THE AUTHOR

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No part of this work has been submitted in support of an application for another degree or qualification.

SYNOPSIS

In this Thesis, details of a proposed method for the elastic-plastic failure load analysis of complete building structures are given. In order to handle the problem, a computer programme in Atlas Autocode is produced. The structures consist of a number of parallel shear walls and intermediate frames connected by floor slabs. The results of an experimental investigation are given to verify the theoretical results and to demonstrate various factors that may influence the behaviour of these structures. Large full scale practical structures are also analysed by the proposed method and suggestions are made for achieving design economy as well as for extending research in various aspects of this field.

The existing programme for elastic-plastic analysis of large frames is modified to allow for the effect of composite action of structural members, i.e. reinforced concrete floor slabs and the supporting steel beams. This modified programme is used to analyse some framed type structures with composite action as well as those which incorporate plates and shear walls. The results obtained are studied to ascertain the influence of composite action and other factors on the load carrying capacity of both bare frames and complete building structures.

The theoretical failure load presented in this thesis does not predict the overall failure load of the structure nor does it predict the partial failure load of the shear walls and slabs but it merely predicts the partial failure load of a single frame and assumes that the loss of stiffness of

For most structures the analysis proposed in this thesis is likely to break down prematurely due to the failure of the slab and shear wall system and this factor must be taken into account in any future work on such structures.

The experimental work reported in this thesis is acknowledged to be unsatisfactory as a verification of the limited theory proposed. In particular perspex was not found to be a suitable material for testing at high loads, micro-concrete may be more suitable.

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NOTATIONS

$\underline{L}$	Applied external load vector, or unit load vector.
$\underline{K}$	Overall stiffness matrix.
$\underline{X}$	Joint Displacements.
fy	Yield stress of steel.
As	area of steel beam.
b	Effective width of concrete slab (Chapter 1, Section 1.3).
t_s	Thickness of concrete slab.
U_c	Cube strength of concrete.
d_n	Depth of plastic neutral axis below top of slab.
M_{pc}	Plastic moment of the composite section.
d	Depth of steel beam.
bf	Width of steel flange.
tf	Thickness of steel flange.
Af	= bf x tf.
tw	Thickness of web.
a,b	First end and second end of the member ab.
Kaa, etc.	Stiffness submatrix or subvector.
h	Hinge number in member data.
λ	Load Factor.
λ_1, λ_2	Load factors at previous and current iterations.
Mp	Plastic moment of the section.
m_1, m_2	Bending moments at previous and current iterations
$M'p$	Reduced plastic hinge moment.
M'_{p1}, M'_{p2}	Reduced plastic hinge moment at previous and current iterations.

n'	Value of "n" when the web is fully yielded.
G, F, n''	Constants for calculation of Mp (Chapter 1).
p	Axial force in the member at a load factor of
p_1, p_2	Axial forces in the member at previous and current iterations.
E	Young's Modulus of Elasticity for steel.
I	Second moment of area of the section.
A	Area of the section
xa, xb	Horizontal deflections at ends a and b of the member.
ya, yb	Vertical deflections at ends a and b of the member.
$\theta_x, \theta_y, \theta_z$	Rotational displacements about x, y, z axis.
$\emptyset$	Stability functions.
θ_a, θ_b	Rotational displacements at ends a and b of the member.
α	Inclination of the member (Chapter 1) or the fraction of a unit external load transmitted to the frames (Chapter 2).
t	Specified tolerance (Chapter 1), or thickness of slab or wall (Chapter 2).
$\underline{P}$	Vector of externally applied wind loads.
$\underline{f}$	Vector of horizontal loads transmitted to the frames.
$\underline{g}$	Vector of horizontal loads transmitted to the grillage of slabs and walls.
$\underline{G}$	Influence coefficient matrix of the grillage of slabs and walls.
K	Total number of frames to be analysed.
$\underline{F}$	Influence coefficient matrix of the frames.
$\underline{a}$	Vector of horizontal deflections at the junction of floors and slabs due to loads applied to the shear walls.

m,n	Total number of floors and frames in the structure (Chapter 2).
d,b,c,q,a	Stiffness coefficients (Chapter 2).
$\underline{K}_{uv}$	Overall stiffness matrix of a wall element.
$\underline{K}_{Lm}$	Overall stiffness matrix of a floor element.
B	Width of the wall or floor slab.
t	Thickness of slab (Chapter 2).
Υ	Constant for the effect of shear on slope deflection equations.
G	Torsional rigidity (Chapter 2).
ψ	$= 1/(1 + 2\Upsilon)$.
p_{ij}	An external load at a given junction ij.
f_{ij}	Frame load at a given junction ij.
g_{ij}	Grillage load at a given junction ij.
β	$= 1 - \alpha$ (Chapter 2).
α_{ij}	The value of α at a given junction ij (Chapter 2)
β_{ij}	The value of β at a given junction ij.
α_o	The initial value of α (Chapter 2).
foij	The initial value of f_{ij} at the junction ij.
α_{oij}	The initial value of αij.
λ_L	Lowest load factor.
ms	Total number of joints in frame s.
hs	Total number of hinges in frame s.
λ_s	Load factor in frame s.
λ_q	Load factor in frame q.
$\underline{Fn}$	New influence coefficient matrix of frames.
$\underline{fn}$	The new vector of frame loads.
α_n	The new value of α

$\underline{f}nij$	The new value of fij.
$\underline{g}nij$	The new value if gij.
Δ_{pij}	An increment.
r_{J1}	The row number corresponding to the unit load.
π_{ri}	The row number in load vector $\underline{R}$ (Chapter 3).
J_{ij}	The joint number at which the unit load is applied to frame i at floor level j.
f_k	The total number of hinges round joint k.
M_{H1}, M_{H2}	Applied hinge moments.
x	Axial loads in the member (Chapter 2).
w	$= \lvert x \rvert / f_Y$ (Chapter 2).
x_m	Axial load in the member at a load factor λ_s.
x_m'	Axial load in the member of the lower load factor λ_L.
M	Applied moment.
Z	Applied horizontal load.
Y	Applied vertical load.
L	Length of the member (Chapters 1 and 2) or the distance between two neighbouring frames. (Chapter 1).
int.pt	Integer part
i	Number of frame
j	Number of floor
l	Length of a secondary beam

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CHAPTER 1

INTRODUCTION

1.1 ANALYSIS OF FRAMED STRUCTURES

The innovation of the digital computers made the analysis of large structures possible. Livesley (11,12) was one of the first to give the application of the computer in the structural engineering problems. By using the matrix displacement method, a relationship was given between the external loads and resulting joint displacements. This was formulated by applying the conditions of joint equilibrium and compatibility and the matrix formulation was given in the form:-

$$\underline{L} = \underline{K}.\underline{X} \qquad \text{......(1.1)}$$

Here $\underline{L}$, $\underline{K}$, $\underline{X}$ are externally applied load matrix, the overall stiffness matrix and the resulting joint displacements respectively. For a given frame the unknown is usually the joint displacements vector $\underline{X}$ while $\underline{L}$ and $\underline{K}$ are known.

The effect of the axial loads were later studied. The stability functions were introduced by Merchant (3,4,5,6,7,8) and tabulated by Livesley (10) and Chandler. It was pointed out that the "overall frame instability" could be ignored for small structures such as one, two or three storey frames, but it would be of great importance for larger frames.

The failure load analysis was dealt with by several authors, (4,5,6,7,8). Merchant (8) et all gave an emprical formulae in orde to determine the collapse load of a frame as a whole. This formulae was similar to that proposed by Rankine for simple load of multi-storey structure was given by Wood (13). The collapse load of the structure was described as a function of the "deteriorated critical load" after yielding had taken place at various plastic hinge locations. Wood observed that the "elastic critical load" would have a "deteriorated value" whenever a plastic hinge developed with a corresponding discount in stiffness. During the formation of the plastic hinge at some point, the "elastic-critical" load value would deteriorate. The formation of plastic hinges and the existence of large compressive axial forces in the members, therefore, resulted in a high reduction in the stiffness of a frame.

The concept of stability was approached by a study of the elastic behaviour in relation to the critical load (11). This was followed by the study of deterioration of stability as plastic zones developed (12). The stability functions (10) were used in this way as mentioned above. Firstly, the élastic behaviour of frames was traced and consequently the approximate behaviour of the frame between the formation of plastic hinges was followed.

Taking the effect of instability into consideration immediately violated the three basic requirements of the simple plastic theory (1, 2,19). Consideration of the axial loads also made the equilibrium conditions different from those assumed by the simple plastic theory, and led to a premechanism collapse by reducing the overall stiffness of the frame. The plastic theory was found to be insufficient to obtain the failure load of tall frames where the instability plays the signitheory (17,18).

The elastic-plastic failure loads of a general framed structure was studied in detail by Jennings, Majid and others (16). The stiffness equations given by Livesley (11) was modified to include the presence of real and/or plastic hinges in the frames. They also studied the effect of axial load in reducing the plastic hinge moment of the members.

The same analysis procedure was later extended by Jennings and Majid (21) to allow for the inclusion of such effects as mentioned above. The iteration procedure was the same as that of Livesley's (11,12). While they were constructing the overall stiffness matrix, the rows and columns corresponding to plastic hinges were inserted to the bottom and the right of the original elastic stiffness matrix. The full matrix operation was used in order to solve the stiffness equations to obtain resulting joint displacements. Therefore, the storage location and machine time required for such an analysis was considerably high. Hence its application limited the size of frames to be analysed.

The size of their matrix was considerably large. This is particularly the case when a large sway frame approaches the point of failure under the combined loading, since many plastic hinges are usually present in the structure at this stage. The demand on the storage locations was also too high, when the full matrix operation was used, because so many zero elements together with the non-zero ones had to be stored.

To overcome the limitations in the programme of Jennings the analysis of large frames. The rows and columns corresponding to plastic or real hinges were placed immediately after the rows and columns of the related joints. Majid and Anderson (23) constructed the overall stiffness matrix in compact form and solved it according to the technique given by Jennings (20).

In "compact storage scheme" (20), two "sequences" were introduced. The "main sequence" stored the elements which appeared between the first non-zero element and that on the leading diagonal, inclusive, in each row within the half band-width of the overall stiffness matrix. The second sequence was called the "address sequence" and was used to record the position of each location which contains the elements on the leading diagonal. The zero elements existing within the irregular half band-width were also stored. The compact method of construction and storage was applied successfully by using the rather important characteristic of the stiffness matrix which is symmetrical about its leading diagonal and have a large number of zero elements. In this way a large number of zeros were left outside the storage and resulted in a considerable reduction in the storage requirements.

Even in the compact storage of the overall stiffness matrix, the width of the half band became sometimes extremely large and it was also found uneconomical. Anderson (24), later modified the same programme (23) to keep the half band-width as small as possible, and also to minimise the increase in the size of the "main sequence".

For instance, for a member connecting the joints a and b, of the same matrix lies below the leading diagonal, and the contributions of a member to that submatrice consists of nine elements in this submatrix. Anderson (24) inserted the rows and columns, related to the hinge, h, at end a, immediately below and to the right of the rows and columns which correspond to the joint a. In addition to this, if a hinge formed at end b, then the row and column corresponding to this hinge were also placed below and to the right of the row and column of joint a respectively. These hinges were considered to be "grouped" around joint a. Only the parts of the row and column subvectors of the hinges existing below and on the leading diagonal within the half-band width were stored.

The column corresponding to a hinge might pass through the half-band width of the joint and hinge rows. Therefore, this column would appear below the row related to the same hinge, so the zero elements in such a column subvector were also stored.

The hinges from more than one member might be grouped around a joint. In this case, the positioning of such hinges were solely dependent upon the number of members containing these hinges.

While predicting the load factor, λ , at which a plastic hinge would form, the variation of bending moment with the load factor was assumed to be linear (16,21). Therefore, an iteration to a constant plastic moment of the section Mp, was carried out, and the prediction of the load factor was made by interpolation or extrapolation. To avoid the mathematical inaccuracies caused when Mp $\cong m_0$, the formulae to predict the load factor was where; λ_1, m_1 and λ_2, m_2 were the load parameters and bending moments at previous and current iterations respectively.

It was shown by Majid and Anderson (23) that the variation of bending moment and plastic moment of the section due to load parameter was not linear, since the axial loads in the members were considerable. It was observed that with λ increasing, both the bending moment and the axial load increased while the plastic moment of the section decreased. This process continued until the bending moments equal to the reduced plastic moment of the section, $M'p_1$ at which a plastic hinge formed. Realising that the variation in the load parameter in this case is not so large a linear variation was adopted.

To overcome the oscillation problem when predicting the load parameters in case of more than one hinge were about to form, an improved prediction was obtained (23,24) by iterating towards the reduced plastic moment, $M'p_1$ of the section at current iteration. However, this increased the computer time. In order to reduce the increased computer time and to overcome oscillation, the prediction of λ was made in the form:-

$$\lambda = \lambda_1 + \frac{(\lambda_2 - \lambda_1)\ (M'p_1 - m_1)}{M'p_1 - M'p_2 + m_2 - m_1} \qquad \ldots\ldots (1.3)$$

where $M'p_1$ and $M'p_2$ are the reduced plastic moments of the section at previous and current iterations. The value of $M'p$ was obtained by the formulaes given below:-

$$Mp' = (Zp - F \times n^2) \times fy \text{ for } n < n' \qquad \ldots\ldots (1.4.a)$$

$$\text{and } Mp' = \left(G\ (1 - n)\ (n'' + n) \right) \times fy \text{ for } n > n' \qquad \ldots\ldots (1.4.b)$$

These are used for universal sections. In the above formulaes:-

n' = the value of n when the web of the section is fully yielded.

G, F, n'' = Section constants given in standard tables (26) of section properties.

The sign of $\frac{\lambda_2 - \lambda_1}{m_2 - m_1}$ was adopted as the sign of the plastic moment. But, $m_2 \cong m_1 \cong M'p$ gave numerical inaccuracy which led to the wrong prediction of laod parameters. Davies (9) found it to be necessary to check $(m_1 \times m_2)$ against $(M'p)^2$. If these were within a specified tolerance, then the sign of m_2 was given to $M'p$.

The axial loads, P, in the member at a predicted load parameter λp was obtained as:-

$$p = \frac{\lambda_2 - \lambda p}{\lambda_2 - \lambda_1} \times p_1 + \frac{\lambda p - \lambda 1}{\lambda 2 - \lambda 1} \times p_2 \qquad \text{......} (1.5)$$

Where p_1 and p_2 are the axial loads in a member at previous and current iterations. These axial loads were used to construct the stiffness matrix (23) in the presence of axial forces.

The iteration procedure used in the above elastic-plastic analysis programmes (16,21,23) were that given in references (11,12). As an example, the iteration procedure followed by Anderson (24) will be described since this was also used in the work done in this Thesis.

For the first iteration, the member forces were set to zero and load parameters were put to equal unity. In this case, the stability functions are made equal to unity. The overall stiffness matrix was constructed, and the stiffness equations were

$$ma = \frac{6EI}{L2} \times \Big((ya \times Cos\alpha - xa \times Sin\alpha) - (yb \times Cos\alpha - xb \times Sin\alpha) \Big) \phi_2 + \frac{4EI}{L} \phi_3 \times \theta_a + \frac{2EI}{L} \phi_4 \times \theta_b \qquad (1.6)$$

$$mb = \frac{6EI}{L2} \times \phi_2 \times \Big((ya \times Cos\,\alpha - xa\ Sin\,\alpha) - (yb \times Cos\,\alpha - xb \times Sin\,\alpha) \Big) + \frac{2EI}{L} \phi_4 \times \theta_a + \frac{4EI}{L} \phi_3 \times \theta_b$$

$$xab = \frac{EA}{L} (xa - xb)\ Cos\,\alpha + \frac{EA}{L} (ya - yb) \times Sin\,\alpha \qquad (1.6)$$

Where ma, mb and xab are the bending moments at end 1 and end 2, and the member axial force respectively. xa, xb are the horizontal displacements of end 1 and end 2 while ya, yb are the vertical displacements of end 1 and end 2, α being the inclination of a member. ϕ_2, ϕ_3 and ϕ_4 are the stability functions, while E, I and L are the Young Modulus of elasticity, second moment of area and length of the member respectively.

These member forces were used in the prediction of load parameters at which a plastic hinge forms. λ was predicted from the eqn. (1.3) and the iteration continued until the condition $|\lambda / \lambda_2 - 1| < t$ was satisfied. Here t is a specified tolerance. The axial load at λ was predicted by eqn. (1.5), to be used in the reformation of the overall stiffness matrix. Meanwhile, the sign of m_2 was given to the reduced plastic moment which was included in the load vector. In the next iteration, the load factor was increased by a small amount to initiate the new iteration. The new sets of joint displacements were obtained which were used in obtaining new member forces by eqn's. (1.6).

in the member and the sign of (m_2) was given to the reduced plastic moment of the section. After each hinge insertion to the frame, the determinant of the overall stiffness matrix of the frame was checked to find out that at some stage, the frame lost all its stiffness after the formation of sufficient hinges, when the determinant of the overall matrix became negative.

1.2 HISTORICAL REVIEW OF THREE DIMENSIONAL ANALYSIS OF STRUCTURES

A brief review of the work done on the analysis of two dimensionsl bare frames in both elastic (3,4,5,6,8,11,13) and elastic plastic range (12,16,17,18,21,22,23,24) including instability (4-8) effects was given in section (1.1). It is generally impossible to rely only on the bare frames in stabilising the structure against the lateral forces. The shear walls were thought to be lateral force resistant elements in the building and first research work (30) on this appeard in the U.S.A. in the design of blast resistant structures. Further research in shear walls (31,33,34) together with bracing walls (29) and rigid frames coupled with shear walls (32, 57) were carried out and is still continuing. The effect of the cladding on the load bearing capacity of the structure was also studied by Bryan and El-Dakhakhni (39,40) and Majid (16).

It was realised that a structure should be stiffened against lateral loads by means of a system composed mainly of flat concrete elements or panels with walls and floor slabs supported by walls. The analysis of structures was mowed from two dimensional conventional analysis into a three dimensional one.

Wilson (35), in their method, ignored the shear walls and assumed that all the frames parallel to one axis were treated to be connected by rigid intermediary at each floor level.

Clough (36) et all, later, extended this method and developed another method for the analysis of the arbitrary systems of shear walls acting in conjunction with frames. Nonetheless they continued to treat three dimensional structures in two dimensions. In order to simulate the behaviour of a three dimensional building, two dimensional frames were assembled into a system of crossed frames. The effect of the torsion in the analysis of the frame members was neglected. To analyse the complete system, the lateral stiffness of each frame was determined and added to each other so as to obtain the total building stiffness. This method was only applicable to the type of building laid out in rectangular grid pattern. The complete structure was assumed to consist of two sets of parallel frames in perpendicular directions. To simplify the problem, floor slabs were assumed to be rigid in their own plane and not to have stiffness normal to this plane. Each floor level was constrained to translate but not to rotate. The axial deformation of both columns and walls were considered, but such an effect was ignored in the beams. Therefore, each plane frame was assumed to have the same displacement at any given floor level. The effects of the shear, flexural and the axial strains of the members were taken into account.

Weaver and Nelson (37) presented a more realistic approach for the three dimensional analysis. A type of structure was model for a "tier building" was built. Frames within the structure were connected by floor slabs which were assumed to be rigid in their own planes.

They followed the modified "tri-diagonalization approach" (35,36) and treated the frames to be in three dimensional configuration rather than conventional two dimensional one. This approach comprised the effect of axial forces in the columns and the torsion in all members. The axial force in the beams was neglected. This method was to construct load and stiffness matrices of a floor taken one storey at a time starting from the top to the bottom storey. From load vector and stiffness matrix, the resulting rigid body displacements were calculated. These consisted of displacements in x and y direction and rotation about z axis by assuming the frames to be rigidly fixed to the foundation. Joint displacements and member forces were obtained from the rigid body displacements. There was no restriction on the symmetry of the loads and frames, thus the assymmetrical arrangement of frames was allowed in the analysis.

Goldberg (41) took the deformations of walls and floors into consideration by assuming that the slabs act as "deep beams" under the combined action of bending moments and shear force in their own plane. The method proposed by him made use of the "slope deflection" equations and was mainly based upon similar concept of "Timoshenko's Beam" (70,71). It was noted that the method was applicable to long narrow tall slab type buildings.

The equilibrium equations for each type of the intersection seven intermediate frames which were connected by means of floor slabs to one another and supported by two end walls. The frames were arranged to be parallel at roughly equal spacing. To reduce the computer time, the frames were assumed to be identical and were substituted by an equivalent frame with two columns.

Three different cases were taken into account in the analysis in which the walls and floors were subject to:-

- combined bending and shear deformations,
- bending deformations only,
- shear deformations only.

Goldberg did not consider the effect of axial deformations in the columns and neglected the resulting side sway of the structure owing to the vertical loading. Twisting of the floors and walls and the out of plane bending of these elements and frames were also ignored. The flexural effect of axial forces, particularly in columns, was not considered.

Majid and Williamson (44,45) gave a method of analysis for the general complete structures consisting of a combination of one or more three basic elements. These components were prismatic members and plate elements with in plane forces and plate elements with out of plane forces respectively.

A computer programme was given which enabled the analysis of a structure in its complete form. This was one of the first programmes to analyse space structure including plate elements under in plane forces. It also included the out of plane forces including bending torsion and cross coupling. The method made use of the finite element technique.

The contribution of each structure consisting of plate elements with in plane forces and plate elements with out of plane forces and the space frame of the structure were grouped together. The overall stiffness matrix was obtained so simply by superimposing the overall stiffness matrix of each three components. The "sparse matrix" technique (42) was utilised in both constructing the matrices and solving the stiffness equations for obtaining the nodal displacements.

They also carried out a set of experiments on two or three storey plane frames with shear wall cladding. The effect of shear walls on the stiffness of bare frames were taken into consideration. By analysing a pitched roof portal shed (16,39,40), the effect of sheeting on the carrying capacity of the frames was demonstrated.

The laterally loaded system comprising of shear walls and parallel intermediate frames similar to "Goldberg Structure" (41) was studied by Rosman (47). The walls at both ends of the structure were identical and the intermediate frames were also identical to one another. The floor slabs which played a role as an interconnecting media between frames and walls were assumed infinitely rigid in their own planes.

The intermediate frames were assumed not to take part in carrying the lateral loads. The rigidity of the walls was also assumed to be much greater than those of the frames.

All the building was replaced by a "substitute system" which consisted of a flexure cantilever resting on an elastically yielding support and a shear cantilever, both continuously connected thin laminae which was extremely rigid in their longitudinal axis and hinged at both ends when dealing with multi-storey structures.

Rosman used differential equations for the solution and ignored the effects of shear deformation in the walls and the flexural deformation in the frames. The deformation of the wall was assumed to be of prime importance. The main purpose was to define the bending moment of the flexure cantilever so as to satisfy the differential equation.

Stamato and Stafford Smith (46) gave an approximate method of three dimensional alysis of complete buildings which were assumed to consist of two dimensional vertical panels such as rigid jointed frames, trusses and shear walls or any sort of mixed arrangement of these. Three dimensional analysis of the structure was reduced to a two dimensional analysis. The components of the building might be arranged in an arbitrary pattern in the plan.

Discontinuity in walls and floor slabs were permitted, beams may also be discontinuous across the width or length of the structure. They gave consideration to the axial strains in both walls and columns. Either to conform the assumptions of slabs being rigid diaphragms in their own plane with no transverse stiffness or in order to satisfy the compatibility conditions at each floor level, the beams were assumed to be axially rigid. That is to say, the axial strains in the beams were ignored.

The vertical forces in the intersections between panels and the horizontal forces transmitted by floor slabs in the intersection between floors and panels were assumed to be the only

Allowance was given for any arbitrary intersection between panels. The angle of intersection might be of any magnitude, but it had to be right angle where the intersections took place at a column, otherwise it would cause the occurance of bending moments along the line of intersections between panels. This bending moment was already assumed to be small, therefore, the occurance of such a bending moment would violate this assumption.

The effect of the torsion in the structure as a whole was ignored. The shear deformations was considered only in the shear walls by including "Poisson's Ratio" in the calculations.

Majid and Croxton (51) proposed a method for the analysis of three dimensional structures consisting of skeletal frames together with a system of grillage of shear walls and floor slabs. It was a method for the wind analysis of complete building structures. The frames and the grillage were taken to be two distinct components of the structure and were analysed independently in order to obtain the influence coefficients of these components. The arrangement of frames was assumed to be parallel to one another. Consideration was also given to the assymetrical arrangements of parallel frames and walls in the plan of the structure.

The floors were considered to be monolithic and of an arbitrary shape at any floor level. The bending of walls and floor slabs in their own plane due to mixed action of flexure and shear were taken into account. The torsion of the whole structure, together with the possible side sway caused by vertical loads were also included.

caused by windloads were ignored. The horizontal displacement of the columns at a certain level in a frame was assumed to be equal. Therefore, the axial forces in the beams were neglected and by which the compatibility equations were satisfied.

The overall stiffness matrix of the grillage was constructed by utilising the concept of the "deep beams". This was used to obtain the "influence coefficients" of the grillage system. The unit load matrix was constructed and the eqn. $\underline{G} = \underline{K}^{-1}\ \underline{L}$ was solved. Here $\underline{G}$ is the grillage influence coefficient matrix, $\underline{K}$ is the overall stiffness matrix and $\underline{L}$ is the unit load matrix acting on the grillage.

The overall stiffness matrix was constructed by using any technique (21,22,24). The influence coefficients matrix of bare frames were obtained by solving the equations $\underline{F} = \underline{K}^{-1}\ \underline{L}$, where F is the influence coefficients matrix of the frame, $\underline{K}$ is the overall stiffness matrix and $\underline{L}$ is the unit load vector acting on the frame.

By utilising the influence coefficients of both grillage and plane frames, the horizontal deflections at junctions of the frames with the grillage and the sway deflections of all the junctions of the bare frames were obtained. The horizontal displacements of the grillage and the intermediate frames at the same junctions of the grillage were set to equal to one another in order to satisfy the compatibility conditions. From the compatibility consideration, the loads transmitted by frames and grillage elements were calculated, and then these were used as load vectors to analyse the grillage and the frames individually.

of the concrete on the behaviour of the complete building structure as a whole were considered. It was shown that the assumption that the slabs were rigid misrepresented the bending moments at the top and the bottom storeys of the structure. The effect of axial thrusts in columns was also included.

This method was only applicable to the type of structure which consists of orthogonal grillage and frames. However, it was rather a simplified method and enabled to reduce the time required for both data preparation and computer analysis considerably. The method was basically a matrix force method and required a small number of equations.

To verify the method, a number of tests were carried out. The structure analysed by Goldberg (41) was reanalysed and the results were compared with each other. The results were also compared with those obtained from finite element solution (44).

Ghali and Neville (50) gave a method for the analysis of shear walls in which the effect of the lateral forces on the structure was studied. In the three dimensional analysis the structure was considered to have an arbitrary arrangement of both shear walls or frames. The structure was considered to have an assembly of monolythic walls, plane walls connected by beams at each floor level and a plane frame.

The floor slabs were assumed to act as a rigid diaphragms (35,46,47) but the floors were not to restraint the angular displacements of any member. By utilising this assumption on the floors, the joint displacements of the structure were reduced to two horizontal deflections with an angle of twist about verof elements in an arbitrary arrangement was given. The procedure was commenced by determining the lateral stiffness of each element as a separate structure. The laterall stiffness of individual elements were then used to construct the overall stiffness matrix of the complete structure. The out of plane and the torsional rigidities of plane frames and plane walls connected by beams at floor levels were ignored. The axial deformation in the beams was also neglected.

Davies (58) proposed a method for the computer analysis of "stressed skin buildings" which consisted of a series of parallel rigid steel frames connected together via purlins and sheeting panels which were supported by end gables at both ends of the building. The frames were arranged roughly at equal spacing.

Davies used a mathematical model to reduce the complete building analysis to that of a two dimensional frame analysis. The joint properties of sheeting panel with purlin was defined by the shear flexibility "c". In the model, the distance between frames and also between outer frames and end gables was assumed to make no difference, therefore, they were moved close together. By considering the relative displacement "Δ" of two adjacent frames, a couple of restoring forces "Δ/c" exerted by sheeting panels was obtained and applied at each floor level. To simulate the coupling effect of the sheeting panels, therefore, these panels were substituted by a series of springs of flexibility "c". These springs connected two adjacent frames through the one leading diagonal of the panels to be in the same directions at each panel.

of the next frame. The same type of connections was repeated at each span.

The end gables were assumed to act as a rigid foundation. In the case of stiffened end gables the same idealisation was made. The restoring forces were found by considering the relative displacement of end frames and flexibility "c".

A computer method was given. The overall stiffness matrix of the complete structure was constructed including the coupling effect of the sheeting. Such an inclusion increased the band width of the matrix considerably. But, the sheeting resulted in considerably large reductions in bending moments of the frames.

By using this technique and a mathematical model, Davies (59) later tried to give a method of analysis for the elastic-plastic stressed skin buildings. The three dimensional analysis of the structure was reduced to an elastic-plastic analysis of two dimensional bare frames.

Proposed method of elastic-plastic analysis was applied to pitched roof and rectangular portal frames buildings. The computer results for both complete structure and bare frame (uncladding)compared with those of experimental results obtained from the tests on bare frame and complete structures. The increasing effect of the sheeting on the load bearing capacity of the structures was demonstrated.

The well known assumptions on the elastic-plastic analysis (16,21,23) were utilised. In addition, the behaviour of the panel sheetings were assumed to be elastic - purely plastic. The effect of instability and strain hardening were not included

1.3 ANALYSIS OF COMPOSITE STRUCTURES

There have been a continuous research in the field of the composite action of floor slabs and supporting steel beams. They were particularly based on the experimental work rather than computer analysis. It was shown by current research that there would be many advantages in using the composite action between steel beams and the concrete slabs. It has already been realised that the effect of the cladding (16,39,40) on the load bearing capacity of the bare frame would be considerably high.

The early tests on the composite action were performed by a bridge company (60). Two steel beams encased in concrete and floor slabs were tested. It was thought that the steel beam and the concrete might really act together in order to form a composite beam.

A review of the research works carried out on the composite actions was given by Viest (60). This review covered most of the research carried out on composite steel-concrete structures between 1920-1958. Viest (61) conducted a number of tests so as to demonstrate the carrying capacity and the behaviour of the steel shear connectors for composite concrete and steel T beams. The steel beams and the concrete slabs can be connected naturally, but it was obvious that this type of connection cannot always be reliable. Therefore, it was recommended to use shear connectors in order to provide the mechanical connection between floors and beams. The shear connectors provided the composite action by preventing the slabs moving away from the beams.

in complicated structures.

An ultimate design method was suggested to be suitable when the relatively large shape factor of the composite section is taken into consideration. But, some limitations would be required to satisfy the restrictions of the rotational ductility of the composite section. It was pointed out that because of the large shape factor of the section, yielding was spread over a large portion of the length of the beam before collapse occurred. By using a large shape factor of 1.55 for the composite beam, a considerable increase in both elastic and plastic section modulus were gained.

Chapman (63), later carried out a number of tests on simply supported composite T beams under both concentrated and distributed load. The loads were applied on the axis of the beam. The elastic neutral axis was considered to be close to the adjacent face of the slab and the steel beam. As the bending moment increased, the bottom flange yielded first which caused the neutral axis to move upward resulting in tensile cracking at the adjacent face of the concrete slab. It was also pointed out that if the plastic neutral axis was close to the beam flange, then full plasticity could not be developed in the steel before the cracking of the concrete.

Barnard (64) et all carried out a number of tests in order to obtain complete sets of values of the bending moment and curvature for a cross-section of a composite beam. This was an extension to the work of Chapman (62, 63).

The high shear stress and the slip at the adjacent face

stress-strain curve for the concrete was assumed to be linear up to first yield in the steel reinforcement. This was predicting the behaviour of the section of the composite beams at yield and at a maximum moment. By assuming the linear variation in stress due to strain in concrete and neglecting the slip at the steel-concrete interface, the possible three conditions at maximum moment were studied. They were as follows:-
The neutral axis is:

- in the steel beam,
- in the concrete slab but steel fully plastic,
- in concrete slab but the steel beam is not fully plastic,

respectively.
At the above conditions, the reinforcement was assumed to be concentrated at a point and to have a negligible area compared to the slab area.

Later, Barnard (65) et all carried out further tests on the plastic behaviour of continuous composite beams. The use of simple plastic theory was justified for the design of most types of continuous composite beams. It was pointed out that the secondary failure could be prevented. They did not give a clear recommendation to analyse and design such a continuous beam by using ultimate strength method. But a method for the prediction of the behaviour of composite sections was described in bending in which the effect of slip was neglected. The proposed method was shown experimentally to be still applicable.

Davies (66) carried out an experimental research into the half scale steel-concrete beams with steel stud connectors. A forcement in the slab was examined. It was noted that the general behaviour of a composite beam was affected by the amount of transverse reinforcement in the slabs. Within the elastic range, the theory gave the stresses to be slightly less than the observed values.

Johnson and Heyman (67) compared the design methods of using simple plastic and plastic composite methods with other conventional design methods. The advantages of the composite plastic design method over the others were shown. It was also noted that the composite actions between a column member and concrete wall elements were significant. In this case a proportion of the axial load could be carried by the concrete elements. This resulted in reducing the section of columns by a considerable amount.

No composite action at the ends of the composite beams was taken into consideration. It was found to be important to check the deflections at working load. Therefore an elastic analysis was necessary. This was carried out by neglecting any slip at interface.

Wood (13) also carried out a large amount of tests to demonstrate the interaction of floors and beams in multi-storey buildings. The simply supported and discontinuous reinforced concrete slabs were tested which were supported by beams of various sizes. The slabs tested by Wood (13) were assumed to be resting on edge beams without any composite action between them.

Majid (69) used an experimental programme in order to satconsiderable advantage would be gained when the effect of composite action was considered. The inclusion of floor slabs was found to be particularly significant as the stiffness of the beam or floor slabs play a significant role in the stability criteria for a multi-storey structure.

This work was mainly concerned with the ultimate load behaviour of beam and floor slabs using steel beam and concrete floor slabs. Three basic modes of collapse of beams and floor slabs were considered. They were:-

- Collapse might happen in the slab and the secondary beam due to weak secondary and main beams,
- Collapse might take place in the floor slabs and main beams also due to weak secondary and weak main beams respectively,
- The independent collapse of the slab might occur.

The failure of the slab and the secondary beam consisted of a "repeating element" of one secondary beam of length (l), and a slab of width equal to the average length of two adjacent bays. In the case of equal bays, the effective slab width was to be equal to the length of the bay, L.

Majid (68) gave a general computer analysis. This was written in Atlas Autocode. Three different types of failure, (68,69) were considered in the programme. The effect of composite action of the stiffness of the supporting beams and, therefore, on the instability of the frame as a whole were also considered. The effect of walls in composite behaviour of frames was not included.

The plastic moment of the composite section was acquired by

Here only the first case will be summarised since it was related to the part of work done in this Thesis. The formulaes were given depending upon the position of plastic neutral axis and derived from the simple equilibrium consideration between tensile and compressive forces in the composite section. In all formulaes, slab reinforcement and the effect of axial forces were neglected.

1 - Plastic neutral axis within the slabs:-

This case happens when:-

$$(fy \times As) \leqslant (b \times ts \times Uc) \qquad \ldots\ldots \quad (1.7)$$

therefore

$$dn \leqslant ts \qquad \ldots\ldots \quad (1.8)$$

The plastic neutral axis position was found by:-

$$dn = (fy \times As) / (b \times Uc) \qquad \ldots\ldots \quad (1.9)$$

where

fy = yield stress of steel beam.

As = the cross-sectional area of steel beam.

b = effective width of floor slabs.

ts = thickness of slab.

Uc = cube strength of concrete which is $\frac{2}{3}$ of the value specified for construction.

dn = the position of the plastic neutral axis within the composite section which is the distance from the top of the slabs to the neutral axis.

The plastic moment of the composite section, Mpc was obtained by:-

$$Mpc = fy \times As \times [\,0.5 \times (d - dn) + ts\,] \qquad \ldots\ldots \quad (1.10)$$

where d = depth of the steel beam.

a - Plastic neutral axis within the top flange of the steel beam:-

This was possible in the case when the following conditions were present:-

either (fy x As) $<$ (b x ts x Uc + 2 x fy x Af) ..... (1.11)

or (fy x As) $>$ (b x ts x Uc) ..... (1.12)

where

Af = the area of the steel flange.

The location of plastic neutral axis was determined by:-

$$dn = ts + \frac{fy \times As - b \times ts \times Uc}{2 \times bf \times fy} \quad \text{..... (1.13)}$$

hence

$$ts < dn < tf + ts$$

where

ts = thickness of slab

tf = thickness of flange

bf = width of flange

Af = bf x tf

After having found the location of the plastic neutral axis in the section, then the plastic-composite moment Mpc was obtained:-

$$Mpc = fy\,[\,0.5 \times As \times (d + ts) - bf \times dn \times (dn - ts)\,] \quad \text{..... (1.14)}$$

b - Plastic neutral axis within the web of the steel beam:-

This case was possible when the following condition was present:-

$$fy \times (As - 2 \times b_f \times t_f) > (b \times ts \times Uc) \quad \text{..... (1.15)}$$

and the distance of the plastic neutral axis from the top of the slab is given by:-

therefore:

$$dn > ts + tf \qquad \ldots\ldots (1.17)$$

Plastic moment of the composite section is then given by:-

$$Mpc = fy \times [\ (0.5 \times As \times (d + ts) - (bf \times tf) \times (ts + tf) - tw \times (dn + tf) \times (dn - ts - tf)] \qquad (1.18)$$

The effect of slab reinforcement was included in references (68,69). The location of the plastic neutral axis and the value of Mpc were found in the same manner followed in the derivation of the above formulaes.

As mentioned before, the effective width of the compression flange of the slabs, b, is taken to be equal to the distance between two intermediate frames. In the case of unequal spacing between frames, then the average of these was taken as "b".

1.4 SCOPE OF THE PRESENT WORK

The main purpose of this Thesis is to present an elastic-plastic method for the wind analysis of a class of multi-storey buildings consisting of a grillage system of parallel floor slabs and shear walls, together with an arbitrary arrangement of parallel skeletal sway frames. The proposed method given in Chapter 2 considers the structure to consist of two distinct components. An accurate approach to this type of structure demands a knowledge of the separate amount of external loads transmitted individually to the frames and the grillage system of floor slabs Hence, the relative stiffness of the frames and the grillage system throughout the loading history of the complete structure up to and including collapse will be taken into account.

action of shear and flexure. Allowance is also made for the torsion of the building as a whole about a vertical axis. The full effect of the axial loads in the stability and the plastic hinge moment of the frames is included. Asymmetrical arrangement of parallel frames and shear walls is permitted.

The method assumes that the thickness of the floor slabs are decided by the vertical dead plus superloads. It is also assumed that the panels act as deep beams in which the out of plane bending under wind loading can be neglected.

Elastic-plastic analysis of complete building structures is not an easy task, since it involves the solution of a large number of equations repeatedly. Therefore, a computer programme, which is described in Chapter 3, is required to handle the problem. In this programme, facilities are provided to reduce the requirements in computer time and storage locations in the analysis.

The results of an experimental investigation are given in Chapter 5 to verify the theoretical work and to demonstrate various factors that may influence the behaviour of these structures. Descriptions of the experimental set up are given in Chapter 4. The practical work carried out here is of two types. Firstly, single storey structures with three internal frames (sometimes two) are tested to collapse. The second type of tests are on two storey structures consisting of three intermediate frames which are also tested to collapse.

Large full scale practical building structures are analysed in Chapter 2 by the proposed method and suggestions are made for achieving design economy to extend research in various as-bare plane frames is modified in Chapter 6 to take the composite effect of the slabs on the plastic hinge moment of the beam into consideration. Thus the accuracy of the elastic-plastic analysis will be improved by including the composite effect of the slabs and the frames. A number of examples taken from practical structures are given to exhibit the advantage gained by including such effects.

The elastic-plastic method proposed for the wind analysis of the complete building structure is combined in Chapter 6 with the method proposed for the composite analysis of the bare frames. In carrying out the elastic-plastic analysis of complete buildings, the composite effect of slabs on the plastic hinge moment of the beams is taken into consideration. To do this, the computer programme written for complete building structures is modified in Chapter 6 to provide the facility to include the composite action.

To demonstrate the effect of the composite action on the load carrying capacity of the complete building, a number of full scale practical structures are analysed in Chapter 6. In these analyses, the effect of slab reinforcement and also the composite action in the columns are ignored.

CHAPTER 2

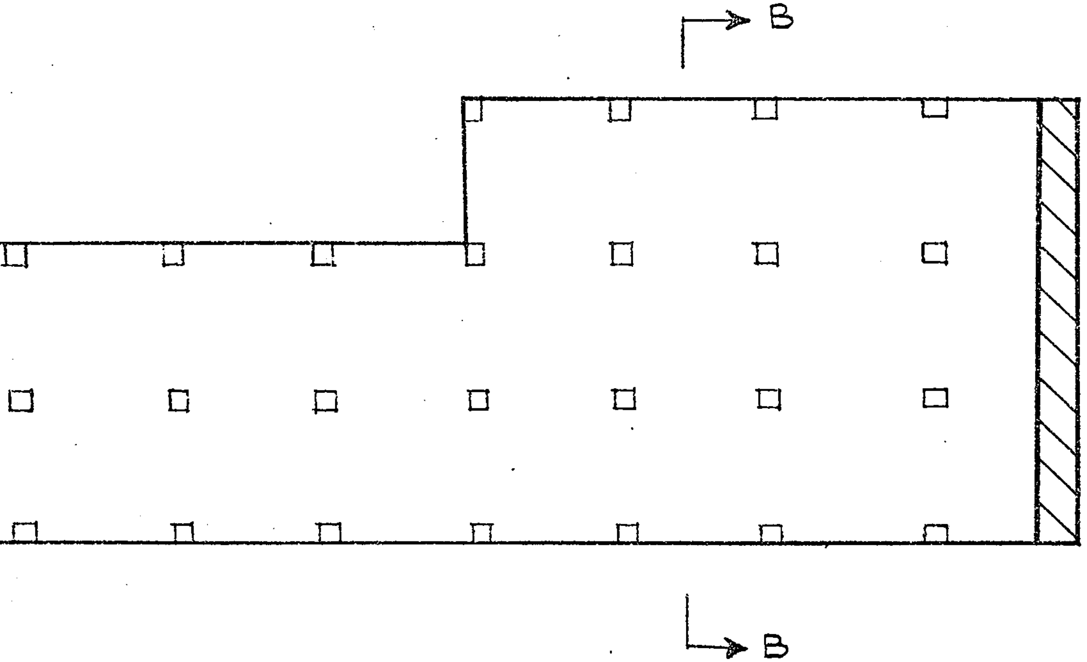
THE ELASTIC-PLASTIC FAILURE LOAD ANALYSIS OF COMPLETE BUILDING STRUCTURES

2.1 Introduction

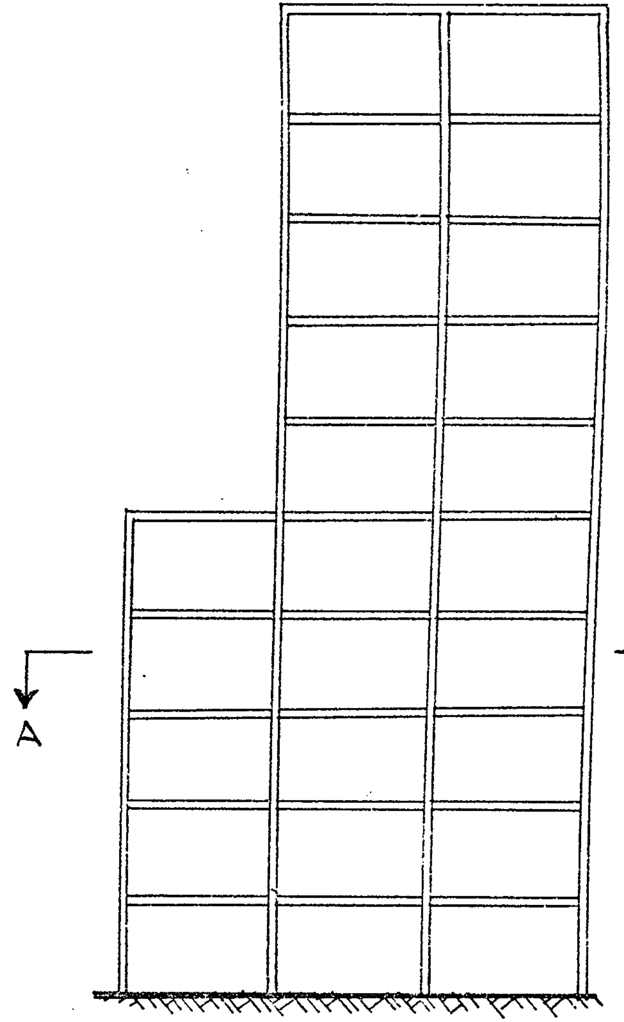
In the field of structural analysis, significant progress was made in the last decade mainly because of using matrix and computer techniques. This facilitated the development of general methods applicable to quite complex problems. For instance, analysts moved away from elastic plane frames into three dimensional skeletal as well as clad structures (35,40,42,44,49). The shortcomings of the simple plastic theory (1) of plane frames, popular in the fifties, was soon recognised and it was replaced by the more realistic elastic-plastic approach including the effect of instability (16,18,21,22,23,24,25).

It is, therefore, natural to expect that, during the present decade, elastic-plastic analysis and design procedures will be extended to deal with complete three dimensional structures. In fact, Davies (58) has already used a two dimensional plane frame analysis to deal with the elastic-plastic analysis of pitched roof portal sheds. Although the corrugated sheeting of the shed was idealised by a simple pin ended member whose stiffness was found experimentally, the results obtained proved to be encouraging.

In this chapter an elastic-plastic method for the wind analysis of a class of multi-storey buildings consisting of a grillage system of parallel floor slabs and shear walls, together with an arbitrary arrangement of parallel skeletal sway



Section A - A



Section B - B

Fig. 2.1. Typical parallel arrangement of walls and slabs

requires a knowledge of the separate amount of external loads transmitted individually to the frames and the grillage system of slabs. This demands the consideration of the relative stiffness of the frames and the grillage system throughout the loading history of the complete structure, up to and including collapse.

2.2 Separation of Wind Loads

The complete structure is considered as if it consists of two distinct components (51). These are the bare frames, made out of steel, and the grillage system of slabs and shear walls which is usually made out of reinforced concrete. Throughout the proportional loading of the complete structure and specifically whenever a plastic hinge develops in one of the frames, the amount of external load transmitted to the bare frames changes. It is, therefore, necessary to evaluate, at each hinge formation, the manner in which the external wind loads are divided between the frames on the one hand and between the frames and the grillage system on the other.

Consider the complete structure, with the sign convention and the degrees of freedom shown in fig. (2.2) which consists of shear walls, slabs and frames. The applied wind load vector $\underline{p}$ at the junctions of the frames and the slabs are divided into a vector $\underline{f}$ transmitted to the frames and another vector $\underline{g}$ transmitted to the grillage system.

Thus

$$\underline{p} = \underline{f} + \underline{g} \qquad \ldots\ldots (2.1)$$

This is the same equation as eqn. (2) in reference (51).

Considering the grillage of floors and shear walls, let $\underline{G}$

expresses the horizontal displacement at each junction due to unit loads applied, one at a time, at the various junctions of the frames and the floors. For instance, matrix $\underline{G}$ is given (51) for the structure shown in the figure (2.2) as:-

$$\underline{G} = \begin{bmatrix} G_{11,11} & & \text{symmetrical} & \\ G_{21,11} & G_{21,21} & & \\ G_{12,11} & G_{12,21} & G_{12,12} & \\ G_{22,11} & G_{22,21} & G_{22,12} & G_{22,22} \end{bmatrix} \begin{matrix} \text{1st floor} \\ \\ \\ \text{2nd floor} \end{matrix} \qquad \ldots(2.2)$$

Each row of matrix $\underline{G}$ includes all horizontal deflections of a given frame junction, when unit loads are applied alternatively at every frame junction of the grillage. The vector of horizontal deflections of these junctions is given by $\underline{G}\,\underline{g} + \underline{a}$ where $\underline{a}$ is the vector of horizontal deflections of the junctions when loads $\underline{w}$, shown in fig. (2.2), are applied to the shear walls. Similarly, the deflections at the floor levels in the frames are given by $\underline{F}\,\underline{f}$, where $\underline{F}$ is the influence coefficients matrix for the frames and which, for instance, is given for the structure shown in the same figure by:-

$$\underline{F} = \begin{bmatrix} F_{11,11} & & & \\ 0 & F_{21,21} & \text{symmetrical} & \\ F_{12,11} & 0 & F_{12,12} & \\ 0 & F_{22,21} & 0 & F_{22,22} \end{bmatrix} \begin{matrix} \text{1st floor} \\ \\ \\ \text{2nd floor} \end{matrix} \qquad \ldots(2.3)$$

Frame   Frame   Frame 1   Frame 2

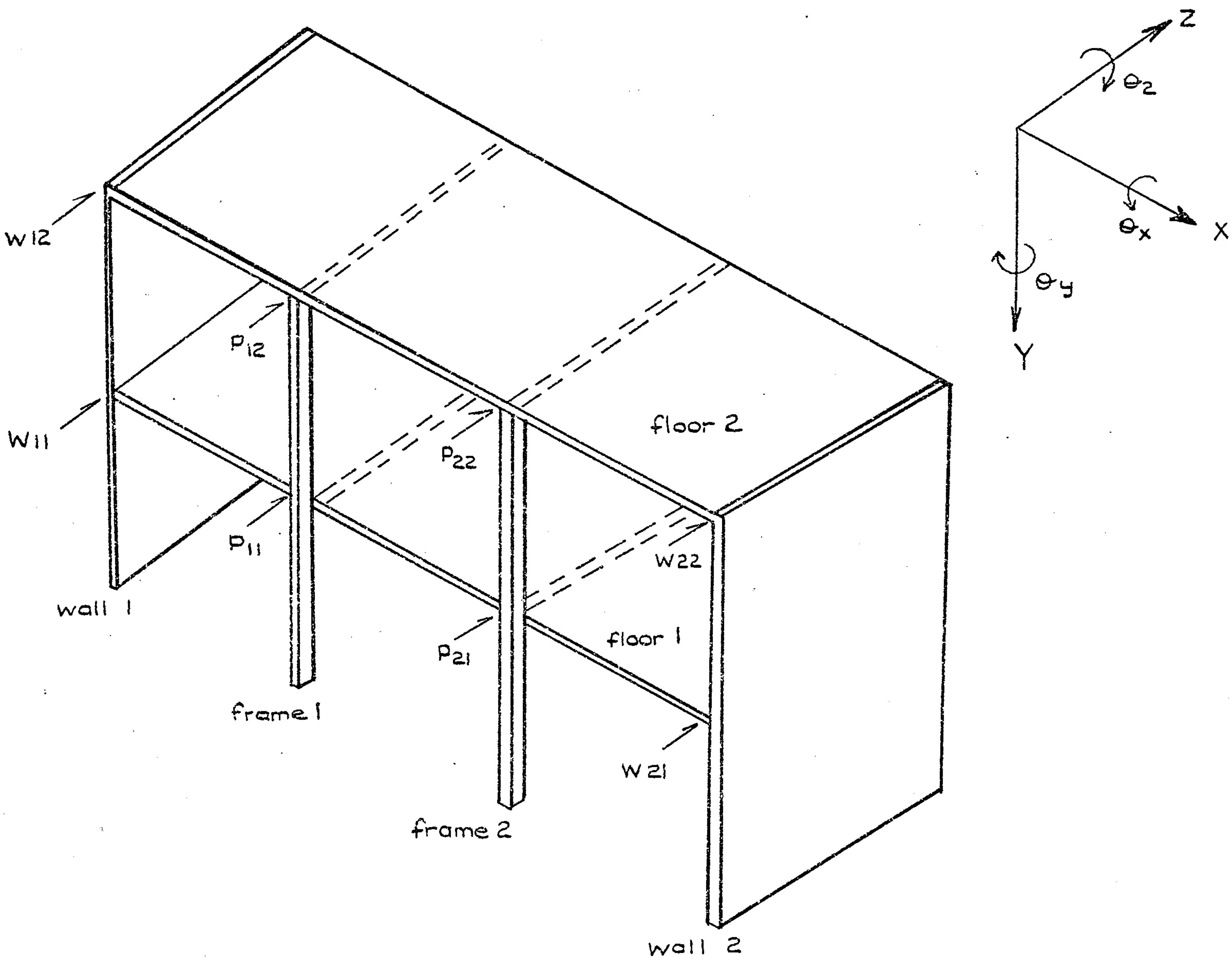


Fig. 2.2 Numbering of walls and frames

In matrix $\underline{F}$ each row contains all the horizontal deflections o
a given frame junction when unit loads are applied alternative
at every floor level of the frames. Matrices $\underline{G}$ and $\underline{F}$ are of
the same order $(m \times n)^2$, where m and n are the total number of
floors and frames in the structure.

When combining the frames and the grillage system, compat
ibility requires that the horizontal deflection of the grillag
and the bare frames at the junctions must be equal, thus:-

$$\underline{G}\,\underline{g} + \underline{a} = \underline{F}\,\underline{f} \qquad \text{...... (2.4)}$$

Using (2.1) and solving for $\underline{f}$ gives:-

$$\underline{f} = (\underline{G} + \underline{F})^{-1} (\underline{G}\,\underline{p} + \underline{a}) \qquad \text{...... (2.5)}$$

The vector $\underline{g}$ can be calculated from eqn. (2.1). Further
details about the form of the influence coefficient matrices a
given in references (51,52) and need not be repeated here.

2.3. The Elastic-Plastic Analysis

The steps for the full analysis of the complete structure
up to collapse are:-

(i) - The frames in the structure are analysed by any conven-
tional method such as the matrix displacement method. In that
case, an inverse transformation of the form $\underline{F} = \underline{K}^{-1}\,\underline{L}$ gives th
influence coefficients, where $\underline{K}$ is the overall stiffness matri
of a frame. Each column of the load matrix $\underline{L}$ for a frame con-
sists entirely of zeroes except for a single unit load. Each
joint in the frame has three degrees of freedom in z, y and θx
directions (see fig. 2.2). Initially, many or all of the fram
may be identical and a single frame analysis may suffice. How
ever, this state of affairs changes as soon as plasticity de-
velops in the frames.

grillage system for the construction of matrix $\underline{G}$. Each joint of the grillage is assumed to have freedom in Z, θ_x and θ_y directions. The contribution of a wall or floor element to the overall stiffness matrix of the grillage can be obtained directly from the slope deflection equations for a deep beam. For instance, the contribution of a shear wall element connecting floors u and v is:-

$$\underline{K}_{uv} = \begin{bmatrix} d & & & & & \text{symmetrical} \\ -c & b & & & & \\ 0 & 0 & q & & & \\ -d & c & 0 & d & & \\ -c & a & 0 & c & b & \\ 0 & 0 & -q & 0 & 0 & q \end{bmatrix} \qquad \text{...... (2.6.a)}$$

at u at v

where $a = 2EI\psi(1-\gamma)/L$

$b = 2EI\psi(2+\gamma)/L$,

$c = 6EI\psi/L^2$,

$d = 12EI\psi/L^3$,

$q = GBt^3/3L$,

$\psi = 1/(1+2\gamma)$,

B = the width of the shear wall,

t = the thickness

The constant γ takes into consideration the effect of shear deformation on the slope deflection equation and is taken as $7.2EI/L^2AG$. Here A is the cross-sectional area of the panel and G is the modulus of rigidity (70). This assumes a parabolic shear stress distribution.

(51). This matrix is:-

$$\underline{K}_{lm} = \begin{bmatrix} d & & & & & \\ 0 & q & & \text{symmetrical} & & \\ -c & 0 & b & & & \\ -d & 0 & c & d & & \\ 0 & -q & 0 & 0 & q & \\ -c & 0 & a & c & 0 & b \end{bmatrix} \quad \ldots\ldots\ldots (2.6.b)$$

where all the coefficients appearing in the above matrix are the same as those of matrix $\underline{K}_{uv}$.

(iii) - Once matrices $\underline{G}$ and $\underline{F}$ are constructed, the inverse transformation, given by equations (2.5), is carried out. This gives the forces $\underline{f}$ transmitted to the frames.

Consider now a given junction ij with an external load pij. The horizontal equilibrium equation at this junction is:-

$$pij = fij + gij \quad \ldots\ldots (2.7)$$

Dividing through by pij we obtain:-

$$1 = {}^{fij}/pij + {}^{gij}/pij \quad \ldots\ldots\ldots (2.8)$$

i.e. $1 = \alpha\, ij + \beta ij$

where $\alpha ij = {}^{fij}/pij$ and

$\beta_{ij} = {}^{gij}/pij$

Here α ij is that fraction of a unit external load which is transmitted to the frame. For a given frame let the initial value of α be α o.

(iv) - Each frame is now considered separately and analysed elasto-plastically to find the load factor λ that is necessary to cause the formation of a plastic hinge. Full account is taken of the effect of the axial loads in the members on the sta

Mp values of the hinges. A brief explanation of this is given in Chapter (1), and full details are given in references (23,24) and need not be repeated here.

(v) - Because the loads $\underline{f}$ acting on each frame are different from frame to frame, the load factor for the formation of a hinge is also different from frame to frame. The lowest load factor λ amongst these is selected as being the one that causes a plastic hinge to form anywhere in the structure.

Consider that under λ the plastic hinge develops in frame i and let the force obtained from equation (2.5) at floor j of the frame be foij and the corresponding α_o value be α_{oij}. The total load at junction ij acting on the frame, at the formation of a hinge in frame i will, therefore be $\lambda_L \cdot f_{oij}$. The external load $\lambda_L p_{ij}$ is thus given by:-

$$\lambda_L p_{ij} = \lambda_L f_{oij}/\alpha_{oij} \qquad \text{......} \ (2.9)$$

Hence, if the external working loads acting on the structure are increased proportionally by a load factor λ_L, a plastic hinge will develop in frame i.

(vi) - This hinge is inserted in the frame. With it, frame i changes qualitatively and becomes different from other initially similar frames. A plastic hinge can only sustain a bending moment equal to its plastic hinge moment Mp. Any increase in the loads acting on frame i does not lead to a similar increase in the bending moment across the plastic hinge. In fact, from now on this hinge responds as a purely frictionless real hinge. Furthermore, as the loads increase, the axial forces in the members of frame i also increase and hence the plastic hinge moment of a hinge continuously decreases.

flexible. As the external loads increase, this frame becomes incapable of sustaining its share of the transmitted loads. Part of this has, therefore, to be transferred to the slabs and transmitted to the other frames and the shear walls.

Before leaving this stage, it is necessary to mention that, to save computer time, more than one plastic hinge can be inserted into the structure. To do this, while selecting λ_L, the load factor at which a hinge develops in any other frame is compared to λ_L. If this is within a specified tolerance of λ_L, then a further hinge is also inserted in that frame. On the othe hand, to safeguard the accuracy of the analysis, only one plastic hinge at a time is permitted to develop in any one frame.

(vii) - The influence coefficient matrix for a frame with plastic hinges is different from a similar one without a hinge. For this reason, every time a hinge is inserted in a frame, it has to be re-analysed under unit loads in order to prepare a new influence coefficient matrix $\underline{F}_n$. During the interval between the formation of one hinge and the next, interest is focussed upon the increments of the applied loads and not upon their total values. Therefore, the response of the plastic hinges, already in the structure, to these load increments is exactly as that of real hinges. Details of the changes in the stiffness matrix of the frames with hinges and the corresponding unit load matrix are given in the next section.

(vii) - Each time a new $\underline{F}_n$ is prepared, the inverse matrix transformation (2.5) is carried out to calculate the new forces $\underline{f}_n$ transmitted to the frames. Equations (2.5) are now in the form:

$$\underline{f}_n = (\underline{G} + \underline{F}_n)^{-1} (\underline{G}\, \underline{p} + a) \qquad \text{...... (2.10)}$$

system is unaltered but matrix ($\underline{G} + \underline{F}_n$) is nonetheless different from ($\underline{G} + \underline{F}$).

(ix) - Once $\underline{f}_n$ is obtained, the new factors α_n and β_n at each junction are calculated from:-

$$\alpha_{nij} = f_{nij}/p_{ij}$$

and $$\beta_{nij} = g_{nij}/p_{ij} \qquad \ldots\ldots \ (2.11)$$

It is noticed that here and in equations (2.10), the values of vector $\underline{p}$ or force $\underline{p}_{ij}$ are unaltered. This is because the actual values of the external loads are immaterial in these calculation and the original working loads can be used throughout.

(x) - Each frame is now treated independently to a fresh elasto-plastic analysis to predict the lowest load factor for the formation of the next plastic hinge in the whole structure. Each existing hinge is now treated as a plastic hinge and the manner in which this is catered for in the stiffness matrix and the applied load vector is also given in the next section.

At each junction ij of a frame, let f_{nij} and f_{oij} be respectively the frame loads at the formation of the previous and current hinges in the structure. The load increment at this junction is $f_{nij} - f_{oij}$ and the increment Δ_{pij} in the external load (acting on the structure) at this junction is given by:-

$$\Delta_{pij} = (f_{nij} - f_{oij}) / \alpha_{nij} \qquad (2.12)$$

From this the applied load factor λ_n at the formation of the current hinge is calculated. Once again, the smallest of these obtained from all the frames is selected.

(xi) - The whole process is repeated until the determinant $|K|$ of the stiffness matrix for one of the frames becomes negative indicating the failure of that frame and hence of the structure bility of each frame, failure usually takes place prior to the formation of a mechanism either in one frame or in the structure as a whole.

The iteration process, for tracing the load deflection history of the entire structure up to collapse, is noticed to be from one hinge to the next. No intermediate analysis between the formation of two successive hinges is necessary. In this manner, a considerable amount of computer time is saved.

Notice that because a general matrix is used for the stiffness of the slabs and the shear walls, it is necessary to carry out a fresh influence coefficient analysis for each hinge. This is necessary in an accurate three dimensional analysis of the structure in order to detect the redistribution of the loads. Davies (58) excluded the stiffness of the slabs in θy and θx directions. The stiffness of the slabs in θy direction is far too large to be neglected. Furthermore, the plane frame approach of Davies excludes the use of any shear walls.

2.4 Load and Stiffness Matrices

To save computer storage, the overall stiffness matrix, for a frame with hinges, is constructed in the manner recommended by Majid and Anderson (23). Fig. (2.3) shows a part of this matrix for the first three joints and three hinges near joints 1 and 3 of a frame. This matrix is simplified considerably by utilising the orthogonal nature of the frames. When using the irregular band width elimination method of Jennings (20), only the portion bound by the double lines is stored and operated upon.

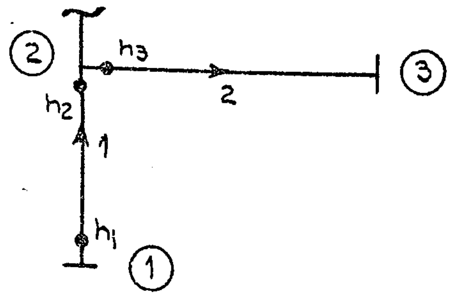
The matrix is used repeatedly in two different manners. Firstly, it is used when the influence coefficient matrix $\underline{F}$ for columns in the applied load vector is equal to the number of storeys in the frame. Each load column consists of zeroes except for a unit horizontal load at the beam level of the floor. The joint where this load is applied is always numbered first. This is to simplify the scanning operation when specifying the row number corresponding to a unit load. Thus, if J_{ij} is the joint number at which the unit load is applied to frame i at floor level j, the row number corresponding to the unit load is given by:-

$$r_{J1} = 3J_{ij} - 2 + \sum_{k=1}^{J_{ij-1}} f_k \qquad \ldots\ldots (2.13)$$

where f_k is the total number of hinges round joint k. In equation (2.13), the summation for $J_{ij} = 1$ is disregarded.

For example, the unit load matrix for the two storey frame, shown in fig. (2.4), is given as:-

$$\underline{L} = \left[\begin{array}{cc} 1 & 0 \\ 0 & 0 \\ 0 & 0 \\ \hline 0 & 0 \\ \hline 0 & 0 \\ \hline 0 & 0 \\ 0 & 0 \\ 0 & 0 \\ \hline 0 & 1 \\ 0 & 0 \\ 0 & 0 \\ \hline 0 & 0 \\ \hline \end{array}\right] \begin{array}{l} \\ \text{Joint 1} \\ \\ h_1 \\ h_2 \\ \\ \text{Joint 2} \\ \\ \\ \text{Joint 3} \\ \\ h_3 \end{array} \qquad \ldots\ldots (2.14)$$



	joint 1			hinge 1	joint 2			hinge 2 & 3		joint 3		
	R_1	y_1	θ_1	θh_1	z_2	y_2	θ_2	θh_2	θh_3	z_3	y_2	θ_3
Z_1	a_1											
Y_1		b_1										
M_1	$-c_1$		d_1									
M_{H1}	$-c_1$		d_1	d_1								
Z_2	$-a_1$		c_1	c_1	$a_1 + b_2$							
Y_2		$-b_1$				$b_1 + a_2$						
M_2	$-c_1$		e_1	e_1	c_1	$-c_2$	$d_1 + d_2$					
M_{H2}	$-c_1$		e_1	e_1	c_1		d_1	d_1				
M_{H3}						$-c_2$	d_2		d_2			
Z_3					$-b_2$					b_2		
Y_3						$-a_2$	$-c_2$		c_2		a_2	
M_3						$-c_2$	e_2		e_2		c_2	d_2

$a = 12EI\,\phi 5/L^3$ $\qquad$ $b. = EA/L$

$c = 6EI\,\phi 2/L^2$ $\qquad$ $d = 4EI\,\phi 3/L$

$e. = 2EI\,\phi 4/L$

Fig. 2.3. Contributions of two members with three hinges to the stiffness matrix of a rectangular frame.

The supports of a frame are fixed and make no contribution to the stiffness matrix. They are all numbered zero in (fig. 2.4). Since each hinge at this stage is treated as a real one, its contribution to the load matrix consists of zeroes which appear under the headings h in equation (2.14).

During the elasto-plastic analysis, the matrix of figure (2.3) is also used repeatedly. On this occasion, the loads are presented in a single vector with the reduced plastic hinge moments M_p', for the hinges appearing at the appropriate places in this vector. These are shown as M_{H1}, M_{H2} etc., in figure (2.3). The reduced plastic hinge moments $M'p$ are obtained from equations (1.4a) and (1.4b), for the universal sections. For the rectangular solid sections, such plastic moments are obtained from:-

$$M_p' = (Z_p - 0.25 \times w^2/B) \times f_y \qquad \ldots\ldots (2.15)$$

where

$w = |x| / f_y$. Here x is the axial force in the member, Z_p the plastic section modulus , B is the width of the section and fy is the yield stress of the section.

The stability functions $\phi_2 - \phi_5$, shown in fig. (2.3), are calculated from current values of the axial loads in the members. Suppose that the load factor at which a hinge develops in frame s is λ_s, that for frame q is λ_q, with λ_q being the lowest load factor λ_L. For frames other than q, it is necessary to recalculate by simple proportion the forces in every member for the case when $\lambda_L = \lambda_q$ is acting on the structure. Thus the axial load x_m in member m o f frame s at λ_L is calculated from:-

$$x_m = x'_m \times \lambda_L / \lambda_s \qquad \ldots\ldots (2.16)$$

x'_m ... the force in member m when frame s is subject to λ_s.

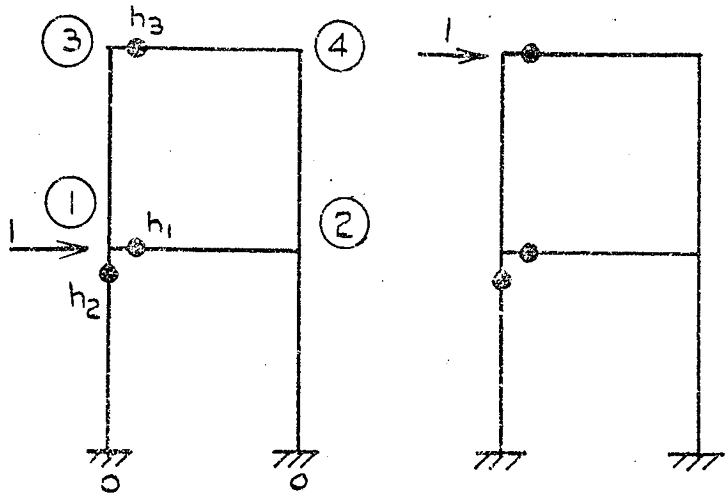


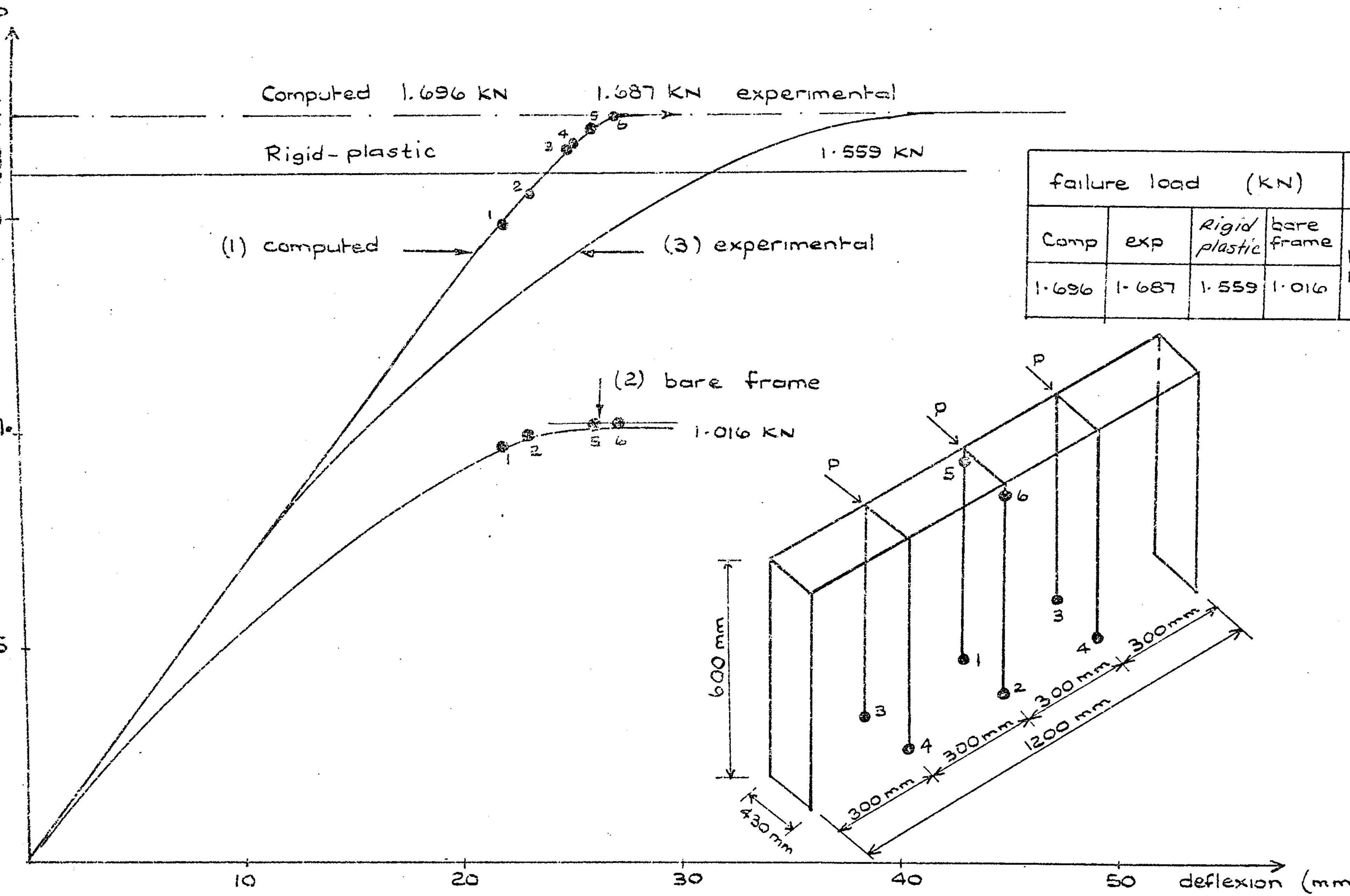
Fig. 2.4. Unit loads acting at alternative floor levels

2.5 Elasto-Plastic Analysis of a Single Storey Structure

As an application, the simple structure shown in figure (2.5 was analysed. For experimental reason (see Chapter 5), the grillage system is made of perspex consisting of two 12.7 mm thick shear walls and a 7.94 mm thick slab. The frames are made of 12.7 mm x 12.7 mm black mild steel bars with Mp = 152.475 KN mm. The overall dimensions of the complete structures are shown in the figure with the slabs being 130 mm wide.

The result of the elasto-plastic analysis of the structure is shown in the figure by graph (1). The first two plastic hinges developed in the central frame. These reduced the capacity of the frame to sustain its share of the wind loads. More load was thus transmitted via the slabs to the outer frames and the shear walls. This caused the second set of hinges 3 and 4 to develop in the outer frames. These too became more flexible and threw the loads back to the grillage system. The complete structure finally collapsed at a load p = 1.696 KN after the formation of two more hinges in the central frame.

In figure (2.5), the elasto-plastic load deflection diagram for one of the bare steel frames is also shown. It is observed that the carrying capacity of the complete structure is 66% higher than that of the bare frame. This structure was also analysed by calculating the initial α_o value for the central frame which was then used in a rigid plastic analysis. This gave the failure load for a sway mechanism as 1.56 KN indicating that α_o exaggerates the amount of load transmitted to this frame and thus this analysis under-estimates the failure load. The experimental load deflection curve presented by graph (3) will be ... in Chapter (5) while discussing the results ob-



failure load (KN)			
Comp	exp	*Rigid plastic*	bare frame
1·696	1·687	1·559	1·016

Fig. 2.5 Load - deflexion diagram for structure 3

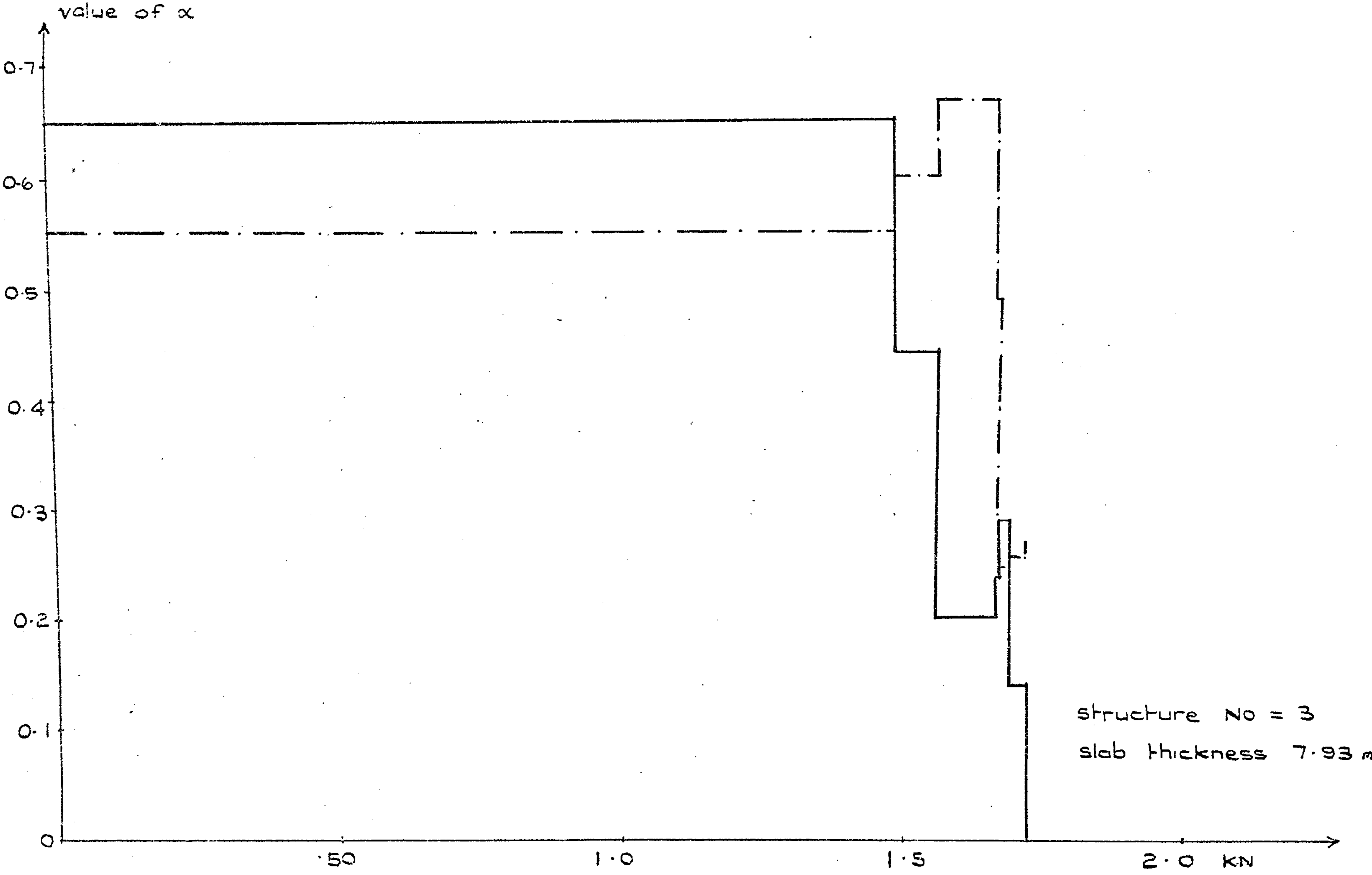


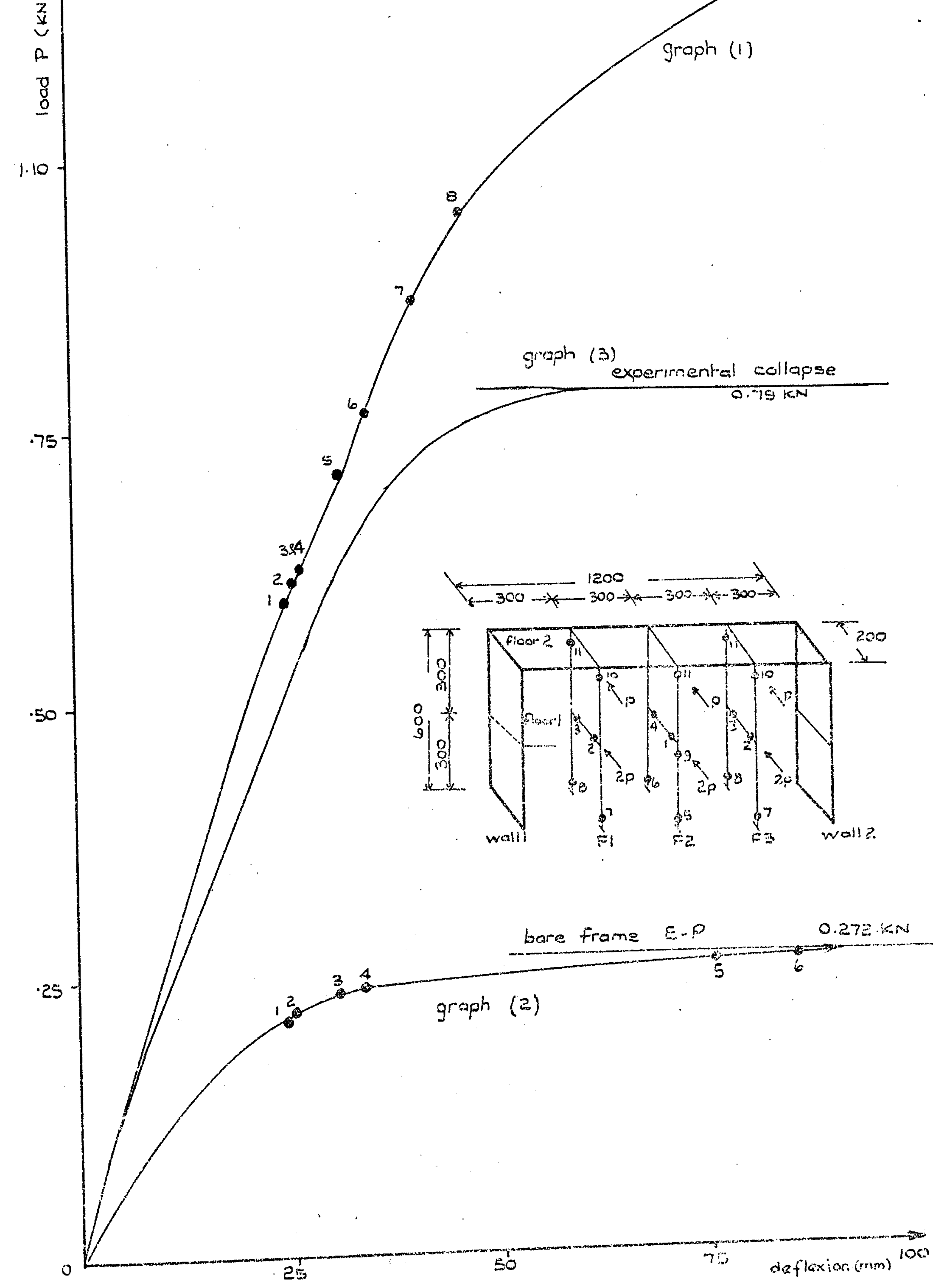
Fig. 2.6 Variation of α with applied load

An investigation into the manner in which the value of α in each frame deteriorates is shown in figure (2.6). As the first hinges develop, the load share of the central frame deteriorates while loads transmitted to the outer frames increase. The figure shows that as plasticity develops in the outer frames, their share of the load also dwindles.

2.6 Elastic-Plastic Analysis of Two Storey Structure

For a further example, the two storey structure shown in figure (2.7) was analysed. For experimental reason (see Chapter 5), the floors and the walls are fabricated from perspex consisting of two 15.875 mm thick shear walls and two 7.94 mm thick slabs. The frames are manufactured from 12.7 x 12.7 mm black mild steel bars with a plastic moment of 122.56 KN mm. The overall dimensions of the structure are shown in the figure with the slabs being 180 mm wide.

Figure (2.7) shows the results of the elastic-plastic analysis of the structure. The first plastic hinge took place in the beam in mid floor of the central frame. This reduced the load bearing capacity of the frame and, therefore, more load was transmitted by slabs to the other frames and shear walls. This resulted in the development of a second and third set of plastic hinges in the outer frames. Formation of these two groups of hinges in the outer frames threw more loads back to the central frame and caused the hinges 4, 5 and 6 to develop in frame 2. These hinges again weakened the central frames and reduced its capacity of holding up its share of the loads, so more loads were transferred to both shear walls and outer frames via floor slabs. Consequently, hinges 7 and 8 developed in frames 1 and 3



deflexion graphs for two storey structure.

This was followed by the formation of hinge 9 in the middle frame, and hinge 10 in the outer frames. Eventually, the structure collapsed at a load of 1.207 KN at the top floor levels after the formation of the 11th group of hinges at all intermediate frames. The last hinges developed in all frames simultaneously and led the outer frames and then complete structure to failure. There were altogether 18 plastic hinges in the whole structure while the intermediate frames had 6 hinges each. Graph (3) presents the experimental load-deflection curve of the structure. This will be discussed in Chapter 5 in more detail.

The elastic-plastic load deflection diagram for one of the bare steel frames analysed independently is shown in figure (2.7 by graph (2). The bare frame had six plastic hinges at collapse and their sequence of formation is also shown with the hinge patterns in the same figure.

In figure (2.8), the deterioration of α values with the formation of plastic hinges is shown. This is given only for the top floor of the outer frames. It can easily be seen from the figure that whenever a hinge occurs in other frames the α values increase and the outer frames take more loads. When a hinge forms in the outer frames, then their α values decrease since these frames become weaker than before. For instance, the first hinge developed in the central frame and caused a sudden increase in α value 0.06 in the outer frames. On the other ha the formation of hinges 2 and 3 resulted in sudden drops in α as shown in the figure. It is shown in the figure that the formation of the 11th group of hinges in the structure caused the value to drop from 0.27 down to zero. At this stage, it can be

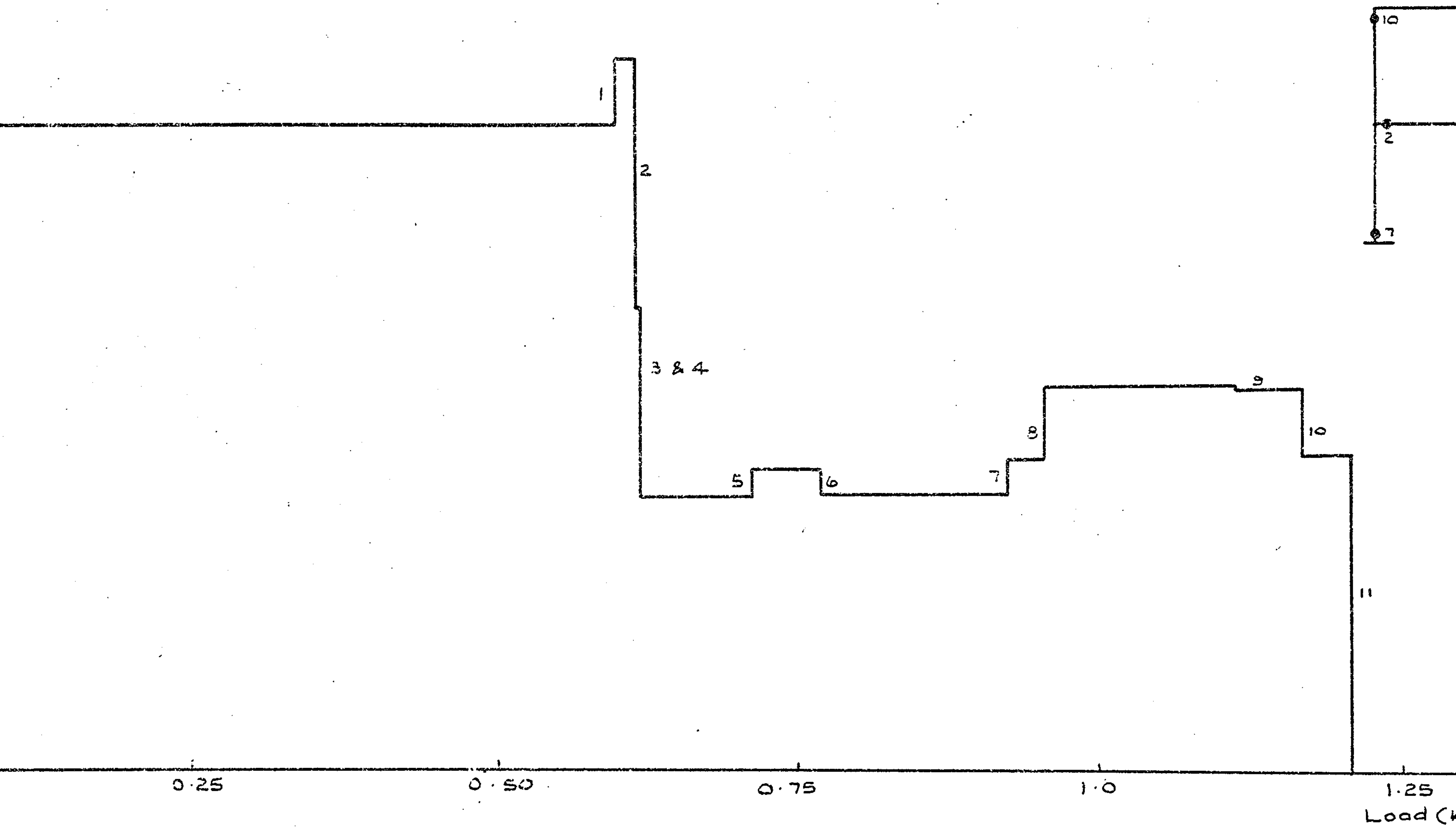


Fig. 2.8 Variation of α for hinges (1st floor 2nd storey)

said that these frames are no longer stable and cannot take any extra loads at all.

2.7 Analysis of Large Structures:

2.7.1 Four Storey Structures

The first group of examples in practical structures analysed by the proposed method consists of 3, 5, 7 and 10 four storey single bay intermediate steel frames connected by reinforced concrete slabs of 152.4 mm thickness which are connected at either end to two reinforced concrete shear walls of 304.8 mm.

In these examples, all the dimensions, the applied external loads and the section properties of the intermediate frames are the same. As an example, a structure consisting of 10 internal frames will be given. Figure (2.9) shows the dimensions of the structure and the frames as well as the working vertical and horizontal loads, while the sectional properties of the frames are listed in table (2.1).

Because of the symmetrical arrangement of the intermediate frames, plastic hinges develop in pairs and symmetry is preserved throughout the analysis. The full effect of the axial loads in reducing the stability of the frames and the plastic hinge moments of the sections was considered. The pattern of hinge formation is striking. This is shown in figure (2.9) where it is seen that hinges spread from the interior of the structure outwards. The first two plastic hinges develop simultaneously in the first floor beams of frames 5 and 6.

Altogether 66 hinges develop before collapse with a maximum of 8 hinges in frames 5 and 6. Only four hinges develop in the outer frames. Collapse takes place before a mechanism develops in any frame. This is because of the loss of stiffness of

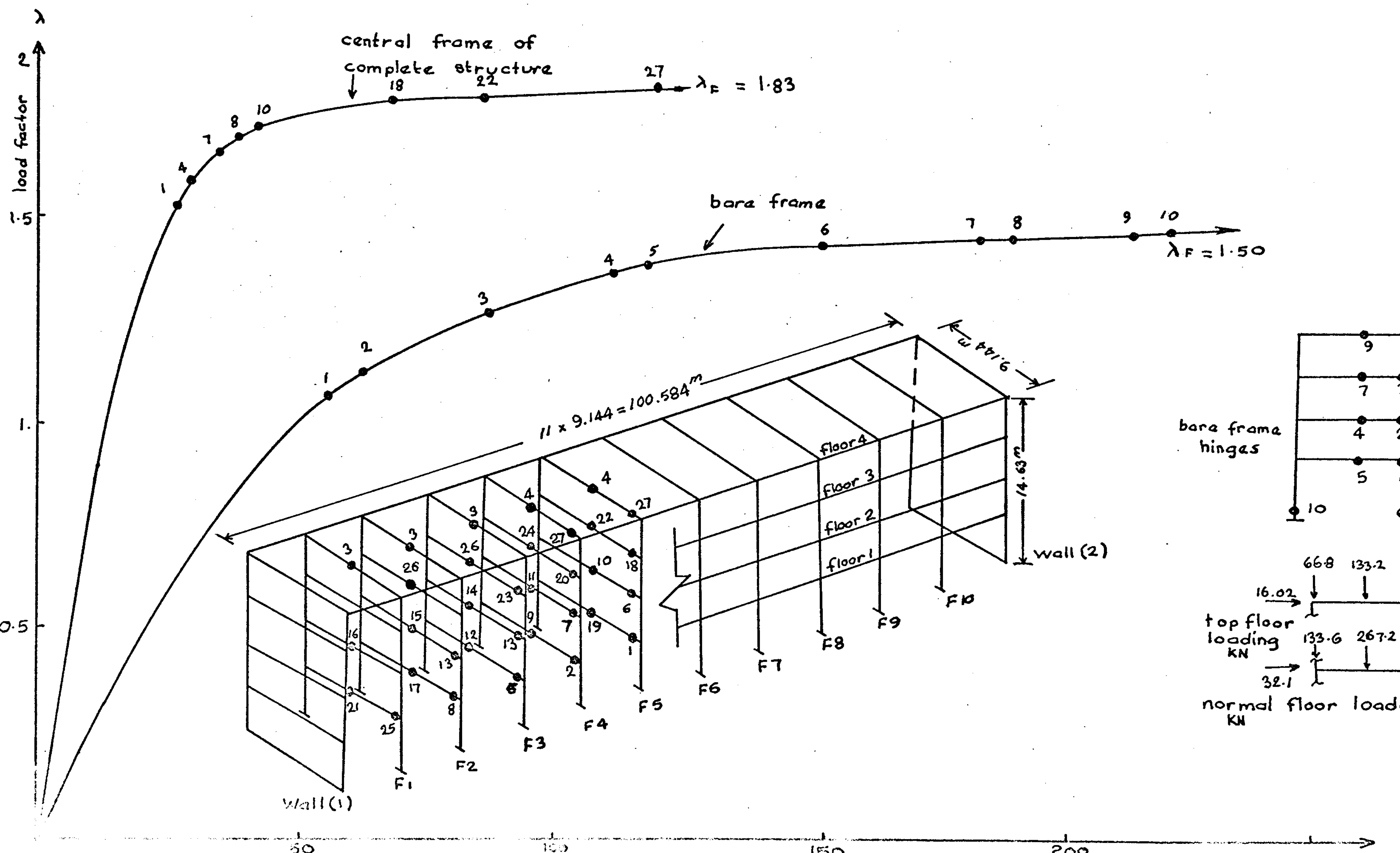


Fig. 2.9 Four storey structure

MEMBER	SECTION
BEAMS	406 x 178 x 60
GROUND FLOOR COLUMNS	305 x 305 x 118
OTHER COLUMNS	254 x 254 x 89

TABLE 2.1

SECTIONS USED FOR THE FRAMES OF THE FOUR STOREY STRUCTURES the frames due to axial load effects. All the hinges develop in the beams because of excessive vertical loading. This also indicates that while the vertical loads are transmitted to the foundations, via the frames, most of the wind loads are transferred by the slabs to the shear walls. The failure load factor of the structure at 1.83 is 22% higher than that of one of the frames analysed independently. This frame collapsed with 10 hinges, also without the formation of a mechanism.

The results obtained from the analysis of the structures with 3, 5, 7 and 10 intermediate frames are summarised and given in table (2.2). This Table shows the load factors of the structures at collapse and the maximum number of plastic hinges occured in one of the frames within the structure as well as the load factor and the number of plastic hinges at collapse of a single bare frame analysed individually. As can be seen in the table, the load factors at collapse of the first three structures are found to be nearly the same which are 1.89 and 26% higher than that of one of the bare frames analysed independently. This is just because of the excessive vertical loads acting on the intermediate frames.

2.7.2 Six Storey Structures

The second group of examples in practical structures consist of complete structures with 3, 5 and 7 intermediate frames of two unequal bays. Bay one is 6.098 m while bay two is 3.049 m. The reinforced concrete slabs were 152.4 mm thick while the shear walls were 3048 mm thick. The dimensions and the applied working loads and the sectional properties are the same for all the frames. As an example, a structure consisting of seven intermediate frames of two unequal bays, shown in fig. (2.10), will

Number of Intermediate Frames	Failure Loads	Maximum No. of Hinges in One Frame	Total No.of Hinges in Structure
3	1.89449	6	16
5	1.89022	8	36
7	1.89433	8	36
10	1.82555	8	66
Bare Frame	1.50	10	10

TABLE 2.2

ANALYSIS OF FOUR STOREY STRUCTURES

be given. The dimensions and the applied working loads are shown in fig. (2.10) and table (2.3) gives the sectional properties.

Once again the structure collapsed before the formation of a mechanism in any frame. Failure took place at a load factor of 1.765 under combined loading with a total of 103 hinges as shown in fig. (2.10). It is noticed that because of the symmetrical nature of the frames, due to unequal bays, the sway deformations of the structure are aggravated and as many as 18 hinges develop in the columns. The failure load factor under combined loading of the whole structure was once again higher than that of a bare frame by 24%.

The results obtained from the analysis of the structures in this group are shown altogether in the table (2.4). This shows not only the load factors, the total number of plastic hinges developed in the structure and the maximum number of plastic hinges in one of the intermediate frames of the structure at failure but also the number of plastic hinges and the load factor at collapse of one bare frame analysed independently.

As shown in the table, the collapse load factor of the structures with the increasing number of the internal frames were found to be nearly the same. This was, once again, because of the excessive vertical loads acting on the bare frames, and also because of the horizontal loads, most of which are taken by the floor slabs and transmitted to the shear walls. All structures in this group of examples collapsed before the development of a mechanism in any of the frames.

To reduce the computer time, the facility of inserting more than one hinge simultaneously was resorted to with both struc-

	MEMBERS	SECTIONS
Beams U.B.	2,7,12,17,22	305 x 165 x 40
	4	254 x 146 x 31
	9	254 x 146 x 37
	14	203 x 133 x 30
	19,24,29	203 x 133 x 25
	27	203 x 133 x 30
Columns U.C.	1,8,13	254 x 254 x 73
	3	305 x 305 x 97
	5,6	203 x 203 x 60
	10,11	203 x 203 x 52
	15	152 x 152 x 37
	16	203 x 203 x 46
	18	203 x 203 x 52
	20,21,23,25,26,28,30	152 x 152 x 37

TABLE 2.3

SECTIONS USED FOR THE FRAMES OF THE SIX STOREY STRUCTURES

Number of Intermediate Frames	Failure Loads	Maximum No. of Hinges in one Frame	Total No. of Hinges in Structure
3	1.76678	14	42
5	1.76794	14	68
7	1.76501	17	103
Bare Frame	1.42	16	16

TABLE 2.4

ANALYSIS OF SIX STOREY STRUCTURES

tures. At one time six hinges were inserted in the four storey structure consisting of 10 intermediate frames and seven in the six storey one with 7 internal frames. In figures (2.9) and (2.10) it is seen that simultaneous hinge development takes place in the early stages of loading. In the advanced stages nearer collapse, hinges are inserted one at a time (two at a time to preserve symmetry). Thus the accuracy of the analysis in the advanced stages was not tampered with.

Computer time is also saved by taking into consideration the symmetrical arrangement of shear walls and frames. In this case, only half of the total number of the frames within the structure is considered, including the frame on the axis of the symmetry, if there is any. For instance, only five frames out of ten frames are analysed in the example on the four storey structure shown in fig. (2.9), and four frames only are analysed in the example on the six storey structure shown in fig. (2.10).

2.8 Conclusion

The carrying capacity of a building depends considerably upon the relative stiffness of the grillage system and the frames at all stages of the loading process. Investigation into the variation of the load transmitted to a frame by evaluating its α value (shown in figures (2.6) and (2.8)) indicates that, as hinges develop, a proportional increase in the applied external loads does not lead to a similar increase in the loads carried by each individual frame. It is this fact that makes a fresh calculation of α , each time a hinge develops, imperative.

Analysis of the large practical frames indicate that most of the wind loads are transferred by the slabs to the shear walls and then to the foundations. The vertical loads on the

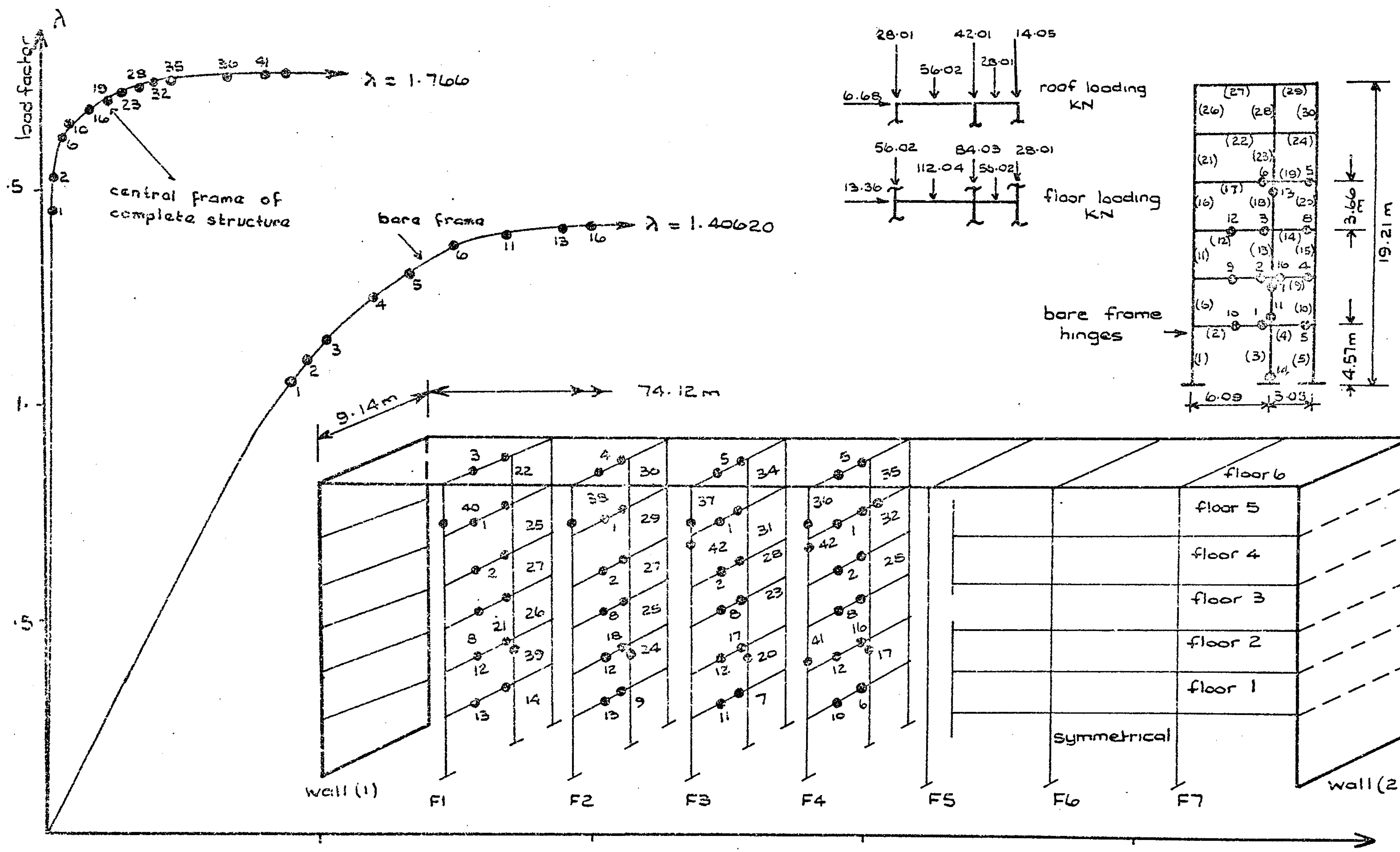


Fig. 2.10. Six storey structure

other hand are carried mainly by the frames. This was also concluded by Creasy (55). However, with vertical loading, the accuracy of the elasto-plastic analysis can be improved by including the composite effect of the slabs and the frames. This will be paid special attention later in Chapter 6.

CHAPTER 3

COMPUTER PROGRAMME FOR THE ELASTIC-PLASTIC ANALYSIS OF COMPLETE BUILDING STRUCTURES

3.1 Introduction

A computer programme for the analysis of the complete structures was written in Atlas Autocode. This computer programme will be described and a flow diagram, together with the steps followed will also be given in this Chapter.

3.2 Elastic Analysis of Structure

It should be noted that an elastic analysis of the structure (51,52) is required in order to provide the data for the subsequent elasto-plastic analysis of the frames. This analysis gives the external horizontal loads transmitted to the frame members at each junction with the floor slabs. This also gives the influence coefficient matrix $\underline{G}$ of the grillage and the column vector of $(\underline{G}\ \underline{P} + a)$ of equation (2.5). The horizontal external loads acting at each floor level of the frames and matrix $\underline{G}$ together with $(\underline{G}\ \underline{p} + a)$ have to be found in advance before an elastic-plastic analysis can be carried out. The rest of the information needed for the structure is obtained from the geometry and the member properties of the structure.

3.3 Programme Data

The data is divided into two groups. One is the data concerning the overall properties of the general structure. The second is the data which is relevant to the intermediate frames.

In the general data (Block 1 of figure 3.1), the symmetry in the arrangement of frames and shear walls is specified. If there is a symmetry, then 0 is used to indicate the symmetrical arrangement. Otherwise, 1 is used for the unsymmetrical arrangement. The type of sections used are also defined, in case of universal section integer 1 is indicated, while integer 2 indicates that the sections are rectangular. The total number of different sections used in the intermediate frames throughout the structure is also read in as data.

The maximum number of joints, the maximum total number of members in any of the frames, Young's Modulus of Elasticity of steel, the applied tolerance, the unit load factor and the maximum number of plastic hinges expected to develop in the analysis up to collapse are also fed into the data. These are followed by the total number of frames and the total number of storeys in the structure. A single load acting at one mid floor level on the structure and the increment to be used in the iterations are also fed in.

The sectional properties of the different sections used in the structure are given. For the universal sections a total of 7 properties are needed. These are the cross-sectional area, second moment of area, the plastic section modulus and the other relevant constants given in the "safe load tables" (26). These are used to calculate the reduced plastic hinge moments of the sections. For the rectangular solid sections only five properties are required for each different section. These are the width and the depth of the sections, apart from the cross-sectional area, second moment of area and the plastic section modulus of the sections.

Following the above information, the column vector ($\underline{G}\ \underline{p} + \underline{a}$) and the influence coefficient matrix of the grillage system, $\underline{G}$, are fed in. The vector ($\underline{G}\ \underline{p}$ + a) is of order (m x n) and $\underline{G}$ is of order $(m \times n)^2$ where m, n are the total number of frames and the total number of storeys respectively. The latter matrix is full which generally has no zero elements. These two matrices become large in size when dealing with large structures. This raises an important problem in solving equations (2.5) and (2.10) by requiring extremely large computer storage and time.

The general data of structure includes the above information. The rest of the information will be given in the data for the intermediate frames.

The second part of data, block 2 of figure (3.1) starts with the total number of real hinges, the total number of joints and the total number of the members. If there are any real hinges in any member of the frame, then the number of the hinges at one end (or either end) of the member have to be stated.

The length, joint numbers of first and second ends and the inclination of the members are given in the data. This is followed by the number of different section types used in each frame. The external loads acting on each joint, in turn, in z, y and θ_x directions are also given in the form of a row vector. This load vector is of order $(3m_s + h_s)$ where m_s and h_s are the total number of joints and the total number of hinges in frame s. The yield stress of each section is also fed into the data to be used in the calculation of the reduced plastic section modulus and hinge moments. The lowest joints of each storey of the frame are also given. This is used in the construction of the unit load matrix. In the second part of the data, the information is given for one frame at a time.

3.4 Description of the Programme

After feeding both the general and the frame data into the programme, the programme procedure is commenced. For each frame in turn the maximum size of the stiffness matrix and the other arrays to be used throughout the analysis are found. If there is no real hinge in the frame, then the hinge numbers at both ends of the members are set to zero, otherwise they have to be read in. The total number of plastic hinges that can possibly form around each joint is calculated. This depends upon the number of members connected to each joint. The lowest non-zero joint connected to each joint is calculated and used in constructing the overall stiffness matrix in the "subroutine stiffmatt". The locations of the elements in the "address sequence" are calculated and the total number of locations required by the "main sequence" is determined. Meanwhile, the initial load parameter is set to zero while the current one is put equal to unity. All these are preparations for the analysis (Block 2 of Fig. (3.1)).

By taking one frame at a time, the member forces are set to zero so as to start the iteration towards the first plastic hinge. By using the zero axial forces, the contribution of each member to the submatrices $\underline{K}_{aa}$, $\underline{K}_{ba}$ and $\underline{K}_{bb}$ are found. The stiffness equations are solved by a technique given by Jennings (20), (Block 4 of Fig. (3.1)). In the first iteration, the "stability functions" and the applied load factor are taken as unity. From this solution the resulting joint displacements are found which are used in the calculation of the new member forces by using equations (1.6). The reduced plastic moment of the section is found by equations (1.4a), (1.4b) or (2.15) depending upon the type of section used. These member forces and the reduced plastic hinge moments are utilised in the prediction of the load factor at which a hinge will possibly form in the frame. The

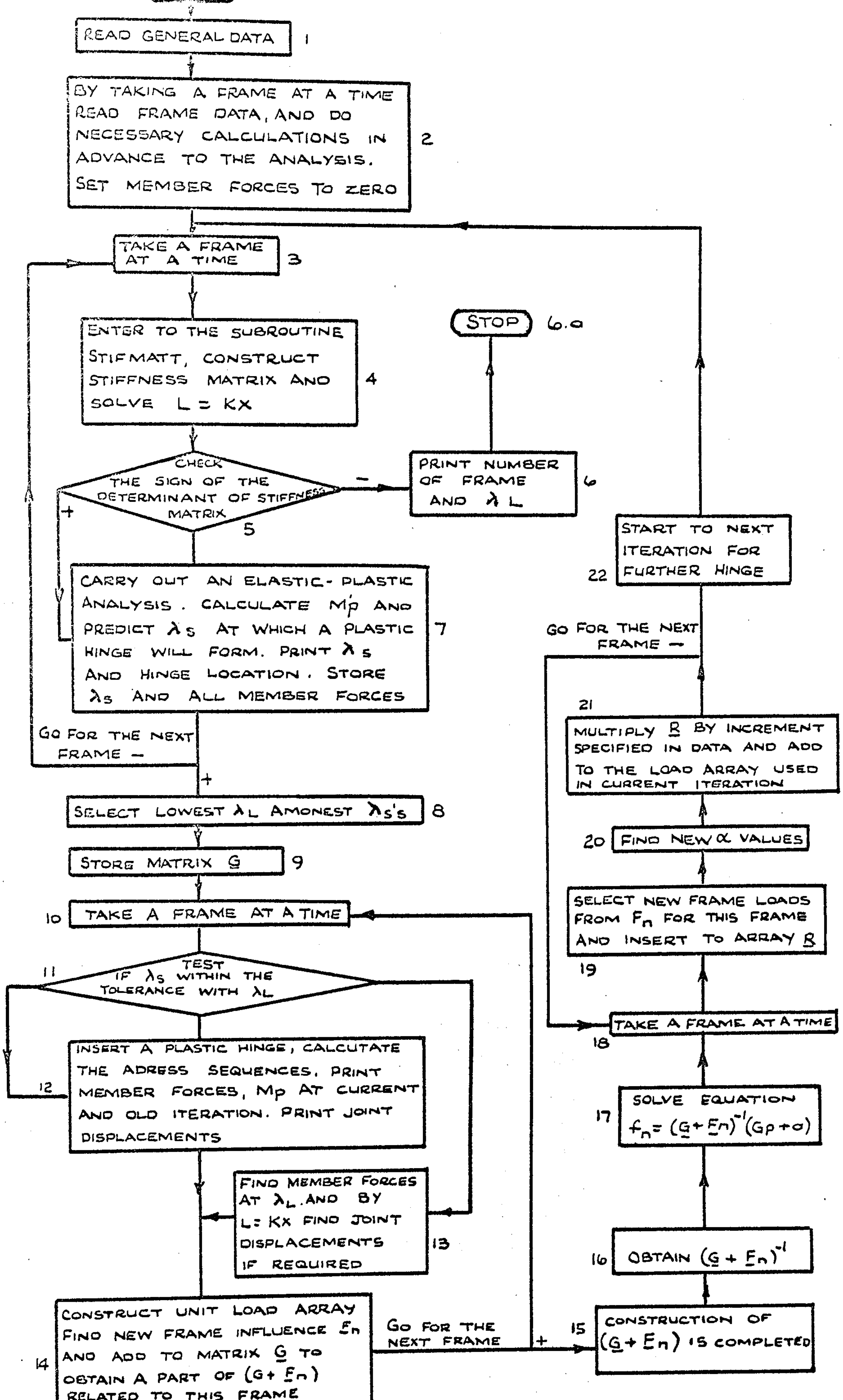


Fig. 3.1. The flow diagram

load factor is predicted by eqn. (1.3), and the member axial loads at this predicted load parameter are calculated by eqn. (1.5). The predicted load parameter is compared with the current load parameter. If this is within the specified tolerance, then a plastic hinge can be inserted in this frame. Otherwise, the old load parameter is set to the current one, and the current member axial loads are replaced by the predicted axial load and a further iteration is started. By using new predicted axial loads, the stability functions $\phi_2 - \phi_5$ are calculated. This time the stability functions have values different from unity. The overall stiffness matrix is constructed once again and solved to yield the new set of joint displacements. The new displacements of the joints are used in the calculation of the member forces which are utilised in the prediction of the new load factor and the axial loads. If the predicted load parameter is within the tolerance of the current load factor, a plastic hinge can form in a certain member of the frame. If not, this iteration continues until the predicted load parameter satisfies the tolerance test. When the tolerance test is satisfied, then the predicted load parameter at which a plastic hinge will be inserted to a frame and the location of plastic hinges are printed out with the number of intermediate frames. The member forces are stored for later use in the programme.

This is carried out for each frame in turn, and the location of a plastic hinge with its load factor are printed out. After each frame is analysed, the load factors at which each frame may have oneplastic hinge each are compared to one another, and the smallest load factor is found amongst them (Block 8 of Fig. (3.1)). This lowest load factor is compared with the load factors for the other frames, (Block 10 of Fig. (3.1)). If any load factors λ_s for frame s is within the stipulated tolerance to λ_L, then a plastic hinge is inserted to this frame too, (Blocks 11,12). The factor λ_L is selected as the load factor at which a plastic hinge forms anywhere in the structure.

If a plastic hinge is inserted to any of the frames, the size of the stiffness matrix changes immediately. The plastic hinge moment is included in the load vector corresponding to this frame. The plastic hinge is included in the stiffness matrix in the manner described briefly in Chapter (1). Inclusion of a hinge causes a change in the number of locations required to store the stiffness matrix and the locations of the "address sequences". Hence, these are re-calculated and used in the reconstruction of the overall stiffness matrix.

The inclusion of any plastic hinges in any of the frames results in a change in the behaviour of the structure. Therefore, a new set of frame influence coefficients $\underline{F}_n$ of equation (2.10) is calculated (Block 14 of Fig. (3.1)). To do this, the unit load vector is constructed by considering one floor at a time in a frame in the manner described in Chapter (2). At this stage, the overall stiffness matrix of the frames are obtained by using zero axial forces in the members. The equation $\underline{F} = \underline{K}^{-1} \underline{L}$ is solved as many times as the number of storeys in each frame. The solution yields the deflections at the lowest joint number at each floor level in z , x and θ_x directions as shown in figure (2.2).

For the frame influence coefficient matrix $\underline{F}_n$, only the displacements in the z direction are required and these are selected amongst the others. The matrix $\underline{F}_n$ is of order $(m \times n)^2$, half of its elements are zeroes. Only its non-zero elements within the matrix $\underline{F}_n$ are detected together with their locations. Once these are found, they are added to the elements within the corresponding locations inside matrix $\underline{G}$. In this manner, only the non-zero elements with their locations are stored, and the total number of locations required by $\underline{F}_n$ is highly reduced. At each frame only the related parts of $(\underline{G} + \underline{F}_n)$ are found (Block 14 of Fig. (3.1)), and by analysing each intermediate frame in turn, the construction of $(\underline{G} + \underline{F}_n)$ is completed (Block 15 of Fig. (3.1)).

This is followed by a matrix inversion routine to obtain $(\underline{G} + \underline{F}_n)^{-1}$, (Block 16). This is used in solving equation (2.10) which yields the new sets of frame loads $\underline{f}_n$, (Block 17 in Fig. (3.1)). $\underline{f}_n$ is a column vector which contains n groups and each group includes m number of elements. Here m, n are the total number of frames and floors respectively. The loads acting at each floor level in each frame in turn are selected from $\underline{f}_n$ and they are placed into the new load array $\underline{R}$ which is of the same order as the load array fed in the data. $\underline{R}$ is initially a null matrix, eventually some of the zero elements in certain rows which correspond to the lowest joint at each floor are substituted by the actual values obtained from the load vector $\underline{f}_n$, (Blocks 18 and 19 in Fig. (3.1)). The row numbers are found by:-

$$\Pi_{ri} = 3J_{ri} - 2 \qquad \text{......} \quad (3.1)$$

where J_{ri} is the lowest joint at which one external load is applied to a frame at floor r.

The new α values are obtained from the loads obtained by using the equation (2.11), (Block 20 of flow diagram shown in Fig. (3.1)). To find the α values at each floor, except the top floor, the new loads are divided by the single load given in the general data of the structure. The value of α at a floor level is found in the same way, but the new load at the top floor is halved. The α values are placed in a matrix which is of order m x n. Here m and n are the total number of the intermediate frames and the storeys in the structure. These values are used in the calculation of the external loads at each hinge formation.

The load array $\underline{R}$ is multiplied by a small increment specified in the general data and put on the load array $\underline{L}$ used in the previous iteration to commence the next iteration, (Blocks 21 and 22). Before leaving this stage, it must be noted that for the next iteration, the new member forces in each frame are calculated from equation 2.9, (Block 13). To do this, the member forces are recalled from the store and divided by the load factor of the frame at this iteration and later multiplied by λ_L of the structure. The iteration procedure is re-started all over again.

The analysis continues until one of the frames within the structure loses its stiffness after the formation of a sufficient number of plastic hinges, but often before a collapse mechanism develops. At each hinge insertion to any of the frames, the determinant of its overall stiffness matrix is checked. If it is negative, then the analysis is terminated and the frame number which loses its stiffness is printed out.

The horizontal external load at each floor level, in particular, the horizontal load at the top floor of the frame which collapsed in the analysis is calculated and the load deflection graphs are produced as in figures (2.5) and (2.7) for the analysis of the simple structures. In these figures, the sequence and the pattern of the hinge formation are also given. In Chapter (2), a number of examples were solved by this computer programme. In figures (2.9) and (2.10), the results obtained from the analysis of large structures were shown, their load-deflection graphs were given.

3.5 <u>The Subroutine Stiffmatt</u>

The subroutine stiffmatt is similar to that of Majid and Anderson (23). It is modified to allow for the analysis of complete building structures and its flow diagram is shown briefly in figure (3.2). At the beginning of the subroutine, the total number of storage locations is calculated for the considered frame and used in defining the size of the arrays to be used in the operation within the subroutine.

This subroutine is used for two different purposes. It is used in the elastic-plastic analysis of the intermediate frames. Secondly, it is also used in the calculation of the frame influences coefficients $\underline{F}_n$ at each iteration. In the second case, all member forces are set to zero while the actual member forces are being stored in the main programme is utilised. The stiffness matrix is formed by disregarding the axial forces, and the stiffness equations (eqn. 1.1) are solved by another subroutine called "compact div" within the "subroutine stiffmatt". To carry out the elastic-plastic analysis, the actual member forces are taken from their stores and used to construct the stiffness matrix. In the solution of the stiffness equations, the actual load for a frame in question is used. The "compact div" routine is the same subroutine of references (23) and (24) which is written to solve the stiffness equations by a technique given by

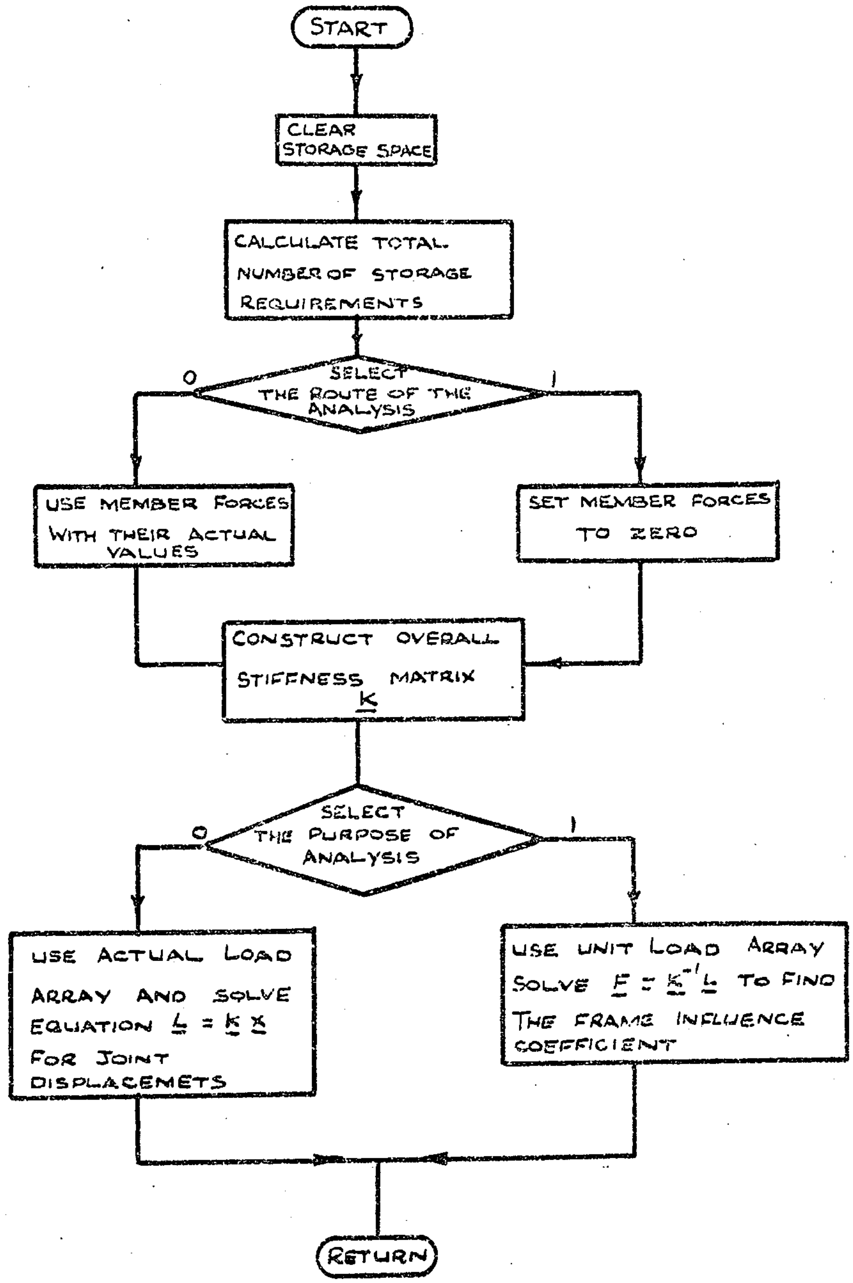


Fig. 3.2. Flow diagram for subroutine stiffmatt

Jennings (20).

3.6 The Effect of Symmetrical Arrangement of Both Shear Walls and Intermediate Frames

The symmetrical arrangement of both shear walls and frames is allowed in the programme. This provides a rather important advantage in the analysis of the structure. Only half the number of intermediate frames are considered. The frame on the axis of symmetry being included. The total number of frames K, to be considered in the analysis is calculated by:-

$$K = \text{int.pt}\,(0.5 \times n + 0.5) \qquad \ldots\ldots \quad (3.2)$$

where n is the total number of the frames within the structure.

The computer time and the total number of storage locations required for such an analysis are considerably saved. On the other hand, the time required for data preparation is also reduced, since only K number of frames are analysed, and their data has to be prepared. In a symmetrical structure, the plastic hinges appear in pairs as shown in figures (2.5), (2.7), (2.9) and (2.10).

When constructing the $(\underline{G} + \underline{F}_n)$ matrix, only the elements of $\underline{F}_n$ corresponding to K frames are found and added to $\underline{G}$. The whole elements of $(\underline{G} + \underline{F}_n)$ are completed automatically from the symmetry consideration.

CHAPTER 4

EXPERIMENTAL WORK

4.1 Introduction

In order to test the validity of the computer programme, a number of tests were carried out. The results obtained from both the theory and the experimental work were compared with each other. Two different groups of tests were conducted. One group was on one storey structures consisting of three frames connected by floor slabs which were supported by two shear walls at either end of the structures. The second group of tests were on two storey structures consisting of three frames. These frames were connected to each other by floor slabs in the same way as for the first group of tests. Altogether, fifteen tests were carried out, eight of which were one storey structures, the rest were two storey structures.

4.2 Preparation of the Structural Model

The frames were constructed from (12.7 mm x 12.7 mm) square section black-mild steel bars except the frames in test 5B which were made out of (9.525 mm x 9.525 mm) square section bars. At each test, the frames were tried to be constructed from a single bar chosen for its evenness. A length of steel bar, long enough for one frame, was taken and the length of each member was marked. The same ends having the same number were welded together with a double V welding to form the frame. With this arrangement, any possible difference in the cross-sectional areas of the members forming a joint was avoided. For the control test, a bar of 508 mm length was cut out of the same bar which was used to construct the frames. At the floor level on the centre line of the beam elements, a steel ring of 10 mm diameter was welded for the application of the loads in vertical direction. The length of the columns were made 50 mm longer than the columns of other storeys. These extra lengths were used in fixing the frames to the test bed mounted on the labaratory wall. The frames were fixed to the plates of the test bed by two steel angles at the foot of each frame as shown in fig. (4.1). and plate 1. Two 12.7 mm x 3 mm nests were engraved on the horizontal arms of the plates and the foot of the frames were inserted in them. The two angles were then bolted together. The other arms of the angles were bolted firmly to the bed plates in order to provide the rigidness at the bases. This type of connection is shown clearly in figure 4.1 and plate 1. In test 5B of one storey structures, the same size of angles and connection at the base were used, but the nests were of 9.525 mm x 3 mm.

The shear walls and floors were manufactured from the "Acrylic" perspex sheets in different thickness. The walls were thicker than the floor slabs. The connection between the walls and floors were made by using a special glue for perspex. A strip of perspex was glued to the inside corner of each wall-floor junction. This strip of perspex was bolted by means of a number of (3 mm) bolts to the wall and the floor. In this manner, a rigid connection was obtained. To provide the fixity and avoid the cracking at the base of the walls, a strip of perspex was glued to the base of the wall on either side. These strips were of (50 mm x width of the wall). The overall length of the walls were also made 50 mm longer at the bottom storey. These extra lengths were used when fixing the walls to the base plates. The base plates of the walls were made of strips of bright mild steel angles. The foot of the wall, at which two

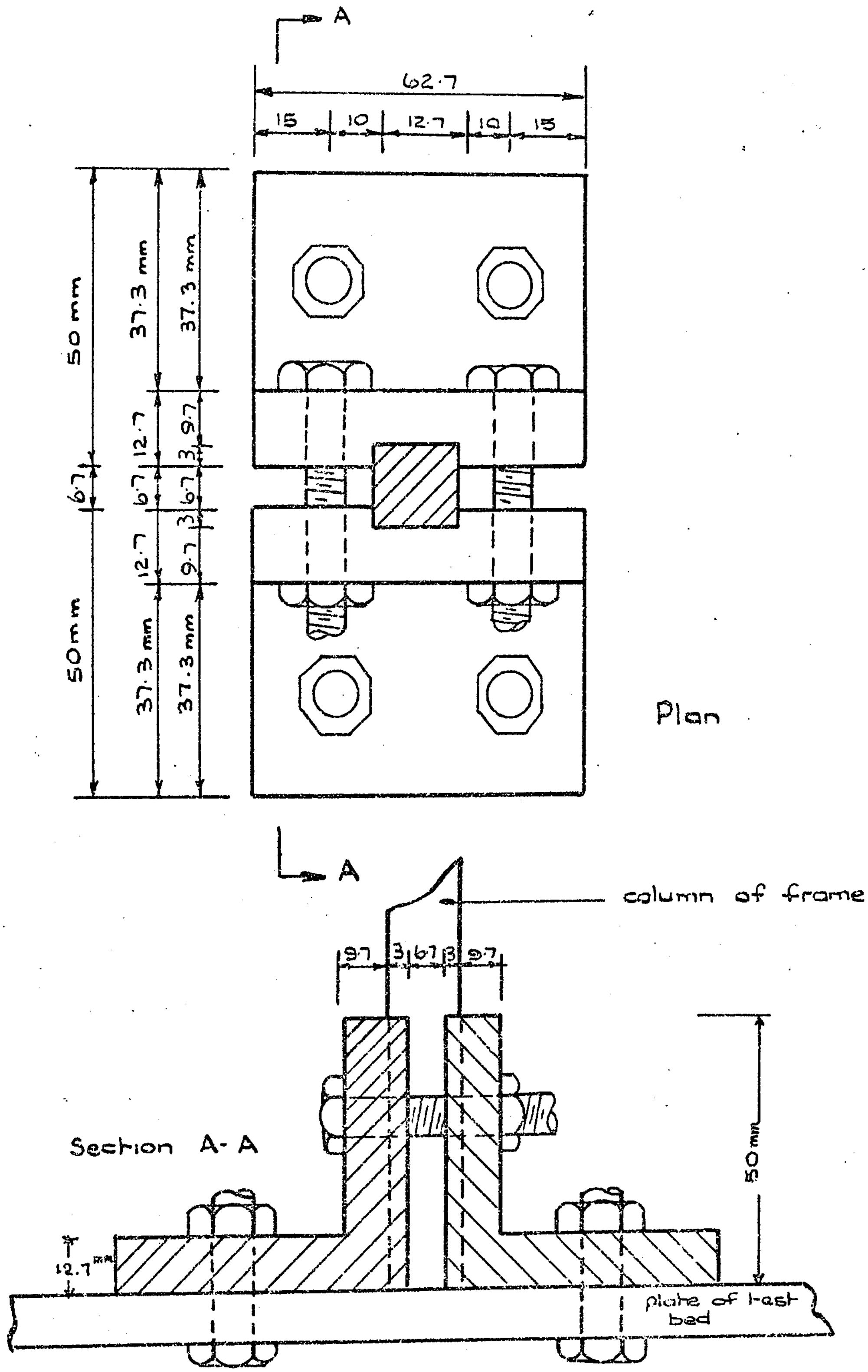


Fig. 4.1. Detail of a fixed base

strips of perspex glued on was inserted between two base plates and were bolted firmly by using (12.7 mm) bolts at 50 mm centre as shown in figure 4.2. The depth of these plates was 50 mm and the thickness was 12.7 mm. The width and length could be anything provided they were not less than the width of the shear walls. The floor slabs were bolted to the beams by using 3 mm bolts.

The model structure was fixed to the test bed. The test bed was made out of 25.4 mm thick steel plate. As can be seen from Plate 2, this plate was securely bolted to two I beams running longditudinally along the laboratory wall. It was considered to be important that no movement of the wall should be allowed during the loading process. Therefore, great care was taken in ensuring that all the components were firmly bolted. The frames were first fixed between the angles and the other arms of the angles at the foot of the frames and were bolted securely to the plate of the test bed. In order to set up the shear walls for uniform fixity, the walls were fixed between the angles and the bolts were tightened. The other arms of the angles were attached to the test bed with the same size of bolts as those used in fixing the walls.

A Dexian frame was built and mounted above the structure to be tested, as can be seen in Plates 2 and 4. This was used to hold the dial gauges M, W, G, N, H etc., in position in order to measure the deflections at each required point on the structure during the tests. This frame was fixed to the same two I beams bolted longditudinally along the laboratory wall, to which the plate of test bed was also bolted rigidly. Therefore, any movement of the labaratory wall or test bed would have no effect

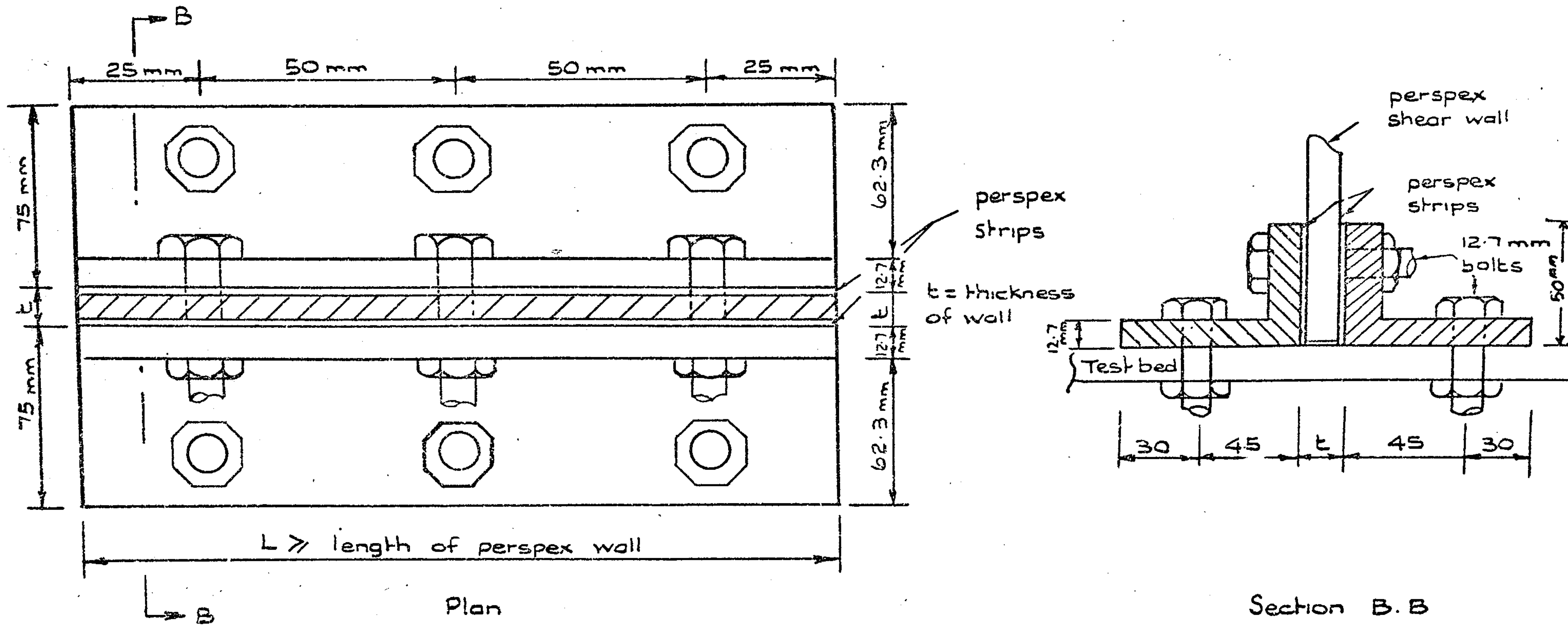


Fig. 4.2. Details of a fixed base of a wall

on the deflections. The dial gauges were of 50 mm travel. A piece of bright mild steel bar of 6 mm diameter, 50 mm long was attached to the dial gauges to avoid the possibility of dial gauges moving off the points where deflections were being recorded.

The loads were applied at each floor frame junction on one side of the structure by using dead weights so that it provided constant loading as yield and creep took place. No loads were applied to the floor junction at the shear walls. While testing the one storey structures, the loads applied at each frame junction of floors were such that a lever system at each loading point was required. The lever system used in the tests is illustrated in figure (4.3) and also shown in Plate 2. The lever arms were made from a 76 mm x 57 mm x 7 [illegible] R.S.J. and the leverage was 1:7. One end of each lever arm was anchored to the *anchorage girder* which was a heavy universal beam bolted securely to the ground by using special grip bolts. To fasten the end of the lever to the anchorage girder a high tensile wire of 2 mm diameter was used. The same size wires were also utilised in transmitting the loads from lever arms to the loading points of the structure. In this manner, the effect of any possible vibration during the test on the structure was avoided. The loads were applied at one end of the lever arms by using hangers with a loading pan. The dead weights applied to each lever were transmitted to the anchorage girder and to the loading points through knife edges on the lever arms as can be seen in figure 4.3. The knife edges at both the loading point and the anchorage end were placed just above the centre line of the lever beam, and the third knife edge was located at one-seventh of the length

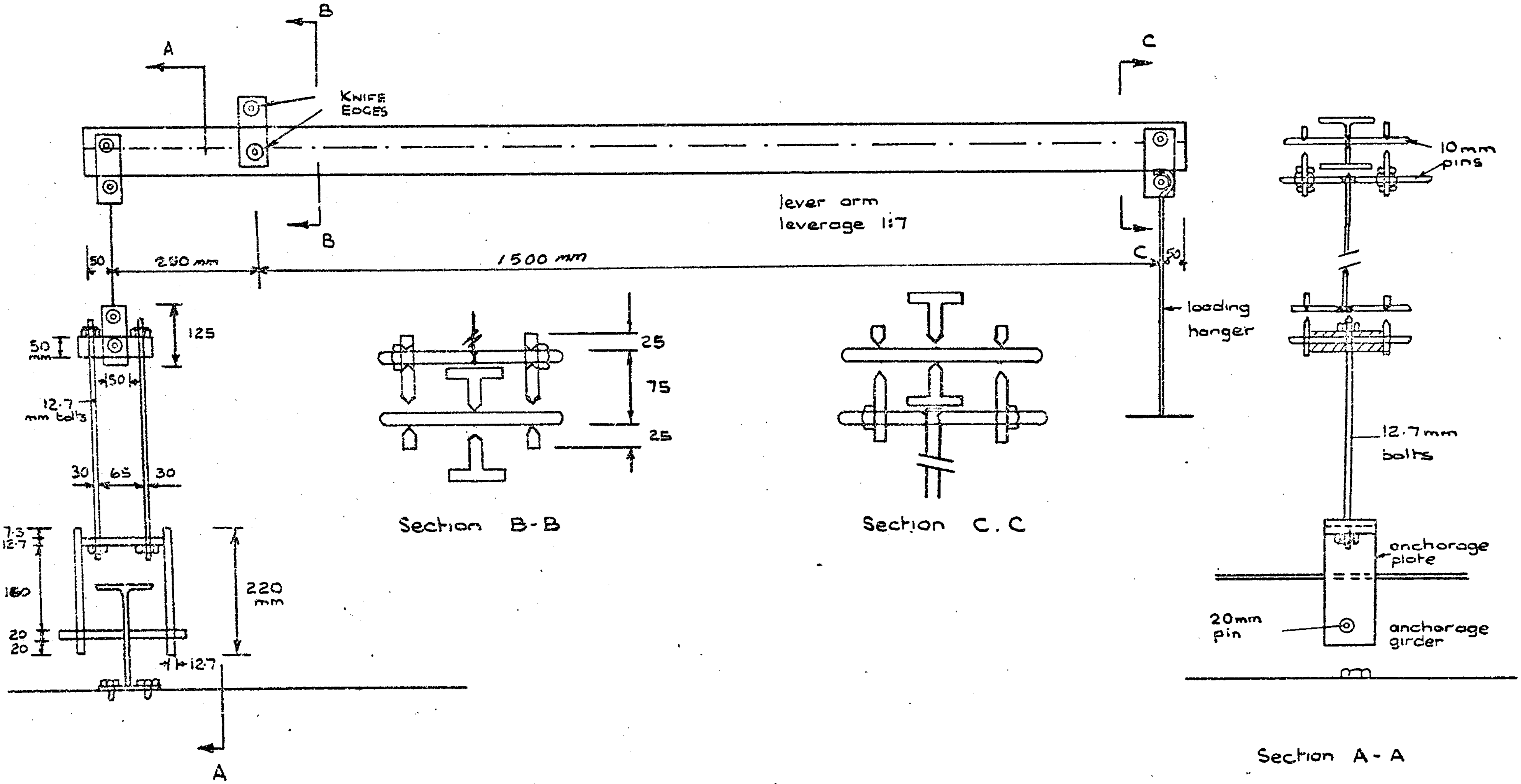


Fig. 4.3. Details of a lever arm

from the anchorage end on the levers just below the centre line. By doing this, the loads were made to act exactly on the centre line of the levers as shown in the same figure.

At the loading points, one pin of 10 mm diameter and 100 mm long passed through the holes of knife edges. The hanger was looped around the same size of pin and those two pins were linked by two silver steel plates. Those two pins were also passed through the knife edges drilled through those two link plates. The plates were 3 mm thick. The same size of pins and plates were also used at the other knife edges. At the anchorage end, one pin went through the hole of the knife edge on the lever arm. The high tensile wire went round the neck opened on the lower pin which was connected to the anchorage girder as shown in figure 4.3. The same type of connection was made between the levers and the loading points on the structure. One pin, as before, passed through the hole of the knife edge and the wire looped around the neck on the other pin. The other end of the wire went through the ring welded to the frames and was clipped by using special grips. These two pins were linked together by two steel plates. The details of the lever arms and full details of the connection points are shown in figure 4.3.

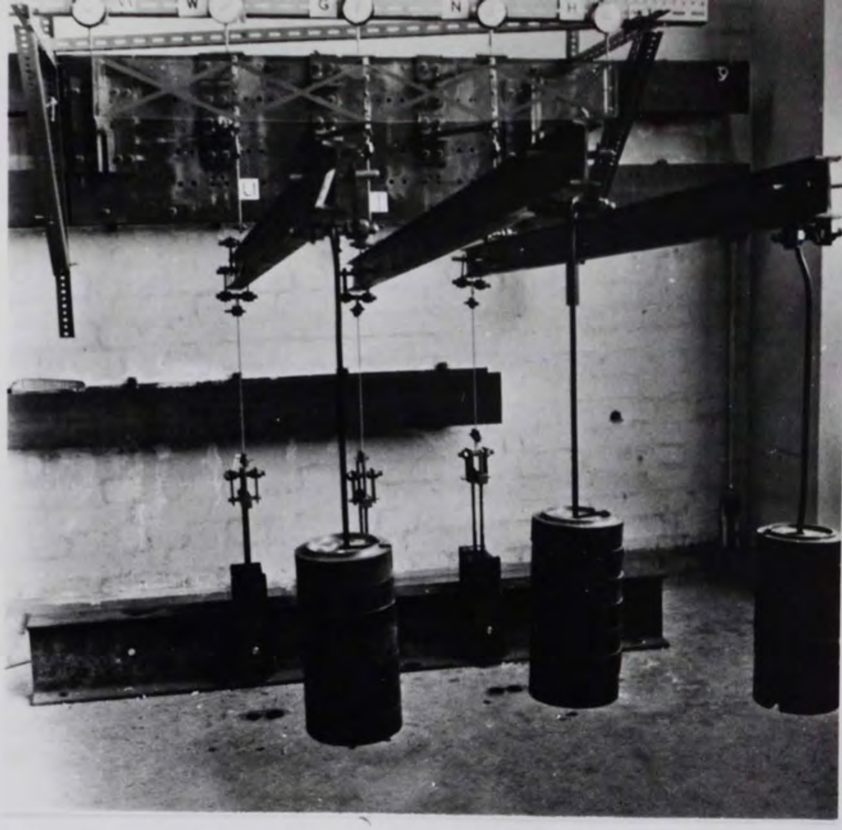
All plates and pins used in the load transfer from the lever arms to both frames and anchorage girder were hardened by tempering in an oven. The knife edges on the levers were also hardened. In this manner, the components of the connections were prevented from being damaged during the test.

The lever arms were initially set up above the level, but during the test, owing to the deflections, it moved downward. If an adjustment was required, it could be conducted by using the two 300 mm long in 12.7 mm diameter bolts utilised in linking the lever arm to the anchorage girder. Such an adjustment could be performed easily by turning the nuts on these two bolts. The anchorage plate was of inverted *U* shape of 12.7 mm thick black-mild steel plates which are shown in figure (4.3) and plate 3. The adjustment bolts are shown in plate by C, D and E. In the same plate A, B and L show the anchorage plates and K shows the anchorage girder. One end of each bolt was linked to the others by a solid steel plate and the other end connected to the anchorage plates. It should be noted that, whenever an adjustment was performed, the dial gauges were reset and a new set of readings were recorded.

4.3 Testing Procedure

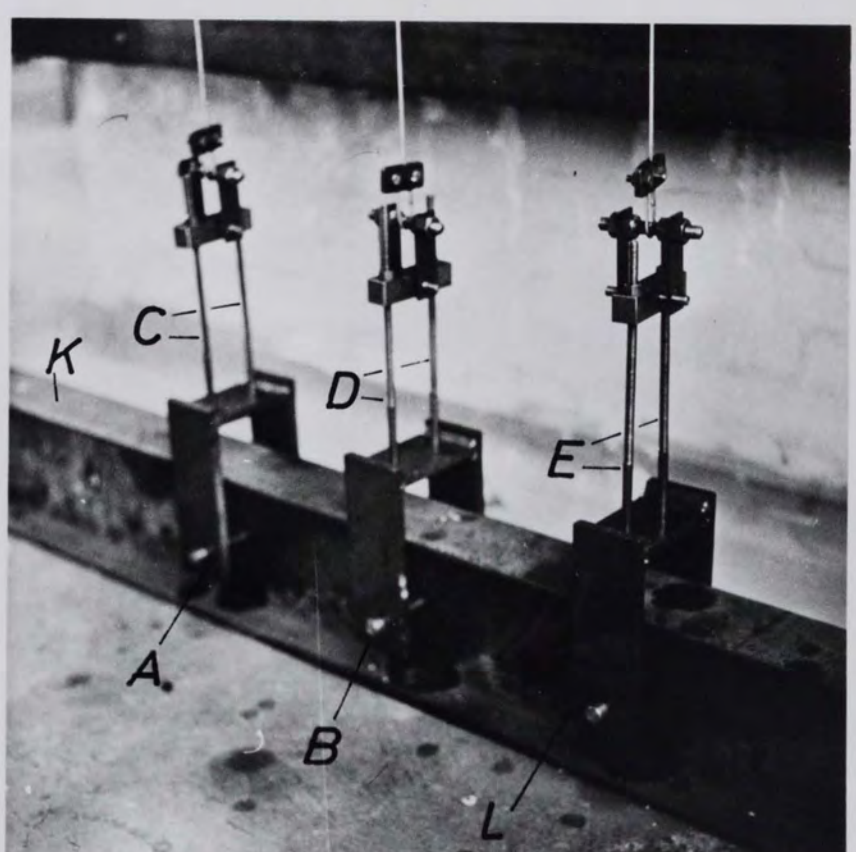
After setting up the model structure on the laboratory wall (see plates 2 and 4), the Dexian frame with dial gauges were fixed above the model. First zero readings were taken, then the lever arms were set up at each loading point and the loading pans, with hangers were located on the levers. All moving parts used in transmission of the loads were oiled to reduce friction. Then a new set of readings were taken. In this way, the self weight, including the loading hanger with the pan, plates, pins and lever arm were taken into account. The effect of these was magnified owing to the 1:7 leverage ratio of the system. The magnified value of the self weight of the lever arms was calculated as 46 kgf., which was the magnitude of the load transferred to a frame via the high tensile wire. Therefore, in effect, at each loading point on the structure, 46 kgf. was applied at the start of a test.

The loading procedure was started by using an increment of



GENERAL VIEW OF ONE STOREY STRUCTURE TEST

PLATE 2



DETAIL OF THE ANCHORAGE OF LEVER ARMS

PLATE 3

4 kgf., which was magnified to 28 kgf., and passed to the loading point on the frame. This increment was gradually reduced to 2 kgf., 1 kgf., and eventually to 0.5 kgf., at each pan as yield progressed and collapse became imminent. After each loading, before taking the new sets of readings, the dial gauges were left for a while until they became steady. That is to say, the dial gauges were left for some ten minutes, at least, in the elastic range, and half an hour after yield progressed. The readings were taken only when the movement of the dial gauges had slowed down to one division (0.01 mm) per minute. The load deflection graph of the centre frame on the top floor level was plotted. Before applying another load, the readings were checked. If there were any slight difference with the previous readings, the average of the two readings was used. Whenever yield commenced at any joint on the frame, this was noted. The occurance of the plastic hinges was traced by observing the cracks on a film of plumber's resin which had been put on both ends of the members of each frame. However, this manner of following the sequence of plastic hinge formation was not very satisfactory. At some tests, the resin was too thick, therefore, the detection of the plastic hinges was found to be difficult. But at collapse, all plastic hinges were obvious and could be seen clearly.

In the tests on the two storey structures, the loads were applied by hangers at each floor level on the frames as shown in plate 4. No loads were applied to the shear walls.

Load procedure was commenced by using an increment of 20 kgf. on the hangers of the first floor and 10 kgf. on the hangers of the second floor of the frames. This initial increment was reduced to 10,8,6,4,2 and 1 kgf. on first floor and correspondingly 5,4,3,2 and 0.5 kgf. on second floors as yield commenced and eventually failure became imminent.

4.4 Control Tests

At least two specimens for each test, cut out from the same bars used to manufacture the intermediate frames, were tested. To carry out such tests, a loading device, seen in figure 4.4 and plate 5, was utilised. The specimens were of 508 mm ($\approx$ 20") long and rested on two supports 457 mm apart. They were loaded in one-third points along the bar. The loads were applied on a hanger looped around a knife edged hole drilled in a plate 140 mm x 70 mm. A pin, 20.00 mm ($\frac{3}{4}$") in diameter and made out of silver steel, passed through a second knife edged hole drilled at the top end of the plate. Both ends of this pin were linked to one-third points of the test specimen by using two more silver steel plates as seen in figure 4.4 and plate 5. The loads acting on the hanger were transmitted to the linkage plates and the specimen through sharp knife edges opened on the plates as shown in the same figure. The specimens were supported on a knife edge (shown as "Z" in plate 5) at one end and on a roller (shown as "R" in the same plate) at the other. The loads were initially applied in increments of 16 kg.f. to the load pan which was gradually reduced to 8,4,2,1 and eventually to 0.5 kgf. as the plastic hinge in the section of the span between the loading points was about to form. Between each loading, ample time was given for the dial gauge to settle. When the gauge settled to a constant rate of 0.01 mm per minute, then the next load was added. The dial gauge was located at mid

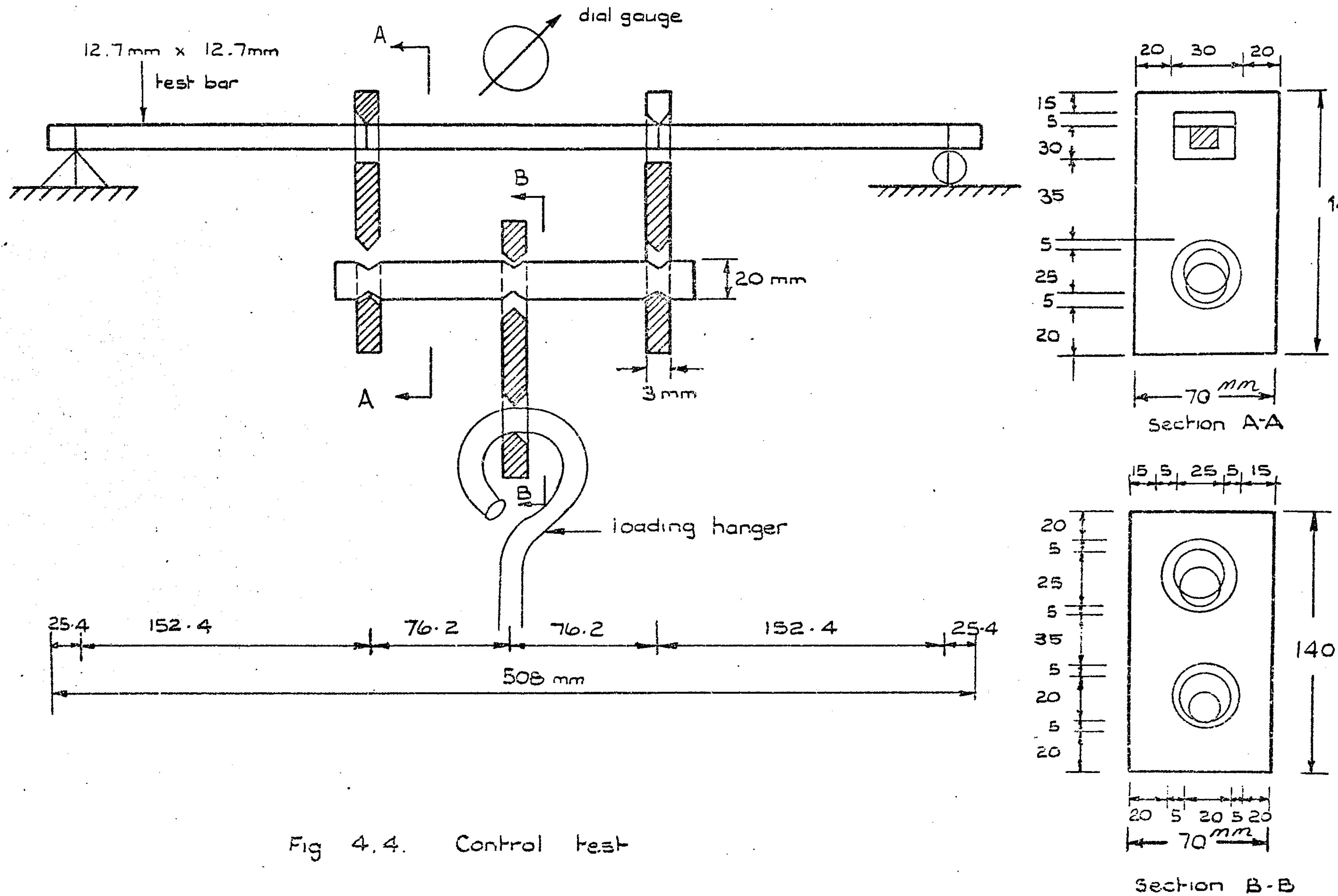
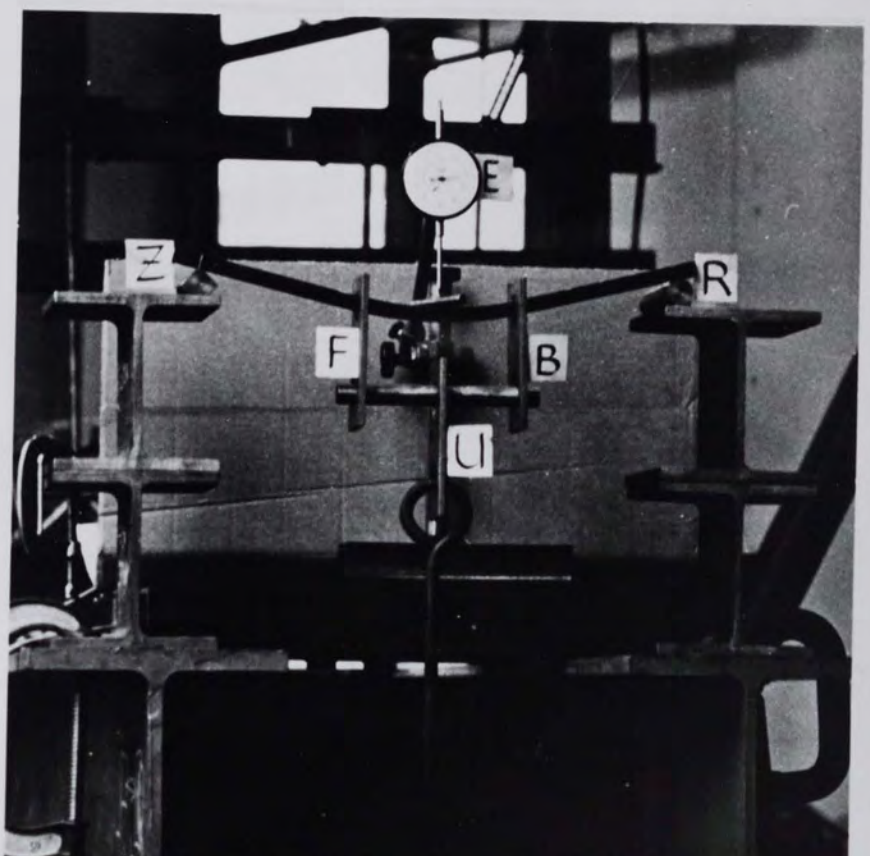


Fig 4.4. Control test

GENERAL VIEW OF TWO STOREY STRUCTURE TEST

PLATE 4



CONTROL TEST

PLATE 5

point of the specimen. After recording each dial gauge reading, the load-deflection graph was plotted. One of the typical load deflection diagrams obtained from a control test carried out for test 4 of the one storey structure is shown in figure 4.5. The same control tests were performed on the specimens cut out from the frames after testing the structure to collapse. One bar cut out of the undistorted part of each frame was subjected to the control test. By taking the average of the results obtained from the control tests carried out for each structure, the plastic moment, Mp and the corresponding yield stress, fy, of the members were calculated. The irregularities of the members were not taken into consideration in the theoretical calculations. However, the variations in the cross-section of the members were avoided by using the values of the plastic moment and yield stress obtained from the control tests. Extreme care was taken in the selection of the members to be as nearly perfect as possible. The magnitude of deflections at collapse were such that, the effect of untrueness was negligible.

The value of Young's Modulus of Elasticity was obtained from the tensile test on at least two specimens cut from the frames used in the same tests. The average value of the test results were taken into account in the calculations.

It should be noted that no control tests were carried out to find the mechanical properties of "Perspex". It has already been emphasised that the mechanical properties of "Acrylic" perspex depend markedly upon the temperature at which they are measured. In reference (72), the mechanical properties were given at 20° C. In the analysis, Young's Modulus of Elasticity in flexure was taken as 2.943 KN/mm^2 as given by the manufac-

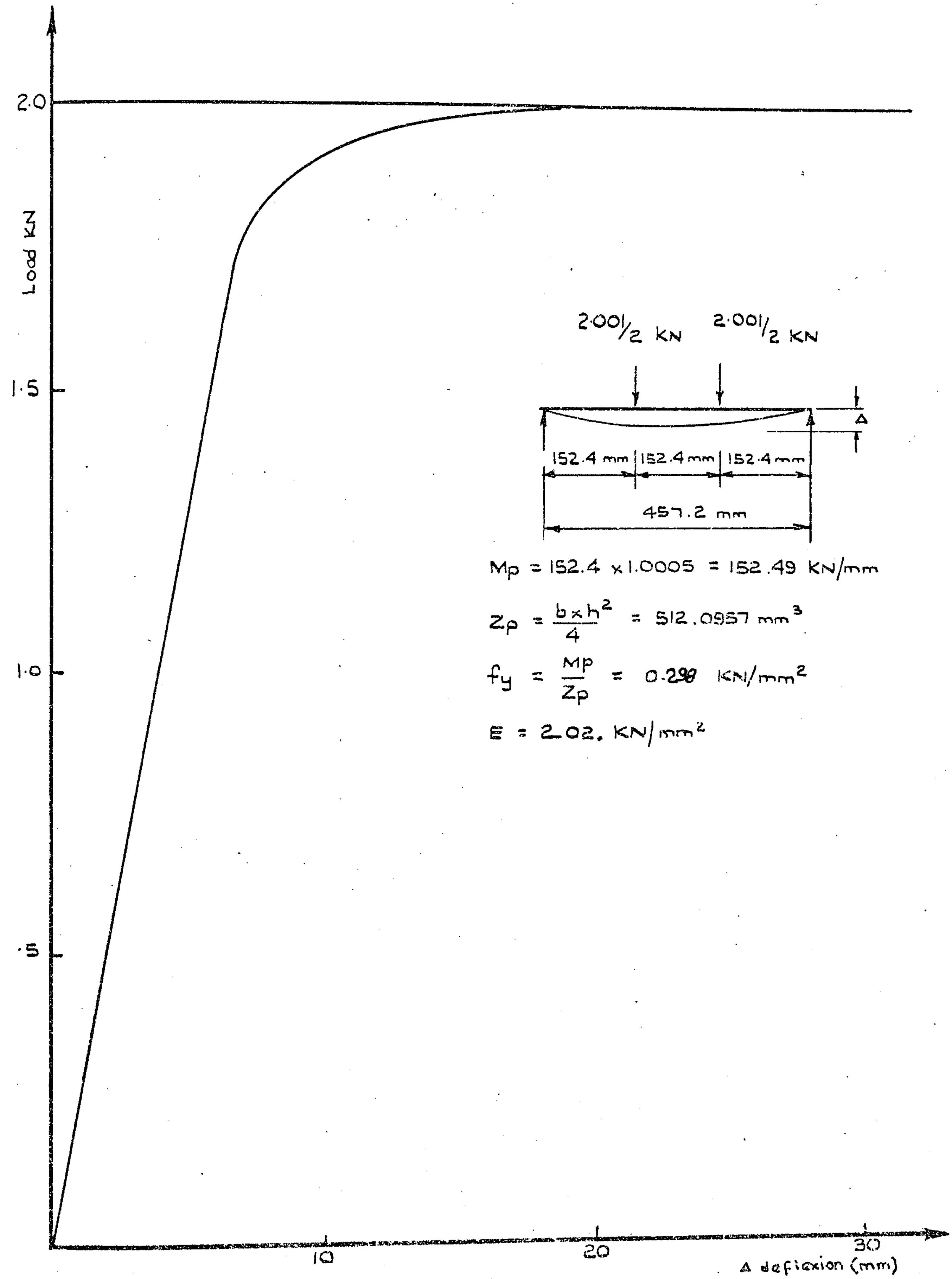


Fig. 4.5 Control test on 12.7 mm x 12.7 mm

turer. By using the Poisson's ratio of 0.35, the torsional modulus of perspex was calculated as 1.090 KN/mm^2.

CHAPTER 5

EXPERIMENTAL INVESTIGATION

5.1 Introduction

To verify the method described in Chapter (2), a series of tests were carried out on single storey and two storey structures. The shear walls and the floor slabs were made of perspex sheets of different thickness. The frames were fabricated from 12.7 mm x 12.7 mm black mild steel bars. The values of plastic moment of the sections and the Young's Modulus of Elasticity were found from the control tests carried out on the bars cut out of the frames used in the tests. In this Chapter, the results of the tests performed will be given and compared to the computed results.

5.2 Results of the Tests on Single Storey Structures

To cover a spectrum of possibilities, eight structures were tested. The overall dimensions of these were kept constant like those given in figure (2.5), but other properties were varied as shown in table (5.1). Structures 1A and 1B were manufactured to be identical so as to examine the consistency of the apparatus and the test procedure. These failed exactly under the same loads. Both structures failed by slab buckling at loads lower than the elasto-plastic analytical values. A discrepancy of 17% was observed with these structures.

The method described in Chapter (2) does not cater for buckling of unstiffened thin slabs but these are not used in practice. The thickness of the slabs in structures 2 and 3 was therefore increased. Both structures gave very favourable results. The overall dimensions of structure 2 are shown in the figure (5.1) with the slabs being 130 mm wide while the other

failure loads					Mp
computer	experiment	rigid plastic	E-P bare frame	R-P bare frame	for steel
1·740	1·580	1·709	0·98	0·99	1·46·51

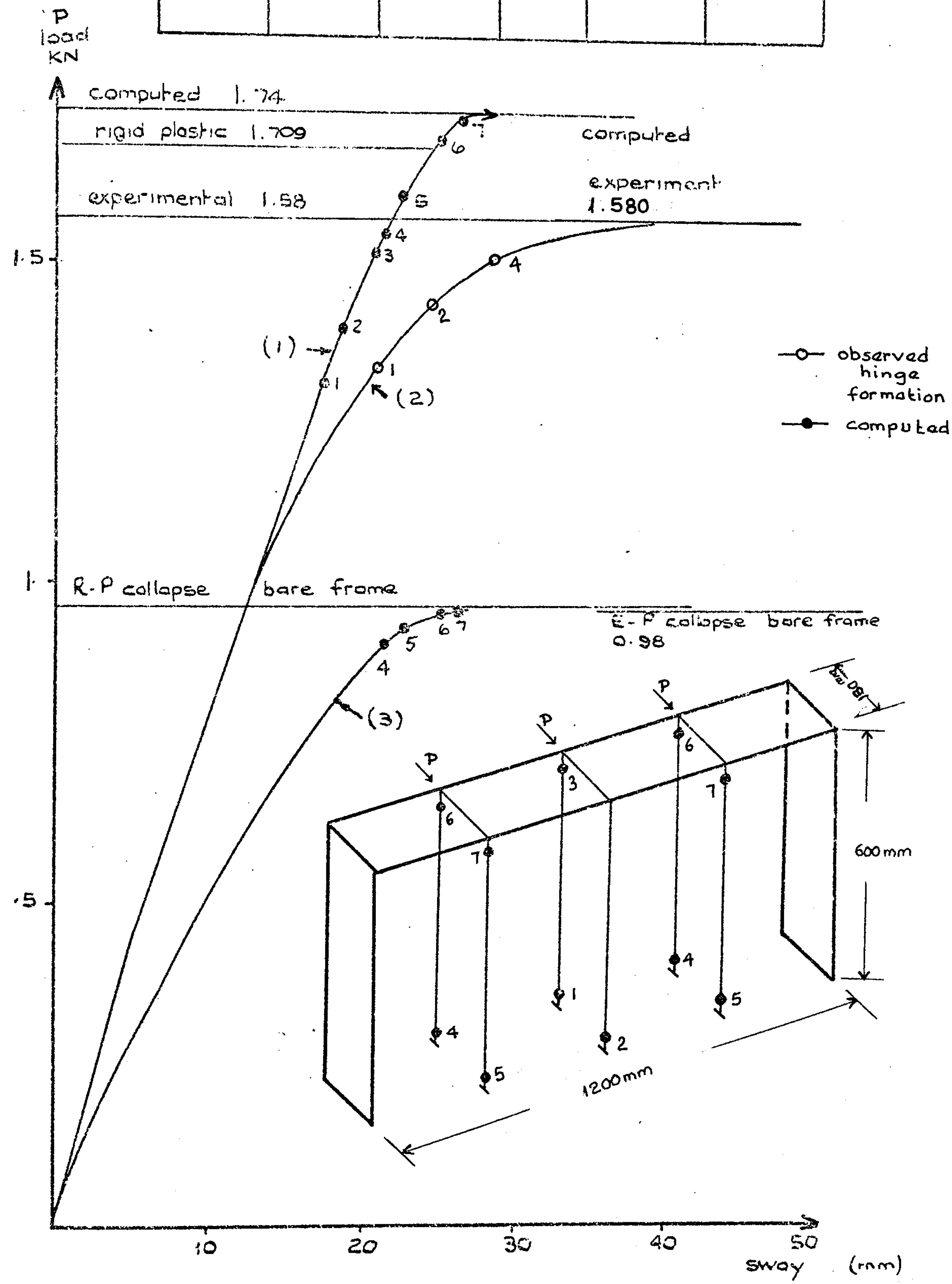
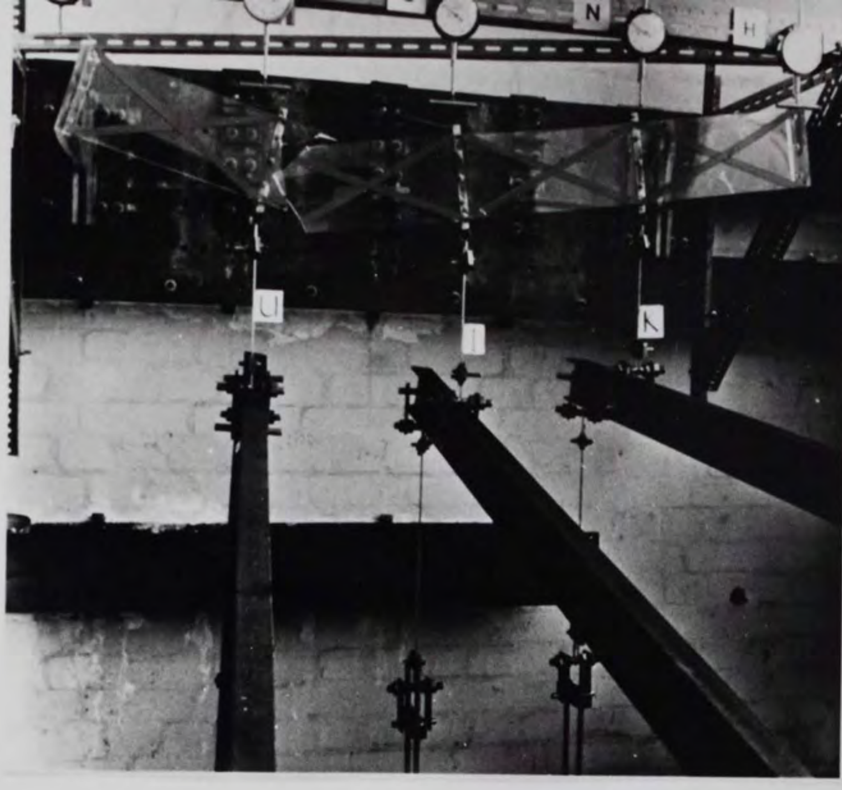


Fig. 5.1 Load - deflexion diagram for structure 2

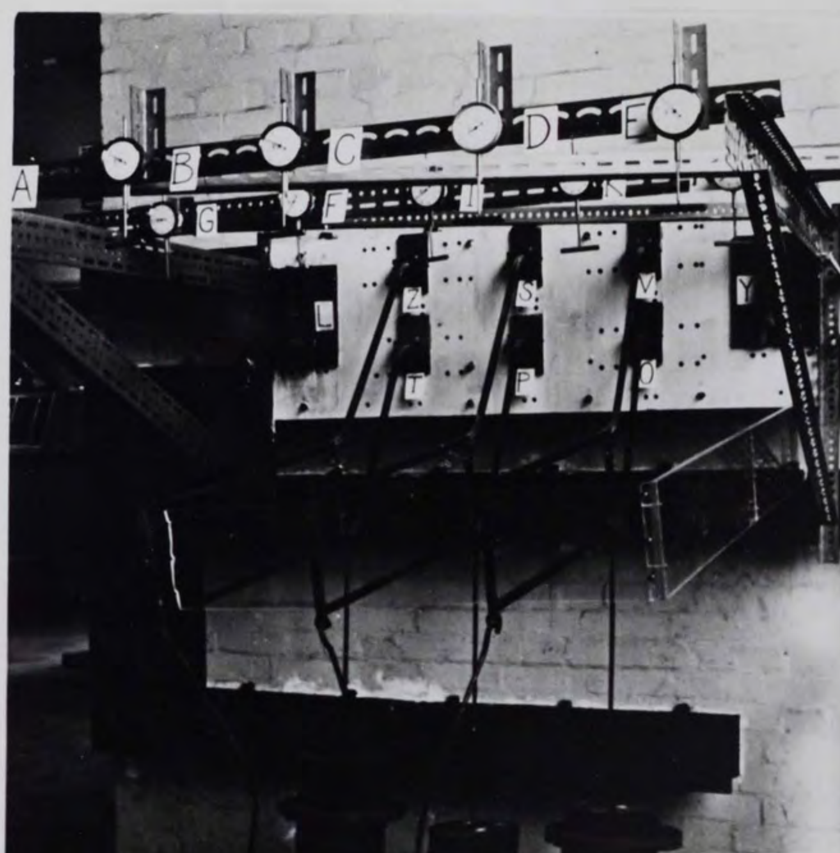
properties are varied as shown in table (5.1). The results of the elastic-plastic analysis of the structure is shown in the figure by graph (1). The first three plastic hinges formed in the middle frame which became so weak and unable to maintain its share of the wind loads. Hence, more load was transmitted to the outer frames by means of the floor slabs. This caused hinges 4, 5, 6 and 7 to develop in the external frames consecutively. Structure 2 collapsed with failure mechanism developing in frame 1 and frame 3 at a load of 1.74 KN. In figure (5.1), the experimental load-deflection graph was shown by graph (2). The model structure collapsed at a load of 1.58 KN owing to excessive buckling in floor slabs. The structure collapsed after the occurance of a sudden crack along the floor and one of the outer frames intersection as shown in plate (6). The formation of the first two hinges in the central frames were traced by cracks occuring on the resin film on the frames. Hinge (1) was visible at a load of 1.373 KN and hinge (2) was visible at a load of 1.451 KN. In the outer frames only hinge (4) was traced which was clearly apparent at a load of 1.550 KN. After that point, the floor slab buckled extremely and led the structure to failure. Therefore, the rest of the hinges were not traced. Structure 2 proved to be 9% too weak.

The rigid plastic collapse load for a sway mechanism is 1.709 KN which is 1.7% below the computed collapse load. The deterioration of α values with the formation of the plastic hinges is shown in figure (5.2). It can be seen in the figure that the development of the last hinge in the outer frames caused the α value to drop down to zero indicating that those frames lose their stiffness at this stage, and are not able to hold up their share of the loads.



COLLAPSE OF ONE STOREY STRUCTURE

PLATE 6



COLLAPSE OF TWO STOREY STRUCTURE

PLATE 7

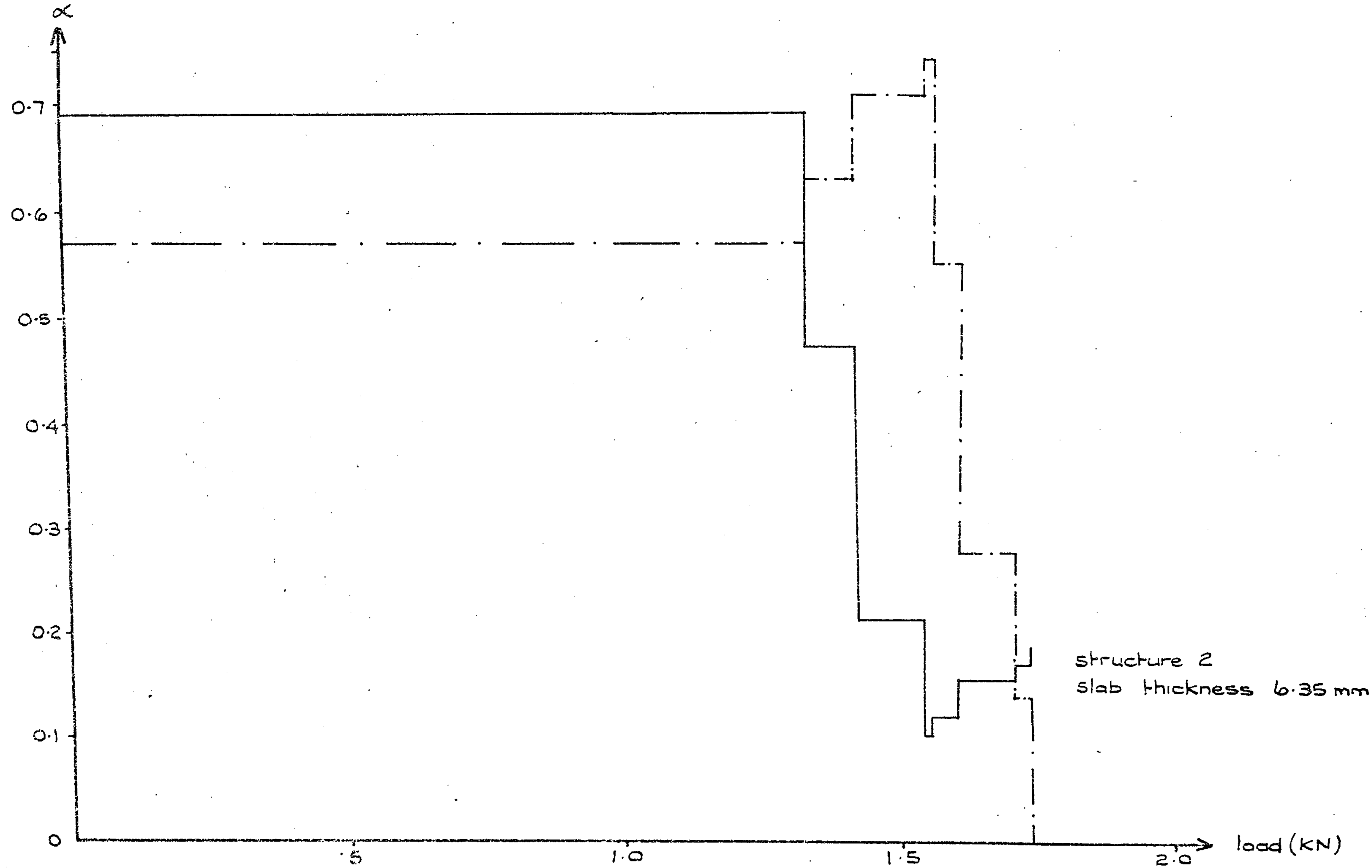


Fig. 5.2. Variation of α due to hinges

The thickness of the floor slabs were increased to 7.93 mm in structure 3. The rigid plastic collapse load for this structure was calculated as 1.56 KN which was 8.7% and 8.1% lower than those of the computed and the experimental collapse loads respectively. The results obtained from the various analysis of this structure are summarised in table (5.1) and the related graphs are presented in figure (2.5). The structure collapsed at the expected theoretical value. The variation in the values of this structure was given also in Chapter (2) by figure (2.6).

The slab thickness was again increased for structures 4A and 4B while for structures 5A and 5B the thickness of the slabs as well as the shear walls was increased further. Structures 4A and 5A both failed at loads much higher than those predicted. It was obvious that with grillage systems as thick as these, it is possible to reduce the number of the intermediate frames. Thus structures 4B and 5B were manufactured with two intermediate frames instead of three. In this manner, while keeping the thickness of the perspex unaltered, the relative stiffness of the slabs was reduced by increasing their longitidunal spans from 300 mm to 400 mm. The load deflection graphs obtained from experimental and theoretical analysis are shown in figure (5.3). The experimental results of structure 4B compare favourably with those obtained theoretically. The rigid-plastic collapse load for a sway mechanism in the middle frame was 1.706 KN. This was found to be lower than both the experimental and the theoretical values. The deterioration of the α values with the formation of the plastic hinges is shown in figure (5.4). The formation of plastic hinges in the structure throughout the analysis was traced up to failure. Altogether, four plastic

computer	experiment	rigid plastic collapse	Ep collapse	R-P collapse
1.824	1.893	1.706	0.82	.83

Mp = 123.34 KN mm

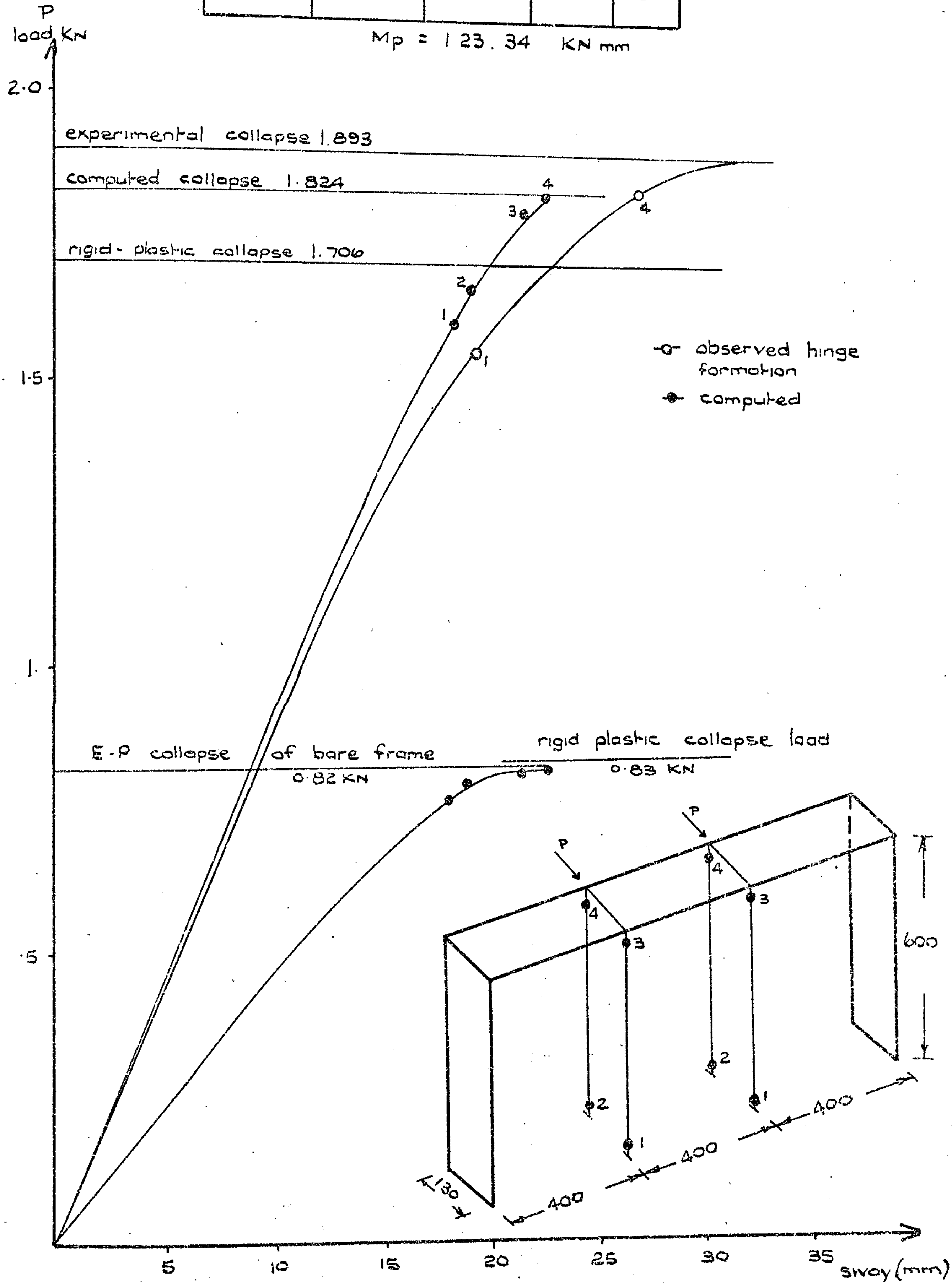


Fig 5.3. Load deflexion diagram for structure 4B

Structure No.	No. of Frames	Slab Thickness mm	Wall Thickness mm	Mp for Steel KN.mm.	Failure Loads (P)			
					Computed KN	Experiment KN	Bare Frame KN	Rigid-Plastic Collapse of Structure KN
1A	3	5.08	12.7	149.50	1.69	1.41	1.00	1.677
1B	3	5.08	"	151.74	1.71	1.41	1.01	1.702
2	3	6.35	"	146.51	1.74	1.58	0.98	1.709
3	3	7.94	"	152.49	1.696	1.687	1.02	1.56
4A	3	9.52	"	122.59	1.588	2.065	0.82	1.508
4B	2	9.52	"	123.34	1.824	1.893	0.82	1.706
5A	3	12.7	19.05	125.58	1.74	2.443	0.84	1.584
5B	2	12.7	19.05	131.00	2.405	2.545	0.84	2.152

$E = 202.02$ KN/mm^2

TABLE (5.1)

THEORETICAL AND EXPERIMENTAL FAILURE LOADS OF ONE STOREY STRUCTURES

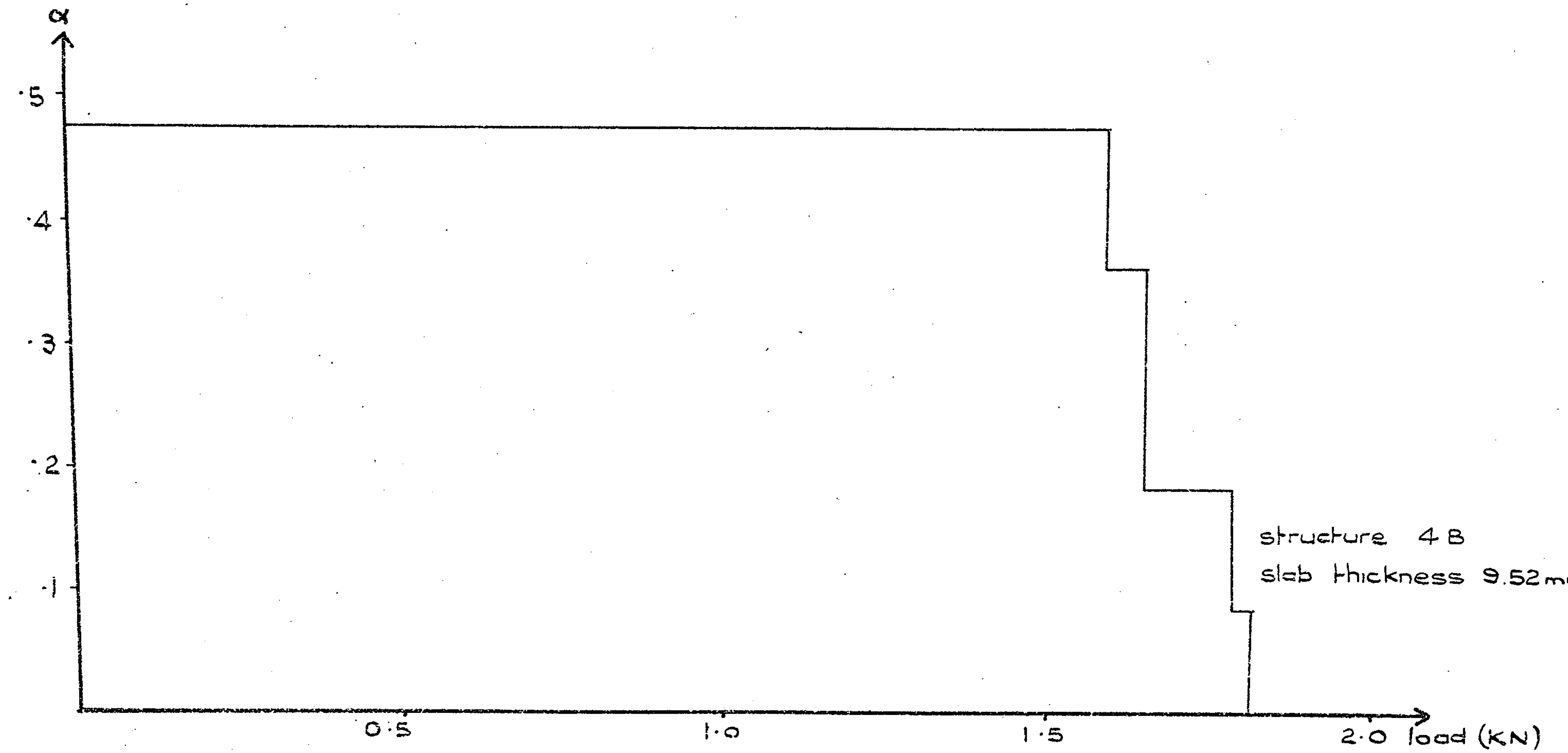


Fig. 5.4 The variation of α with the formation of hinges

hinges developed. During the testing, the formation of the plastic hinges were not traced successfully. At 1.55 KN, hinge (1) was visible. Number 2 hinge was not traced because the plumber resin was too thick. Yield was observed around the top ends of the columns at 1.687 KN. Hinges numbered 2, 3 and 4 were visible at 1.824 KN. The structure failed at 1.893 KN, while the theoretical collapse load was 1.824 KN. A difference of 3.8% was observed with structure 4B.

With structure 5B, the experimental results compare favourably with those obtained theoretically. In structure 5B, the frames were also weakened by manufacturing them from (9.525 x 9.525) mm^2 black mild steel bars. In this test, also the plastic hinges were clearly visible before collapse took place. The model structure collapsed at a load of 2.545 KN. With structure 5B, a discrepancy of 5.83% was observed. Excessive buckling of floor slabs was also observed.

Both structures 4A and 5A collapsed with failure mechanism developing in the outer frames rather than the central one. There were as many as 10 hinges in each structure and considerable strain hardening was observed. This was particularly the case with the outer frames where the spread of plasticity was noticed along the columns. With the simple structures tested, where collapse takes place at the formation of a frame mechanism and with no instability effects or any reduction in Mp due to the axial forces, it is possible for strain hardening to play a significant role. However, this effect is much less with larger and taller structures where failure takes place by a pre-mechanism collapse. Structures 4B and 5B with 8 hinges at collapse also demonstrated strain hardening but to a lesser extent.

5.3 Results of the Tests on Two Storey Structures

The intermediate frames in all tests were made out of 12.7 mm x 12.7 mm black mild steel. The length and the height of the structure were 1200 mm and were kept constant throughout. But other properties were varied as shown in table (5.2). There were altogether three intermediate frames in each model structure. Altogether seven structures were tested to collapse. The results obtained from both computer analysis and the experiments are also summarised in the same table. While carrying out the tests, there were no loads applied to the shear walls. Shear walls and floor slabs were fabricated from "Acrylic Perspex" sheets of various thickness. The ratio between the external loads applied to middle floors and the top floors of the structure was 0.5.

In the first test the width of the structure was 200 mm with the other properties shown in table (5.2). The test was carried out in the manner outlined in the previous Chapter. The experimental load-deflection diagram of the structure was shown in figure (2.7) of Chapter (2), and as can be seen in the figure, the experimental collapse load of 0.79 KN was lower than that of the predicted load of 1.207 KN. This is because of the thin slabs used in the structure which resulted in their buckling. During the test, at the load of 0.49 KN at the top floor level and 0.98 KN at mid-floors, a crack was observed in the floor slabs connecting the middle frame to the outer frames. This crack later propagated and speeded as the loads were increased. Before collapse took place a considerable creep was observed in the perspex. The model structure collapsed at a load of 0.79 KN at the top floors and 1.58 KN at mid-floors. Just before collapse, the hinges at the feet of the centre frame were visible. The formation of the rest of the hinges were not followed, since the film of plumbers resin put on the frames appeared to be too thick. However, all hinges were visible after collapse. The failure of the structure was disastrous as shown in the Plate 7.

The deterioration of the α value at the top floor of one of the outer frames were illustrated in figure (2.8), and the result obtained from the elastic-plastic analysis of the structure was presented in figure (2.7) of Chapter 2. Therefore, there is no need to make further comments on them.

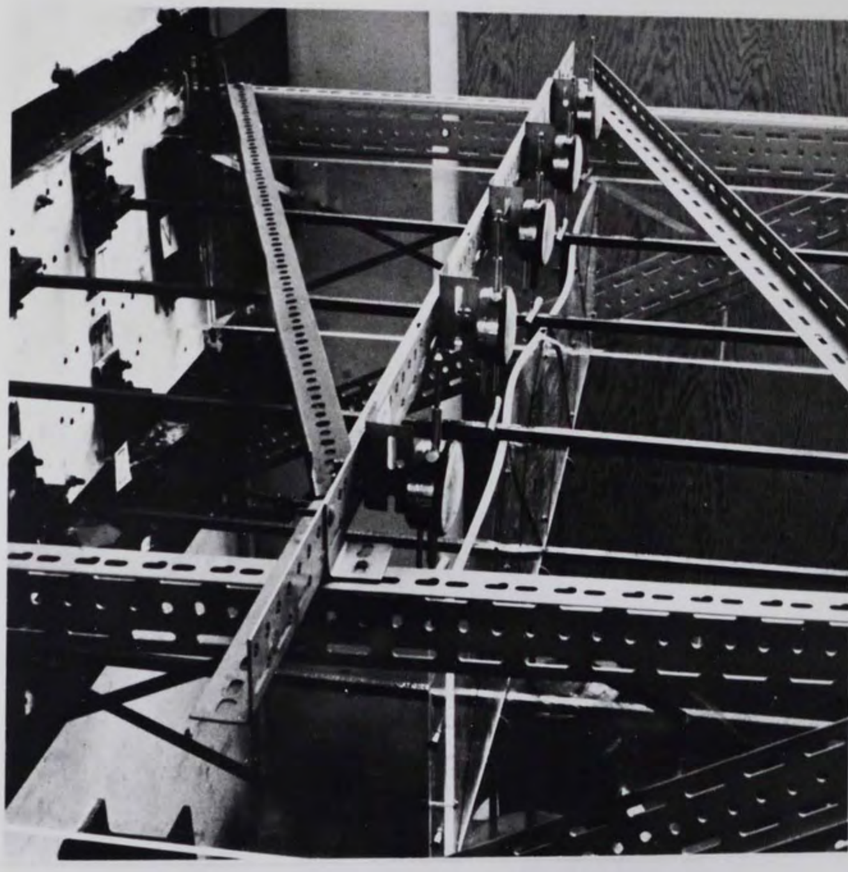
By employing the same thickness of walls and floors, the width of the structure was increased by 50 mm in structure 2 and by 100 mm in structure 3 to find out the effect of the width change on the collapse load of the structure. In structures 2 and 3 the buckling of the slabs, especially in the middle storey floor slabs was experienced as shown in Plate (8). Failure, however, was finally caused by the occurance of cracks in mid-floor slabs. The results of structures 2 and 3 show that an increase in the width of the structure may cause the early failure of the structures, since the grillage system becomes stiffer and attracts most of the loads. At any stage of the loading procedure, buckling in slabs may take place and lead to the occurance of cracks and consequently the collapse of the whole structure.

The effect of the variations of both floor and wall thickness was studied in structures 4, 5, 6 and 7. Structures 4 and 5 were similar to that of structure 2. The only difference was that the thickness of the middle floor was increased to 12.7 mm in structure 4 and 15.88 mm in structure 5. The second floor slabs were kept at the same thickness as

	Thickness of Walls mm	Thickness of Slabs		Width of Frames mm	Width of Walls & Slabs mm	FAILURE LOADS (AT TOP FLOORS)				
		1st Floor mm	2nd Floor mm			(E-P) Computed KN	(E-P) Experimental KN	(E-P) Bare Frame KN	Mp value for Steel KN mm	E KN/mm^2
1	15.875	7.94	7.94	200	180	1.2070	0.789	0.272	122.556	202.02
2	15.875	7.94	7.94	250	230	2.116	0.937	0.272	122.556	202.02
3	15.875	7.94	7.94	300	280	3.478	1.428	0.272	122.556	202.02
4	15.875	12.7	7.94	250	230	2.302	0.976	0.274	123.30	"
5	15.875	15.88	7.94	250	230	3.312	1.044	0.274	123.303	"
6	20.00	12.7	7.94	250	230	2.546	1.373	0.275	124.051	"
7	20.00	15.88	12.7	250	230	2.793	1.667	0.292	131.524	202.02

TABLE (5.2)

THEORETICAL AND EXPERIMENTAL FAILURE LOADS OF TWO STOREY STRUCTURES



BUCKLING OF MIDDLE FLOOR SLAB

PLATE 8

that for the previous structures. Both structures collapsed after the occurance of cracks at one of the bases of the shear walls and at the junction of the wall with the middle floor. This showed that an increase in the thickness of the slabs diverted most of the loads to be transmitted via floors to the walls. These structures failed due to the development of large shear forces in the slabs and the walls. As mentioned in the previous Chapter, the shear walls were fixed to the test plate by bolted clamps. Large shear forces may cause a considerable stress concentration around the holes of the bolts at the wall bases. This can also cause a crack around the holes and an early failure of the complete structure.

To reduce the discrepancy between the computed and the experimental collapse loads, it was decided to increase the thickness of the walls, and structures 6 and 7 were tested. The thickness of the walls in structure 6 was 20.00 mm. The floors, however, were kept to be of the same thickness as in structure 4. This structure collapsed at a load of 1.373 KN with shear cracks developing at the bases. Structure 7 was similar to structure 5 except the walls were 20.00 mm thick. No much improvement was gained from this test. Premature collapse was observed in both structures 6 and 7. Structure 7 collapsed suddenly at a load of 1.667 KN due to the development of a shear crack at the base of one of the walls. Structures 6 and 7 showed that increasing the thickness of the walls may reduce the gap between the experimental and computed collapse loads. It should be noted that in the above tests carried out on two storey structures, it was found impossible to determine the optimum thickness of both wall and floors to provide the simultaneous failure of slabs and frames. With structures 6 and 7 discrepancies of 46% and 40% were obtained. These are far above the experimental collapse loads.

5.4 Conclusions

From the tests carried out on single and two storey structures, the following conclusions may be drawn:-

(i) - The carrying capacity of a building depends considerably upon the relative stiffness of the grillage system and the frames at all stages of the loading process. Investigation into the variation of the load transmitted to a frame by evaluation its α value indicates that, as hinges develop, a proportional increase in the applied external loads does not lead to a similar increase in the loads carried by each individual frame. This can be seen in the figures.

(ii) - In the case of thin slabs, failure of a structure may take place by the buckling of the slabs, but thin slabs are impractical unless stiffened by cross beams which add to the construction cost. With thicker slabs, it is not so easy to select the geometry of a building to ensure a simultaneous failure of the slabs and the frames.

(iii) - Two storey structures with thin plates collapse at an early stage of the tests due to excessive buckling of the floor slabs. Buckling and large shear forces occur along the floor beam intersection and may cause cracks in the slabs. This can lead the structure to failure promptly.

(iv) - The stiffness of the grillage increased with the increase in the width of the structure. This was confirmed by the reduction in the initial α_o values obtained.

(v) - With thicker walls, the structure may take much more loads and reduce the discrepancy observed between the theoretical and experimental collapse loads. It was observed that, with thicker walls and floors, collapse may occur due to high shear stress at the wall bases or anywhere in the grillage system.

(vi) - Comparison of the results also showed that the effect of the grillage system on the load carrying capacity of the structure is considerably high. This could easily be seen in tables (5.1) and (5.2) by comparing the results obtained from bare frames analysed independently.

(vii) - Torsional buckling of walls and slabs was also experienced in the tests. This may cause the premature failure of the structures as well.

(viii) - Considerable creep was also observed in all tests. This may cause the discrepancy between the theoretical and experimental deflections of the structures.

CHAPTER 6

THE COMPUTER PROGRAMME FOR THE COMPOSITE ELASTIC-PLASTIC ANALYSIS OF THE FRAMED STRUCTURES

6.1 Introduction

This Chapter describes the development of the elastic-plastic analysis programme by Anderson (24) to analyse skeletal frames by taking the composite action between floors and the supporting beams into consideration. To do this, Anderson's programme is modified and the plastic moment of the composite section is obtained by making use of the formulaes (68) given in Chapter (1). Previous work carried out by Majid (68) on the composite action of continuous beams and slab floor systems was described briefly in section 1.3 of Chapter (1). A brief review of the work done by Anderson (24) was also given in the same Chapter (section 1.1).

Later in the Chapter, a few examples taken from practical structures are given in order to demonstrate the advantage gained in the load carrying capacity of the frames by the inclusion of composite action. In these examples, the effect of slab reinforcement on the plastic composite moment of the section and the composite action in the columns are ignored.

The elastic-plastic analysis programme of the complete building structure described in Chapter (3) is also modified to allow for the inclusion of the composite action. In this Chapter, the effect of composite action in the complete building structure is also studied. By the modified programme a number of examples taken from the practical structures were analysed to exhibit the advantage gained by the inclusion of such an effect on the carrying capacity of the structures.

6.2 Description of Analysis Programme

At the beginning of the data, it is indicated whether composite action is being taken into consideration in the analysis or not. To do this, (0) is used to show that the composite action is not being considered. In this case, a normal elastic-plastic analysis (23,24) is carried out. Otherwise, an integer, for instance 1, is used to indicate that the composite action will be considered. Following this, the effective width of floor slabs, thickness of slabs and the cube strength of concrete are fed in. In this part of the data, the total number of load cases to be considered in the analysis, and the type of section used in the frames are included.

The general frame data consists of the total number of joints, total number of members, Young's modulus of elasticity of steel, applied tolerance, the unit working load factor, and the total number of plastic hinges expected to form in the frame. The rest of the information required for the analysis is obtained from the frame geometry.

The computer programme is written in Atlas Autocode - Chilton - Didcot and its flow diagram is shown in figure (6.1). Each block of this is labelled from 1 to 26. By following the same programme procedures of references (23,24), the contribution of each member to the submatrices of the overall stiffness matrix is constructed and stored in the manner (23,24) which was briefly described in Chapter 1 (section 1.1). The stiffness equations (1.1) are solved to obtain the resulting joint displacements. These are used to calculate the new sets of member forces from equation (1.6). New member forces are used to predict the load factor for a plastic hinge.

In the calculation of plastic moments of the section, the member is checked whether the composite action is being considered. If it is not, the plastic moment of the section is found by equation (1.4a) and (1.4b). If it is, by checking the sign of the bending moments at each end of the members, it is decided whether composite action is present at these ends. In the case of the composite action, the plastic composite moment of the section is calculated by any of the equations given in Chapter (1) (section 1.3), depending upon the location of the plastic neutral axis within the section.

It must be noted that when a beam is subject to a sagging moment and the slab is in compression, the plastic hinge moment of the beam is increased substantially. On the other hand, hogging moments cause tension in the concrete slab and thus the plastic hinge moment of the beam is considered to be unaffected in this case. Thus each member has two different plastic hinge moment values Mp and Mpc. It is possible that the beam can be subjected to a sagging moment at one end and a hogging moment at the other end. The composite plastic moment of a section is considered when sagging moment is present.

The same iteration procedure of Majid and Anderson (23) is utilised and iteration is continued until the predicted load parameter satisfies the tolerance test. A hinge is then recorded in the member data and $\pm$ plastic hinge moment of the section is included in the load vector. The prediction of load parameter and the axial forces in the members are carried out in the manner which was outlined briefly in Chapter 1 (section 1.1). These predicted axial forces are used in the re-construction of the stiffness matrix in the next iteration. The value of predicted load parameter is increased by a specific amount to com-

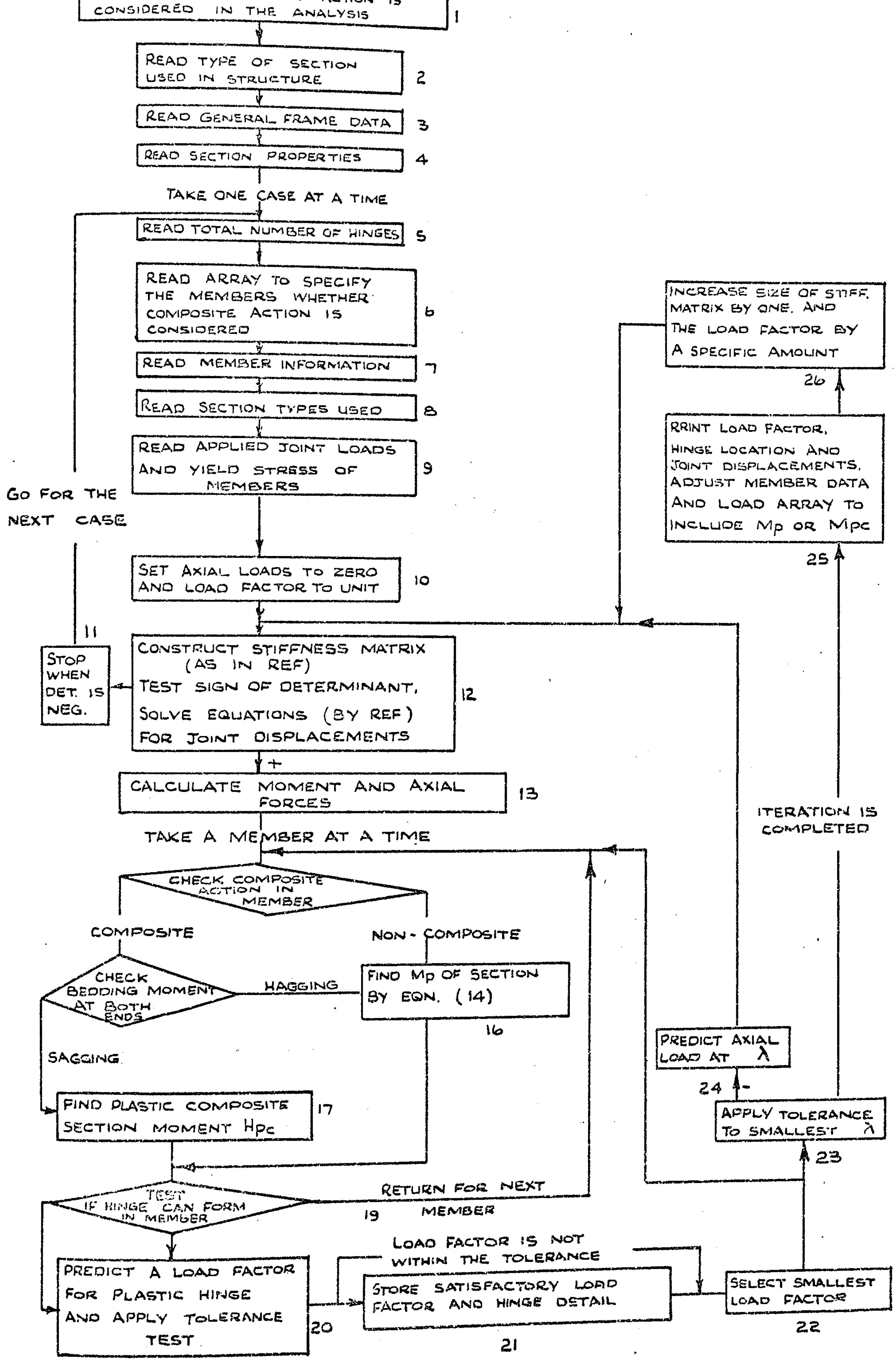


Fig. 6.1. Flow diagram for the composite action analysis programme.

mence the search for the next plastic hinge. The overall stiffness matrix is re-constructed by using the axial forces, and the determinant of which is checked. The analysis is terminated when a negative determinant is obtained, which means that at this load parameter the structure loses its stiffness.

6.3 Examples

6.3.1 Analysis of Four Storey Frames

The first practical structure analysed by considering the effect of composite action is the four storey structure analysed in Chapter (2). As mentioned earlier in that Chapter, the structure may consist of various numbers of intermediate steel frames connected by reinforced concrete slabs of 152.4 mm thickness which are connected at either end to two reinforced concrete shear walls of 304.8 mm thickness. The intermediate frames in this example are arranged at equal spacings of 9.144 m. One of the intermediate frames is taken out of the structure to be analysed elastic-plastically by taking the composite behaviour into account. The effective width "b" of the floor slabs is taken to be of 9.144 m. The composite action is considered only in the beams while it is ignored in the columns. The cube strength of the concrete is taken to be of 2.06 KN/mm^2.

The isolated frame is first analysed under vertical loading. The dimensions of the frame and the working vertical loads are shown in figure (6.2). The sectional properties of the frame were as listed in table (2.2).

The computer analysis shows that the first plastic hinge formed at a load factor of 2.07. The structure collapses at a load factor of 2.98851 after the formation of the 8th hinge before a mechanism developes in the frame. Most of the hinges,

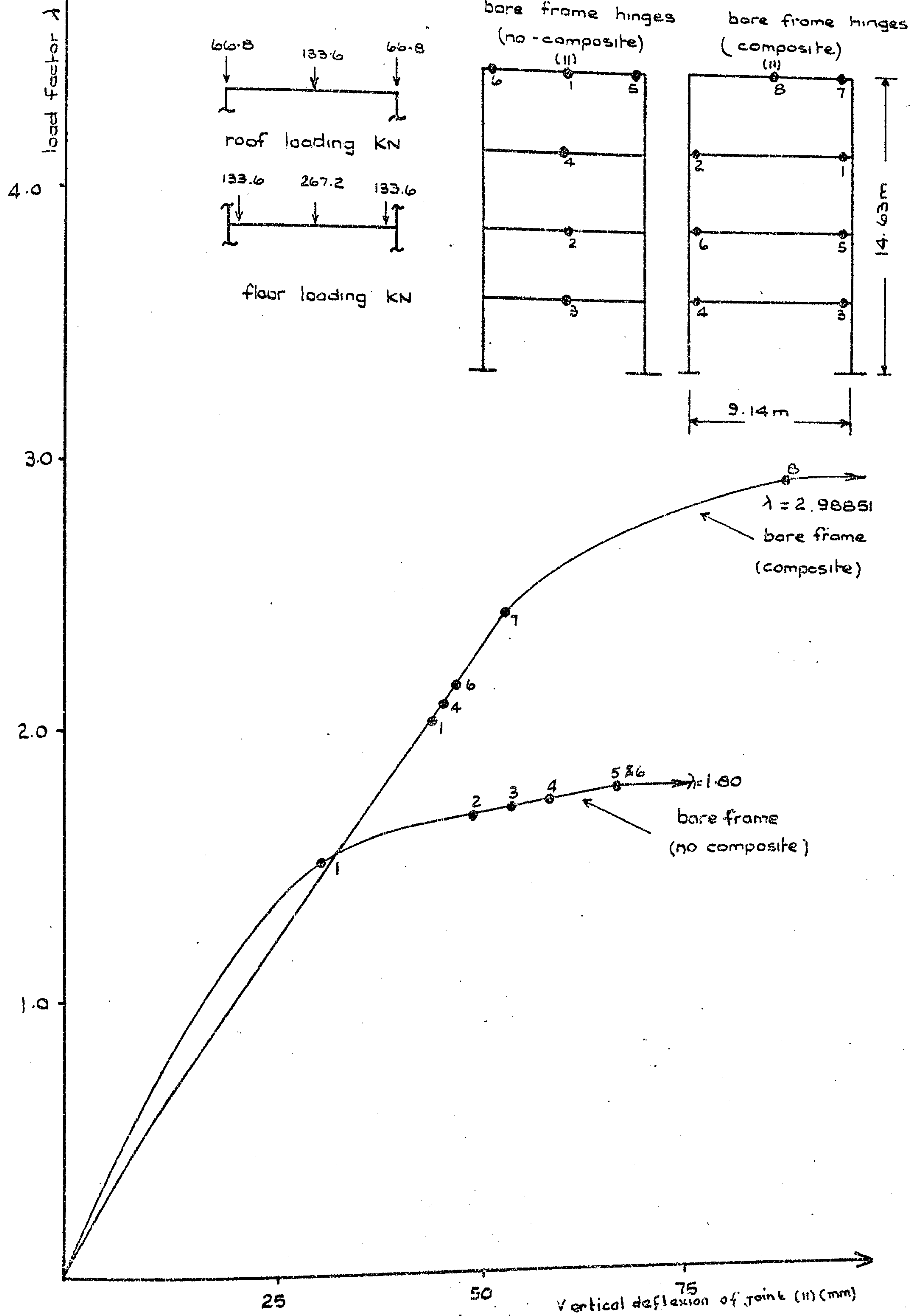
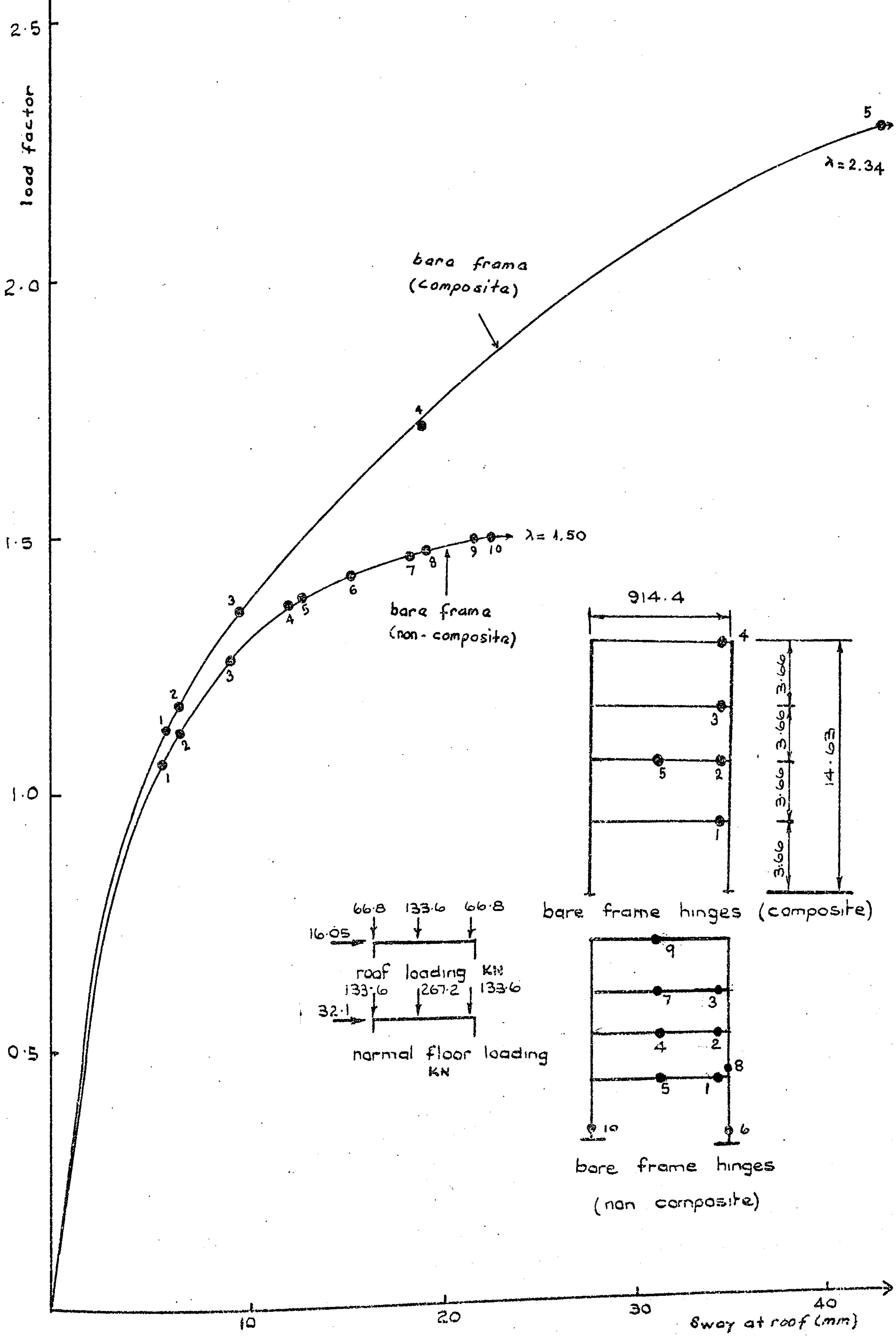


Fig. 6.2. Load deflexion graphs for a four storey frame (under vertical loading)

seven in all, formed at the ends of the beams. The beam in the top floor has one hinge at one end, number 7 hinge. The formation of the seventh hinge caused a sudden increase in the load factor from 2.44017 to 2.98851 at which the eighth plastic hinge formed at the mid-span of the roof beam. This resulted in the collapse of the frame. The vertical deflections of the roof beam at mid-span are plotted against the load factor at each hinge formation. Figure (6.2) shows the load-deflection graph and the plastic hinge pattern at collapse.

The same frame was analysed once again by ignoring the effect of composite action. The first plastic hinge occurs at the mid-span of the top floor beam at a load factor of 1.51 which is lower than that of the first plastic hinge in composite frame by 37%. This time the frame collapses at a load factor of 1.80 which is 66% lower than that of the frame analysed by considering the composite action. This frame collapses with six hinges also without the formation of a mechanism. Four of the hinges formed at the mid-span of the beams, the other two hinges formed at the ends of the top floor beams. It is noticed that while failure of the non-composite frame takes place at a load factor of 1.80, the composite oneis still linear. The load deflection curve and the hinge pattern are shown in the same figure.

Secondly, the isolated frame is analysed under combined loading. Altogether five hinges develop in the frame. Figure (6.3) shows the load-deflection graph, hinge patterns and the sequence of hinge formation. The applied working loads are also shown in the figure. As expected, four of the hinges develop around the column junctions of the beams. After the formation of the fourth hinge at one end of the top floor beam, a considerable increase is observed in the load factor. This leads to


2·5
2·0
1·5
1·0
0·5
load factor
5
λ=2.34
bare frame
(composite)
4
3
2
1
λ= 1.50
1
2
3
4
5
6
7
8
9
10
bare frame
(non-composite)
914.4
3·66
3·66
3·66
3·66
14·63
bare frame hinges (composite)
66·8
133·6
66·8
16·05
roof loading KN
133·6
267·2
133·6
32·1
normal floor loading
KN
bare frame hinges
(non composite)
10
20
30
40
Sway at roof (mm)

the formation of the fifth hinge at the mid-span of the second floor beam at a load factor of 2.33804 which causes the collapse of the frame. This load factor is 55.87% higher than that of the frame analysed by ignoring the effect of composite action. This latter frame collapses after the formation of the tenth hinge at a load factor of 1.50. Altogether four hinges formed at the mid-spans of the beams and three hinges developed at the columns. The rest of the hinges developed at one end of the first, second and third floor beams. The hinge pattern is shown in figure (6.3). Both composite and non-composite frames collapsed without the development of a mechanism.

6.3.2 Analysis of Six Storey Frames

The second practical structure analysed by the programme is the 6 storey structure analysed in Chapter (2), which also consisted of varying numbers of intermediate frames of two unequal bays. The reinforced concrete slabs were 152.4 mm thick. The arrangement of the frames are made at equal spacing of 9.144m. One of the intermediate frames is isolated to be analysed independently by considering the composite action between floors and the supporting beams. The effective width of the slabs is 9.144 m. The "Uc", cube strength of the concrete is taken as 2.06 KN/mm^2.

First of all, the frame is analysed subjected to vertical loads only. The dimensions and the working applied vertical loads are shown in figure (6.4). The sectional properties of the frame were as listed in table (2.3).

The frame collapses at a load parameter of 2.55632. Altogether, 21 plastic hinges developed before collapse as shown in figure (6.4). Eleven of the plastic hinges developed around the column junctions of the beams. The ninth hinge formed at the

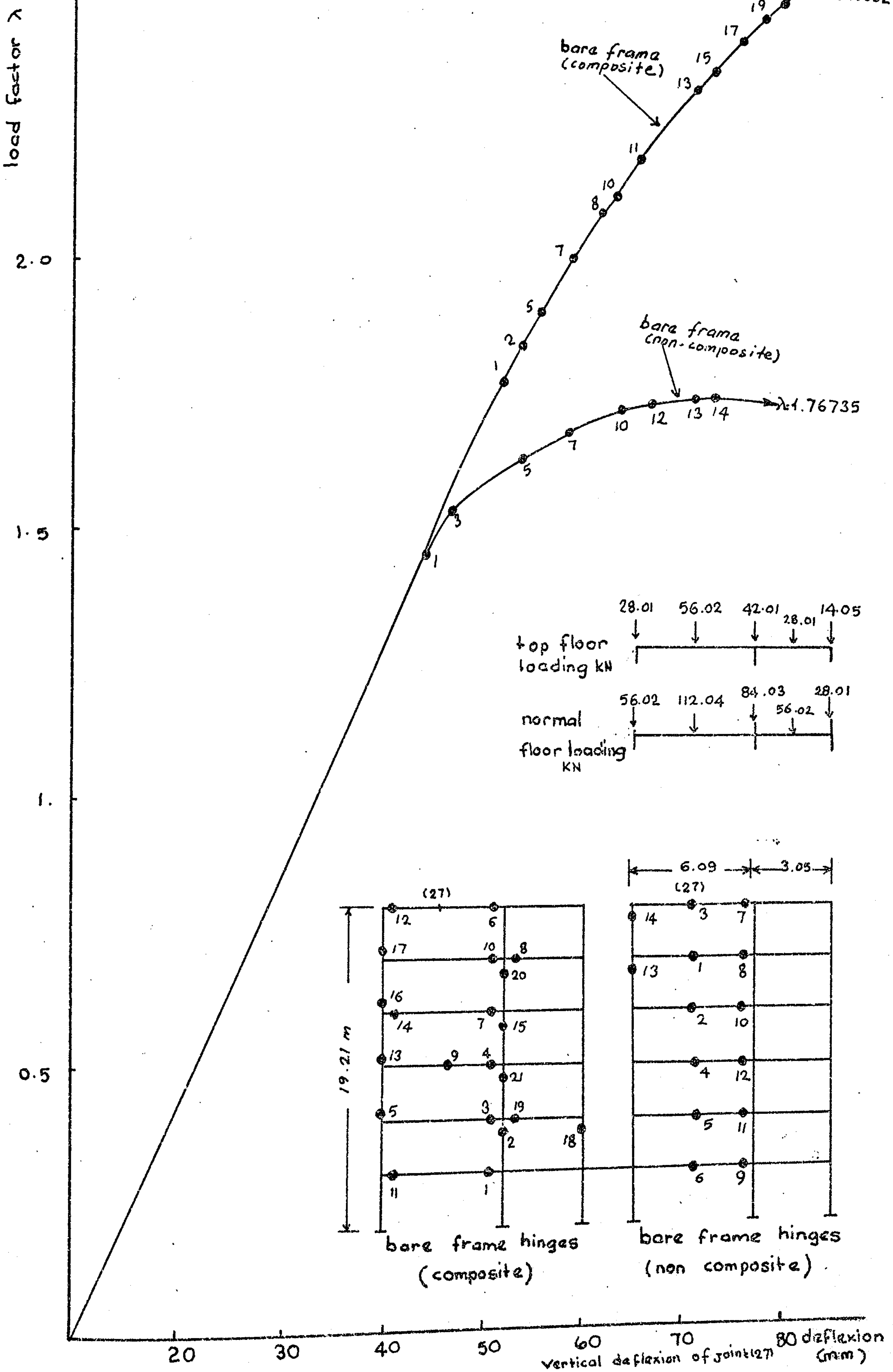


Fig. 6.4 Six storey frame (vertical loading)

mid-span of the beam in the third floor at the wider bay at a load factor of 2.12920. Nine of the hinges formed in the columns. The development of the twenty-first hinge in one of the columns of the third storey caused the failure of the frame. The failure load factor under vertical loading of the frame is higher than that of the same frame analysed by ignoring the effect of composite action, by 45%. This latter frame collapsed with fourteen hinges, also without the development of a mechanism. Its load-deflection graph and the hinge patterns are also shown in figure (6.4). At the load factor of 1.45916, the first hinge occurred in non-composite frame while the composite frame was still elastic. This load factor is 22.7% lower than that of the first plastic hinge in the composite frame. There were two hinges in the columns, six hinges at the mid-span of the beams and a total of six hinges around the column junctions of the beams. At the collapse load factor of non-composite bare frame, 1.76735, the composite frame was still elastic.

The same frame was also analysed under combined loading. This time, the frame collapsed at a load factor of 1.86917 with fifteen plastic hinges. There were no plastic hinges at the mid-span of any beam and all the hinges formed aroung the beam-column junctions. It is noticed that because of the unsymmetrical nature of the frame, the sway deformations of the frame are agravated and as many as five hinges develop in the columns. The failure load parameter of the frame with composite action is, once again, higher than that of the frame without composite action, analysed individually by 33%. This latter frame collapsed at a load factor of 1.40620 with sixteen hinges as shown in figure (6.5). At the collapse load factor of the non-composite

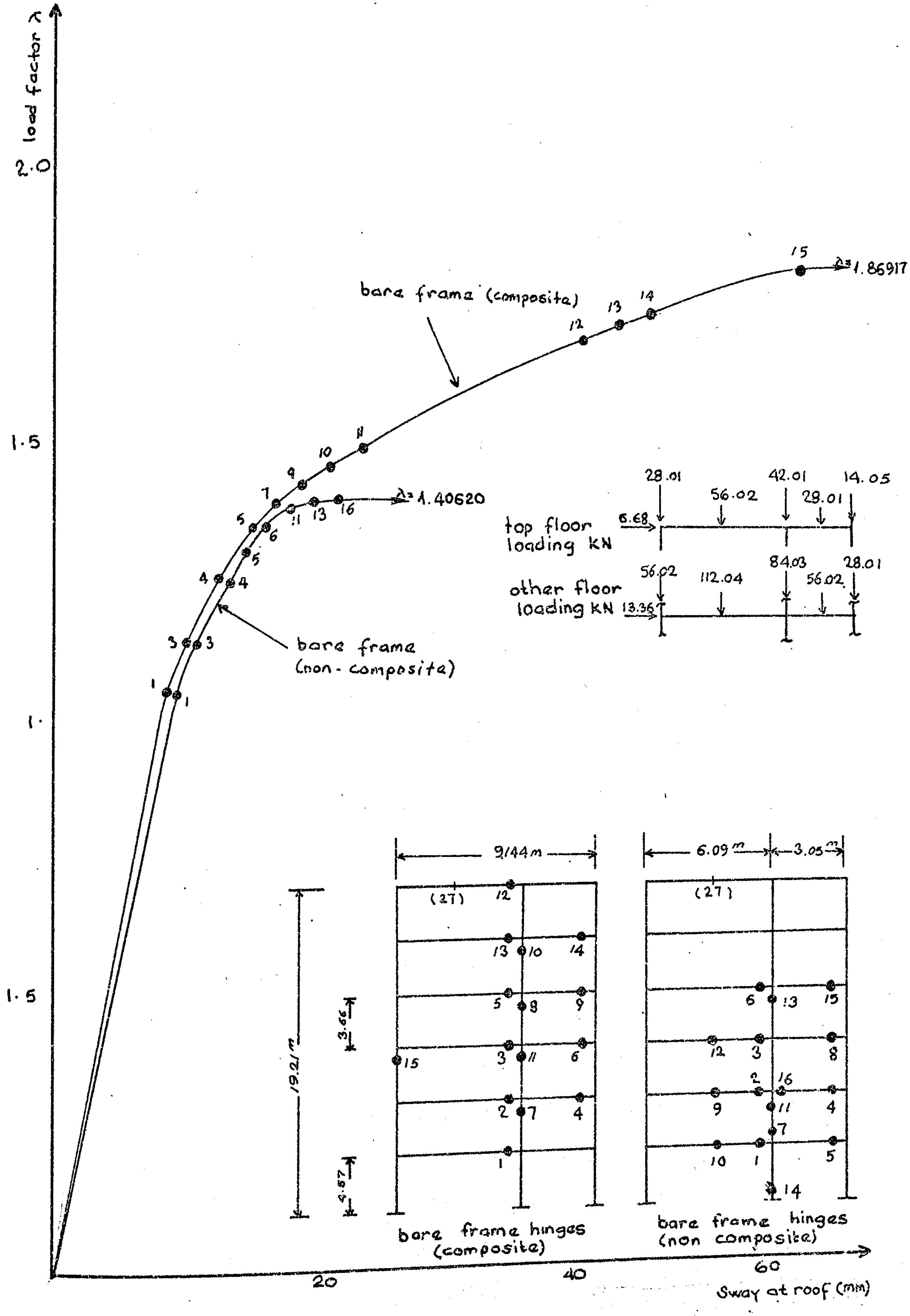


Fig. 6.5 Six storey frame (combined loading)

frame, there were only seven hinges in the composite frame. First four hinges developed in the same members and also at the same load factors.

6.4 The Effect of the Composite Action in the Complete Building Structures

By the modified elastic-plastic analysis programme of the complete building structures, a number of examples are solved to demonstrate the advantage gained, by the inclusion of the composite action, on the load carrying capacity of the structures.

The sample structures analysed by this programme are the four and six storey structures analysed in Chapter (2). The effective width and the thickness of the slabs, and the cube strength of the concrete are the same as those given in Section 6.3.

6.4.1 Analysis of Four Storey Structures

In this group, four separate analysis were carried out, by taking the structure with different number of internal frames each time. The results obtained from the analysis are tabulated and summarised in table (6.1). The table shows the load, the maximum total number of hinges in both structure and one frame at collapse. For comparison purposes, the results obtained from the bare frame analysed individually is also shown in the table. As an example, only the analysis of the structure with ten intermediate frames is briefly described below:-

The dimensions of the structure and the frames as well as the working combined loads is shown in figure (6.6). The sectional properties were as listed before in table (2.1).

Because of the symmetrical arrangement of the intermediate frames, only five of the frames are considered in the analysis. The plastic hinges were searched in those five frames, if any of them had a plastic hinge, the corresponding frame on the other side of the symmetry axis would have a plastic hinge as well. As mentioned earlier in Chapter (2), plastic hinges develop in pairs and symmetry is preserved throughout the analysis. The hinge pattern is shown in figure (6.6), where it is seen that the first plastic hinges appeared at a load factor of 1.52 in the centre frames simultaneously, i.e. frame numbers five and six. Later in the iterations, the hinges spread from the central frames to the outer ones. For instance, the second group of hinges appeared in frame numbers four and seven. The structure collapsed at a load factor of 2.51084 after the formation of fifty-two hinges. The central frames five and six had seven hinges each, frame numbers two, three, four and also seven, eight and nine had five plastic hinges each, and finally the outer frames, frames one and ten had four hinges each at collapse. Only one hinge developed at the mid-span of the first floor beam in frame numbers five and six at the same time and which caused the failure in these frames and, therefore, in the structure. Forty of the plastic hinges formed at column junctions of the beams, ten of them formed in the columns. The collapse load factor of 2.51084 is 38% higher than that of the same structure analysed without composite action. It is also higher than that of the bare frames analysed independently by 67%. It can be seen from table (6.1) that the same load factor is also 7% higher than that of the composite frames analysed individually.

6.4.2 Analysis of Six Storey Structures

Three analysis were performed in this group of examples on the six storey structures with three, five and seven intermediate frames respectively. The results obtained from the

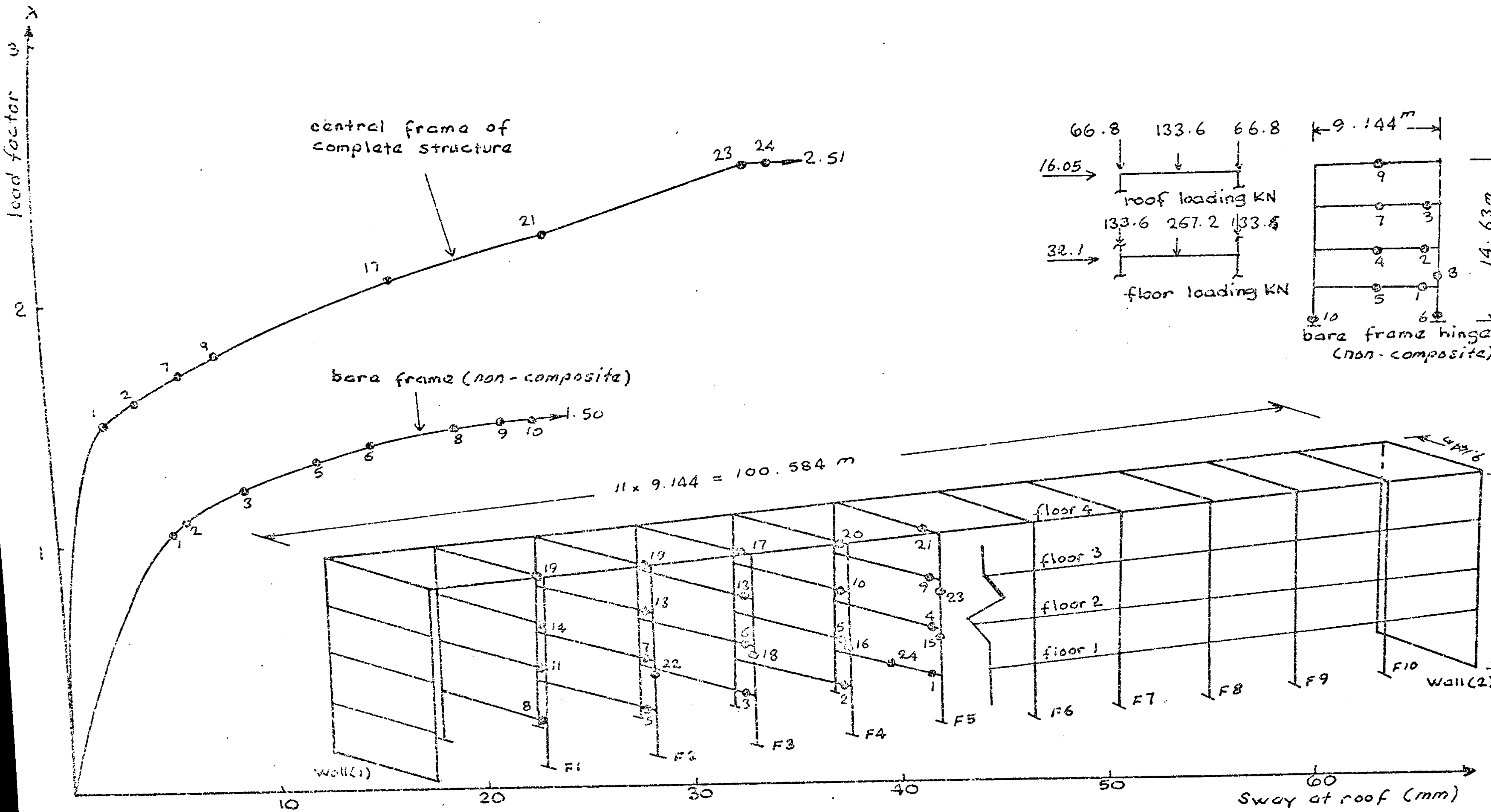


Fig. 6.6 Four storey structure (composite)

Total Number of Intermediate Frames	Failure Loads Composite	Maximum No.of Plastic Hinges	Maximum No. of Plastic Hinges In One Frame	Failure Load / F. Load (Non-Composite Structure)	Failure Load / F. Load of Bare Frame (Comp)	Failure Load of Complete Structure (Non-Comp)
3	2.96473	23	8	1.56	1.267	1.89449
5	2.92454	31	7	1.55	1.2498	1.89022
7	2.75882	43	7	1.45	1.118	1.89443
10	2.51084	52	7	1.38	1.073	1.82555
Bare Frame	2.34 with composite action	-	5	-	1.	1.50 Bare frame with no composite action

TABLE (6.1)

SUMMARY OF RESULTS OBTAINED FROM THE ANALYSIS OF FOUR STOREY STRUCTURES analysis of the bare frames are summarised in table (6.2) which shows the collapse load factors, the maximum number of hinges in both the structure and one of the intermediate frames at collapse. The results obtained from the bare frame analysis are also included in the table. As an example, the analysis of the structure with six storey seven frames is given below:-

The overall dimensions of the structure and the frames, together with the working vertical and horizontal loads were shown in figure (6.7). The sectional properties of frames were as listed before in table (2.3).

The first hinge appeared in the central frame, frame number 4, at a load factor of 1.66710. The structure finally collapsed at a load factor of 2.40983 under combined loading with a total of 114 plastic hinges as can be seen in figure (6.7). Once again, by making use of the symmetrical nature of the structure, the computer time and storage were saved by considering only four intermediate frames in the analysis. Plastic hinges were inserted symmetrically and also in pairs, except in the centre frame. A maximum number of eighteen hinges formed in the central frames and seventeen hinges formed in frame number 2 and 6. The rest of the frames had a total of sixteen hinges each. In this structure, as many as 53 plastic hinges developed in the columns because of the unsymmetrical nature of the intermediate frames. The collapse load factor of the structure is higher than those of the same structure and the bare frame analysed independently, without giving any consideration to the composite action, by 36.5% and 69.7% respectively. This load factor is also 28.9% higher than that of the bare frames analysed in Section(6.3) (sub-section 6.3.2) with composite action.

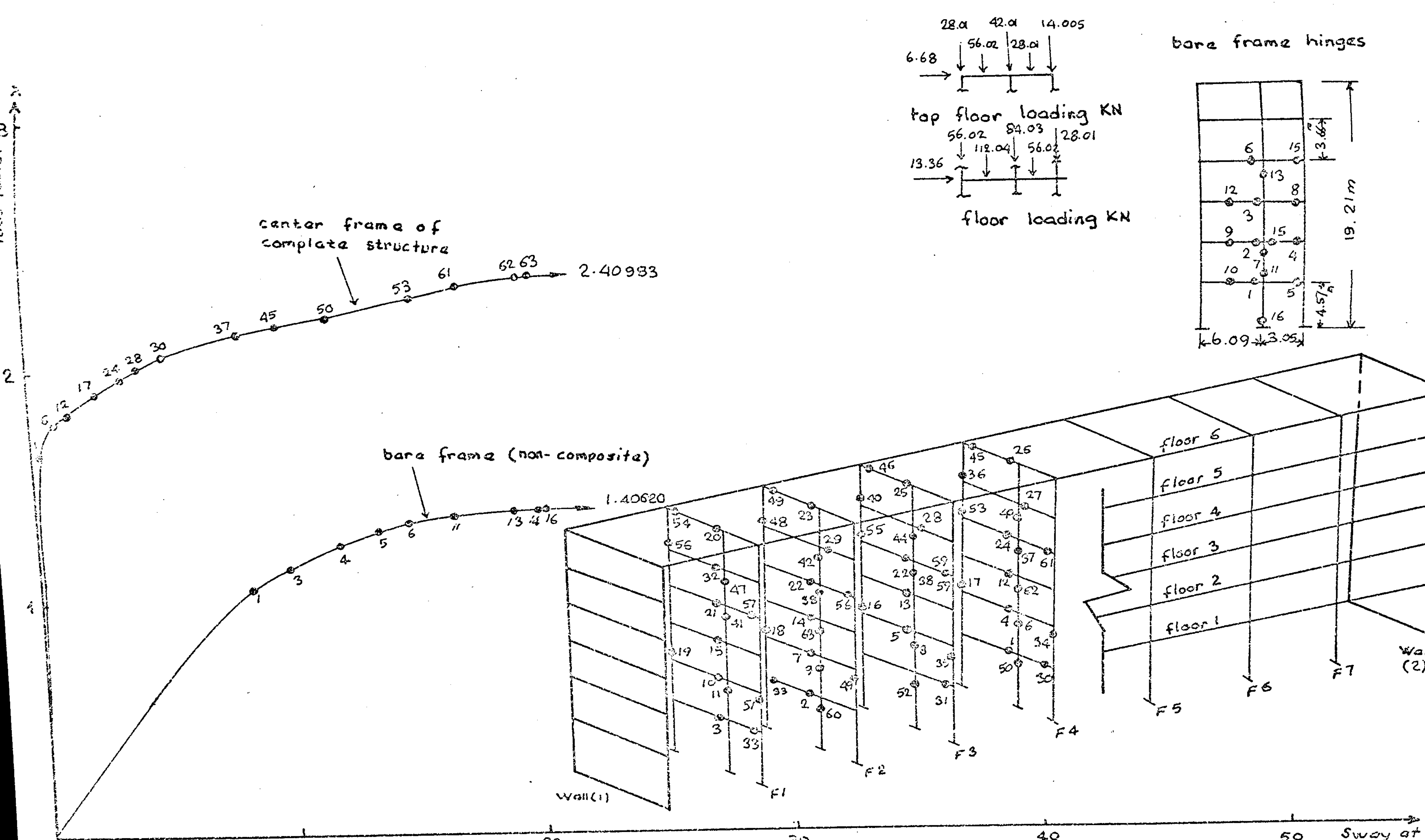


Fig. 6.7. Six storey structure (composite)

Total Number of Frames	Failure Load	Maximum No. of Plastic Hinges in Structure	Maximum No. of Plastic Hinges in One Frame	Failure Load / F. Load (Non-Composite) Structure	Failure Load / F. Load of Bare Frame (Comp.)	Failure Load of Complete Structure Without Composite
3	2.44425	33	11	1.383	1.309	1.76678
5	2.41678	69	17	1.367	1.2934	1.76794
7	2.40983	114	18	1.365	1.289	1.76501
Bare Frame	With composite 1.86917	-	15	-	1.000	Bare Without Composite 1.40620

TABLE (6.2)

SUMMARY OF THE ANALYSIS OF SIX STOREY STRUCTURES

6.5 Conclusion

In this Chapter, an elastic-plastic composite analysis programme for the frames was shortly described and a number of examples were solved. By making use of this programme, the existing programme, described in Chapter (3), was modified to study the effect of the composite action on the behaviour of the complete structures. A number of examples were also given.

By comparing the results obtained from composite and non-composite analysis of both the bare frames and the complete structures, the following conclusions may be drawn:-

(i) - The load carrying capacity of a structure is increased considerably by the inclusion of the composite action.

(ii) - The hinge patterns, the sequence of hinge formation and the total number of hinges developed in the structures were changed. It was observed that the composite action stopped the hinges forming at the mid-spans of the beams, therefore, most of the hinges developed around the beam-column junctions.

(iii) - It is observed that, to increase the number of the intermediate frames, caused the load factors at collapse to drop. This is clearly seen in table 6.1. In the analysis of the four storey complete structures, the collapse load factor of a structure with three frames, 2.96473, decreases while the total number of the intermediate frames is varied from three to ten. For instance, it becomes 2.92454 with five frames, 2.75882 with seven frames and finally 2.51084 with ten intermediate frames. This was because of the effect of the grillage system. With less frames, the grillage is more stiff than the frame system, so more loads are naturally attracted by the grillage system of walls and floor slabs. On the other hand, an increase in the number of frames reduces the relative stiffness of the grillage, therefore, most of the wind loads are transferred to the frames and this causes a drop in the collapse load factors. This phenomenon is also observed in the six storey complete structures but not as much as in the four storey ones. There was a slight decrease in the load factors. For instance, the load factor of the six storey structure with three frames was 2.444425, later it became 2.41678 with five frames and 2.40983 with seven frames.

CHAPTER 7

GENERAL CONCLUSIONS AND SUGGESTIONS FOR FURTHER WORK

A method for the elastic-plastic failure load analysis of complete building structures consisting of a number of parallel frames of any shape made out of steel, together with reinforced concrete floor slabs and shear walls was described in Chapter (2). An accurate approach to them required a knowledge of the separate amount of the external loads transmitted individually to the frames and the grillage system. The relative stiffness of the frames and the grillage system throughout the loading history of the complete structure, up to and including collapse, was considered. Investigation into the variation of the load transmitted to a frame by evaluating its α value showed that the development of plasticity in it caused a sudden drop in its share of the load. Therefore, the stiffening effect of the slabs was shown to be changing. This indicated that, as plastic hinges develop, a proportional increase in the applied external loads did not lead to a similar increase in the loads carried by individual frames as could be seen in figures (2.6) and (2.8). Hence, a fresh calculation of α was required each time a hinge developed.

A number of examples were given so as to demonstrate the effect of the grillage system on the load carrying capacity of the structures. It was shown that the grillage system of the floor slabs and the shear walls increased such a capacity of the complete structures by a considerable amount.

Analysis of a group of practical structures showed that most of the wind loads were transferred by the slabs to the end walls and then to the foundations. The vertical loads on the other hand were carried mainly by the frames. This phenomenon was also concluded by Creasy (55). The large structures analysed in Chapter (2) collapsed before the development of a mechanism in any of the intermediate frames. This is simply because of the effect of the axial loads in the members. The excessive vertical loads dominated the failure loads of the structures. The method proposed in Chapter (2) was applicable only to structures consisting of concrete grillage system and steel frames. But, nowadays, many frames are constructed in concrete. Therefore, a suitable technique for the analysis of the complete structure comprising concrete cladding system and concrete intermediate frames up to failure should be produced.

A general computer programme was developed in the course of the present work in Atlas Autocode which was described in Chapter (3). In the programme an allowance was given for the assymmetrical arrangement of the walls and frames.

The overall stiffness matrix for a frame with hinges was constructed in the manner recommended by Majid and Anderson (23), in order to save computer storage. This matrix was used repeatedly in two different manners. Firstly, it was used when the influence coefficient matrix F for the frames was being constructed. In that case, the number of columns in the applied load vector was equal to the number of storeys in the frame. Each load column consisted of zeroes except for a unit horizontal load at the beam level of the floor. During the elasto-plastic analysis, the stiffness matrix was also used repeatedly. On this occasion, the loads were presented in a single vector with reduced plastic hinge moments Mp′, for the plastic hinges appearing at the appropriate places in this vector. This time the stability functions (10) $\phi_1 - \phi_5$ were also included, with their actual values, in the analysis.

The iteration process adopted in the method presented in Chapter (2), for tracing the load-deflection history of the entire structure up to failure, was from one hinge to the next.

A number of practical large structures were analysed, as examples, by this computer programme and presented also in Chapter (2). In these structures, owing to the symmetrical arrangement of the intermediate frames, plastic hinges developed in pairs and symmetry was maintained throughout the analysis. The full effect of the axial loads in reducing the stability of the frames and the plastic hinge moments of the sections were considered. In the programme, both rectangular and universal sections were allowed to be used in the structures.

The hinge patterns at collapse were also rather interesting. It was noticed that the order of plastic hinge formation changed from frame to frame and the structures collapsed without the development of a mechanism. It was seen that the development of plasticity spread from the internal frames to the outer frames in the structure.

Computer time was saved by facilitating the insertions of more than one hinge simultaneously. It was observed that simultaneous hinge formation occured in the early stage of loading. But in the later stages, nearer to collapse, hinges were inserted one at a time (two at a time to preserve symmetry). In this manner, the accuracy of the analysis in the advanced stages was not violated.

The preparation of an elastic-plastic analysis programme that allowed for shape factors other than unity, strain hardening and large changes of geometry would enable a more accurate analysis. This could result in more economical analysis of the buildings. Therefore, they could be included in the programme.

It was essential to test the validity of the method, proposed in Chapter (2), as extensively as possible. The ideal situation would be to build several full size complete building structures and test them under experimental conditions until failure occurs. This, of course, was impossible owing to the sheer size of the problem. It would be necessary to reduce the size and consider the model structures. Since it was considered that the behaviour of model structures could safely be used to predict the behaviour of full scale tests. The experimental set up and the testing procedure followed in tests were outlined in Chapter (4). The results obtained from theoretical and experimental analysis were compared, in Chapter (5), with the aid of a general computer programme described in Chapter (3).

Most of the single storey structure tests showed an acceptable agreement with the theoretical results as shown in table (5.1). These tests demonstrated that, in the case of thin slabs, collapse of a structure might take place by the buckling of the slabs. The method presented in Chapter (2) did not consider the buckling of the plates. More accurate results might be obtained by the inclusion of such phenomenon in the analysis programme outlined in Chapter (3). However, thin slabs are not practical unless stiffened by cross beams which increase the construction cost. On the other hand, with the thicker slabs, it was found to be too difficult to determine the geometry of a structure to provide for a failure occuring at the same time in the grillage system and the frames. By reducing the number of intermediate frames in structures 4B and 5B of single storey structures tested, good agreements were obtained with the theoretical results by obtaining the discrepancies of 3.8 % and 5.8 % with structures 4B and 5B respectively. The interest should, therefore, be particularly focussed on achieving design economy by reducing the number of the intermediate frames required for a given grillage system. It might be possible to utilise the initial values of the loads transmitted to the central frame of a structure.

A number of tests were also carried out on two storey structures, but none of the test results in this category did agree with the theoretical results. It was shown in table (5.2) that all experimental frames failed at much lower loads. This indicated that the grillage system was collapsing earlier than the frames. As mentioned before, it was quite difficult to decide the optimum thickness of the grillage members, particularly in multi-storey structures, in order to achieve the simultaneous failure of floor slabs and frames. Further tests should, therefore, be carried out on two storey structures to cover all the factors playing a major role in selecting the thickness of the grillage members. This can form a topic of its own when carrying out further research on the optimum thickness of the floor slabs and shear walls.

All the tests carried out on the model structures exhibited high creep. A considerable discrepancy was observed between the theoretical and the experimental deflections. It may be possible to increase the accuracy of the analysis by giving a specific consideration to creep. On the other hand, the above tests could be repeated by using some material to construct the grillage system that might exhibit less or no creep.

There were many advantages to be gained by considering the structure as a composite system rather than a series of individual components. Hence, the well known elastic-plastic analysis programme (23,24) was modified in Chapter (6), to allow for the composite action in the rigidly jointed frames. In practice, wall and floor panels are usually attached to the bare frame. It has already been shown that even light walls and roof cladding had a significant stiffening effect (16,39,40). Moreover, when the floor slabs are connected to the beams by shear connectors to provide the continuity between two components (beams and floor slabs or walls and columns), then the slab and beam (or wall and column) can act together as a composite section and the stiffness of the complete structure is then considerably increased. By the programme outlined in Chapter (6), the four and six storey bare frames were analysed under the vertical and combined loading, in turn, to exhibit the increasing effect of composite action on the failure loads. Note that when a beam is subject to a sagging moment and the slab is in compression, the plastic hinge moment of the beam increases substantially. On the other hand, hogging moments cause tension in the slabs and thus the plastic hinge moment of the beam is considered to be unaffected in this case. Therefore, each member had two different Mp values.

In order to test the validity of the composite action programme, a number of tests should be carried out, and the experimental results should also be compared with the theoretical ones.

There should be many design advantages in using composite action between slabs and beams. The interaction between these two elements is by no means the only one in a structure. Whenever any continuity exists between two components, there is bound to be a certain degree of structural interaction, and basic philosophy of composite design is that this interaction should be recognised and allowed for during the design process. It was usual for specifications to allow for no provision to be made for the effect of cladding and to stipulate that all wind loads should be carried on the bare steel frame. This was already disproved. The grillage system of shear walls and floor slabs play an important role by carrying a substantial portion of the wind loads (Chapter 2). It was also concluded in Chapter (2) that the carrying capacity of a building depends considerably upon the relative stiffness of the grillage system and the frames at all stages of the loading process. Almost all cladding would increase the stiffness of the structure. The effect of cladding should, therefore, be made use of in design. It would be appreciated then that if such an elasto-plastic composite method were devised for designing a steel framework by considering the composite action. To do this, the elastic-plastic design method for sway frames, produced by Majid and Anderson (22) could be utilised. By combining the composite action programme of Chapter (6) and the design programme of Majid and Anderson (22), a composite design programme might be produced.

By combining the two computer programmes outlined in Chapters (3) and (6), a new computer programme was obtained.

This was basically a modification of the programme written for the elastic-plastic failure load analysis of the complete building structures. For the examples, the same type of multi-storey structures of Chapter (2) were analysed by considering the composite action between slabs and beams in complete buildings. The results were compared with the previous ones (without composite action) and the bare frames (with composite action). These results were rather interesting, mainly two facts were observed. Firstly, the carrying capacity of complete structures increased by a considerable amount as could be seen in tables (6.1) and (6.2). Secondly, the hinge patterns were completely different from those of non-composite structures. Due to composite action between slabs and the supporting steel beams, most of the plastic hinges developed at the ends of the beams and columns. Since,composite action prevented the hinges to form at mid-span of the beams where usually a sagging moment is present. It was observed that only one or two hinges could form at the mid span of the beams when the structure is approaching collapse. The structures analysed with composite action also collapsed before the development of mechanisms.

In the computer programmes mentioned in Chapter (6), the properties of the members in the frames were taken from the "Handbook on Structural Steelwork" (26). Thus, the effect of the slabs on the member properties of the frames was not included in the composite action. Only the increasing effect of the slabs on the plastic hinge moment of the section was considered. It was quite right where the hogging moment was present, but it could cause inaccuracy in the analysis where the sagging moment was present. In this case, the stiffness of the structure was underestimated. Thus, in some examples presented in Chapter (6), an excessive side sway was observed while it was compared with the deflections of the bare frame analysed independently without giving any consideration to composite action. A more true approach to the problem would be achieved by considering the joint properties of the composite element, i.e. slabs and beams.

In both programmes mentioned in Chapter (6), the effect of reinforcement on the composite action was not considered that would have an effect on the collapse load. The composite action between wall panels and the steel columns was also not included which would have a considerable effect on load carrying capacity of the structures. These two aspects should, therefore, be taken into account when further work is carried out in this field.

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PUBLISHED WORK

The work presented in Chapters 2 - 5 of this Thesis has been published in the form of a paper, written jointly by Prof. K. I. Majid and the author. The paper, entitled "The Elasto-Plastic Failure Load Analysis of Complete Building Structures", was published in "Proc. Instn. Civ. Engrs.," Vol. 55; September, 1973.

A paper on the composite action between structural members in both bare frames and complete building structures, based on the work presented in Chapter 6, has been submitted to the "Institution of Civil Engineers", for consideration for publication.