EFFECT OF INFILL AND OPENINGS IN SHEAR WALLS ON THE LATERAL RESISTANCE OF TALL BUILDINGS

THESIS

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IN STRUCTURAL ENGINEERING

BY

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بسم الله الرحمن الرحيم

TO MY

DAUGHTER

SALLY

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ABSTRACT

The behaviour of high rise buildings under lateral loads is affected by the presence of the infill walls and openings especially in shear walls.

In this work the effect of the presence of infill using different types of filling materails, position and size of openings were investigated.

A comprehensive review for the previous work in this field has been made. The review covers, analysis of frame-shear wall interaction, analysis of coupled shear wall and analysis of infilled frames.

Modulus of elasticity and Poisson's ratio for three types of brick walls, commonly used locally as infill were determined experimentally. The approach of using an equivalent diagonal strut to replace the infill panel was used in this study.

A computer program based on stiffness matrix method was used to obtain the straining actions and deformations in the various structural elements for the different parametes.

The analysis of the different cases and parameters studied represent the effect on the deformation and the ductility of such structures. The comparison for the results also shows the imparative gain due to adequate choice for the type of infill, openings location and dimensions taking ecconomical aspects into consideration.

Main conclusions reached in this work and recommendations for future studies in this area are presented.

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NOTATIONS

The following notations are those generally used in this thesis, other notations required for the purpose of the analysis will be mentioned following their appearance.

SYMBOLS

H	total height of the building
h	storey height
L	spam of frame
m	spacing of frame
N	number of stories
n_1	number of spacing
n_2	number of span
b	width of shear wall
b_w	opening width
h_w	opening height
A_1	cross-section area of exterior columns
A_2	cross-section area of interior columns
I_1	second moment of inertia of exterior columns
I_2	second moment of inertia of interior columns
I_w	second moment of inertia of connecting beam
H'	height of tested brick
L'	length of tested brick
B	width of tested brick
b_e	effective width of the diagonal strut

t_{panel} thickness of the infill

D length of the diagonal strut

θ angle of panel diagonal to the horizontal

P_w uniform lateral load

E_i Young's modulus of the tested wall

μ_i Poisson's ratio of the tested wall

μ_L local Poisson's ratio of the tested wall

μ_g global Poisson's ratio of the tested wall

δ lateral deflection of structure

X_1, X_2 half width of shear wall segment

Y_1, Y_2 half depth of connecting beam

KM element stiffness matrix

K_r stress-strain matrix

K_{s1} rotational stiffness of spring at end 1

K_{s2} rotational stiffness of spring at end 2

U_{xy} vector of nadal deformation

CHAPTER 1

CHAPTER 1

INTRODUCTION

The definition of tall building depend first on such considerations as where the building is located. A twenty-storey building may be considered tall in Egypt but is not considered tall in New York City. It also depends on who is asking the question. For the structural engineer a tall building can be defined as one whose structural system must be modefied to make it sufficiently economical to resist lateral forces due to wind or earthquakes within the prescribed criteria for strength drift and comfort of the occupants(1).

The early buildings consisted of beam-columns frame system which made the construction of taller buildings relatively expensive and therefore, economically unfeasible. In the early 1950's the introduction of shear wall type of construction, new technology of material, new cosntruction methods and bigger cranes opened up the possibility of using concrete in apartment and office building as high as 30-stories. Taller buildings still remained economically inattractive and technically inadequate because the shear walls were mostly used in the core of the building were relatively small in dimension compared to the height of such buildings leading to insufficient stiffness to resist lateral loads.

Structural system for high-rise buildings with the increasing height, new concepts evolved to economically provide resistance to lateral force due to wind and earthquake. This evolution is shown in figure (1.1).

As can be seen in the figure, in building up to 15-stories frame action usually suffices to provide lateral resistance.

The rigidity of frame building for 20-storey height is mostly insufficient and sway due to wind may begin of control the design. Introduction of shear walls which interact with the frame increases the total rigidity of the building beyond the sum of two individual components. The frame tube consists of a closely spaced grid of exterior columns connected with beams. It is an efficient system to provide lateral resistance without interior columns starting at 30-35 storey. When the sway or wind stresses begin controlling the design, the framed tube is supplemented by a core to create the tube-in-tube system which is essentially a shear wall-frame interactive system with all its advantages when we reach 80-100 stories, the multicall framed tube system is sufficient to provide lateral resistance.

In Egypt the common main structural elements to resist the lateral loads are mostly frames and sometimes shear walls. It is economic to have the interaction between frames and shear walls into consideration in resisting the lateral loads. Some efforts are spent local by certain research workers to evaluate such a combined system under local condition and practices.

The calculated valus for deforamtions and straining actions of structures under the action of lateral loads are noticed not to be equal to the actual values. This fact leads to many factors that were not taken into consideration during design such as:

1- The infill which is used for partitioning and separating the dwellings in buildings.
2- Position and size of openings in shear wall either in vertical or horizontal direction.
3- The configuration of the shear wall at the base.

In this research work studies were carried out to analyze the interaction between frames and shear walls taking into consideration the effect of infill. The infill has been considered as a diagonal bracing in the frame to assist in resisting the lateral deformations caused by wind pressure or seismic actions as shown in figure (1.2). This system which consists of frames and infill is called infilled frames.

In this work modulus of elasticity, poisson's ratio and the global behaviour for three different types of brick walls which are commonly used locally as infill were investigated and evaluated experimentally.

The effect of openings in shear walls is also studied and hence the effect of such openings on the behaviour of the global structure as one unit.

The stiffness matrix method for special plane frame element has been used to analyze the structure with a reasonable accuracy to obtain the deformations and straining actions in the different elements of the structure.

A set of design curves has been developed which enable the designer of such buildings to take into account the effect of different possible types of local infill as well as the geometry of the openings in shear walls in the design of frames and shear wall system. Some recomendations are also presented concerning the limits and conditions under which the conventionally infilled frame in tall buildings can be efficient and how it can be easily evaluated to resist lateral loads.

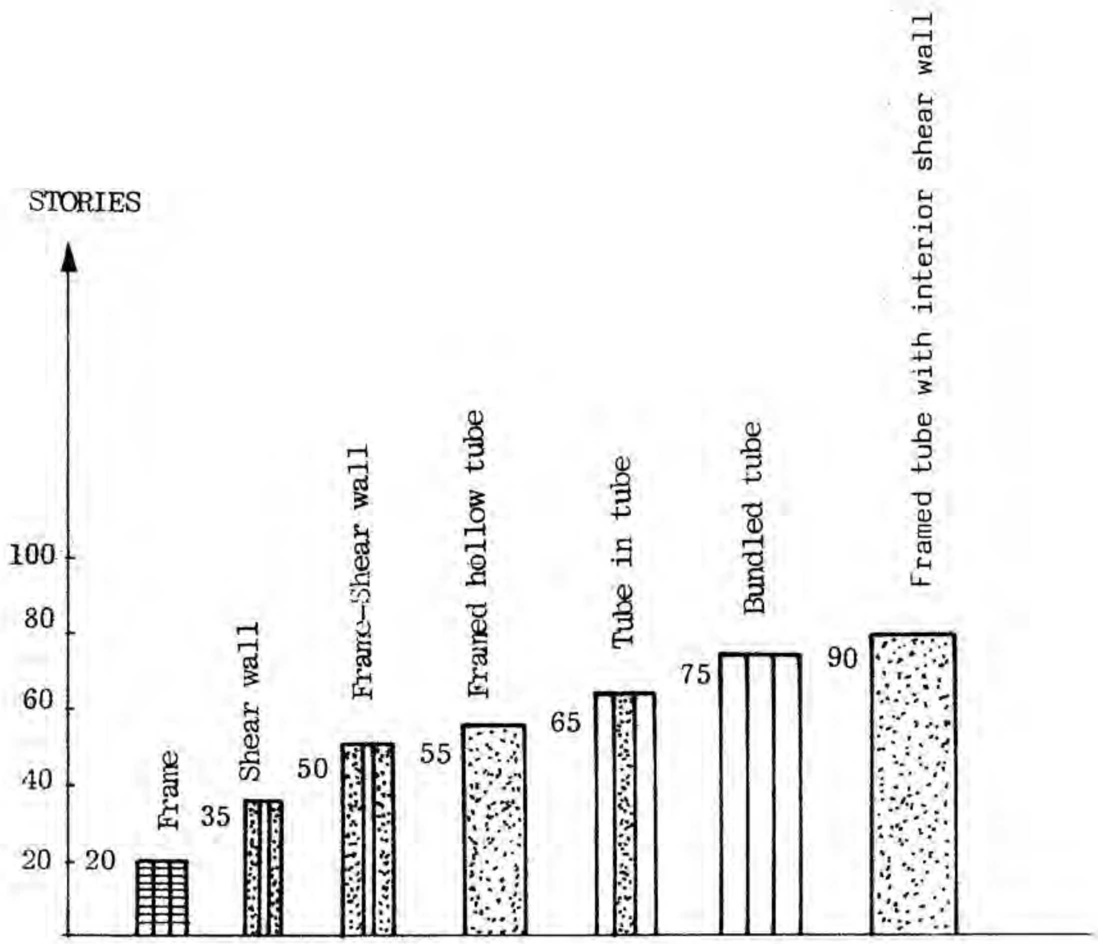
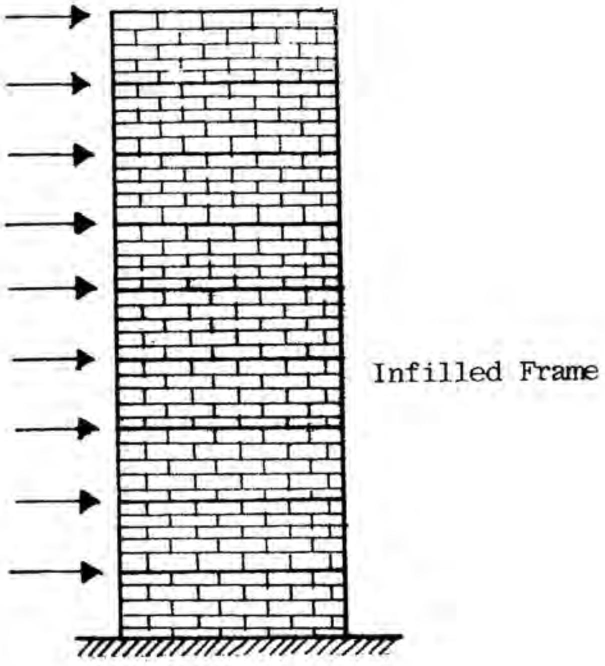
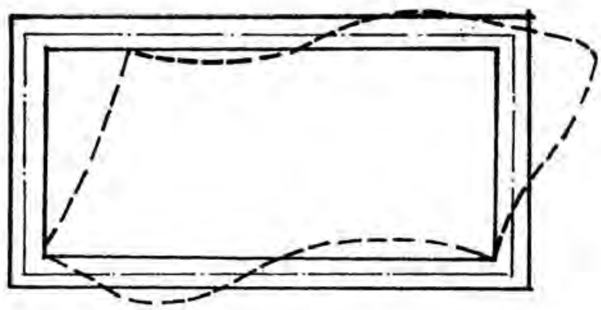
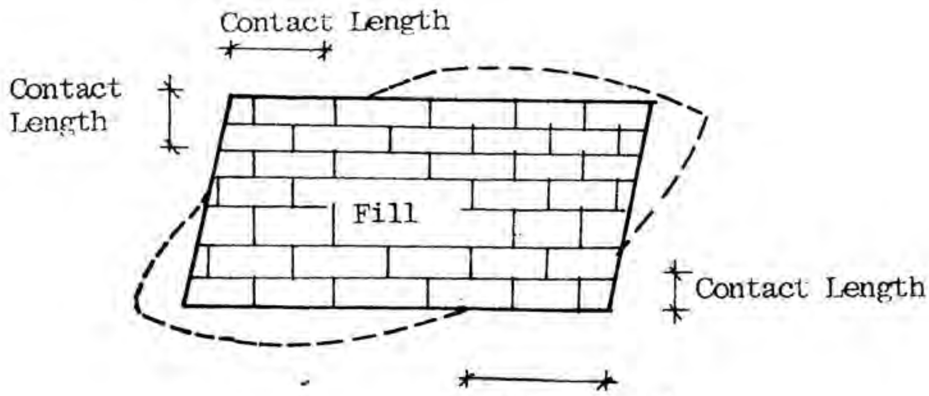


Fig. (1.1) Suitable Systems for Concrete Tall Building.





Deformation of Frames without Infill



Deformation of Fill

Fig. (1-2) Infilled Frame and Deformations

CHAPTER 2

CHAPTER 2

REVIEW OF PREVIOUS WORK

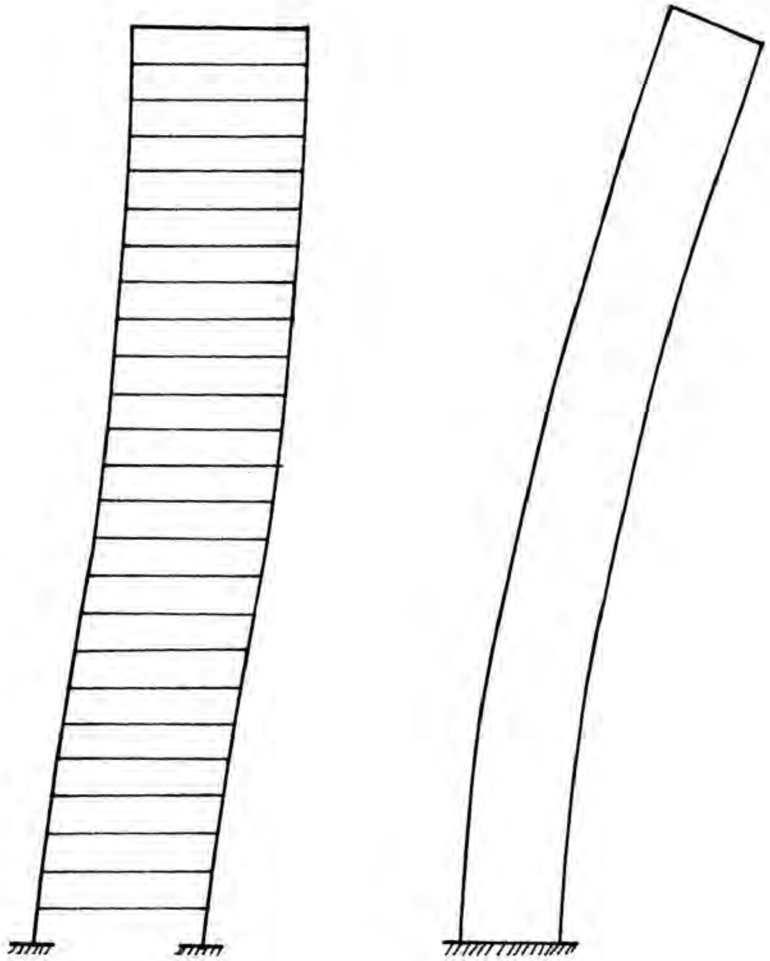
2.1 FRAME - SHEAR WALL INTERACTION

2.1.1 Introduction

Many types of buildings are composed of shear wall coupled to frames. It has been common to design the frame to resist the vertical loads and the shear walls to resist the lateral loads. The strength and stiffness of the shear walls was estimated by the same methods used for frame member.

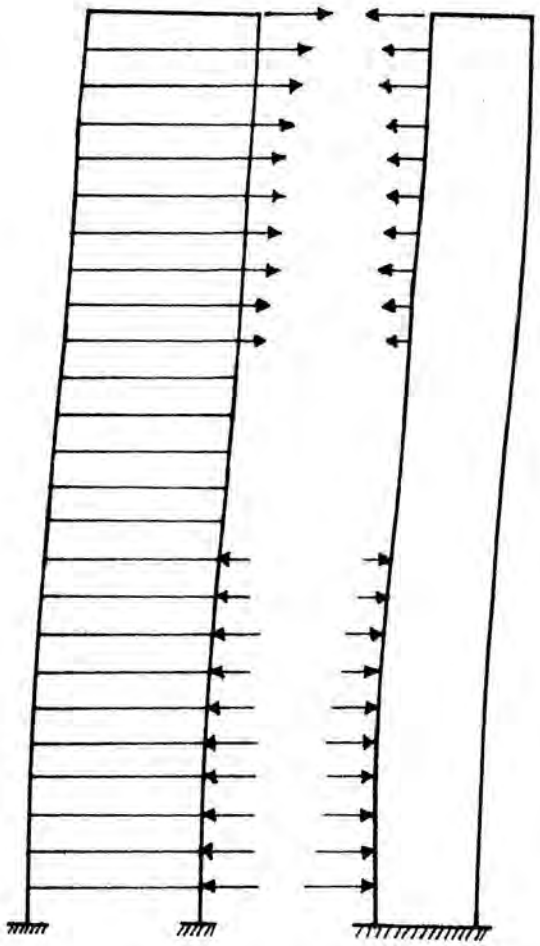
This procedure is acceptable for low buildings, but as more slender structures are considered, the interaction between the shear wall and the frame becomes significant. If the frame alone resisted the lateral loads it would deflect as shown in fig. (2.1.a). On the other hand the free shear wall would deflect as shown in fig. (2.1.b). Since the two systems are connected the redistribution of forces will in a final deflected shape as shown in fig. (2.2).

So the behaviour of multi-storey building composed of frames and shear walls has took a great deal of interest. Methods of elastic analysis have been presented but scarcely attempt has been made to sudy the behaviour of the system in the inelastic range. A comprehensive



a)-Free Frame b)- Free Wall

Fig. (2.1) Free Deflected Shape of Frame and Shear Wall



Combined Frame-Wall

Fig. (2.2) Final Shape of Combined Shear Wall-Frame System review for the previous researchs carries out on frame shear wall structures either in elastic or inelastic analysis are presented in this chapter.

2.1.2 Elastic Analysis of Frame - Shear Wall Interaction

Since 1959 series of method were proposed for elastic analysis of shear wall-frame interaction in high rise buildings.

2.1.2.1 Concrete shear walls combined with rigid frames in multistorey buildings sujected to lateral loads

Cardan[2] studies the combined frame-shear wall structures under lateral loads for the case shown in fig. (2.3). Each storey is isolated by assuming that the columns has an inflection point at mid-height and developed exressions for lateral stiffness of varoius types of beams and columns arrangements.

The moments and forces developed by the various structural arrangements are then assumed to be continously distributed over the height of the shear wall. A differential equation expressing the equilibrium of the shear wall was formulated assuming the structure to have constant properties through out its whole height. This last assumption makes the method unsuitable for most of the practical cases. This method can easly be for manual calculations for symmetrical structures.

2.1.2.2 Frames combined with shear trusses under lateral loads

In this method the shear wall was replaced by an equivalent truss system as shown in fig. (2.4). Bandel[3] derived an expressions for the total internal and external work of the system by using a power series. The constants of the actual deformation functions are to be determined and the solution of a set of simultaneous equations is considered. The equations result from the minimization of the total potential energy of the system. Straining action can then be obtained by evaluation of the deformation. This method can not be easly used manually by the designer and requires a great effort for solving the equations and gives approximate results.

2.1.2.3 Interaction of shear wall with frames in concrete structures under lateral loads

The model consists of a shear wall and a simplified frame system as shown in fig. (2.5). Khan[4] developed an iterative method in which the building is lumped for a structural model. The method accounts for the variation in wall and frame properties. The effect of base rotation, plastic rotation of wall and secondary deflections in the frame had also been discussed. Influence curves were provided to estimate the distribution of shear between the wall and the frame. These curves make the method more suitable for the use by the designer.

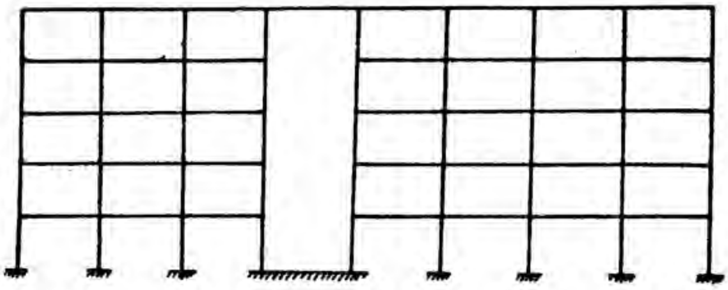


Fig. (2.3) Structure Model by Cardan[2]

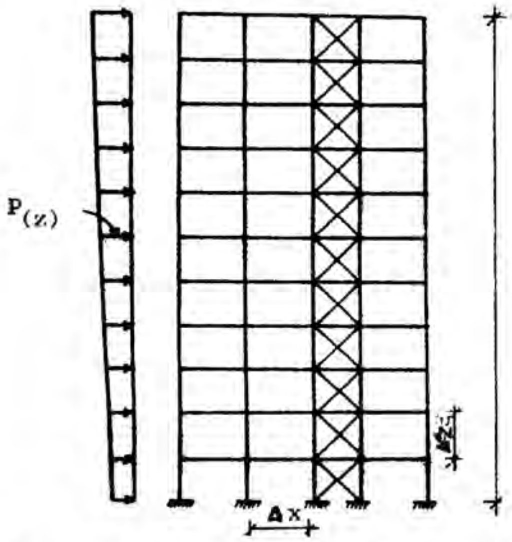


Fig. (2.4) Combined Frame and Truss System

Bandel Model[3]

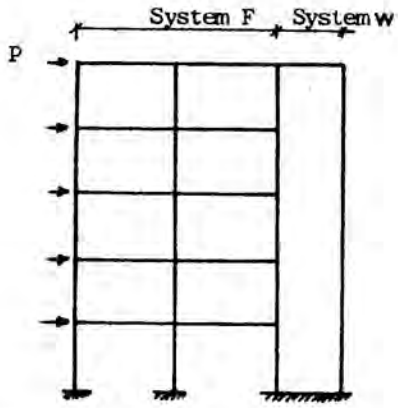


Fig. (2.5) Khan Model[4]

2.1.2.4 Interaction of shear wall frame systems in multistorey buildings

Gould[5] made representation for each storey by a rigid bars connected by extensional and rotational springs as shown in fig. (2.4). The individual storey properties and the effect of adjacent floors are considered in determining the lateral resistance of a particular storey.

The analysis assumes constant storey height and constant shear wall stiffness and neglects the effect of the axial deformations of the columns and shear deformations of the members.

The lateral deformations of the wall are represented by finite difference expressions. This method gives approximate rsults due to approximation adopted in this solution and limitions.

2.1.2.5 Design of combined frames and shear walls

Parme[6] combined the resistance of the shear wall with that of the frame to obtain an expression for the total lateral load on a floor as a function of the stiffness of the members (including the shear wall) and the displacements and rotations of the floor considered and the adjacent floors. The response of the shear wall is formulated in terms of finite difference equations.

Finally, the solution of the resulting set of simultaneous equations determins the lateral displacement of each floor.

The analysis neglects the axial deformation of the columns and the effect of the wall width on the moments in the girders adjacent to the wall.

2.1.2.6 The component stiffness method

Macleod[7] presented the component stiffness method for frame shear wall structure without and with torsion. The main assumption is that the frame takes constant shear, this means that the interaction force between frame and shear wall can be represented by a concentrated force at top.

To use the method for the case when torsional effect is neglected. The basis of this method is that each vertical unit of structure has its stiffness defined only at top, that is only movement at the top of the structure is considered in the solution with only one degree of freedom. The stiffness of a unit is defined as the uniformly distributed load required to produce unit deflection at the top of the frame or for shear wall their moment of inertia, where all units of a structure have similar deflected shape under the same lateral load. The system is treated as wall supported at top by spring as shown in fig. (2.7). But for case of structure

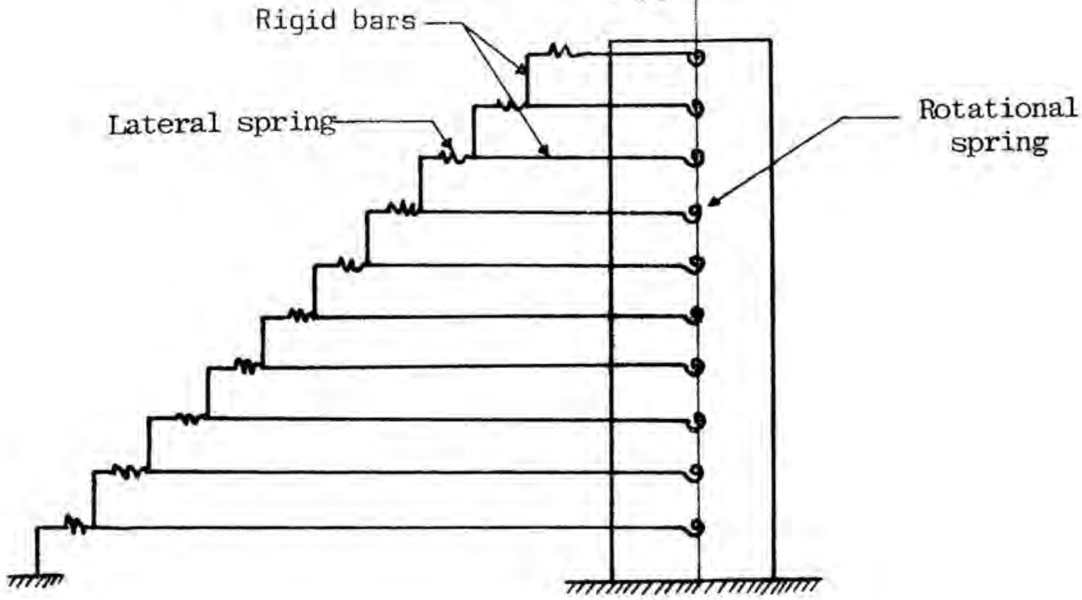


Fig. (2-6) Model by Gould[5]

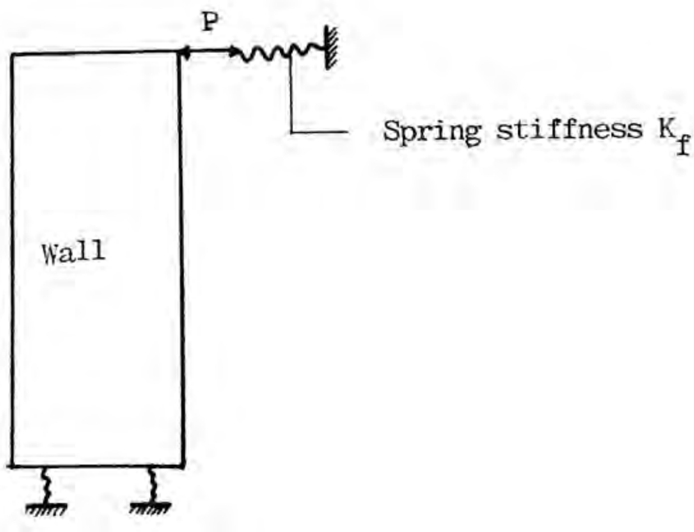


Fig. (2.7) Model by Maclcod[7]

taking into consideration torsional effect. In this case two degrees that correspond to the deformation (Δ , θ) movement and rotation are considered. This method lacks accuracy if the wall is more flexible than the frame but the method can be easly used manually.

2.1.2.7 Simplified analysis of tube-in-tube tall building under lateral load

Bally[8] presented computer analysis of tube-in-tube tall building under lateral load using various models as shown in fig. (2.8). These include the plane and space analysis models in which the effects of flate plate floors-shear walls- frame interaction and soil structure interaction are considered.

In this method two simplified plane models for the analysis of tube-in-tube tall buildings under lateral loads are proposed and varified. In this model , while the inner tube is represented by a one-dimensional cantilever, the portal method is utilized for the analysis representing the outer tube as shown in fig.(2.8.b).

In the second model where soil-structure interaction for buildings on raft foundations is considered, the cantilever tube of the shear walls is assumed supported on a beam on elastic foundation representing the part of foundation considered acting with the shear walls

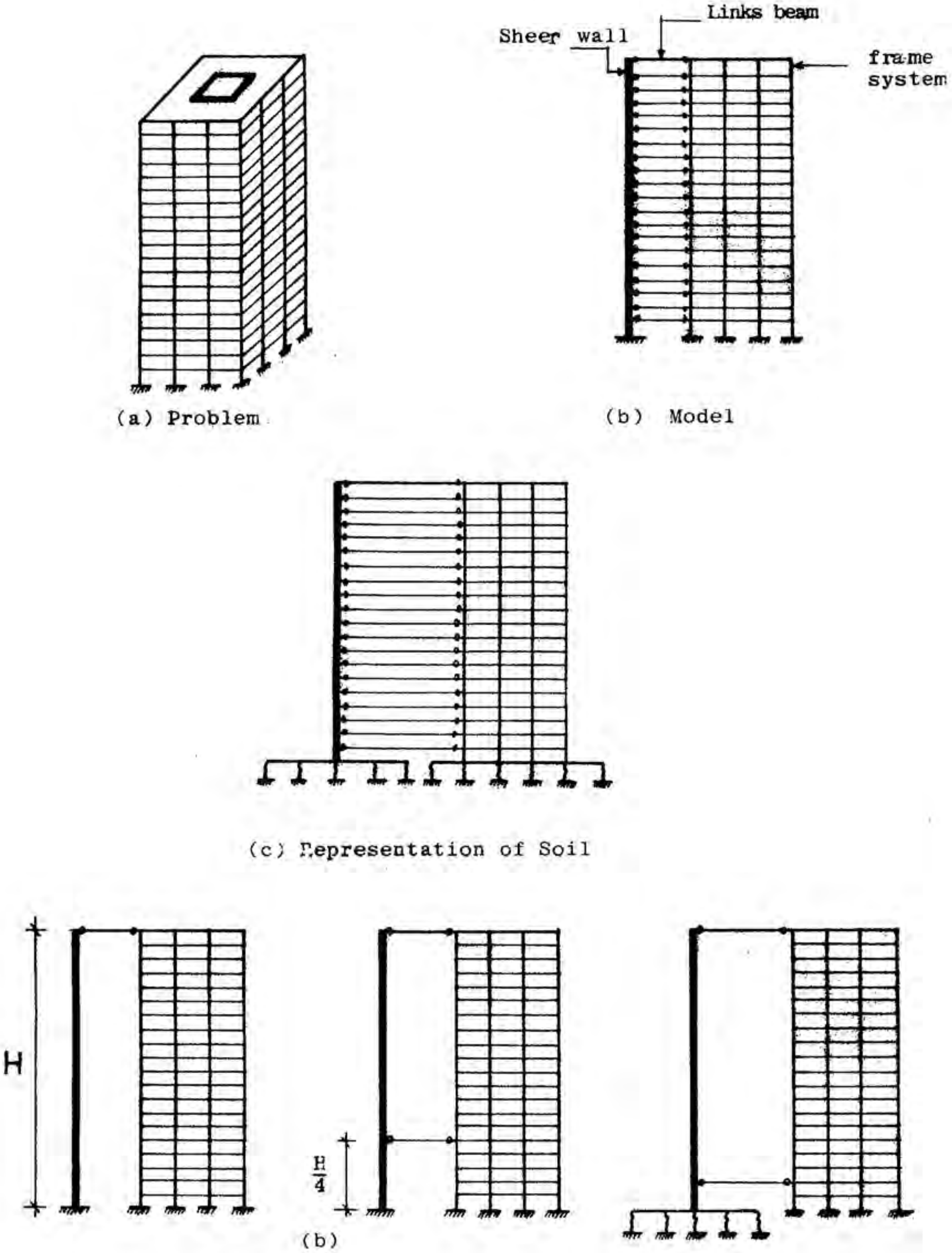


Fig. (2.8) Problem and various Models Used by Sherif Baly[8] together with its subgrade soil as shown in fig. (2.8.c). The interaction between the bending mode of the inner tube and shear mode of the outer tube is conveniently modeled by two rigid links. The first link is positioned at the top storey level fig. (2.8.d). The second link is assumed either at the storey near the quarter of the height in the first model or at the first storey level in the second model where soil-structure interaction is considered to interact as shown in fig.(2.8.d). This method enable the use of the computer programs for analyzing the structure either in plane or space.

2.1.3 Inelastic Analysis of Frame-Shear Wall Interaction

Majumder, Macgregor and Adams[9] developed method for the analysis of frame-shear wall interaction in the inelastic range. In this analysis the actual structure had been replaced by a simplified structural model shown in fig. (2.9).

A conservative method has been used for lumping the actual structure into the model. The model was analyzed taking into account the formation of the plastic hinges in the frame, the inelastic action of the shear wall and the secondary moment caused by load-diplacement ($p - \Delta$) effect.

It is assumed that lateral buckling, lateral torsional buckling and local buckling of the members are

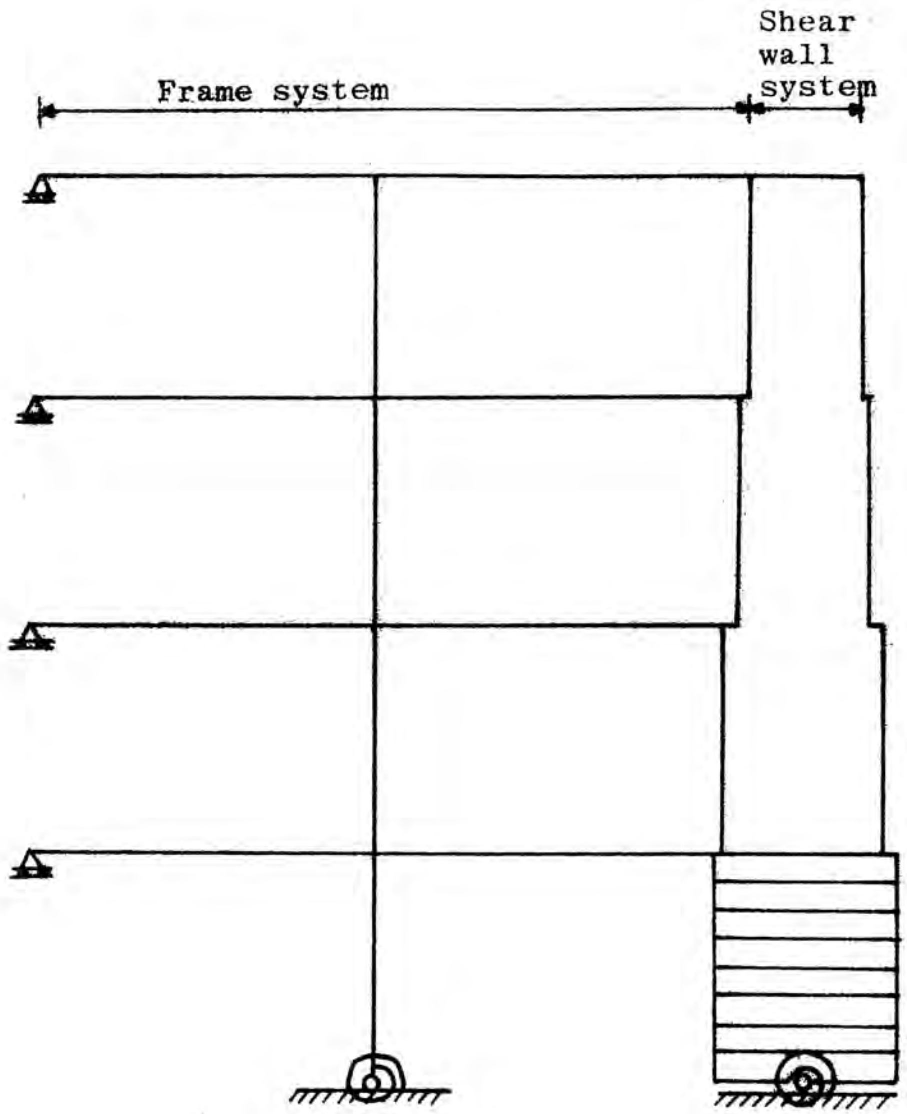


Fig. (2-9) Analytical Model by Adams[9]

prevented. The effect of axial shortening and shear deformation of the members were neglected. This method needs cosniderable computer analysis.

2.2 COUPLED SHEAR WALL

2.2.1 Introduction

A brief review and discussion for the different methods used for the analysis of coupled shear wall subjected to either vertical or lateral loads is presented here in after.

2.2.2 The Continuous Connection Method

Rosman(10) presented simple approximate analysis for many types of shear walls. These analysis deal primarily with the proplem of concentrated lateral load at the top of the wall. Using the continuous connections of individual piers are chosen as the statically redundant functions. Deformations due to bending moment, the contribution of normal forces in the piers and shear forces in the connecting beams are taken into account.

In this method the connecting beams or slabs are replaced by a shear medium connection which is continuous over the full height of the wall as shown in fig. (2.10). This is good assumption for uniform walls with height where the wall sections are stiff compared with the connecting beams. Assuming that the walls will deflect equally and the connecting beams will deflect with a known point of contraflexure. The behaviour of the structure may be expressed as a single second order differential equation which enables a general solution to be obtained.

Coull[11] extended the approximate analysis to take account of shearing deformations of walls and the influence of the flexibility of the wall-beam connection. The results indicate that including the shearing deformations in the analysis of walls has a little effect on the stresses and increases the deflections by a few percent only.

The conventional assumption of a fully rigid joint yields deflections which are on the order of 17% lower than the measured values. For a wall with one row of openings, the use of equivalent length suggested by Michael[12] taking in account of the joint flexibility, yields good agreement between the measured and theoretical deflections, but it is not so good in the case of walls with two rows of openings.

The unconvenients of the method are:-

1- It is assumed that the structure is uniform with height.

2- It is valid only for limited variation in properties of members and in supporting systems.

3- It is unsuitable for non-linear problems.

2.2.3 The Simplified Method

H. Hosny[13] developed a simple method which can be used by the desinger to asses the amount of redistribution of vertical loads in shear wall structures. The analysis is based on the continuous connection approach, simple formula and graphs were derived for laterally restrained walls with one row of openings. A simple criterion was developed for the case of lateral restraint.

A simplified method was developed to reduce shear wall with several rows of openings to that of one row of openings to extend the use of these equations and graphs.

Mr. Hosny concluded that three situations are possible when analysing shear wall system under vertical load:-

1- The foundation are rigid or fairly rigid. In this case the simplified method is good.
2- The superstructure is rigid as compared with the foundation. In such a case the foundation load will be related only to the foundation stiffness.
3- The stiffness of foundation and superstructure are not widely different. In this case the simplified method is less reliable.

On the other hand, the unconvenients of the method is that it deals only for lateral restrained walls and it is especially used to calculate the load at foundation level only.

2.2.4 The Stiffness Method

In this method the walls are to be devided into elements connected only at their ends (nodes). It is assumed that the behaviour of the shear wall is similar to that of these elements. The displacements of any point in an element are assumed to be a function of element nodal displacements which are considered as the unknown parameters. Distributed loads acting on the element boundaries are to be replaced by an equivalent systems of concentrated loads acting on the nodal points. These methods are classified according to the types of elements and to the stiffness matrix formulation.

2.2.4.1 The finite element method

Is a numerical method of analysis that can deal with engineering problems of various boundary and loading conditions. In this method, the structure model consists of a number of finite plane elements connected at discrete set of nodes fig. (2.11.a). The load deflection relationships for each element are expressed in terms of the nodal forces and displacements. The satisfaction of the overall equations of equilibrium and compatiblity at the nodes leads to a set of algebric simultaneous equations.

At each node there can be two or three degree of freedom for plane structure and six degrees of freedom for space structures. Iain. A. Macleod[14] suggested that linear elastic plane stress analysis of shear walls may be carried out by using finite element idealization. When the connecting beams between openings are selender, existing types of elements are not suitable, they can not be connected with line element in bending. He derived a rectangular element which had a rotational degrees of freedom as well as the two translational at each node. Krishnam[15] discussed the results which were obtained by Macleod and suggested an alternative simpler approach for the solution of a problem which requires a rotational degree of freedom at a node. Chiyyaraih[16] illustrate the use of the finite element method in determining stress distribution in shear wall with openings and in predicting more accurate values for the bending moments, axial and shear forces. He presented the effects of changes in material properties such as Poisson's ratio on the stress in shear walls and bending moments and axial forces in the connecting beams.

The advantages of finite element method:-

1- It can be used for complex geometric forms.

2- It is simple in both its physical interpretation and mathematical formulation.

3- Its application is possible due to the availability of computers.

2.2.4.2 The beam method

Smith[17] presented modified method for analyzing symmetrical interconnected shear walls. In this method the walls and beams of the original structure are replaced by line members of the corresponding length and stiffness while the influence of the wall width is reproduced by connecting a stiff arm between the ends of each beam and the center line of the wall as shown in fig. (2.11.b). A stiffness matrix is then formulated for the wide column frame and the deflection and actions solved by standard matrix procedures. The method can take account of walls which vary in thickness up their height, interconnected by beams which may vary in size and spacing from floor to floor, however, it is restricted in use to the analysis of cross walls which at all floors are symmetrical about their verical central section. The method can also take into account the effect of shear in the beams.

2.2.4.3 The frame method

I.A. Macleod[18] and H. Hosny[13] presented a plane frame analysis based on a computer program with some nonstandard elements and is applicable to wide range of problems fig.(2.11.c). Their applications of this technique showed an acceptable correlation with the published results probiding the following advantages:-

1- It can deal with any structural configuration allowing any variation in the type of foundation and in the pattern of loading.
2- The results of displacement and actions are given for each individual member.
3- Shear deformation for all elements can be easily considered.
4- It can be used to account for non-linear behaviour.
5- Any type of connection between the wall and floor beams can be considered.
6- It is considered to give an exact solution, the degree of accuracy can be decided by the designer.
7- The versitality of the stiffness method makes it paricularly useful for analysing unsymmetrical structures.

2.2.5 The Iterative Method

Khan[20] presented an iterative method for calculating the shear forces in the connecting beams when the walls are unequally stressed. If two segments of a shear wall are assumed to be connected by a connected beam and are loaded at each floor in a way that the average stress in two segments is different, then it is expected that some load from segment with heigher stress will be transferred to the one with lower stress.

The amount of load redistributed will be a function of the area of the two segments as well as the moment of inertia and the area of the connecting beam (the shear transfer capacity). He suggested the use of the method of forced convergence iteration (21),(22) to obtain a solution for loaded redistribution.

The main disadvantage of the iterative method is that the boundary condition at the base of the walls is considered as if they were connected to a rigid foundation. However, this is not always the case seince piled foundations are widely used in practice which are considered as elastic supports.

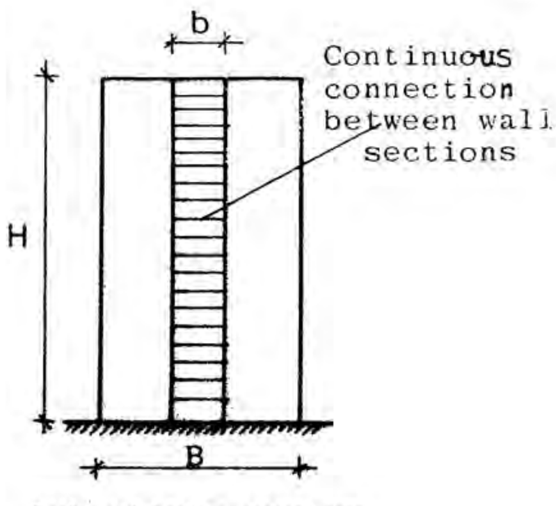
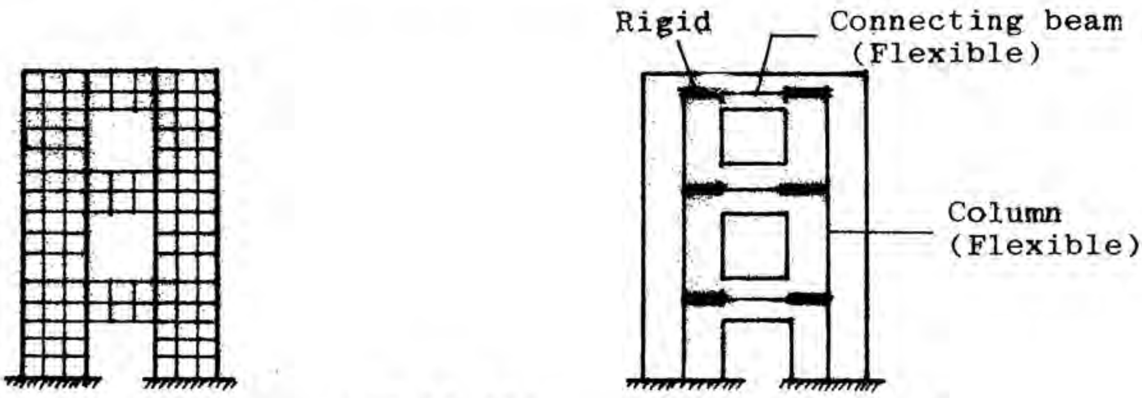
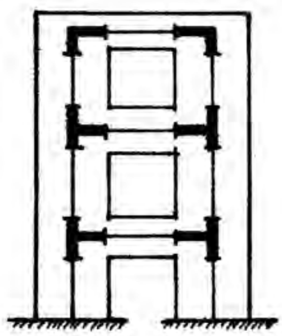


Fig. (2.10) The Continous Connection Method[10]



a- Finite element model[14,15,16]   b- Beam Model[17]



c- Fram Model[13,18]

Fig. (2.11) The Stiffness Method.

2.3 INFILLED FRAMES

2.3.1 Introduction

Since 1949 different tests were perfomed, and several studies have been published on infilled frames using steel frame with concrete infill or of reinforecd concrete, frames with plain concrete or brick infill. The pioneers in this field were Jack R. Benjamin and Harry A. Williams(23,32). They published in 1954 the results of their investigations on the strength and behaviour of infilled shear walls. After that so many researches were carried out to analyse the infilled frames either theoretically and or experimentally(26,28,34,37,39,40,41,42).

2.3.2 Types of Infilled Frames

The resistance of structure to lateral loads depends on the shape of structure, the type of construction, the type of filling materials and the strength of the components(25).

The structure shown in fig.(2.12.b) is similar to that in fig.(2.12.a) except that bracing is providing for full height of one bay. In this case the braced bay acts as a vertical girder to resiste all lateral loads. Barcing may not appear as actual member but may be replaced by infill panels of any material such as sand lime brick, light weight brick, red brick, loam perforted brick and hollow cement brick. Shear connectors somtimes are provided to insure the intergerity of the interaction between infill and frames. The infill may or may not contain openings (window or door) as shown in fig.(2.13).

2.3.3 Representation of Filling Material

The infill can be represented in analysis by one of the two following methods:-

1- Equivalent Strut Method

2- Finite Element Method

2.3.3.1 Equivalent strut method

In this method the filling material is replaced by an equivalent diagonal strut as shown in fig.(2.14). This method has been used in this research work.

Factors affecting the behaviour of infilled frames

The behaviour of infilled frame is affected by the wfollowing factors:-

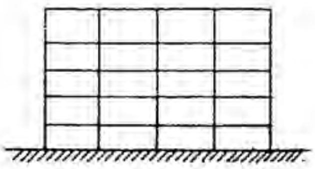
1- Frame infill interaction.

2- Contact length.

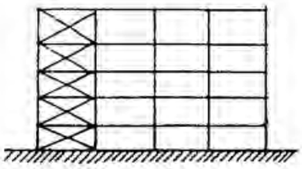
3- Effective width of the equivalent diagonal strut.

4- Relative stiffness of infilled frame.

5- Lateral stiffness of infilled frame.

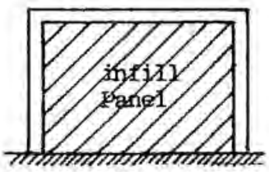


(d) Unbraced Frame

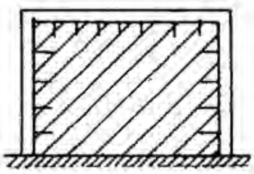


(b) Braced Frame

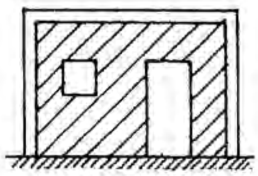
Fig. (2.12) Framed Structures



(a) Infilled Frame



(b) Infilled Frame with Shear Connectors



(c) Infilled Frame with Openings

Fig. (2.13) Different Types of infilled Frames

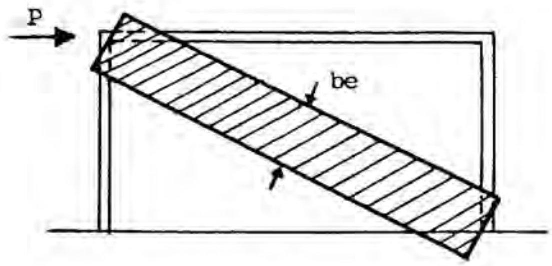


Fig. (2.14) Equivalent Diagonal Strut(26)

1- Frame infill interaction[26]

When an infilled frame is subjected to in-plane loading, the frame members and the infill interact to develop a combined resistance to the load. The members deforming mainly by bending and axial stresses and the infill by in-plane shear and direct stresses. When the load is applied on the infilled frame, the frame transfers the major part of the load to the infill and distribute it to a region arround the position of the load. The consequent interaction and deflection of the infill and the frame caused them mutual contact only in regions around the ends of compressive paths through the infill cracks will develop between frame and infill except in the vincinity of two of the corners where the infill panel will lock into the frame and there will be transmission of compressive force into the infill. At this stage, it is convenient to consider that the infill is acting as a compression diagonal strut within the frame as shown in fig. (2.14). Interaction between frame and infill varies according to a variety of factors including:-

a- Types of filling materials

b- Strength and properties of the frame and the infill

c- Geometry and dimension of the frame

d- Type of loading (static or dynamic)

2. Contact length

Contact length is the length which there is no separation between frame and infill, therefore the infill is represented by an equivalent strut acting along the compressive path of the infill. The length of contact (α) depends on the relative stiffness of the members in flexure and the infill in compression and on the geometry of the panel. B. S. Smith [27] developed the contact length by the analogy with a beam on elastic foundation fig. (2.15). The general solution for the beam on elastic foundation is given by the form;

$$y= e^{\lambda x} (A \cos \lambda x + B \sin \lambda x) + e^{-\lambda x}(C \cos \lambda x + D \sin \lambda x) \qquad \text{............(2.0)}$$

in which

$$\lambda = \sqrt[4]{(K^{`}/EI)} \qquad \text{...........(2.1)}$$

where

K` is the foundation modulus

$$\lambda_n = \sqrt[4]{Ei\ t_{panel} \sin \frac{2\theta}{4EI_1 h}} \qquad \text{............(2.2)}$$

therefore:

$\alpha_h = (\frac{\pi}{2}) \lambda_h$ is the contact length between column and infill

t_{panel} = wall thickness

I^1 = 2nd moment of area of the column

h = height of wall

$\theta = \tan^{-1} \frac{h}{L}$

E_i = elastic modulus of the wall

E = elastic modulus of the frame

Similary for beam member

$$\lambda_b = \sqrt[4]{E_i \, t_{panel} \sin \frac{2\theta}{4EI_2L}} \qquad \ldots\ldots\ldots\ldots(2.3)$$

therefore:

$\alpha_b = \frac{\pi}{2} \lambda_b$ is the contact length between beam and infill

I_2 = 2nd moment of area of the beam

3. Effective width of the equivalent diagonal strut (b_e)

The effective width of the equivalent diagonal strut is the width of an equally stiff uniform strut whose length is equal to the diagonal of the panel and whose thickness is the same as the panel(28). The effective width depends on the rectangularity ratio of the panel L/h where:

L = Length of the panel

h = Height of the panel

B. S. Smith [28] plotted the effective width (b_e) as a proportion of diagonal length for varying side ratios as shown in fig.(2.16.a). The effective width of diagonal strut (b_e) can be calculated as shown in fig.(2.16.b).

The effective width "b_e" can be taken as follows:

$$b_e = \frac{1}{2}\sqrt{\alpha_b^2 + \alpha_h^2} \qquad \text{..........(2.4)}$$

$$= \frac{\pi}{2}\sqrt{\left(\frac{1}{\lambda_b}\right)^2 + \left(\frac{1}{\lambda_h}\right)^2}$$

where

α_h = contact length of the column

α_b = contact length of the beam

In 1984 Liauw and Khan [29] suggested the following simple formula for the estimation of the effective width "b_e"

$$b_e = 0.45\ h \cos\theta \qquad \text{...........(2.5)}$$

where

h = storey height

$\theta = \tan^{-1}\left(\frac{h}{L}\right)$

The effective width for various panel proportions computed from the above equation is compared with that given by B.S. Smith in table (2.1). The results are in close agreement, however the formula suggested by Liauw and Khan is more simple and for that reason it was used in this research.

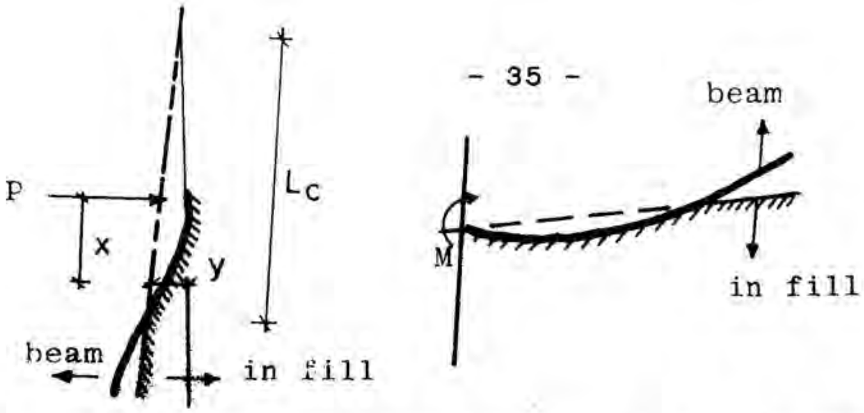


Fig. (2.15) Beam on Elastic Foundation Analogy for Contact Length Between Infill and Frame(27)

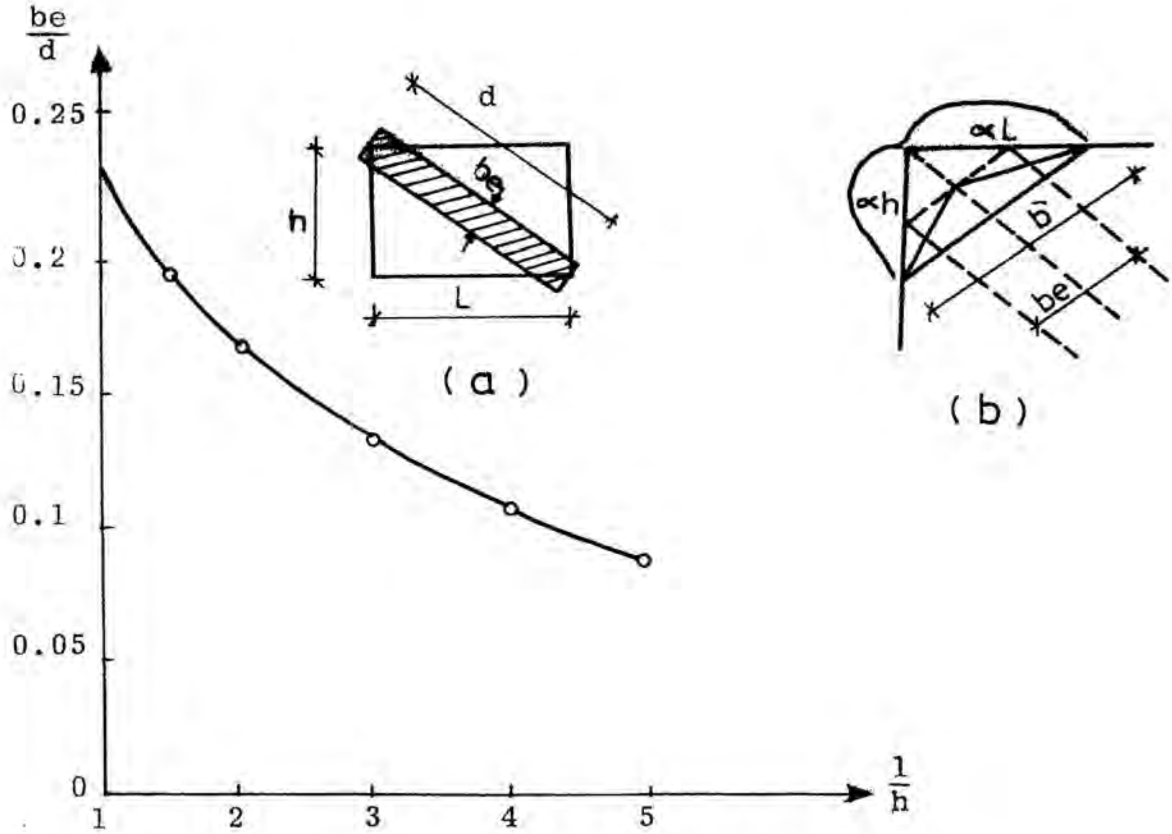


Fig. (2.16) Effective width of Equivalent Diagonal Strut(28)

span/height ratio (L/h)		1	3	5
$\frac{b_e}{d}$	Smith	0.233	0.13	0.086
	Liauw & Khan	0.225	0.135	0.086

Table (2.1) Effective Width of Equivalent Strut "b_e".

4- Relative stiffness of infilled frame

B. S. Smith [26] represented the relative stiffness of infilled frame by the non-dimensional parameter λL.

where

$$\lambda = \sqrt[4]{E_i \, t_{panel} \sin \frac{2\theta}{4EIL}}$$

λL expresses the relative stiffness of a foundation to an interacting beam theses correspond to the infill and frame respectively. Fig. (2.17) represents the relation between the length of contact and relative stiffness of infilled frame where:

$$\frac{\alpha}{L} = \frac{\pi}{2\lambda L} \qquad \text{.............(2.6)}$$

From curve one may conclude that :-

For small value of λL where frame is stiffer relative to the infill, the length of contact is large.

For large value of λL where frame is flexible, the length of contact is small, e.g. the length of contact increases with increasing frame stiffness (small value of λL).

5- Lateral stiffness of infilled frames

The lateral stiffness of infilled frame is obtained by assuming a diagonal strut to replace the infill in which effective width "b_e" is derived previously explained.

R.H. Wood[30] carried out lateral loading tests on steel frames with concrete and hollow clay block infill. The tests showed that the failure in the wall panel took the form of a crack. roughly, diagonal across the panel.

B.S. Smith[28] calculated lateral stiffness of infilled frame theoretically and experimentally. He studied the infilled frames under three different conditions that is diagonally loaded rectangular panels. Laterally loaded single-storey and laterally multi-storey infilled frame.

5.i Rectangular panels diagonally loaded

B.S Smith [28] calculated lateral stiffness theoretically and experimentally and compared the strains along the loaded diagonal obtained from theoretical and experimental work as shown in fig. (2.18).

5.ii Single storey infilled frames laterally loaded

The stiffness of infilled frame depends on whether the surrounding frame has rigid or non rigid joints.

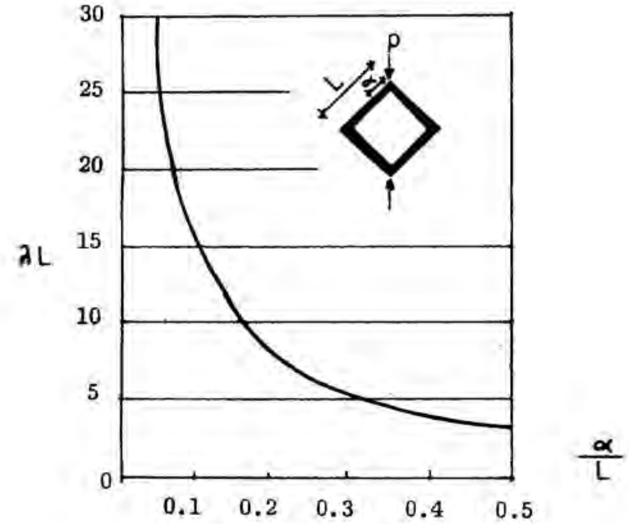


Fig. (2.17) Relation Between the Contact Length and Relative Stiffness of Infilled Frame(26)

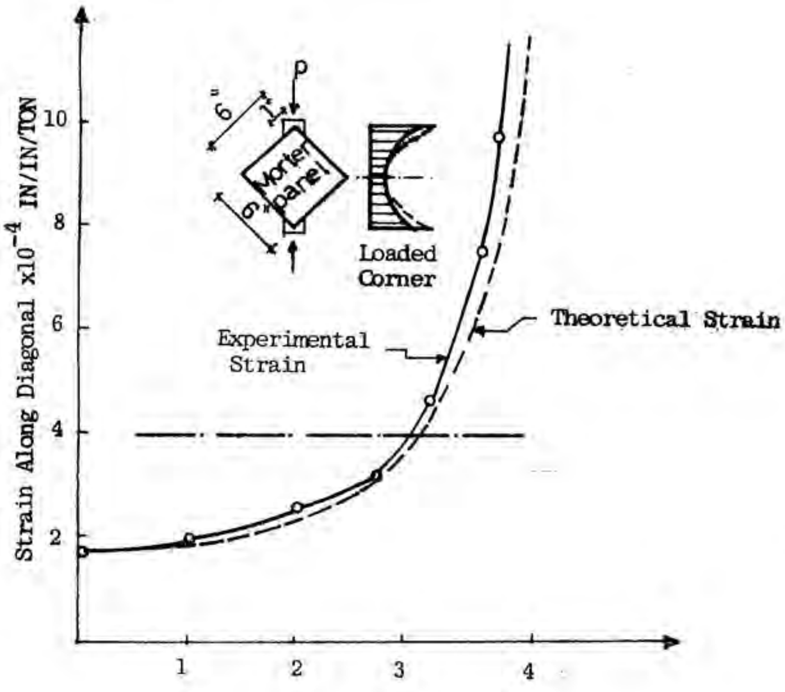


Fig. (2.18) Comparison of Theoretical and Experimental Strain Along Loaded Diagonal of Square Panel(28)

Case of a rigid frame[28]

For the equivalent structure shown in fig.(2.19), lateral stiffness at top of column

$$= \frac{A + B + C}{C\,(A+B)} \qquad \text{..........(2.7)}$$

where

$$A = \frac{h \tan^2\theta}{a_s\, E_s}$$

$$B = \frac{d}{b\, t_{panel}\, E \cos^2\theta}$$

$$C = \frac{h^3\,(3I_1 h + 2I_2 L)}{12\, E_s I_2\,(6I_1 h + I_2 L)}$$

where

$$\theta = \tan^{-1}\frac{h}{L}$$

a_s = cross sectional area of wind ward column

E = elastic modulus of concrete

E_s = elastic modulus of steel

I_1 = 2nd moment of area of beam

I_2 = 2nd moment of area of column

h = height of column from foundation to center line of beam

L = length of beam

Case of non rigid frame

Lateral stiffness at top of column

$$= \frac{1}{\Sigma F^{`2} L^{`}/aE} \quad \ldots\ldots\ldots\ldots\ldots(2.8)$$

where

F` = the force in member due to unit load acting horizontally at top of column

L` = length of member

a = cross sectional area of member

The lateral stiffness for laterally loaded single storey infilled frame was calculated experimentally[28]. Model tests were perfomed on three infilled frames of proportions L/h = 1, 1.5, 2, load - defelection curves are shown in fig. (2.20) for comparison between theoretical and experimental stiffness.

B.S. Smith[28] found that discrepancies of as much as 16% from the theoretical prediction occured. Higher stresses caused a reduction in the elastic modulus. This caused lower panel stiffness than predicted and initial lack of full contact between the infill and frame.

S. Darwish[31] tested five reinforced concrete verandeel frames, two of these frames were infilled with bricks, one with red bricks and the other with sand lime bricks, as shown in fig. (2.21). The result showed that carrying capacity of infilled frames varied from 2 - 2.3

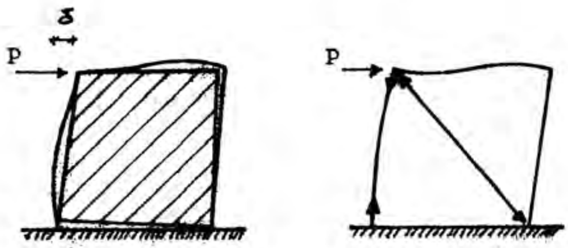


Fig.(2.19) Equivalent Structure For Infilled Frame(28)

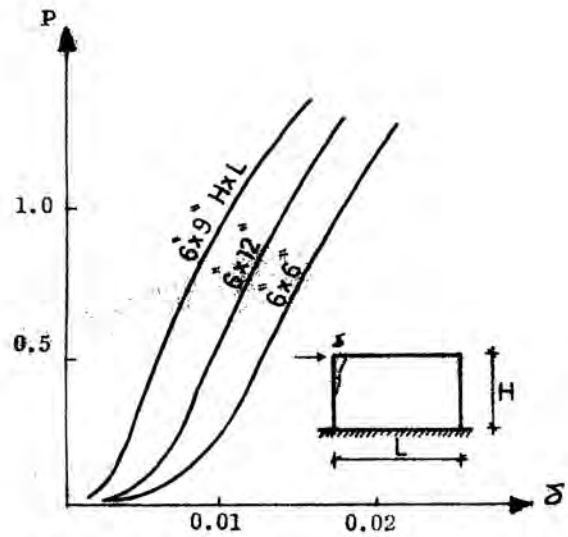


Fig. (2.20) Relation Between Lateral Deflection (δ) and Lateral Load (P)(28)

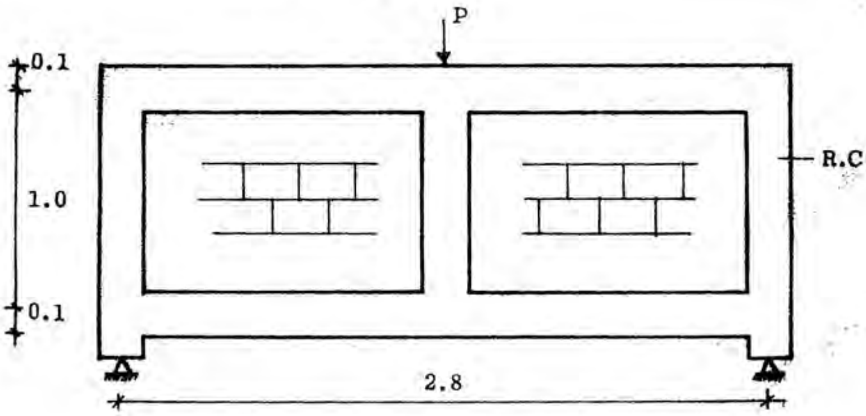


Fig. (2.21) Model Tested by Darwish(31)

times that of the bare frame. The behaviour at the early stages of loading is beam action (90 -95%) plus (10 - 5%) frame action. At higher stages of loading cracks started in the brick walls and the behaviour is 90% truss action plus 10% frame action.

B.S. Smith[26] dealing with another application of the infilled frames proved that the behaviour of a pair beams, one above the other and infilled by a wall can be represented by the equivalent structure as shown in fig.(2.22).

Benjamin & Williams[32] completed their research in this field on large size and scale model brick walls which were tested without bounding frames and with reinforced concrete or structural steel frame. The results showed that the brick wall can effectivelly resist shearing force resulting from lateral load and its resistance depends on the way in which the wall are tied into the structural elements. A typical load-deflection curve was given for a particular type of brick and one type of morter as shown in fig. (2.23).

Further tests were made using cut bricks. Fianlly they suggested a formula predicting the deflection in the panel.

$$\delta = \frac{1.2\ h\ P}{I\ G\ t_{panel}}$$

where

δ = wall deflection

t_{panel} = panel thickness

P = wall shear

h = clear panel height

L = clear panel length

G = shearing modulus of brick composite

5.iii Multi-storey infilled frames laterally loaded[28]

Lateral stiffness of multi-storey frames is calculated theoretically by replacing each infill by an equivalent diagonal strut. In multi-storey frame, it is not necessary to provide rigid beam to column connection if the infill is intended to carry lateral loading. Hence for non rigid connections

$$\text{Lateral stiffness at any floor} = \frac{1}{\Sigma F\, K' L/a\, E} \quad \ldots.(2.10)$$

where

F = total force in the member due to lateral force system on structure, proportional to unit load being applied at the floor at which stiffness is required

K' = the force in the member due to unit lateral force applied only at floor at which stiffness is required

If the connections in the frame are rigid, then this formula still gives a reasonable approximation. The

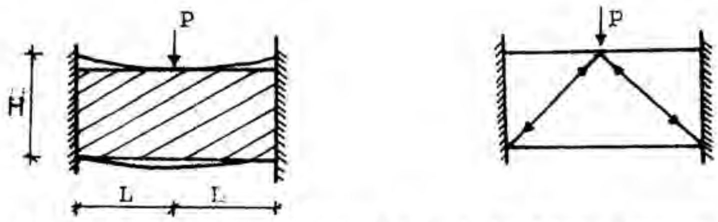


Fig. (2.22) Vertically Loaded in Filled Beams Equivalent Structure(26)

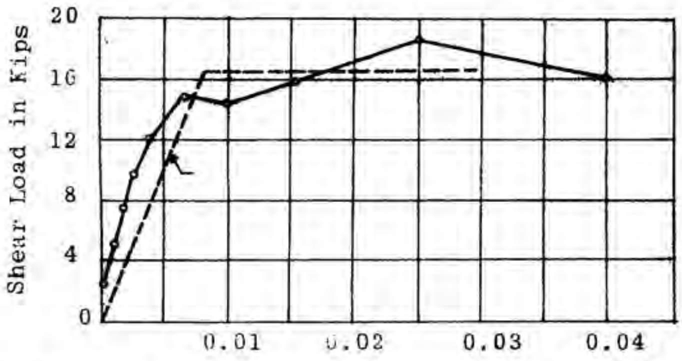


Fig. (2.23) Typical Load Deflection Curve (32)

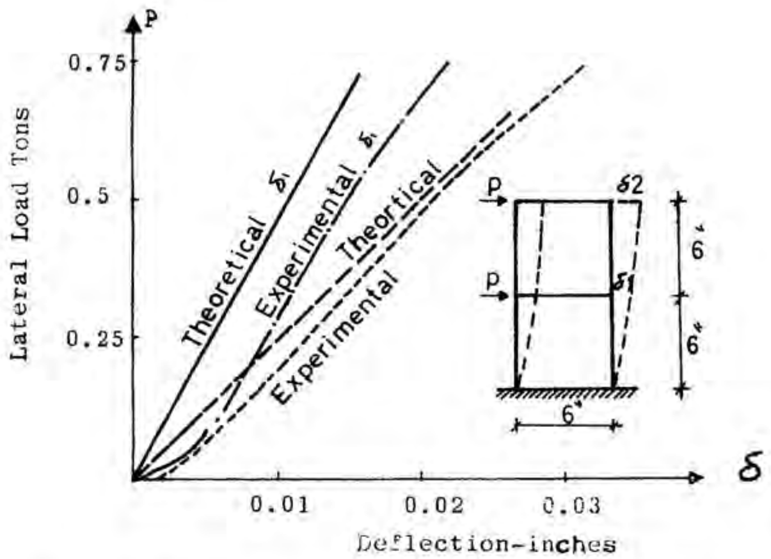


Fig. (2.24) Relation Between Lateral Deflection and Lateral Load for Two-Story Infilled Frame(28) verification of these deductions was made by tests, which were perfomed on a two-storey model infilled frame using "back to back technique" to provide a rigid base. This necessitated making a four-panel frame and loading it . Fig.(2.24) shows the load-deflection diagrams for the experimental and theoretical results.

Holmes[33] studied the case of steel frame with concrete infilling subjected to a horizontal shear force as shown in fig.(2.25). At failure the wall and frame will only be in contact in the vicinity of B & D as shown in fig. (2.26). This figure shows the equivalent structure where the wall panel has been replaced by an equivalent strut BD. The shear force carried by the steel frame alone are shown in fig. (2.27). The change in length of diagonal BD of steel frame due to the forces shown in fig. (2.27) may be calculated as":-

$$\sigma_{BD} = \frac{(P - N_i \cos\theta)\, H^3}{24\ E\ I} \quad (1 + \frac{I_2}{I_1} \cos\theta)\cos\theta \qquad \ldots\ldots\ldots(2.11)$$

$$\sigma_{BD} = e'_c \ d \qquad \ldots\ldots\ldots(2.12)$$

where

e'_c = strain in concrete at failure and vary from 0.0002 to 0.005

$N_i = A\ F_c$

A = cross-sectional area of diagonal strut

f_c = crushing strength of concrete

Horizontal load to cause failure is obtained from equations (2.11) & (2.12).

$$P_u = \frac{24\ E\ I_2\ e`_c d}{H^3\ (1 + I_2/I_1\ \cos\theta)\ \cos\theta} + A\ f_c\ \cos\theta \quad ......(2.13)$$

At failure, horizontal deflection is given by

$$\delta = \delta_{BD}\ \cos\theta \quad ....(2.14)$$

For brick work infilling M. Holmes found that equations (2.13) & (2.14) may be used with the value of $e`_c = 0.005$ as weak concrete infilling and the compressive strength of brick work (f_c) is largerly a function of the properties of the moter joints. A value of $f_c = 450$ Ib/sq. in. is suggested based on the available test results.

2.3.3.2 Infilled frame with shear connectors

Shear connectors can be used in infilled frame as shown in fig. (2.28). The provision of shear connectors increases the length of contact and hardly any boundary cracks appear to the application of aproximately two-thirds of the ultimate load. The distribution of stress is changed, thus increasing the lateral stiffness of infilled frames[34]. Infilled frames with shear connectors could be as much as 67% stiffer than the

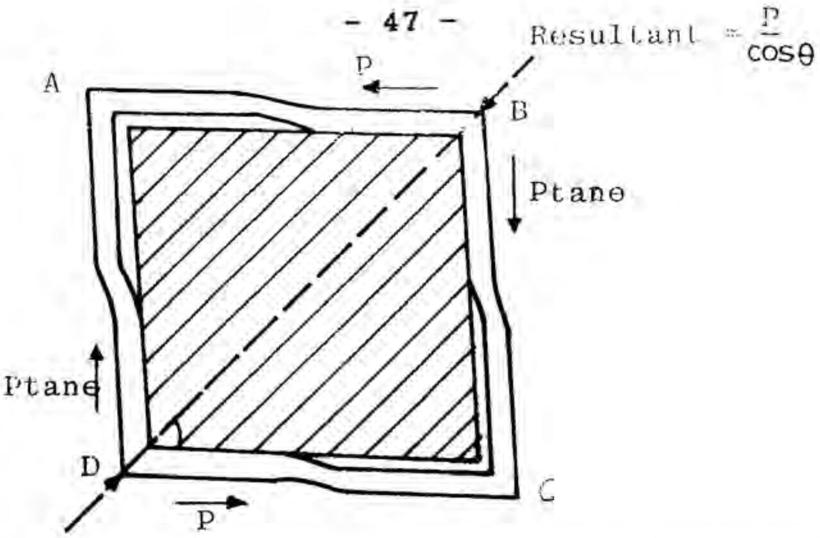


Fig. (2.25) Steel Frame with Concrete Wall in fill(33)

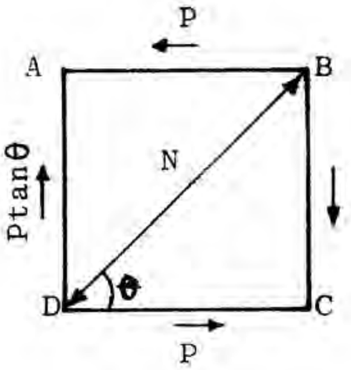


Fig. (2.26) Equivalent Structure For Steel Infilled Frame(33)

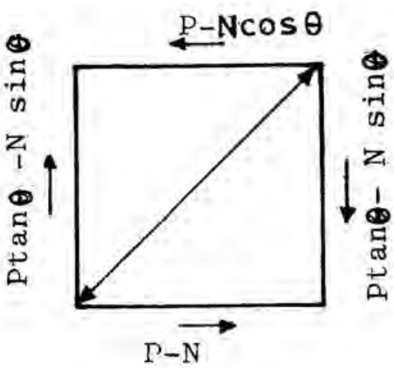


Fig. (2.27) Shear Forces on Steel Frame Only(33)

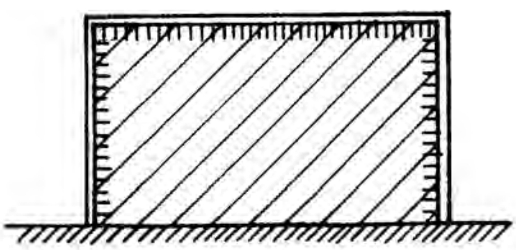


Fig. (2.28) Infilled Frame with Shear Connectors(34,35) infilled framed without shear connectors. Infilled frames with shear connectors are preferable to the infilled frames without connectors for the following reasons:-

1- Infilled frames with shear connectors are stiffer than the infilled without shear connector.

2- Infilled frames with shear connectors failed ultimately due to the crushing of one of the loaded conrners subsequent to the appearance of a diagonal crack at a load less than the ultimate load. This type of frame gives sufficient warning before failure and is desirable from the design point of view.

3- The presence of connectors will probably reduce the risk of lack of fit which is also responsible for decrease in the initial stiffness of the infilled frame.

Fig.(2.29) shows the load deflection curve for square infilled frames with and without shear connectors(35). Fig. (2.30) shows the test results of two infilled frames, one with shear connectors spacing 2" and the other spacing 4". It can be seen from the given figures that the spacing of shear conncetors do not affect the behaviour of infilled frames appreciably(35).

2.3.3.3 Infilled frames with openings

Infilled panels contain door or window openings which will reduce their effectiveness in stiffening the surrounding frame to an extent depending on their size. Experiments by a number of investigators have indicated that centrally located openings may reduce the stiffness of an infilled frame by as much as 50% and 70% respectively[27].

Kadir[27] suggested an approximate method for analysis infilled panels with openings. Experimentally, Kadir found that the load at first cracking was reduced by approximately (50 - 80%) and the ultimate load by (0 - 40%) as compared to a corresponding frame without openings. These results are for various size centrally placed openings in a square infilled panel.

Holmes[33] carried out experimental tests on infilled frame specimens with openings provide in the brickwork filling. These openings resulted in reduction in ultimate load of (30 - 40%).

Coul tested a few infilled frames having central openings with and without reinforcement around the openings. He observed that these openings reduce the stiffness and strength of infilled frames by about (60-70%) and by 45% respectively as compared with a solid infilled panel.

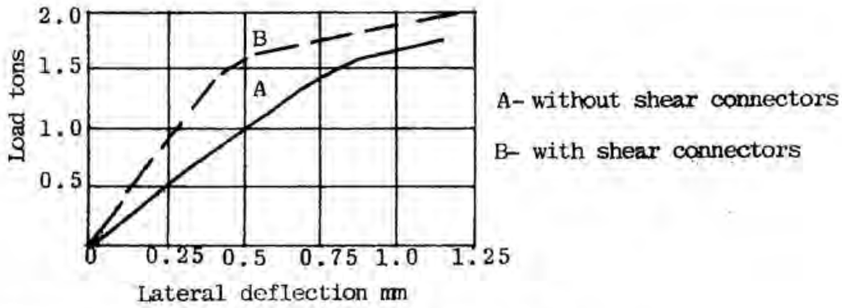


Fig. (2.29) Load-Deflection curves of square infilled frame with and without shear connectors (35)

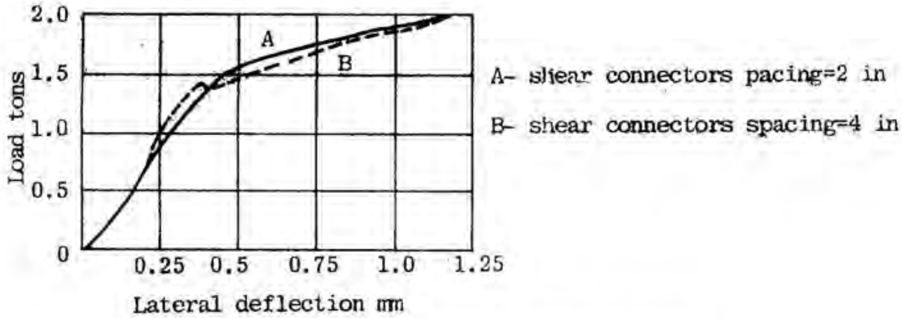


Fig. (2.30) Load-Deflection curves of square infilled frame with shear connectors at different spacing (35)

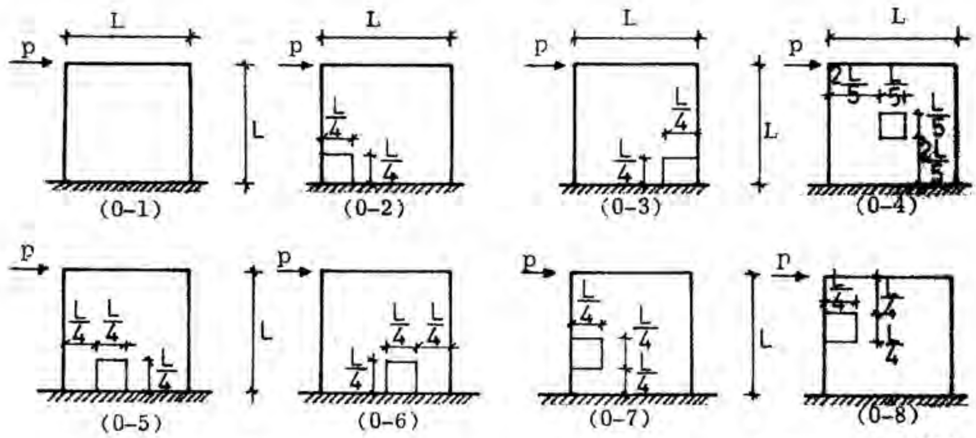


Fig. (2.31) Size, shape and position of openings in model tests (35)

D.V. Mallick and R.P. Grag[35] carried out experimental and theoretical investigations on two types of infilled frames with and without shear connectors. They studied the effect of square openings with sides one-quarter of the side dimension one-fifth the panel length were investigated. Fig. (2.31) shows the size, shape and position of openings studied by D.V. Mallick and R.P. Garg.

Fig. (2.32) shows the load deflection curves together with the failure patterns of square infilled frames with and without shear conenctors for different position of openenings. It is observed that:-
The presence of an opening on either end of the loaded diagonal of an infilled frame without shear connectors reduces its stiffness by about (85 -90%) and lateral strength reduces by about 75% as compared to that of similar infilled frame without openings.

In case of shear connectors openings reduces its stiffness by about (60 - 70%) as compared to that of similar frames without opening. For all these types of frames, the loss of strength and stiffness due to a centrally loaded square opening having side dimensions one-fifth those of the panel is about (25 -50%) as compared to that of similar frames without opening. Hence the openings at either end of the loaded diagonal are structurally unstable.

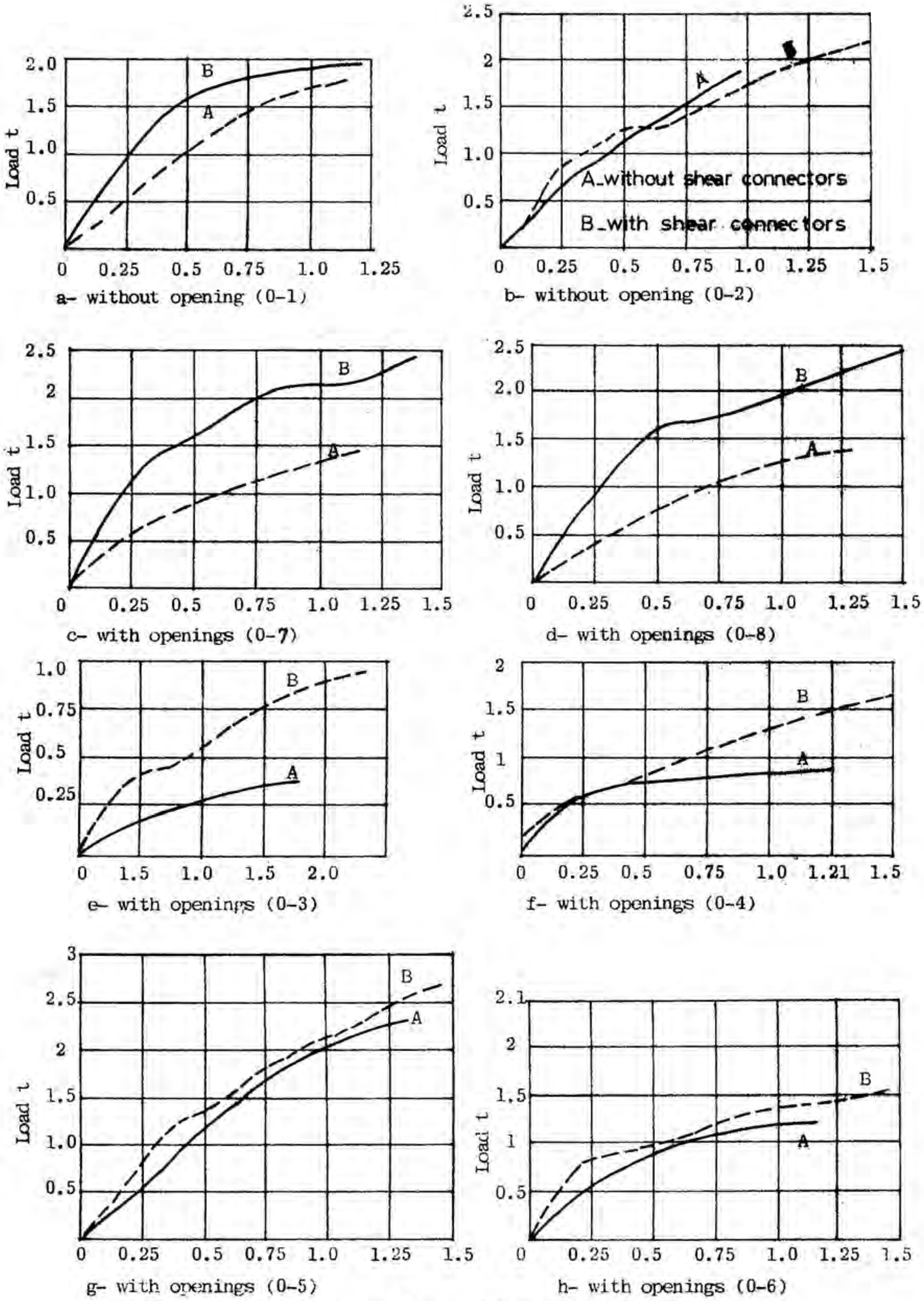


Fig. (2.32) Load-Deflection Curve of Square Infilled Frame

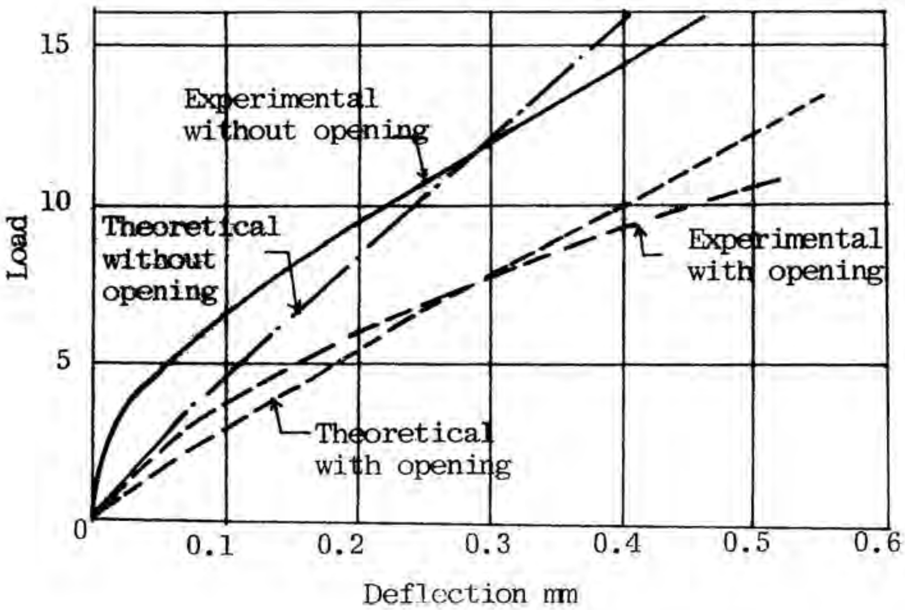


Fig. (2.33) Load-Deflection curve for Theoretical and Experimental Results of Three-bay Two Story Frame with and without opening(24)

G.J.W. King and P.C. Pandey[24] have carried out experimental and theoretical investigations for a number of steel frames with Kaffir D infill. The frames were 12.7 x 12.7 mm. in cross - section and the panels 150 x 150 x 12.7 mm. The results are shown in fig.(2.33).

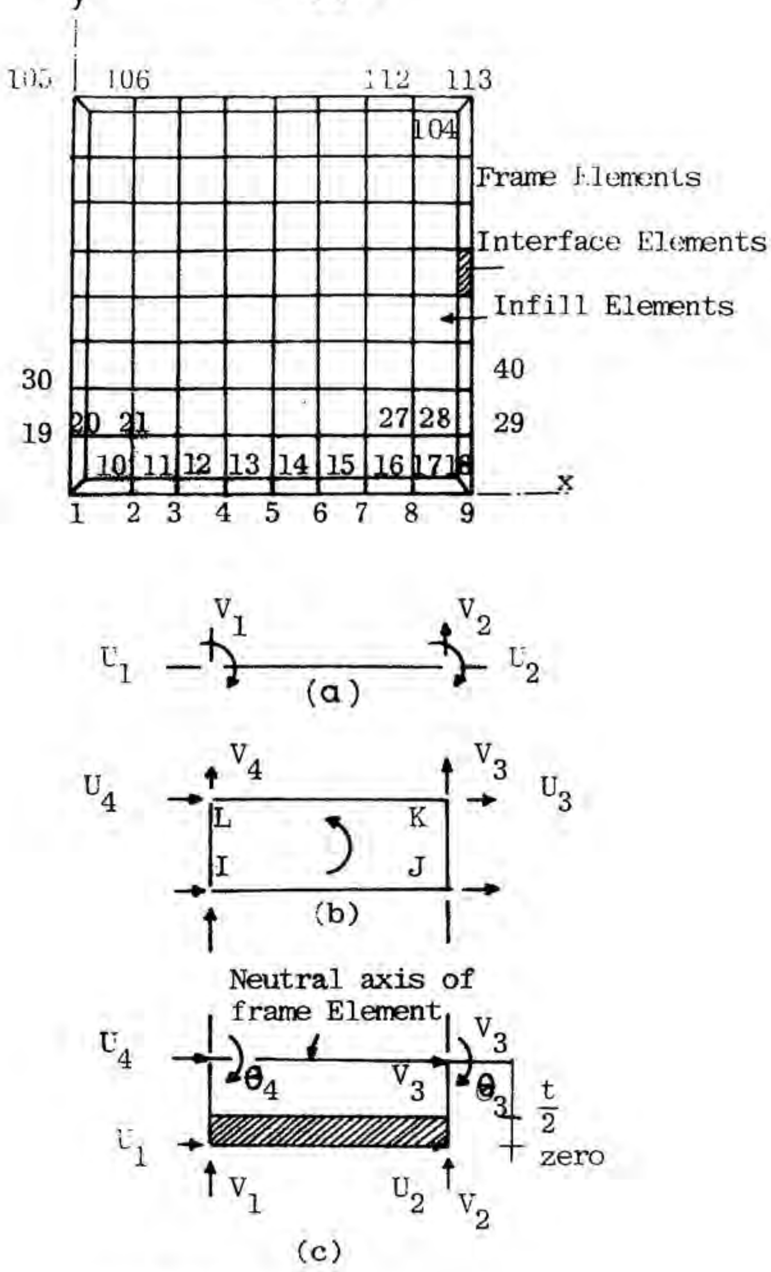
2.3.3.4 Finite Element method

Mallick & Severn[36] represented the infill by plane stress rectangular elements and the frame by axially rigid elements.

Mallick & Garg[35] considered axial deformation in wind ward elements. However, it was assumed that only interaction forces between frames and infill were normal and that the elements of the beam and lee ward column were axially rigid.

Riddington & Smith[37] used the basic four-node rectangular element with two degrees of freedom per node and lineary varying displacement functions along the boundaries.

King & Pandey[24] represented frame members by prismatic bending elements having three degrees of freedom at each node. The infill is idealized as basic four noded rectangular plane stress elements having two degrees of freedom at each node as shown in fig. (2.34).



(a) FRAME ELEMENT

(b) INFILL ELEMENT

(c) INTERFACE ELEMENT

Fig. (2.34) Typical Finite Element Idealization of an Infilled Frame (24)

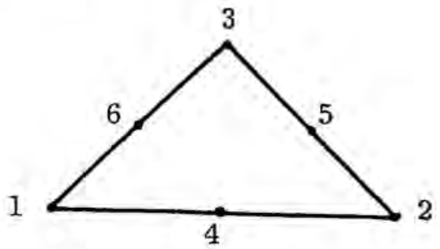


Fig. (2.35) Triangular Element with six nodes by M.H. El-Hozain (38)

The interface between frame and infill is represented by the modified friction element. These elements have three degrees of freedom at the nodes which are connected to the frame elements and two degrees of freedom at the nodes which are connected to the infill elements as shown in fig. (2.34).

M.H. El Hozain[38] represented the infill by plane stress triangular element with six nodes which have two degrees of freedom at each node as shown in fig. (2.35).

2.3.4 Modes of Failure

The survey of the results of previous research work give the following summary for the possible modes of failure.

The collapse of infilled frames is due to either the failure of the frame or the failure in the infill. However, the most common modes of failure are:

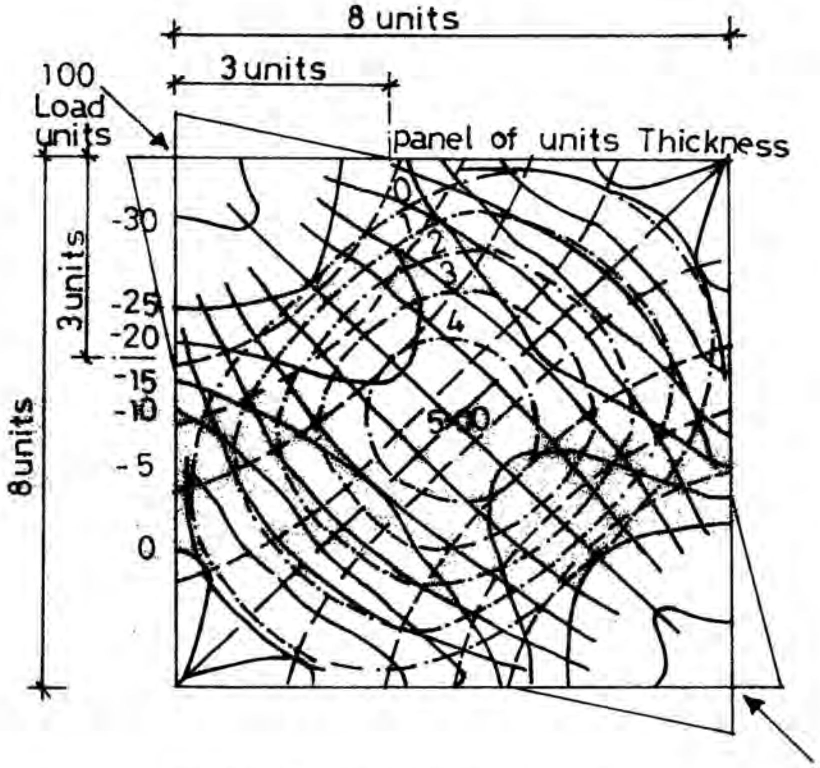
2.3.4.1 Tensile failure[34]

In this case cracks extend from the center of the infill along the diagonal towards the loaded corners. This type of failure is the first sign of failure in all modes except those with the most flexible frames. After cracking load, it was usually possible to increase the diagonal load until the infill collapsed by a compressive failure. This type of failure occurs by principal tensile stress which have maximum valu at the center of the infill panel. If the maximum principal tensile stress reaches the tensile failure stress of the infill materail, the failure occurs.

2.3.4.2 Compressive failure(26)

In more flexible frame models, failure occurs at one of the loaded corners as a region of crashing has quadrant shape bounded by the length of contact as a radii. The compressive failure produces collapse and defined the maximum strength of the infilled frame.

In the stiffest frame, the compressive failure is displaced away from the loaded corners to give a large central region of crashing as shown in fig. (2.36).



Line of Uniform Principal Compressive Stres

Line of Uniform Principal Tensile Stress

Principal compressive stress Trajectory

Principal Tensile Stress Trajectory

Vales of Stress Given in Load Units per Square Length Unit

Fig. (2.36) Modes of Failure[26,39]

CHAPTER 3

CHAPTER 3

EXPERIMENTAL INVESTIGATION FOR INFILL BRICK WALLS

3.1 Introduction

The various functions of brick walls have been well known for a very long time. Recently, there have been significant changes impressive developments in the manner of using brick walls for resisting the lateral load. Limited experimental work has been carried out locally on the various types of bricks and morters, especially, common brick walls.

So in the present work tests were carried out on the three common types of brick walls used in Egypt for determining their global behaviour and their mechanical porperties Young's modulus and Poisson's ratio...etc. and uptill failure. The out put of these tests shall be used in the theoretical analysis and studies carried out in this work.

This work was carried out at the concrete research laboratories, Cairo University.

3.2 Previous Work

Parsan, Hendry and Bradshaw tests[48]

The tests were carried out on a storey-height walls 4.5 inches in thickness. The main conclusions reached are:

1- The typical observed mode of failure by transverse splitting indicated that the tensile strength of the brick and properties of the horizontal joints such as Young's modulus and Poisson's ratio may be of primary importance in determining the strength of brick work.

2- The brick work piers having mortar joint thickness greater than 3/4 inch were weaker than piers having normal joint.

3- Increase in brick work strength of over 60% was observed when every bed joint was reinforced horizontally.

F.EL Refai, A.E. Salama and E.H. Morsy Investigation[49]

They carried out tests on three types of bricks (red bricks, light weight bricks and sand lime bricks) and three morters mixes (cement mortar, lime cement and lime mortar) were used for making masonary specimens with the dimensions shown in fig. (3.1). The main conclusions reached are:

1- Poisson's ratio of mortar is the most influencing property on masonary wall compressive strength.

2- As the rigidity of mortar increases, squeezing out decreases and at equal rigidities the thickness becomes of no effect on masonary strength.

3- When the mortar rigidity increases and becomes higher than that of bricks it introduces lateral restraints at the ends of bricks and consequently the masonary strength increases.

4- Shifting from the mortar softness to the thickness of horizontal joint, experimental measurements showed a considerable reduction in strength when the thickness of horizontal joint is increased especailly when mortar is relatively softer than bricks.

3.3 Theoretical Concepts

It is known that the deformation properties play a major role in defining the brick walls strength in compression. Theoretical analysis is usually based on the lateral interaction between bricks and mortar.

On the baes that bricks are so rigid when compared to mortar, bond between bricks and mortar is stronger than the tensile strength of bricks.

The strength formula $P_{ult.} = \dfrac{\left(\dfrac{a}{a+t_1}\right)\left(\dfrac{f_{tb}}{E_{tb}}\right)}{\left(\dfrac{\mu_m}{E_m}\right)-\left(\dfrac{\mu_b}{E_b}\right)}$

where

$P_{ult.}$ = ultimate compression load

a = brick thickness

t_1 = mortar thickness

f_{tb} = tensile strength of brick

E_{tb} = modulus of elasticity of brick in tension

E_b = modulus of elasticity of brick in comporession

E_m = modulus of elasticity of mortar

μ_m = Poisson's ratio of mortar

μ_b = Poisson's ratio of brick

3.4 Experimental Investigation

Three types of bricks (Hallow cement bricks, loam perforated bricks and sand lime bricks) and portland cement mortar was used to make nine brick wall test specimens (three for each type of bricks) with the dimensions as shown in fig. (3.2) had been investigated in this research work. The model was subjected to vertical uniform load using the 500 ton Amsler Hydraulic machine. The strains at every segment of the wall (each 10 cm.) either in vertical or horizontal direction (local strain E_L) at different load stages were measured using mechanical strain gauges as shown in fig.(3.4). Also the strain of the whole wall either in vertical or horizontal direction (global strain E_g) at different load stages were measured using mechanical strain gauges as shown in fig.(3.5).

3.5 Description of the Test Models

The test models shown in figures (3.6, 3.7, 3.8) is composed of:-

1- Bricks

Three types of bricks were used, the dimensions and weight for brick of each type are given in table (3.1).

2- Mortar

The mixture of ordinary portland cement to sand had the ratio of 1 : 3 by weight. The mixing water was added till a plastic workable mix was obtained.

- **Factors which affect the behaviour of bricks in a wall:**

  a) Brick compressive and tensile strength.

  b)Rectangularity shape and dimesion giving regular bond and uniform joint.

  c)Bed surface levelled to avoid local of stresses concentration.

- **Factors which affect the quality of mortar:**

  a) Type of cement.

  b) Quantity of cement.

  c) Quantity of water.

  d)Petrographic properities, particle shape and size grading of sand.

- **Factors affecting the test:**

  a) Mechanical properties of bricks.

  b) Mechanical properties of mortar.

c) Size and dimensions of test wall.

d) System of joining or bonding.

e) The slenderness of the member.

f) Work man ship.

g) Loading system adequacy.

However, these factors are not the field of this study so they were considerd of some conditions and values for all investigated test walls.

3.6 Test Results and Discussions

1- At the same conditions loam perforated brick walls have a higher compressive strength than Hollow cement brick walls by 100%. Also sand lime brick walls have a higher compressive strength than Hollow cement brick walls by 200%.

This means that the load transfer inside the cement hollow brick wall through the webs and mortar only. Thus the actual loaded area decreases and accordingly the crushing load decreases as given in table below.

Type of brick for test wall	Loaded area cm^2	Crushing load kg.	Crushing stress kg/cm^2
Sand lime	12x52=624	45000	72
Loam perforated	624	30000	48
Hollow cement	624	15000	28

2- From stress-strain curve, the values of Young's modulus for the different types of brick walls are:

Type of brick For test wall	Young's modulus (E) kg/cm^2
Sand lime	66
Loam perforated	20
Hollow cement	11

3- Poisson's ratio varies from segment to segment through the wall either in vertical or horizontal direction. Table below shows arithmatic mean and standard deviation for local and global Poisson's ratio for every type of brick wall specimens.

Type of brick for test wall	Local Poisson's ratio (μ_L)		Global Poisson's ratio (μ_g)	
	Arithmatic mean	Standard deviation	Arithmatic mean	Standard deviation
Sand lime	0.18	0.0065	0.174	0.0052
Loam perforated	0.252	0.0082	0.246	0.0048
Hollow cement	0.278	0.0072	0.276	0.0042

4- The relation between local Poisson's ratio (μ_L) & stresses at top and bottom fiber and center of brick walls shown in fig.(3.10).

4.a. μ_L at top and bottom is smaller than μ_L at center of the wall.

CHAPTER 4

4.b. μ_L and μ_g for stresses up to 48 kg/cm^2 are almost constant and for stresses greater then 48 kg/cm^2, μ_L and μ_g increases with the stresses level increase for sand lime bricks.

For loam perforated bricks μ_L and μ_g for stresses up to 28kg/cm^2 are almost constant and for stresses greater than 28 kg/cm^2 μ_L and μ_g increases with the stresses level increase.

For hollow cement μ_L and μ_g for stresses up to 17 kg/cm^2 are almost constant and for stresses greater than 17 kg/cm^2, μ_L and μ_g increases with the stresses level increase.

5- Fig. (3.12) shows the relation between μ_g and μ_L for different types of brick walls. Take value of the angle with the horizontal for each wall type is as follows:

Type of brick for test walls	The angle ϕ
Sand lime	44^o
Loam perforated	40^o
Hollow cement	43^o

6- Cracks propagation and pattern for various types of brick wall specimens are illustrated in figures (3.13, 3.14, 3.15).

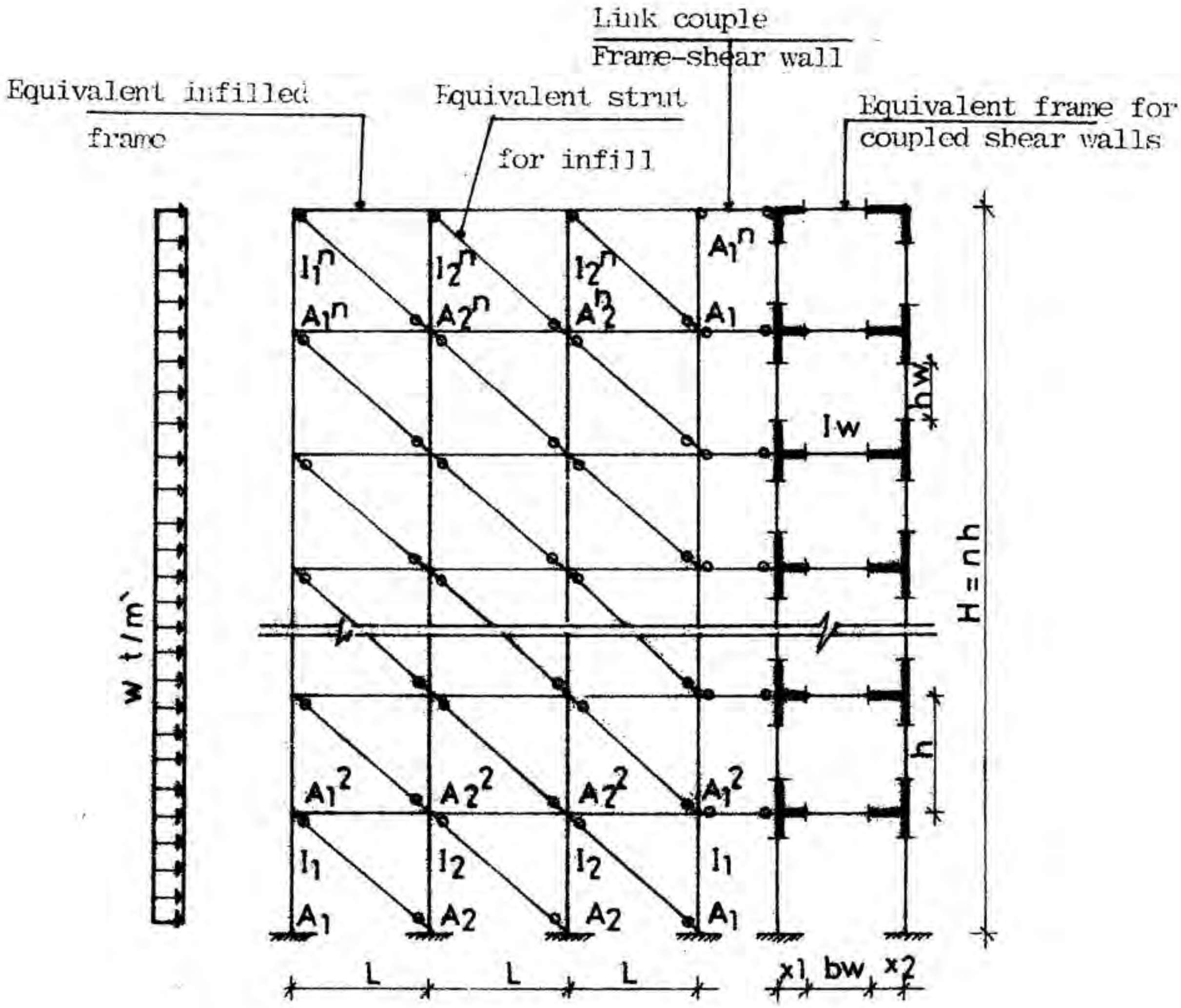


Figure. (4-2) Proposed Model

(t_{panel}). The effective width of the equivalent diagonal strut (b_e) is considered as the width of an equally stiff uniform strut.

The effective width of the diagonal strut (b_e) considered in the analysis of all the investigated models has been calculated using the relationship given by Liauw and Khan[29].

$b_e = 0.45\ h \cos\theta$

in which, h is the storey height and is the angle of panel diagonal to the horizontal.

The three types of filling material commonly used in Egypt have been investigated in this work to explore the effect of the type of bricks infill.

The values of modulus of elasticity and Poisson's ratio for the three types of brick walls considered has been determined experimentally as explained befor (chapter 3).

c) Shear walls:

An equivalent reinforced concrete coupled shear wall of height ($H = nh$), width (b) and the dimensions ofthe openings (b_w, h_w) is used to represent the system of shear wall. The general plane frame element shown in fig. (4.3) is taken to represent the element, of the structure.

d) Loads:

The model is subjected to a uniform lateral load P_w

$P_w = cq$

Where

q is the pressure of wind per m^2 acting horizontally and it vaires according to the height of the building from the ground level as follows:

Height of building ms.	0-8	8-20	20-100	100-200	200
Wind pressure q (kg/cm^2)	50	76	100	125	150

C is a factor determined by aerodynamical tests according to the shape and size of the structure[50].

$C = 1.2 \sin\alpha - 0.4$ for inward surface in case of sheds

$C = 1.6 \sin\alpha - 0.4$ for inward surface in case of towers

$C = -0.4$ for leaward exposed surface

is the angle of inclination of the roof to the horizontal.

4.3 Type of Elements

The method adopted in this work requires the use of plane frame program with the element shown in fig. (4.3) and is applicable to wide range of problems with no restriction on loading.

The frame element is the basic element which can be used for representing both columns, floor slabs or beams, equivalent strut for infill and shear wall with or without openings.

The complete element stiffness matrix has been developed taking into consideration the shear, bending and axial deformation of the flexible part, rotational springs and the rigid parts.

The use of the rotational springs is optional but they can be used to take into account the local deformations or to allow a pin connection between the rigid and flexible parts.

Developement of Element Stiffness [KM][51]:

Basic Stress - Strain Matrix (K_r)

The basic stress-strain matrix (K_r) relates the end actions shown in fig. (4.4.a) to the corresponding deformations, i.e.

$$\begin{Bmatrix} N_a \\ M_a \\ M_b \end{Bmatrix} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 \\ 0 & K_{11} & K_{12} \\ 0 & K_{21} & K_{22} \end{bmatrix} \begin{Bmatrix} \Delta_a \\ \theta_a \\ \theta_b \end{Bmatrix} \quad \text{.......(4.1)}$$

$P_{ab} = K_r \cdot V_{ab}$

Where

E = Young's modulus

A = Cross sectional area

L = Flexible length

A convenient way to formulate the K_{ij} terms is to form the equivalent flexibility relationship and then invert it, i.e. the terms to be established are the f_{ij}'s terms in the following relation:

$$\begin{Bmatrix} a \\ b \end{Bmatrix} = \begin{bmatrix} f_{11} & f_{12} \\ f_{21} & f_{22} \end{bmatrix} \begin{Bmatrix} M_a \\ M_b \end{Bmatrix} \qquad \text{........(4.2)}$$

The f_{ij} terms are dependent on bending, shear and rotational spring deformation as follows:

Bending deformation:

$$f_{11} = f_{12} = \frac{L}{3\ EI}$$

$$f_{12} = f_{21} = -\frac{L}{6\ EI}$$

Where

I = second moment of area of flexible part.

Shear deforamtion:

$$f_{11} = f_{12} = f_{21} = f_{22} = \frac{1}{L\bar{A}G}$$

Where

$\bar{A}$ = Equivalent shear area = ASF x area

$$ASF = \frac{5}{6} \text{ for a rectangular section}$$

$$= \frac{A_{web}}{A_{total}} \quad \text{for I section}$$

$$G = \text{shear modulus} = \frac{E}{2(1+\mu)}$$

μ = Poisson's ratio

Rotational spring:

$$f_{11} = \frac{1}{K_{s1}} \qquad f_{22} = \frac{1}{K_{s2}} \qquad f_{12} = f_{21} = 0$$

Where

K_{s1} = Rotational stiffness of the spring at end 1

K_{s2} = Rotational stiffness of the spring at end 2

$$\begin{Bmatrix} a \\ b \end{Bmatrix} = \begin{bmatrix} \left(\frac{L}{3EI} + \frac{1}{LAG} + \frac{1}{K_{si}}\right) & \left(\frac{L}{6EI} + \frac{1}{LAG}\right) \\ \left(-\frac{L}{6EI} + \frac{1}{LAG}\right) & \left(\frac{L}{3EI} + \frac{1}{LAG} + \frac{1}{K_{s2}}\right) \end{bmatrix} \begin{Bmatrix} M_a \\ M_b \end{Bmatrix}$$

Defining:

$$F_1 = \frac{1}{1+\frac{4EI}{LK_{s1}}} \qquad F_2 = \frac{1}{1+\frac{4EI}{LK_{s2}}} \qquad B = \frac{2(1+\ \ )\,I}{AL^2}$$

$$C_1 = 4\left(\frac{1}{3} + B\right) \qquad C_2 = 4\left(B - \frac{1}{6}\right)$$

Equation (4.2) can be rewritten as:-

$$\begin{Bmatrix} a \\ b \end{Bmatrix} = \frac{L}{4EI}\begin{bmatrix} \left(C_1 + \frac{1}{F_1} - 1\right) & C_2 \\ C_2 & \left(C_1 + \frac{1}{F_2} - 1\right) \end{bmatrix} \begin{Bmatrix} M_a \\ M_b \end{Bmatrix}$$

When $F = 0.0$ the connection is pinned

$F = 1.0$ the connection is fully fixed

Inverting this gives:

$$\begin{Bmatrix} M_a \\ M_b \end{Bmatrix} = \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \begin{Bmatrix} a \\ b \end{Bmatrix} \qquad \text{.........(4.3)}$$

Where

$$K_{11} = \frac{4EI}{LD} (1 - F_2)\ F_1 + C_1F_1\ F_2$$

$$K_{12} = K_{21} = - \frac{4EI}{LD} C_2F_1F_2$$

$$K_{22} = \frac{4EI}{LD} (1 - F_1)\ F_2 + C_1F_1F_2$$

$$D = (1 - F_1 + F_1C_1)(1 - F_2 + F_2C_1) - F_1F_2C_2^2$$

All the terms of K_r are thus established.

Transformation to the system of fig.(4.4.b)

The action of fig. (4.4.a) - P_r are related to those of fig. (4.4.b) - P_{ab} by

$$\begin{Bmatrix} N_a \\ V_a \\ M_a \\ N_b \\ V_b \\ M_b \end{Bmatrix} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & -\frac{1}{L} & -\frac{1}{L} \\ 0 & 1 & 0 \\ -1 & 0 & 0 \\ 0 & \frac{1}{L} & \frac{1}{L} \\ 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} N_a \\ M_a \\ M_b \end{Bmatrix} \qquad \text{.........(4.4)}$$

i.e. $P_{ab} = H^T\ P_r$ and by contragredience

$$U_r = H\ U_{ab} \qquad \text{............(4.5)}$$

Where

U = vector of deformations

Transformation to rigid ends

P_{ab} is related to the actions of fig. (4.4.c) - P_{12} by

$$\begin{Bmatrix} N_1 \\ V_1 \\ M_1 \\ N_2 \\ V_2 \\ M_2 \end{Bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ -Y1 & -X1 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & -Y2 & -X2 & 1 \end{bmatrix} \begin{Bmatrix} N_a \\ V_a \\ M_a \\ N_b \\ V_b \\ M_b \end{Bmatrix} \quad \ldots(4.6)$$

i.e. $P_{12} = T^T P_{ab}$ and by contragredience

$$U_{ab} = T\, U_{12} \quad \ldots\ldots\ldots\ldots(4.7)$$

Rotation to global co-ordinate system

P_{12} are related to the actions of fig.(4.3)- P_{xy} by:

$$\begin{Bmatrix} P_{x1} \\ P_{y1} \\ M_1 \\ P_{x2} \\ P_{y2} \\ M_2 \end{Bmatrix} = \begin{bmatrix} \cos\theta & \sin\theta & 0 & 0 & 0 & 0 \\ -\sin\theta & \cos\theta & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\theta & \sin\theta & 0 \\ 0 & 0 & 0 & -\sin\theta & -\cos\theta & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \begin{Bmatrix} N_1 \\ V_1 \\ M_1 \\ N_2 \\ V_2 \\ M_2 \end{Bmatrix} \quad \ldots(4.8)$$

i.e. $P_{xy} = R^T P_{12}$

and the conrresponding deformations are related by

$$U_{12} = R\, U_{xy} \quad \ldots\ldots\ldots(4.9)$$

Formation of the element stiffness matrix - KM :

Substituting eqs.(4.4) to (4.9) into eq.(4.1) gives

$$P_{xy} = R^T \, T^T \, H^T \, K_r \, H \, T \, R \, U_{xy} \qquad \text{..........(4.10)}$$

i.e. $P_{xy} = K \, M \, U_{xy}$

Where

$KM = R^T \, T^T \, H^T \, K_r \, H \, T \, R$ is the required element stiffness matrix.

The element can be used for representing:-

a- **Frame:**

Use flexible part with length L and fixed ends

i.e. X1 = X2 = Y1 = Y2 = 0 F = 1.0

b- **Strut (truss element):**

Use flexible part with length L and pin connection

i.e. X1 = X2 = Y1 = Y2 = 0 F = 0.0

c- **Solid shear wall:**

Each storey is idealized as one element

i.e. Element length L = storey height h

X1 = X2 = Y1 = Y2 = 0 F = 1.0

d- **Shear wall with openings (coupled shear wall):**

Each beam can be represented by horizontal element with opening width (b_w = L)

X1 = X2 = half width of wall segment

Y1 = Y2 = 0 F = 1.0

Each column can be represented by vertical element with opening height (h_w = L)

X1 = X2 = half depth of connecting beam

Y1 = Y2 = 0 F = 1.0

4.4 Computer Program

The program is prepared to operate on micro computer mashine I.B.M. with an internal storage of 64000 bites. The flow chart for this program is shown in fig.(4.5).

Program out put

The out put of the computer includes mainly, the nodal displacements and nodal forces (bending moments, normal force and shearing forces) for each structural element.

4.5 Case Study

The effect of the different parameters mentioned above, on the straining actions and behaviour of the structure shown in fig.(4.1.a) has been studied. The values of different parameters and the dimensions of various elements are taken as follows to simplify the problem.

4.5.1 Frames

The models consists of reinforced concrete frame of height (H = nh) where N = total number of stories considered . It was taken equal to 10, 15 and 20 stories and h = storey height equals 3.0 m. The values of both L and m were taken 4.0 m. The breadth of the beams was equal to 0.25 m. and the depth of the beams was equal to 0.7 m.

The cross sections for all columns was as shown in table (4.1) to represent practical cases.

4.5.2 Filling materials

Different filling materials having various thickness were considered. The three types of filling materials considered are sand lime bricks, loam perforated bricks and hollow cement bricks. The modulus of elasticity and Poisson's ratio were [(E= 66, 20 & 11 t/cm^2), (μ = 0.175, 0.245 & 0.275)] for each of these fillings respectively. These values were calculated from the experimental results obtained as explained before.

Each filling material has been studied for two panel thickness (t_{panel}) i.e. one brick and half a brick thickness. The length of equivalent diagonal strut (d) was taken equal to 5.0 m. and the effective width of equivalent strut (b_e) was equal to 1.0 m.

4.5.3 Coupled shear wall

The shear wall has one row of openings as shown in fig.(4.1.a). The heights and widths of openings considered in this work are four values as follows:

$$\frac{h_w}{h} = 0,\ 0.4,\ 0.6\ \&\ 0.8$$

$$\frac{b_w}{b} = 0,\ 0.4,\ 0.6\ \&\ 0.8$$

Where

h = storey height = 3.0 m.

b = wall width = 4.0 m.

h_w = opening height

b_w = opening width

The thickness of the shear wall was taken constant for all cases and is equal to 0.25 m.

The flow chart shown in fig. (4.6) illustrates the various cases studied in this investigation.

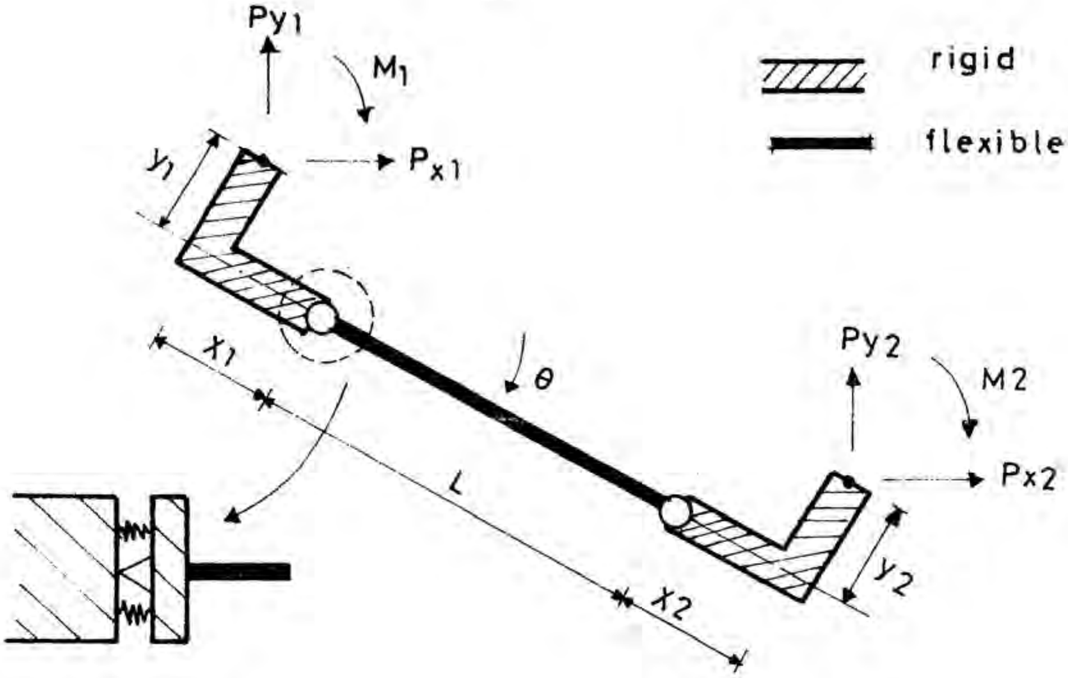


Fig. (4.3): General Form of Element

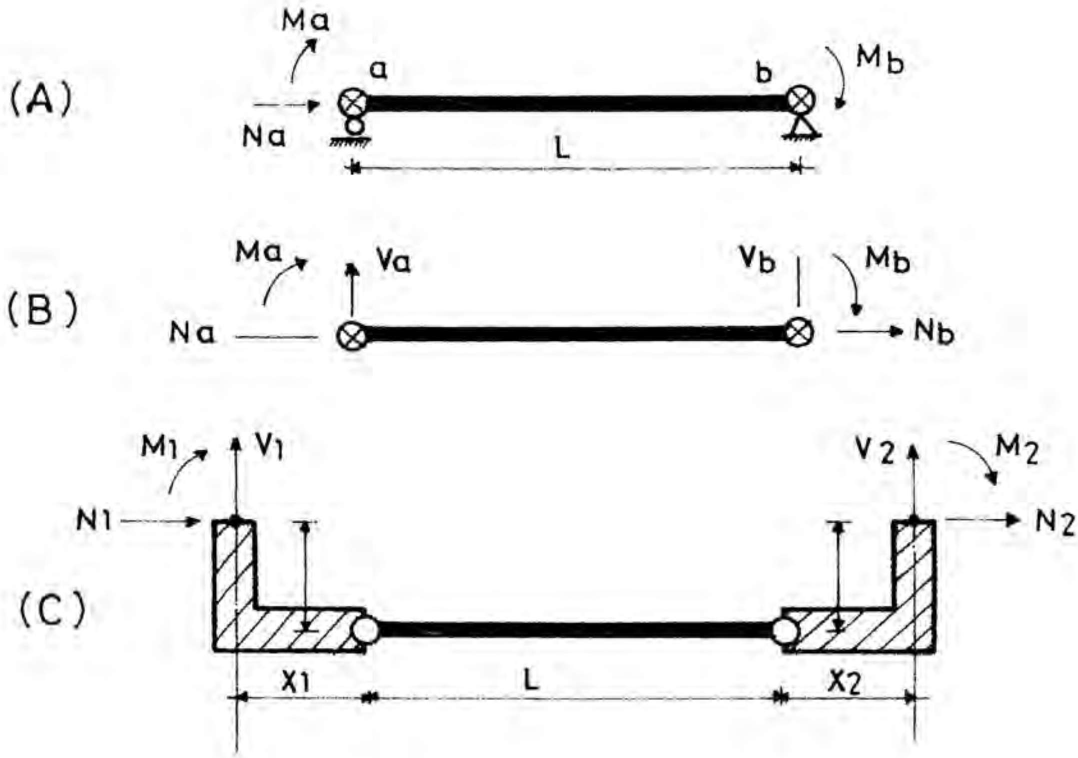


Fig. (4.4): Steps for Forming Element Stiffness

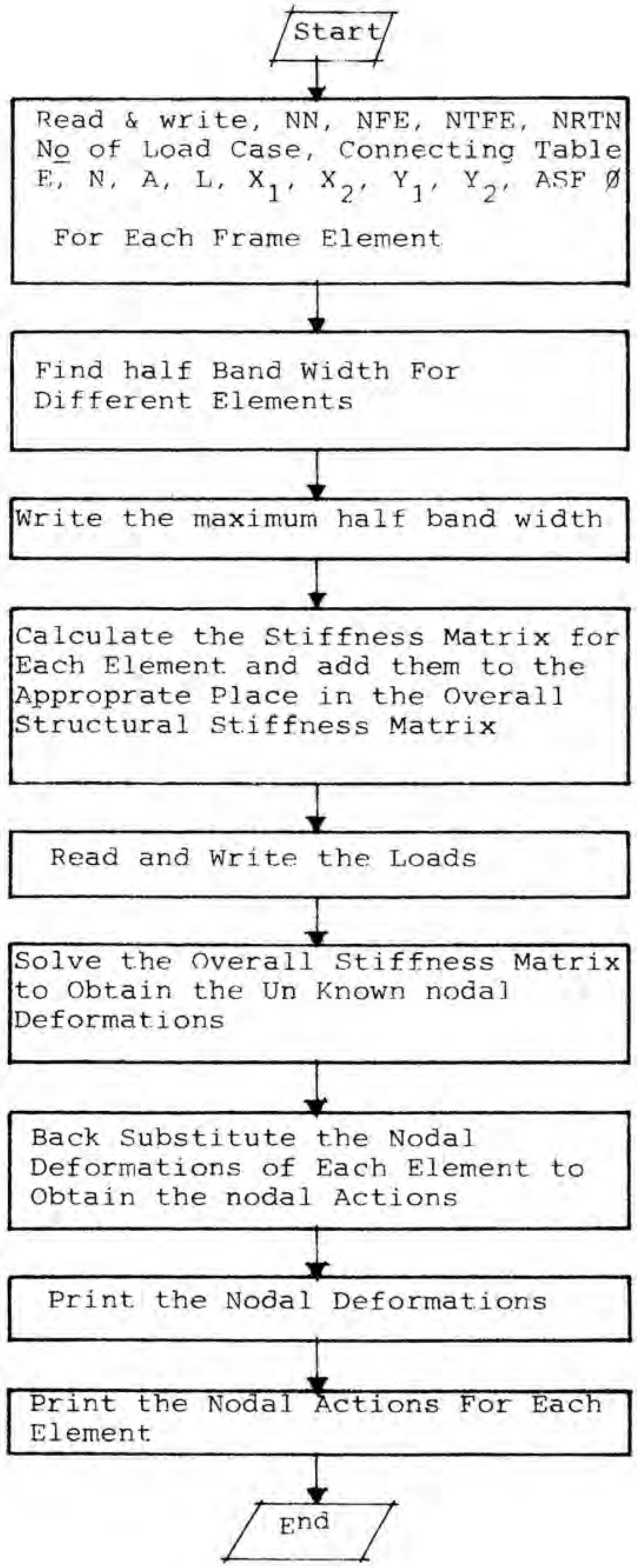
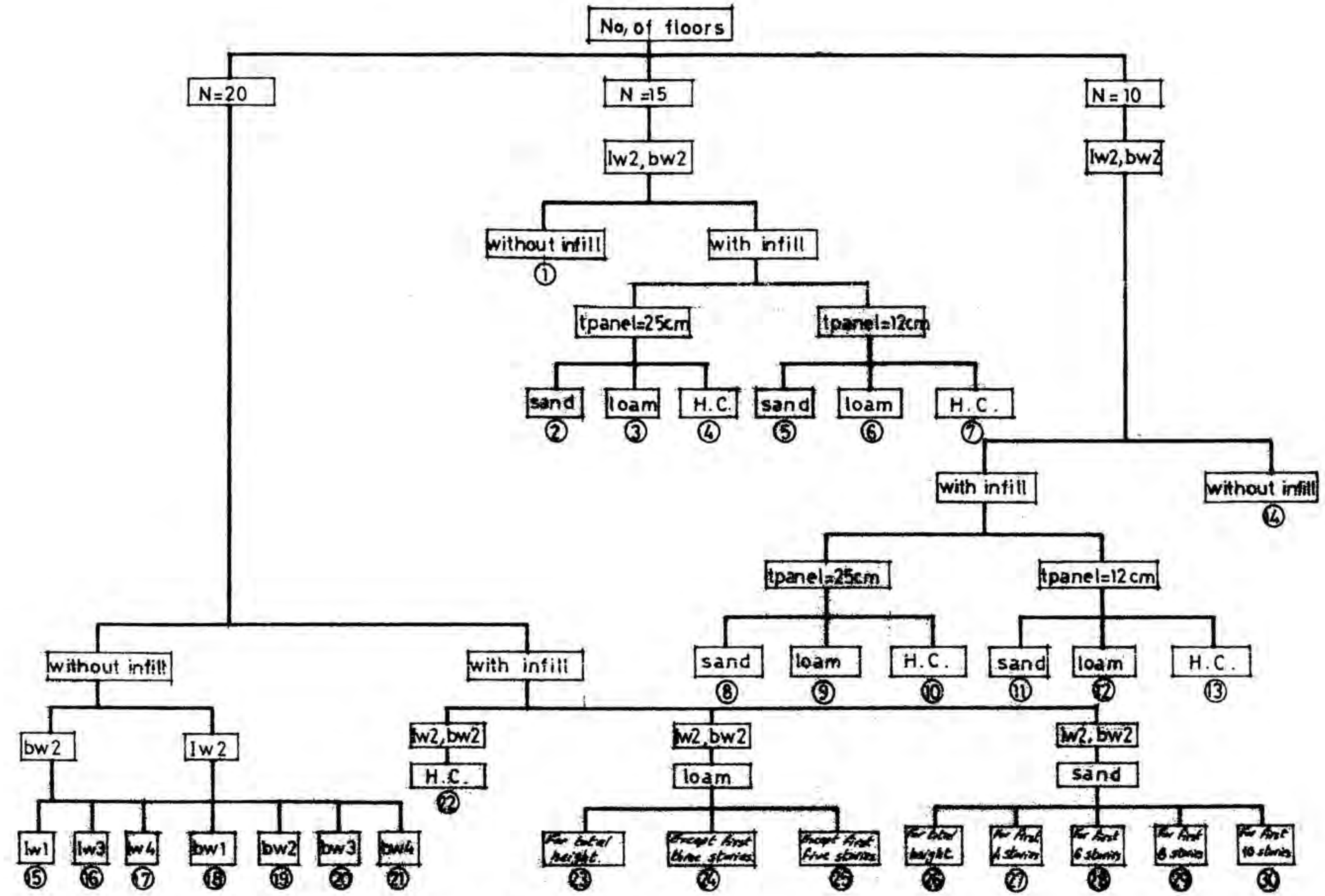


Fig. (4.5) Computer Program Flow Chart



Figure(4-6) Study Cases

N=20

N=15

N=10

No. of stories	1,2	3,4	5,6	7,8	9,10	11,12	13,14	15,16	17,18	19,20
A_1	30x110	30x100	30x90	25x90	25x80	25x70	25x60	25x50	25x40	25x30
A_2	40x140	40x130	35x130	35x120	30x120	30x110	30x100	25x100	25x80	25x60

Table (4.1) Columns cross-section

CHAPTER 5

CHAPTER 5

ANALYSIS AND DISCUSSION OF RESULTS

5.1 Introduction

The behaviour of a reinforced concrete multi-storey building is affected tangibly not only by the skelton conditions, but also by the infilled material and the size of the openings in the shear walls.

The flow chart, fig.(4.7) shows three groups of height of buildings, i.e. ten, fifteen and twenty stories. The panel thickness and types of bricks used as infill are also varied. The effect of variation in opening width to wall width ratio $(\frac{bw}{b})$ are studied. Also the effect of variation in these opening hight to the height of storey $(\frac{hw}{h})$ are investigated.

5.2 Effect of Filling Material and it's Type

The infill panels investigated in this research are made of sand lime, loam and hollow cement bricks. The values obtained experimentally for Young's modulus are (66, 20 and 11 t/cm^2) and for Poisson's ratio are (0.175, 0.245 and 0.275) respectively, and the brick walls are taken one brick, i.e. 25 cm. in thickness to illustrate that effect.

5.2.1 Lateral deflection of structure

The effect of infill and its type on the lateral deflection of structure are varied according to the number of stories. The values of lateral deflection of the structure for all the considered cases are calculated and plotted against the number of stories as shown in figures (5.1) to (5.3).

It is observed that for all the cases investigated the lateral deflection of the structure decreases when infill is used. Also the amount of decrease in deflection depends on the type of infill.

Figure (5.1) shows the lateral deflection of the building against the number of stories (N=10). The values of lateral deflection decrease with the increase of Young's modulus and the decrease of Poisson's ratio of the filling materail. For instance the lateral deflection decreases by about 55% when using sand lime bricks, by about 38% when using loam bricks and by about 28% when using hollow cement bricks.

Figure (5.2) shows the lateral deflection against the number of stories (N=15). The values of lateral deflection decrease by about (44%, 32% and 22%) when using sand lime bricks, loam bricks and hollow cement bricks respectively.

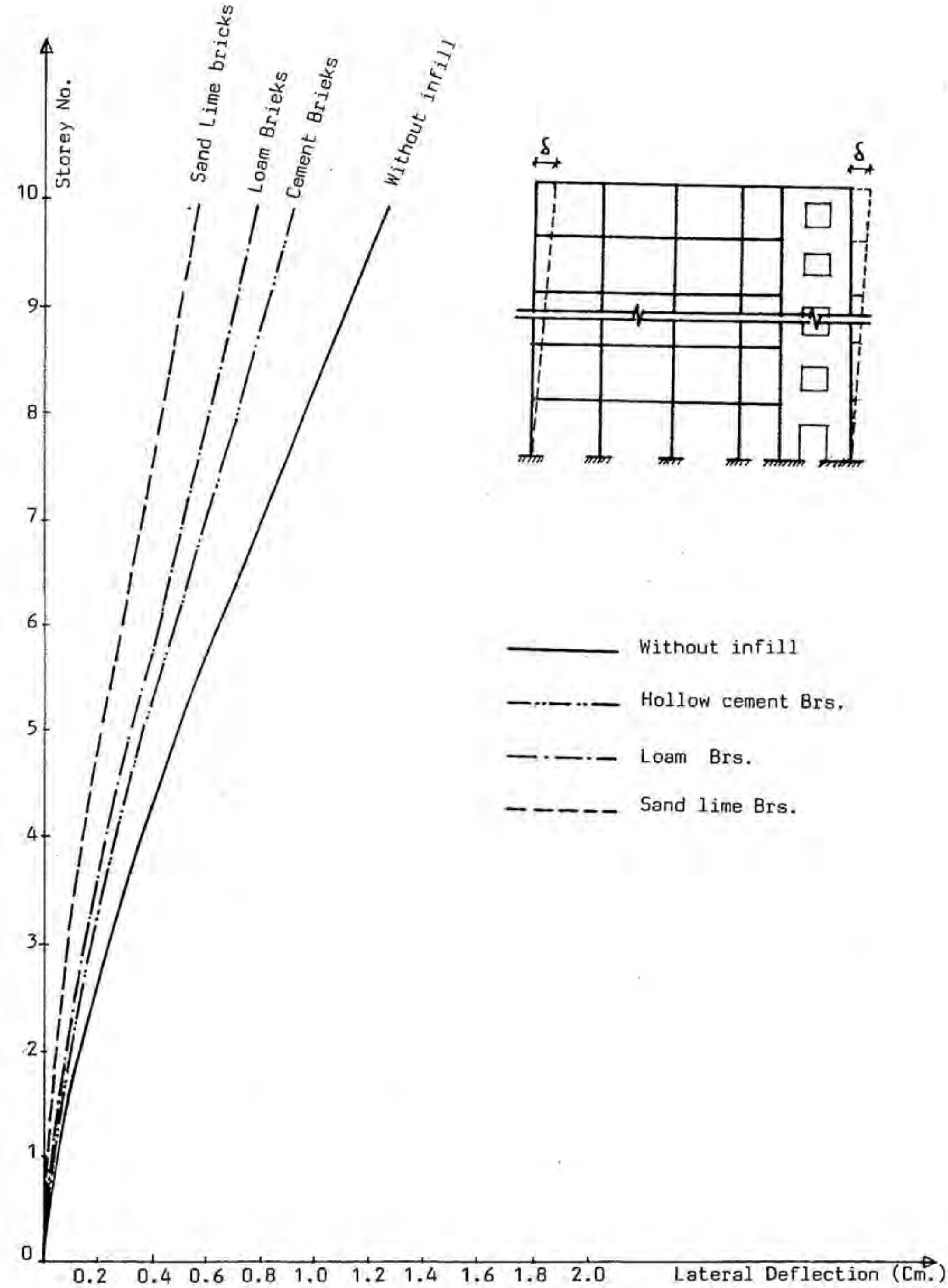


Fig. (5.1) Effect of infill on the building lateral deflection (Total No. of stories N = 10).

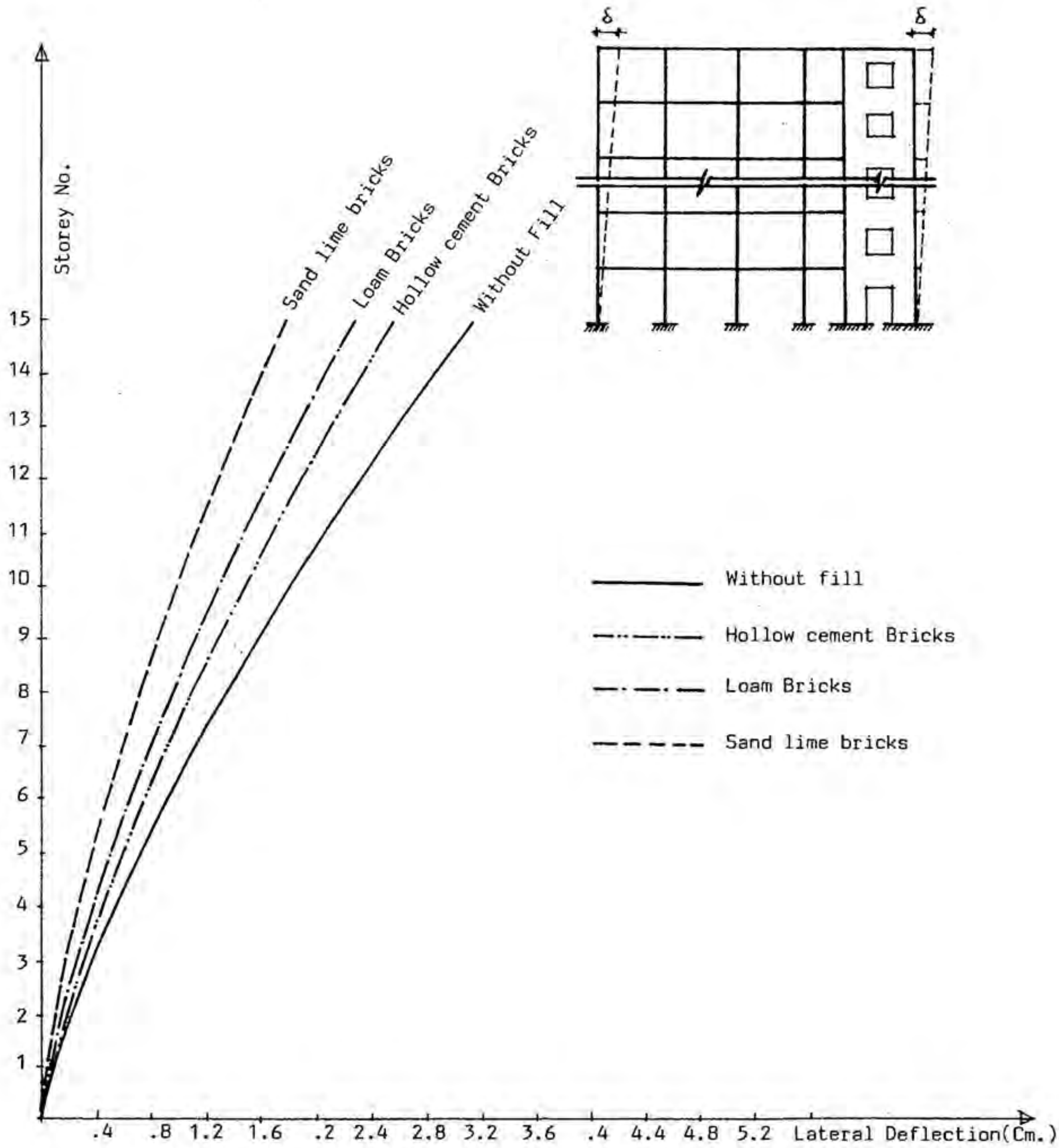


Fig. (5.2) Effect infill on the building Lateral Deflection (Total No. of stories N = 15).

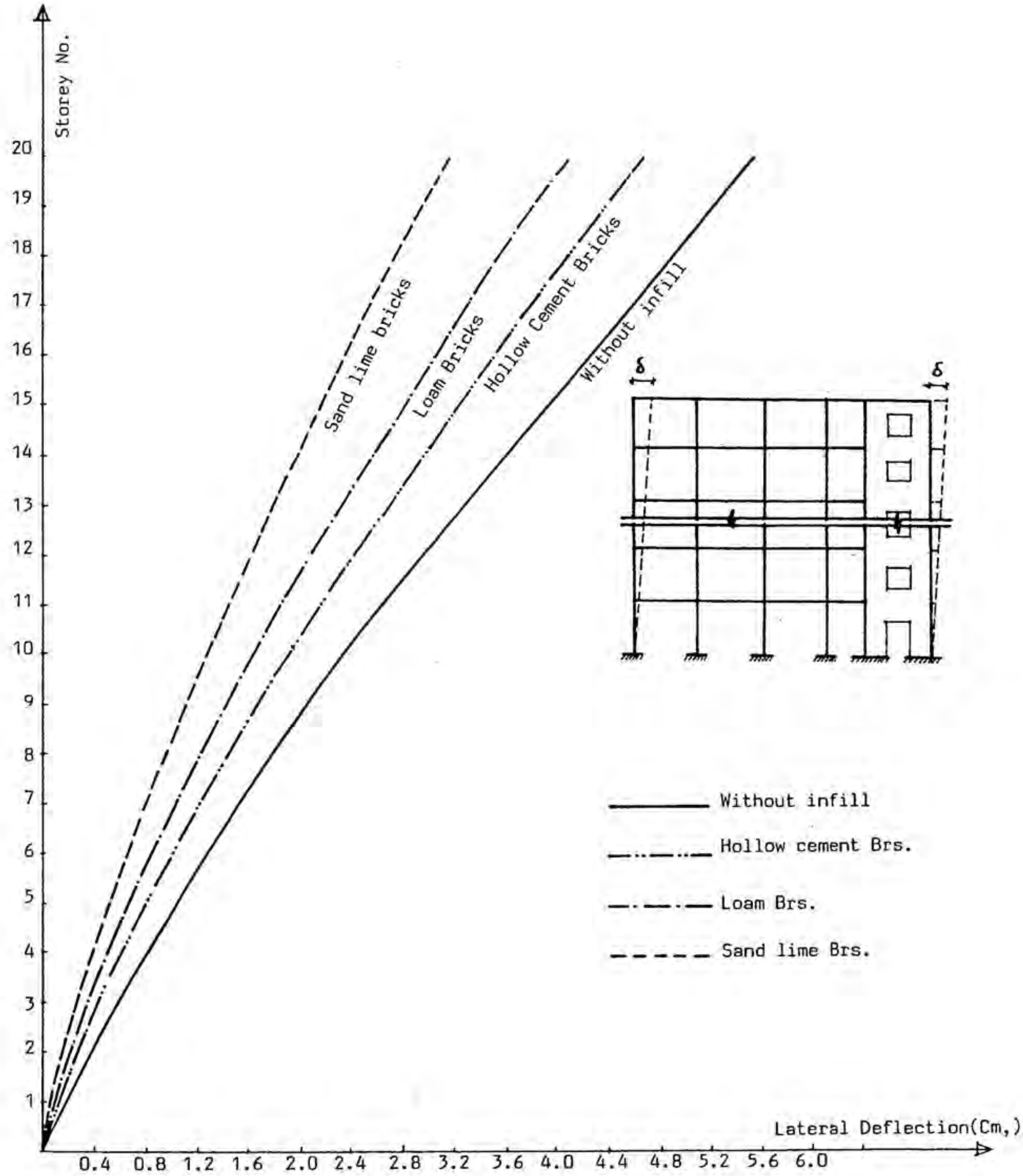


Fig. (5.3) Effect of infill on the building Lateral deflection
(Total No. of stories N = 20).

Figure (5.3) shows the lateral deflection against the number of stories (N=20). The values of lateral deflection decrease by about (38%, 25% and 18%) when using sand lime bricks, loam bricks and hollow cement bricks respectively.

5.2.2 Bending moments in the columns of the building

a- Exterior columns:

The values of maximum bending moment in exterior columns for all the investigated models were calculated and plotted for each storey number as shown in figures (5.4) to (5.6).

Figure (5.4) illustrates the bending moment in exterior columns against the storey number (N=10). The values of maximum bending moment decrease by about (64%,40% and 30%) when using sand lime, loam and hollow cement bricks respectively.

The maximum bending moments decrease by about (60%,38% and 28%) when using sand lime, loam and hollow cement bricks respectively for the case of N=15 as shown in fig.(5.5).

The maximum bending moments decrease by about (52%, 36% and 25%) when using sand lime, loam and hollow cement bricks respectively as shown in fig. (5.6) for the case of (N=20).

b- Interior columns:

The values of bending moments in interior columns for all the investigated models were calculated and plotted against the number of stories as shown in figures (5.7) to (5.9).

Figure (5.7) shows the relation between bending moment and storey number (N=10). The values of maximum bending moments decrease by about (68%, 48% and 40%) when using sand lime, loam and hollow cement bricks respectively.

Figure (5.8) shows the bending moment against the storey number (N=15). The values of maximum bending moments decrease by about (65%, 45% and 35%) when using sand lime, loam and hollow cement bricks respectively.

The values of maximum bending moment decrease by about (56%, 42% and 30%) when using sand lime, loam and hollow cement bricks respectively, in the case of (N=20) as shown in fig. (5.9).

In general, it can be noticed that the maximum bending moment in exterior and interior columns decreases by using infill. Also the amount of decrease in bending moment depends on the type of the infill.

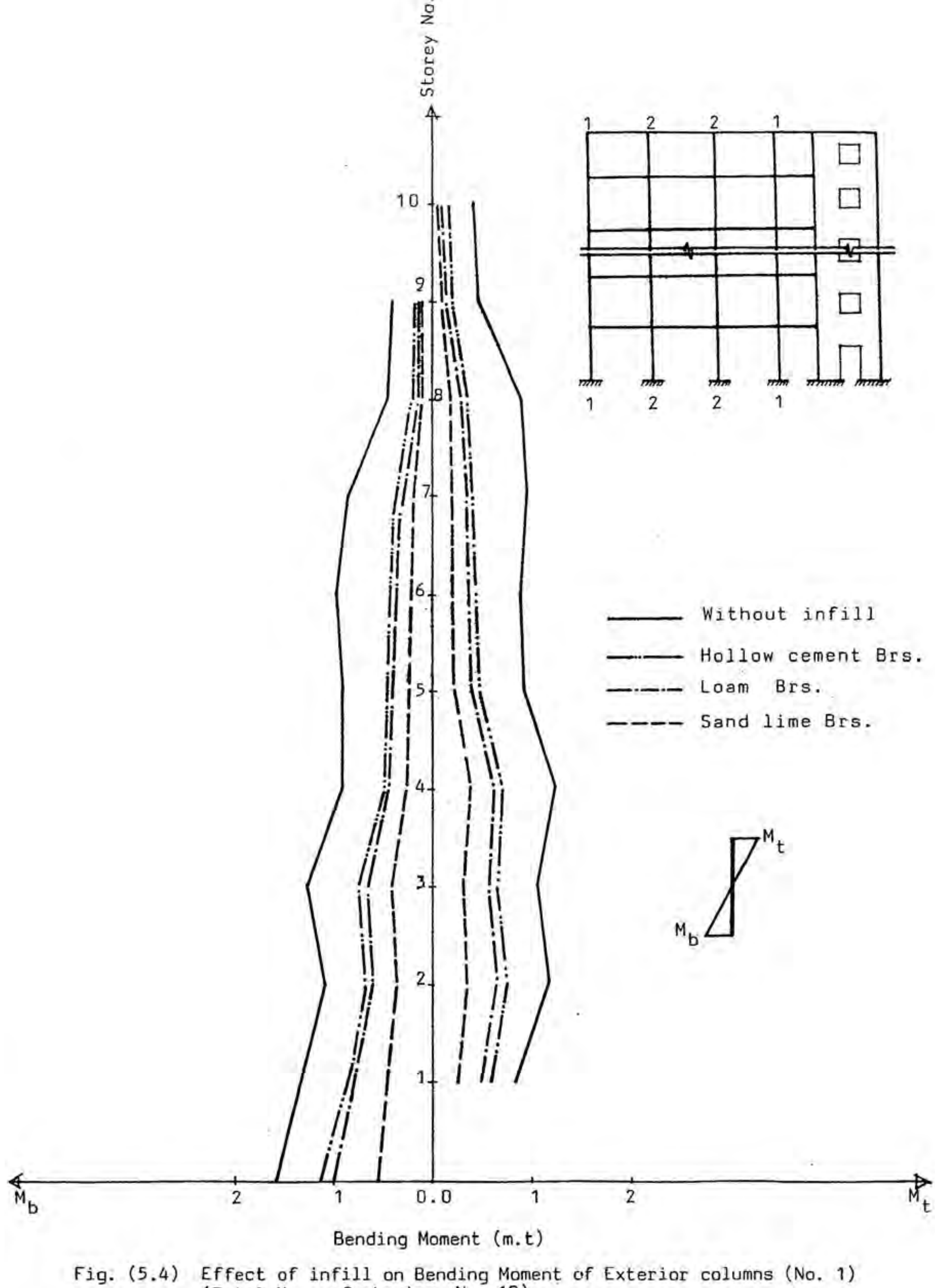


Fig. (5.4) Effect of infill on Bending Moment of Exterior columns (No. 1) (Total No. of stories N = 10).

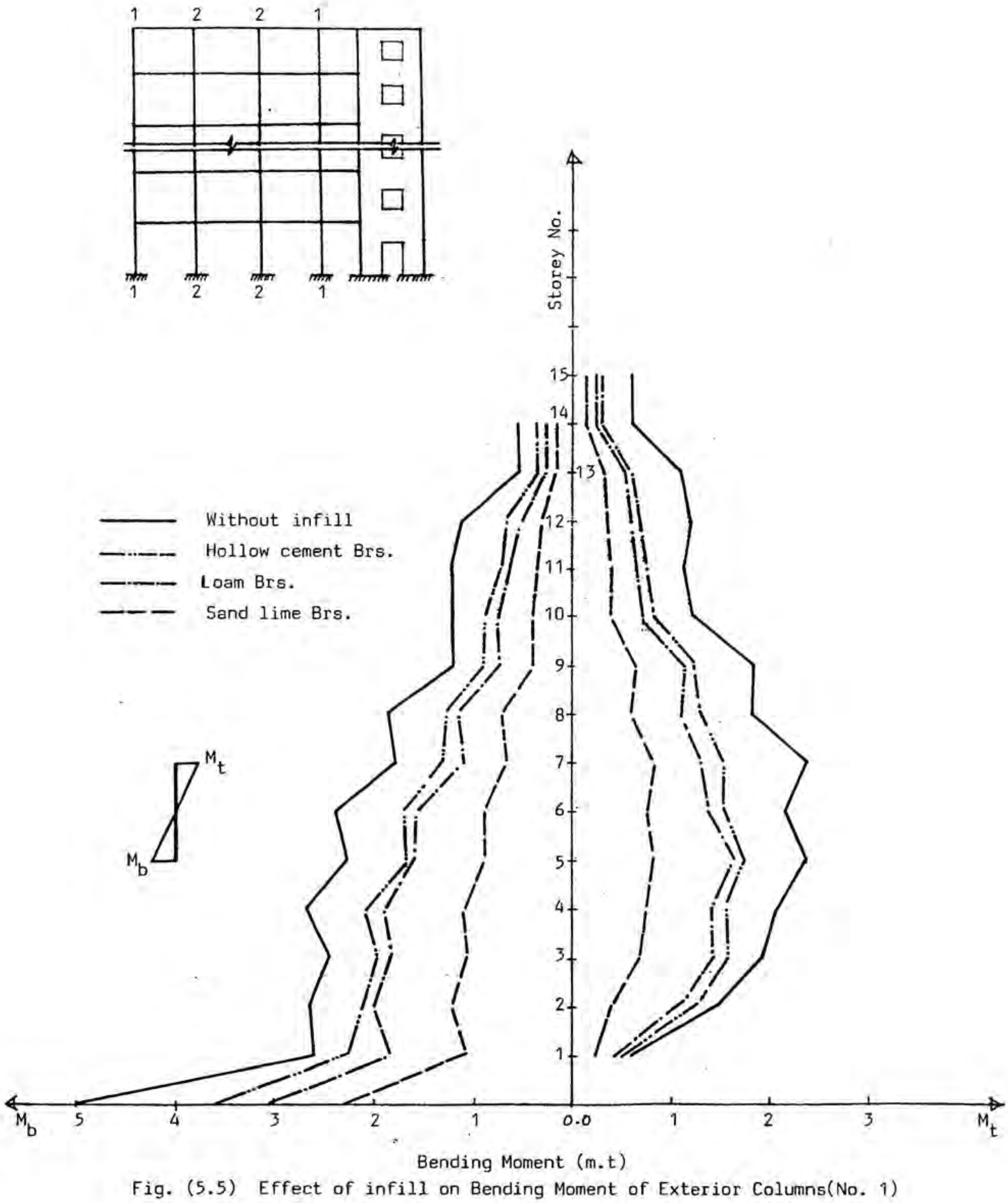


Fig. (5.5) Effect of infill on Bending Moment of Exterior Columns(No. 1)

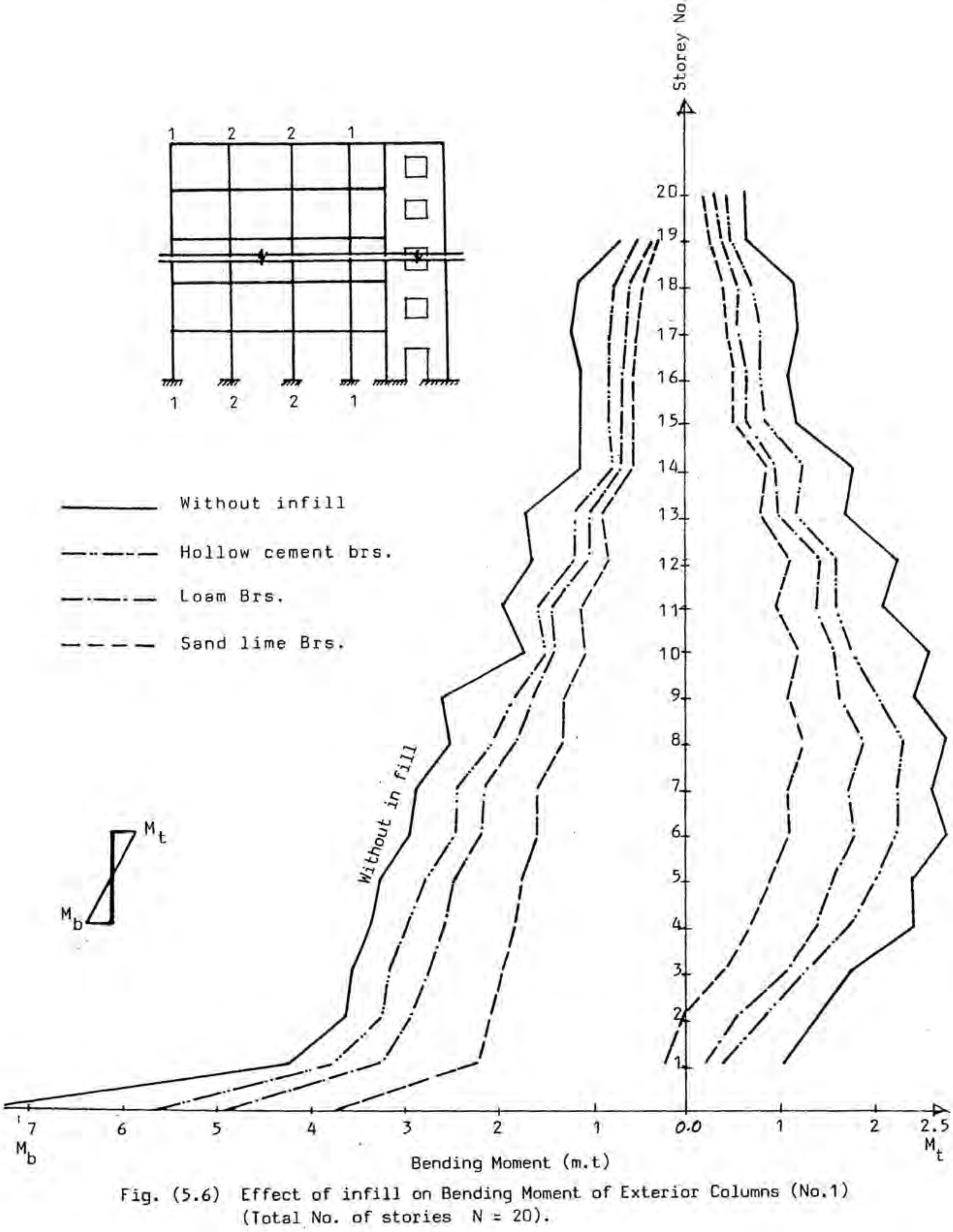


Fig. (5.6) Effect of infill on Bending Moment of Exterior Columns (No.1)
(Total No. of stories N = 20).

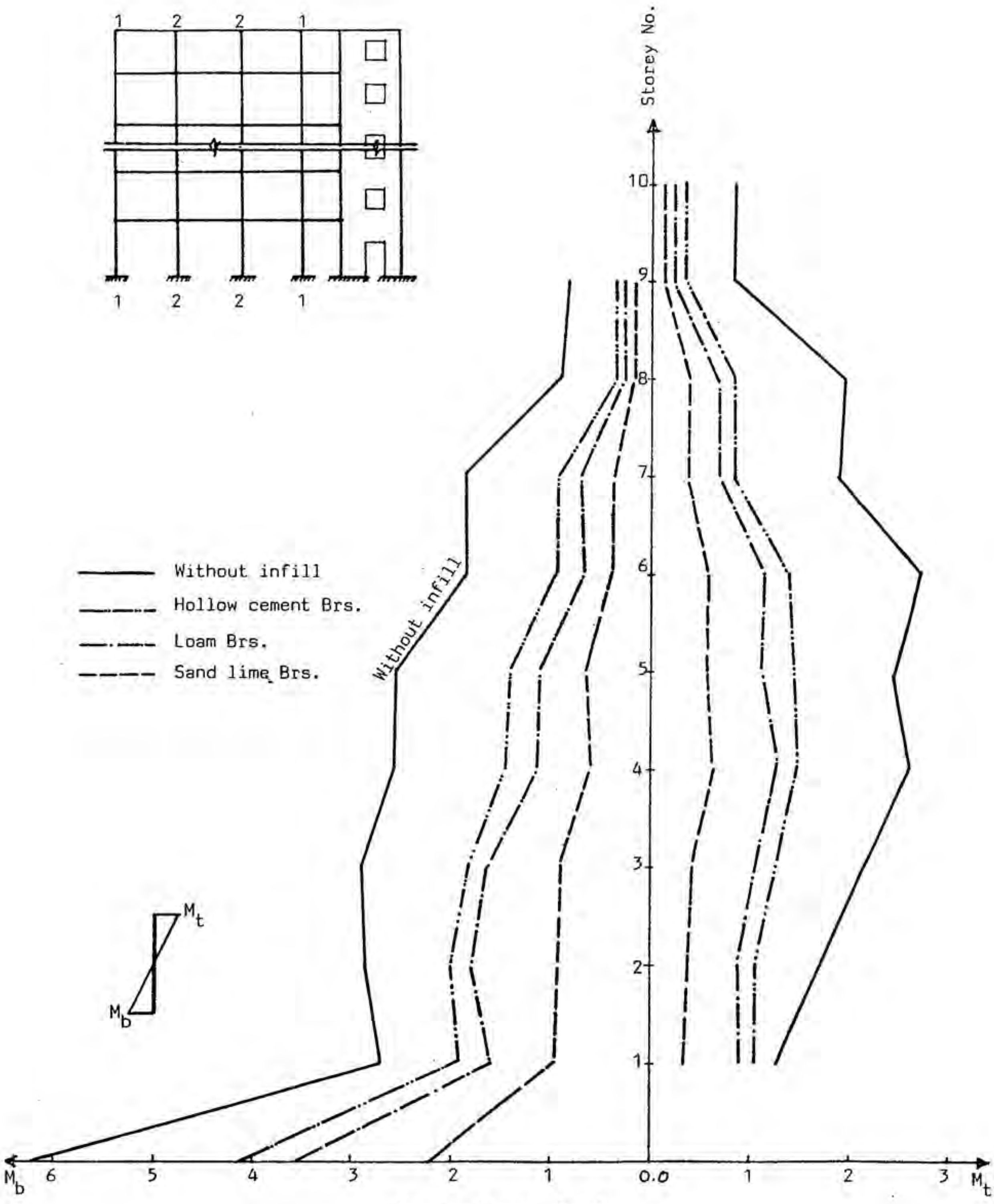


Fig. (5.7) Effect of infill on Bending Moment of Interior columns (No. 1)
(Total No. of stories N = 20).

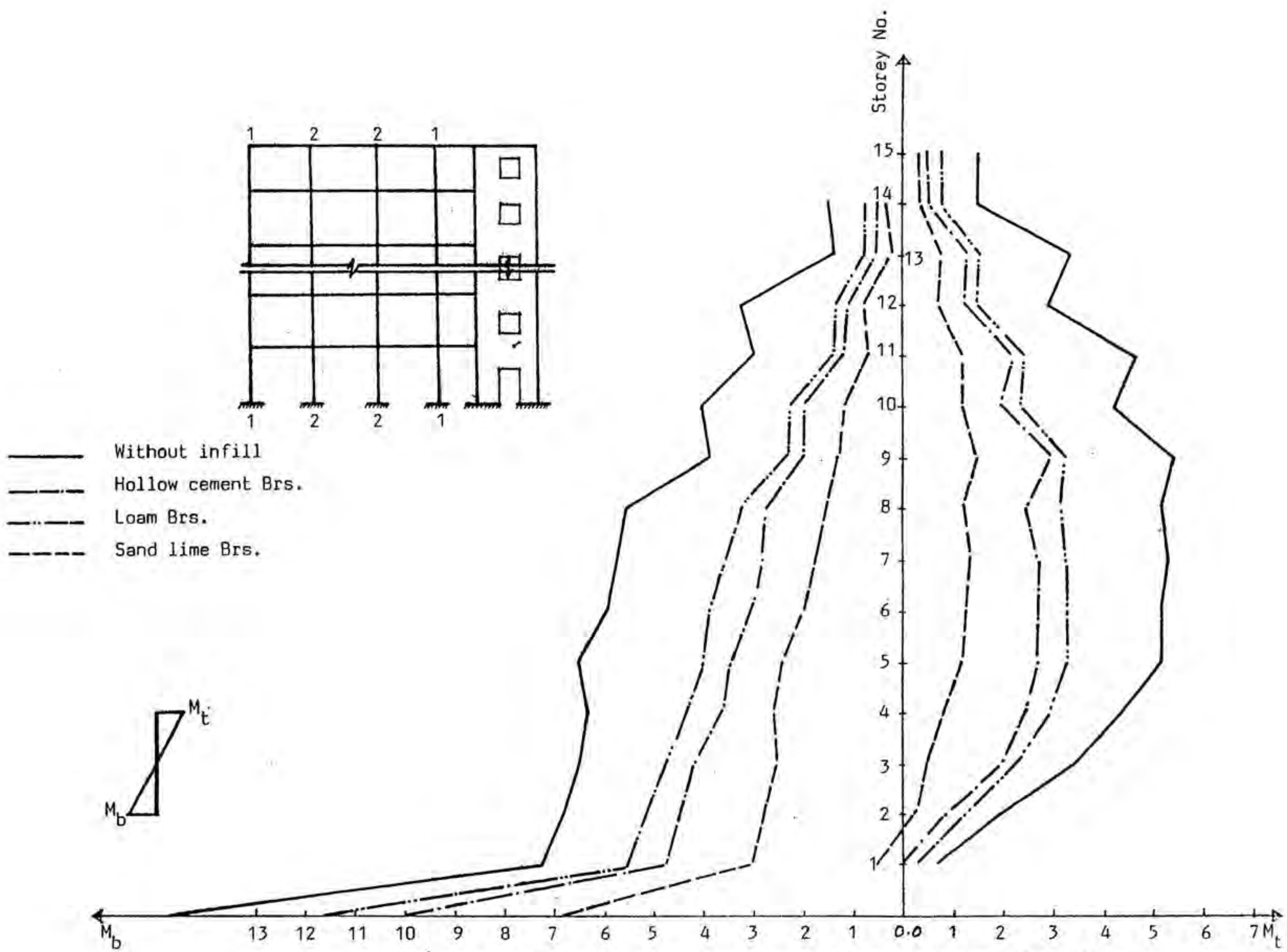


Fig. (5.8) Effect of Infill on Bending Moment of Interior Columns (No. 2)
(Total No. of stories N = 15).

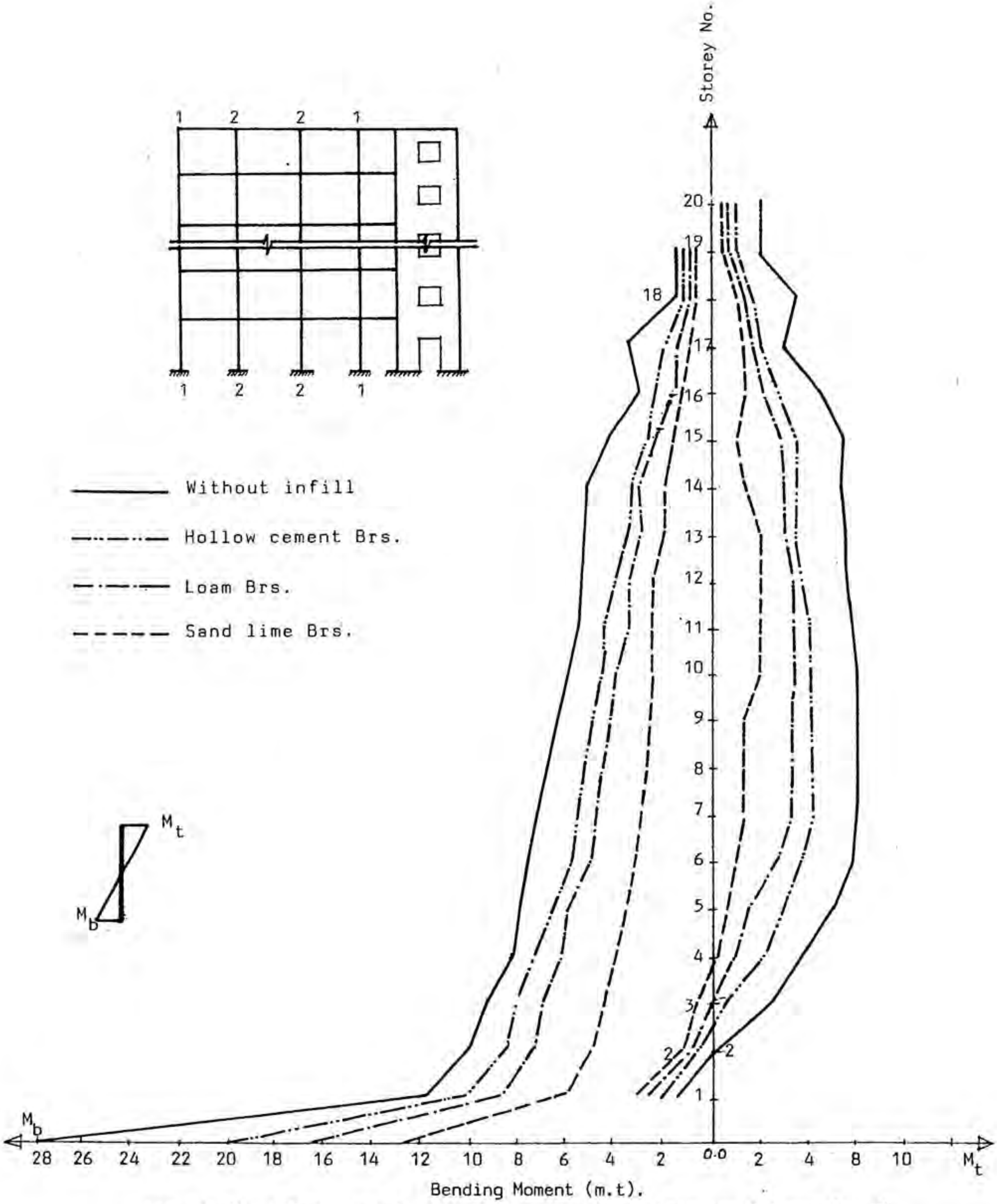


Fig. (5.9) Effect of infill on Bending Moment of Interior Columns (No.2)
(Total No. of stories N = 20).

5.2.3 Bending moments in the beams of the building

a- Exterior beams:

Figures (5.10) to (5.12) represent the relation between the maximum values of bending moments in exterior beams and storey number for different filling materials.

It can be noticed that the values of maximum bending moment in beams at 0.3H decrease when using infill. This decrease depends on the stiffness of filling materials. Figure (5.10) shows that the values of maximum bending moment decrease by about (70%, 45% and 38%) when using sand lime, loam and hollow cement bricks in case of (N=10).

In case of (N=15) the maximum bending moments decrease by about (65%,36% and 25%) when using sand lime, loam and hollow cement bricks as shown in fig. (5.11).

The maximum bending moments decrease by about (55%, 35% and 20%) when using sand lime, loam and hollow cement bricks when (N=20) as shown in figure (5.12).

b- Interior beams:

Figures (5.13) to (5.15) represent the relation between bending moment in interior beams and storey number for different filling materials.

It can be noticed that the maximum bending moments decrease by about (70%, 50% and 42%) if sand lime, loam and hollow cement bricks are used respectively as shown in fig. (5.13) in case of (N=10).

Figure (5.14) shows that the maximum bending moments decrease by about (67%, 40% and 27%) when using sand lime, loam and hollow cement bricks in case of (N=15).

In case of (N=20) the values of maximum bending moment decrease by about (67%, 40% and 28%) when using sand lime, loam and hollow cement bricks respectively as shown in fig. (5.15).

5.2.4 Bending moment in shear walls

It can be noticed that the values of maximum bending moment in shear walls decrease by the increase of Young's modulus and the decrease of Poisson's ratio of the filling materails as shown in table (5.1). For instance the maximum bending moment in shear walls decrease by about (56%, 36% and 30%) when N=10, about (48%, 34% and 22%) when N=15, and decrease by about (48%, 28% and 18%) when N=20 if sand lime, loam and hollow cement bricks are used respectively.

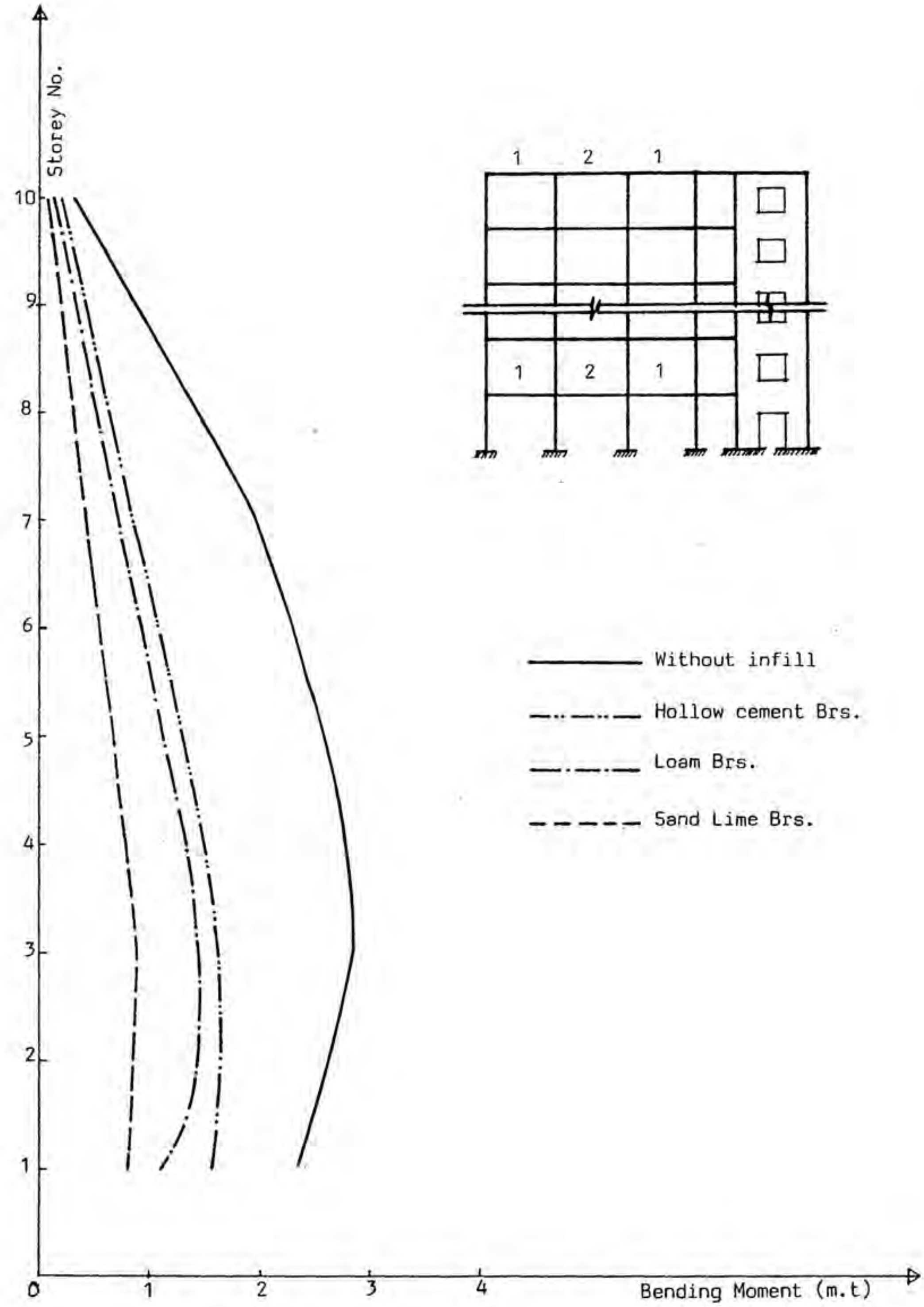


Fig. (5.10) Effect of infill on Bending Moment in an Exterior Beams (No. 1) at Each storey level (Total No. of stores N=10).

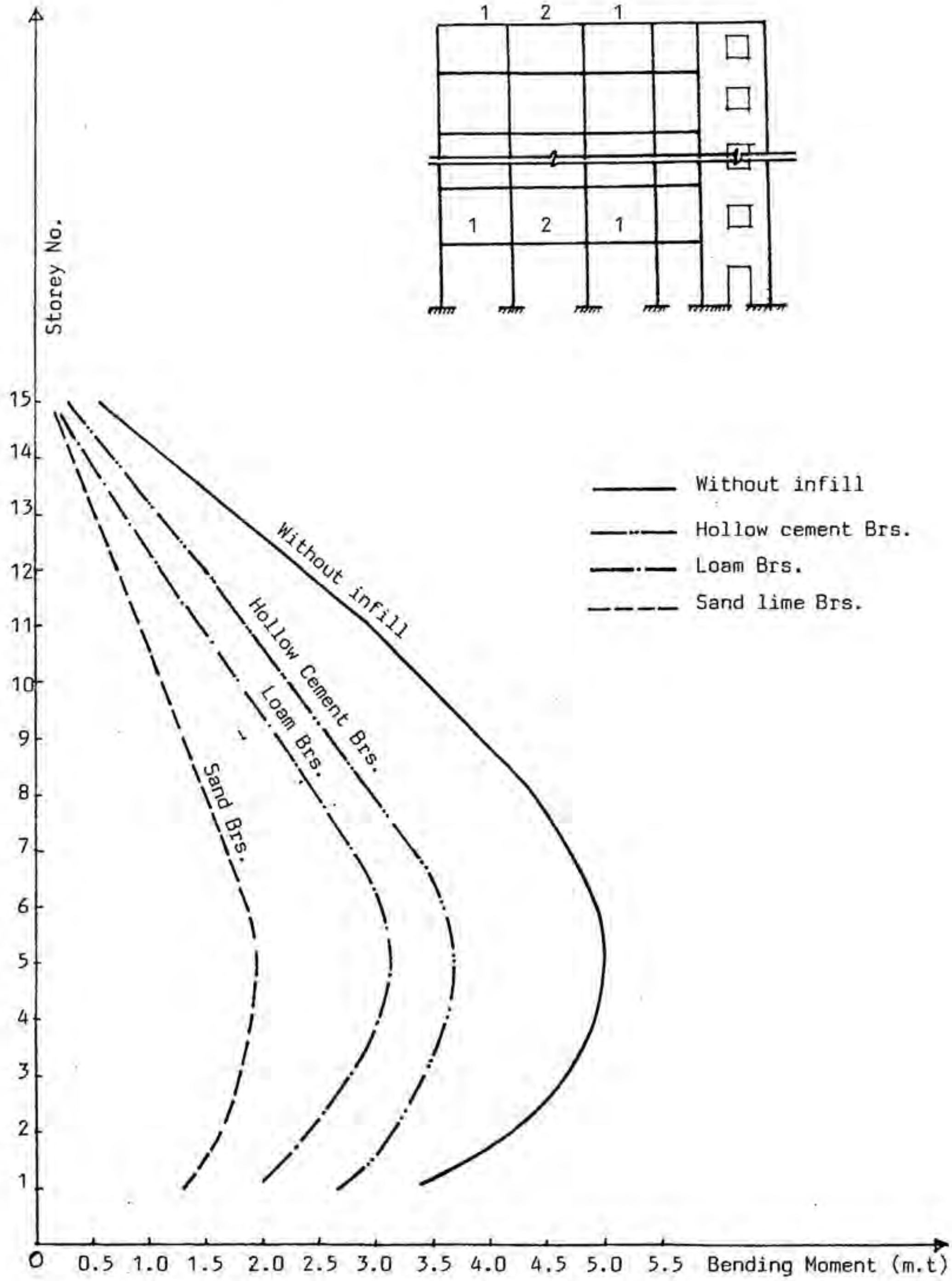


Fig. (5.11) Effect of infill on Bending Moment in an Exterior Beams (No.1) at Each storey level(Total No. of stories N = 15).

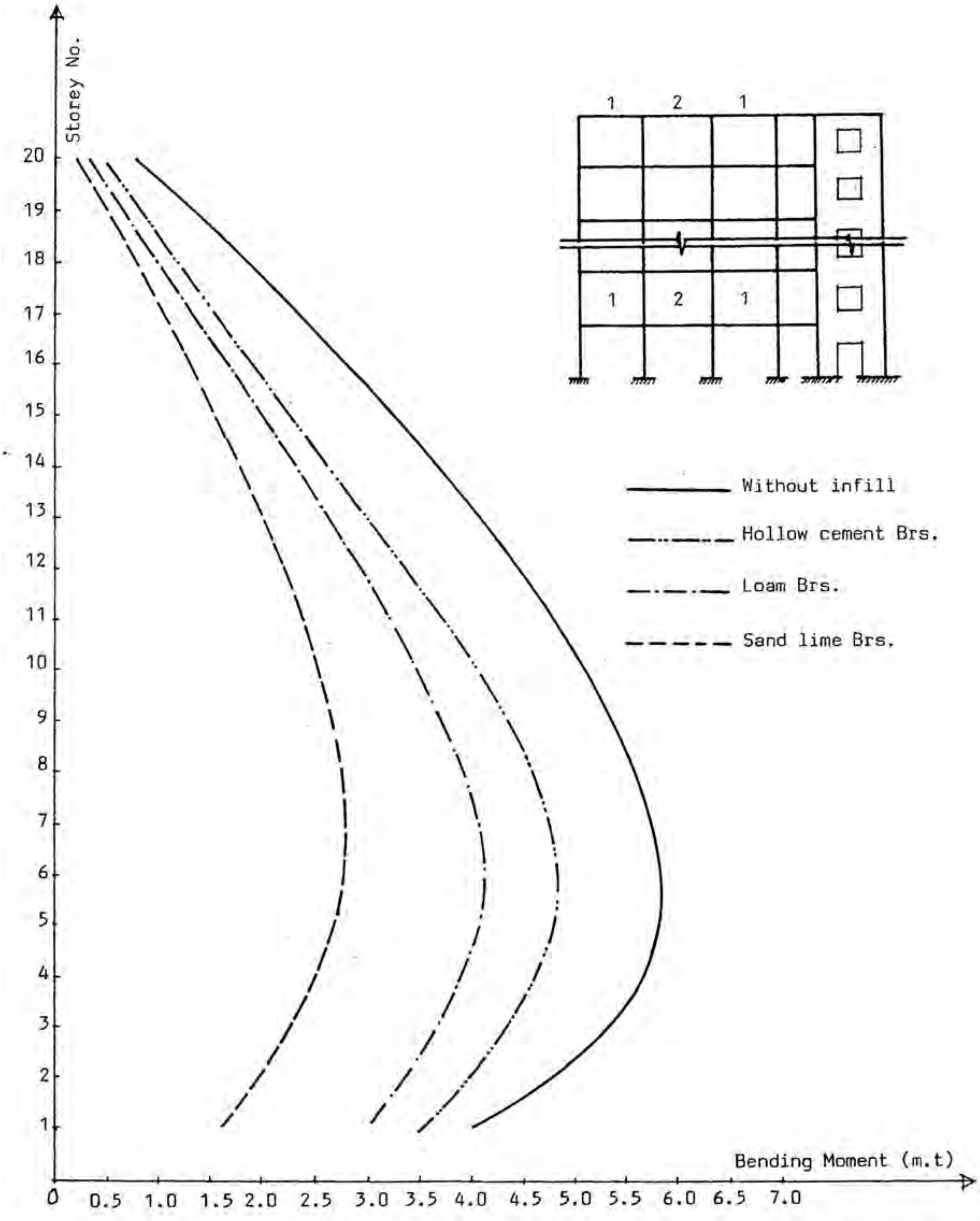


Fig. (5.12) Effect of infill on Bending Moment in an Exterior Beams (No.1) at Each storey level (Total No. of stories N = 20).

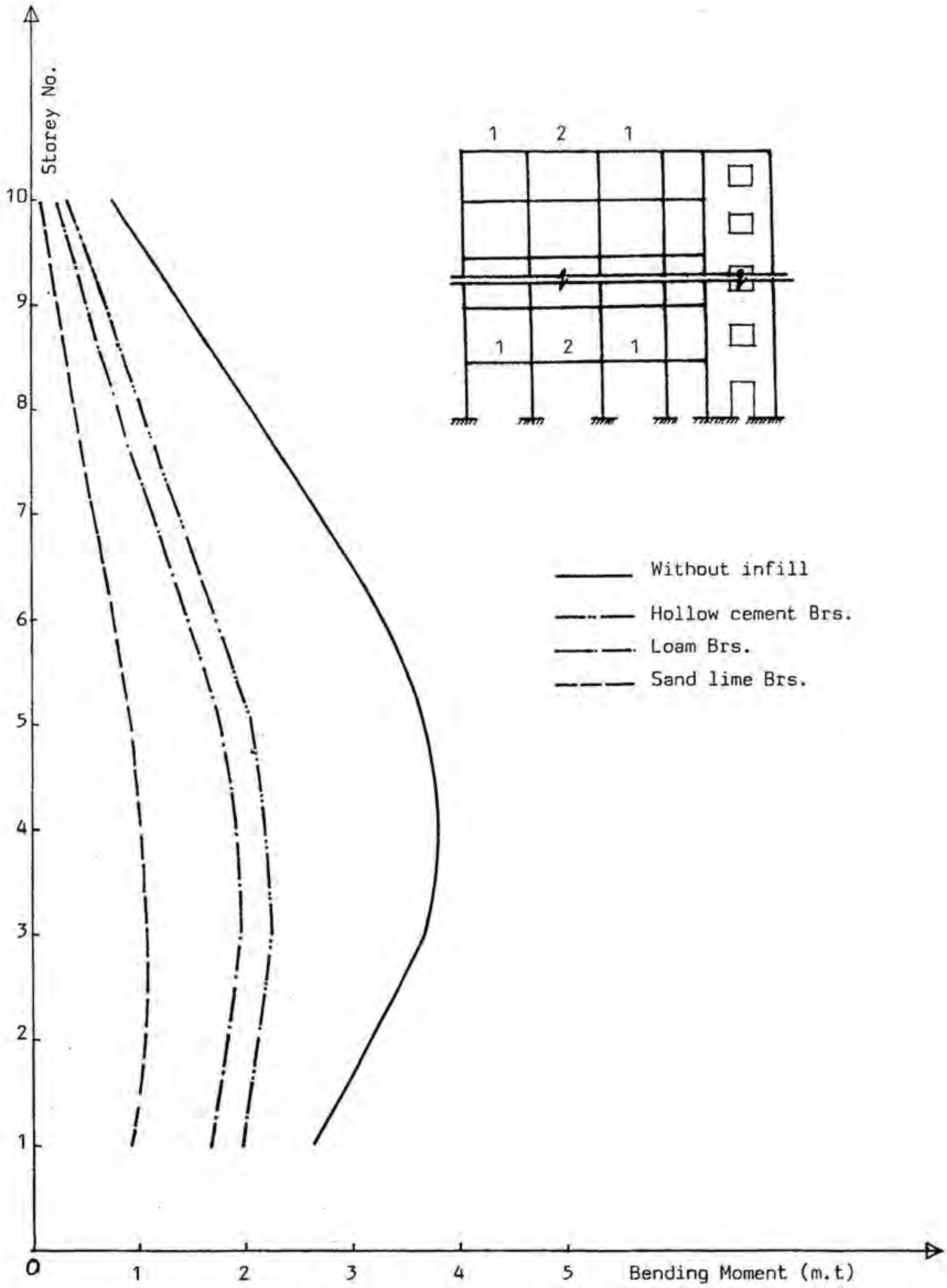


Fig. (5.13) Effect of infill on Bending Moment in an Interior Beams (No. 2) at Each Storey level(Total No.of stories N=10).

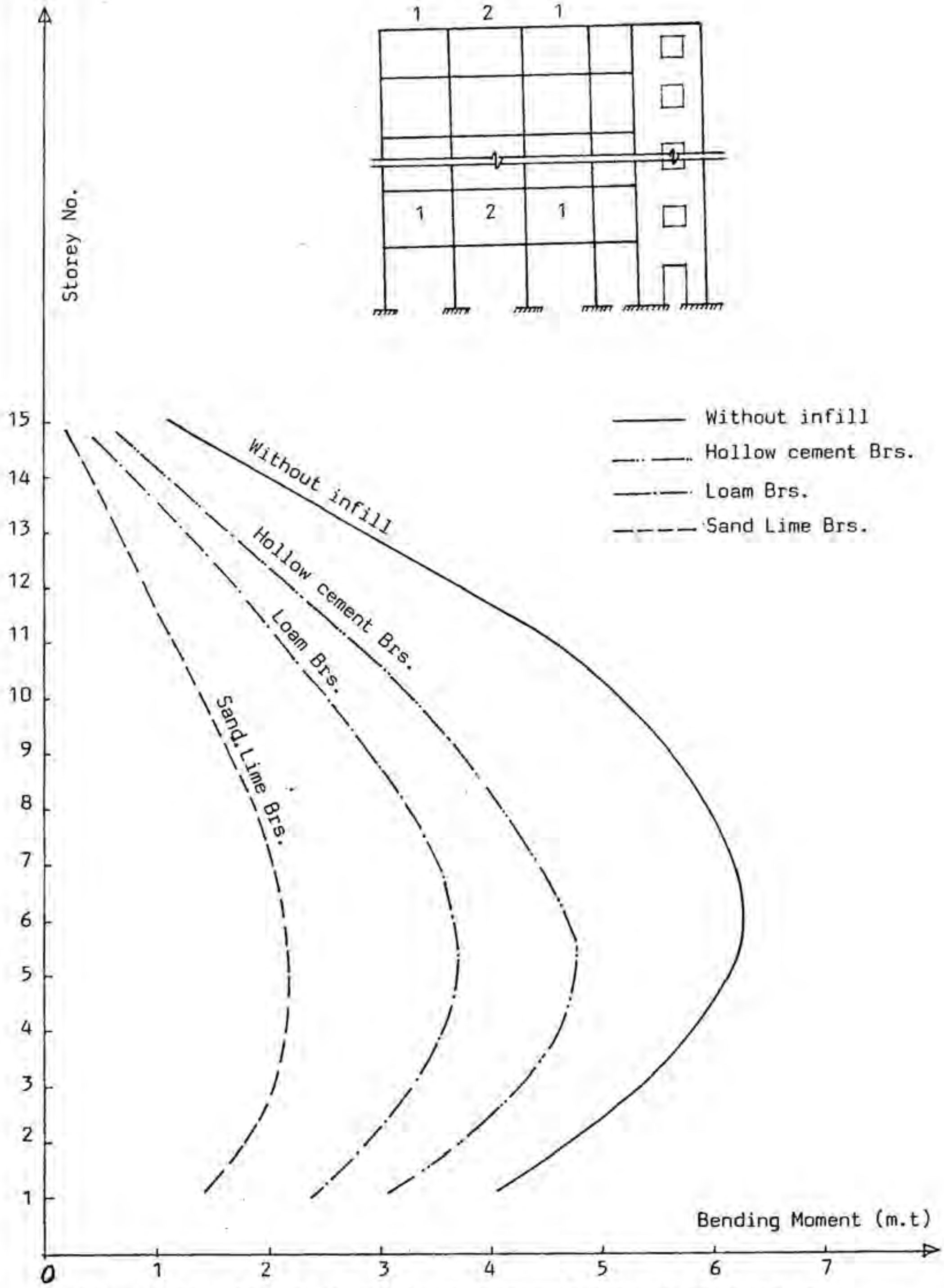


Fig. (5.14) Effect of infill on Bending Moment in an Interior Beams (No. 2) at Each storey level(Total No.stories N=15).

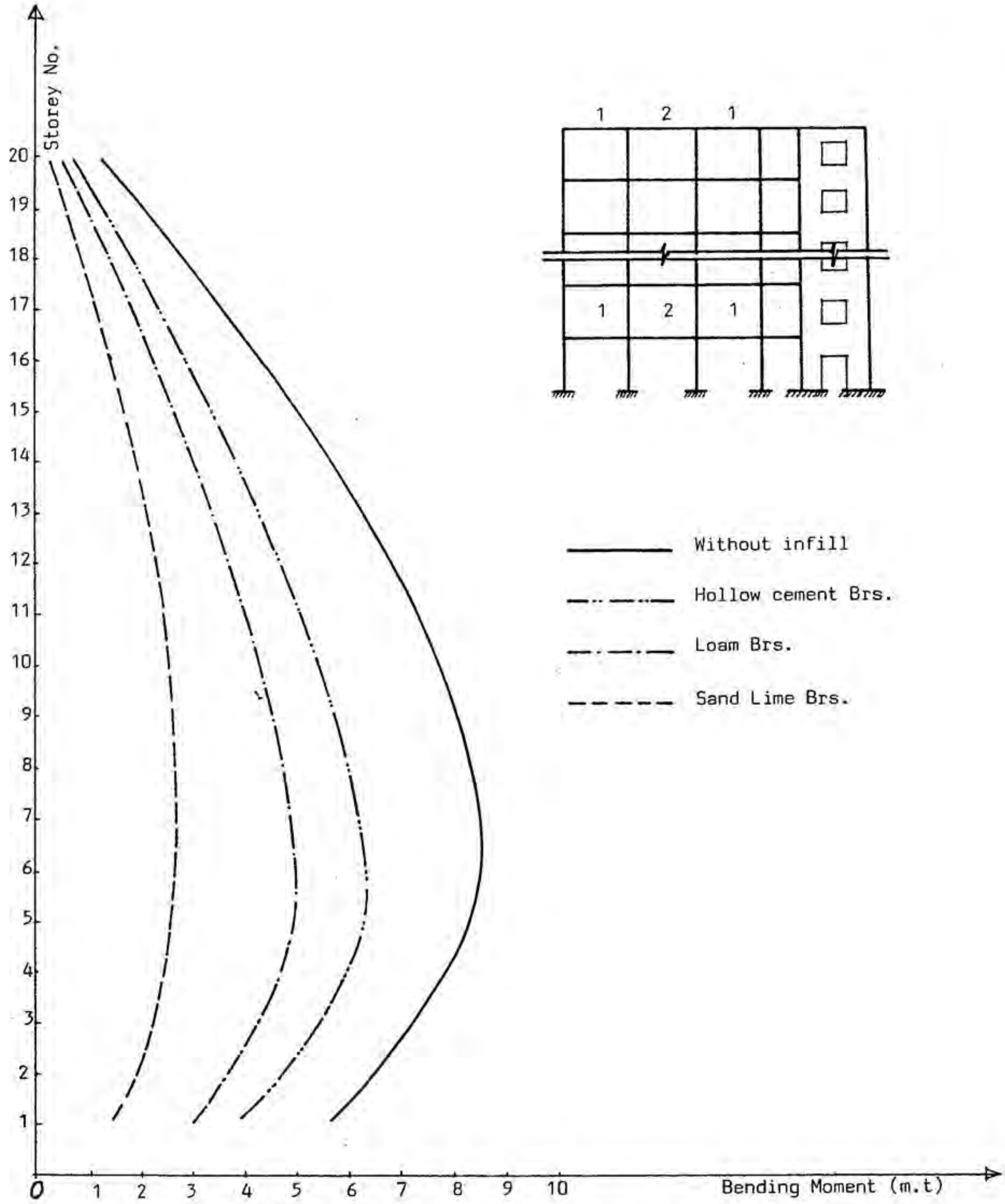


Fig. (5.15) Effect of infill on Bending Moment in on Interior Beams (No. 2) at Each storey level (Total No. of Stories N = 20).

5.2.5 Normal forces in columns

a- Exterior columns:

Table (5.1) shows that the values of maximum normal force increases when the stiffness of the filling material increases. This increase is compression in columns at Lea ward side and tension in columns at wind ward side. For instance the maximum normal force increases by about (78%, 54% and 42%) when N=10, about (76%, 50% and 36%) when N=15 and by about (58%, 48% and 35%) when N=20 if sand lime, loam and hollow cement bricks are used respectively.

b- Interior columns:

Table (5.1) also shows that maximum normal force increases by about (140%, 100% and 75%) if N=10, about (120%, 80% and 65%) if N=15, and by about (110%, 75% and 60%) if N=20 when using sand lime, loam and hollow cement bricks respectively.

5.3 Effect of Infill Panel Thickness

To illustrate the effect of infill panel thickness on the lateral deflection of the structure and on the straining actions in the beams, columns and shear walls, two different thickness were considered, namely one birck (25 cm.) and half a brick (12 cm.).

5.3.1 Lateral deflection of structure

Figure (5.16) presents the relation between the lateral deflection of the structure and the thickness of the panel for sand lime, loam and hollow cement bricks.

It can be noticed that the maximum values of lateral deflection increases with the decrease of the thickness of infill panel. In case of half a brick the lateral deflection is more than that of one brick by about (40%, 38% and 35%) by using sand lime, loam and hollow cement bricks respectively for a ten storey building.

For a fifteen storey building the lateral deflection of half a brick is more than that of one brick if sand lime, loam and hollow cement bricks are used by about (38%, 36% and 32%) respectively.

5.3.2 Bending moments in the columns of the building

The relation between maximum bending moment and the thickness of the infill for sand lime, loam and hollow cement bricks is illustrated in figure (5.17). The values of maximum bending moment decreases with the increase of the thickness of infill panel.

a- Exterior columns:

In case of N=10, the maximum values of bending moment in exterior columns having half a brick infill is more than that of one brick infill of sand lime, loam and hollow cement bricks by about (45%, 30% and 22%) respectively.

In case of N=15, the maximum values of bending moment in exterior columns having half a brick infill increase by about (38%, 30% and 22%) than that of one brick thickness of sand lime, loam and hollow cement bricks respectively, as shown in fig. (5.17).

b- Interior columns:

The maximum bending moment in interior columns having half a brick infill increases by about (52%,34% and 24%) than that of one brick thickness for sand lime, loam and hollow cement bricks respectively in case of N=10.

The maximum bending moment in interior columns having half a brick infill increases by about (38%, 32% and 20%) than that having one brick thickness for sand lime, loam and hollow cement bricks respectively in case of N=15, as shown in fig. (5.18).

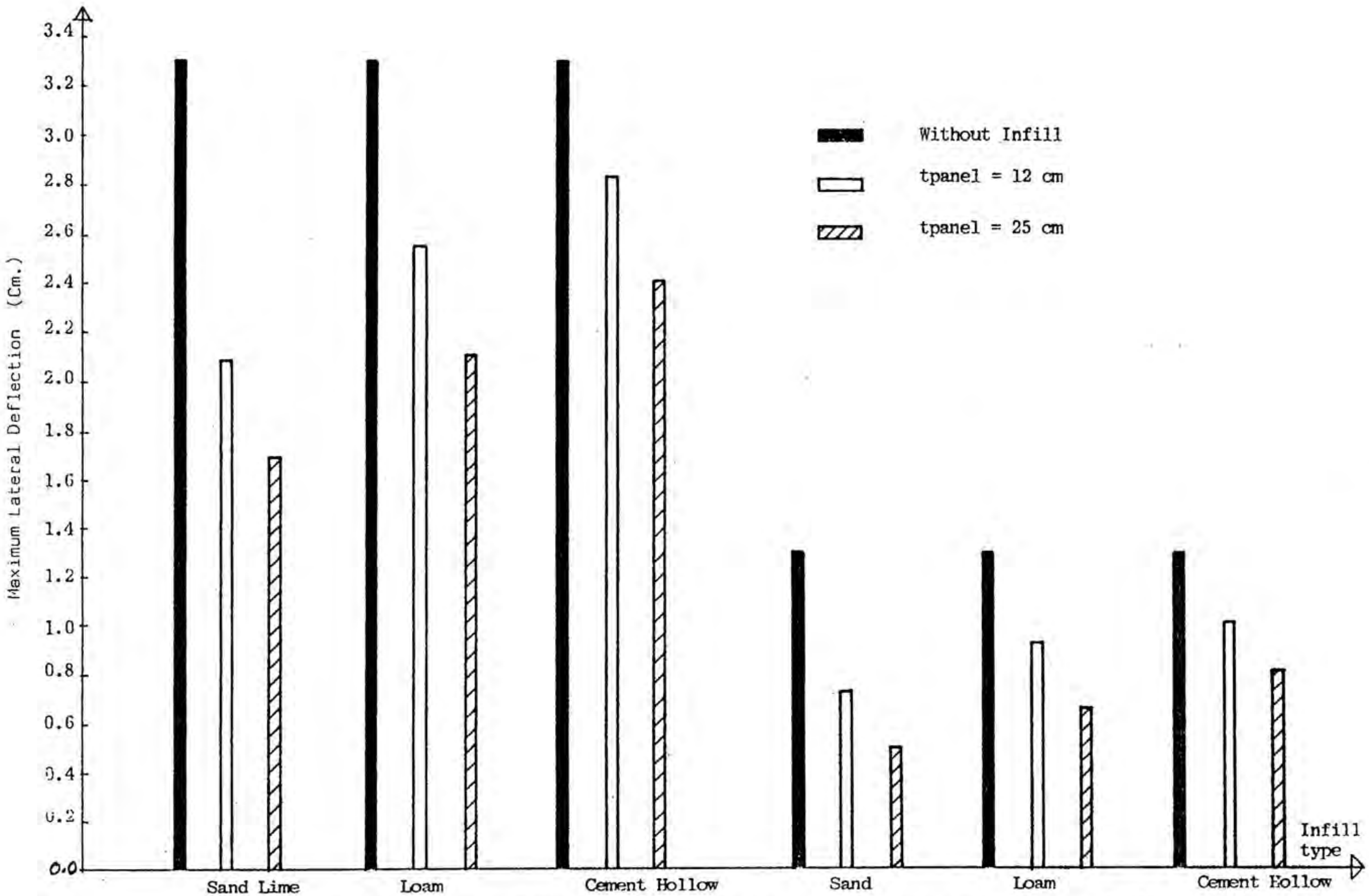


Fig. (5.16): Maximum Lateral Deflection of structure For various panel thickness (tp).

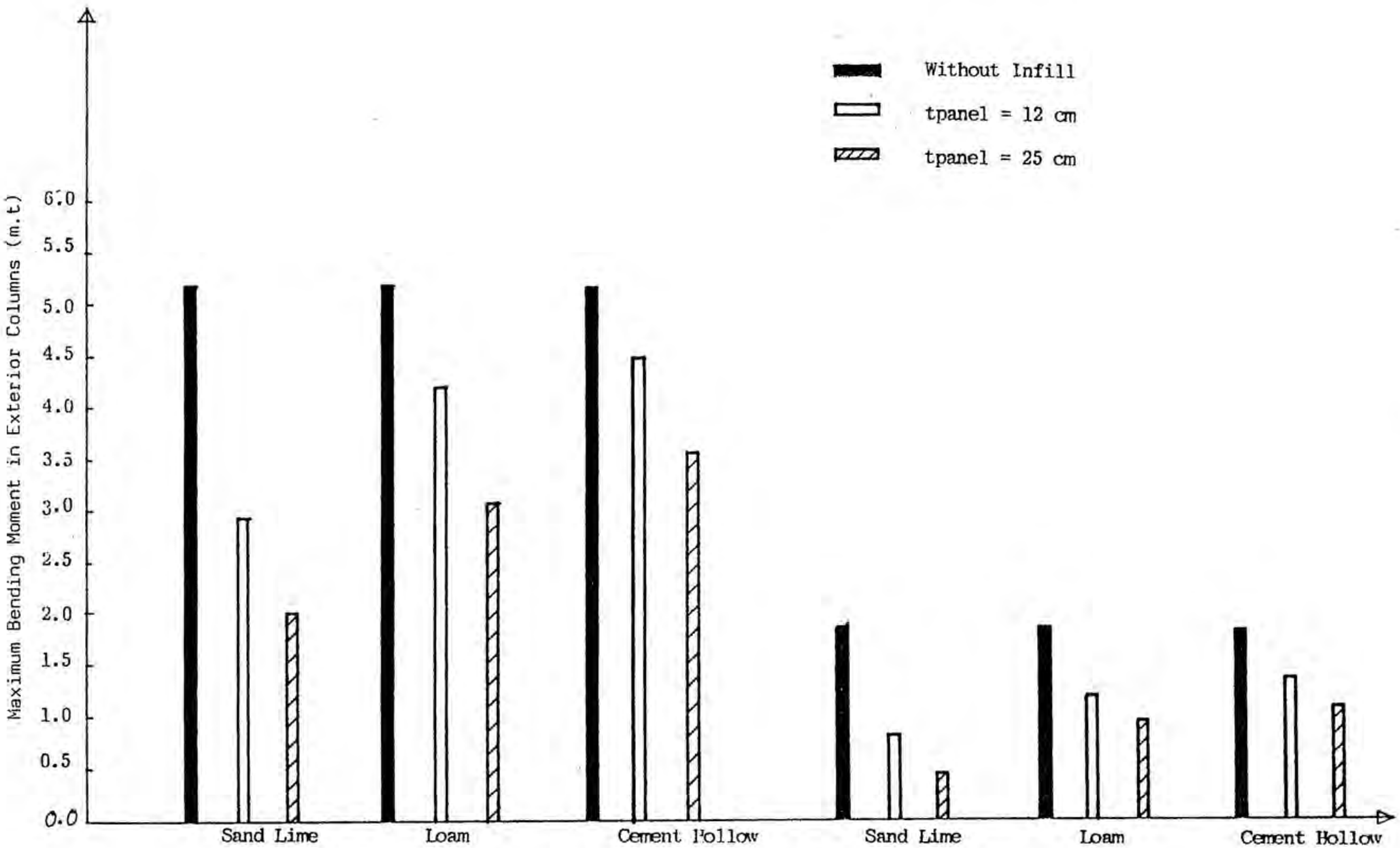


Fig. (5.17): Effect of Panel thickness tp on the Maximum Bending Moment in Exterior Columns (N = 15 & 10).

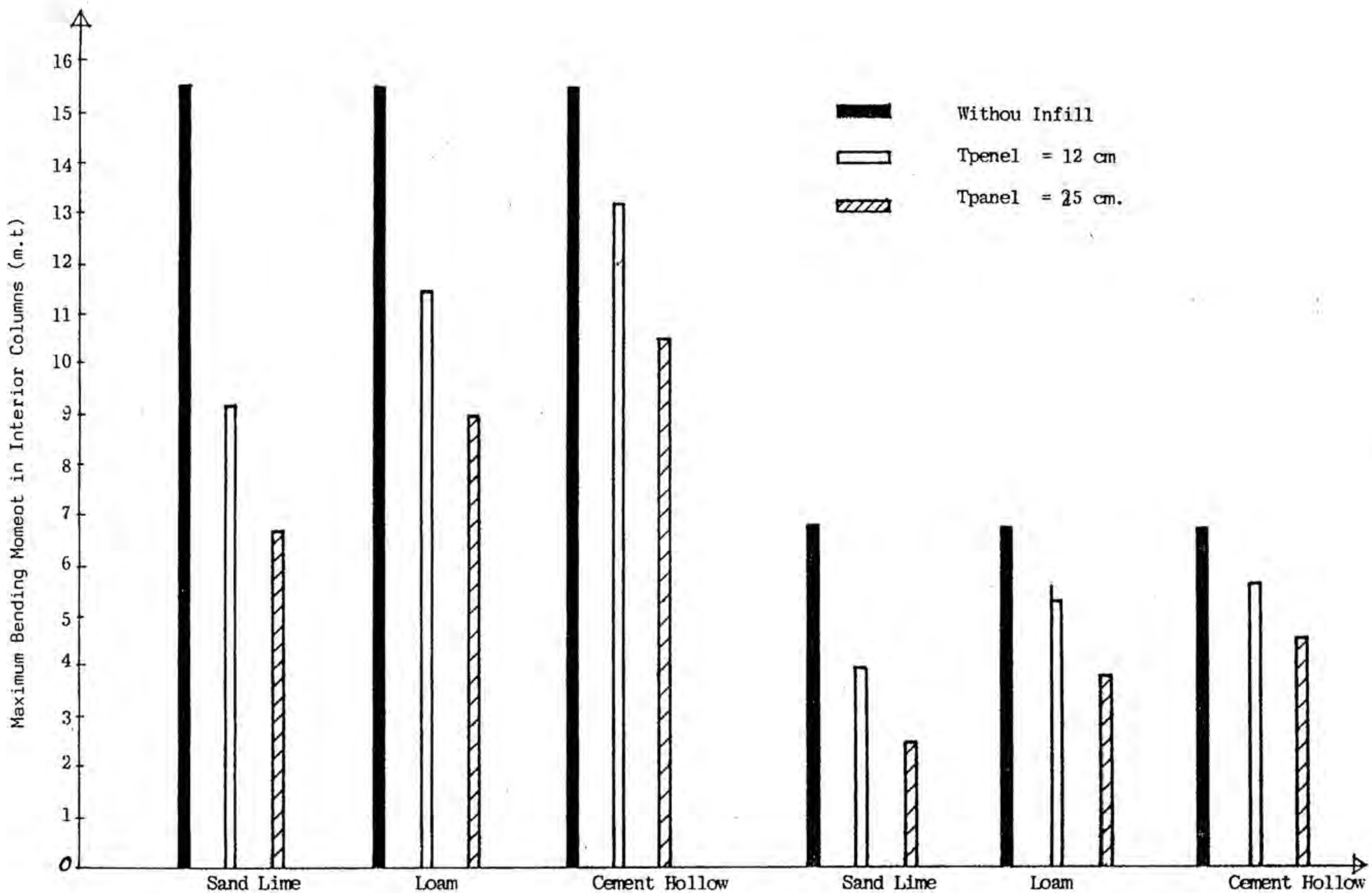


Fig. (5.18): Effect of panel thickness tp on the Maximum Bending Moment in Interior Columns (N = 15 & 10).

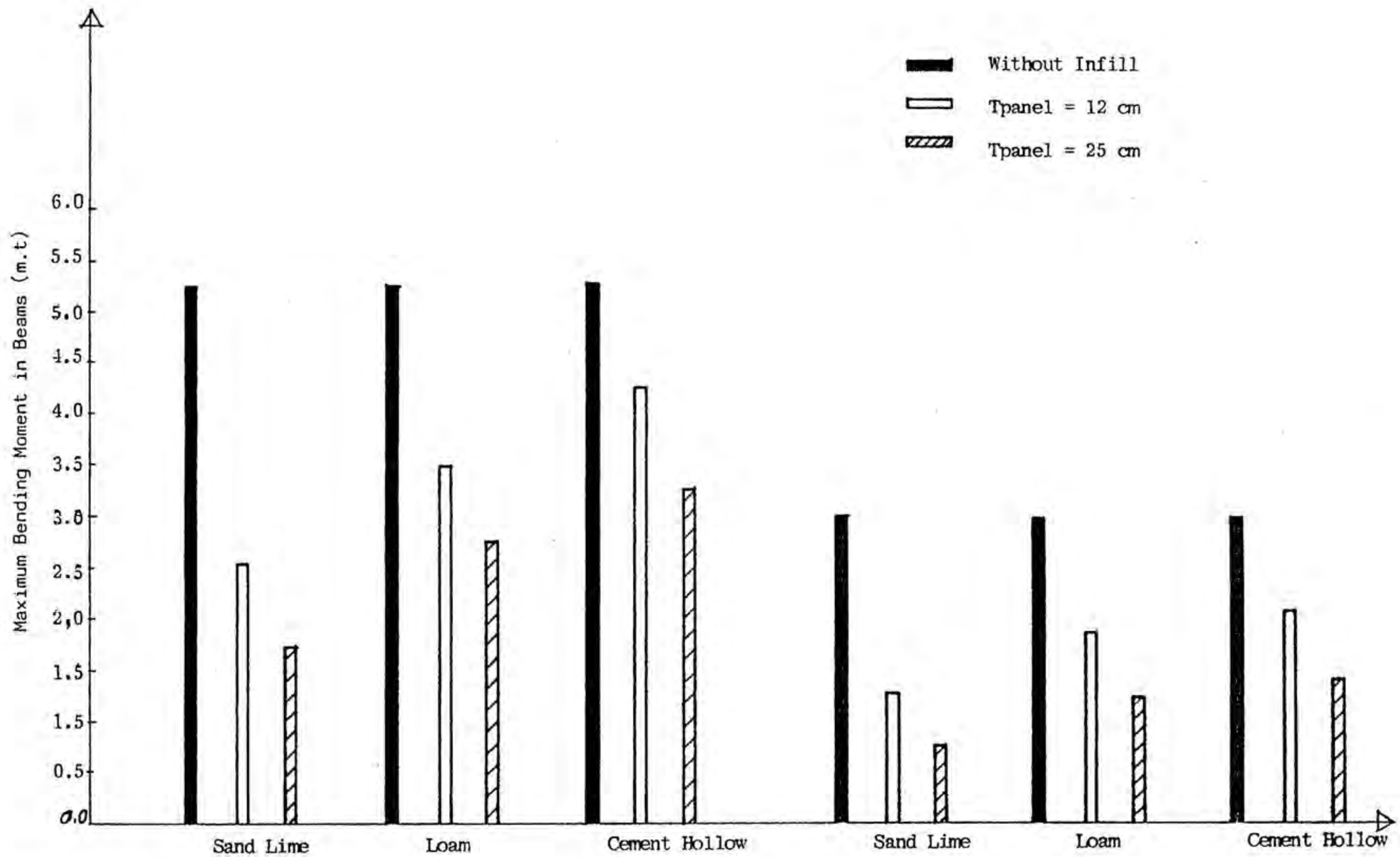


Fig. (5.19: Effect of panel thickness tp on the Maximum Bending Moment In Exterior Beams (N= 15 & 10).

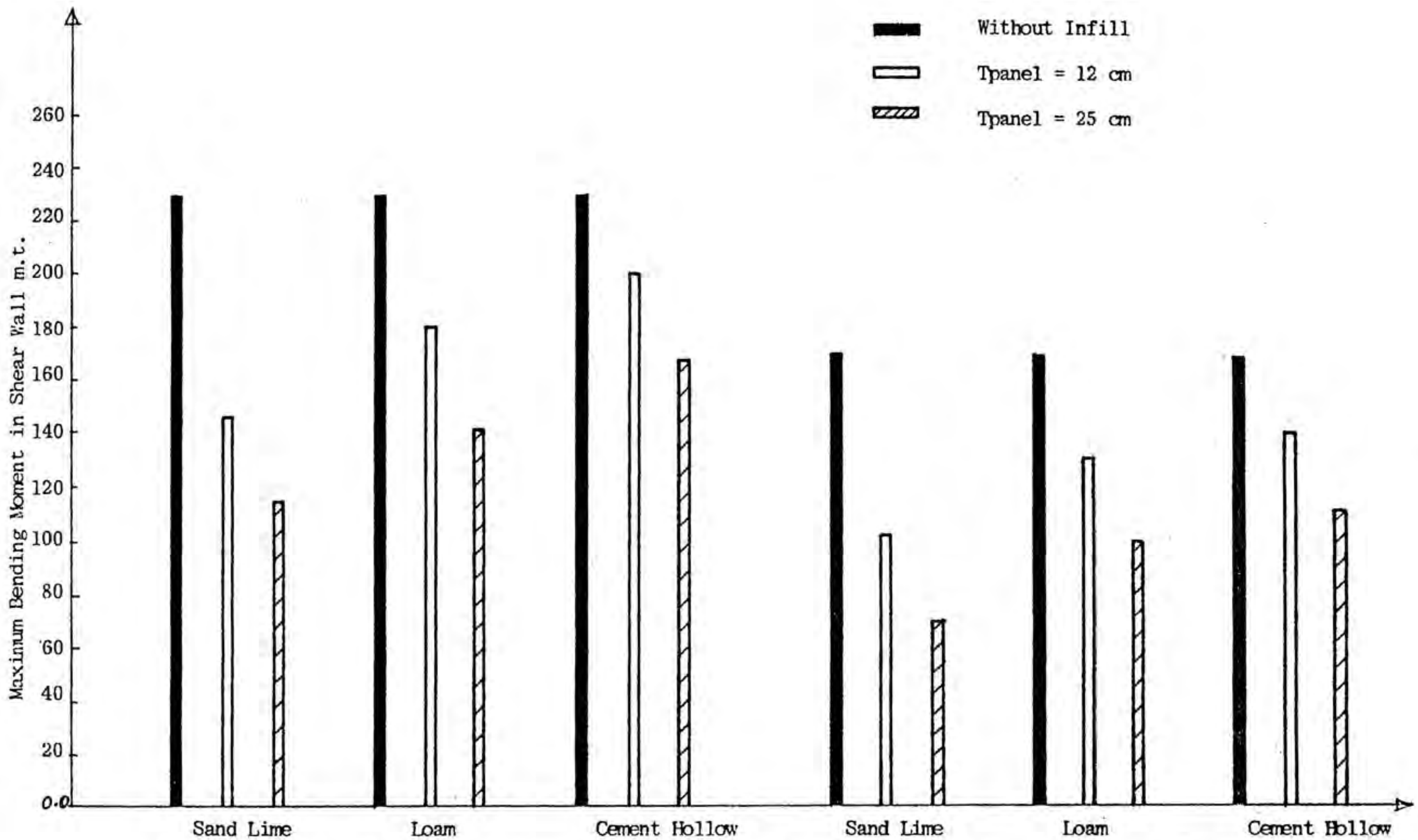


Fig. (5.20): Effect of panel thickness tp on the Maximum Bending Moment In shear wall(N = 15 & 10).

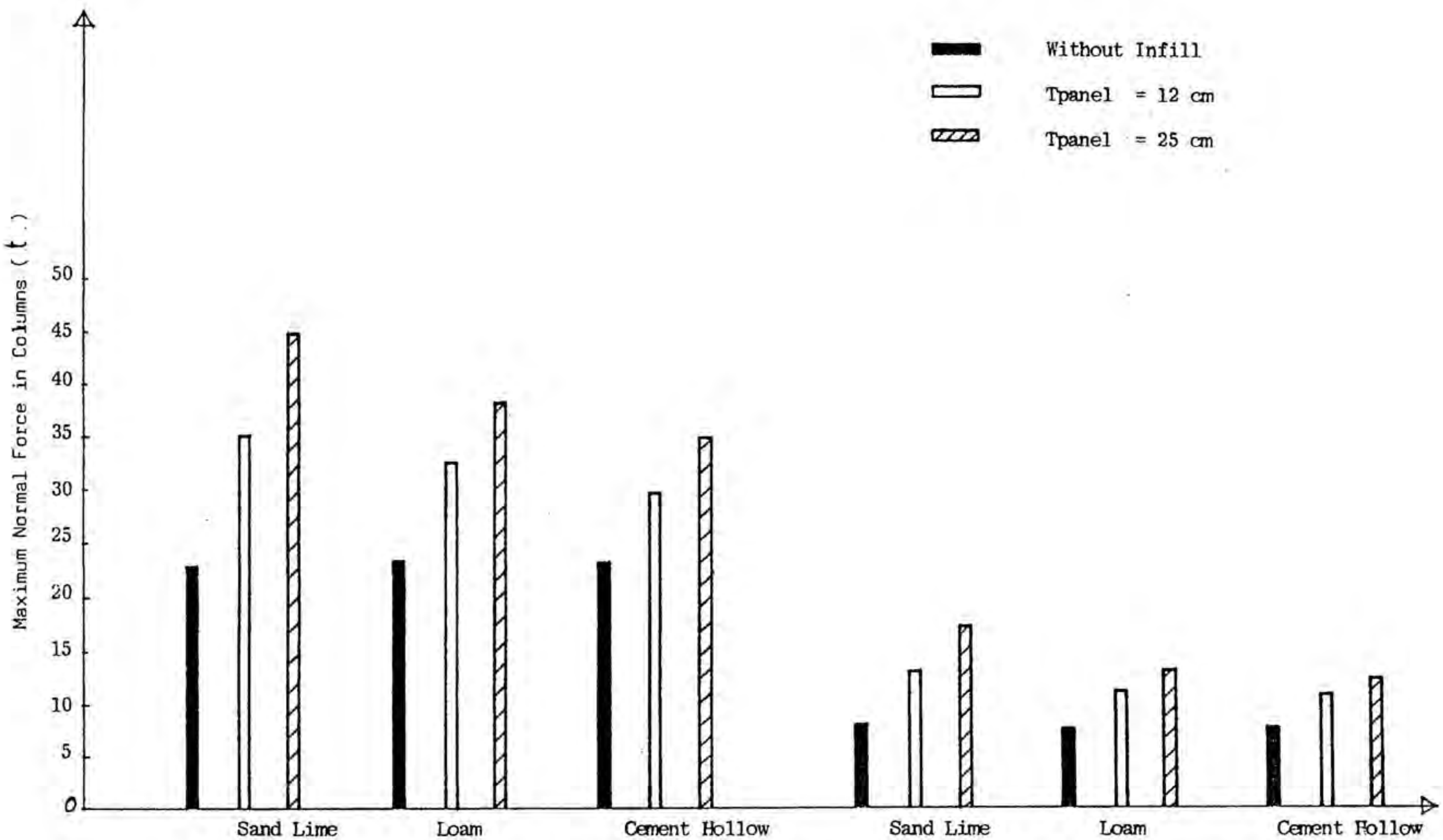


Fig. (5.21): Maximum Normal force In Columns for Various Panel thickness tp (N = 15 & 10).

half a brick decreases by about (28%, 20% and 16%) than that for one brick thickness for sand lime, loam and hollow cement bricks respectively for ten storey building. The maximum normal force either in exterior or interior columns having half a brick decreases by about (26%, 18% and 15%) than that of one brick thickness for sand lime, loam and hollow cement bricks respectively in fifteen storey building.

5.4 Effect of Change of Infill Stiffness Between Different Stories

In this research a twenty storey building was chosen, where the lower stories (10,8,6 and 4) filled with sand lime bricks and the upper stories filled with hollow cement bricks, to illustrate the effect of change of infill stiffness between different stories.

5.4.1 Lateral deflection of structure

Figure (5.22) shows the maximum lateral deflection for all the investigated models. The maximum lateral deflection decreases by about (25%, 24% , 23% and 20%) when filling the lower stories mentioned above with sand lime bricks and the upper stories with hollow cement bricks.

5.4.2 Bending moments in the columns of the building

Figure (5.23) shows the maximum bending moment in exterior or interior columns for all the investigated models. The maximum values of bending moment decrease by about (42%, 41%, 39% and 33%) when filling the lower stories of the building with sand lime bricks and the upper ones with hollow cement bricks.

5.4.3 Bending moments in the beams of the building

The maximum values of bending moment in the exterior or interior beams decrease by about (58%, 47%, 45% and 40%) when filling the lower stories of the building with sand lime bricks and the upper stories with hollow cement bricks as shown in figure (5.24).

5.4.4 Bending moment in shear wall

When the lower stories are filled with sand lime bricks and the upper ones are filled with hollow cement bricks the maximum values of bending moment decrease by about (38%, 35%, 32% and 30%) as shown in figure (5.25).

5.4.5 Normal forces in columns

The maximum values of normal force in exterior or interior columns increase by about (39%,35%, 32% and 30%) when filling the lower stories with sand lime bricks and the upper stories with hollow cement bricks.

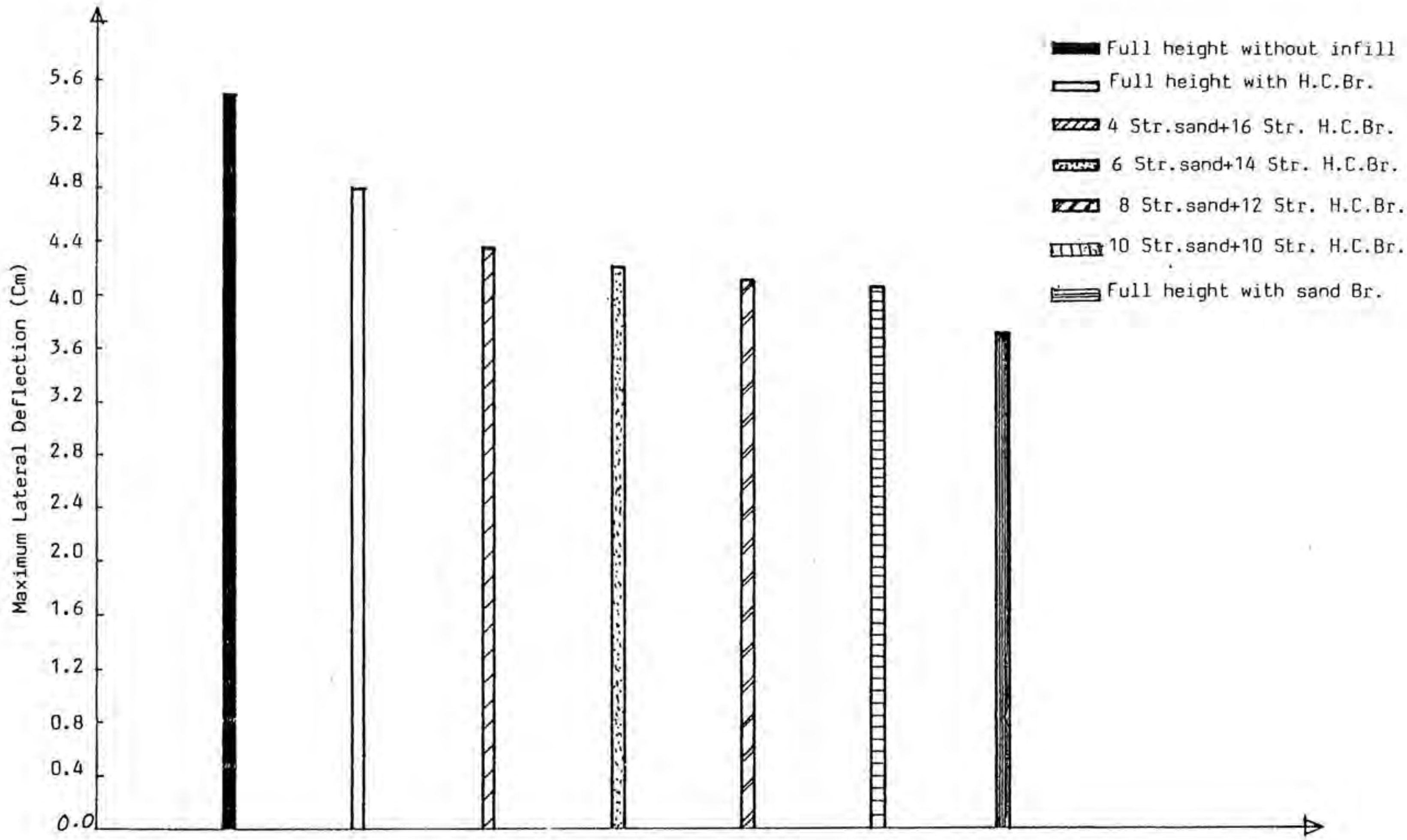


Fig. (5.22) Effect of change of height and type of infill on Maximum Lateral Deflection.

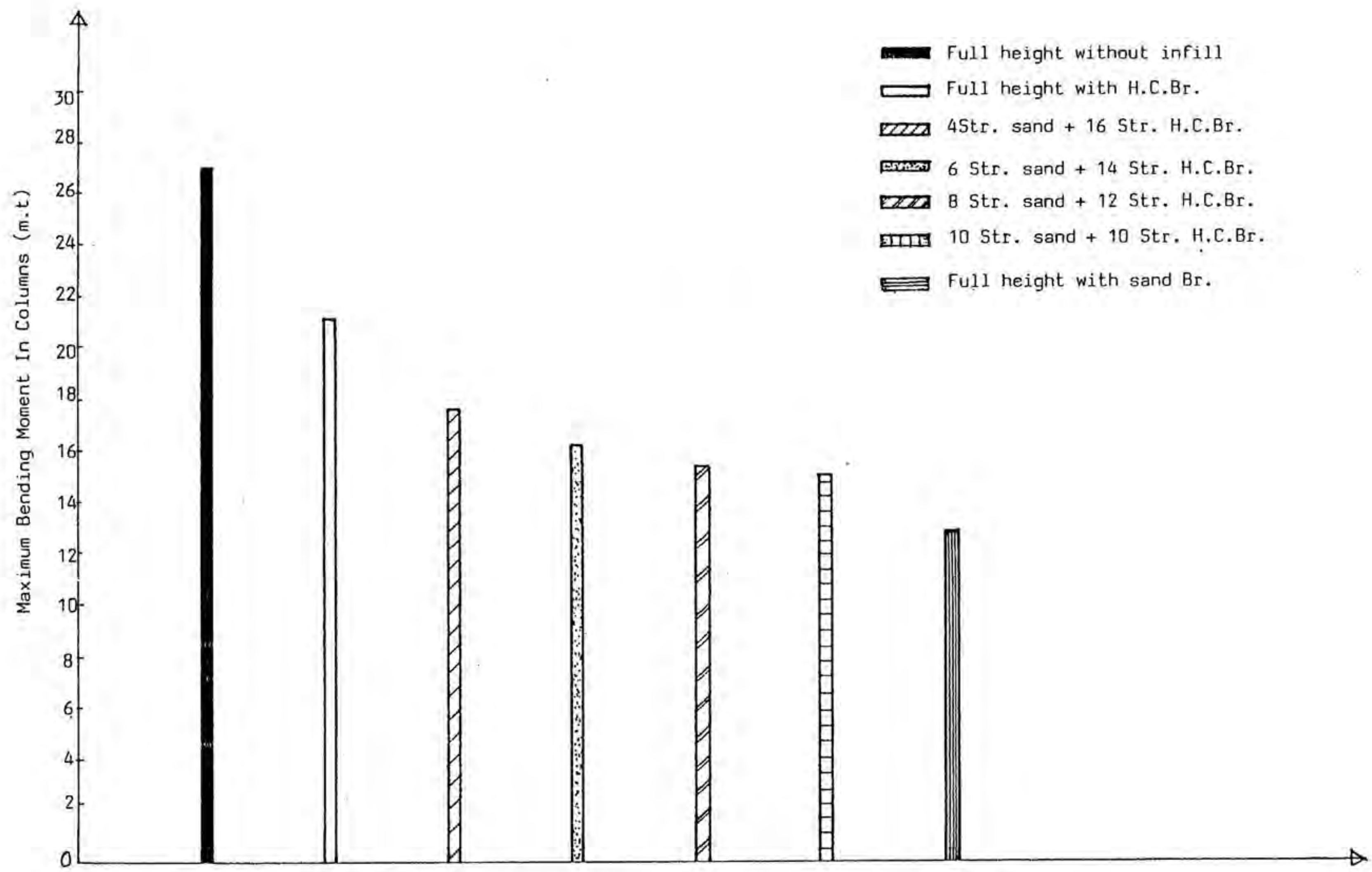


Fig. (5.23) Effect of change of height and type of infill on Maximum Bending Moment In Columns.

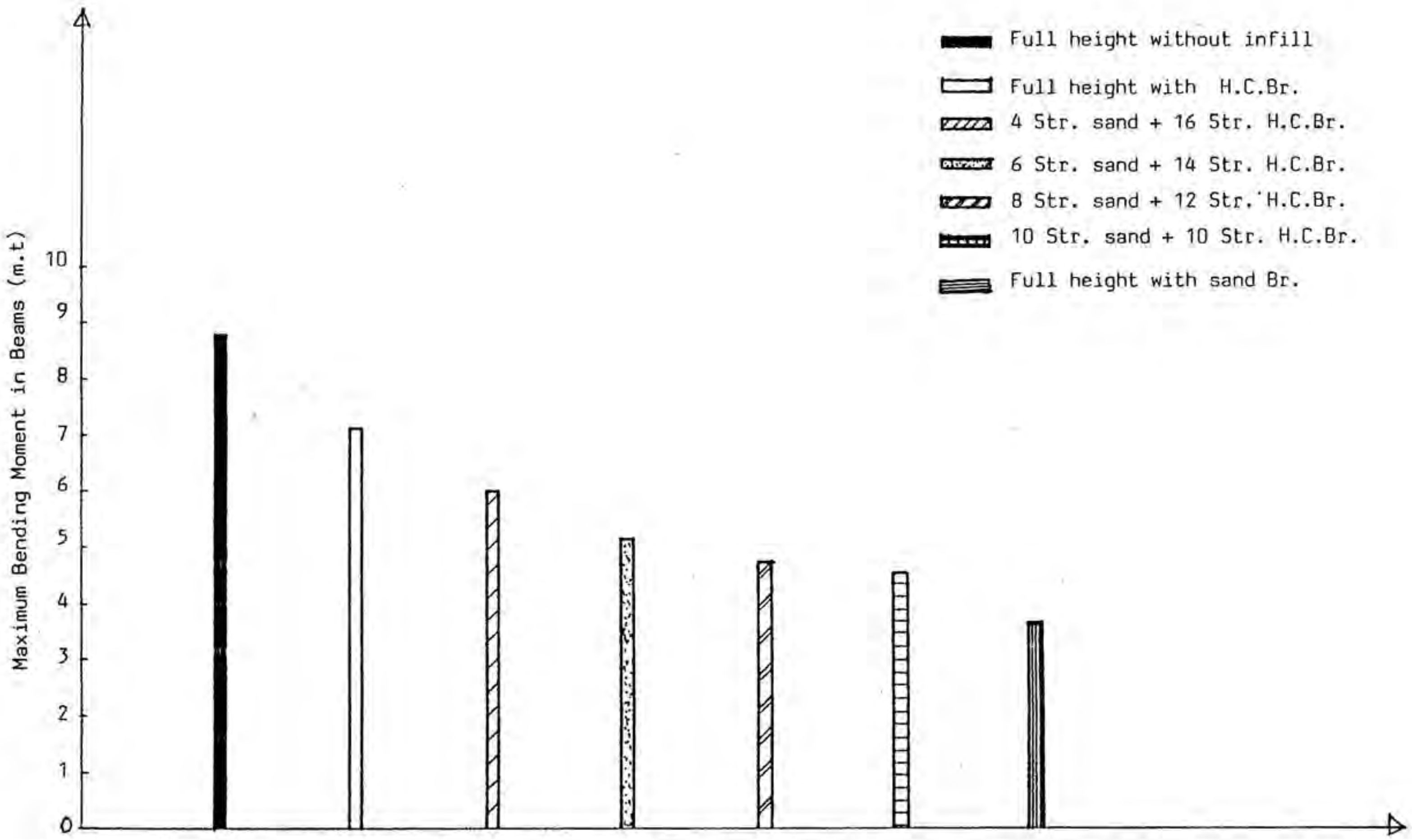


Fig. (5.24) Effect of change of height and type of infill on Maximum Bending Moment In Beams.

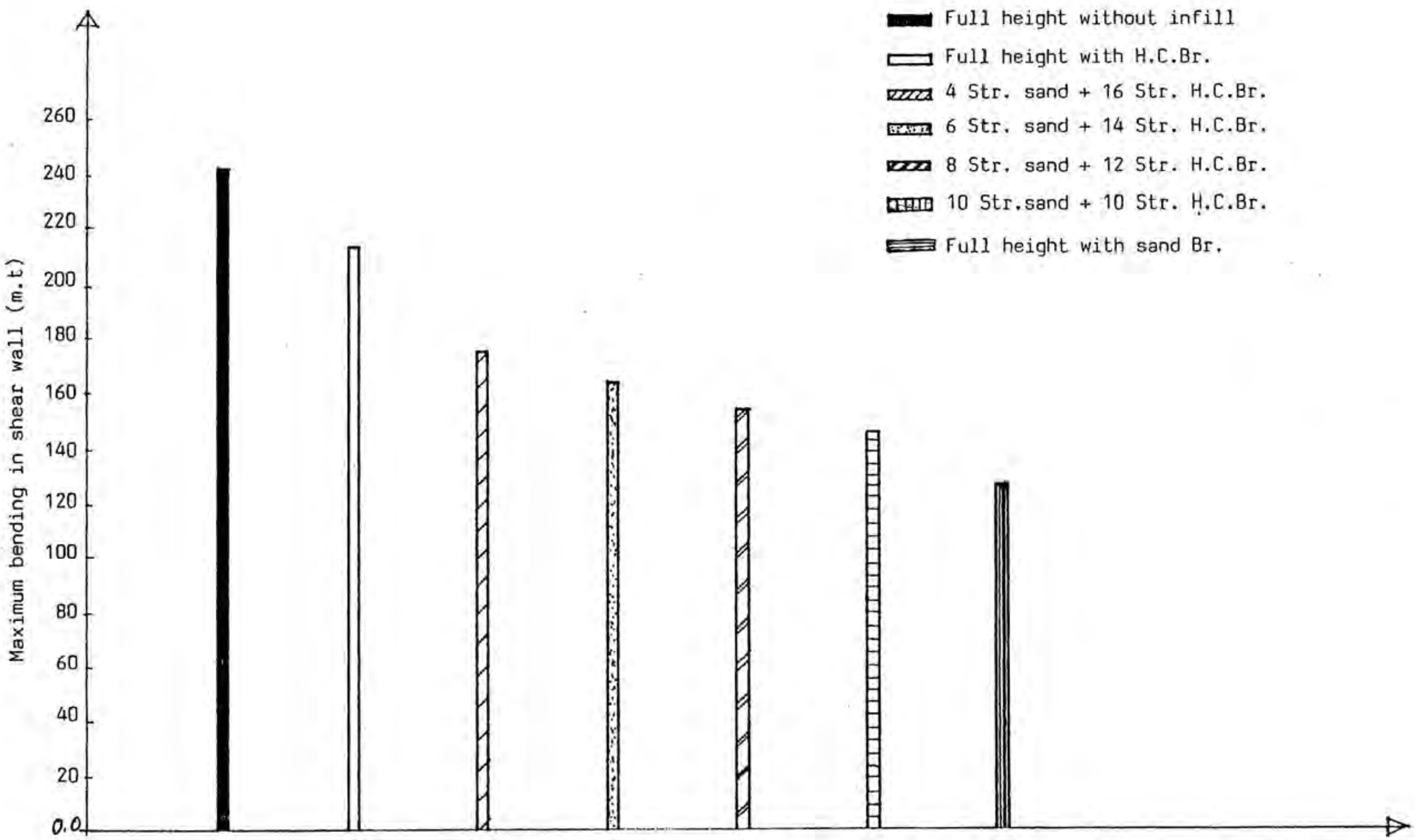


Fig. (5.25) Effect of change of height and type of infill on Maximum Bending Moment in Shear wall.

It can be noticed that there is a small change in the lateral deflection and internal forces when the filling material is sand lime bricks in the lower four to ten stories.

5.5 Effect of Leaving the Lower Stories Without Infill and Filling the Upper Stories with Loam Bricks

A twenty storey building was chosen in this work, where the lower three and five stories left without infill and the upper stories filled with loam bricks, to illustrate its effect.

5.5.1 Lateral deflection of structure

Figure (5.26) shows the maximum lateral deflection for all the investigated models. The maximum lateral deflection decreases by about (18% and 16%) when leaving the lower stories without infill and filling the upper ones with loam bricks.

5.5.2 Bending moments in the columns of the building

The maximum values of bending moment in exterior or interior columns decrease by about (20% and 12%) when leaving the third and fifth lower stories without infill and filling the upper ones with loam bricks as shown in figure (5.27).

5.5.3 Bending moments in the beams of the building

The maximum values of bening moment in exterior or interior beams decreases by about (28% and 18%) when the third and fifth lower stories are left without infill and the upper stories filled with loam bricks, figure (5.28).

5.5.4 Bending moment in shear wall

Figure (5.29) indicates that the maximum values of bending moment decrease by about (10% and 6%) when leaving the same lower stories without infill and filling the above stories with loam bricks.

5.5.5 Normal forces in columns

The maximum values of normal forces in exterior or interior columns increase by about (34% and 26%) when leaving the third and fifth lower stories without infill and filling the upper ones with loam bricks.

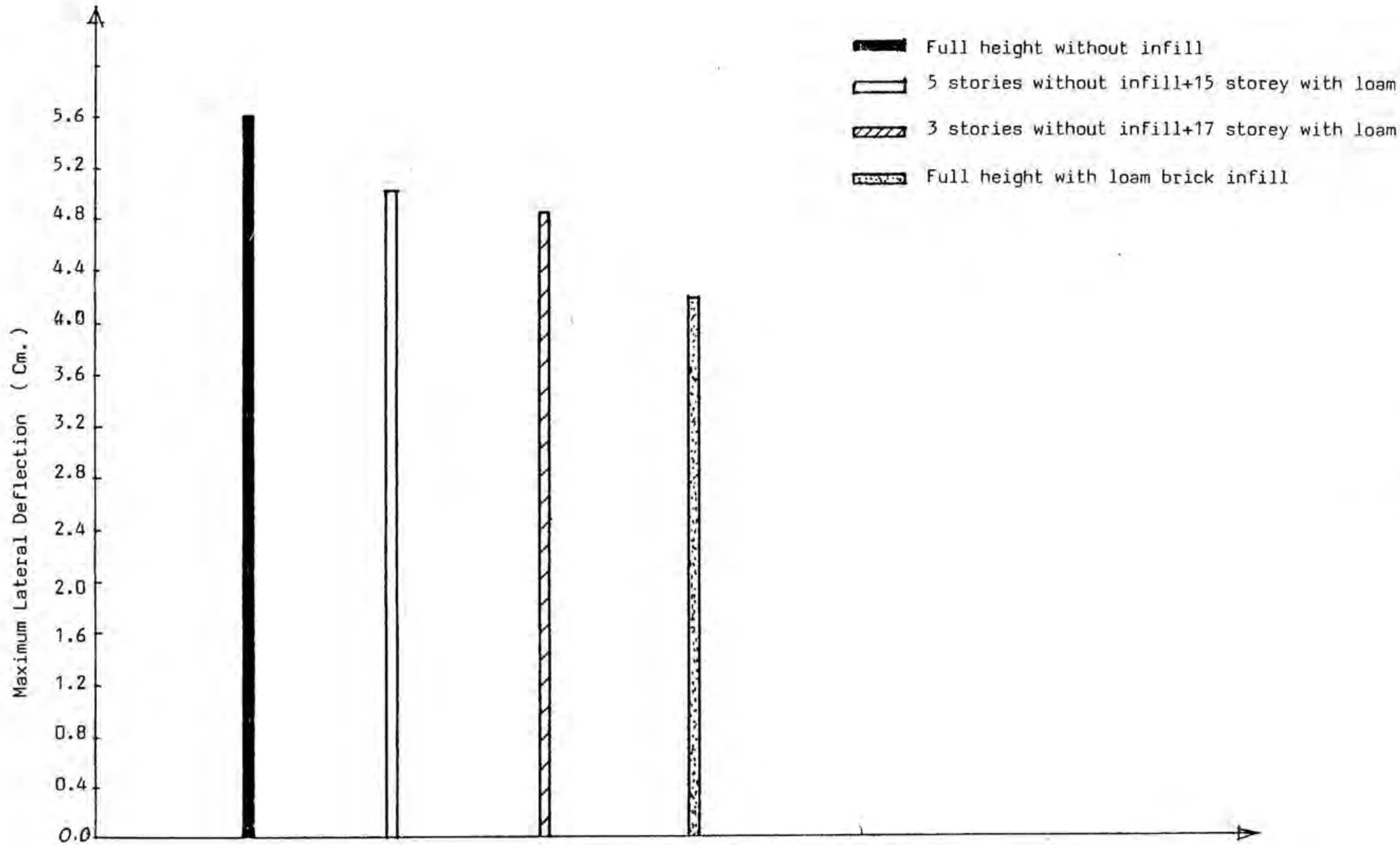


Fig. (5.26) Effect leaving the lower stories without infill and filling the upper stories with loam bricks infill on Maximum Lateral Deflertion.

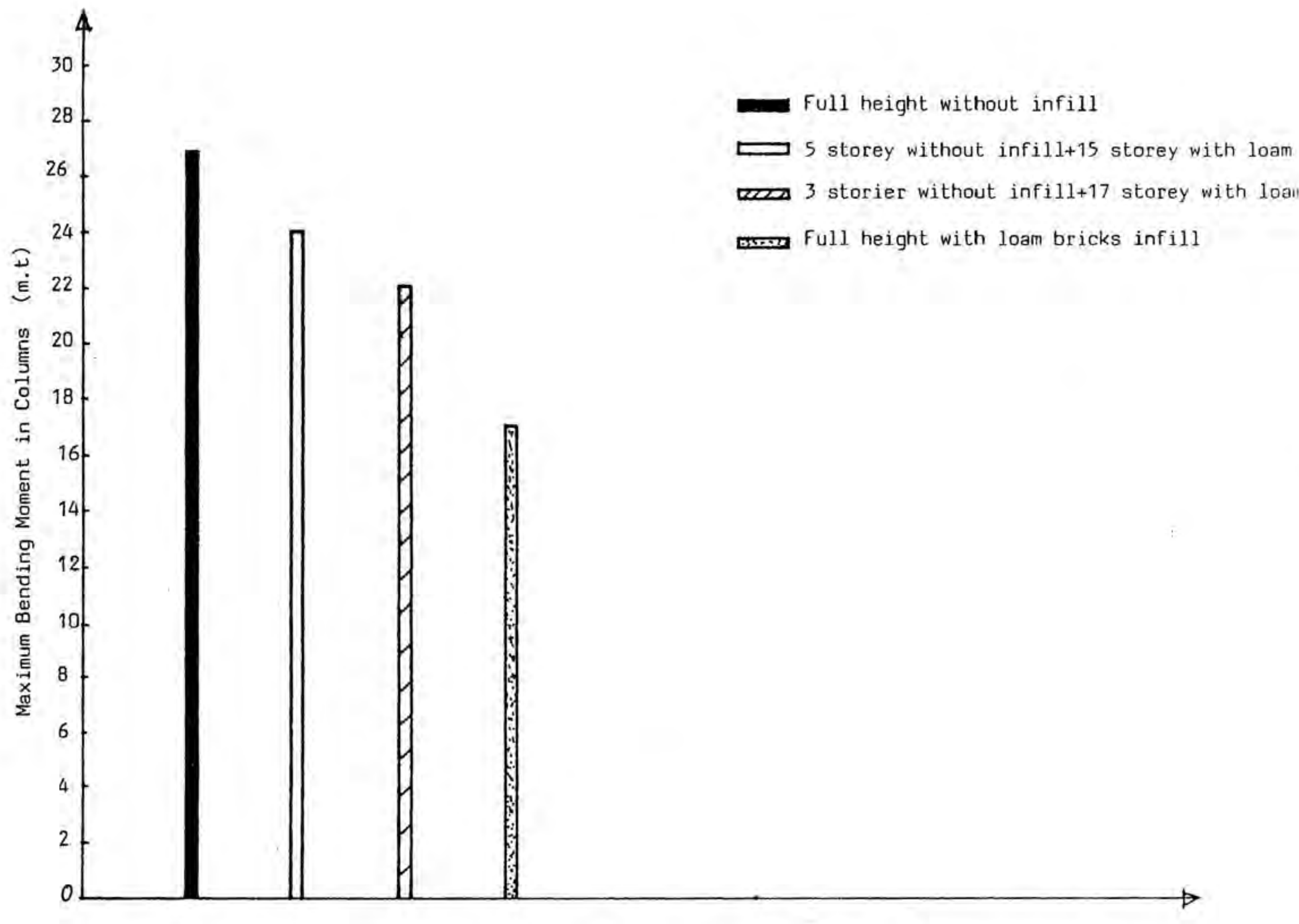


Fig. (5.27) Effect of leaving the lower stories without infill and filling the upper stories with Loam bricks infill on Maximum Bending Moment In Columns.

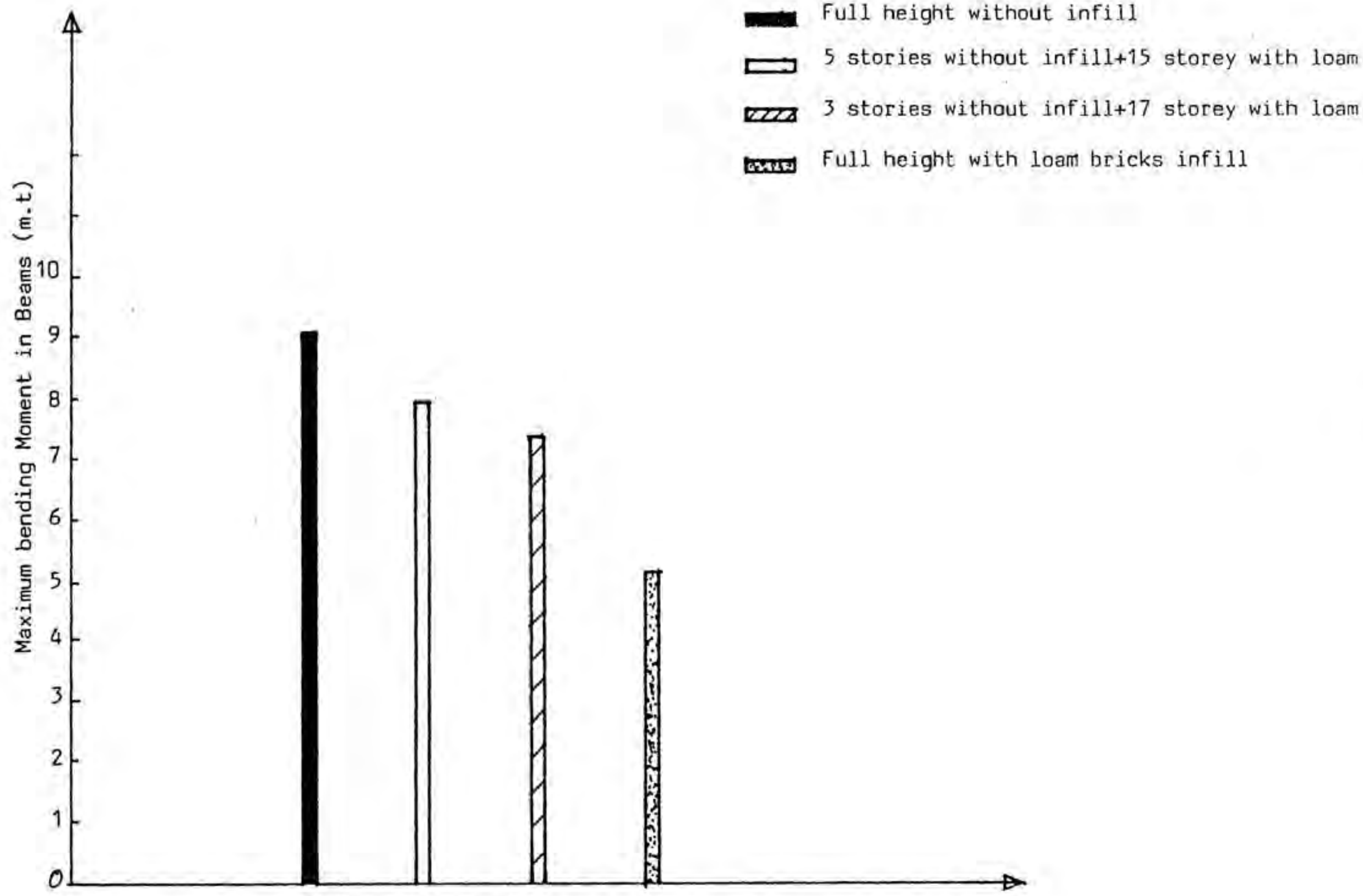


Fig. (5.28) Effect of Leaving the Lower stories without infill and filling the upper stories with loam bricks infill on Maximum Bending Moment in Beams.

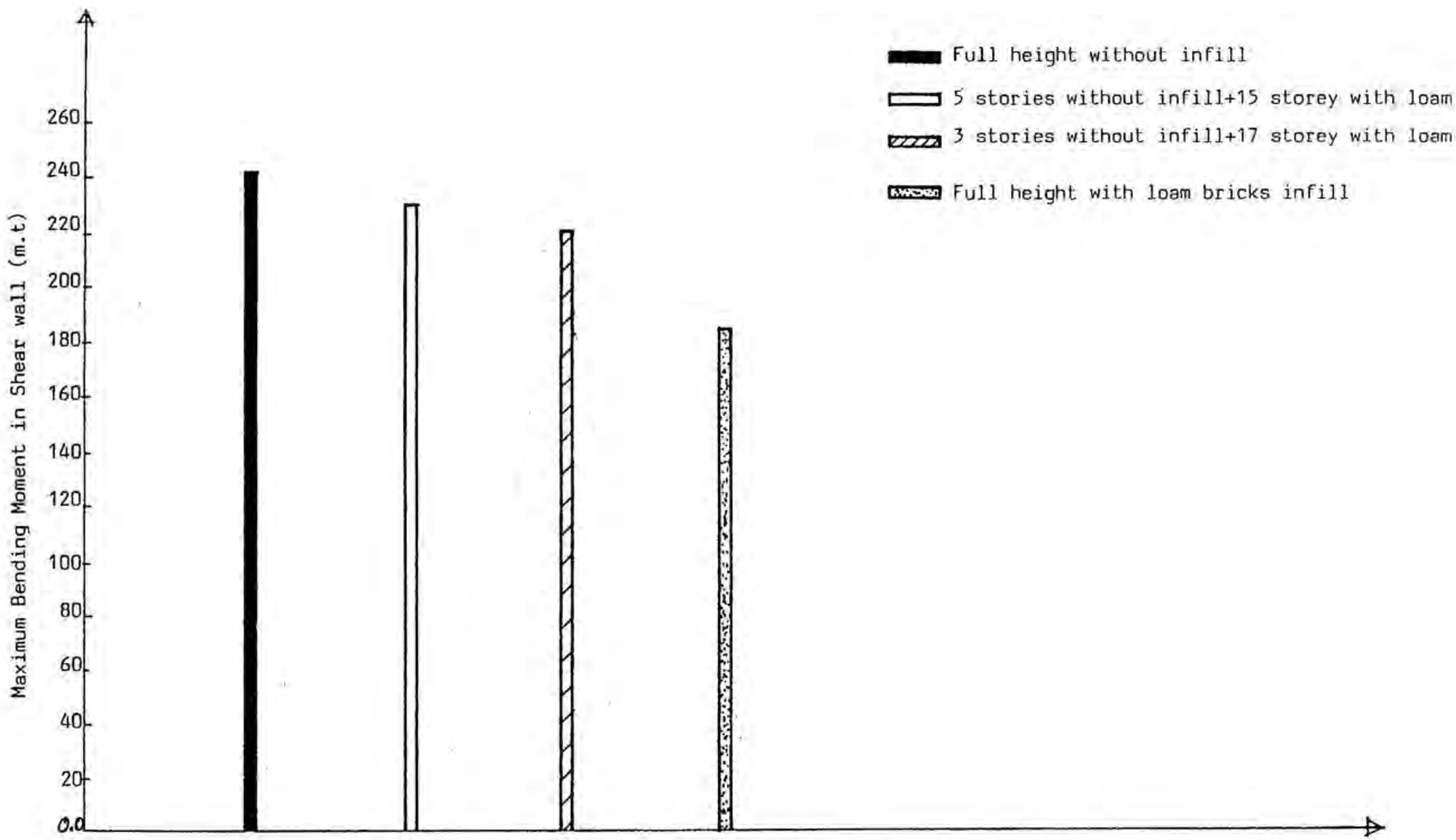


Fig. (5.29) Effect of leaving the lower stories without infill and filling the upper stories with loam bricks infill on Maximum Bending Moment In Shear wall.

5.6 Effect of Openings in Shear walls

In this work a shear wall with one row of openings was taken as example, the heights and widths of openings considered are four values as follows:

$$\frac{hw}{h} = 0,\ 0.4,\ 0.6\ \&\ 0.8$$

$$\frac{bw}{b} = 0,\ 0.4,\ 0.6\ \&\ 0.8$$

5.6.1 Effect of variation in opening height to storey height($\frac{hw}{h}$ ratio)

Four values of openings height to storey height ratio $\frac{hw}{h}$ equal to 0, 0.4, 0.6 & 0.8 were investigated in this work while keeping the ratio of opening width to wall width $\frac{bw}{b}$ constant and equal 0.4. The effect of these changes on the behaviour of the building is represented here after.

5.6.1.1 Lateral deflection of structure

The values of maximum lateral deflection for all considered cases are plotted in figure (5.30.a). The maximum lateral deflection increased by about (38% , 24% and 18%) as $\frac{hw}{h}$ was increased to 0.8, 0.6 & 0.4 respectively.

5.6.1.2 Bending moments in the columns of the building

For $\frac{hw}{h}$ equal to 0.8, 0.6 & 0.4 the values of maximum bending moment in both exterior and interior columns increases by about (36%, 26% and 16%) respectively, figure (5.31.a).

5.6.1.3 Bending moments in the beams of the building

The increase of the ratio $\frac{hw}{h}$ to 0.8, 0.6 & 0.4 increases the values of maximum bending moment in both exterior and interior beams by about (18%, 8% and 6%) respectively, figure (5.32.a).

5.6.1.4 Bending moments in shear walls

Figure (5.33.a) indicates that the maximum values of bending moment in shear walls decrease by about (36%, 20% and 12%) as the ratio $\frac{hw}{h}$ increases to 0.8, 0.6 & 0.4 respectively.

5.6.1.5 Normal forces in columns

The maximum values of normal forces in columns increase by about (18%, 10% and 6%) as the ratio $\frac{hw}{h}$ increases to 0.8, 0.6 & 0.4 respectively.

5.6.2 Effect of variation in opening width to wall width ($\frac{bw}{b}$ ratio)

Four values of opening width to wall width ratio $\frac{bw}{b}$ equal to 0, 0.4, 0.6 & 0.8 were investigated in this work while keeping the ratio of opening height to storey height $\frac{hw}{h}$ cosntant and equal to 0.4. The effect of these changes on the behaviour of the building is represented here after.

5.6.2.1 Lateral deflection of structure

The values of lateral deflection for all the considered cases of $\frac{bw}{b}$ were caluclated and plotted as shown in fig.(5.30.b). The maximum lateral deflection increases by about (48%, 32% and 18%) when the ratio $\frac{bw}{b}$ equal to 0.8, 0.6 & 0.4 respectively.

5.6.2.2 Bending moments in the columns of the building

The maximum values of bending moment in both exterior and interior columns increase by about (50%, 32% and 16%) when $\frac{bw}{b}$ ratio is equal to 0.8, 0.6 & 0.4 respectively as shown in figure (5.31.b).

5.6.2.3 Bending moments in the beams of the building

The maximum values of bending moment in both the exterior and interior beams increase by about (22%, 14%

and 6%) when the width ratio is equal to 0.8, 0.6 & 0.4 respectively as shown in figure (5.32.b).

5.6.2.4 Bending moments in shear wall

Figure (5.33.b) indicates that the maximum values of bending moment in shear wall decrease by about (51%, 28% and 12%) when the $\frac{bw}{b}$ ratio is equal 0.8, 0.6 & 0.4 respectively.

5.6.2.5 Normal forces in columns

The maximum values of normal forces in columns increase by about (26%, 18% and 6%) for $\frac{bw}{b}$ ratio equals to 0.8, 0.6 & 0.4 respectively.

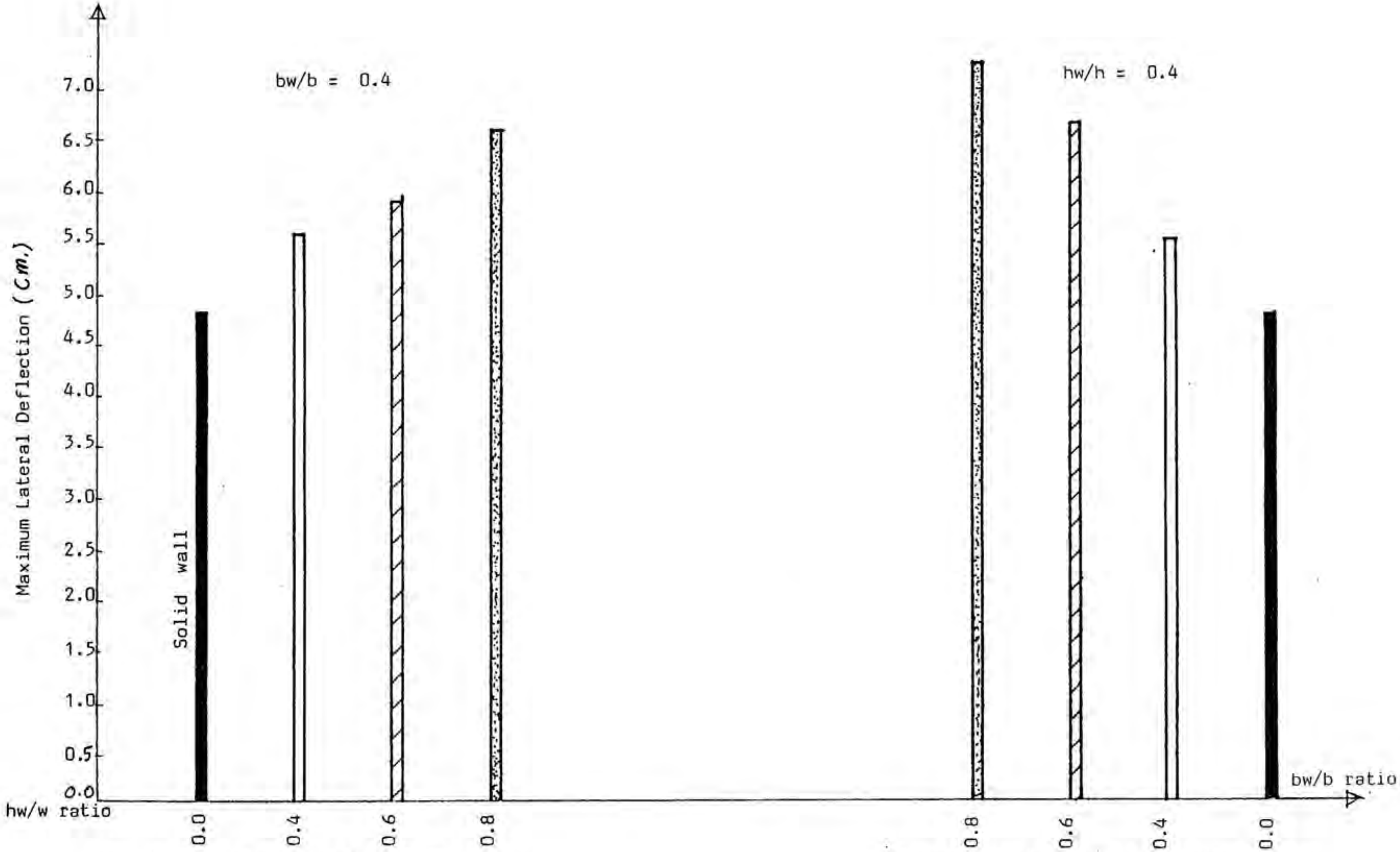


Fig. (5.30-a) Maximum Lateral Deflestion For various opening height to storey height ratio (hw/h).

Fig. (5.30-b) Maximum Lateral Deflection For various opening width to wall width ratio (bw/b).

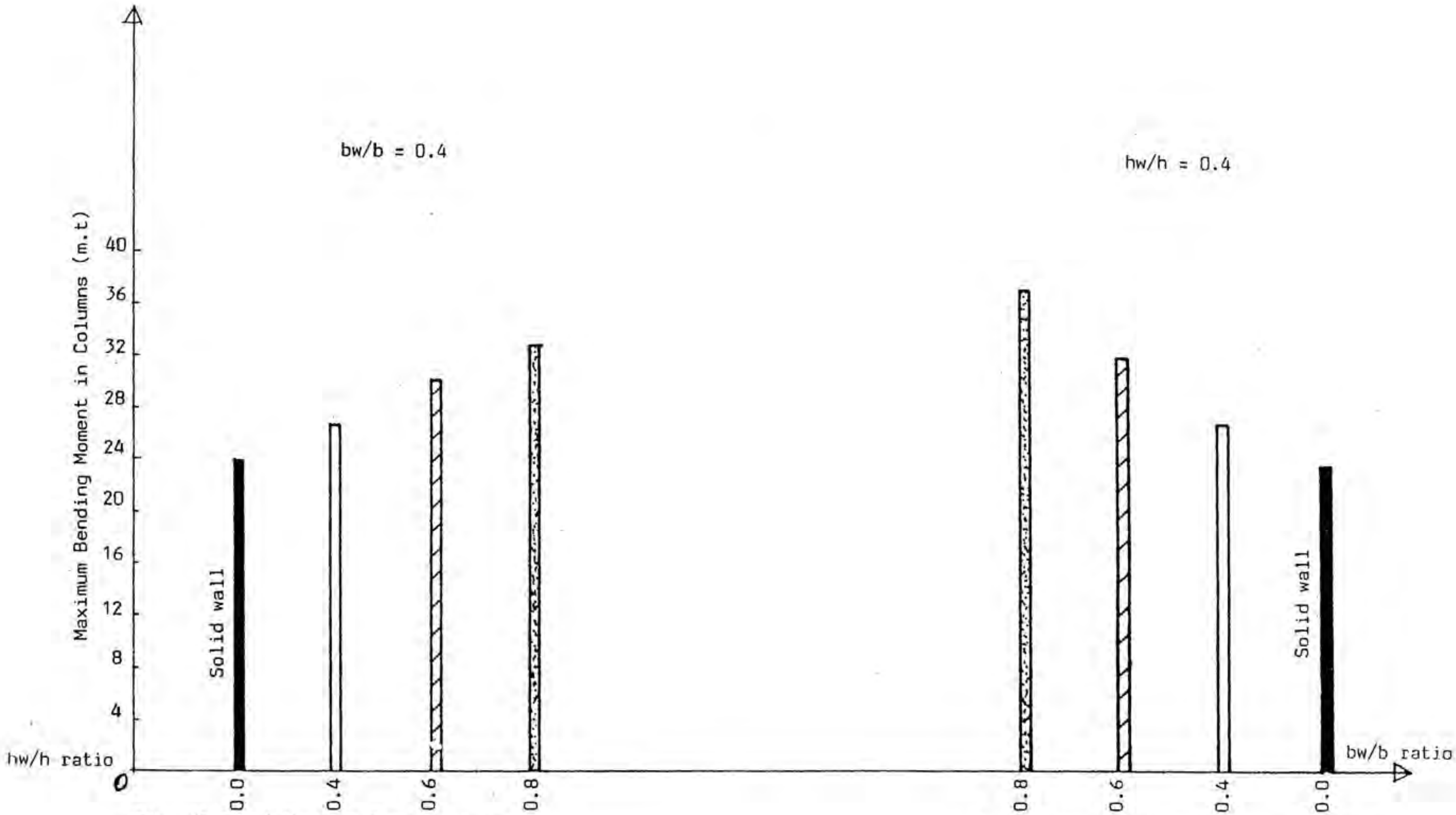


Fig. (5.31-a) Maximum Bending Moment in Columns for various opening height to storey height ratio (hw/h).

Fig. (5.31-6) Maximum Bending Moment in Columns for various opening width to wall width ratio (bw/b).

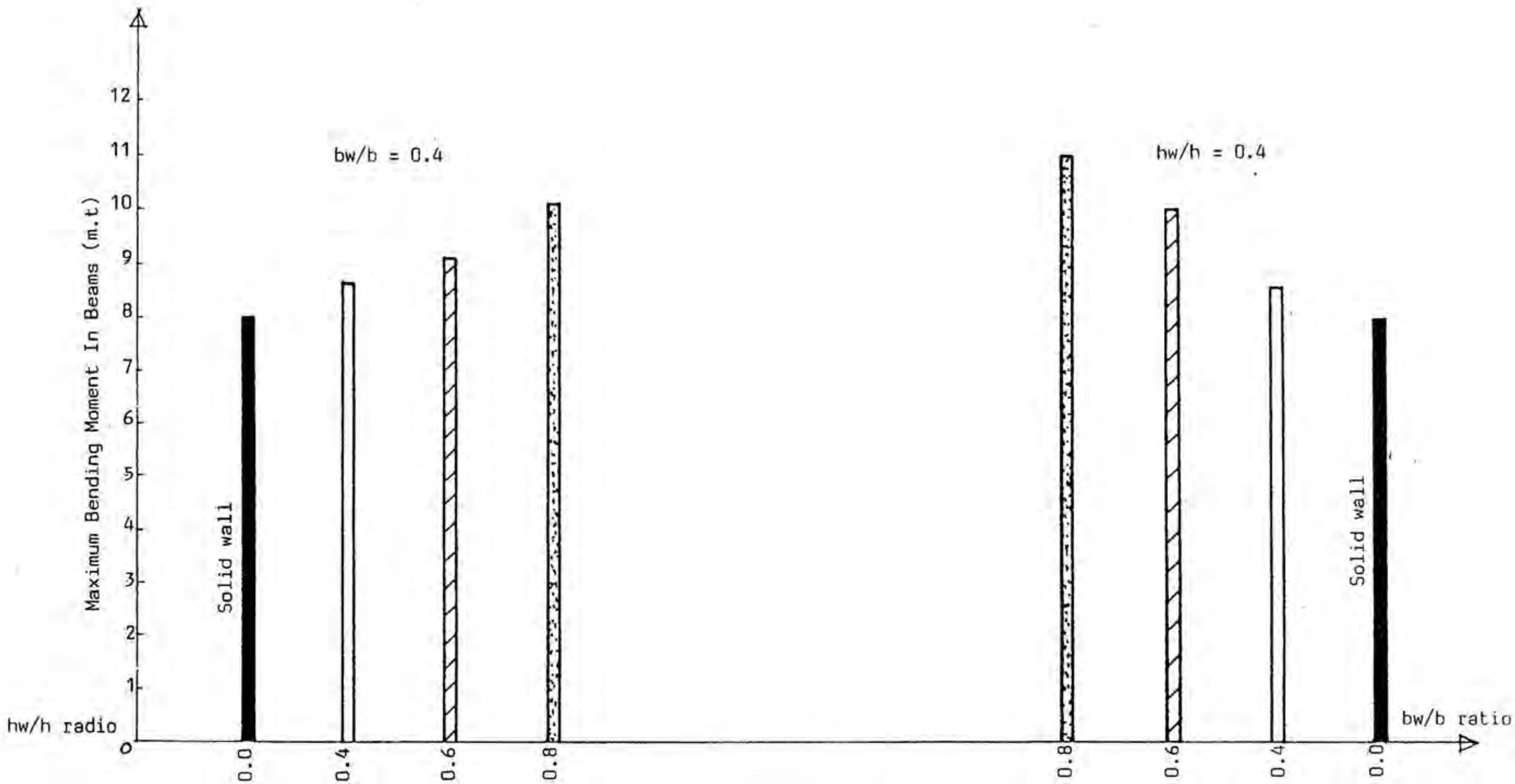


Fig. (5.32-a) Maximum Bending Moment in Beams for various opening height to storey height ratio (hw/h).

Fig.(5.32-b) Maximum Berding Moment in Beams for various opening width to wall width ratio(bw/b).

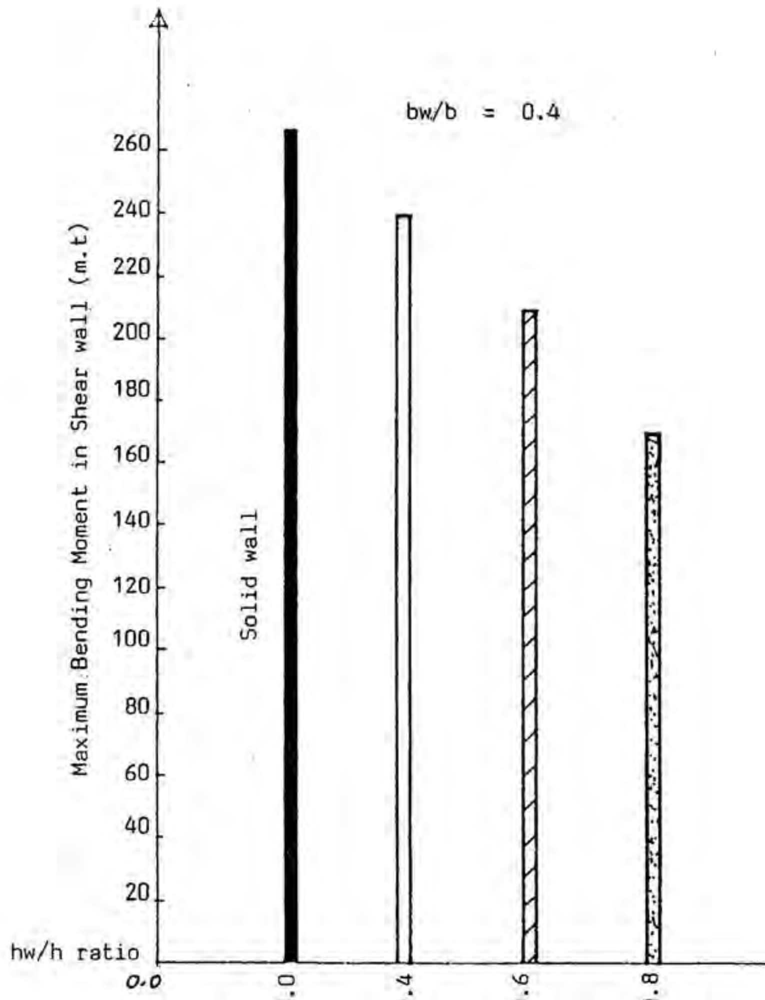


Fig. (5.33-a) Maximum Bending Moment in shear wall for various opening height to storey height ratio (hw/h).

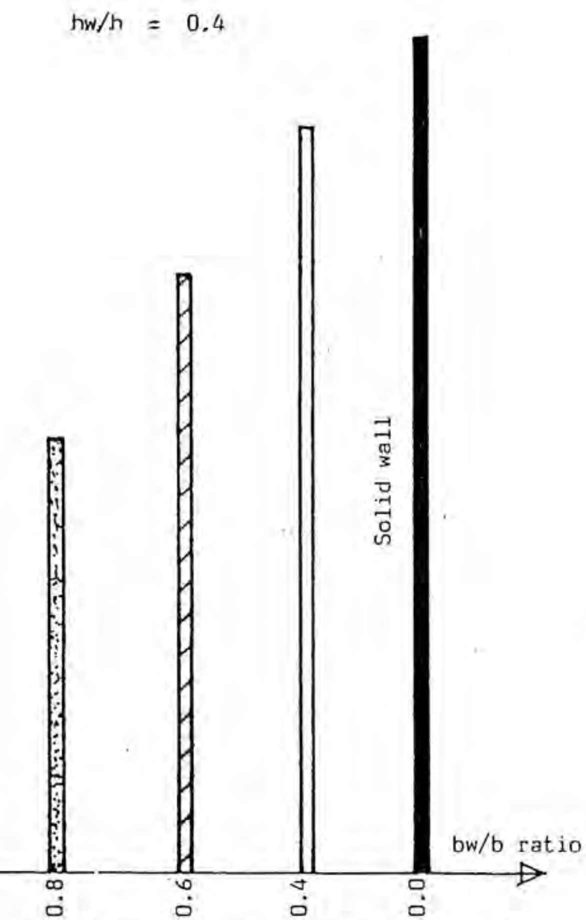


Fig.(5.33-b) Maximum Bending Moment in shear wall for various opening width to wall width ratio (bw/b).

TABLE(5-1) : The effect of filling material and type of bricks on the lateral deflection and internal forces:
(values are calculated for thickness of wall = 25 cm)

Number of stories (N)	Sand Brick [E=66 t/cm², ν= 0.175]								Loam Brick [E= 20 t/cm², ν=0.245]								Hollow Cement Brick [E=11 t/cm², ν=0.275]							
	% decrease in lateral deflection	% decrease in B.M. of Cl.		% decrease in B.M. of B.		% decrease in B.M. of shear wall	% increase in N.F.		% decrease in lateral deflection	% decrease in B.M. of Cl.		% decrease in B.M. of B.		% decrease in B.M. of shear wall	% increase in N.F.		% decrease in lateral deflection	% decrease in B.M. of Cl.		% decrease in B.M. of B.		% decrease in B.M. of shear wall	% increase in N.F.	
		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.
10	55	68	64	70	70	56	140	78	38	48	40	50	45	36	100	54	28	40	30	42	38	30	75	42
15	44	65	60	67	65	48	120	76	32	45	38	40	36	34	80	50	22	35	28	27	25	22	65	36
20	38	56	52	67	55	48	110	58	25	42	36	40	35	28	75	48	18	30	25	28	20	18	60	35

TABLE(5_2): Effect of infill panel thickness on the lateral deflection and internal forces :

(The values given in the table is the increase in half abrick with respect to one brick infill)

Number of stories (N)	Sand Brick [$E=66\ t/cm^2$, $\nu=0.175$]								Loam Brick [$E=20\ t/cm^2$, $\nu=0.245$]								Hollow Cement Brick [$E=11\ t/cm^2$, $\nu=0.275$].							
	% increase in lateral deflection	% increase in B.M. of columns		% increase in B.M. of beams		% increase in B.M. of shear wall.	% decrease in N.F.		% increase in lateral deflection	% increase in B.M. of columns		% increase in B.M. of beams		% increase in B.M. of shear wall.	% decrease in N.F.		% increase in lateral deflection.	% increase in B.M. of columns		% increase in B.M. of beams.		% increase in B.M. of shear wall.	% decrease in N.F.	
		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.		Int.	Ext.	Int.	Ext.		Int. Col.	Ext. Col.
10	40	52	45	50	50	32	28	28	38	34	30	35	35	20	20	20	35	24	22	30	30	12	16	16
15	38	38	38	40	40	25	26	26	36	32	30	32	32	20	18	18	32	20	22	24	24	10	15	15

TABLE(5.3): Effect of change of infill stiffness between different stories on the laterad deflection and internal forces:
(values are calculated for thikness of wall = 25 cm)

Number of lower stories filled by Sand Brick.	Number of upper stories filled by Hollow Cement Brick.	% decrease in Lateral deflection.	% decrease in B.M. of columns.		% decrease in B.M. of beams.		% decrease in B.M. of shear wall.	% increase in N.F. of columns.
			Interior.	Exterior.	Interior	Exterior.		
20	-	38	55	55	67	67	48	55
10	10	25	42	42	58	58	38	39
8	12	24	41	41	47	47	35	35
6	14	23	39	39	45	45	32	32
4	16	20	33	33	40	40	30	30

TABLE(5_4): Effect of lower stories without infill and upper stories with Loambrick infill on the lateral deflection and internal forces:

(values are calculated for thickness of wall = 25 cm)

Number of lower stories without infill.	Number of upper stories filled with loam Brick.	% Decrease in lateral deflection.	% decrease in B.M. of columns.		% decrease in B.M. of beams.		% decrease in B.M. of shear wall.	% increase in N.F. of columns.
			Interior	Exterior.	Interior	Exterior.		
–	20	28	41	41	45	45	28	46
3	17	18	20	20	28	28	10	34
5	15	16	12	12	18	18	6	26

TABLE (5-5):Effect of variation in opening height to storey height ratio ($\frac{hw}{h}$) on the lateral deflectionand internal forces:

$\frac{bw}{b}$	$\frac{hw}{h}$	% increase in lateral deflection.	% increase in B.M. of columns.		% increase in B.M. of beams.		% decrease in B.M. of shear wall.	% increase in N.F. of columns.
			Int.	Ext.	Int.	Ext.		
0.4	0.8	38	36	36	18	18	36	18
	0.6	24	26	26	8	8	20	10
	0.4	18	16	16	6	6	12	6

TABLE(5_6): Effect of variation of opening width to wall width ratio ($\frac{bw}{b}$) on the lateral deflection and internal forces:

$\frac{hw}{h}$	$\frac{bw}{b}$	% increase in lateral deflection.	% increase in B.M. of columns.		% increase in B.M. of beams.		% decrease in B.M. of shear wall.	% increase in N.F. of Columns.
			Int.	Ext.	Int.	Ext.		
0.4	0.8	48	50	50	22	22	51	26
	0.6	32	32	32	14	14	28	18
	0.4	18	16	16	6	6	12	6

CHAPTER 6

CHAPTER 6

CONCLUSIONS

Infilled frames - shear walls interaction under lateral loads have been theoretically studied in this work. The effect of filling materials and its type were investigated, sand lime bricks, loam bricks and hollow cement bricks were considered as filling materials. The effect of openings in shear wall and change in its geometry were investigated.

The stiffness of different filling materials represented by the modulus of elasticity and Poisson's ratio have been experimently determined.

Within the range of the conditions, parameters and limits considered in this work the following main conclusions and recommendations have been reached.

6.1 The Experimental Investigation Carried out in this work

Sand lime, loam and hollow cement bricks represent the most commenly used local types of bricks.

6.1.1 Sand lime bricks have a heigher compressive strength than hollow cement bricks by about 200%. Loam bricks have a heigher compressive strength than hollow cement bricks by about 100%.

6.1.2 Young's modulus for sand lime brick walls is greater than that for hollow cement brick walls by about 600%. Young's modulus for loam brick walls is greater than that for hollow cement brick walls by about 100%.

6.1.3 Poisson's ratio for hollow cement brick walls is greater than that for sand lime and loam brick walls by about 60% and 40% respectively.

6.1.4 Poisson's ratio varies from segment to segment through the same wall in both vertical and horizontal directions. The value of Poisson's ratio at the top and the bottom of the wall is smaller than its value at the center of the wall.

6.1.5 The value of Poisson's ratio is almost constant for the same wall for stress up to 1.5 times the compressive stress and for stress greater than that, Poisson's ratio value increases with the increase of the applied stress.

6.2 Theoretical Studies

The main conclusions and recommendations reached from the theoretical studies carried out in this work can be summarized as follows:

6.2.1 Effect of filling material and its type

a- Lateral deflection:

The values of lateral deflection decrease with the increase in Young's modulus and with the decrease of Poisson's ratio of the filling material. For instance the values of lateral deflection in the case of sand lime brick infill decreases by about 50% and 100% of that recorded in case of loam and hollow cement brick infills respectively. For the same filling material type, the effect of infill on the building lateral deflection decreases with the increase of the height of the building.

b- Bending moment in columns, beams and shear walls:

The values of maximum bending moment recorded in the case of using sand lime brick infill are smaller by about 50% and 100% than those obtained when using loam and hollow cement brick infills respectively.

c- Normal force in columns:

The values of maximum normal force obtained in the case of using sand lime brick infill are heigher by about 50% and 120% than those reached for loam and hollow cement brick infills respectively.

6.2.2 Effect of infill panel thickness

a- Lateral deflection:

The lateral stiffness of the building increases with the increase of the infill panel thickness and consequently the lateral deflection decreases. The values of lateral deflection in case of half a brick is heigher by about 40% from that recorded in case of one brick thickness as infill.

b- Bending moment in columns, beams and shear walls:

The values of bending moments in the different elements increase with the decrease of the panel thickness. This increase depends on the type of filling material and the height of the building having infill.

c- Normal force in columns:

The critical values of normal force in columns decrease with the decrease of the panel thickness. This depends also on the type of filling materail and the height of the building having infill.

6.2.3 Effect of change of infill stiffness between different strories

a- Lateral deflection:

The maximum building lateral deflection when filling the lower ten stories with sand lime bricks and the upper stories with hollow cement bricks are smaller by about (4%, 9% and 25%) than that when filling the lower (8,6 & 4) stories with sand lime bricks and the upper stories filled with hollow cement bricks respectively.

b- Bending moment in columns, beams and shear walls:

The maximum bending moments in the different elements when filling the lower ten stories with sand lime bricks and the upper stories filled with hollow cement bricks are smaller by about (4%, 8% and 25%) than that when filling the lower (8,6 & 4) stories with sand lime bricks and the upper stories filled with hollow cement bricks respectively.

c- Normal force in columns:

The maximum normal force in columns when filling the lower ten stories with sand lime bricks and the upper stories filled with hollow cement bricks are heigher by about (3%, 20% and 30%) than that when filling the lower (8,6 & 4) stories with sand lime bricks and the upper ones filled with hollow cement bricks respectively.

c- Normal force in columns:

For $\frac{bw}{b}$ ratio equals 0.4, the value of maximum normal force in case of $\frac{hw}{h}$ equals to 0.8 increased by about 80% and 200% than that of $\frac{hw}{h}$ equals 0.6 & 0.4 respectively.

6.2.5.2 Effect of variation in opening width to wall width ($\frac{bw}{b}$) ratio

The change in the opening width to wall width ($\frac{bw}{b}$) ratio while keeping the opening height to storey height ratio constant,affects the building behaviour as follows:

a- Lateral deflection:

For $\frac{hw}{h}$ ratio constant and equals 0.4, the lateral deflection in the case of $\frac{bw}{b}$ ratio equals to 0.8 increased by about 50% and 180% than that of the case of $\frac{bw}{b}$ ratio equals 0.6 & 0.4 respectively.

b- Bending moment in columns, beams and shear walls:

For $\frac{hw}{h}$ ratio constant and equals 0.4, the values of bending moment in columns and beams in the case of $\frac{bw}{b}$ ratio equals to 0.8 increased by about 60% and 210% than that of the case of $\frac{bw}{b}$ ratio equals to 0.6 & 0.4 respectively. Also for shear wall having $\frac{hw}{h}$ ratio constant and equals 0.4 and $\frac{bw}{b}$ equals 0.8, the values of bending moment decreased by about 85% and 300% than that of the case of $\frac{bw}{b}$ ratio equals 0.6 & 0.4 respectively.

c- Normal force in columns:

The normal force in columns for $\frac{hw}{h}$ ratio cosntant and equals 0.4 and $\frac{bw}{b}$ equals 0.8 increased by about 200% and 320% than that of the case of $\frac{bw}{b}$ ratio equals 0.6 & 0.4 respectively.

It can be noticed that the effect of opening width to wall width ($\frac{bw}{b}$) ratio has more tangible effect than opening height ($\frac{hw}{h}$) ratio in the values of building lateral deflection as well as on the internal straining actions for different structural elements.

The above conclusions show that the infill materail has a benefical effect on the lateral resistance of high rise buildings. Also the geometry of openings in shear walls should be taken into consideration in the analysis of the frame - shear wall interaction as it may affect the global behaviour of the building.

6.3 Future Studies

Future studies are also required to explore the behaviour of infilled multistorey frames having openings in the infill. Effect of addition of shear connectors between the infill and concrete skeleton with different shapes and types. Also the effect of different plastering types, conditions need additional investigation to explore its contribution to the effect of the infill on the whole building.

9- S.N. Cuhamjumdar, R.P. Nikhed, J.G. Macgregor, P.F. Adams, *Approximate Analysis of Frame - Shear Wall Structures*, Structural Engineering, Report No. 14, May 1969.

10- Rosman, R.,*Approximate Analysis of Shear Walls Subjected to Lateral Loads*, A.C.I. Journal proceedings, Vol.61, June 1964.

11- Alexandar Coull, and Ravi D. *Analysis of perced Shear Walls* Journal of the Struct.Division A.S.C.E., January 1968, pp. 71-82.

12- Michael, D. *The Effect of Local Wall Deformations on the Elastic Interaction of Cross Walls Coupled by Beams*, proceeding, symposium on tall buildings, University of Southampton, England, Pergamon Press, 1967.

13- Hassan Hosny, *Load Distibution in Shear Wall Structures* Ph. D. Thesis, Baisly College of Technology, 1976.

14- Macleod, I.A., *New Rectangluar Finite Element for Shear Wall Analysis*, Journal of the Struct. Division A.S.C.E., March 1969, pp. 399-409.

15- Krishna, M.A., *New Rectangular Finite Element for Shear Wall Analysis* Discussion, Journal of the Struct. Division A.S.C.E., September 1969, pp. 2000-2001.

16- Chiyyarath V. Girijavallabhan, A.M. ASCE, *Analysis of Shear Wall With Openings*, Journal of the Struct. Division A.S.C.E., October 1969, pp. 2093-2103.

17- B.S. Smith, *Modified Beam Method for Anaylzing Symmetrical Interconnected Shear Walls*, ACI Journal, June 1971.

18- I.A. Macleod, *Structural Analysis of Wall Systems*, the Structural Engineering, vol.55, No.11, November 1977, pp. 487-495.

19- Clough, R.W., *The Finite Element Method in Plane Stress Analysis*, proceedings, 2nd. A.S.C.E., Conf. on Electronic Computation, Pittsburgh, pa., September 1960.

20- Kham, F.R., On Some Special Problems of Analysis and Design of Shear Wall Structures, Report on Symposium on tall buildings, Southampton, 1966, pp. 70-82.

21- Khan, F.R. and Sbarounis, J.A., *Interaction of Shear Walls and Frames*, Journal of the Struct. Division A.S.C.E., vol.90, No. ST3, June 1964.

22- Kham, F.R. and Fitel, L.M., *Effect of Columns Exposure in Tall Structures Part B, Analysis and Design*, submitted for publication in Journal ACI.

23- Jack R. Benjamin and Harry A. Williams, *The Behaviour of One-Storey Reinforced concrete Shear Walls*, proc. A.S.C.E., Vol.83, No.ST3, Paper 1254, May 1957.

24- G.J.W. King and P.C. Pandey, *The Analysis of Infilled Frames Using Elements*, proc. I.C.E., Part 2, Vol.65, pp. 749-760, December 1978.

25- A.W. Astill and L.H. Martin, *Elementary Structural Design in Concrete to CP 110*, Edward Book Co., London, 1975.

26- B. Stafford Smith, *The Composite Behaviour of Infilled Frames* Symposium on Tall Buildings, Oxford, Pergamon, 1967.

27- Arnorld W. Hendry, *Structural Brickwork*, published by the Macmillan Press LTD, 1981.

28- B. Stafford Smith, *Lateral Stiffness of Infilled Frames*, proc. A.S.C.E., vol. 88, No.ST6, pp.183-199, December 1962.

29- Liauw, T.C. and Kwam, K.H., *Nonlinear Behaviour of Non-Integral infilled Frames*, Journal of Computers & Structures, Pergamon Press, vol. 18, No.3, pp. 551-550, 1984.

30- R.J. Mainstone, *On the Stiffness and Strngths of Infilled Frames*, proc. I.C.E., Paper No. 7360 S, Supplement IV, pp.57-90. 1971.

31- M.S.F. Darwish, *Reinforced Concrete Frames Infilled with Brick Walls*, Bulletin of the Facullty of Engineering, Cairo University, Civil Engineering, Paper 31, 1973-1974.

32- Jack R. Benjamin and Harry A. Williams, *The Behaviour of One-Storey Brick Shear Walls*, proc. A.S.C.E., vol.84, No.ST4, Paper 1723, July 1958.

33- M. Holmes, *Steel Frames With Brickwork and Concrete Infilling*, proc. I.C.E., vol.19, pp.473-478, 1961.

34- B. Stafford Smith and C. Carter, *A Method of Analysis for Infilled Frames*, proc. I.C.E., vol.44, pp.31-48, September 1969.

35- D.V. Mallick and R.P. Garg, *Effect of Openings on the Lateral Stiffness of Infilled Frames*, proc. I.C.E.,vol.49, pp.193-209, June 1971.

36- D.V. Mallick and R.T. Severn,*The Behaviour of Infilled Frames Under Static Loading*, proc. I.C.E., vol.38, pp.639-656, December 1967.

37- J.E. Riddington and B. Stafford Smith, *Analysis of Infilled Frames Subject to Racking with Design Recommendations*, Structural Engineer, vol.55, No.6, pp.263-268, June 1977.

38- El-Sayed M.H. El-Hozain, *Analysis of Reinforced Concrete Closed Frames with Brick Infill*, Thesis submitted for the Master Degree, Structural Engineering Department, Cario University, 1984.

39- B. Stafford Smith, *Behaviour of Square Infilled Frames*, proc. A.S.C.E., vol.92, No.ST1, pp. 381-403, February 1966.

40- B. Stafford Smith, *Model Test Results of Vertical and Horizontal Loading of Infilled Frames*, ACI Journal, Title No. 65-44, pp. 618-625, August 1968.

41- B. Stafford Smith, C. Carrter and J.R. Choudhury, *The Diagonal Tensile Strength of Brickwork*, Structural Engineer, vol.48, No.6, pp. 219-225, June 1970.

42- Randal Herbert Wood, *The Stabiltiy of Tall Buildings*, proc. I.C.E. Paper No.6280, vol. 11, pp.69-102, 1958.

43- P. Govindan, M. Lakshmipathy, and A.R. Santhakum, *Ductility of Infilled Frames*, ACI Journal, July-August 1986.

44- J.F.A. Moore, MA. BSC, PhD., ARSM *The Resistance of Masonary to Lateral Loading*, The Structural Engineer, vol.63A, No.11, November 1986.

45- Robert M. Henry and Frederic Roll, *Gladding-Frame Interaction*, The Structural Engineer, vol.112, No.4, April 1986.

46- Ezzat M. Morsy, *An Investigation of Mortar Properties Influencing Brickwork Strength*, Thesis Submitted for the Philisophy, Department of Civil Engineering and Building Science, University of Edingurgh, 1968.

47- Nabawe H. Ghareeb, *Analysis of Infilled Frames Under Lateral Loads*, Thesis submitted for the degree of Master, Structural Engineering Department, Cairo University, 1984.

48- Parsan S., Hendry A.W., Bradshaw R.E., *Crushing Tests on Storey Height Walls* $4\frac{1}{2}$ *- Inches Thick*, pro-ceedings of the Brit. Cer. Society No. 41, Load-bearing brickwork, July 1965.

49- F. El-Refai, A.E. Salama, E.H. Morsy, *Analytical and Experimenal Investigation of Masonry in Copression.*

50- Shaker El-Behairy, *Reinforced Concrete Design Handbook.*

51- Iain A. Macleod, *General Frames Element For Shear Wall Analysis*, P.I.C.E. part2, 1976, 61, Dec., 785-790.

APPENDIX (1)

COMPUTER PROGRAM

DATA

- Number of nodes	(NN)
- Number of restrained nodes	(NRSTN)
- Number of degrees of freedom	(N)
- Number of frame elements	(NFE)
- Number of type of frame elements	(NTFE)
- Node freedom array	(NF)
- Frame connection table	(FM)
- Number of load case	(NL)
- Young's modulus of materail	(E)
- Poisson's ratio of material	(μ)
- Cross-section area	(A)
- Length of flexible part	(L)
- Half width of wall segment	(X_1,X_2)
- Half depth of connecting beam	(Y_1,Y_2)
- Spring stiffness	(F_1,F_2)
- Equivalent shear area	(A)
- Shear deformation factor	(ASF)
- Angle of inclination of the member to the horizontal	(θ)

```
5  OPEN "B:DATA1.DAT" FOR INPUT AS #1
10 PRINT "PROCEDURE FORM NF (NN,NRSTN,RSTN,NF)
20 PRINT
30 INPUT "NO OF NODES NN =",NN
40 INPUT "NO OF NRSTN=",NRSTN
50 DIM NF(100 ,3),RSTN(100 ,4)
60 PRINT "    MATRIX RSTN"
70 PRINT "NO OF ROWS = NRSTN,  NO OF COLS = 4"
80 FOR L = 1 TO NRSTN
100 INPUT # 1 , RSTN(L,1),RSTN(L,2),RSTN(L,3),RSTN(L,4)
102 NEXT L
104 FOR I = 1 TO NRSTN
106 PRINT "  NODE NO =  " RSTN(I,1);
108 PRINT "  x =  " RSTN(I,2);
110 PRINT "  y =  " RSTN(I,3);
115 PRINT " theta =  " RSTN(I,4)
120 NEXT I
130 REM NF ARRAY
140 L=1
150 K = 1
160 I = 1
170 FOR I = 1 TO NN
180 FOR J = 1 TO 3
190 IF I = RSTN(L,1) THEN GOTO 230
200 NF(I,J) = K
210 K = K+1
220 GOTO 490
230 IF RSTN(L,2) = 0 THEN NF(I,1) = 0 ELSE
    IF RSTN(L,2) > 1000 THEN GOTO 290
240 IF RSTN(L,2) = 0 THEN GOTO 300
260 NF(I,1) = K
270 K = K+1
280 GOTO 300
290 NF(I,1) = NF(RSTN(L,2) - 1000, 1)
300 IF RSTN(L,3) = 0 THEN NF(I,2) = 0 ELSE
    IF RSTN(L,3) > 1000 THEN GOTO 360
310 IF RSTN(L,3) = 0 THEN GOTO 370
330 NF(I,2) = K
340 K = K+ 1
350 GOTO 370
360 NF(I,2) = NF(RSTN(L,3) - 1000, 2)
370 IF RSTN(L,4) = 0 THEN NF(I,3) = 0 ELSE
    IF RSTN(L,4) > 1000 THEN GOTO 440
380 IF RSTN(L,4) = 0 THEN GOTO 450
410 NF(I,3) = K
420 K = K + 1
430 GOTO 450
440 NF(I,3) = NF(RSTN(L,4) - 1000, 3)
450 L = L + 1
460 IF L = NRSTN + 1 THEN L = NRSTN
480 GOTO 500
490 NEXT J
500 NEXT I
```

```
560 LPRINT "   MATRIX NF"
570 FOR I = 1 TO NN
580 LPRINT "  x = " NF(I,1); "  y = " NF(I,2);
    "  theta = " NF(I,3)
590 NEXT I
610 PRINT "   PROCEDURE FORM MI (CONNECTING TABLE )"
620 INPUT "NO OF FRAME ELE = " ,NFE
630 INPUT "NO OF ROWS IN CON TABLE = ",NCON
635 DIM CON(50  ,5),FM(100,3)
640 REM entering matrix con
650 PRINT "   MATRIX CON"
660 PRINT "   NO OF ROWS = NCON, NO OF COLS = 5"
670 FOR L = 1 TO NCON
680 INPUT #1,CON(L,1),CON(L,2),CON(L,3),CON(L,4),CON(L,5)
710 NEXT L
720 L = 1
730 I = 1
740 IF CON(L,1) = 0 THEN GOTO 790
750 FM(I,1) = CON(L,1)
760 FM(I,2) = CON(L,2)
770 FM(I,3) = CON(L,3)
780 GOTO 830
790 FM(I,1) = CON(L,3)
800 FM(I,2) = CON(L,4)
810 FM(I,3) = CON(L,5)
820 GOTO 900
830 I = I + 1
840 K = I - 1
850 FM(I,1) = FM(K,1) + CON(L,4)
860 FM(I,2) = FM(K,2) + CON(L,4)
870 FM(I,3) = CON(L,3)
880 IF FM(I,2) = CON(L,5) THEN GOTO 900
890 GOTO 830
900 L = L + 1
910 I = I + 1
920 IF L < (NCON + 1) THEN GOTO 740
930 LPRINT "   FRAME CONNECTING TABLE"
940 FOR I = 1 TO NFE
950 LPRINT "END1 = " FM(I,1);
960 LPRINT "END2 = " FM(I,2);
970 LPRINT "TYPE = " FM(I,3)
980 NEXT I
1000 PRINT "   PROCEDURE FIND W"
1010 W = 0
1020 FOR M = 1 TO NFE
1030 FOR I = 1 TO 1
1040 FOR J = 1 TO 3
1050 Z = FM(M,I)
1060 K = NF(Z,J)
1070 IF K = 0 THEN GOTO 1170
1080 FOR P = (I + 1) TO  2
1090 FOR Q = 1 TO 3
1100 X = FM(M,P)
```

```
1110 L = NF(X,Q)
1120 IF L = 0 THEN GOTO 1150
1130 N = ABS(K - L)
1135 IF N > W THEN NW = L
1140 IF N > W THEN W = N
1150 NEXT
1160 NEXT
1170 NEXT
1180 NEXT
1190 NEXT
1200 LPRINT "HALF BAND WIDTH = ", W " AT FREEDOM = ", NW
1210 PRINT
1220 PRINT "   INPUT PROPERTIES OF MEMBERS "
1230 INPUT "   NO OF TYPES OF FRAME ELEMENTS = " ,FT
1240 DIM MD(50,13)
1250 PRINT "     E       V       A         I      L   X1   X2
     Y1   Y2  THETA  ASF   FF1   FF2"
1260 FOR I = 1 TO FT
1270 INPUT # 1 , MD(I,1),MD(I,2),MD(I,3),MD(I,4),MD(I,5),
     MD(I,6),MD(I,7),MD(I,8),MD(I,9),MD(I,10),MD(I,11),
     MD(I,12),MD(I,13)
1280 LPRINT "E=", MD(I,1);
1290 LPRINT "V=", MD(I,2);
1300 LPRINT "A=", MD(I,3);
1310 LPRINT "I=", MD(I,4)
1320 LPRINT "L=", MD(I,5);
1330 LPRINT "X1=", MD(I,6);
1340 LPRINT "X2=", MD(I,7);
1350 LPRINT "Y1=", MD(I,8);
1360 LPRINT "Y2=", MD(I,9);
1370 LPRINT "THETA=", MD(I,10);
1380 LPRINT "ASF=", MD(I,11);
1390 LPRINT "FF1=", MD(I,12);
1400 LPRINT "FF2=", MD(I,13)
1410 NEXT I
1420 PRINT
1430 PRINT "  INPUT LOAD VECTOR "
1440 INPUT "  NO OF JOINT LOADS = ",M
1445 INPUT "  NO OF DEGREES OF FREEDOMS = ",N
1450 DIM R(50 ,3),P(300)
1460 FOR L = 1 TO M
1470 INPUT "  NODE NO = ", R(L,1)
1472 INPUT "  DIRECTION ", R(L,2)
1474 INPUT "  LOAD = ",R(L,3)
1480 NEXT L
1490 LPRINT "  LOADS TABLE "
1500 FOR I= 1 TO M
1510 LPRINT  R(I,1),R(I,2),R(I,3)
1520 NEXT I
1522 FOR L = 1 TO M
1524 S = NF(R(L,1) , R(L,2))
1526 P(S) = R(L,3)
1528 NEXT L
```

```
1530 PRINT
1540 PRINT "  FORM MEMBER STIFFNESS MATRIX KM "
1555 W1 = W + 1
1560 DIM KB(300,40),Q(6,6),QQ(6,6),G(6),KM(6,6)
1570 LPRINT "  NO OF DEGREES OF FREEDOMS = ",N;
1580 PRINT " HALF BAND WIDTH = ",W
1590 FOR M = 1 TO NFE
1595 GOSUB 1600
1597 GOTO 1820
1600 TH = FM(M,3)
1610 E = MD(TH,1): V =MD(TH,2): A =MD(TH,3): I = MD(TH,4)
1620 L = MD(TH,5) : X1 = MD(TH,6) : X2 = MD(TH,7) :
     Y1 = MD(TH,8) : Y2 = MD(TH,9)
1630 TT = MD(TH,10) : ASF = MD(TH,11) : F1 = MD(TH,12) :
     F2 = MD(TH,13)
1640 IF ASF = 0 THEN B = 0
1645 IF ASF = 0 THEN GOTO 1660
1650 B = (2*(1 + V) * I) / (L*L*A*ASF)
1660 C1 = 4*(1 / 3 + B)
1670 C2 = 4*(B - 1 / 6)
1680 FF = F1 * F2
1690 RD = (1 - F1 + F1*C1)*(1 - F2 + F2*C1) - (FF*C2*C2)
1700 A1 = E * A / (L + X1 + X2)
1710 B11 = (4 * E * I / (L * RD))*((1 - F2)*F1 + C1 * FF)
1720 B12 = ( - 4 * E * I / (L * RD)) * FF * C2
1730 B22 = (4 * E * I / (L * RD)) * ((1 - F1)*F2 + C1*FF)
1740 H1 = (B11 + B12) / L : H2 = (B12 + B22 ) / L
1750 TT = TT * .0174533 : C = COS (TT) : S = SIN (TT)
1760 Q(1,1) = A1 * C: Q(1,2) = -A1 * S: Q(1,3) = -A1 * Y1
1770 Q(1,4) = -A1 * C : Q(1,5) = A1 * S: Q(1,6) = A1 * Y2
1780 Q(2,1) =-H1*S: Q(2,2) =-H1*C: Q(2,3) = (H1* X1) +B11
1790 Q(2,4) = H1*S: Q(2,5) = H1*C: Q(2,6) = (H1* X2) +B12
1800 Q(3,1) =-H2*S: Q(3,2) =-H2*C: Q(3,3) = (H2* X1) +B12
1810 Q(3,4) = H2*S: Q(3,5) = H2*C: Q(3,6) = (H2* X2) +B22
1815 RETURN
1820 QQ(1,1) = C : QQ(1,2) = -S / L : QQ(1,3) = -S / L
1830 QQ(2,1) = -S : QQ(2,2) = -C / L : QQ(2,3) = -C / L
1840 QQ(3,1) = -Y1: QQ(3,2) = 1 + X1 / L: QQ(3,3) = X1/ L
1850 QQ(4,1) = -C : QQ(4,2) = S / L : QQ(4,3) = S / L
1860 QQ(5,1) = S : QQ(5,2) = C / L : QQ(5,3) = C / L
1870 QQ(6,1) = Y2: QQ(6,2) = X2 / L: QQ(6,3) = 1 + X2 / L
1880 FOR II = 1 TO 6
1890 FOR J = 1 TO 6
1900 X = 0
1910 FOR K = 1 TO 3
1920 X = X + QQ(II,K) * Q(K,J)
1930 NEXT K
1940 KM(II,J) = X
1950 NEXT J
1960 NEXT II
1970 PRINT "  ELEMENT STIFFNESS MATRIX"
2005 GOSUB 2010
2007 GOTO 2080
```

```
2010 REM FORM  G
2020 FOR II = 1 TO 2
2030 FOR J = 1 TO 3
2040 G(3 * (II - 1) + J) = NF( FM(M,II) , J)
2050 NEXT J
2060 NEXT II
2065 RETURN
2080 Z = 6
2090 GOSUB 3000
2100 NEXT M
2110 GOTO 3130
3000 REM FORM KB
3010 PRINT "FORMING OVERALL STIFFNESS MATRIX KB FROM
     MEMBER",M
3020 FOR I = 1 TO Z
3030 IF G(I) = 0 THEN GOTO 3110
3040 FOR J = 1 TO Z
3050 IF G(J) = 0 THEN GOTO 3100
3060 C = G(J) - G(I) + W + 1
3070 IF C > (W + 1) THEN GOTO 3100
3080 IF C < 1 THEN PRINT "YOU MUST RE-ARRANGE THE NODES"
3090 KB(G(I),C) = KB(G(I),C) + KM(I,J)
3100 NEXT J
3110 NEXT I
3120 RETURN
3130 PRINT
3140 PRINT "PROCEDURE FORM LB , SOLVE LB"
3150 W1 = W + 1
3160 FOR I = 1 TO N
3170 X = 0
3180 FOR J = 1 TO W
3190 X = X + KB(I,J)*KB(I,J)
3200 NEXT J
3205 IF X > KB(I,W1) THEN PRINT "X > KB(I,W1) AT I =",I
3210 KB(I,W1) = SQR( KB(I,W1) - X)
3220 FOR K = 1 TO W
3230 X = 0
3240 IF N < (I + K) THEN GOTO 3310
3250 IF K = W THEN GOTO 3300
3260 FOR L = (W - K) TO 1 STEP -1
3270 S = I + K  :  H = L + K
3280 X = X + KB(S,L) * KB(I,H)
3290 NEXT L
3300 A =I+K: B =W-K+1: KB(A,B) = (KB(A,B) - X)/ KB(I,W1)
3310 NEXT K
3320 NEXT I
3330 REM PROCEDURE SOLVE LB
3340 DIM Y(300 ) , D(300) , GH(6)
3350 Y(1) = P(1) / KB(1,W1)
3360 FOR I =  2 TO N
3370 X = 0
3380 M = I - W - 1
3390 IF I < W1 THEN K = W - I + 2
```

```
3400 IF I <W1 THEN GOTO 3420
3410 K = 1
3420 FOR J = K TO W
3430 X = X + KB(I,J) * Y(M + J)
3440 Y(I) = (P(I) - X) / KB(I,W1)
3450 NEXT J
3460 NEXT I
3470 D(N) = Y(N) / KB(N,W1)
3480 FOR I = (N-1) TO 1 STEP -1
3490 X = 0
3500 IF I > (N - W) THEN L = N
3510 IF I > (N - W) THEN GOTO 3530
3520 L = I + W
3530 M = W + I + 1
3540 FOR J = (I + 1) TO L
3550 X = X + KB(J , (M - J)) * D(J)
3560 NEXT J
3570 D(I) = (Y(I) - X) / KB(I,W1)
3580 NEXT I
3590 LPRINT " NODAL DEFORMATIONS"
3600 FOR I = 1 TO NN
3602 LPRINT " NODE NO = " , I ;
3604 LPRINT " X= ",D(NF(I,1));: LPRINT" Y = ",D(NF(I,2));
3606 LPRINT " THETA = " ,D(NF(I,3))
3610 NEXT I
3620 PRINT
3630 PRINT "BACK SUBSTITUTION "
3635 LPRINT "  MEMBER ACTIONS "
3640 FOR M = 1  TO NFE
3650 GOSUB 1600
3660 GOSUB 2010
3680 FOR II = 1 TO 6
3690 GH(II) = D(G(II))
3700 NEXT II
3710 FOR I = 1 TO 3
3720 X = 0
3730 FOR J = 1 TO 6
3740 X = X + Q(I,J) * GH(J)
3750 NEXT J
3760 P(I) = X
3770 NEXT I
3780 AF = - P(1)
3790 M1 = P(2)
3800 M2 = P(3)
3810 Q1 = - (P(2) + P(3)) / L
3820 Q2 = Q1
3840 LPRINT "  MEMBER "  FM(M,1), FM(M,2);
3850 LPRINT "  AXIAL FORCE ",AF;
3860 LPRINT " SHEAR ",Q1 ;
3870 LPRINT "  MOMENT END 1 ",M1
3880 LPRINT " MOMENT END 2 ",M2
3890 NEXT M
4000 CLOSE # 1
```

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جامعة القاهرة
كلية الهندسة

تأثير الحوائط المائلة والفتحات في الحوائط الخرسانية على مقاومة المباني العالية للأحمال العرضية

رسالة
مقدمة للحصول على درجة الماجستير في الهندسة المدنية

من

جوده محمد محمد غانم

تحت إشراف

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بجامعة حلوان

١٩٨٧

اهـــــداء ....

الــى ابنـــتى الغاليـــــة ...

ســالـــــــى .

شكـــــــر

يتقدم الباحث بخالص الشكر الى **الاستاذ الدكتور / محمد العدوى ناصف أستـــــاذ المنشأت الخرسانية بكلية الهندسة جامعة القاهرة** لاقتراحة موضوع البحث ولاشرافة علـــــى الرسالة وتوجيهاتة القيمة وارشاداتة البناءة أثناء اجراء هذا البحث ٠

كذلك يعرب الباحث عن عميق امتنانه وشكره **للدكتور / أحمد الكفراوى الاستـــــاذ المساعد بكلية الهندسة بالمطرية جامعة حلوان** لقيامة بالاشراف على هذه الرسالة وتوجيهاتـة الصادقـــة ٠

ويتوجه أيضا بعميق الشكر لجميع أفراد أسرة معمل ابحاث الخرسانة جامعة القاهـــرة لحسن تعاونهم فى اتمام الابحاث المعملية ٠

وأيضا يتوجه بعميق الشكر الى رئيس قسم وأعضاء هيئة التدريس بكلية الهندســـــة بالمطرية جامعة حلوان لما أبدوه من تعاون ومساعدة لانجاز هذا العمل ٠

ملخـــــص

يتأثر سلوك المبانى العالية المعرضة لاحمال تؤثر فى الاتجاه العرضى لها بوجـــود الحوائط المملوئة بالطوب وكذا نوعة وأيضا بالفتحات الموجودة بالحوائط الخرسانية الكائنة لمقاومة القص .

لذلك تحتوى الرسالة على دراسة تحليلية للمبانى العالية المكونة من اطارات مـــن الخرسانة المسلحة ذات فتحات مملوئة بأنواع مختلفة من الطوب بالاضافة الى حوائط القـــص الخرسانية وذلك تحت تأثير الاحمال العرضية وتتناول الدراسة تأثير نوع المادة المالئـــة وكذلك تأثير التغير فى سمك هذه المواد المالئة وكذلك تأثير تفريغ الادوار السفلى وحشـــو الادوار العليا وأيضا مدى تأثير استخدام المواد المالئة العالية الجساءة (طوب رملى) وذلك للادوار السفلى .

وتتضمن الدراسة مدى تأثير الفتحات بالحوائط الخرسانية والتغير لهذه الفتحـــات سواء فى الاتجاه الافقى أو الاتجاه الرأسى .

كذلك تم الحصول معمليا على قيم معاير المرونة ونسبة بواسون لثلاث انواع مـــن الطوب الاكثر شيوعا أو استخداما محليا فى الحوائط (طوب رملى ـ طوب طفلى ـ طـــوب اسمنتى مفرغ) .

تحتوى الرسالة على ملخص موجز للدراسات السابقة فى هذا المجال وهى الطرق المختلفة لتحليل المنشأت العالية المكونة من اطارات وحوائط قص خرسانية وكذلك الطرق المختلفـــة لتحليل حوائط القص الخرسانية ذات الفتحات وايضا الانواع المختلفة للاطارات المحشـــوة ، طرق تمثيـــل المادة المالئة ، العوامل المؤثرة فى سلوك هذا النوع من المنشأت ، تأثـــير وجود اتصال بين الاطارات والمواد المالئة بواسطة وصلات القص ، تأثير وجود فتحات فى الحوائط المالئة ، الاشكال المختلفة والمحتملة للانهيار فى هذا النوع من المنشأت .

وقد أستخدم الباحث طريقة العضو القطرى المكافىء ليحل محل الحائط المائـــــــى للهيكل فى تحليل الهياكل المحشوة التى تمت دراستها فى هذا البحث ٠

وقد تم فى هذا البحث تحليل عدد ٣٠ نموذج وذلك لدراسة مدى تأثير المتغيـــــرات المختلفـــة ٠

وقد قام الباحث بتحليل النتائج وعمل المقارنات بينها لجميع الحالات المختلفـــة وكذلك لجميع العوامل المؤثرة على سلوك هذه المنشأت وقد تم التوصل الى استحداث مجموعـــة من المنحنيات تساعد المصمم على أخذ تأثير المادة المالئة والفتحات فى حوائط القـــــص الخرسانية فى الاعتبار عند التصميم لمثل هذه المنشأت وتشمل النتائج مقارنة الازاحة الافقيـــة وبيان مدى تأثيرها بالعوامل المختلفة وكذا قيم العزوم المؤثرة على جميع أعضاء المنشـــــأ الخرسانى وقيم القوى العمودية المؤثرة على الاعمدة وتشمل الرسالة ملخصا موجزا للنتائـــج التى تم التوصل اليها فى هذه الدراسة وكذلك بعض التوصيات للدراسات المستقبلية فى هـــذا المجـــال ٠