EFFECT OF INFILL WALLS ON FRAMES DUE TO SEISMIC LOADING

A Thesis

by

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December, 2005

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I do hereby agree to the style and content of the present dissertation.

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ABSTRACT

In the building construction, framed structures are frequently used due to ease of construction and rapid progress of work. Column and girder framing of reinforced concrete structures is in-filled by masonry infill panels, solid or hollow blocks or concrete blocks. Masonry infill panels have been widely used as interior and exterior partition walls for aesthetic reasons and functional needs. Usually in analysis only the bare frame is modeled ignoring the effect of infills with the assumption that such modeling is rather conservative and computationally more efficient. However such assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of a structure. When an infilled frame is subjected to lateral loading, the infills behave effectively as struts along its compression diagonals to brace the frames. When infill walls are omitted in a particular storey, a weak storey is formed compared to much stiffer other storeys. The ground floor is the most common location, which is usually devoid of infill component due to parking or large commercial spaces, thus resulting in a point of weakness in the structure.

Normally in structural analysis it is considered that the Equivalent Static Force Method is more conservative than more rigorous dynamic procedure for regular structures or structures of smaller height. The present study compares the results of Equivalent Static Analysis and the Response Spectrum Analysis of frames modeled with infills. In this investigation the performance of masonry infill components on frames under different conditions have been studied. The masonry infill has been modeled by equivalent struts. The size, shape and other properties of the equivalent struts have been calculated using different published literature. The drift and flexural behavior of the frame with different combinations of infill walls have been studied and compared using both the Equivalent Static Analysis and Response Spectrum Analysis techniques. A parametric investigation has also been performed varying various parameters of the frames to observe their influences in drift and flexural behaviour of the frames. The present study is aimed at finding out the effect of infills on structures due to horizontal loading, which would lead to safe, economic and durable framed structures against earthquakes.

It is observed that the Equivalent Static Analysis shows higher values for deflection and moment than the Response Spectrum Analysis only for the bare frame. Presence of infill in the frame, however, shows converse results for 6, 9 and 12 storied frames. For 4 storied frames, Response Spectrum Analysis gives higher values for both deflection and moments for the structures with or without infill components than the Equivalent Static Analysis. It is observed that moment increased depending on the increase of the infill percentage. When all bays have infill it shows the highest variation of maximum moment and conservative results are shown by the Response Spectrum analysis than the Equivalent Static one.

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NOTATIONS

a	equivalent masonry strut width
a_{mod}	Modified equivalent width of infill strut (in.)
A_{open}	Area of the opening (in^2)
A_{panel}	Area of the infill panel (in^2)
D	Diagonal length of infill (in.)
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of the reinforcing steel (ksi)
E_m	Modulus of elasticity of the masonry unit.
f'_m	Compressive strength of the entire masonry assemblage
h	Height of the concrete member (in.)
h_m	Height of the masonry infill panel (in.)
hm/t	Slenderness ratio of infill panel
l	Length of the infill panel (in.)
l/h_m	Infill panel aspect ratio
l_{beam}	Distance from the face of the beam to the first beam plastic hinge (in.)
l_{column}	Distance from the face of the column to the first column plastic hinge (in.)
I_{col}	Moment of inertia of the column
n	Modular ratio
t	Thickness of the masonry infill panel
t_{eff}	Net thickness of the masonry panel (in)
w	Unit Weight of masonry assembly
σ	Modulus of Rigidity
$(R_1)_i$	Reduction factor for in-plane evaluation due to presence of openings
$(R_2)_i$	Reduction factor for in-plane evaluation due to existing infill damage
ξ_I	Strength increase factor due to presence of FRP overlay
θ	Angle produced by the strut with the horizontal

CHAPTER-1

INTRODUCTION

1.1 GENERAL

Frame structures are frequently used in multistoried buildings, mainly due to ease of construction and rapid progress of work. Column and girder framing of reinforced concrete, or some times steel, is in-filled by panel of brickwork, block work, cast in place or pre-cast concrete. When an in-filled frame is subjected to lateral loading, the in-fills behave effectively as struts along its compression diagonals to brace the frames. So, the in-fills serving as external walls or internal partitions, contribute some effect on stiffness of the framing system. This contribution totally depends on the properties of in-filled materials.

In the case of masonry in-fills, many uncertainties are involved, because masonry is a composite material consisting of masonry units set in mortar. The units and mortar have different characteristics; masonry exhibits distinct directional properties with potential planes of weakness being created by the low tensile strength at each unit/mortar interface. For resistance to wind and earthquake forces, it is this bond strength at the interface that is important, in both flexure and shear.

The wide range of structural damages observed during several earthquakes in the past is very educative in identifying the structural systems, those which are preferable and those which should be avoided. Seismic forces primarily act on the ground and displace it laterally along with the bases of buildings. If a super-structure is laterally stiff, then it moves together with its base. However a structure that is laterally flexible, experiences a relative displacement with respect to the ground. The extent of the relative displacement depends on its stiffness and the inertia of its masses. Stiffness of a building is reflected by its natural period of vibration.

When a sudden change in stiffness takes place along the building height, the story at which this drastic change of stiffness occurs is called a soft story. According to BNBC (1993) a soft story is the one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three storeys above. The most common form of vertical discontinuity arises due to the unintended effect of infill component. The problem is most severe in structures having relatively flexible lateral load resisting system because the infill can provide a significant portion of the total stiffness.

The ground floor is the most common location, which is usually devoid of infill component due to parking or large commercial spaces. For the architectural reasons and various functional purposes, open spaces are provided at any height of the building. Normally these open spaces have only columns as vertical members and there is no infill component. The adjacent floors, however, possess some infill components in between two columns to create utility spaces. The infill components increase the lateral stiffness and serve as a transfer medium of horizontal inertia forces. From this conception the floors that have no infill component has less stiffness than other floors. This particular floor gives excessive lateral deformation. In seismic design at least a minimum stiffness is ensured through the limitation on the drift, i.e., horizontal relative floor displacement per unit story height. According to UBC (1985) the allowable story drift is 0.005 of the story height. If this limitation is exceeded for any floor then that particular floor forms a soft story. Normally in structural analysis it is considered that the Equivalent Static Analysis is more conservative against ground shaking for regular structures or structures of smaller height. Usually in analysis only the bare frame effect is considered ignoring the effect of masonry infill. According to Tantry etl. (2003) such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of the structures.

From past experience it was observed at Jabalpur, India, that a number of RC frame buildings with brick infill walls, although performed well during an earthquake with only nominal damage, usually cracking of brick infill walls, most of this did not comply with seismic codes for earthquake forces or for seismic detailing. The brick infills apparently played a positive role in the seismic response of such buildings. For buildings that are reasonably symmetric in geometry and do not have significant variation in stiffness and strength in plan and in elevation, brick infill, if intact, acts as a source of strength and stiffness, and leads to improvements in seismic performance. Thus more investigation of the effect of infills on framed structures is very important.

All metropolitan towns in Bangladesh have a large inventory of multistory structures with this type of problem. The present study is aimed at finding out the infill effect on structures due to horizontal loading. This may lead to safe, economic and durable framed structures against earthquakes.

1.2 OBJECTIVES AND SCOPE

The wide range of structural damages observed during several earthquakes in the past is very educative in identifying the structural systems, those which are preferable and those which should be avoided. During an earthquake the nature and amount of displacement of a structure depend on its stiffness and the inertia of its masses. When a sudden change in stiffness takes place along the building height, the story at which this abrupt change of stiffness occurs is called a soft story. The most common form of vertical discontinuity arises due to the unintended effect of infill component. Normally in structural analysis it is considered that the equivalent static analysis is more conservative against ground shaking for regular structures. In the analysis usually only the bare frame effect is considered but not the effect of the infill components. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of the structures having vertical discontinuity of infill components. The major objectives of the research work are as follows:

a) to review the codal provisions of soft story due to absence of infill walls at particular stories.
b) to investigate the change in the amount of moments in beams and columns due to the effect of infill components.
c) to find out the influence of masonry infill wall panel in reinforced concrete framed structures in terms of deformation.

1.3 METHODOLOGY OF THE WORK

Methodology of this research work is summarized below:

i) Available literature are reviewed in order to know the state of the art of behavior of masonry infill walls on framed structures.

ii) Two dimensional frames are studied in this work. These frames are analyzed by the Equivalent Static Method and the Dynamic Analysis Method using Response Spectrum.

iii) The frames are divided in a number of ways, a) Bare frame; b) One bay has infill wall out of three and three bays have infill wall out of five bays; c) Two bays have infill wall out of three and four bays have infill wall out of five bays; d) All bays having infill walls.

iv) The infill walls have different characteristics: a) 125 mm infill wall thickness; b) 250 mm infill wall thickness; c) Combined Compressive strength of the brick masonry infill are also varied in this study. Values of f'_m considered in the study are 2757.9 N/cm^2 (4000 psi) and 5515.8 N/cm^2 (8000 psi).

v) No of story in vertical direction and the panel aspect ratio is also varied.

vi) After analyzing the frames, deflection pattern and variation of moments in columns and beams are observed. These quantities are compared for different infill patterns and analysis procedures.

1.4 ORGANIZATION OF THE THESIS

The thesis is organized into six chapters. Chapter 1 is the current chapter which introduces the work presented in the thesis. Chapter 2 deals with the literature review which includes the characteristics of infilled frame and methods of analysis. Chapter 3 discusses about the computational modeling of the infilled frame. Chapter 4 contains the discussion of various parameters of infilled frame. Chapter 5 is composed of analysis and results and Chapter 6 draws conclusions by summarizing the outcome of the research work and proposed new directions for further research and developments.

CHAPTER 2
LITARETURE REVIEW

2.1 GENERAL

The in-filled building frames consists of Steel or reinforced concrete column and girder frames with in-fills of solid or hollow brick or concrete block. Generally these in-fills are used as partition walls or architectural element and some of they have significant contribution to horizontal stiffness, even though they are considered as nonstructural element. At the time of strong horizontal shaking the infills tend to interact with the building frame. For most of the framed structures in our country masonry walls are used as an infill component due to the simplicity of construction, availability and economical reasons.

During an earthquake, violent shaking is transmitted into a structure at the points where it is attached to the ground. Numerous uncertainties exist in making a realistic evaluation of the maximum expected earthquake motion at a given site. It is virtually impossible to predict with reasonable accuracy where and when the next damaging earthquake will take place, and the fault on which it will be triggered. Even in the area of high seismic vulnerability that has been thoroughly studied geologically, damaging earthquake have occurred due to movement on an otherwise unidentified fault. To help the engineers for designing a sound structure, response spectrum (modal superposition) analysis is used in conjunction with information gathered by instruments from past earthquakes. An engineer can determine how a structure would react to a past real-world earthquake and consider any new design. Response spectrum analysis is a procedure for computing the statistical maximum response of a structure to a base excitation (or earthquake). In this chapter literature regarding the behavior of in-filled frames, rigid frames, braced frames and response spectrum analysis are reviewed.

2.2 REVIEW OF CODAL DEFINATIONS OF SOFT STORY

When a sudden change in stiffness takes place along the building height, the story at which this drastic change of stiffness occurs is called a soft story. According to

BNBC (1993) and UBC (2000) a soft story is the one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three storeys above and in IBC (2000) an extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above. The vertical geometric irregularity shall be considered to exist where the horizontal dimensions of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story.

The most common form of vertical discontinuity arises due to the unintended effect of infill component. Usually in analysis only the bare frame effect is considered ignoring the effect of infill walls and code has no clear suggestion about the infill walls on frames. The problem is most severe in structures having relatively flexible lateral load resisting system because the infill can compose a significant portion of the total stiffness.

2.3 RESPONSE SPECTRUM ANALYSIS

Earthquake accelerograms display the irregularity of the ground accelerations as a function of time. Although the accelerogram provide basic information about the nature of the ground motions, the response spectrum gives a more meaningful characterization for design purposes. It is defined as a graphical representation of the maximum response of a damped single degree of freedom (SDOF) mass spring system with continuously varying natural periods to a given ground excitation (Smith and Coull, 1991).

The value of response spectrum is that it provides a more significant and meaningful measures of the effect of an earthquake motion than just a single value, such as the peak acceleration, does. Although the actual response spectra for earthquake motions are quite irregular, they have the general shape of a trapezoid when plotted in tripartite logarithmic forms as in Fig. 2.1. For design purpose the actual response spectrum is normally smoothed to produce a curve that consists only of straight line portions as shown in Fig. 2.2. The smoothing is performed on a statistical basis, in recognition of the fact that the detailed response spectrum of any future earthquake is unknown. It may be possible to average the response spectra of more than one earthquake to give a more meaningful design input and it is also possible to use probabilistic theory to construct simulated accelerograms and design response spectra (Clough and Penzien, 1975). The linear form is also more appropriate because of the difficulty of calculating exactly what the period of a tall building will be during strong shaking (Smith and Coull, 1991).

During an earthquake, violent shaking is transmitted into a structure at the points where it is attached to the ground. To help engineers design sound structures, response spectrum (modal superposition) analysis is used in conjunction with information gathered by instruments from past earthquakes. An engineer can determine how a structure would react to a past real-world earthquake and consider any new design. Response spectrum analysis is a procedure for computing the statistieal maximum response of a structure to a base excitation (or earthquake). Each of the vibration modes that are considered may be assumed to respond independently as a single degree of freedom system. Design code speeify response spectra which determine the base acceleration applied to each mode according to its period (time in seconds required for a cycle of vibration). The design response spectrum is obtained by multiplying the response spectra curve by a structural performance factor, a risk factor, a zone factor, and limit state factor.

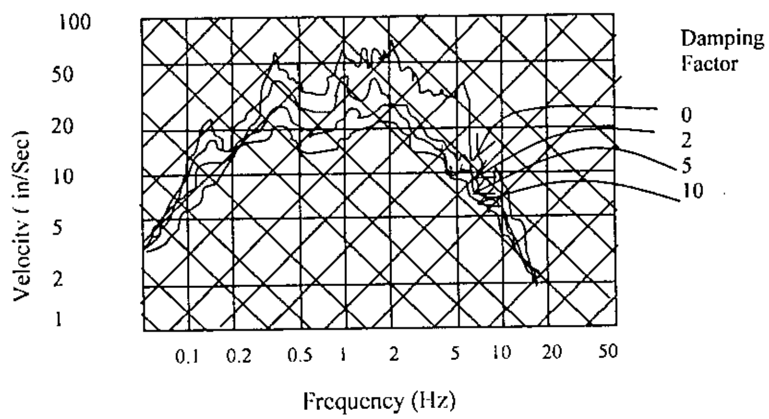


Fig. 2.1 Response Spectra . El Centro Earthquake. N-S Direction (Newmark and Hall, 1981)

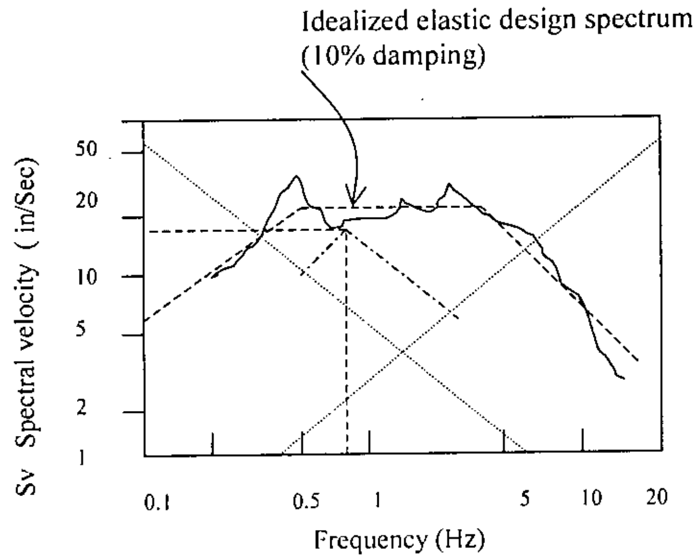


Fig. 2.2 Idealized design response spectrum

Determining the behavior of a structure during an earthquake is basically a vibration problem. The seismic motion of the ground cause the structure to vibrate and the amplitude of vibration of this dynamic deformation and its duration are of concern to the designer. The actual earthquake design criteria must be based on a number of considerations, such as the probability of occurrence of strong ground shaking, the characteristics of the ground motion, the nature of the structural deformation, the behavior of the building materials when subjected to oscillatory strains, the nature of the building damage that may be sustained, and the cost of repairing the damage as compare to the cost of providing additional earthquake resistance (Taranath, 1988). The structural engineering profession has gradually moved toward more exact approaches in the seismic design of multistory building, due to greater understanding of the earthquake phenomenon. Relatively simple methods based on equivalent static loads are no longer satisfactory for predicting improved accuracy in structural response. The critical ground motion characteristics and application of advanced dynamic analysis techniques give improved accuracy in predicting structural response.

There are three choices available for applying the dynamic earthquake loads, namely, the time history, the frequency domain, or the response spectrum methods. Both the time history and frequency domain methods require prescription of a specific ground motion record which requires prediction of the future critical seismic ground motions that may occur at a site during the useful life of a structure. The time history analysis is not always practicable. Though the intention for doing time history analysis seems to be site-specific ground motion studies, which is burdened with numerous uncertainties, it is impractical for most building design project(Murty and Jain, 1994). Therefore, it is prudent to base seismic design on a range of possible earthquake ground motions rather than a single assumed earthquake motion. This is obtained by using a so-called response spectrum, which represents an envelop of upper bound response based on several different ground motion records. The method based on response spectrum, is generally cost effective and therefore is the most widely used approach for representing dynamic earthquake loading.

The word "spectrum" is used to reflect the fact that a broad range of related quantities is summarized on one graph. For a given earthquake record and a given percentage of critical damping, the graph shows related quantities such as acceleration, velocity or deflection for a complete range of spectrum of building periods. The plot of a response spectrum can be explained with reference to the Fig. 2.3 by visualizing it as the response of a series of progressively longer cantilever pendulums with increasing natural periods subjected to a common lateral agitation at the base. Assume that the common base is moved through a ground motion corresponding to the motion that would occur in a given earthquake. A plot of maximum response such as acceleration versus the period of the pendulums will provide the acceleration response spectrum as shown in Fig. 2.3 (Taranath, 1988).

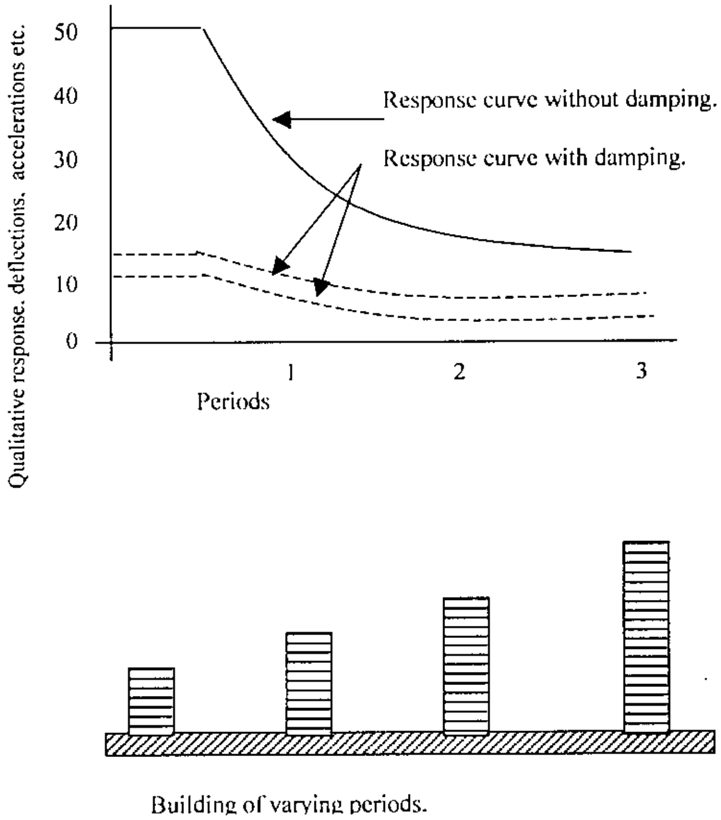


Fig. 2.3 Response Spectrum

Using the ground acceleration as input, a family of response spectrum curves can be generated for various levels of damping, where higher values of damping results in lower spectral response.

A multistory building will have as many modes of vibration as its degrees of freedom. The use of lumped mass model to present the actual distributed mass of a structure is a conventional tool for reducing the infinite degree of freedom of the structure to a manageable few. In multistory buildings it is generally sufficient to assume the masses as concentrated at the floor levels and to formulate the problem in terms of these masses. Since a multistory building has several degrees of freedom, in general it vibrates with as many different mode shapes and periods as its degrees of freedom. Each mode of vibration contributes to the base shear, and for elastic action of the structure, this base shear can be determined by multiplying an effective mass by an acceleration read from the response spectrum for the period of that mode and for the assumed damping. Each mode of vibration has its own characteristics frequency or period of vibration. The actual motion of tall building at any instant is a unique linear combination of its natural or principal modes of vibration. During vibration, the masses of the structure vibrate in phase with displacement as measured from its initial position.

There are three basic approaches used in the development of design spectra as given below:

1. The use of actual earthquake records.
2. The use of smoothed design spectra.
3. Use of unique design spectra reflecting the actual site conditions.

Response Spectrum for Actual Earthquake Records

The generation of a response spectrum curve can be idealized by subjecting a series of damped single degree of freedom mass spring systems with continuously varying natural periods to a given ground excitation. Response spectrum graphs are generated by numerical integration of actual earthquake records to determine maximum values for each period of vibration.

Spectral curves developed from actual earthquake records are quite jagged, being characterized by sharp peaks and troughs. Because the magnitude of this troughs and peaks can vary significantly for different earthquake records and because of the uncertainties of future earthquakes, it is wise to consider several possible earthquake spectra in the evaluation of the structural response for design purposes. Thus, if response to actual recorded earthquakes is to serve as a design basis, analysis should be performed using several selected spectra that are believed to be representative of critical ground motions that may occur at the site.

Smooth Design Spectra

As an alternative to use of several earthquake spectra for design, researchers have developed smooth design spectra that represent approximate upper-bound response envelopes based on critical level of ground motion. The sharp peak in earthquake records indicate the resonant behavior of the system when the natural period of the system coincides with the period of the forcing function, especially for the systems with little or no damping. however as can be seen from the spectra, even a moderate amount of damping has a tendency to smooth out the peaks and reduce the spectral response.

Unique Design Spectra

For specially important structures or where local soil conditions are not amenable to simple classification, the use of recommended smooth spectrum curves is inadequate for final design purpose. In such a case, site-specific studies are performed to determine more precisely the expected intensity and characteristics of seismic motion.

Structural Response

Since the earthquake causes the vibratory motion, which is cyclic about the equilibrium , the structural response is vibratory (dynamic) and it is cyclic about the equilibrium position of a structure. The fundamental natural frequency of most Civil Engineering structures lies in the range of 0.1 sec. to 3.0 sec. This is also the range of frequency content of earthquake generated ground motions.

2.4 SALIENT FEATURES OF BRACED FRAMES

Bracing is a highly efficient and economical method of resisting horizontal forces in a frame structure. A braced bent consists of the usual columns and girders, whose primary purpose is to support the gravity loading, and diagonal bracing members that are connected so that the total set of member from a vertical cantilever truss to resist the horizontal loading. The braces and girders act as the web members of the truss, while the columns act as the chords. Diagonal bracing is inherently obstructive to the architectural plan and can pose problems in the organization of internal space and traffic as well as in locating window and door openings. The most efficient, but also the most obstructive types of bracing are those that form a fully triangulated vertical truss. These includes the single diagonals, double diagonals, and K-braced types.

Some of the braced bent to allow window and doorś opening are shown in Figures 2.4 (c) and (d).

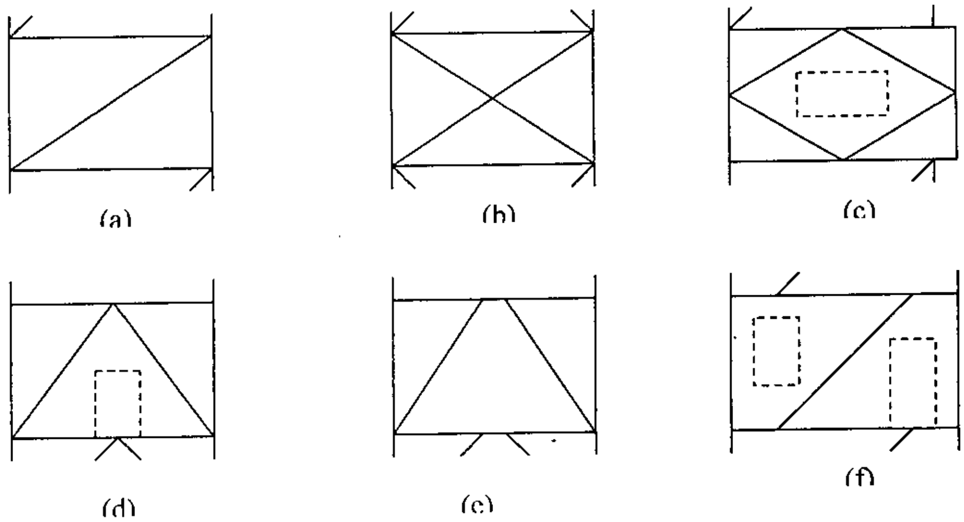


Fig. 2.4 Types of bracing

Bracés are subjected in turn to both tension and compression, because lateral loading on a building is reversible. consequently, they are usually designed for the more stringent case of compression. For this reason, bracing systems with shorter breaches, for example the K-types, may be preferred to the full-diagonal types. As an exception to designing braces for compression, the braces in the double diagonal systems are sometimes assumed to buckle in compression, and each diagonals is designed to carry in tension the full shear in the panel.

A significant advantage of the fully triangulated bracing types given in Figures 2.4 (a) to (f) is that the girder moments and shear are independent of the lateral loading on the structure. Consequently, the floor framing, which in this case is designed for gravity loading only, can be repetitive throughout the height of the structure with obvious economy in the design and construction.

In bracing systems in which the diagonals connect to the girder at a significant distance from the girder ends, for example, those in Figures 2.4 (c), (d) and (e) the girder can be designed more economically as continuous over the connection, thus helping to offset the cost of the bracing. A further advantage of this type of bracing system is that the braches, in having one or both ends connected to the beam, which is relatively flexible vertically; do not attract a significant load as the columns shorten under gravity loading.

Eccentric bracing system (i.e. system in which the braces are not concentric with the main joints) may be used to design a ductile structure for an earthquake resistant steel-framed building. The bracing acts in this usual elastic manner when controlling drift against wind or minor earthquakes. In the event of an overload during a major earthquake, the short link in the beam between the brace connection and the column in Fig. 2.4 (f) and the link in the beam between brace connections in Fig. 2.4 (e), serve as a "fuse" by deforming plastically in shear to give a ductile response of the structure. Such braced systems combine high elastic stiffness and a large inelastic energy dissipation capacity that can be sustained over many cycles.

The roles of the "web" members in resisting shear on a bent can be understood by following the path of the horizontal shear down the bent from story to story. Referring to Fig. 2.5 and considering four typical types of bracing subjected to the total external shear, the is neglecting the lesser effects of the horizontal forces applied locally at the floor levels, the vertical transmission of horizontal shear can be traced.

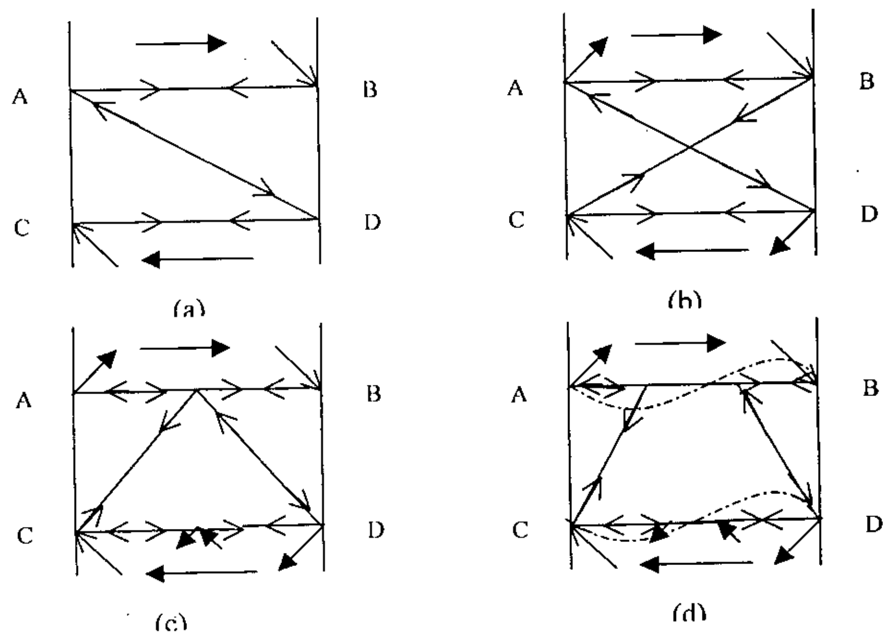


Fig. 2.5 Path of horizontal shear through web members. (a) Single diagonal bracing (b) Double- diagonal bracing (c) K-bracing (d) Story-height Knee bracing.

In Fig. 2.5 (a) the diagonal in each story is in compression, causing the beams to be in axial tension, therefore, the shortening of the diagonals and extension of the beams give rise to the shear deformation of the bent. In Fig. 2.5 (b) the forces in the braces connecting to each end are in equilibrium horizontally, with the beam carrying an insignificant axial load. In Fig. 2.5 (c) half of each beam is in compression and the other half in tension whereas in Fig. 2.5(d) the end parts of the beam are in compression and tension with the whole beam subjected to double curvature bending. With a reverse in the direction of the horizontal load on the structure the actions and deformations in each member of the bracing will also be reversed.

2.5 SALIENT FEATURES OF RIGID FRAME

A rigid-frame high rise structure typically comprises parallel or orthogonally arranged bents consisting of columns and girders with moment resisting joints.

Mainly the bending resistance of the girders, the columns, and their connections influences the horizontal stiffness of a rigid frame, by the axial rigidity of the columns. The accumulated horizontal shear above any story of a rigid frame is resisted by shear in the columns of that story. The shear in columns above and below is resisted by the attached girders, which also bend in double curvature, with points of contra flexure at approximately mid span. These deformations of the columns and girders allow racking of the frame and horizontal deflection in each story. The overall deflected shape of a rigid frame structure due to racking has a shear configuration with concavity up wind, a maximum inclination near the base, and a minimum inclination at the top. The advantage of a rigid-frame are the simplicity and convenience of its rectangular forms. Its unobstructed arrangement, clear of bracing members and structural walls, allows freedom internally for the layout and externally for the fenestration.

The overall moment of the external horizontal load is resisted in each story level by the couple resulting from the axial tensile and compressive force in the columns on opposite sides of the frame (Fig. 2.6). The extension and shortening of the columns cases overall bending and associated horizontal displacements of the structure. Because of the cumulative rotation up the height, the story draft due to overall bending increase with height, while that due to racking tends to decrease. Consequently the contribution to story drift from overall bending may, in the uppermost stories, exceed that from racking. The Contribution of overall bending to the total drift, however , will usually not exceed 10% of that of racking except in very tall, slender, rigid frames. Therefore the overall deflected shape of a high-rise rigid frame usually has a shear configuration (Smith and Coull, 1991).

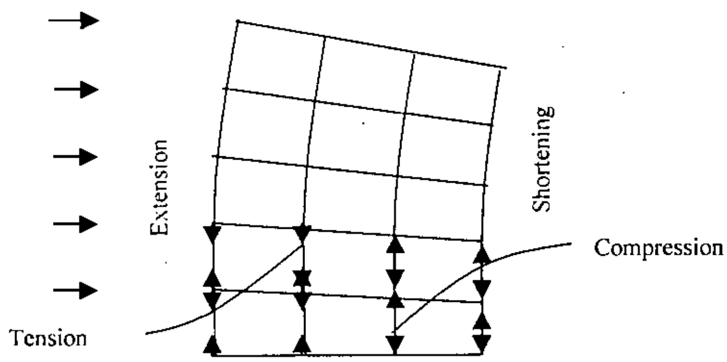


Fig. 2.6 Forces and deformations caused by external moment.

The response of a rigid frame to gravity loading differs from a simply connected frame in the continuous behavior of the girders. Negative moments are induced adjacent to the columns, and positive moments of usually lesser magnitude occur in the mid-span regions. The continuity also causes the maximum girder moments to be sensitive to the pattern of live loading. This must be considered when estimating the worst moment conditions. For example, the gravity load maximum hogging moment adjacent to an edge column occurs when live load acts only on the edge span and alternate other spans, as for (a) in Fig. 2.7. The maximum hogging moment adjacent to an interior column are caused, however, when live load acts only on the spans adjacent to the column, as for (b) in Fig. 2.7. The maximum mid-span sagging moment occurs when live load acts on the span under consideration, and alternate other spans, as for spans AB and CD in Fig. 2.7 (a).

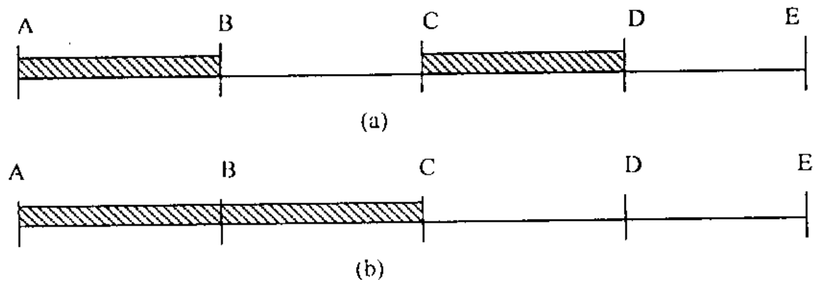


Fig. 2.7 (a) Live load pattern for maximum positive moment in AB and CD and maximum negative moment at A (b) Live load pattern for maximum negative moment at B.

The dependence of a rigid frame on the moment capacity of the columns for resisting horizontal loading usually causes the columns of a rigid frame to be larger than those of the corresponding fully braced simply connected frame. On the other hand, while girders in braced frames are designed for their mid-span sagging moment, girders in rigid-frames are designed for the end-of-span resultant hogging moments, which may be of lesser values. Consequently, girders in a rigid frame may be smaller than in the corresponding braced frame. Such reductions in size allow economy through the lower cost of the girders and possible reductions in story heights. These benefits may be offset, however, by the higher cost of the more complex rigid connections.

2.6 SALIENT FEATURES OF INFILLED FRAME

The infilled frame consists of a reinforced concrete or steel column-and-girder frame with infills of brickwork or concrete blockwork (Fig. 2.8). In addition to functioning as partitions, exterior walls, and walls around stair, elevator, and service shafts, the infill may also serve structurally to brace the frame against horizontal loading (Smith and Coull, 1991). Out of all infill systems, masonry is the most popular form of infill for multi-stored structures with reinforced concrete or steel frames in our country. For at least 3500 years, clay masonry has been particularly noted for its attractive appearance, long life and good load bearing qualities. When properly constructed and detailed it provides one of the most functional walling systems ever developed.

By definition, masonry is a composite material consisting of masonry units set in mortar. Because the units and mortar have different characteristics, masonry exhibits distinct directional properties with potential planes of weakness being created by the low tensile strength at each unit/mortar interface. For resistance to wind and earthquake forces it is this bond strength at the interface that is important, in both flexure and shear.

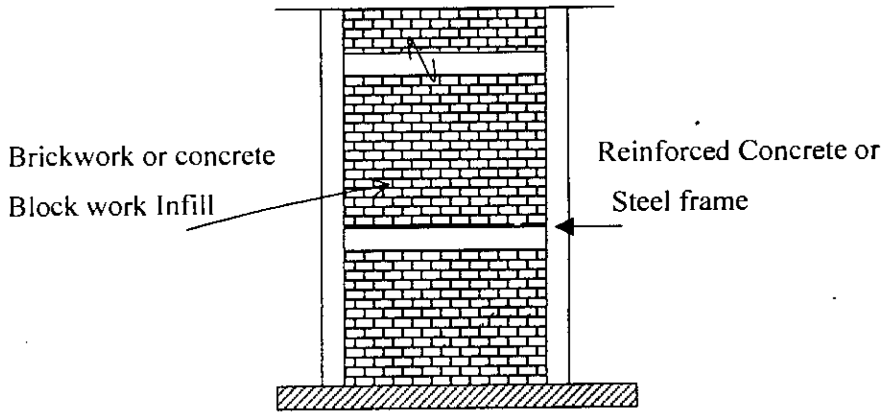


Fig. 2.8 Structural Frame infilled with masonry.

According to Lawrence and Page (1999), unreinforced masonry infill panels have the potential to add considerably to the strength and rigidity of a framed structure if they are designed and detailed for composite action. Interaction between infill and frame depends on the contact area at the interface of the two components. The extent of composite action will depend on the level of lateral load, the degree of bond or anchorage at the interfaces, and geometric and stiffness characteristics of the frame and infill masonry.

In a framed structure, load is transferred from the face-loaded walls to the framing members through their connections. The framing members then act together to resist the lateral force by sway action or braced-truss action, thereby transferring the force to the foundations. When the frame is subjected to a horizontal loading, it deforms with double curvature bending of the Columns and girders. The translation of the upper part of the column in each story and the shortening of the leading diagonal of the frame cause the column to lean against the wall as well as to compress the wall along its diagonal. It is roughly analogous to a diagonally braced frame shown in Fig. 2.9 (Smith and Coull, 1991)

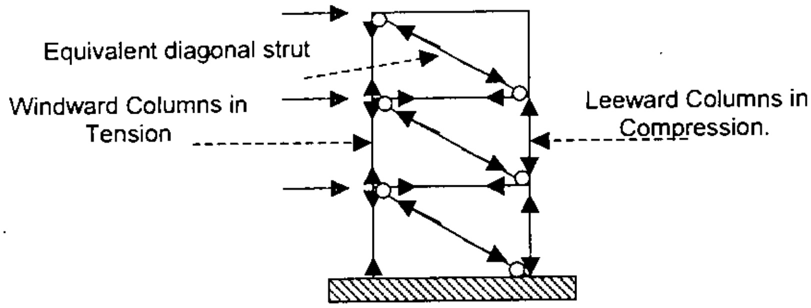


Fig. 2.9 Analogous braced frame.

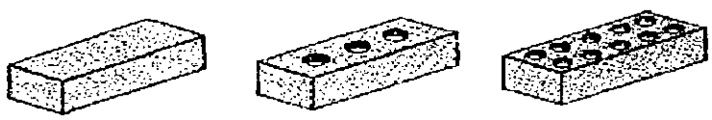
The load carrying capacity of unreinforced masonry wall panels depends upon the dimensions and support conditions, the level of compressive stress in the wall and the tensile strength of the masonry. The presence of door and window openings also has a strong influence on the behavior. In this part of the thesis work, detailed of the infill materials and as well as the infill property will discussed.

2.6.1 Clay Brick

In this study clay brick was considered as infill material because of its availability and economic aspect. Masonry bricks have been utilized for structures since the earliest days of mankind. The masonry units considered in this study are solid clay bricks. Masonry units are available in a variety of sizes, shapes, colors, and textures.

Solid Clay Brick Units

A solid clay masonry unit (Fig. 2.10), as specified in ASTM C 62 (1994) (Specification for Building Brick, Solid Masonry Units Made From Clay or Shale) is a unit whose net cross-sectional area, in every plane parallel to the bearing surface, is 75% or more of its gross cross-sectional area measured in the same plane. A solid brick may have a maximum coring of 25%.



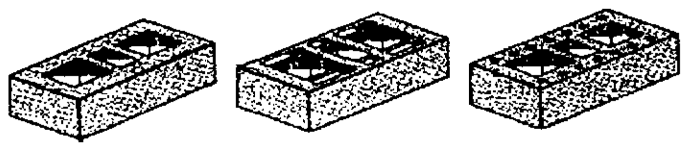
No void. Void 25% or less of the cross sectional area

Fig. 2.10 Solid Clay brick

Solid clay units are specified in ASTM C 62 (1994) and in UBC Standard 21-1 (1994). Building bricks are classified as solid masonry units used where appearance is not a consideration.

Hollow Clay Brick Units

A hollow clay brick unit as specified in UBC Standard 21-1 (1994) and ASTM C 652 (1994), is a unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75% of its gross cross- sectional area measured in the same plane.



Solid shell hollow Brick units. | Double shell hollow Brick units | Cored shell hollow brick units.

Fig. 2.11 Hollow Clay Brick.

General

Bricks are graded according to their weathering resistance. According to ASTM C 62 (1994) and C 216 (1994) the Bricks grade given in below.

GRADE SW (Severe Weathering) Bricks are intended for use where a high and uniform degree of resistance to frost action and disintegration by weathering is desired and the exposure is such that the brick may freeze when permeated with water.

GRADE MW (Moderate Weathering) bricks are used where they will be exposed to temperatures below freezing, but unlikely to be permeated with water, and where a moderate and somewhat non-uniform degree of resistance to frost action is permissible.

GRADE NW (Negligible Weathering) applies to building brick only and is intended for use in backup or interior masonry.

The physical requirements for each grade of solid and hollow brick are compressive strength, water absorption and the saturation coefficient as shown in Table 2.1.

TABLE 2.1 Physical Requirements Solid and Hollow Bricks. (ASTM C 62, C 216 or C 652)

Designation	Minimum Compressive strength (Brick flat wise), psi. Gross area		Maximum water absorption by 5 hour boiling per cent		Maximum saturation co efficient	
	Average of 5 bricks	Individual	Average of 5 bricks	Individual	Average of 5 bricks	Individual
Grade SW	3000	2500	17.0	20.0	0.78	0.80
Grade MW	2500	2200	22.0	25.0	0.88	0.90
Grade NW	1500	1250	no limit	no limit	no limit	no limit

NOTE:

1. Based on ASTM C 62, C 216 or C 652.
2. The saturation coefficient or C/B ratio. is the ratio of absorption by 24- hours submersion in cold water to that after 5-hour submersion in boiling water

2.6.2 Mortar

General

By definition, masonry is a composite material consisting of masonry brick units set in mortar. Mortar is a plastic mixture of materials used to bind masonry brick units into a structural mass. Mortar is an important ingredient in masonry construction because its characteristics have a strong influence on both the strength and durability of the masonry assemblage. It is also the component most susceptible to site problems related to mixing and batching. Mortar must be workable when wet and have sufficient strength and be adequately bonded to the masonry units when set. The tensile bond strength of masonry can vary from zero to more than 1.0 MPa depending on the correct match of mortar and unit properties. Selection of sand, cement, mix composition and admixtures such as air entrained (when appropriate) are of vital importance for the achievement of the required tensile bond strength.

Mortar is used for the following purposes:

i) It serves as a bedding or seating material for the masonry units.

ii) It allows the units to be leveled and properly placed.

iii) It bonds the units together.

iv) It provides compressive strength.

v) It provides shear strength, particularly parallel to the wall.

vi) It allows some movement and elasticity between units.

vii) It seals irregularities of the masonry units.

viii) It can provide color to the wall by using color additives.

ix) It can provide an architectural appearance by using various types of joints.

Historically, mortar has been made from a variety of materials. Plain mud, clay, earth with ashes, and sand with lime mortars have all been used. Modern mortar consists of cementitious materials and well graded sand.

Types of Mortar

The requirements for mortar are provided in ASTM *C 270* (1994), *Mortar for Unit Masonry* and in UBC Standard *No. 2.1* – 15 (1994), *Mortar for Unit Masonry and Reinforced Masonry Other Than Gypsum.* There were originally five types of mortar which were designated as M, S, N, O, and K. The types are identified by every other letter of the word MaSoNwOrK. Type K is no longer referred to in the Uniform Building Code or in ASTM C 270 (1994).

Selection of Mortar Types

The performance of masonry is influenced by various mortar properties such as workability, water retentivity, bond strength, durability, extensibility, and compressive strength. Since these properties vary with mortar type, it is important to select the proper mortar type for each particular application. Tables 2.2 and 2.3 are general guides for the selection of mortar type. Selection of mortar type should also consider all applicable building codes and engineering practice standards.

TABLE 2.2 Mortar Types for Classes of Construction. (ASTM C 270)

ASTM Mortar Type Designation	Construction Suitability
M	Masonry subjected to high compressive loads, severe frost action, or high lateral loads from earth pressures, hurricane winds, or earthquakes. Structures below or against grade such as retaining walls, etc.
S	Structures requiring high flexural bond strength, and subject to compressive and lateral loads.
N	General use in above grade masonry. Residential basement construction, interior walls and partitions. Masonry veneer and non-structural masonry partitions.
0	Non-load-bearing walls and partitions. Solid load bearing masonry with an actual compressive strength not exceeding 100 psi and not subject to weathering.

TABLE 2.3 Guide for the Selection of Masonry Mortars. (ASTM C 270)

Location	Building Segment	Mortar Type	
		Rec.	Alt.
Exterior, above grade	load bearing wall , non load bearing wall, parapet wall.	N 02 N	S or M N or S S
Exterior, at or below grade	Foundation wall, retaining wall, manholes sewers, pavements, walks and patios.	*S3*	M or N3
Interior	Load-bearing wall.	N	S or M
	Non-bearing Partitions.	02	N

NOTE:

1. This table does not provide for many specialized mortar uses, such as chimney, reinforced masonry, and acid-resistant mortars.
2. Type O mortar is recommended for use where the masonry is unlikely to be frozen when saturated or unlikely to be subjected to high winds or other significant lateral loads. Type N or S mortar should be used in other cases.
3. Masonry exposed to weather in a nominally horizontal surface is extremely vulnerable to weathering. Mortar for such masonry should be selected with due caution.
4. Based on ASTM C 270, Table X 1.1. Rec. = Recommended, Alt. = Alternative.

Specifying Mortar

Mortar may be specified by either property or proportion specifications

Property Specifications

Property specifications are those in which the acceptability of the mortar is based on the properties of the ingredients (materials) and the properties (water retention, air content, and compressive strength) of samples of the mortar mixed and tested in the laboratory. Property specifications are used for research so that the physical characteristics of a mortar can be determined and reproduced in subsequent tests. The property requirements for mortar are given in Table 2.4

TABLE 2.4 Property Specifications for Mortar (ASTM C 270)

Mortar	Type	Avg. Comp. Strength at 28 day Min.	Water Retention, Min	Air Content% Max.%	Aggregate Ratic (measured in. damp, loose conditions)
Cement lime	M	2500	75	12	
	S	1800	75	12	
	N	750	75	142	
	O	350	75	142	
Masonry cement	M	2500	75	3	Not less than 2 ¼ and not more than 3 ½ times the sum of the separate volume of cementitious materials.
	S	1800	75	3	
	N	750	75	3	
	O	350	75	3	

NOTE:

1. Laboratory-prepared mortar only.
2. When structural reinforcement is incorporated in cement lime mortar, the maximum air content shall be 12 percent.
3. When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall be 18 percent.

Compressive strength is usually the only property or characteristic which a specifier who is not a researcher would require. Two methods are used to determine the compressive strength of mortar. The first method tests 2" cubes of mortar in compression after having cured for 28 days. The second method, based on UBC (1994) Standard 21-16 *Field Tests Specimens for Mortar,* uses mortar specimens 50 mm in diameter by 100 mm high. These cylinders must have a minimum compressive strength of 1035 N/cm². Although no qualification is made for the age of the compression test cylinders, it may be assumed as 28 days. Table 2.5 is a comparison of the equivalent strength between cylinders and cube specimens for three types of mortar.

TABLE 2.5 Compressive Strength of Mortar (psi) (ASTM C 270)

Mortar type	2'' dia x 4'' high Cylinder specimen	2'' cube specimen
M	2100	2500
S	1500	1800
N	625	750

NOTE:

1. Lesser periods of time for testing may be used provided the relation between early tested strength and the 28-day strength of the mortar is established.

The field strength of mortar should be used only as a quality control test, rather than a quantification evaluation. The in-place mortar strength can be much higher than the test values. The higher in-place strength is attributed to the inherent difficulty in failing the thin and wide (l/4" to 5/4" high by 11/4" to 4" wide) mortar joints in compression. Additionally, the masonry units above and below the mortar joint, as well as the grout, confine the mortar so that the in-place mortar strength is much higher than the strengths of the test specimens. NCMA TEK 107 Laboratory and Field Testing of Mortar and *Grout* and Fig. 2.12 dramatically show that a 3/8" to 5/8" mortar specimen has a strength far exceeding the strength of the 2" test specimens.

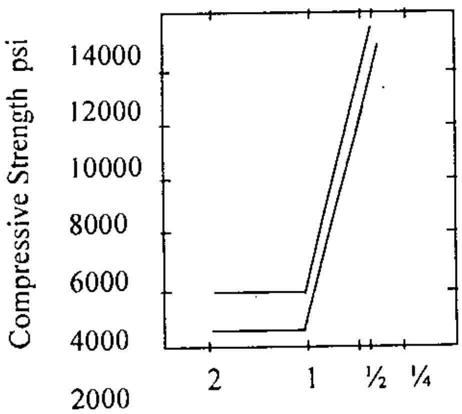


Fig. 2.12 Effect of specimen thickness on compressive strength (NCMA TEK 107)

Because the in-place mortar strength exceeds the cube and cylinder test strengths, mortar will perform well even when tests on mortar are less than the specified strength of the mortar specimens. Additionally, because the in-place strength is quite high, mortar performs well even when the compressive strength of the entire masonry assemblage, f'_m, is higher than the cylinder and cube strengths. In fact, the National Concrete Masonry Association conducted a research project in which the apparent strength of mortar ranged from 500 psi to 3500 psi and the apparent strength of the wall increased only 4%, *(NCMA* TEK *NOTE* 15 (1969) *Compressive Strength of Concrete Masonry).* NCMA TEK *NOTE 70* (1975) *Concrete Masonry Prism Strength* indicates that the compressive strength of masonry structures built with type S or M mortar may be as much as 40% higher than masonry prisms built with type N Mortar. Additionally, tests by the Brick Institute of America indicate that the effect on the prism strength using type M mortar instead of type N may be as high as 34%. Conservatively. Besides compressive strength requirements, it is suggested that the bond shear strength be investigated, particularly where lateral forces from winds or earthquakes must be considered.

The Office of the State Architect of California specifies that the bond shear strength between brick and mortar should be a minimum of 20 psi, after 14 days of curing. The actual bond strength in the wall is usually many times higher than the required 20 psi, since the vertical load on the wall and the development of shear friction also counteract the shear forces. Materials can also greatly affect bond strength, and the time lapse between spreading mortar and placing the masonry unit should be kept to a minimum since the bond of the mortar will be reduced by a long delay in placing the units.

2.6.3 Verification of the Specified Strength, f'_m

The required or specified value, f'_m is used as the basis for structural engineering design and must be obtained or verified in accordance with prescribe code requirements. The Uniform Building Code (1994) has provided the following three methods to verify the specified strength of the masonry assembly, f'_m

a) *Masonry Prism Testing* - UBC Section 2105.3.2

b) *Masonry Prism Test Records* - UBC Section 2105.3.3

c) *Unit Strength Method* - UBC Section 2105.3.4

Selection of f'_m from Code Tables

The specified compressive strength of masonry, f'_m may be selected from Tables 2.6 and 2.7 that are based on the strength of the masonry unit and mortar used. These tables are conservative and higher values may be obtained by conducting prism tests. In order to use full allowable stresses, the masonry units and grout must be tested prior to construction.

TABLE 2.6 Compressive Strength of Masonry Based on the Compressive Strength of the clay Masonry units and type of mortar used in construction. . (ACI / ASCE / TMS Table 1.6. 2.1)

Net area Compressive strength of clay masonry units in psi.		Net area Compressive strength of masonry in psi , f'_m
Type **M** or **S** mortar	Type **N** mortar	
2400	3000	1000
4400	5500	1500
6400	8000	2000
8400	10500	2500
10400	13000	3000
12400	---	3500
14400	---	4000

NOTE:

1. ACI / ASCE / TMS Table 1.6. 2.1

TABLE 2.7 Specified Compressive Strength of Masonry f'_m (psi), based on specifying the compressive strength of masonry units. (ACI 530.1-92/ASCE 6-92/TMS 602-92)

Compressive Strength of Clay Masonry Units (Psi)	Specified Compressive Strength of Masonry, f'_m	
	Type M or S Mortar (Psi)	Type N mortar (Psi)
14,000 or more	5,300	4,400
12,000	4,700	3,800
10,000	4,000	3,300
8,000	3,350	2,700
6,000	2,700	2,200
4,000	2,000	1,600
Compressive Strength of Concrete Masonry Units (Psi)	**Specified Compressive Strength of Masonry, f'_m**	
	Type M or S mortar (Psi)	Type N mortar (Psi)
4,800 or more	3,000	2,800
3,750	2,500	2,350
2,800	2,000	1,850
1,900	1,500	1,350

NOTE:

1. Compressive strength of solid clay masonry units is based on gross area. Compressive strength of hollow clay masonry units is based on minimum net area. Values may be interpolated.
2. Assumed assemblage. The specified compressive strength of masonry f'_m, is based on gross area strength when using solid units or solid grouted masonry and net area strength when using ungrouted hollow units.
3. Mortar for unit masonry, proportion specification. These values apply to Portland cement- lime mortars without added air-entraining materials.
4. Values may be interpolated. In grouted concrete masonry the compressive strength of grout shall be equal to or greater than the compressive strength of the concrete masonry units.

ACI 530.1-92/ASCE 6-92/TMS 602-92 also provides tables for the selection of f'_m based on the strength of the masonry unit and type of mortar used as shown in Tables 2.6 and 2.7 (ACI/ASCE/TMS, (1990) Tables 1.6.2.1 and 1.6.2.2).

2.6.4 Modulus of Elasticity, E_m

General

The physical measure of a material to deform under load is called the modulus of elasticity, E_m. It is the ratio of the stress to the strain of a material or combination of materials as is the case for grouted masonry. Originally, E_m for masonry was the same as for concrete, namely 1000 f'_c or for masonry $1000 f'_m$, This value changed for concrete in the UBC (1967) 33 w $^{1.5}$ $(f'_c)^{0.5}$ to reflect the influence of the unit weight of concrete and the curvature of the stress strain curve.

The value for masonry assemblies was maintained as $E_m = 1000 f'_m$ until 1988 when it was changed to $750 f'_m$. This change recognized that masonry is not as stiff as concrete and has a lower modulus. However, no accommodation was made to further define the E_m based on weight, strength or volume of component materials. Thomas Holm, 1978, of the Solite Corporation, has suggested the equation, $E_m = 22\ w^{1.5} (f'_m)^{0.5}$, to reflect the influence of light weight masonry and the strength of the assembly. Similarly, the Colorado Building Code has recognized that clay masonry has a lower *Em* and thus uses $500 f'_m$ as the modulus of elasticity of clay masonry (Amerhein, 2000).

When using the ACI/ASCE/TMS Masonry Code (1990), the modulus of elasticity is given in the Tables 2.8 and 2.9 (ACI/ASCE/TMS (1990) Specification Tables 1.6.2.1 and 1.6.2.2.)

TABLE 2.8 Specified Compressive Strength of Clay Masonry Assemblages f'_m (psi) ACI/ASCE/TMS (1990)

Compressive Strength of Clay Masonry Units (Psi)	Specified Compressive Strength of Masonry , f'_m	
	Type M or S Mortar (Psi)	Type N Mortar (Psi)
14,000 or more	5,300	4,400
12,000	4,700	3,800
10,000	4,000	3,300
8,000	3,350	2,700
6,000	2,700	2,200
4,000	2,000	1,600

NOTE:

1. Compressive strength of solid masonry units is based on the gross area. Compressive strength of hollow clay masonry units is based on minimum net area. Values may be interpolated.
2. Assumed assemblage. The specified compressive strength of masonry, f'_m, is based on gross area strength when using solid units or solid grouted masonry and net area strength when using ungrouted hollow units.
3. Mortar for unit masonry, proportion specification, as specified in UBC. These values apply to Portland cement-lime mortars without added air-entraining materials.

TABLE 2.9 Clay Masonry f'_m , E_m, n and G values Based on Clay Masonry unit Strength and the Mortar Type. ACI/ASCE/TMS (1990)

Type N Mortar				
Compressive Strength of Clay Masonry (Psi)	Specified Compressive Strength of Clay Masonry Assem-blage f'_m (Psi)	Modulus of Elasticity $E_m = 750\ f'_m$ (psi) E_m maximum = 3,000,000 Psi.	Modular ratio $n = E_s / E_m$ Where E_s=29,000,000 (Psi)	Modulus of Rigidity $\sigma = 0.40\ E_m = 300 f'_m$ (Psi) G (max) = 1,200,000 (psi)
14,000 or more	4,400	3,000,000	9.7	1,200,000
12,000	3,800	2,850,000	10.2	1,140,000
10,000	3,300	2,475,000	11.7	990,000
8,000	2,700	2,025,000	14.3	810,000
6,000	2,200	1,650,000	17.6	660,000
4,000	1,600	1,200,000	24.2	480,000
Type M or S Mortar.				
Compressive Strength of Clay Masonry (Psi)	Specified Compressive Strength of Clay Masonry Asse-mblage f'_m(Psi)	Modulus of Elasticity $E_m = 750\ f'_m$ (Psi) E_m maximum = 3,000,000 Psi.	Modular ratio $n = E_s / E_m$ Where E_s=29,000,000 Psi	Modulus of Rigidity $\sigma = 0.40\ E_m = 300 f'_m$ (Psi) G (max) = 1,200,000 (psi)
14,000 or more	5,300	3,000,000	9.7	1,200,000
12,000	4,700	3,000,000	9.7	1,200,000
10,000	4,000	3,000,000	9.7	1,200,000
8,000	3,350	2,512,500	11.5	1,005,000
6,000	2,700	2,025,000	14.3	810,000
4,000	2,000	1,500,000	19.3	600,000

NOTE:

1. Compressive strength of solid Masonry units is based on the gross area. Compressive strength of the hollow clay masonry units is based on minimum net area. Values may be interpolated.

After found out the property of the masonry unit, now the property of the infill components have been discussed below.

2.6.5 Equivalent Strut Width

In-plane strength predictions of infilled frames are a complex, statically indeterminate problem. The strength of a composite-infilled frame system is not simply the summation of the infill properties plus those of the frame. Great efforts have been invested, both analytically and experimentally, to better understand and estimate the composite behavior of masonry-infilled frames. Polyakov (1960) (work dating back to the early 1950s), Stafford-Smith (1962, 1966, 1969), Mainstone (1971), Klingner and Bertero (1976, 1978), to mention just a few, formed the basis for understanding and predicting infilled frame in-plane behavior. Their experimental testing of infilled frames under lateral loads resulted in specimen deformation shapes similar to the one illustrated in Fig. 2.13.

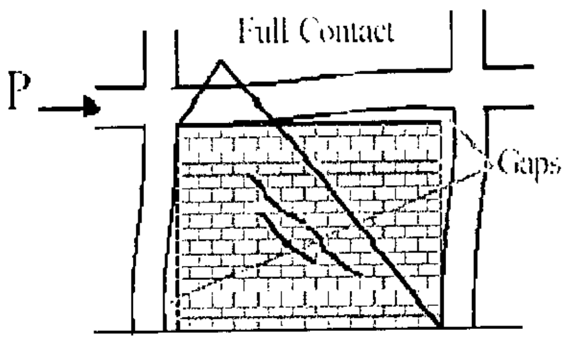


Fig. 2.13 Specimen Deformation Shape

During testing of the specimens, diagonal cracks developed in the center of the panel, and gaps formed between the frame and the infill in the nonloaded diagonal corners of the specimens, while full contact was observed in the two loaded diagonal corners. This behavior, initially observed by Polyakov, lead to a simplification in infilled frame analysis by replacing the masonry infill with an equivalent compressive masonry strut as shown in Fig. 2.14. The equivalent masonry strut of width, a, with same net thickness and mechanical properties (such as the modulus of elasticity E_m) as the infill itself, is assumed to be pinned at both ends to the confining frame.

The evaluation of the equivalent width, a, varies from one reference to the other. The most simplistic approaches presented by Paulay and Priestley (1992) and Angel et al.

(1994) have assumed constant values for the strut width, a, between 12.5 to 25 percent of the diagonal dimension of the infill, with no regard for any infill or frame properties. Stafford-Smith and Carter (1969), Mainstone (1971), and others, derived complex expressions to estimate the equivalent strut width, a, that consider parameters like the length of contact between the column/beam and the infill, as well as the relative stiffness of the infill to the frame.

The expressions used in this chapter have been adopted from Mainstone (1971) and Stafford-Smith and Carter (1969) for their consistently accurate predictions of infilled frame in-plane behavior when compared with experimental results (Mainstone, 1971; Stafford-Smith and Carter, 1969; Klingner and Bertero, 1978; and Al-Chaar, 1998). The masonry infill panel will be represented by an equivalent diagonal strut of width, a, and net thickness, t_{eff}, as shown in Fig. 2.15.

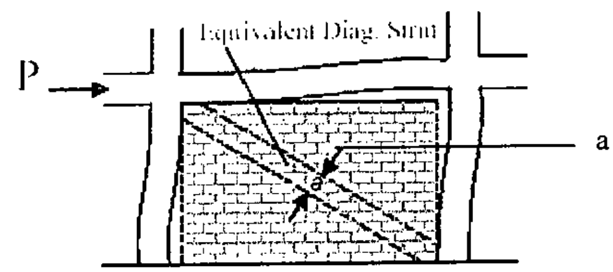


Fig. 2.14. Equivalent Diagonal Strut

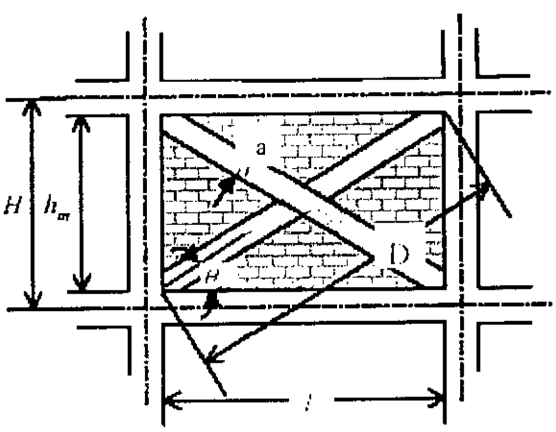


Figure 2.15 Strut Geometry.

The equivalent strut width, a, depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness shall be evaluated using Eq. 2.1 (Stafford-Smith and Carter, 1969):

$$\lambda_I H = H\,[\,(E_m\, t \sin 2\theta) / (4\, E_c\, I_{col}\, h_w)\,]^{1/4} \qquad \text{Eq-2.1}$$

Where t is the thickness of the masonry wall.

Using this expression, Mainstone (1971) considers the relative infill-to-frame flexibility

in the evaluation of the equivalent strut width of the panel as shown in Eq.2.2

$$a = 0.175D(\lambda_I H)^{-0.4} \qquad \text{Eq. 2.2}$$

If there are openings present, existing infill damage, and/or FRP overlay, however, the equivalent strut width must be modified using Eq.2.3

$$a_{mod} = a(R_1)_i\,(R_2)_i\,\xi_I \qquad \text{Eq. 2.3}$$

Where:

$(R_1)_i$ = reduction factor for in-plane evaluation due to presence of openings (Eq 2.5)

$(R_2)_i$ = reduction factor for in-plane evaluation due to existing infill damage (Table 2.10)

ξ_I = Strength increase factor due to presence of FRP overlay.

Although the expression for equivalent strut width given by Equation 2.3 was derived to represent the elastic stiffness of an infill panel, this document will extend its use to determine the ultimate capacity of infilled structures. The strut will be assigned strength parameters consistent with the properties of the infill it represents. A nonlinear static procedure, commonly referred to as a pushover analysis, will be used to determine the capacity of the infilled structure.

Eccentricity of Equivalent Strut

The equivalent masonry strut is to be connected to the frame members as depicted in Fig. 2.16, where the bold double-sided arrow represents the location of the strut in the structural model. The infill forces are assumed to be mainly resisted by the columns, and the struts are placed accordingly. The strut should be pin-connected to the column at a distance l_{column} from the face of the beam. This distance is defined in

Equations 2.4 and 2.5 and is calculated using the strut width, a, without any reduction factors.

$l_{column} = a / \cos\ \theta_{column}$ Eq-2.4

$\tan \theta_{column} = \{ h_m - (a / \cos \theta_{column})\} / l$ Eq-2.5

Using this convention, the strut force is applied directly to the column at the edge of its equivalent strut width, *a*. Figure 2.16 illustrates this concept.

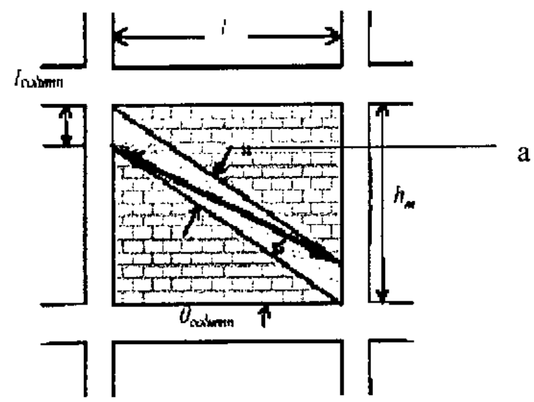


Fig. 2.16 Placement of Strut.

Partially Infilled Frames

In the case of a partially infilled frame, the reduced column length, l_{column}, is equal to the unbraced opening length for the windward column, while l_{column} for the leeward column is defined as usual (Fig. 2.17). The strut width should be calculated from Equation 2.2, using the reduced infill height for h_m in Equation 2.1. Furthermore, the only reduction factor that should be taken into account is $(R_2)i$, which accounts for existing infill damage.

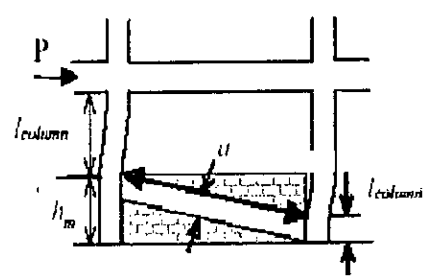


Fig. 2.17 partially infilled frame

Perforated Panels

In the case of a perforated masonry panel, the equivalent strut is assumed to act in the same manner as for the fully infilled frame. Therefore, the eccentric strut should be placed at a distance l_{column} from the face of the beam as shown in Figure 2.18. The equivalent strut width, a, shall be multiplied, however, by a reduction factor to account for the loss in strength due to the opening. The reduction factor, $(R_I)_i$, is calculated using Eq.2.6

$$(R_I)_I = 0.6\,(A_{open}/A_{panel}) - 1.6\,(A_{open}/A_{panel}) + 1 \qquad \text{Eq. 2.6}$$

Where:

A_{open} = Area of the opening (in^2)
A_{panel} = Area of the infill panel (in^2) = $l \times h_m$

Note: If the area of the opening (A_{open}) is greater than or equal to 60 percent of the area of the infill panel (A_{panel}), then the effect of the infill should be neglected, i.e, $(R_I)_i = 0$

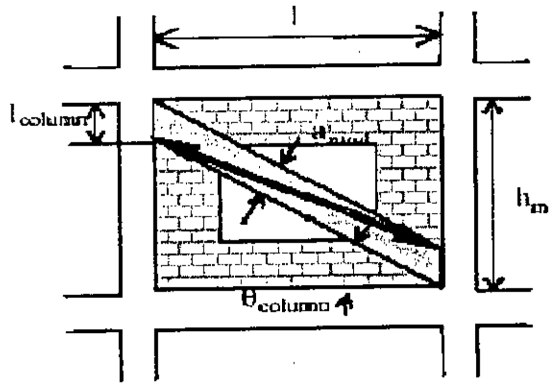


Fig. 2.18 perforated panel

It should be noted that reducing the strut width to account for an opening does not necessarily represent the stress distributions likely to occur. This method is a simplification in order to compute the global structural capacity. Local effects due to an opening should be considered by either modeling the perforated panel with finite elements or using struts to accurately represent possible stress fields as shown in Fig. 2:19.

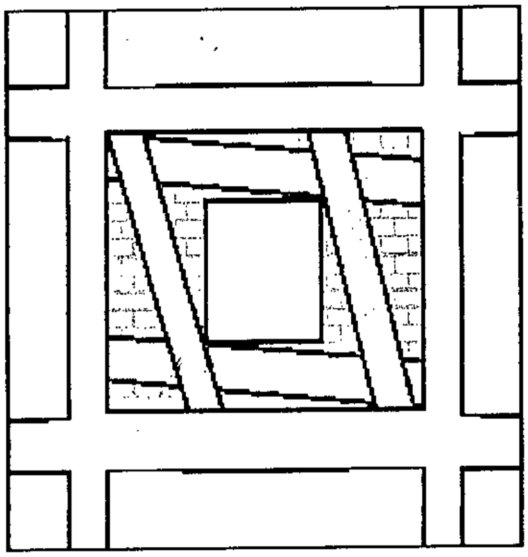


Fig. 2.19 possible strut placement for perforated panel

Existing Infill Damage

Masonry infill panel behavior deteriorates as the elastic limit is exceeded. For this reason, it is important to determine whether the masonry in the panel has exceeded the elastic limit and, if so, by how much. The extent of existing infill damage can be determined by visual inspection of the infill. Existing panel damage (or cracking) must be classified as either: no damage, moderate damage, or severe damage as presented in Fig. 2.21. If in doubt as to the magnitude of existing panel damage, assume severe damage for a safer (conservative) estimate. A reduction factor for existing panel damage ($R_2)i$ must be obtained from Table 2.10. Note that, if the slenderness ratio (hm/t) of the panel is greater than 21, ($R_2)i$ is not defined and repair is required. For panels with no existing panel damage, the reduction factor ($R_2)i$ must be taken as 1.0.

TABLE 2.10 in-Plane Damage reduction Factor

	$(R_2)_i$ for Type of Damage	
h_m/t	Moderate	Severe
≤ 21	0.7	0.4
> 21	Requires repair.	

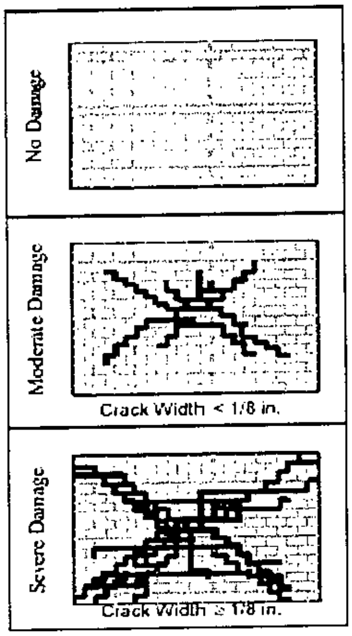


Fig. 2.20 Visual Damage Classification.

Masonry infill crushing strength

The masonry infill crushing strength corresponds to the compressive load that the equivalent masonry strut can carry before the masonry is crushed (R_{cr}). The applied load that corresponds to the crushing strength of the infill is evaluated using Equation 2.7..

$$R_{cr} = a_{mod}\, t_{eff}\, f'_m \qquad \text{Eq.- 2.7}$$

Where:
f'_m = Compressive strength of the masonry (ksi)
t_{eff} = Net thickness of the masonry panel (in).

CHAPTER 3

MODELING OF INFILLED FRAME

3.1 GENERAL

The task of structure modeling is arguably the most difficult one facing the structural analyst, requiring critical judgment and a sound knowing of the structural behavior of the tall building components and assemblies.

3.2 ASSUMPTIONS

An attempt to analyze a high-rise building and account accurately for all aspects of behavior of all the component and materials, even if their size and properties were known, would be impossible. Simplifying assumptions are necessary to reduce the problem. The most common assumptions are as follows.

Materials

The material of the structure and the structural components are linearly elastic. This assumption follows the superposition of forces and deflections and, hence the use of linear methods of analysis. The development of linear methods and their solution by computer have made it possible to analyze large complex statically indeterminate structures.

Components Properties

The effects of secondary structural components and nonstructural components are usually assumed to be negligible and conservative. Although this assumption is generally valid, exceptions occur. The effects of heavy cladding may be not negligible and may significantly stiffen a structure. Similarly masonry infill may significantly change the behavior and increase the forces un-conservatively in a surrounding frame.

Negligible Stiffness

Components stiffness of relatively small magnitude is assumed negligible. The use of this assumption should be dependent on the role of the components in the structures behavior. For example, the contribution of a slabs bending resistance to the lateral load resistance of a column and beam rigid frame structure is negligible, whereas its contribution to the lateral resistance of a flat plate structure is vital and must not be neglected.

3.3 MODELING OF THE FRAME

It is necessary for the intermediate and final stage of design to obtain reasonably accurate estimate of the structures deflection and its member forces. With the wide availability of structural analysis programs and powerful computers it is now possible to solve very large and complex structural models easily.

The major structural analysis programs, typically offer a variety of finite elements for structural modeling. A number of good finite element analysis computer packages are available in the Civil Engineering field. They vary in degree of complexity, usability and versatility. The name of such packages are- STAAD, ABAQUAS, ADINA, ANSYS, DIANA, SAP90. STAAD is one of the powerful and versatile package available for any kind of structural analysis, which has been used for analysis in this study.

In this study, the frames are RCC frames. At first only the bare frame effect is considered, after that the panels are considered infilled by masonry brick units. All panels are not considered infilled at a time. The different parameters that were considered in this study given below,

Z Seismic zone coefficient 0.15

1 Structural importance coefficient 1.0

R Response modification coefficient 12

Soil Type S2

Component Properties

For the convenience of modeling, the infill walls are considered as strut members.(Smith and Coull, 1991) To find out the property of the equivalent strut, it requires to determine the following properties of the structural and non structural items-

a) Modulus of elasticity of concrete E_c value for the column and beam materials.

b) Sectional property (ie. Depth, Width, Moment of inertia, centroid) of the column and beam.

c) Equivalent width of the masonry infill strut "a" (Fig. 3.1).

d) f'_m, compressive strength of the masonry assemble units.

e) E_m, modulus of elasticity of the masonry unit.

In this study, the frames having 4, 6, 9 and 12 story have been studied. The properties of the frame components are given in Table 3.1.

Table 3.1 Properties of the RC Frame Components

No. of Story	Foundation nature	Concrete compressive Strength.	Modulus of Elasticity in ksi	Height		Properties			
				Floor to G.Beam inch	All floors above G.F. inch	Columns		Beams	
						Section	I (in^4)	Section (in)	I(in^4)
4	Fixed support	3500 psi	3587	62.97	118.08	12 x 12	1728	10 x15.75	3255.8
6	Fixed support	3500 psi	3587	62.97	118.08	12 x 12	1728	10 x15.75	3255.8
9	Fixed support	3500 psi.	3587	62.97	118.08	12 x 15	3375	10 x 15.75	3255.8
12	Fixed support	3500 psi.	3587	62.97	118.08	15 x 15	4218.7	10 x 15.75	3255.8

Determination of f'm and Em

In this study masonry bricks are considered as infill material because it is widely used in Bangladesh. Masonry clay bricks are mainly of solid and hollow block types. Solid masonry clay bricks (NW type, according to ASTM C62, 1994) are considered in this study. Specified Compressive Strength of Masonry $f'm$ are based on specifying the compressive strength of masonry units that were given in Table 2.7. For strength of clay masonry units 2757.9 N/cm^2 (4000 psi) and 5515.8 N/cm^2 (8000 psi.), corresponding value of $f'm$ is 1103.16 N/cm^2 (1600 psi) and 1861.6 N/cm^2 (2700 psi.) respectively.

E_m is the ratio of the stress to the strain of a material or combination of materials as is the case for grouted masonry. To find out the value of E_m, considering the ACI/ASCE/TMS

Masonry Code, (1990) the modulus of elasticity is given in the Tables 2.8 and 2.9 (ACI/ASCE/TMS Specification Tables 1.6.2.1 and 1.6.2.2.). According to the Tables 2.8 and 2.9,value of E_m is 827.37 kN/cm^2 (1200 ksi) and 1396.18 kN/cm^2 (2025 ksi) for $f'm$ 1103.16 N/cm^2 (1600 psi) and 1861.6 N/cm^2 (2700 psi) respectively.

Determination of Equivalent Strut Width

The evaluation of the equivalent width, a, varies from one reference to the other. The expressions used in this chapter have been adopted from Mainstone (1971) and Stafford-Smith and Carter (1969) for their consistently accurate predictions of infilled frame in-plane behavior when compared with experimental results (Mainstone 1971; Stafford-Smith and Carter 1969; Klingner and Bertero 1978; and Al-Chaar 1998). The masonry infill panel will be represented by an equivalent diagonal strut of width, a, and net thickness, t_{eff}, as shown in Fig. 3.1.

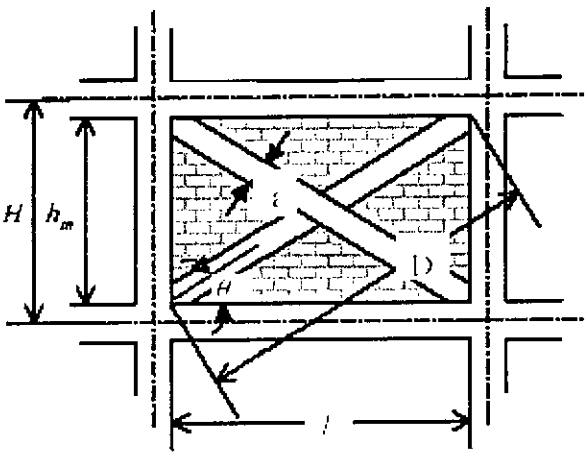


Fig. 3.1 Strut Geometry

The equivalent strut width, a, depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness shall be evaluated using Eq. 2.1. (Stafford-Smith and Carter 1969):

$$\lambda_I H = H\,[\,(E_m\, t \sin 2\theta)\,/\,(4\, E_c\, I_{col}\, h_w)\,]^{1/4} \qquad \text{Eq. 2.1}$$

Using this expression, Mainstone (1971) considered the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel as shown in below:

$$a = 0.175D(\lambda_I H)^{-0.4}$$

Where

a	is the equivalent strut width.
t	thickness of the masonry infill panel.
E_m	Modulus of elasticity of the masonry unit.
E_c	Modulus of elasticity of concrete.
h_w	Clear height of column member.
I_{col}	Moment of inertia of the column.
θ	Angle produce by the strut with the horizontal.

Eccentricity of Equivalent Strut

The equivalent masonry strut is to be connected to the frame as structural members by bold double-sided arrow in Fig. 3.2

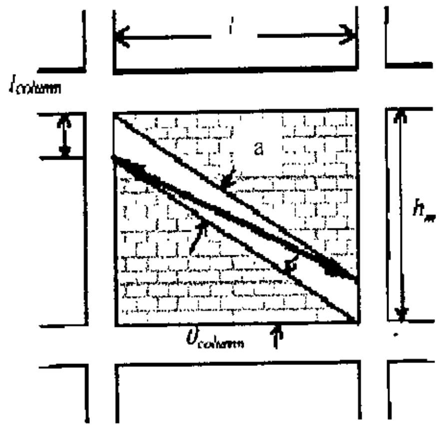


Fig. 3.2 Equivalent Masonry Strut

The strut should be pin-connected to the column at a distance l_{column} from the face of the beam. This distance is defined in Equations 2.4 and 2.5.

$l_{column} = a / \cos\ \theta_{column}$ Eq. 2.4

$\tan \theta_{column} = \{ h_m - (a / \cos \theta_{column})\} / l$ Eq. 2.5

Using this convention, the strut force is applied directly to the column at the edge of its equivalent strut width, a.

Perforated Panels

In the case of a perforated masonry panel, (panel having window and/or door opening) the equivalent strut is assumed to act in the same manner as for the fully infilled frame. Therefore, the eccentric strut should be placed at a distance l_{column} from the face of the beam as shown in Figure 3.3. The equivalent strut width, a, shall be multiplied, however, by a reduction factor to account for the loss in strength due to the opening. The reduction factor, $(R_I)_i$, is calculated using Equation 2.6.

$(R_I)_I = 0.6\ (A_{open} / A_{panel}) - 1.6\ (A_{open} / A_{panel}) + 1$ Eq. 2.6

Where:

A_{open} = Area of the opening (in^2)

A_{panel} = Area of the infill panel (in^2) = $l \times h_m$

If the area of the opening (A_{open}) is greater than or equal to 60 percent of the area of the infill panel (A_{panel}), then the effect of the infill should be neglected, i.e. $(R_I)_i = 0$

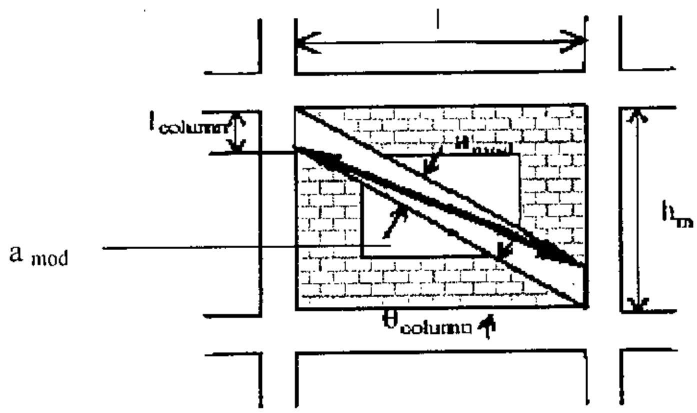


Fig. 3.3 perforated panel

This method is a simplification in order to compute the global structural capacity

Properties of Masonry Infill

Before computer modeling of the structural frame, the dimensions of the frame are selected and its properties calculated. The properties of various types of frames, under study, are given in Figures 3.4 and 3.5. All frames are two dimensional three and five bay RCC frames.

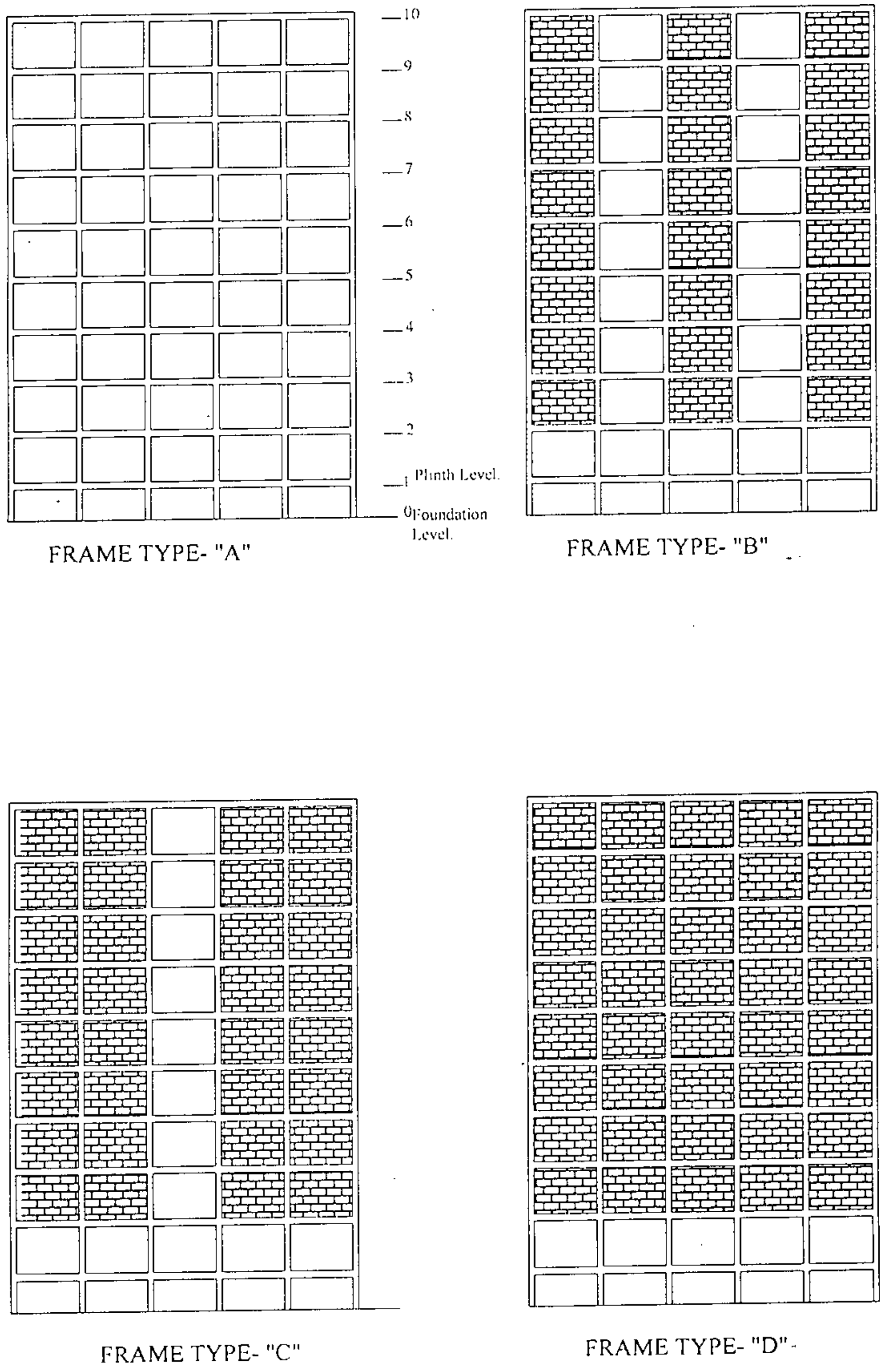


Fig.3.4 5 bay frames of 9 storied buildings

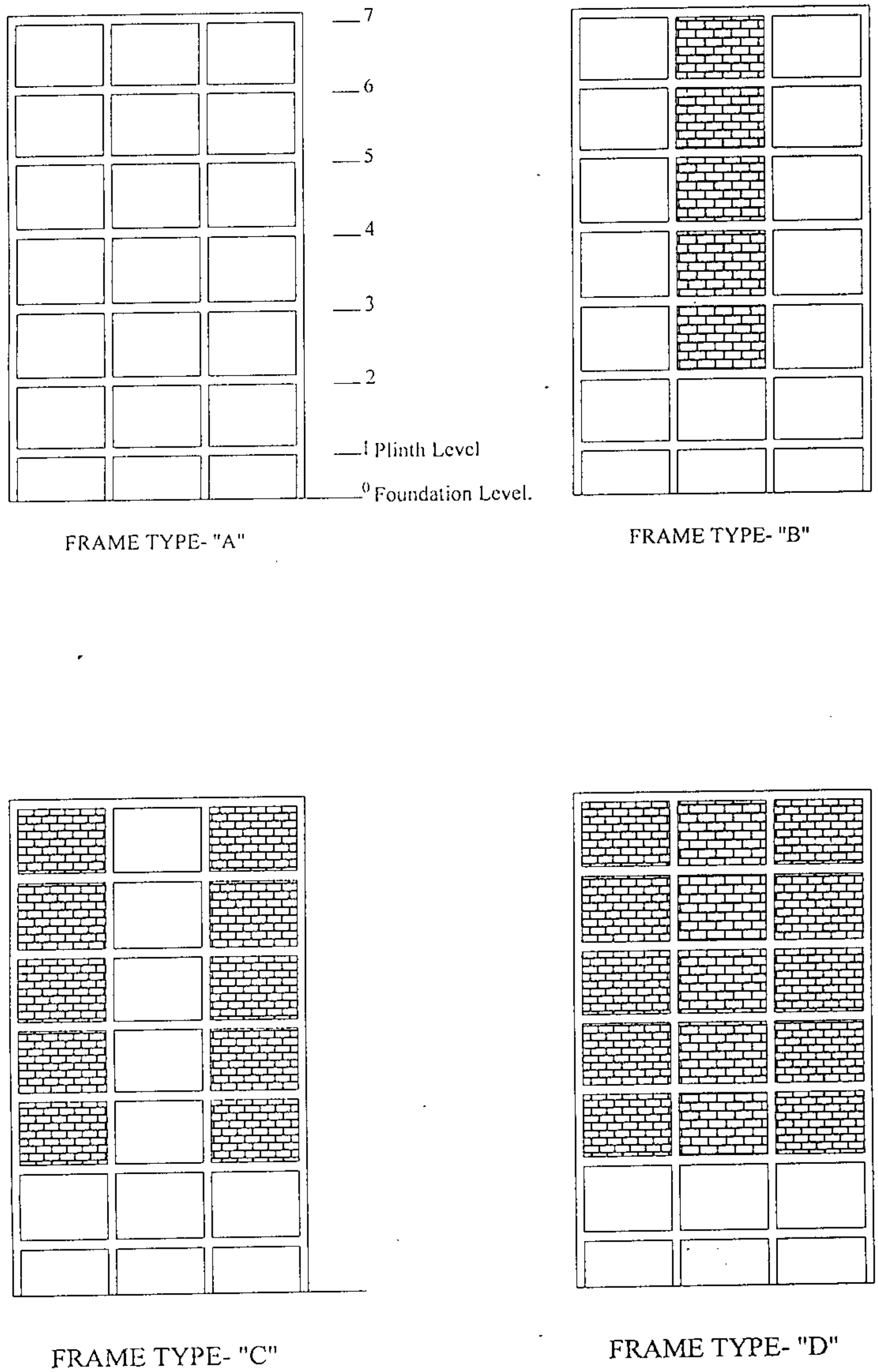


Fig.3.5 3 bay frames of 6 storied buildings

Type A – Fully bare frame, no infill components are considered in this type of frame.

Type B - One bay infill by masonry brick block and the other two bays have no infill components for three bays frame and three bays infill and others two have no infill for five bays frame.

Type C – Two bays have infill of masonry brick block and the rest one has no infill for three bays frame and four bays have infill out of five bays, the rest one has no infill.

Type D – All bays are infilled with masonry brick block.

TABLE 3.2 Properties of frame (having equal bay distance) and infills without opening

No of Story	Column size	I column, in^4	Wall thickness	Values of						$\lambda_1 H$	a in inch	θ_{column}	l_{column}, inch
				f'_m,Psi	E_m,Ksi	E_c, Ksi	H, inch	h_w, inch	θ ,radian				
6	12" x 12"	1728	5	4000	1200	3587	118.08	102.33	0.64347	4.583	18.732	0.5076	21.43
				8000	2025					5.224	17.777	0.513	20.40
			10	4000	1200					5.451	17.477	0.5147	20.08
				8000	2025					6.212	16.586	0.5197	19.11
9	12" x 15"	3375	5	4000	1200	3587	118.08	102.33	0.64347	3.877	20.03	0.5004	22.83
				8000	2025					4.419	19	0.5065	21.73
			10	4000	1200					4.611	18.687	0.508	21.39
				8000	2025					5.255	17.735	0.5133	20.36
12	15" x 15"	4218.75	5	4000	1200	3587	118.08	102.33	0.64347	3.667	20.48	0.4977	23.31
				8000	2025					4.179	19.437	0.5036	22.19
			10	4000	1200					4.361	19.11	0.5055	21.84
				8000	2025					4.970	18.135	0.511	20.79

TABLE 3.3 Center point distance (C_p) of the equivalent strut

No. of story	f'_m	Wall thickness	l_{column} (inch)	C_p (inch)
6	4000	5	21.43	18.59
	8000	5	20.40	18.08
	4000	10	20.08	17.91
	8000	10	19.11	17.43
9	4000	5	22.83	19.29
	8000	5	21.73	18.74
	4000	10	21.39	18.57
	8000	10	20.36	18.05
12	4000	5	23.31	19.53
	8000	5	22.19	18.97
	4000	10	21.84	18.80
	8000	10	20.79	18.27

TABLE 3.4 Properties of frame (having unequal bays) and infills without opening

No of Story	Column size	I *column*, in^4	Wall thickness	Values of						$\lambda_1 H$	a in inch	θ_{column}	l_{column}, inch
				f'_m, Psi	E_m, Ksi	E_c, Ksi	H, inch	h_w, inch	θ, radian				
6	12" x 12"	1728	5	4000	1200	3587	157.5	102.33	0.785	4.631	15.83	0.66	20.02
			5	4000	1200		196.8		0.540	4.488	22.03	0.416	20.77
9	12" x 15"	3375	5	4000	1200	3587	157.5	102.33	0.785	3.917	16.926	0.653	21.31
			5	4000	1200		196.8		0.540	3.796.	23.554	.418	20.45
12	15" x 15"	4218.75	5	4000	1200	3587	157.5	102.33	.785	3.704	17.308	0.65	21.74
			5	4000	120 0		196.8		0.5404	3.59	24.08	0.422	19.44

Table 3.5 Center point distance (C_p) of the equivalent strut. (unequal bay)

No of story	f'_m	Wall thickness	l_{column} (inch)	C_p (inch)
6	4000	5	20.02	17.89
	4000	5	20.77	18.26
9	4000	5	21.31	18.53
	4000	5	20.45	18.10
12	4000	5	21.74	18.75
	4000	5	19.44	17.60

Table 3.6 Properties of frame (having equal bays) and infills with opening

No. of Story	Column size	I $column$, in^4	Wall thickness	Values of						$\lambda_l H$	a_{mod} in inch	θ_{column}	l_{column}, inch
				f'_m,Psi	E_m,Ksi	E_c, Ksi	H, inch	h_w, inch	θ ,radian				
6	12" x 12"	1728	5	4000	1200	3587	118.08	102.33	0.64347	4.583	12.673	0.542	14.79
				8000	2025					5.451	11.824	0.547	13.84
			10	4000	1200					5.224	12.027	0.546	14.07
				8000	2025					6.212	11.221	0.55	13.16
9	12" x 15"	3375	5	4000	1200	3587	118.08	102.33	0.64347	3.877	13.550	0.538	15.78
				8000	2025					4.611	12.643	0.542	14.76
			10	4000	1200					4.419	12.859	0.541	15.00
				8000	2025					5.255	11.998	0.546	14.04
12	15" x 15"	4218.75	5	4000	1200	3587	118.08	102.33	0.64347	3.667	13.856	0.536	16.12
				8000	2025					4.361	12.928	0.541	15.08
			10	4000	1200					4.179	13.150	0.54	15.33
				8000	2025					4.970	12.269	0.545	14.35

CHAPTER 4
PARAMETRIC STUDY

4.1 GENERAL

The composite behavior of an infilled frame is a complex statically indeterminate problem. Structures experience a relative displacement depending on its stiffness and the inertia of its masses. Stiffness of a structure is reflected by its natural period of vibration. When a sudden change in stiffness takes place along the building height , the story at which this drastic change of stiffness occurs is called a soft story. The most common form of vertical discontinuity arises due to the unintended effect of infill component. The problem is most sever in structures having relatively flexible lateral load resisting system because the infill can compose a significant portion of the total stiffness.

The strength of an infill frame is influenced by the interaction of the frame and infill. The distribution of the interaction control the stress distribution in the infill, and, therefore affect its strength and modes of failure. When the infill cracks in initial mode, the frames prevent disintegration, and the infill may resist substantially higher load before finally collapsing by a compressive failure. Thus the mutual interaction of the frame and infill plays an important role in controlling the stiffness and strength of the infill and the problem is examined in terms of their relative properties.

This chapter aims at studying the displacement and moment behavior of the frame. The frame is modeled with or without infill at earthquake loading and the effects of various parameters are also observed.

According to Arnold and Reitherman (1982), buildings with regular and simple configuration with direct load transfer path perform much better during strong shaking. While additional analysis requirements are usually provided for building with irregular configuration. Hence the seismic configuration is an important consideration at the stage of architectural planning of a building.

4.2 OBSERVED PARAMETERS

There are a few materials and geometric parameters which influence the behavior of infilled frame, these parameters are as follows:

- Masonry compressive strength, f'_m
- Panel aspect ratio, (h/l)
- Infill wall thickness.
- Number of story in the frame
- Number of bays
- Position of the infill wall
- Nature of infill wall in the panel (ie. with or without opening)

The general idea of parametric study for a number of independent parameters embodies the fact that in a single instance only one variable should be allowed to vary while other parameters are fixed at some standard value within its range. If we allow two or more parameters to vary at the same time it would cause confusion in the results of the analysis. Hence our investigation specifies a fixed range for all the variables within which the actual work of sensitivity analysis is carried out.

Masonry Compressive Strength, f'_m

Masonry compressive strength is a direct indication of the effect of infill, because it is the core material of the infill component. In this study value of masonry compressive strength is 27.5 and 55 Mpa.

Panel Aspect Ratio

Panel aspect ratio is the direct indication of the effect of frame sway characteristics when the frame is modeled with or without infill. In infilled frames the infill stiffness greatly depends on panel aspect ratio. So it is an important parameter for infilled frame analysis. The panel aspect ratio depends on floor height "h" and span length "l" of the frame. In this analysis the floor height is 3000 mm for all the floors and the span length "l" has a typical values of 4000 mm and it has been studied for different values in the range between 3000 mm to 5000 mm.

Wall Thickness, t

Wall thickness is also a direct indication of the infill stiffness. The compressive strength of masonry has great influcnce on the sway characteristics of the masonry infilled frames. Generally 125 mm and 250 mm thickness walls are used in building structures. So in this study two types of walls (125 mm and 250mm) are considered .

Number of Story in Vertical Direction

Story number primarily depends on owner's consideration and functioning system of the building, and it has a great influence on deflection and flexural behavior of the frame. To find out the effect of number of story, four types of buildings have been studied in this work and that are four, six, nine and twelve storied building frames.

Number of Bays in Horizontal Direction

Number of bays in horizontal direction of a building frame is dependent on its size, shape and functioning purposes. To find out the effect of the bays, three and five number of bays have been considered in this study.

CHAPTER 5

ANALYSIS AND RESULTS

5.1 INTRODUICTION

The main objective of this investigation is to study the effect of infill walls on frames due to horizontal loading in different conditions. The materials that are generally available in Bangladesh have been considered in this investigation. Properties of the materials are given in Table 3.1. In this chapter only the analysis and the related results are presented.

5.2 TYPES OF ANALYSIS

Two types of analysis techniques have been adopted to find out the effects of the infill walls on frames. At first the Equivalent Static Force Method has been used and later the same frame has been analyzed by the Response Spectrum Analysis. Sections 2.5.6 and 2.5.7 of BNBC (1993) have been consulted to find out the equivalent loading and design response spectrum respectively due to earthquake. For regular structures the Equivalent Static analysis is considered conservative.

5.3 LOADS

All structures must be designed to resist gravitational and lateral forces, both permanent and transient, that it will be called on to sustain during its construction and subsequent service life. These forces will depend on the size and shape of the building, as well as on its geographic location. In the present investigation, earthquake loads have been chosen for the horizontal loading ignoring all other probable loads. The design seismic lateral forces are calculated by the Equivalent Static Force Method and the Dynamic Response Method of article 2.5 of BNBC (1993).

5.4 MODELS FOR ANALYSIS

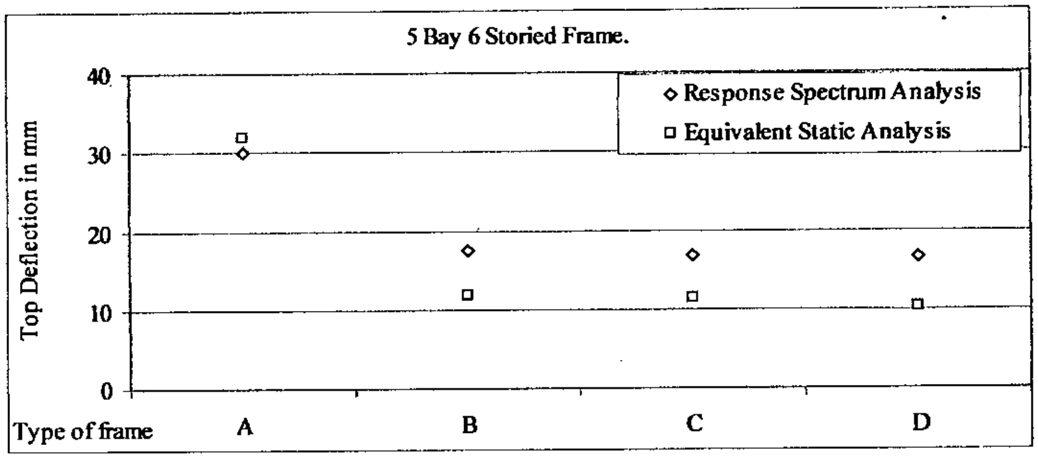
Four types of frames are considered as the basic models in this investigation. Types of the frames are: Type-A, B, C and D (Figures 3.4 and 3.5). Only bare frame effect is considered in Type-A. Type-B frame has one bay infill out of three bays and three bays infill out of five bays. Two bays infill walls out of three bays and four bays infill walls out of five bays are considered in Type-C. In Type-D all bays have infill walls. In all cases of the infill walls, bottom story has no infill components for movement and parking facility of vehicles.

5.5 DRIFT

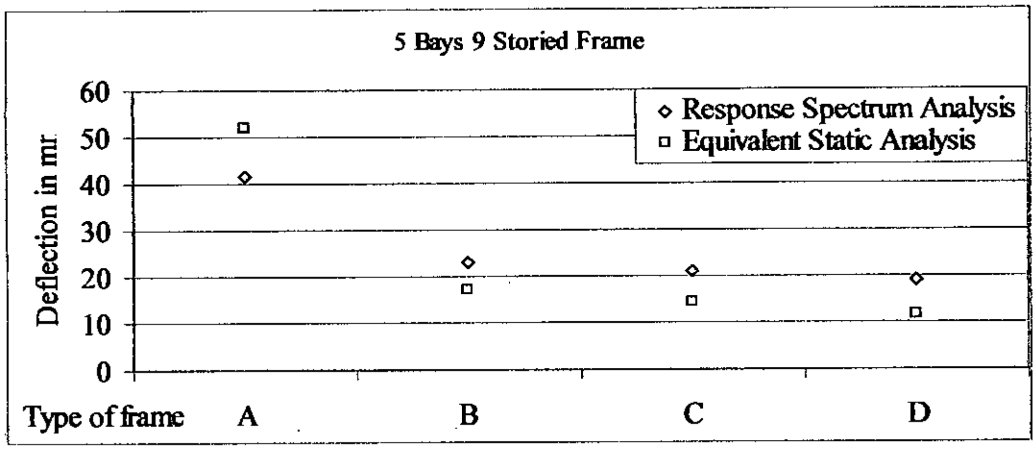
The provision of adequate stiffness, particularly lateral stiffness, is a major consideration in the design of building for several important reasons. In terms of serviceability limit state, deflections must first be maintained at a sufficient low level to allow proper functioning of non structural components, to prevent excessive cracking and consequent loss of stiffness. One simple parameter that affords an estimate of the lateral stiffness of a building is the drift index, defined as the ratio of the maximum deflection at the top of the building to the total height. Design drift index limits that have been used in different countries ranges from 0.001 to 0.005. (Smith and Coull, 1991) Generally lower values should be used for hotels or apartment buildings and higher for office buildings.

5.6 DEFLECTION BEHAVIOUR OF THE FRAME

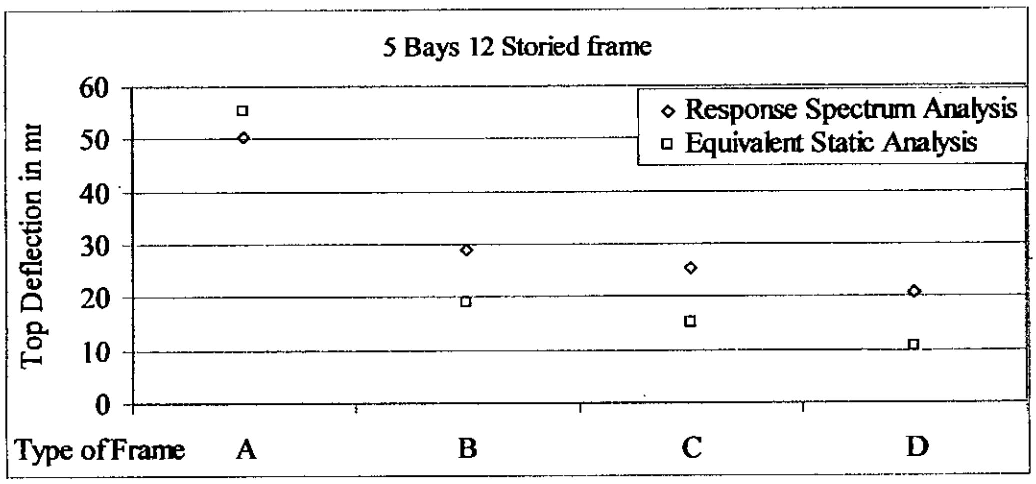
Deflections of bare frame and bare frame with different patterns of infill walls have been summarized and some graphs have been plotted using those results. From Fig 5.1 to 5.3 it is found that the Equivalent Static analysis shows higher values only for bare frame (Type A) but when infill is present in the frame then Response Spectrum Analysis gives higher values of deflection for all other cases except the bare frame. This variation in results is also observed in the variation of number of bays in horizontal direction (3 and 5 bays) and also in the variation of number of stories in vertical direction (6, 9 and 12 stories). For 4 storied frame (Fig. 5.1 d) it is found that the Response Spectrum Analysis result shows higher values than the Equivalent Static Analysis for all type of frames (including the bare frame).



(a) 5 bay 6 storied frame

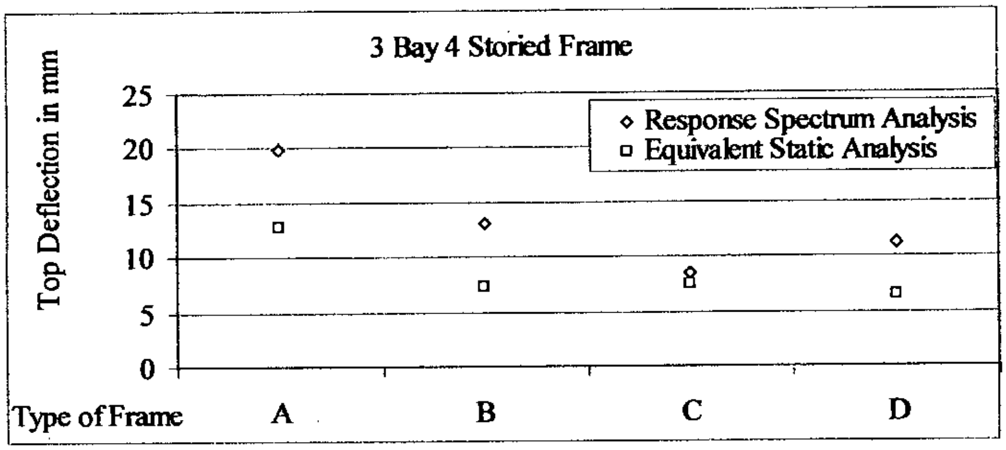


(b) 5 bay 9 storied frame.

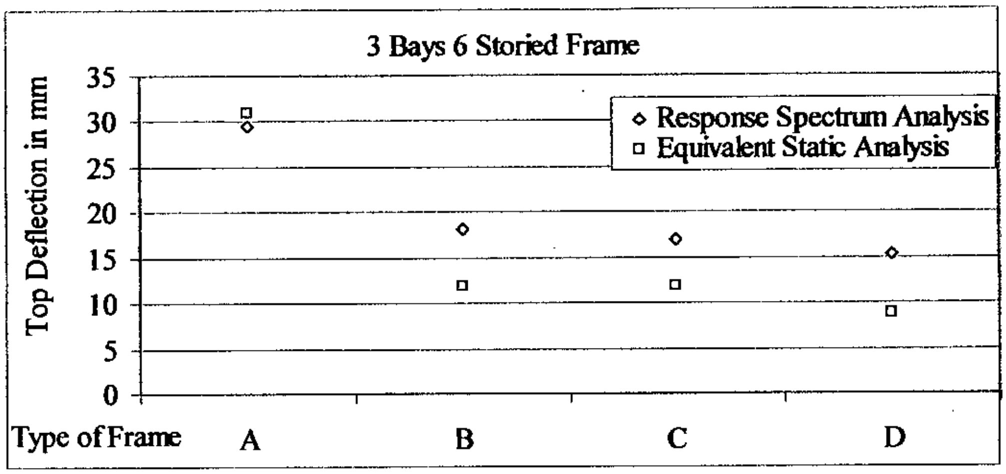


(c) 5 bay 12 storied frame

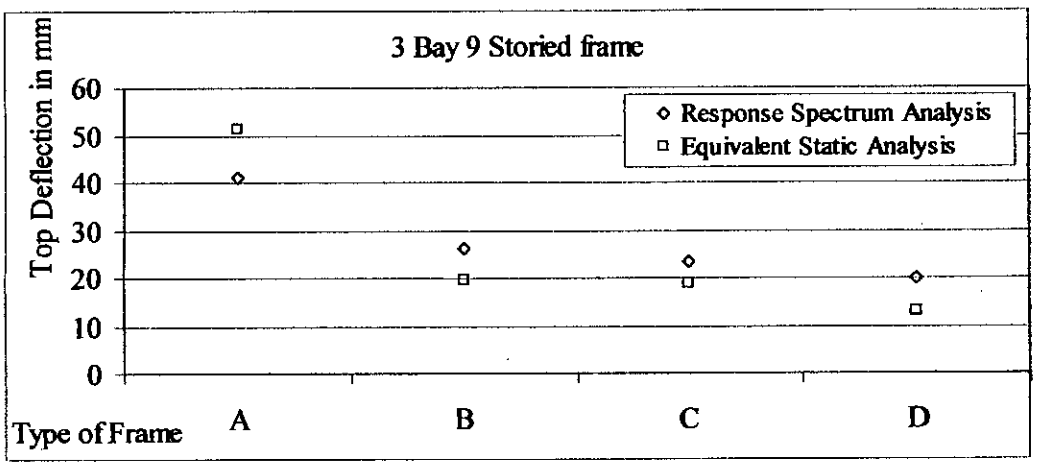
Fig. 5.1 continue



(d) 3 bay 4 storied frame

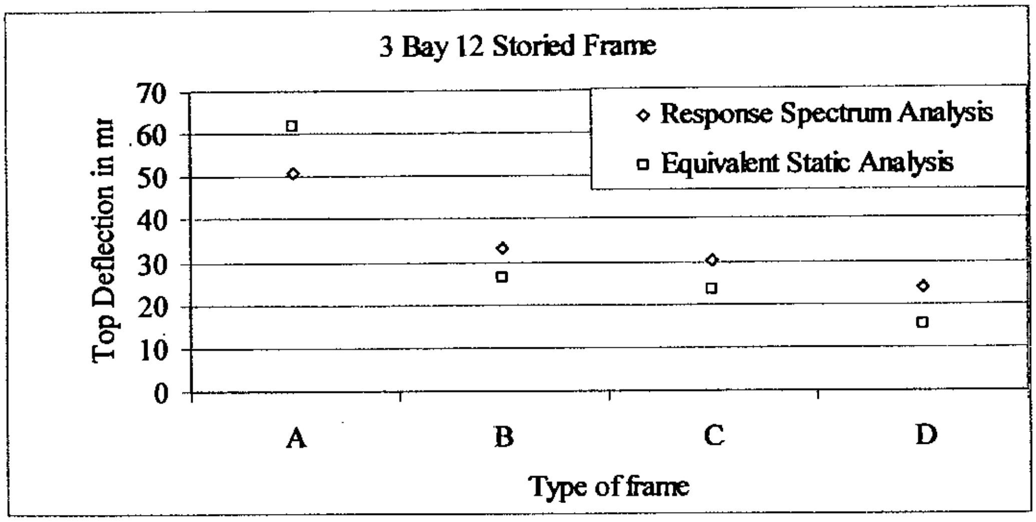


(e) 3 bay 6 storied frame

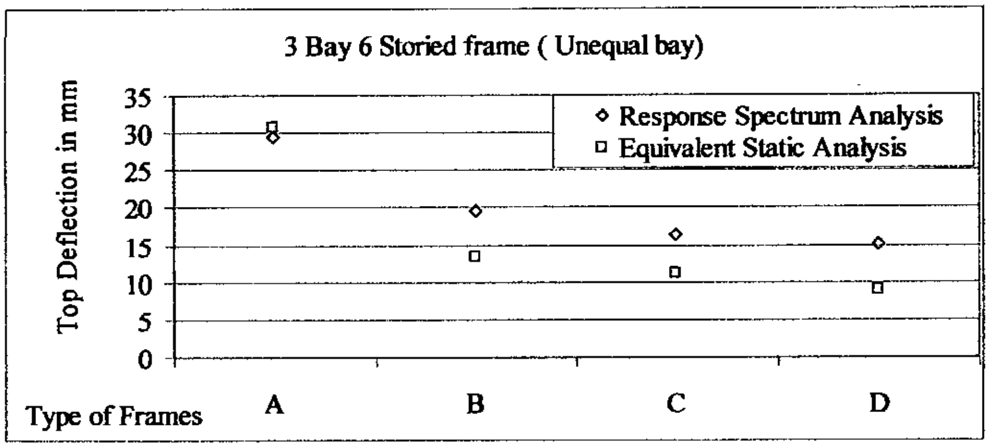


(f) 3 bay 9 storied frame

Fig. 5.1 continue



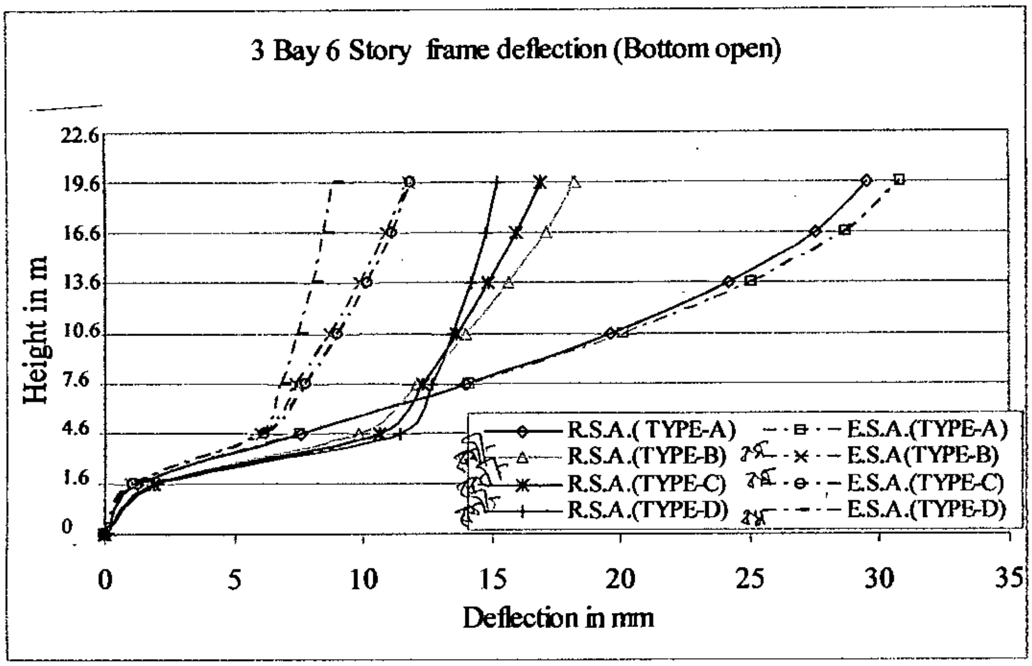
(g) 3 bay 12 storied frame



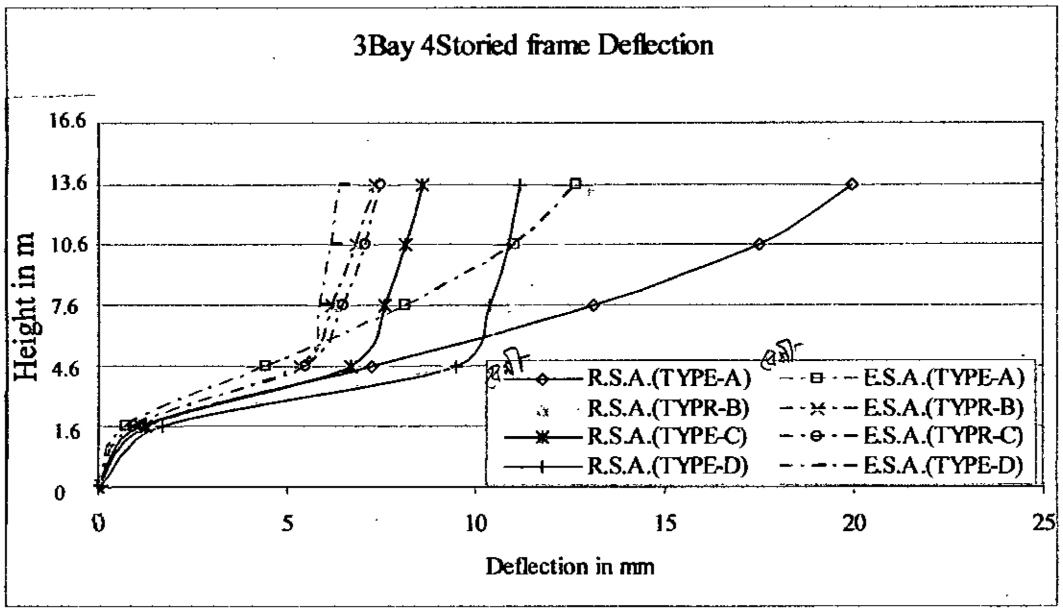
(h) 3 bay 6 storied frame (unequal bay)

Fig. 5.1 Top deflection of different types of frames (Thickness of infill Wall = 125 mm, E_m = 827.37 kN/cm^2(1200 Ksi)

Story wise total deflections of all the frames under consideration have been summarized and some graphs have been plotted using those results. From Fig. 5.2 it is observed that the Equivalent Static analysis shows higher values only for bare frame (Type A) but when infill is present in the frame then Response Spectrum Analysis gives higher values of deflection for all other cases except the bare frame. This variation in results is also observed in the variation of number of bays in horizontal direction (3 and 5 bays) and also in the variation of number of stories in vertical direction (6, 9 and 12 stories). But for 4 storied frame (Fig. 5.2 b) it is found that the Response Spectrum Analysis result shows higher values than the Equivalent Static Analysis for all type of frames (including the bare frame).

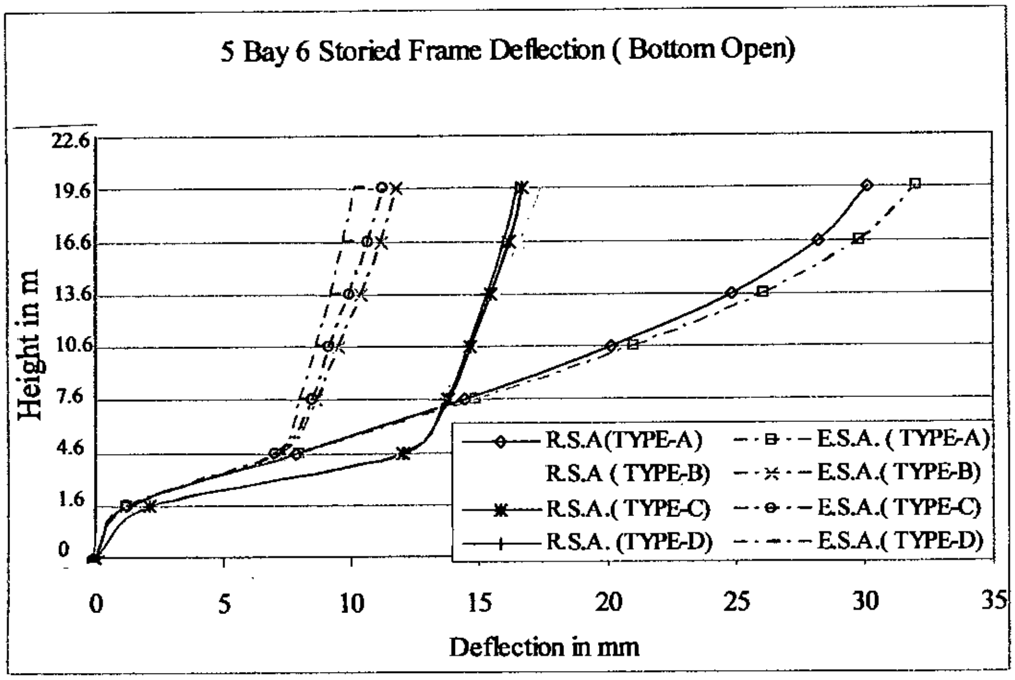


(a) 3 bay 6 storied frame deflection

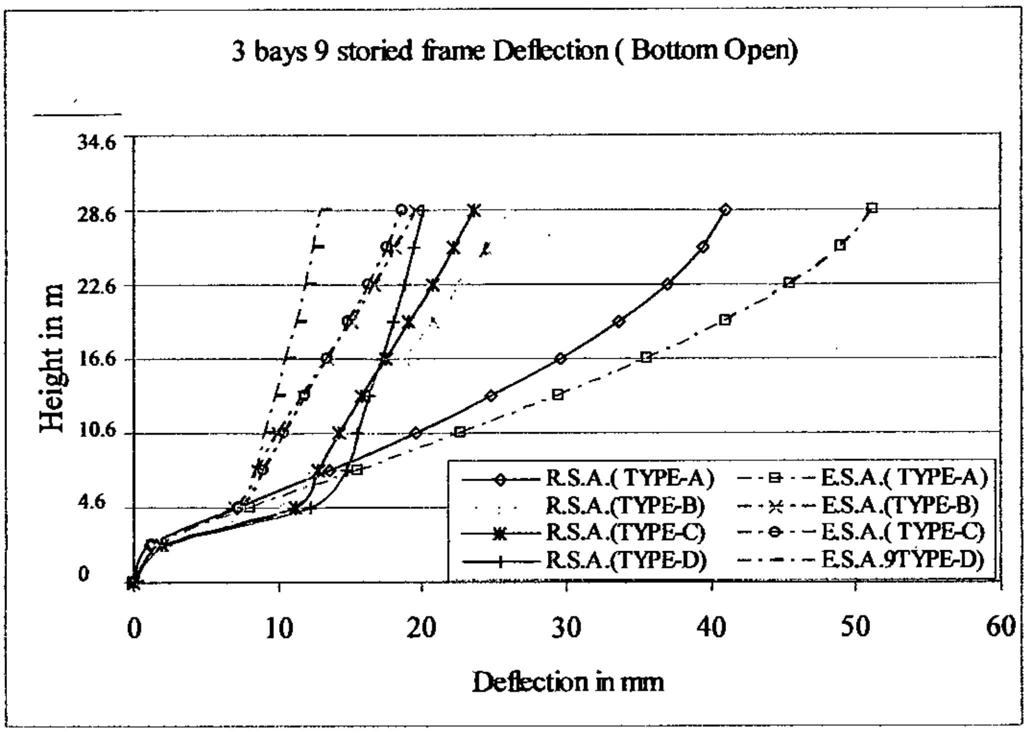


(b) 3 bay 4 storied frame deflection

Fig. 5.2 continue

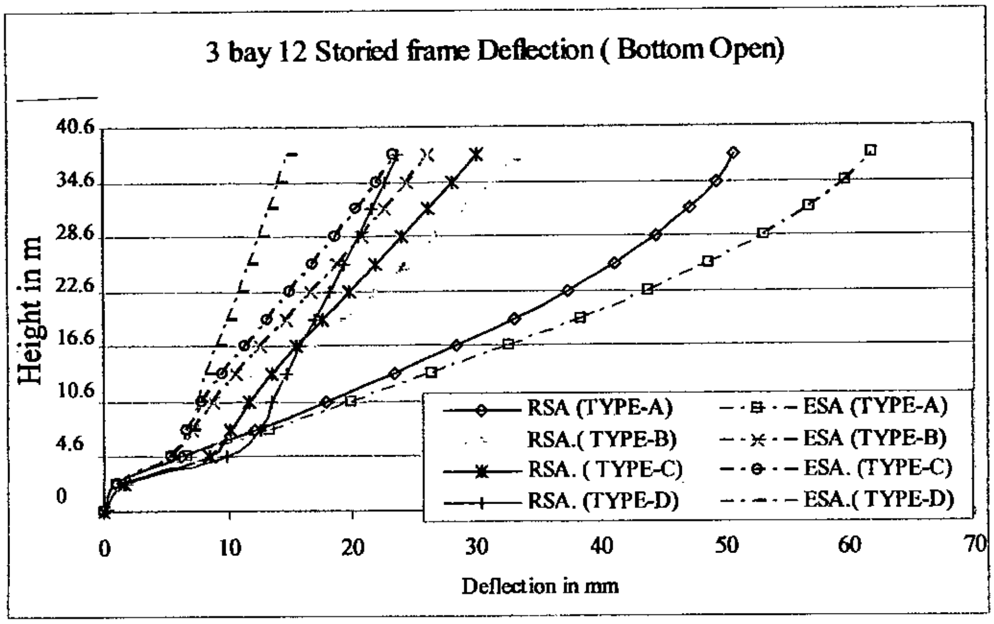


(c) 5 bay 6 storied frame deflection



(d) 3 bay 9 storied frame deflection

Fig. 5.2 continue



(e) 3 bay 12 storied frame deflection.

Fig. 5.2 Story wise deflection of different type of frames, thickness of infill Wall = 125 mm and E_m = 827.37 kN/ cm^2 (1200 ksi)

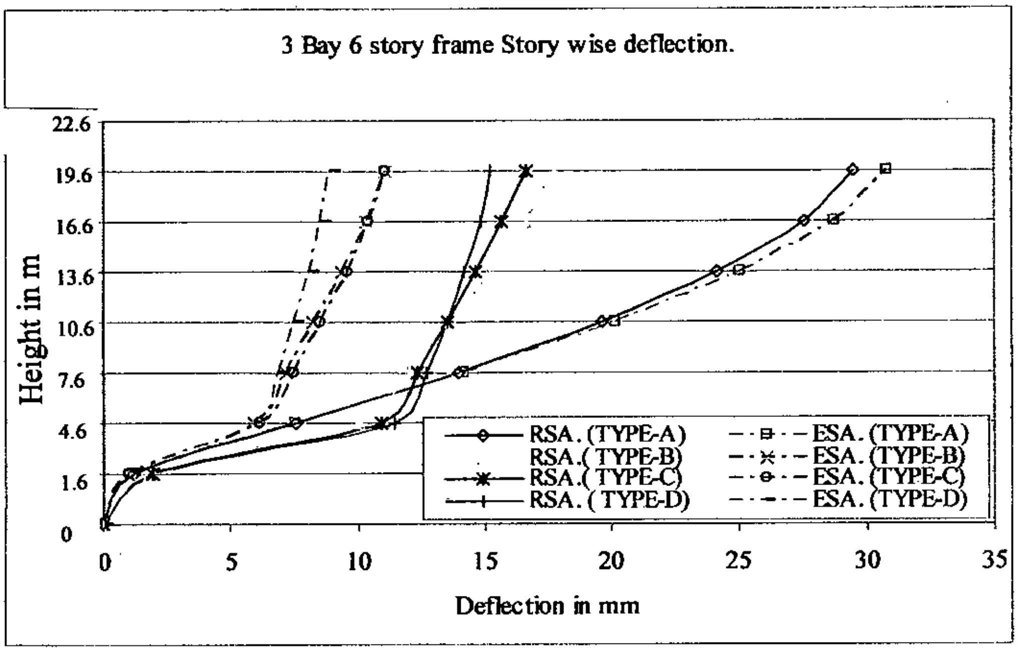


Fig. 5.3 Story wise deflection of different type of frames, thickness of infill Wall = 125 mm, E_m = 1369.18 kN/cm^2 (2025 ksi)

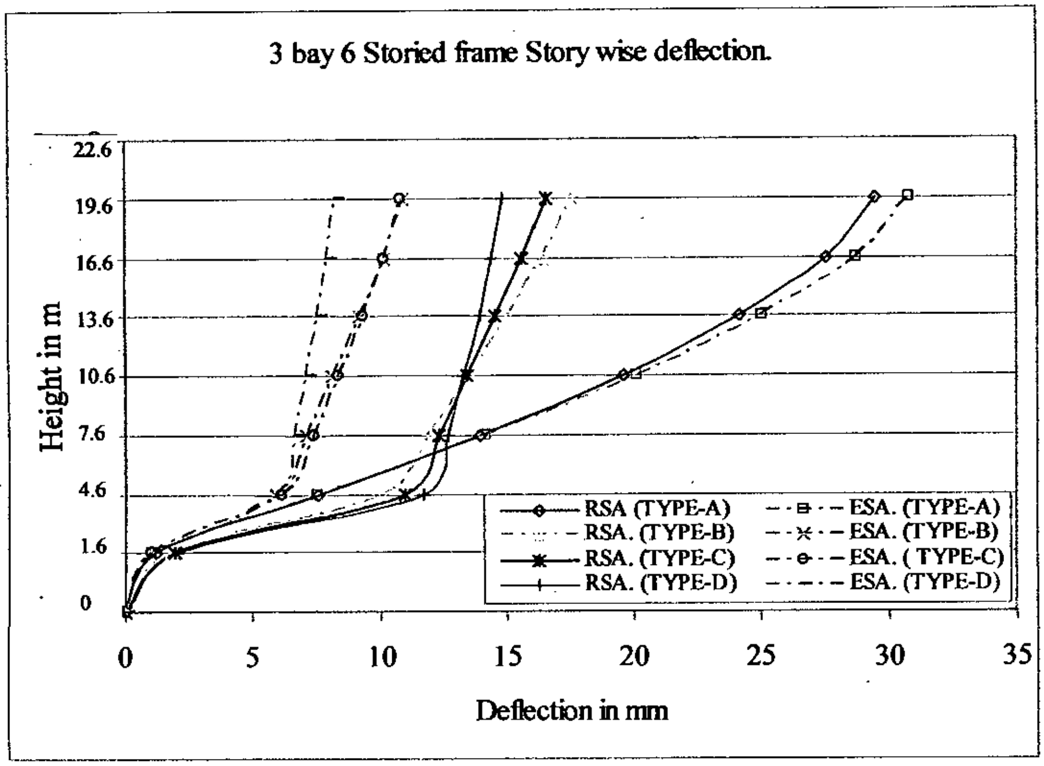


Fig. 5.4 Story wise deflection of different type of frames, thickness of infill wall = 250 mm , E_m = 827.37 kN/ cm^2 (1200 ksi)

Masonry infill is easy, economical and widely used in Bangladesh. Generally 125 mm and 250 mm thickness masonry are used as infill walls in frames. In this study both types of infill walls have been analyzed. At the time of an earthquake the walls act as a compressive masonry strut. Strength of the masonry strut depends on the combined strength of the masonry block and the mortar by which the blocks are bonded with each other. Solid masonry clay brick (NW type, according to ASTM C62, 1994) has been considered in this study. The required or specified value of the compressive strength of masonry, f'_m is used as the basis for structural engineering design and must be obtained or verified in accordance with prescribed code requirements. Specified values of compressive strength of masonry f'_m, based on specific compressive strength of masonry units, are given in Table 2.7. Strength of clay masonry units considered in this investigation are 2757.9 N/cm^2 (4000 psi) and 5515.8 N/cm^2 (8000 psi). Because this type of clay masonry units are available in

Bangladesh, the corresponding values of f'_m are 1103.16 N/cm^2 and 1861.58 N/cm^2 (1600 and 2700 psi) respectively.

E_m is the ratio of the stress to the strain of a material or combination of materials. To find out the value of E_m, considering the ACI/ASCE (1992), the modulus of elasticity is given in Tables 2.8 and 2.9. According to Tables 2.8 and 2.9, values of E_m are 827.37 N/cm^2 (1200 ksi) and 1396.18 N/cm^2 (2025 ksi) for f'_m 1103.16 and 1861.58 N/cm^2 (1600 and 2700 psi) respectively. In analyzing the frames all the values given above have been considered as the input properties.

It has been found from the analysis that the effect of presence of infill wall is more significant than the thickness and E_m values of the wall. At first the 3 bays of 6 storied frames with variable types of infill properties have been studied and compared. The results are summarized in Figures 5.3, 5.4 to 5.7. It is clear from the results that the effect of the wall thickness and E_m is not so pronounced and for this reason, these variables are not considered for other frames.

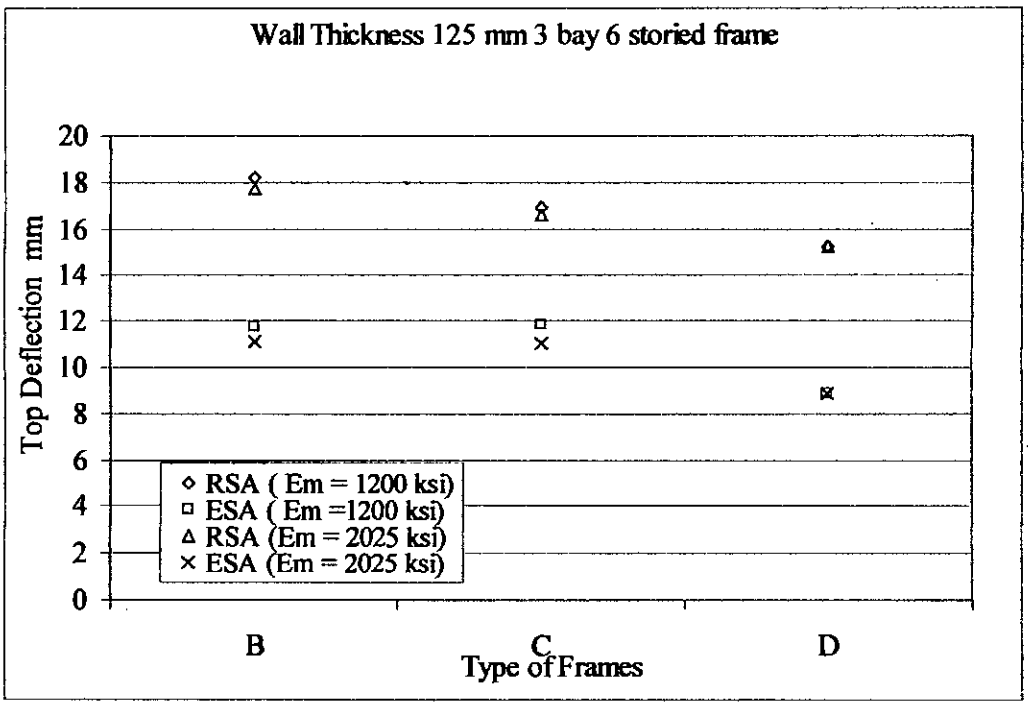


Fig. 5.5 Top Deflection of different type of frames and E_m

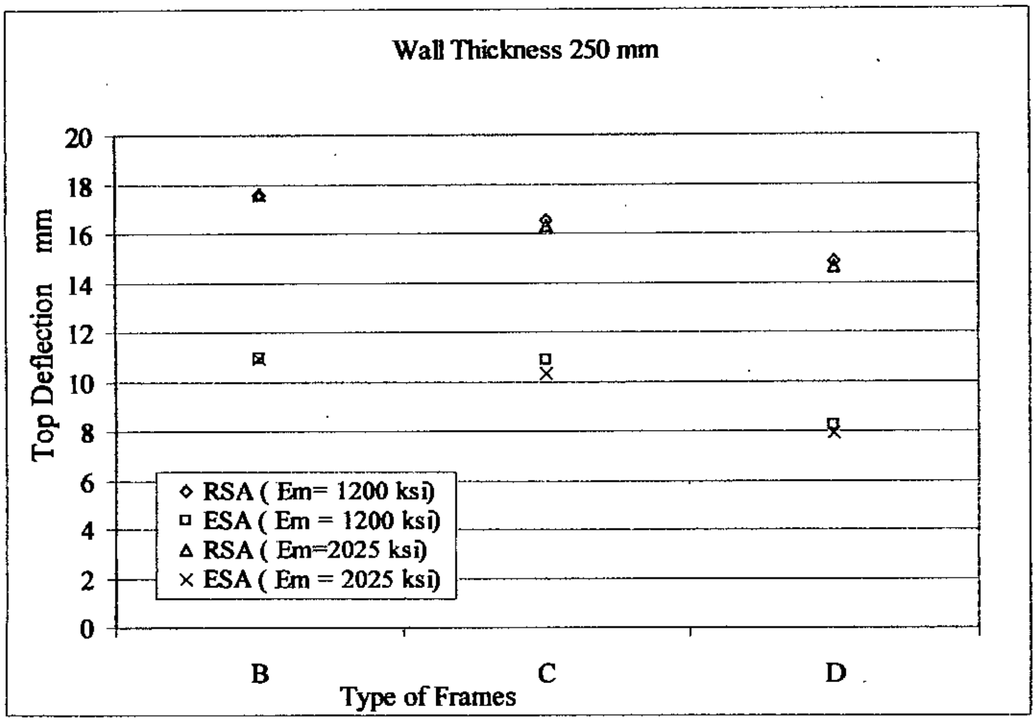


Fig. 5.6 Top Deflection for different type of frames and E_m, 3 bay 6 storied frame

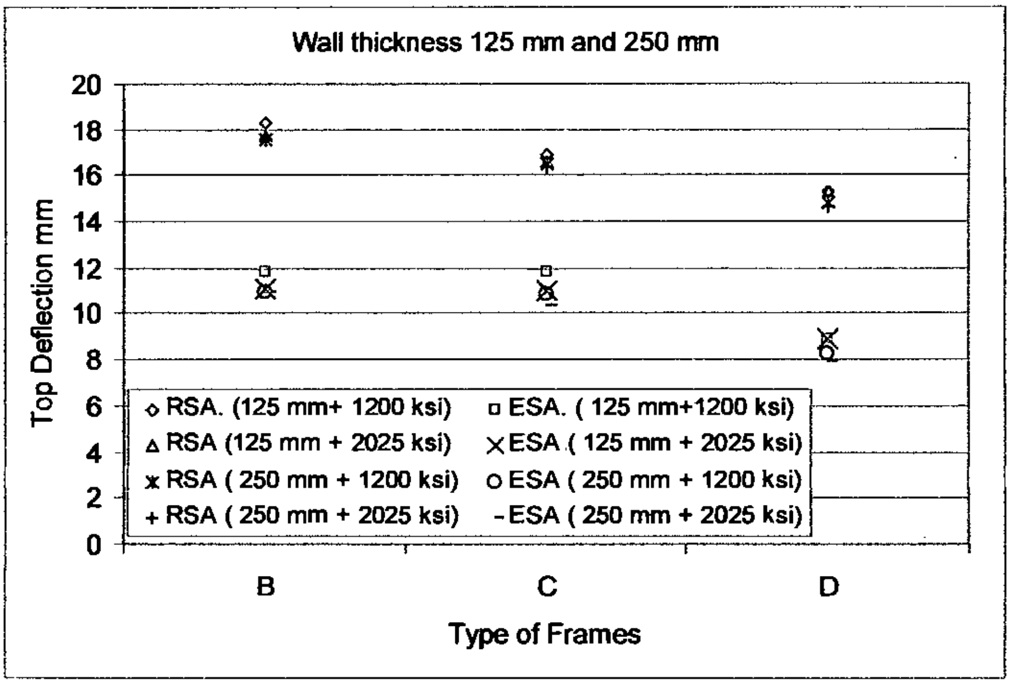
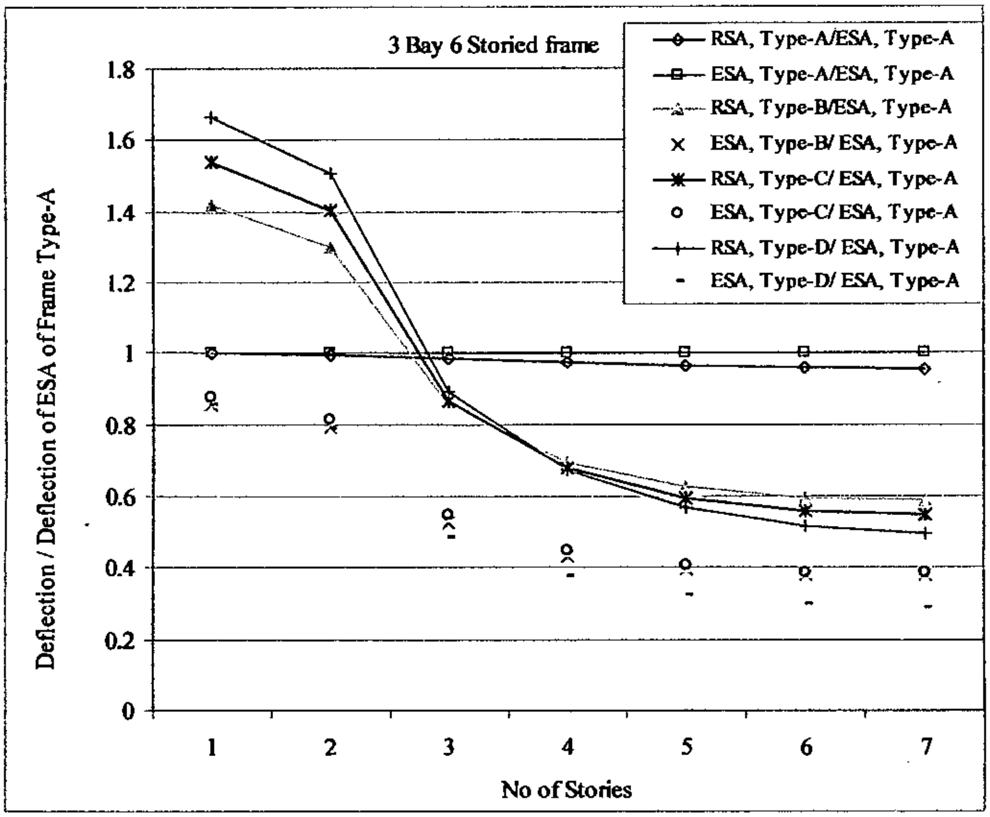


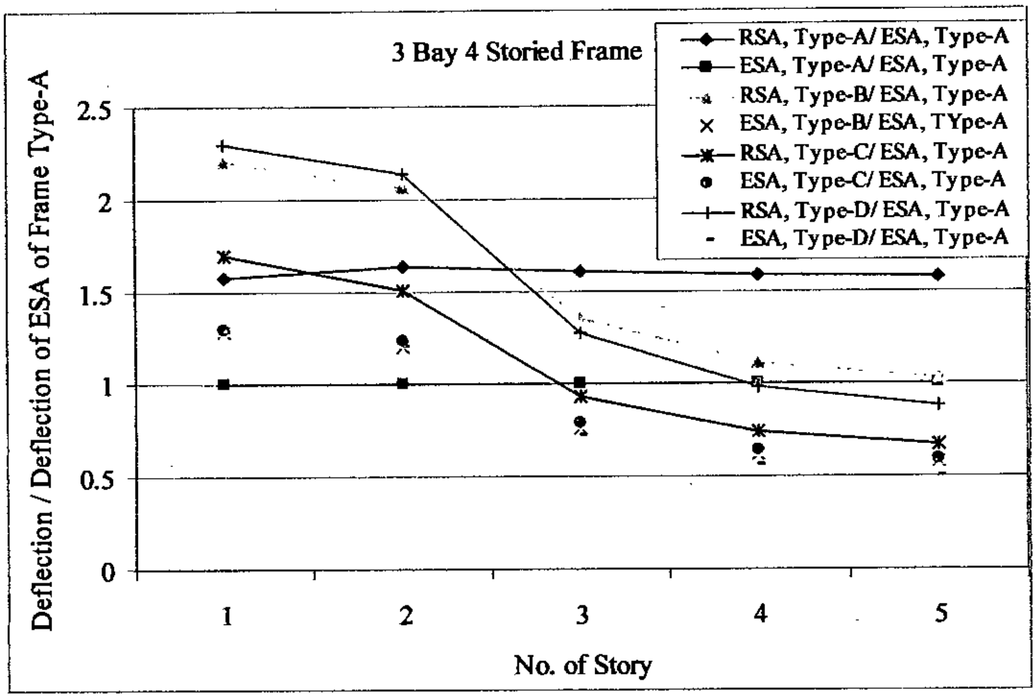
Fig. 5.7 Top Deflection for different type of frames with different wall thickness and E_m (RSA = Response Spectrum Analysis, ESA = Equivalent Static Analysis, 3 bay 6 storied frame.)

The effects of analysis procedure on the story wise deflection of frames of different types are shown in Fig. 5.8. It is found from the figure that the Equivalent Static Analysis (ESA) procedure gives higher deflection values compared to the Response Spectrum Analysis (RSA) procedure only in the case of 6, 9 and 12 storied bare frame (Type A). But when the frames have masonry infill walls, the result are drastically changed, specially at the level below which there is no infill and above which the infill wall is present. It is true in this investigation for both equal and unequal length of bay. For 4 storied frame Response Spectrum Analysis (RSA) procedure gives higher deflection values in all types of frames compared to the Equivalent Static Analysis (ESA) procedure at the level where there is no infill and the other floors have some infill walls.

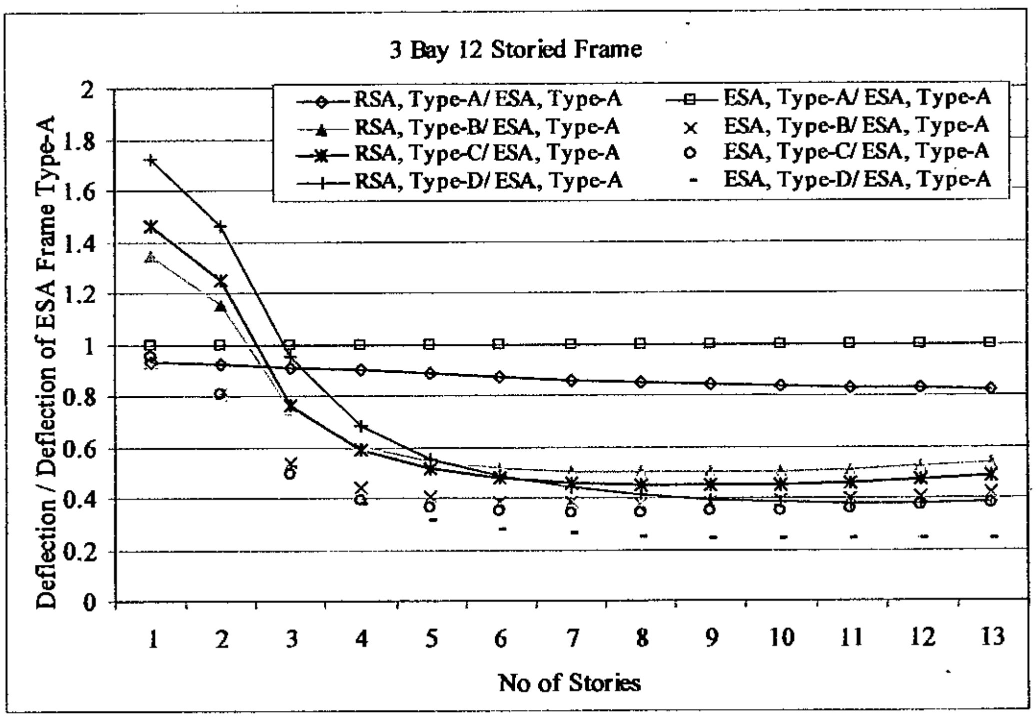


(a) 3 bay 6 storied frame

Fig. 5.8 continue

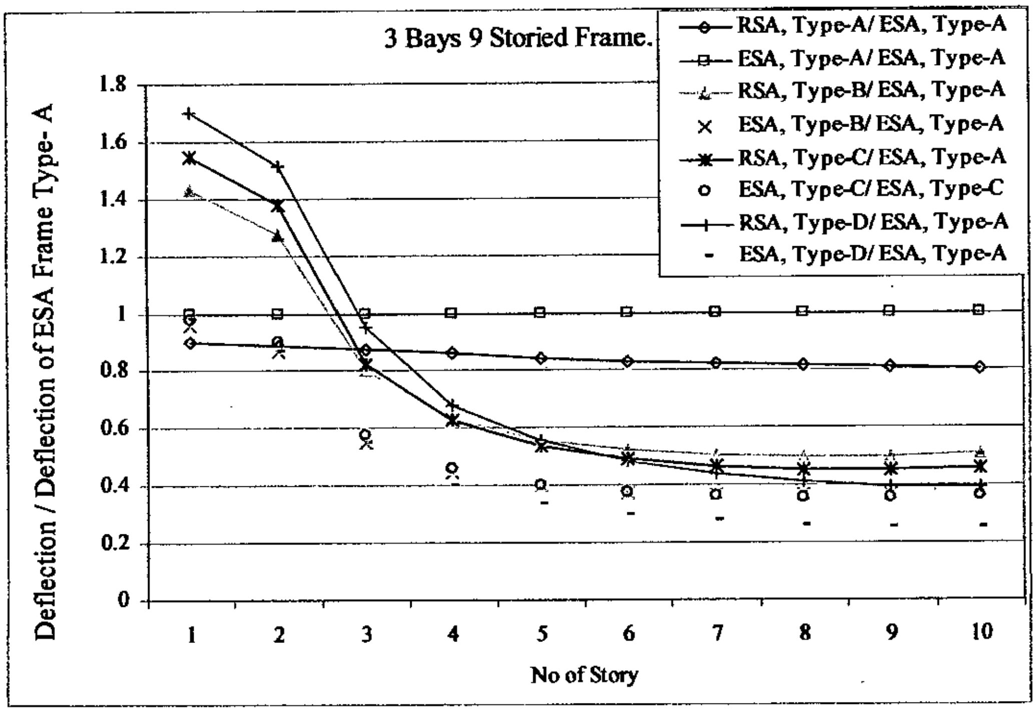


(b) 3 bay 4 storied frame

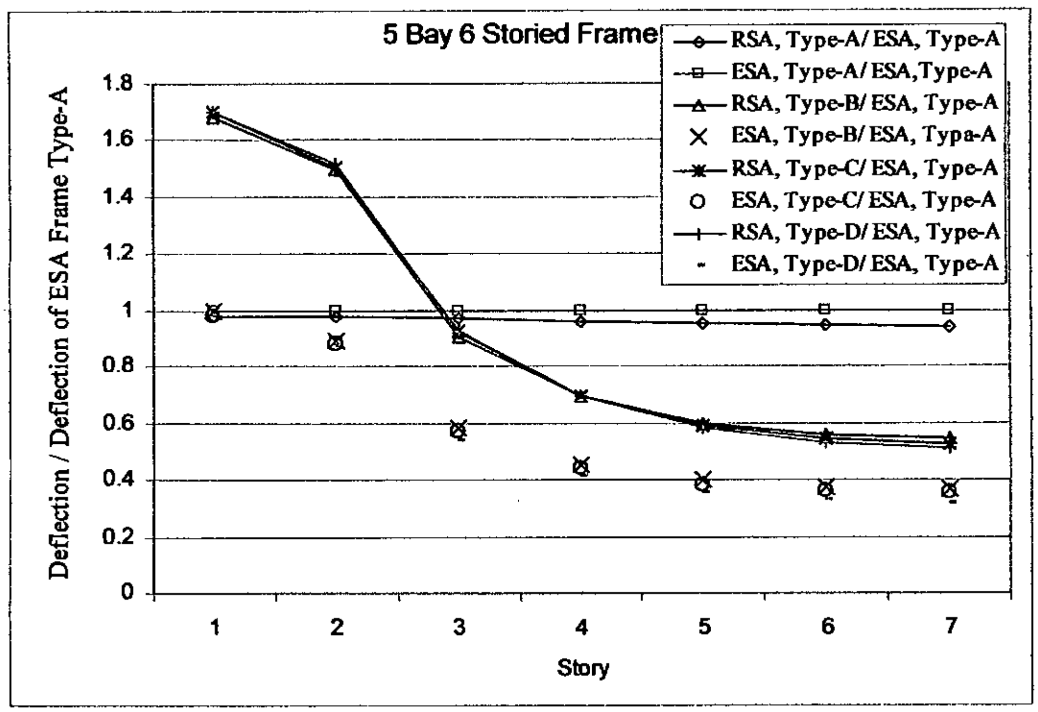


(c) 3 bay 12 storied frame

Fig. 5.8 continue

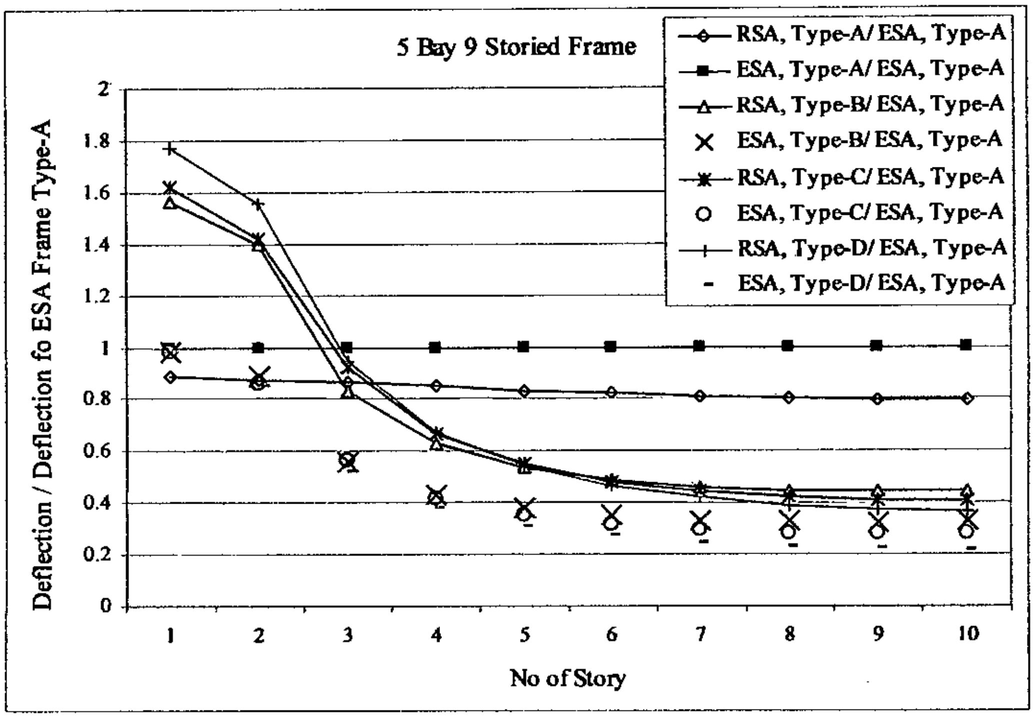


(d) 3 bay 9 storied frame

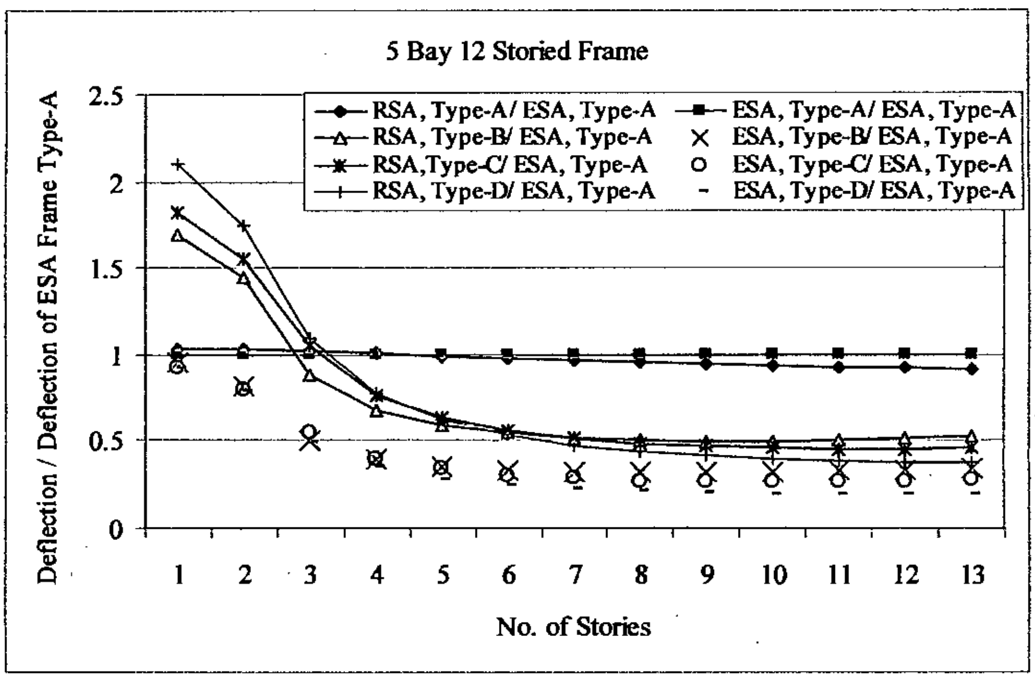


(e) 5 bay 6 storied frame

Fig. 5.8 continue

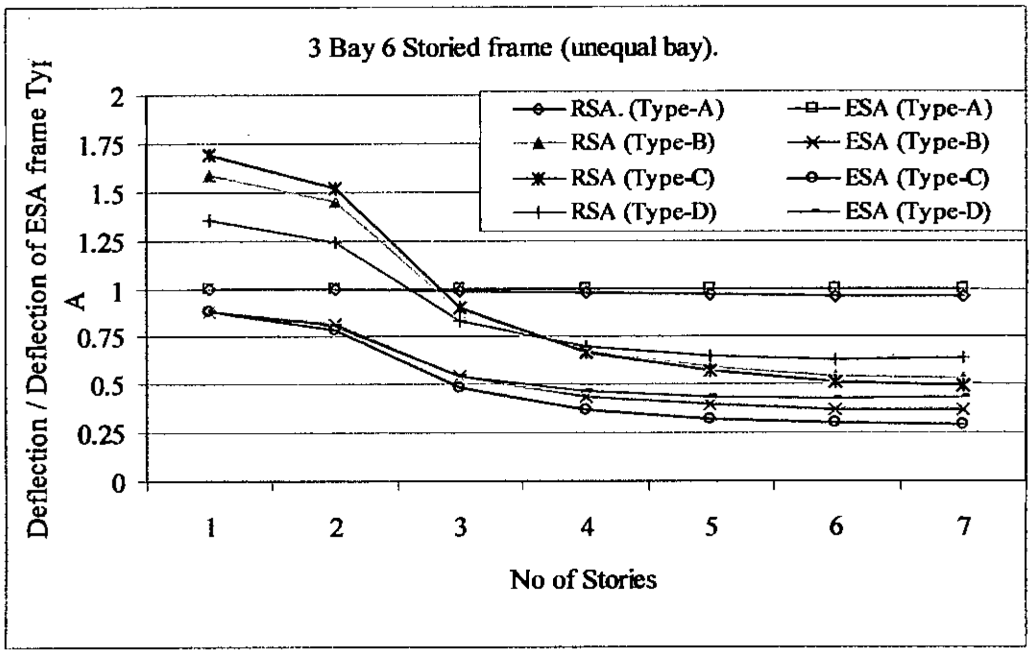


(f) 5 bay 9 storied frame



(g) 5 bay 12 storied frame

Fig. 5.8 continue

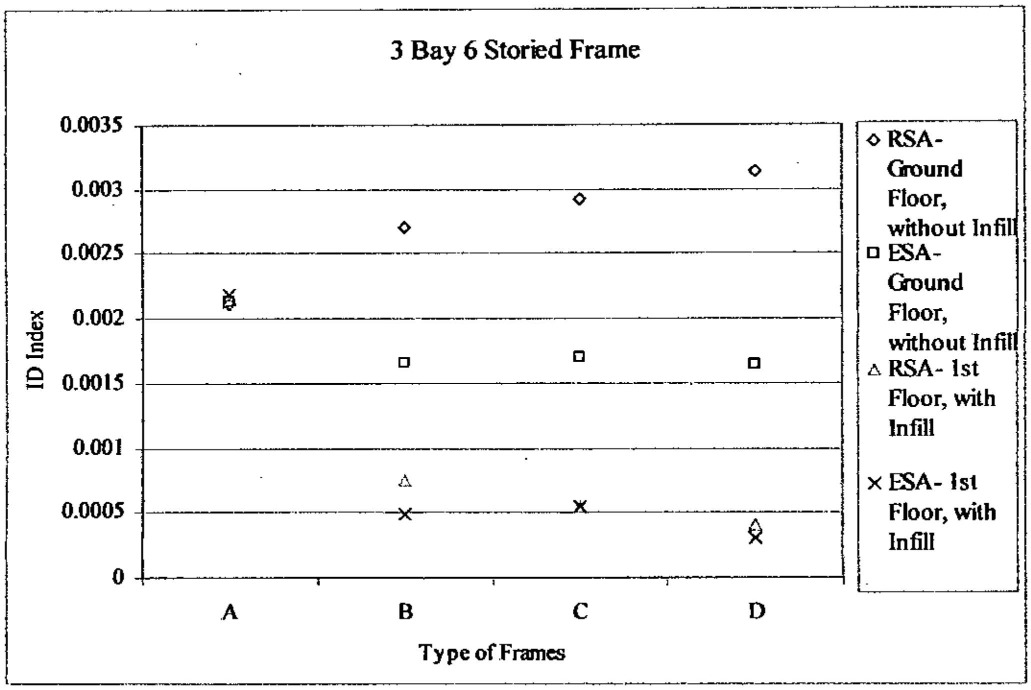


(h) 3 bay 6 storied frame (unequal bay)

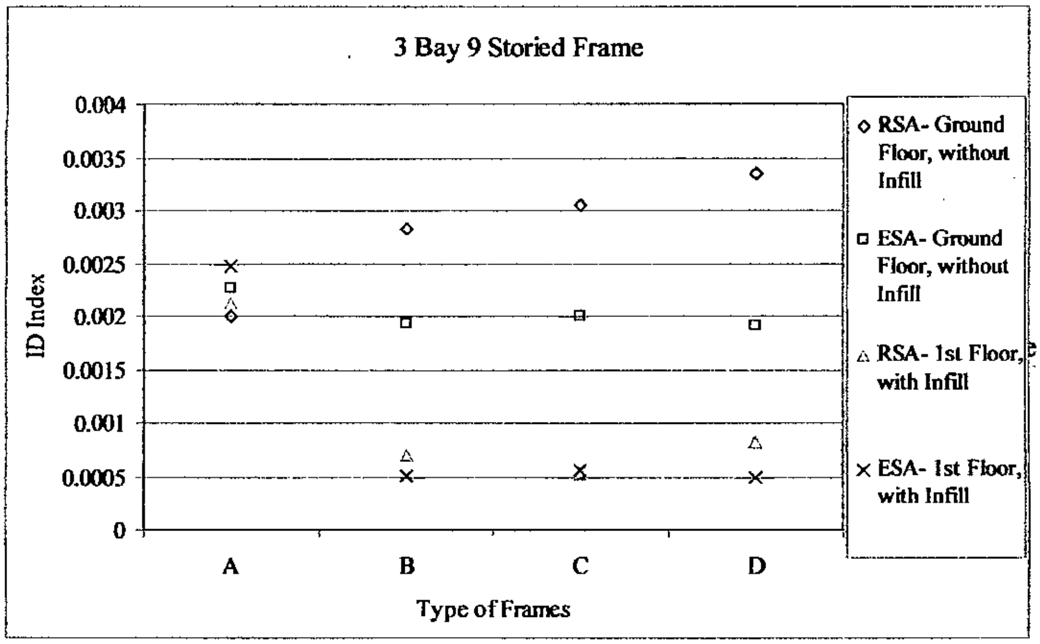
Fig. 5.8 Storey wise deflection Pattern of different frames with respect to Equivalent Static Analysis (ESA) of bare frame

Inter story drift (ID) is one of the most important parameters for serviceability of any structure. In this research it is found that the Equivalent Static Analysis and the Response Spectrum Analysis give different nature of ID in different conditions of the frame. It is found from the Fig. 5.9 that the ID values depend mainly on the presence and the pattern of infill components of the frame. For frame type A the ID values have only 5 to10 % increase in Equivalent Static Analysis with respect to the Response spectrum analysis. But for other types of frames the results are reverse i.e, the ID values attain higher percentage in Response Spectrum Analysis rather than the Equivalent Static Analysis. All these results are shown in Fig. 5.9. In this figure ID is shown only for that story which have no infill and the adjacent story have infill components. The bare frame ID values are also given for the same location.

Figures 5.10 and 5.11 show the ID values for different wall thickness and E_m. In this case it is observed that increase in the wall thickness and E_m values give increasing nature in ID values but in total story deflection or drift (shown in Fig. 5.12 and 5.13) it shows no significant change. After studying the moment nature, the influence of the variable wall thickness and Modulus of Elasticity are finally understood.

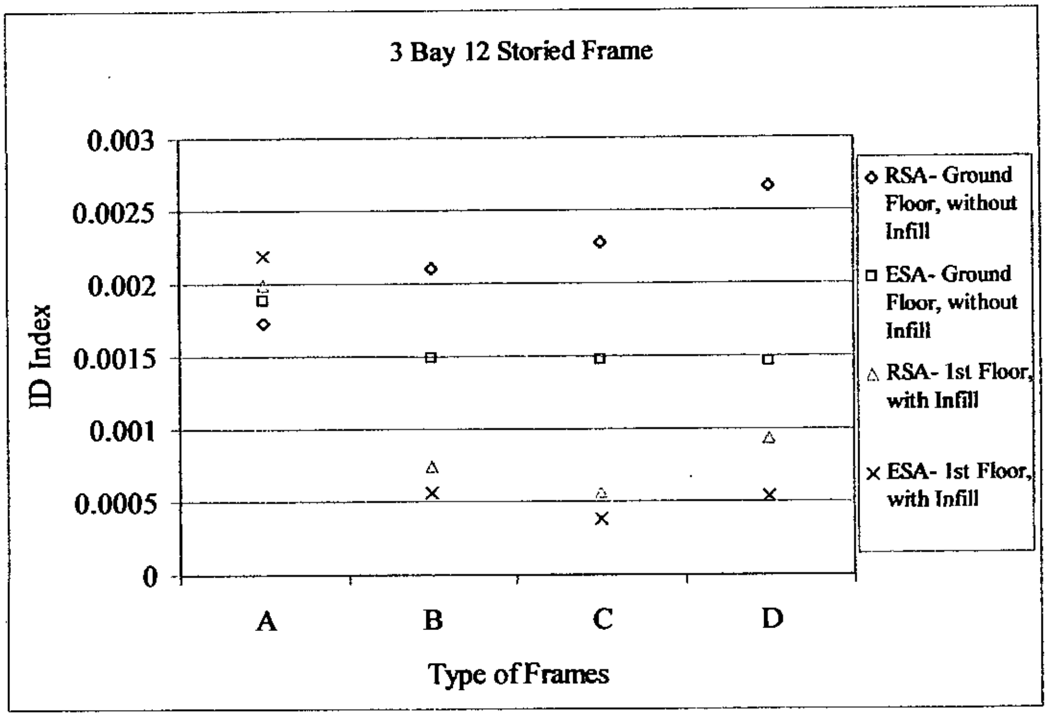


(a) 3 bay 6 storied frame

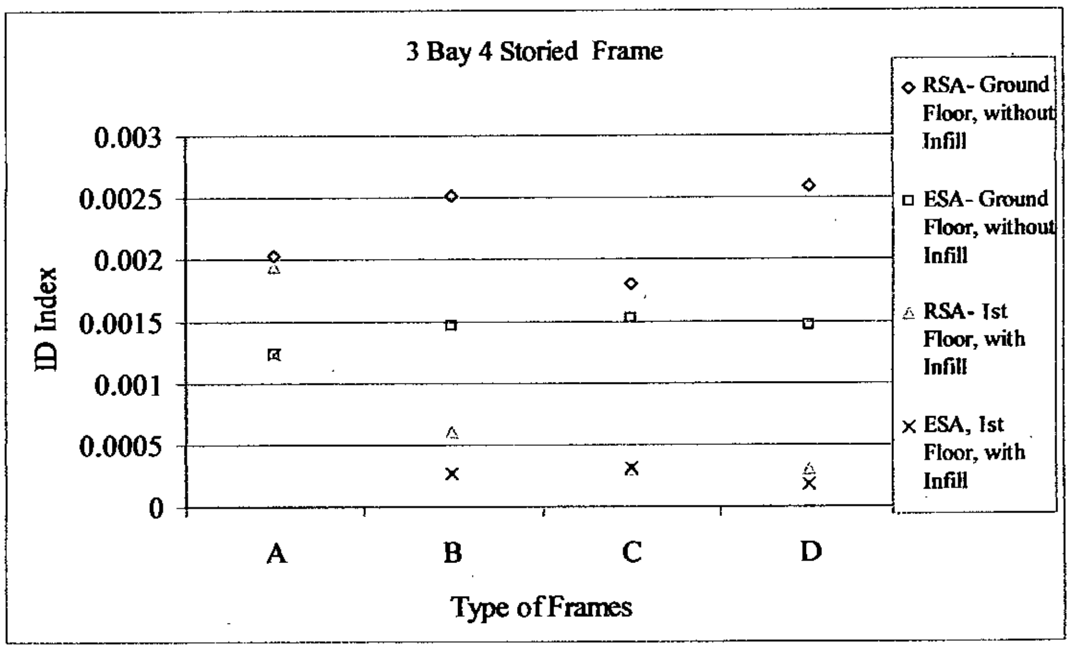


(b) 3 bay 9 storied frame

Fig. 5.9 continue

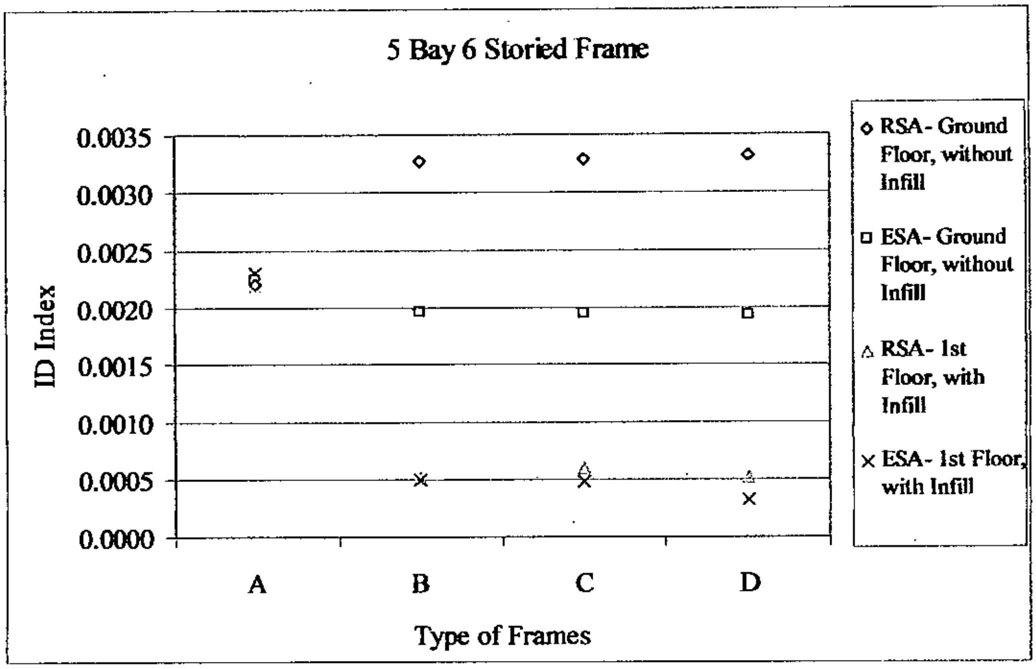


(c) 3 bay 12 storied frame

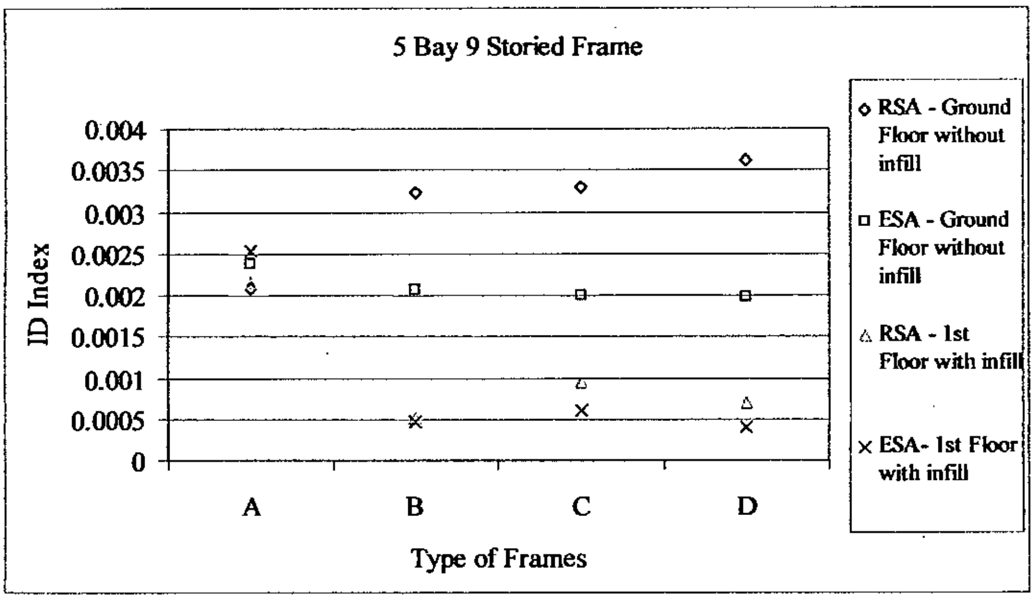


(d) 3 bay 4 storied frame

Fig. 5.9 continue



(e) 5 bay 6 storied frame



(f) 5 bay 9 storied frame

Fig. 5.9 Inter story Drift Index(ID) between of Ground Floor and 1st Floor

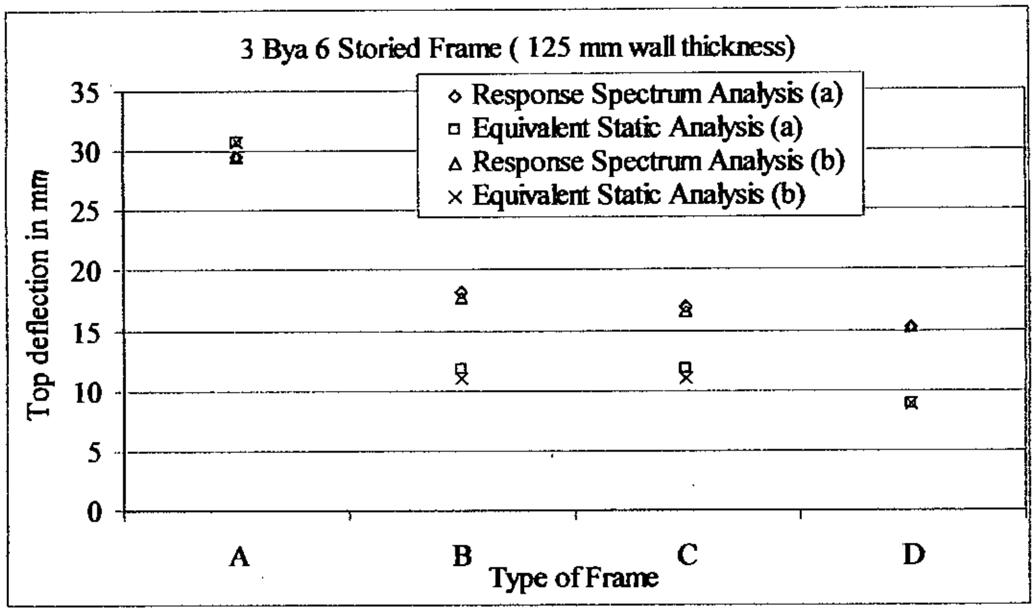


Fig. 5.10 Top Deflection of Frame with different E_m (a = 827.37 kN/ cm^2, b = 1396.18 kN/ cm^2)

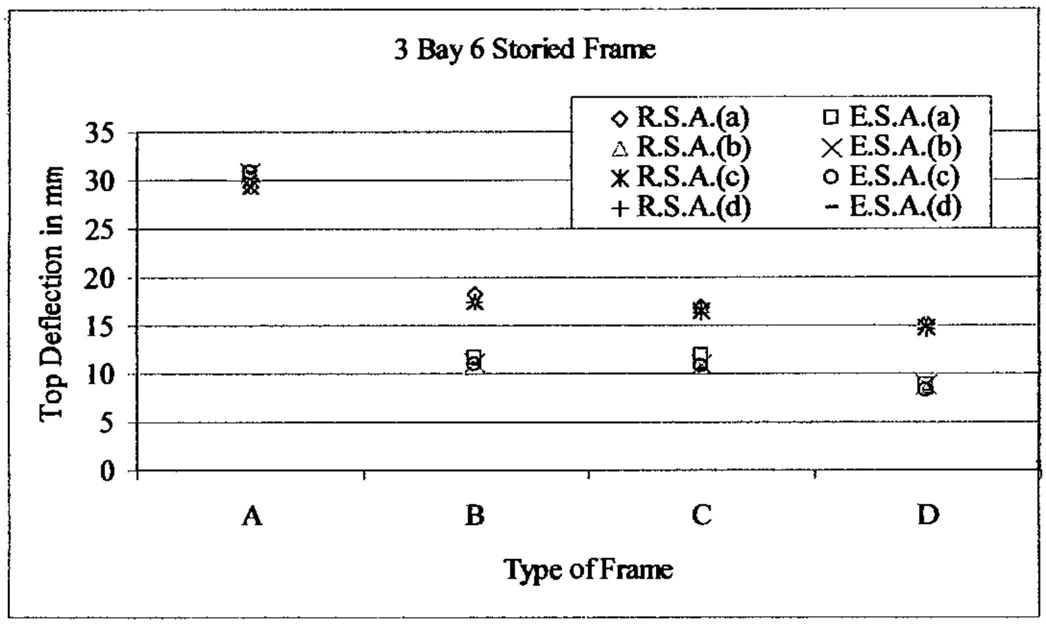


Fig. 5.11 Top Deflection of Frame with different wall thickness and E_m (a = 125 mm and 827.37 kN/ cm^2, b = 125 mm and 1396.18 kN/ cm^2, c = 250 mm and 827.37 kN/ cm^2, d = 250 mm and 1396.18 kN/ cm^2)

5.7 BEHAVIOUR OF THE FRAME DUE TO MOMENT

In this investigation two different analysis techniques have been used to find out the flexural effects on different types of frames due to the presence of infill wall. The results for three bay frames of different infill conditions and different properties of the infill components are summarized in Figures 5.14 to 5.21. From these figures it is found in the case of 6 storied frames, that the effect of thickness on infill components is not so pronounced and variation of E_m does not either produce a significant variation in the maximum moment. Due to this, the effect of variable thickness and value of E_m have not been considered in further analysis. In this study the effect of unequal length of the bays has also been studied. It is found that there is little effect of unequal bay length on moment variation. It is shown in Figures 5.12 and 5.13.

For further study all infill components have same properties (E_m = 827.37 kN/cm^2), thickness (125mm) and equal bay length. Considering these conditions the maximum moment variations are given in Figures 5.22 to 5.32 for both three and five bay frames. From the figure it is found that the amount of moment is increased in the frame due to the Response Spectrum Analysis (RSA), specially when the frames have infill components while some story have no infill. In three and five bay frames the moment increased depending on the increase of the infill percentage. When the frames have infill in all bays (Type D) it shows the highest amount of moment for 6, 9 and 12 storied frames. But for 4 storied frame it is observed from Fig. 5.33 that for any type of frame condition i.e, with or without infill the maximum moment is observed in the results of the Response Spectrum Analysis rather than the Equivalent Static Analysis. From the result it can be concluded that the Response Spectrum Analysis (RSA) procedure is more conservative analysis. Figures 5.34 and 5.35 show the member and member ends of the frames where the maximum moments observe.

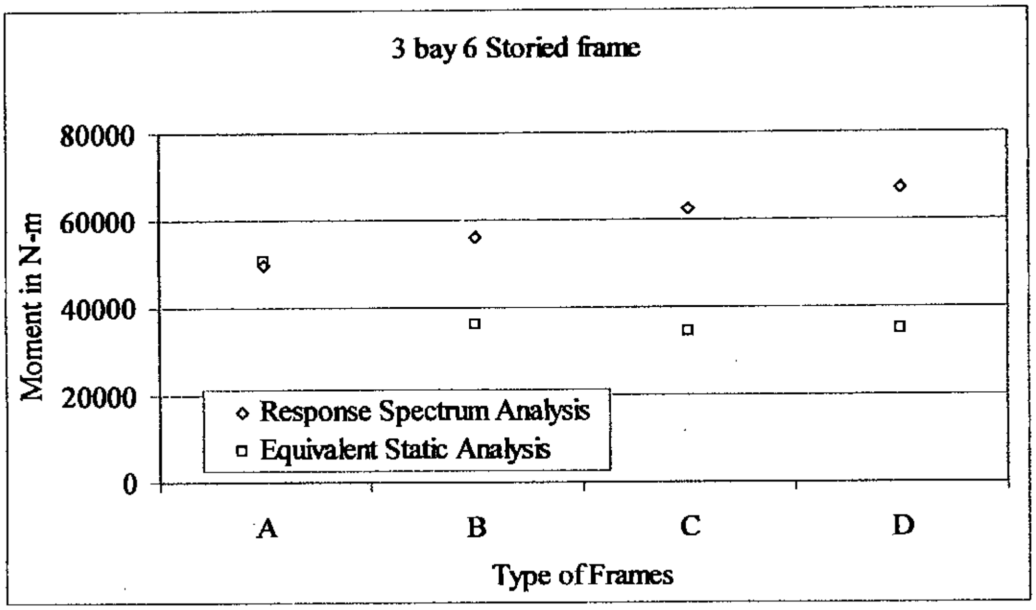


Fig. 5.12 Maximum Beam Moment for unequal bay

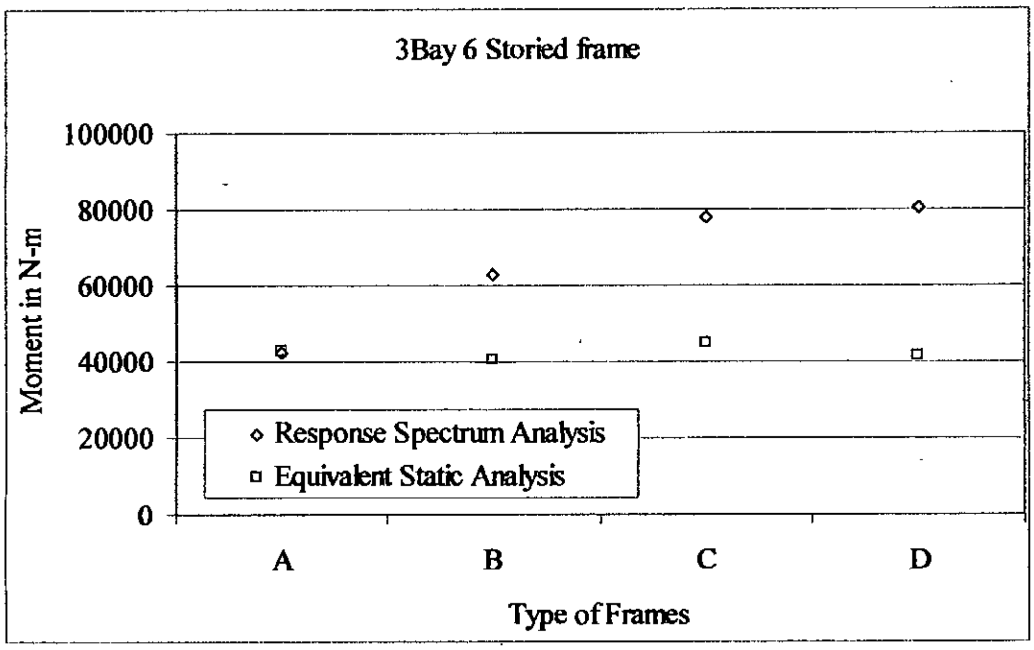


Fig. 5.13 Maximum Column Moment for unequal bay

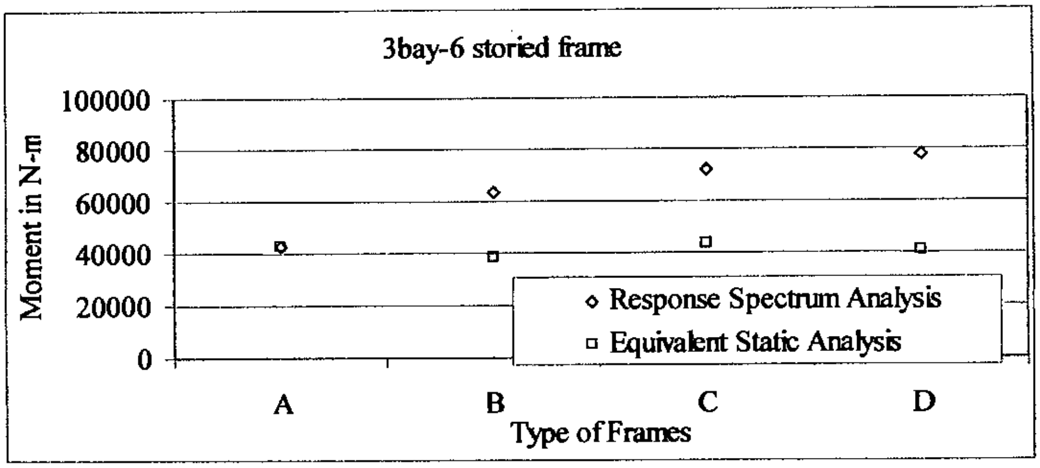


Fig. 5.14 Maximum Column moment, 125 mm wall and E_m = 827.37 kN/cm^2

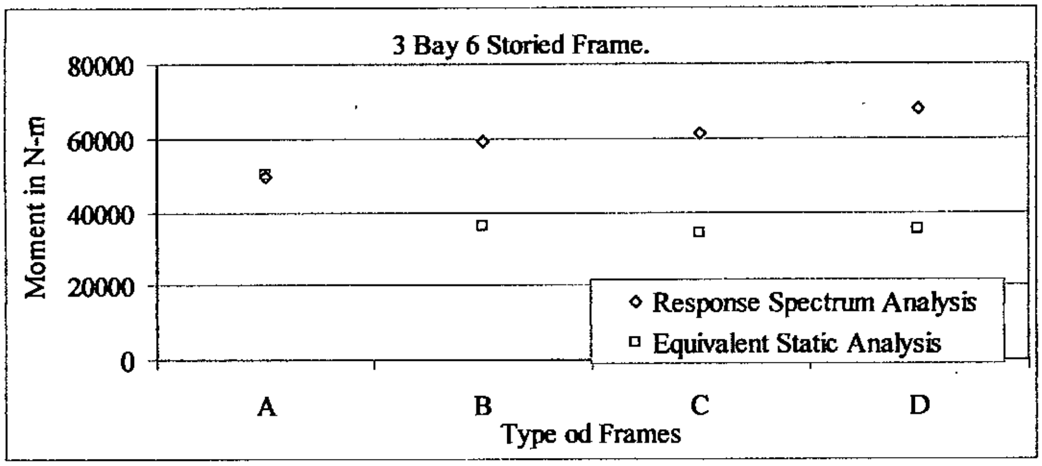


Fig. 5.15 Maximum Beam moment, 125 mm wall and E_m = 827.37 kN/cm^2

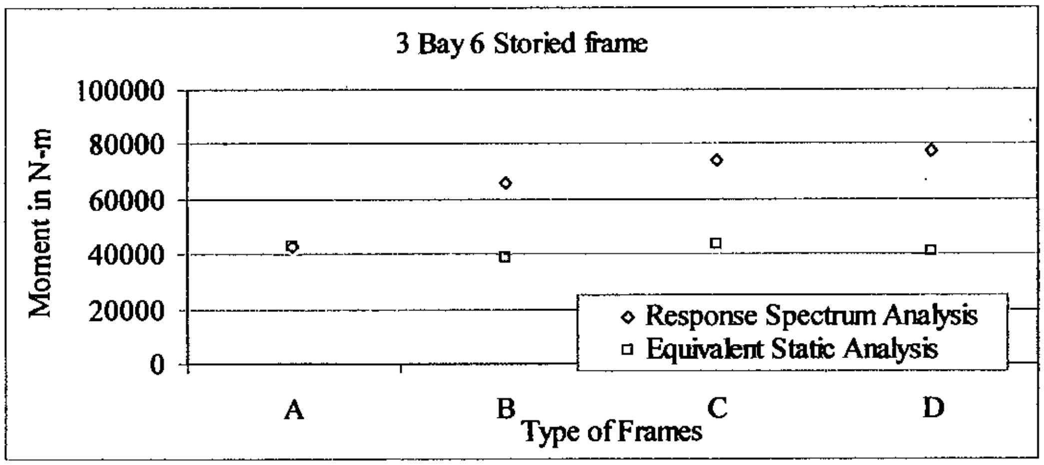


Fig. 5.16 Maximum Column moment, 125 mm wall and E_m = 1396.18 kN/cm^2

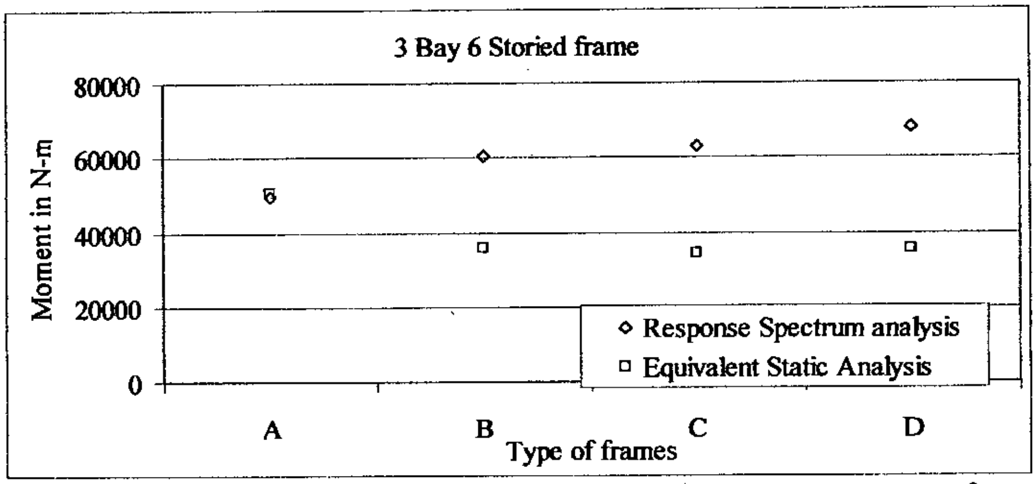


Fig. 5.17 Maximum Beam moment, 125 mm wall and $E_m = 1396.18$ kN/cm^2

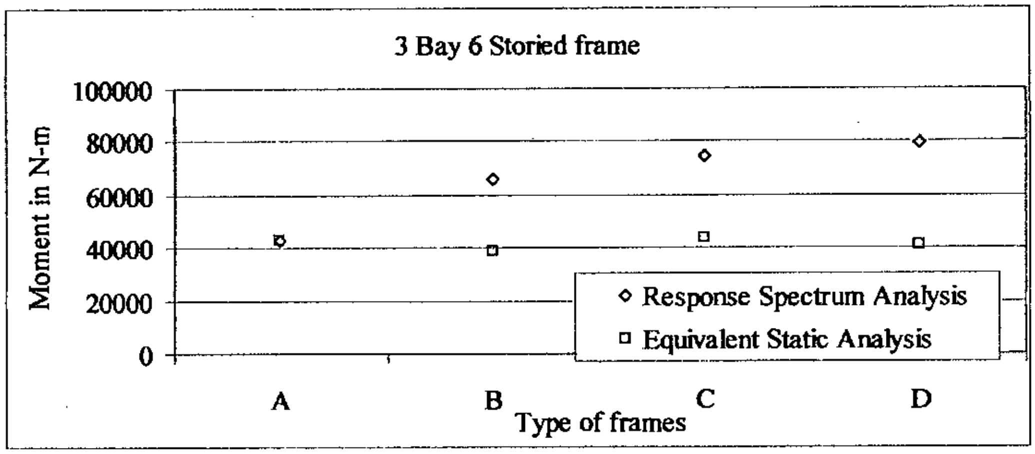


Fig. 5.18 Maximum Column Moment, 250 mm wall and $E_m = 827.37$ kN/cm^2

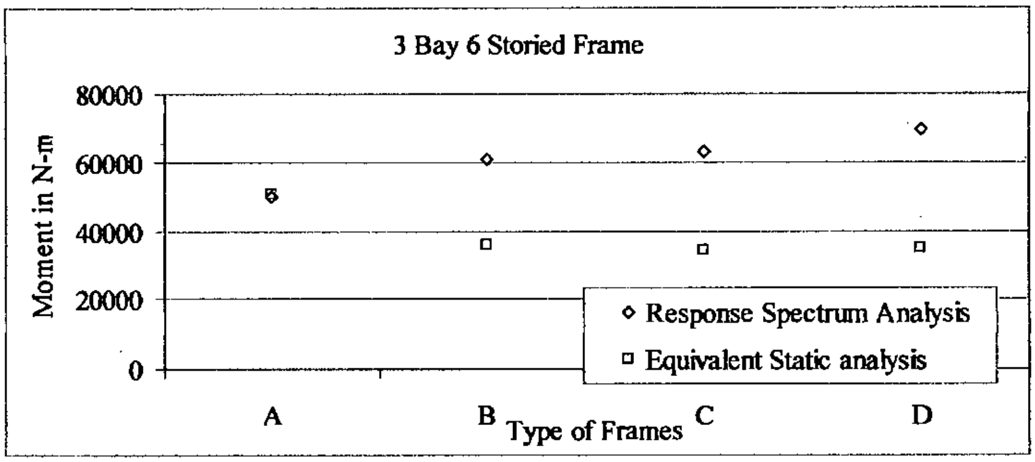


Fig. 5.19 Maximum Beam Moment, 250 mm wall and $E_m = 827.37$ kN/cm^2

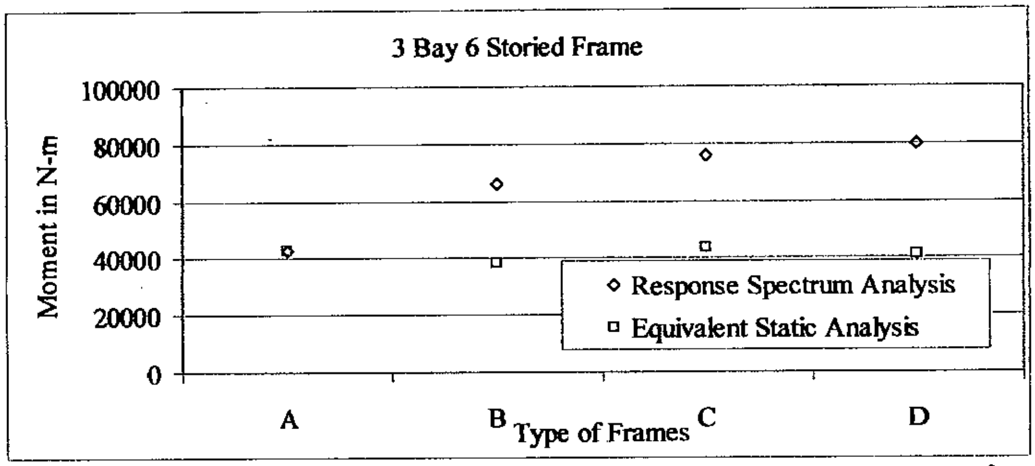


Fig. 5.20 Maximum Column Moment, 250 mm wall and E_m = 1396.18 kN/cm^2

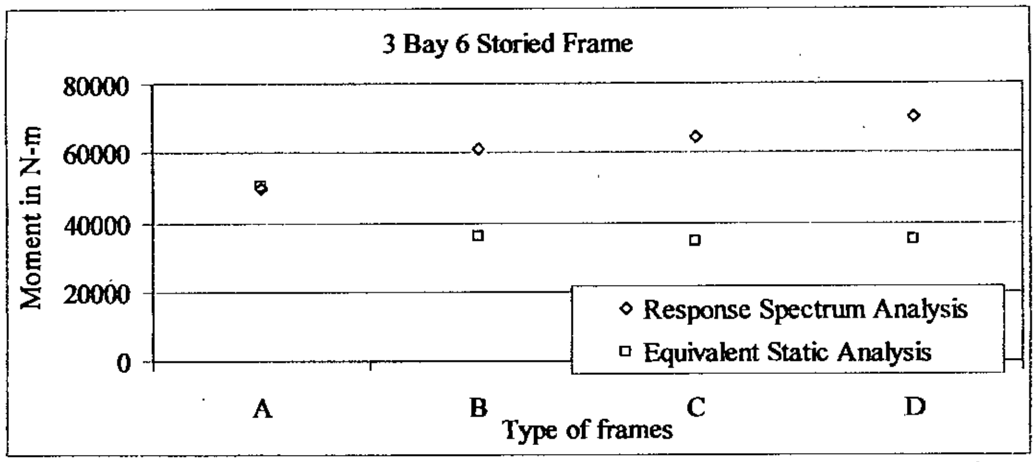


Fig. 5.21 Maximum Beam Moment, 250 mm wall and E_m = 1396.18 kN/cm^2

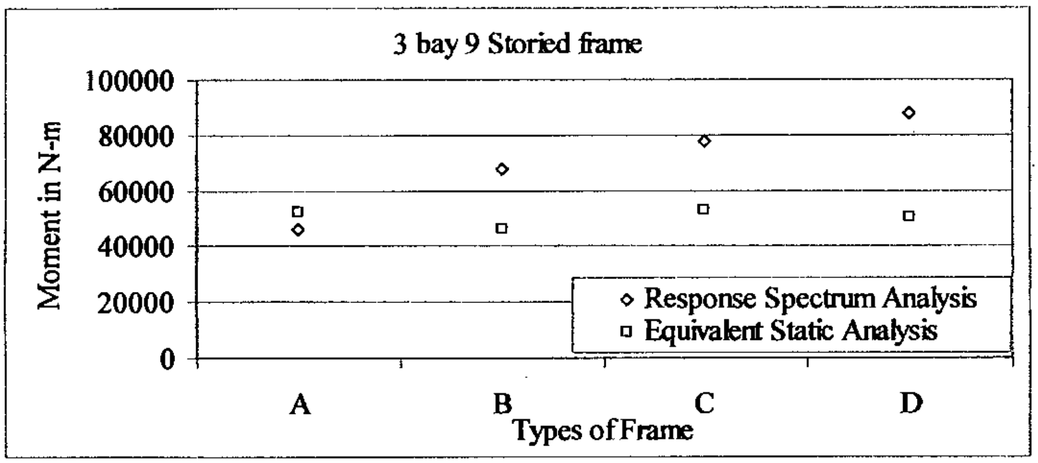


Fig 5.22 Maximum Column Moment, 125 mm wall and E_m = 827.37 kN/cm^2

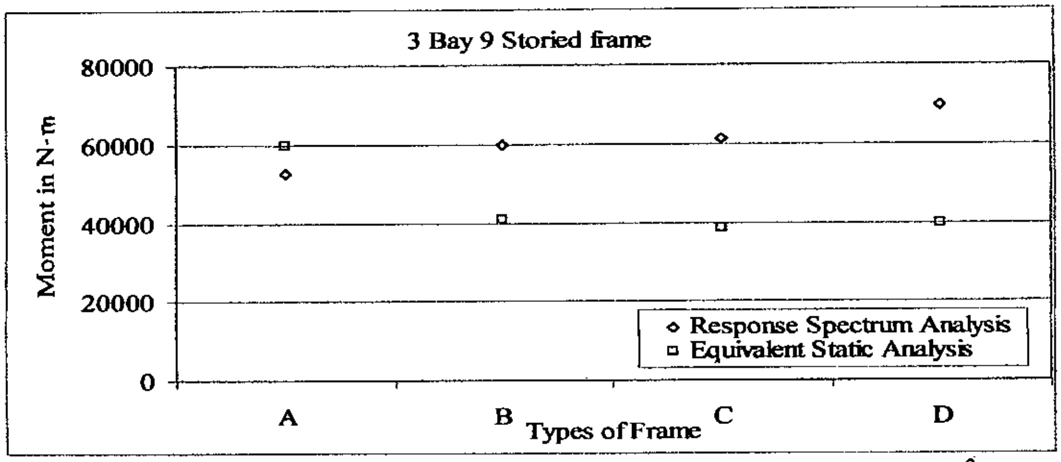


Fig. 5.23 Maximum Beam Moment, 125 mm wall and E_m = 827.37 kN/cm^2

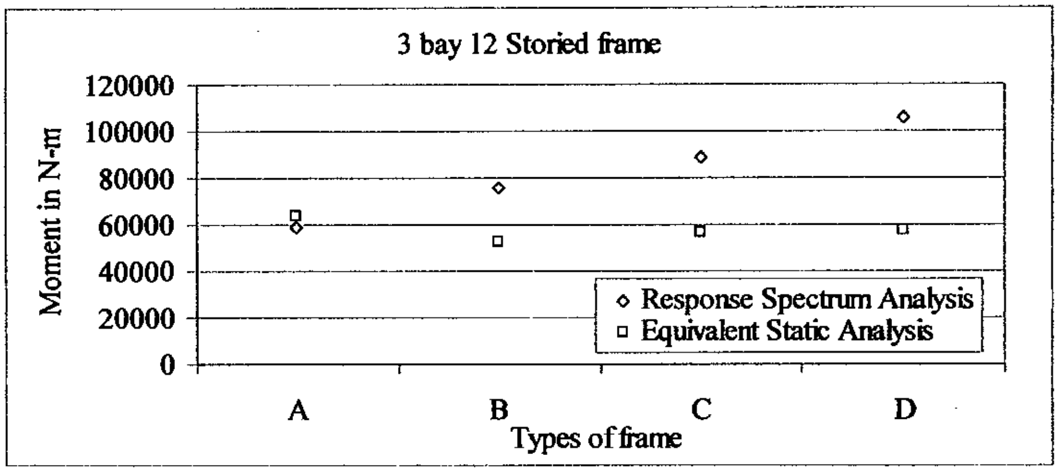


Fig. 5.24 Maximum Column Moment, 125 mm wall and E_m = 827.37 kN/cm^2

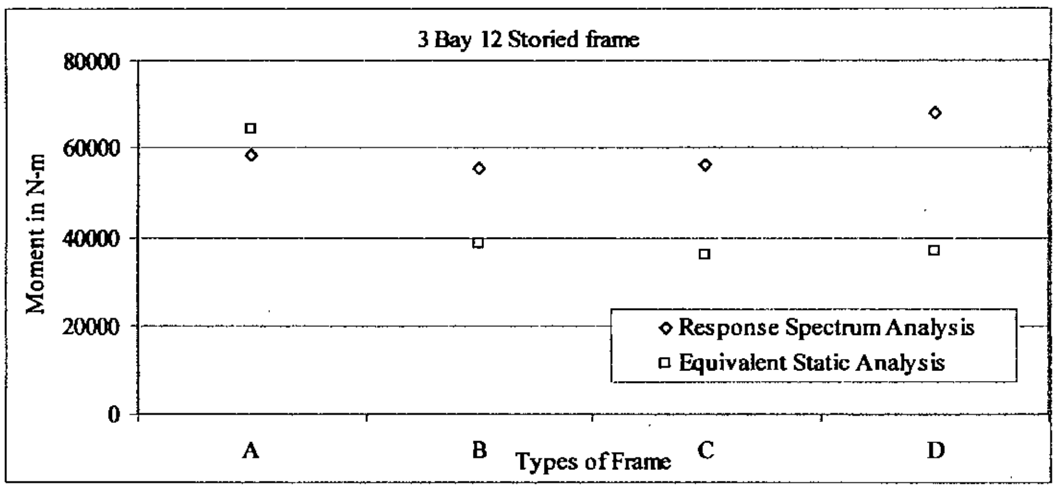


Fig. 5.25 Maximum Beam Moment, 125 mm wall and E_m = 827.37 kN/cm^2

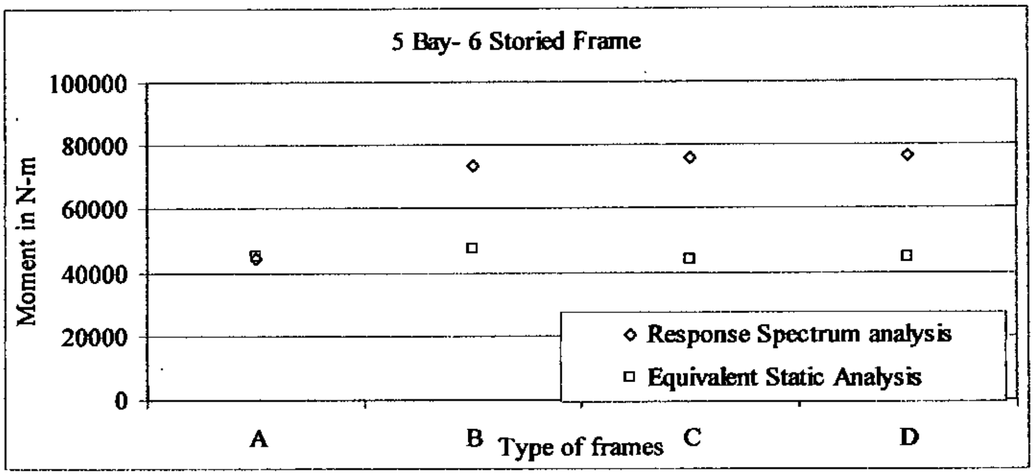


Fig. 5.26 Maximum Column Moment, 125 mm wall and E_m = 827.37 kN/cm^2

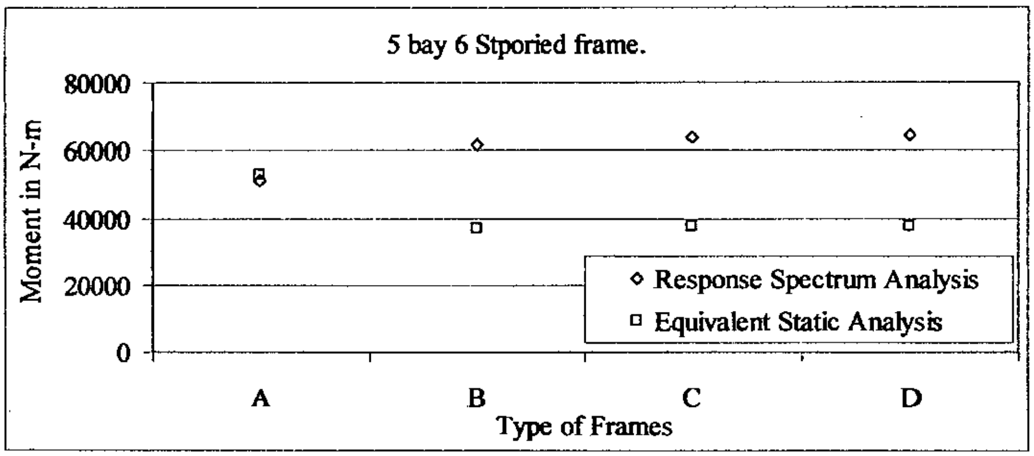


Fig. 5.27 Maximum Beam Moment, 125 mm wall and E_m = 827.37 kN/cm^2

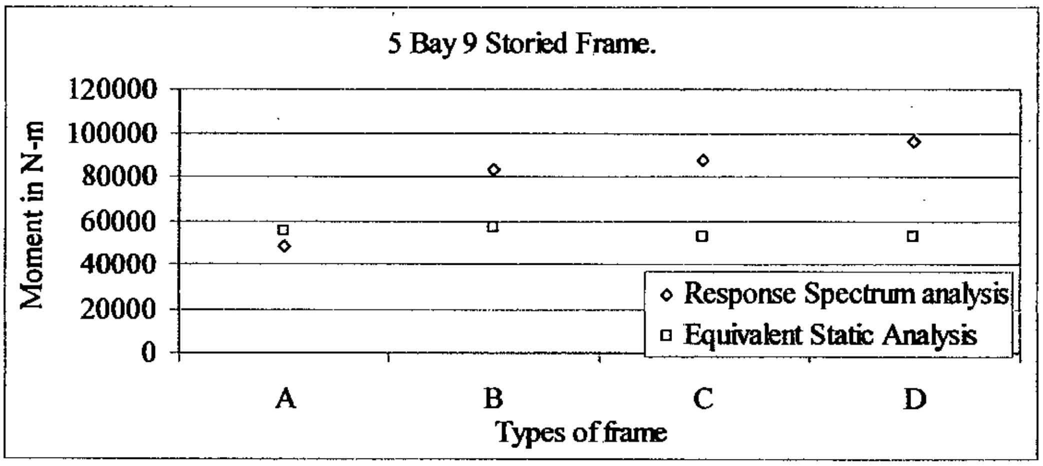


Fig. 5.28 Maximum Column Moment, 125 mm wall and E_m = 827.37 kN/cm^2

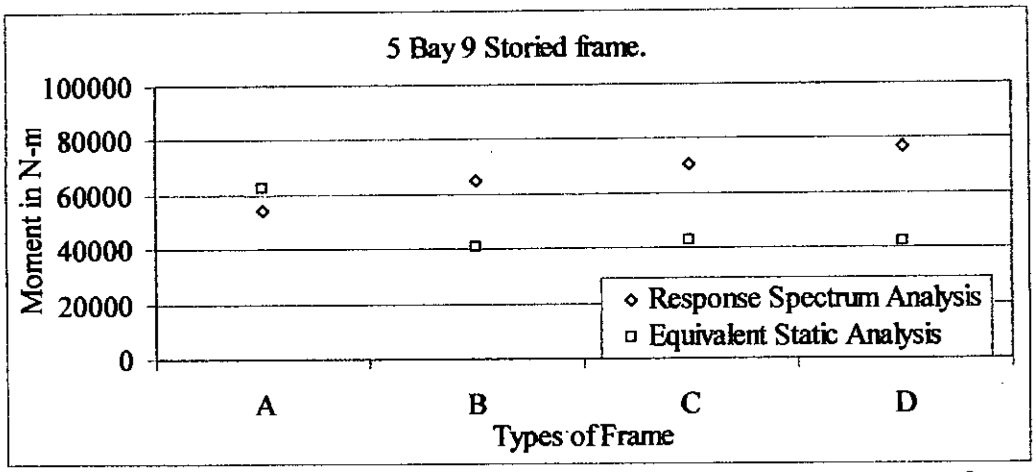


Fig. 5.29 Maximum Beam Moment, 125 mm wall and E_m = 827.37 kN/cm^2

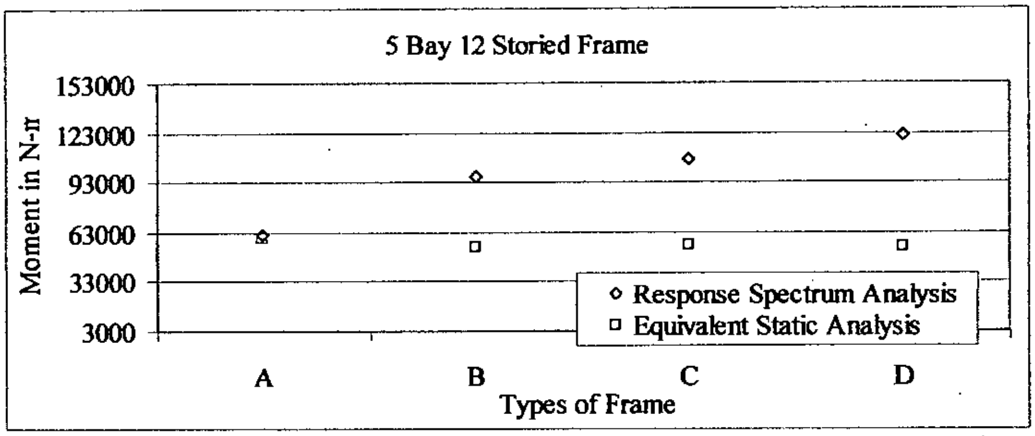


Fig. 5.30 Maximum Column Moment, 125 mm wall and E_m = 827.37 kN/cm^2

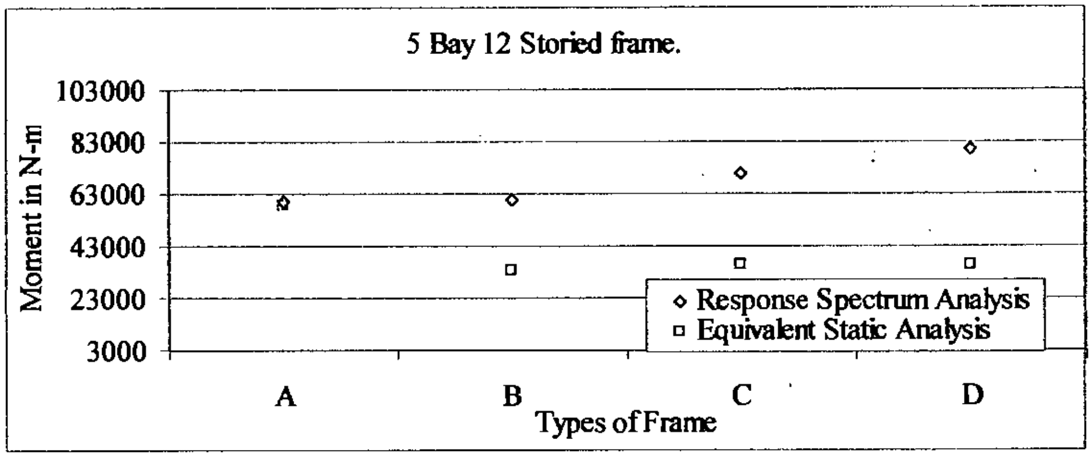
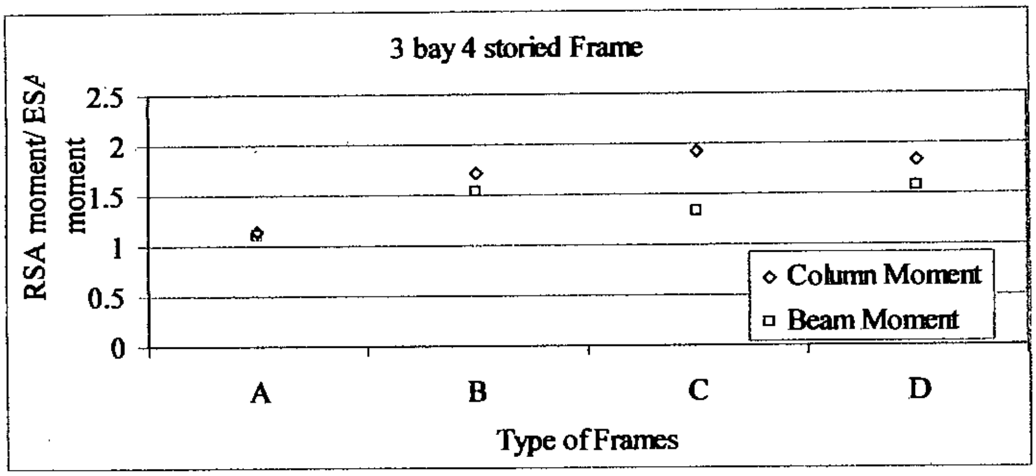
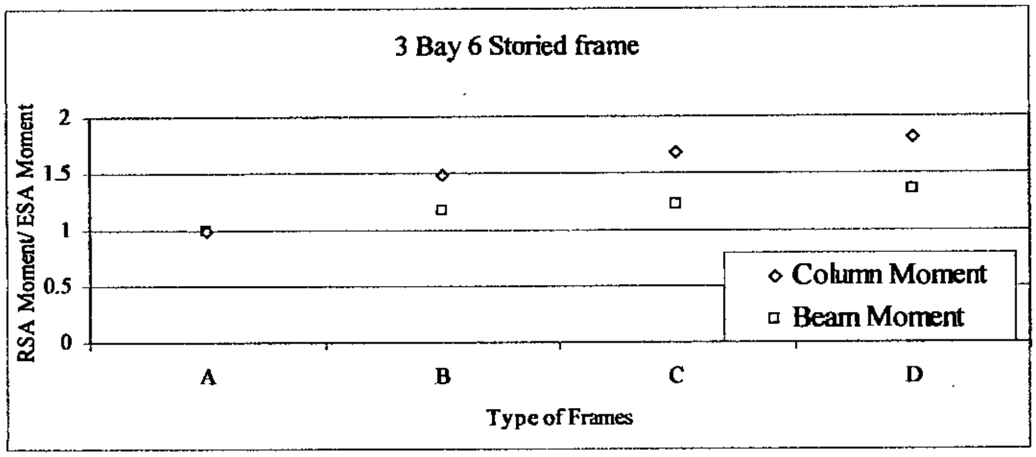


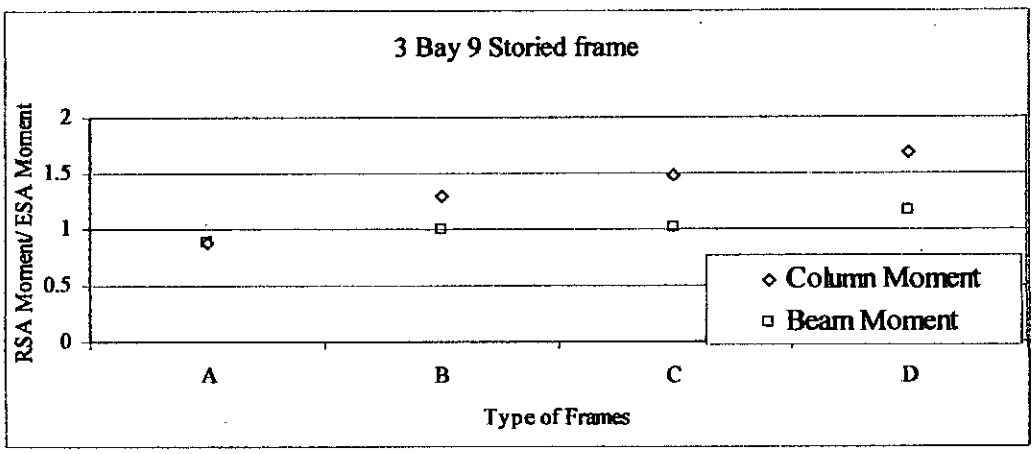
Fig. 5.31 Maximum Beam Moment, 125 mm wall and E_m = 827.37 kN/cm^2



(a) 3 bay 4 storied frame

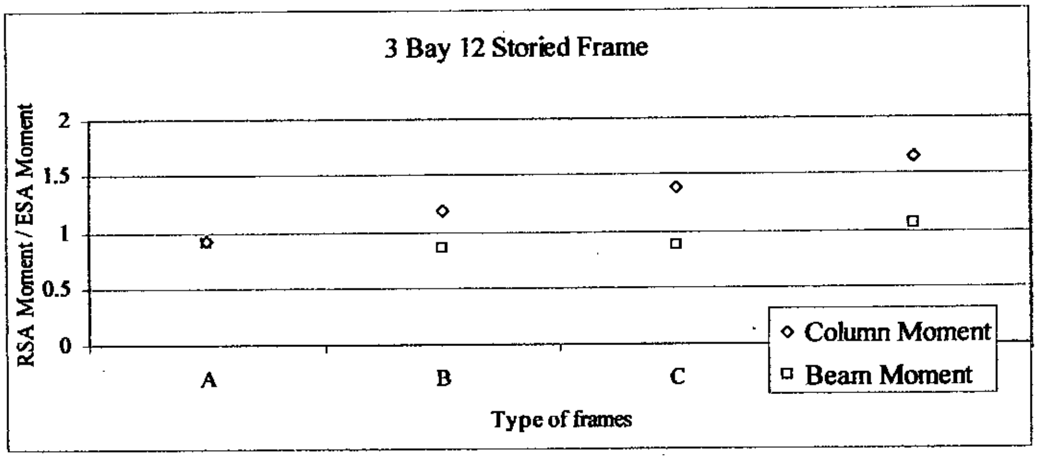


(b) 3 bay 6 storied frame

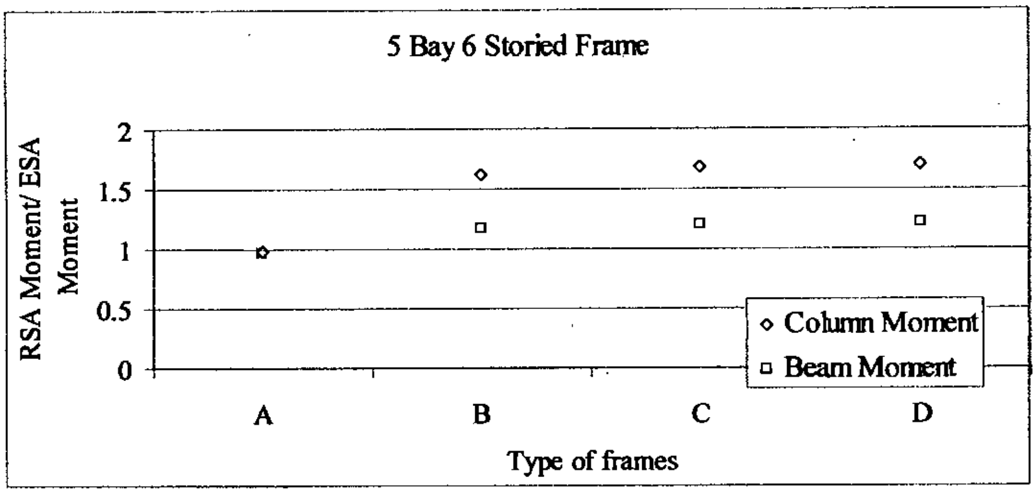


(c) 3 bay 9 storied frame

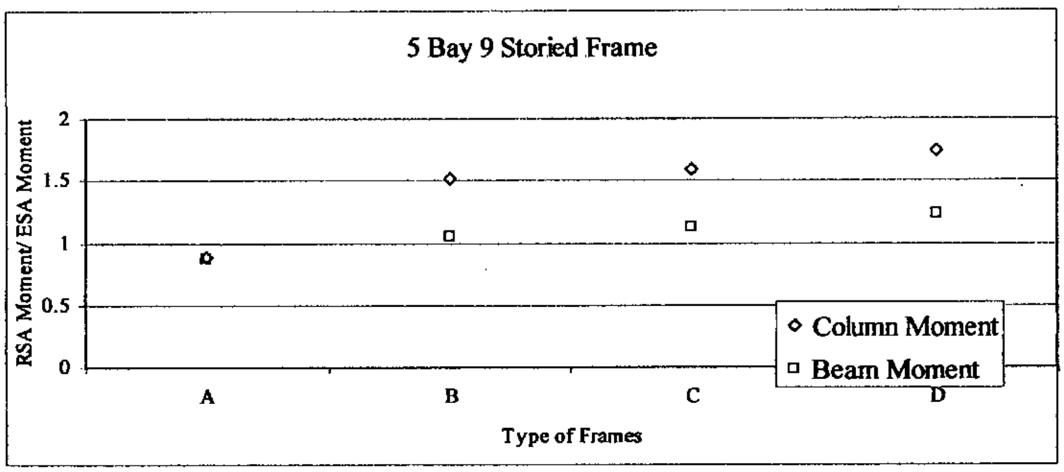
Fig. 5.32 continue



(d) 3 bay 12 storied frame

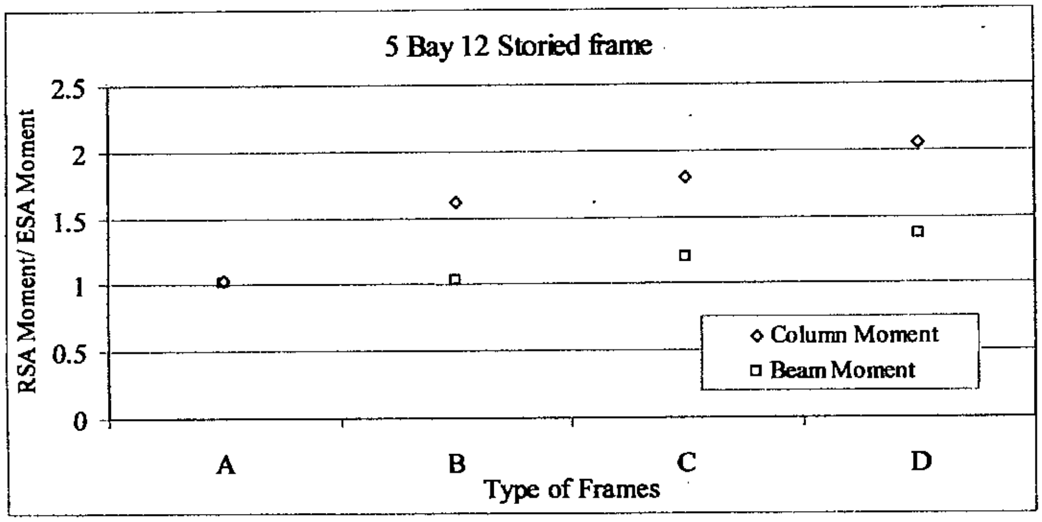


(e) 5 bay 6 storied frame



(f) 5 bay 9 storied frame

Fig. 5.32 continue



(g) 5 bay 12 storied frame

Fig. 5.32 Maximum Moment of Response Spectrum Analysis (RSA) of different frames with respect to Equivalent Static Analysis (ESA) of bare frame

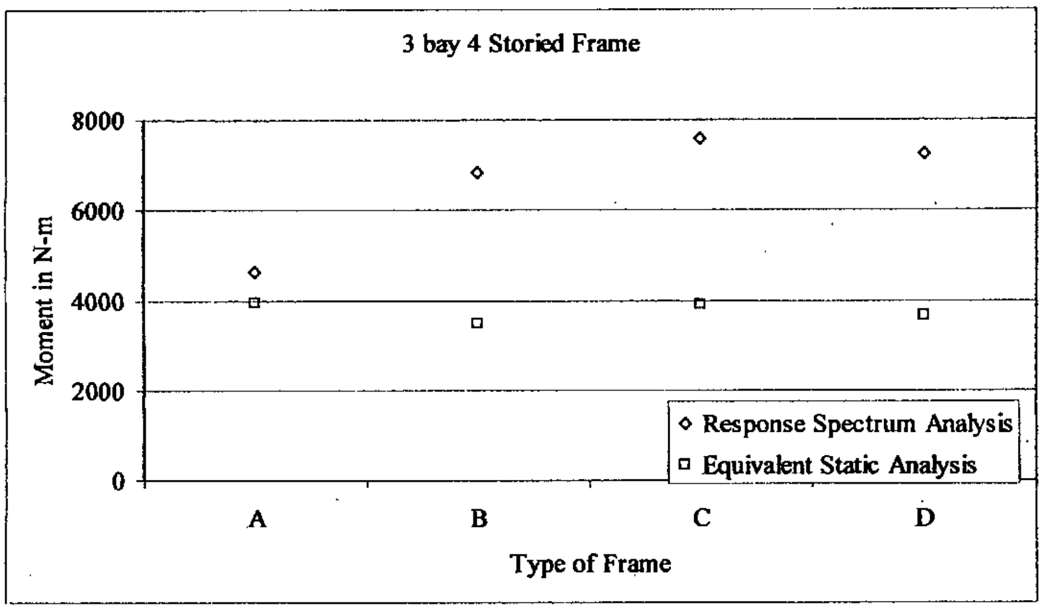


Fig. 5.33 (a) Column Moment

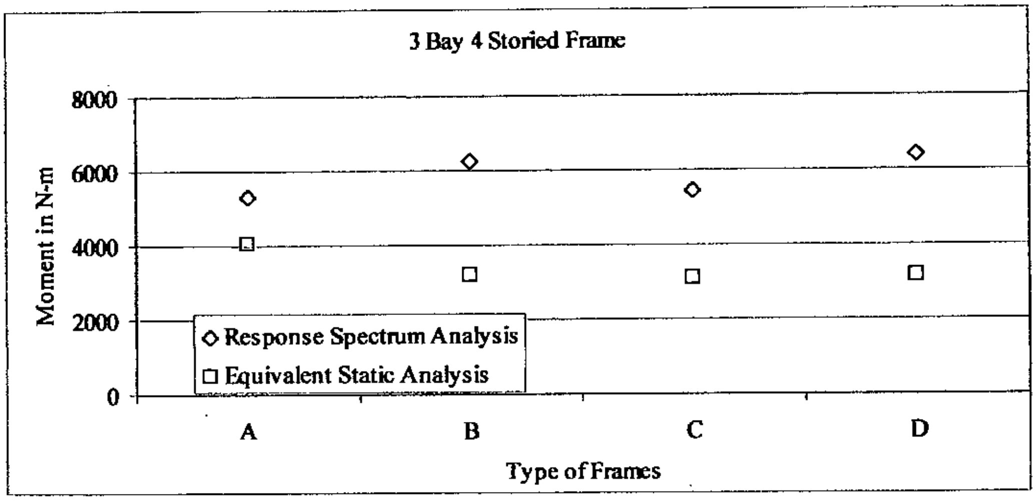


Fig. 5.33 (b) Beam Moment

Fig: 5.33 Maximum Moment of Response Spectrum Analysis (RSA) and Equivalent Static Analysis (ESA)of different type of frames

5.8 BASE SHEAR OF THE FRAME

Base shear of the frame is shown in fig. 5.34 for nine storied frame, all other frames shows the same type of graphs i.e., response spectrum analysis gives higher base shear depending of the infill percentage on the frame with respect to the equivalent static analysis procedure.

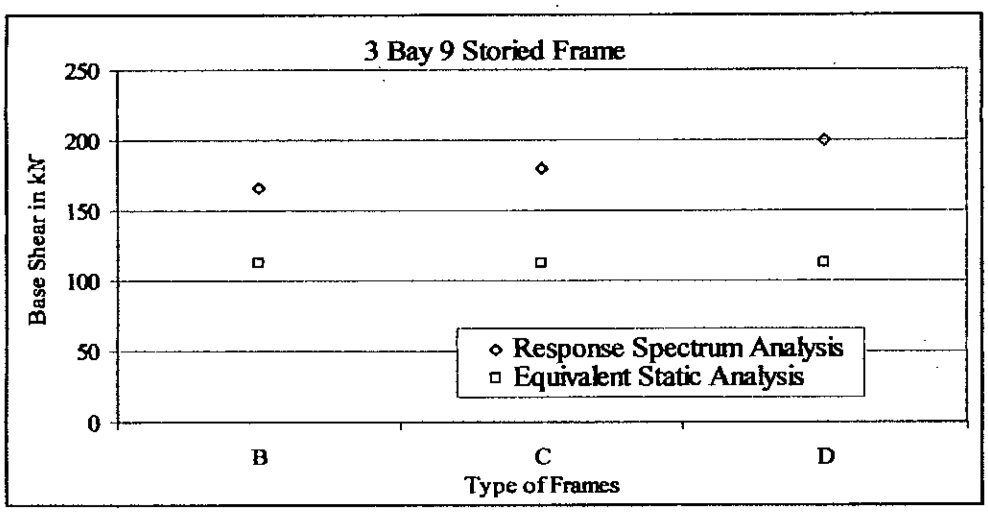
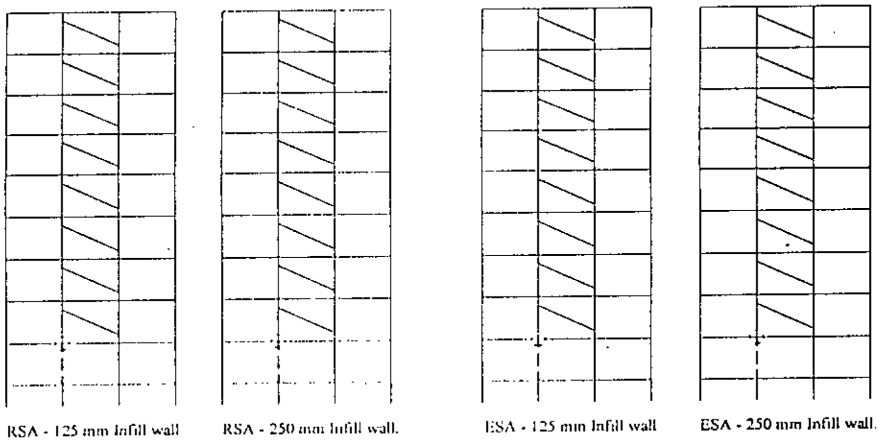
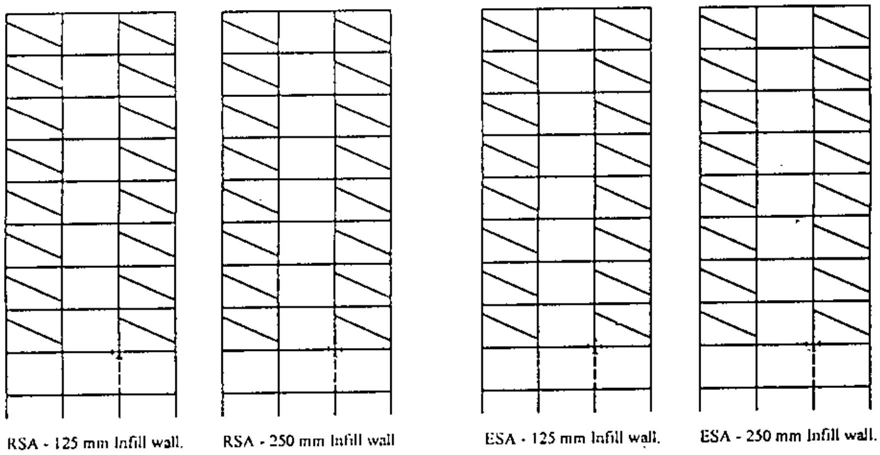


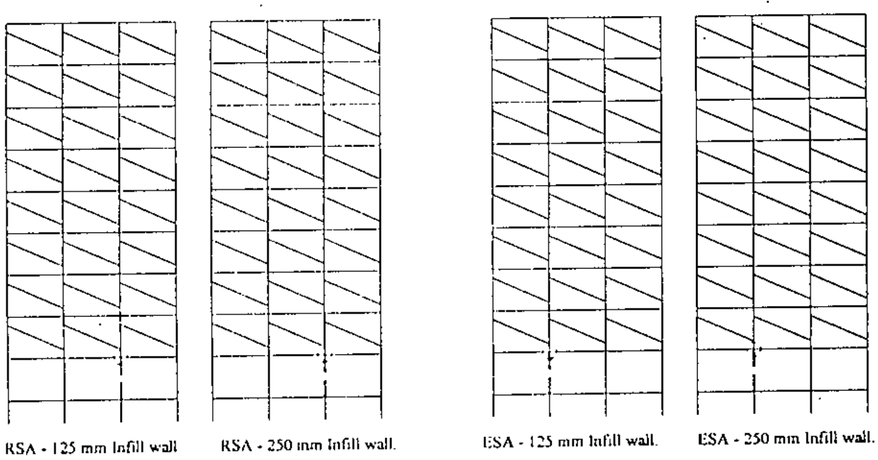
Fig. 5.34 Base shear of Response Spectrum Analysis (RSA) and Equivalent Static Analysis (ESA)of different type of frames.



Maximum Column Moment for 9 story frame (Type-B).

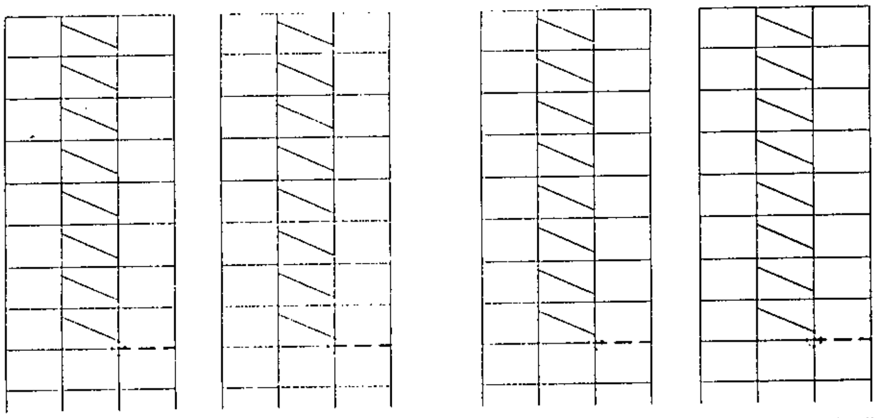


Maximum Column Moment for 9 story frame **(Type-C)**



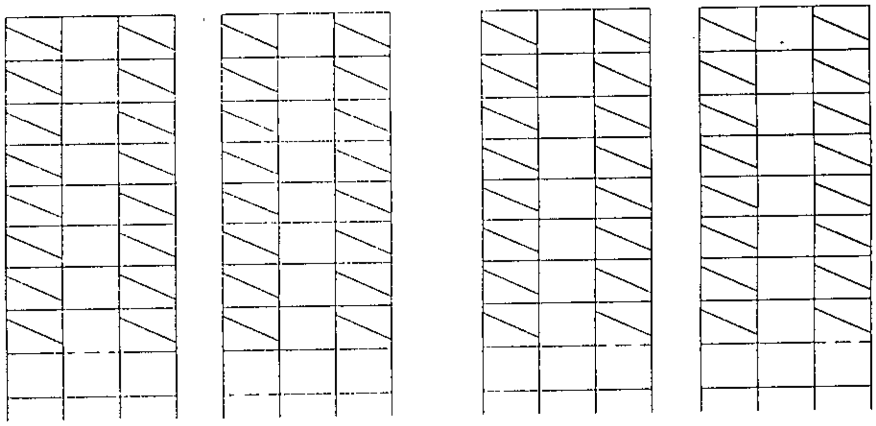
Dotted line indicates the member on which the maximum moment occur and the position of this moment showss by the circle.

Fig. 5.34 Maximum Column Moment for 9 story frame (Type -D).



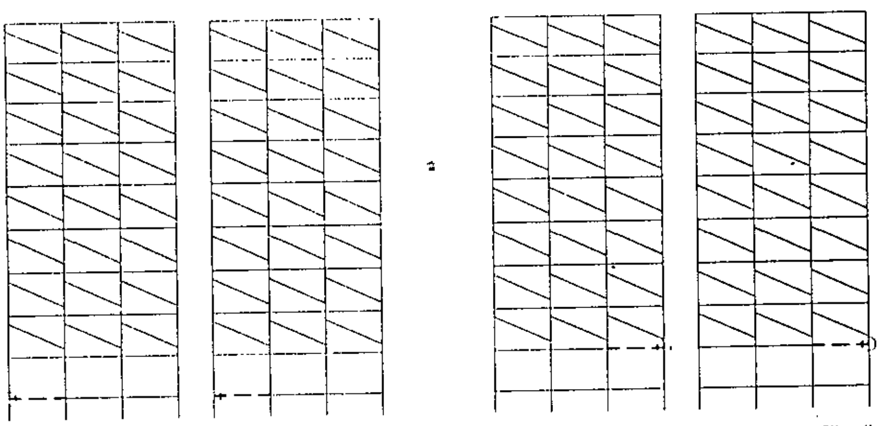
RSA - 125 mm Infill wall RSA - 250 mm Infill wall ESA - 125 mm Infill wall. ESA - 250 mm Infill wall.

Maximum Beam Moment for 9 story frame (Type-B).



RSA - 125 mm Infill wall. RSA - 250 mm Infill wall. ESA - 125 mm Infill wall ESA - 250 mm Infill wall.

Maximum Beam Moment for 9 story frame (Type-C).



RSA - 125 mm Infill wall RSA - 250 mm Infill wall ESA - 125 mm Infill wall. ESA - 250 mm Infill wall.

Dotted line indicates the member on which the maximum moment occur and the position of this moment showss by the circle.

Fig. 5.35 Maximum Beam Moment for 9 story frame (Type D).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

Due to the demand of various facilities like open utility or parking space, structures have some floor without any infill components, while the other floors have some infill walls. In this type of structures the combined behavior of infilled-frame is a complex problem, especially for lateral loading. The Bangladesh National Building Code (BNBC, 1993) provides specification for the lateral loading and also gives some guidance for the analysis techniques (Equivalent Static Analysis, Response Spectrum Analysis and Time History Analysis). BNBC specifies when and why one should consider the Equivalent Static or Response Spectrum Analysis technique for solving a structure. In BNBC it is stated that the Equivalent Static Analysis gives conservative results for regular structures. In the definition of the regular structure, BNBC gives a guide-line (plan and elevation regularities) but nothing is mentioned about the infill walls on the frame. In day-to-day practice of design offices, infills are usually not modeled during analysis. Hence absence of infills in some floor is not regarded as an irregularity as such. In many earthquakes it was found that a large number of structures were failed at column beam joints. These joints point faced a high concentration of Shear and Moment at the time of horizontal loading. The present study has made an effect to find out the condition on which the maximum moments occur and at the same time the Analysis technique on which the designers should resort to. For this purpose four, six, nine and twelve storied frames having three and five bays were studied in this research work. Some frames having no infill components, some having variable patterns of infill with or without window opening were studied. All types of frames were analyzed by Equivalent Static Analysis and Response Spectrum Analysis techniques. Then the results have been compared. It has been observed that the lateral sway was significantly reduced when the infills were considered in both the analysis techniques. The moments were significantly increased when infills were considered in the Response Spectrum Analysis.

6.2 LIMITATIONS

The structural damage observed during several earthquakes in the past is very educative in identifying the structural systems. During an earthquake the nature and amount of displacement of a structure depend on its stiffness and the inertia of its masses. Normally in structural analysis it is considered that the equivalent static analysis is more conservative against ground shaking for regular structures. In the analysis usually only the bare frame effect is considered but not the effect of the infill components. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength and ductility of the structures having vertical discontinuity of infill components. The major limitations of the research work are :

a) The masonry infill panels are modeled as an equivalent struts,
b) There is no variation in the story height of the building frame.
c) All the structural components having the same strength.
d) Analysis performed up to the elastic limit of the material.

6.3 CONCLUSIONS

A study using different numerical analyses was performed on the effect of infill walls on the behavior of frames due to lateral loading. Four, six, nine and twelve story frames having three and five bays were studied. The infill walls were provided in different patterns and conditions with different thickness and compressive strength of the masonry infill in the frame. Strength of the frame components was kept unchanged in all the cases. With the limitation and scope of this study the following conclusions can be drawn.

i) In the Bare Frame Structure, the Equivalent Static Analysis gives conservative values for deflection and moments instead of the Response Spectrum Analysis Technique.

ii) Results of the Equivalent Static Analysis showed variation regarding the effect of the presence of infill walls in frame. In some cases it showed lower values of moments with respect to the bare frame in other cases larger values. Whereas the results of the Response Spectrum Analysis consistently showed greater values of moments of infilled frames with one bare storey compared to the totally bare frame, depending on the amount of masonry infill in the frame. The maximum variation in moment between frame Type A and Type D was observed 200% higher in Response Spectrum Analysis with respect to the Equivalent Static one.

iii) For six, nine and twelve storied frame the Equivalent Static Analysis shows slightly higher amount of moment than the Response Spectrum Analysis in the case of frame type A. But for types B, C and D the results were reverse. For the four story frame, the Response Spectrum Analysis showed higher values of moments than the Equivalent Static Analysis for all types of frames.

iv) The thickness and strength of the infill components have little bearing on the variation of results between the analysis techniques.

v) In case of deflection, the Equivalent Static Analysis showed higher values than the Response Spectrum Analysis for frame type A, for six, nine and twelve storied frames. For all other types of frames i.e, type B, C and D, the Response Spectrum Analysis gave higher values than the Equivalent Static Analysis.

vi) Presence of windows on the infill panels showed some variation in moments and deflection of the frame. However the Response Spectrum Analysis gave conservative results than the Equivalent Static Analysis.

6.4 RECOMMENDATIONS FOR FURTHER STUDY

It is believed that due to the limitations of this present study as is mentioned in section 1.2, a complete guideline for the designers could not be developed here. The present study may be regarded as a preliminary work for an extensive research work on the effect of infill walls on frames due to horizontal loading. Therefore, some guidelines for future theoretical and experimental study on this topic may be recommended. The recommendations are:

i) This analysis may be performed with variations of relative stiffness of beam, with respect to column.

ii) This analysis may be performed by using non linear property of the materials.

iii) Analysis can be made on the basis of Strong column and Weak-beam or Weak-column Strong-beam phenomena.

iv) This analysis may be performed by using various types of infill components.

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APPENDIX A

Maximum Moment of Column and Beams of the Frame

Table A.1 Maximum moment for Column & Beams (3 bays & 6 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam (N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	377.87	378.555	441.88	446.864
B	565.714	342.485	523.367	318.809
C	640.847	384.297	543.209	304.275
D	686.058	361.606	602.566	310.375

Table A.2 Maximum moment records for Column & Beams (3 bays & 9 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam(N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	46576.36	52387.01	52831.81	59989.07
B	68034.34	46016.92	59909.44	40840.82
C	77785.85	53278.87	61467.48	38491.78
D	88170.57	50009.17	69611.02	39799.75

Table A.3 Maximum moment records for Column & Beams (3 bays & 12 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam(N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	59045.93	63900.45	58249.85	64462.49
B	75852.7	52811.6	55373.32	38648.1
C	88405.17	56643.91	56178.89	35809.53
D	105990.1	57459.08	68130.69	37021.5

Table A.4 Maximum moment records for Column & Beams (5 bays & 6 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam(N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	44380.48	45099.3	51472.67	52831.93
B	73465.36	47301.85	61823.73	36691.23
C	76307.32	44116.63	63930.94	37578.91
D	76577.5	44317.57	64476.38	37336.29

Table A.5 Maximum moment records for Column & Beams (5 bays & 9 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam(N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	48366.97	54935.75	54353.15	62330.99
B	83486.38	56784.99	65202.77	40743.57
C	87294.75	52692.55	70528.64	42615.06
D	96227.93	52682.38	77161.12	42326.47

Table A.6 Maximum moment records for Column & Beams (5 bays & 12 story)

Type of Frame	Maximum Moments for Column (N-m)		Maximum Moments for Beam(N-m)	
	Analysis Technique		Analysis Technique	
	Response Spectrum	Equivalent Static	Response Spectrum	Equivalent Static
A	61312.74	59451.08	59934.62	58635.69
B	96371.26	54112.34	60666.21	33964.59
C	106707.4	54584.25	70653.67	36128.06
D	121749.8	54220.77	80058.99	35676.7

Table A.7 Maximum moment records for Columns having different infill thickness and strength. (3 bays & 6 story)

Type of frame	Analysis Technique							
	Response Spectrum		Equivalent Static		Response Spectrum		Equivalent Static	
	Infill Thickness (inch) and Strength (Ksi)							
	5 ; 4	10 ; 4	5 ; 4	10 ; 4	5 ; 8	10 ; 8	5 ; 8	10 ;8
A	377.87	377.87	378.555	378.555	377.87	377.87	378.555	378.555
B	565.714	585.605	342.485	341.643	581.739	585.637	341.937	341.872
C	640.847	662.309	384.297	385.201	658.056	672.218	385.054	385.393
D	686.058	702.057	361.606	360.752	686.378	709.393	361.595	360.041

Table A.8 Maximum moment records for Beams having different infill thickness and strength. (3 bays & 6 story)

Type of frame	Analysis Technique							
	Response Spectrum		Equivalent Static		Response Spectrum		Equivalent Static	
	Infill Thickness (inch) and Strength (Ksi)							
	5 ; 4	10 ; 4	5 ; 4	10 ; 4	5 ; 8	10 ; 8	5 ; 8	10 ;8
A	441.88	441.88	446.864	446.864	441.88	441.88	446.864	446.864
B	523.367	541.161	318.809	318.018	537.149	540.78	317.996	317.975
C	543.209	561.161	304.275	303.286	557.578	569.57	303.486	302.812
D	602.566	617.308	310.375	310.021	602.37	624.24	310.335	309.865

APPENDIX B

MAXIMUM DEFLECTION OF THE FRAMES

Table B.1 Maximum deflection (mm) records for frames having no infill (3 bays & 6 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.2192	1.7272	1.8796	2.032	1.2192	1.0414	1.0668	1.0414
1	7.5692	9.8552	10.668	11.4554	7.5946	6.0198	6.1722	5.969
2	13.97	12.1158	12.2936	12.6746	14.1732	7.4676	7.7978	6.8834
3	19.5834	13.97	13.6144	13.5128	20.1168	8.7376	9.017	7.5184
4	24.1808	15.6972	14.8844	14.224	25.0698	9.9314	10.1854	8.1026
5	27.559	17.145	16.002	14.8082	28.7274	10.9728	11.1506	8.5598
6	29.5148	18.2626	16.9418	15.2654	30.861	11.7856	11.8618	8.89

Table B.2 Maximum deflection (mm) records for frames having no infill (3 bays & 9 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.1684	1.8542	2.0066	2.2098	1.2954	1.2446	1.27	1.27
1	7.1882	10.3124	11.176	12.2682	8.1026	7.0358	7.2898	7.0104
2	13.5636	12.4206	12.7508	14.7574	15.5194	8.5852	8.9916	8.509
3	19.5072	14.3764	14.224	15.494	22.7076	10.0838	10.3886	9.1694
4	24.8412	16.4846	15.7988	16.3576	29.464	11.7348	11.9126	9.906
5	29.591	18.5928	17.4498	17.2212	35.6362	13.4112	13.4112	10.6172
6	33.6804	20.6756	19.0754	18.034	41.0464	15.0876	14.8844	11.303
7	37.0332	22.6568	20.6756	18.796	45.5676	16.6878	16.2814	11.938
8	39.5732	24.511	22.225	19.4818	49.022	18.1864	17.5514	12.5222
9	41.1226	26.1366	23.647	20.116	51.282	19.5326	18.669	13.030

Table B.3 Maximum deflection in mm records for frames having no infill (3 bays&12 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.016	1.4732	1.6002	1.8796	1.6002	1.0414	1.8796	1.016
1	6.223	7.7978	8.4582	9.8806	8.4582	5.461	9.8806	5.3848
2	12.1666	10.033	10.1346	12.6746	10.1346	6.6294	12.6746	6.985
3	17.9578	12.1158	11.7348	13.6144	11.7348	7.8486	13.6144	7.5692
4	23.4188	14.4272	13.589	14.7066	13.589	9.6266	14.7066	8.255
5	28.4988	16.8656	15.5702	15.875	15.5702	11.43	15.875	9.144
6	33.1724	19.3802	17.653	17.0434	17.653	13.2842	17.0434	10.033
7	37.4142	21.9202	19.7866	18.2118	19.7866	15.1384	18.2118	10.922
8	41.2242	24.4094	21.9456	19.3802	21.9456	16.9672	19.3802	11.7856
9	44.5516	26.8478	24.1046	20.5232	24.1046	18.7452	20.5232	12.6492
10	47.2948	29.1592	26.1874	21.6408	26.1874	20.447	21.6408	13.462
11	49.4284	31.3436	28.2194	22.6822	28.2194	22.0218	22.6822	14.224
12	50.8508	33.3502	30.1752	23.6728	30.1752	23.4696	23.6728	14.9098

Table B.4 Maximum deflection (mm) records for frames having no infill (5 bays & 6 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.2446	2.1336	2.159	2.159	1.27	1.27	1.2446	1.2446
1	7.8486	11.9634	12.039	12.141	8.001	7.1628	7.0612	7.0104
2	14.4272	13.5382	13.817	13.716	14.884	8.6614	8.5344	8.001
3	20.1676	14.605	14.681	14.579	21.031	9.5504	9.1948	8.636
4	24.8412	15.6464	15.494	15.341	26.136	10.414	10.007	9.2456
5	28.2194	16.6116	16.205	16.002	29.895	11.2014	10.718	9.779
6	30.1498	17.4498	16.738	16.51	32.029	11.811	11.277	10.185

Table B.5 Maximum deflection (mm) records for frames having no infill (5 bays & 9 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.1938	2.1082	2.1844	2.3876	1.3462	1.3208	1.3208	1.3208
1	7.4422	11.8364	12.065	13.208	8.4836	7.5438	7.2898	7.2644
2	13.9446	13.3858	14.884	15.290	16.129	8.9916	9.1186	8.4582
3	19.9644	14.8082	15.697	15.849	23.495	10.16	9.8298	8.89
4	25.3746	16.3068	16.687	16.51	30.403	11.4554	10.642	9.398
5	30.1244	17.8308	17.678	17.145	36.652	12.7508	11.480	9.8806
6	34.1884	19.3294	18.669	17.703	42.113	14.0208	12.319	10.337
7	37.4904	20.7518	19.583	18.211	46.634	15.2146	13.182	10.744
8	39.9542	22.098	20.421	18.669	50.063	16.3068	13.919	11.125
9	41.402	23.3426	21.183	19.024	52.197	17.272	14.503	11.404

Table B.6 Maximum deflection in mm records for frames having no infill (5 bays & 12 story)

Floor/ Level	Analysis Technique							
	Response Spectrum				Equivalent Static			
	Type of Frame							
	A	B	C	D	A	B	C	D
Base	0	0	0	0	0	0	0	0
GB	1.0414	1.7272	1.8542	2.1336	1.016	0.9652	0.9398	0.9398
1	6.4516	9.0932	9.7282	10.9982	6.2738	5.1308	5.0038	4.9022
2	12.4968	10.795	12.9032	13.4366	12.2682	6.1722	6.7056	6.0452
3	18.3642	12.3952	13.9446	14.1224	18.288	7.2136	7.3152	6.4008
4	23.8506	14.1732	15.1638	14.9606	24.1046	8.5852	8.1534	6.8326
5	28.9306	16.0782	16.4846	15.8242	29.6672	9.9568	9.017	7.3914
6	33.5788	18.034	17.8562	16.6624	34.8996	11.4046	9.9314	7.8994
7	37.7698	19.9898	19.2532	17.4752	39.7256	12.827	10.8458	8.382
8	41.4782	21.9456	20.5994	18.2626	44.069	14.224	11.7856	8.89
9	44.6532	23.8506	21.9202	18.9992	47.879	15.5956	12.7762	9.4234
10	47.244	25.7048	23.1648	19.6596	51.054	16.8656	13.6906	9.8806
11	49.1744	27.4574	24.3586	20.2692	53.5432	18.0848	14.5288	10.287
12	50.419	29.1084	25.4762	20.828	55.245	19.177	15.24	10.6172

APPENDIX C

CALCULATION OF STRUT WIDTH, "*a*"

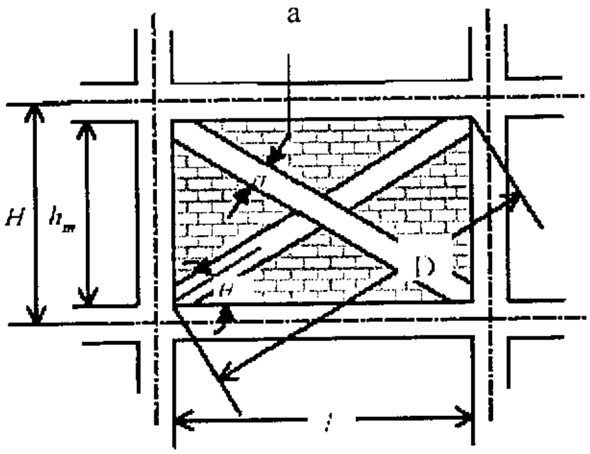

Fig. C.1 Strut geometry

The equivalent strut width, a, depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness shall be evaluated using the following equation.

$\lambda_1 H = H\,[\,(E_m\, t \sin 2\theta) \,/\, (4\, E_c\, I_{col}\, h_w)\,]^{1/4}$

Using this expression, Mainstone (1971) considered the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel as shown in below:

$a = 0.175 D(\lambda_1 H)^{-0.4}$

Where

a	is the equivalent strut width
t	thickness of the masonry infill panel
E_m	Modulus of elasticity of the masonry unit
E_c	Modulus of elasticity of concrete
h_m	Clear height of column member
I_{col}	Moment of inertia of the column
θ	Angle produce by the strut with the horizontal

Eccentricity of Equivalent Strut

The equivalent masonry strut is to be connected to the frame as structural member as shown by bold double-sided arrow in the following figure.

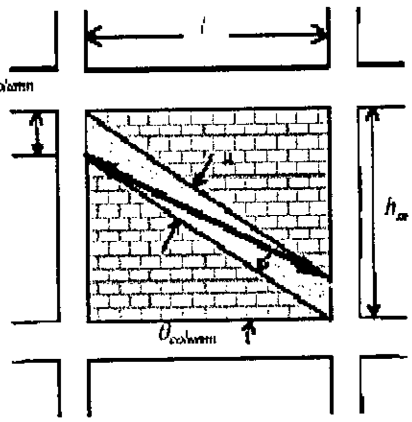

Fig. C.2 Equivalent masonry strut

The strut should be pin-connected to the column at a distance l_{column} from the face of the beam. This distance is defined in Equations 2.4 and 2.5 is calculated using the strut width, a, without any reduction factors.

$l_{column} = a / \cos\ \theta_{column}$ Eq-2.4

$tan\ \theta_{column} = \{ h_m - (a / \cos\ \theta_{column})\} / l$ Eq-2.5

Now a frame having 3 m vertical height from floor to floor, beam thickness of 0.4 m is considered. In horizontal direction the center to center distance of the bays is 4m and the column size is 0.3x 0.3 m in cross section.

$I\ col$ = $(0.3*0.3^3)/12$ = 675000000 mm^4

H = 3000 mm

h_m = Clear height from floor top to beam bottom.

= 3000 - 400 = 2600 mm

E_c = 3586.6 ksi

E_m = 1200 ksi

t = thickness of the infill wall (5") 125 mm

l = 157.45-12 = 145.45" =3694.43 mm

$D = 196.8'' = 5000$ mm

$\tan\theta = 3000/4000$

$\theta = 36.868°$

$\lambda_l H = 116.42\ mm$

$a = 0.175D(\lambda_l H)^{-0.4}$

$= 18.73'' = 475.74$ mm

$\tan\theta_{column} = \{ h_m - (a / \cos\theta_{column}) \} / l$

$\theta_{column} = 0.506$

$l_{column} = a / \cos\theta_{column}$

$= 21.41'' = 543.81$ mm

APPENDIX D

INPUT FILES FOR EQUIVALENT STATIC AND RESPONSE SPECTRUM ANALYSIS.

Input files for 9 story 5 bay Frames having no infill (Equivalent Static Analysis)

```
STAAD PLANE
START JOB INFORMATION
JOB NAME 4m*5=20mwidth-6th
ENGINEER DATE 31-Mar-04
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 0 1.6 0; 3 0 4.60001 0; 4 0 7.60002 0; 5 0 10.6 0; 6 0 13.6 0;
7 0 16.6 0; 8 0 19.6 0; 9 0 22.6 0; 10 0 25.6001 0; 11 0 28.6001 0;
12 4.00001 0 0; 13 4.00001 1.6 0; 14 4.00001 4.60001 0; 15 4.00001 7.60002 0;
16 4.00001 10.6 0; 17 4.00001 13.6 0; 18 4.00001 16.6 0; 19 4.00001 19.6 0;
20 4.00001 22.6 0; 21 4.00001 25.6001 0; 22 4.00001 28.6001 0; 23 8.00002 0 0;
24 8.00002 1.6 0; 25 8.00002 4.60001 0; 26 8.00002 7.60002 0;
27 8.00002 10.6 0; 28 8.00002 13.6 0; 29 8.00002 16.6 0; 30 8.00002 19.6 0;
31 8.00002 22.6 0; 32 8.00002 25.6001 0; 33 8.00002 28.6001 0; 34 12 0 0;
35 12 1.6 0; 36 12 4.60001 0; 37 12 7.60002 0; 38 12 10.6 0; 39 12 13.6 0;
40 12 16.6 0; 41 12 19.6 0; 42 12 22.6 0; 43 12 25.6001 0; 44 12 28.6001 0;
45 16 0 0; 46 16 1.6 0; 47 16 4.60001 0; 48 16 7.60002 0; 49 16 10.6 0;
50 16 13.6 0; 51 16 16.6 0; 52 16 19.6 0; 53 16 22.6 0; 54 16 25.6001 0;
55 16 28.6001 0; 56 20 0 0; 57 20 1.6 0; 58 20 4.60001 0; 59 20 7.60002 0;
60 20 10.6 0; 61 20 13.6 0; 62 20 16.6 0; 63 20 19.6 0; 64 20 22.6 0;
65 20 25.6001 0; 66 20 28.6001 0;
MEMBER INCIDENCES
1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
11 12 13; 12 13 14; 13 14 15; 14 15 16; 15 16 17; 16 17 18; 17 18 19; 18 19 20;
19 20 21; 20 21 22; 21 23 24; 22 24 25; 23 25 26; 24 26 27; 25 27 28; 26 28 29;
27 29 30; 28 30 31; 29 31 32; 30 32 33; 31 34 35; 32 35 36; 33 36 37; 34 37 38;
35 38 39; 36 39 40; 37 40 41; 38 41 42; 39 42 43; 40 43 44; 41 45 46; 42 46 47;
43 47 48; 44 48 49; 45 49 50; 46 50 51; 47 51 52; 48 52 53; 49 53 54; 50 54 55;
```

```
51 56 57; 52 57 58; 53 58 59; 54 59 60; 55 60 61; 56 61 62; 57 62 63; 58 63 64;
59 64 65; 60 65 66; 61 2 13; 62 13 24; 63 24 35; 64 35 46; 65 46 57; 66 3 14;
67 14 25; 68 25 36; 69 36 47; 70 47 58; 71 4 15; 72 15 26; 73 26 37; 74 37 48;
75 48 59; 76 5 16; 77 16 27; 78 27 38; 79 38 49; 80 49 60; 81 6 17; 82 17 28;
83 28 39; 84 39 50; 85 50 61; 86 7 18; 87 18 29; 88 29 40; 89 40 51; 90 51 62;
91 8 19; 92 19 30; 93 30 41; 94 41 52; 95 52 63; 96 9 20; 97 20 31; 98 31 42;
99 42 53; 100 53 64; 101 10 21; 102 21 32; 103 32 43; 104 43 54; 105 54 65;
106 11 22; 107 22 33; 108 33 44; 109 44 55; 110 55 66;
MEMBER PROPERTY AMERICAN
61 TO 110 PRIS YD 0.4 ZD 0.25
1 TO 60 PRIS YD 0.305 ZD 0.381
UNIT INCHES KIP
CONSTANTS
E 3150 MEMB 1 TO 110
POISSON 0.17 MEMB 1 TO 110
DENSITY 8.68e-005 MEMB 1 TO 110
ALPHA 6.5e-006 MEMB 1 TO 110
UNIT METER KN
SUPPORTS
1 12 23 34 45 56 FIXED
DEFINE UBC LOAD
ZONE 0.15 I 1 RWX 12 RWZ 12 S 1.5 PX 0.9028 PZ 0.9028
MEMBER WEIGHT
61 TO 110 UNI -25
LOAD 1 EQUIVALENT STATIC
UBC LOAD X 1
LOAD 2 DEAD LOAD
MEMBER LOAD
61 TO 110 UNI GY -25
PERFORM ANALYSIS PRINT ALL
FINISH
```

Input files for 9 story Frames having no infill (Response Spectrum Analysis)

STAAD PLANE

START JOB INFORMATION

JOB NAME 4m*5=20mwidth-6th

ENGINEER DATE 31-Mar-04

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 0 1.6 0; 3 0 4.60001 0; 4 0 7.60002 0; 5 0 10.6 0; 6 0 13.6 0;
7 0 16.6 0; 8 0 19.6 0; 9 0 22.6 0; 10 0 25.6001 0; 11 0 28.6001 0;
12 4.00001 0 0; 13 4.00001 1.6 0; 14 4.00001 4.60001 0; 15 4.00001 7.60002 0;
16 4.00001 10.6 0; 17 4.00001 13.6 0; 18 4.00001 16.6 0; 19 4.00001 19.6 0;
20 4.00001 22.6 0; 21 4.00001 25.6001 0; 22 4.00001 28.6001 0; 23 8.00002 0 0;
24 8.00002 1.6 0; 25 8.00002 4.60001 0; 26 8.00002 7.60002 0;
27 8.00002 10.6 0; 28 8.00002 13.6 0; 29 8.00002 16.6 0; 30 8.00002 19.6 0;
31 8.00002 22.6 0; 32 8.00002 25.6001 0; 33 8.00002 28.6001 0; 34 12 0 0;
35 12 1.6 0; 36 12 4.60001 0; 37 12 7.60002 0; 38 12 10.6 0; 39 12 13.6 0;
40 12 16.6 0; 41 12 19.6 0; 42 12 22.6 0; 43 12 25.6001 0; 44 12 28.6001 0;
45 16 0 0; 46 16 1.6 0; 47 16 4.60001 0; 48 16 7.60002 0; 49 16 10.6 0;
50 16 13.6 0; 51 16 16.6 0; 52 16 19.6 0; 53 16 22.6 0; 54 16 25.6001 0;
55 16 28.6001 0; 56 20 0 0; 57 20 1.6 0; 58 20 4.60001 0; 59 20 7.60002 0;
60 20 10.6 0; 61 20 13.6 0; 62 20 16.6 0; 63 20 19.6 0; 64 20 22.6 0;
65 20 25.6001 0; 66 20 28.6001 0;

MEMBER INCIDENCES

1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 6 7; 7 7 8; 8 8 9; 9 9 10; 10 10 11;
11 12 13; 12 13 14; 13 14 15; 14 15 16; 15 16 17; 16 17 18; 17 18 19; 18 19 20;
19 20 21; 20 21 22; 21 23 24; 22 24 25; 23 25 26; 24 26 27; 25 27 28; 26 28 29;
27 29 30; 28 30 31; 29 31 32; 30 32 33; 31 34 35; 32 35 36; 33 36 37; 34 37 38;
35 38 39; 36 39 40; 37 40 41; 38 41 42; 39 42 43; 40 43 44; 41 45 46; 42 46 47;
43 47 48; 44 48 49; 45 49 50; 46 50 51; 47 51 52; 48 52 53; 49 53 54; 50 54 55;
51 56 57; 52 57 58; 53 58 59; 54 59 60; 55 60 61; 56 61 62; 57 62 63; 58 63 64;
59 64 65; 60 65 66; 61 2 13; 62 13 24; 63 24 35; 64 35 46; 65 46 57; 66 3 14;
67 14 25; 68 25 36; 69 36 47; 70 47 58; 71 4 15; 72 15 26; 73 26 37; 74 37 48;
75 48 59; 76 5 16; 77 16 27; 78 27 38; 79 38 49; 80 49 60; 81 6 17; 82 17 28;
83 28 39; 84 39 50; 85 50 61; 86 7 18; 87 18 29; 88 29 40; 89 40 51; 90 51 62;
91 8 19; 92 19 30; 93 30 41; 94 41 52; 95 52 63; 96 9 20; 97 20 31; 98 31 42;

```
99 42 53; 100 53 64; 101 10 21; 102 21 32; 103 32 43; 104 43 54; 105 54 65;
106 11 22; 107 22 33; 108 33 44; 109 44 55; 110 55 66;
MEMBER PROPERTY AMERICAN
61 TO 110 PRIS YD 0.4 ZD 0.25
1 TO 60 PRIS YD 0.305 ZD 0.381
UNIT INCHES KIP
CONSTANTS
E 3150 MEMB 1 TO 110
POISSON 0.17 MEMB 1 TO 110
DENSITY 8.68e-005 MEMB 1 TO 110
ALPHA 6.5e-006 MEMB 1 TO 110
UNIT METER KN
SUPPORTS
1 12 23 34 45 56 FIXED
LOAD 1 DL
MEMBER LOAD
61 TO 110 UNI GY -25
LOAD 4 DYNAMIC
JOINT LOAD
1 FX 0
2 FX 50
3 FX 50
4 FX 50
5 FX 50
6 FX 50
7 FX 50
8 FX 50
9 FX 50
10 FX 50
11 FX 50
12 FX 0
13 FX 100
14 FX 100
15 FX 100
16 FX 100
17 FX 100
18 FX 100
19 FX 100
```

20 FX 100
21 FX 100
22 FX 100
23 FX 0
24 FX 100
25 FX 100
26 FX 100
27 FX 100
28 FX 100
29 FX 100
30 FX 100
31 FX 100
32 FX 100
33 FX 100
34 FX 0
35 FX 100
36 FX 100
37 FX 100
38 FX 100
39 FX 100
40 FX 100
41 FX 100
42 FX 100
43 FX 100
44 FX 100
45 FX 0
46 FX 100
47 FX 100
48 FX 100
49 FX 100
50 FX 100
51 FX 100
52 FX 100
53 FX 100
54 FX 100
55 FX 100
56 FX 0
57 FX 50

58 FX 50

59 FX 50

60 FX 50

61 FX 50

62 FX 50

63 FX 50

64 FX 50

65 FX 50

66 FX 50

1 FY 0

2 FY 50

3 FY 50

4 FY 50

5 FY 50

6 FY 50

7 FY 50

8 FY 50

9 FY 50

10 FY 50

11 FY 50

12 FY 0

13 FY 100

14 FY 100

15 FY 100

16 FY 100

17 FY 100

18 FY 100

19 FY 100

20 FY 100

21 FY 100

22 FY 100

23 FY 0

24 FY 100

25 FY 100

26 FY 100

27 FY 100

28 FY 100

29 FY 100

30 FY 100

31 FY 100

32 FY 100

33 FY 100

34 FY 0

35 FY 100

36 FY 100

37 FY 100

38 FY 100

39 FY 100

40 FY 100

41 FY 100

42 FY 100

43 FY 100

44 FY 100

45 FY 0

46 FY 100

47 FY 100

48 FY 100

49 FY 100

50 FY 100

51 FY 100

52 FY 100

53 FY 100

54 FY 100

55 FY 100

56 FY 0

57 FY 50

58 FY 50

59 FY 50

60 FY 50

61 FY 50

62 FY 50

63 FY 50

64 FY 50

65 FY 50

66 FY 50

1 FZ 0

2 FZ 50

3 FZ 50

4 FZ 50

5 FZ 50

6 FZ 50

7 FZ 50

8 FZ 50

9 FZ 50

10 FZ 50

11 FZ 50

12 FZ 0

13 FZ 100

14 FZ 100

15 FZ 100

16 FZ 100

17 FZ 100

18 FZ 100

19 FZ 100

20 FZ 100

21 FZ 100

22 FZ 100

23 FZ 0

24 FZ 100

25 FZ 100

26 FZ 100

27 FZ 100

28 FZ 100

29 FZ 100

30 FZ 100

31 FZ 100

32 FZ 100

33 FZ 100

34 FZ 0

35 FZ 100

36 FZ 100

37 FZ 100

38 FZ 100

39 FZ 100

40 FZ 100

41 FZ 100

42 FZ 100

43 FZ 100

44 FZ 100

45 FZ 0

46 FZ 100

47 FZ 100

48 FZ 100

49 FZ 100

50 FZ 100

51 FZ 100

52 FZ 100

53 FZ 100

54 FZ 100

55 FZ 100

56 FZ 0

57 FZ 50

58 FZ 50

59 FZ 50

60 FZ 50

61 FZ 50

62 FZ 50

63 FZ 50

64 FZ 50

65 FZ 50

66 FZ 50

SPECTRUM SRSS X 0.5 ACC SCALE 0.981 DAMP 0.05

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1.8 0.8; 2 0.75;

PERFORM ANALYSIS PRINT ALL

FINISH