THESIS

EFFECTS OF OPENINGS IN SHEAR WALL ON SEISMIC RESPONSE OF FRAME-SHEAR WALL STRUCTURES

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THESIS

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A Thesis Submitted in Partial Fulfillment of
the Requirements for the Degree of
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The study investigated the accuracy of a simplified method in evaluating the stiffness of shear wall with opening. By comparing the stiffnesses of single story rectangular shear walls of size 5 m × 3 m with different opening sizes and locations, the difference in stiffnesses obtained from the simplified method and finite element method was found less than 20% for the shear wall with opening area of 10 % at the base, regardless of the opening aspect ratio (h_o/b_o). Additionally, finite element models of 6- and 12-story 7 × 3 bays apartment buildings with typical floor plan of 35 m × 15 m and floor height of 3 m with different opening sizes and locations in shear walls were developed. Equivalent static analyses as per IS 1893 (part 1): 2002, were performed for all developed models. Top displacement and base shear in shear walls were compared for all developed structures to evaluate the effects of openings sizes and locations in shear walls, on stiffness of the system. In addition to that, coupling ratio (CR), maximum absolute principal stress, maximum principal stress (S_{max}) in shear walls and vertical stress (S_y) at opening level in shear walls were compared to evaluate the effects of openings on seismic responses of the structures as well as on behavior of the shear walls. The results reveal that the stiffness as well as the seismic responses of the structures is more affected by the size of the openings than their locations in shear wall, for opening area $\leq$ 20%. However, it is significantly affected by openings configurations, for opening area > 20%. In addition to that, it has significant effects on behavior of shear wall.

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8 / May / 08

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Meena Shrestha
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LIST OF ABBREVIATIONS

CR = Coupling Ratio of shear wall
DL = Dead Load
EL = Earthquake Load
Eq. = Equation
FEM = Finite Element Method
IL = Imposed Load
IS = Indian Standard
kN/m^2 = Kilo Newton per square meter
kN/m^3 = Kilo Newton per cubic meter
m = Meter
mm = Millimeter
OMRF = Ordinary Moment Resisting Frame
SMRF = Special Moment Resisting Frame
3D = Three Dimensional
A = Area of web about the axis of bending
A_h = Design horizontal seismic coefficient
b_o = Width of opening in shear wall
d = Base dimension of the building at plinth level, in meter, along the considered direction of lateral force
E = Modulus of elasticity of material
e_h = Horizontal distance from center of boundary element in shear wall to the edge of door
e_v = Vertical distance from floor level to the bottom of window in shear wall
G = Shear modulus
H = Height of building in meter
h = Height of shear wall element
h_i = Height of floor i measured from base
h_o = Height of opening in shear wall

LIST OF ABBREVIATIONS (Continued)

h' = Height of applied shear above the top of shear wall element
I = Importance factor
I_x = Moment of inertia of shear wall about x axis
I_y = Moment of inertia of shear wall about y axis
L = Length of shear wall
n = Number of stories in the building, is the number of levels at which masses are located
Q_i = Design lateral force at floor i
R = Rigidity of shear wall element
S_a/g = Average response acceleration factor
S_{max} = Maximum principal stress in shear wall
S_y = Vertical stress in shear wall
t = Thickness of shear wall
T_a = Approximate fundamental natural time period of vibration in second
V_B = Total base shear
W = Seismic weight of the building
W_i = Seismic weight of floor i
Z = Zone factor
γ = Unit weight in kN/m^3
Δ_{pier} = Displacement of pier formed by opening
Δ_{piers} = Combined displacement of piers
$\Delta_{solid\ strip}$ = Displacement of solid strip
$\Delta_{solid\ wall}$ = Displacement of solid wall
Δ_{wall} = Displacement of wall with opening
ν = Poisson’s ratio

EFFECTS OF OPENINGS IN SHEAR WALL ON SEISMIC RESPONSE OF FRAME-SHEAR WALL STRUCTURES

INTRODUCTION

Nepal is located in one of the active continental collision zones of the world, in the lap of Himalaya where the probability of earthquake occurrence is very high. Many destructive earthquakes have been reported in historical records that have devastated different parts of Nepal in different times. The 15th January 1934 Bihar-Nepal Great Earthquake of magnitude greater than Ms 8 still hunts the mind of many elderly people. In the decades of eighties, two moderate earthquakes with magnitude greater than Ms 6 occurred, one in western Nepal (1980, Bajhang Earthquake) and other in eastern Nepal (1988, Udayapur Earthquake). In addition to that, many small earthquakes occurred frequently. Thus, Nepal is highly susceptible to earthquakes.

Earthquake is one of the nature’s greatest hazards to properties and human lives. It poses a unique engineering design problem. An intense earthquake constitutes severe loading to which most civil engineering structures may possibly be subjected. The number of earthquakes reported worldwide, are usually followed by enormous death and injury. Not only life but also economy that are threatened from this disaster. The approach of engineering design is to design the structures in such a way that it can survive under the most severe earthquakes, during their service lives to minimize the loss of life and the possibility of damage.

Buildings are designed primarily to serve the needs of an intended occupancy. One of the dominant design requirements is therefore the provision of an appropriate internal layout of buildings. Once the functional layout is established, one must develop a structural system that will satisfy the established design criteria as efficiently and economically as possible, while fitting into the architectural layout. The vital structural criteria are an adequate reserve of strength against failure, adequate lateral stiffness and an efficient performance during the service life of the buildings.

An introduction of shear wall represents a structurally efficient solution to stiffen a building structural system because the main function of a shear wall is to increase the rigidity for lateral load resistance. In modern tall buildings, shear walls are commonly used as a vertical structural element for resisting the lateral loads that may be induced by the effect of wind and earthquakes. Shear walls of varying cross sections i.e. rectangular shapes to more irregular cores such as channel, T, L, barbell shape, box etc. can be used. Provision of walls helps to divide an enclose space, whereas of cores to contain and convey services such as elevator. Wall openings are inevitably required for windows in external walls and for doors or corridors in inner walls or in lift cores. The size and location of openings may vary from architectural and functional point of view.

Statement of the problem

During the last few decades, population has increased at an alarming rate in Nepal. This large population not only needs jobs but also needs housing and other infrastructure facilities. Peoples from rural areas are migrating to main cities for searching jobs. The rapid growth of urban population and consequent pressure on limited space, have considerably influenced the city residential development. This massive population increment will put great pressure on agricultural land and middle level cities. Therefore, constructing tall buildings for more space would be the better solution for this urbanization.

At present, Kathmandu and other cities of Nepal are expanding horizontally and haphazardly. In addition to that, there is a need of vertical expansion due to the scarcity of land. This is specially needed for saving agricultural land otherwise there will be only buildings, slums and roads all round and lesser land for agriculture purposes. The high cost of land, the desire to avoid a continuous urban sprawl and the need to preserve important agricultural production have all contributed to drive residential buildings upward.

Nowadays, most of the slender residential buildings are haphazardly constructed in Kathmandu without proper design and construction. On the other hand, the structural designs and reinforcement detailings are not checked and evaluated by concerned authorities. This may lead to huge disaster during earthquake, resulting in loss of lives and properties. The devastation due to the earthquake in 1934 in Nepal is remarkable in the record up till date. The movement of Indian Plate towards the north Himalayan is the cause of seismic waves in this region.

The adverse effect for tall building is the higher lateral loads due to wind and expected earthquake. Thus, shear walls are introduced in modern tall buildings to make the structural system more efficient in resisting the horizontal and gravity loads as well, thereby causing less damage to the structure during earthquake.

Shear walls in apartment buildings will be perforated by rows of openings that are required for windows in external walls or for doors ways or corridors in internal walls. However, the size and location of openings in the shear wall may have adverse effect on seismic responses of frame-shear wall structures. Relative stiffness of shear walls is important since lateral forces are distributed to individual shear wall according to their relative stiffness. Simplified methods for stiffness of shear walls with openings are recommended in several design guidelines. As a designer, it is necessary to know the effects of openings sizes and configurations in shear wall on stiffness as well as on seismic responses and behavior of structural system so that a suitable configuration of openings in shear walls can be made.

OBJECTIVES

Shear walls are generally located at the sides of buildings or arranged in the form of core that houses stairs and lifts. In an apartment building, shear walls may have one or more openings. In most of the apartment building, size and location of openings in shear wall are made without considering its effect on structural behavior of the building. Thus, the main objectives of this study are:

1. To investigate an accuracy of simplified method in evaluating the stiffness/rigidity of shear wall with opening.
2. To study the effects of sizes and locations of openings in shear wall on seismic responses of buildings.
3. To evaluate the behavior of shear wall with openings under seismic loads.

Scope of the study

The scope of this study was limited to the following:

1. The study is carried out on 6- and 12-story 7 × 3 bays frame-shear wall buildings with 5-m span and floor height of 3 meters, using linear elastic analysis.
2. Typical floor plan with same dimensions 35 m × 15 m is used for all model buildings
3. Shear wall of 5 m × 3 m × 250 mm is taken for evaluating the stiffness/rigidity of shear wall with opening using simplified and finite element methods.
4. For the research purpose, opening size is changed in shear wall and location is changed for same opening size.
5. Size and concrete grade used for all structural elements are same for all structural models.

LITERATURE REVIEW

1. General

1.1. Influence of building configuration on seismic response

Generally, the building configuration which is conceived by architects and then accepted by developer or owner may provide a narrow range of options for lateral-load resistant systems that can be utilized by structural engineers. By observing the following fundamental principles relevant to seismic responses, more suitable structural systems may be adopted (Paulay and Priestley, 1992):

1. To perform well in an earthquake, a building should possess simple and regular configurations. Buildings with articulated plans such as T and L shapes should be avoided.
2. Symmetry in plans should be provided, wherever possible. Lack of symmetry in plan may lead to significant torsional response, the reliable prediction of which is often difficult.
3. An integrated foundation system should tie together all vertical structural elements in both principal directions. Foundation resting on different soil condition should preferably be avoided.
4. Lateral force resisting systems with significantly different stiffness such as shear walls and frames within one building should be arranged in such a way that at every level of the building, symmetry in lateral stiffness is not grossly violated. Thus, undesirable torsional effects will be minimized.
5. Regularity in elevation should prevail in both the geometry and the variation of story stiffness.

1.2. Essentials of structural systems for seismic resistance

The primary purpose of all structural members used in buildings is to support gravity loads. However, buildings may also be subjected to lateral forces due to wind and earthquakes. The effects of lateral forces in buildings will be more significant as the building height increases. All structural systems will not behave equally under seismic excitation. Aspects of structural configuration, symmetry, mass distribution and vertical regularity must be considered. In addition to that, the importance of strength, stiffness and ductility in relation to acceptable response must be evaluated in structural system (Paulay and Priestley, 1992).

The first task of the structural designer is to select the appropriate structural system for the satisfactory seismic performance of the building within the constraints dictated by architectural requirements. It is better where possible to discuss architect and structural engineer for alternative structural configuration at the earliest stage of concept development. Thus, undesirable geometry is not locked into the system before structural design is started.

Irregularities in buildings contribute to complexity of structural behavior. When not recognized, they may result in unexpected damage and even collapse of the structures. There are many possible sources of structural irregularities. Drastic changes in geometry, interruptions in load path, discontinuities in both strength and stiffness, disruption in critical region by openings and unusual proportion of members are few of the possibilities. The recognition of many of these irregularities and of conceptions for remedial measures for the mitigation of their undesired effects relies on sound understanding of structural behavior.

1.3. Stiffness and drift limitations

The provision of adequate lateral stiffness is a major consideration in the design of a tall building in seismic zone. Firstly, deflection must be maintained at a sufficiently low level for the proper functioning of non-structural components such as elevators and doors. Secondly, it must be limited to prevent excessive cracking and consequent loss of stiffness, and to avoid any redistribution of load to non structural elements such as partitions, infill, cladding or glazing. Thirdly, the structure must be sufficiently stiff to prevent dynamic motions becoming large enough to cause discomfort to occupants, prevent delicate work being undertaken or affect the sensitive equipments.

One simple parameter that can be used to estimate the lateral stiffness of a building is the drift index. It can be defined as the ratio of the maximum deflection at the top of the building to the total height. In addition to that, the inter-story drift index gives a measure of possible localized excessive deformation. The control of lateral deflections is of particular importance for modern tall buildings in which the traditional reserves of stiffness due to heavy internal partitions and outer cladding have largely disappeared.

Smith and Coull (1991) state that the design drift index limits that have been used on different countries range from 0.001 to 0.005. Generally, lower values would be used for hotels or apartment buildings than for office buildings. Sufficient stiffness must be provided to ensure that the top deflection does not exceed this value under extreme load events. As the building height increases, drift index coefficient should be decreased to the lower end of the range to keep the story deflection to a suitably low level. If excessive, the drift of a structure can be reduced by changing geometric configuration to alter the mode of lateral load resistance. Increasing the bending stiffness of the horizontal members, adding additional stiffness by inclusion of stiffer wall or core members, achieving stiffer connections and even by sloping the exterior columns are some of the solutions.

As per Indian standard, Criteria for earthquake resistant design of structures, IS 1893 (Part 1) : 2002, the story drift in any story due to service load shall not exceed 0.004 times the story height.

1.4. Stiffness/rigidity of shear wall

The stiffness/rigidity of a shear wall in a given direction (R_x or R_y) is defined as a force required per unit displacement in the given direction. Varyani (2002) states that the deflection (Δ_x) as shown in Figure 1 of a wall element regarded as a deep cantilever beam fixed at base due to shear V_x applied in x direction at a height h' from top is composed of deflection due to bending and shear.

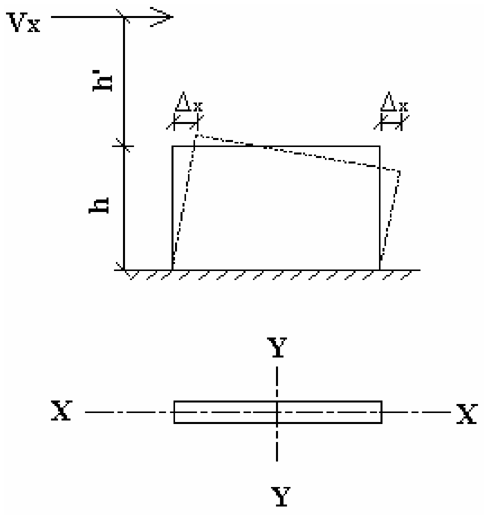


Figure 1 Deflection of cantilever shear wall subjected to lateral load

Source: Varyani (2002)

The deflection (Δ_x) is given by:

$$\Delta_x = \frac{V_x h^3}{3EI_y} + \frac{V_x h' h^2}{2EI_y} + \frac{1.2V_x h}{A_y G} \tag{1}$$

Where,

E = modulus of elasticity of material of shear wall

$G = \frac{E}{2(1+\nu)}$ = shear modulus of material of shear wall

I = moment of inertia of shear wall about the axis of bending

A = area of web about the axis of bending

Δ = deflection due to applied shear in a given direction

h = height of shear wall element

h' = height of applied shear above the top of wall element

Assuming Poisson's ratio (ν) = 0.17 for concrete and V acts at the top of shear wall making $h' = 0$, the rigidity of the wall element in x direction is given by:

$$R_x = \frac{V_x}{\Delta_x} = \frac{1}{\dfrac{h^3}{3EI_y} + 2.81\dfrac{h}{A_y E}} \tag{2}$$

Similarly, the rigidity of the wall element in y direction is given by:

$$R_y = \frac{V_y}{\Delta_y} = \frac{1}{\dfrac{h^3}{3EI_x} + 2.81\dfrac{h}{A_x E}} \tag{3}$$

1.4.1. Solid rectangular shear walls

For solid rectangular shear wall (Figure 2) with t<<L

$$I_y = \frac{1}{12}L^3 t \tag{4}$$

$$I_x = \frac{1}{12}Lt^3 \approx 0 \tag{5}$$

$$A_x = A_y = Lt \tag{6}$$

Substituting the above values in Eq. (2), the rigidity of shear wall in the direction of its length R_x (say R) is given by:

$$R = \frac{Et}{4\dfrac{h^3}{L^3} + 2.81\dfrac{h}{L}} \tag{7}$$

From definition, $R = \frac{1}{\Delta}$

$$\therefore \Delta_{\text{solid wall}} = \frac{4\frac{h^3}{L^3} + 2.81\frac{h}{L}}{Et} \tag{8}$$

1.4.2. Shear wall with openings

Piers in a wall formed by openings may be regarded as fixed at both ends as shown in Figure 2. The bending deflection term $\frac{h^3}{3EI}$ will be reduced to $\frac{h^3}{12EI}$ as presented in Eqs. (2) and (3). Thus, the rigidity of a pier in the direction of its length is given by: (Varayani, 2002)

$$R = \frac{Et}{\frac{h_o^{\ 3}}{l^3} + 2.81\frac{h_o}{l}} \tag{9}$$

Displacement of pier is thus given by:

$$\Delta_{\text{pier}} = \frac{\frac{h_o^{\ 3}}{l^3} + 2.81\frac{h_o}{l}}{Et} \tag{10}$$

Where, h_o is height of opening and l is length of pier.

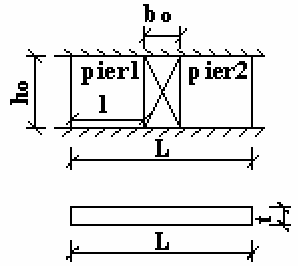


Figure 2 Wall element fixed at both ends

The rigidity of wall with openings may be calculated neglecting the effects of the axial shortening of piers by judicious use of the principles of series and parallel (Varyani, 2002). However, the end condition of the solid strip formed by the opening is not mentioned by Varyani.

Neuenhofer (2006) studied on accuracy of the simplified method for calculating lateral stiffness of shear wall with openings. Finite element algorithm was developed in computing package MATLAB to calculate lateral stiffness of shear wall with opening. Displacement of wall with opening using the simplified method was calculated using the relation given in Eq. (11). Then, the stiffness was obtained as the reciprocal of the displacement of wall.

$$\Delta_{\text{wall}} = \Delta_{\text{solid wall}} - \Delta_{\text{solid strip}} + \Delta_{\text{piers}} \tag{11}$$

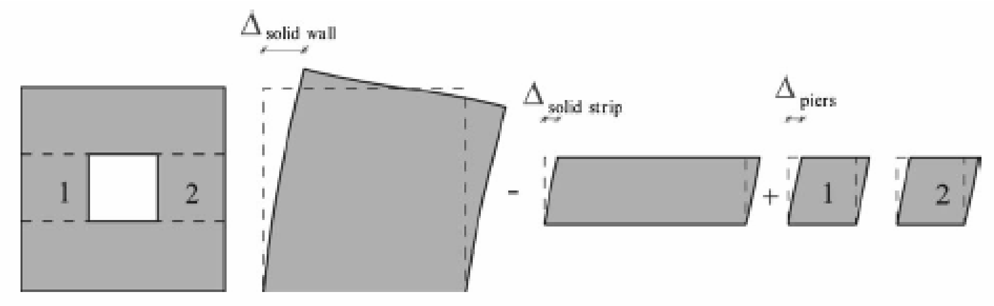


Figure 3 Simplified method for calculating displacement of shear wall with opening
Source: Neuenhofer (2006)

In Figure 3, fixed-fixed action was assumed for piers and solid strip formed by the opening. Stiffness of shear wall with opening obtained using MATLAB was compared with that from the simplified method. It was concluded that the simplified method consistently underestimates the impact of opening on the reduction of lateral stiffness of shear wall. It was also found that manual procedure yields remarkably poor results for walls with small aspect ratio (h/L).

2. Structural systems

From structural engineering view point, the determination of structural systems of a high-rise building involves only the selection of the major structural elements to resist most efficiently the various combinations of gravity and lateral loads. Some of the structural systems used in modern tall buildings are as follows:

2.1. Shear wall structure

A shear wall structure is considered to be one whose resistance to horizontal loading is provided entirely by shear walls. They may act as a vertical cantilever in the form of separate planner walls and as non-planner assembles of connected walls around elevator, stair and service shaft. Shear walls have been the most common structural elements used for stabilizing the building structures against lateral forces. Their very high in-plane stiffness and strength makes them ideally suited for bracing tall buildings.

The usefulness of shear walls in framing of buildings has long been recognized. Walls situated in advantageous positions in a building can form an efficient lateral-force-resisting system, simultaneously fulfilling other functional requirements. When a permanent and similar subdivision of floor areas in all stories is required as in the case of hotels or apartment buildings, numerous shear walls can be utilized not only for lateral force resistance but also to carry gravity loads. In such case, the floor by floor repetitive planning allows the walls to be vertically continuous which may serve simultaneously as excellent acoustic and fire insulators between the apartments. Shear walls may be planar but are often of L-, T-, I-, or U- shaped section (Figure 4) to better suit the planning and to increase their flexural stiffness.

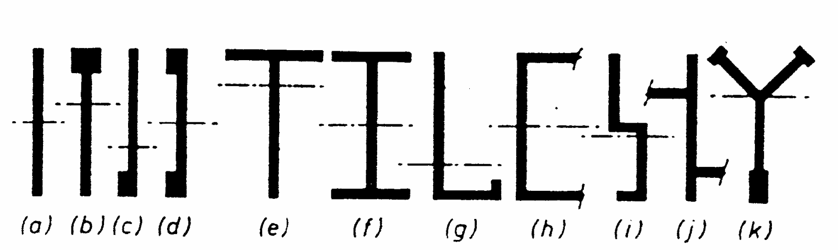


Figure 4 Common sections of shear walls
Source: Paulay and Priestley (1992)

The positions of shear walls within a building are usually dictated by functional requirements. These may or may not suit structural planning. The purpose of a building and consequent allocation of floor space may dictate required arrangements of walls that can often be readily utilized for lateral force resistance. Building sites, architectural interests or client's desire may lead the positions of walls that are undesirable from a structural point of view. However, structural designers are often in the position to advise as to the most desirable locations for shear walls in order to optimize seismic resistance. The major structural considerations for individual shear walls will be aspects of symmetry in stiffness, torsional stability and available overturning capacity of the foundations (Paulay and Priestley, 1992).

2.2. Coupled shear wall structure

A coupled shear wall structure consists of two or more shear walls in the same plane or almost same plane connected by beams or stiff slabs. Normally, openings are required in vertical rows throughout the height of the shear wall. The connections between the wall segments are provided by either connecting beams or floor slabs or a combination of both. The effect of shear connecting members is to cause the set of walls to behave in their plane partly as a composite cantilever, bending about the common centroidal axis of the walls. This results in a horizontal stiffness very much greater than the case where the walls acted as a set of separate uncoupled cantilevers. The presence of the moment resisting connections greatly enhances the efficiency of the wall system. Couple shear walls are more appropriate for concrete construction.

According to Smith and Coull (1991), if a pair of in-plane shear walls is connected by pin ended links that transmit only axial forces between them. Any applied moment will be resisted by sum of individual moments in the two walls, the magnitudes of which will be proportional to the walls flexural rigidities. The bending stresses will then be distributed linearly across each wall with maximum tensile and compressive stresses occurring at the opposite extreme edges (Figure 5d).

On the other hand, if the walls are connected by rigid beams to form a dowelled vertical cantilever, the applied moment will be resisted by the two walls acting as a single composite unit bending about the centroidal axis of the two walls. The bending stresses will then be distributed linearly across the composite unit with maximum tensile and compressive stresses occurring at the opposite extreme edges (Figure 5c). The stiffer the connecting beams, the closer the structural behavior will approach to that of a fully composite cantilever wall.

When the walls deflect under the action of the lateral loads, the connecting beam ends are forced to rotate and displace vertically, so that the beams bend in double curvature and thus resist the free bending of the walls (Figure 6). The bending action induces shears in the connecting beams which exert bending moments of opposite sense to the applied external moments on each wall. The shears also induce axial forces in the two walls, tensile in the windward wall and compressive in the leeward wall.

The overturning moment M due to lateral force, at any level is then resisted by the sum of the bending moments M_1 and M_2 in the two walls at that level and the moment due to coupling action of the axial forces NL.

$$M = M_1 + M_2 + NL \tag{12}$$

Where,

N = the axial force in each wall at that level

L = the distance between their centroidal axes.

The last term NL represents the reverse moment caused by the bending of the connecting beams which opposes the free bending of the individual walls and reaches a maximum when the connecting beams are infinitely rigid.

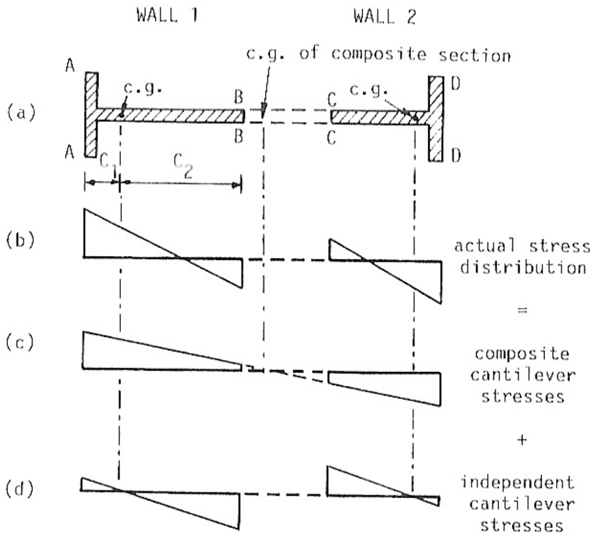


Figure 5 Superposition of stress distribution due to composite and individual cantilever actions to give true stress distribution in walls

Source: Smith and Coull (1991)

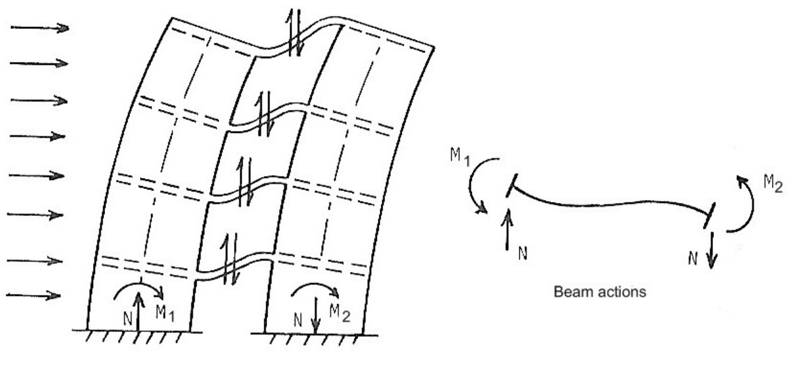


Figure 6 Behavior of laterally loaded coupled shear walls

Source: Smith and Coull (1991)

The action of connecting beams is to reduce the magnitude of the moments in the two walls by causing a proportion of the applied moment to be carried by axial forces. The proportion of the system overturning moment resisted by the coupling action is defined as coupling ratio (CR) which is given by: (El-Tawil, *et. al*, 2002)

$$\mathrm{CR} = \frac{\mathrm{NL}}{\mathrm{M_1 + M_2 + NL}} \tag{13}$$

El-Tawil, *et. al* (2002) said that too small a coupling ratio will yield a system to behave as uncoupled wall while too large a coupling ratio will add excessive stiffness to the system. Thus, it causes couple wall to perform as a single pierced wall. The optimum amount of coupling lies in between these two extremes.

A relatively small axial stress can give rise to a larger moment of resistance because the relatively large lever arm L involves. The maximum tensile stress in the concrete may then be greatly reduced. This makes it easier to suppress the earthquake load tensile stresses by gravity compressive stresses.

2.3. Frame-shear wall structures

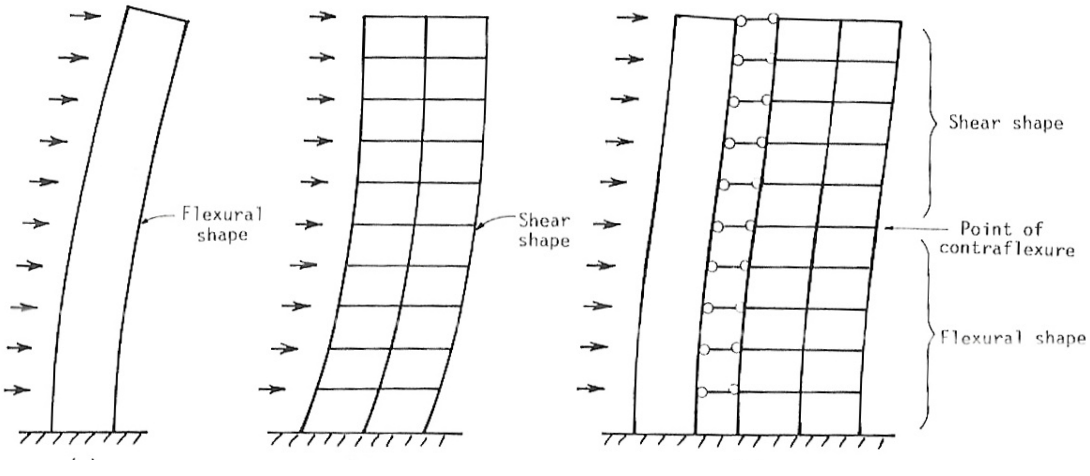
A structure whose resistance to horizontal loading is provided by a combination of shear walls and rigid frames may be categorized as a frame-shear wall structure. The shear walls are often part of the elevator and service cores while the frames are arranged in plan in conjunction with the walls to support the floor system.

When a frame-shear wall structure is loaded laterally, the different free deflected forms of the walls and the frames cause them to interact horizontally through the floor slabs. Consequently, the individual distribution of lateral loading on the shear wall and the frame may be very different from the distribution of the external loading. The horizontal interaction can be effective in contributing to lateral stiffness to the extent that frame-shear wall structure of up to 50 stories or more are economical (Smith and Coull, 1991).

The potential advantage of a frame-shear wall structure depends on the amount of horizontal interaction which is governed by the relative stiffness of the shear walls and frames and the height of structure. It is used to be common practice in the design of high-rise structures to assume that the shear walls resist all lateral loading and to design the frame for gravity loading only. The principal advantages of accounting for the horizontal interaction in designing a frame-shear wall structure are as follows: (Smith and Coull, 1991)

1. The estimated drift may be significantly less than the case where the walls alone are considered to resist the horizontal loading.
2. The estimated bending moments in the shear walls will be less.
3. The columns of the frames may be designed as fully braced.
4. The estimated shears in the frames in many cases may be approximately uniform through the height.

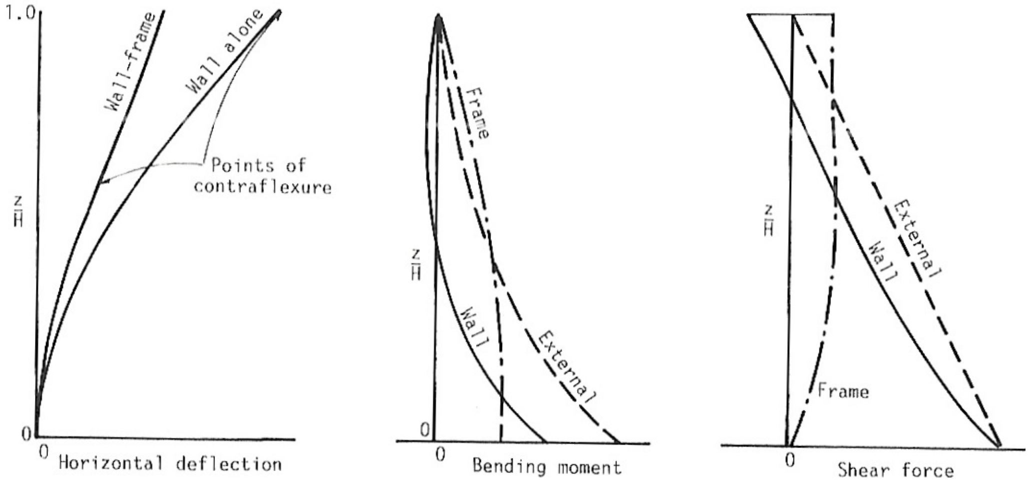
A further understanding of interaction between the shear wall and the frame in a frame-shear wall structure is given by the deflected shapes of a shear wall and a rigid frame subjected separately to horizontal loading (Figures 7a and b). Under the action of horizontal loading, the wall deflects in flexure mode with concavity downwind and a maximum slope at top while the frame deflects in shear mode with concavity upwind and a maximum slope at the base. When the shear wall and frame are connected together by pin ended links and subjected to horizontal loading, the deflected shape of the composite structure has flexural profile in the lower part and a shear profile in the upper part (Figure 7c). Axial forces in the connecting links cause the shear wall to restrain the frame near the base and frames to restrain the shear wall at the top. The deflection curve (Figure 8a) and wall moment curve (Figure 8b) indicate the reversal in curvature with a point of inflexion above which the wall



(a) Free shear wall (b) Free frame (c) Frame-shear wall structure

Figure 7 Interaction between shear wall and frame subjected to uniformly distributed horizontal loading

Source: Smith and Coull (1991)



(a) Deflection diagram (b) Moment diagram (c) Shear diagram

Figure 8 Typical responses of laterally loaded frame-shear wall structure

Source: Smith and Coull (1991)

moment is opposite in sense to that in a free cantilever. Figure 8c shows the shear force is approximately uniform over the height of the frame except near the base. It reduces to a negligible amount at the base. At the top where the external shear force is zero, the frame is subjected to a significant positive shear which is balanced by an equal negative shear at the top of the wall. This is due to the interaction forces acting between the frame and the shear wall.

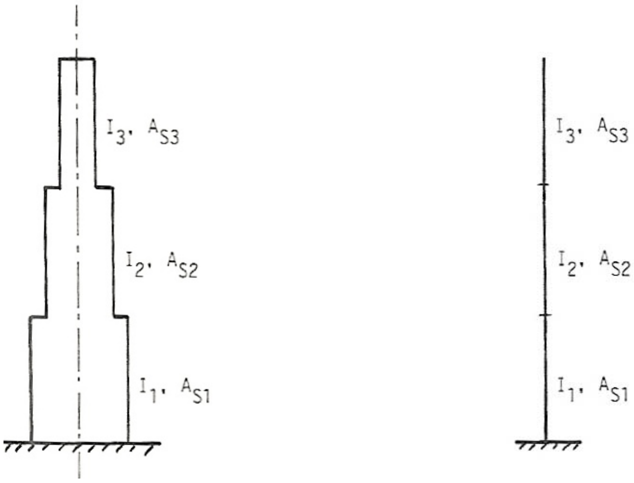
3. Methods of modeling

The objective of analysis is to obtain the responses of a structure to the excitation. The excitations can be direct loads, seismic excitation, thermal changes or deformations due to creep and shrinkage. The response can be the deformations, strains, stress etc. The analytical results are relevant to the model and not to the real structure.

The apparent numerical accuracy of the results of the analysis does not automatically guarantee the accuracy of the responses of the real structure. The results obtained from the software are for the model and not for the real structure. It is therefore imperative that the model should represent the real structure with an appropriate level of detail and likeliness to capture the desired responses as realistically as possible.

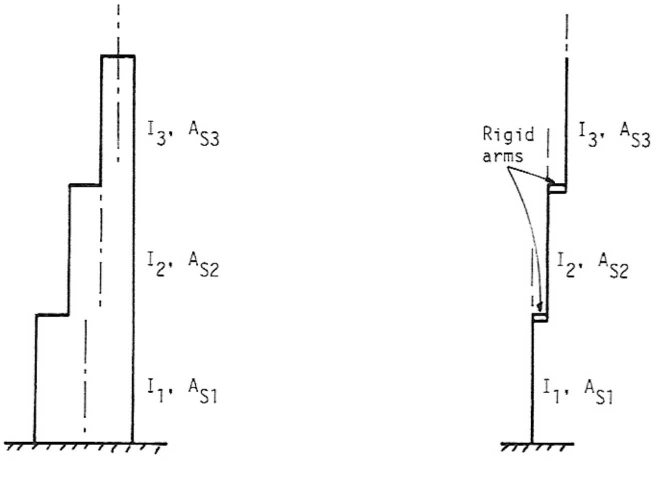
A building's responses to loading is governed by the components that are stressed as the building deflects. Ideally, for ease and accuracy of the structural analysis, only the main structural elements are considered in modeling a building. At the same time, a reasonably accurate assessment of behavior of a proposed structure is necessary to form a properly representative model for analysis.

For approximate analysis, an axially concentric shear wall (Figure 9a) consisting of relatively uniform regions can be modeled by a column located at centroidal axis of the wall (Figure 9b). If the wall is axially eccentric as in Figure 10a, the analogous columns on the respective wall axes should be connected by horizontal rigid arms as shown in Figure 10b (Smith and Coull, 1991).



(a) Axially concentric shear wall (b) Equivalent column model

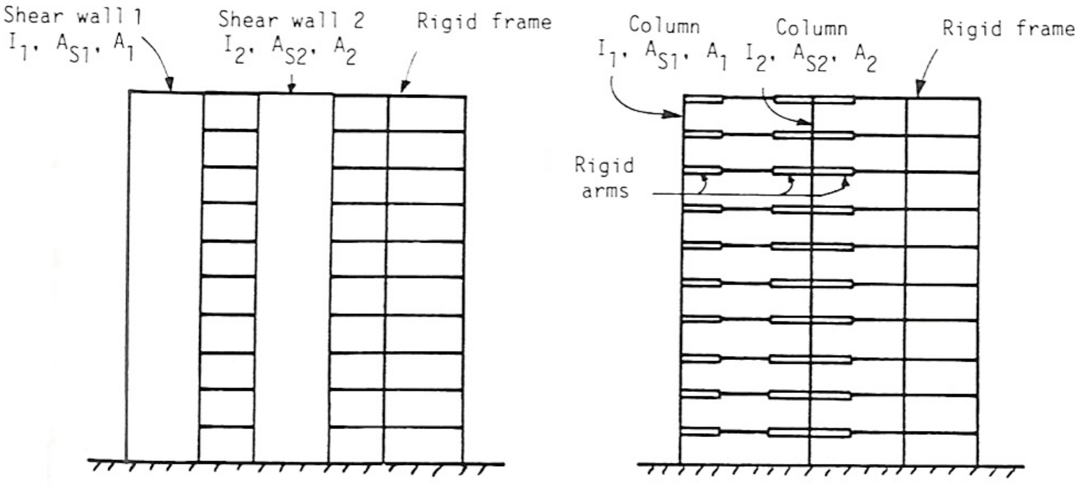
Figure 9 Model for axially concentric shear wall
Source: Smith and Coull (1991)



(a) Axially eccentric shear wall; (b) Equivalent column model

Figure 10 Model for axially eccentric shear wall
Source: Smith and Coull (1991)

If a shear wall has beams connecting to it in-plane causing it to interact vertically as well as horizontally with another shear wall or with other parts of the structure (Figure 11a), the shear wall can be represented by an analogous wide column. This is a column placed at centroidal axis of the wall and assigned to have the wall's inertia and axial area, having rigid arms that join the column to the connecting beams at each framing level (Figure 11b). In this way the rotations and vertical displacements at the edges of the wall are transferred to the connecting beams.

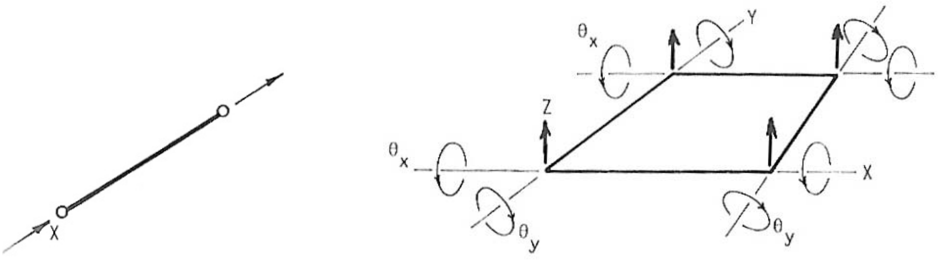


(a) Shear walls and frames joined by beams (b) Equivalent wide-column model

Figure 11 Model for shear walls and frames joined by beams
Source: Smith and Coull (1991)

To obtain analysis results such as structure deflections and member forces as accurately as possible, shear walls with openings can not be well represented by simple equivalent column model and wide column model. The structural model for more accurate analysis results close to exact analysis results as possible should represent in a more detailed way all the major active components of the structure mainly the columns, beams, walls and slabs.

(a) Beam element (b) Quadrilateral membrane element



(c) Truss element (d) Quadrilateral plate bending element

Figure 12 Finite element models for different structural elements
Source: Smith and Coull (1991)

Beam elements (Figure 12a) can be used to represent the beams and columns while the membrane elements (Figure 12b) can be used to represent the shear walls. Since the membrane elements do not have a degree of freedom to represent in plane rotation of their corners, a beam element connected to a node of the membrane element is effectively connected only by a hinge. The remedy for this deficiency can be maintained by adding a fictitious flexural rigid auxiliary beam to the edge of the wall element. Also the truss elements (Figure 12c), quadrilateral plate elements, (Figure 12d) and combined membrane-plate elements can be used to better represent, respectively the truss members, slabs in bending and shear walls subjected to out of plane bending.

4. Past studies on frame-shear-wall structures and shear walls with openings

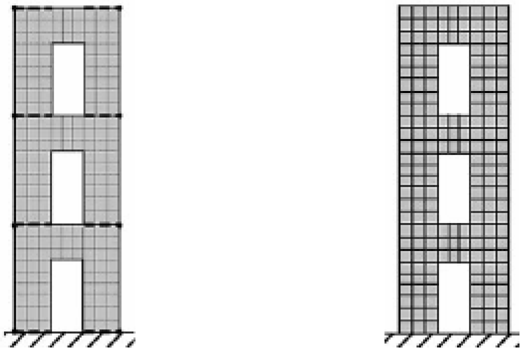
Sharma (1998) studied on various structural systems i.e. moment resisting frames, frame-shear wall and frame-coupled shear wall, concentrically braced frame under seismic excitation, eccentrically braced frame and hybrid structures, using structural analysis software Sap90. From the results it was concluded that design based on drift control criteria generally results into same levels of stiffness whatever may be the structural system and it is advantageous to use correct combination of frame and shear wall to get uniform inter story drift.

Atimtay and Kanit (2006) said that earthquake itself is a great professor to learn the seismic design of buildings and it tells the engineer what to do and what not. From numerous and careful examinations of the wide spread earthquake damage due to Marmara earthquake on August 17, 1999 of 7.4 Richter scale in north western region of Turkey, it was found that no buildings had collapsed which contain shear walls as part of the structural system. It was concluded that successfully designed and constructed building should contain adequate number of shear wall with well detailed reinforcement.

Singh (2000) studied on effect of curtailment of shear walls at different levels on seismic response of 12-story frame-shear wall building. It was suggested that it is better to keep uniform column dimension throughout the building than to proportion the column sizes at intermediate levels. In the first case, the walls can be reduced to 10th floor, there by saving 8.77% of the gross concrete volume with no amendment in material property and size of the full wall structure. However, for same concrete volume, it can be reduced only up to 11th floor in the 2nd case.

Lin and Kuo (1988) had conducted finite element analysis and experimental work to study the ultimate strength of shear wall with openings under lateral load. The test program demonstrated the shear behavior of reinforced concrete walls with different sizes of opening and reinforcing patterns around the opening. In the test program, the different amount and pattern of reinforcement were arranged around the openings so that, overall dimension of boundary element and reinforcement were same for all wall units. The test results indicated that the shear strength contributed by diagonal reinforcement around opening reached 40% of its yield strength, while the shear strength contributed by rectangular arrangement reached 20% of its yield strength. It was also concluded that shear capacity of section is not only affected by width of opening but also affected by depth of opening as well.

Kim and Lee (2003) proposed a method for the analysis of shear wall with openings using super elements (Figure 13a) to model the shear wall. Matrix condensation technique was used to eliminate the degrees of freedom (DOFs) in the shear wall except at end nodes of fictitious beams, added to enforce the compatibility conditions at the boundaries of the super elements. Static and dynamic analyses of shear walls with various sizes of door and window openings were performed to verify the efficiency and accuracy of the proposed method. Fine mesh (Figure 13b) with large number of finite elements to model the shear wall was assumed to be most accurate for verification of the proposed method. The results (top displacements, natural period) of the method were found very close to those obtained from the fine mesh model regardless of the number, size and location of the openings.



(a) Super element used for wall model (b) Fine mesh model using FEM

Figure 13 Models of shear wall with openings using different elements
Source: Kim and Lee (2003)

Qaqish and Daqqaq (2000) studied on effect of small openings on behavior of shear walls and the effect of opening size on behavior of coupled shear walls. It was concluded that opening area approximately less than 0.11 times the wall area surrounded by centerlines of columns and beams can be categorized as small openings. Comparing vertical stresses at the base of shear wall for each size of opening, it was found that the effects of small opening can be neglected on overall state of stress due to opening. It was also concluded that the wall with opening area less than 0.11 times the wall area acts as single cantilever while the wall with that greater than 0.11 times and less than 0.29 times the wall area acts as couple shear walls.

Yanez, Park and Paulay (1992) studied on seismic behavior of reinforced concrete walls with square openings of different size and arrangement under reversed cyclic loading. From experimental results, it was concluded that appropriately designed walls with staggered openings can have the same behavior and ductility as walls with regular openings. It was also concluded that the stiffness of walls is dependent on the size of the openings not on their horizontal locations. However, effects of vertical location were not mentioned. It was suggested that the stiffness of walls without openings can be used for the stiffness of the walls with openings smaller than 10% of the wall area.

El-Tawil *et. al* (2002) studied on effect of coupling ratio on response of hybrid coupled walls system. The results of analysis were examined for evidence of behavioral trend versus the change in coupling ratio in the system. In the study, coupling ratio (CR) is defined as the percentage of overturning moment resisted by the coupling action in the couple wall system. Stiffness of the system was found increased with an increase in coupling ratio.

After studying number of literature, it can be seen that number of experimental and analytical studies had been done for frame-shear wall structures and shear walls with openings. However, present study is different than previous studies in aspect of type of shear wall, openings configurations, loads applied to the structures etc.

RESEARCH METHODOLOGY

The research procedures followed for this study are as described below:

1. Literature review

Many of the literatures related to research work were collected while doing literature survey. Literatures on shear wall structures and related subjects were reviewed.

2. Model design

One of the objectives of this model designing is to ensure that the models represent the characteristics of apartment buildings. These days, high-rise buildings are different in shape, height and functions. This makes each building characteristics different from each others. There are some standards for each kind of high-rise buildings, such as residential, official and commercials. However, for model designing, main factors such as grid spacing, floor shape, floor height and column section were considered. Two buildings with different number of stories, one with 6-story and other with 12-story having same floor plan of 35 m × 15 m dimensions were considered for this study. The floor plans were divided into seven by three bays in such a way that center to center distance between two grids is 5 meters as shown in Figure 14. The floor height of the building was assumed as 3 meters for all floors.

Mostly in apartment buildings, floor plan will be same for all floors. So the buildings were considered with same floor plan in all floors having shear walls at same location. Shear walls of same section were used for same height of buildings throughout the height. Only the size and location of the openings in shear walls were changed for both 6- and 12-story buildings. Tables 1 and 2 show the sizes and locations of door and window openings used in the shear walls of the proposed typical frame-shear wall structures. Horizontally centered door openings of 2.1 m height (Figures 15a and 16a) were increased from 14% to 35% varying the door width ‘b_o’.

Similarly, window openings of 2 m width located at 0.6 m from floor level as shown in Figures 15c and 16c were increased from 12% to 28% varying the opening height 'h_o'. Door openings of size 1 m × 2.1 m i.e. opening area of 14% were eccentrically located at a horizontal distance 'e_h' from center of vertical boundary element in shear wall to the edge of doors as shown in Figures 15b and 16b. The distance 'e_h' was varied from 1 m to 2 m to see the effects of opening locations in horizontal direction. Similarly, window openings of 2 m × 1.2 m i.e. opening area of 16% were located at different vertical distance 'e_v' from floor level to the bottom of windows as shown in Figures 15d and 16d. The distance 'e_v' was varied from 0.3 m to 1.5 m to see the effects of opening locations in vertical direction.

Table 1 Sizes and locations of door openings in shear walls

Horizontally centered door openings of 2.1 m height		**1 m × 2.1 m door openings at different horizontal distance 'e_h'**	
b_o/L	**Opening area (%)**	**e_h (m)**	**Opening area (%)**
0.2	14	1.00	14
0.3	21	1.50	14
0.4	28	1.75	14
0.5	35	2.00	14

Table 2 Sizes and locations of window openings in shear walls

2 m wide window openings at 0.6 m from floor level		**2 m × 1.2 m window openings at different vertical distance 'e_v'**	
h_o/h	**Opening area (%)**	**e_v (m)**	**Opening area (%)**
0.3	12	0.3	16
0.4	16	0.6	16
0.5	20	0.9	16
0.6	24	1.2	16
0.7	28	1.5	16

Rectangular shear wall of size 5 m × 3 m with aspect ratio (h/L) of 0.6 was considered to investigate an accuracy of the simplified method in evaluating the stiffnesses/rigidities of shear walls with opening. Only the openings sizes and configurations were varied in the shear wall section for parametric study. Sizes and locations of openings used for the parametric study are given in Tables 3 and 4, respectively. Vertical location was varied for two different sizes of openings to evaluate the stiffnesses of shear walls with opening at different vertical locations. One is of size 1 m × 1.2 m which is usually used in store rooms, bath rooms and toilets where no more light is required. Other is of size 2 m × 1.2 m usually used in all other rooms where enough light is required.

Table 3 Sizes of openings used in single story rectangular shear walls

	Opening area (%)				
b_o/L	**h_o/h = 0.3**	**h_o/h = 0.4**	**h_o/h = 0.5**	**h_o/h = 0.6**	**h_o/h = 0.7**
0.2	6	8	10	12	14
0.3	9	12	15	18	21
0.4	12	16	20	24	28
0.5	15	20	25	30	35

Table 4 Vertical locations of openings in single story rectangular shear walls

Vertical location of openings 'e_v' in meter			
Opening size 1 m × 1.2 m		**Opening size 2 m × 1.2 m**	
Opening (%)	**e_v (m)**	**Opening (%)**	**e_v (m)**
8	0.0	16	0.0
8	0.3	16	0.3
8	0.6	16	0.6
8	0.9	16	0.9
8	1.2	-	-
8	1.5	-	-

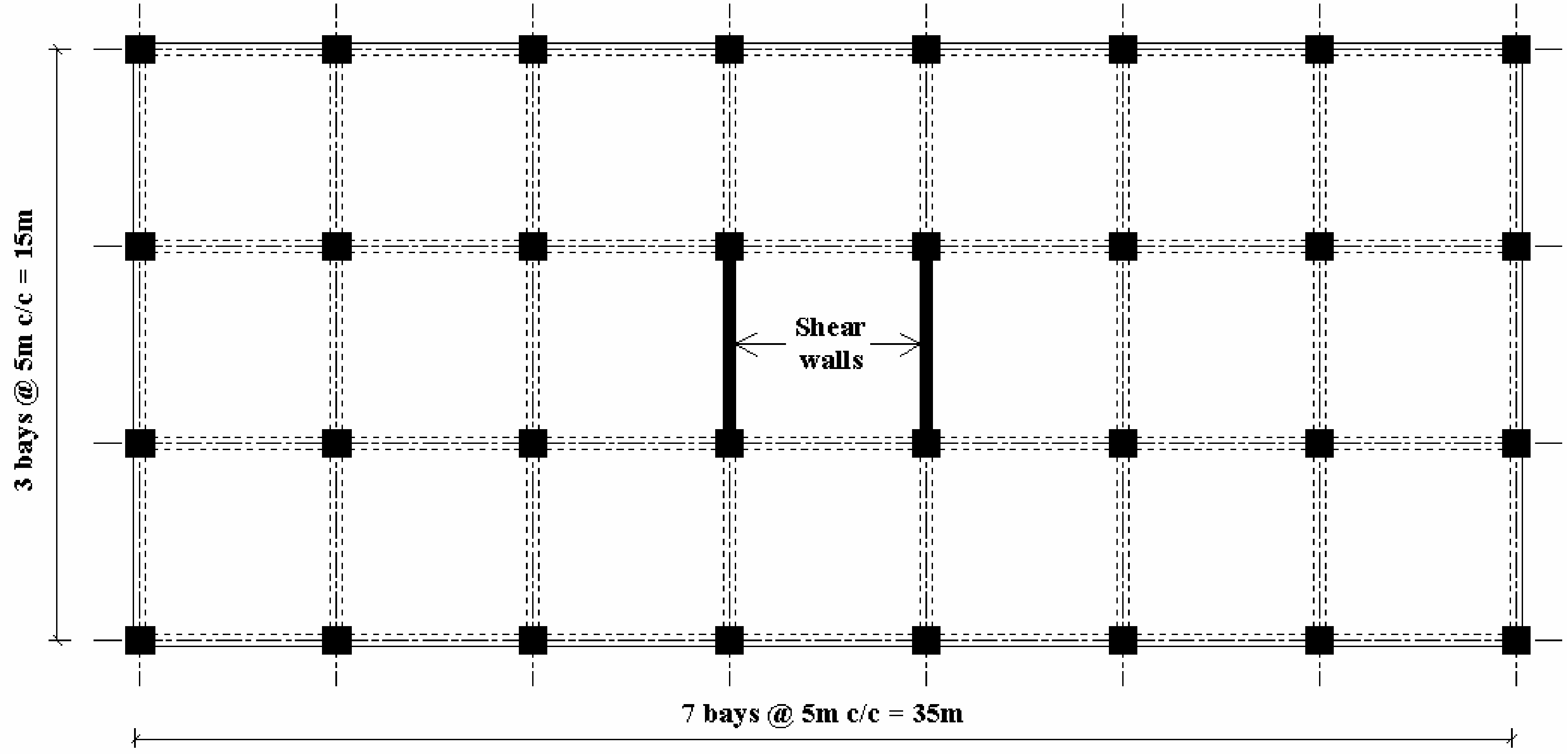


Figure 14 Typical floor plan for 6- and 12-story buildings with frame-shear wall structures

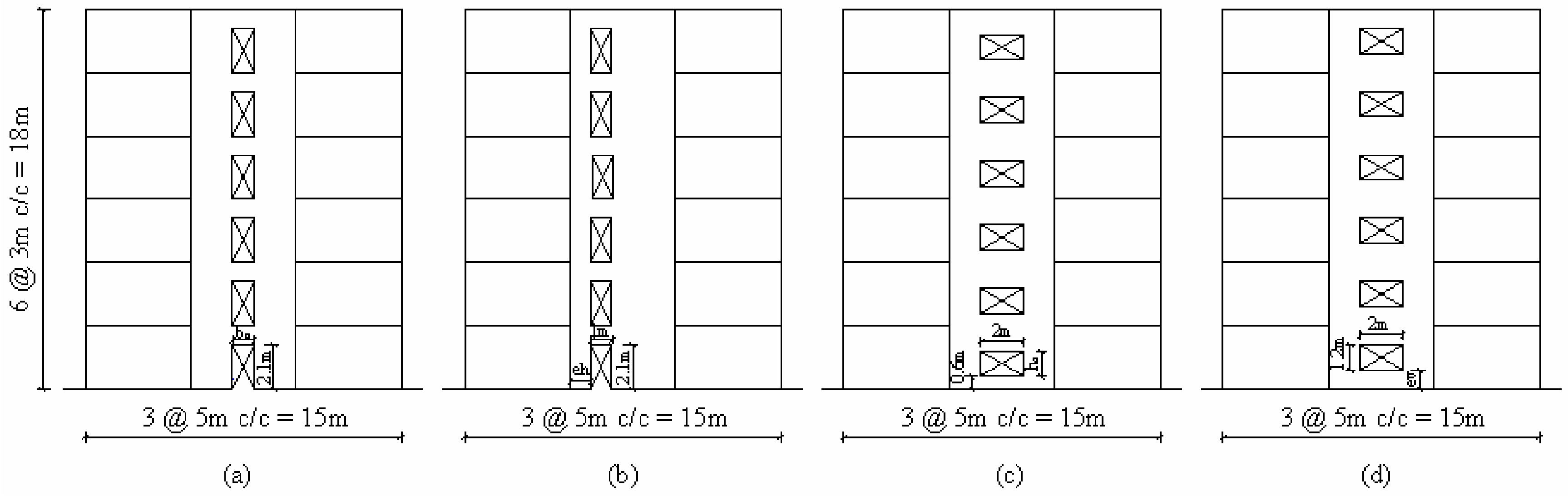


Figure 15 Sections of 6-story buildings with different sizes and locations of door and window openings in shear walls

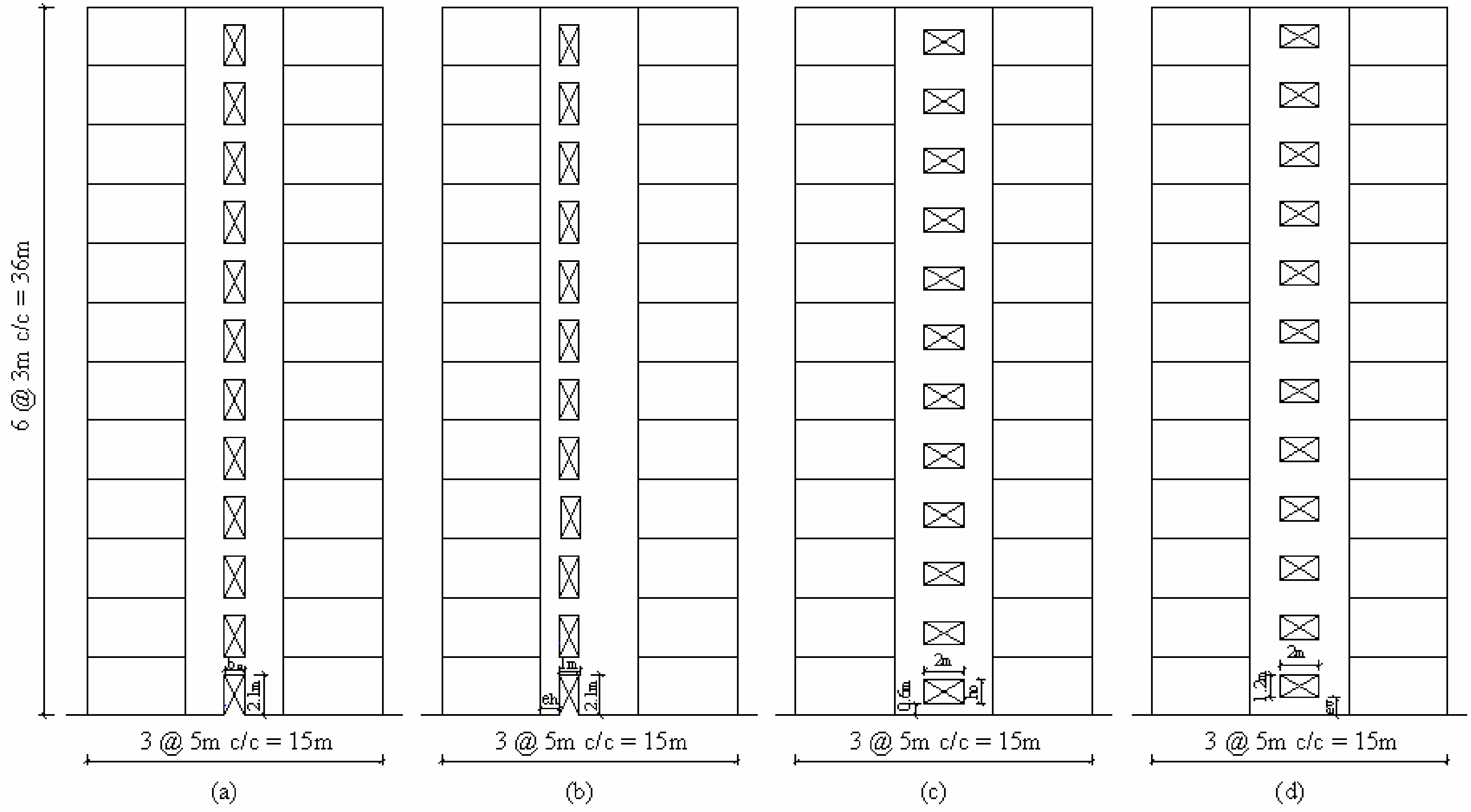
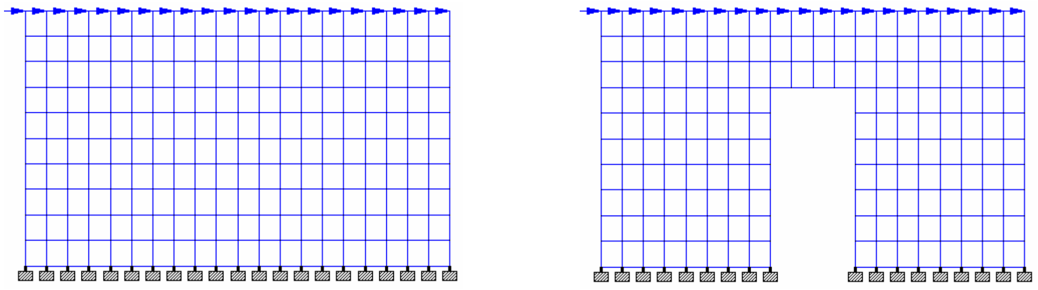


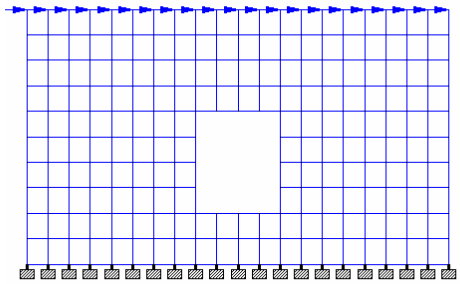
Figure 16 Sections of 12-story buildings with different sizes and locations of door and window openings in shear walls

3. Structural modeling for analysis

Rectangular shear walls with and without opening were modeled using plate elements available in structural analysis program STAAD.Pro to obtain the stiffnesses/rigidities of shear walls using finite element method (FEM), as shown in Figures 17. Uniformly distributed lateral load was applied at the top of shear walls. For parametric study, the percentage of horizontally centered opening at the base of wall was increased from 6% to 14%, 9% to 21%, 12% to 28% and 15% to 35% by increasing the ratio of opening height to shear wall height (h_o/h) from 0.3 to 0.7 for fixed ratio of opening width to shear wall length (b_o/L) of 0.2, 0.3, 0.4, 0.5, respectively. Locations of opening size 1 m × 1.2 m and 2 m × 1.2 m were changed in vertical direction to obtain the stiffnesses of shear walls with same size of opening at different vertical locations.



(a) Without opening (b) With 1 m × 2.1 m opening at base



(c) With 1 m × 1.2 m opening at 0.6 m from base

Figure 17 Finite element models of single story rectangular shear walls

To study the effects of openings sizes and locations in shear walls on seismic responses of buildings, three dimensional (3D) geometric models of the buildings were developed in STAAD.Pro (Appendix F). Beams and columns were modeled as beam elements. Shear walls were modeled as plate elements. Floor slabs were modeled as rigid horizontal plane. At each floor level, an additional node termed as 'master' node with three degree of freedoms, two translations and one rotation about normal to the plane of the floor was introduced. The other corresponding three degree of freedoms of all floor nodes called slave nodes were related to those of the master node at same floor.

Due to time limitations, it was impossible to account accurately for all aspects of behavior of all the components and materials even if their sizes and properties were known. Thus, for simplicity, following assumptions were made for the structural modeling:

1. The materials of the structure were assumed as homogeneous, isotropic and linearly elastic.
2. The effects of secondary structural components and non structural components such as staircase, masonry infill walls were assumed to be negligible.
3. Floors slabs were assumed rigid in plane.
4. Foundation for analysis was considered as rigid.

4. Structural analysis

Initial dimensions of the structural elements for both 6- and 12-story buildings were assumed on the basis of gravity loads and imposed loads. Since, the buildings were assumed as apartment buildings, imposed load of 2 kN/m^2 and load due to floor finish plus partition were taken as 1.5 kN/m^2 as per Indian Standard, IS 875 (part2): 1987. Lateral loads due to earthquake (EL) were calculated considering full dead load (DL) plus 25% of imposed load (IL), using seismic coefficient method given in IS 1893 (Part 1): 2002. In which, imposed load on roof was not considered. Total base shear (V_B) was calculated by:

$$V_B = A_h W \tag{14}$$

where,

W = seismic weight of the building and

A_h = the design horizontal seismic coefficient which is determined by using the following expression:

$$A_h = \frac{ZIS_a}{2Rg} \tag{15}$$

Since the buildings were assumed as apartment buildings with dual system (shear wall and special moment resisting frame) situated in seismic zone V, importance factor (I) = 1, zone factor (Z) = 0.36 and response reduction factor (R) = 5 as given in IS code (Appendix A) were used for lateral load calculations. Assuming medium type of soil on which the foundations rest and the damping ratio of 5% for concrete structure, average response acceleration coefficient (S_a/g) was obtained from Appendix Figure A1, depending on the approximate fundamental natural time period of the structure estimated by:

$$T_a = \frac{0.09H}{\sqrt{d}} \tag{16}$$

In which,

H = height of building in meter

d = base dimension of the building at plinth level, in meter, along the considered direction of lateral force

The design base shear (V_B) computed from the Eq. (14) was distributed along the height of the building as per the following expression:

$$Q_i = V_B \times \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \tag{17}$$

Where,

Q_i = design lateral force at floor i

W_i = seismic weight of floor i

h_i = height of floor i measured from base, and

n = number of stories in the building, is the number of levels at which masses are located

Assuming the floor is capable of providing rigid diaphragm action (floor to be infinitely rigid in the horizontal plane), total shear in any horizontal plane is distributed to the various vertical elements according to their relative stiffness.

After several analyses in STAAD.Pro using equivalent static lateral force analysis for various load combinations given in IS 1893 (Part 1): 2002 (Appendix A), final dimensions of the structural elements for further analyses of the buildings were obtained. The maximum percentage of reinforcements for structural elements was limited to 4% of concrete gross area as per IS 456: 2000. Concrete grade of M25 i.e. characteristic compressive strength (f_{ck}) of 25 N/mm^2 was assumed for all structural elements. Maximum story drift due to design lateral force with partial load factor of 1 was limited to 0.004 times the story height as given in IS 1893 (Part 1): 2002. Similarly, the maximum nominal shear stress in shear wall due to factored loads was limited to $0.17f_{ck}$ for wall with largest opening area used for this study. Then, the developed models were analyzed for each size and location of openings in shear walls to evaluate the effects of openings in shear wall on stiffnesses as well as on seismic responses of the frame-shear wall structures.

Material properties were assumed same for all structural elements used in both 6- and 12-story buildings. The material properties assumed for final structural analysis are as follows:

Modulus of elasticity (E) = 2.5×10^7 kN/m^2

Poisson's ratio (ν) = 0.17

Unit weight of concrete (γ) = 25 kN/m^3

Section properties of structural members used for the final structural analyses of buildings are as follows:

For 6-story buildings:

Column size	: 0.45 m × 0.45 m
Beam size	: 0.25 m × 0.50 m
Shear wall thickness	: 0.25 m

For 12-story buildings:

Column size	: 0.70 m × 0.70 m
Beam size	: 0.30 m × 0.50 m
Shear wall thickness	: 0.30 m

Shear wall of same thickness and material as obtained for 6-story buildings was used to model the single story rectangular shear walls for evaluating an accuracy of simplified method in evaluating the stiffnesses/rigidities of shear walls with various opening sizes and configurations. Linear elastic analyses were performed for all modeled shear walls. The stiffnesses/rigidities of the shear walls using finite element method (R_{FEM}) were obtained for various openings sizes and arrangements in shear walls.

For the stiffnesses/rigidities of walls using the simplified method ($R_{simplified\ method}$), the wall element is regarded as deep cantilever beam fixed at base with shear V applied at the top of the wall. Thus, the displacement of the solid wall was calculated using the relation given by Varayani (2002) i.e. Eq. (8). The solid strip and piers formed by the openings were assumed as fixed-fixed at both ends with reference to the conditions used by Neuenhofer (2006). Thus, the displacements of each pier were calculated using Eq. (10) and the combined displacement of piers was calculated using:

$$\Delta_{\text{piers}} = \frac{1}{\dfrac{1}{\Delta_{\text{pier1}}} + \dfrac{1}{\Delta_{\text{pier2}}}} \tag{18}$$

Similarly, displacement of solid strip was calculated substituting L for l in Eq. (10) as follows:

$$\Delta_{\text{solid strip}} = \frac{\dfrac{h_o^{\,3}}{L^3} + 2.81\dfrac{h_o}{L}}{Et} \tag{19}$$

$$\therefore \Delta_{\text{wall}} = \Delta_{\text{solid wall}} - \Delta_{\text{solid strip}} + \Delta_{\text{piers}} \tag{20}$$

Ratio of stiffnesses and different in stiffnesses obtained from the simplified method and FEM were calculated as:

$$\text{Stiffness ratio} = \frac{R_{\text{wall}}}{R_{\text{solid wall}}} = \frac{\Delta_{\text{solid wall}}}{\Delta_{\text{wall}}} \tag{21}$$

$$\text{Different (\%)} = \left(\frac{R_{\text{simplified method}}}{R_{\text{FEM}}} - 1\right) \times 100 \tag{22}$$

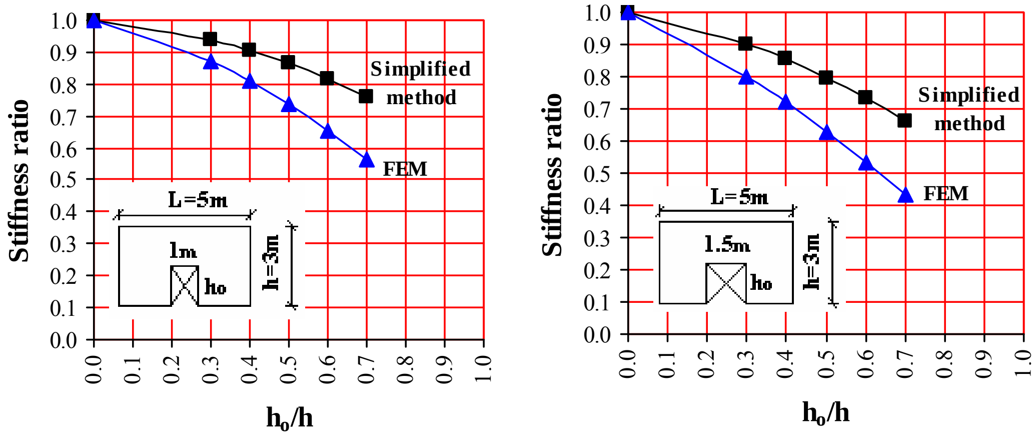
Sample calculation for the stiffnesses/ rigidities of shear walls without and with opening using the simplified method is given in Appendix B.

RESULTS AND DISCUSSIONS

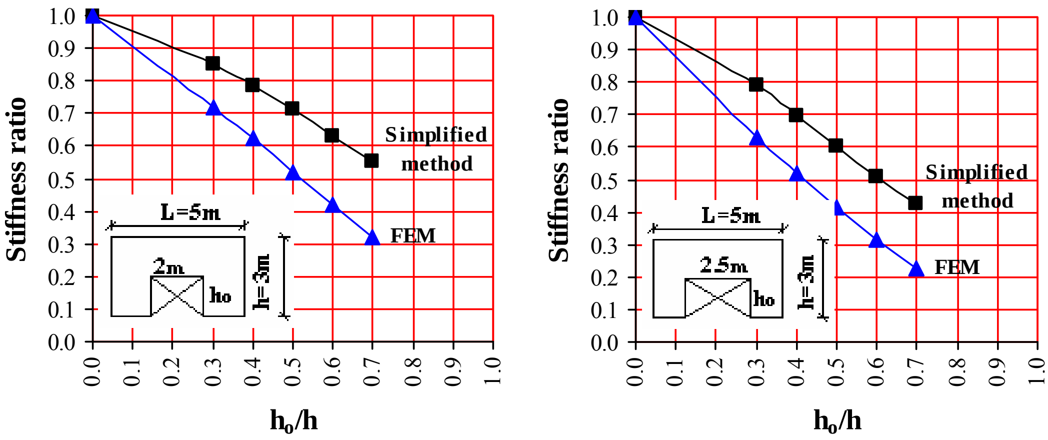
The stiffnesses/rigidities of shear walls with and without opening obtained from both the simplified and finite element methods are compared in the study. The stiffnesses of the walls with opening relative to that of solid wall are presented in Figure 18 in terms of stiffness ratio. In addition to that, percentage different in stiffness of the walls from the simplified method and FEM is presented in Figures 19 and 20.

Figures 18a to 18d reveal that the stiffness of the wall decreases with an increase in the opening size for both the simplified and finite element methods. Using the simplified method, stiffness of wall with 12% opening has been found 82% of that of solid wall with aspect ratio of the opening (h_o/b_o) equals 1.8 (Figure 18a), where h_o is height of the opening and b_o is width of the opening. The stiffness of the wall with the same opening size and aspect ratio, using finite element method has been found 65% of the stiffness of solid wall. It differs in both the methods for walls with different aspect ratio of opening for the same opening area. However, the simplified method often overestimates the stiffness of walls with opening because the fixed-fixed condition assumed for top of the solid strip and piers increases the actual rigidity.

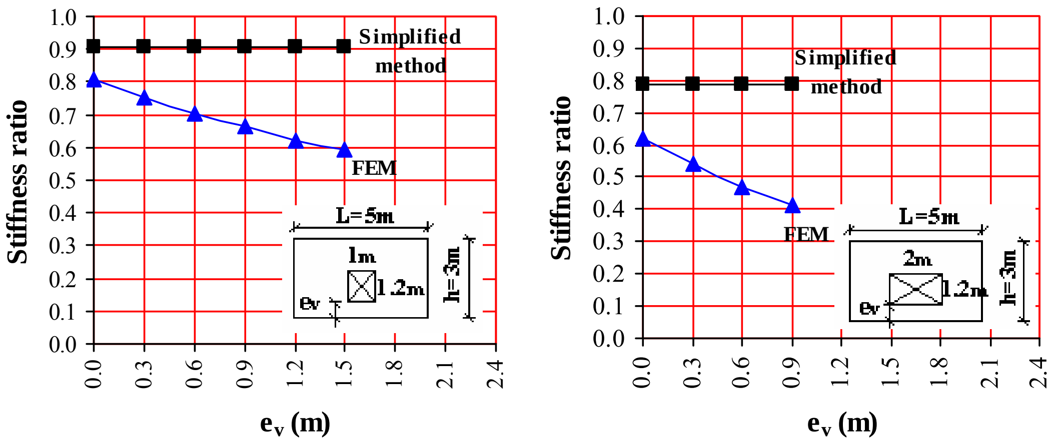
The relative stiffness of shear walls with opening of same size at different vertical locations is a straight line in Figures18e and 18f. It reveals that the stiffness of the wall obtained by the simplified method does not consider the vertical location of opening. In finite element method, it strongly depends on vertical location of opening.



(a) 1 m wide opening (b) 1.5 m wide opening



(c) 2 m wide opening (d) 2.5 m wide opening



(e) 1 m × 1.2 m opening at different e_v (f) 2 m × 1.2 m opening at different e_v

Figure 18 Stiffness of walls with opening relative to that of solid wall

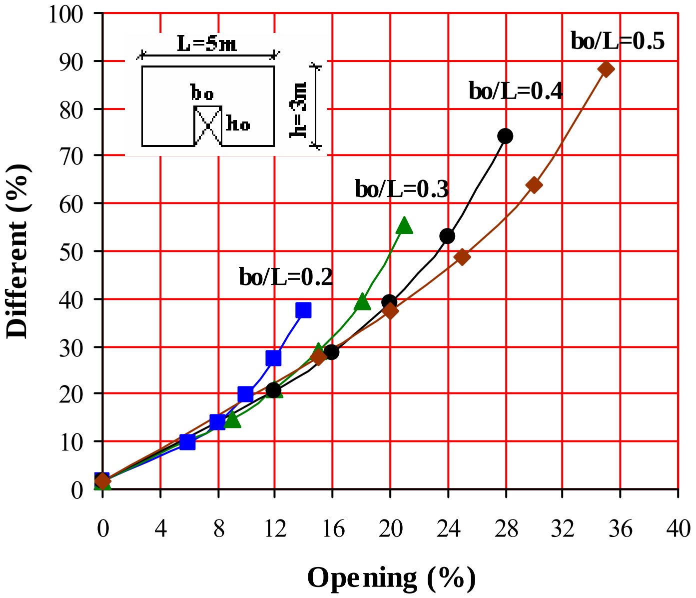


Figure 19 Different in stiffness of walls using the simplified method and FEM, with respect to opening size

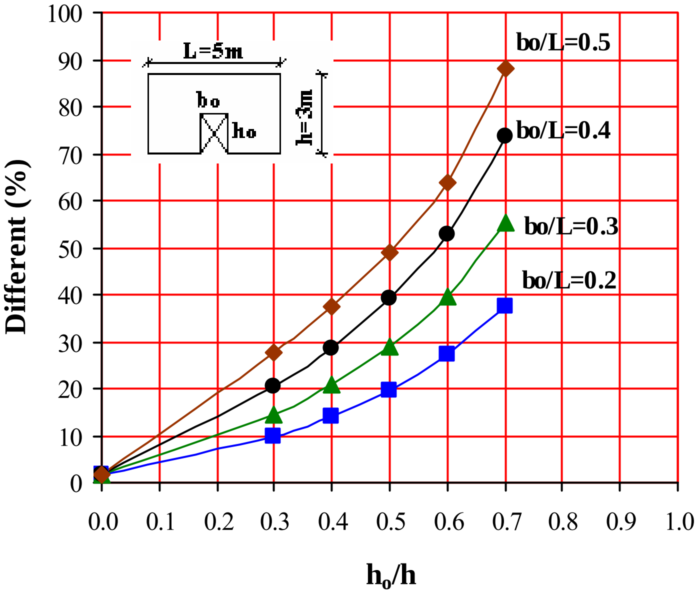
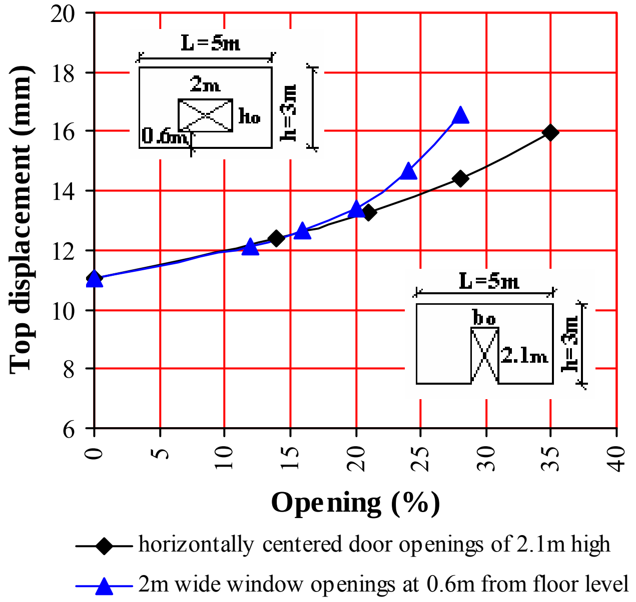


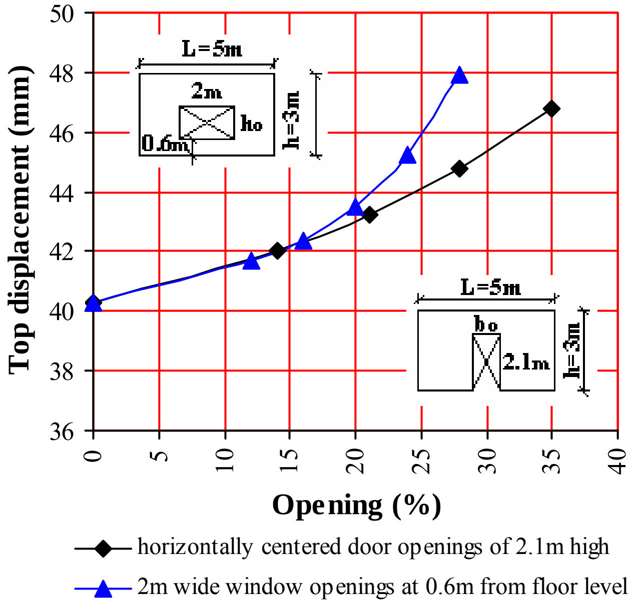
Figure 20 Different in stiffness of walls using the simplified method and FEM, with respect to ratio of opening height to wall height (h_o/h)

Figure 19 and 20 clearly present that an accuracy of the simplified method decreases with an increase in the opening size. For wall with 20% opening, the simplified method overestimates the stiffness of the wall by about 39% for opening aspect ratio (h_o/b_o) of 0.75. However, percentage difference increased to 50% for the wall with the same opening area but higher aspect ratio of 1.3. Thus, the simplified method gives remarkably poor results for wall with higher aspect ratio of opening (h_o/b_o) than that with lower aspect ratio for same opening area. The simplified method can overestimate the stiffness of shear wall by about 30% for wall with maximum opening area of 16% at the base of shear wall. For this area, an aspect ratio of opening (h_o/b_o) has to be less than or equal to 0.6. It can be clearly seen from Figure 19 that the simplified method can overestimate the stiffness of wall by less than 20% for the wall with opening area $\leq$ 10%, regardless of the aspect ratio of opening.

The seismic responses of the frame-shear wall structures such as top displacement of the systems and base shear in shear walls with different openings sizes and locations due to earthquake load are compared in Figure 21 to Figure 26. Thus, it is easy to observe the effects of opening size and its location in shear wall on seismic response of the frame-shear wall structures under seismic excitation.

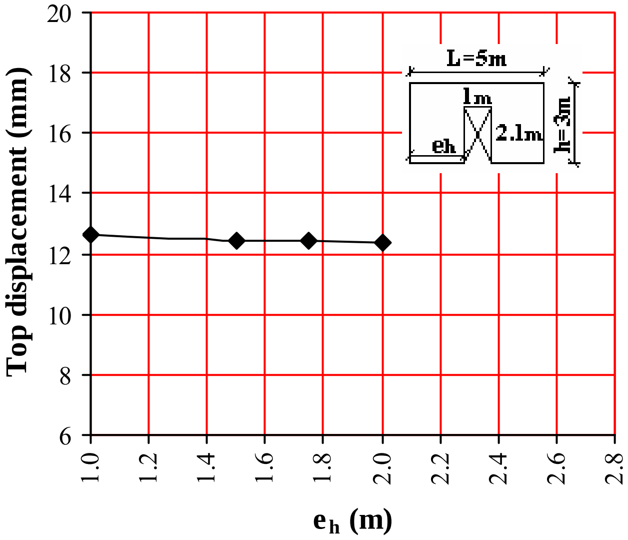


(a) 6-story buildings

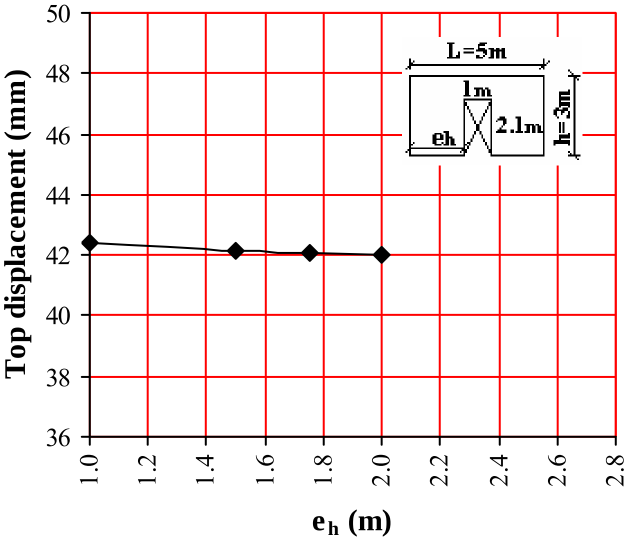


(b) 12-story buildings

Figure 21 Top displacement of frame-shear wall structures with different size of door and window openings in shear walls

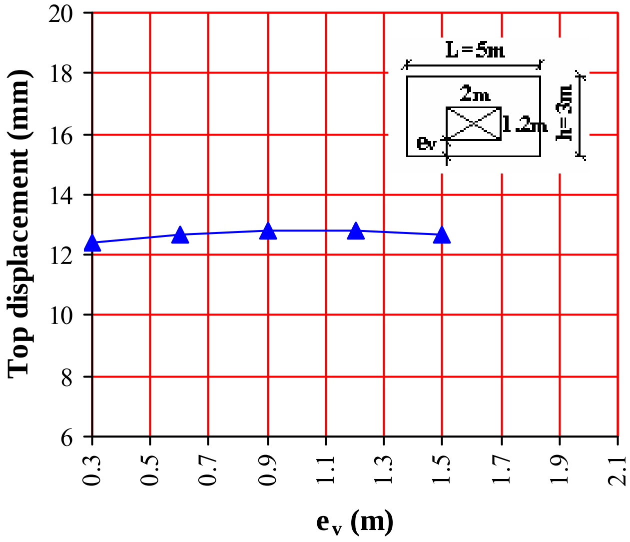


(a) 6-story buildings

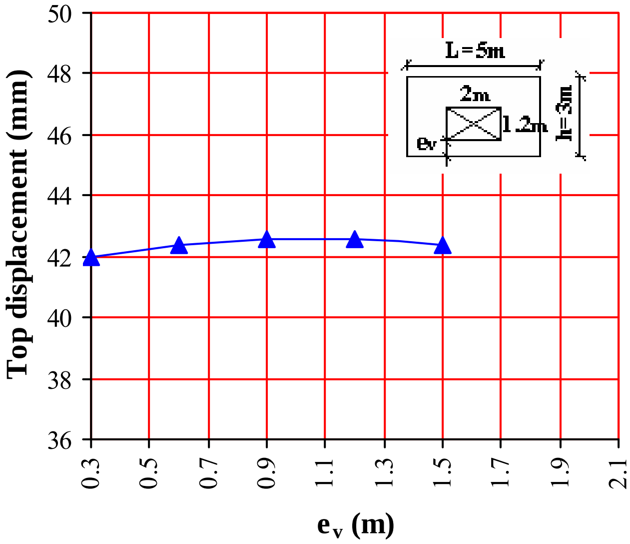


(b) 12-story buildings

Figure 22 Top displacement of frame-shear wall structures with 1 m × 2.1 m door openings at different horizontal locations in shear walls

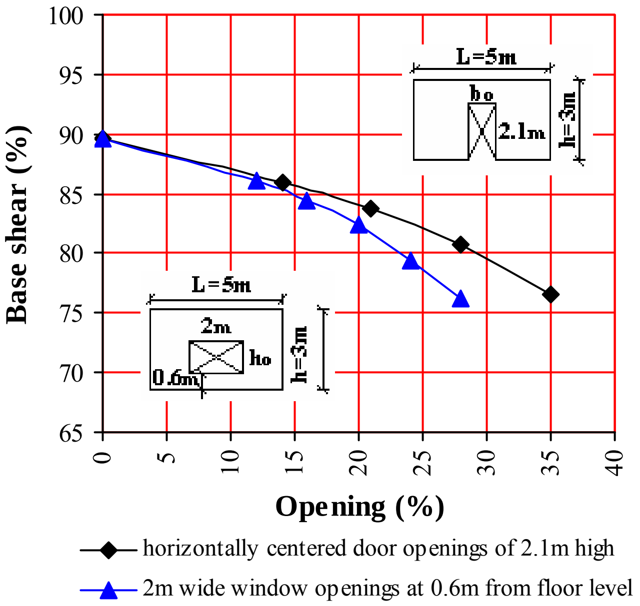


(a) 6-story buildings

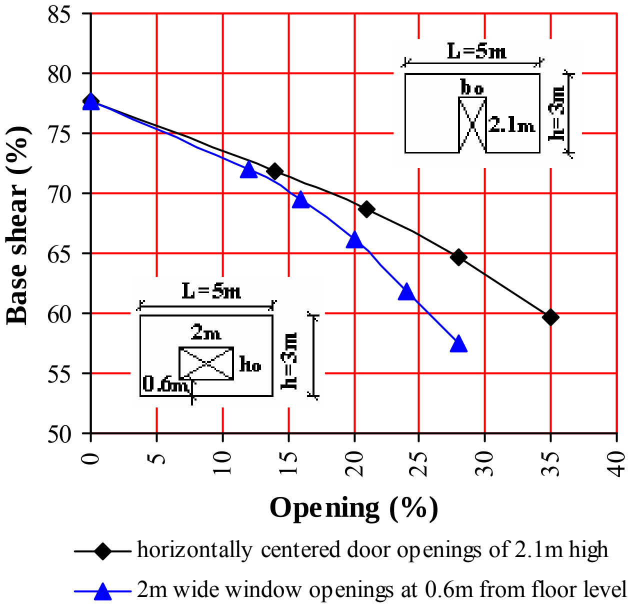


(b) 12-story buildings

Figure 23 Top displacement of frame-shear wall structures with 2 m × 1.2 m window openings at different vertical locations in shear walls

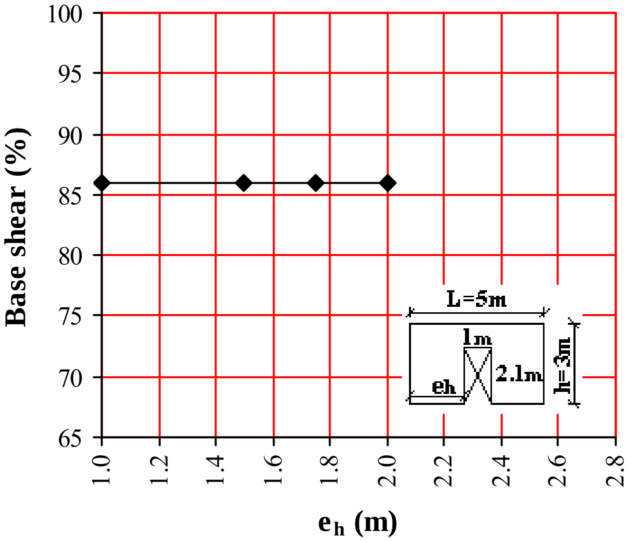


(a) 6-story buildings

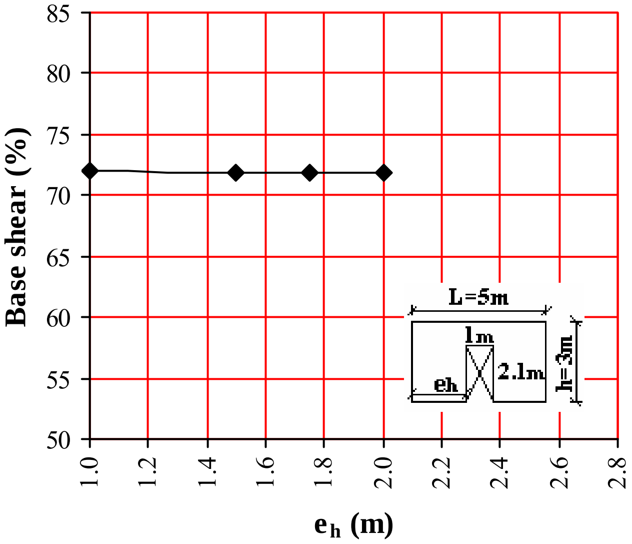


(b) 12-story buildings

Figure 24 Base shear in shear walls with different size of door and window openings

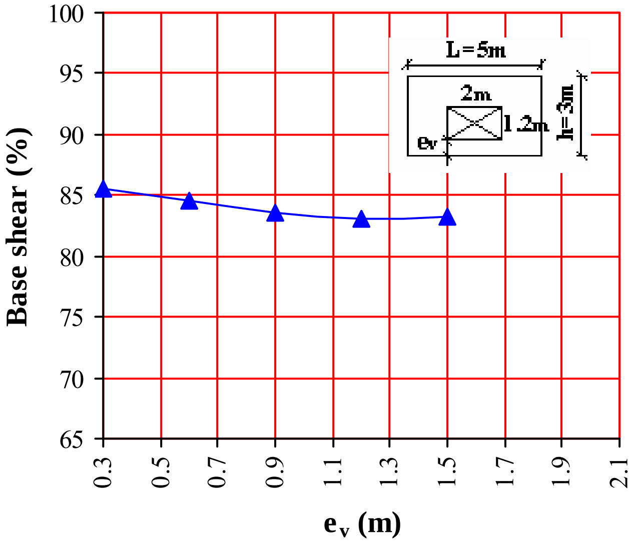


(a) 6-story buildings

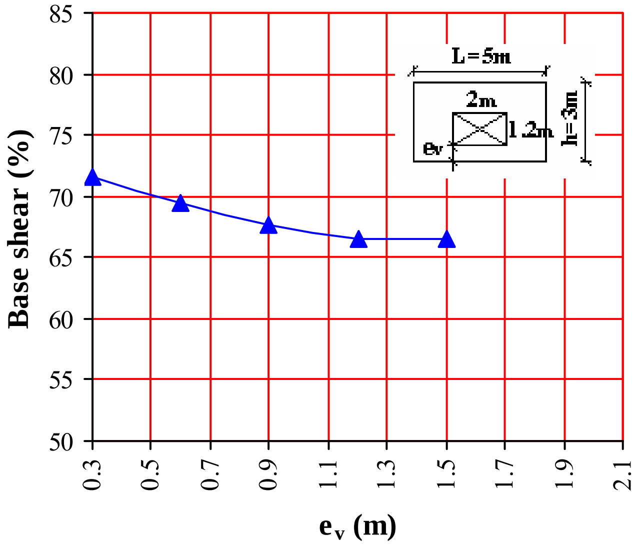


(b) 12-story buildings

Figure 25 Base shear in shear walls with 1 m × 2.1 m door openings at different horizontal locations



(a) 6-story buildings

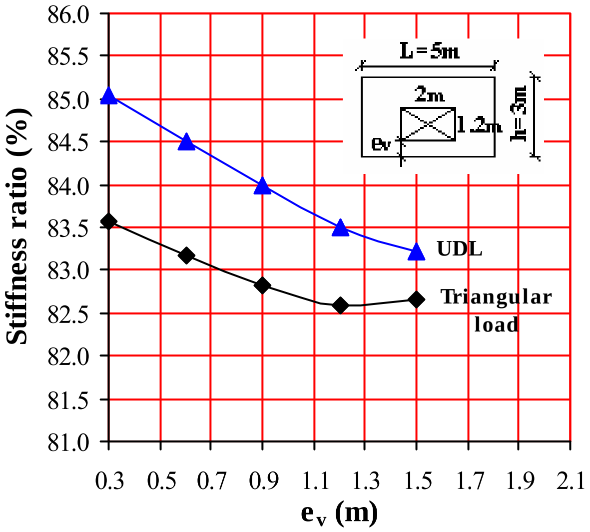


(b) 12-story buildings

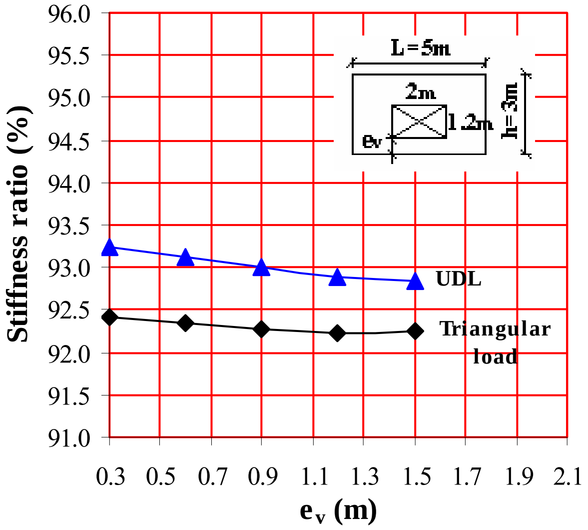
Figure 26 Base shear in shear walls with 2 m × 1.2 m window openings at different vertical locations

Figure 21 shows that the top displacement of both the 6- and 12-story buildings increases with an increase in openings sizes while, base shear in shear walls decreases (Figure 24). It means that the stiffness of the systems decreases with an increase in openings sizes. The stiffness of the system is not only affected by the width of the openings but also affected by the height of the openings. For shear walls with opening area less than or upto 20% of the wall area, the rate of increase in top displacement of the system and decrease in base shear in shear walls with an increase in opening size is nearly the same for different openings arrangements, in both 6- and 12-story buildings. The top displacement of 6- and 12-story buildings (Figure 21) with door openings of 28% in shear walls has been found 14.42 mm and 44.79 mm, respectively. It has been found 16.55 mm and 47.89 mm, respectively for shear walls with same opening area of windows at 0.6 m from floor level. Similarly, base shear in shear walls of 6- and 12-story buildings (Figure 24) has been obtained 80.8% and 64.6%, respectively while it is 76.2% and 57.4% for shear walls with window openings of same area at 0.6 m from floor level. It reveals that base shear in shear walls of 6-story building with window opening of 28% at 0.6 m from floor level is 4.6% less than that with door opening of same size at each floor level. Similarly, it is 7.2% less in case of 12-story building. Thus, the opening arrangement in shear wall has significant effects on the stiffness of the system, when the opening area in the shear walls is larger than 20%.

The effect of vertical location of window openings (Figures 23 and 26) has been found more sensitive than the horizontal location of door openings (Figures 22 and 25), on stiffness of the system. The stiffness of the system decreased with an increase in vertical distance ‘e_v’ from floor level to the bottom of window (Figure 23). However, it increased slightly when the window opening is shifted from 1.2 m to 1.5 m from floor level. The effect has been found same in the separate shear wall of same section and shape used in three dimensional models. The stiffness ratio of 6- and 12-story shear walls with window openings of size 2 m × 1.2 m at different vertical locations, used in three dimensional models, with different loads, is presented in Figure 27. Triangular load with maximum magnitude at top of shear wall and uniformly distributed lateral load (UDL) at the top of shear wall was applied to determine the stiffness of multistory shear walls. Since, the beams and columns sizes are constant for all structures, change in stiffness of the system with changes in opening sizes and locations is due to the changes in stiffness of the shear walls.



(a) Shear wall in 6-story buildings



(b) Shear wall in 12-story buildings

Figure 27 Stiffness of multistory shear wall with 2 m × 1.2 m window openings at different vertical locations, with different load, relative to that of solid wall

For this study, coupling ratios (CR) in shear walls were calculated as the ratio of overturning moment resisted by the coupling action due to axial forces in shear wall to the total overturning moment applied in the shear wall. Results obtained for shear walls with different opening configuration are presented in Figures 28 to 31.

The coupling ratio of shear wall has been found decreased with an increase in opening size in both 6- and 12-story buildings as noticed from Figure 28 and 29, respectively. This is because the stiffness of the coupling system relative to the walls decreases with reduction in concrete area of the wall. Consequently, the moment resisted by the coupling action of the shear wall due to axial forces decreases. Thus the induced vertical shear forces in the connecting beams decreases. Rate of decrease in coupling ratio of the shear wall with an increase in opening width has been found significant (Figure 28) than that with an increase in opening height (Figure 29). Coupling ratio of the wall with door openings at different horizontal locations has been found nearly the same (Figure 30). However, it has been found increased with an increase in vertical distance ‘e_V’ (Figure 31).

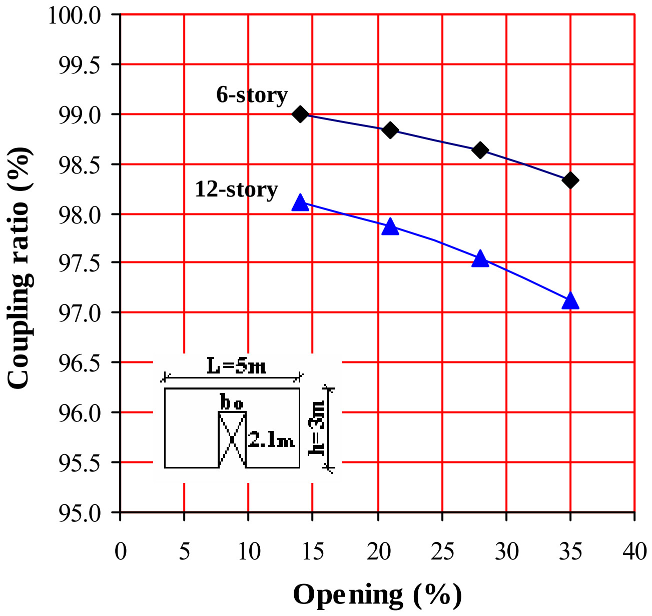


Figure 28 Coupling ratio of shear walls with different sizes of horizontally centered door openings

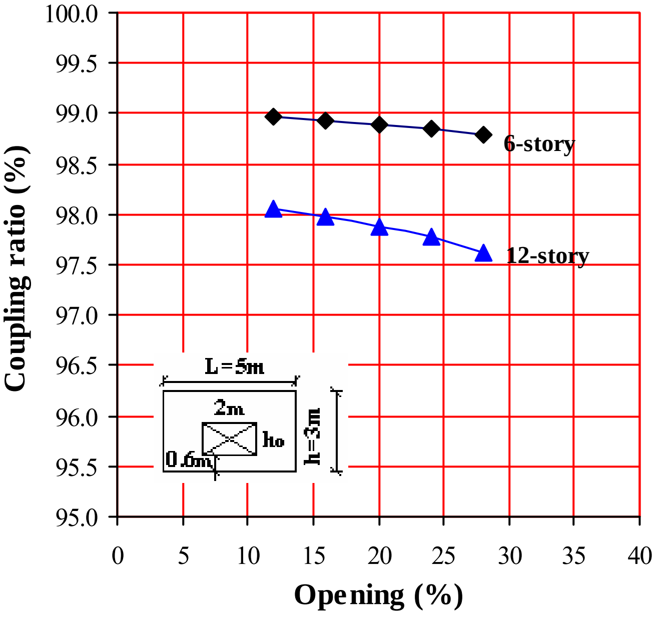


Figure 29 Coupling ratio of shear walls with different sizes of window openings at 0.6 m from floor level

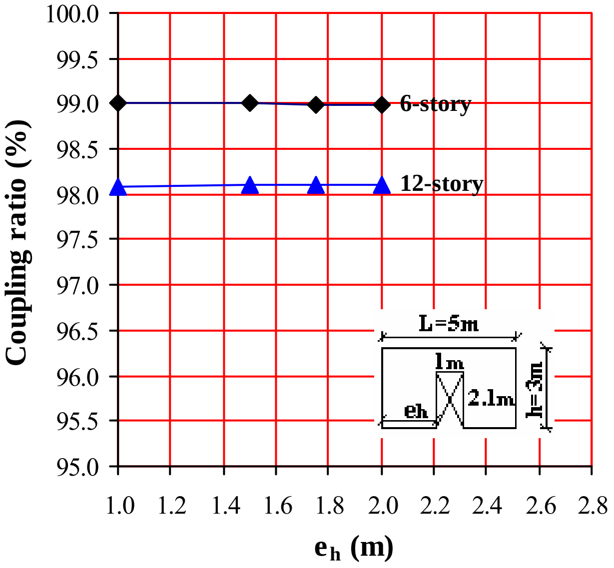


Figure 30 Coupling ratio of shear walls with 1 m × 2.1 m door openings at different horizontal locations

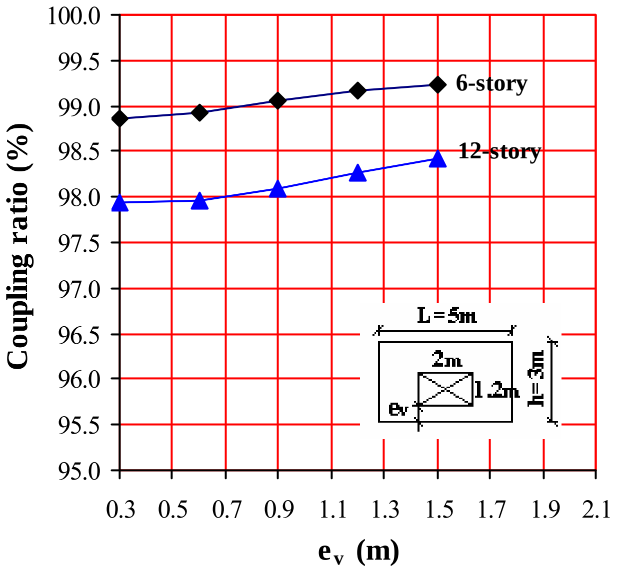


Figure 31 Coupling ratio of shear walls with 2 m × 1.2 m window openings at different vertical locations

Maximum principal stress is one of the parameter that can be used for failure criteria in concrete structures which is brittle in nature. Failure occurs in concrete structure when the maximum principal stress reaches either uniaxial tensile strength or uniaxial compressive strength. Thus, magnitudes and locations of maximum absolute principal and maximum principal stresses in shear walls due to service loads are compared in Tables 5 to 12 and Figures 32 to 35 for all structures. In addition to that, maximum absolute principal stress at top corner of opening at 2nd floor of shear walls, are compared in Figures 36 to 38 and Appendix Tables E1 to E4. Contour diagrams of maximum absolute principal stresses in shear walls are presented in Appendix Figures E1 to E6. Thus, the effects of openings sizes and locations in shear walls on response of frame-shear wall structures can be evaluated.

Figures 32 and Figure 33 show that maximum principal stress (S_{max}) in shear wall increases with an increase in openings sizes in both 6- and 12-story buildings. The maximum principal stress in shear wall with horizontally centered door openings of 14% in 6-story building has been found 6.03 N/mm^2. It increased to 10.98 N/mm^2 when the openings area is increased to 35%. In 12-story building, it is 6.39 N/mm^2

and 11.99 N/mm^2 for door openings of 14% and 35%, respectively. Location of maximum principal stress is at top of opening at 2nd floor (Tables 5 and 6) in shear walls with door openings of different sizes considered in this study. Similarly, for window openings, it is at the top of window at 2nd floor for opening area upto 24% in shear walls of both 6-and 12-story buildings (Tables 7 and 8). However, the location of maximum principal stress shifted to the bottom of window at 3rd floor in 6-story building and to the bottom of window at 4th floor in 12-story building with opening area of 28% in shear walls. This is because the behavior of wall tends to change from coupled wall systems to two independent walls. It is clear from the vertical stress (S_y) diagrams presented in Figures 41d and 42d and is discussed in the following section.

Table 5 Principal stresses in shear walls with horizontally centered door openings of 2.1 m height, in 6-story buildings

bo/L	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location of stress in shear wall**	**S_{max}**	**Location of stress in shear wall**
0.2	14	10.24	Base of wall	6.03	Top of door at 2nd floor
0.3	21	10.57	Base of wall	7.89	Top of door at 2nd floor
0.4	28	11.35	Top of door at 2nd floor	9.56	Top of door at 2nd floor
0.5	35	12.65	Top of door at 2nd floor	10.98	Top of door at 2nd floor

Table 6 Principal stresses in shear walls with horizontally centered door openings of 2.1 m height, in 12-story buildings

bo/L	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location of stress in shear wall**	**S_{max}**	**Location of stress in shear wall**
0.2	14	11.99	Base of wall	6.39	Top of door at 2nd floor
0.3	21	12.78	Top of door at 2nd floor	8.55	Top of door at 2nd floor
0.4	28	14.00	Top of door at 2nd floor	10.42	Top of door at 2nd floor
0.5	35	15.18	Top of door at 2nd floor	11.99	Top of door at 2nd floor

Figure 34 reveals that effect of horizontal locations of 1 m × 2.1 m door can be neglected in maximum principal stress in shear wall. Similarly, vertical location of 2 m × 1.2 m window has negligible effects in maximum principal stress in shear wall (Figure 35).

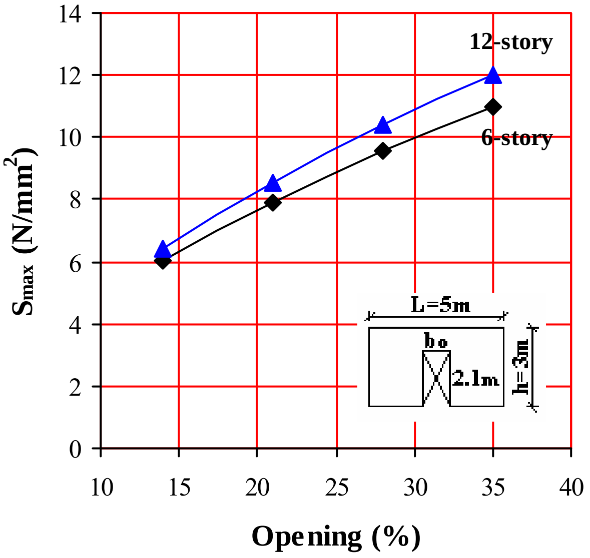


Figure 32 Maximum principal stress in shear walls with different size of doors

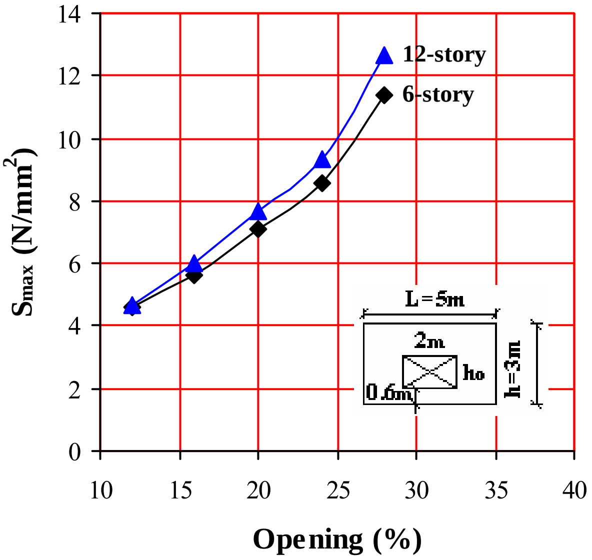


Figure 33 Maximum principal stress in shear walls with different size of windows

Table 7 Principal stresses in shear walls with 2 m wide window openings at 0.6 m from floor level, in 6-story buildings

h_o/h	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location of stress in shear wall**	S_{max}	**Location of stress in shear wall**
0.3	12	10.84	Base of wall	4.62	Top of window at 2nd floor
0.4	16	10.86	Base of wall	5.64	Top of window at 2nd floor
0.5	20	10.83	Base of wall	7.07	Top of window at 2nd floor
0.6	24	10.79	Top of window at 2nd floor	8.55	Top of window at 2nd floor
0.7	28	12.32	Bottom of window at 3rd floor	11.40	Bottom of window at 3rd floor

Table 8 Principal stresses in shear walls with 2 m wide window openings at 0.6 m from floor level, in 12-story buildings

h_o/h	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location of stress in shear wall**	S_{max}	**Location of stress in shear wall**
0.3	12	13.01	Top of window at 2nd floor	4.67	Top of window at 2nd floor
0.4	16	13.24	Top of window at 2nd floor	5.99	Top of window at 2nd floor
0.5	20	13.53	Top of window at 2nd floor	7.68	Top of window at 2nd floor
0.6	24	13.65	Top of window at 2nd floor	9.32	Top of window at 2nd floor
0.7	28	14.39	Bottom of window at 3rd floor	12.67	Bottom of window at 4th floor

The maximum absolute principal stress at the top corner of opening at the 2nd floor in shear wall increased with an increase in opening size as shown in Figures 36. However, it decreased when the window opening is increased from 24% to 28%. Maximum absolute principal stress at the opening corner decreased from 10.36 N/mm^2 to 8.93 N/mm^2 (Figure 37a) when horizontal distance (e_h) from center line of vertical boundary element in shear wall to the edge of doors openings of 14%, is increased from 1 m to 2 m in 6-story buildings. Similarly, it decreased from 13.96 N/mm^2 to 11.47 N/mm^2 (Figure 37b) for the same case in 12-story buildings. However, the effect of vertical location of the window opening is negligible (Figure 38)

Table 9 Principal stresses in shear walls with 1 m × 2.1 m door openings at different horizontal locations, in 6-story buildings

e_h (m)	**Opening (%)**	**Principal stresses in shear walls (N/mm^2)**			
		Max. absolute	**Location of stress in shear wall**	**S_{max}**	**Location of stress in shear wall**
1.00	14	11.01	Top of door at 1st floor	6.79	Top of door at 2nd floor
1.50	14	10.24	Base of wall	6.24	Top of door at 2nd floor
1.75	14	10.23	Base of wall	6.20	Top of door at 2nd floor
2.00	14	10.22	Base of wall	6.03	Top of door at 2nd floor

Table 10 Principal stresses in shear walls with 1 m × 2.1 m door openings at different horizontal locations, in 12-story buildings

e_h (m)	**Opening (%)**	**Principal stresses in shear walls (N/mm^2)**			
		Max. absolute	**Location of stress in shear wall**	**S_{max}**	**Location of stress in shear wall**
1.00	14	13.99	Top of door at 2nd floor	6.89	Top of door at 2nd floor
1.50	14	12.79	Top of door at 2nd floor	6.76	Top of door at 2nd floor
1.75	14	12.15	Top of door at 2nd floor	6.62	Top of door at 2nd floor
2.00	14	11.99	Base of wall	6.39	Top of door at 2nd floor

Table 11 Principal stresses in shear walls with 2 m × 1.2 m window openings at different vertical locations, in 6-story buildings

e_v (m)	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location of stress in shear wall**	**S_{max}**	**Location of stress in shear wall**
0.3	16	10.93	Base of wall	5.20	Top of window at 2nd floor
0.6	16	10.86	Base of wall	5.64	Top of window at 2nd floor
0.9	16	10.69	Base of wall	5.61	Top of window at 2nd floor
1.2	16	10.46	Base of wall	5.16	Bottom of window at 2nd floor
1.5	16	10.20	Base of wall	4.88	Bottom of window at 2nd floor

Table 12 Principal stresses in shear walls with 2 m × 1.2 m window openings at different vertical locations, in 12-story buildings

e_v (m)	Opening (%)	Principal stresses in shear walls (N/mm^2)			
		Max. absolute	**Location at shear wall**	**S_{max}**	**Location at shear wall**
0.3	16	13.12	Top of window at 2nd floor	5.32	Top of window at 2nd floor
0.6	16	13.24	Top of window at 2nd floor	5.99	Top of window at 2nd floor
0.9	16	13.14	Top of window at 2nd floor	6.02	Top of window at 2nd floor
1.2	16	12.88	Top of window at 2nd floor	5.41	Bottom of window at 3rd floor
1.5	16	12.50	Top of window at 2nd floor	5.06	Bottom of window at 2nd floor

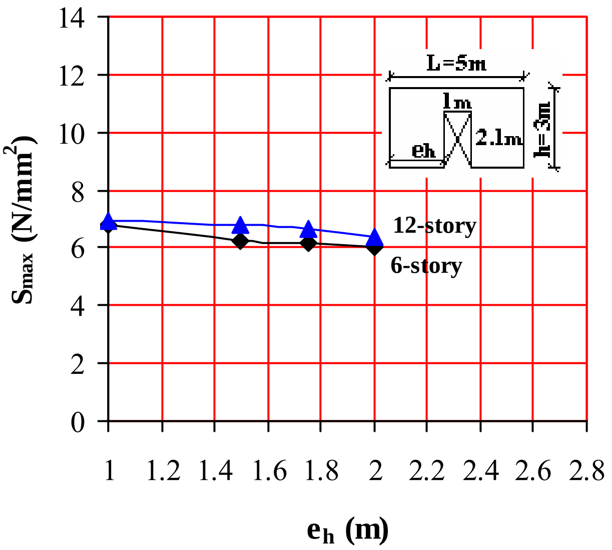


Figure 34 Maximum principal stress in shear walls with 1 m × 2.1 m door openings at different horizontal locations

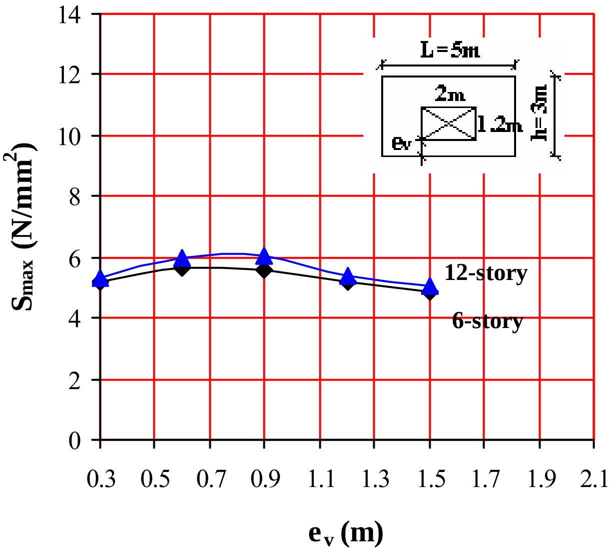
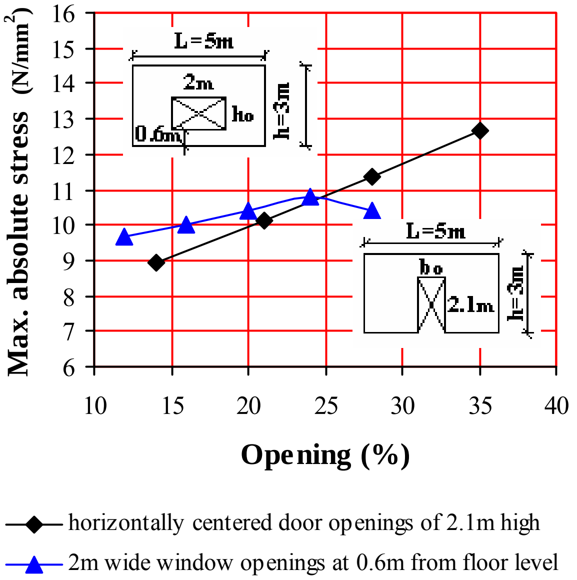
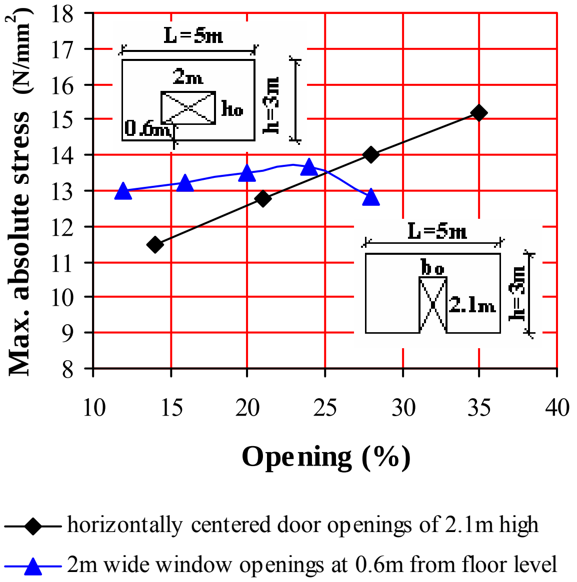


Figure 35 Maximum principal stress in shear walls with 2 m × 1.2 m window openings at different vertical locations

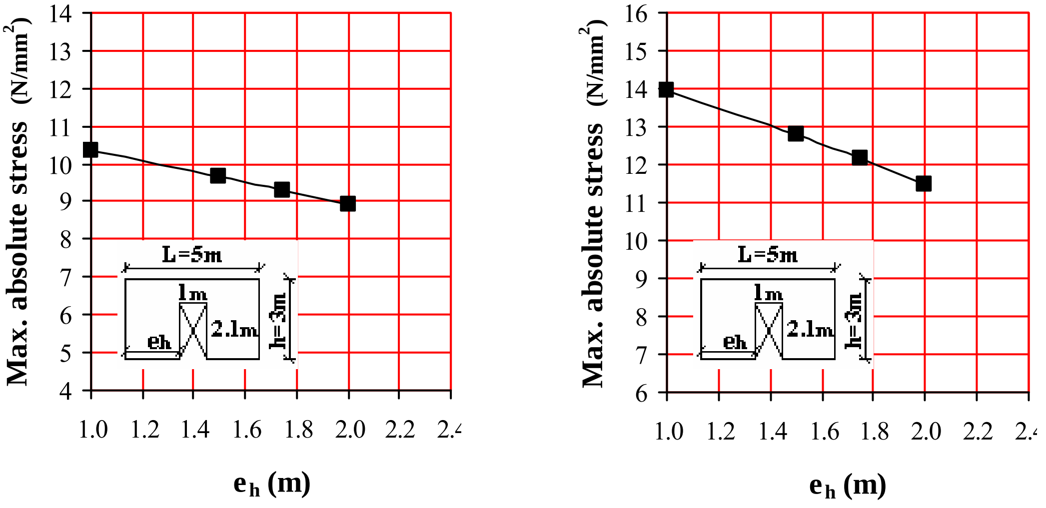


(a) 6-story buildings



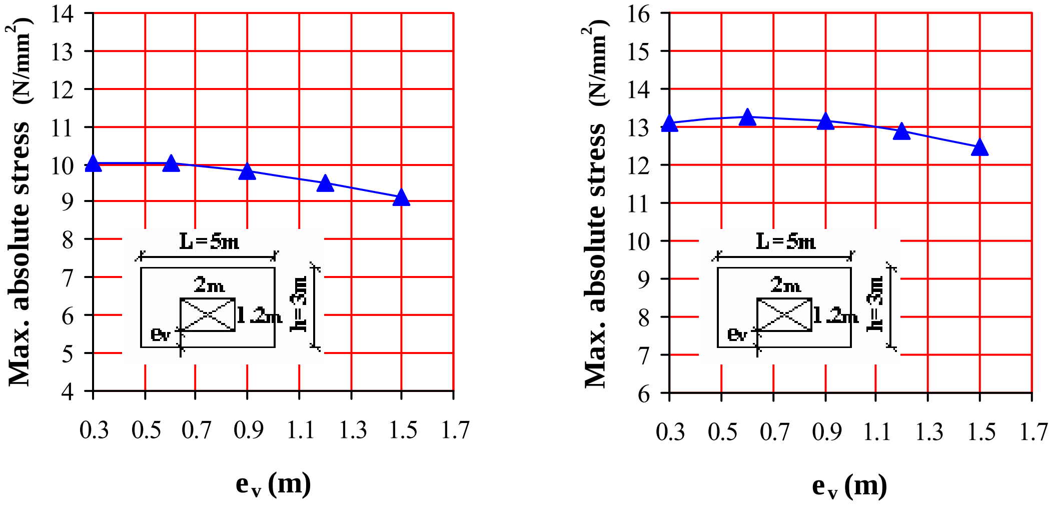
(b) 12-story buildings

Figure 36 Maximum absolute principal stress at top corner of opening at 2nd floor in shear walls with different size of doors and windows



(a) 6-story buildings (b) 12-story buildings

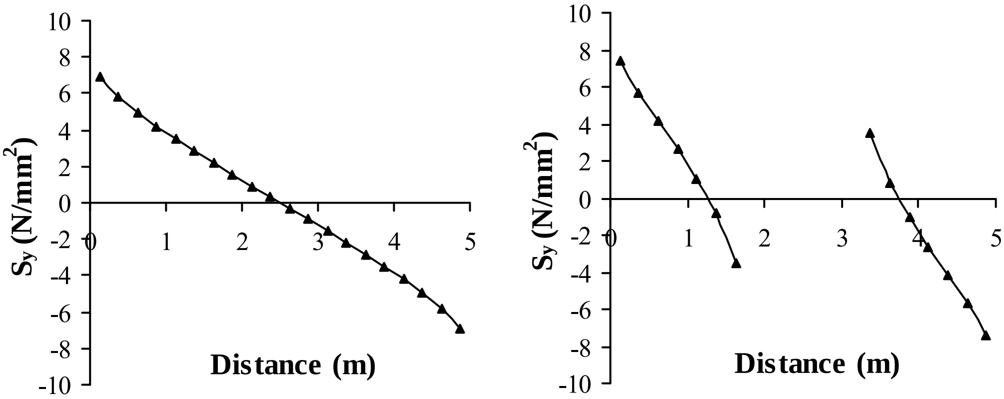
Figure 37 Maximum absolute principal stress at top corner of door at 2nd floor in shear walls with 1 m × 2.1 m doors at different horizontal locations



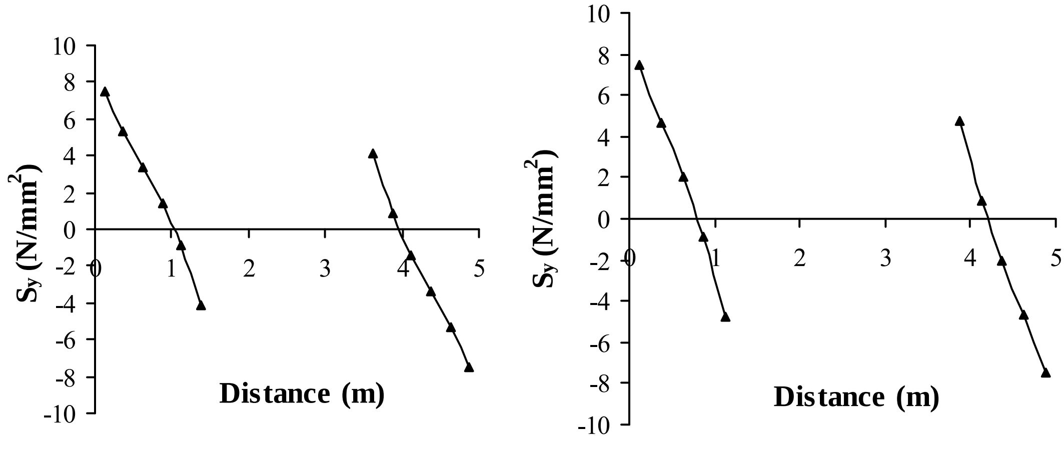
(a) 6-story buildings (b) 12-story buildings

Figure 38 Maximum absolute principal stress at top corner of window at 2nd floor in shear walls with 2 m × 1.2 m windows at different vertical locations

Vertical stresses (S_y) at opening level at 1st floor of shear walls are compared in Figures 39 to 46 to evaluate the behavior of shear wall with and without openings. The pattern of vertical stress diagrams in the shear wall with horizontally centered door openings (Figures 39 and 40) are nearly as coupled wall system in both 6- and 12-story buildings. However, it is closer to two independent shear walls when the opening area of door is 35% in 6-story building.

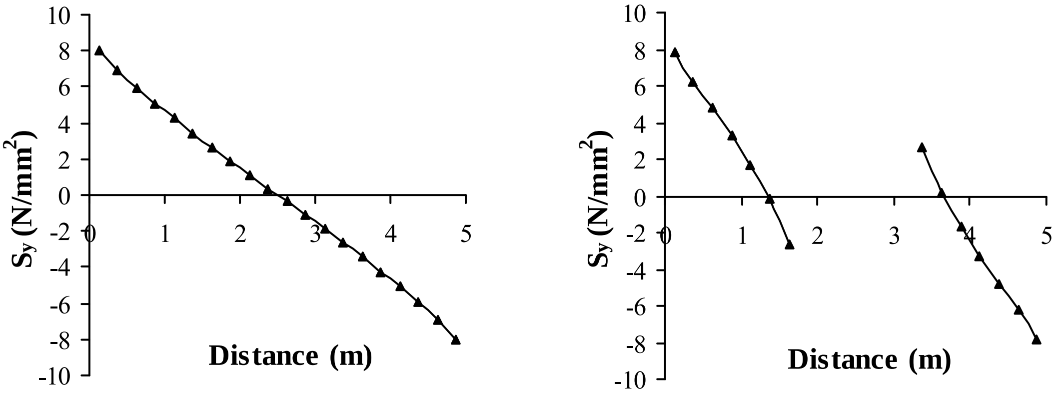


(a) Wall without opening (b) Wall with 1.5 m ×2.1 m (21%) door



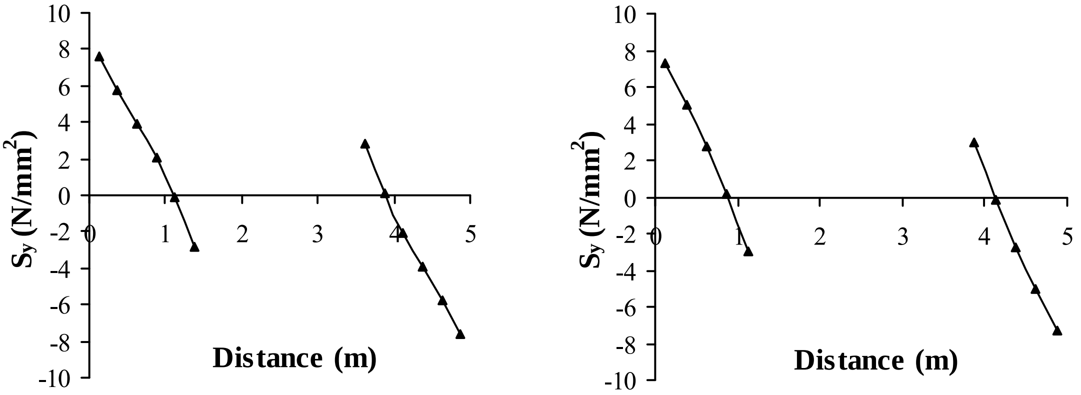
(c) Wall with 2 m ×2.1 m (28%) door (d) Wall with 2.5 m ×2.1 m (35%) door

Figure 39 Vertical stress diagrams in solid shear wall and shear walls with horizontally centered door openings, in 6-story buildings



(a) Wall without opening

(b) Wall with 1.5 m ×2.1 m (21%) door

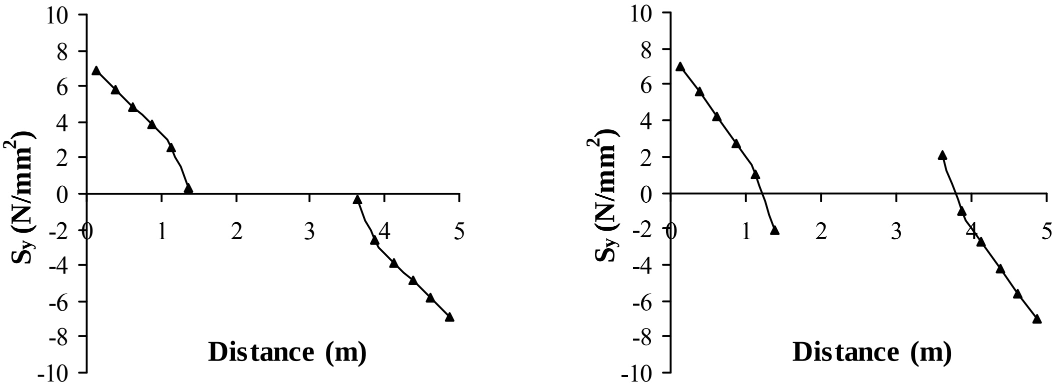


(c) Wall with 2 m ×2.1 m (28%) door

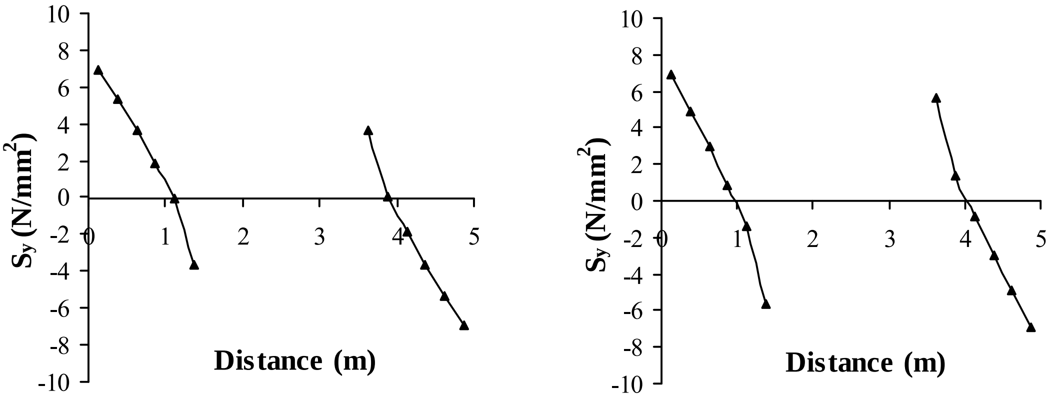
(d) Wall with 2.5 m ×2.1 m (35%) door

Figure 40 Vertical stress diagrams in solid shear wall and shear walls with horizontally centered door openings, in 12-story buildings

Similarly, Figure 42a shows that the behavior of shear wall with window openings of size 2 m × 0.9 m (12%) at 0.6 m from floor level, is closer to single composite shear wall in 12-story building whereas it behaves as coupled wall system in 6-story building (Figure 41a). The behavior changed to coupled wall system when the windows sizes at same location, are increased in vertical direction (Figures 41 and 42). It is closer to two independent shear walls when the window opening is 28% (Figures 41d and 42d). However, the behavior of wall with same size of doors is still similar to coupled wall system. It reveals that the vertical location of opening has significant effect on behavior of shear wall when the opening area is larger than 20%.

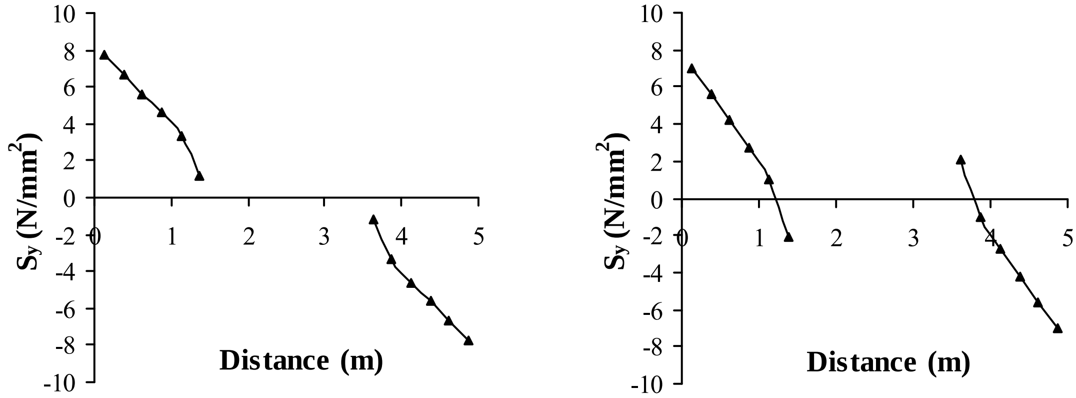


(a) Wall with 2 m ×0.9 m (12%) window (b) Wall with 2 m ×1.5 m (20%) window

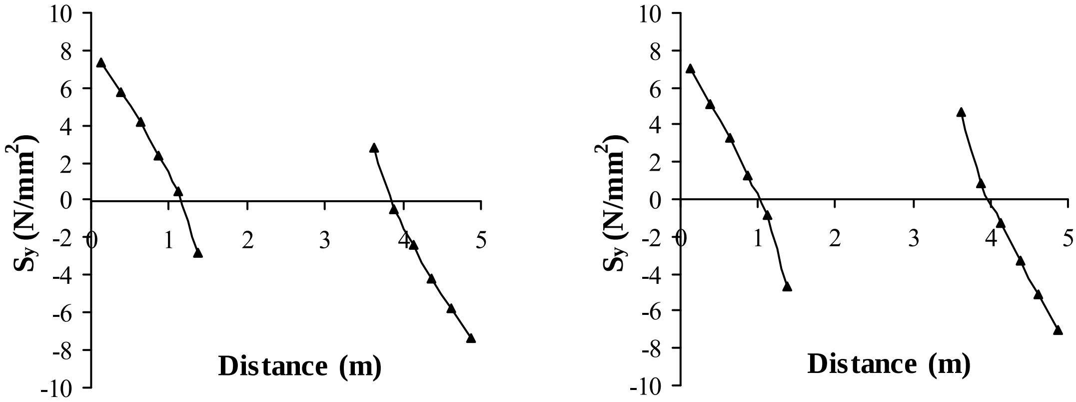


(c) Wall with 2 m ×1.8 m (24%) window (d) Wall with 2 m ×2.1 m (28%) window

Figure 41 Vertical stress diagrams in shear walls with window openings at 0.6 m from floor level, in 6-story buildings



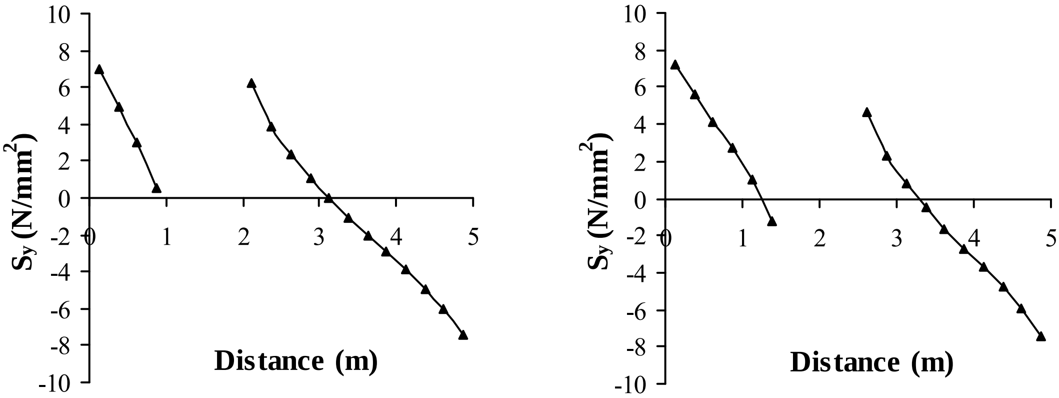
(a) Wall with 2 m ×0.9 m (12%) window (b) Wall with 2 m ×1.5 m (20%) window



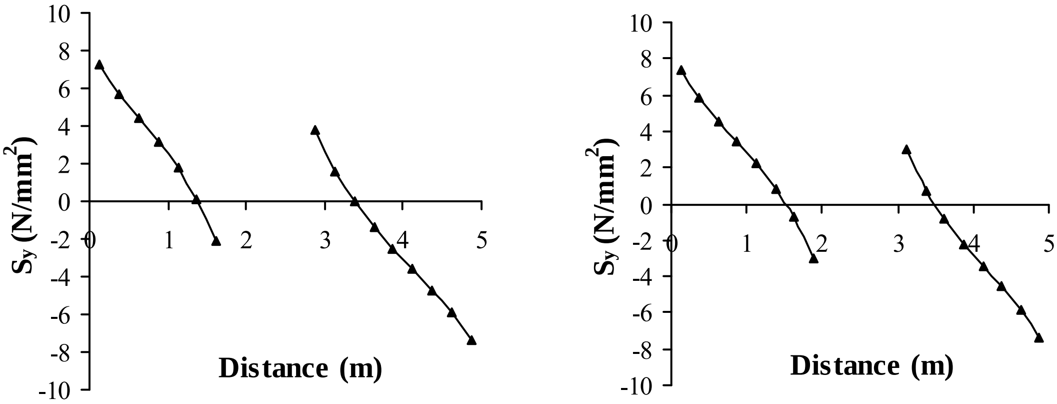
(c) Wall with 2 m ×1.8 m (24%) window (d) Wall with 2 m ×2.1 m (28%) window

Figure 42 Vertical stress diagrams in shear walls with window openings at 0.6 m from floor level, in 12-story buildings

Figures 43 and 44 reveal that shear wall with door openings of 14% at different horizontal locations (e_h) from center of vertical boundary element in shear wall to the edge of the openings behaves as coupled wall system for all the structures. Thus, the effect of horizontal locations of the door openings can be neglected on behavior of shear walls.

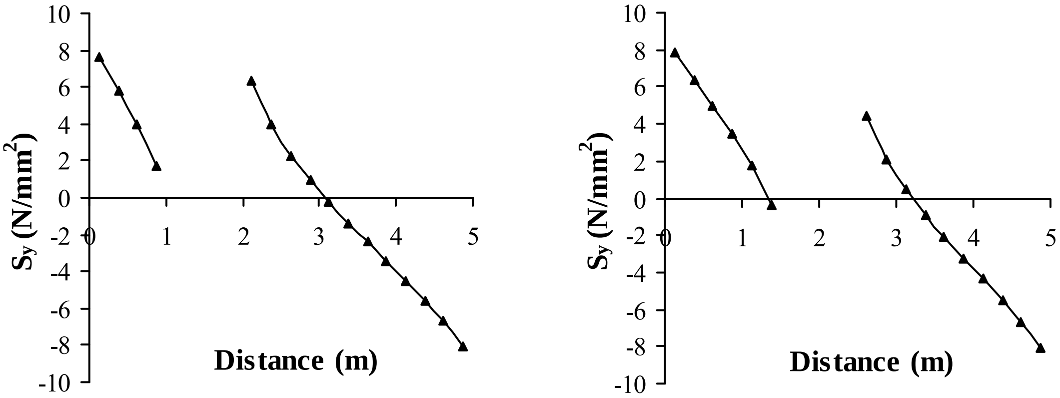


(a) Door at e_h = 1 m (b) Door at e_h = 1.5 m



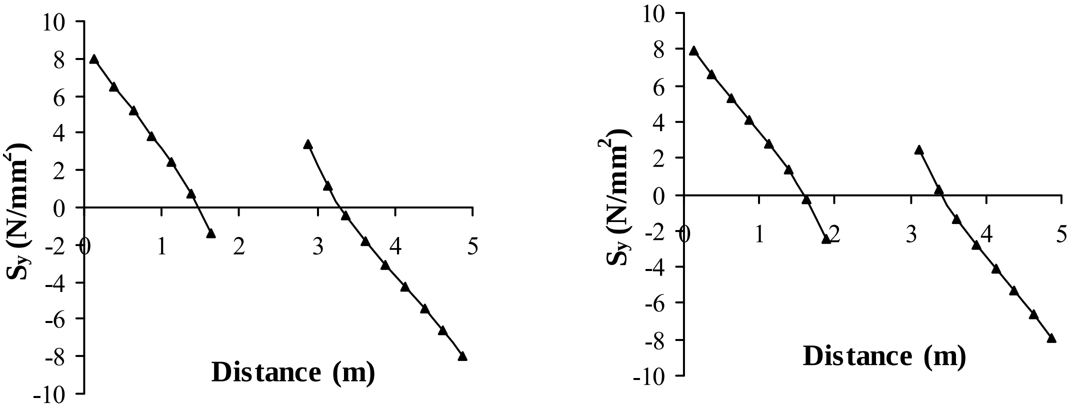
(c) Door at e_h = 1.75 m (d) Door at e_h = 2 m

Figure 43 Vertical stress diagrams in shear walls with door openings of 1 m × 2.1 m (14%) at different horizontal locations, in 6-story buildings



(a) Door at e_h = 1 m

(b) Door at e_h = 1.5 m

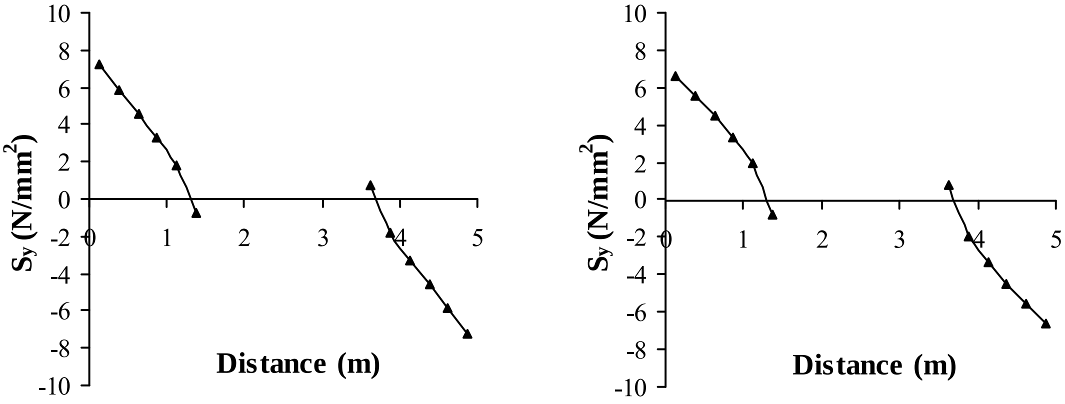


c) Door at e_h = 1.75 m

(d) Door at e_h = 2 m

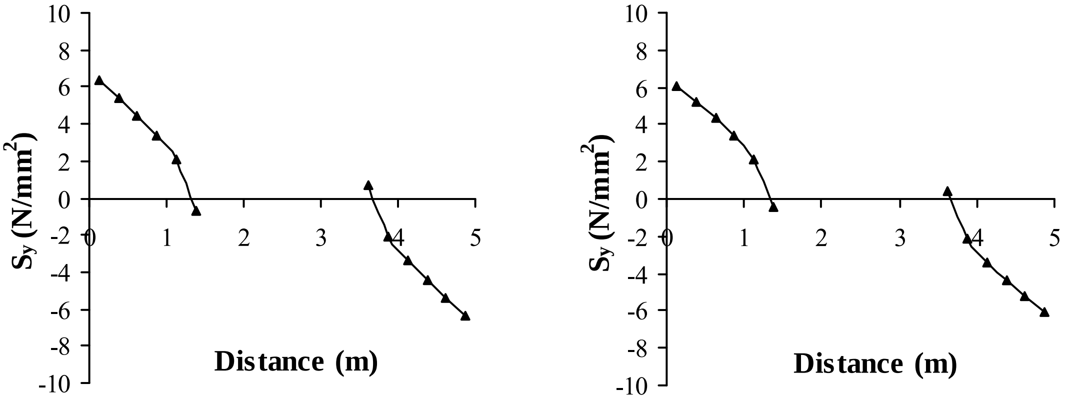
Figure 44 Vertical stress diagrams in shear walls with door openings of 1 m × 2.1 m (14%) at different horizontal locations, in 12-story buildings

Figures 45 and 46 reveal that shear walls with window openings of 16% at different vertical distances from floor level have nearly the same behavior as coupled wall system in both 6- and 12-story buildings. Thus, the effect of vertical locations of window openings of 16% is negligible on behavior of shear wall in frame-shear wall structures.



(a) Window at e_v = 0.3 m

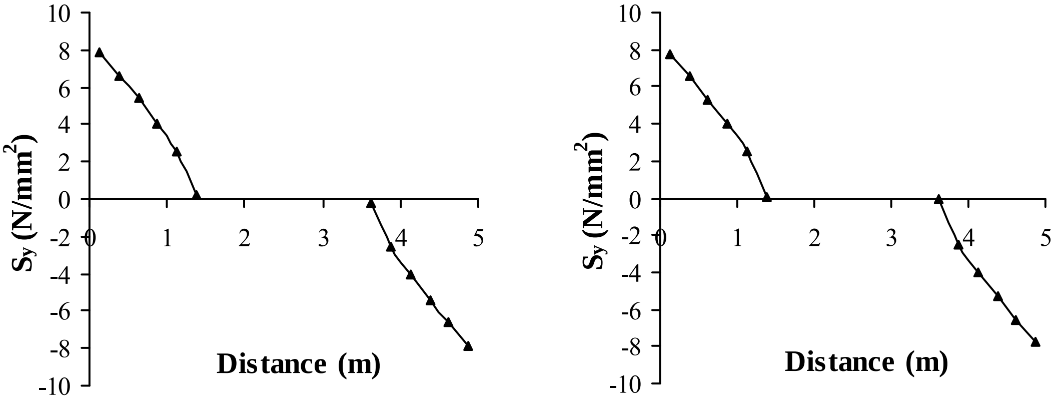
(b) Window at e_v = 0.6 m



(c) Window at e_v = 1.2 m

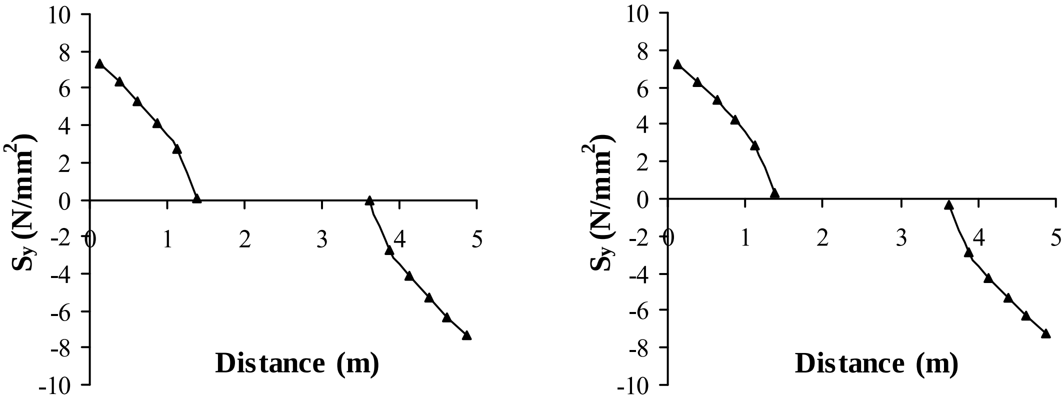
(d) Window at e_v = 1.5 m

Figure 45 Vertical stress diagrams in shear walls with window openings of 2 m × 1.2 m (16%) at different vertical locations, in 6-story buildings



(a) Window at e_v = 0.3 m

(b) Window at e_v = 0.6 m



(c) Window at e_v = 1.2 m

(d) Window at e_v = 1.5 m

Figure 46 Vertical stress diagrams in shear walls with window openings of 2 m × 1.2 m (16%) at different vertical locations, in 12-story buildings

CONCLUSION AND RECOMMENDATIONS

Conclusion

From the above results and discussions, following conclusions can be drawn:

1. It is advisable to limit the simplified method to calculate the stiffness of shear walls with maximum opening area of 10% at the base of shear wall, regardless of the aspect ratio of openings. Thus, the simplified method overestimates the stiffness of wall by less than or equals to 20%. On the other hand, it is beneficial to use FEM for earthquake resistant design of building with shear walls having opening area larger than 10% of the wall area.
2. The stiffness as well as response of frame-shear wall structure is more affected by the size of openings than their locations in the shear walls with opening area $\leq 20\%$. However, it is significantly affected by the opening locations in shear walls with opening area >20%.
3. Coupling ratio (CR) of shear wall decreases with an increase in openings sizes. However, it increased with an increase in vertical distance 'e_v' from floor level to the bottom of window.
4. The behavior of shear wall tends to change from coupled wall system to two independent walls as the opening area is increased. With the same rate of an increment in opening area, the rate of decrease in interaction between walls with an increase in width of doors has been found higher than that with an increase in height of windows with a constant distance between the floor level and the bottom of the window.
5. In the case of maximum absolute principal stress at opening corner in the shear walls, the results indicate that effects of horizontal locations of door openings of 14% is more sensitive than that of the vertical locations of window openings of 16%. It can be concluded that it is better to locate the door openings at center of shear wall. Therefore, maximum principal stress in shear wall as well as maximum absolute principal stress at opening corner is minimized.

Recommendations

Due to the time limitations for the study, different assumptions and limitations have been adopted for simplicity in modeling the proposed structures. In reality, it might affect on results. Thus, all factors which may influence on the behavior of the structures should be considered in the modeling. For the further study, to obtain the real responses of the structures, the following recommendations are made:

1. Since the study was performed for only one type of shear wall, the further investigations should be made for different types of shear walls.
2. For the present study, the analyses were performed for the symmetrical buildings with shear walls at central frames. The further investigations should be made by locating the shear walls at exterior frames.
3. Further investigations should be done for shear walls with different aspect ratio (h/L), in frame-shear wall structures.
4. A flexible foundation will affect the overall stability of the structure by reducing the effective lateral stiffness. So the soil structure interaction should be considered in further study.
5. Shear wall structure have been shown to perform well in earthquakes, for which ductility becomes an important consideration. Thus, the further study should be made considering geometric and material non-linear behavior of the members concerned.

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APPENDICES

Appendix A

Indian standard, IS codes

1. Design seismic base shear (V_B)

The total design seismic base shear (V_B) along any principal direction is given by:

$$V_B = A_h W$$

where,

W = Seismic weight of the whole building, is sum of the seismic weight of all floors.

A_h = Design horizontal acceleration spectrum value which is determined by using the following expression:

$$A_h = \frac{ZIS_a}{2Rg}$$

In which,

Z = Zone factor, is given in Table A1 for maximum considered earthquake and service life of structure in a zone. The factor 2 in the denominator of Z is used to reduce the maximum considered earthquake zone factor to the factor for design basis earthquake.

I = Importance factor, depending upon the functional use of the structures, historical value, economic importance etc. The minimum values of importance factors are given in Appendix Table A2.

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations, is given in Appendix Table A3.

$\frac{S_a}{g}$ = Average response acceleration coefficient for rock and soil sites, is given in Appendix Figure A1 for different soil condition, assuming 5% damping for concrete structure, based on appropriate natural period T_a.

Appendix Table A1 Zone factor, Z

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

Appendix Table A2 Importance factor, I

S.No.	**Structure type**	**Importance factor (I)**
1	Important services and community buildings, such as hospitals, schools, monumental structures, emergency buildings like telephone exchange, television stations, radio stations, fire station buildings, large community halls like cinemas, assembly halls and subway stations, power stations	1.5
2	All other buildings	1.0

Appendix Table A3 Response reduction factor, R for building systems

S.No.	**Lateral load resisting system**	**R**
1	Ordinary RC moment-resisting frame (OMRF)	3.0
2	Special RC moment-resisting frame (SMRF)	5.0
3	Ordinary reinforced concrete shear walls	3.0
4	Ductile shear walls	4.0
5	Ordinary shear wall with OMRF	3.0
6	Ordinary shear wall with SMRF	4.0
7	Ductile shear wall with OMRF	4.5
8	Ductile shear wall with SMRF	5.0

2. Approximate fundamental time period

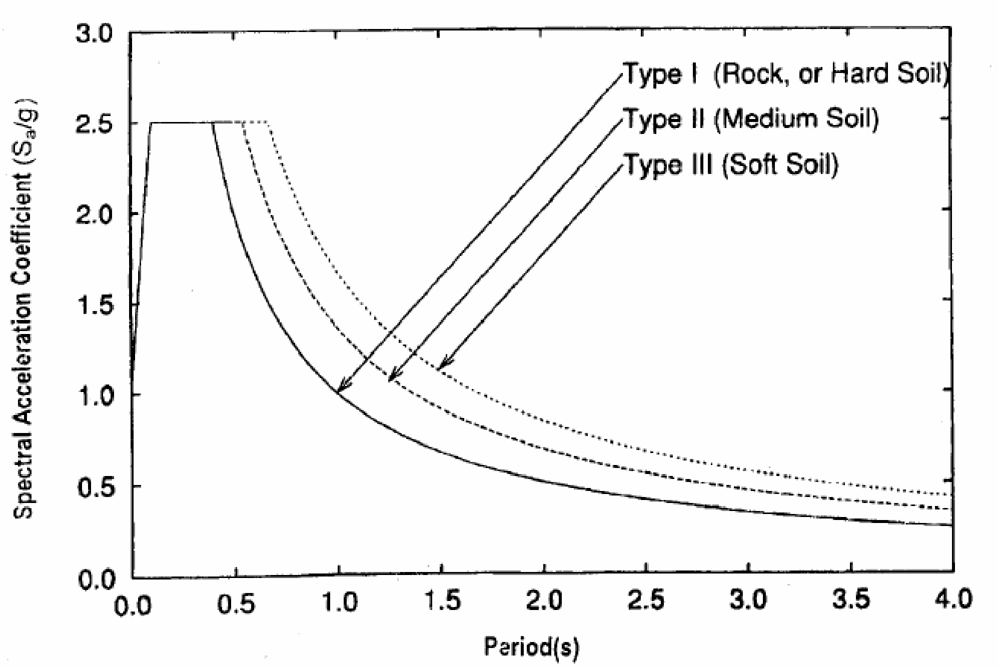
The approximate fundamental natural period of vibration (T_a) in seconds of moment resisting frame structure without brick infill panels is estimated by the empirical relation:

$$T_a = 0.075H^{0.75}$$

Similarly, for all other buildings, it is given by empirical expression:

$$T_a = \frac{0.09H}{\sqrt{d}}$$

Where, H is height of building in meter and d is base dimension of the building at plinth level in meter along the considered direction of lateral force.



Appendix Figure A1 Response spectra for rock and soil sites for 5 percent damping

Source: IS 1893 (Part1): 2002

3. Design imposed load for earthquake load calculation

The earthquake load is calculated for full dead load plus the percentage of imposed load as given in Appendix Table 4.

Appendix Table A4 Percentage of imposed load to be considered in seismic weight calculation

S.No.	**Imposed uniformly distributed floor loads (kN/m^2)**	**Percentage of imposed load to be considered**
1	Upto and including 3.0	25
2	Above 3.0	50

Imposed load on roof is not considered for calculating the design seismic forces of the building.

4. Load combinations to be considered in earthquake resistant design of structures

In the limit state design of reinforced concrete structures with earthquake loads, the following load combinations are considered:

1) 1.5 (DL + IL)
2) 1.2 (DL + IL ± EL)
3) 1.5 (DL + IL ± EL)
4) 0.9 DL ± 1.5 EL

Appendix B

Stiffnesses/rigidities of walls using simplified method

1. Calculation of stiffness/rigidity of solid wall using the simplified method

Height of solid wall (h) = 3 m

Length of solid wall (L) = 5 m

Modulus of elasticity (E) = 2.5×10^7 kN/m^2

Thickness of wall (t) = 250 mm = 0.25 m

From Eq. (8),

$$\Delta_{\text{solid wall}} = \frac{4\frac{h^3}{L^3} + 2.81\frac{h}{L}}{Et} = \frac{4\frac{3^3}{5^3} + 2.81\frac{3}{5}}{Et} = \frac{2.55}{Et}$$

Substituting E = 2.5×10^7 kN/m^2 and t = 0.25 m, rigidity of solid wall is given by:

$$R_{\text{solid wall}} = \frac{1}{\Delta_{\text{solid wall}}} = \frac{Et}{2.55} = 2450980.4\,\text{kN/m} = 2450.98\,\text{kN/mm}$$

2. Calculation of stiffness/rigidity of wall with opening of size 1 m × 2.1 m using the simplified method

Height of solid wall (h) = 3 m

Length of solid wall (L) = 5 m

Height of solid strip (h_o) = 2.1 m

Length of pier (l) = 2 m

Modulus of elasticity (E) = 2.5×10^7 kN/m^2

Thickness of wall (t) = 250 mm = 0.25 m

Using Eq. (19),

$$\Delta_{\text{solid strip}} = \frac{\frac{h^3}{L^3} + 2.81\frac{h}{L}}{Et} = \frac{\frac{2.1^3}{5^3} + 2.81\times\frac{2.1}{5}}{Et} = \frac{1.254}{Et}$$

Using Eq. (10)

$$\Delta_{\text{pier}} = \frac{\frac{h_o{}^3}{l^3} + 2.81\frac{h_o}{l}}{\text{Et}} = \frac{\frac{2.1^3}{2^3} + 2.81 \times \frac{2.1}{2}}{\text{Et}} = \frac{4.108}{\text{Et}}$$

Using Eq. (18)

$$\Delta_{\text{piers}} = \frac{1}{\frac{1}{\Delta_{\text{pier1}}} + \frac{1}{\Delta_{\text{pier2}}}} = \frac{4.108}{2\text{Et}} = \frac{2.05}{\text{Et}}$$

$\Delta_{\text{solid wall}} = \frac{2.55}{\text{Et}}$ from above calculation

Therefore, using Eq. (20)

$$\Delta_{\text{wall}} = \Delta_{\text{solid wall}} - \Delta_{\text{solid strip}} + \Delta_{\text{piers}} = \frac{3.35}{\text{Et}}$$

$$\therefore \text{R}_{\text{wall}} = \frac{1}{\Delta_{\text{wall}}} = \frac{\text{Et}}{3.35} = 1865797.2\ \text{kN/m} = 1865.8\ \text{kN/mm}$$

From Eq. (21)

$$\text{Stiffness ratio} = \frac{\text{R}_{\text{wall}}}{\text{R}_{\text{solid wall}}} = \frac{\Delta_{\text{solid wall}}}{\Delta_{\text{wall}}} = \frac{2.55}{3.35} = 0.76$$

R_{wall} from FEM is obtained as 1356.09 kN/mm. Thus, different in stiffnesses from the simplified method and FEM is given by Eq. (22).

$$\therefore \text{Different (\%)} = \left(\frac{\text{R}_{\text{simplified method}}}{\text{R}_{\text{FEM}}} - 1 \right) \times 100 = 37.6\%$$

Appendix Table B1 Stiffnesses/rigidities of walls with 1 m wide opening

h_o/h	Simplified method		Finite element method		Different (%)
	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	
0	2450.98	1.0	2405.89	1.0	1.9
0.3	2301.03	0.94	2096.08	0.87	9.8
0.4	2222.00	0.91	1947.88	0.81	14.1
0.5	2122.47	0.87	1772.51	0.74	19.7
0.6	2002.73	0.82	1575.56	0.65	27.1
0.7	1865.80	0.76	1356.09	0.56	37.6

Appendix Table B2 Stiffnesses/rigidities of walls with 1.5 m wide opening

h_o/h	Simplified method		Finite element method		Different (%)
	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	
0	2450.98	1.0	2405.89	1.0	1.9
0.3	2209.30	0.90	1927.45	0.80	14.6
0.4	2092.81	0.85	1732.13	0.72	20.8
0.5	1953.64	0.80	1515.98	0.63	28.9
0.6	1795.47	0.73	1285.71	0.53	39.6
0.7	1625.19	0.66	1045.77	0.43	55.4

Appendix Table B3 Stiffnesses/rigidities of walls with 2 m wide opening

h_o/h	Simplified method		Finite element method		Different (%)
	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	
0	2450.98	1.0	2405.89	1.0	1.9
0.3	2090.74	0.85	1734.17	0.72	20.6
0.4	1927.96	0.79	1498.47	0.62	28.7
0.5	1743.38	0.71	1254.62	0.52	39.0
0.6	1546.36	0.63	1011.00	0.42	53.0
0.7	1348.52	0.55	775.86	0.32	73.8

Appendix Table B4 Stiffnesses/rigidities of walls with 2.5 m wide opening

h_o/h	Simplified method		Finite element method		Different (%)
	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	R_{wall} (kN/mm)	$R_{wall}/R_{solid\ wall}$ (kN/mm)	
0	2450.98	1.0	2405.89	1.0	1.9
0.3	1931.04	0.79	1512.35	0.63	27.7
0.4	1710.95	0.70	1245.06	0.52	37.4
0.5	1477.08	0.60	992.57	0.41	48.9
0.6	1248.02	0.51	760.87	0.32	64.0
0.7	1037.01	0.42	551.46	0.23	88.1

Appendix Table B5 Stiffnesses/rigidities of walls with 1 m × 1.2 m opening at different vertical locations

	Simplified method		**Finite element method**		
e_v (m)	**R_{wall} (kN/mm)**	**$R_{wall}/R_{solid\ wall}$ (kN/mm)**	**R_{wall} (kN/mm)**	**$R_{wall}/R_{solid\ wall}$ (kN/mm)**	**Different (%)**
0	2222.0	0.91	1947.88	0.81	14.1
0.3	2222.0	0.91	1816.31	0.75	22.3
0.6	2222.0	0.91	1696.81	0.71	31.1
0.9	2222.0	0.91	1593.78	0.66	39.4
1.2	2222.0	0.91	1500.00	0.62	48.1
1.5	2222.0	0.91	1423.50	0.59	56.1

Appendix Table B6 Stiffnesses/rigidities of walls with 2 m × 1.2 m opening at different vertical locations

	Simplified method		**Finite element method**		
e_v (m)	**R_{wall} (kN/mm)**	**$R_{wall}/R_{solid\ wall}$ (kN/mm)**	**R_{wall} (kN/mm)**	**$R_{wall}/R_{solid\ wall}$ (kN/mm)**	**Different (%)**
0	1927.96	0.79	1498.47	0.62	28.7
0.3	1927.96	0.79	1297.82	0.54	48.5
0.6	1927.96	0.79	1133.09	0.47	70.2
0.9	1927.96	0.79	991.01	0.41	94.5

Appendix C

Top displacement and base shear in shear walls

Appendix Table C1 Top displacement and base shear in shear walls with horizontally centered door openings of 2.1 m height

b_o/L	Opening (%)	6-story buildings		12-story buildings	
		Top displacement (mm)	Base shear in shear walls (%)	Top displacement (mm)	Base shear in shear walls (%)
0.0	0	11.07	89.6	40.31	77.7
0.2	14	12.39	85.9	42.03	71.9
0.3	21	13.28	83.8	43.21	68.7
0.4	28	14.42	80.8	44.79	64.6
0.5	35	15.94	76.6	46.78	59.7

Appendix Table C2 Top displacement and base shear in shear walls with 2 m wide window openings at 0.6 m from floor level

h_o/h	Opening (%)	6-story buildings		12-story buildings	
		Top displacement (mm)	Base shear in shear walls (%)	Top displacement (mm)	Base shear in shear walls (%)
0.0	0	11.07	89.6	40.31	77.7
0.3	12	12.16	86.0	41.71	72.0
0.4	16	12.67	84.5	42.39	69.5
0.5	20	13.44	82.3	43.47	66.1
0.6	24	14.69	79.5	45.26	61.9
0.7	28	16.55	76.2	47.89	57.4

Appendix Table C3 Top displacement and base shear in shear walls with 1 m × 2.1 m door openings at different horizontal locations

e_h (m)	Opening (%)	6-story buildings		12-story buildings	
		Top displacement (mm)	Base shear in shear walls (%)	Top displacement (mm)	Base shear in shear walls (%)
1.00	14	12.61	86.0	42.42	72.0
1.50	14	12.44	85.9	42.12	71.9
1.75	14	12.41	85.9	42.05	71.9
2.00	14	12.39	85.9	42.03	71.9

Appendix Table C4 Top displacement and base shear in shear walls with 2 m × 1.2 m window openings at different vertical locations

e_v (m)	Opening (%)	6-story buildings		12-story buildings	
		Top displacement (mm)	Base shear in shear walls (%)	Top displacement (mm)	Base shear in shear walls (%)
0.3	16	12.40	85.6	41.98	71.6
0.6	16	12.67	84.5	42.39	69.5
0.9	16	12.80	83.6	42.60	67.7
1.2	16	12.81	83.1	42.60	66.5
1.5	16	12.68	83.2	42.39	66.5

Appendix D

Coupling ratio of shear walls

Appendix Table D1 Coupling ratio (CR) of shear walls with horizontally centered door openings of 2.1 m height

b_o/L	**Opening (%)**	**Coupling ratio (%)**	
		6-story buildings	**12-story buildings**
0.2	14	98.99	98.11
0.3	21	98.84	97.87
0.4	28	98.64	97.55
0.5	35	98.33	97.12

Appendix Table D2 Coupling ratio (CR) of shear walls with 2 m wide window openings at 0.6 m from floor level

h_o/h	**Opening (%)**	**Coupling ratio (%)**	
		6-story buildings	**12-story buildings**
0.3	12	98.96	98.06
0.4	16	98.93	97.97
0.5	20	98.89	97.88
0.6	24	98.85	97.77
0.7	28	98.79	97.62

Appendix Table D3 Coupling ratio (CR) of shear walls with 1 m × 2.1 m door openings at different horizontal locations

e_h (m)	**Opening (%)**	**Coupling ratio (%)**	
		6-story buildings	**12-story buildings**
1.00	14	99.00	98.09
1.50	14	99.00	98.11
1.75	14	98.99	98.11
2.00	14	98.99	98.11

Appendix Table D4 Coupling ratio (CR) of shear walls with 2 m × 1.2 m window openings at different vertical locations

e_v (m)	Opening (%)	Coupling ratio (%)	
		6-story buildings	**12-story buildings**
0.3	16	98.87	97.94
0.6	16	98.93	97.97
0.9	16	99.05	98.10
1.2	16	99.16	98.27
1.5	16	99.24	98.42

Appendix E

Maximum absolute principal stress in shear walls

Appendix Table E1 Maximum absolute principal stress at opening corner of shear wall with horizontally centered door openings of 2.1 m height

bo/L	Opening (%)	Maximum absolute principal stress at opening corner (N/mm^2)		Location of opening corner in shear wall
		6-story buildings	**12-story buildings**	
0.2	14	8.93	11.47	Top of door at 2nd floor
0.3	21	10.14	12.78	Top of door at 2nd floor
0.4	28	11.35	14.00	Top of door at 2nd floor
0.5	35	12.65	15.18	Top of door at 2nd floor

Appendix Table E2 Maximum absolute principal stress at opening corner of shear wall with 2 m wide window openings at 0.6 m from floor level

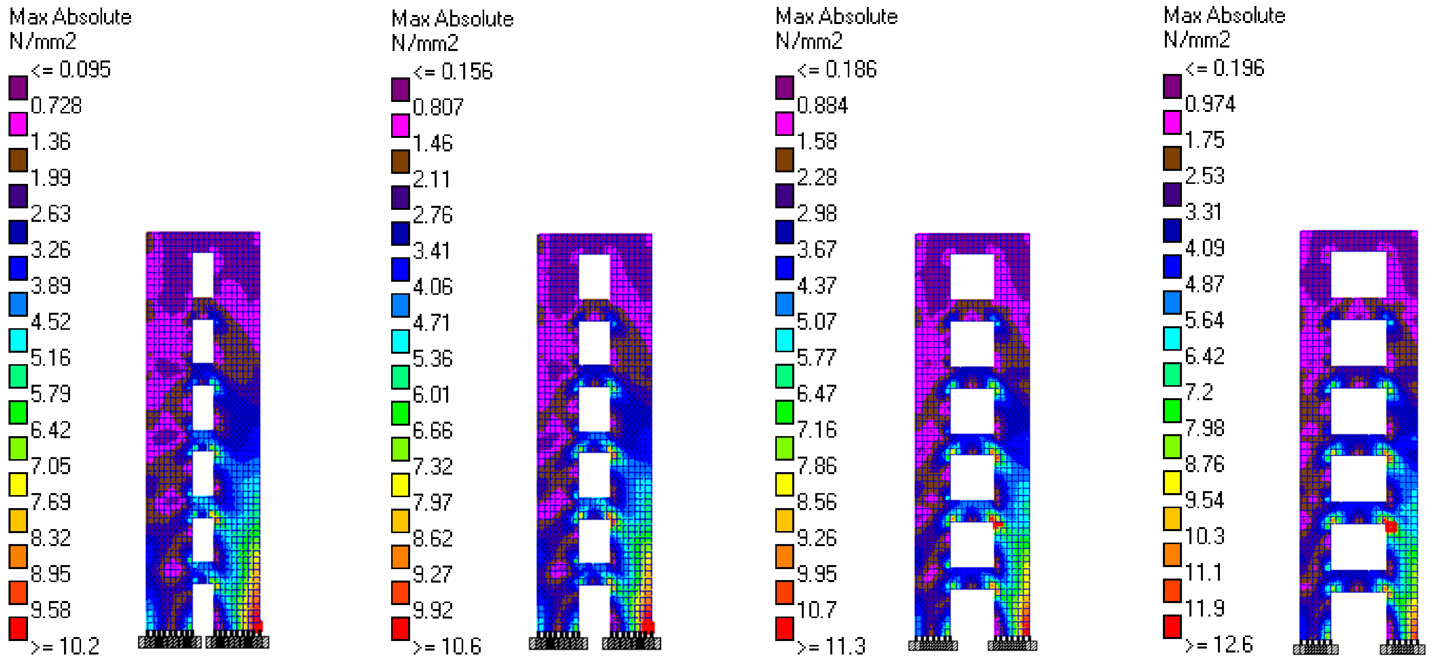
ho/h	Opening (%)	Maximum absolute principal stress at opening corner (N/mm^2)		Location of opening corner in shear wall
		6-story buildings	**12-story buildings**	
0.3	12	9.69	13.00	Top of window at 2nd floor
0.4	16	10.02	13.24	Top of window at 2nd floor
0.5	20	10.43	13.52	Top of window at 2nd floor
0.6	24	10.80	13.65	Top of window at 2nd floor
0.7	28	10.42	12.84	Top of window at 2nd floor

Appendix Table E3 Maximum absolute principal stress at opening corner of shear wall with 1 m × 2.1 m door openings at different horizontal locations ‘e_h’

e_h (m)	Opening (%)	Maximum absolute principal stress at opening corner (N/mm^2)		Location of opening corner in shear wall
		6-story buildings	12-story buildings	
1.00	14	10.36	13.96	Top of door at 2nd floor
1.50	14	9.66	12.79	Top of door at 2nd floor
1.75	14	9.31	12.15	Top of door at 2nd floor
2.00	14	8.93	11.47	Top of door at 2nd floor

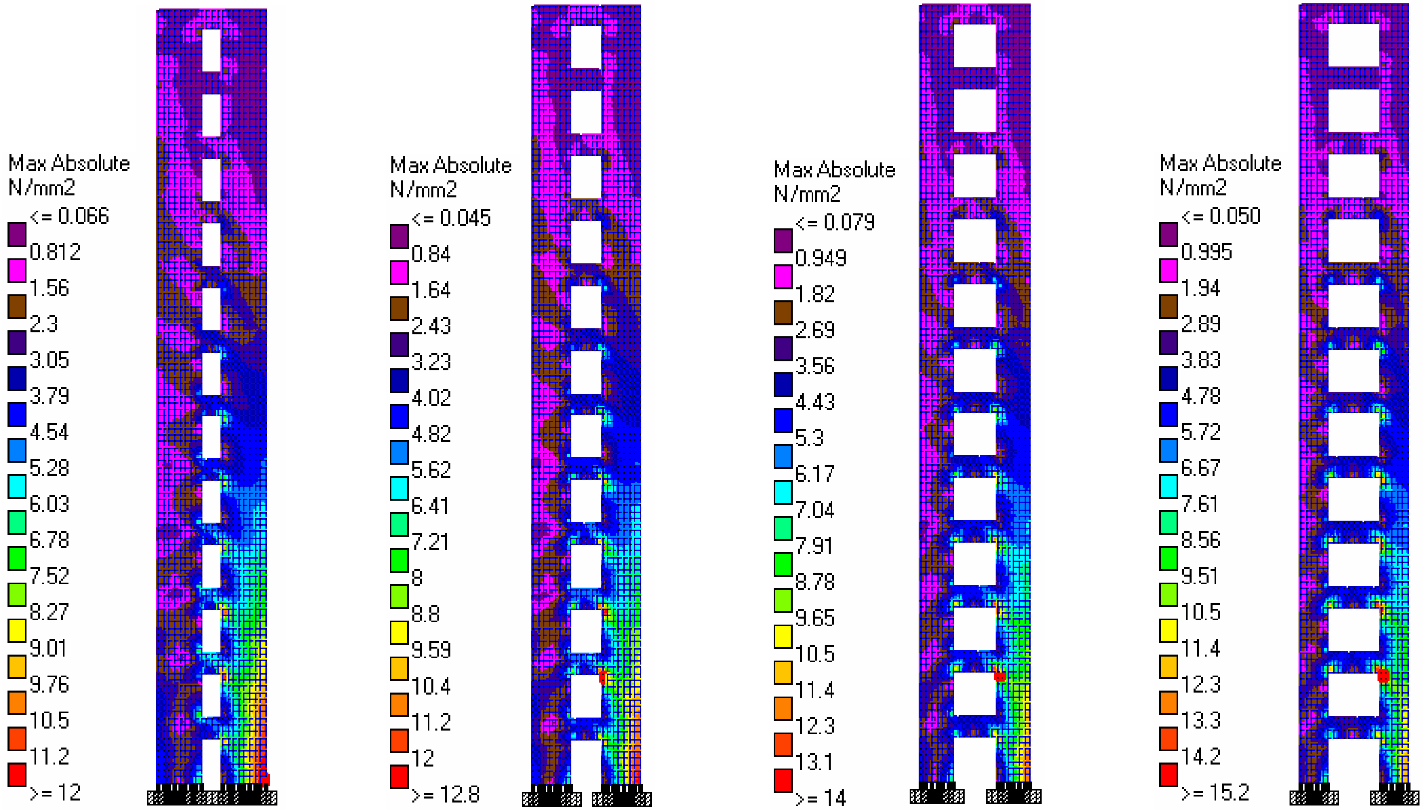
Appendix Table E4 Maximum absolute principal stress at opening corner of shear wall with 2 m × 1.2 m window openings at different vertical locations ‘e_v’

e_v (m)	Opening (%)	Maximum absolute principal stress at opening corner (N/mm^2)		Location of stress at opening corner
		6-story buildings	12-story buildings	
0.3	16	10.03	13.11	Top of window at 2nd floor
0.6	16	10.02	13.24	Top of window at 2nd floor
0.9	16	9.83	13.14	Top of window at 2nd floor
1.2	16	9.53	12.87	Top of window at 2nd floor
1.5	16	9.12	12.49	Top of window at 2nd floor



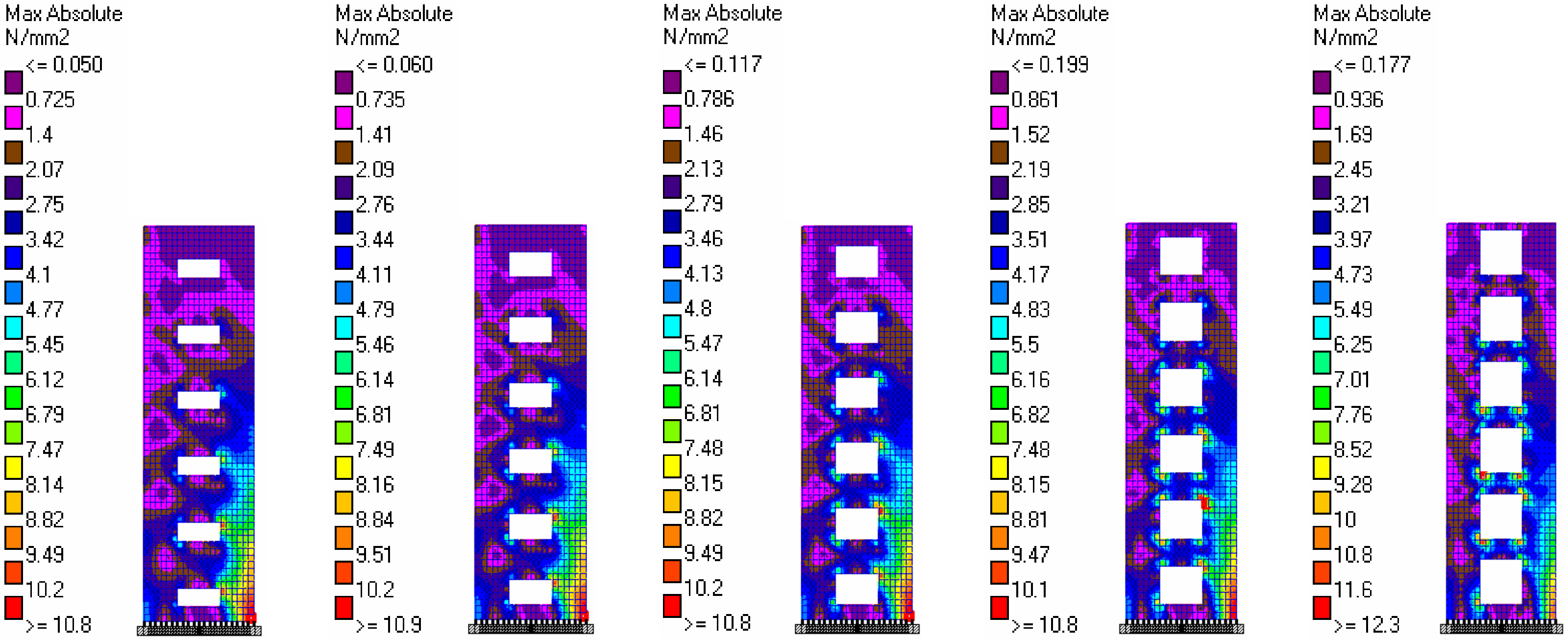
(a) 14% opening (b) 21% openings (c) 28% opening (d) 35% opening

Appendix Figure E1 Maximum absolute principal stress diagrams in shear walls with horizontally centered door openings, in 6-story buildings



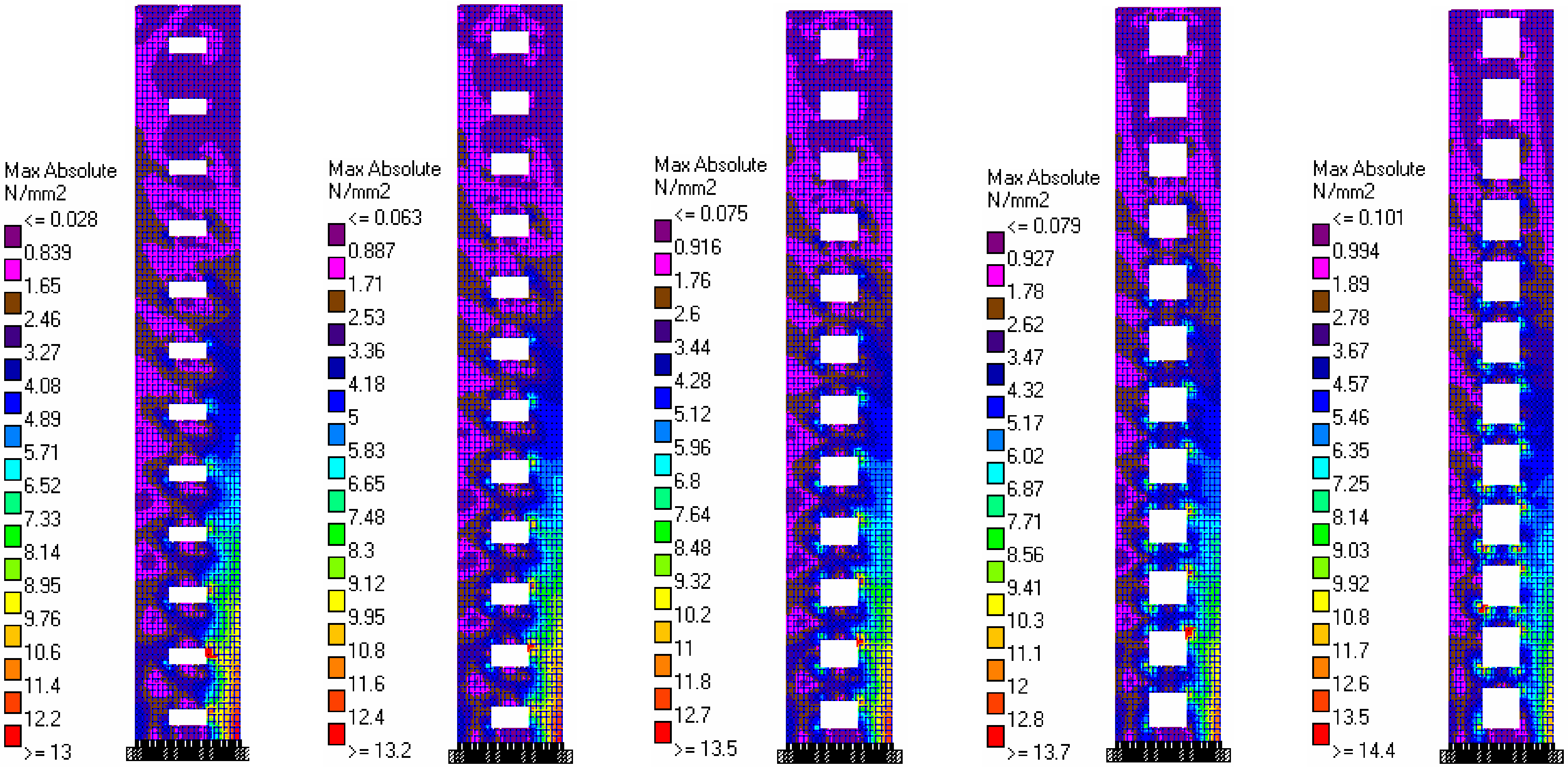
(a) 14% opening (b) 21% opening (c) 28% opening (d) 35% opening

Appendix Figure E2 Maximum absolute principal stress diagrams in shear walls with horizontally centered door openings, in 12-story buildings



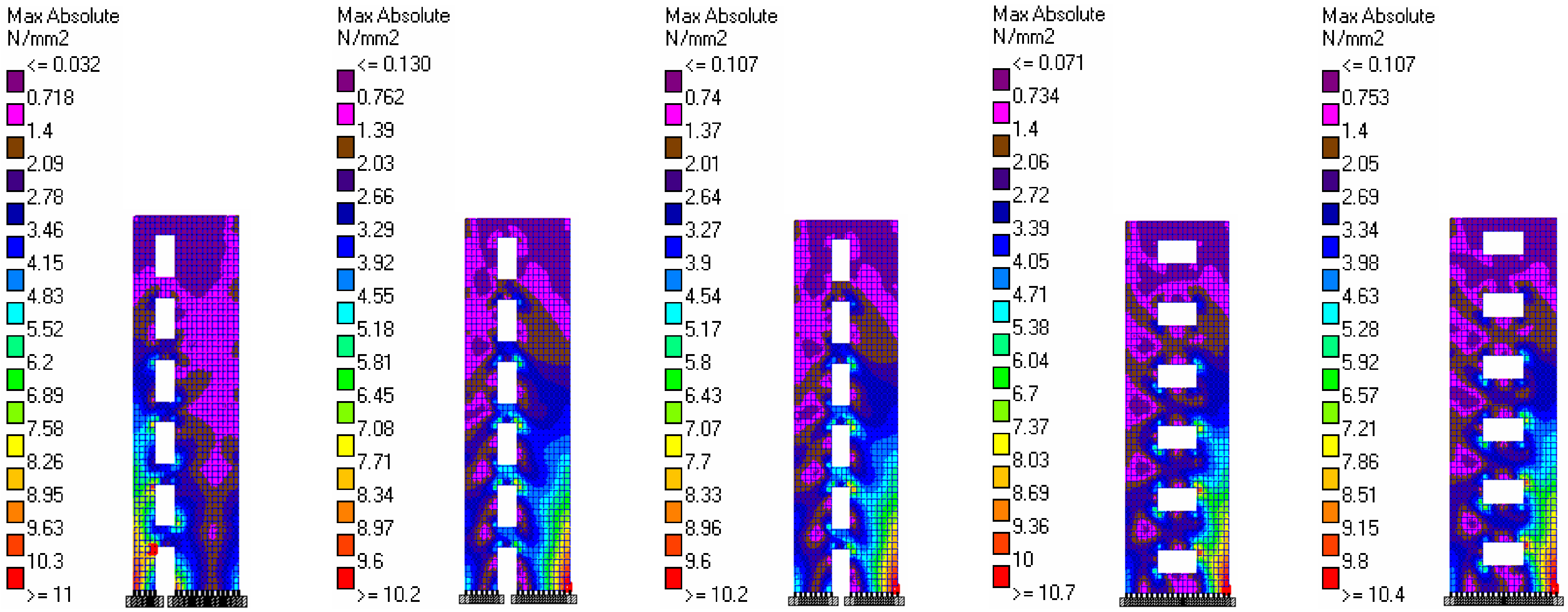
(a) 12% opening (b) 16% opening (c) 20% opening (d) 24% opening (e) 28% opening

Appendix Figure E3 Maximum absolute principal stress diagrams in shear walls with window openings at 0.6 m from floor level, in 6-story buildings



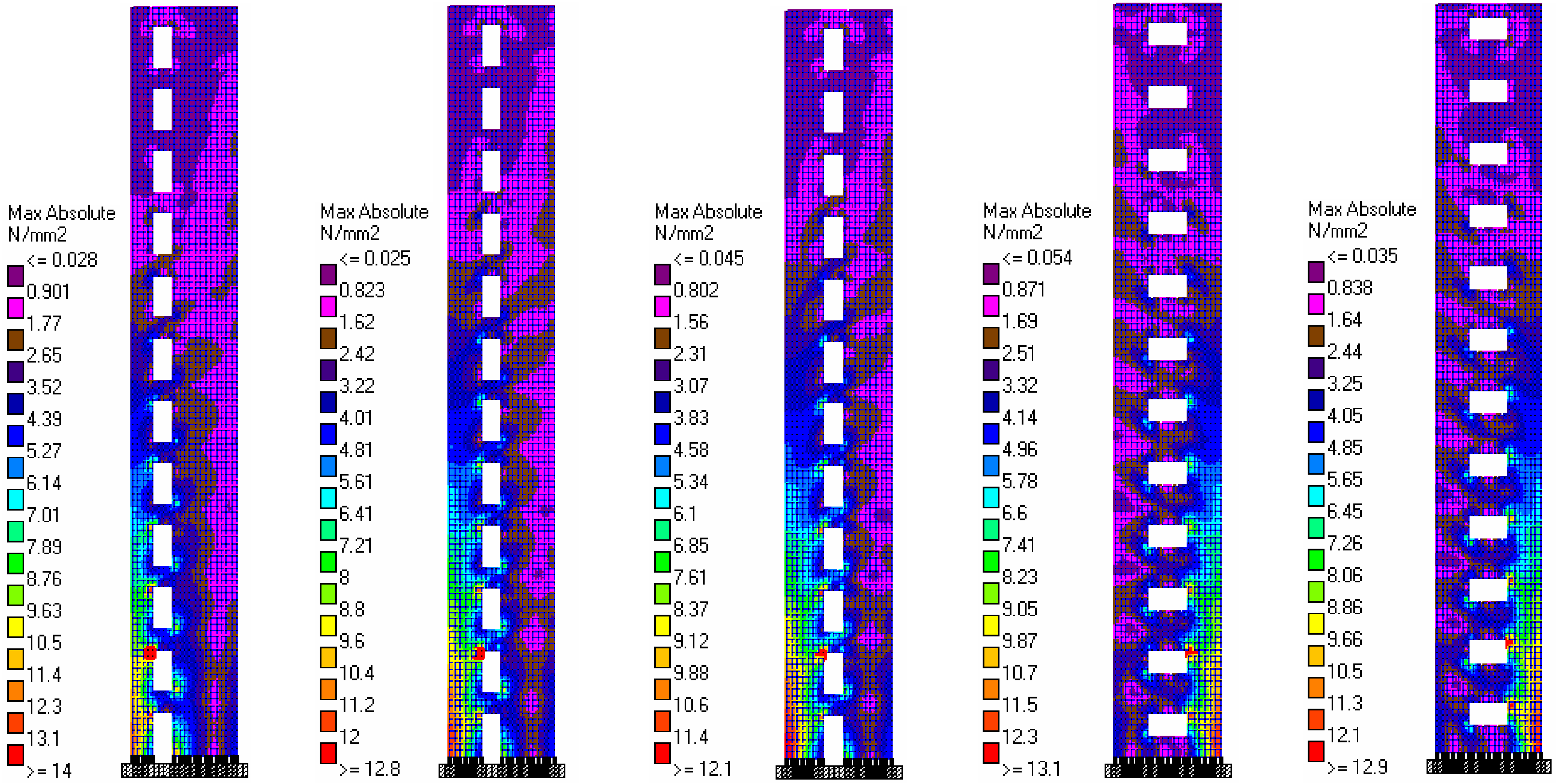
(a) 12% opening (b) 16% opening (c) 20% opening (d) 24% opening (e) 28% opening

Appendix Figure E4 Maximum absolute principal stress diagrams in shear walls with window openings at 0.6 m from floor level, in 12-story buildings



(a) door at e_h =1 m (b) door at e_h =1.5 m (c) door at e_h = 1.75 m (d) window at e_v = 0.9 m (e) window at e_v = 1.2 m

Appendix Figure E5 Maximum absolute principal stress diagrams in shear walls with different openings locations, in 6-story buildings

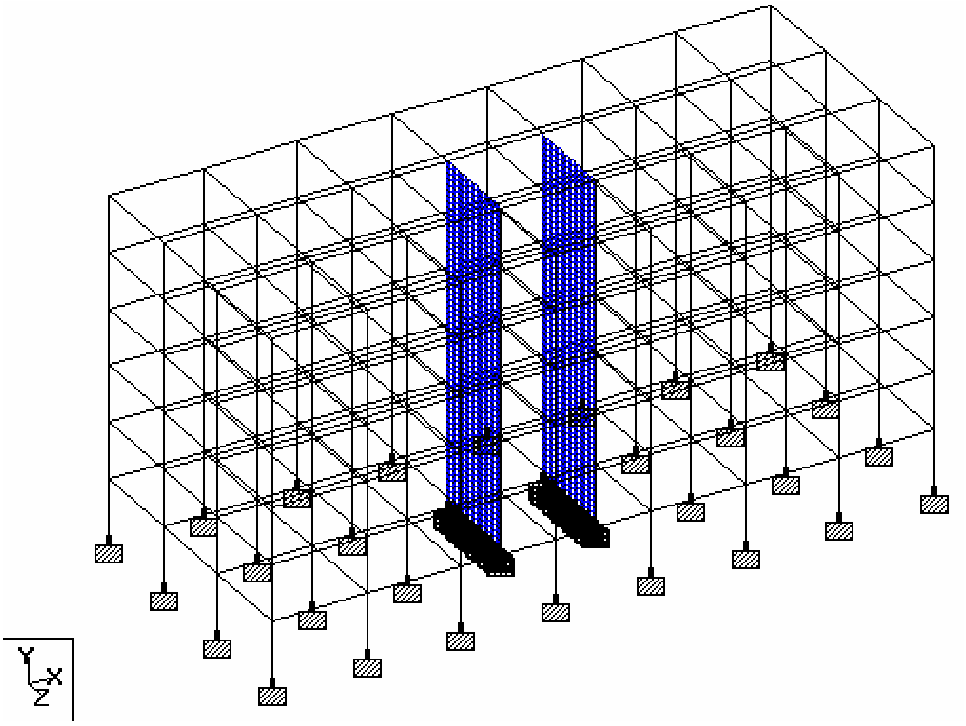


(a) door at e_h =1 m (b) door at e_h =1.5 m (c) door at e_h = 1.75 d) window at e_v = 0.9 m (e) window at e_v = 1.2 m

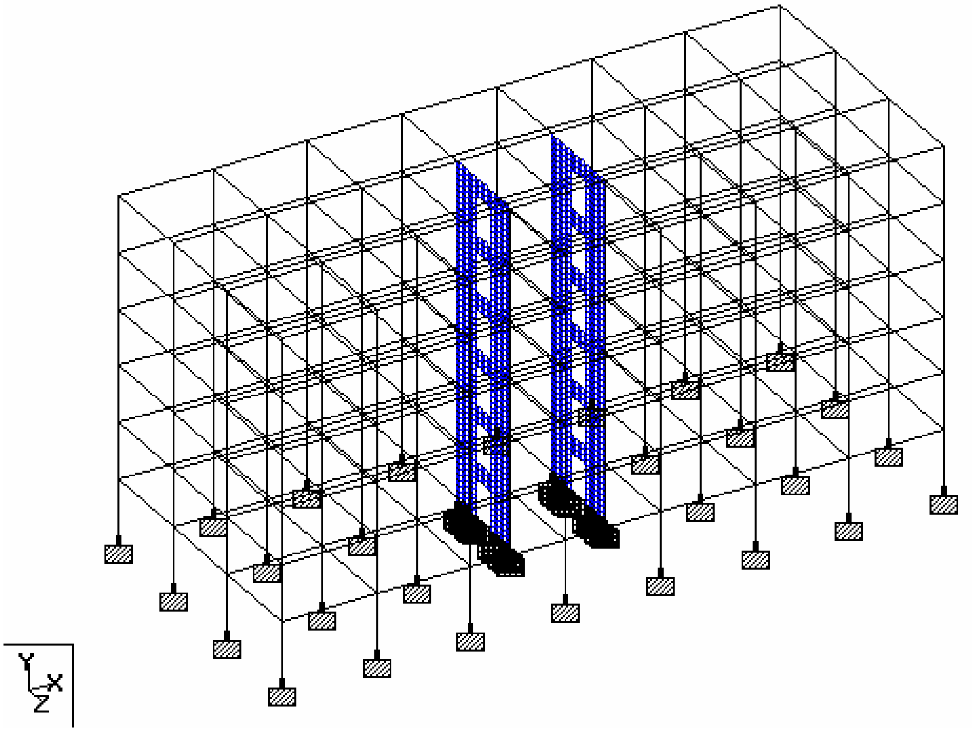
Appendix Figure E6 Maximum absolute principal stress diagrams in shear walls with different openings locations, in 12-story buildings

Appendix F

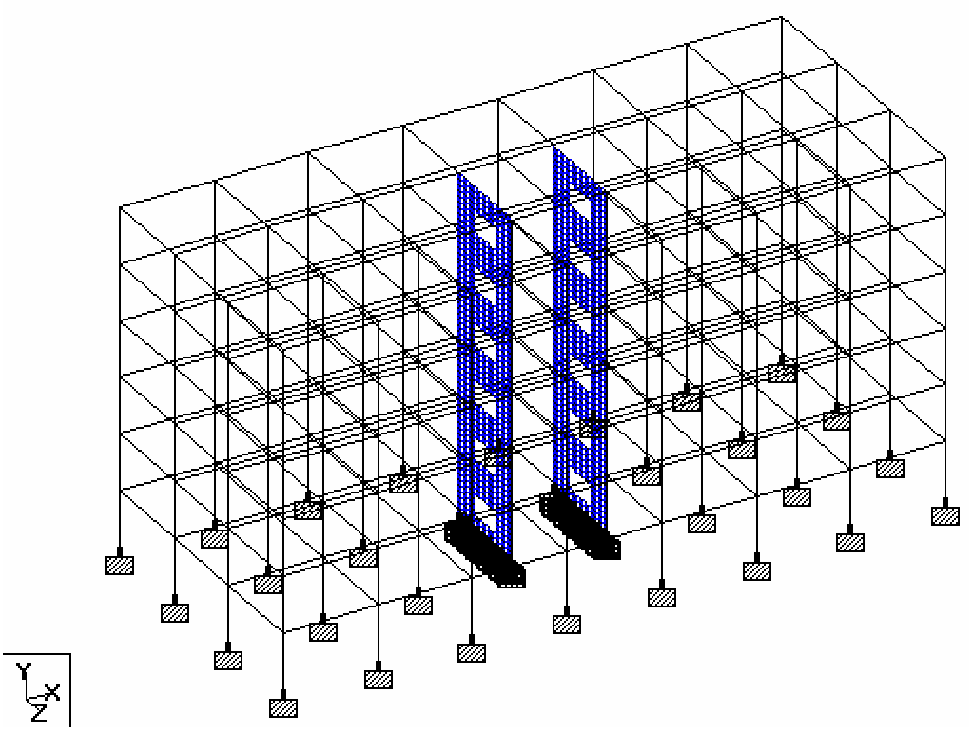
Finite element models



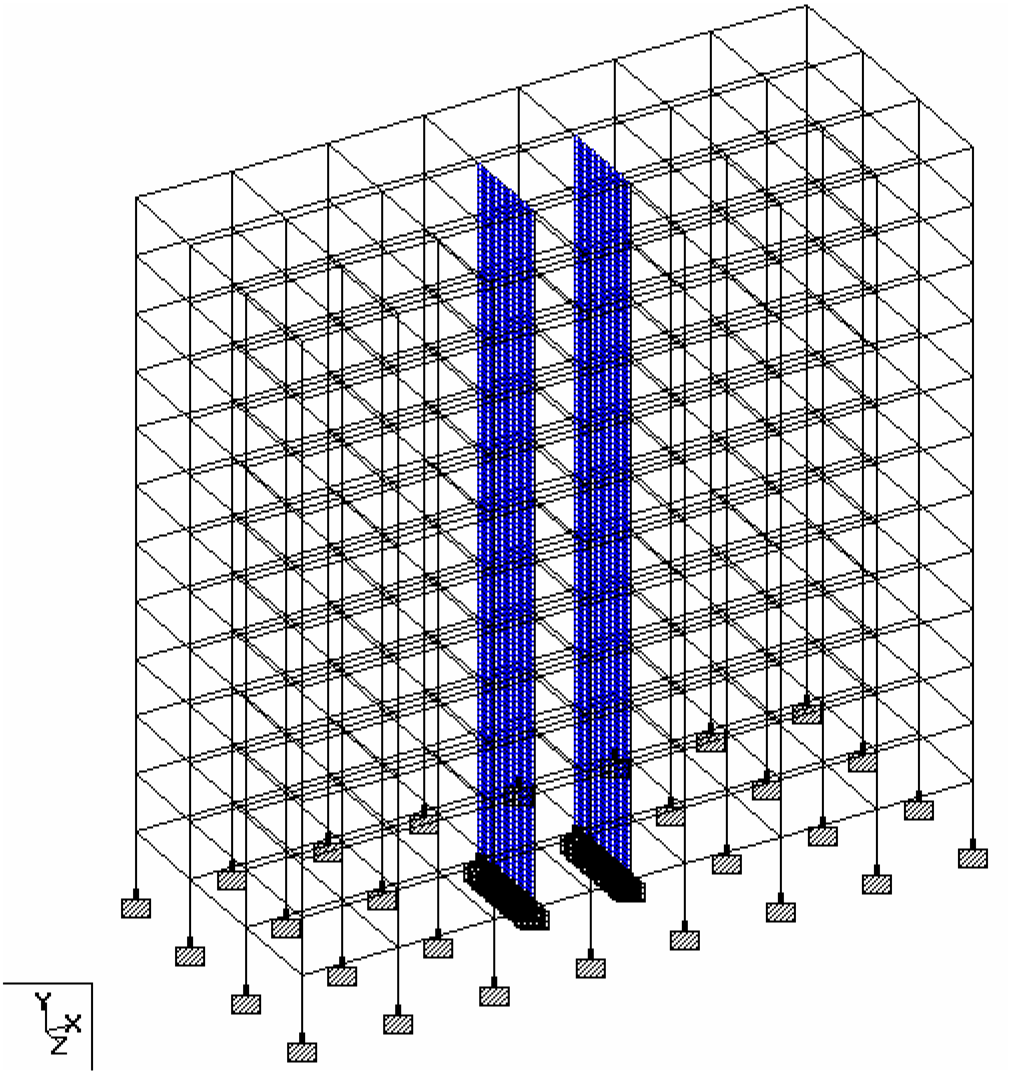
Appendix Figure F1 3D model of 6-story building without opening in shear walls



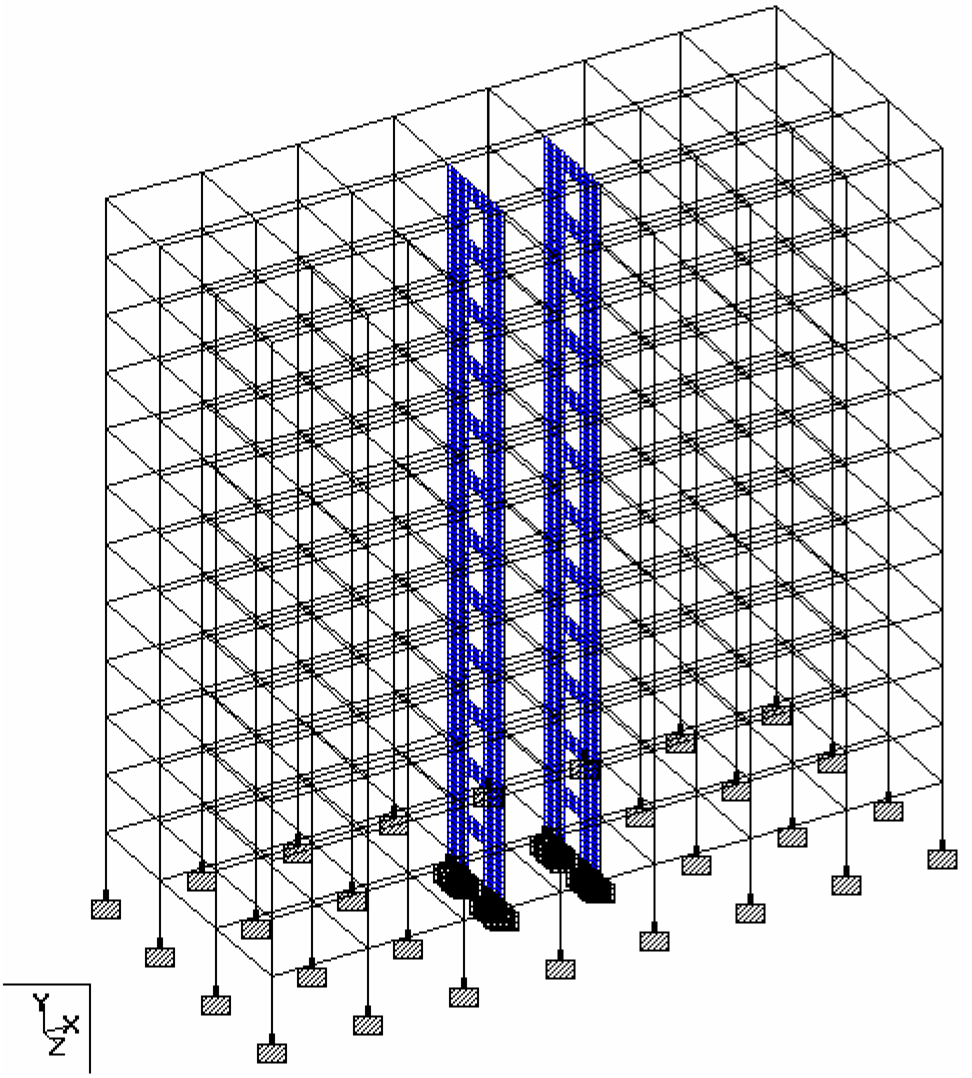
Appendix Figure F2 3D model of 6-story building with door openings in shear walls



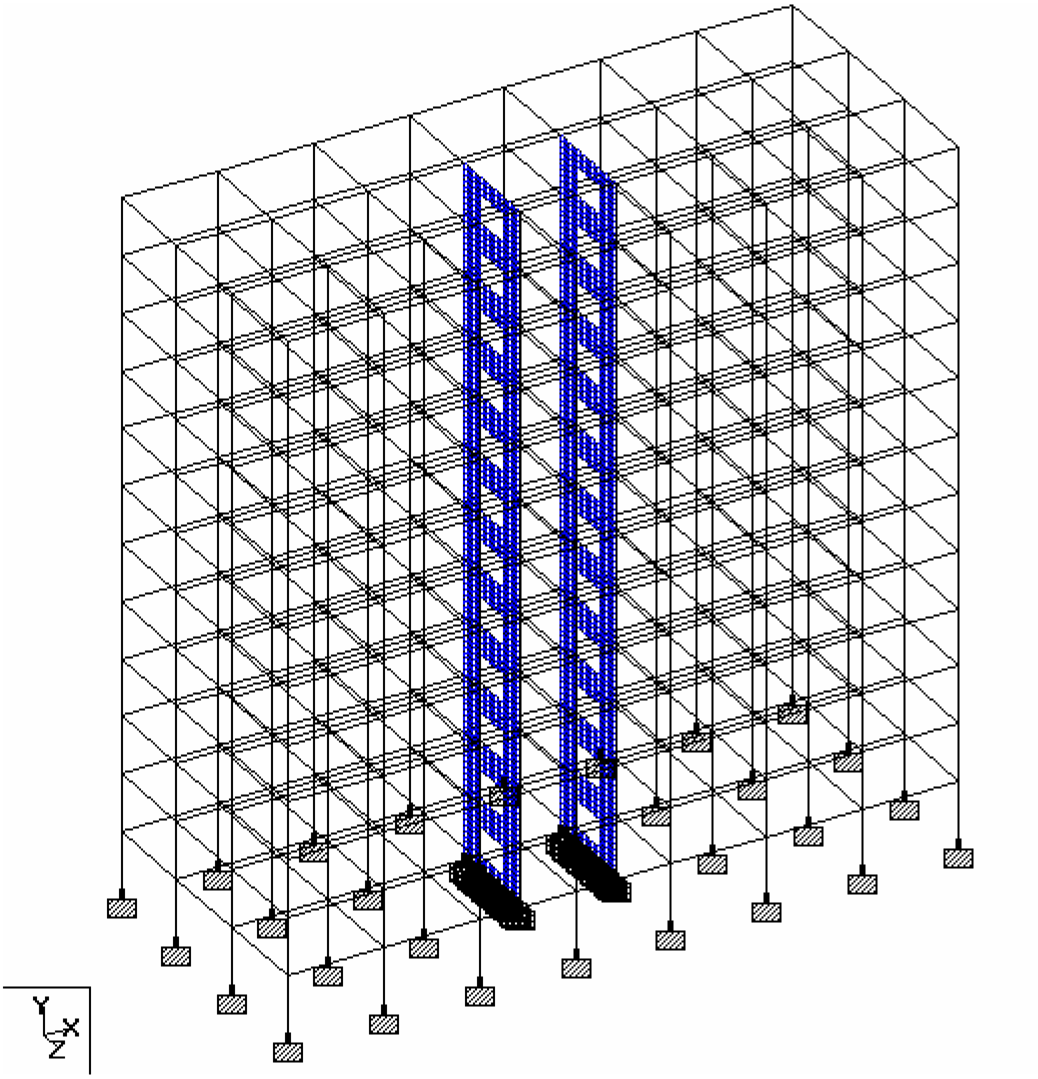
Appendix Figure F3 3D model of 6-story building with window openings in shear walls



Appendix Figure F4 3D model of 12-story building without opening in shear walls



Appendix Figure F5 3D model of 12-story building with door openings in shear walls



Appendix Figure F6 3D model of 12-story building with window openings in shear walls

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