SEISMIC BEHAVIOUR OF FRAME SHEAR WALL BUILDING

A DISSERTATION
Submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
EARTHQUAKE ENGINEERING
(With Specialization in Structural Dynamics)

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation titled "**SEISMIC BEHAVIOUR OF FRAME SHEAR WALL BUILDING**", in partial fulfillment of the requirements for the award of the Degree of **MASTER OF TECHNOLOGY** in Earthquake Engineering, with specialization in **Structural Dynamics** submitted to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, is the record of my own work carried out during the period from August 2003 to June 2004 under the supervision of **Dr. Yogendra Singh**, Assistant Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, India.

This matter embodied in this dissertation has not been submitted for the award of any other degree.

Dated: 30 June, 2004

Place: Roorkee

(KAPIL JAIN)

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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(KAPIL JAIN)

ABSTRACT

Earthquake causes lateral forces in buildings, due to their inertia. A number of structural systems for buildings have been developed, over the years, to resist the lateral load. Shear wall and frame-shear wall systems are quite common for mid-rise buildings. Performance of shear wall buildings during past earthquakes has been observed to be quite satisfactory. In this dissertation an analytical study has been made to estimate the performance of frame shear wall buildings designed as per current IS codes.

A number of options are available for analytical modelling of shear walls. In linear range the shear walls can be modelled either as wide columns or using plane stress elements. Perform 2D/ 3D software provides two elements for non- linear modelling of shear walls. In the present study, five 10 storeyed buildings with different shear wall areas, including a frame building without shear wall have been considered. These buildings have been analysed and designed as per IS codes, using STAAD software. Non-linear behaviour and performance of these building under expected earthquake have been studied using Non-linear Static Push Over Analysis in Perform 3D software.

It has been observed that behaviour of the bare frame building without shear wall is not satisfactory, while the behaviour improves significantly with shear walls. Effect of varying the area of shear wall, on the behaviour of buildings has been studied and optimum shear wall area in terms of the floor area has been estimated. The effect of non-linearity on frame shear wall interaction has also been studied.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Loads acting on the structure are generated either directly by the force of nature or made by man himself; that is there are three basic sources for building loads: gravity load, lateral load and man made.

As a result of gravity, the weight of the building itself produces forces on the structure, called dead load and this load remains constant throughout the building's life span. Lateral forces result from the erratic motion of ground and due to wind. The man made sources of loading may be the variations or shocks generated by cars, elevators, machines, and so on, or they may be movement of people and equipment or the result of blast and impact. [23]

These all forces are mutually dependent. The mass, size, shape, and material of building influence the lateral load and gravity load action.

The lateral forces can be caused by different agencies namely wind, blast or earthquake and magnitudes of such forces are controlled by the intensity of these agents. The structure should have sufficient strength so as to respond satisfactorily to these occasional loads.

The structural system of a building consists of two components, one is horizontal framing system (beam and slab) and other is vertical framing system (walls and columns). Horizontal framing system is primarily responsible for transfer of vertical load and torsional forces to vertical framing system and vertical framing system is responsible for transfer of vertical load and lateral loads to the footing.

1.2 LATERAL LOAD RESISTING SYSTEMS

For proper design of structure an understanding of the behaviour of the structure system under various types of loadings is necessary. With the advent of exceptional computational facilities, a number of softwares are now available to study the behaviour of structure along with its analysis and design. Most commonly employed lateral load resisting systems are introduced here.

1.2.1 Load Bearing Masonry Wall

Bearing wall structures have been of thick, heavy masonry wall construction. Their high weight and inflexibility in plan made them insufficient for multistoried application.

1.2.2 Frame

Frames are those structures which derive their lateral load resistance from the rigidity of connection between beams and columns. The frames are infilled by masonry panels for the purpose of partition. Generally contributions of infills are ignored in lateral load resistance, because under severe shaking due to earthquake these fail and fall apart before the frame is subjected to ultimate load.

Concrete framed structures rely on the formation of plastic hinges in the beams rather than the columns. If the framed structure has not been designed well the sideways motion during earthquake will cause the plastic hinges to form in column. This can cause, in the worst case, the plastic hinges to form in columns of only one storey as the other storey columns may be stronger. If this does occur it will create a need for huge ductility. It is likely that the columns would not have been designed for such demands and so the column will fail. This type of collapse is often seen after major earthquakes and results in storey failure. However if the beams fail first, then a sideways movement can develop which makes much more moderate demands on the ductility of the beams and columns.(Figure1.1) Additionally the ductility required is much easier to be provided in beams than in columns and repair in beam is also easier.[27]

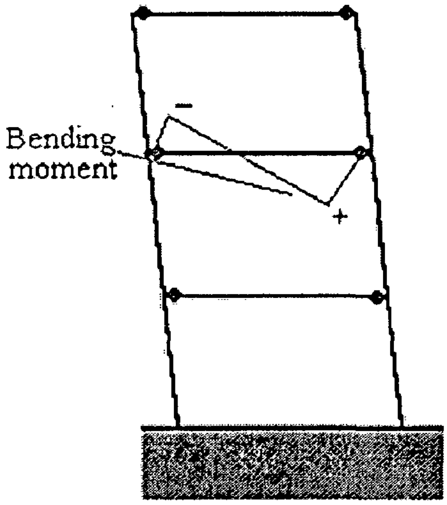

Figure 1.1 Typical Frames.

1.2.3 Shear Wall

A wall that is subjected to lateral loads in its plane is referred to as "Shear Wall". The extent to which these walls shall share the load coming on the structure is governed by the stiffness, the strength of the wall and strength of the enclosing frame. Shear wall enhances strength, stiffness and ductility of the frame. The term shear wall is actually a misnomer as far as high rise building is concerned. Shear walls are designed to resist lateral forces on the building by wind and earthquakes. They may be expressed as exterior or interior walls or cores enclosing elevator shafts or stairways. There does not seem to be any limitation to the geometrical configuration of shear wall systems. The triangle, rectangle, angle, channel and wide flanges are form in which these are used. Shear wall system whether inside or outside a building may be arranged symmetrically or asymmetrically. Figure 1.2 shows the typical arrangement of shear walls in building.

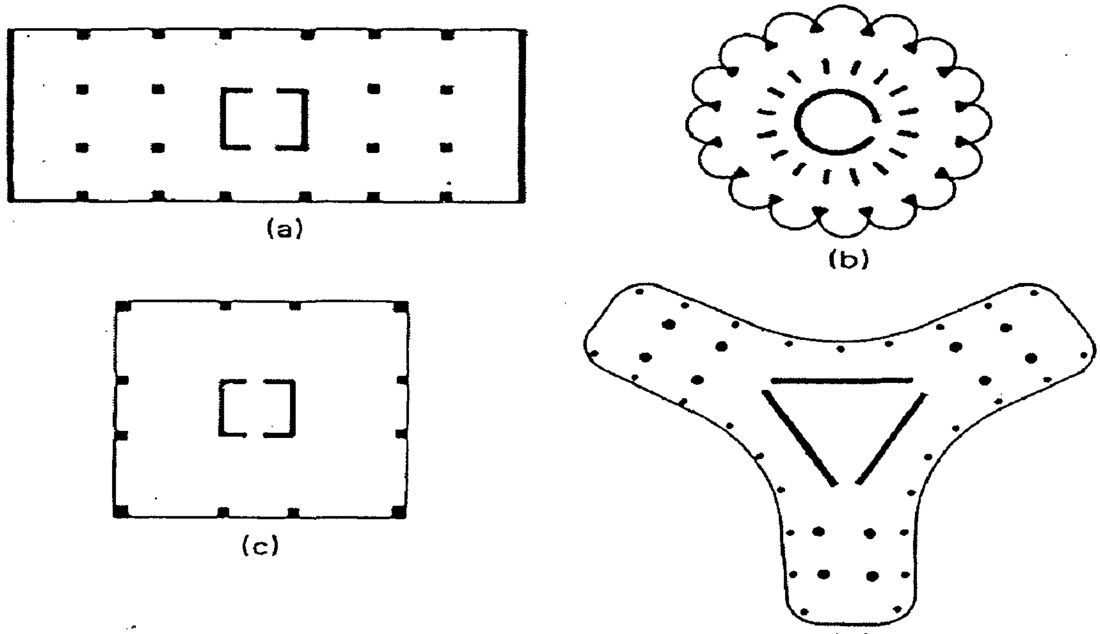

Figure 1.2 Typical Arrangement of Shear Wall in Building [8]

1.2.4 Frame Shear Wall

The great majority of multistory buildings today are in fact, shear wall-frame structure, since elevator shafts, stair wells and central core units of tall buildings are mostly enclosed by shear walls in addition to isolated concrete walls. In a frame with a shear wall, the wall carry a major part of seismic base shear, while the frame is designed to act as a second line of defense against earthquakes, after extensive cracking and/or failure of walls. Shear wall also contributes in resisting the vertical load.

Behaviour of the building consisting of frames with shear wall is more reliable than that of bare frame building, since plastic hinges form in the beams not in the walls. In addition, benefit of reducing lateral sway in the tall building under seismic loading can be availed using shear walls.

1.2.5 Coupled Shear Wall

The terms pierced shear wall and coupled shear wall are used to describe the shear wall with openings. Openings normally occur in vertical row throughout the height of wall and connection between the wall segments is provided by either connecting beams or floor slab or combination of both. Such shear walls behave as two shear walls coupled through the portion of the shear wall between the openings as shown in Figure 1.3. If the openings are very small, their effect on the overall state of stress in a shear wall is minor but large opening has more pronounced effect. The efficiency of the system depends of the strength of the coupling beam. These beams are subjected to very high shear stress.

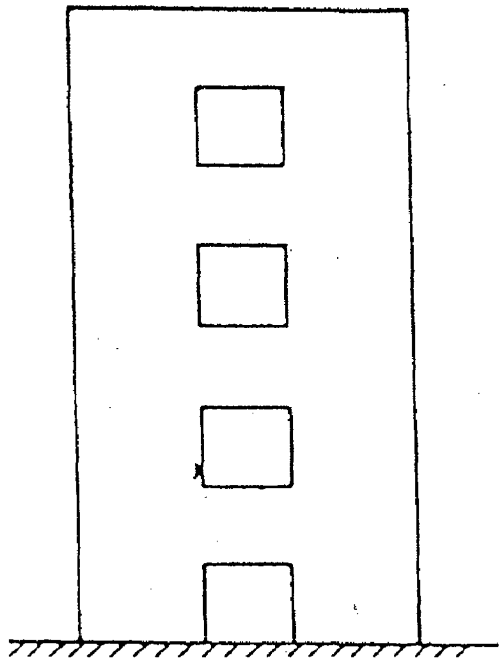

Figure 1.3 Coupled Shear Wall

1.2.6 Frame Tube

In this type of system the exterior walls of building, consisting of closely spaced rectangular grid of the beam and columns rigidly connected together, resist lateral loads through cantilever tube action without using interior bracing. The interior columns are assumed to carry gravity loads only and do not contribute to the exterior tube's stiffness. The stiff floor act as diaphragms with respect to distributing the lateral forces to the perimeter frame panels. Other type of hollow tube buildings have interior cores; It would be ideal in the design of frame tube systems if the exterior walls act as a unit, responding to lateral loads in pure cantilever bending. If this were the case, all columns that make up the tube, analogous to fibers of the beam, would be either in direct axial tension or in compression. The linear stress distribution that would result is indicated by broken line in Figure 1.4.

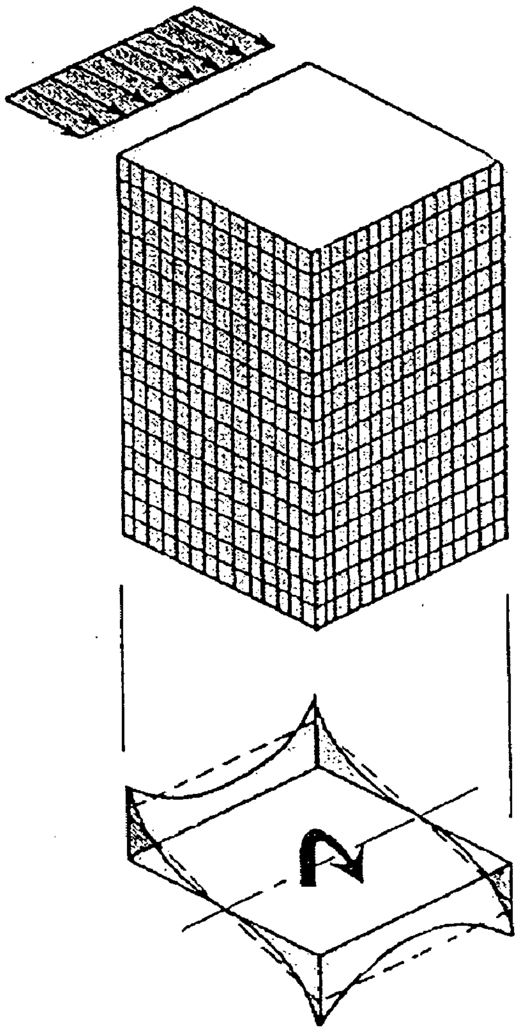

Figure 1.4 Frame Tube Behaviour [23]

However the true behaviour of the tube lies somewhere between pure cantilever and pure frame behaviour. The sides of the tube parallel to the lateral load tend to act as independent multibay rigid frames, given the flexibility of the spandrel beams. This flexibility results in wracking of the frame due to shear, called shear leg. Hence bending take place in column and beams. the effect of shear leg on the tube action result in nonlinear pressure distribution along the column envelope; the column at the corners of the building are forced to take higher share of the load than columns in between (Figure1.4) furthermore the total deflection of the building no longer resembles a cantilever beam, as shear mode deformation becomes more significant.

1.3 BEHAVIOUR OF SHEAR WALL UNDER LATERAL LOADS

Floors act as horizontal diaphragms and distribute lateral load between to the shear walls and frames. It is assumed that floors are sufficient deep and have no major openings, in other words floors are infinitely stiff in their plane and do not distort. The distribution of lateral force is function of the geometrical arrangement of the resisting wall systems.

Deformation of shear wall is different than frame because it deforms in bending mode rather than shear mode as column are deflected. Figure 1.5 shows the deflected shape of shear wall under lateral loading.

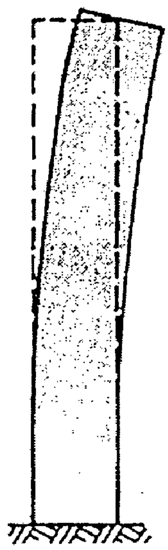

Figure 1.5 Deformation Pattern of Shear Wall Due to Lateral Forces [20]

If the resultant of lateral forces acts through the centroid of a building's relative stiffness, only translational reaction will be generated. The most obvious case is the symmetrical pure shear wall building. In the rigid frame shear wall building, the shear may be assumed to be resisted by the core. This is because the stiffness of shear wall is very high than the lateral stiffness of frame.

The shape and location of shear walls have significant effects on their structural behaviour under lateral loads. A core eccentrically located with respect to building shape has to carry torsion as well as bending and direct shear. However torsion may also develop in buildings with symmetric shear wall arrangements if lateral load does not act through the centroid of building's stiffness. Optimal torsional resistance is obtained with closed core sections. This resistance decreases due to be reduced for the doors, window, and other openings, because the stiffness of the resisting walls decreases.

The torsionally stability of wall system can be examined with aid of Figure 1.6. Many shear walls are open thin walled sections with small torsional rigidities. Hence in seismic design it is customary to neglect torsional resistance of individual walls. Tubler sections are exceptions. Torsional resistance of wall arrangement 1.6(a) (b) and (c) could only be achieved if the laterals force resistance with respect to its weak axis is significant. As this is not the case, these examples represent torsional unstable systems.

Figure 1.6 (d) to (f) show torsionally stable configurations. Even in case of the arrangement in Figure 1.5(d), where significant eccentricity is present under east-west lateral force, torsional resistance can be effectively provide by the actions induced in the plane of short wall. However eccentric system such as Figure 1.6(d) and (f), are particular examples that should not be favored in ductile earthquake-resisting building unless additional lateral force resisting systems, such as ductile frame are also present.

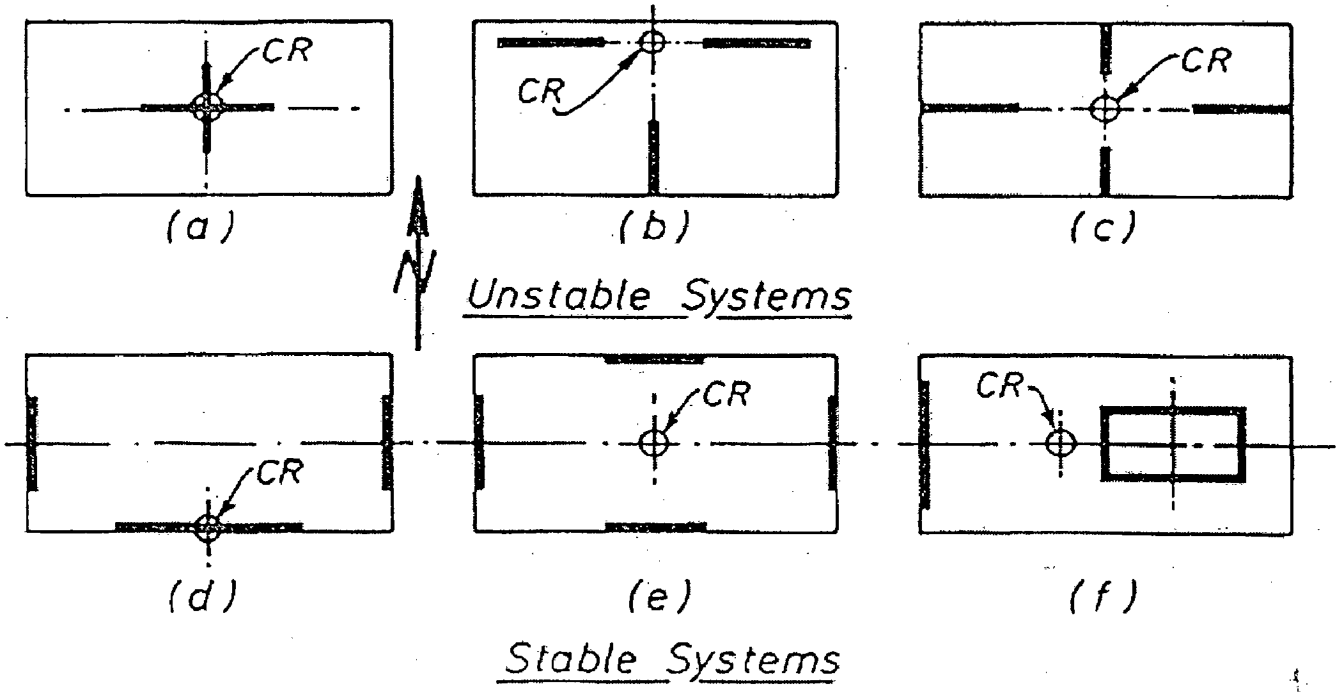

Figure 1.6 Example for Torsional Stability of Wall System [20]

In choosing suitable locations for lateral- force-resisting shear walls, three additional aspects should be considered:

1. For the best torsional resistance, as many of the walls as possible should be located at the periphery of the building.

2. In multistory buildings situated in high-seismic-risk areas, a concentration of the total lateral force resistance in only one or two shear walls is likely to introduce very large forces to the foundation structure, so that special enlarged foundation may be required.

3. The more gravity load can be routed to the foundations via a shear wall, the less will be the demand for flexural reinforcement in that wall and the more readily can foundations be provided to absorb the overturning moments generated in that wall.

Openings in stairwell are sometimes arrange in such a way that an extremely weak shear fibers result where inner edges of the opening line up as shown in Figure 1.7(a). It is difficult to make such connection sufficiently ductile and to avoid early damage in earthquakes and hence it is preferable to avoid the arrangement, a larger space between the staggered opening would allow an effective diagonal compression and tension field to develop after the formation of diagonal cracks(Figure1.7(b)). When suitably reinforced using diagonal reinforcement, distress of region between openings due to shear can be prevented and ductile cantilever response due to flexural yielding at the base only, can be readily enforced.

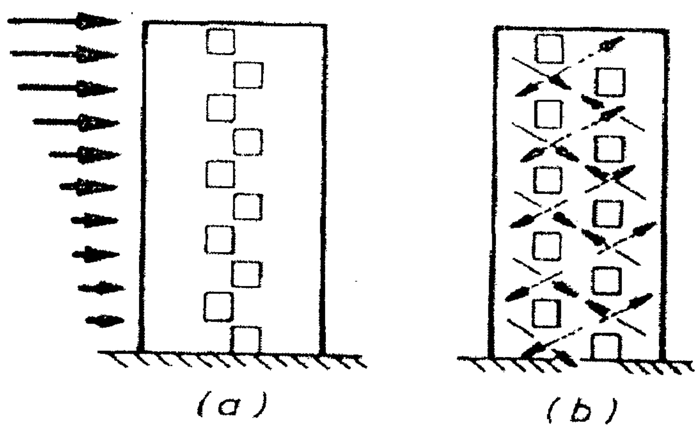

Figure 1.7 Shear Walls as with Opening [8]

1.4 NON LINEAR BEHAVIOUR OF SHEAR WALL

For reliable performance of structure during earthquake, adequate strength and ductility of structure is necessary. Since it is impossible to predict the characteristics of ground motion due to large earthquake, it is impossible to estimate with accuracy the response of structure. However it is possible to provide the structure with the most desirable features like ductility, energy dissipation, desirable failure and damage mechanism. Ductility is defined on the basis of nodal rotation as being equal to $\frac{\theta_{max}}{\theta_y}$ where θ_{max} are the maximum computed rotations at the node and θ_y is the nodal rotation corresponding to yielding at the base. Behaviour of building is change with fundamental time period, flexural strength, and intensity of earthquake, duration of earthquake and frequency characteristics of earthquake.

It has been observed that the horizontal displacements show a consistent increase with increasing fundamental time period (or decreasing stiffness) of the structure. The ductility requirement becomes greater with decreasing fundamental time period. However beyond a certain value of fundamental time period, ductility requirements do not decrease significantly with an increase in period. Rotational ductility requirements increase with increasing stiffness.

For the same fundamental period, the horizontal displacement decreases sharply as the yield level increases. It can be seen that the ductility requirements increase significantly as the yield level decreases.

Slender walls are governed by their flexural strength. They are usually subjected to low nominal shear stress. They develop a predominantly horizontal crack pattern in the lower hinging region after a few cycles of inelastic deformation. After yield of the vertical reinforcement, shear is resisted by interface friction across cracks and dowel action of the vertical reinforcement. The flexural strength of wall is limited by fracture of main flexural reinforcement.[21]

Nominal shear stress V_s is usually computed as V/hd, where V is the base shear, h is the wall thickness, and d is the effective depth between the extreme compression fiber and the centroid of the rebars in tension. Code requirement allow use of the value $0.8l_w$ for d, where l_w is the total length of the wall.

The behaviour of wall subjected to $v_{max} \leq 3\sqrt{f_c}$ [2] is characterized by the formation of predominantly horizontal crack pattern in the lower hinging region of the wall after a few inelastic deformation reversals. Generally, in the wall subjected to low nominal shear $v_{max} \leq 3\sqrt{f_c}$, there is adequate shear transfer capacity even though the crack may intersect it from basically horizontal plane. In wall capable to resisting high nominal shear stresses $(v_{max} \geq 7\sqrt{f_c})$, inclined crack form a diagonal strut system that precludes development of sliding shear. Therefore failure of sliding shear is a possibility only in the range of nominal shear stress between $3\sqrt{f_c}$ and $7\sqrt{f_c}$. Figure 1.8 shows the shear failure modes in squat shear walls and Figure 1.9 shows the failure modes in slender walls.

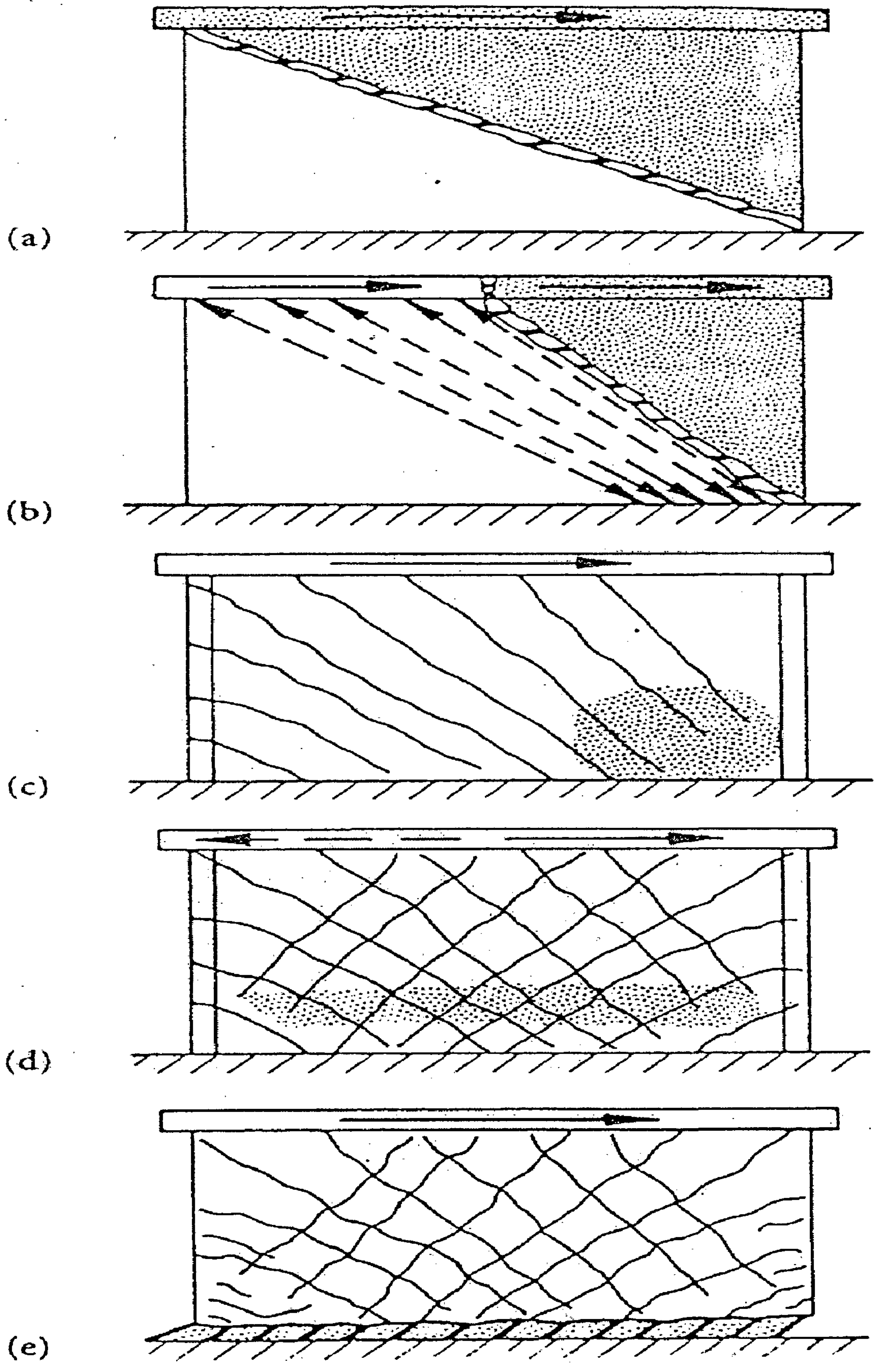

Figure 1.8 Shear Failure Modes in Squat Shear Wall [10]

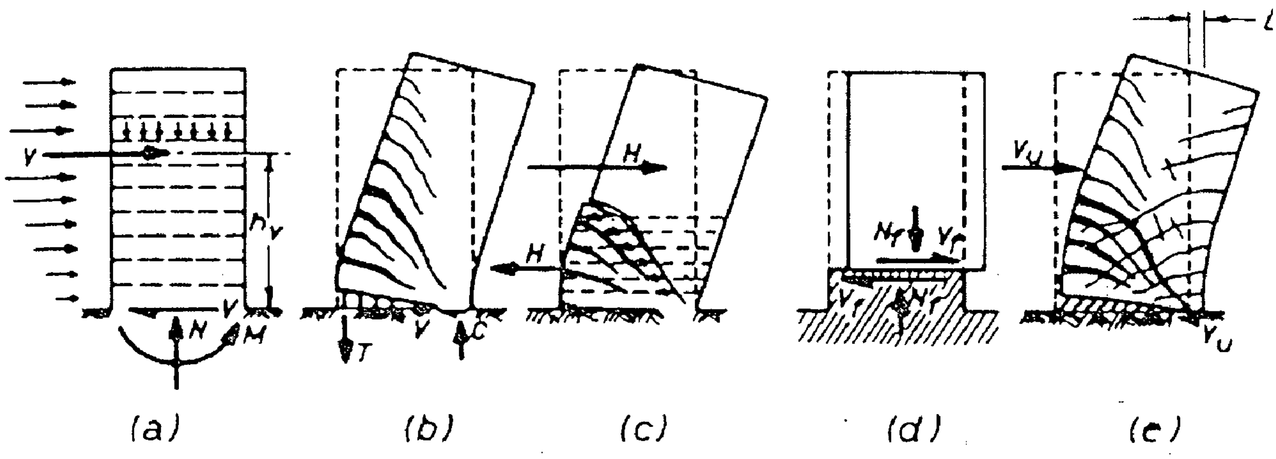

Figure 1.9 Failure Modes in Slender Walls [10]

The various parameters that influence the response of shear walls are as follows:

1. *Height to Width Ratio*: Walls whose height to width ratio is greater than 2, beha like a bending beam while those with a smaller ratio behave like shear beam. F walls having ratio less than 1.0, vertical reinforcement is seen to be more effecti than horizontal reinforcement in resisting the horizontal shear force

2. *Type of Loading*: Shear wall show less flexural strength and deformation und reverse cyclic loading as compared to monotonic loading. Squat shear walls do n show any significant difference in behaviour under monotonic loading a reversed loading.

3. *Flexural Reinforcement*: The vertical reinforcement present in the wall determi the flexural capacity and hence the maximum shear force to which it can subjected.

4. *Shear reinforcement*: Horizontal shear reinforcement is provided to prev diagonal tension failure. It improves the inelastic response of walls subjected

high nominal shear stress by reducing shear deformation. It is ineffective in resisting sliding shear and does not influence the web crushing strength significantly.

5. *Diagonal Reinforcement*: It is used in the web to reduced shear deformation and sliding shear it is particularly used in squat shear wall. It also contributes to flexural strength and results in increased energy dissipation capacity.

6. *Special Transverse reinforcement*: This is required in boundary elements that lie in the potential hinging region of the wall were large inelastic rotations are likely to occur. It provides lateral confinement in concrete in the boundary element support the vertical reinforcement and it improve the shear capacity and stiffness for dowel action of the boundary element.

7. *Concrete Strength*: This affects the extreme fiber compressive capacity, web crushing capacity and shear strength of concrete.

8. *Construction Joint*: Sliding shear failure is observed in poorly constructed joints. They perform satisfactorily if the surface of previously cast concrete is roughened and cleaned so as to remove loose particles and laitance

9. *Axial Compressive stress*: The presence of moderate axial compressive load on a wall that is loaded monotonically or under reversed cyclic loading results in an increase in its flexural capacity and shear strength. It reduces shear distortion and increase the shear stiffness of the hinging region.

10. *Moment to Shear Ratio*: The nominal shear capacity of the wall increases as this ratio decreases. This is primarily because at low ratios flexural yielding is unlikely

to precede web crushing. Wall tests have shown that web crushing shear capacity is a function of shear distortions as well as concrete strength. Shear distortions increase significantly as flexural yielding take place.

1.5 FRAME SHEAR WALL INTERACTION

Usually shear walls are provided with frames. There results a complex interaction between the two structural components.

1.5.1 Elastic Interaction

Figure 1.10 shows the deflected shapes of frame and shear wall.

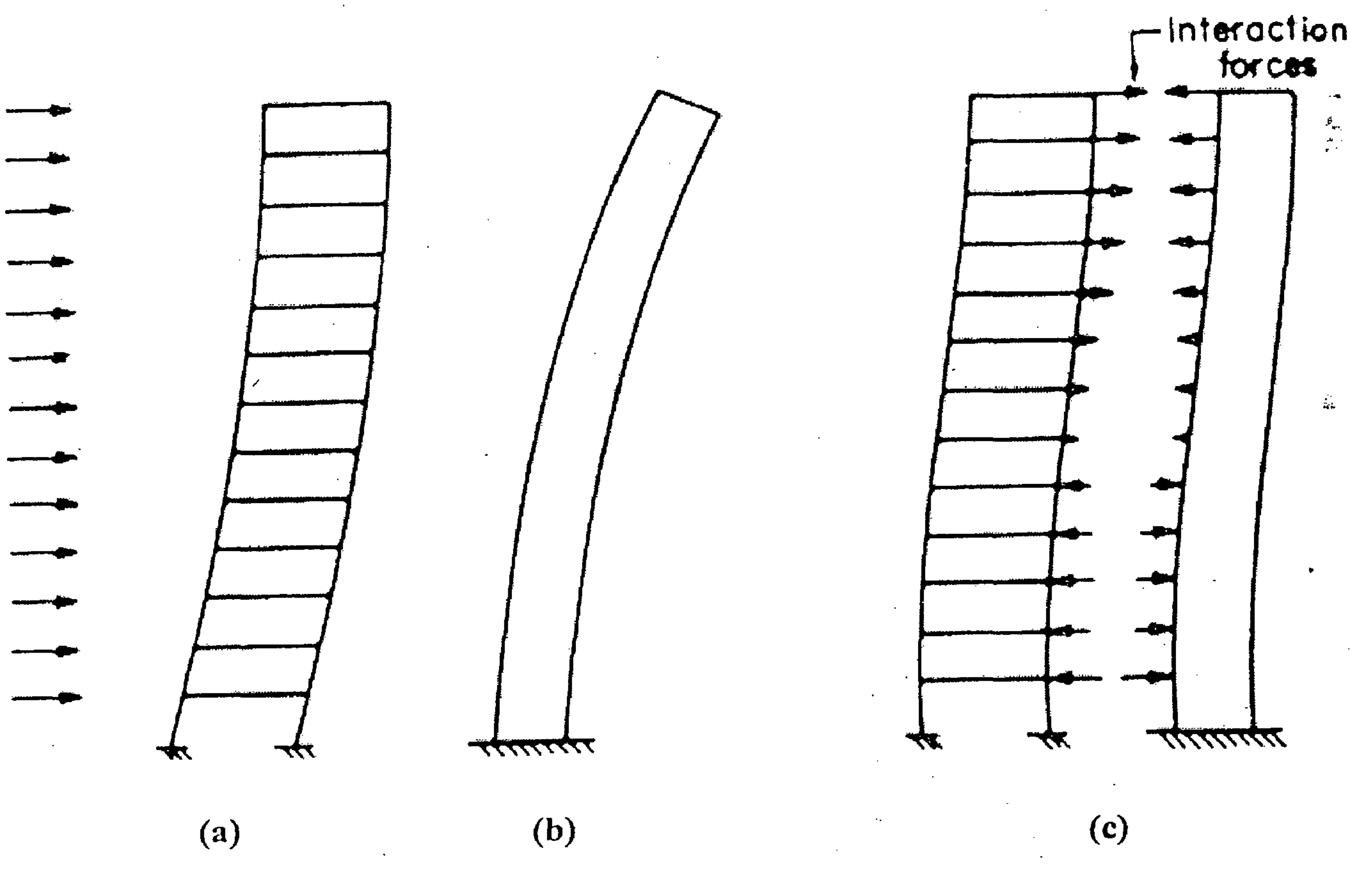

Figure 1.10 Frame Shear Wall Interaction (17)

Rigid Frame shear mode deformation: This deflection is shown in Figure1.10 (a).The slop of the deformation is greatest at the base of the structure where the maximum shear is acting.

Shear wall bending mode deformation: This deflection is shown in Figure 1.10(b). This system act as vertical cantilever beam and deflects as such. The slope of the deflection is greatest at the top of the building, indicating that in this region, the shear wall system contributes the least stiffness.

Interacted frame and shear wall: By superimposing the above shapes, the combined shape is comes in the form of flat S curve. Because of the different deflection characteristics of shear wall and frame, the shear wall is pulled back by the frame in upper portion of the building, and pushed forward in the lower portion. Hence, lateral shear is carried mostly by the frame in upper portion of the building and by the shear wall in lower portion. This interaction is shown in Figure 1.10(c).

1.5.2 Inelastic Interaction

(a)Bending: The moment diagrams for slender shear walls ($h_w/l_w>2$) in dual system under static seismic action have form of Figure 1.11 However, the dynamic response analysis results in moment diagrams with approximately linear variation. Thus the design moment diagram introduced by Paulay has the form of trapezoidal covering the saw like M diagram. The vertical displacement h_{cr} of the envelop aims at ensuring that inelastic deformations, i.e. curvature ductility demand during an earthquake, will be restricted to the base of wall (capacity design).

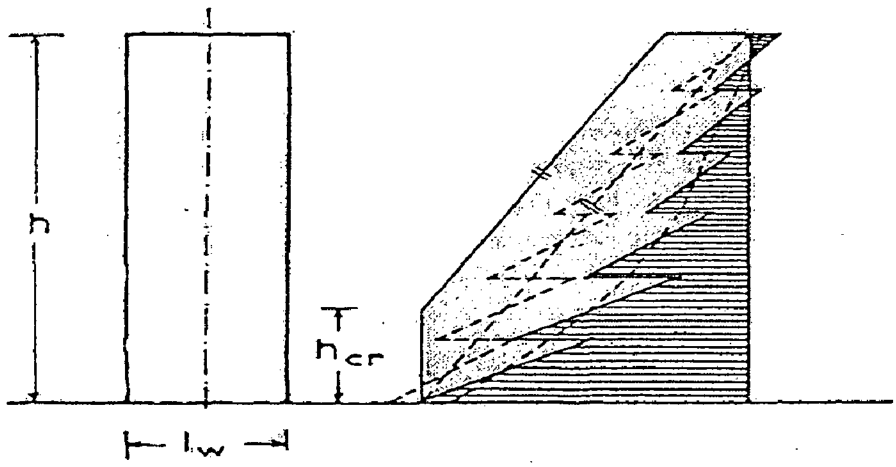

Figure. 1.11 Shear Wall Moment Diagram [21]

The value of h_{cr} above the base of the wall may be estimated as follows.

$$h_{cr} = \max\left(l_w, h_w/6\right) \tag{1.1}$$

but

$$h_{cr} \le \begin{cases} 2l_w \\ h_s \text{ for } n = 6 \text{ storey} \\ 2h_s \text{ for } n = 7 \text{ storey} \end{cases} \tag{1.2}$$

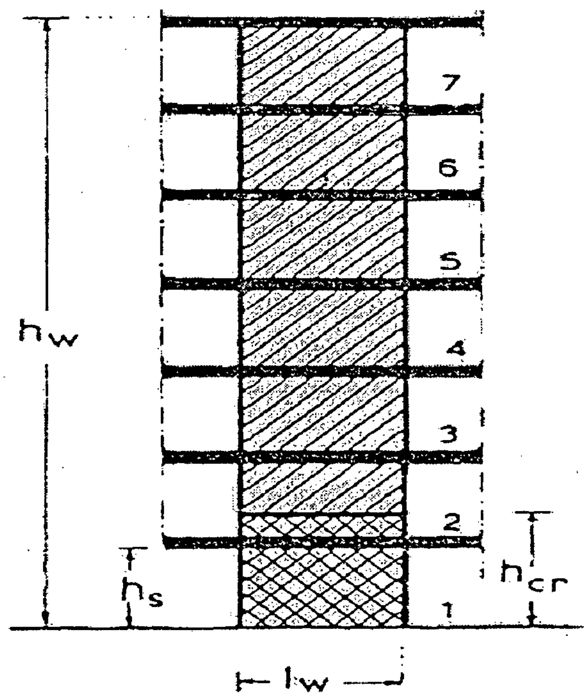

Figure 1.12 Critical Region at the Base of Wall. [21]

The above design moment envelope applies to all ductility categories.

For squat wall ($h_w/l_w \leq 2$) there is no need for a design envelop of bending moments

(b)Shear: in dual systems containing slender walls, in order to account for the uncertainties in the contribution of higher modes, a modified design envelope of the shear force is adopted (Figure 1.13). In addition, inelastic analyses have shown that the resulting shears are much higher than the shear derived from an elastic response analysis. For this reason the design envelop of the shear force along the height of the wall is derived as follows (application of the capacity design criterion)

$$\left[V_{sd} = \varepsilon V_{sd}^{'}\right] \tag{1.3}$$

For $z<1/3h_w$, while for $1/3h_w<z>h_w$ the variation is linear according to Figure 1.13.

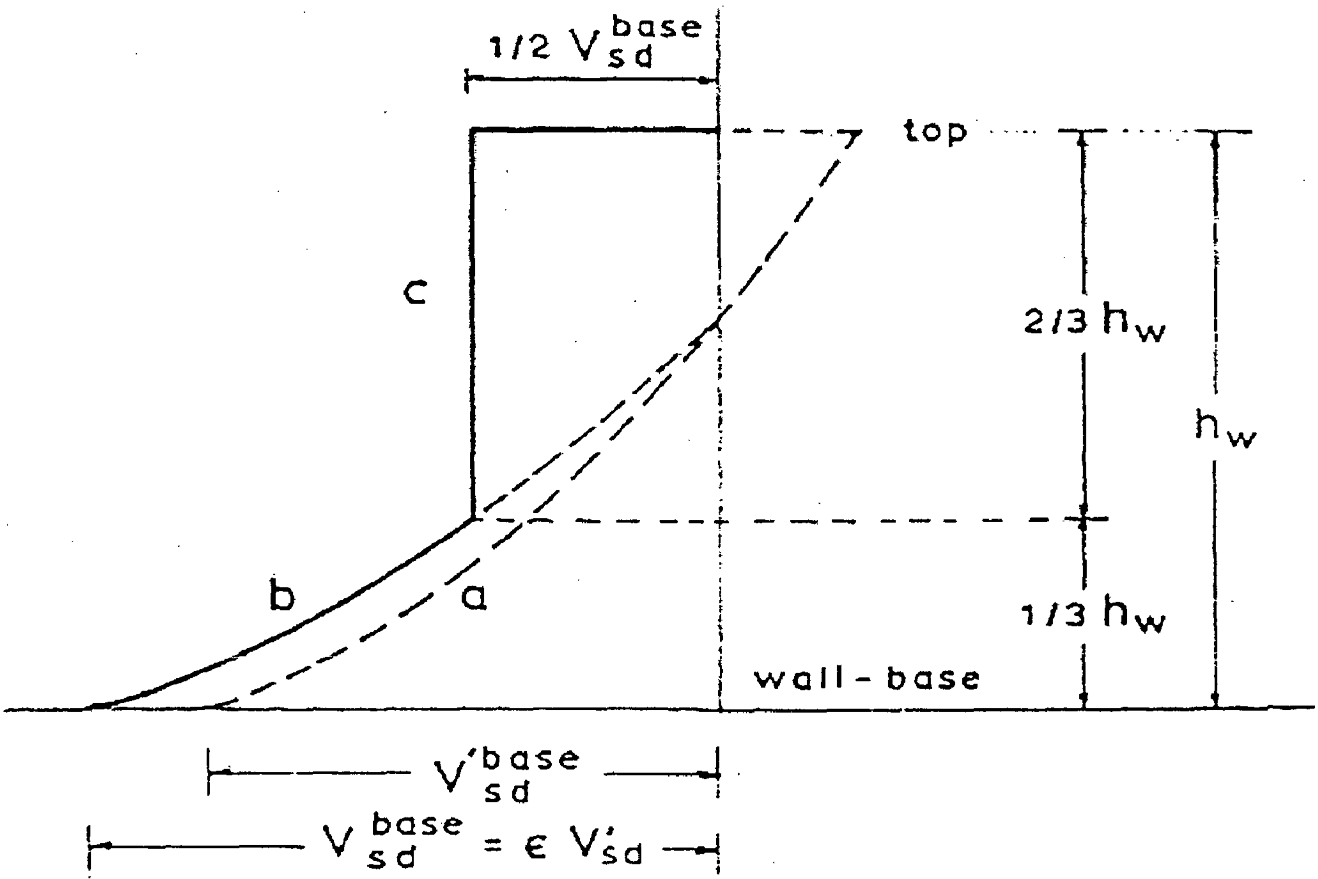

Figure 1.13 Design Envelop for Shear Force in Slender Walls of Dual System[21]

In the above equation V_{sd} is the shear force along the height of wall, obtained from the analysis and ε is the magnification factor, which depend upon ductility class as follow:

1. For DC 'H'(Ductility class-high) and M (Ductility class-medium) the magnification factor ε may be estimated as follows

$$\varepsilon = q\left[\left(\frac{\gamma_{RD}}{q}\frac{M_{RD}}{M_{SD}}\right)^2 + 0.1\left(\frac{S_e(T_c)}{S_e(T_1)}\right)^2\right]^{1/2} \leq q \tag{1.4}$$

   where M_{sd} is the design bending moment at the base of wall, M_{Rd} is the design flexural resistance at the base of the wall, γ_{Rd} a factor equal to 1.25 for DC 'H' and 1.15 for DC 'M', T_1 is the fundamental period of vibration of building, T_c the upper limit period of the constant spectral acceleration branch and $S_e(T)$ the ordinate of elastic response spectrum.

2. For DC 'L'(Ductility class-low) the magnification factor ε may be taken equal to 1.3.

in case of squat walls the design shear V_{Sd} should be taken as follows:

For DC 'H' and 'M'

$$V_{Sd} = \gamma_{Rd}\left(\frac{M_{Rd}}{M_{Sd}}\right)V'_{Sd} \leq qV'_{Sd} \tag{1.5}$$

For DC 'L' the shear force V'_{sd} may be increase by a magnification factor equal to 1.3

1.6 PERFORMANCE OF SHEAR WALL BUILDINGS IN PAST EARTHQUAKES

The observed response of medium high-rise buildings during the latest earthquakes in the USA, Chile, Mexico and Japan have indicated that buildings with structural walls or dual systems of frames and walls behave considerably better during strong shaking. A series of strong motion and ambient vibration records have been obtained in a 22-stories high structural wall Chilean building[25] The structure walls with a total wall area between 3 to 6% of the floor plan area were used. In these structures typical nominal shear stresses in the walls for strong motions was in the range of 0.2 to 0.8 MPa and wall thickness 300 to 400 mm. These walls were located by Whittaker at the perimeter, corridors and around vertical circulation areas. Dynamic properties and response characteristics are identified using parametric and nonparametric system identification techniques. Torsional effects are not important in the observed seismic response but they become relatively important during low level ambient excitations. The basic response characteristics identified are compared with those of a three-dimensional model. The model was determined using typical consulting office assumptions. The agreement between model and experimental records is good for global dynamic parameters, and with further adjustment, seismic response can be modeled with a good degree of accuracy validating a series of modeling assumptions.

Reinforced concrete structures to investigate the physical characteristics of a coupled shear wall structure were taken. The study was intended to qualitatively confirm the following points: (a) stress distribution and deformation properties of the whole structural system, (b) degree of rigidity contributed by the connecting beams and perpendicularly intersecting wall to the coupled shear wall, and (c) the process of the structural failure. The following conclusions were drawn:

- The deformation pattern and stress distribution of this structural type were very similar to those of one-story, one-bay rigid frames with wall-shaped columns and the top rigid connecting beam.

1. Plastic hinges formed at both ends of connecting beams, column base and column top in order of occurrence. Maximum strength of the model was recorded when the plastic hinge yielded around the column top. Collapse of the model resulted from bending and shears failure of the top connecting beams.

2. Since the vertical stress distribution was nearly uniform at a level in the wall perpendicular to loads, it was expected that the full width of the wall was effective as flanges to the wall system if the proportion of the wall elements was similar to that of this model.

3. The effectiveness of reinforcement after initial cracking was such that an increase in strength and high ductility can be anticipated if the structure was properly reinforced.

Two geometrically similar but structurally very different reinforced concrete planar model structures were examined in recent earthquake of California. One was a 7-story shear wall and the second was a 7-story coupled shear wall. The observed performances of these two models were studied, based on comparisons related to the dominant response frequencies and the measured story drifts, base shear and overturning moment. The initial stiffness of the coupled shear model was approximately 40% of that of the shear wall;

however, it degraded very much as soon as diagonal cracking occurred. Thus, for moderate earthquake loads, coupled shear wall may retain an initial high stiffness, which is expected to become much lower after the appearance of damage in the wall. The initial stiffness for the shear wall also degraded during the large displacement response stage, accompanied by moderate damage at the base of the wall. The measured maximum base shear of the shear wall was more than twice that of the coupled shear all, whereas the maximum overturning moment for the shear wall was 50 percent larger than that of the coupled shear wall. [10]

MODELING OF SHEAR WALLS

Several approaches have been used for modeling of shear walls, ranging from equivalent isotropic continuum modeling to discrete frame modeling. Close form solutions have been obtained for shear walls and a number of Finite Elements are available which can be used for modeling of shear walls. Although close form solutions provide important insight into behaviour of shear walls, in this chapter discussion is limited to computer oriented Finite element modeling.

As discussed in the previous chapter, behaviour of shear walls is similar to that of a wide column, which has primarily a bending moment, but shear deformations are also not insignificant. A model of shear wall should take into account both these modes. The following sections describe the various linear and non-linear models for shear walls.

2.1 LINEAR MODELING

Linear modeling of shear wall can be done using following elements.

2.1.1 Equivalent Wide Column Modeling

A shear wall may be modeled as a wide column. The stiffness matrix for the wide column should account for the shear deformations. Stiffness matrix for a wide column element shown in Figure.2.1 is given by the Equation 2.1.

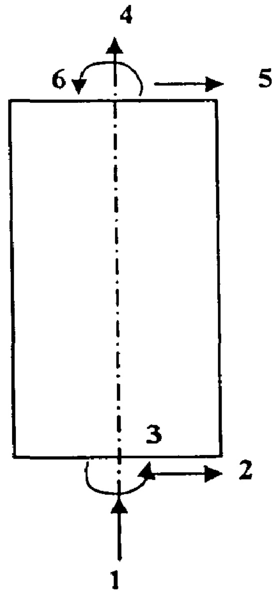

Figure.2.1 Wide Column Element for Modeling of Shear Wall

Using the wide column analogy, coupled shear wall can also be easily modeled on any available computer software as frame, in which the two shear walls are modeled as wide columns along the center line and the beams are having rigid arms at the ends with length equal to half the width of shear wall. The Frame-Shear wall can also be modeled in the similar manner with shear wall replaced by a wide column and a beam joining the frames with shear walls having rigid arm at the end joining the shear wall. The length of rigid arm is again equal to half the width of the wall.

$$[K]=\frac{1}{(1+\alpha)}\begin{bmatrix} \frac{EA}{L} & & & & & \\ 0 & \frac{12EI}{L^3} & & & & \\ 0 & \frac{6EI}{L^2} & 4+\alpha\frac{EI}{L} & 0 & & \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & & \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & (2-\alpha)\frac{EI}{L} & 0 & -\frac{6EI}{L^2} & (4+\alpha)\frac{EI}{L} \end{bmatrix} \tag{2.1}$$

where, $\alpha = \dfrac{12EI}{L^2 GA_R}$, G is shear modulus of elasticity and A_R is the effective shear area.

Modeling of finite size of joints: In case of frame shear wall building with wide columns and/or deep beams the size of the joins is not negligible (Figure 2.2). The beam column member can be replaced by a linear member with rigid ends. The rigid end portions are having infinite stiffness.

The stiffness of member with rigid ends can be obtained as

$$[K] = [H]^T [K][H] \tag{2.2}$$

where, $[K]$ is given by Equation 2.1 and [H] is given as

$$[H] = \begin{bmatrix} 1 & dl & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & -bl \\ 0 & 0 & 0 & 1 \end{bmatrix} \tag{2.3}$$

where *bl, dl,* and *l* are lengths of rigid end at left end, lengths of rigid end at right end and length of the member, respectively, as shown in Figure 2.2..

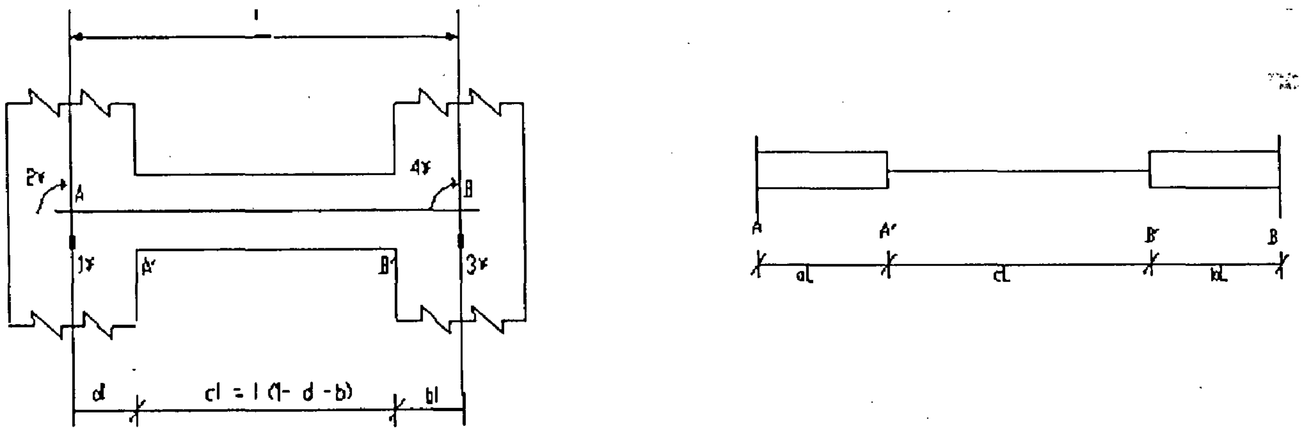

Figure 2.2Beam Member with Finite Size of Joints

2.1.2 Modeling Using Plane Stress Element

Use of plane stress elements is the viable alternative when beam element is unsuitable. When modeling a tall wall one can apply lateral load to either side of wall without affecting the result significantly (due to large depth to span ratio). But with a low rise wall is modelled as plane stress elements a lateral point load will cause two components of deformation. First there will be deformation due to localized stress in the area of load, and second there will be overall deformation due to shear and bending. The following are the basic assumptions in plane stress modeling. [25]

- There is no applied load perpendicular to the plane of shear wall
- There is no restraint to strain in the direction perpendicular to the plane of shear wall

Sometimes shell elements are also used for modeling of shear walls. However, these elements act essentially as plane stress elements as bending stresses in shear walls are negligible.

PLANE 42 and SHELL 63 in ANSYS software and Plate element in STAAD Pro software is available for this type of modeling.[6]

PLANE 42 is used for 2-D modeling of solid structures. The element can be used either as a plane element (plane stress or plane strain) or as an axi-symmetric element. The element is defined by four nodes having two degrees of freedom at each node- translations in the nodal x and y directions (Figure 2.3). The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities. This element can be used only for homogeneous material.

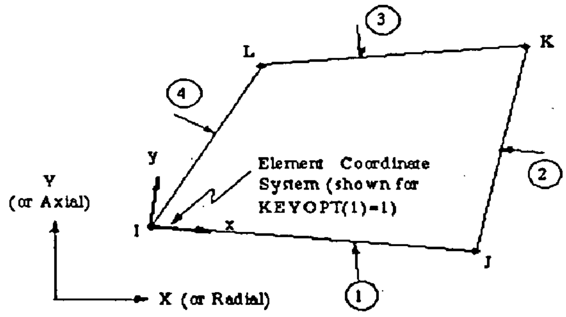

Figure 2.3 Plane 42 Element [10]

The STAAD plate element is based on hybrid finite element formulations. A complete quadratic stress distribution is assumed. The membrane and bending stresses used in the element are shown in Figure 2.4.

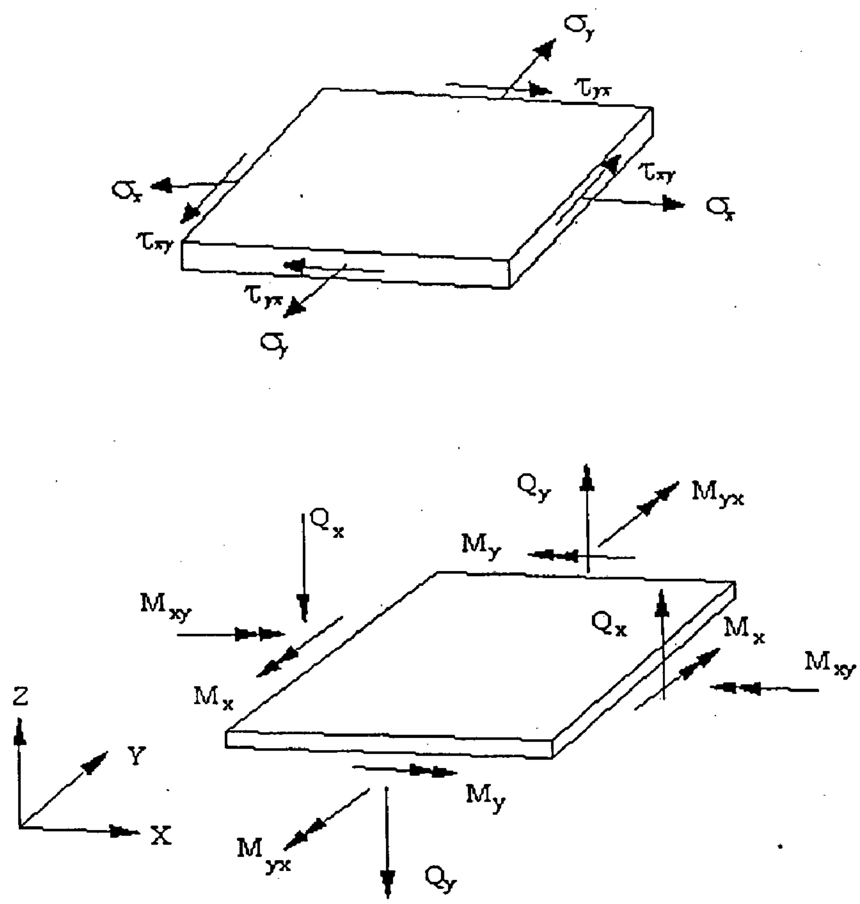

Figure 2.4 Plate Element [6]

SHELL 63 is an 8-noded shell element with six degrees of freedom at each node viz. three translations and three rotations as shown in Figure 2.5. It has both bending and membrane capabilities. The deformation shapes are quadratic in both in-plane directions. The element has plasticity, stress-stiffening, large deflection and large strain capabilities. The element is defined by eight nodes and four thicknesses. This is also useful only for homogeneous material.

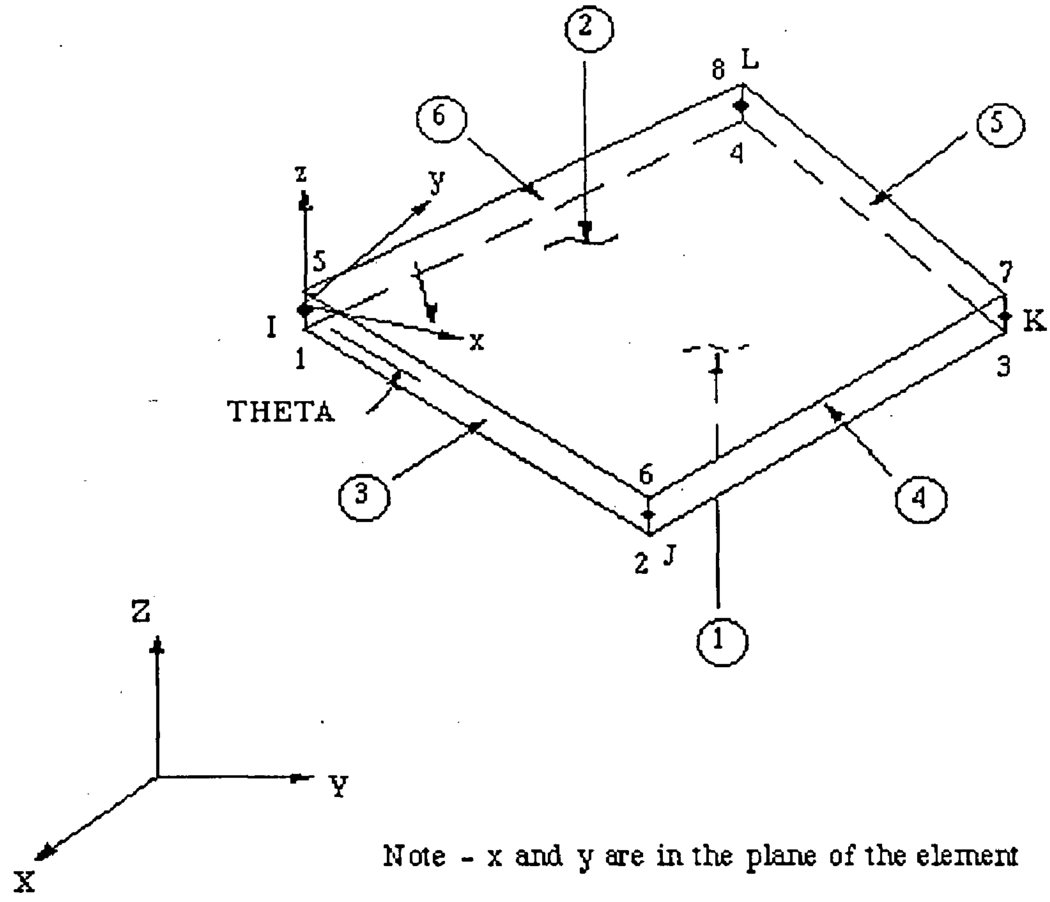

Figure 2.5 SHELL63 Elastic Shell

2.2 NONLINEAR MODELING

Nonlinear modeling of shear walls can be done using following elements.

2.2.1 Modeling using SOLID 65

SOLID 65 element is used for the three dimensional modeling of solids with or without reinforcing bars (Figure 2.6) in ANSYS. Solid is capable of cracking in tension and crushing in compression. In concrete applications, the solid capability of the element may be used to model the concrete while the rebar capacity is available for the modeling reinforcing behaviour. The element is defined by eight nodes having three degrees of freedom at each node - translations in the nodal x, y, and z directions. Up to three different rebar specification may be defined.

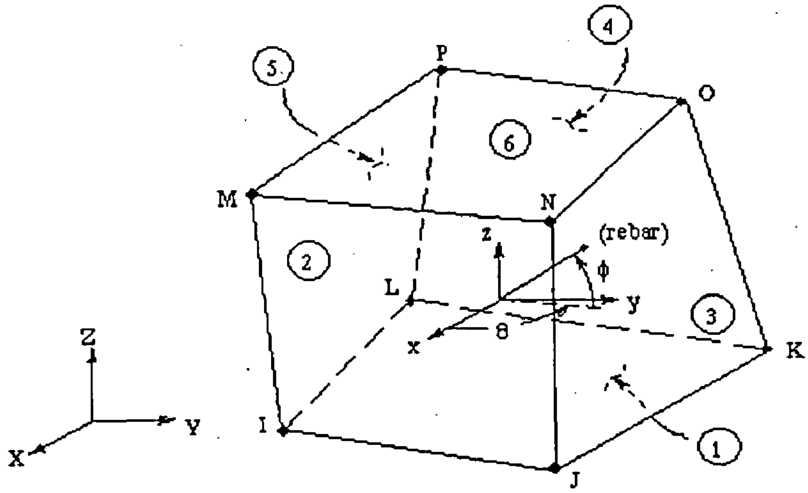

Figure 2.6 SOLID 65 3D Reinforced Concrete Solid Element (10)

2.2.2 Modeling using Perform Shear Wall element

Perform shear wall element is the element available in RAM Perform 2D and RAM Perform 3D. These elements acts as a deep beam element, with bending, axial and shear deformations. For each element we have to specify a shear component and axial bending component. The current version of RAM Perform 3D includes the following components.

(1) Shear components

(a) Elastic

(b) Inelastic

(2) Axial bending components

(a) Elastic

(b) Inelastic, based on fiber representation of the cross section.

For this component we must specify properties for one or more steel and/or concrete material components.

Strength of wall can be checked with the help of strength section. The current version of RAM Perform 3D includes the following strength section for shear wall.

(1) P-M strength section, steel type

(2) P-M strength section, concrete type

(3) Shear strength section

A shear wall element does not have to be rectangular, but it must not be highly distorted. Each element must have clear longitudinal and transverse directions. The following are some key points in modelling of shear walls using Perform shear wall elements:

(1) Each element connects 4 nodes and has 6 degrees of freedom at each node.

(2) Longitudinal in-plane behavior is the most important. Along this direction the element is essentially a deep beam that can be inelastic in bending and/or in shear.

Transverse in-plane behavior is secondary, and is assumed to be elastic. Out-of-plane bending is also secondary, and is assumed to be elastic.

(3) For the purposes of calculating the element stiffness, the cross section depth is assumed to be constant along the element length, based on the element width at its mid-height.

(4) If we specify elastic components for both bending and shear, the element is elastic. If we specify an inelastic shear material and an inelastic fiber cross-section, the element is inelastic in both bending and shear. [22]

2.2.3 Modeling using Perform General Wall element

Perform shear wall element is also available in RAM Perform 2D and RAM Perform 3D. These elements are used for the analysis of complex reinforced concrete walls with irregular openings ("punched" or coupled walls). Each element has 4 nodes with six degree of freedom at each node. In these eight are associated with in-plane deformations these are shown in Figure 2.7. There are also out-of-plane bending deformations, but these are of secondary importance. To modal bending, shear and diagonal compression behaviour, an element consists of five parallel layers. In an actual wall the concrete is in a state of multi-axial stress. For example, there could be combined vertical compression, horizontal tension and shear. The inelastic behavior of a material under multi-axial stress is much more complex than its behavior under uniaxial stress. This is especially true for concrete. The General Wall element does not consider multi-axial stress. Instead it separates the various aspects of behavior into layers with uniaxial stress in each layer.

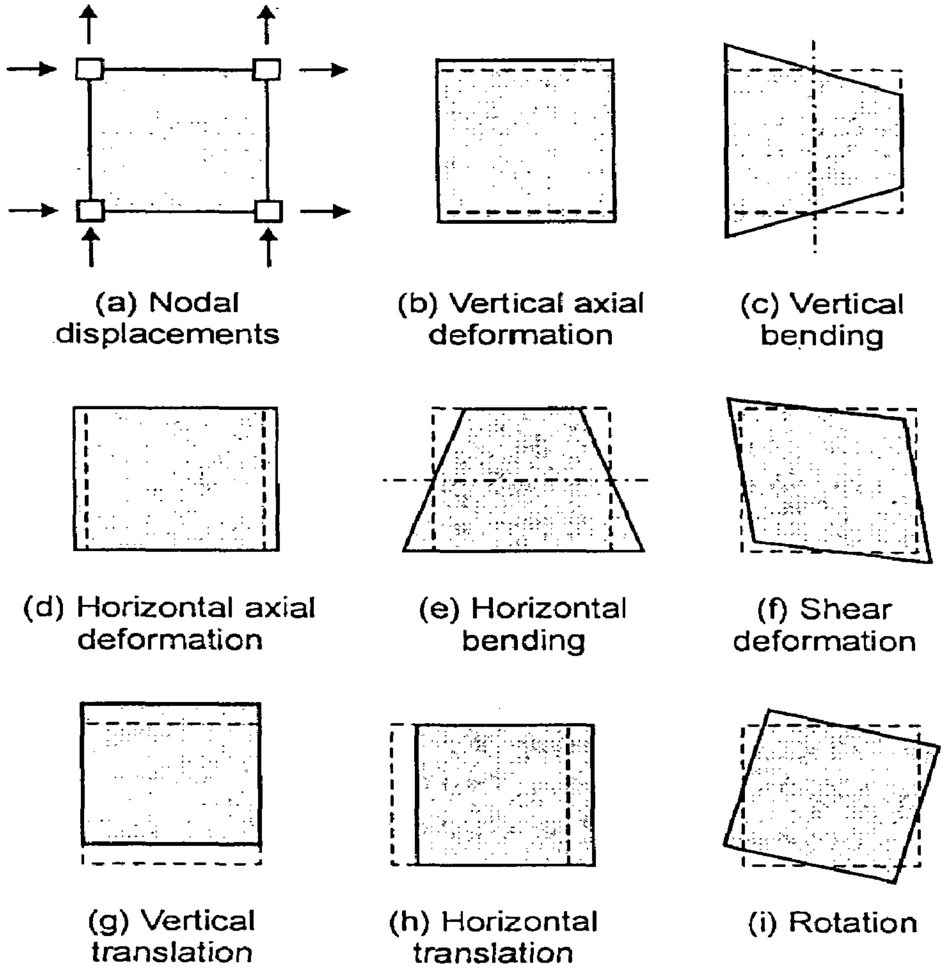

Figure 2.7 Elements In-plane Rotation [22]

NUMERICAL STUDIES

3.1 PARAMETERS OF THE BUILDING

The building used for the study is a G + 9 office reinforced concrete building. Its plan dimensions are shown in Figure 3.1. Building is symmetric in longitudinal direction but in lateral direction it is asymmetric. The analysis and design is considered for the gravitational load coming on the frames of the building and seismological load (earthquake load) for zone IV. The soil conditions are assumed medium and it is supported on raft foundation. Importance factor of building is taken as 1 and reduction factor is taken as 5 for calculation of base shear. Wind load is not considered, as it is very small. Other data which is used for the study is as given in Table 3.1 and dead load[15] is given as per Table 3.2. Imposed load[16] is distributed as per yield line theory and loaded as triangular and trapezoidal load on the beams. Seismic weight is the sum of Dead load and 0.5 times the Live load. All the values are taken from corresponding IS codes.

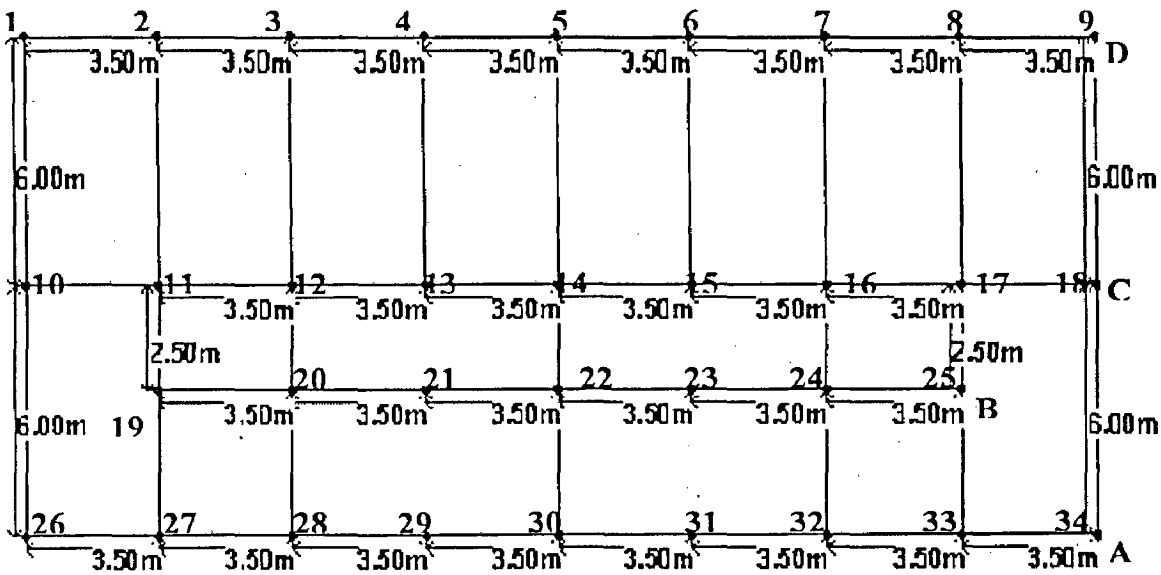

Figure 3.1 Plan of the Building

Table 3.1 Design data for the building

Type of structure	Reinforced concrete structure (G+9)
Zone	IV
Reduction factor	5
Importance factor	1
Soil condition	Medium
Foundation	Raft
Plan size	28mX12 m
Height of building	32m
Story height	3m(other than ground floor)
Ground floor height	4m
Height of parapet wall	1m
Depth of slab	150 mm(including flooring)
External wall	230 mm thick including plaster
Internal wall	150 mm thick including plaster
Shear wall	230 mm
Mud phuska	16.67 kN / m^3
Brick masonry	20.00 kN / m^3
Grade of concrete	M25
Grade of steel	Fe-415
Ceiling and finishing	0.33 kN / m^2
Live Load	4 kN/m^2 for all floor and 1.5 kN/m^2 for roof
RC material	25kN/m^3
E for Steel	$2x10^8$ kN/ m^2
E for Concrete	$2.5x10^7$ kN/ m^2
Poisson ratio for Concrete	0.17

Table 3.2 Dead load

S.No.	Dead Load	Loading Type	Description
1	Beams	Uniform	According to yield line theory and as per unit wt of RC material in Table 3.1
2	Column	Nodal	As per unit wt of RC material in Table 3.1
3	Masonry walls	Uniform	As per unit wt of Brick masonry in Table 3.1
4	Shear walls	Uniform	As per unit wt of RC material in Table 3.1
5	Slab	Trapezoidal or Triangular	Loading as per yield line theory and unit weight as per Table 3.1
6	Mud phuska	Trapezoidal or Triangular	Loading as per yield line theory and unit weight as per Table 3.1
7	Ceiling and Finishing	Trapezoidal or Triangular	Loading as per yield line theory and unit weight as per Table 3.1

The following load combination as given in IS: 1893-2002[14] (Part 1) are considered for the design purpose:

1. 1.5 (DL + IL)
2. 1.2 (DL + IL + EL)
3. 1.2 (DL + IL - EL)
4. 1.5 (DL+EL)
5. 1.5 (DL-EL)
6. 0.9 DL + 1.5 EL
7. 0.9 DL - 1.5 EL

A total number of 17 load combinations are used for the analysis and design.

3.2 LINEAR ANALYSIS AND DESIGN

Modelling of bare frame for the above parameters is done in STAAD Pro. The value of loads which we applied on the building are shown in Table 3.3. Initially sections are selected by rough calculation for the maximum expected axial load for columns and maximum expected bending moment in beams. The final cross-sections of columns and beams are selected to be the minimum required sections for the given loading. The rigid diaphragm action of the slabs has been modelled by fictitious rigid diagonal members at every floor level as shown in Figure 3.2. The stiffness of the fictitious members is considered 1000 times of a typical beam. Seismic weight of building is calculated as per IS 1893:2000. For the present loading conditions seismic weight is dead load with 50% of live load[15]. Response spectrum is used as per Indian code for the above site and soil; conditions. The bare frame has been designed for the base shear and member forces obtained by modal analysis.

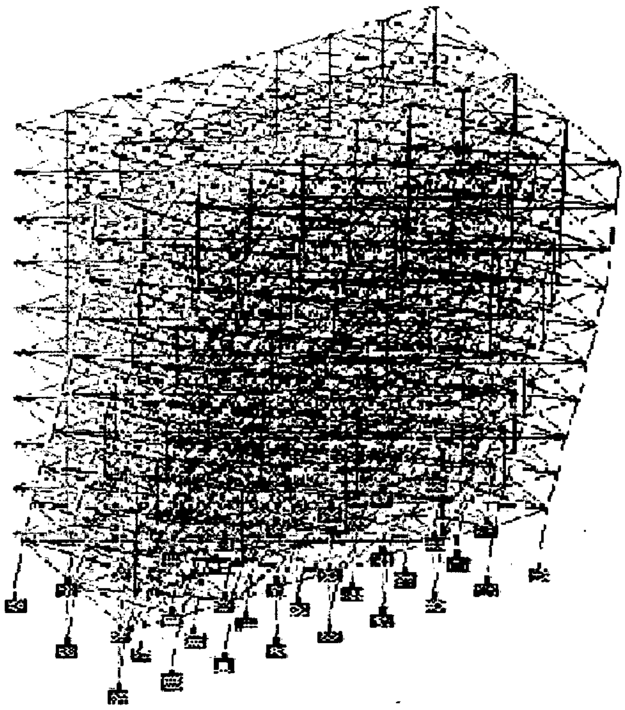

Figure 3.2 (a) First Mode Shape (Transverse)

The time periods of building for first 6 modes are 2.965, 2.647, 2.381, 1.002, 0.9091, and 0.8265 sec. consequently. 95 % mass is participating in these 6 modes. The first 6 mode shapes are shown in Figure 3.2 (a)-(f). Base shear for the building, obtained from analysis is 541 kN in transverse direction and 453.5 kN in longitudinal direction. The final cross sections of beams are 300 mm x 300 mm. The column sizes for first three storeys are 400 mm x 400 mm and for middle three storeys the column sizes are

350 mm x 350 mm. For the upper four storeys the column sizes are 300 mm x 300 mm.

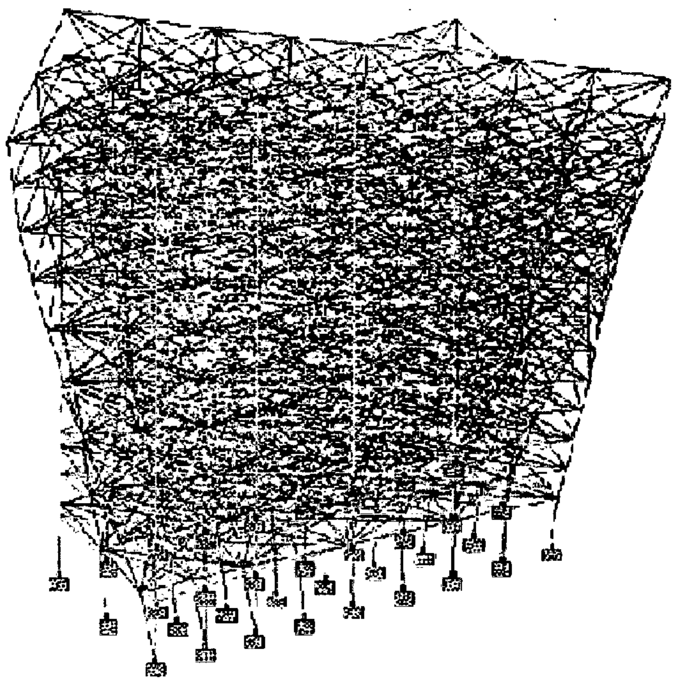

Figure 3.2 (b) Second Mode Shape (Torsion)

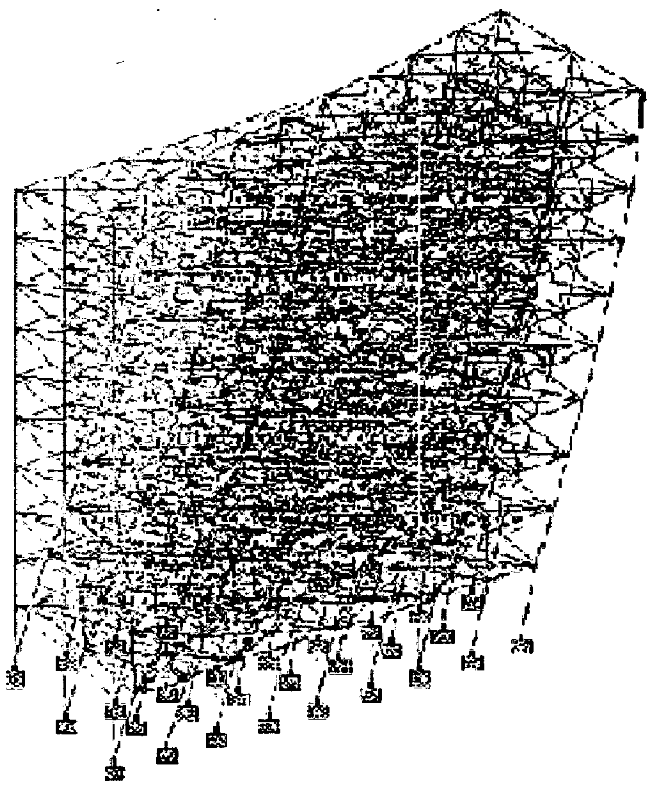

Figure 3.2 (c) Third Mode Shape (Longitudinal)

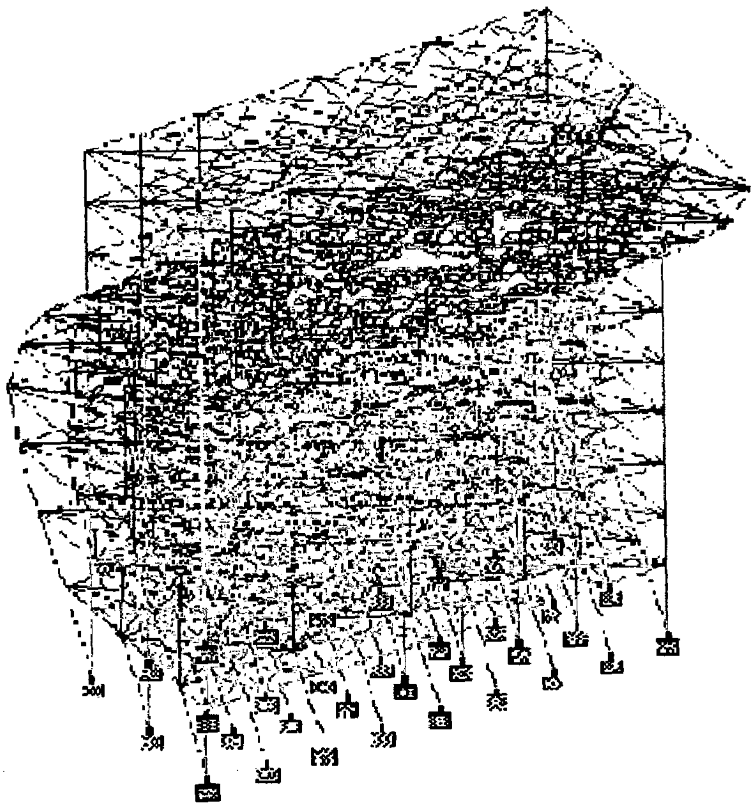

Figure 3.2 (d) Fourth Mode Shape (Transverse)

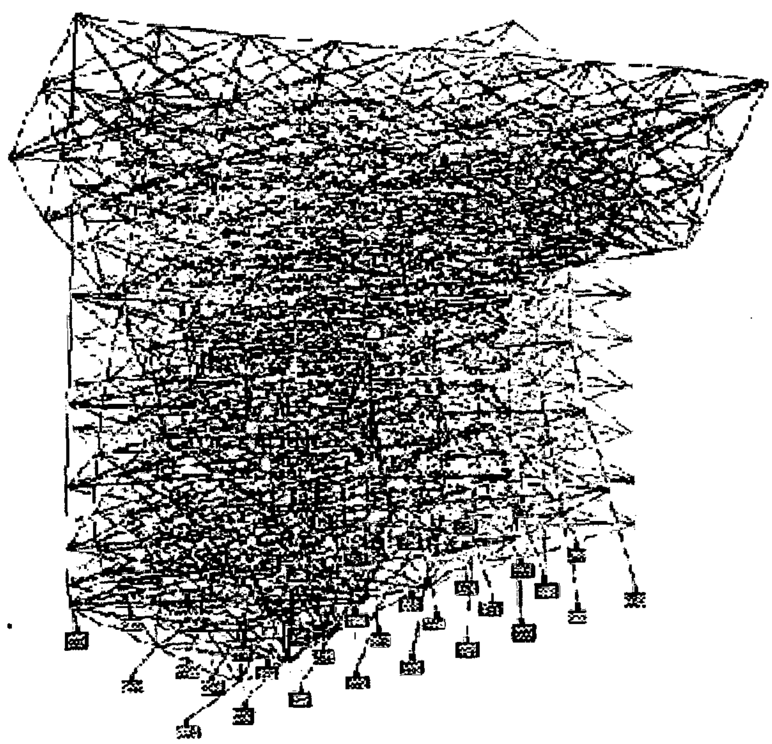

Figure 3.2 (e) Fifth Mode Shape (Torsion)

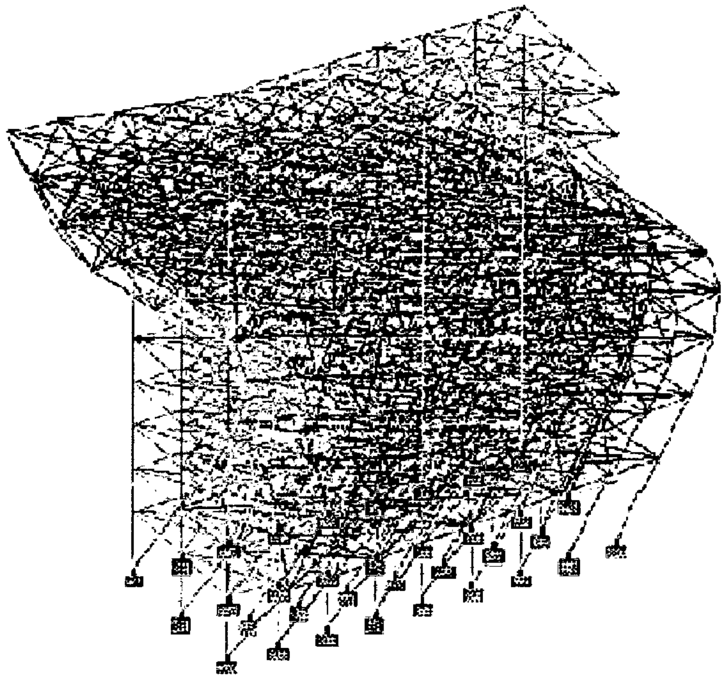

Figure 3.2 (f) Sixth Mode Shape (Longitudinal)

Table 3.3 Calculation of dead load, live load and seismic load

Description	Nos.	L (m)	B (m)	T (m)	H (m)	Intensity	Total Load (kN)
Self Weight							
Beam	10	201	0.3	0.3		2.25 kN/m	4522.5
Slab	10	28	12	0.15		3.75 kN/m2	12600
Mud Pussca	1	28	12	0.15		2.5 kN/m2	840
Finishing & celling	10	28	12			0.33 kN/m2	1108.8
Interior wall	9	121		0.15	2.55	7.65 kN/m	8330.85
Exterior Wall	9	80		0.23	2.55	11.73 kN/m	8445.6
Parapet wall	1	80		0.15	1	3 kN/m	240
Column(1)	34	0.4	0.4		4	16 kN	544
Column(2 - 3)	68	0.4	0.4		3	12 kN	816
Column (4 - 6)	102	0.35	0.35		3	9.1875 kN	937.125
Column (7 - 10)	136	0.3	0.3		3	6.75 kN	918
						TOTAL DEAD LOAD	**39303**
Live Load							
Roof	1	28	12			1.5 kN/m2	504
Floor (1 - 9)	9	28	12			4 kN/m2	12096
						TOTAL LIVE LOAD	**12600**
						Seismic Weight	**45351**

Modelling of building with different shear wall area as percentage of floor area is done in STAAD using wide column modelling. At each floor level a very stiff beam is provided across the shear walls to model the finite width of shear walls. Stiffness of these beams is provided 1000 times the stiffness of a typical beam. Analysis of these models is done using STAAD Pro and Design of shear walls is done manually as per IS 13920. Four buildings with increasing shear wall area (1.8%, 3.6%, 5.4% and 7.2% of floor area, respectively) have been considered in the present study. The plans of these buildings have been shown in Figure 3.3(a) to (d). The percentage reinforcement in shear walls for building 2, 3, 4, 5 is obtained as 0.7%, 0.5%, 0.45% and 0.4%, respectively. Sizes of beams and columns required with these shear walls have been obtained by analysis and design using STAAD Pro. The obtained sizes are shown in Tables 3.4 and 3.5.

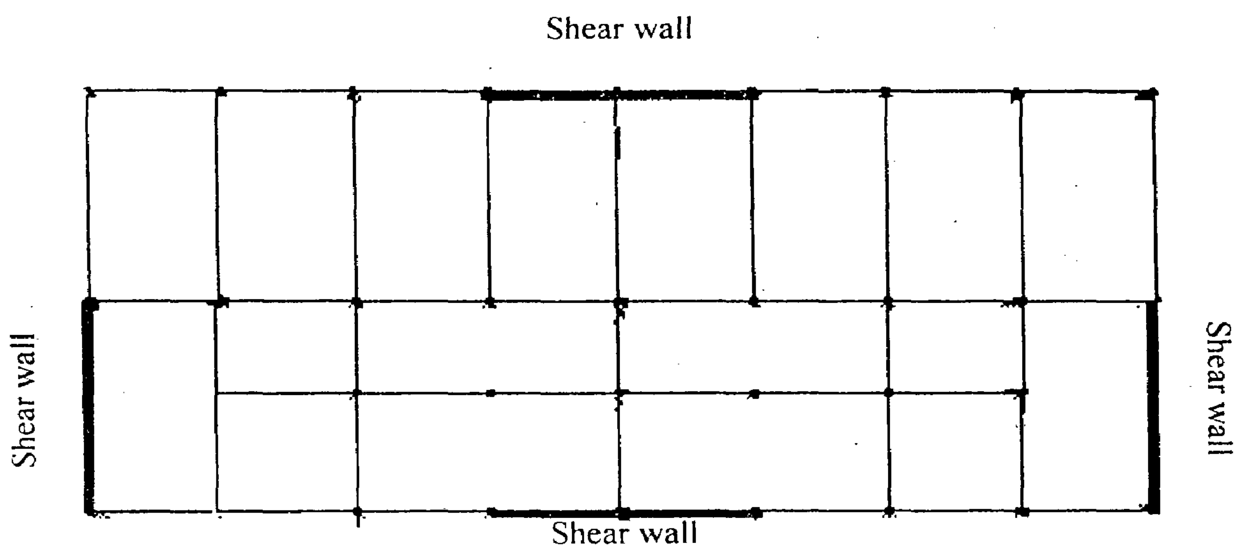

Figure 3.3 (a) Building 2

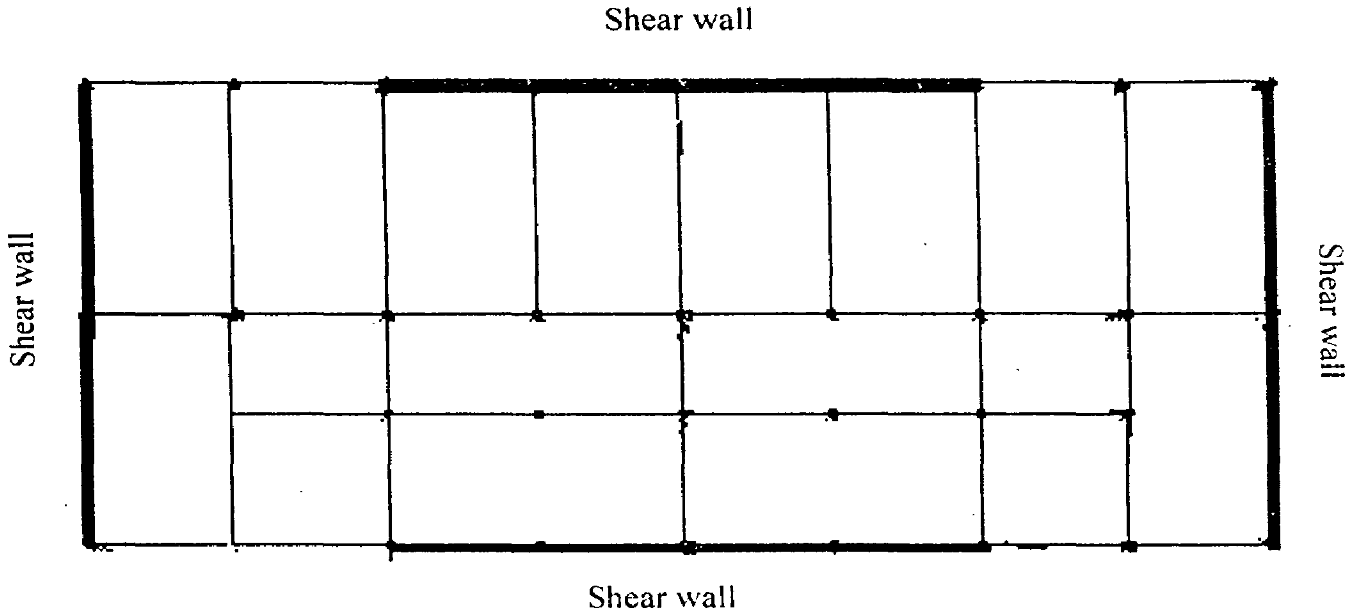

Figure 3.3 (b) Building 3

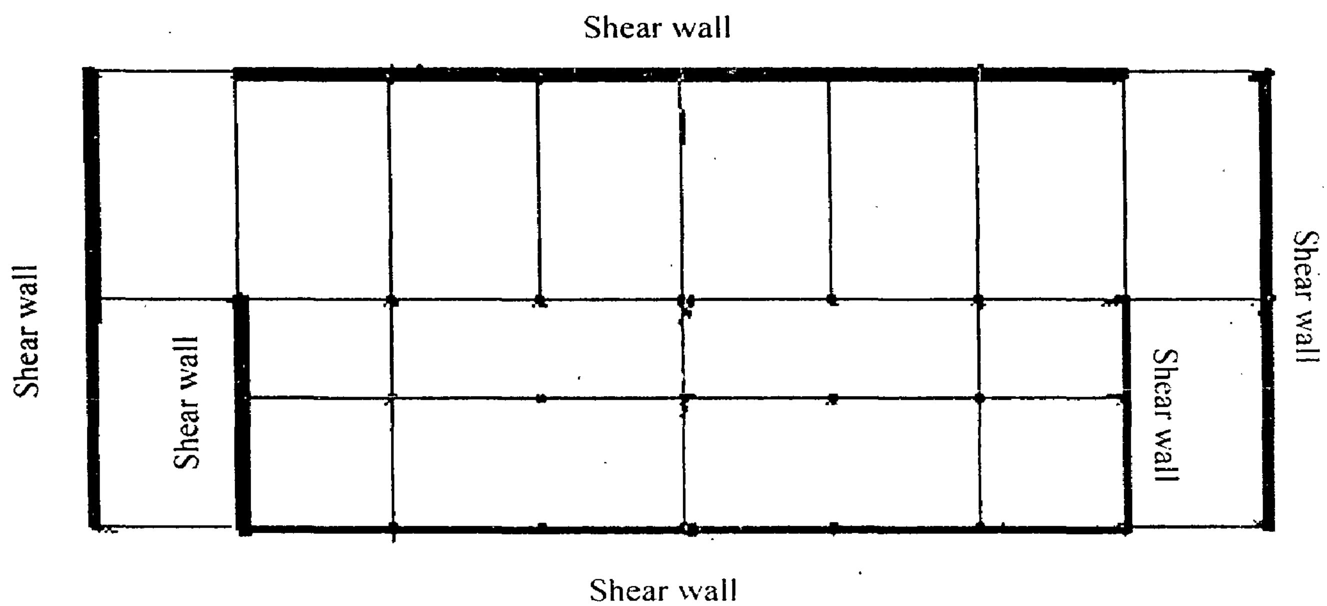

Figure 3.3 (c) Building 4

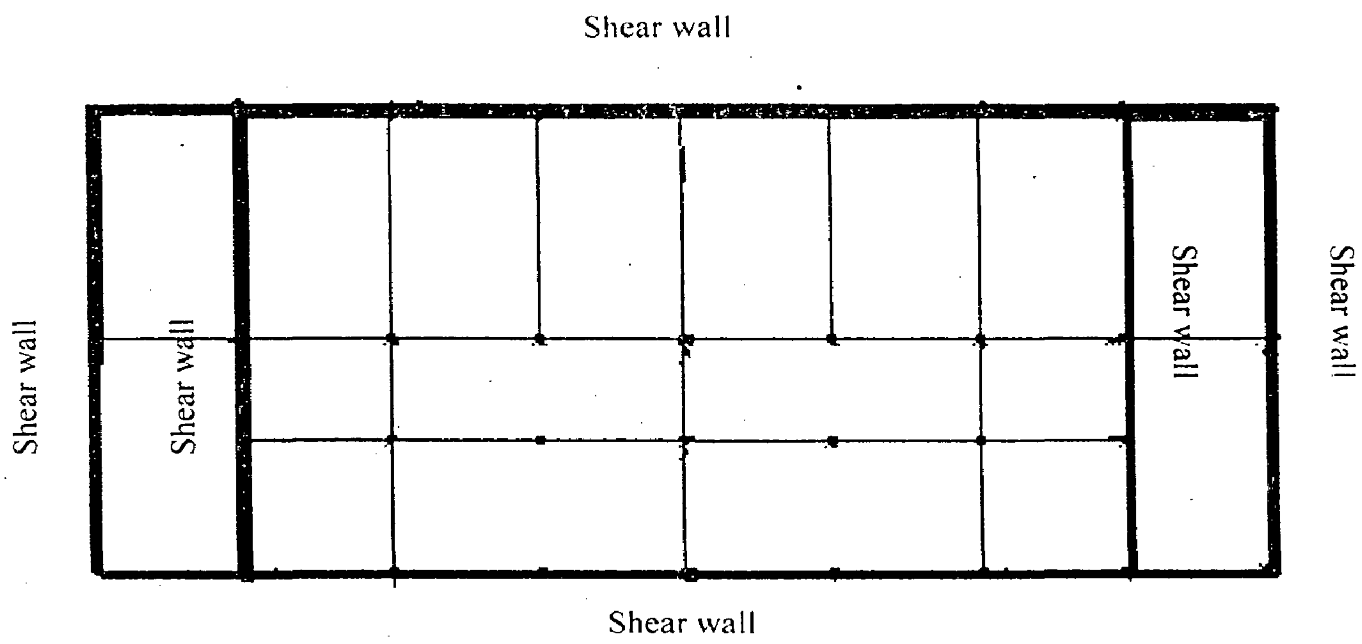

Figure 3.3 (d) Building 5

3.3 NON-LINEAR MODELLING

After designing the buildings using linear analysis, the non-linear modelling of the buildings is done in RAM Perform 3D, using non linear elements for beams, columns and shear walls as described below. The slab is modelled as a rigid horizontal diaphragm. Fixed supports are assumed at ground level.

3.3.1 Beam element

To define the properties for a beam element, first a frame compound component is defined using stiff end zones and FEMA Concrete Beam components with small axial forces and zero bending moment about the vertical axis.

FEMA 273 concrete beam is the basic component used to implement the chord rotation model of the beams. The chord rotation model is the simplest model, with the most limitations. The basic model is shown in Figure 3.4 this is a symmetrical beam with equal and opposite end moments and no loads along the beam length. This model requires the nonlinear relationship between the end moment and end rotation, the end rotation is the rotation from the chord, which eliminates rigid body rotations. A major advantage using this model is that FEMA-273 gives specific properties, including end rotation capacities.

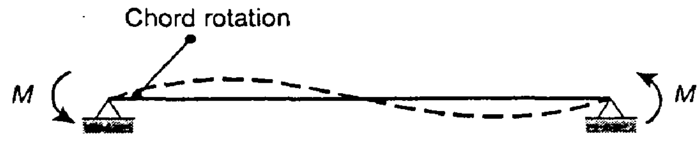

Figure 3.4 Chord Rotation Model

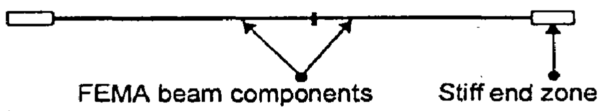

Figure 3.5 Perform 3d Compound Component Model for Beam

Figure 3.5 shows a typical frame compound component for beam. The key parts of this model are the FEMA beam components. These are finite length components with nonlinear properties. The model has two of these components, to allow for the case where the strengths are different at the two ends of the element. Strictly speaking, the chord rotation model applies only to a symmetrical beam member, with equal strengths at both ends and an inflection point at mid-span.

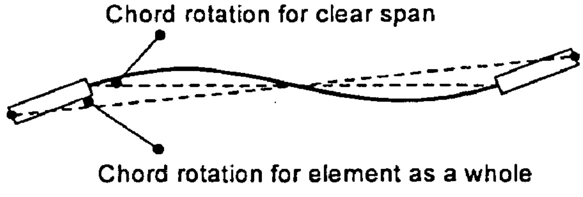

Figure 3.6 Deformed Shape of Compound Component

Figure 3.6 shows the deformed shape for an element with stiff end zones. The chord rotation for the clear span between end zones is larger than for the element as a whole. In RAM-Perform 3D, the rotation used for the chord rotation model is the rotation for the clear span. The default end zone set in RAM which can be used for beam and column elements has a stiffness that is 10 times larger than the body of the component, and an "auto" length that is obtained from the dimensions of the adjacent beams and columns. The beam-to-column connections have been modelled as rigid connections. The deformation capacities for a FEMA beam component, are defined in terms of element end rotations. RAM-Perform 3D calculates the hinge rotations, and converts these to element end rotations for the demand-capacity calculation.

3.3.2 Column element

Column element has been modelled as a Perform 3D frame compound component, which can have large axial forces and biaxial bending moment. Frame compound component for column element contains default end zones and FEMA-273 type concrete column. The component properties have been obtained form FEMA 273 document.

3.3.3 Shear wall element

The different available shear wall elements have already been explained in Chapter 2. As shear wall elements have no in-plane rotational DOF at nodes, modelling of connection of adjoining beams is difficult. For this purpose a rigid beam has been imbedded across shear walls and the adjoining beams are connected to this beam as shown in Figure 3.7.

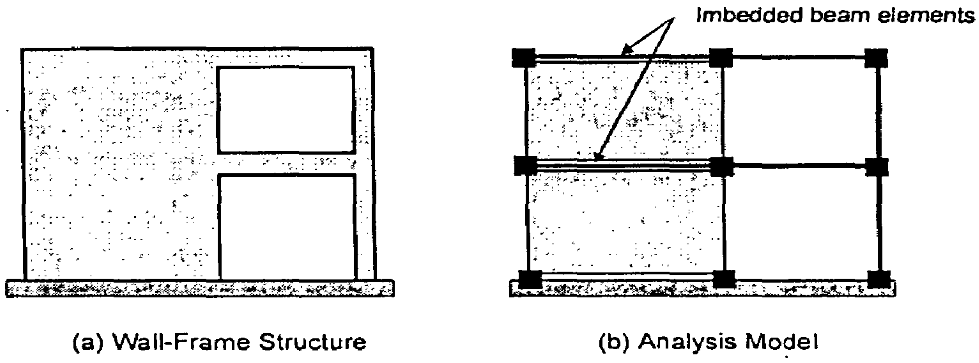

Figure 3.7 Modelling of Connection of Beam with Shear Wall

3.3.4 Component properties

The elements modelled in RAM Perform 3D are to be assigned their nonlinear component properties before the actual nonlinear analysis. These component properties depend upon the design results obtained from STAAD. These are shown in Table 3.4 and 3.5.

The sizes and reinforcement area obtained from the linear analysis and design, has been used to find out the performance levels for beams, columns and shear walls from FEMA 273 document. Three performance levels have been used in the present analysis, viz. Immediate Occupancy level (IO), Life Safety level (LS) and Collapse Prevention level (CP).(7)

Ultimate capacities in axial load, shear force and bending moment have been obtained for the provided sizes and reinforcement using a software developed for this purpose.

The force deformation relationship has been assumed Elastic Perfectly Plastic (Figure 3.8).

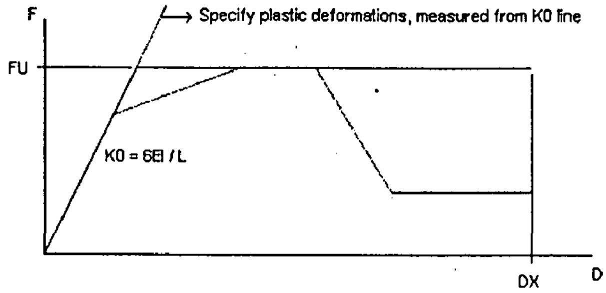

Figure 3.8 Typical Force Deformation Relationship

Table 3.4 Beam cross-sectional properties

L (m)	B (m)	D (m)	Area of steel (%) at End 1 Top	at End 1 Bottom	at End 2 Top	at End 2 Bottom	Ultimate moment (kN m)	Design shear force (kN)
(A) Bare frame								
2.50	0.30	0.30	1.12	1.05	1.12	1.05	132.42	86.00
3.50	0.30	0.30	1.12	1.05	1.12	1.05	138.00	89.00
6.00	0.30	0.30	2.68	1.57	2.68	1.57	200.41	120.00
(B) Building 2								
2.50	0.30	0.30	0.45	0.25	0.45	0.25	40.51	75.00
3.50	0.30	0.30	1.40	0.39	1.40	0.39	115.51	85.00
6.00	0.30	0.30	2.68	1.57	2.68	1.57	220.83	128.00
(C) Building 3								
2.50	0.30	0.30	2.18	1.34	2.18	1.34	33.42	45.00
3.50	0.30	0.30	1.04	0.34	1.04	0.34	91.61	76.00
6.00	0.30	0.30	0.34	0.25	0.34	0.25	183.42	120.00
(D)Building 4								
2.50	0.30	0.30	2.18	1.34	2.18	1.34	33.42	45.00
3.50	0.30	0.30	1.34	0.34	1.34	0.34	69.49	70.00
6.00	0.30	0.30	0.34	0.25	0.34	0.25	193.14	120.00
(E)Building 5								
2.50	0.30	0.30	2.18	1.34	2.18	1.34	33.42	45.00
3.50	0.30	0.30	1.34	0.34	1.34	0.34	64.23	64.00
6.00	0.30	0.30	0.34	0.25	0.34	0.25	204.33	120.00

Table 3.5 *Column cross-sectional properties*

(A)Bare frame

Storey Nos.	Column Size.(m)	Column Nos	Pu(kN)	Mu(kN-m)	Pt(%)	PT(kN)	PC(kN)	PB(kN)	MB(kN)
1,2,3	0.4x0.4	2 - 8,10 -18	3130	120	2.83	1879.12	4213	970	276.5
	0.4x0.4	19 - 25	2700	102	1.42	942.88	3442.8	1075	161.5
	0.4x0.4	1,9,26 - 34	2450	97	0.85	564.4	3131.25	1200	106.5
4,5,6	0.35x0.35	2 - 8,10 - 18	3130	120	2.83	1438.7	3226	723	178
	0.35x0.35	19 - 25	2700	102	1.42	721.9	2636	855	105
	0.35x0.35	1,9,26 To 34	2450	97	0.85	432.12	2397.5	894	69
7,8,9	0.3x0.3	19 - 25, 27 - 33	1156	45	0.9	336.15	1776.92	650	44.2
	0.3x0.3	1,10,26,9,18,34	1500	75	1.6	597.6	1992.3	622	69.3
	0.3x0.3	2 - 8,11 - 17	1670	78	2.52	941.22	2275.2	510	97.2
10	0.3x0.3	19 - 25, 27 - 33,10,17	1152	37	0.9	336.15	1776.92	650	44.2
	0.3x0.3	26, 34	1585	76	2.2	321.7	2176.7	540	88
	0.3x0.3	1 - 9,11 - 17	1830	99.5	3.02	1128	2428.86	490	111.85

(B) Building 2

Storey nos.	Column Size.(m)	Column Nos	Pu(kN)	Mu(kN-m)	Pt(%)	PT(kN)	PC(kN)	PB(kN)	MB(kN)
1	0.4x0.4	C,D	3074	84.3	2.83	1879.12	4212.81	998.31	276.5
	0.4x0.4	B	2415	72	1.41	942.88	3437	1180	160
	0.4x0.4	A	2125	86	0.79	524.56	3098	1195	99
2,3	0.4x0.4	C	2800	79	2.26	1500	3902	1020	232
	0.4x0.4	D	2600	92	1.7	1128	3596	1065	185.5
	0.4x0.4	A,B	2125	86	0.79	524.56	3098.5	1195	99
4,5,6	0.35x0.35	C	2200	68	2.59	1316	3126	728	167

	0.35x0.35	D	2100	71	1.85	940	2816	780	128.5
	0.35x0.35	A,B	1702	70	1.03	523	2473	890	81.5
7,8,9	0.3x0.3	D	1840	98	1.31	665.67	2590	875	98
	0.3x0.3	A,B,C	1540	73	0.66	335.52	2318.5	920	55
10	0.3x0.3	C,D	2100	160	2.95	1500	3277	720	183
	0.3x0.3	A	1750	68	1.03	523.62	2473	900	81
	0.3x0.3	B	1600	73	0.66	335.52	2318	920	60

(C) Building 3

Storey Nos.	Column Size.(m)	Column Nos	P_u (kN)	M_u (kN-m)	P_t (%)	PT (kN)	PC (kN)	PB (kN)	MB (Kn-m)
1	0.35x0.35	C,D	2500	64	3.08	1565	3330	730	191
	0.35x0.35	B	2100	54	1.97	1001	2866.09	780	134.5
	0.35x0.35	A	1710	.43	1.03	523.63	2473	897	81
2,3	0.35x0.35	D,B	2200	56	2.22	1128.6	2970	770	147.5
	0.35x0.35	C	2000	48	1.6	813.4	2711.3	816	115
	0.35x0.35	A	1700	53	0.74	379.7	2351.5	91.5	61
4,5,6	0.3x0.3	B,C,D	1650	40	2.5	933.75	2269	535.5	96.6
	0.3x0.3	A	1200	45	0.89	332.415	1773.85	694	44.2
7,8,9	0.3x0.3	A,B,C,D	1260	58	1.01	377.24	1810.75	660	48.5
10	0.3x0.3	A,B,C,D	1500	61	1.8	672.3	2053	622	75.09

(D) Building 4

Storey Nos.	Column Size.(m)	Column Nos	Pu(kN)	Mu(kN-m)	Pt(%)	PT(kN)	PC(kN)	PB(kN)	MB(kN)
1	0.35x0.35	C	2320	57.5	2.59	1276	3092	730	163
	0.35x0.35	B	1800	45	1.31	666	2590	875	98.5
2	0.35x0.35	C	2000	52	1.65	940.53	2816	781	128
	0.35x0.35	B	1660	68	1.03	523.64	2473	897.11	80.5
4,5,6	0.3x0.3	C	1500	50	1.79	668.567	2050	598.51	75
	0.3x0.3	B	1250	50	0.89	332.415	1774	660	44.2
7,8	0.3x0.3	C,B	1250	55	1.01	377.24	1811	660	48.6
9,10	0.3x0.3	C,B	1400	57	1.6	672.3	2054	598	75.2

(E) Building 5

Storey nos.	Column Size.(m)	Column Nos	Pu(kN)	Mu(kN-m)	Pt(%)	PT(kN)	PC(kN)	PB(kN)	MB(kN)
1	0.35x0.35	C	220	547.5	2.19	1251	2898	712	154
	0.35x0.35	B	1700	43	1.11	601	2370	820	92.3
2	0.35x0.35	C	1800	50	1.45	900.53	2706	761	119
	0.35x0.35	B	1570	64	1.03	503.64	2421	797	74.5
4,5,6	0.3x0.3	C	1430	48	1.59	618.567	2000	568	70
	0.3x0.3	B	1200	48	0.74	302.415	1704	650	42.2
7,8	0.3x0.3	C,B	1050	51	0.89	337.24	1741	640	44.6
9,10	0.3x0.3	C,B	1300	52	1.31	632.3	2004	541	71

In the above Tables, P_u and M_u are the design axial and moment capacity of columns, respectively and P_t is area of steel provided in the column. *PT, PC,* and *PB* refer to member axial capacity in tension, compression and at balance point, respectively and *MB* refers to moment capacity at balance point.

Shear wall is modelled using inelastic steel and inelastic concrete material models. Tensile strength of steel has been considered as 415000 kN/m2 and maximum tensile strain as 0.2. For inelastic concrete properties, compressive strength has been considered as 167000 kN/m2 and compressive strain as 0.0035. Shear material in shear wall has been considered as elastic.

3.4 NON-LINEAR STATIC ANALYSIS

Nonlinear static pushover analysis has been done to estimate the performance levels of different building described above for different levels of ground shaking. In static pushover analysis it is first required to find out the full capacity of the structure by developing a pushover curve or capacity curve for the structure. The ground shaking level is defined in terms of demand diagram. In the present study four levels of ground shaking - Design Basis Earthquake (DBE), 1.2 x DBE, Maximum Considered Earthquake (MCE) and 1.2 x MCE have been considered. The response spectra for the four cases are shown in Figure 3.9.

In addition to these Earthquake Loads 1.2 times DL plus LL has also been applied. To apply the pushover load over the structure, nodes are defined at the geometric center of each floor. Two load patterns – Linear and Parabolic have been used for obtaining the pushover curves in both the directions (H1 – longitudinal, and H2 – transverse) of the buildings. IO, LS and CP limit states have been defined for beams, columns and drift of the building as a whole.

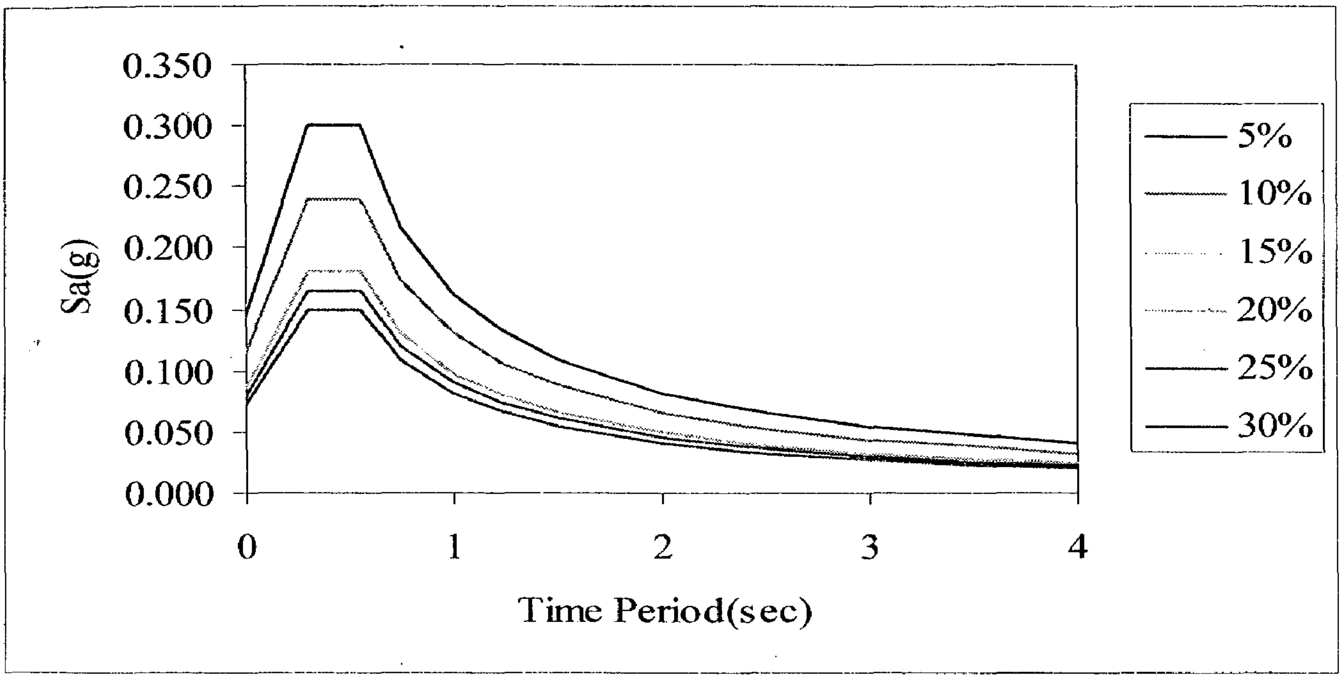

Figure 3.9(a) Response Spectra for DBE

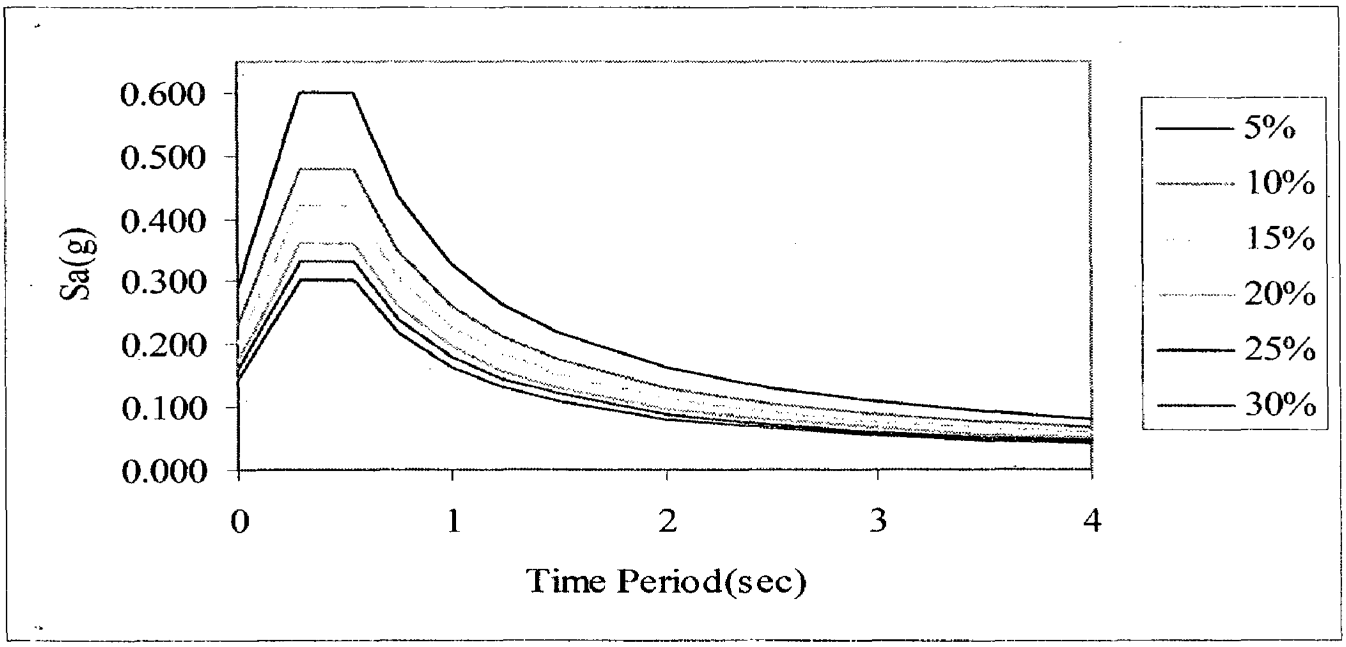

Figure 3.9(b) Response Spectra for MCE

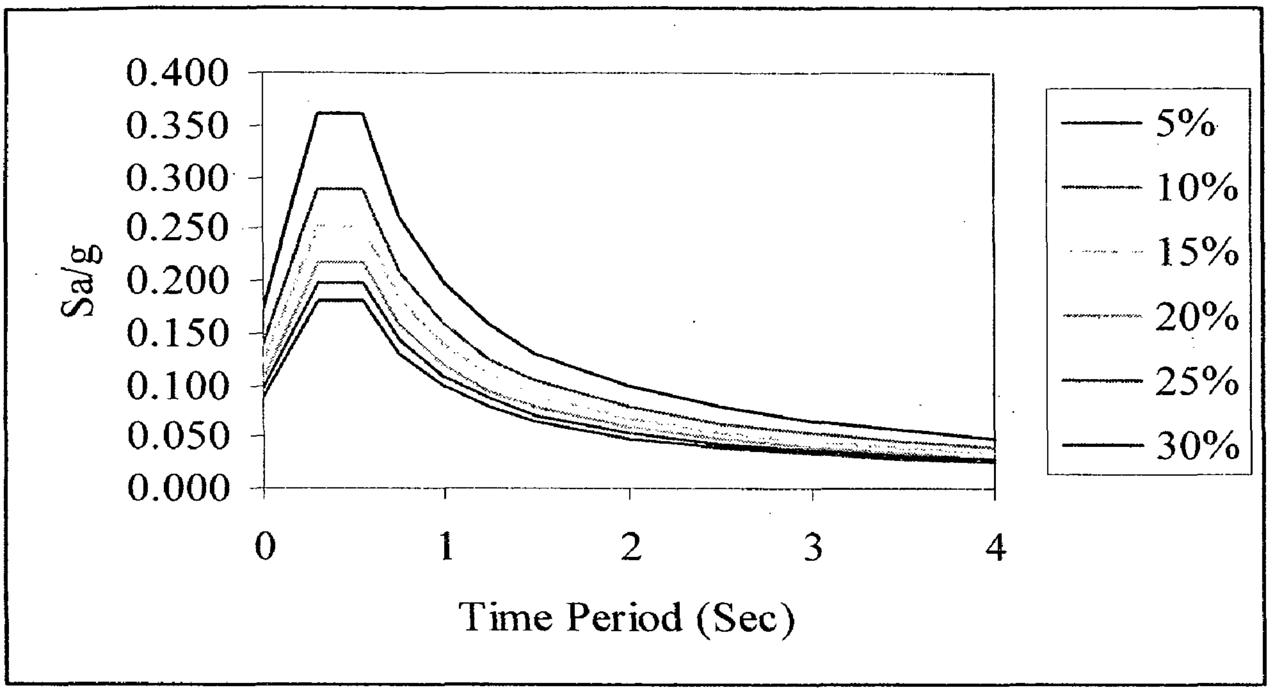

Figure 3.9(c) Response Spectra for 1.2DBE

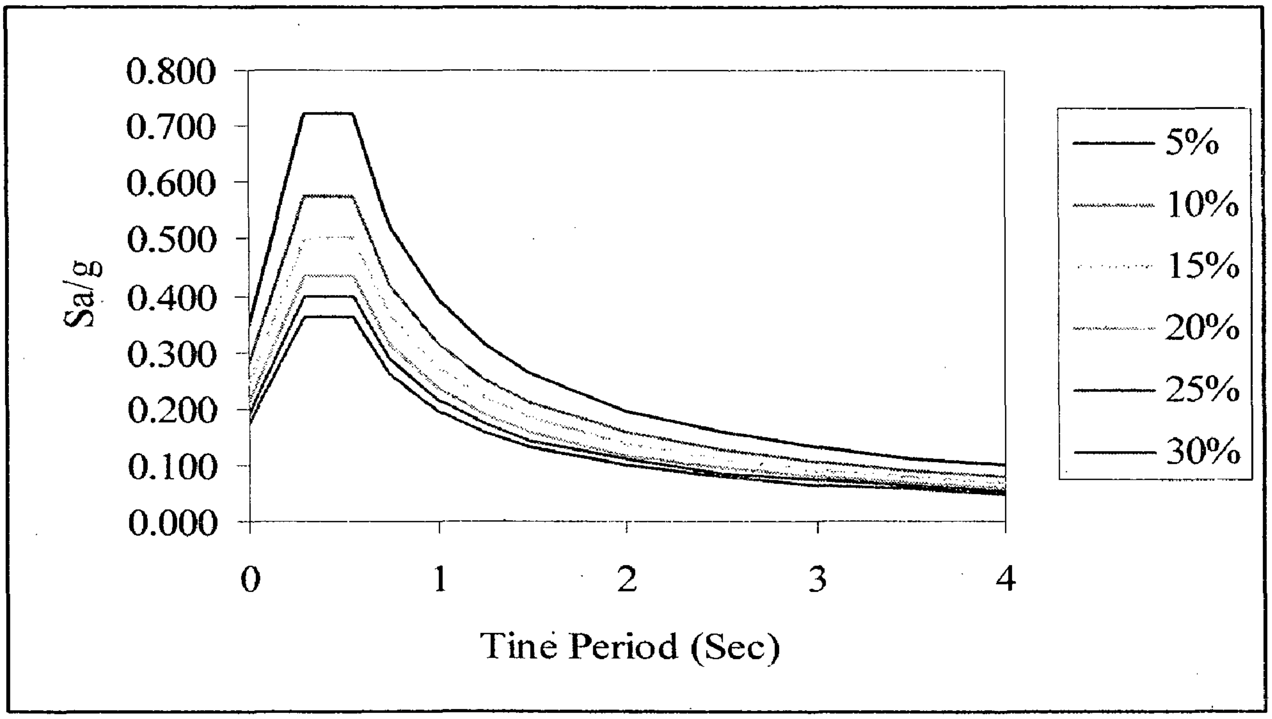

Figure 3.9(d) Response Spectra for 1.2MCE

3.4.1 Bare frame building

Figure 3.10 shows the Pushover curves for bare frame building (Building 1) for linear and parabolic loading patterns in *H1* (longitudinal) and *H2* (transverse) directions of the building. The points marked by star show the performance points for the ground shaking levels mentioned there.

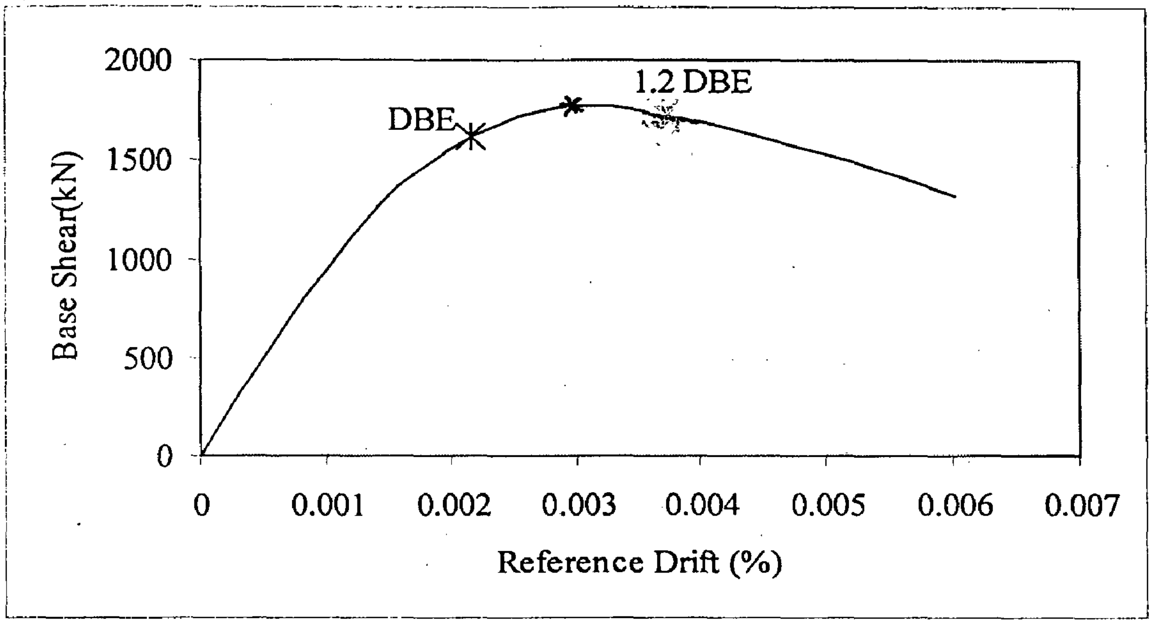

Figure3.10 (a) Pushover Curve for Linear Load Pattern in H1 Direction

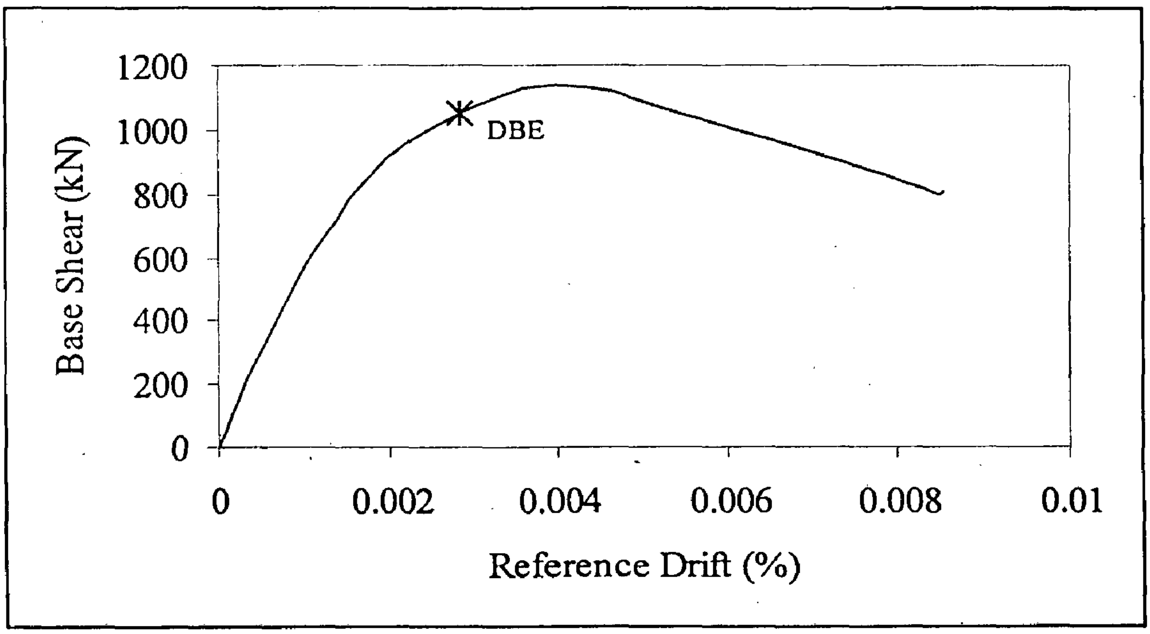

Figure3.10 (b) Pushover Curve for Linear Load Pattern in H2 Direction

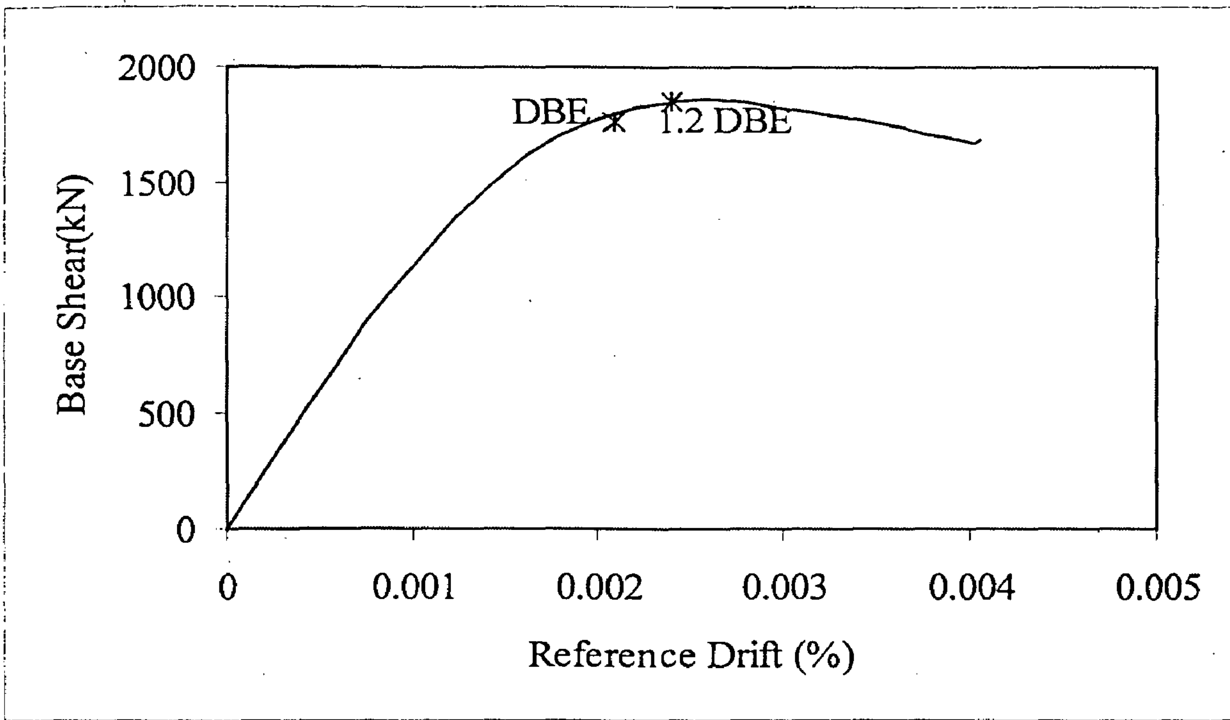

Figure3.10(c) Pushover Curve for Parabolic Load Pattern in H1 Direction

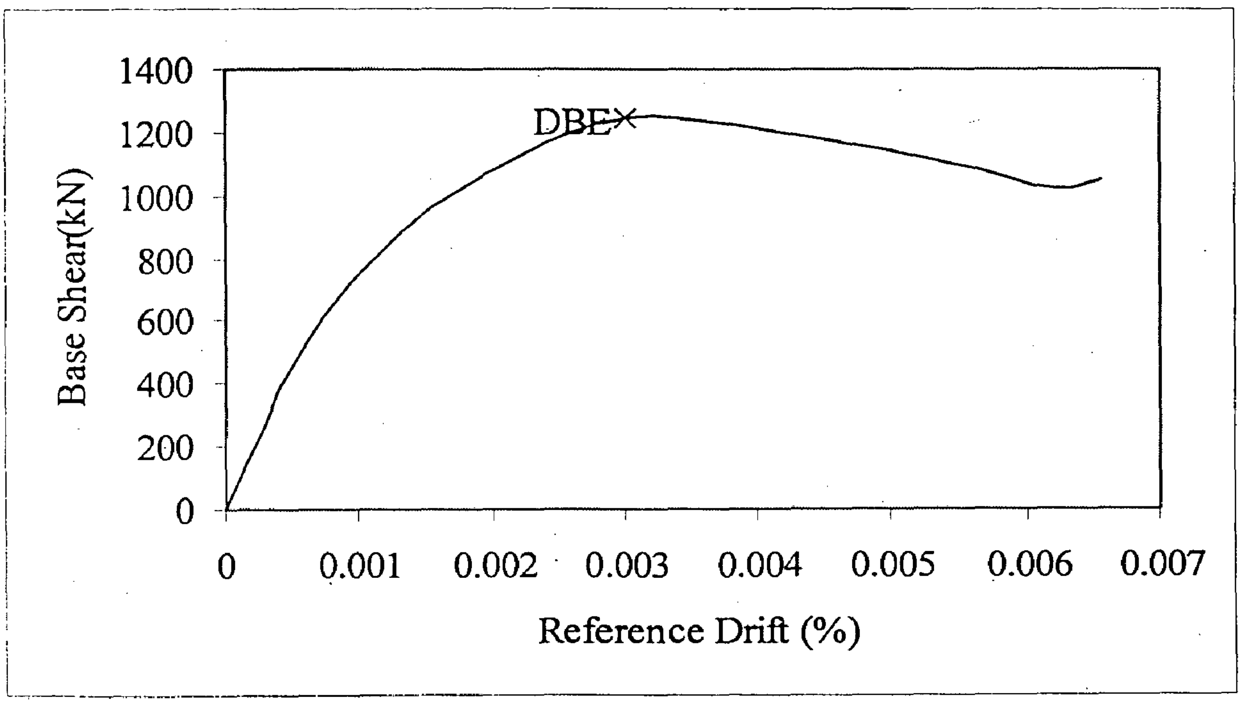

Figure3.10 (d) Pushover Curve for Parabolic Load Pattern in H2 Direction

It can be seen from the above results that for bare frame building there is no significant effect of loading pattern on building pushover curve. The performance points for linear and parabolic loading patterns in H1 direction occur at reference drifts equal to 0.0022 and 0.0021, respectively for DBE and at reference drifts equal to 0.0025 and 0.0024, respectively for 1.2 times DBE. Similar results have been obtained for H2 direction also. For bare frame building, performance points could not be obtained for MCE and 1.2 times MCE ground shaking levels, indicating that the building fails for these ground shaking levels. For further studies only parabolic loading pattern has been considered.

3.4.2 Effect of shear walls

Figures 3.11 and 3.12 show the Pushover curves for building with different percentage of shear wall area (Buildings 2, 3, 4, and 5) for parabolic loading patterns in H1 and H2 directions of the building, respectively. The points marked by star show the performance points for different levels of ground shaking. The detailed pushover curves, demand curves and performance points have been given in Appendix.

Table 3.6 summarizes the limit states corresponding to performance points of these buildings. It can be seen from the figures and table that bare frame building fails for MCE and 1.2 MCE level of ground shaking. By providing shear wall area equal to just 1.8% of floor area, the performance of the building has been improved remarkably and Immediate Occupancy level has been achieved, even for MCE, which is the desired performance level for post earthquake important structures. Similar performance levels have been achieved for buildings with shear wall area equal to 3.6% and 5.4% of floor area. For building with 7.2% shear wall area, the members remain elastic, even for MCE and such a high shear wall area will usually not be required. It can be concluded that shear wall area equal to 1-3% of floor area is sufficient for satisfactory seismic performance of building structures of 10 storey height.

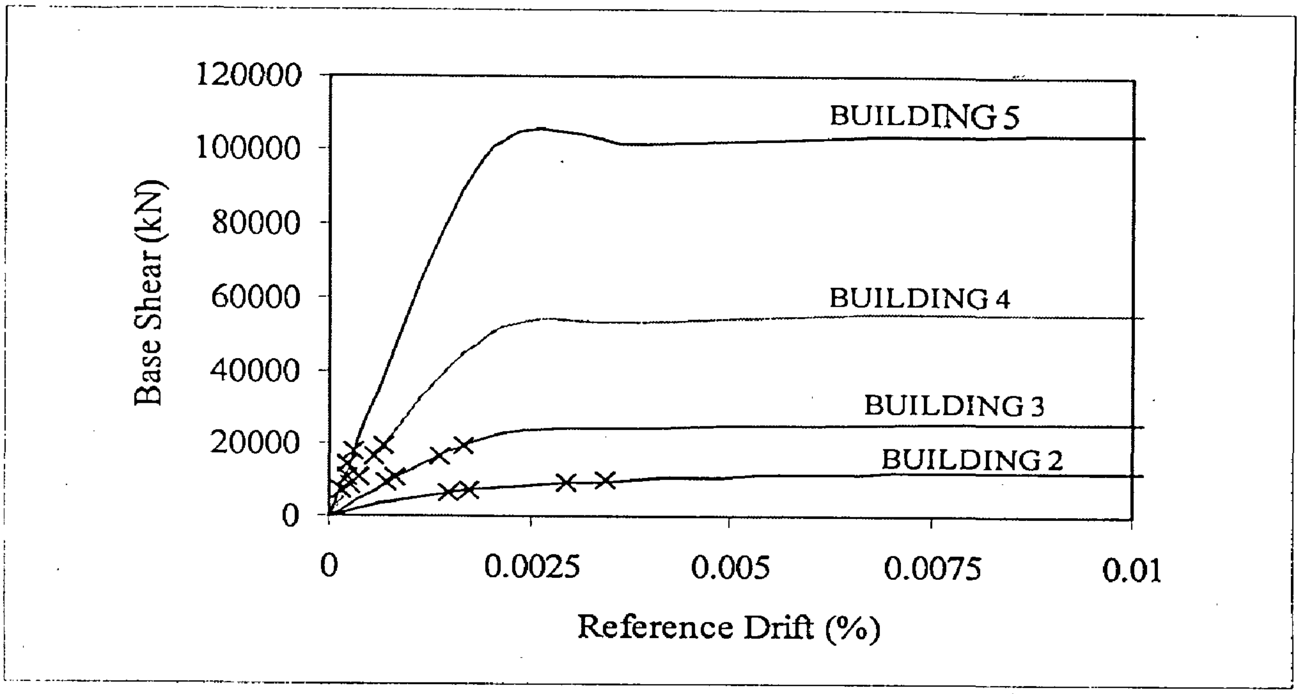

Figure 3.11 Pushover Curves (H1 Direction) of Buildings with Shear Wall

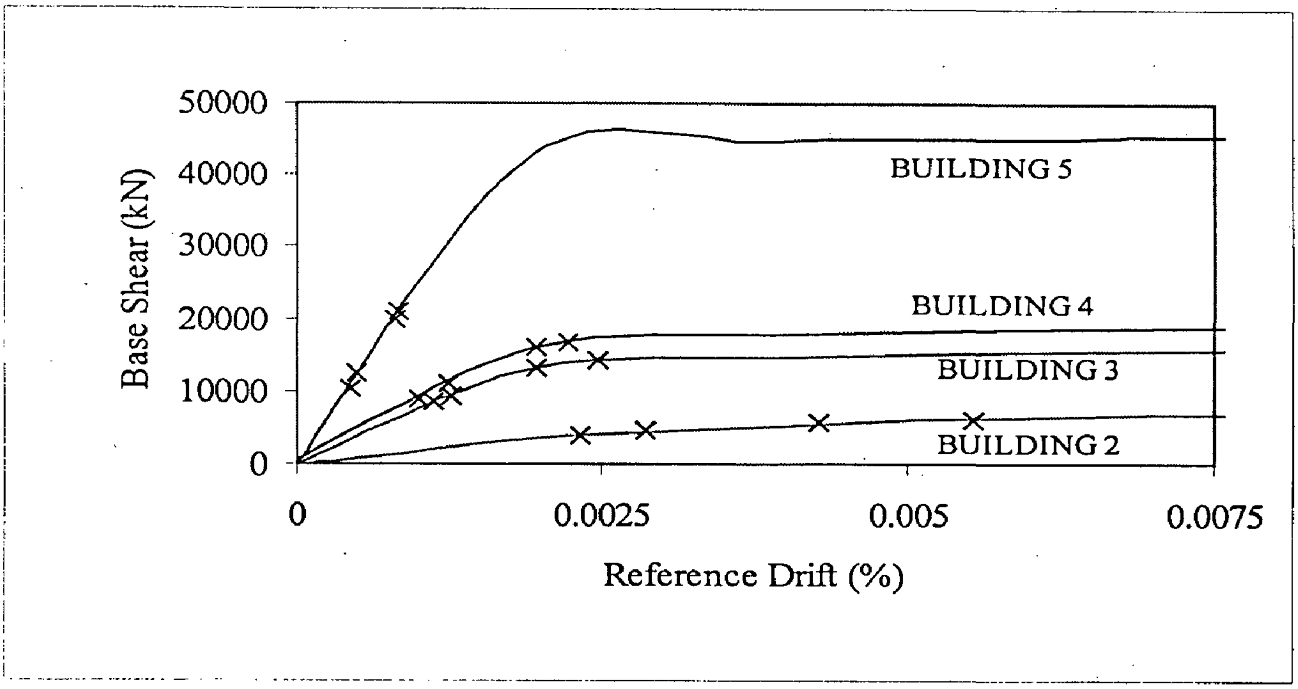

Figure 3.12 Pushover Curves (H2 Direction) of Buildings with Shear Wall

Table 3.6 Performance point Limit States for buildings with and without shear walls

Building No.	Limit States* at Performance Point							
	DBE		1.2 DBE		MCE		1.2 MCE	
	Column	Beam	Column	Beam	Column	Beam	Column	Beam
1.	IO	IO	IO	IO	-	-	-	-
2.	EL	EL	EL	EL	EL	IO	EL	LS
3.	EL	EL	EL	EL	EL	IO	EL	LS
4.	EL	EL	EL	EL	EL	IO	EL	LS
5.	EL	EL	EL	EL	EL	EL	EL	IO

** EL – Elastic, IO – Immediate Occupancy, LS – Life Safety*

3.4.3 Effect of nonlinearity on frame-shear wall interaction

Figures 3.13 and 3.14 show the history of bending moment and shear force, respectively for shear wall in transverse direction of Building 2. Figure 3.15 shows the shear force for the same shear wall at different drifts.

It can be observed from the figures that the bending moment and shear force in the shear wall varies in a complex manner with increasing drift. As more and more numbers of beam and column members in the frames yield, the relative stiffness of the shear wall w.r.t. frame changes. This makes the frame-shear wall interaction, more and more complex. In linear analysis, the frame-shear wall interaction is based only on the relative stiffness of frame and shear wall, but in non-linear analysis it is also influenced by the relative strengths of different members.

Further, it can be seen from Figure 3.13 that not only the bottom portion of the shear wall is subjected to high bending moment; the top portion of the shear wall is also subjected to considerable bending moment. This moment, in the top portion of the shear wall, first increases with increasing lateral load and then starts decreasing after yielding of the shear wall at the base. The bending moment at about one third height of the building from bottom remains almost zero as the bending moment diagram changes sign at this height. No significant change in this height has been observed with increasing lateral drift. Similar trend has been observed for shear force also, as shown in Figure 3.15.

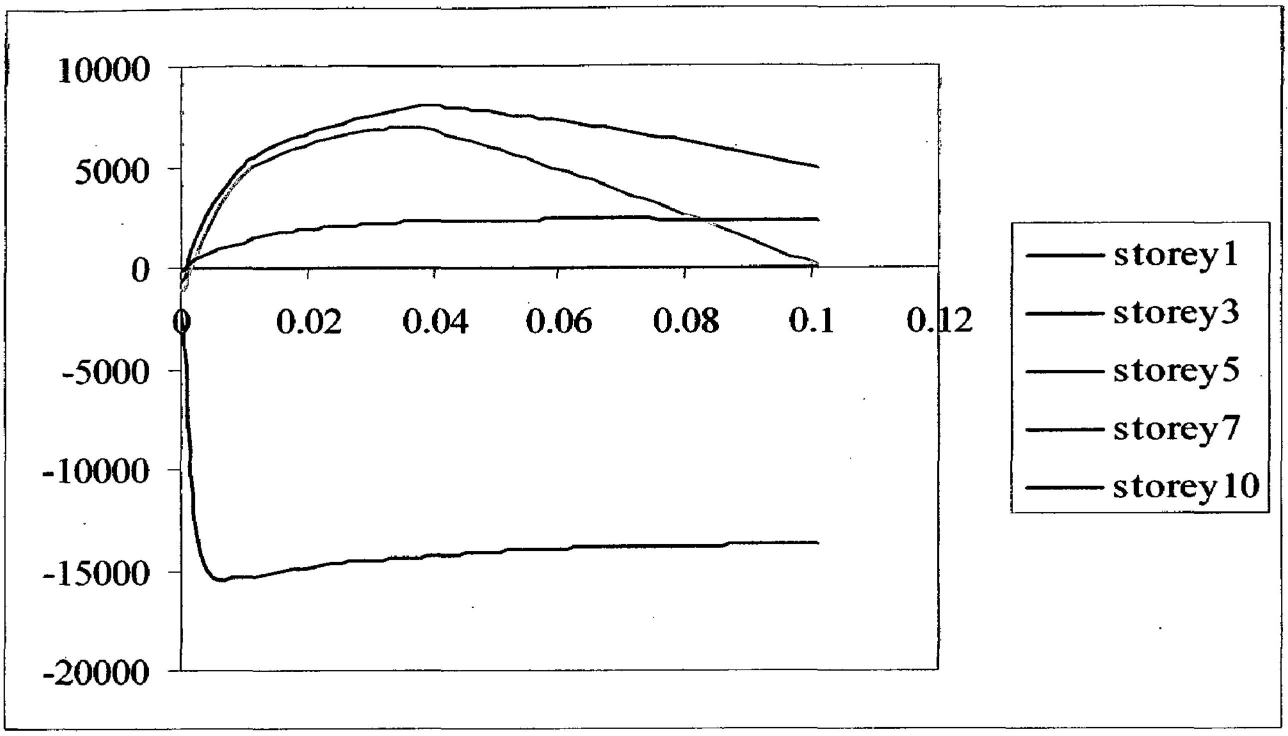

Figure 3.13 Bending Moment History for 6 m Shear Wall (Building2)

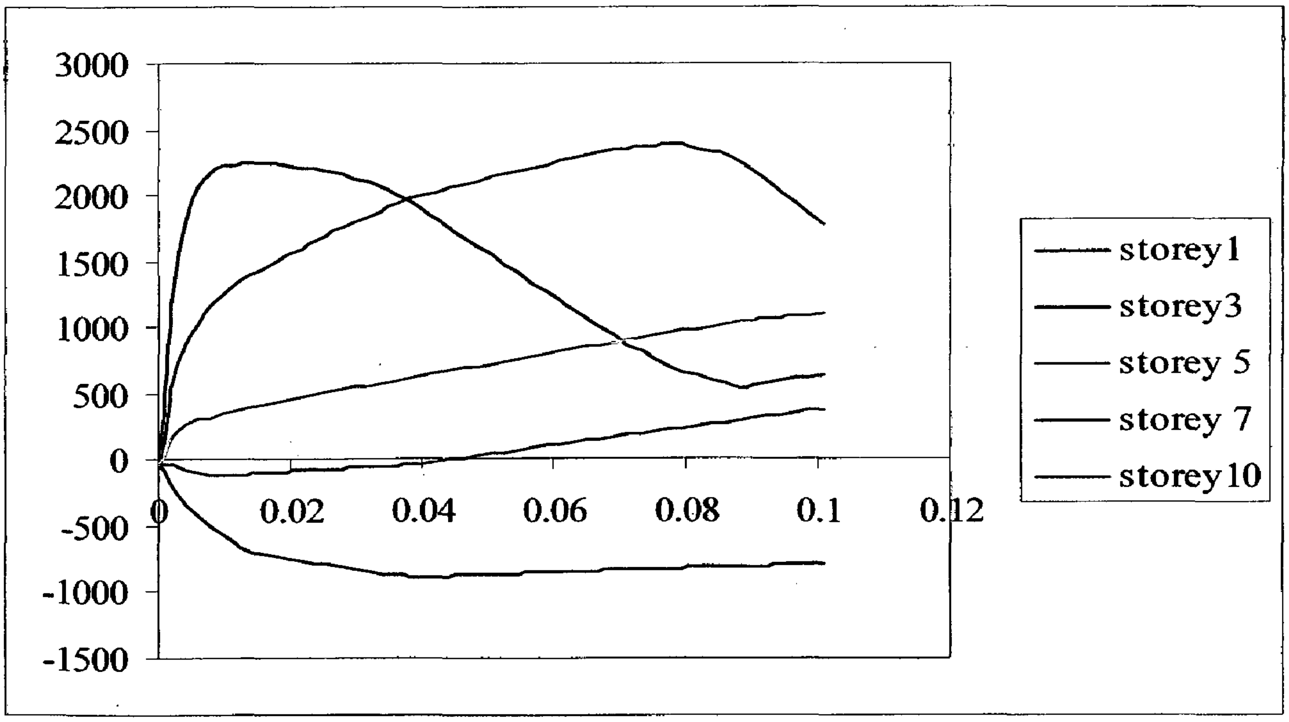

Figure 3.14 Shear Force History for 6 m Shear Wall (Building2)

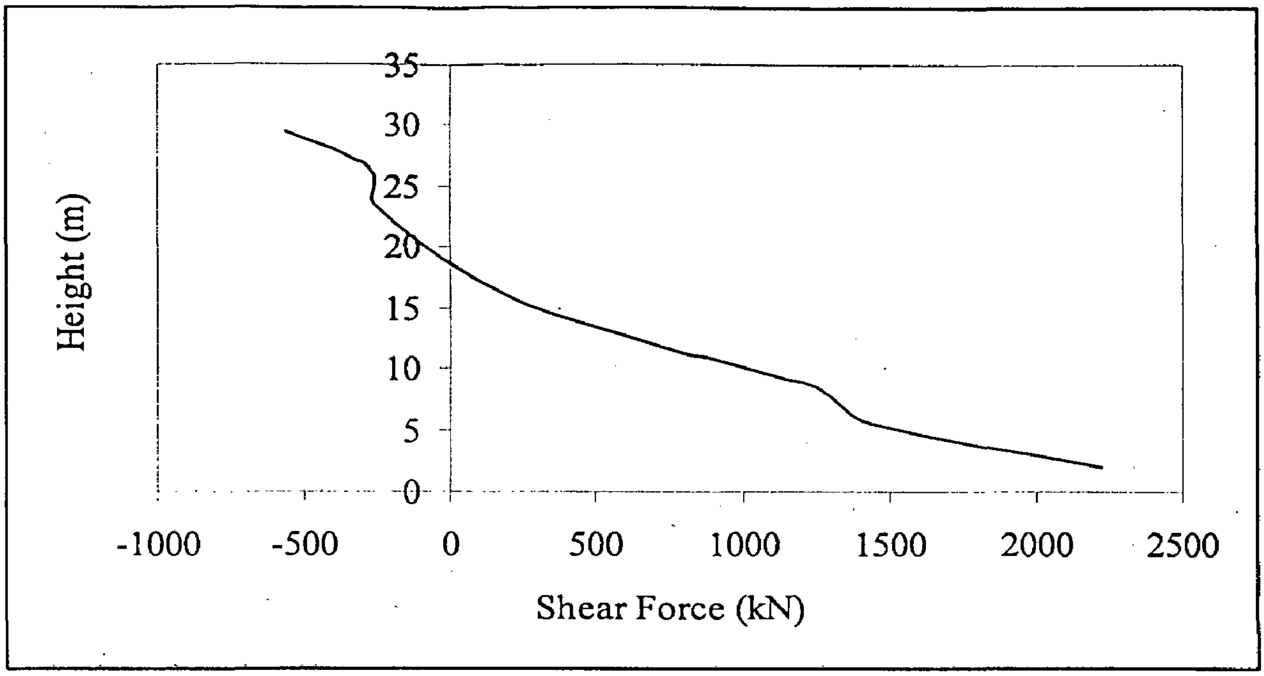

Figure 3.15 (a) Shear Force Diagram for 6 m Shear Wall at Drift 0.01 (Building2)

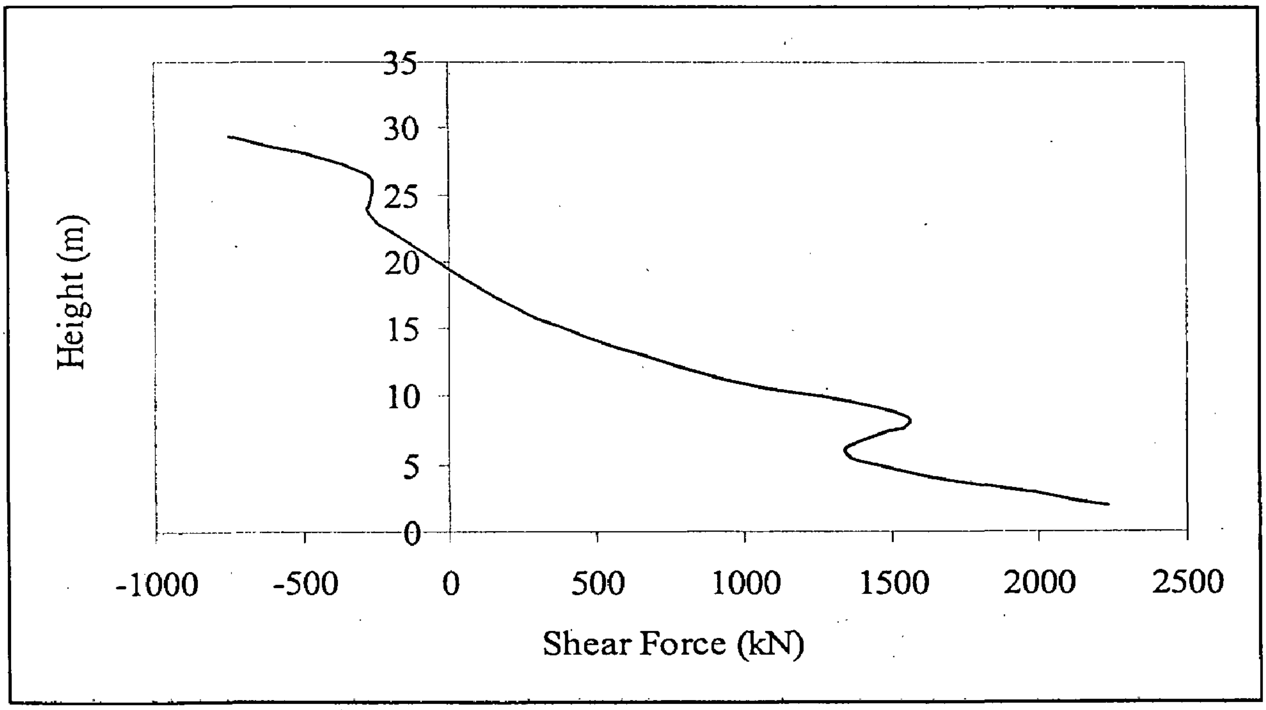

Figure 3.15(b) Shear Force Diagram for 6 m Shear Wall at Drift 0.02 (Building2)

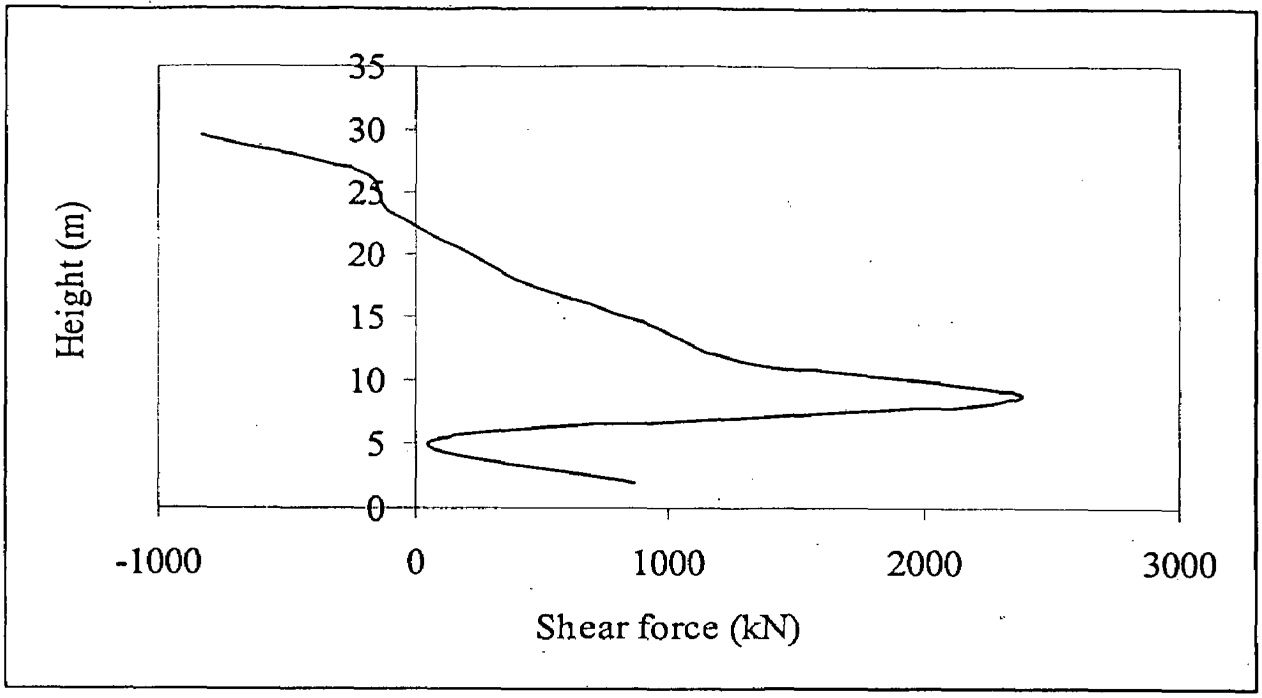

Figure 3.15(c) Shear Force Diagram for 6 m Shear Wall at Drift 0.07 (Building2)

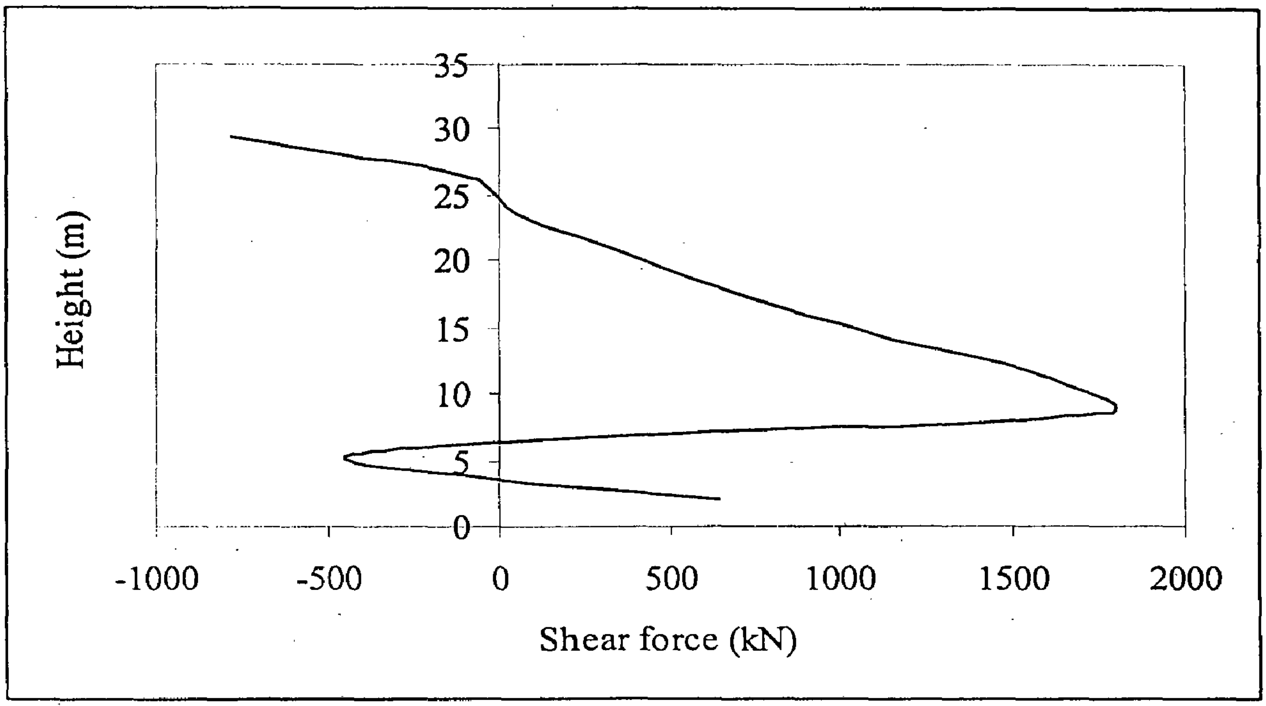

Figure 3.15(d) Shear Force Diagram for 6 m Shear Wall at Drift 0.1 (Building2)

CONCLUSIONS

Behaviour of Frame-shear wall buildings under seismic load has been studied in this dissertation. Five 10 storeyed buildings, including a frame building and the other four buildings with increasing shear wall area have been considered. These buildings have been designed as per current IS codes and their performance has been studied using non-linear static analysis. Following are the main conclusions of the study:

1. In the nonlinear static analysis of the buildings there is no significant effect of loading pattern on building pushover curve.
2. Performance of the frame building is not satisfactory as its fails for MCE. However, performance of frame-shear wall building improves significantly. Buildings with large shear wall area, remain elastic, even under MCE.
3. By providing shear wall area equal to just 1 to 3% of floor area of building the performance of the building has been improved remarkably and Immediate Occupancy level has been achieved, even for MCE, which is the desired performance level for post earthquake important structures, such as hospital.
4. The bending moment and shear force in the shear wall varies in a complex manner with increasing drift. As more and more numbers of beam and column members in the frames yield, the stiffness of the shear wall relative to frame changes. This makes the frame-shear wall interaction, more and more complex. In linear analysis, the frame-shear wall interaction is governed only by the relative stiffness of frame and shear wall, but in non-linear analysis it is also influenced by the relative strengths of different members.

5. Due to the frame-shear wall interaction, not only the bottom portion of the shear wall is subjected to high bending moment; the top portion of the shear wall is also subjected to considerable bending moment. This moment, in the top portion of the shear wall, first increases with increasing lateral load and then starts decreasing after yielding of the shear wall at the base. The bending moment at about one third height of the building, from bottom, remains almost zero as the bending moment diagram changes sign at this height. No significant change in this height has been observed with increasing lateral drift.

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APPENDIX

Pushover curves for the shear wall buildings due to ground shaking DBE, MCE, 1.2 DBE and 1.2 MCE under parabolic pushover loading pattern in longitudinal (H1) and transverse (*H2*) direction.

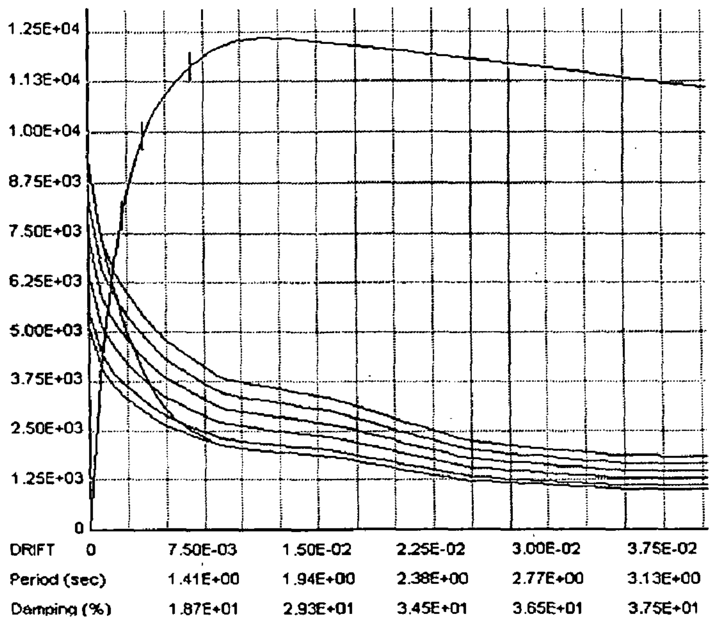


Figure A1 Pushover Curve for Building 2 for DBE in H1

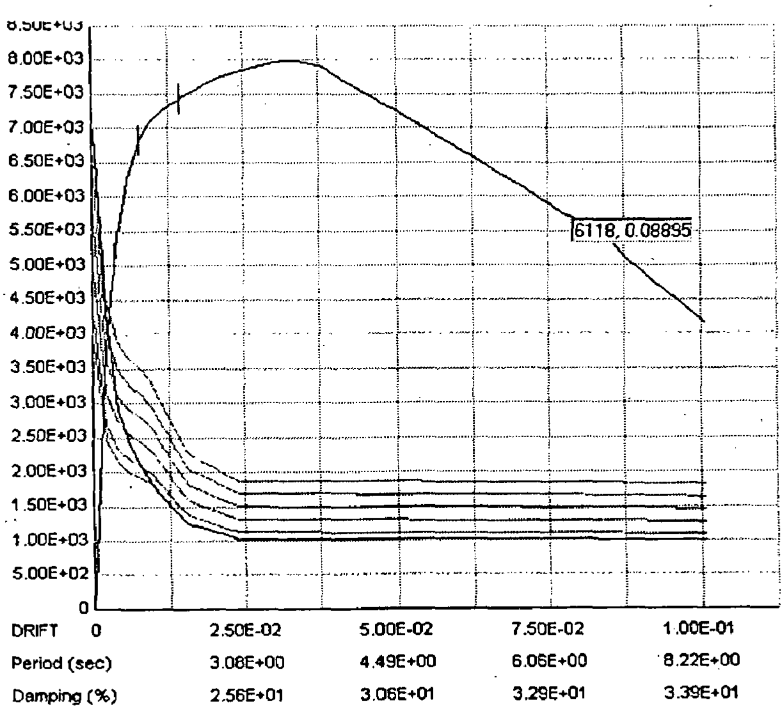


Figure A2 Pushover Curve for Building 2 for DBE in H2

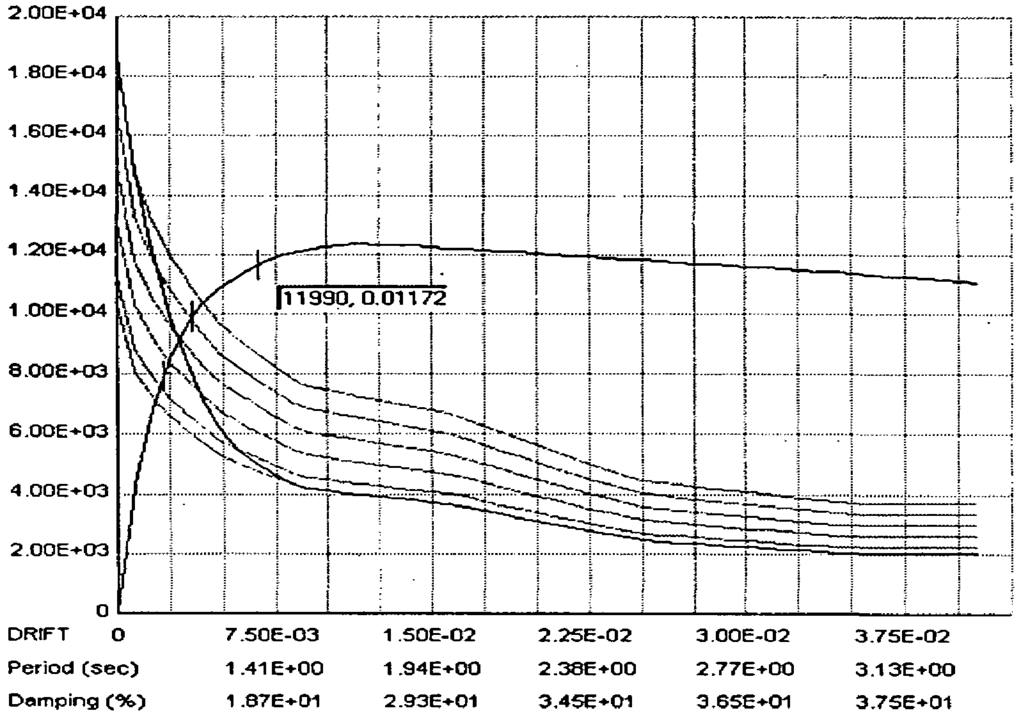


Figure A3 Pushover Curve for Building 2 for MCE in H1

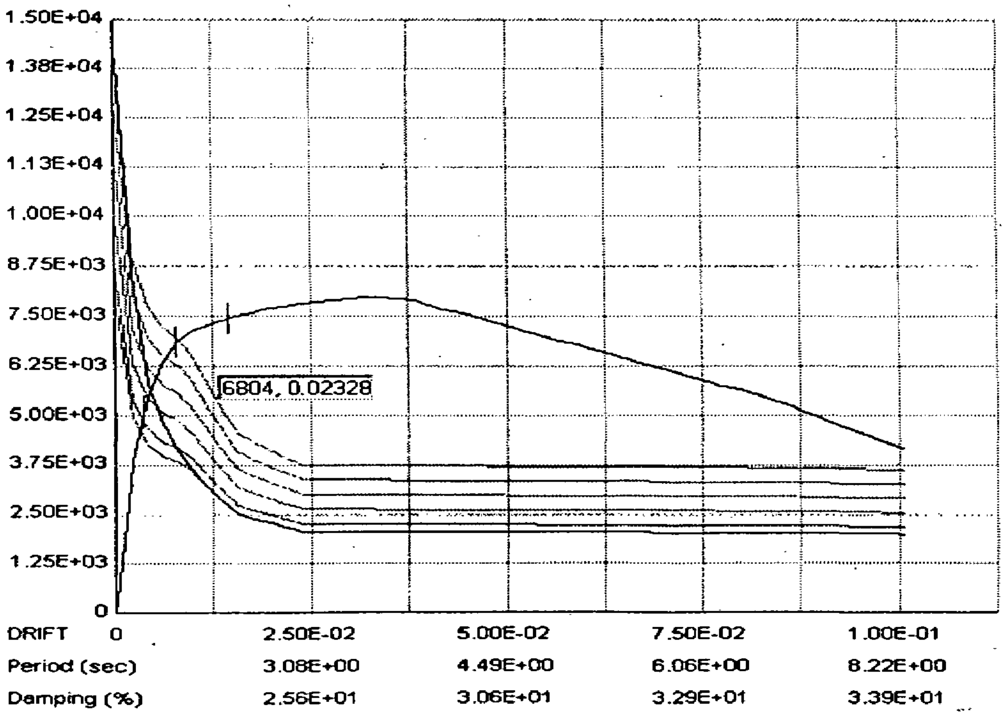


Figure A4 Pushover Curve for Building 2 for MCE in H2

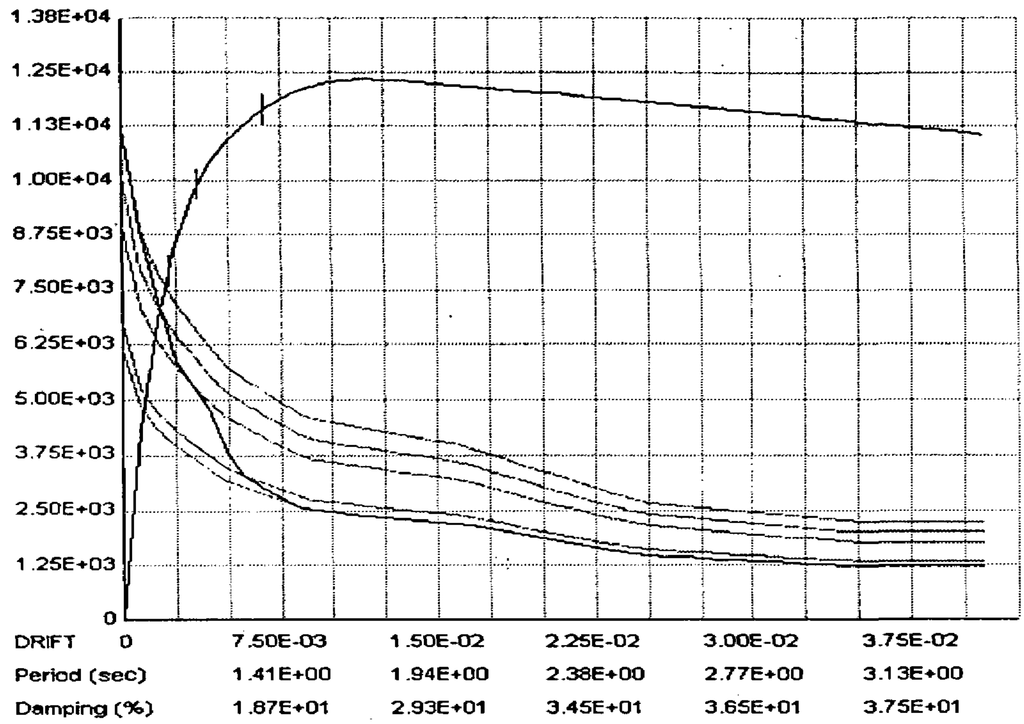


Figure A5 Pushover Curve for Building 2 for 1.2 DBE in H1

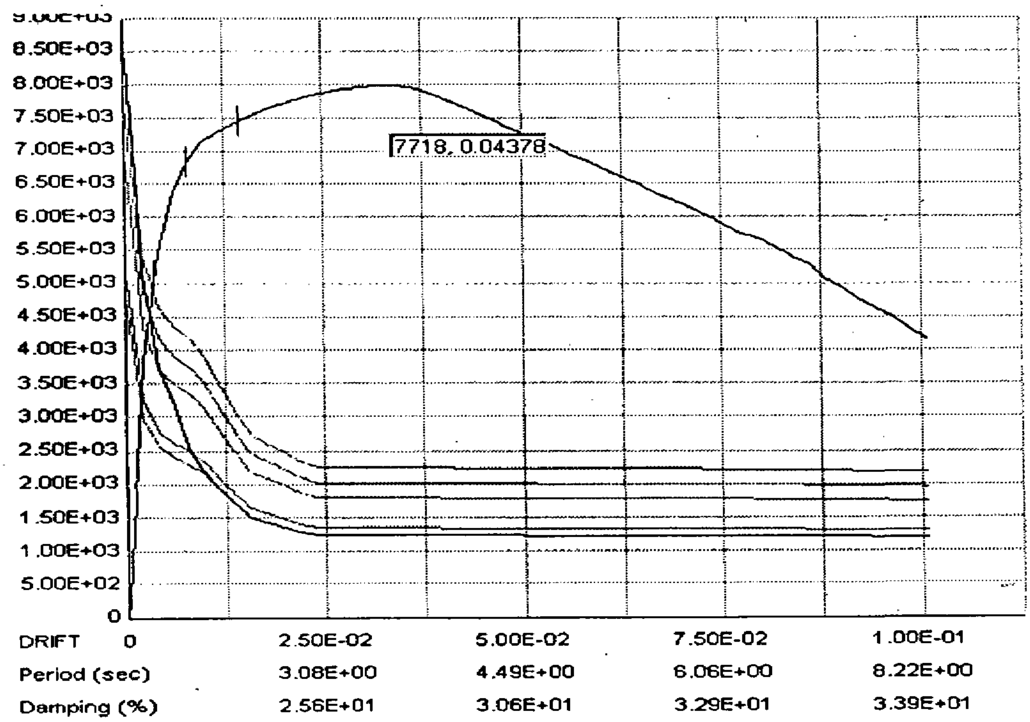


Figure A6 Pushover Curve for Building 2 for 1.2 DBE in H2

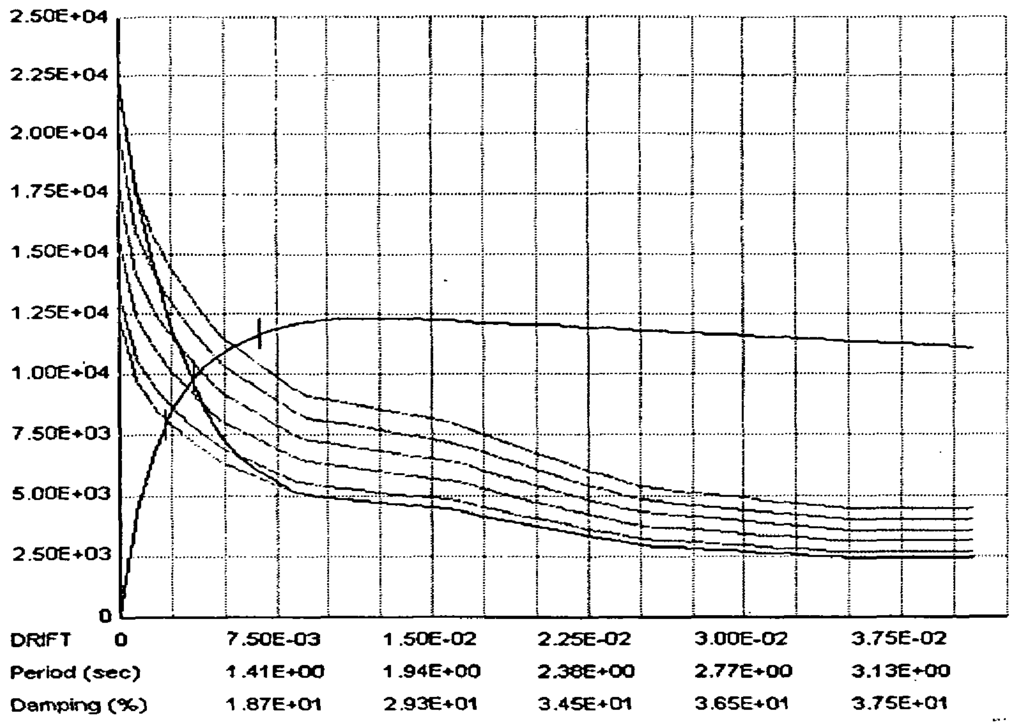


Figure A7 Pushover Curve for Building 2 for 1.2 MCE in H1

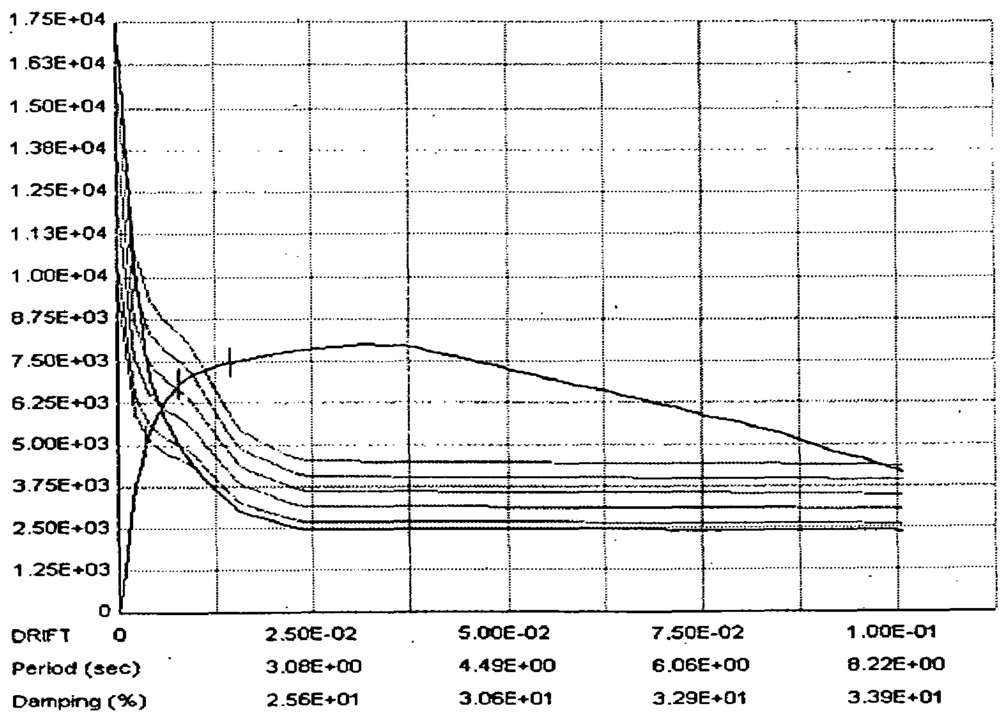


Figure A8 Pushover Curve for Building 2 for 1.2 MCE in H2

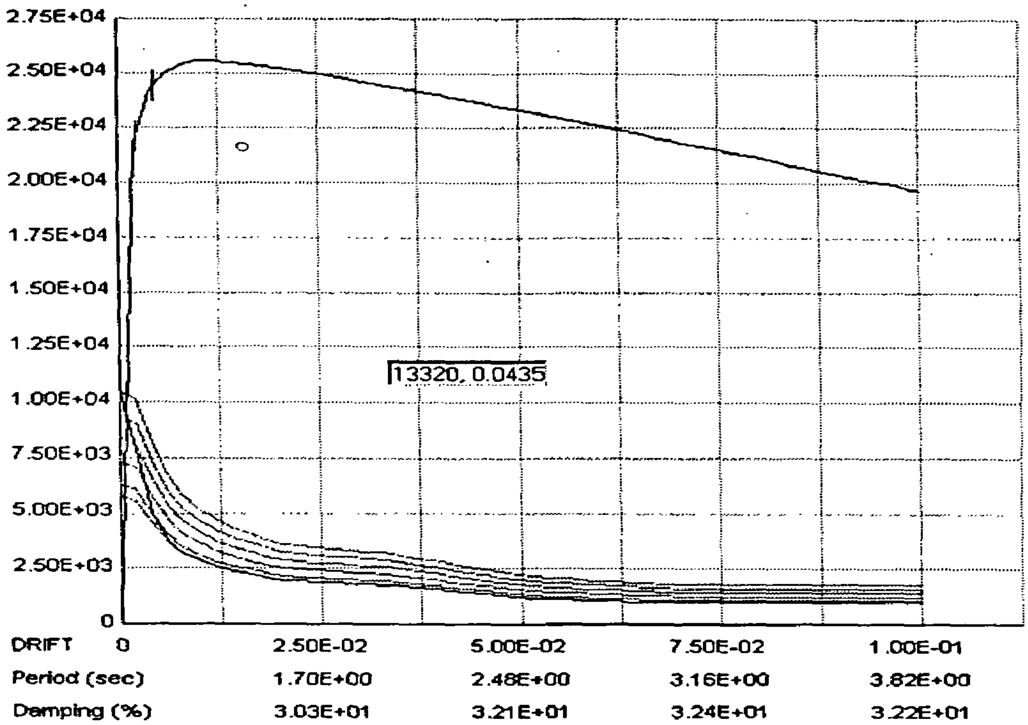


Figure A9 Pushover Curve for Building 3 for DBE in H1

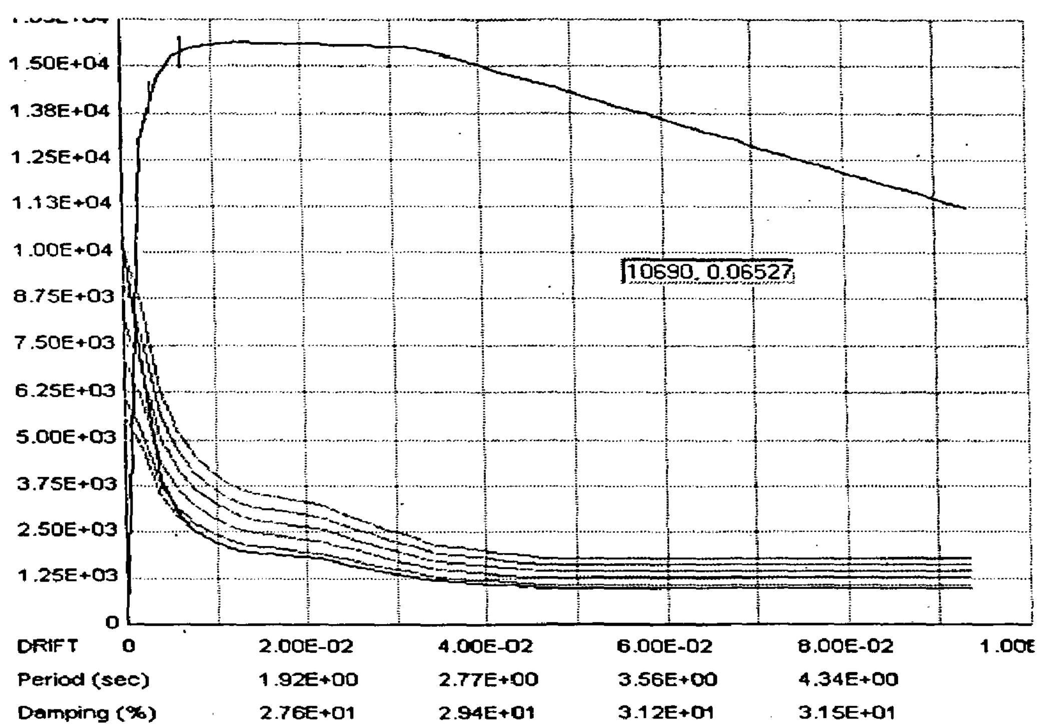


Figure A10 Pushover Curve for Building 3 for DBE in H2

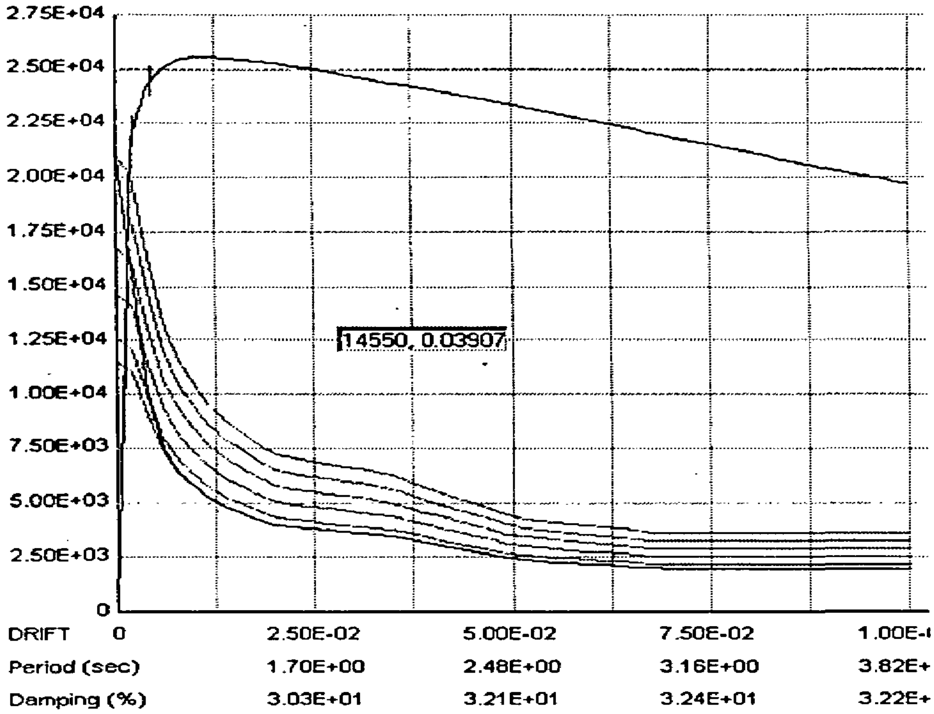


Figure A11 Pushover Curve for Building 3 for MCE in H1

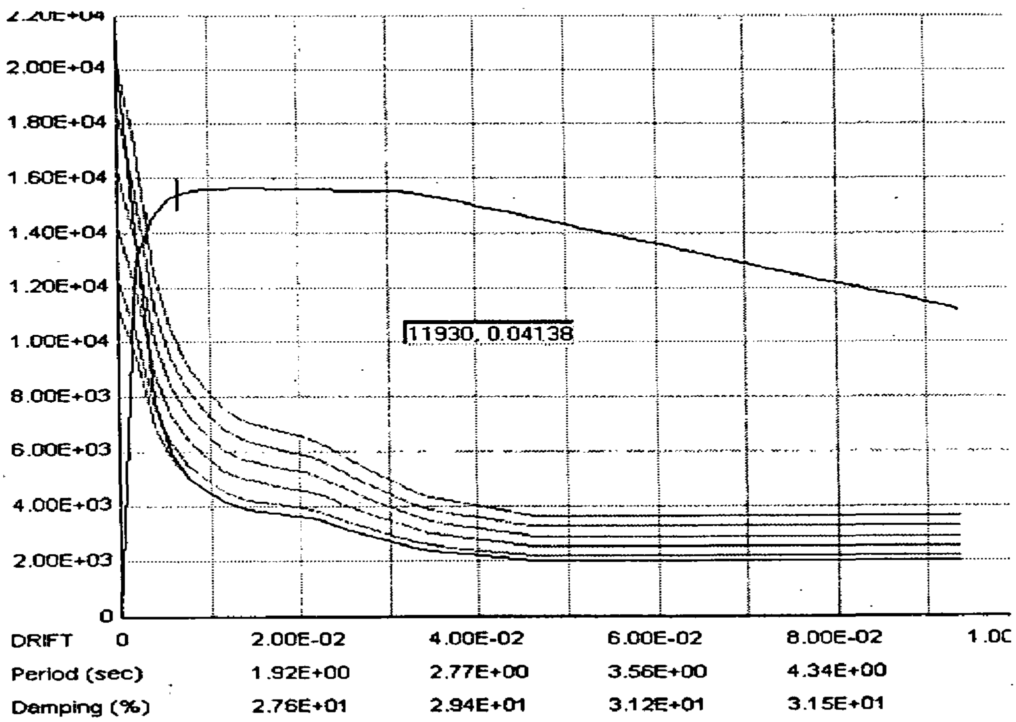


Figure A12 Pushover Curve for Building 3 for MCE in H2

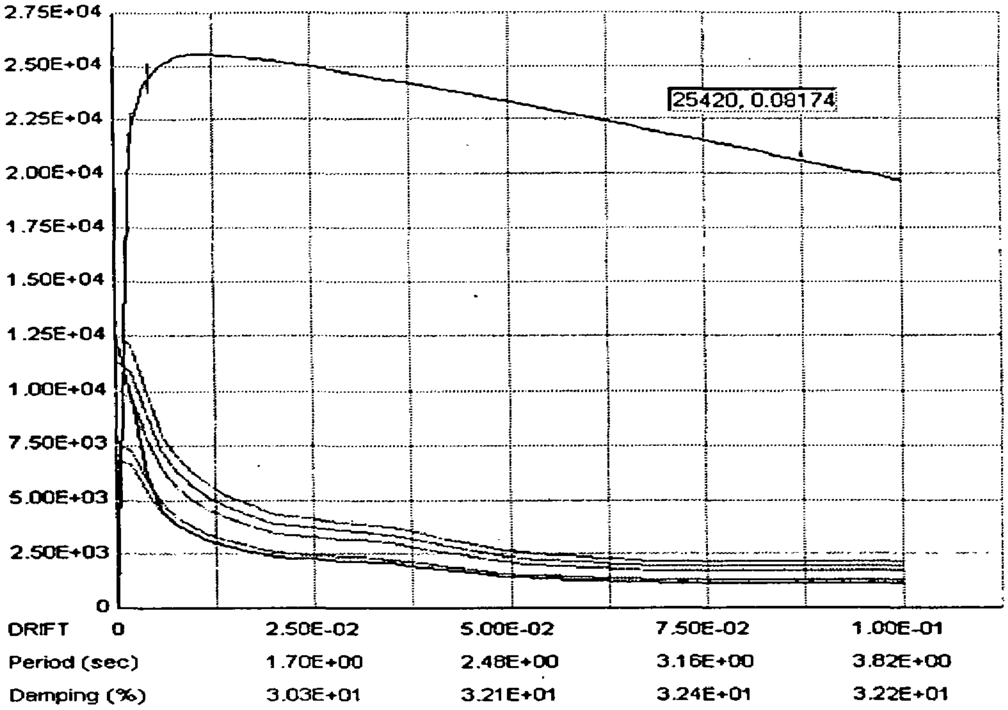


Figure A13 Pushover Curve for Building 3 for 1.2 DBE in H1

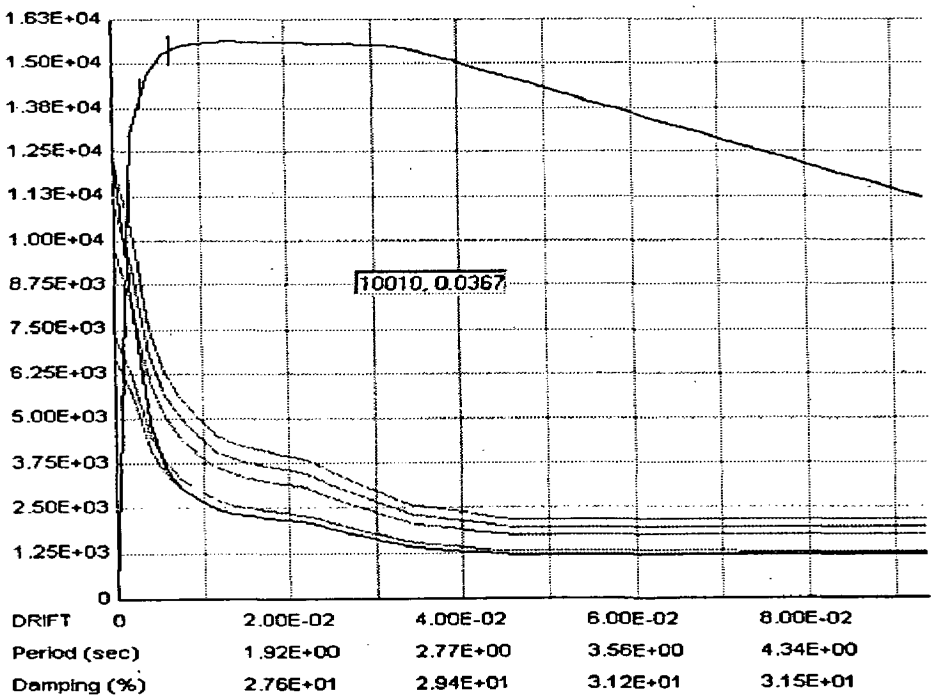


Figure A14 Pushover Curve for Building 3 for 1.2 DBE in H2

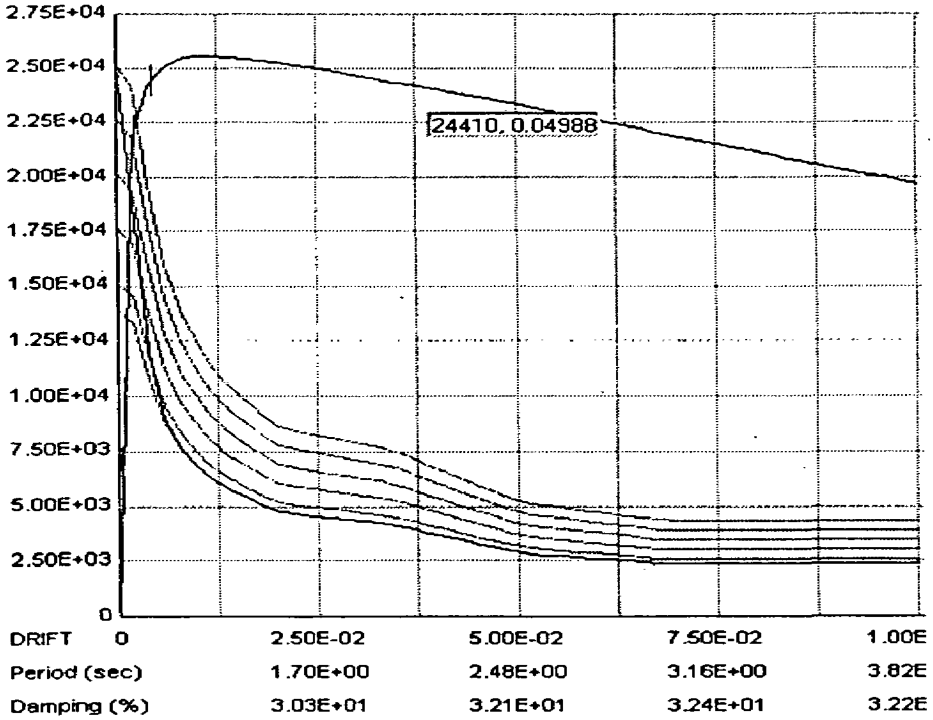
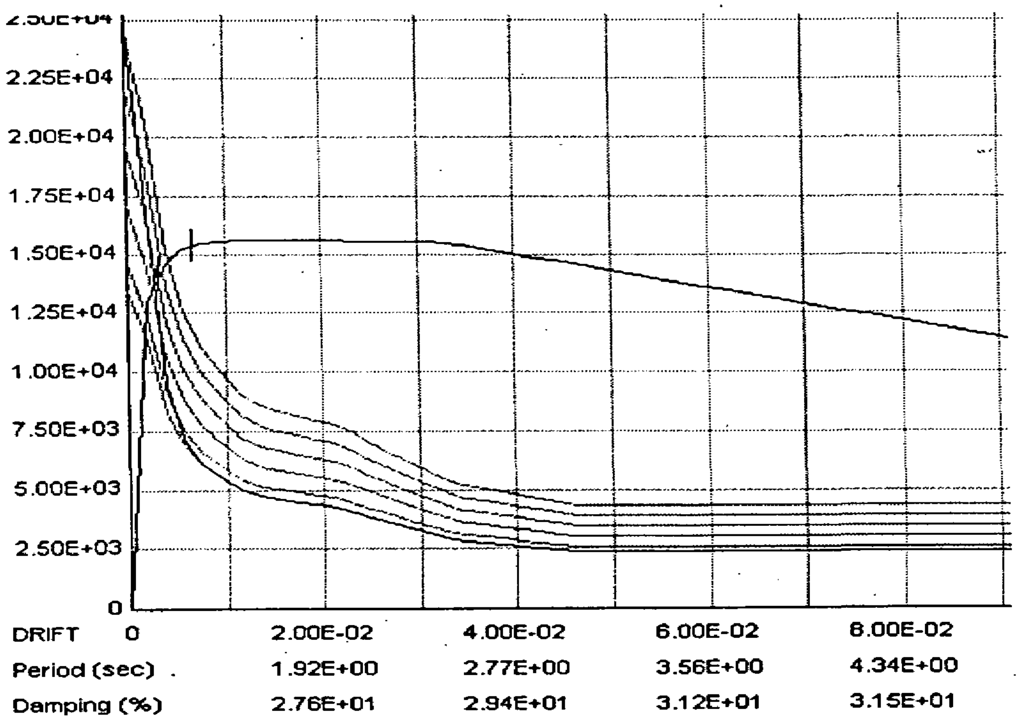


Figure A15 Pushover Curve for Building 3 for 1.2 MCE in H1



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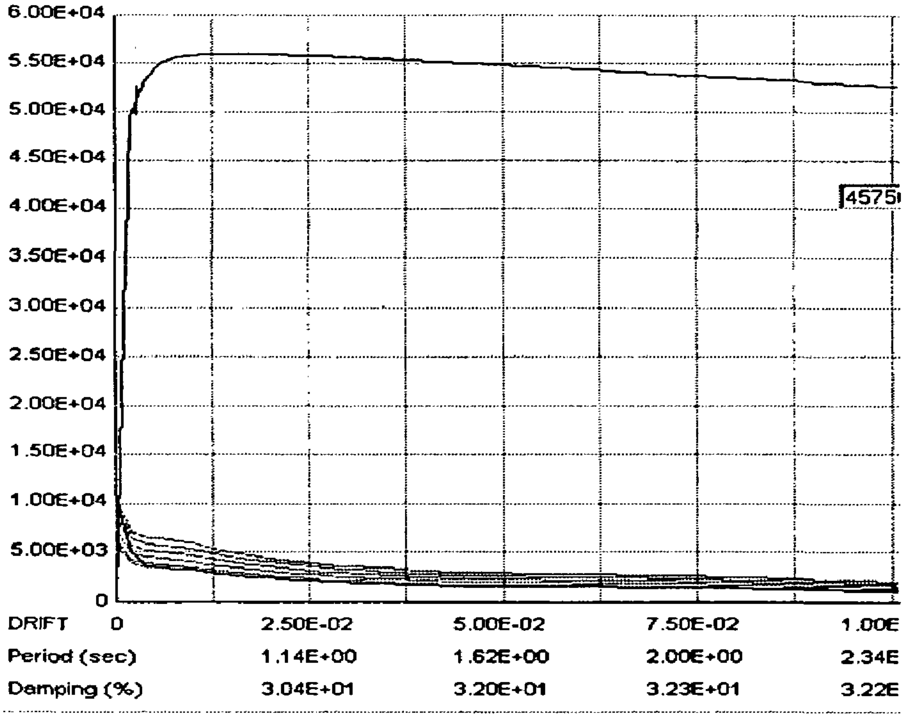


Figure A17 Pushover Curve for Building 4 for DBE in H1

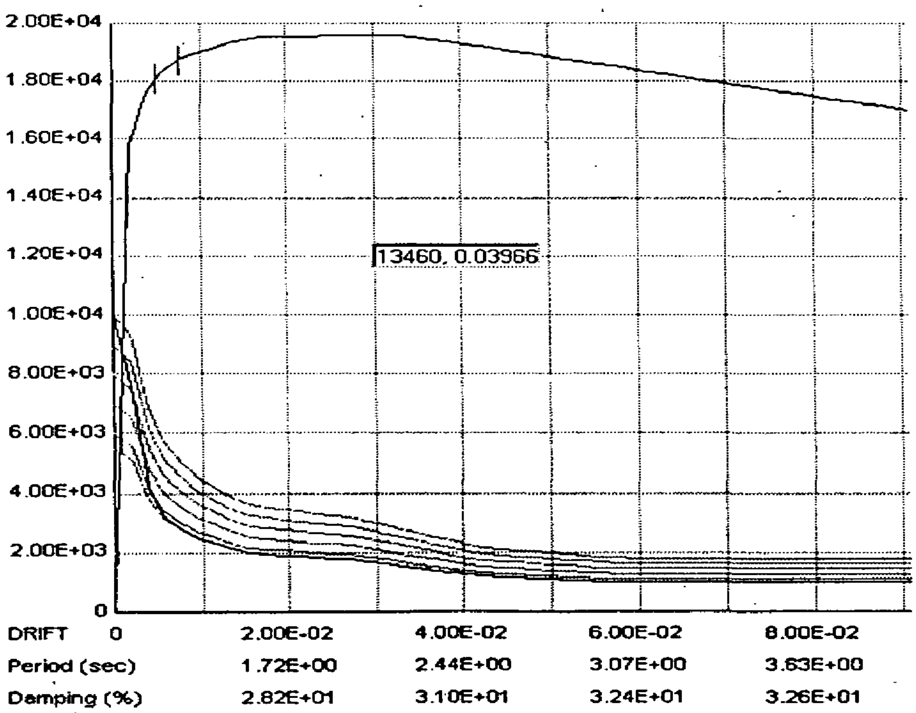


Figure A18 Pushover Curve for Building 4 for DBE in H2

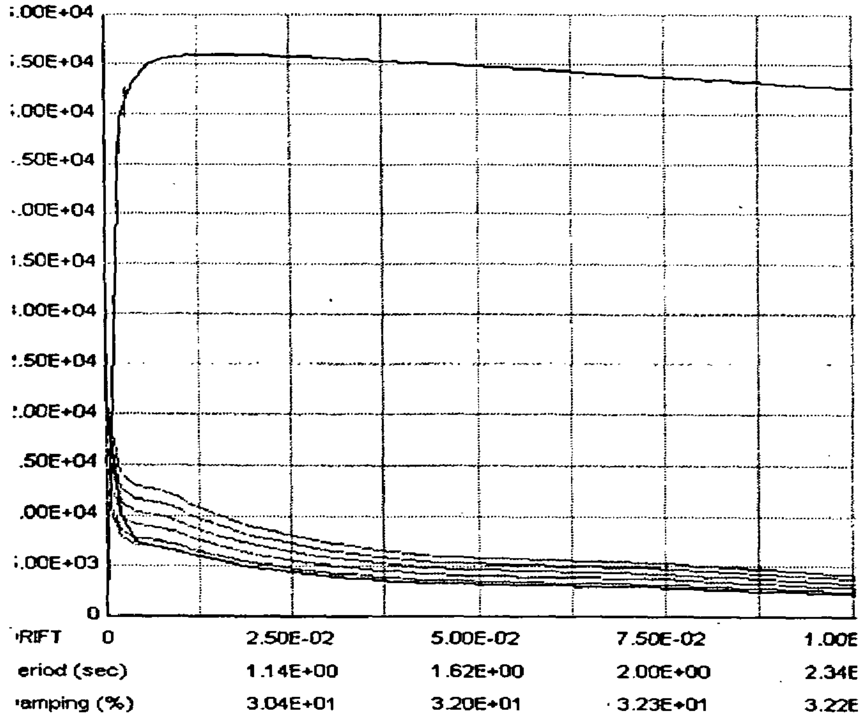
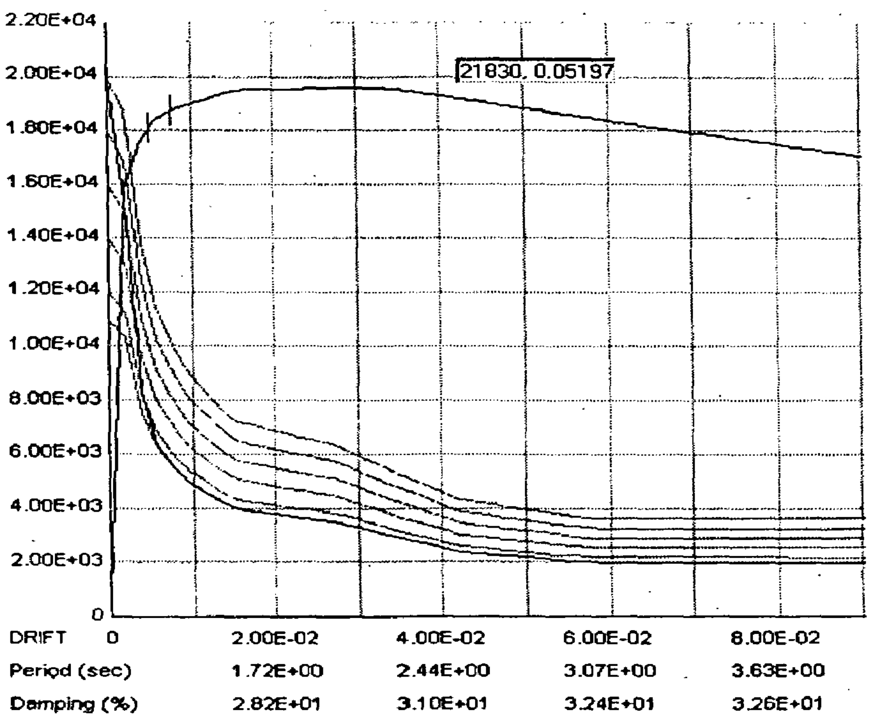


Figure A19 Pushover Curve for Building 4 for MCE in H1



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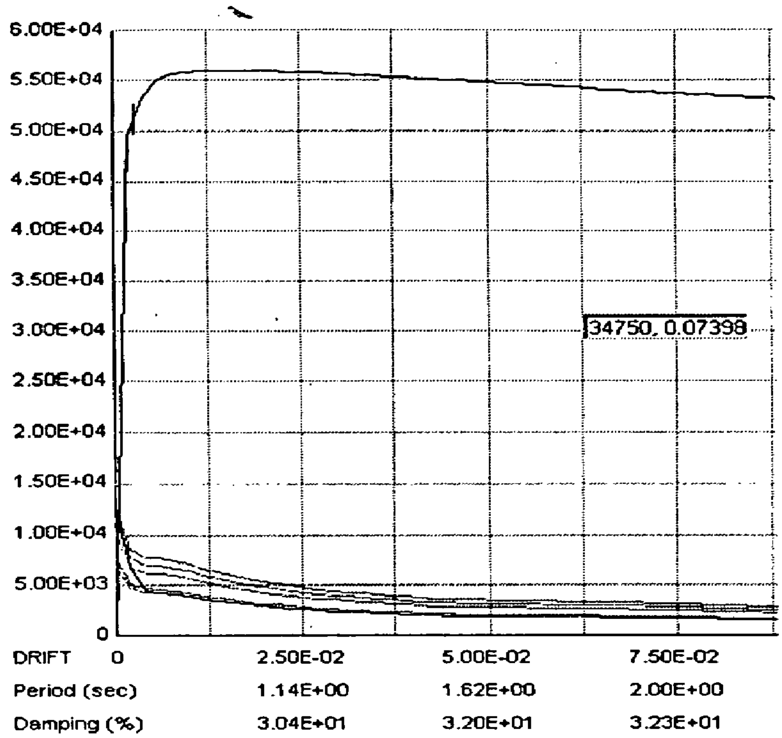

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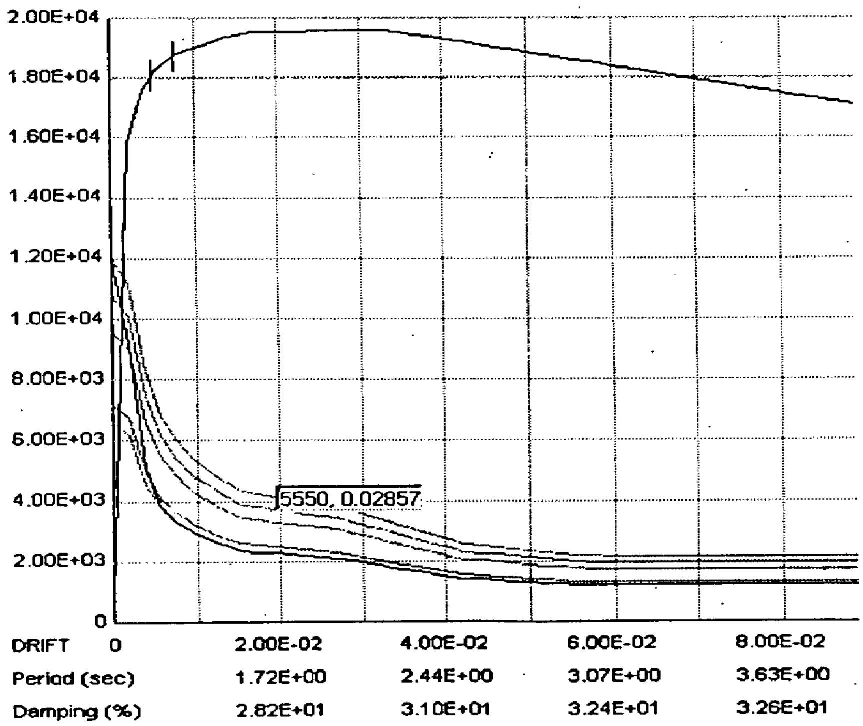

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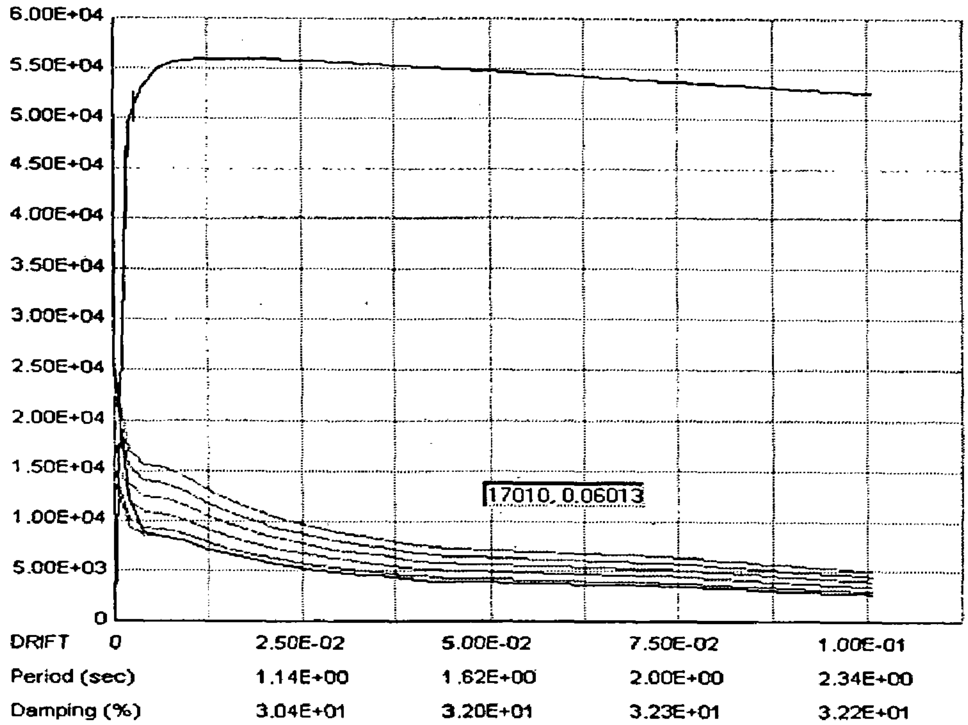
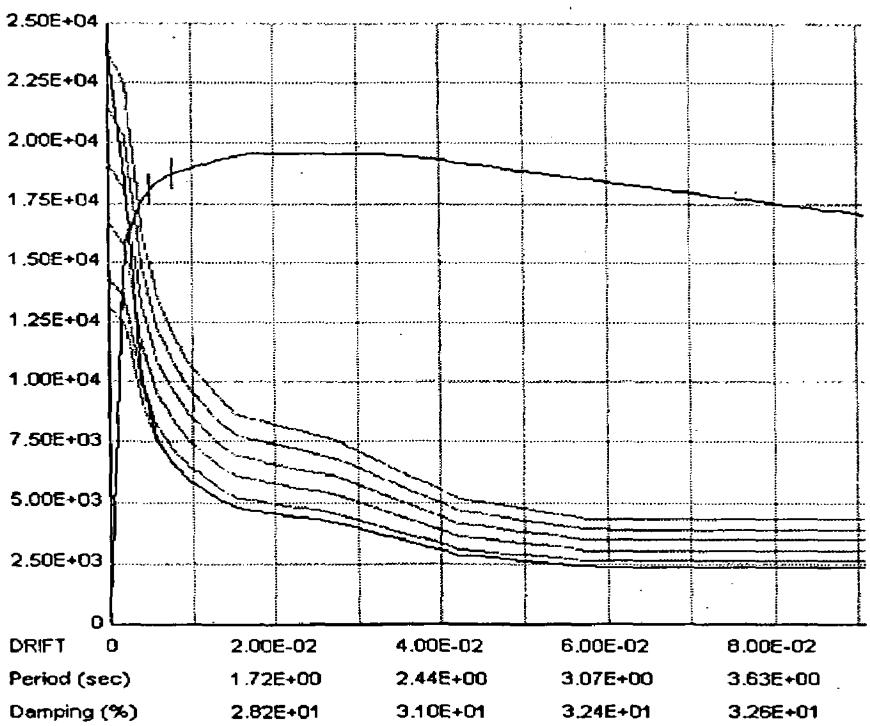


Figure A23 Pushover Curve for Building 4 for 1.2 MCE in H1



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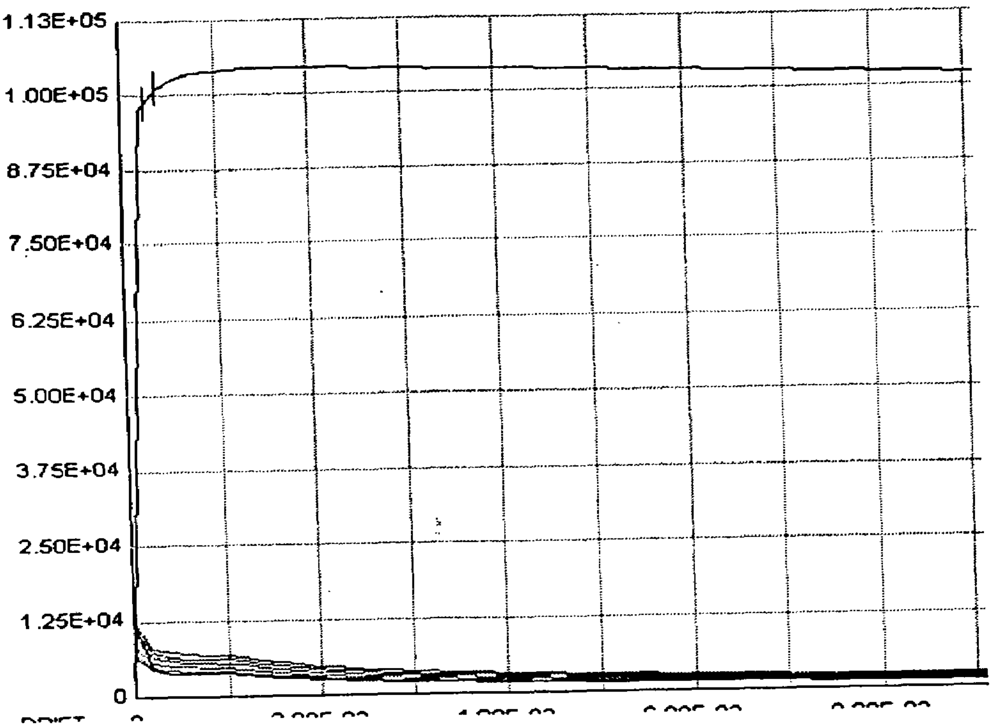
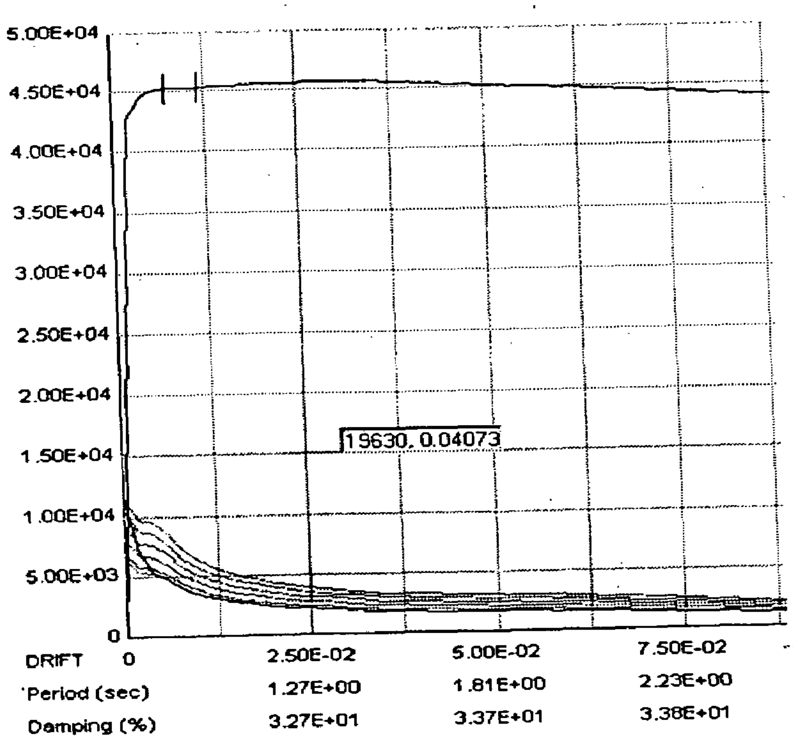


Figure A25 Pushover Curve for Building 5 for DBE in H1



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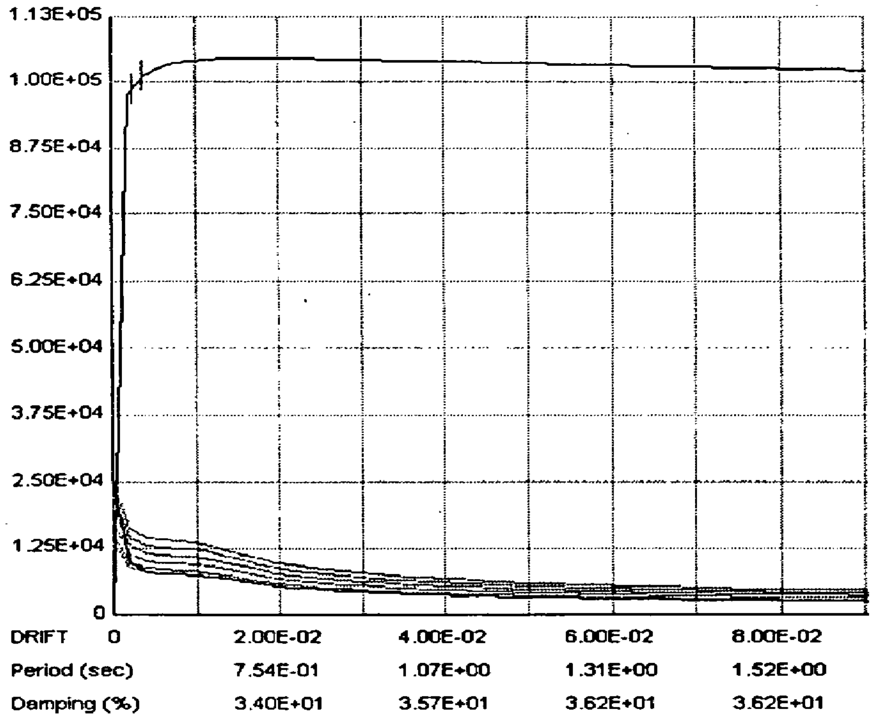

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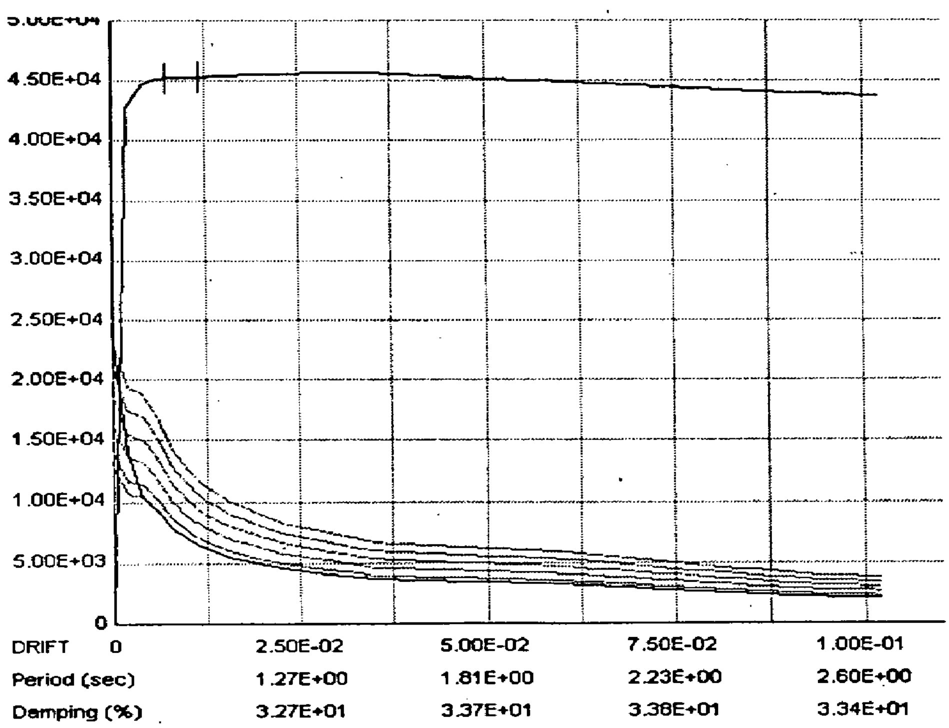

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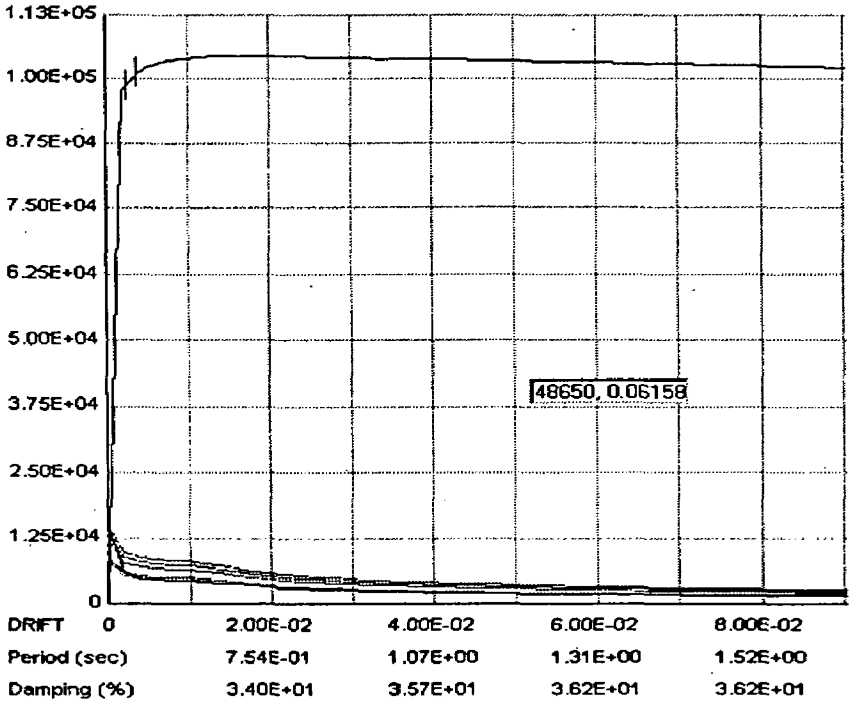

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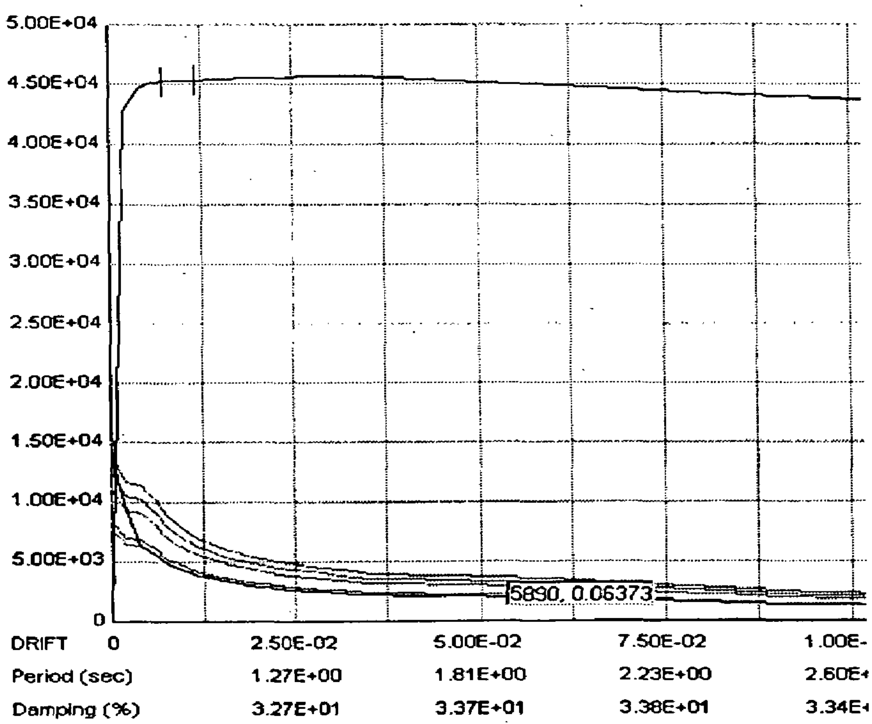

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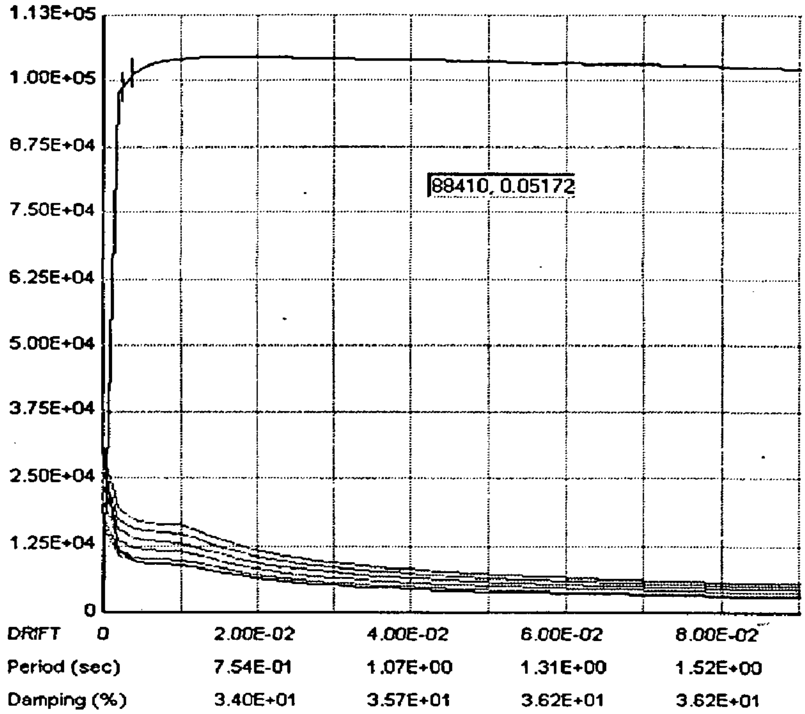

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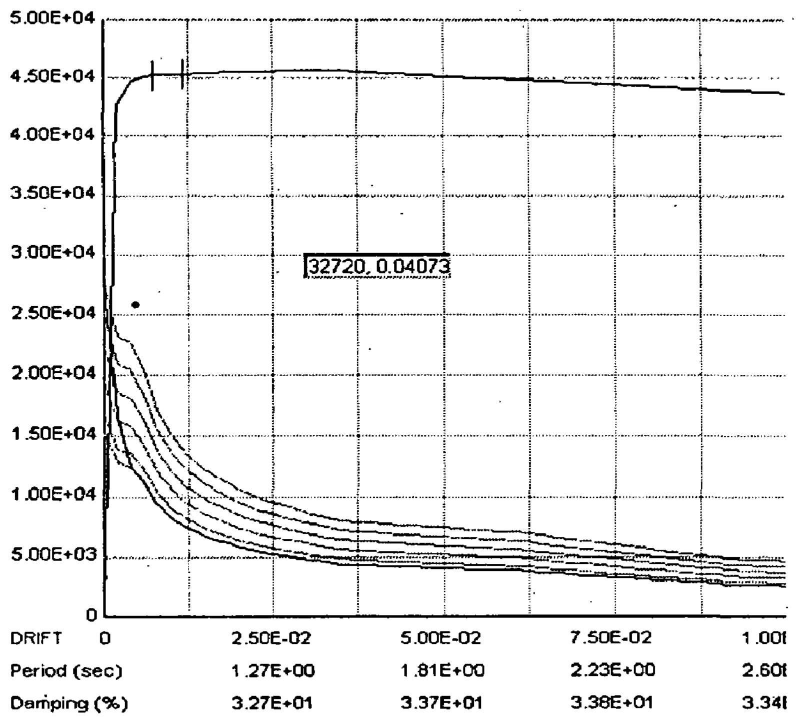

Figure A32 Pushover Curve for Building 5 for 1.2MCE in H2