ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

DEPARTMENT OF CIVIL ENGINEERING

ADDIS ABABA UNIVERSITY

ASSESSMENT OF RIBBED SLAB CONSTRUCTIONS FRAMED ONLY IN ONE DIRECTION AGAINST LATERAL LOAD EXCITATION

BY

MISRAK TEFERA GIZAW

APRIL 2011

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ASSESSMENT OF RIBBED SLAB CONSTRUCTIONS FRAMED ONLY IN ONE DIRECTION AGAINST LATERAL LOAD EXCITATIONS

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By

MISRAK TEFERA GIZAW

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ABSTRACT

Nowadays construction of buildings using ribbed slabs, with hollow blocks, is common from low to high seismic regions of Ethiopia. When investigating some of the buildings constructed and being constructed using such slabs, framing only in one direction is being practiced. In recent years, numbers of earthquakes have also occurred in different parts of the country. The occurrence of these earthquakes has drawn increased attention towards the importance of seismic performance evaluation and improvement of existing structures to reduce seismic risk. In this thesis work, performance evaluation of three case study buildings, constructed using ribbed slabs with hollow blocks framing only in one direction is done using static nonlinear, pushover, analysis. During the conventional analysis of such buildings, the floor slabs are considered to act as a rigid diaphragm. Therefore, in this thesis, the three case study buildings are evaluated for rigidity of the diaphragms. The result of the performance evaluation proved that the buildings do not perform adequately to insure the safety of their dwellers at their earth quake design level. The result of diaphragm flexibility check has also shown that the stiffness of such slabs is not sufficient to act as a rigid diaphragm in the longitudinal direction of the ribs. To improve the performance of the evaluated case study buildings, the introduction of the missing beams were found not to be sufficient. Therefore, addition of shear walls with some defined cross sectional dimension and plan distribution are recommended in addition to the insertion of the missing beams.

Keywords: *Ribbed slabs, framing only in one direction, seismic performance evaluation, static nonlinear analysis, pushover analysis, diaphragm, diaphragm flexibility*

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LIST OF SYMBOLS AND ABBREVIATIONS

ADRS	Acceleration Displacement Response Spectra
a_{pi}	Spectral acceleration coordinate at trial performance point
a_y	Spectral acceleration coordinate at yield
ATC	Applied Technology Council
C_a	Seismic coefficient on elastic response spectrum of UBC 97
CSM	Capacity Spectrum Method
C_n	Column, n= 1, 2, 3…
CP	Collapse Prevention
C_v	Seismic coefficient on elastic response spectrum of UBC 97
d_{pi}	Spectral displacement coordinate at trial performance point
d_y	Spectral displacement coordinate at yield
EBCS	Ethiopian Buildings Code Standard
EC	European Code
EDU	Energy Dissipation Units
ETABS	Extended 3D Analysis of Building Systems
E_o	Energy dissipated by damping
E_{so}	Maximum strain energy
$f_{c'}$	Concrete cylinder compressive strength
FEMA	Federal Emergency Management Agency
f_y	Yield strength
IO	Immediate Occupancy
LS	Life Safety
M_y	Yield Moment
PGA	Peak Ground Acceleration
P-Δ	Forces deformation
RB_n	Roof Beam, n= 1, 2, 3…
SRA	Spectral reduction factor in the acceleration sensitive region of the spectrum
SRV	Spectral reduction factor in the velocity sensitive region of the spectrum
SS	Structure Stability

SW_n	Shear wall, n= 1, 2, 3 …
T	Period
UBC	Uniform Building Code
$ß_{eff}$	effective viscous damping
$ß_{eq}$	equivalent viscous damping
$ß_o$	hysteretic damping
θ_y	rotation at yield
κ	damping modification factor
Δ_{diaph}	in plane diaphragm displacement
ω	modal circular frequency

Chapter 1: Introduction

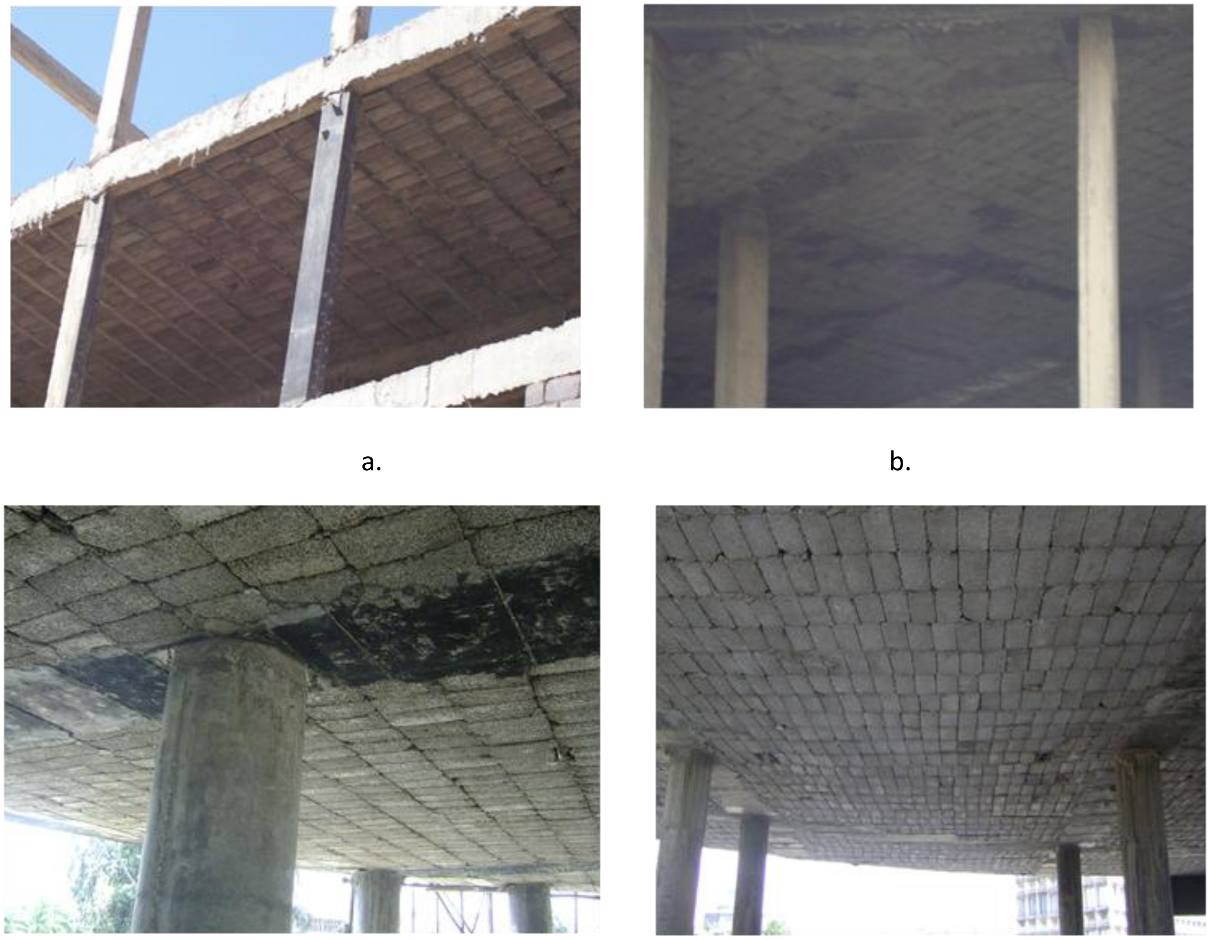
1.1 General

In different countries, seismic performance evaluation of building structures against seismic loads has been done for several reasons. These include, to check the performance of buildings constructed prior to the revision of the seismic building code standards [20] and to insure an appropriate level of safety for the occupants of the buildings. It has also been done to check the performance of existing buildings which are not designed for earthquake forces. This is due to the fact that the seismic performance of such buildings is quite questionable looking at the performance of similar structures in the past earthquakes.

Nowadays, construction of low to high-rise multi story buildings using ribbed slabs with hollow blocks floor system is mostly widespread in Ethiopia. This is maybe due to the advantages of these slabs posses over other conventional types of beam supported slab constructions, such as architectural flexibility, less form work, less workmanship during construction and reduced construction period.

When investigating some of the buildings constructed using ribbed slab with hollow blocks systems, framing only in one direction is being practiced. This is because of considering the ribs to act as a beam in the other direction. Some pictures of the building with such type of framing system are presented in Figure 1.1 for illustration.

In recent years, numbers of earthquakes have occurred in different parts of the country. The occurrence of these earthquakes in recent years has drawn increased attention to the importance of seismic performance evaluation and improvement of the existing structures to reduce seismic risks.



a.

b.

c.

d.

Figure 1. 1 Samples of buildings constructed framing only in one direction

During the conventional analysis of these buildings, floor slabs are considered to act as a rigid diaphragm irrespective of their topping thickness or the availability of supporting beams.

Due to the absence of beams in one direction, there will be large ductility demand on the columns. In addition, because of their flexibility, higher lateral drifts result during lateral load excitations. In turn, this affects the ultimate and serviceability capacity of the building.

In this thesis work , performance evaluation of three case study buildings constructed using ribbed slabs with hollow blocks framing only in one direction will be done by nonlinear static analysis.

1.2 Objective of the thesis

The main objective of this thesis is evaluating the performance of buildings constructed using ribbed slab framed only in one direction, and recommending appropriate measures if the buildings are found to be of poor performance.

The specific objectives of this thesis are:

- investigation of the rigidity of floor diaphragms in the longitudinal direction of the ribs,
- investigation of the performance of the structures against lateral loads,
- And suggestion of appropriate measures in order to strengthen the buildings if they are found to be with inadequate lateral load resistance capacity.

1.3 Organization of the thesis

Chapter 1 presents general description about the thesis including; background, general and specific objectives and thesis overview.

Chapter 2 presents literature survey on details of diaphragm and diaphragm flexibility, performance evaluation of buildings against earthquake, different method of analysis to estimate the demand required to resist a particular earthquake loading and measures which are used to retrofit and strengthen buildings which are inadequate to resist earthquake demands.

Chapter 3 contains general methods and procedures adopted for diaphragm flexibility analysis, and general methods and procedures for performance evaluation and retrofitting mechanisms adopted for evaluated case study buildings.

Chapter 4 contains general description about the case study buildings considered in this thesis work. This includes section properties, longitudinal and transverse reinforcement in every frame member of each building.

Chapter 5 contains the analyses, results analyses, summary and discussion of the analyses adopted in this thesis.

Chapter 6 contains conclusions and recommendations, and the last sections contain references used in this thesis work and Appendices.

Chapter 2: Literature review

2.1 Diaphragm

Floor and roof system in reinforced concrete building act as a diaphragm not only to transmit the inertia forces to the vertical structural systems, but also to insure that those systems to act together in resisting the horizontal actions [16].

Diaphragms shall be classified as rigid, stiff or flexible depending on their stiffness relative to vertical lateral load resisting system during lateral load excitation [3].

The manner and the degree to which the diaphragms participate in transferring lateral loads to the vertical lateral load resisting structures depends on this relative stiffness of the diaphragms. With a rigid diaphragm, the shear forces transmitted from the diaphragm to the vertical elements will be in proportion to the relative stiffness of the vertical elements. Otherwise, the lateral load is distributed according to tributary areas.

In current design practices, horizontal diaphragms of reinforced concrete are typically considered to have infinite stiffness to transmit the lateral loads. This consideration neglects the effect of their in plane movement relative to the vertical lateral load resisting system.

Even though many seismic codes rely on this rigid diaphragm assumption, as per M. Dolce et al., [1] some seismic codes say that the rigid diaphragm hypothesis cannot always be retained. For some geometrical and structural configurations, the actual force distribution among vertical resistance elements can differ considerably from that obtained with the rigid diaphragm hypothesis. The most important factors in this respect are: the ratio of plan dimension related to the position of the vertical elements, the presence and position of holes and/or reentrance in the floors, and the structural systems.

Classification and how to determine this categorization of diaphragms will be discussed in Section 2.1.1.

2.1.1 Classification of Diaphragms

According to EBCS 8 1995, the floor slabs are considered rigid if the in plane deviation of all points of the diaphragm from their rigid body position are less than (5%) of their respective absolute displacements under the seismic load combination.

FEMA 273 classifies diaphragm depending on the ratio of in plane deflection of the diaphragm to the story drift of adjoining vertical lateral force resisting elements. When the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story, it shall be considered as flexible. Whereas, it shall be considered rigid, when the maximum lateral deformation of the diaphragm is less than half of the average inter-story drift of the associated story. Diaphragms that are neither flexible nor rigid shall be classified as stiff. This will be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral force with the story drift of adjoining vertical lateral force resisting elements under equivalent tributary lateral force.

Particularly ribbed slabs, due to the presence of concrete topping and rigid ceiling materials between floors, are usually assumed to produce relatively rigid diaphragms. But, when the thickness of the concrete topping is very small they may not act rigidly. Especially, when the framing in one direction is missing, the floor will be susceptible to the in plane deflection due to the lateral load as well as out of plane deformation due to the gravity loads.

2.1.2 Effect of in plane deflection of diaphragms

Diaphragm flexibility results in an increase in the fundamental period of the building, decoupling of the vibration modes of the horizontal and vertical seismic framing, and modification of the inertia force distribution in the plane of the diaphragm.

2.2 Performance evaluation of building structures

Earthquake or seismic performance is an implementation of a building's or structure's ability to sustain their due functions, such as its safety and serviceability, at and after a particular earthquake exposure [3].

A structure is considered safe depending on the performance objective of the structure in performance based design and analysis. A performance objective specifies the desired seismic performance of the building. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion) [2]. As per ATC40, dual- or multiple-level performance objectives can be created by selecting two or more different desired performances, each for a different level of ground motion.

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building [2].

The earthquake ground motion can be expressed either by specifying a level of shaking associated with a given probability of occurrence (a probabilistic approach), or in terms of the maximum shaking expected from a single event of a specified magnitude on a specified source fault (a deterministic approach). The level of ground motion is expressed in terms of engineering characteristics for use in design. A response spectra or an equivalent series of simulated recordings of earthquake motions are used for this purpose [2].

A complete performance objective will be assigned by the building's owner (for a private building) or the state or local government agency acting as the owner (for a public building) for each building prior to evaluation or retrofitting [2].

There are different mechanisms of evaluation to assess the seismic performance of the existing buildings. These include laboratory works on specimens, in-situ tests on existing buildings or performance evaluation via application of different analytical methods.

2.2.1 Seismic performance analysis

Seismic performance analysis or simply seismic analysis is a major intellectual tool of earthquake engineering which breaks the complex topic into smaller parts to gain a better understanding of seismic performance of building and non-building structures[3].

Various analysis methods, such as elastic (linear) and inelastic (nonlinear) analyses, are available for the analysis of existing concrete buildings.

Elastic or linear analysis methods include static lateral force procedures, dynamic lateral force procedures and elastic procedures using demand capacity ratios.

When the linear static or dynamic procedures are used for seismic evaluation, the design seismic forces, the distribution of applied loads over the height of the buildings, and the corresponding displacements are determined using a linearly elastic analysis. It is difficult to obtain accurate results for structures that undergo nonlinear response through linear procedures.

The fundamental inelastic analysis method is the complete nonlinear time history analysis. Available simplified nonlinear analysis methods referred to as nonlinear static analysis procedures include, the capacity spectrum method (CSM), the displacement coefficient [4] method and the secant method [2]. The nonlinear static analysis, also known as push-over analysis, consists of laterally pushing the structure in one direction with a certain lateral force or displacement distribution until either a specified drift is attained or a structural instability is occurred.

Using one of these analysis procedures, the seismic performance of a structure can be evaluated and the deficiencies or weak elements can be found. But, because linear procedures have limitations and nonlinear dynamic procedures are complicated, nonlinear static analysis is commonly used [6].

2.2.2 Static non linear analyses/pushover analysis

As the name implies, it is the process of pushing horizontally with a prescribed loading pattern incrementally until the structure reaches a limit state.

In static non linear or pushover analysis, CSM uses the intersection of the capacity (pushover) curve and a reduced response spectrum to estimate maximum displacement. The displacement coefficient method (e.g., FEMA-273) uses pushover analysis and a modified version of the equal displacement approximation to estimate maximum displacement. But, the secant method uses substitute structure and secant stiffness to estimate the maximum displacement [2].

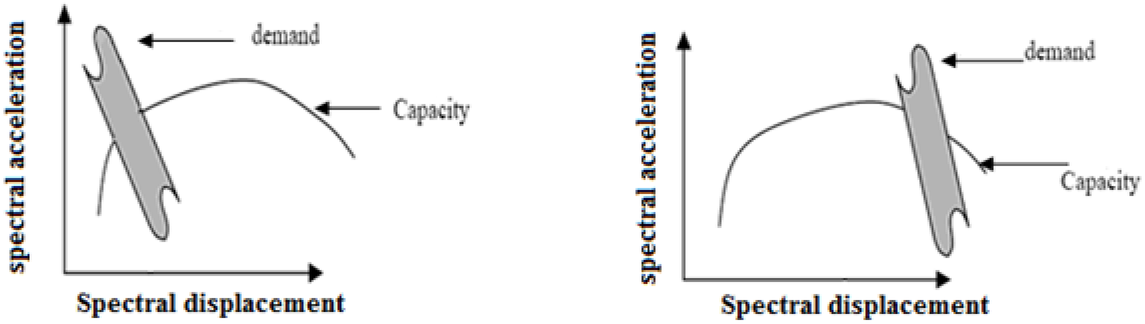
In this research work, the capacity spectrum method is used. This method provides a graphical representation of the global force-displacement capacity curve of the structure (i.e., pushover curve) and compares it to the response spectra representations of the earthquake demands. The graphical representation provides a clear picture of how a building responds to earthquake ground motion, and it provides an immediate and clear picture of how various retrofit strategies, such as adding stiffness or strength, will impact the building's response to earthquake demands [2].

In order to determine the performance of a building using capacity spectrum method of static nonlinear analysis, two basic elements, which are capacity and demand, must be determined. It is from these two key elements that the performance will be determined. They are discussed as follows.

i. **Capacity:** Capacity is a representation of the structure's ability to resist the seismic demand. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits, nonlinear analysis, such as the pushover procedure, is required.
ii. **Demand/displacement:** For a given structure and ground motion, the displacement demand is an estimate of the maximum expected response of the building during the ground motion.
iii. **Performance:** Once a capacity curve and demand displacement is defined, a performance check can be done using the intersection point of the demand and the capacity curves. This intersection point is the performance point of a building structure. This performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand.

It is also usually possible to estimate the performance of a structure from the manner in which the capacity spectrum and demand spectra intersects.

If the demand spectrum intersects the capacity spectrum near the elastic range (Figure 2.1 a), then this shows that the structure has a good resistance to the applied seismic load excitation. Whereas, if the demand spectrum intersects the capacity spectrum with little reserve of strength and deformation capacity (Figure 2.1 b), then it can be concluded that the structure will behave poorly during the imposed seismic excitation and it needs to be retrofitted to avoid future major damage or collapse [1].



a. Good performance

b. Poor performance

Figure 2. 1. Typical Demand Vs Capacity spectrum intersection [1]

Sometimes, the capacity spectrum and demand spectrum of some structures may fail to intersect. This would typically occur when local failure of structural elements prevents the capacity spectrum from intersecting the demand spectrum [2].

I. Procedures to determine capacity

ATC 40 offered the following procedures to determine the capacity curve of a given structure.

1. A computer model of the structure will be created according to the guidelines presented on ATC40 (refer to Appendix A).
2. The gravity and lateral loads will be applied on the structure. The lateral storey forces are applied in proportion to the product of mass and fundamental mode.
3. Member forces for the required combination of lateral and vertical load will be calculated. Then, the lateral force levels will be adjusted so that some element or group of elements is stressed within ten percent of its member strength.

4. After recording the base shear and the roof displacement at this step, the model will be revised using zero or very small stiffness for the yielding elements.

5. A new increment of lateral load to the revised structure will be applied such that another element (or group of elements) yields.

6. The increment of lateral load and the corresponding increment of roof displacement will be added to the previous total to get the accumulated value of base shear and roof displacement in other word, the next point on the capacity curve.

7. The above steps 4, 5 and 6 will be repeated until the structure reaches an ultimate limit such as: instability from P-Δ effects, distortions considerably beyond the desired performance level, an element (or group of elements) reaching a lateral deformation level at which significant strength degradation begins, or an element (or group of elements) reaching a lateral deformation level at which loss of gravity load carrying capacity occurs.

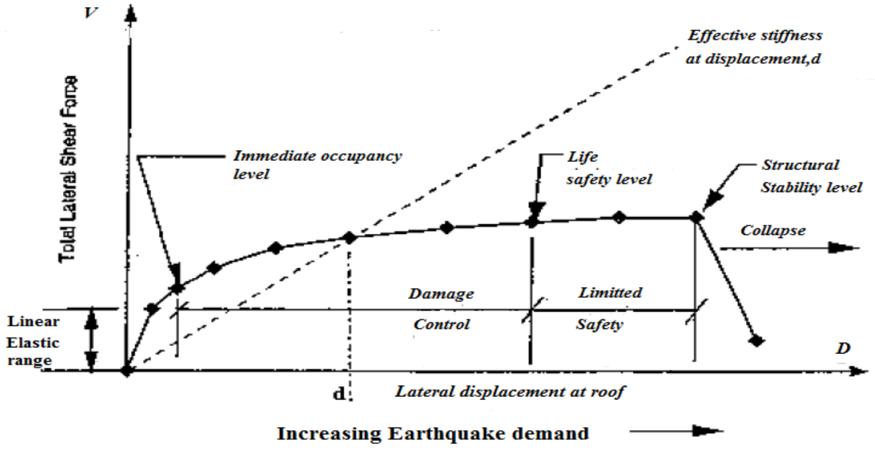


Figure 2. 2. Typical capacity curve [2]

8. The standard Sa versus T format of the capacity curve will be converted to Acceleration–Displacement Response Spectra (ADRS) format. The procedure is given on appendix B.

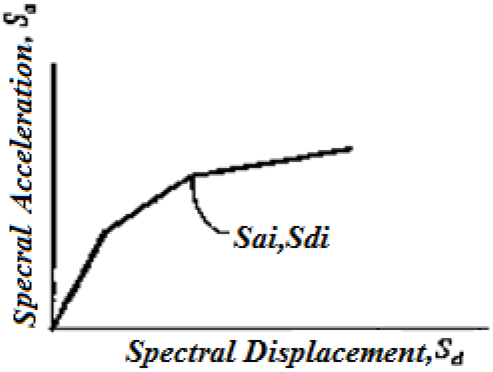


Figure 2. 3. Capacity Spectrum (ADRS format) [2]

II. Procedures to determine demand

The capacity spectrum method is based on finding a point on the capacity that also lies on the appropriate demand/response spectrum reduced for non linear effect.

The demand displacement in the capacity spectrum method occurs at a point on the capacity spectrum called the performance point.

In order to determine this demand point, the elastic response spectrum will be reduced to inelastic response or demand spectrum.

According to ATC 40, the determination of demand/displacement in capacity spectrum method includes the following procedures.

1. Conversion of the traditional Sa versus period (T) format of response spectrum to ADRS (S_a Vs S_d) format.
2. Estimation of the effective damping and appropriate reduction of spectral demand.

This is done by a bilinear representation of the capacity spectrum. Effective damping is a combination of the viscous damping, that is inherent in the structure, and hysteric damping. Hysteretic damping is related with the area inside the loops that are formed when the earthquake force (base shear) is plotted against the roof displacement. Hysteretic damping can be represented as equivalent viscous damping using equations that are available in literatures [2].

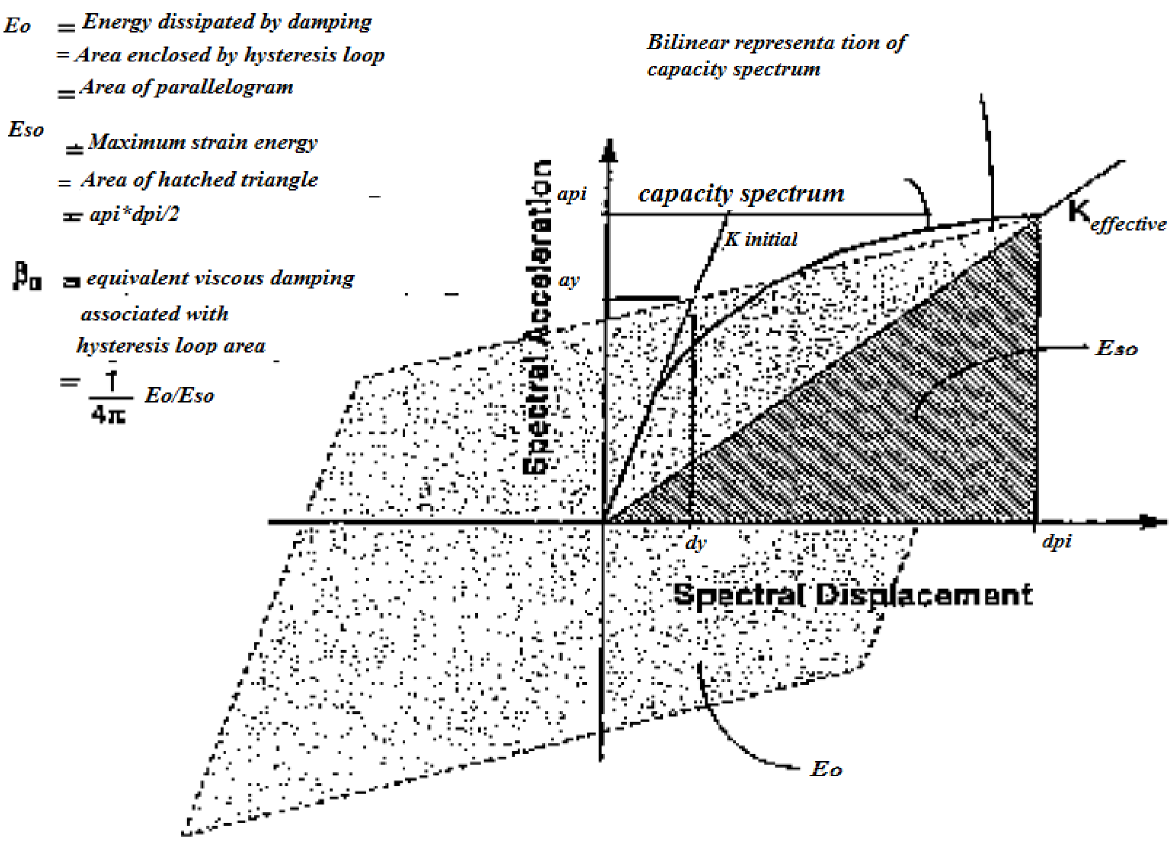


Figure 2. 4 Derivation of Damping For spectral Reduction [2]

The equivalent viscous damping associated with a maximum displacement of d_{pi}, can be estimated from the following equation:

$$ß_{eq}=ß_o+0.05 \qquad (2.1)$$

Where,

$ß_o$ = hysteretic damping represented as equivalent viscous damping

0.05 = 5% viscous damping inherent in the structure (assumed to be constant)

The term $ß_o$ can be calculated as (Chopra 1995) [2]:

$$ß_o= \frac{1}{4\pi} * \frac{Eo}{Eso} \qquad (2.2)$$

Where E_o = energy dissipated by damping (Figure 2.3)

E_{so}= maximum strain energy (Figure 2.3)

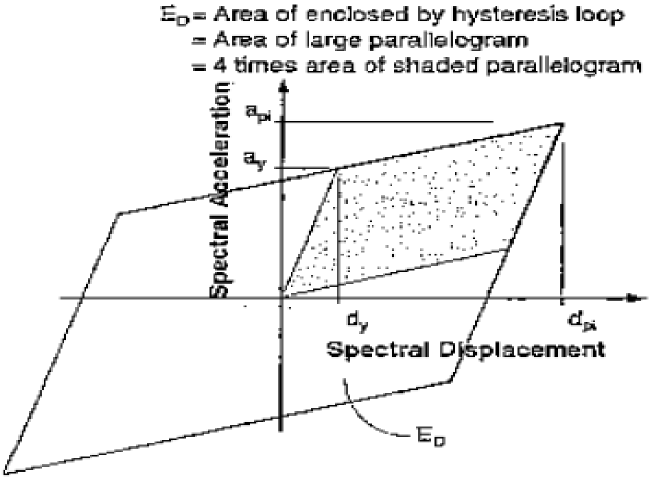


Figure 2. 5 Derivation of Energy Dissipated by Damping, Eo [2]

Referring to Figures 2.3 and 2.4, the term E_o can be derived as:

$$E_o = 4(a_y * d_{pi} - d_y * a_{pi}) \ [2] \tag{2.3}$$

Referring to Figure 2.3, the term E_{so} can be derived as

$$E_{so} = a_{pi} * d_{pi} / 2 \ [2] \tag{2.4}$$

And when $ß_o$ is written in terms of percent critical damping, the equation becomes:

$$ß_o = 63.7 * \frac{(ay*dpi - dy*api)}{api*dpi} \ [2] \tag{2.5}$$

Thus βeq becomes,

$$ßeq = ßo + 5 = 63.7 * \frac{(ay*dpi - dy*api)}{api*dpi} + 5 \ [2] \tag{2.6}$$

Where,

E_o= energy dissipated by damping

E_{so}= maximum strain energy

5 = percentage of viscous damping

The physical significance of the terms E_o and E_{so} in equation 2.2 is illustrated in Figure 2.3. For buildings that are not typically ductile, the concept of effective viscous damping using damping modification factor κ has been introduced in order to be consistent with these previously developed damping coefficients as well as to enable simulation of imperfect hysteresis loops[2].

$$ß_{eff} = \kappa\, ßo + 5 = 63.7\kappa * \frac{(ay*dpi - dy*api)}{api*dpi} + 5 \tag{2.7}$$

The κ factor is a measure of the extent to which the building hysteresis is well represented by the parallelogram, either initially or after degradation. The κ –factor depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking [2].

For simplicity, ATC 40 simulates three categories of structural behavior Type A, Type B and Type C.

Type A represents stable, reasonably full hysteresis loops most similar to Figure 2.5, and is assigned a κ of 1.0. Type B is assigned a basic κ of 2/3 and represents a moderate reduction of area. Type C represents poor hysteretic behavior with a substantial reduction of loop area (severely pinched) and is assigned a κ of 1/3.

3. Numerical determination of spectral reduction

Once $ß_{eff}$ is determined, the reduction factors in the acceleration and velocity regions, SRA and SRV respectively, are determined from equations below.

$$SRA = \frac{3.21 - 0.68 \ln (ßeff)}{2.12} \quad (2.8)$$

SRA is greater than or equal to: 0.3 for type A, 0.44 For type B and 0.56 For type C [2].

$$SRV = \frac{2.31 - 0.41 \ln (ßeff)}{1.65} \quad (2.9)$$

Whereas SRV is greater than or equal to: 0.5 for Type A buildings, 0.56 for Type B buildings, and 0.67 for type C buildings [2].

4. The reduced response spectrum will be plotted.

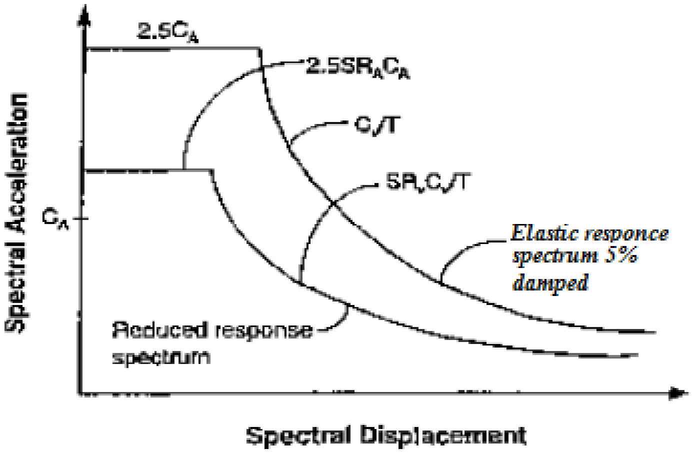


Figure 2. 6 Reduced response spectrum

5. The performance point will be determined.

To find this point where demand and capacity are equal, a point on the capacity spectrum as an initial estimate will be selected. Using the spectral acceleration and displacement defined by this point, the reduction factors will be calculated as it is expressed in steps 2 and 3[2].

If the reduced demand spectrum intersects the capacity spectrum at or near the initial assumed point, then it is the solution for the unique point where capacity equals demand. In other word, when the displacement at the intersection of the demand spectrum and the capacity spectrum, di, is within 5 percent ($0.95d_{pi} <= d_i <=1.05\ d_{pi}$) of the displacement of the trial performance point (api, dpi), (api, dpi) becomes the performance point. If the intersection of the demand spectrum and the capacity spectrum is not within the acceptable tolerance, then a new a_{pi}, d_{pi} point is selected and the process is repeated. Figure 2.7 illustrates this concept [2].

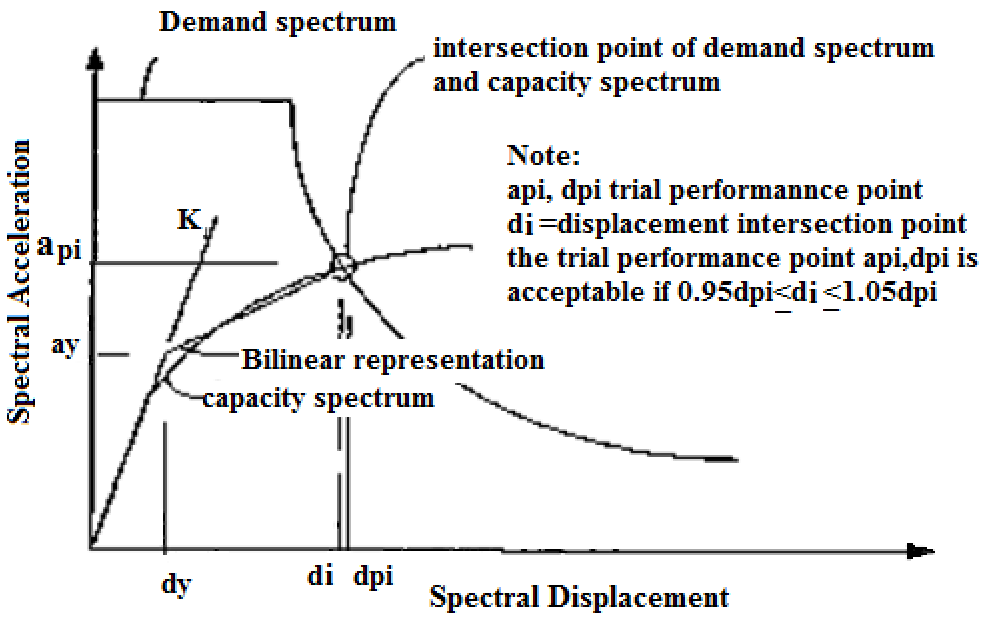


Figure 2. 7 Intersection point of Demand and capacity spectra Within Acceptable Tolerance [2]

2.2.3 Static nonlinear/pushover analysis using ETABS

Nonlinear computer programs like, SAP2000 and ETABS are able to perform a pushover analysis directly. Therefore, the step by step method discussed above will be directly applied by the programs if such nonlinear computer programs are used[2].

In static non linear analysis of ETABS, several types of non linear behavior can be considered. This includes material nonlinearity at discrete, user-defined hinges in frame/line elements, material nonlinearity in the link elements, and geometric nonlinearity in all elements and so on.

Static nonlinear analysis cases are completely independent of all other analysis types in ETABS. The distribution of load applied on a structure for a given static nonlinear case is defined as a scaled combination of one or more of the following:

1. Any static load case.
2. Uniform acceleration acting in any of the three global directions. The force at each joint is proportional to the mass tributary to that joint and acts in the specified direction.

3. A modal load for any Eigen or Ritz mode. The force at each joint is proportional to the product of the modal displacement, the modal circular frequency squared (ω^2), and the mass tributary to that joint, and it acts in the direction of the modal displacement.
4. Any of the methods described in 1, 2 and 3 can be combined.

Depending on the physical nature of the load and the behavior expected from the structure, two different types of control are available for applying the load: force and displacement controls. The Procedure for performing static non linear analysis by ETABS will be given next.

The following general sequence of steps is involved in performing nonlinear static push over analysis by ETABS:

- i. Creation of a building model as it would be done for any analysis.
- ii. Definition of frame hinge properties and assignment to the frame/cable elements.
- iii. Definition of static and dynamic analysis cases that may be needed for steel or concrete design of the frame elements, particularly if default hinges are used.
- iv. Running the analysis cases needed for design (if required).
- v. If any concrete hinge properties are based on default values to be computed by the program, concrete design will be performed so that reinforcing steel is determined.
- vi. Definition of the load cases that are needed for use in the push over analysis, including gravity loads and other loads that may act on the structure before the lateral seismic loads are applied.
- vii. Definition of lateral loads that will be used to push the structure.
- viii. Definition of the nonlinear static analysis cases to be used for pushover analysis, including:
    - a) A sequence of one or more cases that start from zero and apply gravity and other fixed loads using load control. These cases can include geometric nonlinearity.
    - b) One or more push over cases that start from this sequence and apply lateral pushover loads. These loads should be applied under displacement control. The monitored displacement is usually at the top of the structure and will be used to plot the push over curve.

ix. Running of the pushover analysis cases.

x. Revision of the pushover results. Plot of the push over curve, the deflected shape showing the hinge states, force and moment plots, and print or display any other results needed.

xi. Revision of the model if necessary.

Several types of outputs can be obtained from the static nonlinear analysis of ETABS. These include: Plot of Base Reaction versus Monitored Displacement, tabulated values of Base Reaction versus Monitored Displacement at each point along the pushover curve, Plot of Base Reaction versus Monitored Displacement in the ADRS format, tabulated values of the capacity spectrum(ADRS capacity and demand curves), Graphical view of the sequence of hinge formation and the color-coded state of hinge, and graphical view of the member forces and stresses on a step by step basis.

2.2.4 Non-linear modeling of beams and columns

To create the proper behavior of the structure in post elastic state under a particular earthquake, the nonlinear modeling of the structural elements is very essential. Beams may develop inelastic response associated with flexural, shear, development splice and slip of bars embedded in joints. But, usually a response associated with flexure is the controlling mode. Where inelastic flexure is the controlling mode, this response may be represented by using concentrated hinge models [9].

Column element has been modeled as frame component which can have substantial axial forces and biaxial bending moments. The same modeling procedure can be employed, but it has to take in to account the effect of axial load and hence it is necessary to consider P-M-M interaction.

2.3 Mechanism of retrofitting/strengthening

Repair and retrofitting of concrete structures have been attracting the attention of researchers over the last two decades [7]. Once seismic performance evaluation is conducted and the presence of unacceptable seismic deficiencies has been detected, evaluated structures are retrofitted using different mechanisms of retrofitting. Unless the structures are with extremely poor performance, different mechanisms of retrofitting are recommended, evaluated and applied. Otherwise, if the structure under consideration is with very bad or unrecoverable performance, or if the cost of retrofitting is exceedingly large, the structure will have to be demolished.

In order to select proper mechanism of retrofitting for a structure, several factors should be considered and detailed analysis of these factors should be done. The factors include the building's structural characteristics, the intended performance objectives, design and construction cost limits, the project schedule, historic preservation requirements (for historically valuable structures), the effects on building appearance and floor space layout, the effects on building occupancy both during and after project construction, and issues of project risk [2].

In general, these mechanisms of retrofitting will be categorized into two and discussed. The first category includes the systems which modify capacity of the building to withstand earthquake-induced forces and deformations. And the second category includes mechanisms which modify the response of the structure such that the demand forces and deformations are reduced.

2.3.1 Methods applied to change the capacity of the building

i. System completion

System completion approaches are applicable on structures that have the basic components of an adequate lateral force resisting system but lack some details required to make the system complete. Also, they are applicable on structures that miss some of the basic components of lateral force resisting systems[2].

To ensure that the system behaves as intended, system completion mechanisms through provision of the missing detailing or elements will be adopted. Often, this strategy must be implemented together with other strategies to obtain a building with the desired seismic performance capabilities[2].

ii. Strengthening/ stiffening systems

These are the most common seismic performance improvement mechanisms adopted for buildings with inadequate lateral force resisting systems [2].

The effect of strengthening a structure is to increase the amount of total lateral force required to initiate damage events within the structure. If this strengthening is done without increasing stiffness, then the effect is to permit the structure to achieve larger lateral displacements without damage. This will be achieved when relatively local-retrofit measures that strengthen existing elements without greatly altering their stiffness are taken [2].

Since most systems that will strengthen a structure also simultaneously stiffen it, system strengthening and stiffening are nearly always performed as concurrent strategies. Similarly, stiffening techniques also usually result in strength increase [2].

Typical systems employed for stiffening and strengthening include, the addition of new vertical elements, including shear walls, infilled structural walls, bracings/braced frames, buttresses, or moment resisting frames. Some of them are briefly discussed below [2].

a. Shear walls

Shear walls are vertical elements that resist lateral loads in their plane [18]. These systems are often economical and tend to be readily compatible with most existing concrete structures [2]. Addition of shear walls is an extremely effective method of increasing both building strength and stiffness [2].

Shear walls can often result in significant architectural impact through the loss of windows and the introduction of barriers within areas of floor space. They also tend to produce large overturning forces at their bases that may require supplemental foundation work; which is often expensive. Applying large number of shear walls has some adverse effect on the building structure like a significant increase in building mass and therefore increases seismic forces and strength requirements [2].

b. Structural infilled walls

Structural infilled walls are new structural walls attached to the frame or beam elements of reinforced concrete framed building to provide strength and stiffness to the building. Structurally they are shear walls consisting of existing frame working together with the infill.

They can be totally in filled walls or side walls as shown in the figures below. Generally, they can be cast in-situ concrete infilling existing bare frame or replace non structural walls. Steel panels may also be an element to infill the existing frame [2].

This solution requires adequate strength in the existing columns for the overturning forces from the infilled shear walls. The existing column should also have adequate shear capacity as high shear stress will result if the movement occurs between the new infilled panel and the beam [10].

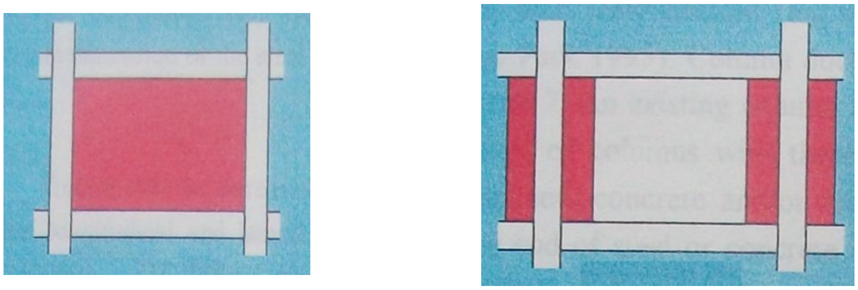


Figure 2. 8 Infilled Walls [8]

c. Braces/Braced frames

Braces or braced frames are steel or RC members provided among the frame of the structure to provide stiffness and strength to the building structure. They can be compression or tension braces, tension and compression braces, steel or concrete braces.

Braced frames provide lower levels of stiffness and strength than do shear walls. But, they add far less mass to the structure than shear walls. They can be constructed with less disruption of the building, result in less loss of light, and have a smaller effect on traffic patterns within the building.

The difficulty encountered in such systems is connection detail of braced frames to a concrete building. This is because relatively large forces must be transferred between the structure and the braced frames [2]. Besides, high shear strength and ductility capacity of the existing beams and columns are required to add the diagonal braces to form a vertical truss of existing beam, column and new diagonals [10].

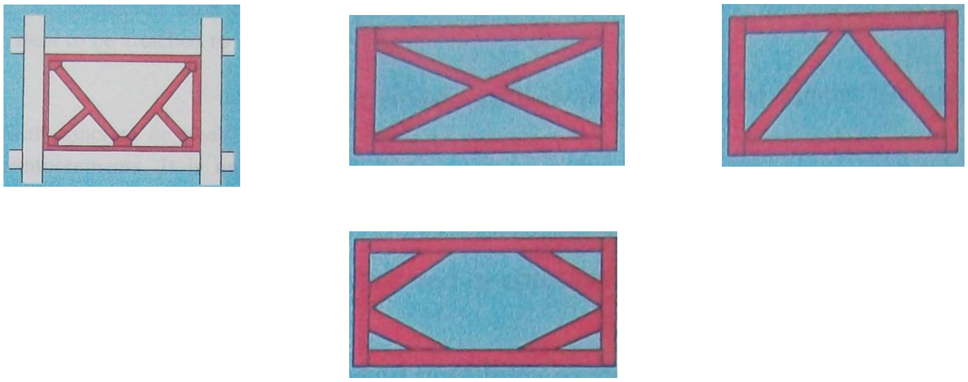


Figure 2. 9 Some of different types of bracings [8]

d. Buttresses

Buttresses are braced frames or shear walls installed perpendicular to an exterior wall of the structure to provide supplemental stiffness and strength. This system is often a convenient one to use when a building must remain occupied during construction, as most of the construction work can be performed on the building exterior, minimizing the inconvenience to the building occupants [2].

The two most difficult problems in strengthening a building by buttresses are:

i. The buttress stability may be critical since it is not actually loaded vertically downwards in the same way that the structure is. The vertical action on the buttress is only its own weight. This increases the possibility of uplifting of the foundations and may even cause over-turning [17].
ii. The connections between the buttresses on the one hand and the building on the other is far from straightforward. To insure full interaction and load sharing when the structure is subjected to lateral actions, the buttress should be connected to the floors and columns at all levels. The connection area will be subjected to unusual levels of stresses that require special attention [17].

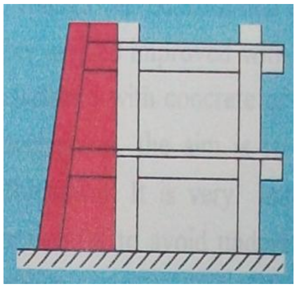


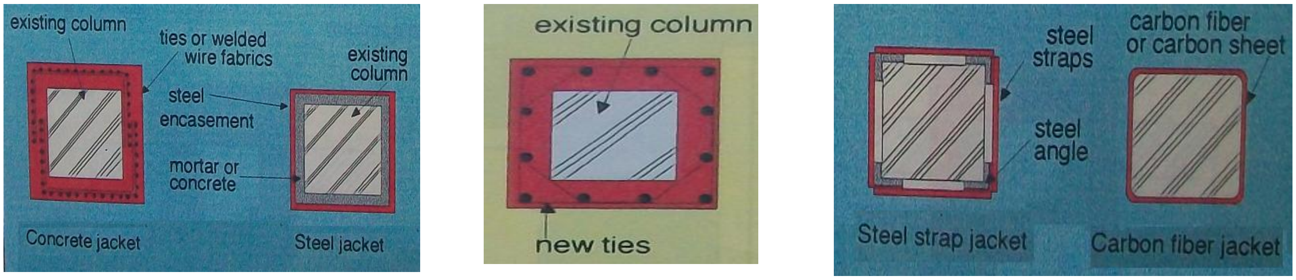
Figure 2. 10 Buttresses (peripheral frame buttresses) [8]

iii. Deformation capacity enhancing systems

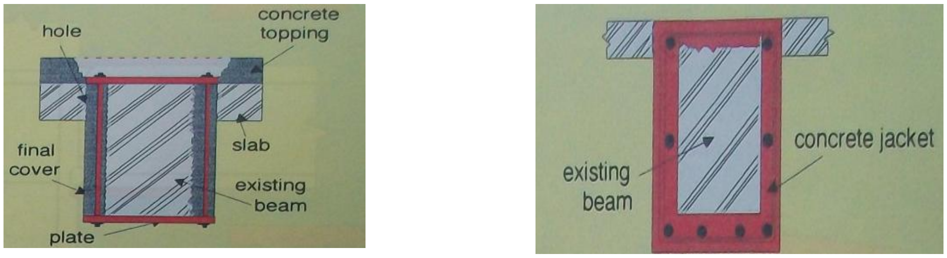
Systematic methods of enhancing deformation capacity include adding confinement to existing elements, making local reductions in stiffness, and providing supplemental support at areas subject to deformation-induced failure.

This strategy is typically most effective when the necessary enhanced deformation capacity can be obtained by modifying only a few existing elements. Otherwise it becomes quite costly and disruptive of occupancy during construction [2].

The deformation capacity of non ductile concrete columns and beams can be enhanced through provision of exterior confinement jacketing. Jacketing may consist of continuous steel plates encasing the existing element, reinforced concrete, and fiber-reinforced plastic fabrics as shown in Figure 2.10.



a. beams



b. columns

Figure 2. 11 Different confining methods to existing RC beams and columns [8]

2.3.2 Systems applied to reduce earthquake demands on a building

This approach involves modification of the response of the structure such that the demand forces and deformations are reduced rather than modifying the capacity of the building to withstand earthquake-induced forces and deformations. In effect, the demand spectrum for the structure, rather than the capacity spectrum, is modified.

Methods for achieving this strategy include reductions in the building's mass and the installation of systems for base isolation and/or energy dissipation. The installation of these special protective systems within a building typically entails a significantly larger investment than do more conventional approaches [2].

1) Base isolation

This approach requires the insertion of compliant bearings within a single level of the building's vertical load carrying system, typically near its base. The bearings are designed to have relatively low stiffness, extensive lateral deformation capacity and may also have superior energy dissipation characteristics [8].

Installation of an isolation system results in a substantial increase in the building's fundamental response period, and potentially, its effective damping[2].

Since the isolation bearings have much greater lateral compliance than does the structure itself, lateral deformation demands produced by the earthquake tend to concentrate in the bearings themselves [2].

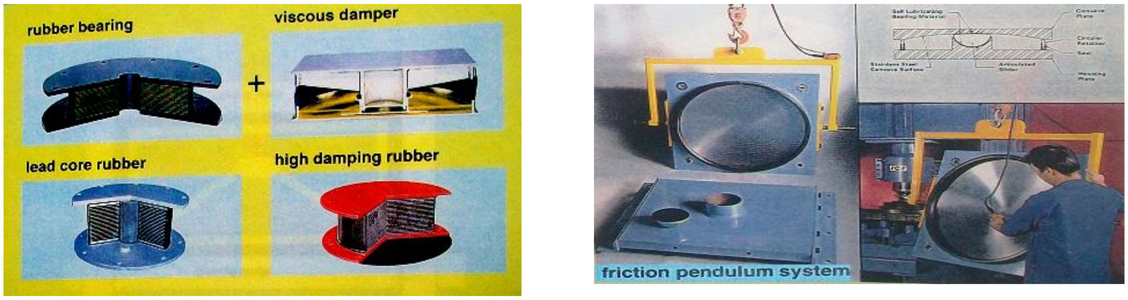


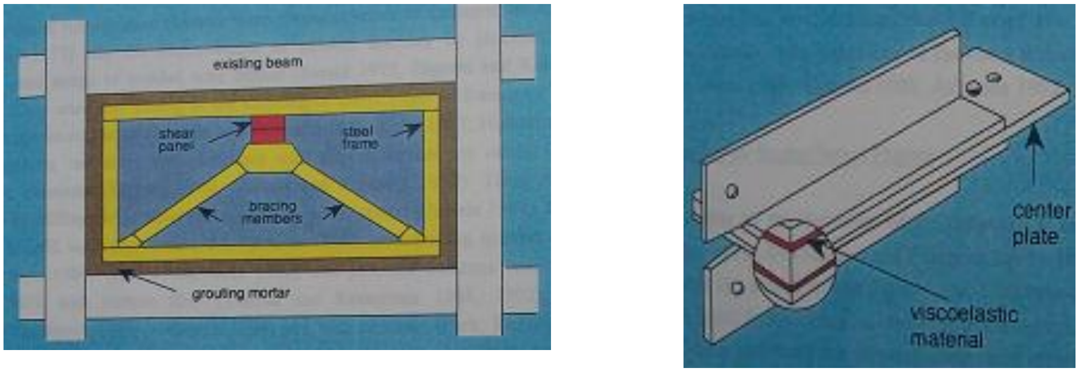
Figure 2. 12 Isolation bearings used for seismic retrofitting [8]

2) Energy dissipation systems

Energy dissipation systems directly increase the ability of the structure to dampen earthquake response in a benign manner, through either viscous or hysteretic damping. This approach requires the installation of energy dissipation units (EDUs) within the lateral force resisting system[2].

The EDUs dissipate energy and in the process reduce the displacement demands on the structure. The installation of EDUs often requires the installation of vertically braced frames to serve as a mounting platform for the units and therefore, typically results in a simultaneous increase in system stiffness[2].

Energy dissipation systems are most effective when installed in structures that have a significant lateral deformation capacity without adequate ductility. The amount of energy dissipated by these systems is directly proportional to the force developed by the individual EDUs and the displacement across the EDUs. If a building is relatively rigid, the energy dissipation system will not be able to effectively dampen its response before damage has occurred. Therefore, these systems are most applicable to frame structures[2].



a. Shear yielding damper

b. Visco elastic damper

Figure 2. 13 Some of Seismic dampers used for seismic retrofitting [8]

3) Mass reduction

The performance of some buildings can be greatly improved by reducing the building mass. Building mass reductions reduce the building's natural period, the amount of inertial force that develops during its response, and the total displacement demand on the structure. Mass can be reduced by removing heavy nonstructural elements, such as cladding, water tanks, and storage. In the extreme cases, mass reduction can be attained by removing one or more building stories.

2.3.3 Preliminary design of retrofitting systems

A preliminary design of retrofitting systems include the approximate size, number and preferred locations for all major elements of the retrofit system including braced frames, shear walls, buttresses, base isolators, and energy dissipation units [2]. The choice of the type, number and size of added elements depend on the particularities of the existing structure and the functional layout of the building [10].

The first step in the preliminary design process is to prepare and overlay demand and capacity spectra for the unretrofitted structure. The performance point for the unretrofitted structure should be determined and the behavior of the structure at this performance point should also be understood [2].

If the structure is incapable of meeting the desired performance objectives at the performance point, the specific deficiencies should be identified. Then, the initial dimensions of retrofitting systems will be determined in two ways depending on the retrofit strategy selected as per ATC40. If the deficiency identified from the performance evaluation indicates that the measure should be taken is only increasing the stiffness and strength or only increasing the energy dissipation capacity of the building by base isolation, ATC 40 has some procedures.

If the strategy is different from increasing the stiffness and strength, base isolation and energy dissipation capacity of the building or if it includes additional measures ATC 40 recommends performing a trial and error process. In this process a retrofit design is assumed, the resulting capacity and demand spectra developed, and the performance point determined. If the assumed design results are in an acceptable performance point for the project performance objectives, then the design is adequate. If not, then the design must be revised and the process repeated or an alternative strategy employed.

Regarding the location and dimensioning of retrofitting structural elements, some general outline principles were given by Prof. Dr. Predrag Gavrilovic [10]. These outlines are helpful for the trial and error process.

As per his study, incorporation of new structural components in an existing building will change the dynamic behavior of the whole space structure considerably during an earthquake. The increase in stiffnesses will also tend to increase the seismic design forces for most typical structures. It also causes considerable distribution of the lateral forces between the earthquake resisting elements. Therefore, it is very important that the most favorable conditions are created whenever possible as follows:

a. Avoid large concentration of forces in members with small strength and/or ductility capacities and locate the strengthening elements uniformly throughout the structure.

b. Improve the distribution of lateral force by reducing the effect of torsion and irregularities.
c. Provide sufficient strength, stiffness and ductility of the individual elements and the whole structure.
d. Provide adequate strength in connection between the existing structures and newly added elements.
e. Provide stiffness compatibility of the existing and newly added elements.

Some examples of favorable added elements are also given for pure skeleton structures with insufficient lateral resistance. Shear walls should be added in architecturally convenient places [10]. Particularly, in long, rectangular buildings the shear walls placed at the corners and oriented along the longitudinal direction limit the deformation due to temperature changes and can produce considerable additional internal forces in the structure. Therefore, it is desirable that the shear walls be oriented in transverse direction at building ends as given in Figure 2.14 [10].

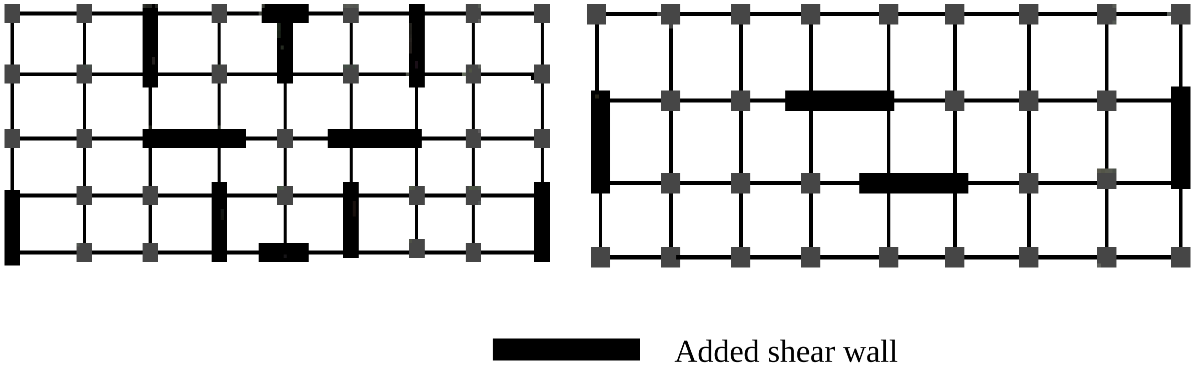


Figure 2. 14 Convenient distribution of added shear walls for long rectangular buildings [10]

Chapter 3: Methods and procedures

3.1 General

This chapter presents the analytical procedures and measures taken for evaluating the flexibility of diaphragm and seismic performance of the three case study buildings in addition to seismic retrofitting mechanisms adopted.

3.2 Evaluation procedures for determining diaphragm flexibility

As it is discussed previously in Section 2.1 diaphragms can be classified as rigid, stiff or flexible depending on their relative stiffness relative to vertical lateral load resisting system during lateral load excitation.

This deformation can be determined by comparing the midpoint in-plane deflection of the diaphragm with the story drift of the adjoining vertical resisting elements under equivalent tributary load as per FEMA 273. But in EBCS, there is only general explanation about the condition when to consider a diaphragm as rigid. Therefore, in this thesis work, to determine the flexibility of the diaphragm, the method according to FEMA 273 is adopted.

To evaluate diaphragm rigidity or flexibility, equivalent static analysis method is used. The lateral load is distributed according to tributary areas.

The gravity loads consist of dead loads from member self-weight, walls on the exterior beams, floor finishes on the ribs and dead loads from the non structural hollow blocks on the ribs. The live load is also considered at each floor level [23].

Design earthquake loads are determined and the base shear is distributed according to the Ethiopian Buildings Code Standard 8 1995 (EBCS8 1995). The earthquake loads are applied according to the tributary area to the buildings.

The beams and columns are the basic lateral load resisting system in the case study buildings. The intermediate beams in one direction are missed.

Normal weight concrete with characteristic cube compressive strength of 25 MPa is used for all members and the specified characteristic yield strength for reinforcements is 400 MPa.

As it is discussed in Section 2.1.1, FEMA 273 classifies floor diaphragms as flexible, stiff, or rigid depending on the ratio of in plane deformation to the average inter- story drift of the corresponding floor. The criteria of FEMA 273 are given Table 3.1.

Usually, the deformation of the diaphragm is measured at the story level below the roof to check the flexibility of a diaphragm [24].

Table 3. 1 FEMA 273 criteria for floor diaphragms classification

Classification	Criteria
Rigid	Δdiaphragm<0.5*ΔStory
Stiff	0.5*ΔStory<Δdiaphragm<2*ΔStory
Flexible	Δ diaphragm>2*ΔStory

3.2.1 Members modeling

Frame elements are used to model the ribbed slabs, the columns and the beams. The ribbed slabs are modeled as a tee beam. But, because of the connections at their ends (case study buildings A and C) and their reduced cross sectional dimensions (case study building B), they are modeled as a simply supported beams.

3.2.2 Stiffness modeling

Under seismic load analysis and design, due to cyclic nature of the earthquake excitation, flexural members exhibit moment reversal along their length. And at any section the moment of inertia, I, will be affected. Hence member stiffness shall be calculated considering shear, flexure, axial behavior and reinforcement slip. The moment of inertia, Ig, should be modified to take in to account the above phenomenon. ATC 40 (Table 9-3) recommends the following member stiffness and has been used in this study.

Table 3. 2 Effective stiffness values

Component	**Flexural Rigidity**	**Shear Rigidity**	**Axial Rigidity**
Beams non pre-stressed	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
column with compression	$0.7\ E_cI_g$	$0.4\ E_cA_w$	E_sA_s

3.3 Performance evaluation of case study buildings

The performance of the buildings under consideration is evaluated using capacity spectrum method of non linear static analysis. The efficiency of the capacity spectrum method as a "displacement-based" analysis tool for seismic assessment was illustrated using the results of a study into the seismic resistance of existing RC frame plus bearing (shear) wall buildings in Switzerland [22].

In this thesis work, this analysis has been performed using ETABS 9.2.0, which is a full-featured program that can be used for building analysis of non linear static and dynamic finite element analysis of building structures.

The gravity loads are applied, and then lateral loading is applied first in X-direction starting at the end of the gravity push, and then in Y-direction again starting at the end of the gravity push.

The concept of plastic hinge is extremely important in the nonlinear analysis. While a concrete element undergoes large deformations in the post-yield stage, it is assumed that all the deformation takes place at a point called "plastic hinge". The plastic hinge properties for beams and columns used in ETABS are based on ATC 40 [2] (Table 9-6 and 9-7) as shown in Figure 3.2. Five points labeled A, B, C, D and E define the load-deformation behavior of the plastic hinge. The values assigned for each of these points vary depending on the types of elements, material properties, longitudinal and transverse steel contents.

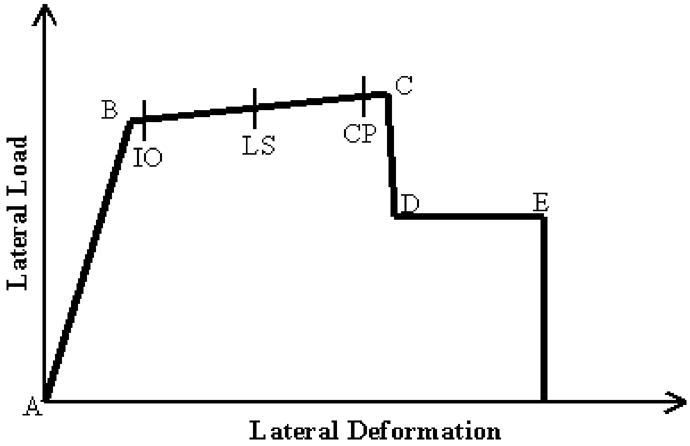


Figure 3. 1 Load-deformation behavior of a typical plastic hinge [19]

The three points labeled IO, LS, and CP (SS) representing immediate occupancy, life safety and collapse prevention (or Structure stability [2]) respectively shown in Figure 3.1, define the numerical acceptance criteria for plastic hinge. Their values used in ETABS are based on ATC 40 (Table 11.3and 11.4).

In this thesis, dual level performance objectives are created by selecting serviceability earthquake SE and design earthquake DE ground motion levels. The operational and life safety building performances for each level of ground motions respectively are selected as per the recommendation of ATC40.

According to ATC40 the SE represents a frequent level of ground shaking that is likely to be felt during the life of the building. It has a mean return period of approximately 75 years. The DE represents an infrequent level of ground shaking that can occur during the life of the building. The DE has a mean return period of approximately 500 years. The DE has the same definition as the level of ground shaking currently used as the basis for the seismic design of new buildings by the UBC [2].

The PGA values given on EBCS8 1995, have a return period of 100 years. This is approximately equal to that of SE level on ATC 40. Due to this all the values of PGAs for different seismic zones of EBCS8 1995 are considered as SE level.

For the DE many researchers presented different values for different seismic zones of the country. Even though there should be still further analysis and study, for seismic Zone of two, PGA of 0.12 to 0.5g are suggested. Whereas for seismic zones of four PGA of 0.19g to 0.75g are suggested. But for this thesis work recommendations given by Samuel Kinde[19] for seismic zone of two which is 0.1g for seismic zone of two is used. This value is twice to that of given on EBCS 8 1995. Extending this for seismic zone of four, PGA of 0.2g is used [19]. Therefore, for seismic zones of two and four, 0.05g and 0.1g are taken as PGA at SE level, and 0.1g and 0.2g as DE level .

To determine the demand curve, the 9.2.0 version of ETABS typically uses UBC 97 design spectrum. Sub Soil class of S_d is used for the performance evaluation. This is because ATC 40 recommends using this soil classification when the soil properties are not known in sufficient detail to determine the Soil Profile Type.

In order to make the results from this spectrum near to that of EBCS 8 elastic response spectrum, the values of C_a and C_v are determined depending on the seismic zone and soil classification under consideration. The values of C_a and C_v, for each seismic zone, are obtained from Tables 4-7 and 4-8 of ATC40 for soil class S_d.

3.4 Retrofitting mechanisms adopted for evaluated case study buildings

Detailed analyses including factors like design and construction costs; effect of building occupancy both during and after construction is beyond the scope of this thesis. Therefore, the governing factors during selection of retrofitting systems are the building structural characteristics after application of the systems and the building floor space layout.

The selected mechanisms are applied on the buildings, and the buildings are analyzed using pushover analysis.

Chapter 4: Case study buildings

4.1 General

Three case study buildings are taken for performance evaluation and diaphragm flexibility check. Two case study buildings (case study building B and C) are located in seismic zone of two while the third building (case study building A) located in seismic zone of four in the course classification of EBCS8. There are no holes, but insignificant reentrance in the floors (case study building B) and the lateral load resisting structural systems on the buildings are moment resisting frames.

The case study buildings include two ground plus five (case study building B and C) and one ground plus two (case study building A) buildings. For case study buildings A and B the ribs are oriented in the longer direction of the building. But, in case study building B the ribs are oriented in the shorter direction of the building. The detailed description of each case study building will be given in the sections below.

4.2 Case study building A

This case study building has a plan dimension of 24.05m by 8.6m. The height of the building, from ground level to the roof is equal to 9.09m. Each storey has equal height of 3.06m except the third storey which is 2.91m. Ribs are spaced in every 600mm.

In this case study building, the cross sectional dimension of all the beams, except the top tie beams, are 480 mm depth and 200 mm width. The roof or top tie beams are of cross section 300mm depth and 200 mm width. The clear cover to the longitudinal reinforcement is 25 mm. The cross sectional dimension of the columns is 400mm by 200mm. The reinforcing detail for this building is given in Table 3.2.

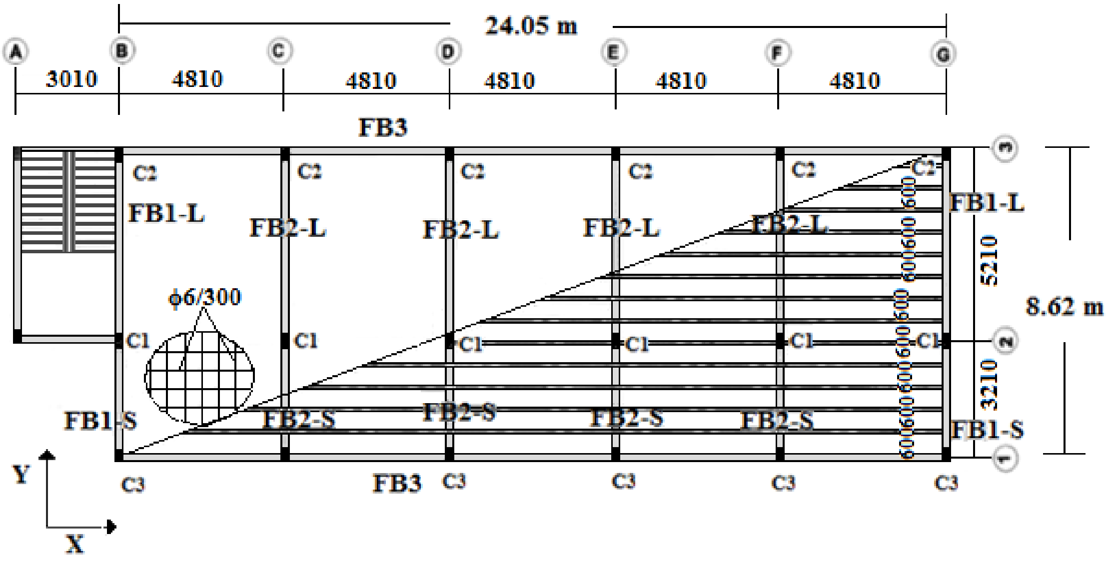
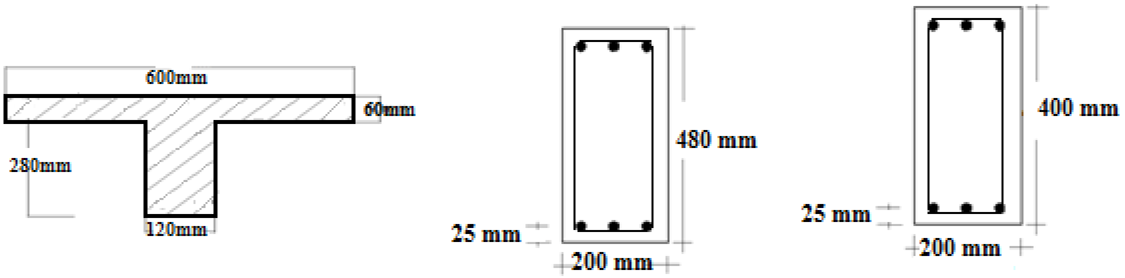


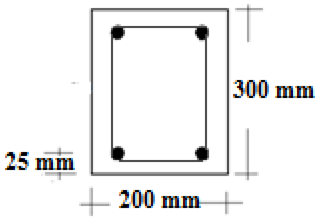
Figure 4. 1 Typical floor plan of case study building A



a) Rib

b) Beam

c) Column



d) Top tie beams

Figure 4. 2 Members’ cross sectional dimentions of case study building A

Table 4. 1 Reinforcement details of beams for case study building A

Typical floor beams		# and size of bars	Typical floor beams		# and size of bars
FB1S	Bottom	2 ø14	FB2L	Bottom	3ø20
	Top	2 ø14		Top	4 ø20
	Transverse	ø10 cc 200		Transverse	ø8 cc 200
FB1L	Bottom	2 ø14	Roof tie beams		# and size of bars
	Top	4 ø14	RB1	Bottom	2 ø14
	Transverse	ø10 cc 130		Top	2 ø14
FB3	Bottom	2 ø14		Transverse	ø8 cc 200
	Top	2 ø14	RB2	Bottom	2 ø14
	Transverse	ø8 cc 200		Top	2 ø14
				Transverse	ø8 cc 200

Table 4. 2 Reinforcement details of columns for case study building A

Column	Floor	# and size of bars	Column	Floor	# and size of bars
C1	First	8ø20	C3	First	6ø16
	Second	6ø20		Second	6ø16
	Third	6ø16		Third	6ø16
C2	First	6ø20			
	Second	6ø16			
	Third	6ø16			

4.3 Case study building B

This case study building has a plan dimension of 27 m in the longer direction, 15.9 m in the shorter and 18.6 m in the longer directions of the transverse direction. The height of the building from ground level to the roof is equal to 15.3m. Each storey has equal height of 3.06m. Rib beams are spaced in every 400 mm, unlike the other two case study buildings. Typical floor plan layout with its dimension, orientation of beams and columns is shown on Figure 4.2.

In this case study building, the cross sectional dimension of all columns is 300 mm by 300 mm. All the intermediate beams have a cross sectional dimension of 300 mm depth and 800 mm width. But, the exterior beams have a cross sectional dimension of 300 mm depth and 500 mm width. The roof or top tie beams are of cross sections 300 mm in depth and 250 mm in width. The clear cover to the longitudinal reinforcement is 25 mm. The reinforcing detail for this building is given in Tables 4.3 and 4.4.

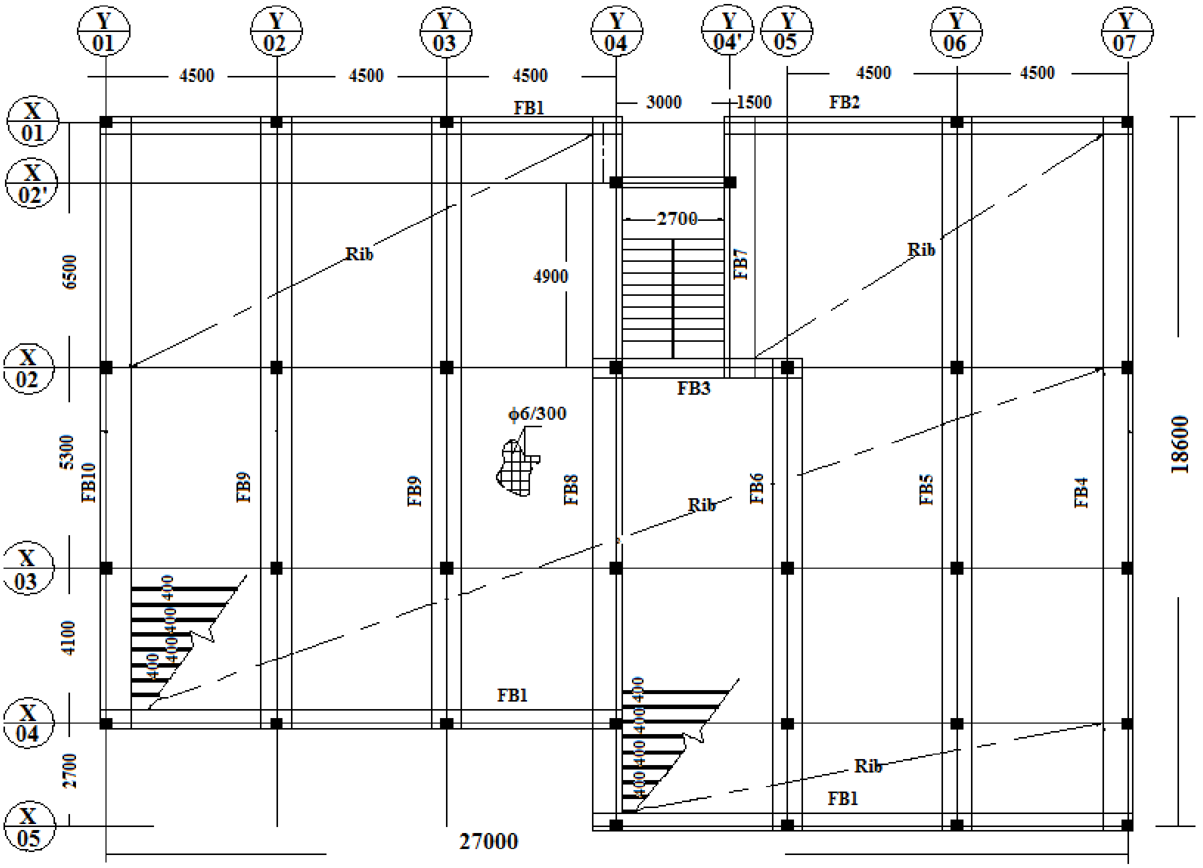
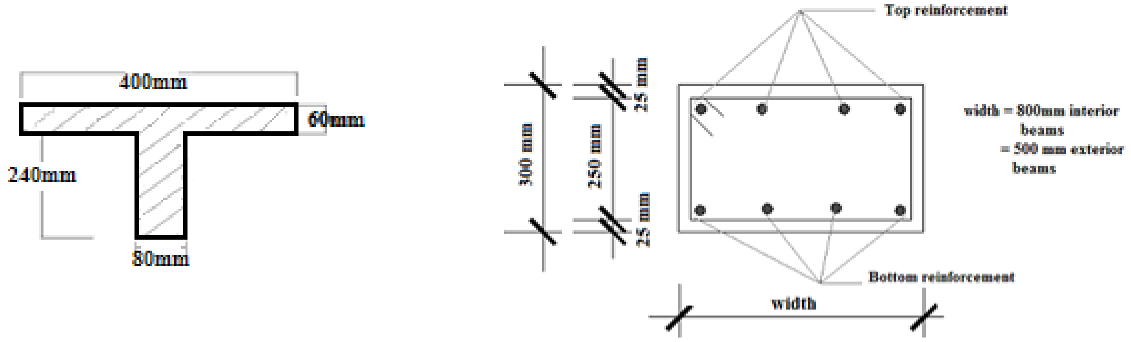
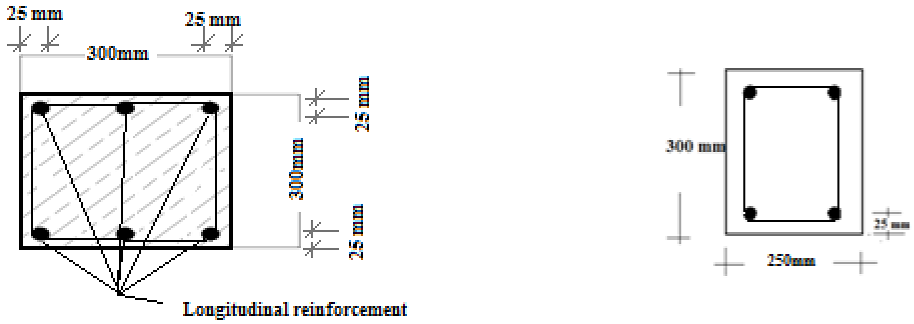


Figure 4. 3 Typical floor plan of case study building B



a) Rib b) Beam



c) Column d) Top tie beam

Figure 4. 4 Members' cross sectional dimensions of case study building B

Table 4. 3 Reinforcement details of columns for case study building B

Column	Floor	# and size of bars	Column	Floor	# and size of bars
C1	First	10ø16	C3	First	10ø16
	Second	10ø16		Second	10ø16
	Third	8ø16		Third	10ø16
	Fourth	6ø14		Fourth	8ø16
	Fifth	6ø14		Fifth	8ø16
C2	First	10ø16			
	Second	8ø16			
	Third	8ø16			
	Fourth	6ø14			
	Fifth	6ø14			

Table 4. 4 Reinforcement details of beams for case study building B

Typical floor beams		# and size of bars	Typical floor beams		# and size of bars
FB1 Axis Y01-Y04	Bottom	4 ø14	FB7 Axis X02-X02'	Bottom	5ø14
	Top	4 ø14		Top	5ø16
	Transverse	ø8 cc 200		Transverse	2ø8 cc 200
FB2 Axis Y04'-Y07	Bottom	5 ø14	FB8 Axis X05-X02	Bottom	5ø14
	Top	5ø14		Top	5ø14
	Transverse	ø8 cc 200		Transverse	2ø8 cc 200
FB3 Axis Y04-Y05	Bottom	5 ø14	FB8 Axis X02-X01	Bottom	5ø14
	Top	5ø14		Top	6ø16
	Transverse	ø8 cc 200		Transverse	2ø8 cc 200
FB4 Axis X05-X02	Bottom	5 ø14	FB9 Axis X04-X03	Bottom	5ø14
	Top	6ø16		Top	5ø14
	Transverse	ø8 cc 200		Transverse	2ø8 cc 200
FB4 Axis X02-X01	Bottom	5 ø16	FB9 Axis X03-X02	Bottom	5ø14
	Top	5ø16		Top	6ø20
	Transverse	ø8 cc 200		Transverse	2ø8 cc 180
FB5 Axis X05-X02	Bottom	5 ø16	FB9 Axis X02-X01	Bottom	7ø16
	Top	6ø16		Top	6ø20
	Transverse	2ø8 cc 180		Transverse	2ø8 cc 180
FB5 Axis X02-X01	Bottom	5 ø20	FB10 Axis X04-X03	Bottom	5ø14
	Top	6ø20		Top	5ø14
	Transverse	2ø8 cc 180		Transverse	2ø8 cc 200
FB6 Axis X05-X03	Bottom	5 ø14	FB10 Axis X03-X02	Bottom	5ø14
	Top	5ø14		Top	5ø16
	Transverse	2ø8 cc 200		Transverse	2ø8 cc 180
FB6 Axis X03-X02	Bottom	6 ø16	FB10 Axis X02-X01	Bottom	6ø16
	Top	6 ø16		Top	5ø16
	Transverse	2ø8 cc 200		Transverse	2ø8 cc 180

4.4 Case study building C

The existing case study building C has a plan dimension of 26.08 m in the longer direction and 15 m in the shorter direction. The height of the building from ground level to the roof is equal to 19 m. Each storey has equal height of 3.8 m. The ribs are spaced at every 625 mm.

A plan view of typical floor of the building showing the beams' as well the columns' distribution is shown in figure 4.3.

The cross sectional dimension of all columns is 400 mm in depth and 250 mm in width. All the beams have cross section of 480 mm in depth and 250 mm in width. The roof or top tie beams are of cross sectional dimensions of 300 mm in depth and 250 mm in width. The clear cover to the longitudinal reinforcement of the columns and the beams is 25 mm. The reinforcing detail for this building is given in Tables 4.5 and 4.6.

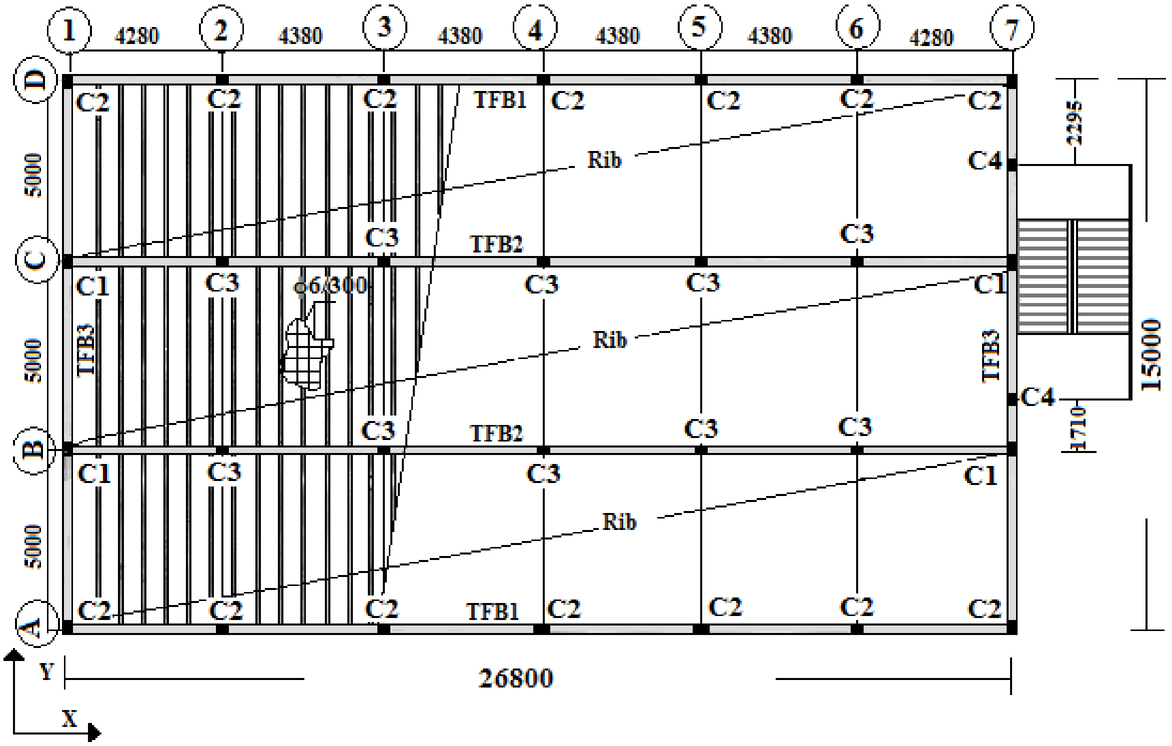
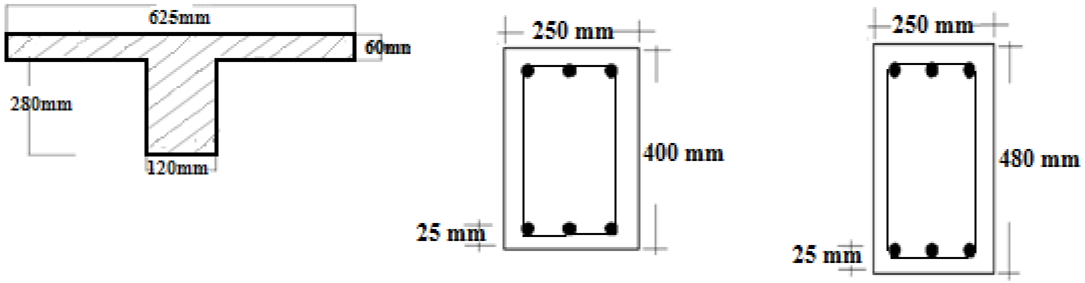
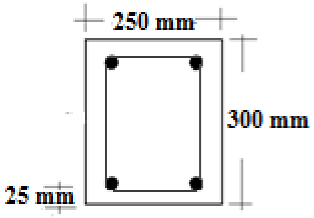


Figure 4. 5 Typical floor plan of case study building C



a) Rib b) Column c) Beam



d) Top tie beam

Figure 4. 6 Members' cross sectional dimensions of case study building C

Table 4. 5 Reinforcement details of beams for case study building C

Typical floor beams		# and size of bars
TFB1 Axis 1-7	Bottom	3 ø14
	Top	3ø16
	Transverse	ø8 cc 200
TFB2 Axis 1-7	Bottom	4 ø16
	Top	3ø20
	Transverse	ø8 cc 200
TFB3 A-B-C-D Axis 1	Bottom	4 ø16
	Top	4ø20
	Transverse	ø8 cc 200
TFB3 A-B-C-D Axis 1	Bottom	3 ø16
	Top	4ø20
	Transverse	ø8 cc 200

Table 4. 6 Reinforcement details of columns for case study building C

Column	Floor	# and size of bars	Column	Floor	# and size of bars
C1	First	8ø20	C3	First	10ø20
	Second	8ø20		Second	10ø20
	Third	8ø16		Third	8ø20
	Fourth	8ø14		Fourth	8ø14
	Fifth	6ø14		Fifth	8ø14
C2	First	10ø20	C4	First	6ø16
	Second	8ø20		Second	6ø16
	Third	8ø20		Third	6ø16
	Fourth	8ø16		Fourth	6ø16
	Fifth	8ø14		Fifth	6ø16

Chapter 5: Analysis and Discussion of Results

5.1 General

The following sections of this chapter discuss observed evaluation of results from diaphragm flexibility check and seismic performance evaluation. The analyses results after provision of retrofitting measures will also be presented and discussed.

5.2 Result of diaphragm flexibility check

The loads are distributed according to tributary areas for the analysis done to do diaphragm flexibility check.

The results are presented for each building models using the three tables given sequentially. Table 5.1 a presents results for case study building A, Table 5.2 present for case study building B, and Table 5.3 presents for case study building C.

The in-plane diaphragm deformation is measured by subtracting the deflection of interior column from that of exterior one, usually at the top floor level below the roof. The average inter-story drift is taken for the story level just below the floor level under consideration [11].

i. Case study building A

Table 5. 1 Diaphragm flexibility for case study building A

Floor	Δdiaph (cm)	Drift (cm)	Δdiaph/Drift	FEMA 273 Classification
2nd	9.004	8.48	1.06	Stiff
1st	5.216	7.67	0.68	Stiff

ii. Case study building B

Table 5. 2 Diaphragm flexibility for case study building B

Floor	Δdiaph	Drift	Δ diaph/Drift	FEMA 273 Classification
4th	10.23	14.16	0.72	Stiff
3rd	8.6	24.3	0.35	Rigid
2nd	5.5	23.9	0.23	Rigid
1st	1.19	19.29	0.06	Rigid

iii. Case study building C

Table 5. 3 Diaphragm flexibility for case study building C

Floor	Δdiaph (cm)	Drift (cm)	Δdiaph/Drift	FEMA 273 classification
4th	27.39	5.48	5	Flexible
3rd	21.25	7.94	2.68	Flexible
2nd	12.45	8.39	1.48	Stiff
1st	3.656	4.31	0.85	Stiff

Even though further analyses is required, the results of the analyses executed show that, unlike two way beam supported reinforced concrete slabs, the floor slabs of the three case study buildings do not exhibit sufficient stiffness to act as a rigid diaphragm. Therefore, the assumption of rigid body motion of the diaphragm does not hold in the longitudinal direction of the ribs. This means during performance evaluation of these buildings, the in-plane deformation of the diaphragms should be included.

5.3 Performance evaluation

ETABS provide several outputs from the static non linear analysis which include plot of pushover curve, demand versus capacity curves from which the performance point is obtained, the deflected shape of the building frame showing the hinge state at each step of pushover analysis, force and moment plots, and any other results as it has been discussed in section 2.2.3.

In this thesis, the capacity spectrum versus demand spectrum intersection point or performance point, and the state of plastic nonlinear hinges formed at this performance point will be used to assess the performance of the three case study buildings.

The performance point represents the maximum structural displacement expected for the demand earthquake ground motion. In other way, the magnitude of the performance point signifies the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion.

The colors of the plastic nonlinear hinges represent the state of nonlinear plastic hinges at each performance level. Purple colored hinges represent hinges below immediate occupancy level, blue hinges beyond immediate occupancy level but below life safety level, light blue hinges beyond life safety but below collapse prevention level, yellow hinges beyond collapse prevention level, and orange and red colored hinges represent the state of hinges beyond collapse prevention state.

In the following three sections, the results, which are the performance point in terms of spectral acceleration S_a and spectral displacement S_d incorporated with the state of plastic non linear hinges at the performance point for the three case study buildings will be presented. The presentation of the results will be done for X and Y directions of each case study buildings.

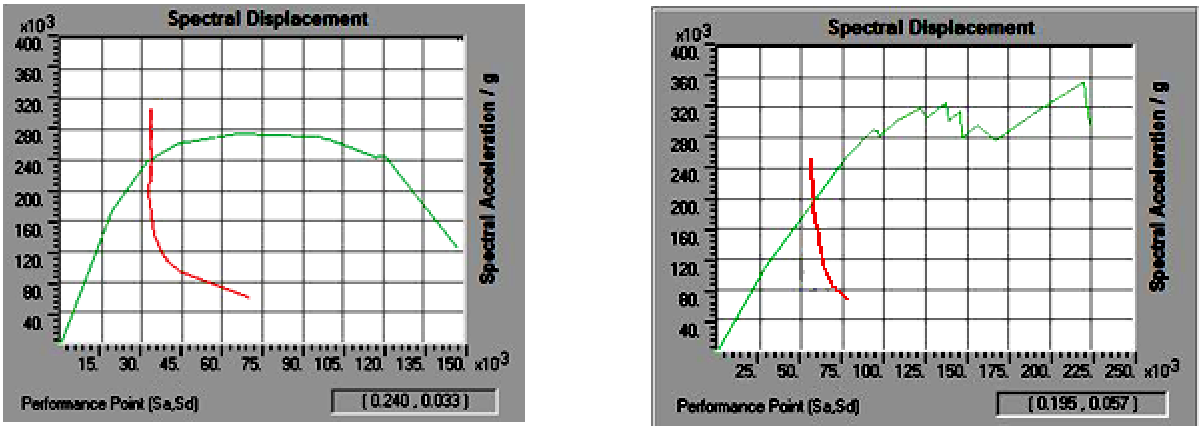
The capacity spectrum is portrayed figure by green color; whereas, the demand spectrum is represented by red color.

5.3.1 Case study building A

This case study building is located in Seismic zones of four. The serviceable earthquake level is taken as Peak Ground Acceleration PGA of 0.1g, and the design earthquake level is taken as PGA of 0.2g (Section 3.3). The resulting capacity versus demand spectra of this case study building, in the two orthogonal directions are shown in Figure 5.1 for serviceable earthquake level, and in Figure 5.2 for design earth quake level.

For the 0.1g PGA, the demand spectrum intersected the capacity spectrum at the performance point of the spectral acceleration, and demand of 0.024 and 0.033 in the longitudinal direction of the ribs, and 0.195 and 0.057 in the transverse direction of the ribs. At these performance points, the resulting state of plastic non linear hinges is shown in Figure 5.2. As it can be observed from the Figures, majority of these hinges in the frames on axes 1 and 3 are beyond the immediate occupancy level. In the transverse direction, the non linear plastic hinge on axes B, C, D, E, F and G near axis 1 are beyond life safety level. This implies that the performance objective of this building at this earthquake level is not fulfilled.

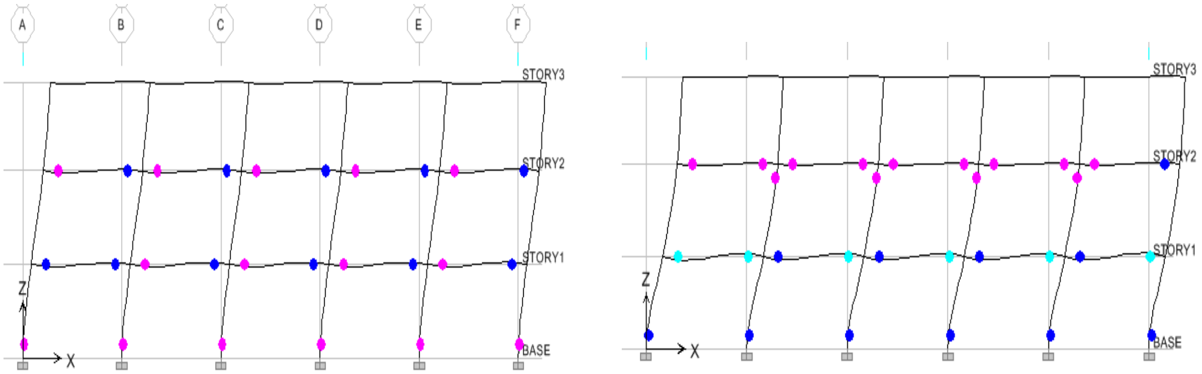
For the 0.20g PGA of earthquake level, the demand versus capacity curve is shown in Figure 5.3. The demand spectrum intersected the capacity spectrum at the performance point of the spectral acceleration, and demand of 0.027 and 0.059 in the longitudinal direction of the ribs, and 0.289 and 0.095 in the transverse direction of the ribs. For this earthquake level, the states of non linear plastic hinges are shown in Figure 5.4. All the nonlinear plastic hinges of beams on axes one and three are beyond life safety level. Besides, all nonlinear plastic hinges of the columns on axis 3, and the exterior columns on axis one of the first story at base level are beyond life safety level; whereas in the transverse direction, the non linear plastic hinge on axes B, C, D, E, F and G near axis one are beyond life safety level, in which the three are beyond collapse prevention level. This implies that at this earthquake level the performance of this case study building is not adequate to insure the well beings of its occupant.



a) X-direction

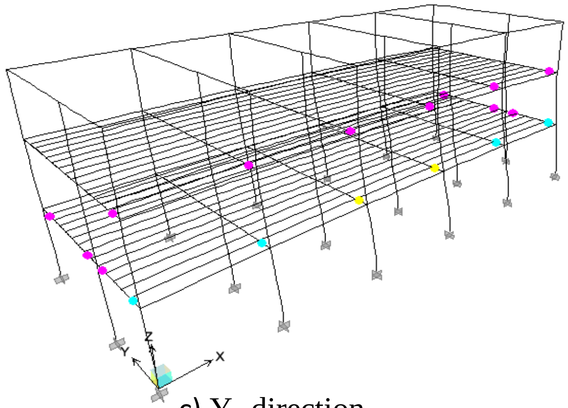
b. Y- direction

Figure 5. 1 Capacity versus demand spectra at PGA of 0.1g



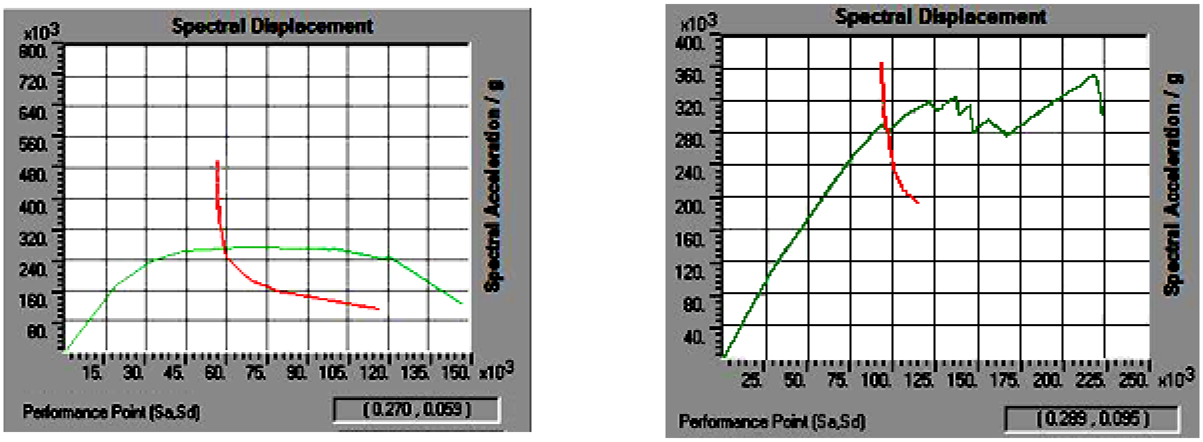
a) Axis 1

b) Axis 3



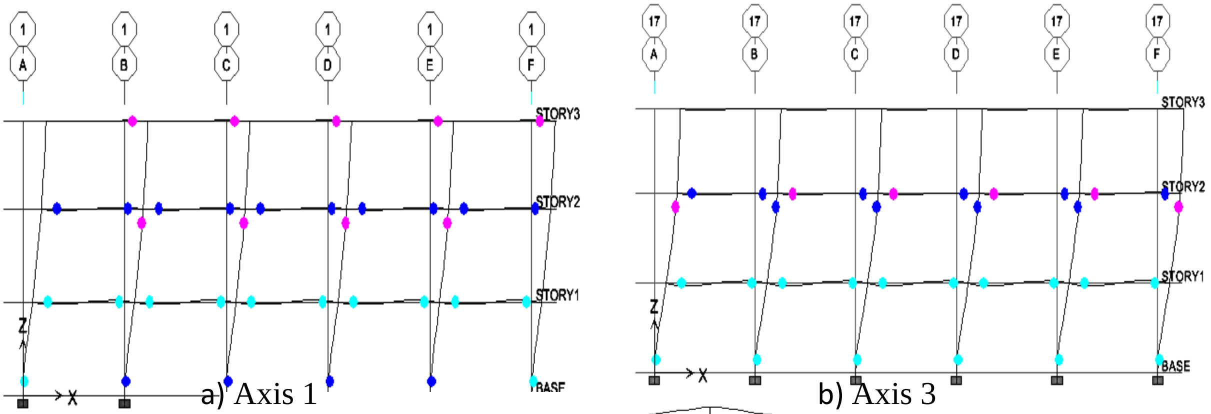
c) Y- direction

Figure 5. 2 State of nonlinear plastic hinges at PGA of 0.1g

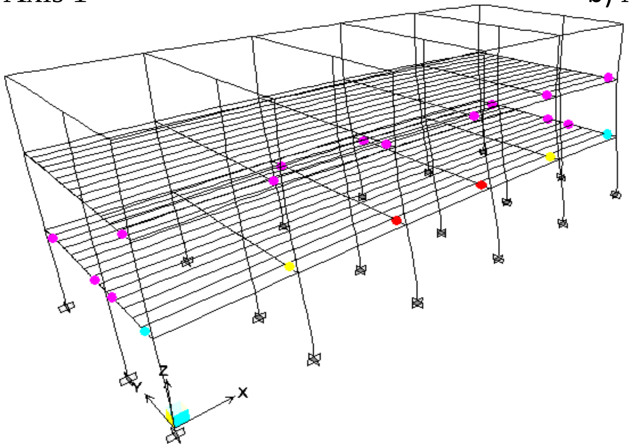


a. X- direction b. Y- direction

Figure 5. 3 Capacity versus demand spectra at PGA of 0.2g



a) Axis 1 b) Axis 3



c) In the Y-direction of the ribs

Figure 5. 4 State of nonlinear plastic hinges at PGA of 0.2g

5.3.2 Case study building B

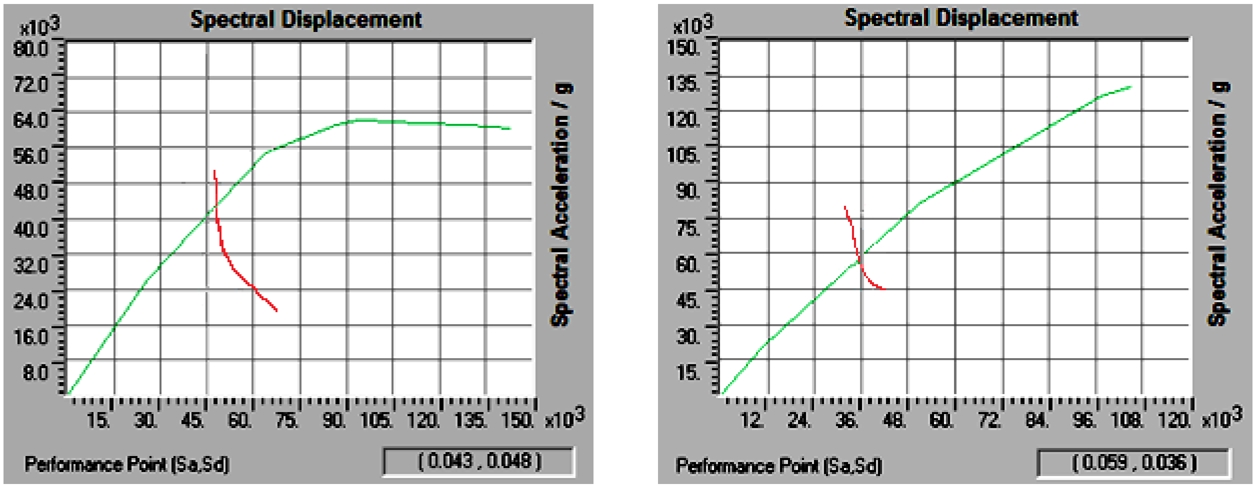
This case study building is located in Seismic zones of four. The serviceable earthquake level is taken as Peak Ground Acceleration PGA of 0.05g, and the design earthquake level is taken as PGA of 0.1g (Section 3.3). The resulting capacity versus demand of this case study building, in the two orthogonal- directions is shown in Figure 5.5 for PGA of 0.05g, and in Figure 5.7 for PGA of 0.1g.

For the 0.05g PGA, the demand spectrum intersected the capacity spectrum at the performance point of the spectral acceleration, and demand of 0.043 and 0.048 respectively in the longitudinal direction of the ribs, and 0.059 and 0.039 in the transverse direction of the ribs. At these performance points, the resulting state of plastic non linear hinges is shown in Figure 5.6 in the longitudinal direction, and in Figure 5.7 in the transverse direction of the ribs.

As it can be seen from the Figure 5.6 a, c and d, state of nonlinear plastic hinges in beams of first and second floor levels beyond immediate occupancy level are inspected. In the transverse direction, the majority of non linear plastic hinges of the beams in axes Y01 to Y07 of first and second story are beyond immediate occupancy level. Some of the hinges are even, beyond life safety level (Figure 5.7). This indicates that this case study building do not meet its performance objective at this earth quake level in both orthogonal directions.

For the 0.10g PGA of earthquake level, the demand versus capacity curve is shown in Figure 5.8. The demand spectrum intersected the capacity spectrum at the performance point in terms of the spectral acceleration, and demand, 0.062 and 0.099 in the longitudinal direction of the ribs, and 0.11 and 0.081 in the transverse direction of the ribs.

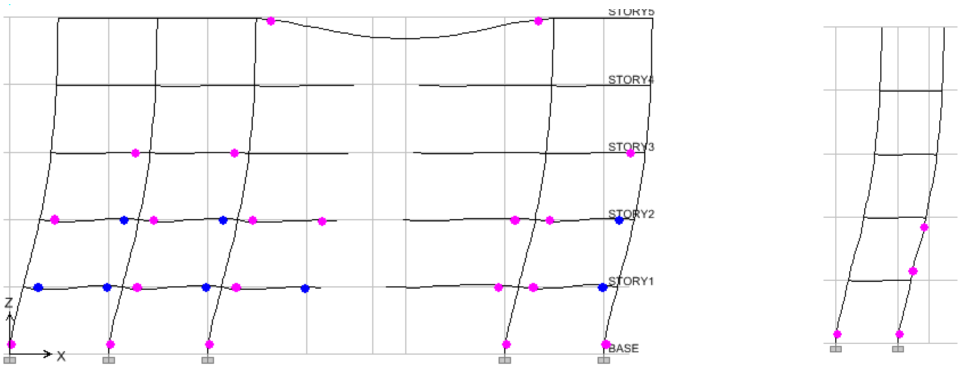
For this earthquake level, the corresponding states of non linear plastic hinges in the direction of the ribs are shown in Figure 5.9 and in the transverse direction in Figure 5.10. Majority of the nonlinear plastic hinges on the columns of axes X01 and X02' in the first floor at the base level are beyond the life safety level. In the direction transverse to the ribs, greater parts of the nonlinear plastic hinges in the beams of the first floor level are beyond life safety level. In the second floor level, the greater part of the non linear plastic hinges on axes Y02 to Y03 are also beyond life safety level. Therefore, for this earthquake level the performance of this case study building is not sufficient to insure the life safety of its occupants.



a. X- direction

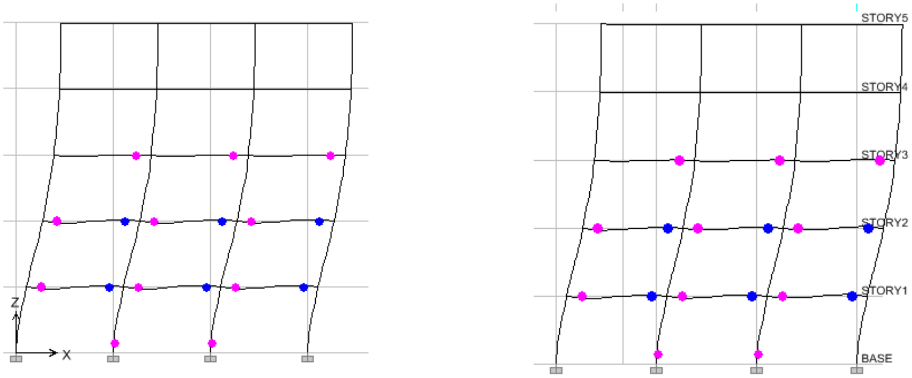
b. Y- direction

Figure 5. 5 Capacity versus demand spectra at PGA of 0.05g



a) Axis X-01

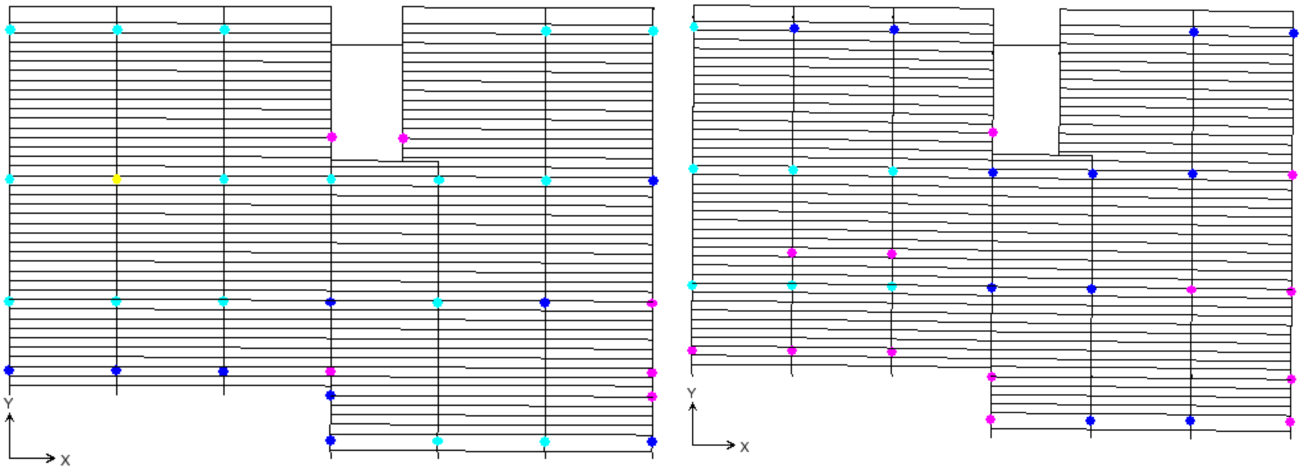
b) X-02’



c) Axis X-04

d) Axis X-05

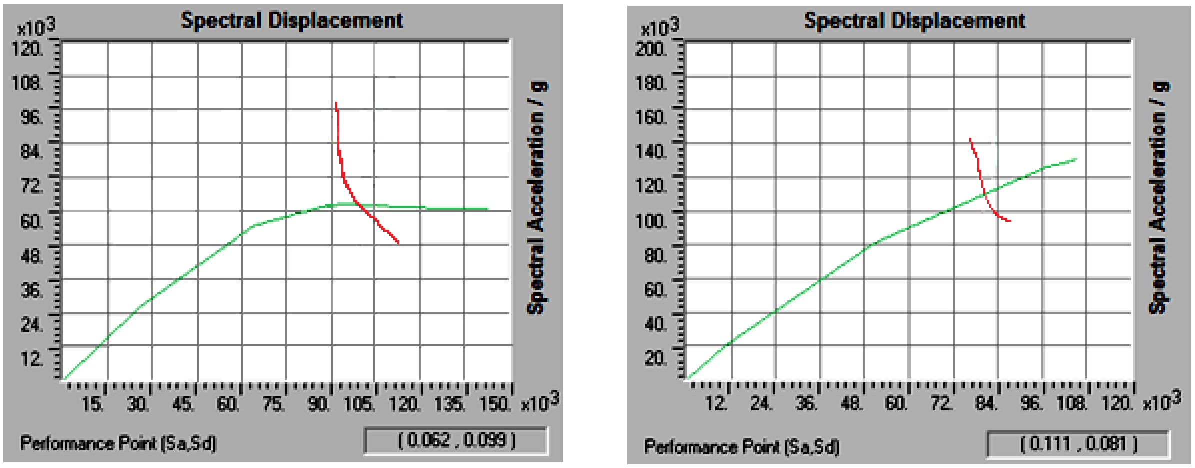
Figure 5. 6 State of non linear plastic hinges in the X-direction at PGA of 0.05g 1



a) First floor b) Second

floor

Figure 5. 7 State of nonlinear plastic hinges in the Y-direction at PGA of 0.05g

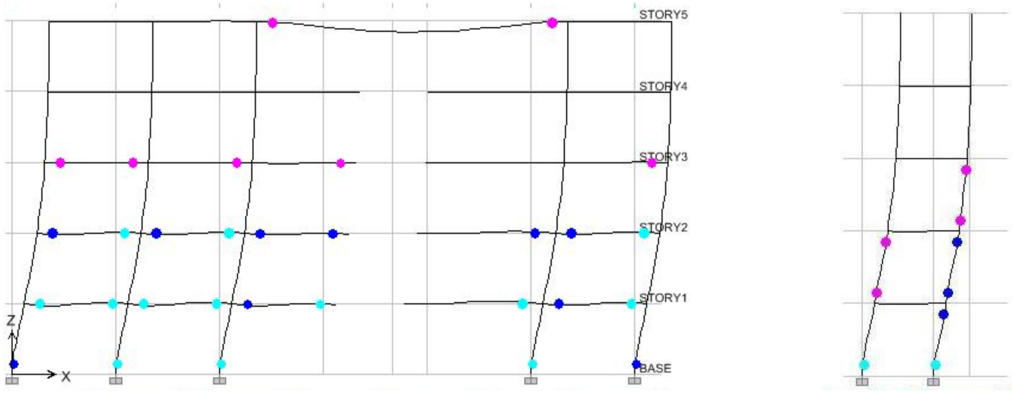


Performance Point (Sa,Sd) (0.062 , 0.099)

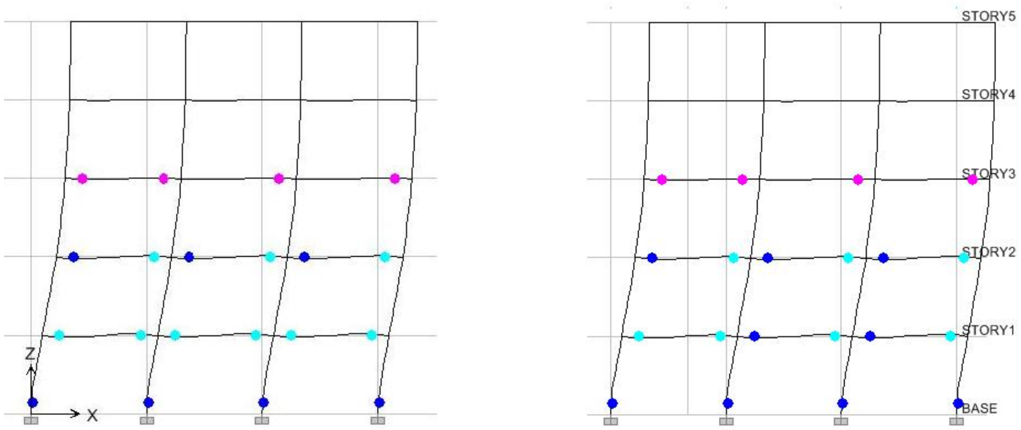
Performance Point (Sa,Sd) (0.111 , 0.081)

a) X- direction b) Y- direction

Figure 5. 8 Capacity versus demand spectra at PGA of 0.1g

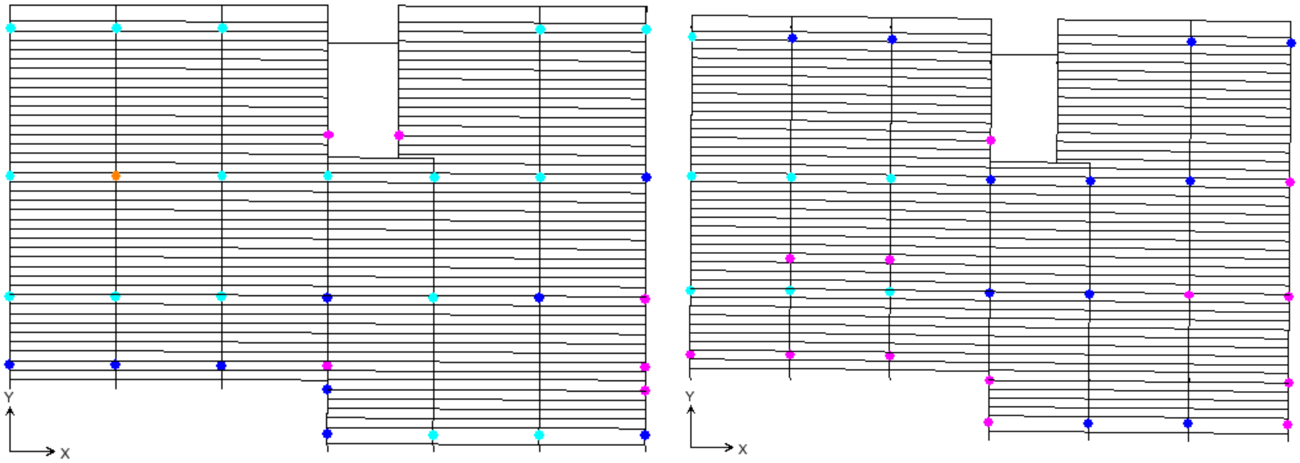


a) Axis X-01 b) Axis X-02’



c) Axes X-04 d) Axis X-05

Figure 5. 9 State of nonlinear hinges in the X-direction at PGA of 0.1g



a) First floor b) Second
floor

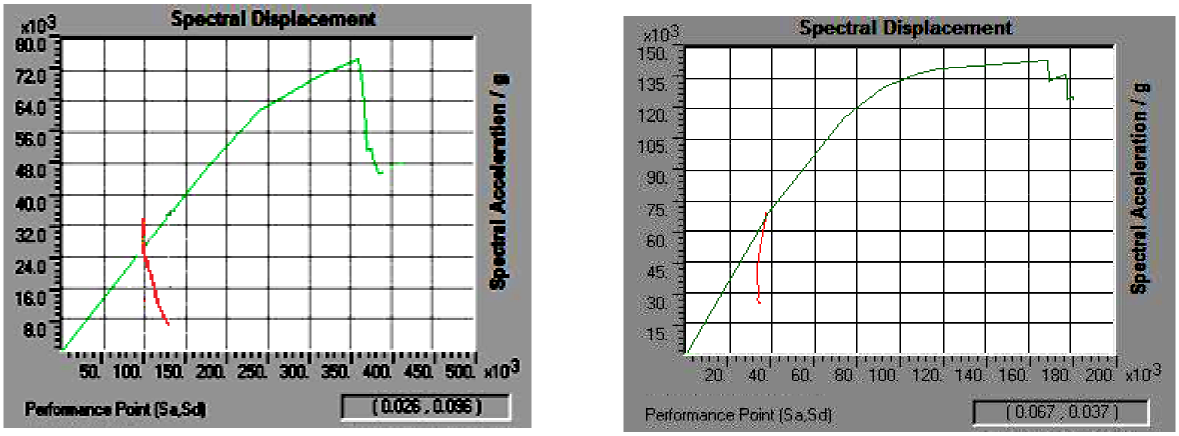
Figure 5. 10 State of nonlinear hinges in the Y-direction at PGA of 0.1g

5.3.3 Case study building C

This case study building is located in Seismic zones of four. The serviceable earthquake level is taken as Peak Ground Acceleration PGA of 0.05g, and the design earthquake level is taken as PGA of 0.1g (Section 3.3). For this case study building, for the 0.05g PGA of earthquake level, the demand versus capacity curve is shown in Figure 5.11.

The demand spectrum intersected the capacity spectrum at the performance point of the spectral acceleration, and demand of 0.026 and 0.096 in the longitudinal direction of the ribs, and 0.067 and 0.037 in the transverse direction of the ribs. For this earthquake level, the states of nonlinear plastic hinges are shown in Figure 5.12. Majority of nonlinear plastic hinges are beyond immediate occupancy level in the first and second floors of two orthogonal directions. Similar to the previous case study buildings, this case study building does not meet its performance objective at this earthquake level.

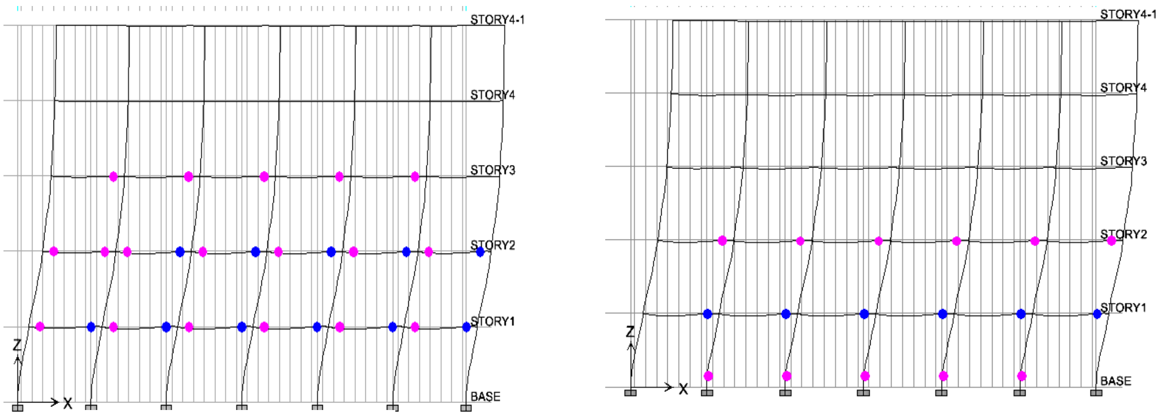
For the 0.1g PGA, the demand spectrum intersected the capacity spectrum at the performance point of the spectral acceleration, and demand of 0.056 and 0.216 in the longitudinal direction of the ribs and 0.12 and 0.079 in the transverse direction of the ribs as shown in Figure 5.13. At these performance points, the resulting state of plastic non linear hinges is shown in Figure 5.14. As it can be observed from the figure, most of the nonlinear plastic hinges assigned in first and second stories on frame axis 7 are beyond the life safety level. In the transverse direction, most of the non linear plastic hinges in all the beams of first story are beyond life safety level. In the second story level, significant number of the non linear plastic hinges on frames of axes A and D are beyond life safety level. This implies that, the majority of the beams in the direction of the ribs are failed to insure the life safety of the occupants.



a) Y- direction

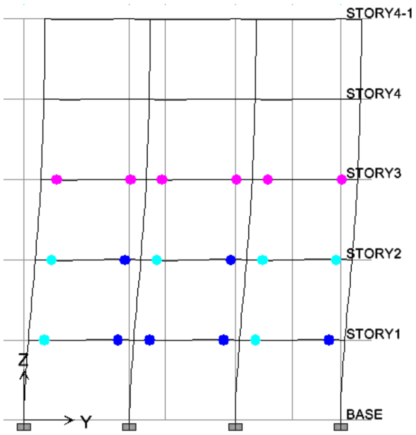
b) X-direction

Figure 5. 11 Capacity versus demand Spectra at PGA of 0.05g



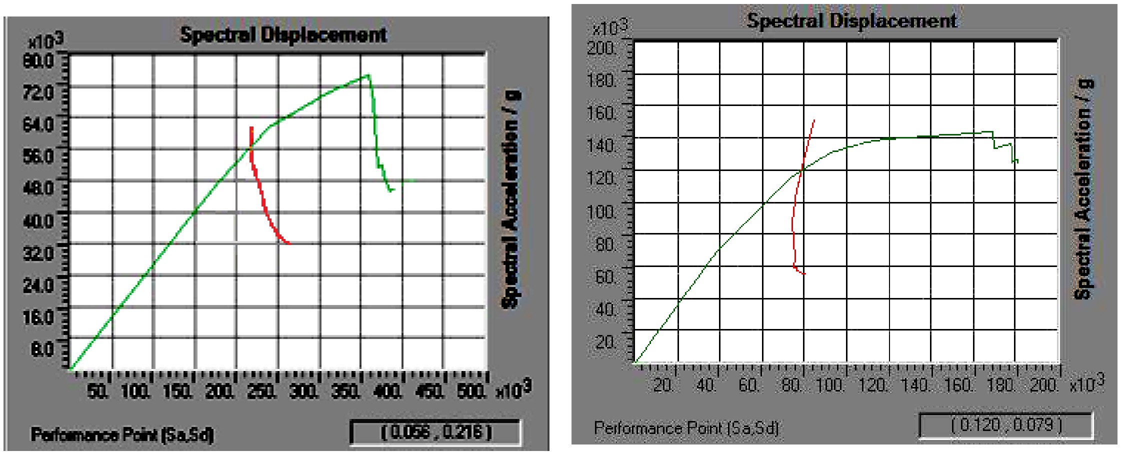
a. Axes A and D

b. Axes B and C



c. Axis 1

Figure 5. 12 Formation of hinges in the X- and Y direction



a) Y-direction

b) X- direction

Figure 5. 13 Capacity versus demand spectra at PGA of 0.05g

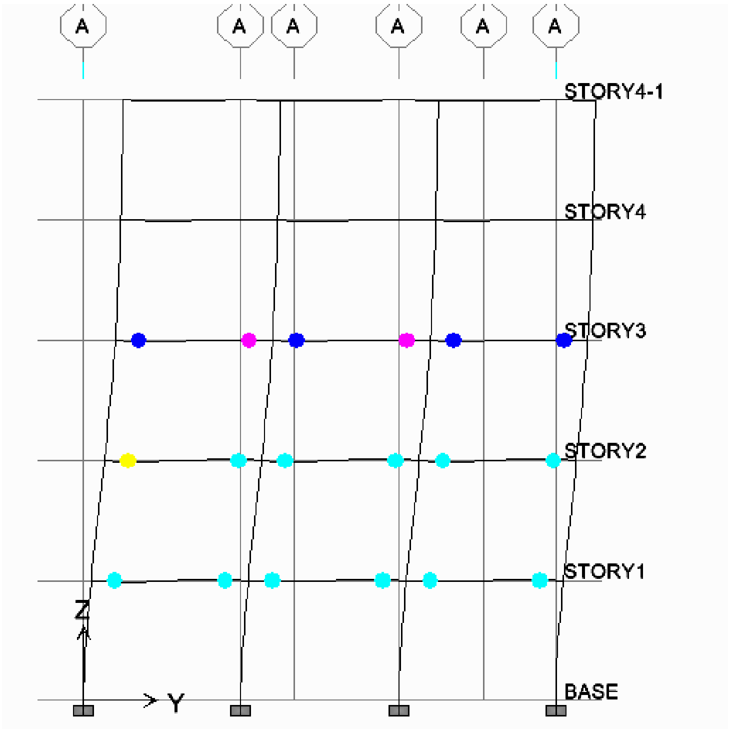


Figure 5. 14 State of nonlinear plastic hinges on beams of axis one at PGA of 0.1g

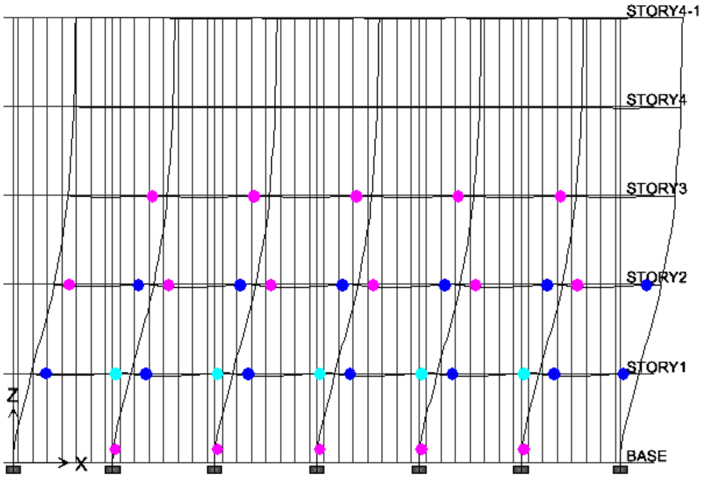


Figure 5. 15 State of nonlinear plastic hinges on beams of axes A and D at PGA of 0.1g

5.4 Retrofitting mechanisms adopted

The major deficiencies of the buildings evaluated in this study are mainly the failure of the beams in the shorter direction (case study buildings A and C). All the hinges beyond life safety level are created in these beams because of the missed beams in this direction. The failure of beams in the transverse direction of the ribs is also seen on case study building B.

In order to improve the seismic resistance of the three case study buildings, the first measure taken is placing of missing beams in all the three case study buildings in order to insure complete transfer of lateral forces to the ground and in order to reduce their flexibility. Pushover analysis has been done after the application of the beams. Depending on the result from this analysis, additional supplemental lateral load resisting systems like shear wall and reinforced concrete jacketing of columns are added, and analysis is performed again.

The added structural systems are made to resist lateral force jointly with that of the structural elements of the already existing structure. Each measure taken for each case study building will be discussed in the next three sections.

5.4.1 Case study building A

For this case study building, an intermediate beam with dimension and detailing similar to FB2L (Table 4.1) is added. This is done to make the stiffness the same with that of the existing beams for lateral load resistance. Then, pushover analysis is performed.

The application of the beams improved the performance of the beams and columns on axis 1 (Figure 4.1). But, this has brought additional demand on the beams on axis 3 and on columns on axis 1 and 2 of first floor. The effect of concrete jacketing of the columns on axis two and three of the first floor was also investigated but this has not improved the performance of the building. Therefore, shear walls are added to the existing building as shown on Figure 5.16. The walls are distributed according to the guidelines in Section 2.3.3 in addition to the architectural and functional floor plan layout of the building considerations. The dimensions of the shear walls are given on Table 5.4.

Table 5. 4 Dimension of shear walls for case study building A

Shear wall	Thickness(m)	Length(m)
SW1	0.250	1.20
SW2	0.250	1.20
SW3	0.250	0.60

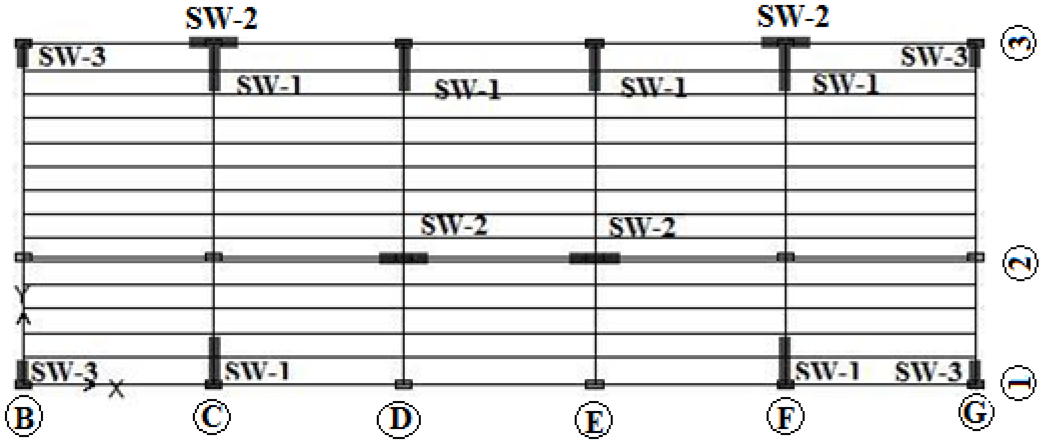
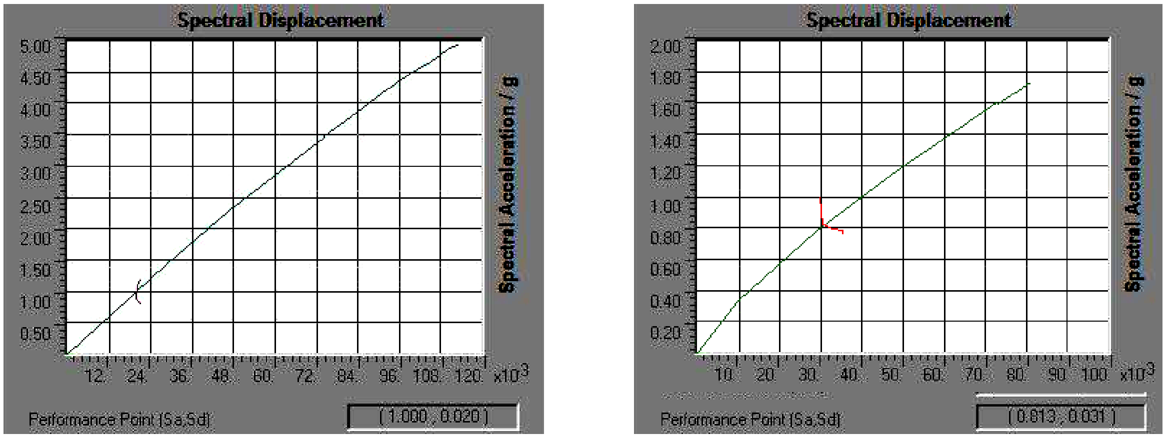


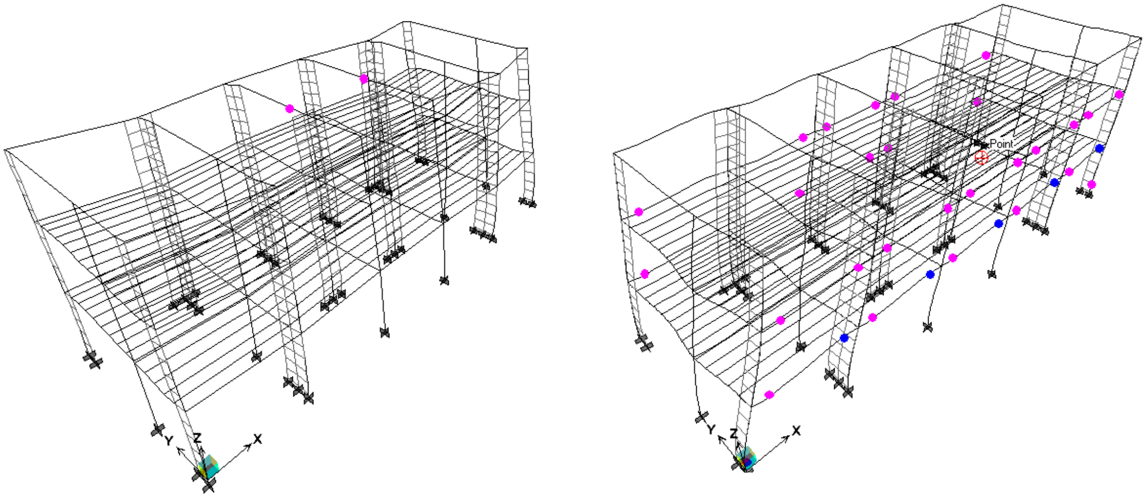
Figure 5. 16 Plan distribution if shear walls added in case study building A

The resulting pushover versus demand spectra showing the performance points for PGA of 0.2g are given on Figure 5.17. The corresponding state of nonlinear plastic hinges for the performance points obtained is shown in Figure 5.18. No nonlinear plastic hinge is observed at the performance points obtained.



a) X-direction b) Y-direction

Figure 5. 17 Capacity versus demand spectra at PGA of 0.2g 2



a) Y- direction b) In the

X-direction

Figure 5. 18 State of nonlinear plastic hinges at PGA of 0.2g

5.4.2 Case study building B

To improve the seismic performance of this case study building, shear walls are added to the existing building according to Figure 5.19. The walls are located and their plan dimensions are fixed considering the functional and architectural layout of the building, which is shown on the floor plan of the building. In addition, they are located to reduce the plan stiffness irregularity effects.

Table 5. 5 Dimension of shear walls for case study building B

Shear wall	Thickness (m)	Length (m)
SW1	0.25	1.2
SW2	0.25	1.5
SW3	0.25	0.7

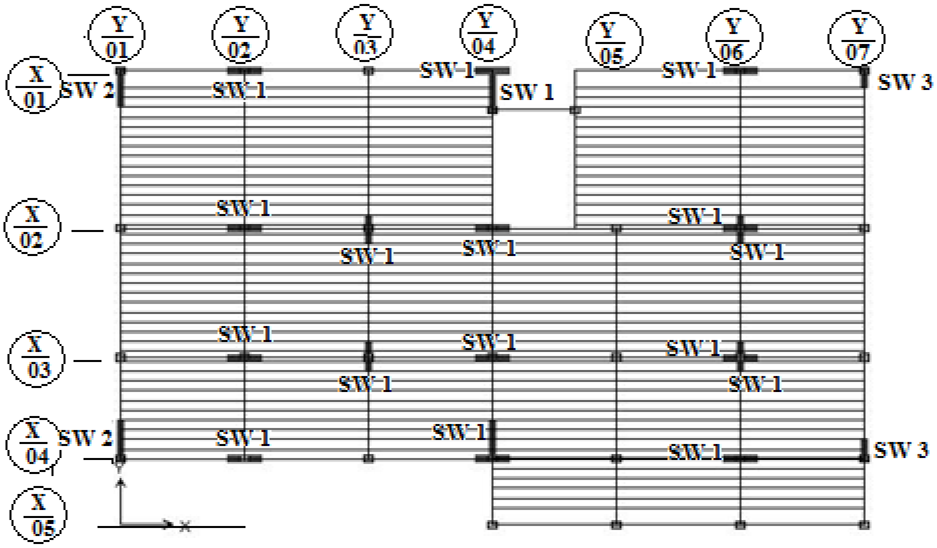
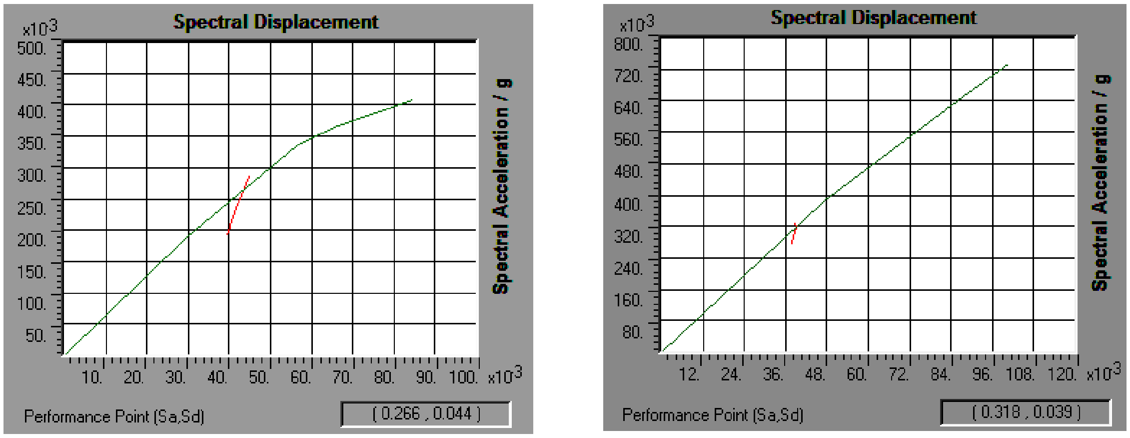


Figure 5. 19 Plan distribution if shear walls added in case study building B

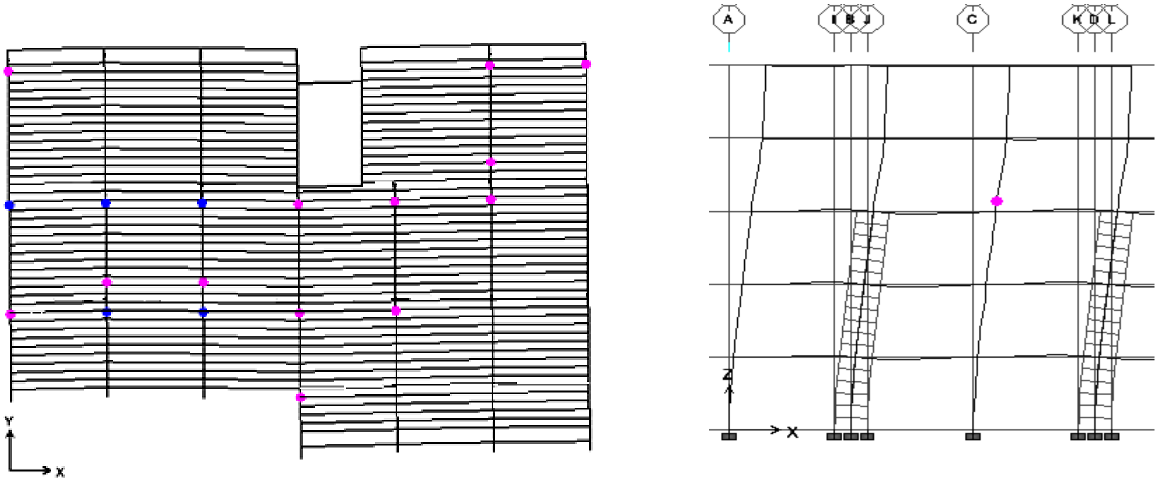
The resulting pushover versus demand spectra, which show the performance points for PGA of 0.1g are given on Figure 5.20. The corresponding state of nonlinear plastic hinges for these performance points obtained is shown in Figure 5.21. There is no any nonlinear plastic hinge observed for these performance points obtained in the floors where the shear walls are available. For the transverse direction of the ribs in the fourth floor, some non linear plastic in the beams with state below life safety level are inspected. Whereas in the longitudinal direction of the ribs, the nonlinear plastic hinges in the columns in between the third and fourth floors with the state below life safety level are seen.



a) X- direction

b) Y- direction

Figure 5. 20 Capacity versus demand spectra at PGA of 0.1g



a) Forth floor

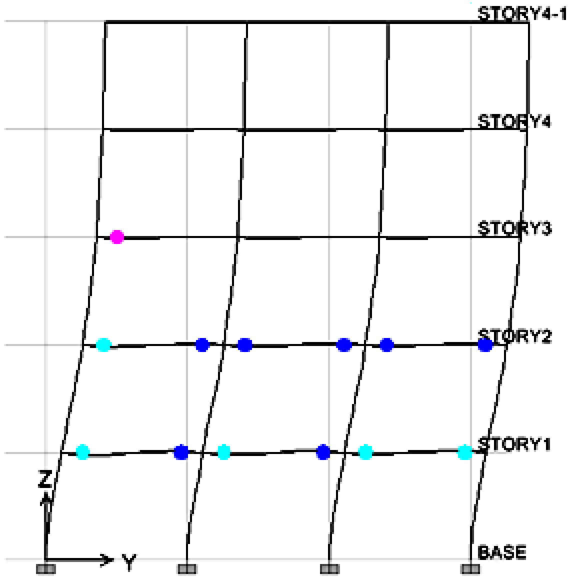
b) Frame on axis X-04

Figure 5. 21 State of nonlinear plastic hinges at PGA of 0.1g

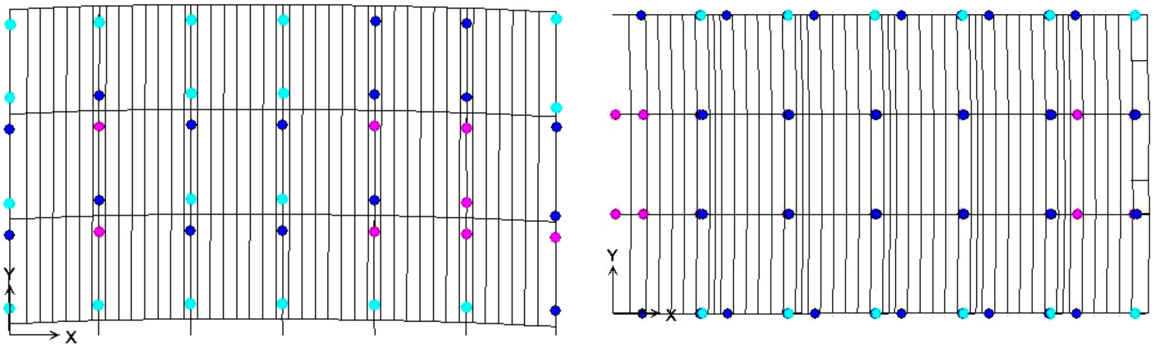
5.4.3 Case study building C

For this case study building, after the addition of intermediate beams with dimensions and detailing similar to TFB3 (Table 4.5), additional pushover analysis is performed. The application of these beams improved the performance of the beams and columns on axis 1

(Figure 4.1). But, some nonlinear plastic hinges in these axes are still beyond life safety level as shown in Figure 5.22 below.



a) Frame on axis 1 X-direction



b) First floor in Y-
direction c) First floor in X- direction

Figure 5. 22 State of nonlinear plastic hinges after addition of beams at PGA of 0.1g

In order to improve the performance of this building, the member-level strengthening approach, like concrete jacketing of columns in the first floors, cannot provide a reliable result on this building. Because, when increasing the strength of some structures, and performing pushover analysis, the capacity of the strengthened members will be improved but some additional members will require local strengthening. The same is true for the above two case study buildings. Therefore, structure-level retrofit is necessary. To enhance the seismic performance of this case study building, shear walls are added in the first and second storey levels according to Figure 5.23. All the shear walls have thickness of 0.25m and length of 3.13m.

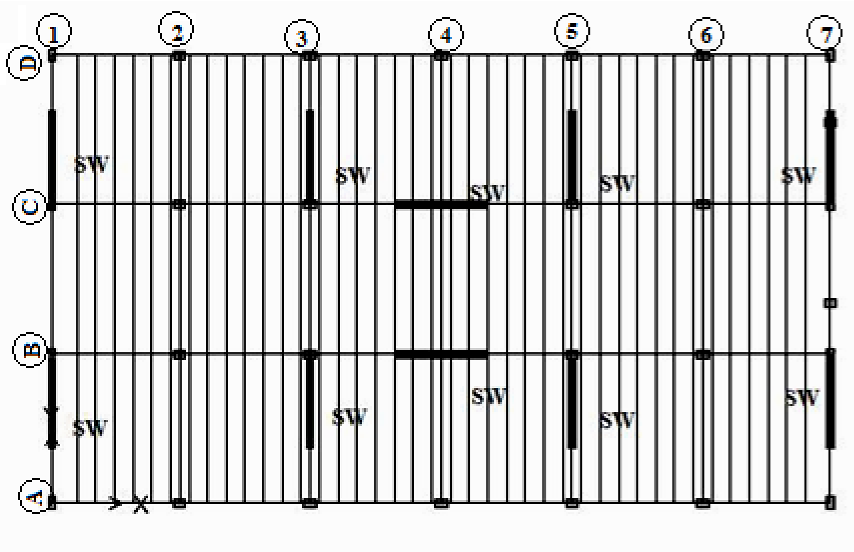
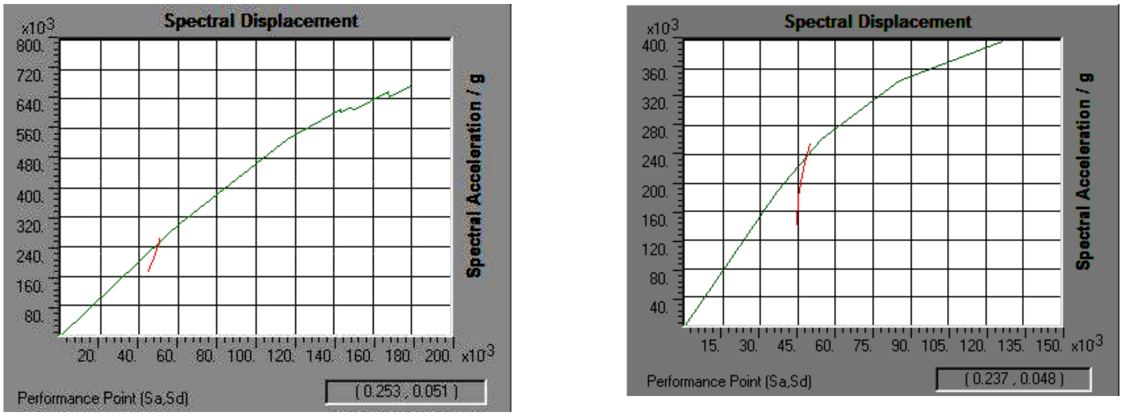


Figure 5. 23 Plan distribution of shear walls added for case study building C

The resulting pushover versus demand spectra showing the performance points for PGA of 0.1g are given on Figure 5.24.



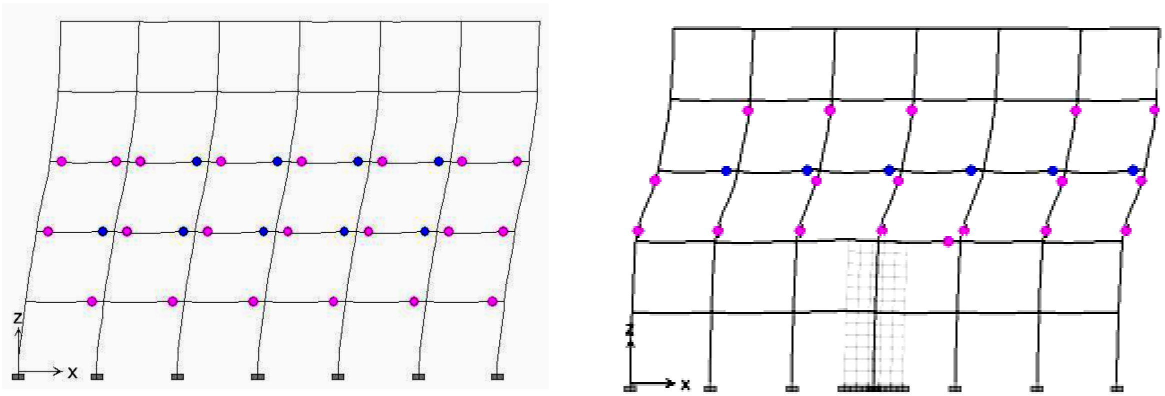
Performance Point (Sa,Sd) (0.253 , 0.051)

Performance Point (Sa,Sd) (0.237 , 0.048)

a) X-direction of ribs b) Y- direction

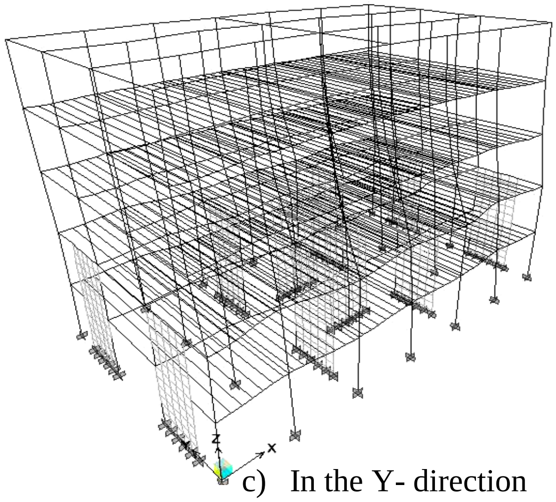
Figure 5. 24 Capacity versus demand spectra at PGA of 0.05g

The state of nonlinear plastic hinges for the performance points obtained at 0.15g of PGA is shown in Figure 5.18. No non linear plastic hinge beyond life safety level is observed at the performance points obtained for this PGA.



a) Axes A and D

b) Axes B and C



c) In the Y- direction

Figure 5. 25 State of nonlinear plastic hinges after addition of shear walls at PGA of 0.1g

Chapter 6: Conclusions and recommendations

6.1 Conclusions

Diaphragm flexibility check of the three case study buildings is carried out using static linear analysis. The analysis results show that, slabs of buildings constructed using ribbed slabs with hollow blocks framing only in one direction do not exhibit sufficient stiffness to act as rigid diaphragm. Therefore, during seismic performance evaluation of these buildings, the in plane deflection of these floor slabs is considered.

Seismic performance evaluation of the three case study buildings is also performed. This is done by capacity spectrum approach of static non linear (pushover) analysis using ETABS V 9.2.0. The ADRS format of pushover (capacity) versus demand curves in the two orthogonal directions for each case study building is determined. And, the intersection of the two spectra, or the performance points are determined for each case study buildings. The state of nonlinear plastic hinges of the frame elements at the performance point of the buildings is used to assess the performance of the buildings.

The performance evaluation results of these buildings demonstrate that these buildings do not fulfill the performance objectives both at the serviceable and design earthquake levels. For the design earthquake level, which is twice to that of their serviceable earthquake level, the performance of all the three case study buildings is not to be relied on to insure the life safety of their occupants. Therefore, all the three case study buildings need retrofitting through adding of supplementary lateral resisting system to perform well in future earthquakes.

For all the three case study buildings, addition of shear walls in the given cross sectional dimensions and plan layout and concrete jacketing of columns, in addition to the insertion of the missed beams, have been tried. But, shear walls have improved the performance of the structure in the required manner in addition to the introduction of the missed beams. Therefore, they are recommended to be used in seismic retrofitting of the three case study buildings.

6.2 Recommendations

Making this thesis as a basis, the following points are recommended.

i. The behavior of every building structure is specific, so that detailed seismic performance evaluation of every building constructed using ribbed floor slabs framing only in one direction should be performed for the future. The evaluation should include site investigation, exact soil data, exact seismic property of the area and so on. By doing so, better performance evaluation of each buildings will be done in a better way. This can help to obtain enhanced mechanism of retrofitting or strengthening for each case study buildings.

ii. Further analyses and studies can be done for determining optimum retrofitting methods and to determine the diaphragm flexibility of such strructures. Detailing of the retrofitting structures proposed in this thesis can also be done.

iii. Because such building structures do not perform well against lateral load excitations, designers, contractors and government officials involving in giving permission for construction should be aware such structures

iv. The Ethiopian building code standards should include guidelines for the seismic evaluation and rehabilitation of buildings.

v. Designers should check the rigidity of floor diaphragms of RC framed buildings which are susceptible for in-plane deformation before performing analysis and design.

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Appendix A: ATC 40 guidelines for developing analytical models

This section presents ATC 40 rules/ guidelines for developing analytical models for existing framed concrete buildings for static nonlinear analysis. The sections address loads, global building modeling, material models, element models and component models, considering stiffness, strength, and deformability. For other types of RC existing buildings, the reader may refer to ATC 40.

i. Loads

Gravity Loads*:* The nonlinear analysis of a structure should include the simultaneous effects of gravity and lateral loads. Gravity loads should include dead loads and likely live loads.

Lateral loads: Lateral loads should be applied in predetermined patterns that represent predominant distributions of lateral inertial loads during critical earthquake response.

Gravity loads should be in place during lateral loading. The effect of gravity loads acting through lateral displacements, the so-called P-Δ effect, should be modeled.

ii. Global building considerations

Analytical models for evaluation or retrofitting must represent complete three-dimensional characteristics of building behavior, including mass distribution, strength, stiffness, and deformability, through a full range of global and local displacements. Two-dimensional models may be used if they adequately represent overall lateral response.

Building Model: The analytical model of the building should represent all new and existing components that influence the mass, strength, stiffness, and deformability of the structure at or near the expected performance point. Elements and components shown that do not significantly influence the building assessment need not be modeled.

Soil-structure Interaction: Effects of soil-structure interaction should be modeled or shown to be insignificant to building assessment.

iii. Element Models

An element is defined as either a vertical or a horizontal portion of a building that acts to resist lateral and/or vertical load.

Beam-Column Frames: The analysis model for a beam-column frame element should represent the strength, stiffness, and deformation capacity of beams, columns, beam-column joints, and other components that may be part of the frame. Beam and column components should be modeled considering flexural and shear rigidities, although the latter may be neglected in many cases. Potential failure of anchorages and splices may require modeling of these aspects as well. Rigid beam-column joints may be assumed, except where joint strength may limit capacity development in adjacent components.

iv. Component Models

Component behavior generally will be modeled using nonlinear load-deformation relations defined by a series of straight-line segments. Figure A.1 illustrates a typical representation. In this figure, *Qc* refers to- the strength of the component and *Q* refers to the demand imposed by the earthquake.

As shown in Figure A.1, the response is linear to an effective yield point, B, followed by yielding (possibly with strain hardening) to point C, followed by strength degradation to point D, followed by final collapse and loss of gravity load capacity at point E. It is acceptable to use a simple bilinear model that includes only line segments A-B-C if the analysis ensures that response does not extend beyond point C for any of the components.

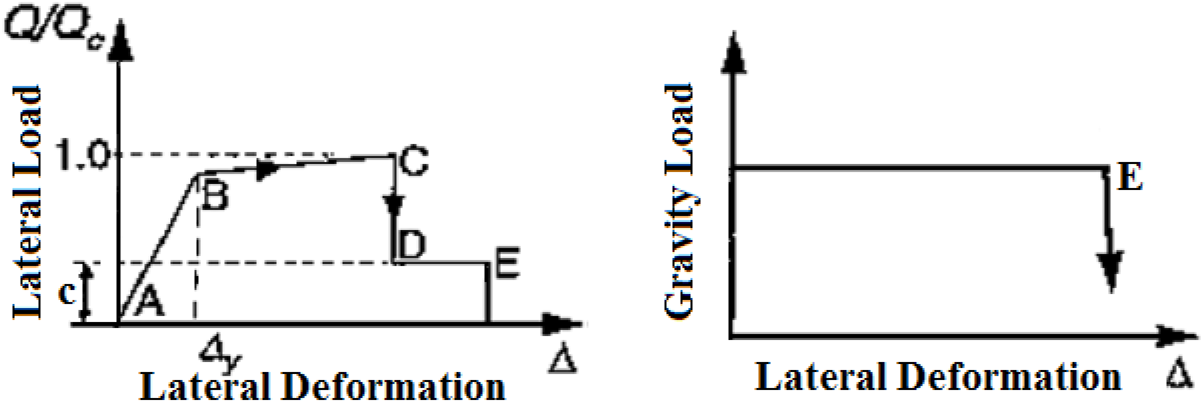


Figure A.1 Generalized Load-Deformation Relations for non degraded components [2]

v. Material Models

The material models should consider all available information, including building plans, original calculations and design criteria, site observations, testing, and records of typical materials and construction practices prevalent at the time of construction. Default assumptions may be required in certain cases where information is unavailable.

vi. Component Initial stiffness

Reinforced concrete component initial stiffness may-be represented by a secant value defined by the effective yield point of the component, as shown by the initial slope in Figure 9-6. For flexure-dominated components, this stiffness corresponds approximately to the fully-cracked stiffness. The stiffness value may be determined as a function of material properties (considering current condition), component dimensions, reinforcement quantities, boundary conditions, and stress and deformation levels. In many cases ,it will be impractical to calculate effective stiffnesses directly from basic mechanics principles. Instead, the effective initial stiffness may be based on the approximate values of Table 9-3 of ATC 40. For non pre-stressed beams and columns, these approximate values are given in Table 3.1 of this thesis.

vii. Component strength

Actions (forces and associated deformations) in a structure are classified as either deformation controlled or force-controlled. Deformation-controlled actions are permitted to exceed elastic limits under applicable earthquake loads. Strengths for deformation controlled actions should be taken equal to expected strengths obtained experimentally or calculated by using accepted mechanics principles. Force-controlled actions are not permitted to exceed elastic limits under applicable earthquake loads.

viii. Component deformability

Deformation limits corresponding to loss of lateral load resistance, and corresponding to loss of gravity load resistance should be defined. Figure A.2 illustrates a generalized load deformation relation applicable to most concrete components. As shown in Figure A.2, there are two ways to define deformations:

Type I: In this curve, deformations are expressed directly using terms such as strain, curvature, rotation, or elongation. The parameters a and b refer to those portions of the deformation that occur after yield, that is, the plastic deformations. Parameters a, b, and c are defined numerically in Tables 9-6 through 9-12 of ATC 40, but for beams and columns ,Tables 9.6 and 9.7 of ATC 40 are presented in Tables A.1 and A.2 of this thesis. (Curve type I is convenient to use when the deformation is a flexural plastic hinge. Most computer programs for inelastic analysis will directly report the flexural plastic hinge rotation in this format so that results can be compared [2])

Type II: In this curve, deformations are expressed in terms such as shear angle and tangential drift ratio. The parameters d and e refer to total deformations measured from the origin. Parameters c, d, and e are defined numerically in Tables 9-6 through 9-12 of ATC 4040 but for beams and columns, Tables 9.6 and 9.7 of ATC 40 are presented in Tables A.1 and A.2 of this thesis.

Beams: Beams may be modeled with concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent important characteristics of reinforced concrete beam components subjected to lateral loading. The model should be capable of representing inelastic response along the component length, except where it is shown by equilibrium that yielding is restricted to the component ends. Where nonlinear response is expected in a mode other than flexure, the model should be able to represent that mode.

Columns: Columns may be modeled with concentrated plastic hinge models, distributed plastic hinge models, or other models whose behavior has been demonstrated to adequately represent. Where nonlinear response is expected in a mode other than flexure, the model should be able to represent that mode. When there are significant axial force variations under the action of earthquake loading, the model should also represent the effects of the variation on stiffness and strength properties.

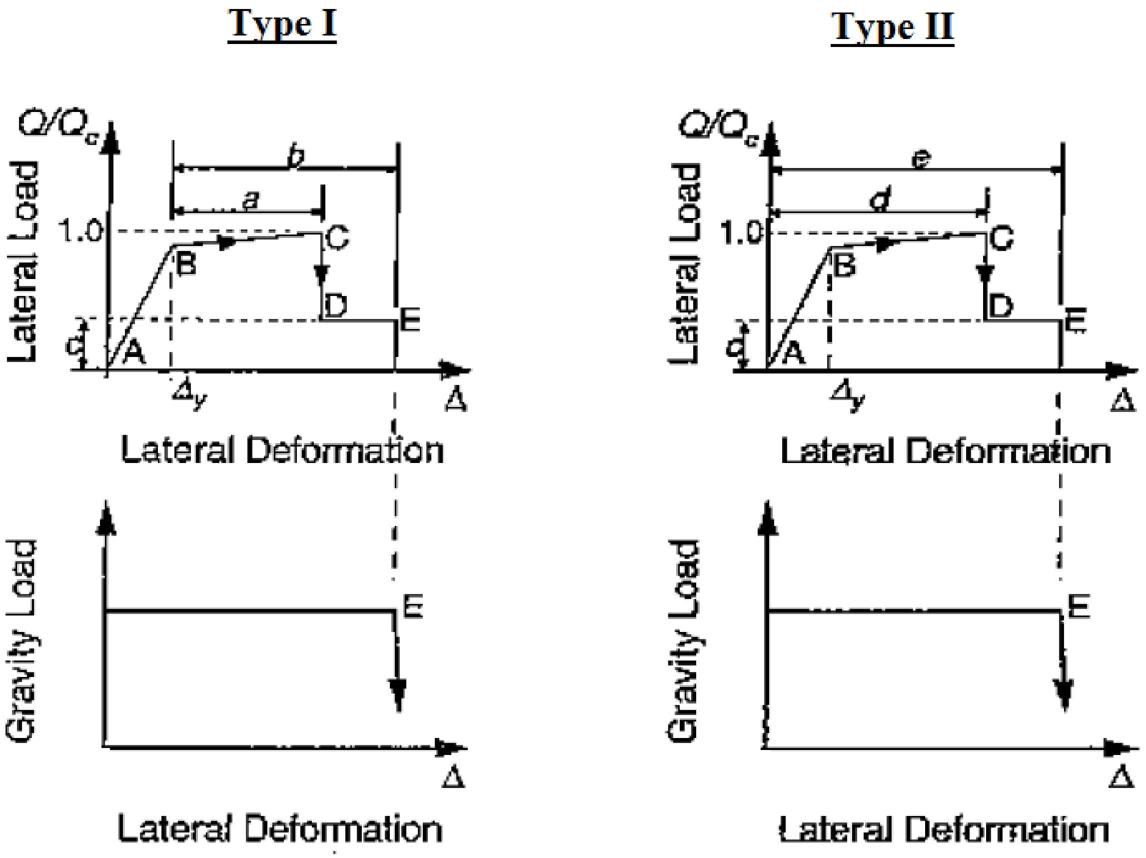
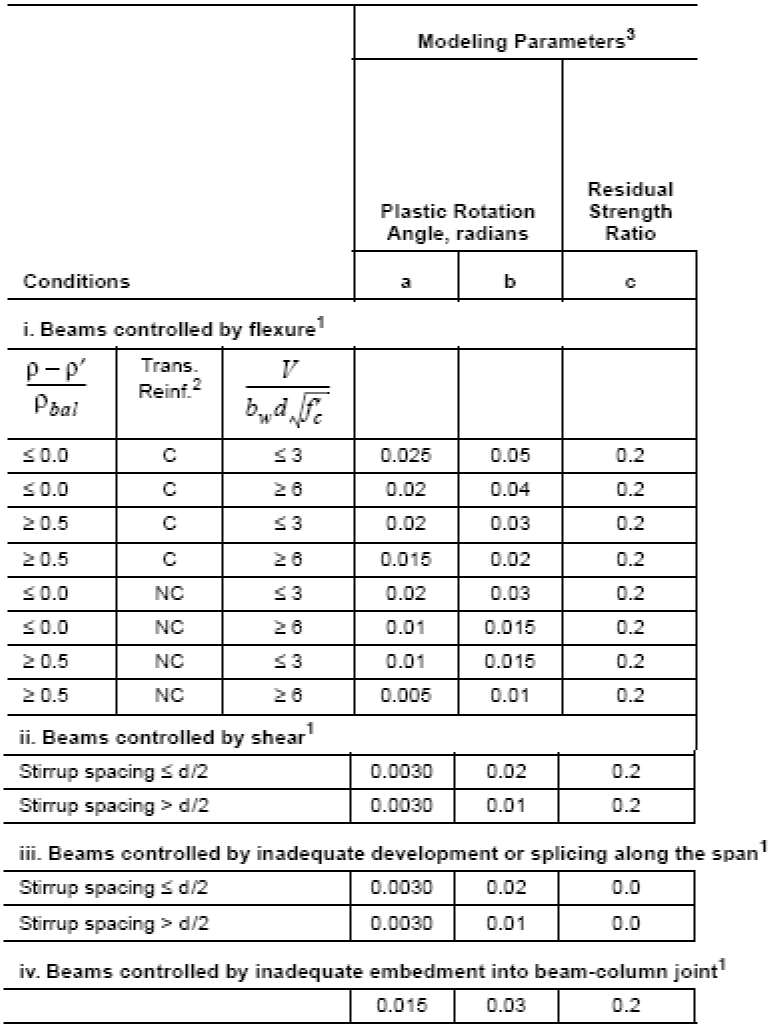


Figure A.2. Generalized Load-Deformation Relations for components [2]

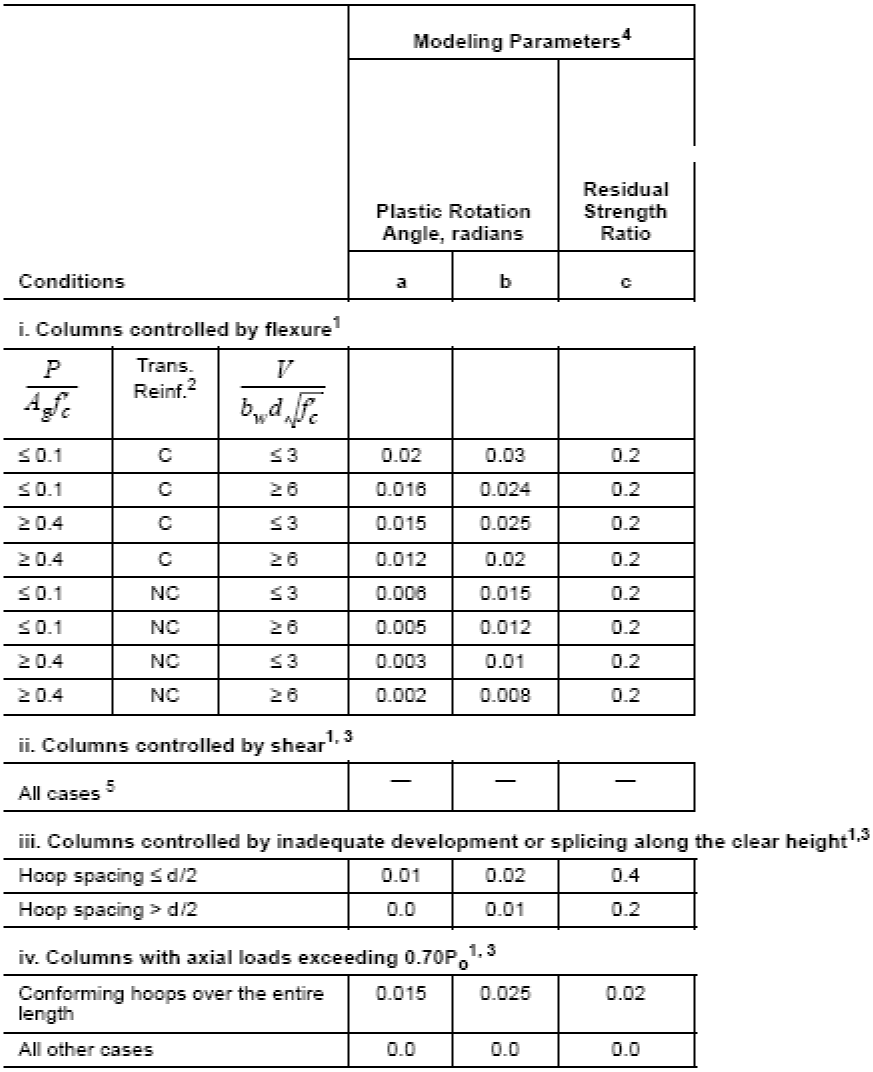
Table A.2 Modeling Parameters for Nonlinear procedures-Reinforced Concrete beams [Table 9.6 0f ATC 40]



			Modeling Parameters[3]		
			Plastic Rotation Angle, radians		Residual Strength Ratio
Conditions			a	b	c
i. Beams controlled by flexure[1]					
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$			
≤ 0.0	C	≤ 3	0.025	0.05	0.2
≤ 0.0	C	≥ 6	0.02	0.04	0.2
≥ 0.5	C	≤ 3	0.02	0.03	0.2
≥ 0.5	C	≥ 6	0.015	0.02	0.2
≤ 0.0	NC	≤ 3	0.02	0.03	0.2
≤ 0.0	NC	≥ 6	0.01	0.015	0.2
≥ 0.5	NC	≤ 3	0.01	0.015	0.2
≥ 0.5	NC	≥ 6	0.005	0.01	0.2
ii. Beams controlled by shear[1]					
Stirrup spacing ≤ d/2			0.0030	0.02	0.2
Stirrup spacing > d/2			0.0030	0.01	0.2
iii. Beams controlled by inadequate development or splicing along the span[1]					
Stirrup spacing ≤ d/2			0.0030	0.02	0.0
Stirrup spacing > d/2			0.0030	0.01	0.0
iv. Beams controlled by inadequate embedment into beam-column joint[1]					
			0.015	0.03	0.2

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. “C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if within the flexural plastic region: 1) closed stirrups are spaced at <=d/3, and 2) for components of moderate and high ductility demand the strength provided by the stirrups (Vs) is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3. Linear interpolation between values listed in the table shall be permitted.
4. V = design shear force

Table A.2 Modeling Parameters for Nonlinear procedures-Reinforced Concrete Columns [Table 9.7 0f ATC 40]



Conditions			Modeling Parameters[4] Plastic Rotation Angle, radians a	b	Residual Strength Ratio c
i. Columns controlled by flexure[1]					
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$			
≤ 0.1	C	≤ 3	0.02	0.03	0.2
≤ 0.1	C	≥ 6	0.016	0.024	0.2
≥ 0.4	C	≤ 3	0.015	0.025	0.2
≥ 0.4	C	≥ 6	0.012	0.02	0.2
≤ 0.1	NC	≤ 3	0.006	0.015	0.2
≤ 0.1	NC	≥ 6	0.005	0.012	0.2
≥ 0.4	NC	≤ 3	0.003	0.01	0.2
≥ 0.4	NC	≥ 6	0.002	0.008	0.2
ii. Columns controlled by shear[1, 3]					
All cases [5]			—	—	—
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]					
Hoop spacing ≤ d/2			0.01	0.02	0.4
Hoop spacing > d/2			0.0	0.01	0.2
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]					
Conforming hoops over the entire length			0.015	0.025	0.02
All other cases			0.0	0.0	0.0

1) When more than one of the conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.
2) Under the heading "transverse reinforcement," "C" and "NC" are abbreviations for conforming and non-conforming details, respectively. A component is conforming if within the flexural plastic hinge region: 1) closed hoops are spaced at <= *d/3,* and 2) for components of moderate and high ductility demand the strength provided by the stirrups *(Vs)* is at least three-fourths of the design shear. Otherwise, the component is considered non-conforming.
3) To qualify, 1) hoops must not be lap spliced in the cover concrete, and 2) hoops must have hooks embedded in the core or must have other details to ensure that hoops will be adequately anchored following spalling of cover concrete.
4) Linear interpolation between values listed in the table is permitted.
5) P = Design axial load
6) V = Design shear force

Appendix B: Conversion to ADRS Spectra [2]

Application of the Capacity-Spectrum technique requires that both the demand response spectra and structural capacity (or pushover) curves be plotted in the spectral acceleration vs. spectral displacement domain. Spectra plotted in this format are known as Acceleration-Displacement Response Spectra (ADRS) after Mahaney, 1993.

Every point on a response spectrum curve has associated with it a unique spectral acceleration, *Sa*, spectral velocity, Sv, spectral displacement, S*d*, and period, *T*. To convert a spectrum from the standard Sa vs *T* format found in the building code to ADRS format, it is necessary to determine the value of S*di* for each point on the curve, Sai, Ti. This can be done with the equation:

$$S_{di} = \frac{T_i^2}{4\pi^2} S_{ai} g$$

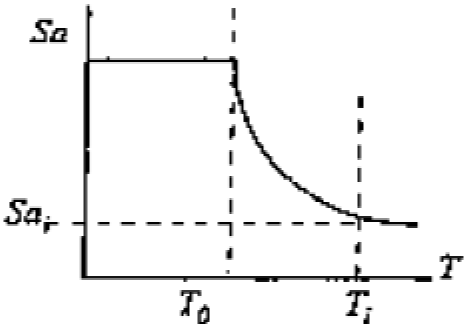


Figure A.3 Standard Format (Sa vs T)

Standard demand response spectra contain a range of constant spectral acceleration and a second range of constant spectral velocity. Spectral acceleration and displacement at period T, are given by:

$$S_{ai} g = \frac{2\pi}{T_i} S_v \qquad S_{di} = \frac{T_i}{2\pi} S_v$$

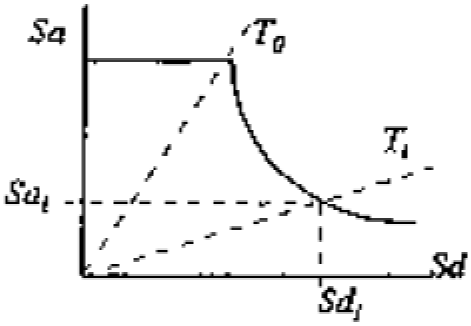


Figure A.4 ADRS Format (Sa Vs Sd)

In order to develop the capacity spectrum from the capacity (or pushover) curve, it is necessary to do a point by point conversion to first mode spectral coordinates. Any point Vi, $\Delta roof$ *on* the capacity curve is converted to the corresponding point Sai, Sdi on the capacity spectrum using the equations:

$$S_{ai} = \frac{V_i / W}{\alpha_1}$$

$$S_{di} = \frac{\Delta_{roof}}{\left(PF_1 \times \phi_{1,roof}\right)}$$

Where a1 and PF1 are respectively the modal mass coefficient and participation factors for the first natural mode of the structure and ϕ_{roof} is the roof level amplitude of the first mode.

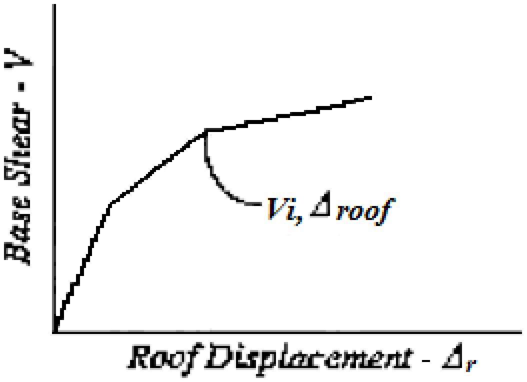


Figure A.5 Capacity curve

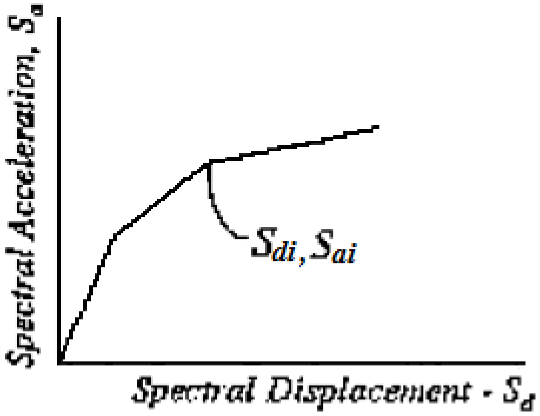


Figure A.6 Capacity spectrum

Appendix: C Verification of ETABS V 9.2.0 for pushover analysis

The objective of this verification part is to demonstrate the performance of ETABS V 9.2.0 for static nonlinear or pushover analysis. This reference column is taken from the work of Kunnath *et al* (1997).

The experimental results of two specimens, namely A1 and A2, are used as a reference for verification. The first specimen, A1, was tested under a monotonically increasing lateral loading condition and the second specimen, A2, was tested under the standard quasi-static loading until its failure condition. The general information of the specimen can be seen in Figure A.7.

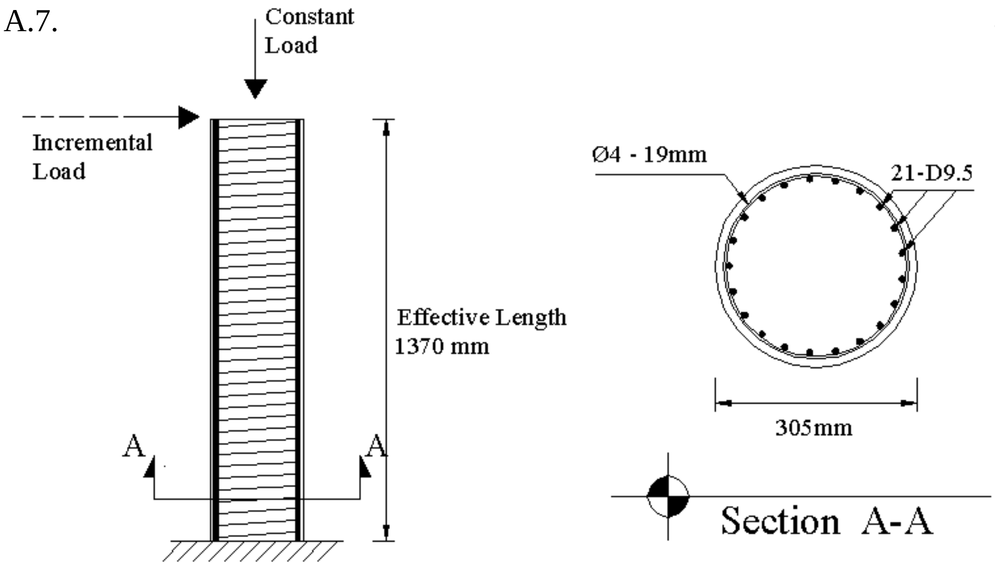


Figure A.7 Detail information of the RC column A1 and A2 specimens Kunnath *et al* (1997).

Specification and Property:

P_{axial} = 200 KN = 44,964.03 lb. (constant)

Concrete

fc' = 29 MPa = 4206.16 lb/in^2

Reinforced Bar

Longitudinal Steel

Number of steel = 21

Diameter = 9.5 mm = #3

fy_{long} = 448 MPa

Transversal Steel (using spiral)

Diameter = 4 mm

Spacing =19 mm

fy_{spiral} = 434 MPa

The above column with the given detailing is modeled in ETABS and pushover analysis is performed. The resulting lateral load versus displacement curves from the experiment is given in Figure A.8. The resulting pushover curve from ETABS nonlinear static analysis is given in Figure A.9 for comparison.

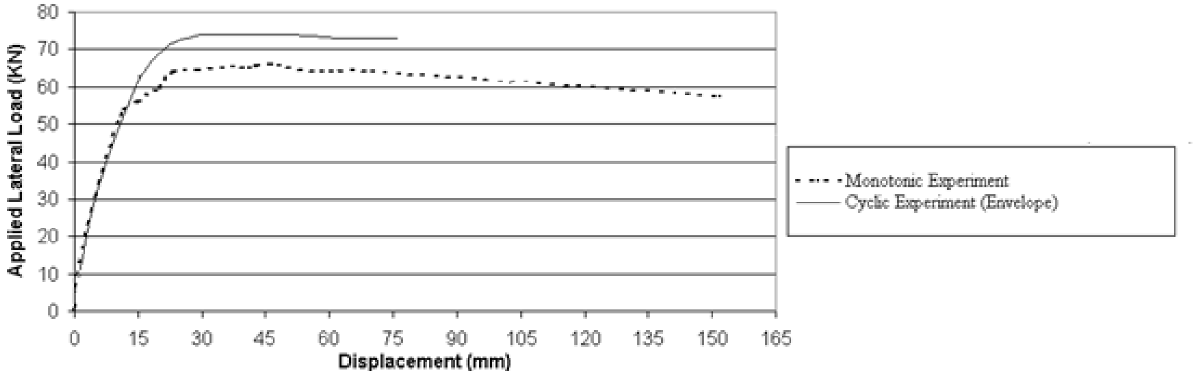


Figure A.8 Load-displacement curve of circular column (Kunnath *et al*)

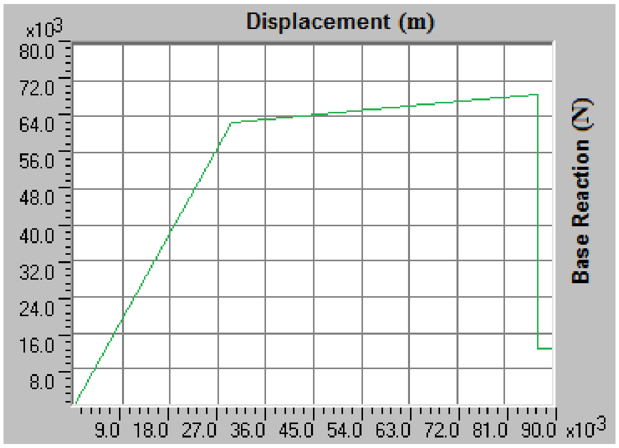


Figure A.9 Base shear versus top displacement curve ETABS static nonlinear analysis

According to the monotonic experimental result, the peak strength of the A1 specimen is 66 KN. The maximum lateral displacement is predicted occurred at 152 mm. On the other hand, from the envelope of the cyclic load, the peak load is 75KN with the maximum lateral displacement of 76mm. the ultimate strength from ETABS is 69 KN and maximum lateral displacement before failure is 87 mm.

Comparing the analytical versus experimental results, the analytical result from ETABS over-predicted the peak column strength by an approximately of 3 KN higher than monotonic experimental result. Conversely, compared to the peak strength of the A2 specimen, it has under estimated the peak column strength value by 6 KN.

Declaration

I hereby declare that the work presented in this thesis which is entitled “Assessment of Ribbed Slab Constructions Framed Only in One Direction against Lateral Load Excitations” is my original work and has not been presented in any other University and that all sources of material used for the thesis have been properly acknowledged.

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Misrak Tefera