ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Effect of Diaphragm Discontinuity in Seismic Response of Multi Storey Reinforced Concrete L-Shape Building

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A Thesis in Structural Engineering

By Minas Moges
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A Thesis
Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled '**Effect of Diaphragm Discontinuity in Seismic Response of Multi-Storey Reinforced Concrete L-Shape Building**' presented by **Minas Moges**, a candidate for the degree of Master of Science and hereby certify that it is worthy of acceptance.

Dr. AsnakeAdamu	-------------------	--------------
Advisor	Signature	Date
Dr.-Ing. Girma Zerayohannse	------------------	--------------
External examiner	Signature	Date
Dr.-Ing. Adil zekaria	-----------------	-------------
Internal examiner	Signature	Date
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Chairman	Signature	Date

UNDERTAKING

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Signature of Student

Minas Moges

Abstract

Most recent seismic code put criteria for classified buildings to account the effect come shape regularity. In the present research the buildings taken are regular in elevation and irregular in plan (L-shape). After checking elevation regularity and fundamental period of vibration satisfaction, the equivalent lateral force method of analysis used for analysis of earthquake resistant buildings. Analysis was carried out reinforced concrete framed building where parametric study made to structure exhibiting different opening location-shape planer with varied projections and varied story height based on capacity design principle. Seismic design makes use of energy absorption and dissipation to reduce the design forces in order to achieve economy by properly accounting behavior factor based on code recommendation. Diaphragm discontinuity plays a great role on the response of building when the building exposed to different types and direction of loads. It affects the performance, top story displacement and base shear of building. In this study, the evaluation of diaphragm discontinuity for elevation regular and plan irregular ductile reinforced concrete structures using nonlinear analysis is investigated. The evaluation was carried out by designing elevation regular and plan irregular structures for different parametric condition (opening location, story height and projection lengths) with the provision of the Ethiopian seismic code and by checking their performance through non-linear analysis using CSI ETABS. In this study parametric study has been conducted using same parameters including seismic location, ground type, and ductility class, loading condition, behavior factor and material type for all models. Moreover, the evaluation of diaphragm discontinuity using different models under the same conditions along with performance comparison of structures designed for same ductility class (DCM) were carried out and the findings results are concluded as follows:- For same stories on different slab opened down positions the top story displacement decreases when pushed to both x-and y-direction, especially at the slab opened at the projection part and the comparison values clearly states that for the slab opened down at an angle, main part and projection part, the base shears have decreased respectively when pushed to the x-direction and y-direction. For same stories on different projection lengths, the top story displacement increased by the same percent when pushed to x-direction and decreased when projection length increase to pushed to y-direction and the base shears decreased almost by same percent for all projections when pushed to x-direction and while the projection length increased, the base shears decreased respectively when pushed to y-direction. For different stories on same slab opened down positions, the slab opened down in all stories led the top story displacement to be decreased approximately by the same percentage when pushed to the x direction and when the story level increased, the top story displacement decreased when pushed to y-direction and the base shears decreased almost by same percent when pushed to the x-direction and the base shears decreased when story height increased respectively, when pushed to the y-direction.

KEY WORD- *Diaphragm, Projection, Opened down, pushover*

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ABBREVIATIONS

MMC	A method of modal combinations
TEC	Turkish Earthquake Code
IBC	International Building Code
POA	Static pushover analysis
EHF	Equivalent Horizontal Force
ΣMRb	Sum of the beam moment of resistance at end
∑MRc	Sum of the design values of the moments of resistance of the columns framing the joint.
P-Δ	Axial load and deflection
P-M	Moment curvature interactions
MRSF	Moment-resisting space frames
DCM	Medium ductility
DCH	High ductility
2nd	Second
e.g	Example
MDOF	Multi degree of freedom
EC8	Euro code eight
FEMA	Finite element analysis
IS	Indian standard code
DL	Dead load
LL	Live load
EQ	Earthquake load
UDL	Uniform distributed load
MPa	Mega pascal
kN/m^3	Kilo Newton per meter cube
Comb	Combination
ES EN	Ethiopian standard European norm
LLRS	Lateral load resist system
ATC	Applied technical council

LIST OF SYMBOLS

As or Ast	Area of reinforcement bar
Ag or Ac	Area of concrete
b	Width of section
Fb	Base shear
P	Applied axial load of column
Sd (T1)	Design spectrum
T	Time
M	Mass
λ	Lambda
Lcr	Length of the critical region
εsy,d	Design value of tension steel strain at yield
ρ	The reinforcement ratios of the tension zone
ρ'	The reinforcement ratios in compression zone
b_C	Cross-sectional dimension of the column
h_W	The depth of the beam
b_W	Minimum width of beam
ν_d	The reduction factor
h	Inter story height
V_{tot}	Total seismic story shear
P_{tot}	Total gravity load
θ	Inter-story drift sensitivity
q_o	The basic value of the behavior factor
M_{i2} and M_{i3}	Geometric imperfection moments,
L	Actual length of the vertical member
e_i	Eccentricity
S	Soil condition
%	Percent
αu/α1	Over strength factor
As	Area of reinforcement

LIST OF SYMBOLS (CONTINUED)

Ec	Modulus of elasticity of concrete
Es	Modulus of elasticity of reinforcing bar
Φ	Curvature
fy	Yield strength of reinforcement bar
Φu	Ultimate curvature
Φ	Diameter of reinforcing bar
Δ	Building top story deflection relative to base
δ	Story deflection relative to the story just below it
ρ	Longitudinal reinforcing bar area ratio

CHAPTER ONE: INTRODUCTION

1.1. Background

In multi-storied framed building, damages from earthquake generally initiates at locations of structural weaknesses present in the lateral load resisting frames. This behavior of multi-stores framed buildings during strong earthquake motions depends on the distribution of mass, stiffness, strength in both the horizontal and vertical planes of buildings. In few cases, these weaknesses may be created by discontinuities in stiffness, strength or mass along the diaphragm. Such discontinuities between diaphragms are often associated with sudden variations in the frame geometry along the length of the building. Structural engineers have developed confidence in the design of buildings in which the distributions of mass, stiffness and strength are more or less uniform. There is a less confidence about the design of structures having irregular geometrical configurations and diaphragm discontinuities.

All parts of the structure between separation joints shall be interconnected to form a continuous path to the lateral force-resisting system, and the connections shall be capable of transmitting the lateral forces induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having strength to resist a force of not less than 5% of the portion's weight [1].

Irregularity of building structure is major problem which leads to disaster during severe earthquake. Irregularities are not avoidable in construction of buildings; however the behavior of structures with these irregularities during earthquake needs to be studied. In order to prevent damages due to irregularity problem, seismic demands must be determined accurately. Several studies have focused on evaluating the response of regular structures.

In the present thesis, the effect of diaphragm discontinuity on the seismic response and performance of a selected common peripherally irregular plan (L) shape multi story building is studied. Multistory building having discontinuity floor diaphragms that considerably weaken slab capacity and affect even distribution of seismic loads to the vertical lateral load resisting elements. Most of the time buildings or structures with floor plan have open down throughout the floor or in some section of the floor like mezzanine floors in a building. The existence of these openings has different architectural function or aesthetic value. Slab opening down (diaphragm discontinuity) which affects rigidity of a diaphragm and distribution of lateral load to the lateral load resisting element. Though we know opening down has adverse effect in building in load transferring mechanism, no rules have been set in all country building codes to prefer place(position) where opening down has less effect, opening down effect relation to story height and projection length relation to opening down.

Among the engineering community involved with the development of seismic design procedures, there is a general belief that the conventional elastic design and analysis methods cannot capture many important aspects that control the seismic performance of structures in severe earthquakes. Moreover, the other powerful tool, inelastic time-history analysis is computationally expensive and not feasible for many cases. Nowadays, engineers are seeking a technique, which would solve the drawbacks described above.

The study for a more useful and rational design process is a big issue for the future. Design has always been a compromise between simplicity and reality. The latter term, reality, seems to be very complex due to big uncertainties in imposed demands and available capacities. The first term, simplicity, is a necessity driven by computational cost and at the same time the limited ability to implement complexity with available knowledge and tools [2].

1.2 Objective and scope of study

1.2.1 Objective of study

1.2.1.1 General objective

The general objective of this thesis is to study the seismic behavior and performance of L-shape reinforced concrete building with floor slab open down.

1.2.1.2 Specific objective

a) To investigate the seismic performance of a multi storied building with slab openings at selected position using nonlinear static analysis (pushover analysis).

b) To investigates the suitable and practical location of openings.

c) Compare and select position of opening down in L-shape building.

d) Evaluate effect of story height and projection in response and performance of L-shape building.

1.2.2 Scope of the study.

This study paper considers the effect of diaphragm discontinuity in L-shape building due to open down. In the study 16 building models selected and analyzed using commercial software ETABS2016.Based on design requirement, all selected building models designed properly. To compare the result nonlinear static (pushover) analysis selected because this method of analysis use to evaluate the performance building by considering nonlinear behavior of material. In this study the following response use to compare, these are top story displacement and base shear. This study is done for reinforced concrete framed multistory building with fixed support conditions

This study is conducted to evaluate problems in structural behavior of buildings during earthquake. An assumption is made in the seismic analysis that all the structures behave linearly elastic during seismic excitations which intern does not clear the non-linearity of the structure. So in order to understand better the non-linear response of the structures this study would be useful.

CHAPTER TWO: LITERATURE REVIEW

2.1 Floor diaphragm and structural function

2.1.1 General feature of diaphragm

The main function of floor and roof systems is to support gravity loads and to transfer these loads to other structural members such as columns and load resisting walls. Furthermore, they play a central role in the distribution of lateral force to the vertical elements of the lateral load resisting system.

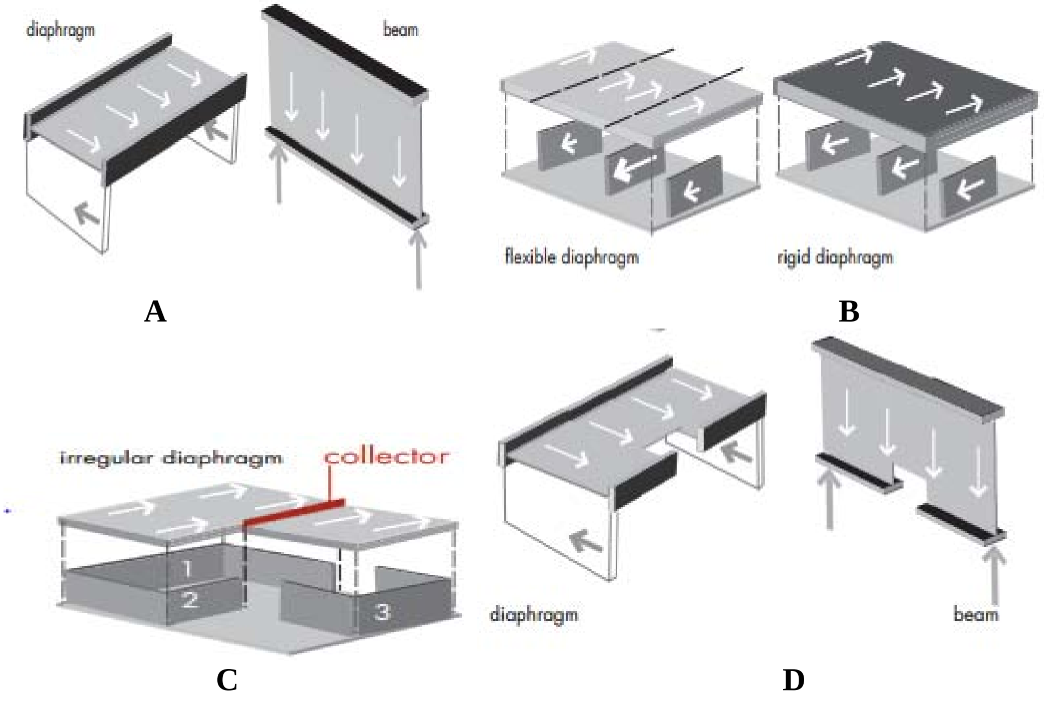
Diaphragm is horizontal-resistance members that transfer lateral forces between vertical resistance elements like shear walls or frames. The diaphragms are generally provided by the floor and roof elements of the building; sometimes, however, horizontal bracing systems independent of the roof or floor structure serve as diaphragms. The diaphragm is an important element in the entire seismic resistance system as describe in Figure 2.1.

The diaphragm can be visualized as a wide horizontal beam with components at its edges, termed chords, designed to resist tension and compression: chords are similar to the flanges of a vertical beam as describe in below Figure 2.1A.

A diaphragm serves in different forms in structural part of a resistant system may act either in a flexible or rigid manner, depending partly on its size means the area between enclosing resistance elements or stiffening beams and also on its material. The flexibility of the diaphragm, relative to the shear walls whose forces it is transmitting, also has a major influence on the nature and magnitude of those forces. With flexible diaphragms made of wood or steel decking without concrete, walls take loads according to tributary areas. With rigid diaphragms mostly concrete slabs, walls share the loads in proportion to their stiffness as showed in Figure 2.1B.

Collectors, drag struts or ties, are diaphragm framing members that collect or drag diaphragm shear forces from laterally unsupported areas to vertical resisting elements as it described in Figure 2.1C below.

Most of time floors and roofs have to be penetrated by staircases, elevator and duct shafts, skylights, and atria. The size and location of these penetrations are critical to the effectiveness of the diaphragm. The reason for this is not hard to see when the diaphragm is visualized as a beam. For example, it can be seen that openings cut in the tension flange of a beam will seriously weaken its load carrying capacity. In a vertical load-bearing situation, a penetration through a beam flange would occur in either a tensile or compressive region. In a lateral load system, the hole would be in a region of both tension and compression, since the loading alternates rapidly in direction based on the activity .This statement described by Figure 2.1D below.



Figur2. 1 Diaphragms for horizontal resistance system [3]

In buildings and other structures which incorporate floors, floors play a very important role in the overall seismic behavior of the structure. They act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. The action of floors as diaphragms is especially relevant in cases of complex and non-uniform layouts of the vertical structural systems, or where systems with different horizontal deformability characteristics are used together.

Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure.

Diaphragms should have sufficient in-plane stiffness for the distribution of horizontal inertia forces to the vertical structural systems in accordance with the assumptions of the analysis particularly when there are significant changes in stiffness or offsets of vertical elements above and below the diaphragm.

The plan configuration shall be compact and each floor shall be delimited by a polygonal convex line. If in plan set-backs exist, regularity in plan may still be considered as being satisfied, provided that these setbacks do not affect the floor in-plan stiffness and that, for each set-back, the area between the outline of the floor and a convex polygonal line enveloping the floor does not exceed 5 % of the floor area.

The in-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements. In this respect, the L, C, H, I, and X plan shapes should be carefully examined, notably as concerns the stiffness of the lateral branches, which should be comparable to that of the central part, in order to satisfy the rigid diaphragm condition [4].

2.1.2 Structural function of diaphragms

Diaphragms serve multiple roles to resist gravity and lateral forces in buildings. Among the roles main are below:-

- **Resist gravity loads** – Most diaphragms are part of the floor and roof framing and therefore support gravity loads.

- **Provide lateral support to vertical elements** – Diaphragms connect to vertical elements of the seismic force-resisting system at each floor level, thereby providing lateral support to resist buckling as well as second-order forces associated with axial forces acting through lateral displacements. Furthermore, by tying together the vertical elements of the lateral force-resisting system, the diaphragms complete the three-dimensional framework to resist lateral loads.

- **Resist out-of-plane forces –** Exterior walls and cladding develop out-of-plane lateral inertial forces as a building responds to an earthquake. Out-of-plane forces also develop due to wind pressure acting on exposed wall surfaces. The diaphragm-to-wall connections provide resistance to these out-of-plane forces.

- **Transfer lateral inertial forces to vertical elements of the seismic force**-Resisting system the floor system commonly comprise most of the mass of the building. Consequently, significant inertial forces can develop in the plane of the diaphragm. One of the primary roles of the diaphragm in earthquake-resistant buildings is to transfer these lateral inertial forces, including those due to tributary portions of walls and columns, to the vertical elements of the seismic force-resisting system.

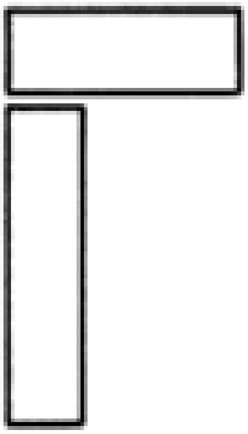
- **Transfer forces through the diaphragm –** As a building responds to earthquake loading, lateral shears often must be transferred from one vertical element of the seismic force-resisting system to another. The largest transfers commonly occur at discontinuities in the vertical elements, including in-plane and out-of-plane offsets in these elements.

- **Support soil loads below grade –** For buildings with subterranean levels, soil pressure bears against the basement walls out-of-plane. The basement walls span between diaphragms, producing compressive reaction forces at the edge of the diaphragms [5].

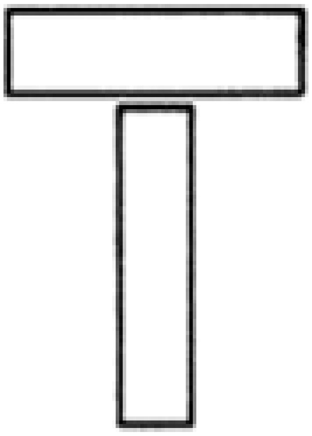
Inappropriate location or large-size openings or stair or elevator cores, atriums, and skylights create problems similar to those related to cutting a hole in the web of a beam. This reduces the ability of the diaphragm to transfer the forces and may cause failure [6].

Buildings having plans with shapes like, L, T, E and Y shall preferably be separated into rectangular parts by providing separation sections at appropriate places. Typical examples are shown in figure2.3.The buildings with small lengths of projections forming L, T, E or Y shapes need not be provided with separation section. In such cases the length of the projection may not exceed 15 to 20 percent of the total dimension of the building in the direction of the projection.

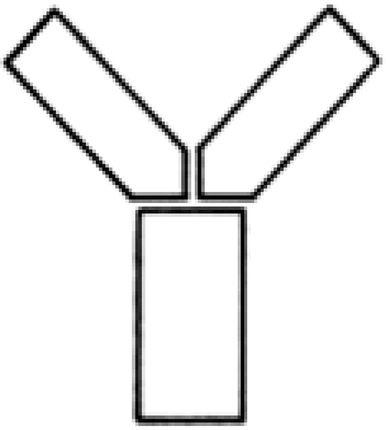
For buildings with minor asymmetry in plan and elevation separation sections may be omitted. Strength in Various Directions The structure shall be designed to have adequate strength against earthquake effects along both the horizontal axes. The design shall also be safe considering the reversible nature of earthquake forces.



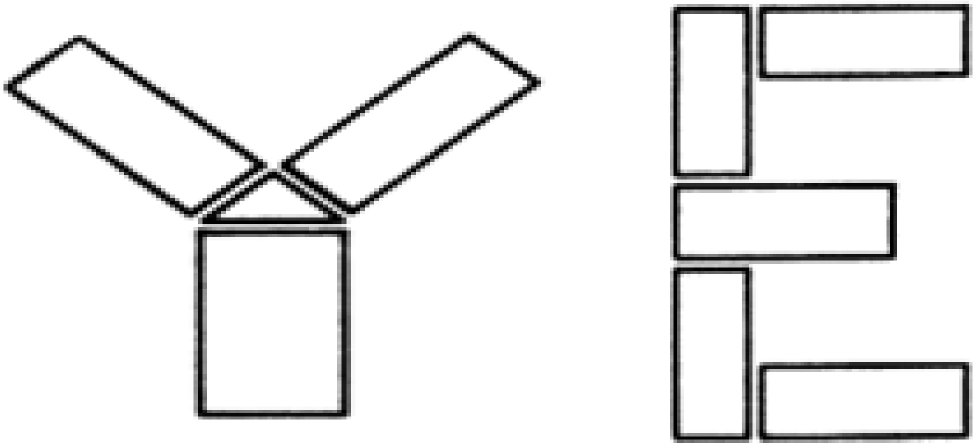
L-SHAPE



T-SHAPE



Y-SHAPE



Y-SHAPE E-SHAPE

Figure2. 2: Typical shapes of building with separation sections [7].

2.1.3 Effects of opening (Diaphragm discontinuity)

Gravity and earthquake loads should flow in a continuous and smooth path through the horizontal and vertical elements of structures and be transferred to the supporting ground. Discontinuities are, however, frequently present in plan and elevation. Sidestepping and offsetting are common vertical discontinuities, which lead to unfavorable stress concentrations. In plan, openings in diaphragms may considerably weaken slab capacities. This reduction of resistance depends on the location, size and shape of the openings. High stress concentrations caused by a large opening for stairwells in a floor slab. Conversely, small openings do not jeopardize the load transfer at a floor level; the diaphragm behaves like a continuous beam under uniform seismic forces. High stress concentrations may also exist at the connection between structural walls and slabs, as well as between columns and flat slabs [8].

2.2 Stiffness and strength requirements

Strong earthquake shaking will induce both vertical and lateral forces in a structure. The lateral forces that tend to move structures horizontally have proven to be particularly damaging. If a structure has inadequate lateral stiffness or strength, these lateral forces can produce large horizontal displacements in the structure and potentially cause instability [9].

2.2.1 Regularity

A structure is regular if the distribution of its mass, strength, and stiffness is such that it will sway in a uniform manner when subjected to ground shaking means the lateral movement in each story and on each side of the structure will be about the same.

Regular structures tend to dissipate the earthquake's energy uniformly throughout the structure, resulting in relatively light but well-distributed damage. In an irregular structure, however, the damage can be concentrated in one or a few locations, resulting in extreme local damage and a loss of the structure's ability to survive the shaking [9].

2.2.2 Configuration

This deficiency category covers configuration irregularities that adversely affect performance. In codes for new buildings, these configuration features are often divided into plan irregularities and vertical irregularities. Plan irregularities are features that may place extraordinary demands on elements due to torsion response or the shape of the diaphragm. Vertical irregularities are created by uneven vertical distribution of mass or stiffness between floors that may result in concentration of force or displacement at certain levels. In older existing buildings, such irregularities were seldom taken into consideration in the original design and therefore normally require retrofit measures to mitigate. In prescriptive evaluation methods, features that qualify as irregularities are defined by rules, similar to the rules used for new buildings.

Evaluation methods that explicitly consider nonlinear behavior will normally identify concentrations of force or displacement due to configuration and the components affected by these concentrations will be shown to have inadequate capacity [10].

2.3 Structural load path of building

Buildings are generally composed of vertical and horizontal structural elements. The vertical elements commonly used to transfer lateral forces to the ground are: shear walls, braced frames and moment-resisting frames. The horizontal elements that distribute lateral forces to the vertical elements are: diaphragms, such as floor and roof slabs and horizontal bracing that transfers large shears from discontinuous walls or braces.

The seismic forces that are proportional to the mass of the building elements are considered to act at their centers of mass. All of the inertia forces originating from the masses on and off the structure must be transmitted to the lateral force-resisting elements, and then to the base of the structure and into the ground.

A complete load path is a basic requirement for all buildings. There must be a complete lateral-force-resisting system that forms a continuous load path between the foundation, all diaphragm levels, and all portions of the building for proper seismic performance. The general load path is as follows. Seismic forces originating throughout the building, mostly in the heavier mass elements such as diaphragms, are delivered through connections to horizontal diaphragms; the diaphragms distribute these forces to vertical force-resisting elements such as shear walls and frames; the vertical elements transfer the forces into the foundation; and the foundation transfers the forces into the supporting soil.

If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements. Interconnecting the elements needed to complete the load path is necessary to achieve good seismic performance. Examples of gaps in the load path would include a shear wall that does not extend to the foundation, a missing shear transfer connection between a diaphragm and vertical elements, a discontinuous chord at a diaphragm's notch, or a reentrant corner, or a missing collector [6].

2.4 Structural analysis

2.4.1 Modeling

1. The model of the building shall adequately represent the distribution of stiffness and mass in it so that all significant deformation shapes and inertia forces are properly accounted for under the seismic action considered. In the case of non-linear analysis, the model shall also adequately represent the distribution of strength

2. The model should also account for the contribution of joint regions to the deformability of the building, e.g. the end zones in beams or columns of frame type structures. Non-structural

elements, which may influence the response of the primary seismic structure, should also be accounted for.

3. In general the structure may be considered to consist of a number of vertical and lateral load resisting systems, connected by horizontal diaphragms.

4. When the floor diaphragms of the building may be taken as being rigid in their planes, the masses and the moments of inertia of each floor may be lumped at the center of gravity [4].

2.4.2 Analysis

Load effects on individual structural members shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility, and both short- and long-term material properties. Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life [1].

2.4.3 Non-linear static (pushover) analysis

2.4.3.1 General

Nonlinear static pushover is analysis type which used to evaluate performance of existing or newly expected building to construct which is performed by subjecting the structure a monotonically increase pattern of lateral loads on it which represents the forces that the structure may experience during earth quake. During incrementally increasing loads different structural elements of a given building elements may yield sequentially.

Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. It may be applied to verify the structural performance of newly designed and of existing buildings for the following purposes:

a) To verify or revise the over strength ratio values $\alpha u/\alpha 1$
b) To estimate the expected plastic mechanisms and the distribution of damage;
c) To assess the structural performance of existing or retrofitted buildings

d) As an alternative to the design based on linear-elastic analysis which uses the behavior factor q should be used as the basis of the design [4].

2.4.4 Structural performance hierarchy

As the structural concept for a tall building is being developed, clearly identify zones or elements where nonlinear response is anticipated. Capacity design concepts are a good starting point when considering desirable system and element actions.

While a strict application of capacity design may not be practical or even warranted in the final design, early consideration of these principles will help establish a clear hierarchy to the anticipated building response and will serve to guide the development of the design, which will later be confirmed through nonlinear response history analysis.

A primary aim of the preliminary design should be to select a target yielding mechanism that is practical within the ductility limits of available structural components. For frame or braced frame structures, yielding that is well distributed over the height is preferred to yielding that is concentrated in one or few stories.

For core-wall structures, a targeted flexural yielding mechanism that distributes flexural yielding over the lower stories just above a podium may be acceptable. Another aim of the preliminary design is to target yielding to occur in components that are reliably capable of ductile response.

Desirable modes of inelastic response include, but are not necessarily limited to, the following:

- Flexural yielding in reinforced concrete beams, slabs, shear (structural) walls, and conventionally reinforced coupling beams with relatively slender proportions
- Yielding of diagonal reinforcement in diagonally reinforced concrete coupling beams
- Tension yielding in structural steel braces and steel plate shear walls, and tension/compression yielding in buckling-restrained braces
- Post-buckling compression in structural steel braces that are not essential parts of the gravity-load system, and whose buckling does not compromise system behavior

- Shear yielding in structural steel components such as panel zones in moment frames, shear links in eccentric braced frames, and steel coupling beams
- Yielding of outrigger elements, while protecting the axial-load-resisting capacity of gravity-load-carrying outrigger columns
- Yielding in ductile fuses or energy dissipation devices controlled rocking of foundations where designs require yielding in components such as gravity-load-carrying columns, for example at the intersection of a frame column with a basement wall or a frame column with roof beams, the design should provide details, possibly beyond the minimum requirements of the Building Code, that ensure adequate behavior at such yielding locations.

These yielding locations should be brought to the attention of the structural performance evaluation [11].

2.5 Structural response to a seismic action

2.5.1 Building drift

Drift is generally defined as the lateral displacement of one floor relative to the floor below. Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Stress or strength limitations in ductile materials do not always provide adequate drift control, especially for tall buildings with relatively flexible moment-resisting frames or narrow shear walls. Total building drift is the absolute displacement of any point relative to the base. Adjoining buildings or adjoining sections of the same building may not have identical modes of response, and therefore may have a tendency to pound against one another. Building separations or joints must be provided to permit adjoining buildings to respond independently to earthquake ground motion [6].

Story drift is expressed as the difference of the deflections at the top and bottom of the story under consideration: this is also often expressed as a ratio between the deflection and the story, or floor-to floor height as illustrated Figure 2.3.

Drift limits serve to prevent possible damage to interior or exterior walls that are attached to the structure and which might be cracked or distorted if the structure deflects too much laterally, creating racking forces in the member. Thus the IBC requires that drift be limited in typical buildings to between 0.02 and 0.01 times the building height, depending on the occupancy of the building. For a building that is 30 feet high, drift would be limited to between 3.6 inches and 7.2 inches depending on the building type. When the earthquake-induced drift is excessive, vertical members may become permanently deformed; excessive deformation can lead to structural and nonstructural damage and, ultimately, collapse

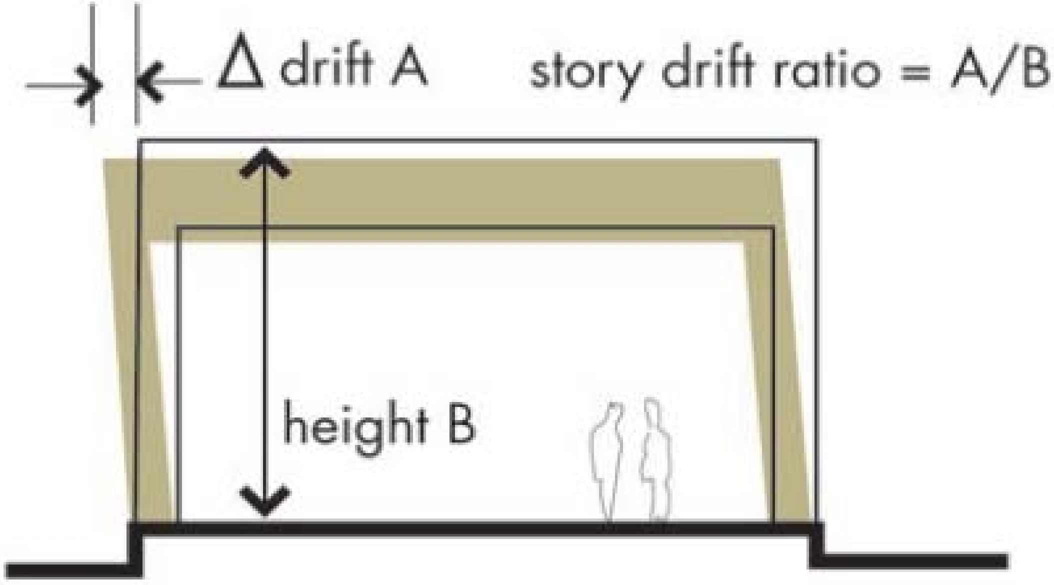


Figure 2. 3 Story drift [3]

2.5.2 Natural period of vibration

The ground shaking during an earthquake contains a mixture of many sinusoidal waves of different frequencies, ranging from short to long periods. The time taken by the wave to complete one cycle of motion is called period of the earthquake wave. In general, earthquake shaking of the ground has waves whose periods vary in the range 0.03 - 33sec. Even within this range, some earthquake waves are stronger than the others. Intensity of earthquake waves at a particular building location depends on a number of factors, including the magnitude of the earthquake, the epicentral distance, the type of ground that the earthquake waves traveled through before reaching the location of interest and rigidity of the structure, flexible building undergoes larger relative horizontal displacements than rigid building[13].

2.5.3 Lateral force distribution

Floor diaphragms in reinforced concrete buildings are typically modeled as rigid during the design phase and so the effect of in-plane diaphragm flexibility on the structure is often not considered. For the rigid diaphragm model, the diaphragm has equal in-plane displacements along its entire length under lateral load such that horizontal forces are transferred to the vertical LLRS proportional to the relative stiffness of each frame.

A flexible diaphragm, however, exhibits in-plane bending due to lateral load, resulting in additional horizontal displacements along its length. This can lead to damage of the diaphragm due to high flexural stresses along its boundaries. This flexibility also increases the lateral load transfer to frames that were not designed to carry these additional lateral loads based on a rigid diaphragm model. If this effect is sizeable, it can lead to overloading of structural elements [14].

2.5.4 Capacity curve

The relation between base shear force and the control displacement should be determined by pushover analysis for values of the control displacement ranging between zero and the value corresponding to 150% of the target displacement, the control displacement may be taken at the Centre of mass of the roof of the building [4].

2.5.5 Base shear force

Base shear is an estimate of maximum expected lateral force that will occur due to ground seismic motion at base of the structure. It depends on:-

- Soil condition at the site
- Potential source of seismic activity
- Level of building ductility nature
- Fundamental period of vibration
- Mass of building that expose to seismic

The seismic base shear force Fb, for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$$\mathrm{Fb} = \mathrm{Sd}\ (\mathrm{T1}).\mathrm{m}.\lambda \qquad (2.1)$$

Where

Sd (T1) is the ordinate of the design spectrum at period T1;

T1 is the fundamental period of vibration of the building for lateral motion in the direction; m is the total mass of the building, above the foundation or above the top of a rigid basement

λ is the correction factor, the value of which is equal to: λ = 0,85 if T1 < 2 TC and the building has more than two stories, or λ = 1,0 otherwise[4].

2.5.6 Hinge

Each beam and column element requires hinge elements to properly analyze the nonlinear behavior of the structure. The hinge elements were assigned to either end of the column and beam elements. As the intensity of the earthquakes increases, the moment experienced within the beams and columns also increases. At some point, the moment experienced by these elements will exceed the capacity and a hinge will form, the load will shift, and the system will release energy due to the hysteretic moment rotation behavior assumed for plastic hinges [15].

Hinge properties are used to define nonlinear moment-rotation behavior that can be assigned to discrete locations along length of frame objects. These hinges are used during static nonlinear analysis. The number of hinges affects computation time, so it is strongly recommended that to assign hinge where the occurrence of nonlinear behavior is highly probable.

2.5.7 Story displacement

Due to lateral load structure displace to horizontal direction but the magnitude of displacement depends on structure type, magnitude of lateral and nature of material which the structure made.

The displacement of frame obtained from linear static pushover analysis used to understand the displacement capacity and stiffness capacity of building. At every deformation step, the plastic hinge location can determine and hinge state also shows.

2.5.8 Safety ratio

The place in graph where capacity curve meet the demand curve we call it performance point. So using this control point in all building the performance can measure. The ratio of base shear at performance point to design base shear can call safety ratio. The one has highest safety ratio can take building in good performance. Since safety is one of the prime important parameter is to be considered in seismic design

2.6 Previous studies on seismic response of building

There are a lots research studies done by different researchers on irregularities of building both in plan and elevation. These works are summarized as follows.

R. BENTO et. al on their paper studied non-linear static procedures in performance based seismic design. The performance of a structural system can be evaluated resorting to non-linear static analysis. This involves the estimation of the structural strength and deformation demands and the comparison with the available capacities at desired performance levels. This paper aims at evaluating and comparing the response of two reinforced concrete building systems by the use of different methodologies namely the ones described by the ATC-40 and the FEMA-273 and by the Euro code 8 design code using nonlinear static procedures, with described acceptance criteria. The methodologies are applied to a 4 and 8 story frames system, both designed as per the Euro codes in the context of Performance Based Seismic design procedures .The result showed that non-linear static analysis more appropriate for low rise and high frequency structures, i.e. for structures that vibrate primarily in the fundamental mode and those analysis method may be expose design weaknesses that may remain hidden in an elastic analysis, such as: weaknesses due to story mechanisms, excessive deformation demands and strength irregularities; [16].

Erol KALKAN1 and Sashi K. KUNNATH on their paper studied method of modal combinations for pushover analysis of buildings nonlinear static procedures are finding widespread use in performance based seismic design since it provides practitioners a relatively simple approach to estimate inelastic structural response measures. However, conventional nonlinear static procedures using lateral load patterns recommended in FEMA-356 do not adequately represent the effects of varying dynamic characteristics during the inelastic response or the influence of higher modes. To overcome these drawbacks, they used method of modal combinations. A method of modal combinations (MMC) that implicitly accounts for higher mode effects is investigated in this paper. MMC is based on invariant force distributions formed from the factored combination of independent modal contributions. The validity of the procedure is validated by comparing response quantities (inter-story drift and member ductility demands) using other pushover methods and nonlinear time history analyses. The validation studies are based on evaluation of three existing steel moment frame buildings: two of these structures were instrumented during the Northridge earthquake thereby providing realistic support motions for the time-history predictions. The result from the investigation show that the method of modal combinations provides a basis for estimating the potential contributions of higher modes when determining inter-story drift demands and local component demands in multistory frame buildings subjected to seismic loads [17].

Boonyapinyo, V. et. al on their paper studied seismic performance evaluation of reinforced-concrete buildings by static pushover and nonlinear dynamic analyses. The prediction of inelastic seismic responses and the evaluation of seismic performance of a building structure are very important subjects in performance-based seismic design. The seismic performances of reinforced-concrete buildings evaluated by nonlinear static analysis (pushover analysis and modal pushover analysis) and nonlinear time history analysis are compared.

A finite element model that can accurately simulate nonlinear behavior of building is formulated by considering several important effects such as p-delta, masonry in-fill walls, soil-structure interaction, and beam-column joints that can be considered rigid zones with joint failure due to poor detailing of joints. Both global responses such as system ductility demand and local response such as inter-story drift are investigated is done on this study. A numerical example is performed on a 9-story reinforced concrete building. Finally, the global and local responses obtained from the modal pushover analysis are compared with those obtained from the nonlinear dynamic analysis of MDOF system. The results show that the modal pushover analysis is accurate enough for practical applications in seismic performance evaluation when compared with the nonlinear dynamic analysis of MDOF system [18].

M.T. Al Harashet. al on their paper studied inelastic seismic response of reinforced concrete buildings with floor diaphragm openings. As they stated floor has great role in carrying, transferring of gravity load and also it distributes earthquake induced load to load resisting system by diaphragm action. In reinforced concrete buildings, the in-plane flexibility of the floor diaphragms is often ignored for simplicity in practical design (i.e., the floor systems are frequently treated as perfectly rigid diaphragms). Past research, which is acknowledged in recent building standards, has shown that this assumption can result in considerable error when predicting seismic response of reinforced concrete buildings when diaphragm plan aspect ratio is greater than 3:1.

In order to investigate the influence of diaphragm openings on the seismic response of reinforced concrete buildings; two 3-story reinforced concrete buildings are designed as a building Frame System. Each building is analyzed with and without floor openings -4 cases. The inelastic behavior of the buildings is investigated under both static lateral loads (push-over) and dynamic ground motions (time-history), where a suite of three well-known earthquakes is scaled to model moderate ground motions. The diaphragm parametric study conducted involves two opening size/locations and two lateral load resisting frames stiffness/locations, where three types of floor diaphragm models (rigid, elastic, and inelastic) are assumed. It was concluded that in order to capture the seismic response of reinforced concrete buildings with floor diaphragm openings accurately; it is necessary to use an inelastic diaphragm model [19].

M. Piazza et. al on their paper studied the role of in-plane floor stiffness in the seismic behavior of traditional buildings. The structural behavior of an existing masonry building subjected to seismic action, is strongly affected by the in-plane stiffness of the floors, and by the connections between the horizontal diaphragms and the masonry walls. The study aim to experimentally evaluate the behavior of timber floor refurbished using different techniques, with special regard to the in-plane stiffness. The size adopted for the specimens are 5 m span and 4 m width. Taking into account the size of the specimens, and the need to determine the in-plane strength and stiffness of the floor, a special test set-up has been designed and adopted in order to allow the free in-plane deformation of the floor itself subjected to lateral load: the load configuration applied to the floor simulates the effect of seismic action on the floor. The experimental phase of the research aims to calibrate engineered models that can be used for studying existing structures. The result shows the in-plane stiffness of the floors strongly affects the structural behavior of an existing masonry building subjected to seismic action. It defines the seismic distribution of forces on lateral walls and the request displacement for verifying the out–of–plane mechanism of the walls. [20].

Pu YANG And Yayong WANG on their paper studied a study on improvement of pushover analysis. The static pushover analysis is becoming popular as a simplified computer method for seismic performance evaluation of structures. This method implies that the response of the structure is only controlled by the first mode, and the mode keeps constant during time history. Several examples illustrate that the structural maximum responses under-estimated the influence of higher modes compared to the results obtained from dynamic analysis. There will be a noticeable error especially for the structure with long period or when a local mechanism forms, then the dynamic properties of the structures changes accordingly. In this paper, a lateral load pattern is taken as the approximation of the distribution of the inertia force obtained from results of dynamic analysis of the story equivalent MDOF system of the structure, which is time dependent.

Then the static pushover analysis is used to analyze the structure step by step until the top displacement reaches the target one. It is indicated from the analysis of several frame structures that, story displacement and base shear force calculated by the presented method agree better with the results obtained from dynamic analysis than the general pushover analysis method, and it takes much less computer time than from the dynamic analysis [21].

ASHVIN G. SONI et. al on their paper studied effect of irregularities in buildings and their consequences. Many buildings in the present scenario have irregular configurations both in plan and elevation. So it is necessary to identify the performance of the structures to withstand against disaster for both new and existing one. Structures experience lateral deflections under earthquake loads.

Magnitude of these lateral deflections is related to many variables such as structural system, mass of the structure and mechanical properties of the structural materials. This is due to the irregularities in plan or elevation or in both, all multistoried buildings be analyzed as three dimensional system using IS standard. The paper discusses the performance evaluation of reinforced concrete buildings with irregularity. Structural irregularities are important factors which decrease the seismic performance of the structures. The study as a whole makes an effort to evaluate the effect of vertical irregularity on reinforced concrete buildings, in terms of dynamic characteristics and the influencing parameters which can regulate the effect on Story Displacement, Drifts of adjacent stories, Excessive Torsion, Base Shear, etc. obtained result, the base frame (regular) develops least storey drifts while the building with heavy loading on 4th and 7th stores shows maximum storey drifts on the storey levels. Hence, this is the most vulnerable to damages under this kind of loading. The buildings with irregularities also showed unsatisfactory results to some extent. The result proves that irregularities in buildings are harmful for the structures and it is important to have simpler and regular shapes of frames as well as uniform load distribution of load around the building. [22].

Monish, S., &Karuna, S on their paper studied a study on seismic performance of high rise irregular reinforced concrete framed buildings. Earthquakes are known as one of the most unpredictable and devastating of all natural disasters; however the unpredictable nature of occurrence of these earthquakes makes it difficult to prevent loss of human lives and destruction of properties, if the structures are not designed to resist such earthquake forces. In this paper attempt has been made to study two types of plan irregularities namely diaphragm discontinuity and re-entrant corners in the frame structure. These irregularities are created as per clause 7.1 of IS code. Various irregular models were considered having diaphragm discontinuity and reentrant corners which were analyzed using ETABS to determine the seismic response of the building. The models were analyzed using static and dynamic methods, parameters considered being displacement, base shear and fundamental natural period. From the present study the model which is most susceptible to failure under very severe seismic zone is found, modeling and analysis is carried out using ETABS. The result indicates irregular structural configurations are affected severely during earthquakes especially in high seismic zones and the results obtained from response spectrum method are accurate, when compared with results of equivalent static method, since the method is based only on empirical formula [23].

J.Sreenathet. al on their paper studied effect of diaphragm discontinuity in the seismic response of multi-story building. Many buildings in the present scenario have irregular configurations both in elevation and plan. It is necessary to identify the performance of the structures to withstand against disaster for both new and existing buildings.

In this study an attempt is made to try to know the difference between building with diaphragm discontinuity and a building without diaphragm discontinuity. This present paper makes a humble effort to portrait the behavior of the five different multi storied buildings with diaphragm openings under response spectrum analysis using ETABS v 9.7.4 To achieve this objective, various models with varying diaphragm openings were analyzed and compared for seismic parameters like base shear, maximum story drifts, and response spectrum results[24].

M. Moeini et. al on their paper studied investigation into the floor diaphragm flexibility in rectangular reinforced concrete buildings. Building structures are typically designed on the assumption that the floor systems serve as rigid diaphragms that span between the vertical resisting elements. Such an assumption is normally perfectly adequate for the seismic analysis of most buildings, but some structural forms, typically those comprising long, thin floor plans and perimeter lateral resisting elements, can exhibit significant in plane flexibility in their floor systems.

The dynamic behavior of this latter class of structures is dissimilar to the behavior expected of typical structures and can lead to unexpected force and drift patterns. The main purpose of this paper is to use the finite element method to investigate the influence of floor diaphragm flexibility on the behavior of concrete structures. Initially a parametric study is undertaken on a variety of reinforced concrete structures with rectangular plan form, perimeter shear walls and slabs that contain openings. In the second part of the paper, a number of response spectrum analyses are performed on the structures assuming both rigid and flexible diaphragm assumptions. A regression analysis is then performed on the results to obtain an error formula that can be used to estimate the error involved in using the rigid diaphragm assumption on structures similar to those tested. The result indicates for buildings without shear walls, the rigid-floor model is as accurate as the flexible model even for irregular floor systems. This is due to the fact that the in-plane stiffness of the slab is much larger than the out-of-plane column stiffness. For buildings with shear walls, the rigid-floor and flexible-floor analyses can differ greatly due to the very large lateral stiffness of the shear wall system. [25].

R. Pinho et. al on their paper studied modeling of the horizontal slab of a 3d irregular building for nonlinear static assessment. The manner in which the horizontal slab in 3D buildings, or better the rigid diaphragm effect, is modeled may influence the response of the structure when subjected to seismic action.

In this study different ways of modeling the rigid diaphragm effect are tested and compared, through the application to the irregular 3D SPEAR building, a full-scale specimen tested under pseudo-dynamic conditions at JRC Ispra, representing typical old three-story buildings. The dynamic properties of the different models (periods, modes of vibration and effective modal mass percentages) are analyzed, and the corresponding interaction with seismic action is studied by means of nonlinear static and dynamic analyses; different response measures, such as capacity curves, inter story drifts and displacements are evaluated for the two orthogonal directions. The results of this study show that the most accurate and reliable way of modeling the floor's behavior is the Rigid Diaphragm with Lagrange Multipliers nodal constraints model [26].

Franklin Y. Cheng & Jeffrey Ger on their paper studied performance of seismic design aids for nonlinear pushover analysis of reinforced concrete and steel bridges. Nonlinear static monotonic (pushover) analysis has become a common practice in performance-based bridge seismic design.

This study provides an overview of a newly published book "Seismic Design Aids for Nonlinear Pushover Analysis of Reinforced Concrete and Steel Bridges" by the authors of this paper, which fills the need for a complete reference on pushover analysis for practicing engineers. This technical reference provides five different nonlinear element stiffness formulation methods, ranging from the simplest to the most sophisticated, suitable for engineers at various levels of nonlinear structural analysis experience. The authors also provide a downloadable computer program, Analysis of Reinforced-Concrete and Steel Structures), that allows readers to perform their own pushover analyses [27].

M. K. Rahman et. al on their paper studied nonlinear static pushover analysis of an eight story RC frame-shear wall building. The recent seismic events have led to concerns on safety and vulnerability of RC buildings, which were designed only for gravity loads in the past devoid of any ductile detailing of joints.

This paper presents a 3D nonlinear static analysis for seismic performance evaluation of an existing eight-story reinforced concrete frame-shear wall building. The building has a dome, reinforced concrete frame, elevator shafts and ribbed and flat slab systems at different floor levels. The seismic displacement response of the RC frame-shear wall building is obtained using the 3D pushover analysis. The 3D static pushover analysis was carried out using SAP2000 incorporating inelastic material behavior for concrete and steel. Moment curvature and P-M interactions of frame members were obtained by cross sectional fiber analysis using XTRACT. The shear wall was modeled using mid-pier approach. The damage modes include a sequence of yielding and failure of members and structural levels were obtained for the target displacement expected under design earthquake and retrofitting strategies to strengthen the building were evaluated. The result indicates, pushover analysis of building show deficient to resist seismic loading. Formation of hinges clearly shows that the members of the building are designed purely for gravity loads as with a small increment of displacement, most of the members start yielding. Pushover curves show non-ductile behavior of the building, because almost all the seismic load is carried by the shear walls and at very small displacement, hinges start forming in shear walls. This indicates that strengthening of the shear walls in the building is required [28].

Pramodini NaikandSatishAnnigeri their paper studied performance evaluation of RC building. In the study first the building designed according to Indian standard and then they used static nonlinear pushover analysis to evaluate the performance of building. This study highlights the importance of carrying out the performance based design reinforced concrete building buildings and investigates the seismic performance of 9 story residential building. The building designed as per IS standard and seismic evaluation is carried out using non linear pushover analysis. The analysis for pushover analysis carried out by ETABS V9 software for both default and user defined hinge frame models. To predict the global response of building they used to these parameter; capacity curve, hinge location, ductility ratio and safety ratio

From the result of safety ratio and base shear force at a performance point it can be seen that is over safe and hence the capacity of building further can be reduced some extent to achieve economy in design. It is seen that the building model with user defined hinges more ductile. Therefore, using user defined hinges for accurate performance evaluation [29].

CHAPTER THREE: MODELING, ANALYSIS AND DESIGN OF DIFFERENT L-SHAPE BUILDINGS

3.1 General

This study aims to assess the seismic performance of common peripherally plan irregular (L-shape) building with diaphragm discontinuity. To study building structural behavior represented by six, nine, ten and twelve story building. This selected building must be design properly using capacity design principles first.

In this chapter, buildings selected for the study are described where structural members size, material properties, seismic loading parameters, capacity design principles and design detailing rules used for design of selected building models according to ES EN are introduced.

3.2 Modeling

Modeling and design of building for parametric study are created using structural design and analysis software ETABS2016.2.1. The frame section and slab size in models fulfills the minimum serviceable requirements as per the requirements of Ethiopian building code standard.

A total of Sixteen (16) building models are analyzed and designed for same ground acceleration, soil type and ductility classes for the purpose of the objectives of this research. Different floor plans for models and all data that are necessary to analyze the models are summarized and briefed below. All models selected are regular in elevation but irregular in plan.

3.3 Loads and load combinations

In this research the loads applied on the structures are dead load (self-weight, floor finishing and partition wall and external wall load), live load and seismic loads. The load combinations are created as per ES EN -1998 -2015 A total of 64 load combinations are considered for all models and the maximum action effects are used for analysis and design of the buildings. The dead loads considered in this project are the self-weight of the structure, floor finishing and partition load of 2.7kN/m2 and external wall load of 14kN/m2. The live load is taken from the structural function of the buildings and is chosen as 4kN/m2 assuming the floor is used for lecture hall.

3.3.1 Seismic load

The seismic load parameter at each zones described in ES EN 1998:1-2015.acordingle to this guideline the seismic hazard map is divided in to 5 zones, where the ratio of the design bedrock acceleration to the acceleration of gravity for the respective zone indicated below.

Table3. 1 Bed rock Acceleration Ratio ao

Zone	5	4	3	2	1	0
$a_o = \frac{ag}{g}$	0.2	0.15	0.1	0.07	0.04	0

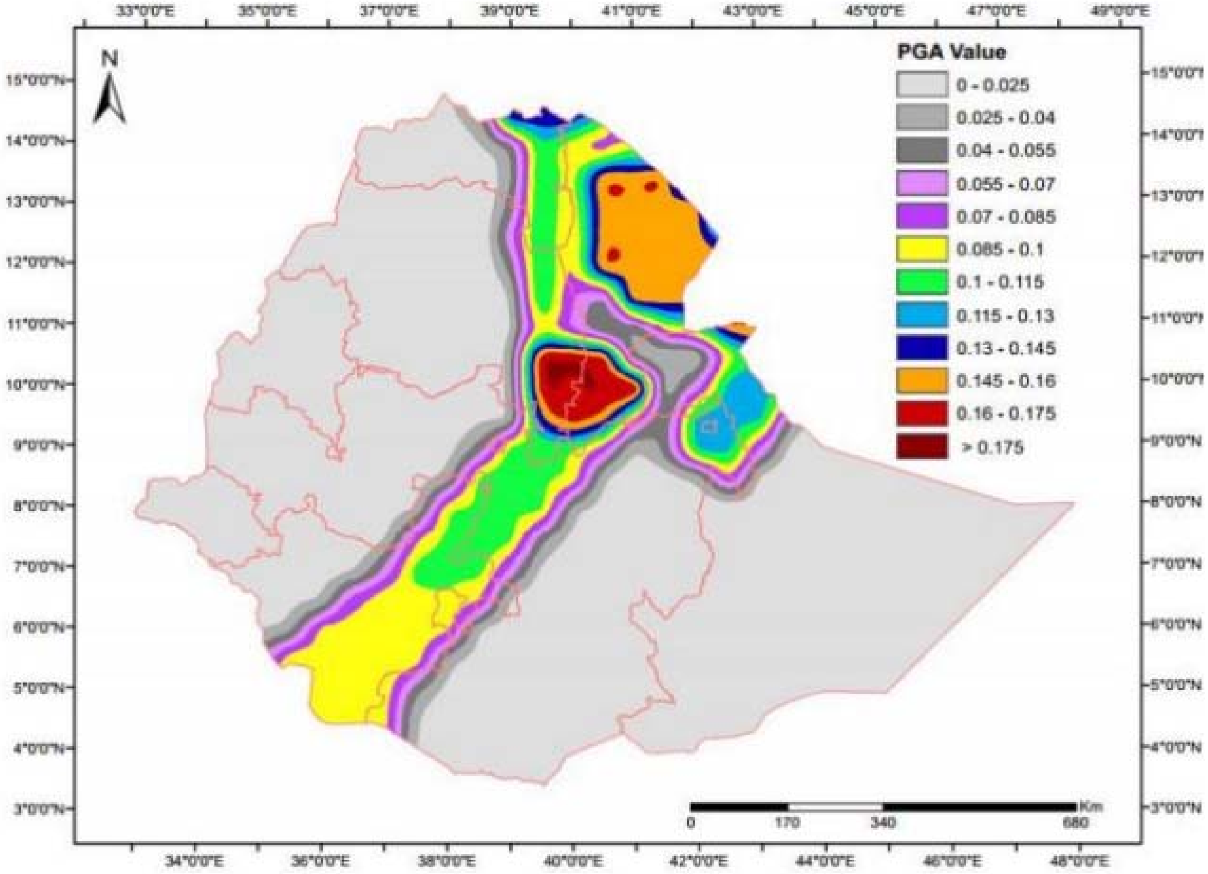


Figure3. 1 Ethiopia's Seismic hazard map in terms of peak ground acceleration

In linear analysis and design of building to find seismic design force for we use, type 1 elastic response spectrum ,also the values of parameter are shown below

Table3. 2 Values of the parameters describing the recommended Type 1 elastic response spectra

Ground type	S	T_B (s)	T_C (s)	T_D (s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.2	0.6	2.0
D	1.35	0.2	0.8	2.0
E	1.4	0.15	0.5	2.0

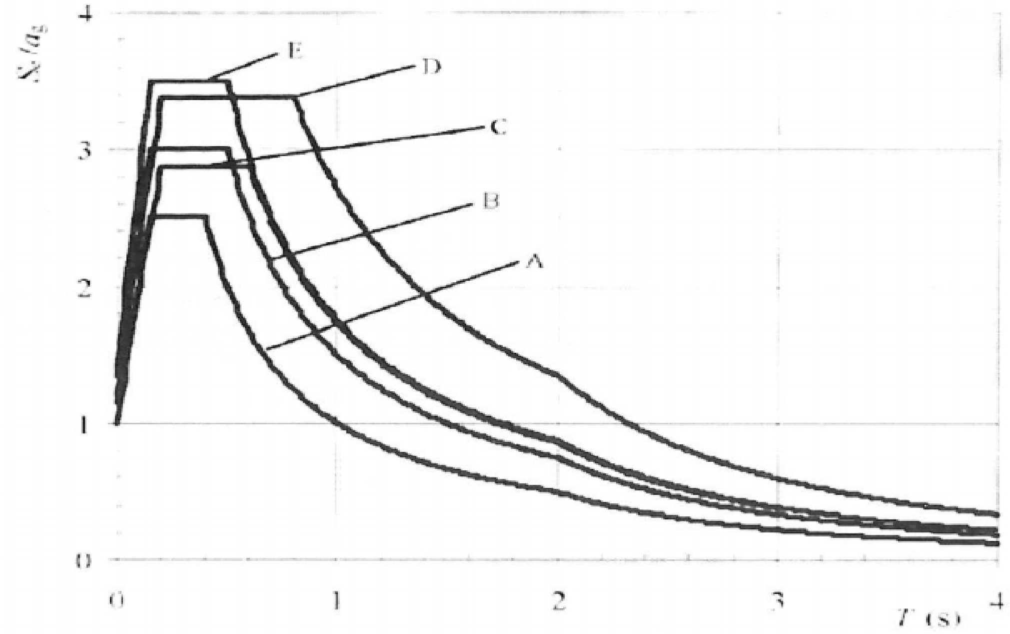


Figure3. 2: Recommended type 1 elastic re 1 elastic response spectra for ground types A to E (5% damping) [4].

3.3.2 Load combination

According to ES EN 1998-1:2015 for linear static analysis and design, the code recommended the following load combination:-

$$\mathrm{DL} + \mathrm{LL} \tag{3.1}$$

$$1.35\mathrm{DL} + 1.5\mathrm{LL} \tag{3.2}$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm \mathrm{EQX1} \pm 0.3\mathrm{EQYI} \ \pm \mathrm{IMPX}\&Y \tag{3.3}$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm \mathrm{EQX1} \pm 0.3\mathrm{EQY2} \pm \mathrm{IMPX}\&Y \tag{3.4}$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm \mathrm{EQX2} \pm 0.3\mathrm{EQY1} \pm \mathrm{IMPX}\&Y \qquad (3.5)$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm 0.3\mathrm{EQX2} \pm 0.3\mathrm{EQY2} \pm \mathrm{IMPX}\&Y \qquad (3.6)$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm 0.3\mathrm{EQX1} \pm \mathrm{EQY1} \pm \mathrm{IMPX}\&Y \qquad (3.7)$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm 0.3\mathrm{EQX1} \pm \mathrm{EQY2} \pm \mathrm{IMPX}\&Y \qquad (3.8)$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm 0.3\mathrm{EQX2} \pm \mathrm{EQY1} \pm \mathrm{IMPX}\&Y \qquad (3.9)$$

$$\mathrm{DL} + 0.3\mathrm{LL} \pm 0.3\mathrm{EQX2} \pm \mathrm{EQY2} \pm \mathrm{IMPX}\&Y \qquad (3.10)$$

3.4 Accounting imperfection in design of building

Euro code 2 and Euro code 3 requires frame and member imperfections (global and local) be accounted for either in the analysis stage or in the design stage.

Accounting global and local imperfections in the analysis stage of a numerical model can be done by either:

- Modeling the global and/or local imperfections explicitly in the model
- Applying horizontal uniformly distributed load along vertical member length for local imperfection only applicable for Euro code three
- Applying Equivalent Horizontal Force at each story for global imperfection.
- Using the buckling mode shapes as initial geometry only applicable for Euro code three

Accounting global and/or local imperfections in the analysis stage by modeling the frames and members straight or crooked, although can be done, will be impractical and time consuming. Similarly, applying horizontal uniformly distributed load on vertical members to account for local imperfection is only practical when checking the effect on a single vertical element. These methods will not be discussed here.

In Euro code two and three, global imperfections can be accounted for in the design stage as eccentricity on isolated vertical members. Local imperfections in euro code three are accounted for in the design stage by using the relevant buckling shape factors in member buckling check.

3.4.1 Geometric imperfections

ES EN 1992-1-1:2015.The unfavorable effects of possible deviations in the geometry of the structure and the position of loads shall be taken into account in the analysis of members and structures. Imperfections may be represented by an inclination, θ_l, given by:

$$\theta_i = \theta_0 \; \alpha_h \; \alpha_m \tag{3.11}$$

Where;

θ_0= is the basic value given as 0.005 (= 1/200)

α_h is the reduction factor for length or height:

$$\alpha_h = 2/\sqrt{l}; 2/3 \leq \alpha_h \leq 1 \tag{3.12}$$

l is the length or height [m]

α_m is the reduction factor for number of members:

$$\alpha_m = \sqrt{0.5(1+1/m)} \tag{3.13}$$

m is the number of vertical members contributing to the total effect

Horizontal load due to geometric imperfection applied on center of diaphragm and the loads in both principal directions calculated as:

$$H_i = \theta_i (N_b + N_a)/2 \tag{3.14}$$

where

N_a and N_b are longitudinal forces that contributing to lateral force.

H_i horizontal load due to geometric imperfection in both principal direction that applied on center of diaphragm.

3.5 Behavior factor for horizontal seismic action

The purpose defining behavior factor, to avoid explicit inelastic structural analysis in design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements and/or other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one, henceforth called a "design spectrum". This reduction is accomplished by introducing the behavior factor q. The value of behavior factor define by the following equation

$$q = q_o \, k_w \tag{3.15}$$

Where;

q_o=is the basic value of the behavior factor, dependent on the type of the structural system and on its regularity in elevation

kw= is the factor reflecting the prevailing failure mode in structural systems with walls.

Concrete buildings designed in accordance with are classified in two ductility classes DCM (medium ductility) and DCH (high ductility), depending on their hysteretic dissipation capacity.

Both classes correspond to buildings designed, dimensioned and detailed in accordance with specific earthquake resistant provisions, enabling the structure to develop stable mechanisms associated with large dissipation of hysteretic energy under repeated reversed loading, without suffering brittle failures.

Table3. 3 Basic value of behavior factor, qo, for regular in elevation

Structural type	DCM	DCH
Frame system, dual system, coupled wall system	$3.0\alpha_u/\alpha_1$	$4.5\alpha_u/\alpha_1$
Uncoupled wall system	3.0	$4.0\alpha_u/\alpha_1$
Torsion ally flexible system	2.0	3.0
Inverted pendulum system	1.5	2.0

But in this research studies the lateral force resisting mechanism of models used frame system and plan irregular. For buildings which are not regular in plan the approximate value of α_u/α_1 that may be used when calculations are not performed for its evaluation are equal to the average of 1.0 and 1.3 therefore for ductility class medium α_u/α_1=1.15, behavior factor, q=3*1.15=3.45(ES EN 1998:1-2015)[4].

3.6 Description of parametric study

Parametric studies carried out for different number of story, which is believed to verify response of a structure with diaphragm discontinuity/opening at different location of slab. Depending on the above parametric study, the following responses of structures are shown;

- Maximum lateral displacement both x and y direction
- Base shear in both x and y direction using pushover analysis

The structures were analyzed by assumptions; existing diaphragms are semi-rigid and analysis were carried out by taking different representative models to compare the response using advance structural software (ETABS), further more will discuss in detail below.

3.7 Description of building structure

In order to achieve the aim of this study; three different story level reinforced concrete frame building and beam bounded reinforced concrete solid slab, six, nine, ten and twelve story buildings are chosen. All selected buildings are same in story height, space between column in all direction, slab type and thickness, beam and place of construction, grade of concrete and rebar, loading condition and service and floor height. These buildings are chosen intentionally so that the selected case study building can provide a critical structural layout for evaluating diaphragm discontinuity/opening for actual diaphragm stiffness assumption.

For present study purpose, the structural system used for these buildings is taken as concrete moment-resisting space frames, and the soil type is considered as class A.

The ductility class of the building is taken as medium (DCM). Furthermore, the design bed rock acceleration (a_g) has been taken as 0.15g which corresponds to seismic zone (Zone IV**)** [4].

Detail structural and material description of building structures for this research are summarized in Table 3.4-3.6 below, which consists building code, number of stories, opening location and projection parts length relative main parts length in percent.

To verify the effect of diaphragm discontinuity 16 structures are chosen and analyzed as discussed in the Table below.

Table3. 4 Structures for parametric study-number of story as parametric study

Code	No of stories	No spans to x-direction	No spans to y-direction	Bays width both x and y direction(m)	Length of projection parts relative to main parts in (percentage)	Opening location	Building description
M5	6	4=main building 2=projection	11=main building but 4 span projection part parallel to main building	4	20	Angle	six- story with opening down
M5*	6	4=main building 2=projection	11=main building, 4 span projection part parallel to main building	4	20	No opening	six- story without opening down
M6	9	4=main building 2=projection	11=main building, 4 span projection part parallel to main building	4	20	Angle	nine- story with opening down
M6*	9	4=main building 2=projection	11=main building, 4 span projection part parallel to main building	4	20	No opening	Nine- story without opening down
M7	12	4=main building 2=projection	11=main building, 4 span projection part parallel to main building	4	20	Angle	twelve -story with opening down
M7*	12	4=main building 2=projection	11=main building, 4 span projection part parallel to main building	4	20	No opening	twelve -story without opening down

Table3. 5 Structures for parametric study-opening location varied

Code	No of stories	No spans to x direction	No spans to y direction	Bays width both x and y direction(m)	Length of projection parts relative to main parts in (percentage)	Opening location	Building description
M1	10	4=main building 4=projection part	12=main building but 4 spans projection part parallel to main building	4	41.7	No opening	ten -story without opening down
M2	10	4=main building 4=projection part	12=main building but 4 spans projection part parallel to main building	4	41.7	Angle	ten -story with opening down
M3	10	4=main building 4=projection part	12=main building but 4 spans projection part parallel to main building	4	41.7	Main part center	ten -story with opening down
M4	10	4=main building 4=projection part	12=main building but 4 spans projection part parallel to main building	4	41.7	Projected part center	ten -story with opening down

Table3. 6 Structures for parametric study-percentage of projection in constant open down at angle.

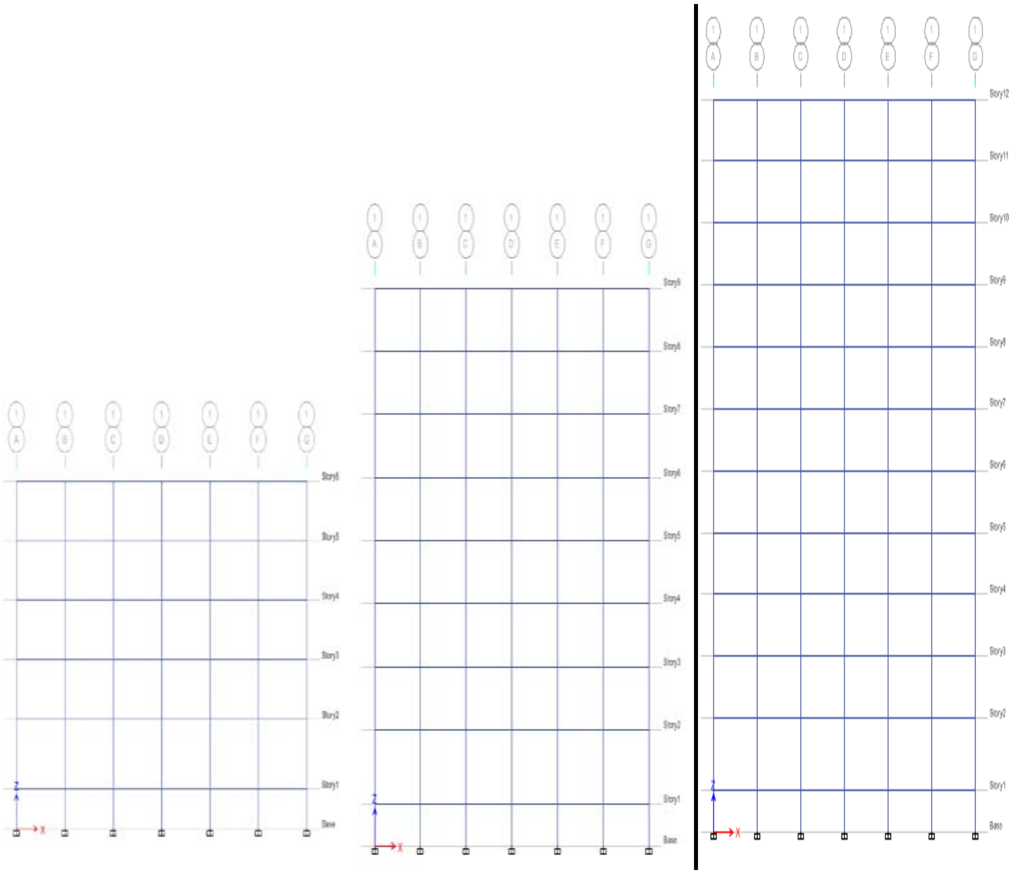
Code	No of stories	No spans to x-direction	No spans to y-direction	Bays width both x and y direction(m)	Length of projection parts relative to main parts in (percentage)	Opening location	Building description
M8*	10	4=main building 1=projection part	10=main building but 4 spans projection part parallel to main building	4	10	No opening	ten -story without opening down,10% projection
M8	10	4=main building 1=projection part	10=main building but 4 spans projection part parallel to main building	4	10	Angle	ten -story with opening down,10% projection
M9*	10	4=main building 2=projection part	10=main building but 4 spans projection part parallel to main building	4	20	No opening	ten -story without opening down,20% projection
M9	10	4=main building 2=projection part	10=main building but 4 spans projection part parallel to main building	4	20	Angle	ten -story with opening down,20% projection
M10 *	10	4=main building	10=main building but 4 spans projection part	4	30	No opening	ten -story without opening

		3=projection part	parallel to main building				down,30% projection
M10	10	4=main building 3=projection part	10=main building but 4 spans projection part parallel to main building	4	30	Angle	ten -story with opening down,30% projection

To evaluate the response of selected building structure lots of design parameters are considered in ETABS software. In any software providing wrong data or missing single parameter gives wrong response. In this regard, careful considerations made in the input information where the basic design parameters are given in Table 3.1-3.3. Also sample drawing of 2D and floor plan view of each building structure are shown Figure 3.3-3.5.

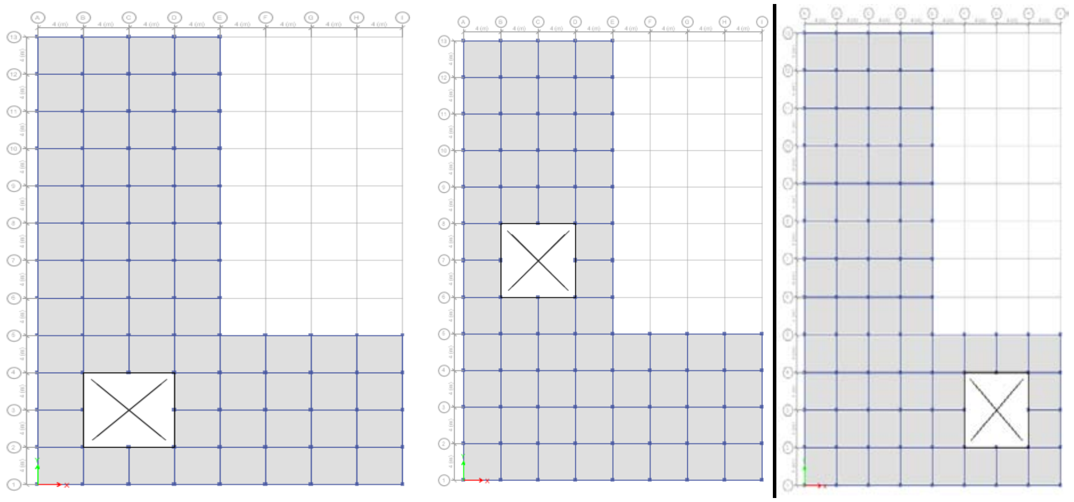
Table3. 7 Details of selected building structure

Building parameters	Details
Location where building construct	Debre Berhan
Usage	lecture halls
Concrete	C25/30Mpa for structural members
Reinforcement bar	S400Mpa for longitudinal and S300 for confinement rebar
Seismic zone	IV
Slab thickness	150mm
Number of story	6,9,10 and 12
Wall load(line load in all beam)	14kN/m
Floor finish and part ion load	2.7kN/m2
Density of concrete	24kN/m3
Damping of structure	5 percent
Poisson ratio	0.2
Structural system of building	Moment resisting reinforced concrete frame fixed at base
Size of diaphragm discontinuity	Constant size for all model
Geometry of building	Plan irregular
Ductility class	Medium(DCM)
Importance class of building	Class (III) and value of 1.2.
Soil class	A



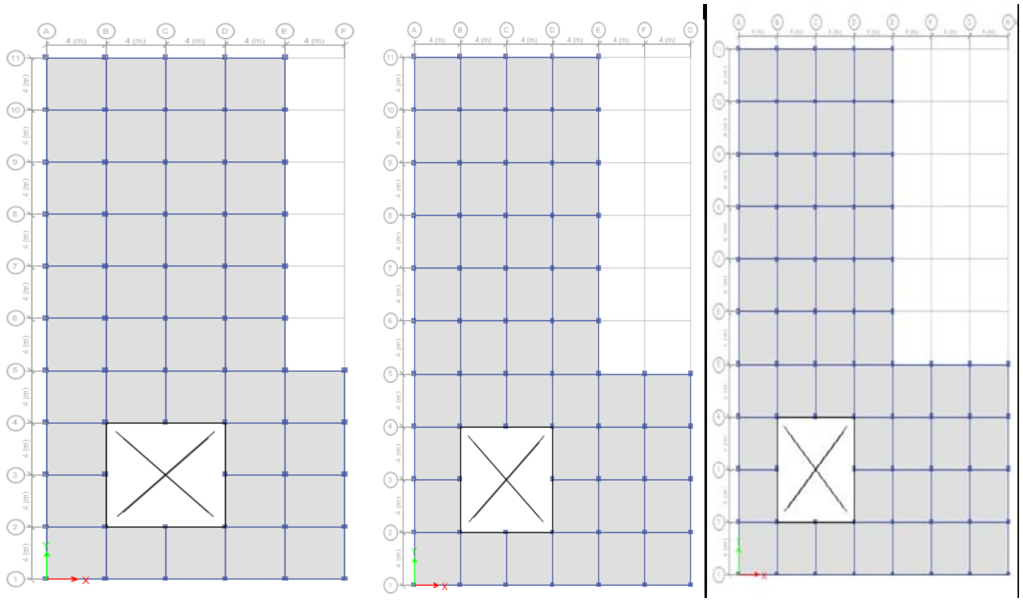
(a)Model M5 (b) Model M6 (c) Model M7

Figure3. 3 Elevation view of the models- number of story as parametric study



(a)Model M2 (b) Model M3 (c) Model M4

Figure3. 4 Plan view models-opening location as parametric study



(a)Model M8 (b) Model M9 (c) Model M10

Figure3. 5 Plan view of the models- projection length as parametric study

3.7.1 Material properties

Normal-weight concrete (24kN/m^3) with a characteristic cube compressive strength of 30MPa and cylindrical strength of 25Mpa and yield strength of 400MPa for longitudinal reinforcement and300MPa for shear reinforcement are used.

3.7.2 Vertical and horizontal element system

Vertical and horizontal structural elements used for the parametric study is reinforced concrete frame system without any shear wall and bracing, which consists of reinforced concrete column as vertical element and beam as horizontal elements. In this research rectangular shape of structural elements (column and beam) are used.

3.7.3 Loading

Uniform live load of 4kN/m^2 (assumed building service for lecture halls) and partition wall load and floor finishing dead load of 2.7kN/m^2,wall load on beam(line dead load) 14kN/m, ,and also permanent dead load of the structure is computed using the software using unit weight of concrete**.**

3.8 Analysis verification and 2^{nd} order effect consideration

On design of structures subjected to seismic action according ES EN: 1998-2015 is intended to ensure these objectives:

1). safeguarding human lives;

2). limit structural damage;

3). important structures for civil protection remain operational.

In this context, it is necessary that structures are designed for non-occurrence of local or global collapse to a seismic level with small probability of occurrence, designated as design seismic action, and is controlled the damage level in the same structure for a seismic action with minor intensity and higher probability of occurrence than the design seismic action. The damage should not correspond to disproportionately high costs when compared to the structure itself. These two requirements are accomplished,

1). Ultimate limit states: The no-collapse requirement (ultimate limit state) under the seismic design situation is considered to have been met if the following conditions regarding resistance, ductility, equilibrium, foundation stability and seismic joints are met.

A) Resistance condition

The following relation shall be satisfied for all structural elements including connections and the relevant non-structural elements:

$$E_d = R_d \tag{3.16}$$

E_d= is the design value of the action effect, due to the seismic design situation including, if necessary, second order effects.

$R_{d=}$ is the corresponding design resistance of the element, calculated in accordance with the rules specific to the material used (in terms of the characteristic values of material properties fyk and partial factor γm) and in accordance with the mechanical models which relate to the specific type of structural system.

B) Second-order effects (P-Δ effects)

Second-order effects (P-Δ effects) need not be taken into account if the following condition is fulfilled in all stores:

$$\theta = \frac{\mathrm{Ptot.dr}}{\mathrm{Vtot.h}} \qquad (3.17)$$

θ= is the inter-story drift sensitivity coefficient;

P_{tot}= is the total gravity load at and above the story considered in the seismic design situation;

dr= is the design inter-story drift, evaluated as the difference of the average lateral displacements ds at the top and bottom of the story under consideration

V_{tot}=is the total seismic story shear; and

h= is the inter story height.

If $0.1 < \Theta \leq 0.2$, the second-order effects may approximately be taken into account by multiplying the relevant seismic action effects by a factor equal to 1/(1 - θ). The value of the coefficient θ shall not exceed 0.3.

2. Damage limitation: The damage limitation requirement is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action corresponding to the no-collapse requirement: in accordance with Limitation of inter story drift, if the drift limits satisfied in the following.

a) For buildings having non-structural elements of brittle materials attached to the structure:

$$\mathrm{dr}\,\nu \leq 0.005\mathrm{h} \qquad (3.18)$$

b) For buildings having ductile non-structural elements:

$$\mathrm{dr}\,\nu \leq 0.0075\mathrm{h} \qquad (3.19)$$

c) For buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$$\mathrm{dr}\,\nu \ \leq 0.010\mathrm{h} \qquad (3.20)$$

Where

dr= is the design inter story drift

h= is the story height;

ν = is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement. The value of the reduction factor ν may also depend on the importance class of the building. Implicit in its use is the assumption that the elastic response spectrum of the seismic action under which the “damage limitation requirement” should be met. The values to be ascribed to ν for use in a country may be found in its National Annex. I n Ethiopia their commended values of ν are 0.4 for importance classes III and IV and ν = 0.5 for importance classes I and II [4].

3.9 Parametric studies of cases

Analysis result and set discussion on selected 16 representative structures that are categorized in three parametric studies are present hereafter.

Table3. 8 List of parameters in each three case

Cases	Parametric study
Case-1	Number of stories as a parameter
Case-2	Percentage length of projection relative to main parts of building
Case-3	Opening location

3.10 Analysis of building

After modeling every building structures and loading expected load performing preliminary design carried using equivalent static analysis method for all models. After performing equivalent linear static analysis, the corresponding ultimate and damage limit state requirements of ES EN 1998:1-2015 code are checked and respected. The tasked performed summarized presented in the Table below.

The size / stiffness of the elements was obtained by a process of iteration as the demand of the ductility classes for each model and optimized sections are obtained keeping the satisfaction of the limits provided in the design codes. In all cases, the sections are selected to satisfy all the capacity design requirements. All of the following design information's are checked to in the required limit like longitudinal reinforcement, shear reinforcement, beam column capacity, tensional reinforcement, , shear failure to have safe structure to resist loads.

Table3. 9 Analysis output and drifts calculation for DCM, six stories building without diaphragm discontinuity

STORY	P(kN)	V(KN)	AVG dr	θ	dr*v	0.005h	Check	
							ULS	SLS
STORY6	14465.75	-1683.72	21.244	-0.06084	8.4976	15	OK	OK
STORY5	29574.63	-3109.14	15.301	-0.04852	6.1204	15	OK	OK
STORY4	44683.5	-4239.65	14.789	-0.05196	5.9156	15	OK	OK
STORY3	59779.25	-5078.09	10.611	-0.04164	4.2444	15	OK	OK
STORY2	74964.55	-5634.38	6.575	-0.02499	2.63	17.5	OK	OK
STORY1	84504.43	-5754.36	3.14	-0.02306	1.256	10	OK	OK

Table3. 10 Analysis output and drifts calculation for DCM, nine stories building without diaphragm discontinuity

STORY	P(kN)	V(KN)	AVG dr	θ	dr*v	0.005h	Check	
							ULS	SLS
STORY9	14465.75	-1311.25	30.374	-0.1117	12.1496	15	OK	OK
STORY8	29574.63	-2499.35	26.306	-0.10376	10.5224	15	OK	OK
STORY7	44683.5	-3535.77	17.377	-0.0732	6.9508	15	OK	OK
STORY6	59792.38	-4420.52	14.943	-0.06737	5.9772	15	OK	OK
STORY5	74897.65	-5154.24	13.576	-0.06576	5.4304	15	OK	OK
STORY4	90022.13	-5739.39	11.889	-0.06216	4.7556	15	OK	OK
STORY3	105273.4	-6179.9	11.287	-0.06409	4.5148	15	OK	OK
STORY2	120960.9	-6476.82	10.734	-0.05728	4.2936	17.5	OK	OK
STORY1	131176.1	-6542.02	2.498	-0.02504	0.9992	10	OK	OK

Table3. 11 Analysis output and drifts calculation for DCM, twelve story building diaphragm discontinuity without opening

STORY	P(kN)	V(KN)	AVG dr	Θ	dr*v	0.005h	Check	
							ULS	SLS
STORY12	14465.75	-1196.56	35.587	-0.14341	14.2348	15	OK	OK
STORY11	29574.63	-2133.97	30.97	-0.14307	12.388	15	OK	OK
STORY10	44683.5	-2212.09	28.027	-0.18871	11.2108	15	OK	OK
STORY9	59792.38	-2361.92	26.955	-0.22746	10.782	15	OK	OK
STORY8	74897.95	-2683.65	26.149	-0.24326	10.4596	15	OK	OK
STORY7	75019.61	-2778.15	25.092	-0.22586	10.0368	15	OK	OK
STORY6	95285.61	-2846.18	23.63	-0.2637	9.452	15	OK	OK
STORY5	100779.9	-2887.41	22.272	-0.25912	8.9088	15	OK	OK
STORY4	126503.7	-3100.24	21.92	-0.29815	8.768	15	OK	OK
STORY3	142191.2	-3284.97	20.294	-0.29281	8.1176	15	OK	OK
STORY2	168419.9	-3292.58	11.802	-0.17248	4.7208	17.5	OK	OK
STORY1	179395.1	-3355.47	2.057	-0.05499	0.8228	10	OK	OK

Table3. 12 output and drifts calculation for DCM, ten stories building without opening

STORY	P(kN)	V(KN)	AVG dr	Θ	dr*v	0.005h	Check	
							ULS	SLS
STORY10	18678.68	-1359.89	31.708	-0.14517	12.6832	15	OK	OK
STORY9	36291.88	-2610.49	29.779	-0.138	11.9116	15	OK	OK
STORY8	54882.78	-3716.92	26.36	-0.12974	10.544	15	OK	OK
STORY7	73473.67	-4679.92	20.285	-0.10616	8.114	15	OK	OK
STORY6	92058.71	-5500.69	21.856	-0.12193	8.7424	15	OK	OK
STORY5	110696.1	-6181.51	21.419	-0.12785	8.5676	15	OK	OK
STORY4	129468.7	-6725	18.386	-0.11799	7.3544	15	OK	OK
STORY3	148611.7	-7128.99	17.667	-0.12276	7.0668	15	OK	OK
STORY2	168054	-7387.85	19.644	-0.12767	7.8576	17.5	OK	OK
STORY1	179945.8	-7438.9	3.664	-0.04432	1.4656	10	OK	OK

Table3. 13 : Analysis output and drifts calculation for DCM, ten story building 10% projection without opening

STORY	P(kN)	V(KN)	AVG dr	θ	dr*v	0.005h	Check	
							ULS	SLS
STORY10	12281.45	-945.991	34.293	-0.1484	13.7172	15	OK	OK
STORY9	25206.03	-1816.25	30.529	-0.14123	12.2116	15	OK	OK
STORY8	38130.6	-2586.65	29.785	-0.14636	11.914	15	OK	OK
STORY7	51055.18	-3257.18	28.504	-0.14893	11.4016	15	OK	OK
STORY6	63977.05	-3828.37	24.376	-0.13578	9.7504	15	OK	OK
STORY5	76915.71	-4302.73	22.121	-0.13181	8.8484	15	OK	OK
STORY4	89990.81	-4684.76	21.571	-0.13812	8.6284	15	OK	OK
STORY3	103538.4	-4968.89	20.4	-0.14169	8.16	15	OK	OK
STORY2	117086	-5149.59	19.828	-0.12881	7.9312	17.5	OK	OK
STORY1	125404.6	-5185.39	2.408	-0.02912	0.9632	10	OK	OK

Table3. 14 Analysis output and drifts calculation for DCM, ten story building 20%projection without opening

STORY	P(kN)	V(KN)	AVG dr	θ	dr*v	0.005h	Check	
							ULS	SLS
STORY10	14070.32	-1228.3	36.585	-0.1397	14.634	15	OK	OK
STORY9	28140.64	-2075.1	33.657	-0.15214	13.4628	15	OK	OK
STORY8	42210.96	-2812.999	32.199	-0.16106	12.8796	15	OK	OK
STORY7	56281.28	-3542.171	30.389	-0.16095	12.1556	15	OK	OK
STORY6	70384.39	-4163.394	29.402	-0.16569	11.7608	15	OK	OK
STORY5	84651.19	-4679.291	22.427	-0.13524	8.9708	15	OK	OK
STORY4	99396.26	-5094.67	16.289	-0.10593	6.5156	15	OK	OK
STORY3	114141.3	-5603.453	11.189	-0.07597	4.4756	15	OK	OK
STORY2	129089.5	-5673.12	5.95	-0.03868	2.38	17.5	OK	OK
STORY1	137350	-5785.89	0.714	-0.00847	0.2856	10	OK	OK

Table3. 15 Analysis output and drifts calculation for DCM, ten story building 30%projection without opening

STORY	P(kN)	Vx(KN)	AVG dr	θ	dr*v	0.005h	Check	
							ULS	SLS
STORY10	14465.75	-1113.66	31.684	-0.13718	12.6736	15	OK	OK
STORY9	29681.81	-2137.94	25.743	-0.11913	10.2972	15	OK	OK
STORY8	44897.88	-3044.67	19.337	-0.09505	7.7348	15	OK	OK
STORY7	60113.94	-3833.86	17.279	-0.09031	6.9116	15	OK	OK
STORY6	75326.7	-4506.17	16.363	-0.09118	6.5452	15	OK	OK
STORY5	90560.71	-5064.46	15.605	-0.09301	6.242	15	OK	OK
STORY4	105952	-5513.98	12.977	-0.08312	5.1908	15	OK	OK
STORY3	121894.6	-5848.24	10.685	-0.07424	4.274	15	OK	OK
STORY2	137837.1	-6060.81	9.545	-0.06202	3.818	17.5	OK	OK
STORY1	147597.3	-6102.81	3.655	-0.0442	1.462	10	OK	OK

3.11 Capacity design

(As per ES EN 1998-1:2015)In field of earthquake engineering design capacity used as a design tools, because of nonlinear response and energy absorption ability greater than 1.5. Building structure subjected to strong earthquake force and use linear analysis with reduced the seismic design force require to use capacity design rule.

The main energy dissipating structural elements of building in lateral force resisting system are deign and detailing according to the capacity design principles to ensure elastic response of lateral deformation.

To achieve capacity design, apply the following capacity design rules:

1.Brittle failure or other undesirable failure mechanisms (e.g. concentration of plastic hinges in columns of a single story of a multistory building, shear failure of structural elements, failure of beam-column joints, yielding of foundations or of any element intended to remain elastic) shall be prevented, by deriving the design action effects of selected regions from equilibrium conditions, assuming that plastic hinges with their possible over strengths have been formed in their adjacent areas.

2.The primary seismic columns of frame or frame-equivalent concrete structures should satisfy sum of the design values of the moments of resistance of the columns framing the joint greater than or equal to 1.3 times the sum of the design values of the moments of resistance of the beams framing the joint with the following exemptions.

a) In plane frames with at least four columns of about the same cross-sectional size, it is not necessary to satisfy expression above in all columns, but just in three out of every four columns.

b) At the bottom story of two-story buildings, if the value of the normalized axial loads νd does not exceed 0.3 in any column.

3. Slab reinforcement parallel to the beam and within the effective flange width specified, should be assumed to contribute to the beam flexural capacities taken into account for the calculation of ∑MRb, if it is anchored beyond the beam section at the face of the joint.

Capacity design rules guide to have weak beam with strong column, beam sway mechanism, involves plastic hinging at all beams ends either plastic hinging at column bottom.

3.11.1 Design action effects

The analysis result value of bending moment and axial force and shear force are obtained from linear static analysis of selected building structure by exposing seismic effect. These design values for primary seismic beam and column are determined as follows

1. Beam

In primary seismic beams the design shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam under:

a) The transverse load acting on it in the seismic design situation and

b) End moments Mi, d (with i=1, 2 denoting the end sections of the beam), corresponding to plastic hinge formation for positive and negative directions of seismic loading.

The plastic hinges should be taken to form at the ends of the beams or (if they form their first) in the vertical elements connected to the joints into which the beam ends frame. End moments Mi,d may be determined as follows:

$$M_{\mathrm{i,d}} = \gamma_{\mathrm{Rd}} M_{\mathrm{Rb,i}} \min\left(1, \frac{\sum M_{\mathrm{Rc}}}{\sum M_{\mathrm{Rb}}}\right) \qquad (3.21)$$

where

γ_{Rd} is the factor accounting for possible over strength due to steel strain hardening, which in the case of DCM beams may be taken as being equal to 1.0;

M_{Rb},i is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action

;ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively. The value of ΣM_{Rc} should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

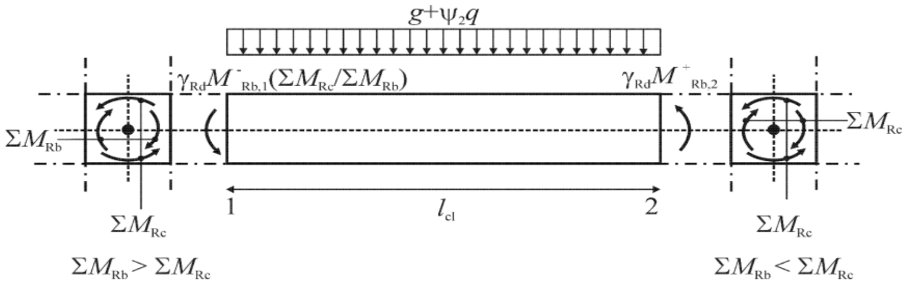


Figure3. 6 : Capacity design values of shear forces on beams [4].

2. Columns

In primary seismic columns the design values of shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments Mi,d (with i=1,2 denoting the end sections of the column), corresponding to plastic hinge formation for positive and negative directions of seismic loading. The plastic hinges should be taken to form at the ends of the beams connected to the joints into which the column end frames, or (if they form their first) in the columns. End moments Mi,d may be determined from the following expression:

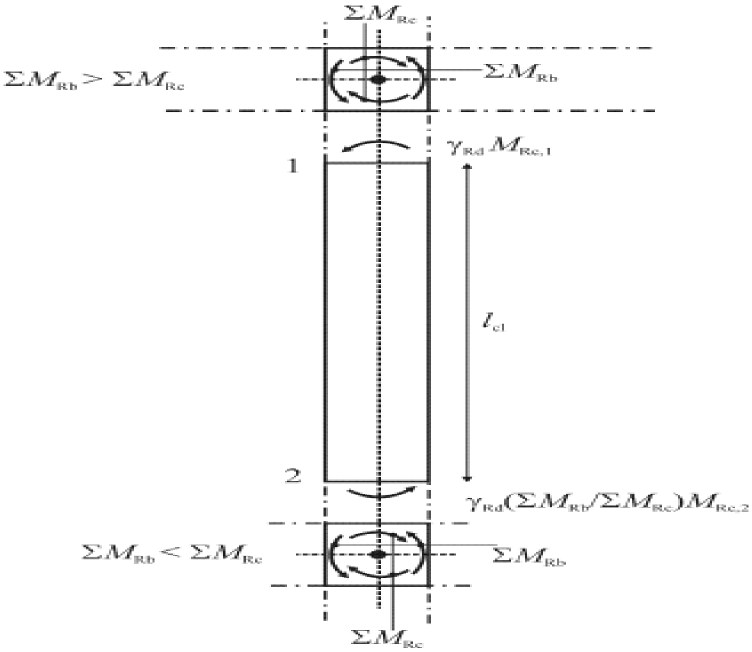


Figure3. 7 Capacity designs shear force in columns [4].

$$M_{\mathrm{i,d}} = \gamma_{\mathrm{Rd}} M_{\mathrm{Rc,i}} \min\left(1, \frac{\sum M_{\mathrm{Rb}}}{\sum M_{\mathrm{Rc}}}\right) \tag{3.22}$$

where

γ_{Rd} is the factor accounting for over strength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1.1;

M_{Rci} is the design value of the column moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;ΣMRc and ΣMR bare as defined in above.

3.11.2 Global and local ductility condition

To ensure local and global ductility of structure the Ethiopian building code stated as follows

1. It shall be verified that both the structural elements and the structure as a whole possess adequate ductility, taking into account the expected exploitation of ductility, which depends on the selected system and the behavior factor.

2. Specific material related requirements shall be satisfied, including, when indicated, capacity design provisions in order to obtain the hierarchy of resistance of the various structural components necessary for ensuring the intended configuration of plastic hinges and for avoiding brittle failure modes.

3. In multi-story buildings formation of a soft story plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft story.

4. In frame buildings, including frame-equivalent ones with two or more stores, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns: Expression below should be satisfied in two orthogonal vertical planes of bending, which, in buildings with frames arranged in two orthogonal directions, are defined by these two directions. It should be satisfied for both directions (positive and negative) of action of the beam moments around the joint, with the column moments always opposing the beam moments. If the structural system is a frame or equivalent to a frame in only one of the two main horizontal directions of the structural system, then expression below should be satisfied just within the vertical plane through that direction.

$$\sum \mathrm{MRc} \geq 1.3 \sum \mathrm{MRb} \tag{3.23}$$

Where

$\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint. The minimum value of column moments of resistance within the range of column axial forces produced by the seismic design situation should be used in expression (3.20);

$\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint. When partial strength connections are used, the moments of resistance of these connections are taken into account in the calculation of $\sum M_{Rb}$.

To achieve the local ductility the Ethiopian building code(ES EN1998-1:2015) specified the minimum and maximum limits so every design must be guide specified parameter to ensure the design locally ductile for DCM beam and column as follows:

A) Beam

1. The minimum width of beam b_w of a primary seismic beam shall satisfy the following expression:**.**

$$b_w \leq \min \{b_c + h_w; 2b_c\} \tag{3.24}$$

Where

h_w is the depth of the beam and b_c is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam.

2. The regions of a primary seismic beam up to a distance $l_{cr} = h_w$ (where h_w denotes the depth of the beam) from an end cross-section where the beam frames into a beam column joint, as well as from both sides of any other cross-section liable to yield in the seismic design situation, shall be considered as being critical regions.

3. The reinforcement ratio of the tension zone ρ does not exceed a value ρ_{max} equal to:

$$\rho_{\max} = \rho' + \frac{0.0018}{\mu_{\varphi}\varepsilon_{\mathrm{sy,d}}} \cdot \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd}}} \tag{3.25}$$

With the reinforcement ratios of the tension zone and compression zone, ρ and ρ', both normalized to bd, where b is the width of the compression flange of the beam. $\varepsilon_{sy,d}$ is the design value of tension steel strain at yield; If the tension zone includes a slab, the amount of slab reinforcement parallel to the beam within the effective flange width

4. Along the entire length of a primary seismic beam, the reinforcement ratio of the tension zone, ρ, shall be not less than the following minimum value ρ_{min}:

$$\rho_{min} = 0.5\left(\frac{f_{ctm}}{f_{yk}}\right) \tag{3.26}$$

5. At the compression zone reinforcement of not less than half of the reinforcement provided at the tension zone is placed, in addition to any compression reinforcement needed for the ULS verification of the beam in the seismic design situation.

6. In the critical regions of primary seismic beams, diameter d_{bw} of the hoops (in millimeters) shall be not less than 6. The spacing, s, of hoops (in millimeters) shall not exceed

$$s = \min\{h_w/4;\ 24d_{bw};\ 225;\ 8d_{bL}\} \tag{3.27}$$

Where

D_{bL} is the minimum longitudinal bar diameter (in millimeters); and h_W the beam depth (in millimeters).

7. The first hoop shall be placed not more than 50 mm from the beam end section

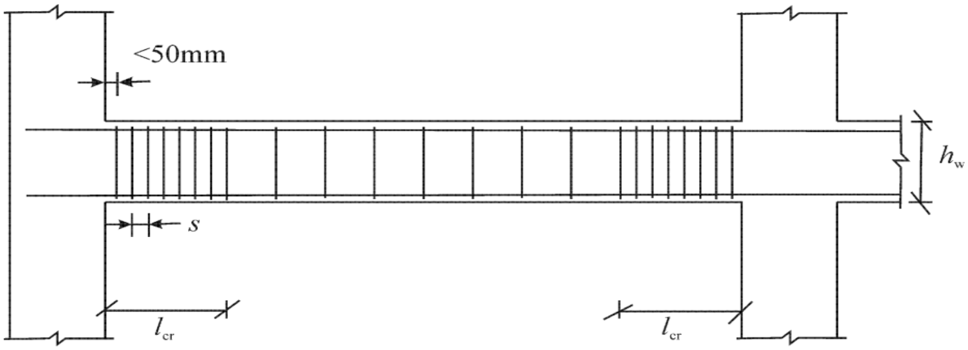


Figure3. 8 Transverse reinforcement in critical regions of beams [4].

B) Column

1.minimum width of column ,Unless θ≤ 0,1 means θ≥0.1 the cross -sectional dimensions of primary seismic columns should not be smaller than one tenth of the larger distance between the point of contra flexure and the ends of the column, for bending within a plane parallel to the column dimension considered.

2. The total longitudinal reinforcement ratio ρ shall be not less than 0,01 and not more than 0,04. In symmetrical cross-sections symmetrical reinforcement should be provided ($\rho = \rho'$).

3. The regions up to a distance l_{cr} from both end sections of a primary seismic column shall be considered as being critical regions.

4. The length of the critical region lcr (in meters) may be computed from the following expression:

$$l_{cr} = \max\left\{h_c\ ;\ l_{cl} / 6;\ 0.45\right\} \tag{3.28}$$

Where h_c is the largest cross-sectional dimension of the column (in meters); and l_{cl} is the clear length of the column (in meters).

5. Diameter d_{bw} of the hoops (in millimeters) shall be not less than 6. The spacing, s, of the hoops (in millimeters) in column does not exceed:

$$s = \min\{b_o/2;\ 175;\ 8d_{bL}\} \tag{3.29}$$

Where

b_o (in millimeters) is the minimum dimension of the concrete core (to the centerline of the hoops); and d_{bL} is the minimum diameter of the longitudinal bars (in millimeters).

6. A minimum value of ω_{wd} equal to 0.08 should be provided within the critical region at the base of the primary seismic columns.

7. If for the specified value of $\mu\varphi$ a concrete strain larger than ε_{cu2}=0,0035 is needed anywhere in the cross-section, compensation for the loss of resistance due to spelling of the concrete shall be achieved by means of adequate confinement of the concrete core, on the basis of the properties of confined concrete

8. In primary seismic columns the value of the normalized axial force ν_d shall not exceed 0.65.

9. At least one intermediate bar shall be provided between corner bars along each column side, to ensure the integrity of the beam-column joints.

10. The regions up to a distance l_{cr} from both end sections of a primary seismic column shall be considered as being critical regions [4].

3.12 Design outputs

In the design of building sample the followings have been checked and kept in the required limit as per Ethiopian design code.

1. For ultimate limit state verification, inter story drift has been checked

2. Damage limitation, damage limitation requirement has been checked.

3. In frame fulfillments of strong column/weak beam capacity design rule, with over strength factor greater than 1.3 has been checked.

4. Axial force ratio of base column has been checked, value in DCM less than 0.65.

5. Design parameters maximum and minimum requirements have been checked.

CHAPTER FOUR: NON-LINEAR ANALYSIS AND EVALUATION BUILDING PERFORMANCE

4.1 General

This study aims to assess the seismic performance of common peripherally plan irregular (L-shape) building with diaphragm discontinuity. To assess the effect of diaphragm discontinuity represented by six, nine, ten and twelve story respectively. This selected building seismic performance will be evaluated by considering reinforced elements. In this research structural irregularity occur due to plan and diaphragm discontinuity.

4.2 Modeling

4.2.1 General

Modeling of considered building in ETABS2016 is done using nonlinear static (pushover) analysis. Currently Nonlinear static pushover analysis has become commonly used method to determine the nonlinear behavior structure using structural software using this method, capacity curve is obtained which shows the relation between base shear and roof displacement. Pushover analysis is numerically demanding and may cause numerical difficulties for all software used to run the analysis if the selected building is complex. Therefore, to complete analysis and reduce running time attempts is made by selecting less complex structure. To find more approximate result, any linear elements should be modeled with least possible amount of meshing. Hinges are assigned at location where nonlinear behavior expected.

The basic step of analysis is determination of primary and secondary structural elements. Primary structural elements should be completely modeled in the entire analysis but secondary elements which do not significantly contribute to the buildings lateral force resisting system, are not included to the analysis. In building modal elements that yield much earlier compared to the rest, may be numerically difficult to run. Therefore, due to weakness of these elements or not to be the main components of the lateral load resisting system to carry the loads along may be modeled as pin-end.

4.2.2 Steps in pushover analysis

The following steps are included in pushover analysis these are creating modal of structure, run the analysis and review analysis results.

Create basic computer structural modal of building as usual manner (without pushover data) using ETABS graphic interface so it is easy and quick task among other steps.

1. Define all properties and acceptance criteria for the pushover hinges.
2. Locate the pushover hinges on a given model by selecting one or more frame members and assign them one or more hinge properties and hinge considering end-offsets from supports.
3. Define pushover load case based on type loads imposed on modeled structure these loads may be both gravity and lateral loads or one of the two.
4. Run the basic static analysis then run static nonlinear pushover analysis.
5. Then display pushover curve.
6. Display capacity spectrum curve
7. Immediately seeing the new capacity spectrum plot, we can interactively modify the magnitude of earth quake and damping ratio. The performance point value of analyzed structure is defined by the intersection of capacity curve and single demand spectrum curve.
8. Review the pushover displaced shape and sequence of hinge formation on structural elements on step-by-step.
9. And also review member forces on a step-by-step basis.
10. Print the output of pushover analysis in tabular forms for the entire modal or for selected structural element of building. These outputs include joint displacement at each step of pushover, frame member force at each step of pushover,+ and hinge force, displacement and state at each step of the pushover analysis[4].

4.3 Nonlinear behavior of structural elements

The nonlinear behavior of a building structure depends on the nonlinear responses of the elements that are used in the lateral force resisting system. Therefore, before applying any nonlinear analysis method on a building structure, the nonlinear behavior of such elements must be clearly described and evaluated. ATC-40 and FEMA-356 codes define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 3.2, the five points (A, B, C, D and E) which are used to define the hinge rotation behavior of reinforced concrete members and the acceptance criteria on a force versus deformation diagram are given. In this diagram, points marked as IO, LS and CP represent immediate occupancy, Life safety and Collapse prevention, respectively.

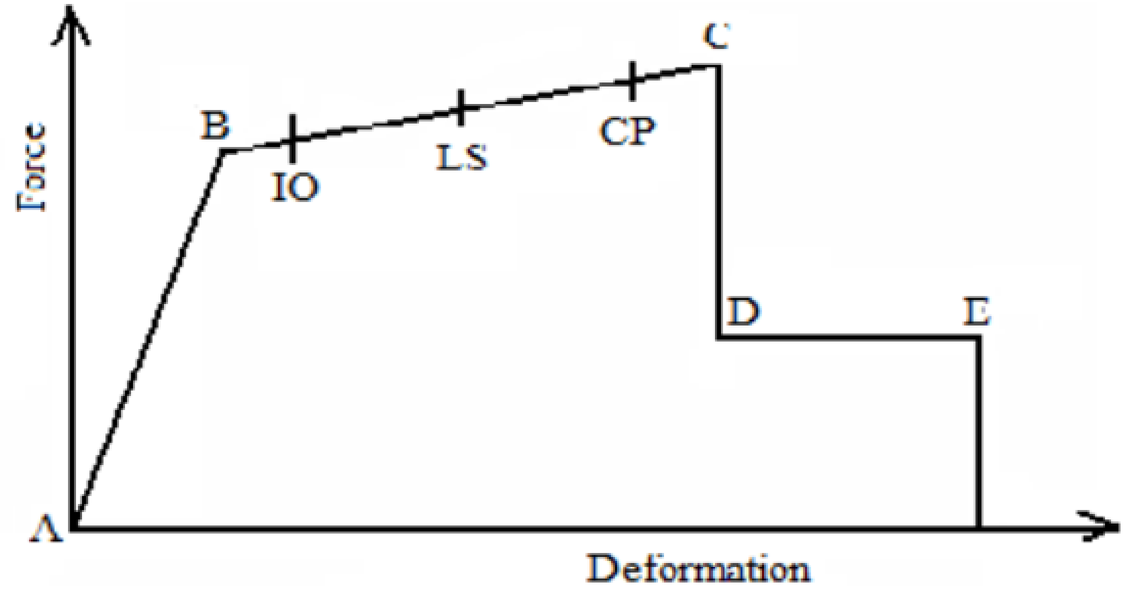


Figure4. 1 Acceptance criteria on a force versus deformation diagram [30]

The load-deformation relation is defined by linear response (or elastic response) until point B. At point B, the member yields and again a linear response is observed with a reduced stiffness between the points B and C. At point C, a sudden reduction in the load resistance of the element occurs and the graph drops to point D. The residual resistance is observed until point E, where the final loss of resistance takes place. The initial slope of this diagram between points A and B defines the elastic stiffness of the structure. In the analyses carried out in this study, the second slope between points B and C is taken as 10% of the initial slope. Point C in this diagram represents the ultimate strength of the element where the significant stiffness degradation begins.

The above-mentioned nonlinear response of the structural member is called hinge property which is defined symmetrically in order to include the reversals to the calculations. For the modeling of nonlinear response of an element, ATC-40 and FEMA-356 express the parameters A, B and C in Figure 4.1 by defining plastic rotation angles [31]

4.4 Verification of ETABS (using default and user defined hinges) for non-linear analysis using experimental Results.

The frame specimen frame was developed and tested by Vecchio and Emara (1992). The frame was flexure-critical with well-confined cross sections; span-depth ratios of beams were larger at 8.75 and higher column axial loads were applied.

Three members were used to create sectional models of the beam, column, and base members. The frame was loaded to a lateral displacement of 155 mm and then unloaded to a net lateral load of zero [32]. The 2D frame geometry, loading, cross-sectional detail of beam and column of frame used in the experiment is shown in figure 4.2 below.

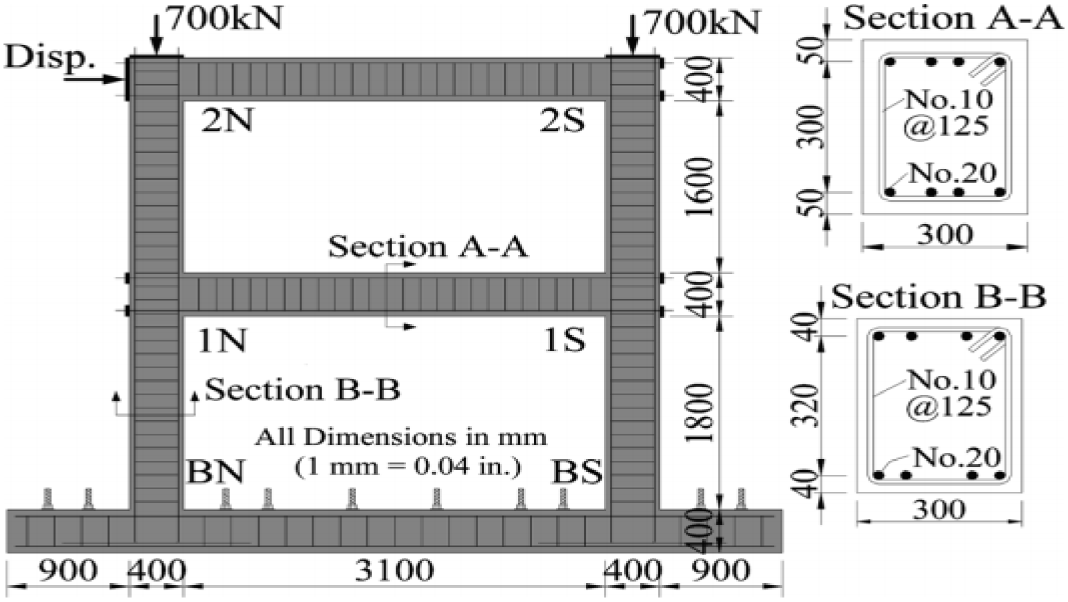


Figure4. 2 Structural details of Vecchio and Emara(1992)frame.

Table 4. 1: Parameters in ground column used for moment-curvature generation

Axis	Longitudinal Rebar	Stirrup	Axial force(kN)	Section(mm)
GCOL	8Φ20	Φ8c/c150mm	-974.6	300x400
FCOL	4Φ20	Φ8c/c200mm	-959.4	300x400

Table 4. 2: Parameters in ground column used for moment-curvature generation-section capacity

Id	Section(mm)	My	Ky	Mu	Ku
GCOL	300x400	230.3	0.0000129	240	0.000025
FCOL	300x400	192.8	0.0000125	199	0.0000245

Table 4. 3:hinge property data for column at plastic hinge point

Id	Lp	B		C		D		E	
	0.5h	My	θy	Mu	θu	0.2my	θu	0.2my	2θu
GCOL	0.2	230.3	2.58E-06	240	0.000005	46.06	0.000005	46.06	0.00001
FCOL	0.2	192.8	0.0000025	199	0.0000049	38.56	0.0000049	38.56	9.8E-06

Table 4. 4: Scale factor for rotation and moment for column

Id	A		B		C		D		E	
	M	Θ	m	θ	M	θ	m	θ	M	2θ
GCOL	0	0	1	0	1.042118975	0.00000242	0.2	0.00000242	0.2	0.00000484
FCOL	0	0	1	0	1.032157676	0.0000024	0.2	0.0000024	0.2	0.0000048

Table 4. 5:Parameters in beam used for moment-curvature generation

Axis	Story	Joint	section(mm)	Longitudinal rebar		Stirrups
				Top	Bottom	
axis 1	FB2	4	300X400	3Φ16	2Φ16	Φ8 C/C 100
		3	300X400	2Φ16	3Φ16	Φ8 C/C 100
axis 1	FB1	2	300X400	5Φ16	2Φ16	Φ8 C/C 100
		1	300X400	2Φ16	5Φ16	Φ8 C/C 100

Table 4. 6: Parameters in beam used for moment-curvature generation-section capacity

Id	Section(mm)	My	Ky	Mu	Ku
FB1,L	300x400	115.3	8.5E–06	130.4	0.000081
FB1,R	300x400	50.73	7.3E–06	60.11	0.00011
FB2,L	300x400	69.7	7.69E–06	85.68	0.000102
FB2.R	300x400	50.98	7.28E-06	60.99	0.0001103

Table 4. 7:hinge property data for beam at plastic hinge point

Id	lp=0.5h	B		C		D		E	
	0.5h	my	θy	Mu	θu	0.2my	θu	0.2my	2θu
FB1,1	0.2	115.3	1.7E–06	130.4	0.0000162	23.06	0.0000162	23.06	3.24E-05
FB1,2	0.2	50.73	1.46E–06	60.11	0.000022	10.146	0.000022	10.146	0.000044
FB2,3	0.2	69.7	1.54E–06	85.68	0.0000204	13.94	0.0000204	13.94	4.08E-05
FB2.4	0.2	50.98	1.46E-06	60.99	2.206E-05	10.196	0.00002206	10.196	4.41E-05

Table 4. 8:Scale factor for rotation and moment for beam

Id	A		B		C		D		E	
	M	Θ	m	θ	M	θ	m	θ	M	2θ
FB1,1	0	0	1	0	1.130962706	0.0000145	0.2	0.0000145	0.2	0.000029
FB1,2	0	0	1	0	1.184900453	0.00002054	0.2	0.00002054	0.2	0.00004108
FB2,3	0	0	1	0	1.229268293	0.000018862	0.2	0.000018862	0.2	0.000037724
FB2.4	0	0	1	0	1.19635151	0.000020604	0.2	0.000020604	0.2	0.000041208

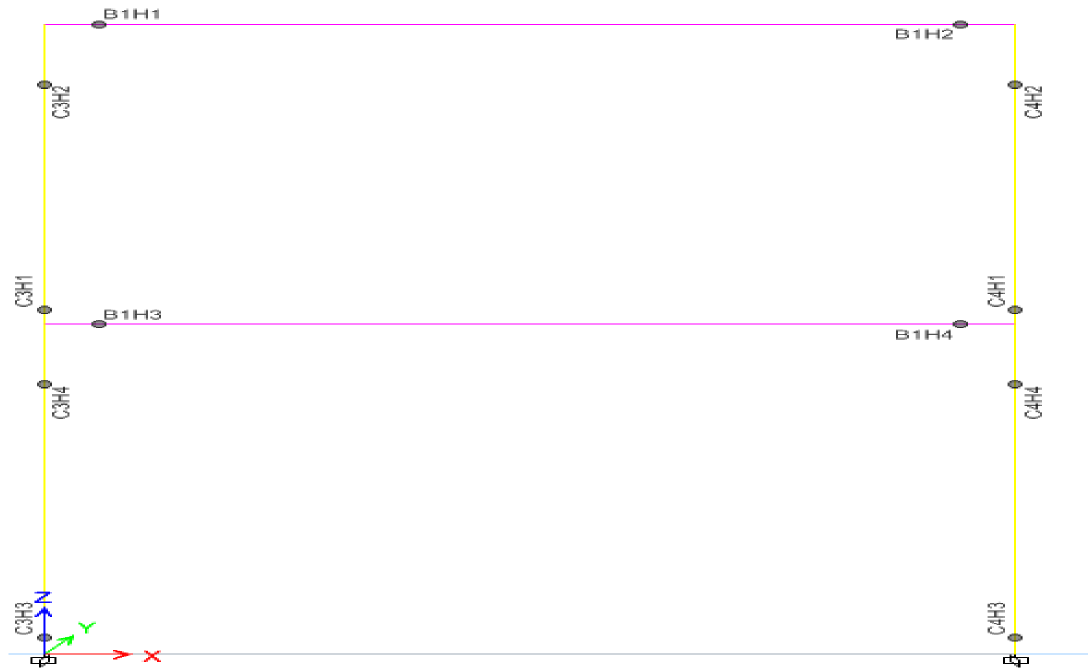


Figure4. 3: Default hinge properties

Figure4. 4:User defined hinge property

4.4.1. Moment Curvature Data of Frame Section that was done using SAP2000 Section Designer.

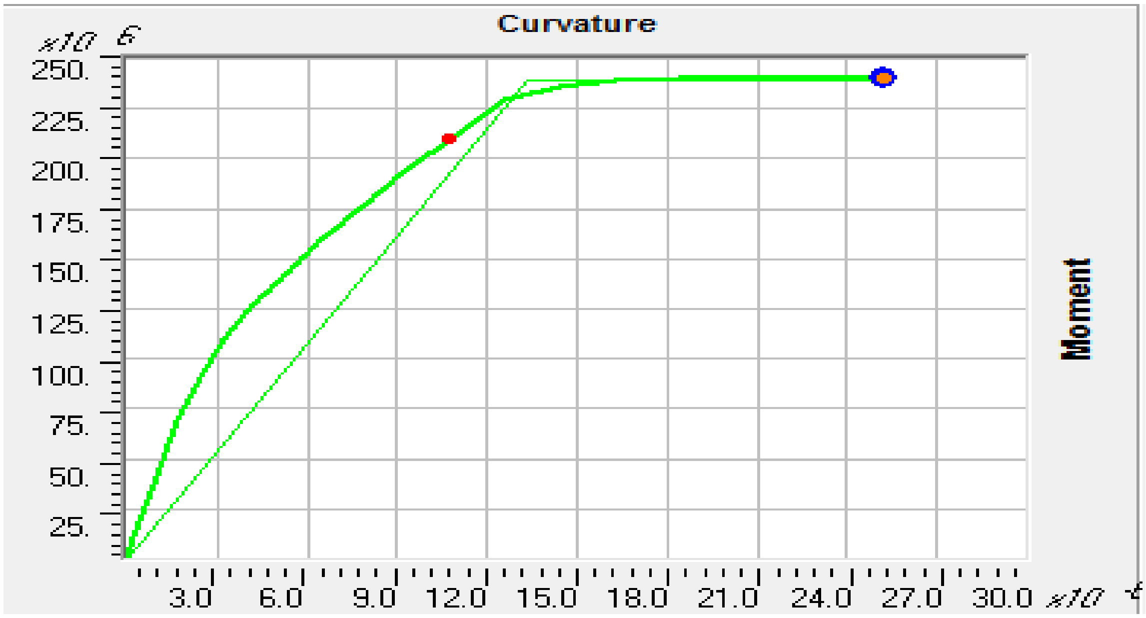


Figure4. 5:Ground column moment curvature

Table4. 9 Pushover base shear and top story displacement using user defined and default hinge

Default hinge results	
Displacement (mm)	Base shear force (kN)
0.001	0
20.001	220.8215
22.348	246.7359
27.566	277.4908
36.328	293.6149
56.328	302.7594
58.785	303.9494
42.617	99.7399

User defined hinge results	
Displacement (mm)	Base shear force (kN)
0.001	0
17.717	195.6004
32.363	274.5366
41.319	286.6282
61.319	287.0819
81.319	287.5744
101.319	288.0767
121.319	288.5822
130.469	288.8141
114.009	54.4551

Experimental	
Displacement(mm)	Base Shear(kN)
0	0
5	50
10	100
15	150
20	200
25	250
40	300
60	305
75	305
100	305
125	300
146	295
140	210

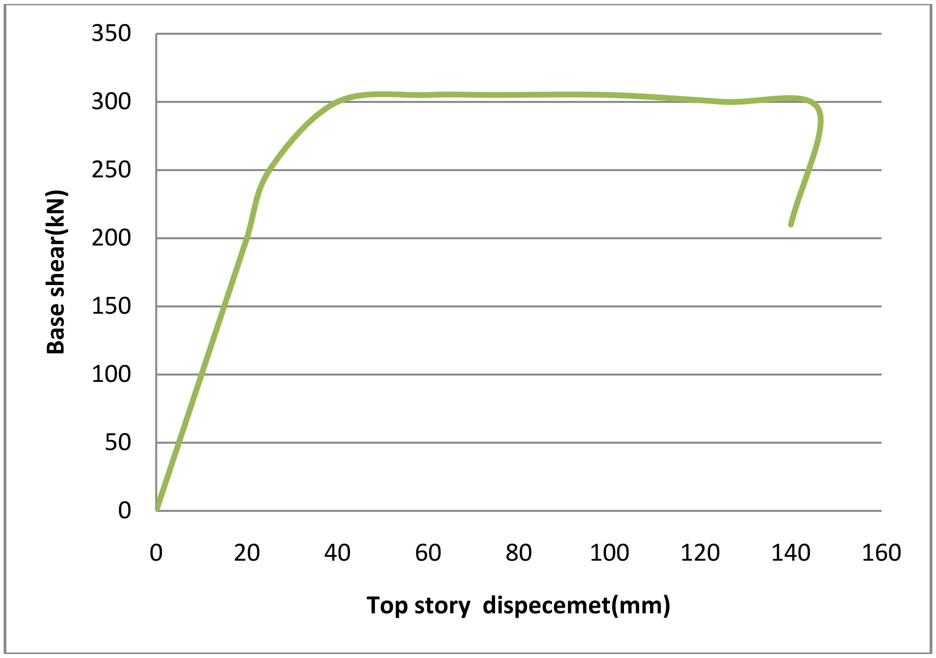


Figure4. 6: pushover curve for experimental

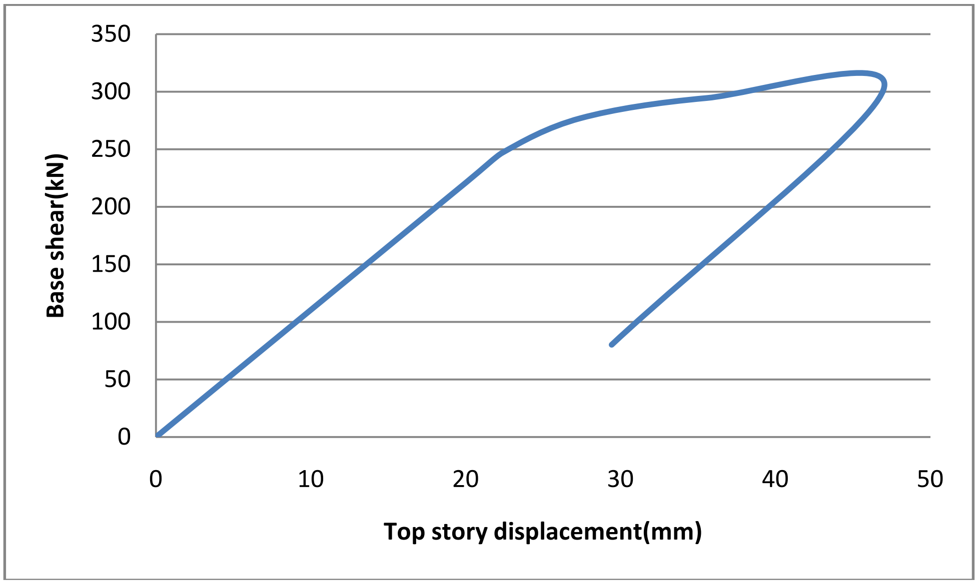


Figure4. 7:Capacity curve using default hinge

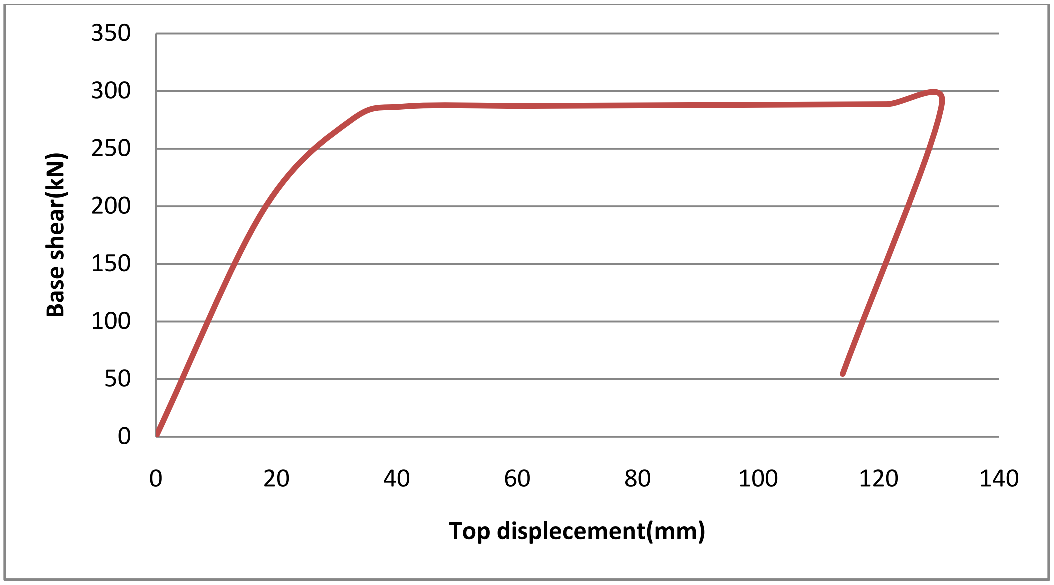


Figure4. 8:Capacity curve using user define

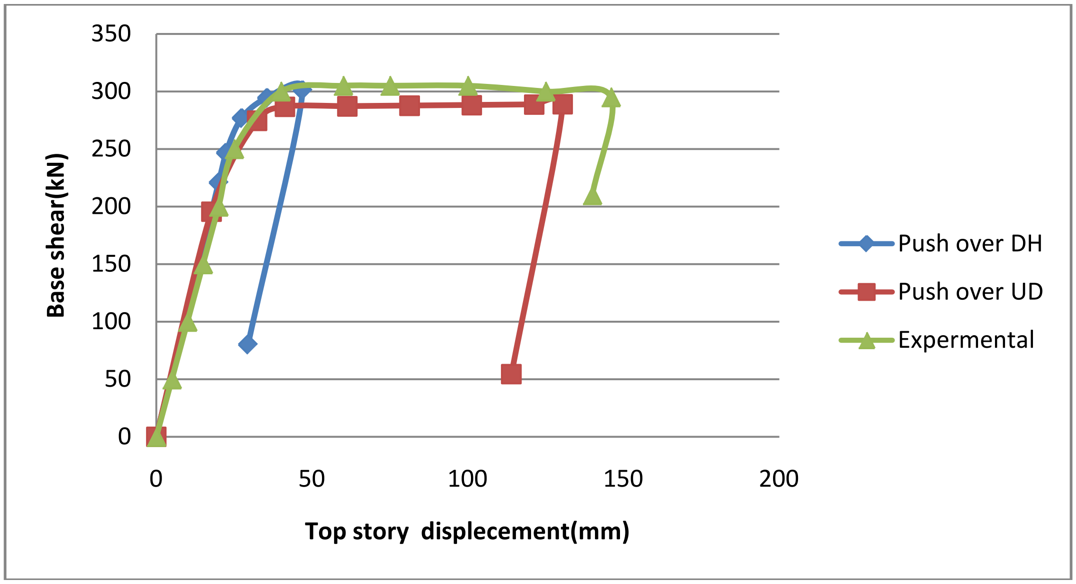


Figure4. 9:Capacity curve comparison of Experimental, user defined and defaults hinge

4.4.2. Result and Conclusion

As we can see the results from the comparison curve in Figure 4.9 the experimental and user defined hinge properties analytical results difference is very small. But the use of default hinge properties the output has great difference to experimental and user defined hinge properties of analytical output. based on the above results when default hinges used in performance evaluation of structures, the deformation capacity of structure is undermined as compare to experimental and user defined hinges properties analytical output.

The result difference using user defined and default hinges may be default hinge properties used analytical analysis not considering the effect of confinement and concrete cover properly. But for this thesis default hinge assignment for all models applied because applying the same method has no effect on conclusion.

4.5 Parameters use to evaluate the models

The analysis gives different output to express global and local behavior of model after analysis. The analysis result may generated in tabular or graphic form among the result generated from pushover analysis are story maximum displacements, base shear of pushover and ratio base shear at performance point in pushover analysis to design base in linear static analysis. They are chosen as the basic performance indicator and requirements to compare the response of buildings.4.6 Results

4.6.1 Opening location

The first parametric study is carried out by considering the variation opening location as a parameter and making the other constant.

4.6.1.1 Story maximum displacements

ETABS2016 provides a simple table in the summary output with story maximum displacements. This provides indication of maximum to check displaced distance. The maximum displacements due to push-X in X-direction and push-y in y-direction are:

Table 4. 10:Maximum story displacement(mm)due to push x

Story	M1	M2	M3	M4
Story10	354	346	344	342
Story9	341	330	330	339
Story8	329	313	309	316
Story7	293	283	284	284
Story6	252	242	242	244
Story5	208	193	194	200
Story4	159	150	148	151
Story3	112	103	99	102
Story2	61	49	46	50
Story1	20	16	14	17
Base	0	0	0	0

Table 4. 11:Maximum story displacement(mm)due to push y

Story	M1	M2	M3	M4
Story10	414	434	410	238
Story9	400	422	396	231
Story8	382	401	378	218
Story7	348	368	346	196
Story6	307	327	302	167
Story5	254	277	256	132
Story4	204	222	207	114
Story3	151	168	154	68
Story2	95	111	102	35
Story1	15	27	26	8
Base	0	0	0	0

4.6.1.2 Story shear

ETABS2016 provides a simple table in the summary output with story shear. This provides indication of maximum shear that developed at each floor. The maximum story shear due to push-X in X-direction and push-y in y-direction are:

Table 4. 12 Story shear in X (kN) due to push-X

Story	M1	M2	M3	M4
Story10	1760	1584	1600	1580
Story9	3876	3626	3652	3622
Story8	5990	5666	5704	5656
Story7	8102	7701	7756	7694

Story6	10212	9744	9811	9732
Story5	12346	11796	11876	11784
Story4	14500	13877	13973	13862
Story3	16692	15996	16104	15978
Story2	18920	18142	18264	18127
Story1	20234	19433	19568	19414
Base	20321	19918	19654	19502

Table 4. 13 Story shear in X (kN) due to push-y

Story	M1	M2	M3	M4
Story10	2174	2204	2208	1638
Story9	4516	4522	4410	3258
Story8	6850	6782	6611	4877
Story7	9884	9042	8813	6496
Story6	11524	11303	11016	8116
Story5	13877	13577	13236	9745
Story4	16262	15884	15484	11398
Story3	18684	18228	17770	13076
Story2	21144	20612	20094	14784
Story1	22604	22042	21488	15812
Base	22692	22135	21576	15875

4.6.2 Number of stories as a parameter

The second parametric study is carried out by considering the variation of number of stories as a parameter and making the other constant.

4.6.2.1 Story maximum displacements

ETABS 2016 provides a simple table in the summary output with story maximum displacements. This provides indication of maximum check displaced distance. The maximum displacements due to push-X in X-direction and push-y in y-direction are:

Table 4. 14 Story displacement in X (mm)

Story	M5*	M5
Story6	169	166
Story5	159	154.164
Story4	140	135.912
Story3	112	107.422
Story2	76	72.3
Story1	26	23.857
Base	0	0

Table4.14 Story displacement in Y (mm)

Story	M5*	M5
Story6	106	104
Story5	99	99
Story4	88	84
Story3	62	62
Story2	37	37
Story1	8	8
Base	0	0

Table 4. 16 Story displacement in X (mm)

Story	M6*	M6
Story9	316	309
Story8	305	297
Story7	285	276
Story6	245	247
Story5	216	208
Story4	169	160
Story3	119	111
Story2	68	61
Story1	33	29
Base	0	0

Table 4. 15 Story displacement in Y (mm)

Story	M6*	M6
Story9	354	310
Story8	341	744
Story7	322	239
Story6	293	206
Story5	252	175
Story4	203	138
Story3	151	100
Story2	99	63
Story1	32	19
Base	0	0

Table 4. 17 Story displacement in X (mm)

Story	M7*	M7
Story12	442	432
Story11	433	420
Story10	412	402
Story9	384	380
Story8	346	360
Story7	302	301
Story6	258	251
Story5	214	207
Story4	167	161
Story3	120	107
Story2	61	59
Story1	12	9
Base	0	0

Table 4. 18 Story displacement Y(mm)

Story	M7*	M7
Story12	497	396
Story11	485	391
Story10	466	381
Story9	435	356
Story8	398	326
Story7	353	287
Story6	307	250
Story5	260	211
Story4	211	171
Story3	154	121
Story2	97	79
Story1	27	26
Base	0	0

4.6.2.2 Story Shear

ETABS2016 provides a simple table in the summary output with story shear. This provides indication of maximum shear that developed at each floor. The maximum story shear due to push-X in X-direction and push-y in y-direction are:

Table 4. 19 Story shear in push X (kN)

Story	M5*	M5
Story6	2446.583	2315.799
Story5	4946.865	4684.422
Story4	7447.148	7053.045
Story3	9955.961	9430.05
Story2	12528.56	11869.86
Story1	14055.45	13346.87
Base	14056.85	13348.24

Table 4. 20 story shear in push Y (kN)

Story	M5*	M5
Story6	2088.026	2027.37
Story5	4221.882	4100.986
Story4	6355.738	6174.603
Story3	8496.875	8255.556
Story2	10692.45	10391.49
Story1	11995.57	11684.54
Base	11996.76	11685.74

Table 4. 21 Story shear in push X (kN)

Story	M6*	M6
Story9	2115	1983
Story8	4724	4422
Story7	7291	4870
Story6	9856	9325
Story5	12426	11753
Story4	15007	14212
Story3	17637	16716
Story2	20376	19331
Story1	22032	20944
Base	22139	21044

Table 4. 22 Story shear in push Y (kN)

Story	M6*	M6
Story9	2592	2116
Story8	5341	4220
Story7	8096	6324
Story6	10849	8431
Story5	13602	10537
Story4	16374	12658
Story3	19192	14814
Story2	22031	17070
Story1	22910	18457
Base	23026	19549

Table 4. 23 Story shear in push X (kN)

Story	M7*	M7
Story12	1939	1767
Story11	4278	4002
Story10	6618	6234
Story9	8959	8473
Story8	11300	10709
Story7	13654	12954
Story6	16038	15236
Story5	18454	17544
Story4	20893	19873
Story3	23366	22239
Story2	25968	24732
Story1	27566	26297
Base	27610	26430

Table 4. 24 Story shear in push Y (kN)

Story	M7*	M7
Story12	2472	2093
Story11	5006	4168
Story10	7543	6240
Story9	10077	8319
Story8	12616	10393
Story7	15168	12488
Story6	17749	14592
Story5	20368	16742
Story4	23000	18909
Story3	25688	21108
Story2	28504	23423
Story1	31012	23875
Base	31489	24998

4.6.3 Percentage of projection relative to main parts of building length.

The third parametric study is carried out by considering the variation percentage of projection length as a parameter and making the other constant.

4.6.3.1 Story maximum displacements

ETABS2016 provides a simple table in the summary output with story maximum displacements. This provides indication of maximum to average ratio to check displaced distance. The maximum displacements due to push-X in X-direction and push-y in y-direction are

Table 4. 25 Story displacement in X(mm)

Story	M8*	M9*	M10*	M8	M9	M10
Story10	394	392	400	388	386	394
Story9	380	381	385	375	374	376
Story8	359	356	362	350	351	351
Story7	324	325	328	320	317	319
Story6	282	282	285	275	274	274
Story5	231	234	234	222	224	224
Story4	182	183	182	171	172	170
Story3	132	133	131	123	122	122
Story2	77	76	78	66	65	65
Story1	23	22	23	15	14	14
Base	0	0	0	0	0	0

Table 4. 26 Story displacement in Y (mm)

Story	M8*	M9*	M10*	M8	M9	M10
Story10	422	419	438	296	356	394
Story9	410	408	424	287	340	380
Story8	387	386	403	269	326	362
Story7	356	354	367	244	296	332
Story6	313	310	323	218	257	290
Story5	263	260	270	172	214	238
Story4	210	207	218	133	166	191
Story3	161	160	168	98	128	150
Story2	105	104	112	56	80	94
Story1	22	20	23	10	18	20
Base	0	0	0	0	0	0

4.6.3.2 Story shear

ETABS2016 provides a simple table in the summary output with story shear. This provides indication of maximum shear that developed at each floor. The maximum story shear due to push-X in X-direction and push-y in y-direction are:

Table 4. 27 Story shear in push x(kN)

Story	M8*	M9*	M10*	M8	M9	M10
Story10	1300	1434	1590	1128	1270	1420
Story9	2952	3235	3539	2698	2993	3289
Story8	4606	5028	5492	4269	4716	5150
Story7	6262	6829	7443	5844	6432	7029
Story6	7912	8622	9395	7418	8152	8892
Story5	9580	10430	11359	9001	9896	10879
Story4	11283	12283	13368	10616	11668	12708
Story3	13016	14161	15411	12270	13474	14671
Story2	14765	16051	17472	13940	15298	16650
Story1	15800	17184	18690	14950	16390	17850
Base	15871	17258	18774	15120	16476	17984

Table 4. 28 Story shear in push Y(kN)

Story	M8*	M9*	M10*	M8	M9	M10
Story10	1616	1733	1919	1349	1629	1855
Story9	3350	3610	3999	2672	3250	3698
Story8	5119	5492	6080	3998	4861	5540
Story7	6860	7380	8152	5327	6483	7384
Story6	8612	9264	10237	6649	8101	9227
Story5	10374	11149	12324	7983	9732	11086
Story4	12186	13089	14459	9357	11395	12982
Story3	14020	15066	16640	10751	13094	14918
Story2	15864	15053	18829	12147	14814	16868
Story1	16963	18231	20120	13009	15839	18044
Base	17079	18422	20149	13059	16038	18128

4.7 Discussion of results

4.7.1 Slab open down location as a parametric

4.7.1.1 Story maximum displacements

Suggested maximum drift at the top of buildings vary between H/50 and H/2000 where H is the height of the building. A limiting value for the maximum displacement within the elastic limits was obtained as a function of the height of a story, the stiffness of a story, number of stories, the yield strain of steel εy and the maximum allowable concrete strain εc. However, the value H/50 Therefore, obtained values are within limits. Below figures are curves maximum displacements vs.story levels for push X and push Y.

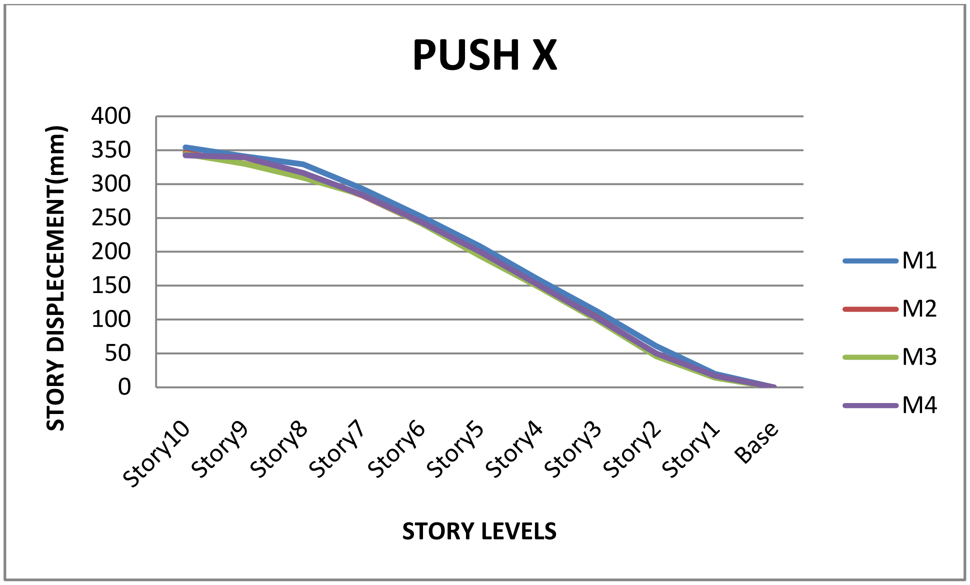


Figure4. 10:Comparison maximum story displacements due to push X.

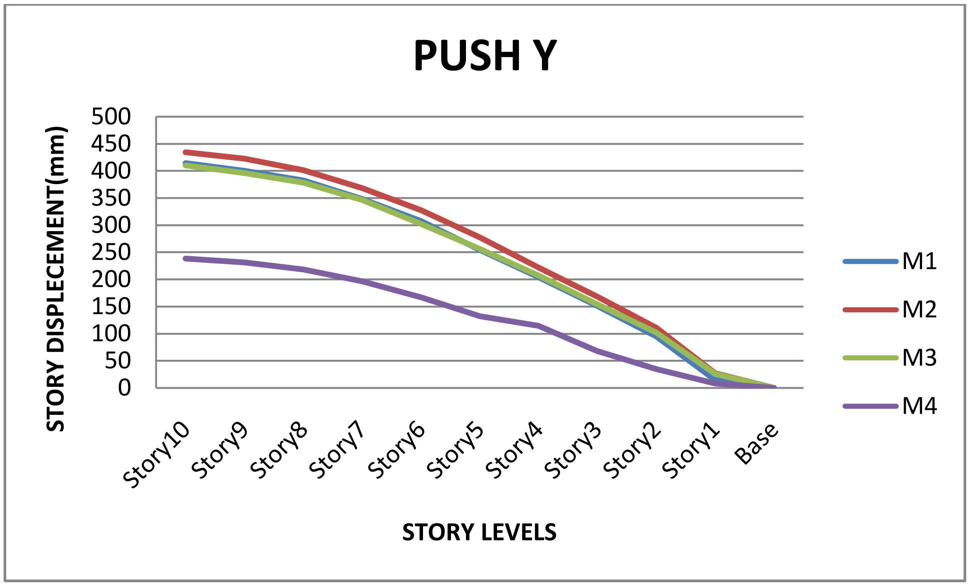


Figure4. 11:Comparison Maximum story displacements due to push Y.

The above curve figure 4.10 reveals that the interrelation among same stories on different slab opened down positions at an angle, main and projection parts decrease the top story displacement by 2.3%, 2.8% and 3.4%, respectively. Therefore, it clearly confirms that the slab opened down decreases top story displacement, especially for the slab opened at the projection part when pushed to x direction.

The above curve figure 4.11 reveals that the interrelation among same stories on different slab opened down positions at an angle, the top story displacement increases by 4.8%, but when slab opening down at the main and projection parts decrease top story displacement by 1% and 42.5% respectively. Therefore, the study clearly depicts that the slab opened down has a significant effect on the projection part when pushed to y direction.

4.7.1.2 Base shear

The base shears obtained from Pushover analyses are as mentioned in tabulation convert to bar-charts are compared below:

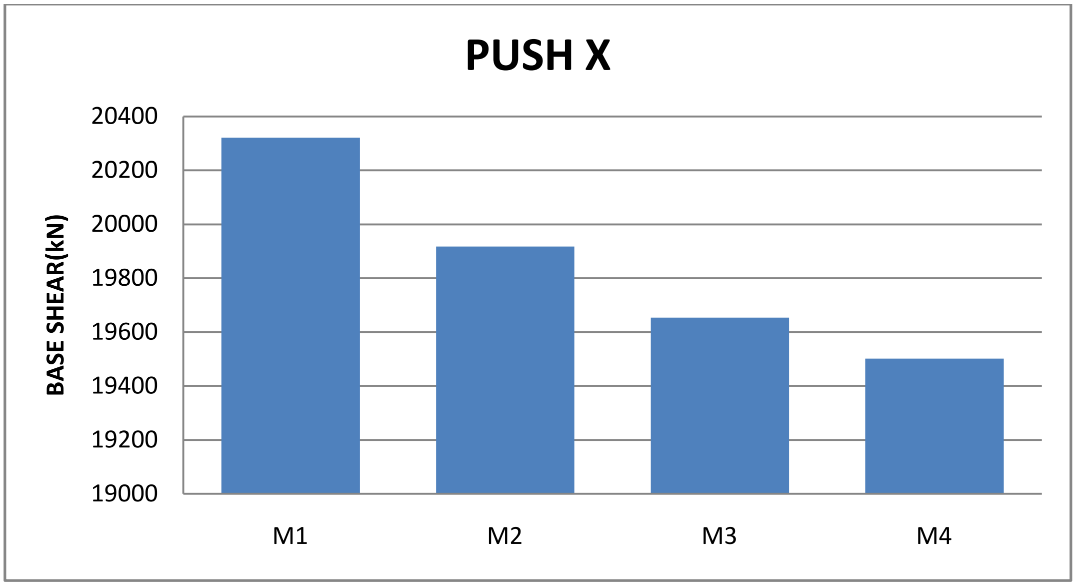


Figure4. 12:Comparison base shears when pushed to x

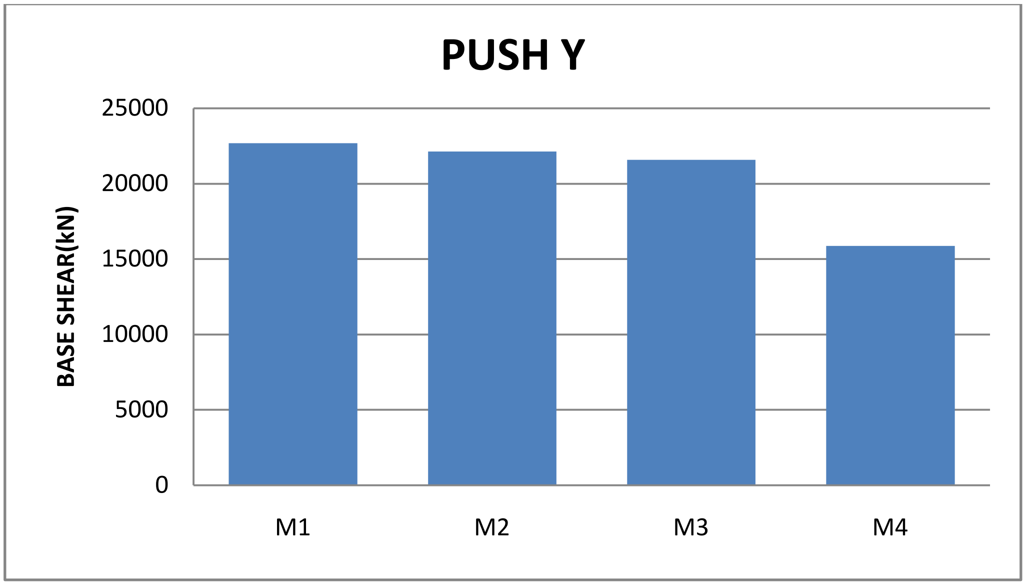


Figure4. 13:Comparison base shears when pushed to y direction

As stated above on the Bar-graph 4.12, the comparison values clearly reveals that the slab opened down at an angle, main part and projection part the base shears have decreased by 2%, 3.3%,and 4% respectively, when pushed to the x-direction.

Similarly, as per the bar-graph 4.13, the slab opened down at an angle, main and projection parts the base shears have decreased by 2.5%, 4.9%, and 30% respectively when pushed to y direction.

4.7.2 Projection length as a parametric

4.7.2.1 Story maximum displacements

Suggested maximum drift at the top of buildings vary between H/50 and H/2000 where H is the height of the building. A limiting value for the maximum displacement within the elastic limits was obtained as a function of the height of a story, the stiffness of a story, number of stories, the yield strain of steel εy and the maximum allowable concrete strain εc. Therefore, obtained values are within limits. Below figures are curves maximum displacements vs. Story levels for push X and push Y.

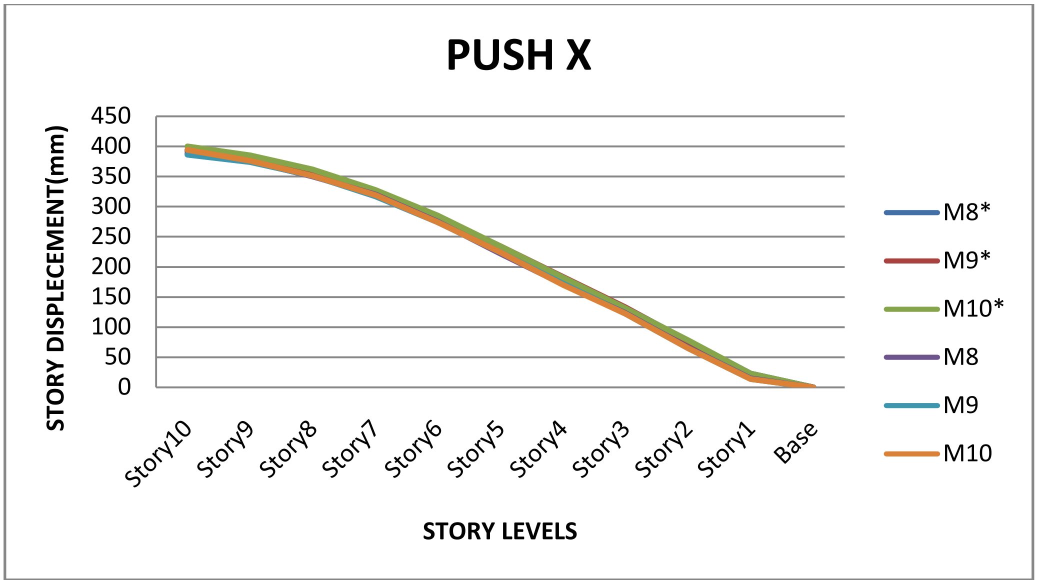


Figure4. 14:Comparison maximum story displacements due to push X

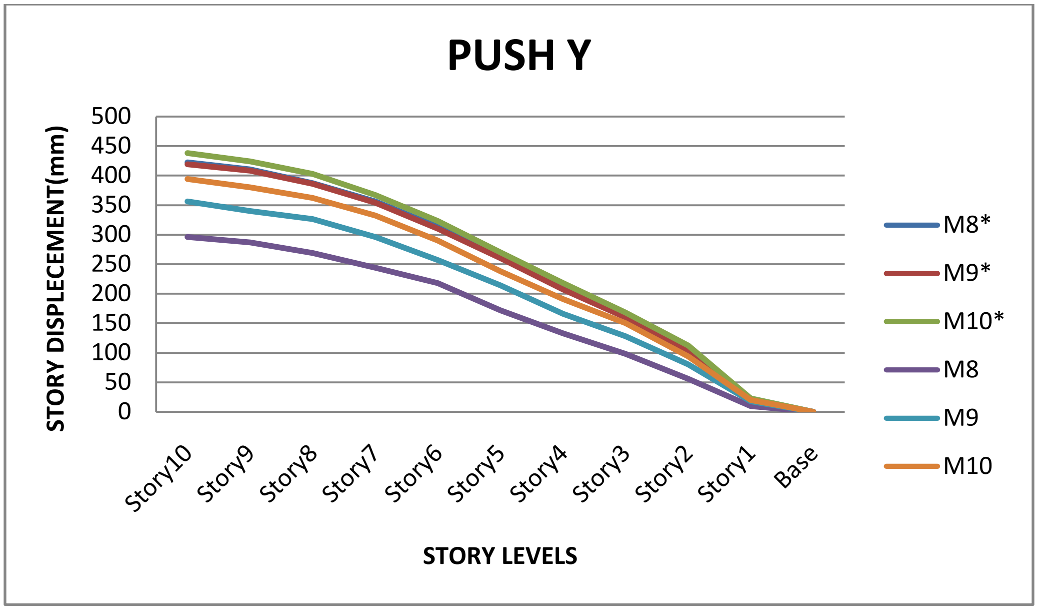


Figure4. 15:Comparison maximum story displacements when pushed to Y direction

As per the figure 4.14, when three different structures which are without slab opened down taken as reference and compared with three slab opened down at angle of 10%, 20% and 30% ,for each projection length respectively, the top story displacement increased by 1.5% for each projections. Therefore, the analysis revealed that top stories displacements increased by same percent which is 1.5% for all projection when push to x directions.

As per the figure 4.15, when three different structures which are without slab opened down taken as reference and compared with three slab opened down at angle of 10%, 20% and 30% for each projections respectively, the top story displacement decreased by 30%, 15% and 10% respectively. Therefore, the analysis depicts that the top story displacement decreased whilst projection length increases when pushed to y direction.

4.7.2.2 Base shear

The base shears obtained from Pushover analyses are as mentioned in tabulation convert to bar-charts are compared in Figure 4.16:

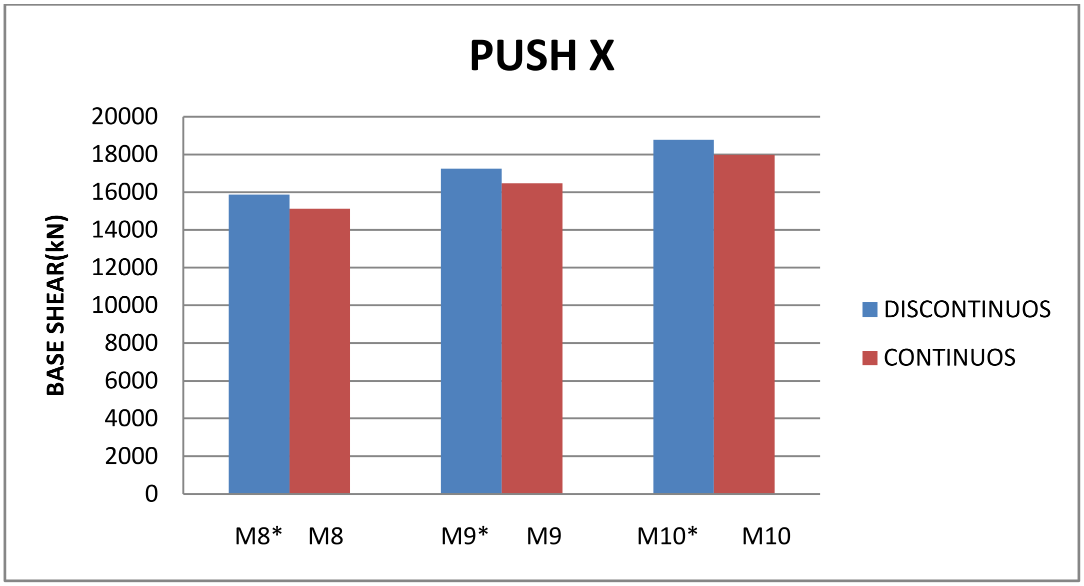


Figure4. 16:Comparison base shears for due to push X.

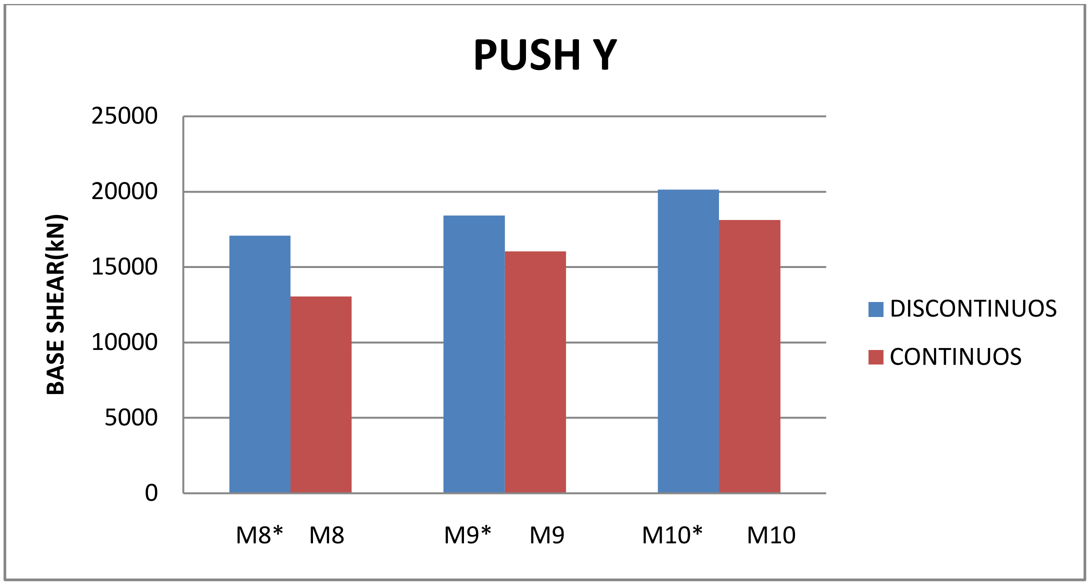


Figure4. 17:Comparison base shears for due to push Y

As per the figure 4.16, when the slab opened down structure is at angle of 10%,20% and 30% projection, the base shears have been decreased by 4.7%,4.5%and 4.2% respectively when pushed to in the x-direction.

As per the figure 4.17, when the slab opened down structure is at angle of 10%,20% and 30% projection, the base shears have been decreased by 23.5%, 13%,and 10% respectively, when pushed to the y direction.

4.7.3 Story height as a parametric

4.7.3.1 Story maximum displacements

Suggested maximum drift at the top of buildings vary between H/50 and H/2000 where H is the height of the building. A limiting value for the maximum displacement within the elastic limits was obtained as a function of the height of a story, the stiffness of a story, number of stories, , the yield strain of steel εy and the maximum allowable concrete strain εc. Therefore, obtained values are within limits. Below figures are curves maximum displacements vs. story levels for push X and push Y.

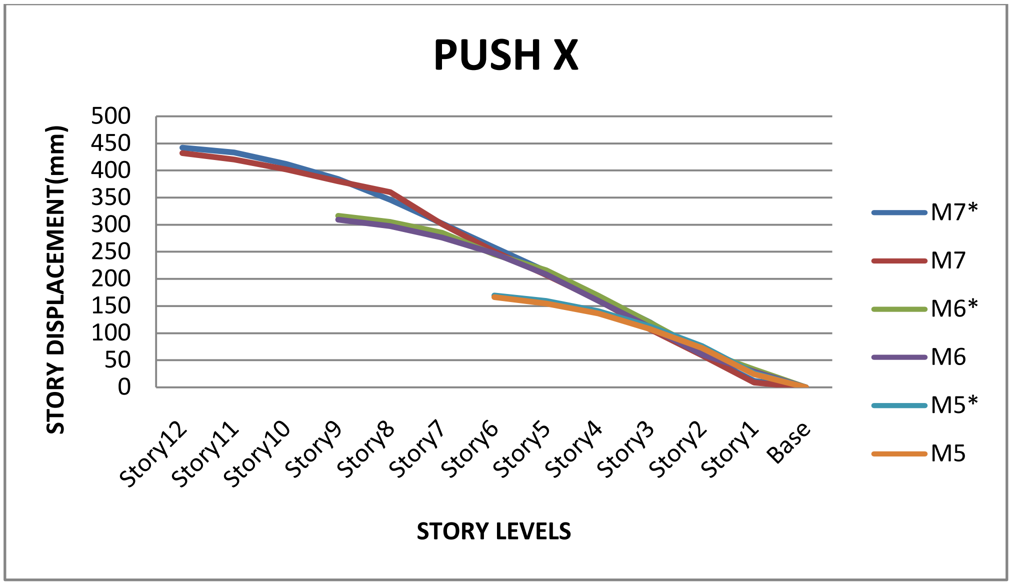


Figure4. 18: Comparison maximum story displacements due to push X.

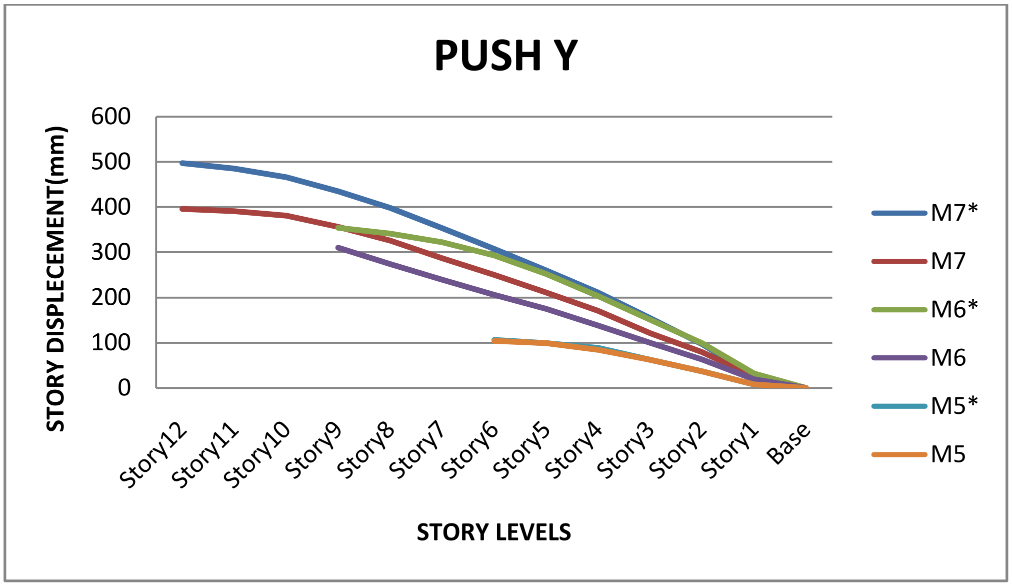


Figure4. 19:Comparison maximum story displacements when pushed to y direction.

As per the figure 4.18, when the slab opened down at angle for story 6, 9 and 12, in constant projection, top story displacement decreased by 1.8%, 2.2% and 2.3% respectively referenced to without slab opened down for each story height. Therefore, the analysis revealed, having slab opened down in all stories led the top story displacement decreased approximately by the same percentage when pushed to the x direction.

As per the figure 4.19, when the slab opened down at angle for story 6, 9 and 12, in constant projection, top story displacement decreased by 1.9%, 12.4% and 20.3% respectively referenced to without slab opened down for each story height. Therefore, the analysis revealed, when the story level increased, the top story displacement decreased when pushed to y direction.

4.7.3.2 Base shear

The base shears obtained from pushover analyses are as mentioned in tabulation convert to bar-charts are compared below

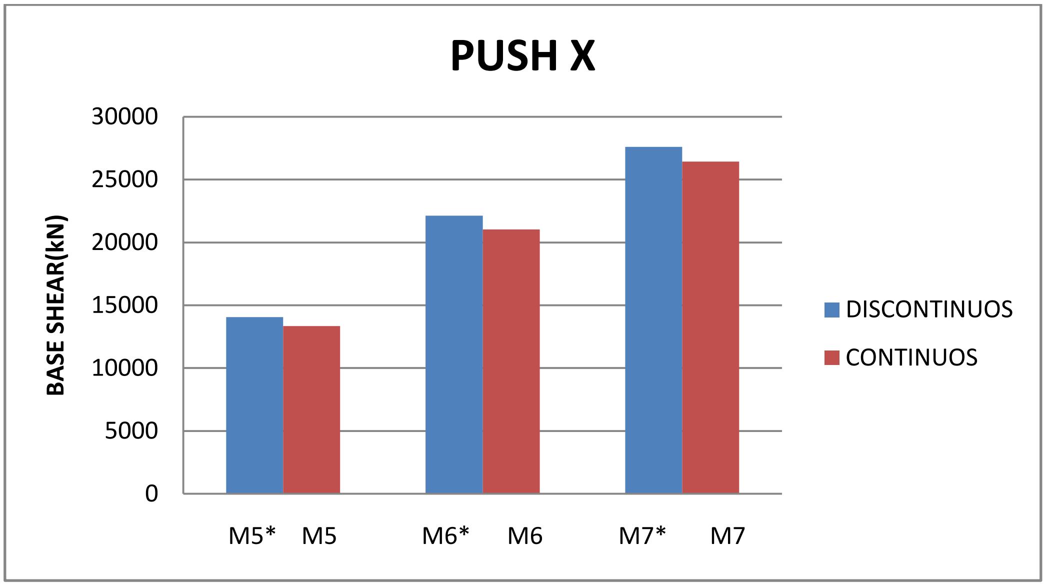


Figure4. 20:Comparison base shears for due to push X.

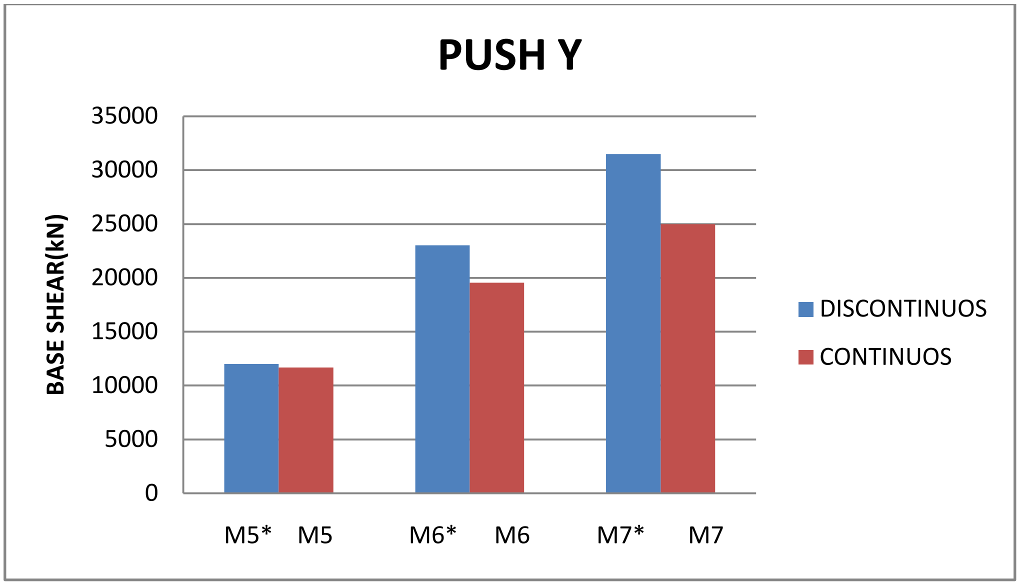


Figure4. 21:Comparison base shears for due to push Y.

As per the values in figure 4.20, when the slab opened down structure of the stories 6, 9 and 12 referenced with the same structure without slab opened down for each stories, the base shears decreased by 5%, 4.9% and 4.3% respectively when pushed to the x direction. The analysis shows the base shears decreased almost by the same percentage (5%).

As per the values in figure 4.21, when the slab opened down structure of the stories 6, 9 and 12 referenced with the same structure without slab opened down for each stories, the base shears decreased by 2.6%, 15.1% and 20.6 % respectively when pushed to the y direction. The analysis shows the base shears decreased when the story height increases.

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

5.1 Conclusion

Generally, the research has come up with the following conclusions after detail analysis of the findings from the raw data for each parameter's (Opening locations, Story height and Projection lengths) used throughout the course of study:-

Same stories on different slab opened down positions: - Top story displacement decreased, especially for the slab opened at the projection part when pushed to x direction and also significantly decreased top story displacement when pushed to y-direction for slab opened down at projection part.

1. **Same stories on different projection lengths:-**Top story displacement increased almost by same percent for all increment of projection length when pushed to x-direction and the analysis depicts that the top story displacement decreased whilst projection length increases when pushed to y direction.
2. **Different stories on same slab opened down positions**: - Slab opened down in all stories led the top story displacement to be decreased approximately by the same percentage when pushed to the x direction and when the story level increased, the top story displacement decreased when pushed to y direction.
3. **Same stories on different slab opened down positions**: - The comparison values clearly reveals that the slab opened down at an angle, main part and projection part, the top story displacement have decreased respectively when pushed to the x-direction. And also the top story displacement have decreased for all models when pushed to y- direction, especially has a significant decrease effect when slab opened down at projection part.
4. **Same stories on different projection lengths**:-. The study reveals that base shears decreased almost by same percent for all projections when pushed to x-direction and while the projection length increased, the base shears decreased respectively when pushed to y-direction.
5. **Different stories on same slab opened down positions**: - The base shears decreased almost by same percent when pushed to the x-direction and the base shears decreased when story height increased respectively, when pushed to the y-direction.

6. **Same stories on different slab opened down positions**: - The comparison values clearly reveals that the slab opened down at an angle, main part and projection part, the base shears have decreased respectively when pushed to the x-direction. Similarly the slab opened down at an angle, main and projection part, the base shears have decreased respectively when pushed to y- direction, especially has a significant decrease effect when slab opened down at projection part.

5.2. Recommendation

Here-under are the recommendations based on the study findings:-

- Evaluation of performance of irregular structures for both in plan and in elevation, using non-linear analysis.
- Evaluation of performance of studied buildings, the load transfer from slab to beam has to be carried out manually in both linear static and push over analysis
- The evaluation of performance of building for dual and wall systems to be carried out, as this research has been conducted only for the moment resisting frames.
- Evaluation of performance of irregular structures for both in plan and in elevation, using dynamic analysis in the first place and then non-linear analysis to find-out the response of the structures.
- Evaluation of the models responses used for the study by taking account other factors, like different slab types, column types, column spacing’s and others.
- The study recommends other researches to be conducted for any discrepancies emerged in this research.

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APPENDICES

Appendix A: Design Sections and Reinforcements for Sample Models with slab

Figure Appendix 5. 1: Sample design sections of six- story building along axis-1

Table 5.1: Sample design sections and Reinforcements and axial force ratios of base columns for six story building model

Location	Longitudinal Rebar	Stirrup	Axial force in base column(kN)	Section(mm)	N/Ag*Fcd
Corner	8 Φ16	8 ΦC/C150mm	1000	400x400	0.441
Edge	8Φ16	8 ΦC/C150mm	1476	500x500	0.417
Interior	12Φ16	8 ΦC/C150mm	2300	500x500	0.649

Appendix B: Beam column capacity ratios of the sample model with slab

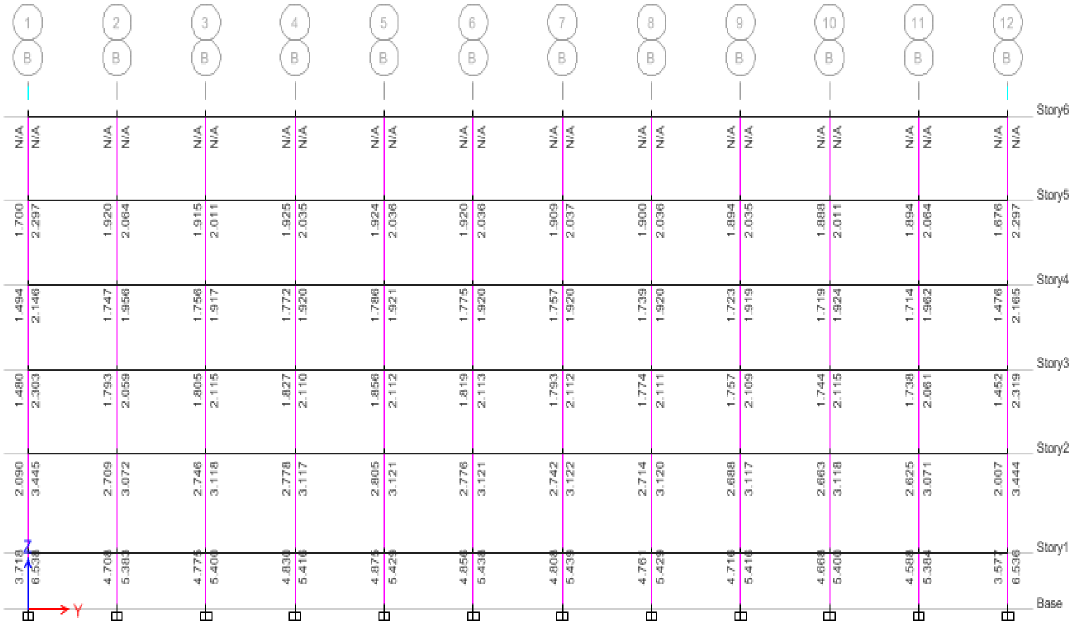

Figure Appendix 5. 2: Sample column / beam capacity ratios for six story building model along axis B

Appendix C: Default hinge assignment sample in beam and column

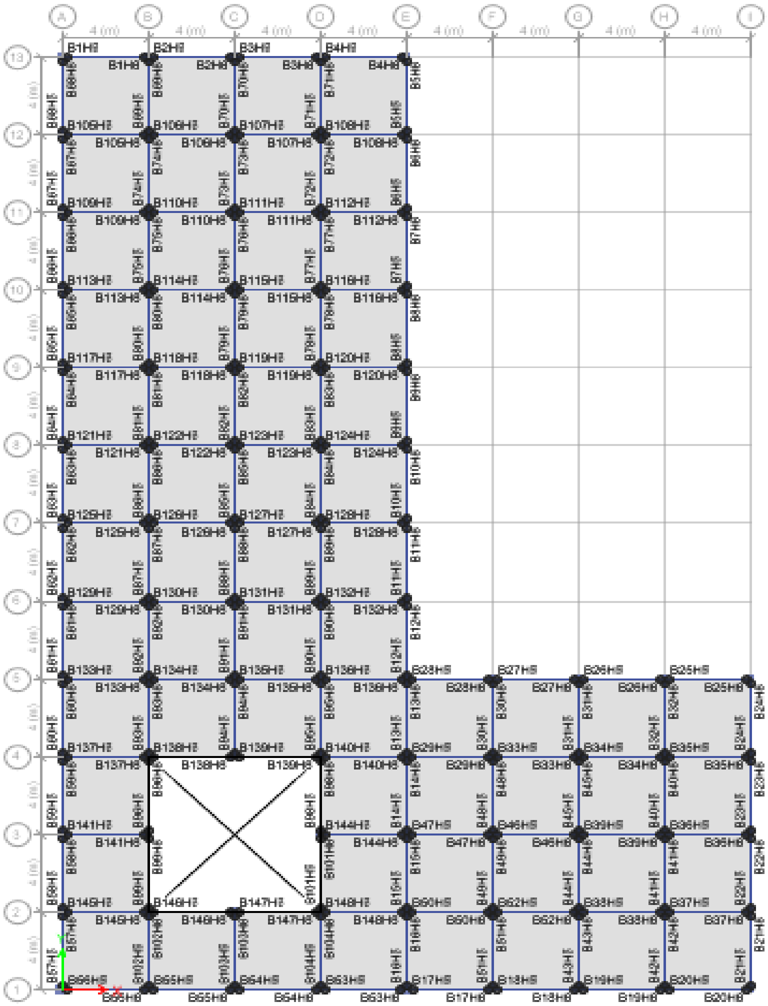

Figure Appendix 5. 3: Default hinge assignment sample in beam for opening at angle when open down location as parametric study.

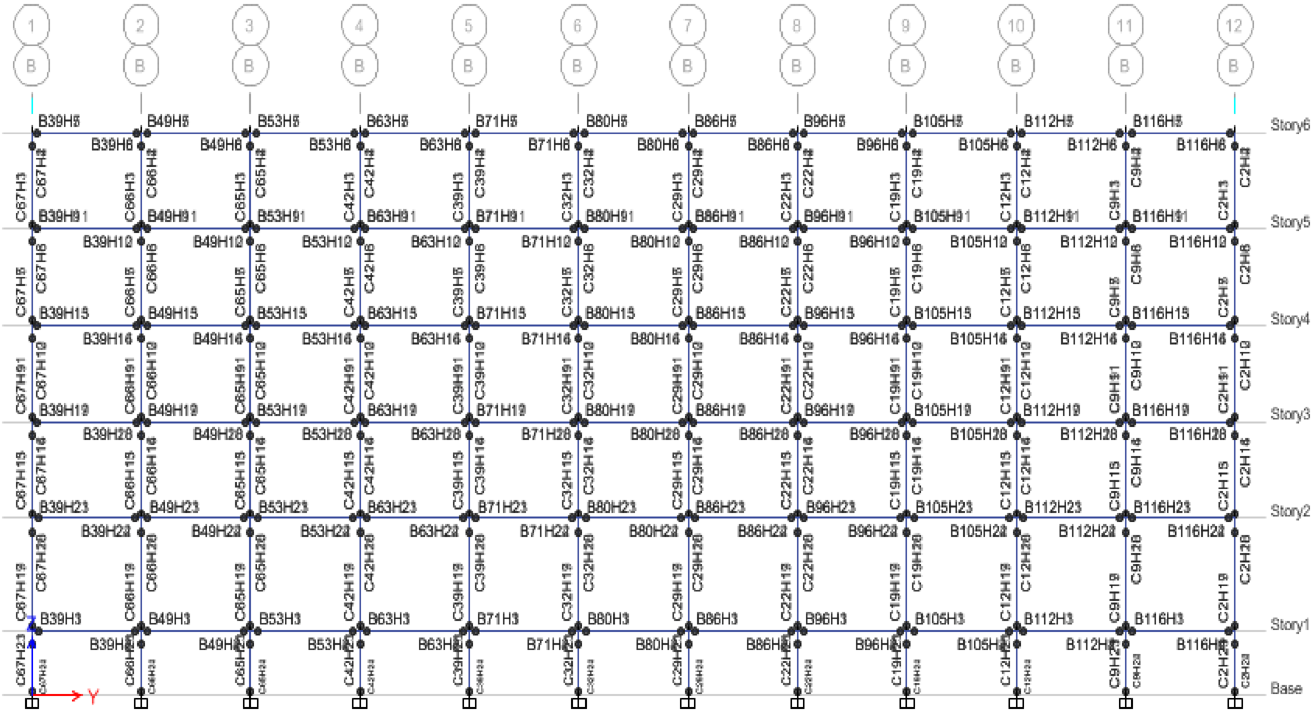

Figure Appendix 5. 4: default hinge assignment sample in beam and column end for open down at angle for six- story model when story height as parametric study along axis B.

Table Appendix 5. 2: horizontal load (imperfection load) sample in six-story building

Story	Load case/combo		θ_0	α_h	α_m	m	N	θ_ι	Hi
Roof	Gravity for Seismic	Top	0.005	0.67	0.71	70	14465.75	0.002379	34.40679
5	Gravity for Seismic	Top	0.005	0.67	0.71	70	22341.75	0.002379	53.13985
4	Gravity for Seismic	Top	0.005	0.67	0.71	70	37450.63	0.002379	89.07631
3	Gravity for Seismic	Top	0.005	0.67	0.71	70	52552.94	0.002379	124.9972
2	Gravity for Seismic	Top	0.005	0.67	0.71	70	67758.15	0.002379	161.1628
1	Gravity for Seismic	Top	0.005	0.67	0.71	70	80480.43	0.002379	191.4227