ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

EVALUATION OF THE EFFECT OF SHEAR WALL ARRANGEMENT IN DUAL SYSTEMS SUBJECTED TO LATERAL LOADS

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in Partial fulfilment of the Requirements for the Degree of Master of Sciences in Structural Engineering

By

Abel Yoseph

September 2009

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ABSTRACT

Dual structural systems are still efficiently used by civil and structural engineers to address the public demand for residential buildings, schools, hospitals and office buildings. Furthermore, they are effective in resisting lateral loads by combining the advantages of their constituents. However, dual structural systems with moment frames are designed to carry lateral forces in longitudinal direction entirely while structural walls are positioned in shorter side of the building to resist transversal lateral loads are challenged by Northridge earthquake. After then, this type of dual system is cited by Erdy Charles k. (2007) as dangerous combination of shear walls and moment resisting frames. Hence, in an effort to address this problem, comparative analysis among the sample dual system buildings with different shear walls arrangement is carried out and overall seismic performance of the dual system is evaluated using non-linear push over analysis method. Three cases of shear wall arrangements for each sample building are used to examine the effect of different shear walls arrangement on the whole system, their interaction with frames and their out-of-plane bending resistance. Results of non-linear push over analysis are obtained for each case in the form of capacity curves and plastic hinge formation patterns. The observation of these results shows that the overall seismic resistance capacity is dependent on the shear wall location. Also it reveals that the shear walls are vulnerable to local failure due to bending moment in their out-of- plane direction, regardless of their location. Therefore, caution should have to be taken during designing of this type of dual system in order to avoid premature collapse of shear wall in its out-of-plane direction, which may perhaps leave the building defence less to transversal components of earth quake load.

DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in any other University and that all sources of materials used for the thesis have been duly acknowledged.

Name	Abel Yoseph
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Date of submission	September 2009

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List of Notations

Symbols	Descriptions
S	the site coefficient
L_i	the length along the direction of eccentricity
h_i	height of the building
W_i	dead load or weight of the building
E	young's modulus of elasticity
I	second moment of inertia
θ_p	plastic rotation
Φ_u	ultimate curvature
Φ_y	yield curvature
l_p	plastic hinge length
h	section depth in the direction of loading
A	initial point on moment-curvature relation
B	yielding curvature point on moment-curvature relation
C	ultimate curvature point on moment-curvature (M-κ) relation
D	point corresponding to sudden fall of moment on M-κ curve
E	point corresponding to rupture on M-κ curve
IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention
dp	displacement coordinate of the performance point
δt	target displacement

1.0 INTRODUCTION

1.1 Background

The need for high-rise buildings is inevitable while the land in the rapidly developing cities and towns are becoming scarce due to increasing number of population. In addition to the scarcity of the land, some part of our world is always stricken by natural disaster like earthquake, hurricane, tornado, tsunami etc. This will compel the structural engineers to design buildings using the building system with good resistant of lateral loads. One of these building systems is dual system where shear wall (structural wall) is used in combination with moment resisting frame. The extent to which a shear wall contribute to the resistance of overturning moments, story shear forces and story torsion depends on its geometric configuration, orientation & location within the building [15]. Therefore, the arrangement of shear wall in the building with dual structural system is essential in order to have building with good resistance of lateral loads.

The dual system with moment frame to resist lateral load in longitudinal direction and shear walls at each end in short direction was approved as a major structural category to resist earthquakes by Structural Engineers Association of California (SEAOC) with the basic concept that the end shear walls will take care of the earthquake force component in the short direction while the moment frame will resist the longitudinal component acting in the longitudinal direction of the structure [7]. However, this type of structure was challenged in the January 17, 1994, by Northridge earthquake. One of the best examples of dual system building which is failed by the Northridge earthquake was Kaiser Permanente office building whose Shear walls at wing collapsed while the moment frame of the building didn’t crack as shown in fig 1.1.

Figure 1-1 Collapsed end shear wall by Northridge earthquakes, 1994. Kaiser Permanente Office Building, Granada Hills, CA. (Photo courtesy of the University of California Library at Berkeley.)

It is believed that the moment frame pushed over and destroyed the shear wall, leaving the structure defenceless against the lateral force component in the short direction, which caused the building to be declared dangerous to use it [7].

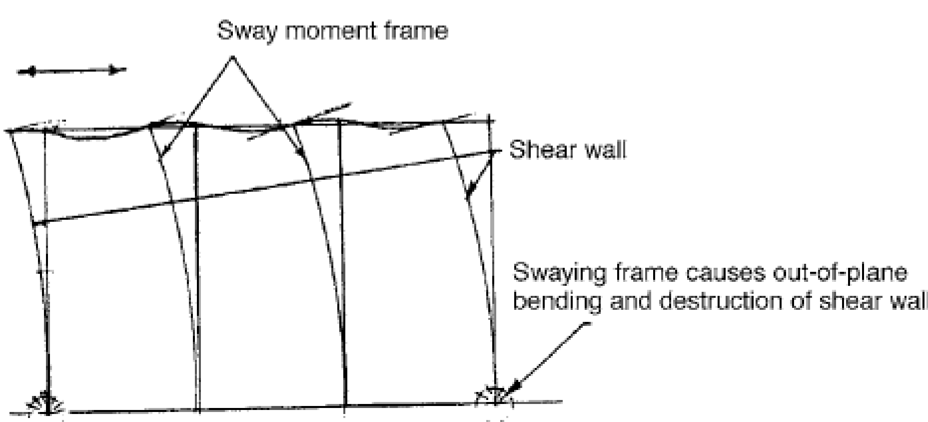


Figure 1-2 Hinge mechanism of moment frame and end shear wall in dual system.

The other example of the same dual building system which has failed a challenge of Northridge earthquake is Champagne Towers, an upscale high-rise apartment building overlooking the Santa Monica Bay in Southern California. When the 1994 Northridge earthquake hits the building, the building responds with the sound of an explosion and with violent and uncontrollable sway. Main load-bearing columns damaged with diagonal cracks up to 0.5 inch while the shear wall didn't show any fracture as shown on fig 1.3 [7].

Figure 1-3 Reinforced-concrete columns severely damaged in the Northridge earthquake, Champagne Towers, Santa Monica, CA.

Consequently, designing & selecting arrangement of shear wall using elastic analysis and with static consideration is misleading. According to Erdey Charles K. [7], 80% of high-rise hospital structures in California estimated to be constructed with the type of dual system mentioned above.

The intention of this thesis is therefore; to investigate and evaluate the effect of the arrangement of shear walls in a dual system. The dual system building with the moment frames are designed to resist the lateral forces in longitudinal direction entirely while shear walls are located in shorter side of the building to take care of the lateral loads that come in transverse direction is used for this study. It is going to be done, by carrying out a comprehensive literature survey and analysis of sample buildings.

1.2 Purpose of the study

The main objective of this thesis is to evaluate the effect of shear wall arrangements in the dual system where the moment frames are designed to carry lateral forces in longitudinal direction entirely while structural walls are situated in shorter side of the building to resist lateral load that comes in transverse direction. This study is carried out by conducting a comprehensive literature survey and by making comparative analysis among the sample buildings with different shear wall arrangement (by placing it at different location). More specifically, this thesis examines whether the dual structural system as specified above have better performance if shear wall was placed somewhere in the middle rather than at the end along the longitudinal direction of the building.

The other objective of this study is to analyse and evaluate the resistance of shear wall to out-of-plane bending for dual structural system mentioned as dangerous combination by Erdey Charles K. [7].Out-of-plane bending resistance of shear walls of different arrangement in the dual system will be compared and its output will be discussed.

2.0 REVIEW OF DUAL STRUCTURAL SYSTEM UNDER LATERAL LOAD

2.1 LATERAL LOADS

2.1.1 General

Loads are a primary consideration in any building design because they define the nature and magnitude of hazards or external forces that a building must resist to provide reasonable performance (i.e., safety and serviceability) throughout the structure's useful life. The anticipated loads are influenced by a building's intended use (occupancy and function), configuration (size and shape), and location (climate and site conditions). Ultimately, the type and magnitude of design loads affect critical decisions such as material selection, construction details, and architectural configuration. Thus, to optimize the value (i.e., performance versus economy) of the finished product, it is essential to apply design loads realistically [14].

There are different types of design loads based on the way they are generated. These include dead (gravity) loads, live loads & lateral loads. Lateral loads are the loads whose main component is horizontal force acting on the structure. They are mostly caused due to environmental loads or natural hazards. Also they are non-static or dynamic in nature. Typical lateral loads would be a wind load against a facade, an earthquake, the earth pressure against a beach front retaining wall or the earth pressure against a basement wall or induced horizontal force due to temperature difference (thermal effect) or centrifugal effect. Most lateral loads vary in intensity depending on the building's geographic location, structural materials, height and shape. Wind loads and earthquake loads are the major constitute of the lateral forces on a structure.

2.1.2 Wind Loads

2.1.2.1 Cause of Winds

As the sun shines on the earth at different parts of the land and sea, it changes temperature of the earth's surface at different speeds. This results in high and low pressure areas and leads to the lift and fall of air passes across the entire globe. Due to the angle of the earth while rotating, the majority of the heat falls upon the middle of the world (equator) and much less towards the ice caps of the northern and southern hemisphere this means that as the warm air rises on the equator the cold air is pulled in from the ice caps. This spreads the warmth across the globe and results in moving air patterns which is called wind.

Also when the temperature of adjacent regions becomes unequal, the warmer air tends to rise and flow over the colder and heavier air. Winds initiated in this way are usually greatly modified by the earth's rotation which will produce extreme weather changes such as thunderstorms, tornadoes, tropical storms and hurricanes.

2.1.2.2 Nature of Wind loads

Wind loads constitute what many engineers call environmental loads on a structure which is a product of the interaction of wind with structure. It is non-static loads on a structure at highly variable magnitudes. usually wind is composed of a massive amount of eddies of varying sizes and rotational characteristics carried along in a general stream of air moving relative to the earth’s surface. These eddies give wind its gusty or turbulent character as shown in figure 2-1[14].

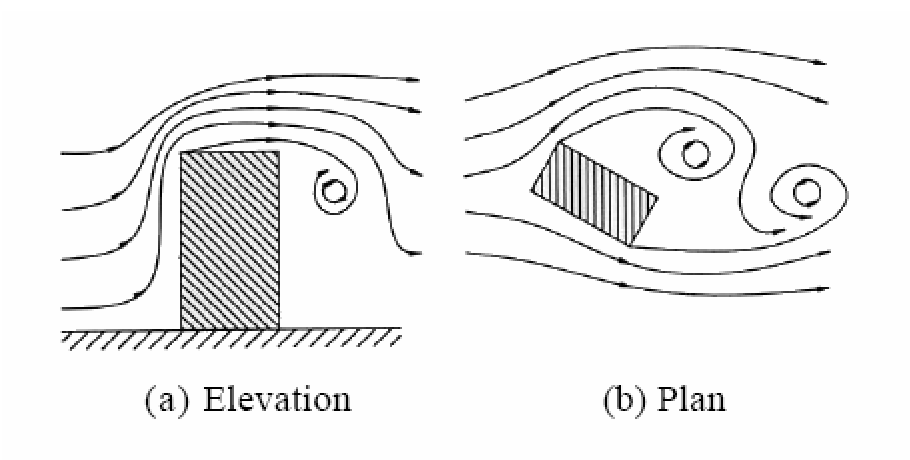


Figure 2-1 Formation of eddies due interaction of wind with buildings

Large wind pressure fluctuations can occur on the surface of a building as winds collide with building. As a result, large aerodynamic loads are imposed on the structural system and intense localised fluctuating forces act on the facade of such structures. Under the collective influence of these fluctuating forces, a building tends to vibrate in rectilinear and torsional modes, as illustrated in figure 2-2 [14].

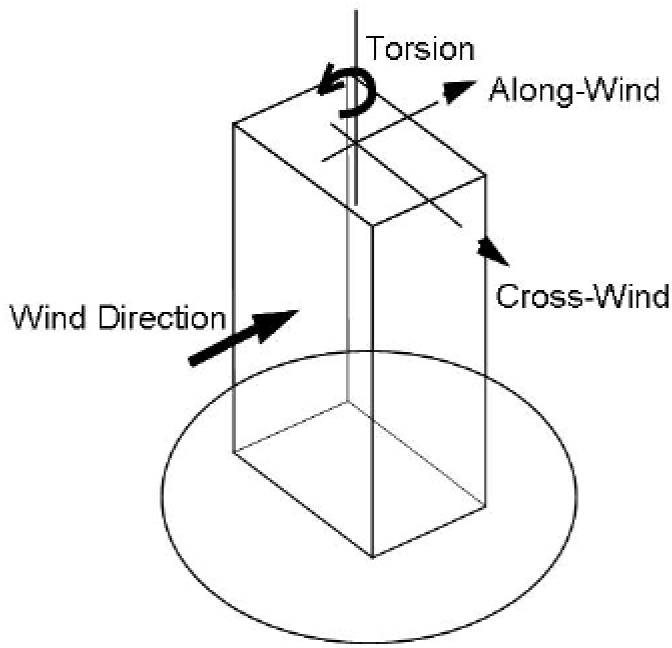


Figure 2-2 Wind response directions

Wind loads are considered at two different scales .On a large scale; the loads are produced on major structural systems that sustain wind loads from more than one surface of the building, including the shear walls and diaphragms that create the lateral force-resisting system as well as the structural systems such as trusses that experience loads from two surfaces of the building [14].

On a smaller scale, pressures are greater on localized surface areas of the building, particularly near abrupt changes in building geometry (e.g., eaves, ridges, and corners), by affecting the loads carried by components and cladding (e.g., sheathing, windows, doors, purlins, studs) [14].

2.1.2.3 Determination of Wind Loads on Buildings

The wind load is an external force, the magnitude of which depends upon the height of the building, the velocity of the wind and the amount of surface area that the wind "attacks”. The determination of wind loads for the design of residential buildings is based on a simplification given by the codes’ wind provisions (for example EBCS1, 1995; IBC 2003, Sec.1609). Therefore, the wind loads are not an exact duplicate. The wind action is represented either as a wind pressure or a wind force and it is assumed to act normal to the surface except where otherwise specified [8].

There are two procedure required to determine design wind loads on a residential building and its components according EBCS1, 1995. The first one is a simplified and static analysis procedure which is set by codes (including EBCS1, 1995). It applies to the structures whose structural properties do not make them susceptible to dynamic excitation. This procedure can also be used for design of mildly dynamic structures whose dynamic coefficient is less than 1.2. The dynamic coefficient takes into account the reduction effects due to the lack of correlation of pressure over the surfaces as well as the magnification effects due to the frequency content of turbulence close to the fundamental frequency of the structure. Its value depends upon the type of the structure (concrete, steel, composite), the height of the structure and its breadth.

The second procedure is a detailed dynamic analysis procedure which is required for the structures that are likely susceptible to dynamic excitation and for the structures whose value of the dynamic coefficient is greater than 1.2 [8]. In such cases, wind tunnel procedure, which is a real time air pressure testing, is used for determination of wind loads on the building. The wind load determination methods in many codes are considered without taking into account the wind-borne debris protection; however, for regions where hurricanes & tornadoes usually occurred, the building envelope (i.e., windows, doors, sheathing, and especially garage doors) should carefully designed for the required pressures.

2.1.2.4 Wind-Resistant Design Philosophy

Unlike earthquake, almost all wind hazards (hurricanes, tornados & tropical storms) are relatively predictable. In the case of wind, excitation is an applied pressure or force on the façade of the structure. The loading is dynamic but the response is nearly static for most structures. Also deformations are monotonic (unidirectional) except for tornados. Therefore most structures are designed to respond elastically under factored wind loads except for tornadoes which can't be resisted by economically feasible design. As a result, proper bracing should be provided to lessen the damage to the structures that are in near miss from a tornado.

2.1.3 Earthquake Loads

2.1.3.1 Causes of Earthquakes

Earthquake is a shaking of the earth's surface caused by rapid movement of the earth's rocky outer layer. Earthquakes occur when energy stored within the earth, usually in the form of strain in rocks, suddenly releases. This energy is transmitted to the surface of the earth by earthquake waves. Most earthquakes are caused by the sudden slip along geologic faults. The faults slip because of movement of the earth's tectonic plates. This type of earthquake is referred as tectonic earthquakes. Other than tectonic earthquake, there are many different types of earthquakes based on the way they are generated. Earthquakes may caused by dilatancy in the crustal rocks, explosions, volcanoes, collapse of mine [2].

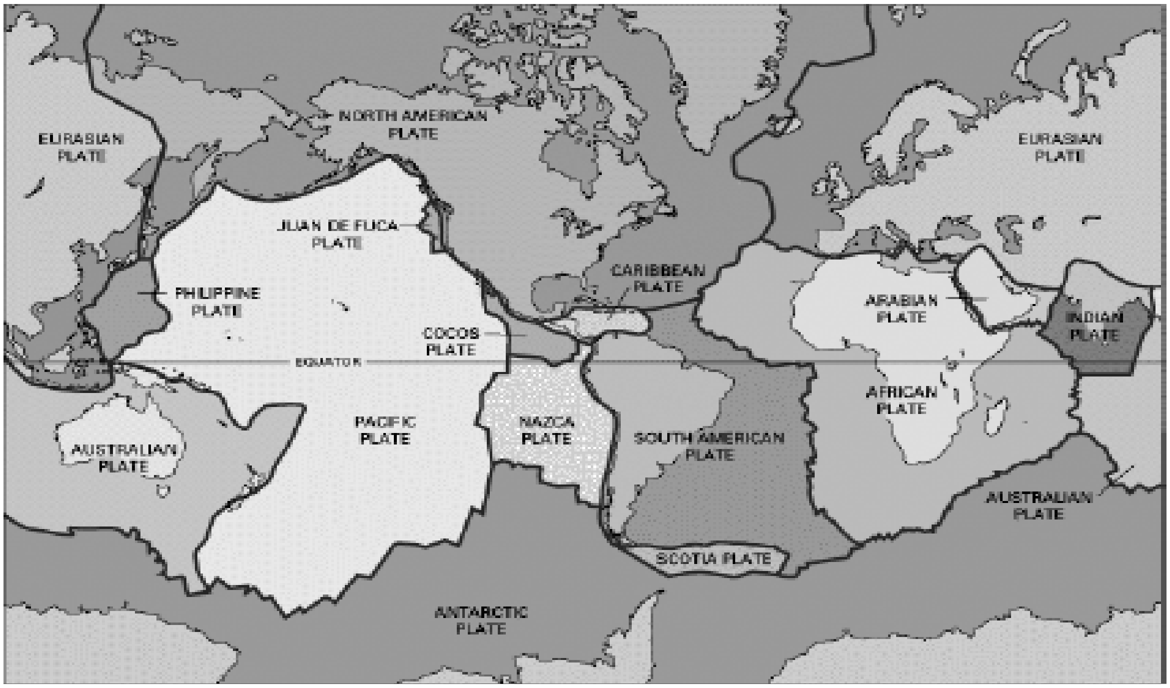


Figure 2-3 Tectonic plates and world-wide distribution of earthquakes [2]

2.1.3.2 Nature and Determination of Earthquake loads

Earthquake loads are another non-static type of lateral loads. They are very complex, uncertain, and potentially more damaging than wind loads. It is quite fortunate that they do not occur frequently. The earthquake creates ground movements that can be categorized as a "shake," "rattle," and a "roll." Every structure in an earthquake zone must be able to withstand all three of these loadings of different intensities. Although the ground under a structure may shift in any direction, only the horizontal components of this movement are usually considered critical in a structural analysis. It is assumed that a load-bearing structure which supports properly calculated design loads for vertical dead and live loads are adequate for the vertical component of the earthquake.

Both, earthquake loads and wind loads are designed as if they are horizontally applied to the structural system. The wind load is considered to be more of a constant force while the earthquake load is almost instantaneous. The magnitude of earthquake load depends up on the mass of the structure, the stiffness of the structural system and the acceleration of the surface of the earth.

According EBCS-8 [10], there are two types of analysis for determining the seismic effects depending on the structural characteristics of the building .These are

- Equivalent Static analysis: for building that satisfies the criteria for regularity in plan & elevation. Additionally the buildings should have fundamental periods of vibration in the two main directions less than 2 second.
- Dynamic analysis: it is applicable to all types of Buildings.

2.1.3.3 Earthquake Resistant Design Philosophy

Severity of ground shaking at a given location during an earthquake can be minor, moderate and strong. Thus relatively speaking, minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely. For instance, on average annually about 800 earthquakes of magnitude 5.0-5.9 occur in the world while about 18 for magnitude range 7.0-7.9. So we should design and construct a building to resist that rare earthquake shaking that may come only once in 500 years or even once in 2000 years, even though the life of the building may be 50 or 100 years [2].

Structural engineering do not attempt to make earthquake proof buildings that will not get damaged even during the rare but strong earthquake since designing such kind of buildings will be too expensive. Instead the engineering intention is to make buildings that resist the effects of ground shaking, although they may get damaged severely but would not collapse during the strong earthquake. These kinds of buildings will assure safety of people and contents, and by this means a disaster is avoided. This is a major objective of seismic design codes throughout the world.

The general philosophy of earthquake resistant building design is that:

A) Under minor but frequent shaking, the structural members of the buildings that carry vertical and horizontal forces should not be damaged; however non-structural buildings parts that do not carry load may sustain repairable damage.
B) Under moderate but occasional shaking, both structural & non-structural members may sustain repairable damage.

C) Under strong but rare shaking, the structural and non-structural members may sustain severe damage, but the building should not collapse.

Earthquake resistant design is therefore concerned about ensuring that the damages in buildings during earthquakes are of acceptable amount, and also that they occur at the right places and in right amounts [3]. Structural members of the building should be ductile enough to avoid collapse.

2.2 DUAL STRUCTURAL SYSTEMS

2.2.1 Historical Background of Dual Structural Systems

Even though using reinforced concrete for construction is started in the early 1900s, the structural system employed at that moment was the traditional beam-column frame system. This traditional beam-column system made the construction of taller buildings very expensive and economically impractical. Because of this scenario, the reinforced concrete buildings height was limited to only a few stories [16].

In the early 1950s, new structural systems (frame-wall systems) are introduced and the use of reinforced concrete in apartment and office buildings as high as 30 stories made possible. This new structural system is referred to us dual or hybrid structural system. It is effective in resisting lateral loads in addition to the gravity load compared to beam-column frame system. Dual structural systems combine the advantage of their constituent elements. Ductile frames, interacting with walls, can provide a significant amount of energy dissipation particularly in the upper stories of a building. On the other hand, as result of the large stiffness of walls, good story drift control during an earth quake can be achieved and the development of story mechanism involving column hinges as in that of soft stories can readily be avoided [15].

2.2.2 Moment resisting frames

Moment-resisting frames (MRFs) are structures that resist applied forces primarily by bending of their members and connections. They can provide large open spaces without the obstruction usually caused by braces or shear walls. Furthermore, because of their flexibility and relatively long period of vibration, they usually draw smaller seismic forces than the comparable braced or shear wall systems.

Moment resisting frames can be classified in to three categories based on their structural detailing. These are special moment resisting frames (SMRF), intermediate moment-resisting frames (IMRF) and ordinary moment-resisting frames (OMRF). For highly seismic zone (Zone 3 & 4), UBC 1997 requires the usage of SMRF type reinforced concrete moment resisting frame in building construction.

Reinforced concrete special moment resisting frames (SMRF) are used as part of seismic force-resisting systems in buildings that are designed to resist earthquakes when flexibility is desired in architectural space planning. Beams, columns, and beam-column joints in moment frames are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements result in a frame capable of resisting strong earthquake shaking without significant loss of stiffness or strength. These additional requirements in special moment resisting frames improve the seismic resistance in comparison with less strictly detailed intermediate and ordinary moment resisting frames [13].

The proportioning and detailing requirements for moment resisting frames are intended to ensure that inelastic response is ductile. Three main goals that should have to be achieved in order to make sure that the moment resisting frames are ductile include: (1) to achieve a strong-column and weak-beam design that spreads inelastic response over several stories; (2) to avoid shear failure; and (3) to provide details that enable ductile flexural response in yielding regions.

2.2.2.1 Principle of a Strong-column and Weak-beam Frame Design

When a building sways during an earthquake, the distribution of damage over height depends on the distribution of lateral drift. If the building has weak columns, drift tends to concentrate in one or a few stories (figure 2.4a), and may exceed the drift capacity of the columns. On the other hand, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed (figure 2.1c), and localized damage will be reduced. Additionally, it is important to recognize that the columns in a given story support the weight of the entire building above those columns, whereas the beams only support the gravity loads of the floor of which they form a part; therefore, failure of a column is of greater consequence than failure of a beam. Recognizing this behaviour, building codes specify that columns be stronger than the beams that frame into them [13]. This strong-column and weak-beam principle is fundamental in achieving safe behaviour of frames during strong earthquake ground shaking.

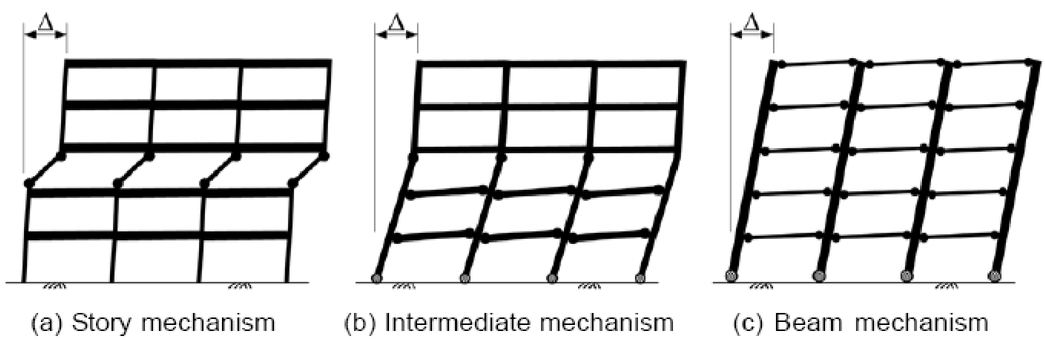


Figure 2-4 Design of special moment frames aims to avoid the story mechanism (a) and instead achieve either an intermediate mechanism (b) or a beam mechanism (c) [13].

In designing moment resisting frame, often the principle of strong-column and weak-beam is implemented in order to make sure that plastic hinging occurs in the beams and in order to ensure that the frame is capable of dissipating significant energy while remaining stable in the inelastic region. The stability in this context is defined as the ability of the frame to maintain its elastic level of resistance throughout the entire inelastic range of response.

2.2.2.2 Avoiding shear failure

Ductile response requires that members yield in flexure, and that shear failure be avoided. Shear failure, especially in columns, is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity. Column shear failure is the most frequently cited cause of concrete building failure and collapse in earthquakes [13].

2.2.2.3 Proper Detailing for Ductile Behavior

Plain concrete has relatively small usable compressive strain capacity (around 0.003), and this might limit the deformability of beams and columns of moment resisting frames. Strain capacity can be increased ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete as it is loaded in compression, and this confining action leads to increased strength and strain capacity. Hoops typically are provided at the ends of columns, as well as through beam-column joints and at the ends of beams as shown on fig 2.5 [13].

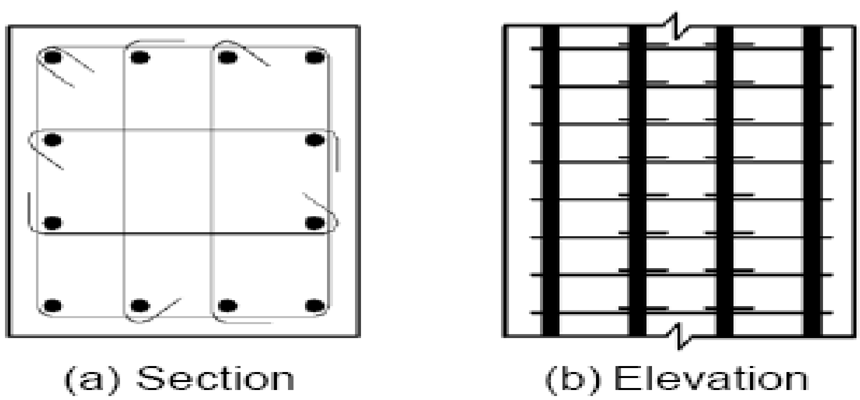
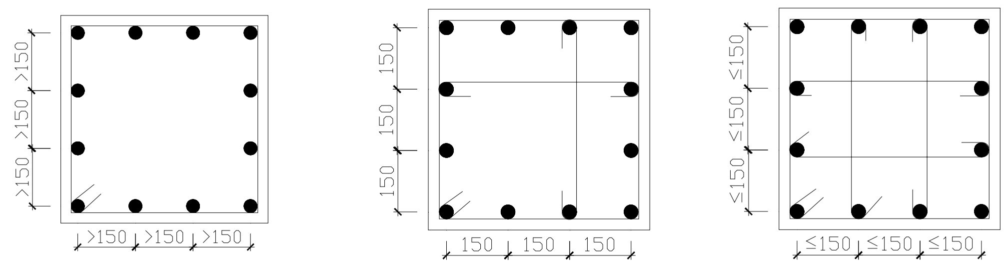


Figure 2-5 Hoops confine heavily stressed cross sections of columns and beams, with (a) hoops surrounding the core and supplementary bars restraining longitudinal bars, all of which are (b) closely spaced along the member length

Column hoops should be configured with at least three hoops or crosstie legs restraining longitudinal bars along each face as shown fig 2-6 c. A single perimeter hoop without crossties (fig 2-6 a), legally permitted by many codes for small column cross sections, but it is discouraged in moment resisting frame because confinement effectiveness is low.



(a) Poorly confined (b) Improved confinement (c) Well confined

Figure 2-6 Different hoop configurations with different degrees of confinement

Generally, moment resisting frames also can be used in dual structural systems that combine moment resisting frames with shear walls. In this study, the moment resisting frame designed to carry lateral load in longitudinal direction while shear wall positioned to take care of lateral load in transverse direction is considered.

2.2.3 Reinforced Concrete Shear walls

Shear walls are vertical elements in the lateral force resisting system. They transmit lateral forces from the diaphragm above to the diaphragm below or to the foundation. Shear walls might be considered as analogous to a cantilever plate girder standing on end in a vertical plane where the wall performs the function of a plate girder web, the floor diaphragms function as web stiffeners, and the integral reinforcement of the vertical boundaries functions as flanges. The distribution of shear forces is proportional to the moment of inertia of the cross sections of the walls. The displacements in each floor or level are the result of the flexural deformations in the walls [16].

Shear walls may be subjected to both vertical (gravity) and horizontal (wind or earthquake) forces. The horizontal forces are both in plane and out of plane. walls will be designed to withstand all vertical loads and horizontal forces, both parallel to and normal to the flat surface, with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall, whether or not intended as part of the lateral force resisting system, is subjected to lateral forces unless it is isolated on three sides (both ends and top), in which case it is classified as non-structural. Any wall that is not

isolated will participate in shear resistance to horizontal forces parallel to the wall, since it tends to deform under stress when the surrounding framework deforms [15].
The extent to which individual shear wall contribute resistance to overturning moment, story shear force and story torsion depend on its geometric configuration, orientation and location in the plan of building [15]. Designing of reinforced concrete shear wall considers torsional stability of the building system, the arrangement (location) of the walls and the flexural and torsional stiffness of individual walls. Also they need to be designed and constructed by ensuring their ductility. Seismic provisions of building codes in various countries provide guidelines for ductile detailing of reinforced concrete shear walls. In a reinforced concrete shear wall, steel reinforcing bars are to be provided in regularly spaced vertical and horizontal grids [15].

Generally reinforced concrete shear walls provide large strength and stiffness to buildings in the direction of their orientation, which reduces lateral sway of the building and thereby reduces damage to structural and non-structural components. Also special detailing is required for reinforced concrete in high seismic regions.

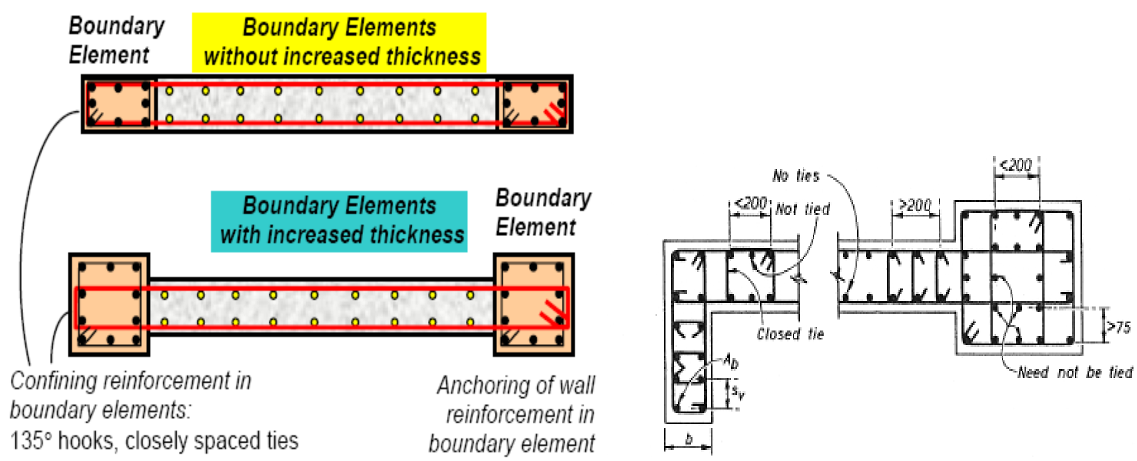
2.2.3.1 Type of Reinforced Concrete Shear walls

Reinforced concrete wall may be considered as short or squat when the ratio of its effective height to its length less than 7. Otherwise, it shall be considered as a slender [9]. Squat shear walls are mostly used in seismic resistance of low-rise buildings. Also sometimes they are placed in high-rise building, where they make a major contribution of lateral load resistance when extending only over few stories above foundation level [15]. According to Paulay and Priestley [15], Squat shear walls can be categorized as elastic walls, rocking walls and ductile walls based on their response characteristics.

Also reinforced concrete shear walls can be categorized as coupled or uncoupled (non-coupled) shear walls based on the way they resist lateral loads. When lateral loads are resisted by independent actions of individual walls, those walls are referred to us as uncoupled shear walls. The connecting element among the uncoupled walls assumed to be non-bending resistant elements or pin-ended links. In many practical situations, many shear walls are connected to each other in bending-resisting members or coupling beams. Then applied moment will be resisted by the two walls acting as single composite units. These types of shear walls are termed as coupled shear walls [16].

2.2.3.2 **Proper Shear Walls Detailing for Ductile Behavior**

Just like reinforced concrete (RC) beams and columns, RC shear walls also perform much better if designed to be ductile. Overall geometric proportions of the wall, types and amount of reinforcement, proper provision of confinement and connection with remaining elements in the building affects the ductility of the shear walls. Confining reinforcement is provided in compression region of potential plastic hinge zone of shear walls to confine concrete and to avoid buckling in longitudinal reinforcement as shown in figure 2-7[15].



a) Section of shear wall b) types of hoops used for confining

Figure 2-7 Provision of confining reinforcement of shear wall shown in (a) section with (b) different types of hoops used for confining shear wall

2.2.4 Reinforced concrete dual structural system

Dual structural system combines the advantages of its constituent. Under the action of lateral forces, a frame will deform primarily in a shear mode where as a wall will behave like a vertical cantilever with primary flexural deformations. Compatibility of deformations requires that frames and walls sustain at each level essentially identical lateral displacements as shown in figure 2-8.

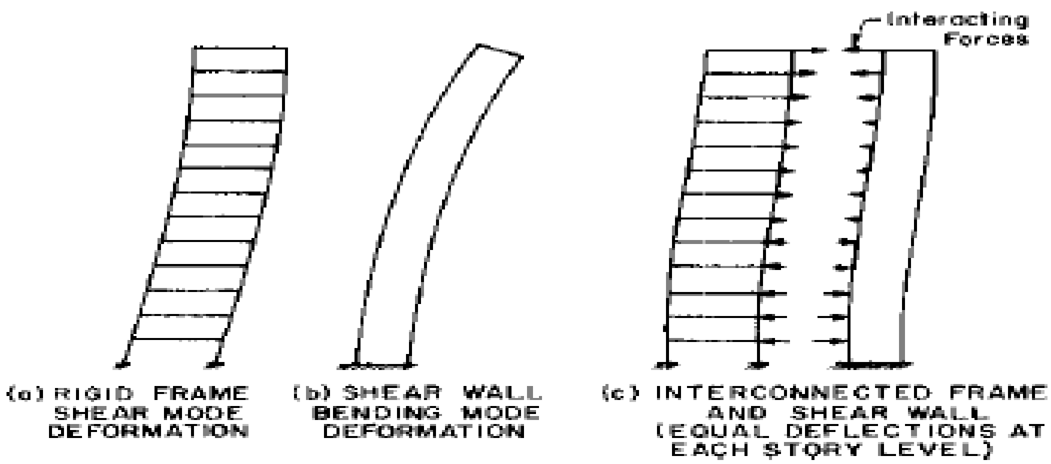


Figure 2-8 Deformation patterns due to lateral forces of a frame, a wall element and a dual system

Shear walls and frames share the resistance of story shear forces in lower stories but tend to oppose each other at higher level. According Pauely & Priestely [15], shear walls provide high contribution to moment and shear force resistance of dual structural system in lower stories but its contribution of moment and shear force diminishes with height & the contribution goes to moment resisting frames in upper stories. The mode of sharing the resistance to laterals forces between walls and frames of dual system is also strongly influenced by the dynamic response characteristics and development of plastic hinges during major seismic events and it may be quite different from the predicted by an elastic analysis [15].

2.2.5 Modes of Failure in Dual structural system

In dual structural system, shear walls are connected to either moment resisting frames or shear walls by coupling beams. Therefore, mode of failure of dual structural systems depends on ductility, stiffness and strength of its constituents (frames, walls or coupling beams). They may fail in flexure, shear or combined action of flexure and shear.

2.3 NONLINEAR PUSHOVER ANALYSIS OF REINFORCED CONCRETE STRUCTURES

2.3.1 General

As the world moves towards the implementation of performance based engineering philosophies in seismic design of civil structures, structural engineers have to perform nonlinear analyses of the structures more often than before for the designing and maintenance purpose. As a result, nonlinear pushover analysis becomes very attractive method since it is easy to use, fairly accurate and suitable in a design office setting usage. It is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking.

Non-linear pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Also it allows tracing the sequence of yielding and failure on members and structural levels as well as the progress of overall capacity curve of the structure. In this research, ATC-40, FEMA-273 &FEMA-356 are used as the reference document for performing the nonlinear push over analysis or nonlinear static procedure.

2.3.2 **Definition of Nonlinear pushover analysis**

According Federal Emergency Management Agency document 273 [11], Nonlinear Pushover Analysis or Non-Linear Static Procedure is defined as a non – linear static approximation of the response that a structure will undergo when subjected to dynamic earthquake loading. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material non – linearity of an existing or previously designed structure and monotonically increasing those loads until the peak response of the structure is obtained on capacity curve (a base shear vs. roof displacement plot) as shown in figure 2-9. From this plot and other parameters representing the expected or design earthquake, the maximum deformations that the structure is expected to undergo during the design seismic event can be estimated.

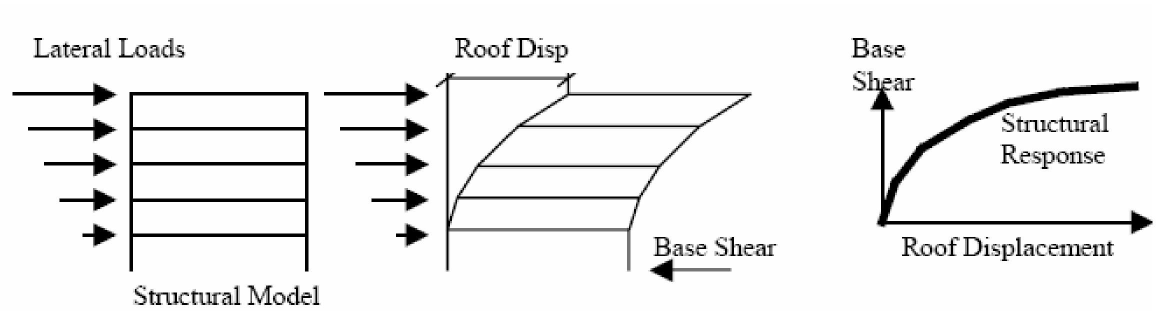


Figure 2-9 Static approximation used in the pushover analysis

2.3.3 Description of Building Performance Level

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquakes serviceability of the building [1]. Hence, building performance can be described qualitatively in terms of the safety afforded to building occupants, during and after the event; the cost and feasibility of restoring the building to pre-earthquake condition; the length of time the building is removed from service to effect repairs; and economic, architectural, or historic impacts on the larger community [11].

The building performance level is a function of the post event conditions of the structural and non-structural components of the structure. Each building performance level consists of a structural performance level, which defines the permissible damage to structural systems, and a non-structural performance level, which defines the permissible damage to non-structural building components and contents [11].

According Federal Emergency Management Agency document 273[11] & 356 [12], the structural performance level is defined as the post – event conditions of the structural building components, which is divided into three levels and two ranges while the non-structural performance level is defined as the post – event conditions of the non-structural components, which is divided into five levels. Consequently, the performance level of building is a combination of the performance level of both structural and non-structural components. Even though there is several performance level of building based on possible combination of structural and non-structural component's performance level, the more common & well established building performance levels are four. These are Operational, Immediate Occupancy, Life Safety and Collapse Prevention level of performances. Some common building performance levels are shown in figure 2-10 and the expected damage on each level is given in table 2.1(taken from table 2.3 of FEMA 273)

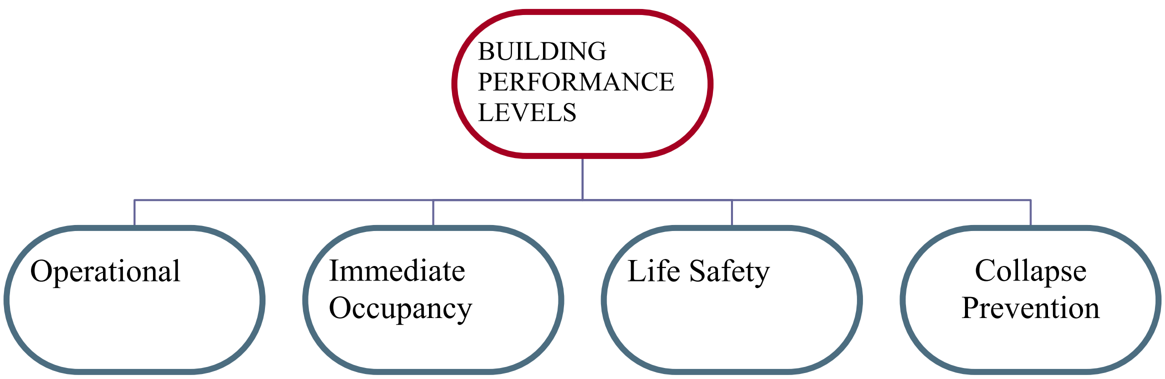


Figure 2-10 some common building performance levels

Table 2-1 Damage Control and Building Performance Levels (taken from table 2-3, FEMA 273)

	Building Performance Levels			
	Collapse Prevention Level	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. In fills and un braced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional.
Nonstructural components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.	Negligible damage occurs. Power and other utilities are available, possibly from standby sources.
Comparison with performance intended for buildings designed, under the NEHRP Provisions, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.	Much less damage and lower risk.

The owner, architect, and structural engineer can now decide what building performance level they want their building to achieve after a range of ground shakings which are expected to occur at a given design location.

2.3.4 Seismic Hazard

An important parameter that must be determined for the pushover analysis is the seismic hazard of a given location. The most common earthquake damage to buildings is caused by the ground shaking. The knowledge of seismic hazard or earth quake ground motion is one of the requirements in setting basic safety performance objective. It is combined with a desired performance level to form a performance objective. The earthquake ground motion can be expressed either by specifying a level of shaking associated with a given probability of occurrence (a probabilistic approach), or in terms of the maximum shaking expected from a single event of a specified magnitude on a specified source fault (a deterministic approach).

ATC-40 [1] ,FEMA -273[11] & FEMA- 356[12] sets three levels of ground shaking for basic safety performance objective with different definition and categorization as shown in table 2-2.

Table 2-2 Seismic hazard levels defined for basic safety performance objective

Earthquake hazard levels according ATC-40	Description of the seismic hazard level	Earthquake hazard levels according FEMA 273 & FEMA 356	Description of the seismic hazard level
The Serviceability Earthquake (SE):	Ground motion with a 50 % chance of being exceeded in a 50-year period	The Serviceability Earthquake	Earthquake with any defined probability of exceedance in 50 years
The Design Earthquake (DE):	Ground motion with a 10 % chance of being exceeded in a 50-year period	Basic Safety Earthquake 1 (BSE-1)	Earthquake ground motion With a 10% probability of exceedance in 50 years (10%/ 50 year).
The Maximum Earthquake (ME):	Maximum level of ground motion expected within the known geologic framework due to a specified single event (median attenuation), or the ground motion with a 5 %chance of being exceeded in 50 year period	Basic Safety Earthquake 2 (BSE-2), also termed as Maximum Considered Earthquake (MCE) ground shaking	Earthquake ground motion With a 2% probability of exceedance in 50 years (2%/ 50 year).

In addition to ground motion levels described in the table 2-2, performance objectives may be formed considering earthquake ground shaking hazards with any defined probability of exceedance, or based on any deterministic event on a specific fault. For the specific site, ground shaking hazard is determined using a specific study of the faults and seismic source zones that may affect the site, as well as evaluation of the regional and geologic conditions that affect the character of the site ground motion caused by events occurring on these faults and sources.

2.3.5 Vertical Distribution of the Lateral Loads in Pushover analysis

In non-linear pushover analysis, there are three types of vertical distribution of lateral loads [4]. These are

I) **"Uniform" Distribution or Rectangular Distribution:** The lateral force at any story is proportional or equal to the mass at that story, i.e.

$$F_i = \frac{m_i}{\sum m_i}$$

Where: F_i: lateral force at i-th story

m_i: mass of i-th story

II) **Equivalent Lateral Force (ELF) Distribution:** The lateral force at a floor is computed from the following formula

$$F_i = \frac{m_j h_j^k}{\sum m_i h_i^k}$$

Where: m_j is the mass of jth floor,

hj is the height of the jth floor above the base floor, and

k =1 for fundamental period T1 ≤ 0.5 sec,

k = 2 for fundamental period T1 > 2.5 sec; and

Linear interpolation shall be used to estimate values of K for intermediate values T. The lateral load computed from this formula has a triangular pattern which is similar to a pattern of lateral loads obtained from Elastic first mode analysis or different national codes. Also Lateral load obtained from FEMA-273 is categorized under this type of load which is used in this research. This load pattern is effective for the regular building whose mass participation above 75% in the first mode.

III) **Multi-Modal or SRSS Distribution:** The lateral load pattern considers the effects of elastic higher modes of vibration for long period and irregular structures and the lateral force at any story is calculated as Square Root of Sum of Squares (SRSS) combinations of the load distributions obtained from the modal analyses of the structures.

2.3.6 Non Linear Pushover Analysis Procedure

Pushover analysis can be performed as either force-controlled or displacement controlled depending on the physical nature of the load and the behavior expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought (such as in seismic loading), where the magnitude of the applied load is not known in advance, or when the structure can be expected to lose strength or become unstable.

Nonlinear version of SAP2000 can model nonlinear behavior of structures and perform pushover analysis directly to obtain capacity curve for two and/or three dimensional models of the structure. When such programs are not available or the available computer programs could not perform pushover analysis directly, a series of sequential elastic analyses are performed and superimposed to determine a force displacement curve of the overall structure.

There are different simplified non-linear pushover analysis methods to determine the primary elements of a performance-based design procedure, i.e demand, capacity and performance. The first one is Capacity Spectrum Method (CSM), for which ATC-40[1] is used as a guideline. Capacity spectrum method is a nonlinear static analysis procedure that provides a graphical representation of the expected seismic performance of the existing or retrofitted structure by the intersection of the structure's capacity spectrum with a response spectrum (demand spectrum) representation of the earthquake's displacement demand on the structure. The intersection is the performance point, and the displacement coordinate (d_p) of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard as shown in figure 2-11[1].

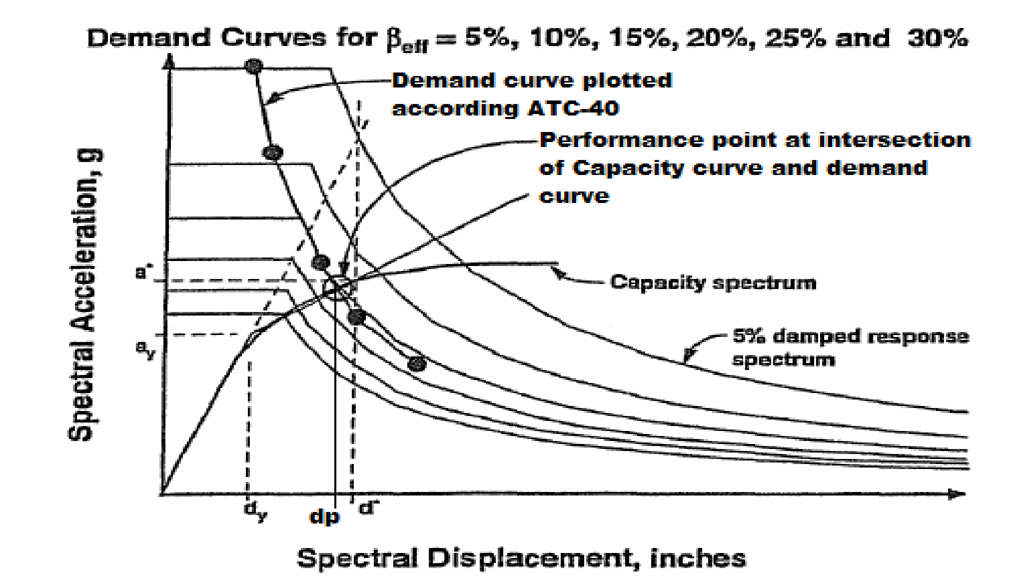


Figure 2-11 Performance point obtained by capacity spectrum procedure (ATC-40, 1996)

The other method of non-linear pushover analysis procedure is Displacement Coefficient Method (DCM).FEMA-273 [11] & FEMA-356 [12] are used as guideline for displacement coefficient method .It is a nonlinear static analysis procedure that provides a numerical process for estimating the displacement demand on the structure, by using a bilinear representation of the capacity curve and a series of modification factors, or coefficients, to calculate a target displacement as shown as figure 2-11. The point on the capacity curve at the target displacement in displacement coefficient method is the equivalent of the performance point in the capacity spectrum method [1].

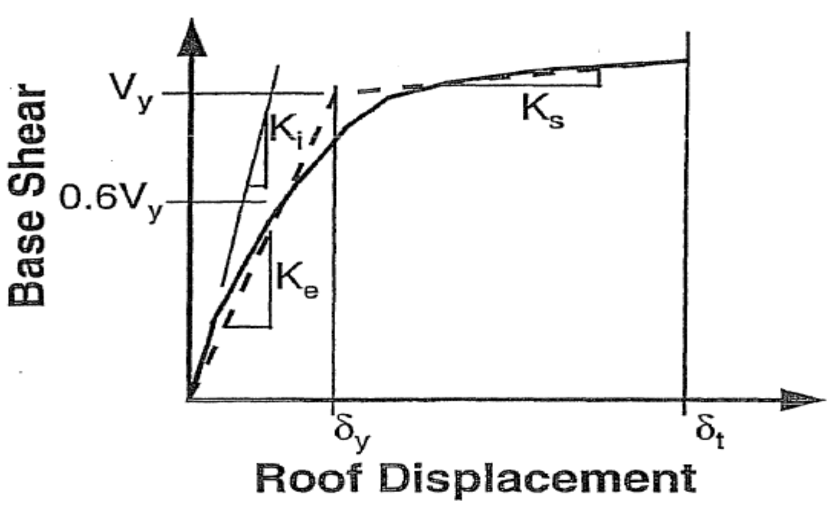


Figure 2-11 Bilinear representation of capacity curve for displacement coefficient method

3.0 MODELING AND ANLAYSIS OF SHEAR WALL ARRANGEMENT

3.1 General

The research methodology was started with problem identification on dual structural system arranged in such way that moment resisting frames resist lateral load in longitudinal direction while shear walls at each end in short direction take lateral load in transverse direction and setting up the objectives of study. All related literature is reviewed and the background information is collected for this research. The major objective of this study is to evaluate the effect of shear wall arrangement in dual structural system which is stated as dangerous combination by Erdey Charles K [7].

Sample buildings which can represent the building type & structural system mentioned above are selected to demonstrate the effect of locating shear wall in different locations. Three types of building structures are established for parametric study. The buildings are assumed to cover the same plan area of 25 x10 meter as shown in figure 3-1 but with different story number and column size. The size of columns increased realistically as the number of story is increased. Each building model has three cases of study based on shear wall arrangement.

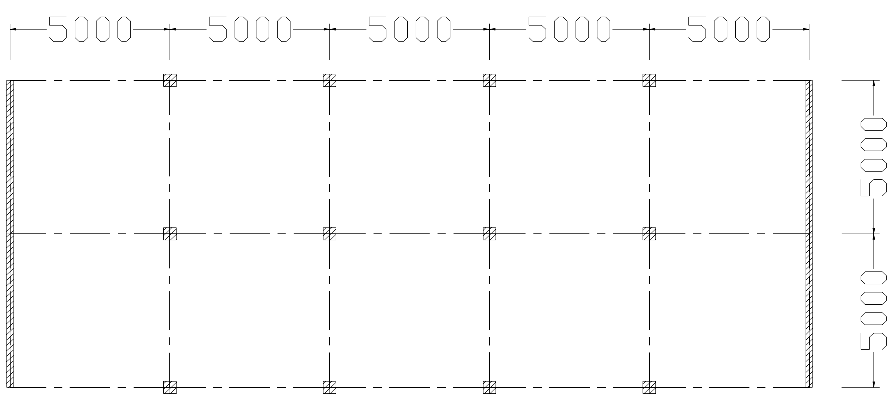


Figure 3-1 Sample building structure used for this study (dimension is in mm)

3.2 Description of the building structure

Three building structures are considered for this study. The detail of each building model is discussed as follows

Building model 1: The first model is a five-story dual structural system with moment resisting frames are assumed to carry lateral loads in longitudinal direction while shear wall at each end in short direction will take lateral loads in the transverse direction as shown in figure 3-1 and figure 3-2.The assumed size of columns in this building are 400 x 400 mm and the size of beams are 400 x 400 mm. The width of shear wall is 200 mm while its length is equal to the length of the building in the short direction [i.e. 10 meter]. It is assumed that the columns and shear walls are rigidly connected to the floor slabs, where as the floor slabs act as rigid diaphragm in both directions. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid.

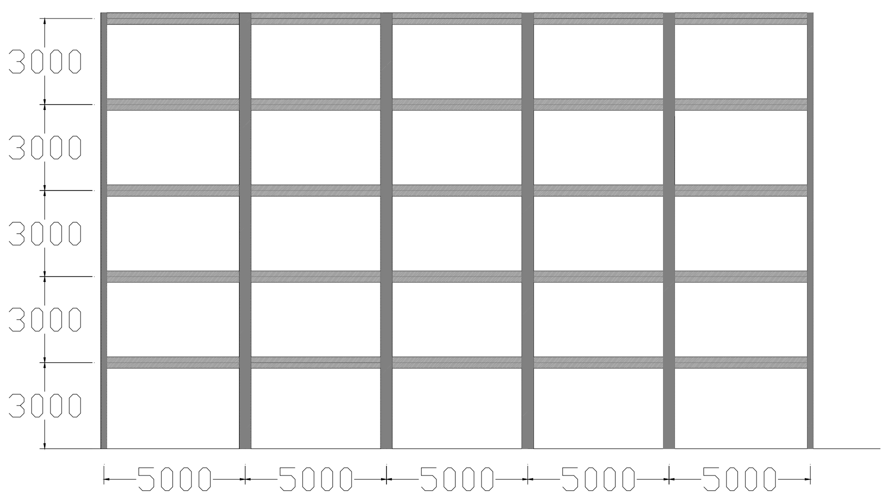


Figure 3-2 Elevation of building model 1 used for this study (dimension is in mm)

Building model 2: It is a ten- story dual structural system with moment resisting frames are assumed to carry lateral loads in longitudinal direction while shear wall at each end in short direction will take lateral loads in the transverse direction as shown in figure 3-1 and figure 3-3.The assumed size of columns in this building are 500 x 500 mm and the size of beams are 400 x 400 mm. The width of shear walls is 200 mm while their length is equal to the length of building in the short direction [i.e. 10 meter].Also it is assumed that columns and shear walls are rigidly connected to the floor slabs, where as the floor slabs act as rigid diaphragm in both directions. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid.

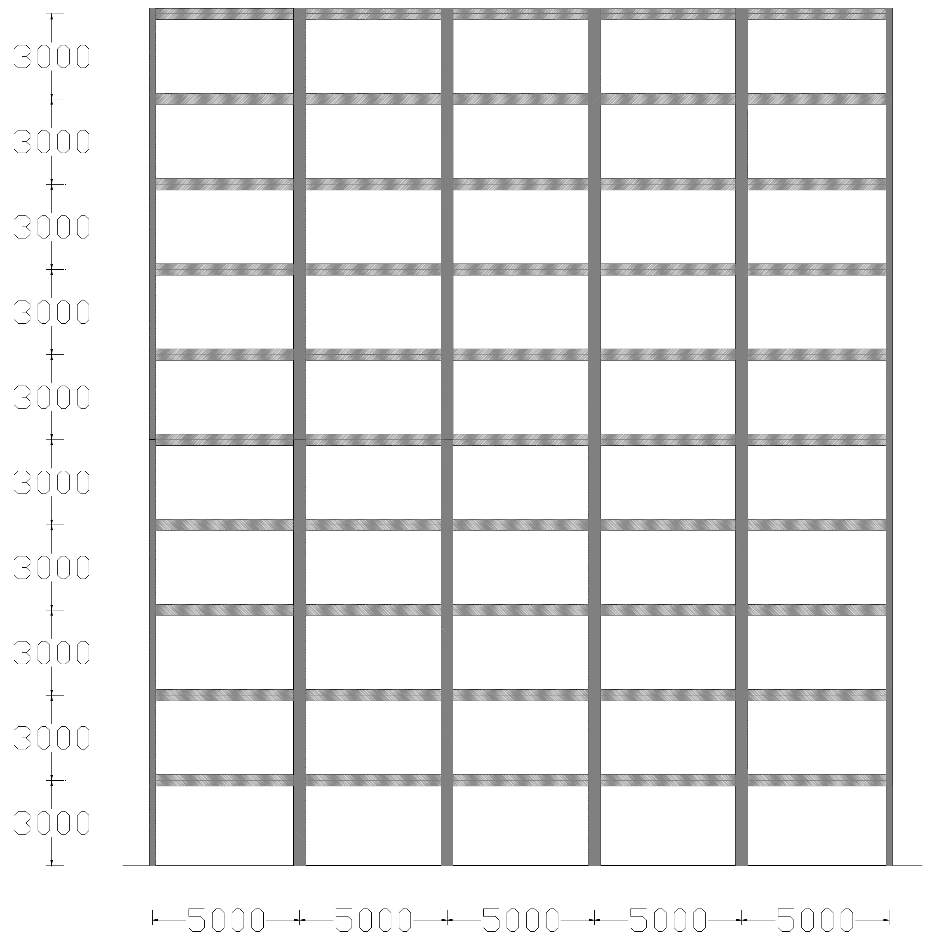


Figure 3-3 Elevation of building model 2 used for this study (dimension is in mm)

Building model 3: It is a twenty-story dual structural system with moment resisting frames are assumed to resist lateral loads in longitudinal direction while shear walls at each end in short direction will carry lateral loads in transverse direction as shown in figure 3-1 and figure 3-3.The assumed size of columns in this building are 600 x 600 mm and the size of beams 400 x 400 mm. Also it is assumed that columns and shear walls are rigidly connected to the floor slabs, where as the floor slab acts as rigid diaphragm in both directions. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid.

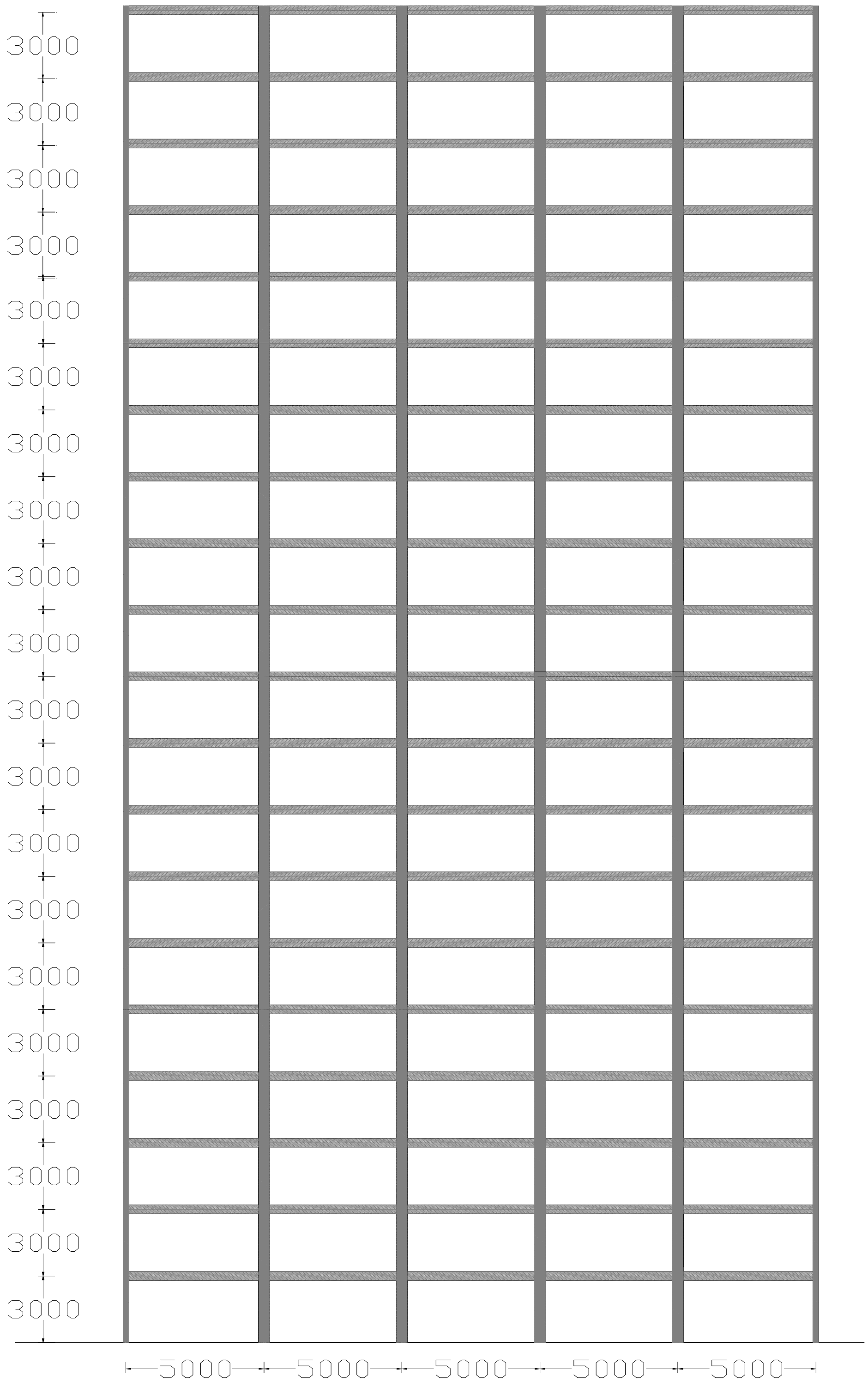


Figure 3-4 Elevation of building model 3 used for this study (dimension is in mm)

3.2.1 Description of case study

Based on the shear wall arrangements, there are three cases of study for each sample building model. In all cases, shear walls are arranged symmetrically, so that torsional effects are not introduced in the sample buildings.

Case I: Dual structural system with shear wall placed at the end of the building in the short direction as shown in figure 3-5. 80% of California high rises hospitals are estimated to be these types of buildings [7].

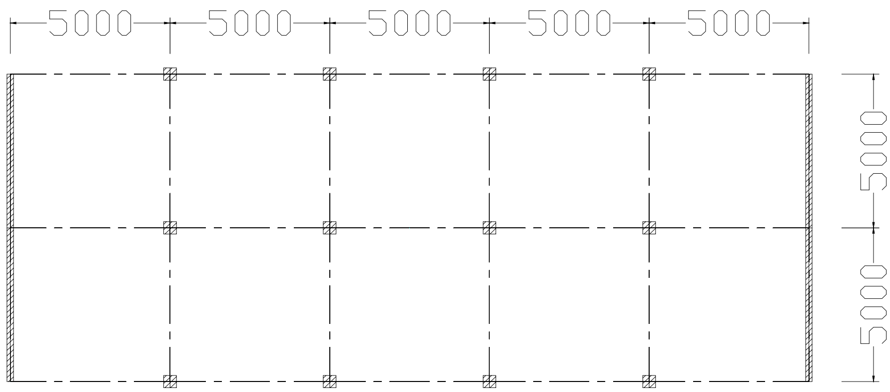


Figure 3-5 Shear walls are placed at both end of the building in the short direction

Case II: Dual structural system with shear wall placed symmetrically at distance of 7.5 m from the centre of mass of the building as shown in figure 3-6.

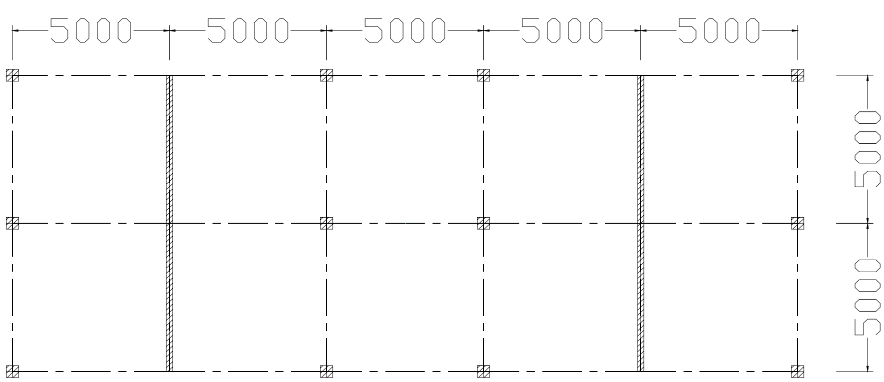


Figure 3-6 Shear walls are placed at 7.5 m far from the centre of mass of the building in the short direction on both sides

Case III: Dual structural system with shear wall placed symmetrically at distance of 2.5 m from the centre of mass of the building as shown in figure 3-7

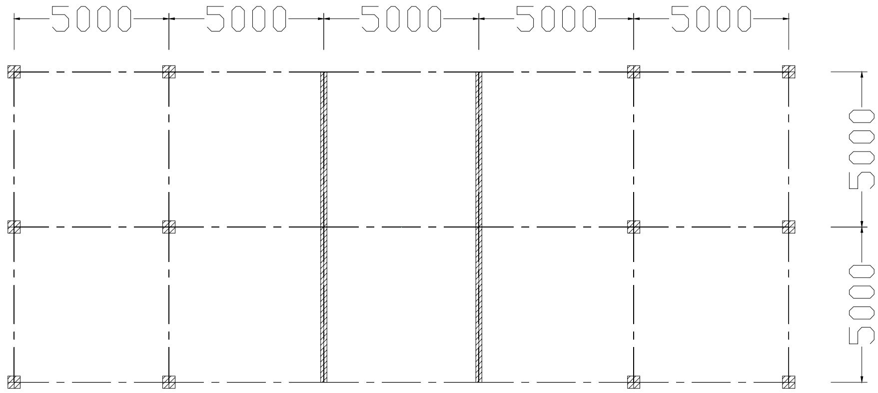


Figure 3-7 Shear walls are placed at 2.5 m far from the centre of mass of the building in the short direction on both sides.

3.2.3 Designing of sample building models

Each sample buildings are designed according to the EBCS-2 [9] and EBCS-8 [10] seismic design requirements. They are assumed as hospital buildings located in high seismic zone, which is zone 4 according to EBCS-8[10]. A subsoil class C is also adopted to obtain the site coefficient S. Equivalent static method is used for obtaining lateral loads. Each building model is designed for the type of Case I shear wall arrangement. Additional Eccentricities in order to cover uncertainties in the location of masses, which may induce accidental torsional effect, are considered in designing the sample building models as specified by EBCS-8[10]. Since the plan areas of the building model floors are the same, all floors will have the same centre of mass at the geometric centre. The point at which lateral load is applied in order to account for accidental torsional effect of each floor is shown table 3.1.Seismic loads are computed based on equivalent static procedure and the distribution of the lateral force over the height of the building is calculated and shown in appendix A.2.

Table 3-1 Additional Eccentricities to account for accidental torsional effect of each floor

Floor level	Centre of mass(X)	Centre of mass(Y)	Accidental Eccentricities in long direction in m (±0.05Li)	Accidental Eccentricities in short direction in m (±0.05Li)
Roof Level	0	0	±0.5	±1.25
Other Floor	0	0	±0.5	±1.25

3.3 Analytical techniques of evaluating the shear wall arrangement

On this research, the effects of shear wall arrangement in the dual system building shown in pervious section is evaluated by comparing the overall seismic performance of the building for each case of sample models. Seismic performance of those sample buildings can be evaluated by carrying out non-linear push over analysis, which could be used to determine the lateral load resisting capacity of structure and the maximum level of damage in the structure at the ultimate load. Therefore the performance based parameters used to evaluate the shear wall arrangements are lateral load resisting capacity curve and plastic hinge mechanism.

In non-linear pushover analysis, the behavior of the structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This is a very convenient representation in practice, and can be visualized easily by the engineer. Using the roof displacement for the capacity curve is a widely accepted practice all over the world. Furthermore, performance point or target displacement is one of non-linear pushover analysis parameters which may used for performance evaluation purpose. The other parameter in pushover analysis, which is used for performance evaluation, is plastic hinge mechanism. The hinging patterns provide information about local and global failure mechanisms in the structure. Also it shows the extent of damage that the structure has suffered in relative to established performance level.

3.4 Modeling approach

Modeling rules presented in chapter nine of ATC-40[1] is used as a guide for modeling the structure considered for this study. SAP 2000 is utilized to run non-linear push over analysis. It is one of the powerful computer programs which have a capability to perform non-linear push over analysis as either force-controlled or displacement-controlled. The other advantage of using SAP2000 is that it considers the effect of geometric non-linearity of the structure (i.e. P-Δ effect) simultaneously with non-linear pushover analysis.

3.4.1 Computation of lateral load and its vertical distribution

Seismic hazard level of Basic Safety Earthquake 1 (BSE-1) specified in FEMA 273 [11] is used for determination of lateral load acting on structure for non-linear pushover analysis. A computed Lateral load is applied with Equivalent Lateral Force (ELF) Distribution pattern or inverted triangular pattern. The distribution of the lateral force over the height of the building is presented in table 3-2 to 3-4.

Table 3-2 Distribution of lateral load over the height of the building model 1

Story Level	Height (h_i)	Dead load (W_i)	W_ih_i	Cvx	*Fx*
Roof Level	15.00	1445.50	21682.50	0.24	1950.70
4th Floor Level	12.00	2345.50	28146.00	0.31	2532.20
3rd Floor Level	9.00	2345.50	21109.50	0.23	1899.15
2nd Floor Level	6.00	2345.50	14073.00	0.15	1266.10
1st Floor Level	3.00	2345.50	7036.50	0.08	633.05
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		13173.00	92047.50		***8281.18***

Table 3-3 Distribution of lateral load over the height of the building model 2

Story Level	Height (h_i)	Dead load (W_i)	$W_ih_i^{1.1}$	Cvx	*Fx*
Roof Level	30.00	1445.50	60932.85	0.13	1177.06
9th Floor Level	27.00	2345.50	88051.26	0.18	1700.91
8th Floor Level	24.00	2345.50	77351.33	0.16	1494.22
7th Floor Level	21.00	2345.50	66784.65	0.14	1290.10
6th Floor Level	18.00	2345.50	56368.33	0.12	1088.88
5th Floor Level	15.00	2345.50	46124.94	0.10	891.01
4th Floor Level	12.00	2345.50	36085.67	0.07	697.08
3rd Floor Level	9.00	2345.50	26296.76	0.05	507.98

Story Level	Height (hi)	Dead load (W_i)	$W_ih_i^{1.3}$	Cvx	*Fx*
2nd Floor Level	6.00	2345.50	16834.56	0.03	325.20
1st Floor Level	3.00	2345.50	7853.60	0.02	151.71
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		24900.50	482683.96		***9324.13***

Table 3-4 Distribution of lateral load over the height of the building model 3

Story Level	Height (h_i)	Dead load (W_i)	$W_ih_i^{1.3}$	Cvx	*Fx*
Roof Level	60.00	1445.50	296220.23	0.07	752.60
19th Floor Level	57.00	2345.50	449648.09	0.11	1142.40
18th Floor Level	54.00	2345.50	419128.63	0.10	1064.86
17th Floor Level	51.00	2345.50	389113.83	0.09	988.61
16th Floor Level	48.00	2345.50	359624.31	0.08	913.68
15th Floor Level	45.00	2345.50	330682.88	0.08	840.15
14th Floor Level	42.00	2345.50	302314.87	0.07	768.08
13th Floor Level	39.00	2345.50	274548.72	0.06	697.54
12th Floor Level	36.00	2345.50	247416.52	0.06	628.60
11th Floor Level	33.00	2345.50	220954.86	0.05	561.37
10th Floor Level	30.00	2345.50	195205.96	0.05	495.95
9th Floor Level	27.00	2345.50	170219.12	0.04	432.47
8th Floor Level	24.00	2345.50	146052.86	0.03	371.07
7th Floor Level	21.00	2345.50	122777.99	0.03	311.94
6th Floor Level	18.00	2345.50	100482.33	0.02	255.29
5th Floor Level	15.00	2345.50	79278.25	0.02	201.42
4th Floor Level	12.00	2345.50	59315.89	0.01	150.70
3rd Floor Level	9.00	2345.50	40808.51	0.01	103.68
2nd Floor Level	6.00	2345.50	24089.74	0.01	61.20
1st Floor Level	3.00	2345.50	9783.47	0.00	24.86
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		48355.50	4237667.05		***10766.48***

3.4.2 Modeling the sample building for non-linear push over analysis

The other basic part of this research is to model the sample buildings so that push over analysis can be carried out. In SAP2000, a frame element is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics to perform non-linear push over analysis of column-beam frame system. But the software has a limitation in analyzing shear walls, therefore the shear walls in dual structure system are represented by equivalent “wide columns”.

According Smith& Coull [16], shear walls connected by beams to other part of the structure can be modeled by vertical stack of beam element located at the centoridal axis of the wall with fictitious rigid horizontal beam elements attached at the framing levels to represent the effect of the wall’s width. Also ATC-40[1] states that solid wall can be represented with equivalent wide column element located at the centreline of the wall using multi, spring models, truss models, or planar finite elements.

Therefore analogous column should have similar axial area, inertia & shear area as that of shear wall. It is important that these fictitious rigid horizontal beams are given properties that are relatively rigid in the plane of each wall panel but not out of plane. To achieve this, the rigid beam with depth equal to the floor-to-floor height is used in this study with adjusting its properties.

The nonlinear behavior of beams and columns is represented by assigning concentrated plastic hinges at member ends where flexural yielding is assumed to occur. Flexural characteristics of beams and columns, defined by moment-rotation relationships, assigned as moment hinges at the ends of the frames. There are three types of hinge properties in SAP2000 [5]. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements. When these hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for each and every hinge [5].

SAP2000 implements the plastic hinge properties described in FEMA-356[13] or ATC-40 [1]. As shown in Figure 3-8, five points labeled A, B, C, D, and E define the force–deformation behavior of a plastic hinge. The values assigned to each of these points vary depending on the type of element, material properties, longitudinal and transverse steel content, and the axial load level on the element [1].

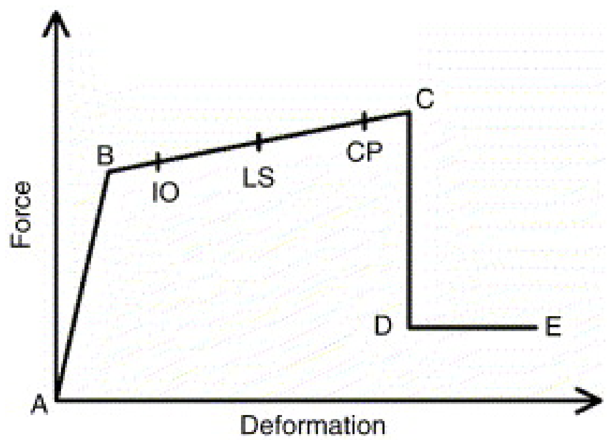


Figure 3-8. Force–deformation relationship of a typical plastic hinge

The definition of user-defined hinge properties requires moment–curvature analysis of each element. The points B and C on Figure 3-8 are related to yield and ultimate curvatures respectively. Since deformation ductility is not a primary concern, the point B is not the focus, and it is obtained using approximate component initial effective stiffness values according to ATC-40 [1]; 0.5EI and 0.70EI for beams and columns, respectively..

Moment-curvature relationships of beams, columns and shear walls were calculated based on the section and material properties given in Appendix B to define the user-defined force-displacement characteristics of the members. The input required for SAP2000 is the moment–rotation relationship instead of the moment–curvature relationship [6]. Therefore, moment–curvature relationship has to be converted to moment–rotation relationship for the five points labeled as A, B, C, D and E shown on figure 3-8. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures.

Thus

$$\theta_p = (\Phi_u - \Phi_y) l_p$$

Where:

l_p: Plastic hinge length
Φ_y: Yield curvature
Φ_u : Ultimate curvature
θ_p : Plastic rotation

Several plastic hinge lengths have been proposed in different literature but Paulay T. and Priestely M.J.N. [15] proposed that plastic hinge length can be approximated as 0.5h where h is a section depth. Also ATC-40 [1] states that the plastic hinge length, lp = h/2 where h is the section depth in the direction of loading, is an acceptable value that usually gives conservative results. In this study, user-defined plastic hinges, which are obtained from Moment-curvature relation, are assigned to frame elements. Plastic hinge length is taken as half of the section depth of the frame element in the direction of loading.

Following the calculation of the ultimate rotation capacity of an element, acceptance criteria, which are labeled as IO, LS, and CP in figure 3-8, are defined. IO, LS, and CP stand for Immediate Occupancy, Life Safety, and Collapse Prevention, respectively. In this study, these three points defined as a point corresponding to 10%, 40%, and 80% use of plastic hinge deformation capacity. Also flexural hinge properties were used and assigned to frames and shear walls because it is intended to evaluate the out-of-plane bending resistance of shear walls in specified type of dual structural system.

4.0 RESULTS AND DISCUSSIONS

4.1 General

The interior frames and shear walls of five, ten and twenty story buildings were considered in pushover analyses to represent dual structural system of reinforced concrete (RC) buildings mentioned in this study. Frame elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns with user defined plastic hinge properties obtained from moment-curvature relation. Shear walls are modeled with equivalent wide column because SAP2000 doesn't carry out pushover analysis for wall elements. Also all buildings are pushed until roof displacement become 4% of the height of the building.

In pushover analysis, the behavior of the structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. Also the plastic hinge formation mechanisms and its pattern will provide information about local and global failure mechanisms in the structure. Consequently the results of non-linear pushover analysis, i.e. capacity curve and plastic hinge mechanisms, are shown and discussed as follows;

4.2 Capacity curves

The three resulting capacity curves or base shear force vs. roof displacement curves for all three cases of arrangement of shear wall in the three buildings are shown in figure 4-1, figure 4-2 and figure 4-3. The three curves of all cases in each building show similar features. They are initially linear but start to deviate from linearity as the beams and the columns undergo inelastic actions. When the buildings are pushed well into the inelastic range, the curves become linear again but with a smaller slope in all three buildings, the capacity curve of shear wall arrangement of Case I is higher than that of Case II and Case III, the capacity curves of shear wall arrangement of Case II and Case III don't have big difference for all three buildings as shown in figure 4-1, figure 4-2 and figure 4-3.

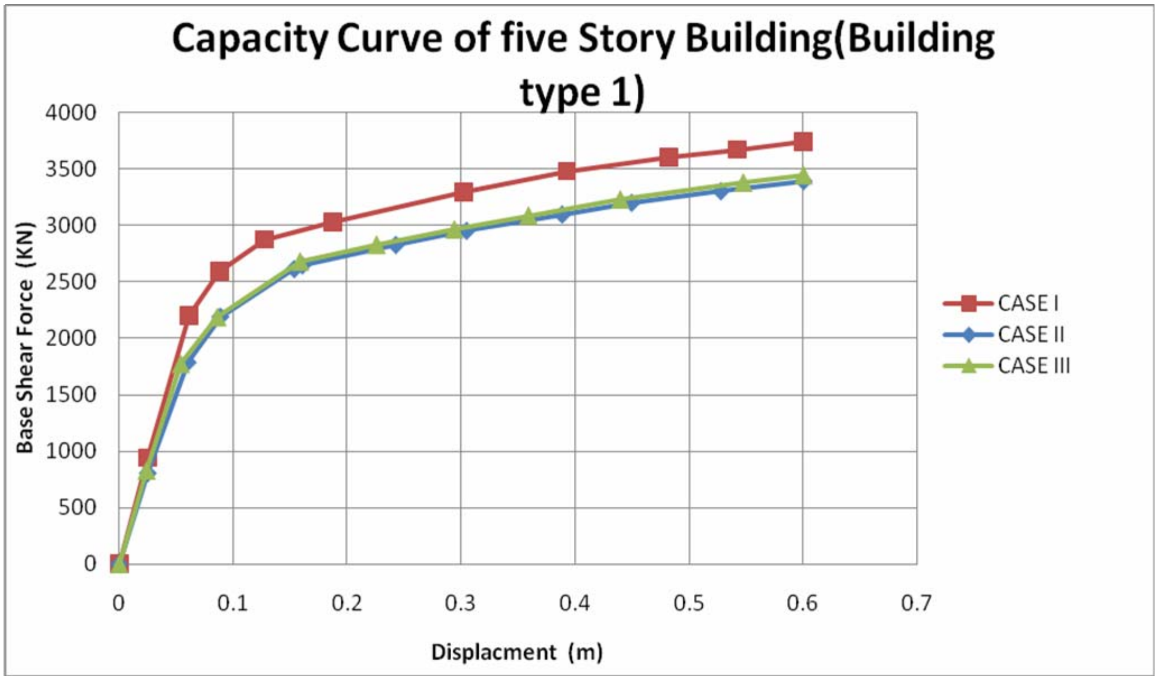


Figure 4-1 capacity curve of all three cases of 5 Story Building model type1

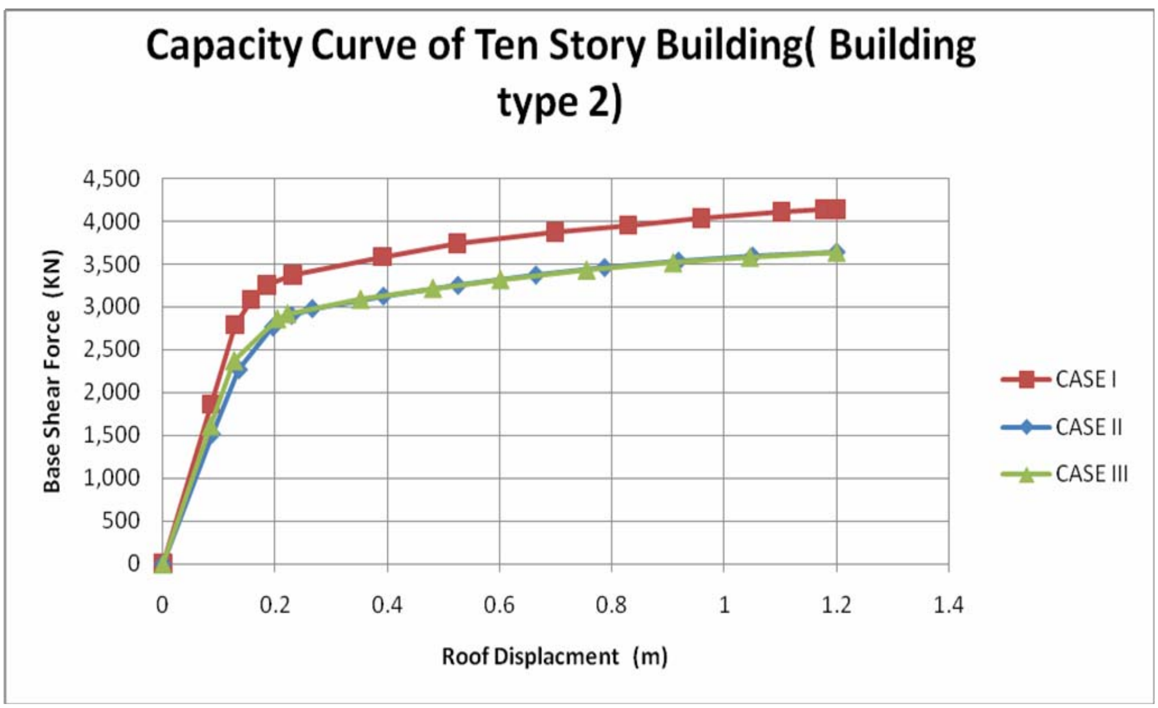


Figure 4-2 capacity curve of all three cases of 10 story Building model type 2

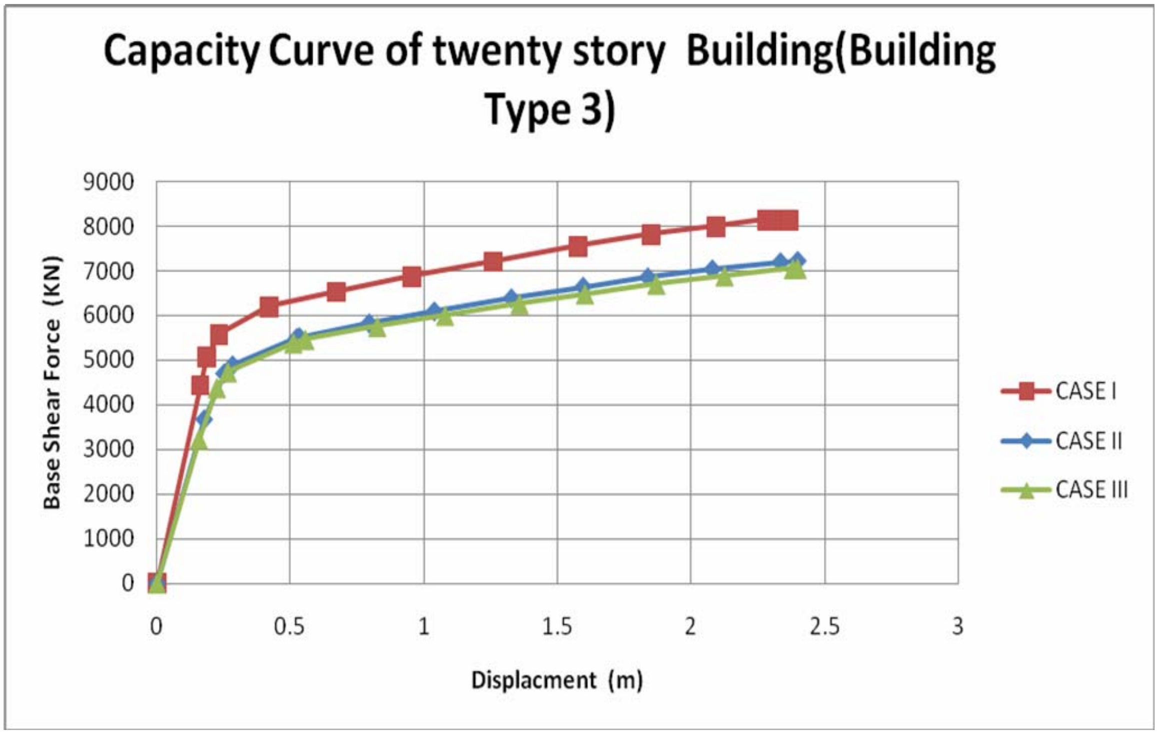


Figure 4-3 capacity curve of all three cases of 20 story Building model type 3

SAP2000 compute the performance points, the points at which demand curve intersects the capacity curve, by capacity spectrum method according to ATC-40. Comparison of performance points is made among all cases of each building. Also the global yielding point of each case is obtained in order to check the proximity of performance point from the elastic range. The global yielding point corresponds to the displacement on the capacity curve where the system starts to soften. Table 4-1, 4-2 and 4-3 shows the performance point and the global yielding point.

Table 4-1 Performance and Global yielding point of all three cases for building type 1

	Performance point		Global Yielding point	
	base Shear force (KN)	displacement(m)	Base Shear force (KN)	displacement(m)
Case 1	2367.019	0.073	2228.5	0.0602
Case 2	2029.146	0.077	1809.5	0.0605
Case 3	1987.548	0.071	1767.8	0.0543

Table 4-2 Performance and Global yielding point of all three cases for building type 2

	Performance point		Global Yielding point	
	Base shear force (KN)	displacement(m)	base Shear force (KN)	displacement(m)
Case 1	2911.0	0.141	2932.8	0.1364
Case 2	2619.7	0.178	2345.2	0.1395
Case 3	2582.6	0.159	2395.6	0.1302

Table 4-3 Performance and Global yielding point of all three cases for building type 3

	Performance point		Global Yielding point	
	base Shear force (KN)	displacement(m)	base Shear force (KN)	displacement(m)
Case 1	5217.01	0.2	5114.6	0.186
Case 2	4143.187	0.211	3840	0.181
Case 3	4356.791	0.223	4187	0.205

4.3 Plastic hinges mechanism

As the building pushed laterally or undergo earthquake-induced lateral displacements, frames and shear walls will experience yielding at the regions where subjected to large moments. They will form plastic hinge due to large inelastic curvatures at those regions. Plastic hinge formation mechanisms and hinging patterns for the three buildings have been obtained at different displacements levels or pushover steps. The hinging patterns are plotted and shown on appendix D. Comparison of the hinging patterns reveals that the patterns for the three buildings are quite similar. Plastic hinge formations start with beam ends, then propagate to upper stories and continue with yielding of columns and shear walls at lower stories. The amounts of damage in the three buildings are similar. in all buildings, columns yielding occur in lower story at events B, IO and LS damage level while shear walls at lower story and beams exhibit yielding at event point B, IO, LS, CP and C damage level. Summary of number of plastic hinges at different damage levels of each case is shown in appendix C.

Plastic hinge patterns of all cases of shear wall arrangement in each building are examined for different pushover steps to get information about local and global failure mechanisms in the structure. The roof displacement corresponding to event point B, IO, LS, CP and C damage level of shear walls at lower story have been obtained. Their magnitudes corresponding to the same damage levels are compared among Case I, Case II and Case III for each building. Table 4-4 shows the roof displacements at different damage levels of shear walls at lower story in out-of-plane direction.

Table 4-4 Computed Roof Displacements for different damage level of shear wall

	Roof Displacements at different Damage level of shear wall (m)			
Building Type 1	Immediate occupancy(IO)	Life safety (LS)	Collapse prevention (CP)	Ultimate curvature(point C)
Case1	0.088	0.188	0.302	0.449
Case2	0.086	0.226	0.388	0.480
Case3	0.089	0.243	0.439	0.540
Building Type2				
Case1	0.231	0.524	0.699	0.919
Case2	0.266	0.526	0.909	1.046
Case3	0.352	0.601	0.909	1.051
Building Type3				
Case1	0.673	0.954	1.577	1.850
Case2	0.795	1.328	1.839	2.080
Case3	0.825	1.358	1.870	2.120

4.4 Discussion of results

From capacity curves and plastic hinge mechanisms of the building models, the following findings were observed:

1. In all buildings, building with Case I arrangement of shear wall has high seismic resistance capacity than the building with Case II and Case III shear wall arrangements while the seismic resistance capacity of building due to shear wall arrangement of Case II and Case III are similar.

2. The performance point, where the demand curve intersects the capacity envelope, of Case I shear wall arrangement is near to the elastic range compared to Case II and Case III shear wall arrangements. Thus, the building with shear wall arrangement of case I has better performance of seismic resistance than others.
3. Plastic hinge mechanism patterns of all cases in three buildings are similar except there is a difference among the magnitude of roof displacements of all cases at the same shear wall‘s damage levels respectively in each building.
4. Comparisons of hinging roof displacement shows that case III shear walls arrangement will undergo large displacement before shear wall yields to event point B, IO, LS, CP and C damage level respectively than case I and case II shear wall arrangements. Also Case II shear wall arrangement will exhibit large hinging roof displacement than case I shear wall arrangement.
5. In all cases of the three buildings, severe rupture beyond collapse prevention (CP) damage level in shear walls at lower story occurs before columns with further pushing of the building. As a result, dual structural building with combination of moment resisting frame to carry lateral load in longitudinal direction while shear walls are placed to resist lateral load in short direction will suffer local failure due to collapse of shear wall in its out-of- plane direction.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The performance of reinforced concrete dual structural system specified in this study is investigated using the non-linear pushover analysis. The effect of placing or arranging shear walls in different locations along longer direction of building is assessed in terms of the whole building performance. After careful inspection and comparison of the output of non-linear pushover analysis for different arrangement of shear walls, the following conclusions are drawn:

- Among the three arrangements of the shear walls, the one with shear walls are placed at each end of the building in the short direction has better seismic resistance capacity of the longitudinal component of earthquake load.
- Despite different arrangement or location of shear walls in this particular dual system, shear walls will experience damage beyond collapse prevention level before columns from large longitudinal component of earth quake load.
- The roof displacements at different damage level of shear walls in out-of- plane direction are influenced by shear wall’s arrangement or location in the building. When the shear walls are placed at the outer side or at each end of the building, they will undergo immediate occupancy (IO), life safety (LS) and Collapse prevention (CP) damage level with smaller roof displacement than the shear wall placed in an interior of dual building system respectively.

Therefore, buildings with dual structural system in this study have better overall seismic resistance when shear walls are placed at each ends of the buildings but with larger risk of local failure of shear walls in out of plane direction at lower stories.

5.2 Recommendations

Dual system with frames are designed to resist lateral load in longitudinal direction while shear walls are intended to carry seismic load in transverse direction is susceptible to local failure of shear wall in out-of- plane direction. Hence, Caution should have to be taken in using this type of dual system in high seismic zone for building like Hospital, Fire brigade and chemical store, in order to reduce the environmental damage and loss of human life during or after earthquake. It is advisable to carry out evaluation of overall seismic performance and local failure mechanism of already constructed building with this type of dual system based on its importance factor. According to the evaluation results and performance objectives of the building, it should be maintained or retrofitted.

Shear walls are considered as secondary elements in their out-of -plane direction because they resist or carry inconsiderable amount of lateral loads due to small stiffness. Considering only primary structural element of the building in design and analysis of dual system specified above is unsafe since local failure of shear wall in out-of-plane direction will leave the building defence less to transversal components of earthquake load. Therefore, engineers should have to make sure that local failure didn't occur in out-of-plane direction of shear walls for required performance objective in this particular type of dual system building. Also they should have to check the effect of interaction of shear walls with frames, after they undergo in to plastic range, for any type of dual system building they are going to design.

This study engaged in evaluating the effect of different shear wall arrangement in reinforced concrete dual system where moment resisting frames carry lateral load in longitudinal direction while shear walls resist lateral loads in transverse direction. Hence, an extensive study containing other types of dual system could be carried out to enhance the knowledge regarding the interaction between frames and shear walls, mode of sharing the resistance to lateral loads in inelastic range and the effect of development of plastic hinge in out-of-plane direction of shear walls. Also only the longitudinal components of earthquake loads are used to evaluate the effect of different shear wall arrangement on the overall performance of the building in this study.

Therefore, the effect due to transversal components of earthquake loads on this particular type of dual system could be investigated in the extended research.

This study is limited by assigning solid shear walls in all cases of arrangement while traditionally shear wall arrangement in Case II and Case III must consist of two coupled shear walls due to functional requirement. This limitation doesn’t change the result of this study but in extended study, the effect of coupling beam and coupled shear wall could be introduced.

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APPENDIX A
LATERAL LOAD COMPUTATION FOR DESIGNING AND NON-LINEAR PUSHOVER ANALYSIS

A.1 DETERMINATION OF CHARACTERISTICS DEAD LOAD OF THE BUILDINGS

Project :	Hosiptal			Page :	1
Client :	For the Purpose of studying			Designed By :	Abel Yoseph
Project No. :	Building model I	Date :	December,2008	Checked By :	

EARTH QUAKE ANALYSIS (USING EBCS-8:1995)

Determination of Characteristic Dead Load of the Building

(A) GROUND FLOOR LEVEL

HCB WALL, EXTERNAL	
Unit Weight,γ =	20.00
Wall Length, all =	50.00
Wall Height =	3.00
Wall Thickness =	0.20
Weight (kN) =	600.00

BEAMS		COLUMNS	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Beam Length, all =	115.00	Column Length, all =	12.00
Beam Depth =	0.40	Column Depth =	0.40
Beam Width =	0.40	Column Width =	0.40
Weight (kN) =	460.00	Weight (kN) =	48.00

SHEAR WALL		SLAB	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Sh. Wall Length, all =	20.00	Slab Length =	25.00
Sh. Wall Height =	3.00	Slab Width =	10.00
Sh. Wall Thickness =	0.20	Slab Thicknes =	0.15
Weight (kN) =	300	Weight (kN) =	937.50

TOTAL DEAD LOAD = 2345.50 KN

(B) TYPICAL FLOOR LEVEL

HCB WALL	
Unit Weight,γ =	20.00
Wall Length, all =	50.00
Wall Height =	3.00
Wall Thicknes =	0.20
Weight (kN) =	600.00

BEAMS		COLUMNS	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Beam Length, all =	115.00	Column Length, all =	12.00
Beam Depth =	0.40	Column Depth =	0.40
Beam Width =	0.40	Column Width =	0.40
Weight (kN) =	460.00		48.00

SHEAR WALL		SLAB	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Sh. Wall Length, all =	20.00	Slab Length =	25.00
Sh. Wall Height =	3.00	Slab Width =	10.00
Sh. Wall Thickness =	0.20	Slab Thicknes =	0.15
Weight (kN) =	300	Weight (kN) =	937.50

TOTAL DEAD LOAD = 2345.50 KN

(C) ROOF LEVEL

HCB WALL	
Unit Weight,γ =	20.00
Wall Length, all =	0.00
Wall Height =	3.00
Wall Thicknes =	0.20
Weight (kN) =	0.00

BEAMS		COLUMNS	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Beam Length, all =	115.00	Column Length, all =	12.00
Beam Depth =	0.40	Column Depth =	0.40
Beam Width =	0.40	Column Width =	0.40
Weight (kN) =	460.00	Weight (kN) =	48.00

SHEAR WALL		SLAB	
Unit Weight,γ =	25.00	Unit Weight,γ =	25.00
Sh. Wall Length, all =	0.00	Slab Length =	25.00
Sh. Wall Height =	3.00	Slab Width =	10.00
Sh. Wall Thickness =	0.20	Slab Thicknes =	0.15
Weight (kN) =	0	Weight (kN) =	937.50

TOTAL DEAD LOAD = 1445.50 KN

CALCULATION SHEET

A.2 COMPUTATION OF LATERAL LOAD FOR DESIGNING AND ITS DISTRUBUTION

A.2.1 FIVE STORY BUILDING (BUILDING MODEL1)

Project :	Hospital			Page :	1 of Earthquake Analysis
Client :	For the Purpose of studying			Designed By :	Abel Yoseph
Project No. :	Building model I	Date :	December,2008	Checked By :	

Earthquake Analysis

Location of the Building = Awassa

Structural Regularity =	Plan	Yes!	
	Elevation	Yes!	**Use Static Analysis**

Zone	4		(EBCS 8-1995 TABLE 1.3)
$\alpha o=$	0.10	Bedrock Acceleration.	(EBCS 8-1995 TABLE 1.1)
I=	1.40	Importance Factor	(EBCS 8-1995 TABLE 2.4)
$\alpha = \alpha_o I =$	0.14		(EBCS 8-1995 ARTICLE 1.4.2.2(5))
S =	1.50	Site Coefficient from Subsoil Condition	(EBCS 8-1995 TABLE 1.2)
H =	15.00	Height of the Building.	
$C_1 =$	0.050		(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$T_1=C_1H^{3/4} =$	0.381	Fundamental Period of Building ,in sec <2sec OK!!	(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$\beta = 1.2S/T^{2/3} \leq 2.5 \Rightarrow$	2.50	Design Response Factor	(EBCS 8-1995 ARTICLE 1.4.2.2(6))
$\gamma_o=$	0.20	Basic Value of Behavior Factor	(EBCS 8-1995 TABLE 3.2)
$K_D=$	2.00	Factor Reflecting Ductility Class	(EBCS 8-1995 ARTICLE 3.3.2.1(4))
$K_R=$	1.00	Factor Reflecting Regularity in Elevation	(EBCS 8-1995 ARTICLE 3.3.2.1(6))
$\alpha o=\Sigma Hw/lw =$	1.50	$Kw=0.5\alpha o =$ 0.75 ≤ 2.5	
$K_W=$	1.50	Factor Reflecting Prevailing Failure Mode	(EBCS 8-1995 ARTICLE 3.3.2.1(7))
$\gamma = \gamma_o K_D K_R K_W \leq 0.70$	0.60	Behaviour Factor	(EBCS 8-1995 ARTICLE 3.3.2.1(1))

$\mathbf{S_d(T_1) = \alpha\beta\gamma =}$	0.21	Design Spectrum	(EBCS 8-1995 ARTICLE 1.4.2.2(4))
$\mathbf{F_b = S_d(T_1)W =}$	2766.33	Seismic Base Shear Force	(EBCS 8-1995 ARTICLE 2.3.3.2.2 (1))
$\mathbf{F_t = 0.07T_1F_b =}$	73.80	Concenterated Force at Top	(EBCS 8-1995 ARTICLE 2.3.3.2.3 (2))

Base Shear Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	W_ih_i	F_i
Roof Level	15.00	1445.50	21682.50	708.04
4th Floor Level	12.00	2345.50	28146.00	823.31
3rd Floor Level	9.00	2345.50	21109.50	617.49
2nd Floor Level	6.00	2345.50	14073.00	411.66
1st Floor Level	3.00	2345.50	7036.50	205.83
Ground Floor Level	0.00	2345.50	0.00	0.00
		13173.00	92047.50	2766.33

CALCULATION SHEET

A.2.2 TEN STORY BUILDING (BUILDING MODEL2)

Project :	Hospital			Page :	2 of Earthquake Analysis
Client :	For the Purpose of studying			Designed By :	Abel Yoseph
Project No. :	Building Model II	Date :		Checked By :	

Earthquake Analysis

Location of the Building = Awasa

Structural Regularity = Plan Yes!

Elevation Yes! **Use Static Analysis**

Zone	4		(EBCS 8-1995 TABLE 1.3)
$\alpha o=$	0.10	Bedrock Acceleration.	(EBCS 8-1995 TABLE 1.1)
I=	1.40	Importance Factor	(EBCS 8-1995 TABLE 2.4)
$\alpha = \alpha_o I =$	0.14		(EBCS 8-1995 ARTICLE 1.4.2.2(5))
S =	1.50	Site Coefficient from Subsoil Condition	(EBCS 8-1995 TABLE 1.2)
H =	30.00	Height of the Building.	
$C_1 =$	0.050		(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$T_1=C_1H^{3/4} =$	0.641	Fundamental Period of Building ,in sec <2sec OK!!	(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$\beta = 1.2S/T^{2/3} \leq 2.5 \Rightarrow$	2.42	Design Response Factor	(EBCS 8-1995 ARTICLE 1.4.2.2(6))
$\gamma_o=$	0.20	Basic Value of Behavior Factor	(EBCS 8-1995 TABLE 3.2)
$K_D=$	2.00	Factor Reflecting Ductility Class	(EBCS 8-1995 ARTICLE 3.3.2.1(4))
$K_R=$	1.00	Factor Reflecting Regularity in Elevation	(EBCS 8-1995 ARTICLE 3.3.2.1(6))
$\alpha o=\Sigma Hw/lw =$	3.00	$Kw=0.5\alpha o =$ 1.5 ≤ 2.5	
$K_W=$	1.50	Factor Reflecting Prevailing Failure Mode	(EBCS 8-1995 ARTICLE 3.3.2.1(7))
$\gamma = \gamma_o K_D K_R K_W \leq 0.70$	0.60	Behaviour Factor	(EBCS 8-1995 ARTICLE 3.3.2.1(1))

$S_d(T_1) = \alpha\beta\gamma =$	0.20	Design Spectrum	(EBCS 8-1995 ARTICLE 1.4.2.2(4))
$F_b = S_d(T_1)W =$	5065.44	Seismic Base Shear Force	(EBCS 8-1995 ARTICLE 2.3.3.2.2 (1))
$F_t = 0.07T_1F_b =$	227.26	Concenterated Force at Top	(EBCS 8-1995 ARTICLE 2.3.3.2.3 (2))

Base Shear Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	W_ih_i	F_i
Roof Level	30.00	1445.50	43365.00	810.05
9th Floor Level	27.00	2345.50	63328.50	851.08
8th Floor Level	24.00	2345.50	56292.00	756.51
7th Floor Level	21.00	2345.50	49255.50	661.95
6th Floor Level	18.00	2345.50	42219.00	567.39
5th Floor Level	15.00	2345.50	35182.50	472.82
4th Floor Level	12.00	2345.50	28146.00	378.26
3rd Floor Level	9.00	2345.50	21109.50	283.69
2nd Floor Level	6.00	2345.50	14073.00	189.13
1st Floor Level	3.00	2345.50	7036.50	94.56
Ground Floor Level	0.00	2345.50	0.00	0.00
		24900.50	360007.50	**5065.44**

CALCULATION SHEET

A.2.3 TWENTY STORY BUILDING (BUILDING MODEL3)

Project :	Hospital			Page :	3 of Earthquake Analysis
Client :	For the Purpose of studying			Designed By :	Abel Yoseph
Project No. :	Building model III	Date :	December,2008	Checked By :	

Earthquake Analysis

Location of the Building = Awasa

Structural Regularity = Plan Yes!

Elevation Yes! **Use Static Analysis**

Zone	4		(EBCS 8-1995 TABLE 1.3)
αo=	0.10	Bedrock Acceleration.	(EBCS 8-1995 TABLE 1.1)
I=	1.40	Importance Factor	(EBCS 8-1995 TABLE 2.4)
$\alpha = \alpha_o I$ =	0.14		(EBCS 8-1995 ARTICLE 1.4.2.2(5))
S =	1.50	Site Coefficient from Subsoil Condition	(EBCS 8-1995 TABLE 1.2)
H =	60.00	Height of the Building.	
C_1 =	0.050		(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$T_1 = C_1 H^{3/4}$ =	1.078	Fundamental Period of Building ,in sec <2sec OK!!	(EBCS 8-1995 ARTICLE 2.3.3.2.2)
$\beta = 1.2S/T^{2/3} \leq 2.5 \Rightarrow$	1.71	Design Response Factor	(EBCS 8-1995 ARTICLE 1.4.2.2(6))
γ_o=	0.20	Basic Value of Behavior Factor	(EBCS 8-1995 TABLE 3.2)
K_D=	2.00	Factor Reflecting Ductility Class	(EBCS 8-1995 ARTICLE 3.3.2.1(4))
K_R=	1.00	Factor Reflecting Regularity in Elevation	(EBCS 8-1995 ARTICLE 3.3.2.1(6))
$\alpha o = \Sigma Hw/lw$ =	6.00	$Kw = 0.5\alpha o$ = 3 ≤ 2.5	
K_W=	1.50	Factor Reflecting Prevailing Failure Mode	(EBCS 8-1995 ARTICLE 3.3.2.1(7))
$\gamma = \gamma_o K_D K_R K_W \leq 0.70$	0.60	Behaviour Factor	(EBCS 8-1995 ARTICLE 3.3.2.1(1))

$S_d(T_1) = \alpha\beta\gamma$ =	0.14	Design Spectrum	(EBCS 8-1995 ARTICLE 1.4.2.2(4))
$F_b = S_d(T_1)W$ =	6954.48	Seismic Base Shear Force	(EBCS 8-1995 ARTICLE 2.3.3.2.2 (1))
$F_t = 0.07T_1F_b$ =	524.74	Concenterated Force at Top	(EBCS 8-1995 ARTICLE 2.3.3.2.3 (2))

Base Shear Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	W_ih_i	F_i
Roof Level	60.00	1445.50	86730.00	916.44
19th Floor Level	57.00	2345.50	133693.50	603.80
18th Floor Level	54.00	2345.50	126657.00	572.02
17th Floor Level	51.00	2345.50	119620.50	540.25
16th Floor Level	48.00	2345.50	112584.00	508.47
15th Floor Level	45.00	2345.50	105547.50	476.69
14th Floor Level	42.00	2345.50	98511.00	444.91
13th Floor Level	39.00	2345.50	91474.50	413.13
12th Floor Level	36.00	2345.50	84438.00	381.35
11th Floor Level	33.00	2345.50	77401.50	349.57
10th Floor Level	30.00	2345.50	70365.00	317.79
9th Floor Level	27.00	2345.50	63328.50	286.01
8th Floor Level	24.00	2345.50	56292.00	254.23
7th Floor Level	21.00	2345.50	49255.50	222.45
6th Floor Level	18.00	2345.50	42219.00	190.67
5th Floor Level	15.00	2345.50	35182.50	158.90
4th Floor Level	12.00	2345.50	28146.00	127.12
3rd Floor Level	9.00	2345.50	21109.50	95.34
2nd Floor Level	6.00	2345.50	14073.00	63.56
1st Floor Level	3.00	2345.50	7036.50	31.78
Ground Floor Level	0.00	2345.50	0.00	0.00
		48355.50	1423665.00	6954.48

CALCULATION SHEET

A.3 COMPUTATION OF LATERAL LOAD FOR PUSHOVER ANALYSIS AND ITS VERTICAL DISTRUBUTION

A.3.1 FIVE STORY BUILDING (BUILDING MODEL1)

Project :	**Hospital**	**Page :**	**1 of PUSHOVER ANALYSIS**
Client :	**For the Purpose of studying**	**Designed By :**	**Abel Yoseph**
Project No. :	**Building model I** **Date :** **December,2008**	**Checked By :**	

PUSH OVER ANALYSIS

$H =$ 15.00 Height of the Building.

$C_1 =$ 0.050

$T_1 = C_1H^{3/4} =$ 0.381 → $T \leq 0.5$ sec Therefore, k=1.0 (FEMA273-1997 ARTICLE 3.3.1.2)

Assumption: BSE-1 (10%/50 year) is used.

Zones of High Seismicity (S_{xs} & S_{x1} interms of g) $S_{xs} \geq$ 0.5 $S_{x1} \geq$ 0.2 (FEMA273-1997 ARTICLE 2.6.3.1)

A 5% damped response spectrum is used → $B_s =$ 1.0 $B_1 =$ 1.0 (FEMA273-1997 ARTICLE 2.6.1.5,Table 2-15)

$T_o = S_{x1}Bs/ SxsB1 =$ 0.400 Transition period of Response Spectrum (FEMA273-1997 ARTICLE 2.6.1.5,EQ 2-10)

For 0.2To<T<To, **Sa** = Sxs/ Bs = 0.500 Response spectrum acceleration (FEMA273-1997 ARTICLE 2.6.1.5)

For 0.1<T< To , $C_1 =$ 1.032 Modification factor (FEMA273-1997 ARTICLE 3.3.1.3 (A))

For 0.1<T< To , $C_2 =$ 1.219 Modification factor to stiffness degrdation (FEMA273-1997 ARTICLE 3.3.1.3 (A))

$C_3 =$ 1.000 Modification factor to P-Δ effects

$V = C_1C_2C_3S_aW =$ **8281.18** KN (FEMA273-1997 ARTICLE 3.3.1.3 (A))

$F_x = C_{vx}V$

$$Cvx = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i} \longrightarrow Cvx = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

(FEMA273-1997 ARTICLE 3.3.1.2 (B))

Lateral Force Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	W_ih_i	Cvx	*Fx*
Roof Level	15.00	1445.50	21682.50	0.24	1950.70
4th Floor Level	12.00	2345.50	28146.00	0.31	2532.20
3rd Floor Level	9.00	2345.50	21109.50	0.23	1899.15
2nd Floor Level	6.00	2345.50	14073.00	0.15	1266.10
1st Floor Level	3.00	2345.50	7036.50	0.08	633.05
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		13173.00	92047.50		***8281.18***

CALCULATION SHEET

A.3.2 TEN STORY BUILDING (BUILDING MODEL2)

Project :	**Hospital**	**Page :**	**2 of PUSHOVER ANALYSIS**
Client :	**For the Purpose of studying**	**Designed By :**	**Abel Yoseph**
Project No. :	**Building Model II**	**Checked By :**	

EARTHQUAKE LOAD FOR PUSH OVER ANALYSIS

$H =$ 30.00 Height of the Building.

$C_1 =$ 0.050

$T_1 = C_1 H^{3/4} =$ 0.641 → $T \geq 0.5$ sec Therefore, k= 1.1 (FEMA273-1997 ARTICLE 3.3.1.2)

Assumption: BSE-1 (10%/50 year) is used.

Zones of High Seismicity (S_{xs} & S_{x1} interms of g) $S_{xs} \geq$ 0.5 $S_{x1} \geq$ 0.2 (FEMA273-1997 ARTICLE 2.6.3.1)

A 5% damped response spectrum is used → $B_s =$ 1.0 $B_1 =$ 1.0 (FEMA273-1997 ARTICLE 2.6.1.5,Table 2-15)

$T_o = S_{x1} Bs / SxsB1 =$ 0.400 Transition period of Response Spectrum (FEMA273-1997 ARTICLE 2.6.1.5,EQ 2-10)

For T>To, **Sa** = Sx1/ (B1T)= 0.312 Response spectrum acceleration (FEMA273-1997 ARTICLE 2.6.1.5)

For T> To , $C_1 =$ 1.000 Modification factor (FEMA273-1997 ARTICLE 3.3.1.3 (A))

For T> To , $C_2 =$ 1.200 Modification factor to stiffness degrdation (FEMA273-1997 ARTICLE 3.3.1.3 (A))

$C_3 =$ 1.000 Modification factor to P-Δ effects

$V = C_1 C_2 C_3 S_a W =$ 9324.13 KN (FEMA273-1997 ARTICLE 3.3.1.3 (A))

$F_x = C_{vx} V$

$$Cvx = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \longrightarrow Cvx = \frac{w_x h_x^{1.1}}{\sum_{i=1}^{n} w_i h_i^{1.1}}$$

(FEMA273-1997 ARTICLE 3.3.1.3 (B))

Lateral Shear Force Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	$W_i h_i^{1.1}$	Cvx	*Fx*
Roof Level	30.00	1445.50	60932.85	0.13	1177.06
9th Floor Level	27.00	2345.50	88051.26	0.18	1700.91
8th Floor Level	24.00	2345.50	77351.33	0.16	1494.22
7th Floor Level	21.00	2345.50	66784.65	0.14	1290.10
6th Floor Level	18.00	2345.50	56368.33	0.12	1088.88
5th Floor Level	15.00	2345.50	46124.94	0.10	891.01
4th Floor Level	12.00	2345.50	36085.67	0.07	697.08
3rd Floor Level	9.00	2345.50	26296.76	0.05	507.98
2nd Floor Level	6.00	2345.50	16834.56	0.03	325.20
1st Floor Level	3.00	2345.50	7853.60	0.02	151.71
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		24900.50	482683.96		***9324.13***

CALCULATION SHEET

A.3.3 TEN STORY BUILDING (BUILDING MODEL2)

Project :	Hospital			Page :	3 of PUSHOVER ANALYSIS
Client :	For the Purpose of studying			Designed By :	Abel Yoseph
Project No. :	Building model III	Date :	December,2008	Checked By :	

PUSH OVER ANALYSIS

H = 60.00 Height of the Building.

C_1 = 0.050

$T_1=C_1H^{3/4}$ = 1.078 → T≥0.5 sec Therefore, k= 1.3 (FEMA273-1997 ARTICLE 3.3.1.2)

Assumption: BSE-1 (10%/50 year) is used.

Zones of High Seismicity (S_{xs} & S_{x1}interms of $S_{xs}\geq$ 0.5 $S_{x1}\geq$ 0.2 (FEMA273-1997 ARTICLE 2.6.3.1)

A 5% damped response spectrum is used → B_s= 1.0 B_1= 1.0 (FEMA273-1997 ARTICLE 2.6.1.5,Table 2-15)

$T_o=S_{x1}Bs/\ SxsB1$= 0.400 Transition period of Response Spectrum (FEMA273-1997 ARTICLE 2.6.1.5,EQ 2-10)

For T>To, **Sa** = Sx1/ (B1T)= 0.186 Response spectrum acceleration (FEMA273-1997 ARTICLE 2.6.1.5)

For T> To , C_1= 1.000 Modification factor (FEMA273-1997 ARTICLE 3.3.1.3 (A))

For T> To , C_2= 1.200 Modification factor to stiffness degrdation (FEMA273-1997 ARTICLE 3.3.1.3 (A))

C_3= 1.000 Modification factor to P-Δ effects

$V=C_1C_2C_3S_aW$ = <u>10766.48</u> KN (FEMA273-1997 ARTICLE 3.3.1.3 (A))

$F_x = C_{vx}V$

$$Cvx = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \longrightarrow Cvx = \frac{w_x h_x^{1.3}}{\sum_{i=1}^{n} w_i h_i^{1.3}}$$

(FEMA273-1997 ARTICLE 3.3.1.3 (B))

Psedo Lateral Shear Force Distiributed over the Height of the Building

Story Level	Height (h_i)	Dead load (W_i)	$W_ih_i^{1.3}$	Cvx	*Fx*
Roof Level	60.00	1445.50	296220.23	0.07	752.60
19th Floor Level	57.00	2345.50	449648.09	0.11	1142.40
18th Floor Level	54.00	2345.50	419128.63	0.10	1064.86
17th Floor Level	51.00	2345.50	389113.83	0.09	988.61
16th Floor Level	48.00	2345.50	359624.31	0.08	913.68
15th Floor Level	45.00	2345.50	330682.88	0.08	840.15
14th Floor Level	42.00	2345.50	302314.87	0.07	768.08
13th Floor Level	39.00	2345.50	274548.72	0.06	697.54
12th Floor Level	36.00	2345.50	247416.52	0.06	628.60
11th Floor Level	33.00	2345.50	220954.86	0.05	561.37
10th Floor Level	30.00	2345.50	195205.96	0.05	495.95
9th Floor Level	27.00	2345.50	170219.12	0.04	432.47
8th Floor Level	24.00	2345.50	146052.86	0.03	371.07
7th Floor Level	21.00	2345.50	122777.99	0.03	311.94
6th Floor Level	18.00	2345.50	100482.33	0.02	255.29
5th Floor Level	15.00	2345.50	79278.25	0.02	201.42
4th Floor Level	12.00	2345.50	59315.89	0.01	150.70
3rd Floor Level	9.00	2345.50	40808.51	0.01	103.68
2nd Floor Level	6.00	2345.50	24089.74	0.01	61.20
1st Floor Level	3.00	2345.50	9783.47	0.00	24.86
Ground Floor Level	0.00	2345.50	0.00	0.00	0.00
		<u>48355.50</u>	<u>4237667.05</u>		***10766.48***

CALCULATION SHEET

APPENDIX B
SECTIONS, THEIR MOMENT –CURVATURE RELATIONS AND ASSIGNED PLASTIC HINGE PROPERTIES

B.1 FIVE STORY BUILDING (BUILDING MODEL1)

- Shear wall

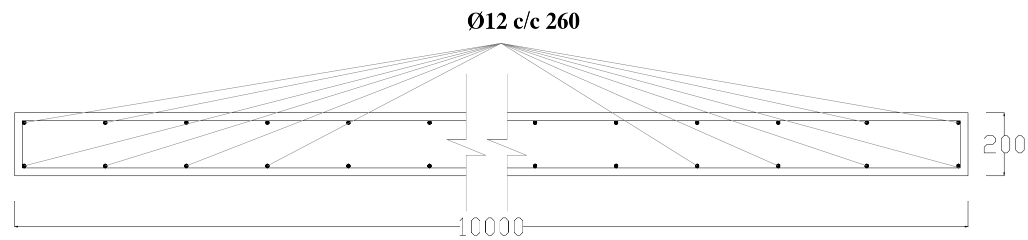


Figure B.1.1 Section for shear wall

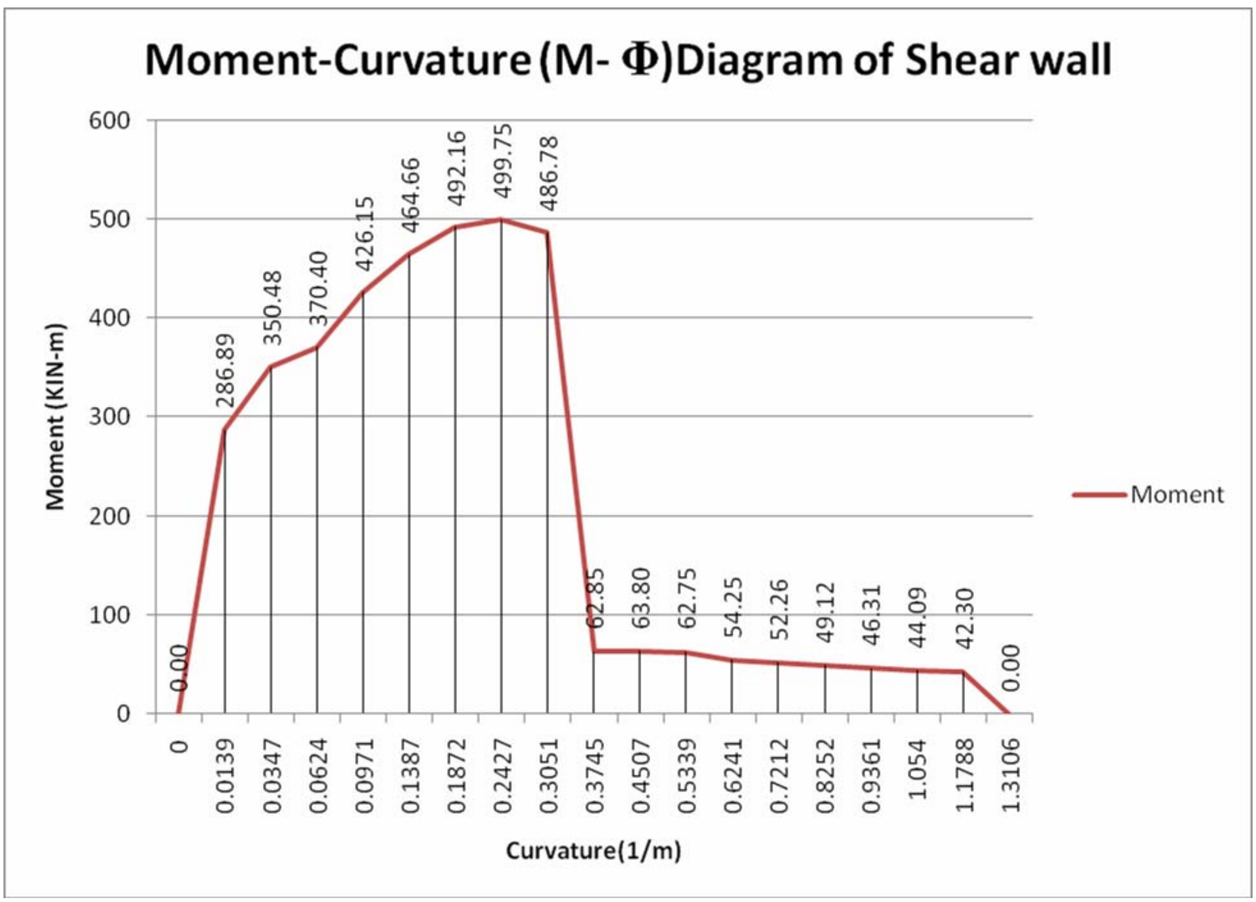


Figure B.1.2 Moment –Curvature (M-Φ) Diagram for shear wall in out-of-plane direction

Table B.1.1 Assigned plastic hinge properties of shear wall in out-of-plane direction

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	286.89	1.00	0.01
IO*	0.03			0.03
LS*	0.12			0.12
CP*	0.24			0.24
C	0.31	486.78	1.70	0.31
D	0.37	62.85	0.22	0.37
E	0.72	62.00	0.22	0.72
Yield Moment (Cf)		287		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Column

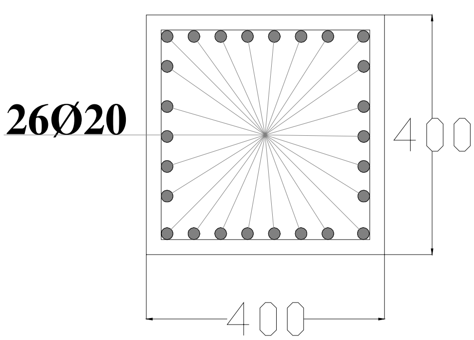


Figure B.1.3 Section for column

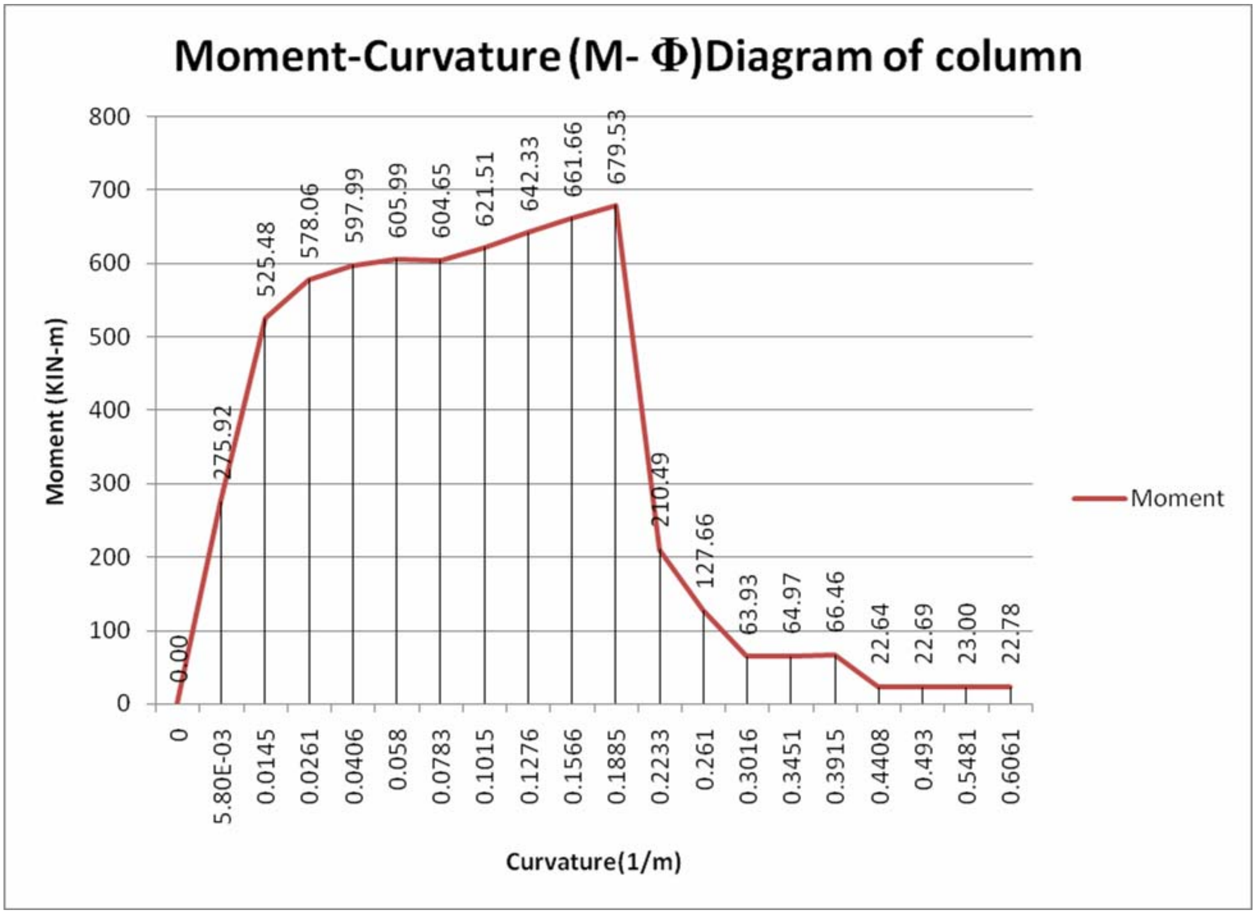


Figure B.1.4 Moment –Curvature (M-Φ) Diagram for column

Table B.1.2 Assigned plastic hinge properties of column

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	525.48	1.00	0.01
IO*	0.02			0.02
LS*	0.08			0.08
CP*	0.15			0.15
C	0.19	679.50	1.29	0.19
D	0.22	210.00	0.40	0.22
E	0.26	210.00	0.40	0.26
Yield Moment (Cf)		525		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Beam

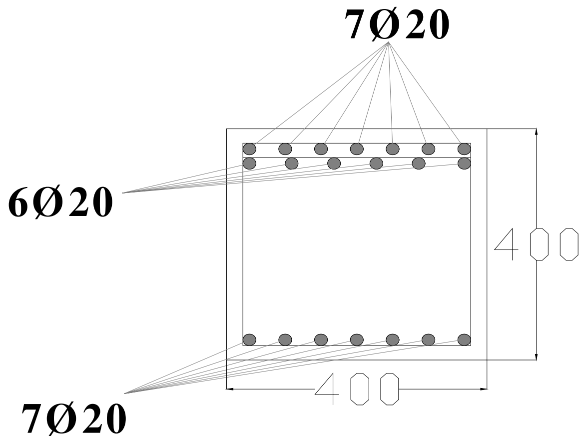


Figure B.1.5 Section for beam

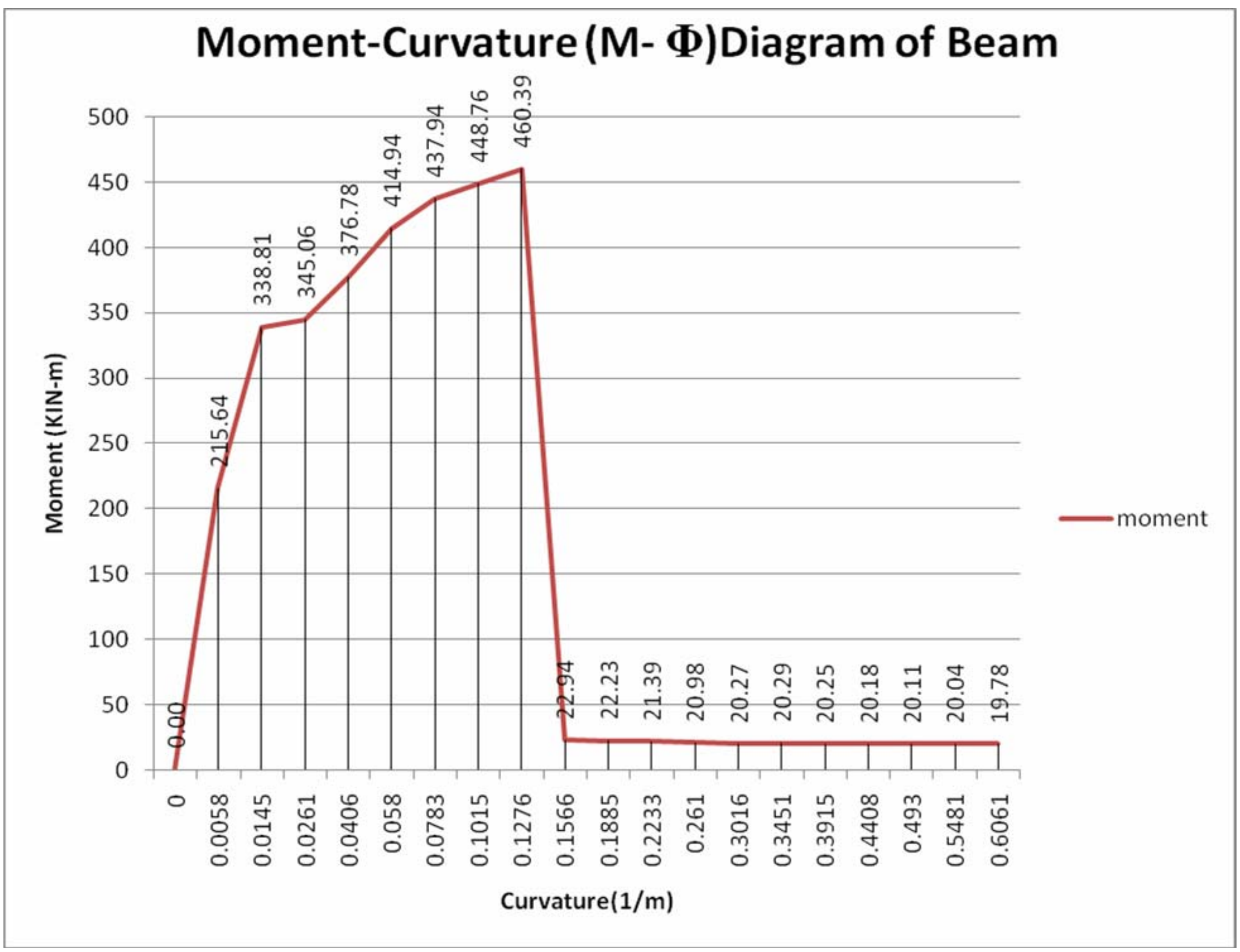


Figure B.1.6 Moment –Curvature (M-Φ) Diagram for beam

Table B.1.3 Assigned plastic hinge properties of beam

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	215.60	1.00	0.01
IO*	0.01			0.01
LS*	0.05			0.05
CP*	0.10			0.10
C	0.13	460.39	2.14	0.13
D	0.16	22.94	0.11	0.16
E	0.61	23.00	0.11	0.61
Yield Moment (Cf)		216		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

B.2 TEN STORY BUILDING (BUILDING MODEL2)

- Shear wall

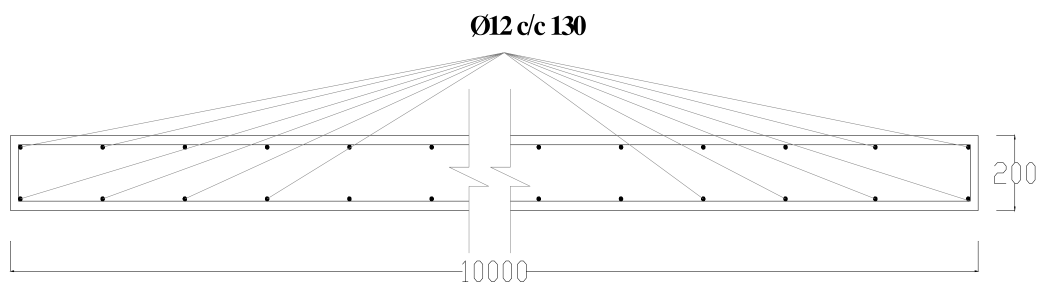


Figure B.2.1 Section for shear wall

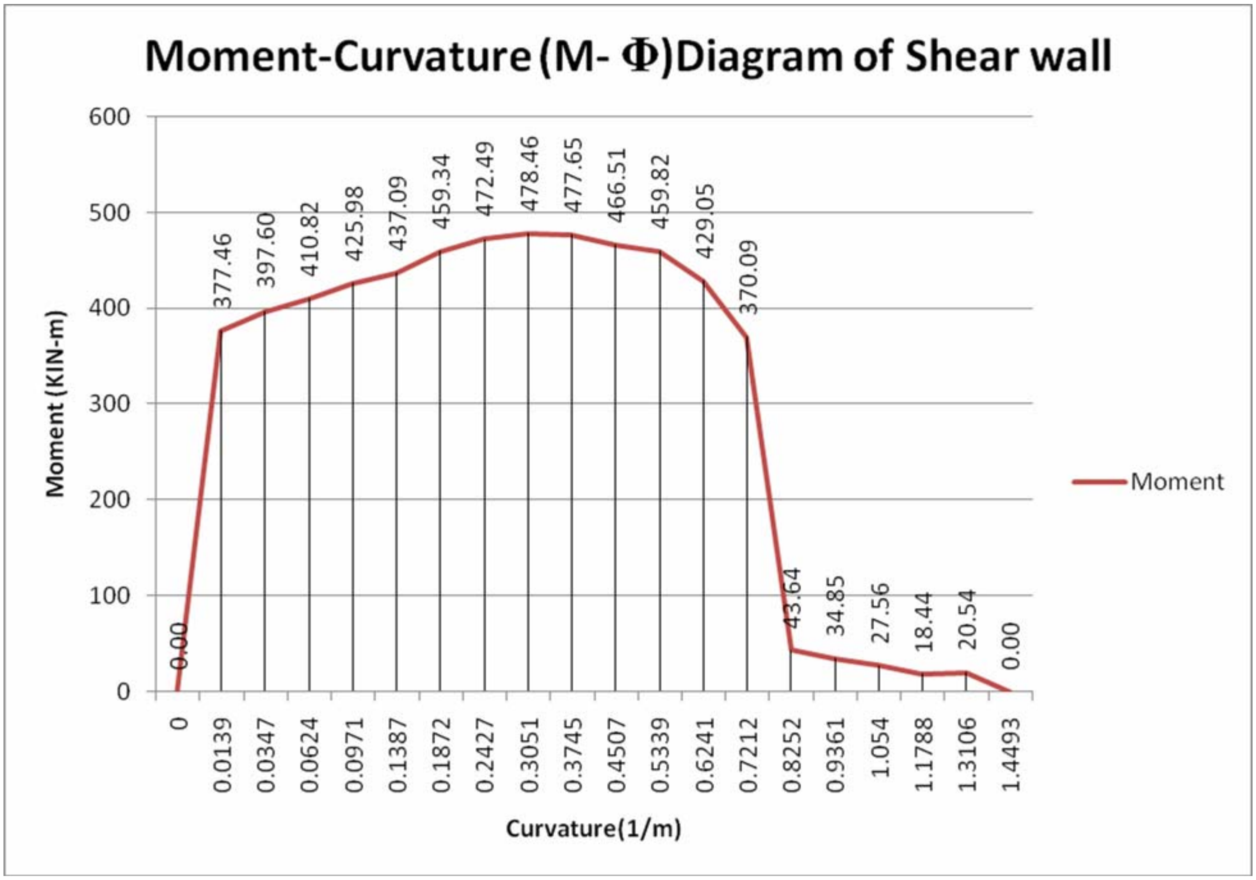


Figure B.2.2 Moment –Curvature (M-Φ) Diagram for shear wall in out-of-plane direction

Table B.2.1 Assigned plastic hinge properties of shear wall in out-of-plane direction

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	377.46	1.00	0.01
IO*	0.05			0.05
LS*	0.21			0.21
CP*	0.43			0.43
C	0.53	459.82	1.22	0.53
D	0.83	45.00	0.12	0.83
E	0.94	45.00	0.12	0.94
Yield Moment (Cf)		377		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Column

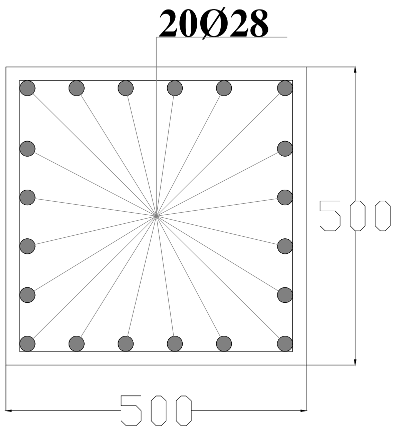


Figure B.2.3 Section for column

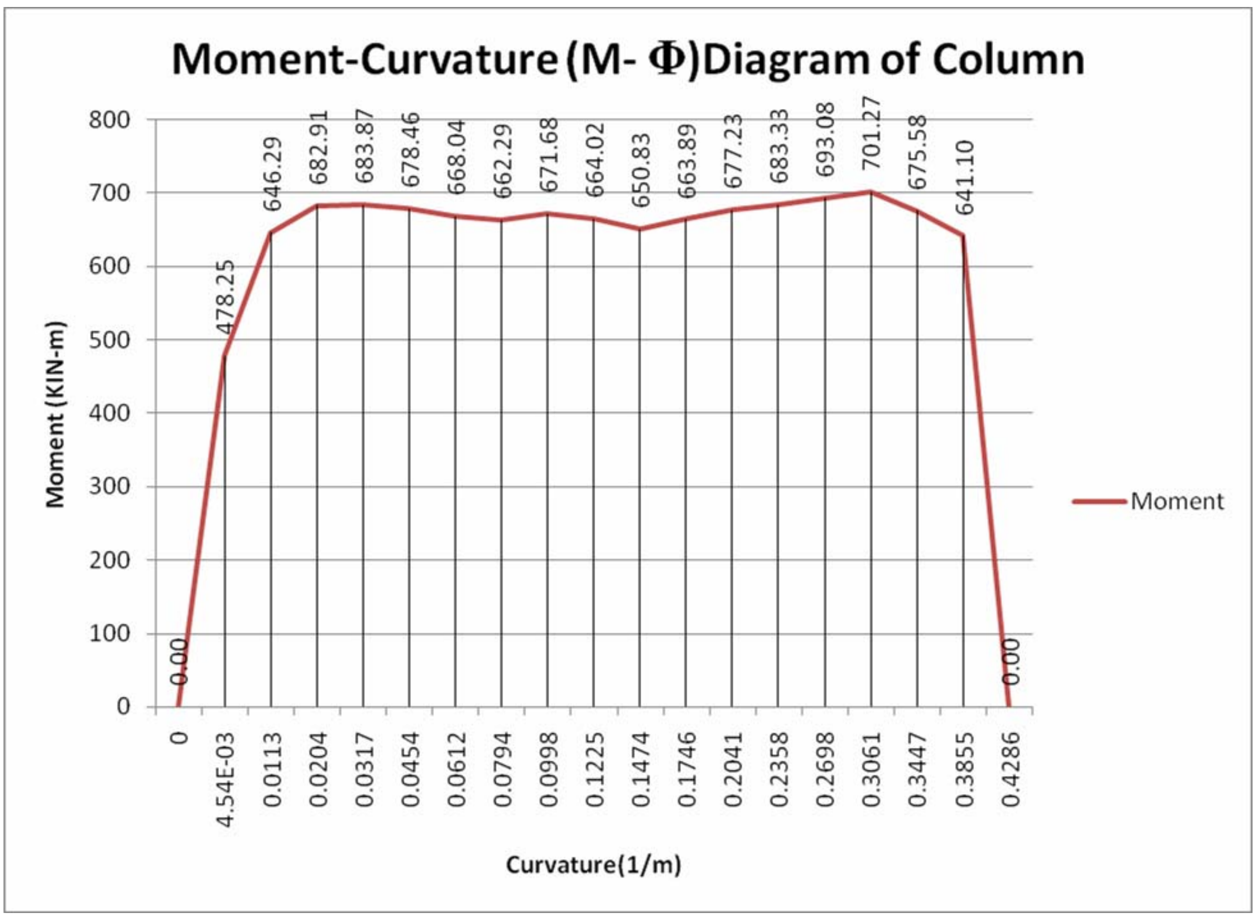


Figure B.2.4 Moment –Curvature (M-Φ) Diagram for column

Table B.2.2 Assigned plastic hinge properties of column

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	646.29	1.00	0.03
IO*	0.03			0.03
LS*	0.12			0.12
CP*	0.24			0.24
C	0.31	701.27	1.09	0.31
D	0.40	200.00	0.31	0.40
E	0.43	200.00	0.31	0.43
Yield Moment (Cf)		646		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Beam

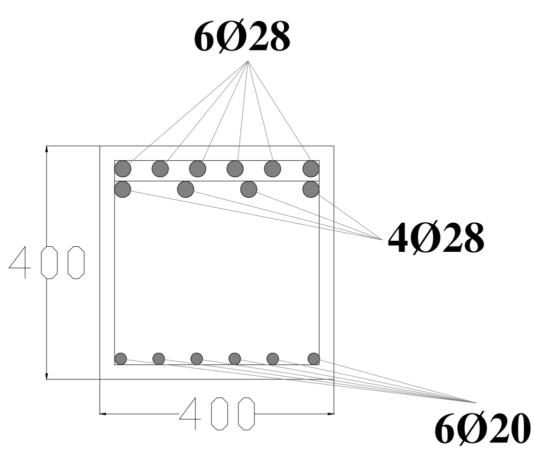


Figure B.2.5 Section for beam

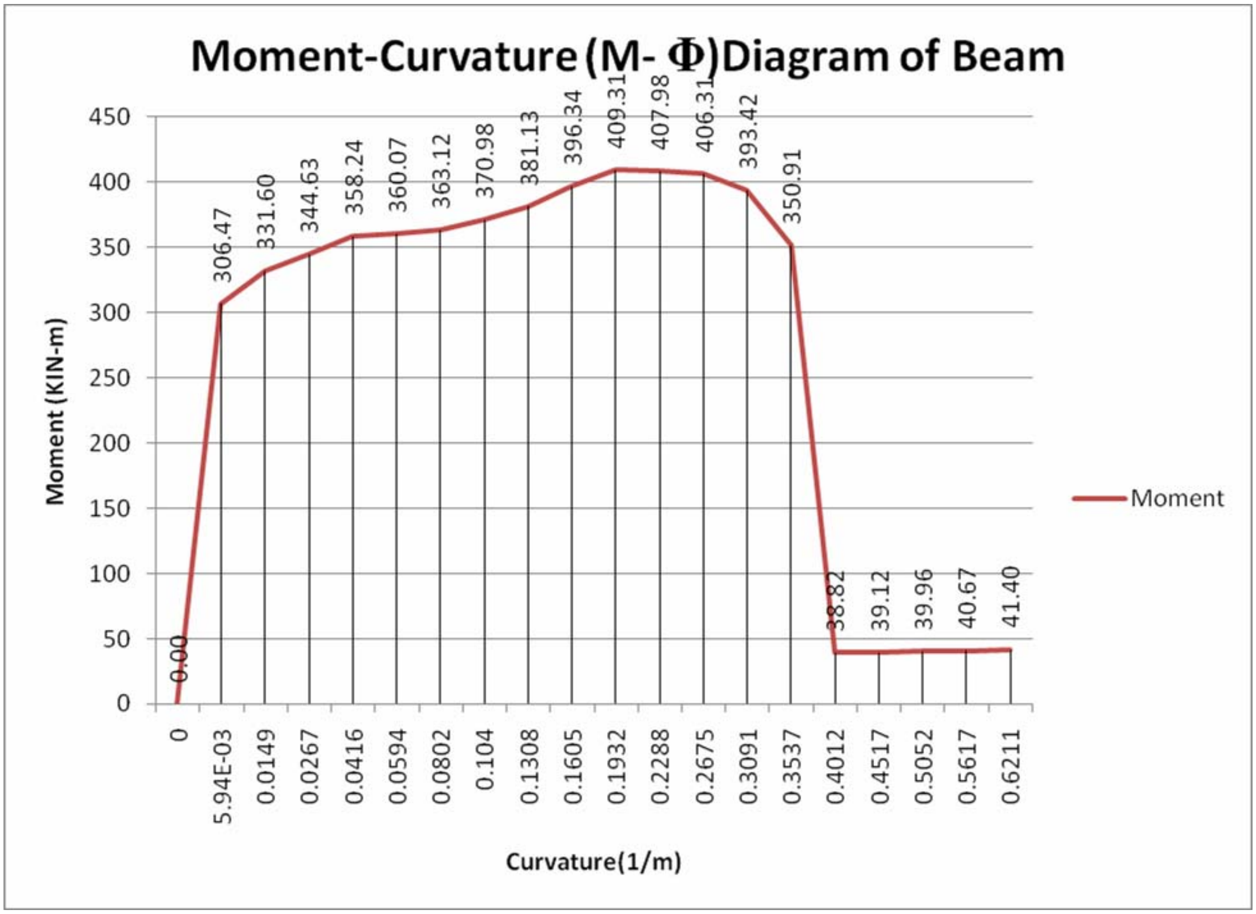


Figure B.2.6 Moment –Curvature (M-Φ) Diagram for beam

Table B.2.3 Assigned plastic hinge properties of beam

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	306.47	1.00	0.01
IO*	0.02			0.02
LS*	0.09			0.09
CP*	0.18			0.18
C	0.23	407.98	1.33	0.23
D	0.40	38.82	0.13	0.40
E	0.45	39.12	0.13	0.45
Yield Moment (Cf)		306		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

B.3 TWENTY STORY BUILDING (BUILDING MODEL3)

- Shear wall

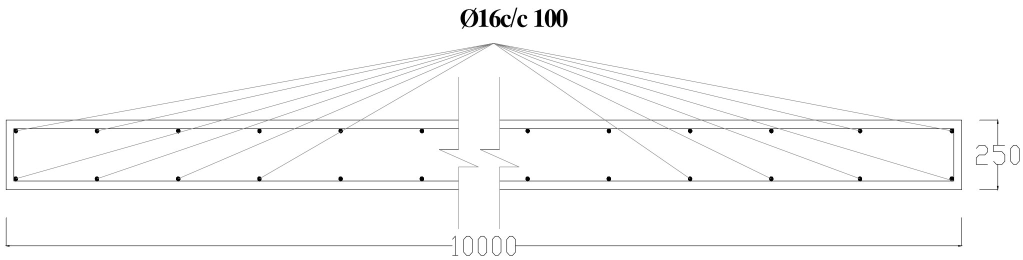


Figure B.3.1 Section for shear wall

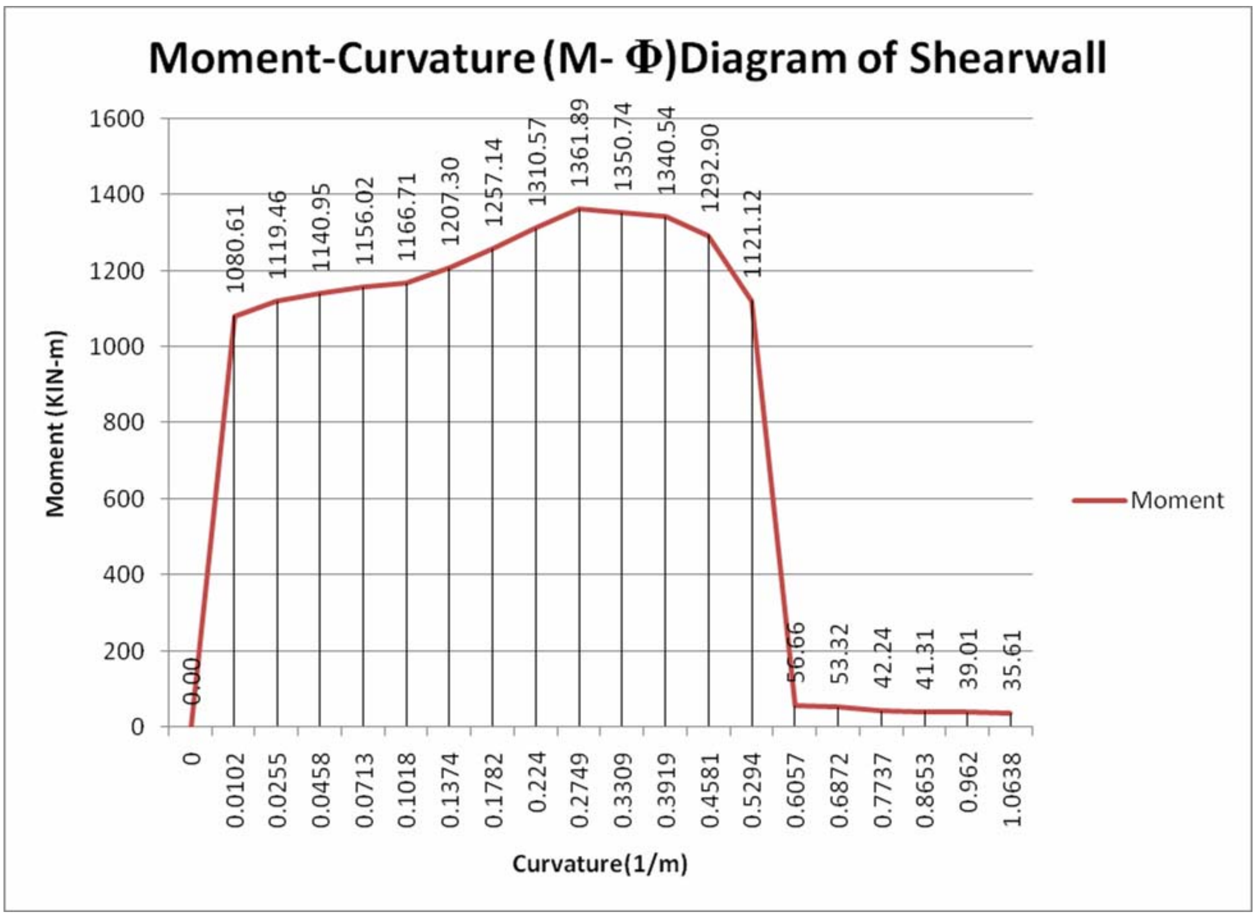


Figure B.3.2 Moment –Curvature (M-Φ) Diagram for shear wall in out-of-plane direction

Table B.3.1 Assigned plastic hinge properties of shear wall in out-of-plane direction

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	1080.61	1.00	0.01
IO*	0.04			0.04
LS*	0.16			0.16
CP*	0.31			0.31
C	0.39	1340.54	1.24	0.39
D	0.61	56.66	0.05	0.61
E	0.69	53.32	0.05	0.69
Yield Moment (Cf)		1081		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Column

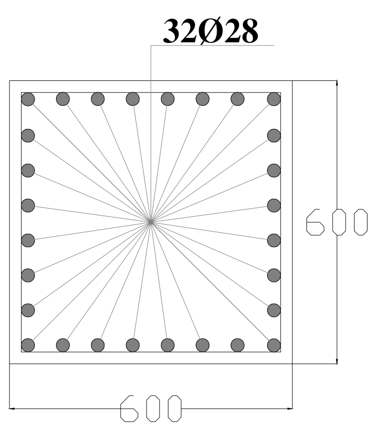


Figure B.3.3 Section for column

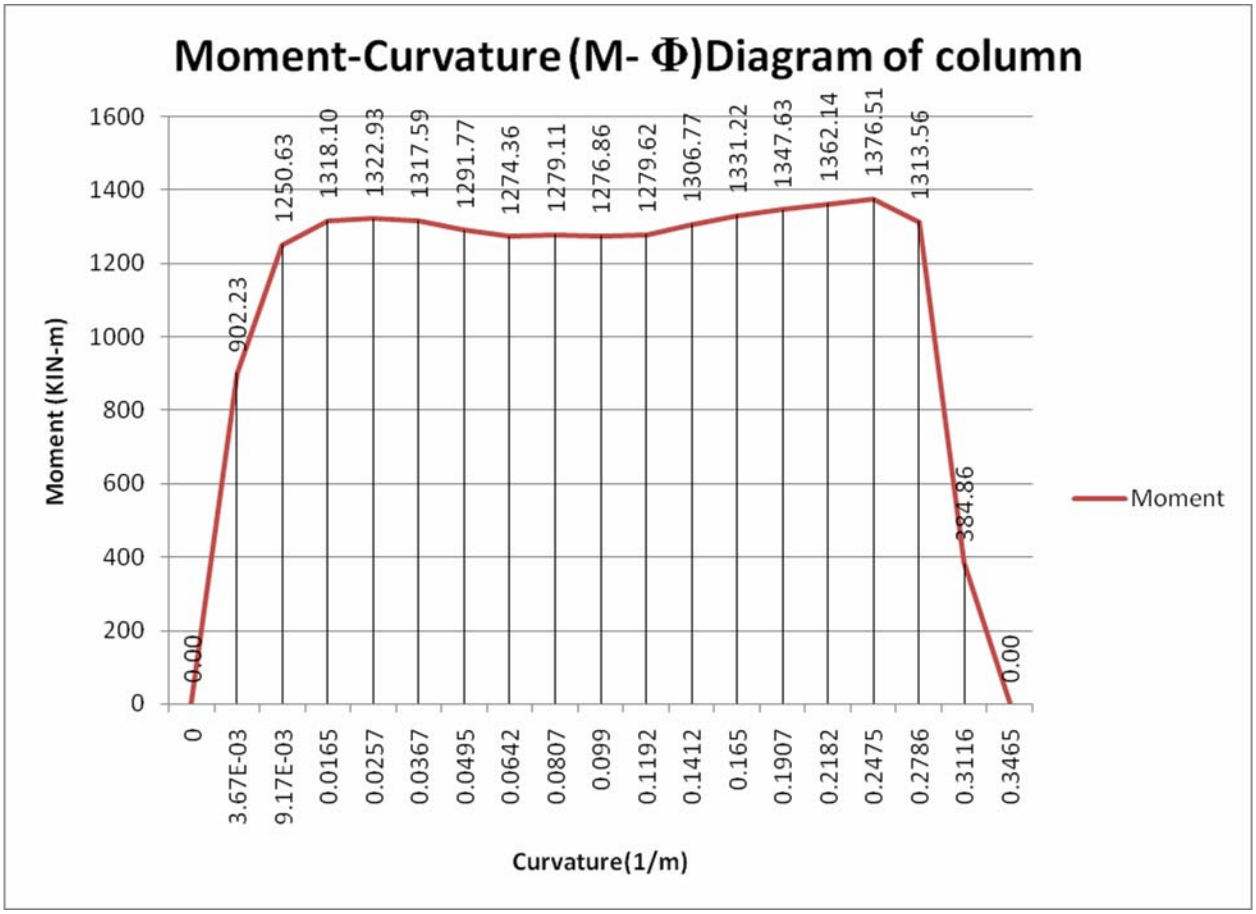


Figure B.3.4 Moment –Curvature (M-Φ) Diagram for column

Table B.3.2 Assigned plastic hinge properties of column

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.01	1250.63	1.00	0.03
IO*	0.02			0.02
LS*	0.10			0.10
CP*	0.20			0.20
C	0.25	1376.51	1.10	0.25
D	0.31	384.86	0.31	0.31
E	0.35	380.00	0.30	0.35
Yield Moment (Cf)		1251		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

- Beam

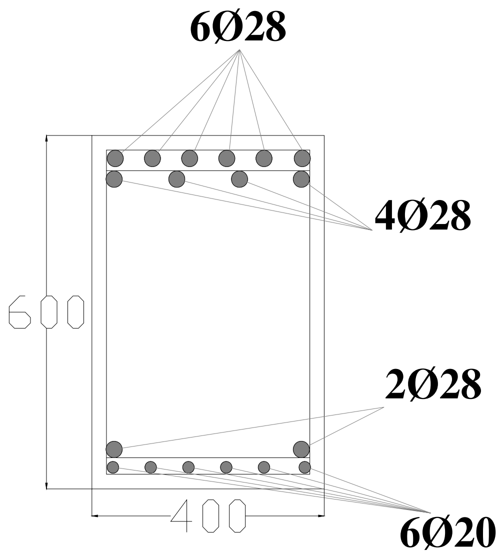


Figure B.3.5 Section for beam

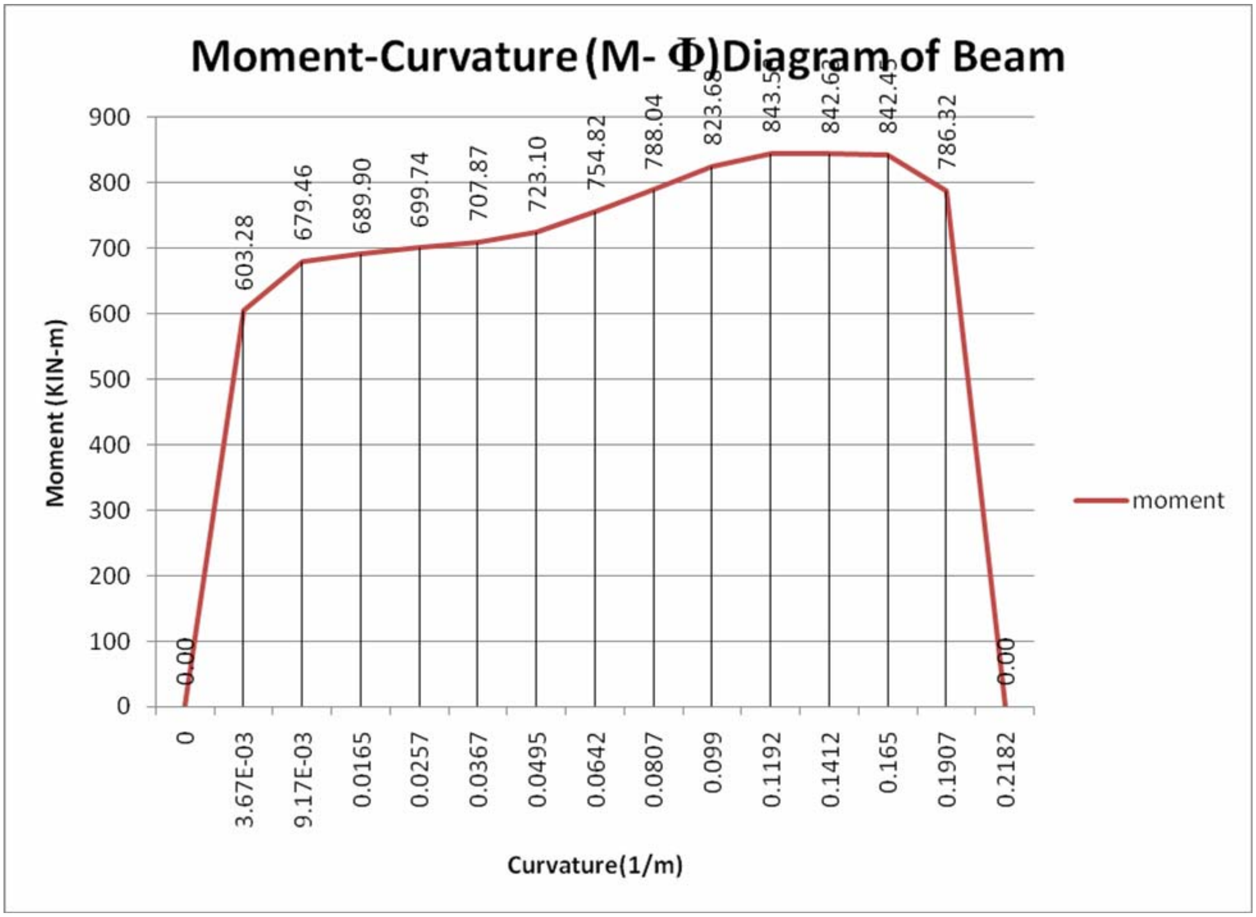


Figure B.3.6 Moment –Curvature (M-Φ) Diagram for beam

Table B.3.3 Assigned plastic hinge properties of beam

point	Curvature(1/m)	Moment (KN-m)	Moment /SF	Curvature/SF
A	0.00	0.00	0.00	0.00
B	0.00	603.28	1.00	0.02
IO*	0.02			0.02
LS*	0.07			0.07
CP*	0.13			0.13
C	0.17	842.45	1.40	0.17
D	0.20	400.00	0.66	0.20
E	0.22	400.00	0.66	0.22
Yield Moment (Cf)		603		
Yield Curvature (Cf)		1		

NOTE *: IO, LS and CP are taken as 10%, 40% and 80% ultimate curvature at point C

APPENDIX C

SUMMARY OF NUMBERS OF PLASTIC HINGING AT DIFFERENT DAMAGE LEVEL

C.1 FIVE STORY BUILDING (BUILDING MODEL1)

Table C.1.1 summary of plastic hinging at different damage level for case I

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	410	0	0	0	0	0	0	0	410
1	0.025123	944.653	408	2	0	0	0	0	0	0	410
2	0.061278	2198.735	376	34	0	0	0	0	0	0	410
3	0.08786	2592.261	334	30	46	0	0	0	0	0	410
4	0.127205	2876.237	310	24	76	0	0	0	0	0	410
5	0.187205	3031.668	308	8	40	54	0	0	0	0	410
6	0.301769	3297.407	284	28	6	38	54	0	0	0	410
7	0.39237	3480.005	274	16	28	26	66	0	0	0	410
8	0.481702	3602.443	262	24	30	8	84	2	0	0	410
9	0.541702	3675.084	262	4	50	4	88	2	0	0	410
10	0.6	3744.482	250	14	52	0	92	2	0	0	410

Table C.1.2 summary of plastic hinging at different damage level for case II

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	410	0	0	0	0	0	0	0	410
1	0.02502	804.686	408	2	0	0	0	0	0	0	410
2	0.059999	1786.744	378	32	0	0	0	0	0	0	410
3	0.088679	2191.691	346	34	30	0	0	0	0	0	410
4	0.153398	2613.096	326	10	58	16	0	0	0	0	410
5	0.160478	2646.582	320	16	58	16	0	0	0	0	410
6	0.242508	2826.561	306	14	34	56	0	0	0	0	410
7	0.304541	2951.59	298	14	30	24	44	0	0	0	410
8	0.388269	3096.489	296	2	32	32	48	0	0	0	410
9	0.449249	3199.924	288	10	30	18	62	2	0	0	410
10	0.527487	3302.812	280	16	20	14	78	2	0	0	410
11	0.6	3392.319	274	18	18	20	78	2	0	0	410

Table C.1.3 summary of plastic hinging at different damage level for case III

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	410	0	0	0	0	0	0	0	410
1	0.023988	828.073	409	1	0	0	0	0	0	0	410
2	0.053926	1769.911	385	25	0	0	0	0	0	0	410
3	0.085998	2185.919	344	30	36	0	0	0	0	0	410
4	0.158477	2678.056	321	15	52	22	0	0	0	0	410
5	0.225608	2828.109	301	19	42	48	0	0	0	0	410
6	0.293774	2962.283	298	8	38	36	30	0	0	0	410
7	0.358744	3084.182	295	7	26	40	42	0	0	0	410
8	0.439271	3227.695	287	9	27	31	56	0	0	0	410
9	0.547123	3376.176	281	12	13	24	78	2	0	0	410
10	0.6	3442.879	271	16	19	22	80	2	0	0	410

C.2 TEN STORY BUILDING (BUILDING MODEL2)

Table C.2.1 summary of plastic hinging at different damage level for case I

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	820	0	0	0	0	0	0	0	820
1	0.085278	1868.602	818	2	0	0	0	0	0	0	820
2	0.128943	2790.587	772	48	0	0	0	0	0	0	820
3	0.157292	3085.879	710	110	0	0	0	0	0	0	820
4	0.185502	3259.291	692	104	24	0	0	0	0	0	820
5	0.231153	3377.052	660	70	90	0	0	0	0	0	820
6	0.390507	3588.827	644	36	140	0	0	0	0	0	820
7	0.52473	3739.644	604	50	66	100	0	0	0	0	820
8	0.69882	3877.132	578	70	54	116	2	0	0	0	820
9	0.82882	3958.941	576	58	46	72	68	0	0	0	820
10	0.959357	4039.605	570	40	66	30	110	4	0	0	820
11	1.102053	4107.632	568	38	70	26	108	10	0	0	820
12	1.177153	4140.646	568	38	68	28	96	22	0	0	820
13	1.2	4143.427	564	42	68	28	96	22	0	0	820

Table C.2.2 summary of plastic hinging at different damage level for case II

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	740	0	0	0	0	0	0	0	740
1	0.087541	1516.852	738	2	0	0	0	0	0	0	740
2	0.135501	2270.509	694	46	0	0	0	0	0	0	740
3	0.196043	2768.214	636	85	19	0	0	0	0	0	740
4	0.229305	2904.84	606	91	43	0	0	0	0	0	740
5	0.266256	2983.806	588	74	78	0	0	0	0	0	740
6	0.39285	3129.421	567	43	130	0	0	0	0	0	740
7	0.525655	3255.153	560	38	72	70	0	0	0	0	740
8	0.6645	3376.139	542	33	53	112	0	0	0	0	740
9	0.786802	3462.993	525	39	52	104	20	0	0	0	740
10	0.918576	3538.94	503	56	57	58	66	0	0	0	740
11	1.050458	3597.779	497	49	70	38	78	8	0	0	740
12	1.2	3645.102	495	40	64	41	90	10	0	0	740

Table C.2.3 summary of plastic hinging at different damage level for case III

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	820	0	0	0	0	0	0	0	820
1	0.08492	1614.617	818	2	0	0	0	0	0	0	820
2	0.126858	2376.552	760	60	0	0	0	0	0	0	820
3	0.20382	2867.254	663	106	51	0	0	0	0	0	820
4	0.221905	2926.01	632	134	54	0	0	0	0	0	820
5	0.352125	3092.796	621	79	120	0	0	0	0	0	820
6	0.480917	3221.72	590	86	93	51	0	0	0	0	820
7	0.601179	3322.734	584	76	67	93	0	0	0	0	820
8	0.754716	3434.438	564	76	62	104	14	0	0	0	820
9	0.908736	3520.554	539	93	64	54	70	0	0	0	820
10	1.045665	3583.447	530	94	62	45	87	2	0	0	820
11	1.2	3644.811	522	92	61	45	98	2	0	0	820

C.3 TEN STORY BUILDING (BUILDING MODEL3)

Table C.3.1 summary of plastic hinging at different damage level for case I

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	1640	0	0	0	0	0	0	0	1640
1	0.161423	4447.99	1636	4	0	0	0	0	0	0	1640
2	0.186062	5071.87	1562	78	0	0	0	0	0	0	1640
3	0.233262	5576.83	1454	186	0	0	0	0	0	0	1640
4	0.419486	6193.87	1350	140	150	0	0	0	0	0	1640
5	0.672307	6532.27	1316	76	248	0	0	0	0	0	1640
6	0.954286	6872.3	1296	56	112	176	0	0	0	0	1640
7	1.258898	7215.66	1284	44	92	220	0	0	0	0	1640
8	1.5773	7555.18	1256	58	56	116	154	0	0	0	1640
9	1.849485	7812.47	1216	76	74	68	196	10	0	0	1640
10	2.09222	7999.73	1194	94	76	44	216	16	0	0	1640
11	2.280472	8135.86	1174	114	56	48	196	52	0	0	1640
12	2.303212	8145.24	1174	114	56	48	170	78	0	0	1640
13	2.347373	8149.44	1174	114	56	48	144	104	0	0	1640
14	2.367114	8145.14	1174	114	56	48	118	130	0	0	1640

Table C.3.2 summary of plastic hinging at different damage level for case II

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	1640	0	0	0	0	0	0	0	1640
1	0.176767	3670.451	1637	3	0	0	0	0	0	0	1640
2	0.251427	4695.659	1507	133	0	0	0	0	0	0	1640
3	0.283242	4879.661	1452	188	0	0	0	0	0	0	1640
4	0.523933	5492.459	1372	106	162	0	0	0	0	0	1640
5	0.533577	5510.345	1362	115	163	0	0	0	0	0	1640
6	0.795019	5819.656	1341	65	212	22	0	0	0	0	1640
7	1.038874	6083.697	1315	69	115	141	0	0	0	0	1640
8	1.327633	6378.424	1296	68	78	198	0	0	0	0	1640
9	1.596335	6637.511	1277	76	56	119	112	0	0	0	1640
10	1.838945	6850.306	1239	89	72	73	161	6	0	0	1640
11	2.080152	7025.081	1226	96	62	74	166	16	0	0	1640
12	2.335507	7184.978	1209	101	74	56	172	28	0	0	1640
13	2.4	7215.052	1206	98	77	59	145	55	0	0	1640

Table C.3.3 summary of plastic hinging at different damage level for case III

Step	Displacement (m)	BaseForce (KN)	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	0	0	1640	0	0	0	0	0	0	0	1640
1	0.157091	3228.966	1637	3	0	0	0	0	0	0	1640
2	0.224104	4384.306	1540	100	0	0	0	0	0	0	1640
3	0.265723	4737.731	1472	168	0	0	0	0	0	0	1640
4	0.510825	5386.615	1362	125	153	0	0	0	0	0	1640
5	0.555442	5464.473	1353	121	166	0	0	0	0	0	1640
6	0.825339	5753.996	1329	75	189	47	0	0	0	0	1640
7	1.07889	6004.025	1299	83	103	155	0	0	0	0	1640
8	1.358278	6264.892	1280	82	80	197	1	0	0	0	1640
9	1.60318	6483.287	1263	95	50	122	110	0	0	0	1640
10	1.870419	6706.774	1231	107	65	64	167	6	0	0	1640
11	2.125885	6889.621	1213	122	49	68	174	14	0	0	1640
12	2.381702	7050.067	1193	134	55	58	166	34	0	0	1640
13	2.4	7059.061	1193	132	57	58	152	48	0	0	1640

APPENDIX D

PLASTIC HINGES MECHANISM AND ITS PATTERNS

Table D.1. Symbol of Damage Level used to show the pattern

Damage level	Yielding(B)	Immediate Occupancy (IO)	Life safety (LS)	Collapse prevention (CP)	Ultimate point C
Symbol	▲	◆	▬	●	★

D.1 FIVE STORY BUILDING (BUILDING MODEL1)

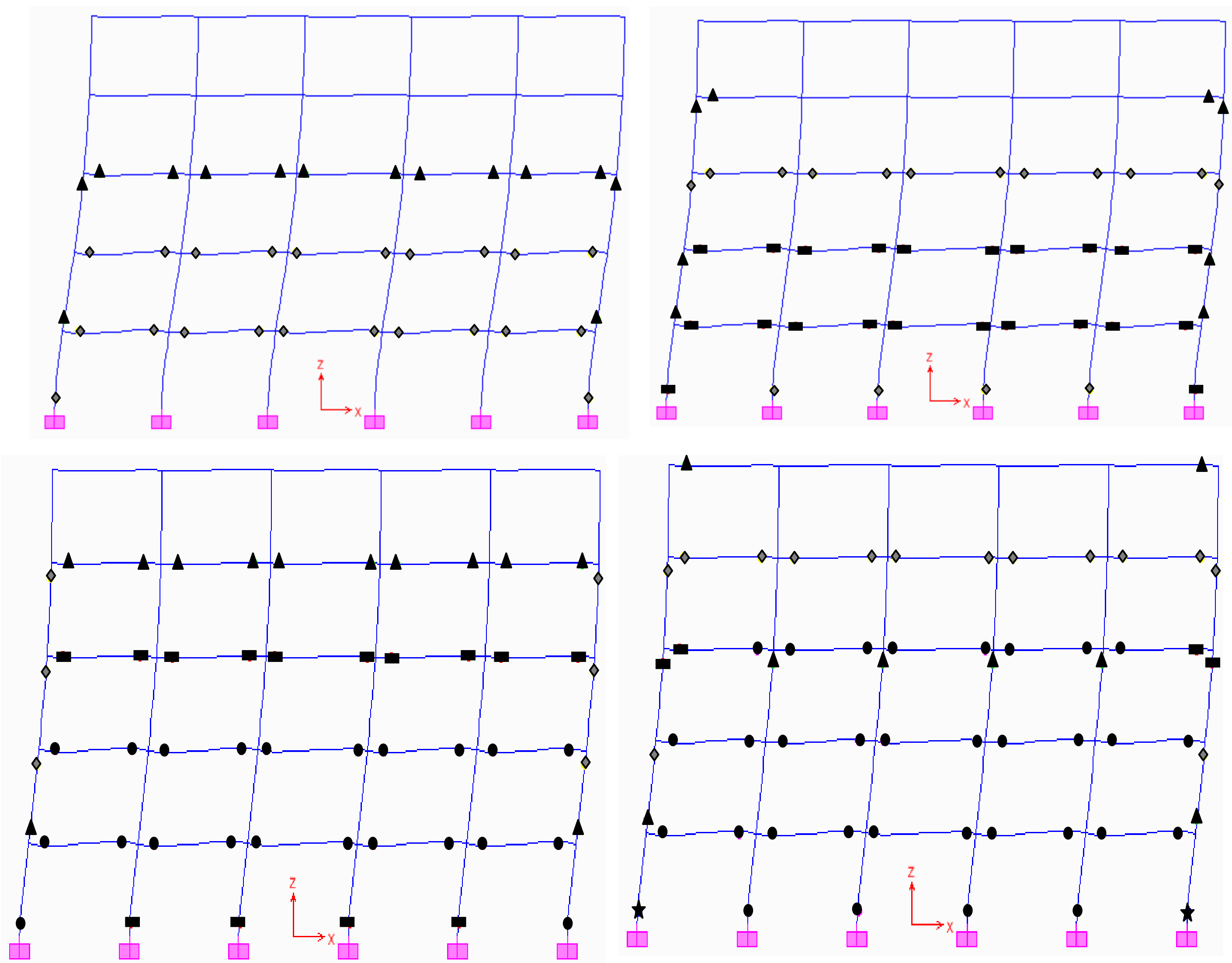


Figure D.1.1 Hinge patterns of Case I shear wall arrangement for different displacements levels

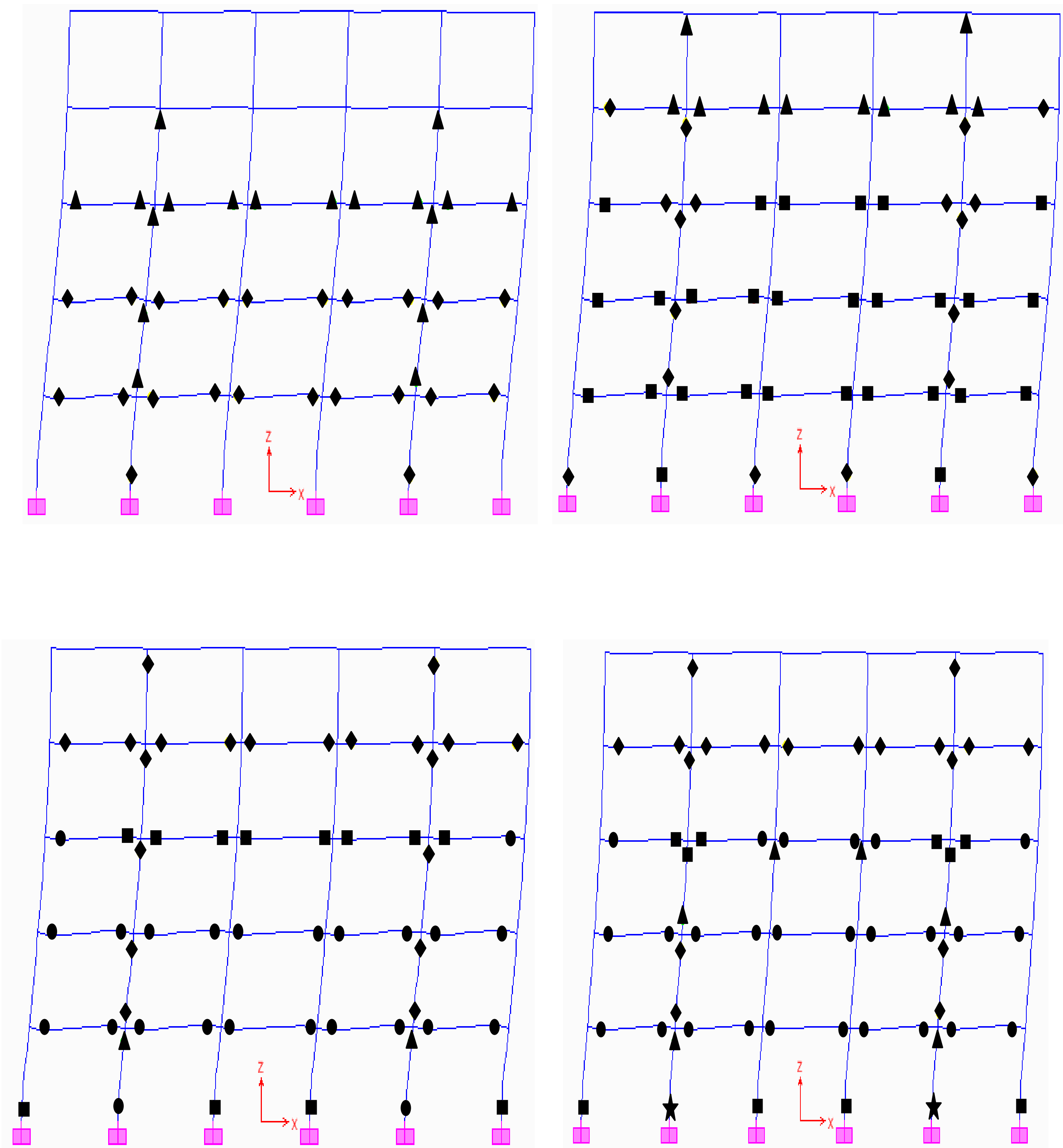
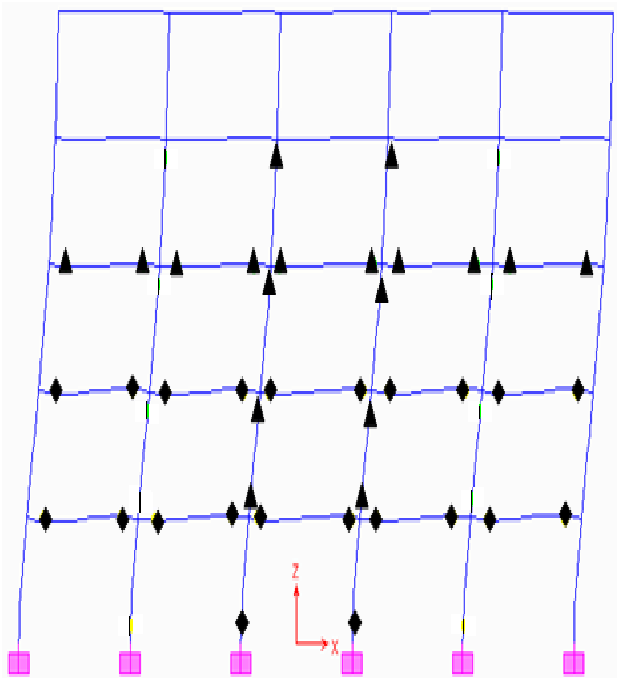


Figure D.1.2 Hinge patterns of Case II shear wall arrangement for different displacements levels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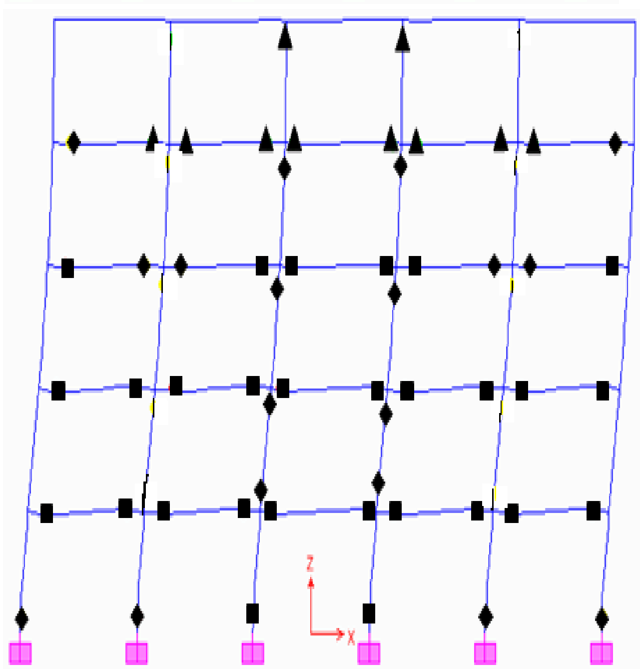
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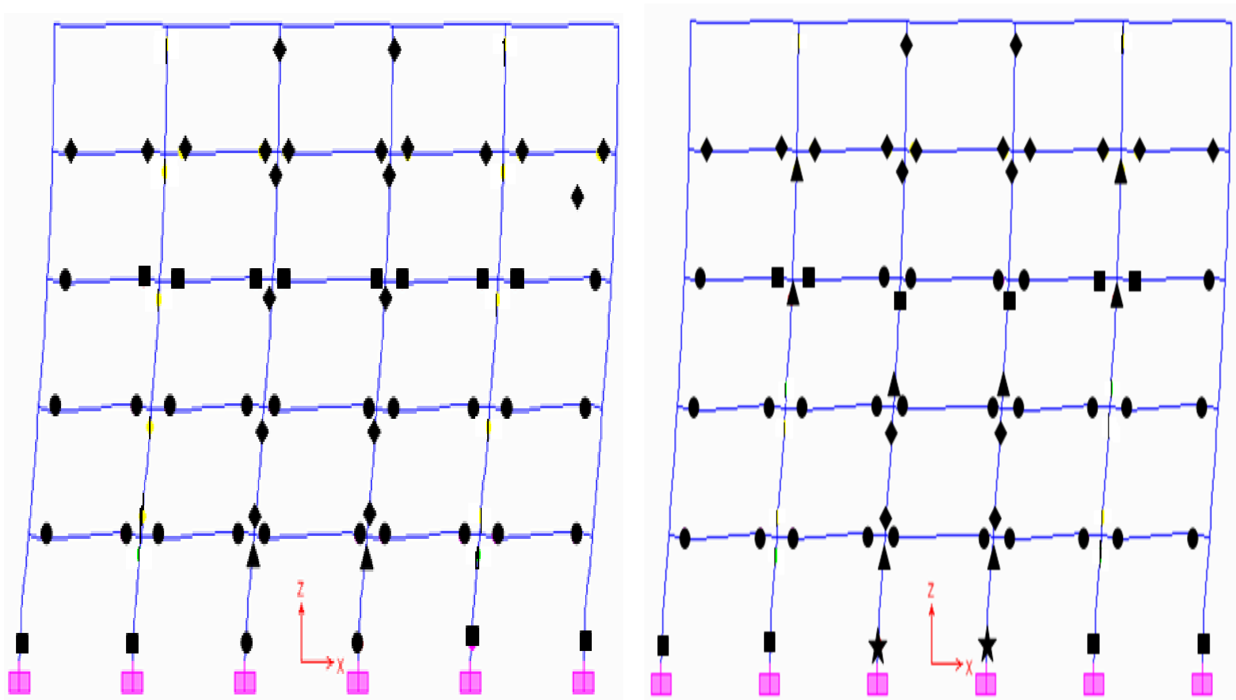
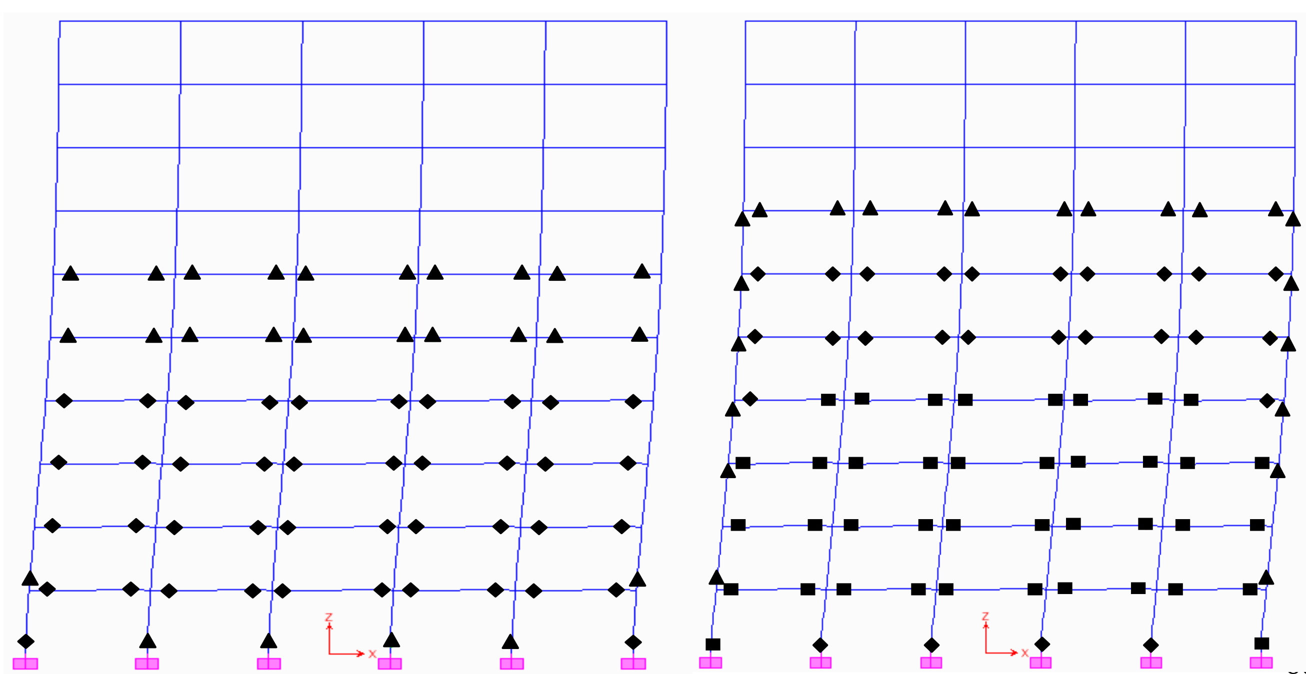


Figure D.1.3 Hinge patterns of Case III shear wall arrangement for different displacements levels

D.2 TEN STORY BUILDING (BUILDING MODEL2)



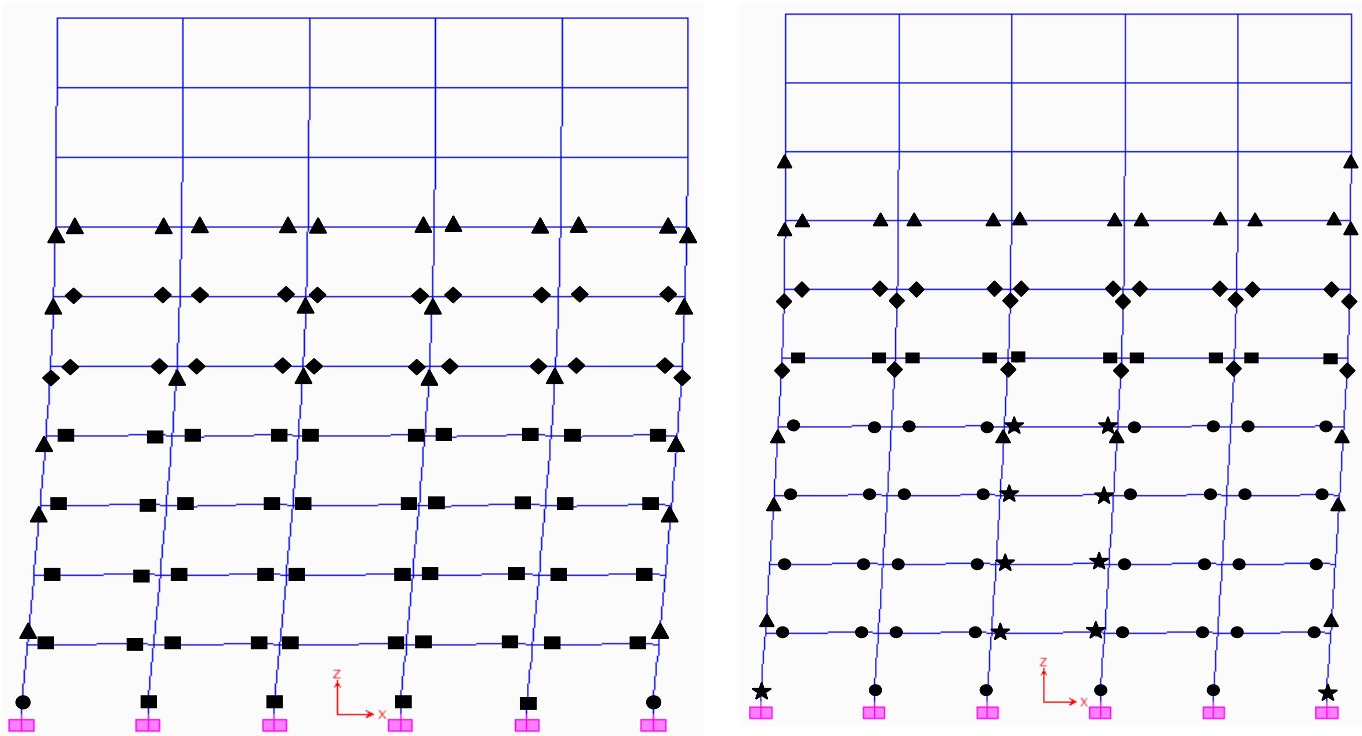
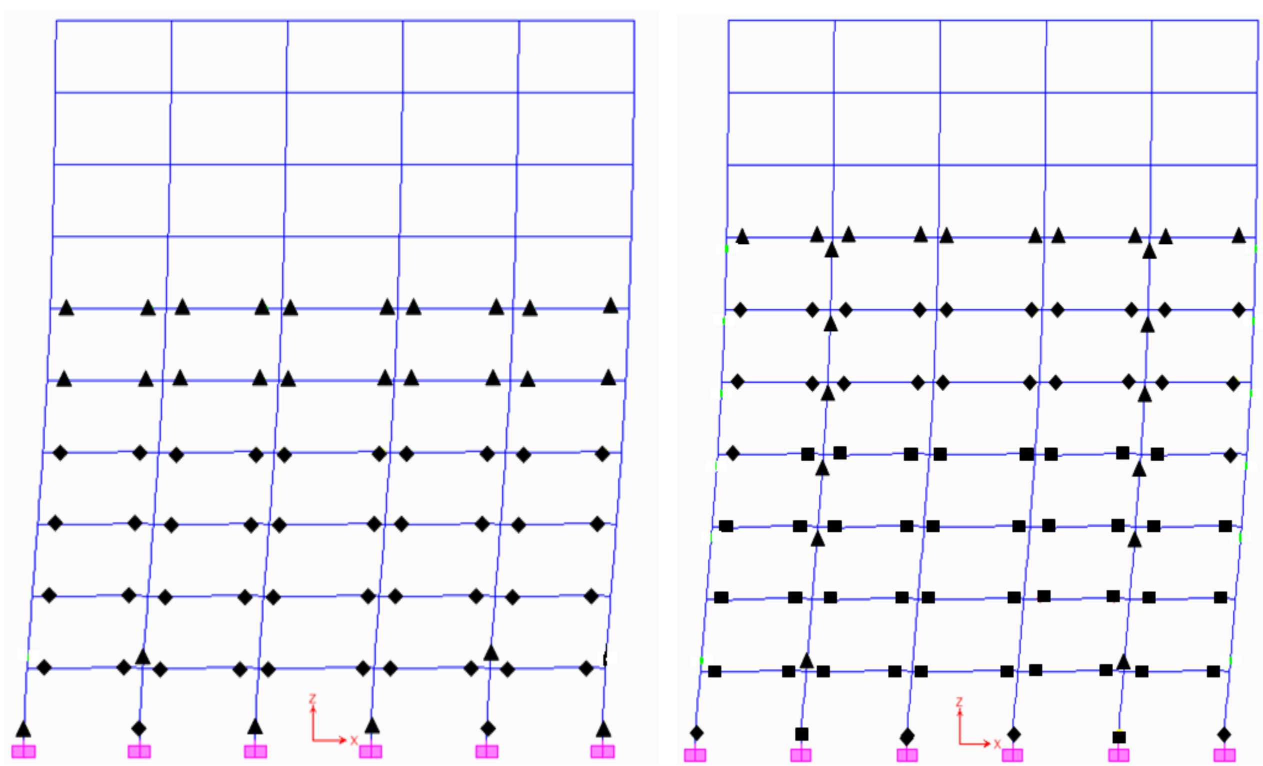


Figure D.2.1 Hinge patterns of Case I shear wall arrangement for different displacements levels



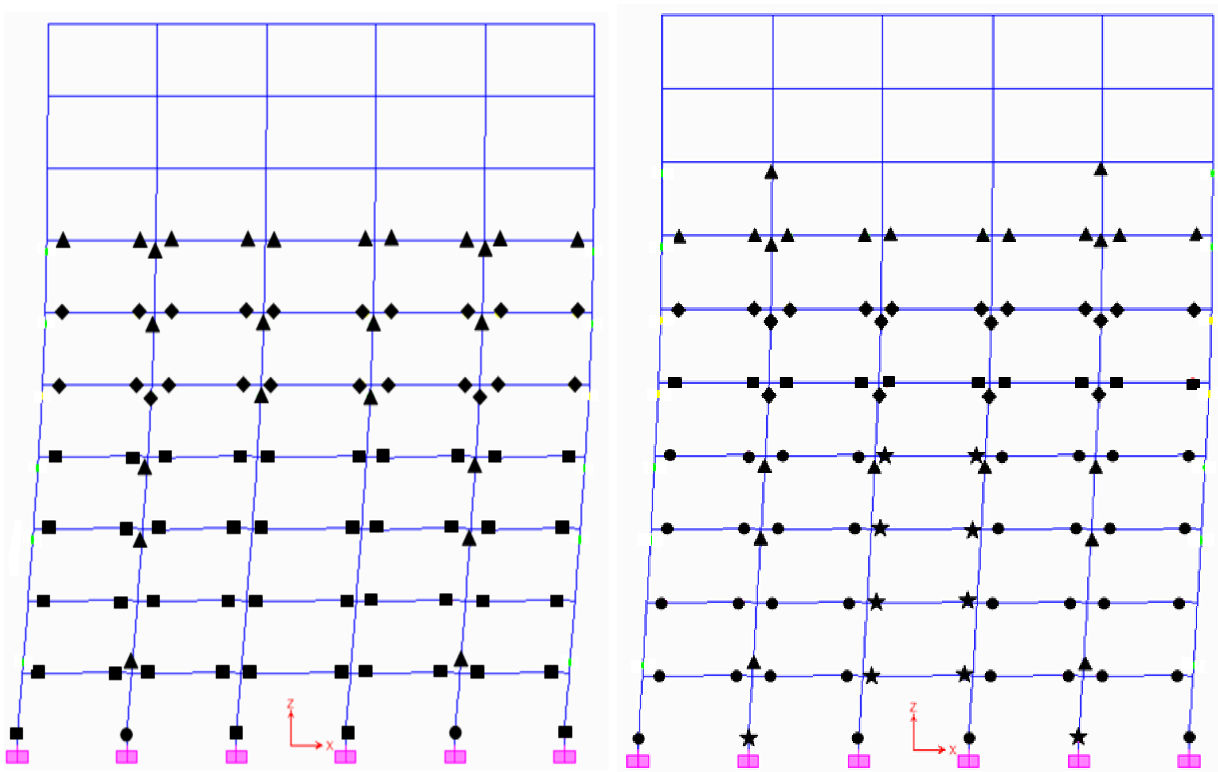
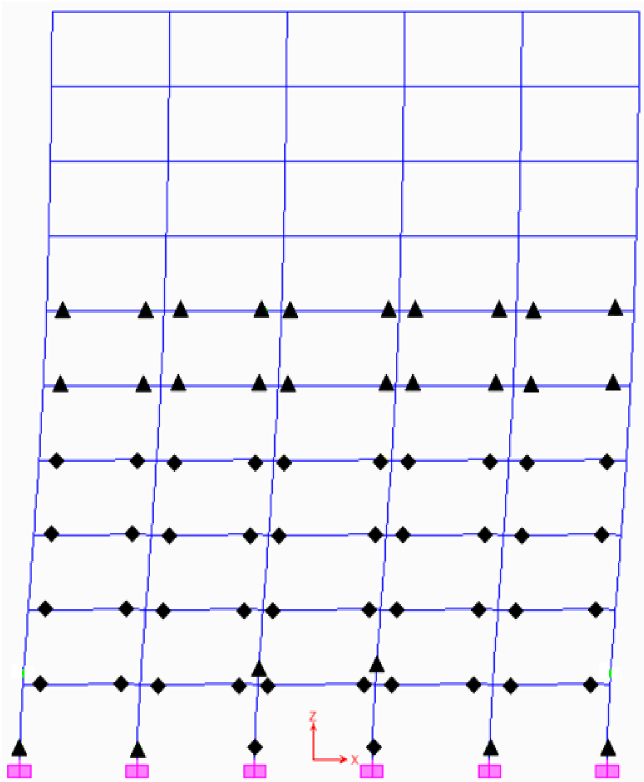


Figure D.2.2 Hinge patterns of Case II shear wall arrangement for different displacements levels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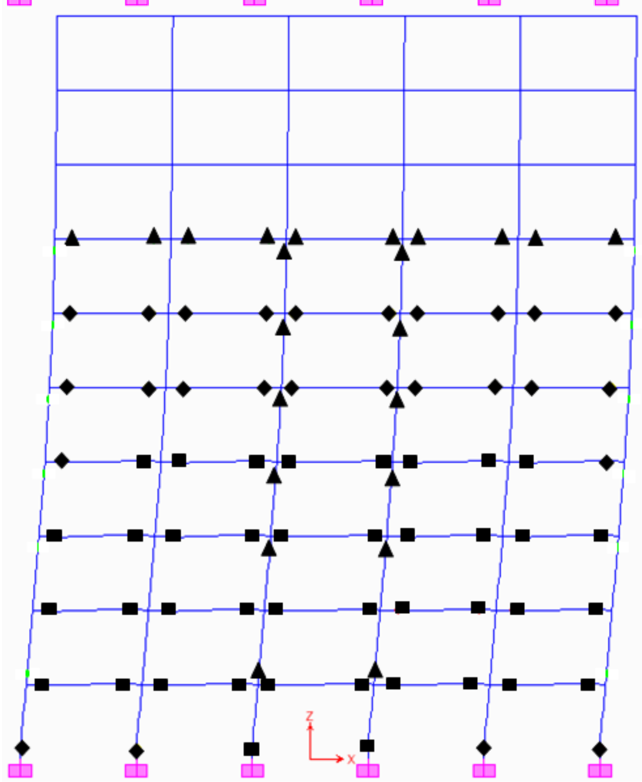
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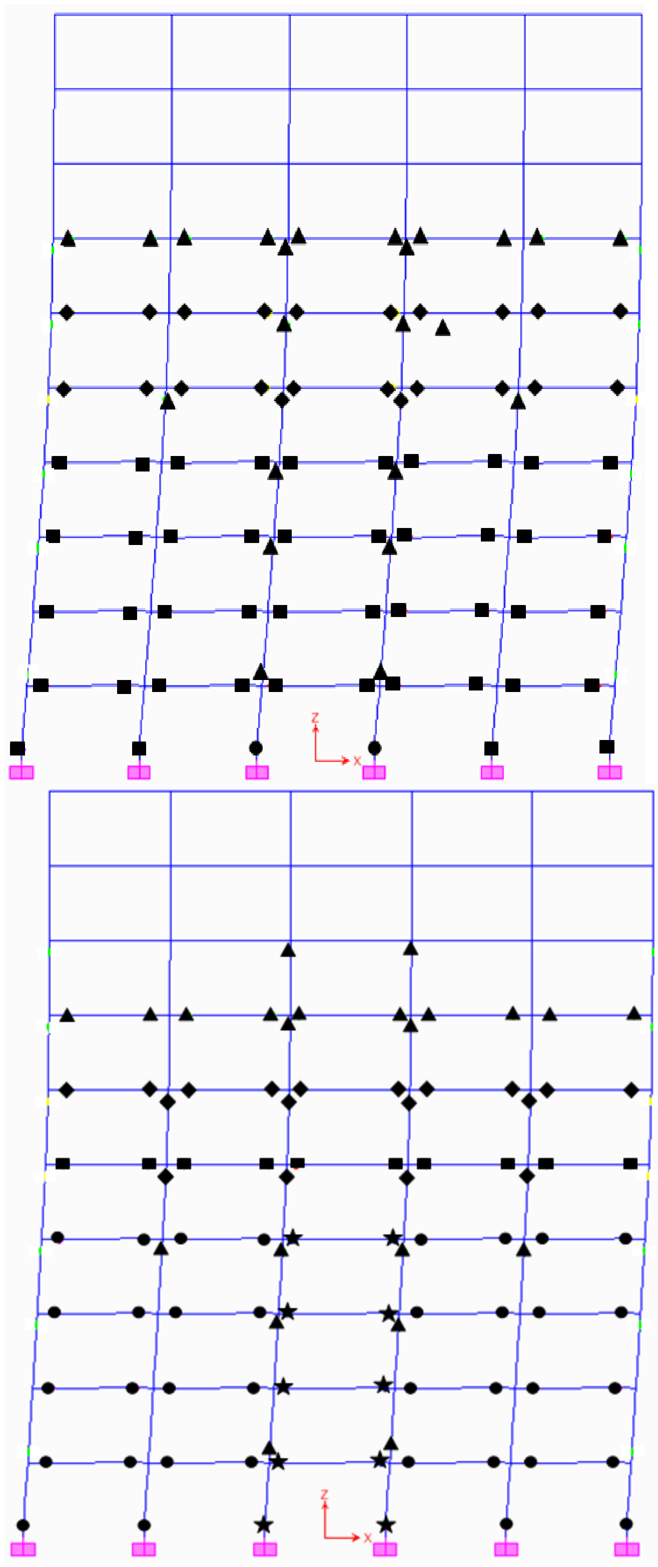

Figure D.2.3 Hinge patterns of Case III shear wall arrangement for different displacements levels

D.3 TWENTY STORY BUILDING (BUILDING MODEL3)

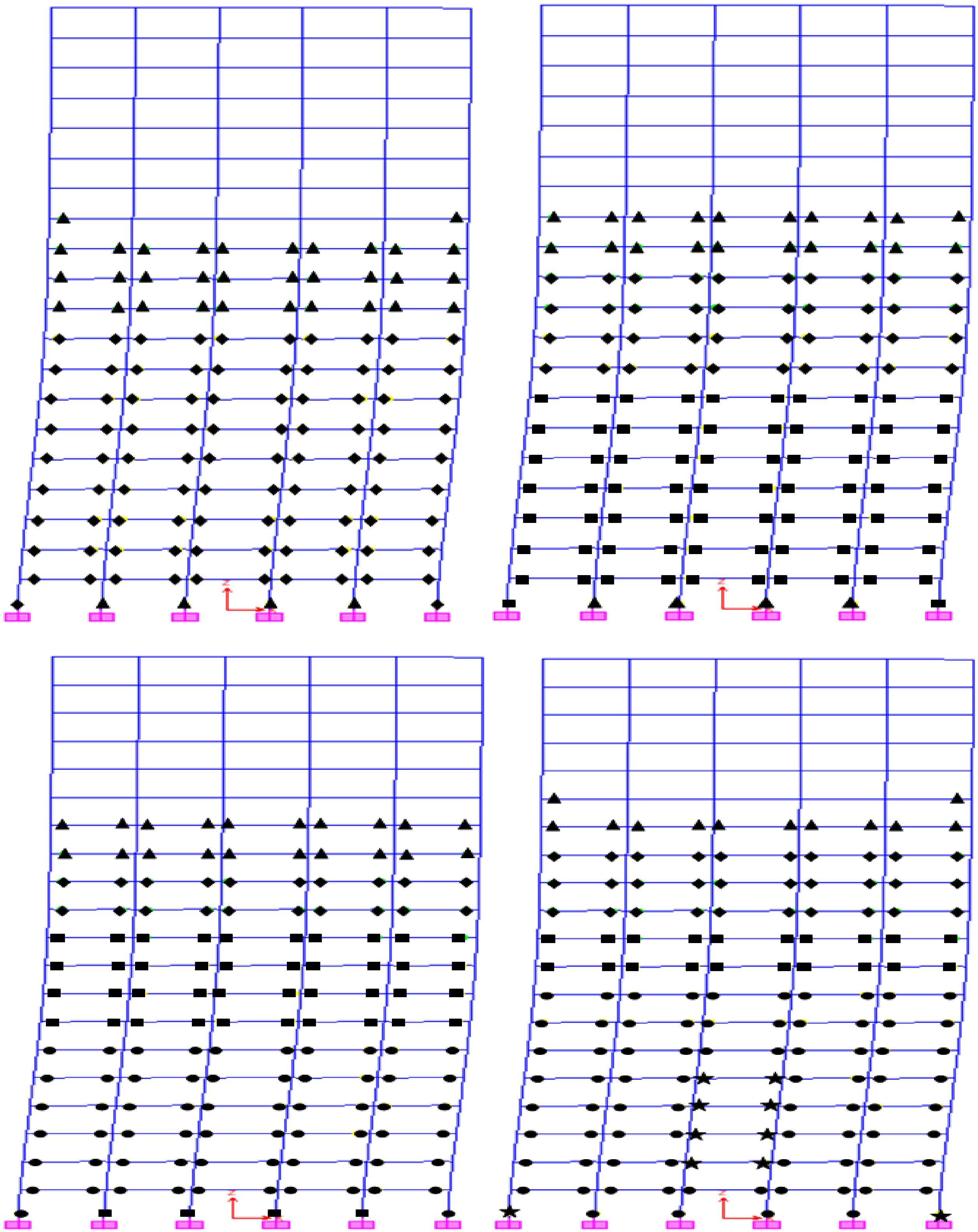

Figure D.3.1 Hinge patterns of Case I shear wall arrangement for different displacements levels

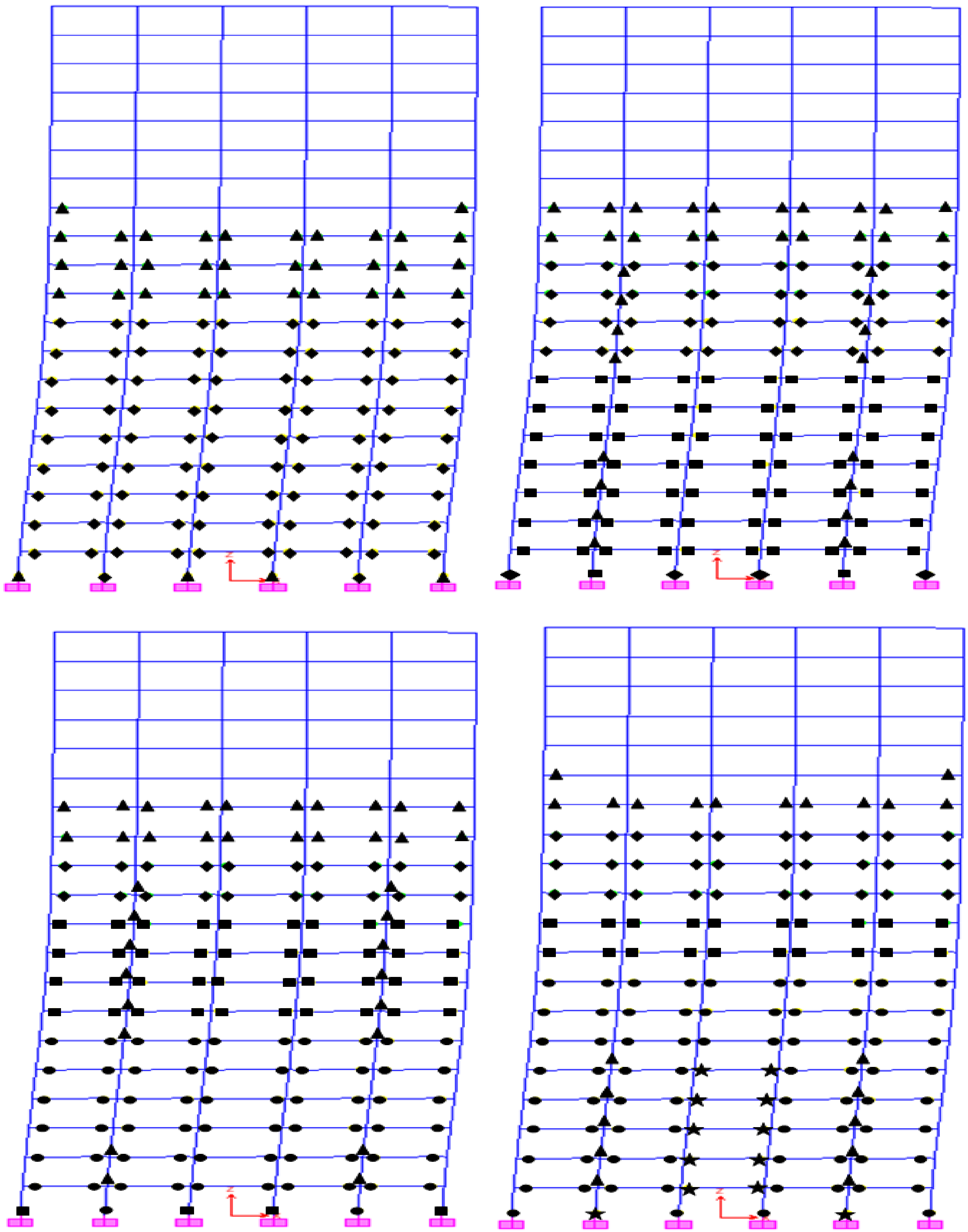

Figure D.3.2 Hinge patterns of Case II shear wall arrangement for different displacements levels

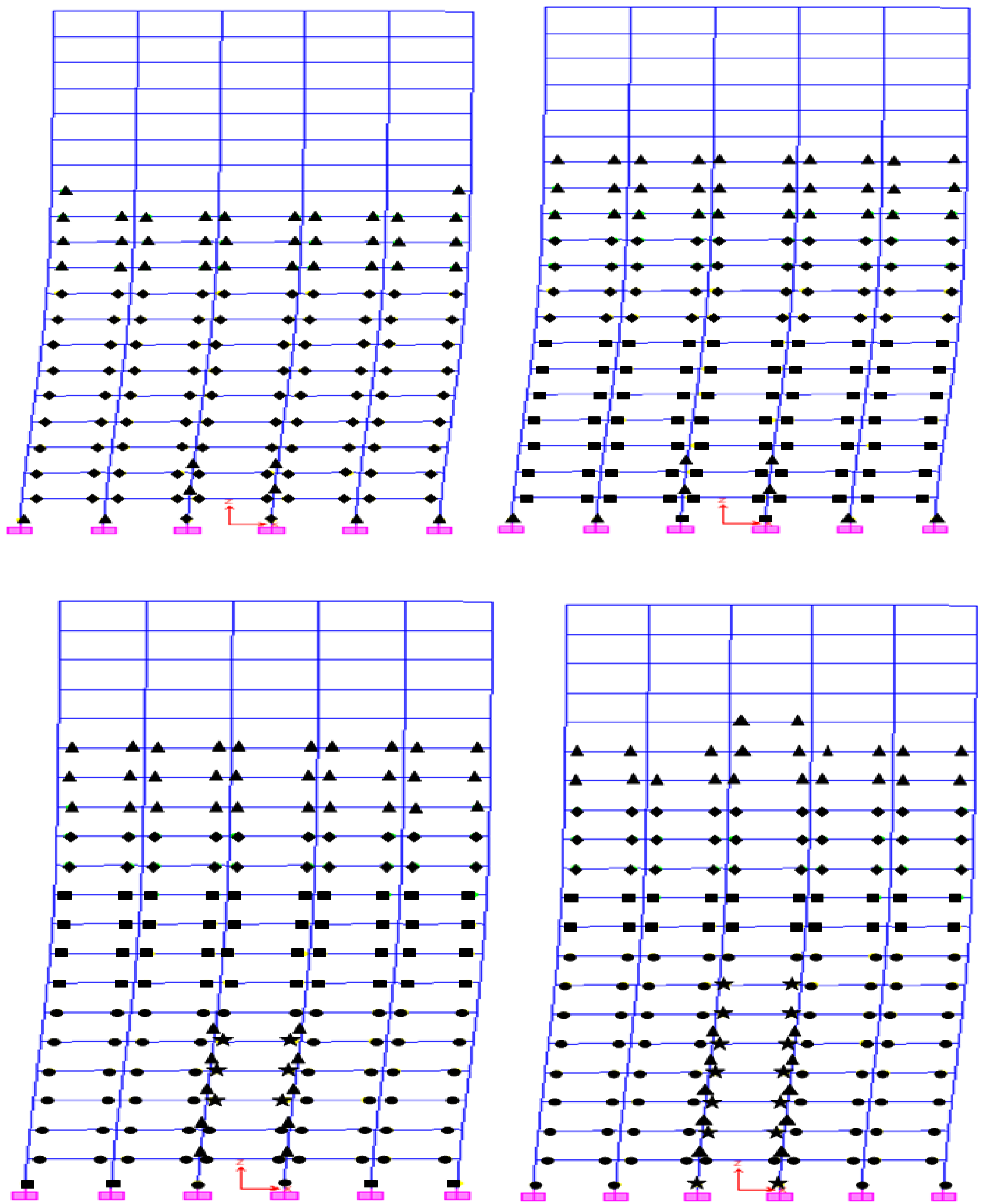

Figure D.3.3 Hinge patterns of Case III shear wall arrangement for different displacements levels