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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

ASSESSMENT OF THE DESIGN AND PERFORMANCE OF PRIMARY AND SECONDARY SEISMIC MEMBERS IN REINFORCD CONCRETE DUAL SYSTEM

A thesis submitted to the school of graduate studies at Addis Ababa Institute of Technology in partial fulfillment of the requirement for the degree of masters of Science in Structural Engineering

By

Daniel Abrha Teklu

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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.

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My mother, who has believed in me and had full confidence on me, deserves the highest recognition for this achievement. Her unconditional love and support has always given me the courage to keep on going.

To God, who has kept his eyes on me in every step I have taken; my praise and psalms are beyond words

ABSTRACT

From recent earthquake experiences, buildings with structural walls do have satisfactory performance against collapse. However, this is not always true for dual structures composed of both structural walls and frames resisting seismic actions. This begs for a detailed study on the performance of dual structures. Some studies show that the elastic response of dual buildings is mainly governed by the wall response and the frame contribution could be neglected, which then makes them a secondary seismic member. This scenario is reversed while the members are within their inelastic nature. These variations make it difficult to select and identify primary and secondary members based on stiffness limitation specified in codes which is only based on the linear response of the members. Without clearly identifying the distinction between these members in a structure, and without clearly knowing the boundary between these seismic members it is difficult to follow the simplified approaches of analysis in practice, in which lateral load is resisted by some selected and properly detailed members.

Hence, to address the effect of considering frames as secondary seismic members in RC wall-frame system, comparative analysis among sample dual system buildings has been carried out in the following thesis work. Efforts have been made to evaluate the overall seismic performance of the buildings using non-linear push over analysis method. For each building under study two cases are examined. In the first case frames and shear walls are designed and detailed as primary seismic members. In the second case however, frames are designed as secondary seismic members and shear walls are designed and detailed to be primary seismic resistance contributors. The results of non-linear push over analysis were obtained for each case in the form of capacity curves, global yielding point and performance point, plastic hinge formation patterns, and base shear distribution among frames and walls.

The observation of these results shows that the overall seismic resistance capacity of buildings reduces when frames are considered as secondary seismic members. In conclusion of this study; in RC wall-frame dual system, it is important to design both frames and walls as primary seismic members.

In addition, frames take up larger bases shear in the inelastic range than what is estimated in the linear analysis, which makes it incorrect to regard these members as secondary seismic members according to most building codes specification of base shear resistance proportion

between primary and secondary members. To minimize the effect of neglecting frames as secondary members, the stiffness limitation specified in code should be revised.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1. Background

Buildings with structural walls and frames as major lateral resistance contributors are known as dual system buildings. Frames are groups of beams and columns connected with each other by rigid joints. The deformation in frames and walls are not similar. Frame deformation is governed by the shear mode, whereas the walls behave as cantilevering elements supported at the foundation level hence their deformation is governed by the flexure mode. Due to this deformation variation between frames and walls, the structural walls are subjected to a pull by the frames at the top stories and a push at the bottom of a dual system building [18]. This shows that the frames highly contribute to the seismic resistance in the top story where their cross-section will be much larger than what is actually needed for gravity loading. On the other hand, the structural walls resist most of the lateral loads at the lower stories.

According to building codes currently in practice, enough capacity should be provided for structures in order to resists seismic forces without collapse. It is accepted that the structure may suffer damage when subjected to intense earthquake. However, the forces used to design the structures are defined based on elastic analysis, in which the damage that the structures may experience is considered in an approximate way. In reality the forces to which structures are subjected not only depend on the characteristics of the excitation but also on the dynamic properties of the structures in the linear and nonlinear range of behavior [19]. Therefore, it is not possible to estimate the behavior of structures through linear analysis when they are exposed to seismic forces that make them behave nonlinearly. Generally, to know the real behavior of structures it is required to adopt nonlinear step by step analysis techniques.

Recently, researches have been carried out to approximate the nonlinear behavior of building structures subjected to earthquakes using simplified methods of evaluation. A research carried out in 1995 by Camilo can be taken as a good example [19]. In this research it has been made possible to evaluate the behavior of regular buildings, with the restriction that they do not include in their formulations the contribution to the response of higher modes of vibration, which is of importance for structures of considerable height. Another aspect that these methods ignore is the diminishing lateral stiffness of members when subjected to intense earthquakes, with increasing levels of base shear, which in turn causes change in the distribution of the equivalent lateral forces to which the structure is subjected.

Although an elastic analysis gives a good indication of the elastic capacity of structures and helps to identify where first yielding will occur, it cannot predict failure mechanisms and does not account for redistribution of forces during progressive yielding [17]. As opposed to elastic analysis procedures, inelastic analysis procedures help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation is attempts to help engineers better understand how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with elastic procedures

Codes also contain procedures for linear and non-linear, dynamic or static analysis. In practice however, considering the financial and time constraints, the simplified methods are the ones mostly practiced. On the other hand, Performance-Based Seismic Design (PBSD) assesses structural behavior under seismic loads and tries to maximize the utility of the structure at a minimum expected cost. The main feature of PBSD is to evaluate seismic response in terms of displacements and not forces, which are the primary indicators of a structural damage.

Due to the random nature of seismic action and the uncertainties of the post elastic behavior of concrete structures, the overall uncertainty in analysis procedures is substantially higher than that which is considered in analysis without seismic actions. Therefore measures shall be taken to reduce uncertainties related to the structural configuration, analysis procedures, resistance of structures and the local and global ductility [26]. In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting systems are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. In modern codes, designers can select certain members for energy dissipation (primary seismic members), and the remaining members are considered as secondary members which will basically resist gravity load. The major question is: to what extent is the designer free to assign certain members as secondary or primary?

Studies show that, in dual system the contribution of frames and walls are quite different in the elastic and inelastic range [16]. As the structure behaves nonlinearly, frame contribution will increase. Thus nonlinear response of dual system which is designed based on code provision and capacity design should be assessed to check if the predefined plastic mechanism works or not.

"*In the wall-frame buildings, due to its higher stiffness in the elastic analysis, wall easily attracts higher portion of the horizontal loads. That makes it a "primary seismic element" and frames are designed for practically no horizontal loads. However, when the frames are exposed to expected non-linear displacements of the system then induced forces are much greater than the ones obtained during the linear elastic analysis. And frames suffer due to these increased loads since they are under designed*" [16].

The fact that the frames are subjected to greater loads in the inelastic range makes it difficult to select and identify primary and secondary members. Without clearly knowing the classification boundary between these seismic members it is difficult to follow the simplified approaches, in which lateral load is resisted by some selected and properly detailed members. On the other hand, Secondary seismic elements are severely penalized by being required to remain elastic in the seismic design situation.

The intention of this thesis is therefore; to assess and evaluate the design and performance of Primary and secondary seismic members of Reinforced concrete (RC) dual system. The following two general cases have been investigated in the following study in order to evaluate the outcome of categorizing some members as secondary and the effectiveness of members assigned to be primary.

1) All members are designed and detailed as primary seismic members for Ductility Class Medium (DCM).
2) Shear walls are designed and detailed as primary members, whereas columns and beams are considered as secondary seismic members.

1.2. Objective of the study

The main objective of this study is to assess the design and performance of primary and secondary seismic members of RC dual system using nonlinear static analysis. This study is carried out by conducting a comprehensive literature survey and by making comparative analysis among sample buildings with different story heights and different percentage share of frames against base shear.

The following points describe the objectives of this thesis work:

1) To understand design of a structure in accordance to Euro codes, Capacity design rules.

2) To assess and follow non-linear analyses procedures, i.e. the Pushover Analysis.
3) To evaluate the response of primary and secondary members and their contribution on the performance of the structure by using pushover analysis. This includes addressing the following issues:
   - Ways in which secondary system stiffness limitation requirement influence the geometry and design of the structure
   - Detailed studies on the consequence of modeling frame as secondary seismic element.
   - Effectiveness of energy dissipating members

2. THEORETICAL BACKGROUND

2.1. Seismic action determination and design philosophies

2.1.1. Determination of seismic action

The size of an earthquake can be measured in terms of intensity and magnitude. Intensity is the measure of damage to structures, ground surface effects, and reactions of humans to earthquake shaking. It is non-instrumental or qualitative method because it is dependent on density of population, familiarity with earthquake and type of structures.

Although there are several scales developed in pre-instrumental times, Modified Mercalli Intensity (MMI) scale is the most commonly used. It is a subjective scale defining the level of shaking at specific sites on a scale of I to XII.

Table 2-1: Modified Mercalli Intensity Scale of 1931 (after Wood and Neumann 1931) [13]

I	Not felt except by a very few under especially favorable circumstances
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing track. Duration estimated
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, and doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rock noticeably
V	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving motor cars

VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed over banks
XI	Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly
XII	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air

Unlike intensity, magnitude is instrumental, quantitative and objective scale used to measure the size of earthquake. For the first time earthquake magnitude is defined by Richter (1935). Then after, several other magnitudes have been defined. Surface wave magnitude (Ms), body wave magnitude (Mb), and moment magnitude (Mw) have been widely in use.

Earthquake loads are non-static (dynamic) type of lateral loads. They are very complex, uncertain, and potentially more damaging than other lateral loads (e.g. wind loads). It is quite fortunate that they do not occur frequently. Earthquakes create ground movements that can be categorized as a "shake," "rattle," or a "roll." Every structure in an earthquake zone must be able to withstand all three of these loadings with different intensities. Even if 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. This is because of the assumption 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.

Earthquake loads are designed as if they are horizontally applied to the structural system. And it is considered as an instantaneous force (not constant). The mass of the structure, the stiffness of the structural system and the acceleration of the surface of the earth are factors affecting the magnitude of earthquake loads.

According to Euro codes 8 [1] depending on the structural characteristics of the building, there are two types of analysis for determining the seismic effects.

- Lateral force method (Equivalent Static analysis): for building that satisfies the criteria for regularity in plan & elevation and structures whose fundamental periods of vibrations in the two main directions are less than two seconds.
- Dynamic analysis: it is applicable to all types of Buildings.

2.1.2. Earthquake resistance 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. An attempt to make earthquake proof buildings that will not get damaged even during the rare but strong earthquake events is unrealistic and will not be economical. Thus the objective in designing earthquake resistant buildings is to avoid collapse during strong earthquake. This implies that the buildings are designed to resists the effect of ground shakes, although they may get damaged severely.

The general philosophy of earthquake resistant building design is that:

1) Under minor but frequent shaking, the structural members of the buildings that carry vertical and horizontal forces should not be damaged; however non-structural building parts that do not carry load may sustain repairable damage.
2) Under moderate but occasional shaking, both structural & non-structural members may sustain repairable damage.
3) Under strong but rare shaking, the structural and non-structural members may sustain severe damage, but the building should not collapse.

Therefore, Earthquake design philosophy is mainly 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.

2.2. Dual structural systems

There are many types of lateral load resisting systems in buildings. Most of these systems can be categorized under the following three groups:

1) Frame system
2) Wall system
3) Dual system (Wall – Frame system)

2.2.1. Frame System

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. In an earthquake, a frame with suitable proportions and details can develop plastic hinges that will absorb energy and allow the frame to survive actual displacements that are larger than the displacement calculated in an elastic-based design. In modern moment frames, the ends of beams and columns, being the locations of maximum seismic moment, are designed to sustain inelastic behavior associated with plastic hinging over many cycles and load reversals.

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 [20].

The proportioning and detailing requirements for moment resisting frames are intended to ensure that the 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. Wall System

Shear walls, as the name implies, resist lateral forces primarily in shear. In the analysis of shear walls, it is customary to consider the shear taken by the length of the wall and the flexure taken by vertical reinforcement added at each end, much as flexure in diaphragms is designed to be taken by chords at the edges. Squat walls that are long compared to their height are dominated by shearing behavior. Flexural forces require only a slight local reinforcement at each end. Slender walls that are tall compared to their length are usually dominated by flexural behavior, and may require substantial boundary elements at each end. When the earthquake direction being considered is parallel to a shear wall, the wall develops in plane shear and flexural forces as described above.

When the earthquake direction is perpendicular to a shear wall, the wall contributes little to the lateral force resistance of the building and it is subjected to out-of-plane forces tending to separate it from the rest of the structure. Thus for tall walls compared to their length, the behavior of a shear wall is opposite to what its name suggest. Such shear walls primarily resist the lateral load in flexure with very little shear deformations. The deformation of shear wall is different than that of frame. Therefore, when used in conjunction with frame, shear wall results in complex interaction with the resultant lateral load on the shear wall and frame varying in a complex manner along the height.

2.2.3. Dual system (Wall –Frame)

Pure frame lateral load resisting systems are not recommended in high seismicity areas for more than three or four storey RC buildings because moment resisting frames are relatively flexible and exhibit larger ductility demands when subjected to severe seismic action, which make them sensitive to second order effects. As a result, during construction of such frames careful detailing is required to achieve sufficient ductility capacity at critical zones. In practice though, it is difficult or rarely achievable. Therefore from structural point of view, it is usually preferred to include RC walls in the lateral load resisting systems to have increased stiffness and reduced deformation demands. It is also easier to construct structural wall with adequate flexural and shear strengths than that of ductile frame system. [5]

Dual system or hybrid system are most commonly used terms to indicate that the lateral load resisting system is provided by combined contribution of frames and shear walls. This structural system combines the advantageous features of the constituent elements.

Ductile frame interacting with walls can provide a significant amount of energy dissipation, when required, particularly in the upper stories of the building. And also, as a result of the large stiffness of walls, good story drift control during an earthquake can be achieved, and the development of story mechanisms involving column hinges (i.e. soft stories), can readily be avoided.

As discussed in Paulay T, and Priestely M.J.N on Seismic Design of Reinforced concrete & Masonry Buildings [4]; even if it is advantageous to use wall – frame lateral load resisting system, it is only recently that research effort has been directed toward developing relevant seismic design methodologies. This research, involving analytical studies of existing building and experimental work, using static and shake table tests, and has indicated a potential for excellent inelastic seismic response. Under the action of lateral forces, a frame will deform primarily in a shear mode, whereas a wall will behave like a vertical cantilever with primary flexural deformations, as shown if figure 2-5 (b) and (c). Compatibility of deformations requires that frames and walls sustain at each level essentially identical lateral displacements [figure 2-5 (d)]. Due to the preferred displacement mode of the two elements shown in fig 2-5 (b) and (c), it is found that the walls and frames share in the resistance of story shear forces in the lower stories, but tend to oppose each other at higher levels.

It is also indicated that the mode of sharing the resistance to lateral force between walls and frames of dual system is influenced by the dynamic response characteristics and development of plastic hinges during a major seismic event, and it may be quite different from that predicted by an elastic analysis. As a result, simplified elastic analysis procedures are likely to be misleading in case of dual system. Particularly, the most recent practice of allocating portions of the lateral forces to the frames and the remaining to the walls each of which are then independently analyzed is entirely inappropriate.

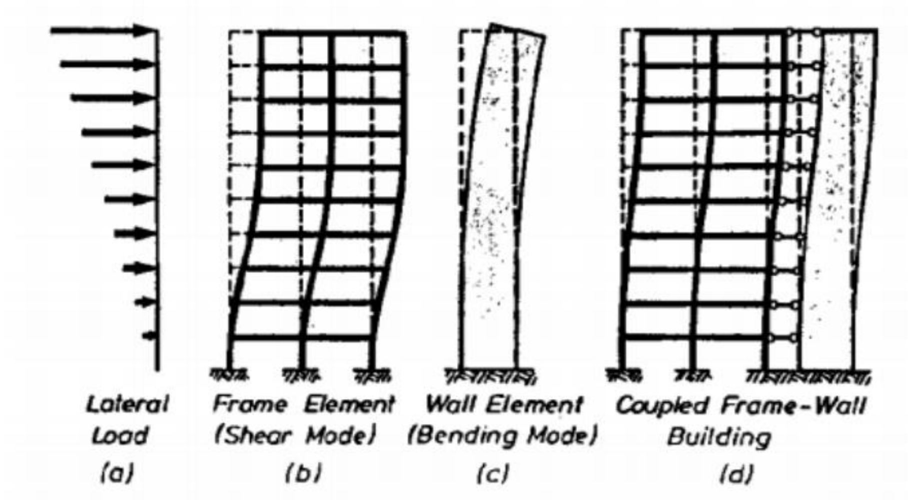


Figure 2-1: Deformation pattern due to lateral force [4]

2.2.4. Wall frame interaction

The behavior, and design, of dual wall frame systems has similarities to that of coupled walls. The typically large stiffness variation between the frames and the walls will mean that walls yield at significantly lower lateral displacements than the frames do, and hence distribution of lateral force between walls and frames based on initial elastic stiffness has little relevance to ductile response of the structure. Consequently, as suggested by Paulay T [14], similar freedom is available to the designer in choosing the share of lateral resistance provided by walls and frames as has been suggested for coupled walls. The designer may choose a portion of the base shear force carried by the frames based on experience and judgment rather than on elastic analysis based on generally invalid estimates of wall and frame stiffness. Typically the portion of base shear carried by the frames will be between 15 % and 50% of the total base shear – rather less than for frame action with coupled walls – but the value will depend on the size of the walls, and the relative numbers of frames and walls in the structural configuration.

According to a paper presented on the 14th World Conference on Earthquake Engineering, by Sigmund, Guljasand, and M.Hadzima-Nyarko [16], the experience provided by the recent earthquakes showed that buildings with structural walls have had satisfactory performance for collapse prevention. Conversely, some of the dual system buildings (wall-frame) were rated as severely damaged therefore implying an urgent need for strength assignments redefinition between the structural components. The elastic response of dual buildings is mainly governed by the wall response and the frame contribution could be neglected (less than 15% of the base shear what makes them a secondary member). In the inelastic range the situation could drastically change and frame contribution increases (even up to the 50%) but these elements are then under-designed and not capable to carry the increased load.

2.3. Earthquake response analysis

Seismic analysis consists of the determination of structural response during an earthquake. For seismic performance evaluation a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. The different methods used throughout this study are described as follows.

2.3.1. Linear analysis

In this study Lateral force method (Equivalent static analysis) is applied for linear analysis and design. The lateral force method is a relatively easy, less time-consuming, and can be adopted for several types of structures within its limitation. Furthermore, it is a straight-forward method for determining forces and displacements of structures excited by the earthquake. Moreover, the contributions of the higher modes are not accounted without affecting the global response, i.e. the base shear and overturning moment.

Limitation of the procedure

The lateral force method is only applied when the effects of higher modes are insignificant and the fundamental translational mode in the direction of the applied lateral forces governs the response. Euro code 8 [1] gives the following restrictions:

- The fundamental period of the building is less than 2sec or 4 times the corner period T_c
- The building must fulfill the requirement for regularity in plan and elevation

Fundamental Period and Base shear

Euro code 8 [1] promotes different approaches to find the fundamental period, T_1, which estimate or define its determination analytically. It must be noted that the linear static method can only be used on an elastic building model. The seismic shear above the foundation or the top of a rigid basement (base shear), F_b, is separately determined in horizontal directions X and Y, on the basis of the 1_{st} translational mode period and direction of interest. Thus,

$$F_b = S_d(T_1).m.\lambda$$

Where: $S_d(T_1)$ is the design spectral acceleration at the fundamental period, m is the total mass of the building and λ is a correction factor defined by the number of stories in the building and the fundamental period.

Pattern of Lateral Loads

The base shear defined by the equation below is the resultant of a set of inertia forces on the masses m_i associated with degree of freedom *i* in the horizontal direction. Those lateral forces are defined as:

$$F_i = F_b \cdot \frac{h_i . m_i}{\sum (h_i . m_i)}$$

Where: mi is the mass and hi is the height of the floor

There are several studies which focus on the comparisons of seismic code provisions in different codes. The comparisons are mostly based on importance factor, zone factor, time period, and structural system factor.

According to a study by Noor, Ansary and Seraj [15], it is concluded that almost all building codes, considered on the study (Uniform Building Code [UBC] 1994, National Building Code of Canada [NBC] 1995 and Building Standard Law of Japan [BSLJ]) adopt a similar definition for the various coefficients in the equation of the base shear determination using the equivalent static method. However a direct comparison of seismic forces is not possible because there are large differences in the seismic intensity from country to country, leading to differences in the design values of zone factor.

The requirements of equivalent static method of analysis are primarily intended to provide life safety, not property protection, at the maximum expected earthquake level. Observation of structural system responding in the inelastic range indicate that as the structure yields, the period, damping, and other dynamic properties change, often substantially. The effect of this change in dynamic properties is that while the force levels actually experienced by the structure are greater than those used in the design, they are less than those that would occur in a fully elastic response. The more ductile the system's performance is, the greater its capacity to accommodate inelastic displacements and forces.

The development of earthquake resistance design regulation is considered to be revised on a regular basis. It is well recognized that structures designed by current codes undergo large inelastic deformations during major earthquakes. However, lateral force distributions given in the seismic design codes are typically based on results of elastic-response studies.

In the inelastic range of structural behavior (which is expected to be severe for large values of the behavior factor, q), local ductility demands may not be adequately captured by code-prescribed linear types of analysis methods. As a result, the code-compliant seismic design process becomes inherently less reliable in accounting for and properly verifying the expected local ductility demands for structural layouts of increased complexity [12]. Generally, seismic analysis is more reliable when non-linear methods are applied and deformations in post-elastic domain are determined. Furthermore, the linear static analysis does not account for the variation of the modal properties when the structure responds in the post-elastic domain.

2.3.2. Non-linear static (Pushover) analysis

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty five years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post-elastic behavior.

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached.

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 a structure. When such programs are not available or the available computer programs could not perform pushover analysis directly, a series of sequential elastic analyses is performed and superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bi-linear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

Depending on the physical nature of the load and the behavior expected from the structure, pushover analysis can be performed as either force-controlled or displacement-controlled. When the load is known (such as gravity loading) and the structure is expected to be able to support the load, force-controlled option is useful. 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

In force controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. To overcome this problem pushover analysis is performed as displacement-controlled [17]. In which the magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement.

Advantages of Pushover Analysis

Pushover analysis is used to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provides information on many response characteristics that cannot be obtained from an elastic-static or elastic-dynamic analysis [17]. Such as:

- The overall structural behaviors and performance characteristics.
- The sequential formation of plastic hinges in the individual structural elements constituting the entire structure.
- When a structure is to be strengthened through a rehabilitation process, it allows us to selectively reinforce only the required members maximizing the cost efficiency
- Provides good estimate of global and local inelastic deformation demands for structures that vibrate primarily in the fundamental mode.
- Verification of the completeness and adequacy of load path

- Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range
- Estimates of inter-story drifts and its distribution along the height

Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

Limitations of Pushover Analysis

- Deformation estimates obtained from a pushover analysis may be grossly inaccurate for structures where higher mode effects are significant.
- In most cases it will be necessary to perform the analysis with displacement rather than force control, since the target displacement may be associated with very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.
- Pushover analysis relates damage with only the lateral deformation of the structure, neglecting duration effects, number of stress reversals and cumulative energy dissipation demand.
- The procedure does not take into account the progressive changes in modal properties that take place in a structure as it experiences cyclic non-linear yielding during earthquake.
- The most critical concern that the pushover analysis may detect only the first local mechanism that will form that will form in an earthquake and may not expose other weaknesses that will be generated when the structure's dynamic characteristics change after formation of first local mechanism.

2.3.3. Lateral load distribution for pushover analysis

There are three types of vertical distribution of lateral loads for pushover analysis [21].

1) **"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, and m_i: mass of i-th story

2) **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 j^{th} floor, h_j is the height of the j^{th} 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.

Since Lateral load obtained from FEMA-356 [8] is categorized under this type of load, the lateral force distribution method is adopted in this research. This load pattern is effective for regular buildings whose mass participation is above 75% in the first mode.

3) **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.

2.3.4. 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 [2]. Therefore, 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 [8].

The building performance level is a function of the post event conditions of the structural and non-structural components of the structure [8]. 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.

Based on Federal Emergency Management Agency documents, FEMA-273 [7] and FEMA-356 [8], 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 nonstructural performance level is defined as the post-event conditions of the non-structural components, which is divided into five levels. As a result, the performance level of building is a combination of the performance level of both structural and non-structural components.

Based on possible combination of structural and non-structural components of a building, there are several performance levels. However, the well-established and commonly used building performance levels are operational (O), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The following table which is taken from table C1-2 of FEMA 356 [8] shows expected damages on each level.

Table 2-2: Damage Control and Building Performance Levels (From FEMA-356 table C1-2)

	Target Building Performance Levels			
	Collapse Prevention Level (5-E)	**Life Safety Level (3-C)**	**Immediate Occupancy Level (1-B)**	**Operational Level (1-A)**
Overall Damage	**Severe**	**Moderate**	**Light**	**Very Light**
General	Little residual stiffness and strength, but loadbearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-loadbearing 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.	Less damage and lower risk.	Much less damage and lower risk.

Primary elements of a performance based design procedure (i.e. demand, capacity, and performance) can be determined using different simplified non-linear analysis methods including Capacity spectrum method (CSM) and Displacement Coefficient Method (DCM).

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. The demand curves are shown for different effective damping values (βeff.). Figure 2-6, taken from Applied Technology Council (ATC-40) shows the performance point obtained by CSM.

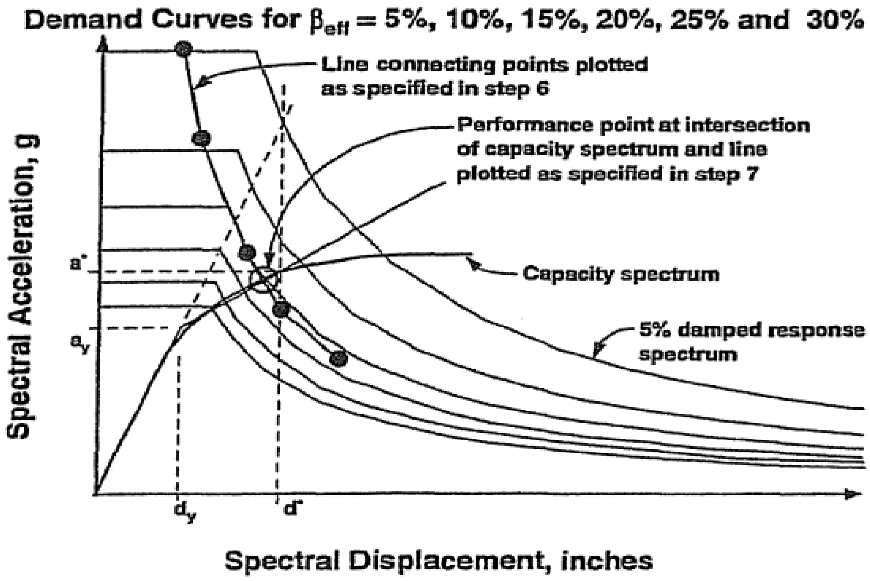


Figure 2-2: Performance point obtained by capacity spectrum procedure (ATC-40, 1996)

Displacement Coefficient Method is the other non-linear analysis procedure for which FEMA-273 [7] & FEMA-356 [8] are used as guidelines. This method provides 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. 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 [2].

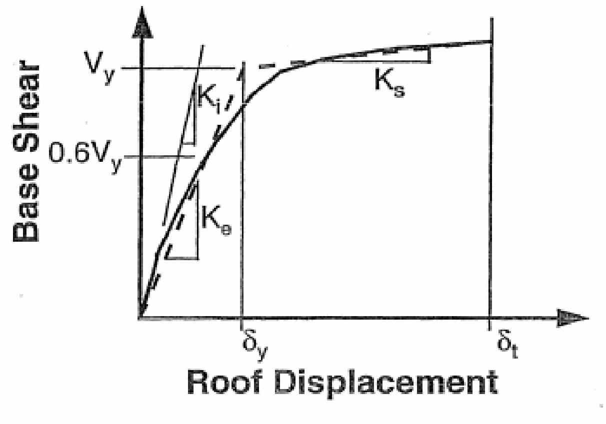


Figure 2-3: Bilinear representation of capacity curve for displacement coefficient method

2.4. Seismic members

2.4.1. Definition of primary and secondary seismic members

According to different building codes, primary seismic members are members which are designed and detailed to dissipate earthquake induced forces. Whereas, secondary seismic members are only designed to carry gravity loads. The definitions for primary and secondary members in different codes have similar interpretations. The classification of members into “primary” and “secondary” (as they are called in Euro codes) is equivalent to the old-time distinction in US seismic design codes for new buildings of members which are part of the lateral- (or seismic-) load-resisting system from those that are not. In Euro code 8 [1] the qualification “seismic” has been added to “primary” or “secondary”, to make it clear that the differentiation applies only for the seismic action.

According to New Zealand building code [3], all members in a building normally participate in carrying the applied vertical load but not all members are necessarily designed to resist applied lateral forces from wind or earthquake. It is important that the secondary members are designed and detailed to conform to the deformations that may be imposed on them by the primary system. These deformations can cause significant localized lateral forces to be developed between vertical load resisting members and horizontal floor diaphragms.

2.4.2. The need to classify members as primary and secondary seismic elements

Most current seismic codes recognize that certain structural members may have a secondary role and contribution to earthquake resistance. The main objective of this distinction is to allow for some simplification of the seismic design by not considering secondary elements in the structural model used for seismic analysis of the buildings [6].

The contribution of “secondary members” to stiffness and resistance against seismic actions is not included in the structural model for the seismic analysis of new buildings. Only the primary members are designed and detailed for earthquake resistance, in accordance with all the relevant rules in the seismic design code. “Secondary members” are fully considered and designed for the non-seismic actions and are subject to special verifications under the design seismic action and the concurrent gravity load.

In seismic assessment and retrofitting of existing buildings, “secondary members” are less important for the performance and safety of the whole structure. Therefore, more severe seismic damages in “secondary members” are accepted. Accordingly, their verification criteria for the seismic action are relaxed compared to those of “primary members”. According to Euro code 8 [1] in existing or retrofitted buildings the contribution of “secondary members” to stiffness and resistance against seismic actions should be neglected in the model for linear seismic analysis, like in new buildings, but should be included in a nonlinear analysis model. Note, however, that, unless “secondary members” are fully included in the model for the seismic actions, it is not easy to check whether they meet their (relaxed) compliance criteria. [6]

Other reason for the designer to consider some of the members of new building designed for ductility as “secondary” is when they are not within the scope of the rules for seismic design based on energy dissipation and ductility. For instance, flat slab frames and post-tensioned girders are not covered by seismic design codes.

The designer may also want to consider as “secondary” those members of a new building that – owing to architectural constraints – cannot be made to conform to the seismic design rules for geometry, dimensioning or detailing for energy dissipation and ductility.

On the other hand, from point of view of economy, a structural system that cannot be utilized in its entirety for the engineered earthquake resistance of the building is a waste of resources.

2.4.3. Constraints on secondary seismic members

Now based on the definition given in most building codes, it is clear that secondary seismic members do not participate in the lateral load resisting system, but it is up to the designer to decide which members should be consider as "secondary".

To establish a limit to the discretion of the designer of a new building, Part 1 of Euro code 8 [1] gives three conditions to be met:

- The total contribution to lateral stiffness of all "secondary members" should not exceed 15% of that of all "primary" ones.
- The characterization of some of the structural members as "secondary" should not change the classification of the structure from irregular to regular.
- "Secondary members" should meet the special requirements applying to them

2.4.4. Modeling of Secondary seismic members

In seismic design of new buildings based on linear analysis the strength and stiffness of "secondary members" against lateral loads is neglected for the seismic action, but it may be considered for all other actions (e.g. gravity loads).

Then, two different structural models may be used for the linear analysis.

1) A model that completely neglects the contribution of "secondary members" to lateral stiffness.
2) Another which includes fully the "secondary members".

Then the seismic deformation demands in the "secondary members" may be obtained in a two-step procedure:

1) The elastic deformation demands in the "secondary members" due to the design seismic action are estimated from a linear seismic analysis using Model no. 2 (with the design spectrum, but removing afterwards the effect of the behavior factor, *q*, from displacements and deformations).
2) The outcome of Step I for storey *i* is multiplied by the ratio of interstorey drifts in that storey using Model no. 1 in the linear analysis for the design seismic action to those using Model no. 2.

According to M.N. Fardis [6], using two different structural models is not convenient, especially if linear analysis and design take place in an integrated computational environment. Among other problems, we have to use in the verifications. If it is not needed to design the building for another lateral action, e.g., wind, it may be possible to use in some cases a single structural model for the seismic action and for gravity loads. In which "secondary members" are included only with those properties that are essential for their gravity-load-bearing function.

2.4.5. Capacity design of structures

In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting systems are chosen and suitably designed and detailed for energy dissipation under severe imposed deformations. Parts of a structure intended to remain elastic in all events are designed so that under maximum feasible actions corresponding to over strength in the plastic hinges, no inelastic deformation should occur in those regions. A clear distinction is made with respect to the nature and quality of detailing for potentially plastic regions and those which are to remain elastic in all events. Primary aim of the capacity design will be to prohibit formations of a soft story and as a corollary, to ensure that the only mechanism shown in the following figure, fig 2-8 (a) can develop.

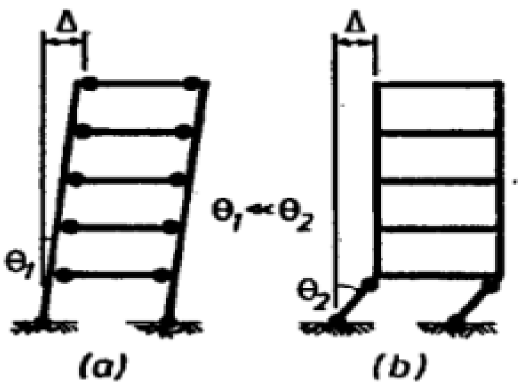


Figure 2-4: Comparison of energy dissipating mechanisms

As per the discussion of Paulay T. and Priestley [4], on the capacity design for dual system: The dominant feature of the capacity design strategy is the prior establishment of a rational hierarchy in strength between components of the structural system. Accordingly, the approach to the design of each primary lateral force resisting components of a dual system, which is to be protected against yielding or a brittle failure, will be carried out.

3. MODELING AND ANALYSIS

3.1. General

Model buildings taken in this research are selected to fulfill the requirements of regularity both in plane and elevation, so that the equivalent static lateral force method can be used when conducting linear seismic analysis. The structures are modeled in 3 dimensional platforms. As it has been discussed in the previous chapters, two separate models with different features regarding secondary members have been analyzed. For the fully detailed models, all members (frames and shear walls) are designed to be primary seismic members and detailed to satisfy the Ductility Class Medium (DCM) requirements. However, in the partially detailed models the shear walls are designed as primary seismic members satisfying the requirements of DCM, whereas beams and columns are considered to be secondary seismic members.

To assess the design and performance of primary and secondary seismic members of RC dual system, six types of building structures are established in two different categories for parametric study. In the first category three buildings are modeled with varying number of storeys but keeping the percentage share of frames against lateral load to be with in the same range. In the second category however, three buildings are modeled for identical number of storeys by varying percentage share of frames against lateral load.

In all of the models, the size of columns and shear walls increased realistically as the number of story is increased. Each model has two cases of study based on contribution of members against lateral load, i.e. fully detailed and partially detailed.

3.2. Materials

The material properties that are allowed for DCM structures are of Class B or C for reinforcement and Class C16/20 or higher for concrete [1]. Therefore, reinforcement of class B-400 and concrete grade of C20/25 has been used. The partial safety factor for concrete γ_C = 1.5 and for steel γ_S = 1.15, were set.

For the nonlinear model, Mander stress-strain relation is adopted for concrete with compressive strain of 0.002 at maximum stress and ultimate strain of 0.0035. The kinematic nonlinear model is used for reinforcement with strain at onset strain hardening of 0.01 and ultimate strain capacity of 0.1.

3.3. Loads

Slab panels are first designed and load is transferred to the beams according to coefficient method of slab analysis. For linear analysis earthquake load is calculated assuming soil class C and location of the buildings to be in high seismic zone. For the nonlinear static analysis the inverted triangle lateral load pattern is used and calculated according to FEMA 356 [8]. Appendix A and Appendix B shows lateral load calculation for linear and non-linear analysis respectively.

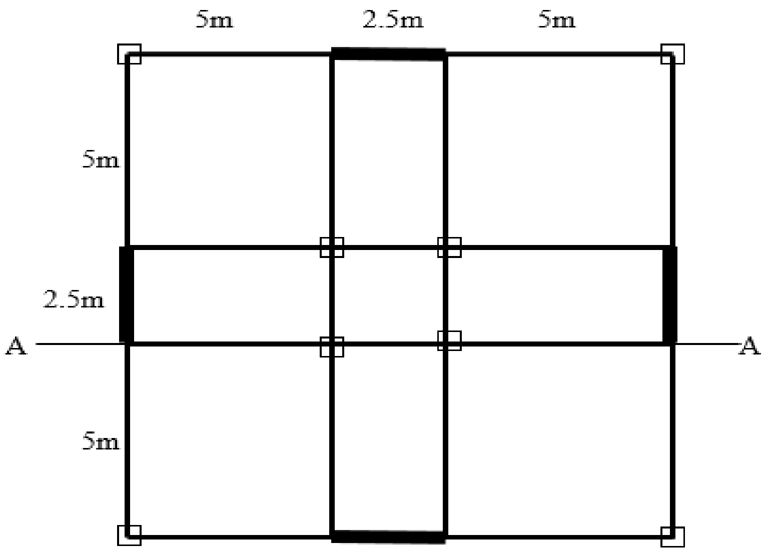
3.4. Description of the buildings

On every model building considered in this study, members were sized based on the percentage contribution of frames and shear walls against lateral load. In all cases frames carry below 15 % of the total base shear (based on linear analysis, to satisfy the requirements of secondary members). Therefore, to arrive at the finally considered member sizes several trials has been made and the sizes of members presented on the detail description of each model are the final cross sections. Since the shear walls carry more than 85 % of the lateral load, the buildings are categorized under wall equivalent structural system.

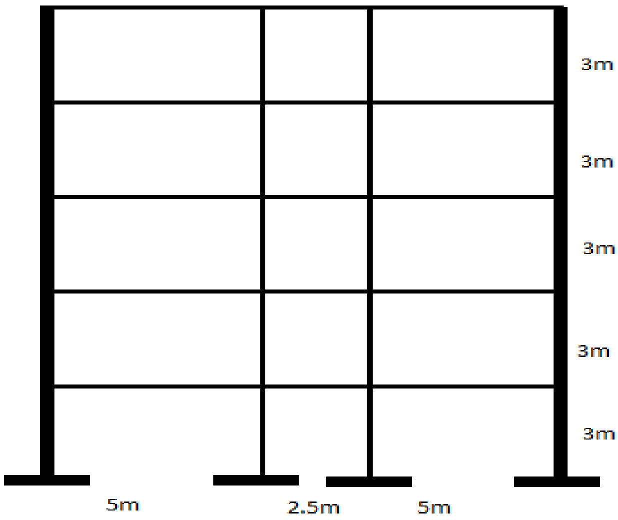
3.4.1. Category 1: Varying number of stories

In the first category three buildings are modeled having 5, 9 and 13 storeys. The percentage contribution of members against lateral load is kept in the same range in all three models. Design results of these models are presented in appendix D.

Model 1-1: The first model is a five-story dual structural system having plan and elevation view as shown in figure 3-1. The size of columns in this building is 400 x 400 mm and the size of beams is 300 x 300 mm. The thickness of shear wall is 200 mm while its length is equal to 2.5 m. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid. The plan area of this model is 12.5 m X 12.5 m. The linear analysis result shows that 89.15 % of the base shear is taken by shear walls and 10.85 % is taken by frames.



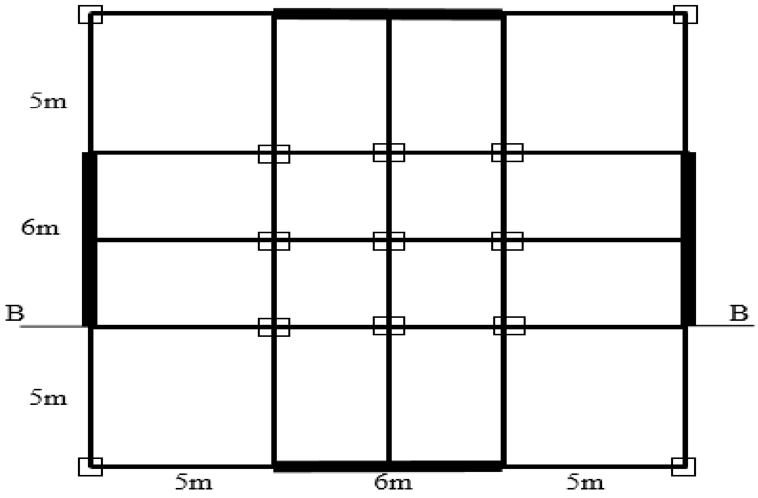
(a) plan view of model 1



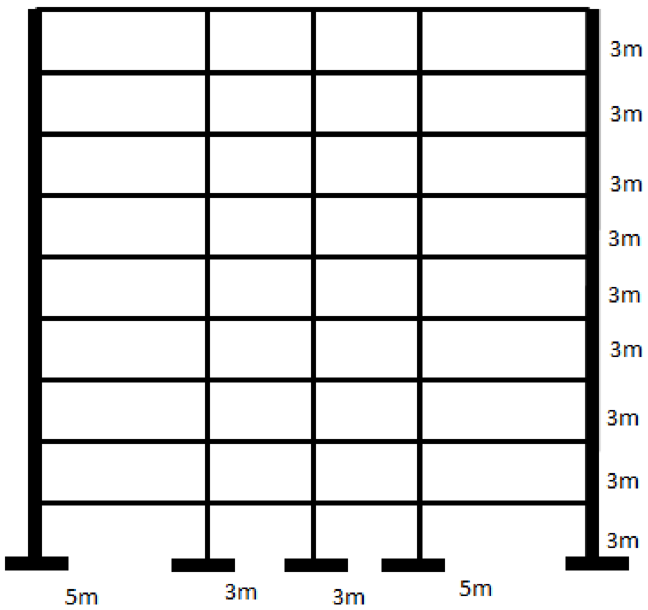
(b) Section A-A

Figure 3-1: Plan and section view of Model 1-1

Model 1-2: The second model is a nine-story dual structural system having plan area of 16 m by 16 m. plan and elevation view is shown in the figure 3-2. The sizes of columns used in this building are 400 X 400 mm, 500 X 500 mm, and 600 x 600 mm varying from top to bottom storeys. The size of beams is 300 x 300 mm. The thickness of shear wall is 200 mm while its length is equal to 6 m. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid. According to linear analysis results, frames take 11.27 % of the total base shear and shear walls take 88.73%.



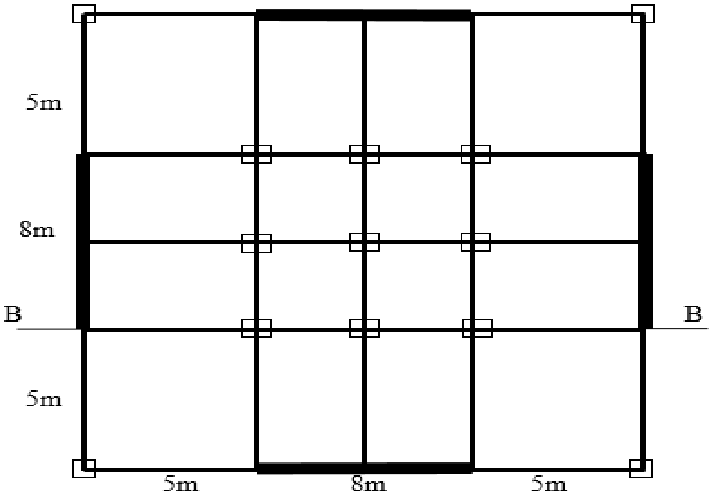
(a) Plan view of model 2



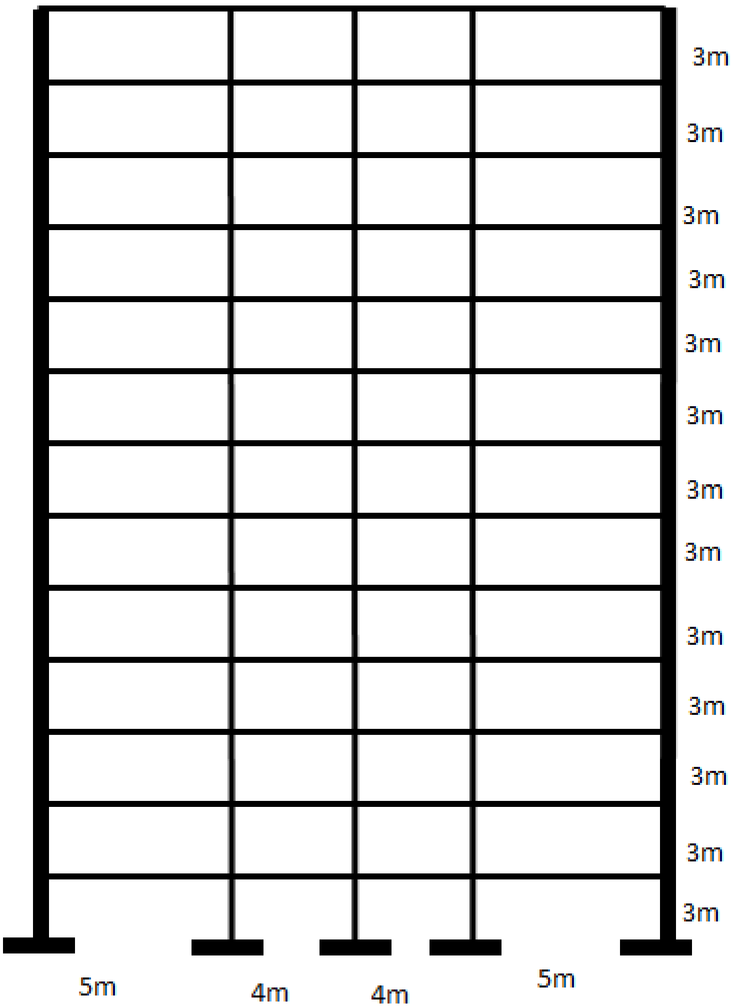
(b) Section B-B

Figure 3-2: Plan and section view of Model 1-2

Model 1-3: The third model is a thirteen-story dual structural system having plan area of 18m X 18 m as shown in the figure 3-3. The sizes of columns in this building are 400 X 400 mm, 500 X 500 mm, 600 x 600 mm, and 700 X 700 mm and the size of beams is 400 x 400 mm. The thickness of shear wall is 250 mm while its length is 8 m. The floor height is taken as 3.0 meters throughout the building. The foundation is assumed to be structurally rigid. Linear analysis results show that frames take 11.15 % of the total base shear and shear walls take 89.85%.



(a) Plan view of model 3



(b) Section B-B

Figure 3-3: Plan and section view of Model 1-3

3.4.2. Category 2: Varying percentage share of members against lateral load

In this category the number of storeys is kept the same while the percentage share of members against lateral load is varying. The three models used in this category have 9 storeys and identical floor plan and elevation. The floor plan and elevation for models used in this category is similar with the second model of category 1 (shown in figure 3-2). The percentage share of frames against lateral load is 9.82 %, 11.27 %, and 13.12% for building model 1, model 2, and model 3 respectively.

3.4.3. Description of case study

Based on the contribution of members to lateral force, there are two 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 1: Fully detailed model

In this case all members are designed to carry both gravity and lateral load and detailed to satisfy the requirements of DCM. The frames carry less than 15 % of the total base shear, whereas the remaining base share is carried by shear walls. Since all the members in these models are detailed, the case study name in this documentation is fully detailed models.

Case 2: Partially detailed model

Similar to case 1 frames carry not more than 15 % of the base shear and the remaining percent is carried by shear walls. However, the contribution of beams and columns for lateral load is neglected and they are designed as secondary seismic members, whereas, Shear walls are designed as primary seismic members and detailed to satisfy the DCM requirements.

3.4.4. Design of sample building models

Each sample building is designed according to the Euro codes seismic design requirements. They are assumed as an office buildings located in a high seismic zone. 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 both cases, where the contribution of secondary members against lateral load is neglected and considered. Second-order effect is checked by evaluating sensitivity factor for the selected behavior factor. (Appendix C shows calculations of sensitivity index coefficient)

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 Euro code. Since the plan areas of each floor of the building models are the same, all floors will have the same center of mass at the geometric center. The point at which lateral load is applied in order to account for accidental torsional effect of each floor is taken as 5 % of the length of the building. 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.

Based on clause 5.4.2.1(1) of Euro code 8[1], the design values of bending moments and axial forces are obtained from the analysis of the structure for the seismic design situation. Capacity design requirements for columns in bending at beam/column joints do not apply in this study, as the buildings are classified as wall-equivalent structural system. However, according to clause 5.4.2.3(1) of Euro code 8 [1], the design values of shear forces are determined in accordance with the capacity design rule, on the basis of the equilibrium of the column under end moments that correspond to the formation of plastic hinges at the ends of the beams connected to the joints into which the column end frames, or at the ends of the columns (wherever they may form first).

In 5.4.2.3(1) the end moments are defined as $M_{i,d} = \gamma_{Rd} M_{Rc,i} \min (1, \sum M_{Rc} / \sum M_{Rb})$, where γ_{Rd} is a factor accounting for over-strength due to steel strain hardening and confinement of the concrete in the compression zone of the section, $M_{Rc,i}$ is the design value of the column moment of resistance at end i, $\sum M_{Rc}$ and $\sum M_{Rb}$ are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively ($\gamma_{Rd} = 1.1$ for DCM).

3.5. Evaluation techniques

On this research, the effects of neglecting contribution of frames (secondary seismic members) against lateral load in the RC dual system building shown in pervious section is evaluated by comparing the overall seismic performance of the fully detailed and partially detailed buildings. Seismic performance of 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 parameters used to evaluate the design and performance of primary and secondary seismic members are lateral load resisting capacity curve, global yielding point and performance point, plastic hinge mechanism, and percentage share of base shear among frames and walls.

The behavior of the structure in nonlinear pushover analysis 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. In addition to this, performance point or target displacement is one of non-linear pushover analysis parameters which may be used for performance evaluation purpose. The other parameter in pushover analysis, which is used for performance evaluation, is plastic hinge mechanism. The hinging pattern provides 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.

3.6. Modeling approaches

Modeling rules presented on Applied Technology Council (ATC)-40 [2] is used as a guide for modeling the structure considered for this study. To run non-linear push over analysis SAP 2000 is utilized. 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. SAP2000 considers the effect of geometric nonlinearity of the structure (i.e. P-Δ effect) simultaneously with non-linear pushover analysis, this also makes the software advantageous.

In order to validate pushover analysis results of SAP2000, verification is made with pushover results of frame elements presented by Baris Binici and Ahmet Yakut [24] in which results are obtained by incremental step by step procedure. Pushover analysis result verification is presented in appendix G.

3.6.1. Computation of lateral load and its vertical distribution

Seismic hazard level of Basic Safety Earthquake 1 (BSE-1) specified in FEMA 356 [8] 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 buildings for pushover analysis is presented in Appendix B.

3.6.2. Modeling sample buildings

In order to get correct results from the push over analysis the sample building should be properly modeled. 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. Even though the software has an established way of analyzing shear walls it has its own limitations. Therefore the shear walls in the dual structural systems under study are represented by equivalent “wide columns”.

According to Smith& Coull [11], shear walls connected by beams to other part of the structure can be modeled by vertical stack of beam element located at the centroidal 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 [2] states that solid wall can be represented with equivalent wide column element located at the centerline 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. Based on a study on comparison of practical approaches for modeling shear walls in structural analyses of buildings, rigid arm with one height story depth give the most consistent results in comparison with shell elements models [25]. Therefore in this study the rigid beam with depth equal to the floor-to-floor height is used. Thickness of the rigid arm is considered as the thickness of the wall itself.

To represent the nonlinear behaviors, concentrated plastic hinges are assigned for beams and columns 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. In SAP2000 three types of hinge properties are available [9]. 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.

The plastic hinge properties described in FEMA-356 or ATC- 40 are implemented in SAP2000. As shown in Figure 3-4, five points labeled A, B, C, D, and E define the force–deformation behavior of a plastic hinge. Type of element, material properties, longitudinal and transverse steel content, and the axial load level on the element are factors varying the values assigned to each of these points.

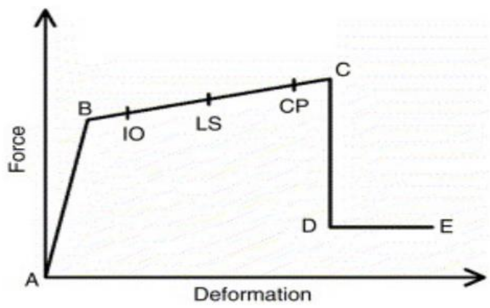


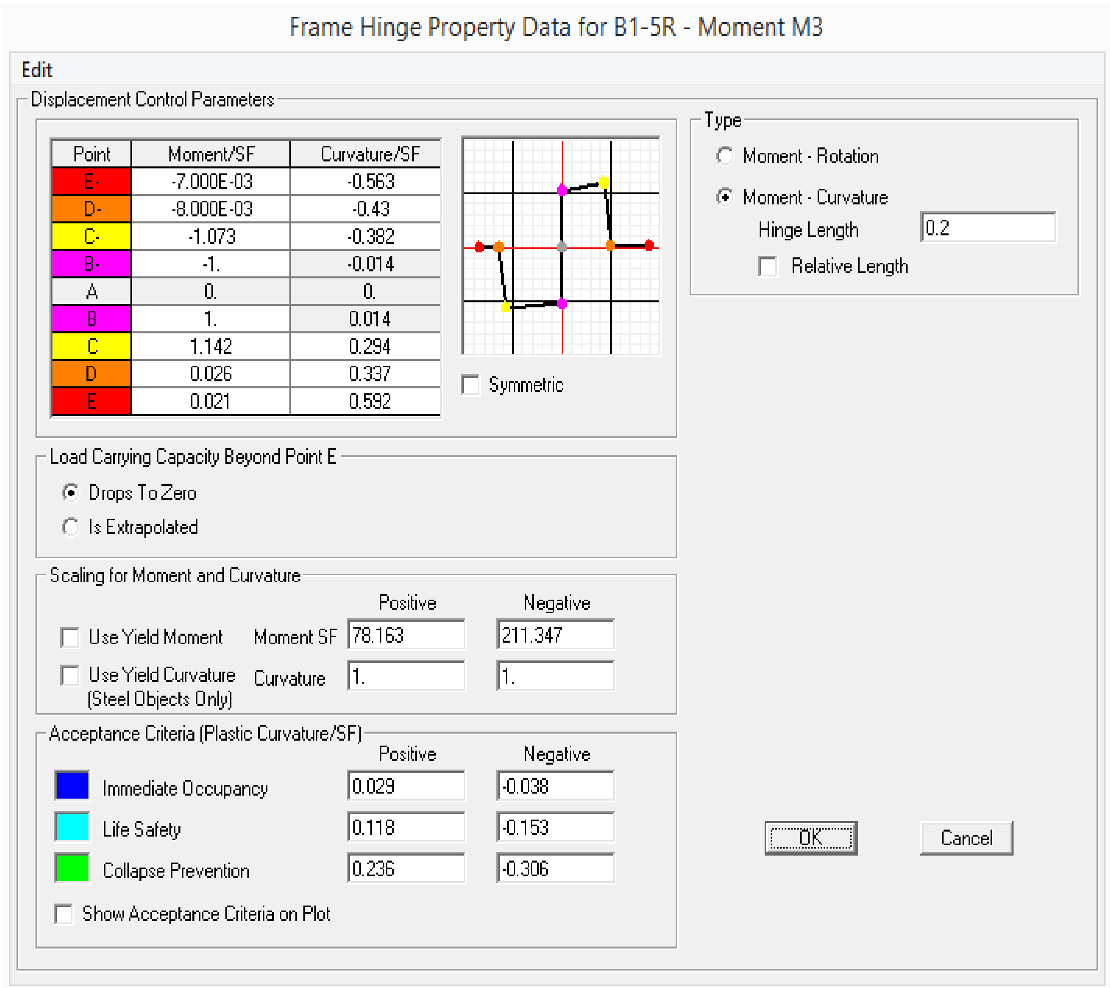
Figure 3-4: Force deformation relation of typical frame element

Moment–curvature analysis of each element is required to define user-defined hinge properties. The points B and C on Figure 3-4 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; 0.5EI and 0.70EI for beams and columns, respectively.

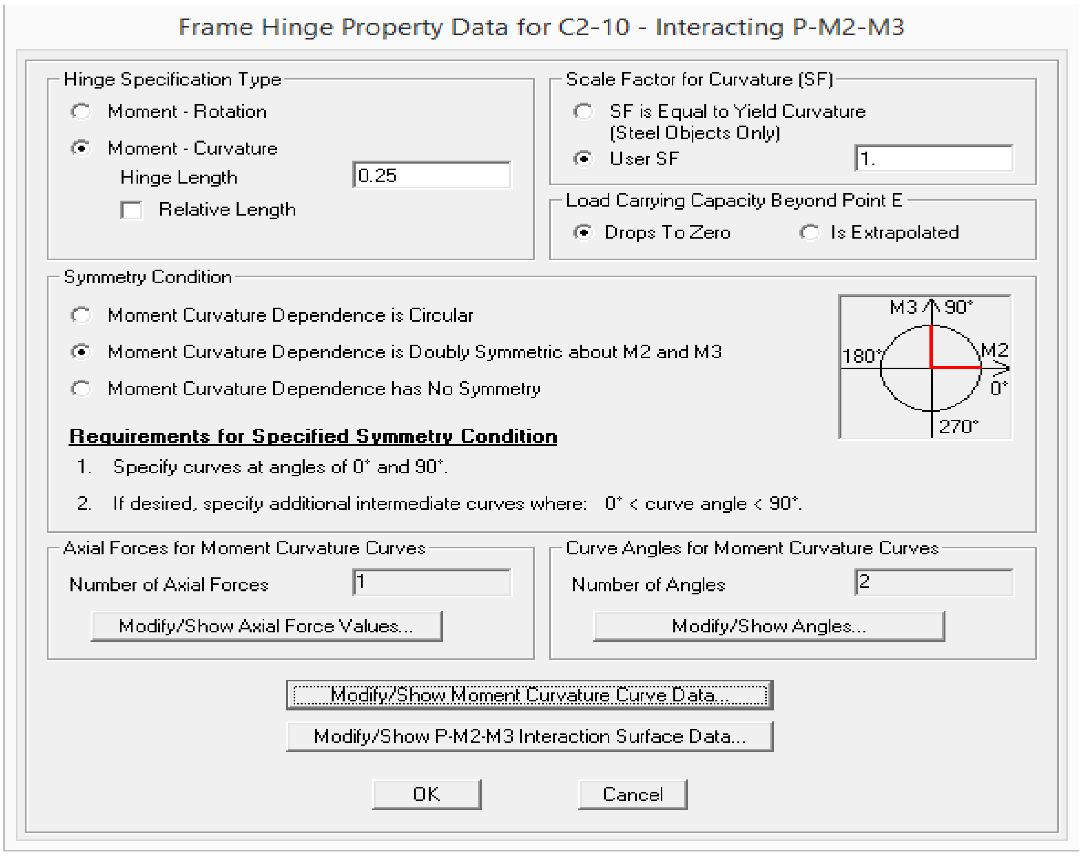
User-defined force-displacement characteristics of plastic hinges were utilized to perform pushover analyses. Moment-curvature relationships of beams, columns and shear walls were calculated from design results of linear analysis to define the user-defined force-displacement characteristics of the members. For this purpose, the axial forces in beams were assumed to be zero. The column axial forces were assumed to be constant during an earthquake and the axial forces due to dead load and 30% live load were used to calculate the moment curvature relationships of columns.

Cross sectional properties of RC members can be calculated manually or using software's, such as Response 2000 and SAP 2000 section designer. Due to its simplicity, SAP 2000 section designer was utilized to determine cross sectional properties of RC members after sample verification is made using response 2000 and manual calculation following the procedures discussed by James k. Wight, and James g. Macgregor [23]. Moment curvature verification is shown on appendix E.

The nonlinearity of members can be considered by defining plastic hinges with hysteretic relationships automatically based on FEMA-356 (FEMA-356, 2000) or on CALTRANS Flexural Hinges (Caltrans, 2009), or manually following one of the solutions: uncoupled M2/M3 or interaction PMM (P-M_2-M_3) hinges. On this study flexural hinges are assigned manually: M3 hinges are assigned on beams, interaction PMM hinges are assigned on columns, and based on direction of push PM hinges are assigned for shear walls. Hinge properties assigned for typical beam, column, and shear wall is shown in appendix F. Typical M3 and typical PMM hinge assignments dialogue boxes of SAP2000 are shown in figure 3.5 a and b.



(a) M3 hinge assignment dialogue box



(b) PMM hinge assignment dialogue box

Figure 3-5: User defined hinge assignment dialogue box (SAP2000)

The input required for SAP2000 is the moment–rotation relationship instead of the moment–curvature relationship. 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 3-4. Plastic hinge length is used to obtain ultimate rotation values from the ultimate curvatures.

$$\theta_p = (\varphi_{ult} - \varphi_y) * l_p$$

Where: l_p: Plastic hinge length

φ_{ult}: Ultimate curvature

θ_p: Plastic rotation

Several plastic hinge lengths have been proposed in different literature but Paulay T. and Priestely M.J.N [4] proposed that plastic hinge length can be approximated as 0.5h where h is the section's depth. Also ATC-40 states that the plastic hinge length, l_p = h/2 where h is the section depth in the direction of loading, is an acceptable value that usually gives conservative results.

For analytical models of shear walls and wall segments FEMA 356 recommends the value of lp to be equal to 0.5 times the flexural depth of the element, but less than one story height for shear walls and less than 50% of the element length for wall segments. For columns supporting discontinuous shear walls, lp shall be set equal to 0.5 times the flexural depth of the component.

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 as suggested by the sources mentioned above. Following the calculation of the ultimate rotation capacity of an element, Acceptance criteria (plastic def./SF), which are labeled as IO (Immediate Occupancy), LS (Life Safety), and CP (Collapse Prevention) in figure 3-4, are defined. In this research, these three points defined as a point corresponding to 10%, 40%, and 80% use of plastic hinge deformation capacity.

In SAP2000 three different member unloading methods are available to remove the load that the hinge was carrying and redistribute it to the rest of the structure (As shown in figure 3-6).

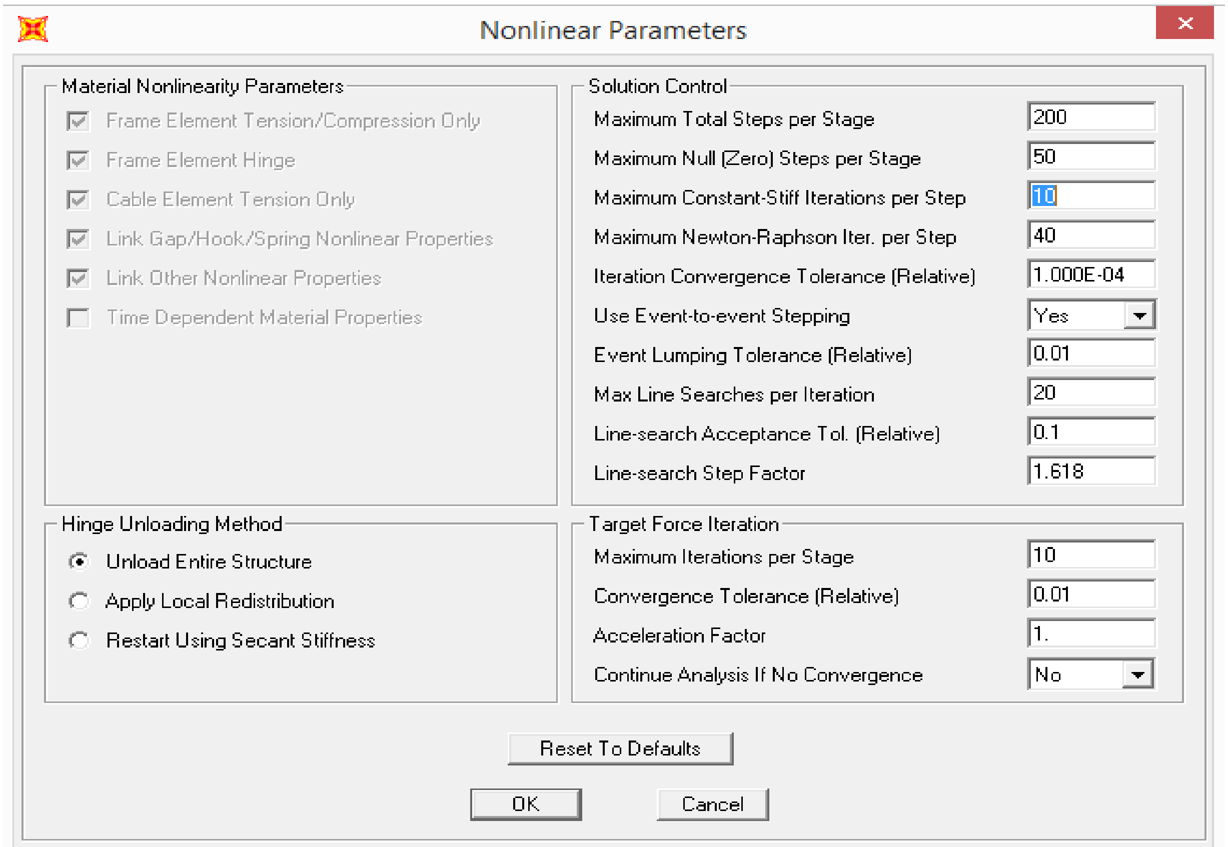


Figure 3-6: Nonlinear parameter dialogue box (SAP2000)

In the "Unload Entire Structure" option, when the hinge reaches point C on its force displacement curve (Figure 3-4) the program continues to try to increase the base shear. The analysis proceeds if the lateral deformation is increased due to the increased base shear. Otherwise, base shear is reduced by reversing the lateral load on the whole structure until the force in that hinge is consistent with the value at point D on its force-displacement curve. Since the base shear is reduced, all elements are unloaded and lateral displacement is lowered. After the hinge is fully unloaded, base shear is again increased, lateral displacement begins to increase and other elements of the structure carry the load that was removed from the unloaded hinge. This method will fail if hinge unloading requires large reductions in the applied lateral load and two hinges compete to unload (one hinge requires the applied load to increase while the other requires the load to decrease).

Instead of unloading the entire structure, the “Apply Local Redistribution” option unloads only the element containing the hinge. If the analysis proceeds by reducing the base shear when a hinge reaches point C, unloading of the hinge is performed by applying a temporary, localized, self-equilibrating, internal load that unloads the element [10]. After the hinge is unloaded, the temporary load is reversed and transfers the removed load to neighboring elements. If two hinges in the same element compete to unload (where one hinge requires the temporary load to increase while the other requires the load to decrease) this method will fail to proceed.

In the case of "Restart Using Secant Stiffness" option, all hinges that become nonlinear are reformed using secant stiffness properties whenever any hinge reaches the yielding point. When the stress in a hinge under gravity load is large causing the secant stiffness to be negative, this method may fail. However, this method gives solutions where the two methods mentioned above fail due to nearly horizontal negatively sloped hinges [10].

From these three methods of unloading, “Unload Entire Structure” option is the most efficient method and uses moderate number of total null steps. However, pushover curves obtained from each method have the same base shear capacity and maximum lateral displacement [17]. In this study, “Unload Entire Structure” option is adopted to obtain the capacity curves.

4. RESULT AND DISCUSSION

4.1. General

In this study six models were considered in two categories for pushover analysis to represent the required dual structural system. In the first category, three building models having 5, 9, and 13 storeys were considered by keeping the percentage share of members against base shear in the same range. In the second category the number of storeys was kept to be 9, whereas the percentage share of frames against lateral load is varying to be 9.82 %, 11.27 %, and 13.12% for building model 1, model 2, and model 3 respectively.

In all models the contribution of shear walls against lateral load in the linear analysis is above 85 %, whereas frames carry less than 15 % of the total base shear. Each model was designed and analyzed for two cases. In the first case, all members are designed as primary seismic members (fully detailed) and in the second case, only shear walls are designed as primary seismic members and frames are considered to be secondary seismic members (partially detailed).

To represent the nonlinearity in the finite element model prepared using SAP2000, user defined concentrated plastic hinges properties obtained from moment-curvature relation are used at both ends of each elements. M3, interacting PMM, and interacting PM hinges are defined for beams, columns, and shear walls respectively. Mid pier model having a rigid arm depth equal to a storey height is used to model shear walls.

Before drawing conclusions from the pushover curves obtained, one should be well aware of the limitations of the software package used (SAP2000). As suggested by Belejo et al. (2012), all the tested SAP2000 modeling approaches were unable to reproduce the softening effect of the curve in the post-peak range, since the analysis stops at the maximum base shear. In addition to this, the influence of nonlinear parameters for solution control on the capacity curves must be checked. Particularly, attention must be given to the definition of the maximum null steps per stage and the iteration convergence tolerance. During the nonlinear procedure when a frame hinge is trying to unload, when an event triggers another event and when iteration does not converge and a smaller step size is attempted, null steps will occur. An excessive number of null steps may indicate that the solution is suddenly stopped due to catastrophic failure or numerical sensitivity [22]. To make sure that the equilibrium is achieved at each step, the iteration convergence tolerance is used.

In pushover analysis, capacity curves (Base shear Vs roof displacement) and plastic hinge mechanisms are parameters used to describe the performance and characteristics of the structure. Analysis results of the non-linear pushover analysis are presented as follows.

4.2. Capacity curve

Capacity curves are obtained for building models of category 1 and category 2 when pushed up to roof displacement of 4% of their respective height. Sensitivity of the capacity curve to the nonlinear solution control parameters is checked.

4.2.1. Capacity curve for category 1

The base shears vs. roof displacement for all models in this category are shown in figure 4-1, figure 4-2, and figure 4-3. In all models the curves show similar features: initially linear, then as the members undergo inelastic actions there is deviation from linearity, and become linear again with small slopes when pushed well into the inelastic range. For the first and the third models, the capacity curve of the partially detailed case is higher. For the second model however, the capacity curves are almost the same. At roof displacement of 4% of the height of the buildings, when frames are considered as secondary seismic members the base shear increase by 12.29 %, 0.273 %, and 5.22 % for model 1, model 2, and model 3 respectively. As a result, in terms of load caring capacity, in the partially detailed case the selected primary members (shear walls) resist lateral load as good as or even better than fully detailed cases. However, as discussed later on the performance and global yielding points and Plastic hinging patterns larger damage levels are observed on the partially detailed cases.

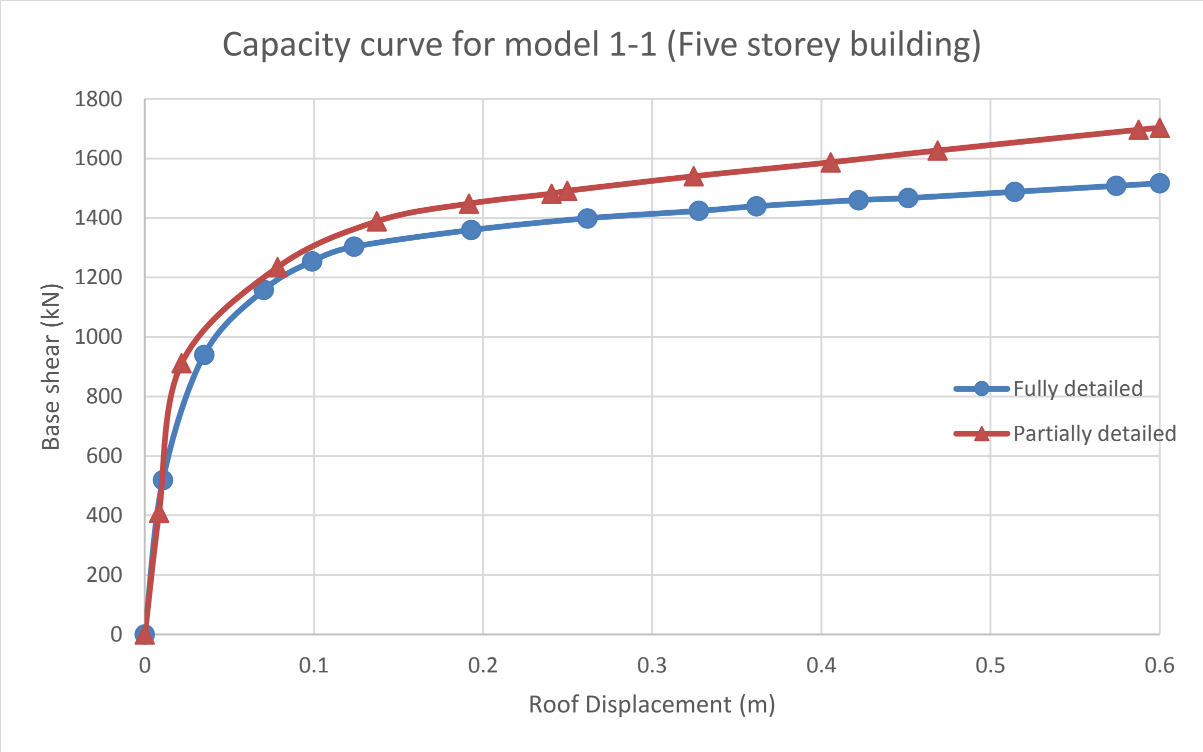


Figure 4-1: Capacity curve for model 1-1 (5 storey building)

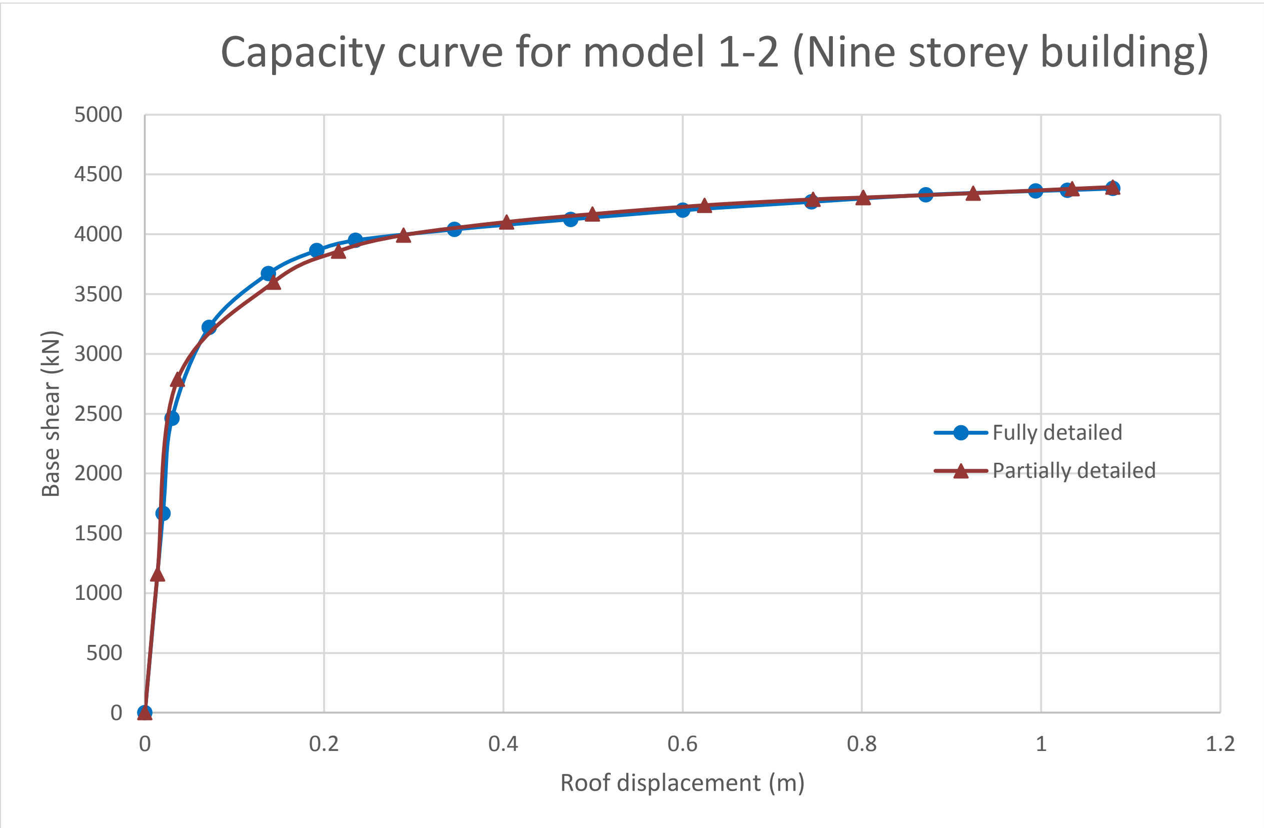


Figure 4-2: Capacity curve for model 1-2 (9 storey building)

For model 3 the capacity curves reach their ultimate base shear and declines slightly. As a result ductility can be compared using the yield and ultimate displacements. For the fully detailed case Δy=0.088 m and Δu=1.360 m as shown on table 4-17, and for the partially detailed case Δy=0.10 m and Δu=1.212 m as shown on table 4-18. The capacity curve of the partially detailed case starts to decline prior to the fully detailed case. More importantly, fully detailed case is 13.20 % more ductile than the partially detailed case.

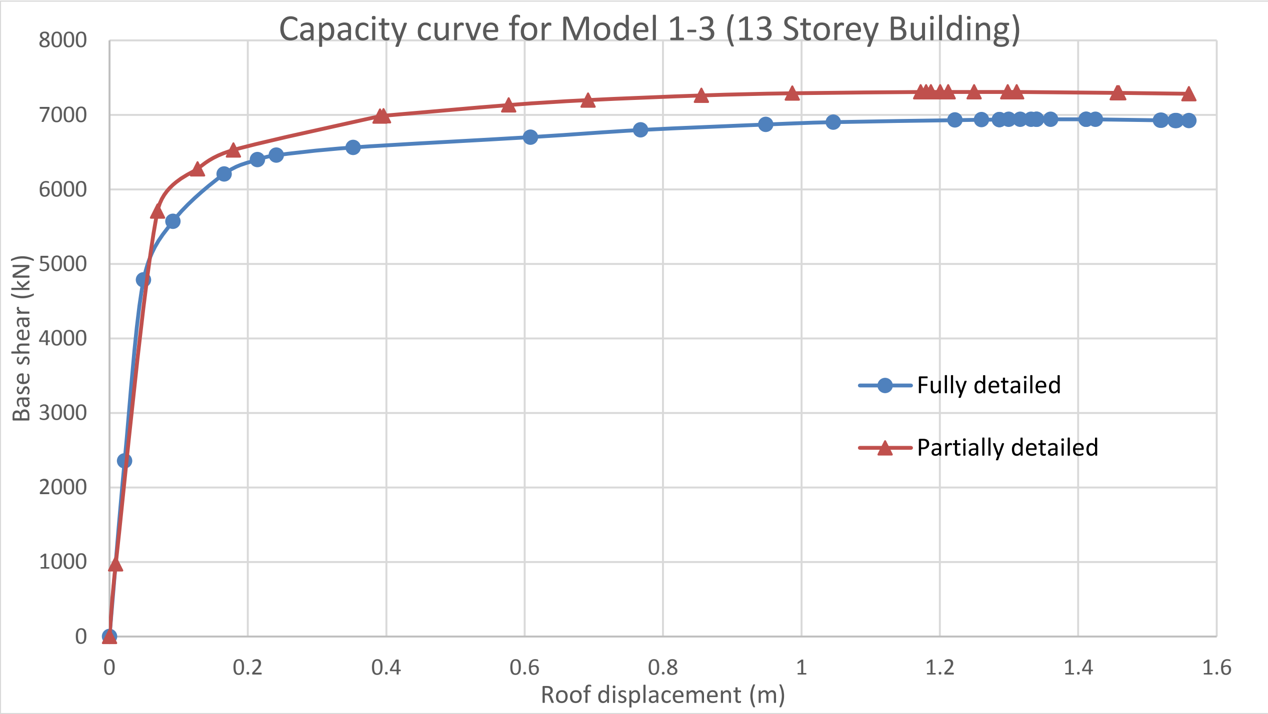


Figure 4-3: Capacity curve for model 1-3 (13 storey building)

4.2.2. Capacity curve for category 2

The base shears vs. roof displacement for all models of this category are shown in figure 4-4, figure 4-5, and figure 4-6. In all three models the curves show similar features: initially linear, then as the members undergo inelastic actions there is deviation from linearity, and become linear again with small slopes when pushed well into the inelastic range. For the first model the capacity curve of the partially detailed case is visibly higher. Whereas for model 2 and model 3 the capacity curves are almost similar especially on the later steps. Comparing the capacity curves of the three models of this category, the difference between the fully detailed and the partially detailed is bigger when the percentage share of the frames (based on linear analysis) against base shear reduces. At roof displacement of 4% of the height of the buildings, the base shear increase by 4.39 %, 0.273 %, and 0.40 % for the partially detailed cases of model 1, model 2, and model 3 respectively.

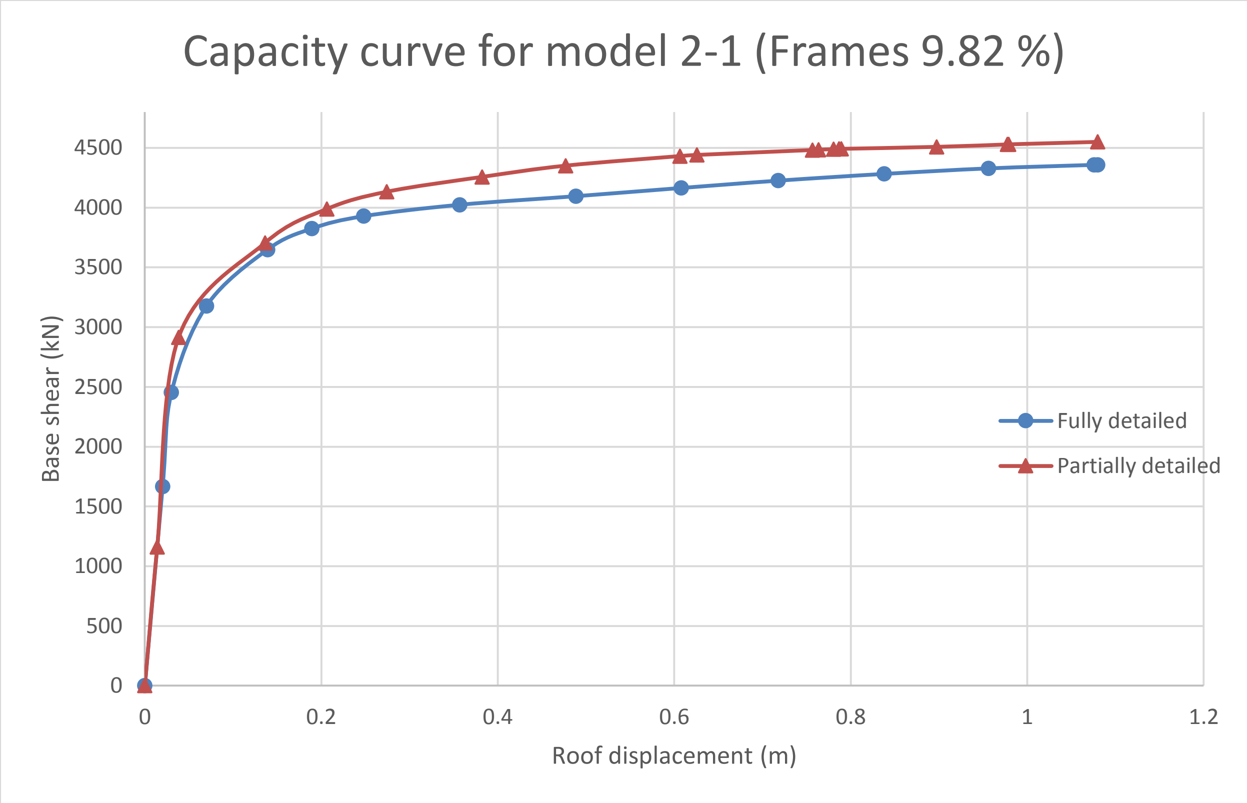


Figure 4-4: Capacity curve for model 2-1 (Frames 9.82 %)

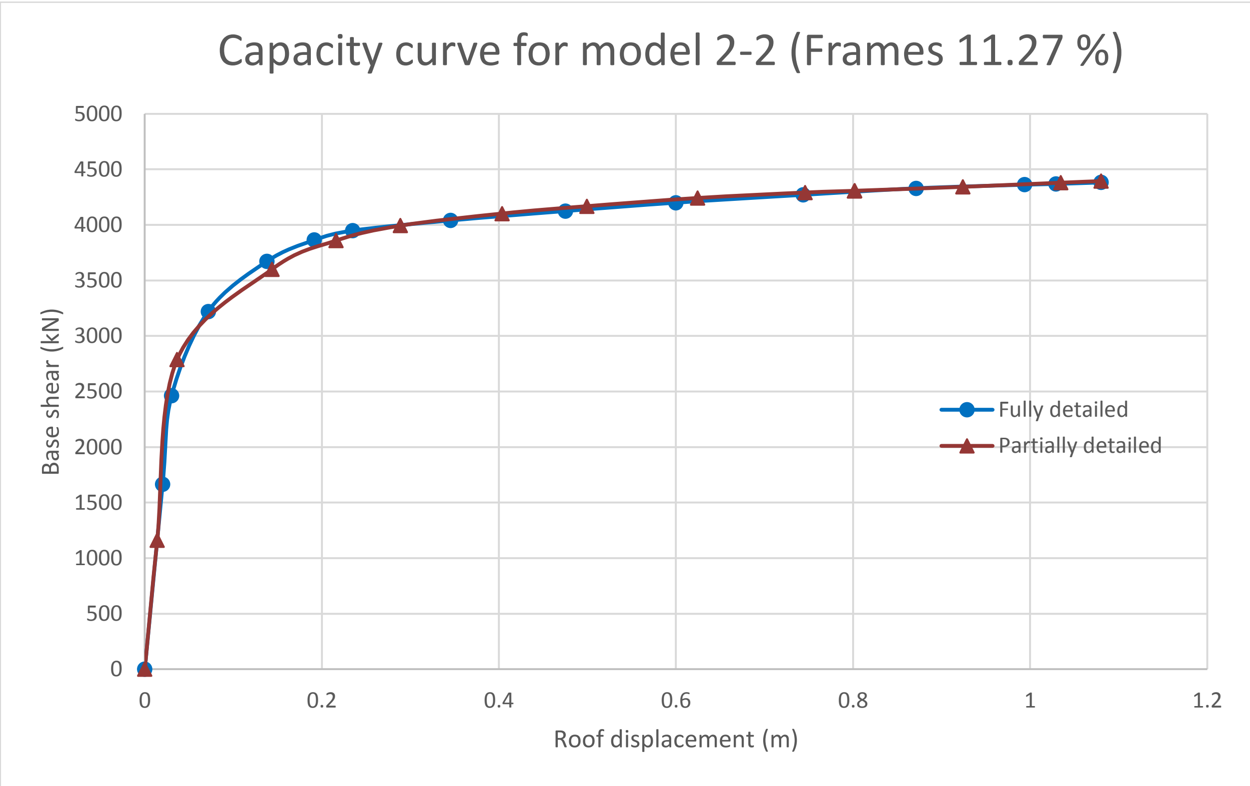


Figure 4-5: Capacity curve for model 2-2 (Frames 11.27 %)

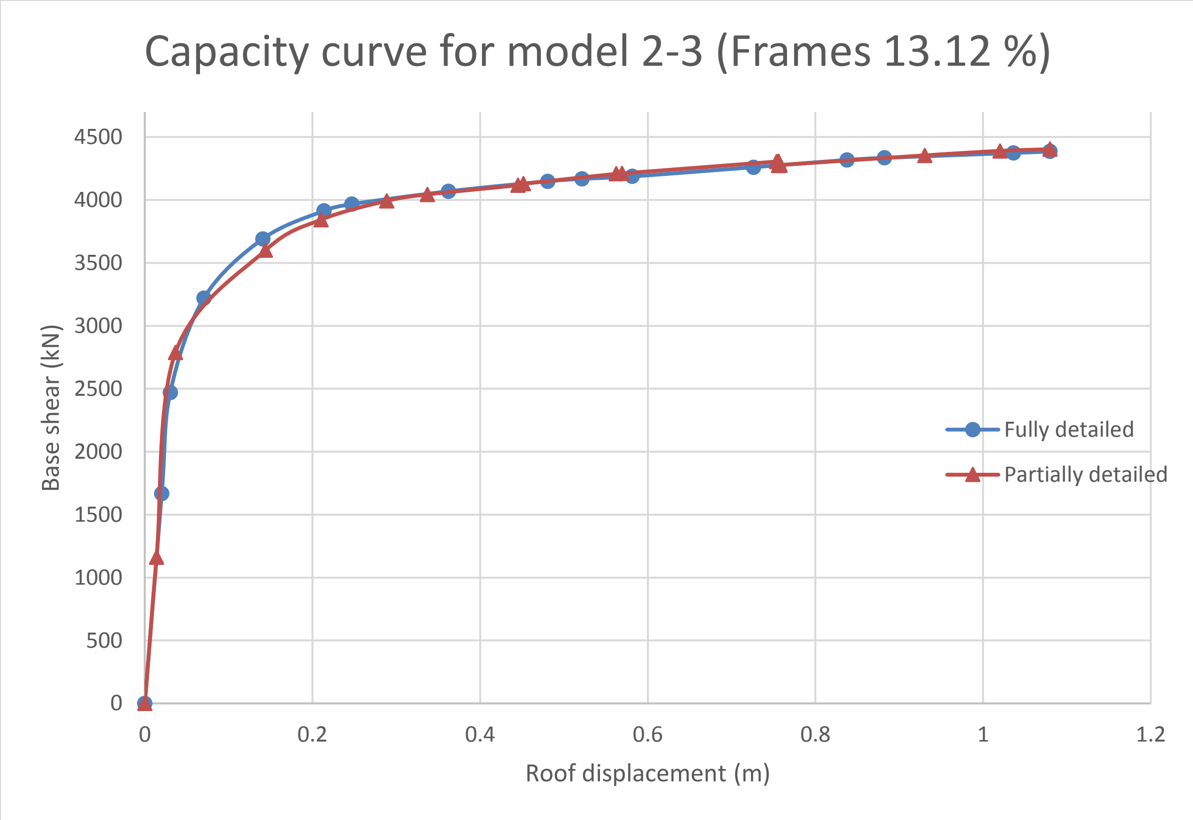


Figure 4-6: Capacity curve for model 2-3 (Frames 13.12 %)

4.3. Performance and global yielding point

In SAP2000 the point at which demand curve intersects the capacity curve can be computed using capacity spectrum method according to ATC-40. This point is known as performance point. Comparison of performance points is made between the two cases of each building. And in order to check the proximity of performance point from the elastic range, the global yielding point of each case is obtained. The global yielding point corresponds to the displacement on the capacity curve where the system starts to soften. As the performance point is closer to the global yielding point the better will be the performance of the building. In most cases of all the models of both category 1 and 2, the performance and the global yielding points are closer in case I showing better performances (i.e. for the fully detailed cases the performance point is near to elastic range).

4.3.1. Performance and global yielding point for category 1

The performance and global yielding points of each model in this category are shown in table 4-1, 4-2 and 4-3. Comparing the fully detailed and partially detailed cases of each models, in model 1 and 2 the performance point is closer to the global yielding point in the fully detailed cases than the partially detailed cases. Proximity of these points is the same for model 3. Therefore, considering frames and walls as primary seismic members improves the seismic resistance performance of RC dual system buildings covered in this study in terms of requiring the performance point to be near to the elastic range.

Table 4-1: Performance and Global yielding point for building model 1-1

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I (Fully detailed)	1031.257	0.050	954.576	0.0388
Case II (Partially detailed)	1055.115	0.047	921.356	0.0217

Table 4-2: Performance and Global yielding point for building model 1-2

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I (Fully detailed)	3102.744	0.065	2535.593	0.0341
Case II (Partially detailed)	3074.597	0.074	2774.237	0.0341

Table 4-3: Performance and Global yielding point for building model 1-3

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I (Fully detailed)	5495.348	0.088	4888.136	0.0579
Case II (Partially detailed)	6005.192	0.100	5694.273	0.0703

4.3.2. Performance and global yielding point for category 2

The performance and global yielding points of each model in this category are shown in table 4-4, 4-5 and 4-6. Similar to building models considered in category 1, the global yielding point is closer to the performance point in the fully detailed models than the partially detailed models. This shows better performance of fully detailed models irrespective of the variation in the percentage share of frames against base shear.

Table 4-4: Performance and Global yielding point for building model 2-1

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I (Fully detailed)	3084.178	0.065	2490.848	0.0341
Case II (Partially detailed)	3198.047	0.073	2931.525	0.037

Table 4-5: Performance and Global yielding point for building model 2-2

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I	3102.744	0.065	2535.593	0.0341
Case II	3074.597	0.074	2774.237	0.0341

Table 4-6: Performance and Global yielding point for building model 2-3

Cases	Performance point		Global yielding point	
	Base shear force (kN)	Displacement (m)	Base shear force (kN)	Displacement (m)
Case I	3112.985	0.065	2520.678	0.0341
Case II	3074.334	0.074	2806.780	0.037

4.4. Base shear distribution

Base shear distribution among frames and shear walls clearly shows that the wall contribution diminishes as the plastic deformation increases. In the elastic range shear wall takes more than 85 % of the base shear, when the building is pushed up to roof displacement of 4% of its height, shear wall contribution drops significantly and contribution of frames increases.

4.4.1. Base shear distribution for category 1

Base shear distributions between frames and shear walls for this category are shown on tables 4-7, 4-8, and 4-9. When frame contribution for the fully detailed and partially detailed case are compared for each model in the inelastic range, the percentage contribution of frame is larger in fully detailed cases. Therefore, detailing frames as primary seismic members improve the response of frames as well as the overall performance of the building against lateral load.

Comparing base shear distribution considering the variation in number of storeys, contribution of frames is larger for building model with smaller number of storeys. At the roof displacement of 4 % of the respective heights of the models, contribution of detailed frames against base shear reaches 56.54 %, 43.03 %, and 41.67 % for 5 storey, 9 storey, and 13 storey building models respectively.

In both cases, the contribution of frames against base shear significantly increases in the initial steps, and then keeps increasing slightly in the later steps. Table 4-7 shows that the contribution frames reach up to 56.54 % for fully detailed case and 43.46 % for the partially detailed case.

Table 4-7: Base shear distribution between frames and shear walls for Model 1-1

Model 1: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	56.28	462.61	518.89	10.85	89.15
2	361.16	578.95	940.11	38.42	61.58
3	409.28	748.71	1157.98	35.34	64.66
4	439.68	814.66	1254.33	35.05	64.95
5	464.37	839.09	1303.46	35.63	64.37
6	526.494	833.10	1359.59	38.72	61.28
7	576.98	821.99	1398.97	41.24	58.76
8	613.19	810.66	1423.85	43.07	56.93
9	648.80	790.86	1439.67	45.07	54.93
10	701.23	759.03	1460.26	48.02	51.98
11	719.04	747.87	1466.91	49.02	50.98
12	771.71	716.48	1488.19	51.86	48.14
13	835.07	673.01	1508.08	55.37	44.63
14	857.54	659.06	1516.60	56.54	43.46
Model 1: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	44.32	364.22	408.54	10.85	89.15
2	113.89	797.92	911.81	12.49	87.51
3	233.152	1001.43	1234.58	18.89	81.11
4	286.89	1101.99	1388.88	20.66	79.34
5	327.34	1121.07	1448.40	22.60	77.40
6	351.90	1129.85	1481.75	23.75	76.25
7	363.87	1127.47	1491.34	24.40	75.60
8	420.90	1119.45	1540.35	27.32	72.68
9	483.78	1103.48	1587.26	30.48	69.52
10	572.08	1054.86	1626.94	35.16	64.84
11	721.98	974.72	1696.70	42.55	57.45
12	740.09	962.94	1703.03	43.46	56.54

Table 4-8 shows base shear distribution between frames and wall at different stages of the pushover analysis. Contributions of frames in the fully detailed case reach up to 43.03 % for fully detailed case and 32.77 % for the partially detailed case.

Table 4-8: Base shear distribution between frames and shear walls for Model 1-2

Model 2: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	188.077	1478.502	1666.58	11.29	88.71
2	279.895	2181.664	2461.56	11.37	88.63
3	1180.036	2041.399	3221.44	36.63	63.37
4	1224.937	2447.459	3672.40	33.36	66.64
5	1275.477	2588.304	3863.78	33.01	66.99
6	1322.884	2625.912	3948.80	33.50	66.50
7	1372.324	2667.935	4040.26	33.97	66.03
8	1466.085	2658.849	4124.93	35.54	64.46
9	1571.6	2628.783	4200.38	37.42	62.58
10	1728.682	2542.249	4270.93	40.48	59.52
11	1837.866	2491.636	4329.50	42.45	57.55
12	1866.971	2494.841	4361.81	42.80	57.20
13	1873.188	2495.487	4368.68	42.88	57.12
14	1885.665	2496.845	4382.51	43.03	56.97
Model 2: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	131.051	1030.21	1161.26	11.29	88.71
2	244.291	2542.534	2786.83	8.77	91.23
3	660.432	2939.71	3600.14	18.34	81.66
4	801.928	3057.05	3858.98	20.78	79.22
5	890.766	3102.70	3993.47	22.31	77.69
6	992.912	3109.63	4102.54	24.20	75.80
7	1056.525	3112.06	4168.59	25.34	74.66
8	1146.065	3096.33	4242.40	27.01	72.99
9	1233.312	3058.25	4291.57	28.74	71.26
10	1262.918	3043.96	4306.88	29.32	70.68
11	1263.363	3043.67	4307.03	29.33	70.67
12	1332.7	3010.03	4342.73	30.69	69.31
13	1405.173	2974.93	4380.11	32.08	67.92
14	1440.085	2954.38	4394.47	32.77	67.23

For the building model having 13 storeys, contribution of frames against base shear reaches 41.67 % and 23.75 % in the fully detailed and partially detailed cases respectively. Therefore, irrespective of variation in number of storeys, detailed frames can take increased base shear than what is estimated from linear analysis.

Table 4-9: Base shear distribution between frames and shear walls for Model 1-3

Model 3: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	263.482	2092.474	2355.96	11.18	88.82
2	546.7	4238.73	4785.43	11.42	88.58
3	1880.125	3691.45	5571.58	33.74	66.26
4	2226.977	3979.083	6206.06	35.88	64.12
5	2210.465	4187.509	6397.97	34.55	65.45
6	2155.245	4302.747	6457.99	33.37	66.63
7	2080.952	4481.402	6562.35	31.71	68.29
8	2486.908	4213.888	6700.80	37.11	62.89
9	2740.728	4055.909	6796.64	40.32	59.68
10	2810.017	4060.12	6870.14	40.90	59.10
11	2837.282	4064.774	6902.06	41.11	58.89
12	2877.972	4050.56	6928.53	41.54	58.46
13	2885.187	4048.85	6934.04	41.61	58.39
14	2886.78	4049.441	6936.22	41.62	58.38
15	2886.782	4049.442	6936.22	41.62	58.38
16	2888.813	4048.762	6937.58	41.64	58.36
17	2891.441	4047.218	6938.66	41.67	58.33
18	2891.89	4047.185	6939.08	41.68	58.32
19	2892.529	4046.921	6939.45	41.68	58.32
20	2892.551	4046.917	6939.47	41.68	58.32
21	2893.928	4046.365	6940.29	41.70	58.30
22	2893.779	4046.366	6940.15	41.70	58.30
23	2893.787	4046.361	6940.15	41.70	58.30
24	2895.262	4044.076	6939.34	41.72	58.28
25	2886.123	4040.392	6926.52	41.67	58.33
26	2886.2	4040.181	6926.38	41.67	58.33
27	2885.663	4038.264	6923.93	41.68	58.32
28	2885.64	4038.163	6923.80	41.68	58.32
29	2884.066	4037.531	6921.60	41.67	58.33

Model 3: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	108.613	862.562	971.18	11.18	88.82
2	588.708	5120.522	5709.23	10.31	89.69
3	1406.45	4869.321	6275.77	22.41	77.59
4	1504.931	5023.456	6528.39	23.05	76.95
5	1233.089	5345.515	6578.60	18.74	81.26
6	1354.72	5488.651	6843.37	19.80	80.20
7	1486.127	5500.58	6986.71	21.27	78.73
8	1552.075	5501.3	7053.38	22.00	78.00
9	1656.562	5459.151	7115.71	23.28	76.72
10	1690.519	5454.669	7145.19	23.66	76.34
11	1709.658	5455.106	7164.76	23.86	76.14
12	1710.548	5454.52	7165.07	23.87	76.13
13	1711.195	5453.964	7165.16	23.88	76.12
14	1711.272	5453.926	7165.20	23.88	76.12
15	1711.319	5453.891	7165.21	23.88	76.12
16	1711.715	5453.478	7165.19	23.89	76.11
17	1712.191	5452.921	7165.11	23.90	76.10
18	1714.136	5451.037	7165.17	23.92	76.08
19	1707.905	5445.073	7152.98	23.88	76.12
20	1707.672	5445.232	7152.90	23.87	76.13
21	1696.149	5445.124	7141.27	23.75	76.25

4.4.2. Base shear distribution for category 2

Table 4-10, table 4-11, and table 4-12 shows base shear distribution of building models mentioned in this category. In all models the percentage share of frames against base shear is larger in the fully detailed cases than the partially detailed cases. From these results it can be inferred that as the percentage share of frames against base shear in the elastic range increases (from results of linear analysis), the more the performance of the building will be affected when frames are considered as secondary seismic members.

The contributions of frames against base shear in the linear analysis for the first model building of this category were 9.82%. Table 4-10 shows the percentage contribution of frames is larger in the fully detailed case (41.54 %) than the partially detailed case (34.29 %).

Table 4-10: Base shear distribution between frames and shear walls for Model 2-1

Model 1: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	163.826	1502.086	1665.91	9.83	90.17
2	243.061	2210.115	2453.18	9.91	90.09
3	1110.501	2066.413	3176.91	34.96	65.04
4	1155.246	2493.643	3648.89	31.66	68.34
5	1183.648	2641.293	3824.94	30.95	69.05
6	1227.569	2701.099	3928.67	31.25	68.75
7	1303.554	2720.042	4023.60	32.40	67.60
8	1378.103	2717.374	4095.48	33.65	66.35
9	1468.968	2695.235	4164.20	35.28	64.72
10	1564.533	2660.901	4225.43	37.03	62.97
11	1683.138	2598.658	4281.80	39.31	60.69
12	1782.059	2546.013	4328.07	41.17	58.83
13	1809.607	2547.812	4357.42	41.53	58.47
14	1810.511	2547.898	4358.41	41.54	58.46
Model 1: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	114.127	1046.414	1160.54	9.83	90.17
2	239.511	2675.017	2914.53	8.22	91.78
3	816.649	2888.371	3705.02	22.04	77.96
4	906.914	3081.797	3988.71	22.74	77.26
5	1007.135	3125.884	4133.02	24.37	75.63
6	1152.686	3104.245	4256.93	27.08	72.92
7	1298.252	3052.604	4350.86	29.84	70.16
8	1417.009	3011.516	4428.53	32.00	68.00
9	1431.441	3006.743	4438.18	32.25	67.75
10	1487.763	2992.855	4480.62	33.20	66.80
11	1495.257	2988.099	4483.36	33.35	66.65
12	1497.768	2989.023	4486.79	33.38	66.62
13	1499.339	2989.161	4488.50	33.40	66.60
14	1500.257	2989.371	4489.63	33.42	66.58
15	1519.382	2988.869	4508.25	33.70	66.30
16	1539.073	2989.429	4528.50	33.99	66.01
17	1539.705	2989.508	4529.21	33.99	66.01
18	1559.893	2989.758	4549.65	34.29	65.71

For building model 2 contribution of frames against base shear increase up to 43.03 % in the fully detailed case, and 32.77 % for the partially detailed case.

Table 4-11: Base shear distribution between frames and shear walls for Model 2-2

Model 2: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	188.077	1478.502	1666.58	11.29	88.71
2	279.895	2181.664	2461.56	11.37	88.63
3	1180.036	2041.399	3221.44	36.63	63.37
4	1224.937	2447.459	3672.40	33.36	66.64
5	1275.477	2588.304	3863.78	33.01	66.99
6	1322.884	2625.912	3948.80	33.50	66.50
7	1372.324	2667.935	4040.26	33.97	66.03
8	1466.085	2658.849	4124.93	35.54	64.46
9	1571.6	2628.783	4200.38	37.42	62.58
10	1728.682	2542.249	4270.93	40.48	59.52
11	1837.866	2491.636	4329.50	42.45	57.55
12	1866.971	2494.841	4361.81	42.80	57.20
13	1873.188	2495.487	4368.68	42.88	57.12
14	1885.665	2496.845	4382.51	43.03	56.97
Model 2: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	131.051	1030.21	1161.26	11.29	88.71
2	244.291	2542.534	2786.83	8.77	91.23
3	660.432	2939.71	3600.14	18.34	81.66
4	801.928	3057.05	3858.98	20.78	79.22
5	890.766	3102.70	3993.47	22.31	77.69
6	992.912	3109.63	4102.54	24.20	75.80
7	1056.525	3112.06	4168.59	25.34	74.66
8	1146.065	3096.33	4242.40	27.01	72.99
9	1233.312	3058.25	4291.57	28.74	71.26
10	1262.918	3043.96	4306.88	29.32	70.68
11	1263.363	3043.67	4307.03	29.33	70.67
12	1332.7	3010.03	4342.73	30.69	69.31
13	1405.173	2974.93	4380.11	32.08	67.92
14	1440.085	2954.38	4394.47	32.77	67.23

For model 3 frames share 13.12% of the total base shear in the linear analysis. However, in the nonlinear range frames take 42.87 % of the total base shear in the fully detailed case and 34.84 % in the partially detailed case.

Table 4-12: Base shear distribution between frames and shear walls for Model 2-3

Model 3: Base shear distribution for Fully detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	219.139	1448.324	1667.46	13.14	86.86
2	327.457	2144.44	2471.90	13.25	86.75
3	1197.014	2022.685	3219.70	37.18	62.82
4	1227.964	2463.81	3691.77	33.26	66.74
5	1288.344	2625.534	3913.88	32.92	67.08
6	1316.728	2650.549	3967.28	33.19	66.81
7	1398.113	2671.168	4069.28	34.36	65.64
8	1492.708	2656.25	4148.96	35.98	64.02
9	1516.782	2651.803	4168.59	36.39	63.61
10	1534.752	2652.675	4187.43	36.65	63.35
11	1682.314	2578.084	4260.40	39.49	60.51
12	1807.062	2511.506	4318.57	41.84	58.16
13	1831.483	2503.627	4335.11	42.25	57.75
14	1868.361	2505.168	4373.53	42.72	57.28
15	1880.405	2505.468	4385.87	42.87	57.13
Model 3: Base shear distribution for Partially detailed case					
Step	Frame Reaction (kN)	Wall Reaction (kN)	Total Base shear (kN)	Frame Reaction (%)	Wall Reaction (%)
1	152.719	1009.348	1162.07	13.14	86.86
2	282.19	2505.597	2787.79	10.12	89.88
3	685.708	2915.074	3600.78	19.04	80.96
4	824.344	3019.62	3843.96	21.45	78.55
5	922.3	3070.836	3993.14	23.10	76.90
6	962.223	3082.233	4044.46	23.79	76.21
7	1009.208	3108.358	4117.57	24.51	75.49
8	1024.956	3104.113	4129.07	24.82	75.18
9	1125.97	3083.519	4209.49	26.75	73.25
10	1133.425	3078.821	4212.25	26.91	73.09
11	1304.987	3005.427	4310.41	30.28	69.72
12	1308.302	3002.727	4311.03	30.35	69.65
13	1301.059	2974.299	4275.36	30.43	69.57
14	1303.089	2974.268	4277.36	30.46	69.54
15	1305.26	2974.206	4279.47	30.50	69.50
16	1449.075	2906.107	4355.18	33.27	66.73
17	1520.109	2869.286	4389.40	34.63	65.37
18	1533.932	2868.704	4402.64	34.84	65.16

Comparing base shear distribution between frames and walls for building models in this category, considering frames as primary seismic members increases the percentage share of frames and overall performance of buildings.

4.5. Plastic hinge mechanism

Frames and shear walls will experience yielding at the regions where they are subjected to large moments when the building is pushed laterally or subjected to earthquake induced displacements. In those regions which are subjected to larger moment, Plastic hinges are formed due to large inelastic curvatures. For all the buildings, plastic hinge formation mechanisms and hinging patterns have been obtained at different displacement levels or pushover steps.

Comparing hinging patterns for the two cases for all sample buildings, in the fully detailed cases plastic hinge formation start with beam ends, then forms on upper stories of columns. And when the building is pushed further, hinges are formed on columns and shear walls at lower stories. In the partially detailed cases however, some of the columns in the upper stories experience larger damage levels prior to beams since the frames in this case are not checked to satisfy capacity design rules. Plastic hinging patterns for typical frames is presented on appendix H to show the variation of hinging patterns for the fully detailed and partially detailed case.

4.5.1. Plastic hinge mechanism for category 1

Comparing number of plastic hinges of different damage levels for all the three models considered in this category, larger numbers of hinges are at higher damage levels in the partially detailed cases.

Tables 4-13 and 4-14 show the hinge state details at each step of the analysis for fully detailed case and partially detailed cases of building model 1 respectively. It can be seen that for the Performance Point for the fully detailed case (shown on section 4.3), taken as step 3 (which actually lies between steps 2 and 3), 99.375% of hinges are within IO performance level. However, at the performance point of the partially detailed case (taken as step 3) of this model, 97.50 % of hinges are within IO performance level.

Table 4-13: Summary on number of plastic hinges for model 1-1: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	320	0	0	0	0	0	0	0	320
1	0.01063	518.89	318	2	0	0	0	0	0	0	320
2	0.035025	940.111	286	34	0	0	0	0	0	0	320
3	0.070361	1157.982	241	77	2	0	0	0	0	0	320
4	0.098911	1254.336	224	80	16	0	0	0	0	0	320
5	0.123673	1303.455	205	75	40	0	0	0	0	0	320
6	0.192875	1359.588	184	63	73	0	0	0	0	0	320
7	0.261597	1398.972	159	66	95	0	0	0	0	0	320
8	0.32747	1423.853	145	66	99	10	0	0	0	0	320
9	0.361527	1439.668	139	68	85	28	0	0	0	0	320
10	0.422064	1460.26	128	75	82	35	0	0	0	0	320
11	0.451389	1466.905	128	69	79	44	0	0	0	0	320
12	0.514402	1488.19	126	65	73	56	0	0	0	0	320
13	0.574402	1508.081	126	53	75	58	8	0	0	0	320
14	0.6	1516.601	126	52	69	63	10	0	0	0	320

Table 4-14: Summary on number of plastic hinges for model 1-1: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	320	0	0	0	0	0	0	0	320
1	0.008369	408.539	316	4	0	0	0	0	0	0	320
2	0.021751	911.811	264	56	0	0	0	0	0	0	320
3	0.078535	1234.582	227	85	8	0	0	0	0	0	320
4	0.137208	1388.881	205	80	35	0	0	0	0	0	320
5	0.191696	1448.402	187	58	75	0	0	0	0	0	320
6	0.240532	1481.747	185	45	90	0	0	0	0	0	320
7	0.249657	1491.34	183	47	90	0	0	0	0	0	320
8	0.324522	1540.35	175	33	104	8	0	0	0	0	320
9	0.405536	1587.257	153	55	83	29	0	0	0	0	320
10	0.468818	1626.935	147	61	79	33	0	0	0	0	320
11	0.58756	1696.702	123	71	63	59	4	0	0	0	320
12	0.6	1703.027	123	69	63	59	6	0	0	0	320

Hinge state details at each step of the analysis for fully detailed and partially detailed cases of building model 2 are shown on tables 4-15 and 4-16 respectively. Comparing number of hinges at different damage levels at roof displacement of 1.08 m (4% of the height of the building models), 98.19 % and 96.14 % of the hinges are within collapse © performance level for the fully detailed and the partially detailed case respectively.

Table 4-15: Summary on number of plastic hinges for model 1-2: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.020107	1666.577	879	3	0	0	0	0	0	0	882
2	0.0302	2461.559	830	52	0	0	0	0	0	0	882
3	0.071587	3221.436	769	113	0	0	0	0	0	0	882
4	0.137921	3672.395	655	209	18	0	0	0	0	0	882
5	0.191554	3863.78	596	183	103	0	0	0	0	0	882
6	0.234914	3948.797	556	200	126	0	0	0	0	0	882
7	0.345236	4040.26	507	161	214	0	0	0	0	0	882
8	0.475047	4124.935	480	118	266	18	0	0	0	0	882
9	0.599996	4200.383	455	89	288	50	0	0	0	0	882
10	0.743589	4270.931	442	79	239	122	0	0	0	0	882
11	0.871321	4329.504	427	83	199	157	16	0	0	0	882
12	0.993816	4361.811	420	87	158	193	8	16	0	0	882
13	1.029193	4368.674	419	82	159	190	16	16	0	0	882
14	1.08	4382.508	416	74	164	192	20	16	0	0	882

Table 4-16: Summary on number of plastic hinges for model 1-2: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.01401	1161.26	873	9	0	0	0	0	0	0	882
2	0.036418	2786.823	756	126	0	0	0	0	0	0	882
3	0.143579	3600.143	619	228	35	0	0	0	0	0	882
4	0.216098	3858.974	537	264	81	0	0	0	0	0	882
5	0.288696	3993.341	490	259	133	0	0	0	0	0	882
6	0.403486	4102.418	448	209	209	16	0	0	0	0	882
7	0.499392	4168.587	413	188	261	20	0	0	0	0	882
8	0.624572	4242.4	380	177	268	57	0	0	0	0	882
9	0.745702	4291.567	358	171	279	60	14	0	0	0	882
10	0.801327	4306.882	352	169	279	68	14	0	0	0	882
11	0.801749	4307.202	352	169	279	68	14	0	0	0	882
12	0.924024	4342.732	343	143	278	86	18	14	0	0	882
13	1.034587	4380.113	338	129	257	122	18	18	0	0	882
14	1.08	4394.474	336	129	247	119	17	34	0	0	882

Table 4-17: Summary on number of plastic hinges for model 1-3: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	1274	0	0	0	0	0	0	0	1274
1	0.021904	2355.959	1272	2	0	0	0	0	0	0	1274
2	0.049293	4785.429	1113	161	0	0	0	0	0	0	1274
3	0.091703	5571.574	981	293	0	0	0	0	0	0	1274
4	0.165467	6206.061	852	374	48	0	0	0	0	0	1274
5	0.213715	6397.975	761	402	111	0	0	0	0	0	1274
6	0.241177	6457.991	719	358	197	0	0	0	0	0	1274
7	0.351924	6562.354	682	332	260	0	0	0	0	0	1274
8	0.608302	6700.794	649	135	436	54	0	0	0	0	1274
9	0.767682	6796.638	637	136	428	72	0	0	0	1	1274
10	0.948294	6870.138	610	109	335	195	24	0	0	1	1274
11	1.046175	6902.057	600	109	297	243	2	22	0	1	1274
12	1.221984	6928.53	581	121	214	285	48	24	0	1	1274
13	1.260261	6934.038	580	121	196	304	44	28	0	1	1274
14	1.286205	6936.219	579	116	184	322	44	28	0	1	1274
15	1.286252	6936.227	579	116	183	323	44	28	0	1	1274
16	1.299582	6937.577	579	116	177	329	44	28	0	1	1274
17	1.316329	6938.658	578	117	169	337	30	42	0	1	1274
18	1.331925	6939.075	578	117	168	338	24	48	0	1	1274
19	1.339639	6939.452	578	117	166	340	22	50	0	1	1274
20	1.33981	6939.467	578	117	166	340	22	50	0	1	1274
21	1.360123	6940.292	577	118	164	342	20	52	0	1	1274
22	1.411291	6940.142	577	117	147	360	8	64	0	1	1274
23	1.411502	6940.149	576	118	147	360	8	64	0	1	1274
24	1.424842	6939.339	576	118	145	362	0	72	0	1	1274
25	1.518604	6926.515	572	121	144	362	2	72	0	1	1274
26	1.520321	6926.381	572	121	144	362	2	72	0	1	1274
27	1.540145	6923.927	571	122	144	362	2	72	0	1	1274
28	1.541912	6923.801	571	122	144	362	2	72	0	1	1274
29	1.56	6921.596	570	123	139	367	2	72	0	1	1274

Table 4-18: Summary on number of plastic hinges for model 1-3: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	1274	0	0	0	0	0	0	0	1274
1	0.009029	971.177	1268	6	0	0	0	0	0	0	1274
2	0.069452	5709.23	957	317	0	0	0	0	0	0	1274
3	0.127368	6275.771	783	465	26	0	0	0	0	0	1274
4	0.179102	6528.388	718	475	81	0	0	0	0	0	1274
5	0.391088	6983.074	591	353	318	12	0	0	0	0	1274
6	0.395963	6987.99	586	354	320	14	0	0	0	0	1274
7	0.57686	7131.323	542	314	358	60	0	0	0	0	1274
8	0.691554	7197.994	531	284	389	70	0	0	0	0	1274
9	0.855495	7260.33	511	269	368	106	20	0	0	0	1274
10	0.986584	7289.808	505	239	298	188	24	20	0	0	1274
11	1.17213	7306.455	486	229	272	223	18	46	0	0	1274
12	1.180236	7306.759	486	225	276	223	18	46	0	0	1274
13	1.18729	7306.852	486	219	282	221	20	46	0	0	1274
14	1.200621	7306.889	483	216	288	221	20	46	0	0	1274
15	1.212005	7306.903	482	215	290	221	20	46	0	0	1274
16	1.249294	7306.884	477	203	292	234	22	46	0	0	1274
17	1.297891	7306.803	476	196	290	242	24	46	0	0	1274
18	1.3112	7306.866	476	194	291	243	12	58	0	0	1274
19	1.456347	7294.667	458	199	270	275	4	68	0	0	1274
20	1.458474	7294.595	457	200	269	276	4	68	0	0	1274
21	1.56	7282.964	434	220	244	287	17	72	0	0	1274

For building model 3 Summary on number of plastic hinges are shown on table 4-17 and 4-18. At roof displacement of 4% of the height of the building, 94.11 % of hinges are within CP performance level. For the partially detailed case however, 93.01 % of hinges are within CP performance level.

4.5.2. Plastic hinge mechanism for category 2

Similar to category 1, in all the three models considered in category 2 larger number of hinges are at higher damage levels in the partially detailed cases.

Table 4-19: Summary on number of plastic hinges for model 2-1: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.020118	1665.912	879	3	0	0	0	0	0	0	882
2	0.030119	2453.178	830	52	0	0	0	0	0	0	882
3	0.069734	3176.915	780	102	0	0	0	0	0	0	882
4	0.138997	3648.891	654	210	18	0	0	0	0	0	882
5	0.189359	3824.942	601	180	101	0	0	0	0	0	882
6	0.248103	3928.667	549	205	128	0	0	0	0	0	882
7	0.356693	4023.598	498	159	225	0	0	0	0	0	882
8	0.48864	4095.478	479	117	266	20	0	0	0	0	882
9	0.607791	4164.201	453	83	296	50	0	0	0	0	882
10	0.717759	4225.433	445	76	245	116	0	0	0	0	882
11	0.837736	4281.796	435	77	227	127	16	0	0	0	882
12	0.956117	4328.072	419	90	159	194	4	16	0	0	882
13	1.07583	4357.418	412	93	147	194	20	16	0	0	882
14	1.08	4358.41	412	93	145	196	20	16	0	0	882

Table 4-20: Summary on number of plastic hinges for model 2-1: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.014015	1160.541	873	9	0	0	0	0	0	0	882
2	0.038255	2914.529	750	132	0	0	0	0	0	0	882
3	0.136229	3705.019	621	241	20	0	0	0	0	0	882
4	0.206329	3988.711	543	267	72	0	0	0	0	0	882
5	0.274242	4133.021	501	262	119	0	0	0	0	0	882
6	0.382242	4256.929	454	234	180	14	0	0	0	0	882
7	0.476771	4350.856	419	195	248	20	0	0	0	0	882
8	0.606396	4431.05	382	180	267	53	0	0	0	0	882
9	0.62551	4440.629	374	182	270	56	0	0	0	0	882
10	0.756734	4481.962	356	169	277	66	14	0	0	0	882
11	0.763484	4484.7	352	173	276	67	14	0	0	0	882
12	0.780525	4488.86	350	168	279	69	16	0	0	0	882
13	0.786624	4490.569	349	167	281	69	16	0	0	0	882
14	0.789379	4491.695	349	165	283	69	16	0	0	0	882
15	0.897379	4508.25	342	146	280	82	18	14	0	0	882
16	0.977617	4528.503	341	133	266	110	16	16	0	0	882
17	0.978986	4529.212	341	132	267	110	16	16	0	0	882
18	1.08	4549.651	335	129	254	116	16	30	1	1	882

Table 4-21: Summary on number of plastic hinges for model 2-2: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.020107	1666.577	879	3	0	0	0	0	0	0	882
2	0.0302	2461.559	830	52	0	0	0	0	0	0	882
3	0.071587	3221.436	769	113	0	0	0	0	0	0	882
4	0.137921	3672.395	655	209	18	0	0	0	0	0	882
5	0.191554	3863.78	596	183	103	0	0	0	0	0	882
6	0.234914	3948.797	556	200	126	0	0	0	0	0	882
7	0.345236	4040.26	507	161	214	0	0	0	0	0	882
8	0.475047	4124.935	480	118	266	18	0	0	0	0	882
9	0.599996	4200.383	455	89	288	50	0	0	0	0	882
10	0.743589	4270.931	442	79	239	122	0	0	0	0	882
11	0.871321	4329.504	427	83	199	157	16	0	0	0	882
12	0.993816	4361.811	420	87	158	193	8	16	0	0	882
13	1.029193	4368.674	419	82	159	190	16	16	0	0	882
14	1.08	4382.508	416	74	164	192	20	16	0	0	882

Table 4-22: Summary on number of plastic hinges for model 2-2: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.01401	1161.26	873	9	0	0	0	0	0	0	882
2	0.036418	2786.823	756	126	0	0	0	0	0	0	882
3	0.143579	3600.143	619	228	35	0	0	0	0	0	882
4	0.216098	3858.974	537	264	81	0	0	0	0	0	882
5	0.288696	3993.341	490	259	133	0	0	0	0	0	882
6	0.403486	4102.418	448	209	209	16	0	0	0	0	882
7	0.499392	4168.587	413	188	261	20	0	0	0	0	882
8	0.624572	4242.4	380	177	268	57	0	0	0	0	882
9	0.745702	4291.567	358	171	279	60	14	0	0	0	882
10	0.801327	4306.882	352	169	279	68	14	0	0	0	882
11	0.801749	4307.202	352	169	279	68	14	0	0	0	882
12	0.924024	4342.732	343	143	278	86	18	14	0	0	882
13	1.034587	4380.113	338	129	257	122	18	18	0	0	882
14	1.08	4394.474	336	129	247	119	17	34	0	0	882

Table 4-23: Summary on number of plastic hinges for model 2-3: Case I

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.020096	1667.461	879	3	0	0	0	0	0	0	882
2	0.030305	2471.898	830	52	0	0	0	0	0	0	882
3	0.070426	3219.697	773	109	0	0	0	0	0	0	882
4	0.140794	3691.772	652	212	18	0	0	0	0	0	882
5	0.213715	3913.876	584	185	113	0	0	0	0	0	882
6	0.247042	3967.279	545	209	128	0	0	0	0	0	882
7	0.362442	4069.281	497	158	227	0	0	0	0	0	882
8	0.480609	4148.958	478	120	266	18	0	0	0	0	882
9	0.521599	4168.586	461	107	282	32	0	0	0	0	882
10	0.581523	4187.425	452	101	291	38	0	0	0	0	882
11	0.726088	4260.399	440	82	246	114	0	0	0	0	882
12	0.837754	4318.567	432	80	228	126	16	0	0	0	882
13	0.88242	4335.111	420	91	192	163	14	2	0	0	882
14	1.036236	4373.527	417	79	166	188	16	16	0	0	882
15	1.08	4385.872	416	65	173	192	20	16	0	0	882

Table 4-24: Summary on number of plastic hinges for model 2-3: Case II

Step	Displacement (m)	BaseForce (kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
0	0	0	882	0	0	0	0	0	0	0	882
1	0.014005	1162.065	873	9	0	0	0	0	0	0	882
2	0.036403	2787.787	757	125	0	0	0	0	0	0	882
3	0.143972	3600.782	621	226	35	0	0	0	0	0	882
4	0.210311	3843.966	547	253	82	0	0	0	0	0	882
5	0.288537	3993.136	490	255	137	0	0	0	0	0	882
6	0.337045	4044.455	469	243	162	8	0	0	0	0	882
7	0.445045	4117.564	429	203	232	18	0	0	0	0	882
8	0.451773	4129.069	427	203	234	18	0	0	0	0	882
9	0.562362	4208.179	396	175	259	52	0	0	0	0	882
10	0.569112	4210.815	394	177	259	52	0	0	0	0	882
11	0.753719	4305.674	355	176	278	59	14	0	0	0	882
12	0.756151	4308.117	355	174	279	59	14	0	1	0	882
13	0.756152	4273.895	355	172	281	59	14	0	0	1	882
14	0.756785	4275.895	355	171	282	59	14	0	0	1	882
15	0.757981	4278.001	355	171	280	61	14	0	0	1	882
16	0.930682	4354.823	340	146	263	98	20	14	0	1	882
17	1.020426	4390.576	332	143	256	114	20	16	0	1	882
18	1.08	4403.819	329	132	258	112	20	30	0	1	882

Similar to building models considered in category 1, results of summary of plastic hinges for models in category 2 shows that larger number of hinges are at higher damage level in partially detailed models than the fully detailed ones. Therefore, irrespective of the variation in number of storeys and percentage share of frames against base shear, considering frames as secondary seismic member affects the performance of buildings.

4.6. Discussion of results

The results obtained above unanimously show that buildings will well detailed frames function much better than those whose frames are considered as secondary members. From the very beginning the demarcation between primary and secondary members depending on the percentage of base shear the frames and structural walls share should not be limited to results obtained from elastic analysis. As it has been discussed above, analysis in the non-linear range should be carried out in order to fully understand how a certain building behaves. In the sample buildings modeled and analyzed in this thesis work, the results show that when the buildings reach their inelastic range, the percentage of base shear the frames take up increases in folds. This means, when the members are exhibiting such level of deformation, they will be subjected to larger forces than the ones considered in the design. Hence, care should be taken when regarding members as primary or secondary solely based on their elastic performance in the structural system.

Even in the elastic range, it can be seen from the results presented above and in the annex that for building model 1, 2 and 3 considered in category 2, the percentage share of frame against base shear was 9.82 %, 11.27 % and 13.12 % respectively. As the percentage share of secondary seismic members (i.e. frames, in which their contribution against lateral load is neglected in case II of each model) against lateral load increases, it is quite risky to totally ignore their contribution.

The capacity curve shows that in the partially detailed case the selected primary members (structural walls) carry base shear as good as the fully detailed models but with higher damage levels. Even though the number of storeys and the member's stiffness were varying in the two categories under consideration, the relative results were found to be the same. This implies that, irrespective of the number of storeys in a building and the percentage share of the base shear between the walls and frames, having the beams and columns fully detailed helps to achieve a better resistance in the overall structure. As it has been simply put in the above result, the performance point of the fully detailed models are near to the elastic range, and on the other hand larger damage levels are observed for the partially detailed models and hence the overall performance of the structure reduces as well when frames are considered as secondary seismic members.

The comparison between fully detailed and partially detailed scenarios has shown that frames become more part takers when they are fully detailed. This has an advantage in obtaining an efficient design. Rather than having very stiff and highly reinforced walls that take up almost the entire base shear induced, having frames with significant contribution to the overall stiffness of the structure helps in avoiding localized failures.

Having mentioned these basic facts that can be observed from the results presented, the comparison between the two cases in each sample building shows that having well detailed frames positively affects the overall performance of the structure when subject to seismic loading.

In addition to the no collapse limit state design, damage limitation is a crucial factor that needs to be considered in building design. The occupants of the building must not be in dangers that may take their lives or cause a severe damage to their lively hood. How quickly the building can regain its original state, and the cost for retrofitting works also matter in analyzing a building's performance. Formation of hinges at preferred locations helps in attaining this. Hinge formation on beams near to beam column joint avoids occurrence of a soft story. On the other hand, when hinges form on columns near to the beam-column joint, the consequences are harsh. As it has been observed in the results above, members with fully detailed frames follow the correct hierarchy in hinge formation, whereas those whose frames are considered as secondary members have a hinge formation that precedes any other location on the top story columns.

The following points briefly summarize the results obtained from all models considered in this research based on the capacity curves, performance and global yielding points, base shear distribution between frames and shear walls, and plastic hinge mechanisms of the building models:

1) For all building models, at the performance point the partially detailed case has larger number of hinges at higher damage levels than the fully detailed case. Therefore,

neglecting frames as secondary seismic member highly affects the overall performance of buildings against lateral load.

2) For building model 1, 2 and 3 considered in category 2 (varying percentage share of members against base shear), in the linear analysis the percentage share of frame against base shear were 9.82 %, 11.27 % and 13.12 % respectively. As the percentage share of secondary seismic members (i.e. frames, in which their contribution against lateral load is neglected in case II of each model) against lateral load increases, it is quite risky to neglect them.

3) From the results of base shear distribution between frames and shear walls, it is observed that the wall contribution diminishes as the plastic deformation increases. In the linear analysis, in all the models shear walls take above 85% of the total base shear. However, in the inelastic range the contribution of frames increase up to 56.54 %, 43.03%, and 41.67 % for building models having 5, 9, and 13 storeys respectively. Thus as the buildings experience inelastic deformation, the frames do not represent “secondary seismic members”. Comparing the two cases for each building models, frames take larger percent of base shear in the fully detailed cases. Due to this, instead of neglecting frames as “secondary seismic members” it is more advantageous to detail them to resist seismic action since they may experience forces which are larger than what is estimated in the linear analysis.

4) For all the models, the performance point is near to the elastic range for the fully detailed case when compared with partially detailed case. Thus, fully detailed case gives better performance of seismic resistance.

5) Plastic hinge mechanism patterns of the partially detailed case shows that most of the columns in the upper and lower stories experience higher damage levels prior to beams connected to them because the frames in this case are not checked to satisfy capacity design rules.

6) Shear walls in all cases of each model do not experience larger damage level before columns. Therefore, local failure is not a trait.

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

In this study, the design and performance of primary and secondary seismic members of RC dual system is investigated using nonlinear pushover analysis. The effect of considering frames as “secondary seismic members” (when their contribution against lateral load is below 15%) is assessed by comparing overall performance of the model buildings. After detail inspection and comparison of the output from the nonlinear analysis results of each model for the fully detailed and partially detailed case, the following conclusions are drawn:

- Even if the contribution of frames against base shear in the linear analysis is below 15 %, considering these frames as secondary seismic members will reduce the overall performance of the buildings irrespective of the number of storeys. Therefore, in RC dual system it is quite risky to consider frames as “secondary seismic members” based on the linear analysis stiffness limitations.
- In the inelastic range, the contribution of shear walls against lateral loads significantly reduces and contribution of frames increases. Thus as the buildings experience inelastic deformation, the frames do not behave as “secondary seismic members”.
- Frames in the inelastic range are subjected to forces larger than what is estimated from linear analysis. Thus considering these members as primary members is advantageous in order to get better performance of the building even when the forces are increased beyond what is expected in the linear analysis.
- In both fully and partially detailed frame cases shear walls do not experience larger damage levels before columns. Therefore, there is no risk against local failure. However, in partially detailed case columns may experience larger damage levels before beams as they are not checked for capacity design rules.

Therefore, Buildings with RC dual system mentioned in this study have better overall seismic performance when both frames and shear walls are detailed as primary seismic members.

5.2. Recommendations

The stiffness limitation given in Euro codes in order to consider specific members as "secondary seismic members" is determined from linear analysis. According to this limitation, in RC dual system frames can be considered as "secondary seismic members" if they take less than 15% of the total base shear. In which case, the building is designed to relay its lateral load resisting system only by its shear walls. However, as we can see from this research, considering frames as "secondary seismic members" affects the overall performance of the building. Therefore, it is generally recommended to utilize both frames and shear walls to carry lateral loads by designing and detailing them as primary seismic member.

When the contribution of frames against base shear in the linear analysis reduces, the effect of considering them as "secondary seismic member" on the performance of the building is reversed according to non-linear analysis results. Thus to minimize this effect, the stiffness limitation given in codes should be studied and modified considering nonlinear responses of structures.

On this study flexural and interacting hinges are defined for members. Therefore, further study can be made including shear hinges as well in order to capture shear failures.

This study focuses in evaluating the performance of primary and secondary seismic members of RC dual system for regular and non-sway buildings. Hence, an extensive study containing irregular and sway buildings could be carried out to enhance the knowledge regarding the effect of selecting specific members as "secondary seismic members" on the overall performance of structures.

To evaluate the performances, buildings in this study are pushed up to roof displacement of 4% of their respective height. Hence studies should be made to know performances of buildings when pushed beyond 4%.

This study encompasses buildings designed for DCM, further study could be made by using DCH and compare the effect with DCM.

In addition to the performances of primary and secondary seismic members of RC dual system, it is also an interesting topic to compare the economic benefit of utilizing all existing members in the lateral load resisting system.

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APPENDICES

Appendix A: lateral load computation for linear analysis and designing

A.1. General Formulas used to determine seismic weight and base shear.

$$Q_G = \sum G_{kj} + \sum \varphi.\psi_{2i}.Q_{ki} \quad \text{(EN 1998:1 Section 3.2.4)}$$

Where: Q_G = Total seismic weight

$\sum G_{kj}$ = Total seismic dead load

Q_{ki} = Live load and $\varphi.\psi_{2i}$ = Live load participation factors

- Recommended values of $.\psi_{2i}$ for office building (Category B) is 0.3. (EN 1990:2002 Table A1.1)
- For roof $\varphi = 1$ and for other storeys $\varphi = 0.8$ (EN 1998:1 Table 4.2)
- Variable loads are taken from EN 1991-1-1:2002 for office building (Category B), $3kN/m^2$ for all floors except the roof. For roof category H (which is not accessible except for maintenance) variable load is take as $1kN/m^2$

And Base shear is calculated using $F_b = S_d(T_1).m.\lambda$ (EN 1998:1 section 4.2.3.3)

Where: $m =$ total mass of the building above foundation or top of rigid basement

$\lambda = 0.85$ if $T_1 \leq 2Tc$ and more than 2 storeys, $\lambda = 1$ otherwise

$T_1 =$ fundamental period of vibration of the building for lateral motion in the direction considered. Based on Euro code, for buildings with height up to 40 m, $T_1 = C_1(H)^{3/4}$. Where in this study, $C_1 = 0.05$ based on the load resisting system. And H is height of the building.

$S_d(T_1)$ is the design spectrum given in section 3.2.2.5 of EN 1998:1, this parameter is calculated based on Ground type, Ground acceleration, and behavior factor

A.2. Determination of lateral load for model 1

A.2.1 Seismic weight calculation

Dead load calculation for typical floor plan

HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	10	m
Thickness=	0.2	m	Thickness=	0.2	m
Weight=	480	kN	Weight=	150	kN
Columns			**Beams**		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	24	m	Length=	90	m
Depth=	0.4	m	Depth=	0.3	m
Width=	0.4	m	Width=	0.3	m
Weight=	96	kN	Weight=	203	kN
Slab					
unit weight=	25	kN/m3			
Length=	13	m			
Width=	13	m			
Thickness=	0.2	m			
Weight=	586	kN	Total DL=	1514	KN

Dead load calculation for roof level

HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	m
Length=	40	m	Length=	10	m
Thickness=	0.2	m	Thickness=	0.2	m
Weight=	240	kN	Weight=	75	kN
Columns			**Beams**		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	12	m	Length=	90	m
Depth=	0.4	m	Depth=	0.3	m
Width=	0.4	m	Width=	0.3	m
Weight=	48	kN	Weight=	202.5	kN
Slab					
unit weight=	25	kN/m3			
Length=	12.5	m			
Width=	12.5	m			
Thickness=	0.15	m			
Weight=	585.94	kN	Total DL=	1151.44	KN

Dead load calculation for ground floor plan

HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	M
Length=	40	m	Length=	10	M
Thickness=	0.2	m	Thickness=	0.2	M
Weight=	240	kN	Weight=	75	kN
Columns					
unit weight=	25	kN/m3			
Length=	12	m			
Depth=	0.4	m			
Width=	0.4	m			
Weight=	48	kN	total DL=	363	KN

Total seismic load					
Typical floor plan		Roof level		Ground level	
total DL (kN)=	1514.4375	total DL (kN)=	1151.4375	total DL (kN)=	363
Total LL (kN)=	112.5	Total LL (kN)=	46.875	Total LL (kN)=	0
Total W (kN) =	1626.9375	Total W (kN)=	1198.3125	Total W (kN)=	363
Total seismic weight of building model 1 = 8069.0625 kN					

A.2.2 Base shear Determination and distribution over the height of the building

$F_b = S_d(T_1).m.\lambda$

$T_1 = C_1(H)^{3/4} = 0.05*(15)^{3/4} = 0.381\sec$

For ground Type C, $T_B = 0.2\sec$, $T_C = 0.6\sec$, $T_D = 2\sec, S = 1.15$ (EN 1998:1 Table 3.2)

Since $T_1 \le \begin{cases} 4Tc = 2.4Sec \\ 2\sec \end{cases}$ Lateral force method can be used

$S_d(T_1) = a_g.S.\frac{2.5}{q}$ for $T_B \le T \le T_C$

$a_g = 0.981$ (Taken value of pick ground acceleration for high seismic zone)

$q = q_0.k_w$ (Behavior factor, $q_0 = 3\alpha_u/\alpha_1$

For wall equivalent system $\alpha_u/\alpha_1 = 1.2$

Calculated value of $k_w = 1$

Thus $q = q_0.k_w = 3*1.2*1 = 3.6$

$S_d(T_1) = a_g.S.\frac{2.5}{q} = 0.981*1.15*2.5/3.6 = 0.7986$

$F_b = S_d(T_1).m.\lambda = 0.7986*(8069.0625/9.81)*0.85 = 547.44kN$

Base shear Distribution over the height of the building				
Storey	Height(m)	mi (kN)	hi*mi	Fi (KN)
Roof	15	1198.31	17974.69	147.34
4th	12	1626.94	19523.25	160.04
3rd	9	1626.94	14642.44	120.03
2nd	6	1626.94	9761.63	80.02
1st	3	1626.94	4880.81	40.01
Ground	0	363.00	0.00	0.00
	Total	8069.06	66782.81	547.44

A.3 Determination of lateral load for model 2

A.3.1 Seismic weight calculation

Dead load calculation for typical floor plan 1,2,3					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	24	m
Thickness=	0.2	m	Thickness=	0.2	m
Weight=	480	kN	Weight=	360	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	136	m
Depth=	0.6	m	Depth=	0.3	m
Width=	0.6	m	Width=	0.3	m
Weight=	351	kN	Weight=	306	kN
Slab					
unit weight=	25	kN/m3			
Length=	16	m			
Width=	16	m			
Thickness=	0.2	m			
Weight=	1280	kN	Total DL=	2777	kN

Dead load calculation for typical floor plan 4,5,6					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	24	m
Thickness=	0.2	m	Thickness=	0.2	m
Weight=	480	kN	Weight=	360	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	136	m
Depth=	0.5	m	Depth=	0.3	m
Width=	0.5	m	Width=	0.3	m
Weight=	243.75	kN	Weight=	306	kN
Slab					
unit weight=	25	kN/m3			
Length=	16	m			
Width=	16	m			
Thickness=	0.2	m			
Weight=	1280	kN	Total DL=	2669.75	kN

Dead load calculation for typical floor plan 7,8					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	M
Length=	40	m	Length=	24	M
Thickness=	0.2	m	Thickness=	0.2	M
Weight=	480	kN	Weight=	360	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	136	M
Depth=	0.4	m	Depth=	0.3	M
Width=	0.4	m	Width=	0.3	M
Weight=	156	kN	Weight=	306	kN
Slab					
unit weight=	25	kN/m3			
Length=	16	m			
Width=	16	m			
Thickness=	0.2	m			
Weight=	1280	kN	Total DL=	2582	kN

Dead load calculation for roof level					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	m
Length=	40	m	Length=	24	m
Thickness=	0.2	m	Thickness=	0.2	m
Weight=	240	kN	Weight=	180	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	19.5	m	Length=	136	m
Depth=	0.4	m	Depth=	0.3	m
Width=	0.4	m	Width=	0.3	m
Weight=	78	kN	Weight=	306	kN
Slab					
unit weight=	25	kN/m3			
Length=	16	m			
Width=	16	m			
Thickness=	0.2	m			
Weight=	1280	kN	Total DL=	2084	kN

Dead load calculation for ground floor plan					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	M
Length=	40	m	Length=	24	M
Thickness=	0.2	m	Thickness=	0.2	M
Weight=	240	kN	Weight=	180	kN
Columns					
unit weight=	25	kN/m3			
Length=	19.5	m			
Depth=	0.6	m			
Width=	0.6	m			
Weight=	175.5	kN	Total DL=	595.5	KN

Total seismic load					
Typical floor plan 1,2,3		Typical floor plan 4,5,6		Typical floor plan 7,8	
Total DL (kN)=	2777	Total DL (kN)=	2669.75	Total DL (kN)=	2582
Total LL (kN)=	184.32	Total LL (kN)=	184.32	Total LL (kN)=	184.32
Total W=	2961.32	Total W=	2854.07	Total W=	2766.32
Ground level		Roof level			
Total DL (kN)=	595.5	Total DL (kN)=	2084		
Total LL (kN)=	0	Total LL (kN)=	76.8		
Total W=	595.5	Total W=	2160.8		
Total seismic weight of building model 2 = 25,735.11 kN					

A.3.2 Base Shear Determination and distribution over the height of the building

$F_b = S_d(T_1).m.\lambda$

$T_1 = C_1(H)^{3/4} = 0.05*(27)^{3/4} = 0.59\sec$

For ground Type C, $T_B = 0.2\sec$, $T_C = 0.6\sec$, $T_D = 2\sec$, $S = 1.15$ (EN 1998:1 Table 3.2)

Since $T_1 \le \begin{cases} 4Tc = 2.4Sec \\ 2\sec \end{cases}$ Lateral force method can be used

$S_d(T_1) = a_g.S.\frac{2.5}{q}$ for $T_B \le T \le T_C$

$a_g = 0.981$ (Taken value of pick ground acceleration for high seismic zone)

$q = q_0.k_w$ (Behavior factor, $q_0 = 3\alpha_u/\alpha_1$

For wall equivalent system $\alpha_u/\alpha_1 = 1.2$

Calculated value of $k_w = 1$

Thus $q = q_0.k_w = 3*1.2*1 = 3.6$

$S_d(T_1) = a_g.S.\frac{2.5}{q} = 0.981*1.15*2.5/3.6 = 0.7986$

$F_b = S_d(T_1).m.\lambda = 0.7986*(25735.11/9.81)*0.85 = 1745.97kN$

Base shear Distribution over the height of the building				
Storey	Height(m)	mi (kN)	hi*mi	Fi (KN)
Roof	27	2160.80	58341.60	279.41
8th	24	2766.32	66391.68	317.96
7th	21	2766.32	58092.72	278.22
6th	18	2854.07	51373.26	246.04
5th	15	2854.07	42811.05	205.03
4th	12	2854.07	34248.84	164.03
3rd	9	2961.32	26651.88	127.64
2nd	6	2961.32	17767.92	85.09
1st	3	2961.32	8883.96	42.55
Ground	0	595.50	0.00	0.00
	Total	25735.11	364562.91	1745.97

A.4 Determination of lateral load for model 3

A.4.1 Seismic weight calculation

Dead load calculation for typical floor plan 1,2,3,4,5					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	480	kN	Weight=	600	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	148	m
Depth=	0.7	m	Depth=	0.4	m
Width=	0.7	m	Width=	0.4	m
Weight=	477.75	kN	Weight=	592	
Slab					
unit weight=	25	kN/m3			
Length=	18	m			
Width=	18	m			
Thickness=	0.2	m			
Weight=	1620	kN	Total DL=	3769.75	kN

Dead load calculation for typical floor plan 6,7,8					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	480	kN	Weight=	600	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	148	m
Depth=	0.6	m	Depth=	0.4	m
Width=	0.6	m	Width=	0.4	m
Weight=	351	kN	Weight=	592	
Slab					
unit weight=	25	kN/m3			
Length=	18	m			
Width=	18	m			
Thickness=	0.2	m			
Weight=	1620	kN	Total DL=	3643	KN

Dead load calculation for typical floor plan 9,10,11					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	480	kN	Weight=	600	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	148	m
Depth=	0.5	m	Depth=	0.4	m
Width=	0.5	m	Width=	0.4	m
Weight=	243.75	kN	Weight=	592	kN
Slab					
unit weight=	25	kN/m3			
Length=	18	m			
Width=	18	m			
Thickness=	0.2	m			
Weight=	1620	kN	Total DL=	3535.75	KN

Dead load calculation for typical floor plan 12					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	3	m	height=	3	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	480	kN	Weight=	600	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	39	m	Length=	148	m
Depth=	0.4	m	Depth=	0.4	m
Width=	0.4	m	Width=	0.4	m
Weight=	156	kN	Weight=	592	kN
Slab					
unit weight=	25	kN/m3			
Length=	18	m			
Width=	18	m			
Thickness=	0.2	m			
Weight=	1620	kN	Total DL=	3448	kN

Dead load calculation for roof level					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	240	kN	Weight=	300	kN
Columns			Beams		
unit weight=	25	kN/m3	unit weight=	25	kN/m3
Length=	19.5	m	Length=	148	m
Depth=	0.4	m	Depth=	0.4	m
Width=	0.4	m	Width=	0.4	m
Weight=	78		Weight=	592	kN
Slab					
unit weight=	25	kN/m3			
Length=	18	m			
Width=	18	m			
Thickness=	0.2	m			
Weight=	1620	kN	Total DL=	2830	KN

Dead load calculation for ground floor plan					
HCB wall			Shear wall		
unit weight=	20	kN/m3	unit weight=	25	kN/m3
height=	1.5	m	height=	1.5	m
Length=	40	m	Length=	32	m
Thickness=	0.2	m	Thickness=	0.25	m
Weight=	240	kN	Weight=	300	kN
Columns					
unit weight=	25	kN/m3			
Length=	19.5	m			
Depth=	0.7	m			
Width=	0.7	m			
Weight=	238.875	kN	Total DL=	778.875	KN

Total seismic load					
Typical floor plan 1,2,3,4,5		Typical floor plan 6,7,8		Typical floor plan 9,10,11	
Total DL (kN)=	3769.75	Total DL (kN)=	3643	Total DL (kN)=	3535.75
Total LL (kN)=	233.28	Total LL (kN)=	233.28	Total LL (kN)=	233.28
Total W=	4003.03	Total W=	3876.28	Total W=	3769.03
Typical floor plan 12		Roof level		Ground level	
Total DL (kN)=	3448	Total DL (kN)=	2830	Total DL (kN)=	778.875
Total LL (kN)=	233.28	Total LL (kN)=	97.2	Total LL (kN)=	0
Total W=	3681.28	Total W=	2927.2	Total W=	778.875
Total seismic weight of building model 3 = 50,338.435 kN					

A.4.2 Base shear Determination and distribution over the height of the building

$F_b = S_d(T_1).m.\lambda$

$T_1 = C_1(H)^{3/4} = 0.05*(39)^{3/4} = 0.7803\sec < 2\sec$, Lateral force method can be used

For ground Type C, $T_B = 0.2\sec$, $T_C = 0.6\sec$, $T_D = 2\sec$, $S = 1.15$ (EN 1998:1 Table 3.2)

$$S_d(T_1) \quad \text{is} \quad \begin{cases} = a_g.S.\dfrac{2.5}{q}\left[\dfrac{T_C}{T}\right] \\ \geq \beta.a_g \end{cases}$$

β is 0.2 (Recommended value in EN 1998:1)

$a_g = 0.981$ (Taken value of pick ground acceleration for high seismic zone)

$q = q_0.k_w = 3*1.2*1 = 3.6$ (Similar with model 1 and model 2)

$$S_d(T_1) \begin{cases} = a_g.S.\dfrac{2.5}{q}\left[\dfrac{T_C}{T}\right] = 0.981*1.15*\dfrac{2.5}{3.6}\left[\dfrac{0.6}{0.7803}\right] = 0.6024 \\ \geq \beta.a_g = 0.2*0.981 = 0.1962 \end{cases}$$

$F_b = S_d(T_1).m.\lambda = 0.6024*(50338.435/9.81)*0.85 = 2627.45\text{kN}$

Base shear Distribution over the height of the building				
Storey	Height(m)	mi (kN)	hi*mi	Fi (KN)
Roof	39	2927.20	114160.80	296.91
12th	36	3681.28	132526.08	344.68
11th	33	3769.03	124377.99	323.48
10th	30	3769.03	113070.90	294.08
9th	27	3769.03	101763.81	264.67
8th	24	3876.28	93030.72	241.96
7th	21	3876.28	81401.88	211.71
6th	18	3876.28	69773.04	181.47
5th	15	4003.03	60045.45	156.17
4th	12	4003.03	48036.36	124.93
3rd	9	4003.03	36027.27	93.70
2nd	6	4003.03	24018.18	62.47
1st	3	4003.03	12009.09	31.23
Ground	0	778.88	0.00	0.00
	Total	50338.44	1010241.57	2627.45

Appendix B: lateral load computation for nonlinear pushover analysis

B.1 General Formulas used to calculate lateral load for pushover analysis

Lateral load for pushover analysis is calculated the following formulas taken from FEMA 356

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{W_x h_x^{\ k}}{\sum_{i=1}^{n} W_i h_i^{\ k}} \quad \text{(Vertical distribution factor)}$$

$Wi =$ Portion of total building weight W located on or assigned to floor level i.

$Wx =$ Portion of total building weight W located on or assigned to floor level x.

$h_i =$ Height from the base to floor level i.

$h_x =$ Height from the base to floor level x.

$$k = \begin{cases} 2 & \text{for} \quad T \geq 2.5\,\text{sec} \\ 1 & \text{for} \quad T \leq 0.5\,\text{sec} \end{cases}$$

$$V = C_1.C_2.C_3.C_m.S_a.W \quad \text{(Pseudo lateral load)}$$

- $C_1 =$ Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.
- $C_2 =$ Modification factor to represent the effect of pinched hysteresis shape, stiffness degradation, and strength deterioration on maximum response
- $C_3 =$ Modification factor to represent increased displacements due to dynamic $P-\Delta$ effects
- $C_m =$ Mass factor
- $S_a =$ Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.
- $W =$ Seismic weight (Dead load and portion of live load)

B.2 Lateral load calculation for pushover analysis for Model 1

$$T_1 = C_1 (H)^{3/4} = 0.05 * (15)^{3/4} = 0.381\sec$$

For $T \leq 0.5\sec\, k = 1$

$$V = C_1.C_2.C_3.C_m.S_a.W$$

- $C_1 = \begin{cases} 1.5 & \text{for} \quad T < 0.1 \\ 1.0 & \text{for} \quad T > T_s \end{cases}$ Where: $T_s = S_{x1}B_s / S_{xs}B_1$

  For high seismic zone $S_{xs} = 0.5g$ and $S_{x1} = 0.2g$ (section 1.6.3.1 of FEMA 356)

  $B_s = B_1 = 1$ (From table 1-6 of FEMA 356)

  Thus $T_s = S_{x1}B_s / S_{xs}B_1 = 0.2/0.5 = 0.4\sec$

  Using linear interpolation $C_1 = 1.03167$
- $S_a = S_{xs} / B_s$ for $T < T_s$ (From section 1.6.1.5 of FEMA 356)

  $S_a = S_{xs} / B_s = 0.5g$
- $C_2 = 1$ (From table 3-3 of FEMA 356)
- $C_3 = 1$ (P-Δ effect is verified)
- $C_m = 0.8$ (Table 3-1 of FEMA 356)
- $W = 8069.06kN$

$$V = C_1.C_2.C_3.C_m.S_a.W = 3329.84kN$$

Distribution of lateral load to floor level for pushover analysis

Storey level	Height(m)	Wi (KN)	$Wi.h_i^k$	Cvx	Fx (KN)
Roof	15	1198.31	17974.69	0.2692	896.23
4	12	1626.94	19523.25	0.2923	973.44
3	9	1626.94	14642.44	0.2193	730.08
2	6	1626.94	9761.63	0.1462	486.72
1	3	1626.94	4880.81	0.0731	243.36
Ground	0	363.00	0.00	0.0000	0.00
	Total=	8069.0625	66782.8125		3329.84

B.3 Lateral load calculation for pushover analysis for Model 2

$T_1 = C_1 (H)^{3/4} = 0.05 * (27)^{3/4} = 0.59 \sec$, Using linear interpolation $k = 1.045$

$V = C_1.C_2.C_3.C_m.S_a.W$

- $C_1 = \begin{cases} 1.5 & \text{for} \quad T < 0.1 \\ 1.0 & \text{for} \quad T > T_s \end{cases}$ Where: $T_s = S_{x1} B_s / S_{xs} B_1$

  For high seismic zone $S_{xs} = 0.5g$ and $S_{x1} = 0.2g$ (section 1.6.3.1 of FEMA 356)

  $B_s = B_1 = 1$ (From table 1-6 of FEMA 356)

  Thus $T_s = S_{x1} B_s / S_{xs} B_1 = 0.2 / 0.5 = 0.4 \sec$

  For $T > T_s$, $C_1 = 1$
- $S_a = S_{x1} / B_1 T$ for $T > T_s$ (From section 1.6.1.5 of FEMA 356)

  $S_a = S_{x1} / B_1 T = 0.5 / 0.59 = 0.84746g$
- $C_2 = 1$ (From table 3-3 of FEMA 356)
- $C_3 = 1$ (P-Δ effect is verified)
- $C_m = 0.8$ (Table 3-1 of FEMA 356)
- $W = 25735.11 kN$

$V = C_1.C_2.C_3.C_m.S_a.W = 17447.58 kN$

Distribution of lateral load to floor level for pushover analysis					
Storey level	Height(m)	Wi (KN)	$Wi.h_i^k$	Cvx	Fx (KN)
Roof	27	2160.80	67668.99	0.1634	2851.31
8	24	2766.32	76599.01	0.1850	3227.59
7	21	2766.32	66622.60	0.1609	2807.22
6	18	2854.07	58509.23	0.1413	2465.36
5	15	2854.07	48359.30	0.1168	2037.68
4	12	2854.07	38300.90	0.0925	1613.85
3	9	2961.32	29421.77	0.0711	1239.72
2	6	2961.32	19259.87	0.0465	811.54
1	3	2961.32	9334.20	0.0225	393.31
Ground	0	595.50	0.00	0.0000	0.00
	Total=	25735.11	414075.8743		17447.58

B.4 Lateral load calculation for pushover analysis for Model 3

$T_1 = C_1(H)^{3/4} = 0.05*(39)^{3/4} = 0.7803\sec$, using linear interpolation $k = 1.14015$

$V = C_1.C_2.C_3.C_m.S_a.W = 25804.69\text{kN}$

- $C_1 = \begin{cases} 1.5 & \text{for} \quad T < 0.1 \\ 1.0 & \text{for} \quad T > T_s \end{cases}$ Where: $T_s = S_{x1}B_s / S_{xs}B_1$

  For high seismic zone $S_{xs} = 0.5g$ and $S_{x1} = 0.2g$ (section 1.6.3.1 of FEMA 356)

  $B_s = B_1 = 1$ (From table 1-6 of FEMA 356)

  Thus $T_s = S_{x1}B_s / S_{xs}B_1 = 0.2/0.5 = 0.4\sec$

  For $T > T_s, C_1 = 1$

- $S_a = \dfrac{S_{x1}}{B_1 T} = \dfrac{0.5}{0.7803} = 0.64078g$ for $T > T_s$ (Section 1.6.1.5 of FEMA 356)
- $C_2 = 1$ (From table 3-3 of FEMA 356), $C_3 = 1$ (P-Δ effect is verified)
- $C_m = 0.8$ (Table 3-1 of FEMA 356), and $W = 50338.44\text{kN}$

Distribution of lateral load to floor level for pushover analysis

Storey level	Height(m)	Wi (KN)	$Wi.h_i^k$	Cvx	Fx (KN)
Roof	39	2927.20	190767.64	0.1207	3114.24
12	36	3681.28	218986.40	0.1385	3574.90
11	33	3769.03	203031.43	0.1284	3314.44
10	30	3769.03	182124.93	0.1152	2973.15
9	27	3769.03	161509.84	0.1022	2636.61
8	24	3876.28	145232.22	0.0919	2370.88
7	21	3876.28	124722.12	0.0789	2036.06
6	18	3876.28	104619.85	0.0662	1707.90
5	15	4003.03	87762.56	0.0555	1432.70
4	12	4003.03	68048.31	0.0430	1110.87
3	9	4003.03	49019.45	0.0310	800.23
2	6	4003.03	30874.36	0.0195	504.02
1	3	4003.03	14008.08	0.0089	228.68
Ground	0	778.88	0.00	0.0000	0.00
	Total=	50338.44	1580707.19		25804.69

Appendix C: Second order (P-Δ) effect checks

C.1 General Formula used to check second order effect

According to EN 1998:1, Second-order effects (P-Δ effects) need not be taken into account if the interstorey drift sensitivity coefficient: $\theta \leq 0.1$.

$$\theta = \frac{P_{tot}.d_r}{V_{tot}.h}$$

Where: θ is the interstorey drift sensitivity coefficient.

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation.

d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration and calculated in accordance with 4.3.4

$$\text{Real diplacement} = q.\ (\text{Displacement from Linear Analysis})$$

$$q = \text{Behaviour factor}$$

V_{tot} is the total seismic storey shear; and

h is the interstorey height

C.2 Second order effect checks for Model 1

Storey	displacement (mm)	real d (mm)	dr (mm)	P,total (kN)	Vi (kN)	Vtotal (kN)	Height (mm)	θ
5th	11.214	40.3704	9.504	2,035.46	147.34	147.34	3000	0.043764
4th	8.574	30.8664	10.0008	5,754.66	160.04	307.38	3000	0.06241
3rd	5.796	20.8656	9.6336	9,473.86	120.03	427.41	3000	0.071179
2nd	3.12	11.232	7.7076	13,193.07	80.02	507.43	3000	0.066799
1st	0.979	3.5244	3.5244	16,912.26	40.01	547.44	3000	0.036294
Ground	0	0	0					
				Total=	547.44			

C.3 Second order effect checks for Model 2

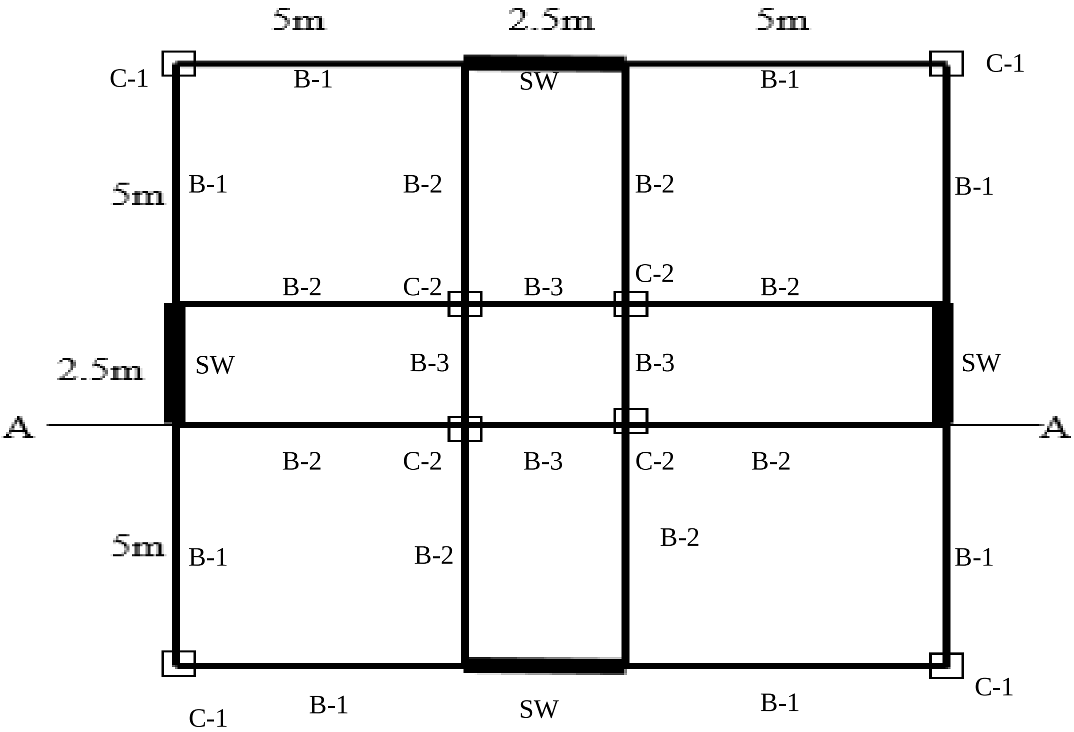
Storey	displacement (mm)	real d (mm)	dr (mm)	P,total (kN)	Vi (kN)	Vtotal (kN)	Height (mm)	θ
9th	20.88	75.15	10.27	4,012.58	279.41	279.41	3000	0.049183
8th	18.02	64.88	10.50	10,818.08	317.96	597.38	3000	0.063390
7th	15.10	54.37	10.57	17,623.58	278.22	875.59	3000	0.070913
6th	12.17	43.80	10.35	24,393.10	246.04	1,121.63	3000	0.075004
5th	9.29	33.46	9.82	31,491.23	205.03	1,326.66	3000	0.077706
4th	6.57	23.64	8.83	38,553.39	164.03	1,490.69	3000	0.076130
3rd	4.11	14.81	7.29	45,615.56	127.64	1,618.33	3000	0.068494
2nd	2.09	7.52	5.19	52,894.90	85.09	1,703.43	3000	0.053695
1st	0.65	2.33	2.33	60,174.25	42.55	1,745.97	3000	0.026758
Ground	0.00							
				Total=	1,745.97			

C.4 Second order effect checks for Model 3

Storey	displacement (mm)	real d (mm)	dr (mm)	P,total (kN)	Vi (kN)	Vtotal (kN)	Height (mm)	θ
13th	23.702	85.3272	7.5996	6,092.57	296.91	296.91	3000	0.051980882
12th	21.591	77.7276	7.7976	15,893.87	344.68	641.59	3000	0.064389336
11th	19.425	69.93	7.9128	25,695.20	323.48	965.07	3000	0.070226616
10th	17.227	62.0172	8.0532	35,753.19	294.08	1259.15	3000	0.076222906
9th	14.99	53.964	8.1108	45,812.24	264.67	1523.82	3000	0.081281446
8th	12.737	45.8532	8.0064	55,869.21	241.96	1765.77	3000	0.084441125
7th	10.513	37.8468	7.8084	66,240.94	211.71	1977.48	3000	0.087187567
6th	8.344	30.0384	7.4196	76,612.64	181.47	2158.95	3000	0.087764153
5th	6.283	22.6188	6.786	86,984.34	156.17	2315.12	3000	0.084988623
4th	4.398	15.8328	5.958	97,726.81	124.93	2440.05	3000	0.079541575
3rd	2.743	9.8748	4.8384	108,469.24	93.70	2533.75	3000	0.069043573
2nd	1.399	5.0364	3.4272	119,211.72	62.47	2596.22	3000	0.052456112
1st	0.447	1.6092	1.6092	129,954.18	31.23	2627.45	3000	0.026530439
Ground	0				0.00			
				Total=	2627.45			

Appendix D: Design results

D.1 Design results for 5 storey building (model 1 in category 1)



Plan view of 5 storey building

D.1.1 Design result for beams (section: 300X300 mm)

Model 1 , Design result of primary Beams			
Storey	Location	Reinforcement Top	Reinforcement Bottom
B-1			
5	Left	5φ14	2φ14
	Right	6φ14	3φ14
2,3,4	Left	5φ20	5φ14
	Right	5φ20	5φ14
1	Left	7φ16	4φ14
	Right	5φ20	6φ12
B-2			
5	Left	4φ16	2φ14
	Right	6φ16	3φ14
1,2,3,4	Left	5φ18	3φ14
	Right	5φ20	6φ14
B-3			
1,5	Left	5φ12	2φ12
	Right	5φ12	2φ12
2,3,4	Left	5φ14	2φ14
	Right	5φ14	2φ14

Model 1 , Design result of Secondary Beams			
Storey	Location	Reinforcement Top	Reinforcement Bottom
B-1			
5	Left	3φ12	2φ12
	Right(M)	2φ12	3φ14
1,2,3,4	Left	4φ12	2φ12
	Right(M)	4φ18	7φ18
B-2			
5	Left(M)	2φ18	6φ16
	Right	7φ14	2φ16
1,2,3,4	Left(M)	2φ12	5φ20
	Right	5φ20	2φ12
B-3			
5	Left	6φ12	2φ12
	Right	6φ12	2φ12
1,2,3,4	Left	5φ16	2φ16
	Right	5φ16	2φ16

D.1.2 Design result for Columns

Model 1: Design results of primary columns				
	Cross sections		Reinforcement	
Storey	b (mm)	h (mm)	C-1	C-2
5	400	400	8φ16	8φ16
4	400	400	8φ16	8φ16
3	400	400	8φ16	8φ16
2	400	400	8φ16	8φ16
1	400	400	8φ16	8φ16

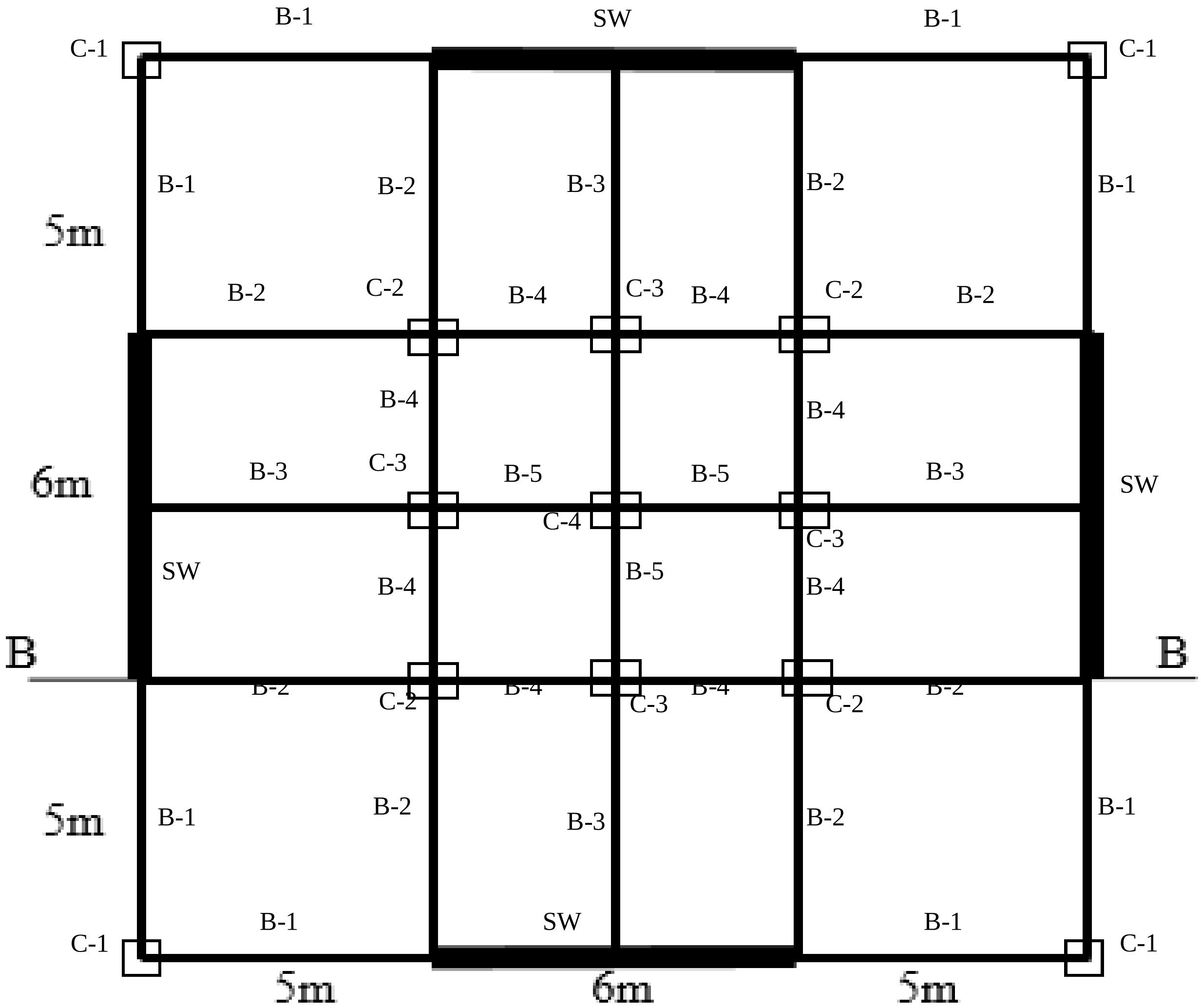
Model 1: Design results of secondary columns				
	Cross sections		Reinforcement	
Storey	b (mm)	h (mm)	C-1	C-2
5	400	400	4φ12	4φ12
4	400	400	4φ12	4φ12
3	400	400	4φ12	4φ12
2	400	400	4φ12	4φ12
1	400	400	4φ12	4φ12

D.1.3 Design result for Shear walls (length = 2.5 m, height 3m)

Model 1: Design result of Shear wall for fully detailed case				
Storey	b (mm)	Vertical reinforcement	Horizontal reinforcement	Med/Mrd
5	200	φ12 c/c 300 mm	φ8 c/c 250 mm	0.76
4	200	φ12 c/c 300 mm	φ8 c/c 250 mm	0.84
3	200	φ12 c/c 200 mm	φ8 c/c 250 mm	0.92
2	200	φ12 c/c 170 mm	φ8 c/c 250 mm	0.97
1	200	φ12 c/c 130 mm	φ8 c/c 250 mm	0.98

Model 1: Design result of Shear wall for partially detailed case				
Storey	b (mm)	Vertical reinforcement	Horizontal reinforcement	Med/Mrd
5	200	φ12 c/c 170 mm	φ8 c/c 250 mm	0.97
4	200	φ12 c/c 120 mm	φ8 c/c 250 mm	0.98
3	200	φ14 c/c 100 mm	φ8 c/c 250 mm	0.9
2	200	φ14 c/c 100 mm	φ8 c/c 250 mm	0.99
1	200	φ14 c/c 80 mm	φ8 c/c 250 mm	1

D.2 Design results for 9 storey building (model 2 in category 1)



Plan view of 9 storey buildings

D.2.1 Design result for beams (Section: 300X300 mm)

Model 2 , Design result of primary			
Storey	Location	Reinforcement Top	Reinforcement Bottom
B-1			
9	Left	5ϕ16	2ϕ16
	Right	7ϕ14	3ϕ14
5,6,7,8	Left	7ϕ18	5ϕ16
	Right	7ϕ18	5ϕ16
4	Left	7ϕ18	5ϕ16
	Right	5ϕ20	6ϕ14

Model 2 , Design result of Secondary Beams			
Storey	Location	Reinforcement Top	Reinforcement Bottom
B-1			
9	Left	3ϕ12	2ϕ12
	Right(M)	2ϕ12	6ϕ14
1,2,3,4,5,6,7,8	Left	6ϕ12	2ϕ12
	Right(M)	5ϕ12	6ϕ20

Model 2 , Design result of primary Beams			
Storey	Location	Reinforcement	
		Top	Bottom
B-1			
3	Left	5ϕ20	6ϕ14
	Right	5ϕ20	4ϕ16
1,2	Left	6ϕ18	5ϕ14
	Right	6ϕ18	6ϕ12
B-2			
9	Left	5ϕ16	2ϕ14
	Right	7ϕ16	4ϕ14
7,8	Left	7ϕ16	3ϕ16
	Right	6ϕ20	6ϕ16
4,5,6	Left	5ϕ18	3ϕ14
	Right	8ϕ18	5ϕ18
1,2,3	Left	5ϕ18	3ϕ14
	Right	8ϕ18	7ϕ16
B-3			
9	Left	7ϕ14	2ϕ16
	Right	6ϕ16	4ϕ12
7,8	Left	5ϕ20	5ϕ14
	Right	5ϕ20	5ϕ14
5,6	Left	5ϕ20	6ϕ12
	Right	5ϕ20	4ϕ16
1,2,3,4	Left	5ϕ20	4ϕ14
	Right	5ϕ20	4ϕ16
B-4			
9	Left	4ϕ14	2ϕ12
	Right	6ϕ12	2ϕ14
8	Left	6ϕ12	2ϕ14
	Right	6ϕ14	3ϕ12
4,7	Left	5ϕ14	2ϕ14
	Right	6ϕ14	3ϕ12
5,6	Left	5ϕ14	2ϕ14
	Right	5ϕ16	2ϕ16
3	Left	5ϕ14	2ϕ14
	Right	4ϕ16	2ϕ14
2	Left	6ϕ12	2ϕ12
	Right	5ϕ14	2ϕ14
1	Left	5ϕ12	2ϕ12
	Right	4ϕ14	2ϕ14
B-5			
9	Left	4ϕ14	2ϕ12
	Right	4ϕ14	2ϕ12
4,7,8	Left	5ϕ14	2ϕ14
	Right	6ϕ14	3ϕ12
5,6	Left	4ϕ16	2ϕ14
	Right	6ϕ14	3ϕ12
3	Left	5ϕ14	2ϕ14
	Right	4ϕ16	2ϕ14
2	Left	6ϕ12	2ϕ12
	Right	5ϕ14	2ϕ14
1	Left	5ϕ12	2ϕ12
	Right	5ϕ12	2ϕ12

Model 2 , Design result of Secondary Beams			
Storey	Location	Reinforcement	
		Top	Bottom
B-2			
9	Left(M)	5ϕ14	6ϕ18
	Right	7ϕ16	5ϕ14
7,8	Left(M)	5ϕ16	6ϕ20
	Right	7ϕ18	5ϕ16
4,5,6	Left(M)	5ϕ16	6ϕ20
	Right	6ϕ20	5ϕ16
2,3	Left(M)	6ϕ16	6ϕ20
	Right	6ϕ20	2ϕ18
1	Left(M)	2ϕ16	6ϕ20
	Right	8ϕ18	6ϕ16
B-3			
9	Left(M)	2ϕ18	5ϕ18
	Right	6ϕ16	4ϕ14
7,8	Left(M)	4ϕ16	5ϕ20
	Right	5ϕ20	4ϕ16
4,5,6	Left(M)	4ϕ16	8ϕ16
	Right	8ϕ16	4ϕ16
2,3	Left(M)	4ϕ16	8ϕ16
	Right	7ϕ18	5ϕ16
1	Left(M)	4ϕ16	5ϕ20
	Right	7ϕ18	5ϕ16
B-4			
9	Left	7ϕ12	2ϕ14
	Right	3ϕ14	2ϕ12
7,8	Left	7ϕ16	7ϕ12
	Right	3ϕ14	2ϕ12
4,5,6	Left	6ϕ18	7ϕ12
	Right	3ϕ14	2ϕ12
2,3	Left	7ϕ18	5ϕ16
	Right	2ϕ12	3ϕ14
1	Left	8ϕ16	5ϕ16
	Right	2ϕ12	3ϕ14
B-5			
9	Left	7ϕ14	4ϕ12
	Right	3ϕ14	2ϕ12
7,8	Left	7ϕ16	4ϕ16
	Right	3ϕ14	2ϕ12
5,6	Left	6ϕ18	4ϕ16
	Right	3ϕ14	2ϕ12
4	Left	6ϕ18	4ϕ16
	Right	6ϕ18	4ϕ16
2,3	Left	6ϕ18	4ϕ16
	Right	2ϕ12	3ϕ14
1	Left	5ϕ20	4ϕ16
	Right	2ϕ12	3ϕ14

D.2.2 Design result for Columns

Model 2: Design results of primary columns												
	C-1			C-2			C-3			C-4		
Storey	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.
9	400	400	8φ16	400	400	8φ16	400	400	8φ16	400	400	8φ16
8	400	400	8φ16	400	400	8φ16	400	400	8φ16	400	400	8φ16
7	400	400	8φ16	400	400	8φ16	400	400	8φ16	400	400	8φ16
6	500	500	8φ20	500	500	8φ20	500	500	8φ20	500	500	8φ20
5	500	500	8φ20	500	500	8φ20	500	500	8φ20	500	500	8φ20
4	500	500	8φ20	500	500	8φ20	500	500	8φ20	500	500	8φ20
3	500	500	8φ20	600	600	12φ20	600	600	12φ20	600	600	12φ20
2	500	500	8φ20	600	600	12φ20	600	600	12φ20	600	600	12φ20
1	500	500	8φ20	650	650	16φ20	600	600	12φ20	600	600	12φ20

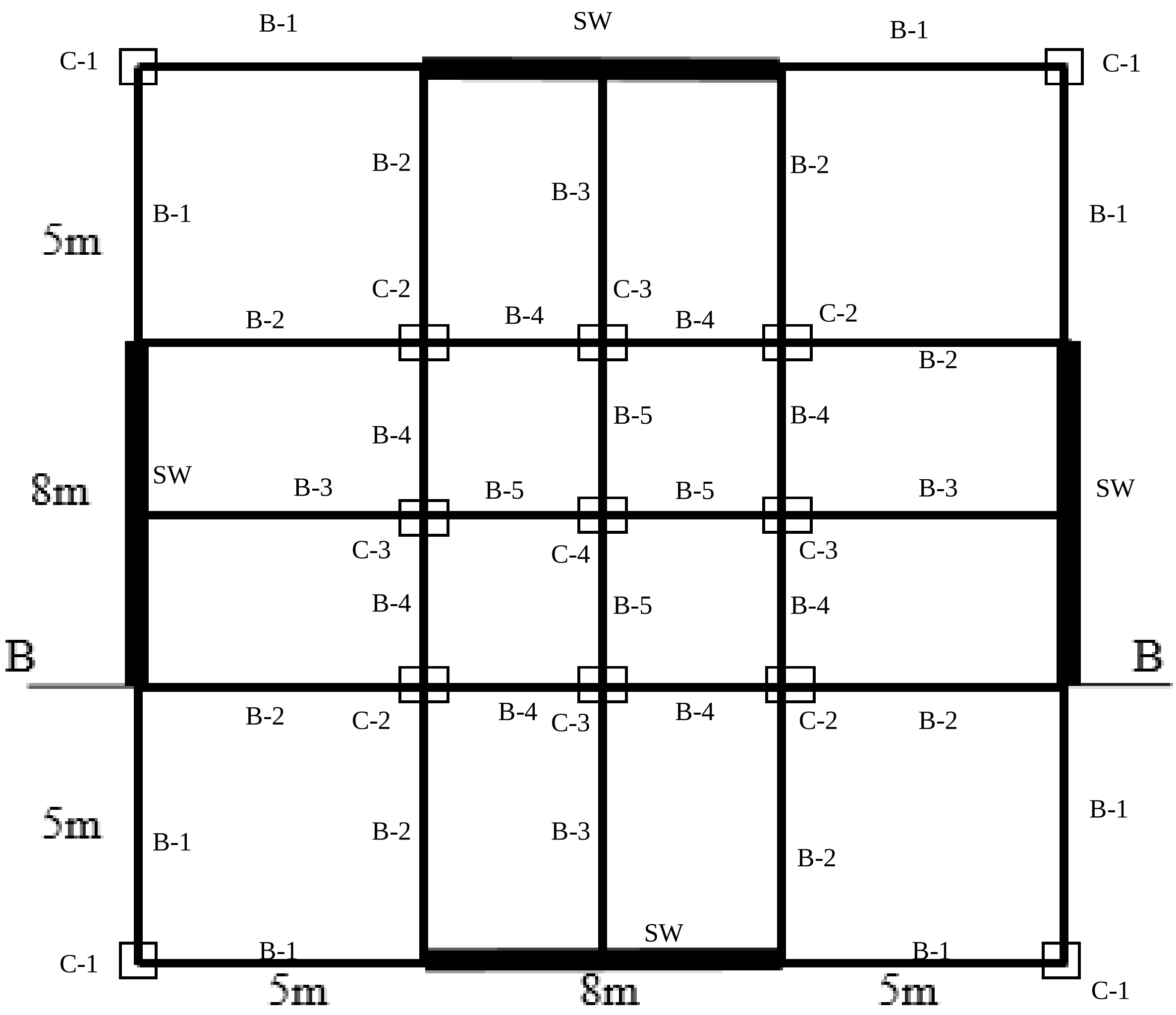
Model 2: Design results of Secondary columns												
	C-1			C-2			C-3			C-4		
Storey	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.	b (mm)	h (mm)	Reinf.
9	400	400	4φ12	400	400	4φ12	400	400	4φ12	400	400	4φ12
8	400	400	4φ12	400	400	4φ12	400	400	4φ12	400	400	4φ12
7	400	400	4φ12	400	400	4φ12	400	400	4φ12	400	400	4φ12
6	500	500	8φ12	500	500	8φ12	500	500	8φ12	500	500	8φ12
5	500	500	8φ12	500	500	8φ12	500	500	8φ12	500	500	8φ12
4	500	500	8φ12	500	500	8φ14	500	500	8φ12	500	500	8φ12
3	500	500	8φ12	600	600	8φ12	600	600	8φ12	600	600	8φ12
2	500	500	8φ12	600	600	8φ12	600	600	8φ12	600	600	8φ12
1	500	500	8φ12	650	650	12φ12	600	600	8φ12	600	600	8φ12

D.2.3 Design result for shear walls (length = 6m, height = 3m)

Model 2: Design result of Shear wall for fully detailed case				
storey	b (mm)	Vertical	Horizontal	Med/Mrd
9	200	φ12 c/c 250 mm	φ8 c/c 250 mm	0.9
8	200	φ12 c/c 250 mm	φ8 c/c 250 mm	0.89
7	200	φ12 c/c 200 mm	φ8 c/c 250 mm	0.91
6	200	φ12 c/c 160 mm	φ8 c/c 250 mm	0.93
5	200	φ14 c/c 160 mm	φ8 c/c 250 mm	0.93
4	200	φ16 c/c 200 mm	φ8 c/c 250 mm	0.94
3	200	φ16 c/c 170 mm	φ8 c/c 250 mm	0.98
2	200	φ16 c/c 140 mm	φ8 c/c 250 mm	0.97
1	200	φ16 c/c 110 mm	φ8 c/c 250 mm	0.98

Model 2: Design result of Shear wall for partially detailed case				
storey	b (mm)	Vertical	Horizontal	Med/Mrd
9	200	φ14 c/c 200 mm	φ8 c/c 250 mm	0.9
8	200	φ14 c/c 200 mm	φ8 c/c 250 mm	0.96
7	200	φ14 c/c 160 mm	φ8 c/c 250 mm	0.96
6	200	φ16 c/c 160 mm	φ8 c/c 250 mm	0.94
5	200	φ16 c/c 120 mm	φ8 c/c 250 mm	0.96
4	200	φ20 c/c 200 mm	φ8 c/c 250 mm	0.95
3	200	φ20 c/c 170 mm	φ8 c/c 250 mm	0.96
2	200	φ20 c/c 160 mm	φ8 c/c 250 mm	0.97
1	200	φ20 c/c 130 mm	φ8 c/c 250 mm	0.98

D.3 Design results for 13 storey building (model 3 on category 1)



Plan view of Model 3

D.3.1 Design result for beams (Section: 400X400 mm)

Model 3 , Design result of primary			
Storey	Location	Reinforcement	
		Top	Bottom
B-1			
13	Left	4ϕ20	3ϕ14
	Right	4ϕ20	3ϕ14
12	Left	8ϕ18	5ϕ14
	Right	7ϕ18	4ϕ14
11	Left	7ϕ20	4ϕ16
	Right	7ϕ18	4ϕ14
9,10	Left	9ϕ18	6ϕ14
	Right	7ϕ18	4ϕ14

Model 3 , Design result of Secondary Beams			
Storey	Location	Reinforcement	
		Top	Bottom
B-1			
13	Left	2ϕ18	2ϕ12
	Right(M)	2ϕ12	8ϕ12
6,7,8,9,10, 11,12	Left	5ϕ14	2ϕ12
	Right(M)	3ϕ16	8ϕ16
4,5	Left	3ϕ18	2ϕ12
	Right(M)	3ϕ16	8ϕ16
1,2,3	Left	6ϕ12	2ϕ12
	Right(M)	3ϕ16	8ϕ16

Model 3 , Design result of primary			
Storey	Location	Reinforcement	
		Top	Bottom
B-1			
6,7,8	Left	8ϕ20	7ϕ14
	Right	7ϕ18	4ϕ14
5	Left	9ϕ18	6ϕ14
	Right	8ϕ16	5ϕ12
4	Left	7ϕ20	4ϕ16
	Right	5ϕ20	5ϕ12
3	Left	8ϕ18	5ϕ14
	Right	6ϕ18	5ϕ12
2	Left	7ϕ18	4ϕ14
	Right	5ϕ18	4ϕ12
1	Left	5ϕ20	5ϕ12
	Right	6ϕ16	3ϕ12
B-2			
13	Left	5ϕ16	3ϕ12
	Right	7ϕ14	3ϕ12
12	Left	5ϕ18	2ϕ16
	Right	6ϕ18	2ϕ18
11	Left	5ϕ18	2ϕ16
	Right	5ϕ20	5ϕ12
10	Left	4ϕ20	2ϕ16
	Right	7ϕ18	4ϕ14
8,9	Left	5ϕ18	2ϕ16
	Right	7ϕ18	4ϕ14
6,7	Left	5ϕ18	3ϕ12
	Right	6ϕ20	6ϕ12
5	Left	7ϕ14	3ϕ12
	Right	6ϕ20	6ϕ12
4	Left	4ϕ18	2ϕ14
	Right	7ϕ18	6ϕ12
2,3	Left	4ϕ18	2ϕ14
	Right	6ϕ20	6ϕ12
1	Left	6ϕ14	2ϕ12
	Right	8ϕ18	5ϕ14
B-3			
13	Left	6ϕ18	5ϕ12
	Right	6ϕ14	2ϕ14
12	Left	6ϕ20	6ϕ12
	Right	5ϕ18	2ϕ16
10,11	Left	6ϕ20	6ϕ12
	Right	7ϕ16	3ϕ14
9	Left	6ϕ20	6ϕ12
	Right	6ϕ18	2ϕ18
8	Left	7ϕ18	4ϕ14
	Right	6ϕ18	5ϕ12
6,7	Left	7ϕ18	4ϕ14
	Right	5ϕ20	5ϕ12
5	Left	8ϕ16	5ϕ12
	Right	5ϕ20	5ϕ12
4	Left	5ϕ20	5ϕ12
	Right	5ϕ20	5ϕ12

Model 3 , Design result of Secondary Beams			
Storey	Location	Reinforcement	
		Top	Bottom
B-2			
13	Left(M)	2ϕ16	7ϕ16
	Right	6ϕ16	2ϕ12
9,10, 11,12	Left(M)	2ϕ18	7ϕ18
	Right	6ϕ18	2ϕ16
7,8	Left(M)	2ϕ18	7ϕ18
	Right	5ϕ20	2ϕ12
6	Left(M)	2ϕ18	7ϕ18
	Right	8ϕ16	2ϕ16
2,3,4,5	Left(M)	2ϕ18	7ϕ18
	Right	7ϕ18	2ϕ18
1	Left(M)	3ϕ16	8ϕ16
	Right	8ϕ16	3ϕ16
B-3			
13	Left(M)	2ϕ14	5ϕ18
	Right	5ϕ16	2ϕ12
9,10, 11,12	Left(M)	3ϕ16	8ϕ16
	Right	7ϕ16	2ϕ16
5,6,7,8	Left(M)	3ϕ16	8ϕ16
	Right	6ϕ18	2ϕ18
1,2,3,4	Left(M)	2ϕ16	6ϕ18
	Right	8ϕ16	3ϕ16
B-4			
13	Left	4ϕ16	5ϕ14
	Right	6ϕ14	2ϕ12
12	Left	7ϕ14	2ϕ12
	Right	5ϕ16	2ϕ12
9,10,11	Left	6ϕ16	2ϕ12
	Right	6ϕ14	2ϕ12
8	Left	5ϕ18	2ϕ14
	Right	6ϕ14	2ϕ12
7	Left	5ϕ18	2ϕ14
	Right	4ϕ16	2ϕ12
5,6	Left	7ϕ16	2ϕ16
	Right	5ϕ14	2ϕ12
4	Left	7ϕ16	2ϕ16
	Right	6ϕ12	2ϕ12
3	Left	6ϕ18	2ϕ16
	Right	6ϕ12	2ϕ12
2	Left	6ϕ18	2ϕ16
	Right	4ϕ14	2ϕ12
1	Left	6ϕ18	2ϕ16
	Right	5ϕ12	2ϕ12
B-5			
13	Left	4ϕ16	2ϕ12
	Right	4ϕ16	2ϕ12
10,11,12	Left	6ϕ16	2ϕ12
	Right	6ϕ14	2ϕ12
9	Left	6ϕ16	2ϕ12
	Right	4ϕ16	2ϕ12

Model 3 , Design result of primary			
Storey	Location	Reinforcement	
		Top	Bottom
B-3			
3	Left	7ϕ18	4ϕ14
	Right	6ϕ18	5ϕ12
2	Left	7ϕ16	4ϕ12
	Right	7ϕ16	3ϕ14
1	Left	7ϕ16	4ϕ12
	Right	6ϕ18	2ϕ18
B-4			
13	Left	6ϕ12	2ϕ12
	Right	6ϕ14	3ϕ14
12	Left	6ϕ14	2ϕ14
	Right	6ϕ16	4ϕ12
11	Left	6ϕ14	2ϕ14
	Right	5ϕ18	3ϕ14
9,10	Left	5ϕ16	2ϕ16
	Right	7ϕ16	2ϕ18
7,8	Left	6ϕ16	2ϕ16
	Right	7ϕ16	5ϕ12
5,6	Left	6ϕ12	3ϕ14
	Right	7ϕ16	2ϕ18
4	Left	7ϕ14	2ϕ16
	Right	5ϕ18	3ϕ14
3	Left	5ϕ16	3ϕ12
	Right	6ϕ16	4ϕ12
2	Left	5ϕ16	3ϕ12
	Right	5ϕ16	2ϕ16
1	Left	6ϕ14	2ϕ14
	Right	6ϕ14	2ϕ14
B-5			
13	Left	6ϕ12	2ϕ12
	Right	4ϕ16	2ϕ14
12	Left	5ϕ16	2ϕ14
	Right	6ϕ16	3ϕ14
11	Left	7ϕ14	4ϕ12
	Right	6ϕ16	4ϕ12
9,10	Left	6ϕ16	4ϕ12
	Right	5ϕ18	3ϕ14
7,8	Left	6ϕ16	3ϕ14
	Right	7ϕ16	2ϕ18
5,6	Left	6ϕ16	4ϕ12
	Right	5ϕ18	4ϕ12
4	Left	6ϕ16	4ϕ12
	Right	6ϕ16	4ϕ12
3	Left	7ϕ14	2ϕ16
	Right	6ϕ16	4ϕ12
2	Left	5ϕ16	3ϕ12
	Right	5ϕ16	3ϕ12
1	Left	6ϕ14	2ϕ14
	Right	6ϕ14	2ϕ14

Model 3 , Design result of Secondary Beams			
Storey	Location	Reinforcement	
		Top	Bottom
B-5			
8	Left	5ϕ18	2ϕ14
	Right	4ϕ16	2ϕ12
7	Left	5ϕ18	2ϕ14
	Right	5ϕ14	2ϕ12
6	Left	7ϕ16	2ϕ14
	Right	5ϕ14	2ϕ12
4,5	Left	7ϕ16	2ϕ14
	Right	6ϕ12	2ϕ12
2,3	Left	6ϕ18	2ϕ16
	Right	4ϕ14	2ϕ12
1	Left	6ϕ18	2ϕ16
	Right	5ϕ12	2ϕ12

D.3.2 Design result for Columns

Model 3: Design results of primary columns						
Storey	Cross sections		Reinforcement			
	b (mm)	h (mm)	C-1	C-2	C-3	C-4
13	400	400	20φ14	12φ14	12φ14	12φ14
12	400	400	12φ14	12φ14	12φ14	12φ14
11	500	500	8φ20	8φ20	8φ20	8φ20
10	500	500	8φ20	8φ20	8φ20	8φ20
9	500	500	8φ20	8φ20	8φ20	8φ20
8	600	600	12φ20	12φ20	12φ20	12φ20
7	600	600	12φ20	12φ20	12φ20	12φ20
6	600	600	12φ20	12φ20	12φ20	12φ20
5	700	700	16φ20	16φ20	16φ20	16φ20
4	700	700	16φ20	16φ20	16φ20	16φ20
3	700	700	16φ20	20φ20	16φ20	16φ20
2	700	700	16φ20	24φ20	16φ20	16φ20
1	700	700	16φ20	24φ25	24φ20	20φ20

Model 3: Design results of secondary columns						
Storey	Cross sections		Reinforcement			
	b (mm)	h (mm)	C-1	C-2	C-3	C-4
13	400	400	4φ12	4φ12	4φ12	4φ12
12	400	400	4φ12	4φ12	4φ12	4φ12
11	500	500	8φ12	8φ12	8φ12	8φ12
10	500	500	8φ12	8φ12	8φ12	8φ12
9	500	500	8φ12	12φ12	8φ12	8φ12
8	600	600	8φ12	8φ12	8φ12	8φ12
7	600	600	8φ12	12φ12	8φ12	8φ12
6	600	600	8φ12	16φ20	12φ12	8φ12
5	700	700	12φ12	12φ12	12φ12	12φ12
4	700	700	12φ12	16φ20	12φ12	12φ12
3	700	700	12φ12	20φ20	16φ16	12φ12
2	700	700	12φ12	24φ25	16φ20	12φ12
1	700	700	12φ12	24φ25	20φ20	16φ12

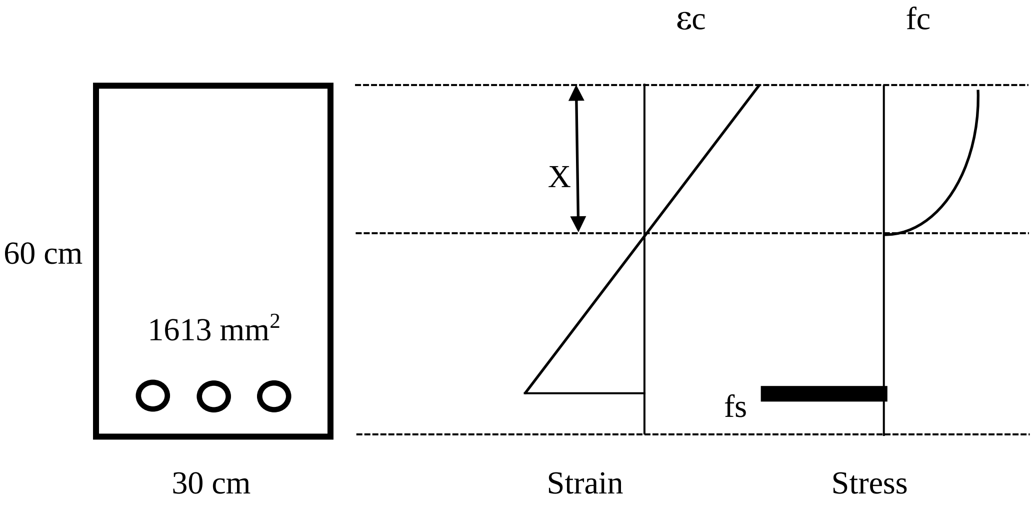
D.3.3 Design result for shear walls (length = 8m, height = 3m)

Model 3: Design result of Shear wall for fully detailed case				
storey	b	Vertical	Horizontal	Med/Mrd
13	250	φ12 c/c 300 mm	φ8 c/c 200 mm	0.83
12	250	φ12 c/c 300 mm	φ8 c/c 200 mm	0.76
11	250	φ12 c/c 300 mm	φ8 c/c 200 mm	0.76
10	250	φ12 c/c 300 mm	φ8 c/c 200 mm	0.81
9	250	φ12 c/c 250 mm	φ8 c/c 200 mm	0.88
8	250	φ14 c/c 250 mm	φ8 c/c 200 mm	0.89
7	250	φ14 c/c 180 mm	φ8 c/c 200 mm	0.94
6	250	φ16 c/c 180 mm	φ8 c/c 200 mm	0.89
5	250	φ16 c/c 150 mm	φ8 c/c 200 mm	0.95
4	250	φ20 c/c 200 mm	φ8 c/c 200 mm	0.99
3	250	φ20 c/c 160 mm	φ8 c/c 200 mm	0.98
2	250	φ20 c/c 140 mm	φ8 c/c 200 mm	0.96
1	250	φ20 c/c 120 mm	φ8 c/c 200 mm	0.96

Model 3: Design result of Shear wall for partially detailed case				
storey	b	Vertical	Horizontal	Med/Mrd
13	250	φ14 c/c 200 mm	φ8 c/c 200 mm	0.91
12	250	φ14 c/c 200 mm	φ8 c/c 200 mm	0.91
11	250	φ14 c/c 200 mm	φ8 c/c 200 mm	0.96
10	250	φ16 c/c 200 mm	φ8 c/c 200 mm	0.92
9	250	φ16 c/c 180 mm	φ8 c/c 200 mm	1
8	250	φ20 c/c 200 mm	φ8 c/c 200 mm	0.92
7	250	φ20 c/c 160 mm	φ8 c/c 200 mm	0.95
6	250	φ20 c/c 140 mm	φ8 c/c 200 mm	0.99
5	250	φ20 c/c 130 mm	φ8 c/c 200 mm	1
4	250	φ25 c/c 180 mm	φ8 c/c 200 mm	0.95
3	250	φ25 c/c 160 mm	φ8 c/c 200 mm	0.98
2	250	φ25 c/c 150 mm	φ8 c/c 200 mm	0.99
1	250	φ25 c/c 130 mm	φ8 c/c 200 mm	0.95

Appendix E: Moment curvature verification

- Beam section taken from “Reinforced concrete mechanics and design", by James K. Wight and James G. Macgregor, six edition,



- **Material properties**

  Concrete fc’=27.59 MPa (C-35)

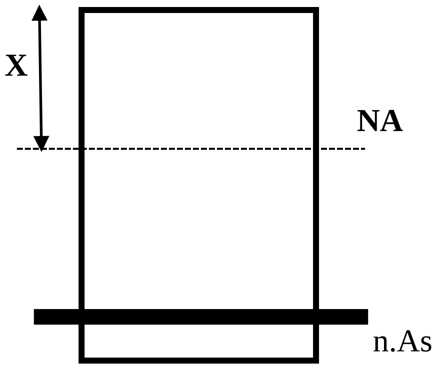
  Reinforcement fy = 413.685 MPa

- **Manual Calculation**

$$n = \frac{Es}{Ec} = \frac{200}{25.02} = 8$$

$$Ec = 15210\sqrt{fc'} = 255076.56\, kg/cm^2 = 25.023\, Gpa$$
$$fc' = 281.2436\, kg/cm^2$$

✓ **Compute Neutral Axis depth (NA)**



$$\sum M_{N-A} = 0$$

$$bx^2/2 = n.As.(d-x)$$

$$d = 540mm$$

$$\Rightarrow x = 176.77mm$$

- **Compute Icr, Ig, fr, and Ig**

$$Icr = Iconc + Isteel = \frac{1}{3}bx^3 + n.As.(d - x)^2 = 2254867cm^4$$

$$fr = 2.\sqrt{fc'} = 2\sqrt{281.2436} = 33.541kg / cm^2 = 3.29N / mm^2$$

$$Ig = \frac{bh^3}{12} = \frac{300*600^3}{12} = 5.4*10^9 mm^4$$

- **Compute Mcr and ϕcr**

$$Mcr = \frac{fr.Ig}{y_t} = \frac{3.29*5.4*10^9}{600-176.77} = 41.98kNm$$

$$\phi cr = \frac{Mcr}{Ec.Ig} = 3.1068*10^{-4} / m$$

- **Compute ϕy and My**

$$\phi y = \frac{\varepsilon y}{d - x} = \frac{(fy / Es)}{d - x} = 5.6945*10^{-3} / m$$

$$My = As.f_{yd}.d.(1 - k / 3)$$

$$k = \sqrt{(n.\rho)^2 + 2n\rho} - n\rho$$

$$\rho = As / bd$$

$$My = As.f_{yd}.d.(1 - k / 3) = 321.01kNm$$

- **Compute ϕu and Mu**

$$\phi u = \frac{\varepsilon u}{C} = 0.031367$$

$$\varepsilon u = 00035$$

$$C = \frac{a}{\beta_1} = 111.58$$

$$a = \frac{As.fyd}{0.85.fc'.b} = 94.845mm$$

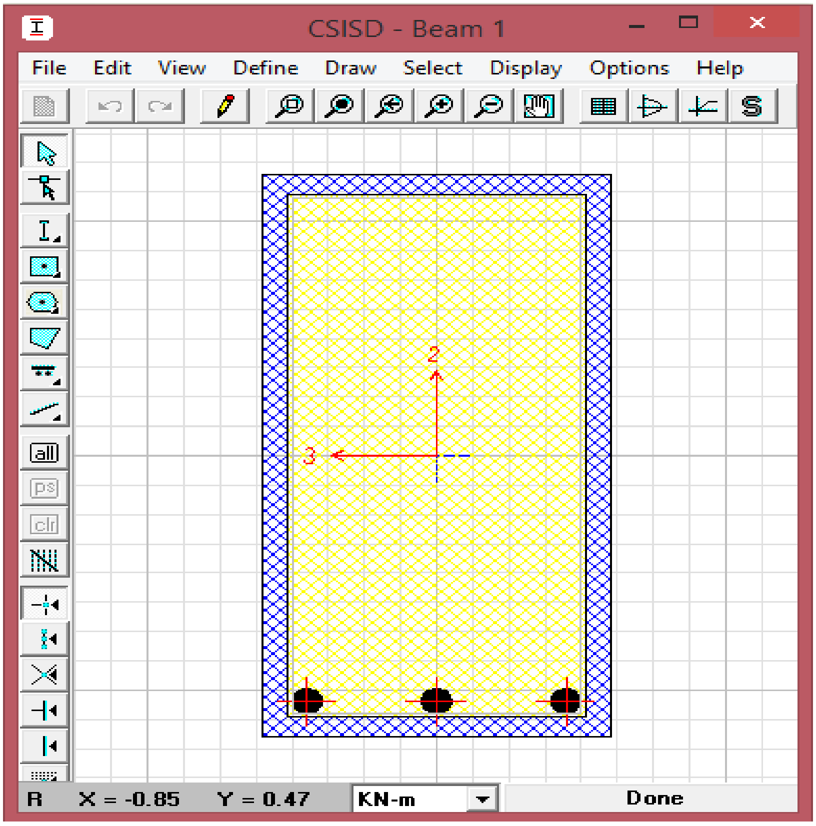
$$\beta_1 = 0.85$$

$$Mu = As.fy.(d - a / 2) = 328.68$$

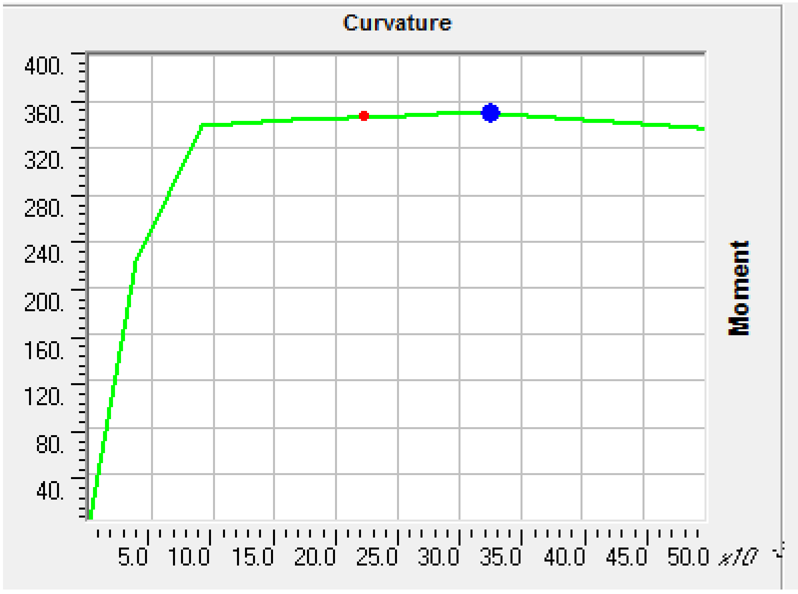
- **Summary**

	Φ (1/m)	M (kNm)
Crack	$3.1068*10^{-4}$	41.98
Yield	$5.6945*10^{-3}$	321.01
Ultimate	$3.1367-10^{-2}$	328.68

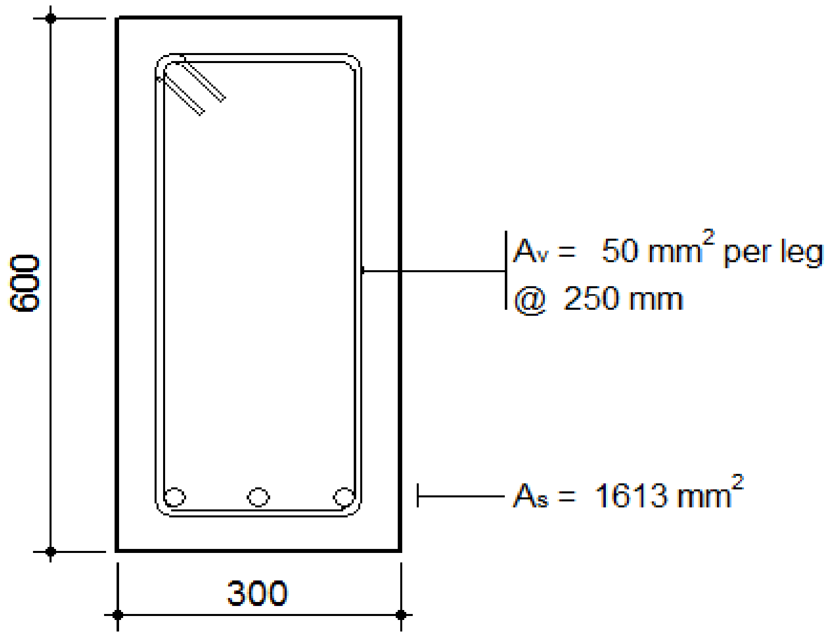
- **Moment curvature relation from SAP2000 section designer**

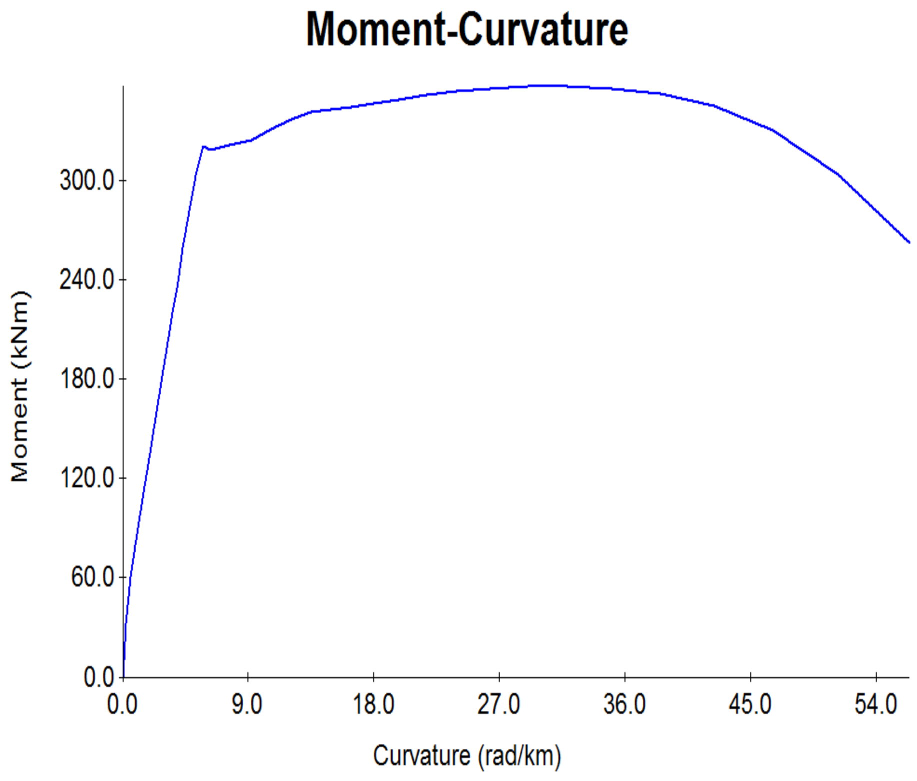


Sap 2000	
Curvature. (1/m)	Moment (kNm)
0	0
0.0007656	46.9855
0.001914	117.4222
0.003445	210.0495
0.005359	322.4493
0.007656	337.2006
0.0103	341.032
0.0134	343.6453
0.0168	345.4126
0.0207	346.3843
0.0249	348.2841
0.0295	350.6031
0.032	351.7131
0.0398	348.335
0.0456	341.8451
0.0517	337.198
0.0582	333.8652



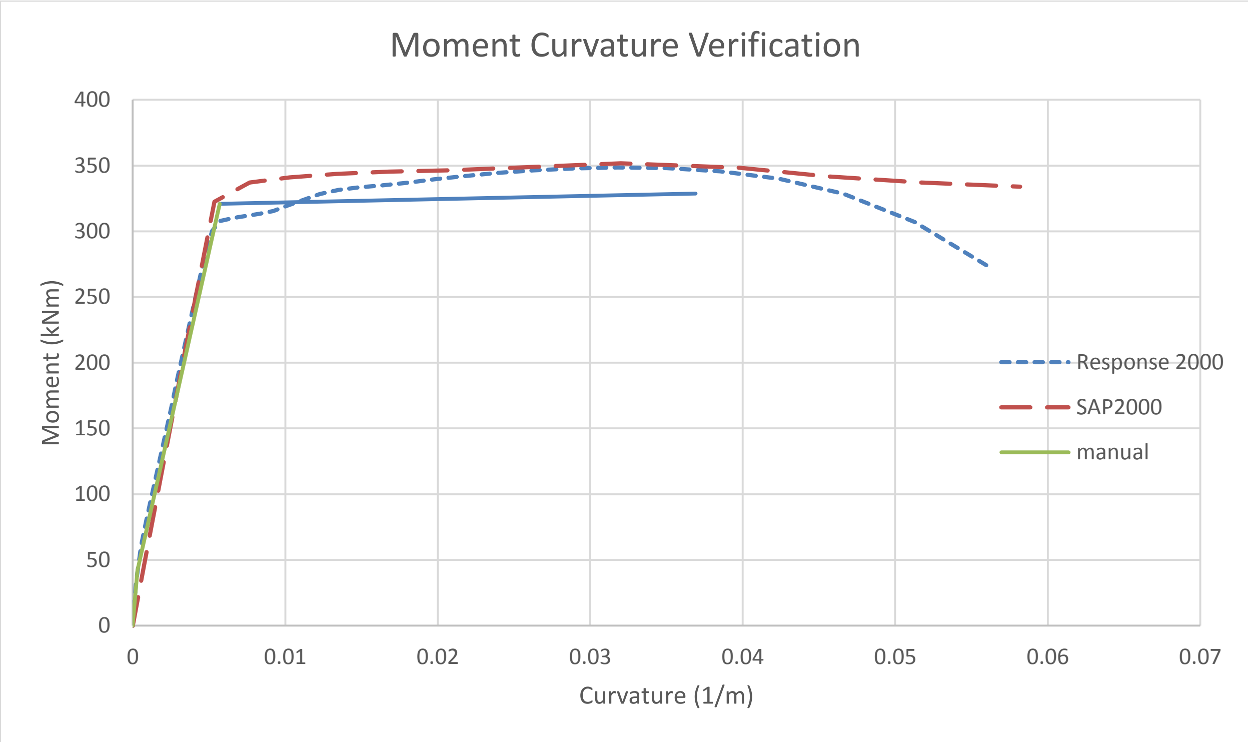
- **Moment curvature relation from Response 2000**





Response 2000	
Curvature. (1/m)	Moment (kNm)
0	0
0.000221	32.359
0.000554	60.874
0.000887	79.782
0.001221	97.551
0.001554	115.127
0.001887	132.704
0.002221	150.278
0.002554	167.878
0.002887	185.466
0.003221	203.045
0.003554	220.576
0.003909	239.238
0.0043	259.609
0.00473	281.893
0.005203	300.293
0.005724	307.797
0.006296	309.334
0.006926	310.754
0.007618	312.045
0.00838	313.363
0.009218	315.391
0.01014	319.529
0.011154	323.778
0.01227	328.284
0.013496	331.409
0.014846	333.435
0.016331	334.864
0.017964	336.893
0.01976	339.555
0.021736	342.054
0.02391	344.354
0.026301	346.295
0.028931	347.859
0.031824	348.561
0.035006	348.027
0.038507	345.546
0.042358	339.943
0.046593	328.574
0.051253	307.135
0.056378	271.394

- **Moment curvature comparison**

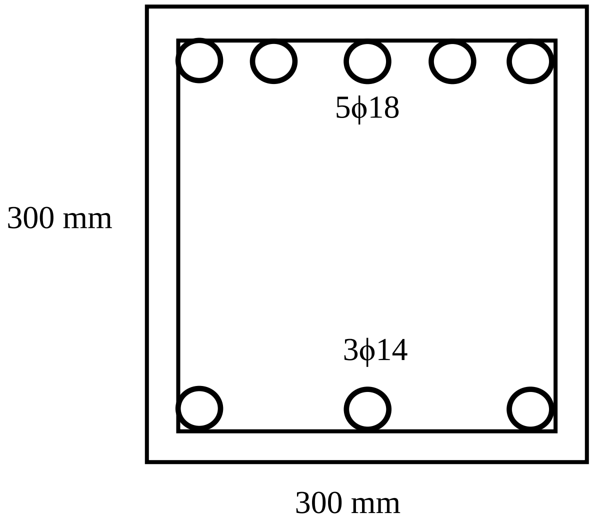


The result extracted from SAP2000 and Response 2000 are the same and slightly larger than results obtained from manual calculation.

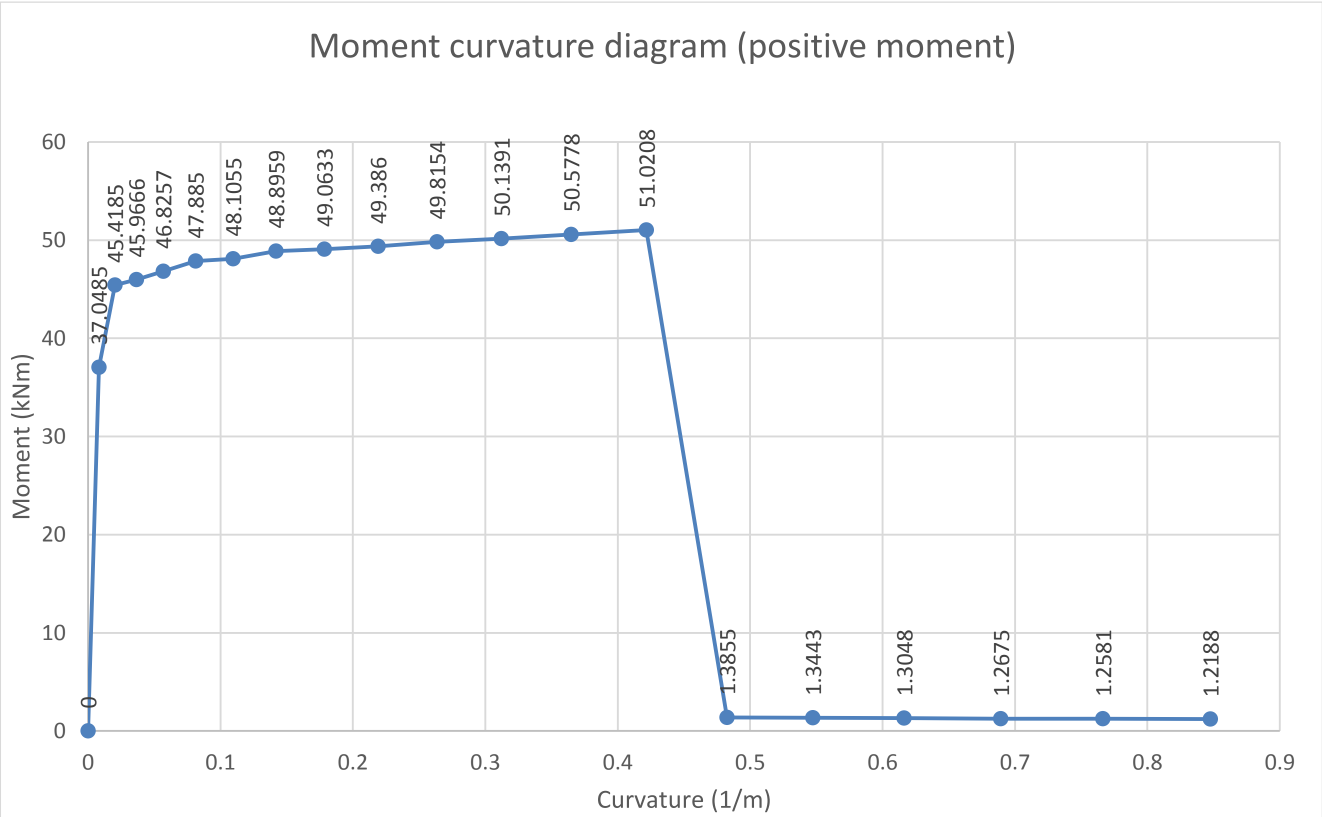
Appendix F: Hinge properties

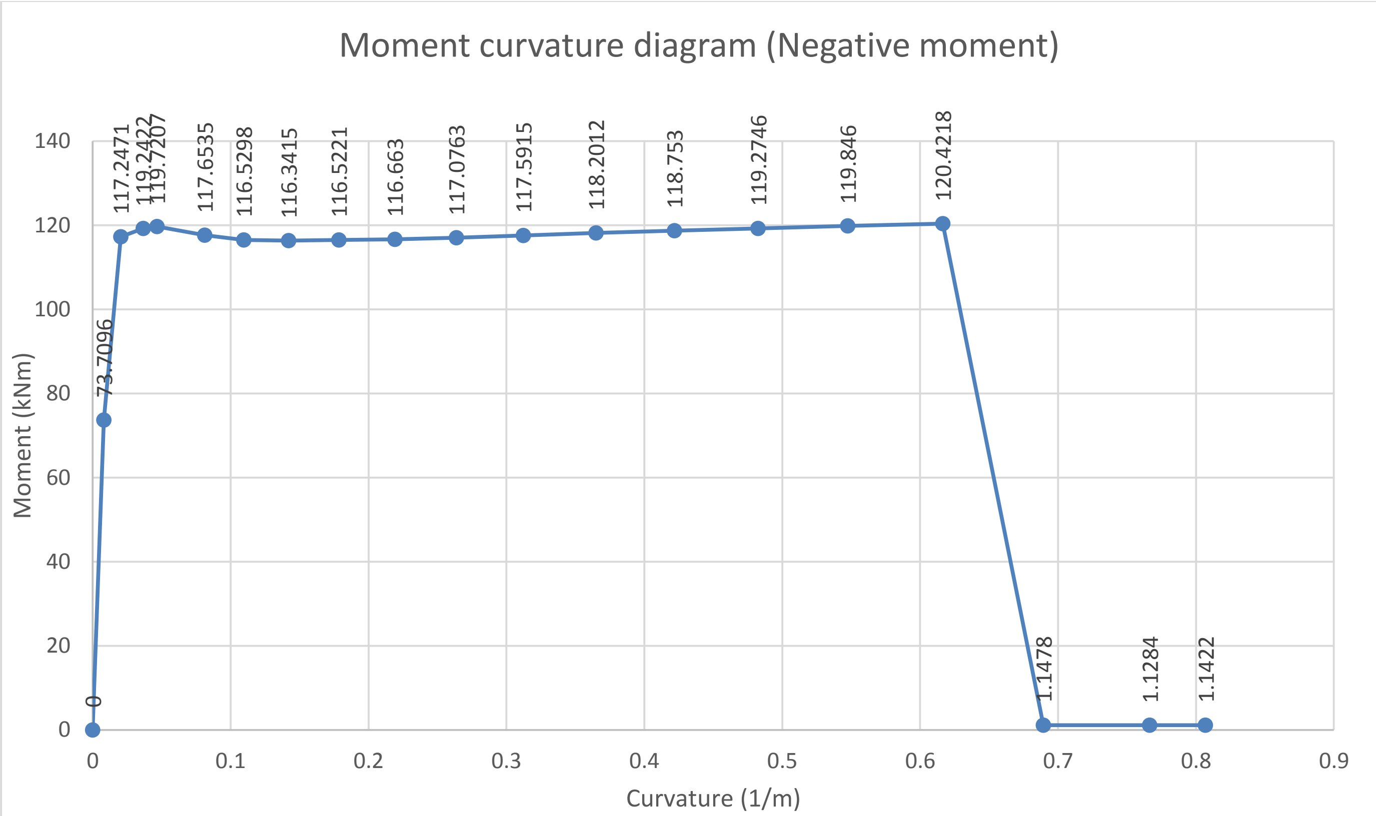
F.1 Hinge properties for typical beam

- **Beam section (Category 1: Model 1 fully detailed: B2-ST 1L)**



- **Moment curvature diagram**





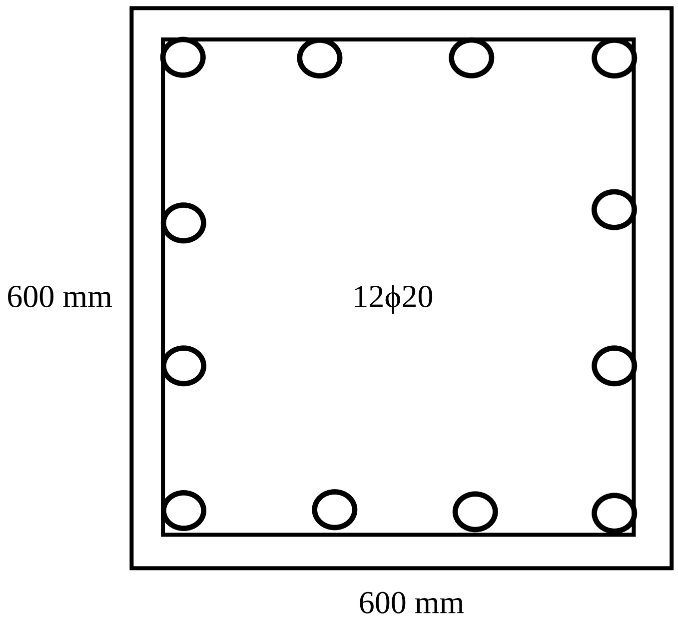
- **Assigned plastic hinge properties**

Point	Curvature(1/m)	Moment (kNm)	Moment/SF	Curvature/SF
E-	0.8069	1.1422	-0.010	-0.807
D-	0.6893	1.1478	-0.010	-0.689
C-	0.6163	120.4218	-1.027	-0.616
B-	0.0203	117.2471	-1.000	-0.020
A	0.000	0.000	0.000	0.000
B	0.0203	45.4185	1.000	0.020
C	0.4217	51.0208	1.123	0.422
D	0.4825	1.3855	0.031	0.483
E	0.8475	1.2188	0.027	0.848

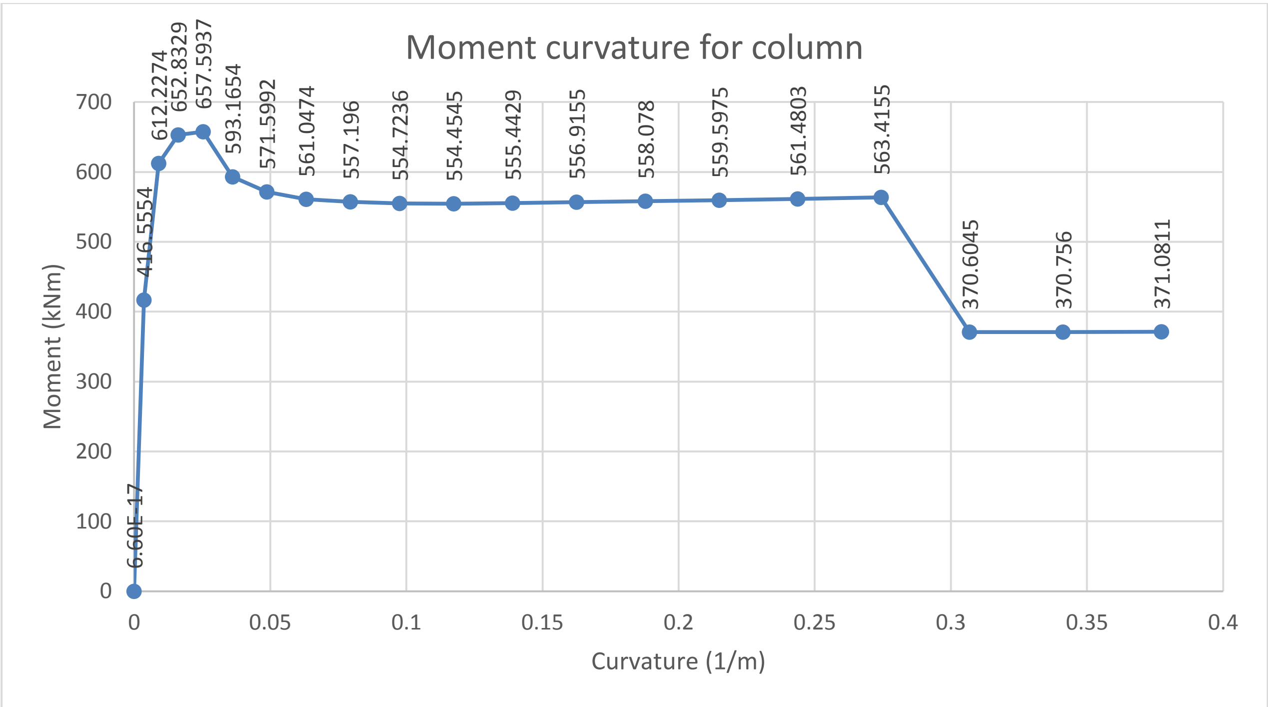
	Positive	Negative
Yield moment (Cf.)=	45.419	117.247
Yield Curvature (Cf.)=	1.000	1.000
Acceptance criteria (plastic deformation/SF)		
IO (10% ϕ ultimate)	0.042	-0.062
LS (40% ϕ ultimate)	0.169	-0.247
CP (80% ϕ ultimate)	0.337	-0.493

F.2 Hinge properties for typical column

- **Column section (Category 1: Model 3, fully detailed: C1-ST 8)**



- **Moment curvature diagram (For axial force = -1287 kN)**



- **Assigned plastic hinge properties**

Point	Curvature(1/m)	Moment(kNm)	Moment/SF	Curvature/SF
A	0.000	0.000	0.000	0.000
B	0.009028	563.4155	1.000	0.009
C	0.2744	563.4155	1.000	0.274
D	0.3069	370.6045	0.658	0.307
E	0.3774	371.0811	0.659	0.377
Yield moment (Cf)=		563.416		
Yield Curvature (Cf)=		1.000		
Acceptance criteria (plastic deformation/SF)				
IO		0.027		
LS		0.110		
CP		0.220		

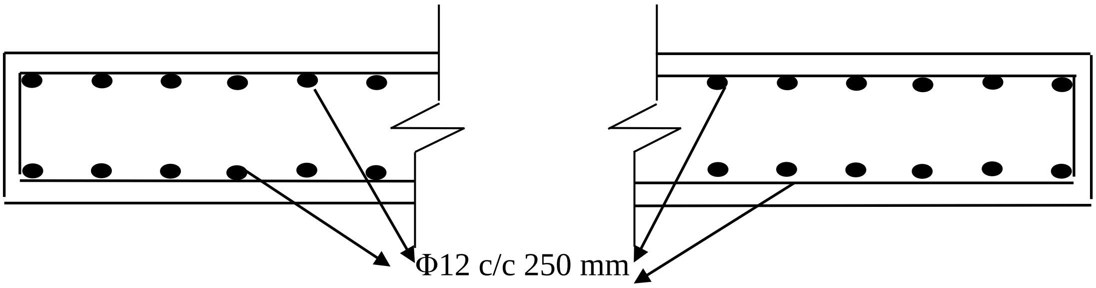
- **Interaction surface (PMM)**

Curve 1 (0. degrees)				Curve 2 (45. degrees)				Curve 3 (90. degrees)			
Point	P	M2	M3	Point	P	M2	M3	Point	P	M2	M3
1	-1.0000	0.0000	0	1	-1.0000	0.0000	0.0000	1	-1.0000	0	0.0000
2	-0.8619	0.3562	0	2	-0.9467	0.1124	0.1124	2	-0.8619	0	0.3562
3	-0.7619	0.5676	0	3	-0.8843	0.2362	0.2362	3	-0.7619	0	0.5676
4	-0.6513	0.7483	0	4	-0.7716	0.3940	0.3940	4	-0.6513	0	0.7483
5	-0.5322	0.8982	0	5	-0.6165	0.5376	0.5376	5	-0.5322	0	0.8982
6	-0.3997	1.0000	0	6	-0.4091	0.6222	0.6222	6	-0.3997	0	1.0000
7	-0.2934	0.9766	0	7	-0.2201	0.5995	0.5995	7	-0.2934	0	0.9766
8	-0.1904	0.8964	0	8	-0.0610	0.4862	0.4862	8	-0.1904	0	0.8964
9	-0.0789	0.7461	0	9	0.0745	0.3079	0.3079	9	-0.0789	0	0.7461
10	-0.0011	0.5819	0	10	0.1664	0.1320	0.1320	10	-0.0011	0	0.5819
11	0.2163	0.0000	0	11	0.2163	0.0000	0.0000	11	0.2163	0	0.0000

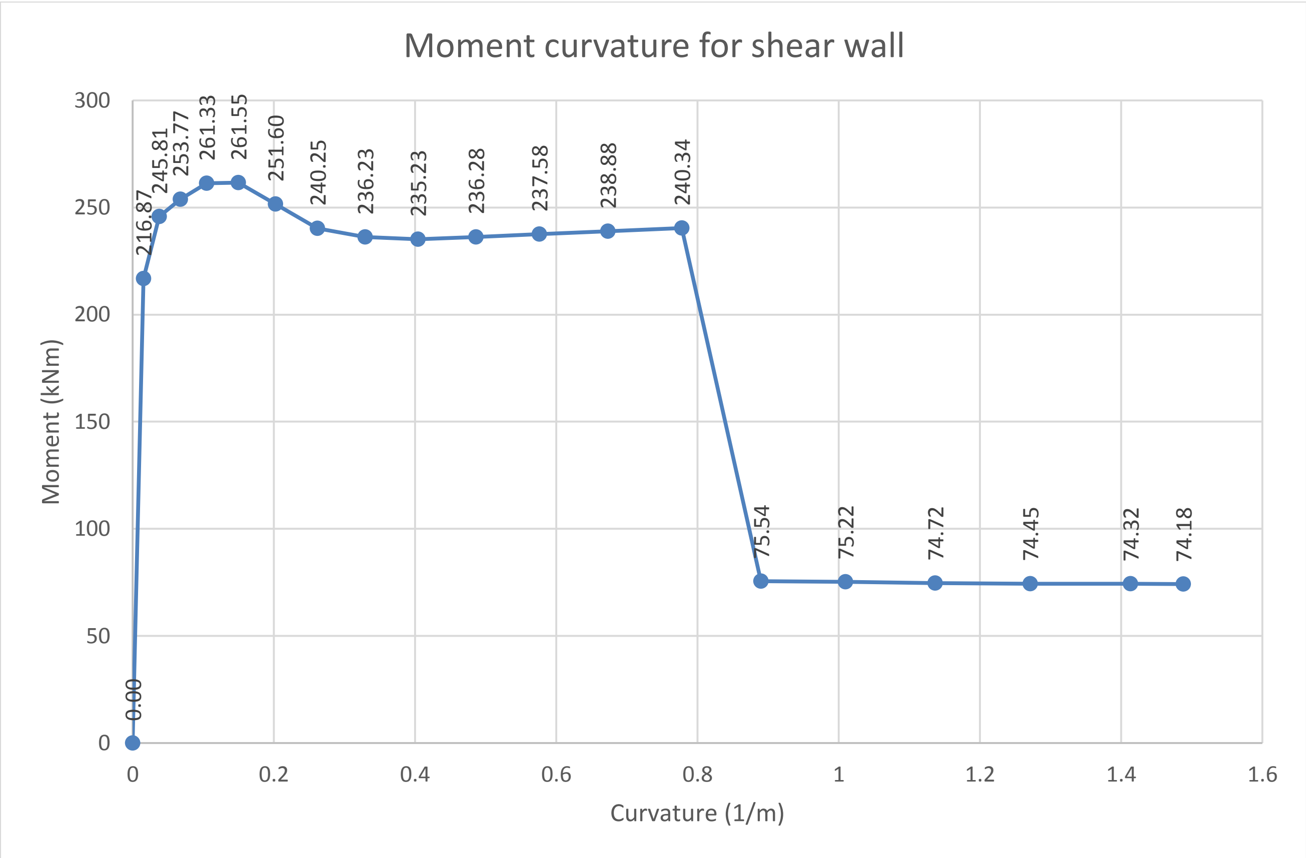
Scale factor for all curves		
P (kN)	M2 (kNm)	M3 (kNm)
6060.00	604.83	604.83

F.3 Hinge properties for typical shear wall

- **Shear wall section (Category 1: model 2, fully detailed: Storey 9 out of plane)**



- **Moment curvature diagram (For axial force = -807 kN)**



- **Assigned plastic hinge properties**

Point	Curvature(1/m)	Moment(kNm)	Moment/SF	Curvature/SF
A	0.000	0.000	0.000	0.000
B	0.0374	240.3363	1.000	0.037
C	0.7775	240.3363	1.000	0.778
D	0.8897	75.5411	0.314	0.890
E	1.4877	74.1841	0.309	1.488
Yield momentv(Cf)=		240.336		
Yield Curvature (Cf)=		1.000		
Acceptance citeria (plastic def/SF)				
IO		0.078		
LS		0.311		
CP		0.622		

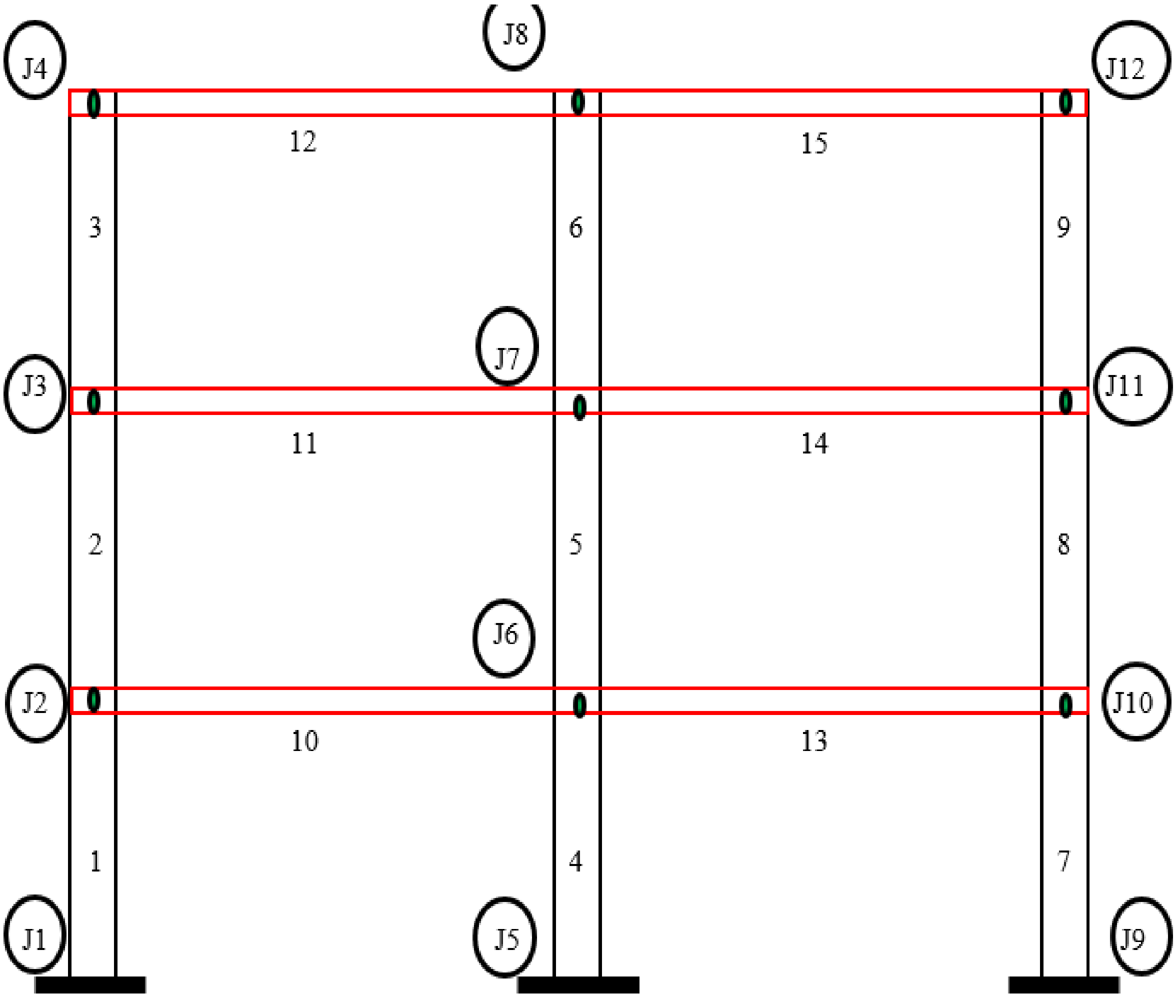
- **Interaction surface (PMM)**

Curve 1: 0. degrees				Curve 2: 45. degrees				Curve 3: 90. degrees			
Point	P	M2	M3	Point	P	M2	M3	Point	P	M2	M3
1	-1.0000	0.0000	0	1	-1.0000	0.0000	0.0000	1	-1.0000	0	0.0000
2	-0.8906	0.3513	0	2	-0.9027	0.3146	0.0192	2	-0.8675	0	0.3847
3	-0.7963	0.5976	0	3	-0.8050	0.5770	0.0199	3	-0.7548	0	0.6403
4	-0.6963	0.7883	0	4	-0.7015	0.7790	0.0204	4	-0.6337	0	0.8300
5	-0.5912	0.9222	0	5	-0.5928	0.9188	0.0216	5	-0.5051	0	0.9514
6	-0.4786	1.0000	0	6	-0.4765	0.9981	0.0222	6	-0.3713	0	1.0000
7	-0.3617	0.9935	0	7	-0.3561	0.9885	0.0226	7	-0.2966	0	0.9352
8	-0.2445	0.8911	0	8	-0.2358	0.8781	0.0221	8	-0.2140	0	0.8091
9	-0.1274	0.6938	0	9	-0.1155	0.6667	0.0225	9	-0.1096	0	0.5910
10	-0.0103	0.4001	0	10	0.0049	0.3543	0.0225	10	0.0352	0	0.2389
11	0.1099	0.0000	0	11	0.1099	0.0000	0.0000	11	0.1099	0	0.0000

Scale factor for all curves		
P (kN)	M2 (kNm)	M3 (kNm)
17890.00	14394.00	512.46

Appendix G: Pushover analysis verification

- **Model frame**



- **Assumptions**
  - ✓ Constant Axial Load on Columns for Analysis Steps
  - ✓ Rigid-plastic with no hardening or softening moment-rotation behavior for columns and beams
  - ✓ Plastic hinging occurs when moment capacity is within 5% tolerance
  - ✓ Load combinations 1.0 DL + 0.3 LL and 1.0 DL + 0.3 LL+1.0EQ to compute axial load levels

- **Data**
  - ✓ **Material properties**

    Steel (fyd=495 Mpa)

    Concrete (fcd=25 Mpa)

    Clear cover=5 cm

    E=2.779E+4 MPa

  - ✓ **Loading**

    DL = 15 kN/m for beams on the first and second storey

    DL = 10 kN/m for beans on the top storey

    LL = 2 kN/m for all beams

  - ✓ **Cross section and reinforcement**

    Beams: 250 X 500 mm, 3ϕ10 at the top and 3ϕ10 at the bottom

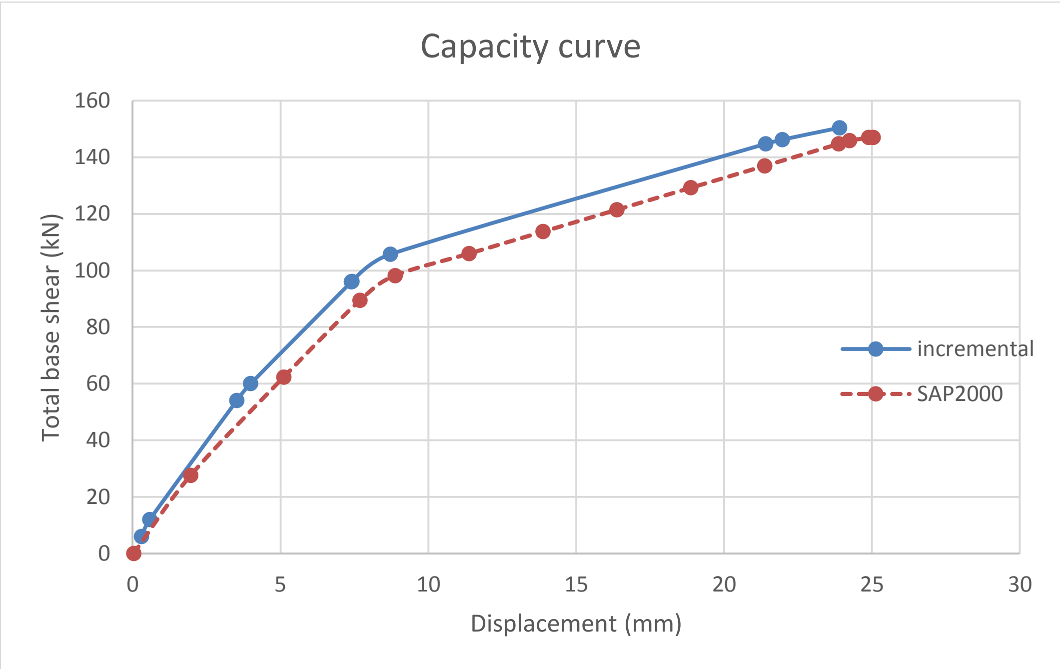
    Columns: 600 X 600 mm, 10 ϕ10

- **Section Capacities**

Idealized member moment curvature relations for estimated axial load level				
Member	N (kN)	My (kNm)	Φy (rad/m)	Φu (rad/m)
1	-83.786	124	0.0055	0.111
2	-51.347	115.5	0.0056	0.115
3	-19.872	107.5	0.0056	0.119
4	-253.392	166	0.0059	0.085
5	-158.905	143	0.0060	0.099
6	-64.797	119	0.0060	0.113
7	-124.104	133.5	0.0056	0.105
8	-77.747	122	0.0057	0.112
9	-31.201	110	0.0054	0.118
10	5.606	49	0.0073	0.103
11	1.421	50	0.0069	0.102
12	-17.233	53	0.0069	0.099
13	5.606	49	0.0073	0.103
14	1.421	50	0.0069	0.102
15	-17.233	53	0.0069	0.099

- **Capacity curves for incremental and SAP2000 Pushover analysis results**

pushover analysis results					
Incremental			SAP2000		
Step	Displacement (mm)	Base shear (kN)	Step	Displacement (mm)	Base shear (kN)
1	0.2947	6	0	0.036	0
2	0.5812	12	1	1.962	27.539
3	3.5212	54	2	5.11	62.295
4	3.9904	60	3	7.685	89.379
5	7.4004	96	4	8.875	98.185
6	7.41317	96.12	5	11.375	105.94
7	8.71317	105.72	6	13.875	113.694
8	21.40317	144.72	7	16.375	121.449
9	21.969	146.22	8	18.875	129.203
10	23.90917	150.42	9	21.375	136.958
			10	23.875	144.712
			11	24.236	145.832
			12	24.892	146.946
			13	25.036	147.018



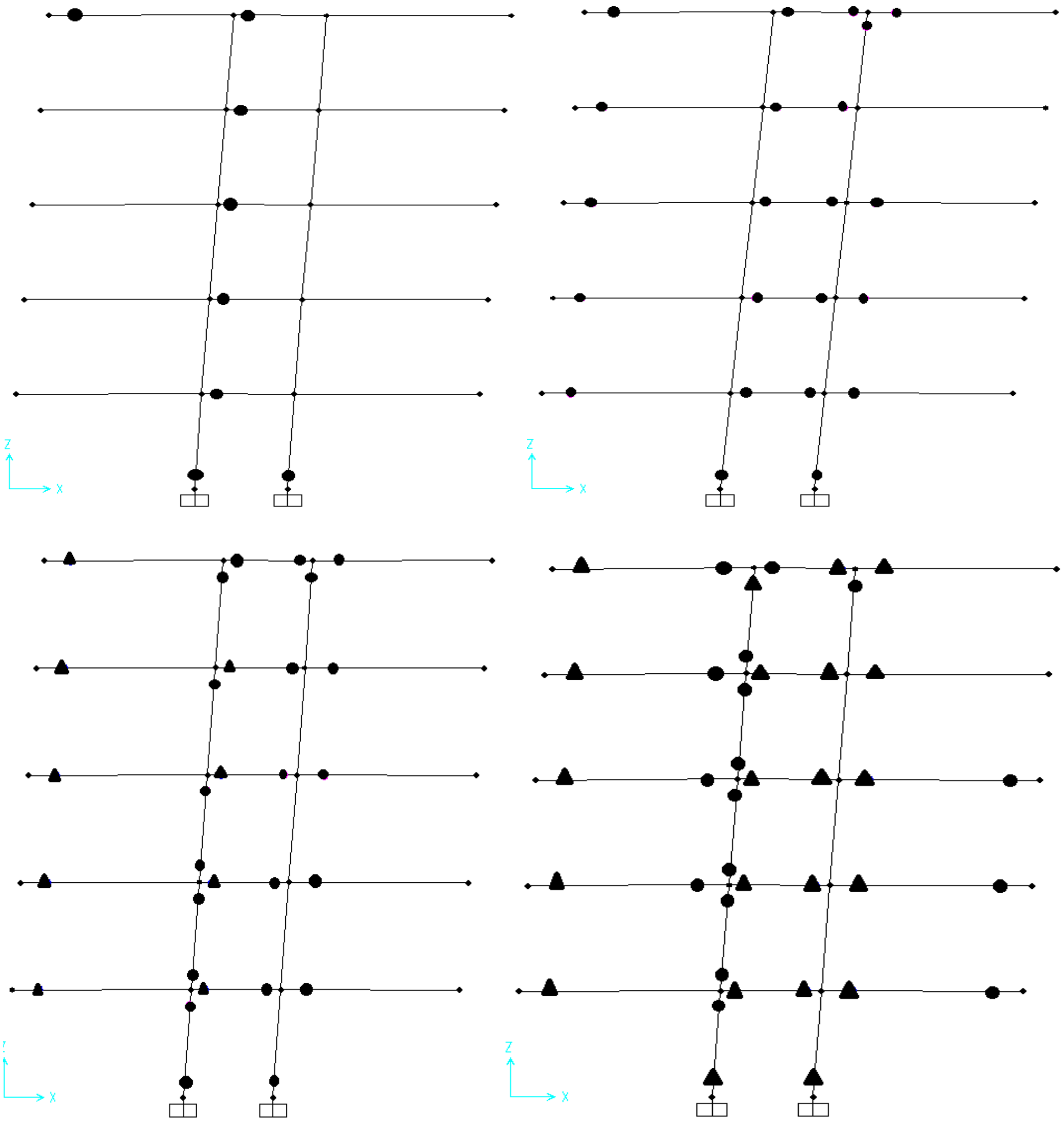
Appendix H: Plastic hinging patterns at different damage levels

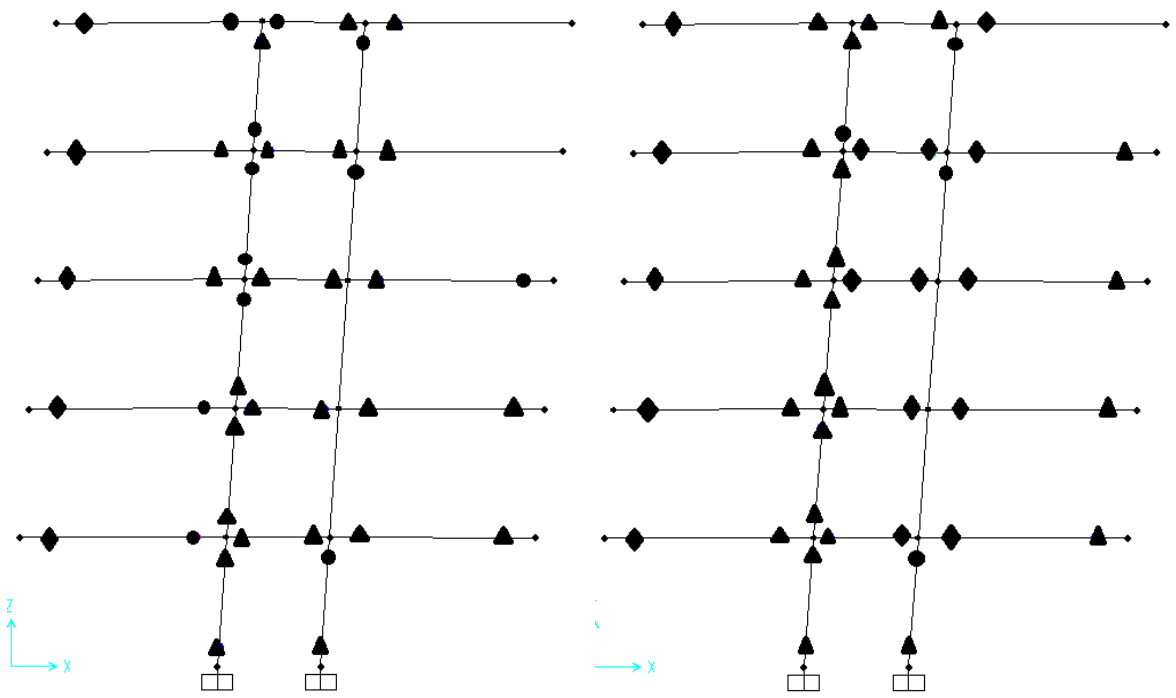
H.1: Symbol of Damage Level used to show the hinging pattern

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

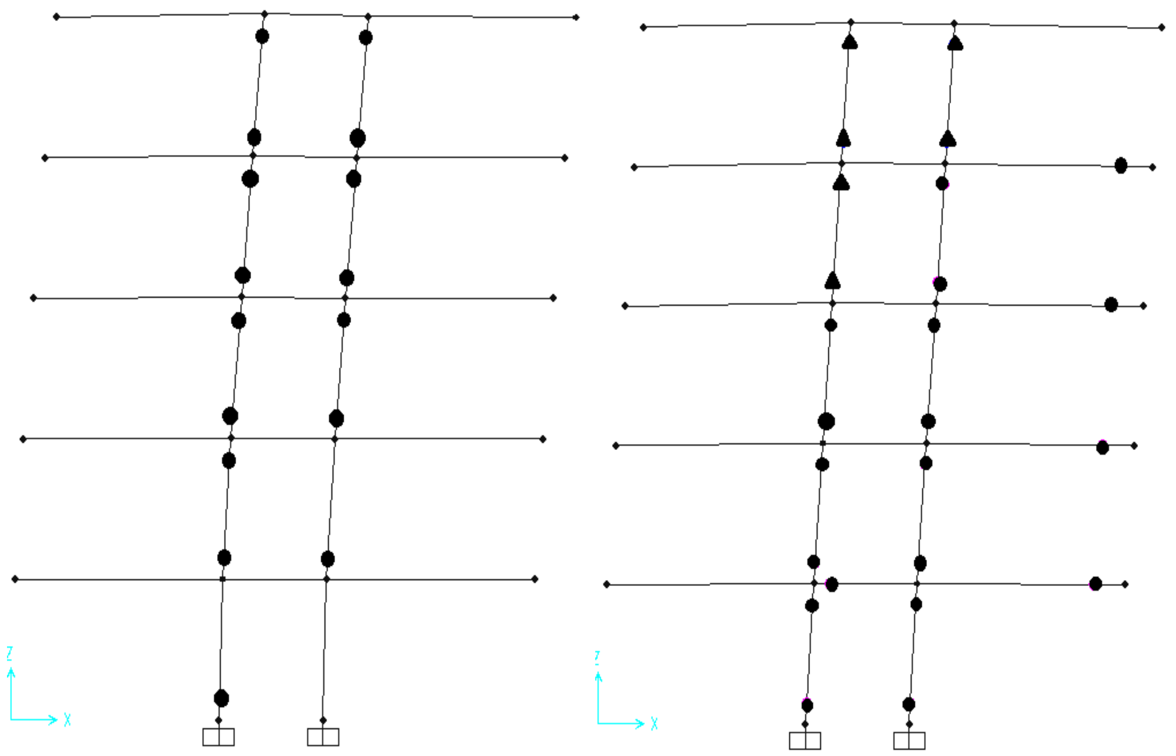
H.2: Five storey building (Category 1, Model 1)

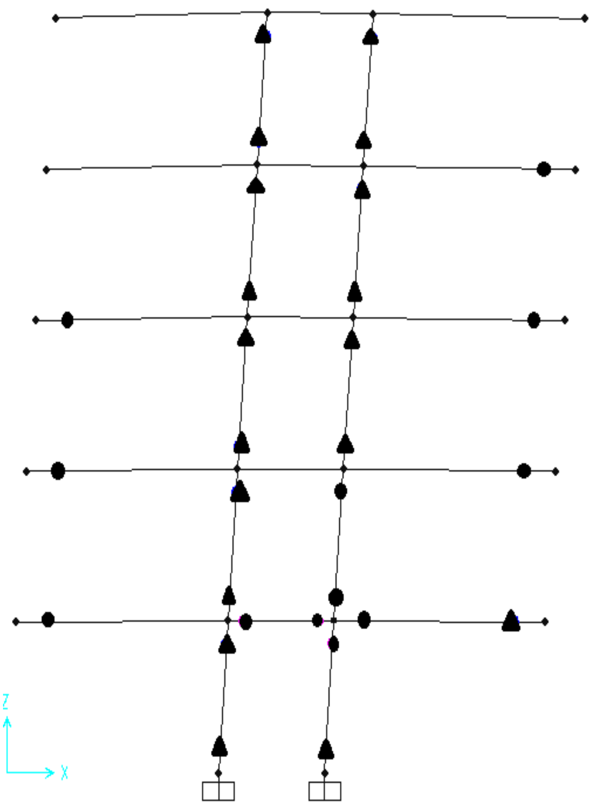
H.2.1: Hinge patterns of Case I (Fully detailed) for different displacements levels

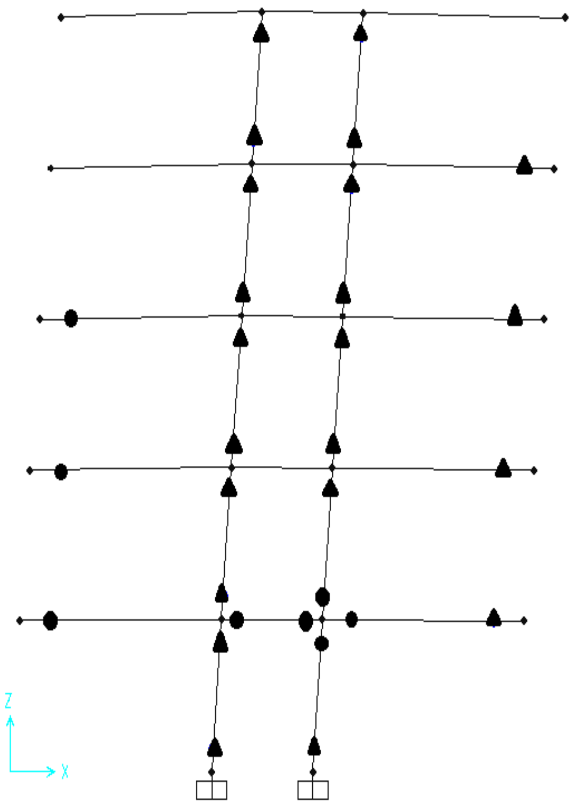


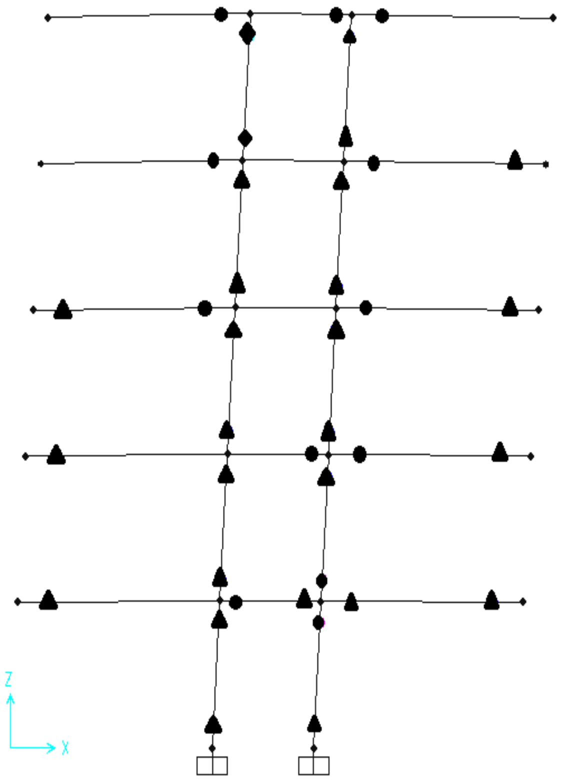


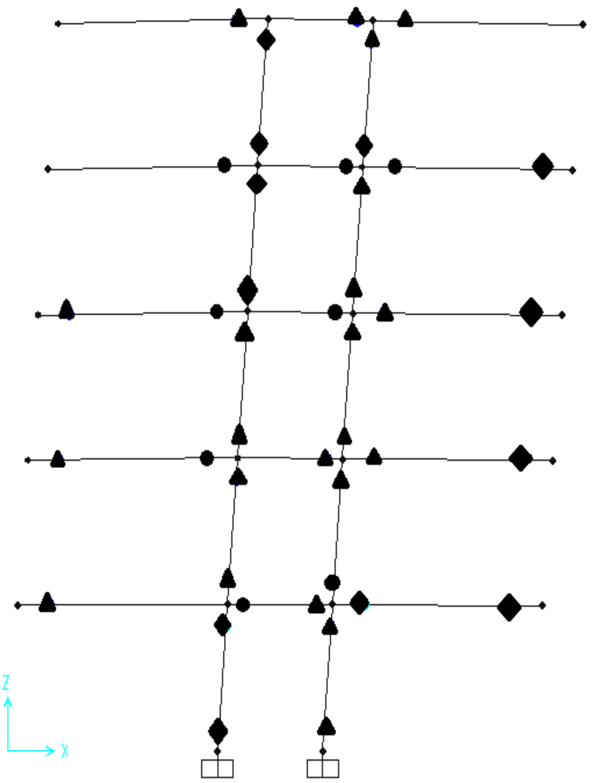
H.2.2: Hinge patterns of Case II (Partially detailed) for different displacements levels




Z
X

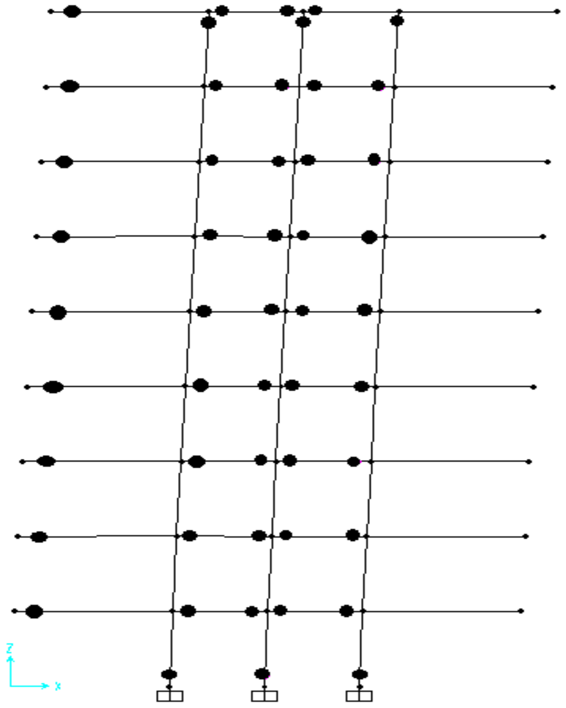

Z
X

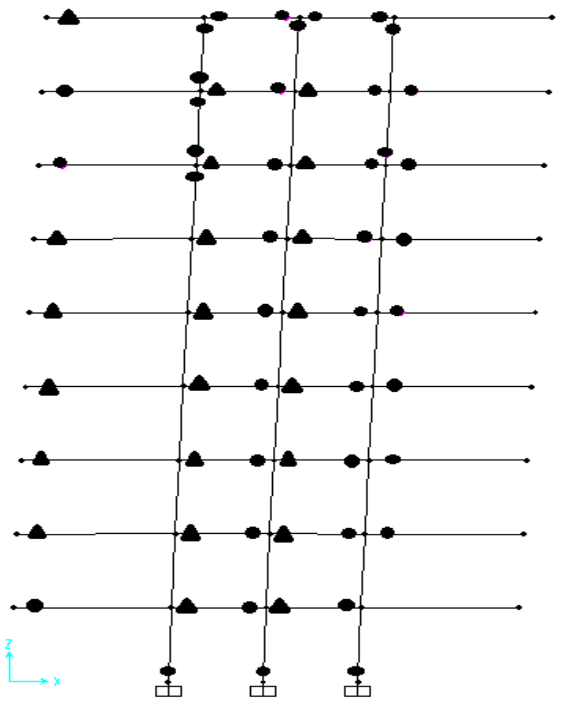

Z
X

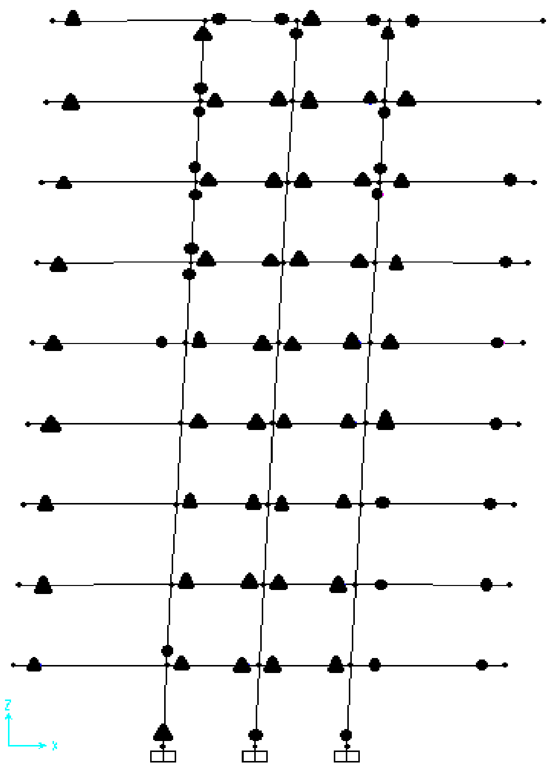

Z
X

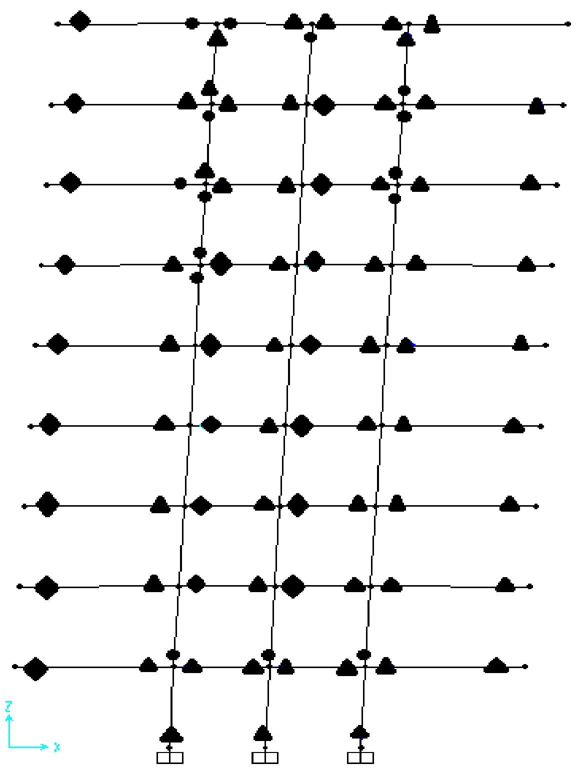
H.3: Nine storey building (Category 1, Model 2)

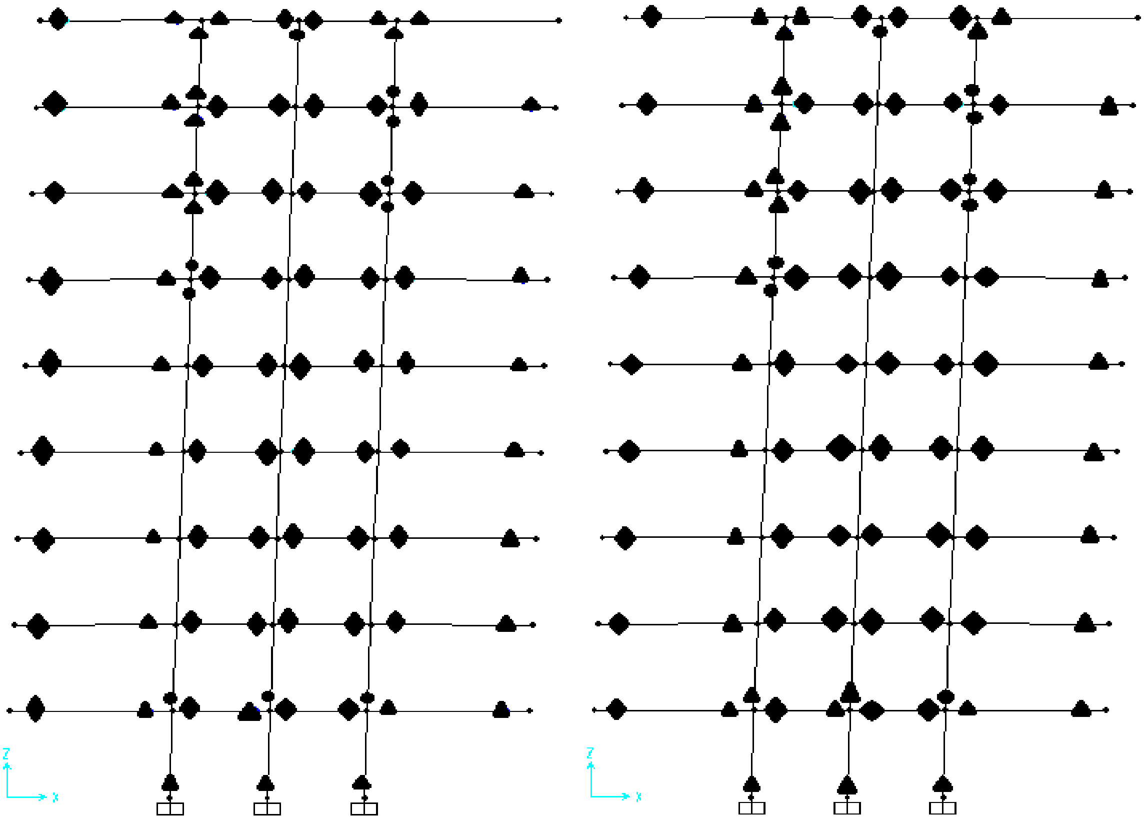
H.3.1: Hinge patterns of Case I (Fully detailed) for different displacements levels



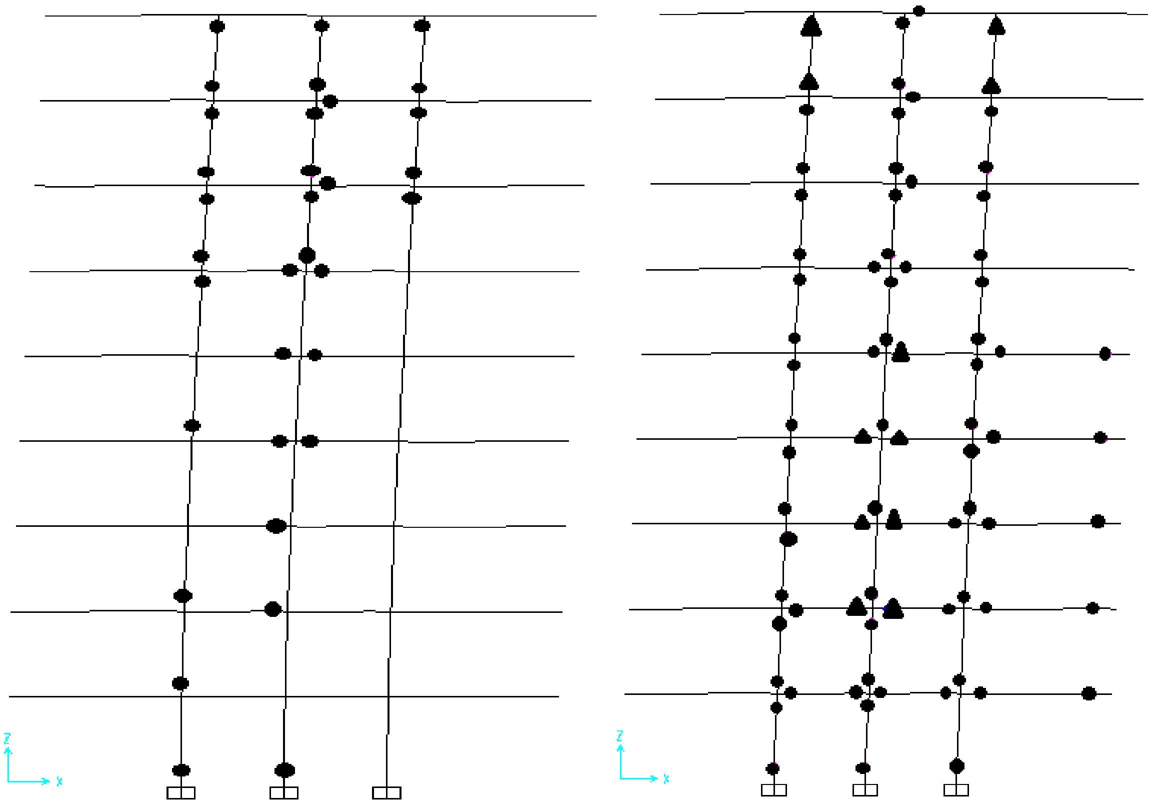


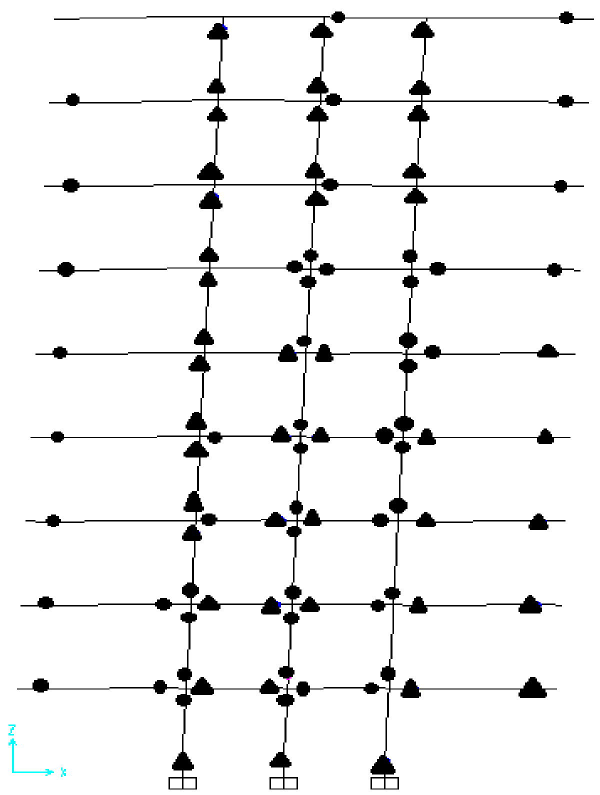


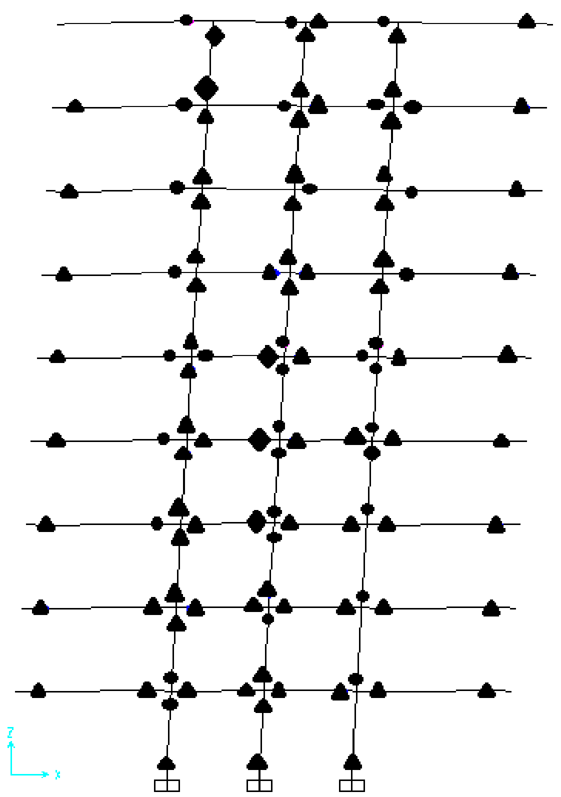


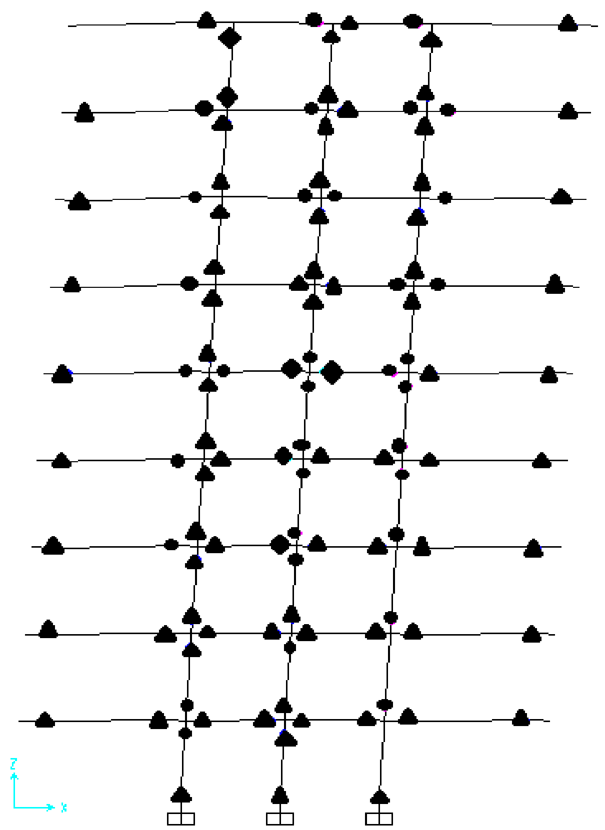


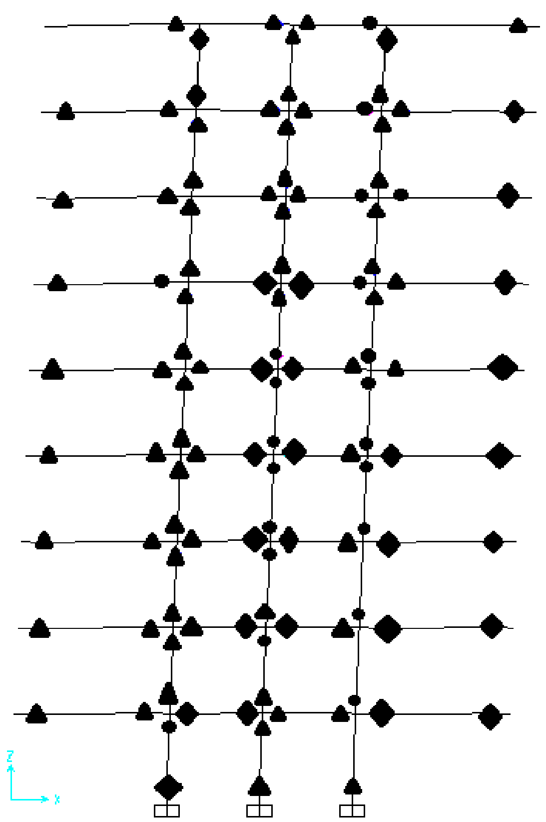
H.3.2: Hinge patterns of Case II (Partially detailed) for different displacements levels





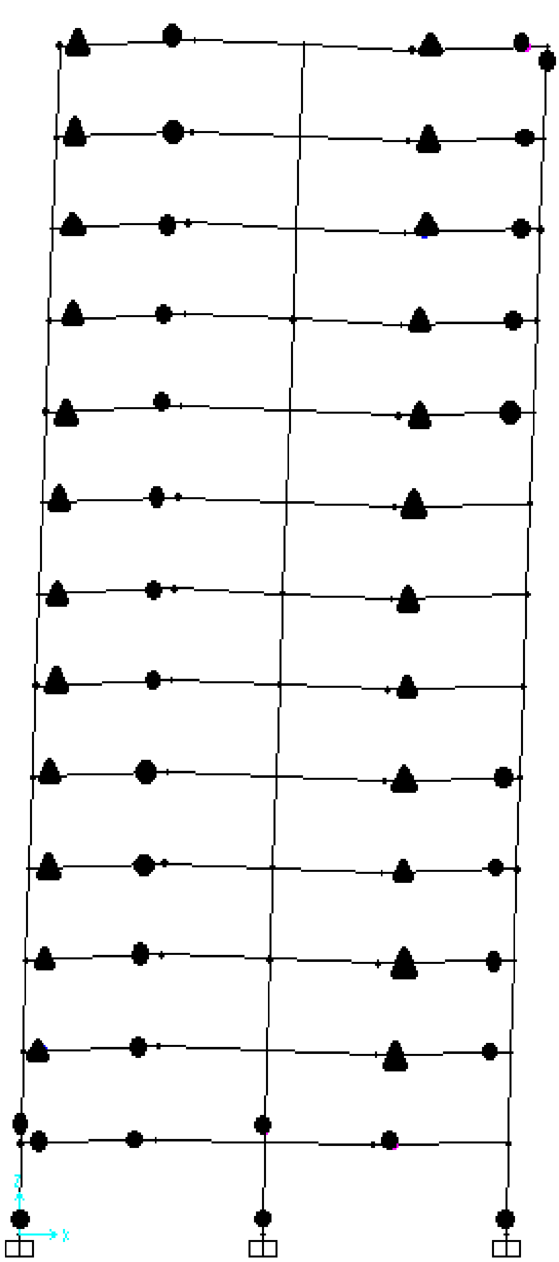


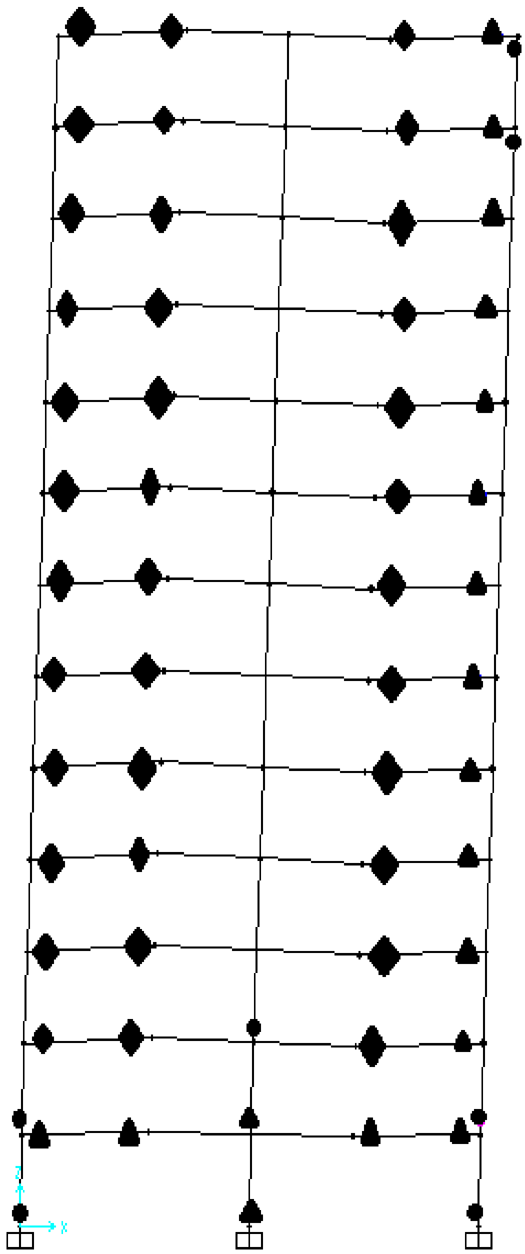


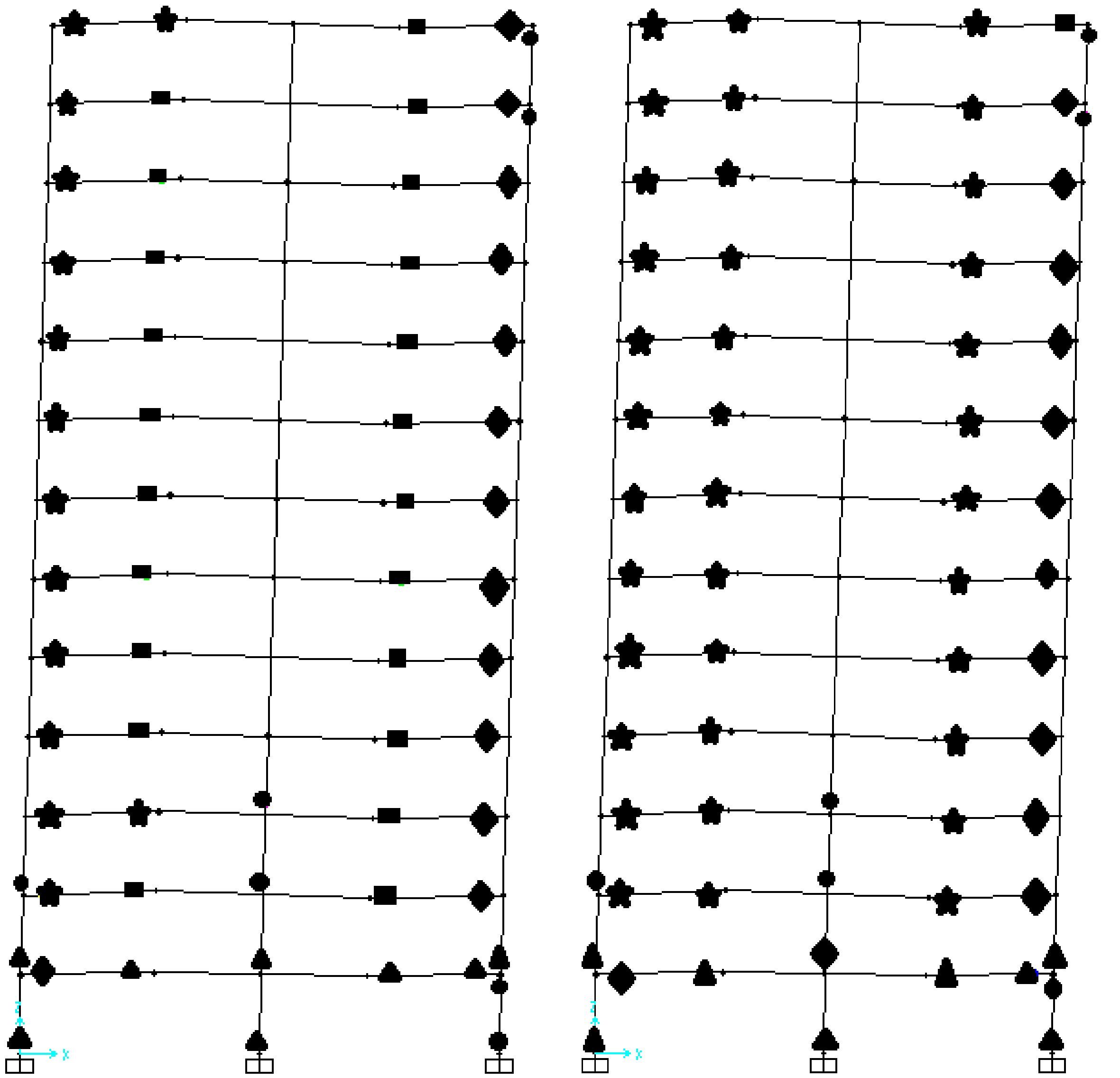


H.4: Thirteen storey building (Category 1, Model 3)

H.4.1: Hinge patterns of Case I (Fully detailed) for different displacements levels







H.4.2: Hinge patterns of Case II (Partially detailed) for different displacements levels