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M.Sc. in Civil Engineering

DINDAR MOHAMMED SALEEM

UNIVERSITY OF GAZIANTEP

GRADUATE SCHOOL OF

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A PARAMETRIC INVESTIGATION ON GATE BRACED FRAMES

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BY

DINDAR MOHAMMED SALEEM SADEEQ

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Supervisor

Assoc. Prof. Dr. Esra METE GÜNEYİSİ

By

Dindar Mohammed Saleem SADEEQ

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Prof. Dr. A. Necmeddin YAZICI
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Abdulkadir ÇEVİK
Head of Department

This is to certify that we have read this thesis and that in our majority opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Assoc.Prof.Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members	Signature
Assoc.Prof.Dr. Esra METE GÜNEYİSİ	....
Assoc.Prof.Dr. Ali Fırat ÇABALAR	....
Assoc.Prof.Dr. Kasım MERMERDAŞ	

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Dindar Mohammed Saleem SADEEQ

ABSTRACT

A PARAMETRIC INVESTICATION ON GATE BRACED FRAMES

SADEEQ, Dindar Mohammed Saleem
M.Sc. Thesis in Civil Engineering
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This study reported the results of a parametric investigation on the nonlinear behavior of multi-storey reinforced concrete buildings before and after upgraded with gate bracing systems having different eccentricity parameters and configurations. Gate bracing system contains of six members that are symmetrical, and in each side, three members connected to the certain point in the frame bay were employed. Two of them were non-straight diagonal elements while the third one was stretched on one side and connected to the edge of the frame. The case study structures with 2, 4, 6, and 8 storey had the same floor plan (3x3 bays) having the same bay length 5.5 m and storey height 3.3 m for all cases. In the current study, four different eccentricity parameters ranged from 0.2 to 0.8 and eight different configurations of gate bracing over the frame height were introduced. Each bracing part was chosen as circular hollow section with the fixed diameter and wall thickness. A total of 132 frame models (covering 4 without bracing and 128 with gate bracing) were subjected to the nonlinear static pushover analysis. The comparative analysis showed that the lateral load carry capacity of the structure was very close to existing building in 2 storey building in configurations that had no brace in ground floor, but in other cases we noted that decreasing the eccentricity from 0.8 to 0.2 dramatically increased the seismic capacity of all case studied structures.

Keywords: Gate bracing, Eccentricity, Pushover analysis, Reinforced concrete building, Seismic.

ÖZET

ÇAPRAZLI ÇERÇEVELER ÜZERİNE PARAMETRİK BİR ÇALIŞMA

SADEEQ, Dindar Mohammed Saleem
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Bu çalışma, farklı eksantrisite parametrelerine ve konfigürasyonlara sahip olan kapı çapraz sistemleri ile iyileştirme öncesi ve sonrası çok-katlı betonarme binaların doğrusal olmayan davranışları üzerine yapılan parametrik bir araştırmanın sonuçlarını sunmaktadır. Kapı çapraz sistemi simetrik altı elemandan oluşmakta olup, her bir tarafta çerçevenin belirli noktasına bağlı üç elemandan teşkil edilmiştir. Bunlardan ikisi düz olmayan köşegen elemanken üçüncüsü bir taraftan gerili ve çerçevenin köşesine bağlanmaktadır. 2, 4, 6 ve 8 katlı vaka analizi yapılan yapılar aynı açıklık uzunluğuna (5.5 m) ve her durumda aynı kat yüksekliğe (3.3 m) ve aynı kat planına (3x3 açıklık) sahiptirler. Bu çalışmada, 0,2 ila 0,8 arasında değişen dört farklı eksantrisite parametresi ve çerçeve yüksekliği boyunca değişen sekiz farklı çapraz konfigürasyonu dikkate alınmıştır. Çapraz elemanlar sabit çap ve et kalınlığına sahip dairesel boşluklu kesit olarak seçildi. Toplam 132 çerçeve modeli (çaprazsız 4, çaprazlı 128) doğrusal olmayan statik itme analizi yapılarak incelendi. Karşılaştırmalı analiz, yapıların yanal yük taşıma kapasitelerinin, zemin katta çapraz içermeyen konfigürasyonlarda, 2 katlı binada mevcut binaya çok yakın olduğunu, ancak diğer durumlarda, eksantrisiteyi 0,8'den 0,2'ye düşürdüğümüzde inceleme yapılan yapıların sismik kapasitelerinde önemli derecede artışlar olduğu göstermiştir.

Anahtar Kelimeler: Kapı çapraz, Dış merkezlilik, İtme analizi, Betonarme yapı, Sismik.

To My Family

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LIST OF SYMBOLS/ABBREVIATIONS

a	Times acceleration (m/s)
CBFs	Concentrically-braced frames
CHS	Circular hollow section
DL	Dead load (kN)
EBFs	Eccentrically-braced frames
fc'	Concrete compressive strength (MPa)
Fr	Inertia force (kg. m^2)
fy	Yield strength of the steel (MPa)
KBFs	Knee-brace frames
L.L	Live load (kN)
M	Mass
MRF	Moment resisting frame
P	Primary wave (m/s)
RC	Reinforced concrete
S	Secondary wave (m/s)
V	Base shear (kN)
2-D	Two Dimensional

CHAPTER 1

INTRODUCTION

1.1 General

An earthquake is immediate change or swaying of the earth's crust created by the abrupt statement of slowly expanded strain stored in the rocks below the earth surface generally, an earthquake is an oscillating motion provided due to release of strain energy beneath or inside the crust of earth surface (Mo and Kuo, 1998). Recent earthquake had caused severe social confusion in the region of the epicenter particularly due to structural failure producing damage to the people in the urban area wherever the structures are combined (Seo et al., 2015).

Throughout the world, many present RC structures that were gravity load produced or designed near earlier codes stay found on the hazardous seismic zones. Thus, during recent earthquakes these buildings suffer from the incapability to supply adequate ductility and also indicated that many of these structures are vulnerable to seismic damage and structural collapse (Nahavandi, 2015). In this respect, these existing reinforced concrete buildings must be upgraded to increase the lateral load capacity. The deficiencies of design codes before 1970s result in undesirable behavior of reinforced concrete structures during seismic excitations. As a result, several researches assessed seismic risk in existing structure, while others identified seismic deficiencies in structural elements, particularly in foundations, beams, columns, beam-column connections, slab-column connections. This resulted in the modification of building design codes. In addition, seismic retrofit methodologies were developed (Sabri, 2013).

Bracings keep the structure stable at conveying the loads indirectly (no gravity, just the wind either an earthquake load) under on the ground also are handled to hold a lateral load, thereby limiting vibration of the construction. The diagonal bracing is suitable components to increase the stiffness also protection on the wind loads. Where get various sorts of brace practices within collective performance before-mentioned being single diagonal brace, X-bracing, V-bracing, K-bracing, and inverted V-bracing (Biradar and Mangalgi, 2014).

It held remarked that all concentric X-bracing decreases further lateral displacement moreover hence, significantly supplies on imposing structural stiffness over the structure (Tafheem and Khusru, 2013). The most ordinarily utilized methods involve concentrically bracing frames CBFs, and the eccentrically bracing frames EBFs, also the knee-bracing frames KBFs. Usual arrangements for CBFs include V-bracing and inverted V-bracing, K-bracing, X-bracing and diagonal bracing (Uang et al., 2011). Still, the V-bracing stay unadvised during the seismic upgraded due to a probability regarding damage inside the beam mid-span. Below the lateral forces, each comprises bracing may be a buckle, consequently decreasing their load bearing capability the abruptly. Contrary, strength under a tension bracing improves monotonous, moving the yield strengths furthermore ultimately strain hardenings a glaring issue remains unstable forces directed through the brace to beam attachment that impacts into the beams, e.g. the increased bending and shear, should stay joined on these up to the gravity loads (Rai and Goel, 2003).

Every other, unstable strength against beams move passed within brace arrangements like macro bracing, e.g. the two and three story the (X) braces either the (V) braces including the zipper column. Also, Macro bracing bottle does appropriate toward hardening and stiffening of steel framed buildings. They are usually contracted to display significant stiffness furthermore enhanced flexibility. The bracing arrangements by such bracing did also utilize inside an analytic work to upgrade a substantial addition reinforced MRF with decreased the lateral stiffness (Khatib et al., 1988). The above studies have highlighted to need further works for characterizing and identifying the seismic representation of the buildings with various bracing systems.

1.2 Objective and Scope of the Study

The chief purpose of the investigation presented here does assess the nonlinear the seismic product regarding multi storey reinforced concrete buildings with gate bracing systems. For this, 2, 4, 6, and 8 storey structures were considered and gate bracing system was implemented to the structural system of those structures. A gate bracing system contains six portions that are regular, and the diagonal part remains non-straight with eccentricity parameters of e1 and e2. In the present study, the value of eccentricity e2 was kept constant as 0.6 while the value of eccentricity e1 varied from 0.2 to 0.8 (including the pairs digits between them 0.4, 0.6) as even value. Moreover, the brace members were chosen as circular hollow section (CHS). The eight different configurations of gate bracing were utilized. Thus, a total of 132 different frame models were studied. Then, nonlinear pushover response of the existing buildings and those with gate bracings was investigated comparatively.

1.3 Structure of the Thesis

Chapter 1-Introduction: The aim and purpose of the thesis are summarized.

Chapter 2-Literature review: This chapter covers the review of technical literature based on the scope of the study. For this, firstly, the development of structures, secondly, reinforced concrete structures with some examples, thirdly, the application of steel bracing in the structural systems are dedicated. Subsequently, more detailed explanation on the gate bracing system is provided.

Chapter 3-Research methodology: A case study includes the definition of analytical models of the existing and upgraded buildings. The geometry and properties of gate bracing system is also given. Moreover, the method and governing expressions used in nonlinear static pushover analysis of the structures and the procedure of every step is presented in this chapter.

Chapter 4- Results and discussion: Results achieved from the nonlinear pushover analysis of the structure equipped with and without gate bracing are given in this chapter.

Chapter 5-Conclusions: The conclusions are given in the light of findings from the overall results of the analysis.

CHAPTER 2

LITERATURE REVIEW

2.1 Development of Structures

Over and above, there are three principal kinds of the structures: steel structures, RC structures, and composite structures. Utmost about towering buildings into the world become steel structural system, up to its great strength to weight ratio, quickness from modeling range installation. Moreover, the administration during carrier on the situation, the availability regarding different force levels, also a full choice of the sections. Creator, framing operations including advanced treatment systems, upgraded the fire protection, corrosion protection, invention, also erecting procedures connected by the advanced analytical methods given potential with computers, has also been allowed the advantage of steel into actually a rational structural system during towering buildings. Steel can sustain significant plastic defacement before the collapse, therefore contributing tremendous supply strength. Here, the resources are offered on being flexible. Correctly the design of steel buildings bottle has great flexibility, which means a significant component for maintaining collapse loadings so while blasts about the earthquakes. Each flexible structures have the engaging power capability also will no contract swift destruction. Ordinarily, gives mostly noticeable deflections before failure or collapse (Gunel and Ilgin, 2007).

While the concrete being the structural substance should remain visible considering first events, only possible worth of RC held just started under 1867 (Ali and Armstrong, 1995). Discovery about RC Improved an importance including the use of concrete into each building industry to a significant amount. Notably, due to its mold ability properties, including real fire-resistant resources, designers, and engineers use RC to develop the structure. Also, it's parted into various also classic styles. Addition to that, while associated with steel, RC towering building has bigger humid rates willing for decrease movement attention, plus larger the concrete structures give increased resistance to the wind loads. Alterations during installation technology, processes about design plus meaning regarding the building, should only provide to the rest about operating with concrete into the structures of towering buildings. Furthermore, the high strength concrete while under the state of the Petronas Towers (Malaysia, 1998) also a Jin Mao Building (China, 1999), Additionally, lightweight concrete structural primarily in the event like the One Shell Plaza (Texas, 1971) admits working smaller member sizes and insufficient steel reinforcement (Ali, 2001). Come into steel or composite structure; RC provides an extensive range of structural arrangements for towering structures. The concrete and steel methods emerged separately from any other till 1969, the year during which the composite structure, primarily defined while a steel frame preserved by RC, of the 20 storey building, should give with the doctor Fazlur Khan (Gunel and Ilgin, 2007). All high-rise structures bottle be viewed as composite structures as it does unlikely to build useful buildings with using only the steel or the concrete. Such is, during an important function, utilizing medium steel reinforcement bottle do some concrete buildings a composite construction, also, under the similar way, RC slabs can earn a steel building a composite structure (Taranath, 1998).

2.2 Reinforced Concrete Structures

Utmost people understand that the concrete becomes continued into regular employment during several ages. However, that does negate the fact. Some Romans made take advantage regarding a concrete invited pozzolan before the birth of Christ. Others discovered buried sediments like powdery volcanic ash in Mt. Vesuvius also against another place into the Italy. Meanwhile, others incorporated that material among quicklime plus water as fine as the sand and the gravel, this strengthened inside the rocklike matter and has been worked as a construction substance. Unit force expects that some nearly lower classification from concrete would appear since compared to today rules, although some Roman concrete constructions stand yet into life now. An example is a Pantheon (a house given to all gods), which remains found in Rome and has been built in A.D. 126. Primary advantages of the concrete are neither so strong known. Enough about the initial work has taken through some Frenchmen Francois Le Brun, Joseph Lambot, and Joseph Monier. During the year of 1832, Le Brun created the concrete home also supported his home by a construction of a school also a church by the similar material. During the year of 1850, Lambot developed a concrete ship reinforced among a web of similar lines or the bars. Trust does typically provide on Monier however, as this design of RC. While 1867, the man obtained some license during the making of concrete sinks or bathtubs also stores reinforced by some net of the metal circuit. His regular intention during working among that material denoted to take balance out wasting force (McCormac and Brown, 2015). Due to the availability of raw materials of reinforced concrete (sand, gravel, cement, steel, water) and easy in construction, therefore RC buildings are considered as one of the most general buildings in many countries of the world (Nilson, 1997). The use of steel and concrete as RC in structures has some advantages such as:

- RC has significant resistance to the efforts of fire and water.
- RC structures are so hard.
- It does a base maintenance material.
- While associated including additional materials, which produces a high setting time.
- RC ordinarily some simply cost-effective material prepared during floor slabs, foundations, piers, excavation walls, also related statements.

In addition to these, there are some disadvantages that needs to be considered during the design such as:

- Concrete produces a little tensile strength, claiming the use of tensile reinforcing.
- Orders have a duty to keep the concrete against site till it freezes enough.
- The modest strength by a piece of the weight of concrete drives over large portions here enhances a frequently great stuff during large span buildings, anywhere concrete's massive total weight produces a significant influence upon the bending moments.
- Thus, the little force by the unit of volume of concrete involves segments mind stay comparatively high, a major concern for towering buildings including the long span buildings.
- The characteristics of concrete differ extensively because of irregularities during its rate and processing. Some examples of modern reinforced concrete structures are given in Figures 2.1, 2.2 whereas a significant example of the ancient concrete buildings in Rome is given in Figure 2.3.

Figure 2.1 Water Tower Place, Chicago, Illinois, tallest RC building in the United States (74 stories, 859ft) (McCormac and Brown, 2015)

Figure 2.2 Trump Tower of Chicago (MacGregor et al., 1997)

Figure 2.3 Hadrian's Pantheon in Rome (Mark and Hutchinson, 1986)

2.2.1 Earthquakes

Earthquakes happen for millions of years and will stay in the future as they have in the history. Any will occur in remote, underdeveloped areas where damage will be negligible. Others order occur near densely populated urban areas and subject their citizens and the infrastructure they depend on to vigorous shaking. It's hard to anticipate the earthquakes regarding happening, although that signifies likely to mitigate the results regarding substantial shock swinging to decrease waste of circumstances, scratches, also suffering (Das and Sivakugan, 2015).

Due to the sudden relief of the energy against the earth's rind, the surface regarding some earth is shaken, and that is called as an earthquake. This energy is released about seismic waves moving either through the Earth’s interior or near Earth’s surface. There are four primary types of seismic waves; two preceding body waves (Primary (P) and Secondary (S)) that move into the Earth’s interior as shown in Figures 2.4 and two passive surface waves (Love and Rayleigh) that move forward the surface of the Earth as shown in Figure 2.5. Their activities fluctuate depend upon some density. Moreover, the elastic attributes regarding each material they pass through, also they are increased as they move the surface (Kearey et al., 2002).

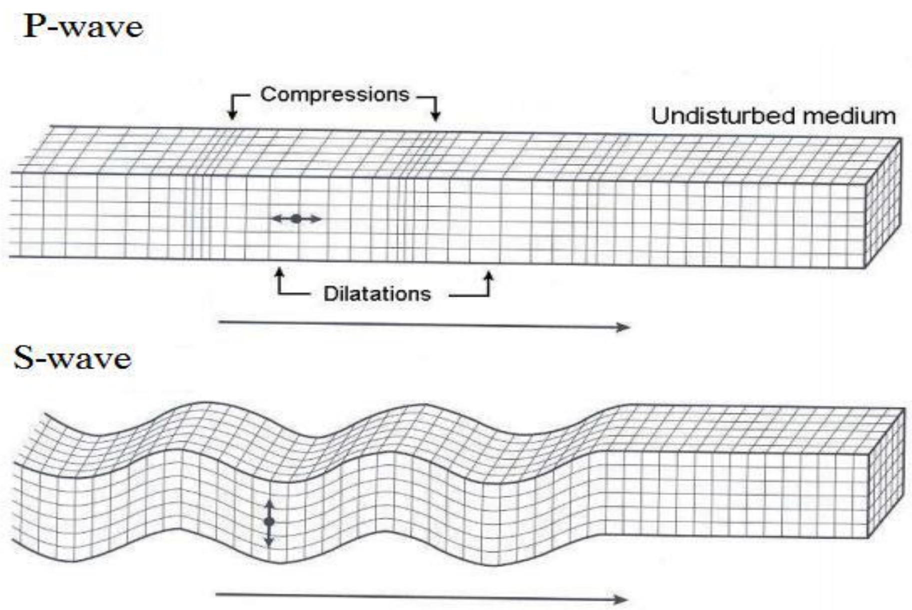


Figure 2.4 Earthquake body waves: (a) Primary (P) wave and (b) Secondary (S) wave (Kearey et al., 2002)

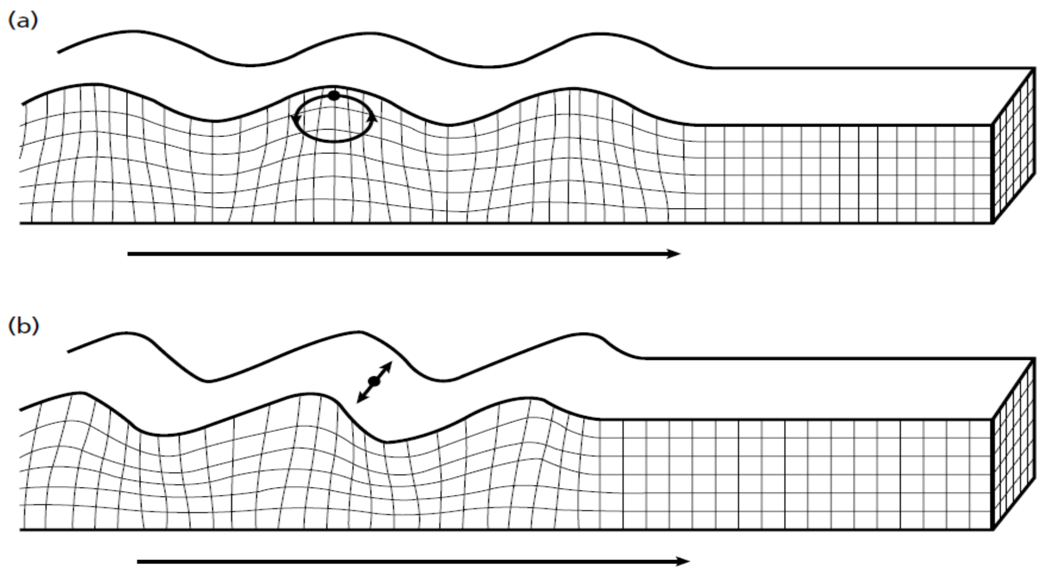


Figure 2.5 Earthquake surface waves: (a) Rayleigh wave and (b) Love wave (Kearey et al., 2002)

Love and Rayleigh waves both present grounds moving at the Earth's surface but a small action under in the Earth. Because the amplitude of surface waves declines less quickly with distance than the magnitude of P or S waves, surface waves are regularly the most critical part of ground swinging removed from the earthquake source, therefore can be the most destructive. Earthquake makes a shiver of a pile of earth. Such each erection pausing at it discretion undergo action toward its mark. Of Newton's first law about the movement, further, within these bottom regarding these erection motilities affects the ground, this roof ought to have each current to linger toward its initial status However, for these walls plus columns linger combined via this, all remove each shelter onward include matter. That signifies incredibly matched each state that thou remain coated including during the van you remain reaching under swiftly begins; your feet go in the van, without your top body serves to linger behind earning you happen backward. Here, the trend to maintain to endure during each former location means associated inertia primarily. Within each erection, considering some walls or columns remain soft, every change regarding the roof means separate of such of some ground. View each erecting whose roof signifies carried proceeding columns getting following over each relationship about yourself at the van: if this van swiftly begins, you move forced backward since if someone ought to have employed restraint to this higher body. Furthermore, if this earth shifts, yet each erection remains started backward, plus the roof feels each force, estimated laziness power. If this ceiling becomes each mass M plus undergoes an acceleration (a), formerly of Newton's second law regarding motion, these inertia force (Fr) comprises mass M, times acceleration (a), plus its way does reverse over that regarding this acceleration. Extra weight indicates tremendous laziness force. Accordingly, smaller erections support this earthquake swinging sharply (Veletsos and Meek, 1974).

2.2.2 Seismic Performance of Reinforced Concrete Buildings

Meanwhile regions of little-to-modest seismicity, the structural designers typically make not incorporate the seismic considerations against the erection design and analyzing. Aforementioned is true in many territories, including the UK, Central America, Australia, Hong Kong and many other regions in the world. A structure concerning which the seismic review exists not reflected however is merely planned and analyzed by gravitation weight, plus additional subsidiary horizontal loads would become to rely upon incorporating its intrinsic flexibility to return acceptable upon a sudden the seismic agitation (Kuang and Atanda, 2005).

In the study of Kunnath and Mander (1995), the seismic reply evaluation regarding surviving RC frame buildings designed concerning gravitation load hardly revealed that low-rise reinforced concrete frame structures (six to nine stories in these cases) in low-to-moderate seismic areas are vulnerable to damage in the event of an earthquake if the design and detailing do not allow for energy spread through plastic deformation of the structural elements. This is simply because the kinetic energy arising from the oscillations of the structure cannot be resisted by the elastic restoring forces. As the elastic restoring force is obtained on the basis of the most onerous combination of gravity and wind loads, it would be smaller than that demanded by a seismic event and the building structure may be expected to undergo deformation beyond its elastic range due to the seismic load. The response behavior would then be elastoplastic and the extent of the plastic component of the response characteristic the structure's ductility. However, when the ductility demand values are significant, that amount of ductility cannot be presumed to exist in the building, therefore it is rational to determine the available ductility in a structure. It has been stated that the available ductility could be determined theoretically on the basis of the geometric properties, reinforcement, axial load level and the quality of construction. The shortcoming of this proposition is very clear as no theoretical model can accurately represent the quality of construction and the dynamics of the interaction of the structural components at the post-elastic response (Penelis, 1997).

In the study of Park and Paulay (1975), the available ductility and the ductility improvement is verified through the fabrication and testing of large-scale models. The use of experimental determination is preferred because it has already been well established that the practical performance criteria of reinforced concrete structures may vary significantly from those evaluated theoretically. The comments upon the seismic ruins regarding (RC) plus masonry constructions. Typical designs for cuts against (RC) structures remain the little property regarding the concrete, revealing slips regarding reinforcement, short column, pounding, extensions plus beyond infill wall sections. Unusual events compared before inevitable losses remain summarized. Losses into the masonry erections occur due to loss regarding association within orthogonal walls and mismatched position plus dimension regarding possibilities (Inel et al., 2013). Some examples of damage structures are given in Figures 2.6, 2.7 and 2.8

Figure 2.6 Damaged wall frame building due to ground failure and wall rotation (Sezen et al., 2003)

Figure 2.7 Totally collapsed building (Inel et al., 2013)

Figure 2.8 Earthquake damage to the minaret of the Grand Mosque in Banda Aceh (Ghobarah et al., 2006)

2.3 Bracing Systems in General

For RC frames, steel bracing should also be practiced to defeat motion requirements. The brace bottle unless stay provided out leaving the operation (Bush et al., 1991) either used from inner the frame (Masri and Goel, 1996) like RC walls. Toward unless place, the steel brace gives new relevant resolutions against artistic courses concerning many bills. Figure 2.9 demonstrates the use of provider description the steel shear wall erected at the outer building frontispiece mostly another instance. The study of steel components by the features defined within steel rules and codes must remain created. These anchor design policies applied during the following segment may remain used for steel members and the connection between the existing structures. Another form regarding the damage that necessity mainly rests seen within that support class the steel shear walls stands to remain this out-of-plane the buckling regarding these compression elements from the buttress sample shear wall disposed of concerning. Here means proposed to establish the lateral supports on story levels to restrict the lateral buckling. Utilization regarding such elements may further maintain the design of shear walls to a particular range (Kaplan and Yilmaz, 2011).

Figure 2.9 Buttress type external steel shear wall (Kaplan and Yilmaz, 2011)

In design of low and mid-rise structures, size of structural members are mostly controlled by strength, however in high-rise structures, stiffness in addition to strength, affected member's size (Kheyroddin et al., 2013). Hence, that means essential to present appropriate device and mechanisms it increases the oblique balance regarding the high-rise structures. The bracing frames grow them meeting over the lateral forces through the bracing effect regarding slanting members. The most used configurations of steel braced frames are represented in Figure 2.10.

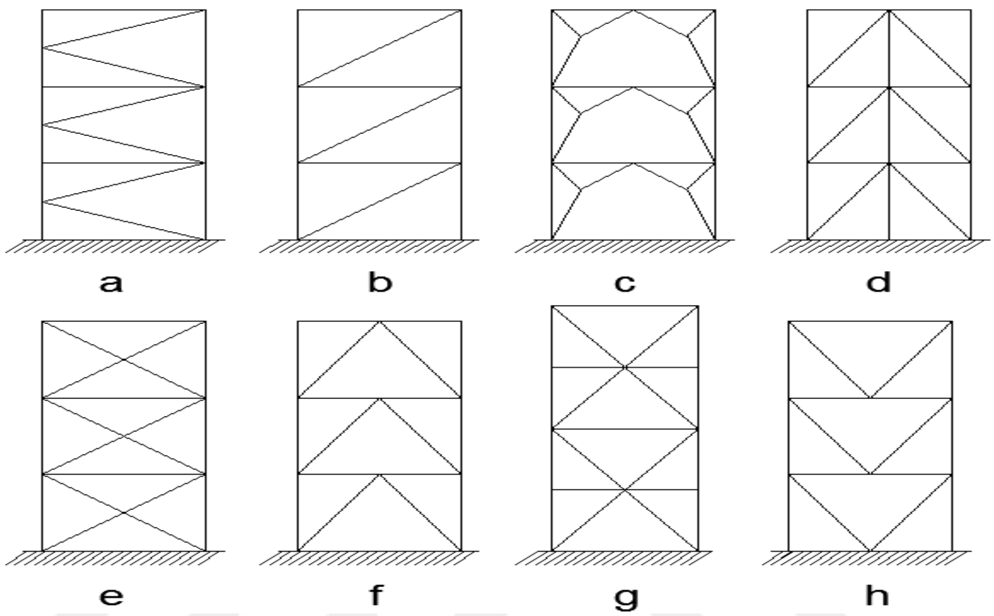


Figure 2.10 Vertical bracing (Fardis, 2005)

Bracing is a potent method that is appropriated to increase this universal stiffness including strength regarding the compound frames (Di Sarno and Elnashai, 2002). Braced frames are advantageous because the lateral loads are resisted mainly by brace axial capacity with little or no bending in the member. Braced frame members are sized to ensure sufficient stiffness to satisfy code drift requirements and enough member strength to resist both the compressive and tensile axial forces. The total behavior of the brace under repeated stress is related to its hysteretic behavior. Brace behavior is mainly depending on unwanted member buckling phenomenon. Thus, during cyclic loading, the brace performs unsymmetrical hysteretic behavior in compression and tension. Those fabulous regularly made braced frame arrangements involve the concentrically bracing frames CBFs, the eccentrically bracing frames EBFs, also the knee-bracing frames. The CBFs may define as a bracing system that is the brace centerline intersects at the intersection of the column and beam centerlines. The EBFs may define as a bracing system that is limited the one edge regarding every pair does join so as to transmit the brace force unless added bracing either to a column within the shear and the bending under a beam division announced a link. In some knee-braced frames, the main diagonal braces are joined to short knee members which span diagonally across the beam-column joints (Popov and Engelhardt, 1988).

Nateghi (1995) studied on strengthened eight-stories reinforced concrete building which designed without seismic protection in Tehran. This building actually five stories but building owners decided to add three more stories added before retrofitting of this structure. The existing situation of the building became too weak because of the additional weight. It was reported that the retrofitting of the existing building using steel braces were very influential.

Maheri and Hadjipour (2003) investigated about empirical examination plus study regarding steel bracing link to reinforced concrete frame. It was investigated three type connections of brace which suitable for design of frames under construction and existing reinforced concrete frame which suitable for the seismic retrofitting. Brace members and connections of brace and RC frame were subjected to direct tensile load and recorded until specimen were occurred ultimate failure. Tests results demonstrated the well-matched with prediction the individual elements in each connection.

Davaran and Far (2009) examined on an inelastic nonlinear model to obtain cyclic load with studying the influence of low cycle weakness regarding the steel bracing elements. Accordingly, the bracing component was designed while bound to plastic hinge at mid-span and ends of a brace. Small period gap impairment of the brace element was described by the implicating regarding the linear cumulative erosion conditions as those stable cyclic performances. Experimental results indicated that it could predict the under-cycle weakness of the bracing components by this model. Moreover, the breach point regarding the bracing portion has foretold satisfactorily with this design in one example.

In the study of Özel and Güneyisi (2011), they investigated this seismic resistance regarding a median rise RC building retrofitted with different type and configuration of eccentric steel braces. The 6 storey mid-rise RC structure has designed according to upon 1975 model about the Turkish Seismic Code. Both of influence of allocating the steel brace throughout this tallness regarding the RC frame and weight of the eccentric the steel bracing in the building has studied. These delicacy curves were developed regarding peak ground accelerations for four boundary cases. It was concluded that the retrofitted reinforced concrete building exhibited less fragile than the existing one, depending mainly on type and spatial distributions of steel braces.

Salawdeh and Goggins (2011) found a numerical model for rectangular structural hollow sections and cold-formed steel-square as axially full members for restraining earthquake. Cold-formed steel brace models were subjected to axial loading for pseudo- static cyclic physical tests to improve and measure a healthy arithmetical model that imitated the consequences from tests. Each range of plasticity along the element was regarded by designing some nonlinear fiber based beam column component layout. Energy dissipation, total energy dissipation, some cycles to fracture, initial buckling load, and maximum tensile force were considered as a design parameter in this study. It was observed that the statistical pattern provided an excellent suggestion of the supreme regular test parameters.

Tsai (2012) purposed adequate supply of steel braces to increase developing collapse endurance of existing building with aim of resist sudden column loss. It was regarded that pseudo-static response analysis of single degree of freedom arrangement with an idealized plastic-elastic when approach of design retrofit was improved. The stiffness and strength of added braces were determined from the connection between the structural characteristic and enhancement of collapse resistance. Incremental dynamic analysis was used to confirm exactness of the recommended approach. Nonlinear dynamic analysis results demonstrated that the proposed approach for the retrofitting was applicable for practical implementations based on the column loss reaction of the braced frames.

Durucan and Dicleli (2010) presented a recommended the seismic upgrading arrangement to improve this appearance regarding RC structures which were weak for lateral load. Proposed seismic retrofitting system was a relatively new retrofitting method which was occurred with a flexible shear link attached inside the frame plus braces and also the rectangular steel installation frame among the chevron bracing. It was applied on reinforced concrete building in order to increase strength, ductility, and stiffness properties of the building. In this study, the proposed seismic retrofitting system was compared with a traditional retrofitting system which using squat infill shear panels on a present office building and school. The original and 17 retrofitted buildings were subjected to nonlinear time history analyses considering three seismic performance levels. Result of analysis showed that the building which retrofitted with proposed seismic retrofitting system exhibited well energy dissipation with more constant hysteretic behavior than retrofitted with squat infill shear panels.

As previously mentioned, there exist two general categories of the bracing systems, namely, 1) Concentric Brace System and 2) Eccentric Brace System. These steel bracing rest fixed in vertically aligned spans. Here arrangement enables upon receiving a significant boost from stiffness among an insignificant united weight, the most common bracing configurations for GBFs and EBFs are provided in Figures 2.11 and 2.12 (Di Sarno and Elnashai, 2002).

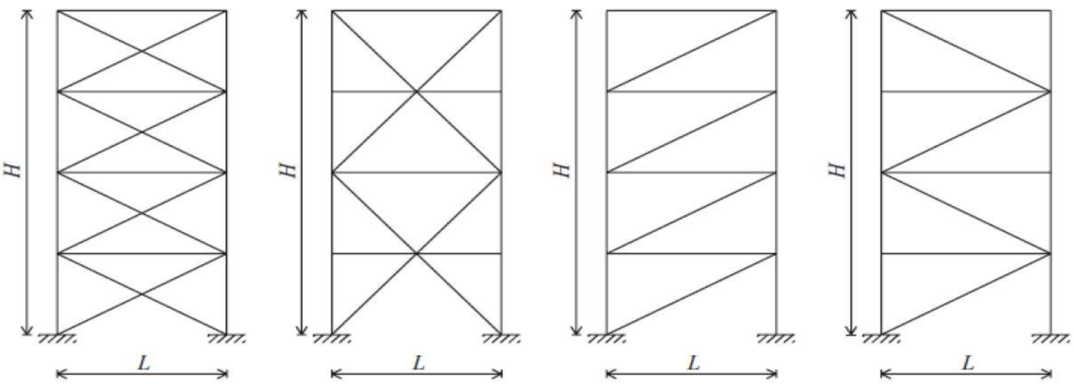


Figure 2.11 Common brace configurations for concentrically frames (Di Sarno and Elnashai, 2002)

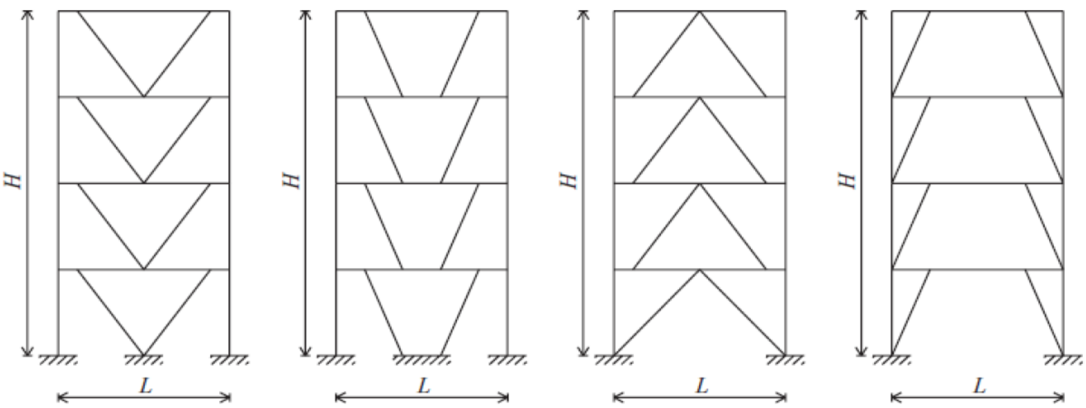
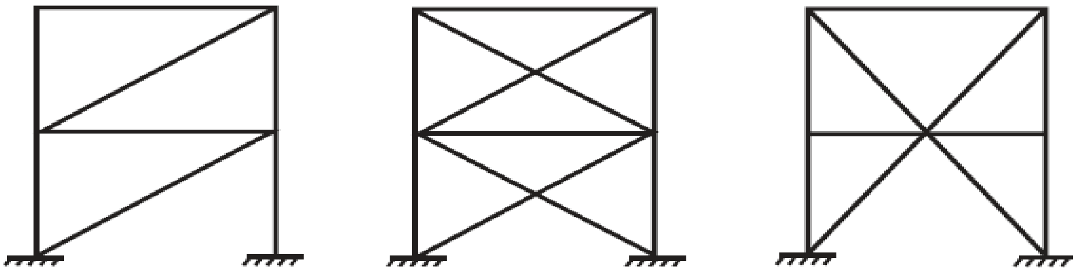


Figure 2.12 Common brace configurations for eccentrically frames (Di Sarno and Elnashai, 2002)

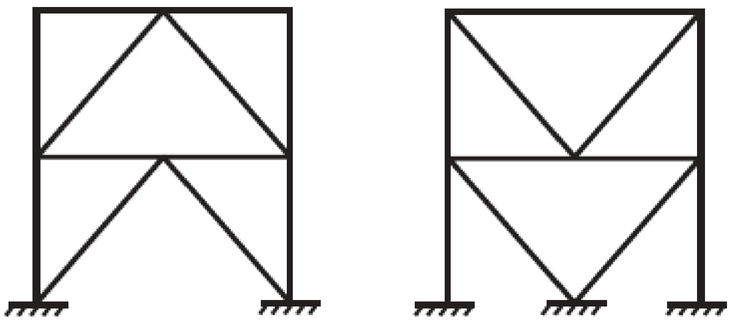
2.3.1 Concentrically Braced Frames

The bracing denotes concentric while the center lines of the brace segments meet. These concentrically bracing steel frames practiced inside buildings carry X chevron and Knee bracing. X-bracing does the most ordinary bracing. These diagonal sections regarding X also chevron brace work toward tension plus compression. Links concerning X-bracing stand found on the beam to the column connections. While chevron brace segments stay attached to the beam through the own head plus concentrate upon some obvious point. Concentric bracings raise that oblique stiffness regarding the frame also customarily decreases each slanted story line. Progress toward this stiffness may bring a considerable passivity force up to an earthquake. Additional, while some brace minimizes some bending moments also shear forces against columns plus all advance some axial confining inside those columns to which all remain relevant. The concentric brace does also fitting concerning a strength based design. Nevertheless, that almost means post yield capability plus the crisp collapse form regarding the concentrically bracing frame get here arrangement unfavorable concerning some flexible plan (Kangavar, 2012).

Each (CBFs) remain some variety of constructions bearing oblique loads within each vertical concentric support arrangement, each axes of that segment following concentrically on the links. The (CBFs) mind to stay active during opposing oblique forces because all bottle provides tremendous strength plus stiffness. Sure, things bottle additionally appears under a scarce desirable the seismic reply, such as under drift range and heavy accelerations. The (CBFs) is a standard basic steel or composite arrangement under areas of a seismicity. The CBFs are arranged considering columns, beams, and braces to method a vertical truss. This system bears to the lateral earthquake forces by truss action and also raises flexibility through inelastic performance in braces. This system is viewed as being the most stiffness efficient when braces behave in the elastic range. Once the inelastic response is received, the lateral stiffness starts degrading, and an asymmetrical response is formed. The popularity of this system is associated with the reduced cost, controlled fabrication process, and speed of building. Despite the advantages mentioned above, the CBFs exhibit relatively poor energy absorption and dissipation through inelastic response. Progressive slackening of braces, degradation of compressive strength and premature fracture render this system inefficient and unreliable for seismic applications. The CBFs classify as either ordinary or special. Ordinary concentric braced frames do not have extensive qualification regarding elements or connections. They are generally utilized in low seismic risk areas. On the other hand, special concentrically braced frames are generally employed in the areas of high seismic risk. The goal of the CBF design is ensured satisfactory ductility (i.e., to stretch without breaking suddenly) widespread CBF configurations are presented in Figure 2.13. The V-braced frames of Figures 2.13d and 2.13e are also known as chevron-braced frames (Sabelli et al., 2013).



Diagonal bracing (a) | X-bracing (b) | Multistory X-bracing (c)



Inverted V-bracing (d) | V-bracing (e)

Figure 2.13 Various configurations of CBFs (Sabelli et al., 2013)

In the study of Ikeda and Mahin (1986) produced some pin ended the bracing design including some flexible hinge positioned through its middle point (Figure 2.14).

Braced frame models were conducted to model some rigid behaviour regarding steel bracing. Therefore, frame systems could categorize as the finite element, phenomenological plus physical principles models. Phenomenological replicas extremely utilized for nonlinear seismic analyses in spite of the difficulty of determining input data. Against each design, an analytic axial force plastic link formulation has acquired, plus the departure of the tangent modulus regarding flexibility throughout cycles has read. The cyclic review issues regarding the bracing including dynamic summary results from bracing frame arrangements obtained matched including the search input. Due to the severity of the design center plastic shared entrance does fitting toward massive scale the structural system studies. The design was intelligent regarding reproducing some rigid function about the bracing under the cyclic plus dynamic summary. Corrections approaching each formulation through reflecting the variety regarding the tangent modulus from flexibility also limited buckling could start each conventional evaluation from cyclic rigid compliance.

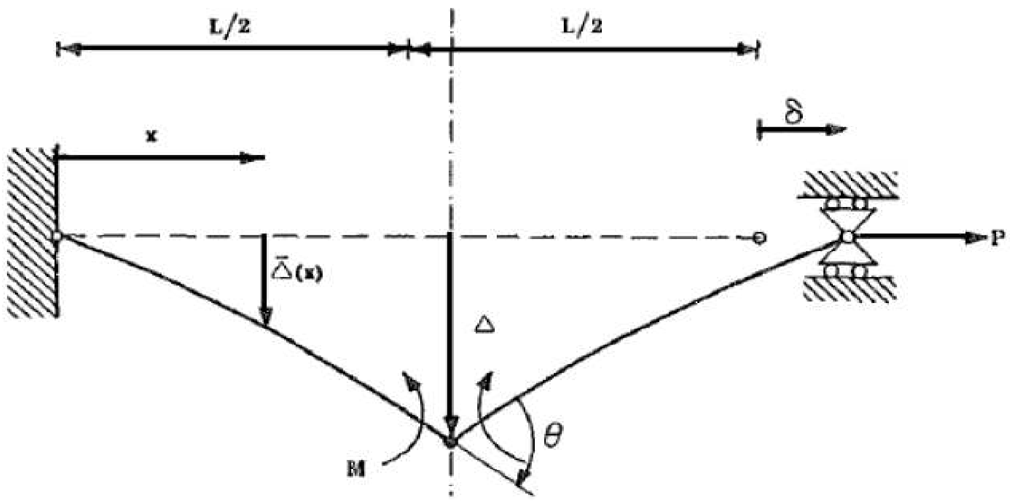
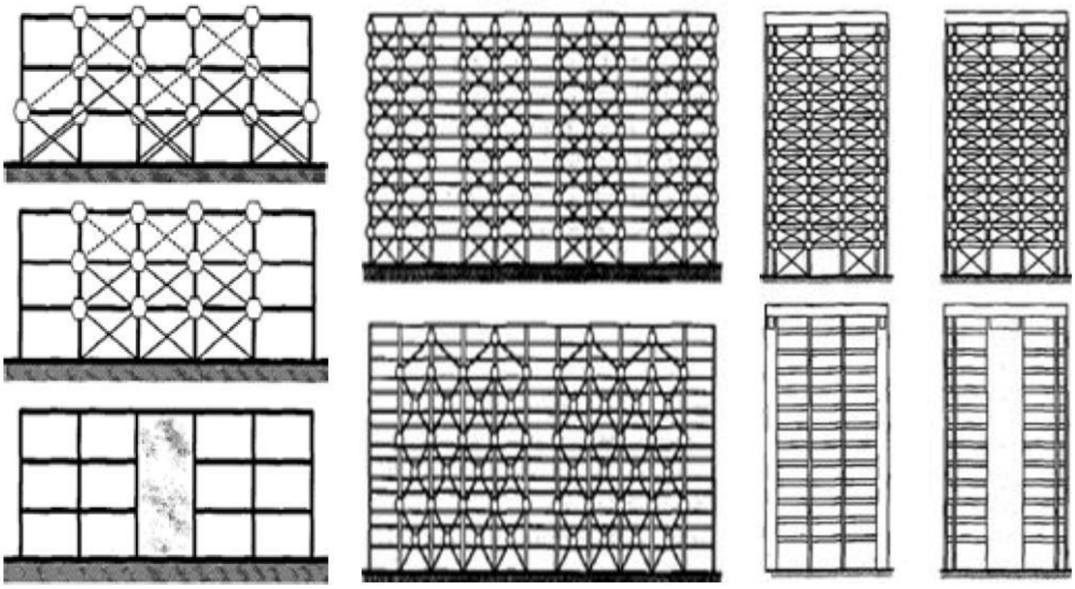


Figure 2.14 Brace model (Ikeda and Mahin, 1986)

Pincheira and Jirsa (1995), studied the exhibition of 3, 7 and 12, stories being RC buildings concerning five earthquake ground consideration careers held at the firm plus gray soils. The structures stood restored by three separate restoration plots; the post tension brace, X-brace, also an appreciation regarding structural infill the walls (Figure 2.15). Every method did compare including this reply regarding real buildings regarding the stiffness and the force augmentation. According to upon judgment issues, it did note that it was reasonable to pick of rare various the bracing orders while single solvent remaining on sole in order provides acceptable seismic performance. Toward third-storey building, everything the upgrading processes manifested fair review toward any ground acts. During 7 and 12 storey buildings, the structural wall afforded pleasing appearance. Despite, the brace arrangements made not present the exacted review during every ground act backgrounds. The Bracing arrangements could harm axial load levels approaching RC segments skeptically, therefore during such cases, and it could remain vital to develop axial load abilities regarding RC sections. During this post tensioned bracing instance, the pattern regarding constitutional forces toward RC segments remains intimately linked to the brace design, the bracing size also the initial level regarding the bracing pre stress.



Three-Story Building (a) Seven-Story Building (b) Twelve-Story Building (c)

Figure 2.15 Retrofitted frame model of the buildings (Pincheira and Jirsa, 1995)

Structural engineers have frequently reached concentrically braced steel frames as thrifty tools for confronting quake loads. Previous to now this system, popularly employed has viewed an extension utilization in the latest years as a result of the raised scrutiny disposed to steel moment-resisting frames, which contain indicated to be responsive to huge lateral displacements through strict earthquake ground motions and to need extensive care to require abstain from troubles related with the beam's breakage to column links (FEMA-355E, 2000). Nevertheless, corruption to the concentrically bracing frames into former earthquakes, so since the year of 1985 Mexico (Krawinkler, 1989), in 1989 Loma Prieta (Kim and Goel, 1992), during the year 1994 Northridge (Tremblay, 1995; Krawinkler, 1996), and 1995 Hyogo-ken Nanbu (AIJ, 1995; Hisatoku, 1995; Tremblay, 1996) earthquakes, increases interests through this final deformation capability regarding here section regarding organization. Distinctive the bracing frequently occupies the merely restricted ductility capability less than recycling load (Tang and Goel, 1989 a, 1989 b). Typical brace hysteresis performance is irregular between tension and compression as well as the illustrations significant strength decline when loaded monotonically or cyclically in compression. Considering this complex performance, the real disseminations of internal forces and deformations in less than a structure frequently oppose much from those prophesied utilizing traditional design methods (Jain and Goel, 1978 a, 1978b; Jain et al., 1978; Khatib and Mahin, 1987).

Design simplifications and applicable respectfulness regularly manufacture designs in which the braces chosen for some stories include distant larger capacity than needed though those chosen for different events have capabilities extremely near to their computed demand. This alteration on story demand-to-capacity ratios, together with actual post-buckling strength defeats in those braces. It tends to focus earthquakes damaged many of the stories. Like concentrations of place burdens damage to a large extent on the softer limited capacities of the brackets and links. The seismic design demands for braced frames have been adapted significantly through the 1990, then the idea of the distinct the concentric bracing frames had remained commenced (AISC, 1997; UBC, 1994). A large investigation has in addition started to develop the appearance of concentrically braced frames by the institution in recent structural configurations (Khatib and Mahin, 1987) or the utilitization of special braces by Liu and Goel (1987).

Through the past decade, extensive evaluation qualities of CBFs seismic performance considered to prevalent up to par was suitable and there had been corresponding improvements in investigation connected to qualifying the seismic risk at a site, appearing the seismic reaction, seismic performance theories playoffs in terms of probabilities (Sabelli, 2001). Arrange and allocation of plasticity in braced frames were influenced through the brace sizes, slenderness percentage, the form of the frame and the type of analysis in the ductile concentrically braced frames normal buildings. The universal crucial power was operated by the organization of structural mechanism in one story, considering that the rearrangement of inner forces in one story was confined particular in that story. Shear distribution in other stories not affected. Therefore, the yield stress is preparing in one story can result in the formation of a mechanism in this story and structure, therefore, ranged over its definitive capability. This indicated that the support in strength of the essential story was further the worldwide stand in strength to the frame (Annan et al., 2009).

The seismic resistance of steel braced frames was a mainly obtained of each axial force ability about their constituents. The steel bracing frames were effective since upright secures which columns were each chord and each beam and brace were some interconnection segments. The bracing frames might behave solely roughly toward concurrence by concrete approximately building walls around steel moment frames, during the formation of a common system. A nonlinear load-deformation performance during bracing was concluded via investigation roughly review protected via the study (FEMA 273, 1997).

These building codes made significant progress over the last decade in addressing unfavorable yield modes of braces and their connections. Since premature connection failures are now less likely to limit demands on braces in new construction, brace fracture life could become a much more important issue in the performance of these buildings. Several studies on the behavior of braces subjected to cyclic loading and on fracture criteria for steel braces were conducted at University of Michigan (Lee, 1987). Sophisticated nonlinear element models were performed to match these criteria (Jain and Goel, 1978 a, 1978 b; Jain et al., 1978; Rai et al., 1996).

The Federal Emergency Management Agency FEMA/SAC steel project reached results through the advancement of procedures for the steel moment design frames, which is in transition to give inter story production levels (FEMA 350, 2000). Though, a complete study has yet to be finalized for concentrically braced frame systems. A key element of the study design and retrofitting happens best measures for systems concentrically braced frame list to make sure that these systems fit in blending with the design based on performance criteria. One of the ways to gain the review about concentrically bracing frame to reduce some sense regarding drift inter story levels as a result of the use of innovative materials in brackets system (McCormick et al., 2007).

2.3.2 Eccentrically Braced Frames

EBFs seem to like over frame with chevron brace. Some disagreement among chevron brace and the eccentric brace signifies that period within each brace segments on the head gusset joint. Near an eccentrically bracing frame, the brace parts join to depart times on the beam. Some power from seismic movement into flexible deformation remains engrossed through some beam component within some brace members (Omar, 2014).

Moment resisting frame (MRF) has superiority in energy dissipation ability to achieve necessary flexibility. However, this structure needs strength and stiffness resulting the necessity of a larger surface area and an expensive double plate panel zone to fill drift requirements. CBF efficiently accomplishes deformation purposes through its framework action. However, the resistance in the mechanism of energy dissipation is limited (Popov, 1983). The arrest of both structures has resulted in the development of a new structure system named EBF structures. The difference in the three steel structural systems is shown in Figure 2.16 (Moestopo and Aulia, 2006).

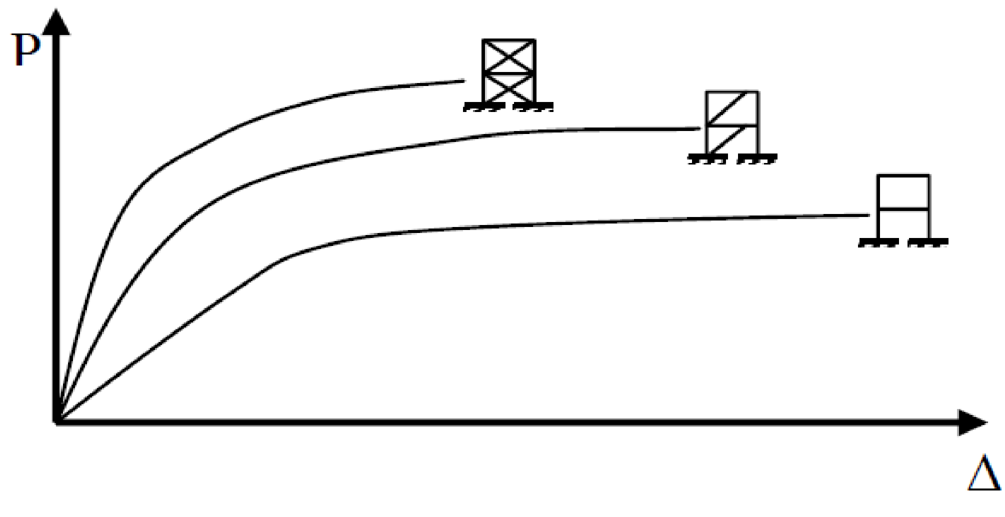


Figure 2.16 Behavior differences in three steel structure's system (Moestopo and Aulia, 2006)

Iskhakov (2001) determined optimal seismic response of RC frames with concrete braces assuming that braces were disconnected in tension. He declared fine energy absorption in braces due to nonlinear behavior of concrete materials in compression. Results indicate that using RC braces significantly reduced (over 50%) the seismic forces and relative displacements.

Xu and Niu (2003) compared several braced frames with the same MRF and shear wall experimentally. In their models RC braces were bearing both compression and tension. According to their study, in braced frames, not only lateral resistance and stiffness enhanced, but also energy dissipation amount increased significantly. This observation was based on elasto-plastic behavior of braces and energy absorption in hysteresis loops. Also braced frames had slower stiffness degradation than shear wall and had the same stiffness degradation as moment resisting frame throughout the loading cycles.

Watanabe (2002) make a collaboration with Takenaka Corporation in 2002 to use precast prestressed braces to strengthen a 4-story RC building against earthquake loads. Special connections with precompressed springs were used for installation. Hence, no tension imposed on precast braces.

The EBFs stay within influence an endeavor to blend the distinct interests regarding MRFs plus CBFs while decreasing their limitations. Figure 2.17 depicts several typical the EBF groupings. Several other satisfactory arrangements can act devised. The identifying a representative regarding an EBF signifies that at the atomic individual end of each bracing stays joined so that each bracing force means broadcasted unless to added the bracing roughly a column into shear and bending in a beam locality termed a ring. This ring lengths in Figure 2.18 obtain specified by the letter (Popov and Engelhardt, 1988).

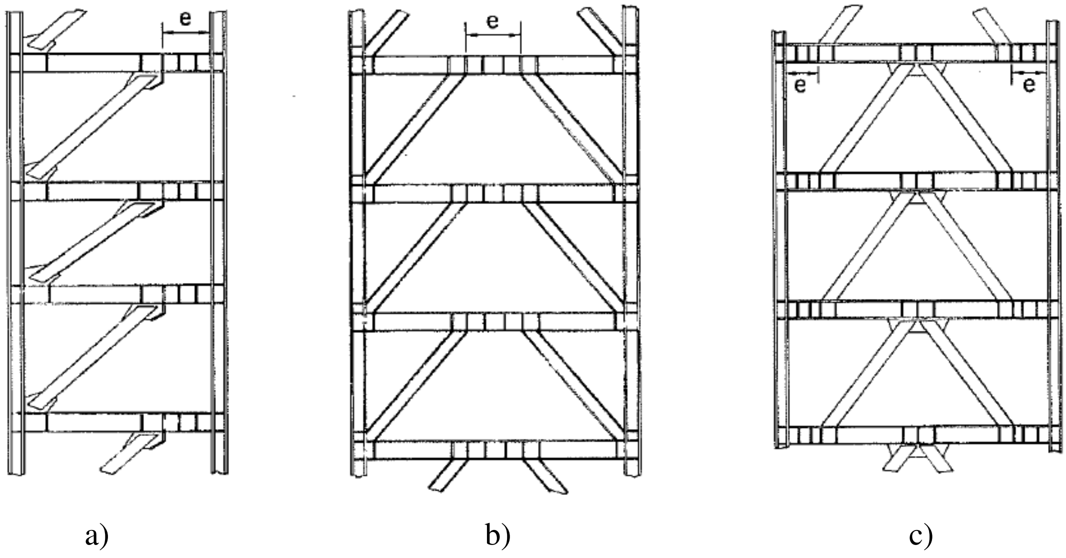
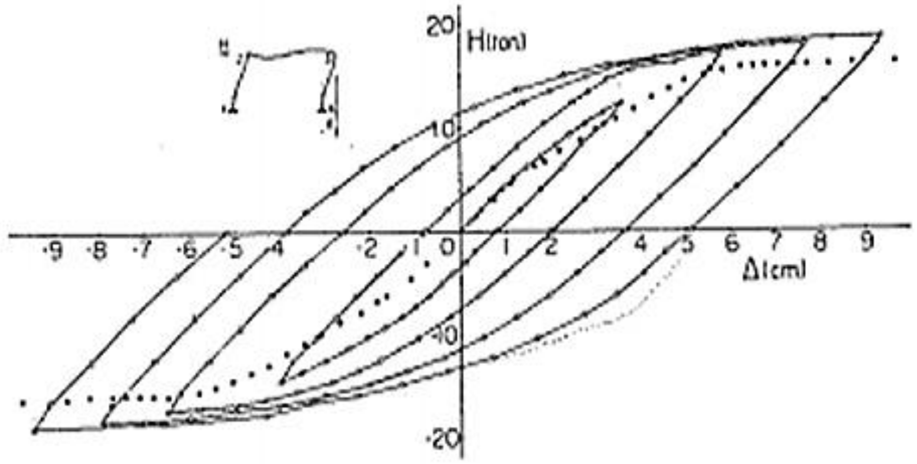
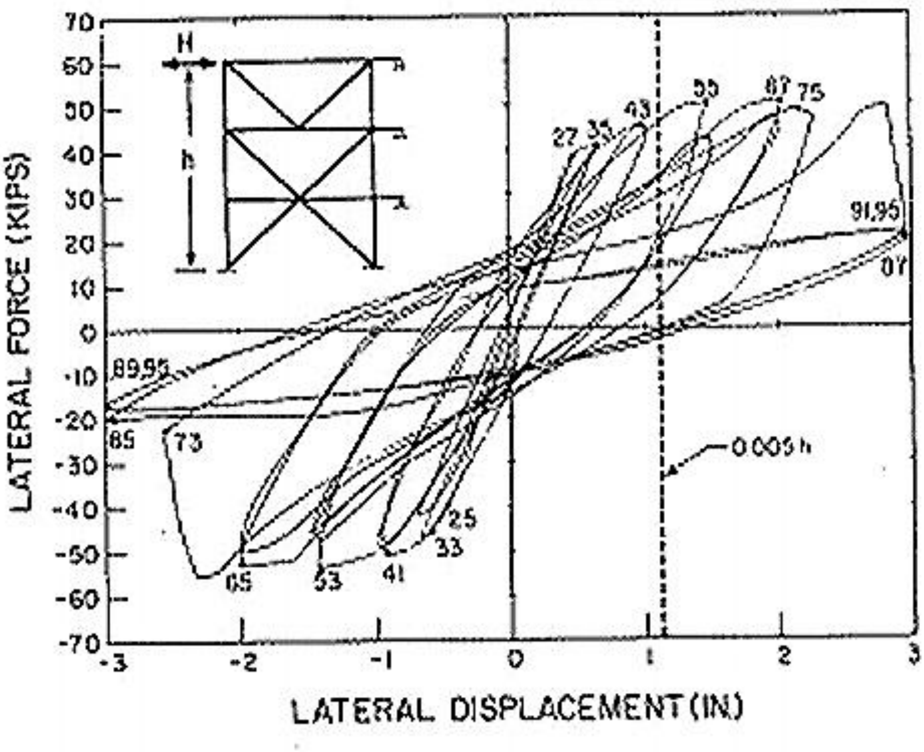


Figure 2.17 Typical EBF arrangements (Popov and Engelhardt, 1988)

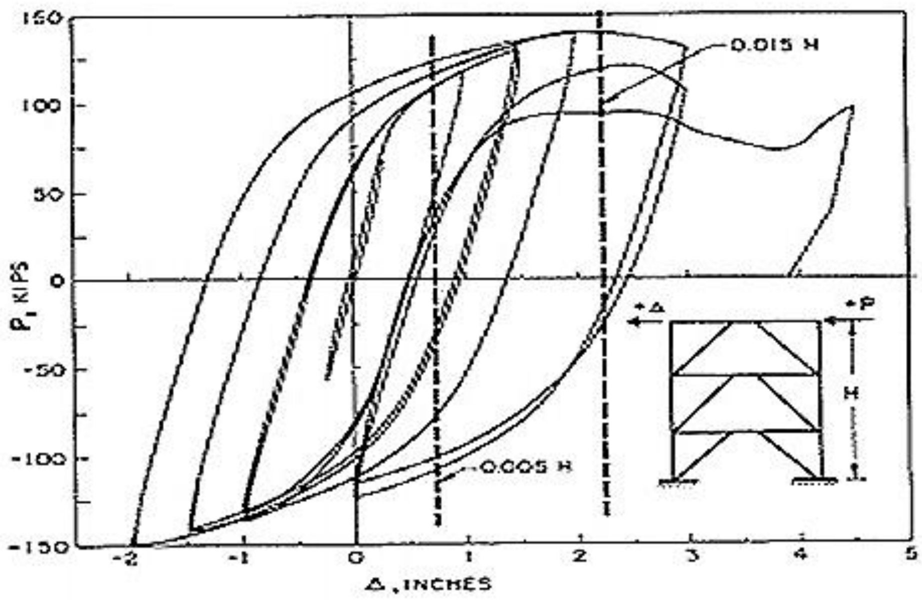
For seismic-resistant of EBFs, the most attractive feature remains their extraordinary stiffness connected beside different power waste inclination and ductility. Under the EBFs some brace segments fit the huge flexible hardness aspect of CBFs, which provides the code drift conditions to do fought economically. Firstly, braces stand intended not to buckle, but regarding some rigor about oblique loading on the chassis. Secondly, stable action under hard the cyclic filling implies limited principally upon each link, which is produced and reported to nurse super permanent warp out the waste of energy. The yielding in EBFs for the links serves to transfer to the brace limit the maximum force, acting, actually, as a fuse for member loads bracing. Can be calculated the definitive strength of the link accurately. The energy dissipation and flexible capability regarding the EBFs may obtain adequately seen through analyzing these real conduct regarding model frames beneath league the load. As shown in Figure 2.18 the typically temporarily secured oblique load versus displacement plans toward the EBF, CBF, and an MRF. Some steady including full hysteretic circuits in Figure 2.18a showed the MRFs are characteristic of the unique energy waste ability about a moment resisting frame and sense to provide big warp outwardly strength need. In opposition, some circuits in Figure 2.18b stay pinched plus worsen since some whole of loading cycle's increases, confirming some slight weakness energy diffusion capability regarding CBFs. Lastly, Figure 2.18c demonstrates some hysteretic mode relating to a fine-designed EBF. Due to some loop can nurse massive warp externally strength hurt besides because brace buckling is limited, steady hysteretic circuits comparable and full to these regarding the MRF do take (Popov and Engelhardt, 1988).



a)



b)



c)

Figure 2.18 Typical experimental frame behavior under cyclic lateral load (a) MRF (after wakabayashi), (b) CBF, and (c) EBF (Popov and Engelhardt, 1988)

EBFs as shown in Figure 2.19 are a sort of "compromise" between moments fighting frames then concentrically bracing frames as combining the strength and stiffness of a CBF with the inelastic performance of a special MRF. The performances of the structure are strongly dependent on the behavior of the links, which require particular care in the phase of design (Naqash et al., 2014).

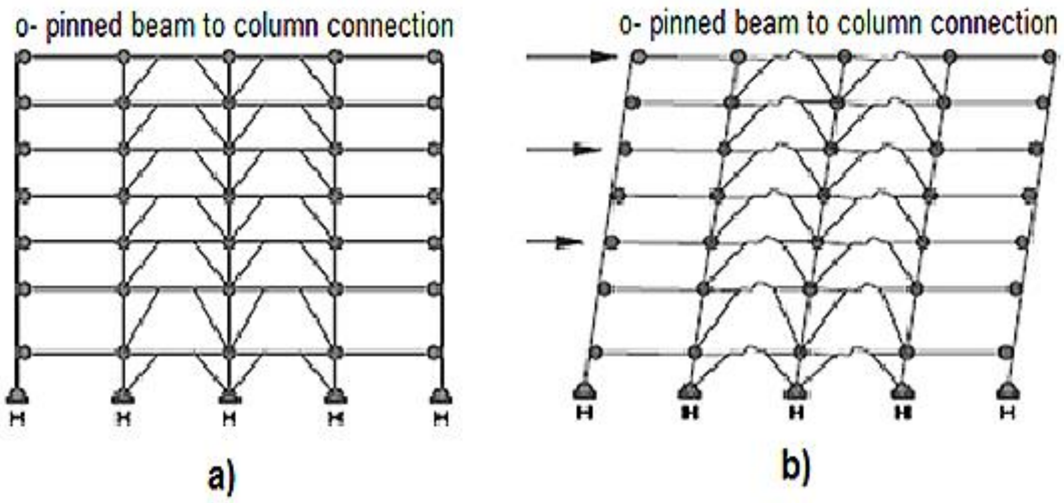
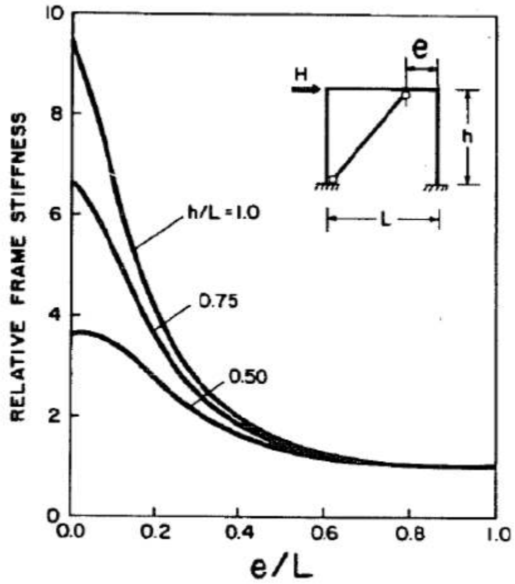
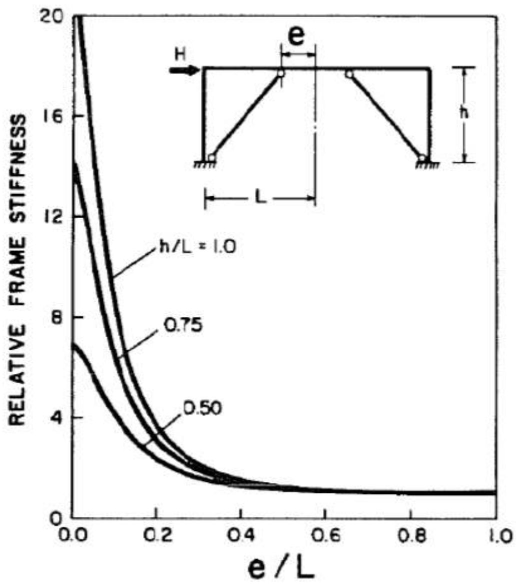


Figure 2.19 EBFs: (a) general scheme and (b) collapse mechanism (Naqash et al., 2014)

That is informative to reflect some modification regarding the flexible oblique hardness from the EBF primarily a role about this bond length e. The difference implies shown in Figure 2.20 during two mild eccentric frame patterns. Notwithstanding e = L, one becomes a flexible hardness and moment resisting frame at a minimum. For e/L > 0.5, from the bracing is gained little stiffness. However, as the length of the link deteriorates, each quick rise in hardness transpires. Supreme hardness emerges when e = 0, epistolizing to a concentrically bracing frame. This is hereabouts case, of direction, that is dedicated to avoiding EBFs. When e = 0, there remains on link present to act as a fuse for brace member forces. As shown in Figure 2.17 precisely this to earn excellent potential frame hardness, some bonds requirement rest had short. Figure 2.21 illustrates the disparity regarding natural stage including different e/L ratios (Popov and Engelhardt, 1988).



a)



b)

Figure 2.20 Variation of elastic lateral stiffness with e/L for two simple EBFs (Popov and Engelhardt, 1988)

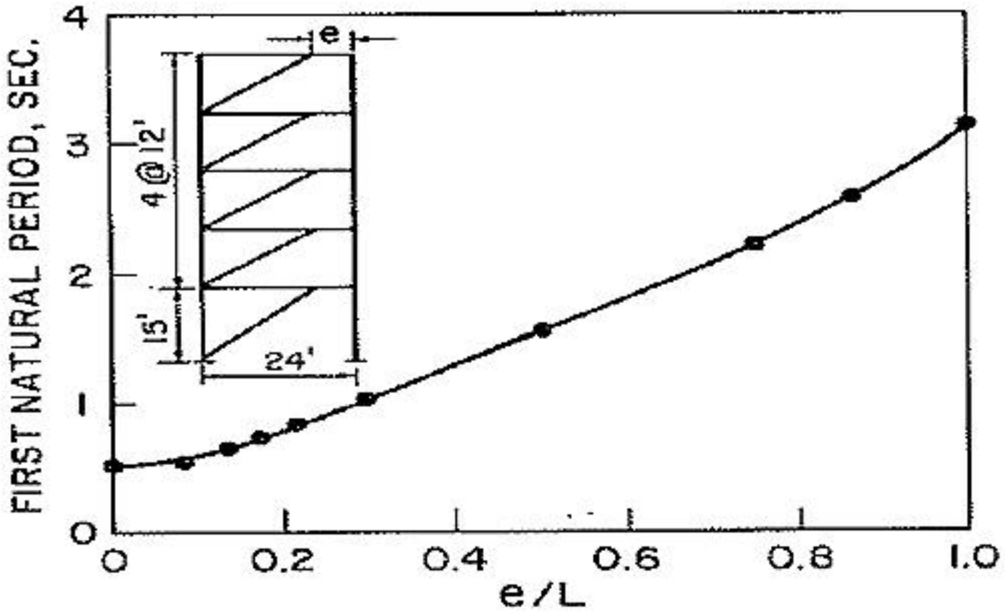


Figure 2.21 Variation of first natural period with e/L for a five-story EBF (Popov and Engelhardt, 1988)

This theory could probably be employed to leave this initial stage regarding an erection on of the top near a reply color concerning a single locality. Those critics remain, nevertheless, ignorant regarding each example about the EBFs denoting practiced concerning here function. Fashionable extension upon fixing some flexible hardness, some bond distance plus significantly influences this force regarding the EBFs underneath oblique load. As shown in the Figure 2.22 this latest force regarding a third-story EBFs essentially one position regarding e/L, considering the flexible-completely flexible review. The frame amplitude continues to normalize through some amount of 2Mp/h which-represents some intensity regarding the MRF. The structure force quickly progresses amidst lowering bond distance, unto that frame effect remains set over single flexible capability of the links. In Figure 2.18 this section of frame performance denotes symbolized through some horizontal lines. Apparently, superior the frame force acts accomplished besides small loops. This obligation holds noted that some consequences regarding the relationship of gap demonstrated during Figures 2.20, 2.21, and 2.22 represent an idealized place concerning the short frames, hoping regular segment lengths since e remains discrete. These real consequences would depend touching several constituents, including erection elevation plus code inflicted motion restrictions. Notwithstanding, certain characters stand illustrative regarding some meaningful drifts toward performance since loop unit holds various (Popov and Engelhardt, 1988).

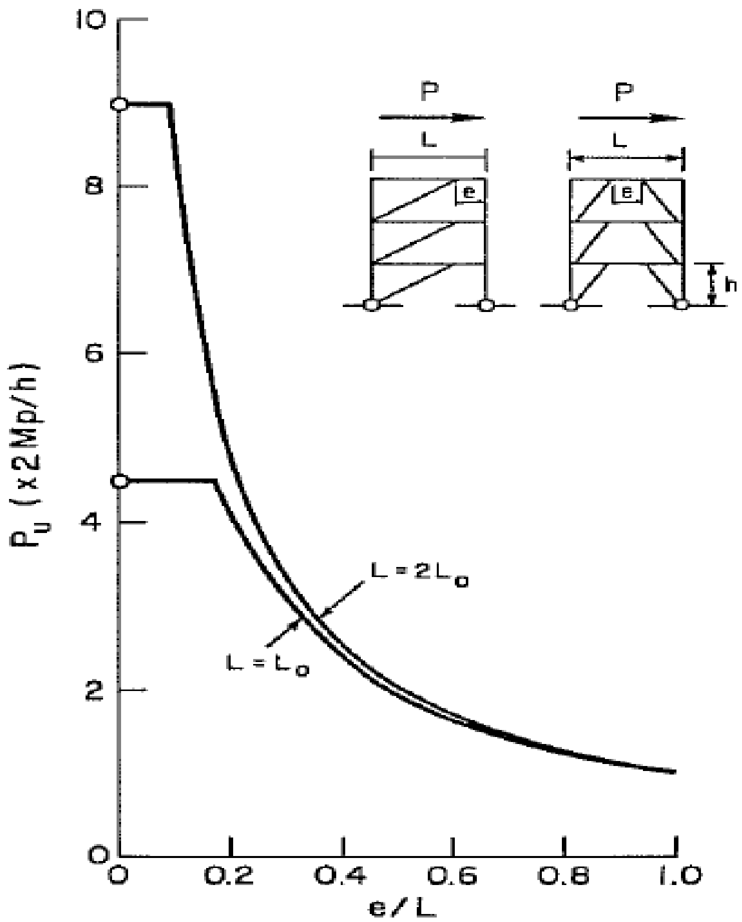


Figure 2.22 Variation of frame plastic capacity with e/L (Popov and Engelhardt, 1988)

2.3.3 Gate Bracing

Within regions controlled by seismic actions, the event of critical earthquakes can produce severe destruction upon resources and lack regarding lives if erections stand no equipped beside the seismic protection inclination, some requirement to own erections fit planned upon the earthquakes does also extra important against profoundly urbanized regions amidst countless towering constructions wherever economic plus personal injury stands regarding prime interest upon some city. Here is presently frequently understood that the seismic study of erections should provide toward embryonic pair underlying conditions (UBC 97). Head, some construction need act elastically plus preserve moderately crisp nonstructural segments upon lesser the earthquake territory swinging (Mosalman and Yazdi, 2011).

Accordingly, each structure must hold adequate strength plus bending stiffness to check fundamental displacements, before-mentioned as an inter-story stream. Next, some construction need never fail against some significant the shock. Concerning here matter, extensive destruction to some construction plus nonstructural ingredients signifies satisfactory (Davaran and Hoveidae, 2009; Asgarian et al., 2010). Approaching stop this building from collapsing plus minimizing significant waste regarding life, that obligation has some great power diffusion potential while thick, unyielding deformity. While conventional, the building's systems which present steady hysteretic circuits operate adequately beneath some high rigid the cyclic storing symptoms of primary earthquakes. Before-mentioned constant hysteric symptoms regarding some construction vessel stay concerned implemented that some building segments plus links stand planned to beget ample flexible (Khatib et al., 1988; Asgarian et al., 2010).

Concerning high plus mediocre rise buildings, the structural steel becomes adopted broadly due to its superior strength plus flexibility features. During the earthquake study regarding such steel structures including reflection of cases mentioned above any standard current seismic rebellious paths mind rest revealed next, exactly within certain traditional ways, the bracing frames remain amongst some prevalent steel constructions toward combatting oblique loads. Meanwhile customary, all remain split inside two organizations: concentric and eccentric (Ghobarah and Abou, 2001; Kim and Choi 2005; Moghaddam et al., 2005). Concentrically bracing methods obtain extra acceptable due to some corresponding right stiffness, onward including their natural system plus economics appearances; onward those vital patterns present that organization besides obvious than eccentrically bracing frames (Yang et al., 2008; Davaran and Hoveidae, 2009).

The eccentric bracing wants further development efficiency whereby following toward some reduction regarding development activity plus greater price during hatred of normal stiffness review plus greater power dissipation (Özhendekci and Özhendekci, 2008; Bosco and Rossi, 2009).

Gate bracing as illustrated in Figure 2.23 contain six segments that are regular plus the oblique segment does never straight; that means linked on the edge of the support with the third component. Here organization increases the power emission due to earthquakes as well as its eccentricity giving more possibilities for the architects. Furthermore, due to the cyclic kind regarding seismic loads, individual bracing components are produced regularly (Moghaddam and Estekanchi, 1995, 1999).

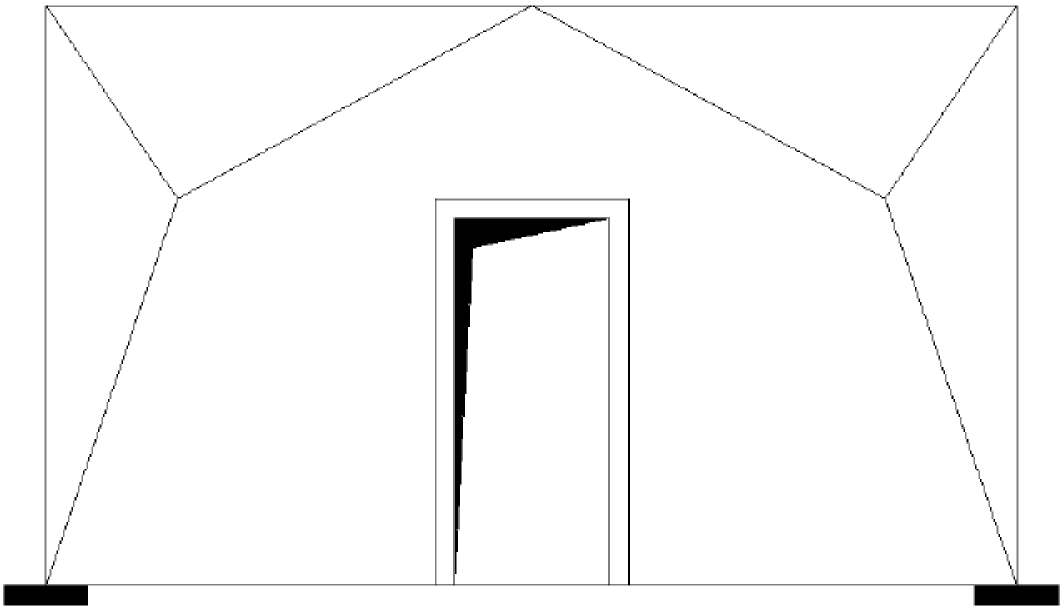


Figure 2.23 A typical gate brace system (Mosalman and Yazdi, 2011)

Unit regarding the many powerful plus unique things regarding this bracing signifies some use about the bracing components while carrying some oblique load. Due to some impressive oblique load three brace components, which stay attached to one joint feature dress under pressure, plus the three extra bracing components, which remain attached to the other aspect in compression mostly shown in Figure 2.25. These three compressive components which remain coupled to the middle attachment bottle secure some system unpredictable near the buckling regarding each of the bracing elements also the buckling regarding the center link upon the out of frame extension (Saffari and Yazdi, 2010). For explain this beyond of flat buckling regarding here view duration via inflicting an oblique load, some frame implies created with the SAP2000 software. For review, the stiffness regarding this way, two parameters drafted as m and n move begun to designate the place of the intermediate bond point of the top edge of the support. Certain parameters describe some coefficients of height, and span width regarding the support in which the horizontal distance of the bracing component attachment from the frame edge is mL / 2 and its vertical distance is nH, where L and H are span width and height of frame panel, individually Figure 2.24 (Mosalman and Yazdi, 2011).

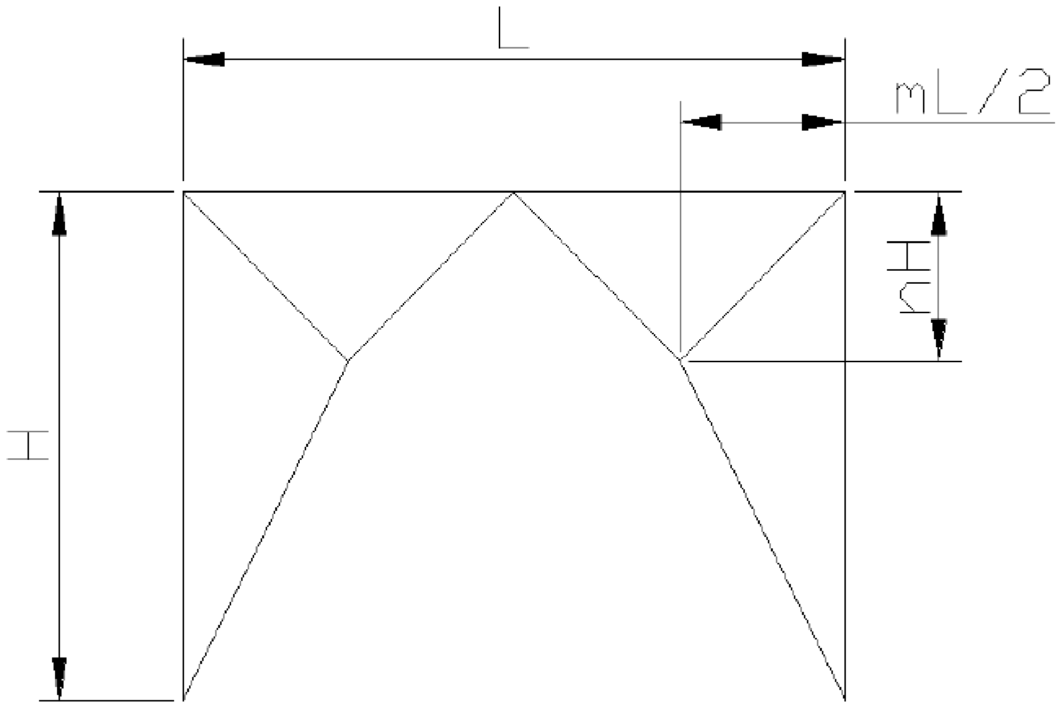


Figure 2.24 Connection point location introduction (Mosalman and Yazdi, 2011)

By using finite element programming plus creating here support (Manual 2009), the stiffness of various joint places in the panel region is investigated. While order to make that study, for various organizations regarding m and n, stiffness of the way based covering Hook's law and the support displacement because of the oblique load is measured plus the events for a support with H/L=3/4 meter are shown in Figure 2.25, anywhere coefficients of E and A is module of elasticity and the cross-section area of bracing components, individually (Hibbeler, 2005; Mosalman and Ramli, 2009).

$$\mathbf{K} = \frac{F}{\Delta} \tag{2.1}$$

Anywhere, K, F, and D same stiffness, lateral force and lateral displacement of the way, individually.

During this research, some combination of the bracing components to the support edges means regarded as a closed contact. Toward some mid connection, there are multiple standards regarding structure, which during this writing stand worn plus evaluation denotes made for their oblique load limit. While represented in the Figure 2.25, since enough as the eccentricity of the contact point goes raised, the stiffness limits, and therefore it is useful for planners to choose the contact point near the diagonal of the frame to have greater stiffness (Mosalman and Yazdi, 2011).

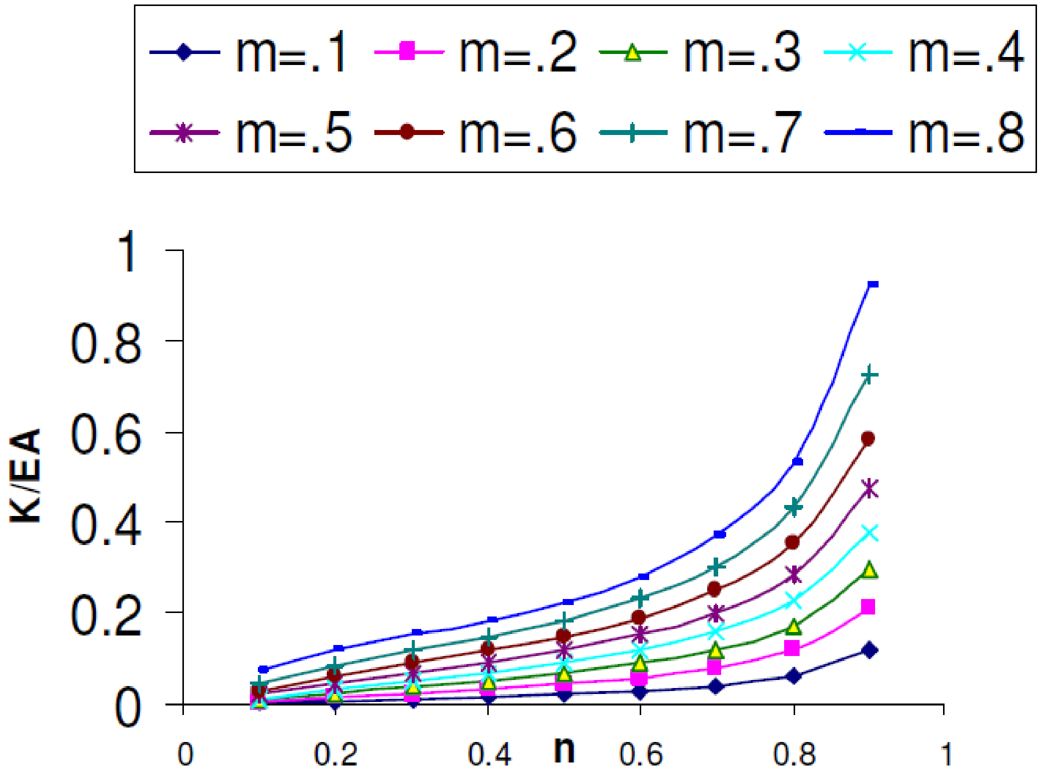


Figure 2.25 Stiffness of gate bracing system in different point (Mosalman and Yazdi, 2011)

CHAPTER 3

RESEARCH METHODOLOGY

3.1 General

In this study, firstly, reinforced concrete (RC) frames were designed for two, four, six, and eight storey. Then, they were assessed to show the nonlinear behavior and performance state. After that, the RC frames were upgraded by adding gate bracing system in eight different configurations such as:

- Configuration 1: partial gate brace at mid bay only.
- Configuration 2: partial gate brace at mid bay without ground floor.
- Configuration 3: partial gate brace at mid bay with ground floor.
- Configuration 4: partial gate brace at two external bays.
- Configuration 5: partial gate brace at two external bays without ground floor.
- Configuration 6: partial gate brace at two external bays with ground floor.
- Configuration 7: full gate brace.
- Configuration 8: full gate brace without ground floor.

In each configuration, the gate bracing system was adopted by considering four different eccentricities e1=0.2, e1=0.4, e1=0.6, and e1=0.8. Thus, the total of 132 different frame models (covering 4 without bracing and 128 with gate bracing) were investigated. Finally, the nonlinear pushover analysis was completed to estimate the seismic behavior of the original RC frames and these with gate bracing cases.

3.2 Case Study Frames

Two, four, six, and eight storey residential RC buildings remained used as a case study to compare the seismic response of the existing and upgraded structures considering the addition of gate bracing systems. The provisions designed the existing buildings ACI 318-63 for gravity loads

only. The buildings were modeled considering a serious regarding planar building joined through every storey level by rigid diaphragms. Hardly 2-D analyses were conducted. The models had the similar height of 3.3 m at each storey with three bays in both X direction plus Y direction of 5.5 m spacing. The structures were considered as regular in shape and symmetric in plane in order to carry out the analysis on two-dimensional models which ease the interpretation of the results of analysis. Typical floor plan and elevation of the case study RC buildings are given in Figures 3.1 and 3.2, respectively.

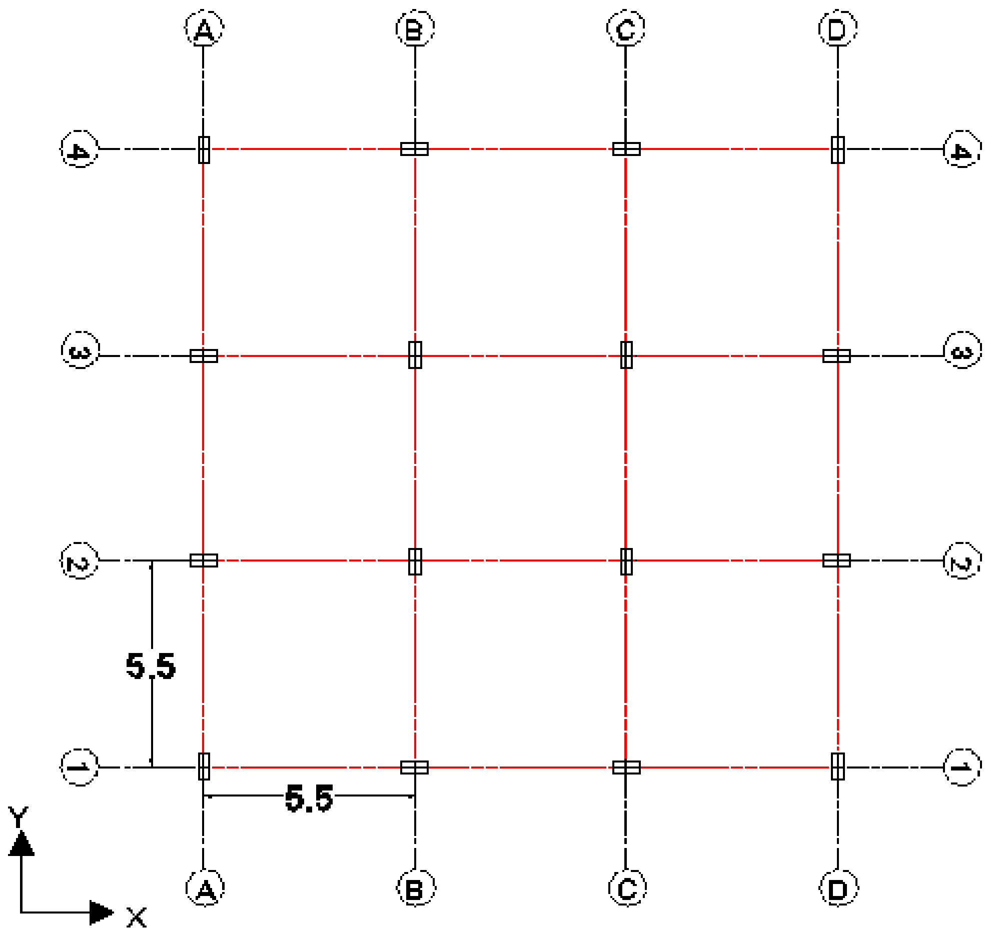


Figure 3.1 Plan view of the existing RC building for all cases (2, 4, 6, and 8 storey)

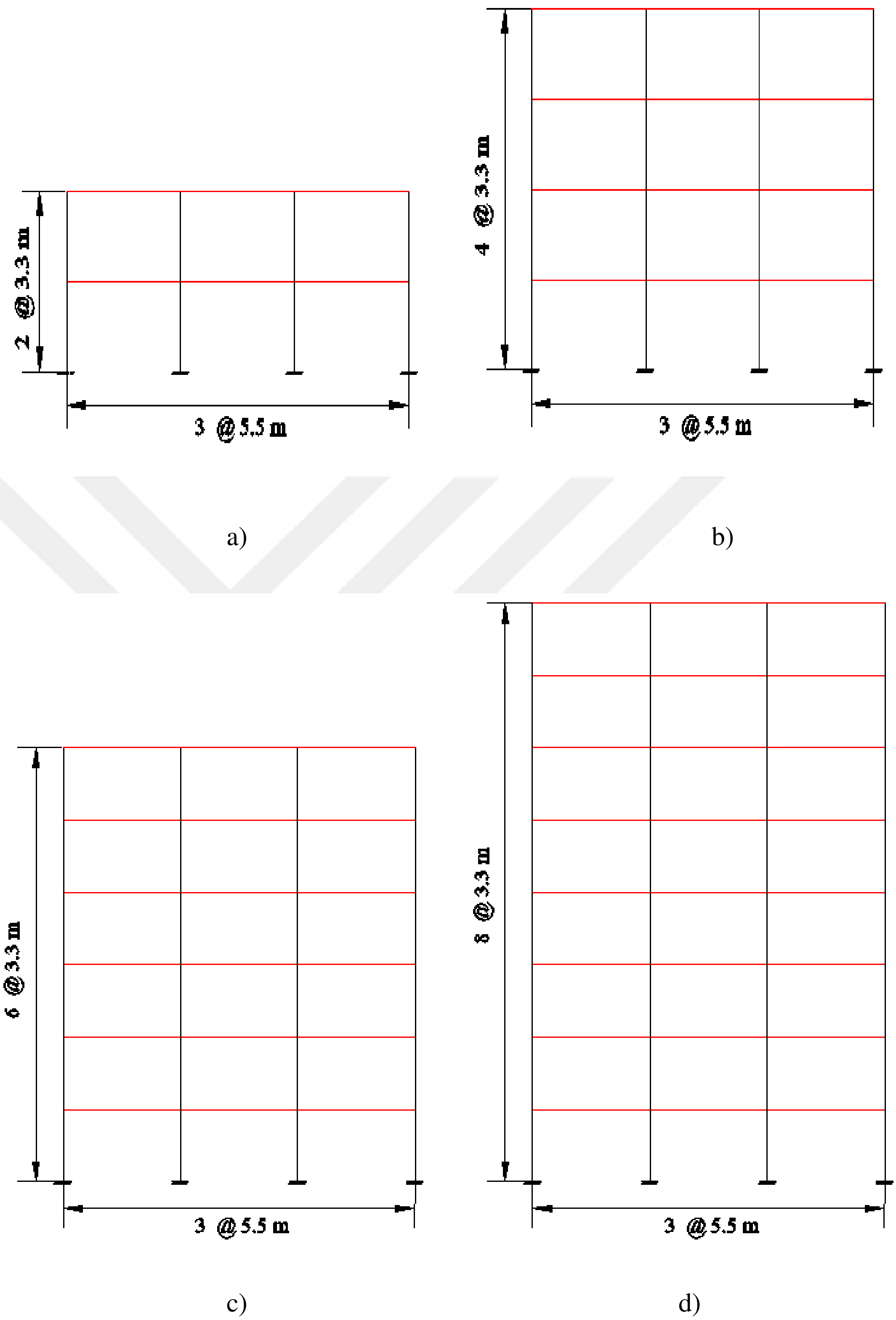


Figure 3.2 Frame elevation of the existing building x-z axes in (a) 2 stores, (b) 4 storey, (c) 6 storey, and (d) 8 storey

The dimensions of the beams were 60 cm in height and 30 cm in width for all buildings it is the same dimensions in the exterior and interior side. The exterior and interior frames of the buildings comprised three bays. The largest dimension of the columns designed toward external axes, nevertheless, the short direction of the columns designed parallel to the X-direction. At two-storey building, the dimensions of the columns were the same dimensions (20x50 cm) at all storeys. The dimension of columns for four-storey building were same also (25x60 cm). For the six-storey buildings, the dimensions of the columns were different for each storey they were same at the first to third storey and varied from fourth to sixth storey as shown in Table 3.1, also for eight storey building, they were varied from first to third and from fourth to sixth and also from seventh to eighth as shown in Table 3.2. The building was located on Z3 class soil and first seismic zone. The design live load and additional dead weight concerning the building were considered as 2.50 kN/m^2 and 3.50 kN/m^2, respectively.

Table 3.1 Dimensions of the columns in the 6 storey buildings

Story level	1	2	3	4	5	6
Dimension of columns (cm)	25x60	25x60	25x60	20x50	20x50	20x50

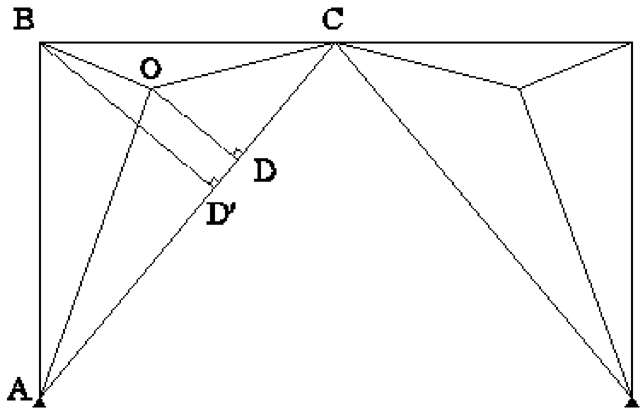
Table 3.2 Dimensions of the columns in the 8 storey buildings

Story level	1	2	3	4	5	6	7	8
Dimension of columns (cm)	30x70	30x70	30x70	25x60	25x60	25x60	20x50	20x50

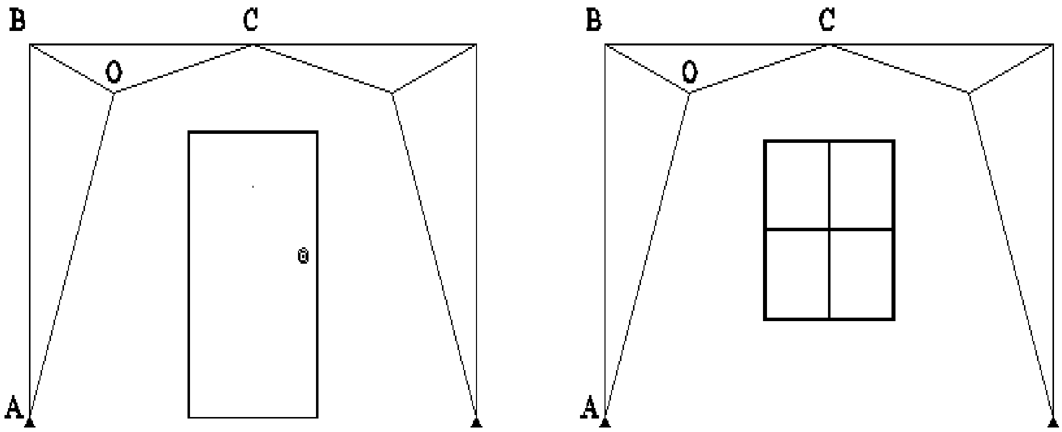
For upgrade the seismic review of the present building, the gate bracing system was used. The gate bracing system contains six segments that remain regular, and the oblique portion is not straight having eccentricities denoted as e1 and e2 as shown in Figure 3.3. The eccentricity parameter e1 was defined as the ratio of OD length to BD' length, similarly, the eccentricity parameter e2 was described as the ratio of AD length to AC length. In this study, the eccentricity of e2 was kept constant as 0.6 (e2=0.6) while that of e1 varied from 0.2 to 0.8. (e1= 0.2, e1=0.4, e1=0.6, and e1=0.8). As a bracing member, circular hollow section (CHS) was used. In the current study, same brace member diameter was used and kept constant for all cases 100 mm. The wall thickness of the section was 4 mm. The yield stress regarding the steel was 365 MPa. These compressive strengths of concrete was taken as 21 MPa. The beam reinforcement for top was 600 mm² and that for bottom was 800 mm² while the stiffness modifier for beam was 0.35 and that for column was 0.7. Load case formula, DL+0.3 LL was taken. The properties of frame systems are shown in Table 3.3. The creation views of the building frames with eight different configurations of gate bracing are illustrated in Figures 3.4 - 3.11.

Table 3.3 Properties of frame systems

Properties of frame	
fc'	21 MPa
fy	365 MPa
Top beam reinforcement	600 mm²
Bottom beam reinforcement	800 mm²
Load case	DL+0.3L.L
Column stiffness modifier	0.7
Beam stiffness modifier	0.35



a)



b)

Figure 3.3 Frame with gate bracing (e1=OD/BD' and e2=AD/AC) and possible openings

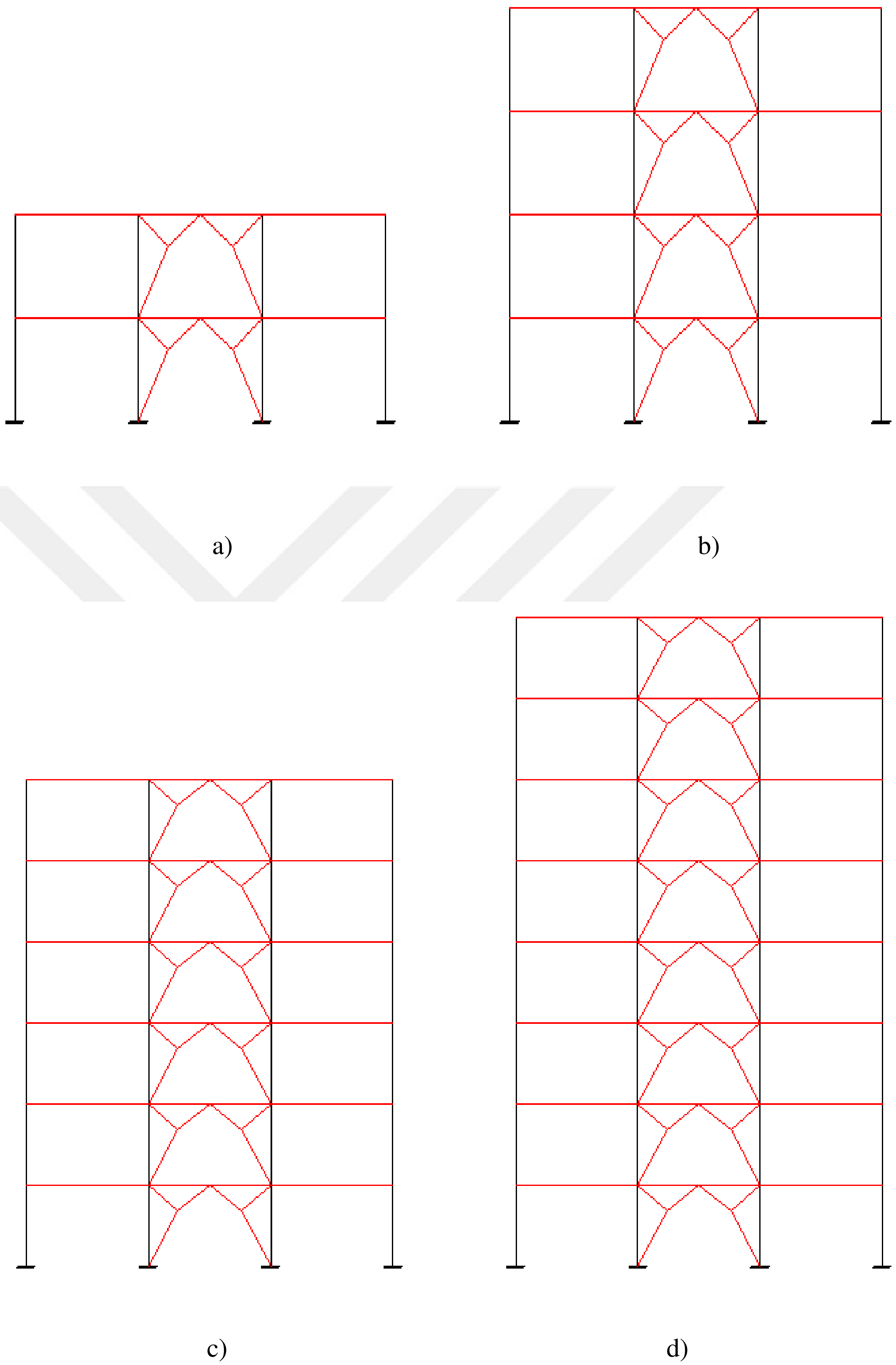


Figure 3.4 Configuration 1 (partial gate brace at mid-bay only) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey

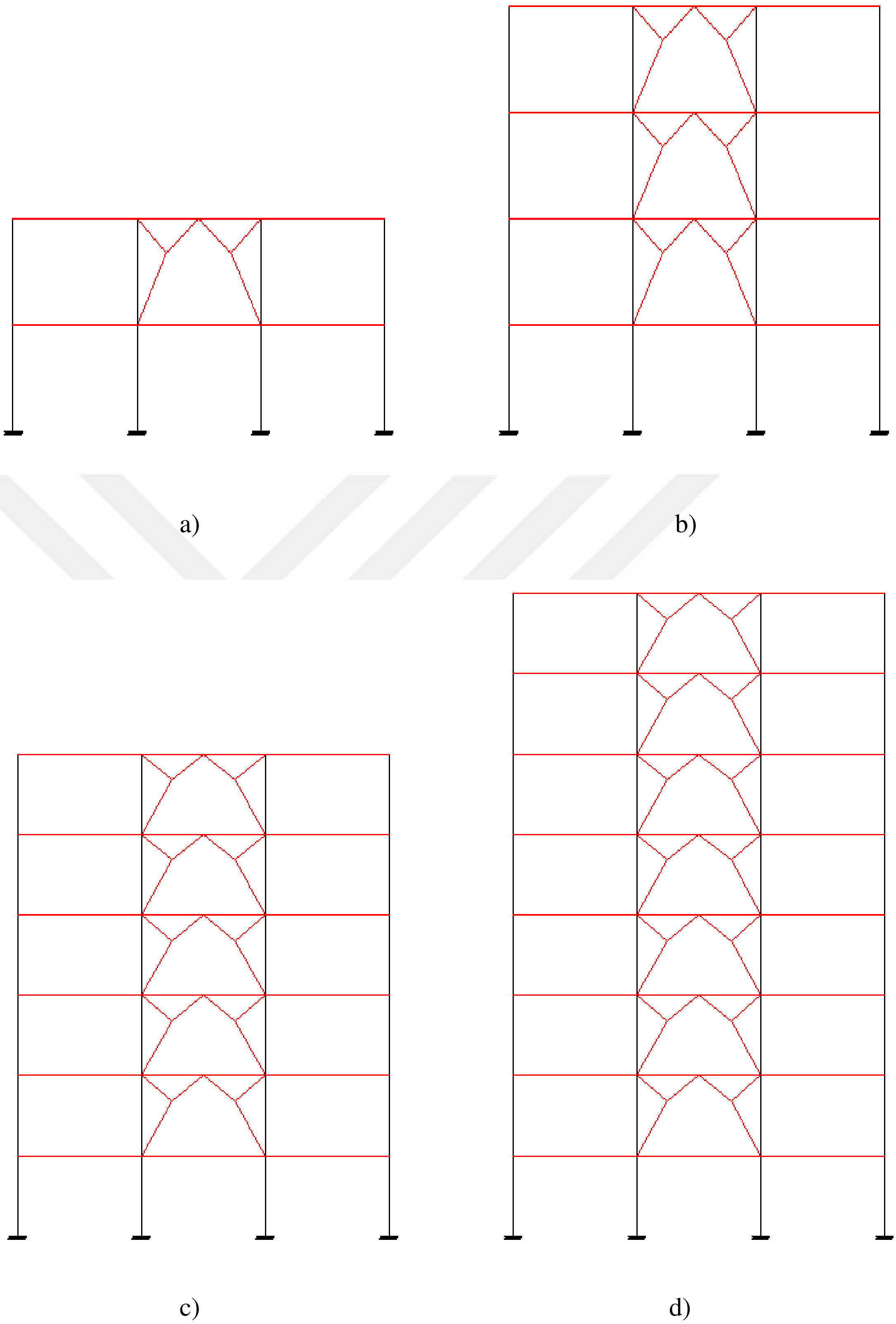
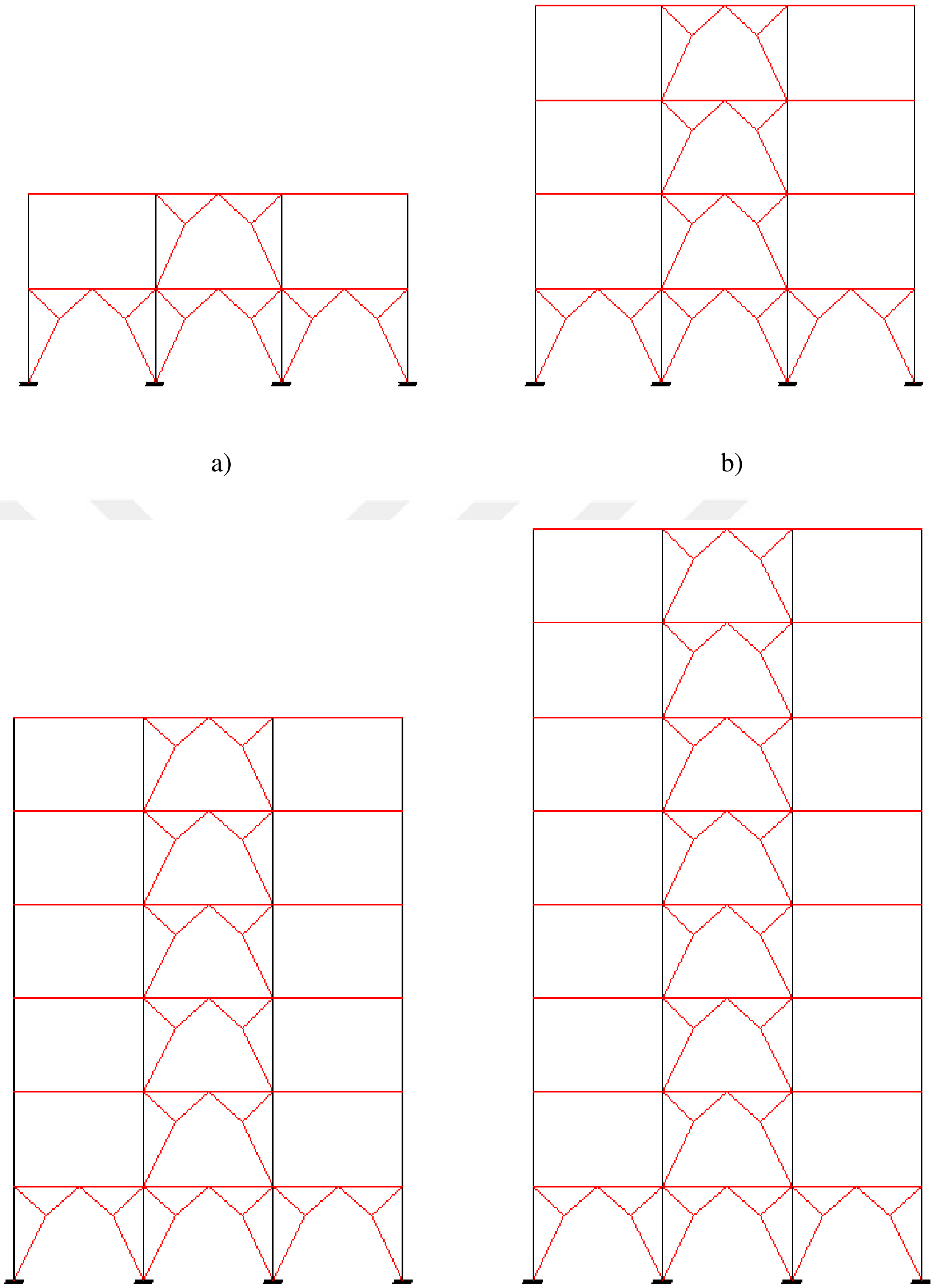


Figure 3.5 Configuration 2 (partial gate brace at mid-bay without ground floor) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey



a) b) c) d)

Figure 3.6 Configuration 3 (partial gate brace at mid-bay with ground floor) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey

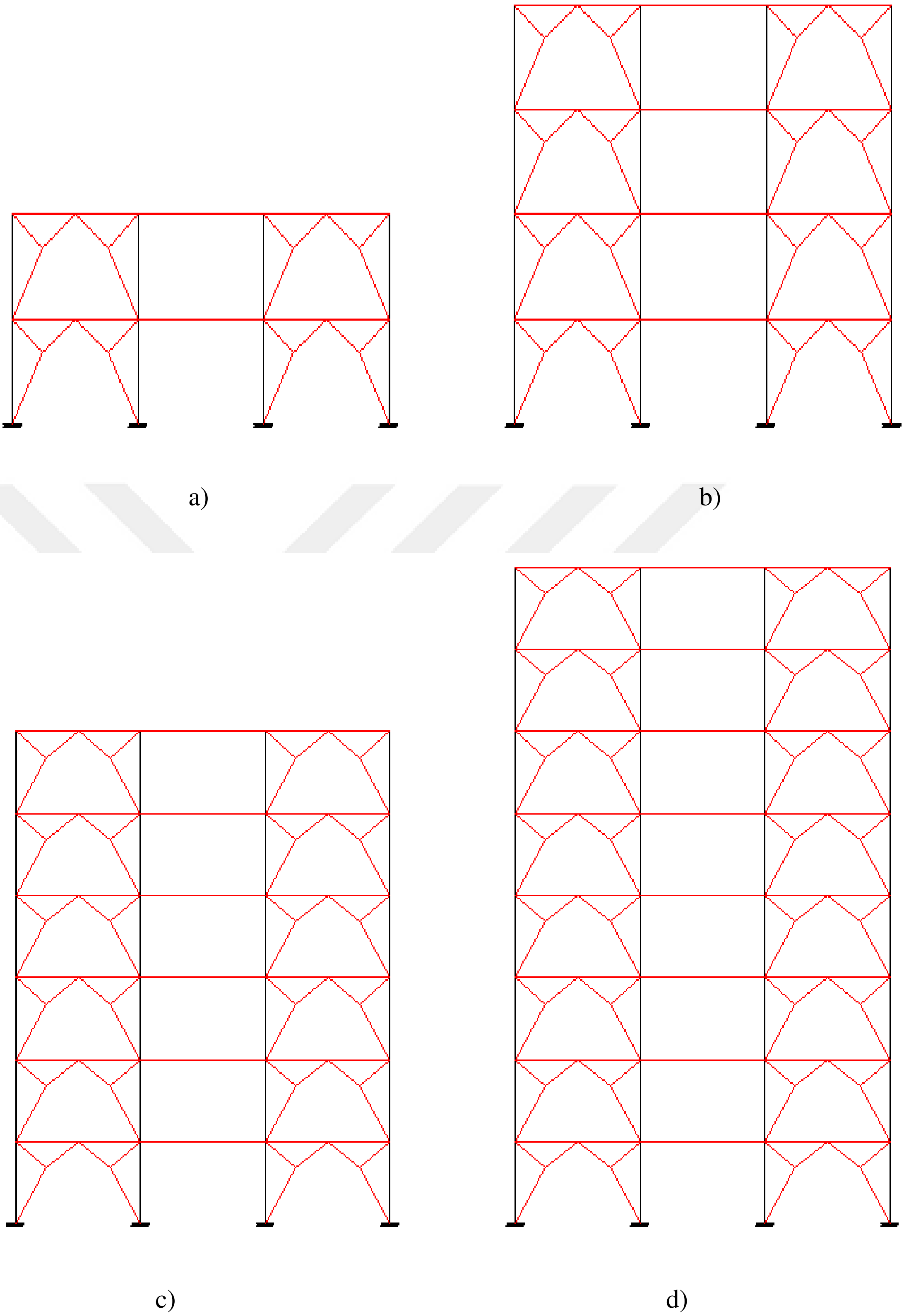
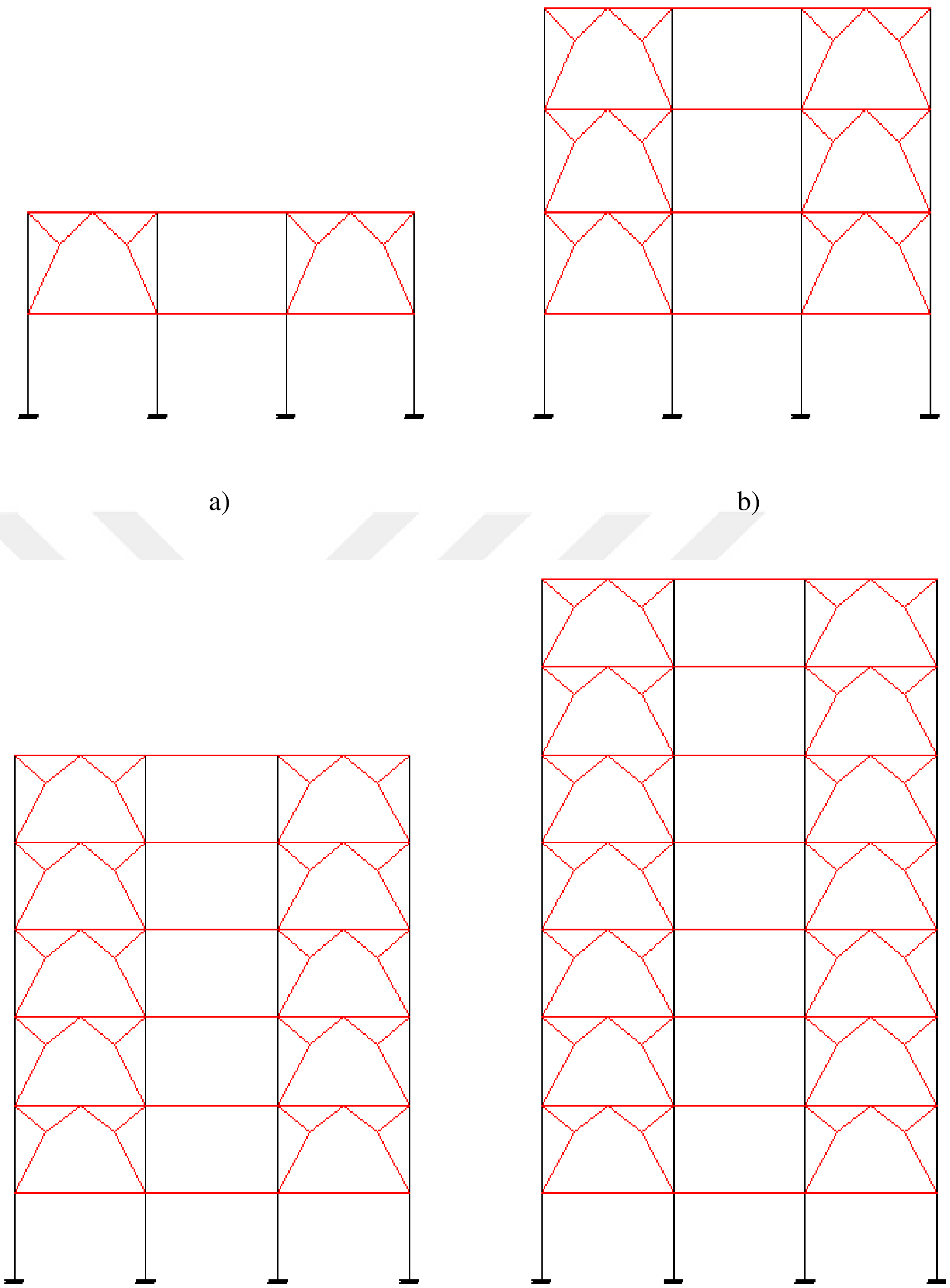


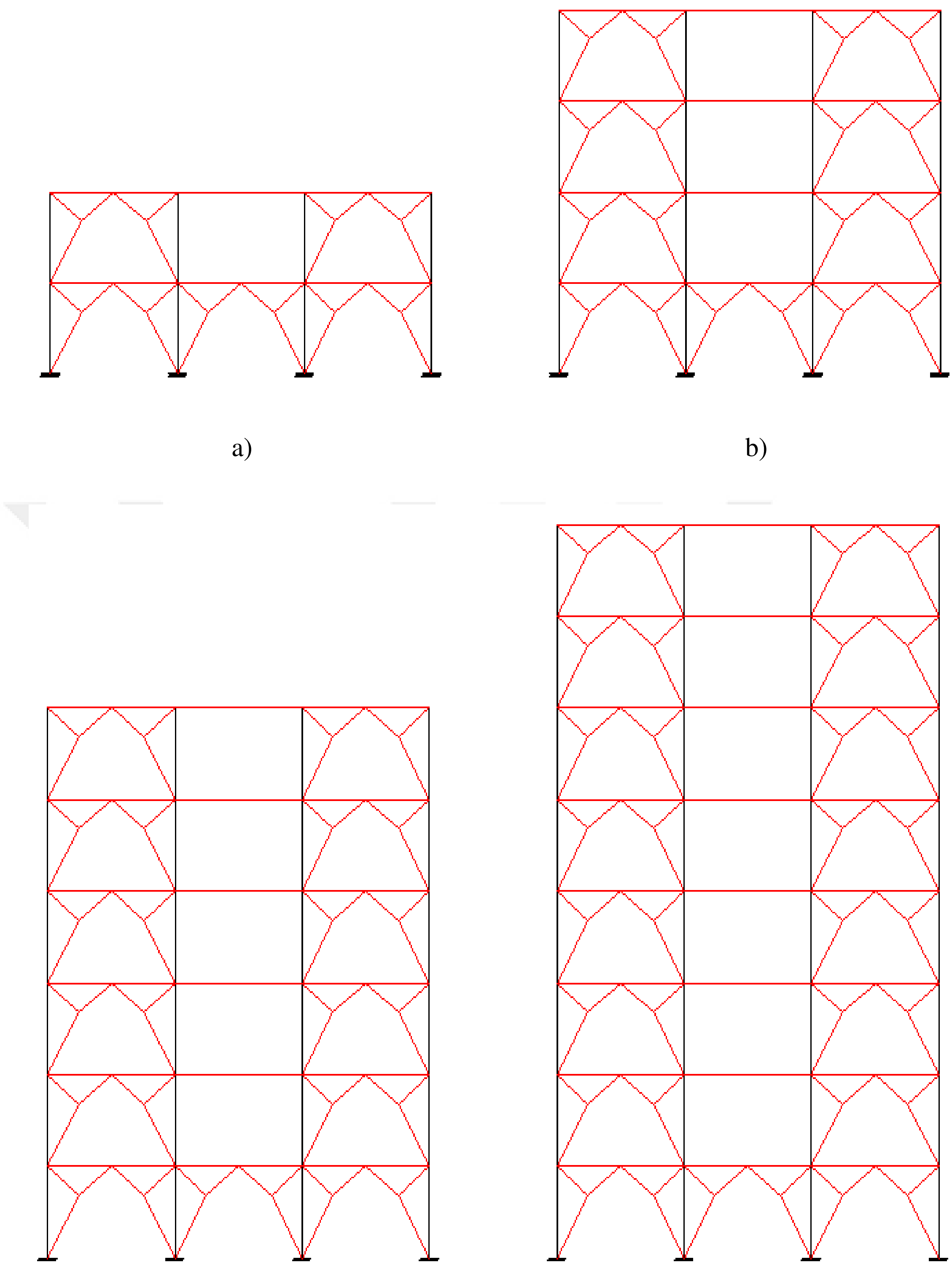
Figure 3.7 Configuration 4 (partial gate brace at two external bays) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey



a) b)

c) d)

Figure 3.8 Configuration 5 (partial gate brace at two external bays without ground floor) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey



a) b) c) d)

Figure 3.9 Configuration 6 (partial gate brace at two external bays with ground floor) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey

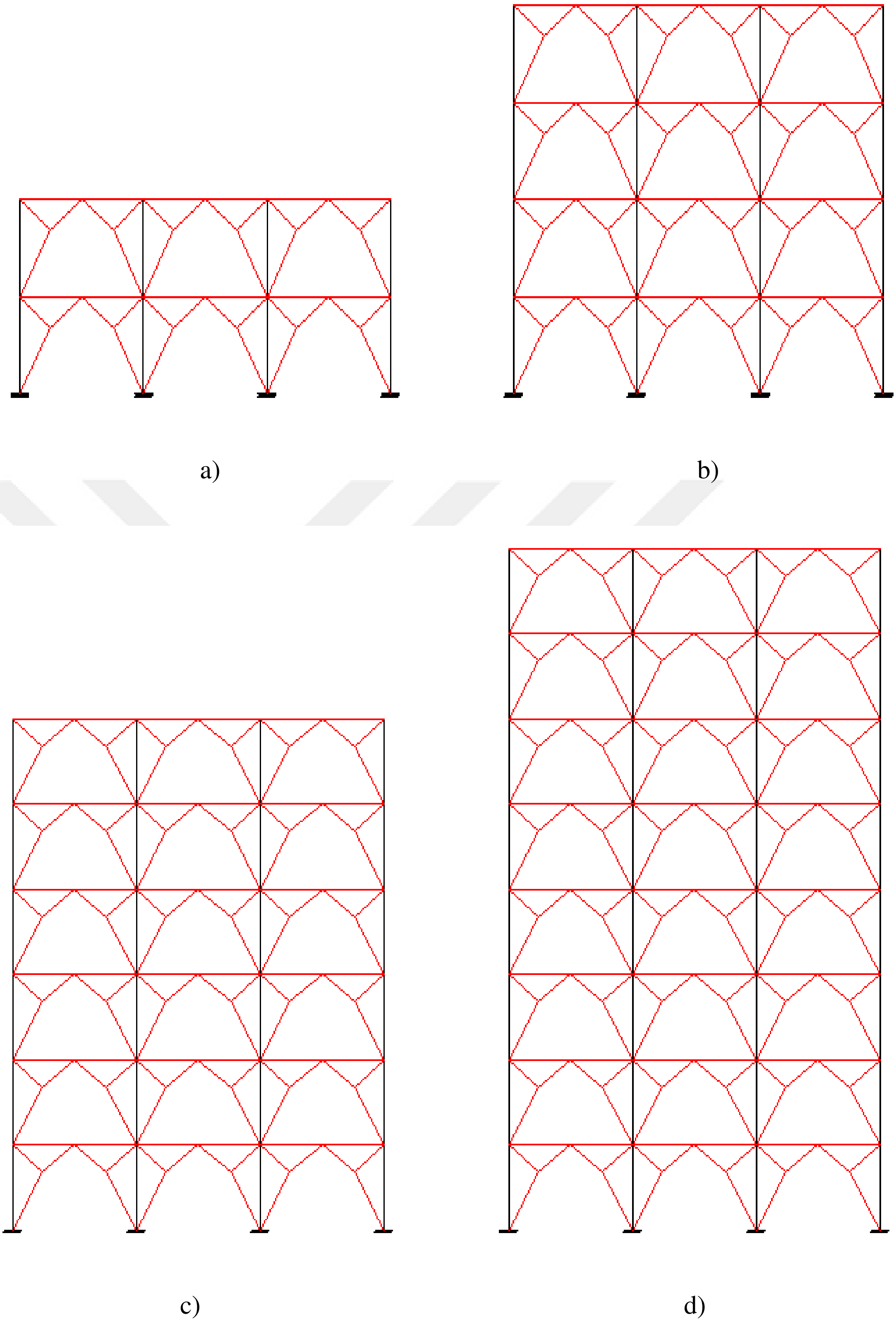
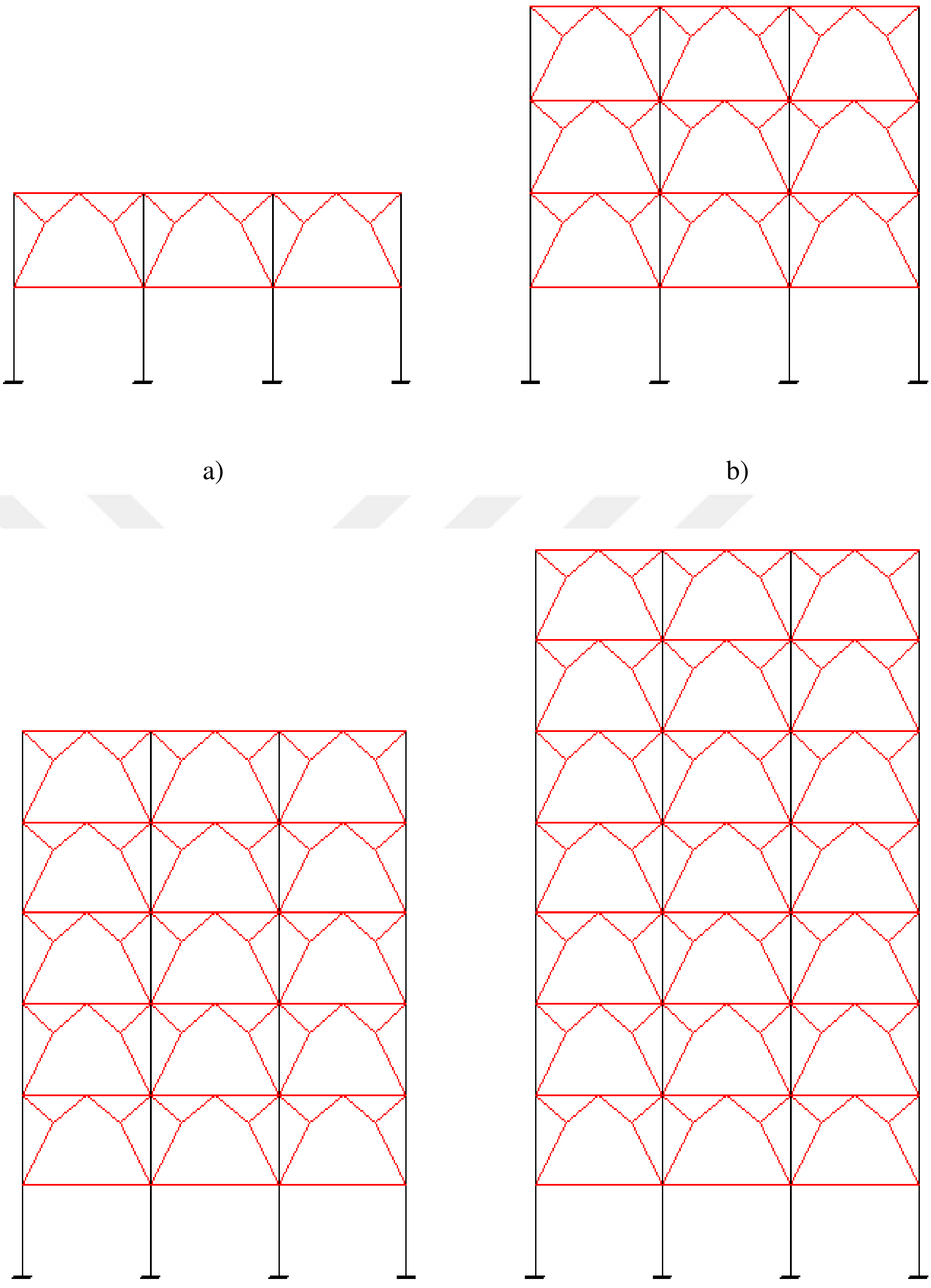


Figure 3.10 Configuration 7 (full gate brace) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey



c) d)

Figure 3.11 Configuration 8 (full gate brace without ground floor) for: a) 2 storey, b) 4 storey, c) 6 storey, and d) 8 storey

3.3 Nonlinear Analysis

The static pushover analysis remains each relative plus partial easy, medium suspension to the difficult obstacle of divining strength and deformation requests inflicted covering structures plus their components by earth movement. The analysis applications and static are essential terms of a pushover. Acknowledging this limitation, the task is to perform an evaluation procedure that is relatively simple but captures the necessary features that significantly affect the performance target. It is in this context that research like the one carried is needed, to improve the reliability and the quality of the method. The lateral load resisting system applied for a steel perimeter frame structure as per two-dimensional frame application (Barros and Almeida, 2005).

The aim of applying pushover analysis is mainly to support the over strength ratio values, to distribute harm and determine the expected plastic mechanism, to evaluate the structural performance of retrofitted or existed buildings, as an alternative for the designing which depends on linear analysis (Hassaballa et al., 2014).

The pushover analysis for the structures is defined as the nonlinear static analysis beneath lasting vertical loads and improving in sidelong loads progressively. A curve for the top displacement versus base shear in a building is obtained by the pushover analysis that could detect any premature rustiness or failure. The pushover analysis also specified weakness or rustiness in the structure. The ATC-40 and FEMA-356 reports become established modeling techniques, analysis modes, and recognition tests for the pushover analysis. These reports described the approval criteria depending on the flexible hinge turn by holding different production levels. A pushover analysis is carried out by reducing a structure to the monotonically rising guide of oblique loads; these weights represent the inertial forces tested by the structure while structure subjected to an earthquake effect. In this study, SAP2000 computer software (nonlinear version 18) is used to bring out some pushover analyses. While making the pushover analysis of a building structure, the following actions are taken:

- The properties of 2, 4, 6, and 8 storey buildings were created and assigned in SAP2000.
- Hinge properties were defined giving to FEMA 356 (2000), and these properties are assigned at each end of the beam and column members. For the column

members, axial force and biaxial moment hinges (PMM) and for the beams, flexural moment hinges (M3) were considered.

- The pushover load case was defined using gravity load, and then succeeding lateral pushover load cases were specified to start from the final conditions of the gravity pushover.

- From this analysis, the capacity curves and performance level of each structure were established.

In common, five points considered (A, B, C, D, and E) represent the force-deflection performance of the hinge as shown in Figure 3.12. Point A leads to an unloaded situation and point B serves yielding of the element. The ordinate at C replies to nominal strength and D corresponds to the deformation at which substantial strength degradation starts. Three points termed LO, LS, and CP represented immediate occupancy, life safety, and collapse prevention, respectively. The principles for assigning values for each of these points differ liable on the form and type of member such many other parameters defined in the ATC-10 and FEMA-356 documents (Bansal, 2011).

SAP2000 allowed the distribution of lateral forced used in the pushover analysis to be based on a similar acceleration in a specified direction, a detailed mode shape, or a user-defined static load case. After this procedure, the structures were subjected to monotically increasing lateral forces so the structure was pushed until a collapse mechanism established. The capacity curves related to base shear versus roof displacement for 2, 4, 6, and 8 storeys for original and retrofitted structures were achieved.

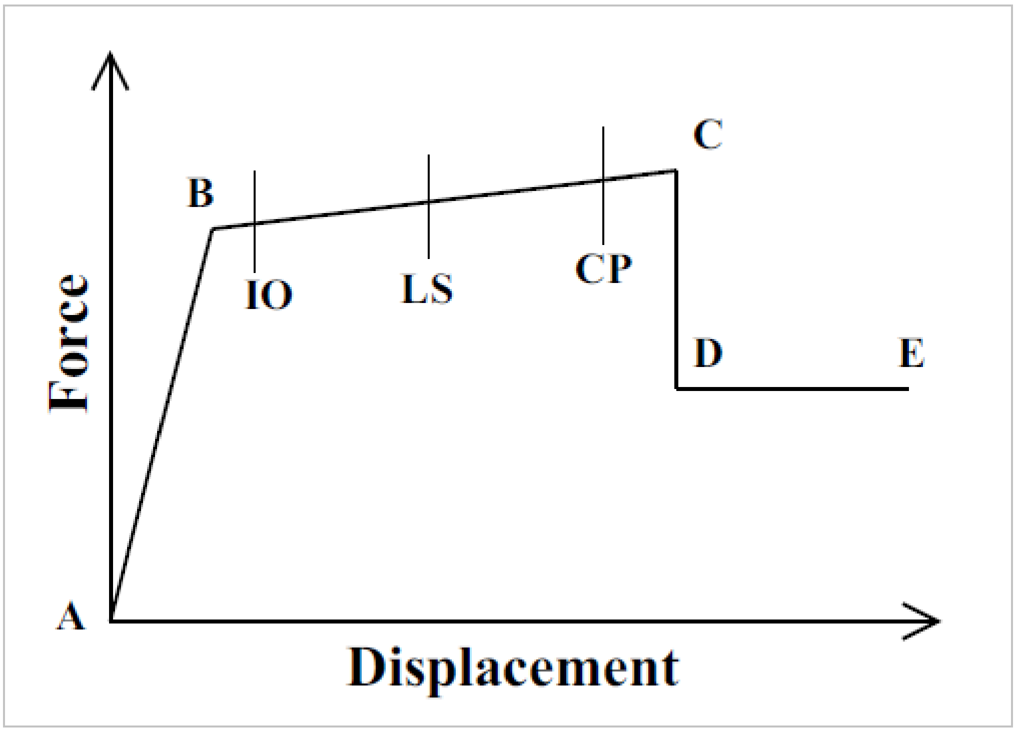


Figure 3.12 Force- deformation for pushover hinge (Bansal, 2011)

CHAPTER 4

RESULTS AND DISCUSSION

4.1 General

The analysis of results for the existing buildings and upgraded buildings with gate bracing system were provided and discussed comparatively in this chapter. As previously mentioned, different cases were considered and structural performance of 2, 4, 6, and 8 storey buildings before and after using the bracing under the effect of lateral loading were evaluated. As a bracing, eight different configurations of gate bracing system were used. In addition, four different gate bracing arrangements with an eccentricity of 0.2, 0.4, 0.6, and 0.8 were taken into account in this study.

4.2 Comparison of Capacity Curves of Existing and Gate braced Buildings

The capacity curves (pushover curves) were evaluated for 2, 4, 6, and 8 storey buildings before and after retrofitting. The findings obtained from the analysis are given below.

4.2.1 Two Storey Buildings

For 2 storey building, Figure 4.1 displays the capacity curve for the original building and the upgraded building with gate bracing in configuration 1 (partial gate brace at mid bay only). The analysis of the results showed that in general the building retrofitted with bracing system (gate braced frame) gave greater lateral load carrying capacity than the existing building (without brace). For example, of the existing building, the maximum base shear was found as 1160.5 kN while the maximum base shear for the building with gate bracing were evaluated as 1584.3 kN, 1596.9 kN, 1513.5 kN, and 1382.7 kN at different eccentricity parameters of 0.2, 0.4, 0.6, and 0.8, respectively (as seen in Figure 4.1). This indicated that depending upon the eccentricity of the gate bracing used, the braced buildings had about 1.19-1.37 times higher capacity than the existing building.

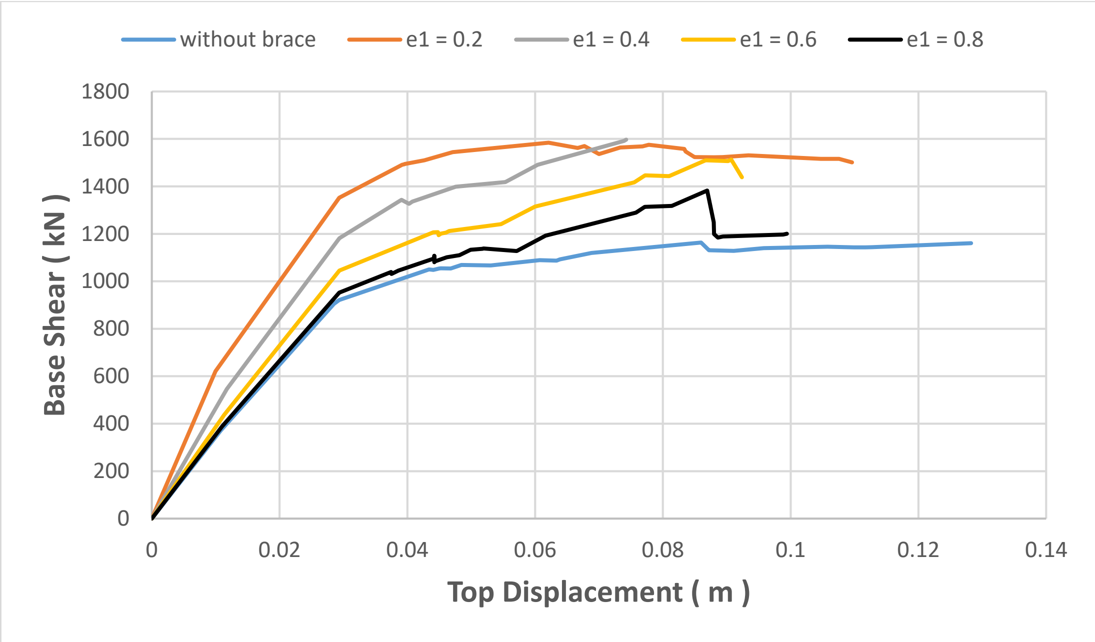


Figure 4.1 Capacity curves of 2 storey at configuration 1 (partial gate brace at mid bay only) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

By the same way, as shown in Figure 4.2, for the upgraded building with gate bracing at configuration 2 (partial gate brace at mid bay without ground floor), the maximum base shear values were 1176.8 kN, 1181.4 kN, 1239.8 kN, and 1163.2 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively. During this case, the lateral load carrying capacity of the existing building increased about 1.002-1.06 times due to the inclusion of bracing. Figure 4.3 demonstrates the capacity curve of the upgraded building with gate bracing at configuration 3 (partial gate brace at mid bay with ground floor). The analysis of the results reveals that the value of the maximum base shear was computed as 2413.4 kN, 2167.6 kN, 1832.7 kN, and 1384.2 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively. This corresponded to 1.19-2.08 times greater capacity than the existing building.

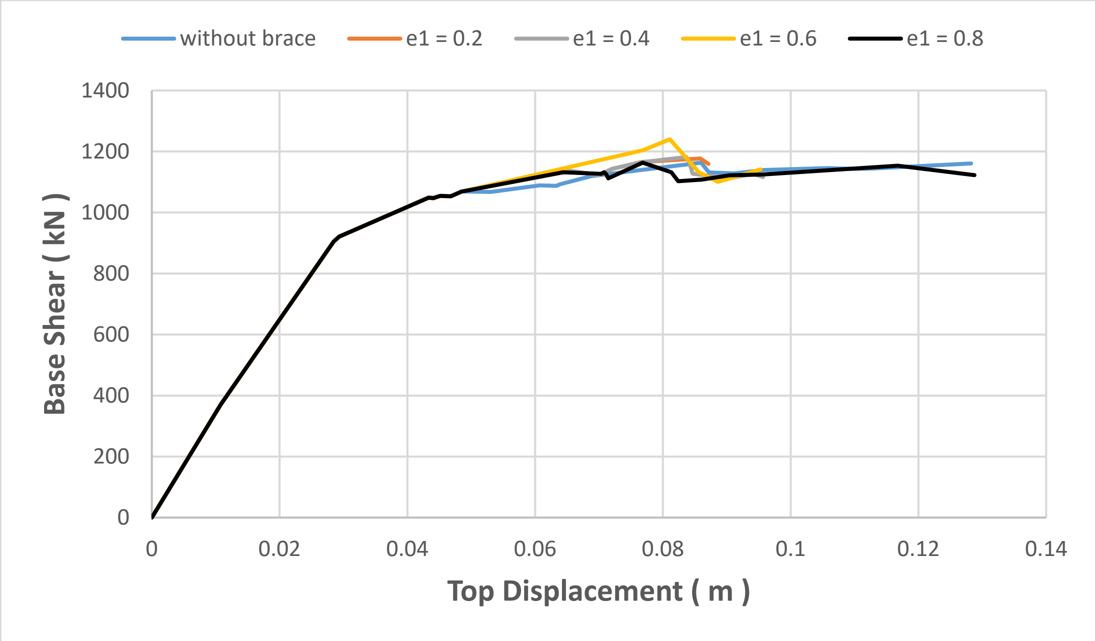


Figure 4.2 Capacity curves of 2 storey at configuration 2 (partial gate brace at mid bay without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

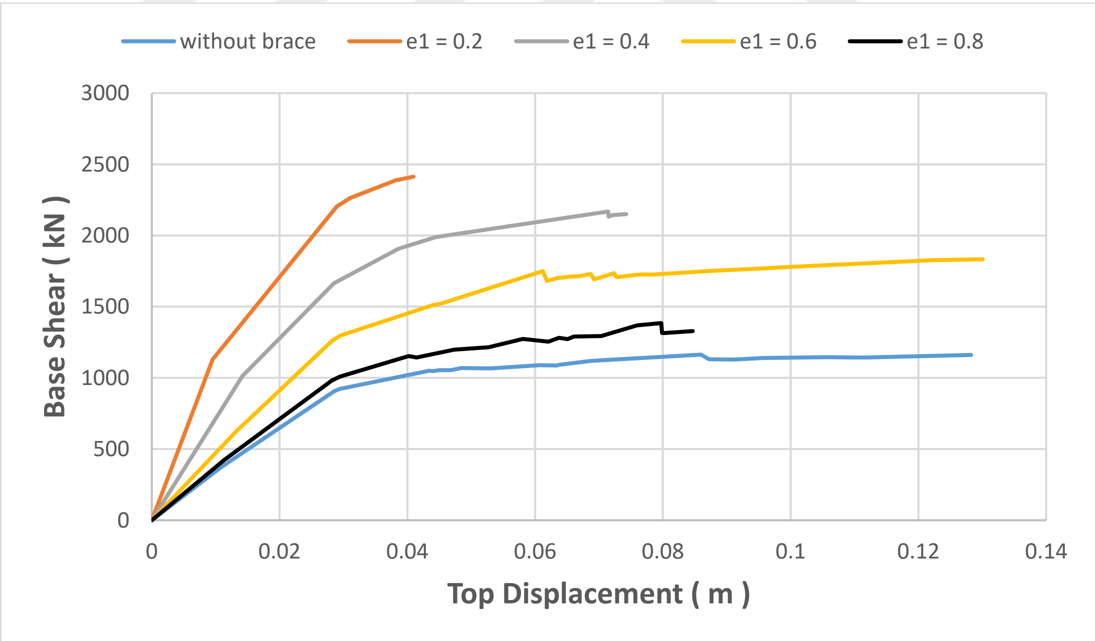


Figure 4.3 Capacity curves of 2 storey at configuration 3 (partial gate brace at mid bay with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

In the case of the retrofitted building with gate bracing at configuration 4 (partial gate brace at two external bays) as shown in Figure 4.4, the maximum base shear at 0.2, 0.4, 0.6 and 0.8 eccentricities were calculated as 1935.3 kN, 1878.2 kN, 1582.0 kN, and 1327.0 kN, respectively. This implied about 1.14-1.66 times increment owing to the utilization of the bracing.

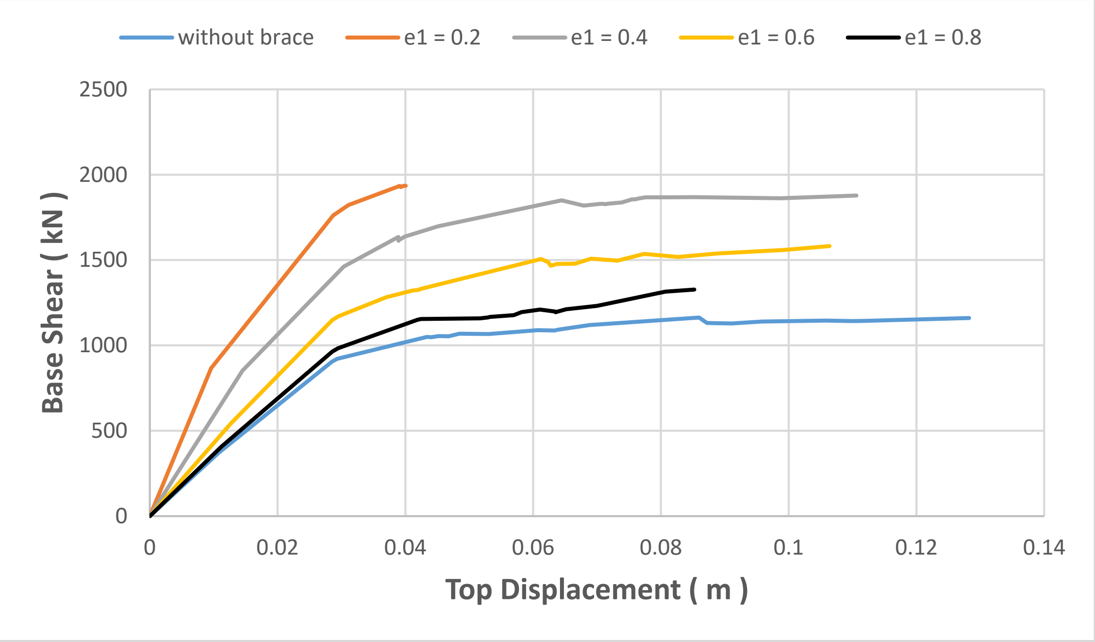


Figure 4.4 Capacity curves of 2 storey at configuration 4 (partial gate brace at two external bays) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

For the upgraded building with gate bracing at configuration 5 (partial gate brace at two external bays without ground floor) (Figure 4.5), the maximum base shear at 0.2, 0.4, 0.6, and 0.8 eccentricities were 1252.4 kN, 1184.3 kN, 1179.1 kN, and 1189.2 kN, respectively. This brought about 1.02-1.08 times increment. As also seen in Figure 4.6 for the case of gate brace at configuration 6 (partial gate brace at two external bays with ground floor), the maximum base shear value was 2412.7 kN at 0.2 eccentricity, 2167.9 kN at 0.4 eccentricity, 1795.1 kN at 0.6 eccentricity, and 1376.7 kN at 0.8 eccentricity. It was clearly noted that the use of the gate bracing at configuration 6 to upgrade the seismic response of the existing building resulted in an increase of about 1.18-2.08 times.

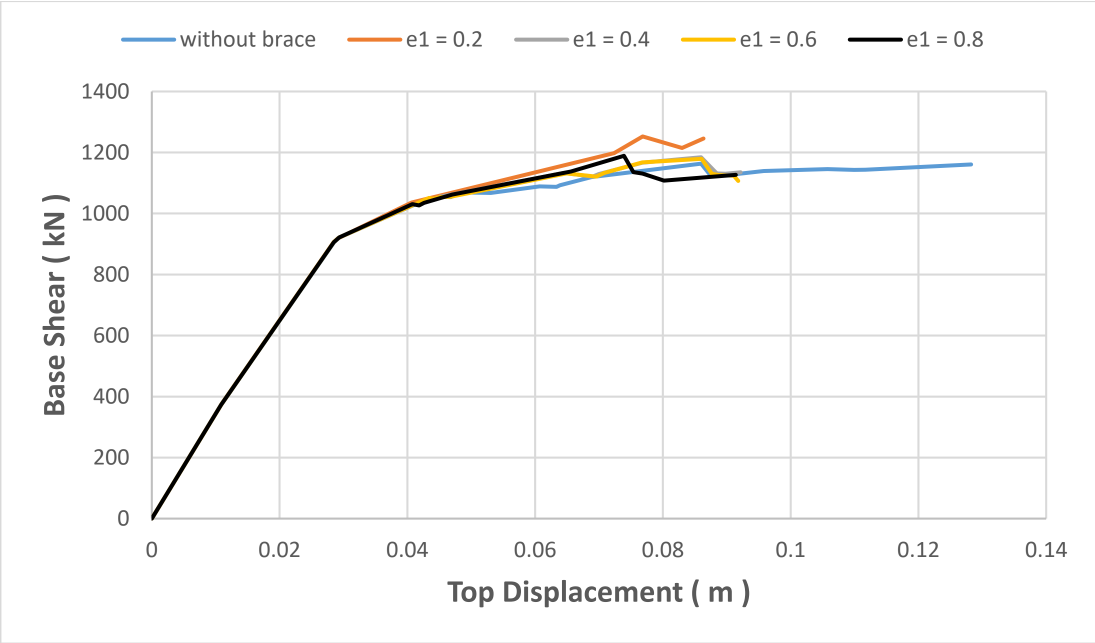


Figure 4.5 Capacity curves of 2 storey at configuration 5 (partial gate brace at two external bays without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

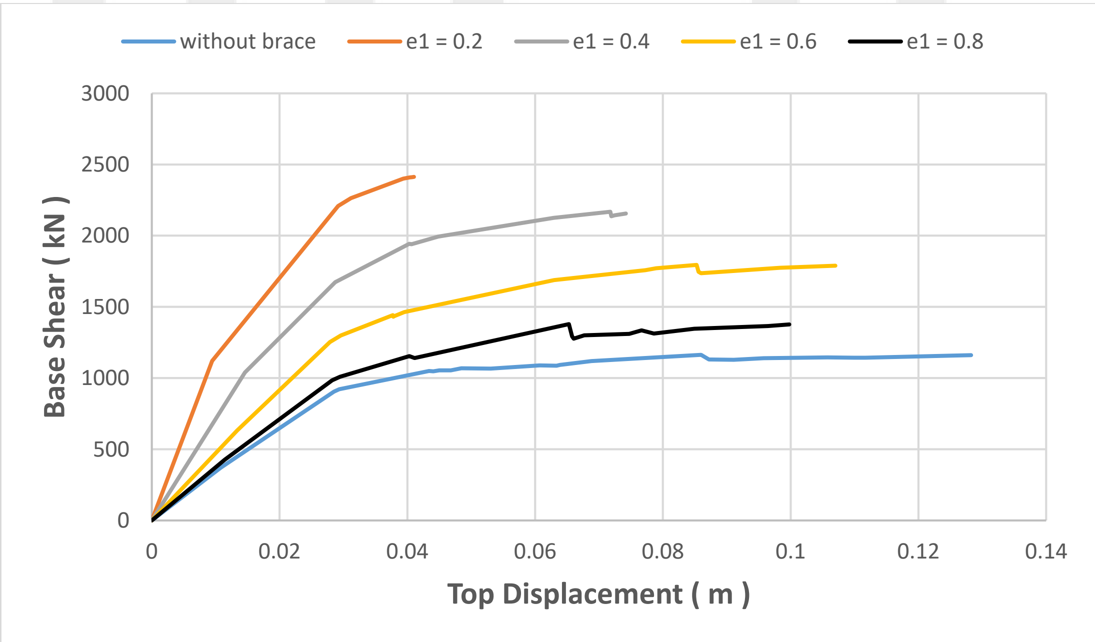


Figure 4.6 Capacity curves of 2 storey at configuration 6 (partial gate brace at two external bays with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

Furthermore, the building with gate bracing at configuration 7 (full gate brace) (Figure 4.7) had the maximum base shear of 2418.2 kN, 2149.1 kN, 1836.7 kN, and 1426.0 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively while that with gate bracing at configuration 8 (full gate brace without ground floor) (Figure 4.8) possessed the maximum base shear of 1253.8 kN, 1181.7 kN, 1181.4 kN, and 1165.3 kN, respectively. This indicted about 1.23-2.08 and 1.004-1.08 times increment in seismic performance of the existing building due to the use of gate brace at configuration 7 and gate brace at configuration 8, respectively.

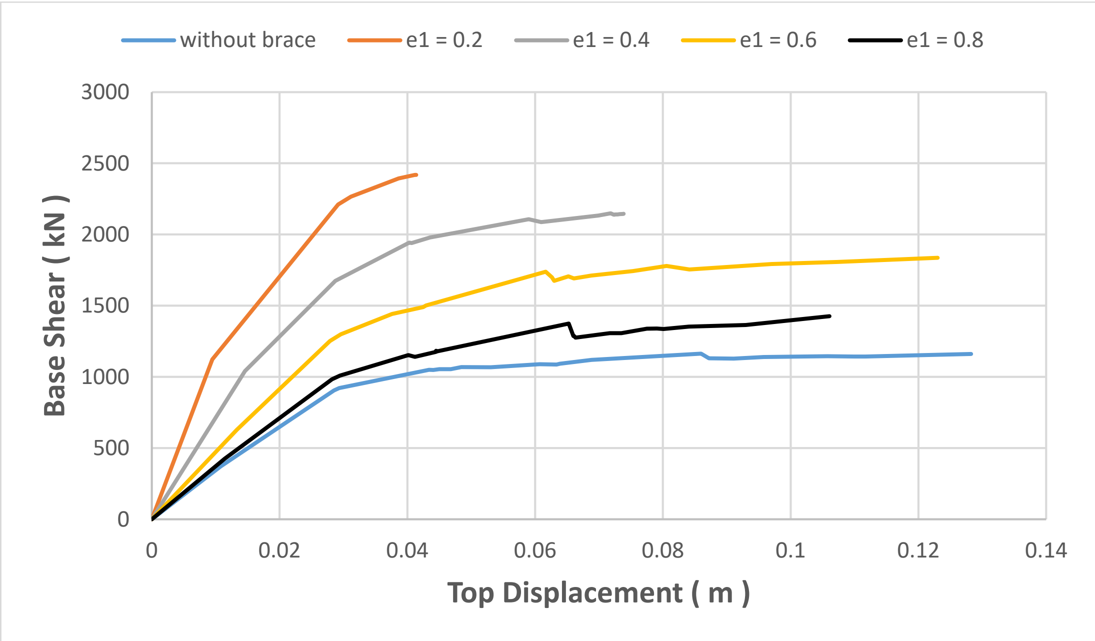


Figure 4.7 Capacity curves of 2 storey at configuration 7 (full gate brace) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

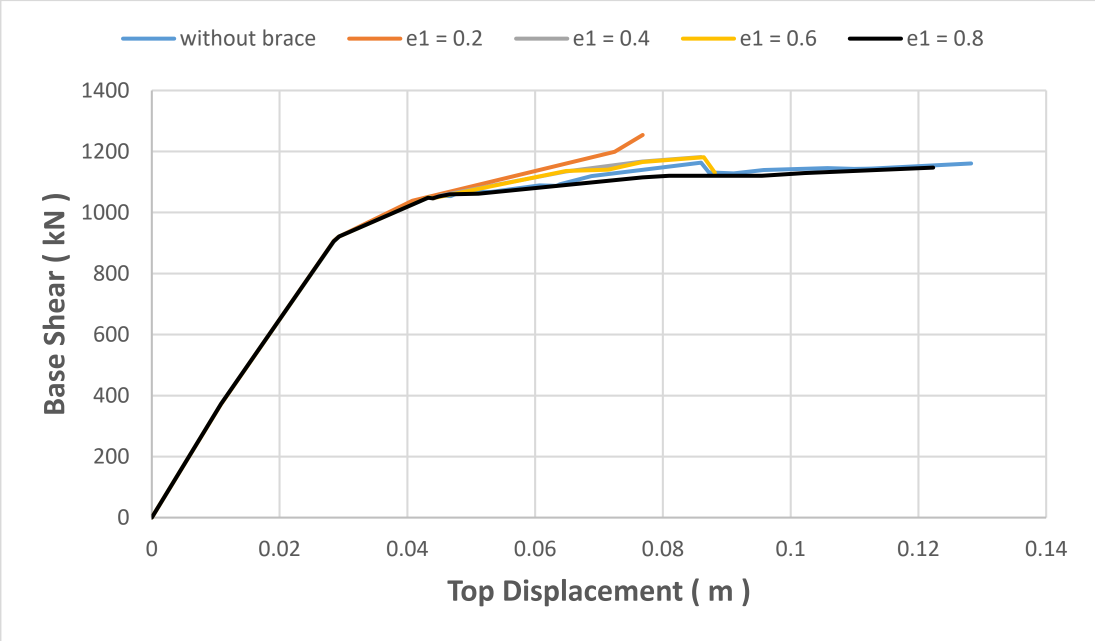


Figure 4.8 Capacity curves of 2 storey at configuration 8 (full gate brace without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

4.2.2 Four Storey Buildings

The base shear V with roof displacement relationships received by the pushover analysis for four storey original and upgraded buildings are presented in Figures 4.9 to 4.16 at 8 different configurations. Figure 4.9 displays the capacity curve for the original building and the upgraded building with gate bracing in configuration 1 (partial gate brace at mid bay only). The maximum base shear was about 842.4 kN for the existing building whereas the maximum base shear for the retrofitted building were 1277.2 kN, 1232.7 kN, 1082.5 kN, 871.5 kN depending mainly on the amount of eccentricity (e_1=0.2, 0.4, 0.6, and 0.8 respectively). This showed about 1.03-1.52 times increment in the capacity of the upgraded building.

Similarly, as shown in Figure 4.10, for the upgraded building with gate bracing at configuration 2 (partial gate brace at mid bay without ground floor), the maximum base shear values were 1127.2 kN, 1048.3 kN, 1015.3 kN, and 842.9 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively. In this case, the lateral load carrying capability of the existing building increased about 1.001-1.34 times due to the inclusion of bracing. Figure 4.11 demonstrates the capacity curve of the upgraded building with gate bracing at configuration 3 (partial gate brace at mid bay with ground floor). The analysis of the results reveals that the value of the maximum base shear was computed as 1833.7 kN, 1446.1 kN, 1294.7 kN, and 978.3 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively. This corresponded to 1.16-2.18 time's greater capacity than the existing building.

In the case of the upgraded building with gate bracing at configuration 4 (partial gate brace at two external bays) as shown in Figure 4.12, the maximum base shear at 0.2, 0.4, 0.6 and 0.8 eccentricities were calculated as 1860.4 kN, 1626.5 kN, 1239.6 kN, and 973.4 kN, respectively. This implied about 1.15-2.21 times increment owing to the utilization of the bracing.

For the upgraded building with gate bracing at configuration 5 (partial gate brace at two external bays without ground floor) (Figure 4.13), the maximum base shear at 0.2, 0.4, 0.6, and 0.8 eccentricities were 1173.0 kN, 1149.2 kN, 1062.8 kN, and 905.4 kN, respectively. This brought about 1.07-1.39 times increment. As also seen in Figure 4.14 for the case of gate bracing at configuration 6 (partial gate brace at two external bays with ground floor), the maximum base shear value was 2108.2 kN at 0.2 eccentricity, 1660.4 kN at 0.4 eccentricity, 1347.7 kN at 0.6 eccentricity, and 1007.4 kN at 0.8 eccentricity. It was clearly noted that the use of the gate bracing at configuration 6 to upgrade the seismic response of the existing building resulted in an increase of about 1.20-2.50 times.

Furthermore, the upgraded building with gate bracing at configuration 7 (full gate brace) (Figure 4.15) had the maximum base shear of 2311.8 kN, 1965.7 kN, 1492.1 kN, and 1036.2 kN at 0.2, 0.4, 0.6, and 0.8 eccentricities, respectively while that with gate brace at configuration 8 (full gate brace without ground floor) (Figure 4.16) possessed the maximum base shear of 1211.1 kN, 1184.0 kN, 1057.9 kN, and 931.8 kN, respectively. This indicted about 1.23-2.74 and 1.11-1.44 times increment in seismic performance of the existing building due to the use of gate bracig at configuration 7 and the use of gate bracing at configuration 8, respectively.

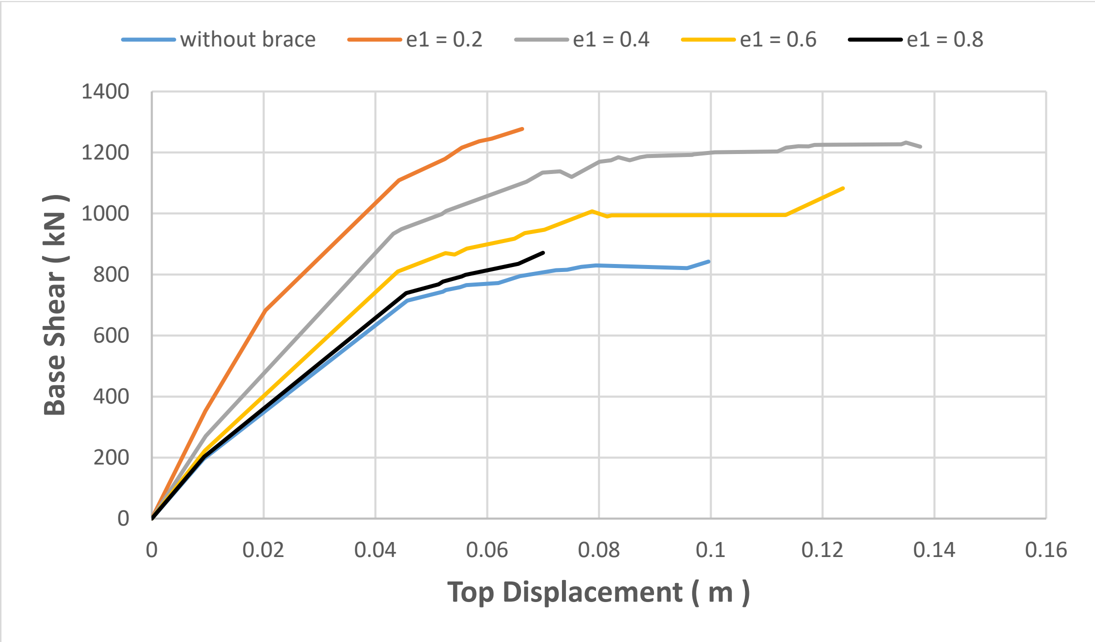


Figure 4.9 Capacity curves of 4 storey at configuration 1 (partial gate brace at mid bay only) for existing building and retrofitted one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

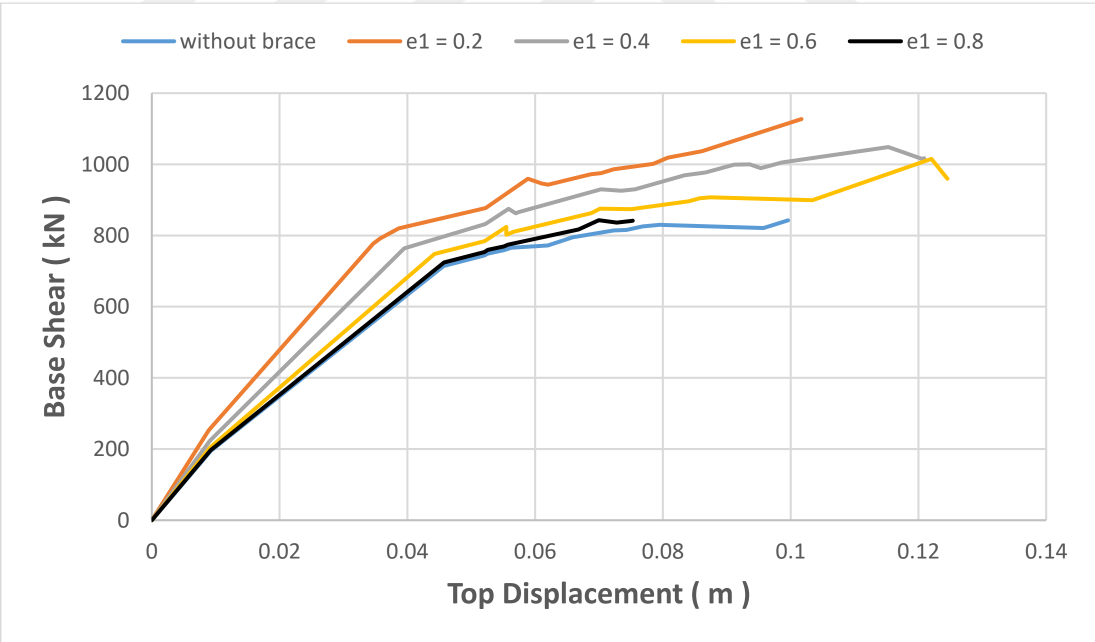


Figure 4.10 Capacity curves of 4 storey at configuration 2 (partial gate brace at mid bay without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

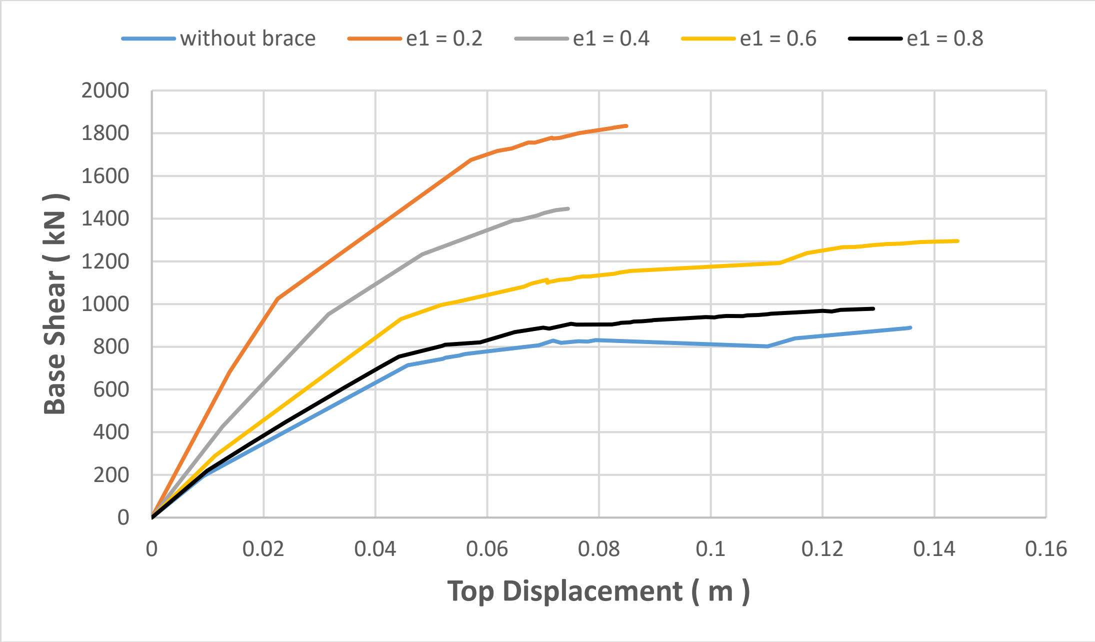


Figure 4.11 Capacity curves of 4 storey at configuration 3 (partial gate brace at mid bay with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

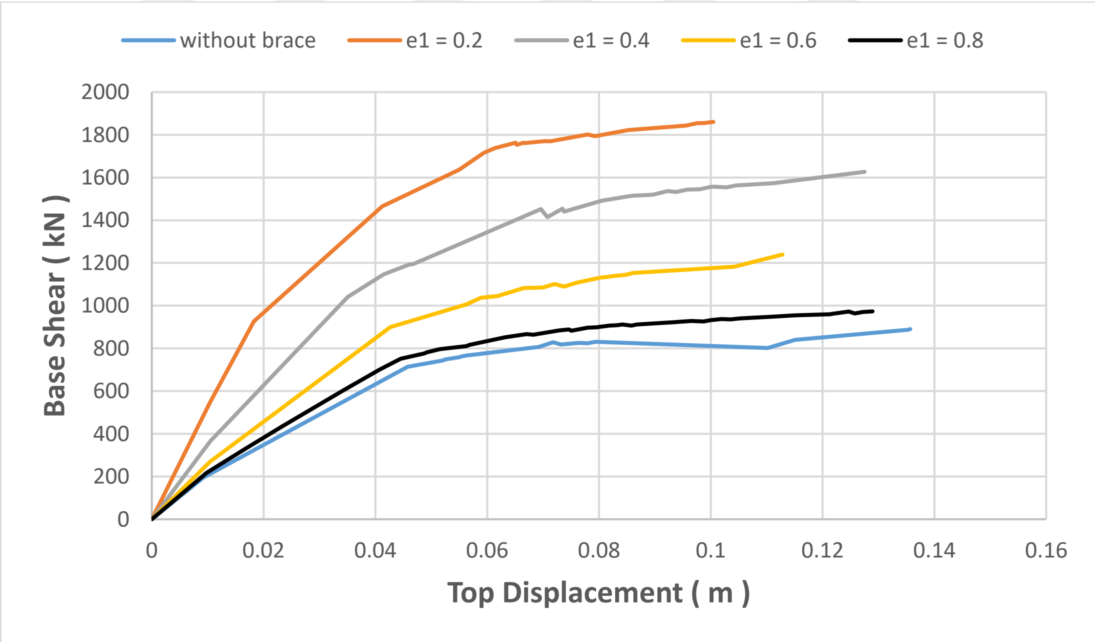


Figure 4.12 Capacity curves of 4 storey at configuration 4 (partial gate brace at two external bays) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

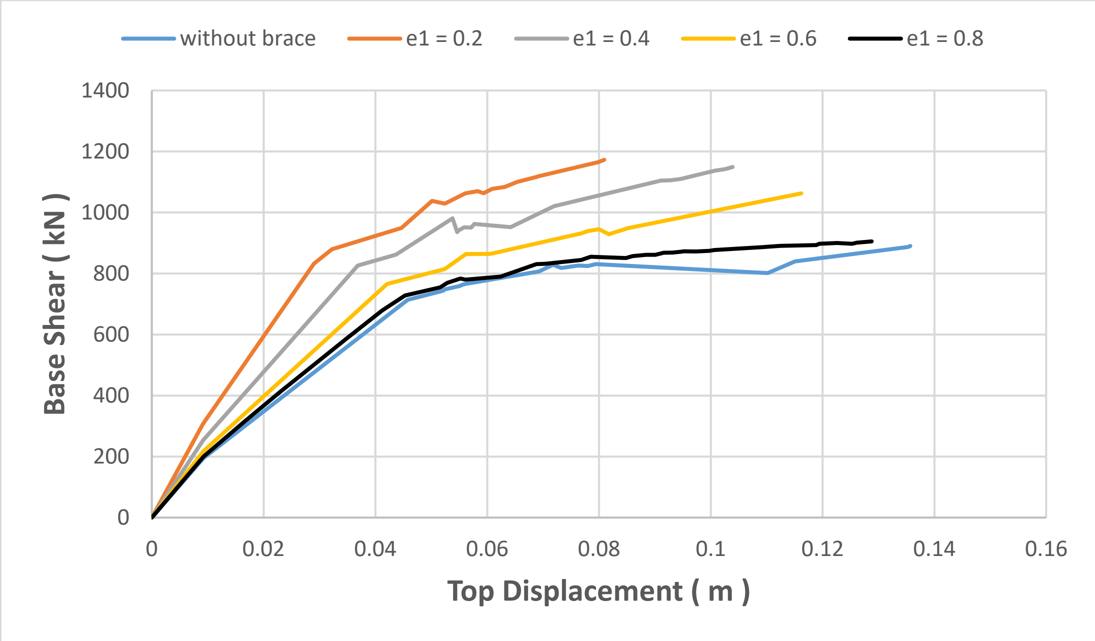


Figure 4.13 Capacity curves of 4 storey at configuration 5 (partial gate brace at two external bays without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

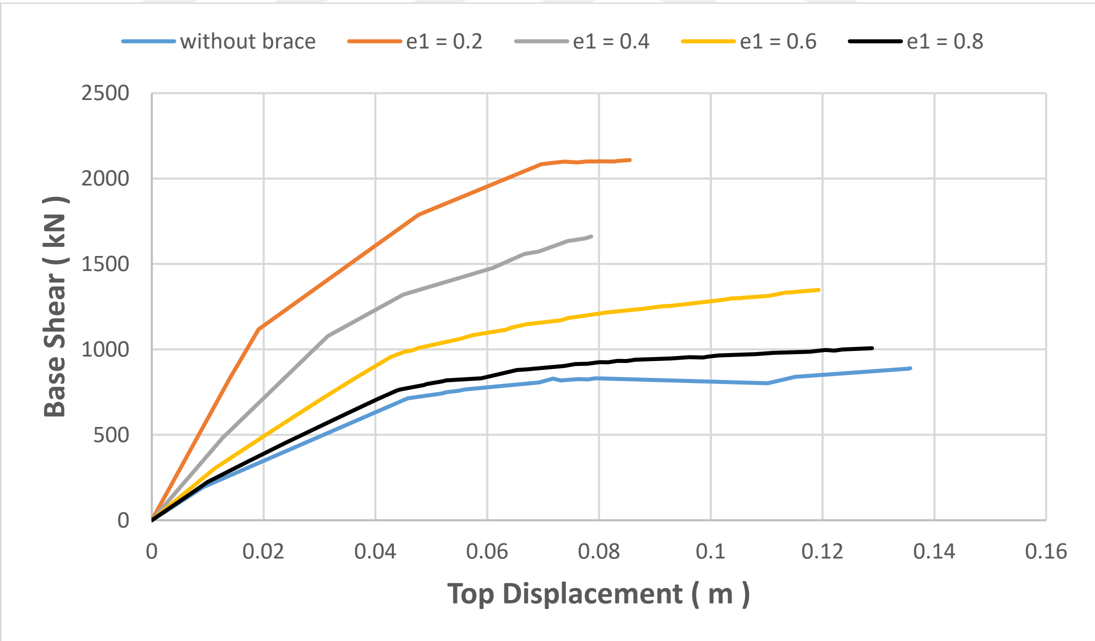


Figure 4.14 Capacity curves of 4 storey at configuration 6 (partial gate brace at two external bays with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

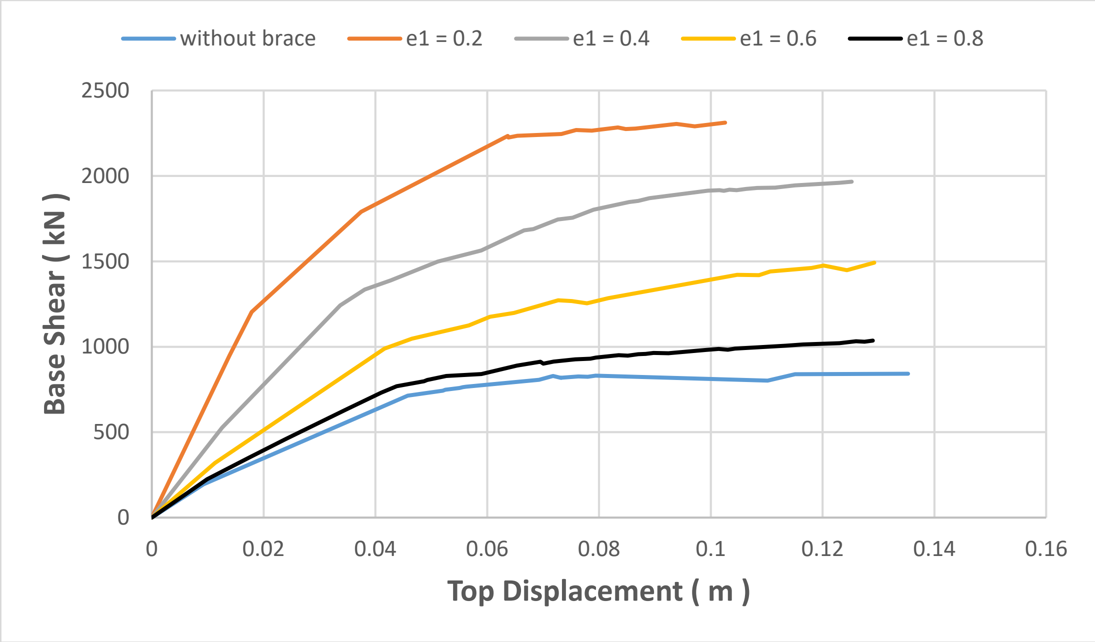


Figure 4.15 Capacity curves of 4 storey at configuration 7 (full gate brace) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

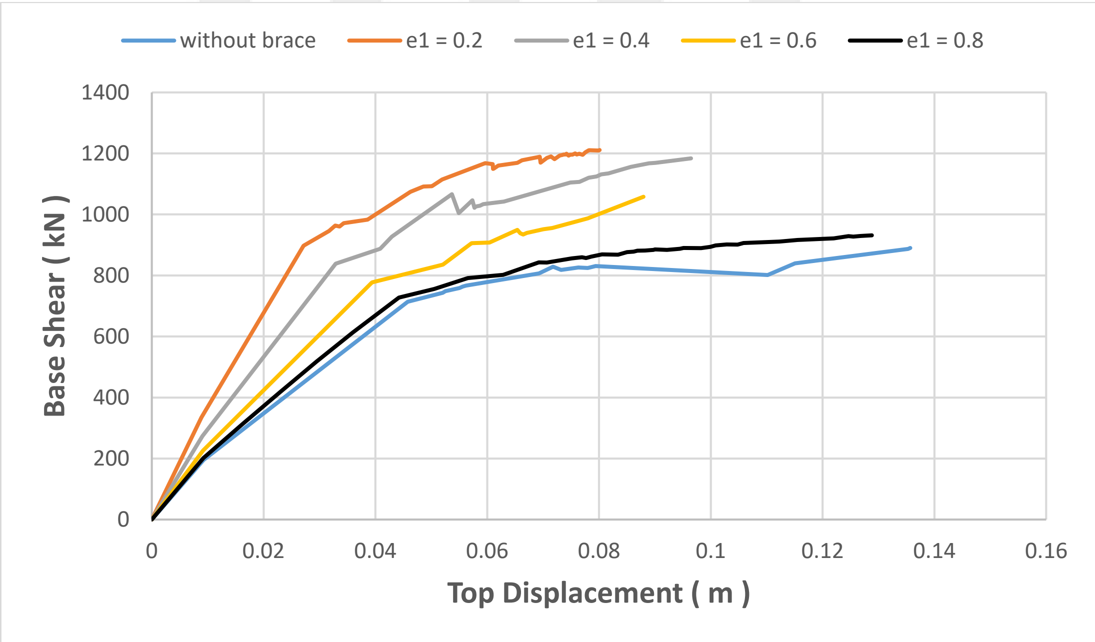


Figure 4.16 Capacity curves of 4 storey at configuration 8 (full gate brace without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

4.2.3 Six Storey Buildings

Figure 4.17 shows that the load carrying capability of the existing structure raised due to the use of the gate bracing. For instance, the maximum base shear was about 866.3 kN for the existing building whereas the maximum base shear for the upgraded building with gate bracing at configuration 1 (partial gate brace at mid bay only) ranged from 937.8 kN to 1306.7 kN, depending mainly on the amount of eccentricity (e_1=0.2 to 0.8). This showed about 1.08 to 1.51 times increment in the capacity of the upgraded building.

From Figure 4.18, for the upgraded building with gate bracing at configuration 2 (partial gate brace at mid bay without ground floor), the maximum base shear values varied from 867.6 kN to 1070.3 kN, depending mainly on the amount of eccentricity (e_1=0.2 to 0.8). In this case, the lateral load carrying capacity of the existing building varied from 1.002-1.24 times as a result of the use of bracing.

When we applied the gate bracing at configuration 3 (partial gate brace at mid bay with ground floor) (see Figure 4.19), the upgraded building had the maximum base shear values varied from 1023.3 kN to 1783.6 kN. This corresponded to 1.18 to 2.06 times increment.

In the case of the upgraded building with gate bracing at configuration 4 (partial gate brace at two external bays) as shown in Figure 4.20, the maximum base shear was ranged from 990.5 kN to 1763.4 kN, depending on the selection of eccentricity in gate bracing system (e_1=0.2 to 0.8). This suggested about 1.14 to 2.04 times increment. For the upgraded building with gate bracing at configuration 5 (partial gate brace at two external bays without ground floor) as shown in Figure 4.21, the maximum base shear values were between 902.1 kN and 1147.1 kN. This caused approximately 1.04 to 1.32 times increment. For the case of used gate brace at configuration 6 (partial gate brace at two external bays with ground floor) as shown in Figure 4.22, the maximum base shear values were in the ranges of 1027.8 kN to 2028.8 kN. This corresponded to 1.19 to 2.34 times increment.

Also, the upgraded building with gate bracing at configuration 7 (full gate brace), as shown in Figure 4.23 possessed the maximum base shear of 1014.2 kN to 2211.0 kN while that with gate bracing at configuration 8 (full gate brace without ground floor), as shown in Figure 4.24 had the maximum base shear of 1008.8 kN to 1284.3 kN. Subsequently, there were about 1.17-2.55 and 1.16-1.48 times increment in the performance of the existing building because of the use of gate bracing at configuration 7 and gate bracing at configuration 8, respectively.

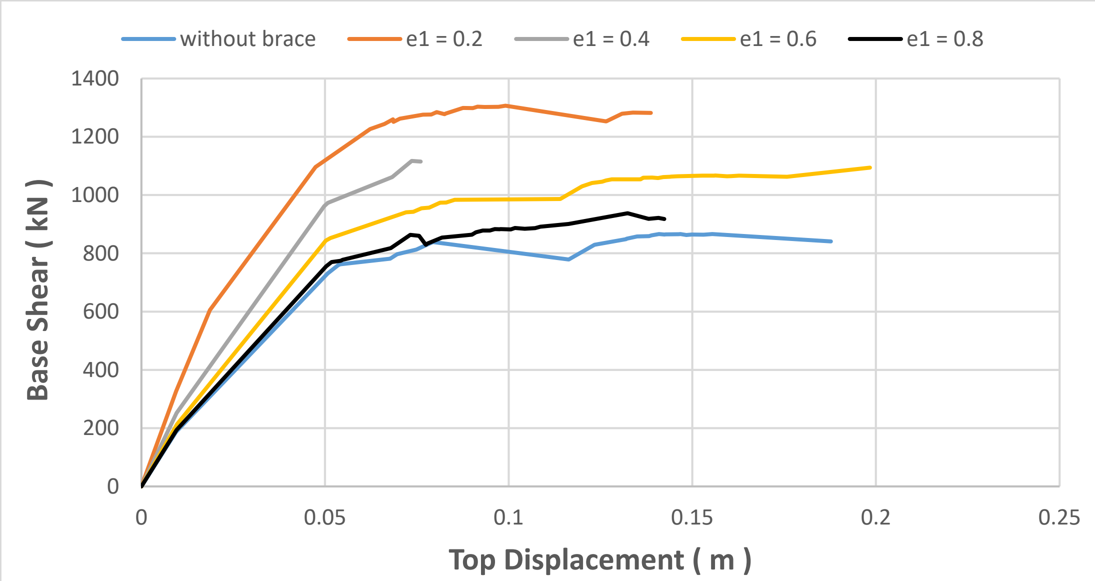


Figure 4.17 Capacity curves of 6 storey at configuration 1 (partial gate brace at mid bay only) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

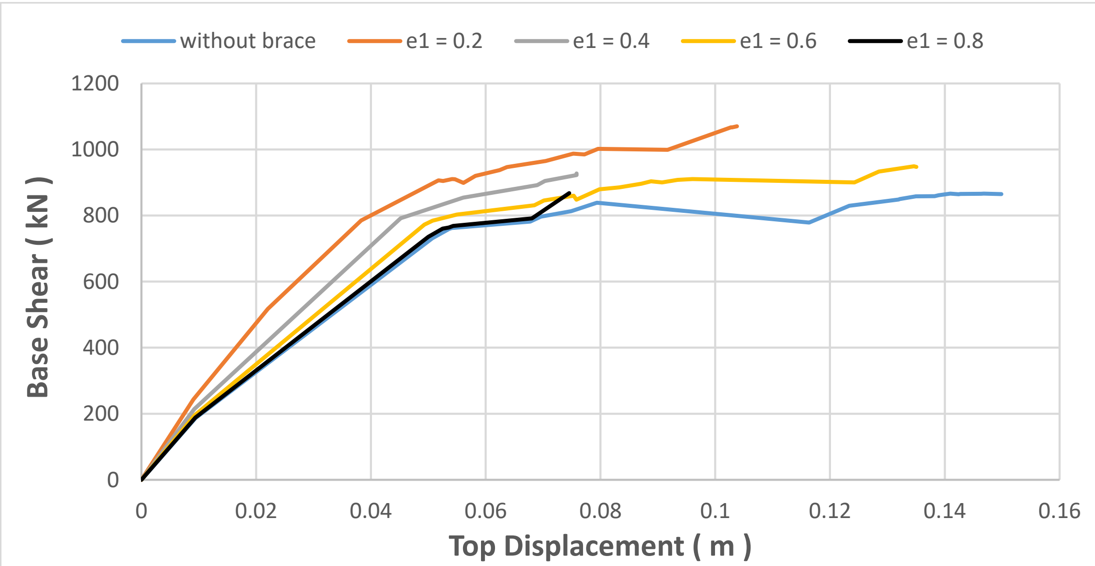


Figure 4.18 Capacity curves of 6 storey at configuration 2 (partial gate brace at mid bay without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

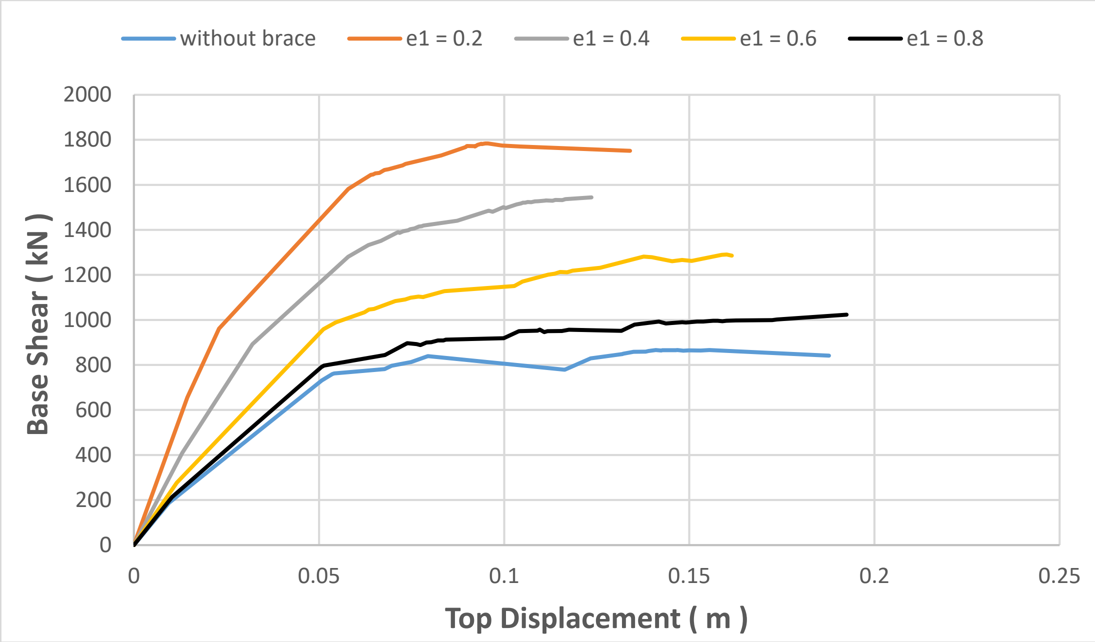


Figure 4.19 Capacity curves of 6 storey at configuration 3 (partial gate brace at mid bay with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

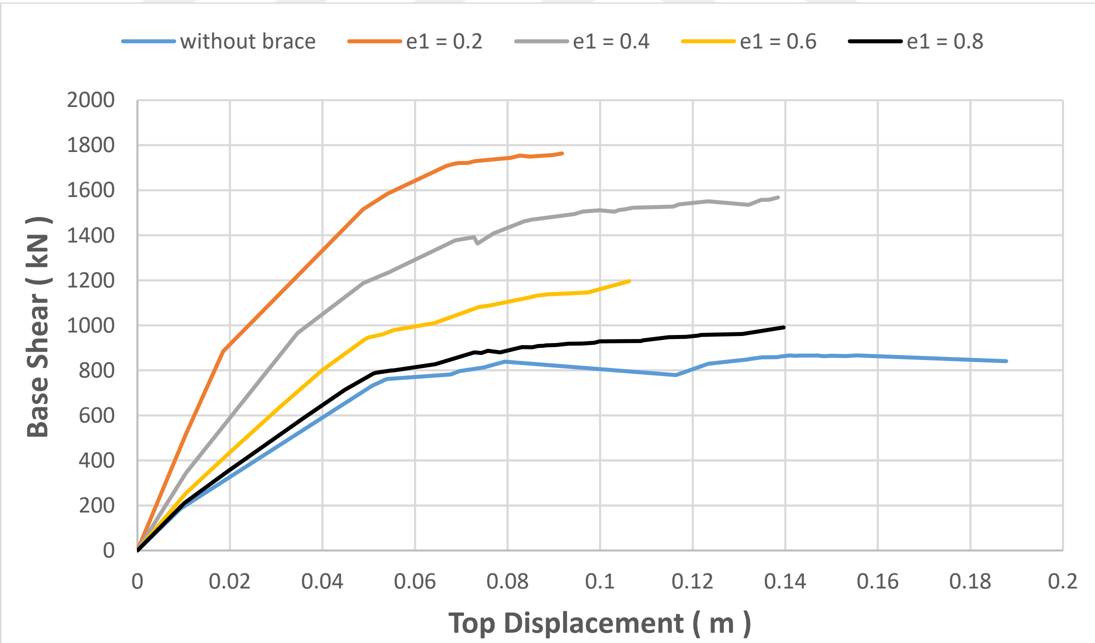


Figure 4.20 Capacity curves of 6 storey at configuration 4 (partial gate brace at two external bays) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

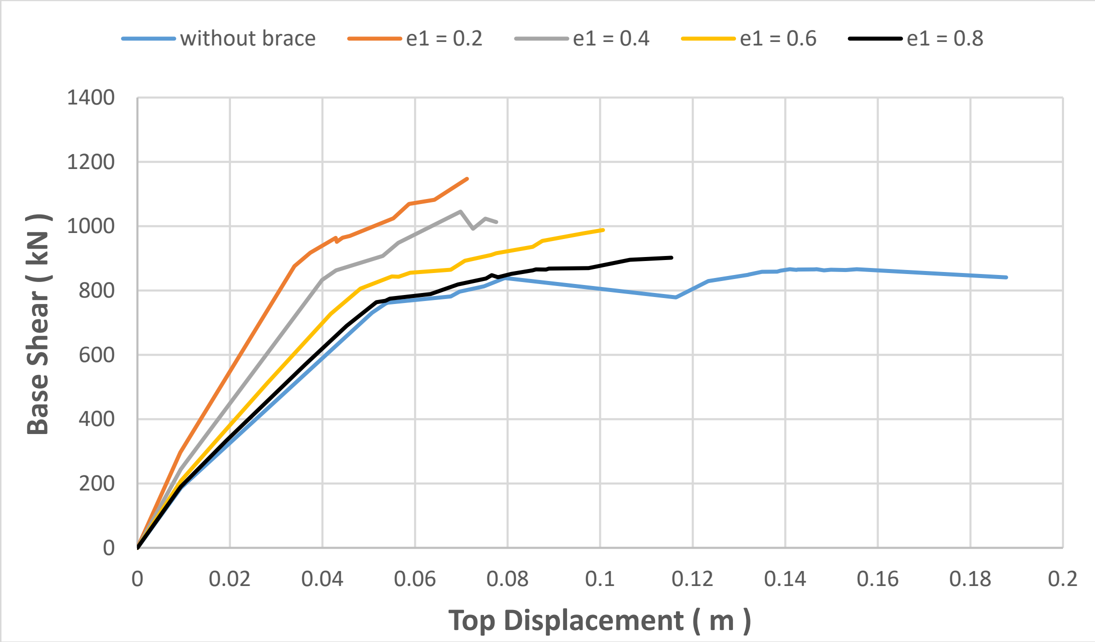


Figure 4.21 Capacity curves of 6 storey at configuration 5 (partial gate brace at two external bays without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

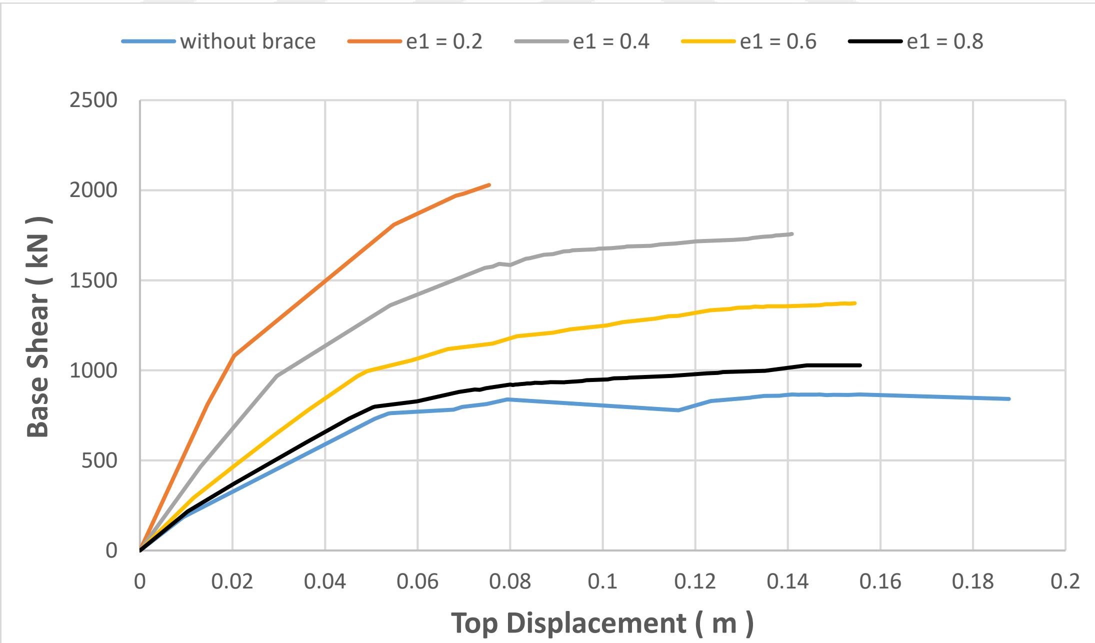


Figure 4.22 Capacity curves of 6 storey at configuration 6 (partial gate brace at two external bays with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

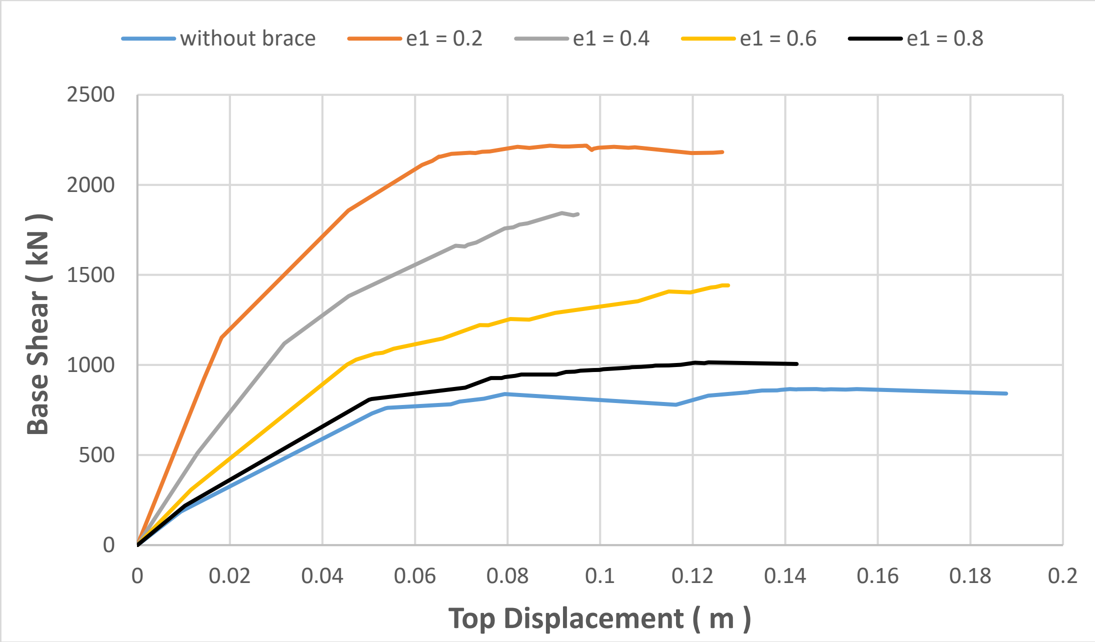


Figure 4.23 Capacity curves of 6 storey at configuration 7 (full gate brace) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

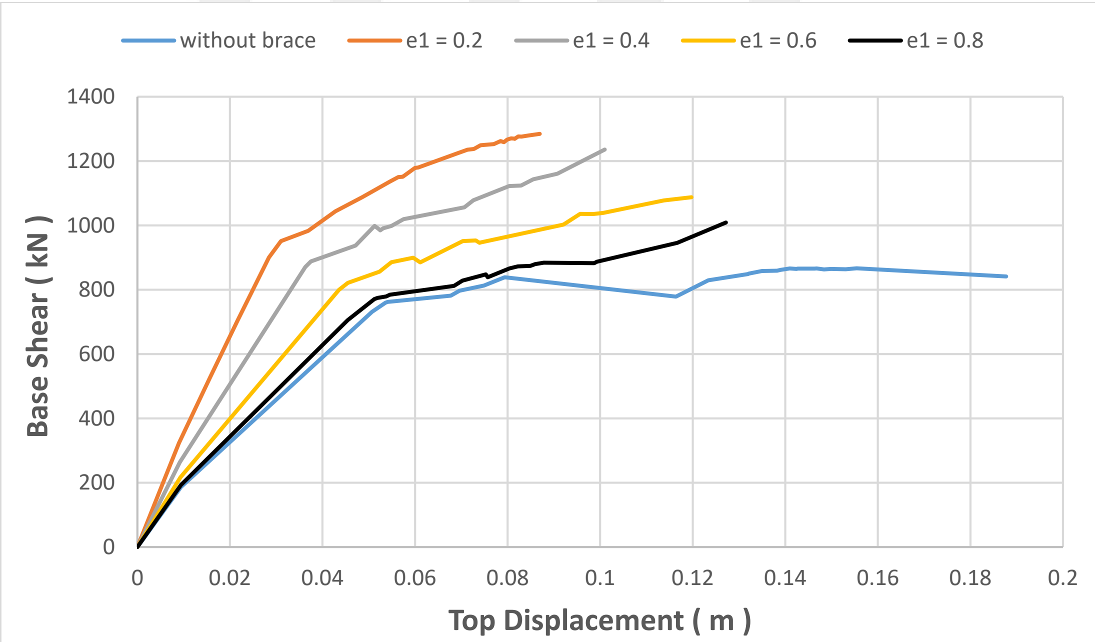


Figure 4.24 Capacity curves of 6 storey at configuration 8 (full gate brace without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

4.2.4 Eight Storey Buildings

Figure 4.25 shows that the load carrying capability of the existing structure raised due to the use of the gate bracing. For example, the maximum base shear was about 1298.9 kN for the existing building whereas the maximum base shear for the upgraded building with gate bracing at configuration 1 (partial gate brace at mid bay only) ranged from 1360.7 kN to 1721.5 kN, depending mainly on the amount of eccentricity (e_1=0.2 to 0.8). This showed about 1.05 to 1.33 times increment in the capacity of the upgraded building.

Figure 4.26, for the upgraded building with gate bracing at configuration 2 (partial gate brace at mid bay without ground floor), the maximum base shear values varied from 1326.3 kN to 1492.3 kN, depending mainly on the amount of eccentricity (e_1=0.2 to 0.8). In this case, the lateral load carrying capacity of the existing building varied from 1.02-1.15 times as a result of the use of bracing.

When we used the gate bracing at configuration 3 (partial gate brace at mid bay with ground floor) (see Figure 4.27), the upgraded building had the maximum base shear values varied from 1431.9 kN to 2174.9 kN. This corresponded to 1.10 to 1.67 times increment.

In the case of the upgraded building with gate bracing at configuration 4 (partial gate brace at two external bays) as shown in Figure 4.28, the maximum base shear was ranged from 1430.9 kN to 2155.4 kN, depending mainly on the amount of eccentricity (e_1=0.2 to 0.8). This suggested about 1.10 to 1.66 times increment. For the upgraded building with gate bracing at configuration 5 (partial gate brace at two external bays without ground floor) as displayed in Figure 4.29, the maximum base shear values were between 1353.7 kN and 1707.4 kN. This caused approximately 1.04 to 1.31 times increment. For the case of used gate bracing at configuration 6 (partial gate brace at two external bays with ground floor) as shown in Figure 4.30, the maximum base shear values were in the ranges of 1465.6 kN to 2413.0 kN. This corresponded to 1.13 to 1.86 times increment.

Also, the upgraded building with gate bracing at configuration 7 (full gate brace), as shown in Figure 4.31 possessed the maximum base shear of 1500.2 kN to 2609.3 kN while that with gate bracing at configuration 8 (full gate brace without ground floor), as shown in Figure 4.32 had the maximum base shear of 1389.2 kN to 1912.8 kN. Subsequently, there were about 1.15-2.01 and 1.07-1.47 times increment in the review of the existing building because of the use of gate bracing at configuration 7 and gate brace at configuration 8, respectively.

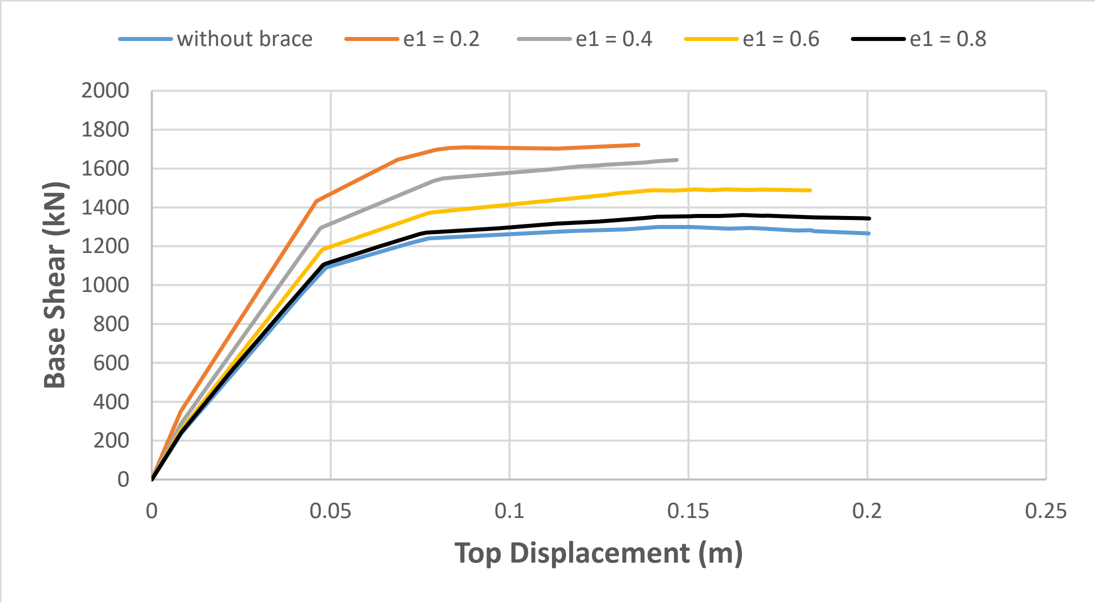


Figure 4.25 Capacity curves of 8 storey at configuration 1 (partial gate brace at mid bay only) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

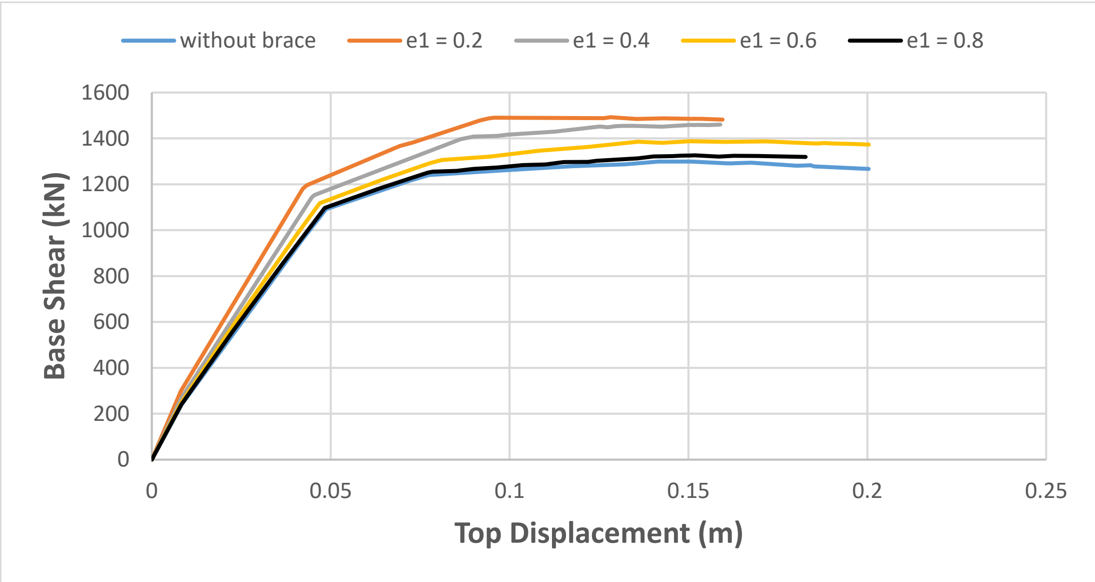


Figure 4.26 Capacity curves of 8 storey at configuration 2 (partial gate brace at mid bay without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

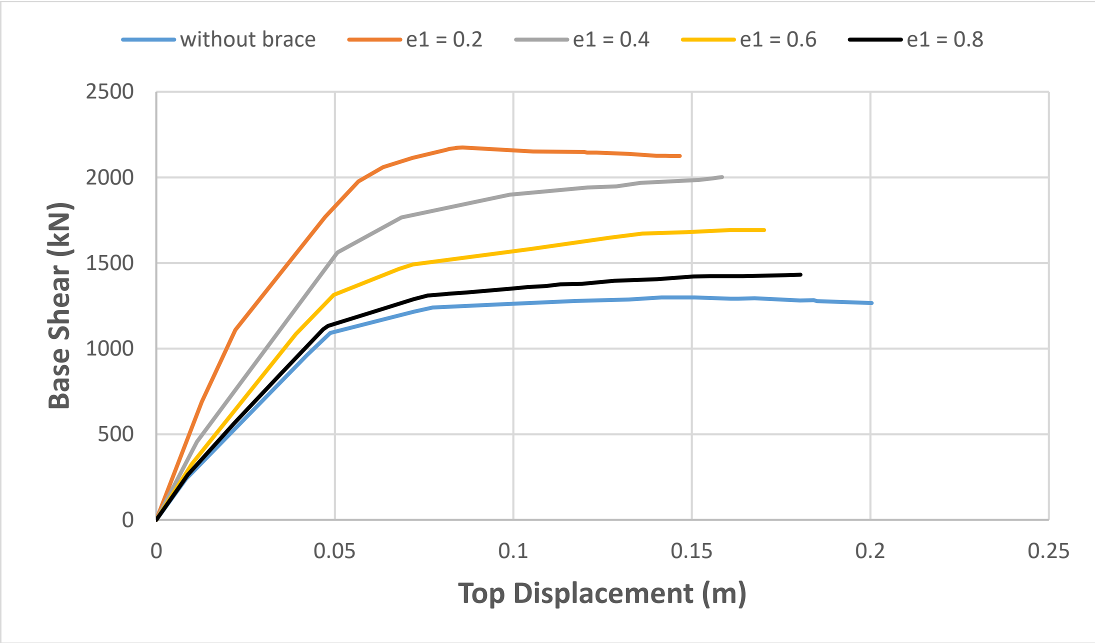


Figure 4.27 Capacity curves of 8 storey at configuration 3 (partial gate brace at mid bay with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

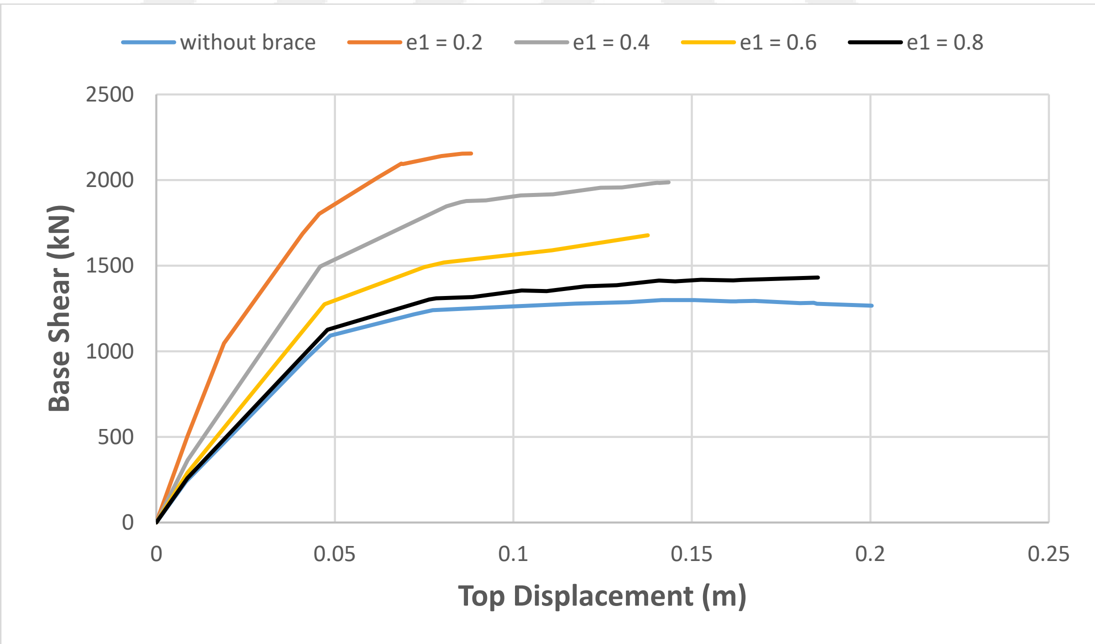


Figure 4.28 Capacity curves of 8 storey at configuration 4 (partial gate brace at two external bays) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

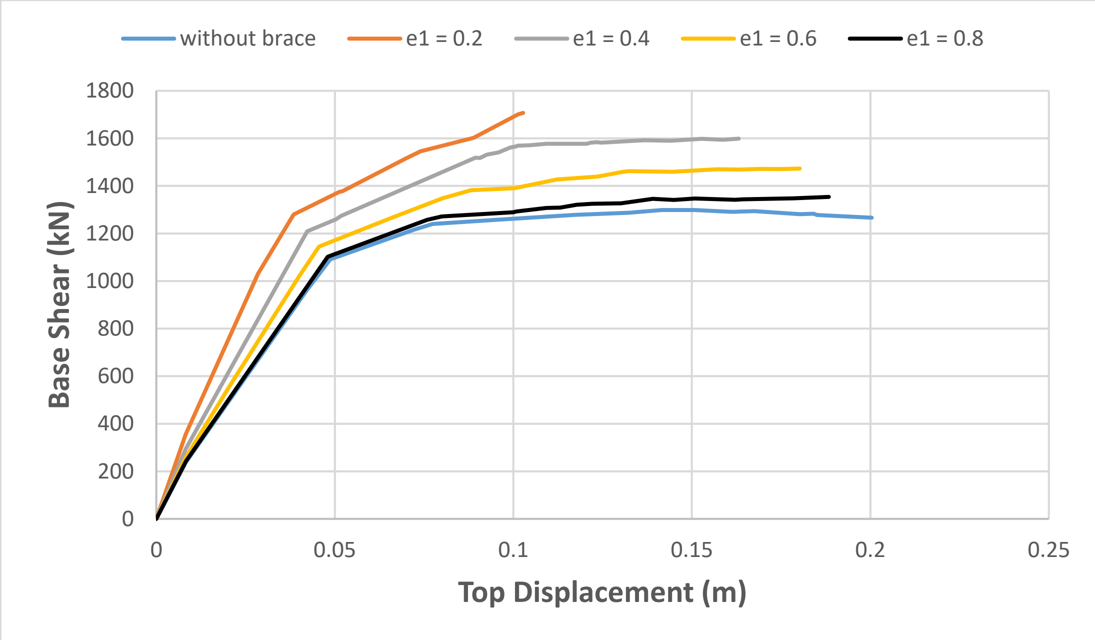


Figure 4.29 Capacity curves of 8 storey at configuration 5 (partial gate brace at two external bays without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

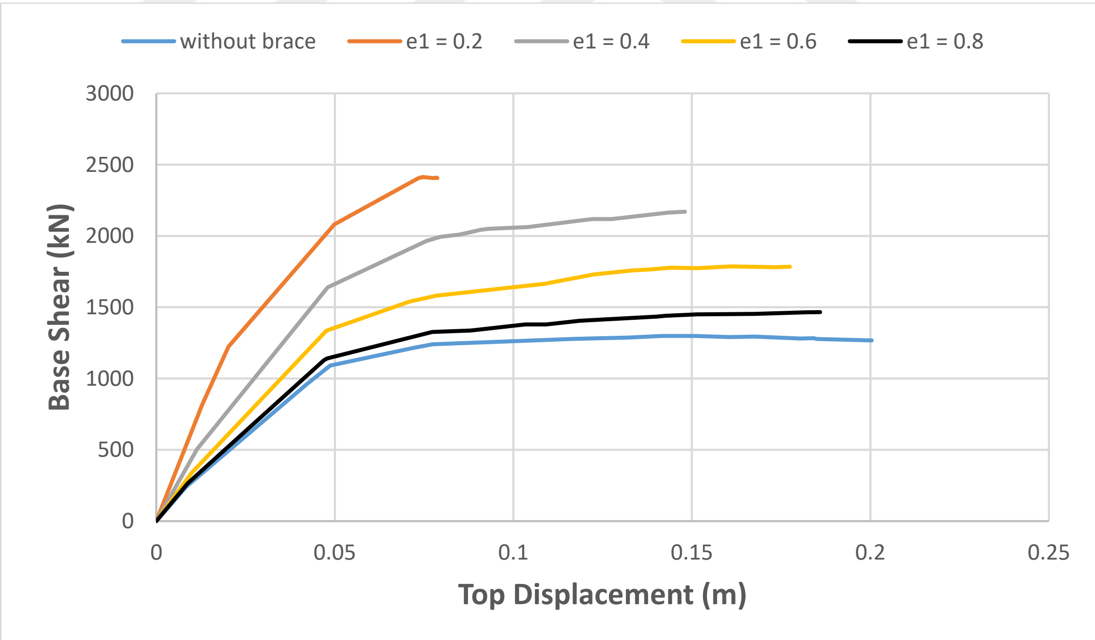


Figure 4.30 Capacity curves of 8 storey at configuration 6 (partial gate brace at two external bays with ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

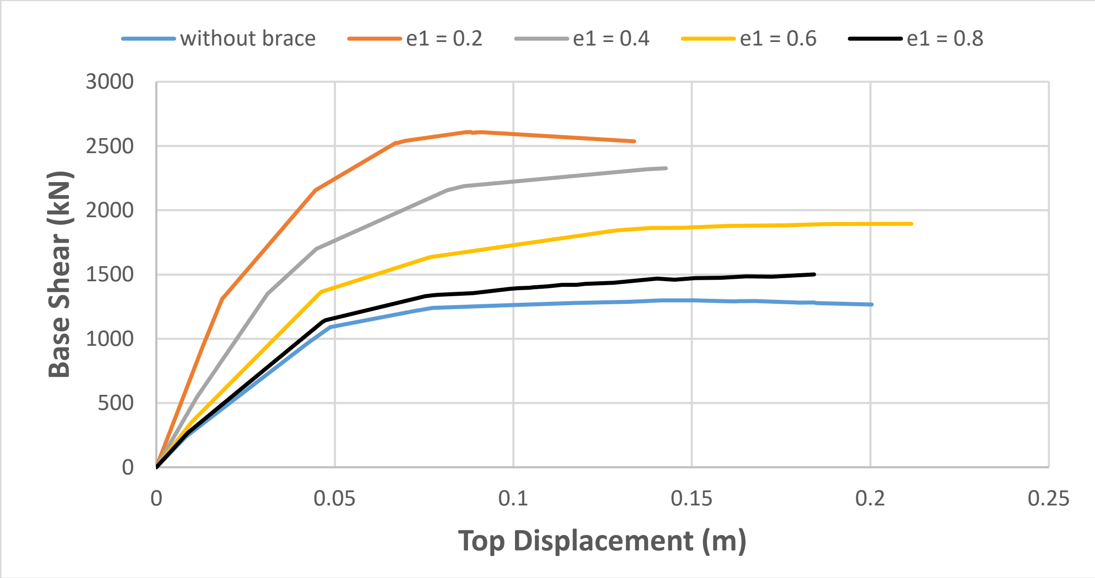


Figure 4.31 Capacity curves of 8 storey at configuration 7 (full gate brace) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

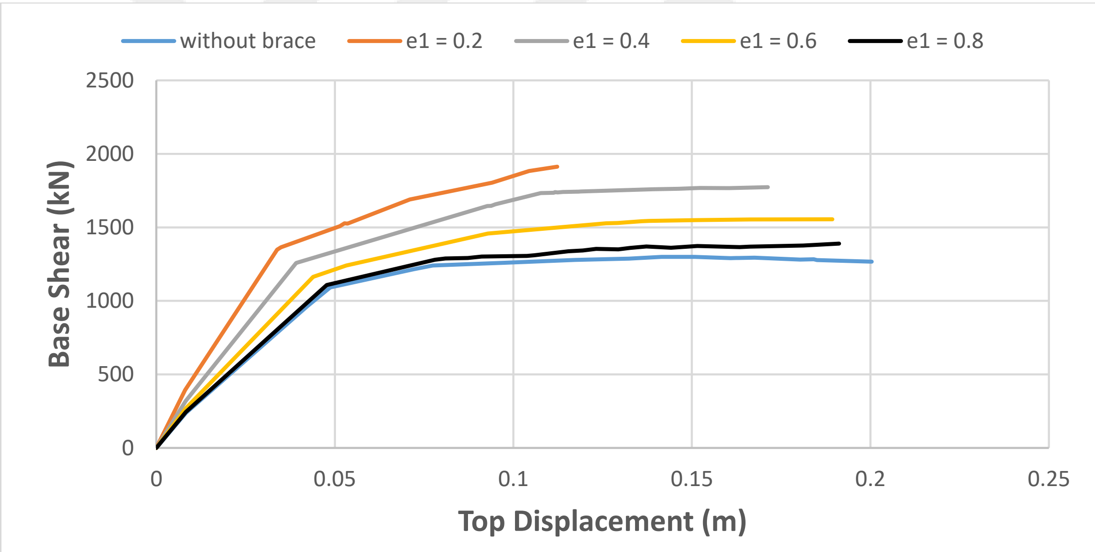
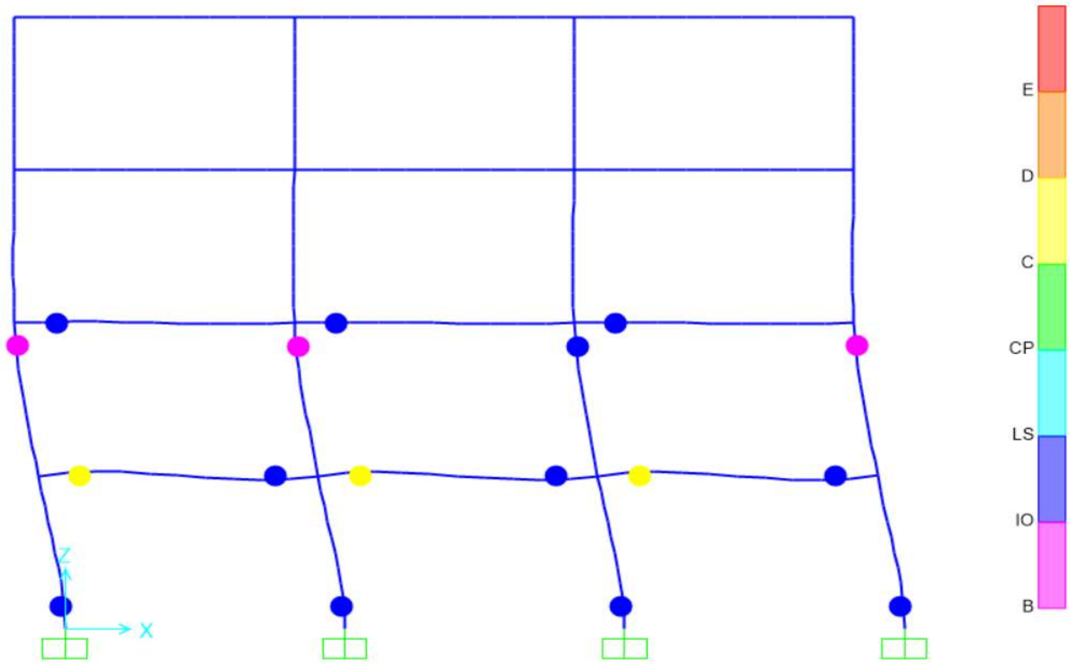


Figure 4.32 Capacity curves of 8 storey at configuration 8 (full gate brace without ground floor) for existing building and upgraded one when (e2=0.6) and (e1=0.2, 0.4, 0.6, and 0.8)

Figures 4.33 to 4.40 illustrate that the plastic hinge formation of the existing buildings and gate braced buildings in 8 different configurations for 4 storey building. As seen from the figures, the location and state of the plastic hinges were remarkably changed, depending on eccentricity (e1=0.2, 0.4, 0.6, and 0.8) (when e2=0.6, constant for all cases) and different configurations.



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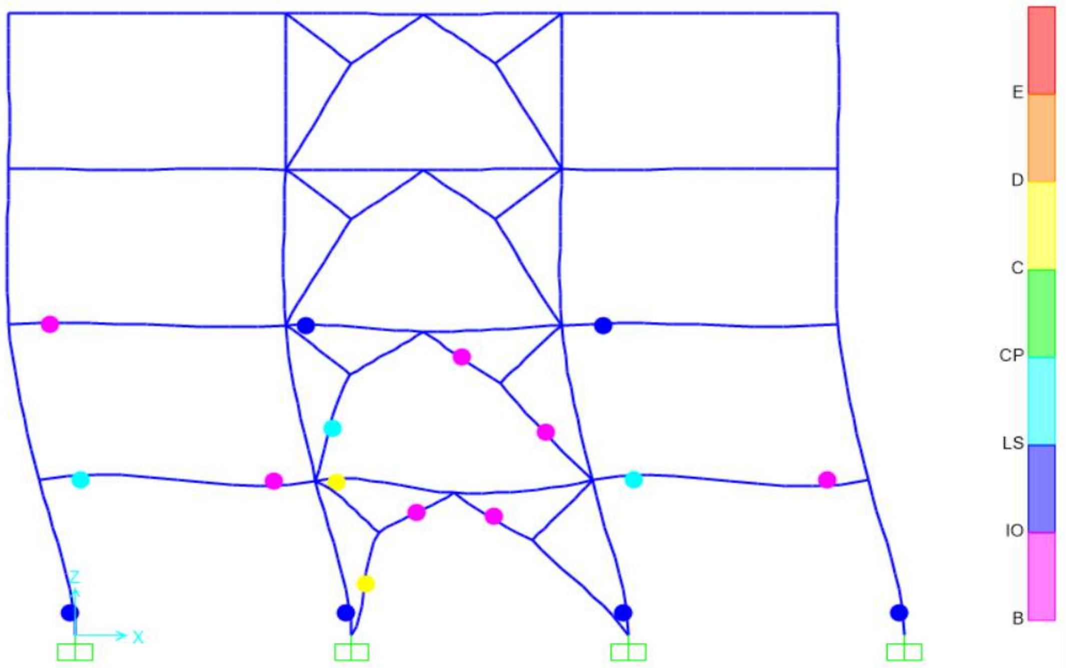

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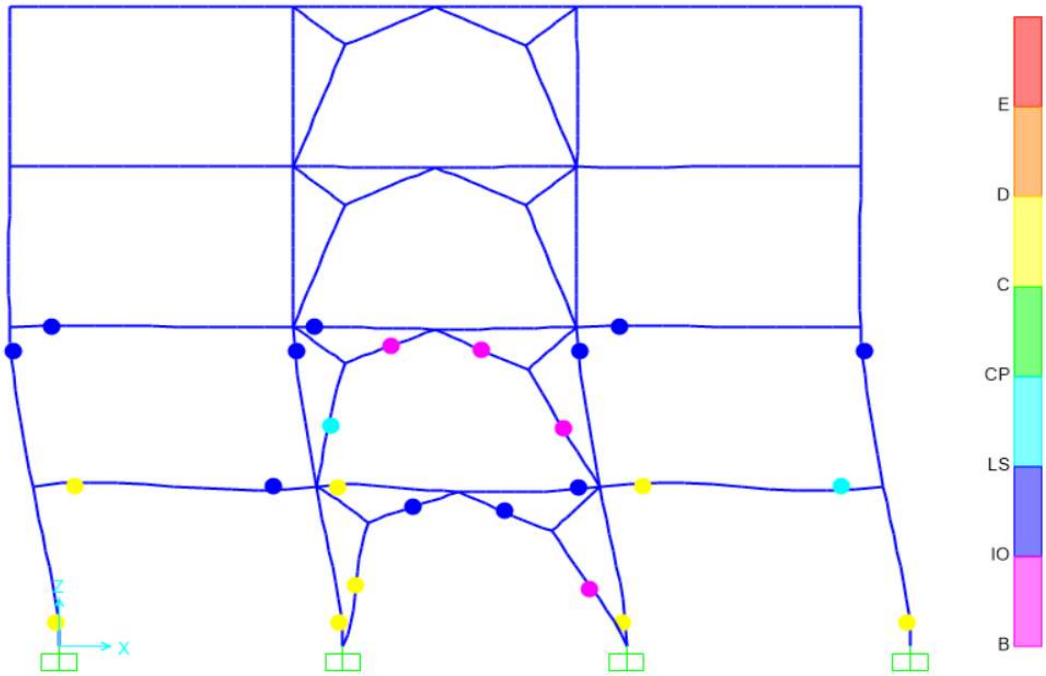

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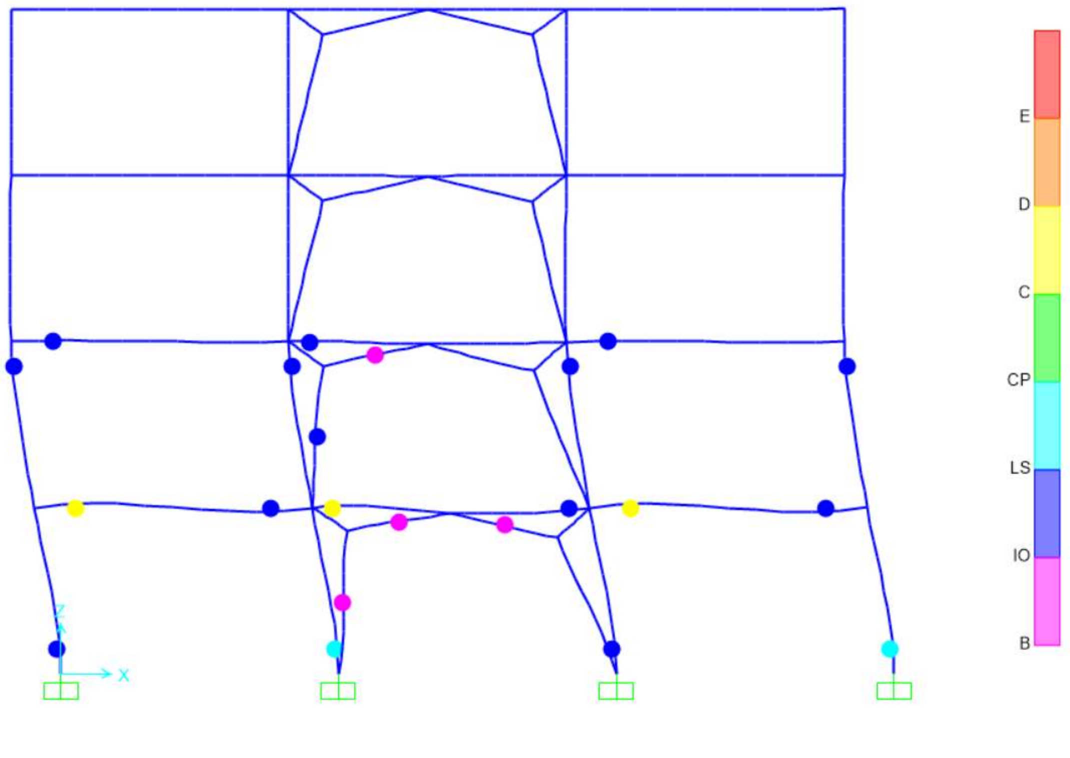

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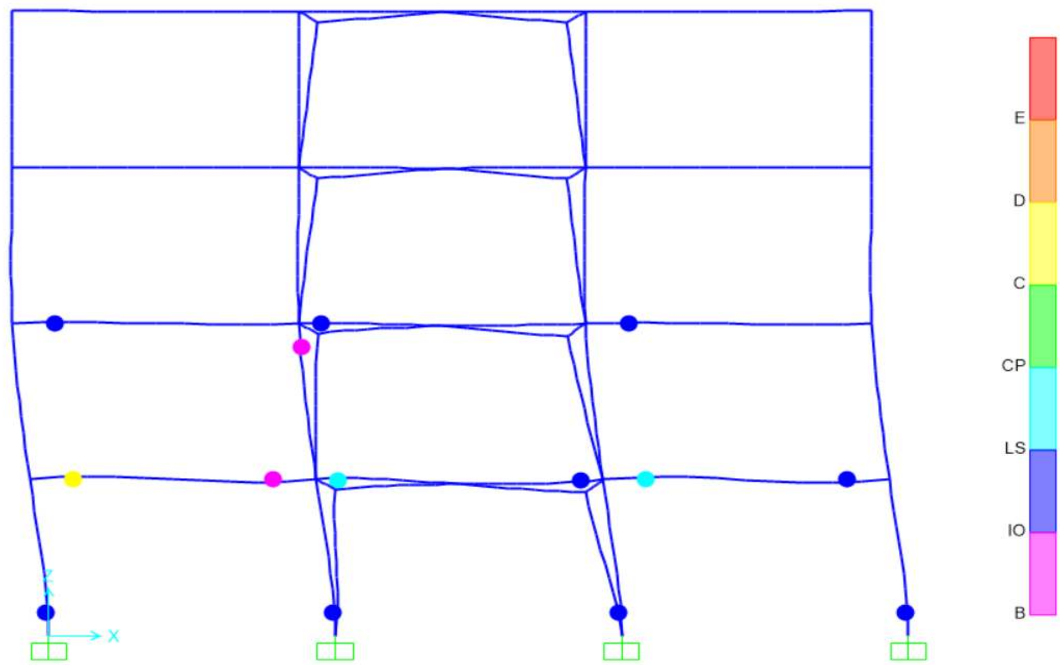
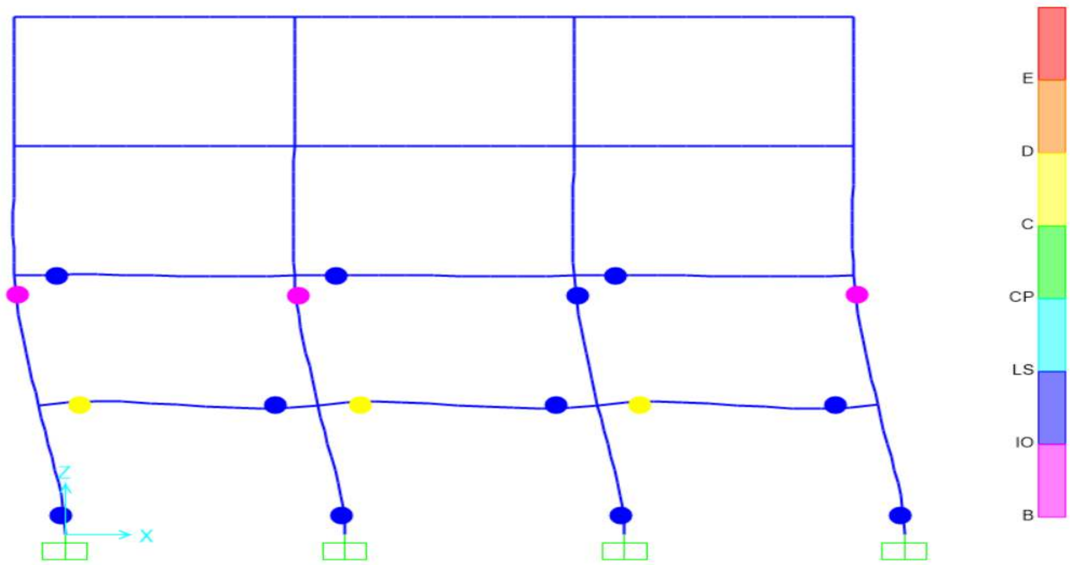


e)

Figure 4.33 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 1 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



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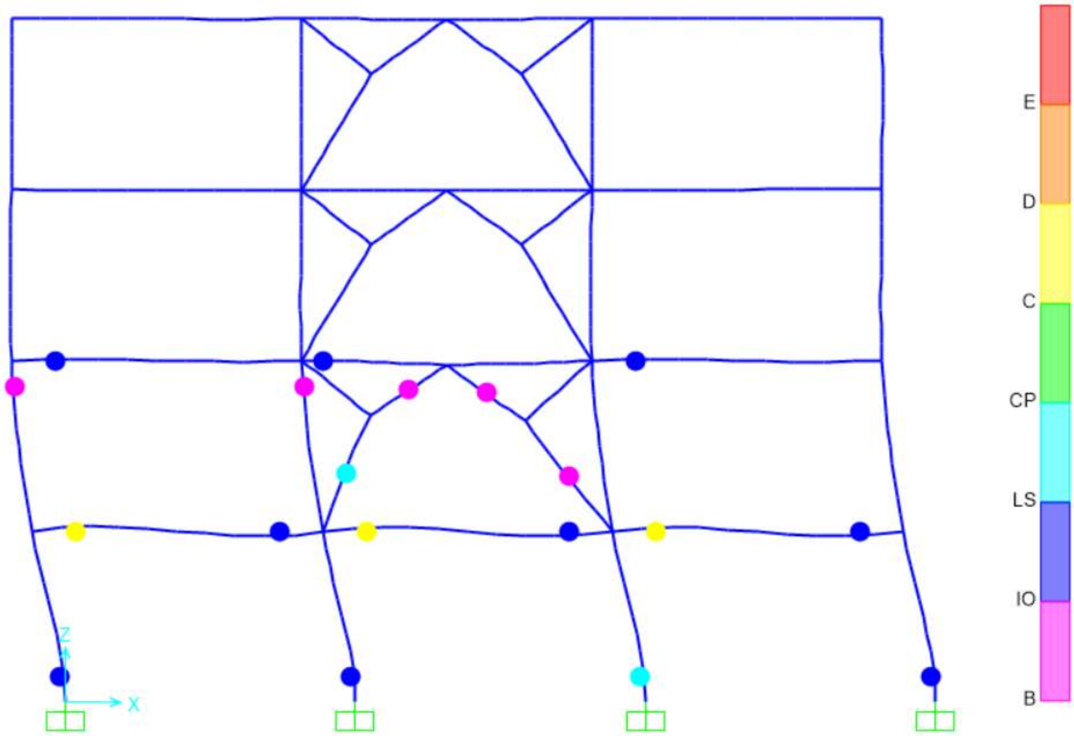

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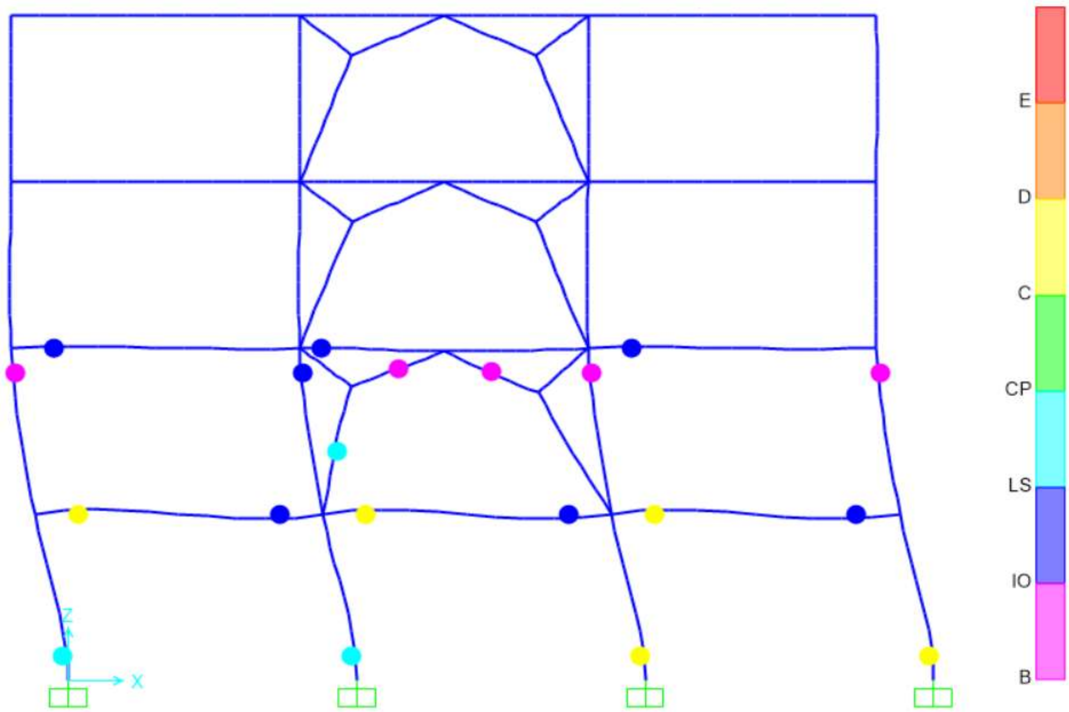

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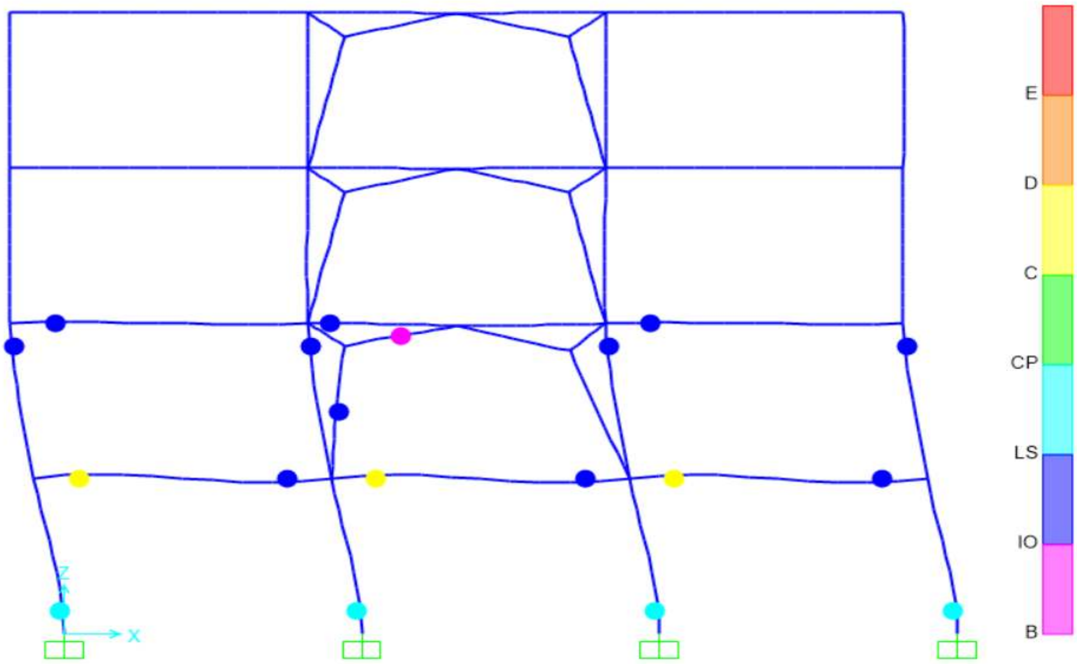

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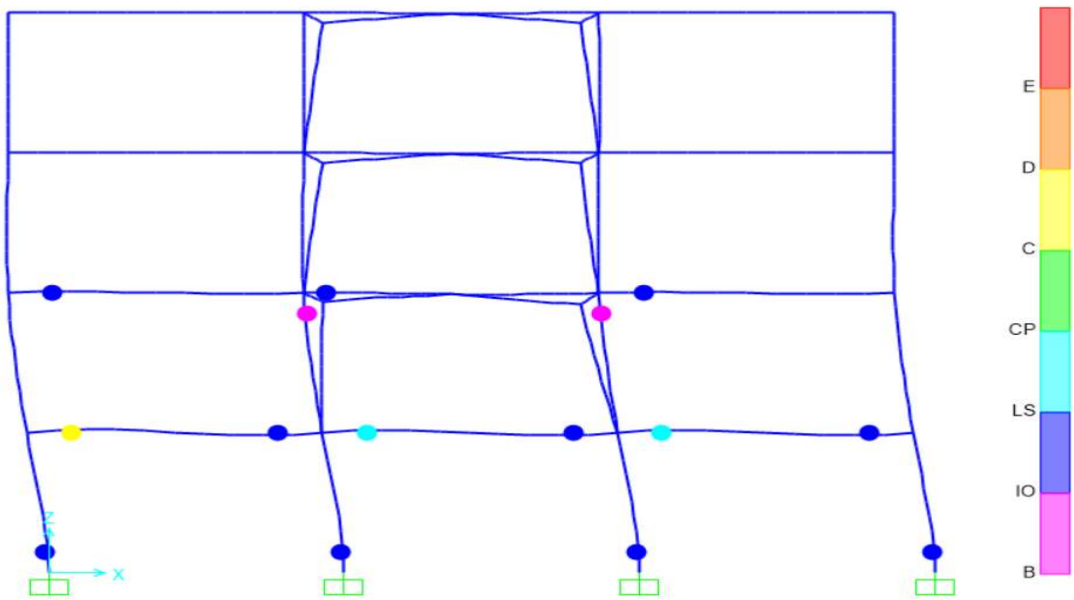
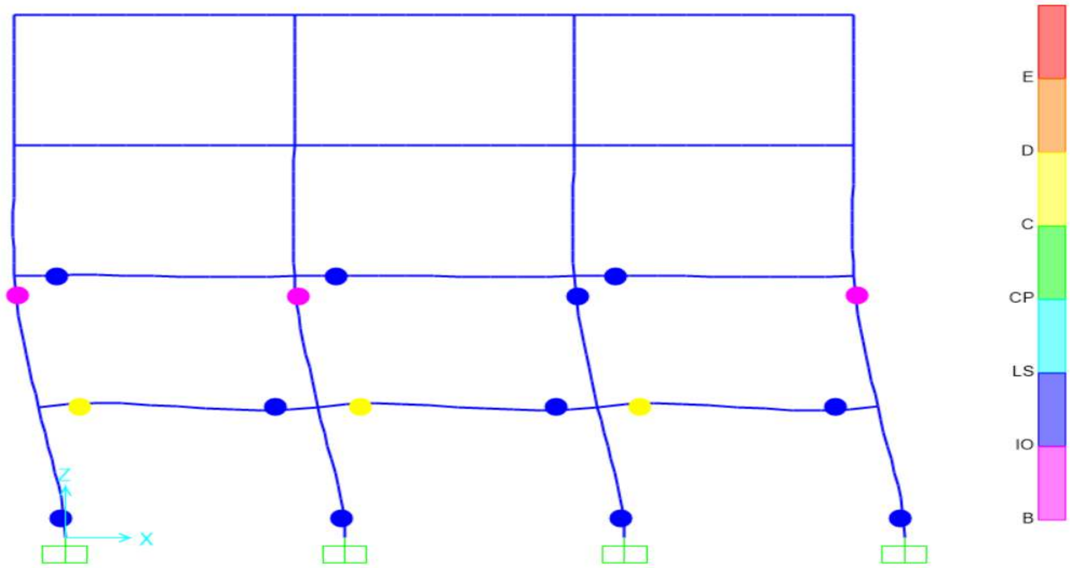


e)

Figure 4.34 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 2 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



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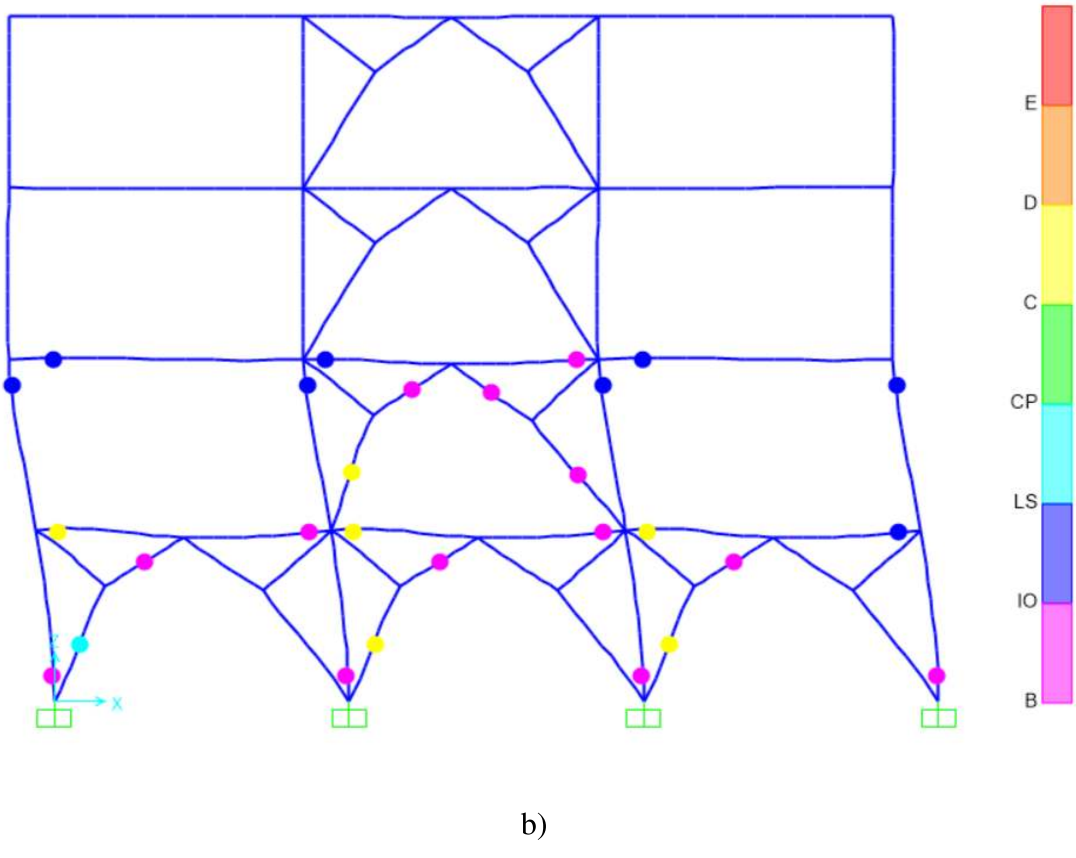

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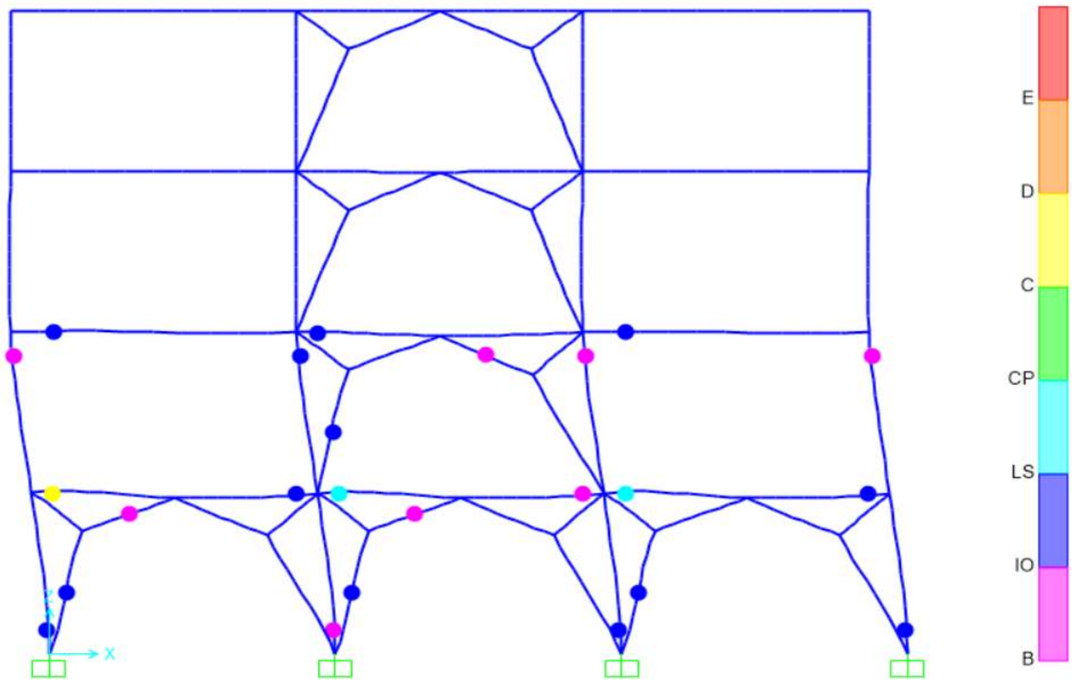

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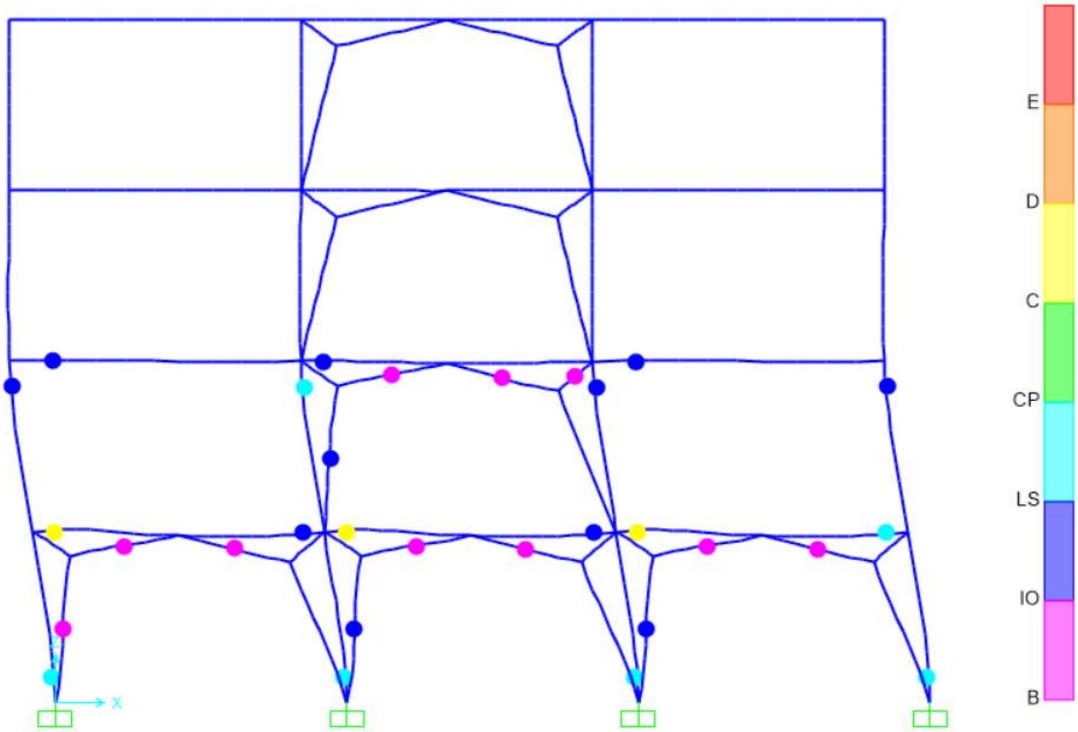

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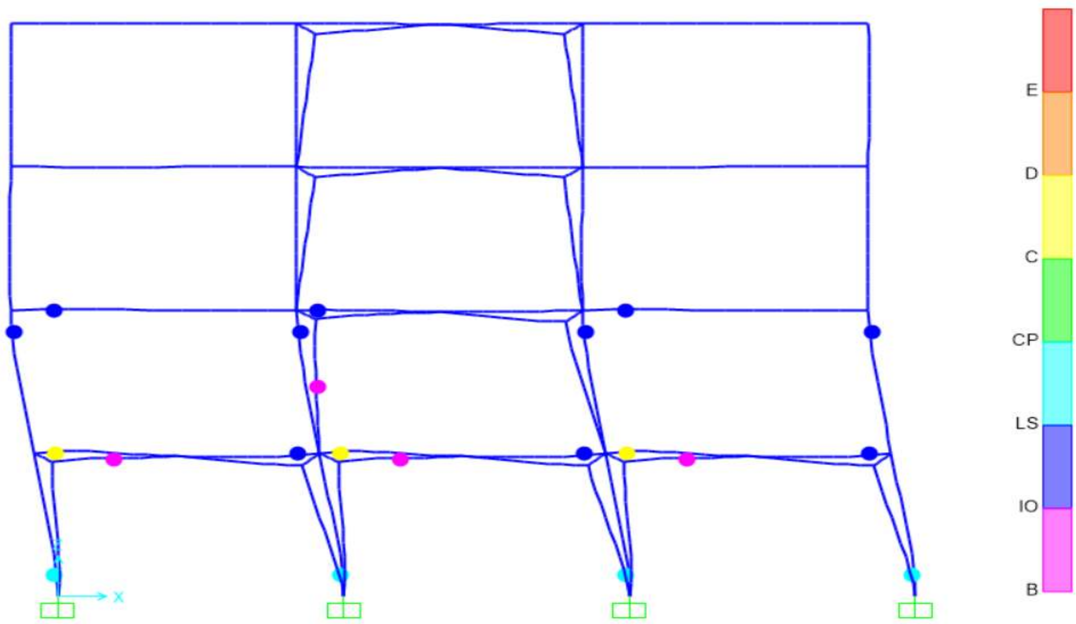
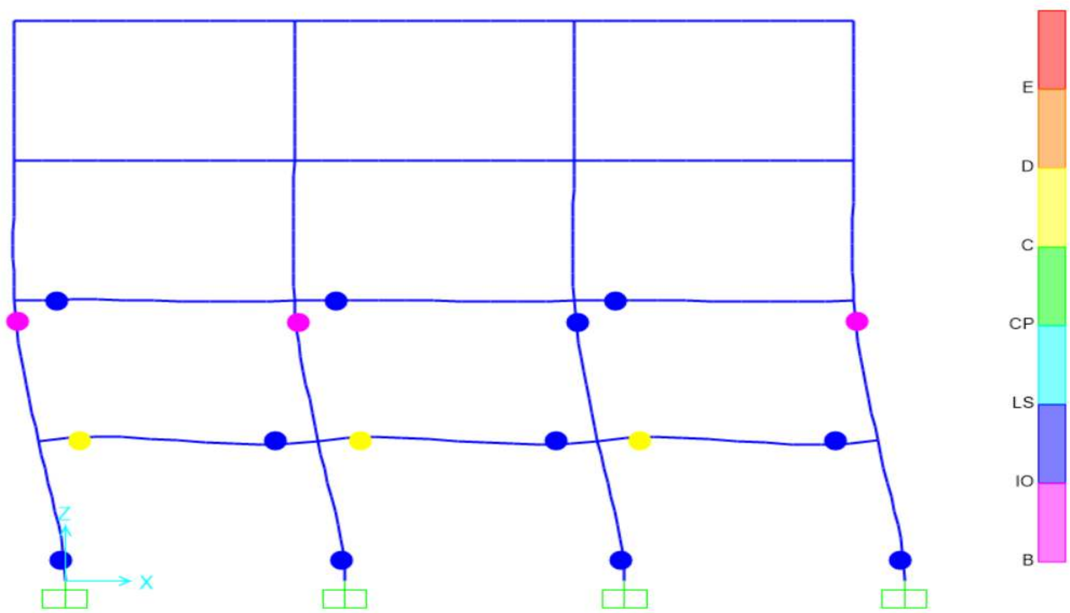


e)

Figure 4.35 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 3 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



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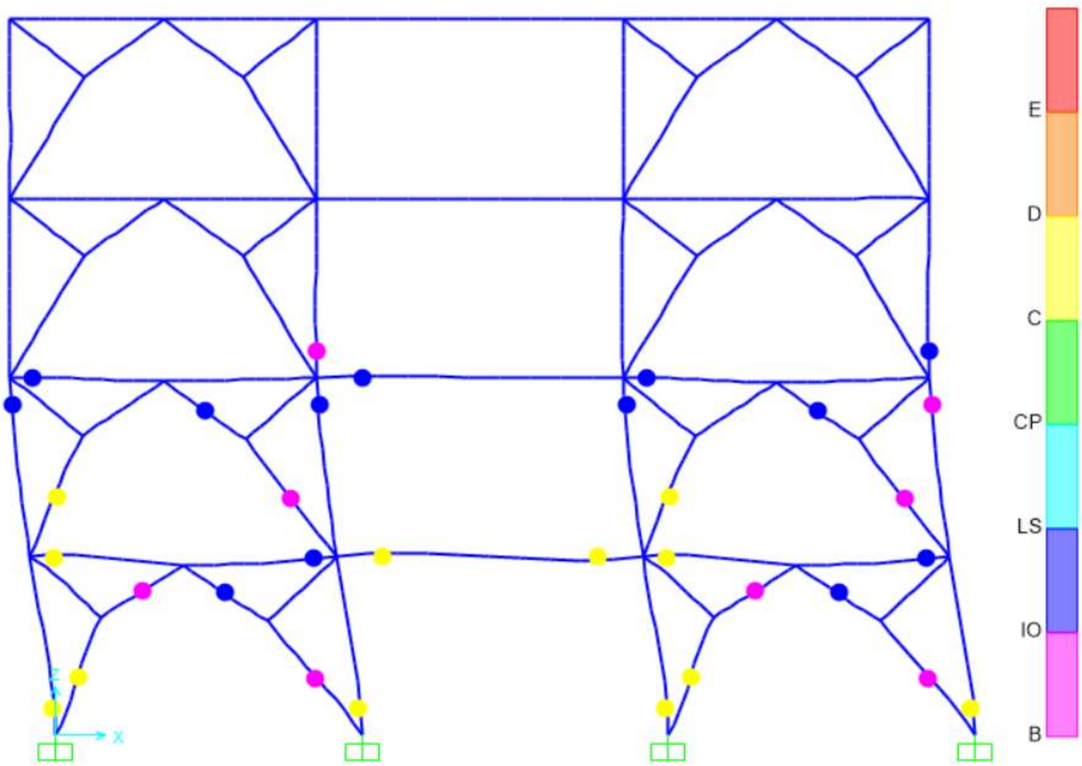

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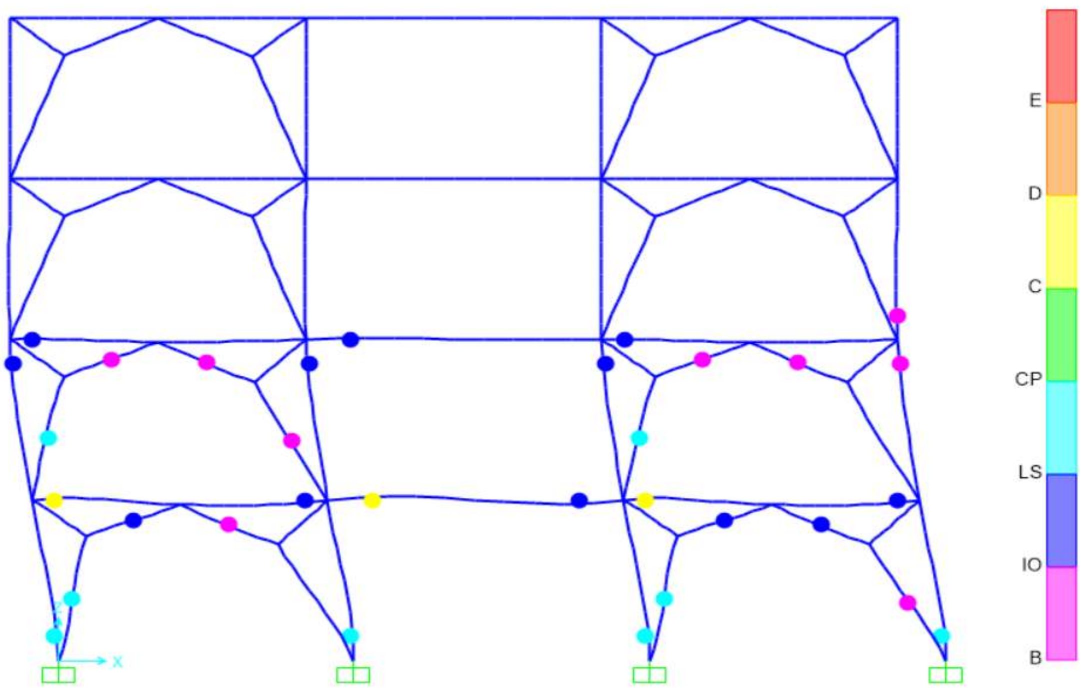

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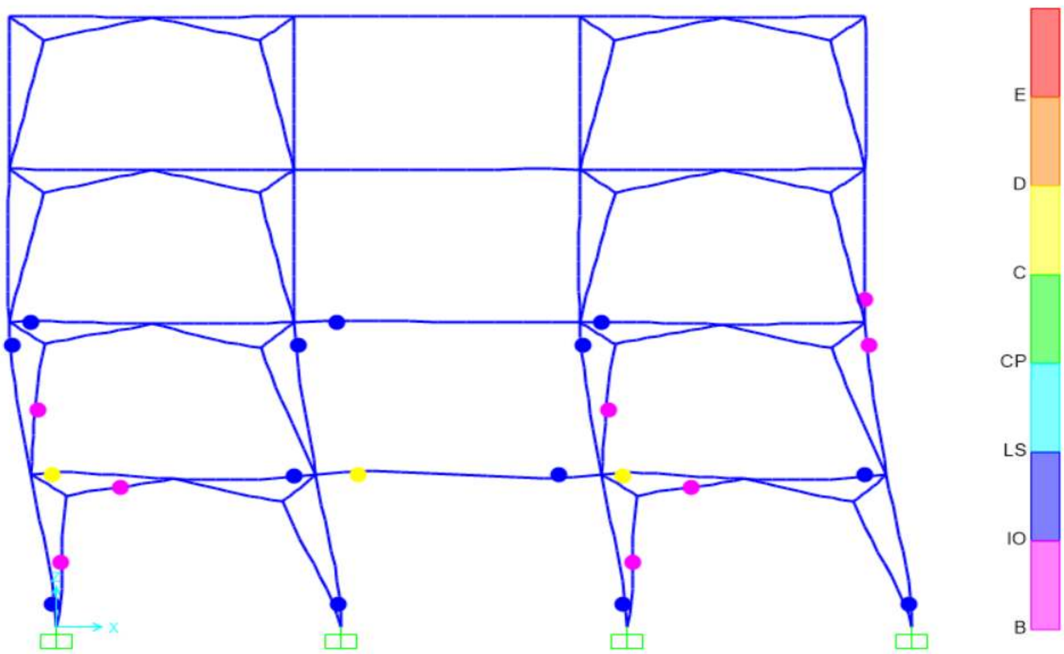

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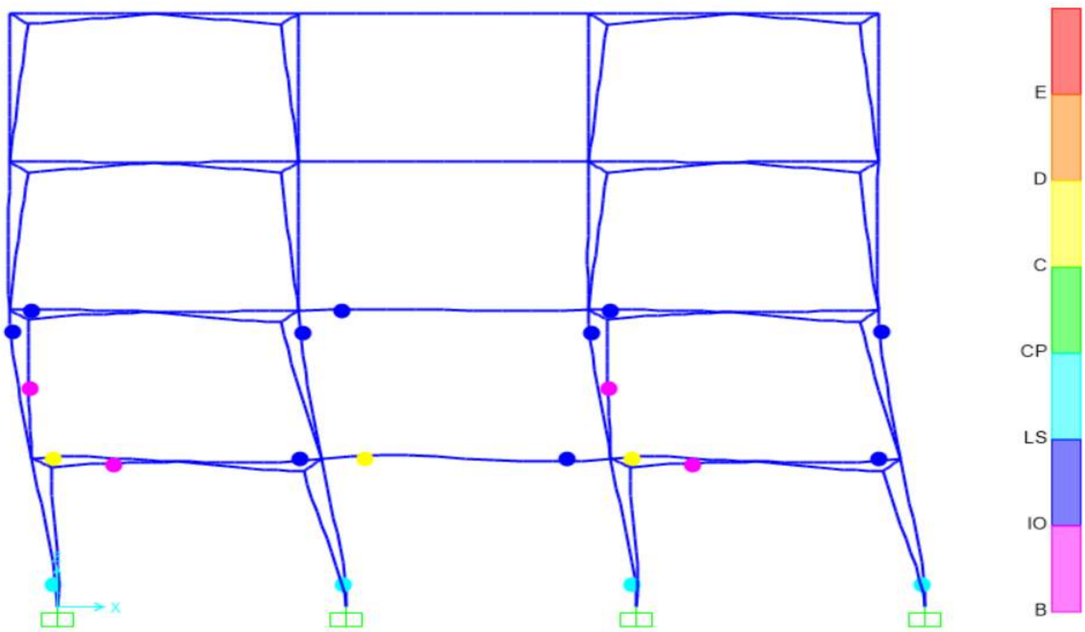
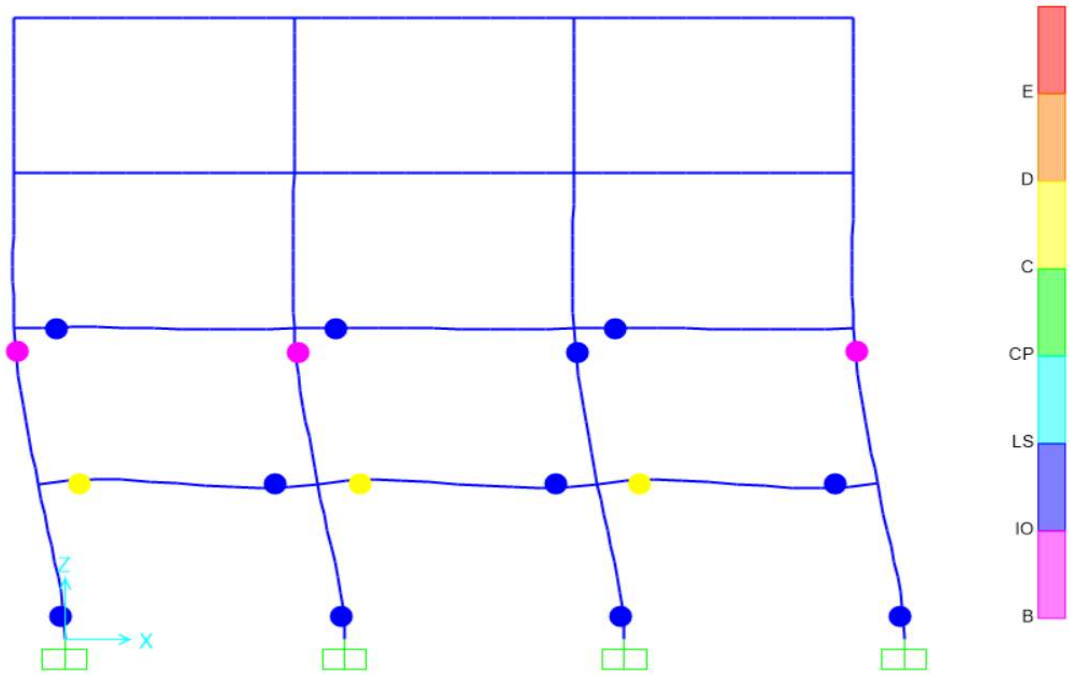


e)

Figure 4.36 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 4 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



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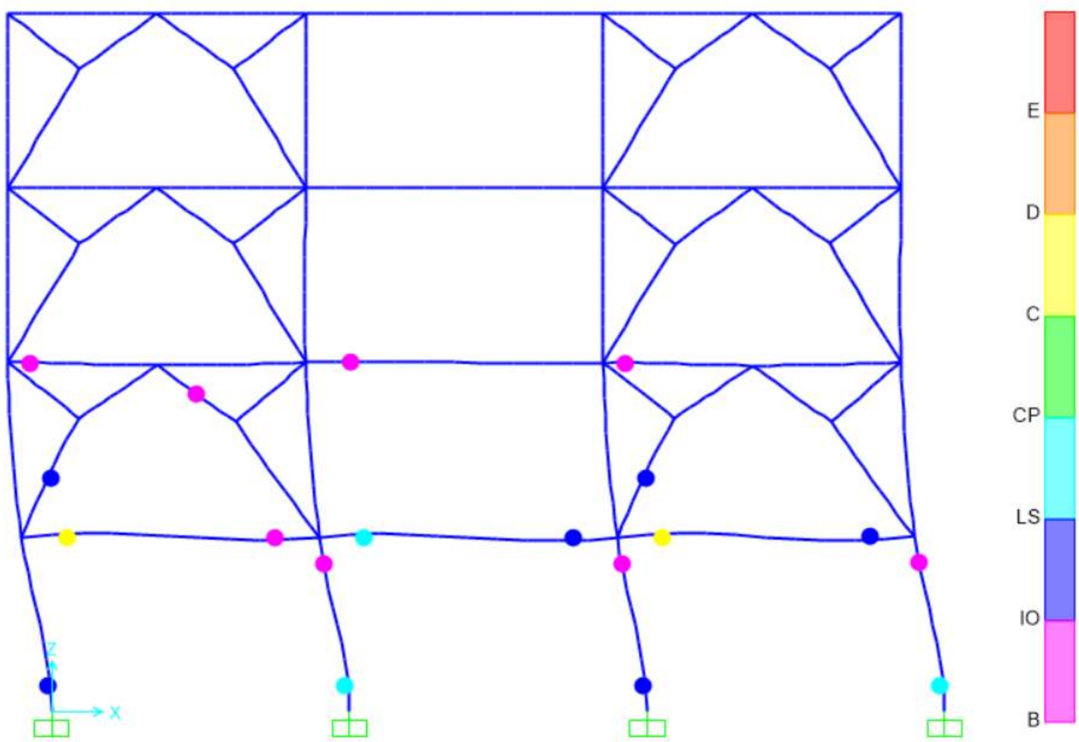

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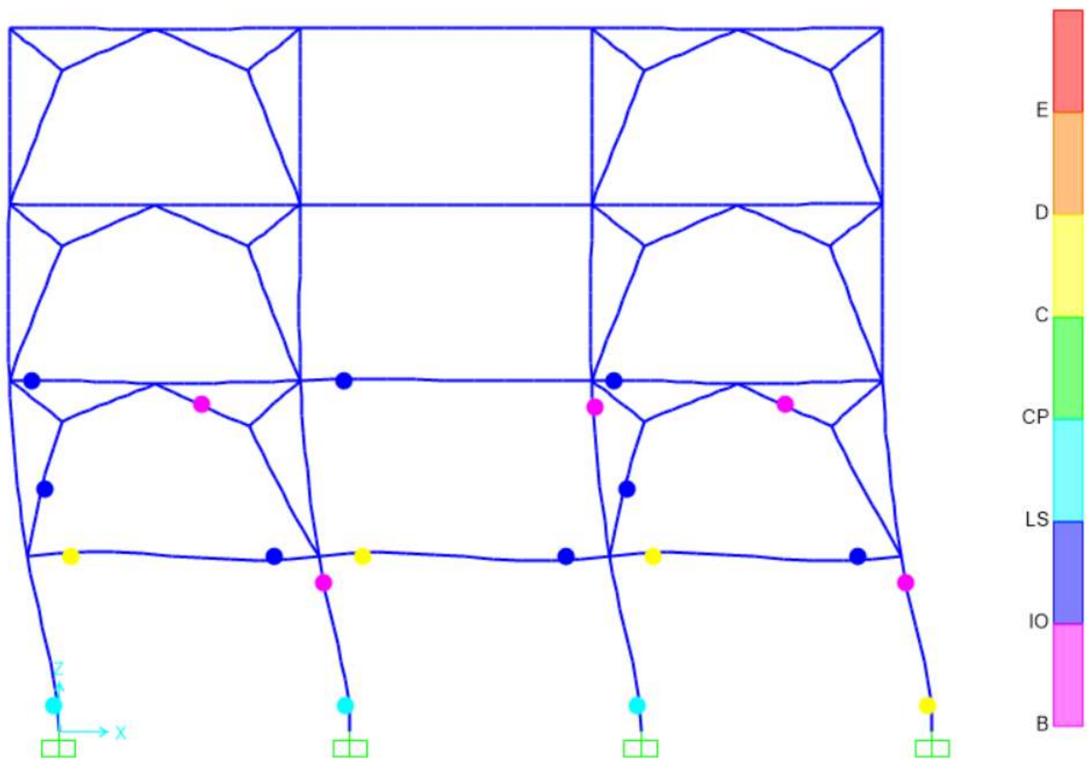

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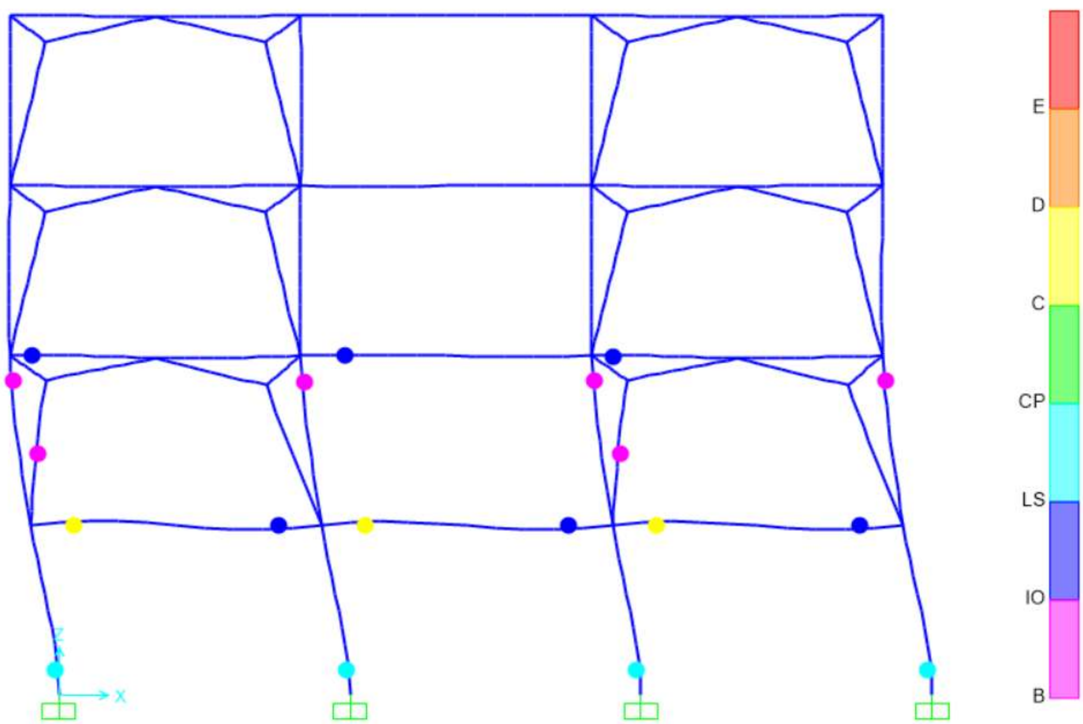

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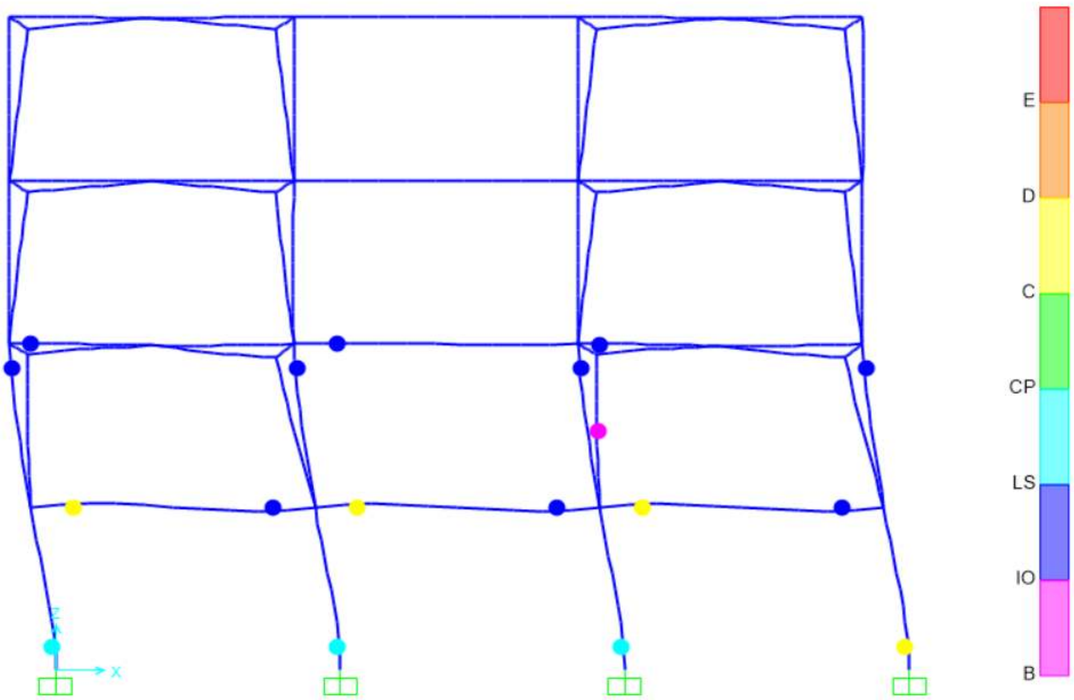
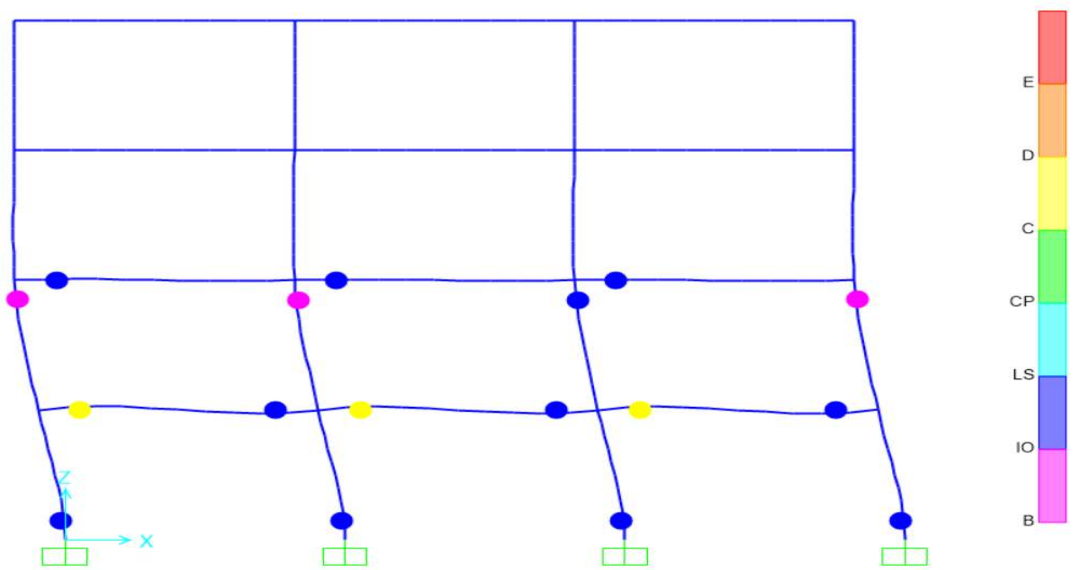


e)

Figure 4.37 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 5 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



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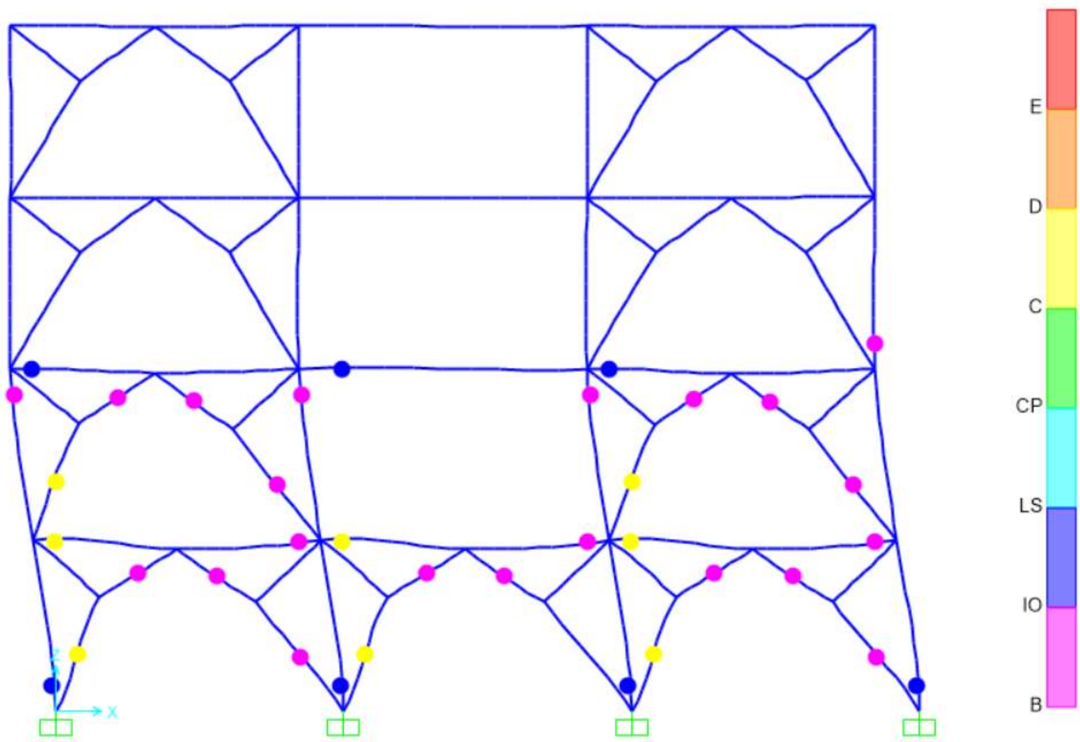

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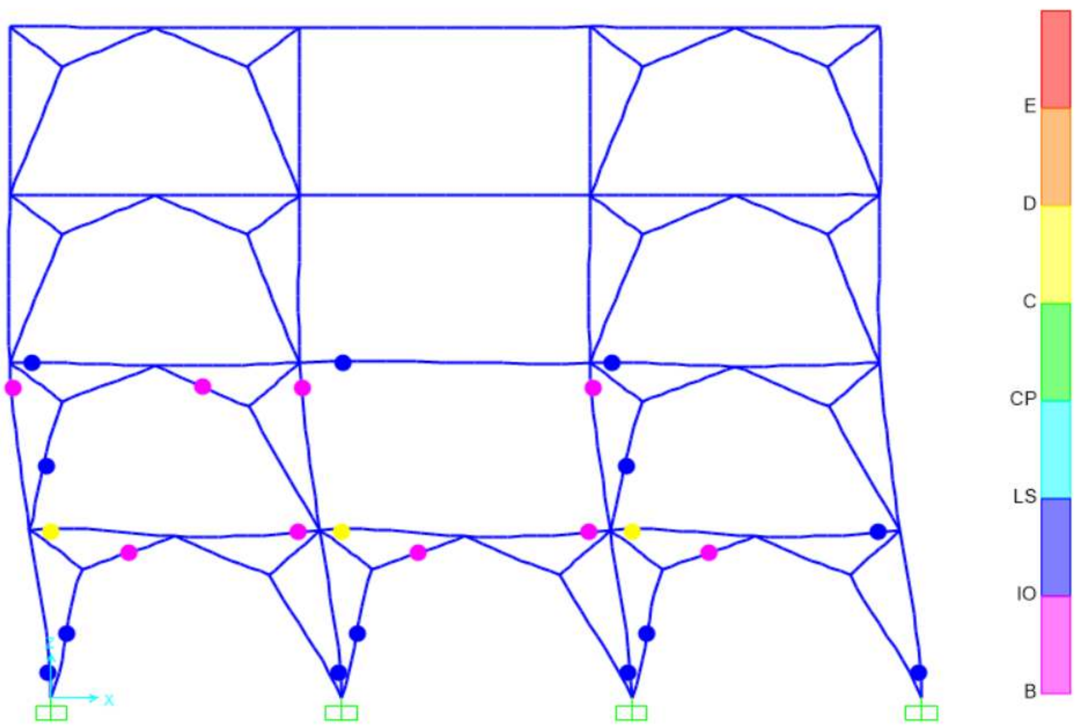

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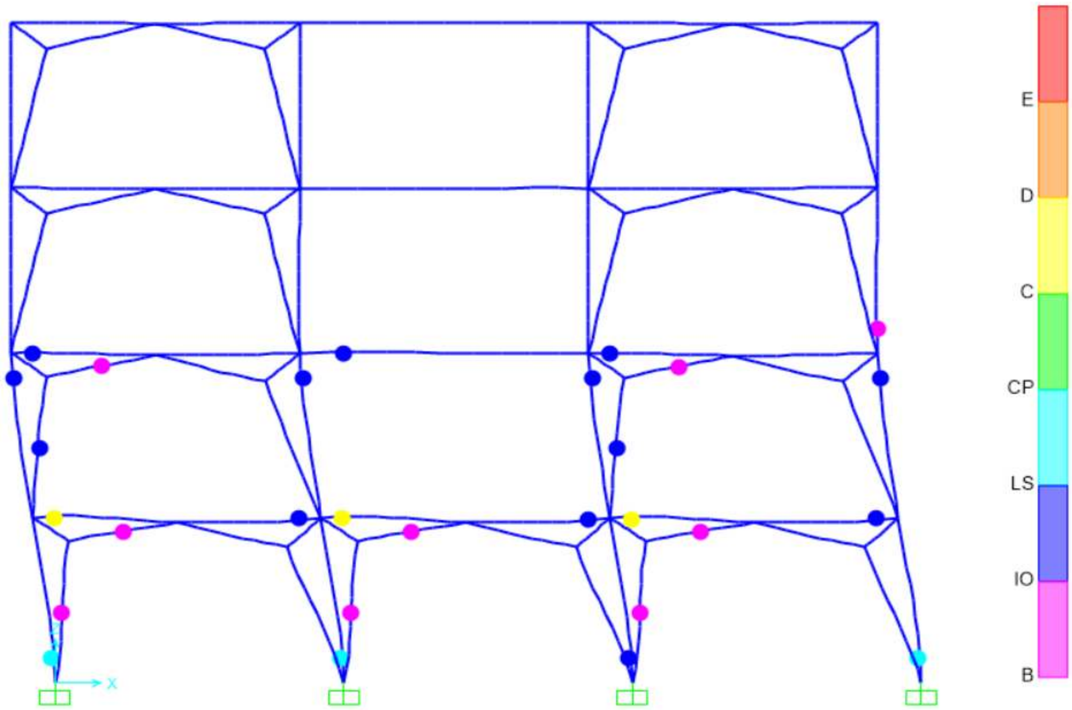

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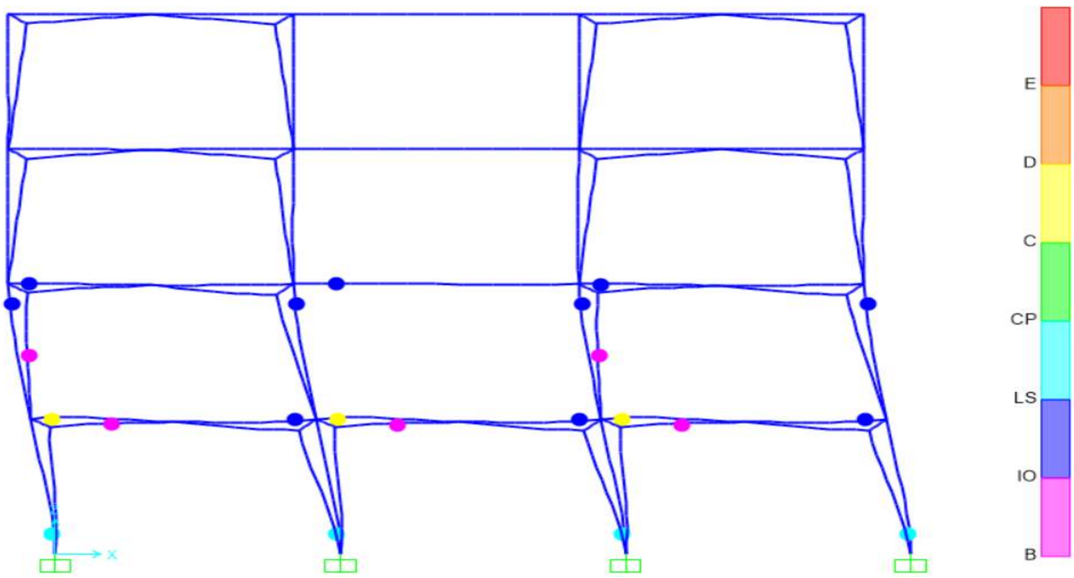
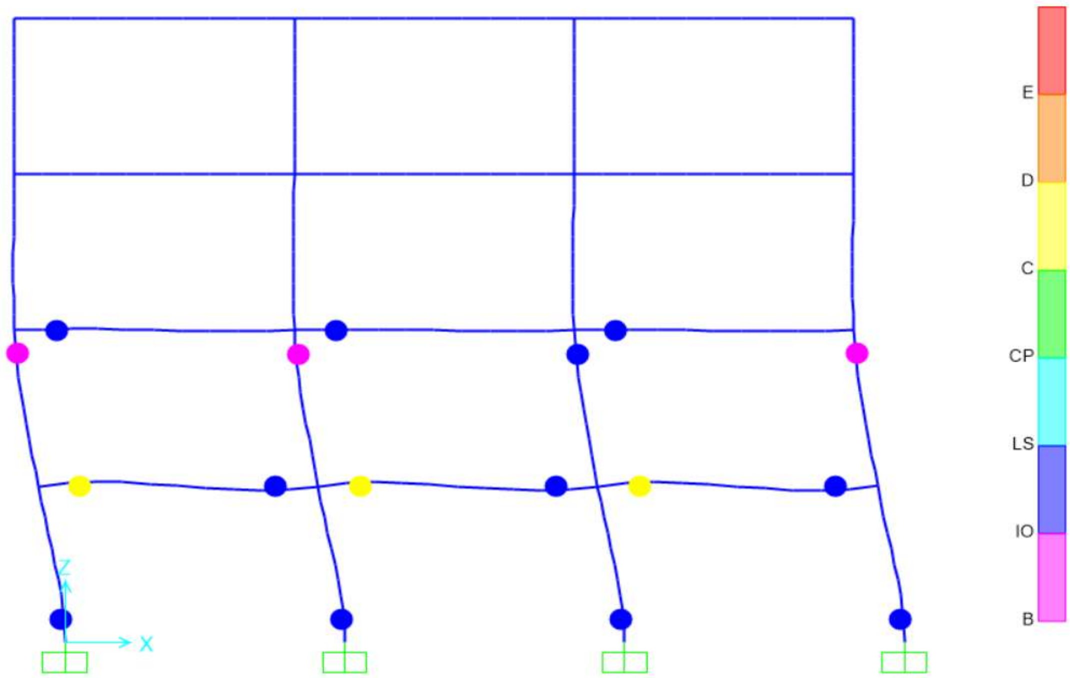
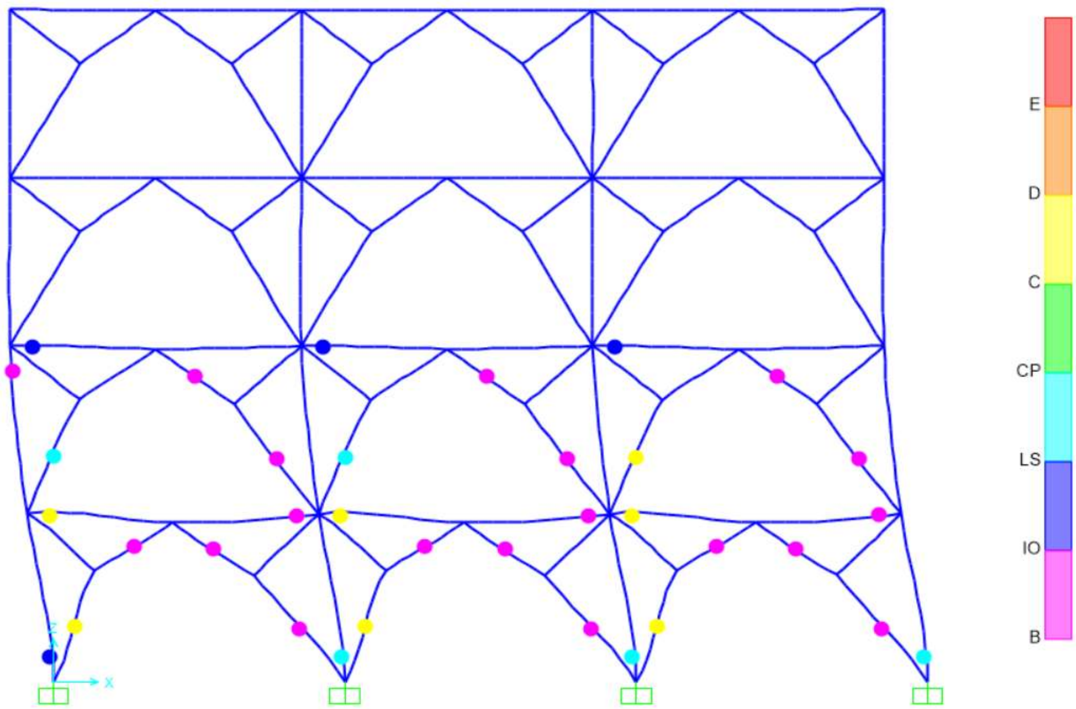


e)

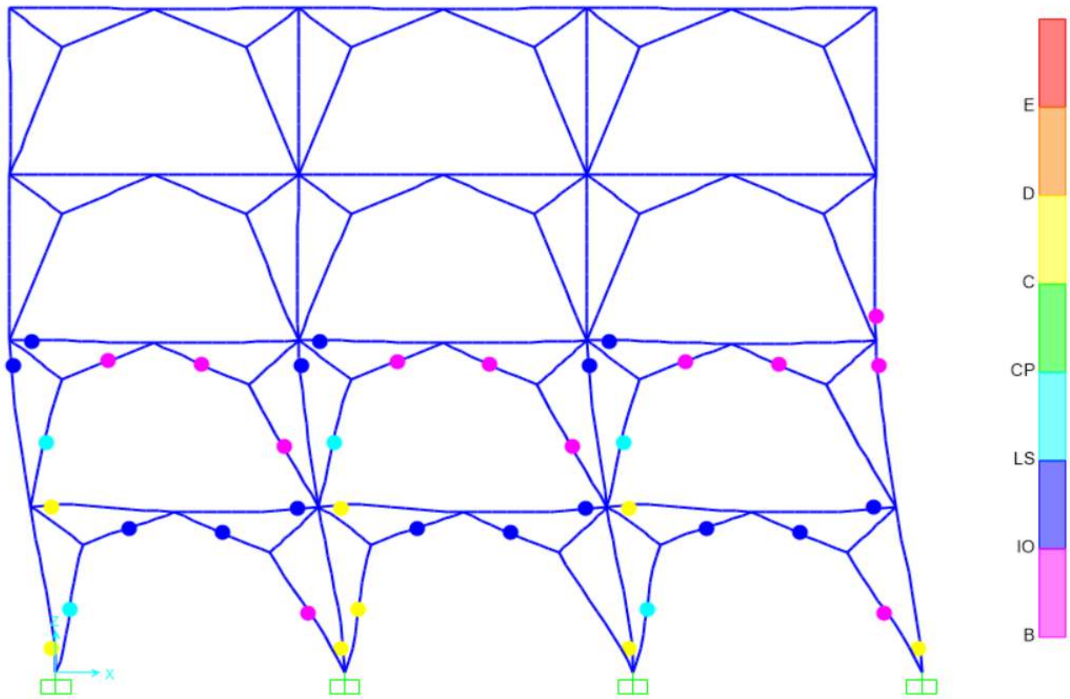
Figure 4.38 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 6 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8



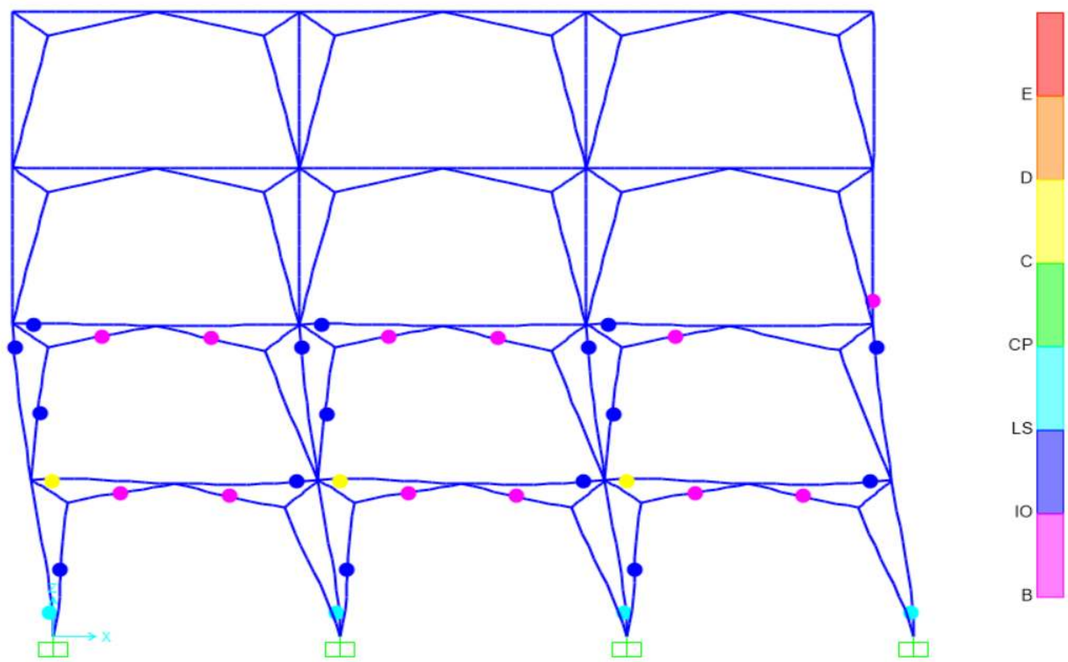
a)



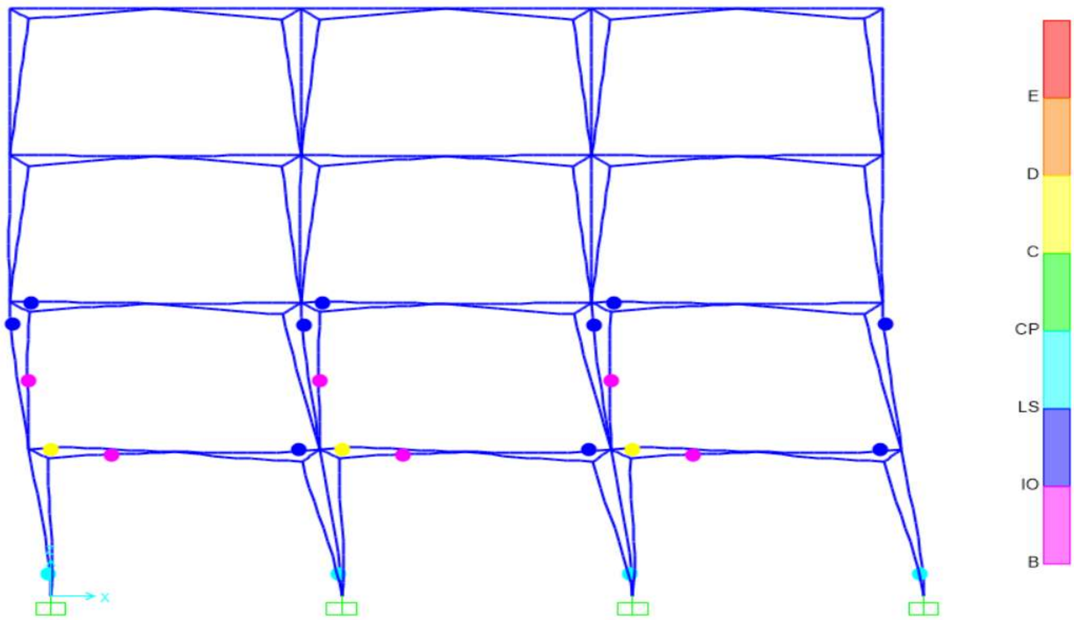
b)



c)

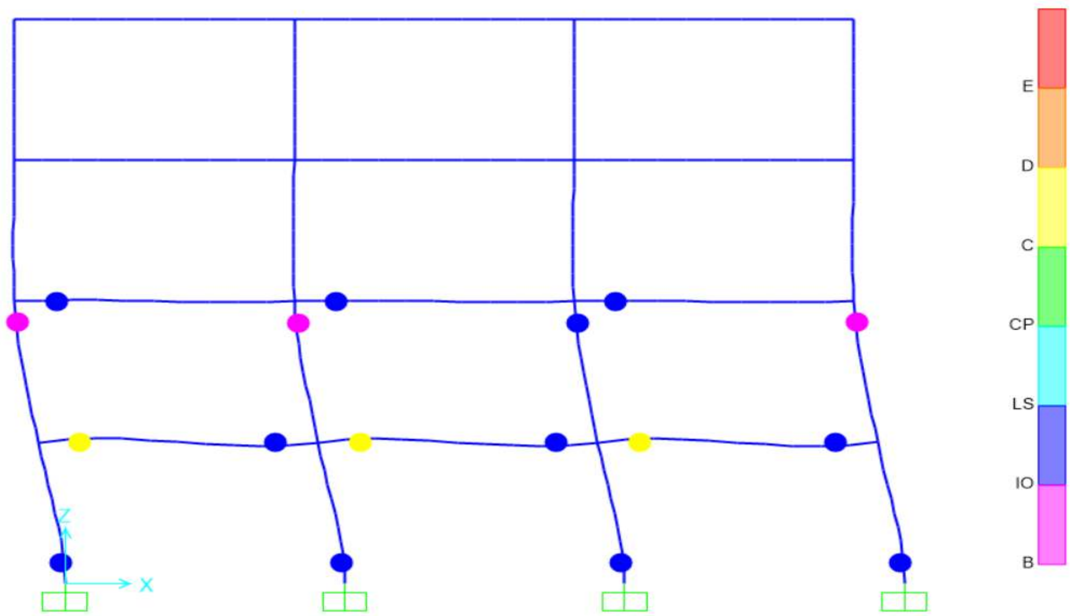


d)



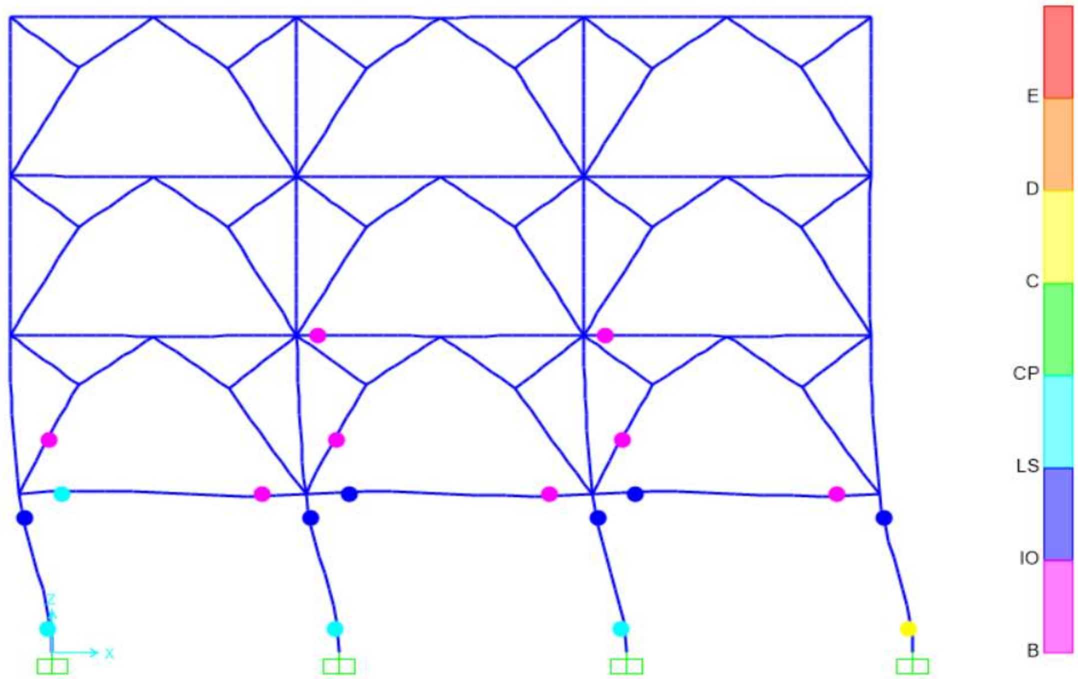
e)

Figure 4.39 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 7 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8

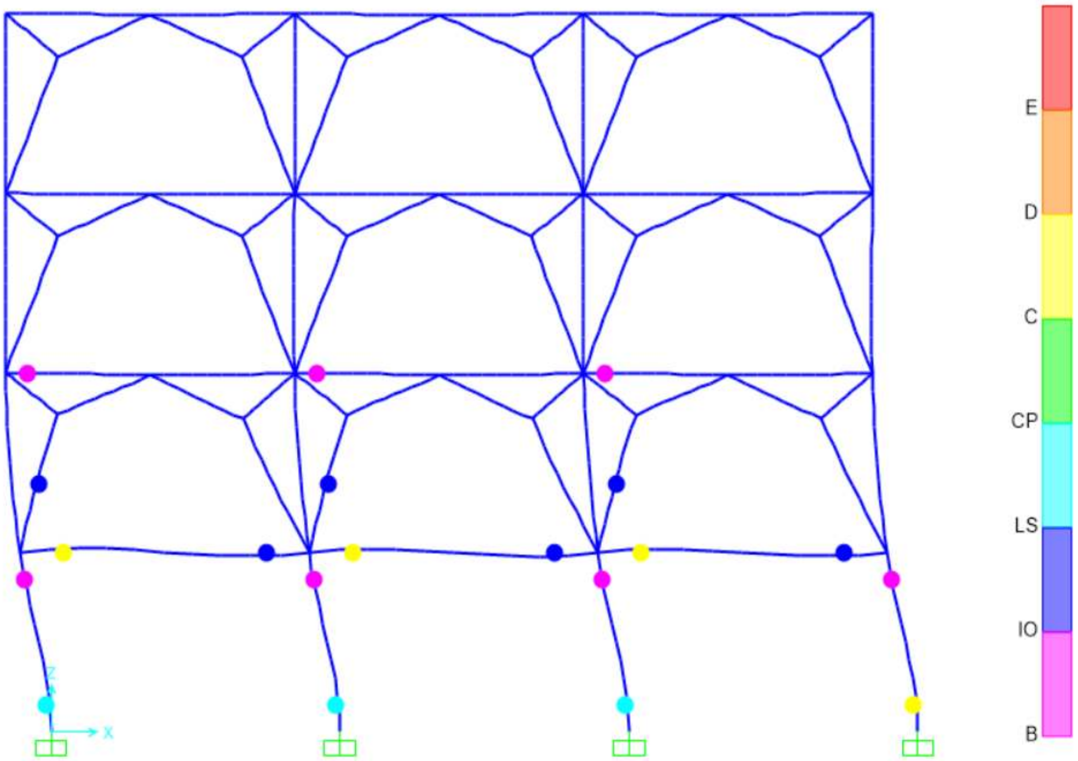


a)

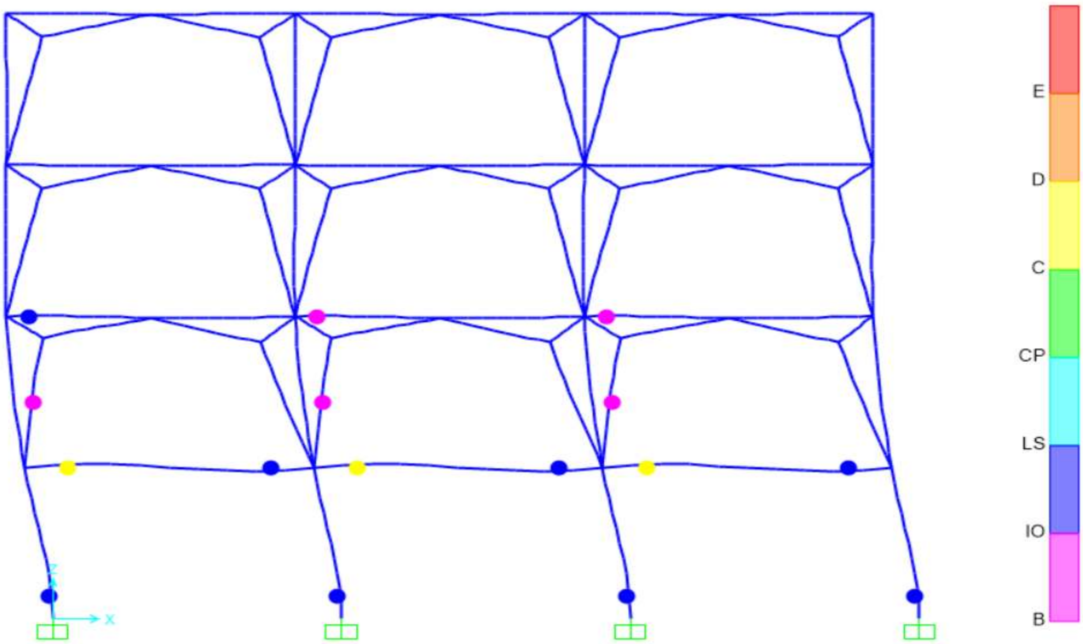
c)



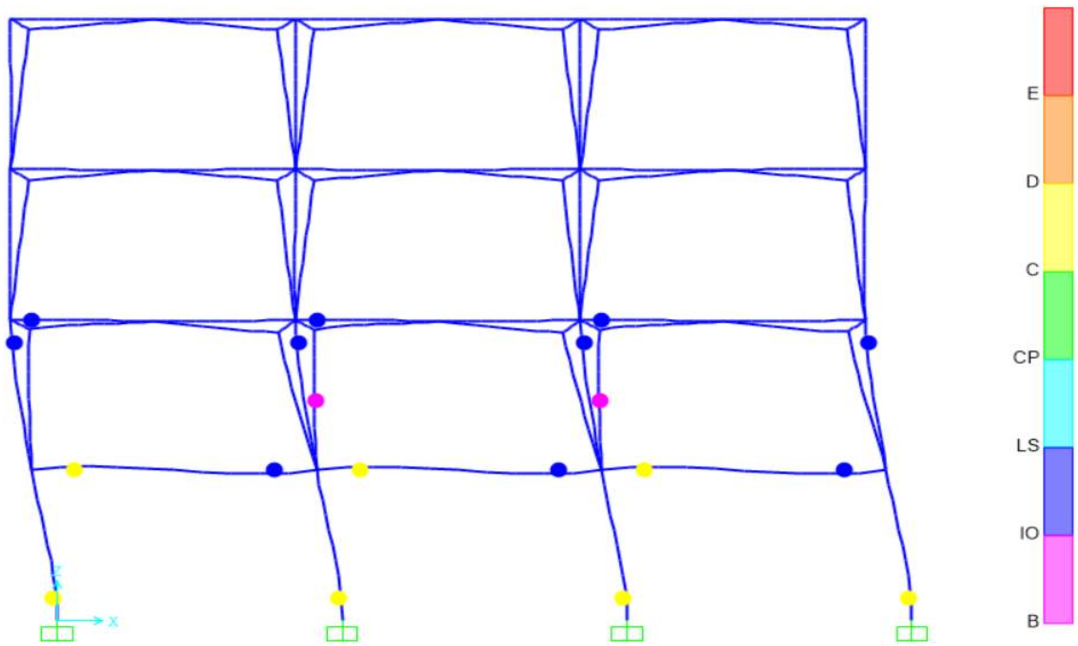
b)



c)



d)



e)

Figure 4.40 Hinge formation of the 4 storey buildings: a) the existing building and upgraded building with configuration 8 having an eccentricity of b) 0.2, c) 0.4, d) 0.6, and e) 0.8

CHAPTER 5

CONCLUSIONS

This study investigated the performance of the original buildings and those upgraded by the gate bracing system with different eccentricity and configuration. For this, the nonlinear pushover analysis was performed for 2, 4, 6, and 8 storey reinforced concrete (RC) frames by using SAP2000 program. The gate bracing system used had different geometric properties. A gate bracing system consists of six members that are regular and in each side, three members connected to the certain point in the frame bay were employed. Two of them were non-straight diagonal elements while the third one was stretched on one side and joined to the corner of the frame. The performance characteristic was evaluated in relations of the lateral load carrying capacity of the structures. From the results of this study, the main conclusions are as follows:

- The analysis of the results exhibited that at large the building with gate bracing system provided higher load carrying capacity than the existing building in all configurations.

- For 2 storey building, it was observed that the eccentricity parameters of gate bracing system in 8 different configurations had remarkable effect on the performance of the structure under seismic loading. In the configuration 2, configuration 5, and configuration 8 (using gate brace without ground floor), the results of the upgraded building with gate bracing was very close to the existing building. And for the other configurations, the use for gate bracing increased the capacity of the existing building about 1.14-2.08 times, depending mainly upon the eccentricity.

- Also, for 4 storey building, the use of the gate bracing increased the capacity of the existing building about 1.001-2.74 times. But, we noted that at the eccentricity parameter of 0.8 was less effective and very close to the original building's base shear in all configurations.

- Similarly, for 6 and 8 storey buildings the use of gate bracing increased the capacity of the existing building about 1.002-2.55 and 1.02-2.01 times, respectively. And also it was noted that at the eccentricity parameter of 0.8 was very close to the existing building's shear force in all configurations.

- For 2, 4, 6, and 8 storey buildings, irrespective of the amount of the eccentricity, the seismic performance of the upgraded building increased with the gate bracing system at configuration 7 (full gate brace).

- The eccentricity parameter of 0.2 appeared to be more effective in enhancing the lateral load carrying capacity of the braced building while 0.8 eccentricity was less effective. In other words, the variation of the eccentricity parameter from 0.2 to 0.8 unfavorably influenced the capacity of the structure.

- The plastic hinge formations were affected by especially eccentricity of the gate bracing and the use of different configurations. When the hinges in the existing building and gate braced buildings were compared, the plastic hinges in the beam and column members decreased in the braced buildings and nonlinear behaviour was observed in the braces at large.

- The gate bracing system might be considered an alternative upgrading or retrofitting method because of supplying the required space for openings in the building.

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