AN ASSESSMENT OF POUNDING EFFECT IN R.C. FRAMED STRUCTURES

A DISSERTATION

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OF

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IN

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CANDIDATE'S DECLARATION

I, **Kishan Singh**, Roll No. 2K/20/STE/12 student of **M.Tech. (Structural Engineering)**., hereby declare that the project Dissertation titled "**AN ASSESSMENT OF POUNDING EFFECT IN R.C. FRAMED STRUCTURES**" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of degree of **Master of Technology in Structural Engineering**, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship or other similar title or recognition.

Kishan Singh

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Date: 30-05-2022

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CERTIFICATE

I, hereby certificate that the Project titled "**AN ASSESSMENT OF POUNDING EFFECT IN R.C. FRAMED STRUCTURES**" which is submitted by **Kishan Singh**, Roll No. **2K/20/STE/12**, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of degree of **Master of Technology in Structural Engineering**, is a record of the project work carried out by the student under my supervision. To the best of my knowledge this work has not been submitted in part or full for any degree or diploma to this university or elsewhere.

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ABSTRACT

Pounding preventive gaps are usually not provided between buildings standing adjoining to each other in cities having high land prices or which are unplanned. Such buildings may collide to sustain major structural damages including reduction in overall stiffness of the structure which in extreme cases may lead to complete collapse under seismic activity. Various strengthening measures which can be adopted is proposed along with the optimum position of installation near the beam-column location. This paper discusses about the installation of Buckling resistant bracings (BRB), steel bracings and shear wall which proved to be beneficial in mitigating the pounding phenomena and greatly reduces the lateral deflection. Here two models are considered having different structural dynamics property each is analysed firstly with bare frame and then after properly strengthening with bracings and shear wall. Five spectral matched and matched ground motions are also provided for analysis. The method used for analysis after placing bracings or shear wall includes majorly non-linear pushover analysis to check the possible hinges and capacity of structure and then non-linear time history analysis is performed to know the actual behaviour of the buildings after strengthening. The analysis is repeated by shifting the bracings and shear wall at various locations in both the buildings one after another.

The result was checked by comparing the sum of lateral displacements of both the buildings at concerned storey level before and after the strengthening is done. From the result various conclusion are drawn to get the optimum locations of Buckling resistant bracingss, steel bracing and the shear wall. It is found that all the locations of bracings were capable of mitigating pounding effect and the one which has least value is taken as the optimum location.

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ABBREVATIONS

SMGM	=	Spectrally matched ground motion
IDR	=	Inter drift ratio
BRB	=	Buckling resistant bracings (BRB)
IS	=	Indian Standard
EQx	=	Earthquake in X axis direction
DL	=	Dead load
LL	=	Live load
SDL	=	Super Dead Load
ATC	=	Applied technology council
FEMA	=	Federal Emergency Management Agency
ISMB	=	Indian Standard Medium Weight Beam
LS	=	Life Safety
IO	=	Immediate Occupancy
CP	=	Collapse Prevention

CHAPTER 1

INTRODUCTION

1.1 GENERAL

From the emergence of global technology in the 21st century particularly in urban areas where the rate of development of infrastructure projects is showing increasing trend which results in huge population migrating to urban areas for this purpose which eventually increases the demand of land and hence it's cost. A considerable percentage of city area lies in unplanned zone where buildings are present in close proximity to it's adjacent one that in order to utilize proper land area buildings are built together without considering any gap even for the thermal expansion as thermal joints. This results in the congestion of both population and land area availability per building.

Availability of less land area creates problem at the time of unpredicted events of earthquake which may result in collision of one building with the other. The collision magnitude becomes critical when both the buildings have different heights that is they will have different dynamic properties result in abrupt behavior of building resulting in high lateral storey displacements

Major cities of India lies either in seismic zone IV or in V which creates a hazard to the life of people and are vulnerable for any future haphazardical events. This brings the idea to seismically design the buildings which can perform better during the designed seismic event by ensuring proper durability and serviceability of the building.

Indian standards has prescribed a set of guidelines in IS 1893:2016(Part I) for separation of adjoining structure under seismic effect, IS 4326:2013 for dynamic analysis of residential buildings having overall height more than 40m, IS 1893:2015 (Part IV) for separation of adjoining industrial buildings. It has specific guidelines majorly for low rise buildings but for high rise building having overall height more than 40m code recommends the use of dynamic analysis for the structure in order to maintain a seismic gap between two buildings.

When there are more than two buildings in a row which is common case in city blocks, the problem of pounding appears quite different since the interior buildings are subjected to two-sided imparts.

Pounding is normally related with a huge impact force developed due to the relative velocities of storey. Many research is going on to evaluate this impact force and it's effect on building's overall stiffness but in our study a attention has been made to avoid the development impact force at the point of contact by making building more stiff against it's lateral displacement. For this purpose, various checks and methods are adopted in the present study which are discussed in various chapters below.

Codes of various countries propose a minimum separation gap which needs to be provided but it appears to be ineffective in many cases due to insufficient amount of land.

1.2 IMPORTANCE OF THE ASSESEMENT

Buildings in urban areas are present at very close proximity due to high population density and high land prices which result in avoidance in providing seismic gap between two buildings by the owners to gain extra land and floor area.

Structure sustains several disturbances during the entire course of design life which may be due external events like earthquake, high winds, impact loadings like blast loading hence such loads either has high magnitude or high return frequency resulting in weakening of its strength and structure becomes relatively more flexible compared to when it was designed initially. This decreases overall performance of the building and can sustain higher lateral storey deflection values in future.

Below listed reasons demands the need to preventing pounding by proving bracings and shear walls at optimum locations possible.

(a) To prevent the collision between adjoining buildings.

(b) To increase the life span of the building.

(c) To provide best bracings among R.C.C and steel material.

(d) To locate bracings and shear wall only at certain important locations and not everywhere making building's strengthening cost less.

(e) Material savings if the optimum locations to place building is known.

(f) Savings in cost for future repair and damages which may occurr due to various future loadings.

1.3 OBJECTIVE

The objective of the work is as follows:

1) To minimise the pounding effect between given buildings using Steel bracing and to determine the optimal location of placing the same.
2) To minimise the pounding effect between given buildings using Buckling resistant bracing and to determine the optimal location of placing the same.
3) To minimise the pounding effect between given buildings using Shear wall and to determine the optimal location of placing the same.

Based on the above analyses this study intends to Compare the performance of Steel Bracing, Buckling resistant Bracing and shear wall to mitigate the pounding effect.

1.4 CODAL RECOMMENDATIONS

Many standard codes and guidelines failed to explain the pounding phenomena and the given provisions in them are not sufficient to control the pounding at desired level. Hence the need of dynamic analysis arises to discuss it.

Still some Indian standards code for building design recommends as discussed below.

As per IS 4326:2013 ,

- Buildings having different heights and hence different dynamic properties should be provided with some gap in between them to prevent pounding during earthquake occurrence.
- Dynamic analysis is to be performed if building height is more than 40m and based on their sum of storey dynamic deflection the gap should be provided and should be more than that.
- It is not necessary to provide separation gap below plinth level and footing which can be made continuous also.

A per IS 1893:2015(Part IV),

- Separation joint present in the same building or building adjacent to each other should be separated by as length equal to amount R(reduction factor) times the absolute sum of the maximum storey displacements to prevent collision.
- To above separation length replace R to R/2 add 25mm to the final value if the adjacent structures are at sane elevation level.

1.5 ANALYSIS METHODS

There are four procedures for seismic analysis of the buildings.

(a) Linear Static Procedure,

(b) Linear Dynamic Procedure,

(c) Nonlinear Static procedure and

(d) Nonlinear DynamicProcedure.

1.5.1 Linear Static Procedure

In this procedure, the distribution of seismic forces over the height of the building, and the corresponding internal forces and displacements are determined using a linearly elastic staticanalysis. Here, the building is modeled with linearly elastic stiffness and equivalent viscous damping. The static lateral storey shear forces represents the design earthquake demand imparted on the building and their sum is equal to the total base shear which is acting at thebase of the building. If the response is elastic, then internal forces will match to those forceswhich are expected during the design earthquake and if the response is inelastic, then the internal forces that are calculated on an elastic basis will be greater than internal forces developed while yielding.

1.5.2 Linear Dynamic Procedure

In this procedure, the distribution of seismic forces over the height of the building, and the corresponding internal forces and displacements are determined using a linearly elastic dynamic analysis. This method is almost similar to that of the previous one. The only difference is that the calculations of the response of the structure are determined with the help of modal spectral analysis or time history analysis. It is expected that this method will produce displacements and internal forces which will cross those response which is likely todevelop during yielding. This method consists of the Response Spectrum method and Timehistory analysis method.

1.5.2.1 Response Spectrum Method

In this method, the response analysis takes into account those modes that considers atleast 90% of the participating mass of the building in each of the building's horizontal directions.Damping of 5% is considered due to BRB structure. The peak responses can be member forces, displacements, storey shears and base reactions for each mode of response and it canbe evaluated by SRSS (square root sum of squares) method or the CQC (complete quadraticcombination) method.

1.5.2.2 Linear Time History Method

The requirement are same as that of Response Spectrum Analysis. Earthquake data as groundmotion input is used for carrying out this analysis. For each earthquake records, response parameters need to be evaluated. The maximum of the above are taken for design of structures. If more number of earthquake records are used, the average of the responses shallbe considered.

1.5.3 Nonlinear Static Procedure

In this procedure, the structure is subjected to a target objectives and resulting requirements are evaluated. The structure is subjected to increasing lateral load or displacements upto thatcondition when building collapse condition arises. The target displacement means the highest displacement which is likely to be occurred during the design earthquake.

1.5.3.1 Pushover analysis

Due to recent advancement in performance based seismic design, the nonlinear static pushover analysis procedure became an important topic for structural engineers. Pushover analysis is a static nonlinear procedure in which the loading is continuously increased upto the failure condition. Static pushover analysis helps to find out structure's real strength. Themodelling techniques are clearly mentioned in ATC-40 and FEMA-273 documents.Thecriteria for nonlinear hinges are given in these documents. Figure 1.1 shows three performance level named IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention) which are used to define the force deformation behavior of the hinges.

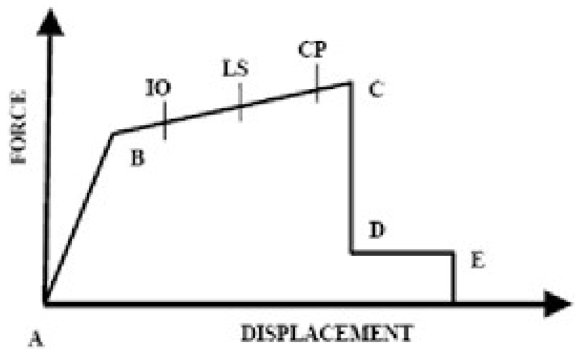


FIGURE 1.1 Force- deformation curve for Pushover Hinge. (Source: Chapter 06, FEMA 273)

1.5.4 Nonlinear Dynamic Procedure

In this procedure, the distribution of seismic forces over the height of the building, and the corresponding internal forces and displacements are determined using an inelastic dynamicanalysis. The modelling approaches are similar to those for the nonlinear static procedure. The only difference is that the response parameters are calculated using Time history analysis. The response of the structure are determined through dynamic analysis using earthquake data records.

1.5.4.1 Time History analysis

Nonlinear time history analysis helps in the verification of structure's performance. It is used to evaluate the system performance. Time history method calculates building response at discrete time steps using time histories as ground motion. For the nonlinear time history analysis, spectrum matched ground motion records should be selected from actual earthquakes considering magnitude, distance, site conditions and other parameters that control the ground motion characteristics.

1.5.4.1.1 Inter-storey Drift

Interstorey drift ratio (IDR) is defined as the subtraction of storey displacement of upper storey minus storey displacement of lower storey divided by its storey height. It is calculatedfrom time history analysis by finding the drift for each time for each storey and then the maximum absolute value is taken as the interstorey drift of that particular storey. The Figure 1.2 shows interstorey drift diagram.

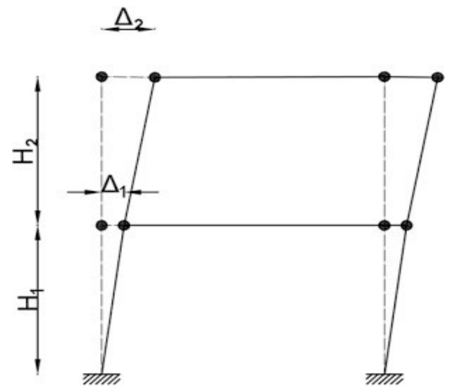


FIGURE 1.2 Interstorey Drift. (Source: Sustainability 2015, 7(10), 14287-14308).

1.6 DESIGN PROCEDURE AS PER IS 1893 (PART 1):2002

According to the code, the total design seismic base shear (V_B) along any principle directionis given by the following expression:

$$V_B = A_h .W$$

Where,

A_h=Design horizontal acceleration spectrum.

W = Seismic weight of the building.

The design horizontal acceleration spectrum (A_h) is given by

$$Ah=(Z*I*Sa)/(2*R*g)$$

Where

Z= Zone factor for the maximum considered earthquake (MCE). It is divided by 2 because maximum considered earthquake (MCE) is to be converted into Design Basis Earthquake (DBE).

I= Importance factor depending on the functional use of the structure.

R= Response reduction factor to take into account the damage suffered by the building dueto earthquake.

$\frac{Sa}{g}$= Average response acceleration coefficient for rock or soil sites based on appropriate natural periods and damping of the structure.

Figure 1.3 shows response spectra for different soil conditions for 5% damping

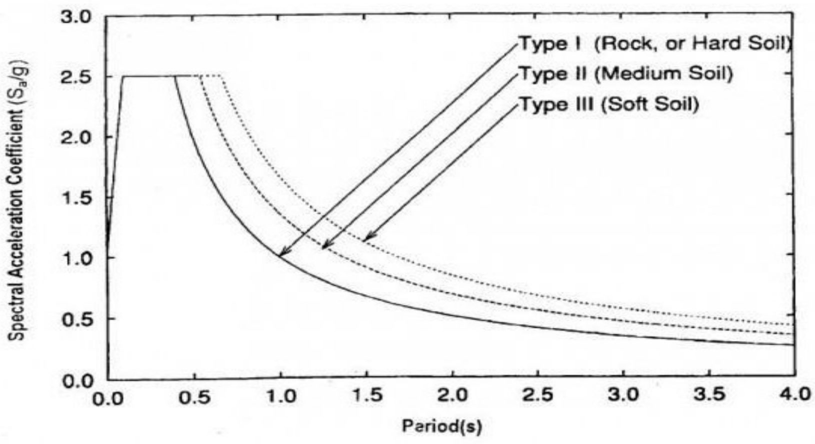


FIGURE 1.3 Response spectra for rock and soil sites for 5% damping. (Source IS 1893:2002)

1.6.1 Fundamental natural time period

The fundamental natural period of vibration (T_a) in seconds, of a moment resisting RC building without brick infill panels is given by

$$T_a = 0.075\ h^{0.75}$$

And for frames with brick infill panels, time period is given by

Where,

h= Height of building, in metre.

d= Base dimension of the building at the plinth level in metre, along the considered directionof the lateral force.

1.6.2 Distribution of design force

The computed base shear (V_B) shall be distributed along the height of the building as per thefollowing expression:

$$Q_i = V_B \frac{Wihi^2}{\sum_{j=1}^{n} Wj\ \ hj^2}$$

Where,

Q_i= Design lateral force at i^{th} floor. W_i= Seismic weight of i^{th} floor.

h_i= Height of floor i measured from base.

n= Number of level at which the masses are located.

1.7 BUILDING RESPONSE UNDER EARTHQUAKE

1.7.1 Building frequency and period

The acceleration of the building mainly depends on the input ground motion's frequency andbuilding's natural frequency. When both the frequencies become equal to each other, the response of the building reaches the highest level of amplitude and resonance condition arises. Resonance condition results in increase in building's response due to which buildingsuffer heavy damage from ground motion at a frequency close to its own natural frequency.

1.7.2 Building stiffness

As the height of the building increases, the natural period also increases. Taller building ismore flexible than shorter building.

1.7.3 Ductility

Ductility is the ability to withstand deformation without failure. To resist strong ground motion, the building must have enough ductility to undergo large displacement without failure.

1.7.4 Damping

All buildings have some inherent damping. Damping is the process due to which the amplitude of vibration reduces in due time. Earthquake resistant design and construction employ added damping devices like shock absorbers to amplify artificially the inherent damping of a building.

1.8 STRUCTURE OF THESIS

Chapter 1 initiates with the introduction part which briefs about the idea of pounding, it's history and significance. It also discusses about the need and importance of pounding.

Chapter 2 states the overview of various researches conducted and papers available on pounding phenomena. It gives an idea of codal provisions on pounding other than the Indian codes.

Chapter 3 deals with the methods available to check and study pounding mitigation measures, particularly the steps taken to achieve aim using ETABs v19 software.

Chapter 4 discusses about the objective and scope of the study. This would be required to achieve our aim.

Chapter 5 includes modelling and design consideration. It's purpose is to create a problem statement and to model the structure for further analysis.

Chapter 6 and Chapter 7 gives the comparison on the outcomes of model when designed without providing strength to the one which has been strengthened and analysed.

Finally, chapter 8 and Chapter 9 gives the result for optimum positioning of bracings and shear wall and then the conclusion is drawn out of it with some possible future works.

CHAPTER 2

LITERATURE REVIEW

Zhe Qu, Shoichi Kishiki, Yusuke Maida, Hiroyasu Sakata, Akira Wada (2015) stated about A new buckling restrained braced frame system was proposed in a previous study for reinforced concrete frames, which was featured by the zigzag configuration of buckling restrained braces to ease the steel-to-concrete connection. Experimental tests were conducted to establish realistic numerical models of the brace connections in the proposed system. With these numerical models, a nonlinear dynamic analysis of a prototype building was conducted to investigate the seismic behavior of the new braced frame system. The results indicate that the buckling restrained braces in the new system are efficient in reducing the responses of the building, even if the nonlinearity of the brace connection is considered. Furthermore, the strength demands for the brace connections are significantly influenced by higher modes of the system after the braces yield. The influences of nonlinearity of BRB connections on the seismic responses of the proposed system are assessed through nonlinear time-history analysis. The bolt-and-corbel connections for BRBs in the prototype building are proportioned according to the models derived from the test results to make sure that the selected properties are reasonable and practical. Five analysis cases with different sets of connection properties are studied to show that the flexibility of concrete corbels may lead to an increase in the inter-story drift of the entire building. The elastic deformation of post-tensioned bolts has little effect on the global responses, because the fact that the local tensile force in bolts arises from higher mode vibrations and does not coincide with peak inter-story drifts of the building. Higher mode effects are also responsible for the significant tensile force demand on the bolt connection, which may be overlooked from a static point of view and may lead to unsafe bolt design.

Mohamed A. N, Abdel-Mooty and Nasser Z. Ahmed (2017) discussed about the use of localized interconnections to prevent pounding in existing structure. For this a three-dimensional finite element modelling is done and analysed using non-linear time history method. Various cases of building heights and configuration are taken and the buildings are interconnected at different storey level to check the efficiency of the connection. 0.15g scaled El Centro ground motion was considered for this study. To solve the non-linear dynamic equation fast non-linear technique is used in ETABS software within the finite element programme method. The inter-connections were designed in order to minimise the impact force during seismic effect. Three types of connections were proposed namely steel plate connector, slab connector and column connector. To control the free vibrations of both the building the optimal location of connection was at the top floor of the shorter building in order to get the least seismic pounding effect. The findings also include that there is no or minimal effect of seismic pounding on same height buildings and the connections installed proof to be very effective in this case. Also, the additional thermal stresses at inter-connection joints and possibility pounding is greatly controlled when the floors are interlinked at an alternate vertical distances at various floor levels. The author failed to control vibration of shorter structure when it is connected with the taller one due to high seismic force being transferred through the joints. Also, there is scope to perform similar analysis with inter-storey pounding case when column of one building collides with the slab level of the others.

H. Naderpour, R. C. Barros, S. M. Khatami, and R. Jankowski(2015) suggested the usefulness of the impact force in knowing seismic damage caused by pounding to buildings and then based on its analysis necessary measures can be taken. Various parametric studies were done by varying gap size between adjoining buildings, coefficient of restitution, impact spring element stiffness and the impact velocities. To verify this parametric study, a non-linear numerical visco-elastic model was considered on a two single degree of freedom system. Four different ground motions were taken out of which San Fernando (1971) has the highest peak ground acceleration. It was analysed that impact force decreases by increasing gap size between the buildings, increasing the coefficient of restitution, decreasing the storey impact velocity and by decreasing the stiffness of the buildings. Relation between coefficient of restitution and damping coefficient proved to be efficient in describing pounding between buildings using non linear viscoelastic model of impact forces.

A. Formisano, A. Massimilla, G. Di Lorenzo, R. Landolfo (2020) stated the use of system external steel bracing arrangement of concentric nature to prevent the higher lateral displacements by performing seismic retrofit of the structure. A live building construction project as a case study was chosen to retrofit using N2 method by taking approach of capacity spectrum method. Emphasis was given in analysing the seismic performance of three type of models namely bare frame (BF),infilled frame (IF) and pilotis frame (PF), and to reduce the bracing to BRB surface interaction at coupling locations. Cross (X)bracings was used for this study. It was found that all type of models namely BFX, IFX, PFX with external cross bracings attached proved the aim of the study giving capacity/demand displacement ratios compared to the bare frame between 3.71 to 4.25,2.08 to 2.90 and 2.38 to 3.91 respectively. Finally analysis shows that most effective working of cross bracing is with the bare frame structure.

Konstantinos V. Spiliopoulos (1992) studied earthquake induced pounding in adjacent buildings. In this study, response of the buildings situated in a row is analyzed and studied.Amplifications of the response due to pounding is found to depend mainly on period ratios,mass ratios and different heights of the adjacent buildings. When masses of both the adjacentbuildings are similar, the stiffer building's response is more than the flexible one and whenthere is huge differences in masses, the building with lesser mass is more penalized than thestiffer building. Serious problems can be caused to the buildings of unequal heights. Due todifference in masses and periods, the small building is greatly overstressed. When the lowerbuilding is stiff and massive, the upper part of the taller building is greatly penalized.

Karanth P. (2016) studied effects of pounding in buildings. This study includes preventive techniques by introducing RC wall and optimizing it. For the analysis, 9 storey building consist of conventional beam column structural system adjacent to the 7 storey building consists of flat slab system is taken. The stiffness of the beam column frame systemis more than the flat slab system. RC wall prevents the buildings from collision. It is better to leave set back/safe separation gap according to FEMA 273-1997 when the buildings are in early stage of design. If buildings are old and are not in a stage to provide safe separationgap, then prevention measure should be taken by using retrofitting's like introducing new RC wall, Cross bracings, Dampers etc.

Abbas Moustafa (2014) studied damage assessment of adjacent buildings under earthquake loads. Here, input energy, energy dissipation and damage indices are used to study pounding of buildings. Damage indices represents damage in the actual buildings through damage states. They investigated the importance of giving separation gap and the building's yield strength on structure's response and damage indices. It is found that with the decrease in separation gap, damage indices increases. Further it is found that the energydissipation is higher in fixed base buildings than the buildings which are base isolated.

Konstantinos V. Spiliopoulos (1992) studied measures against earthquake pounding between adjacent buildings. In this paper, to deal with the problem of pounding, they provided an alternative way to the code specified separation distance. Use of filling of the gaps between the building with a material, connecting them structurally, or by using bumperwalls have been studied. Filling of gaps with an absorbing material did produce any favorable effects on the response of the building, but the accelerations are got greatly reduced. Structural connection was also not found suitable as it not only increases the response and penalize one of the two structures, while benefiting the other. Out of these, only bumper walls proved to be the best alternative to the seismic separation problem.

R. Sabelli, S. Mahin, C. Chang (2003) depicted the ground motion and structural characteristics that control the earthquake response of concentrically braced steel frames and identify improved design procedures and code provisions. The seismic response of three and six story concentrically braced frames utilizing buckling-restrained braces is conducted. A brief discussion is provided regarding the mechanical properties of such braces and the benefits of their use. Results of detailed nonlinear dynamic analyses are then examined for specific cases as well as statistically for several suites of ground motions in order to characterize the effect on key response parameters of various structural configurations and proportions. Results from this phase of the overall study indicate that Buckling-restrained braces provide an effective means for overcoming many of the potential problems associated with special concentric braced frames. To accentuate potential difficulties with this system, numerical modelling and design assumptions were intentionally selected in this investigation to maximize predicted brace demands and the formation of weak stories. Nonetheless, the predicted behaviour is quite good, with significant benefits relative to conventional braced frames and moment resisting frames. For the cases studied to date, response is not sensitive to R factors selected in the range of 6 and 8. Response appears to be sensitive to proportioning suggesting that further improvements in response may be obtained by better estimation of a structure's dynamic properties.

CHAPTER 3

METHODOLOGY

3.1 DEFINE THE 3-D MODEL

- Set design codes to Indian Standard for concrete and steel.
- Define the grid pattern as per the desired aspect ratio.
- Define material property, section dimension based on experience and chose the option "reinforcement to be checked" while defining beam column section property.
- Repeat step third for all kind of element to be used in structure like beam,column,slab,shear,wall,diaphragm, autoselect steel bracing option, autoselect BRB frame option etc.
- Assign all defined sections over desired location of grid lines to generate a 3-D skeleton. Assign fixed support at the base.
- Assign all the defined material properties to respective elements.

3.2 DEFINE LOAD PATTERNS

- Define load patterns like deal load, live load, earthquake load in X and Y direction as per IS1893:2016, super dead load.
- Add mass source with DL+LL+SDL, having multiplier of 0.25 for LL and for rest to be 1.
- Define the target response spectrum function as per IS 1893:2016 (Part I) and chose seismic coefficients as per site's specification like type of soil, response reduction factor, importance factor, zone factor etc.
- Now import time history functions from saved folder and define it also for both X and Y direction.

- For matching the imported time history ground motion with the target response spectrum use match to response spectrum option and select both the target and actual ground motion.
- Start the spectral matching first in frequency domain and if the spectral does not matches chose the time domain option and now it should be matched. This step is called scaling of ground motion for both X and Y axis and this has to be used in nonlinear time history function.
- Now go to load cases where dead, live, EQx, EQy, modal case was already defined. Add response spectrum in both X and Y directions by choosing target response spectrum and appropriate scale factor.
- Under load case add gravity load case of nonlinear static nature. Also add pushover load case with scale factor -1 with displacement control method.
- Now add time history load cases with nonlinear direct integration method and selecting nonlinear static gravity load. Define suitable scale factor, geometric nonlinearity option as P-Delta plus large displacements. The output time step should be chosen as per the time length for which ground motion is considered. Repeat it for both the lateral directions.
- Under load combination chose default design combo as concrete frame design and while applying steel bracing, shear wall also check the option of steel frame design and concrete shear wall design respectively.

3.3 ASSIGNING PLASTIC HINGES

- It is needed to perform capacity analysis and to know failure criteria under nonlinear stage of loading and material.
- Plastic hinges is to be assigned at beam, column, bracing arrangement as per FEMA 41-13 specifications.
- First select the desired element for which hinge is to be assign. Then go to assign tab, select hinges.
- Locate hinges at 0 and 1 relative locations from the end in beam and column but in case of bracing chose 0.5.

- The loading pattern to be selected is GRAVITY of nonlinear static type.
- Appropriate option to be selected while defining hinge at each type of elements.

3.4 MODEL CHECK & RUNNING LOAD CASSES

- First check model for any type of preliminary error. Once okay proceed to next step
- In advance SAPfire option chose multithreaded if analyzing nonlinear load cases.
- Then go to set load cases to run option. Select all or customize according to the analysis being performed.
- Linear load cases take almost no time to run the analysis while if running nonlinear static or nonlinear time history load cases takes some time depending on complexity of the structure.
- While running linear load cases the modal participation ratios must be checked and should greater than 90 percent from all modes. If not then modify number of modes under modal cases accordingly.
- Also check for the base shear from equivalent static method to get matched with response spectrum method.
- If base shear is not matching do scale modification as per IS 1893:2016(Part I).
- Once the linear analysis and design is complete one can proceed to nonlinear analysis.
- Similarly for nonlinear cases of static type, the hinges formation must be checked at each mode and all the analysis as per codal provisions of FEMA & ATC 40 has to be carried out to completely analyse the structure.

3.5 OBSERVE THE ANALYSED RESULTS

- All the required outputs like joint displacements, storey drift, joint acceleration, base reactions, modal participation ratios, shear force and bending moment diagrams etc.,

- All above results can be easily viewed under display tab for show table section.
- Chose structure output data and tick the appropriate option to view the results.
- The output data can also be viewed in Microsoft Excel software using export option under file tab.
- The graphical form of data can be represented which can help in knowing the trend of analysed results so that a proper conclusion can be drawn.

3.6 FINAL BUILDING DESIGN

- Upon successful analysis the building is to be designed as IS 456:2000 for concrete frame element and IS 800:2007 for steel elements.
- This operation can be performed under concrete frame design tab where the start design check can be chosen.
- Upon design analyse the result for each element type and check for any possible failure.
- If any element fails re-design, it by modifying it’s section property and re-run the design option.
- Finally the building is designed to take appropriate loads under seismic activity and from the joint displacement result obtained it was find that pounding is greatly prevented after optimizing the frame with strengthening elements.

CHAPTER 4

MODELLING & DESIGN CONSIDERATIONS

4.1 MODELLING

The current study considers two buildings, one having 8 storey and the other has 5 storey. All the buildings are fixed in the base. The height of the bottom storey is taken 3.5m from the plinth level and rest storey being 3m height. While the plinth level is kept at 1m form the fixed support.

Different plans are also proposed for both the types of buildings. The details of the buildings are given below in Table.4.1

No. of storeys	No. of bays along X direction	No. of bays along Y direction	Bay length along X (m)	Bay length along Y (m)	Beam (mm)	Column (mm)
8	4	5	4	4	550x350	750x550
5	3	5	4	4	450x300	600x400

TABLE 4.1 Building details.

The plan and elevation of the two buildings are given below in Figure 5.1 and 5.2.

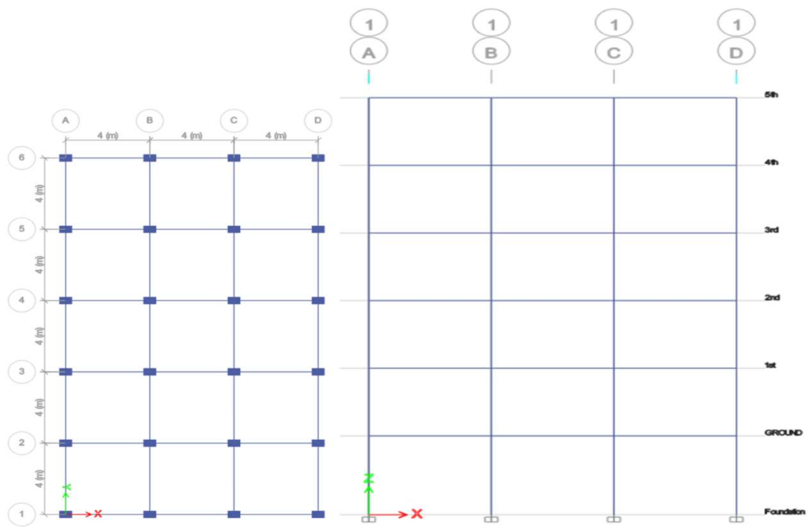


FIGURE 4.1 Plan and elevation of 5 storey building.

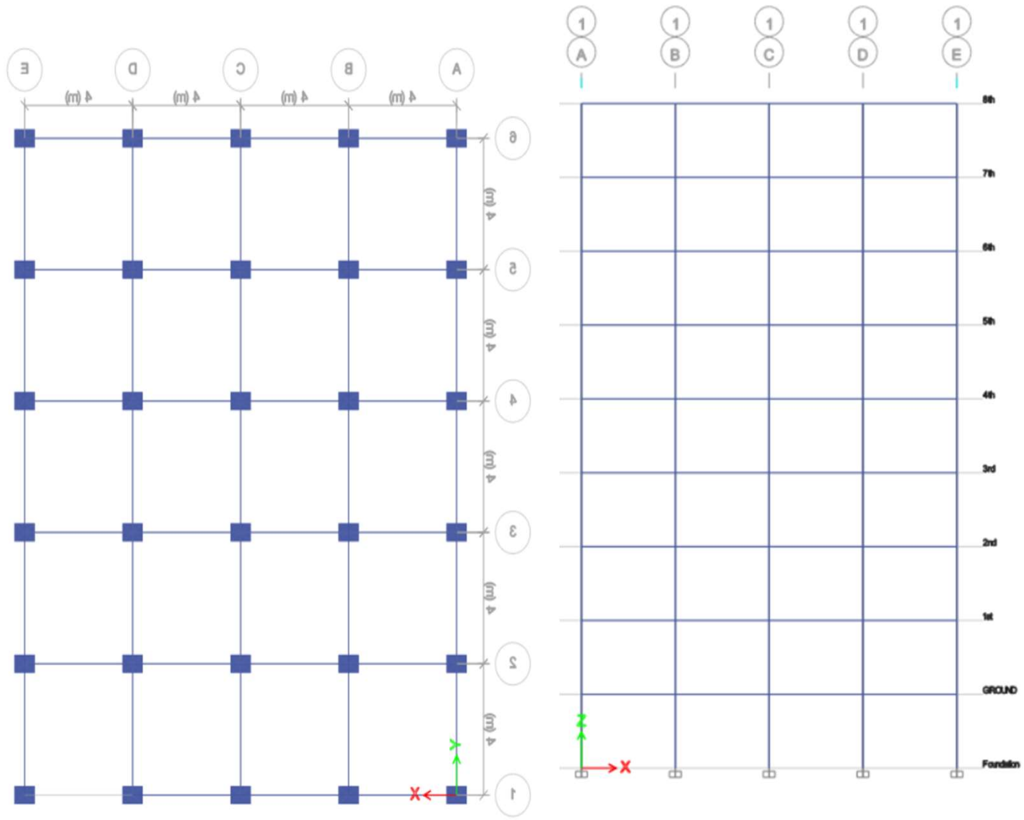


FIGURE 4.2 Plan and elevation of 8 storey building.

4.1.1 Material property considered

Materials taken for the design are concrete of characteristic strength for the beam as 30 MPa and for the column as 40 MPa, reinforcement steel of yield strength for longitudinal main reinforcing bars of 500 MPa and for confinement shear bar of 415 MPa.

4.1.2 Other details considered

Floor and roof slabs are modeled as a rigid diaphragm element having thickness of 125mm for five stories and 140mm for eight stories buildings. 2.5 kN/m2 of live load on floor and 1kN/m2 of live load on roof of eight storey building while the live load of 3kN/m2 on floors and 0.9kN/m2 on roof of five storey buildings is considered. The overall seismic weight is calculated as per the codal provisions of IS 1893:2016(part-I). The unit weight of concreteis taken as 25 kN/m3. The earthquake zone V and medium type soil is considered in the study. Since it is BRB structure, 5% damping is considered. The building is a residential building having importance factor of 1 with response reduction factor of 5.

4.1.3 Load combinations considered

(a) DL+LL

(b) 1.2 (DL+LL±EQx)

(c) 1.2 (DL+LL±EQy)

(d) 1.5 (DL±EQx)

(e) 1.2 (DL±EQy)

(f) 0.9 DL±1.5EQx)

(g) 0.9 DL±1.5EQx)

Whenever other load cases like time history in X and Y direction , response spectrum in X and Y direction, pushover in X and Y direction are taken for analysis ,it has to be made auto-assign after choosing concrete frame design as per IS 1893:2016(Part-I) while defining load patterns in ETABS v19 software.

4.2 PUSHOVER ANALYSIS

For this analysis, a separate joint is created on the roof at centre of gravity of the building to find out the roof displacement at that joint as pushover curve evaluates base shear vs roof displacement curve. Pushover analysis is done considering both the lateral directions that is X and Y direction. Nonlinear hinges are assigned to all the members at 0 and 1 relative distance for beams and columns. Finally when the analysis is done, the capacity of the building to take the base shear can be evaluated.

4.3 TIME HISTORY ANALYSIS

4.3.1 Earthquake data considered

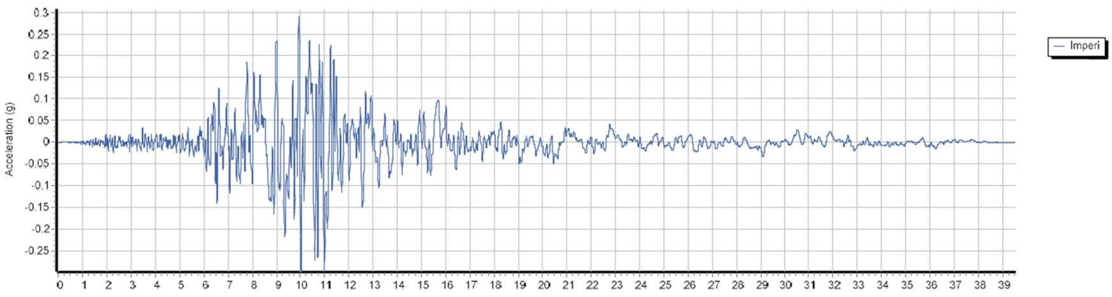
For nonlinear seismic analysis, the ground motion has to be represented through time histories. Five Spectrum Matched Ground Motion (SMGM) has been generated using seismoMatch v22 and ETABS v19 software . For this five different earthquake records are taken from PEER (pacific earthquake engineering research center) ground motion database site and are converted into SMGMs (Spectrum Matched Ground Motion) by

ETABS v19 software by taking target response spectrum as per the IS 1893:2016 (part-I). The Table 4.2 below shows the earthquake location, date of occurrence, its magnitude and duration of occurrence.

S.NO	DATE	PLACE	MAGNITUDE (Mw)	DURATION OF ANALYSIS (sec)
SMGM 1	09.02.1971	San Fernando	6.61	17.19
SMGM 2	16.01.1995	Kobe Japan	6.9	19.17
SMGM 3	19.05.1940	Imperial Valley-02	6.95	32.29
SMGM 4	09.04.1968	Borrego Mtn	6.63	15.95
SMGM 5	22.03.1957	San Francisco	5.28	23.82

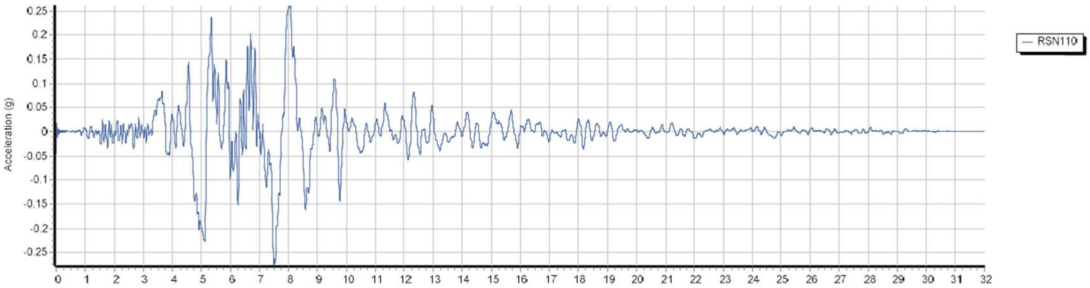
TABLE 4.2 Details of spectrum matched ground motion used

The acceleration vs time graphs of above ground motions considered in this study are shownbelow in the following figures.



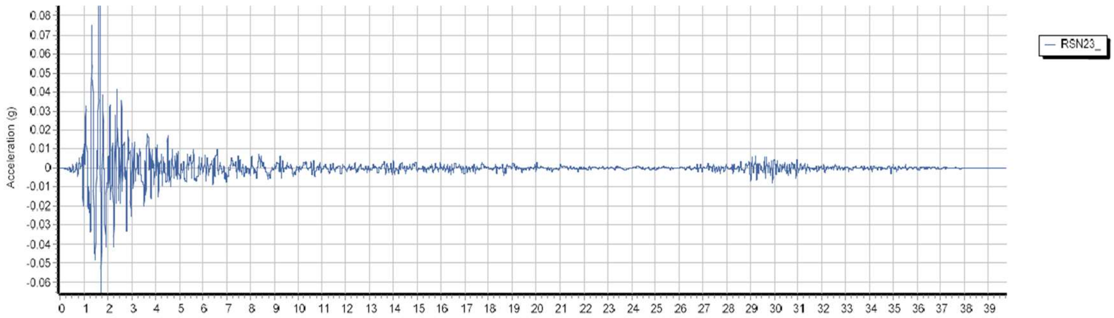
Time (sec)

FIGURE 4.3 Imperial Valley-02 Ground motion



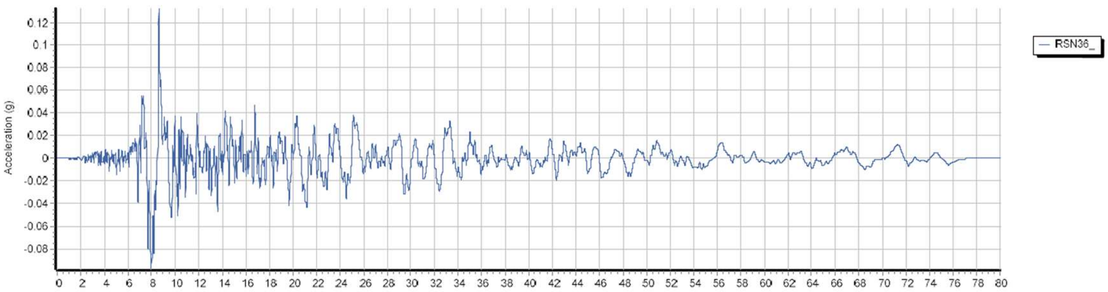
Time (sec)

FIGURE 4.4 Kobe Japan Ground motion



Time (sec)

FIGURE 4.5 San Francisco Ground motion



Time (sec)

FIGURE 4.6 Borrego Mtn Ground motion

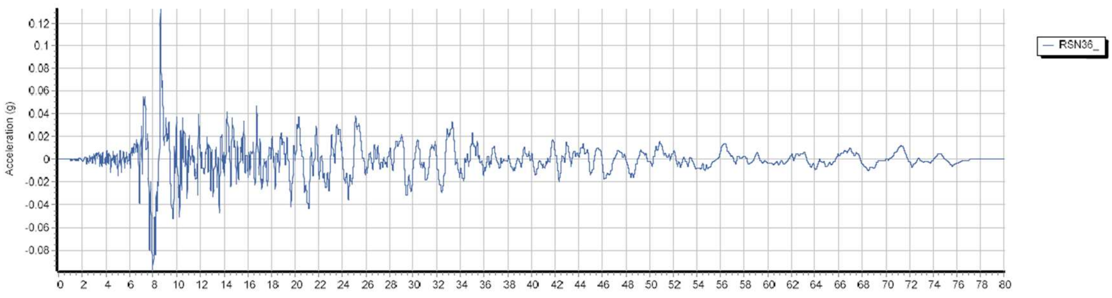


FIGURE 4.7 San Fernando Ground motion

4.4 BRACING

Bracing is used as a measure to control the severity of pounding and hence it is intensively being used in the present study. Two types of bracings are used which is discussed below in Table 4.3 & Table 4.4.

4.4.1 BRB bracing

BRB bracings is used by selecting auto-select option of composite section type in ETABS. Its's properties is discussed as below in Table 4.3

STOREY	MATERIALS	SECTION (mm)
8 storey	M30 & ISMB 200	260x380
5 storey	M30 & ISMB 125	210x320

TABLE 4.3 B R B Bracing properties.

4.4.2 Steel Bracings

Steel Bracing is provided in ETABS under auto-select section property which has properties as given in Table 4.4 below.

STOREY	MATERIAL	SECTION (mm)	Depth of section (mm)
8 storey	Fe250	ISMB 250	250 mm
5 storey	Fe250	ISMB 150	150 mm

TABLE 4.4 S t e e l Bracing properties.

4.5 SHEAR WALL

In this study RC shear wall has been used in between the two columns but very minor gap has been kept in between the outer face of the column and outer face of the shear wall in both the opposite sides of it. This minor gap will be filled by some compressive material in order to not leave those gaps blank. The properties of the shear wall are given below in Table4.5.

SECTION (mm)	**MATERIALS**
2500x230	M30 Unconfined concrete and Fe415

TABLE 4.5 Shear wall properties.

CHAPTER 5

BARE FRAME POUNDING

5.1 GENERAL

In the present study two building of entirely different seismic behaviours are considered. The analysis part is divided into linear and nonlinear analysis. First of all the buildings are designed on the basis of linear seismic analysis and if found safe, then only it should be analysed for nonlinear analysis otherwise the members should be redesigned.

5.2 LINEAR SEISMIC ANALYSIS

This analysis considered two adjacent buildings having different dynamic responses for gravity loading and are designed through IS 456:2000 guidelines. After analysis it is found out that all the members are safe and the forces as base reactions are properly matched as per the IS 1893:2016(Part-I).

Initial scale considered for both eight and five storey building were 9806.65 but after multiplying it with the scale factor based on ratios of base shear based on the equivalent static method to the response spectrum method, the new scale was found out to be 11605.1 in X direction and 11463.54 in Y direction for eight storeys building while it was 11467.7 in X direction and 11239.6 on Y direction for the five-storey building.

5.3 NONLINEAR SEISMIC ANALYSIS

5.3.1 Nonlinear Static analysis (Pushover analysis)

The pushover graphs plotted shows the changes in the behavior of the buildings modelled for both the types of buildings. The changes in the base shear of the buildings or in other words the capability of the buildings to take base shear is noted and are discussed here. The pushover analysis is done for both mode proportional and mass proportional load. Out of these two, the one which is having greater roof displacement is considered and are shown here for both along short andlong direction. Here the short direction is along 'X' and long direction is along ‘Y’. Following Figures 5.1 and 5.2, shows the pushover curves carried out for buildings.

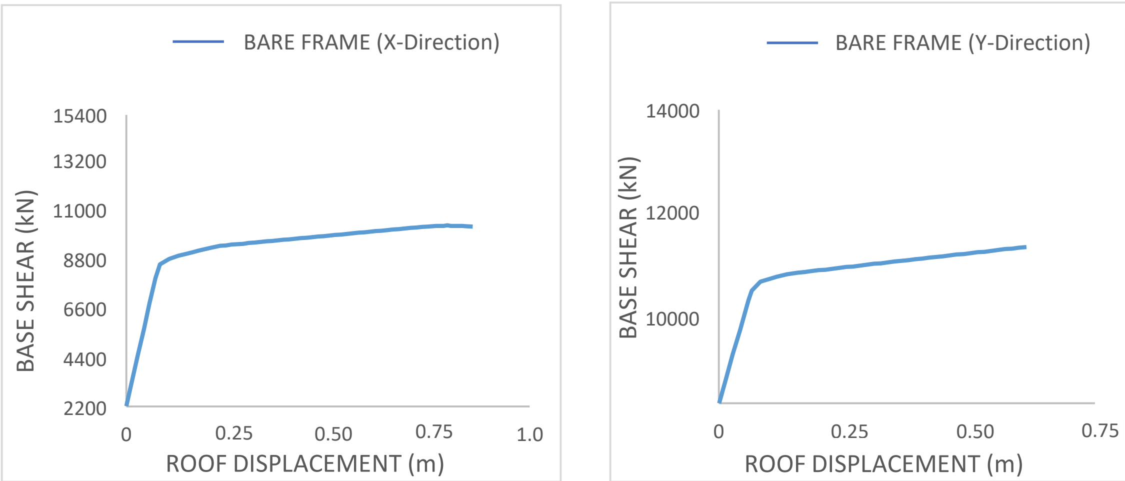


FIGURE 5.1 Pushover curve of 8 storey building along short and long direction respectively.

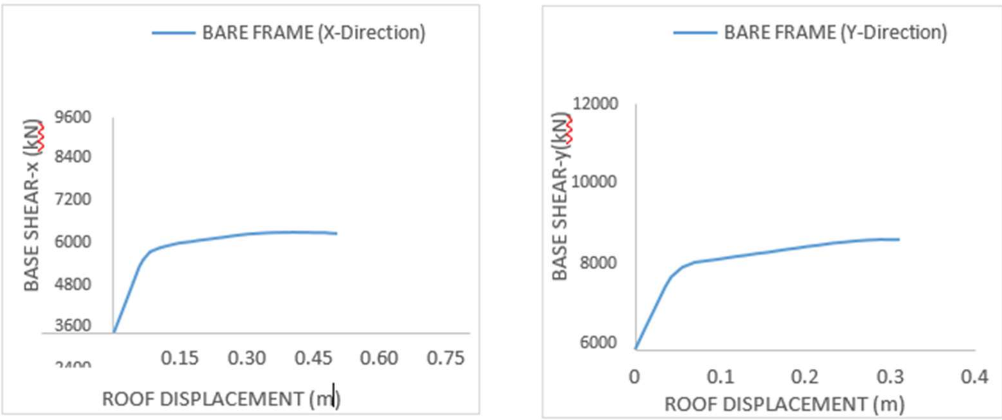


FIGURE 5.2 Pushover curve of 5 storey building along short and long direction respectively.

Long direction will provide more lateral stiffness than the short direction. Therefore, from the pushover analysis, it is clearly seen that the base shear of a building increases and roof displacement decreases in case of building along Y direction as compared to building with along X direction. The base shear variation for the two types of frames are shown below in Table 5.1 and Table 5.2.

STOREY	BARE FRAME BASE SHEAR (kN)
8 STOREY	~9000
5 STOREY	~4200

TABLE 5.1 Base shear variation along short direction.

STOREY	BARE FRAME BASE SHEAR (kN)
8 STOREY	~9400
5 STOREY	~5200

TABLE 5.2 Base shear variation along long direction.

Actually pushover analysis is also used for finding out building's performance level (IO, LSor CP) as per the target objectives but since the present study does not deal with the Unified Performance Based Design (UPBD), therefore this part isnot discussed here. The present study is based upon forced based method or codal method where the design provisions are as per IS 1893:2016 (part I).

5.3.2 Nonlinear dynamic analysis (Time history analysis)

After carrying out pushover analysis, next the buildings are subjected to nonlinear dynamicanalysis or nonlinear time history analysis (NLTHA) which has been carried out with 5 Spectrum Matched Ground Motions (SMGMs). The SMGMs used in this study are given in details in chapter 5 and are named as SMGM1, SMGM2, SMGM3, SMGM4, and SMGM5 The performance of the buildings has been evaluated for Maximum Considered Earthquake(MCE) level. From this analysis, various response parameters like maximum storey displacements, interstorey drift and time history last step hinges are evaluated. The main objective of this study i.e. to observe pounding, the maximum positive displacement of fifth floor level of eight storey building and the maximum negative displacement of roof level offive storey building along short direction (X) are evaluated and if the summation of these displacements exceed the initial separation gap provided (120 mm) then the pounding will occur. Since in this study, pounding will occur along short direction, therefore the main concern of this study is only along short direction. The pictorial representation of pounding phenomenon between adjacent buildings are shown below in Figure5.3.

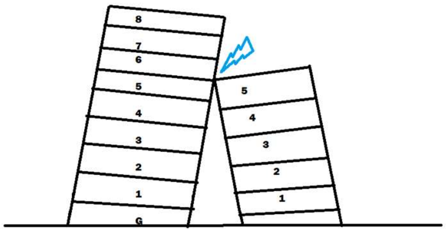
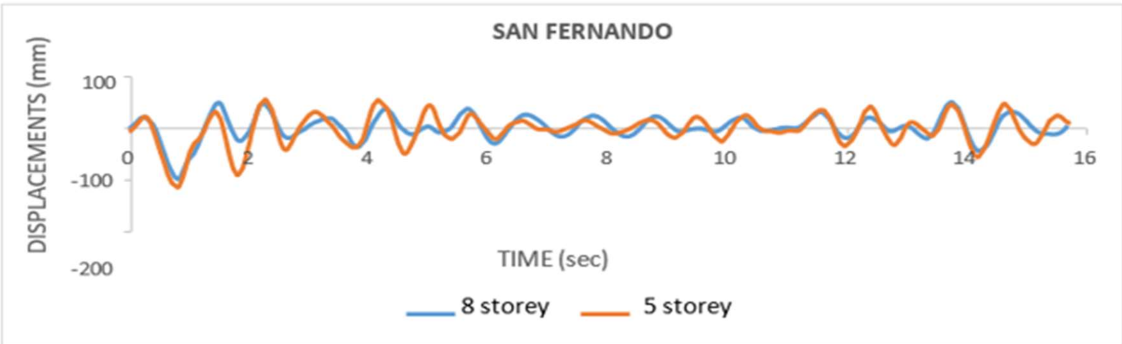
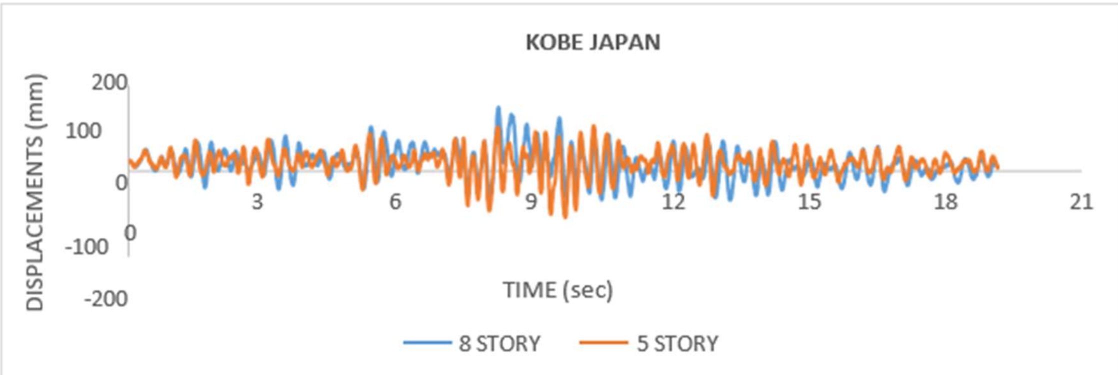


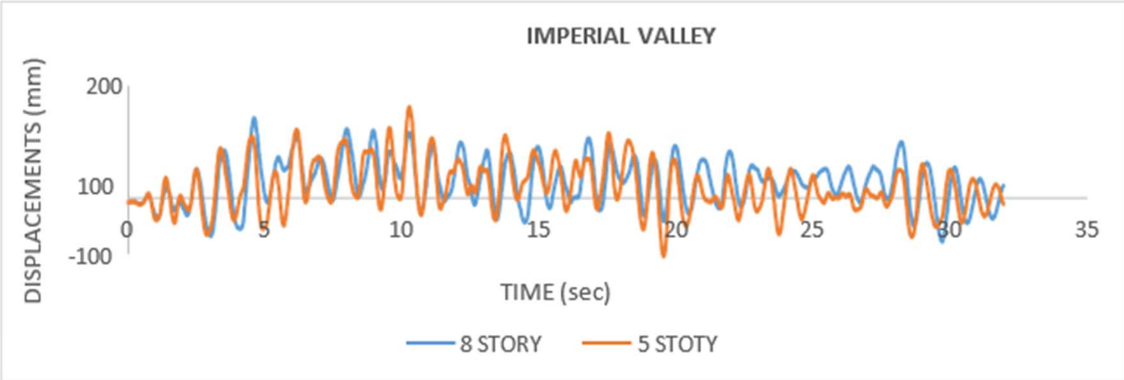
FIGURE 5.3 Occurance of pounding phenomenon.

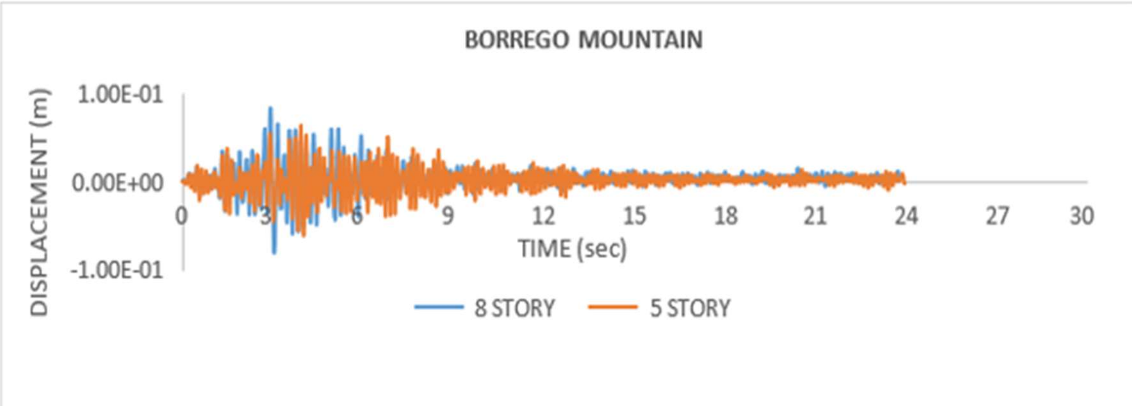
5.3.2.1 Variation of displacement with time

The maximum positive displacement of fifth floor level of eight storey building and the maximum negative displacement of roof level of five storey building from various SMGMs used are shown below in Figure 5.4.









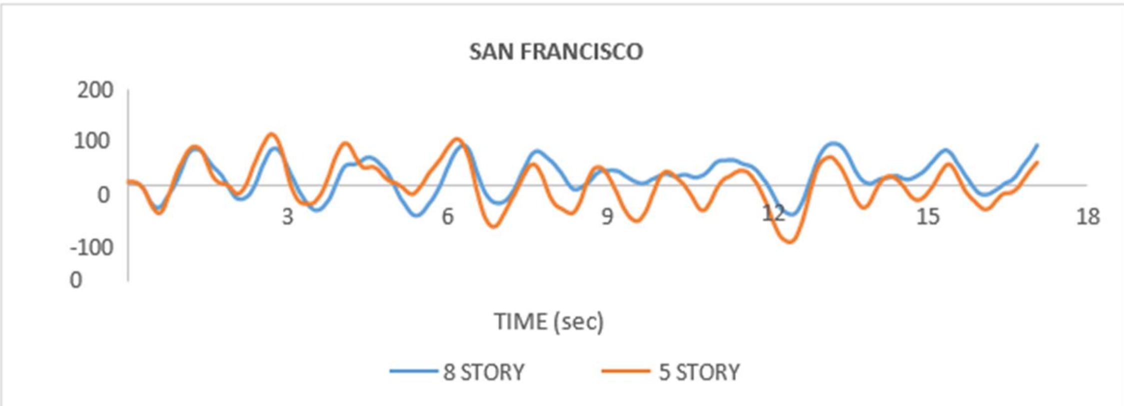


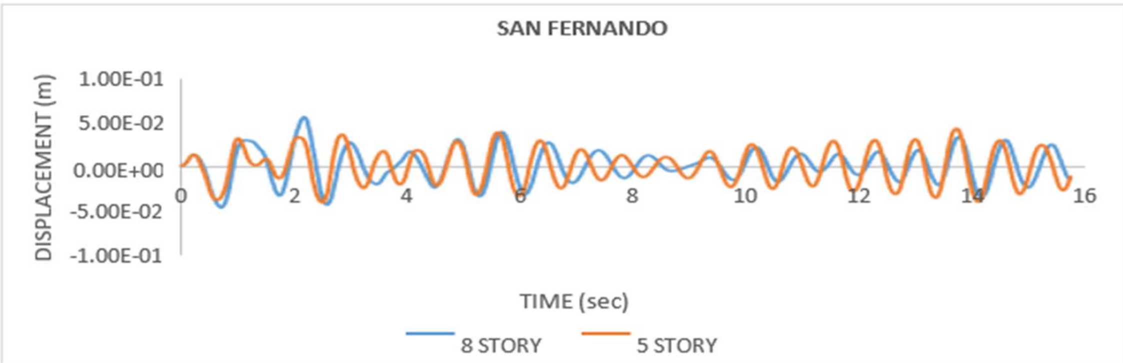
FIGURE 5.4 Displacement vs time graphs of SMGMs used (X Direction-Bare frame).

From the above graphs, the maximum positive and maximum negative displacements are noted and are given below in Table 5.3.

GROUND MOTION	8 STOREY (mm)	5 STOREY (mm)	SUMMATION (mm)
San Fernando	53.18	113.02	166.20
Kobe Japan	106.24	106.45	212.69
Imperial Valley	138.47	87.20	225.67
Borrego Mtn	80.91	112.98	193.89
San Francisco	81.39	124.52	205.91

TABLE 5.3 Maximum positive and negative displacement of 8 storey and 5 storey respectively (X- direction Bare frame).

Here, in all the SMGM records, the summation of the displacements are exceeding the initial separation gap provided, therefore the buildings are colliding with each other. Now again the displacements vs time graphs for both 8 and 5 storey buildings along Y direction are evaluated for bare frames and are shown below in Figure5.5.



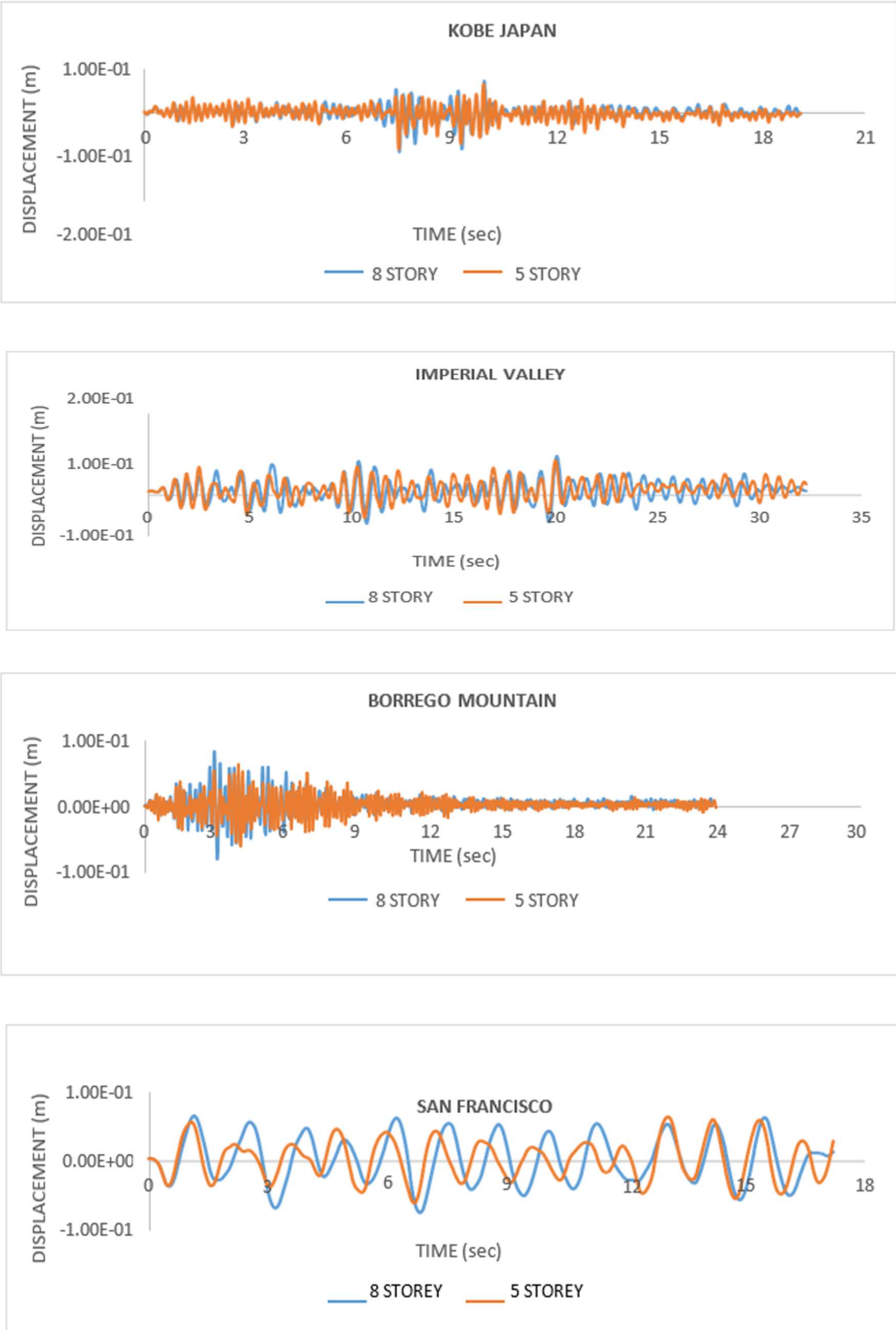


FIGURE 5.5 Displacement vs time graphs of SMGMs used (Y direction-bare frame).

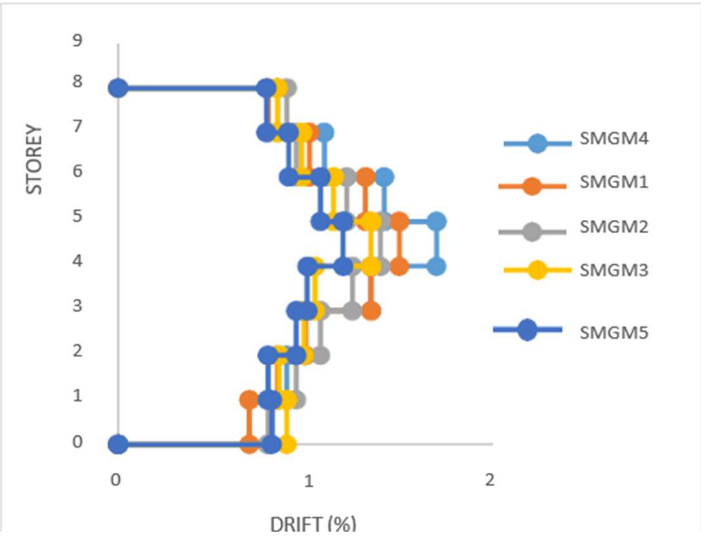
From the above graphs, the maximum positive and maximum negative displacements are noted and are given below in Table 5.4.

GROUND MOTION	8 STOREY (mm)	5 STOREY (mm)	SUMMATION (mm)
San Fernando	59.54	38.58	98.12
Kobe Japan	46.01	66.45	112.46
Imperial Valley	65.52	50.17	115.70
Borrego Mtn	57.4	42.6	100.1
San Francisco	41.46	43.40	84.86

TABLE 5.4 Maximum positive and negative displacement of 8 storey and 5 storey respectively (Y-Direction bare frame).

In this case pounding between buildings is prevented since the displacements are less and so is not the point of discussion here. Therefore, from now onwards all discussion would be done for X-direction pounding prevention. Figure 5.6 and Figure 5.7shows the IDR for both the type of moment resisting frames.

5.3.2.2 Interstorey Drift Ratio (IDR)



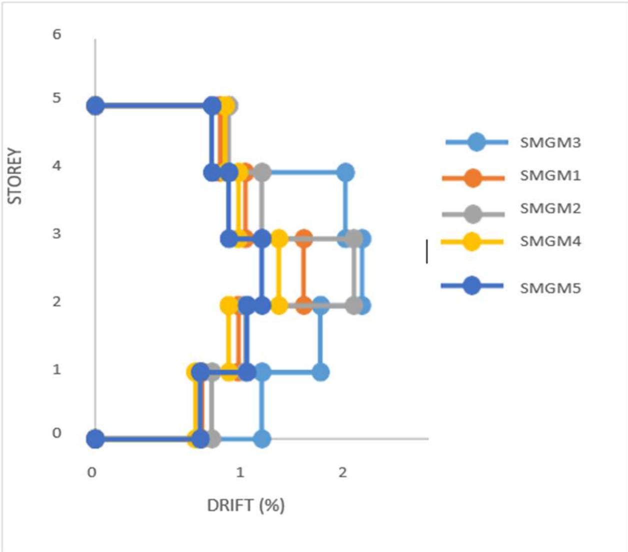


FIGURE 5.6 Interstorey Drift Ratio (IDR) of 8 storey and 5 storey respectively (X-direction bare frame).

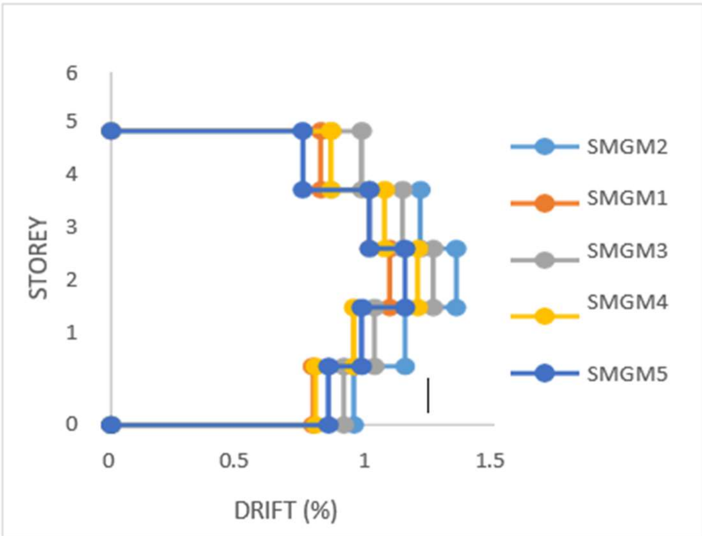


FIGURE 5.7 Interstorey Drift Ratio (IDR) of 8 storey and 5 storey respectively (Y-direction bare frame).

The interstorey drift ratio values varies according to different SMGMs used. Out of these, maximum IDR for 8 storey building is found as 1.78% for shorter direction and 1.50% for longer direction. Similarly the maximum IDR for 5 storey building is found as 1.6% for shorter direction and 1.42% for longer direction with bare frame. Since in performance based design, the target drift value for LS is taken in between 1.5-2%, therefore if the buildings are designed for LS performance level, then the separation gap provided will not be enough to prevent collision.

5.3.2.3 Storey displacements

In 5.3.2.1, only the maximum displacement for concerned floor level is found out i.e. 5^{th} floor level of both 8 storey and roof level of 5 storey building. Now the maximum displacement of roof level of both the buildings and the corresponding lower storey displacements are evaluated and are shown below in Figure 5.8 and Figure 5.9.

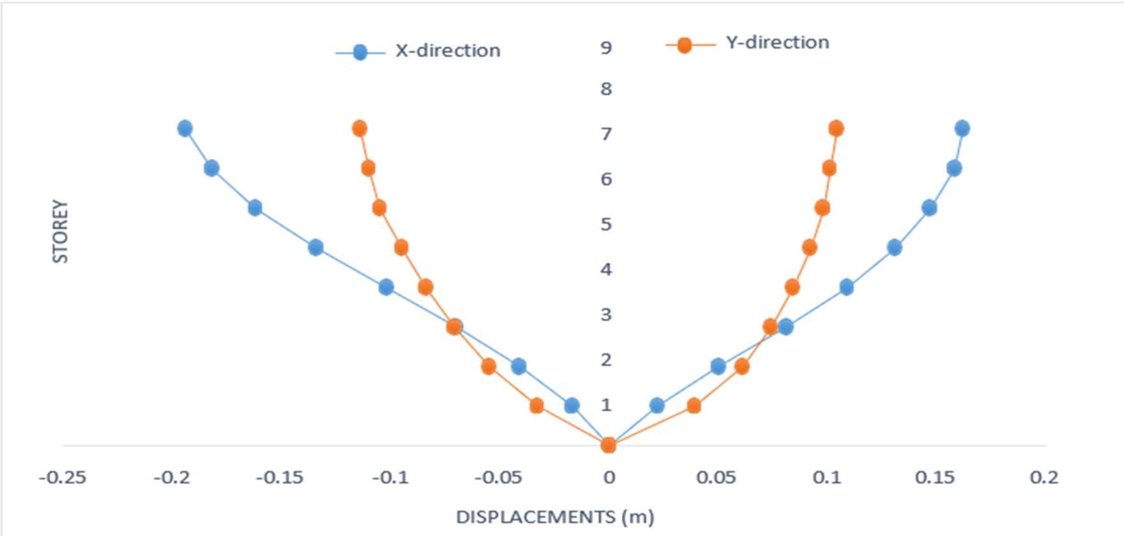


FIGURE 5.8 Maximum storey displacements (8 storey-Bare frame).

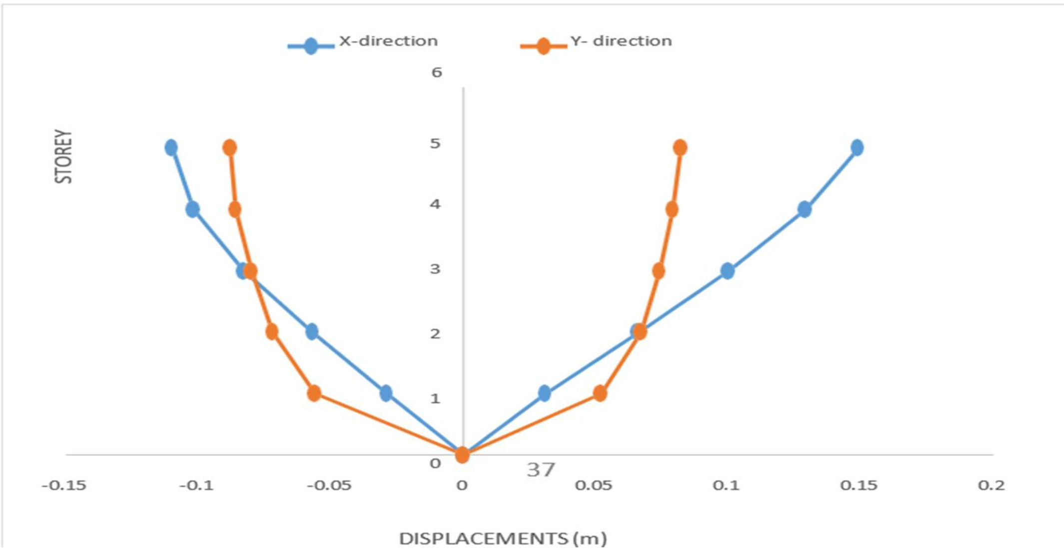


FIGURE 5.9 Maximum storey displacements (5 storey-Bare frame).

CHAPTER 6

POUNDING MITIGATION AND PERFORMANCE CHECK

6.1 GENERAL

Since it's known that severe damage to the structure can happen under strong ground motions during pounding, it may lead to discomfort to the people residing inside it , damage to the non-load bearing elements or it may collapse due to strong impact force of collision, therefore some cost effective preventive measures need to be implemented like bracings, shear wall, dampers etc. In this study BRB cross bracings, Steel cross bracings and RC shear wall has been used. The configuration, material and sectional properties for both bracings and shear wall are given in details in chapter 5. Also the positioning of bracings and shear wall are studied thoroughly by putting them at various locations in the buildings and observe the response of the buildings and out of these various positioning, the best location of bracing and shear wall has been discussed in this study. In the previous chapter it is noticed that the building's performance level attained was Life safety which was not suitable for preventing pounding and therefore the drift of the buildings should be reduced. IDR value for all the locations are not evaluated in this study but rather the positioning in which the displacement gets mostly reduced is considered for evaluating IDR and observe the drift in that particular type of positioning. Thus,the bare frame structure has been laterally strengthen using various cost-effective techniques as discussed in this chapter.

6.2 STRENGTHENING METHODS

In bare frame structure, the cross bracings and shear wall are installed throughout the heightof the structure and they are placed at four number of bays along short direction such that the symmetry of the structure gets maintained. Unsymmetrical location of bracings and shearwall are avoided in order to prevent torsion in the structure. A total of six type of locations of both bracings and shear wall has been analyzed and the structure's response parameters are observed. Here 8 storey has 4 bays and 5 storey has 3 bays.

Therefore building configuration is not same for 8 storey and 5 storey building along short direction and thus same location of bracings and shear wall for both the buildings is not possible as it will hamper symmetry. Hence both the buildings have their own bracing and shear wall locations and it has been given below,

6.2.1 BRB bracing

6.2.1.1 Location 1

Here the cross bracings are provided at two outer face of the building at coordinate Y=0m and Y=20m which is shown in Figure 6.1.

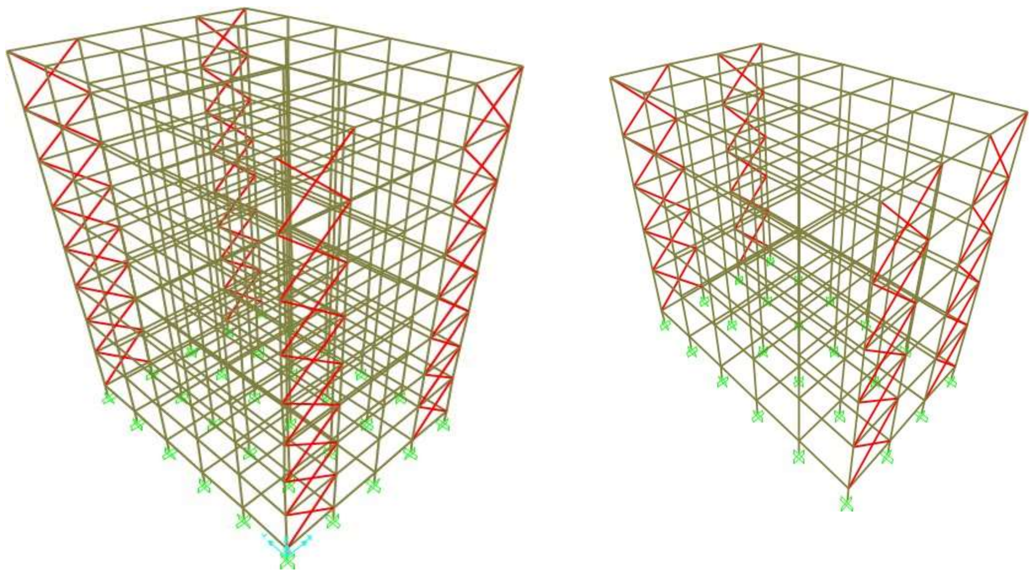


FIGURE 6.1 BRB bracing location no. 1 of 8 storey and 5 storey respectively

6.2.1.1.1 Pushover analysis

Here the base shear vs roof displacement curve for both the buildings are evaluated and are shown below in Figure 6.2.

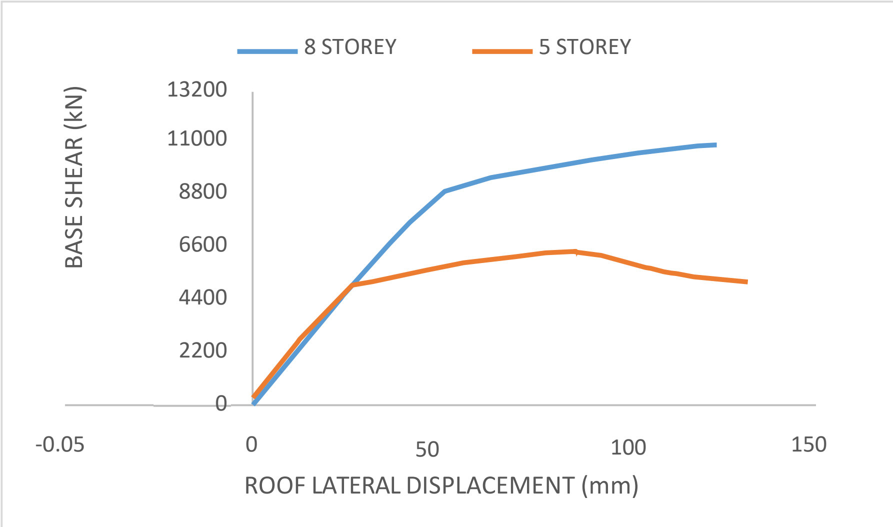
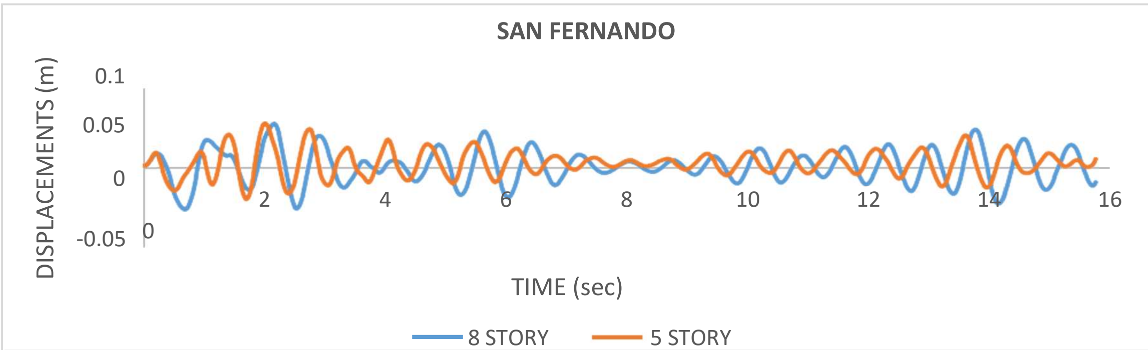


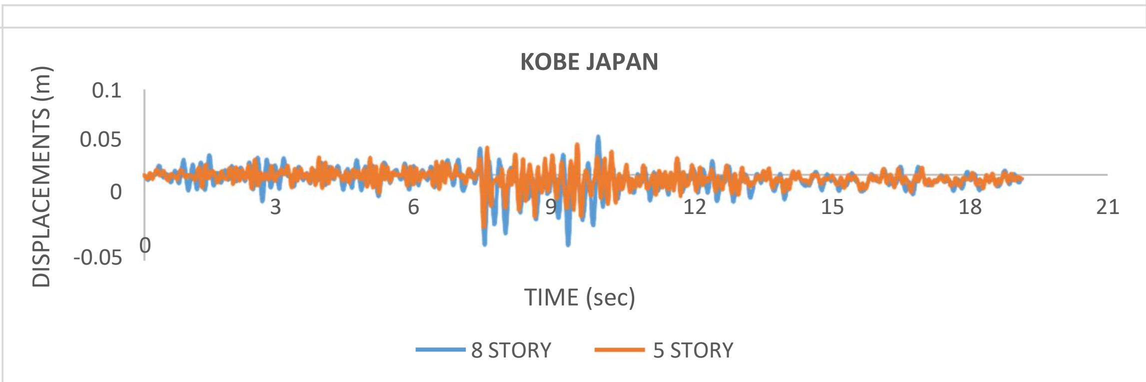
FIGURE 6.2 Pushover curve for location no. 1 (Bare frame).

6.2.1.1.2 Time history analysis

6.2.1.1.2.1 Variation of displacement with time

As discussed in chapter 6, to check if pounding between buildings is occurring, the maximum positive and negative displacement of concerned floor level is evaluated and are shown below in Figure 7.3.





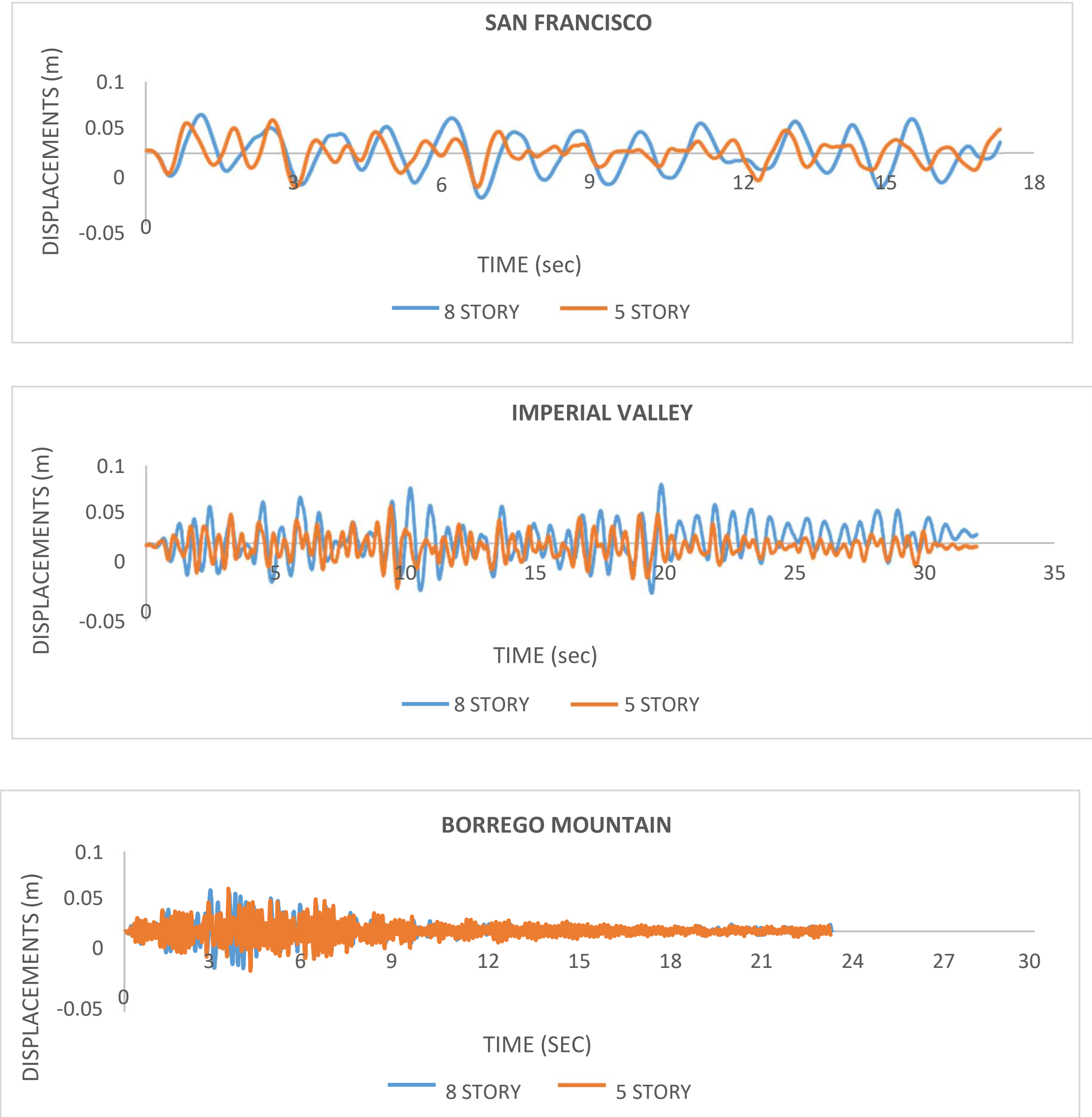


FIGURE 6.3 Displacement vs time graphs of SMGMs used (BRB bracing in bare frame).

From the above graphs, the maximum positive and maximum negative displacements arenoted and are given below in Table 6.1.

GROUND MOTION	8 STOREY (mm)	5 STOREY (mm)	SUMMATION (mm)
San Fernando	47.76	38.49	86.25
Kobe Japan	44.08	58.11	102.19
Imperial Valley	53.59	40.63	94.22
Borrego Mtn	47.25	44.94	92.19
San Francisco	49.13	49.67	98.80

TABLE 6.1: Maximum positive and negative displacement of 8 storey and 5 storey respectively (BRB bracing in bare frame).

In this case, pounding is prevented in all the SMGM records where the summation of positive and negative displacement of 8 storey building and 5 building are not exceeding the separation gap of 120 mm. Therefore this type of positioning is suitable for preventing pounding. Similarly different location of positioning of bracings are studied and are compared for best location in the next chapter. In that case also, the maximum positive and maximum negativedisplacement of concerned floor level of both the buildings are evaluated to observepounding and instead of showing the graphs, only the summation of displacements are shown in table 7.2 in order to avoid similar pattern of graphs. Also the pushover analysis has been carried out and the curves are shown in the next chapter where comparisons among the different positioning are being made. So, the different positionings are shown below in the following figures.

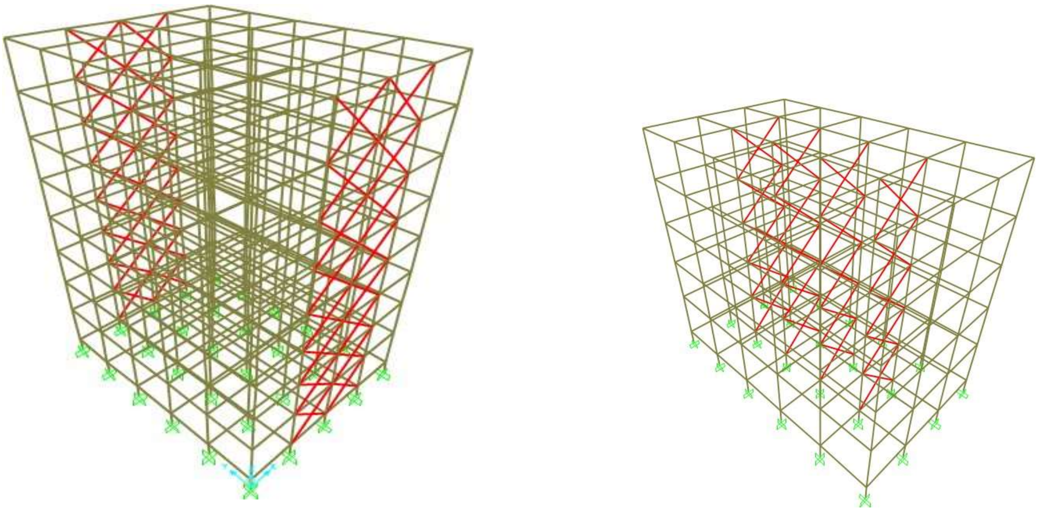


FIGURE 6.4 BRB bracing Location no. 2.

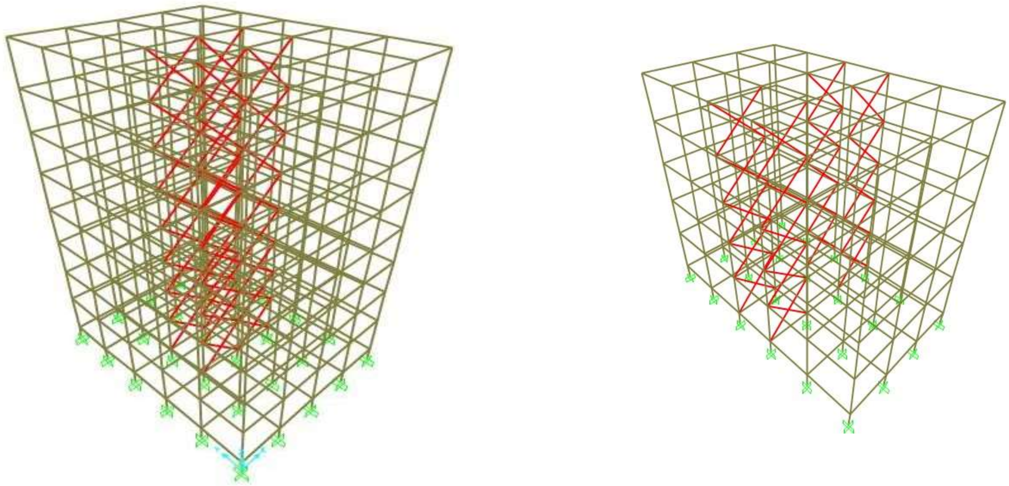


FIGURE 6.5 BRB bracing Location no. 3.

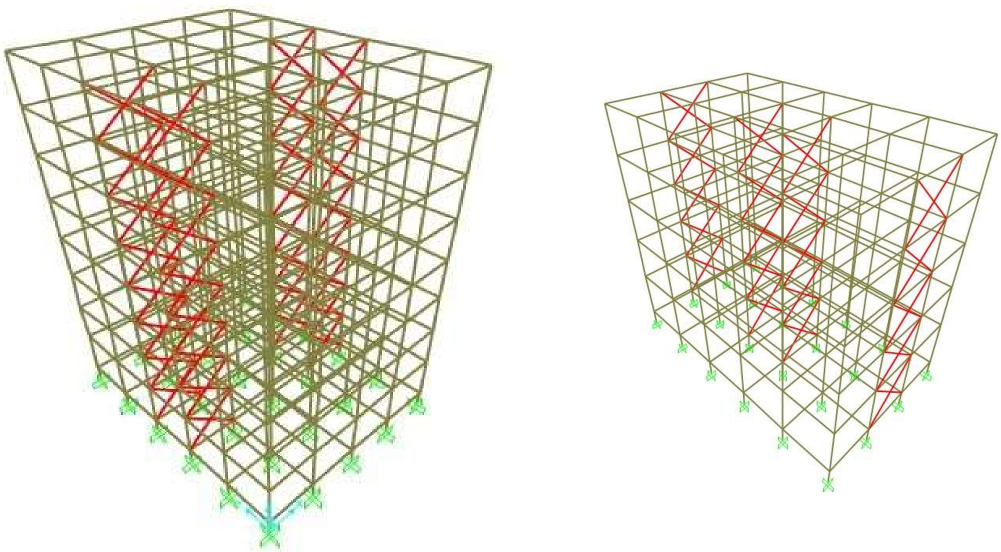


FIGURE 6.6 BRB bracing Location no. 4.

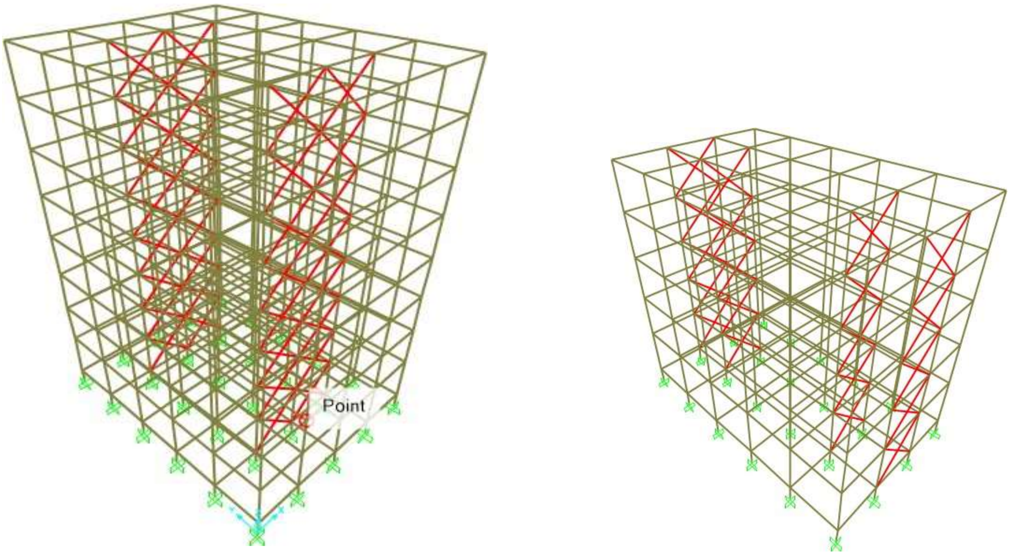


FIGURE 6.7 BRB bracing Location no.5.

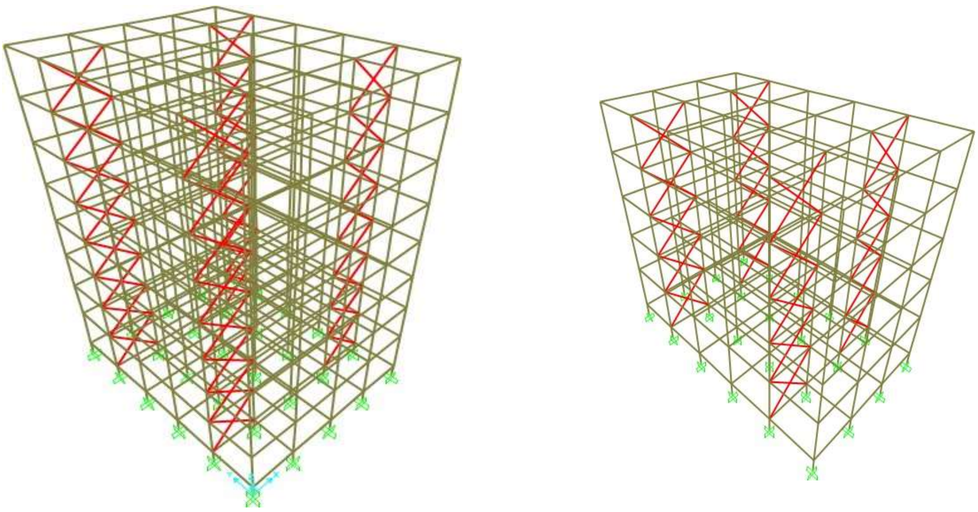


FIGURE 6.8 BRB bracing Location no. 6

LOCATION	SAN FERNANDO	KOBE JAPAN	IMPERIAL VALLEY	BORREGO MTN	SAN FRANCISCO
Location 2	71.19	85.92	96.37	85.55	82.87
Location 3	72.65	88.29	96.05	85.0	83.44
Location 4	75.26	97.90	96.26	91.68	82.35
Location 5	69.97	85.28	97.14	85.72	82.50
Location 6	76.26	94.54	99.59	91.07	86.15

TABLE 6.2: Summation of maximum positive and maximum negative displacement of 8 storey and 5storey building for 5 different SMGMs records (BRB bracing in bare frame).

After analyzing all the type of positioning of bracing, it is found that all the positioning are suitable for preventing pounding between adjacent buildings. But not all the types are giving equal response of the buildings. So out of these 6 positioning, the positioning in which the building's response is found less, is going to be the best location and has been discussed herein the next chapter. Now the shear wall has been used and is discussed below.

6.2.2 Shear wall

Just like bracings which is discussed in section 6.2.1, six type of positioning of shear wall has been studied and both the pushover and time history analysis are being carried out. The pushover curves are discussed in the next chapter and the maximum positive and maximumnegative displacement of 8 and 5 storey building are evaluated from the time history analysisbut similarly as discussed in previous section, only the summation of the displacements are shown in table 6.3 in order to avoid similar pattern of graphs. Following figures show different types of locations of shear wall.

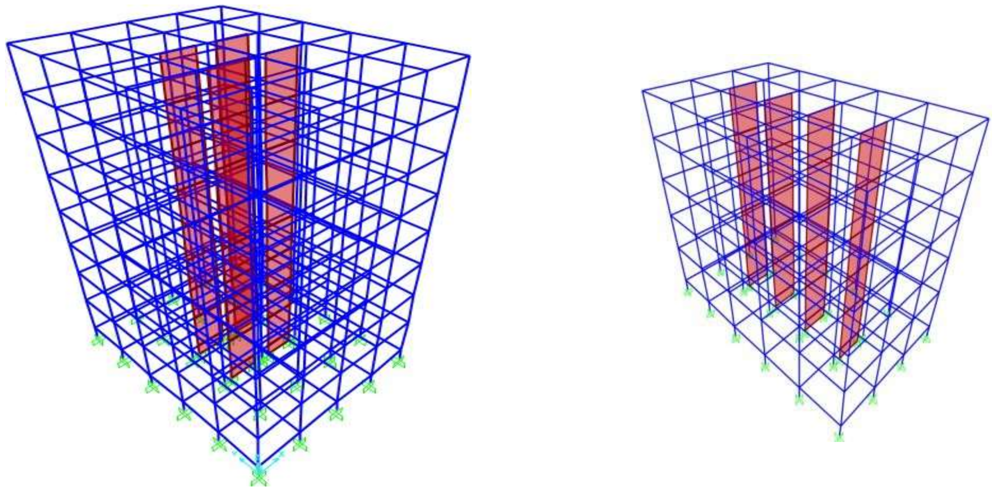


FIGURE 6.9 Shear wall location no. 1.

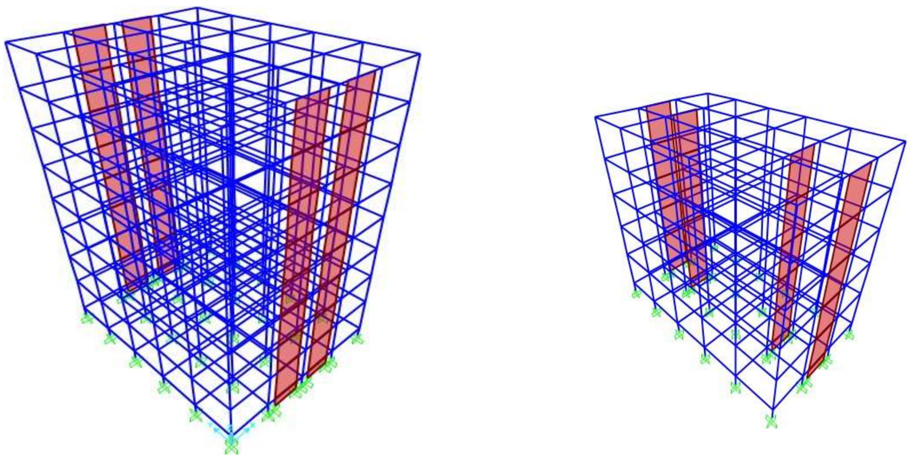


FIGURE 6.10 Shear wall location no. 2

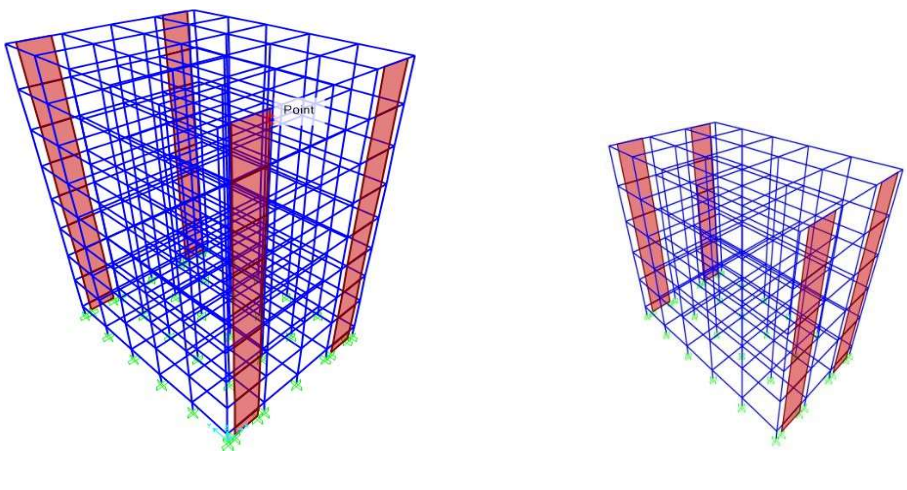


FIGURE 6.11 Shear wall location no. 3

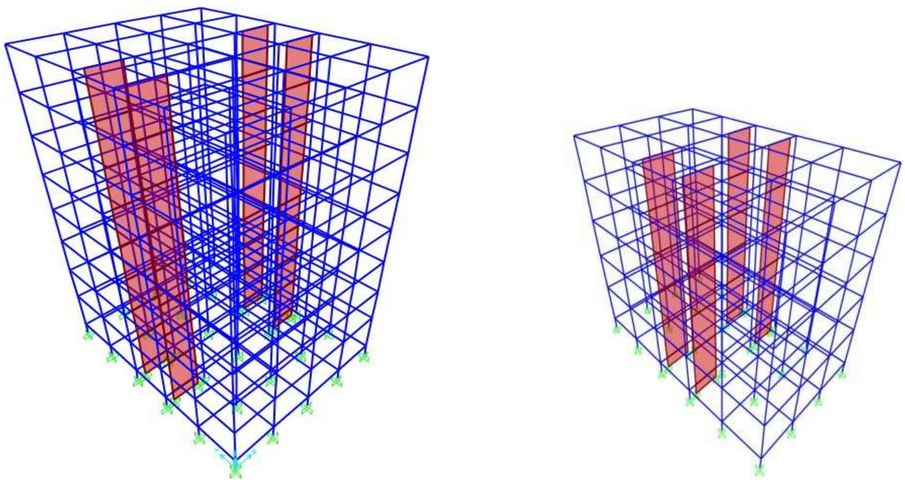


FIGURE 6.12 Shear wall location no. 4

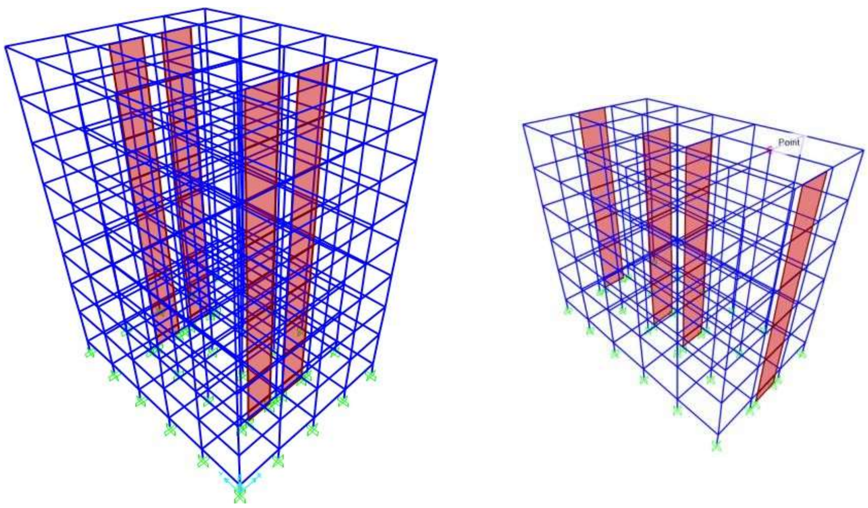


FIGURE 6.13 Shear wall location no. 5

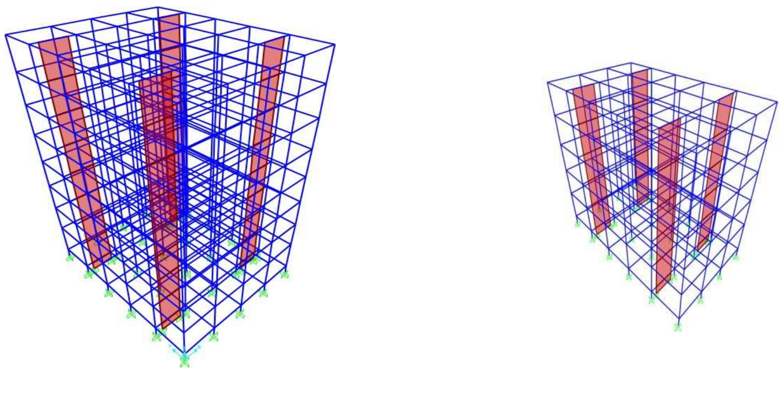


FIGURE 6.14 Shear wall location no. 6.

LOCATION	SAN FERNANDO	KOBE JAPAN	IMPERIAL VALLEY	BORREGO MTN	SAN FRANCISCO
Location 1	76.83	76.88	77.75	76.65	61.44
Location 2	76.17	76.72	83.27	75.53	61.55
Location 3	66.86	64.32	81.71	65.69	68.68
Location 4	78.88	89.48	98.08	83.65	83.24
Location 5	73.19	72.07	87.36	74.19	64.99
Location 6	75.51	87.31	96.52	79.99	81.47

TABLE 6.3: Summation of maximum positive and maximum negative displacement of 8 storey and 5storey building for 5 different SMGMs records (Shear wall in bare frame).

After analyzing all the type of positioning of shear wall, it is found that all the positioning are suitable for preventing pounding between adjacent buildings. Similarly not all the types are giving equal response of the buildings. So out of these 6 positioning, the positioning in which the building's response is found less, is going to be the best location and has been discussed here in the next chapter. With this note, locations of bracings and shear wall for bare frame structure has been completed and out of these the best location of placing them is discussed in the next chapter. Now the steel bracing in bare frame buildings has been discussed in the following section.

6.3 STEEL BRACING

In this type of frame structure, the steel cross bracings are installed throughout the height of the structure. It is located in such a way that the symmetryof the structure gets maintained. Unsymmetrical location of steel bracings is avoided in order to prevent torsion in the structure. Here 8 storey has 4 bays and 5 storey has3 bays and therefore same location of steel bracings for both the buildings is possible as done in earlier two cases. A total of 6 type of locations of steel bracings in 8 storey building and in 5 storey building has been analyzed and the structure'sresponse parameters are observed. Hence both the buildings have steel bracing locations and it has been given below.

6.3.1 Steel Bracing

Following figures show different positioning of steel bracings.

Bracings are provided at four corner joints of both 8 storey and 6 storey building

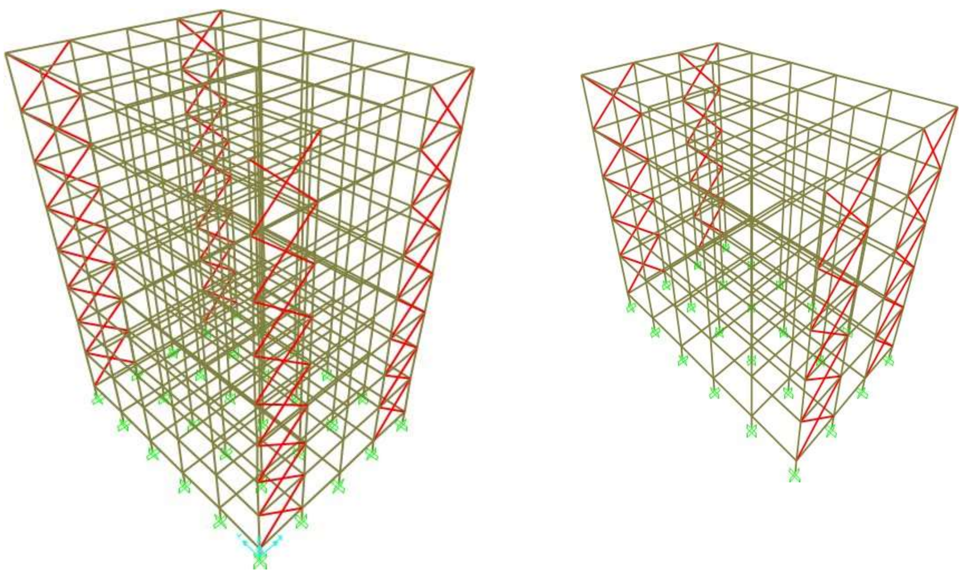


FIGURE 6.15 Steel bracing location no. 1

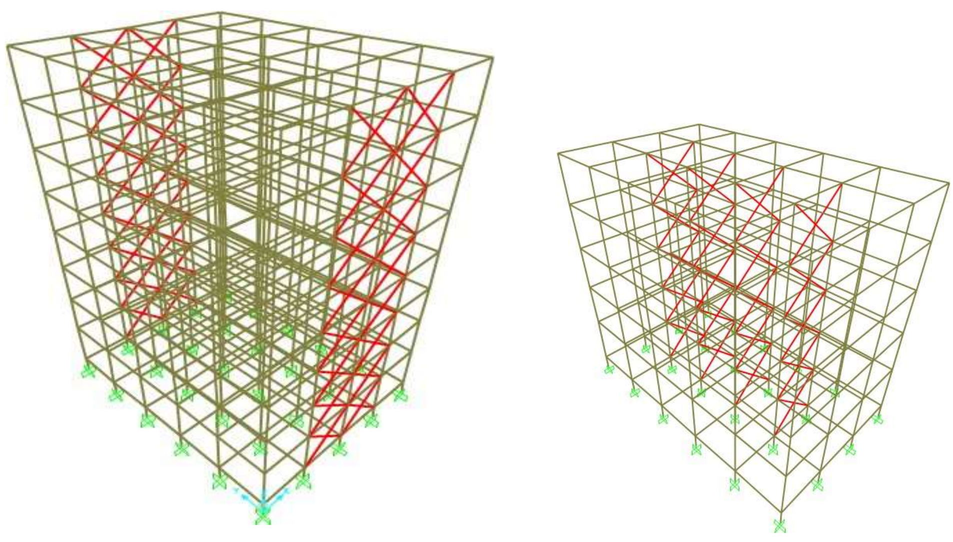


FIGURE 6.16 Steel bracing location no. 2.

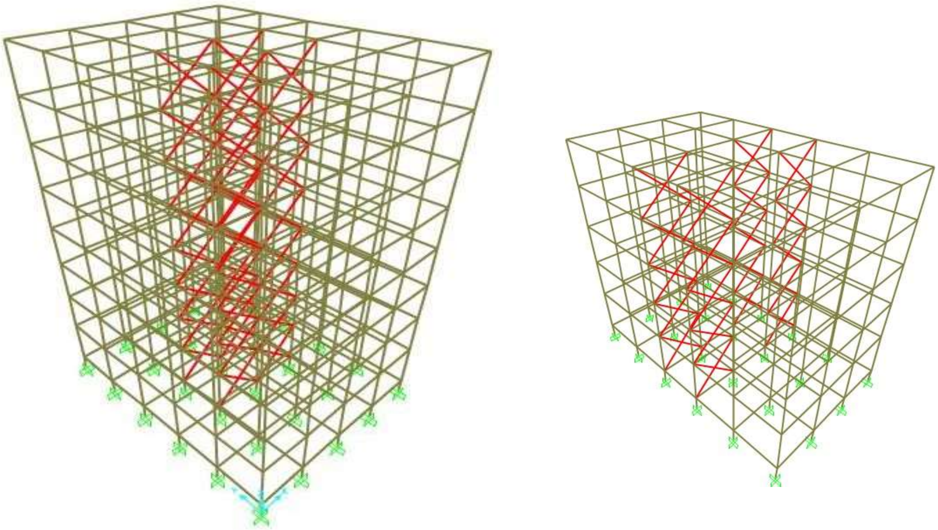


FIGURE 6.17 Steel bracing location no. 3

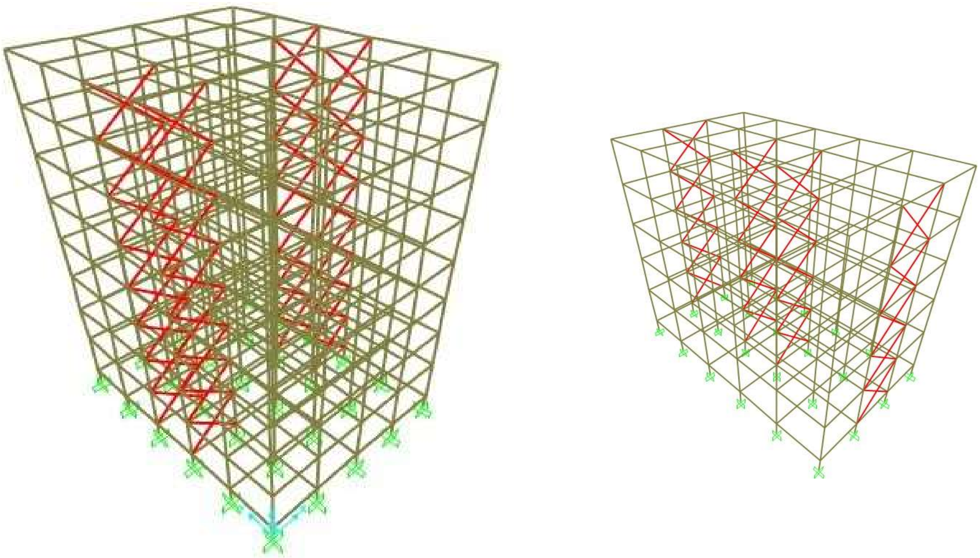


FIGURE 6.18 Steel bracing location no. 4

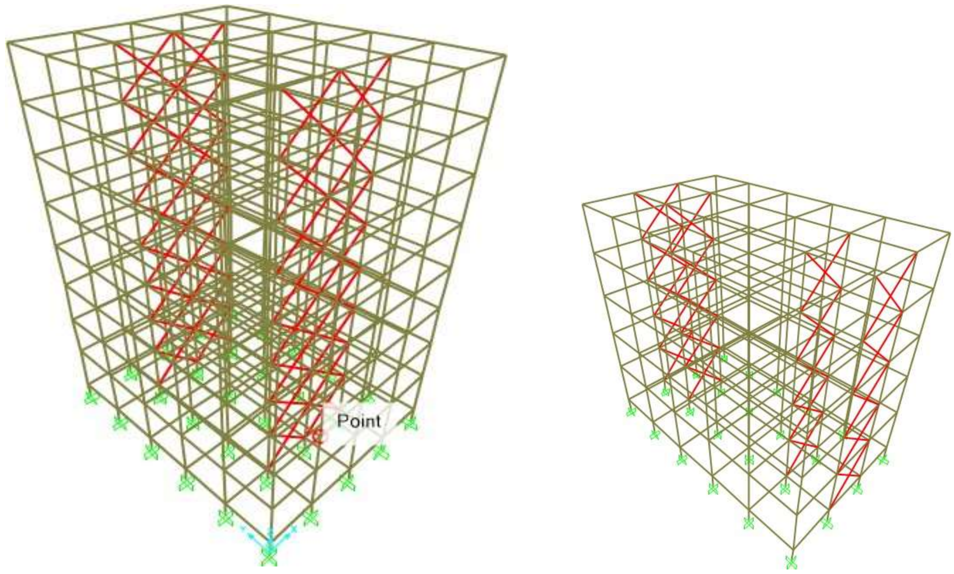


FIGURE 6.19 Steel bracing location no. 5

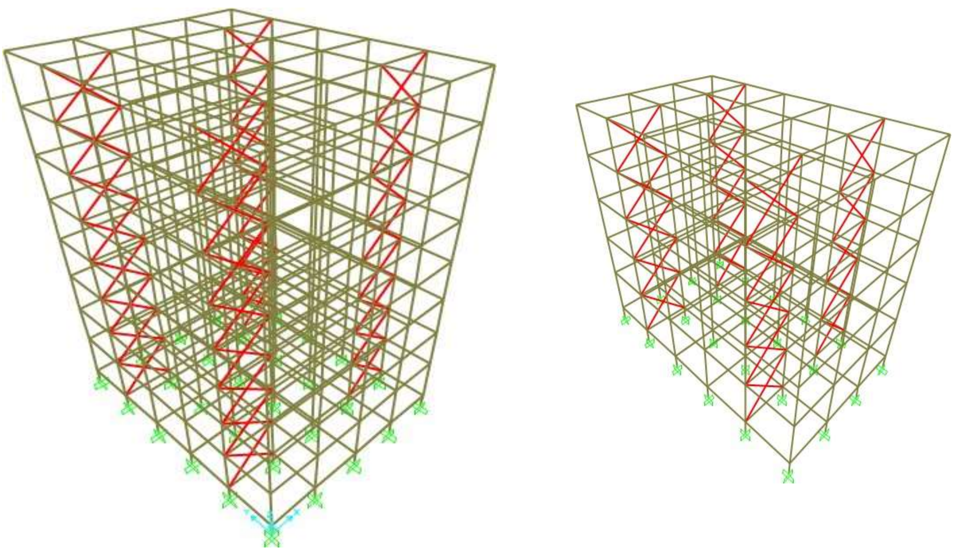


FIGURE 6.20 Steel bracing location no. 6

LOCATION	SAN FERNANDO	KOBE JAPAN	IMPERIAL VALLEY	BORREGO MTN	SAN FRANCISCO
Location 1	82.54	101.03	110.25	86.72	94.60
Location 2	82.88	100.54	109.96	86.73	94.32
Location 3	82.86	99.96	110.20	86.90	93.65
Location 4	80.05	98.22	108.96	85.16	92.41
Location 5	81.57	99.27	110.20	85.92	93.62
Location 6	81.01	99.05	109.32	85.80	93.59

TABLE 6.4: Summation of maximum positive and maximum negative displacement of 8 storey and 5storey building for 5 different SMGMs records (Steel Bracings)

After analyzing all the type of positioning of bracing, it is found that all the positioning are suitable for preventing pounding between adjacent buildings. But not all the types are givingequal response of the buildings. So out of these 6 positioning, the positioning in which the building's response is found less, is going to be the best location and has been discussed herein the next chapter.

With this note, locations of BRB bracings, shear wall and Steel bracings for frame structure has been completed and out of these the best location of placing them is discussed in the next chapter

CHAPTER 7

RESULTS AND OPTIMIZATION

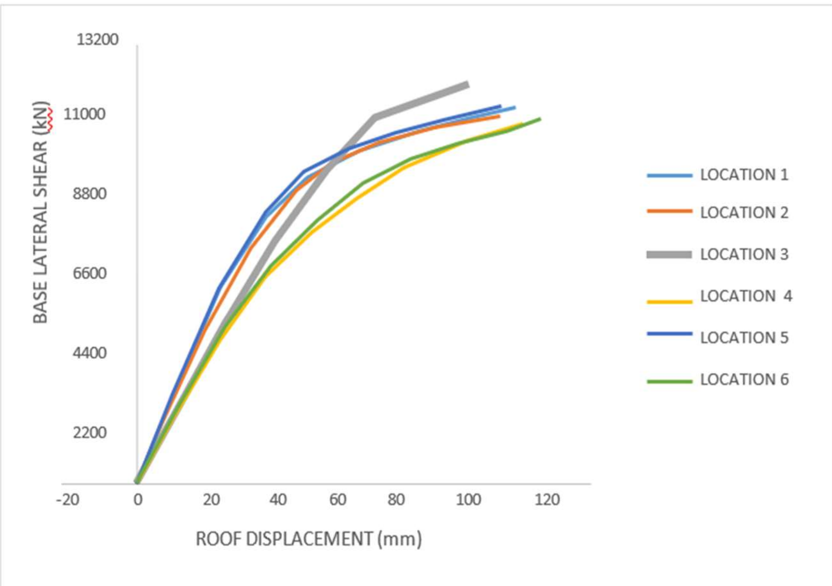
7.1 GENERAL

In the study presented, a total of six type of locations of BRB bracings, steel bracings and shear wall has been studied for moment resisting frames and it has been found that all the locations proved successful in preventing pounding. Now the best location of both type of bracings and shear wall for both the buildings are studied by evaluating the building's displacements and observe the type of positioning in which minimum displacement is recorded. Also, the base shear carrying capacity of the buildings for all the type of positioning are evaluated by nonlinear static analysis which is pushover analysis and are shown below.

7.2 FRAME WITH BRB BRACINGS

7.2.1 Pushover analysis (BRB bracing in bare frame)

Figure 7.1 shows pushover curves for different locations of both the buildings.



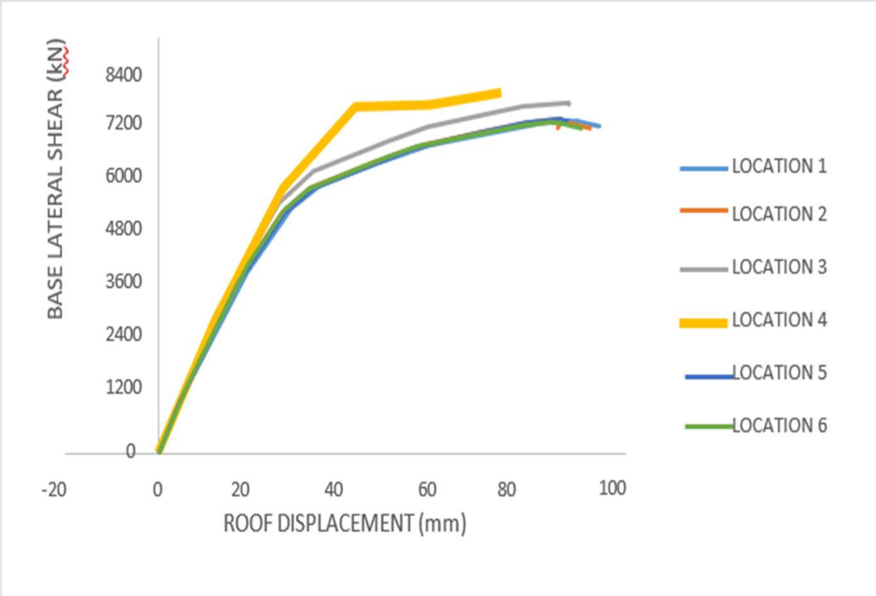


FIGURE 7.1 Pushover curve for different positioning type of 8 storey and 5 storeys (BRB bracing in bare frame)

From the pushover curves, the curve which is showing minimum roof displacement is taken as the more stiffen building. Here in this case, location number 3 of 8 storey building and location number 4 of 5 storey building proved to be the best location of placing bracing.

7.2.2 Storey displacements (BRB bracing in bare frame)

Since we are concerned with the maximum positive displacement of 8 storey building and maximum negative displacement of 5 storey building, therefore the maximum positive roofdisplacement of 8 storey building and the corresponding lower storey displacements are evaluated. Similarly, the maximum negative roof displacement of 5 storey building and their corresponding lower storey displacements are also evaluated and they are compared for minimum displacement. Figure 7.2 and Figure 7.3 show storey displacements.

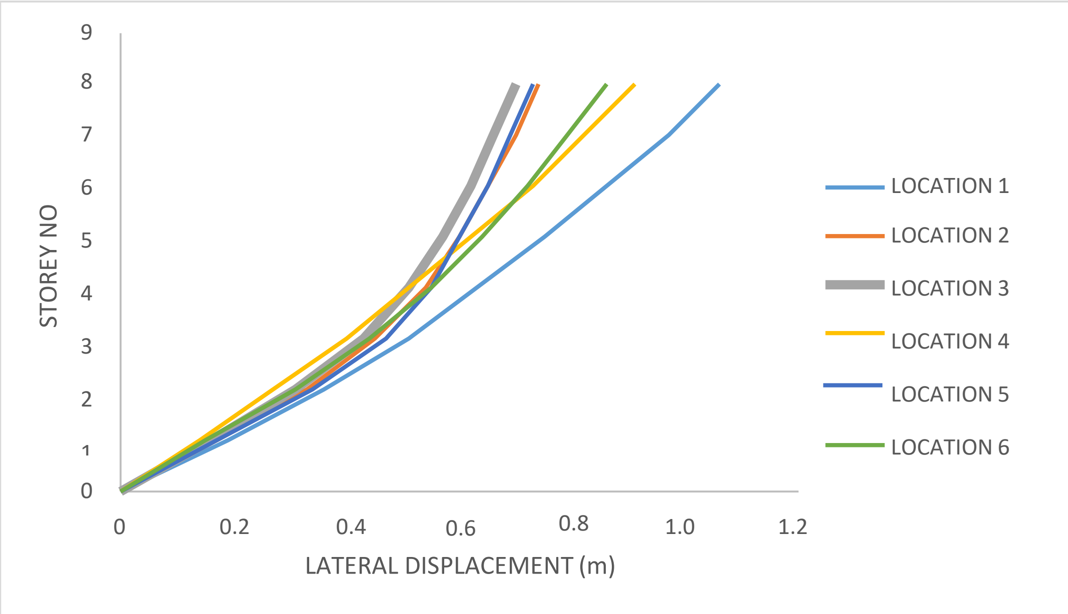


FIGURE 7.2 Maximum positive displacement of 8 storey building (BRB bracing in bare frame).

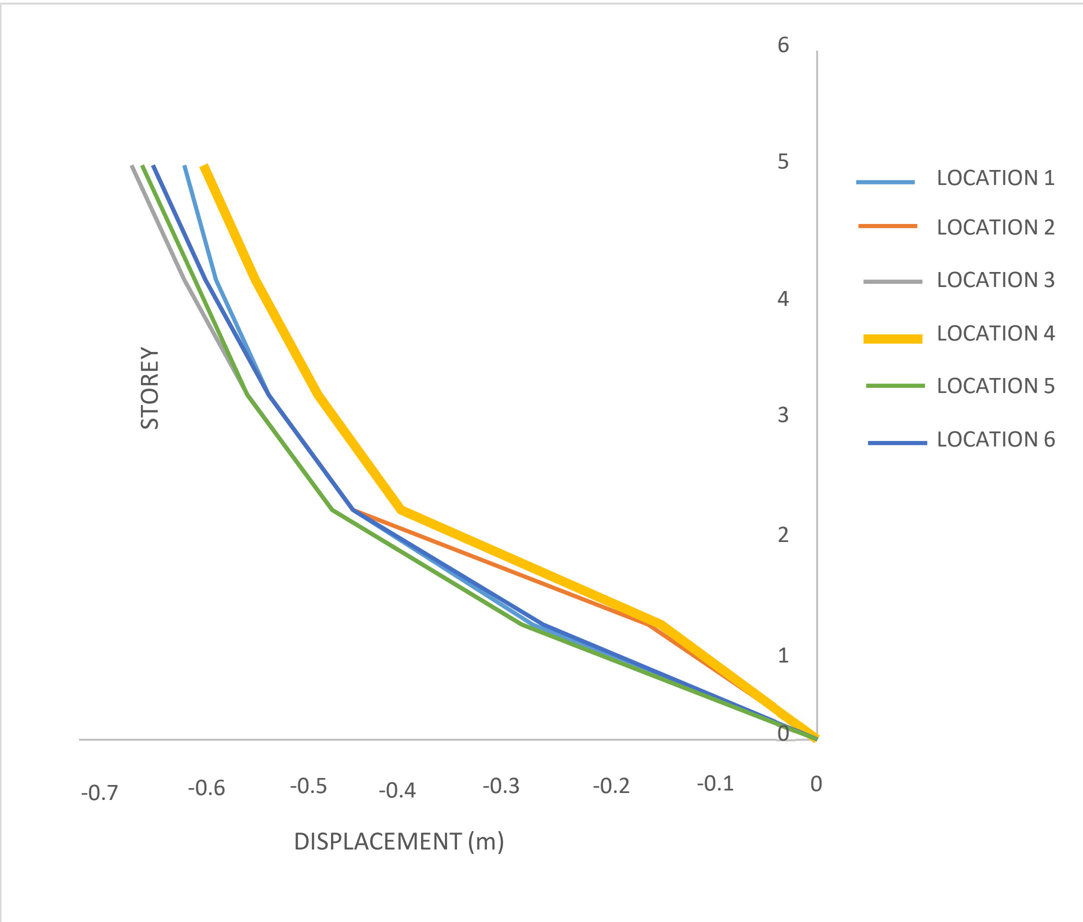


FIGURE 7.3 Maximum negative displacement of 5 storey building (BRB bracing in bare frame).

Here also it is clearly seen that for 8 storey building,location number 3 proved quite impressive as after analyzing, this locaton has got the minimum displacement. Similarly for5 storey building, location number 4 has got the minimum displacement. Now if the IDR value is checked for both the building with that particular best location, then we can know the performance level attained by the buildings. Therefore, the IDR value for different SMGMs records of location 3 of 8 storey building and location 4 of 5 storey building are shown below in Figure 7.5.

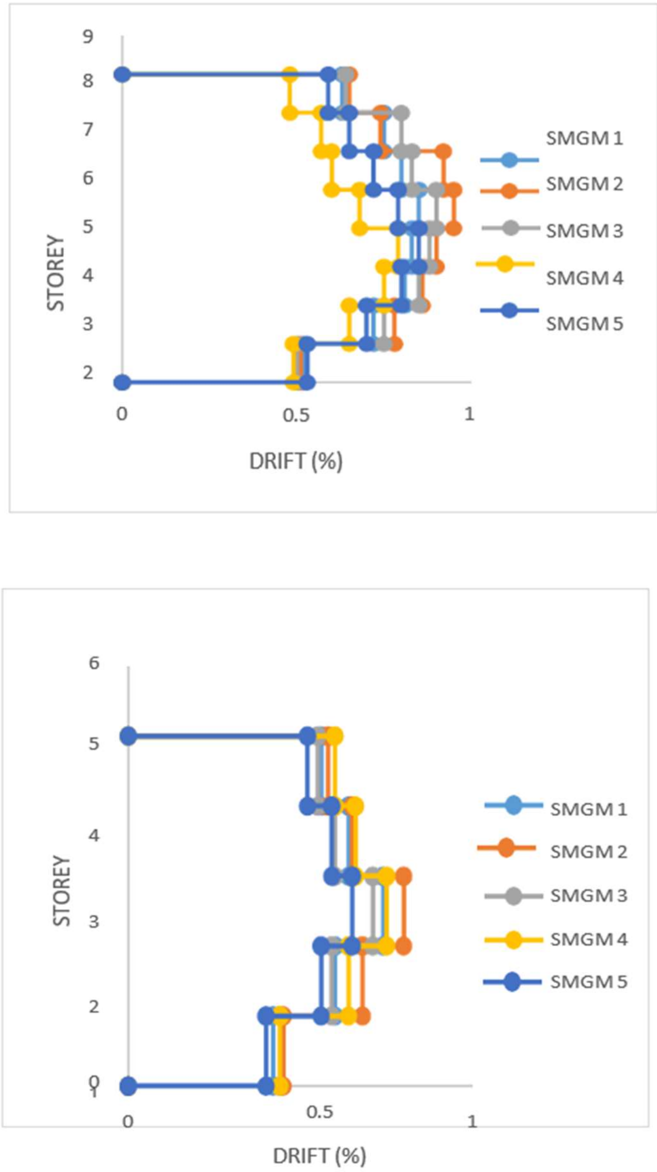


FIGURE 7.4 Interstorey Drift Ratio (IDR) of 8 storey (Location 3) and 5 storey (Location 4) respectively(BRB bracing in bare frame).

The interstorey drift ratio values varies according to different SMGMs used. Out of these, maximum IDR for 8 storey building is found as 0.92% and maximum IDR for 5 storey building is found as 0.77%. Since the drift attained is below 1.5%, therefore the IO performance level is achieved in order to stiffen the structure for preventing pounding.

Also the time history last second hinge formation of that particular best location is checkedin Figure 8.5 in order to ensure that no nonlinear hinges should form on columns. The SMGMs records in which worst kind of hinges are forming is taken into consideration and are shown here. Hence it is found that only IO level hinges are forming on beams and columns are free from hinges. Bracings are also forming hinges but this is only built for strengthening the frame structure which it is maintaining well in this study.

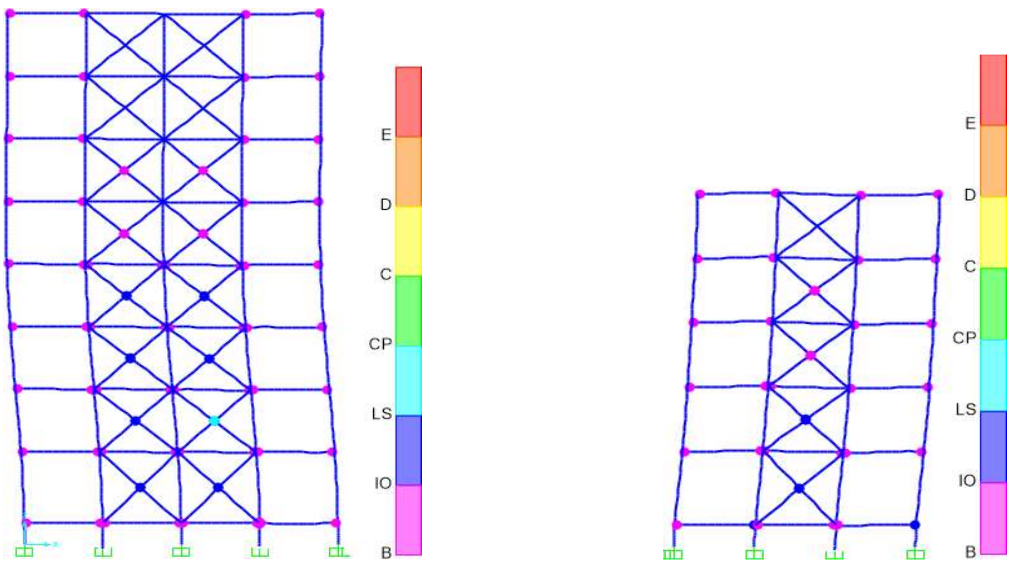
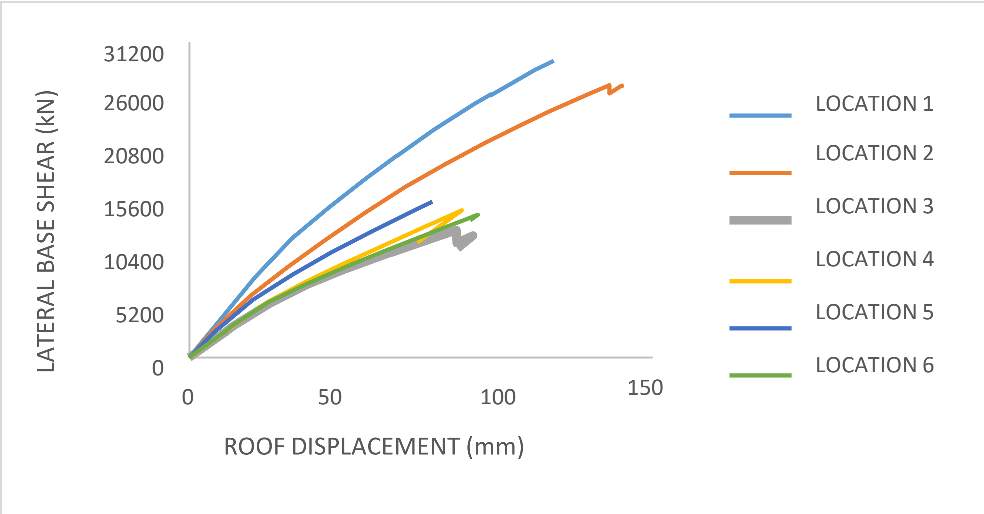


FIGURE 7.5 Last time second hinge formation of 8 storey and 5 storey respectively (BRB bracing in bareframe).

7.3 FRAME WITH SHEAR WALL

7.3.1 Push over analysis (Shear wall in bare frame)



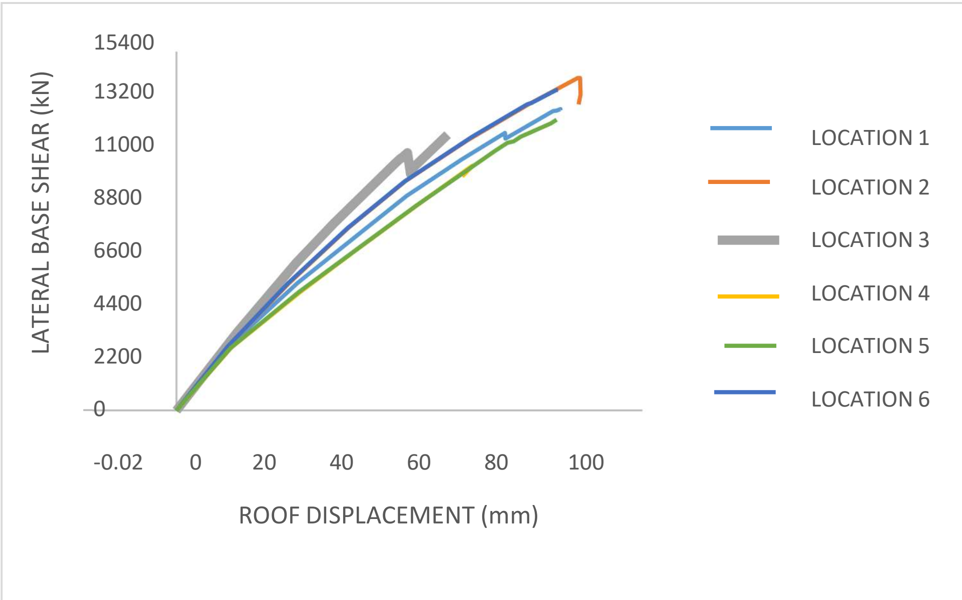


FIGURE 7.6 Pushover curve for different positioning type of 8 storey and 5 storey respectively (Shear wall in bare frame).

From the pushover curves shown in Figure 7.6, the curve which is showing minimum roof displacement is location number 3 of 8 storey building and location number 3 of 5 storey building which is proved to be the best location of placing shear wall.

7.3.2 Storey displacements (Shear wall in bare frame)

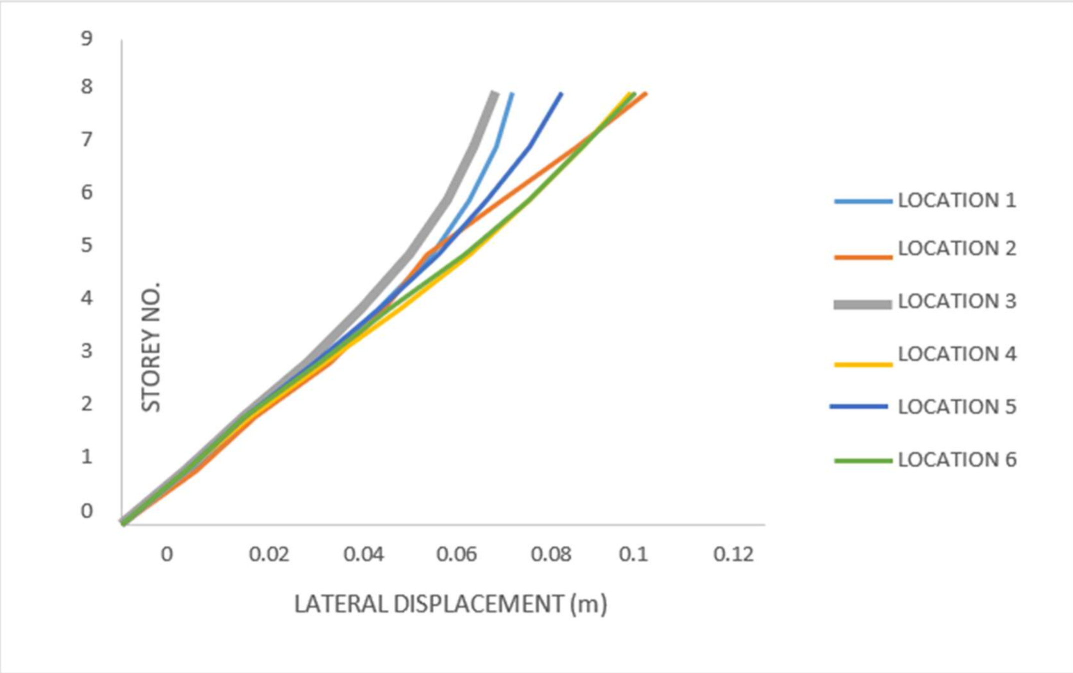


FIGURE 7.7 Positive displacement of 8 storey building (Shear wall in bare frame).

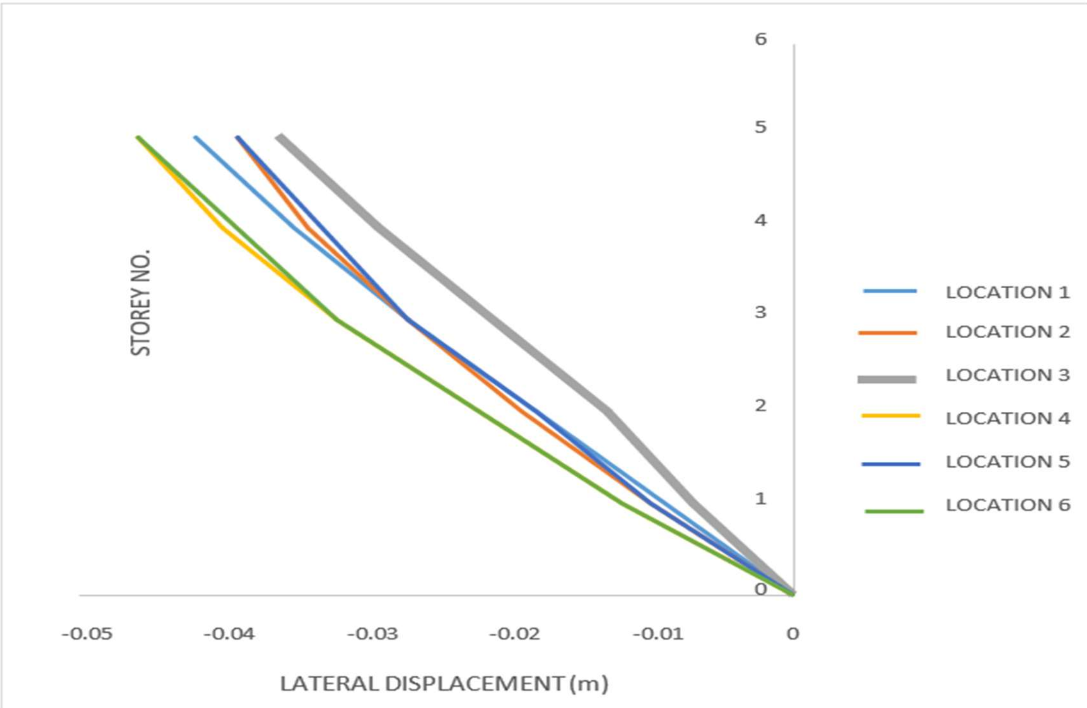


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FIGURE 7.8 Negative displacement of 5 storey building (Shear wall in bare frame).

Figure 7.7 and Figure 7.8 are clearly showing that for 8 storey building, location number 3 proved quite impressive as after analysing this location has got the minimum displacement.Similarly for 5 storey building, location number 3 has got the minimum displacement. The IDR value for different SMGMs records of location 3 of 8 storey building and location 3 of 5 storey building are shown below in Figure 7.9.

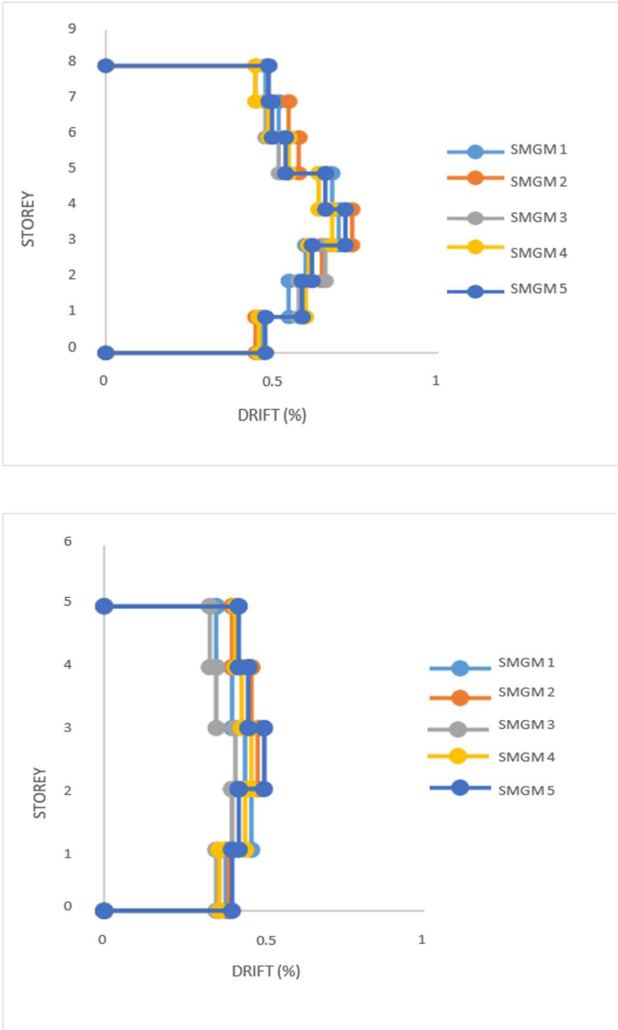


FIGURE 7.9 Inter storey Drift Ratio (IDR) of 8 storey (Location 3) and 5 storey (Location 3) respectively(Shear wall in bare frame).

The interstorey drift ratio values varies according to different SMGMs used. Out of these, maximum IDR for 8 storey building is found as 0.71% and maximum IDR for 5 storey building is found as 0.46%. Since the drift attained is below 1.5%, therefore the IO performance level is achieved. Also the time history last second hinge formation of that particular best location is checked in Figure 7.10 in order to ensure that no nonlinear hinges should form on columns. The SMGMs records in which worst kind of hinges are forming istaken into consideration and are shown here. Here it is found that only IO level hinges are forming on beams and columns are free from hinges. Similarly shear wall are also forming hinges but this is only built for strengthening the frame structure which it is maintaining well in this study.

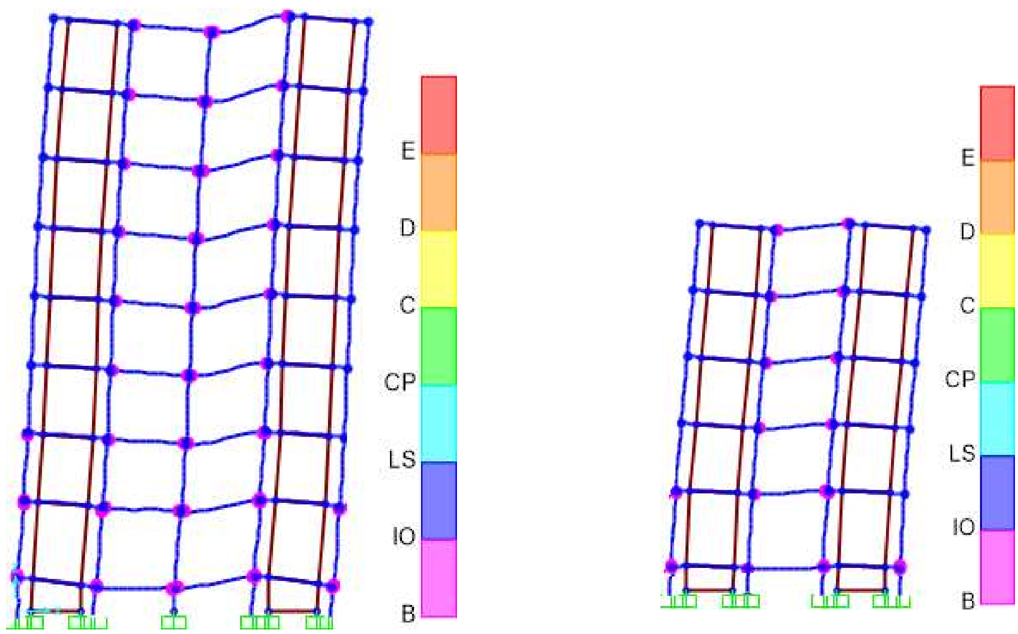


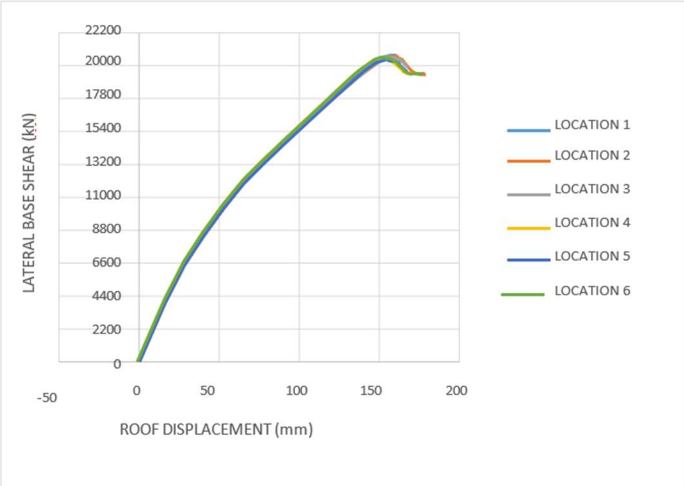
FIGURE 7.10 Last time second hinge formation of 8 storey and 5 storey respectively (Shear wall in bareframe).

Both the BRB bracings and shear wall are being studied for best location in bare frame structure and now similarly for the other type of bracings in bare frame structure i.e, steel bracing are studied for best location in the following section.

7.4 FRAME WITH STEEL BRACING

7.4.1 Pushover analysis (Steel bracing in bare frame)

From the pushover curves shown in Figure 7.11, it is found that all the positioning are showing almost equal responses to the buildings and it is difficult to choose the best one which is showing minimum roof displacement.



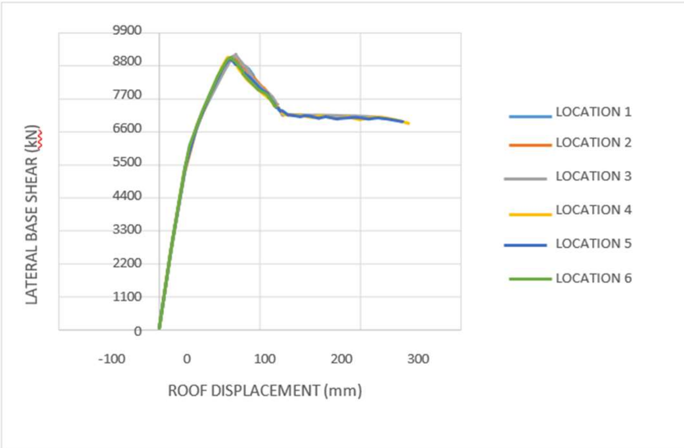


FIGURE 7.11 Pushover curve for different positioning type of 8 storey and 5 storey respectively (Steel bracing in bare frame).

From the pushover curves shown in Figure 8.11, it is found that all the positioning are showing almost equal responses to the buildings and it is difficult to choose the best one which is showing minimum roof displacement.

7.4.2 Storey displacements (Steel bracing in bare frame)

Figure 7.12 and Figure 7.13 show storey displacements which indicates that all the locations are showing almost equal amount of displacements but location number 4 of both 8 storey and 5 storey building shows some good results as compared to other locations. So, it is chosen as the best location of placing steel bracings.

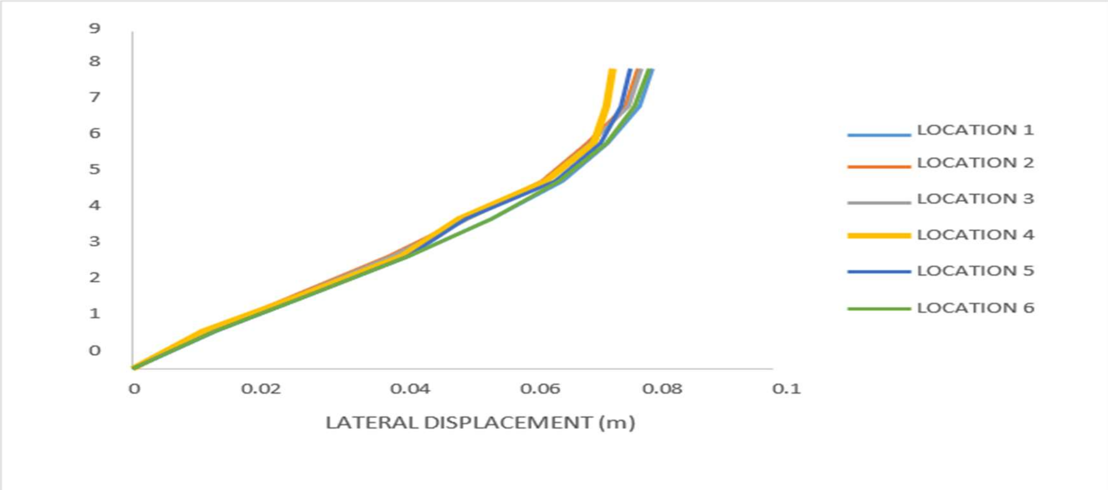


FIGURE 7.12 Positive displacement of 8 storey building (Steel bracings in bare frame)

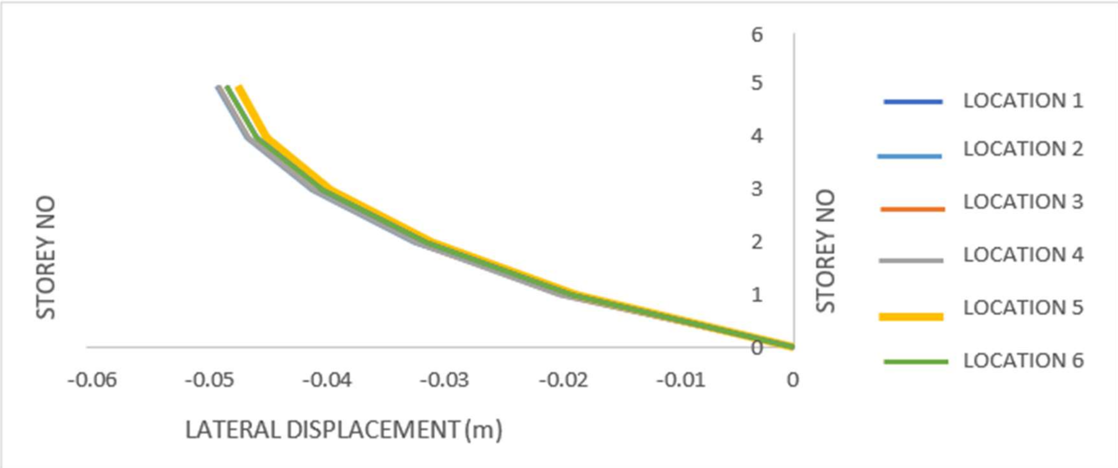
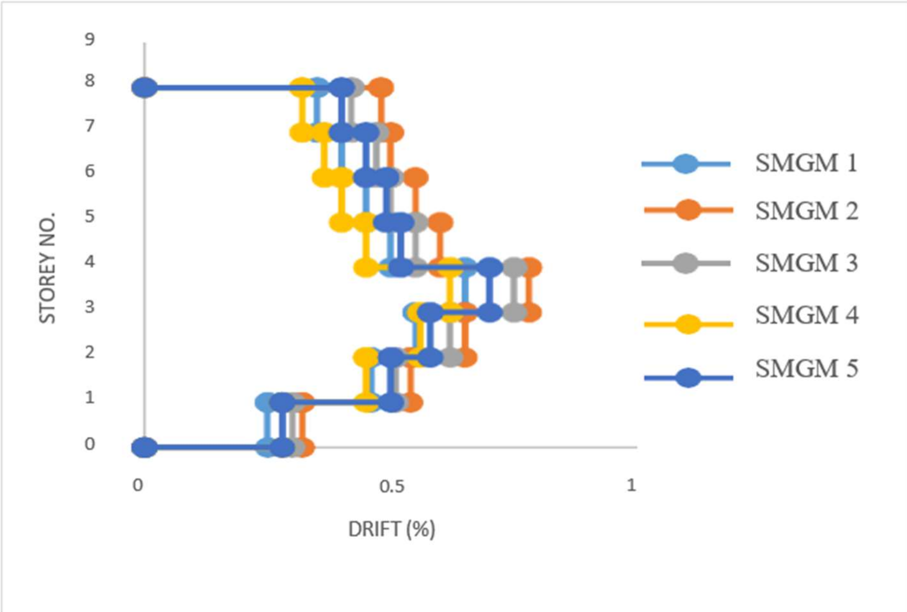


FIGURE 7.13: Negative displacement of 5 storey building

(Steel Bracing in bare frame).

The IDR value for different SMGMs records of location 4 of 8 storey building and location4 of 5 storey building are shown below in Figure 7.14.



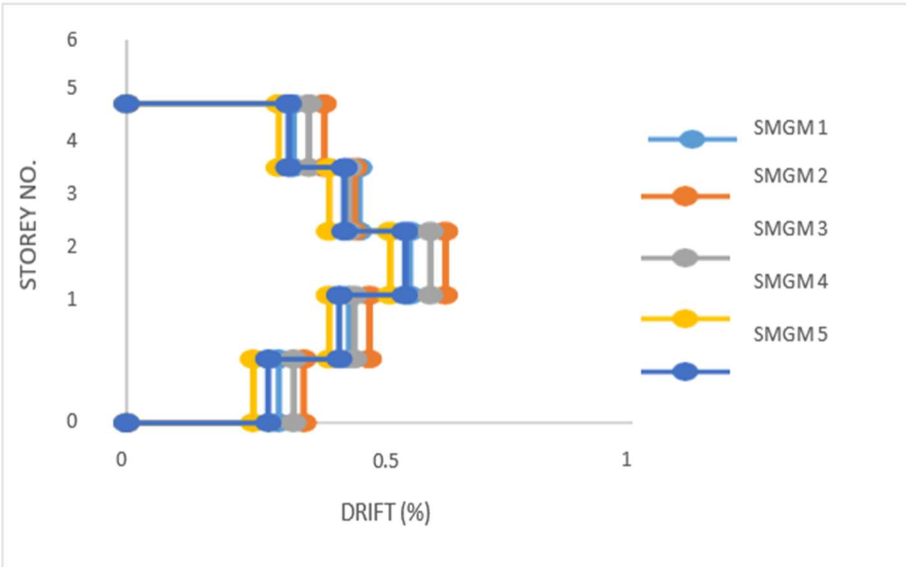


FIGURE 7.14 Interstorey Drift Ratio (IDR) of 8 storey (Location 4) and 5 storey (Location 4) respectively(Steel Bracing in bare frames).

Out of these, maximum IDR for 8 storey building is found as 0.76% and maximum IDR for5 storey building is found as 0.60%. Since the drift attained is below 1.5%, therefore the IO performance level is achieved. Also the time history last second hinge formation of that particular best location is checked in Figure 7.15 in order to ensure that no nonlinear hinges should form on columns. The SMGMs records in which worst kind of hinges are forming istaken into consideration and are shown here. Here it is found that only IO level hinges are forming on beams and columns are free from hinges.

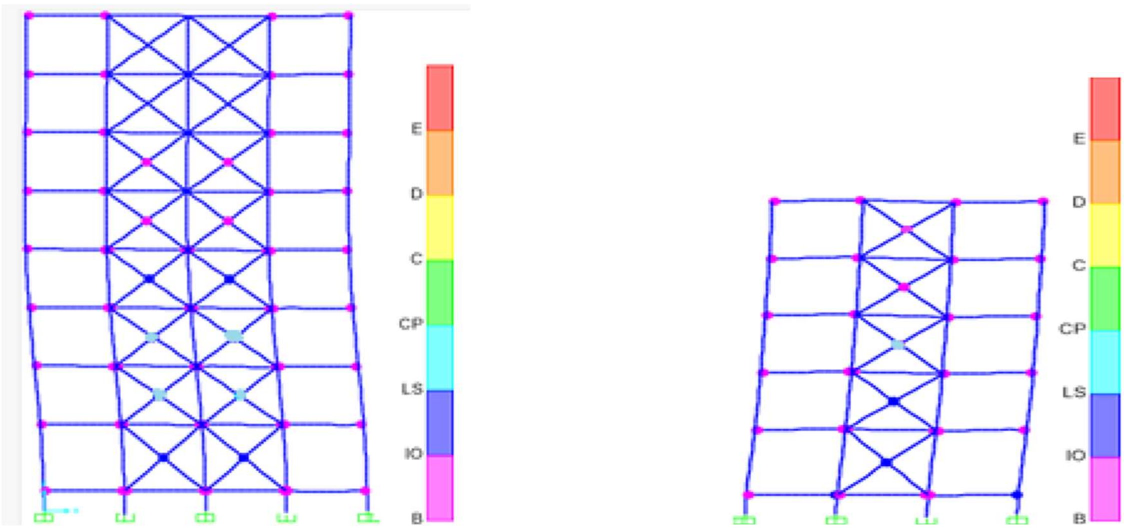


FIGURE 7.15 Last time second hinge formation of 8 storey and 5 storey respectively (Steel Bracing in bare frame).

Hence the study of different positioning of BRB bracing, Steel bracing and shear wall for both 8 storey and 5 storey building has been completed and now it is clearly visible that which type of positioning of bracing and shear wall should be used for better response of the buildings. Now the present study is concluded in the next chapter.

7.5 COMPARISON OF THE THREE APPROACHES

Form the results obtained in the previous chapters, we can summarize as discussed below:

Percentage variance in the sum of displacements at the concerned storey level, i.e., the 5th floor of an 8-story structure and the roof level of a 5-story building, employing BRB bracing, RC shear walls, and steel bracing against no bracing is shown in table 7.1 below.

Methods of strengthening				
	No bracing	BRB bracing	RC shear wall	Steel bracing
Summation of displacements(mm)	225.67	99.59	83.27	94.60
Percentage variation (%)	-	55.86	63.11	58.08

TABLE 7.1: Percentage variation in displacement value using various approaches.

CHAPTER 8

CONCLUSIONS AND FUTURE WORKS

8.1 CONCLUSIONS

After thorough research on the topic of pounding reduction methods using BRB bracing, RC shear wall and steel bracing, we can conclude that:

1) The percentage reduction in storey displacement with BRB bracing in building is 55.86 percent, indicating that BRB bracing is effective in reducing pounding.
2) The percentage reduction in storey displacement utilising RC shear wall in building is 63.11 percent, indicating that RC shear wall is effective in reducing pounding.
3) Use of steel bracing in building reduces storey displacement by 58.08 percent, indicating that it is also effective in moderating the pounding effect.
4) The optimal location of installation of BRB bracing to prevent pounding is location 3 in 8 storey building while location 4 in 5 storey building.
5) The optimal location of installation of RC shear wall to prevent pounding is location 3 in 8 storey building while location 3 in 5 storey building.
6) The optimal locations of installation of Steel bracing to prevent pounding is location 4 in 8 storey building while location 4 in 5 storey building.
7) The most effective approach for reducing pounding in buildings is determined to be RC shear walls, followed by steel bracings, and finally BRB bracings.

8.2 FUTURE WORKS

The present study deals with only BRB bracing, Steel bracings and RC shear wall. The use of RC-steel composite connector on storey level between both the buildings can also be an alternative solution. The various types of dampers like Friction dampers, viscous dampers, Metallic damper, Lead Injection Damper (LED) etc. can also be used to stiffen the structure and helps to prevent pounding.

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