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Effects in Buildings Due To Seismic Pounding

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I. INTRODUCTION

1.1 General

Investigations of past and recent earthquake damage have illustrated that the building structures are vulnerable to severe damage and/or collapse during moderate to strong ground motion. An earthquake with a magnitude of six is capable of causing severe damages of engineered buildings, bridges, industrial and port facilities as well as giving rise to great economic losses. Several destructive earthquakes have hit Egypt in both historical and recent times from distant and near earthquakes. The annual energy release in Egypt and its vicinity is equivalent to an earthquake with magnitude varying from 5.5 to 7.3. Pounding between closely spaced building structures can be a serious hazard in seismically active areas. Investigations of past and recent earthquakes damage have illustrated several instances of pounding damage (Astaneh-Asl et al.1994, Northridge Reconnaissance Team 1996, Kasai& Maison 1991) in both building and bridge structures. Pounding damage was observed during the 1985 Mexico earthquake, the 1988 Sequenay earthquake in Canada, the 1992 Cairo earthquake, the 1994 Northridge earthquake, the 1995 Kobe earthquake and 1999 Kocaeli earthquake. Significant pounding was observed at sites over 90 km from the epicenter thus indicating the possible catastrophic damage that may occur during future earthquakes having closer epicenters. Pounding of adjacent buildings could have worse damage as adjacent buildings with different dynamic characteristics which vibrate out of phase and there is insufficient separation distance or energy dissipation system to accommodate the relative motions of adjacent buildings. Past seismic codes did not give definite guidelines to preclude pounding, because of this and due to economic considerations including maximum land usage requirements, especially in the high density populated areas of cities, there are many buildings worldwide which are already built in contact or extremely close to another that could suffer pounding damage in future earthquakes. A large separation is controversial from both technical (difficulty in using expansion joint) and economical loss of land usage) views. The highly congested building system in many metropolitan cities constitutes a major concern for eismic pounding damage. For these reasons, it has been widely accepted that pounding is a desirable phenomenon that should be prevented or mitigated zones in connection with the corresponding design ground acceleration values will lead in many cases to earthquake actions which are remarkably higher than defined by the design codes used up to now. The most simplest and effective way for pounding mitigation and reducing damage due to pounding is to provide enough separation but it is sometimes difficult to be implemented due to detailing problem and high cost of land. An alternative to the seismic separation gap provision in the structure design is to minimize the effect of pounding through decreasing lateral motion (Kasaiet al. 1996, Abdullah et al. 2001, Jankowski et al 2000, Ruangrassamee & Kawashima 2003,Kawashima & Shoji 2000), which can be achieved by joining adjacent structures at critical locations so that their motion could be in-phase with one another or by increasing the pounding buildings damping capacity by means of passive structural control of energy dissipation system or by seismic retrofitting.

The focus of this study is the development of an analytical model and methodology for the formulation of the adjacent building-pounding problem based on the classical impact theory, an investigation through parametric study to identify the most important parameters is carried out. The main objective and scope are to evaluate the effects of structural pounding on the global response of building structures; to determine the minimum seismic gap between buildings and provide engineers with practical analytical tools for predicting pounding response and damage. A realistic pounding model is used for studying the response of structural system under the condition of structural pounding during elcentro earthquakes for medium soil condition at seismic zone V. Two adjacent multi-story buildings are considered as a representative structure for potential pounding problem. Dynamic and pushover analysis is carried out on the structures to observe displacement of the buildings due to earthquake excitation. The behavior of the structures under static loads is linear and can be predicted. When we come to the dynamic behaviors, we are mainly concerned with the displacements, velocity and accelerations of the structure under the action of dynamic loads or earthquake loads. Unpredictability in structural behaviors is encountered when the structure goes into the post-elastic or non-linear stage. The concept of push over analysis can be utilized for estimating the dynamic needs imposed on a structure by earthquake ground motions and the probable locations of the failure zones in a building can be ascertained by observing the type of hinge formations. The strength capacity of the weak zones in the post-elastic range can then be increased by retrofitting.

For the purpose of this study, SAP2000 has been chosen, a linear and non-linear static and dynamic analysis and design program for three dimensional structures. The application has many features for solving a wide range of problems from simple 2-D trusses to complex 3-D structures. Creation and modification of the model, execution of the analysis, and checking and optimization of the design are all done through this single interface. Graphical displays of the results, including real-time animations of time-history displacements, are easily produced.

1.2 Seismic Pounding Effect between Buildings

Pounding is one of the main causes of severe building damages in earthquake. The non-structural damage involves pounding or movement across separation joints between adjacent structures. Seismic pounding between two adjacent buildings occur

- during an earthquake
- different dynamic characteristics
- adjacent buildings vibrate out of phase
- at-rest separation is insufficient

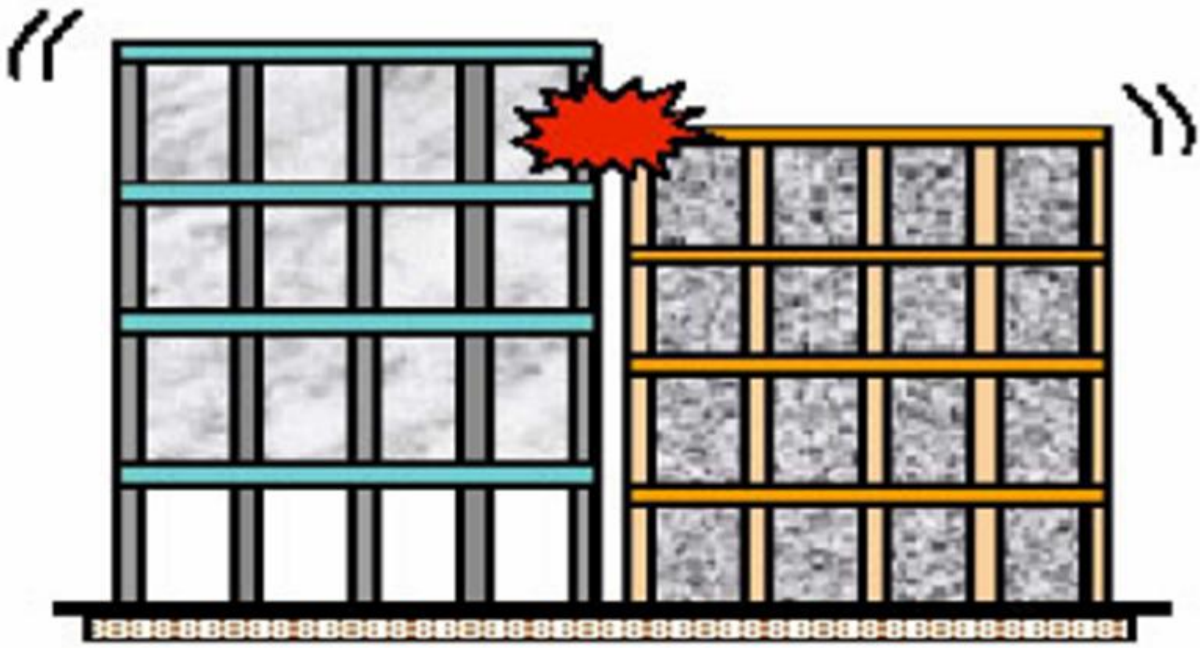


Fig 1.1 Seismic Pounding between Adjacent Buildings.

A separation joint is the distance between two different building structures - often two wings of the same facility - that allows the structures to move independently of one another.

A seismic gap is a separation joint provided to accommodate relative lateral movement during an earthquake. In order to provide functional continuity between separate wings, building utilities must often extend across these building separations, and architectural finishes must be detailed to terminate on either side. The separation joint may be only an inch or two in older constructions or as much as a foot in some newer buildings, depending on the expected horizontal movement, or seismic drift. Flashing, piping, fire sprinkler lines, HVAC ducts, partitions, and flooring all have to be detailed to accommodate the seismic movement expected at these locations when the two structures move closer together or further apart. Damage to items crossing seismic gaps is a common type of earthquake damage. If the size of the gap is insufficient, pounding between adjacent buildings may result in damage to structural components the buildings.

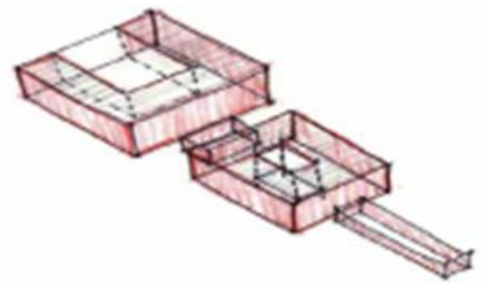


Fig 1.2 A diagram of a Roman house from the first century AD.

1.2.1 Required Seismic Separation Distance To Avoid Pounding

Seismic pounding occurs when the separation distance between adjacent buildings is not large enough to accommodate the relative motion during earthquake events. Seismic codes and regulations worldwide specify minimum separation distances to be provided between adjacent buildings, to preclude pounding, which is obviously equal to the relative displacement demand of the two potentially colliding structural systems. For instance, according to the 2000 edition of the International building code and in many seismic design codes and regulations worldwide, minimum separation distances (Lopez Garcia 2004) are given by ABSolute sum (ABS) or Square Root of Sum of Squares (SRSS) as follow:

$$S = U_a + U_b \qquad \text{ABS (1)}$$
$$S = (U_a^2 + U_b^2) \qquad \text{SRSS (2)}$$

where S = separation distance and U_a , U_b = peak displacement response of adjacent structures A and B, respectively.

Bureau Of Indian Standards clearly gives in its code IS 4326 that a Separation Section is to be provided between buildings. Separation Section is defined as `A gap of specified width between adjacent buildings or parts of the same building, either left uncovered or covered suitably to permit movement in order to avoid hammering due to earthquake `. Further it states that ` For buildings of height greater than 40 meters, it will be desirable to carry out model or dynamic analysis of the structures in order to compute the drift at each storey, and the gap width between the adjoining structures shall not be less than the sum of their dynamic deflections at any level.`

Thus it is advised to provide adequate gap between two buildings greater than the sum of the expected bending of both the buildings at their top, so that they have enough space to vibrate. Separation of adjoining structures or parts of the same structure is required for. Structures having different total heights or storey heights and different dynamic characteristics. This is to avoid collision during an earthquake. Minimum width of separation gaps as mentioned in 5.1.1 of IS 1893 : 1984, shall be as specified in Table 1.1 The design seismic coefficient to be used shall be in accordance with IS 1893 : 1984

SL No.	Type of Constructions	Gap Width/Storey, n mm for Design Seismic Coefficient □h =0.12
i).	Box system or frames with shear walls	15.0
ii).	Moment resistant reinforced concrete frame	20.0
iii).	Moment resistant steel frame	30.0

Table 1.1: Minimum width of separation gaps as mentioned in 5.1.1 of IS 1893 : 1984

NOTE: Minimum total gap shall be 25 mm. For any other value of □h the gap width shall be determined proportionately.

1.3 Methods of Seismic Analysis of a Structure

Various methods of differing complexity have been developed for the seismic analysis of structures. The three main techniques currently used for this analysis are:

1. Dynamic analysis.

- Linear Dynamic Analysis.
- Non-Linear Dynamic Analysis.

2. Push over analysis.

1.3.1 Dynamic Analysis

All real physical structures, when subjected to loads or displacements, behave dynamically. The additional inertia force from, Newton's second law, are equal to the mass times the acceleration. If the loads or displacements are applied very slowly then the inertia forces can be neglected and a static load analysis can be justified. Hence, dynamic analysis is a simple extension of static analysis.

The force equilibrium of a multi-degree-of-freedom lumped mass system as a function of time can be expressed by the following relationship:

F(t)i + F (t)d + F (t)s=F (t) (1)

in which the force vectors at time t are

F(t)i is a vector of inertia forces acting on the node masses
F (t)d) is a vector of viscous damping, or energy dissipation, forces
F (t)s is a vector of internal forces carried by the structure
F (t) is a vector of externally applied loads

Equation (1) is based on physical laws and is valid for both linear and nonlinear systems if equilibrium is formulated with respect to the deformed geometry of the structure.
For many structural systems, the approximation of linear structural behavior is made in order to convert the physical equilibrium statement, Equation (1), to the following set of second-order, linear, differential equations:

M u.. (t)a + C u. (t)a + K u (t)a =F (t) (2)

in which M is the mass matrix (lumped or consistent C), is a viscous damping matrix (which is normally selected to approximate energy dissipation in the real structure) and K is the static stiffness matrix for the system of structural elements.

The time-dependent vectors **u (t)a, u. (t)a**) and **u.. (t)a** are the absolute node displacements, velocities and accelerations, respectively.

For seismic loading, the external loading F(t) is equal to zero. The basic seismic motions are the three components of free-field ground displacements (u(t)ig)that are known at some point below the foundation level of the structure. Therefore, we can write Equation (12.2) in terms of the displacements u (t)a, velocities u. (t)a and Accelerations u.. (t)a that are relative to the three components of free-field ground displacements. Therefore, the absolute displacements, velocities and accelerations can be eliminated from Equation (2) by writing the following simple equations:

u (t)a = u (t)a + Ix u (t)xg + I y u (t)yg + I z u (t)zg
u. (t)a = u. (t)a + Ix u. (t)a xg + I y u. (t)a yg + I z u. (t)a u (t)zg
u.. (t)a = u.. (t)a + Ix u.. (t)a xg + I y u.. (t)a yg + I z u.. (t)a u (t)zg

where Ii is a vector with ones in the "i" directional degrees-of-freedom and zero in all other positions. The substitution of Equation (3) into Equation (2) allows the node point equilibrium equations to be rewritten as

M u.. (t)a +C u. (t)a + K u (t)a =(- Mx u.. (t)a xg - **My**u.. (t)a yg - **Mz** u.. (t)a zg **)**

The simplified form of Equation is possible since the rigid body velocities and displacements associated with the base motions cause no additional damping or structural forces to be developed.

1.3.1.1 Response Spectrum Analysis

The response spectrum technique is really a simplified special case of modal analysis. The modes of vibration are determined in period and shape in the usual way and the maximum response magnitudes corresponding to each mode are found by reference to a response spectrum. The response spectrum method has the great virtues of speed and cheapness. The basic mode superposition method, which is restricted to linearly elastic analysis, produces the complete time history response of joint displacements and member forces due to a specific ground motion loading [1,2]. There are two major disadvantages of using this approach. First, the method produces a large amount of output information that can require an enormous amount of computational effort to conduct all possible design checks as a function of time. Second, the analysis must be repeated for several different earthquake motions in order to assure that all the significant modes are excited, since a response spectrum for one earthquake, in a specified direction, is not a smooth function.

There are significant computational advantages in using the response spectra method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode using smooth design spectra that are the average of several earthquake motions. In this analysis, the CQC method to combine these maximum modal response values to obtain the most probable peak value of displacement or force is used. In addition, it will be shown that the SRSS and CQC3 methods of combining results from orthogonal earthquake motions will allow one dynamic analysis to produce design forces for all members of the structure.

1.3.2 Nonlinear Dynamic Analysis

Nonlinear Dynamic analysis can be done by direct integration of the equations of motion by step by step procedures. Direct integration provides the most powerful and informative analysis for any given earthquake motion. A time dependent forcing function (earthquake accelerogram) is applied and the corresponding response–history of the structure during the earthquake is computed. That is, the moment and force diagrams at each of a series of prescribed intervals throughout the applied motion can be found. Computer programs have been written for both linear elastic and non-linear inelastic material behavior using step-by-step integration procedures. One such program is SAP2000 in which three–dimensional non-linear analyses can be carried out taking as input the three orthogonal accelerogram components from a given earthquake, and applying them simultaneously to the structure.

1.3.3 Push over Analysis

The non-linear static procedure or simply push over analysis is a simple option for estimating the strength capacity in the post-elastic range. This procedure involves applying a predefined lateral load pattern which is distributed along the building height. The lateral forces are then monotonically increased in constant proportion with a displacement control node of the building until a certain level of deformation is reached. The applied base shear and the associated lateral displacement at each load increment are plotted. Based on the capacity curve, a target displacement which is an estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the building at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking

The most frequently used terms in pushover analysis as given in ATC-40 are:

1.3.3.1 Capacity curve

It is the plot of the lateral force V on a structure, against the lateral deflection d, of the roof of the structure. This is often referred to as the 'push over' curve. Performance point and location of hinges in various stages can be obtained from pushover curves as shown in the fig. The range AB is elastic range, B to IO is the range of immediate occupancy IO to LS is the range of life safety and LS to CP is the range of collapse prevention.

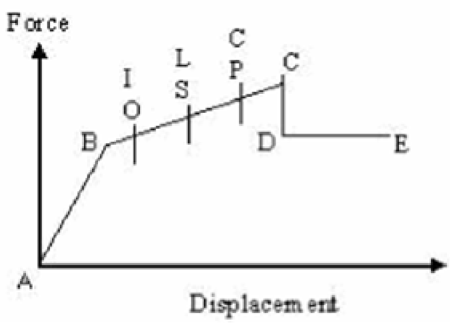


Fig 1.3 Different stages of plastic hinge

1.3.3.2 Capacity-spectrum

It is the capacity curve transformed from shear force vs. roof displacement (V vs. d) coordinates into spectral acceleration vs. spectral displacement (Sa vs. Sd) coordinates.

1.3.3.3 Demand

It is a representation of the earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or deformations that the structure is expected to undergo. This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

1.3.3.4 Demand - Spectrum

It is the reduced response spectrum used to represent the earthquake ground motion in the capacity spectrum method.

1.3.3.5 Displacement-based analysis

It refers to analysis procedures, such as the non linear static analysis procedures, whose basis lies in estimating the realistic, and generally inelastic, lateral displacements or deformations expected due to actual earthquake ground motion. Component forces are then determined based on the deformations.

1.3.3.6 Elastic response spectrum

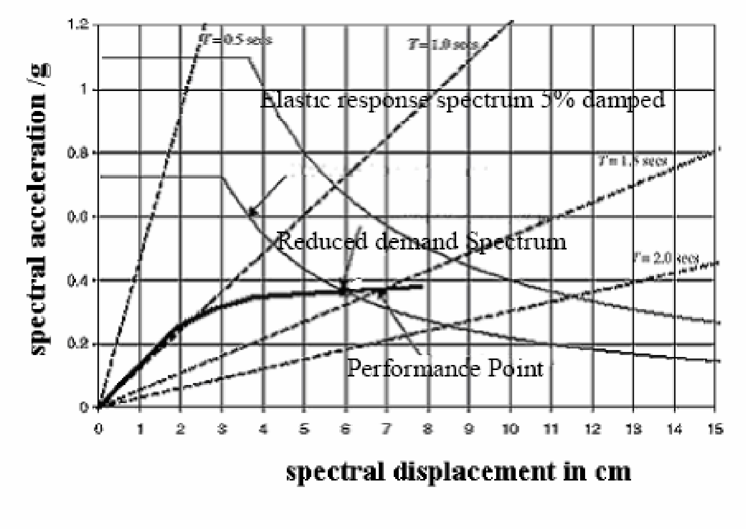
It is the 5% damped response spectrum for the (each) seismic hazard level of interest, representing the maximum response of the structure, in terms of spectral acceleration Sa, at any time during an earthquake as a function of period of vibration T.

1.3.3.7 Performance level

A limiting damage state or condition described by the physical damage within the building, the threat to life safety of the building's occupants due to the damage, and the post earthquake serviceability of the building. A building performance level is that combination of a structural performance level and a nonstructural performance level

1.3.3.8 Performance point

The intersection of the capacity spectrum with the appropriate demand spectrum in the capacity spectrum method (the displacement at the performance at the performance point is equivalent to the target displacement in the coefficient method). To have desired performance, every structure has to be designed for this level of forces. Desired performance with different damping ratios have been shown below:



Capacity spectrum
Fig 1.4 Determination of performance point

1.3.3.9 Yield (effective yield) point

The point along the capacity spectrum where the ultimate capacity is reached and the initial linear elastic force-deformation relationship ends and effective stiffness begins to decrease.

1.3.3.10 Building Performance levels

A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post earthquake serviceability of the building.

1.3.3.10.1 Immediate occupancy

The earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral forces resisting systems of the building retain nearly all of their pre- earthquake characteristics and capacities. The risk of life threatening injury from structural failure is negligible.

1.3.3.10.2 Life safety

The post-earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial collapse remains. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building, although the damage may not always be economically repairable.

1.3.3.10 Collapse prevention level

This building performance level consists of the structural collapse prevention level with no consideration of nonstructural vulnerabilities, except that parapets and heavy appendages are rehabilitated.

1.3.3.11 Primary elements

Refer to those structural components or elements that provide a significant portion of the structure's lateral force resisting stiffness and strength at the performance point. These are the elements that are needed to resist lateral loads after several cycles of inelastic response to the earthquake ground motion.

1.3.3.12 Secondary elements

Refer to those structural components or elements that are not, or are not needed to be, primary elements of the lateral load resisting system. However, secondary elements may be needed to support vertical gravity loads and may resist some lateral loads.

1.4 Objectives of Study

This Thesis aims at computing the minimum seismic gap between buildings for rigid floor diaphragm idealizations by dynamic and pushover analysis using SAP2000 Nonlinear. The principal objectives of the study are as follows:

1. Generation of three dimensional models of buildings for rigid floor diaphragm idealization to analyze dynamic and pushover analysis using SAP2000 Nonlinear
2. Performing linear and non-linear dynamic analysis of rigid floor diaphragm idealization for medium soil at Zone V.
3. Analyzing the displacement of buildings for Four Storey (G+4) and Eight Storey(G+8) building cases to permit movement, in order to avoid pounding due to earthquake by Linear and Non-linear Dynamic Analysis.
4. Performing Pushover analysis for rigid floor diaphragm idealization for three lateral load pattern on the models.
5. Comparison of pushover curves and capacity spectrums of rigid floor diaphragm idealizations for pushover analyses.
6. Comparison of displacement profiles for frames at different locations for rigid floor diaphragm idealization.

1.5 Organization of Thesis

The thesis is reported in five chapters. Chapter 1 aims at giving a glimpse of general concepts and objectives of the present study. Chapter 2 presents the available literature. Chapter 3 gives the details of the model under study, modeling aspects considered and discusses the procedure for rigid floor idealizations of buildings using dynamic and pushover analyses. Chapter 4 presents the results of rigid floor diaphragm idealizations for dynamic and pushover analysis. Chapter 5 outlines the conclusions and scope for future work

II. REVIEW OF LITERATURE

2.1 General

A series of integrated analytical and experimental studies has been conducted to investigate the seismic gap between adjacent buildings located in regions of high seismic risk. When a building experiences earthquake vibrations its foundation will move back and forth with the ground. These vibrations can be quite intense, creating stresses and deformation throughout the structure making the upper edges of the building swing from a few mm to many inches dependent on their height size and mass. This is uniformly applicable for buildings of all heights, whether single storeyed or multi-storeyed in highrisk earthquake zones. In Mexico earthquake it was observed that buildings of different sizes and heights vibrated with different frequencies. Where these were made next to each other they created stresses in both the structures and thus weakened each other and in many cases caused the failure of both the structures. Pounding produces acceleration and shear at various story levels that are greater than those obtained from the no pounding case. Pounding between closely spaced building structures can be a serious hazard in seismically active areas. Also, increasing gap width is likely to be effective when the separation is sufficiently wide practically to eliminate contact.

After a brief evaluation of methods currently standard in engineering practice to estimate seismic gap between buildings, nonlinearities in the structure are to be considered when the structure enters into inelastic range during devastating earthquakes. To consider this nonlinearity effects inelastic time history analysis is a powerful tool for the study of structural seismic performance. A set of carefully selected ground motion records can give an accurate evaluation of the anticipated seismic performance of structures. Despite the fact that the accuracy and efficiency of the computational tools have increased substantially, there are still some reservations about the dynamic inelastic analysis, which are mainly related to its complexity and suitability for practical design applications. Moreover, the calculated inelastic dynamic response is quite sensitive to the characteristics of the input motions, thus the selection of a suite of representative acceleration time-histories is mandatory. This increases the computational effort significantly. Nonlinear static procedures are enlightened due to their simplicity and, its accuracy is towards time history analysis.

Viviane Warnotte summarized basic concepts on which the seismic pounding effect occurs between adjacent buildings. He identified the conditions under which the seismic pounding will occur between buildings and adequate information and, perhaps more importantly, pounding situation analyzed. From his research it was found that an elastic model cannot predict correctly the behaviors of the structure due to seismic pounding. Therefore non-elastic analysis is to be done to predict the required seismic gap between buildings.

Robert Jankowski addressed the fundamental questions concerning the application of the nonlinear analysis and its feasibility and limitations in predicting seismic pounding gap between buildings. In his analysis, elastoplastic multi-degree-offreedom lumped mass models are used to simulate the structural behavior and non-linear viscoelastic impact elements are applied to model collisions. The results of the study prove that pounding may have considerable influence on behavior of the structures.

Shehata E. Abdel Raheem developed and implemented a tool for the inelastic analysis of seismic pounding effect between buildings. They carried out a parametric study on buildings pounding response as well as proper seismic hazard mitigation practice for adjacent buildings. Three categories of recorded earthquake excitation were used for input. He studied the effect of impact using linear and nonlinear contact force model for different separation distances and compared with nominal model without pounding consideration.

ANAGNOSTOPOULOS SA, SPILIOPOULOS KV studied the earthquake induced pounding between adjacent buildings. They idealized the building as lumped-mass, shear beam type, multi-degree-of-freedom (MDOF) systems with bilinear forcedeformation characteristics and with bases supported on translational and rocking springdashpots. Collisions between adjacent masses can occur at any level and are simulated by means of viscoelastic impact elements. They used five real earthquake motions to study the effects of the following factors: building configuration and relative size, seismic separation distance and impact element properties. It was found that pounding can cause high overstresses, mainly when the colliding buildings have significantly different heights, periods or masses. They suggests a possibility for introducing a set of conditions into the codes, combined with some special measures, as an alternative to the seismic separation requirement.

Hasan et al. [17] presented a simple computer based pushover analysis technique for performance based design of building frameworks subject to earthquake loading. The concept is based on conventional displacement method of elastic analysis. To measure the degree of plastification the term plasticity factor was used. The standard elastic and geometric stiffness matrices for frame elements are progressively modified to account for non-linear elastic-plastic behavior under constant gravity loads and incrementally increasing lateral loads.

Korkmaz and Sari [24] studied the performance of structures for various load patterns and variety of natural periods by performing pushover and nonlinear dynamic time history analysis and concluded that for taller structures pushover analysis is underestimating seismic demands.

ATC-40 Vol. 1, 2 [1] provides step by step procedures for seismic evaluation of new and existing RC buildings using nonlinear static procedures.

FEMA-273 [13] provides guidelines for seismic rehabilitation of buildings (both new and existing).

FEMA-274 [14] gives commentary for FEMA-273.

FEMA-356 [15] provides guidelines for seismic rehabilitation of buildings

2.2 Outcomes of Literature Review

From the available literature it was observed that most of the studies are confined on study of 2D frames and simple 3D structures with one story and one bay. The relative areas in which the dynamic and pushover analysis can be applied were discussed. Only a limited number of published works on comparison of use of dynamic and pushover analysis to find out the seismic gap between buildings.

Thus, after reviewing the existing literature it was felt that a comparative study on seismic pounding effect on buildings by dynamic and pushover analysis is required.

III. STRUCTURAL MODELING AND ANALYSIS

3.1 General

In order to evaluate the Seismic gap between buildings with rigid floor diaphragms using dynamic and pushover procedures two sample building was adopted The details of the building are reproduced in section 3.2.

The finite element analysis software SAP2000 Nonlinear [31] is utilized to create 3D model and run all analyses. The software is able to predict the geometric nonlinear behavior of space frames under static or dynamic loadings, taking into account both geometric nonlinearity and material inelasticity. The software accepts static loads (either forces or displacements) as well as dynamic (accelerations) actions and has the ability to perform eigenvalues, nonlinear static pushover and nonlinear dynamic analyses.

3.2 Details of the Models

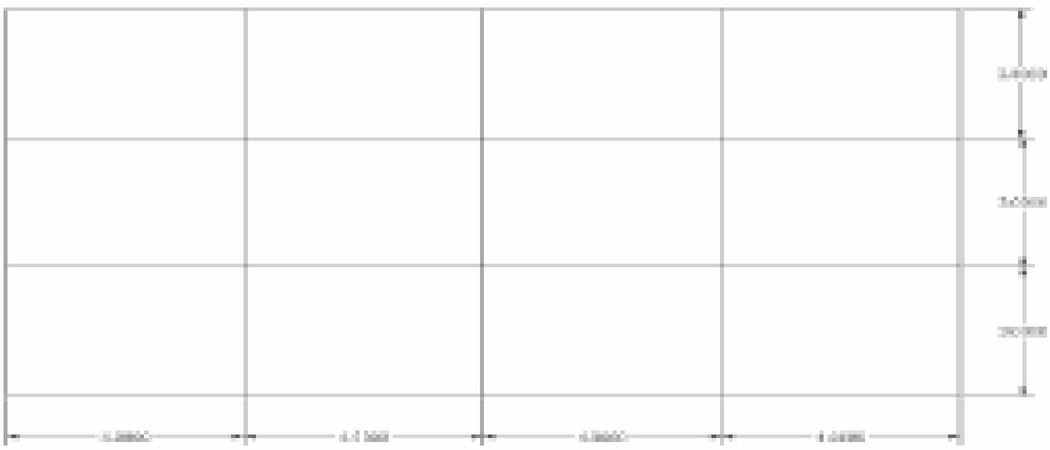
The models which have been adopted for study are asymmetric four storey(G+4) and eight storey (G+8) buildings. The buildings are consist of square columns with dimension 500mm x 500mm, all beams with dimension 350mm x 250mm. The floor slabs are taken as 125mm thick. The foundation height is 1.5m and the height of the all four stories is 3m. The modulus of elasticity and shear modulus of concrete have been taken as E = 2.55 ×107 kN/m2 and G = 1.06 ×107 kN/m2.

Three models have been considered for the purpose of the study.

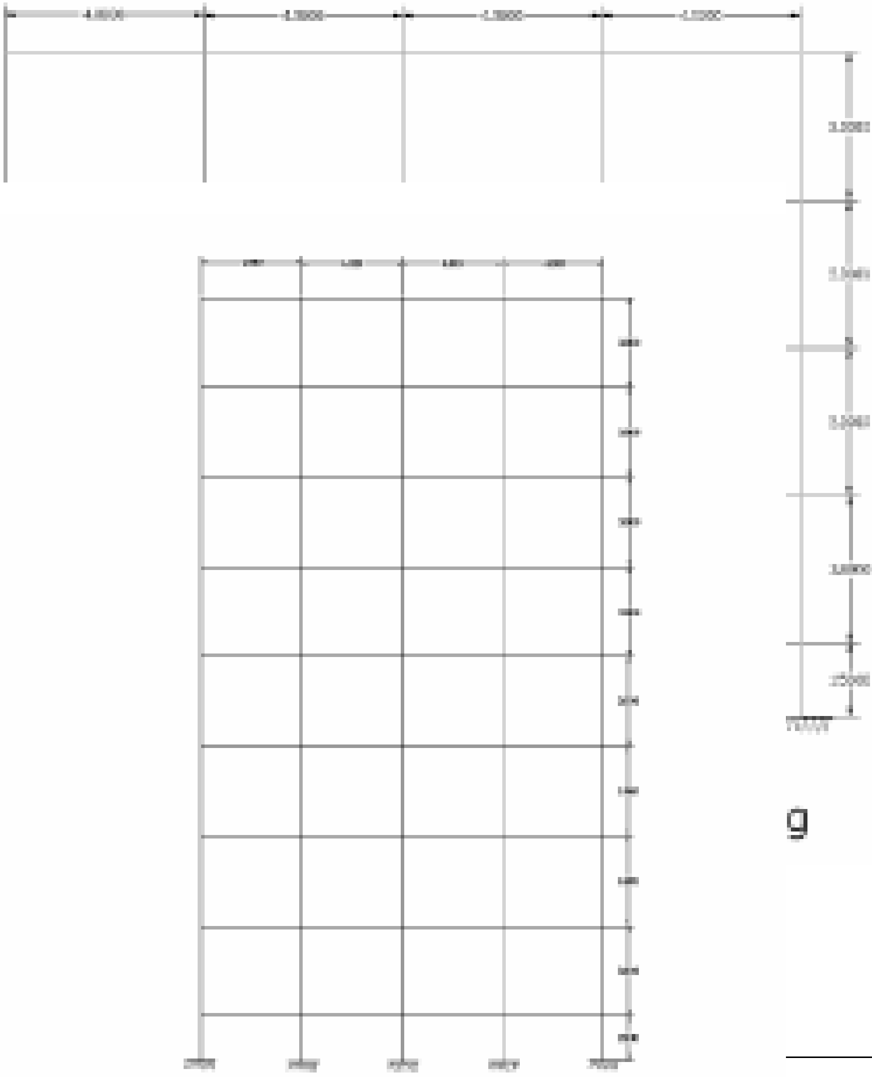
1. Four storey(G+4) adjacent building with equal floor levels.

2 Eight storey(G+8) adjacent buildings with Unequal floor levels.

The plan and sectional elevation of the two buildings are as shown below.



PLAN



Elevation for eight storey(G+8) building

Fig 3.1 Plan and elevation of the two model buildings.

3.2.1 Defining the material properties, structural components and modeling the structure:
Beam, column and slab specifications are as follows:

Column	500mm x 500mm
Beam	350mm x 250mm
Slab thickness	125mm
Reinforcement	
Columns	8-25 mm bars
Beams	4-20 mm bars at both top and bottom

The required material properties like mass, weight density, modulus of elasticity, shear modulus and design values of the material used can be modified as per requirements or default values can be accepted.

Beams and column members have been defined as 'frame elements' with the appropriate dimensions and reinforcement.

Soil structure interaction has not been considered and the columns have been restrained in all six degrees of freedom at the base.

Slabs are defined as area elements having the properties of shell elements with the required thickness. Slabs have been modeled as rigid diaphragms.

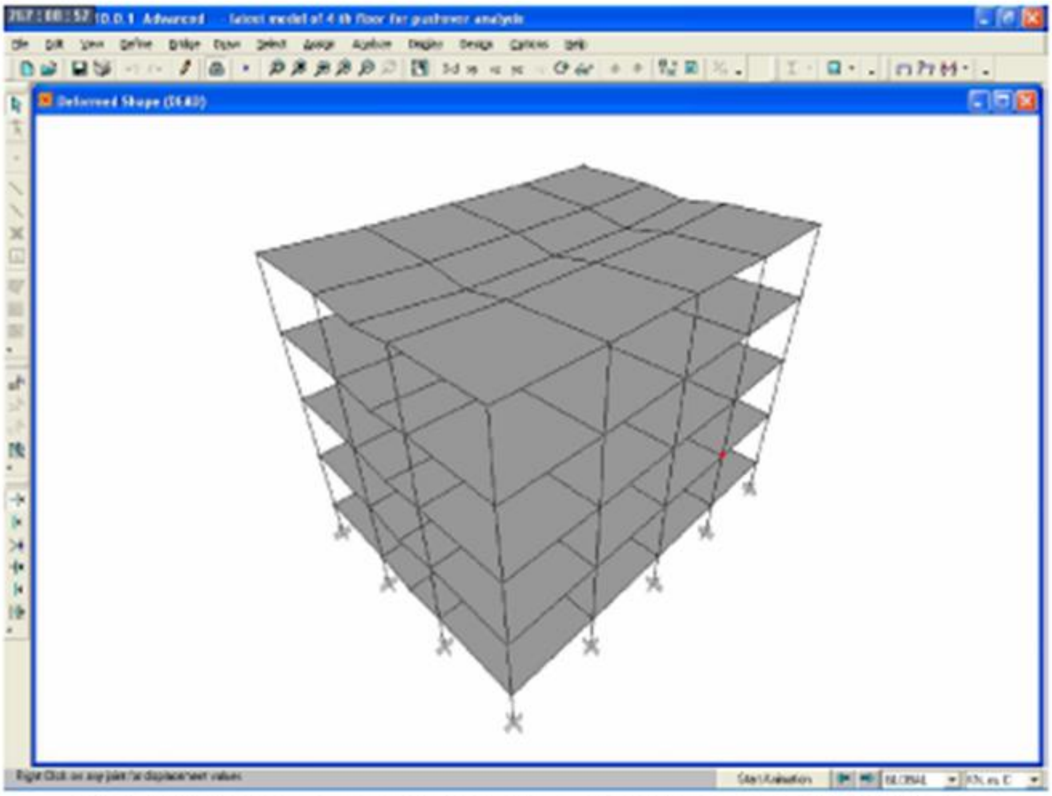


Fig 3.2 3-D view of the four storey (G+4) building created in SAP2000

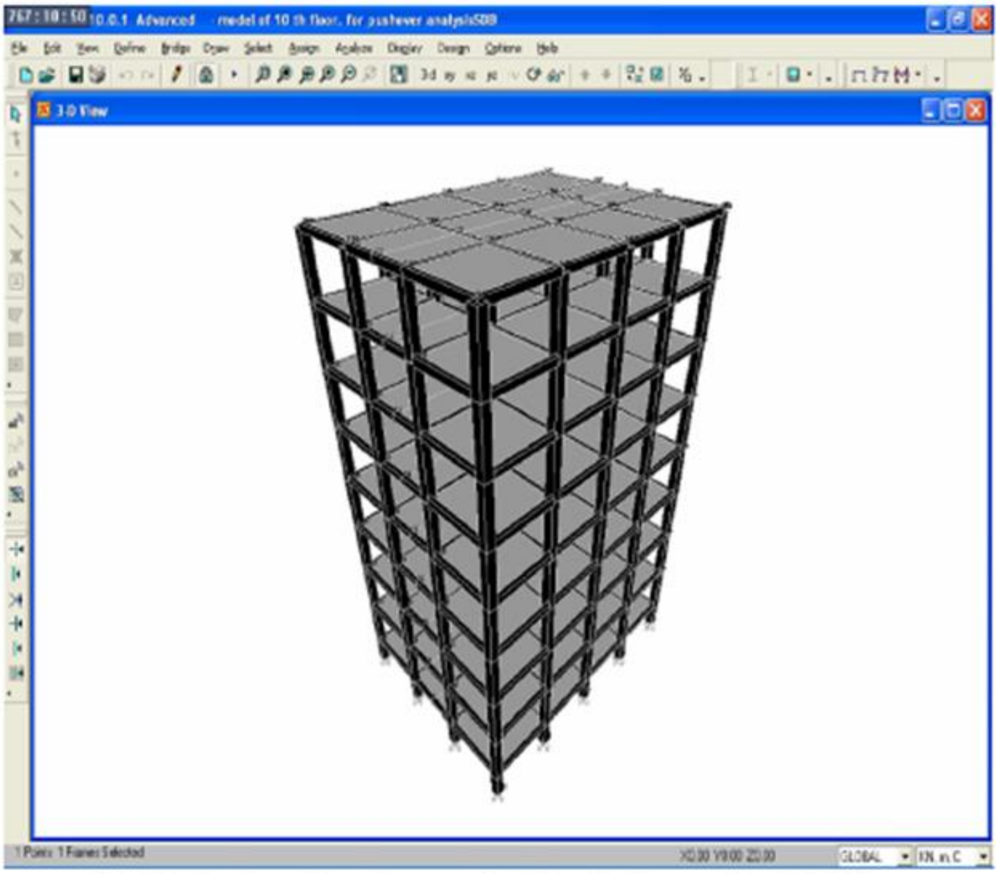


Fig 3.3 3-D view of the eight storey building (G+8) created in SAP2000

3.2.2 Assigning loads.
After having modeled the structural components, all possible load cases are assigned. These are as follows:

3.2.2.1 Gravity loads
Gravity loads on the structure include the self weight of beams, columns, slabs, walls and other permanent members. The self weight of beams and columns (frame members) and slabs (area sections) is automatically considered by the program itself. The wall loads have been calculated and assigned as uniformly distributed loads on the beams.

Wall load = unit weight of brickwork x thickness of wall x height of wall.
Unit weight of brickwork = 20KN/m3
Thickness of wall = 0.125m

Wall load on roof level =20 x 0.125 x 1=2.50KN/m (parapet wall height = 1m)
Wall load on all other levels = 20 x 0.125 x 3 = 7.50KN/m (wall height = 3m)

Live loads have been assigned as uniform area loads on the slab elements as per IS 1893(Part 1) 2002
Live load on roof 2 KN/m2
Live load on all other floors 3.0 KN/m2

As per Table 8, **Percentage of Imposed load to be considered in Seismic weight calculation**, IS 1893 (Part 1) 2002, since the live load class is up to 3 KN/m2 , 25% of the imposed load has been considered.

Quake loads have been defined considering the response spectra for medium soil as per IS 1893 (Part 1) 2002.

3.2.2.1.1 Defining load combinations:
According to IS 1893 (Part 1) 2002 for the limit state design of reinforced and prestressed concrete structures, the following load combinations have been defined

1.5(DL+LL)
1.2(DL+LL+EL)
1.2(DL+LL-EL)
1.5(DL+EL)
1.5(DL-EL)
0.9DL+1.5EL
0.9DL-1.5EL

DL- Dead Load
LL- Live load
EL- Earthquake load.

3.2.2.2 Earthquake lateral loads

The design lateral loads at different floor levels have been calculated corresponding to fundamental time period and are applied to the model. The method of application of this lateral load varies for rigid floor and flexible floor diaphragms. In rigid floor idealization the lateral load at different floor levels are applied at centre of rigidity of that corresponding floor in the direction of push in order to neglect the effect of torsion.

While idealizing the floor diaphragms as flexible, the design lateral load at all floors is applied such that the lateral load at each floor is distributed along the length of the floor in proportion to the mass distribution.

In our case, the slabs have been modeled as rigid diaphragms and in this connection, the centre of rigidity at each floor level has been determined and the earthquake lateral loads have been applied there.

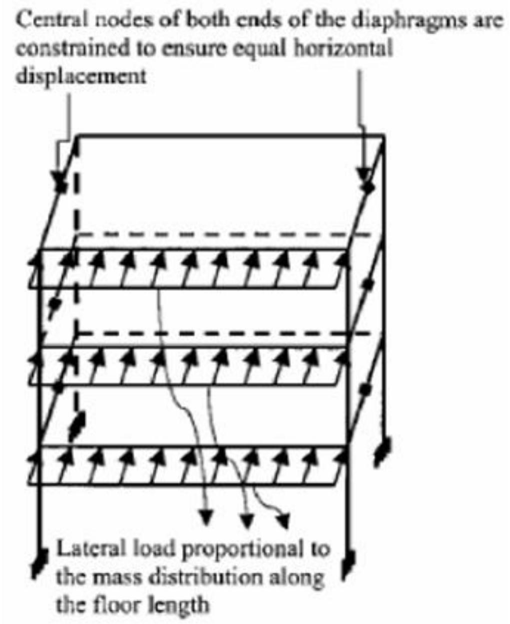


Fig. 3.4 Lateral loads for rigid floor diaphragm

3.2.3 ANALYSIS OF THE STRUCTURE

Namely three types of analysis procedures have been carried out for determining the various structural parameters of the model. Here we are mainly concerned with the behavior of the structure under the effect of ground motion and dynamic excitations such as earthquakes and the displacement of the structure in the inelastic range.

The analyses carried out are as follows:

- Response Spectrum Analysis
- Time History Analysis.
- Pushover analysis.

3.2.3.1 Response Spectrum Analysis

Here we are primarily concerned with observing the deformations, forces and moments induced in the structure due to dead, live loads and earthquake loads. The load case ‘Dead’ takes care of the self weight of the frame members and the area sections. The wall loads have been defined under a separate load case ‘Wall’ and the live loads under the case ‘Live’. Analysis is carried out for all three cases for obtaining the above mentioned parameters.

Modal analysis is carried out for obtaining the natural frequencies, modal mass participation ratios and other modal parameters of the structure. Response spectrum analysis of the three models are done in the zone V where

Z = 0.36 considering zone factor v
I = 1.0 considering residential building.
R = 5.0 considering special RC moment resistant frame (SMRF)
Sa/g = 2.5

For the Seismic pounding effect between adjacent buildings, response spectrum analysis is carried out using the spectra for medium soil as per **IS 1893 (Part 1) 2002.**
The spectral acceleration coefficient (Sa/g) values are calculated as follows.

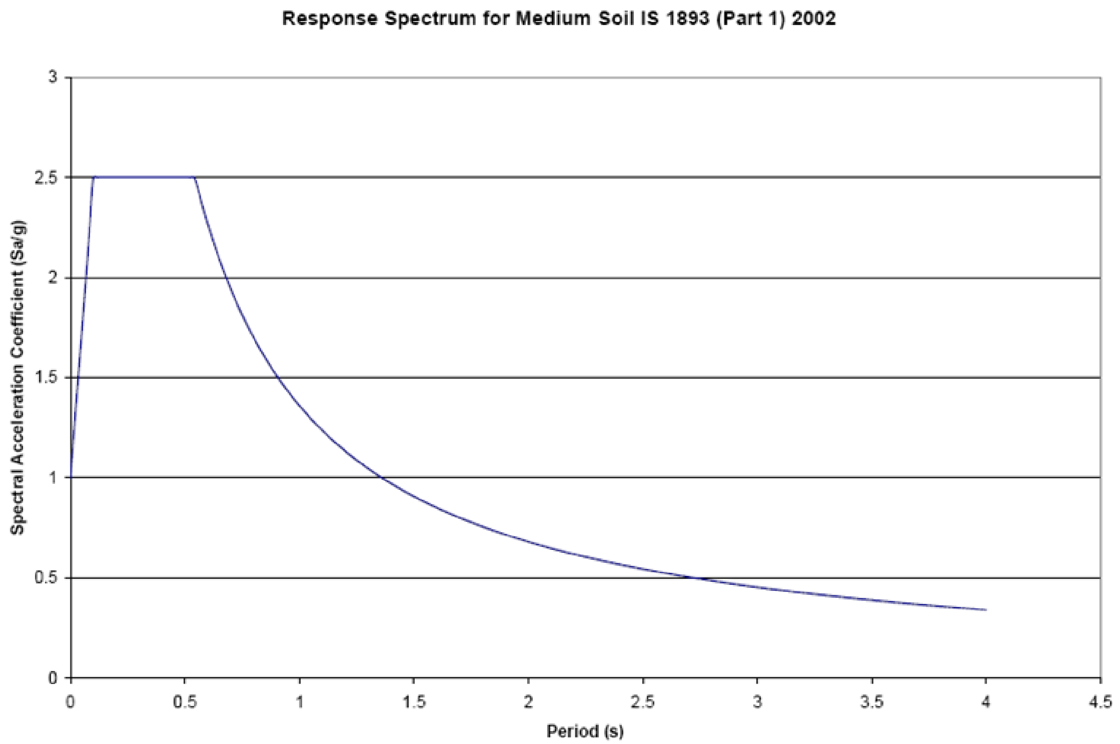
For medium soil sites,
Sa/g = 1 + 15T, (0.00 □ T □ 0.10), (T= time period in seconds)
= 2.50, (0.10 □ T □ 0.55)
= 1.36/T, (0.55 □ T □ 4.00)

The values of the spectral acceleration coefficient (Sa/g) for the time periods of 0.00 to 4.00 seconds calculated as per the above equations and the plot of spectral acceleration coefficient (Sa/g) Vs. Period are as shown.

Period (s)	Spectral Acceleration Coefficient (Sa/g)	Period (s)	Spectral Acceleration Coefficient (Sa/g)	Period (s)	Spectral Acceleration Coefficient (Sa/g)	Period (s)	Spectral Acceleration Coefficient (Sa/g)
0	1	0.94	1.446808511	1.88	0.723404255	2.82	0.482269504
0.02	1.3	0.96	1.416666667	1.9	0.715789474	2.84	0.478873239
0.04	1.6	0.98	1.387755102	1.92	0.708333333	2.86	0.475524476
0.06	1.9	1	1.36	1.94	0.701030928	2.88	0.472222222
0.08	2.2	1.02	1.333333333	1.96	0.693877551	2.9	0.468965517
0.1	2.5	1.04	1.307692308	1.98	0.686868687	2.92	0.465753425
0.12	2.5	1.06	1.283018868	2	0.68	2.94	0.462585034
0.14	2.5	1.08	1.259259259	2.02	0.673267327	2.96	0.459459459
0.16	2.5	1.1	1.236363636	2.04	0.666666667	2.98	0.456375839
0.18	2.5	1.12	1.214285714	2.06	0.660194175	3	0.453333333
0.2	2.5	1.14	1.192982456	2.08	0.653846154	3.02	0.450331126
0.22	2.5	1.16	1.172413793	2.1	0.647619048	3.04	0.447368421
0.24	2.5	1.18	1.152542373	2.12	0.641509434	3.06	0.444444444
0.26	2.5	1.2	1.133333333	2.14	0.635514019	3.08	0.441558442
0.28	2.5	1.22	1.114754098	2.16	0.62962963	3.1	0.438709677
0.3	2.5	1.24	1.096774194	2.18	0.623853211	3.12	0.435897436
0.32	2.5	1.26	1.079365079	2.2	0.618181818	3.14	0.433121019
0.34	2.5	1.28	1.0625	2.22	0.612612613	3.16	0.430379747
0.36	2.5	1.3	1.046153846	2.24	0.607142857	3.18	0.427672956
0.38	2.5	1.32	1.03030303	2.26	0.601769912	3.2	0.425
0.4	2.5	1.34	1.014925373	2.28	0.596491228	3.22	0.422360248
0.42	2.5	1.36	1	2.3	0.591304348	3.24	0.419753086
0.44	2.5	1.38	0.985507246	2.32	0.586206897	3.26	0.417177914
0.46	2.5	1.4	0.971428571	2.34	0.581196581	3.28	0.414634146
0.48	2.5	1.42	0.957746479	2.36	0.576271186	3.3	0.412121212
0.5	2.5	1.44	0.944444444	2.38	0.571428571	3.32	0.409638554
0.52	2.5	1.46	0.931506849	2.4	0.566666667	3.34	0.407185629
0.54	2.5	1.48	0.918918919	2.42	0.561983471	3.36	0.404761905
0.56	2.428571429	1.5	0.906666667	2.44	0.557377049	3.38	0.402366864
0.58	2.344827586	1.52	0.894736842	2.46	0.552845528	3.4	0.4
0.6	2.266666667	1.54	0.883116883	2.48	0.548387097	3.42	0.397660819
0.62	2.193548387	1.56	0.871794872	2.5	0.544	3.44	0.395348837

0.64	2.125	1.58	0.860759494	2.52	0.53968254	3.46	0.393063584
0.66	2.060606061	1.6	0.85	2.54	0.535433071	3.48	0.390804598
0.68	2	1.62	0.839506173	2.56	0.53125	3.5	0.388571429
0.7	1.942857143	1.64	0.829268293	2.58	0.527131783	3.52	0.386363636
0.72	1.888888889	1.66	0.819277108	2.6	0.523076923	3.54	0.384180791
0.74	1.837837838	1.68	0.80952381	2.62	0.519083969	3.56	0.382022472
0.76	1.789473684	1.7	0.8	2.64	0.515151515	3.58	0.379888268
0.78	1.743589744	1.72	0.790697674	2.66	0.511278195	3.6	0.377777778
0.8	1.7	1.74	0.781609195	2.68	0.507462687	3.62	0.375690608
0.82	1.658536585	1.76	0.772727273	2.7	0.503703704	3.64	0.373626374
0.84	1.619047619	1.78	0.764044944	2.72	0.5	3.66	0.371584699
0.86	1.581395349	1.8	0.755555556	2.74	0.496350365	3.68	0.369565217
0.88	1.545454545	1.82	0.747252747	2.76	0.492753623	3.7	0.367567568
0.9	1.511111111	1.84	0.739130435	2.78	0.489208633	3.72	0.365591398
0.92	1.47826087	1.86	0.731182796	2.8	0.485714286	3.74	0.363636364
Period (s)	Spectral Acceleration Coefficient (Sa/g)						
3.76	0.361702128						
3.78	0.35978836						
3.8	0.357894737						
3.82	0.356020942						
3.84	0.354166667						
3.86	0.352331606						
3.88	0.350515464						
3.9	0.348717949						
3.92	0.346938776						
3.94	0.345177665						
3.96	0.343434343						
3.98	0.341708543						
4	0.34						

Table no 3.1 spectral acceleration coefficients (S_a/g) Vs. Period.



3.2.3.1.1 Response spectrum analysis in SAP 2000

The step by step procedure is as follows

- Defining quake loads under the load type ‘quake’ and naming it appropriately.
- Defining response spectrum function as per IS 1893 (Part 1) 2002. The values of Sa/g Vs. T shown in Table 3.1 can be linked in the program in the form of a data file.
- Modifying the quake analysis case with the appropriate analysis case type, applied loads and scale factors.
- Running the analysis**.**

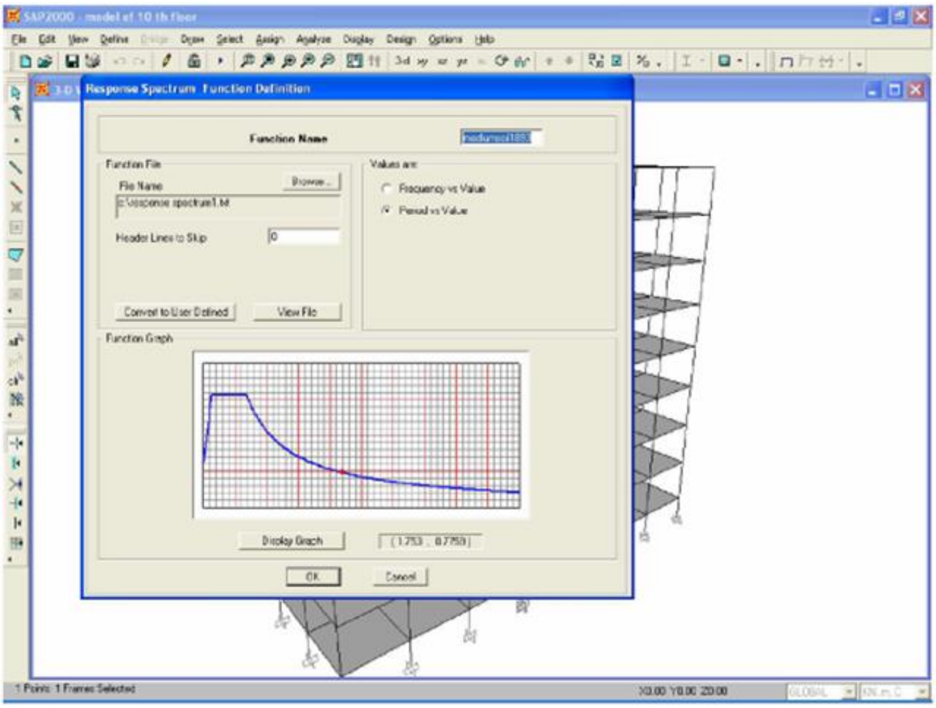
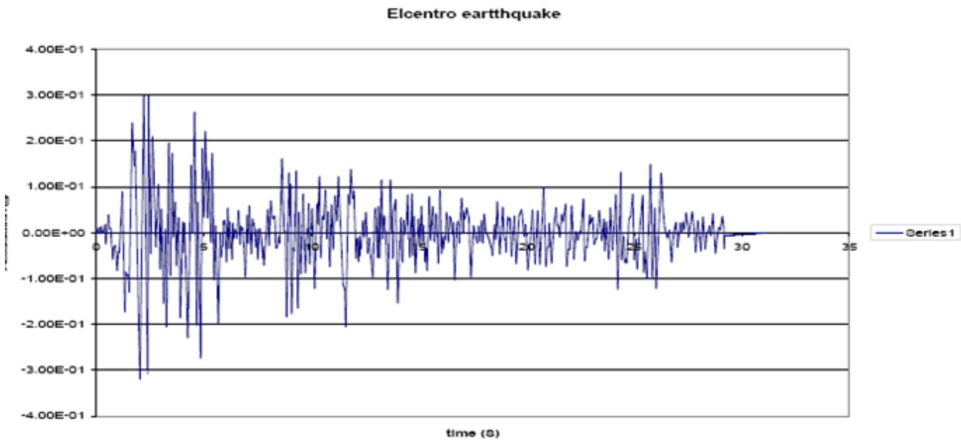


Fig 3.6 Defining response spectrum function (S_a/g) Vs. Period (table 3.1) in SAP2000

3.2.3.2 Time History Analysis of the structure

Time Hisotrey analysis has been carried out using the Imperial Valley Earthquake record of May 18, 1940 also known as the Elcentro earthquake for obtaining the various floor responses. The record has 1559 data points with a sampling period of 0.02 seconds. The peak ground acceleration is 0.319g. Newmark's direct integration method has been adopted and the mass and stiffness proportional coefficients have been calculated taking into account the frequency of the structure in two consecutive modes in the same direction.



Peak ground acceleration (PGA) – 0.319g

3.2.3.2.1 Time history analysis in SAP 2000

The step by step procedure is as follows

- Defining a time history function by adding a function from file. In our case, the Elcentro earthquake record of 1940 has been linked to the program.
- Defining a separate analysis case under the load type 'quake' with the appropriate analysis case type i.e. linear direct integration time history.
- Applying earthquake acceleration values from the defined time history function.
- Specifying the damping coefficients by calculating the mass and stiffness proportional coefficients as per the equations mentioned above or inputting the frequency or time periods of two consecutive modes of the structure in the same direction whereby the program itself calculates the required damping coefficients.
- Specifying a direct integration method in the program. In our case, we have adopted Newmark's direct integration method.
- Running the analysis.

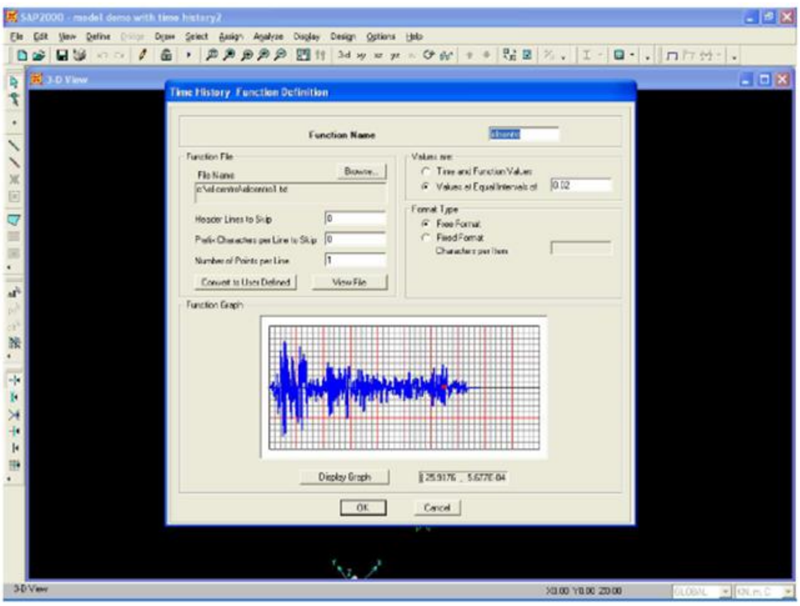


Fig: 3.8 Defining time history function (Elcentro, 1940) in SAP2000

3.3.3.3 Push over analysis

Push over analysis is a static, non-linear procedure that can be used to estimate the dynamic needs imposed on a structure by earthquake ground motions**.** In this procedure a predefined lateral load pattern is distributed along the building height. The lateral forces are then monotonically increased in constant proportion with a displacement control at the control node of the building until a certain level of deformation is reached.

For this analysis nonlinear plastic hinges have been assigned to all of the primary elements. Default moment hinges (M3-hinges) have been assigned to beam elements and default axial-moment 2-moment3 hinges (PMM-hinges) have been assigned to column elements. The floors have been assigned as rigid diaphragms by assigning diaphragm constraint.

3.3.4.3.1 Lateral load calculations

From Modal analysis fundamental time period of the structure have been found to be in mode 1—0.550391 sec and in mode 2—0.5034076 sec.The base shear has been calculated by running the response spectrum analysis. The lateral loads for rigid floor diaphragms have been calculated using parabolic lateral load patterns as adopted from IS 1893(part1)2002Para], triangular and uniform lateral load patterns from ATC-40 [1].

3.3.4.3.2 Seismic Weight of the Building

The Seismic Weight of the whole building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus appropriate amount of imposed load. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.

floor height from ground level in m	Seismic weight Wi in KN
13.5	2636.0
10.5	2636.0
7.5	2636.0
4.5	1600.5

Table 3.2 Seismic weight of the Four Storey Building

Total Seismic weight of the Model 1 Building W=9508.5 KN

floor height from ground level in m	Seismic weight Wi in KN
25.5	2636.0
22.5	2636.0
19.5	2636.0
16.5	2636.0
13.5	2636.0
10.5	2636.0
7.5	2636.0
4.5	1600.5

Table 3.3 Seismic weight of the Model 2Building

Total Seismic weight of the Model 2 Building W=20,052.5 KN

3.3.4.3.3 Base shear calculations

The total design lateral force or design Seismic base shear (Vb) have been obtained from SAP2000 using Response Spectrum Analysis as per IS1893 (part 1)- 2002[19].

The response spectrum ordinates used are for type (medium soil) for 5%damping and for seismic zone-v. The design seismic base shear (Vb) has been calculated using procedure given in IS 1893(part 1)-2002 as follows.

$$Vb = Ah\ W$$

Where Ah is the design horizontal seismic coefficient and is given by

$$Ah = \frac{ZI\ \{Sa / g\}}{2R}$$

Where Z = Zone factor given in table 2 of IS 1893-2002
I = Importance factor given in table 6 of IS 1893-2002
R = Response reduction factor given in table 7 of IS 1893-2002
Sa/g =Average response acceleration coefficient.

CALCULATION
For Four Storey Building

As per clause 7.1 of IS 1893(part 1) 2002 the fundamental time period of vibration (Ta) is

Ta = .075h0.75 Where h = height of the building
Ta =0.075x12.75
=0 .484 sec

From the response spectrum graph (**fig 3.8**)**,** Average response acceleration coefficient (Sa/g) is found to be 2.5

ZI {Sa / g}
Ah
=
2R
.36x1.0x2.5
=
2x5.0
= 0.09

Here Z = 0.36 considering zone factor v
I = 1.0 considering residential building.
R = 5.0 considering special RC moment resistant frame (SMRF)
Sa/g = 2.5

The design base shear in x-direction have been calculated using code provisions is
Vb =*Ah W*
=0.09x9508.5
=855.765 KN

For Eight Storey Building

As per clause 7.1 of IS 1893(part 1) 2002 the fundamental time period of vibration (Ta) is
Ta = *.075h0.75* Where h = height of the building
Ta =0.075x24.75
=0 .8132 sec

From the response spectrum graph (**fig 3.8**)**,** Average response acceleration coefficient (Sa/g) is found to be 1.65

ZI {Sa / g}
Ah
=
2R

.36x1.0x1.65
=
2x5.0
= 0.059706

Here Z = 0.36 considering zone factor v
I = 1.0 considering residential building.
R = 5.0 considering special RC moment resistant frame (SMRF)
Sa/g = 1.65

The design base shear in x-direction have been calculated using code provisions is
Vb =*Ah W*

=0.059706x20,052.5
=1,197254 KN

3.3.4.3.4 Lateral load profiles

IS 1893 (Part 1) 2002 parabolic lateral load (PLL) at floor ‘i’ is given by--
Wi h i
2
Qpi=Vb n
 Wj hj 2
j=1
Triangular lateral load (TLL) at floor ‘i’ is given by
Wi h i
Qpi=Vb n
 Wj hj
j=1
Uniform lateral load (ULL) at floor ‘i’ is given by
Wi
Qpi=Vb n
 Wj
j=1
where *Q*= lateral loads as per IS: 1893-2002 and ATC-40 at each floor level
W = total seismic weight the structure
W= seismic weight of floor i
h= height of floor measured from base
n = is the number of levels at which the masses are lumped.

Lateral force calculations in each storey
Vertical distribution of lateral forces as per IS 1893-2002 presented in table 3.4
Vertical distribution of lateral forces as per IS: 1893-2002 and ATC-40 are presented in

Table 3.4-3.5.

Floor height from h_i ground level (m)	Parabolic Lateral Load (KN)	Triangular lateral Load (KN)	Uniform Lateral Load (KN)
4.5	29.096	68.3032	144.025
7.5	133.242	187.412	237.218
10.5	261.265	262.463	237.218
13.5	431.904	337.428	237.218

Table 3.4 Distribution of lateral loads on different floors for Four Storey Building

Floor height from h_i ground level (m)	Parabolic Lateral Load (KN)	Triangular lateral Load (KN)	Uniform Lateral Load (KN)
4.5	6.704	27.656	95.559
7.5	31.008	75.427	157.319
10.5	60.820	105.358	157.319
13.5	100.449	136.486	157.319
16.5	150.135	166.418	157.319
19.5	209.758	196.349	157.319
22.5	279.199	227.470	157.319
25.5	358.697	257.40	157.319

Table 3.5 Distribution of lateral loads on different floors for Eight Storey Building

3.3.4.3.5 Push over analysis in SAP2000

The step by step procedure for buildings with rigid floor diaphragm is as follows:

- A three dimensional computer model was generated.
- Linear static, modal and response spectrum analysis were performed for specified response spectrum.
- The base shear from response spectrum analysis is used for calculating the design lateral loads.
- Centers of rigidity at various floor levels are calculated and are applied to the model.
- The calculated lateral load is distributed along the height of the building.
- The lateral loads at different floor levels are applied at centre of rigidity of the respective floor level.
- The rigid floor condition is given to the floors at different levels.
- The primary elements are identified and plastic hinges are assigned. The beam elements are assigned with plastic hinge as given in ATC-40 and FEMA – 273, 356. The beam elements are assigned with moment (M3) hinges and the column elements are assigned with axial load, moment in 2 and 3 – directions (PMM) hinges.
- Pushover analysis cases are then defined. The first case is for dead and live loads starting from zero initial conditions (unstressed state). The second case is defined for the calculated lateral loads and starts from the end conditions of the previous state. Non-linear parameters are defined as per requirements or default values are accepted.
- Analysis is then run and pushover curves are obtained.
- For the model with bracings, default axial hinges are defined in the bracings keeping the other parameters same and push over analysis is carried out.

3.3.4.3.6 Pushover Cases

Gravity and lateral push cases are considered for analysis. A set of lateral loads PUSH_IS, PUSH_TRI, and PUSH_UNI are given in Table 5.1 as per IS: 1893-2002, ATC – 40 and FEMA – 356 and are applied at corresponding floor levels.

3.3.4.3.7 Load Combinations for Pushover Analysis

Pushover analysis has been performed for lateral pushes in two orthogonal directions for parabolic load pattern as per IS: 1893-2002 and FEMA-356 along the height of the building. Lateral load combinations for different load cases are shown in Table 5.1.

Push over cases	Dead	Push_IS	Push_TRI	Push_UNI
GRAVX	1	0	0	0
Push_IS	0	1	0	0
Push_TRI	0	0	1	0
Push_UNI	0	0	0	1

Table 3.6 Lateral load combinations for different load cases for rigid floor diaphragms

IV. RESULTS AND DISCUSSION

4.1 General

SAP2000 is used to compute the response of a four (G+4) and eight storey (G+8) buildings for rigid floor diaphragm Linear Dynamic (response spectrum), Non Linear Dynamic (time history) and push over analysis.

Results from Response Spectrum analysis are observed for the natural frequencies and modal mass participation ratios and Displacements of the joints to determine the seismic pounding gap between adjacent structures of two models.

Results from time history analysis have been used to observe and compare the floor responses of all the two models. Pushover curves and capacity spectrum curves results have been used to observe and compare the displacement of the buildings in the performance point for three different lateral load patterns.

4.2. Response spectrum analysis

Response spectrum analysis has been carried out as per the response spectra mentioned in IS 1893(part1) 2002. The displacements for a particular joint at the top floor for two models have been tabulated as below

4.2.1 Analysis of Four storey buildings (G+4)

Load Combinations	Displacements in m		
	Longer (X)	Shorter (Y)	Vertical (Z)
1.5(DL+LL)	$-4.316*10^{-18}$	$-4.916*10^{-18}$	-0.011
1.2(DL+LL+EL)	0.0184	0.0184	-0.0004
1.2(DL+LL-EL)	0.0184	0.0184	-0.0004
1.5(DL+EL)	0.023	0.023	-0.0004
1.5(DL-EL)	0.023	0.023	-0.0004
0.9DL+1.5EL	0.023	0.023	-0.0001
0.9DL-1.5EL	0.023	0.023	-0.0001

Table 4.1 Displacement at the top floor in m for four storey buildings

4.2.2 Analysis of Eight storey buildings (G+8)

Load Combinations	Displacements in m		
	Longer (X)	Shorter (Y)	Vertical (Z)
1.5(DL+LL)	$-9.612*10^{-18}$	$2.667*10^{-18}$	-0.0026
1.2(DL+LL+EL)	0.0378	0.0378	-0.0015
1.2(DL+LL-EL)	0.0378	0.0378	-0.0028
1.5(DL+EL)	0.0472	0.0472	-0.0014
1.5(DL-EL)	0.0472	0.0472	-0.0014
0.9DL+1.5EL	0.0472	0.0472	-0.0005
0.9DL-1.5EL	0.0472	0.0472	-0.0005

Table 4.2 Displacement at the top floor in m for eight storey buildings

Conclusion

Response spectrum result for pounding case is observed. From the above result it have been seen that considering equal floor levels between adjacent buildings the maximum displacement is for Four storey buildings (G+4) is 0.046m against the 0.08m seismic gap between the adjacent buildings provided as per IS 4326-2005 and for Eight storey buildings (G+8) is 0.096 which is much less then the seismic gap provided between the adjacent buildings as per IS 4326-2005.

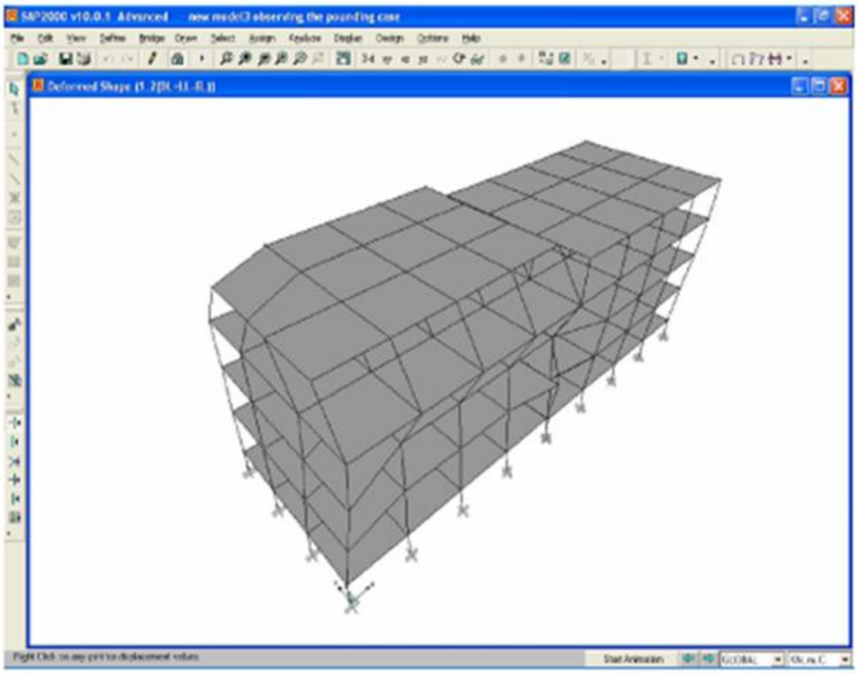


Fig 4.1 Deformed shape of two adjacent buildings due to less seismic gap.

4.3 Time History Analysis

Time history analysis has been carried out using the Imperial Valley Earthquake record of May 18, 1940 also known as the Elcentro earthquake for obtaining the various floor responses.

4.3.1 Analysis of Four storey building (G+4)

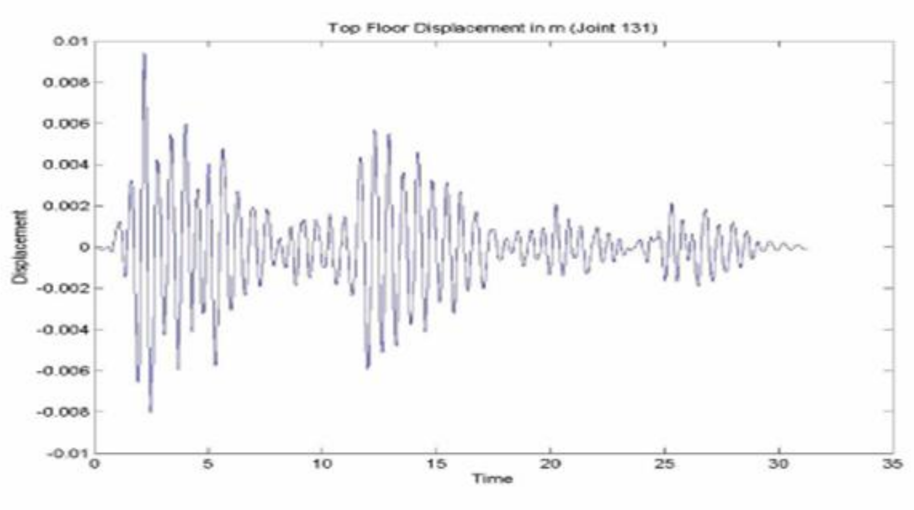


Fig: 4.2 Dispalcement time history of top floor (shorter direction) – model 1 (peak – 0.009437m)

Peak roof displacement of four storey building as obtained from time history analysis in SAP2000 is 0.009437m

4.3.2 Analysis of Eight storey building (G+8)

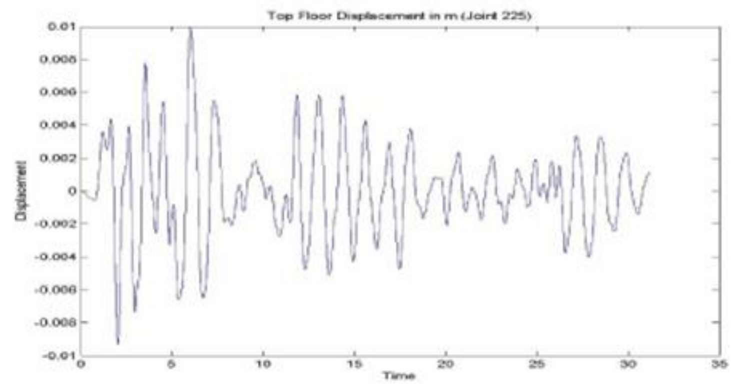


Fig: 4.3 Displacement time history of top floor (shorter direction) – model 2 (peak – 0.009947m)

Peak roof displacement of eight storey building as obtained from time history analysis in SAP2000 is 0.009947m

Conclusion

From the above records, it can be observed that all the three models have exhibited amplified responses for the top floor. The eight storey building has exhibited maximum response between the two models.

4.4 Pushover Analysis

Pushover analysis of an asymmetric building with rigid floor idealization for three different lateral load patterns; parabolic, triangular and uniform is carried out. All the analyses are performed for different nonlinear cases and the results namely; pushover curves, capacity spectrums, displacement are compared.

4.4.1 Pushover curves for four storey building

Pushover curve is a plot of *base shear* vs. *roof displacement* (*V* vs. *D*). It is also known as capacity curve. This curve gives idea about the base shear induced in the structure at performance point. The pushover curves for different lateral load cases for rigid floor idealization for four storey buildings are plotted and are shown in Figs. 4.4– 4.6

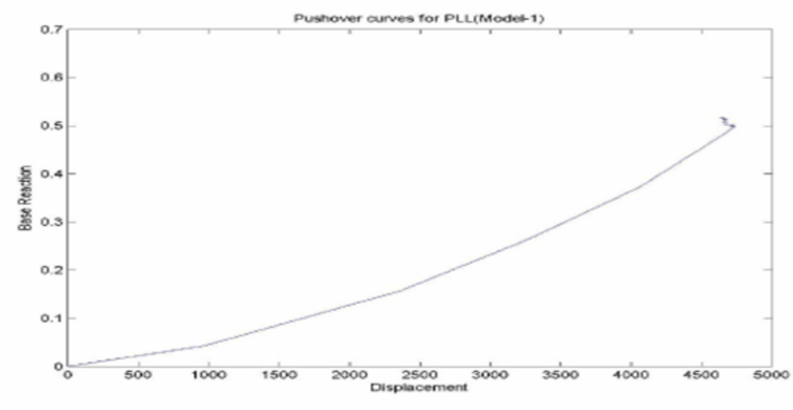


Fig 4.4 Pushover curve for parabolic lateral load pattern

4.4.3 Comparison of pushover curves

Fig. 4.10-4.11 shows the comparison of pushover curves for different lateral load distributions based on rigid floor diaphragm assumption. It is observed that the pushover curve is influenced by lateral load pattern.

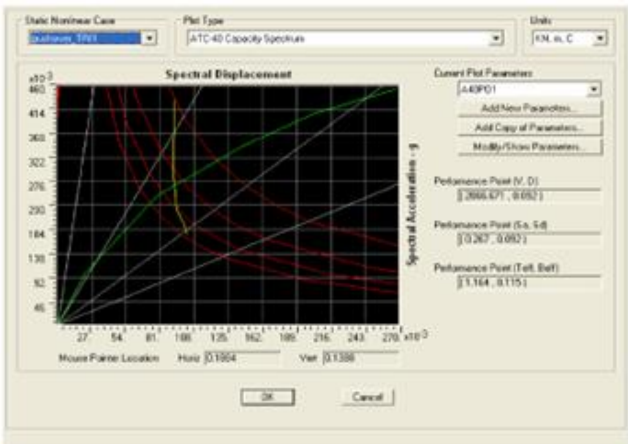


Fig.4.13 Capacity spectrum of four storey building for Triangular Load Pattern.

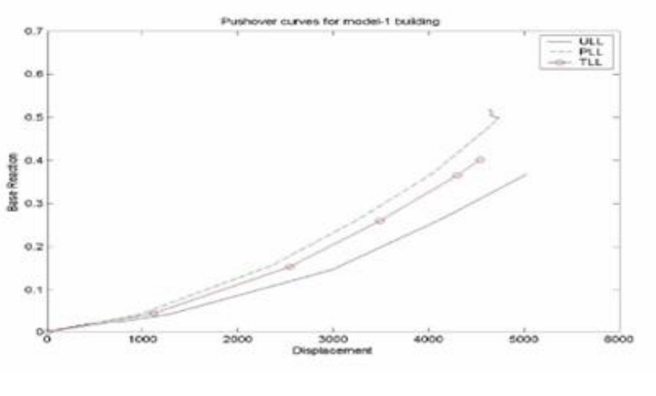


Fig. 4.10 Pushover curves of rigid floor diaphragm for different lateral load patterns of four storey building.

Fig. 4.11 Pushover curves of rigid floor diaphragm for different lateral load patterns of eight storey building.

It is observed that for any base shear, the displacement using rigid floor diaphragm idealization for parabolic load pattern more than that triangular and uniform lateral load pattern.

4.4.4 Capacity spectrum

Capacity spectrum is the capacity curve transformed from base shear vs. roof displacement coordinates into spectral acceleration vs. spectral displacement (Sa vs. Sd) co-ordinates. The performance point is obtained by superimposing demand spectrum on capacity curve transformed into spectral coordinates. To have desired performance, every structure has to be designed for the spectral acceleration corresponding to the performance point.

The capacity spectrum for four storey building is shown in fig 4.12-4.14

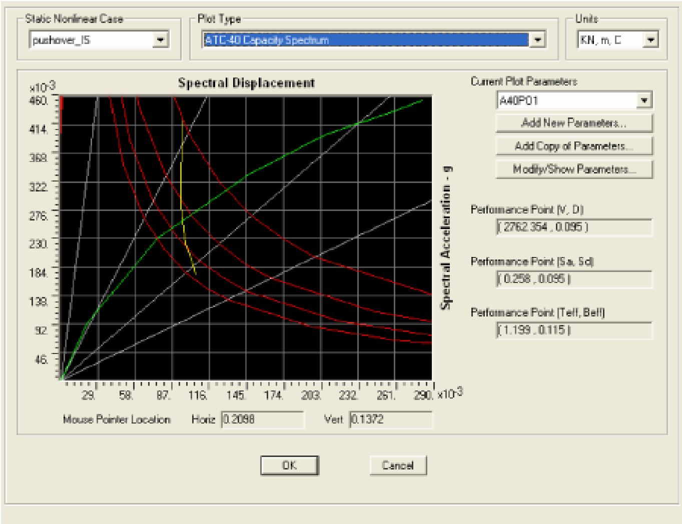


Fig.4.12 Capacity spectrum of four storey building for Parabolic Load Pattern

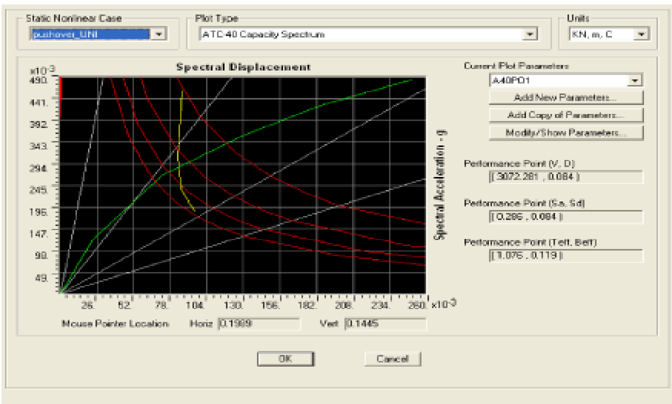


Fig.4.14 Capacity spectrum of four storey building for Uniform Load Pattern.

CASE	V in kN	D in m	S_a	S_d
PLL	2763.34	0.095	0.258	0.095
TLL	2866.71	0.092	0.267	0.092
ULL	3072.28	0.084	0.286	0.084

Table 4.3 Base Shear, Displacement, Spectral Acceleration and Spectral Displacement at Performance point.

From Table 4.3 it is evident that for rigid floor diaphragm idealization the *base shear, displacement, spectral acceleration and spectral displacement* at performance point are governed by the lateral load profile. For parabolic load pattern displacement is greater than triangular load pattern which has again larger displacement than uniform load pattern.

The capacity spectrum for eight storey building are shown in fig 4.15-4.17

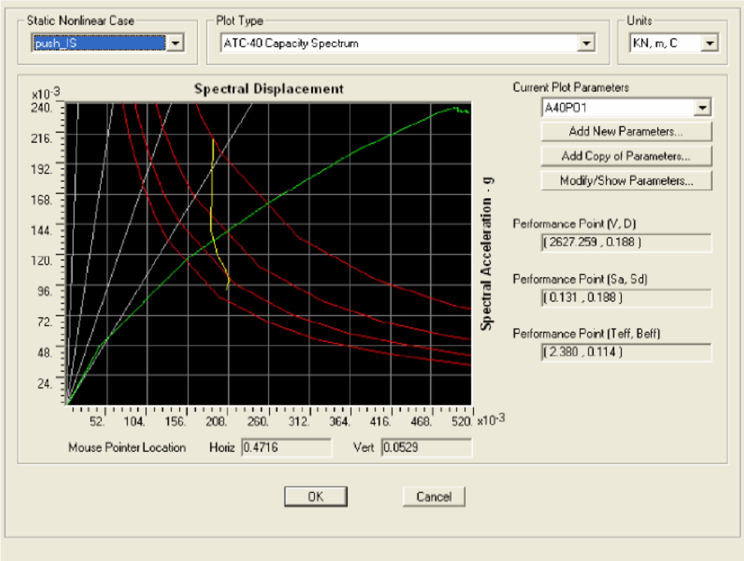


Fig.4.15 Capacity spectrum for eight storey building for Parabolic Load Pattern

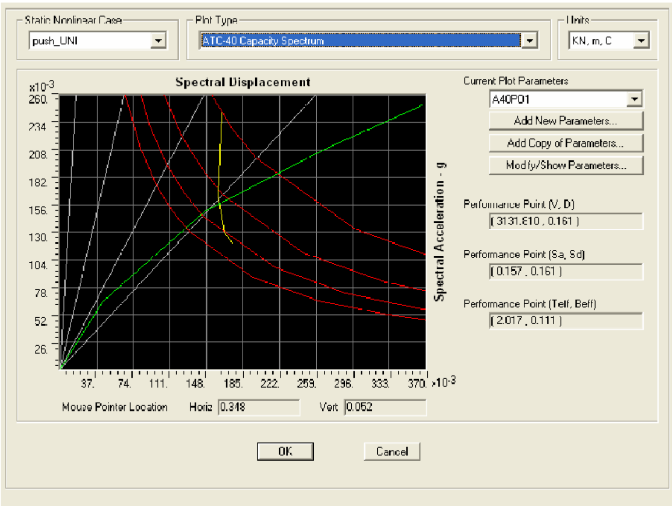


Fig.4.17 Capacity spectrum eight storey building for Uniform Load Pattern

CASE	V in kN	D in m	S_a	S_d
PLL	2627.259	0.188	0.131	0.188
TLL	2757.214	0.177	0.138	0.177
ULL	3131.810	0.161	0.157	0.161

Table 4.4 Base Shear, Displacement, Spectral Acceleration and Spectral Displacement at Performance point

From the table 4.4 it is shown that displacement due to parabolic load pattern is maximum as compared to other load pattern. At the performance point the maximum displacement is found to be 0.188m for parabolic load pattern.

Base shear is an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. Base reaction is greater in uniform load pattern. Therefore the displacement at the performance point of the buildings is minimum among the three lateral load patterns. From the analysis results, it is observable that the model 1 with four storey building has a maximum displacement is 0.095 and for model 2 with eight storey building has a maximum displacement is 0.188m at the performance point.

V. CONCLUSION

The purpose of this study has been to analyze seismic pounding effects between buildings and to observe the structural behaviour in the post elastic range. For this, SAP2000, a linear and non-linear static and dynamic analysis and design program for three dimensional structures has been used. Dynamic analysis has been carried out to know about the deformations, natural frequencies, and time periods, floor responses displacements. The non-linear static procedure or simply push over analysis is carried out to estimate the displacement at the performance point of the structure in the post-elastic range. The models that have been studied are ***1. Four storey (G+4) building 2. Eight storey (G+8) buildings*** of which have been created in SAP2000.

The first phase of the study involves the creation and analysis of the model and Linear dynamic analysis(Response Spectrum Analysis) for medium soil condition has been carried out on those models to observe displacement at the joint of the structure. Depending upon the analysis results, modification of the same for the purpose of no pounding is carried out on those models. Based on the observations from the analysis results, the following conclusions can be drawn. Response Spectrum analysis gives result that the two models have displacement within the permissible limit for seismic pounding between adjacent buildings with the seismic gap provided as per IS 4326-2005. It was found that minimum seismic gap can be provide 0.012m per storey between two four storey building and two eight storey building for no seismic pounding between buildings.

In the second phase of the project Nonlinear dynamic analysis with Elcentro earthquake excitation data as input is carried out on those models to observe the behaviour of the structure under earthquake excitation. The floor responses due to earthquake excitation in the Eight storey building is higher than the Four Storey building.

In pushover analysis three different lateral load patterns are used; parabolic, triangular and uniform. Based on the results obtained from these analyses, the following conclusions are drawn for the buildings under study. From the pushover curves obtained for three lateral load pattern shows displacement of the both buildings is maximum for parabolic lateral load pattern among all three lateral load pattern. Similarly, the displacements at performance point obtained from capacity spectrum for three lateral load patterns on the two buildings with rigid floor diaphragm follow the same trend. The maximum displacements of the buildings obtained from pushover analysis are higher than the results obtain