"Effect of Adding Shear wall & Bracings for Prevention of Seismic Pounding"

A PROJECT

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IN

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Under the supervision of

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HIMACHAL PRADESH INDIA

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CERTIFICATE

This is to certify that the work which is being presented in the project title **"Effect of Adding Shear Wall & Bracings for prevention of Seismic Pounding"** in partial fulfillment of the requirements for the award of the degree of Bachelor of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Mahtab Alam & Shivendra Pratap Singh Rautela** during a period from July 2015 to May 2016 under the supervision of **Mrs. Poonam Dhiman** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

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ABSTRACT

Collision between adjacent structures due to insufficient gap was witnessed in almost every major earthquake since 1960’s. The adjacent buildings collide and collapse during moderate to strong ground vibration caused by earthquakes. Separation distance of many buildings is inadequate to accommodate their relative motion, so buildings vibrate out of phase and collapse. Among the possible structural damage the seismic induced pounding has been commonly observed phenomenon. Major seismic events during the past decade such as those that have occurred in Loma prieta (1989 earthquake), Nepal (April 25, 2015), Central Western India (2001) have continued to demonstrate the destructive power of earthquakes, with destruction of engineered buildings, bridges, industrial and port facilities as well as giving rise to great economic losses. In the work presented here Time History & Response Spectrum of two R.C. buildings was carried out. The gap provided between the models was 0.3 meters. It was found that the two buildings were pounding on each other. By the use of symmetrical shear wall & steel bracings in these building models, seismic pounding was effectively prevented.

CONTENTS Page

List of Tables

List of figures

Chapter 1

INTRODUCTION

INTRODUCTION

1.1 General

A quake with a magnitude of six is capable of causing severe damage. Several destructive earthquakes have hit India in both historical and recent times. The annual energy release in India and its vicinity is equivalent to an earthquake with magnitude varying from 5.5 to 7.3. When two structures are close together, it is expected that they will pound against each other. This situation can be easily seen in highly populated cities. Many studies were made about structural pounding considering single degree of freedom. Pounding is a highly nonlinear phenomenon and a severe load condition that could result in high magnitude and short duration floor acceleration pulses in the form of short duration spikes, which in turn cause greater damage to building contents. Pounding is critical on the responses of the stiff system, especially when the system is highly out-of-phase. Essentially, in-phase systems exhibit displacement amplifications that are much closer to one; independent of model type .The pounding effect can be reduced in two ways:

1) By placing elastic materials between adjacent buildings or by reinforcing structural systems with cast-in-place reinforced concrete (RC) walls.

2) By providing a safe separation distance between adjacent buildings.

The highly congested building system in many metropolitan cities constitutes a major concern for seismic pounding damage. For these reasons, it has been widely accepted that pounding is an undesirable 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, 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. The main objective and scope are to determine the minimum seismic gap between buildings the effect of the different structures configurations on seismic performance is investigated. The lateral resistance of the structure was improved by installing Shear Walls & Bracing elements. A realistic pounding model is used for studying the response of structural system under the condition of structural pounding during El Centro earthquakes for medium soil condition at seismic zone IV. Two adjacent multi-story buildings are considered as a representative structure for potential pounding problem. Dynamic 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 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 strength capacity of the weak zones in the post-elastic range can then be increased by improving lateral resistance of the structure.

For the purpose of this study, Staad pro, SAP2000 have been chosen, a linear and non-linear static and dynamic analysis and design programs 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 the programs.

1.2 Seismic Pounding Effect between Buildings

Pounding is one of the main causes of severe building damages in earthquake as shown in figure 1.1. 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

Fig 1.1 Seismic Pounding between Adjacent Buildings.

A seismic gap is a separation joint provided to accommodate relative lateral movement during an earthquake as shown in figure 1.2. 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. Damage to items crossing seismic gaps is a common type of earthquake damage.

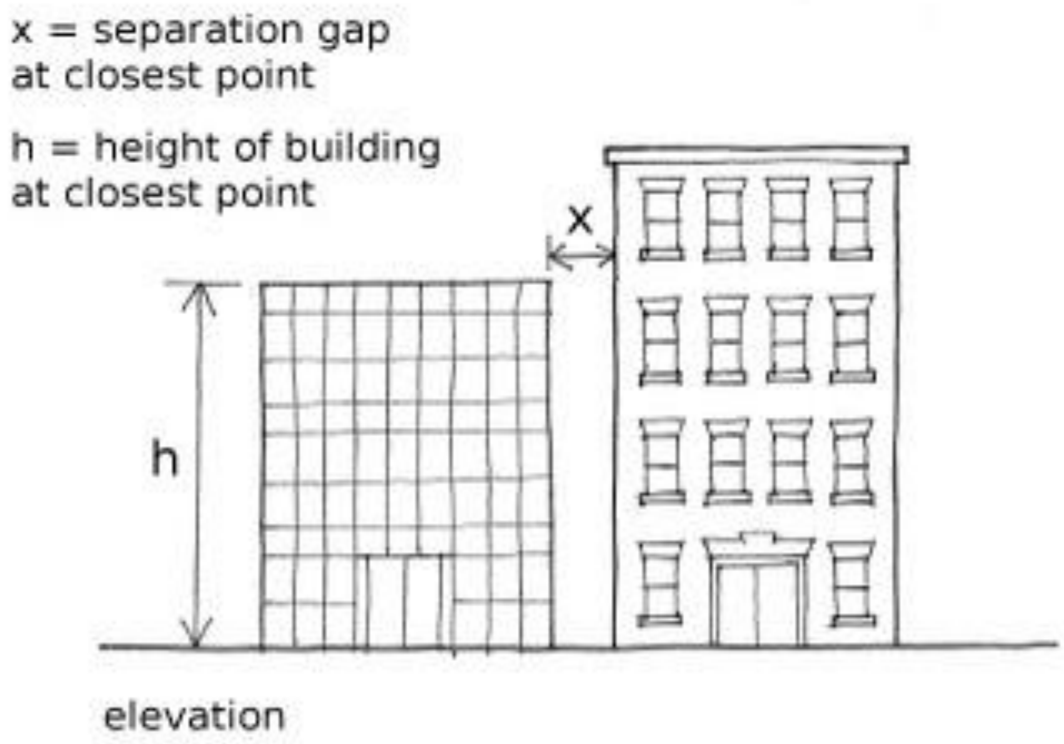


Fig 1.2 Seismic Gaps between Adjacent Buildings.

1.2.1 Required Seismic Separation Distance to Avoid Pounding

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

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

.

Sl. N0.	**Type of Construction**	**Gap Width/Storey, in mm for Design Seismic Coefficient** αh =0.1
1	Box system or frames with shear walls	**15.0**
2	Moment resistant reinforced concrete frame	**20.0**

IS1893:2007 Part1 mentioned that, separation should be R times the sum of displacements. R may be replaced by R/2 when two buildings are
at same levels, where R is response reduction factor (Clause 7.11.1) [6]. As per FEMA: 273-1997: Separation distance between adjacent structures shall be less than 4% of the building height and above to avoid pounding, also the equations for calculating gap are

$$S = U_a + U_b (ABS) \qquad (1)$$

$$S = \sqrt{U_a^2 + U_b^2} (SRSS) \qquad (2)$$

Where S = separation distance and U_a, U_b= peak displacement response of adjacent structures A and B. This method is most popular so in this study this method is adopted as other methods gives somewhat conservative values. In figure 1.3 shows Seismic Gap calculation by Square Root Sum of Square method.

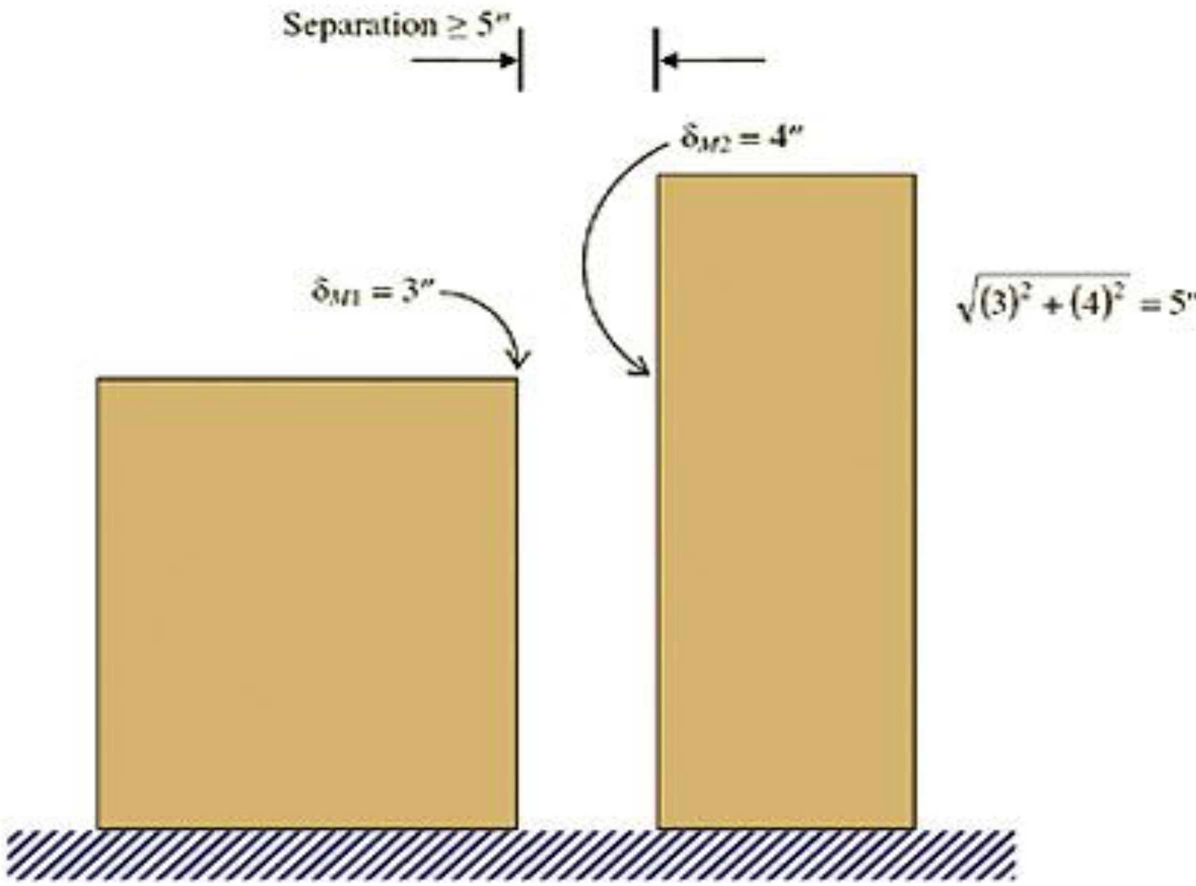


Fig 1.3 Seismic Gap by SRSS method.

1.3 PROVISON OF IS CODE 1893-2002 FOR SEISMIC ANLYSIS

The following assumptions shall be made in the earthquake resistant design of structures:

a)Earthquake causes impulsive ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady-state sinusoidal excitations will not occur as it would need time to buildup such amplitudes.

NOTE— however, there are exceptions where resonance-like conditions have been seen to occur between long distance waves and tall structures founded on deep soft soils.

b) Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.

c) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition (IS 456, IS 1343 and IS 800).

In the limit state design of reinforced and prestressed concrete structures, the following load combinations shall be accounted for:

1) 1.5(DL+IL)

2) 1.2(DL+ZL+EL)

3) 1.5(DL+EL)

4) 0.9DL* 1.5EL

Structure shall be determined by the following expression:

$$\boxed{Ah = ZISa/2Rg}$$

Z=Zone Factor

I= Importance factor

R= Response reduction Factor

Sa/g= Average response acceleration coefficient

1.4 METHODOLOGY

This study is carried out by analyzing reinforced concrete frames using linear static analysis, response spectrum analysis and linear time history analysis in SAP 2000 v16 software. Seismic and pounding responses of two multi-storey structures are studied in aspects of displacement and pounding force. Type of pounding being analyzed is the pounding effect where shorter building collides to adjacent taller building. For linear methods the building in earthquake zone IV is considered and for Time History function, ground excitation data of El Centro earthquake is chosen.

1.5 STRUCTURAL MODELING AND ANALYSIS

In order to observe pounding of multi-storey building (5 storey) is selected. Building is linear statically analyzed in STADD PRO as shown in fig. 14. The building is having 4 bays in X-direction and 3 bays in Z-direction. Width of the bays is 5m each and height of each storey is 3.2m and foundation height is 4.2m. Columns having size (0.23×0.45) m^2, beam are (0.23*0.40) m^2 and slab of thickness 0.125m. The building is located in Seismic zone V, the soil is medium stiff and the entire building is supported on raft foundation. The RC frames are in filled with bricks masonry. The lumped weight due to dead load is 12kN/m^2 on floors and 10kN/m^2 on the roof. The floors carry a live load of 4kN/m^2 on floors and 1.5 kN/m^2on the roofs.

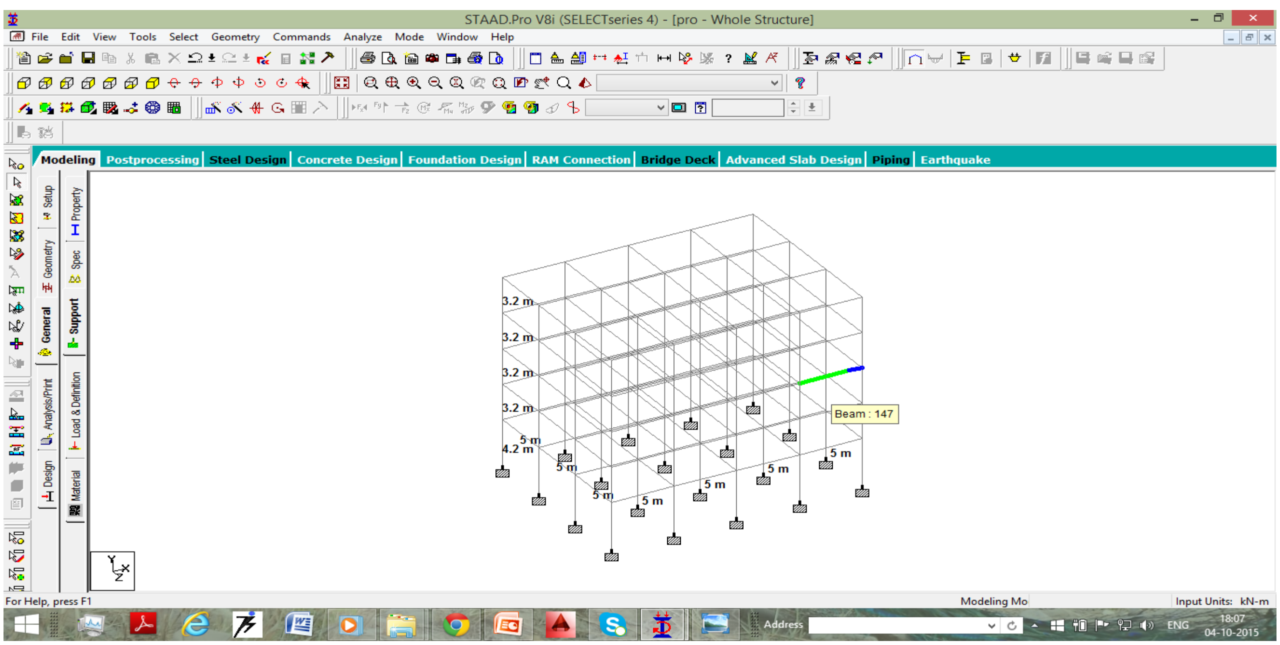


Fig. 1.4 Screen Shot of Modal in Staad.

<u>SOLUTION:</u>

Design Parameters:

For seismic zone 5, the zone factor is 0.36 (Table 2 of IS: 1893-2002).

Being an office building, the importance factor, I is 1 (Table 6 of IS:1893).

The response reduction factor, R, is 5 (Special RC moment resisting frame Table 7 of IS:1893)

Seismic Weights:

The floor area is 15×20=300sq.m.

Live load class is 4kN/sq.m, only 50 % of the live load is lumped at the floor and at the Roof no live lood is to be lumped.

FLOOR

W1=W2=W3=W4 =300×(12+0.5×4)

=4,200Kn

ROOF

W5 =300×10

=3,000kN

Total Seismic weight of the structure,

W =(4×4,200)+3,000

=19,800kN

Fundamental Period:

Lateral load resistance is provided by moment resisting frames infilled with bricks masonry panels. Hence, approximate fundamental natural period:

(Clause 7.6.2. of IS:1893)

EL in X-Direction;

T = 0.09h/square root d

= 0.09×17/square root 20

=0.342 sec

S_a/g = 2.5 (as per IS:1893)

A_h = $(ZI/2R)\times(S_a/g)$

=0.09

Design base shear:

*V_b $=A_h\times W$

=0.09×19,800

=1,782kN

EL in Z-Direction:

T $=0.09h/\sqrt{d}$

$=0.09\times17/\sqrt{15}$

=0.395sec

S_a/g =2.5

A_h =0.09

V_b =1,782kN

1.5 ANALYSIS & RESULT

- Time Period for x as per 1893 Loading = 0.342 seconds.
- Time Period for z as per 1893 Loading = 0.395 seconds.
- Sa/g = 2.5 as per IS 1893-2002.
- Load factor = 1.
- V_b = 1782kN.

Table 1.2: Lateral Force in X & Z directions in each floor of the building frame.

Storey Level	Height (m)	Weight (kN)	$Wi*hi^2/1000$	$Wi*hi^2/\sum Wi*hi^2$	Lateral Force in X-Direction	Lateral Force in Z-Direction
5	17	3,000	867	0.387	632.458	632.458
4	13.8	4,200	799.8	0.357	583.472	583.472
3	10.6	4,200	471.9	0.211	344.250	344.250
2	7.4	4,200	230	0.103	167.774	167.774
1	4.2	4,200	74.1	0.033	54.046	54.046
$\sum$			**2,235.8**	**1.091**		

Chapter 2

REVIEW OF LITERATURE

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 storied or multi-storied in high-risk 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.

S.K. Duggal on his profound interest on structures gave a detailed description about reinforced concrete buildings in his book "Earth quake resistant design of structures "describing a wall in a building which resist lateral loads originating from wind or earthquakes are known as shear walls". He considered flexural strength in the wall to be dominant force based on which design of structure to be carried out in tall shear walls. He described in detail about various types of shear walls with their load bearing capacities as per code requirements.

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 force deformation characteristics and with bases supported on translational and rocking spring dashpots. Collisions between adjacent masses can occur at any level and are simulated by means of viscous elastic 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.

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 of height (G+11) & (G+7). The relative areas in which the dynamic can be applied were discussed. Only a limited number of published works on comparison of use of dynamic 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.

Chapter3

DYNAMIC ANALYSIS

3.1 GENERAL

Structural analysis is mainly concerned with finding out the behavior of a physical structure when subjected to force. This action can be in the form of load due to the weight of things such as people, furniture, wind, snow, etc. or some other kind of excitation such as an earthquake, shaking of the ground due to a blast nearby, etc. In essence all these loads are dynamic, including the self-weight of the structure because at some point in time these loads were not there. The distinction is made between the dynamic and the static analysis on the basis of whether the applied action has enough acceleration in comparison to the structure's natural frequency. If a load is applied sufficiently slowly, the inertia forces (Newton's second law of motion) can be ignored and the analysis can be simplified as static analysis. **Structural dynamics**, therefore, is a type of structural analysis which covers the behavior of structures subjected to dynamic (actions having high acceleration) loading. Dynamic loads include people, wind, waves, traffic, earthquakes, and blasts. Any structure can be subjected to dynamic loading. Dynamic analysis can be used to find dynamic displacements, time history, and modal analysis.

A dynamic analysis is also related to the inertia forces developed by a structure when it is excited by means of dynamic loads applied suddenly (e.g., wind blasts, explosion, earthquake).

All real physical structures, when subjected to loads or displacements, behave dynamically. The additional inertia forces, 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.

All real structures potentially have an infinite number of displacements. Therefore, the most critical phase of a structural analysis is to create a computer model, with a finite number of mass less members and a finite number of node (joint) displacements that will simulate the behaviour of the real structure. The mass of a structural system, which can be accurately estimated, is lumped at the nodes. Also, for linear elastic structures the stiffness properties of the members, with the aid of experimental data, can be approximated with a high degree of confidence. However, the dynamic loading, energy dissipation properties and boundary (foundation) conditions for many structures are difficult to estimate. This is always true for the cases of seismic input.

3.1 Method of Dynamic Analysis

The method of dynamic analysis used here are -:

1) Time History Method., and

2) Response Spectrum Method.

3.1.1 Time History Method

Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non linear. Time history analysis is used to determine the dynamic response of a structure to arbitrary loading.

3.1.2 Response Spectrum Method

The Response Spectrum is a method of estimation of maximum responses (acceleration, velocity and displacement) of a family of SDOF systems subjected to a prescribed ground motion. The RSM utilizes the response spectra to give the structural designer a set of possible forces and deformations a real structure would experience under earthquake loads.

Response Spectrum method, being time consuming and tedious process, most of time, it resort to computer applications. Now while, modelling the structure, in most of available software's, usually, we model the space frame, neglecting the in-fill wall stiffness. These results in flexible frames, and due to which, in most of Cases, the program gives a higher Time Period and results into lower base shear.

3.1.2.1 Definition of Response Spectrum

For three dimensional seismic motion, the typical modal Equation is rewritten as

$$\ddot{y}(t)_n + 2\zeta_n \omega_n \dot{y}(t)_n + \omega_n^2 y(t)_n = p_{nx}\ddot{u}(t)_{gx} + p_{ny}\ddot{u}(t)_{gy} + p_{nz}\ddot{u}(t)_{gz}$$

where the three *Mode Participation Factors* are defined by $p_{ni} = -\phi_n{}^T \mathbf{M}_i$ in which i is equal to x, y or z. This section will address the modal combination problem due to one components of earthquake motion acting at the same time.

For input in one direction only, Equation (3.1) is written as

$$\ddot{y}(t)_n + 2\zeta_n \omega_n \dot{y}(t)_n + \omega_n^2 y(t)_n = p_{ni}\ddot{u}(t)_g \qquad (3.2)$$

Given a specified ground motion $\ddot{u}(t)_g$, damping value and assuming $p_{ni} = -1.0$ it is possible to solve Equation (3.2) at various values of ω and plot a curve of the maximum peak response $y(\omega)_{MAX}$. For this acceleration input, the curve is by definition the *displacement response spectrum* for the earthquake motion.

A plot of $\omega\, y(\omega)_{MAX}$ is defined as the ***pseudo-velocity spectrum*** and a plot of $\omega^2 y(\omega)_{MAX}$ is defined as the ***pseudo-acceleration spectrum.***

These three curves are normally plotted as one curve on special log paper. However, these pseudo values have minimum physical significance and are not an essential part of a response spectrum analysis. The true values for maximum velocity and acceleration must be calculated from the solution of Equation (3.2).

There is a mathematical relationship, however, between the pseudo-acceleration spectrum and the total acceleration spectrum. The total acceleration of the unit mass, single degree-of-freedom system, governed by Equation (3.2), is given by

$$\ddot{u}(t)_T = \ddot{y}(t) + \ddot{u}(t)_g \tag{3.3}$$

Equation (3.2) an be solved for y (t) and substituted into equation (3.3) which yields

$$\ddot{u}(t)_T = -\omega^2 y(t) - 2\xi\omega\dot{y}(t) \tag{3.4}$$

Therefore, for the special case of zero damping, the total acceleration of the system is equal to $\omega^2 y(t)$. For this reason, the ***displacement response spectrum*** curve is normally not plotted as modal displacement $y(\omega)_{MAX}$ vs ω. It is standard to present the curve in terms of *S(w)* vs. a period *T* in seconds, where

$$S(\omega)_a = \omega^2 y(\omega)_{MAX} \quad \text{and} \quad T = \frac{2\pi}{\omega}$$

(3.5a) and (3.5b)

The pseudo-acceleration spectrum, *S* (w)a , curve has the units of acceleration vs. period which has some physical significance for zero damping only. It is apparent that all response spectrum curves represent the properties of the earthquake at a specific site and are not a function of the properties of the structural system. After estimation is made of the linear viscous damping properties of the structure, a specific response spectrum curve is selected.

3.1.2.2 Typical response spectrum curves

A ten second segment of the Loma Prieta earthquake motions, recorded on a soft site in the San Francisco Bay Area, is shown in Figure 3.1 . The record has been corrected, by use of an iterative algorithm, for zero displacement, velocity and acceleration at the beginning and end of the ten second record. For the earthquake motions given in Figure 3.1a, the response spectrum curves for displacement and pseudo-acceleration are summarized in Figure 3.2a and 3.2b

The velocity curves have been intentionally omitted since they are not an essential part of the response spectrum method. Furthermore, it would require considerable space to clearly define terms such as peak ground velocity, pseudo velocity spectrum, relative velocity spectrum and absolute velocity spectrum.

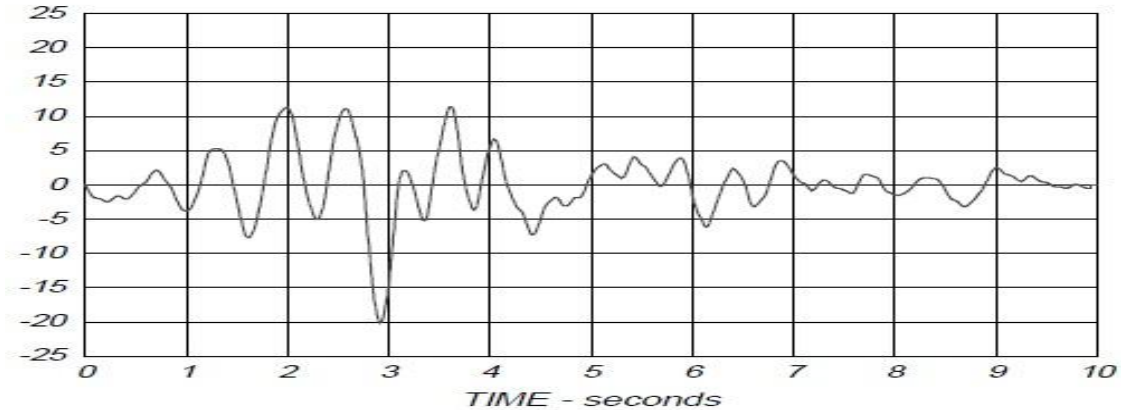


Figure 3.1a Typical Earthquake Ground Acceleration- Period of Gravity

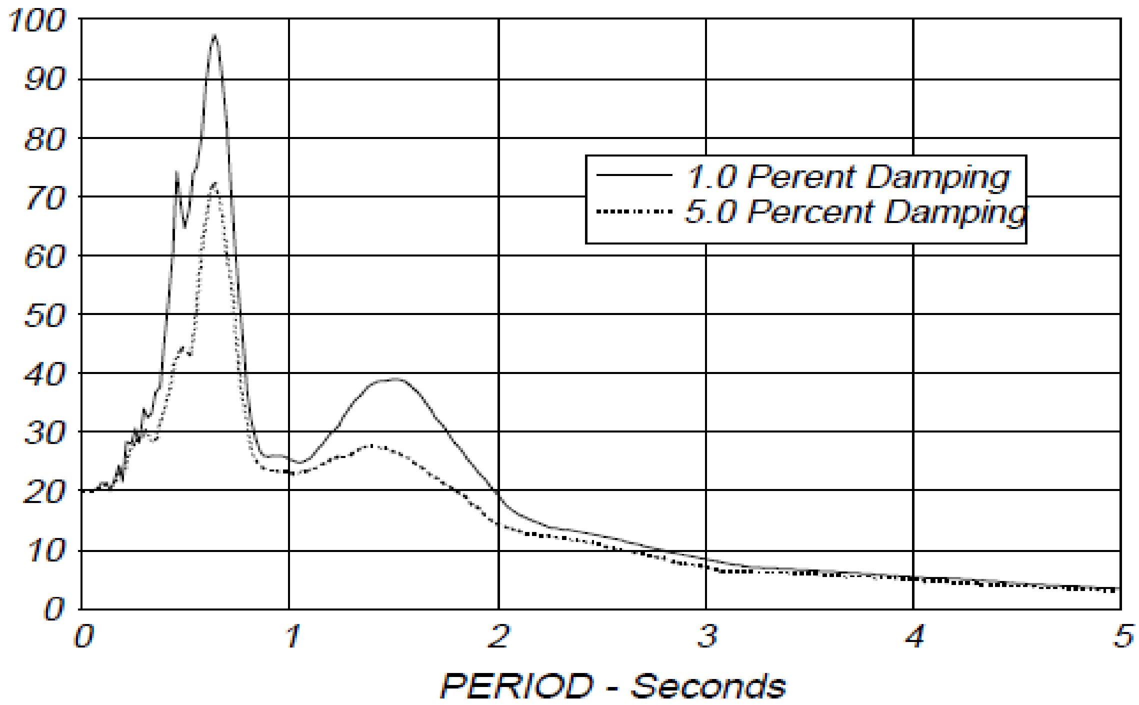


Figure 3.1b Pseudo Acceleration Spectrum, $S_a = \omega^2 y(\omega)_{MAX}$ **- Percent of Gravity**

The maximum ground acceleration, for the earthquake defined by Figure 3.1a, is
20.01 percent of gravity at 2.92 seconds. It is important to note that the pseudo acceleration spectrum, shown in Figure 3.2b, has the same value for a very short period system. This is due to the physical fact that a very rigid structure moves as a rigid body and the relative displacements within the structure are equal to zero as indicated by Figure 3.2a. Also, the behaviour of a rigid structure is not a function of the viscous damping value.

For long period systems, the mass of the one-degree-of-freedom structure does not move significantly and has approximately zero absolute displacement.
Therefore, the relative displacement spectrum curves, shown in Figure 15.2a, will converge to 11.62 inches for long periods and all values of damping. This type of real physical behaviour is fundamental to the design of base isolated structures.

The relative displacement spectrum, Figure 3.2a, and the absolute acceleration spectrum, Figure 3.2b, have physical significance. However, the maximum relative displacement is directly proportional to the maximum forces developed in the structure. Figure 3.2b, the absolute acceleration spectrum, indicates maximum values at a period of 0.64 seconds for both values of damping. Also, the multiplication by

ω 2 tends to completely eliminate the information contained in the long period range.
Since most structural failures, during recent earthquakes, have been associated with soft sites, perhaps we should consider using the relative displacement spectrum as the fundamental form for selecting a design earthquake. The high frequency, short period, part of the curve should always be defined by

$$y(\omega)_{MAX} = \ddot{u}_{g\,MAX} / \omega^2 \qquad \text{or} \qquad y(T)_{MAX} = \ddot{u}_{g\,MAX} \frac{T^2}{4\pi^2} \tag{3.8}$$

Where $\ddot{u}_{g\,MAX}$ is the peak ground acceleration.

Chapter 4

STRUCTURE MODELLING & ANALYSIS

4.1 General

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

4.2 Details of Model

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

1. Twelve storey (G+11) building frame with equal floor level.
2. Eight storey (G+7) building frame with equal floor level.

Dynamic responses of buildings are studied. Regular model consist of symmetry in plan as well as elevation. The SRSS method is employed to get dynamic responses for 5% damping for two models.

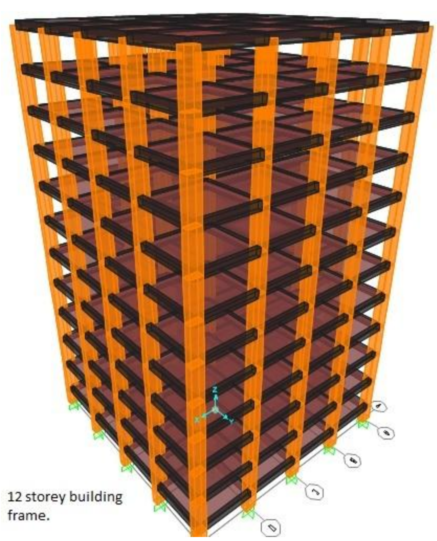


Fig. 4.1(a): 12 storey frame in SAP2000

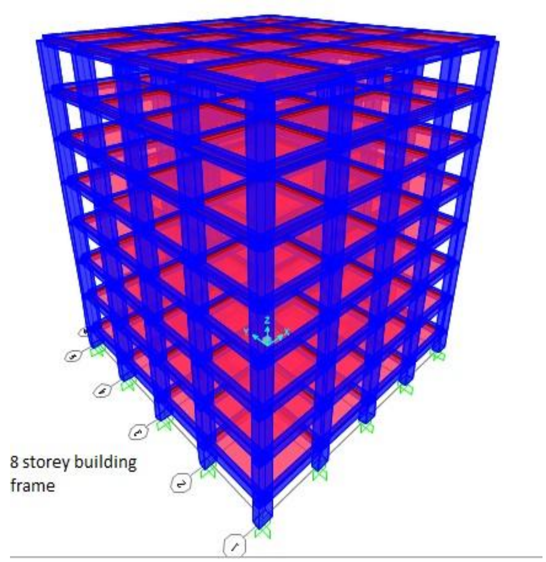


Fig. 4.1(b): 8 storey frame in SAP2000

4.3 Defining the material properties, structural components and modelling the structure:

Beam, column and slab specification are as follows:

Column	550mm x 1000mm
Beam	350mm x 600mm
Slab thickness	125 mm

<u>Reinforcement</u>

Column	8-25mm bars
Beam	4-20mm bars at both top and bottom

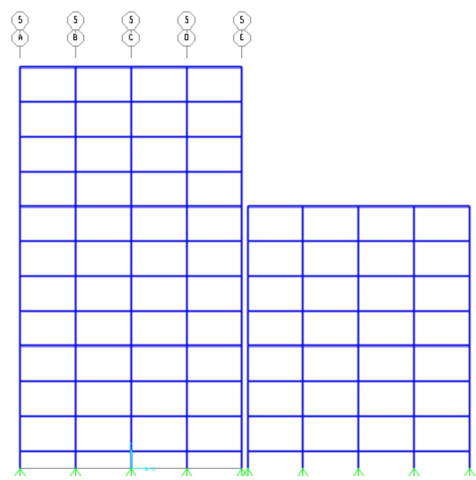


Fig. 4.2: Frame of buildings in SAP

The required material properties like mass, weight density, modulus of elasticity, shear modulus and design values of 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 appropriate dimensions and reinforcements.

Soil Structure interaction has not been considered and the columns have been restrained in all six degree 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.

4.4 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.

4.5 Analysis of the structure

Namely two types of analysis procedure have been carried out for studying seismic pounding effect. Here we are mainly concerned with the behaviour of the structure under the effect of ground motion and dynamic excitations such as earthquakes and the displacement of the structure inelastic range.

The analyses carried out are as follow:-

1. Response Spectrum Analysis.
2. Time History Analysis.

4.5.1 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. 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 two models are done in the zone IV where

Z = 0.24 considering zone factor IV

I = 1 for residential buildings.

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 (S_a/g) values are calculated as follows. For medium soil sites,

$S_a/g = 1 + 15T$, $(0.00<T<0.10)$,

(T= time period in seconds)

$= 2.50$, $(0.10<T<0.55)$

$= 1.36/T$, $(0.55<T<4.00)$

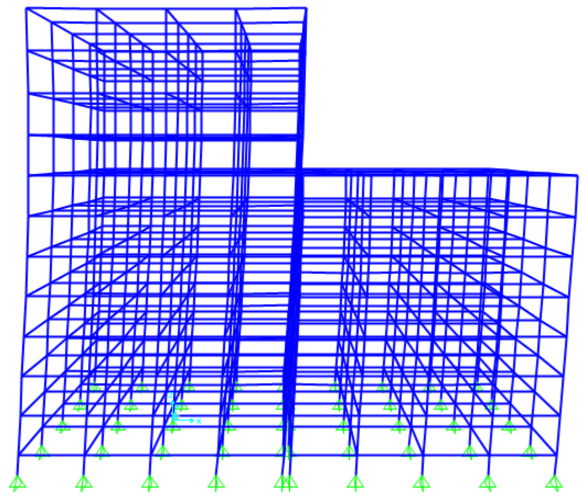


Fig 4.3: Deformed Shape in Response Spectrum Loading.

4.5.1.1 Response Spectrum Analysis in SAP 2000

The step by step procedure is as follows:-

1. Defining quake loads under the load type 'quake' and naming it appropriately.
2. Defining response spectrum function as per IS 1893 (Part 1) 2002.
3. Modifying the quake analysis case with the appropriate analysis case type, applied loads and scale factors.
4. Running the analysis.

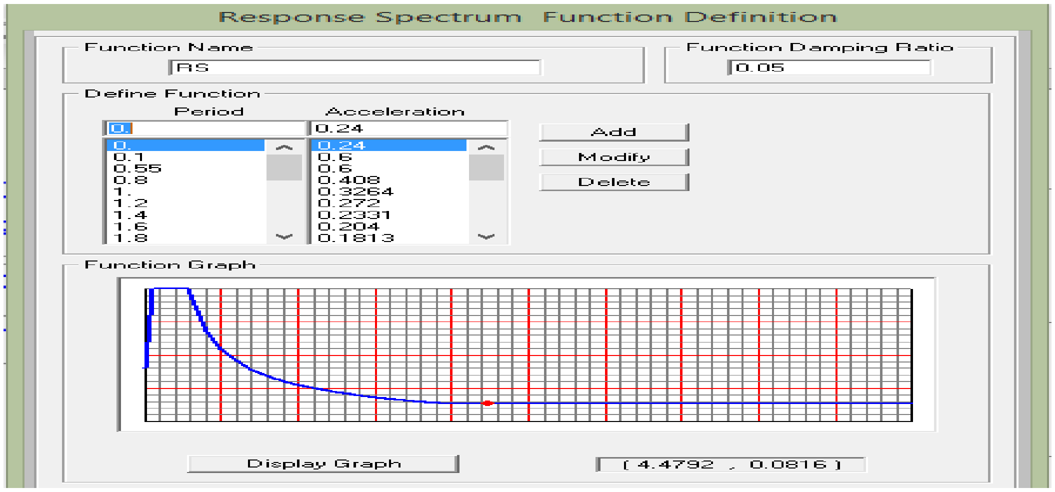


Fig. 4.4(a): Graph of Spectural Acceleration (S_a/g) Vs Periods(s) for various types of soil condition.

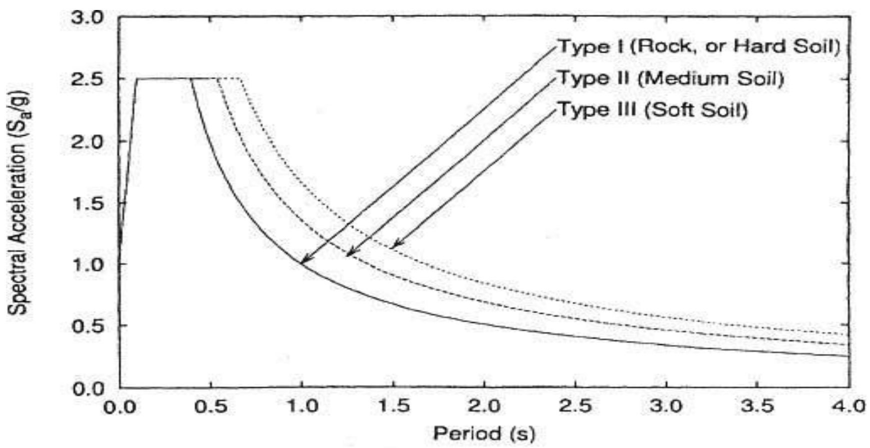


Fig. 4.4(b): Defining Soil Condition as per IS 1893:2002

4.5.2 Time History Analysis in SAP2000

Time history analysis has been carried out using the Imperial Valley Earthquake record of May 18, 1940 also known as the El Centro earthquake for obtaining the floor response.The record has 1559 data points with a sampling period of 0.02 seconds.
Figure below show north-south component of El Centro eartquake data.

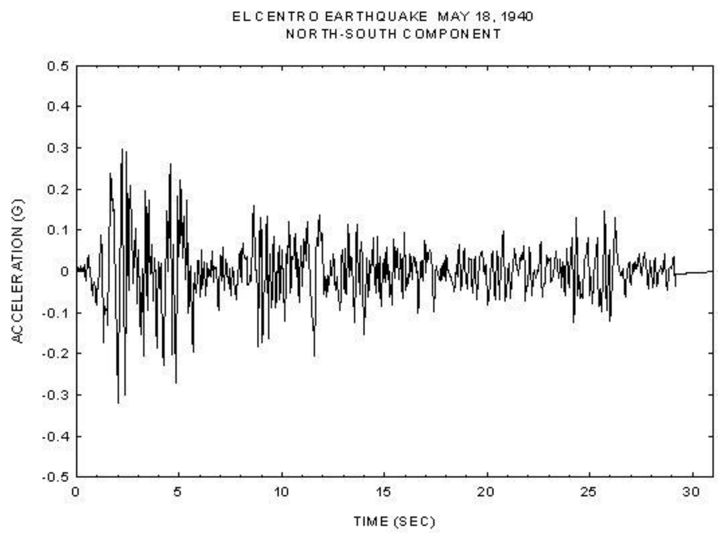


Fig: 4.5 Time history plot of El Centro earthquake.

4.5.2.1 Time History Procedure

The step by step procedure is as follow:-

1. Defining a time history function by adding a function from file. In our case,the El Centro earthquake record of 1940 has been linked to the program.
2. Defining a separate analysis case under the load type 'quake' with the appropriate analysis case type i.e. linear direct integration time history.
3. Applying earthquake acceleration values from the defined time history function.
4. 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 coefficient.
5. Running the analysis.

Chapter 5

Shear Wall and Bracing

5.1 SHEAR WALL

Shear walls are vertical elements of the horizontal force resisting system. Shear walls are -constructed to counter the effects of lateral load acting on a structure. In residential construction, shear walls are straight external walls that typically form a box which provides all of the lateral support for the building. When shear walls are designed and constructed properly, and they will have the strength and stiffness to resist the horizontal forces.

Lateral forces caused by earthquake, wind and, in addition to the weight of structure and occupants; create powerful twisting (torsion) forces. These forces can literally tear (shear) a building apart. Reinforcing a frame by attaching or placing a rigid wall inside it maintains the shape of the frame and prevents rotation at the joints. As part of an earthquake resistant building design; these walls are placed in building plans reducing lateral displacements under earthquake loads.

Shear wall buildings are usually regular in plan and in elevation. However, in some buildings, lower floors are used for commercial purposes and the buildings are characterized with larger plan dimensions at those floors. In other cases, there are set backs at higher floor levels. Shear wall buildings are commonly used for residential purposes and can house from 100 to 150 inhabitants per building.

5.1.1 PURPOSE OF CONSTRUCTING SHEAR WALLS

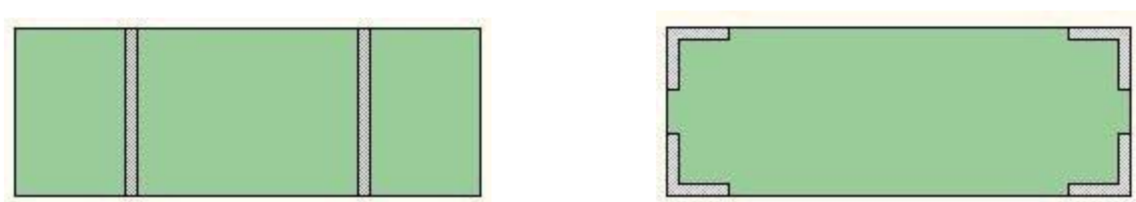
The walls are structurally integrated with roofs / floors (diaphragms) and other lateral walls running across at right angles, thereby giving the three dimensional stability for the building structures.

Shear wall structural systems are more stable. Because, their supporting area (total cross-sectional area of all shear walls) with reference to total plans area of building, is comparatively more, unlike in the case of RCC framed structures.
Earthquake forces produce large displacement, vibration and large stresses in building which leads to building an unsafe and causing discomfort to the occupants.Shear walls are easy to construct, because reinforcement detailing of walls is relatively straight-forward and therefore easily implemented at site.

5.1.2 SHEAR WALL- APPLICATION ON FRAMES.

A simplified analytical model is proposed for modeling the nonlinear response of flexural-yielding reinforced concrete walls using standard structural analysis software. The program SAP2000 is used to implement the proposed model for evaluating structural response by means of nonlinear response history analysis.Shear walled frame building is chosen for study purpose because shear wall is an efficient way of stiffening the structure. Ground motion enters the building and creates inertial forces which move the floor diaphragms. This movement is resisted by the shear walls.

Shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings. Structurally, the best position for the shear walls is in the centre of each half of the building. This is rarely practical, however, since it dictates the utilization of the space, so they are positioned at the ends. This shape and position of the walls give good flexural stiffness in the short direction, but relies on the stiffness of the frame in the other direction.



(a) (b)

Fig. 5.1: Shear wall arrangement in building frame

The above two shear walls are applied in the building frames using SAP2000.

5.1.3 SHEAR WALL REPRESENTATION IN SAP2000

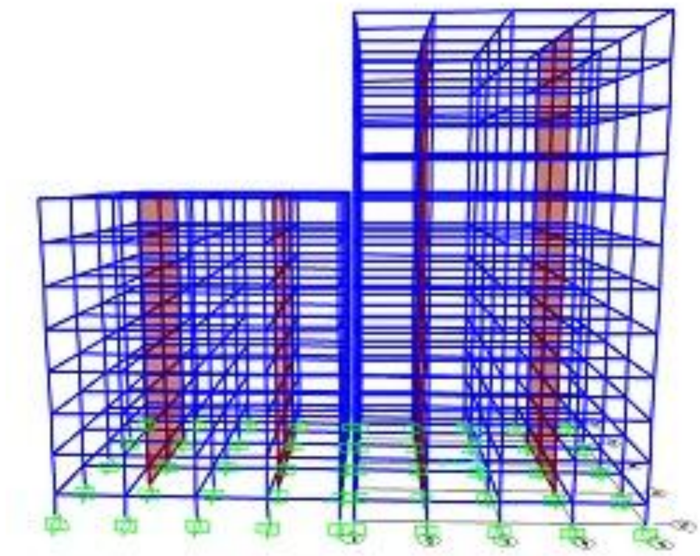


Fig. 5.2(a): Shear combination 1(interior)

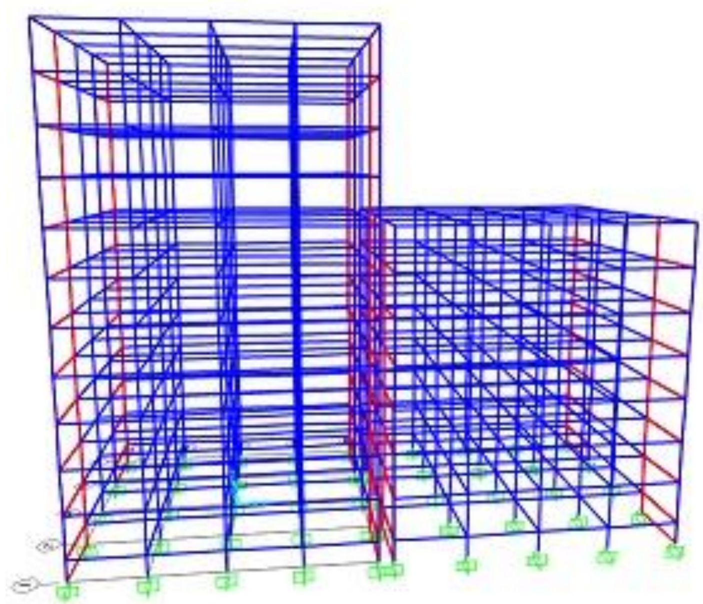


Fig. 5.2(b): Shear combination 2 (exterior)

5.2 BRACED FRAMES (VERTICAL BRACING)

The slenderness of a high-rise building makes the structure extremely sensitive to lateral loadings from wind and earthquake. Thus high-rise buildings need to be carefully designed in order that a stiff, stable and light-weight structural form is achieved. A bracing system is able to efficiently support the structure under complex loading conditions.

Bracing is another way to take care of horizontal loads. The simplest method is to place a diagonal brace, nodes are designed as leads. The transfer of horizontal loads down to one of the supports takes place in the braces direction in the form of either axial tension or compression depending on the direction of the horizontal load. This means that the axial stiffness of the frame members is what is resisting lateral loads. When subjected to a horizontal load, in an X-brace, one of the diagonals will be subjected to compression while the other is in tension

There are many different types of bracing. While the most common and one of the most effective is the X-bracing, this takes a lot of space in the structure which makes little room left for openings. There are also eccentrically braced systems that provides different shapes and openings, they have good ductility for resisting seismic forces but provide less stiffness than the concentric braced frame.

While using a completely braced frame system in a high-rise building, the stability is very good. This system has a few major drawbacks however. The weight of the building when completely braced becomes massive, with a lot of different pieces to fit together. Another drawback is the limitations in terms of space for windows and doors, the bracing means the openings have to fit in accordingly which also means that the ability to form an architectural expression would be less.

5.2.1 Bracing in SAP2000

The two frames model has their exterior columns braced(X-bracing). The bracing is applied along the height of the frames. The elements defined in bracing have same properties of beam members, in both the frames. The figure below shows frame models in SAP2000.

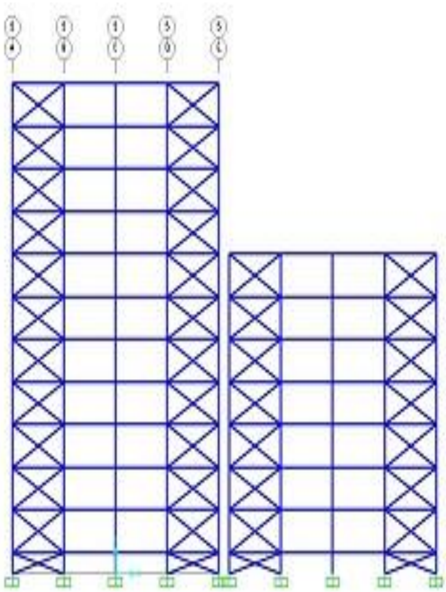


Fig. 5.3(a): x-y plane view.

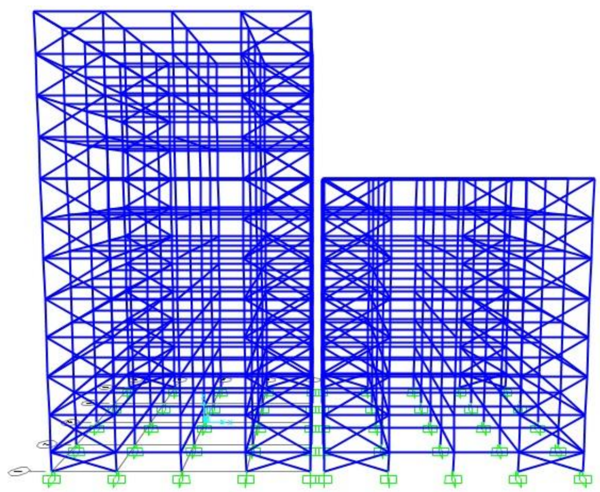


Fig. 5.3(b): 3-D view of braced frames.

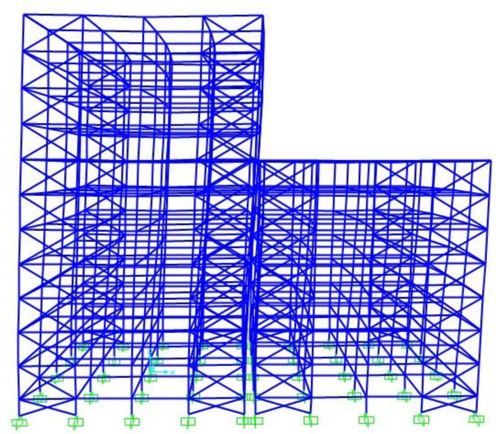


Fig. 5.3(c): Lateral sway of frames under seismic action.

Chapter 6

RESULTS AND DISCUSSION

6.1 RESULT

Results from Response Spectrum analysis & time history analysis have been used to observe and compare the floor responses of the two models for the natural frequencies and modal mass participation ratios and Displacements of the joints to determine whether the seismic pounding is present. The results obtained from shear wall and braced columns are compared with bare frames in the table 6.5.

6.1.1 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. The table shows total storey drifts without considering the Response reduction factor(R).

Table 6.1: Table showing displacement at different floor joints for two models

BUILDINGS	JOINT (pt.obj.)	DISPLACEMENT (U1)
1.Twelve Storey (G+11)	273	0.0846 m
2.Eight Storey (G+7)	334	0.0512 m
		0.1358 m

The joints mentioned in the table (273,334) are the top end points of the frames.
Figure 6.1, 6.2 are the response spectrum curve of the two models- explaining the displacement of the joints of the respective frames, with respect to time.

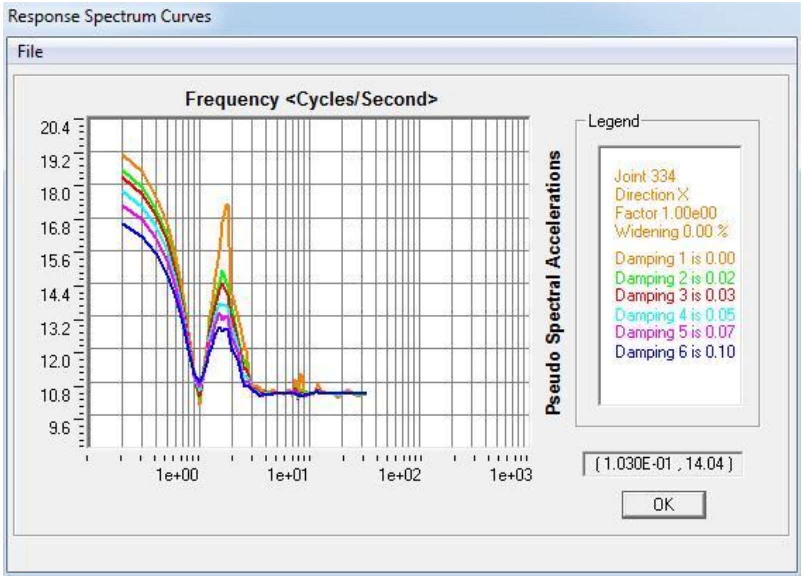


Fig. 6.1: Response Spectrum Curve for twelve storey (G+11) frame.

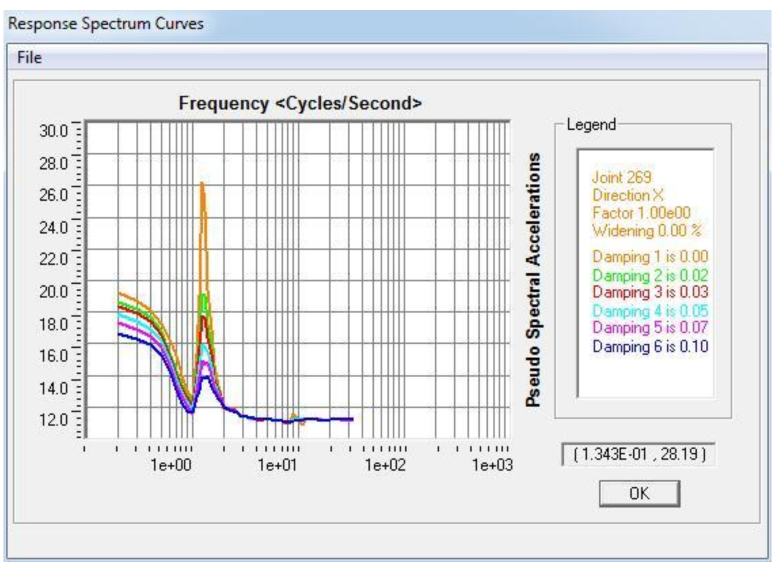


Fig. 6.2: Response Spectrum Curve for eight storey (G+7) frame.

6.1.2 Time History Analysis

Time history analysis has been carried out using the Imperial Valley Earthquake record of May 18, 1940 also known as the El Centro earthquake for obtaining the various floor responses. The table below shows the combined lateral displacement of 0.3238 meters without considering the response reduction factor (R).

Table 6.2: Table showing displacement at different floor joints for two models

BUILDINGS	JOINT (pt.obj.)	DISPLACEMENT (U1)
1.Twelve Storey (G+11)	273	0.2346 m
2.Eight Storey (G+7)	334	0.0892 m
		0.3238 m

Figure 6.3 below shows plot of time history comparison of twelve (G+11) and eight (G+7) storey frame.

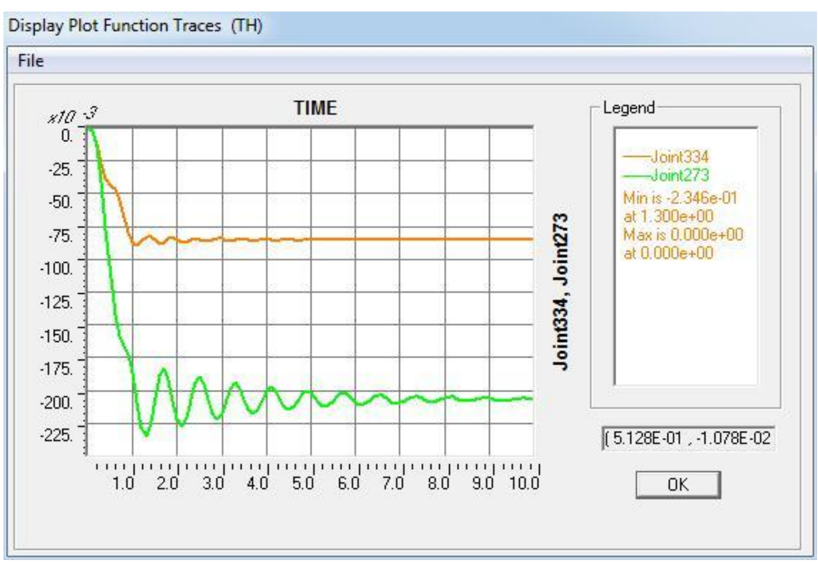


Fig. 6.3: Plot Function of two building frame at top surface.

6.1.3 Shear wall Analysis

Response Spectrum analysis is applied in frames to compare the value of lateral sway. Table 6.3 shows the lateral displacement of two frames when two combinations of shear walls are compared. Both the combination reduced the lateral sway and avoided pounding.

Table 6.3: Shear Wall Combinations

Shear Wall Combination	Combination 1	Combination 2
Lateral Sway(m)	0.0266	0.1029

6.1.4 Bracing Analysis

The displacement obtained by bracing. Table 6.4 shows the lateral displacement of two frames when bracing is applied.

Table 6.4: Table showing lateral displacement of two frames when bracing is applied.

BUILDINGS	JOINT (pt.obj.)	DISPLACEMENT (U1)
1.Twelve Storey (G+11)	273	0.0399 m
2.Eight Storey (G+7)	334	0.0107 m
		0.041 m

The joints (273,334) are the top nearest points of the two building frames. On application of bracing the lateral displacement is 0.041m, hence it avoided the seismic pounding to occur.

6.2 DISCUSSION

Clause no. 7.11.3 of IS 1893-2002 states that two adjacent buildings, or two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount R times the sum of the calculated storey displacement, to avoid damaging contact when two units deflect toward each other.
As per table 7, Response Reduction Factor, R for building systems. The value of R is 3.0 for Ordinary RC moment-resisting frame (OMRF).

6.2.1 Response Spectrum

Multiplying R with displacement, the minimum seismic gap shall be provided is 3×0.1358=0.4084 m. The gap provided between two buildings was 0.4 meters. The seismic gap calculated according to IS 1893-2002 shall be 0.4084 meters. Hence, seismic pounding will occur between two building frames.

6.2.2 Time History Analysis

Multiplying R with displacement, the minimum seismic gap shall be provided is 3×0.3238=0.9714 m. The gap provided between two buildings was 0.4 meters. The seismic gap calculated according to IS 1893-2002 shall be 0.9714 meters, which is much more than the gap provided. Hence, seismic pounding will occur between two building frames.

6.2.3 Comparative Study of Bare Frames, Shear Wall & Bracing

Table 6.5: Comparative Study of Bare frames, Shear Wall & Bracing in respect of lateral sway.

FRAME TYPE	BARE FRAME(Time History)	SHEAR WALL	BRACING
LATERAL SWAY(m.)	0.9714	0.0266	0.041

The above table compares the lateral displacement of three different models. Time History reading is chosen as it gave more accurate value. The lateral displacement in bare frame is more, but decreases when lateral load resistant systems are applied in the system. This explains the importance of the lateral load resistant system, as shear wall is more suitable than bracing, since it reduced the displacement to a small value, avoiding Seismic Pounding in the models.

Chapter 7

CONCLUSIONS

CONCLUSIONS

Based on the observations from the analysis results, the following conclusions can be drawn:

1. The purpose of this study was to analyze seismic pounding effects between buildings, taking different configuration of the structures. Linear Static analysis of the structure provided the value of base shear during earthquake, along the height of the structure.
2. The gap provided between the two bare frames was 0.4 meters, under dynamic analysis the frames pounded with each other.
3. Lateral Load resistant structures helped in reducing the building sway, hence reducing the risk of pounding.
4. The shear walls are one of the most effective building elements in resisting lateral forces during earthquake. Similarly bracing of the structures have kept them safe during strong earthquake shocks, high speed winds.
5. The steel shear wall system has relatively high initial stiffness, thus very effective in limiting the drift.

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