Research Article | Impact Factor: 0.621 | ISSN: 2319-507X
Rahul Phuke, IJPRET, 2014; Volume 2 (9): 181-194 | IJPRET

INTERNATIONAL JOURNAL OF PURE AND APPLIED RESEARCH IN ENGINEERING AND TECHNOLOGY

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DESIGN OF RC BUILDINGS AGAINST SEISMIC POUNDING

PROF. RAHUL M. PHUKE[1], MONALI G.INGLE[2]

1. Asso. Prof., College of engineering and technology, Akola.
2. 2- M.E (Struc), II nd Yr, COET, Akola.

Accepted Date: 27/02/2014 ; Published Date: 01/05/2014

Abstract: Pounding of neighbouring construction of structures due to seismic excitation increases the damage of structural components oreven causes collapse of structures. Among the possible building damages, earthquake induced pounding has been commonly observed in several earthquakes. Therefore it is imperative to consider pounding effect for structures. This study aims to understand the response behavior of adjacent buildings with dissimilar heights under earthquake induced pounding. Investigations of past and recent earthquake damage have illustrated in the literature review that the building structures are vulnerable to severe damage and/or collapse during moderate to strong ground motion. It is concluded that pounding forces are not completely absorbable because of their high values but their effects on structure can be reduced by placing elastic materials between adjacent buildings or by reinforcing structural systems with cast-in-place reinforced concrete (RC) walls.

Keywords: Seismic pounding; Adjacent building; Energy dissipation; Seismic design

Corresponding Author: PROF. RAHUL M. PHUKE

PAPER-QR CODE

Access Online On:

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How to Cite This Article:

Rahul Phuke, IJPRET, 2014; Volume 2 (9): 181-194

INTRODUCTION

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.

I.1 Seismic Pounding Effect between Buildings

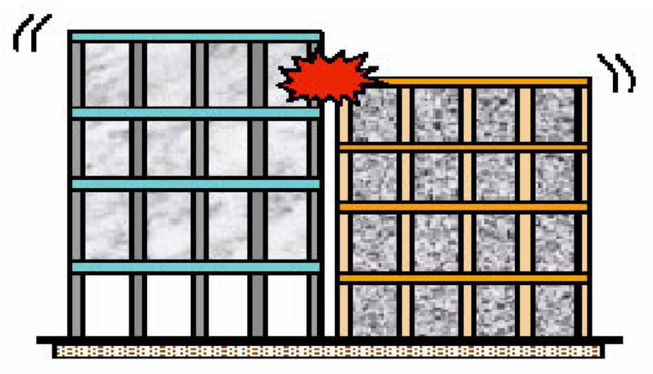
Pounding is one of the main causes of severe building damages in earthquake. The non-structural damage involves pounding or movement across separation joints between adjacent structures. Seismic pounding between two adjacent buildings occur:

· during an earthquake

· different dynamic characteristics

· adjacent buildings vibrate out of phase

· at-rest separation is insufficient



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.This experiment involved testing of a steel building scheduled for demolition in Northbrook, Illinois. The demolition team tore out four selected columns from the building to simulate the sudden column removal that leads to progressive collapse. The structure was instrumented with strain gauges that recorded the change in strain in various structural members while the columns were removed. The author instrumented the beams and columns in the building, managed the testing, and analyzed the recorded data.

I.2 Required Seismic Separation Distance To Avoid Pounding

The 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 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 αh =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

NOTE — Minimum total gap shall be 25 mm. For any other value of αh the gap width shall be determined proportionately.

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:

$S = U_a + U_b$ ABS (1)

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

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

I.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:

$$\mathbf{F(t)_i + F(t)_d + F(t)_s = F(t)}$$

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

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:

$$\mathbf{M\,\ddot{u}(t)_a + C\,\dot{u}(t)_a + K\,u(t)_a = F(t)}$$

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 $\mathbf{\dot{u}(t)_a}$, $\mathbf{\dot{u}(t)_a}$ and $\mathbf{\ddot{u}(t)a}$ are the absolute node displacements, velocities and accelerations, respectively.

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. 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.

I.3.1.2 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.

I.3.2 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 consideredre presentative 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**:**

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.

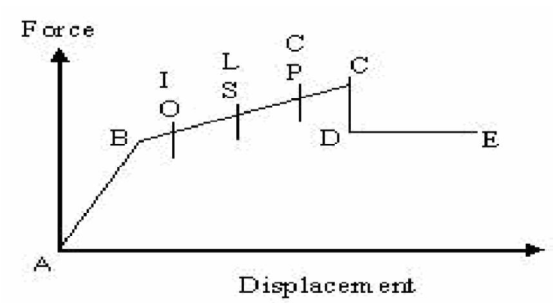


Fig 1.3 Different stages of plastic hinge

Capacity-spectrum

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

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.

Demand -spectrum

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

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.

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.

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

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: minimize dust emissions. Burning of waste shall not be allowed.

Diesel Capacity spectrum

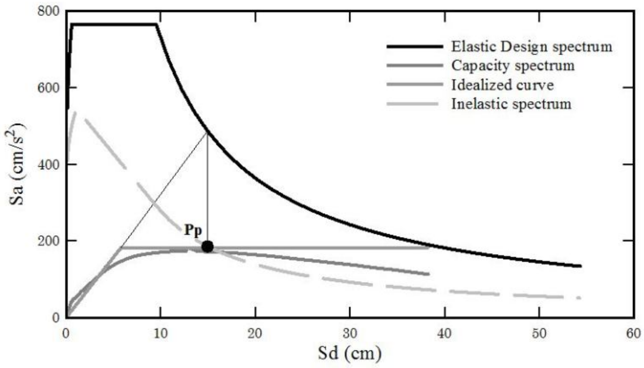


Fig : Determination of performance point

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.

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.

II STRUCTURAL MODELING AND ANALYSIS

The finite element analysis software SAP2000 Nonlinear I s 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. 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.

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.

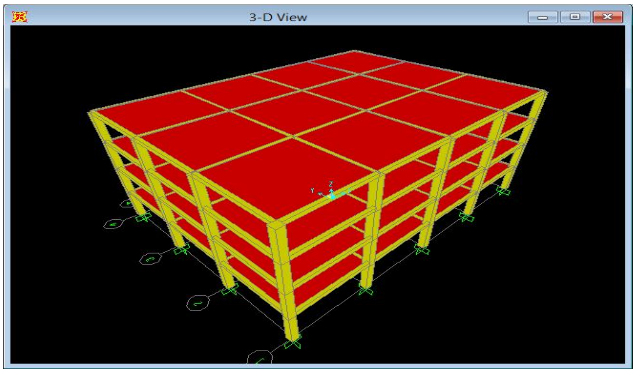


Fig 3-D view of the four storey (G+4) building

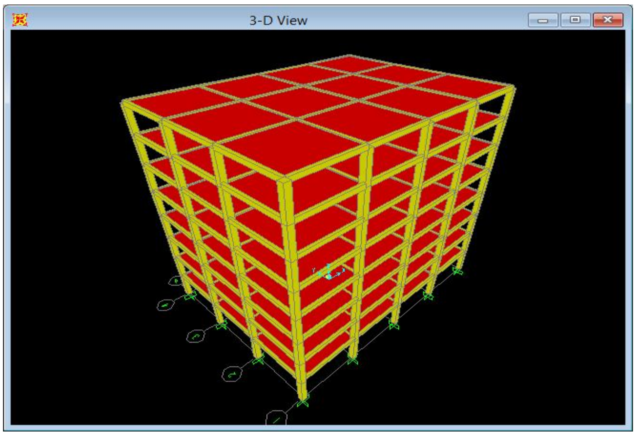


Fig :3-D view of the eight storey building (G+8)

Assigning loads.

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

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.

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) DL- Dead Load

1.2(DL+LL+EL) LL- Live load

1.2(DL+LL-EL) EL- Earthquake load.

1.5(DL+EL)

1.5(DL-EL)

0.9DL+1.5EL

0.9DL-1.5EL

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.

Analysis of the structure

The analyses carried out are as follows:

Response Spectrum Analysis

Time History Analysis.

Pushover analysis.

III RESULTS AND DISCUSSION

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. The displacements for a particular joint at the top floor for two models have been tabulated as below

Analysis of Four storey buildings (G+4)

Load combinations	Displacements in mm		
	Longer (x)	Shorter (y)	Vertical (z)
1.5(DL+LL)	-4.316*10-18	-4.916*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

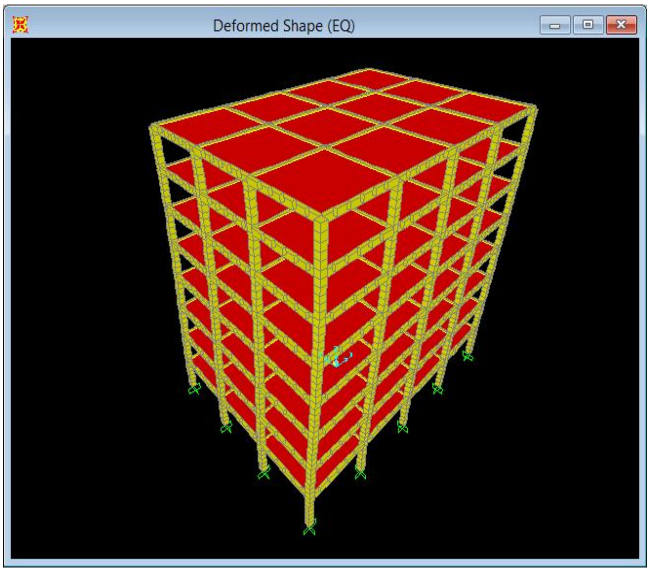
0.9DL-1.5EL	0.023	0.023	-0.0001

Displacement at the top floor in m for four storey buildings

Analysis of Eight storey buildings (G+8)

Load combinations	**Displacements in mm**		
	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

Displacement at the top floor in m for eight storey buildings



Deformed shape o adjacent buildings due to less seismic gap

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.

CONCLUSION

The purpose of this study has been to analyze seismic pounding effects between buildings and to observe the structural behavior 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. 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.

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