Effect of Seismic Pounding on Impact Forces Between Adjacent Buildings

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Abstract*- In our country, after every earthquake, we come across the reality that the earthquake doesn't kill people; it is due to unsafe buildings which causes damage to property and life. In India, about 60% area is susceptible to damaging levels of seismic hazards. We can't avoid the future earthquakes but the preparedness and safe building construction practices for earthquakes can certainly reduce the extent of damage and loss of both property and life. Performance based design of structure is the need in present time.*

In our present work, an attempt is made to study seismic behavior of buildings without floating columns. To study various static and dynamic properties due presence of without floating columns has been studied. Dynamic analysis is carried out by response spectrum method time history method to find dynamic parameters like impact forces. Non–linear analysis is carried out by push-over analysis approach. And presence of hinges has been carried for performance based design of the structure. The analysis is carried out by using SAP2000 Software.

Keywords- Seismic Pounding, Impact forces, SAP: 2000

I. INTRODUCTION

An earthquake is the shaking of the surface of the earth, resulting from the sudden release of energy in the earth's lithosphere that creates seismic waves. Earthquakes can range in size from those that are so weak that they cannot be felt to those violent enough to toss people around and destroy whole cities. The seismicity or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time. At the earth's surface, earthquakes manifest themselves by shaking and sometimes displacement of the ground. When the epicenter of a large earthquake is located offshore, the seabed may be displaced sufficiently to cause a tsunami. Earthquakes can also trigger landslides, and occasionally volcanic activity. Earthquakes are caused mostly by rupture of geological faults, but also by other events such as volcanic activity, landslides, mine blasts, and nuclear tests. An earthquake's point of initial rupture is called its focus or hypocenter. The epicenter is the point at ground level directly above the hypocenter.

Amongst the natural hazards, earthquakes have the potential for causing the greatest damages. Since earthquake forces are random in nature & unpredictable, the engineering tools needs to be sharpened for analyzing structures under the action of these forces. Adjacent buildings with insufficient separation, having different dynamic characteristics may vibrate out of phase during earthquakes causing pounding between them. The pounding of structures may lead to severe damage and even result in complete collapse.

Interactions between neighboring, inadequately separated buildings or bridge segments have been repeatedly observed during earthquakes. This phenomenon, often referred to as earthquake-induced structural pounding, may result in substantial damage or even total destruction of colliding structures during severe ground motions. Structural pounding is a complex phenomenon involving plastic deformations at contact points, local cracking or crushing, fracturing due to impact, friction, etc. Forces created by collisions are applied and removed during a short interval of time initiating stress waves, which travel away from the region of contact. The process of energy transfer during impact is highly complicated, which makes the mathematical analysis of this type of problem very difficult.

Performance based design is gaining a new dimension in the seismic design philosophy wherein the near field ground motion (usually acceleration) is to be considered. Earthquake loads are to be carefully modelled so as to assess the real behavior of structure with a clear understanding that damage is expected but it should be regulated. In this context pushover analysis which is an iterative procedure shall be looked upon as an alternative for the orthodox analysis procedures. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. To turn this promise into a reality, a comprehensive and well-coordinated effort by professionals from several disciplines is required. Performance based engineering is not new. Automobiles, airplanes, and turbines have been designed and manufactured using this approach for many decades. Generally, in such applications one or more full-scale prototypes of the structure are built and subjected to extensive testing. The design and manufacturing process is then revised to incorporate the lessons learned from the experimental evaluations. Once the cycle of design, prototype manufacturing, testing and redesign is successfully completed, the product is manufactured in a massive scale.

What makes performance-based seismic engineering (PBSE) different and more complicated is that in general this massive payoff of performance-based design is not available. That is, except for large-scale developments of identical buildings, each building designed by this process is virtually unique and the experience obtained is not directly transferable to buildings of other types, sizes, and performance objectives. Therefore, up to now PBSE has not been an economically feasible alternative to conventional prescriptive code design practices. Due to the recent advances in seismic hazard assessment, PBSE methodologies, experimental facilities, and computer applications, PBSE has become increasing more attractive to developers and engineers of buildings in seismic regions. It is safe to say that within just a few years PBSE will become the standard method for design and delivery of earthquake resistant structures. In order to utilize PBSE effectively and intelligently, one needs to be aware of the uncertainties involved in both structural performance and seismic hazard estimations. The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are identified. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

The concept of performance based seismic design approach has become the future direction for seismic design codes. In this approach, nonlinear analysis procedures become important in determining the patterns and extent of damage to assess the structure’s inelastic behavior and failure pattern in severe seismic events. Static pushover analysis is a simplified nonlinear procedure wherein the pattern of earthquake is applied incrementally to the structural frame until a plastic collapse mechanism is formed. The methodology adopts the lumped plasticity approach, identifying the extent of inelasticity through the formation of nonlinear plastic hinges assigned at the ends of the frame elements while the incremental loading is applied. In other words, determination of desired structural response that satisfies both global level (i.e. system level) and local level (i.e. element level) response is needed. This is possible with trial and error approach, with available aid of standard engineering software. It has been recognized that, for the performance assessment of structure, the important parameters that needs to be evaluated are: Vertical load carrying capacity of structure, Lateral strength, Inter storey drift ratios (IDRs), and Ductility demands in terms of inelastic displacement demand ratio (IDDR).

3. METHODOLOGY

Sr. No	METHODOLOGY
3.1	Building Considered for the Analytical Study.
3.2	Modelling of the Buildings.
3.3	Loads Acting on Buildings.
3.4	Seismic Analysis of Buildings Using IS 1893(Part 1) – 2016.
3.5	Calculation of Lateral Forces.
3.6	Analysis of data for all models
3.7	Comparison based on various parameters.
3.8	Results and Conclusion

3.1 Building Considered for the Analytical Study The present work of seismic analysis is carried out for reinforced concrete moment resisting building frame having same and different area situated in zone II for varying vertical asymmetry. The analysis is carried out using SAP2000.

The buildings considered in this study are buildings without Floating Columns.

3.2 Modelling of the Buildings

The building is modelled using the finite element based software SAP2000. The analytical models of the building include all components that influence the mass, strength, stiffness and deformability of structure. The building structural system consists of beams, columns, slab, walls, and foundation. The non-structural elements that do not significantly influence the building behavior are not modelled.

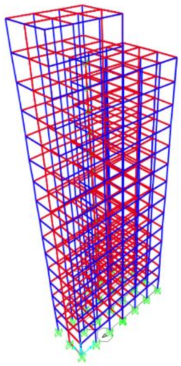


Figure 3.1:- Model in SAP 2000

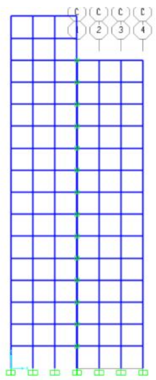


Figure 3.2:- Cross section of model

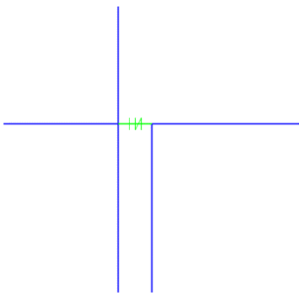


Figure 3.3:- Seismic gap

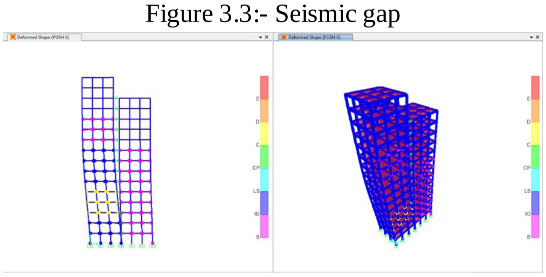


Figure 3.4:- Extrude view

3.3 Loads Acting on Buildings

Loads acting on buildings are mainly gravity loads and lateral loads.

3.3.1 Gravity Loads

Gravity loads include self-weight of building, floor finish which is taken as 1.66 kN/m^2 and live load which is taken as 2 kN/m^2 as per IS 875(part-II) for a bay frames that would be acting on the structure in its working period. We have also considered wall load as imposed load on beams as 7.26 kN/m. and on slab as 7.46kN/m

3.3.2 Lateral Loads

In contrast to the vertical load, the lateral load effects on buildings are quite variable and increases rapidly with increase in height. Most lateral loads are live loads whose main component is horizontal force acting on the structure. Typical lateral loads would be a wind load, an earthquake load, and an earth pressure against a beachfront retaining wall. Most lateral loads vary in intensity depending on the buildings, geographic location, structural material, height and shape.

3.3.2.1 Earthquake Load

Earthquake loading is a result of the dynamic response of the structure to the shaking of the ground. Earthquake loads are another lateral live load. They are very complex, uncertain and potentially more damaging than wind loads. It is quite fortunate that they do not occur frequently. The earthquake creates ground movements that can be categorized as a “shake”, “rattle” and “roll”. Every structure in an earthquake zone must be able to withstand all three of these loadings of different intensities. Although the ground under a structure may shift in any direction, only the horizontal components of this movement are usually considered critical in analysis.

The magnitude of horizontal inertia forces induced by earthquakes depends upon the mass of structure, stiffness of the structural system and ground acceleration. In our presented study the magnitude of wind load is negligible as compared to the seismic load hence the value of wind load is neglected while only seismic load is considered.

The structural system of a building consists of two components, one is horizontal framing system (beam and slab) and other is vertical framing system (walls and columns). Horizontal framing system is primarily responsible for transfer of vertical loads and tensional forces to vertical framing systems that is responsible for transferring the vertical loads and lateral forces to the footing.

Lateral Load Resisting Systems

Gravity loads are the primary loading on a building. As a building becomes taller, it must have adequate strength and stiffness to resist lateral loads imposed by wind and earthquake. As the height of building increases, the additional stiffness is required to control the deflection. A tall building essentially comprises several vertical cantilevers tied together by the floor slabs. Under horizontal loading, each cantilever bends about its own axis, but deforms with other cantilevers owing to the in-plane rigidity of the floor slabs. The various types of vertical cantilevers used in building are rigid frame, braced frame, wall. These individually or in combination form the structural system which resists the lateral loads in a building. The structural systems used in tall buildings are:

1. Rigid frame
2. Braced frame
3. Shear wall.

3.4 Seismic Analysis of Buildings Using IS 1893(Part 1) – 2016

Currently there are two methods by which the magnitude and the distribution of the earthquake induced lateral forces are estimated on the structures. These methods of analysis enables the designer to estimate design forces due to earthquake in multi-storied buildings.

A) Equivalent static method of analysis.
B) Dynamic analysis

The dynamic analysis is of two types:

a) Response spectrum method.
b) Time history analysis.

Seismic codes are unique to a particular region or country. In India, IS 1893 is the main code that provides outline for calculating seismic design force. This force depends on the mass and seismic coefficient of the structure and also depends on properties like seismic zone in which structure lies, importance of the structure, its stiffness, the soil on which it rests and its ductility. Part I of IS 1893: 2016 deals with assessment of seismic loads on various structures and buildings. Whole code focuses on the calculation of base shear and its distribution over height. Depending on the height of the structure and zone in which it belongs, type of analysis i.e. static and dynamic analysis is performed.

3.5 Calculation of Lateral Forces

Lateral forces are calculated by equivalent static and dynamic analysis.

3.5.1 Equivalent Static Analysis

The total design lateral force or design base shear along any principal direction is given in terms of design horizontal seismic coefficient and seismic weight of the structure. Design horizontal seismic coefficient depends on the zone factor of the site, importance of the structure, response reduction factor of the lateral load resisting elements and the fundamental period of the structure.

Following procedure is generally used for the equivalent static analysis:

i) Calculation of lumped weight.
ii) Calculation of fundamental natural period.

The fundamental natural period of vibration (T_a) in seconds of a moment resisting frame building,

$T_a = 0.075\, h^{0.75}$ (without brick infill panel
$T_a = 0.09\, h/\sqrt{d}$(with brick infill panels)

Where

h = Height of the building
d = Base dimension of the building at the plinth level in m ,along the considered direction of the lateral force.

iii) Determination of base shear (V_B) of the building.

$$V_B = A_h \times W$$

where

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

Isthe design horizontal seismic coefficient, which depends on the seismic zone factor (Z), importance factor (I), response reduction factor (R) and the average response acceleration coefficient (S_a/g). S_a/g in turn depends on the nature of foundation soil (rock, medium or soft soil sites), natural period and the damping of the structure.

iv) Lateral distribution of design base shear

The design base shear V_B thus obtained is then distributed along the height of the building using a parabolic distribution expression:

$$Q_i = V_B \frac{W_i \ h_i^{\ 2}}{\sum_{j=1}^{n} W_j h_j^{\ 2}}$$

Where, Q_i is the design lateral force, W_i is the seismic weight, h_i is the height of the i^{th} floor measured from base and n is the number of stories in the building.

3.5.2 Dynamic Analysis of Buildings

Dynamic analysis shall be performed to obtain design seismic forces for various lateral load resisting elements under any of the following conditions:

- For regular buildings, if the height is greater than 40m in zones IV and V or greater than 90m in zone II and III
- For irregular buildings, if the height is more than 12m in zone IV and V and more than 40m in zone II and III.

Dynamic analysis can be performed either by time history method or response spectrum method.

3.5.2.1 response Spectrum Method

In response spectrum method the peak response of the structure is calculated from model combination, where the following two methods can be used.

a)Square Root of Sum of Square (SRSS) Method

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda K)2}$$

where, λk = Absolute value of quantity in mode k
r = Number of modes being considered.

b)Complete Quadratic Combination Method

$$\lambda = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{r} \lambda_i \rho_{ij} \lambda_j}$$

where, λi = Response quantity in mode
Pij = Cross modal coefficient
λj = Response quantity in mode j

ρ_{ij} = cross-modal coefficient

$$\frac{8\zeta^2(1+\beta)\beta^{1.5}}{(1+\beta^2)^2 + 4\zeta^2\beta(1+\beta)^2}$$

where, ξ = Modal damping ratio in fraction
β = Frequency ratio = ωj/ωi
ωi = Circular frequency in i^{th} mode
ωj = Circular frequency in j^{th}mode

3.5.2.2 Time History Analysis

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history. It is concerned with the calculation of structural response as a function of time when a system is subjected to a given ground acceleration.

Time-history analysis is used to determine the dynamic response of a structure to arbitrary loading. The dynamic equilibrium equations to be solved are given by,

$$Ku(t) + \mu C(t) + M^{\varepsilon}(t) = r(t)$$

where, K is the stiffness matrix, C is the proportional damping matrix, M is the diagonal mass matrix, u, µand Ɛ,are the relative displacements, velocities, and acceleration with respect to the ground, and r is the applied load. Any number of time–history analyses is performed in a single execution of the program. Each history can differ in the load applied and in the type of analysis to be performed. Three types of time-history analysis are,

- Linear transient - In this the structure starts with zero initial condition or with conditions at end of a previous linear transient history that is specified.
- Periodic transient-The initial conditions are adjusted to be equal to those at the end of the period of analysis.
- Nonlinear transient -The structure starts with zero initial conditions or with the end of a previous nonlinear transient history that is specified.

All elements are assumed to behave linearly for the duration of the analysis.

Loading:

The load, r (t), applied in a given History may be an arbitrary function of space and time. It can be written as a finite sum of spatial load vectors pi multiplied by time functions, f1 (t), as,

$$r(t) = \sum_i f_i(t) p_i$$

The program uses load Cases or acceleration loads to represent the spatial load vectors. The time functions can be arbitrary functions of time or periodic functions such as those produced by wind or earthquake loading. When acceleration loads are used, the displacements, velocities, and accelerations are all measured relative to the ground. The time functions associated with the acceleration loads m_x, m_y, and m_z are the corresponding components of uniform ground acceleration, u_{gx}, u_{gy} and u_{gz}.

The dynamic analysis of structure in time history analysis amplitude of response is plotted against the time period for each structural member that gives the exact forces in the members.

3.5.3 Non Linear Static Procedure

The Nonlinear static method involves the application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, then the loads must be applied in both the positive and negative directions. Different methods for obtaining performance point through NSP are given below.

A) Capacity Spectrum Method
B) Displacement Coefficient Method

3.5.3.1 Hinges

Hinges are points on a structure where one expects cracking and yielding to occur in relatively higher intensity so that they show high flexural (or shear) displacement. These are locations where one expects to see cross diagonal cracks in an actual building structure after a seismic motion, and they are found to be at the either ends of beams and columns, the 'cross' of the cracks being at a small distance from the joint – that is where one is expected to insert the hinges in the beams and columns of the corresponding computer analysis model. Hinges are of various types namely- flexural hinges, shear hinges and axial hinges. The first two are inserted into the ends of beams and columns. Since the presence of masonry infills have significant influence on the seismic behaviour of the structure, modelling them using equivalent diagonal struts is common in PA, unlike in the conventional analysis, where its inclusion is a rarity. The axial hinges are inserted at either ends of the diagonal struts thus modelled, to simulate cracking of infills during analysis. Basically a hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads. For example, a flexural hinge represents the moment-rotation relation of a beam of which a typical one is as represented in adjacent fig. AB represents the linear elastic range from unloaded state A to its effective yield B, followed by an inelastic but linear response of reduced stiffness from B to C. CD shows a sudden reduction in load resistance, followed by a reduced resistance from D to E, and finally a total loss of resistance from E to F. Hinges are inserted in the structural members of a framed structure typically as shown in Fig.3.5. These hinges have non-linear states defined as 'Immediate Occupancy' (IO), 'Life Safety' (LS) and 'Collapse Prevention' (CP) within its ductile range. This is usually done by dividing B-C into four parts and denoting IO, LS and CP, which are states of each individual hinges.

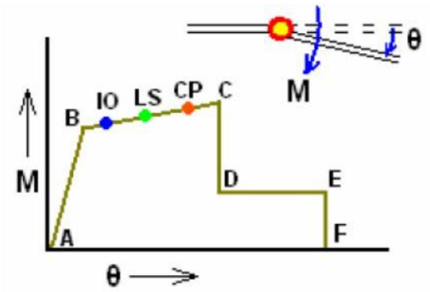


Figure 3.5: A Typical Flexural Hinge Property, showing IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention)

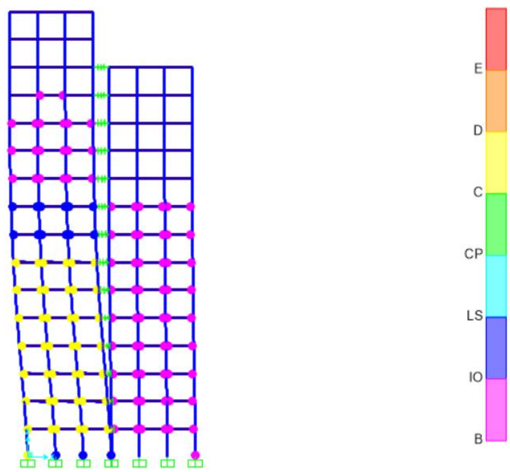


Figure 3.6: Typical Locations of Hinges in a Structural Model

3.5.3.2 The Two Stage Design Approach

Although hinge properties can be obtained from charts of average values included in FEMA356, ATC-40 and

FEMA 440, for accurate results one requires the details of reinforcement provided in order to calculate exact hinge and one has to design the structure in order to obtain the reinforcement details. This means that Pushover Analysis is meant to be a second stage analysis. Thus the emerging methodology to an accurate seismic design is: (1) first a linear seismic analysis based on which a primary structural design is done; (2) insertion of hinges determined based on the design and then (3) a pushover analysis, followed by (4) modification of the design and detailing, wherever necessary, based on the latter analysis.

In Pushover Analysis, in the global sense, it is the base shear (V_b) vs roof top displacement (Δ_{roof}top, taken as displacement of a point on the roof, located in plan at the centre of mass), plotted up to the termination of the analysis. At a local level, it is the hinge states to be examined and decided on the need for its redesign or a retrofit. PA can be useful under two situations: When an existing structure has deficiencies in seismic resisting capacity (due to either omission of seismic design when built, or the structure becoming seismically inadequate due to a later up gradation of the seismic codes) is to be retrofitted to meet the present seismic demands, PA can show where the retrofitting is required and how much. In fact this was what PA was originally developed for, and for which it is still widely used

3.5.3.3 The Single Degree of Freedom idealization

One of the fundamental simplifications underlying the concept of PA is that it considers the structure as a single degree of freedom (SDOF) system. And that means the structure model, with numerous joints with lumped masses, is assumed to be equivalent to a single vertical strut fixed at bottom with a single (but considerable) mass lumped at the top. Equations have been developed (ATC-40, FEMA 440) to arrive at this 'equivalent' damping ratio β (see Appendix), and also time period T (both continuously changing due to the weakening of hinges in course of the analysis) at any particular point in course of the progress of the analysis, having known only the instantaneous Δ_{roof} top and Vb of the structure.

3.5.3.4 The Acceleration Displacement Response Spectra

Another of the innovative concepts incorporated in the PA is the Acceleration Displacement Response Spectra (ADRS) representation, which merges the $V_b$vsΔ_{roof} top plot with the Response Spectrum (RS) curve. This is possible due to a relation connecting V_b, $\Delta_{rooftop}$ and T. First the $V_b$vs$\Delta_{rooftop}$ Cartesian has to be transformed to what is called spectral acceleration (S_a) vs spectral displacement (S_d) using the relations (ATC-40, 1996)

$$Sa = \frac{V_b / W}{(M_k / M)} \cdot g \qquad (1)$$

$$Sd = \frac{\Delta_{rooftop}}{P_k \phi_{k,rooftop}} \qquad (2)$$

where M_k, P_k and $\varphi_{k,rooftop}$ (using the notation of IS:1893-2016) are modal mass, mode participation factor and modal amplitude at rooftop respectively for the first mode (k=1). M and W are the total mass and weight of the building. This is effectively converting the acceleration and displacement of the building to that of the equivalent SDOF System. Next the RS graph, having axes S_a and T has to be converted using the relation in ATC-40

$$Sd = \frac{T^2}{4\pi^2} Sa \qquad (3)$$

Using the above relation, the time period T represented by any radial line drawn from the origin through the point (S_d, S_a) can be found. The two transformed plots, one that of $V_b$vs$\Delta_{rooftop}$ and the other the RS curve ;now known as the capacity and demand curves respectively ;can be superimposed to get the ADRS plot.

The PA has not been introduced in the Indian Standard code yet. However the procedure described in ATC-40 can be adapted for the seismic parameters of IS:1893-2016. The RS curve in ATC-40 is described by parameters C_a and C_v, where the curve just as in IS:1893-2016, is having a flat portion of intensity 2.5 Ca and a downward sloping portion described by C_v/T (Fig.3.7). The seismic force in IS:1893-2002 is represented by ZI/2R X S_a/g , where S_a/g is obtained from the RS curve which on the other hand is represented by 2.5 in the flat portion and the downward sloping portion by 1/T, 1.36/T and 1.67/T for hard, medium and soft soils respectively (Fig.3.8). On comparison it can be inferred that C_a = Z/2 and $C_v$is either of Z/2, 1.36·Z/2 and 1.67·Z/2 for hard, medium and soft soils respectively, for DBE (Design Base Earthquake – which is the one meant for design). Here 'I' (the importance factor as per Table 6 of IS:-1893-2016) is not considered, since in PA, the criteria of importance of the structure is taken care of by the performance levels (of IO, LS and CP) instead. R is also not considered since PA is always done for the full lateral load.

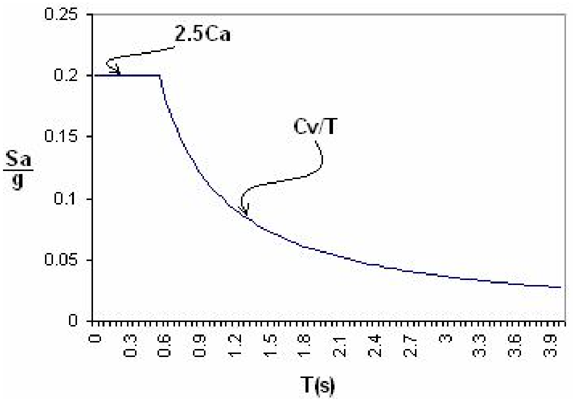


Figure 3.7: Response Spectrum curve described in ATC-40

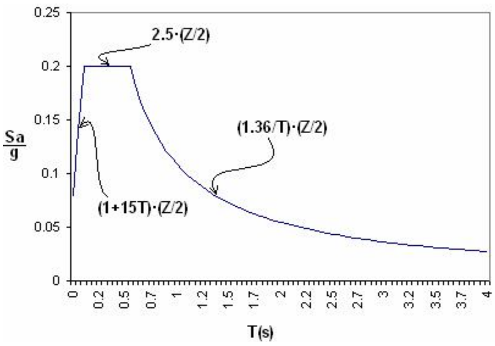


Figure 3.8: Response Spectrum curve described in IS: 1893 – 2016

3.5.3.5 Construction of Bilinear Representation of Capacity Spectrum

A bilinear representation of capacity spectrum is needed to estimate the effective damping and appropriate reduction of spectral demand. Construction of bilinear representation requires the definition of performance point api, dpi. This is the trial performance point which is estimated by the engineer to develop reduced response spectrum. If the reduced response spectrum is found to intersect the capacity spectrum at the estimated point api, dpi, then that point is the performance point. To construct the bilinear representation draw one line up from the origin at the initial stiffness of the building. Draw a second line back from the trial performance point, api, dpi. Slope of the second line should be such that when it intersects the first line, at point ay, dy, the area designated A1 in the figure 3.9 is approximately equal to the area designated A2 .i.e. area under capacity spectrum will be equal to area under its bilinear representation. To calculate the total damping of the system, we need to represent the hysteresis damping of the system as equivalent viscous damping. The total damping is represented as equivalent viscous damping represented by β_{eq}

$$\beta_{eq}= \beta_o+.05 \quad - (1)$$

Where, βo = hysteretic damping represented as equivalent viscous damping and 0.05 is the assumed 5 percent viscous damping of the system. The term

$$\beta_o= (1/4\pi)(ED/ESO) \quad - (2)$$

Where, ED is the energy dissipated by the structure in a single cycle of motion, that is, the area enclosed by a single hysteresis loop. ESO is the maximum strain energy associated with that cycle of motion, that is, the area of the hatched triangle.

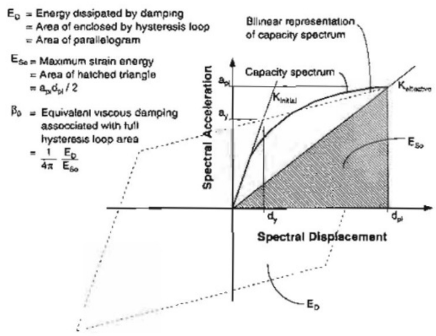


Figure 3.9: Derivation of Damping for Spectral Reduction

Once the equivalent viscous damping of the system is calculated, the response spectrum input by the user is reduced to obtain the reduced response spectrum. For this spectral reduction values in the constant velocity range, SRV and constant acceleration range SRA are determined

$$SRA= [3.21-(0.681x \ln \beta_{eq})] /2.12 \quad - (3)$$

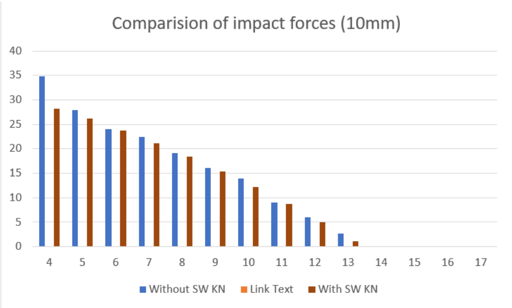
$$SRV= [2.31- (0.41 \text{ x } \ln \beta_{eq})] \quad - (4)$$

In above mentioned methods we have used “Nonlinear static analysis” to study the various parameters like Base reactions, Impact forces etc. Also the “Response Spectrum method” and “Time history analysis” is carried out to study the dynamic properties of structure.

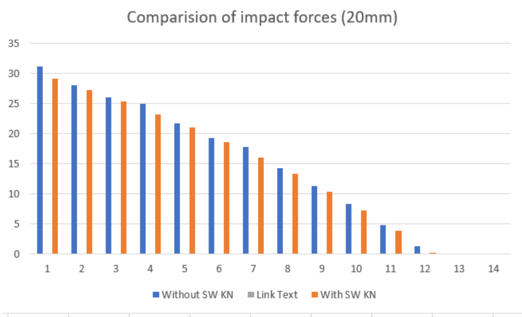
IV. OBSERVATIONS

4.1 Comparison of Impact Forces for structures with different areas:

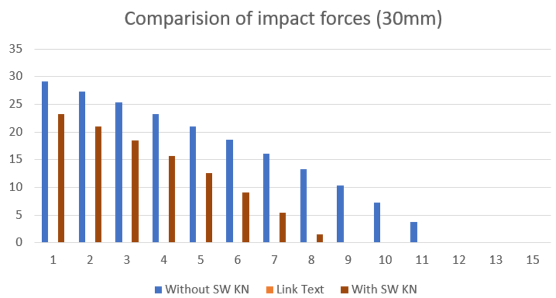
1. For 10mm separation gap.



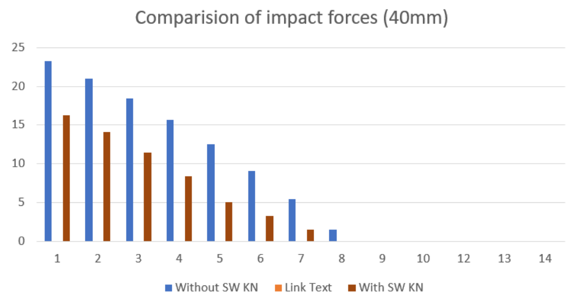
2. For 20mm separation gaP:



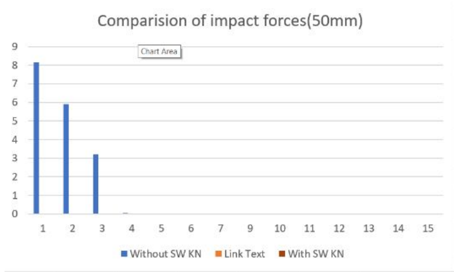
3. For 30mm separation gap:



4. For 40mm separation gap:

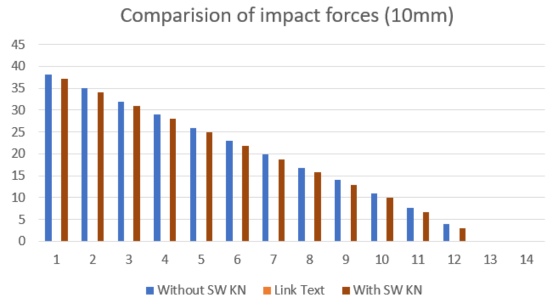


5. For 50mm separation gap:



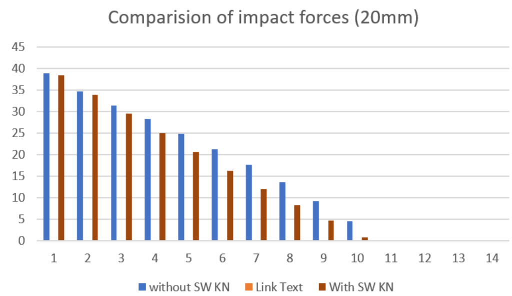
4.2 Comparison of Impact Forces for structures with same areas:

1. For 10mm separation gap

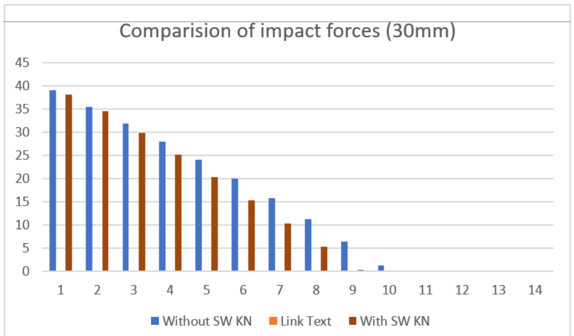


.

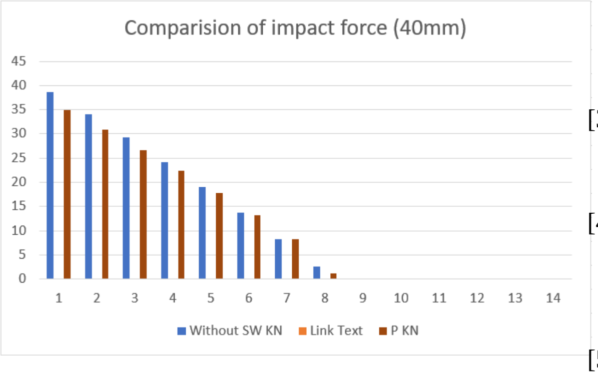
2. For 20mm separation gap.



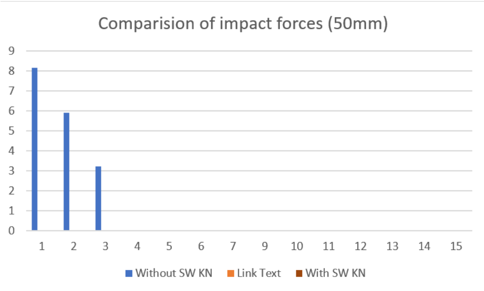
3. For 30mm separation gap.



4. For 40mm separation gap.



5. For 50mm separation gap.



V. CONCLUSION

1) In the fixed base adjacent buildings, pounding effects are found at few top stories in case of bay frame without shear wall and bare frame with shear wall.
2) Also the maximum pounding force occurs at the top for fixed based adjacent building where as it get reduced with the addition of shear wall.
3) In case of bay frame without shear wall, it shows the maximum pounding force and reduces with the increase in the separation gap between adjacent buildings.

VI. FUTURE SCOPE

In the current situation, as urbanization has led to increase in land rates so there should optimum utilization of land. This however will led to construction of structures close to each other. Thus such structures will undergo adverse effects, caused due to earthquakes. As a result to sustain these structures and to avoid any damage of property and life this study is required.

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