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Performance of High Rise Reinforced Concrete Structures With and Without Infill Walls

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ABSTRACT

The most of human occupation is acquired by residential and office buildings for our living and working purposes. To fulfill the demand of urbanization high rise buildings becomes necessary which are more vulnerable to the effects of earthquakes. Despite of having similar structural frame, size and shape residential and office buildings exhibit different infill wall positions. Use of brick infills in building construction is most common practice in India, this practice results in the function of the masonry brick infill wall as a bracing system to stiffen and enhance the strength of the structure. The eight storey model analysis results show that the infill walls increase the stiffness and strength of structure and the use of masonry infill walls located in between the columns of RC framed structures plays a major role in the damage and collapse of buildings during strong earthquakes. This thesis highlights the performance of models at different seismic zones and compares the behavior of bare and infilled reinforced concrete structures by non linear static pushover analysis .The analysis is done by using the guidelines given for infill in following codes (IS 1893:2016 (Part –I), ATC-40 and FEMA-356) on SAP2000 software. Infill wall modeled as equivalent diagonal strut in which the width of struts of each infill panel is evaluated. Simultaneously the non-linear hinge properties also evaluated and assigned to prepared models on SAP. It is observed that brick infill walls present in RC frame building reduce the structural drift but increase the strength and stiffness.

INTRODUCTION

An earthquake force is very unpredictable and different compare to other loads. There is lots of life damage if structure is not designed for earthquake forces when earthquake hits to the RC structure. In the past many countries like Nepal in the year 2015, hit by an earthquake ofintensity 7.8M_w. It has killed more than 8800 and injured 23000 people and Hundreds of thousands of people were made homeless with entire villages flattened.The study has been conducted to evaluate the behavior of RC bare and infill structures for all zones.

Most studies on infill wall behavior aim to understand their part in terms of strength in the assessment of the resistant capacity of existing buildings. Whereas less attention is paid to infill wall - RCC frame interaction modeling in order to evaluate their influence on RCC frame response

The weakness in structures is exposed by an earthquake event. An earthquake force is a very peculiar force and behaves quite differently than other types of loads, such as gravity and wind loads. It strikes the weakest spot in the whole three dimensional building. This should be an eye opener for designers and builders. Due to ignorance in design and poor quality of constructions it results in many weaknesses in the structure which causes serious damage to life and property. Masonry infill is used to fill the gap between the vertical and horizontal resisting elements of building frames, assuming that the infill will not take part in resisting any kind of load either axial or lateral. Hence, its significance in the analysis of frames is generally neglected. In fact, an infill wall considerably enhances the rigidity and strength of the frame structure. It has been observed through various researches, that the frame considering no infill has comparatively lesser stiffness and strength than the infill frame and therefore their ignorance cause failure of many multi-storey buildings when subjected to seismic loads.

A large number of reinforced concrete and steel building are constructed with masonry infill. The term infilled frame is used to denote a composite structure formed by the combination of moment resisting plane frame and infill walls. The masonry can be of bricks, concrete units, or stones. Since they are normally considered as architectural elements, their presence is often ignored by engineers. Infill masonry is considered as non-structural element, it will not take part in resisting any kind of load either axial or lateral; hence its significance in the analysis of RC (reinforced concrete) frame is generally neglected. But, infill enhances the strength and rigidity of the structure. The main reason of failure is the stiffening effect of infilled frame that changes the basic behavior of building during earthquake and creates new failure mechanism.

Building without an open storey, i.e. those with infilled walls generally performed much better even they undergo damage in the form of infill separation from the frame and the diagonal cracking of the infills. Infill walls of unreinforced masonry, contributed significantly to the lateral strength and stiffness of building. In normal residential buildings, the storey height and the panel lengths are nominal and brick infills have generally not shown vulnerability to out of plane collapses. Some conclusions of report were:

General Philosophies of Earthquake Design

The general philosophies for earthquake resistant building are

To prevent non-structural damage in minor earthquake. A minor earthquake occurs frequently in the service life of structure.

To prevent the structural damage and minimize non-structural damage in moderate earthquake which may occasionally occur; and

To avoid collapse or serious damage in the major earthquakes which may rarely occurs.

However code only requires building to be designed for one ultimate force level. In effect, buildings are explicitly design only for third criterion. The extensive damage and unpredictable economic losses caused by the 1994 Northridge Earthquake, had stimulated designers and owners to consider how design philosophy outlined above can be implemented to meet criteria (1) and (2), and to protect the building owner’s economic investment.

Necessity of Non Linear Analysis.

The equivalent lateral force obtained from seismic design codes of various countries isbased on elastic response of structure. The structure is designed for much less force than actual force. According to Li, Y.R. (1996) nonlinear analysis of building is done because:

The internal forces determined from elastic analysis under the code specified static forces are quite different from those produced during the inelastic response of the structure during the earthquake.

The elastic analysis method prescribed by the code does not give the amount of inelasticity or ductility and its location in building. As a result, ductility details must be provided throughout the structural member and at every connection Elastic story drifts found by the code-specified forces gets amplified by multiplication factors prescribed by the codes. The elastic storey drift will be quite different from the actual inelastic storey drifts. Thus keeping the amplified story drifts within the prescribed limits may not result in the intended damage control and safety against instability. Distribution of internal forces and deformations in yielding structures can be obtained through inelastic time history analyses of structures subjected to earthquake motions.

Behavior of In filled Frames

The modeling of the behavior of in filled frames under lateral loading (and primarily earthquake-induced loads) is a complex issue because the structures exhibit a highly nonlinear response that is caused by the interaction of the masonry-infill panel and the surrounding frame. It results in several modes of failures, which has a different failure load and hence a different ultimate capacity and overall behavior.

Various Aspects of Infill

Masonry infill in reinforced concrete buildings causes several effects under seismic loading.

Unequal distribution of lateral forces in the different frames of a building results in overstressing of some frames.

Vertical irregularities in strength and stiffness-soft storey or weak storey it results in higher inter storey drifts and higher ductility demand of RC elements of the soft storey in comparison to remaining stories.

Infill causes horizontal irregularities which causes significant amount of unexpected torsion forces since the centre of rigidity is moved towards the stiffer in filled frames of increased stiffness and as a result occurrence of very large rotation and large displacement in the extreme bare frame.

The effect of short column in in filled frame the full storey slender column whose clear height is reduced by its part-height contact with a relatively stiff masonry infill wall, which constraints its lateral deformation over the height of contact resulting in premature brittle failure of columns.

The out-of-plane and in-plane failure of masonry infill results in the cause of casualties.

Various Failure Modes of Infill

According to Asteries, P. G et al (2011) different failure modes of masonry-in filled frames were proposed on the basis of both experimental and analytical results produced during the last five decades which can be classified into five distinct modes these are as follows:

The corner crushing (CC) mode, which represents crushing of the infill in at least one of its loaded corners, as shown in Figure 1.2(a). This mode is usually associated with in filled frames consisting of a weak masonry-infill panel surrounded by a frame with weak joints and strong members.

The diagonal compression (DC) mode, which represents crushing of the infill within its central region, as shown in Figure 1.2(a). This mode is associated with a relatively slender infill, in which failure results from out-of-plane buckling of the infill.

The sliding shear (SS) mode, which represents horizontal sliding-shear failure through bed joints of a masonry infill, as shown in Figure 1.2(b).

This mode is associated with an infill of weak mortar joints and a strong frame. The diagonal cracking (DK) mode, which is seen in the form of a crack across the compressed diagonal of the infill pane l and which often takes place with simultaneous initiation of the SS mode, as shown in Figure no. 1.2(b). This mode is associated with a weak frame or a frame with weak joints and strong members in filled with a rather strong infill

LITERATURE REVIEW

Murty, C. V. R and Jain, S. K. (2000), this paper shows the experimental study of RCframe with & without infill masonry. The test is carried out by applying the cyclic displacement-controlled loading to the 12 single-bay single story models, out of which 2 are bare and 10 are in filled with masonry. Out of 10, 5 are unreinforced masonry infill (URM) frame models and 5 are reinforced masonry (RM) infill frame models.

Ghosh, A. K., and Amde, A. M. (2002) has found out the same order of occurrence ofthe five distinct failure modes based on the finite-element method and including interface elements at the frame-infill interface. Of the five modes, only the CC and SS modes are of practical importance because most infills are not slender and the second mode (DC) is therefore not favoured. The fourth mode (DK) should not be considered a failure mode because of the post cracking capacity of the infill to carry additional load.

Mondal, G. and Jain, S. K. (2008), he studied the effect of openings like door andwindow on the stiffness of infill. According to him Fema 356 and ATC 40 have some provision to take the effect of infill into consideration by modeling the infill as diagonal struct.

Ko, et al. (2010), in this paper, shake table test is performed on single-storey RCstructures considering with and without unreinforced masonry panel to evaluate the out- of-plane behavior of masonry panels. Four full-scaled single-storey models are prepared, one with pure frame, two frames with confined masonry panels having different thickness and one with infill panel.

Ju et al. (2012), this paper shows the experimental study of steel moment frame with RCinfill wall. The pushover analysis is done on four proposed steel moment resisting RC infill frames models, one with bare frame, one with ordinary RC infill walls, two with side slits between RC wall and frame member.

Fiore, A. et al (2012), he studied the global and local response of building under theearthquake. He performed finite element analysis and compare the result with experimental data in order to evaluate the local effect of the frame and under laying the influence of friction at the frame infill interface.

Al-Chaar et al. (2002) tested four 1/2-scale, single-story, masonry infilled non-ductile concrete frames and one bare frame, under monotonic static loading. These Specimens were different in terms of the number of spans (one, two, and three) and infill type (brick and concrete block).

Lee and Woo (2002) performed dynamic and static monotonic tests on a 1/5-scale two-bay three-story masonry infilled non-ductile concrete frame. They reported a large increase in stiffness, strength, and inertial force (due to added mass) of the infilled frame compared to the bare frame. They also observed that the deformation capacity of the infilled frame remains almost the same as the bare frame.

Anil and Altin (2007) conducted cyclic tests on nine 1/3-scale one-bay, one-story partially infilled ductile RC frames with different configurations and locations of the openings. They observed an increase in the strength and stiffness of the infilled frames compared to the bare frame. Their results also showed an increase in the strength of the infilled frame when the aspect ratio (infill length to the height ratio) increases.

Y. Sanada el al. (2011) studied the effects of nonstructural brick infills on the seismic performance of reinforced concrete (R/C) buildings. Experimental and analytical studies were conducted focusing on an Indonesian earthquake-damaged building due to the 2007 Sumatra earthquakes. A brick wall was extracted from the earthquake damaged building and transported to Japan from Indonesia to experimentally evaluate its seismic performance.

Stavridis et al. (2012) dynamically tested a 2/3-scale, two-bay, three-story masonry in filled non-ductile RC frame, as a representation of a 1920's building in California, under scaled historical ground motion records. This frame was fully infilled in one span and had window openings in the other span. They reported shear failure of concrete columns as well as considerable (but still repairable) damage of the structure, when the intensity of the spectra becomes 43% greater than maximum considered earthquake (MCE1) for Los-Angeles area. They concluded that the selected archetypical in filled frame can behave safely under strong ground motions.

T. Mahdi and V. Bahreini (2013) evaluated the nonlinear seismic behavior of intermediate moment-resisting reinforced concrete (RC) space frames with unsymmetrical plan in three, four and five stories buildings with and without considering the masonry infill (Masonry In fill).

ANALYSIS OF INFILL WALLS IN RC STRUCTURE

Introduction

There is large uncertainty in the accuracy of RC structures with infill because the modulus of elasticity of concrete and infill are difficult to determine. They vary with stress level, loading condition (static or dynamic), material strength, and age. For this reason the elastic modulus value specified by different design codes for different grades of material differ from one another. This implies there is considerable difference in actual and estimated fundamental time periods of structure and hence seismic force experienced by it.

Method of Analysis

According to the most of codes the structures with regular geometry perform well during the earthquake. The unsymmetrical placement of infill will cause irregularities in structures. Various codes propose to do the static analysis for regular structures and nonlinear analysis for irregular structures. Most of the codes does not calculate the seismic forces based on dynamic analysis because it does not differ much with code prescribed minimum value that is based the empirical estimate of natural time period. This may lead to the design of structure for underestimated forces.

Modelling of Infill Wall In Rc Structure

To take into the account the modeling of infill in structure we need to model the effect of infill in structure. However it is difficult to model as built structures due to various constrains because of various physical parameters associated with it. Even if all physical parameters are known like coefficient of friction between frame and infill, amount of separation, material properties there is no guarantee that the real structure behaves similar to the model. However the structural behavior also depends on material of construction, workmanship, quality and etc.
Modeling of infill can be done in two ways

Micro element modeling

Micro element modeling is the finite element modeling where the frame elements, masonry element, contact surface, separation, slipping are model to achieve to results. This method gives the better result but it is lengthy and cumbersome method.

Macro element modeling

According to this procedure no distinction between the individual masonry units and joints is made, and masonry is considered as a homogeneous, isotropic. While this procedure may be preferred for the analysis of large masonry structures, it is not suitable for the detailed stress analysis of a small panel, due to the fact that it is difficult to capture all its failure mechanisms.

Parametric Study

In Macro modeling the infill is model as diagonal strutsin frame. As per IS 1893 (2016) - Part-1,URM infill walls shall be modeled by using equivalent diagonal struts as below:

Ends of diagonal struts shall be considered to be pin-jointed to RC frame;

For URM infill walls without any opening, width w_{ds} of equivalent diagonal strut shall be taken as:

$w_{ds} = 0.175\alpha_h^{-0.4}L_{ds}$

Where,

$$\alpha_h = h\ \left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4E_f I_c H}}\right)$$

Earlier In 1958, Polyakov suggested the possibility of considering the effect of the infilling in each panel as equivalent to diagonal bracing and this suggestion was later taken up by Holmes who replaced the infill by an equivalent pin-jointed diagonal strut made of the same material and having the same thickness as the infill panel and a width equal to one third of the infill diagonal length. The 'one-third' rule was suggested as being applicable irrespective of the relative stiffness of the frame and the infill.

Homes (w) = $\frac{d}{3}$

Where:
d = diagonal length (m).
Stafford Smith and Carter (Smith and Carter 1969) carried out number of experiment on masonry infill with steel frame. He proposed the evaluation of equivalent width, λ_h as function of relative panel to infill frame stiffness

$$\lambda_h = h_c \sqrt[4]{\frac{E_m t \sin 2\phi}{4E_f I_c H_{\inf}}}$$

Main stone based on his experimental and analytical study is proposed the empirical formula for the calculation of equivalent strut for masonry in filled RC structure. The Main stone's empirical formula is also adopted by the Fema 356. The width according to the Main stone is

$$w = (0.175 * \lambda_h^{-0.4}) * d$$

Where

λ_h = aspect ratio
d = diagonal length (m)

Liauw and Kwan (1984), adopting values for the angle θ equal to 25° and 50° (typical for practical engineering purposes), also proposed a semi empirical expression for calculating the equivalent width,

$$w = \frac{0.95 \sin 2\phi}{2 * \sqrt{\lambda_h}}$$

Where
λ_h = aspect ratio
Φ = the a tan (h/l)

Width of infill strut for current study is calculated by FEMA 356 method of pin jointed strut and Stafford Smith and Carter method.

METHODOLOGY OF PUSHOVER ANALYSIS FOR RC STRUCTURE

Introduction

Pushover Analysis option will allow engineers to perform pushover analysis as per FEMA 356 and ATC-40. Pushover analysis is a static, nonlinear procedure using simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement at each increment, until the structure or collapse condition. (sermin, 2005)

Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components is recorded. Pushover analysis can provide a significant insight into the weak links in seismic performance of a structure.

Limitations of Pushover Analysis

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

Element Description of SAP2000

In SAP2000, a frame element is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics of individual frame elements are modeled as hinges represented by a series of straight line segments. A generalized force-displacement characteristic of a non-degrading frame element (or hinge properties) is considered in SAP2000.

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.

Methods of Analysis

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures. (sermin, 2005)

Elastic Methods of Analysis

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes. In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R".

Inelastic Methods of Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

BUILDING PERFORMANCE LEVELS AND RANGES(ATC, 1997A)

Performance Level: The intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

Performance Range: A range or band of performance, rather than a discrete level.

Designations of Performance Levels and Ranges: Performance is separated into descriptions of damage of structural and nonstructural systems; structural designations are S-1 through S-5 and nonstructural designations are N-A through N-D.

Building Performance Level: The combination of a Structural Performance Level and a Nonstructural Performance Level to form a complete description of an overall damage The four Building Performance Levels are Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. These levels are discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur. Each Building Performance Level is made up of a Structural Performance Level that describes the limiting damage state of the structural systems and a Nonstructural Performance Level that describes the limiting damage state of the nonstructural systems. Three Structural Performance Levels and four Nonstructural Performance Levels are used to form the four basic Building Performance Levels listed above. Other structural and nonstructural categories are included to describe a wide range of seismic rehabilitation intentions. The three Structural Performance Levels and two Structural Performance Ranges consist of: •S-1: Immediate Occupancy Performance Level •S-2: Damage Control Performance Range (extends between Life Safety and Immediate Occupancy Performance Levels) •S-3: Life Safety Performance Level •S-4: Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels) • S-5: Collapse Prevention Performance Level In addition, there is the designation of S-6, Structural Performance Not Considered, to cover the situation where only nonstructural improvements are made. The four Nonstructural Performance Levels are:

- N-A: Operational Performance Level
- N-B: Immediate Occupancy Performance Level
- N-C: Life Safety Performance Level
- N-D: Hazards Reduced Performance Level In addition, there is the designation of

STRUCTURAL PERFORMANCE LEVELS (ATC, 1997A)

Immediate Occupancy Performance Level (S-1)

Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness.

Life Safety Performance Level (S-3)

Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical.

Collapse Prevention Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure and to more limited extent degradation in vertical-load-carrying capacity.

ANALYSIS AND RESULTS OF RC STRUCTURE MODEL IN ALL ZONES

Structure Specification

Type of structure	Multistory frame
Zone	II III IV V
Layout	As shown in fig
Number of storeys	Eight
Floor to floor height	3 m
Live load	3.5
Materials	M 20 and fe415
Size of column	400 x 400 mm for 1 to 4 storey 300 x 300 mm for 4 to 8 storey
Size of beam	230 x 300 mm
Total depth of slab	120 mm
Soil type	Medium
Floor finish	0.5 KN/m^2
Terrace water proofing	1.5 KN/m^2

Models on SAP2000

A regular eight storey structure is model in the structural analysis programme (SAP2000). The longitudinal direction of three storey structure is 12 m and in transverse direction of structure is 9 m. The building has 4 bays in X direction and

3 bays in Y directions. In sap two types of model one with bare frame and the other with infill frame. The following figures shows the various model in the SAP2000.

LINEAR ANALYSIS

Introduction

Seismic analysis is a subset of structural analysis and is the calculation of the response of the building structure to earthquake and is a relevant part of structural design where earthquakes are prevalent. The seismic analysis of a structure involves evaluation of the earthquake forces acting at various level of the structure during an earthquake and the effect of such forces on the behaviour of the overall structure. The analysis may be static or dynamic in approach as per the code provisions.

Thus broadly we can say that linear analysis of structures to compute the earthquake forces is commonly based on one of the following three approaches.

1. An equivalent lateral procedure in which dynamic effects are approximated by horizontal static forces applied to the structure. This method is quasi-dynamic in nature and is termed as the Seismic Coefficient Method in the IS code.
2. The Response Spectrum Approach in which the effects on the structure are related to the response of simple, single degree of freedom oscillators of varying natural periods to earthquake shaking.
3. Response History Method or Time History Method in which direct input of the time history of a designed earthquake into a mathematical model of the structure using computer analyses.

Equivalent Static Analysis

This is a linear static analysis. This approach defines a way to represent the effect of earthquake ground motion when series of forces are act on a building, through a seismic design response spectrum. This method assumes that the building responds in its fundamental mode. The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces. In the equivalent static method, the lateral force equivalent to the design basis earthquake is applied statically. The equivalent lateral forces at each storey level are applied at the design 'centre of mass' locations. It is located at the design eccentricity from the calculated 'centre of rigidity (or stiffness)'. The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in meters) and height of the building from the support is represented as h (in meters). The design acceleration coefficient for different soil types as per latest IS 1893-2016 part-1 is :

The design base shear is to be distributed along the height of building as per IS 1893: 2016. The design lateral force at floor i is given as follows,

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

Response Spectrum Method

In order to perform the seismic analysis and design of a structure to be built at a particular location, the actual time history record is required. However, it is not possible to have such records at each and every location.

Further, the seismic analysis of structures cannot be carried out simply based on the peak value of the ground acceleration as the response of the structure depend upon the frequency content of ground motion and its own dynamic properties. To overcome the above difficulties, earthquake response spectrum is the most popular tool in the seismic analysis of structures.

Response Spectra

Response spectra are curves plotted between maximum response of SDOF system subjected to specified earthquake ground motion and its time period (or frequency). Response spectrum can be interpreted as the locus of maximum response of a SDOF system for given damping ratio.

Response spectra thus helps in obtaining the peak structural responses under linear range, which can be used for obtaining lateral forces developed in structure due to earthquake thus facilitates in earthquake-resistant design of structures.

RESULTS OF ANALYSIS

Equivalent static analysis and Response spectrum analysis results of G+7 RC structure Results obtained by equivalent static analysis and response spectrum analysis is shown below

Comparison of baseshear by ESA and RSA

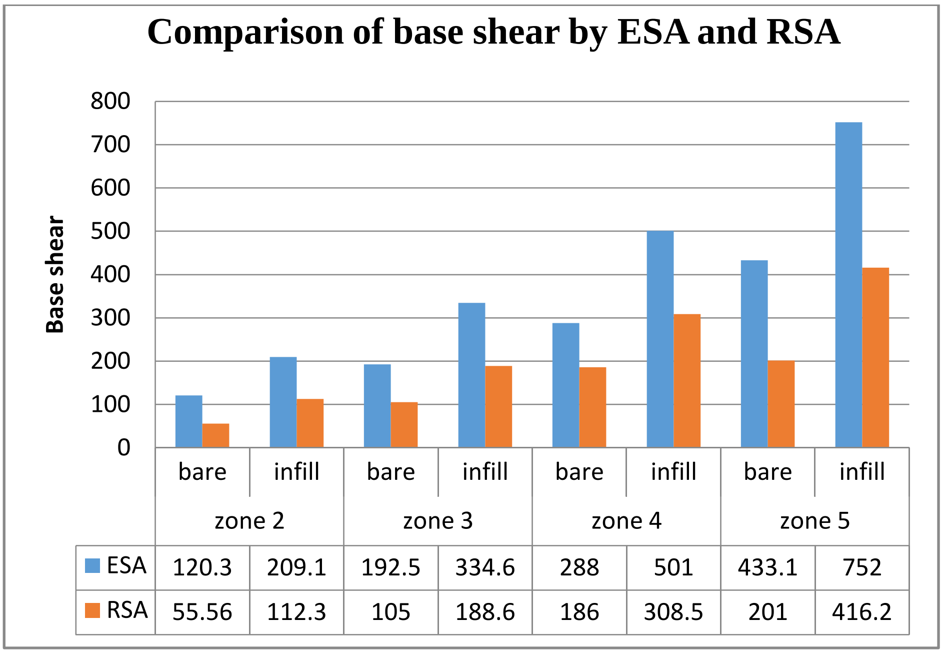

Fig: Comparison of base shear by ESA and RSA

From the above graph we can see that base shear value from response spectrum is very less, according to clause 7.8.2 IS 1893:2016 it has been given that in dynamic analysis the design base shear (V_b) shall be compared with a base shear calculated using a fundamental period Ta. Where V_b is less than all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\frac{\bar{V}_b}{V_b}$

RESULTS OBTAINED BY NONLINEAR ANALYSIS FOR BARE AND INFILL RC STRUCTURE

Comparison of pushover curve of bare and infill RC frame in all zones

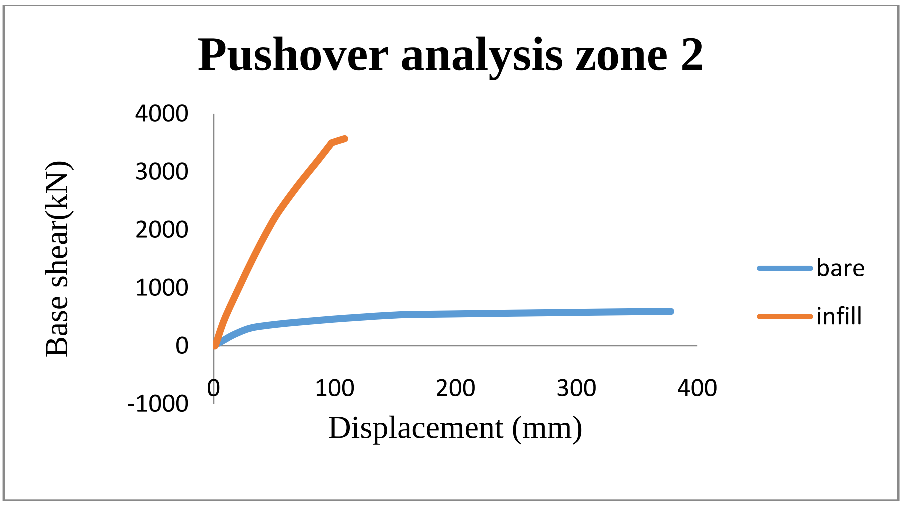

Fig: Pushover curve in zone 2

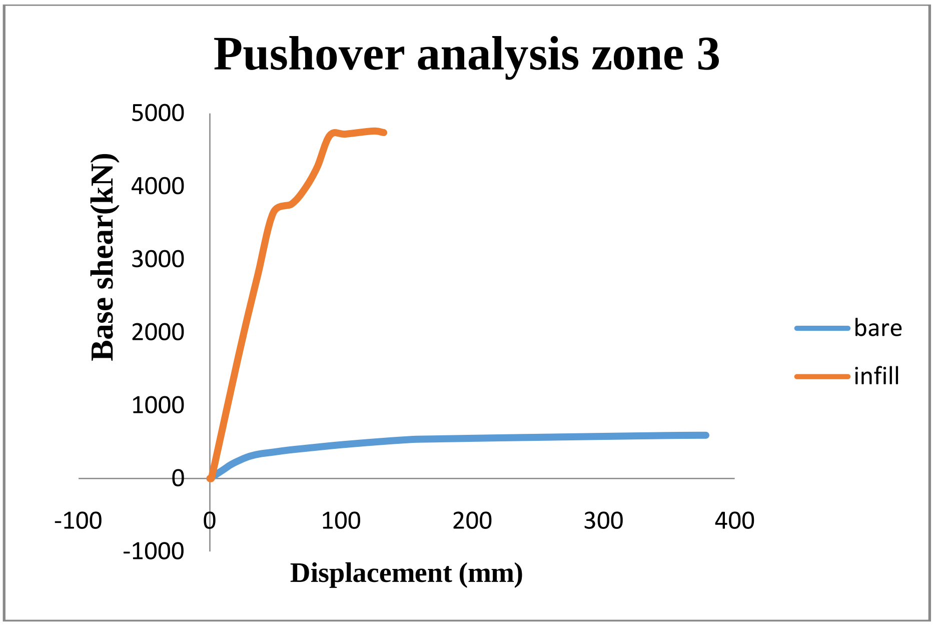

Fig : Pushover curve in zone 3

Salient features of pushover curve

The beam and column elements are modelled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. The frames are modelled with default hinge properties to study the possible differences in the results of pushover analysis. The base shear capacity and hinge formation mechanism for models with the default hinges at performance point and ultimate strength level are shown in below Table 1

Table 1: Hinge Formation Mechanism At Performance Point And Ultimate Strength Level

Zone		Case	BS	Disp	B	IO	LS	CP	C	D	E	TOTAL
V	Bare	P P	596.51	154	123	78	0	0	0	0	0	201
		Ultimate	670	384	50	40	5	60	68	4	0	227
	Infill	P P	4971.4	70.5	266	5	0	0	0	0	0	271
		Ultimate	6040	152.88	204	38	12	11	4	0	1	280
IV	Bare	P P	576.56	161.57	122	80	0	0	0	0	0	202
		Ultimate	644.391	391	82	28	14	48	23	2	0	197
	Infill	P P	4394	60.23	244	2	0	0	0	0	0	246
		Ultimate	5774	148.36	256	8	4	0	0	0	0	268
III	Bare	P P	542	176.65	119	80	0	0	0	0	0	199
		Ultimate	590	378	60	50	22	43	59	2	0	234
	Infill	P P	3640	48.44	270	0	0	0	0	0	0	270
		Ultimate	4735	132	264	8	0	0	0	0	0	272
III	Bare	P P	543.2	184	114	74	2	0	0	0	0	190
		Ultimate	589	373	46	45	14	58	76	2	0	241
	Infill	P P	1860	41.04	256	0	0	0	0	0	0	256
		Ultimate	3569	108.25	260	8	0	0	0	0	0	268

CONCLUSIONS

Design of structure plays important role during the earthquake and the effect of infill should be considered while designing of the structure. In present study infill is modeled as the diagonal strut. The width of infill strut is obtained from fema 354.

The behavior of the infill frame is different from the bare frame. The time period of infill frame structure is less compare to the bare frame structure. Lesser is the time period more will be the force and hence the in filled frame structure should be design considering the effect of infill in the structure. Bare frame displaces more compare to the infill frame and more displacement found in zone five bare frames. Base shear in bare frame is less compare to the infill frame. The inter-storey drift is more in bare frame compare to the infill frame.

Capacity of infillstructure is more compare to bare frame and different in different zones i.e. II III IV V, more vulnerable zone for earthquake is zone V.It requires more capable structure to resist lateral forces compare to other zone structures in the above study we found that zone V has higher capacity from capacity spectrum curves and it goes on decreasing with subsequent lower zone values.

The salient features of pushover curve for various cases are summarized in table 1. Table 1 gives information about number of hinges formed at different performance levels, base shear, and roof displacement for the four zone cases considered. It is observed that in filled RC structure in zone V building shows high lateral resistance compared to all other zone buildings.

Future Scope And Study

Building with unsymmetrical and irregular location of infill walls can be analyzed with the procedures explained in this thesis. This can provide detail view of structural behavior for changing location of infill wall in a particular storey or in several stories. This can help a designer to predict the structural response for such buildings. Performance based design of structure can be done Building with the unsymmetrical plan can also be analyzed with procedure explained in this thesis.

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