Siesmic Strengthening of Residential Building with Open Ground Story using Push over Analysis using Etabs Software

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Abstract: During the past Earthquakes, it has been observed that the open bottom storey is one of the most common problems causing server damage in the structural members leading to complete collapse. Open ground storey feature is mostly preferred in urban areas where there is congestion for parking space and hence architect is forced to provide the same inside the building, thus marking building unsafe. Such type of buildings shall be designed carefully following the provision laid out in IS 1893.
In this report, a study is carried out to improve the performance of a 6 storey structure with open ground storey. The seismic performance of the same is improved by increasing the moment carrying capacity of ground storey columns, providing wall infill in some portions, Combination of the above two

The performance of the structure is determined by using the Non linear Pushover analysis.

The main parameters are intense drift and lateral displacement.
The study shows that the seismic performances of the structure can be improved significantly by combination of constructing wall infill and higher design moments in columns. In case the damage occurs in wall only, it can be reconstructed and structure can be repaired easily.

Key words: E-Tabs,6 stories building, plans.

1. Introduction

In the recent major earthquakes, it is noticed that the seismic risk in urban areas is increasing and the infrastructure facility is far from socio-economically acceptable levels. There is an urgent need to reverse this situation and it is believed that one of the most promising ways of doing this is through the Performance Based Earthquake Engineering (PBEE) in which the structural design is based on the predicted performance of the structure during an earthquake. The Performance Based Earthquake Engineering (PBEE) also known as the Performance Based Seismic Engineering (PBSE) is a rapidly growing idea that is present in all guidelines that were recently published: Vision 2000 (SEAOC, 1995), ATC40(ATC, 1996), FEMA-273 (FEMA, 1997), and SAC/FEMA350(FEMA, 2000a). PBEE
implies design, evaluation, construction, monitoring the function and maintenance of engineered facilities whose performance under seismic loads responds to the diverse needs and objectives of owners, users and society. In loose terms, it requires that a building be designed to meet specific performance objectives under the action of the frequent or the rarer seismic events that it may experience in its lifetime. So, a building with a lifetime of 50 years may be required to sustain no damages under a frequent, “50% in 50 years” event, e.g., one that has a probability of 50% of being exceeded in the next 50 years. At the same time it should be able to remain repairable, despite sustaining some damage, during a “10% in 50 years” event and remain stable and life safe for rare events of “2% in 50 years”, although, subsequently, it may have to be demolished. Obviously such performance objectives can be better tailored to a building’s function, e.g., being stricter for a hospital that needs to remain operational even after severe events, while being more relaxed for less critical facilities, flexible and able to suit each building owner’s needs (respecting a minimum safety of INTERNATIONAL JOURNAL OF CIVIL AN STRUCTURAL ENGINEERING Volume 1, No 4. A general methodology was formulated in an effort to involve all the variables that may affect the performance such as seismic hazard, damage measures, collapse, financial losses or length of downtime due to damage, engineering demands such as story drifts, floor accelerations, etc., (Krawinkler and Miranda, 2004).perhaps the most costly earthquake in U.S. history, and other major earthquakes around the world which occurred at the end of the 20th century? This PBSD of buildings has been practiced since early in the twentieth century, England, New Zealand, and Australia had performance based building codes in place for decades. The International Code Council (ICC) in the United States had a performance code available for voluntary adoption since 2001 (ICC, 2001). The InterJurisdictional Regulatory Collaboration Committee (IRCC) is an international group representing the lead building regulatory organizations of 10 countries formed to facilitate international discussion of performance based regulatory systems with a focus on identifying public policies, regulatory infrastructure, education,

and technology issues related to implementing and managing these systems. In 1989, the FEMA funded project was launched to develop formal engineering guidelines for retrofit of existing buildings started, ATC,1989. The initial design document, NEHRP Guidelines for the Seismic Rehabilitation of Existing Buildings, FEMA 273, therefore contained a range of formal performance objectives that corresponded to specified levels of seismic shaking. The performance levels were generalized with descriptions of overall damage states with titles of Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. . These levels were intended to identify limiting performance states important to a broad range of stakeholders by measuring: the ability to use the building after the event; the traditional protection of life safety provided by building codes; and, in the worst case, the avoidance of collapse. Following the Northridge event, the structural engineers association of California (SEAOC,1995) developed a PBSD process, known as vision 2000, which was more generalized than that contained in FEMA 273 but used similarly defined performance objectives. Over the 10year period after publication of FEMA 273, although intended for rehabilitation of existing builings,the performance objectives and accompanying technical data in ASCE 41 responded to the general interest in PBSD and have been used for the design of new buildings to more reliable performance objectives than perceived available from prescriptive code provisions. ASCE 41 is considered to represent the first generation of performance based seismic design procedures

According to latest seismic code, over 60% of landmass is prone to moderate to severe earthquake events and 82% of population is long in these areas. Though the first seismic zone map and the earthquake resistant features for masonry buildings developed in 1930's. Seismic risk is gradually increasing due to unawareness of the proper construction methods for effective performance of building. Hence earthquake resistant structures are required.

After the 2001 earthquake the Indian middle class saw for the first time, multi-storey buildings fall like a pack of cards, and realized that these housing types are similar to the ones in which they are living or have plans to retire into. The Central and State governments announced numerous plans and activities. It was hoped that India would now have an effective programme for earthquake safety and that most (if not all) new constructions would now comply with seismic codes.

Discussions with professional colleagues around the country and the messages posted on the discussion forum of the Structural Engineers Forum of India (www.sefindia.org) clearly show that a huge number of unsafe buildings continue to be built every day in different cities and towns. After the 2001 earthquake, many municipal authorities have started asking the structural engineer (and others such as architects and builders) to certify that the building complies with seismic codes. Unfortunately, such certificates are easy to procure, sometimes on payment of small money, and need not have any correlation with how a building is built. Until the municipal authorities start enforcing measures to ensure that the building indeed complies with codes, false certificates will continue to be issued for a variety of reasons. The country is going through a major development phase wherein infrastructure is being added at an unprecedented pace. It is a great opportunity to ensure that all new infrastructures comply with seismic requirements. Unfortunately, this is not happening

THE GREAT INDIAN EARTHQUAKES

Within the last two hundred years, India has experienced five great earthquakes, each with Richter magnitude exceeding 8. The regions where these occurred are as follows:

- 1819 Kutch, Gujarat
- 1897 Assam ,
- 1905 Kanga, Himachal Pradesh ,
- 1934 Bihar-Nepal
- 1950 Assam-Tibet

Some special effects of these earthquakes are described here. The Assam Earthquake of 1897 this earthquake had its epicentre near Shillong. It is supposed be one of the largest earthquakes in the world, and has been assigned magnitude 8.7. The earth heaved in the most frightful manner, causing massive landslides and widespread floods. At some places land was displaced on the surface up to 12meters. Along the Chedrang River several waterfalls and lakes developed. More than 1500 people lost their lives in this thinly populated area. The Kangra Earthquake of 1905 this earthquake had twin epicentres – in the Kangra-Kulu and the Mussoorie-Dehradun valleys. It caused several large landslides, rock falls and large scale changes in the flow of water in springs, streams and canal.

2. Performance based design

Loma Prieta and the 1994 Northridge earthquakes, the structural engineering community and building owners began to question the effectiveness of current building codes to protect property (Gong, 2003). Seismic codes at that time were prescriptive and primarily concerned with life safety, their primary

In recognition of the different performance demands possible for different building types, in 1993 the Federal Emergency Management Agency (FEMA) provided funding to various organizations to develop the NEHRP guidelines; namely FEMA 273 (1997) and FEMA 274 (1997), for Seismic Rehabilitation of Buildings (ATC 58-2, 2003). The organizations in charge of developing the guidelines were the Applied Technology Council (ATC), American Society of Civil Engineers (ASCE), and the Building Seismic Safety Council (BSSC). These guidelines laid down the foundation for the PBD philosophy, which were primarily created for seismic assessment and rehabilitation of existing structures. Later in 1994, FEMA also awarded the Structural Engineers Association of California (SEAOC) a project to develop a framework for the PBD of new buildings, extending the concepts of FEMA 273. The project was known as VISION 2000 (ATC 58-2, 2003).

Currently, the PBD philosophy is widely accepted and used for assessing the performance of existing and new buildings, subjected to seismic loads. PBD assessment provides a 12good understanding of a structure's behavior, and allows building owners to have a better idea of a building's damages at different levels of earthquake intensity. The PBD philosophy can be defined as multi-level design that not only has explicit concern for the performance of a building at the ultimate-strength limit states, but also at intermediate and serviceability limit states (Hasan et al., 2002). In this philosophy, the design criteria are expressed in terms of the specified performance objectives that are chosen depending on the performance expected for the structure. A performance objective involves the combination of the structure's expected performance level with a seismic hazard (Bertero and Bertero, 2002). That is, a performance objective dictates the intensity of the seismic hazard that the building will be subjected to, and the limit damage the building should experience. A performance level is a discrete damage state, selected from among a number of damage possibilities (Gong, 2003). FEMA 273 (1997) describes three performance levels for structural components and four for non-structural components, which are combined to generate four performance levels for the assembled building. For the latter, the most common and representative performance levels in the design and rehabilitation of buildings are Operational (OP), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). At the OP level, the building is expected to be suitable for normal use and occupancy after an earthquake, and the risk to life safety in the building is extremely low, but some non-essential services may not function (FEMA 273, 1997). Buildings at the IO performance level are safe to reoccupy after an earthquake. However, non-structural systems may not function due to either lack of electrical power or damage of the equipment. Although the building may require some reparations before re-occupancy, minimal or no damage to structural elements is expected and only minimal damage to non-structural components is expected (FEMA 273, 1997). Buildings at the LS level undergo extensive damage to structural and non-structural components, and reparations must be done before re-occupancy. Although reparations may be costly, risk to life safety is low in buildings meeting this performance level (FEMA 273, 1997). Buildings in the CP level have reached a state of impeding partial or total collapse, and they may have suffered a significant loss of strength and stiffness with some permanent lateral deformation. Yet, the major components of the gravity load carrying system should continue carrying the gravity load demands (Gong, 2003). This building may be dangerous to life safety due to the failure of non-structural components. Most buildings at this performance level are considered complete economical losses.

A seismic hazard at a given site is represented by ground motions and its associated probability of occurrence (Bertero and Bertero, 2002). FEMA 273 (1997) identifies four seismic hazard levels with different mean return periods rounded to 2500, 500, 225 and 75 years, respectively. These seismic hazard levels are usually represented by their probability of exceedance in a 50 year period (i.e. 2%/50, 10%/50, 20%/50, and 50%/50 for severe to light ground motion intensities, respectively).

The performance objectives can range from minimum code requirements (e.g., the OP performance level for a 50%/50 year seismic hazard, and the LS performance level for a 10%/50 year seismic hazard) to high performance requirements (e.g., the OP performance level for a 2%/50 year seismic hazard) (Bertero and Bertero, 2002). The described high performance objective, poses high demands on buildings, since the building must remain operational for the largest seismic hazard. FEMA 273 (1997) proposes three different performance objectives for the rehabilitation of structures: basic, enhanced and limited safety. Each cell represents a performance objective which is the result of combining a performance level with a seismic hazard. Also shown in the table are the different multi-performance objectives that should be satisfied by a structure in accordance with its importance, such as ordinary building, essential building or hazardous facility. For instance, a hazardous facility should meet the OP and IO performance levels for 10%/50 and 2%/50 earthquake hazards, respectively.

The nonlinear static analysis procedure, better known as pushover analysis, is simple to apply and often yields good results for structures with a predominant fundamental period of vibration. However, pushover analysis should not be used for analysing structures for which higher-mode vibration effects are significant, such as structures of irregular plan, structures with irregular distribution of their mass along their height, and structures with seismic isolation devices (FEMA 273, 1997).

3. Performance based design

In the performance-based design process, design professionals, owners, and other stakeholders jointly identify the desired building performance characteristics at the outset of a project. As design decisions are made, the effects of these decisions are evaluated to verify that the final building design is capable of achieving the desired performance. It initiates with selection of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring damage or loss for identified earthquake hazards. Decision-makers including owners, developers, design professionals, and building officials will typically participate in the selection of performance objectives. This process may consider the needs and desires of a wider group of stakeholders, including prospective tenants, lenders, insurers, and the general public. The needs and opinions of others can have an indirect impact on the design of a building, but these groups generally do not have an opportunity to directly participate in the design process. Once performance objectives are selected, designs must be developed and the performance capability determined. As a minimum, basic building design information includes:

(1) The location and characteristics of the site;
(2) Building size, configuration, and occupancy;
(3) Structural system type, configuration, strength, and stiffness; and
(4) Type, location, and character of finishes and non-structural systems.

Performance assessment is the process used to determine the performance capability of a given building design. In performance assessment, engineers conduct structural analyses to predict building response to earthquake hazards, assess the likely amount of damage, and determine the probable consequences of that damage. Following performance assessment, engineers compare the predicted performance capability with the desired performance objectives. If the assessed performance is equal to or better than the stated performance objectives, the design is adequate. If the assessed performance does not meet the performance objectives, the design must be revised or the performance objectives altered, in an iterative process, until the assessed performance and the desired objectives match. Scope Seismic performance assessment (is the portion of the performance-based design process that is the primary focus of the methodology and recommended procedures contained herein. Seismic performance is expressed in terms of potential casualties, repair and replacement costs, repair time, and unsafe placarding resulting from Performance assessment is the primary focus of the general methodology and recommended procedures contained herein. The methodology can be expanded to consider additional consequences such as environmental impacts, and could be adapted to assess performance for other hazards and extreme loading conditions, but such enhancements are beyond the scope of the current version of the methodology. Implementation of the methodology requires basic data on structural and non-structural component vulnerability.

In the process of developing the building performance assessment method, three key aims should be kept in mind as follow:
(1) Subjectivity of assessment should be reduced to a minimum
(2) Assessment should provide consistently reliable result when used on similar buildings
(3) Result should offer a meaningful indication of the building’s total performance Before embarking on the development of the assessment system, efforts have to be made to address the important components or ingredients of a performance assessment.

This is to ensure that the pressing practical problems and thorny technical issues encountered in planning and executing the assessment would be adequately resolved (Berks, 1986). There are a few requirements for performance assessment systems that should be taken into consideration as follow:

1. Methodological Transparency This would allow access and understanding of assumptions, data and other methodological issues that would affect the outcome of assessments and subsequent ratings (Zimmerman, 2004). It would be beneficial to the user of the results as it allows them to make conscious choices and meaningful comparisons. For the building professionals, this means an avenue for them to improve their performance and compete more effectively.
2. Focus on performance Building performance assessment methodologies should be as far as possible fully performance based and quantifiable. The reason being that assessment on the basis of prescriptive technical features would typically prevent buildings without these features from obtaining a good assessment result regardless of actual performance (Zimmerman, 2004). However, it can be advantageous to include “feature-specific” assessment as features can have added contribution to building performance provided that the performance of fundamental attributes in the building are satisfied. The inclusion of features that enhance building performance in the assessment system could serve as a “bonus” category to reward and differentiate the high performance buildings.
3. Easily accessible measures the parameters to be measures should be easily obtained or accessed. It should not require expensive, difficult or disruptive data collection procedures where possible. They also need to be reliable, valid and easy to analyze and the results obtained from the system should be consistent (Becker, 1990)

4. Measures should not be only focused on one aspect the scope of assessment should not focus solely on one narrow aspect of building performance (Becker, 1990). On the contrary, they should represent a broad range of indicators which together can provide a holistic measure of performance that are meaningful to the occupants as well as the organization. In addition, the performance assessment tools should show the change in performance over time, even through the building's service life (Douglas, 1996).

4. Methodology

4.1 LOAD CALCULATION

Table 1: structure details

Dimension in X direction	24m
Dimension in Y direction	24m
Storey height	18m
Live load (typical)	3kN/sq.m
Live load (terrace)	1.5kN/sq.m
Floor finish	Hyderabad
Type of soil	3 soft
Z: zone factor	0.24
I: importance factor	1
R:response reduction factor	3
Column dimension	0.3x0.3
Beam dimension	0.25x0.3
Slab thickness	0.12m
Wall thickness	0.23m
Concrete grade	25
Steel grade	415
Wall load (2.7m height)	12.42kN/m
Wall load internal (2.7m height)	8.1kN/m
Parapet wall (1m height)	4.4kN/m

Table 2: calculation of seismic loads for third and fourth floor

	length	width	Height	density	Numbers	Weight
	(m)	(m)	(m)	kN/m^3		kN
Slab load	24	24	0.12	25	1	1728
Beam (X)	24	0.23	0.3	25	7	289.8
Beam (Y)	24	0.23	0.3	25	7	289.8
Column 1	3	0.3	0.3	25	49	330.98
Floor finish	24	24	1	1.5	1	864.0
Wall load external	24	1	1	12	4	1152.0
Wall load internal	24	1	1	7	10	1680.0
Live load	24	24	1	3	1	1728.0
						6766.4
					Total(25%of live load)	6334.35

Table 3: calculation of seismic loads for the first floor

	length	width	height	density	Numbers	Weight
	(m)	(m)	(m)	kN/m^3		kN
Slab load	24	24	0.12	25	1	1728
Beam (X)	24	0.23	0.3	25	7	289.8
Beam (Y)	24	0.23	0.3	25	7	289.8
Column 1	3	0.375	0.375	25	49	516.8
Floor finish	24	24	1	1.5	1	864.0
Wall load external	24	1	1	12	4	1152.0
Wall load internal	24	1	1	7	10	1680.0
Live load	24	24	1	3	1	1728.0
						6952.4
					Total(25%of live load)	6520.4

Table 4: calculation of seismic loads for ground floor

	length	width	height	density	Numbers	Weight
	(m)	(m)	(m)	kN/m^3		kN
Slab load	24	24	0.12	25	1	1728
Beam (X)	24	0.23	0.3	25	7	289.8
Beam	24	0.2	0.3	25	7	289.

(Y)		3				8
Column 1	2.25	0.375	0.375	25	49	387.6
Floor finish	24	24	1	1.5	1	864.0
Wall load external	24	1	1	6	4	576
Wall load internal	24	1	1	3.5	10	840
Live load	24	24	1	3	1	1728.0
						5407.2
					Total(25%of live load)	4975..2

Table 5: calculation of seismic loads for top floor

	Length	width	height	density	Numbers	Weight
	(m)	(m)	(m)	kN/m^3		kN
Slab load	24	24	0.12	25	1	1728
Beam (X)	24	0.23	0.3	25	7	289.8
Beam (Y)	24	0.23	0.3	25	7	289.8
Column 1	1.5	0.3	0.3	25	49	165.4
Floor finish	24	24	1	1.5	1	864.0
Wall load external	24	1	1	6	4	576
Wall load internal	24	1	1	3.5	10	840
Live load	24	1	1	5	4	480
						5233
					Total seismic load	5232.98

Table 6: calculation of seismic loads for second floor

	length	width	height	density	Numbers	Weight
	(m)	(m)	(m)	kN/m^3		kN
Slab load	24	24	0.12	25	1	1728
Beam (X)	24	0.23	0.3	25	7	289.8
Beam (Y)	24	0.23	0.3	25	7	289.8
Column 1	1.5	0.375	0.375	25	49	258.4
Column 2	1.5	0.3	0.3	25	49	165.4
Floor finish	24	24	1	1.5	1	864.0
Wall load external	24	1	1	12	4	1152
Wall load internal	24	1	1	7	10	1680
Live load	24	24	1	3	1	1728
						6859.4
					Total(25%of live load)	6427.37

Table 7: calculation of shear force in X direction

EL in X direction	
T=0.09H/D^0.5	0.33
Sa/g	2.5
Ah	0.1
EL in Y direction	
T=0.09H/D^0.5	0.33
Sa/g	2.5
Ah	0.1

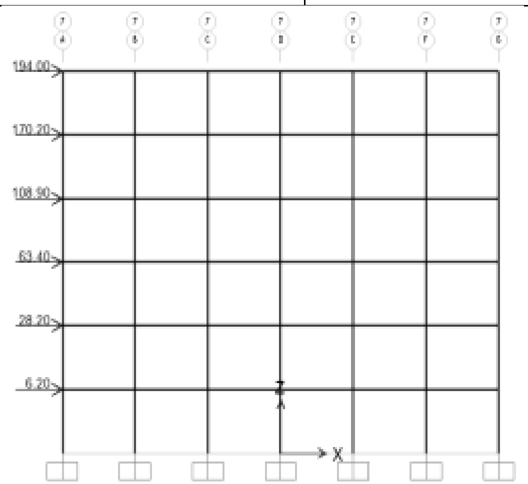


Fig: Figure showing the lateral forces.

Modelling of wall infill was done by using 3-Strut model. Three strut model gives the response of the structure closer to the actual behaviour of infill in continuum modelling and better than single strut model (Kaushik et al.2008)

Response (lateral displacements, storey drift) of three models bare frame, RC frame with full infill, soft bottom storey were obtained. The calculated loads which were considered to obtain the response are as follows.

Along X Direction			**Joint Force (kN)**					
Frame	**Stiffness (kN/m)**	**Dis. Factor**	**GF**	**I Floor**	**II Floor**	**III Floor**	**IV Floor**	**Top Floor**
Frame A	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame B	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame C	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame D	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame E	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame F	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0
Frame G	4589.3	0.14	6.2	28.2	63.4	108.9	170.2	194.0

5. Results and Conclusions

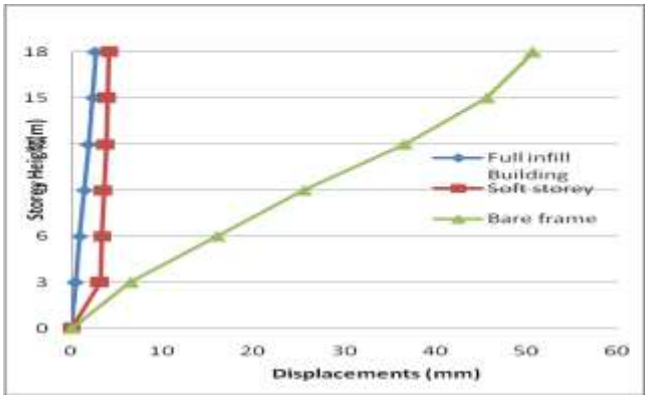
- Time periods and frequencies are as follows

Model Type	Time Period (Sec)	Frequency(Sec)
Bare Frame Building	1.20728	0.82831
Full infill Building	0.25617	3.90362
Building with Soft bottom Storey	0.55064	1.84607

❖ Obtained displacements and storey drifts are

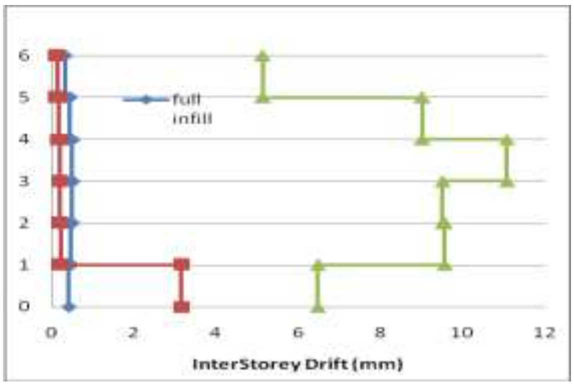
- Displacement Values

Height of building	Bare Frame	Full Infill	Soft bottom storey
0	0	0	0
3	0.42418	3.15502	6.47749
6	0.89694	3.37451	16.0343
9	1.39161	3.58072	25.53755
12	1.89761	3.79127	36.60685
15	2.34342	3.97832	45.62817
18	2.66858	4.11873	50.76261



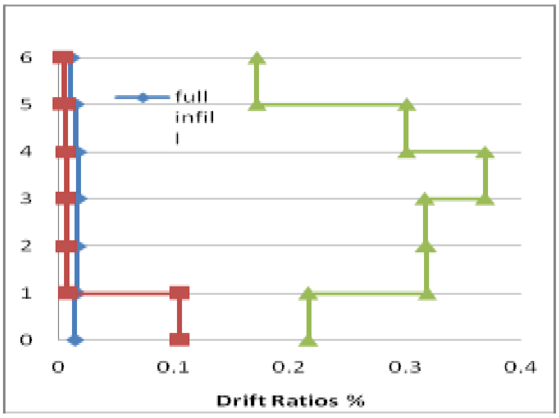
- Respective Inter storey Drift

Storey Level	Bare Frame	Full Infill	Soft Bottom Storey
1	0.42418	3.15502	6.47749
2	0.47276	0.21949	9.55681
3	0.49467	0.20621	9.50325
4	0.506	0.21055	11.0693
5	0.44581	0.18705	9.02132
6	0.32516	0.14041	5.13444



- Drift Ratios

Storey Level	Bare Frame	Full Infill	Soft Bottom Storey
1	0.014139	0.105167	0.215916
2	0.015759	0.007316	0.31856
3	0.016489	0.006874	0.316775
4	0.016867	0.007018	0.368977
5	0.01486	0.006235	0.300711
6	0.010839	0.00468	0.171148



Conclusions

1. The seismic response of open ground storey structure is poor and can lead to sudden collapse of structure because of hinge formation in ground floor column. The stiffness and strength of OGS is 2.8 and 3.9 times, respectively, lesser than full infill frame. The S F and BM also in OGS are much higher compared to full infill frame.
2. The member level retrofitting done by increasing column capacity al one cannot improve the seismic response significantly compare to wall retrofitting and combined retrofitting.
3. The study done clearly shows that strut modelling of brick infill should be done in numerical analysis to check the exact non linear response of the structure. The response of structure will change more significantly if wall provided in structure is asymmetric. Open ground storey should be avoided as far as possible, because design provision given in seismic code shows member level retrofitting which improves the nonlinear response not much significantly. The design provisions provided in seismic codes need to be modified for design provisions of soft storey.

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