Response of Tall Buildings with Symmetric Setbacks under Earthquake Loading

Abstract

Earthquakes are one of nature's greatest hazards to life. Reinforced concrete structures built in zones of low seismicity such as Sri Lanka have not had seismic effect taken into consideration in the design. The seismic performance evaluation and upgrading for non-seismic designed building structures is the most urgent issue for seismic hazard mitigation. The behaviour of a building during earthquakes depends critically on the shape, size and geometry of the building. A setback is a common geometric irregularity consisting of abrupt reduction of floor size in multi-storey buildings above certain elevations. Setbacks usually arise from urban design demands for illumination and aesthetic requirements. Computer program simulations are very valuable in testing a wide range of building types especially under disaster loads which are difficult and economically not viable to analyse with experimental methods.

This paper explores three-dimensional nonlinear dynamic responses of a typical tall building under earthquake loading, with and without setbacks. These 20 storey reinforced concrete buildings were first designed for normal (dead, live and wind) loads. The influence of the setbacks on the lateral load response due to earthquake loading in terms of peak deflections, accelerations, inter-storey drift and bending moments at critical locations (including hinge formation) was then investigated. Structural response predictions were performed with a commercially available three-dimensional finite element analysis program using non-linear direct integration time history analyses.

Results obtained for buildings with different setbacks are compared and conclusions made. It is revealed that the detrimental effects for seismic response due to symmetric setbacks are not that significant, for PGA values of 0.1g to 0.15g. Abrupt changes in moments and shears are experienced near the levels of the setbacks. Further it is revealed that there needs to be a balance between the stiffness and mass of the building to get the optimum response under seismic loading. Finally, these analyses give an indication of the integral seismic resistance of typical reinforced concrete wall-frame structures although specifically not designed for seismic loading. This is of significance to Sri Lanka, a zone with low seismicity and having many urban high-rise buildings that are not originally designed for seismic loading.

Keywords: Earthquake loading; Setbacks; Tall buildings; Dynamic analysis; Hinge formation

1. Introduction

Earthquakes can be considered as one of the worst natural disasters since they can occur at any place without any warning. They are caused mainly by the fracture of the crust of the earth or by the sudden movement along an already existing fault. They pose a direct threat to humans, when they cause major landslides or tsunamis: for instance the 26th December 2004 tsunami that hit many Asian countries including Sri Lanka killed approximately 275,000 people, making it the deadliest tsunami in recorded history. More commonly, the earthquake becomes a dangerous phenomenon only when it is considered in relation with the collapse of structures. This is because the structural system is designed basically for gravity loads and not for the horizontal inertia loads that are generated due to ground accelerations during an earthquake.

One of the recent earthquake events felt by most Sri Lankans was on 7th December, 1993 at 2:24 am., and it was a surprise for many people. The epicentre of this earthquake was located about 170 km west of Colombo and the magnitude as measured according to Richter scale was 4.7 (Abayakoon, 1998).

According to Dissanayaka and Mohadevan (2005), many of the buildings in Colombo have been constructed using reinforced concrete and therefore, naturally have some shear capacity to tolerate low level seismic shaking. In Sri Lanka, reinforced concrete structures are designed in accordance with BS 8110. Since there is a general perception that UK will not experience earthquakes, no specific provision has been made in design and detailing in British Standards to enhance the earthquake resistance. The reinforcement detailing used in all these buildings are generally based on those recommended in Standard Method of Detailing Structural Concrete, published by the Institution of Structural Engineers. These details do not take any precautions against the cyclic loading that occur in earthquakes.

Until recently seismic design of most structures was based on a static analysis using a set of lateral forces assumed to represent the actual (dynamic) earthquake loading. In the absence of commercial software appropriate for dynamic analysis of three-dimensional structures, as well as of the expertise for using whatever software of this type was available, most codes of practice clearly promoted the simpler static procedure (Kappos, 2002). However, the last two decades were marked by a massive introduction of more advanced software packages, running on increasingly more powerful hardware. As a consequence, in modern codes such as EC8, dynamic analysis is adopted as the reference method, and its application is compulsory in many cases of practical interest (Kappos, 2002).

A setback is a common geometric irregularity consisting of an abrupt reduction in the floor area of multistorey buildings above certain elevations. Setbacks are common in urban environments for several reasons: the three most common are zoning requirements that upper floors be set back to preserve light and air to adjoining sites, functional requirements for smaller floors at higher levels, and aesthetic requirements relating to the form of the building (Arnold and Reitherman, 1982). Structurally, a setback represents a large change in the stiffness of the lateral load resisting system. This change in rigidity may result in complex load and moment variations, which are difficult to predict without sophisticated analytical methods. These important features can be captured with the aid of 3D finite element modelling and with dynamic analysis procedures.

The response of real structures when subjected to a large dynamic input involves significant nonlinear behaviour. Dynamic inelastic analysis of three dimensional models of buildings enables more realistic assessment of their performance under unpredictable time varying earthquake loads. Inelastic behaviour is associated with hinge forming in some critical locations of the buildings. Occurrence of these hinges must be predicted and controlled in order to prevent collapse of the building.

This paper reports 3D nonlinear dynamic analyses of typical high-rise buildings under earthquake loading. These buildings have been designed for normal (dead, live and wind) loads, with obvious deficiencies and vulnerabilities to earthquake excitation. The influence of the setbacks on the lateral load response due to a probable earthquake in terms of peak deflections, accelerations, inter-storey drifts and bending moments at critical locations (including hinge formation) is investigated.

2. Objectives and Methodology

2.1 Objective

The intent of the study is to analyze the relative performance of typical 20 storey Reinforced Concrete (RC) buildings with and without setbacks subjected to an earthquake excitation scaled down to be of a peak ground acceleration appropriate for Sri Lankan conditions.

2.2 Description of the buildings used in the study

After a preliminary study on wall-frame buildings of different heights, a typical reinforced concrete office building of 20 storeys' height was selected for dynamic analysis, because it represents a typical high-rise building in Sri Lanka; also, 20 storeys is the limit beyond which wind rather than earthquake action dominates the lateral loading. All the 20 storey buildings had a storey height of 3.5 m and a constant building width of 42.0 m for both bases and towers. By changing the depth (in plan), and adding setbacks at different levels 8 different configurations were selected. The typical floor plan of the buildings is shown in Figure 1 and the different configurations selected are given in Table 1. Building Nos. 7 and 8 had very deep setbacks and are defined only in Table 1. Building Nos. 7 and 8 are rather unrealistic in layout, and chosen only to explore the theoretical progression of setback parameters. Computer generated 3D models of all the buildings are shown in Figure 2.

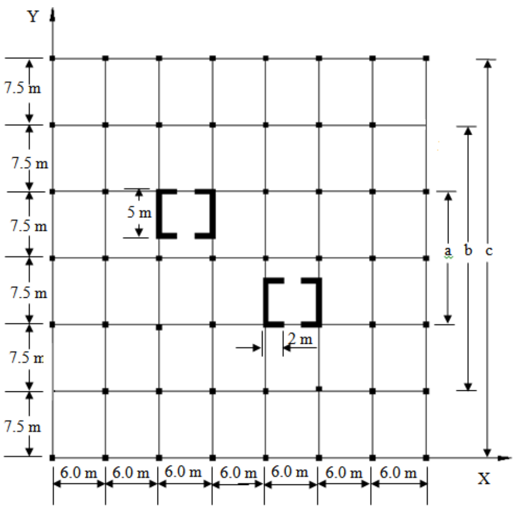


Figure 1: Typical floor plan for the buildings (element sizes not to scale)

Table 1: Building configuration guide (see Figure 1)

Building ID Number	***Base dimensión (m)***	***Top dimensión (m)***	***Setback level***

1	*c = 45*	*c = 45*	*Top*
2	*c = 45*	*b = 30*	*12*
3	*c = 45*	*a = 15*	*8*
4	*b = 30*	*b = 30*	*Top*
5	*b = 30*	*a = 15*	*12*
6	*a = 15*	*a = 15*	*Top*
7	*d = 105*	*a = 15*	*4*
8	*e = 195*	*a = 15*	*2*

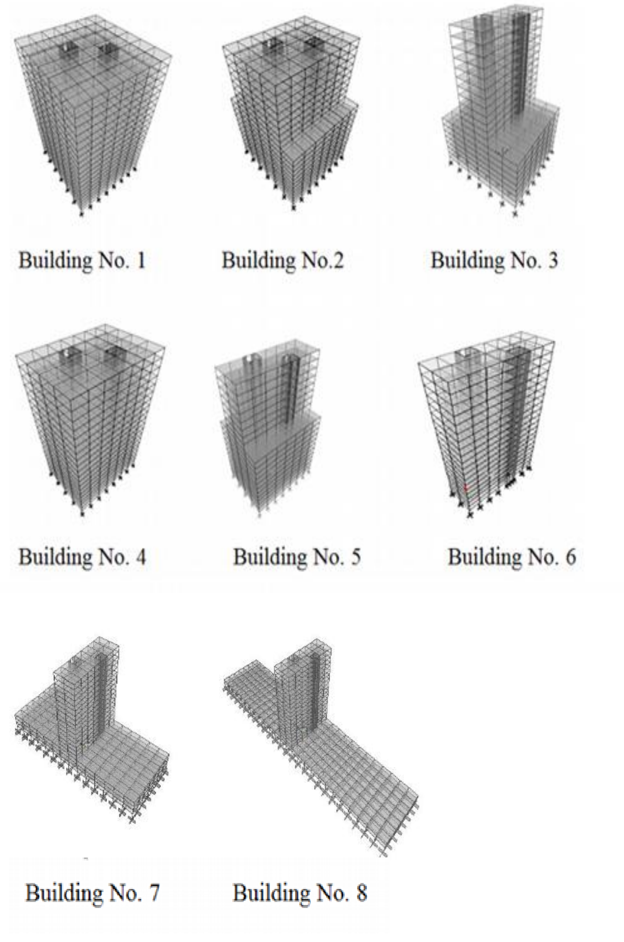


Figure 2: 3D models of buildings

The dimensions of the beams are 600 mm x 400 mm, while those of the columns are 800 mm x 800 mm up to the 12^{th} storey and 600 mm x 600 mm beyond that. The column dimensions in the bases of Building Nos. 7 and 8 are 300 mm x 300 mm. The floor slab thicknesses are 175 mm and shear wall thicknesses 250 mm. The material properties of the concrete used had a compressive strength of 30 N/mm^2, a Young's modulus of 24 kN/mm^2, a Poisson's ratio of 0.2, and a density of 24 kN/m^3

2.3 The feasibility of using real accelerograms for seismic design

Real accelerograms are the most valuable inputs for deriving design seismic loads. The use of real earthquake accelerograms carries the reassuring knowledge that the input motion is a genuine record of shaking actually produced by an earthquake. Before applying a real accelerogram record in an earthquake analysis, the record is often scaled to suit the local seismicity. According to Elnashai and Antoniou (2000), the accelerograms can only be scaled in terms of amplitude, simply multiplying the entire time-history by a single factor. According to Corderoy and Thambiratnam (1993), the time period over which a particular ground acceleration is applied is just as important as its amplitude and period in determining the behaviour of the building to which it is applied.

2.4 Selection of the earthquake input motion

The earthquake record selected to investigate the dynamic seismic response of the building models is the north-south component of the ground motion recorded at El Centro earthquake of May18, 1940. This earthquake recorded a maximum acceleration of 0.32g, a maximum ground velocity of 13.7 in./sec and a maximum ground displacement of 9.3 inches (Arnold and Reitherman, 1982). In this seismic study the buildings were subjected to the first 15 seconds of the well known earthquake input motion recorded at El Centro, 1940, N-S component, scaled down by the factor 0.3 to have a PGA of 0.1g to suit the local seismic conditions.

For buildings and other structures of complex shapes, the seismic forces should be applied in the most unfavourable direction using a three-dimensional model of the structure (Paz, 1994). The direction of interest in the present study is the y direction of the building for which there are 8 in-plane frames as indicated in Figure 3.5. Hence the seismic input was given in the form of base acceleration values at a constant time step of 0.02 s in the y direction for all the buildings under consideration.

2.5 Computer modelling

The 3-dimensional models of the complete buildings were created using SAP2000. Non-linear representation of the columns and beams was done to accommodate simulation of plastic hinges. Most computer programs for seismic analysis of buildings use point hinge models to represent the inelastic behaviour of R/C members. In such a model, inelasticity is permitted only at predetermined sections, which in the case of seismic loading, are the member ends (Kappos, 1991). Standard point hinge modelling which assumes hinges at the two ends was used for beam and column elements with proper account for yield moment-axial force interaction for columns. As such, Moment hinges were assigned for beam elements and PMM (axial force and biaxial moment) hinges for column elements, at the two ends of beams and columns respectively.

2.6 Damping

In seismic analysis problems, a proper specification of damping is important to obtain accurate results since structural members have some levels of inherent capability to minimize vibration by damping. Researchers and structural engineers have studied building vibration data and accelerometer earthquake responses and have recommended values of damping to be used in structural dynamic analyses (Hart and Wong, 2000). As such, the damping ratio selected for the analysis is 5% of critical, a value that is considered typical of reinforced concrete buildings for ULS.

2.7 P-Delta effects

According to SAP2000 (1997), the P-Delta effect refers specifically to the nonlinear geometric effect of a large tensile or compressive direct stress upon transverse bending and shear behaviour. SAP2000 is capable of handling geometric nonlinearity in the form of P-Delta effects. Hence P-Delta effects are also included in the earthquake analysis of the building models with SAP2000.

2.8 Static analysis

A static analysis was carried out for each building for dead, imposed and wind loads. After performing the static analyses for the dead, imposed and wind loads with SAP 2000, the design of reinforcement for the structural members was carried out again with SAP2000 to conform to UBC 97 criteria. The material properties of the concrete used had a compressive strength of 30 N/mm^2, a Young's modulus of 24 kN/mm^2, a Poisson's ratio of 0.2, and a specific weight of 24 kN/m^3.

2.9 Dynamic inelastic time-history analysis

The most sophisticated level of analysis available to the designer for the purpose of predicting design forces and displacements under seismic attack is dynamic inelastic time-history analysis. This involves stepwise solution in the time domain of the multi-degree-of-freedom equations of motion representing a multistorey building response. It requires one or more design accelerograms representing the design earthquake (Paulay and Priestley, 1992).

A nonlinear direct integration time history analysis was selected to obtain the response of these buildings under the selected seismic loading. The main response parameters considered in these seismic analyses are maximum interstorey drift, maximum top acceleration and maximum top displacement. Eight different building configurations having different setback configurations namely, Building Nos. 1, 2, 3, 4, 5, 6, 7 & 8 (see Figure 3) were selected to examine the seismic response. For all the structures, the first fifteen seconds of the factored El Centro excitation was input in the y direction, along which there are eight in-plane frames, to investigate the dynamic response of the models. To get consistent results for the 3D building models, the time step size had to be reduced to 0.001 s for seismic analyses. Hence the non-linear direct integration time history analyses were run for a duration of 15 s with 15,000 time steps for all the building models. Run times for the analyses were in the range of 1 to 4 days.

3. Results

3.1 General

A summary of the maximum responses obtained from the analyses for all the buildings under consideration are presented in Table 2

Table 2: Maximum response figures for the buildings studied

Building No.	*No. 1*	*No. 2*	*No. 3*	*No. 4*	*No. 5*	*No. 6*	*No. 7*	*No. 8*
Fundamental Period (s)	*2.096*	*1.719*	*1.499*	*1.929*	*1.518*	*1.729*	*1.697*	*1.716*
Top displacement (mm) *Time taken (s)*	*90.66* *5.598*	*63.65* *12.06*	*63.7* *11.91*	*77.42* *6.483*	*65.07* *11.91*	*88.63* *6.372*	*69.05* *6.424*	*70.75* *6.321*
Top acceleration (m/s^2) *Time taken (s)*	*1.631* *5.659*	*1.621* *5.659*	*2.407* *2.420*	*1.831* *2.420*	*2.233* *2.420*	*2.383* *2.420*	*3.391* *2.420*	*3.137* *2.420*
Inter-storey drift (mm)	*6.63*	*5.68*	*5.66*	*4.81*	*5.08*	*5.36*	*5.65*	*4.83*
No. of Column hinges formed	*42*	*23*	*04*	*31*	*03*	*24*	*10*	*03*
No. of Beam Hinges formed	*166*	*03*	*0*	*35*	*0*	*0*	*0*	*0*

3.2 Displacement and acceleration response

Figures 5.3 illustrate time history responses of maximum top displacement and maximum top translational acceleration obtained for building No. 4 under the factored El Centro excitation.

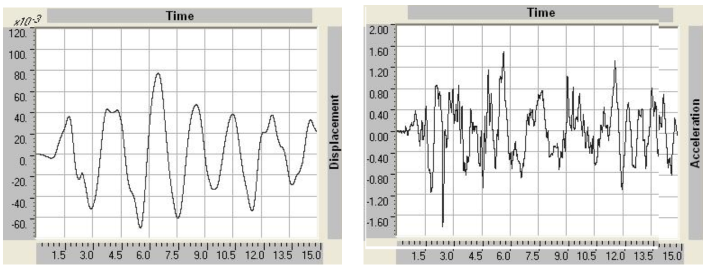


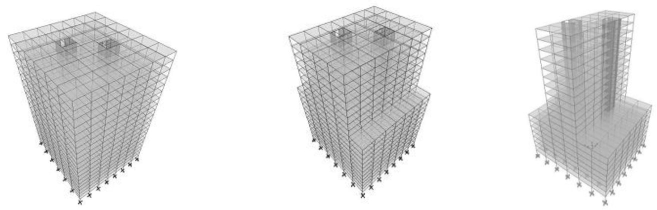
Figure 3: Maximum displacement and acceleration time histories for Building No. 4

Some interesting observations can be made from the results summarized in Table 2 and from the máximum displacement and acceleration time histories obtained for the eight buildings. Generally, the maximum acceleration response occurs immediately after the excitation, while the maximum displacement occurs at a later stage in the time history. This is true for all the building configurations, except for Building No. 1, in which both maxima occur almost at the same time.

It should be noted that careful examination of response figures for all the buildings under consideration reveals that the greatest top displacement of 90.66 mm is 1/772 of building height and the largest inter-storey drift of 6.63 mm is 1/527 of storey height. For conventional structures, the preferred acceptable range of drift index is approximately 1/650 to 1/350 (Smith & Coull, 1991). Hence it is clear that all the buildings under consideration lie within the acceptable range for earthquake analysis. Also, all hinges formed were in the strain-hardening region and did not constitute danger of collapse.

3.3 Comparison of response parameters for buildings having identical base dimensions

Figure 4 illustrates the comparison of maximum interstorey drift, maximum top displacement and maximum top acceleration for Building Nos. 1, 2 and 3, all having identical base dimensions. Building No. 2 has a setback at the 12th floor while building No. 3 has one at the 8th floor.



Building No. 1 Building No. 2 Building No. 3

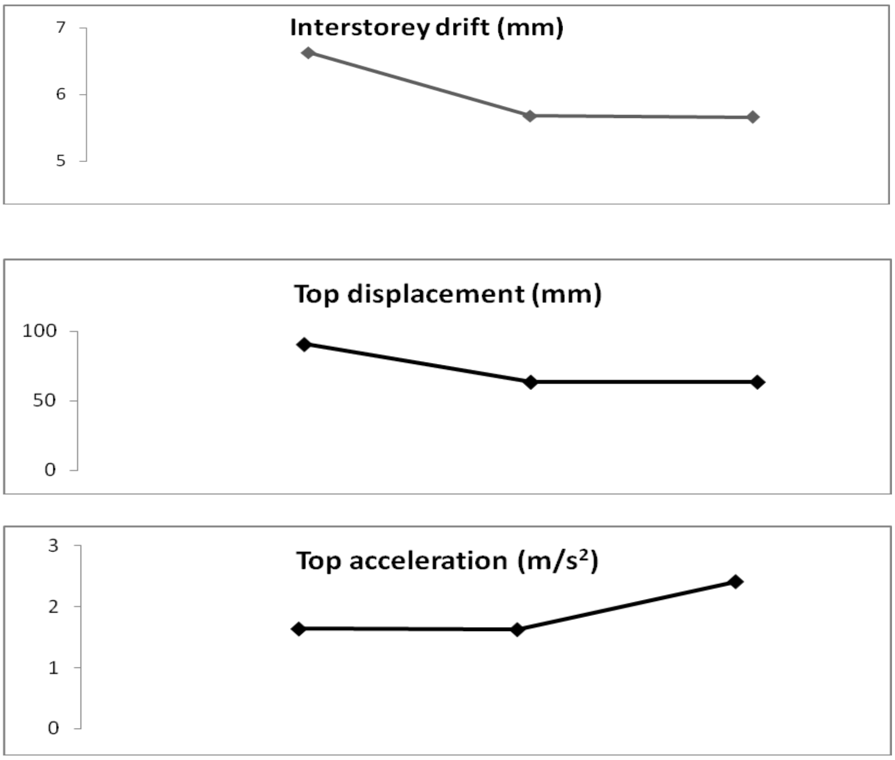


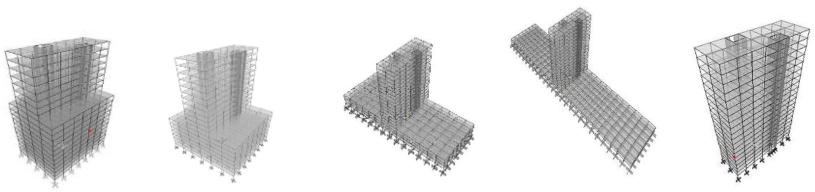
Figure 4: Comparison of response for building Nos 1, 2 & 3

It is seen from Figure 4 that the interstorey drift and top displacement follow similar trends, with the worst performance in Building No.1 which has the highest mass, and as such the highest inertia. Obviously it attracts the highest earthquake force, giving the highest response values for both interstorey drift and top displacement. On the contrary, the top acceleration response shows a reverse trend, with the worst performance in Building No. 3, which has the narrowest tower part. This may be due to the fact that, being the most flexible building out of the three, Building No. 3 is the most severely vibrated building.

When overall performance with respect to the three response parameters is considered, Building No. 2 gives the best performance, having somewhat lower response figures. Although it has a setback at 12th storey, its tower part is deeper than Building No. 3, hence giving a lower top acceleration value.

3.4 Response of the narrowest tower with increasing depths of podium

Of great importance is the comparison of response for Building Nos. 5, 3,7, 8 & 6, all of which have the narrowest tower (15m deep), but with the height of the 15 m tower part increasing from 28 m for the Building No. 5 to 70 m for the Building No. 6, as illustrated in Figure 5.16.



No. 5 No. 3 No. 7 No 8 No 6

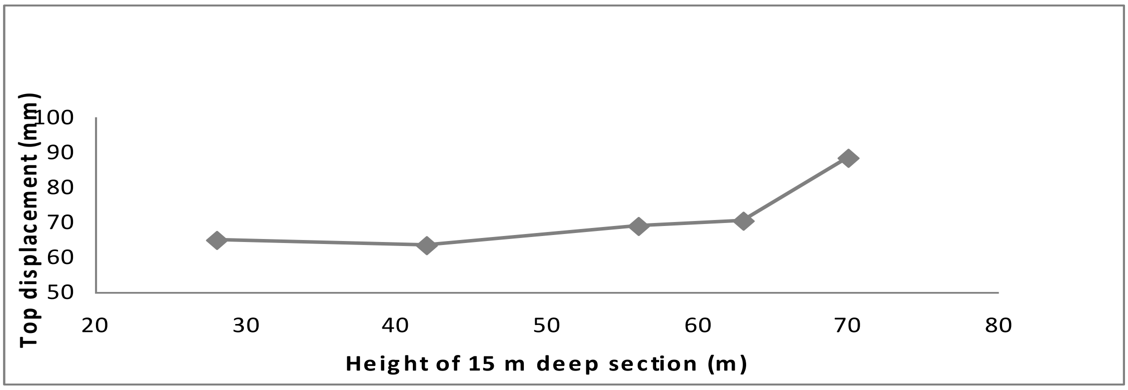


Figure 5: Comparison of top displacement for building Nos. 5, 3,7, 8 & 6

Figure 5 shows the variation of máximum top displacement for the above mentioned set of buildings. Generally, it is seen that there is a trend of increase in displacement with the increase in height of the 15m deep tower section as can be seen from Figure 5. This may be due to the fact that as the tower height increases the building becomes more slender and flexible, giving higher top displacement values, as the horizontal movement of top stories will become higher with the increase in tower height.

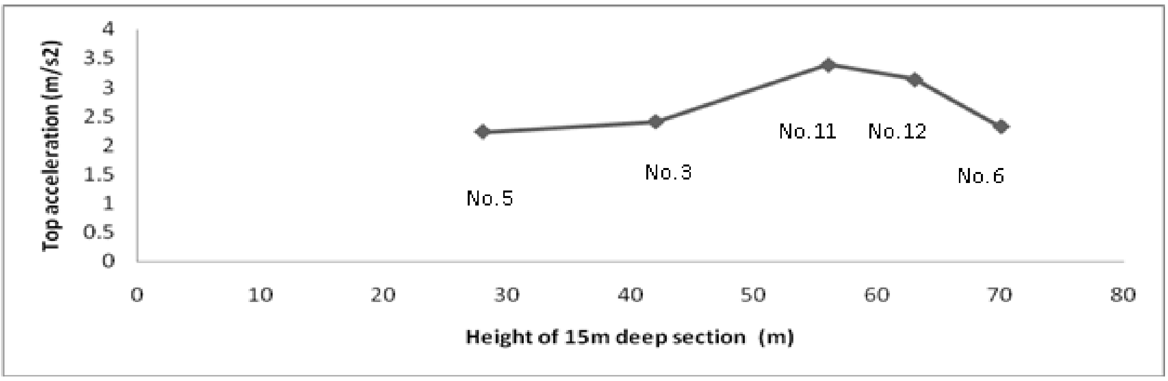


Figure 6: Comparison of top acceleration for building Nos. 5, 3,7, 8 & 6

Figure 6 shows the maximum top lateral acceleration response for the same set of buildings. Here, a somewhat different pattern is obtained, with Building No.7 giving the highest acceleration value. Building No. 5, which has the shortest tower height gives the lowest acceleration value, presumably because it is the building having the minimum unstiffened tower height, out of the five buildings under consideration.

3.5 Column moment and column shear variation near setback level

In the seismic analysis, the variation of column moment and column shear near the setback level of setback buildings is of special interest.

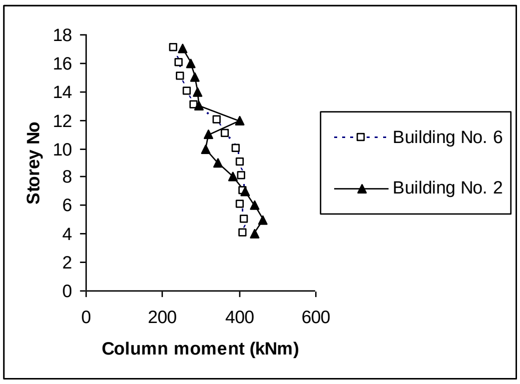


Figure 7: Comparison of column moment variation near setback level

Figure 7 shows the variation of column moments in the vicinity of the setback in Building No. 2, compared to Building No. 6 having no setback. The abrupt increase in column moment just below the setback level in Building No. 2 demonstrates the complex moment variation taking place at that level, compared to the somewhat uniform variation seen in Building No. 6, representing uniform buildings without any setback. A similar variation was seen in the column shear variation too. Similar results were obtained for the other setback buildings also near the setback level.

3.6 Comparison of response parameters when subjected to a higher peak ground acceleration

In order to check the response of these buildings when subjected to a higher earthquake intensity, Building No. 1, which showed the worst response for the previous analysis, and Building No. 3 having a setback with same base dimensions as Building No. 1, were selected. These two buildings were then subjected to the same El Centro excitation, but appropriately factored such that the peak ground acceleration in this case increased to 0.15g.

Table 3 presents the comparison of maximum responses obtained for building Nos. 1 and 3 for the two cases of PGA 0.1g and 0.15g.

Table 3: The comparison of maximum responses for the two cases

Building No.	*No. 1*	*No. 3*	*%change for No. 3 with respect to No. 1*
Top displacement (mm)			
0.1g	*90.66*	*63.7*	*-29.7%*
0.15g	*119.9*	*84.39*	*-29.6%*
Top acceleration (m/s^2)			
0.1g	*1.631*	*2.407*	*47.6%*
0.15g	*2.084*	*3.188*	*53.0%*
Inter-storey drift (mm)			
0.1g	*6.63*	*5.66*	*-14.6%*
0.15g	*7.82*	*5.93*	*-24.2%*
No. of column hinges formed			
0.1g	*42*	*04*	
0.15g	*48*	*20*	
No. of beam hinges formed			
0.1g	*166*	*0*	
0.15g	*537*	*25*	

As illustrated in Table 3, as far as top displacement and top acceleration are concerned, the percentage change in response figures for Building No. 3 compared to Building No. 1 are more or less the same for the two cases, namely PGA of 0.1g and 0.15g. However, when percentage change in interstorey drift is considered, the 0.15g case gives higher percentage reduction than the 0.1g case, showing a 24.2% decrease when Building No. 3 is compared with Building No. 1.

The drift index for both buildings lies within the acceptable range even at a PGA of 0.15g. Note here that Building No. 1 gave the worst performance under El Centro factored to a PGA of 0.1g. Also, all hinges formed were in the strain-hardening region and did not constitute danger of collapse. As such it is seen that these buildings, when subjected to El Centro scaled down to a PGA of 0.15g, also perform fairly well, without catastrophic collapse.

4. Conclusions

(1) When a building is subjected to a seismic excitation, the maximum acceleration response occurs immediately after the event and maximum displacement occurs at a later stage in the time history.

(2) The abrupt change in the rigidity of the lateral load resisting system in tall setback buildings leads to abrupt changes in the moments and shears at the setback level. This becomes more pronounced when shear walls are cut off at the setback level

(3) Twenty storey tall buildings with shear walls and frames that are designed for just normal loads perform reasonably well, without catastrophic collapse, when subjected to a seismic excitation having a PGA of 0.1g. A PGA of 0.15g was also found to be safe for two of the buildings. Some attention would perhaps need to be paid to detailing, in order to enhance ductility

(4) Generally, lighter buildings having less mass and hence less inertia suffers less damage in terms of maximum top displacement, maximum interstorey drift and number of hinges formed, under earthquake loading. On contrary, maximum top acceleration depends more on the flexibility of the building than the mass, resulting in higher acceleration values for more flexible buildings. The presence of setbacks does not seem to give a significant difference in response as compared with the difference seen with the change of mass. Anyway, the top displacement response increases with the increase in mass non homogeneity

(5) The maximum acceleration response appears to be sensitive to the percentage of setback, resulting in somewhat higher values when the percentage of setback increases, causing an increase in vertical mass non-homogeneity.

(6) A reduction in the rigidity of a building by too much will have adverse effects on seismic response, even though the inertia may also reduced thereby.

(7) If a building is to be designed in an area prone to earthquakes, there needs to be a balance between the stiffness and the mass of the building to get the optimum response, when subjected to an earthquake loading. Generally, having a setback with a high mass non homogeneity may not be beneficial for a building designed in an earthquake prone area.

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