

**RESEARCH ARTICLE**

# **Response of tall buildings with symmetric setbacks under blast loading**

**I.N. Jayatilake<sup>1</sup>, W.P.S. Dias<sup>1\*</sup>, M.T.R. Jayasinghe<sup>1</sup> and D.P. Thambiratnam<sup>2</sup>**

<sup>1</sup> *Department of Civil Engineering, Faculty of Engineering, University of Moratuwa, Katubedda.*

<sup>2</sup> *School of Urban Development, Queensland University of Technology, Brisbane, Australia.*

Revised: 30 November 2009 ; Accepted: 18 January 2010

**Abstract:** This study explores three-dimensional nonlinear dynamic responses of typical tall buildings with and without setbacks under blast loading. These 20 storey reinforced concrete buildings have been designed for normal (dead, live and wind) loads. The influence of the setbacks on the lateral load response due to blasts in terms of peak deflections, accelerations, inter-storey drift and bending moments at critical locations (including hinge formation) were investigated. Structural response predictions were performed with a commercially available three-dimensional finite element analysis programme using non-linear direct integration time history analyses. Results obtained for buildings with different setbacks were compared and conclusions made. The comparisons revealed that buildings having setbacks that protect the tower part above the setback level from blast loading show considerably better response in terms of peak displacement and inter-storey drift, when compared to buildings without setbacks. Rotational accelerations were found to depend on the periods of the rotational modes. Abrupt changes in moments and shears are experienced near the levels of the setbacks. Typical twenty storey tall buildings with shear walls and frames that are designed for only normal loads perform reasonably well, without catastrophic collapse, when subjected to a blast that is equivalent to 500 kg TNT at a standoff distance of 10 m.

**Keywords:** Blast loading, dynamic analysis, hinge formation, setbacks, tall buildings.

## **INTRODUCTION**

Blasts that result from bomb explosions have become a new threat to buildings designed for normal static loads. Under blast loading, buildings are subjected to the loads that are quite different from those governing their primary design in both magnitude and direction. Thus a better understanding of the behaviour of high-rise buildings under blast loads is of prime importance, because there

are many buildings that may be under threat of blast loading although not originally designed for the same.

A setback is a common geometric irregularity consisting of an abrupt reduction in the floor area of multistorey buildings above certain elevations. Setbacks may be introduced 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<sup>1</sup>. A building with a setback can be considered to be made up of two parts. The part of the structure above the setback level is the tower and that below is the base. Depending on the location of the tower relative to the base, one can also classify setback structures into those with symmetric setbacks and asymmetric setbacks. Uniform Building Code (UBC) 1988<sup>2</sup> requires dynamic analysis for all vertical irregularities. The New Zealand code NZS 4203: 1984<sup>3</sup> requires dynamic analysis for all setbacks in which the tower has a dimension of less than 75% of the corresponding dimension of the lower floor. According to this code, dynamic analysis is required whenever modes other than the fundamental are likely to be significant<sup>4</sup>. Structures with irregular plans, vertical setbacks or soft storeys will cause no additional problems if a realistic three dimensional computer model is created for their analysis<sup>5</sup>.

A number of studies on the lateral load response of setback buildings have been carried out by researchers. The effect of setbacks on the lateral load response of symmetric and asymmetric high-rise shear wall buildings was investigated by Rutenberg and Dickman<sup>6</sup>. Only the static response of setback buildings has been considered.

\*Corresponding author (priyan@civil.mrt.ac.lk)

It was shown that the shear deformation of walls and in-plane flexibility of floor slabs appreciably affect the distribution of lateral loads among the walls in the vicinity of the setback.

A study carried out by Tso & Yao<sup>7</sup> to evaluate the seismic load distribution in buildings with eccentric setbacks subjected to seismic loading, both with an equivalent static load approach and with the response spectrum approach, has shown that the static approach could not simulate the higher modal contributions; nor could it simulate the inertial floor torques caused by the first mode of vibration.

Tremblay & Poncet<sup>8</sup> studied the influence of mass irregularity on building seismic response for an eight storey concentrically braced steel frame with different setback configurations using the equivalent static force procedure and the response spectrum analysis method. The study revealed that the analysis of irregular structures could be improved by using dynamic analysis.

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, explosive 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 the 3D nonlinear dynamic analyses of typical high-rise buildings under blast loading. These buildings have been designed for normal (dead, live and wind) loads, with obvious deficiencies

and vulnerabilities to blast attack. The influence of the setbacks on the lateral load response due to blasts in terms of peak deflections, accelerations, inter-storey drifts and bending moments at critical locations (including hinge formation) is investigated.

## METHODS AND MATERIALS

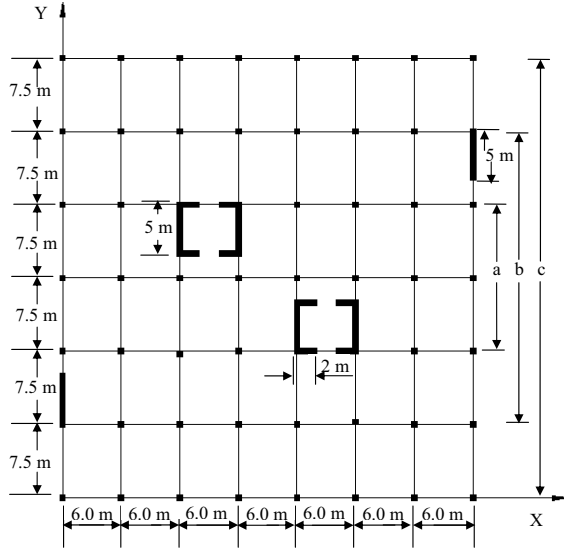
*Objective:* The intent of this study is to analyze the relative performance of typical 20 storey reinforced concrete buildings with and without setbacks subjected to blast loading caused by a close-in surface explosion.

*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 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 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), adding setbacks at different levels and adding perimeter shear walls, 12 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. 11 and 12 had very deep setbacks and are defined only in Table 1. Building Nos. 11 and 12 are rather unrealistic in layout, and chosen only to explore the theoretical progression of setback parameters.

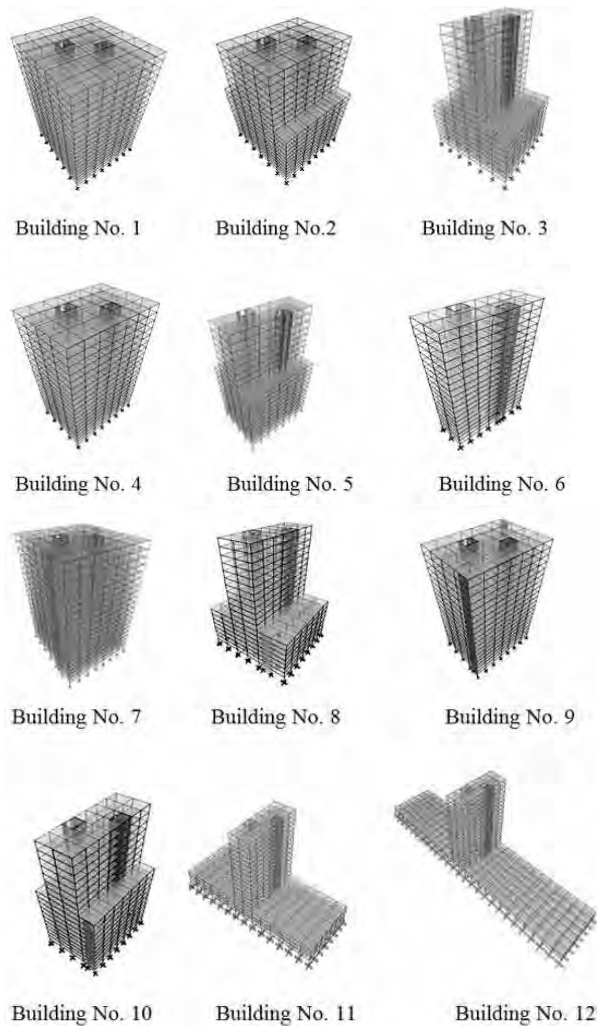
In Table 1, Type A refers to buildings with only a centre core. Type B is provided with two additional shear walls on the perimeter as shown in Figure 1. Such perimeter shear walls are often used to increase lateral load resistance and/or to create a good distribution of

**Table 1:** Building configuration guide (see Figure 1)

Building ID number	Base dimension	Top dimension	Number of storeys exposed to blast	Type A/B
1	c	c	20	A
2	c	b	12	A
3	c	a	8	A
4	b	b	20	A
5	b	a	12	A
6	a	a	20	A
7	c	c	20	B
8	c	a	8	B
9	b	b	20	B
10	b	a	12	B
11	d = 105m	a	4	A
12	e = 195m	a	2	A



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



**Figure 2:** 3D models of buildings

lateral load resisting elements. It should be noted that the shear walls will be included only in buildings with a depth of at least “b” (see Figure 1); they extend vertically either to the top of the building or to the setback level, depending on the depth of setback. Computer generated 3D models of all the buildings are shown in Figure 2.

The storey level and the depth of the setbacks in setback buildings were selected to ensure that the tower part is completely protected from the blast. 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<sup>th</sup> storey and 600 mm x 600 mm beyond that. The column dimensions in the bases of Building Nos. 11 and 12 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<sup>2</sup>, a Young’s modulus of 24 kN/mm<sup>2</sup>, a Poisson’s ratio of 0.2, and a density of 24 kN/m<sup>3</sup>.

*Static analysis:* A static analysis was carried out on 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 SAP 2000 to conform to UBC 97 criteria. Grade 30 concrete and a reinforcement yield strength of 460 MPa were used as material strengths.

*Modal analysis:* A modal analysis was performed and mode shapes examined. In the modal analysis run, the first 12 modes were extracted along with their frequencies. To get the lateral translational mode participation for buildings, modes up to a maximum of the 6<sup>th</sup> mode had to be considered (in Building No. 4). When designing high-rise buildings it is often necessary to consider more modes than just the fundamental in order to account for 90% of the modal mass<sup>9</sup>. To get 90% mass participation, it was necessary to go up to a maximum of the 9<sup>th</sup> mode, in Building No. 4. This was due to the asymmetric nature of the structural model, in which there could be significant torsion. As such the integration time step had to be reduced to 0.001 s to get convergence.

*Blast analysis:* The two equally important parameters that directly influence the blast loading on a structure are the charge weight and the standoff distance. The charge weight can be expressed in terms of an equivalent mass of TNT. Ambrosini *et al.* suggest that 200-500 kg of TNT corresponds to the medium range of terrorist attacks to buildings<sup>10</sup>. For most civilian buildings situated in urban settings large standoff distances are unattainable. These buildings will be exposed to more localized, high intensity blast pressures.

When an explosion occurs at or very near the ground surface it is treated as a hemispherical surface burst. In the majority of cases, terrorist activity has occurred in built-up areas of cities, where devices are placed on or very near the ground surface<sup>11</sup>. Kingery and Bulmash<sup>12</sup> have developed equations to predict air blast parameters from spherical air bursts and from hemispherical surface bursts. These equations are widely accepted as engineering predictions for determining free-field pressures and loads on structures<sup>13</sup>.

According to Yandzio and Gough,<sup>11</sup> in a small scale explosion that is often characterized by a short loading duration, blast loading is considered to act only on the front face of the building. It is usually adequate to assume that the decay of blast overpressure is linear. For the positive overpressure phase, a simplification is made where the impulse of the positive phase of the blast is preserved and the decay of the overpressure is assumed to be linear<sup>11</sup>.

The non-planar nature of the air-blast wave is important in a close range explosion. Here the assumption of a planar incident wave front is not applicable (as the explosion is close by and the building is tall). Hence, the effect of incident angle on the reflective impulse is significant. For a particular angle of incidence  $\alpha$ , the reflected impulse  $i_{ra}$  can be evaluated using the equation proposed by Lorenz<sup>14</sup>.

$$I_{ra} = i_s (1 + \cos \alpha - 2 \cos^2 \alpha) + i_r \cos^2 \alpha$$

where  $i_s$  = impulse of incident wave; and  
 $i_r$  = impulse of normal reflected wave (zero angle of incidence)

In this study it is considered that the buildings are subjected to a surface blast that is equivalent in yield to 500 kg of TNT at a standoff distance of 10 m and symmetrical with respect to the blast loaded face. The 500 kg charge weight marks the upper boundary of TNT weight used in the medium range of terrorist attacks to buildings<sup>10</sup>; the 10 m standoff distance is around the largest that is practically possible in urban settings. The direction of interest is the Y direction of the building (see Figure 1), along which there are 8 in-plane frames.

For dynamic analysis of structures, the blast effects are most conveniently represented by a loading-time history that is applied to the structural members as transient loading<sup>15</sup>. The magnitude and the pressure-time history of the blast load were calculated using the empirical equations given by Kingery and Bulmash<sup>12</sup>.

It was assumed that time varying triangular forces were acting on each beam-column joint on the front face of the building. These pulses have zero rise time and decay linearly as shown in Figure 3. Blast loads were calculated separately for each joint of the front face of the building, taking into account the distance to each joint from the source of explosion and the angle of incidence. The variation in the time of arrival of the blast waves at various points, depending on the distance to the joint, was also considered in constructing and applying loading functions. Loading function durations varied in the range 6 ms to 35 ms, depending on the standoff and angle of incidence value for the joint of interest.

According to TM5-1300,<sup>16</sup> the effects of damping are hardly ever considered in blast design because (i) damping has very little effect on the first peak of response, which is usually the only cycle of response that is of interest; (ii) the energy dissipated through plastic deformation is much greater than that dissipated by normal structural damping; and (iii) ignoring damping is a conservative approach. Hence, damping was not included in the numerical models.

*Computer modeling and analysis:* Computer modelling of the buildings was performed using the finite element software SAP2000 (Non-linear version 8)<sup>17</sup>. The 20 storey reinforced concrete buildings modelled were wall-frame structures composed of columns, beams, slabs

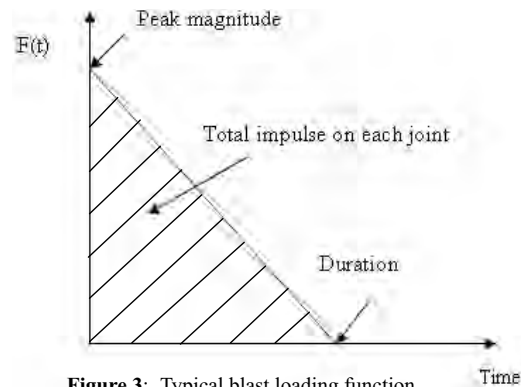


Figure 3: Typical blast loading function

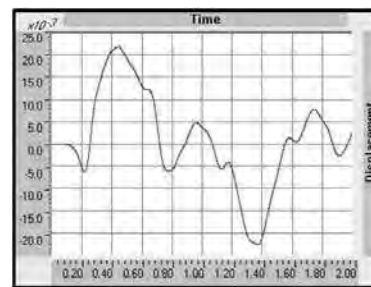


Figure 4: Top displacement time history for Building No. 2

and shear walls, having a shear core in the middle. The columns and beams were modelled as frame elements while the slabs and shear walls were modelled as shell elements. The columns were assumed to be fixed at their bases. A detailed three-dimensional model was employed because of the geometrical non homogeneity of the buildings, and the asymmetry of structural elements. The 3-dimensional models of the complete buildings were created using SAP2000. This software is able to represent material non-linearity of frame elements to model yielding and post-yield behaviour through plastic hinges. Non-linear representation of the columns and beams was employed to accommodate simulation of plastic hinges. Moment hinges were assigned for beam elements at the two ends. To account for the axial force - biaxial moment interaction, coupled axial force and biaxial moment (PMM) hinges were assigned to column elements at the two ends of columns. Coupled PMM hinges yield depending on the interaction of axial force and bending moments at the hinge location<sup>17</sup>. Default hinge properties are based on Applied Technology Council, USA (ATC)-40<sup>18</sup> and Federal Emergency Management Agency, USA FEMA-273<sup>19</sup> criteria<sup>17</sup>. Preliminary analysis runs using distributed blast pressures on the front face beams and columns indicated that mid height hinges in front face columns would not be formed before the top and bottom hinges<sup>20</sup>.

The most general approach for solving the dynamic response of structural systems is direct numerical integration of the dynamic equilibrium equations. For most real structures which contain stiff elements, a very small time step is required to obtain a stable solution<sup>21</sup>. Reducing the integration time step will increase the accuracy, and generally a time step size which is less than 0.01 times the dominating period is selected. Building No. 4, which requires the highest number of modes to

get the lateral translational mode has the relevant period value (period of the 6<sup>th</sup> mode) of 0.54 s. Hence the time step had to be of the order of 0.005 seconds. But when the analyses were run with step sizes 0.005 s and 0.001 s, the results were not in good agreement. Hence to get consistent results for the 3D building models, the time step had to be reduced to 0.0005 s. The non-linear direct integration time history analyses were run for a duration of 2 s with 4000 time steps for all the buildings, and encompassed one cycle of structural response.

## RESULTS AND DISCUSSION

### Overall response results

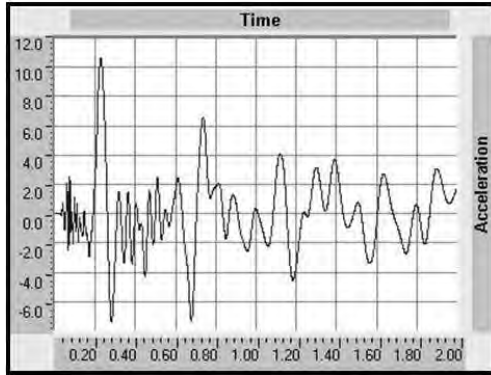
The building response is characterized by that of the fourth in-plane frame from the left hand side (see Figure 1), as this was found to have the maximum response. The maximum response values of all buildings obtained from the analyses are presented in Tables 2 and 3. Typical time histories obtained for top lateral displacement and acceleration are shown in Figures 4 and 5 respectively.

For all building configurations, the maximum acceleration response occurs immediately after the blast, while the maximum displacement occurs at a later stage in the time history. Careful observation of displacement time histories reveals that buildings without setbacks have a more regular variation of displacement than buildings with setbacks. The irregularity is more prominent for Building Nos. 11 and 12 that have deeper setbacks. Lateral acceleration histories do not display a significant difference.

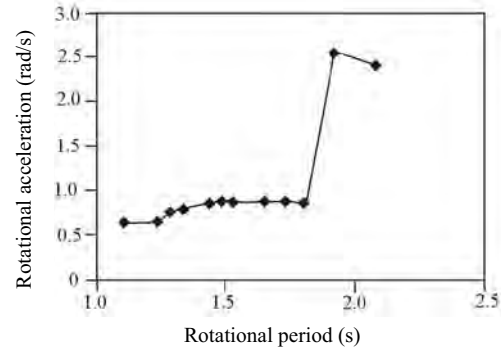
The variation of rotational acceleration with the rotational period of the buildings under consideration

**Table 2:** Maximum response values for Building Nos. 1 - 6

Building Number	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
Base depth (m)	45	45	45	30	30	15
Tower depth (m)	45	30	15	30	15	15
Perimeter shear walls (Y/N)	N	N	N	N	N	N
Fundamental period (s)	2.096	1.719	1.499	1.929	1.518	1.729
Top displacement (mm)	27.68	22.61	21.79	36.4	31.4	61.0
Time taken (s)	(1.388)	(1.369)	(1.214)	(0.4160)	(0.551)	(0.4415)
Top acceleration (m/s <sup>2</sup> )	12.23	11.19	12.18	13.17	13.11	29.4
Time taken (s)	(0.2275)	(0.225)	(0.219)	(0.2135)	(0.2040)	(0.2450)
Inter-storey drift (mm)	5.04	4.67	4.95	6.72	5.96	7.70
No. of column hinges	60	61	61	49	55	72
No. of beam hinges	0	0	0	7	2	71



**Figure 5:** Top acceleration time history for Building No. 2



**Figure 6:** Variation of rotational acceleration (rad/s<sup>2</sup>) with rotational period (s)

**Table 3:** Maximum response values for Building Nos. 7 - 12

Building Number	No. 7	No. 8	No. 9	No.10	No. 11	No. 12
Base depth (m)	45	45	30	30	105	195
Tower depth (m)	45	15	30	15	15	15
Perimeter shear walls (Y/N)	Y	Y	Y	Y	Y	Y
Fundamental period (s)	1.974	1.447	1.821	1.415	1.697	1.716
Top displacement (mm)	25.97	20.84	33.97	30.3	9.74	5.63
Time taken (s)	(1.306)	(1.195)	(0.3765)	(1.16)	(0.399)	(0.439)
Top acceleration (m/s <sup>2</sup> )	11.21	11.13	12.62	12.05	9.276	11.44
Time taken (s)	(0.2205)	(0.2115)	(0.1895)	(0.213)	(0.238)	(0.3455)
Inter-storey drift (mm)	4.96	4.35	5.72	5.44	3.17	2.58
No. of column hinges	52	54	42	46	26	103
No. of beam hinges	0	0	5	2	0	0

**Table 4:** Variation of maximum rotational acceleration with rotational period

Building Number	First rotational mode	First rotational period (s)	Rotational acceleration (rad/s <sup>2</sup> )
5	3rd mode	1.121	0.63
10	3rd mode	1.258	0.65
8	3rd mode	1.293	0.74
3	3rd mode	1.325	0.78
2	3rd mode	1.494	0.88
11	3rd mode	1.541	0.89
12	2nd mode	1.580	0.85
6	2nd mode	1.678	0.87
9	2nd mode	1.754	0.88
7	2nd mode	1.819	0.84
4	1st mode	1.929	2.53
1	1st mode	2.096	2.38

is shown in Figure 6. It is seen that there is a general trend of an increase in rotational acceleration with the increase in rotational period (see Table 4). Buildings having a rotational mode as the fundamental mode (i.e. Building Nos. 1 and 4) have comparatively much higher rotational accelerations; those with the rotational mode as the second mode have somewhat higher rotational accelerations than those with the rotational mode as the third one.

Having a setback for a given set of base dimensions reduces the fundamental period of the building. This decrease is clearly seen in the sequence of Building Nos. 1, 2 and 3 in Table 2. Adding perimeter shear walls reduces the fundamental period of the building. This is true for all four comparisons of buildings with and without perimeter shear walls, namely Building Nos. 1 and 7; Nos. 3 and 8; Nos. 4 and 9; and Nos. 5 and 10.

A setback that protects the tower part from blast loading results in better performance with respect to maximum displacement, acceleration and inter-story drift, compared to a building with the same base but no setback. No significant difference is seen with regard to the number of hinges formed. The best relative performance with respect to peak top displacement is seen in Building No. 3, when compared to Building No. 1 (having identical base dimensions without any setback) with 21.3 % reduction. Building No. 2, which has a wider and shorter tower part, gives better performance with respect to top acceleration when compared to Building No. 3 with a narrower and taller tower part, presumably because the former tower is stiffer.

When Building Nos. 1, 4 and 6 are compared, all without setbacks but having different depths, Building No. 6 gives the worst performance in all the response parameters investigated, obviously due to the fact that it is the most flexible and slender building. It gives the

largest number of hinges, with their formation distributed throughout the height, in contrast to the far fewer hinges in the other two buildings that have hinge formation concentrated in the first three levels. The decrease in hinge formation in Building No. 4 compared to Building No. 1 is because the former has two bays less than the latter. Other than for this apparent anomaly in hinge formation, Building No. 1 (i.e. the widest of the three) gives the best performance in other response parameters.

Of great importance is the comparison of response for Building Nos. 6, 5, 3, 11 & 12, all of which have narrow towers with an increasing degree of “shelter” from projecting bases. Recall that the projecting base depths and heights have been defined such that the tower is completely shielded from the blast. Table 5 illustrates these comparisons. It is interesting to note how the maximum response values decrease dramatically with the decrease in the number of exposed storeys in these buildings. The best performance is shown in Building No. 12, in which the least number of storeys are exposed to the direct blast loading. Compared to Building No. 6 it has a 90.8% reduction in top displacement. However, its top acceleration is somewhat greater than that of Building No. 11.

The adding of perimeter shear walls results in slightly better performance with respect to maximum displacement and acceleration. The best performance is recorded in Building No. 7 compared to Building No. 1, with a maximum displacement reduction of 6.2% and maximum acceleration reduction of 8.3%. Adding perimeter shear walls also gives better performance with respect to the maximum inter-storey drift, yielding a maximum reduction of 14.9% when Building No. 9 is compared with Building No. 4 (both without setbacks and having identical dimensions). A reduction in hinge formation is also seen through the addition of shear walls in all comparisons.

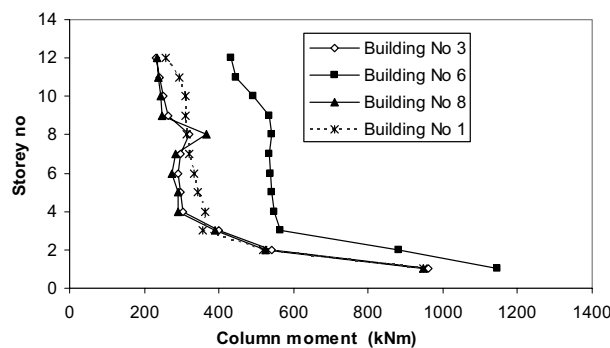


Figure 7: Comparison of column moment variation near setback level

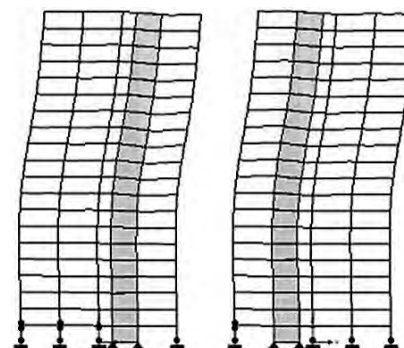


Figure 8: Hinge formation in 4th and 5th in-plane frames in Building No. 4 (Blast on LHS)

**Table 5:** Response for buildings with narrowest tower and increasing depth of base

Building number	No. 6	No. 5	No. 3	No. 11	No. 12
Base depth (m)	15	30	45	105	195
Number of exposed floors	20	12	8	4	2
Top displacement (mm)	61.0	31.4	21.79	9.74	5.63
Top acceleration (m/s <sup>2</sup> )	29.4	13.11	12.18	9.276	11.44
Interstorey drift (mm)	7.70	5.96	4.95	3.17	2.58
% Top displacement reduction compared with Building No. 6	0	48.5	64.38	4.0	90.8
% Top acceleration reduction compared with Building No. 6	0	55.4	58.6	68.4	61.1
% Inter-storey drift reduction compared with building No. 6	0	22.6	35.7	58.8	66.5

It should be noted that the greatest top displacement of 61.0 mm is 1/1147 of building height and the largest inter-storey drift of 7.70 mm is 1/454 of storey height – see Building No. 6 in Table 2. Also, all hinges formed were in the strain hardening region and did not constitute danger of collapse. This shows therefore that 20 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 blast that is equivalent to 500 kg TNT at a standoff distance of 10 m. Some attention would perhaps need to be paid to detailing, in order to enhance ductility.

### Localized shear wall effects

Figure 7 shows the variation of column moments in the vicinity of the setback in Building Nos. 3 and 8, and also Building Nos. 1 and 6, having no setback. The column considered is the middle column of the fourth in-plane frame from the left hand side in Figure 1. Of special interest is the column moment variation in Building Nos. 3 and 8 near the setback level. The abrupt increase in column moment near the setback level demonstrates the complex moment variation taking place at the setback level, compared to the uniform variation seen in Building Nos. 1 and 6, representing uniform buildings without any setback. A similar variation was seen in the column shear distribution too. Building Nos. 3 and 8 both have identical configurations with setbacks, the only difference being the addition of two perimeter shear walls in Building No. 8. When the column moments in Building Nos. 3 and 8 are compared, the abrupt moment change near the setback level is more significant in Building No. 8. The

reason would be the sudden curtailment of the perimeter shear wall in Building No. 8 above the setback level, creating a more abrupt change of rigidity in the lateral load resisting system in that building, compared to the rigidity change in Building No. 3.

Figure 8 shows two in-plane frames of the same building (Building No. 4), having the same standoff distance from the explosion. Comparison of these frames reveals that the in-plane frame having its shear wall closer to the explosion suffers less damage in terms of the number of hinges formed, when compared to the frame having its shear wall further away from the explosion.

### CONCLUSION

Based on the results of the analyses, the following major conclusions are made for typical 20 storey reinforced concrete buildings subjected to a blast equivalent to 500 kg TNT (the probable maximum from a medium range terrorist attack) at a standoff distance of 10 m (the likely maximum in an urban environment).

- (1) Buildings having setbacks that protect the tower part from blast loading show better performance in terms of peak displacement, peak acceleration and inter-story drift when compared to buildings without setbacks.
- (2) The best performance in terms of peak displacement, peak acceleration and inter-storey drift is achieved in the configuration in which the least number of floors are exposed to the direct blast loading.

(3) 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 also cut off at the setback level.

(4) Rotational accelerations depend, in general, on the periods of the rotational modes, high accelerations being obtained when the rotational mode is the fundamental one.

(5) Frames having shear walls closer to the explosion suffer less damage in terms of the number of hinges formed, when compared with similar frames having shear walls further away from the explosion.

(6) 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 blast that is equivalent to 500 kg TNT at a standoff distance of 10 m.

---

## References

1. Arnold C. & Reitherman R. (1982). *Building Configuration and Seismic Design*. Wiley, New York.
2. International Conference of Building Officials (1988). *Uniform Building Code: UBC 1988*. ICBO, Whittier, California, USA.
3. Standards Association of New Zealand (1984). *Code of Practice for General structural Design Loading for Buildings: NZS 4203*. Wellington, New Zealand.
4. Nassiri E. (1993). Review of code provisions and current practices for inelastic analysis and design of buildings subjected to earthquakes. *Research Report 93-4*, Physical Infrastructure Centre, Queensland University of Technology, Queensland, Australia.
5. Paz M. (1997). *Structural Dynamics: Theory and Computation*. Chapman & Hall, New York.
6. Rutenberg A. & Dickman Y. (1993). Lateral load response of setback shear wall buildings. *Engineering Structures* **5**: 47-54.
7. Tso W.K. & Yao S. (1994). Seismic load distribution in buildings with eccentric setback. *Canadian Journal Civil Engineering* **21**: 863-871.
8. Tremblay T. & Poncet L. (2005). Seismic performance of concentrically braced steel frames in multistory buildings with mass irregularity. *Journal of Structural Engineering* **131**: 1363-1375.
9. Wilkinson S.M. & Hiley R.A. (2006). A non-linear response history model for the seismic analysis of high-rise framed buildings. *Computer and Structures* **84**: 318-329.
10. Ambrosini D., Luccioni B., Jacinto A. & Danesi R. (2005). Location and mass of explosive from structural damage. *Engineering Structures* **27**: 167-176.
11. Yandzio E. & Gough M. (1999). *Protection of Buildings Against Explosions*. The Steel Construction Institute, Berkshire, UK.
12. Kingery C.N. & Bulmash G. (1984). Air blast parameters from TNT spherical air burst and hemispherical surface burst. Technical report ARBRL-TR-02555, U.S. Army Armament Research and Development Center, New Jersey, USA.
13. Remennikov A.M. (2003). A review of the methods for predicting bomb blast effects on buildings. *Journal of Battlefield Technology* **6**: 1-6.
14. Lorenz R. (1982). *Derivation of the reflected impulse as a function of the angle of incidence*. Naval Surface Weapons Centre, U.S. Armament Research & Development Center, Dover, New Jersey, USA.
15. Besahara F.B.A. (1994). Modeling of blast loading on above ground structures-1, general phenomenology and external blast. *Computer and Structures* **51**: 585-596.
16. U.S. Department of the Army TM5-1300 (1990). Structures to resist the effects of accidental Explosions: Technical Manual 5-1300. U.S. Department of the Army, USA.
17. SAP 2000 (1997). *Integrated Finite Element Analysis and Design of Structures*, Computers and Structures, Inc. Berkeley, California, USA.
18. Applied Technology Council ATC-40. (1996). Seismic evaluation and retrofit of concrete buildings, Volume 1, ATC-40 Report. Applied Technology Council, Redwood City, California, USA.
19. Federal Emergency Management Agency 273. (1997). *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*. Developed by the building seismic safety council for the federal emergency management agency, Washington, D.C.
20. Jayatilake I.N. (2008) Influence of symmetric setbacks on the performance of high-rise buildings under blast and earthquake loading. *PhD thesis*, University of Moratuwa, Katubedda.
21. Wilson E.L. (2002). *Three-Dimensional Static and Dynamic Analysis of Structures*. Computers and Structures Inc., Berkeley, California, USA.