13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1212

SURVEY AND ASSESSMENT OF SEISMIC SAFETY OF MULTI-STOREYED BUILDINGS IN GUWAHATI, INDIA

Pradip SARKAR[1], Biju Kumar PATIR[2], Devdas MENON[3]

SUMMARY

Guwahati city falls in the highest seismic risk zone (Zone V) in India. Some severe earthquakes have occurred in this region in the past (notably in 1897 and 1950). Until about 1950 or so, the typical construction in the entire northeast region comprised simple single storey 'Assam type' structures which possess good earthquake resistant features. With growing urbanization, RC framed construction has become the standard construction practice in Guwahati during the course of the last five decades. Most of the high-rise constructions in Guwahati have come up only in the past 12 years, and they have not yet been tested for their resistance to a high intensity earthquake. The present paper assesses the safety of such buildings in Guwahati in the event of a severe earthquake of intensity prescribed in IS: 1893:2002 [1].

A field survey of multi-storeyed buildings in Guwahati was carried out and a preliminary assessment of seismic vulnerability made. From the field survey, information was collected on the current design and construction practices, the quality of materials and constructions, soil properties and general perception of the architects, engineers and builders at Guwahati. Most of the structures are ordinary moment-resisting frames with soft ground storey. In very few buildings, RC shear walls have been adopted as lateral load resisting systems. Ductile detailing has been incorporated only in a few of the recently constructed buildings.

Three typical existing multi-storeyed buildings are analysed using the capacity spectrum method in order to assess their vulnerability. It is found that under seismic loading the demands on many members exceed their capacities and the performance objective is not satisfied. The results provide useful information relating to identification of deficient members that need to be retrofitted.

INTRODUCTION

The vulnerability assessment of multi-storeyed buildings under future earthquake in any region is a complex problem. A detailed evaluation requires a detailed survey for the area under study and detailed analyses. In this paper, the study has been focused on Guwahati, the capital city of Assam. It is the political, commercial and socio-cultural centre of the north-eastern part of India. The entire region is

1 Graduate Student, Structural Engineering Division, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India.

2 Formerly Graduate Student, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India.

3 Professor, Structural Engineering Division, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India.

located close to the foothills of the Himalayas. The Indian sub-continent, which forms part of the Indo-Australian plate, is known to be pushing against the Eurasian plate along the Himalayan belt, and the neighbouring regions which include the city under consideration fall in the highest seismic risk zone (Zone V) in India. Some severe earthquakes have occurred in this region in the past (notably in 1897 and 1950). But most of the high-rise constructions (3-10 storey buildings) in Guwahati have come up only in the past 12 years, and have not yet been tested for their resistance to a high intensity earthquake. However, the Gujarat earthquake (26th January, 2001) exposed the seismic vulnerability of such multi-storeyed buildings in the Gujarat region. In this context, it is necessary to evaluate the seismic performance of existing multi-storeyed buildings in Guwahati and other cities in India. Surveys and analyses conducted in Guwahati, as reported in this paper, have revealed that most of the existing reinforced concrete (RC) framed and masonry load-bearing structures do not meet the earthquake resistant requirements of latest Indian codes (IS1893: 2002 [1], IS13920: 1993 [2], IS 4326: 1993[3]).

Surveys were conducted in order to collect the details of existing multi-storeyed buildings, such as detailed drawings of the buildings and construction practices. Several buildings have been analysed to estimate their seismic vulnerability. In this paper, three case studies are presented to demonstrate the serious inadequacies in the current RC framed building design practice in this region

EVALUATION OF BUILDING CONSTRUCTION IN GUWAHATI

It is believed that until about 1850 or so, the typical construction in the entire northeast region comprised simple single storey structures, which possess good earthquake resistant features. The roof was light (and sloping), made of thatch. The structure was framed and made of timber, with the trunks of trees used to form the pillars, and wooden logs framing into the pillars at the eaves' level. The walls were made of spliced bamboo, plastered with mud. Construction using brick masonry was confined to a few important structures. The use of brick masonry with corrugated iron sheet roofs gained popularity due to its superior resistance to accidental fires.

However, the great Assam earthquake of 1897 had flattened almost all the brick masonry structures in the township of Guwahati. Learning from this devastation, the so-called 'Assam type' construction (Figure 1) evolved. It is a combination of the traditional timber framed construction with half brick masonry cladding. The single storey 'Assam type' construction became a common structural and architectural form, which was not only adequately earthquake resistant but also suitable for the prolonged rainfall in the region. The Assam earthquake of 1950 left the 'Assam type' single storey constructions mostly untouched, while causing severe damage to the few buildings that had two or three stories.

With growing urbanisation, the 'Assam type' single storey constructions were replaced with multi-storey constructions to develop residential, commercial and institutional establishments. The earliest multi-storeyed framed constructions to have come up in Guwahati belong to the period 1965-1970. However, it is only since 1990 that the construction of multi-storeyed buildings had gained momentum.

FIELD SURVEY

Field survey involves site visits to physically inspect buildings and search for architectural and structural drawings and other documents that may describe the structural characteristics of the building. It also includes meeting practising engineers, architects and organizations involved in the construction in order to collect information on the construction practices and the quality of construction. The present composition of buildings in Guwahati is as follows. Out of 100 buildings, about 50 are single-storey 'Assam type' construction, which is known to have good seismic resistance. About 25-35 buildings are of 2-4 storeys and about 10 are of 5-6 storeys, using the RC framed construction. Buildings with higher storeys do exist, but they are relatively rare and the typical storey height varies from 2.6 m to 3.2 m. With the experience of the past earthquakes, even the single storey construction in Guwahati is made of RC framed construction with plinth beams. In few buildings, RC shear walls have been adopted. The infill walls are made of brick masonry, but they are only half brick walls (125 mm thick after plastering), and are considered non-load bearing. All walls are invariably provided with RC bands at lintel level. RC columns are provided at corners and junctions of partition walls, and they are tied together with RC beams at the slab levels. Although conceptually this framing arrangement is perfect, a possible deficiency lies in the detailing of the beam-column joints. Also, the sizes of the column and beam members, and their reinforcements, are not always based on detailed analysis and design. Foundations are usually in the form of isolated footings or combined footings. Rafts are occasionally used. Under-reamed piles (up to 8-15 m depth) with pile caps and tie beams are also used in some cases. The soil stratum is, in general, silty-clay, with an average bearing capacity of about 120 kN/m^2. Ground improvement using stone columns (without vibro-technique) has also been done in some cases to ensure that the bearing capacity is not less that 100 kN/m^2.

Figure 1: Typical Modern Assam Type House in Guwahati

Number of so-called “non-engineered” multi-storeyed buildings exists in Guwahati. Figure 2 shows a building located in a “commercial” area, which initially existed as a four-storey building and later expanded up to seven storeys. The columns are fairly slender, and it is fairly certain that the building has poor lateral load resistance. However, there is increasing awareness among professional builders regarding the need to adopt properly engineered construction for multi-storeyed buildings. But most of these recent high-rise engineered buildings have open ground storey, and the increased vulnerability of such ‘soft storey’ construction has not been understood. In some cases, isolated multi-storeyed buildings are positioned so close to one another as to render them vulnerable to possible ‘pounding’ in the event of a high intensity high duration earthquake.

Physical inspection of the individual buildings was helpful for preliminary evaluation of the building as well as for detailed analysis, with regards to the level of concrete deterioration. When architectural drawings were not available, locations of infill walls were postulated. Similarly, in the absence of geo-technical data, the type of soil was assumed.

Figure 2: Addition of extra stories

Rebar detailing was not complete in the available structural drawings. Particularly, details of the length and location of beam and column rebar lap splices were not mentioned in any of the drawings, and so it was assumed that the buildings, as constructed, did not comply with the ductile detailing requirements of IS 13920: 1993 [2].

CASE STUDIES

The three buildings presented in this paper are residential, ordinary moment resisting RC framed buildings in Guwahati. Figure 3 shows the typical floor plans of the three buildings, B1, B2 and B3. Building B1 is a Ground+4 storey building (15.7 m height from the ground level), Building B2 is a Ground+5 storey building (18.9 m height from the ground level) and Building B3 is a Ground+6 storey building (20.7 m height from the ground level). In all of the buildings, infill walls are absent in the ground storey (open ground storey) for car parking. Plan dimensions of Buildings B1, B2 and B3 are $13.95m \times 25.2m$, $17.5m \times 24.7m$ and $11.95m \times 14.4m$ respectively. M15 grade of concrete and Fe 415 grade of steel were used for the construction of all three buildings.

As visual inspection did not reveal concrete deterioration in any of these buildings, the designed characteristic strength of M15 concrete was used in the analyses. The floor slabs in a building were assumed to act as rigid diaphragms. This assumption ensure integral action of all frames. The slabs are 150mm thick for all the floor levels in all three buildings. The infill-wall thickness for all the buildings was assumed to be 120 mm for both the exterior and interior walls, as is the practice in Guwahati. The subsoil was assumed to be medium (Type II), as geotechnical data were not available.

Preliminary Evaluation

The buildings were checked by the method of rapid visual screening given in FEMA 154 [4]. The screening procedure is based on a “sidewalk survey” approach and a data collection form. The procedure is derived from seismic hazard analysis and provides basic structural hazard scores and performance modifiers, which are dependent on seismic hazard of the site, building system and features affecting seismic performance. The data collection form was suitably modified to match Indian construction practice. The hazard scores of FEMA 154 [4] were used without modification. Research on hazard analysis is necessary to modify these scores as per the Indian construction practice. For all of these buildings, rapid screening results indicate the requirement of detailed analysis.

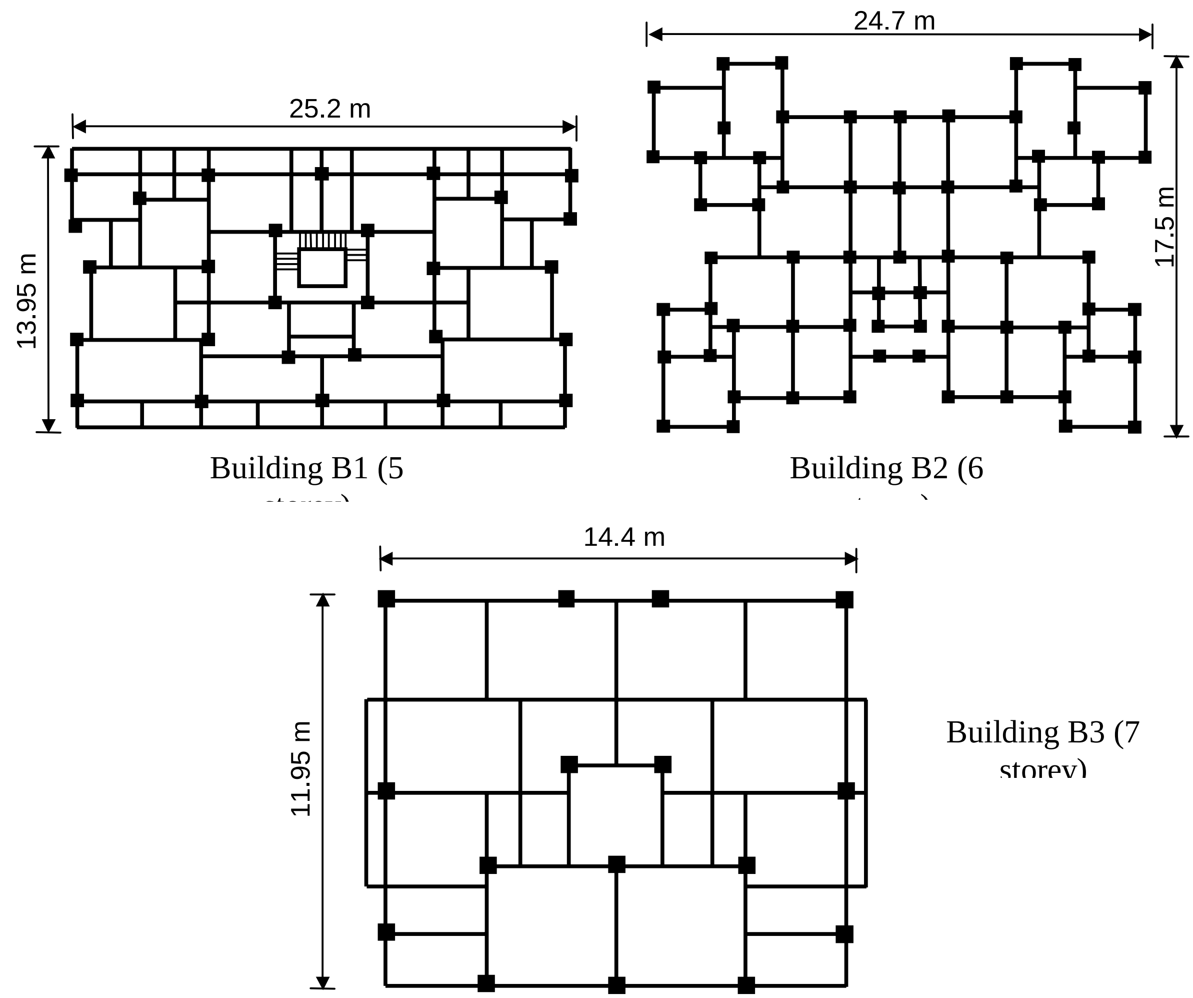

Figure 3: Typical floor plans of Building B1, Building B2 and Building B3.

Before undertaking a detailed analysis of the buildings, "evaluation statements" modified from FEMA 178 [5] were assessed. The evaluation statements are based on the configuration and the type of the building frame. The configuration includes structural redundancy, plan and vertical regularities, whereas the type of frame refers to the strong-column weak-beam frame, parallel frame system, and continuity of the frame.

The evaluation statements pose a variety of queries, which if true, suggest that the building incorporates seismic resistance features. When the statements are false, additional analysis is recommended to examine potential seismic deficiencies. A simple method to calculate the storey stiffness and strength was devised. It showed that in Buildings B1 and B2, the ground storey has less than 60% stiffness and strength as compared to that of the first storey because of the absence of infill walls. Both of these two buildings indicate extreme soft and weak storey. For Building B3, only a few of the walls in the first and the upper storeys lie in a lateral load resisting frame. Hence, their stiffness and strength were not included in the calculated storey stiffness and strength. Thus, any storey has stiffness and strength more than 90% of those of the adjacent storeys.

Detailed Analysis

All three buildings were analysed using equivalent static method (linear static method) and response spectrum method (linear dynamic method) according to IS 1893:2002 [1]. Pushover analysis (non-linear static method) was also carried out. The pushover analysis provides an insight into the structural aspects which control the performance during earthquakes. It also provides data on the strength and ductility of a building. The analyses were done using the finite element analysis software, SAP2000. All the three analyses expose various design weaknesses that are present in a building.

A performance based approach was adopted to evaluate the vulnerabilty of the buildings. The performance based approach identifies building performance level under an anticipated earthquake level. The building performance is broadly categorized under the levels of (a) collapse prevention, CP, (b) life safety, LS, and (c) immediate occupancy, IO. The two commonly used earthquake levels are design basis earthquake (DBE) and maximum considered earthquake (MCE). For the present buildings, LS under DBE was selected as the basic safety objective.

Structural Modelling

The analytical models of the buildings include all components that influence the mass, strength and stiffness. The non-structural elements and components that do not significantly influence the building behaviour were not modelled. Beams and columns were modelled as frame elements with the centerlines joined at nodes. Rigid offsets were provided from the nodes to the faces of the columns or beams. The stiffness for columns and beams were taken as $0.7EI_g$ accounting for the cracking in the members and the contribution of flanges in the beams.

The weight of the slab was distributed to the surrounding beams as per IS 456: 2000 [6], Clause 24.5. The mass of the slab was lumped at the centre of mass location at each floor level. This was located at the design eccentricity (based on IS 1893:2002 [1]) from the calculated centre of stiffness. Design lateral forces at each storey level were applied at the centre of mass locations independently in two horizontal directions (*X*- and *Y*- directions). When an infill wall is located in a lateral load resisting frame the stiffness and strength contribution of the infill wall were considered by modelling it as an equivalent compression strut (Smith [7]). When infill walls are not located in a frame the walls were not modelled as equivalent strut. For Building B3 none of the infill walls was modelled. The weight and mass of all the infill walls were of course considered in the models. Staircases and water tanks were not modeled for their stiffness but their masses were considered in the static and dynamic analyses. Building B1 has a lift core made up of RC walls. But it was not considered in the model because it is not integrally connected either to the floor diaphragms or to the lateral load resistant frames. The design spectrum for medium soil as specified in IS 1893:2002 [1] was used for the analyses. The effect of soil-structure interaction was ignored in the analyses. The foundation system for Building B1 is a pile foundation with groups of 2, 3 or 4 piles. In the model, fixity is considered at the top of the pile caps. In the case of Buildings B2 and B3, the foundation system is strip footing with central beams for groups of 2 or 3 columns. In the model, fixity is considered at the top of the central beam.

The building periods of vibration were determined from an eigenvalue analysis using SAP 2000. The periods for the first three modes and their predominant directions of vibration are presented in Table 1. The first ten modes were considered for the dynamic analysis (response spectrum method), which gives more than 99% mass participation in both the horizontal directions. The SRSS method of modal combination was used for analysis.

Table 1: Time periods and predominant directions of vibration: first 3 modes

	Mode 1	Mode 2	Mode 3
Building 1	0.73s (Translation X)	0.69s (Translation Y)	0.38s (Rotation Z)
Building 2	1.33s (Translation Y)	1.15s (Translation X)	0.53s (Rotation Z)
Building 3	0.98s (Translation X)	0.96s (Translation Y)	0.30s (Bending X)

The eigen value analyses reveal that in both the buildings (which have good plan 'regularity'), the torsional mode of vibration has only marginal participation, and as per ATC 40 [8] recommendations, the "pushover analysis" is particularly valid for these structures.

For pushover analysis, beams and columns were modelled with concentrated plastic hinges at the column and beam faces, respectively. Beams have only moment (M3) hinges, whereas columns have axial load and byaxial moment (PMM) hinges. The normalised moment-rotation relations and the acceptance criteria for the hinges were obtained from ATC-40 [8] according to the reinforcement provided and the design shear and axial forces. The moment-rotation relations were taken as symmetric in the positive and negative sides of the bending moment axis. The default ACI 318-95 interaction surface (with ϕ =1) was considered for the column hinge. In the absence of complete data, to be conservative, the beam and column hinges were considered to be non-compliant to IS 13920: 1993 [2] for all the cases. As the shear strengths of all the beams and columns were found to be more than the respective shear demands (from equivalent static and response spectrum methods), no shear hinge was modeled in the frame elements. The equivalent struts were modeled with axial hinges (entire length of the strut was considered as hinge length), that have a brittle load-deformation relation only for compression. Geometric nonlinearity of the structure due to P-Δ effect was considered in the pushover analyses.

Results and Discussion

Design seismic base shears ($\overline{V_B}$) were calculated using IS 1893:2002 in the X- and Y- directions (EQ_x and EQ_y). Base shear from response spectrum analysis (V_B) was calculated from the modal combination of first ten modes (EQ). V_x and V_y are the components in X- and Y- directions, respectively. As V_B was less than $\overline{V_B}$, the seismic force demands in the frame elements from response spectrum analysis were scaled up by a factor equal to the ratio of the two base shears $(\overline{V_B}/V_B)$. Table 2 shows the comparison between V_B and $\overline{V_B}$ for the three buildings.

Table 2: Comparison of Base Shear V_B and $\overline{V_B}$ for the three buildings.

	Building B1		Building B2		Building B3	
	$V_x(kN)$	$V_y(kN)$	$V_x(kN)$	$V_y(kN)$	$V_x(kN)$	$V_y(kN)$
Equivalent Static $(\overline{V_B})$						
EQ_x	3039	3039	1182	1182	1996	1996
Response Spectra (V_B)						
EQ	2092	2170	778	794	1246	1205
$\overline{V_B}/V_B$	1.45	1.40	1.52	1.49	1.60	1.66

For beam sections, positive and negative bending moment capacities and shear capacities were calculated and were compared with the demand. For both the buildings a number of beam sections were found to have deficient flexural capacity. However, all the sections have sufficient shear capacity.The adequacy of each column section for flexure and axial compression was checked using a 3D interaction surface for axial compression and bi-axial bending generated according to IS 456:2000 [6]. The maximum demand from all the load combinations was plotted along with the corresponding interaction surface. A straight line was drawn joining the demand point and the origin. The ratio of the length of the straight line to the distance between the point of intersection of the straight line with the interaction surface and the origin is termed as the capacity factor.

Analysis results show that a number of column sections do not satisfy the code criteria as the capacity factor exceeds one. Shear force demand for each column section was compared to the corresponding shear capacity. For none of the columns, shear demand exceeds shear capacity. For the struts in Building B1 and Building B2, axial force demands exceed the respective capacities in almost all storeys. The storey drift for every storey due to the design lateral force, with partial load factor of 1.0, was calculated. For each building, it satisfies the code limitation of 0.4% of the storey height. The ground storeys of Building B1 and B2 has fairly high drift compared to the other storeys. This indicates exessive ductility demand in the ground storey columns, which may lead to collapse by soft storey mechanism.

Pushover analyses were performed with monotonically and proportionally increasing lateral loads distributed according to IS 1893: 2002 [1], with simultaneous action of gravity load. For each building, pushover analyses were performed independently in the X- and Y-directions. To achieve life safety (LS) performance level under DBE, the target displacement at the roof of each building was taken as 1.2% of the building height. For Building B1 the target displacement was 0.188m whereas for Building B2 and Building B3, it was 0.235m and 0.248m, respectively.

Figure 5 shows the pushover curves for the three buildings. All the buildings have sufficiently large strength as compared to $\overline{V}_B$, but Buildings B1 and B2 have failed to achieve the target displacements prior to the formation of mechanisms. However, Building B3 reached the target displacement marginally.

In a pushover analysis, when the demand spectrum for DBE is plotted along with the capacity spectrum in an Acceleration Displacement Response Spectrum format, the two curves may meet to give a performance point. Capacity spectrum corresponds to the base shear versus roof displacement curve. For Buildings B1 and B2, the pushover analyses in either direction failed to give performance points before the formation of mechanisms, indicating unacceptable performance and need to be retrofitting. It needs to be retrofitted. The flexural hinges in ground storey columns went beyond life safety. But in the case of Building B3, the pushover analyses in both the directions gave performance points, before the mechanisms formed. The drift corresponding to the performance point is well below the ‘life safety’ limit and there is no hinge in the structure at the performance point beyond the life safety. Beyond the performance points, the capacity goes on increasing till the mechanism forms, whereas the demand reduces due to increased damping and equivalent time period. So, the building has achieved life safety performance level under DBE. It does not need any global retrofitting. Some of the frame sections do not satisfy the code criteria according to elastic analysis and these need local retrofitting.

CONCLUSIONS

The paper describes the engineering practices, types of constuction, and seismic awareness of a seismically sensitive city Guwahati in India. It also demonstrates the vulnerability status of the existing buildings through three detailed case studies. Field survey is a primary part of the vulnerability assessment of existing multi-storeyed building and visual inspection was made to ascertain the missing data.

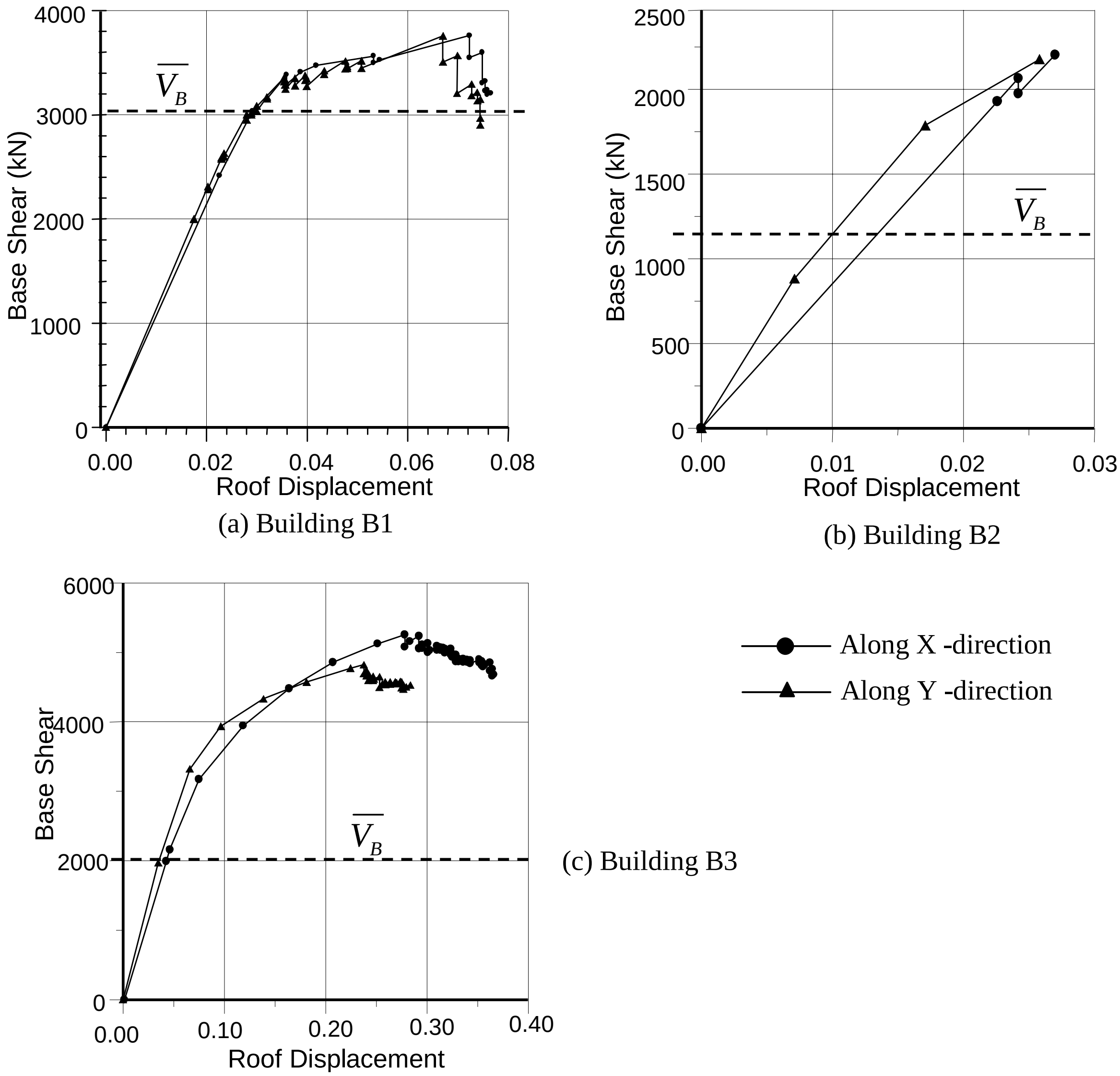

Figure 5: Pushover curves for *X*- and *Y*- direction.

The buildings were analysed using rapid screening method (FEMA 154 [4]) followed by preliminary evaluation (FEMA 178 [5]), detailed analyses using equivalent static method, response spectrum method (IS 1893: 2002 [1]) and pushover analysis (ATC 40 [8]). It was found that none of the three buildings satisfy the requirements of the current IS codes. But the pushover analysis results reveal that while one of the buildings has sufficient strength and ductility at global and local levels, the other two buildings failed to achieve the desirable performance. In general, most of the buildings, especially with open ground storey, need to be retrofitted.

ACKNOWLEDGMENT

The results presented in this paper are based on the work supported by the Department of Science and Technology, Government of India. The authors are grateful for this support.

REFERENCES

1. IS 1893: Part 1 (2002); "Criteria for earthquake resistant design of structures", Bureau of Indian Standards, New Delhi.
2. IS 13920 (1993); "Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces Code of Practice", Bureau of Indian Standards, New Delhi.
3. IS 4326 (1993); "Criteria for earthquake resistant design of structures", Bureau of Indian Standards, New Delhi.
4. FEMA 154 report (1997), "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook" (Second edition)", Federal Emergency Management Agency, Washington DC.
5. FEMA 178 report (1992), "NEHRP Handbook for seismic evaluation of existing buildings", Federal Emergency Management Agency, Washington DC.
6. IS 456:2000; "Plane and Reinforced Concrete-Code of Practice", Bureau of Indian Standards, New Delhi.
7. Smith, S. B. and Carter, C. (1969), "A method of analysis of in-filled frames" Proceedings, Institution of Civil Engineers, London, Vol. 44, pp. 31-48
8. ATC 40, Volume 1 (1996) "Seismic evaluation and retrofit of concrete buildings", Applied Technology Council, Seismic Safety Commission, State of California.