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SEISMIC PERFORMANCE OF Y-SHAPED 10-STOREY DELHI SECRETARIAT BUILDING

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ABSTRACT

There are concerns about the performance of some strategic government buildings situated in highly seismic zones of India in the present scenario. Delhi Secretariat Building is one of the lifeline buildings identified for seismic evaluation and retrofitting by the "Delhi Govt. Earthquake Safety Initiative". Seismic evaluation of Delhi Secretariat, a 10 storey Y-shaped building using pushover analysis is presented here. The analysis brings out the deficiencies in the seismic performance of the building.

Introduction

The recent earthquakes in Kashmir, Gujarat, and other earthquakes of the past, have exposed the seismic vulnerability of not only 'non-engineered' structures, but also of several so-called 'engineered' structures in India, such as multi-storey buildings, many of which have been severely damaged. Surveys and analyses conducted in some cities located in seismic prone zones in India have revealed that many of the existing reinforced concrete (RC) buildings do not meet the earthquake resistant requirements of the latest Indian Standard (IS) codes. As such, there is an obvious and urgent need to assess the seismic vulnerability of existing buildings in India.

The Delhi Secretariat building is one of the lifeline buildings identified for seismic evaluation and retrofit by the Delhi Govt. Earthquake Safety Initiative. The building is Y-shaped in plan with three identical wings equally spaced around a central core that houses govt. offices, auditorium, gallery, and cafeteria. The central core is detached from the adjoining wings with a 20 mm separation joint throughout the height of the building. Considering the possibility of pounding it was decided to join the core with the three wings. Pushover analysis of the four individual parts and also integrated building is carried out using FEMA procedure.

Building Details

The building (Fig. 1) is a 10 storey office building located in New Delhi (Zone IV). It is Y-shaped in plan with three identical wings equally spaced around a central core. The building was constructed in 1980-1982. Fig. 2 shows a typical floor framing plan of the building. The building has varying dimension up the height of building and is stepped by one (or more) grid out of 10 total storeys (Fig. 2). Storey height is

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nearly 6 m for the first two storeys and 3.15 m for all other storeys. The central core is detached from the adjoining wings with a 20 mm separation joint throughout the height of the building. The overall ground profile around the building is level.

Figure 1. Delhi Secretariat building.

All columns in the three wings are rectangular and the column size reduces from 500 mm × 1000 mm at the ground floor to 300 mm × 1000 mm at the top floor. The percentage of reinforcement also reduces from approximately 2% for the section at the ground level to 0.6% for the section at the top floor. The columns in the central core are of rectangular, circular and hexagonal types. Most of the beams are 300 mm × 750 mm and 300 mm × 600 mm in size. According to the structural drawing, the building does not have proper ductile detailing as per IS 13920:1993, as the building construction pre-dates this code.

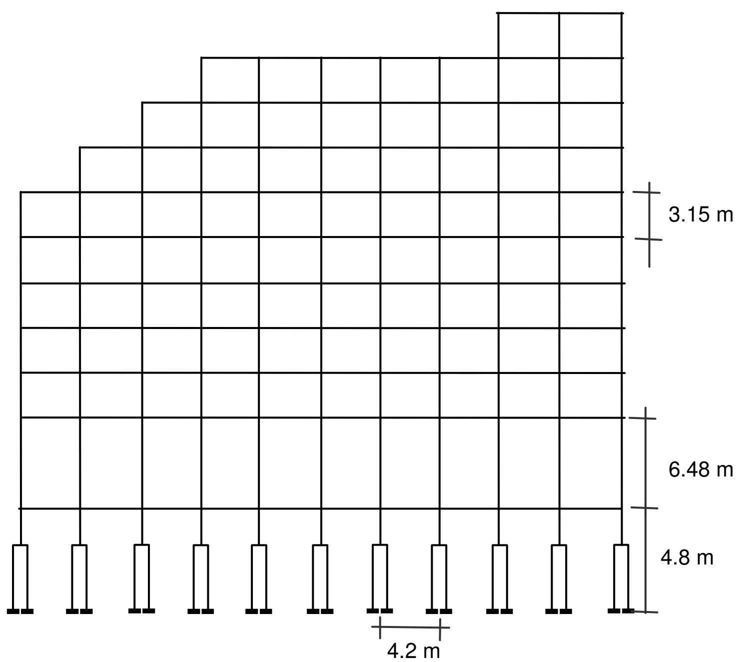


Figure 2. Elevation of a typical frame in the wing.

The concrete slab thickness is 120 mm. Light weight concrete blocks are used for partition walls. The partition walls are not connected integrally to the frame. The central core plan is almost identical for all the floors. It is nearly hexagonal in plan. It consists of a solid structural wall around the elevated lift core and other frame elements.

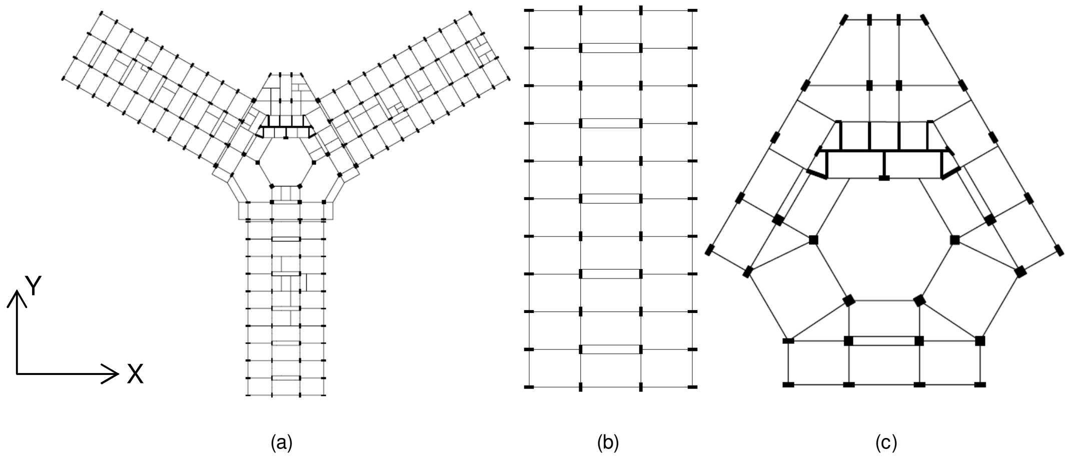


Figure 3. Sketch of a typical floor framing plan (a) integrated building (b) wing (c) central core.

The foundation system consists of multiple piles (24 m deep piles of diameter 500 mm and 450 mm) and pile cap (1.2 m thick). The pile caps are connected by 900 mm $\times$ 900 mm beams. As per the soil report, the soil is not uniformly graded but not vulnerable to liquefaction.

A sketch of the typical floor framing plan for the building is shown in Figure 3 (a). The plan of a typical floor for one of the three identical wings (Wing-Y) and the central core is shown in Figure 3 (b) and 3 (c) respectively.

Structural Modelling

The building is modelled in SAP2000NL which has a facility to carry out nonlinear static pushover analysis. Different grades of concrete (M20/M25/M30) were considered for different elements according to the structural drawing. All of the reinforcing bars are of Fe-415 grade. Beams and columns were modelled as frame elements with the centrelines joined at nodes. According to the available plan, the centreline of the beam and columns are eccentrically aligned. But in modelling, these eccentricities were ignored. The rigid beam-column joints were modelled by giving end offsets at the joints. The floor slabs were assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements. The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams as per IS 456:2000. Staircases and machine floor were not modelled for their stiffness but their masses were considered in the analysis. The shear wall around the lift core is modelled as single column element with a cross-section similar to that of the shear wall and rigid beams were connected to all the beams joining the shear wall.

The piles are modelled as frame elements. The pile cap is assumed as rigidly connected to the pile and the columns. The connection between pile and the column is established by providing the weld constraints in SAP2000NL. The effect of soil-structure interaction is ignored in the analyses.

The lateral force distribution along the height of the building $Q_i = \dfrac{w_i h_i^2}{\sum w_i h_i^2}$ is calculated according to IS 1893:2002 and used for pushover analyses. Pushover analyses performed independently in two orthogonal X and Y directions. The load distribution is dependent on the period of the building in two orthogonal directions.

The force-deformation curves in flexure and shear were obtained from the reinforcement details given in the drawing and were assigned in all the columns and primary beams. The flexural hinges (M3) and shear hinges (V2) were assigned for the beams at two ends. Flexural hinges (PMM) and shear hinges (V2 and V3) were also given for all the columns at upper and lower ends. A typical force-deformation curve is given in the Figure 5. The points A, B, C, D and E are marked on the curve: B is the point at which the section yields; at point C, unloading occurs up to point D, which is the point where the section reaches its residual capacity and then it starts deforming up to point E with a residual capacity. The other salient points are IO, LS and CP that correspond to immediate occupancy, life safety and collapse prevention respectively equally spaced in the region BC. Hinge properties of a typical beam and column is shown in Table 1. The moment (*M*) and corresponding rotation (θ) and shear force (V) and corresponding shear displacement (δ) at points A – E are given in the Table 1.

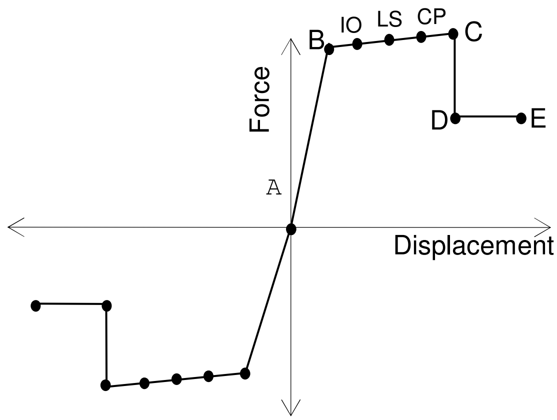


Figure 5. Force-deformation curve.

Table 1. Hinge properties of typical beam and column.

	Beam				Column			
Points	M (kNm)	Θ (rad)	V (kN)	δ (m) $10^{(-4)}$	M (kNm)	Θ (rad)	V (kN)	δ (m) $10^{(-4)}$
A	0	0	0.00	0	0.0	0.000	0	0
B	171.7	0	316.24	0	1.0	0.000	504.67	0
C	188.9	0.02	332.05	0. 20	1.1	0.015	529.90	0.21
D	34.34	0.02	63.25	0. 21	0.2	0.015	100.93	0.21
E	34.34	0.035	63.25	5.48	0.2	0.025	100.93	5.89

Using the Modified Mander (Fardis et al. 2001) model of stress-strain curves for concrete and the stress-strain curve for steel as per IS 456: 2000, for a specific confining steel, moment curvature curves were generated for each beams and columns (for different axial load levels) through a computer program. The moment rotation curves were then derived for plastic hinge length of 0.5D as per the established practice (Park and Paulay 1975). The PMM interaction surface was calculated for all the columns using IS 456:2000 and used for the flexural hinge modelling.

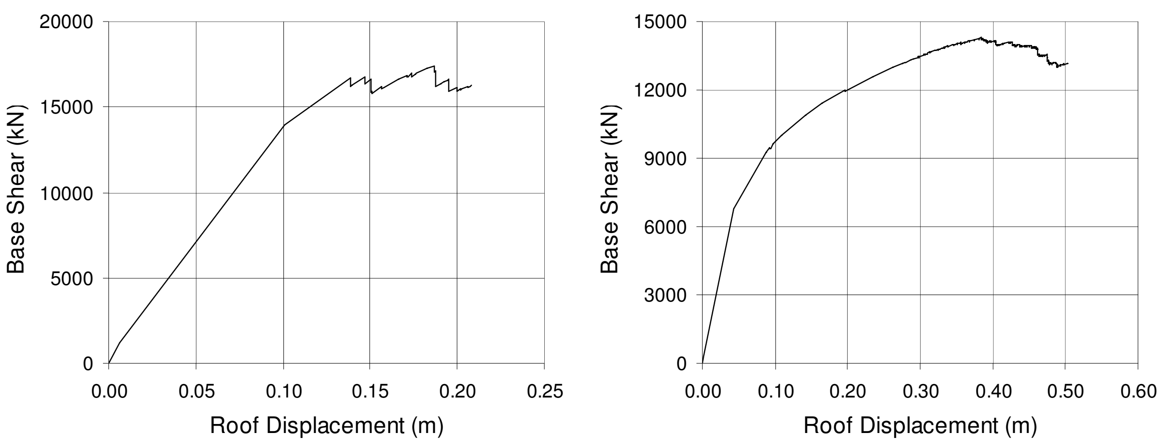
Shear force-deformation curves for beams and columns were modelled to assign shear hinges. Yield shear strength is calculated using IS 456:2000 and the ultimate shear strength is taken as 5% more than yield shear strength and residual shear strength is taken as 20% of the yield shear strength for modelling of the shear hinges. Yield shear deformation is calculated using shear stiffness of the cracked member as per the procedure given in Park and Paulay (1975) and ultimate shear deformation is taken as 1.5 times the yield deformation.

Pushover Analysis

One of the three identical wings (wing-Y) and the central core were analysed separately and then the entire building was analysed assuming that all four parts are integrated. Initially, pushover analysis is done for the gravity loads (DL+0.25LL) incrementally under load control. The lateral pushover analysis (PUSH-X and PUSH-Y) was followed after the gravity pushover, under displacement control. The building is pushed in lateral directions until the formation of collapse mechanism. The capacity curve (Base shear versus Roof displacement) is obtained in X and Y directions. The demand and capacity is expressed in an ADRS format in the same plot to check the performance of the building. The demand curves were obtained for design basis earthquake (PGA = 0.18g) and maximum considered earthquake (PGA = 0.24g) expected for the location as per IS-1893:2002.

Figure 6 shows the pushover curve for wing-Y. Pushover curve shows that the base shear capacities of ‘Wing-Y’ along X- and Y- directions are 17,417 kN (i.e, 17% of total weight) and 14,297 kN (i.e, 14% of total weight) respectively. Maximum roof displacements along X- and Y- directions are 208 mm (0.52% of the building height) and 500mm (1.25% of the building height) respectively. Target displacements for the Wing-Y were calculated as per FEMA-356 and presented in Table 2. The table shows that the displacement capacity of Wing-Y is more than the corresponding demand (FEMA-356).

The demand and capacity spectrum are plotted in ADRS format as per ATC-40. The performance is assessed for two performance levels, Life safety (LS), under design basis earthquake (DBE) and collapse prevention (CP) under maximum credible earthquake (MCE). The performance point is achieved in this case (Wing-Y) for both of the earthquake levels. The mode failure is found to be storey mechanism in the upper storey columns (at 27.05-30.20m level). The column capacities in the higher floors are found inadequate.

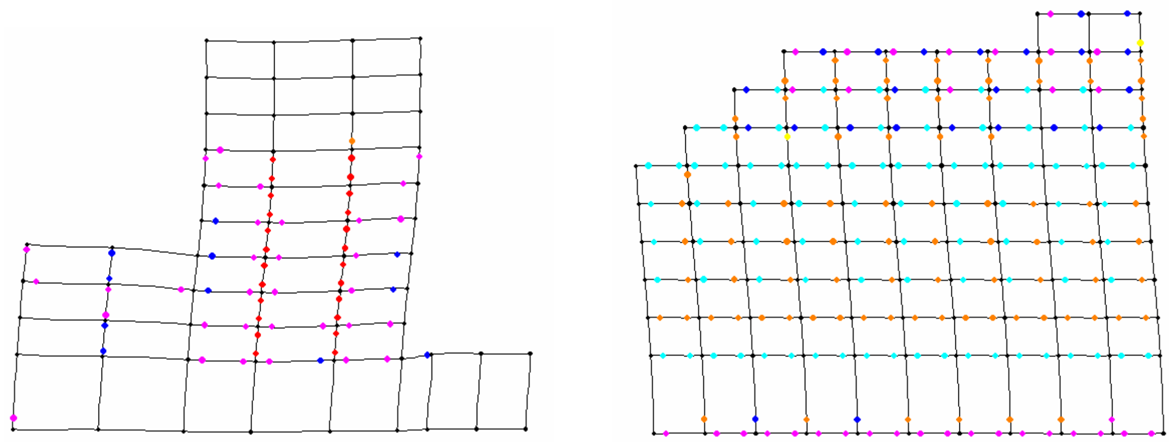


(a) Push - X

(a) Push - Y

Figure 6. Pushover curves for the Wing-Y.

Figure 7 explains the failure mode of the structure in the pushover analysis of Wing-Y. Figure 7b clearly shows the formation of the storey mechanism at the upper floor levels. The concentration of the hinges shown in Figure 7a may be due to the discontinuity in X-frame of the structure at that level. Only the columns in the inner frames have failed because the weaker direction of those columns is oriented along the loading direction.



(a) Push - X (b) Push - Y

Figure 7. Distribution of hinges at the typical frames of Wing-Y at collapse.

Table 2. Displacement demand and capacity (in m) of Wing-Y (FEMA-356).

	Displacement demand for Life Safety (LS)	**Displacement demand for Collapse Prevention (CP)**	**Displacement capacity**
X-direction	0.110	0.159	0.208
Y-direction	0.115	0.167	0.500

Along the X-direction, Wing-Y is found to experience a rotation in the plan due to the difference in height in two sides of the wing. The maximum displacement (Envelope) along the X-direction at all the storey levels was found out and is plotted in the Figure 8 along with the displacement at the centre of mass.

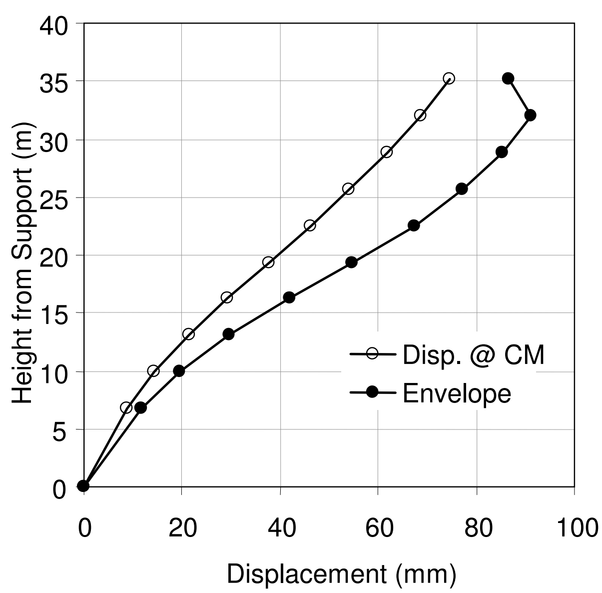
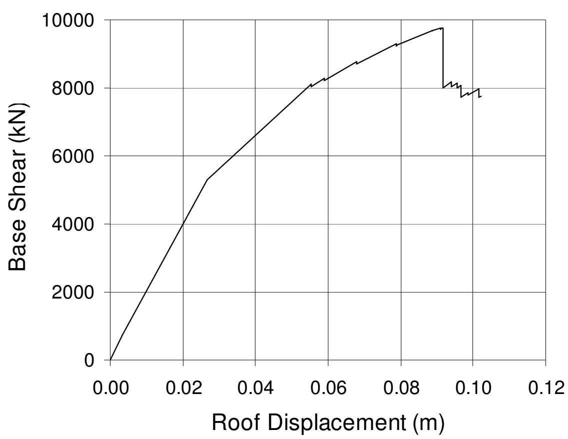


Figure 8. Storey displacement for Wing-Y at an intermediate step of Push-X.

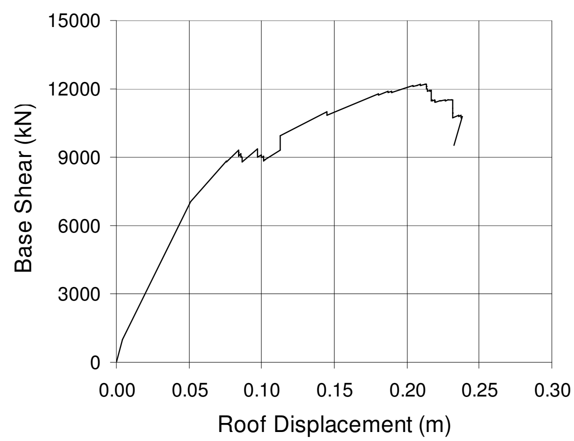
For the central core, the pushover curve (Figure 9) shows that the base shear capacities along X- and Y- directions are 9,770 kN (i.e, 11% of total weight) and 12,236 kN (i.e, 14% of total weight) respectively. Maximum roof displacements along X- and Y- directions are 102 mm (0.25% of the building height) and 238 mm (0.6% of the building height) respectively. Similar to the Wing-Y target displacements for the central core were calculated and presented in Table 3. The table shows that the central core of the building has less displacement capacity in X-direction than the corresponding displacement demand for collapse prevention performance level (FEMA-356). When capacity spectrum of central core is compared with the demand spectrum (ATC-40) it is observed that the central core is failed to reach the performance point under MCE level of earthquake. However, it achieves the performance point under DBE level of earthquake. The failure of the central core is mainly characterised by collapse of piles in axial compression in the core region.

Table 3. Displacement demand and capacity (in m) of Central Core (FEMA-356)

	Displacement demand for Life Safety (LS)	**Displacement demand for Collapse Prevention (CP)**	**Displacement capacity**
X-direction	0.108	0.156	0.102
Y-direction	0.104	0.152	0.238



(a) Push - X

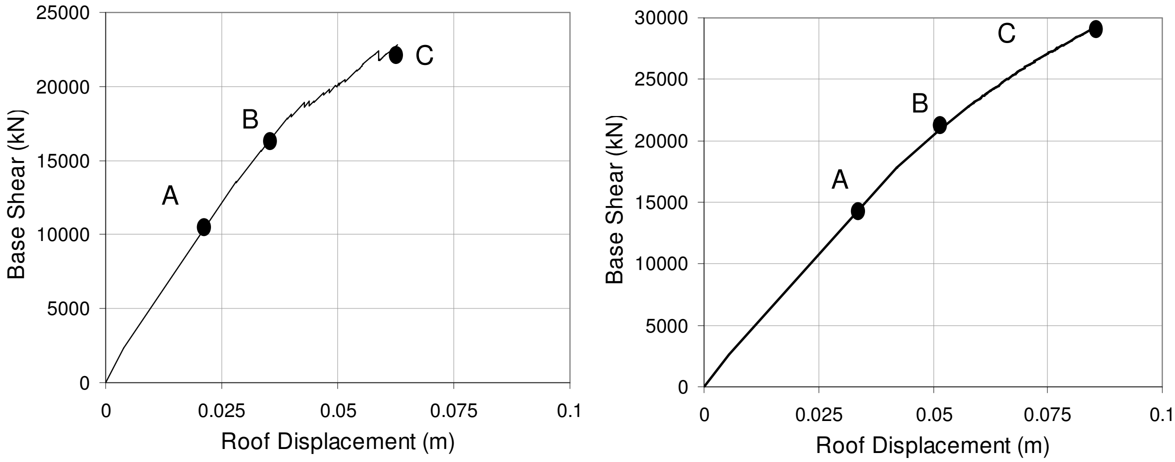


(a) Push - Y

Figure 9. Pushover curves for the Central Core.

The building is also analysed assuming that all four parts are integrally connected. Figure 10 shows the pushover curve of the integrated building. The status, location and number of hinges at points A, B and C are shown in Table 4. The pushover curves show that the building has base shear capacity more than compared to the design base shear based on equivalent static method in both the directions. The maximum roof displacement under gone is 0.19%H and the corresponding lateral load capacity is 6.7%W in X-direction and these values are 0.27%H and 8.6%W in the Y-direction. The building is found to be failed by forming the axial hinges in the piles in compression in some of the pile in the core region in addition to a storey mechanism at the upper storey levels.

It is observed that there is no performance point found in any of the pushover cases when the capacity and demand curve compared as per ATC-40. Target displacement calculated for different performance level (LS and CP) along two orthogonal directions as per FEMA 356 and presented in Table 5. The maximum roof displacement achieved by the building is also shown. The table shows that the building failed to reach the target displacement for the both X- and Y- directions.



(a) Push - X (a) Push - Y

Figure 10. Pushover curves for the integrated building.

Table 4. Status of hinges at points A, B and C.

Point	Push X		Push Y	
	Remarks	**Number of hinges**	**Remarks**	**Number of hinges**
A	Axial hinges in piles (B-IO)	3	Axial hinges in piles & moment hinges in top storey columns (B-IO)	20
B	Moment hinges at top storey columns(B-IO)	20	Moment hinges in top two storeys (IO-LS)	44
C	Storey mechanism at top storey (D-E)	11	Moment hinges in top three storeys (D-E)	85

Table 5: Displacement demand and capacity (in m) of the integrated building (FEMA-356).

	Displacement demand for Life Safety (LS)	**Displacement demand for Collapse Prevention (CP)**	**Displacement capacity**
X-direction	0.098	0.143	0.063
Y-direction	0.109	0.158	0.086

Discussions

The pushover analysis results show that the individual parts of the building are able to achieve performance point except for one case of the central core (along X direction under MCE). The mode of failure is found to be storey mechanism in the upper storey columns (at 27.05-30.20m level) for the wing and a foundation failure for the central core. The column capacities in the higher floors are found inadequate.

The analysis for the integrated building shows that the failure of the piles under the central core leads to the premature collapse of the structure. In addition to failure of the piles, a storey mechanism at the top storey is also observed.

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