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PUSHOVER ANALYSIS OF A 19 STORY CONCRETE SHEAR WALL BUILDING

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SUMMARY

Pushover analysis was performed on a nineteen story, slender concrete tower building located in San Francisco with a gross area of 430,000 square feet. Lateral system of the building consists of concrete shear walls. The building is newly designed conforming to 1997 Uniform Building Code, and pushover analysis was performed to verify code's underlying intent of Life Safety performance under design earthquake. Procedure followed for carrying out the analysis and results are presented in this paper.

INTRODUCTION

Design of civil engineering structures is typically based on prescriptive methods of building codes. Normally, loads on these structures are low and result in elastic structural behavior. However, under a strong seismic event, a structure may actually be subjected to forces beyond its elastic limit. Although building codes can provide reliable indication of actual performance of individual structural elements, it is out of their scope to describe the expected performance of a designed structure as a whole, under large forces. Several industries such as automotive and aviation, routinely build full-scale prototypes and perform extensive testing, before manufacturing thousands of identical structures, that have been analyzed and designed with consideration of test results. Unfortunately, this option is not available to building industry as due to the uniqueness of typical individual buildings, economy of large-scale production is unachievable.

With the availability of fast computers, so-called performance based seismic engineering (PBSE), where inelastic structural analysis is combined with seismic hazard assessment to calculate expected seismic performance of a structure, has become increasingly feasible. With the help of this tool, structural engineers too, although on a computer and not in a lab, can observe expected performance of any structure under large forces and modify design accordingly. Nonlinear response history analysis is a possible method to calculate structural response under a strong seismic event. However, due to the large amount of data generated in such analysis, it is not considered practical and PBSE usually involves nonlinear static analysis, also known as pushover analysis. From research viewpoint, while PBSE is still in developmental stage where improved analysis techniques are being researched (Gupta [1], Gupta [2],

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Chopra [3-4], Miranda [5], Farrow [6]), from application viewpoint, it has reached a stage where established procedures and guidelines are available for practicing engineers (ATC-40 [7], FEMA-273 [8], FEMA-356 [9], Naeim [10]).

BUILDING DESCRIPTION AND MOTIVATION FOR PUSHOVER ANALYSIS

Building analyzed is a nineteen story (18 story + basement), 240 feet tall slender concrete tower located in San Francisco with a gross area of 430,000 square feet. Unique features of the slender concrete tower presented challenges for seismic design. Typically, a 240 feet tall concrete building in seismic zone 4 would have a lateral system that combines shear walls and moment frames. However, two architectural features made the use of moment frames difficult. First, the 60 feet long open bays limited the number of possible moment frames. Second, on the southeast side two of the perimeter columns are discontinued at the 6th story and six new columns are introduced that slope for the lowest six stories at an angle of about 20 degrees from vertical. These sloped columns connect to transverse walls through horizontal transfer elements at the 6th story and put considerable gravity-induced horizontal loads on the lateral system at that level. Due to limited number of available columns and large horizontal loads from the sloped columns, the moment frames were abandoned.

Thus lateral system of the building consists of shear walls with arrangement as shown in a typical floor plan in Fig. 1. Typically shear walls are approximately 15 inch thick at the 15th story and above, and 27 inch thick at the 14th story and below, except for the transverse walls in the eastern half of the building where approximately 36 inch thick shear walls are present at the 6th story and below. Typical floor plan is of rectangular shape and measures approximately 350 x 60 feet. An isometric view of building is shown in Fig. 2, with some of the gravity beams and columns removed for clarity.

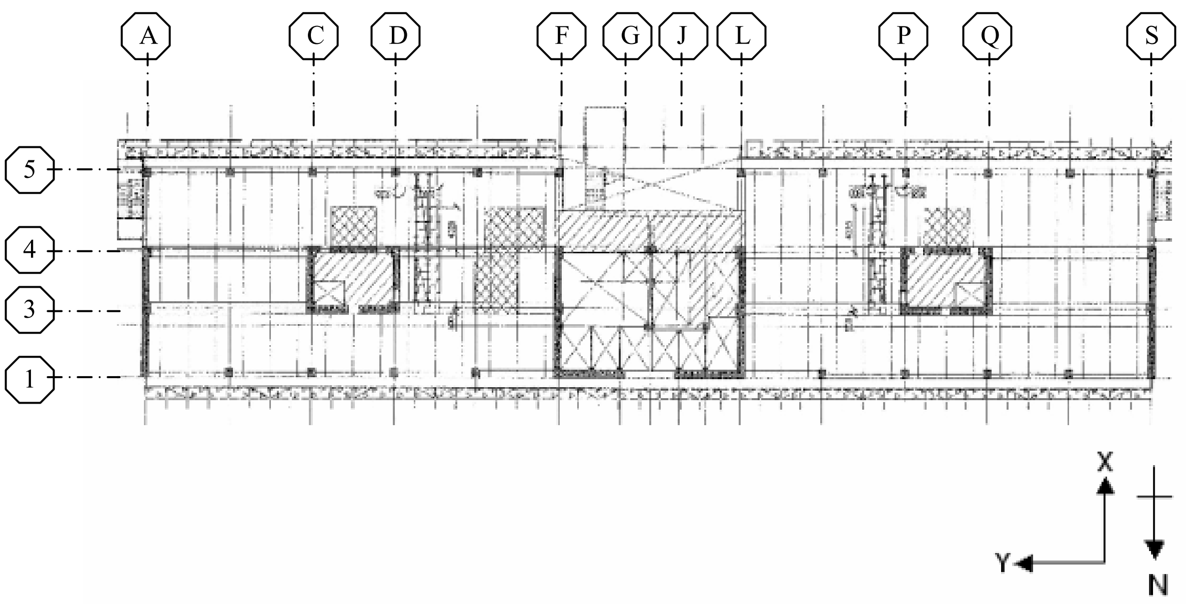

Figure 1. Typical floor plan and wall arrangement

The building was designed per 1997 Uniform Building Code (UBC) [11] with an intended performance objective of Life Safety under design seismic event. However, due to the following reasons a concern was felt regarding whether the building would meet Life Safety performance level intended by the code.

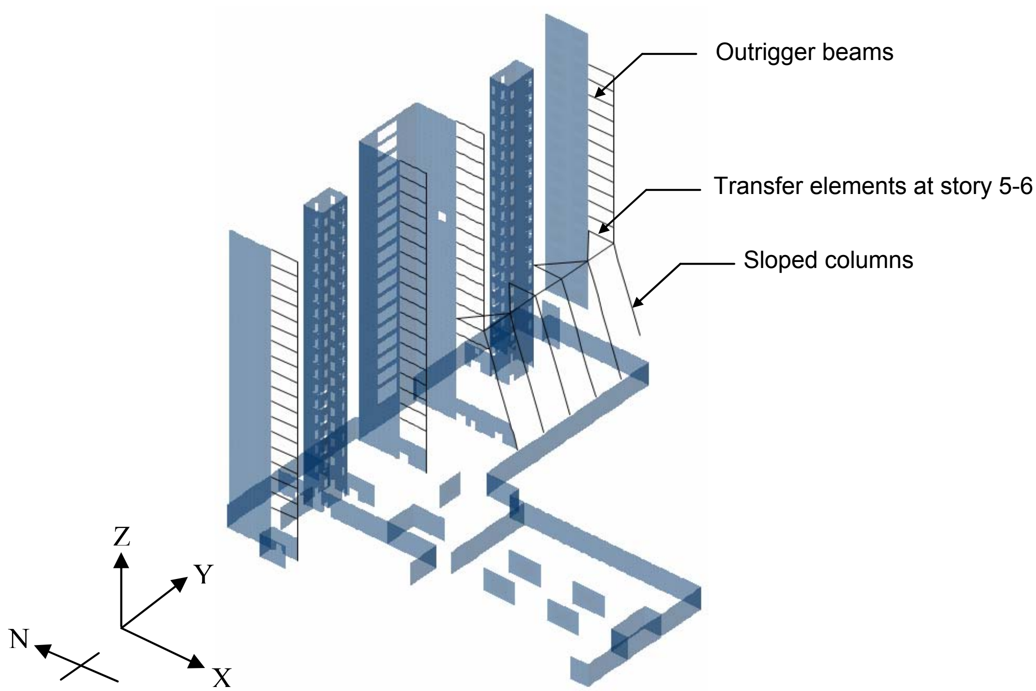

Figure 2. Isometric view of analyzed concrete tower building

The building height is 240 feet which is equal to code limit for concrete shear wall buildings according to 1997 UBC. A more recent 2003 International Building Code (IBC) [12] limits the maximum height of a concrete shear wall building to 160 feet. Furthermore, geotechnical studies indicated a site-specific design response spectrum (of 475 year return period or 10% probability of exceedance in 50 years) with spectral accelerations approximately 40% higher than the 1997 UBC design response spectrum in the relevant time period range for the building (Fig. 3).

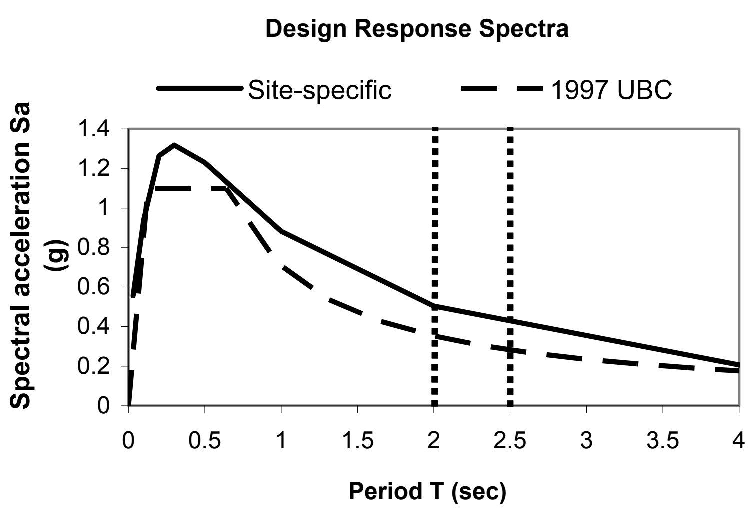

Figure 3. Response spectra for design earthquake

Time period range of 2-2.5 seconds is considered relevant for the building based on its modal properties and effective period at the performance point from pushover analysis. Due to above reasons, a decision was made to verify building's performance using pushover analysis.

MODELING AND ANALYSIS TECHNIQUES

As seen from the floor plan, due to shape of middle core walls, separate pushover analysis in positive and negative transverse direction is needed. Transverse walls at the 6th story and below in eastern half of the building are thicker, thereby requiring separate pushover analysis in positive and negative longitudinal direction as well. Due to the slender shape of floor plan, consideration of mass eccentricity is important for transverse pushover cases. With these considerations in mind, 8 separate pushover analyses were performed as shown in Fig. 4. Uniform lateral load pattern was used where lateral load at each floor is proportional to total floor mass. Software SAP2000 version 7 [13] and ETABS version 7 [14] were used for analysis. SAP2000 was used to perform pushover analysis and ETABS was used to calculate hinge properties of shear wall and elastic analysis. For all lateral elements, cracked section was assumed with an effective stiffness equal to 50% of gross section. Additional modeling techniques of different type of elements are described in the following.

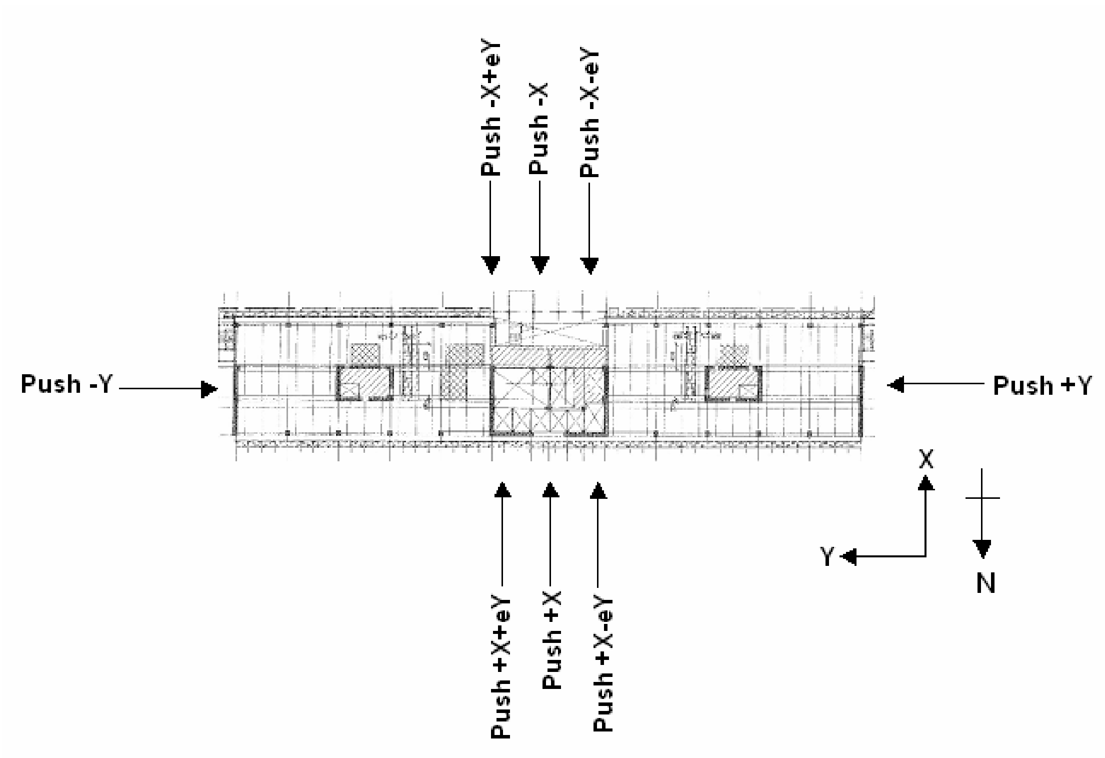

Figure 4. Different cases of pushover analysis performed

Modeling of shear walls

Given that all shear walls in the building are slender with wall height-to-length ratio well above 3 and therefore seismic response of the shear walls is expected to be dominated by flexure, as well as because modeling nonlinear behavior in SAP2000 pushover analysis is limited to frame elements, the shear walls were modeled as equivalent frame elements.

In order to provide connectivity between walls, the equivalent frames were connected at the floor level with rigid links on the side of the wall without any opening, or with beams with rigid end offsets to model spandrels above wall openings. Figure 5 illustrates this modeling technique for the longitudinal walls in the middle core.

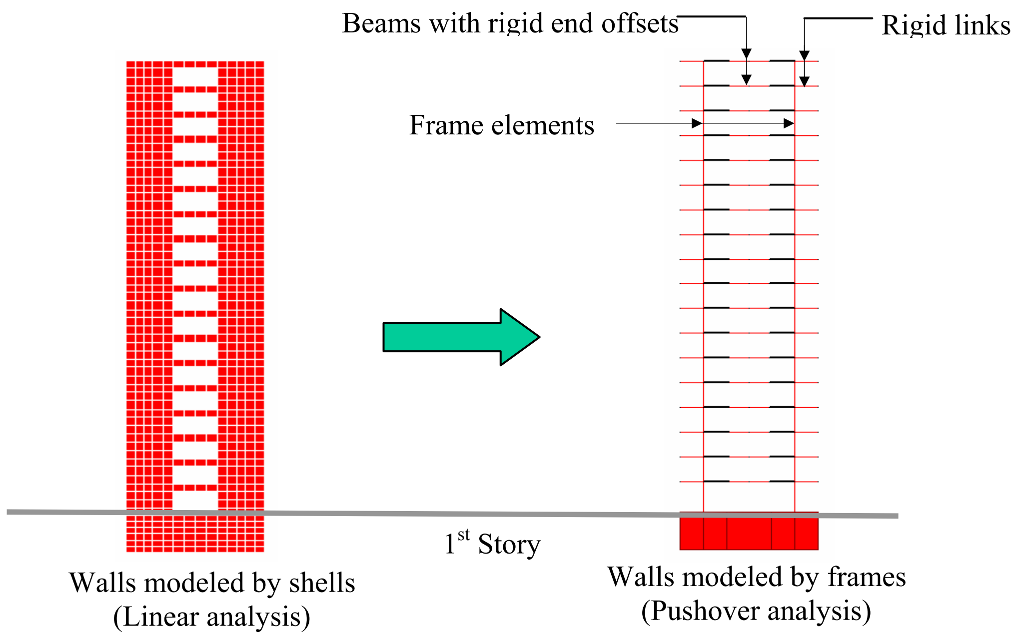

Figure 5. Shear wall modeling in SAP2000 for pushover analysis

Above procedure was applied to model walls above the 1st story. As seen from Fig. 2, since there are many additional walls at the 1st story (basement), stress level is low and nonlinear behavior is not expected. Therefore walls at the 1st story were modeled as shell elements without any nonlinearity. Figure 6 shows pushover analysis model in SAP2000 where shear walls are modeled by frame elements.

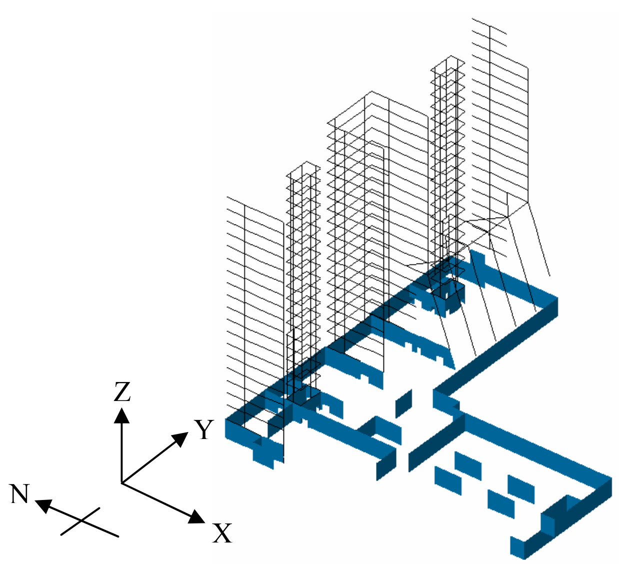

Figure 6. Isometric view of pushover analysis model of building

Nonlinear hinge assignment
In order to model nonlinear behavior in any structural element, a corresponding nonlinear hinge must be assigned in the building model. Nonlinear hinges were assigned to the following structural elements expected to undergo inelastic deformation:

Shear walls
Typically, PMM hinges with axial force-moment interaction were assigned at the wall ends near floor levels and shear hinges were assigned at the mid-height level of walls. At the lowest story, PMM hinges were assigned at the mid-height level and shear hinges were assigned at the lower end of walls.

Spandrels
Spandrels are coupling beams above wall openings connecting two walls that are either in same plane or in perpendicular planes. Moment hinges were assigned at the spandrel ends and shear hinges were assigned at the center of span.

Sloped columns
PMM hinges were assigned at the columns ends and at a few equally-spaced intermediate points.

Gravity columns
To ensure gravity load carrying capability at performance point, PMM hinges are assigned at the ends and shear hinges are assigned at the mid-height level of gravity columns.

Elements without nonlinear hinges
Nonlinear hinges were not assigned to the following structural elements in SAP2000 pushover model:

Gravity beams
Gravity beams were modeled pin-ended.

Outrigger beams
Outrigger beams are gravity beams connecting some of the shear walls and gravity columns. These will be subjected to moment under an earthquake. Since these beams are designed for gravity loads, and expected to develop early flexural hinges, their contribution to lateral stiffness and strength of building is likely to be negligible. Moreover, presence of some weak elements that develop early hinges compared to other elements makes pushover analysis numerically difficult for software. After verifying this behavior from preliminary pushover runs, these beams were also modeled pin-ended.

Transfer elements at story 5-6
Transfer elements are present on the southeast side of the building. These elements connect sloped columns with transverse shear walls. Integrity of these elements is considered critical. Consequently, nonlinear hinges were not assigned to these and these were designed to remain elastic under the pushover forces developed at the performance point.

Nonlinear hinge property calculation
Nonlinear hinge properties, as assigned in SAP2000 model, were calculated as described in the following.

Shear wall PMM hinge
For any given shear wall, PMM hinge property was calculated in the following two steps:

1. PMM interaction surface: PMM interaction surface determines the load at which a shear wall section becomes inelastic and forms a hinge. For a given wall section geometry, material and reinforcement arrangement, PMM interaction surface was calculated using ETABS section designer module. Several of the wall sections are unsymmetrical, and result in different P-M interaction curves in opposite directions. For these unsymmetrical wall sections, the appropriate P-M interaction curve corresponding to the direction of pushover was used in defining hinge property.

2. Moment-plastic rotation (M-θ_p) relation: M-θ_p relation for a shear wall section consists of plastic rotation and corresponding moments as ratio of yield moment. This relation affects the behavior of a section once a hinge forms there. All values needed to define M-θ_p relation may be obtained by following FEMA or ATC guidelines. In this work, values for θ_p were calculated based on the FEMA guidelines and corresponding M values were read from the moment-curvature curves of wall sections, under design gravity load. Moment-curvature curves of wall sections were obtained from ETABS section designer module, which uses stress-strain curve for concrete as suggested by Kent and Park (Park [15]). Plastic hinge length required for this calculation was based on FEMA guidelines.

Shear wall shear hinge
Shear hinge property was entirely defined by nominal shear strength without any reduction factor. This was calculated per 1997 UBC. Shear hinges in all elements were considered force-controlled with no ductility and development of any shear hinge was considered undesirable for the design objective.

Other hinges
FEMA and ATC guidelines were generally followed in calculating the hinge properties for other elements.

Verification of pushover analysis model
In pushover analysis model, as shear walls were modeled by frames rather than shell elements, it was considered important to verify that pushover analysis model is a good representation of the building. For this purpose, pushover analysis model was compared with linear analysis model where shear walls were modeled by shell elements. A comparison of modal periods of the two models is given in Table 1. It can be seen from the table that a good match is achieved between the two models. Although not presented here, mode shapes were also compared and found to be similar.

Table 1: Comparison of linear and pushover analysis models

MODE	**Modal period (sec)**		
	ETABS [1]	**SAP2000 Imported** [2]	**SAP2000 Pushover** [3]
1	2.33	2.40	2.55
2	2.18	2.23	2.30
3	1.70	1.74	1.89

1. Model for linear analysis in ETABS with walls modeled by shells
2. ETABS model imported in SAP2000
3. Model for pushover analysis in SAP2000 with walls modeled by frames

RESULTS

Strengthening requirements

Upon performing the various pushover runs as shown in Fig. 4, shear hinges were found to develop at a few wall and spandrel locations which was considered undesirable for the performance objective. By performing trial runs with arbitrarily increased shear strength of the shear hinges at these locations, shear strengthening requirement was quantified as a factor of original shear strength. These requirements are listed for walls and spandrels in Table 2 and 3, respectively.

Table 2: Required wall shear strengthening

Wall location	Maximum required shear strength increase factor
GL L/1-4, Story 7	1.04
GL F/1-4, Story 7-9	1.21
GL C/3-4, Story 3	1.13
GL 1/F-G, Story 4	1.07

Table 3: Required spandrel shear strengthening

Spandrel location	Maximum required shear strength increase factor
GL 3, between P-Q, Story 2	1.48
GL 3, between P-Q, Story 3	1.46
GL 3, between P-Q, Story 4	1.35
GL 3, between P-Q, Story 5	1.18
GL 3, between C-D, Story 3	1.37
GL 3, between C-D, Story 4	1.42
GL 3, between C-D, Story 5	1.38
GL 3, between C-D, Story 7	1.07
GL 1/J-G, Story 5-6	1.01
GL 1/J-G, Story 7-14	1.03
GL 1/J-G, Story 15-18	1.08
GL 1/J-G, Story 19	1.16

Performance of strengthened building

After strengthening the building in shear as listed in Table 2 and 3, capacity curves obtained from pushover analysis of the building were found to meet the demand curve. At the performance points thus obtained, several flexural hinges were formed in the building. However, the plastic rotations of the hinges developed, as calculated by SAP2000, were checked and found to be within the limits suggested by FEMA and ATC guidelines for the intended design objective of Life Safety. For illustration, Fig. 7 shows capacity curves and performance points for two of the pushover runs. Figure 8 shows the deformed shape and hinges developed in the building at the performance point for these runs. It has been indicated by Chopra [3] that pushover analysis may be inherently limited in accurately computing hinge plastic rotations. Therefore story drift may be a more relevant indicator of building performance. Average story drift at performance point of different pushover runs is shown in Table 4. Average of story drift from different pushover runs was found to be below the value recommended by the FEMA guidelines.

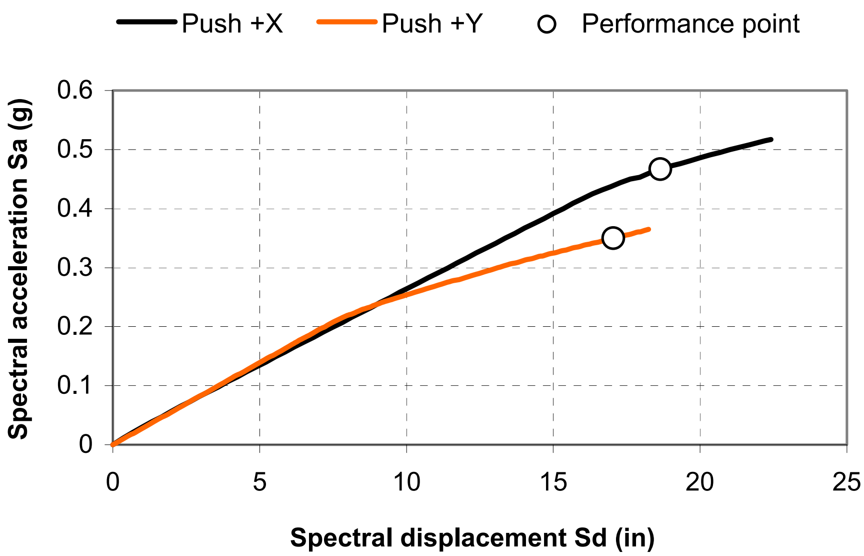

Figure 7. Pushover capacity curve and performance point

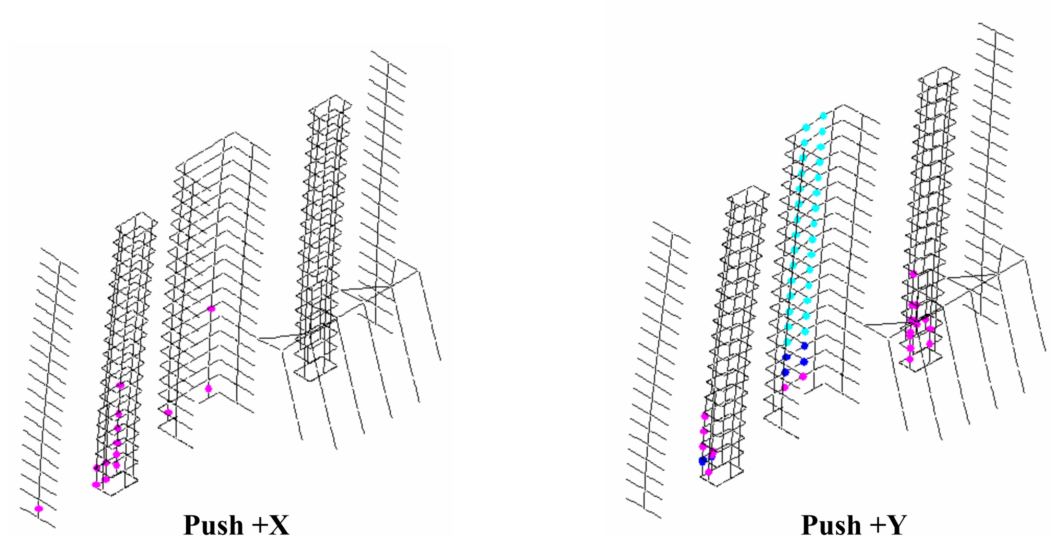

Figure 8. Building deformation and hinge development at performance point

Table 4: Average story drift at performance point

No.	Pushover analysis case	Roof displacement (in)	Average story drift (%)
1	Push +X	30.1	1.05
2	Push –X	26.2	0.91
3	Push +X+eY	29.5	1.03
4	Push –X+eY	25.4	0.89
5	Push +X–eY	30.4	1.06
6	Push –X–eY	26.7	0.93
7	Push +Y	25	0.87
8	Push –Y	25.3	0.88

Avg. X drift % = 0.98
Avg. Y drift % = 0.88

DISCUSSION

Based on the lessons learned during this analysis, and the results presented above, a discussion is presented in this section.

Analysis

Pushover analysis is numerically demanding and may cause numerical difficulties for the software used to run the analysis. Simplifying the model as much as possible is helpful in completing the run and reducing the run time. As the analysis process may usually involve several runs to evaluate effect of the various parameters, it is important to be able to complete a pushover run in less time. Any linear elements should be modeled with least possible amount of meshing. For example, in this work, due to wall openings, the linear model with shells to model walls had dense meshing. For connectivity, this dense meshing was continued to the shells modeling floors as well. Although walls were modeled by frame elements in the pushover model, very large run time for pushover analysis was required due to dense floor meshing. The run time significantly reduced when less dense mesh was used to model floors.

Hinges should be assigned to any location where nonlinear behavior is expected, even when nonlinear behavior is later not observed at many of these locations. Having more hinges than necessary does not slow down analysis and it ensures that nonlinear behavior is captured.

A model with some elements that yield much earlier compared to the rest may be numerically difficult to run. This may happen when elements that are not the main components of lateral system, are modeled with hinges. As these elements are expected to yield early in an earthquake, an easy solution may be to model these as pin-ended. However, building behavior should be verified to not have changed significantly in this process. This may be done by comparing the capacity curves for the model with pin-ended element to that of the original model. Furthermore, these elements should be detailed to yield in a ductile manner.

Rigid end offsets significantly influence model behavior and force distribution between elements. In shear wall buildings where pushover model uses frame elements to model shear walls, the clear span of spandrels and any slender columns formed due to wall openings is usually much smaller than the center-to-center span. These elements should be modeled with rigid end offsets and nonlinear hinges should be assigned outside of the offset.

Design

As shown previously, shear strengthening requirement was found to be approximately 0-50% for spandrels and 0-20% for walls, over code design. As stated in SEAOC Blue Book [16], code design per 1997 UBC does not ensure that flexural strength is reached before shear strength. As seen from this analysis, even for tall and slender buildings, design per 1997 UBC may result in shear walls failing in shear rather than flexure. As shear failure in slender shear walls is force-controlled with limited ductility, this behavior may be a hindrance to achieving the design objective. Therefore an additional check should be made on shear walls and spandrels design to have their minimum shear strength corresponding to their flexural strength. Flexural strength based on the code design was found to be adequate as inelastic deformations typically remained within the acceptable limits.

CONCLUSION

Pushover analysis is a useful tool of Performance Based Seismic Engineering to study post-yield behavior of a structure. It is more complex than traditional linear analysis, but it requires less effort and deals with much less amount of data than a nonlinear response history analysis. Pushover analysis was performed on a nineteen story concrete building with shear wall lateral system and certain unique design features. Utilizing the results from this analysis, some modifications were made to the original code-based design so that the design objective of Life Safety performance is expected to be achieved under design earthquake.

REFERENCES

1. Gupta A, Krawinkler H. "Estimation of seismic drift demands for frame structures." Earthquake Engineering Structural Dynamics, 2000, 29(9):1287-1305.
2. Gupta B, Kunnath SK. "Adaptive spectra-based pushover procedure for seismic evaluation of structures." Earthquake Spectra, 2000, 16(2):367-392.
3. Chopra AK, Goel RK. "A modal pushover analysis procedure to estimate seismic demands for buildings: theory and preliminary evaluation." PEER Report 2001/03, Pacific Earthquake Engineering Research Center, College of Engineering, University of California Berkeley, 2001.
4. Chopra AK, Goel RK. "A modal pushover analysis procedure for estimating seismic demands for buildings." Earthquake Engineering Structural Dynamics, 2002, 31(3): 561-582.
5. Miranda E, Garcia JR. "Evaluation of approximate methods to estimate maximum inelastic displacement demands." Earthquake Engineering Structural Dynamics, 2002, 31(3): 539-560.
6. Farrow KT, Kurama YC. "SDOF demand index relationships for performance-based seismic design." Earthquake Spectra, 2003, 19(4):799-838.
7. ATC-40. "Seismic evaluation and retrofit of concrete buildings." Volume 1 and 2. Applied Technology Council, California, 1996.
8. FEMA-273. "NEHRP guidelines for the seismic rehabilitation of buildings." Federal Emergency Management Agency, Washington DC, 1997.
9. FEMA-356. "Prestandard and commentary for the seismic rehabilitation of buildings." Federal Emergency Management Agency, Washington DC, 2000.
10. Naeim F (Ed). "The seismic design handbook." Chapter 15. Kluwer Academic Publishers, MA, 2001.
11. 1997 Uniform Building Code. International Conference of Building Officials, CA, 1997.
12. 2003 International Building Code. International Code Council, Inc, 2002.
13. SAP2000 nonlinear version 7.44. Computers and Structures, Inc, CA.
14. ETABS nonlinear version 7.22. Computers and Structures, Inc, CA.
15. Park R, Paulay T. "Reinforced concrete structures." John Wiley & Sons, Inc, NY, 1975.
16. SEAOC Blue Book. "Recommended lateral force requirements and commentary." Structural Engineers Association of California (SEAOC), CA, 1999.