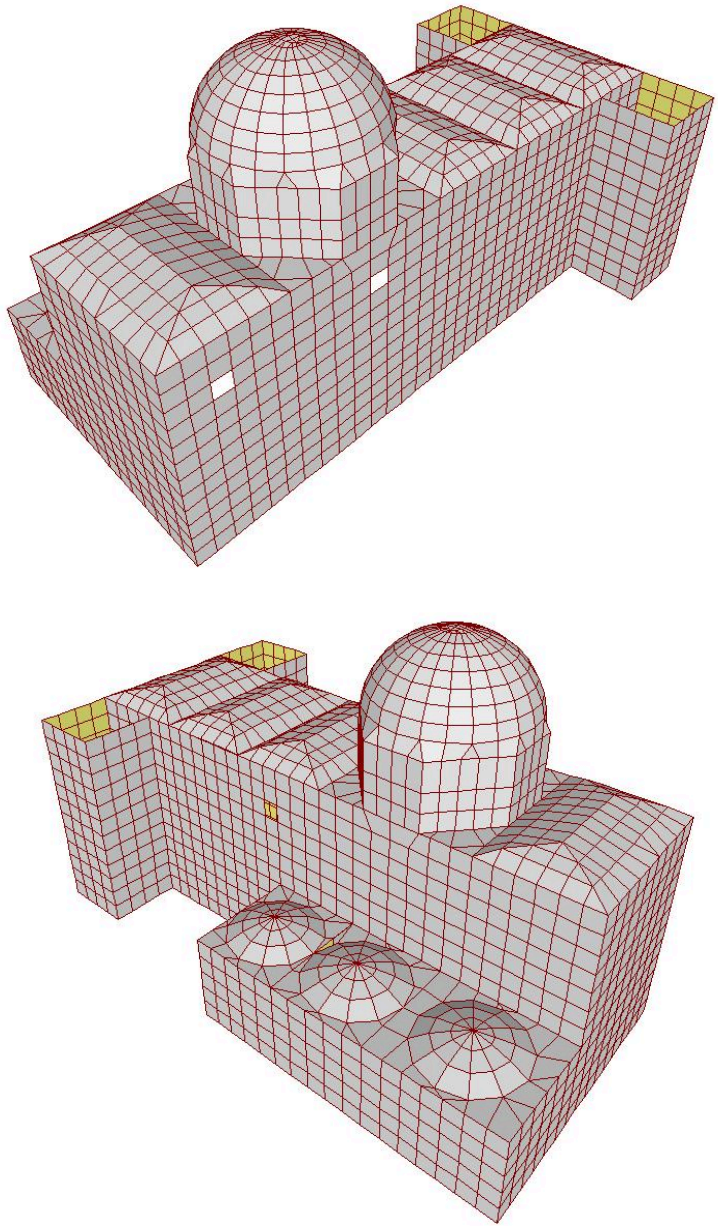

Figure 6. Finite element model of the reconstructed structure.

The stress values between the collapsed model and the repaired model are given respectively for comparison.

In the collapsed model given in Figure 7, the maximum stress values observed at the bottom corners of the collapsed walls are within the range of 240 kN/m^2 – 280 kN/m^2. These values change to the range of 238 kN/m^2 – 300 kN/m^2 at the model of the repaired structure, as it can be seen in Figure 8. While the difference at the corners of the walls are seen as negligible, the compressive stress values at the base of the wall show a drastic increase, from the range of 120 kN/m^2 – 160 kN/m^2 in the model of the collapsed structure to the range of 300 kN/m^2 – 362 kN/m^2 in the model of the repaired structure.

The stress values of the northern wall before and after repair are also given in Figure 9 and Figure 10. It is, once again, possible to see the increase at the stress levels at the base of the wall of the repaired structure.

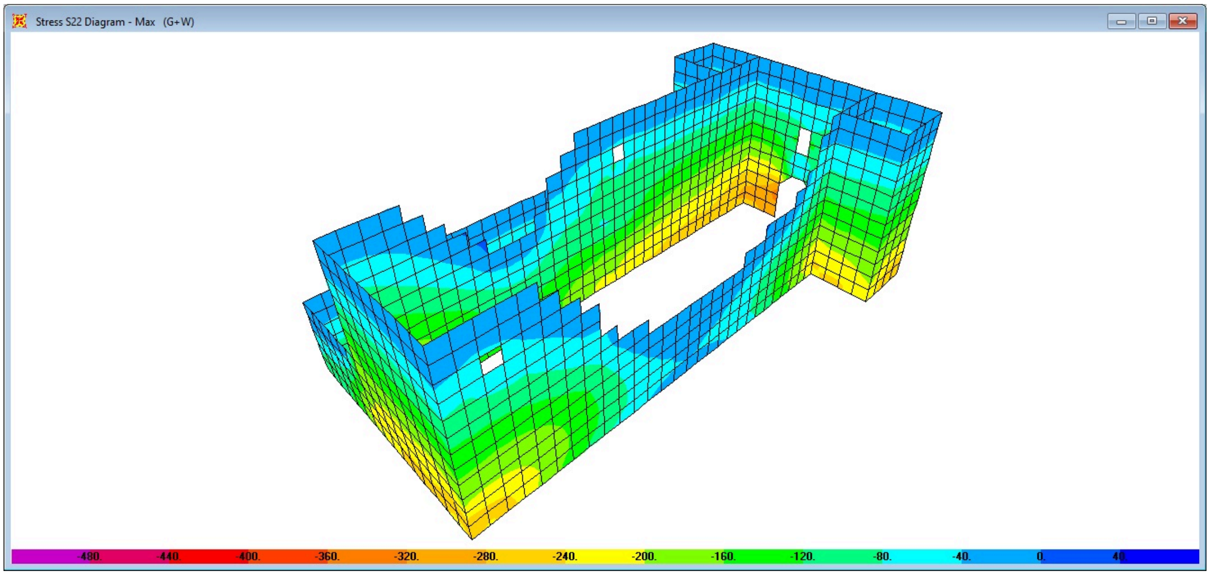

Figure 7. Stress distribution due to combination of gravity and wind force analysis in the collapsed structure.

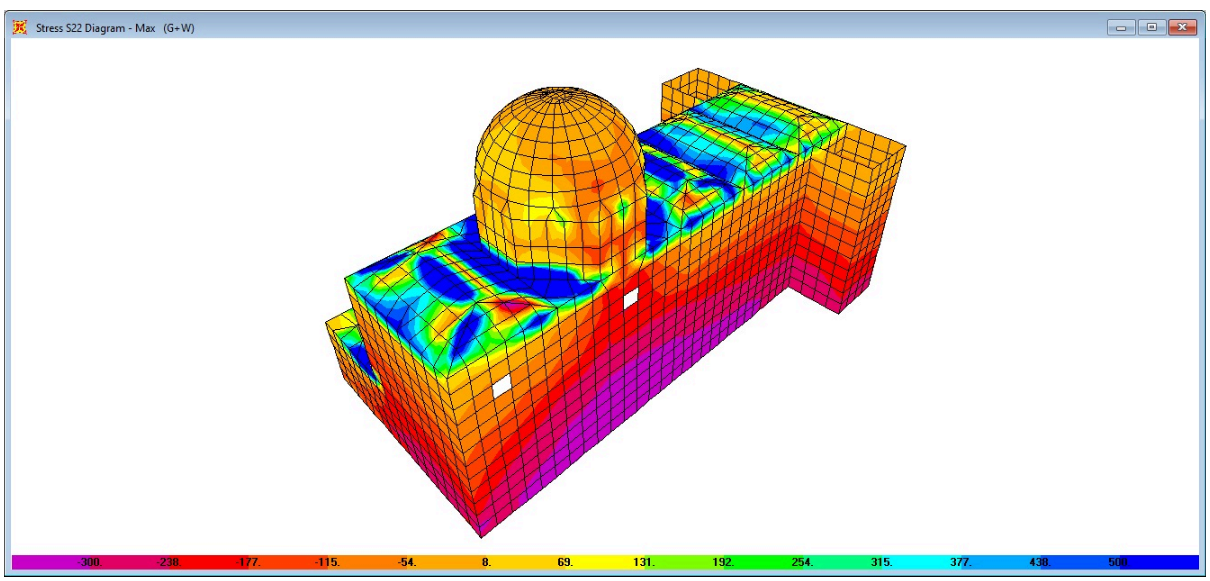

Figure 8. Compression and tensile stress distribution due to gravity and wind analysis in the repaired structure.

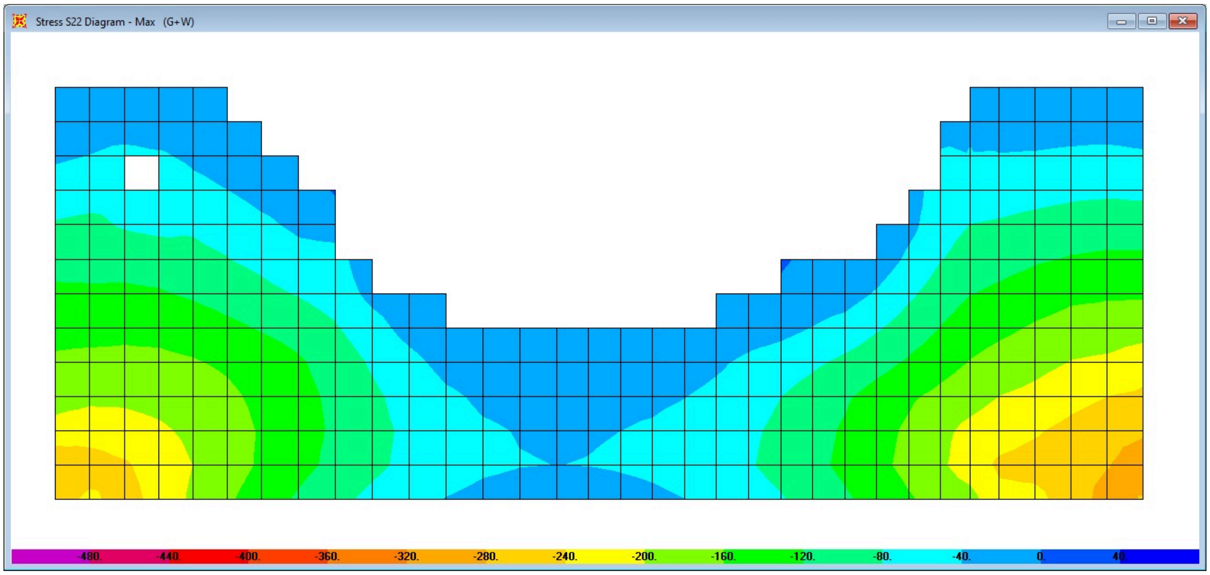

Figure 9. Compressive stress distribution due to gravity and wind analysis in the collapsed structure.

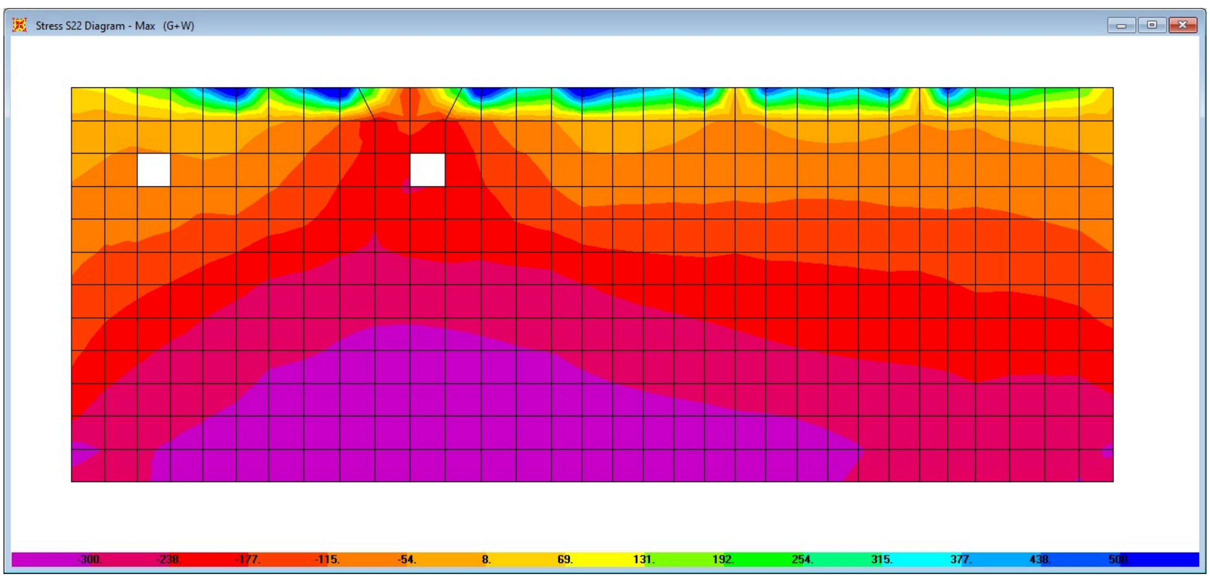

Figure 10. Compressive stress distribution due to gravity and wind analysis in the repaired structure.

CONCLUSIONS

Structural interventions on historic monuments have serious effects on their structural behavior and performance. Understanding the current condition of the building is the most important step in determining the safety level and evaluating the structural needs of the building. Proper documentation, accurate drawings, visual assessments, and the maximum possible amount of information about the background of the building is needed to evaluate its structural performance.

The northern wall of the San Jose Mission Church collapsed in 1868 and the roof caved in 1874. Considering the stability of the structure (Giuffre, 1990), material deterioration can be the most probable reason for this collapse. Given the long period of time, support settlements are not considered as serious to cause instability. Besides, San Jose mission complex has not been exposed to major site work to affect soil conditions and consequently affect the stability of the foundations of the church. On the other hand, the structure was abandoned and was exposed to various other possible threats for a long time until the restoration was started in 1937.

Finite element analysis is a very useful tool in this process as it enables a quick assessment and indication of the vulnerable parts of a structure, which may need future intervention. Due to the lack of seismic incidents in the region, the structure was analyzed only under gravity loads and wind loads. Should the structure be located in a seismic zone, dynamic analysis would be of utmost importance to determine its vulnerability. For San Jose Mission Church, regarding the results of the gravity and wind analysis, it could be said that the structure is safe; and the increase in the compressive stress values at the base of the wall due to the addition of a reinforced concrete dome and vaults do not pose a threat to its overall stability.

While studying on the structural performance of the building, it should be considered that any possible deterioration in the structural materials would cause weakening in the overall structural system. The mechanical properties that have been taken from literature based on research on similar structures may not reflect the actual performance of the materials. The most accurate results could be obtained by testing the materials. Though they are not generally authorized due to reasons concerning the cultural value of the monument or time and cost issues, semi destructive and destructive tests will provide the most reliable results to determine the mechanical characteristics this structure. Therefore, further studies on Mission San Jose church could involve material tests for actual mechanical properties of the structural materials.

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Windstorm Resilience of a 46-Story Office Building

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Abstract

This paper discusses the predicted performance of a 46-story office building subjected to wind loads. The lateral force resisting system in the building consists of a central core of reinforced concrete shear walls, and the gravity framing system consists of a composite floor system supported by structural steel beams and columns. Wind loads on the building envelope were determined using experimental tests on a scale model of the building in an atmospheric boundary layer wind tunnel. The structural response was determined using a finite element model of the building generated in SAP2000 software. Windstorm resilience was predicted by correlating the structural response (story drift, floor acceleration, and roof acceleration) to the sensitivity of both structural and non-structural building components. The FEMA P-58 software and tools, originally developed for seismic applications, were used to determine the building's fragility components and to simulate performance. Results showed that the wind-generated motion of the building was severe with substantial damage to cladding for the basic wind speed of 115 mph. Significant repair cost and repair time for the reinforced concrete shear walls was anticipated, in addition to repair costs and time associated with non-structural and other structural components.

INTRODUCTION

Background. In the United States buildings are conventionally designed using ASCE 7-10 procedures (ASCE 2010) to resist wind loads. Two of the ASCE 7-10 design procedures (the directional and envelope procedures) use equivalent static loads based on codified pressure coefficients based on the results of wind tunnel tests. The third procedure (wind tunnel testing) is often used for tall and slender buildings, or irregularly-shaped buildings. The latter procedure is important to capture dynamic behavior, such as vortex shedding or galloping response characteristics.

The conventional design of "ordinary" (i.e. risk category II) buildings for required strength is based on wind loads that have a 700-year mean recurrence interval (MRI), roughly corresponding to wind loads with a 7% probability of being exceeded in 50 years. For serviceability design, the story drift and wind-induced motion of the building is limited under wind loads that have a 10-year to 25-year MRI. In both limit states, an elastic design approach is employed: structural components are proportioned to have an available strength that is elastic, to have a stiffness that meets the drift limits, and proportioned to minimize unacceptable building motion. However, wind analysis and design methods for ensuring acceptable performance of a building are less developed.

Objective and Scope. The objective of this study was to assess the performance of a tall office building subjected to strength-level wind loads. In this study, the strength level wind loads are based on a 115 mph basic wind speed, corresponding to a 700-year MRI for most of the interior of the United States in ASCE 7-10.

The assessment was accomplished in two steps. In the first step, the building's nonlinear responses of drift and acceleration were predicted by analyzing a structural model of the building subjected to strength-level wind load histories. Wind tunnel tests were used to determine the wind load histories. In the second step, the FEMA P-58 methodology (FEMA 2012a,b), initially developed for seismic assessment, was used for wind assessment. Quantities and fragilities of structural and non-structural building components were determined using the FEMA P-58 Normative Quantity Estimation Tool (FEMA 2012c). Repair cost, repair time, and the probability of an unsafe placard being placed on the building were predicted via Monte Carlo simulations using the FEMA P-58 software, performance assessment tool "*PACT*" (FEMA 2012d).

METHODS

Building Description. The 46-story office building has a five-bay by three-bay, 150 ft by 100 ft, rectangular plan (Figure 1) and is 600 ft tall. The first story height is equal to 15 ft. Upper story heights are equal to 13 ft. The lateral force resisting system consists of reinforced concrete shear walls in the service core. Figure 2 shows a perspective view of the three-dimensional model of the building (Figure 2a) and shear wall core (Figure 2b). There are three types of shear walls. The first wall type is a 30-in. thick wall that extends from the foundation up to the 10th story (Figure 3a). The second wall type is a 21-in. thick wall that extends from the 10th story up to the 27th story, with one set of walls that terminate at the 15th story (Figure 3b). The third wall type is an 18-in. thick wall that extends from the 27th story to the 46th story, with one set of walls that terminate at the 32nd story (Figure 3c). Each wall type has two curtains of #10 rebar (vertical) spaced at 8, 10, and 12 in., respectively.

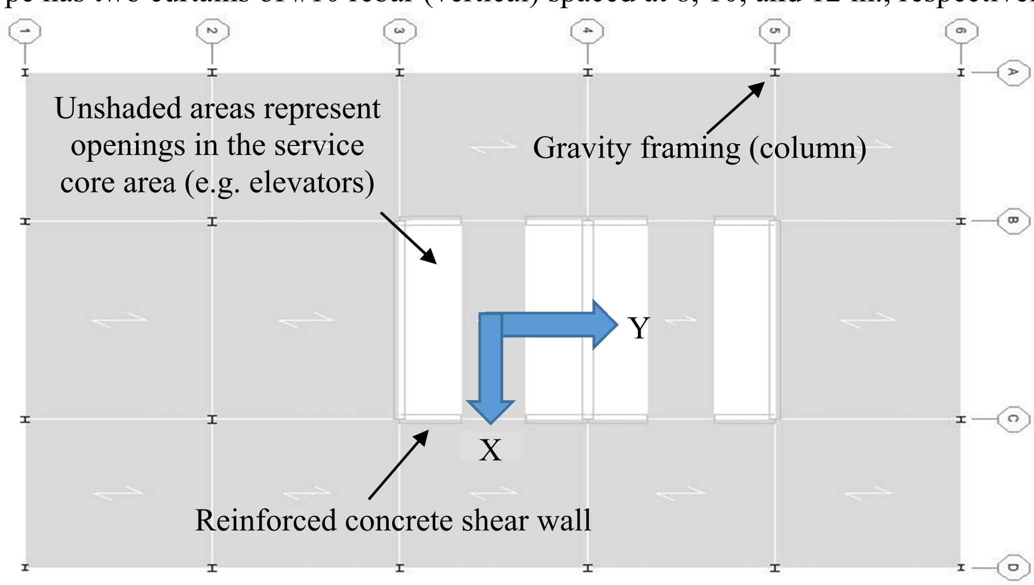
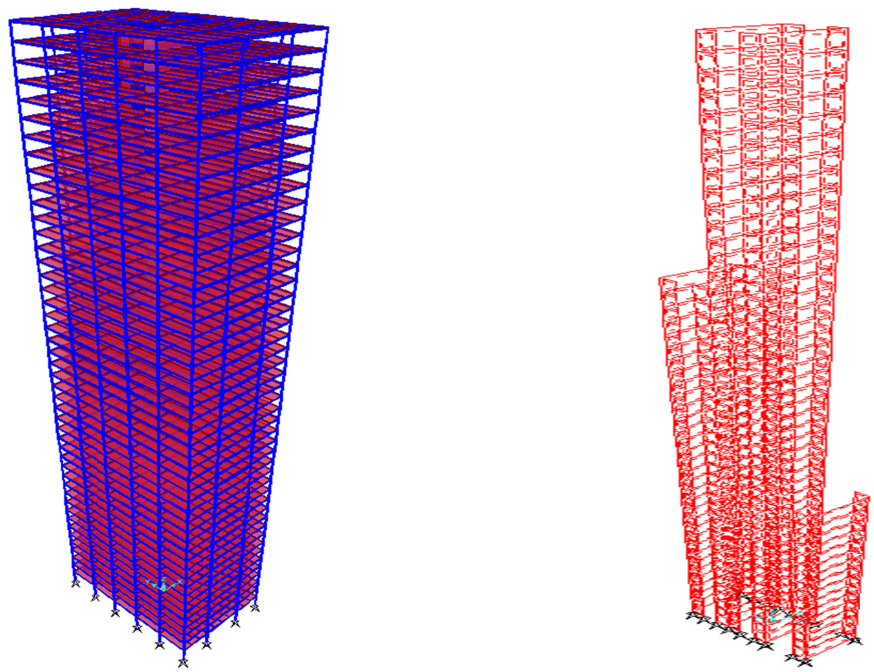
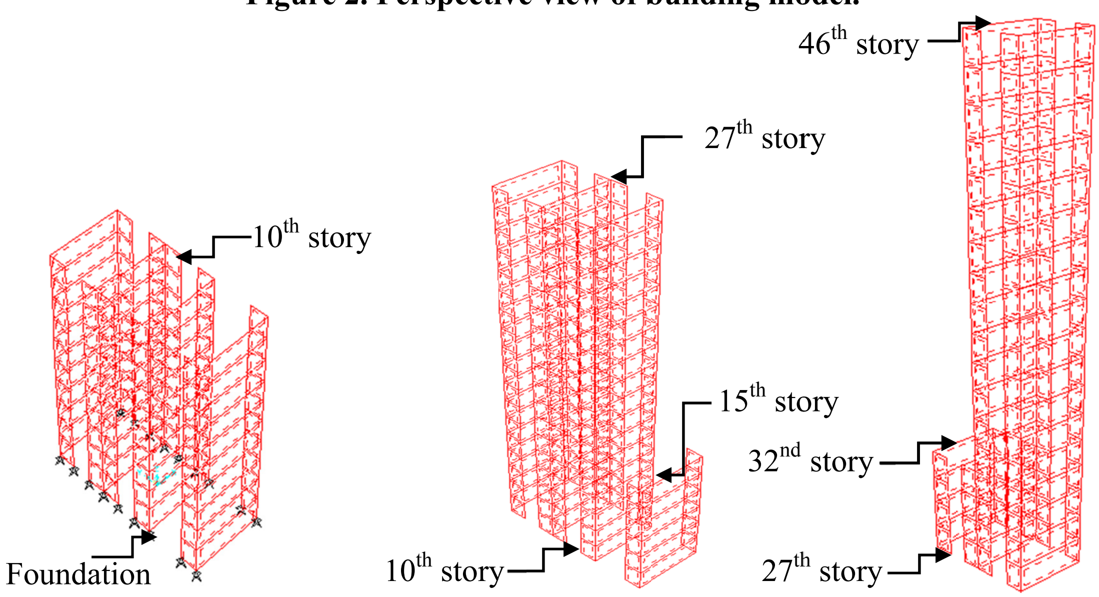


Figure 1. Typical plan view of building (plan at the 16th story).



a) Structural system b) Reinforced concrete shear walls

Figure 2. Perspective view of building model.



a) Type 1 (30-in.) b) Type 2 (21-in.) c) Type 3 (18-in.)

Figure 3. Reinforced concrete shear wall types.

The gravity framing system consists of suspended concrete slab on 3.5-in steel deck composite floor system supported by grade 50 wide-flange structural steel beams and columns. The total dead weight of the building is 77,380 kips, which is equivalent to a building "density" of approximately 8.6 lbf/ft^3. Steel columns range in size from a W14x22 to a W14x550. Beams vary in size from a W16x26 to a W24x68. A detailed description of the building is available in the associated thesis (Ghebremariam 2016).

Analytical Model. The building's structure was modelled using SAP2000 finite element software (CSI 2014). Table 1 summarizes the representation of the building's structural components, material types and typical sizes of the elements including their design strength used in the analytical model. Geometric nonlinearity (*P-Δ* effects) due to gravity loads was included. A nonlinear static load case including all gravity loads with linear material properties was used for determining the stiffness matrix to be used for nonlinear or linear analysis cases. This was equivalent to using a nonlinear static gravity load case, invoking its nonlinear material data in a geometrically linear model.

Table 1. Representation of Building's Structural Elements.

Component of Building	Material	Frame/Area Sections	SAP Element	Material Specification
Gravity Frame Elements	Steel	Ranges between W8x15 and W24x68	Beam	A992-50
		Ranges between W14x22 and W14x550	Column	
	Concrete	60"x18" (depth by thick.)	Beam	$f'_c = 4$ ksi
		60"x21"	Beam	$f'_c = 5$ ksi
		60"x30"	Beam	$f'_c = 6$ ksi
Floor	Concrete	Membrane Type-Light Weight Concrete 3.5	Slab	$f'_c = 4$ ksi
Shear Wall	Concrete	Shell-Thin Type Wall 18"	Wall	$f'_c = 4$ ksi
		Shell-Thin Type Wall 21"	Wall	$f'_c = 5$ ksi
		Shell-Thin Type Wall 30"	Wall	$f'_c = 6$ ksi

The nonlinear stress-strain behavior of the concrete and steel reinforcement materials was represented in the model (Figure 4). The structural steel beams and columns in the gravity framing were modeled using linear behavior. The floor decks at each story level were modeled as rigid diaphragms. The shear walls were designed using ASCE 7-10 for a basic wind speed equal to 115 mph with a terrain exposure category B. Strain compatibility analysis method was used for the shear wall design and the walls were modeled in SAP2000 as a nonlinear layered shell.

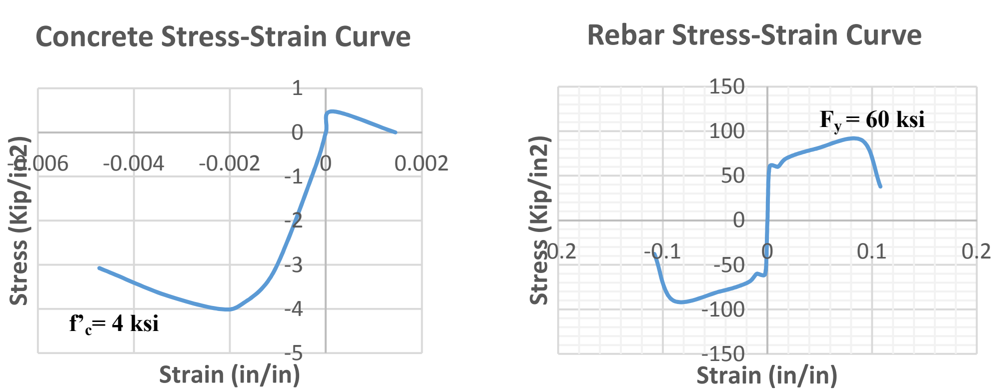


Figure 4. Typical idealized stress-strain behavior.

Wind Time History. Wind load histories were determined using experimental tests on a scale model of the building in an atmospheric boundary layer wind tunnel. The wind was applied to the model approaching from three different directions: 180°, 260°, and 80°. The wind loads for the analytical model were scaled from the test based on the wind speed of interest (115 mph). Ten unique wind load records were produced for the longitudinal direction (X), transverse direction (Y) and torsional direction (Z) at story 4, 10, 15, 20, 25, 30, 35, 39, 43, and story 45.

Figure 5 shows a typical wind load history for a wind direction of 180°, 115 mph speed, applied longitudinally (x) at story 5 ($F_{x1\_180\_115}$). The time history loads from the wind tunnel tests were modified (to avoid an initial impulse load) as follows: the records were initially ramped up for the first 300 seconds and applied constantly for the next 300 seconds with a sharp drop to zero at the last time step. The full scale storm duration was one hour with an output time step size of 0.204 s.

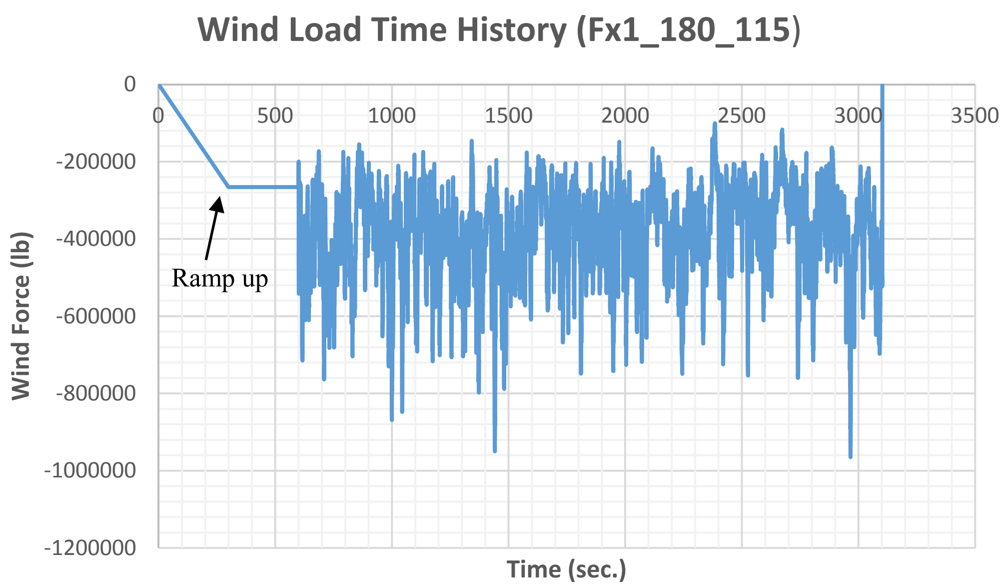


Figure 5. Typical wind load history (story 4, 180°, and 115 mph).

Preliminary Analysis. A geometric nonlinear analysis to account for gravity loads was conducted first. The analysis used a load combination of 1.05D + 0.25L, where D is equal to the floor dead load, and L is equal to the occupancy design live load. The load case was intended to represent the *expected* gravity loads during a windstorm, and is the same load case for seismic collapse analyses (FEMA 2009).

The natural periods of vibration and mode shapes of the building were determined using a Ritz vector mode analysis. Figure 6a shows the first mode of vibration. The computed fundamental period of vibration from SAP2000 was equal to 5.1 s. This value is approximately 11% smaller compared to a typical estimate for typical buildings up to 300 ft tall (5.7 s.), but is reasonable considering that analytical models usually lead to calculated periods of vibration that are on the order of 10% to 30% smaller than estimates. The reason for the difference in calculated period is possibly due to additional stiffness from gravity frames, cladding, and architectural walls, as well as the fact that the building height is double the reference height.