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SEISMIC EVALUATION AND UPGRADE OF A 15-STORY COMPOSITE STEEL-REINFORCED CONCRETE HOSPITAL BUILDING

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Abstract

After the 1994 Northridge earthquake, the state of California passed a law requiring all existing hospital buildings to be seismically evaluated and upgraded. Initially the buildings were required to meet basic life safety requirements before 2008. To remain in operation beyond 2030, they must show an improved level of performance. This paper describes advancing the analysis of a 15-story hospital tower in San Francisco from the initial basic life safety evaluation, to a modern dual performance criteria analysis. The hospital is a composite steel-reinforced concrete building built in the early 1950s. The lateral force resisting system consists of composite steel and reinforced concrete pier-spandrel frames on the perimeter with composite shear walls at the ends of the building wings. In the early 2000s, the building was analyzed using a three-dimensional nonlinear model made of centerline frame elements with lumped plastic hinges. The previous evaluation followed the Nonlinear Static Procedure (NSP) in FEMA 356 document, while using a Modal Pushover Analysis (MPA) to account for higher mode and torsional effects. The current building evaluation uses the Nonlinear Dynamic Procedure (NDP) in ASCE 41-13 standard, to show the structure meets the dual performance objectives of Damage Control (DC) at the BSE-1E (20%/50yrs) and Collapse Prevention (CP) at the BSE-2E (5%/50yrs), using 11 ground motion records at each seismic hazard level. In order to evaluate the building for improved performance using the NDP, many of the previous modeling assumptions were carried forward, and refined. The modeling parameters and acceptance criteria that define the unique composite concrete hinge backbone curves were verified through sub-assemblage modeling. The soil spring modeling was updated for dynamic analysis to achieve a rational hysteretic response. The reinforced concrete diaphragms were modeled with a combination of linear elastic shell elements, and nonlinear shells to allow some redistribution of loading in high stress regions. Similar overall building performance is observed when comparing the global results from the NDP to the previous MPA procedure. With localized retrofit extents, the building in general meets the dual performance objectives.

Keywords: ASCE 41; Composite Structure; Modal Pushover Analysis; Nonlinear Dynamic Procedure; Seismic Retrofit

1. Introduction

Following the 1994 Northridge earthquake in southern California, 12 hospital buildings were "red tagged" as unsafe to occupy and numerous others were damaged resulting in approximately $3 billion in losses [1]. In response, the state of California passed a series of regulations requiring the seismic evaluation and upgrade of all acute care hospital buildings. The first milestone of the regulation required all buildings to meet the basic life safety requirement before 2008. To remain in operation beyond the year 2030, all hospital buildings must show an ability to continue operations following an earthquake with limited interruption.

A 15-story hospital tower in San Francisco was initially seismically evaluated in the mid 2000s to confirm it met the basic life safety requirements. This assessment was state of the art at the time, using a full nonlinear three-dimensional model, and following the newly developed Modal Pushover Analysis procedure by Chopra and Goel [2]. The building was shown to meet this initial performance goal, but in order to remain in operation beyond the year 2030, it must now be re-evaluated to a higher performance criteria as illustrated in Fig. 1. This criteria requires limited damage during a minor seismic event and the ability to retain some margin against collapse during a major seismic event.

In order to show an improved level of performance, a nonlinear response history analysis was deemed necessary. Many of the modeling assumptions from the previous Modal Pushover Analysis were extended to the new analysis, while others required verification through sub assemblage modeling, or further refinement. Before embarking on the full dynamic analysis, the original modal pushover analysis was re-run at the new hazard levels to determine feasibility. The resulting building drifts and identified deficiencies from this analysis are compared to the results from the nonlinear dynamic analysis.

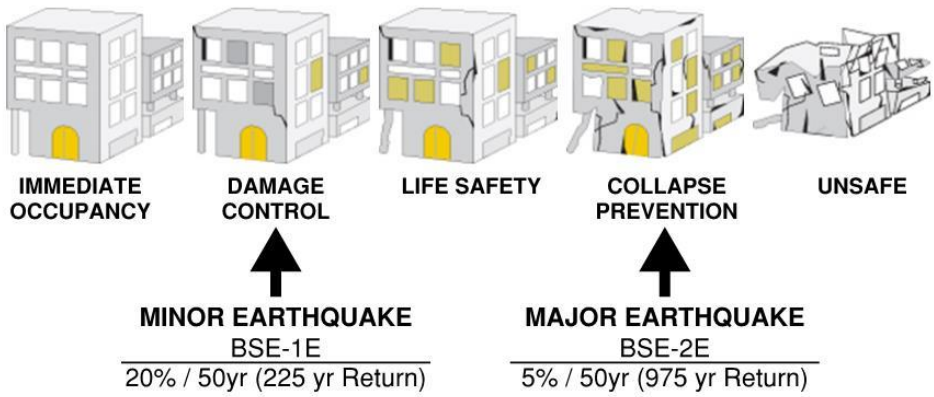


Fig. 1 – Targeted Performance Criteria

2. Building Description

The building is a 15 story hospital building with one basement level located in San Francisco, California. The building was constructed in the early 1950s, and is cruciform in shape with a large setback at the sixth floor. The hospital has undergone some minor modifications over the years including a light frame addition on the 6th floor in the 1960s, infilling and cutting select new openings to connect to an adjacent building in the 1970s, and a new light frame penthouse on the roof of the north wing in the 1990s.

Typical of large buildings of this era, it was constructed first with a complete steel frame, and then cast in concrete. Fig. 2 shows the original building under construction, with the steel frame leading, and the concrete walls and floor slabs following. The gravity system of the building is a 5-inch reinforced concrete slab supported by steel beams and columns, both wrapped in wire mesh reinforcing and encased in concrete. All concrete above grade is light-weight, while the basement walls and footings below are cast with normal weight concrete. The building is supported on concrete spread footings founded between 10 and 20 feet below the basement level on firm undisturbed soils, with shallow bedrock.

Composite steel and reinforced concrete shear walls at the exterior of the building provide the primary lateral load resistance. The walls are typically 12 inches thick with two curtains of reinforcing detailed to either wrap around the steel framing or welded to it to provide composite behavior. At the ends of each wing are generally solid shear walls, while the walls along the long faces of the building are pierced with regularly spaced openings forming a pier-spandrel system. The wall piers are typically 90 inches long, and “T-shaped” in plan with a steel WF section column encased at the center. The spandrels are typically 6-feet deep steel truss girders, centered at each floor level, and spanning 22 feet between steel WF columns (approximately 14’-6” clear span). Most walls are continuous from the foundation to the roof.

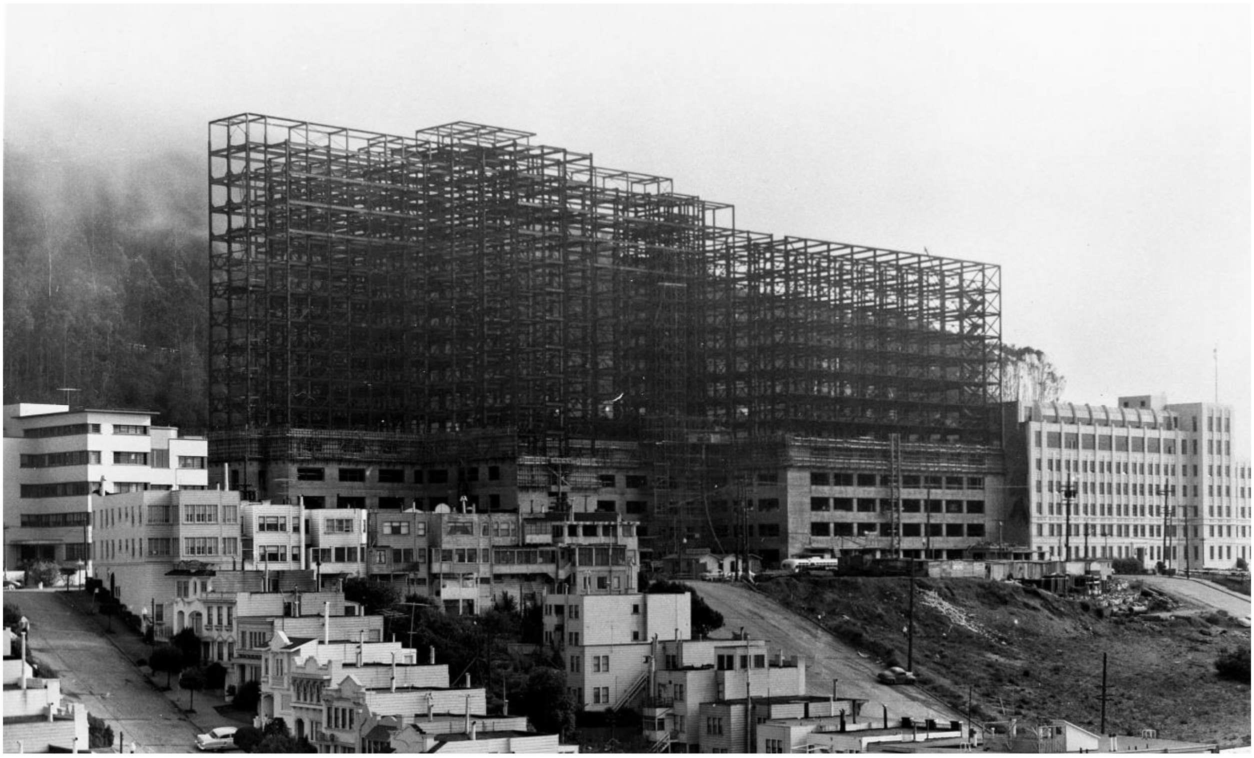


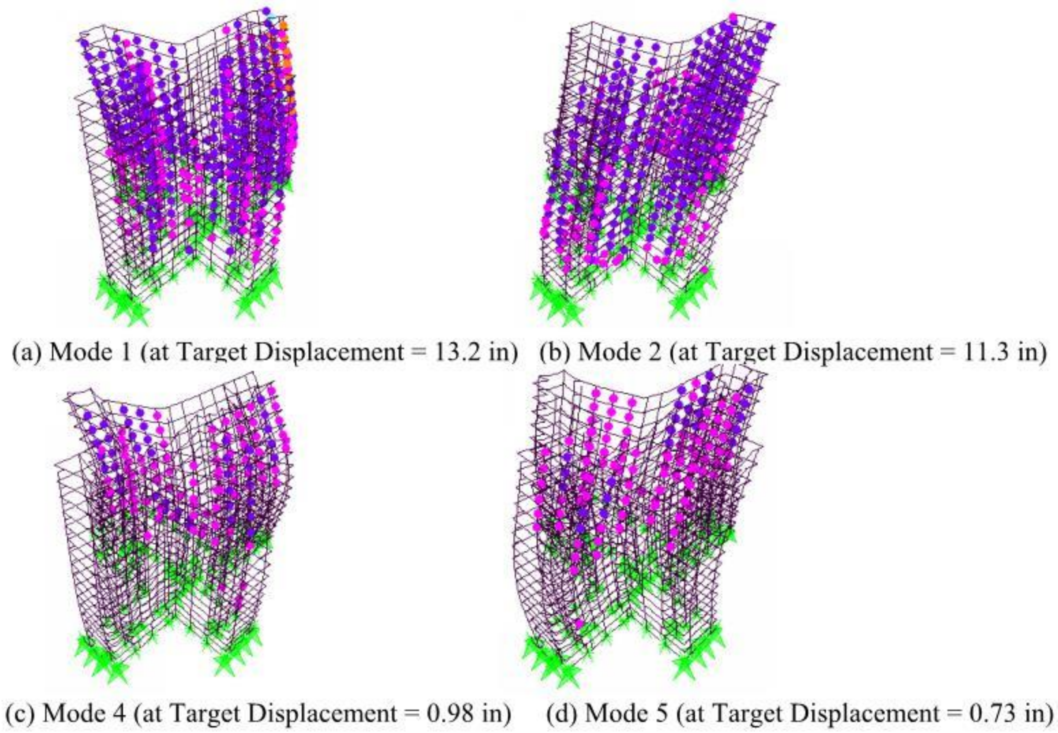
Fig. 2 – 15 Story Hospital Tower in Construction (circa 1952)

3. Prior Life Safety Seismic Evaluation

The hospital tower was previously seismically evaluated in the mid-2000s using the FEMA 356 document. The building was evaluated to the basic Life Safety criteria at the BSE-1 earthquake hazard level, which is defined as the smaller of the 10% in 50-year probabilistic hazard, and 2/3 of the Maximum Credible Earthquake (MCE). A number of methods were considered for the building evaluation including nonlinear static and dynamic analyses. A three-dimensional (3D) nonlinear dynamic analysis would have provided the most rigorous method to estimate building response, but at the time it was deemed computationally not practical for such a large and complex building. Instead, a Modal Pushover Analysis (MPA) was chosen because it provided a satisfactory balance between accuracy and practicality. The MPA procedure was able to capture the 3D lateral-torsional response of the irregular plan layout and incorporate the higher mode effects of the tall building, that would not have been possible with a traditional nonlinear static pushover [3].

The strength and stiffness of the composite steel and reinforced concrete shear walls, piers, and spandrels were determined through a number of representative section analyses. The standard guidelines provided in FEMA 356 for determining the effective elastic stiffness of conventional reinforced concrete sections were deemed not appropriate for this composite construction. Instead moment-curvature analyses were performed on representative sections with the embedded steel shapes explicitly included. The effective linear elastic stiffness was determined at the point of first yield. A similar section analysis was used to determine the Axial Force-Moment (P-M) diagrams for the flexural strength of the section.

The three-dimensional building model was assembled in SAP 2000 [4]. The composite shear walls, piers and spandrels were modeled using centerline frame elements with rigid end offsets, and lumped plastic hinges. FEMA 356 defined the shapes of the force-deformation curves and the acceptable hinge rotations based on those for reinforced concrete sections [5]. The model utilized rigid diaphragms, with the weight of each story lumped at the center of mass together with its mass moment of inertia. The entire model was supported on nonlinear vertical soil springs meant to capture the rocking behavior of the foundations during uplift. The building response and plastic hinge patterns for the first four translational modes used to evaluate the building are shown in Fig 3.



(a) Mode 1 (at Target Displacement = 13.2 in) (b) Mode 2 (at Target Displacement = 11.3 in)

(c) Mode 4 (at Target Displacement = 0.98 in) (d) Mode 5 (at Target Displacement = 0.73 in)

Fig. 3 – Plastic Hinge Patterns at First Four Modes of MPA

4. Seismic Evaluation for Improved Performance

In order for the hospital to remain in operation beyond the year 2030, the state of California requires that it be evaluated and upgraded to meet an improved level of seismic performance. This criteria is defined as limited damage during a minor event to minimize building repair time and operation interruption. During a major event, the structure must retain some margin against collapse with the understanding that significant structural damage, including large permanent lateral deformations may occur, and that the building will likely not be safe to re-occupy. Over the past 10 to 15 years, advancements in building analysis programs have now made it a practical option to conduct a dynamic nonlinear response history analysis for such a large and complex building. Not only was it now practical, but in order to rigorously prove that such a large and complex building met the improved dual performance criteria, this method was deemed the most prudent.

A site-specific hazard analysis was completed to determine the level of ground shaking at both the minor and major seismic events. The minor earthquake is defined as that having less than 20% chance of exceedance in 50 years (a 225 year return period), and the major earthquake is defined as that having less than a 5% chance of exceedance in 50 years (a 975 year return period). These earthquakes are termed the BSE-1E and the BSE-2E respectively in the ASCE 41-13 Standard for Seismic Evaluation and Retrofit of Existing Structures [6]. For each hazard level, a suite of 11 bi-directional ground motions were selected and scaled. The response spectrums for each suite of ground motions are shown in Fig. 4.

Much of the modelling assumptions from the previous Modal Pushover Analysis were able to be used in this building evaluation. Model geometry, most of the centerline frame modeling, and the effective strength and stiffness of the composite steel-concrete walls were all verified and input into the new building model created in CSI-PERFORM-3D [7]. Additional sub-assemblage modeling of the composite steel and reinforced concrete spandrels was completed to confirm that the force deformation relations provided in ASCE 41-13 provided sensible estimates of the nonlinear response. The nonlinear soil springs were modeled with additional gap and drag elements to achieve the appropriate hysteretic response under ground motion excitation. Finally, the diaphragms were explicitly modeled; first as semi-rigid, and then with nonlinear elements in areas of higher stress to better capture the local force redistribution and the torsional response of this cruciform shaped building.

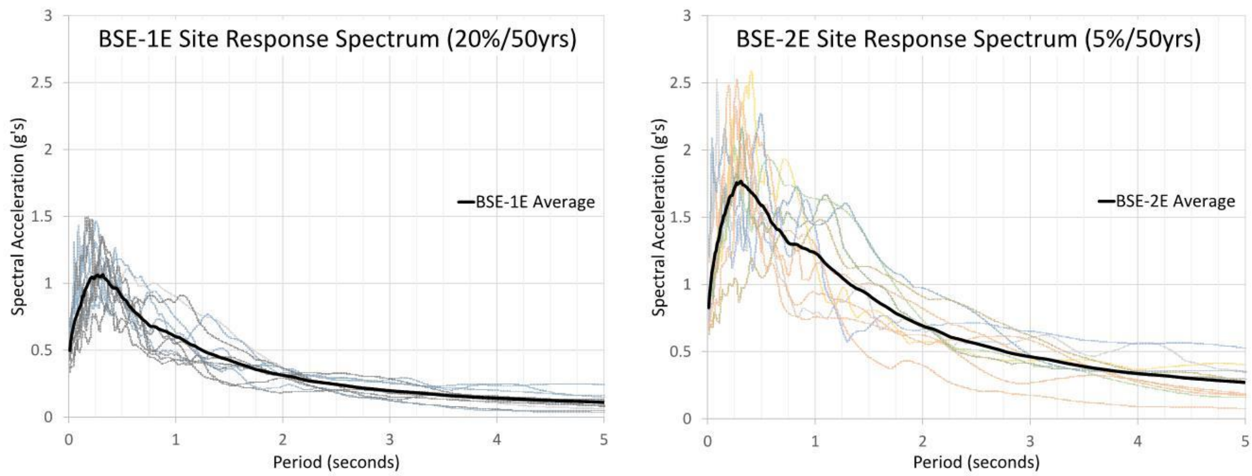


Fig. 4 – Response Spectrum for Dual Hazards

4.1 Sub Assemblage Modeling

For the composite steel-concrete walls, the previous building evaluation used the nonlinear force-deformation relationship values published in FEMA 356. These values are for typical reinforced concrete walls with nonconforming transverse reinforcement and unconfined boundary elements. It was assumed that these values would provide a practical and conservative estimate of the overall ductility of the composite system for the basic Life Safety performance objective. This previous assumption had to be validated before proceeding to evaluate the building for improved performance. A sub-assemblage study of a typical concrete encased steel spandrel truss was used to evaluate and justify the adequacy of these modeling assumptions for the flexural hinge properties used in the PERFORM-3D building model.

Three representative models were developed and analyzed using nonlinear static pushover analyses. Sub-assemblage "A" represented the simplified model included in the overall building analysis; a single centerline frame element with lumped plastic hinges at each end following the force-deformation curves published in the ASCE 41-13 standard. Sub-assemblage "B" modeled the embedded steel truss directly using frame elements with nonlinear fiber sections, in parallel with the reinforced concrete section modeled as a centerline frame element with lumped plastic hinges. Sub-assemblage "C" modeled the entire spandrel truss section using shell elements with nonlinear fibers for the concrete, rebar, and truss chords.

Fig. 5 plots the response of all three sub-assemblage models, along with their respective acceptance criteria limits for Damage Control (DC) and Collapse Prevention (CP) (based on component strain limits). The plot shows that both refined sub-assemblage models exhibit larger maximum moment capacity, and overall greater deformation capacity than the first, simple centerline frame sub-assemblage. Model "B", the fiber truss model, shows a shorter peak strength plateau, but as the hinges in the reinforced concrete frame lose strength, the load is able to redistribute to the fiber truss elements retaining at least 40% of the maximum moment capacity until the truss diagonal reaches its compression limit (corresponding to concrete spalling at approximately 0.04 radians). Model "C", the fiber shell model, shows a considerably longer peak strength plateau, with strength loss occurring when the truss chord reached the tensile strain limit of 5%. It was determined from these sub-assemblage models that it is reasonable and conservative to use the modeling parameters and acceptance criteria published in ASCE 41-13 for the composite steel-concrete spandrel trusses in the global building model.

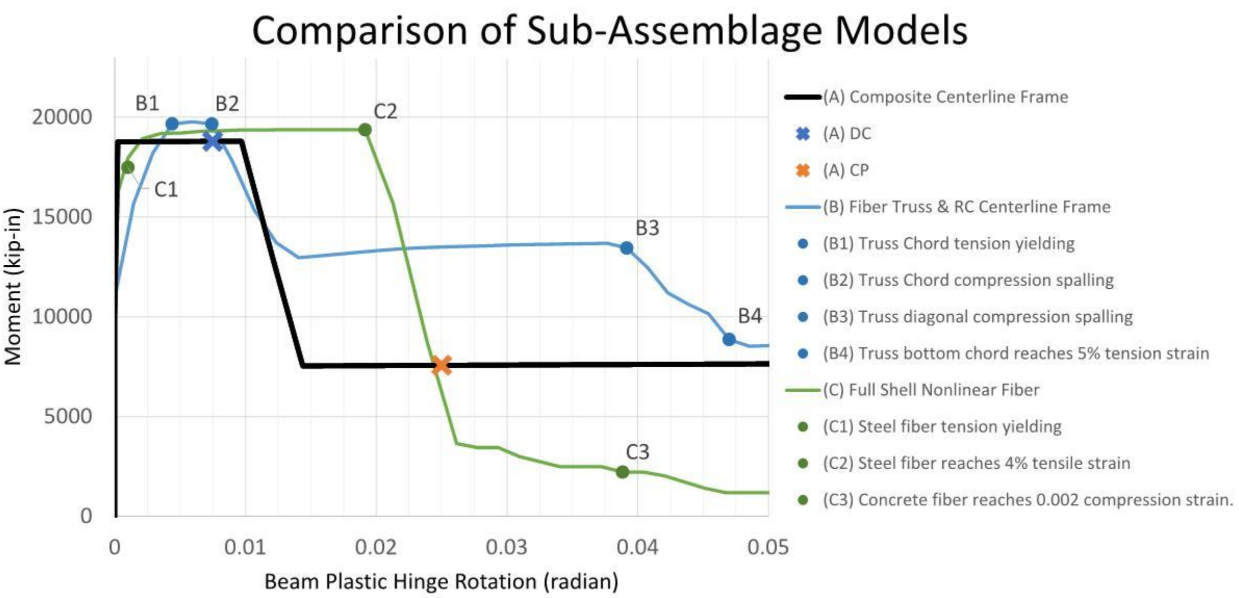


Fig. 5 – Sub-Assemblage Modeling

4.2 Soil Springs

Nonlinear vertical soil springs in the model capture the rocking behavior of the building, particularly at the end walls. The previous nonlinear static analysis calculated equivalent soil springs per FEMA 356 based on the dimensions of the spread footing, depth of embedment, and estimated effective shear modulus of the soil. The ultimate bearing and the uplift capacities (based on uplifting the weight of the spread footing and soil directly above) were also included to create an elastoplastic spring element. The previous static analysis was able to use just one element to model this spring. In order to allow the springs to uplift and gap during dynamic loading without accumulating tensile strains, the soil springs had to be remodeled with multiple elements.

The NIST report for Soil-Structure Interaction of Building Structures [8], provides some practical guidance on complex foundation modeling, and appropriate hysteretic response. For this foundation type, they detail a system of (3) elements; a Compression Spring in series with a Tension Bar and a Gap Element (the latter two elements act in parallel). The Compression Spring defines the elastic stiffness and bearing capacity of the foundation, the Tension Bar is a rigid-plastic element that defines the uplift capacity of the foundation, and the Gap Element allows the whole system to uplift and cycle back down to bearing without accumulating tensile strains. Fig. 6 shows a representative section showing the basement wall and shallow foundation for the hospital we are modeling. It also shows as a schematic of the soil spring configuration used, and the resulting hysteretic response.

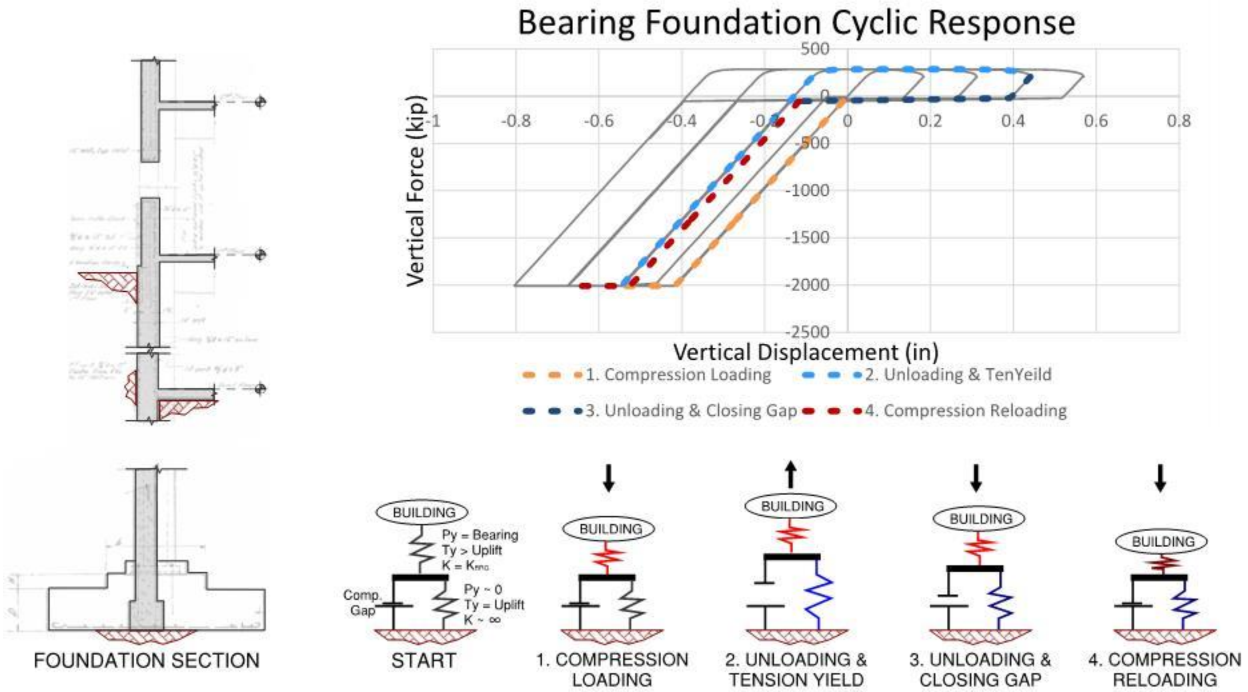


Fig. 6 – Soil Spring Modeling

4.3 Diaphragm Modeling

The previous building evaluation relied on a rigid diaphragm assumption. Based on preliminary analyses, we determined that the diaphragm strength and flexibility could have a significant effect on the lateral-torsional response of the building. Furthermore, by including the diaphragm in the model this would allow us to directly evaluate the acceptability. We began by modeling the diaphragm with elastic shell elements, and then ran the analysis checking the force levels at critical locations to determine where some minor yielding and force redistribution might occur. The diaphragms were found to exceed their elastic strengths at select locations in the upper floors; particularly at the central core of the building, and at locations next to the stiffer end shear walls where large floor openings occurred.

At these identified high stress locations, the diaphragms were modeled with nonlinear shell elements and the steel gravity framing had partially restrained axial hinges assigned to their connections. The capacity of the shear and flexure fibers of the slab shells were based on the strength of the reinforced concrete flat slab, reduced for stresses due to gravity loads. The material force-deformation curves were defined as those for reinforced concrete shear walls with low axial load per the ASCE 41-13 standard. The steel gravity floor framing act as diaphragm drags, and required axial hinge releases to allow for in plane flexure deformations. The steel beams are bolted and riveted to the steel columns with partially restrained moment connections, which have rotation-based modeling parameters and acceptance criteria defined in the ASCE 41-13 standard. These parameters were translated to relative axial deformations by multiplying the rotation dimensions by one half the beam depth. The resulting acceptable axial elongations at Damage Control ranged from approximately 0.07in to 0.20in and at Collapse Prevention they ranged from approximately 0.13in to 0.63in, depending on the depth of the connection and the detailing.

5. Comparison to Preliminary Modal Pushover Analysis

Before endeavoring on a full nonlinear dynamic analysis, the original model was used to re-run the modal pushover analysis (MPA) at the two new hazard levels. This was done to rapidly determine the feasibility of this building to meet the new performance criteria, and to identify possible deficiencies early. The plots in Fig. 7 show the building drifts estimated using the original model following the MPA procedure, overlaid on drift profiles later obtained from the nonlinear dynamic procedure (NDP). Comparing these early drift predictions to the results of the more refined analysis, we can see that they were able to provide a reasonable approximation of the maximum building drifts. In the north south direction, the MPA drift profile follows a similar shape as the dynamic analysis, but generally underestimates the drifts, particularly at the upper levels. In the east west direction, the MPA procedure produced a more conservative drift profile, with an exaggerated irregularity at the 6th floor where the building setback occurs. From these drift profiles and the hinge deformations estimated in the Modal Pushover Analysis, the building was identified as a good candidate for full evaluation and upgrade.

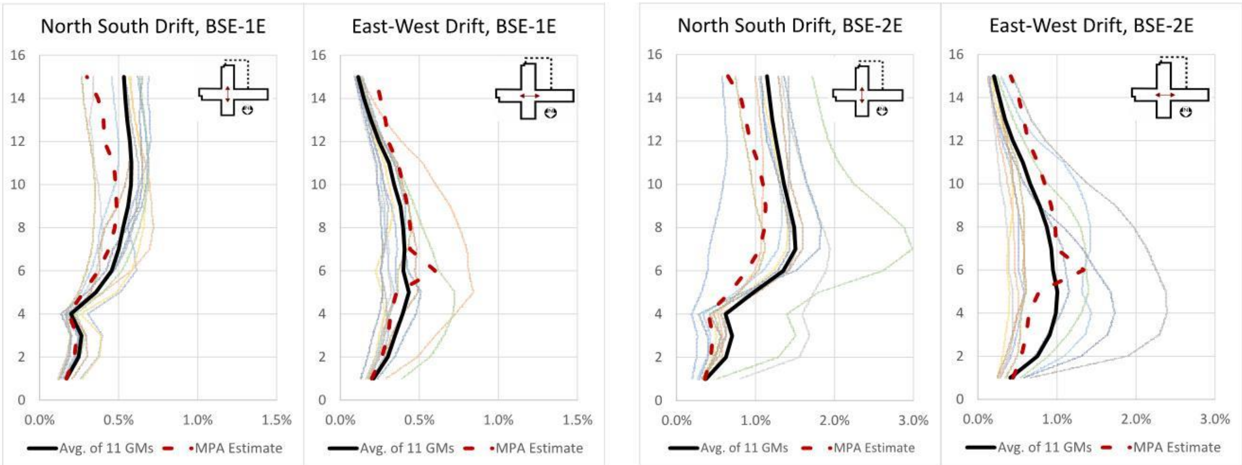


Fig. 7 – Comparison of Building Drift Profiles under MPA and NDP

6. Conclusions

Early on, during the preliminary MPA, the diaphragms were identified as a possible deficiency to monitor. The building was confirmed to have some torsional response due to the cruciform plan layout. This resulted in the wings tending to twist about the central core, concentrating shear and flexural demands. At these same locations near the core, there are significant openings in the diaphragm for mechanical shafts, stairs and elevators. The west wing in particular is longer than the others creating higher demands, and the main stairwell aligns with the elevator and mechanical shafts, creating a weak net section. The detailed dynamic analysis confirmed that the narrow segment of diaphragm at the top two floors of the west wing was overstressed. The proposed retrofit solution details external diaphragm bracing using buckling restrained braces (BRB) that collect north-south transverse diaphragm shear forces in the west wing and deliver them to the in plane concrete shear walls on the south wing. Fig. 8 schematically highlights the deficiency and the proposed retrofit.

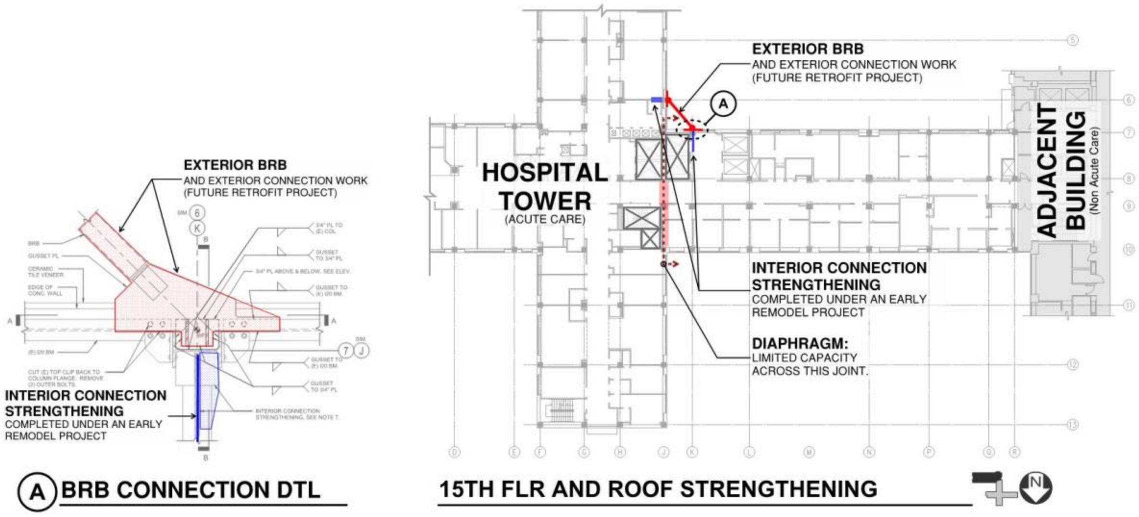


Fig. 8 – Schematic of Proposed Diaphragm Strengthening

The use of the MPA procedure allowed us to rapidly and accurately evaluate the feasibility of this tall complex building meeting the improved performance goals mandated by the state of California. Many of the modeling assumptions from this previous analysis, were then able to be refined and carried forward to run a nonlinear dynamic analysis of the building. Sub-assemblage modeling provided confidence in the assumptions for the nonlinear hinges defined for this unique composite construction. Improvements to the soil spring modeling and nonlinear diaphragm modeling allowed us to better capture the dynamic behavior, yielding, and force redistribution in the building.

7. Acknowledgements

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7. References

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