SEISMIC RISK ASSESSMENT AND STATE-OF-THE-ART SEISMIC RETROFIT OF TWO TOWERS IN SACRAMENTO, CA

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Abstract

Two multistory office buildings in Sacramento, California were investigated for their expected performance in future earthquakes. The structures were constructed in the 1970s per the 1967 edition of the Uniform Building Code (UBC), use perimeter reinforced concrete moment frames to resist seismic loading. The buildings are rectangular in plan and have certain characteristics that adversely affect their seismic performance, the presence of a soft-story response at the first floor (approximately 50% taller than typical floors), and limited ductility typical of buildings of that era. Risk analysis showed that for the towers the Probable maximum loss (PML or the 90-percentile loss) exceeded 20%. Nonlinear response history analysis (NLRHA) of the towers was conducted and showed that in the existing configuration, the story drift ratios (SDRs) at the first floor exceeded 2%, shear hinging of the first-floor beams was expected and that the SDRs would need to be reduced to approximately 1.4% for the first floor to limit the extent of nonlinear response. Seismic retrofit included addition of 300-kip viscous dampers in both directions to the first floor of the building. Analysis showed that the retrofitted structure had a first floor SDR of approximately 1.3% and that the soft story response and plastic shear hinging of first floor beams were mitigated. FEMA P-58 analysis of the retrofitted buildings were then conducted using the results—SDR, story acceleration, and residual drifts—from the NLRHA. It was seen that the PML was significantly reduced and was now less than 20% which is the acceptability threshold.

Keywords: Seismic dampers, Seismic risk analysis, Non-ductile concrete, soft-story, seismic retrofit

1 Introduction

The office towers investigated, located in downtown Sacramento, CA, are of very similar construction. For the purposes of this study, the west tower was specifically modeled and evaluated. The west tower is a 14-story reinforced concrete moment frame building constructed in 1971. The building has a plan dimension of approximately 150 x 81 ft. The building is rectangular in plan. Typical floors measure 11 ft in height, whereas, the first floor is 17 ft tall.

Gravity loading is resisted by a system consisting of 8-inch thick post-tensioned 2-way flat slab and reinforced concrete columns bearing on a pile foundation system.

The lateral load is resisted by a system of reinforced concrete perimeter moment frames. Fig. 1 presents the plan view of a typical floor and the location of the lateral load resisting system. Columns are 28-in square and support 18x40 in. beams on the second floor and 18x30 in. beams at the floors above. In the longitudinal direction, there are five 29-ft long bays on each side (10 bays total), and in the transverse direction, there are three 26-ft long bays on each side (6 bays total).

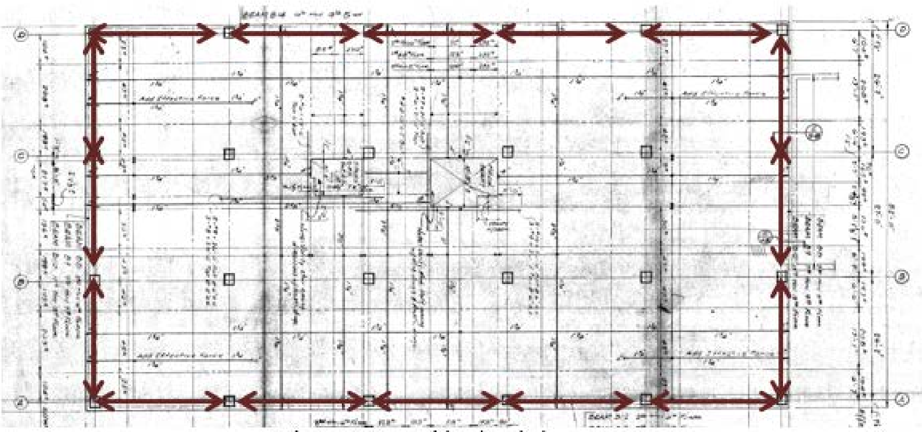


Figure 2. Lateral load resisting system

Fig. 1 –Lateral load resisting system

The building site is classified as Class D. For the building site and soil conditions, the USGS [1] provides the following design parameters: short period acceleration of 0.53g, and 1-sec acceleration of 0.31g. These values are used to construct the seismic demand for the building. The seismicity at the site is considered moderate. Site visits to the building had shown that the structure has not experienced any noticeable damage from past earthquakes, including the recent 2014 South Napa event

2 Building characteristics

Reinforced concrete buildings constructed prior to adoption of new seismic codes are usually classified as having low ductility. In other words, such buildings do not have the means to resist the seismic energy that is imparted to them by earthquakes. The low-ductile reinforced concrete buildings have performed poorly in past earthquakes. The structure under consideration has several key design features that enhance its earthquake resistance, including the following:

- Structural configuration. The building is regular in plan, with no re-entrant corners or vertical off-sets. Regular buildings have performed well in past earthquakes.
- Close stirrup spacing at the beam-to-column joints. The plans show stirrup spacing of 4-in. on center for beams and tie spacing of 3.25-in. on center at the joints. The reinforcement is shown with 135-degree

hooks. Such close spacing of transverse reinforcement would prevent the buckling of reinforcement at the location of highest seismic stress.

The structure under consideration has several key design features that reduce its earthquake resistance, including the following:

- Soft-story response. The first story of the building is approximately 50% taller than the stories above. Buildings with such configuration can be vulnerable to earthquake damage because the deformation and damage is concentrated at the first floor, while good design typically results in uniform distribution of lateral deformation among all floors.
- Transverse reinforcement. The beams have stirrup spacing of 18-in. and 13-in. on center near midspan at the second level and above, respectively. The mid-height column tie spacing is 12-in. on center. These values exceed the current code limits and can lead to premature failure in some members.
- Shear capacity of beams. Beams are constructed of lightweight concrete and use No. 3 or 4 transverse bars spaced 18 in. or 13 in. on center at middle third of the members, thus having limited shear capacity. Modern codes attempt to mitigate shear failure by requiring ductile flexural damage prior to shear failure.
- Splices and development length. The tension lap splices for the beams do not meet the current code requirements. The column #18 to #14 longitudinal bar splices use cold-welded couplers. Inadequate splice and development length can lead to bar pullout and prevent reinforcement from reaching its capacity.
- Column ties. The code requires that every other longitudinal reinforcement have a tie around it. This requirement is not met for the Type D columns shown in the plans. This code requirement is intended to prevent buckling of longitudinal reinforcement at locations of high seismic loading.
- Joint eccentricity. Eccentricity in the line of action between beams and columns will amplify loading on the members.

3 Seismic performance of existing building

3.1 Overview

ASCE 41-13 [2] provides comprehensive requirements for seismic evaluation and upgrade of existing buildings and was used for this structure. Computer program ETABS [3] was used to prepare a three-dimensional mathematical model of the building; see Fig. 2. This model was used to assess the performance of the existing building moment frames. Nominal material properties, spans and member sizes specified in the original construction documents were used in analysis. Dimensions were based on centerline dimensions provided in the drawings. Gravity loading on the building is composed of member self-weights, design live load and additional dead load to account for non-structural elements such as flooring, ceiling, and duct work, which is distributed uniformly on floor slabs. The concrete floor diaphragms are modeled as rigid, meshed shell elements. The seismic loading was based on values obtained from the USGS web site [1] for the design earthquake (475-year event).

Building codes allow for both linear and nonlinear analysis. When linear analysis is used, there are certain conservatisms built into the results to account for the modeling and analysis assumptions. By contrast, nonlinear analysis attempts to model the behavior of the building and its components in greater detail, resulting in greater accuracy of the results, thusly requiring less conservatism. For this structure, nonlinear analysis was utilized to compute capacities and the principal of equal displacement was applied to demands. In other word, displacement-based (or performance-based), rather than force-based, methodology was employed.

3.2 Story drift ratios

Fig. 3 presents the computed drift ratios at the design earthquakes. Drift ratios are one of the most telling parameters in evaluating the response of a building, as they correlate directly to the demand on structural members and drift-sensitive structural components, such as partitions. The building codes place limits on drifts at the design-level earthquake.

Fig. 2 –Mathematical model of the building

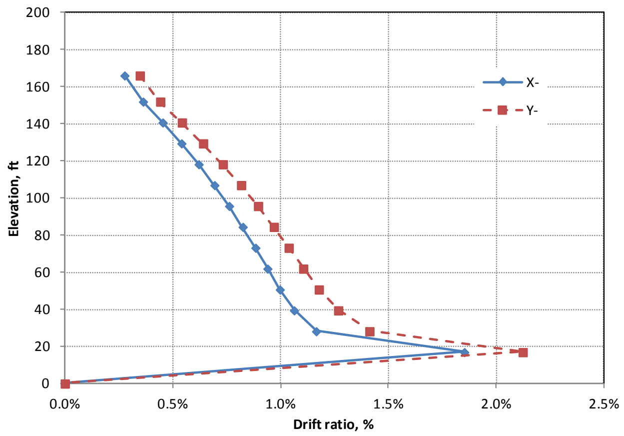


Fig. 3 –Drift ratios at design earthquake

Fig. 3 reveals the following:

- Drift ratio in the Y- (transverse) direction are larger than in the X- (longitudinal) direction. This is because there are fewer moment frame bays in the Y-direction.
- Drift ratios at the first floor are the largest because of the soft story present at this level. First floor drift in the Y-direction exceeds 2%.

For multistory non- or low-ductile reinforced concrete moment frame buildings, the target drift ratio is typically set at approximately 1% to 1.5%. At 1% or below, the structure is unlikely to experience any damage. The 1.5% value is referred to as nearly elastic—implying that there will be some small level of nonlinearity but the damage is likely to be localized and minor. A review of Fig. 3 shows that upgrade measures should be considered for the first floor or two. For multistory non-ductile reinforced concrete buildings, drifts need to be kept to 1.5% or lower.

3.3 Pushover analysis

Preliminary nonlinear analysis of the structure was conducted. For this analysis, it was assumed that all reinforcement as shown in the plans will be fully developed and that bending nonlinearity would only occur near the joints. Additionally, given the low capacity of concrete beams in shear, the model incorporated nonlinear elements at midspan of the beams. Key findings are summarized in Table. 1. The deformed shapes of the perimeter frame for the building in its existing condition during design earthquake is shown in Fig. 4.

Table. 1 Key pushover analysis results

		Displacement, mm		**SDR %**	
Step	**Level**	**X-**	**Y-**	**X-**	**Y-**
Target displacement	Roof	432	508	1.9	2.2
	First	99	114		
Onset of damage	Roof	356	330	1.6	1.4
	First	81	74		

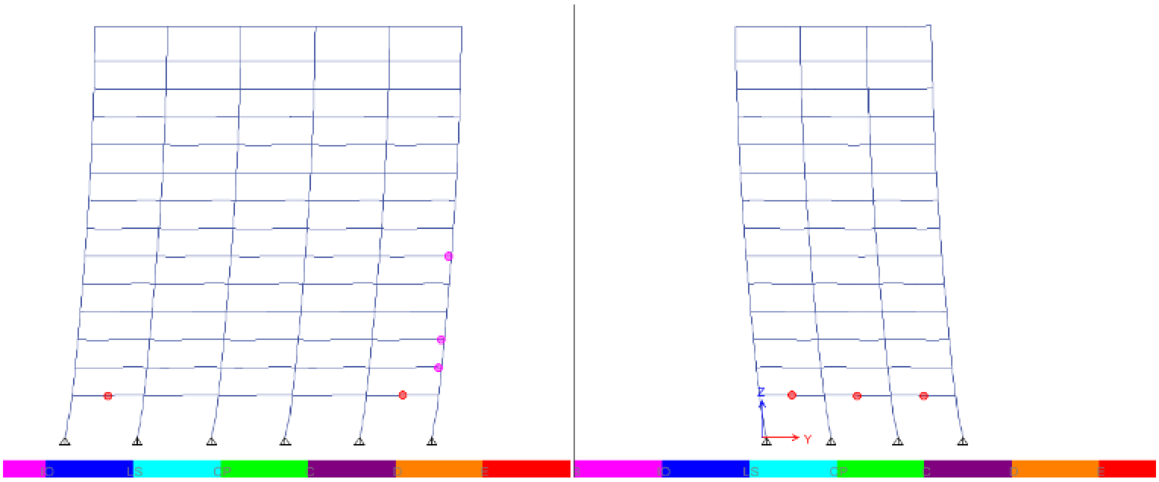


Fig. 4 –Building deformed shape

Examination of Table. 1 and Fig. 4 indicates the following:

- The building in its existing condition (without upgrade) will experience damage when subjected to the design earthquake
- The major damage will be primarily limited to the first floor. Since damage is concentrated at one level only, this can lead to instability and collapse.
- If the first-floor displacement is reduced below approximately 74 mm. (1.4% drift), then damage is essentially eliminated.

4 Seismic upgrade with fluid viscous dampers

Fluid viscous dampers (see Fig. 5 [4]) were used as the upgrade solution. Dampers possess the following characteristics: i) Are maintenance free and have been widely used in upgrade of reinforced concrete buildings with proven reliability; ii) Minimize the need for strengthening of existing members and foundations; iii) Can be aesthetically integrated into the building architectural features.; iv) Minimize disruption to building occupancy; and v) Are cost-effective. For this project, dampers were selected with the following properties:

- Velocity exponent = 0.5
- Damper nominal design force = 1560 kN

As required by ASCE 41-13 [2] all dampers were subjected to production testing conducted by the manufacturer. Fig. 6 shows the force-velocity relation for the 350-kip dampers.

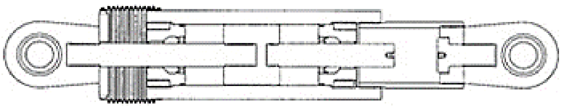


Fig. 5 Damper cross Section

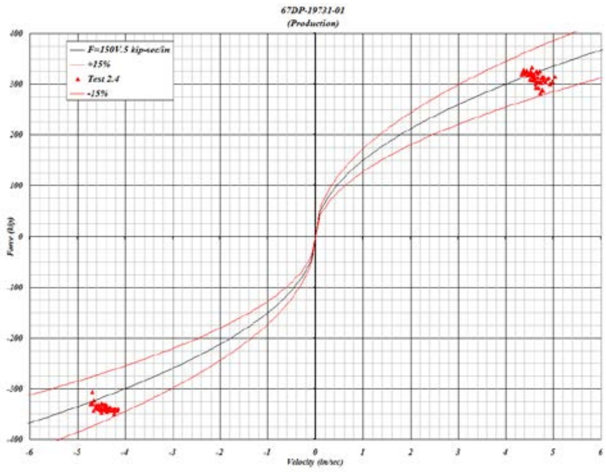


Fig. 6 –Force-velocity relation

5 Seismic upgrade evaluation

5.1 Overview

The analytical model of the building was revised by adding 8 dampers per floor for the bottom story; see Fig. 7 and Fig. 8. Analysis was conducted to assess the efficacy of the proposed upgrade. Three pairs of recorded accelerations from past California earthquakes were synthesized to correspond closely to the types of motions that can be anticipated at the building site during a design-level earthquake. Maximum values were then selected for assessment of the upgraded building model.

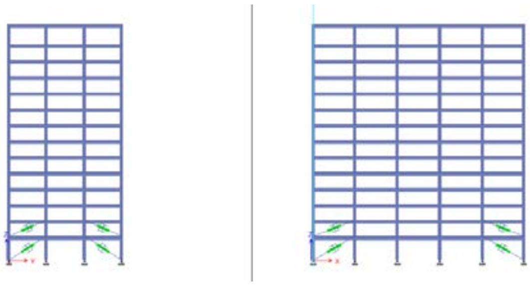


Fig. 7 –Revised building model with dampers

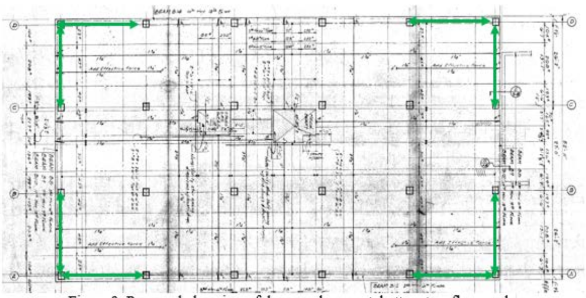


Fig. 8 –Damper placement, bottom story

5.2 Findings

Fig. 9 presents the computed story displacements for the upgraded model. The displacement values of Table. 1 corresponding to the approximate thresholds for linear response (no or minor damage) at the first floor and roof levels are also shown as arrows in the figure. It is noted that displacements of the upgraded model are within the acceptable limits. Fig. 10 presents the computed drift ratios for the existing and upgraded building. The efficacy of the proposed upgrade can be evaluated by noting the following:

- The soft-story response at the first floor is significantly reduced. The drift ratio at the first floor was on the order of twice that of the typical floors above, and this amplification is now reduced by approximately 60%.
- Drift ratio at first floor is approximately 1.2%. As such, no or only minor yielding of concrete members is expected.

The effectiveness of the damper upgrade solution can further be seen in Fig. 11, where the significant reduction in first floor displacement and nonlinear structural damage can be seen. As shown in Fig. 12, viscous dampers dissipate a significant amount of the earthquake input energy. In the absence of dampers, such energy must be absorbed by the existing reinforced concrete members through nonlinear action and structural damage.

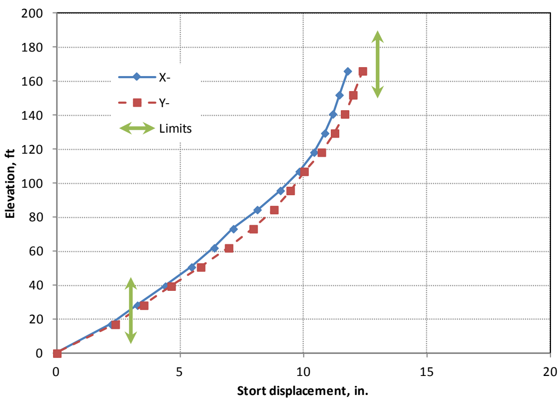


Fig. 9 –Computed story displacements

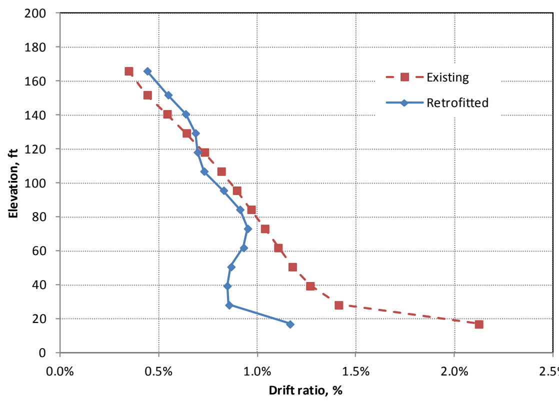


Fig. 10 –Computed drift ratios (Y direction)

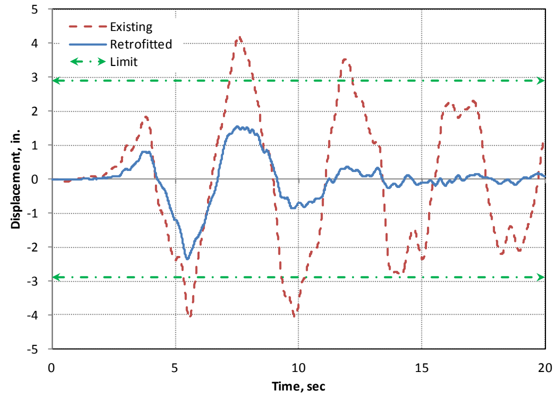


Fig. 11 –Displacement of existing and upgraded models, First floor, Y-direction

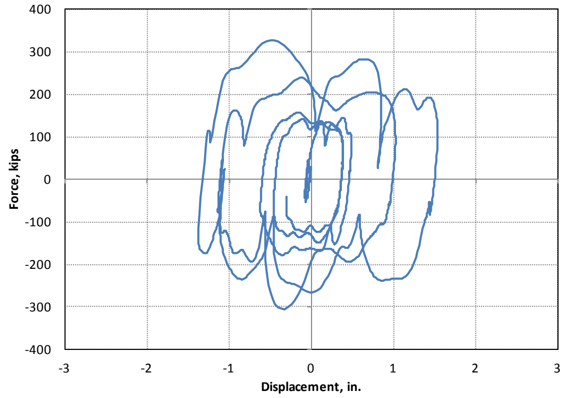


Fig. 12 –First floor damper force-displacement response

6 Risk (PML) analysis

Probabilistic risk analysis was conducted to compute the probable maximum loss (90% confidence PML) [5] and the scenario expected loss (50% confidence SEL) of the structure before and after upgrading. A similar analysis was conducted previously by URS Corporation. The results are presented in Table. 2 for both studies. The following is noted:

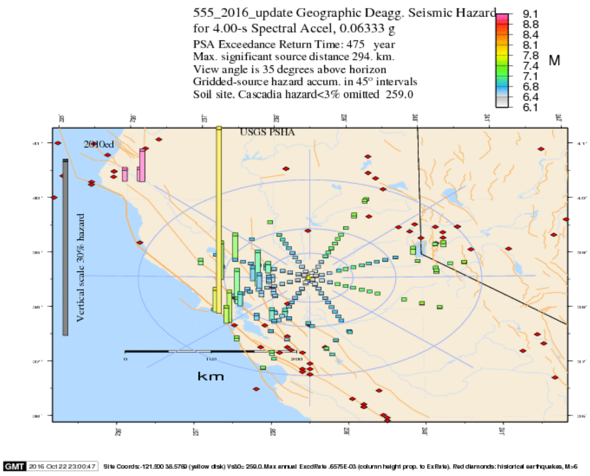
Table. 2 Scenario-based risk analysis (design earthquake)

	PML	**SEL**
Existing	31	14
Upgraded	19	9

- The reduction in PML and SEL for the upgraded building is more pronounced due to the differences in upgrade approaches proposed. The URS proposed upgrade consisted of wrapping the mid-sections of all frame beams and columns. While this approach is effective in providing confinement and increasing shear capacity of the concrete members, it does not reduce the seismic demand or soft-story behavior. With the proposed upgrade using dampers, the soft-story behavior is mitigated and demand on the existing members is reduced to near-elastic levels. Thus concerns regarding the member ductility are no longer applicable.

The FEMA P-58 [6] and [7] methodology is a probabilistic approach that combines the site-specific hazard, building properties, and exposure to estimate key response parameters, including the 90th-percentile repair cost in the event of the design (475-year return period) earthquake. The simulation for this project included 10,000 Monte Carlo analyses.

The site hazard was based on USGS data. The de-aggregation data from contributing faults is presented in Fig. 13 and the hazard curve for the site is presented in Fig. 14.



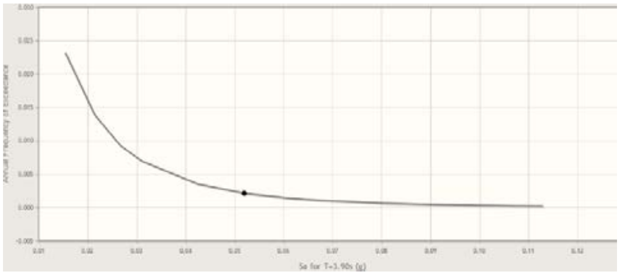


Fig. 13 –De-aggregation for the seismic hazard

Fig. 14 –Site hazard curve

The building was defined as a 14-story reinforced concrete moment frame structure, with lateral frames located at the building perimeter. It is of 1970s vintage, Risk Category II and has office occupancy. The input data for building properties were based on the results from the structural analysis of the building retrofitted with viscous dampers. The key input parameters include story drift ratio, peak floor acceleration, and residual drift; see Fig. 15 through Fig. 16. Building capacity was based on the FEMA 154 [8] checklist. The default values for the collapse fragility were used. The building contents (structural and nonstructural) were based on typical contents for this class of buildings. The P58 default fragility functions were used. The median and 90% repair costs for the design earthquake were computed as approximately 6% and 12%, respectively; see Table 3.

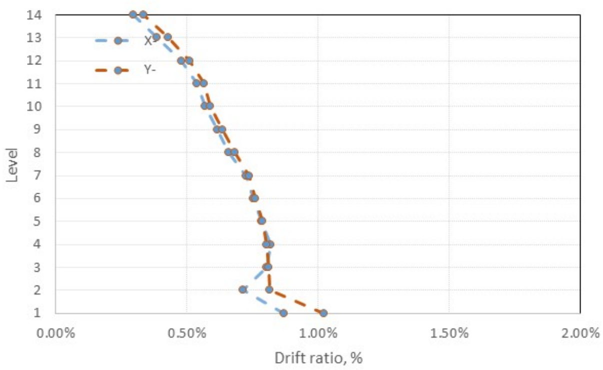


Fig. 15 –Story drift ratios

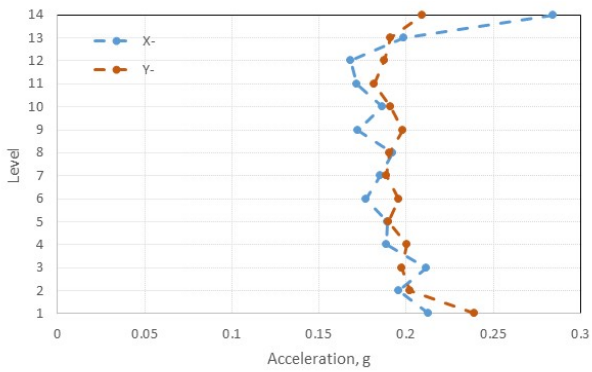


Fig. 16 –Peak floor accelerations

Table. 3 Median and 90% repair cost

Sa (3.6s) (g)	Mean repair cost	Median repair cost	90th percentile repair cost
8%	12%	6%	12%

7 Construction

The construction for the seismic retrofitting of the towers has been completed. Fig. 17and Fig. 18 present a portion of construction plans showing the damper elevations and connection to the existing concrete members, respectively. Fig. 19 presents photographs of retrofitted building

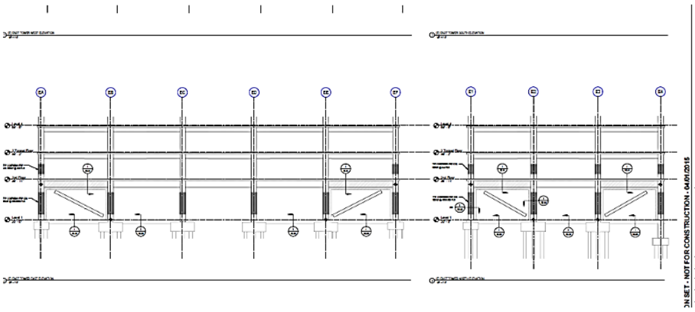


Fig. 17 –Elevation view of damper placement

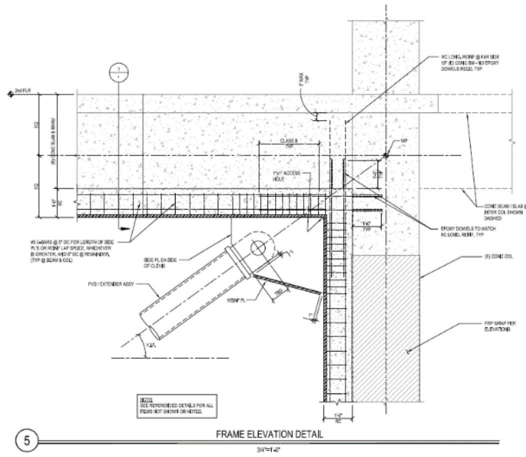


Fig. 18 –Connection details for dampers

Fig. 19 –Seismic retrofitting using viscous dampers

8 Conclusions

The preliminary probable response of the office towers in Sacramento to earthquake loading was investigated using a combination of advanced structural analysis and risk assessment. Analysis and evaluation showed the following:

- The buildings in their existing configurations had PML values that exceed 20 and will likely experience moderate to significant damage in the event of a design-level earthquake.

- The building structure had a moderate degree of nonductile detailing as well as soft-story behavior at the ground floor which pose significant hazards during earthquakes.
- The buildings were effectively upgraded with fluid viscous dampers.
- The upgraded solution mitigated the critical building deficiencies and reduce the PML.

9 Acknowledgements

Mr. Lon Determan of Miyamoto International served as the project manager for this project and his significant contributions are acknowledged.

10 References

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