Seismic Retrofit of a Pre-Northridge Steel Moment Frame Building Using ASCE 41-17

David Gonzalez, S.E.
Simpson Gumpertz & Heger

Kevin O'Connell, S.E.
Simpson Gumpertz & Heger

Guzhao Li, Ph.D., S.E.
Simpson Gumpertz & Heger

Emily McCarthy, Ph.D., P.E.
Simpson Gumpertz & Heger

Russell A. McLellan, P.E.
Simpson Gumpertz & Heger

Abstract

This case study outlines the retrofit approach and challenges to bring an existing seventeen-story, 335,000 sq. ft., steel-framed building with typical Pre-Northridge steel moment-resisting connections constructed circa 1970 into general compliance with ASCE 41-17. The building is located in Los Angeles County.

The voluntary seismic upgrade objectives are to meet the ASCE 41-17 Basic Performance Objective for Existing Buildings (BPOE) and reduce the Probable Maximum Loss (PML) to less than 20%, based on the Scenario Upper Loss (SUL) for an earthquake hazard with a 475-year return period.

The building consists of a fifteen-story tower above-grade with steel moment frames on all four sides of the perimeter. The moment connections are Pre-Northridge Welded Unreinforced Flange – Welded Web (WUF-W). Due to an architectural setback of the perimeter steel beams, the second-floor level does not have moment connections. This, combined with a tall first-story, presents a soft-story issue that was mitigated in the original design with extremely large columns from the first to third floor and large beams with an additional WT bottom flange haunch at the third floor. There are two parking levels partially below-grade constructed with a gravity steel frame and precast concrete shear walls around the perimeter. The building relies on the concrete over metal deck diaphragm at the Plaza level to transfer the lateral demands from the superstructure to the perimeter precast concrete shear walls.

We performed nonlinear static and nonlinear time history analyses following ASCE 41-17. The preferred seismic retrofit approach involves installing piston-type fluid viscous dampers (FVDs) to protect the existing moment connections and reduce demands on existing columns, column splices, and column bases. Limited foundation retrofit of parking garage precast shear walls was required.

This paper presents the methodology followed to evaluate vulnerable WUF-W connections and panel zones using the latest provisions of ASCE 41-17. Database scripting techniques were utilized to extract information from the vast output files which significantly reduced the post-processing time. The project was peer-reviewed by a team of structural and geotechnical engineers. Construction is now complete.

Introduction

Building Description

The building is a seventeen-story steel frame superstructure with two levels of partially underground parking constructed circa 1970. The building's lateral force resisting system (LFRS) comprises perimeter steel moment-resisting frames utilizing "pre-Northridge" Welded Unreinforced Flange - Welded Web (WUF-W) beam-column moment connections.

Photo 1. Exterior View of Building During Construction

The building is rectangular in plan, with the tower having overall dimensions of approximately 106 ft in the north-south direction by 211 ft in the east-west direction (Figure 1). Two partially subterranean levels have overall dimensions of approximately 268 ft in the north-south direction by 211 ft in the east-west direction.

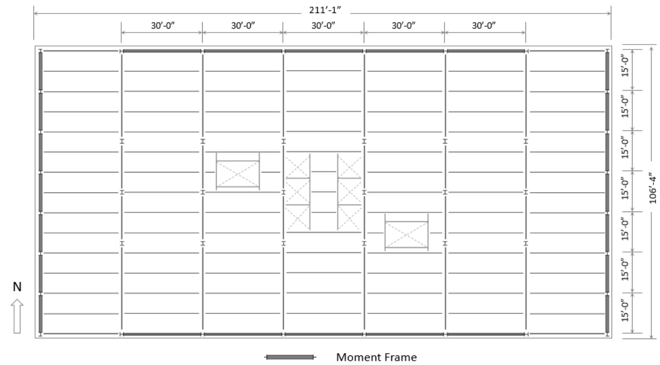

Figure 1. Typical Floor (3rd to 16th) Framing Plan (simplified for clarity)

The elevated floors consist of a 3-1/4 in. lightweight concrete fill slab reinforced with welded wire mesh over steel deck spanning between steel wide flange beams. The lowest basement level floor is a concrete slab-on-grade. Steel-wide flange beams and columns vertically support the floors and roof. Foundations consist of isolated spread footings beneath interior columns and strip footings beneath perimeter retaining walls.

Lateral loads are distributed to the perimeter steel moment frames by the metal deck and concrete-filled slab acting as a diaphragm. The east and west moment frame lines have seven bays with a typical bay width of 15 ft. The north and south moment frame lines have five bays with a typical bay width of 30 ft. All columns in the moment frames are oriented on the strong axis. The first story height is 20 ft-6 in. The height of the above stories is 13 ft-1 in. Typical moment connections consist of complete joint penetration (CJP) welds between the beam flanges and the column flanges and welded shear tabs. The column splices consist of partial joint penetration (PJP) welds and erection plates.

Objective and Approach

The structural performance objective of this voluntary seismic retrofit design is to meet the structural requirements of the ASCE 41-17 Basic Performance Objective for Existing Buildings (BPOE). The evaluation of nonstructural components to meet BPOE is not within the scope of this voluntary retrofit. The subject building is classified as Risk Category II. In meeting BPOE, we found the building's Probable Maximum Loss (PML) to be less than 20%, based on the Scenario Upper Loss (SUL) for an earthquake hazard with a 475-year return period.

We performed both linear dynamic analysis and nonlinear static (pushover) analysis for the existing building to identify critical structural deficiencies (such as columns splices, panel zones, and substructure shear walls) and confirm that seismic retrofit is required to meet the selected BPOE. Then, following the ASCE 41-17 Nonlinear Dynamic Procedure (NDP), the retrofit components are designed, modeled, and refined to reach compliance with the selected BPOE. The seismic retrofit intends to reduce potential damage by improving building performance.

Seismic Ground Motions

Site-specific response spectra with 5% effective damping for the use with the linear dynamic procedure (LDP) were developed by the project geotechnical consultant for both BSE-1E (Basic Safety Earthquake 1) and BSE-2E (Basic Safety Earthquake 2) conditions, as shown in Figure 2.

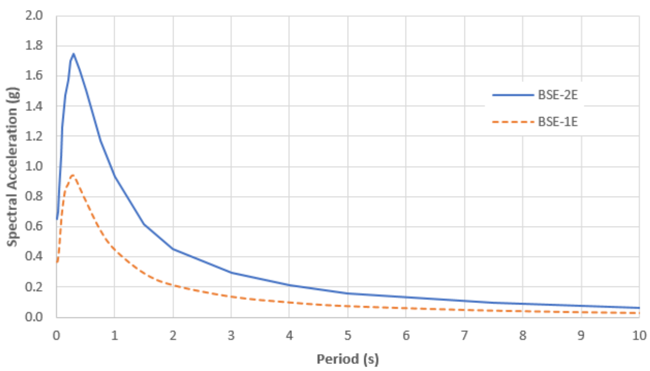

Figure 2. Site-Specific Response Spectra (5% damping) for the Use with LDP

The project geotechnical consultant also provided us with eleven pairs of acceleration-time histories for the BSE-2E and BSE-1E, respectively, for the use with NDP per the requirements of ASCE 41-17. Each set of time histories consists of two orthogonal horizontal ground motion components (rotated to fault normal and fault parallel for near-fault events and modified to the building axes, i.e., N-S and E-W directions). The geotechnical consultant selected the acceleration-time histories using the following criteria (among others) based on the results of disaggregation of hazard of various periods:

- Earthquake magnitude range: selected records from 6.0 to 7.5 (local), 7.0 to 8.0 (distant)
- Joyner- Boore (R_{jb}) distance: selected records both from 0 to 25 kilometers (local), and 25 to 200 kilometers (distant)
- Fault mechanism: selected records from strike-slip, reverse, and reverse-oblique
- Initial scale factor for seed time history: 0.2 to 4.0

Structural Deficiencies and Seismic Retrofit

Deficiencies identified using LDP

To preliminarily identify deficiencies in the building's LFRS, we performed a linear dynamic analysis of the moment frames following the ASCE 41-17 LDP requirements. We created CSI ETABS model, as shown in Figure 3, for the linear dynamic analysis. We explicitly modeled the moment frames and used rigid diaphragms with mass lumped at the center to model each floor. We also included the substructure and lower-level shear walls in the model.

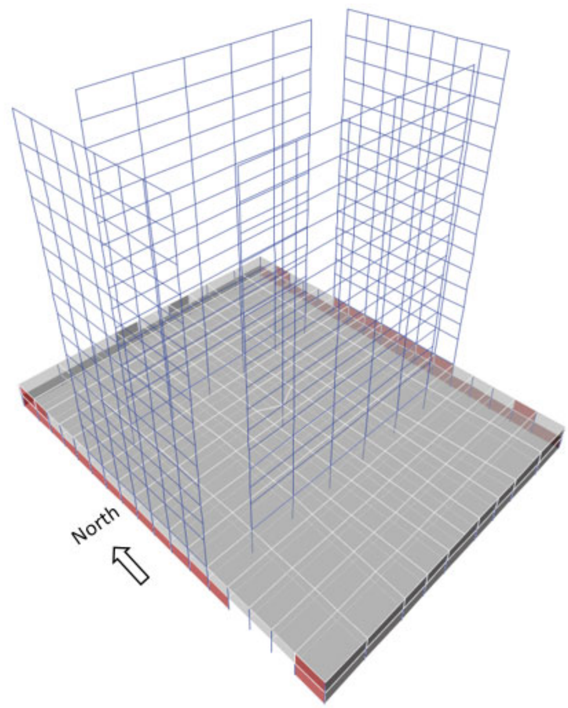

Figure 3. Overview of the ETABS Model (Existing Building)

The analysis results indicated potential deficiencies in the moment frames' panel zones and column splices. The substructure analysis (podium diaphragm, collector elements, shear wall panels) indicated an insufficient capacity for some shear wall panels.

Deficiencies confirmed using NSP.

To confirm the deficiencies (panel zones, column splices) in the moment frames identified using LDP, we performed a nonlinear static (pushover) analysis for the existing building following the ASCE 41-17 nonlinear static procedure (NSP) requirements.

We created a three-dimensional model using CSI PERFORM-3D V7.0.1. The PERFORM-3D model, as shown in Figure 4, consisted of the building's superstructure and substructure, capturing its geometry, mass distribution, and elastic-plastic force-deformation behavior.

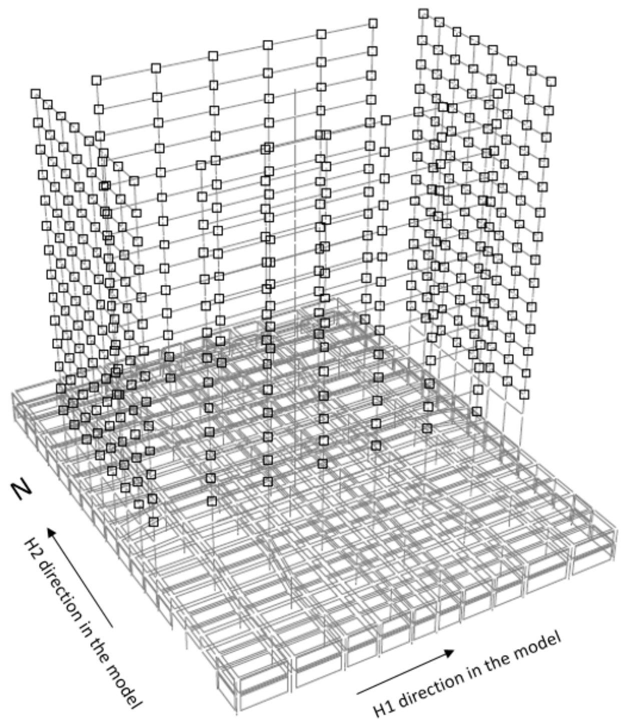

Figure 4. Overview of the PERFORM-3D Model (Existing Building)

We modeled all diaphragms above the first-floor level as rigid. We modeled the two floors in the substructure as semi-rigid to better represent the force transfer from the superstructure to the substructure. We modeled the concrete panel shear walls below the ground floor using elastic planar shell elements. We modeled all the other primary and secondary elements below the ground floor using elastic sections with end moment releases as applicable.

We modeled the panel zones using connection panel zone elements with a rotational spring representing the panel zone shear mechanism. A typical frame compound element may include an auto rigid end zone (at both ends), a WUF-W moment hinge (at both ends), or a P-M column hinge (at both ends), and an elastic beam or column section between the beam or column hinges. We also included strength sections in the compound element to confirm elastic behavior at locations where plastic hinges are not expected, such as column splice locations.

For deformation-controlled components, such as WUF-W moment hinge, P-M column hinge, and panel zone elements, we defined their parameters and acceptance criteria at different performance levels following the ASCE 41-17 requirements.

For pre-Northridge WUF-W moment connections with panel shear to shear strength ratio (V_{pz}/V_y) greater than 1.10, ASCE 41-17 introduced an upper bound limit for the panel zone Life Safety (LS) and Collapse Prevention (CP) performance level acceptance criteria, as shown in ASCE 41-17 Equation 9-19. The upper bound limit is 50% of the panel zone LS and CP acceptance criteria as listed in Table 9-7-2 of ASCE 41-17. There was no such limit for the previous versions of ASCE 41. It is worth noting that panel zone performance with pre-Northridge WUF-W moment connections meeting the ASCE 41-13 acceptance criteria may not satisfy the ASCE 41-17 requirements due to this upper bound limit. The resulting CP plastic rotation for the panel zones in our project was between 0.5% and 1.5%, which is generally between 3θy and 6θy, where θy is the yield rotation of the panel zone shear mechanism. In ASCE 41-13 the CP limit is 12θy, which would have allowed two to four times the plastic rotation relative to ASCE 41-17. A further consequence of the reduction in the plastic rotation is a reduction in strength. Strain hardening is significant for panel zones. The default panel zone strain hardening slope is 6% in ASCE 41, with higher slopes acceptable based on test data. When the panel zone is defined with significant ductility and strain hardening, the ultimate strength of the panel zone often becomes large enough to allow for distributed yielding in the adjacent beams and columns. In our case, almost no beams yielded, mainly because the panel zones could not develop the yield strength of the beams. Panel zone shear is an established ductile mechanism. The additional limitations on the panel zone rotation introduced in ASCE 41-17 are meant to capture the limit state of the excessive panel zone deformation causing column "kinking," which fractures the beam-column flange welds. So, it is a moment connection failure tracked by the panel zone, because once that weld is fractured, the panel zone is no longer engaged. Unintuitively to the authors, there is no strength loss or residual strength defined for this limit state.

For force-controlled components, we defined their strength limits following applicable standards or references. We followed the acceptance criteria for strength checks per the ASCE 41-17 requirements.

The column splices consist of PJP welds and erection plates. Typical pre-Northridge PJP column splices with insufficient toughness may experience brittle fracture before reaching their nominal strength, and therefore, need to be evaluated based on strength limits. We calculated the critical column flange tensile stresses at splice locations following the guidelines as described in Appendix A of NIST GCR 17-917-46v2 (2017). The column splice P-M-M strength component parameters are defined based on the calculated critical flange tensile stresses.

We evaluated the existing building for BSE-2E hazard level with CP objective. We considered a loading pattern proportional to the fundamental modal shape in each horizontal direction for the pushover analysis lateral load following the ASCE 41-17 requirements. We calculated the target roof displacement in each direction per the ASCE 41-17 requirements using 5% damping response spectra for the BSE-2E hazard level. Figure 5 shows the structural capacity curves in both directions with P-delta effects included from our preliminary pushover analysis.

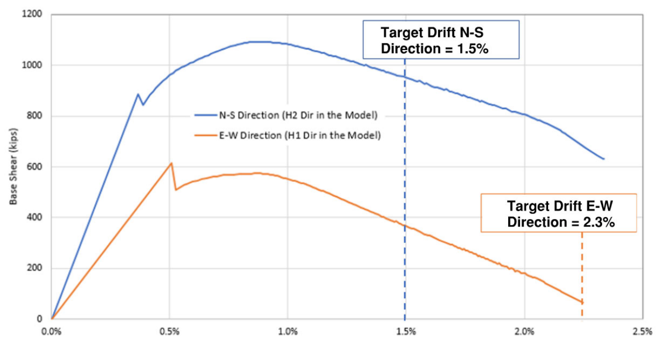

Figure 5. Structural Capacity Curves (in both directions) for the Existing Building

We calculated deformation and force demands at the target roof displacement for selected deformation-controlled and force-controlled components using the PERFORM-3D analysis model. We compared these demands with the CP performance level acceptance criteria for the selected components. We concluded that the structural performance of the existing building does not meet the CP objective under BSE-2E hazard conditions primarily due to unsatisfactory performance of the panel zone components (deformation-controlled) and column splice components (force-controlled), indicating seismic retrofit is required for the building to meet the selected BPOE.

Selected seismic retrofit.

The preferred seismic retrofit approach involves installing piston-type fluid viscous dampers (FVDs) to protect the existing moment connections and reduce demands on existing columns, column splices, and column bases. Limited foundation strengthening of parking garage precast shear walls was also required.

Our NDP analysis results using the upper bound damper properties indicated high axial stress in the columns on the north and south moment frame lines requiring strengthening of some columns. We designed ¼" to ½" thick steel faceplates extending between flanges on the inside face of these columns to address this issue.

We considered an approach that included enhancing existing beam-column moment connection by replacing the CJP welds or adding a double plate to the panel zone to achieve better ductility particularly for the limit state where column kinking may cause premature fracture of the beam to column welds. We quickly found enhancing the moment connections to not be financially feasible for this project.

FVD design and Nonlinear Time History Analysis

FVD design

Our seismic retrofit uses FVDs provided by Taylor Devices which behave according to the follow equation:

$$F = C_N V^{\propto} \qquad \text{(Equation 1)}$$

where, F = damper force, C_N = nonlinear damping coefficient, V = velocity, α = damping constant of 0.3.

Our preliminary FVD design objective was to select a damping coefficient C_N to be used in the initial response history analysis, based on the damper configuration shown in Figures 6 and 7.

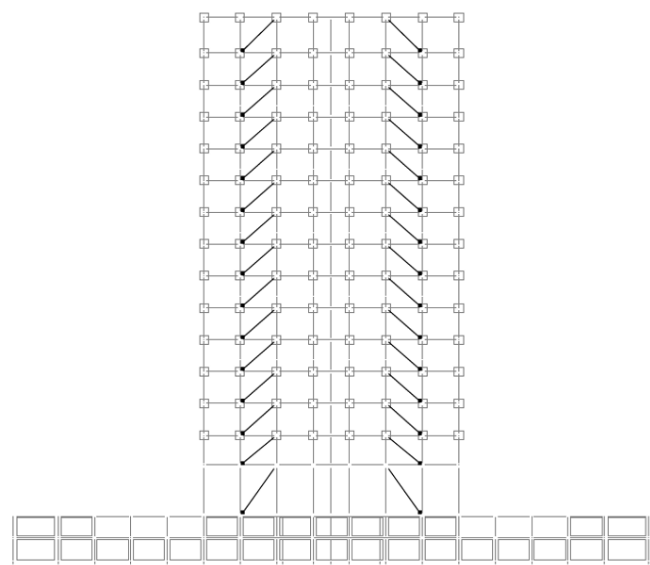

Figure 6: FVD Configuration (East and West Elevation)

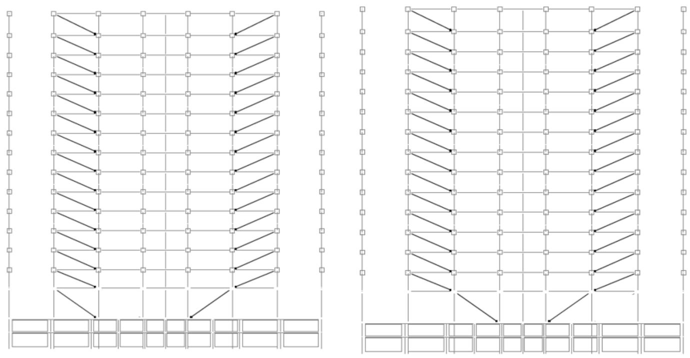

Figure 7: FVD Configuration (North Elevation on the left, South Elevation on the right)

We made the following assumptions in our preliminary FVD design:

1. The fundamental modal shape from the linear dynamic analysis is reasonable for the initial sizing of dampers used in the nonlinear response history analysis
2. The viscous damping ratio of the structure equals the viscous damping ratio contributed from the fundamental mode
3. The 1st mode viscous damping is equal to the target effective damping of the structure (β_{eff}) minus 2% of damping in the structural frame

The estimated target roof displacements for the existing building with 5% effective damping at the BSE-2E hazard level are:

- E-W direction: $\delta \approx 55$ inches ($\Delta \approx 2.25\%$) with an initial fundamental period of 5.32 seconds
- N-S direction: $\delta \approx 37$ inches ($\Delta \approx 1.51\%$) with an initial fundamental period of 3.66 seconds

To resolve the panel zone performance issues entirely, we estimated the target drift in the E-W direction needs to be reduced to 0.9% (equivalent to 22.2 inches of roof displacement), and the target drift in the N-S direction needs to be reduced to 0.75% (equivalent to 18.5 inches of roof displacement). Theoretically, these target drifts can be achieved by continuously increasing the effective damping ratios in the system. In reality, it would not be practical to achieve this goal since any effective damping ratio greater than 30% would not be effective to further reduce the story drift for this particular structure. As a result, we limit the effective damping ratio in the system to 25% (including 2% modal damping) in the preliminary design.

We followed the procedures described in MCEER 00-0010 for the preliminary damper design. With 25% effective damping (23% viscous damping), and assuming the nonlinear damper coefficient, C_N, is a constant for all the devices in each direction, we selected the following damping coefficient for the preliminary design based on our calculations:

- All the dampers in the E-W direction, C_N= 150 (kips-sec/inch)
- All the dampers in the N-S direction, C_N= 250 (kips-sec/inch)

Note that the actual viscous damping ratio from nonlinear response history analysis based on the preliminary damper design may not match the desired viscous damping ratio of 23% as described above. This is anticipated since the preliminary design was based on the assumptions described in this section (based on fundamental modal shape from linear analysis).

Nonlinear time history analysis

We used our NSP PERFORM-3D model for our nonlinear time history analysis, with the difference being that we included PERFORM-3D viscous bar elements as a representation of the FVD and applied the geotechnical consultant's site-specific ground motions in accordance with ASCE 41-17. The viscous bar elements resist only axial load and comprise a viscous damper element and an elastic bar element in series.

ASCE 41-17 Chapter 15 Section 15.3.2 states that the nominal design properties for dampers used as energy dissipation devices shall be factored by 1.2 for an upper bound analysis and 0.85 for a lower bound analysis to account for device property variations. Therefore, we performed three sets of analyses (upper bound, lower bound, and nominal), each comprised of eleven pairs of acceleration-time histories for the BSE-2E condition.

Our final damper design resulted from an iterative process of checking the overall system response and component demands for compliance with the BPOE objective. We checked beam moment hinges (deformation check), column P-M hinges (deformation check), panel zones (deformation check), column compression (strength check), column splices (strength check), and bean moment demand (strength check) for the collapse prevention limit state considering the BSE-2E time histories. We finalized our FVDs into two damper sizes split across floors. For all dampers our damping constant is $\alpha = 0.3$. Figure 8 shows the dissipated energy in the system for one sample acceleration-time history with maximum total energy of 189,800 kip-in. The energy in fluid viscous dampers accounts for about 70% of the maximum total energy for this particular time history. Final damper configurations are shown in Figures 9, 10, and 11.

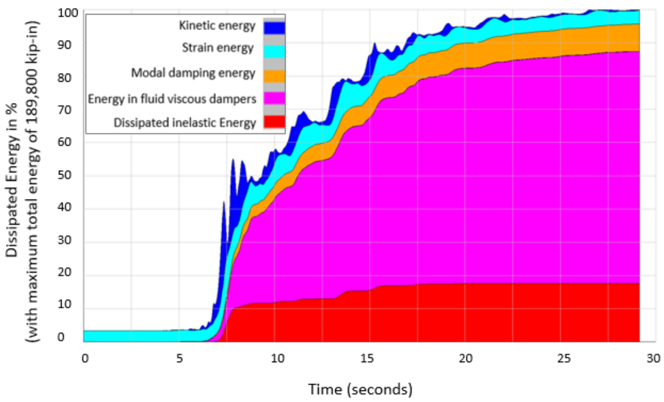

Figure 8. Dissipated Energy in the System for One Sample Time History

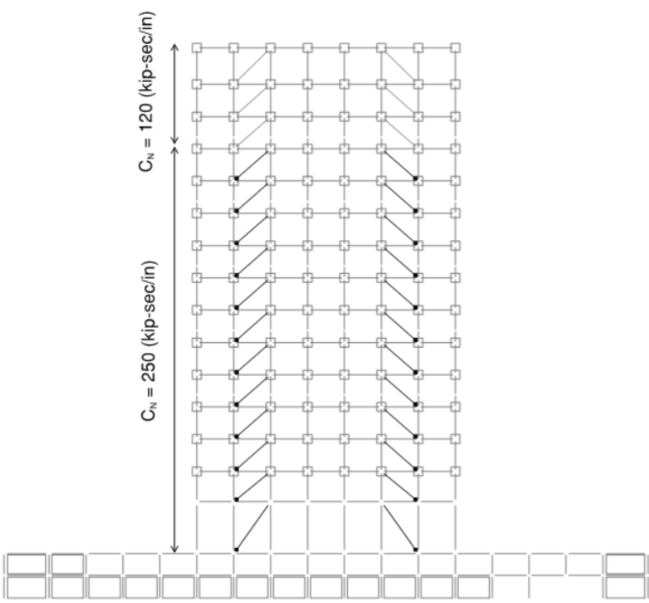

Figure 9. Damper Design (East and West Elevations)

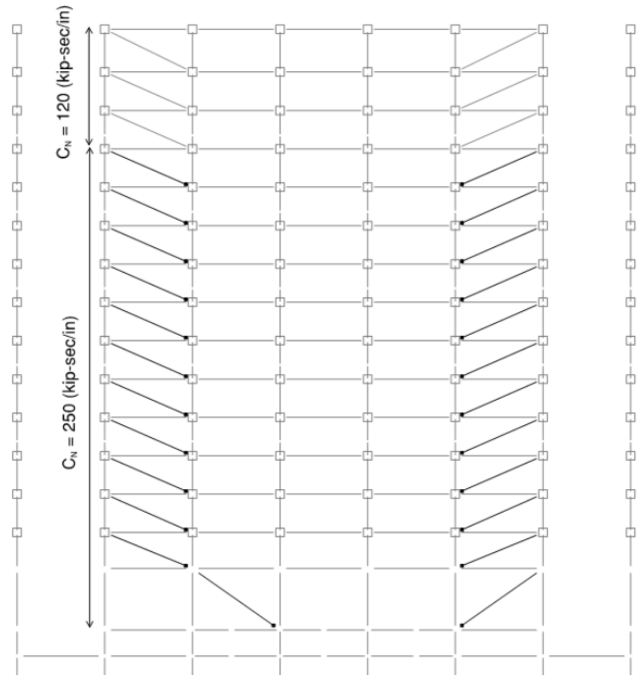

Figure 10. Damper Design (South Elevation)

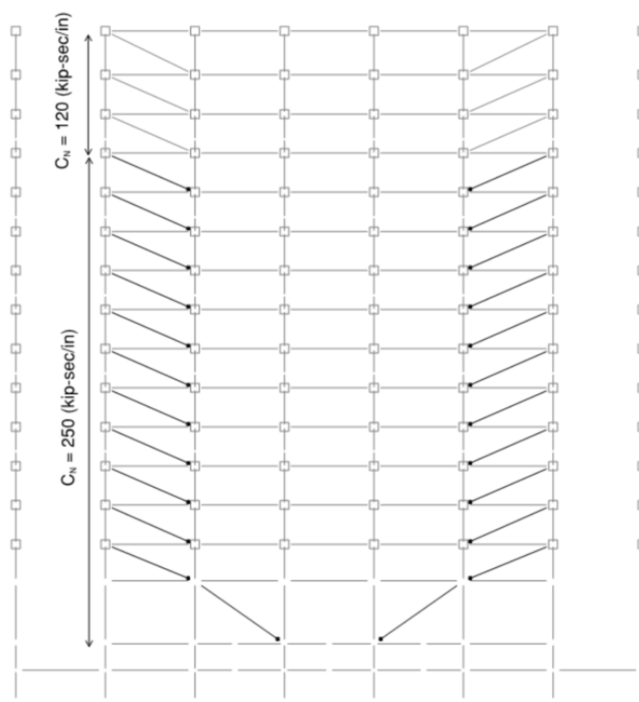

Figure 11. Damper Design (North Elevation)

Analysis results

The nonlinear time history analysis shows improved structural system response and better compliance with the BPOE requirements overall. Even so, a very small number of panel zone components still do not meet the CP performance level acceptance criteria for the BSE-2E condition. In addition, the added dampers apply high demands to some columns, necessitating some further retrofitting of a subset of those elements.

Across all analyses and all-time histories, all the following components meet the CP performance level acceptance criteria for the BSE-2E condition:

- Beam moment hinges (deformation check)
- Column P-M hinges (deformation check)
- Panel Zones (deformation check)
- Column splices (strength check)

The panel zone performance is controlled by the lower bound damper property model that has larger story drifts. Controlling locations are primarily at lower levels in the interior bays of the moment frame lines. The addition of dampers reduced drift but increased the axial stress ratio in the columns. In ASCE 41-17 Eq. 9-19, the ductility of the panel zone is a function of the axial stress ratio. However, using the maximum axial stress ratio to determine the panel zone acceptance criteria is too conservative because the maximum panel zone deformation demands and maximum axial stress ratios are out of phase. Because the dampers have nonlinear damping properties, they are not perfectly out of phase with the story drift. We determined the acceptance criteria for critical panel zones at every step of every time history to determine the DCR at every step of every time history, which was very computationally time-consuming but produced the most accurate execution of the panel zone evaluation. Although some individual time history analyses show high demand-to-capacity ratios (DCRs), when looking at the average demand across all time histories, we noted eight (8) panel zones in the North and South elevations that did not quite meet the CP acceptance criteria. Despite this, we concluded, and the peer reviewer agreed, that the expected global building performance meets the intent of BPOE.

The upper bound damper properties control the column compression performance. Controlling locations are the outermost columns of the North and South moment frames, between the 3rd and 10th floors. We designed faceplates (0.5-in to 0.25-in thick) extending between flanges on the inside face of the column only to add compression capacity, which reduced the maximum DCR from 1.12 to 0.87.

In addition to accounting for the upper bound and lower bound damper properties in the nonlinear time history analyses, ASCE 41-17 section 15.3.2 states that the damper component is required to sustain a force and displacement associated with 130% of the maximum calculated velocity for devices analyzed in the BSE-2E time history analyses. Therefore, we determined the maximum damper axial forces by averaging the demand in each damper at each time history across either the upper bound, lower bound or nominal bound analyses. Using Equation 1, we extracted the corresponding velocity at the maximum demand, amplified it by 1.3, and recalculated the damper forces (maximum stroke remains the same). The final dampers supplied by Taylor Devices are sized to accommodate these amplified forces.

For the analysis model, including dampers with nominal properties, we also performed the nonlinear time history analysis using the eleven pairs of acceleration-time histories for the BSE-1E condition. The results indicate that all the components meet the LS performance level acceptance criteria.

We chose to ignore the stiffness contribution of the gravity frames as well as the contribution of the slab to the moment connection capacity. Given that our flexural hinges in the beams representing the moment connection weren't yielding, there was no need to consider the contribution of the slab. Had the building in its pre-retrofitted condition been sufficiently close to meeting the performance objective, we would have considered the stiffness and strength contributions of the gravity frames.

Advanced pre and post processing techniques were employed on this project similar to the techniques described in the 2019 SEAOC Convention Paper "Using Computer Automation to Simplify Seismic Evaluation of Existing Buildings' by Moore, et. al. We developed programming tools to read modelling parameter and acceptance criteria information from spreadsheets and rapidly create definitions in the nonlinear analysis model. Users familiar with PERFORM 3D know that its solver allows for better computing performance relative to other available nonlinear analysis software, however, tedious inputs can make executing a large model unwieldy, leading to both time and quality assurance challenges. We employed programming techniques on this project that effectively eliminated these data inputting challenges. For post processing, we developed tools to rapidly extract the data from the binary output files of PERFORM 3D with excel functions. NLTHA inherently have massive amounts of data as calculations are performed by the software in a model with hundreds of nonlinear elements, over thousands of time steps, for multiple time history runs. This analysis for this building would be expected to take many months to complete. Because of the programming tools employed on this project we were able to drastically reduce the analysis time in the project and meet the owner's aggressive timeline needs.

Photo 2. FVD in Longitudinal Direction of Building

Photo 3. FVD in Transverse Direction of Building

Improved Resiliency

Moment frame buildings are often susceptible to substantial damage due to the large peak inelastic story drift relative to other structures. The high inelastic story drifts trigger damage states with high cost and repair time consequences. Pre-Northridge Welded Steel Moment Frames often have significant collapse and excessive residual story drift potential, further increasing their damageability and downtime of the building on top of safety concerns. In meeting the requirements of the ASCE 41-17 BPOE, we found that we were drastically reducing the inelastic story drift and seismic base shear with our fluid viscous damper retrofit. Additionally, we expect the damper's ability to reduce floor accelerations to further mitigate damage, particularly to nonstructural components.

To understand how we are improving the resiliency of the building by reducing damageability and downtime, we performed studies using Haselton Baker Risk Group's Seismic Performance Prediction Platform (SP3). We used the Advanced SP3-RiskModel, which allows the user to input Engineering Demand Parameter (EDPs) directly from the NLTHA used to determine the damage state and associated consequences for damageable components in the building. The structural response from the NLTHA inputed into the SP3 model is shown in Figures 12, 13, 14, and 15. The three EDPs used in our damageability are: (1) Median Peak Floor Acceleration (PFA), (2) Peak Story Drift Ratio (SDR), and (3) Median Residual Drift (RD). Categories of damageable components include Structural, Exterior Finishes, Partition Walls, Other Nonstructural, Ceilings, Lighting, Elevators, Piping, and HVAC. We studied the improved resiliency of the building using the hazards included in our performance objective, events with a return period of 224 years (BSE-1E) and 975 years (BSE-2E), each with 11 earthquake simulations.

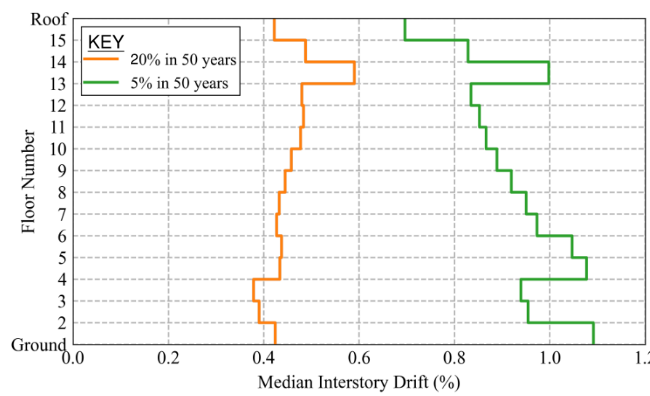

Figure 12. Retrofitted Building Median Story Drift-Longitudinal Direction (from SP3 report)

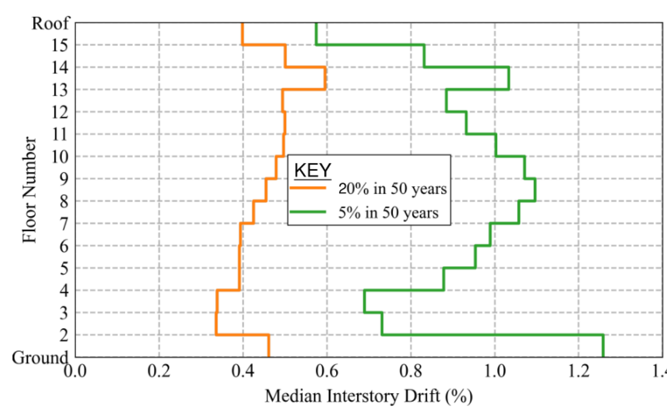

Figure 13. Retrofitted Building Median Story Drift-Transverse Direction (from SP3 report)

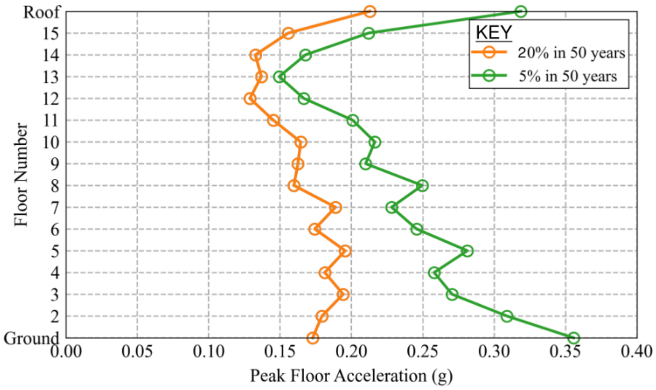

Figure 14. Retrofitted Building Median Peak Floor Acceleration- Longitudinal Direction (from SP3 report)

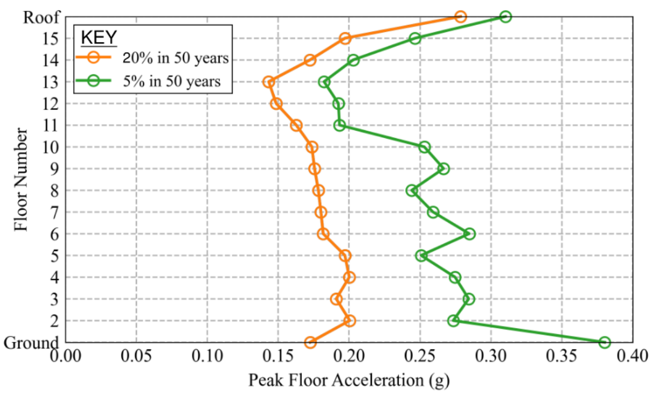

Figure 15. Retrofitted Building Median Peak Floor Acceleration- Transverse Direction (from SP3 report)

In its pre-retrofitted condition, the building had a significant collapse risk for the 975-year seismic event, making it difficult to compare to the performance of the building in its retrofitted condition as safety is the primary concern for the building. In its retrofitted condition, not only was the collapse risk mitigated, but the anticipated damageability and downtime in the 975-year event were found to be favorable. The Scenario Upper Loss (SUL) is less than 20%, and the building downtime is expected to be as low as six months.

For the more frequent event with a 224-year return period, there is a relatively low collapse risk in the pre-retrofitted building allowing for a comparison of performance to the retrofitted building not skewed by large additional costs from the risk of collapse or excessive story drift. As expected, the damageability is drastically reduced in the retrofitted condition, with the SUL approximately three times less than that of the pre-retrofitted condition. In the retrofitted condition, functional recovery could conceivably occur within three months while it would be closer to a year in its pre-retrofitted condition.

It should be understood that downtime is also a function of impeding factors such as inspection, financing, permitting, engineering mobilization, and contractor mobilization. However, the performance of the structure allows for expedited recovery. Also, our study is based on conservative assumptions on the damageable components of the building. Even better performance is possible by identifying and mitigating the key contributing components to damageability and downtime.

Construction Considerations

The project team learned many practical lessons throughout the project, and we offer a few suggestions here for the reader when using dampers for retrofit.

Scheduling and Lead Time

The size and length of the FVDs required on this project resulted in a substantial lead time for product delivery to the project site. Although this is often the responsibility of the general contractor to manage, we worked with the project team to provide FVD specification information (size and length) well in advance and a final design that allowed for material ordering and construction sequencing to minimize the schedule impact.

FVD Size and Pin-to-Pin Length

Before completion of the full structural design, including the connections and damping properties of the FVDs, we provided the contractor with the damper size (diameter) and approximate (over-estimated by several inches as a contingency) pin-to-pin length to allow the damper manufacturer to order and begin custom fabrication of the raw material with the intention that the contractor would determine the specific pin-to-pin length, based on actual field verified dimensions, during an early soft demolition phase. Unfortunately, unanticipated delays in the soft demo reduced the effectiveness of this plan. However, it was still effective in allowing a staged fabrication and delivery of the FVDs to the project site.

Connection Construction Sequencing

Although often an after-thought, careful consideration was given to the detailing of the FVD gusset plate connections to allow for schedule-benefitting construction sequencing.

One-sided complete penetration welds, completed from the interior of the building, were used for welding the gusset plates to the beams and columns to allow the exterior glazing to remain in place. Early sequencing plans by the contractor included the removal of the exterior glazing at each FVD connection. Detailing the connections to allow the glazing to remain in place saved substantial time in the construction schedule.

We designed the connections with a single gusset plate and matching filler plates and tang plates to connect to the FVD clevis. The connection allowed for the installation of the gusset plate, followed by the field welding of the filler plate and tang plate after delivery and the final positioning of the FVD. The intention was to account for small variations of the existing column spacing or story height without requiring an overly precise FVD pin-to-pin dimension. However, the contractor chose to shop-weld the filler plate and tang plate to one side of the FVD and then field measure the pin-to-pin length, which did not optimize the sequencing but did allow for precise manufacturing of the FVD lengths.

Damper Material Finish

The owner chose to proceed with a FVD with an integral damper extender provided by the damper manufacturer rather than a typical damper field bolted to a steel fabricator-provided damper extender (typically an HSS member). The process used to fabricate and finish the steel surface of the integral damper extenders on this project varied from the standard process for a typical damper due to the size and length specified. This resulted in a damper material finish that was different than can be achieved with smaller dampers. Although the structural performance of the FVD is unaffected, it is important to manage client expectations in this regard.

Stroke Indicators

We requested the manufacturer add stroke indicators (also called scratch plates) to each FVD. These stroke indicators will allow structural engineers, the building owner, and the damper manufacturer to determine the damper stoke that occurs during an earthquake leading to a better understanding of the damper performance, story drift, and overall building performance. The manufacturer installed stroke indicators on these dampers, and this small added cost should provide valuable data after future seismic events.

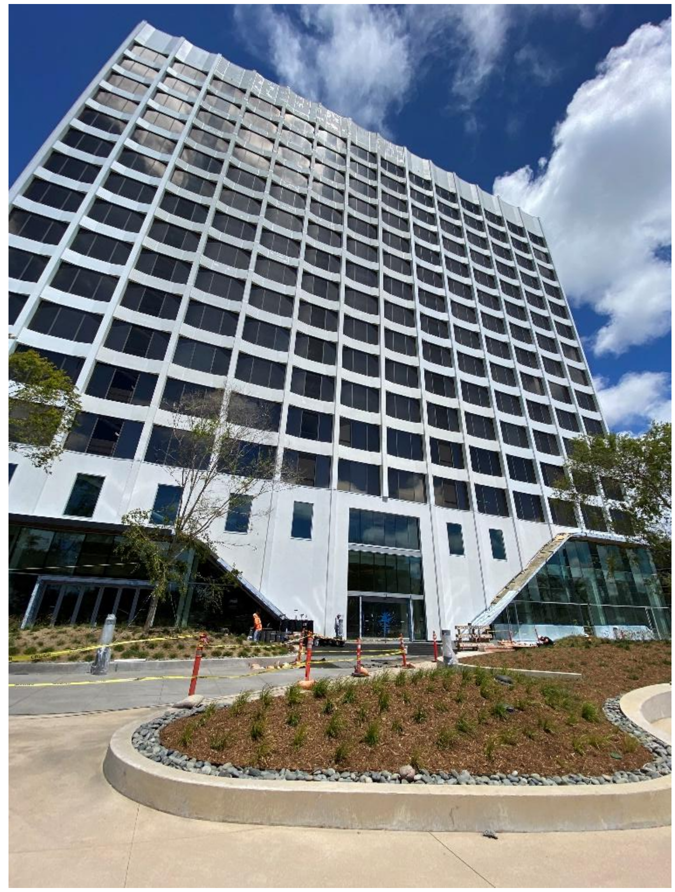

Photo 4. Exterior View of Completed Building

Closing Remarks

This project included the voluntary seismic retrofit of a seventeen-story steel moment-resisting frame building with FVDs following ASCE 41-17. Even though the changes to panel zone criteria in ASCE 41-17 resulted in substantially more supplemental damping than the previous ASCE 41-13 criteria, this case study shows that fluid viscous dampers are an effective way to protect flexible structures. The additional damping introduced into the structural system reduces drifts and protects elements susceptible to large displacements without requiring significant strengthening of foundations, column bases, or diaphragms. Beyond generally meeting BPOE criteria, the building's expected damageability and downtime are also significantly reduced. With a continued reduction in computing cost, it is feasible to effectively process large amounts of data using automated tasks generated by commercial or proprietary applications to assist in the successful analysis and design of seismic retrofit projects.

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