Improving structural performance of an existing EBF and moment frame structure using fluid viscous dampers

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ABSTRACT

Improving the performance of existing buildings is a critical function for engineers and one of the most impactful ways we can improve the sustainability of building structures. By re-using and upgrading existing building stock, we help to extend the usable life of structures that otherwise would pose safety risks to our communities.

This paper presents a case study of the voluntary seismic retrofit of 651 Gateway Boulevard in South San Francisco, California. The existing 17-story steel framed office building uses a core of eccentrically braced frames (EBFs) and perimeter moment frames as the lateral force resisting system. The building was designed and constructed per the 1982 Uniform Building Code, and therefore includes pre-Northridge deficiencies in the beam-column welds, non-ductile partial penetration welded column splices, and EBF link connections susceptible to fracture.

Nonlinear response-history analysis was used to evaluate the seismic performance of the structure before and after retrofitting, including randomized distribution of element properties to capture uncertainty in the built condition. The nonlinear model included the principal gravity girders, and nonlinear soil-foundation flexibility, considering the potential behaviour of all elements of the force path to the precast pile foundations.

Early retrofit solutions that involved stiffening the structure were explored, but the resulting foundation work and remaining beam-column connection work were costly, therefore fluid viscous dampers (FVDs) were explored. Ultimately, the retrofit solution with FVDs reduced story drift to reduce rotation demands on beam-to-column connections within code limits. The lower levels of column splices are still susceptible to fracture, and these were retrofitted.

1 INTRODUCTION

1.1 Existing Building Summary

651 Gateway Boulevard in South San Francisco, California is an existing 17-story steel-framed office building (Figure 1). The structure is trapezoidal in plan, with floor plates of approximately 1,670 m^2 (18,000 ft^2). The framing system consists of concrete fill over metal deck supported by steel framing, with a foundation system of pile caps and precast driven piles. The lateral force resisting system for the structure is a dual system consisting of an eccentrically braced frame (EBF) core and perimeter moment frames.

The building was designed in accordance with the 1982 Uniform Building Code (UBC) and constructed in the mid-1980s, and therefore contained pre-Northridge deficiencies in the connections of EBF and moment frame beams, as well as inadequate column splices.

Figure 1: Pre-retrofit building at 651 Gateway Boulevard in South San Francisco, California.

1.2 Pre-Northridge welded frame deficiencies

The Northridge, California earthquake of 1994 revealed systematic deficiencies in the seismic performance of steel moment frame buildings with welded connections. Prior to the earthquake, engineers in the United States regarded steel moment frame buildings as highly ductile systems, with expected damage limited to flexural yielding and plastic rotation of steel beams and columns. Reconnaissance following the earthquake found that these buildings experienced brittle fractures of beam-column connections at very low plastic demands, including in buildings that experienced only moderate levels of ground shaking. Fractures tended to originate in complete-joint-penetration (CJP) welds from the beam bottom flange to the column, and subsequent studies conducted as part of the SAC joint venture identified other features of the connection that contributed to the brittle behaviour including (FEMA 355D, 2000):

- Welds that were made used self-shielded flux-core arc welding (FCAW-SS) at high deposition rates. The resulting welds commonly have low notch toughness and inclusions or defects.
- Welds at the bottom flange were performed as a downhand field weld with the welder seated on the beam top flange. To perform the weld, passes are interrupted at the beam web, and often welders performed all of the welds on one side of the flange instead of alternating sides. The process frequently results in slag inclusion and other defects that can precipitate cracking.
- Backing bars were typically left in place after completion of the weld, which made detection of weld flaws difficult.
- The configuration of the joint results in the transfer of very high shear and moment demands on the welded joint.

Research has identified a similar risk of weld fracture in the link beams of eccentrically braced frames located adjacent to columns (Okazaki, 2004). Beam-column welds at the link beam are subject to high strain demands, and examples using Pre-Northridge welding standards have performed poorly in laboratory tests. Pre-Northridge column splices commonly used partial-joint-penetration welds at column flanges. Research has shown that when such welds experience net tension from earthquake shaking, the unfused portion of the flange behaves as a pre-existing crack subject to brittle fracture (Bruneau and Mahin, 1990). The fractures are exacerbated using Pre-Northridge weld consumables with low notch toughness.

Our preliminary review of the existing 651 Gateway building showed that the building included these three connection deficiencies typical of Pre-Northridge welded steel construction:

- Beam-column connections use typical weld-unreinforced flange-bolted web (WUF-B) connections without notch tough electrodes.
- Column splices use partial-joint-penetration welds without notch tough electrodes.
- Eccentrically braced frames are configured with some link beams located adjacent to columns.

1.3 Project Scope

The scope of the project involved major upgrades to the architectural, structural, and MEP systems to convert the building from traditional office space to laboratory and life science occupancies. The entire building was vacated and completely gutted for both interior and exterior renovations.

Figure 2: Architectural rendering of proposed building upgrades for 651 Gateway.

Architecturally, the existing precast exterior cladding was removed and replaced with a full height curtainwall system (Figure 2). A portion of the Level 17 floor slab was removed to create a double-height atrium space at the top of the building, and slab extensions at various other levels were added to create exterior terrace and architectural pop-out features. New canopy structures extending out from the second-floor framing were added above the building entries. The MEP systems were completely replaced, with new equipment, ductwork, and piping throughout the building, as well as ground-floor enclosures for new electrical generators and chemical storage tanks.

The structural scope involved design to accommodate the architectural and MEP upgrades, stiffening of the floor systems to accommodate laboratory space, as well as a voluntary seismic retrofit to improve the building performance. The adaptive reuse of the space from office to life science/laboratory did not classify as an occupancy change and therefore a seismic evaluation was not required. The structural engineer of record (SEOR), IMEG, did collaborate with the client to highlight the benefits of a voluntary retrofit, including improved seismic performance, reduction in operational downtime for tenants following a seismic event, prolonged lifespan for the building, and a safer and more resilient structure.

2 EVALUATION OF THE EXISTING BUILDING

2.1 Seismic evaluation approach and criteria

As a part of the project to repurpose and renovate the 651 Gateway office building as a modern life-sciences laboratory, we conducted a seismic evaluation and retrofit program as a voluntary seismic upgrade permitted by the 2016 California Building Code. The retrofit was designed to provide collapse prevention seismic performance at the code-designated BSE-2E hazard level (equivalent to the 1,000-year uniform hazard). The evaluation used nonlinear response history analysis (NLRHA) with 11 pairs of two-component ground motions, and followed the requirements of the ASCE/SEI 41-17 (ASCE 41) Standard, with the following exceptions:

- A known shortcoming of ASCE 41 is the reliance on component force and deformation criteria to assess global performance of the building structure. In our analysis, we monitored ASCE 41 component acceptance criteria, but additionally monitored global performance and aimed to explicitly model collapse modes by including component strength and stiffness degradation in the nonlinear model.
- For selected components, we explicitly modelled the variability in fracture capacity using randomized assignment of force-deformation backbone properties. Our approach differed from ASCE 41, which specifies deterministic force and deformation capacities for all components.

2.2 Beam-column connection modelling

ASCE 41 defines the moment-rotation backbone and acceptance criteria for Pre-Northridge WUF-B beam-column connections using deterministic functions related to beam depth. The relation between depth and rotation capacity at fracture represents the general trend observed in laboratory tests of similar connections (FEMA 355D, 2000). Using deterministic force-deformation backbones is justified for well-understood ductile behaviours. However, such a deterministic approach cannot account for the randomness of damage observed in actual buildings with Pre-Northridge beam-column connections (Maison and Bonowitz, 1999). In both post-earthquake reconnaissance and in laboratory tests, connection fracture capacity is highly variable, resulting from localized stress concentrations and unique weld defects. For fracturing behaviour with rapid strength loss, the use of a deterministic approach can result in unrealistic results (e.g. all beams of a given fracturing and suddenly losing strength simultaneously).

For this evaluation, we used an approach proposed by Maison and Bonowitz (1999) to explicitly model the variability in fracture capacity of welded beam-column connections by treating the rotation capacity of each connection as an independent random variable. We defined the mean flange fracture rotation using the deterministic model from ASCE 41. The distribution of plastic rotation capacity was evaluated using test results for beams of similar depth from the SAC joint venture database of connection test results (Bonowitz, 1998) as shown in the left side of Figure 3. To simplify the analysis, connections were assigned to one of three bins representing the distribution of plastic rotation capacity (Figure 3, center).

We assigned the binned connection properties to the building model using a randomized distribution representing the probability of occurrence for each capacity bin (i.e. based on the fraction of area under the frequency distribution occupied by each bin) (Figure 3, right). The NLRHA used multiple randomized distributions to evaluate the building response. For the 651 Gateway building, the global response was found to not be sensitive to changes in the random distribution.

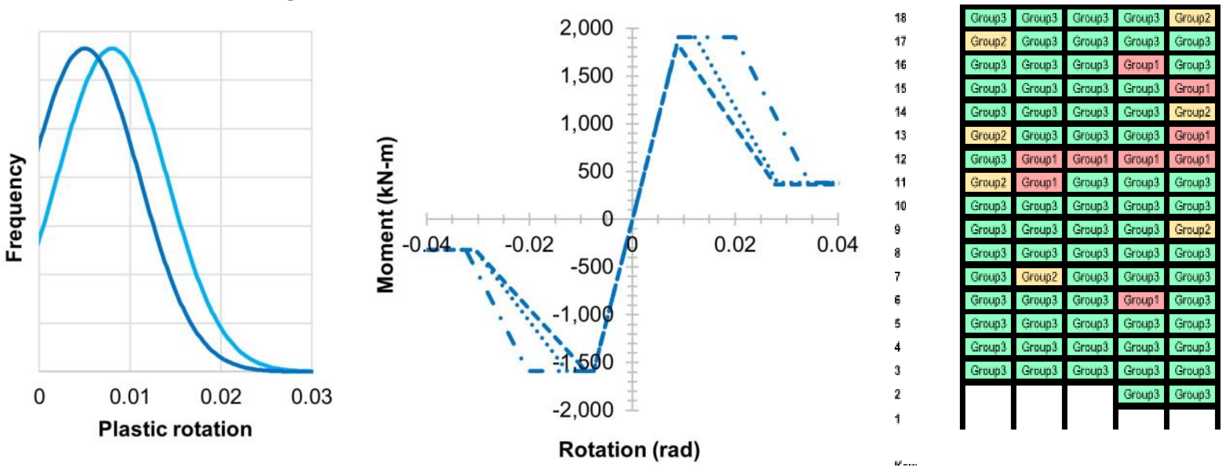

Figure 3: Beam column fracture modelling. Left, distribution of top and bottom flange fracture capacity. Center, binned moment-rotation backbones for a representative beam size. Asymmetry is due to composite action with concrete floor slab. Right, randomized assignment of connection backbone property to a representative moment frame elevation.

2.3 Column splice modelling

We calculated the fracture stress capacity of partial penetration welded steel column splices using an approach developed by Stillmaker (2016). The method calculates an effective fracture stress capacity at each splice weld as a function of material properties, section geometry, and weld size. The approach is calibrated to the results of laboratory tests and finite element fracture mechanics models.

Splice axial-moment interaction response was explicitly included in the NLRHA using fibre sections at each splice location (Figure 4, left). Individual splice fibres used asymmetrical axial stress-strain behaviour, as predicted by Stillmaker (Figure 4, right). Typically, the model predicted greater fracture capacity for partial penetration welds at the column web than at the flanges. The model captures the interaction of column axial and bending demands, but it did not represent the loss of shear capacity that accompanies weld fracture and splice opening in tension. Typical fracture response involved an individual fibre at the flange tip fracturing, leading to the progressive fracture of adjacent fibres and eventually the column web.

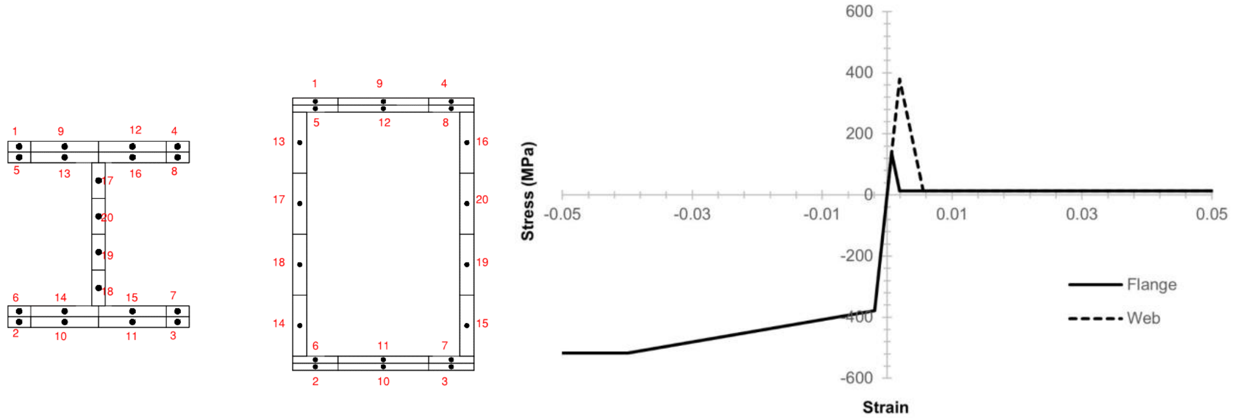

Figure 4: Column splice modelling approach. Left, fibre discretization of wide flange and box columns. Right, an example of fibre axial stress-strain behaviour predicted by Stillmaker (2016).

2.4 Existing building behaviour modes of nonlinear response

In the NLRHA, the existing building developed a multi-story concentration of drift from brittle connection fractures. The mean SRSS interstory drift was almost 0.06 (Figure 5), reflecting a near collapse state, with

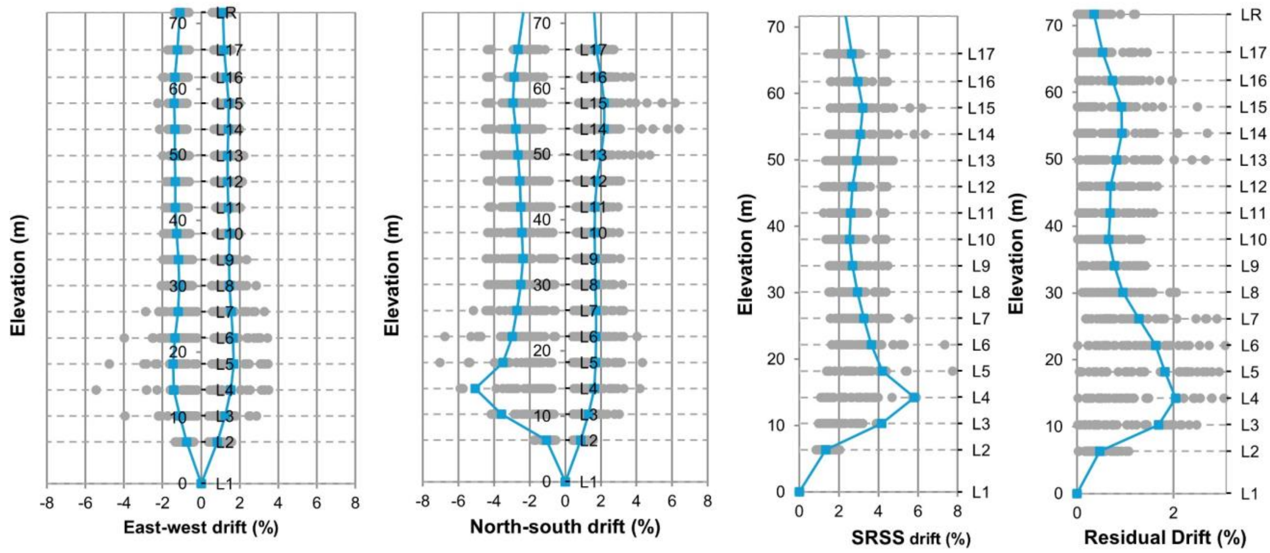

Figure 5: Building multi-story fracture response. Left, East-West interstory drift. Center-Left, North-South interstory drift. Center-Right, SRSS interstory drift. Right, residual drift.

mean residual drift exceeding 0.02. On a component level, welded beam-column connections fractured over the full height of the building, with a concentration of excessive plastic rotation occurring at Level 3 and Level 4. The mean beam-column plastic rotation demand at multiple floors exceeded the estimated rotation capacity of the existing shear connection to the beam web, and we interpreted that many of beams would have experienced a loss of gravity resistance.

The concentration of drift was exacerbated by column splice fractures occurring between Level 2 and Level 3. At this story, all of the moment frame and eccentrically braced frame column splices fracture under combined axial tension and bending. Link beams of eccentrically braced frames experience a similar concentration of deformation and fracture (Figure 6).

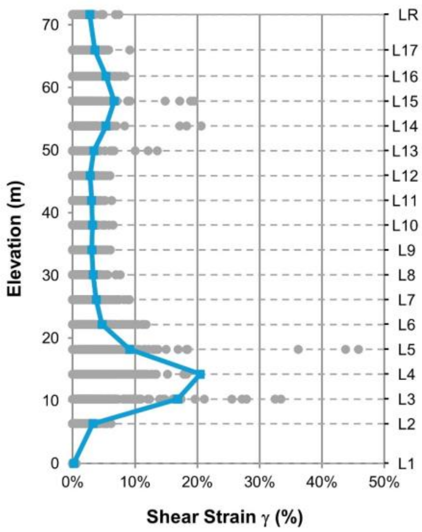

Figure 6: EBF link beam shear strain demand.

3 RETROFIT SOLUTIONS

3.1 Retrofit Options Explored

Several factors were taken into consideration in the preliminary stages of the retrofit design. The asymmetric footprint of the trapezoidal building meant that torsional behaviour under lateral loading would be a significant design consideration. Additionally, the desire to maintain the column-free interior of the floor plate for tenant flexibility meant that the additional of interior frames was avoided. Lastly, with an existing deep foundation system, it was desirable to minimize or avoid costly retrofit of the foundations.

Considering the height of the building, the addition of supplemental bracing systems would have stiffened the structure, in turn attracting higher seismic loads. The increased demand would have adversely affected the existing structural elements and resulted in higher overturning forces on the foundations. Therefore, a retrofit approach using buckling-restrained braces (BRBs) was quickly eliminated from consideration. A retrofit scheme which did not stiffen the structure, but which reduced drift-imposed demands on the lateral elements, was needed to achieve the goals for the seismic upgrades. FVDs became a viable alternative that would meet these objectives.

3.2 Overview of Fluid Viscous Dampers

Fluid Viscous Dampers (FVDs) are passive damping devices which utilize the relative movement between two points in a structure, usually two adjacent floors, to push a machined piston head through a viscous fluid. The movement generates heat as the fluid is forces through and around the piston head, dissipating energy. FVDs have unique attributes that make them ideal for retrofit applications.

1. FVDs do not introduce additional stiffness into the structure, thereby not altering the building period and increasing the seismic response of the structure.
2. Because FVDs do not add stiffness to the structure, they are able to be staggered without creating stiffness irregularities as with traditional lateral force resisting systems.
3. FVDs are velocity dependent devices, which means that they reach their peak output force when the building is at the undeformed state (zero strain) and have zero output force when the building is at peak displacement. This means that the forces introduced by the dampers are not additive to the lateral system demands directly, although there is more overlap when nonlinear dampers are used.

The fundamental equation describing the behaviour of an FVD is

$$F = CV^{\alpha} \tag{1}$$

where F = damper output force; C = damping coefficient; V = velocity; and α = damping exponent.

For this project a constant damping exponent of 0.4 was used and the damping constant, C, was varied to produce the desired building performance.

3.3 Schematic Level Damper Design

A standard approach to damper design is to use stiffness proportional damping to guide the distribution of dampers up the height of the building. The concept is to achieve a semi-uniform level of energy dissipation up the height of the building where softer floors move more and therefore can have a lower damping constant (and force) and stiffer floors move less and have higher damping constants. As a starting place, this can be implemented with linear methods using the Modal Strain Energy method (Ramirez et al., 2001).

Preliminary damping constants were determined using the Modal Strain Energy method to reduce the maximum interstory drift below 2% at the BSE-2E level. These preliminary values were then implemented in ETABS and nonlinear response history analysis was used on the elastic structure, but with nonlinear dampers, to optimize the damper properties. The 2% interstory drift target was still used as a proxy performance criteria at this schematic level,.

One chevron damper bay, each bay containing two dampers, was placed on each side of the trapezoid. Because of the torsional behaviour, it was determined that having smaller dampers on the short side of the trapezoid and larger dampers on the long size was the most impactful. Along the short side, damping constants varied from 1,025 to 773 kN-(s/m)$^{0.4}$ with output forces around 500 kN up the full height. On the other three sides, the damping constants varied from 8,300 to 4,750 kN-(s/m)$^{0.4}$ with output forces from 3350 kN at the bottom to 2500 kN at the top. This preliminary design achieved the 2% drift target in the elastic structure based on the average of 11 ground records and provided a strong proof of concept and order of magnitude cost for the damper solution – enough to progress the design forward as the preferred retrofit solution.

3.4 NLRHA design of FVDs

From here, the damper design was iteratively revised, including layout and properties, using the full NLRHA model with beam, column and link hinges. The final layout is shown in Figure 7. The final configuration included the following:

- Typical bays have dampers in an inverted chevron configuration (Figure 8). This configuration maximizes the number of locations where vulnerable bottom flange welds of the existing moment frame are protected by the addition of a gusset plate at the beam-column joint.
- Damper bays are staggered over the height of the building reducing the accumulation of axial tension columns and eliminated the need for any foundation retrofit.
- The total amount of damping was increased. Along the short side, damping constants varied from 3,866 to 483 kN-(s/m)$^{0.4}$ with output forces from 2,690 kN at Level 1 to 268 kN at the Roof. On the other three sides, the damping constants varied from 7,732 to 1,933 kN-(s/m)$^{0.4}$ with output forces from 4,213 kN at Level 1 to 1030 kN at the Roof.

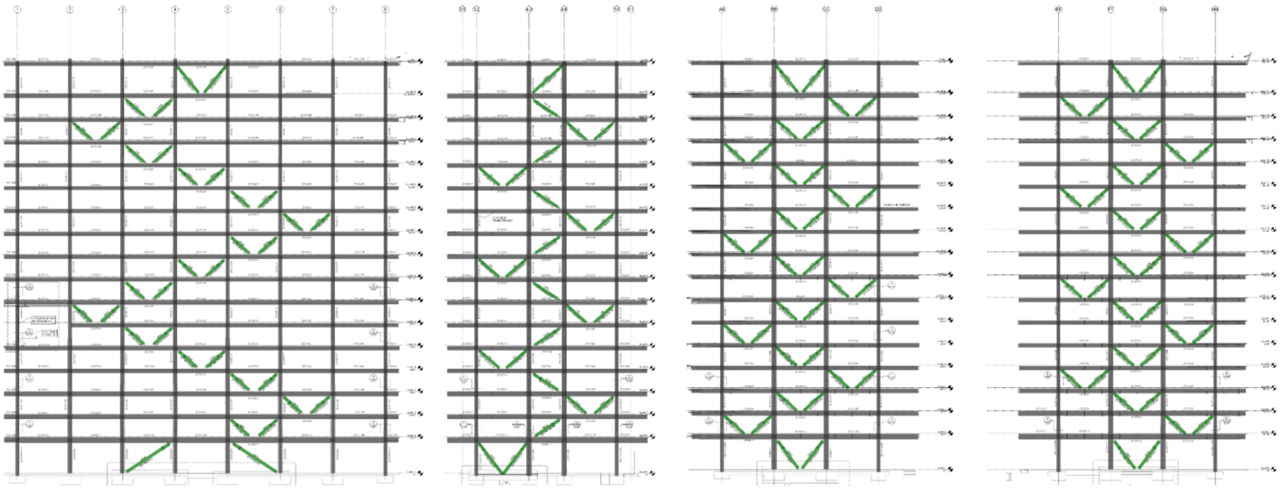

Figure 7: Final layout of fluid viscous dampers on each of the four sides of the trapezoidal building.

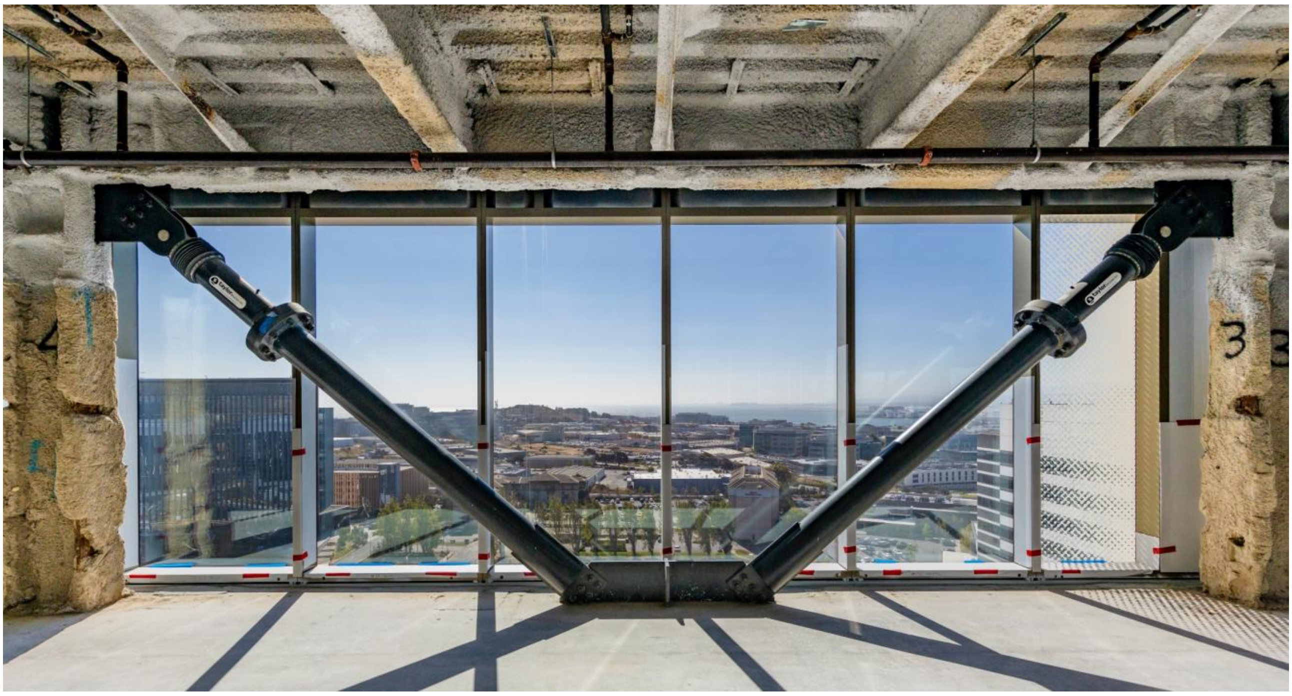

Figure 8: Typical retrofit bay with dampers oriented in an inverted chevron configuration.

3.5 Column splice retrofitting

The addition of FVDs in a staggered pattern reduced the net axial tension on columns, but the retrofitted building still experienced fractures in some welded column splices. Conventional approaches in the United States for retrofitting deficient splices in steel columns typical focus on increasing the splice strength, often by grinding to remove the existing welds and replacing them with new complete-joint-penetration welds. Alternatively, retrofits have added new flange plates spanning the weld. Such strengthening retrofits are effective, however, NLRHA showed that they increase the demands at splices higher in the building. As a result, splice strengthening retrofits often needed to be applied over multiple stories.

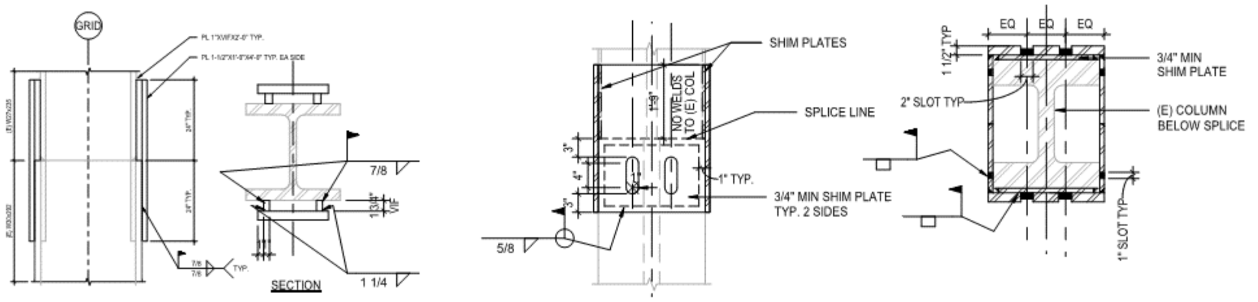

Figure 9: Column splice retrofit details. Left, conventional strengthening at flanges. Right, sleeve detail allowing column fracture without horizontal offset.

For 651 Gateway, the design uses a combination of conventional strengthening splice retrofits and an innovative sleeve retrofit (Figure 9, right). Conventional strengthening of splices occurs at the buildings' perimeter moment frames where access to all sides of the columns was possible. At interior eccentrically braced frames, which have less contribution to the building lateral resistance, we used a sleeved retrofit. In this alternative detail, a built-up steel tube was installed around the column splice. The plates comprising the tube were welded to the column below the splice, and the section of column above the splice was left unattached but confined. The detail allowed columns to fracture if they experience large axial tension demands but restrain the column against horizontal movement so that it returns to its initial position.

3.6 Response of the retrofitted building

The addition of FVDs eliminated the damaging concentration of interstory drift that dominated the response of the existing structure. Figure 10 shows a comparison of the response of the retrofitted structure and the existing structure. Mean maximum story drift in the north-south direction was reduced from almost 0.06 to less than 0.02. Residual drifts were similarly reduced; mean maximum SRSS residual drift was reduced from 0.02 to 0.004.

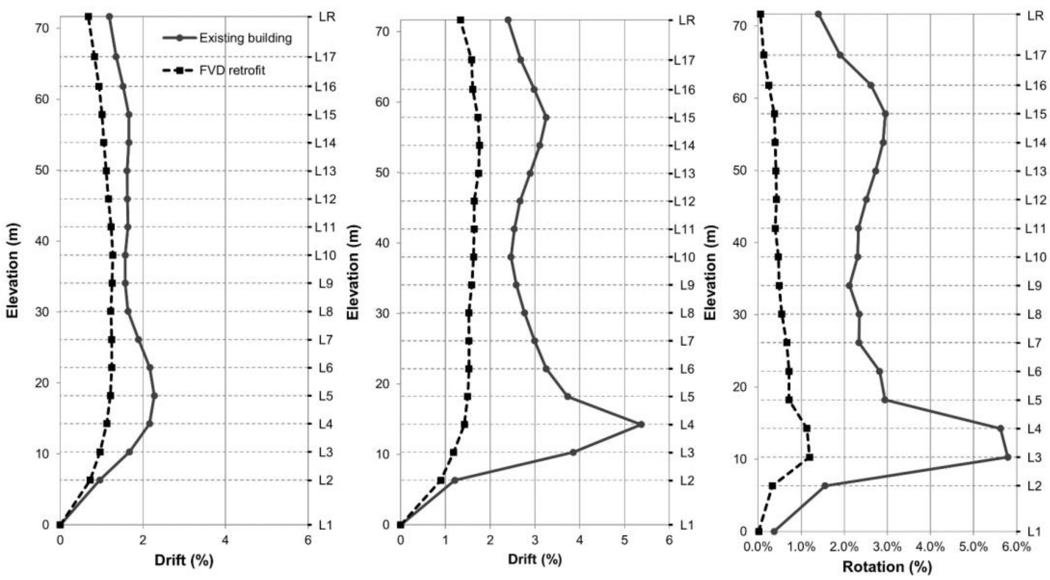

Figure 10: Comparison of existing building and retrofitted building response. Left, north-south interstory drift. Center, east-west interstory drift. Right, plot of beam hinge plastic rotation over the height of the building.

On a component level, welded beam-column connections experienced reduced plastic rotation demand, protecting against fracture and a loss of gravity capacity. Mean maximum rotation demand drops from nearly 0.06 radians for the existing building to just over 0.01 radians in the retrofitted condition (Figure 10, right).

Due to link beam geometry, a number of eccentrically braced frame link beam-column connections continue to experience weld fracture after retrofitting; however, their loss of strength does not negatively affect the global response of the building.

4 CONCLUSIONS

The use of FVDs as the retrofit solution for this structure produced drastic improvements in the seismic performance of the building. With the exception of relatively minor seismic upgrades to existing column splices, the damper solution eliminated the need to strengthen or retrofit nearly any other existing element, including moment frames, eccentrically braced frames, base plates, and foundations. The seismic performance of the structure will most certainly be put to the test in the next major earthquake, and it is expected to behave significantly better than the pre-retrofit condition.

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