maturity of 0.9 can be achieved at iteration 110, a maturity of 0.91 can be achieved at iteration 299, and a maturity of 0.93 can be achieved at iteration 499. The corresponding fitness of the best solutions are 0.0107, 0.0107, and 0.0108. It is concluded that a maturity of about 0.9 is sufficient to achieve reasonable solutions. Maturities above that require more iterations and therefore increase the cost of optimization.

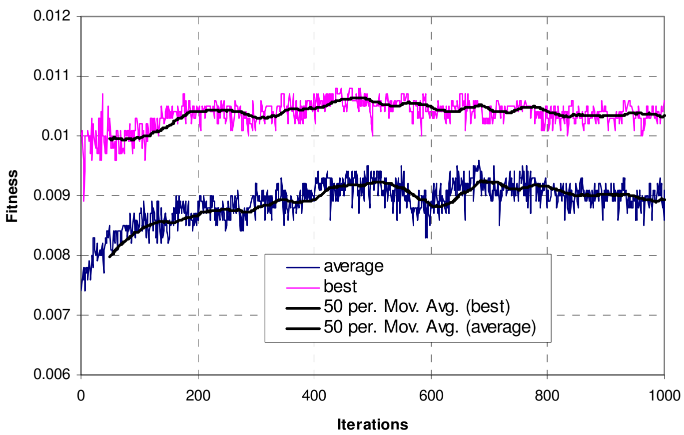

Figure 8 Variation of the best solution fitness and average population fitness with iterations for example building II

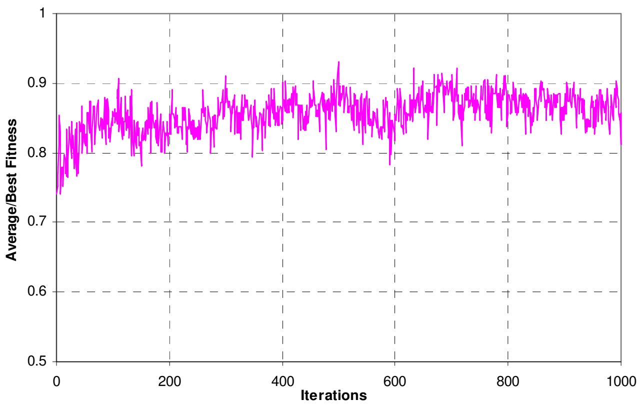

Figure 9 Variation of the average/best fitness with iterations for example building II

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Viscous Dampers Used to Renovate Twin 17-Story State Buildings

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Abstract

This paper describes the structural investigations and subsequent structural renovations that were performed on the twin 17-story State Office Buildings 8 and 9 located four blocks south of the State Capitol in Sacramento, California. The renovation designs conform to a performance criteria established by the California Department of General Services (DGS) with the intent that structural damage would be moderate and Risk to Life would be minor during a large earthquake.

Introduction

The seismic resistance capability of these two buildings was initially evaluated in 1997 by CYS Structural Engineers, Inc. (CYS) as part of the post-Northridge Earthquake effort by the California Department of General Services (DGS) to identify state-owned buildings that should undergo seismic resistance improvement. The primary focus of this DGS program was to identify buildings that have serious seismic deficiencies and establish a prioritized list of such buildings so that the correcting problem buildings could be addressed in a rational sequence.

Figure 1: Office Buildings #8 and #9

The two buildings were designed in the mid-1960's, probably using the 1964 UBC. Seismic design forces required by that code for these buildings would be about 60 percent of the seismic forces required by the 2001 CBC, the code of jurisdiction at the time the subject renovation design for these buildings was undertaken. During this initial evaluation, CYS determined that each building has a significant torsional irregularity. Their structural systems were far too flexible from the third floor up and significant seismic force resistance deficiencies existed throughout the buildings. Deficiencies and mitigation design will be discussed below.

Figure 2: Structural Isometric

Building Descriptions

The towers are virtually identical, approximately 254 ft. tall, approximately 144 ft. square and have full basements. They were constructed as steel-frame buildings with perimeter moment frames from the 3rd floor to the roof, and concrete shear walls from the basement to the 3rd floor. The steel moment frames were constructed with 33” and 36” deep wide flange beams and heavy 14” wide flange columns. The moment frame columns clear-span (unbraced) outside of the first and second story exterior walls and extend through the basement to the mat foundation. Most of the concrete shear walls are offset inward from the perimeter moment frames approximately 6 ft on three sides, and approximately 43 ft. on one side, see figure 3. The shear walls that are offset inward 6 ft (at three sides) are discontinued just below the first-floor slabs and transfer horizontal seismic forces to the basement perimeter walls through the first-floor slab. The discontinued walls impose large vertical seismic forces onto concrete piers located at the basement. The floors are constructed as either concrete over metal deck or concrete flat slab supported by a steel purlin and girder floor framing system. The exterior curtain wall system is located approximately two feet outside the perimeter steel frames from the third floor to the roof. The structure also has an architectural precast concrete cladding system that is located about 8.5 feet outside of the perimeter frames and is supported on cantilevered steel beams. The cantilevered beams that support the precast cladding pass through the perimeter moment frame girders at various locations. The buildings are supported by 7 ft. thick mat slabs 14.5 ft. below the finish grade, and a utilities tunnel connects the structures at the basement level.

Material Properties and Original Design Loads

The original, as-built, properties of the building materials are not fully described in available documents. The missing properties were conservatively assumed, following FEMA 356 guidelines and considering the type of material described in the documents and the typical historical local building practice at the time. The gravity design loads are listed in available documents, but the seismic and wind design loads are not. The available documents describe all dimensions and structural systems sufficiently to allow performance of a detailed seismic evaluation of the building.

The available documents do not address the basis for the seismic design of the structure. The design was probably carried out under the 1964 UBC, which prescribed a static lateral-force analysis based on the height distribution of a minimum total shear at the base [2314(d) 1964 UBC].

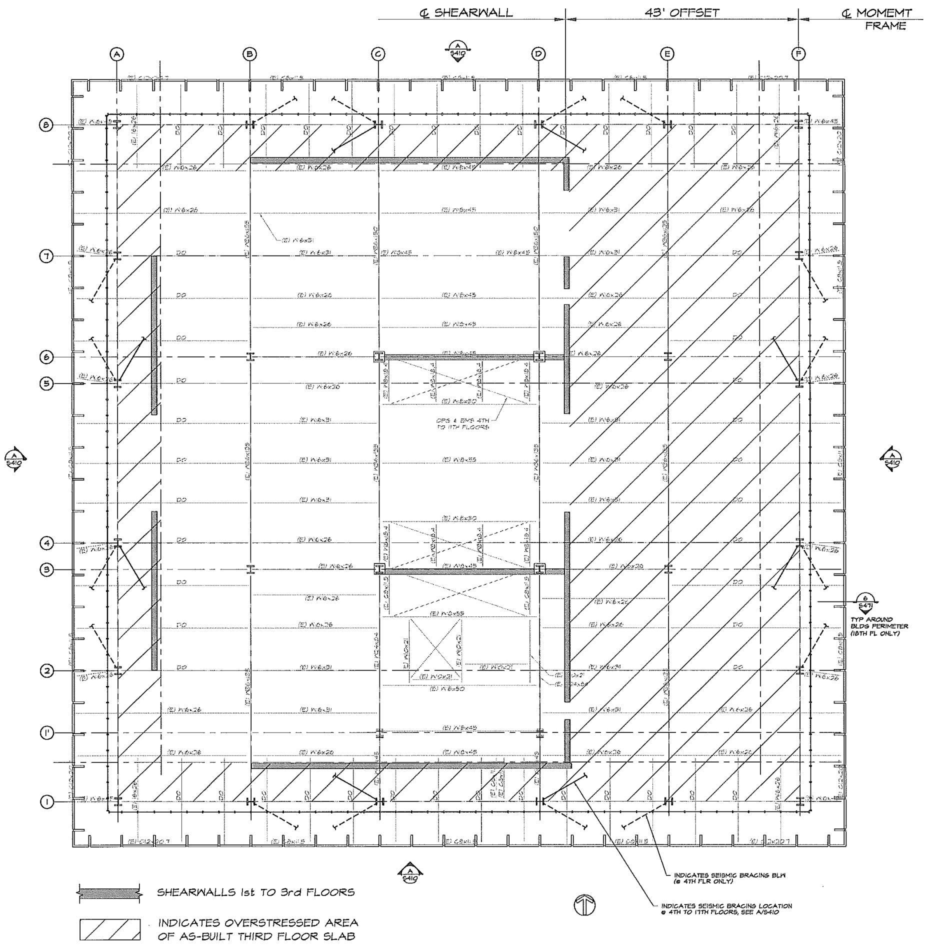


Figure 3: 3rd Through 18th Floor Framing Plan

Initial Investigation and Deficiencies

The 1997 seismic evaluation was conducted on Office Buildings 8 and 9 by CYS following guidelines detailed in FEMA 178. This evaluation revealed a significant torsional irregularity at the first and second stories and substantial deficiencies in the steel moment frames, concrete shear walls and their coupling beams, concrete diaphragms and collectors, and concrete columns/piers supporting discontinuous shear walls at the basement. The consensus opinion was that, during a code-level seismic event, risk to life would be substantial, all systems would be disrupted and the building would have to remain vacated during repairs (if cost-effective). The cost for the schematic structural retrofit schemes developed by CYS was reportedly estimated by the DGS Program Manager to be about $11 million (1997 dollars).

Remodel/Renovation Contract

By mid-2002, DGS had finished preliminary planning for a complete renovation/ remodel for the interiors of the two buildings and invited proposals from design teams for a final design. DGS selected Hammel Green & Abrahamson, Inc. (HGA) as the design Architect, with CYS included as the structural engineering consultant for the seismic renovation that was to be included in the project.

Renovation/remodel designs were performed simultaneously for the two buildings, but construction for the projects was phased sequentially with only one building vacated at a given time. Occupants of Building 8 were relocated to other buildings in Sacramento and construction on No. 8 and a new two-story connecting lobby building was completed in August, 2008. Occupants of Building 9 were then relocated to No. 8 and construction was initiated on No. 9. Structural renovation on No. 9 is nearing completion at press time and final build-out is scheduled for August, 2010.

Requirements and Basis of Design Criteria

HGA's contract with DGS specified that the design satisfy the requirements of the current pertinent codes. The structural renovation design was required to conform to the 2001 California Building Code (2001 CBC). Thus, due to the torsional irregularities in the buildings, a mandatory seismic evaluation, and design retrofit, if needed, is required [1640A.2(1), 2001 CBC]. To comply with the 2001 California Building Code (CBC), Chapter 16A, Division VI-R, a full seismic retrofit of Office Buildings 8 and 9 is required. The 2001 CBC requires that the buildings must meet an essential life-safety level of performance. This level of performance is presumed to be achieved when:

a) The building has some margin against either total or partial collapse.
b) Major structural elements have not fallen or been dislodged causing a life-safety threat.
c) Non-structural systems or elements that are heavy enough to cause severe injuries either within or outside the building have not been dislodged causing a life-safety threat.

Furthermore, DGS' criteria for this project requires that the building performance during a code-level seismic event would be such that risk to life would be minor and structural collapse would not occur.

Current Seismic Hazard

The buildings occupy the block surrounded by P (North), 8th (East), Q (South) and 7th (West) Streets (latitude: *38.574*° N; longitude: 121.500° W). This site is not located within a State-designated Alquist-Priolo Fault Zone where site-specific studies addressing the potential for surface fault rupture are required and no known active faults traverse the site. However, the site lies within Seismic Zone 3 [1629A.4.1, 2001 CBC], just 37.126 km (23.069 miles) Northeast from Seismic Zone 4. The nearest faults to the site are the Foothills and the Great Valley fault systems, located approximately 42 km to the East and 43 km to the Southwest, respectively. A detailed seismic hazard analysis was conducted for the site, generating six sets of three time-history earthquake acceleration components each. Three sets define a Design Basis Earthquake (DBE), and three define a Maximum Considered Earthquake (MCE), or seismic events with 10% and 2% probability of exceedance in 50 years, respectively. It was required that the components of these sets of time histories be statistically independent from each other and from those of the other sets with a correlation not exceeding 15%. These two earthquake hazard levels define two Basic Safety Earthquakes: BSE-l (~10%/50 year) and BSE-2 (~2%/50 year).

Seismic Retrofit Design (Mitigation) Criteria

As prescribed, CBC Method B was the basis for evaluation and design [1643A.1, 2001 CBC] using special procedures as provided by FEMA 267 [1647A.1.1(4), 2001 CBC]. FEMA 267 has been modified to FEMA 267A16 and FEMA 267B, and then superseded by FEMA 351, which was ultimately used in this project to establish the deformation (interstory drift) capacity criteria for the steel moment frames. According to Method B [1648A, 2001 CBC], the following has been implemented in the procedures used:

- The approach, models, analysis procedures, assumptions on material and system behavior, and conclusions have been peer reviewed by ABS Consulting at every major phase of the project [1648A.2, 2001 CBC].
- The basis for using and the specific values of load factors, demand/capacity modification factors, and measures of inelastic deformation have been consistently applied [1648A.2.1.1, 2001 CBC] according to FEMA 351 and FEMA 356.
- Three distinctly representative earthquake records with simultaneous loadings in the three building principal directions were applied to the base of the building to carry out dynamic time-history analyses, and maximum response parameters were used for evaluation and design [1648A.2.1.2, 2001 CBC], as appropriate, for each level of seismic demand.
- Ground motion characterization follows FEMA 273 guidelines [1648A.2.2.1, 2001 CBC] adequately developed for Method B time-history analyses as superseded by FEMA 356.

Schematic Phase Work

The schematic design phase analyses validated most of the previously determined deficiencies. A detailed three-dimensional model was constructed and analyzed with ETABS using three separate time-history data sets for a Design Basis Earthquake (DBE) and three time-history data sets for a Maximum Considered Earthquake (MCE). Major deficiencies found were: steel moment frames did not meet required level of performance, drift confidence levels were significantly below the recommended 50% minimum, floor deck capacities at the 3rd floor appeared deficient in transferring the lateral loads from the moment frames, diaphragm connections to the shear walls were inadequate, several shear wall segments were deficient for shear forces and concrete basement piers that support discontinuous shear walls were deficient. A mitigation scheme was proposed that consisted of a chevron brace configuration with friction dampers to reduce the drift and dampen the seismic energy delivered to the structure. This preliminary scheme did stiffen the structure but was found by later analysis to not dissipate as much energy as viscous dampers would and also would allow excessive forces in too many primary components.

Retrofit Selection, Software, Analysis Procedure

Final Retrofit Configuration. Since the building's steel moment-frame configuration lends itself to be braced, a seismic mitigation scheme based on the bracing of the moment frames was selected. By bracing the moment frames, the drift is reduced, but the loads on the frame columns are increased. To simultaneously reduce the drift and the column loads, and dissipate the seismic loading effect on the structure, a bracing system consisting of steel braces with in-line viscous dampers was selected.

The brace/damper configuration consisting of diagonal tube sections (HSS) in line with viscous dampers was designed to resist a design force derived from a design velocity induced by the stroke of the damper. This design velocity is equivalent to 130% of the maximum damper axial velocity [9.3.1.1, FEMA 356] enveloped over all the earthquake time histories used. All HSS-braces in a single floor were designed for the maximum design force occurring on that floor, and all elements of the bracing configuration, such as connecting flange, gusset and clevis plates, fasteners and welds, were designed for that design force.

Computer Analysis Two-Dimensional Model Investigation. To speed-up the process of determining an effective brace/ damper configuration, a two-dimensional model was developed from the three-dimensional model, consisting of a single perimeter moment-frame tuned to respond to the same first mode of vibration of the three-dimensional model, and approximately the next two modes and as many of the higher modes as possible. This process was instructive, as it helped debug the process and the post-processing software.

Three-Dimensional Model Analysis Fine-Tuning. The brace/damper configuration selected was analyzed in the three-dimensional model and a process of fine-tuning the damper characteristic parameters was carried out. The fine-tuning was carried out by analyzing the model for different sets of damping coefficients, c, and damper velocity exponents, α, to achieve the minimum moment-frame column demand/capacity ratio for a maximum allowable drift ratio. To increase the number of possible damper vendors, two different damper velocity exponents were considered, $0.30 \leq \alpha \leq 1.00$ and $\alpha=0.15$.

Seismic Evaluation – Final Results

Inter-Story Drift and Base Shear. The inter-story drift for the retrofitted structures are shown in Figure 5 with $\alpha=0.4$ and $\alpha=0.15$ dampers. Figure 5 demonstrates that the inter-story drift is below the maximum acceptable level defined by FEMA 351 (Table 3-2). The building's time domain response to one of the strongest MCE loads (MCEG42) was recorded for the building moment frame located on gridline F. On this time domain response it was observed that the building responds primarily to the first mode and responds in slower motions than the as-built configuration. Thus, the damping configuration reduces the higher mode participation, which results in slower motions and lower stress levels. Total story loads (for $\alpha=0.4$) were observed demonstrating that the building still has a strong torsional modal response and that the base shear in the direction where it is maximum is 8.48% and 11.80% of the total weight of the building for the DBE and the MCE loads, respectively. This is still higher than that for which the building was designed, but was determined to be acceptable for all existing elements.

Existing Moment Frame Members. Through the use of reviews with time-domain results, it was determined that the selected brace/damper configuration reduces combined forces to acceptable levels at all moment-frame beam members and all moment-frame columns except two 4^{th} –level corner columns.

Existing Reinforced Concrete Elements. The combined load forces are reduced by the dampers to acceptable levels at all **shear wall segments** except two elevator walls in the 17^{th} and 18^{th} stories. Also, the combined forces imposed on **concrete basement piers** by the damped structures are acceptable if the tops of the piers are confined. Confinement is provided using Fiber Reinforced Plastic (FRP) wrap and drilled FRP anchors at the pier tops as shown in Figure 10.

New Steel Braces and Viscous Dampers. By choice and for simplicity, all HSS brace/viscous damper segments were designed for the same characteristic parameters. For the final fine-tuned computer model, all dampers on a building level were designed for the maximum absolute axial brace force obtained at that level. Table 1 summarizes the main characteristics for the α=0.4 and α=0.15 brace/damper configurations. Figure 4 displays the maximum brace/damper forces.

BRACE/DAMPER CONFIGURATION DESIGN CHARACTERISTICS					
		COEFFICIENTS		CAPACITIES	
BRACE	HSS SECTION	VELOCITY α	DAMPING C, kips/(fps)$^{\alpha}$	FORCE P_{des}, kips	STROKE Δ_{des}, in
Single	HSS7x7x5/8	0.40	400	155-275	4.25
Cross	HSS10x6x5/8	0.40	400	243	3.25
Single	HSS6x6x5/8	0.15	150	115-139	4.75
Cross	HSS10x6x5/8	0.15	150	135	3.75
NOTE: For a viscous damper, $P=cv^{\alpha}$, such that $P_{des}=c(1.3v_{max})^{\alpha}$.					

Table 1. Brace/Damper Configuration Characteristics for Office Buildings 8 and 9.

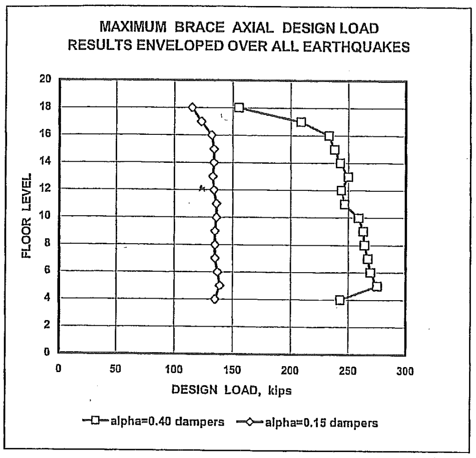
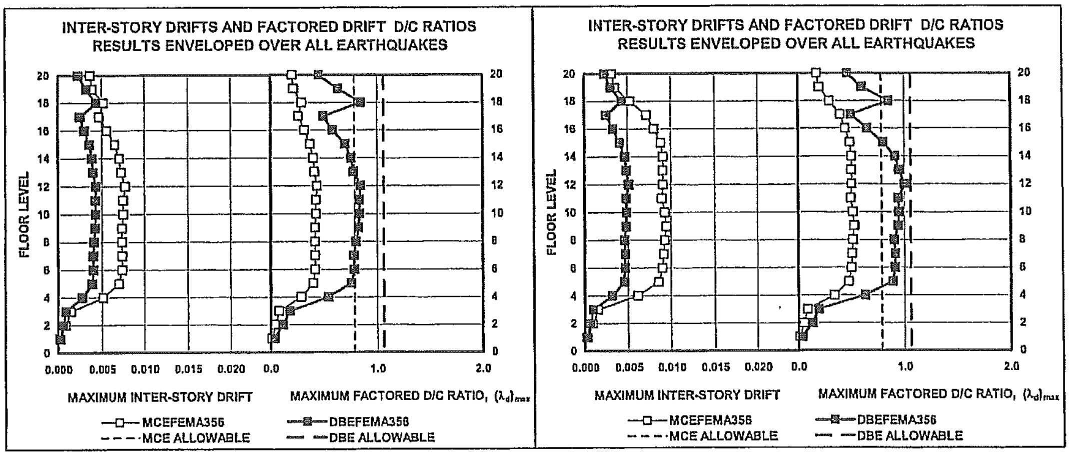
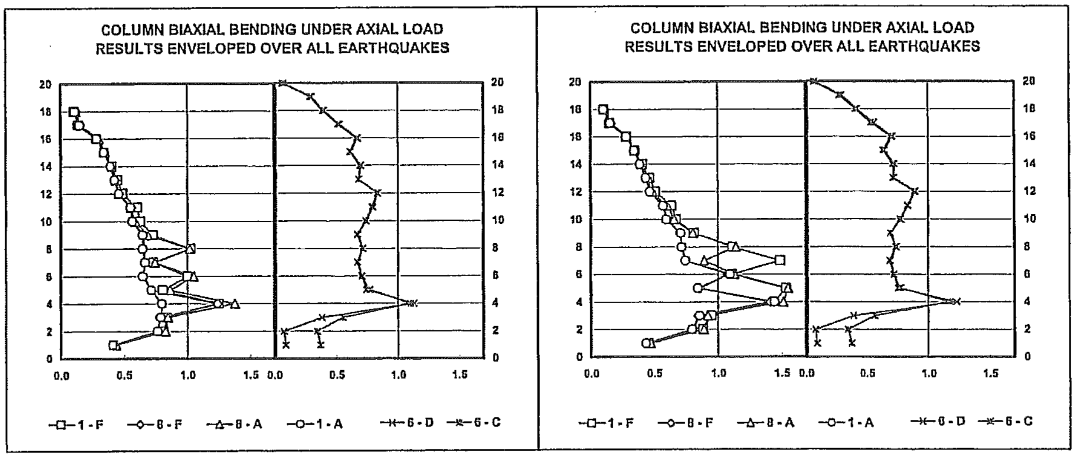


Figure 4: Brace/Damper Configuration Axial Loads for the Retrofitted Office Buildings 8 and 9



(a) Bracing configuration with α=0.40 dampers. (b) Bracing configuration with α=0.15 dampers.

Figure 5: Maximum Factored Inter-Story Drift Demand-to-Capacity Ratio for the Retrofitted Office Buildings 8 and 9



(a) Bracing configuration with α=0.40 dampers. (b) Bracing configuration with α=0.15 dampers.

Figure 6: Maximum Factored Column Demand-to-Capacity Strength Ratio for the Retrofitted Office Buildings 8 and 9

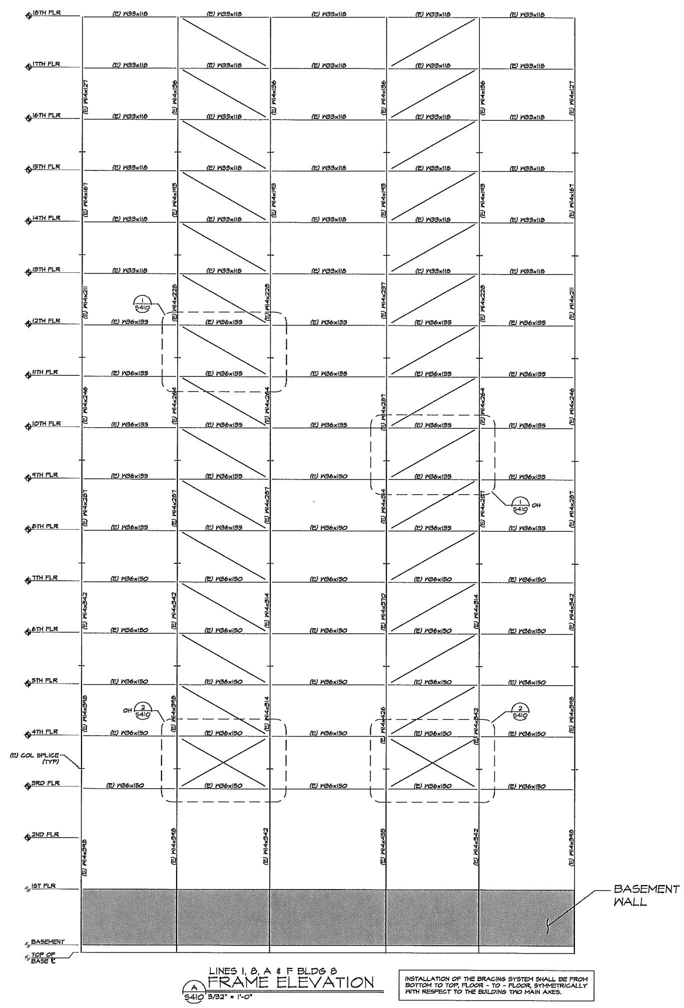


Figure 7: Brace/Damper Configurations

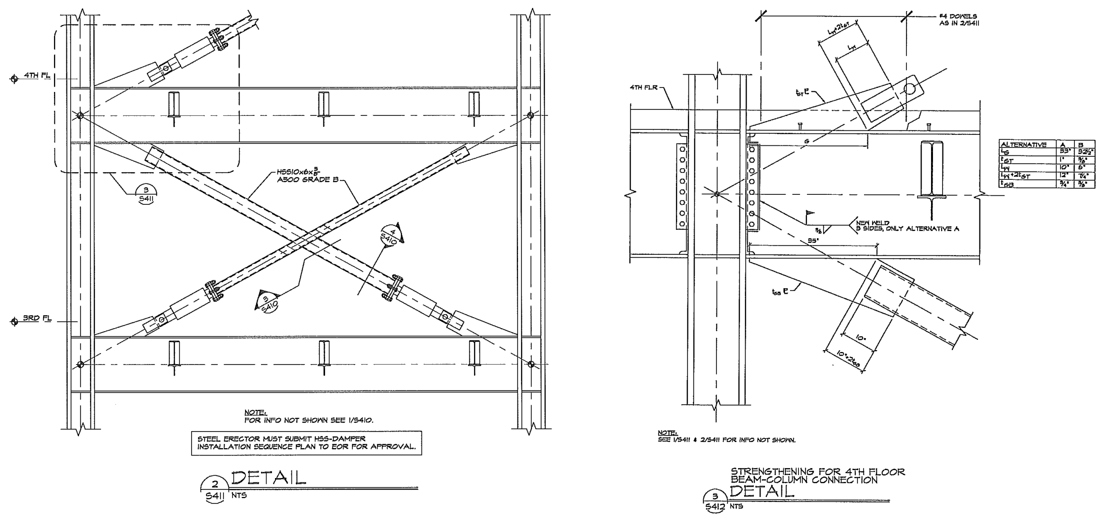


Figure 8: Crossed Seismic Bracing