

# SEISMIC RETROFIT OF TWO MOMENT FRAME STEEL STRUCTURES

F. Medina<sup>(1)</sup> and K.A. Luttrell<sup>(2)</sup>

<sup>(1)</sup> Principal, FME Engineering, San Francisco, California, USA, francisco@cal.berkeley.edu. <sup>(2)</sup> Principal, CYS Structural Engineers, Inc., Sacramento, California, USA.

## Abstract

Two 18-level twin towers with a total of 53,900 m<sup>2</sup> of usable covered space located in Sacramento, California were designed using a 1966 code, inadequate to accommodate current seismic code prescriptions resulting from the updated seismic hazard estimated for the site. The building structures include steel gravity frames from a concrete mat foundation to roof, perimeter steel moment frames, and asymmetrically-recessed concrete shear walls from foundation to third floor. Seismic forces must transfer from the perimeter frames to the shear walls through third-floor slabs. Three of the first-story shear walls are discontinued at the first-floor slabs and must transfer horizontal seismic forces to basement walls through the first-floor slabs, and vertical forces onto concrete piers located at the basements. As a result of current-code higher seismic demands, the buildings were found seismically deficient due to: pre-Northridge steel moment-frame joints, first and third-floor slab shear capacities, shear transfer connections between slabs and walls, deficient basement concrete piers supporting shear walls, first- and second-story wall shear capacities, and excessive flexibility of the steel frames.

A number of conventional and advanced seismic retrofit schemes were studied to provide drift control to: (a) avoid unacceptable damage to the moment frame joints; and (b) limit the higher seismic loads that the structural members would undergo in the case of a destructive earthquake. These studies involved state-of-the-art approaches such as site-specific time-history seismic demands, three-dimensional non-linear time-history analysis, and project-specific software tools developed for time-history FEMA design checks. This latter approach allowed us to confidently accept the seismic demand defined for the structures in a way that enveloped results would not. By strengthening the perimeter frames with diagonal braces and in-line viscous dampers, the seismic safety requirements are met. The final retrofit design, a massive viscous-damper configuration, was distributed from the third floor throughout the height of the buildings. This configuration restricted the extent of retrofitting to sections of the building with easy access, limiting the required strengthening of the seismically-deficient reinforced-concrete elements below the third floor. The diagonal brace with in-line viscous damper configuration included sliding cross braces defining a novel design not used before in this type of applications. The solution reduced the estimated seismic retrofit construction cost from USD 15.2 million to an amount just above USD 3.3 million. The 136 dampers designed, per building, were subjected to a rigorous testing protocol, and subsequently installed in the newly-renovated buildings, currently in use.

Keywords: Seismic protection; seismic retrofit; energy dissipation; viscous dampers; moment-frame steel building.

## 1. Introduction

## **1.1 Project description**

The buildings, Office Buildings Number 8 and Number 9 (OB8&9), located in downtown Sacramento, are owned by the State of California and managed by the Department of General Services (DGS). The buildings were designed in 1966; construction started in 1967. A preliminary seismic evaluation carried out on the buildings in 1997, following FEMA 178 [1], revealed structural deficiencies requiring that the building be seismically rehabilitated. DGS required the seismic rehabilitation to satisfy the current applicable codes' specifications and the risk levels for State-operated non-essential buildings. The structural retrofit design followed the 2001 California Building Code (2001 CBC) [2]. DGS provided structural [3-5] and architectural [6] drawings. In the absence of other documents (construction inspection documents, construction specifications, structural calculations, original soils report), these drawings were the basis of the retrofit design reported herein. The structural drawings indicated that both buildings were identically designed, except for a few structural



members, accommodating distinct minor architectural features. Therefore, the structural procedures described herein were carried out for one building and used in the other with minor plan changes.

## **1.2 Building description**

The twin, nearly identical, 18-level buildings contain over 296,415 square-feet each, of useable covered space, each including a full basement, recessed second floors, and two-level penthouses. The structures are supported by a continuous concrete mat foundation. The exterior cladding consists of precast concrete, including horizontal and vertical panels from the third floor up (see Fig. 1). The structural features of the buildings include 22 supporting steel columns bolted to the foundation and distributed on four perimeter moment-frames (Fig. 2) and ten *gravity* columns connected to the moment frame by steel girders in one direction only. Reinforced concrete shear walls are located at the basement perimeter in line with the moment-frame columns, and also, in the building interior, from the foundation to the third floor (Fig. 3). Three additional shear walls in the first and second stories are asymmetrically set back from the top of the first floor slab to the underside of the third floor. Horizontal seismic forces must transfer from the perimeter frames to the shear walls through the third-floor slab. The three first-story shear walls are discontinued below the first-floor slabs and must transfer horizontal seismic forces onto concrete piers located at the basement perimeter walls through the first-floor slab and vertical seismic forces onto concrete piers located at the basement perimeter walls through the first-floor slab and vertical seismic forces onto concrete piers located at the basement.



Figure 1 – Steel moment-frame twin towers: OB8&9 (view from the North).







Figure 2 - Perimeter moment-frame and basement wall at gridline F. Structural elevation from the three-dimensional structural model developed for OB8&9.



# 2. Original Design and Retrofit Mitigation Criteria

## 2.1 Material properties and design loads

The original as-built properties of the building materials are not fully described in the documentation available. The properties missing were conservatively assumed following FEMA 356 [7], FEMA 351 [8], and considering the type of material described in the documents and the typical historical local building practice at the time. The static design loads are listed on the available documentation, but the earthquake and wind design loads are not. The original and assumed material properties and the original static design loads are presented elsewhere [9].

## 2.2 Original seismic design criteria

The design was probably carried out under the 1964 UBC [10], which prescribed a static lateral-force analysis based on the height distribution of a minimum total shear at the base. An improved method [11], not significantly different, was also used at the time in California (SEAOC blue book). Thus, the total lateral seismic design forces assumed to act non-concurrently in the direction of each of the structure main axes are

$$V = KCW \le 0.1KW \tag{1}$$

where K=0.67,  $C=0.05/T^{\frac{1}{3}} \le 0.1$ , and T=0.1N, such that N is the number of stories above grade. Load combinations did not prescribe a factor modifying the seismic force in the case of steel structures. This type of lateral static-load seismic design is still considered in the current California code for simpler structures, such that

$$0.11C_a IW \le V = C_v IW/(RT) \le 2.5C_a IW/R; \quad T \le 0.4C_t h^{\frac{3}{4}}$$
(2)

where  $C_a=0.36$ ,  $C_t=0.035$ ,  $C_v=0.54$ , I=1, R=4.5, and T is the first natural period of the building. In addition, accidental torsional effects must be included. The fundamental periods of the building in the two horizontal directions are 4.46 and 4.41 seconds (Section 3.4.2). Thus, for the purpose of evaluating and comparing the original seismic design base-shear with the current code base-shear, assuming conservatively that the steel moment-frame structure is only, from the third story, 15 stories high, Eqs. (1) and (2) yield

$$V/W = 0.0293 \le 0.067;$$
 SEAOC blue book  
 $0.0396 \le V/W = 0.0460 \le 0.200;$  2001 CBC (3)

Thus, if the original design followed the then current state-of-best-practice; that is, the SEAOC blue-book recommended lateral force requirements, the 1001 CBC prescribes an earthquake load at least 57% higher than the one used for design.

## 2.3 Updated seismic hazard and updated earthquake loads

The site lies in Seismic Zone 3 (Fig. 4), about 37 km Northeast from Seismic Zone 4. The nearest faults 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 [12], 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 events respectively with 10% and 2% probability of exceedance in 50 years (10%/50yr and 2%/50yr). Fig. 5 shows the site-specific horizontal and vertical component acceleration spectra, and the acceleration time-histories for one of the MCE earthquake time-histories sets developed. 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-1 (~10%/50yr) and BSE-2 (~2%/50yr).

## 2.4 Seismic retrofit design (mitigation) criteria

The building was seismically evaluated and analyzed, and the seismic retrofit designed according to 2001 CBC, chapter 16A, division VI-R. The purpose was to provide a minimum level of seismic performance at the essential life-safety level, as defined in the code. Method B, as prescribed by the code, was the basis for evaluation and design using procedures provided by FEMA 267 [13], subsequently superseded by FEMA 351

[8], which was ultimately used to establish the inter-story drift (drift hereafter) capacity criteria for the steel moment frames. According to Method B, the following was implemented in the procedures used:

- The approach, models, analysis procedures, assumptions on material and system behavior, and conclusions were peer reviewed at every major phase of the project. The resulting retrofit design was further reviewed by a blue ribbon panel. Both reviews were carried out independently and reported directly to DGS.
- The basis for using, and the specific values of, load factors, demand/capacity modification factors, and measures of inelastic deformation have been consistently applied according to FEMA 351 and FEMA 356.
- Three distinctly representative earthquake records with simultaneous loadings on 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, as appropriate, for each level of seismic demand.
- The reinforced-concrete shear wall stiffnesses used were one-half of those given by the actual cross sections.
- All concrete-encased steel was conservatively assumed non-composite and the concrete weight and mass added linearly to the model elements encased by concrete. The weight and mass was similarly added to the model elements partially encased by concrete and/or covered by precast panels.
- Framing element lengths used were equal to the distance from joint-center to joint-center for analysis, but equal to the clear-span distance for capacity calculations; the joint configuration was elastically modeled.
- Foundation flexibility was conservatively ignored.
- Ground motion characterization follows FEMA 273 [14] guidelines adequately developed for Method B timehistory analyses as superseded by FEMA 356.



2003) for OB8&9's site.

horizontal and vertical response spectra (upper panel). Site-specific time-histories for the three components of a matched MCE: seed, 1989 Loma Prieta earthquake, Gilroy Station 4 (three lower panels).

FEMA 356's Life Safety (LS) structural performance level criterion satisfies the 2001 CBC essential lifesafety level. Furthermore, FEMA 356 structural performance level is defined as Collapse Prevention (CP), when in the post-earthquake damage state "the structure continues to support gravity loads but retains no margin against collapse." The Basic Safety Objective is a rehabilitation that achieves the dual goals of LS for the BSE-1



and CP for the BSE-2. This was the fundamental seismic rehabilitation criterion used. Thus, FEMA 356 was the primary guide followed to define seismic demand and strength capacities. On the other hand, FEMA 351 was used to define the moment frame drift capacity to verify that the joints did not deform excessively.

## 2.5 Preliminary analysis

The schematic design phase analyses carried out on a detailed three-dimensional model confirmed most of the previously-determined deficiencies. A mitigation scheme, estimated [15] at \$15,233,964, excluding mark-ups, construction phasing, escalation and contingencies, was proposed to reduce these deficiencies as follows:

- A chevron-brace configuration with friction dampers to reduce the drift and dampen the seismic forces in the building structure from the 3<sup>rd</sup> to the 12<sup>th</sup> floors on all intermediate 24-foot bays around the perimeter frame.
- Increase the 3<sup>rd</sup> floor diaphragm shear capacity at the connection to the offset shear walls.
- Increase the moment-frame corner column capacity from the 1<sup>st</sup> to the 3<sup>rd</sup> floor using steel plates, removing and replacing the concrete encasement and precast panel cover.
- Remove the  $3\frac{1}{2}$ -thick topping slab on the  $3^{rd}$  floor and replace with lightweight concrete.
- Strengthen existing concrete basement piers by adding a shear wall between the piers, removing the end of the piers and replacing with a ductile reinforced section connected to the ends of the new shear walls.

## 3. Seismic Evaluation – Preliminary Results

## 3.1 Analysis procedure and software tools

A seismic mitigation scheme based on the bracing of the moment frames was implemented. 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 earthquake loading effect on the structure, a bracing system consisting of steel braces with in-line viscous dampers was proposed.

**3.1.1 Computer model.** The existing building and the proposed bracing configuration were represented in a detailed structural ETABS [16] model. The steel and concrete members were modeled to include the spatial stiffness, weight and mass distributions. There was no weight or mass lumping, except for the precast panel fins. All steel was structurally modeled according to AISC [17-18] section properties as frame elements. Splices were included in the model, *i.e.*, where a splice occurs at mid-story column height, a node was defined, and two different sections were modeled at each side of the node. The following reinforced concrete members were modeled: (a) basement piers as a combination of column frame and wall shell elements; (b) shear walls as wall shell elements; and (c) floor slabs, elevator shaft slabs on the 11<sup>th</sup> and 18<sup>th</sup> floors, and concrete over metal decks as floor shell elements. The shell elements were sufficiently discretized to represent the spatial stiffness, weight and mass distributions. The wall shell elements were grouped as ETABS piers and spandrels to retrieve the equivalent internal forces. Significant wall and floor openings were included in the model. The concrete encasing steel columns and beams was considered as distributed linear weight and mass loads on the corresponding modeled elements. No composite action was considered. In addition, architectural, mechanical and electrical features contributing to the dead load of the structure such as soffits, ceiling (electrical and mechanical), floor pads, curtain glass walls, roofing material, etc., were also included appropriately as area or linear weight and mass. Live and partition gravity design loads were defined according to the code.

**3.1.2 Computer analyses.** The models developed were subjected to nonlinear dynamic analyses to account for the nonlinear damper characteristics, which were modeled by ETABS' link elements with three-dimensional properties, including dimensions, weight and mass (given by the damper manufacturers), but only nonlinear axial damping characteristics. All other model elements were considered linear. These analyses were carried out with an approximate Ritz modal decomposition, CQC modal superposition, and 5% of critical damping for all the modes considered. The bracing system characteristics (instantaneous stiffness, viscous damper coefficient, c, and viscous damper velocity exponent,  $\alpha$ ) were fine-tuned to achieve the maximum earthquake load dissipation possible for the maximum level of drift allowable. The fine-tuning was carried out measuring column strength



and drift demand-to-capacity ratios. Once fine-tuning was completed, the structural members that were still overstressed were retrofitted to resist the maximum seismic demand.

**3.1.3 Analysis and design check.** A several-step systematic procedure was followed, as shown in Table 1, for studying the existing building structural dynamic behavior and devising a seismic retrofit configuration to mitigate the deficiencies found. Table 2 shows the procedure followed to analyze the retrofitted structure and finalize the design of the retrofitted building.

Table 1 – Analysis and design check procedure for OB8&9 existing building structure.

### STEPS FOR THE EXISTING CONDITION

- 1. Build a detailed model of the existing building.
- 2. Conduct a linear dynamic analysis on the model.
- 3. Retrieve internal forces for all building structural elements and building drifts for all floors, independently enveloped over all DBE and all MCE loads.
- 4. Post-process the data retrieved in 3.: compute the maximum factored demand-to-capacity ratio,  $\lambda_{max}$ , for all elements.
- 5. Identify all steel elements whose  $\lambda_{max}$  are unacceptable ( $\lambda_{max}$ >1.0); identify building seismic deficiencies.
- 6. Repeat 5. for all vertical reinforced concrete members (basement piers, shear walls, elevator shaft walls) and critical horizontal reinforced concrete slab members (first-story slabs connecting set-back walls with perimeter basement walls and third-story slabs connecting set-back walls to moment frames).
- 7. Devise a seismic mitigation configuration to address the seismic deficiencies found.

Table 2 – Analysis and design check procedure for OB8&9 retrofitted building structure.

### STEPS FOR THE RETROFITTED CONDITION

#### Repeat until Convergence:

- 1. Modify the model to implement the seismic mitigation configuration and architecturally-driven remodeling changes (new stairs, floor fills, and new floor and wall openings).
- 2. Conduct a non-linear dynamic analysis on the model.
- 3. Retrieve internal forces for all steel column elements and building drifts for all floors, independently enveloped over all DBE and all MCE loads.
- 4. Post-process<sup>1</sup> the data retrieved in 3.: compute  $\lambda_{max}$  for all columns; compute the drift factored demand-to-capacity ratio,  $\lambda_d$ , for all floors.
- 5. Modify the damper characteristic properties and repeat 1. to 4. until minimum  $\lambda_{max}$  in all columns is achieved, such that all floor  $\lambda_d$ 's are less or equal to the maximum allowable  $\lambda_d$ .
- 6. Identify all column elements whose  $\lambda_{max}$ >1.0; retrieve the corresponding enveloped internal forces for each earthquake.
- 7. Post-process<sup>1</sup> the data retrieved in 6.: compute the enveloped  $\lambda_{max}$ ,  $(\lambda_{max})_{env}$ , for all columns and each earthquake.
- 8. Identify all column elements and each of the corresponding earthquakes for which  $(\lambda_{max})_{env}$ >1.0; retrieve the internal forces as functions of time for each of these columns.
- 9. Post-process<sup>1</sup> the data retrieved in 8.: compute  $\lambda$  in the time domain,  $\lambda(t)$ , for each column and each earthquake independently.
- 10. Identify all column elements for which  $\lambda_{\max}(t)>1.0$ ; design a retrofit scheme to strengthen these columns at the locations where  $\lambda_{\max}(t)>1.0$ .

#### Once Convergence Is Achieved:

- 11. Retrieve the internal forces for all existing steel members, other than columns (moment-frame beams, main girder beams, floor beams, all other beams, and brace members), independently enveloped over all DBE and all MCE loads.
- 12. Post-process<sup>1</sup> the data retrieved in 11.: compute  $\lambda_{max}$  for all existing steel members other than columns.
- 13. Repeat 6. to 10. for all existing steel members other than columns.
- 14. Retrieve the internal forces for all vertical reinforced concrete members and critical horizontal reinforced concrete slab members, independently enveloped over all DBE and all MCE loads.
- 15. Post-process<sup>2</sup> the data retrieved in 14.: compute  $\lambda_{max}$  for all the reinforced concrete elements.
- 16. Identify all the reinforced concrete elements for which  $\lambda_{max}$ >1.0; design a retrofit scheme to strengthen these elements at the locations where  $\lambda_{max}$ >1.0.
- 17. Retrieve the internal forces for all new steel brace and damper elements, independently enveloped over all DBE and all MCE loads.
- 18. Post-process<sup>1</sup> the data retrieved in 17.: compute brace  $\lambda_{max}$ , and obtain damper design forces and strokes.

<sup>1,2</sup> Notes: To compute  $\lambda_{max}$ ,  $(\lambda_{max})_{env}$  and  $\lambda(t)$ , and implement the FEMA 351 and FEMA 356 acceptance criteria on the massive data generated by the analyses, two software systems were developed to post-process steel<sup>1</sup> and reinforced concrete<sup>2</sup> elements after the basic internal force data were retrieved from the ETABS-generated database. The software can post-process enveloped results and results in the time-domain.



### 3.2 Seismic demand

Following FEMA 356, two gravity loads,  $Q_G$ , were defined

$$Q_G = 1.1(Q_D + \frac{1}{4}Q_L + Q_S); \quad Q_G = 0.9Q_D$$
 (4)

where  $Q_D$ ,  $Q_L$ , and  $Q_S$  (=0) are the dead, unreduced live and effective snow loads, respectively. The gravity loads were combined with the earthquake load,  $Q_E$ , for deformation-,  $Q_{UD}$ , and force-controlled actions,  $Q_{UF}$ ,

$$Q_{UD} = Q_G + Q_E; \quad Q_{UF} = Q_G + Q_E/(C_1 C_2 C_3 J)$$
 (5)

respectively, where  $C_1=1$ ,  $C_2=1$ ,  $C_3=1$ , and where J=1 for components of the lateral-force resisting system, but also conservatively assumed equal to 1.0 for all other components. The load combinations were carried out for the resulting earthquake enveloped actions, *i.e.*,  $\max(Q_E)$  and  $\min(Q_E)$ , separately for each set of earthquake loads, DBE and MCE, as well as for the resulting earthquake time-domain action,  $Q_E(t)$ , for each earthquake load. From the earthquake loads described in Section 2.3, twelve sets of earthquake loads, six DBE and six MCE, were defined by interchanging the horizontal components on each time-history developed.

### 3.3 Seismic (capacity) acceptance criteria

**3.3.1 Existing steel moment-frame joints.** The moment frame joints conserve their integrity if the drift remains bounded within a certain confidence level, CL, for a given uncertainty factor,  $\beta_{UT}$ . The drift demand, *d*, resulting from the load combinations, Eqs. (4) and (5), is measured by,

$$\lambda_d = \gamma \gamma_a d / (\phi C_d) \tag{6}$$

where  $\gamma$  and  $\gamma_a$  are the drift-angle demand-variability and analysis-uncertainty factors, respectively; the drift angle capacity,  $\phi C_d$ , is defined by the resistance factor,  $\phi$ , and the drift capacity,  $C_d$ . Table 3 shows FEMA 351's global demand-to-capacity ratio parameters used.

DRIFT DEMAND	γ	Ya	ø	C, rad	CL	$\beta_{UT}$	$(\lambda_d)_{max}$	
IO* (DBE)	1.60	1.04	0.85	0.01	0.50	0.20	1.060	
CP (MCE)	1.80	1.10	0.60	0.06	0.90	0.55	0.785	
FEMA 351 Table	3-9	3-8	3-10	3-10	3-7	3-11	3-6	
FEMA 351 does not define a LS acceptance criterion, but a more restrictive								
one, Immediate Occ	upancy	(IO), add	pted for	this proj	ect.			

Table 3 – OB8&9 global inter-story drift demand-to-capacity ratio parameters (for a Type 2 connection).

**3.3.2 Existing steel and reinforced concrete members.** FEMA 356 factored demand-to-capacity ratio for deformation- and force-controlled actions respectively are

$$\lambda_{UD} = Q_{UD} / (m \kappa Q_{CE}) \le 1; \quad \lambda_{UF} = Q_{UF} / (\kappa Q_{CL}) \le 1$$
(7)

where  $Q_{CE}$  and  $Q_{CL}$  are the element expected and lower-bound strength capacities, respectively;  $\kappa=1$  is the knowledge factor; and *m* is the demand modifier or acceptance criteria modification factor, *m*-factor hereafter. The *m*-factor ( $\geq 1.0$ ) is used to increase the element capacity, or reduce the demand, to account for ductility on all building components that remain linear under the action of the earthquake loads. All existing building components were considered linear. The capacities were calculated using AISC LRFD [22] in the case of steel members and ACI 318 [24] in the case of concrete members, considering all the AISC's and ACI's resistance factors  $\phi=1$ . These calculations were implemented in the software specifically developed for this project. The strength parameters used were derived directly from FEMA 356 and the existing condition. In the case of reinforced concrete, the stiffness assumptions suggested by FEMA 356 were followed.

**3.3.3 New steel members.** All new steel members were designed under current code standards using AISC LRFD. The new bracing 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 enveloped over all the earthquakes. Furthermore, all HSS-braces in a single floor were designed for the maximum design force occurring on any



brace on that floor. In addition, all elements of the bracing configuration, such as connecting flange, gusset and clevis plates, as well as fasteners and welds, were designed for that design force.

## **3.4 Seismic evaluation of existing structure**

**3.4.1 Three-dimensional structural model and quality control.** The three-dimensional model, shown in Fig. 6, consisted of 9,963 nodes, 13,409 frame elements, and 7,353 shell elements, generating a system of 61,476 simultaneous equations stored in a 512 MB stiffness matrix. Considering the size of the model, a quality control procedure was devised [9]. This procedure allowed verifying the integrity of the model, as it was being developed and modified to accommodate seismic mitigation alternatives and architecturally-driven changes. The model was also revised thoroughly, element by element, at two project stages, after schematic design and after the bracing configuration solution was found to be satisfactory. The total weight of the building generated by the model was checked for each story by independent hand calculations carried out from the as-built drawings.





**3.4.2 Period distribution and mass participation ratios.** The as-built model was analyzed using an 800-mode Ritz decomposition. Significant Ritz modes with mass participating ratios greater or equal than 2% are shown in Table 4. The Ritz period distribution shows: (a) both horizontal directions have similar dynamic characteristics; (b) the first two horizontal modes include strong rocking components (89% of mass participation), implying that the earthquake axial load on the perimeter columns is significant; (c) the motions have strong coupled torsional and horizontal modal components at the same Ritz periods; (d) vertical modes become important at Ritz periods below 0.36 seconds and before 90% of the mass participates in the response; and (e) 90% of mass participation is achieved after the 662<sup>nd</sup> mode for the horizontal degrees of freedom.

**3.4.3 Inter-story drift evaluation.** For the load combinations given by Eqs. (4) and (5), two sets of results were retrieved from the massive data (5,152 MB) generated by each ETABS run. Each set consisted of the resulting envelopes for all six DBE and all six MCE load combinations, the maximum drifts in both horizontal directions, and the total story loads. Using Eq. (6), two  $(\lambda_d)_{max}$  per floor were computed, one for the DBE-enveloped load combination and one for the MCE-enveloped load combination (Fig. 7). Fig. 7 shows that the drift is above the maximum acceptable level defined by FEMA 351.

**3.4.4 Structural steel member seismic evaluation.** The internal forces of all column elements were retrieved at several stations within each column. Two sets of results consisting of the envelopes for all DBE and all MCE load combinations were retrieved and then processed to obtain  $\lambda_{max}$  for each column. Details of the calculations of  $\lambda_{max}$  are presented elsewhere [9]. Fig. 8 shows  $\lambda_{max}$  for the most stressed building columns. The moment-frame corner columns were overstressed over most of the building height, including the second level, where the columns are encased in concrete and covered with concrete precast panels, making any local strengthening expensive. In addition, all of the interior columns were overstressed at the third floor and next to the mid-rise elevator shaft on the 11<sup>th</sup> floor. The latter is due to the combination of a short column effect and the



shearing of a rigid slab at mid-column height. Similarly,  $\lambda_{max}$  for the moment-frame beams (including the perimeter beams supporting the recessed second floor) were obtained. In this case, it was observed that  $\lambda_{max}$  for bending or shear did not exceed 0.67. No other steel member response was studied for the existing building.

MODE	RITZ	MODAL MASS PARTICIPATION (%)				ACCUMULATED MASS PARTICIPATION (%)								
WODE	PERIOD	DISF	PLACEN	IENT	R	ΟΤΑΤΙΟ	DN	DIS	PLACEM	ENT		ROTATION	N	FORM
n	$T_n$ , sec	$(m_{ux})_n$	(m <sub>uy</sub> ) <sub>n</sub>	$(m_{uz})_n$	$(m_{rx})_n$	(m <sub>ry</sub> ) <sub>n</sub>	$(m_{rz})_n$	$\sum_n (m_{ux})_n$	$\sum_n (m_{uy})_n$	$\sum_n (m_{uz})_n$	$\sum_n (m_{rx})_n$	$\sum_{n} (m_{ry})_{n}$	$\sum_n (m_{rz})_n$	
1	4.457	57.76	0.00	0.00	0.00	89.09	0.00	57.76	0.00	0.00	0.00	89.09	0.00	horizontal
2	4.405	0.00	57.62	0.00	89.35	0.00	0.05	57.76	57.62	0.00	89.35	89.09	0.05	horizontal
3	3.027	0.00	0.03	0.00	0.05	0.00	60.09	57.76	57.64	0.00	89.39	89.09	60.14	torsional
4	1.616	8.82	0.31	0.00	0.02	0.53	0.00	66.58	57.96	0.00	89.41	89.62	60.14	horizontal
5	1.615	0.32	8.64	0.00	0.53	0.02	0.00	66.90	66.60	0.00	89.94	89.64	60.14	horizontal
6	1.138	0.00	0.00	0.00	0.00	0.00	9.20	66.90	66.60	0.00	89.94	89.64	69.34	torsional
7	0.943	0.00	3.60	0.00	0.65	0.00	0.00	66.90	70.20	0.00	90.59	89.64	69.34	horizontal
8	0.930	3.68	0.00	0.00	0.00	0.69	0.00	70.58	70.20	0.00	90.59	90.33	69.34	horizontal
11	0.699	0.00	0.00	0.00	0.00	0.00	3.42	70.59	70.20	0.21	90.59	90.34	72.76	torsional
211	0.356	0.00	0.00	21.74	0.00	0.06	0.00	74.76	74.07	28.21	91.75	91.80	76.22	vertical
215	0.345	0.00	0.01	2.31	0.65	0.00	0.00	74.80	74.29	31.53	92.94	91.85	76.31	vertical
218	0.345	0.00	0.00	7.94	0.09	0.06	0.00	74.80	74.30	41.49	93.85	91.92	76.31	vertical
395	0.240	0.00	0.00	3.16	0.00	0.01	0.00	76.65	75.54	55.73	94.71	94.98	78.20	vertical
397	0.238	0.00	0.00	3.33	0.00	0.00	0.03	76.66	75.54	59.60	94.72	94.99	78.23	vertical
605	0.134	2.03	0.01	0.00	0.00	0.09	0.00	86.56	78.93	78.19	96.97	97.38	82.86	horizontal
628	0.119	0.01	4.00	0.02	0.28	0.01	0.04	88.74	85.96	79.37	97.45	97.61	83.98	horizontal
800	0.002	0.00	0.00	0.61	0.00	0.00	0.00	99.99	99.99	99.89	97.91	97.92	90.60	vertical
	Accumula	ited mas	s ≥ 90%	for			<i>n</i> =	662	645	750	7	8	763	

Table 4 – Significant Ritz modes and corresponding participating mass ratios for OB8&9 three-dimensional structural model.

**3.4.5 Reinforced concrete member seismic evaluation.** The internal forces of selected reinforced concrete elements were retrieved. It was observed that the basement L-shaped concrete piers supporting the set-back shear walls were overstressed. Also, a number of tall pier wall segments, the walls at the elevator shaft on the 11<sup>th</sup>, 17<sup>th</sup> and 18<sup>th</sup> floors, and a number of spandrel wall segments were overstressed. The diaphragm connections to the moment-frame beams at the third floor were overstressed. With the exception of the basement piers and the walls at the elevator shafts, the concrete members were not as highly overstressed as the moment-frame steel members. Thus, the focus of the seismic mitigation was placed on reducing the moment frame loads and drifts.

### 3.5 Three-dimensional model results

**3.5.1 Preliminary analysis and model fine-tuning.** The bracing configuration selected was implemented in the three-dimensional model developed and a process of fine-tuning the damper characteristic parameters was carried out. Thus, the model was analyzed for different sets of viscous damping parameters, c- $\alpha$ , to achieve the minimum moment-frame column  $\lambda_{max}$  for a maximum allowable drift ratio. To increase the number of possible damper vendors, two different damper velocity exponents were considered,  $0.30 \le \alpha \le 1.00$  and  $\alpha = 0.15$ . As this exponent approaches 0, the analysis convergence becomes challenging; thus achieving the optimal solution is difficult. Then, one series of analyses were carried out to fine-tune the pair  $c - \alpha$  for  $0.30 \le \alpha \le 1.00$ , and another to fine-tune c for a fixed  $\alpha$ =0.15. There is not a unique solution for any particular bracing configuration; thus, achieving the optimal, most cost-efficient, solution is nearly impossible. However, this fine-tuning process allows choosing a good overall solution, better than all the others discarded in the process. To simultaneously simplify the process and minimize the cost of the dampers, all dampers were considered to have the same characteristic parameters. The interior two columns embedded in the mid-rise elevator shaft concrete structure at the 11<sup>th</sup> floor presented a major obstacle. It was not possible to reduce the stresses on these columns without a high level of damping delivered by highly stiff dampers above and below the concrete or by dislodging the columns completely from the concrete. This latter solution was adopted, and the structural model was modified to reflect this change. No results from this phase are included herein. The final bracing configuration chosen was an outward diagonal pattern with sliding cross braces on the third floor (Fig. 9), preferred over a more commonly-used zigzagging pattern. The sliding cross braces are not connected at the cross point. This configuration reduced the loads on the frame connections, but increased the loads on the columns. This increase



was marginal and affected only two corner columns, a cost-effective resulting effect. In addition, this configuration preserved open bays at corners (favored by supervisors) and centers (where meeting rooms are located).



BERAME ELEVATION Figure 9 - OB8&9 typical perimeter moment-frame structural elevation showing story drift. (b) Column strength. the bracing configuration.

22.7

demand-to-capacity ratio for the retrofitted OB8&9 (brace-damper configuration with  $\alpha$ =0.40). (a) Inter-

strength ratio, function of time, for the most overstressed column (occurring at the 3rd floor) of the retrofitted OB8&9 ( $\alpha$ =0.40). Earthquake load given by: (a) MCEG41; (b) MCEG42.

3.5.2 Final analyses. Once the results of the fine-tuning were considered satisfactory, a detailed process to obtain the demand-to-capacity ratios was carried out for all steel and selected reinforced concrete components of the building. For a unique bracing configuration, two HSS-damper assembly sets were obtained, one for dampers with velocity exponent  $\alpha$ =0.40 and another for  $\alpha$ =0.15. Only the results for  $\alpha$ =0.40 are presented herein. To verify the validity of the non-linear dynamic analyses based on a Ritz modal approximation, a series of analyses were conducted, each for a different number of Ritz modes (400, 600, 800, and 1,200). Key results were tracked using a convergence ratio, defined as

$$u_R = |(value_{current} - value_{previous})/value_{previous}|$$
(8)

It was observed [9] that the first twelve natural periods, the column maximum factored demand-to-capacities, and the damper forces and strokes converged at a rate of less than 1%.

## 4. Seismic Evaluation – Final Results

## 4.1 Inter-story drift evaluation

Fig. 10a shows that the drift is below the maximum acceptable level defined by FEMA 351. Table 5 shows the confidence level implied by the maximum drift calculated, higher than the minimum required. The building's time domain response to one of the strongest MCE loads (MCEG41) was recorded for the moment frame located on gridline F. Figs. 12 show the structural response simulations of the frame on gridline F in the time domain under MCEG41 for the as-built and the retrofitted building. On this time domain response, it is observed that the retrofitted building responds primarily to the first mode and sways in slower motions than the as-built model.



Thus, the damping configuration reduces the higher mode participation, resulting in slower motions, reduced drift, and lower stress levels. The building still has a strong torsional modal response. In the direction where it is maximum, the base shear is 8.48% and 11.80% of the total weight of the building for the DBE and the MCE loads, respectively, significantly higher than that for which the building was designed. This result confirms that the energy going into the building system as a result of the earthquake load remains approximately the same. However, the story shear was reduced substantially above the third floor, especially for the MCE loads where the average difference between the reduced shear and the shear for the existing building is 62%, *i.e.*, close to the level of the difference from the original design earthquake loads and the current code-prescribed loads (Section 2.2). The maximum drift obtained at the 11<sup>th</sup> floor is 0.007837 radians. Thus, creating a 1"-gap between the columns and the concrete walls at the top of the mid-rise elevator shaft slab, enough to decouple the steel and concrete slab.

Table 5 – Confidence level, CL, on the structural performance of the retrofitted OB8&9 (brace-damper configuration with  $\alpha$ =0.40) under the DBE and MCE design loads.

DEMAND	$\beta_{UT}$	$(\lambda_d)_{\max}$	CL
DBE	0.20	0.849	0.863>0.50
MCE	0.55	0.431	0.993>0.90

Table 7 – Bracing configuration characteristics for the retrofitted OB8&9 ( $\alpha$ =0.40).

		COEFFI	CIENTS	CAPACITIES			
BRACE	HSS SECTION	TION VELOCITY DAMPING		FORCE	STROKE		
		α	c, kips/(fps) <sup>α</sup>	P <sub>des</sub> , kips	$\Delta_{des}$ , in		
Single	HSS7×7×5/8	0.40	400	155-275	4.25		
Cross	HSS10×6×5/8	0.40	400	243	3.25		
<b>NOTE:</b> For a viscous damper, $P = cv^{\alpha}$ , such that $P_{des} = c (1.3v_{max})^{\alpha}$ .							

Table 8 – Seismic retrofit design needed to bring OB8&9  $(\alpha=0.40)$  under the seismic criteria adopted.

NEW/RETROFITTED MEMBER	RETROFIT	FLOOR	QUANTITY (per bldg.)
STRUCTURAL NEW:			
HSS-Damper Braces	-	3RD to 17TH	128
Brace/Column-Beam Connections	_	3RD to 17TH	256
STRUCTURAL RETROFIT:			
Piers	brace leg	BASEMENT	6
Elevator Shaft Wall	separate from steel	11TH	2
Elevator Shaft Wall	strengthen	17TH, 18TH	2
Moment-Frame Columns	strengthen at bottom	3RD	4
Collector Beams	add brace	2ND, 3RD, 18TH	8
Panel Support Beam	add brace	ROOF	1
ARCHITECTURAL RETROFIT:			
C-Section Panel Support Beams	strengthen	18TH	16
Precast Panel Supports	strengthen	DECK	192

Table 6	i – Max	kimum fa	ctored	demai	nd-to-	capa	acity	ratios	for
column	biaxial	bending	under	axial	load	for	the	retrofit	ted
OB8&9	(α=0.40	)).							

			ENV	TIME		
LOAD, Q <sub>G</sub>	FLOOR	COLUMN	ALL	EACH EARTH	IQUAKE	DOMAIN
			$\lambda_{\rm max}$	Time History	$(\lambda_{max})_{env}$	$\lambda_{\max}(t)$
	3RD	C35	1.251	MCEG41	1.197	1.071
				MCEG42	1.216	1.043
	4TH	C79	1.120	MCEG41	1.094	0.727
				MCEG42	1.088	0.974
				MCEUC1	1.024	0.820
				MCEUC2	1.085	1.006
(T				MCEYE2	1.029	0.690
.25	3RD	C79-2	1.135	MCEG41	1.111	1.006
+				MCEG42	1.112	0.943
DL				MCEUC1	1.002	0.812
1.1				MCEUC2	1.068	0.984
	3RD	C164	1.377	MCEG41	1.305	1.081
				MCEG42	1.315	1.067
				MCEUC1	1.227	0.960
				MCEUC2	1.253	0.939
				MCEYE1	1.117	0.976
				MCEYE2	1.123	0.999
MAXIMUM	AT:					
Corners			1.377		1.315	1.081
Perimeter Frame			1.236		1.197	1.006
Building Interior			1.181		1.157	0.919
COLUMNS	WITH $\lambda_{max}$	<u>,&gt;1:</u>				. 7
Corners			7		5	2
Perimeter	Frame		32		27	2
Building In	terior		10		10	0

## 4.2 Existing structural steel seismic evaluation

**4.2.1 Column members.** Fig. 10b shows  $\lambda_{max}$  for the most stressed building columns. The column overstress was significantly reduced. In particular, the moment-frame corner column overstress was mostly reduced to the third level. The 2<sup>nd</sup> and 11<sup>th</sup> floor column overstress was eliminated. To further reduce the number of columns in need of retrofit,  $\lambda_{max}$  as a function of time,  $\lambda_{max}(t)$ , was obtained for each earthquake, only for the elements with  $(\lambda_{max})_{env} > 1.0$ . First,  $(\lambda_{max})_{env}$  was obtained for each of the DBE and MCE loads, separately. Then, only for those elements and earthquake loads for which  $(\lambda_{max})_{env}$  remained >1.0,  $\lambda_{max}(t)$  was obtained. After this procedure was completed, it was observed that only two columns were overstressed, as shown in Table 6, which summarizes the procedure results. Fig. 11 shows  $\lambda(t)$  for the most stressed corner column.

4.2.2 Beam and brace members. All beams were assumed in biaxial bending under axial load. Bracing was



considered different in both axes, as appropriate. It was observed that for the moment-frame beams,  $\lambda_{max}$  did not exceed 0.60. Four collector beams framing onto columns embedded at the end of shear walls on the 2<sup>nd</sup>, 3<sup>rd</sup> and 18<sup>th</sup> floors were subjected to lateral-torsional buckling resulting in  $\lambda_{max}$ >1.0. This was mitigated by bracing the collector beams. One beam located at a corner of the observation platform, supporting the precast panels, was overstressed. When  $\lambda_{max}$  for this beam was calculated in the time domain,  $\lambda_{max}(t)$ <1.0. Thus, there was no need to retrofit any existing beams or braces.



Figure 12 – Structural response simulations for the OB8&9 frame on gridline F in the time domain under the strongest threedimensional earthquake load applied (MCEG41). (a) Existing (as-built) building. (b) Retrofitted building.

## 4.3 Existing reinforced concrete seismic evaluation

It was observed that the basement piers supporting the set-back shear walls were no longer overstressed if the full length of the pier legs took the load, which was materialized by bracing the pier leg. The maximum shear on any of these columns was less than 21 kip in any one axis, well below capacity. For all wall segments grouped as piers and spandrels, and diaphragms grouped as section cuts  $\lambda_{max}$  was obtained for the two load combinations resulting from Eq. (4). In general, it was observed that the overstress was reduced to three pier wall segments. A biaxial bending under axial load check was carried out on the concrete section with the actual reinforcing bar configuration. After these additional analyses, it was concluded that only the walls at the elevator shaft on the 17<sup>th</sup> and of strengthening. The overstress on the diaphragm connections to the moment-frame beams at the third floor was also reduced. The capacity of the connection was based on dowels acting in friction; however, the moment-frame beams and contiguous slabs were part of the same pour, *i.e.*, continuous. Therefore, the connections were deemed to be satisfactory.

### 4.4 New steel braces and viscous dampers

The retrofit is based on a bracing configuration consisting of HSS sections with in-line viscous dampers. The brace element model used was a non-linear link element in-line with a frame element. The axial characteristics of the link element were assumed non-linear and variable to carry out the fine-tuning described. For the final, fine-tuned model, the maximum absolute value of axial force within each floor was considered as the design force for all the dampers on that level. Table 7 shows a summary of the main characteristics of this configuration. The HSS-damper assemblies require a connecting flange and connections to the beam column joint. The assemblies were only connected to the beams to streamline the design of the gusset.



## 5. Conclusions

In addition to the bracing configuration on the moment frames, only a few members needed mitigating retrofit designs to increase the overall capacity of the building to withstand the earthquake loads considered. Rather than increasing the overall building strength to resist the upgraded earthquake loads, the capacity of the building to dissipate these loads was increased. Table 8 shows a summary of the new and retrofitted members.

By increasing the level of engineering and analysis, significant direct and indirect cost savings were realized. A novel structural design, built next to the most overstressed members, reduced the seismic demands significantly on those members and the rest of the structure. Thus, Office Buildings 8 and 9 were seismically rehabilitated: (a) conforming to current codes; (b) conforming to the risk level for State-operated non-essential buildings; (c) minimizing direct retrofit construction costs; (d) minimizing retrofit construction work time, further reducing construction costs, by minimizing demolition and implementing a clean and streamlined energy-dissipation-based seismic rehabilitation solution; (e) minimizing seismic strengthening of the existing building structural members; and (f) minimizing architectural changes, resulting in architecture as unobtrusive as possible for the end user. The cost of all the seismic retrofit proposed for both buildings was finally estimated [28] at \$3,345,277, excluding mark-ups, construction phasing, escalation and contingencies; that is, an \$11.9 million savings with respect to the schematic design phase estimate.

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