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INNOVATIVE SEISMIC RETROFIT OF TWO HIGH-RISE BUILDINGS WITH UNIQUE CHALLENGES

Saiful Islam[1], MatthewSkokan[2], Sampson Huang[3]

ABSTRACT

This paper presents case study of two existing high-rise buildings, each with its own unique challenges, where innovative seismic retrofit solutions were used to mitigate the critical deficiencies. The case study buildings include a 14-story concrete residential tower and 14-story suspended floor slab building, both located in California in highly active seismic zones. The buildings were designed and constructed in the 1960's and both buildings were determined to pose a significant collapse hazard in the event of a major earthquake. Each of the buildings with its unique structural characteristics and design and construction constraints required unique seismic retrofit solutions. In one case an exterior solution consisting of Buckling Restrained Brace Frames was used while the other case primarily involved use of fluid viscous dampers at the interior of the building.

Case Study I: 14-Story Nonductile Concrete Tower

The first case study building is a 14-story existing concrete post-tensioned moment frame residential building located in Los Angeles, California. It was constructed around 1972 but was designed per the 1965 edition of the Uniform Building Code. The building has a relatively small rectangular floor plate with plan dimensions of approximately 75 ft x 105. The gravity system of the building consists of a post-tensioned concrete flat slab and the lateral system of the building consists of perimeter post-tensioned concrete moment frames with non-ductile detailing of the beams and columns. The slabs, column, and beams are all constructed using 5,000 psi lightweight concrete. The existing foundation system consists of isolated spread footings.

[1] Managing Principal, Saiful/Bouquet Structural Engineers, 385 E. Colorado Blvd., Suite 200, Pasadena, CA 91101
[2] Vice President, Saiful/Bouquet Structural Engineers, 385 E. Colorado Blvd., Suite 200, Pasadena, CA 91101
[3] Principal, Saiful/Bouquet Structural Engineers, 385 E. Colorado Blvd., Suite 200, Pasadena, CA 91101

Analysis of the existing building identified seismic concerns that needed to be addressed to achieve structural life-safety performance objective. The primary seismic concerns included:

1. *Presence of Non-ductile Post-Tensioned Concrete Moment Frames:* The existing lateral bracing system of the tower consists of perimeter non-ductile concrete post-tensioned moment frames. Pre-1976 concrete moment frame buildings such as this one were detailed and constructed without the proper reinforcing to provide adequate post-yield deformation capacity. The additional compression in these beams provided by the post-tensioning has the effect of further reducing the ductility of the beams.
2. *Excessive Building Deflection*: The results from the nonlinear time history analysis of the existing building showed that the tower will experience an maximum story drift of as much as 2.5% during the design earthquake (475-yr) event. These deformations were considered excessive in light of the "non-ductile" detailing of the building.
3. *Excessive Joint Shear*: Analysis also indicated that joint shear demands at several existing moment frame interior beam-column joints are over permissible joint shear stress limits due to the significant quantity of longitudinal reinforcement provided in the existing beams. While a significant amount of ties were provided in the beam-column joints (#5 bars at 2" spacing), the primary concern was the potential brittle crushing of the concrete in the joint core due to diagonal compression.

The seismic retrofit of the 14-story building was approached as a voluntary seismic upgrade with the following stated objectives: (1) mitigate the major seismic deficiencies in the building, and (2) meet the life safety/collapse prevention intent of the current building code. In addition to addressing the structural seismic deficiencies, the owner required that a seismic retrofit solution be developed that is: (1) aesthetically acceptable (since it is one of the taller buildings in the area), (2) be non-intrusive, (3) minimize interior construction work, and (4) allow open views for the occupants.

In order to meet these objectives, a performance-based design approach was utilized whereby the seismic retrofit design followed a two-stage process:

1. The seismic retrofit scheme was first designed based on linear dynamic response spectrum analysis using the design (475-year) response spectrum developed. An "R" value of 8.0 was used to reduce the force demands on the new elements.
2. The performance of the building system, including the retrofit design, was then verified by nonlinear time history analyses. To meet the intent of the building code, a set of two seismic performance objectives were established: (a) "Life Safety" performance for a Design-Basis (475-yr) earthquake (denoted EQ-III), and (b) "Collapse Prevention" for a Maximum Considered (2475-yr) earthquake (denoted EQ-IV). Table 1 shows the seismic design criteria used on this project.

The proposed seismic upgrade consisted of adding new Buckling Restrained Brace Franes (BRBF's) on the exterior of the building to improve the overall lateral load resisting capability as well as to reduce the building drift. The new exterior BRBF's are located on each side of the existing building as shown in Figure 1a. The beams and columns of the new BRBF frames are constructed of reinforced concrete with steel embed plates which are used to connect the braces to the concrete frame. The new concrete frame is detailed as a moment frame to provide additional redundancy and mechanism for added energy dissipation. The buckling restrained braces used were manufactured by Nippon Steel and had design axial capacities varying anywhere between 230 kips (Type B1) to 700 kips (Type B5).

Table 1 Building Acceptance Criteria for Nonlinear Verification Analyses

Components	EQ-III	EQ-IV
Buckling Restrained Braces		
Axial Strain (From Tests)	2.12% (Type B1) 3.04% (Type B2-B5)	2.12% (Type B1) 3.04% (Type B2-B5)
Cumulative Plastic Ductility (From Tests)	400 (Type B1) 1260 (Type B2-B5)	400 (Type B1) 1260 (Type B2-B5)
Existing Beams Plastic Rotation	0.01 rad.	0.015 rad.
New Beams Plastic Rotation	0.02 rad.	0.025 rad.

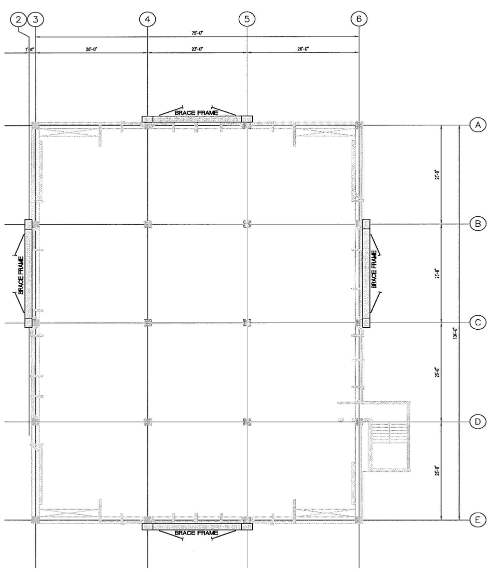


Figure 1 (a) Typical floor plan

Figure 1(b) Building after retrofit

The introduction of the exterior BRBF's effectively addressed the major seismic deficiencies of the existing building. Figure 1b shows the building after retrofit. Stable energy dissipation characteristics and non-degrading stiffness and strength behavior are particularly attractive features of the BRBF system for this building since the existing lateral system is comprised of non-ductile concrete moment frames with poor energy dissipation capacity. The addition of the new exterior concrete beams and columns over the existing moment frame also mitigated the excessive joint shear concern. The use of cast-in-place concrete beams and columns also allowed an easier and more reliable connection (via dowels) to the existing concrete elements of the building. The use of braces minimized obstruction to windows and allowed natural light into the dorm rooms. The exterior application of the BRBF system was very beneficial as it allowed work to be done from the outside without interior of the building.

A three-dimensional nonlinear analysis model of the existing building and retrofit elements was created using the RAM Perform-3D computer program. The beams, columns and basement shear walls of the existing structure and the beams, columns and braces of the new BRBF's were modeled using nonlinear elements. The new and existing concrete beams were modeled using frame elements with moment-rotation hinges at each end. The corner columns which were subjected to significant axial loads and biaxial bending were modeled using frame elements with nonlinear fiber cross-sections at each end. All other columns were modeled using frame elements having moment-rotation hinges with P-M interaction surfaces. The new buckling-restrained braces were modeled using Perform-3D BRBF nonlinear elements.

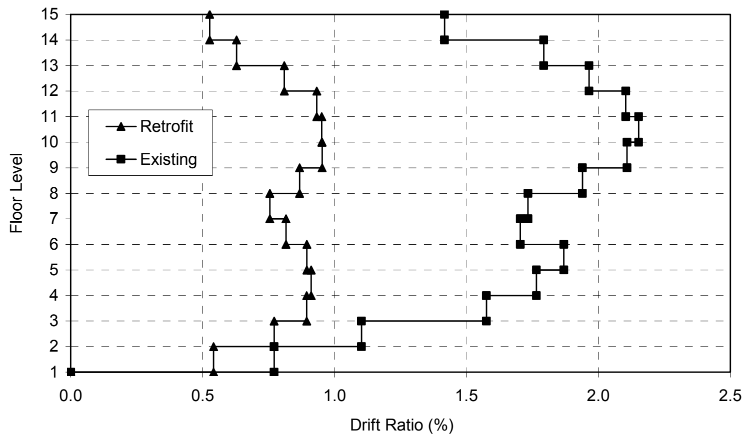


Figure 2 Interstory drift demand in east-west direction for EQ-III (SARAT)

The seismic performance verification of the seismic retrofit scheme was based on a series of nonlinear time history analyses performed using the seven acceleration time history records for the EQ-III (475-year) and EQ-IV (2500-year) seismic hazard levels. The maximum inter-story drift calculated for the EQ-III analyses is approximately 0.8% and that for the EQ-IV analyses approximately 1.3%, which is well within acceptable limits. Figure 2 shows a comparison of the interstory drift demands on the existing and retrofit building in one direction for a EQ-III ground motion time history. The results show the effectiveness of the BRBF in reducing seismic drift (drift reduced by nearly 50%). The BRBF's were found to resist approximately 70% of the total story shear. Table 2 provides a summary of the axial strain, ductility and cumulative plastic ductility demands on the buckling-restrained braces which are well within the acceptance criteria.

Table 2 Buckling restrained brace demands (Average of 7)

Brace Mark	Brace Nominal Capacity (k)	No. of Braces per Frame	Floor Level	EQIII			EQIV		
				Axial Strain	Ductility	CPD	Axial Strain	Ductility	CPD
B1	230	8	11th-Roof	1.06%	9.7	---	1.45%	13.2	---
B2	380	6	8th-11th	1.31%	12.0	---	1.74%	15.8	---
B3	450	6	5th-8th	1.80%	16.5	129	2.49%	22.8	192
B4	570	6	2nd-5th	1.30%	11.9	---	1.92%	17.6	---
B5	700	2	Grnd-2nd	0.79%	7.2	---	1.55%	14.2	---

CPD = Cumulative Plastic Ductility Demand

Case Study II: 11-Story Suspended Floor Building

The second case study building is an 11-story tower built in 1972 in Northern California. This building is one of a handful of lift slab/suspended floor buildings built in the 1960's and early 1970's in California that utilized a very unique structural system. These buildings utilized a patented structural system, which at that time represented an innovative and relatively inexpensive method of construction. The primary structural elements of the system are two reinforced concrete core towers from which steel framed floors with metal deck and concrete are suspended. The floors were assembled on the ground and jacked into place. The floors were attached with shear pins to tapered steel hanger straps that were either supported from steel roof trusses spanning between the cores or were draped over the core walls as is the case on this building. The suspended floors (hung from the straps) were not physically attached to the core walls. There was a gap of approximately four to five inches between the floor and the core. A square steel bumper bar was welded to the floor girders at each corner of the cores to bridge this gap and allow lateral forces generated by the suspended floors to transmit to the supporting concrete core towers. These small replaceable bars were intended to fail in a strong motion thus allowing the floors to sway freely like a pendulum and dissipate energy.

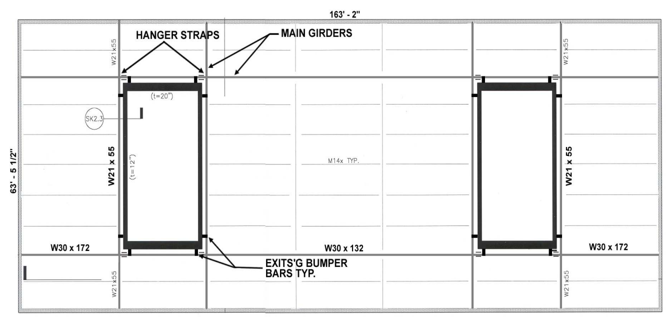


Figure 3. Typical Floor Framing Plan

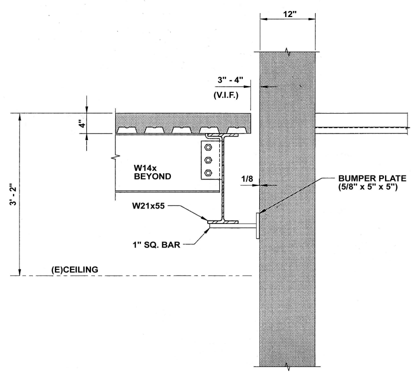


Figure 4. Bumper Bar Detail

The tower is approximately 166'-6" tall. The floors consists of lightweight concrete over metal deck (1-1/2" deck plus 2-1/2" concrete) supported by wide flange beams. Figure 3 shows the typical floor framing of the suspended floors and the location of the hanger straps and bumper bars. Each suspended floor is supported by hangers at eight locations, four at each of the two core walls. The hangers consists of 2-1/2" thick steel plates tapering from 25" at the roof to 10" at the first suspended floor level. At each floor the straps are inserted through slots in the flanges of the main floor member and connected to the main floor beam via 2-3/4" diameter high-strength bolts in double shear. A total of sixteen bumper bars per floor, eight in each direction, bridge the gap between the suspended floors and the core shear walls as shown in Figures 3 & 4. The bumper bars are welded to the bottom flange of the steel beams and are approximately one-inch square and have a 5"x 5"x 5/8" plate at the core wall end. There is approximately 1/16 to 1/8" gap between the bumper plate and the face of the core shear walls. The two core walls support the entire weight of the structure and provide all of its lateral resistance. The rectangular cores measure 20' by 36' with 12" thick walls on the long sides and 20" thick walls on the short sides. In the long direction, the 12" thick core walls accommodate openings at each floor. The foundation system for the core walls consists of a 51' by 60' by 6' thick mat foundation reinforced both at the top and bottom.

The original design intent appears to have relied on these bumpers to prevent significant swinging of the floors during wind and minor to moderate earthquakes. However, in the event of strong earthquake ground shaking, these bumper bars were expected to “break” thus allowing the floors to swing like a pendulum from the top of the cores and dissipate energy in the process.

Analyses Performed

Two levels of site-specific earthquake hazard, Basic Safety Earthquake 1 (BSE-1, 475 yr eq.) and Basic Safety Earthquake 2 (BSE-2, 2475 yr eq.), were used to analyze the building. Three time history sets, each having two orthogonal horizontal components and a vertical component, was developed for each of the two seismic hazard levels. Several models were developed and analyzed to understand and bound the behavior of the building. These included performing pseudo nonlinear time-history analysis of the building (using SAP 2000) with and without soil springs and limit state and deformation analyses of the individual core shear walls.

Figure 5 shows the three-dimensional computer model of the tower. The mass of the building was modeled as discrete lumped masses across the floors to properly account for the vertical ground motion affect, which was an important design consideration. Pseudo nonlinear dynamic time-history analysis was performed using linear elastic material properties of existing as well as new structural elements and nonlinear properties for the element representing the gap between the floors and the core walls, the soil springs and the fluid viscous dampers (for retrofit scheme). Nonlinear gap elements were used to model the gap between the floors and the core walls in the retrofit scheme. The gap element is a compression-only element, which becomes effective only when the floor physically touches the core walls.

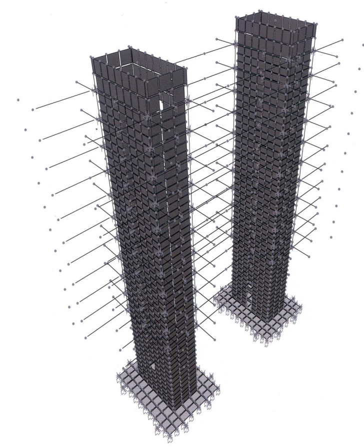


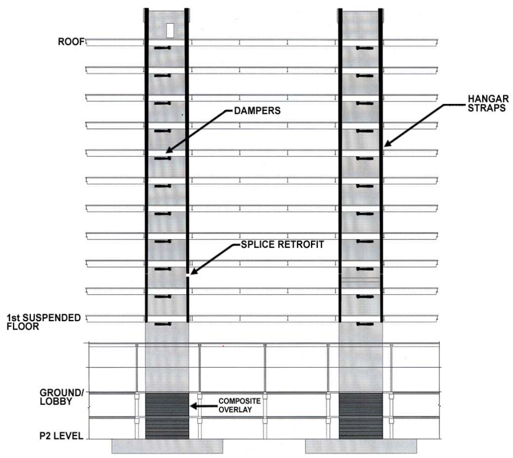
Figure 5. Computer Model

A gap element was modeled at the four corners of the core walls at the location of the bumper bars.

Soil-structure interaction was identified early on in the design process as an important design consideration because of the high aspect ratio of the core walls. This phenomenon has two main consequences: (a) once a part of the mat looses contact with the soil, the compressive stress on the soil can be high and (b) it relieves seismic demands on the super-structure because a part of the lateral sway is caused by the rigid-body motion. In order to analytically capture these phenomena, soil-structure interaction was considered in the model. A finite-element mesh of the mat was created in place of the fixed base with nonlinear compression-only gap elements modeled at each node. An idealized bilinear elasto-plastic load deformation behavior with an ultimate bearing pressure of 40 ksf, a vertical bearing stiffness of 107 pci for the BSE-1 and 70 pci for the BSE-2 earthquake was used per geotechnical engineer’s recommendations.

A detailed analysis of the existing tower identified the following major seismic concerns:

- *Inadequate Force Transfer Mechanism between the Suspended Floors and the Core Shear Walls:* The existing bumper bars were determined to be inadequate to transfer the seismic forces generated during a major seismic disturbance. Once the bumper bars "failed" the suspended floors was going to swing and impact the core shear walls with significant momentum, leading to local and possibly global damage.
- *Inadequate Capacity of the Core Shear Walls:* The existing core walls had inadequate shear and ductility capacity. Since the two core walls not only served as the only lateral resisting element but were also supporting the weight of the entire tower, the structural integrity of the walls was a concern. Strength and stiffness degradation of the walls in regions of moderate to high ductility demand was a critical concern.



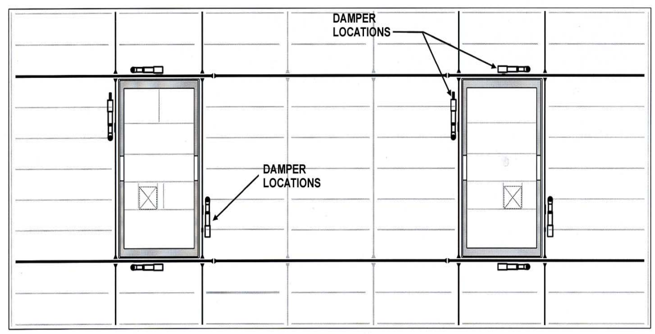


Figure 6. Proposed Retrofit

Figure 7. Floor Plan & Damper Locations

Seismic Retrofit

The seismic strengthening scheme was developed using a performance-based approach. The stated goal for the retrofit was to meet the Life Safety Structural Performance Level for BSE-1 demands. However, the building was also analyzed to see if it could meet the Collapse Prevention Structural Performance Level for BSE-2 demands. Both conventional and non-conventional retrofit schemes were explored at the early stages of the design. The schemes were explored for both fixed-base and flexible-base conditions. The conventional scheme consisted of providing a connection between the floors and the core walls and enhancing the shear capacity of the core walls for eight floor levels using composite overlay and enhancing the flexural capacity using new column pilasters at each corner of the core walls.

The non-conventional scheme finally selected (Figures 6 & 7) consisted of i) removal of all existing bumper bars, ii) addition of fluid viscous dampers at each floor between the suspended floors and the core walls, iii) enhancing the shear strength of the core shear walls using composite fiber overlay at the lowest two parking levels, iv) increasing the flexural capacity of the core wall at the lowest three levels by adding new concrete pilasters to the existing core walls, and v) strengthening of the hanger strap splice connections between the second and third suspended floors. The non-conventional scheme was selected over the conventional retrofit scheme primarily based on cost and tenant disruption considerations. An added benefit of the non-conventional scheme was the superior seismic performance.

In the non-conventional scheme, four viscous dampers were added in each principal direction at each suspended floor level. The dampers were modeled using the SAP2000 damper element with the damper force expressed as $F = CV^{\mu}$, where C = damping coefficient for the device = 30 kip-(sec/in), V = maximum relative velocity between ends of the device, μ = exponent for nonlinear velocity dependent damping device = 0.3. In order to optimize the size of the dampers, detailed parametric studies were performed. Various values of C (10, 20, 30, 40 etc.) and velocity exponent, μ (0.3, 0.4, 0.5, etc.) and combinations thereof were investigated before deciding on the final values.

Rigid versus Flexible Foundation

Analysis of the fixed-base condition indicated that the existing mat foundation supporting the individual core walls would not provide the necessary fixity. Tension piles were considered briefly to provide the fixity but the number of piles and associated foundation work required was quite extensive. Modeling the flexibility of the mat foundation allowed the foundation to rock, thereby reducing the seismic force and ductility demands on the concrete core walls by as much as 60%. Although, this came at the expense of approximately 25 to 30% increase in overall roof displacement of the tower, a significant portion of this displacement was due to the rigid body rotation of the mat foundation and thus did not translate into additional rotational or curvature ductility demand at the base of the walls. Also the number of floors potentially pounding the core walls in the more extreme BSE-2 ground motions was determined to be far less with flexible foundation (no pounding of the floors for BSE-1). Accordingly, it was decided to allow the foundation to rock as long as the soil pressure and deformation was within the acceptable limits.

Core Shear Walls

The results of the pseudo nonlinear time-history indicated that the seismic performance of the core walls will be dictated by flexural yielding at the base of the wall. The shear capacity of the walls, assuming minimal to no degradation, was found to be more than the anticipated demands. The maximum flexural demand-capacity ratio for the BSE-2 earthquake was determined to be less than 2 in the long direction but approximately 2.5 in the short direction of the core walls. Although these values were not considered to be high and were within acceptable limits, given the uncertainties involved with the ground motion, the soil-structure interaction, and the performance of nonconforming lap splices and other reinforcing details, it was decided to increase the flexural capacity of the wall to limit the maximum demand-capacity ratio to less than 2. This was achieved relatively easily by adding two concrete pilasters between the foundation and the mezzanine level on each of the long faces of the core walls. The shear capacity of the walls at these two levels was also enhanced using composite fiber overlay. This was done to mitigate the potential concern with significant degradation of the shear capacity of the existing walls with increase in flexural ductility demand.

A detailed displacement and moment-curvature ductility demand and capacity analysis was also performed for the core walls. The total lateral displacement of the core wall is made up of contributions from three different mechanisms, displacement due to rigid body rotation of the foundation Δ_R (i.e., rocking of the foundation), displacement due to elastic deformation of core wall Δ_E, and post-yield displacement Δ_P. A plastic hinge length of approximately 20' was assumed to calculate the post-yield displacement. For each wall and for each set of time-history analyses, the contribution of each component was calculated along with the maximum curvature and ductility demand at the base of the wall. Only the elastic and post-yield deformation components were considered in ductility demand calculations. Figure 8 shows the moment capacity at the base of the core wall in the short direction of the core walls as a function of the total roof displacement minus the contribution from foundation rocking. For the El Centro BSE-2 earthquake case, the total roof displacement was approximately 45 inches, out of which 11.4 inches was due to foundation rocking. Figure 10 shows that the core wall is expected to yield at roof displacement of approximately 22.7 inches (Δ_R= 11.4", Δ_E= 11.3"). At the maximum roof displacement of 45 inches (Δ_P= 22.4") the displacement ductility demand will be approximately 3 and the curvature ductility demand approximately 5.5. The maximum compressive strain is expected to be approximately 0.0015 in/in, well below the concrete strain limit of 0.003 in/in typically assumed for unconfined concrete.

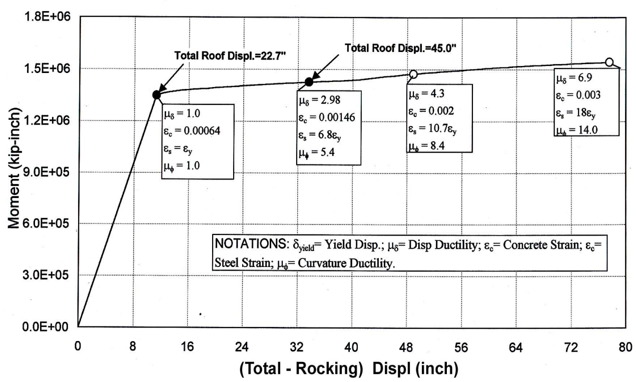


Figure 8. Core Displacement Ductility Curve
El Centro BSE-2

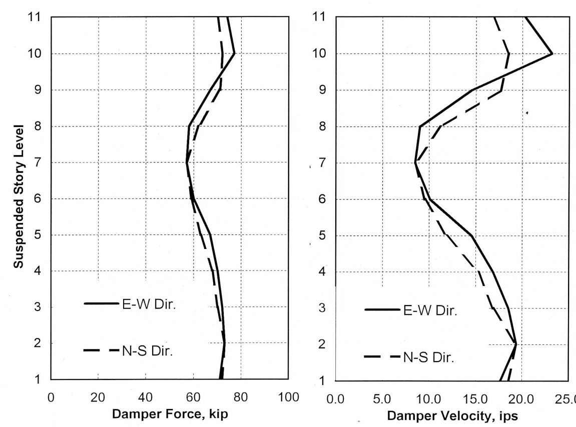


Figure 9. Damper Force & Velocity Envelope
for BSE-2

Damper Design

Figure 9 shows the damper force envelopes for the BSE-2 earthquake. All of the dampers in the building were selected to have the same damper force and damper stroke capacity. Damper stroke was conservatively set equal to 5 inches (since the physical gap between the existing floor and the core walls was on the order of 4 inches) and the maximum damper force capacity was set equal to 100 kips. Given the number of the dampers in this building, per FEMA-356 they had to be designed for forces associated with a velocity equal to 130% of the maximum calculated velocity from BSE-2 earthquake. The components and connections transferring forces to the energy dissipation devices were designed to remain elastic for this increased force.

In order for the proposed retrofit design to work, the existing bumper bars between the suspended slabs and the core walls had to be removed. With the removal of the bumper bars there was a concern that the installed fluid viscous dampers will get activated during regular and frequent wind condition which will damage the dampers. To mitigate this concern a "wind-restraint" feature was devised and incorporated into the design of the dampers. The "wind-restraint" system consists of an external friction mechanism device that prevents the dampers from being activated below a predetermined force, which was set equal to the design wind load prescribed in the 1997 Uniform Building Code. This ensured that the dampers would essentially behave as a rigid element in wind and minor earthquakes and act as fluid viscous dampers during moderate and major earthquakes.

Conclusions

This paper presented case studies of two existing high-rise buildings where innovative applications of seismic retrofit solutions were used to mitigate the seismic deficiencies in the buildings with each building posing unique challenges. Unconventional thinking and approach was the key to finding solutions which was not only cost-effective and practical, but could be implemented within all the constraints imposed by the project requirements.

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