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ANALYSIS AND RETROFIT OF TWO VERY UNIQUE SUSPENDED FLOOR TOWERS USING PERFORMANCE BASED APPROACH

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SUMMARY

This paper describes the analysis performed to predict the seismic performance of two unique suspended floor slab buildings and the seismic retrofit schemes for each building. The two 14-story towers, located in regions of high seismicity, were designed and built in the early 1960's and 1970's. The primary structural element of each tower consists of two reinforced concrete cores that provide the only lateral resistance and support the entire weight of the building. Each core wall is supported by a mat foundation, with the 1960's tower having additional support provided by piles. The floors of the 1960's tower are suspended by means of 24 hanger straps attached to two roof trusses that are in turn supported by the concrete cores. The 1970's tower eliminated the need for the roof trusses by suspending the floors from only 8 straps draped over the concrete cores. Due to the method of construction, a gap is left between the suspended floors and the core walls. In the 1960's tower, the floors were not connected to the core walls, however, in the 1970's tower the gap is bridged by a nominal steel bar welded to the floor girders at each corner of the core, providing the only means for lateral force transfer.

Nonlinear analyses including the effects of soil-structure interaction were performed using site-specific ground motions to evaluate the performance of the lateral and gravity systems. The two buildings were found to perform differently because of differences in detailing, core wall reinforcement, and the presence of the roof trusses and piles in the 1960's tower. A seismic strengthening solution for each tower was developed following a performance-based design approach. One of the towers was unoccupied, with plans to be converted from office use to residential space; therefore, implementing a disruptive retrofit solution was not a concern as long as it was the least expensive. A conventional retrofit solution was selected that structurally connects the suspended floors to the core walls, strengthens selected roof truss members and strengthens the cores at the lower levels. The other tower was fully leased and required a solution that could be implemented with minimal disruption to the office tenants. This was achieved by means of an innovative hi-tech retrofit solution, which introduced 88 fluid viscous dampers installed horizontally to bridge the gap between the suspended floors and the core walls and adding composite overlays to strengthen the core walls at the lower floors.

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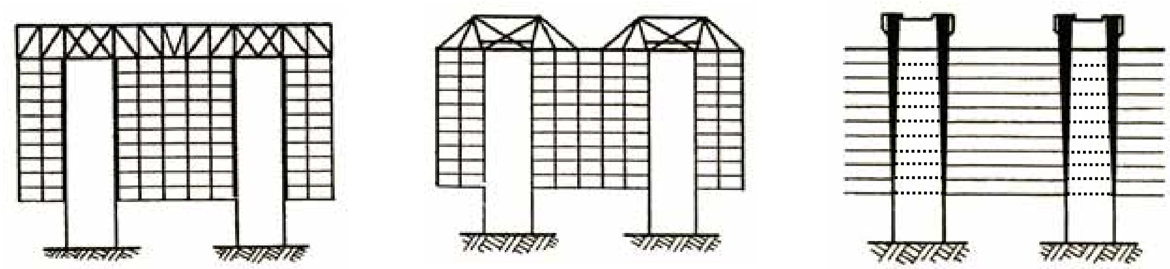
SUSPENDED SLAB SYSTEMS

Several suspended-floor structures were built in California between the late 1960's and the early 1970's. 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 are suspended steel frame floors with metal deck and concrete floor slabs. The suspended slab structural system evolved over time and as shown in Figure 1, there were primarily three versions of the system, each one utilizing a different method for supporting the floors. Floors were assembled on the ground and jacked into place starting at the top of the structure, where they were attached with shear pins to tapered steel hanger straps that were either supported from steel roof trusses, as shown in Figures 1(a) and 1(b), or were draped over the core walls as shown in Figure 1(c). As the suspended slab system evolved, the mechanism for supporting the floor from the concrete cores was simplified and the number of hanger straps was reduced, resulting in a more cost effective, yet less redundant, structural system. The straps allowed the floors to hang down from the cores, but a gap of anywhere from one to five inches was left between the floor and the core. In some buildings, 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. In other buildings, no lateral force transfer mechanism was provided, thus allowing the floors to sway freely even under regular wind excitations.



(a) Large Roof Trusses & 24 hanger straps **(b) Small Roof Trusses & 16 hanger straps** **(c) Concrete Saddles & 8 hanger straps**

Figure 1. Evolution of the Suspended Slab System

This paper describes the analysis performed to predict the seismic performance of two case study buildings and the seismic retrofit schemes for each building. The first case study building, called "Building A" in this paper, uses the least redundant suspended slab system with only 8 hanger straps draped over the concrete cores as shown in Figure 1(c). The second building, called "Building B" in this paper, uses the most redundant of the suspended slab system with 24 hanger straps supported from a large roof truss as shown in Figure 1(a).

BUILDING A

Building Description

The Building A tower, shown in Figure 2, was constructed in 1972 in the bay area of northern California. The building has a total of eleven suspended floors with the first suspended floor located above a two level base structure. The 11-story tower is approximately 166'-6" tall measured from the ground to the top of the main roof. The tower floors consist 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

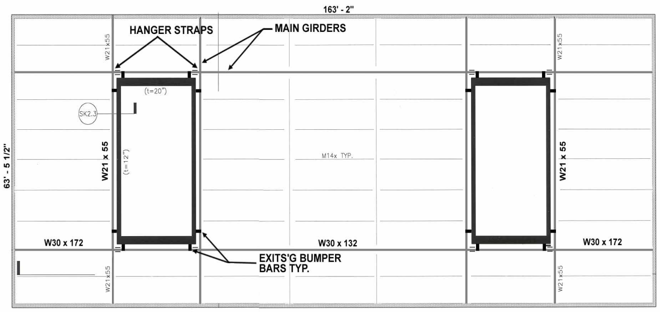


Figure 2. Photo of Building A **Figure 3. Typical Floor Framing Plan for Building A**

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. The straps are connected to steel plates welded to the main floor beam via 2-3/4" diameter high-strength bolts in double shear. The straps are spliced with ten one-quarter inch thick laminated plates, which straddle over the top of the concrete core walls. 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. In this particular building, the gap is ranges between three and five inches.

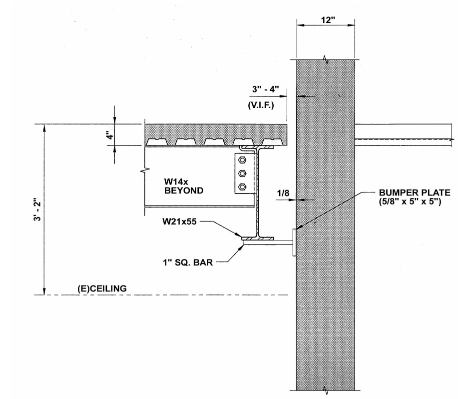


Figure 4. Typical Existing Bumper Bar Detail for Building A

The bumper bars 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 core shear wall. The other end of each bumper bar is welded to the bottom flange of the steel beams. The original design intent appears to be to rely on these bumpers for wind and during minor to moderate ground shaking to prevent significant swinging of the floors. However, in the event of strong ground shaking, these bumper bars are expected to buckle or fracture thus allowing the floors to swing like a pendulum from the top of the cores and dissipate energy in the process. 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 core walls are constructed of 3,000 psi (specified minimum) normal weight concrete. The walls are reinforced with #4 bars and #5 bars at 12" on center each face horizontally for the 12" and 20" thick walls respectively. 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.

Seismic Ground Shaking Hazard

Two levels of site-specific earthquake response spectra, Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2), were established according to the FEMA-356 [1] guidelines. Three sets of acceleration time histories, each set containing two orthogonal horizontal components and a vertical component, were developed for the BSE-1 and BSE-2 seismic hazard levels. These time histories were selected and scaled (in amplitude only) from available strong motion recordings from prior earthquakes. The historical earthquakes selected are the El Centro Array Station 8 record (1979 Imperial Valley Eq.,

M_w 6.6), the Joshua Tree Fire Station record (1992 Landers Eq. M_w 7.4), and the Hollister-South St. & Pine Dr. record (1989 Loma Prieta M_w 7.0). For each pair of horizontal ground motion components, the square root of sum of the squares (SRSS) of the 5-percent damped response spectrum of the scaled horizontal components was constructed. The motions were scaled such that the average value of the SRSS spectra did not fall below 1.3 times the 5% damped spectrum by more than 10% for the applicable range of periods. For Building A, the period range of interest is between 0.2 and 2.4 seconds. Figure 5 shows the BSE-1 and BSE-2 response spectra along with the response spectra of one of the horizontal components of the selected ground motion time histories.

Analyses Models

Several types of analyses were performed to understand and bound the behavior of the building. These include dynamic time-history analysis of the building with and without soil springs (soil-structure interaction) and limit state and deformation analyses of the individual core shear walls. The three-dimensional model of the building was created using the SAP2000 [2] commercial software package. Figure 6 shows the three-dimensional computer model of the tower. The mass of the building was modeled as discrete lumped masses across the floors since the effects of vertical ground motion are considered in the analyses. Dynamic time-history analysis was performed using linear elastic material properties and effective stiffness properties of all existing and new structural elements except that nonlinear elements were used for the gap between the floors and the core walls, the soil springs and the fluid viscous dampers. The pseudo nonlinear dynamic analysis was deemed to be appropriate in this case because the retrofit scheme was developed so as to keep the core shear walls essentially elastic, having a demand-to-capacity ratio less than 2 for the BSE-2 earthquake.

Since it is expected that the bumper bars will buckle or fracture during a moderate to strong ground shaking at the site, the suspended floors will swing, the gap between the floors and the core walls was modeled using a nonlinear gap element. The gap is a compression-only element, which becomes effective only when the floor physically touches the core walls. A gap element was modeled at the four corners of the core walls at the location of the bumper bars.

Four viscous dampers were modeled 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

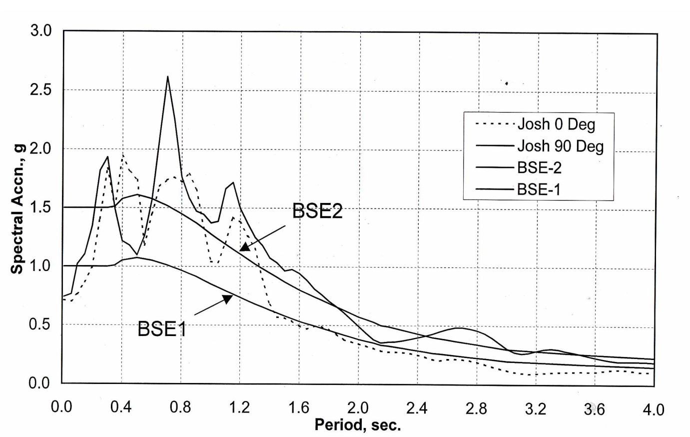


Figure 5. 5% Damped Horizontal Response Spectrum at Building A site

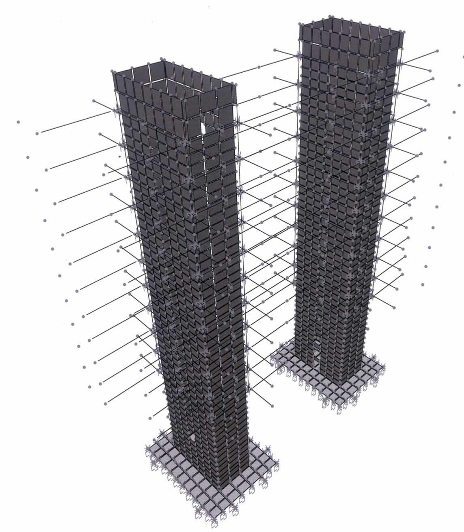


Figure 6. Computer Model for Building A

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.

Given the high aspect ratio of the core walls and the absence of basement restraints, it is clear that some rocking of the foundation is to be expected. 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 response is due to the rigid-body motions. 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 pounds per cubic inch for the BSE-1 earthquake and 70 pounds per cubic inch for the BSE-2 earthquake was suggested by the soils engineer for the analysis. The vertical bearing stiffness parameters were developed using the FEMA-356 method and reflects the anticipated soil degradation during strong ground shaking.

Existing Building Analysis

A detailed analysis of the existing tower identified several major seismic concerns that are expected to result in extensive structural damage and even potential structural collapse during a major seismic disturbance at the site. The primary seismic concerns include the following:

- *Inadequate Force Transfer Mechanism between the Suspended Floors and the Core Shear Walls:* The existing bumper bars are inadequate to transfer the seismic forces generated during a major ground shaking. Once the bumper bars break the suspended floors will 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 have inadequate shear and ductility capacity. Since the two core walls provide not only the entire seismic resistance but also are the only gravity support system for the building, the structural integrity of the core shear walls is a substantial concern. In addition, there is a significant concern about the strength and stiffness degradation of the walls in the regions of high ductility demand.
- *Overturning Resistance of the Foundation*: It is expected that the building will experience some rocking of the mat foundation, however, there is some concern about the stability of the building.

Retrofit Building Analyses

The seismic strengthening scheme was developed using a performance-based approach and was not intended to meet all the prescriptive requirements of the current building code. The "target" goal for rehabilitation is to essentially achieve the Basic Safety Objective (BSO) as defined in the FEMA-356. The scope of the voluntary seismic strengthening for this project, however, was limited to structural elements of the building and did not address the nonstructural elements of the building. In order to achieve the BSO objective, the building must meet the Life Safety Structural Performance Level (S-3) for BSE-1 demands and the Collapse Prevention Structural Level (S-5) for BSE-2 demands. The BSE-1 earthquake is considered to be a major seismic event and the BSE-2 earthquake an extreme seismic event. During the development of the retrofit scheme, it became apparent that given the uncertainties there is a remote possibility that the collapse prevention threshold may not be achievable during a BSE-2 earthquake. Since the BSE-2 earthquake is considered to be an extreme event and that there would be a very large cost increase to eliminate this remote possibility for such an extreme event, it was decided to accept the implied risk provided the building meets the Life Safety Structural Performance for BSE-1 earthquake.

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 shown in Figures 7 & 8 consists of i) fluid viscous dampers at each floor between the suspended floors and the core walls, ii) enhance the shear strength of the core shear walls using composite fiber overlay at the lowest two parking levels, iii) increasing the flexural capacity of the core wall at the lowest three levels by adding new concrete pilasters to the existing core walls, and iv) strengthening of the hanger strap splice connections between second and third suspended floors.

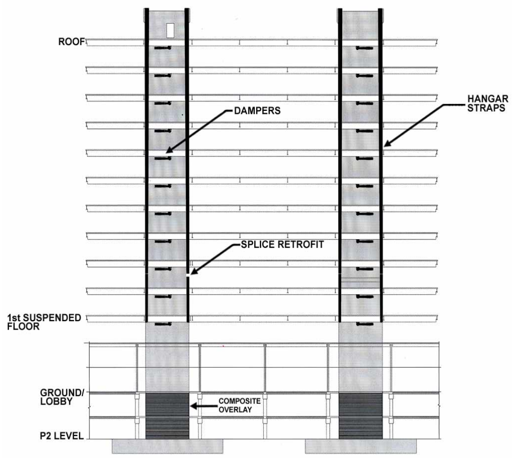


Figure 7. Building A - Core Wall Elevation Proposed Retrofit

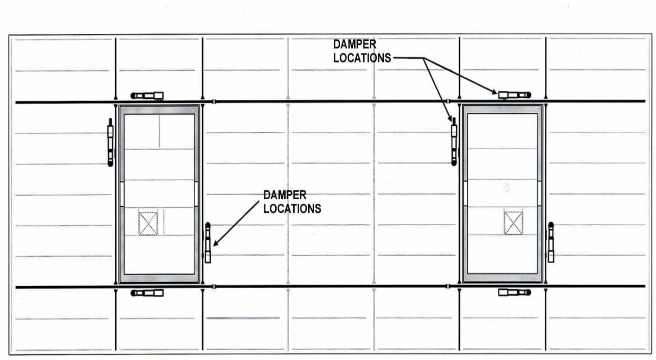


Figure 8. Building A - Floor Plan & Damper Locations

Rigid versus Flexible Foundation
Analysis of the fixed-base condition clearly indicated that the existing mat foundation supporting the individual core walls would not provide the necessary fixity assumed in the fixed-base analysis. 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 comes at the expense of approximately 25 to 30% increase in overall roof displacement of the tower, a significant portion of this displacement is due to the rigid body rotation of the mat foundation and thus does 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 is far less when considering the foundation flexibility (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 given by the soils engineer.

Conventional versus Non-Conventional Retrofit Scheme
Table 1 provides a comparison of some of the key response parameters for both conventional and non-conventional retrofit schemes along with that for the existing building condition. The results, which have been normalized with respect to the existing building condition, are presented for the analysis incorporating the nonlinear soil-structure interaction effects. The existing building case assumes that the existing bumpers have failed and the floors are free to swing within the gap between the core walls and the edge of the slab. As is evident from the results presented in Table 1, both conventional and non-conventional retrofit scheme significantly improves the seismic performance of the tower. The most significant benefit of the retrofit schemes is the reduction in seismic shear demand on the core walls (by as much as 40 to 50%) and essentially eliminating the floor impact concern. It should be noted that for the

passive damping scheme even when the suspended floors do impact, the impact force is only a fraction of that in the existing configuration. The non-conventional scheme was selected over the conventional retrofit scheme primarily based on cost and tenant disruption considerations. The construction cost for the non-conventional retrofit scheme was approximately $12 per sq. ft., while the cost of the conventional scheme was estimated to be approximately $18 per sq. ft. In addition, the conventional retrofit scheme would result in a major disruption to most of the tenant occupied floors, much more than that required for the non-conventional retrofit scheme.

Table 1. Response Parameters Normalized to Existing Building Condition - Building A.

Response Parameters	Existing Building	Conventional Retrofit	Non-Conventional Retrofit
Base Shear	100%	50% ~ 80%	45% ~ 70%
Base Overturning Moment	100%	95% ~ 100%	90% ~ 95%
Roof Displacement	100%	95% ~ 110%	90% ~ 95%
Tension in Hanger Straps	100%	103%	95%
Floor Impact	100%	N/A	BSE 1: 0% BSE 2: 10%

* Range reflects results for different time-history runs and BSE 1 and BSE 2 earthquake.

Core Shear Walls
Figure 9 shows the story shear and moment distribution for the core walls for the flexible foundation analysis in east-west direction i.e., short direction of the core walls. The ground floor and the first suspended floor of the Tower correspond to level 2 and level 5 respectively. The existing shear and flexural capacity of the core walls is also shown on the same figure. The results clearly suggest 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, is larger than the anticipated demands. The flexural demand will exceed the capacity suggesting the potential for plastic hinging at the lower floor levels of the core wall. The maximum flexural demand-capacity ratio for the BSE-2 earthquake is less than 2 in the north-south direction (i.e. long direction of the core walls) but is approximately 2.5 in the east-west direction. Although these values are not considered to be high and are 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.

Systematic 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 need to be considered in ductility demand calculations. Figure 10 shows the moment capacity at the base of the core wall in the east-west direction (i.e., 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, from 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.

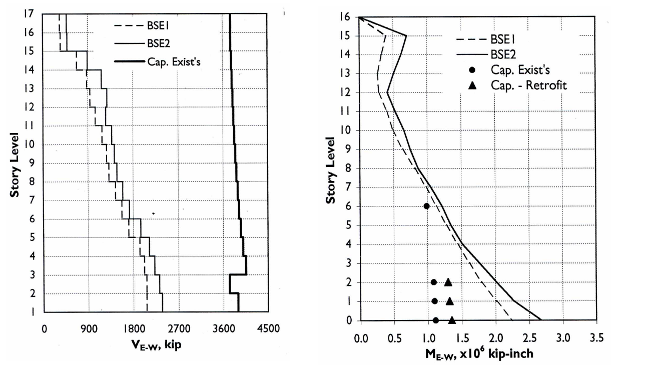


Figure 9. Story Shear & Moment Distribution for core walls – El Centro / Building A

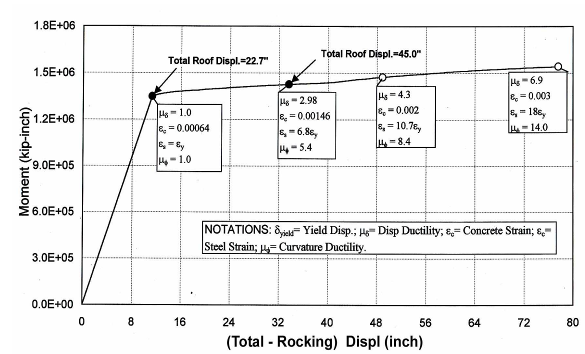


Figure 10. Core Displacement Ductility Curve El Centro BSE-2 / Building A

Damper Design

The design, analysis, quality control, and testing of the fluid-viscous dampers were essentially based on the requirements of FEMA-273 [3], FEMA-356 and NEHRP 2000 [4]. Figure 11 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 gap between the existing floor and the core walls would not permit more than 4 inches of stroke and the maximum damper force capacity was set equal to 100 kips. FEMA-356 requires that if 4 or more energy dissipation devices are provided in a given story in a symmetric fashion then the dampers shall be capable of sustaining the forces associated with a velocity equal to 130% of the maximum calculated velocity for that device from the BSE-2 earthquake. The components and connections transferring forces between the energy dissipation devices shall be designed to remain elastic for this increased force.

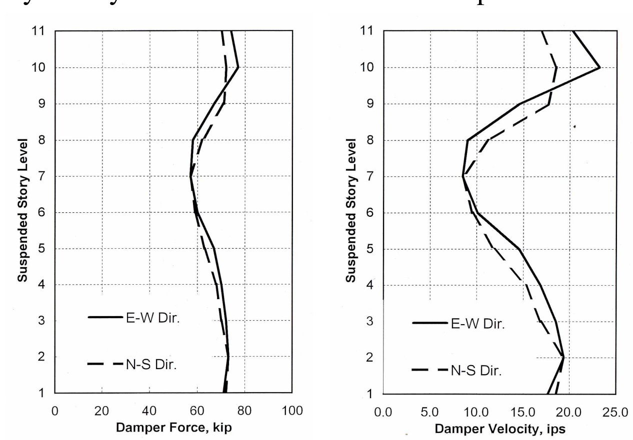


Figure 11. Damper Force & Velocity Envelope for BSE-2 / Building A

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. Once the bumper bars are removed and the fluid viscous dampers are installed, the damper will be continuously activated during regular and frequent wind conditions. 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. The force above which the dampers would be activated was set slightly greater than the wind design load prescribed in the 1997 Uniform Building Code [5]. Below this force limit, the dampers would essentially behave as a rigid element.

BUILDING B

Building Description

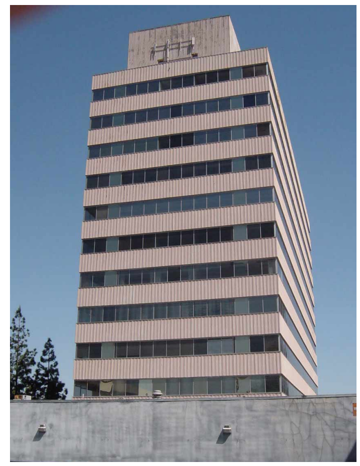


Figure 12. Photo of Building B

The Building B, shown in Figure 12, was constructed in 1962 in the Los Angeles area. The building is an 11-story office building having a total of ten suspended floors. The tower is seismically separated from a low-rise office building and a parking structure. The building is symmetrical with typical floor plan dimensions of 162' by 60'. The tower is approximately 166' tall measured from the foundation to the top of the roof truss. Similar to the floor of Building A, the floors consist of lightweight concrete fill over metal deck supported by wide flange beams and channel members. This building was part of the first generation of suspended slab buildings constructed and as such, hanger straps at 24 locations support the floors. The typical hanger straps are composed of 2" thick steel plates tapering in width from 11" at the roof level to 4-1/8" at the lowest suspended floor level. The hanger straps are connected to a roof truss system that extends the length of the building in the longitudinal direction and is supported by the concrete core walls. The roof truss is comprised of 14" deep steel wide flange members with bolted end connections. The gap between the floors and the core walls is approximately one to two inches for this building and there is no connection in between.

The two core walls that support the entire weight of the structure and provide all of the lateral resistance for the building are the same as the core walls of Building A in terms of dimensions and wall thickness, however, the steel wall reinforcement was substantially greater. The 12" walls were reinforced with #7 vertical bars and #6 horizontal bars, and the 20" walls were reinforced with #10 to #14 vertical bars and #4 horizontal bars. The spacing of the bars varied over the height of the walls between 4" and 10". In addition, 10-#18 bars were added in the corners of the core walls for boundary reinforcement. The foundation for each core walls consists of a 64' by 45' by 7' thick mat foundation with 148 Raymon step-taper piles.

The concrete for the core walls and the mat foundation was specified as 3,000 psi lightweight concrete. A material testing program was developed where concrete cores were sampled from the core walls of the building. The average maximum compressive strength obtained from the concrete cylinder tests was 2,940 psi, slightly below the specified strength. The steel for the roof truss members and the hanger straps was specified as ASTM A-36 steel and was assumed to have a specified yield strength of 40 ksi and an expected strength of 44 ksi (10% overstrength). The steel reinforcement in the concrete walls was specified as ASTM A-15 billet steel and was assumed to have a 45 ksi yield strength.

Seismic Ground Shaking Hazard

Similar to Building A, two levels of site-specific response spectra, BSE-1 and BSE-2, were developed for the project site according to the FEMA-356 guidelines, as shown in Figure 13. In addition, three sets of acceleration time histories, each set containing two orthogonal horizontal components and a vertical component, were developed for earthquake level. These time histories were selected and modified by a spectral matching procedure that modifies the amplitude and frequency content of the time history recordings to match the target response spectrum. The earthquakes were selected from available strong motion recordings from prior earthquakes. The earthquake recordings selected are the El Centro Array Station 7 record (1979 Imperial Valley Eq., M_w 6.6), the Newhall Fire Station recording (1994 Northridge Eq. M_w 6.7), and the Hollister-South St. & Pine Dr. record (1989 Loma Prieta M_w 7.0).

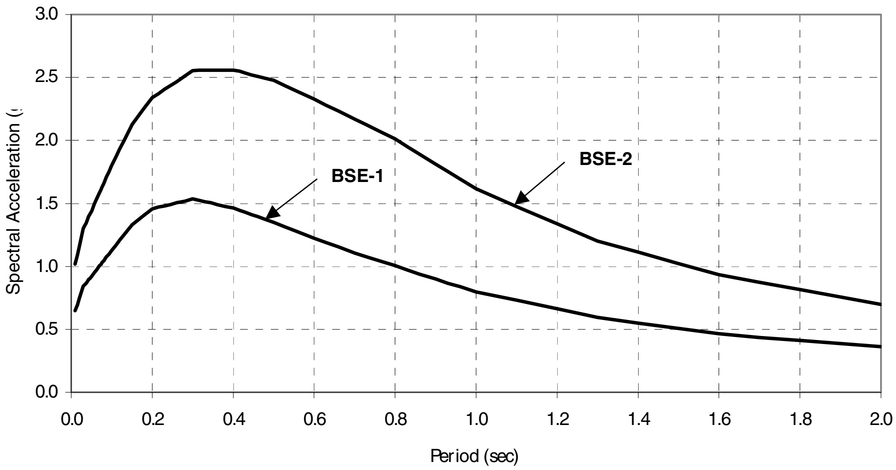


Figure 13. 5% Damped Horizontal Response Spectrum at Building B Site

Existing Building Analysis
Several analyses were performed to understand the behavior of Building B. In one set of analyses, the superstructure was assumed to be essentially elastic, such that the building could be modeled using linear elements having effective stiffness based on the recommendations in the FEMA-356 document. Three-dimensional computer models were created using the SAP2000 computer program as shown in Figure 14. Dynamic time history analyses were performed for the building, where in one case a fixed-base for the core walls was assumed and in another case, the flexibility of the mat foundation system was modeled. The mass of the suspended floors was modeled as distributed discrete lumped masses, such that the effects of vertical ground motions can be properly considered.

The evaluation of the Building B existing structure was based on a target performance goal of meeting the FEMA-356 Life Safety Structural Performance Level (S-3) for the BSE-1 ground motions. Based on the dynamic time history analyses performed, the following aspects of the structural system were evaluated: i) shear and flexural demands on the core walls, ii) shear demands on the coupling beams, iii) axial force demands on the hanger straps, iv) axial force demands on the roof truss members and v) the shear and flexure of the mat foundation. Figure 15 shows the distribution of shear and moment demands on the core walls in the north-south and east-west directions for the Newhall earthquake record and the capacity of the walls considering full development of the steel reinforcement and partial development of the reinforcement due to inadequate lap splices. In

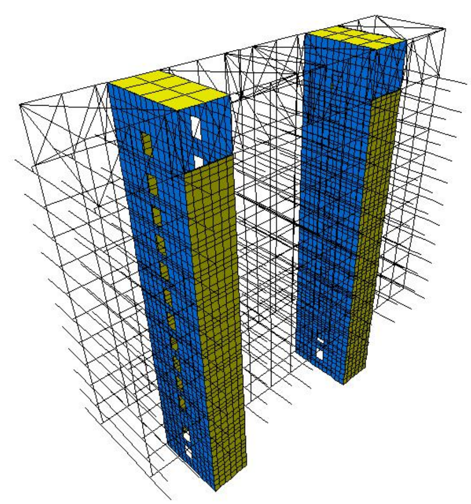
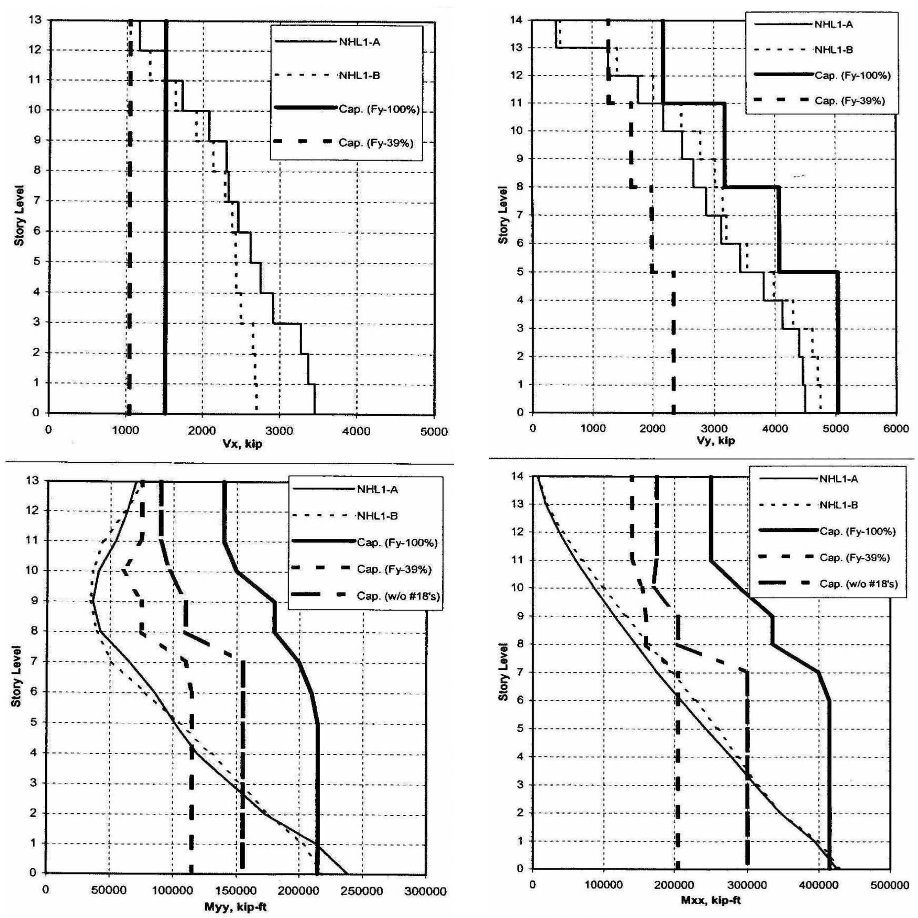


Figure 14. Computer Model for Building B



(a) Short Direction of Core **(b) Long Direction of Core**

Figure 15. Story Shear & Moment Distribution for Core Walls - Newhall / Building B

addition, the flexural capacity of the walls was calculated considering that the #18 boundary bars did not contribute to the strength of the walls since it was obviously clear that a 24 bar diameter lap splice would not be able to develop a #18 bar. In Figure 15, the demands are provided for two different orientations of the earthquake records, 0 degrees denoted as "A" and 90 degrees denoted as "B". Figure 15(a) shows that the short direction of the core walls are clearly governed by shear, with a maximum demand-capacity ratio between 2 and 3, compared to the flexural behavior of the walls where the maximum demand-capacity ratio is between 1 and 1.5. Figure 15(b) shows that the governing limit state is not quite as clear as for the short direction. The maximum demand-capacity ratios for shear and for flexure are both between 1 and 2.

Seismic Retrofit Scheme

From the analysis of the existing building condition, several seismic deficiencies were identified that are concerning from both a life safety and a collapse prevention perspective. The towers were unoccupied and the owner was planning to convert the building from office to residential usage. Therefore, implementing a conventional retrofit solution that would normally be very disruptive to the occupants was not a concern. A non-conventional retrofit scheme incorporating viscous dampers similar to that for Building A was found to be impractical to implement since the gap between the core walls and the suspended floors was only 1" to 2". The primary seismic concerns and the seismic mitigation measures are described in the following. Figure 16 shows an elevation of the building identifying many of the key aspects of the retrofit solution.

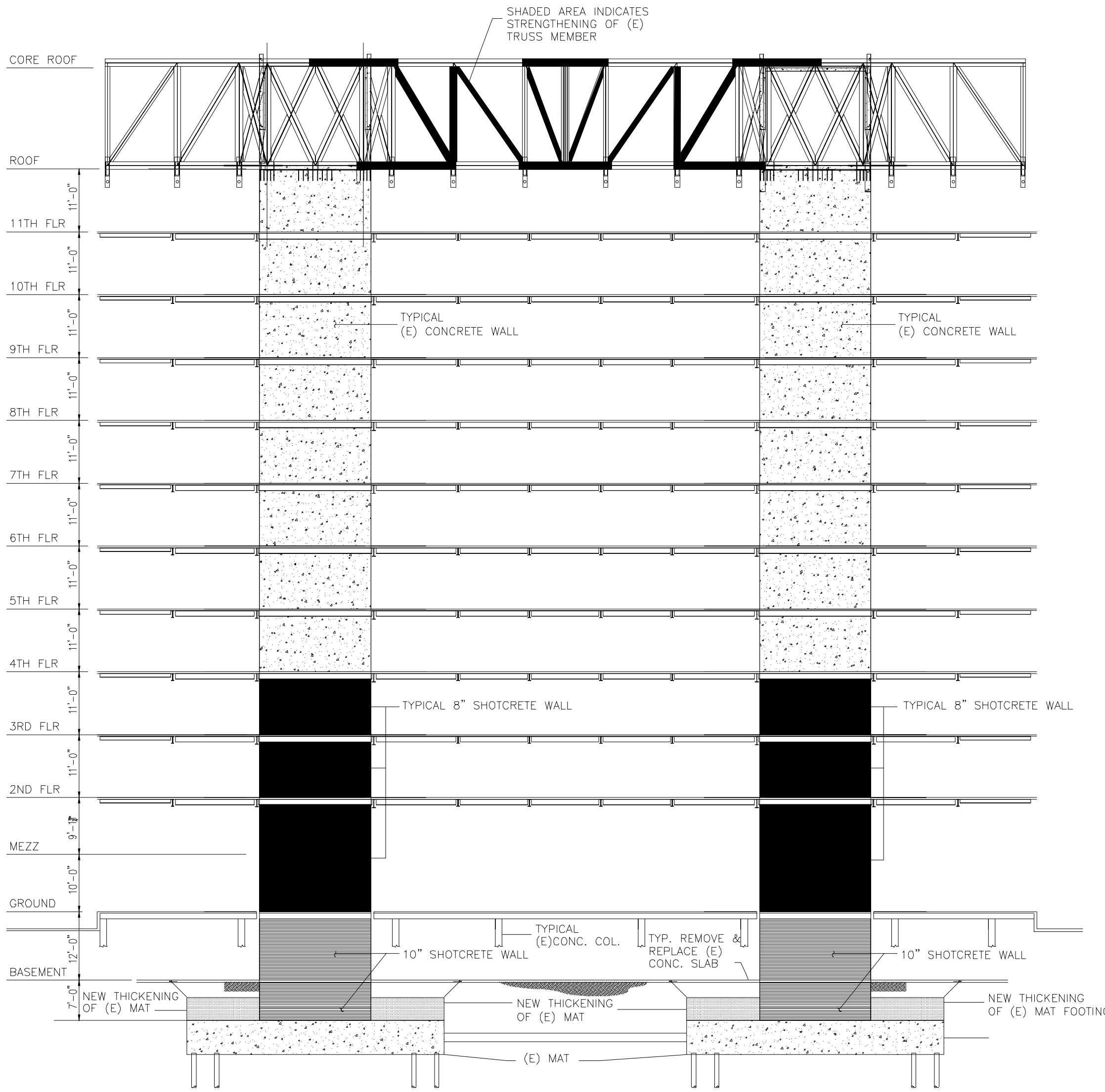


Figure 16. Proposed retrofit scheme for Building B

- *Inadequate Axial Strength of the Roof Truss Members*: The roof truss acts as a cap beam for the core walls in the east-west direction of the building, coupling the behavior of the walls. Therefore, for horizontal seismic motions the roof truss members are stressed due to this coupling effect, in addition to the stresses due to gravity loads from the hanger straps and the seismic forces from vertical ground motion. Several roof truss members have inadequate strength for the gravity and seismic demands as identified in Figure 16. The axial strength of these members will be increased by the addition of plates to the wide-flange truss members.
- *Inadequate Force Transfer Mechanism between the Suspended Floors and the Core Shear Walls:* There is no means for transferring seismic forces from the suspended floors to the core walls and during a major seismic disturbance it is expected that pounding may occur resulting in significant

damage. Plate connections will be provided at each floor connecting the floor framing system to the concrete core walls.

- *Inadequate Capacity of the Core Shear Walls:* The existing core walls have inadequate shear and ductility capacity. Again, as for Building A, since the two core walls provide not the only resistance for seismic and gravity loads, there is a substantial concern for the structural integrity of the core shear walls. In order to strengthen, minimize strength degradation and prevent global collapse of the core walls, 8" to 10" thick shotcrete walls will be added to all four sides of the core walls at the lower levels as shown in Figure 16.
- *Inadequate Capacity of the Foundation*: The existing mat foundation has insufficient strength for the flexural demands imposed by the piles resisting overturning moments on the foundation. New concrete will be added to the existing mat foundations in order to enhance the shear and flexural strength.

CONCLUSIONS

Due to the very unconventional structural system of the two case study buildings presented in this paper, any seismic retrofit solution developed for these buildings must be innovative, effective and cost-sensitive. In the case of Building A, it was also required that the retrofit solution be developed such that the disruption to the occupants be minimal. In addition, it was essential that the analysis technique used to evaluate the existing building condition and the retrofit solution be advanced enough to incorporate the unique features of the building so that the seismic performance of the building could be accurately predicted. The use of sophisticated analysis techniques incorporating the soil-structural interaction effects allowed the design team to analyze the buildings, accurately predict the building response and also to identify the seismic deficiencies in the building.

Building A utilized the least redundant of the suspended floor slab systems and was fully occupied at the time of construction. Since the gap between the suspended floors and the core walls was large enough to accommodate viscous damper elements, the application of the energy dissipation devices appeared to be the most effective and innovative method for mitigating the seismic hazard. The introduction of fluid viscous dampers reduced the seismic demand by as much as 60%. The analysis clearly demonstrated that the soil-structure interaction significantly altered the seismic performance of the tower and significantly benefited the project. Compared to a conventional retrofit scheme, the non-conventional scheme not only provided better performance but also was far less expensive ($12 as opposed to $18 per square feet) and was less disruptive/intrusive.

Building B was one of the earlier suspended floor slab buildings constructed and thus was the most redundant of the building types. The seismic deficiencies of this case study building were mitigated by a conventional retrofit scheme consisting of the application of shortcrete to the lower level of the core walls, the strengthening of the roof truss members and the addition of connections elements to complete the load transfer mechanism. Using viscous dampers in a retrofit scheme similar to that for Building A was not practical since the gap between the suspended floors and the core walls is only 1" to 2". Since the building was unoccupied and slated for complete interior renovation, the proposed conventional retrofit scheme was the least expensive and met the needs of the building owner.

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