Seismic Retrofit of a Landmark Structure Using a Mass Damper

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ABSTRACT

The landmark Theme Building at the Los Angeles International Airport was evaluated for seismic loading. This structure is comprised of a reinforced concrete annular core and four steel arches placed at 90-degree orientations. Performance based engineering was used to assess the seismic performance of the concrete core. A detailed mathematical model of the structure was analyzed using site-specific acceleration histories. Analysis showed that the concrete core had insufficient flexural and shear capacity to resist the seismic loading. A comprehensive retrofit strategy consisting of increasing the capacity and lowering the demand was employed. The focus of the seismic retrofit is a tuned mass damper placed at the roof of the structure to reduce the seismic demand. Additional strengthening for flexure and shear are also incorporated in design. The retrofitted structure met its performance goal. Confidence analyses were conducted to assess the probability of reaching non-ductile limit states.

INTRODUCTION

The iconic Theme Building at the Los Angeles International airport is a well known structure featured in many movies; see Figure 1. A comprehensive investigation was undertaken to assess the performance of the structure and its components to dynamic loading. Wind and seismic loads were considered. The concrete core, supporting exterior walls, floor slabs, steel supporting arches, and wind stability cables were investigated and retrofitted as necessary. The evaluation and the ensuing voluntary seismic upgrade of the main part of the structure are presented in this paper.

DESCRIPTION OF THE BUILDING

The LAX Theme Building is a landmark structure at the Los Angeles International Airport. The building was constructed in 1959. The structure is comprised of a concrete core and a system of steel arches. Figure 2 presents the elevation view of the building showing the concrete core and the steel arches. The overall height of the structure is 144 ft extending from ground, at elevation 90.5 ft, to the apex of the arches at elevation 234.5 ft.

FIGURE 1 - PHOTOGRAPH OF THE BUILDING

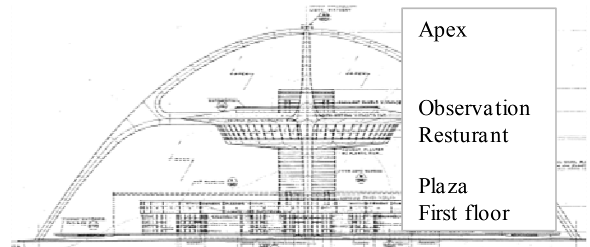

FIGURE 2 - ELEVATION VIEW OF THE THEME BUILDING

The Concrete Core

The concrete core is approximately 108 ft tall and extends from base to the roof at elevation 198 ft. Access to the building is provided at the first floor at 107.5 ft and at plaza level at 124 ft. At the elevation 167 ft, there is the restaurant and entertainment area. The observation slab is located at elevation 179.5 ft. The concrete core is connected to the four arches at the observation level. Figure 3 presents the section cut of the concrete core. The first floor and plaza slabs are connected to the concrete core using a system of steel angles and slotted bolts. These floors are supported by an extensive number of independent concrete walls. Only limited seismic mass from these levels is transferred to the concrete core.

The concrete core consists of a 17-ft diameter annular wall and a system of internal rectangular walls; see Figure 4. At the base, the core thickness is 16 in. and reduced to 12 in. at the first floor. The core is supported by a mat foundation and a system of 128 steel H piles.

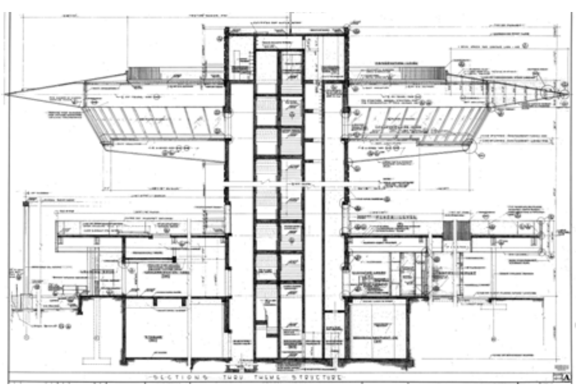

FIGURE 3 - ELEVATION VIEW OF THE CONCRETE CORE

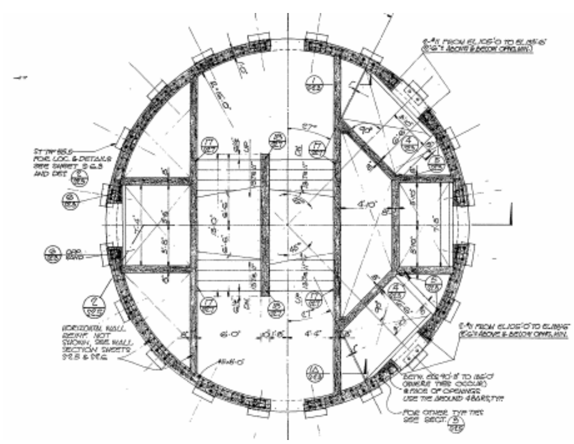

FIGURE 4 - CROSS SECTION OF CONCRETE CORE

The annular wall is the main component resisting the seismic forces. Structural drawings specify 4-ksi normal weight concrete (NWC) up to elevation of 140 ft and 3-ksi lightweight concrete (LWC) above. Longitudinal reinforcement consists of two curtains of #11 bars up to 135 ft, reduced to #9 bars to 170 ft, and changed to #5 bars above. Longitudinal reinforcement is spaced at 12.5 in. on center. Typical splice of longitudinal reinforcement is 25 times the bar diameter. Splices occur at the foundation, first floor, plaza, at 135 ft, and at 170 ft. Transverse reinforcement are #5 bars at 24 in on center. There are two hoops at each location, and the hoops have an overlap of 25 times the bar diameter. Additional trim reinforcement is provided at the elevator or staircase openings. Typical cove for reinforcement is specified at two in. To allow access, openings were cut into the annular wall at various elevations. These openings are not symmetric with respect to the principal orthogonal axis of the core and as such could affect the flexural and shear capacity of the core.

CONDITION ASSESSMENT OF THE MAIN BUILDING

Geotechnical Investigation

Using the available site condition, past seismic events, and active faults that could produce large motions at the site, site-specific response spectra were prepared [Van Beveren & Butelo, 2007] and peer reviewed [JP Singh, 2007]. The (Design Earthquake or 475-year event) DE spectrum is anchored at 0.4g and has a peak spectral acceleration of 0.92g. The spectral peaks are similar to the values computed using the ASCE-SEI 7-05 [ASCE, 2005] procedure based on the mapped acceleration of the USGS web site [USGS, 2007]. The fault-normal (FN) and fault-parallel (FP) components have similar spectral amplitudes up to periods of 2-2.5 sec. Three pairs of spectrum-compatible motions were developed based on the seeds from past earthquakes of similar magnitude and site condition. Recorded earthquake acceleration traces were synthesized such that their response spectra closely matched the target spectra. The computed response spectrum (average of FN and FP components) and the target spectrum are shown in Figure 5.

Material Testing

Comprehensive material tests of the structural components were conducted [Twining Laboratories, 2007]. The testing comprised of sampling concrete cores, reinforcement coupons, and reinforcement splices. ASCE-SEI 41-06 [ASCE, 2006] requirements for comprehensive testing were followed. 37 concrete cores were extracted and tested. The annular wall has compressive strengths of 5.0 and 4.6 ksi; for locations were 4.0 and 3.0 ksi nominal values were respectively specified. The measured splice lengths equaled to or exceeded the nominal values and reinforcement coupons had an average yield and tensile strengths of 50 and 75 ksi, respectively.

Dynamic Field Tests

Field tests were conducted by the University of California at Los Angles [Nigbor and Wallace, 2007] to determine the dynamic properties of the structure. This data was then used to verify the accuracy of the mathematical model of the building and to design the building retrofit. Field tests consisted of ambient vibration surveys and forced vibration (sine sweep and sine hold) tests. For the force-vibration tests, a concrete pad was cast and anchored at the observation level. Two 10-kip capacity shakers were used. The shakers were placed and ran to excite the core mode shapes in both of the lateral and torsional directions. The structure was subjected to low amplitude sinusoidal loading and the acceleration data was collected using 51 accelerometers. The data was collected in the time domain. It was transferred to the frequency domain and the building frequencies and mode shapes were computed. This core has a frequency of approximately 2.5 Hz. The fundamental core mode is presented in Figure 6. Note that due to presence of more flexible LWC at the upper levels, the fundamental mode deviates from the typical cantilever mode shape observed in concrete towers constructed of similar material through the height.

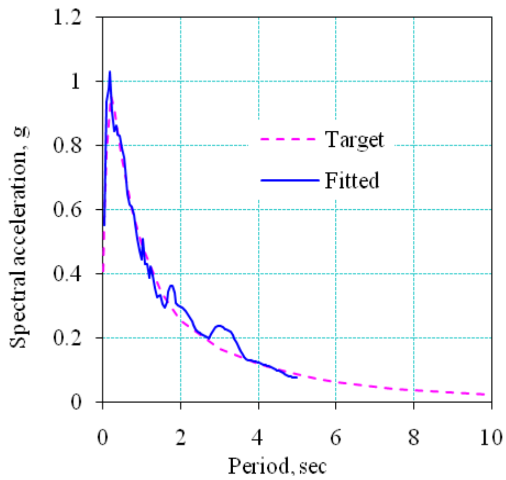

FIGURE5 - SITE-SPECIFIC TARGET, AND COMPUTED RESPONSE SPECTRA

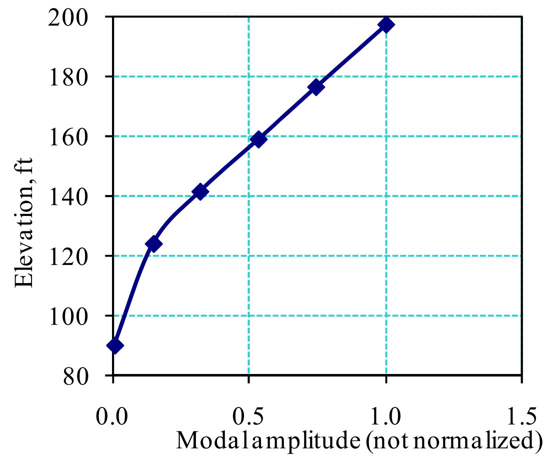

FIGURE 6 - FUNDAMENTAL MODE OF THE CONCRETE CORE

STRUCTURAL EVALUATION

ASCE-SEI 41-06 guidelines were used to assess the seismic performance of the building and to evaluate the effectiveness of the proposed retrofit. The nonlinear dynamic procedure (NDP) was used. Three-dimensional mathematical models of the structure were prepared and were subjected to site-specific motions. The flexural and shear demand were extracted from analysis and compared with computed capacity of the complex cross section at critical elevations.

The performance objective for this structure was selected to be collapse prevention (CP) for the design earthquake (DE). This is the event with a recurrence interval of 475 years (10% probability of exceedance in 50 years). The excepted level of damage corresponds to major flexural and shear cracks, failure around openings, and large permanent drifts. This performance criterion is lower than is usually used. This selection reflected the voluntary nature of the retrofit. More importantly, although, the Theme Building is a unique structure, it is by no means the most important structure for this facility. The recourse is more appropriately earmarked for evaluation and retrofit of more essential structure such as the terminals and the air control tower.

To ensure acceptable performance, non-ductile modes of failure were checked and mitigated. Typical of this vintage concrete building, this structure has poor reinforcement detailing that does not meet the current requirements to ensure ductile behavior.

The existing reinforcement has insufficient splice length. All splices have a nominal splice length of 25 times the bar diameter. The splice lengths were inadequate. ACI 318 [ACI, 2008a] requires much larger splices for bigger bars embedded in light weight concrete. This mode of failure was eliminated by retrofitting the splice locations or by using reduced yield strength for the reinforcement.

ACI 371 [ACI, 2008b] was used to compute the shear capacity of the concrete core. The concrete core has limited shear capacity because of two factors. The openings in the core wall disturb the shear flow path and hence significantly reduce the available shear capacity. Since most openings were oriented parallel to one of the core's principal directions, the shear capacity in that direction was significantly less than the flow in the orthogonal directions. Furthermore, when LWC was used for all the walls, ACI 318 specifies a reduction of 0.75 for shear capacity calculations with LWC. In addition, the shear capacity of transverse reinforcement had insufficient splice length and low volumetric ratio. This mode of failure was accounted for by either increasing the shear capacity at critical elevations or by reducing the seismic demand.

CAPACITY CALCULATIONS

The reduced yield strength, as a function of provided splice length and ACI required development length, was computed per ASCE –SEI 41-06; the software program xSection [Mahan, 2007] was used to compute the flexural capacity of the concrete core at various elevations. The cross section was modeled using fiber elements. Figure 7 presents the analysis results for a typical elevation with openings. The compressive area (shown in black) is shown at the top of the core section. The strain corresponding to the compressive strength was set at 0.002. Typical moment-curvature results are presented in Figure 8.

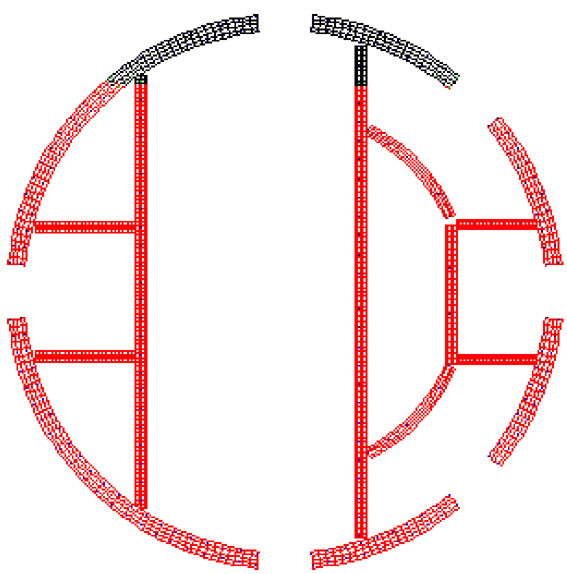

FIGURE 7 - FIBER MODEL

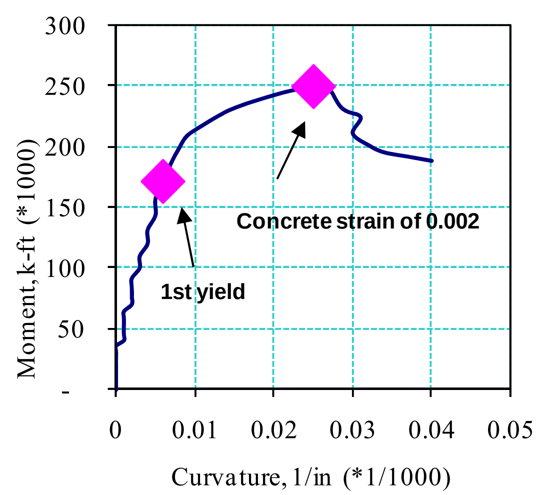

FIGURE 8 - MOMENT CURVATURE RELATION

MATHEMATICAL MODELS

Overview

Computer program SAP (version 11) [CSI, 2008] was used to prepare mathematical models of the structure. All pertinent mass and stiffness components were incorporated in the models. Two models were used in analyses. Mode 1 was a three-dimensional stick (frame) model used for design. The section properties were based on that of the concrete core and interior walls. Model 2, see Figure 9, was a comprehensive and detailed (shell and frame element) model used for final verification.

The use of model 1 for design is justified since the concrete core is much stiffer than the steel arches and hence it governs the response. Since the models have the same seismic mass (5,500 kips) and fundamental concrete core frequency (2.5 Hz); they can be considered to be dynamically equivalent.

The computed core mode shape from analytical models and the measured mode shape from field tests are presented in Figure 10. Note that the analytical model closely tracks the field measured fundamental mode shape. It is noted that the field data is obtained at low amplitude of excitation when cracking of the concrete core is not significant. Furthermore, at the time of tests, construction scaffolding was in place. These factors were accounted for in the mathematical model by specifying a large (0.8) cracked factor (Icr/Ig) and incorporating the mass of the scaffolding.

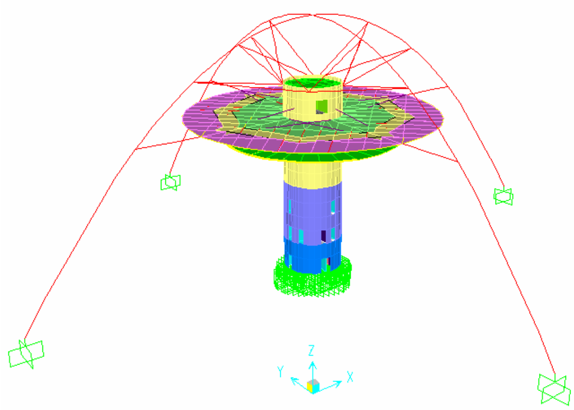

FIGURE 9 - MODEL 2

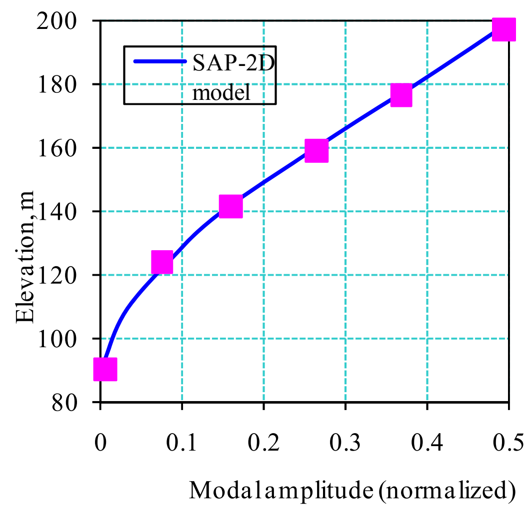

FIGURE 10 - FUNDEMENTAL MODE SHAPE

PERFORMANCE OF THE EXISTING BUILDING

Figure 11 presents the distribution of shear demand and capacity along the height of the concrete core. The shear demands were computed from response history analyses. The shear capacity was computed based on the ACI provisions. The shear demands exceeded capacity along most of the height of the core and thus the building will not be able to withstand the DE event. Figure 12 presents the distribution of bending moment demand and capacity along the height of the core. The flexural demands were the computed from analysis and the capacity was derived from xSection analyses. The flexural demands exceeded capacity in the bottom half of the building.

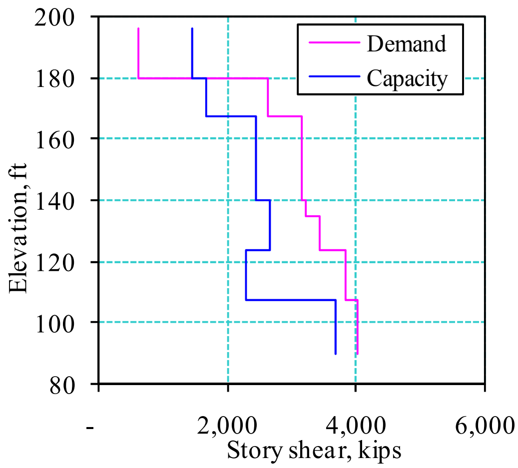

FIGURE 11 - SHEAR PROFILE

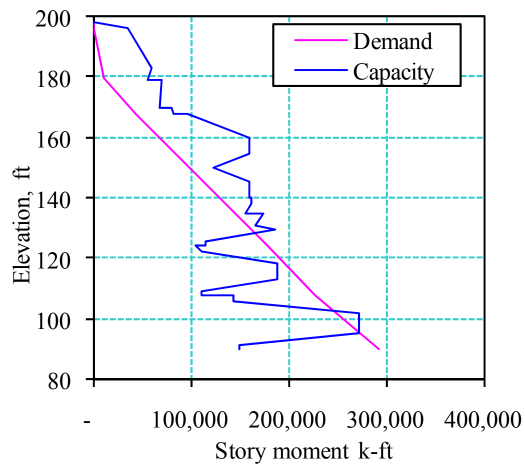

FIGURE 12 - MOMENT PROFILE

SEISMIC RETROFIT

Overview

Both conventional and innovative seismic retrofits were investigated. The conventional retrofit of the building would consist of adding a layer of concrete to the outside core of the structure to increase the flexural and shear capacity of the core. The innovative retrofit consists of adding a tuned mass damper (TMD) to the top of the core. The TMD option was selected because it was less expensive, protected the building's architectural features, and minimized building closure.

The addition of TMD will alter the fundamental mode of the concrete core by introducing two modes. In one, the TMD is in-phase with the concrete core, whereas, in another mode, the TMD motion is out-of-phase with the concrete core. As a result, most of seismic motion is taken up by the TMD and reducing drifts and seismic demand of the concrete core. A high-damped TMD with a mass ratio (defined as mass of TMD to the concrete core) of 20% was selected. This large mass corresponds to 25% of the mass in the fundamental mode and was selected to get approximately 30-40% reduction in the responses.

TMD Properties

Consider the concrete core and the TMD mass attached to a SDOF system by elastic and viscous elements. The result is a couple, 2-DOF system. Since the damping matrix is not mass or stiffness proportional. The resulting eigen value problem would have two complex mode shapes. The coupled equation of motion can be written as:

$$M\ddot{u} + C\dot{u} + Ku = p(t) - md(\ddot{u} + \ddot{u}_d) \tag{1}$$

Therefore, the TMD mass serves to reduce the applied loading. For MDOF systems, the structure is approximated by a generalized SDOF system whose modal properties are that of the fundamental mode of the structure. For application, the fundamental mode is normalized to have unit participation.

For seismic excitation, when many input frequencies are present, the optimal TMD properties are obtained from numerical analysis. One should note that optimizing one response quantity will not necessarily optimize other responses. Sadek et al [1997] optimized the TMD properties by equating the modal damping ratio in the two complex conjugate modes. Randall et al [1981] all developed optimization equations based on numerical simulations for SDOF systems to select TMD properties. Villaverde [2002] has studied multistory buildings retrofitted with tuned mass dampers. The author has examined both analytical simulations and shake table tests. Most of the emphasis was on the TMD systems with smaller mass ratios. Results similar to the other references were obtained.

LAX Theme Building TMD

The existing structure produces a complicated system for TMD optimization. Since the structure is lighter and more flexible over its top half, due to the LWC core, wall, and slabs, its fundamental modal mass is only approximately 68% of total mass. Additionally, this structure differs from a typical multi-story structure. Consequently, the TMD properties were initially selected based on the values suggested by the previous researches. However, the properties were further optimized for this specific structure by conducting a comprehensive analysis simulation program.

The TMD will be mounted at the top of the core; see Figure 13. A concrete slab will be placed and the core walls will be extended to accommodate the TMD; see Figure 14. The TMD mass will be supplied by a system of steel plates, The TMD will weigh approximately 1200 kips. Eight lead rubber bearings will be used to supply the TMD stiffness. The TMD damping will be provided by eight fluid viscous dampers.

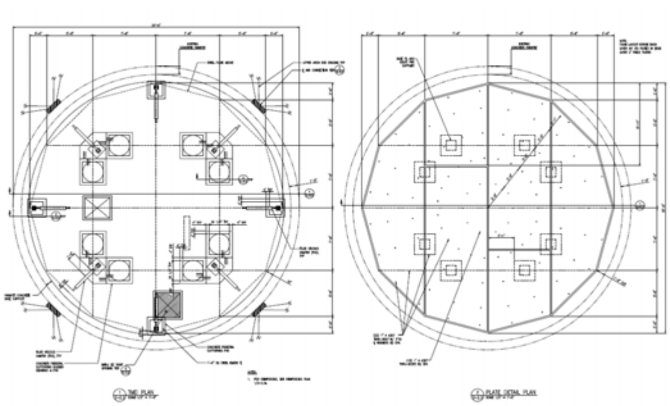

FIGURE 13 - PLAN VIEW OF THE TMD

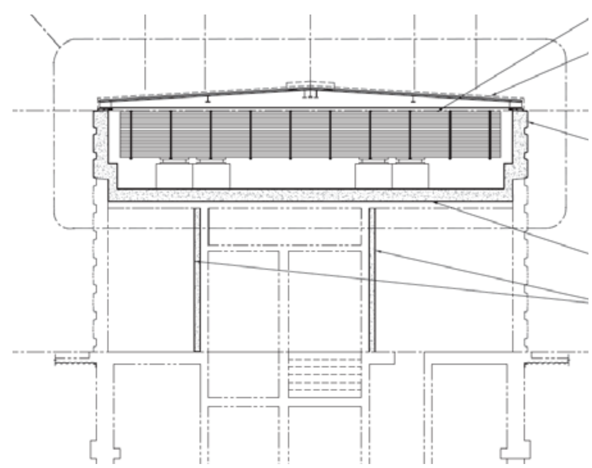

FIGURE 14 - ELEVATION VIEW OF TMD

Production tests of the rubber bearings and viscous dampers have been completed. Shown in Figure 15 is the force-displacement response of a typical bearing [Dynamic Isolation Systems, 2007]. Figure 16 presents the force-displacement response of a damper [Taylor Devices, 2007].

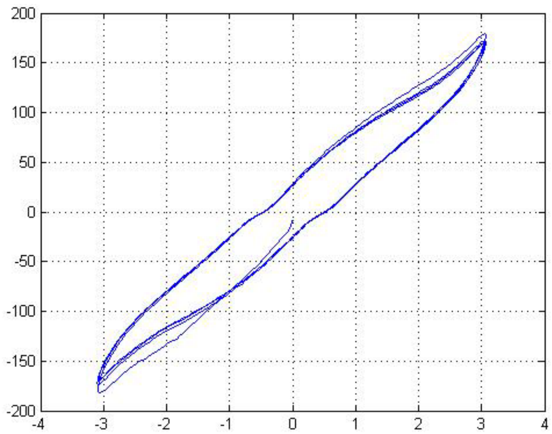

FIGURE 15 - FORCE DISPLACEMENT RESPONSE OF BEARING

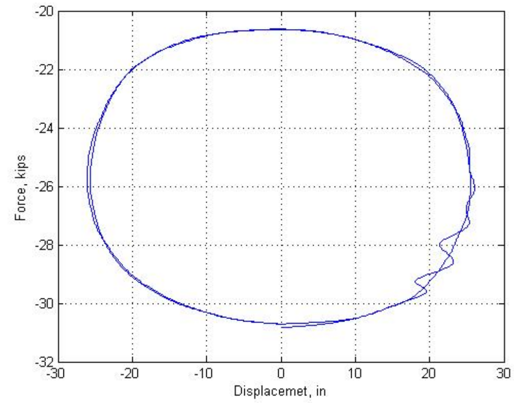

FIGURE 16 - FORCE-DISPLACEMENT RESPONSE OF DAMPER

Retrofit of Lap Splices

The reinforcement splices at the three lowest elevations were retrofitted by providing additional confinement. ACI 318 development length depends on the confinement. Such confinement can be provided by drilling holes, pre-compression the cross section using headed reinforcement, and then grouting the holes [Patterson and Mitchell, 2003]. By providing full confinement, the lap splices in these locations met the ACI requirements and as such, the reinforcement is expected to reach its full capacity. The numbers of threaded bolts, size of anchor plates, horizontal and vertical distribution of the rods were based on the experimental data and were chosen to ensure that the reinforcement could reach its capacity and that the section could reach its full flexural capacity beyond a concrete strain of 0.002.

Retrofit For Shear

At the Plaza floor and at the restaurant, there are large openings in the concrete core, which resulted in significant reduction of shear capacity along one of the core's principal direction. It has been proposed to add fiber reinforced polymer (FRP) sheets to provide an additional shear capacity of approximately 400 kips. This would increase confidence in meeting performance level and safety factor.

RESPONSE OF RETROFITTED STRUCTURE

Figure 17 presents the shear response of the structure. The capacity values are shown along the orientation with the smallest capacity (most openings). Note that addition of TMD has resulted in significant reduction in shear demand throughout the height of the structure. The demand to capacity ratios (DCRs) are all below 1.0. However, at two locations, these values are close to unity. To enhance performance, the added FRP will significantly reduce the shear demand at these two elevations.

Figure 18 presents the bending moment distribution along the height of the concrete core for the retrofitted structure. The flexural demands are less than the capacity. In particular, only minor yielding of the reinforcement is expected at one level. At all other locations, the flexural response will result in steel stresses below the yield value.

Figure 19 and Figure 20 present the response at the top of the core (displacement and acceleration, respectively) along of one axis for one of the DE acceleration records. The displacement response is normalized with respect to the height of the core. Drift and force demands were reduced by approximately 30 %.

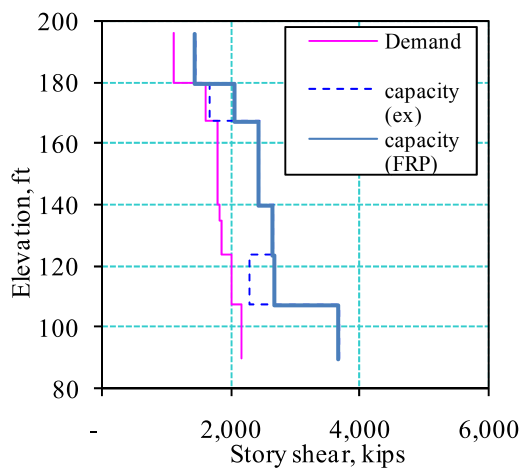

FIGURE 17 - SHEAR PROFILE

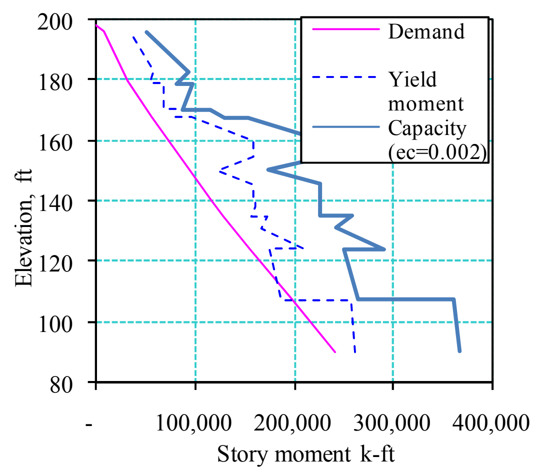

FIGURE 18 - FLEXURAL PROFILE

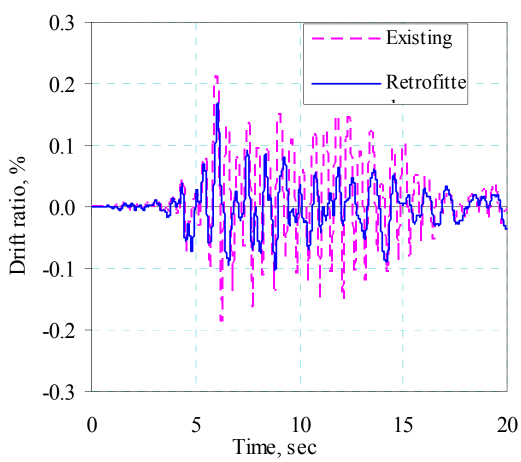

FIGURE 19 - ROOF DISPLACEMENT

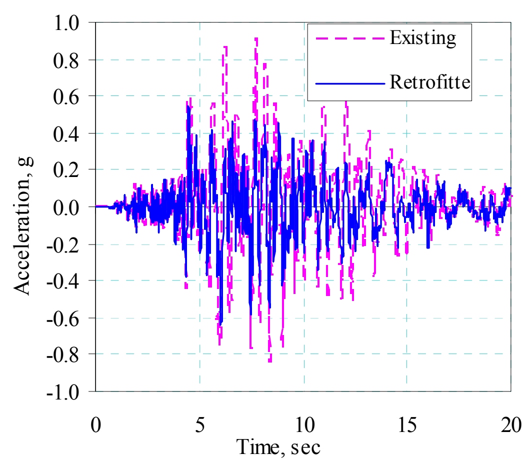

FIGURE 19 - ROOF ACCELERATION

The response of the TMD as a unit is obtained from Model 1. 0 presents the force-displacement response for the TMD stiffness and damper components. The data is shown for the eight stiffness or damper elements combined. The force-displacement response for one of the typical bearing and a damper unit was extracted from Model 2, and was used for the design of the individual units. The stiffness of individual bearings is approximately one-eighth of the TMD stiffness. The individual damper force is approximately one-fourth of that of the TMD damper because four dampers will provide resistance along each axis.

Figure 20 depicts the absolute displacement of the TMD and the roof of the core obtained from one of the DE simulations. The relative motion between the TMD and the roof of the core is of interest. The absolute value of this quantity is presented in Figure 21. It is anticipated that the motion of the TMD relative to the roof will be approximately 4 in. To allow for this motion, a gap of 12 in. is provides between the circumference of the TMD and the inside of the concrete core.

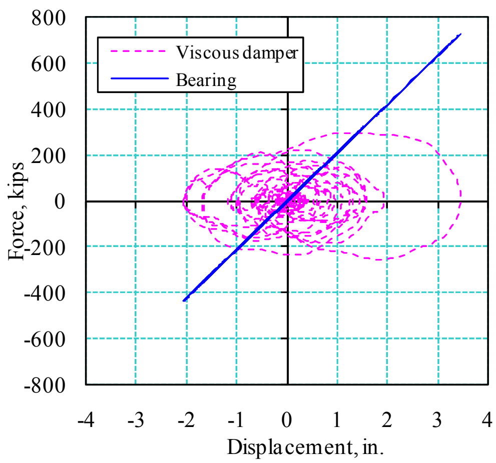

FIGURE 20 - TMD RESPONSE

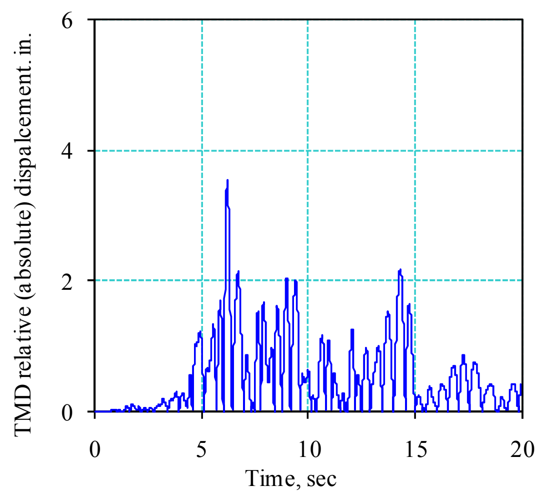

FIGURE 21 - TMD RELATIVE MOTION

CONFIDENCE LEVEL CALCULATIONS

The FEMA 351 [FEMA, 2000] methodology was used to develop confidence levels for not exceeding the non-ductile limit states. FEMA 351 procedure is intended for steel moment framed buildings. However, at the time of evaluation, no similar approach was available for concrete framed structures. To apply this to the concrete tower of the Theme Building, the procedure's approach to non-ductile modes (such as column axial load or non-ductile moment connections) were utilized to assess the performance of the core for reinforcement pull out and shear failure.

In the FEMA method, the confidence level (CL) of meeting a performance goal is computed from:

$$CL = f(\lambda, k, \beta_{UT}) \tag{2}$$

$$\lambda = \frac{\gamma\gamma_a}{\phi} DCR \tag{3}$$

where λ is the confidence index parameter, k is the hazard parameter (equal to 3 in California) and β_{UT} is the vector sum of all logarithms of standard deviations in all demand and capacity , γ and γa are the demand and analysis uncertainty factors, ϕ is the uncertainty in predicting capacity, and DCR is the computed demand to capacity ratio from analysis. Given the computed uncertainties, an overall uncertainty values (β_{UT}) of 0.2 to 0.3 were used in analysis. Using this value, the CL for a number of selected performances were then computed and listed in

Table 1.

limit state	elevation	Condition	CL
shear	167.5 ft	As-is	45%
shear	167.5 ft	Add FRP	80%
shear	124 ft	As-is	55%
shear	124 ft	Add FRP	80%

TABLE 1 – LIMIT STATES PROBABILITY

CONSTRUCTION SCHEDULE

The retrofit construction has begun (see Figure 22). In the first phase the upper arches and the cables will be retrofitted. This is followed by the installation of the TMD at the roof of the core. It is anticipated that the construction will be completed in 2009.

FIGURE 22 - A RECENT CONSTRUCTION PHOTOGRAPH

SUMMARY AND CONCLUSIONS

Seismic evaluation of the LAX Theme Building showed that the reinforced concrete core, which is the main lateral load resisting element of the structure, had deficiencies consistent with its construction vintage. These included non-ductile details such as lack of confinement, low shear capacity and short length of main reinforcement splices. These deficiencies would likely result in severe damage to the structure in the event of major earthquake. A voluntary seismic upgrade was implemented using both increased capacity and reduction in demand.

The increased flexural capacity was achieved by rehabilitating the splices at vulnerable lower level elevations. It is also proposed to add FRP at two critical locations along the core axis with the lowest shear capacity to provide additional safety. Although mot part of the current scope, the client is investigating such implementation in the rehabilitation scope.

The centerpiece of the seismic retrofit is the addition of a TMD at the roof of the core. The TMD was sized to obtain a reduction of approximately 30% for a number of response quantities.

The proposed retrofit was more cost-effective than a conventional scheme and minimized alternations to the appearance of the building and its closure.

The retrofitted structure met its performance goal and there was moderate to high confidence of satisfactory performance in a major earthquake

ACKNOWLEDGEMENTS

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