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SEISMIC RETROFIT OF THE LAX THEME BUILDING WITH A MASS DAMPER

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ABSTRACT

The Theme Building at the Los Angeles International Airport (LAX) is a landmark structure featured in many movies. The main load bearing component of the structure is a core consisting of a reinforced concrete annular wall and interior concrete walls. A detailed mathematical model of the structure was prepared and analyzed using site-specific acceleration histories. Detailed geotechnical investigations, comprehensive material testing, and in-situ dynamic tests were performed to calibrate the model of the structure. Performance based engineering was used to assess the seismic performance of the core. Analysis showed that the concrete core would be subject to large seismic forces and had insufficient flexural and shear capacity to resist the loading. Non-ductile shear failure and reinforcement pull out were identified. A voluntary and comprehensive retrofit consisting of increasing the structure capacity and reducing the seismic demand was evaluated. A tuned mass damper was selected as the main retrofit option to reduce the seismic demand. Such retrofit allowed preserving the unique architectural features of the building. Parametric studies were conducted to optimize the properties of the damper. In addition, steel headed bars were added at lap splice locations for compliance with the current code requirements and to mitigate reinforcement pullout. It was also recommended to add fiber reinforced polymer at critical elevations to augment the existing shear capacity of the core. Reliability-based analyses were conducted and showed that the confidence in avoiding non-ductile modes of failure was significantly increased.

Introduction

The iconic Theme Building at the Los Angeles International airport (LAX) is a well known structure; see Figure 1. A comprehensive investigation was undertaken to assess the performance of the structure and its components due to dynamic loading from wind and seismic loads. The concrete core, supporting exterior walls, floor slabs, steel support 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.

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Figure 1. Photograph of the building.

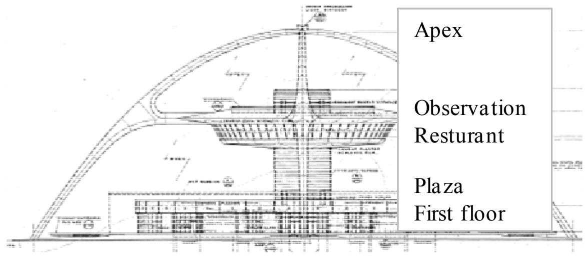

Figure 2. Elevation view of the building.

Description of the Building

The LAX Theme Building was constructed in 1959 and 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.

The Concrete Core

The concrete core is approximately 108 ft tall and extends from base to the roof at elevation 198 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, where access to the building is provided, are supported by a number of independent concrete walls and 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 is reduced to 12 in. above the first floor. The annular wall is the main component resisting the seismic forces. Normal weight concrete is used up to elevation 140 ft and lightweight concrete is used above. Longitudinal reinforcement consists of two curtains with bars varying from #11 at the base to #5 at the roof. Typical lap splice of longitudinal reinforcement is 25 times the bar diameter. Transverse reinforcement consists of two layers at 24-in. spacing. Openings were cut into the annular wall at various elevations

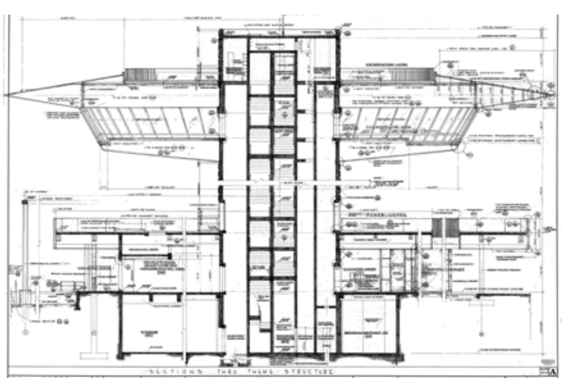

Figure 3. Elevation view of the core.

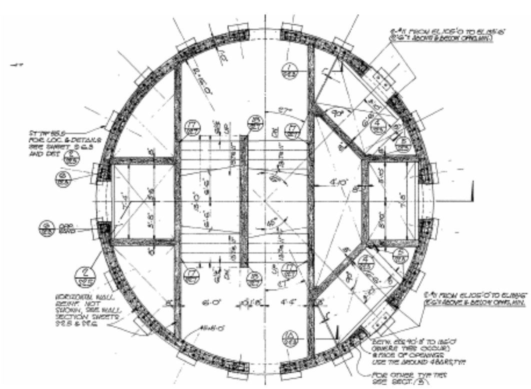

Figure 4. Cross section of concrete core.

Condition Assessment

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 and Butelo 2007) and peer reviewed (Singh 2007). The Design Earthquake (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 by ASCE/SEI 7-05 (ASCE 2005) based on the mapped accelerations of the USGS web site (USGS 2007). Three pairs of spectrum-compatible motions were developed. The computed (average of FN and FP components) and the target spectra are shown in Figure 5.

Material Testing

Comprehensive material testing 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 and 37 concrete cores were tested. The annular wall has compressive strengths of 5.0 and 4.6 ksi; for locations specified with nominal strengths of 4.0 and 3.0 ksi.

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. The field tests consisted of ambient vibration surveys and forced vibration 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 core had a frequency of approximately 2.5 Hz; see Figure 6 . The upper levels of the structure are more flexible due to presence of light-weight concrete at these levels.

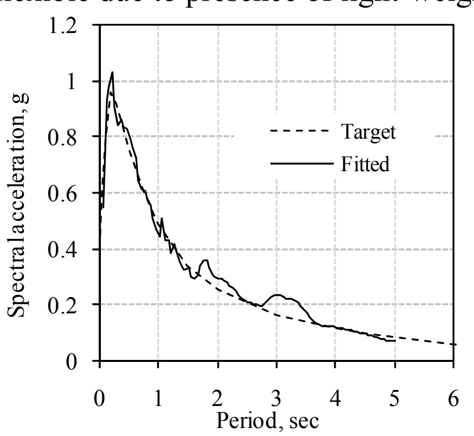

Figure 5. Response spectra.

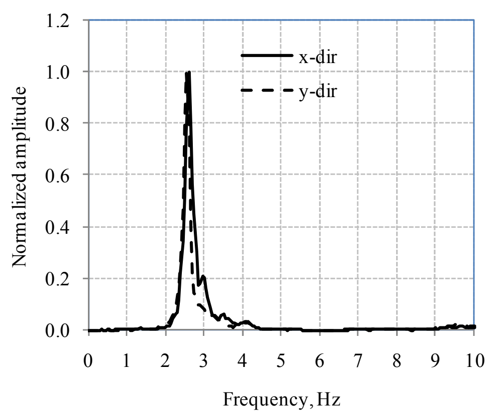

Figure 6. Fundamental mode.

Structural Evaluation and Performance Target

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) with site-specific ground motions was used. The performance objective for this structure was selected to be collapse prevention (CP) at DE level

Capacity Calculations

Typical of vintage concrete buildings, this structure has poor reinforcement detailing which does not meet the current code 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; ACI 318 (ACI 2008) requires longer lap splices. ACI 371 (ACI, 2008) was used to compute the shear capacity of the concrete core. The presence of light-weight concrete and the openings in the core wall reduced the nominal capacity of the annular core. The software program xSection (Mahan 2007) was used to compute the flexural capacity of the concrete core at various elevations. The effect of short splice lengths was taken into account and 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.

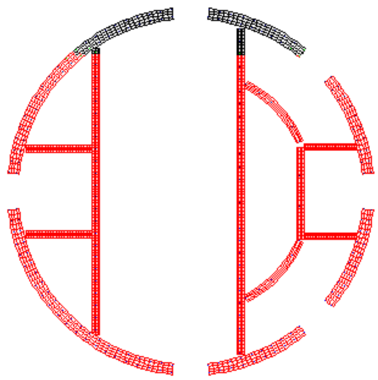

Figure 7. Fiber model.

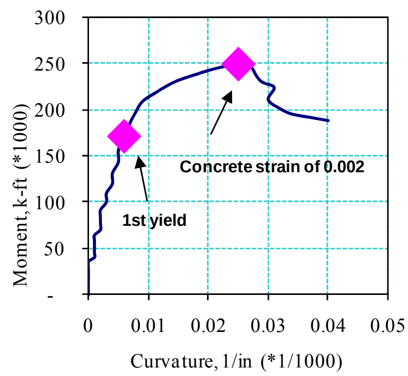

Figure 8. Moment curvature relation.

Mathematical Model of the Building

Program SAP (CSI 2008) was used to prepare mathematical models of the structure. Two three-dimensional models were used in analyses. Mode 1 was stick (frame) representation used for design. The section properties were based on that of the concrete core and interior walls. Model 2, see Figure 9, was a detailed (shell and frame element) model used for final verification. The models had similar mass and were dynamically equivalent with a fundamental frequency of 2.5 Hz; similar to field data. The computed core mode shape from analytical models and the measured mode shape from field tests were closely correlated as shown in Figure 10.

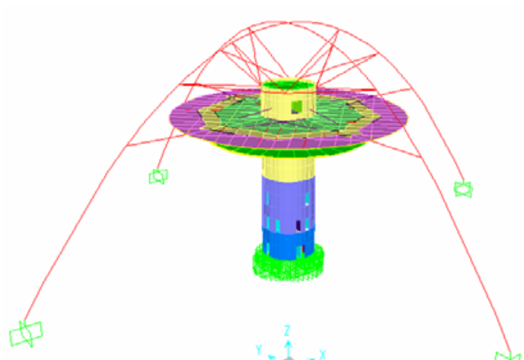

Figure 9. Model 2.

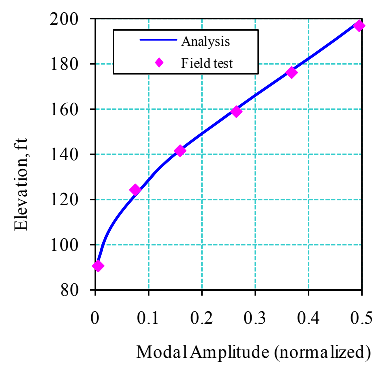

Figure 10. Fundamental mode shape.

Performance of the Existing Structure

Figure 11 and Figure 12 present the distribution of shear and moment demands and capacities along the height of the concrete core. The shear demands were computed from response history analyses and the shear demands exceeded capacity along most of the height of the core. The flexural demands exceeded capacity in the bottom half of the building. Hence, it is expected that the structure will sustain major damage during the design earthquake.

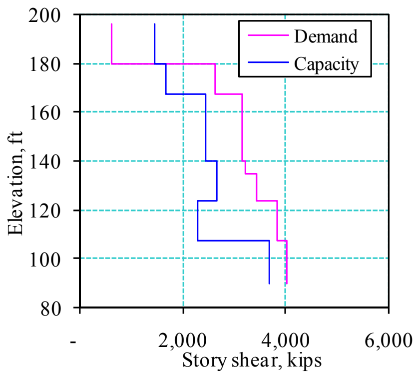

Figure 11. Shear profile.

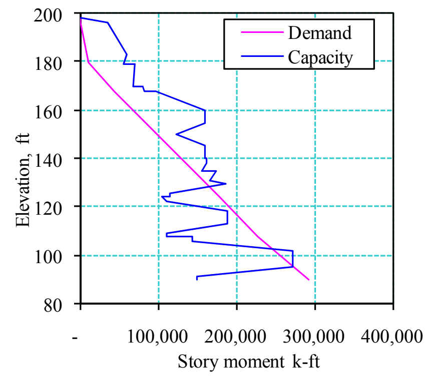

Figure 12. Moment profile.

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 mass damper (MD) to the top of the core. The MD option was selected because it was cost effective, protected the building's architectural features, and minimized business interruption.

The addition of a MD will alter the response of the concrete core by introducing two damped modes in which the motion is primarily concentrated in the MD and reducing drifts and seismic demand of the concrete core. A high-damped MD with a mass ratio of approximately 20% was selected to achieve approximately 30-40% reduction in the response.

MD properties

For seismic excitation, when many input frequencies are present, the optimal TMD properties are obtained from numerical analysis. Sadek et al. (1997) optimized the TMD properties by equating the modal damping ratio in the two complex conjugate modes. Randall et al. (1981) 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, and has examined both analytical simulations and shake table tests.

LAX Theme Building MD

The existing structure produces a complicated system for MD optimization. Since the structure is lighter and more flexible over its top half, its fundamental modal mass is approximately 68% of total mass. Additionally, this structure differs from a typical multi-story structure. Hence, the properties were optimized by conducting analysis simulations similar to the ones undertaken by the researchers previously mentioned. The MD 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 motion of the MD relative to the concrete core; see Figure 14. The MD mass, stiffness, and damping will be provided by steel plates, rubber bearings, and fluid viscous dampers. Figure 15 and Figure 16 present the test results for a typical bearing and damper.

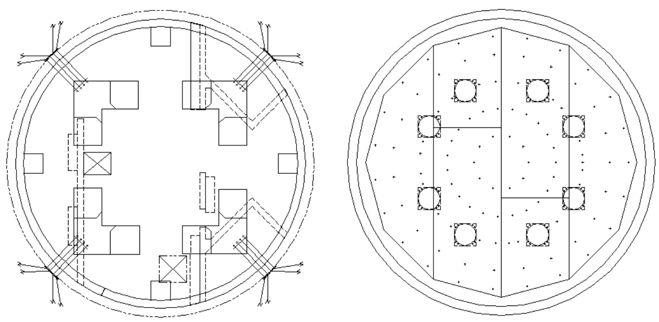

Figure 13. Plan view of the TMD.

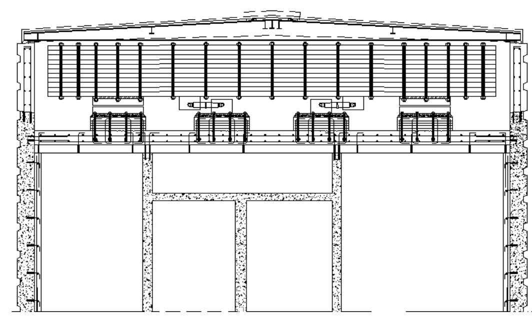

Figure 14. Elevation view of TMD.

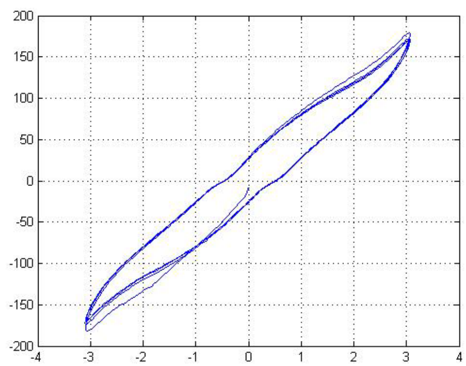

Figure 15. Hysteresis response of bearing.

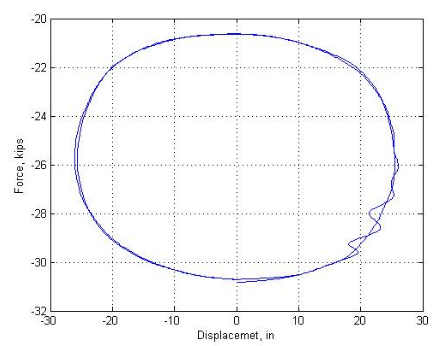

Figure 16. Hysteresis response of damper.

Retrofit of Lap Splices

The reinforcement splices at the three elevations were retrofitted by providing additional confinement. The retrofit was comprised of drilling holes, placing headed reinforcement, and then grouting the holes (Patterson and Mitchell 2003). This served to compress the wall laterally. 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, see Figs. 17 and 18.

Figure 17. Lap splice retrofit.

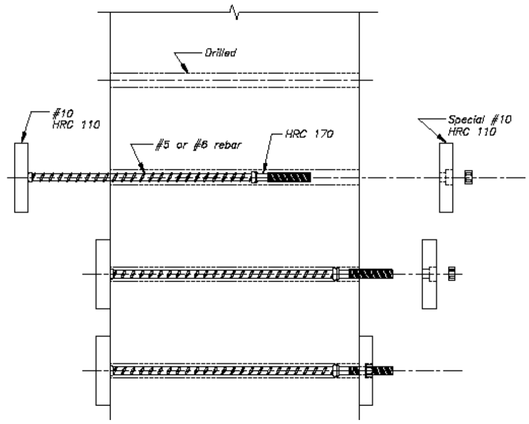

Figure 18. Lap splice retrofit.

Retrofit for Shear

At the Plaza floor and the restaurant level, there are large openings in the concrete core. These openings result in significant reduction of shear capacity along one of the core's principal directions. 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 the design performance level and safety factor.

Response of Retrofitted Structure

Figure 19 and Figure 20 present the shear and flexural responses of the retrofitted structure. Note that addition of MD 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 for the shear profile. To enhance performance, the added FRP will significantly reduce the shear demand at these two elevations. The flexural demands are less than the capacity. In particular, only minor yielding of the reinforcement is expected at one elevation. At all other locations, the flexural response will result in steel stresses below the yield value.

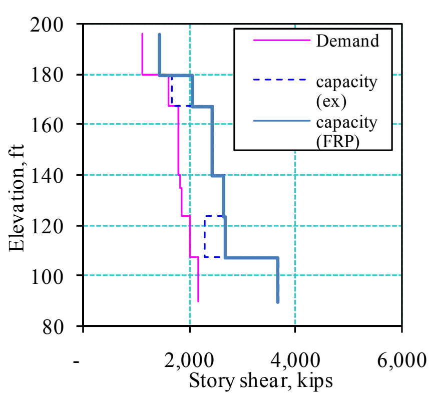

Figure 19. Shear profile.

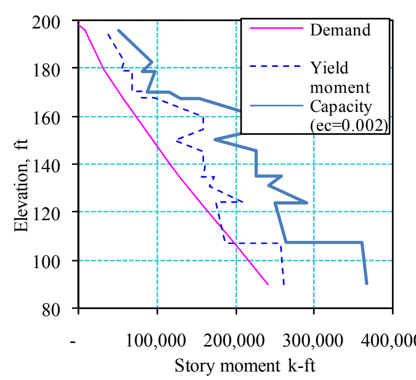

Figure 20. Moment profile.

Figure 21 and Figure 22 present the response at the top of the core (displacement and acceleration, respectively) 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 percent.

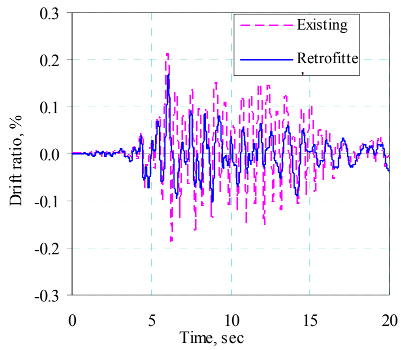

Figure 21. Drift response.

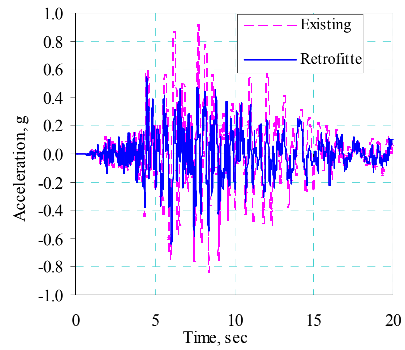

Figure 22. Acceleration response.

Confidence Level Calculations

The FEMA 351 (FEMA 2000) methodology was used to develop confidence levels for not exceeding the non-ductile limit states. The FEMA 351 procedure is intended for steel moment framed buildings. To apply this to the concrete tower of the Theme Building, the procedure's approach to non-ductile modes 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:

$$CL = f(\lambda, k, \beta_{UT}) \quad (1)$$

$$\lambda = \frac{\gamma\gamma_a}{\phi} DCR \quad (2)$$

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. Figure 23 presents the results for the confidence level calculations at the observation level. Note that adding the FRP will increase the confidence level of meeting performance from approximately 40% to 80% for shear.

Table 1. Limit State probability analysis

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%

Construction Schedule

The retrofit construction is currently underway (Figure 24). In the first phase the upper arches and the cables will be retrofitted. This is followed by the installation of the MD at the roof of the core.

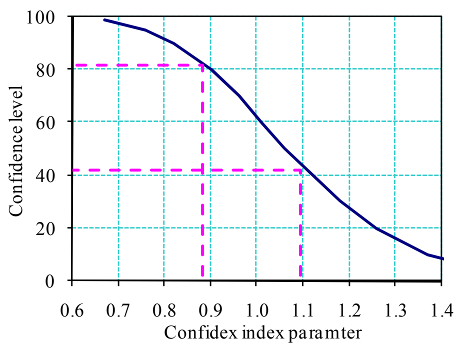

Figure 23. Confidence level analysis.

Figure 24. Construction photo.

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 deficiencies 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 not part of the original scope, the client is investigating such implementation in the rehabilitation scope.

- The centerpiece of the seismic retrofit is the addition of a MD at the roof of the core. The MD was sized to obtain a reduction of approximately 30% for 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

Mr. Jaime Garza of Miyamoto International was the project manager for this project and was responsible for the successful completion of the retrofit program. Dr. R. Nigbor of NEES-UCLA was responsible for the field testing and data analysis reported in this paper. The significant contributions of Messrs. Glenn Ito, David McCombs, Scott Markle, and Jeff Moore of Los Angeles World Airports, Engineering and Project Management Division to this project are kindly acknowledged. The assistance of Messrs. Virgil Aoanan of VCA Engineers, and Millard A. Lee, of Gin Wong Associates is hereby acknowledged.

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