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EXPERIMENTAL/COMPUTATIONAL EXPLORATION OF RETROFIT STRATEGIES FOR THE PIEDRAS BLANCAS LIGHT STATION

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ABSTRACT

The Piedras Blancas Light Station tower, an unreinforced clay brick masonry structure located on the central coast of California, began operation in 1875. In addition to being registered as a historic landmark for many years the light station was elevated to the status of National Monument in 2017. Originally standing 30.48 meters (100 feet tall), the top three levels of the masonry tower were removed in early 1949 after sustaining significant damage in an earthquake on December 31, 1948. Today the light station tower remains operational as an active aid-to-navigation. With the recent recognition as a historic national monument, the impetus to restore the light station to its original configuration has gained traction.

In order to assess finite element models of the light station tower the Piedras Blancas Light Station Association requested Ultra-low Forced Vibration Testing (UL-FVT). UL-FVT, recently validated by a full-scale NSF-NEESR project, is a low cost, mobile alternative to conventional forced vibration testing. The light station tower UL-FVT results correlated favorably with the computational results. This external review provided public confidence in the engineering assessments and the future rehabilitation.

Potential retrofit options include both interior and exterior steel space frames and reinforced concrete walls as well as unbonded post-tensioned tendons cored though the existing tower walls. Interior retrofit options are less efficient than the exterior options and occupy the already limited interior space. Consequently an exterior reinforced concrete wall with brick veneer to replicate the historic masonry brick provides the best balance of cost and historic preservation.

Keywords: Historic; Seismic; Retrofit; Monument; Forced-Vibration-Testing

1. INTRODUCTION

The Piedras Blancas Light Station tower, a circular twin tapered unreinforced clay brick masonry wall structure, is located at 15950 Cabrillo Highway on the scenic central coast of California in close proximity to the National Historic Landmark, Hearst Castle, and approximately 5.5 miles northwest of the city of San Simeon, see Figure 1. The light station tower began operation in 1875. Originally standing 30.48 meters (100 feet) tall including the 3.05 meters (10 feet) tall pedestal at the base, the tower sustained extensive damage in an earthquake on December 31, 1948. The epicenter of the magnitude 4.6 earthquake was approximately 9.67 kilometers (6 miles) offshore. Even through the earthquake accelerations were not equivalent to the design level values of current codes, the damage to the tower was significant enough for the Coast Guard to demolish the top 9.14 meters (30 feet) of the tower including the watch room and the lantern room as well as the lens, lowering the tower height to its current measure of 21.34 meters (70 feet), see Figure 2. Today the light station tower remains operational as an active aid-to-navigation.

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Figure 1. Scenic view from the top of the Piedras Blancas Light Station tower

In addition to being part of the federally designated Piedras Blancas Light Station Outstanding Natural Area and listed on the National Register of Historic Places for many years, the light station was elevated to the status of National Monument in 2017 under the broader umbrella of the California Coastal National Monument, which stretches over 1770 kilometers (1,100 miles) along the California coast. With this recognition as a Historic National Monument, the impetus to restore the light station tower to its original configuration has gained traction with the local community.

(a) Original configuration (PBLSA, 2017) (b) Current configuration

Figure 2. Piedras Blancas Light Station tower

1.1 Recent Seismic Activity

The last significant earthquake in the region occurred on December 22, 2003. The epicenter of the moment magnitude, M_w, 6.5 earthquake was approximately 11.3 kilometers (7 miles) northeast of the city of San Simeon, approximately due east of the light station tower. Considering the magnitude of the event and the close proximity, the light station tower performed remarkably well, however, the main reason for this satisfactory performance was due to the southeast direction of fault slip away from the tower, resulting in lower acceleration demands at the tower location than in the forward directivity direction. The near fault ground motions were particularly strong in the city of Paso Robles where numerous unreinforced masonry buildings were heavily damaged including collapse of a few structures. The light station tower sustained negligible visible damage in the event as evidenced by the fact that the tower was not discussed in the two prominent reports on the earthquake by the Earthquake Engineering Research Institute (Lynn et al. 2004) and the California Seismic Safety Commission (CSSC 2004), both of which detailed damage to structures, lifelines and roadways.

1.2 PBLSA Plans for the Light Station Tower

With multiple fault lines in the near vicinity to the light station the seismic risk to the tower is high. The Piedras Blancas Light Station Association (PBLSA) has been evaluating potential retrofit schemes for more than a decade. The PBLSA recently hired Shoreline Engineering, a structural engineering firm in Morro Bay, California that has extensive experience with the design and rehabilitation of marine facilities, to assess options for the retrofit of the Piedras Blancas Light Station tower. Shoreline Engineering assessed the tower demands and capacities according to the California Historic Building Code (CHBC), the California Existing Building Code (CEBC) and the California Building Code (CBC) as each applies to the evaluation using the 2012 United States Geological Survey (USGS) seismic maps. The spectral acceleration due to the Maximum Considered Earthquake (MCE) or approximately 2500 year return period event is 1.35g and the Design Basis Earthquake (DBE) or 2/3 of the MCE event is 0.9g. An importance factor, I_e, of 1.5 was applied since the structure is used for essential services. A response modification factor, R, of 1.5 was also applied. This low response modification factor implies very limited ductility due to the brittle nature of unreinforced clay brick masonry. With equal importance and response modification factors, the tower is expected to remain essentially elastic in the design level earthquake assuming the structure can resist the seismic demands.

1.3 UL-FVT of the Light Station Tower

In order to validate finite element models of the light station tower the Piedras Blancas Light Station Association (PBLSA) requested Ultra-low Forced Vibration Testing (UL-FVT). UL-FVT, recently validated by a full-scale NSF-NEESR project (McDaniel et. al., 2014), is a low cost, mobile alternative to conventional forced vibration testing. In the past few years, the authors have developed and implemented (Archer et.al. 2011; McDaniel et. al. 2010) a unique type of Forced Vibration Testing (FVT) for low-rise building structures using ultra-low force amplitudes. UL-FVT is accomplished by placing a small portable (~100 lb.) harmonic shaker on the top of a structure and recording the resulting floor accelerations throughout the structure using highly sensitive accelerometers. The structures tested using UL-FVT include steel and concrete moment resisting frames, steel concentric braced frames, reinforced concrete moment resisting frames, and reinforced concrete shear walls. The largest structure tested to date is a 5-story 180,000 ft^2 library building (Rendon et. al. 2012). The Piedras Blancas Light Station tower is an ideal candidate for UL-FVT with the concrete topping slab and a relatively high natural frequency of vibration.

2. LIGHT STATION TOWER ASSESSMENT

2.1 Visual Assessment and Material Testing of the Light Station Tower

Unknown material properties are typically conservatively estimated with low default values provided by design codes. In order to obtain more realistic material properties for the unreinforced masonry, in situ testing was performed on the tower masonry. Flat jack tests and masonry shear tests were conducted to determine the masonry compression strength and the shear strength of the mortar joints, respectively. In place masonry shear test results varied from 214 psi to 342psi. These values are high for the masonry of the era (Atkinson-Noland 2007), pointing to one of the key reasons why the light station tower has withstood multiple seismic events for over the past century. Flat jack testing determines the vertical in situ compressive strength in a masonry wall. The flat jack test results yielded a masonry compression strength, f'_m, equal to 1000 psi.

Visual inspection of the tower highlighted a few key deficiencies that need to be addressed in addition to seismic deficiencies. Visible deterioration of the exterior clay masonry bricks and mortar as well as exterior cracking in the tower face has been covered by painting of the tower in 2012, however, photographs of the tower prior to 2012 document the need to address these issues. The majority of the exterior tower face cracks originate from the door at the tower base and the windows along the tower elevation where stress concentrations are increased due to the openings in the tower walls. The interior and exterior tower walls are separated by an annular ventilation space that was capped with a reinforced concrete slab after the 1948 earthquake. Capping of the ventilation space has led to inadequate ventilation and therefore the potential for deterioration of the faces of the exterior and interior masonry walls along the ventilation space. Inadequate support for the tower stair landings was also documented in the visual inspection.

2.2 Computational Assessment of the Light Station Tower

Three dimensional computational finite element models (see Figure 3) were created by Shoreline Engineering to assess the seismic demands on the current tower configuration as well as potential retrofit

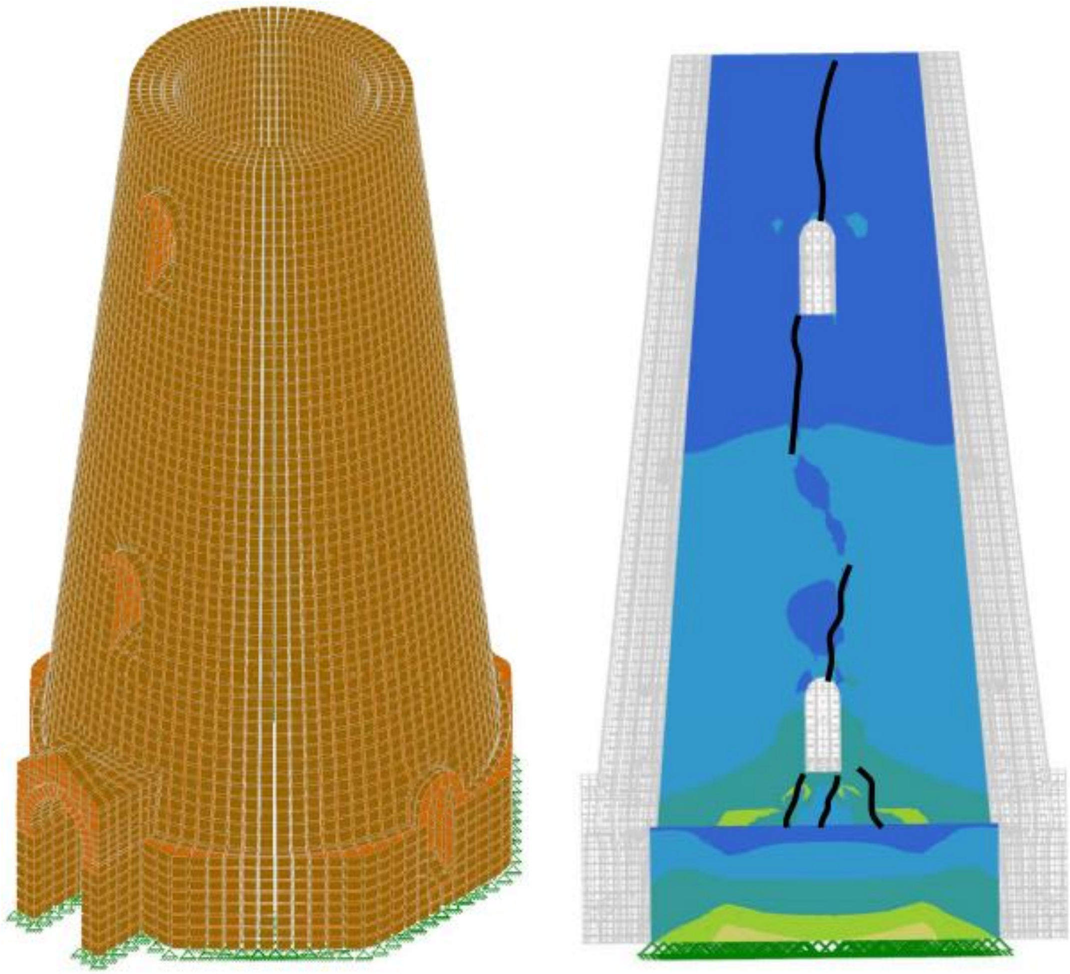


Figure 3. Computational model of tower and crack mapping comparison to stresses (RISA, 2016)

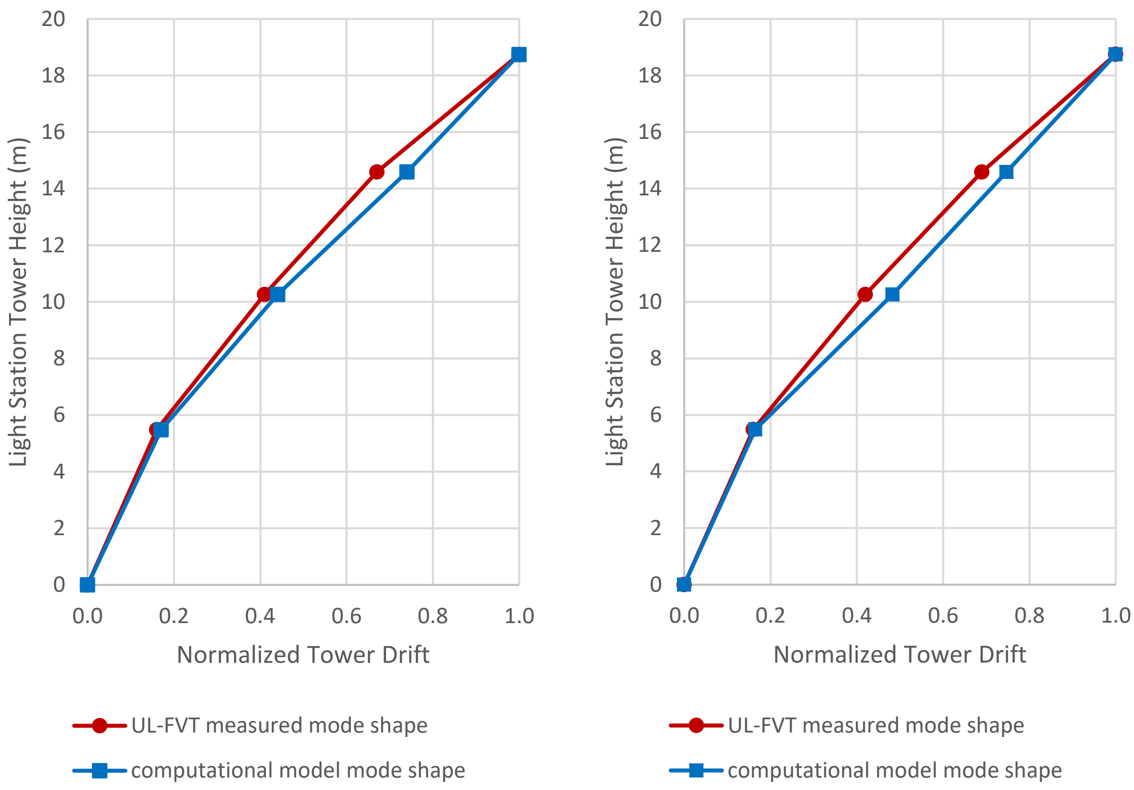
options for the tower. Eight node brick elements were used to model the variation in the tower wall thickness, the observed crack patterns in the tower walls as well as the annular ventilation space running the full height of the tower. The ventilation space location and condition were estimated since access to the space was closed off by the capping of the tower with the reinforced concrete slab after the top three levels of the tower were removed in 1948. Observed crack patterns were compared to the stress demands from the computational model results.

Deficiencies identified during the seismic evaluation include inadequate tension and shear capacity of the masonry walls, inadequate resistance to overturning of the tower and inadequate resistance to sliding at the tower base.

2.3 UL-FVT Assessment of the Light Station Tower

Ambient vibrations were measured first to provide a rough estimate of the tower fundamental frequencies. The maximum ambient vibration readings ranged between 20-30 micro-g's at the tower top. Next a linear shaker was placed in the optimal location on the light station tower top to excite the structure to approximately 3000 micro-g's, or roughly 100 times the ambient vibration levels, allowing for clear identification of the tower resonant response (see Figure 4). With very low damping, dynamic amplification in the range of 50-100 times is predicted by standard structural dynamics equations (Chopra, 2012). Next, Ultra-Low Forced Vibration Testing (UL-FVT) was used to experimentally determine the fundamental frequencies and mode shapes. These experimental fundamental frequencies and mode shapes (see Figure 5) were compared to the computational results which correlated favorably. This external review of the tower finite element model provided public confidence in the engineering assessments and the future rehabilitation decision making process. UL-FVT will also be conducted after the tower is retrofit as a comparison for the retrofitted tower finite element model.

Figure 4. UL-FVT frequency and mode shape experimentation



(a) Mode 1 (b) Mode 2

Figure 5. Normalized UL-FVT and computational mode shape comparison

3. LIGHT STATION TOWER RETROFIT ASSESSMENT

3.1 Light Station Tower Retrofit Considerations

A variety of rehabilitation schemes are currently being evaluated by Shoreline Engineering to restore the Piedras Blancas Light Station tower to its original height of 30.48 meters (100 feet), from full restoration of the tower top with clay brick masonry to light weight replica options that maintain the historic character of the structure without adding the full weight of clay brick masonry. The full restoration of the tower top with clay brick masonry while clearly providing the most historically accurate solution would also lead to the highest seismic demand/capacity ratios on the existing tower structure. Any unnecessary additional weight will only exacerbate the current masonry deficiencies, therefore keeping the additional weight to a minimum is a high priority. Fortunately, the tower tapers in thickness from approximately 1m thick walls at the base to approximately 0.7 m thick walls at the tower top, continuing the tower taper with the restoration to 30.48 m with a light weight façade to include the original cast iron ornamentation as seen in Figure 6 will keep the additional weight to a minimum.

Increasing the tower height will increase the fundamental periods (decrease the fundamental frequencies) of the tower. However, this increase in period will have little effect on the overall design seismic acceleration demand since the corner period at the edge of the design acceleration response spectrum plateau is estimated by the USGS to be 0.37 seconds. The computational modeling predicted a fundamental period of approximately 0.2 seconds and the UL-FVT measured a fundamental period of approximately 0.15 seconds. The UL-FVT period is expected to be low since the cracking in the masonry at the design level event will increase the period of the structure beyond the UL-FVT measured period. However, an increase in the period beyond 0.37 seconds due to restoring the tower to its original configuration is unlikely, therefore, the pre- and post-retrofit design accelerations will likely remain unchanged.

3.2 Retrofit Options for the Light Station Tower

Currently public access to the interior of the tower including ascending and descending the tower stairs is restricted due to the high stress demand/capacity ratios in the masonry. Since public funding will be included in the restoration of the tower, any rehabilitation scheme needs to focus on reopening the tower for public access to the interior of the tower, not simply stabilizing the tower against collapse in the event of a design level earthquake. Currently design level seismic demands create stresses that exceed the strength of the unreinforced clay brick masonry. Overturning of the light station tower in a seismic event and sliding at the tower base are global deficiencies that are likely to occur under the design level earthquake as well. Options for mitigating these deficiencies include the following:

Interior schemes:

Interior steel space frame - This retrofit schemes involves anchoring a full height steel space frame to the interior face of the existing tower wall. The steel space frame would compromise the historic character of the interior of the tower, however, the exterior of the tower would remain aesthetically intact. Modification to the stair landings would be necessary to accommodate the steel interior space frame.

Interior shotcrete concrete shear wall – This retrofit scheme involves anchoring a new concrete shear wall to the face of the existing interior wall of the tower, providing a concrete option similar to the interior steel space frame option. The stair landings would be cast into the concrete wall with anchors providing adequate load transfer.

The disturbance of the internal historic fabric of the tower and intrusion on the limited interior tower space by the internal space frame/concrete shear wall prompted Shoreline Engineering to seek exterior retrofit options.

Exterior schemes:

Exterior steel space frame – This retrofit scheme involves anchoring a full height steel space frame to the exterior face of the existing tower wall. This option was quickly deemed unrealistic due to the historic nature of the tower even though it would be the most cost effective option.

Vertical post-tensioned steel tendons – This retrofit scheme involves installation of unbonded post-tensioned steels tendons in the exterior section of the tower wall. A radial pattern of unbonded post-tensioned steel rods would be anchored at the 4th landing (existing tower top) and anchored at the tower base. This option protects the integrity of the historic nature of the light station tower including the exterior and interior tower walls, however, this is also the most expensive option proposed. In addition the large thickness of the tower walls, exceeding 1m (3 feet) at the tower base, would require numerous post-tensioned tendons to apply the necessary compressive stresses needed to overcome any tensile stress demands on the masonry.

Exterior concrete wall with brick veneer – This retrofit scheme involves sand blasting the existing exterior of the tower to remove any deteriorated mortar and clay brick masonry. A 152.4 mm (6 inch) reinforced concrete shear wall would encompass the tower with 50.8 mm (2 inch) veneer brick to replicate the historic exterior. The overall diameter at the base would increase approximately 4% with this option. With a goal of historic preservation and a cost effective design, this exterior retrofit option is likely to be the choice of the PBLSA. The retrofit will be designed to sustain the added weight from the restoration of the top 9.14m (30 feet) of the light station tower with a light weight composite replica in the range of 25-30% of the weight of a full clay brick masonry restoration.

Figure 6. Original light station tower top 9.14 m (30 feet)

The design of each retrofit option described above includes resisting the additional demands from the weight of a light weight replica of the top 9.14 meters (30 feet) of the tower. In the case of any of the retrofit options, the following additional rehabilitation steps are required:

- Removal of the 15.24 centimeter (6 inch) concrete slab on the tower top to allow access to the annular ventilation shafts and to the renovated tower top.

- Connection of the twin unreinforced clay brick masonry walls (exterior wall and interior wall) of the tower to ensure that the tower walls act as a unit rather than individually. This would enhance both local slenderness deficiencies for the interior wall and global tower instabilities. One option is to install stainless steel helical anchors horizontally on an 18" grid alternating from the interior and exterior of the tower. This would connect the inner tower walls to the exterior tower walls by passing through the inner tower walls and annular ventilation space and anchoring into the exterior tower walls. A borescope investigation of the annular ventilation spaces recently identified a highly irregular pattern to the ventilation spaces, with less than expected ventilation space on the east side of the tower and no ventilation space found on the west side of the tower. Therefore, grouting the ventilation spaces is recommended prior to installing the horizontal anchors.

- Anchoring of the base of the tower to the underlying bedrock is also necessary. This can be accomplished by coring anchors 20-30' below the surface using in a radial pattern around the tower circumference to resist global overturning of the tower about the foundation and sliding of the tower base during seismic events.

- A challenge to including a lightweight replica of the tower top will be to ensure deformation compatibility between the existing tower and the addition both in plan and elevation. Keeping the tower cross-section from deforming in an oval shape rather than the preferred circular deformation and keeping the first mode cantilever behavior in elevation influenced the retrofit decision making process.

3.3 UL-FVT Testing of the Light Station Tower Post-Retrofit

Physical testing (UL-FVT) of the Piedras Blancas Light Station tower during and after the retrofit is a useful tool to confirm the validity of both the retrofit scheme and the computation model used in the tower retrofit design. The one aspect of the retrofit that the UL-FVT will not confirm is the anchoring of the tower to the underlying bedrock. The entire base of the tower is in a high state of compression both before and after the retrofit work. The addition of the anchors will only affect the mode shapes and periods (frequencies) of the structure if any part of the base is put into tension (or if it slid) such as during a large seismic event. Thus the presence of the anchors is unlikely to be detected.

The addition of an exterior concrete wall will both stiffen the existing structure and add mass. Both changes will affect the mode shapes and natural periods (frequencies) and will be easily detected by UL-FVT. More importantly the UL-FVT will confirm the validity of the computational model used in the tower retrofit design. As was done for the existing structure, several accelerometers will be placed over the height of the tower and the resulting mode shapes and natural periods will be compared to the computational predictions. Furthermore, accelerometers will be placed on the inside and outside face of the tower wall in several locations to confirm the bond between the existing and new walls. A lack of bond will show up as a differential in the accelerations between the two faces.

The removal of the concrete slab diaphragm at the top of the existing tower to provide access to the new top may introduce ovalling (deformation of the circular cross-section). If present this will be detected in the mode shapes predicted by the computational model and will be confirmed using UL-FVT by placing several horizontal accelerometers at the elevation of the former slab.

The lightweight tower top will affect the mode shapes and natural periods (frequencies) of the overall tower primarily through the additional mass. It will also become an extension to the tower and will likely add additional modes of vibration; some modes may have little interaction with the tower. The change in the tower modal response will show up in the computational model of the entire retrofitted tower and top and will be confirmed through UL-FVT. For the UL-FVT, additional accelerometers will be placed throughout the new tower top to capture the modal response.

4. CONCLUSION

The Piedras Blancas Light Station tower originally stood 30.48m (100 feet) tall when it began operation in 1875. After an earthquake in 1948 the top 9.14m (30 feet) of the tower were removed by the United States Coast Guard. Since then the tower has performed remarkably well considering the high seismicity of the region including a moment magnitude 6.5 event in 2003. Efforts to restore the tower to the original configuration have been ongoing for years, however, the recent elevation of the light station tower to the level of National Monument will help with the necessary fundraising. Restoration of the tower to its original configuration will be costly due to the balancing of the high seismic demands of the region and the historic preservation requirements. In order to raise public confidence in the engineering assessment of the current tower configuration and retrofit options, Ultra-low Forced Vibration Testing (UL-FVT) was conducted to determine the tower fundamental periods (frequencies) and mode shapes for validation of the three dimensional computational finite element models. Ultra-Low FVT (UL-FVT) was accomplished by placing a small portable (~100 lb.) harmonic shaker on the top of the light station tower and recording the resulting accelerations throughout the structure using highly sensitive accelerometers.

Retrofit options for the Piedras Blancas Light Station tower include both internal and external steel space frames or reinforced concrete walls as well as unbonded post-tensioned tendons cored though the existing tower walls. The interior retrofit options protect the historic nature of the exterior of the tower while attempting to minimize the historical impact on the tower interior. Although unbonded post-tensioned tendons in the exterior wall running full height and anchored at the tower top and the tower base would provide the least impact on the historic nature of the tower the estimated cost is prohibitive. Unfortunately, internal retrofit options are less efficient than the exterior and post-tensioned options and occupy the already limited interior space. Consequently an exterior reinforced concrete wall with brick veneer to replicate the historic clay masonry brick provides the best balance of cost and historic preservation. Once the Piedras Blancas Light Station tower retrofit is completed, UL-FVT will be conducted on the tower to confirm the validity of both the retrofit scheme and the computational models used in the tower retrofit design.

5. ACKNOWLEDGEMENTS

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