FORCED VIBRATION TESTING AND RETROFIT OF THE HISTORIC UNREINFORCED MASONRY POINT SUR LIGHT STATION

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Abstract

The Point Sur Light Station, located on the Big Sur Coast of California, includes a historic unreinforced masonry lighthouse building constructed in 1889. For nearly a century the lighthouse illuminated the rugged coastline with a first-order Fresnel lens, the largest of the Fresnel lenses with the longest broadcast. The lens stands over 8 feet (2.4 m) tall and 6 feet (1.8 m) in diameter, weighing nearly 5 tons (4500 kg). In 1978 the Fresnel Lens was replaced with a modern aero beacon that continues to serve as an active aid-to-navigation. Recently, the Central Coast Lighthouse Keepers applied to the United States Coast Guard for approval to repatriate the original first-order Fresnel lens to the lighthouse. The United States Coast Guard has approved the request, making this the only first-order Fresnel lens that has been approved for repatriation into its original setting. To accommodate the large increase in weight and to protect the fragile lens, assessment of the seismic performance of the unreinforced masonry lighthouse is necessary to develop and implement the most appropriate rehabilitation scheme. In order to provide an independent assessment of the dynamic response of the lighthouse, non-destructive forced vibration testing was conducted to experimentally determine the lighthouse's natural frequencies and mode shapes, key parameters needed to assess the seismic vulnerability of the structure and to calibrate the finite element models of the building.

The repatriation of the first-order Fresnel lens necessitates additional research into the pre- and post-retrofit dynamic response of the Point Sur Light Station unreinforced masonry lighthouse to determine the most appropriate rehabiliation approach. The lighthouse is comprised of a main tower housing the aero beacon as well as two attached rooms that frame into the midheight of the tower, the fog room and the radio room. These three sections of the lighthouse form a T-section in plan. In addition to this plan irregularity, the rigid diaphragm of the main tower and the flexible diaphragms of the two attached rooms provide unique challenges for assessing the dynamic response of the building. Calibrating an accurate finite element model of the lighthouse with the nondestructive forced vibration testing is necessary to capture the difference in the dynamic response of potential rehabilitation schemes aimed at stabilizing the out-of-plane response of the gabled walls. This research on the Point Sur Lighthouse will help advance the state-of-the-art retrofitting techniques of historic unreinforced masonry structures by integrating both nondestructive forced vibration testing and finite element modeling.

Keywords: retrofit; unreinforced masonry; heritage structure, forced vibration testing, finite element modeling

1. Introduction

The goal of this research is to determine seismic rehabilitation and retrofit options for the Point Sur Lighthouse using ultra-low forced vibration testing (UL-FVT) [1] and finite element computer modeling. The lighthouse is a historic unreinforced sandstone masonry (URM) building located in an area of high seismicity, therefore it is particularly susceptible to damage in the event of strong ground motions. Similar research has been performed by the authors on the historic Piedras Blancas Lighthouse [2] farther south and will be used to inform the work done here. As indicated by the high height-to-thickness ratios, ranging between 7.7 and 13.5 at the peak of the tallest wall, the URM walls are generally expected to perform very poorly in seismic events. High inertial forces and the low strength of the masonry and mortar system contribute to poor performance both in-plane and out-of-plane. In the case of the Point Sur Lighthouse, the main area of concern is the out-of-plane dynamic behavior of the walls. The large thickness of the walls and light wood-framed roof relieve much of the burden a strong ground motion normally places on the in-plane strength of the walls. Another area of concern is the weak structural connections that are important for proper lateral load flow.

The historic character of the building makes designing the rehabilitation and retrofit options difficult for a number of reasons. Any structural improvements that are made must preserve the historic fabric of the building, which would limit the workable areas to the building's interior only. The simple floorplan and plain architectural features of both the interior and exterior of the lighthouse make the design of any structural improvements to be minimally intrusive a challenge. Funding for the rehabilitation of the lighthouse comes from both public and private preservation societies and is extremely limited, making the design even more challenging. The Point Sur Lighthouse was evaluated according to ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings [3] and the IEBC 2015: International Existing Building Code [4] in order to highlight key deficiencies needing rehabilitation to mitigate major damage during strong ground shaking. This research will focus on the evaluation of those solutions with respect to their effect on the dynamic behavior of the building. Any solution needs to be minimally invasive, cost effective, and avoid adding load to the foundations as the lighthouse is positioned on steep and possibly unstable cliffs on two sides.

2. Point Sur Lighthouse

The Point Sur Light Station is located along California State Route 1 between Monterey and San Luis Obispo. The lighthouse was first activated on August 1, 1889 after 11 years of petitioning the U.S. Lighthouse Service Board for funds and three years of preparations and construction. The U.S. Coast Guard assumed responsibility of the light station in 1939 and automated the lighthouse in 1974, up until which time the lantern and fog signal were operated by a lighthouse keeper and assistants. The light station is on the National Register of Historic Places and is part of the Point Sur State Historic Park [5].

The lighthouse consists of three main sections in plan: the fog room, tower, and radio room, shown to scale in Figure 2. The fog room is located furthest to the northwest and is roughly 33 feet by 40 feet (10.1 m by 12.2 m) in plan. The unreinforced stone masonry walls are roughly 20 inches (0.51 m) thick and range from almost 14 feet (4.3 m) high to over 22 feet (6.7 m) high at the gabled walls. The signal room is roughly 16 feet by 17 feet (4.9 m by 5.2 m) in plan with walls 20 inches thick extending between 14 feet (4.3 m) and 19 feet (5.8 m). The tower is roughly 15 feet by 17 feet (4.6 m by 5.2 m) in plan and a combined brick and stone cross section extends more than 32 feet (9.8 m) from the ground level, topped by the glass lantern room. The lighthouse sits on or near bedrock close to the edge of a steep cliff with portions of the foundation extending farther below the finish floor than others. The original Fresnel lens weighs 4.33 kips (2000 kg) and the entire lens assembly weighs 9.57 kips (4500 kg) [5]. Even though the lens assembly was removed for safekeeping in 1978, the lighthouse was analyzed with and without the lens due to the plans for future repatriation.

Fig.1 – Photos of the Lighthous

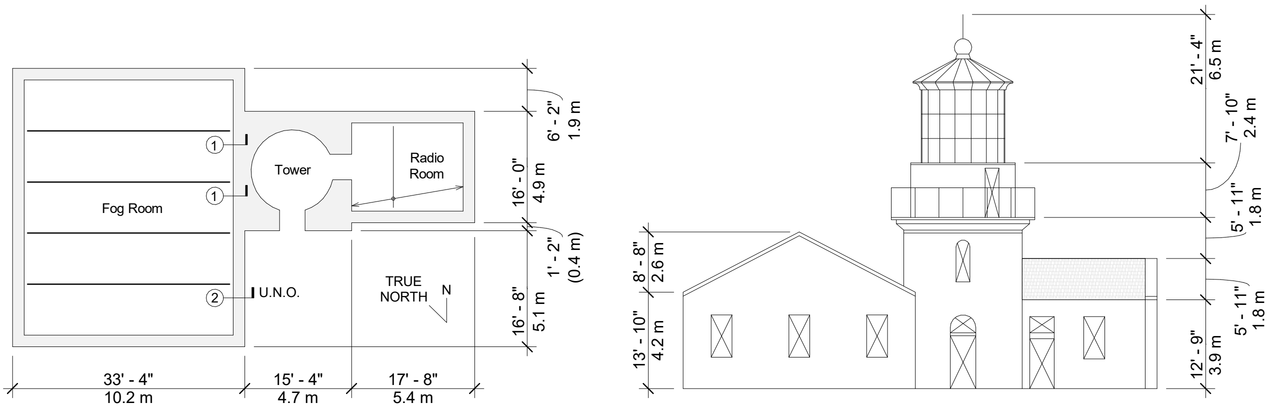

Fig. 2 – Lighthouse Plan and South Elevation

The portions of the lighthouse that are currently most at risk are the gabled walls of the fog room, found on the north and south elevations. Even though all walls of the building apart from the tower are approximately 20 inches thick, the gabled walls have the largest height-to-thickness ratio of 13.5, exceeding the IEBC limit of 12, high enough to indicate a strong risk of poor out-of-plane dynamic behavior in an earthquake. Height-to-thickness evaluations assume a proper floor/roof connection at the levels above and below the wall, which the gabled walls do not have due to the gravity only connection to the roof diaphragm. An adequate and properly connected roof diaphragm allows the wall to act more like a simply supported beam rather than a cantilever as assumed in the code-allowed limit height-to-thickness ratios. An adequate roof diaphragm also helps to distribute out-of-plane loads of the walls to other areas of the building and eventually to the lateral force resisting system. This is not the case in the lighthouse because the timber roof diaphragm itself lacks the necessary strength and stiffness, and its connections, shown in Figure 3, are exceedingly deficient for lateral load transfer. One benefit of the light wood framed roof assembly is that it does not add significant inertial load compared to the surrounding masonry walls.

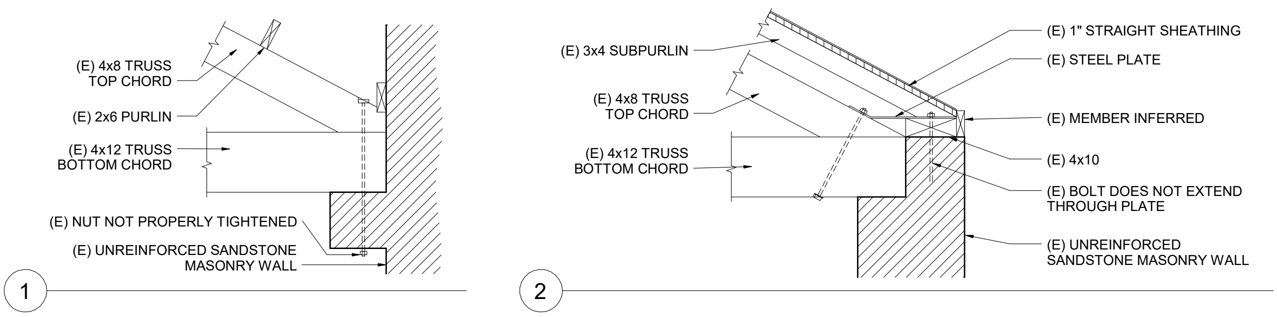

Fig. 3 – Existing Truss Connection Details

3. Seismic Rehabilitation Evaluations

The lighthouse was evaluated according to ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings [3], and the IEBC 2015: International Existing Building Code [4], which outlines procedures of various levels of investigation depending on the engineer's scope of work. The most basic screening, Tier 1, was used to check for continuous load path and an adequate lateral force resisting system. The next level, Tier 2, was used to investigate proportions such as height-to-thickness ratios, aspect ratios, and component strengths. The final evaluation, Tier 3, was used to design the solutions to the deficiencies identified in the previous two screenings.

The recommendations for strengthening the lighthouse according to the ASCE 41 Tier 1-3 evaluations are as follows: Add blocking and sheathing to the diaphragm to increase strength and strengthen connections between the diaphragm and walls to allow lateral loads to be properly transferred. Add cross ties to the bays of diaphragm closest to the gabled walls to distribute their out-of-plane loads to the diaphragm. Add a brace along the top of the gabled walls to bring the height-to-thickness ratio within the code-allowed limit and reduce wall deflections out-of-plane at the gable peak. Add regularly spaced HSS strong backs to the gabled walls to compensate for the wall's low strength capacity out-of-plane. A strong back is a vertical member installed directly adjacent to and anchored into a wall to mitigate out-of-plane structural deficiencies. An alternative to the large steel strong backs that would have a similar effect is regularly spaced bundles of rebar or post-tensioned cables in cored-out shafts inside the wall. One issue with options that involve coring into the walls is that it could be expensive and labor intensive but also substantially increase the loads to the foundations. These strengthening solutions and others were modeled in RISA-3D, a finite element computer program, and their effects on the dynamic behavior of the building were studied. The results are presented in Section 5.

There are other unreinforced masonry buildings at the light station that are currently being restored and seismically rehabilitated. The buildings have similar structural deficiencies as those present in the lighthouse, for example weak diaphragms, and inadequate wall to diaphragm connections. One of the main ways the deficiencies are being addressed is by adding new chords and collectors anchored into the walls with a series of anchor bolts, steel angles, and tube sections. The new connections and members are being added in small cavities in the walls and in between existing floor/roof framing members and will eventually be covered up by wall plaster to preserve the architectural style of the interior. The work that has already been done to other buildings on the property provides a precedent for what is feasible in the lighthouse even if that work may not be covered by wall plaster. For example, a similar chord/collector member and connection may be designed and installed in the lighthouse along the top of the walls as part of the strengthening of the diaphragm and lateral system.

The lighthouse was subjected to UL-FVT in order to characterize its dynamic behavior. UL-FVT is nondestructive and very versatile in its applications. The testing that was conducted used a 30 pound (14 kg) linear mass shaker placed on the top landing in the tower of the lighthouse and piezoelectric accelerometers were placed at strategic locations around the building to measure the resulting accelerations, shown in Figure 4. A frequency sweep was used to find the natural frequencies of the building in the two orthogonal directions and accelerations were recorded once steady state vibration was reached. The natural frequency found when shaking the lighthouse in the north-south direction was 9.6 Hz and the natural frequency found when shaking the lighthouse in the east-west direction was 9.7 Hz.

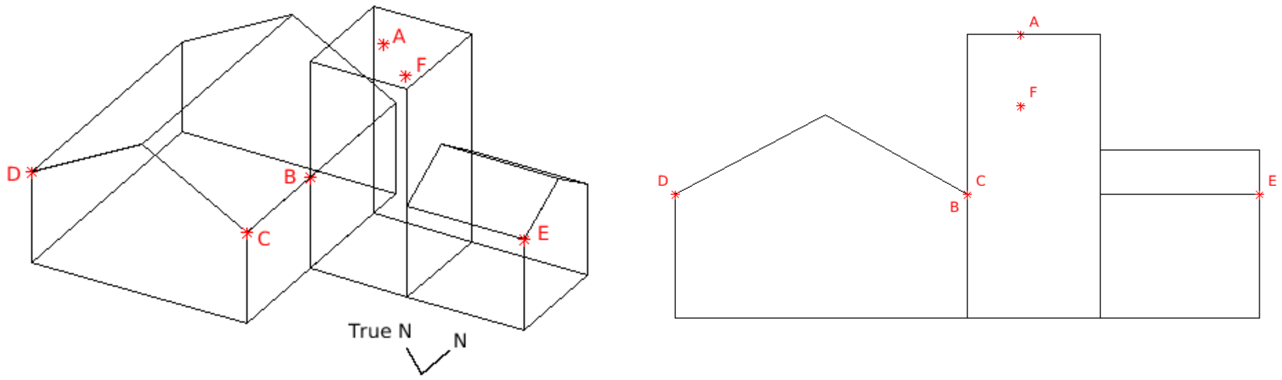

Fig. 4 – Locations of Accelerometers in Isometric View and South Elevation

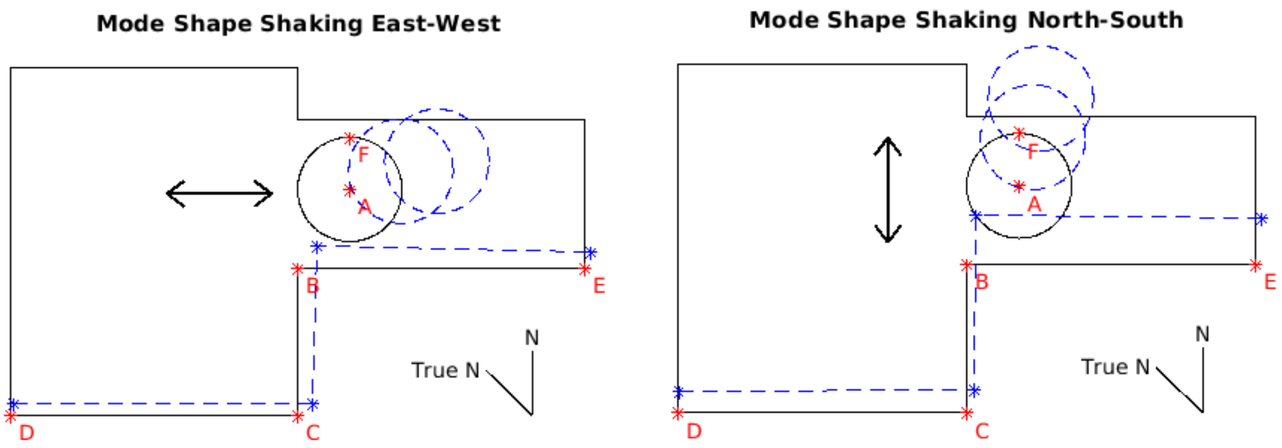

Fig. 5 – Graphical Mode Shapes Derived from UL-FVT Data

A mode shape describes the dynamic response throughout a structure as a set of relative quantities. Accelerations taken at each accelerometer location, labeled in Figure 4 with an asterisk and tagged A-F, are proportional to the relative displacements during steady state vibration. These displacement values for each direction and each mode were normalized by setting the largest acceleration as one for each mode and scaled up for visualization. It is possible to glean some important information from the UL-FVT mode shapes alone. When shaking the building in either the east-west direction or the north-south direction, the accelerometer nearest the mass shaker, accelerometer A, always shows the highest accelerations in the shaking direction with a small amount of acceleration in the perpendicular direction. The perpendicular acceleration might be accounted for by the shaker's placement at a location other than the centroid of the tower, but more likely the cause of the perpendicular acceleration is the mode shape itself and the irregular T-section in plan.

When shaking in the east-west direction, there is minimal east-west movement of location D while there is large east-west movement at location A. Accelerations were measured at two different heights in the tower, A and F, and their movements were similarly proportioned between east-west and north-south components. This shows that there is a slight twist occurring in the tower because the shaking is not aligned with the modal axis of the tower. There is a similar amount of movement between locations C and E, and that movement has roughly equal east-west and north-south components. Location B also displays similar east-west and north-south movement, and that movement is larger than the movement at C and E. Compared to shaking in the north-south, the responses at the accelerometers B through E were 2-2.5 times smaller than the responses at accelerometers A and F. This is evidence of a weak and discontinuous diaphragm and out-of-plane dynamic behavior of the walls. The shaker excited the tower into the fog room, bending the adjoining wall and producing a similar response in B and C. A stiffer diaphragm would likely produce a response at D that looks similar to the response at B and C because the diaphragm would transfer the load of the intruding tower and bending wall to the opposite wall. The continuity of the load path to D in terms of both strength and stiffness is largest factor in the small response at that location. An increase in the strength and stiffness of the diaphragm would aid in transferring full inertial loads of the connected elements in the event of strong ground motions. Although the trusses span east-west, they are ineffective in transferring tensile east-west loads between the walls because of the inadequate lateral strength of the connections.

When shaking in the north-south direction, all locations experience a majority of their movement in the north-south direction with only minimal movement in the east-west direction. Locations A and F have similar but not equal proportions of east-west and north-south movement, which again shows there is a slight twist that occurs in the tower. This twist may be attributed to the walls abutting into the tower's base in different directions and the off centeredness of the circular core that runs its height, but more likely it is because the modal axis and shaker axis are not aligned which results in participation of other modes. Locations C and D have roughly equal movement as do locations B and E, only less. In general, shaking in the north-south produced a much greater response than shaking in the east-west with the same force, possibly due to the higher stiffness of the walls in-plane than out-of-plane. The response at B is comparable to the responses at A and F when shaking in the north-south direction but it is much smaller when shaking in the east-west direction. Accelerometer C is located in the same wall plane as accelerometer B and its response is also large compared to shaking in the east-west direction, though it may be lessened because of the large doorway that reduces the wall section considerably between the accelerometer locations. Again, the weak and discontinuous diaphragm and out-of-plane strength of the walls likely explain the small response at D, since strength and stiffness of the diaphragm is low enough that there is practically no east-west response at D. The north-south response at D is roughly equal to that at C, which may be because of the bending in the walls that occurs out-of-plane.

4. Analytical Model

An important component of this research is the finite element modeling of the lighthouse. Once the dynamic behavior of the computational model accurately represents that currently found in the building, retrofit solutions may be implemented into the model and their effects on the dynamic behavior can be assessed.

The lighthouse was modeled in RISA-3D using frame, shell, and solid elements. The focus of the modeling was on the walls because the out-of-plane dynamic behavior is critical to this research and the walls make up the majority of the mass of the building. Meshing the solids in the walls and tower is necessary in order for components to converge to an accurate stiffness and mass distribution and to achieve proper connectivity in corners and doorways. Meshing is also important for modeling the interaction between the tower section's circular brick inner surface and the rectangular sandstone outer surface. The effects on the dynamic behavior of meshing are shown in Figure 6 below, where the coarse mesh is the minimum meshing necessary for proper connectivity of all elements and the fine mesh is the result of breaking the solids into smaller solids roughly one foot by one foot (0.3 m x 0.3 m) in each dimension wherever possible. The fine mesh proved to be satisfactory later on in the modeling process once material properties were also fine-tuned and natural frequencies adequately corresponded with those found in the UL-FVT testing.

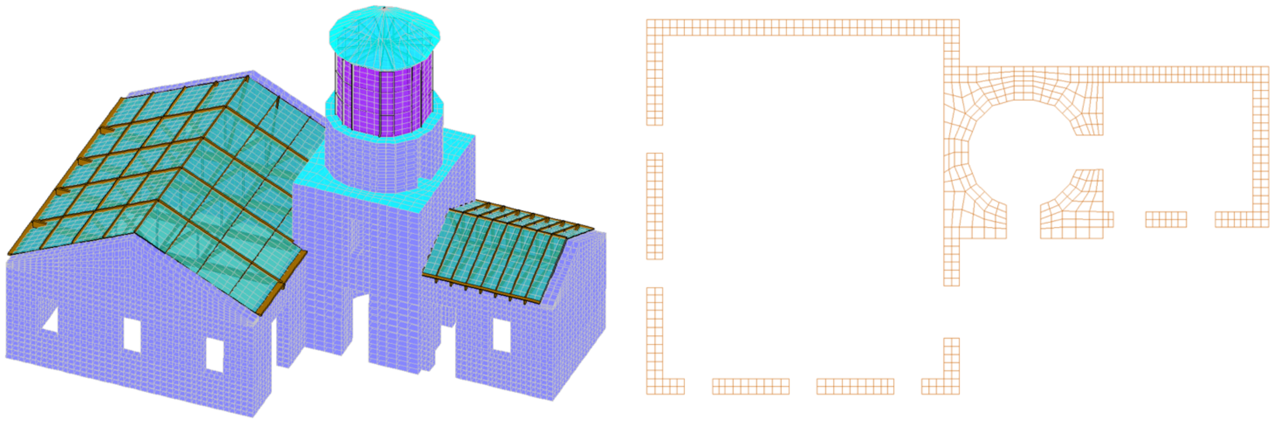

Fig. 6 – Fine Model Mesh in Isometric View and Plan

Table 1 – Coarse and Fine Mesh Modal Frequencies

	Coarse Mesh		Fine Mesh		
Direction	Freq. (Hz)	Period (s)	Freq. (Hz)	Period (s)	ΔFreq. (%)
E-W	32.82	0.030	13.42	0.075	59%
N-S	31.90	0.031	14.67	0.068	54%

Table 1 shows the changes in frequency for the fundamental modes of the lighthouse in both the north-south direction and the east-west direction due to a change in the density of the solid element meshing. The change in mesh size yielded a large change in natural frequency, more than 50% for the fundamental modes in both directions, as well as a change in the order of the fundamental modes.

In addition to the mesh size of the solids in the model, other factors that had a significant effect on the dynamic behavior were the values of the modulus of elasticity for the sandstone and brick. Sourer et al. [7] provide a summary of data from multiple tests for sandstone material properties as well as results from tests of full-scale walls of similar type and construction as those found in the Point Sur Lighthouse. The walls were tested past their peak axial capacities both in and out-of-plane. The walls in the lighthouse are not expected to reach their axial capacities but the range of values for modulus of elasticity is still useful because the walls may have become similarly degraded due to their age. Furtmüller and Adam [8] extracted and tested historical brick and mortar samples from buildings constructed in a range of years that includes the year the lighthouse was constructed, providing the means and standard deviations for brick density and modulus of elasticity among other properties. The purpose of the tests was to determine a more accurate numerical model of the brick's material properties for more efficient structural analysis. The high and low values, shown in Table 3, for the modulus of elasticity of sandstone construction from Sourer et al. were used in an attempt to bracket the true value in the lighthouse and the corresponding effects on the dynamic behavior. Only the mean brick density and modulus of elasticity were used from Furtmüller and Adam.

Table 2: Modal Frequency Range Due to Variance in Parameters

	Sandstone Masonry E = 107 ksi (737 Mpa)		Sandstone Masonry E = 340 ksi (2343 Mpa)		
Direction	Freq. (Hz)	Period (s)	Freq. (Hz)	Period (s)	ΔFreq. (%)
E-W	8.68	0.115	12.26	0.082	41%
N-S	9.48	0.106	13.02	0.077	37%

Table 2 shows the changes in frequency for the fundamental modes of the lighthouse in both the north-south direction and the east-west direction due to a change in the modulus of elasticity of the sandstone. The change in natural frequencies between the two values of modulus of elasticity is close to the estimated value, the square root of the ratio of the moduli of elasticity. The estimated percent change in natural frequency is 56%, while the change in natural frequency for the fundamental modes in the computational model is slightly less at 41% and 37% for the east-west and north-south directions, respectively.

UL-FVT data collected by the authors shows the lighthouse's natural frequency when shaking in the east-west direction is 9.6 Hz and the natural frequency when shaking in the north-south direction is 9.7 Hz. These frequencies were reproduced closely in the finite element model when using the lower limit for the sandstone modulus of elasticity found in Sourer et al, E = 107 ksi (737 MPa). One reason why the order of the modes is not the same between UL-FVT data and the finite element model is because the forced vibration testing was limited to the tower and the out-of-plane modes of the gabled walls were not fully activated by shaking in the tower. The inadequate roof diaphragm and inadequate connections between the walls and roof diaphragm may have also contributed. The mode found by UL-FVT shaking in the east-west direction is similar to modes three and four in the finite element model, where the majority of the tower's movement is in the east-west direction with a small portion in the north-south direction. The mode found by UL-FVT shaking in the north-south direction is similar to modes five and six in the finite element model, where the majority of the tower's movement is in the north-south direction and the walls in the radio room on the right side of the tower also exhibit a large response. However, it is not possible to pinpoint exactly which modes are represented by the UL-FVT data because they may be a combination of the modes identified in the finite element model.

It is difficult to accurately model the dynamic behavior of the lighthouse for a number of reasons. Modeling any existing building, let alone a building that is over 130 years old, is especially difficult because it is impossible to know the exact construction of the building. The simplicity of the lighthouse in plan and elevation was advantageous in this process but there were still assumptions that needed to be made to bring the model to completion. Modeling elements and the connections between those elements was particularly challenging in RISA-3D, but in general the RISA-3D model was able to mimic the response accurately enough for the purposes of this research.

5. Proposed Solutions

Even though the natural frequencies and mode shapes that were produced in the finite element model varied locally from the ones measured with UL-FVT testing, they may be assumed to accurately capture the overall dynamic behavior of the lighthouse. Because of this assumption, the changes in dynamic behavior of the lighthouse after a retrofit solution has been implemented in the finite element model can also be assumed to represent the changes in dynamic behavior of the lighthouse after a retrofit solution has been implemented in the actual structure. Three main retrofit cases were chosen for this research and modeled in RISA-3D: an as-built lighthouse with only strengthened connections which was important to calibrate all finite element models to the measured UL-FVT data, a lighthouse with (2) HSS 8x8x3/8 in. (HSS 203.2x203.2x9.5 mm) strong backs at each gabled wall of the fog room shown in Figure 8, and a lighthouse with (1) HSS 12x6x1/2 in. (HSS

304.8x152.4x12.7 mm) brace near the gable peak and (4) HSS 4x4x1/2 in. (HSS 101.6x101.6x12.7 mm) diamond braces in the roof plane at each gabled wall of the fog room, shown in Figure 9.

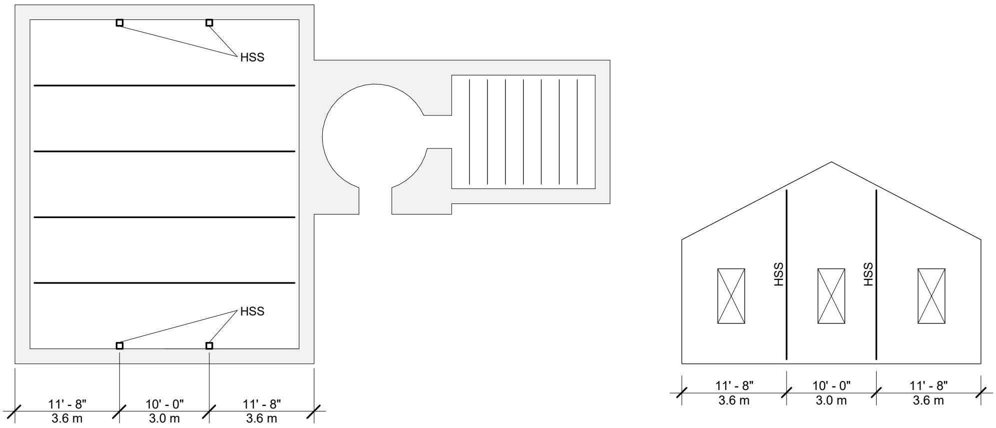

Fig. 8 – Strong Backs in Plan and Elevation

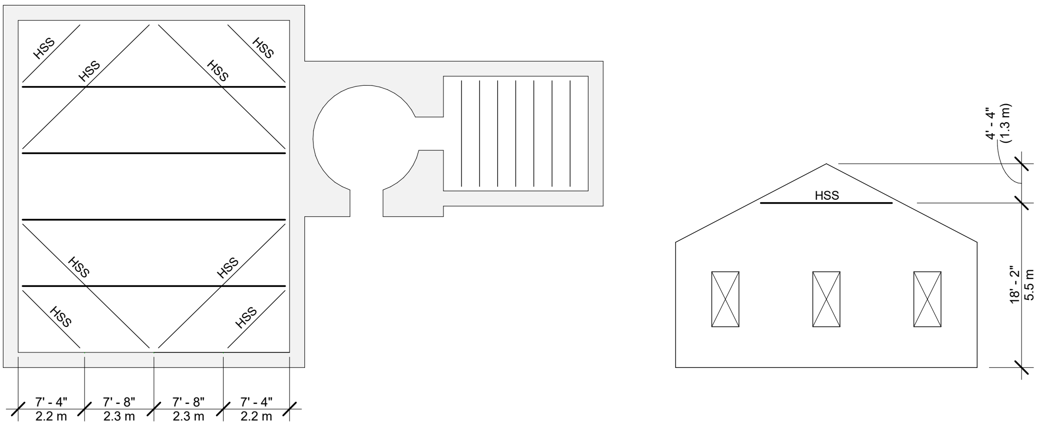

Fig. 9 – Diamond Braces in Plan and Gable Brace in Elevation

While the finite element model of the current lighthouse represents the overall dynamic behavior of the lighthouse as it stands, it is important to note that the strength of the connections in the model is assumed to be greater than what is in the lighthouse currently due to limitations in the finite element modeling program. The finite element model of the unmodified lighthouse is therefore analogous to what the lighthouse would be if only connection strengthening was implemented in the structure. The motivation for installing strong backs at the gabled walls is only to strengthen the walls out-of-plane. The retrofit scheme involving the strong backs did not include any additional strengthening besides the assumed strengthening of lateral connections between the diaphragm and the walls. The main motivation for installing steel members in two of the schemes was to brace the wall sufficiently against out-of-plane movement by using the relatively large stiffness of the tower and perpendicular walls. The brace at the peak of the gabled wall helps to distribute the bracing action of the roof braces evenly across the upper portion of the wall and works to bring the wall's height-to-thickness ratio within the code allowed limit.

The dominant response in the first two modes of all three of the retrofitted lighthouse models is the out-of-plane dynamic behavior of the gabled walls. Higher modes begin to show the effectiveness of the diamond brace scheme in particular. Even though the difference in the effects of the retrofitting schemes can be discerned from the mode shapes, the mode shapes are not the most convincing tool in gauging the overall success of each of the retrofitting schemes. Mode shape comparisons work well to identify differences in the dynamic behavior of components or sections of the building between retrofit options, but they are inadequate to compare the dynamic behavior of the entire structure. One reason for this is that the mode shape represents relative quantities only within the mode shape itself so modes may be erroneously compared at different scales. Normalizing mode shapes to unity is helpful but may also affect the ease of direct comparison, especially between mode shapes of different models.

Another tool that was used to compare the retrofit schemes in addition to mode shapes is the subjection of the models to a linear static design level earthquake to induce displacements with units that are directly comparable. Each of the three finite element models was subjected to the Basic Safety Earthquake-1 for New Building Standards (BSE-1N) according to ASCE 41-17 in each orthogonal direction modeled as a percentage of the self-weight, and the simplified results are shown in Figure 10 below. The goal of subjecting the models to an earthquake was to have a quantitative comparison between the effects of each of the retrofit solutions since mode shapes represent relative values only within each of the modes. The data represented in Figure 10 are the out-of-plane wall displacements along the height of the gabled wall where the gabled wall is tallest.

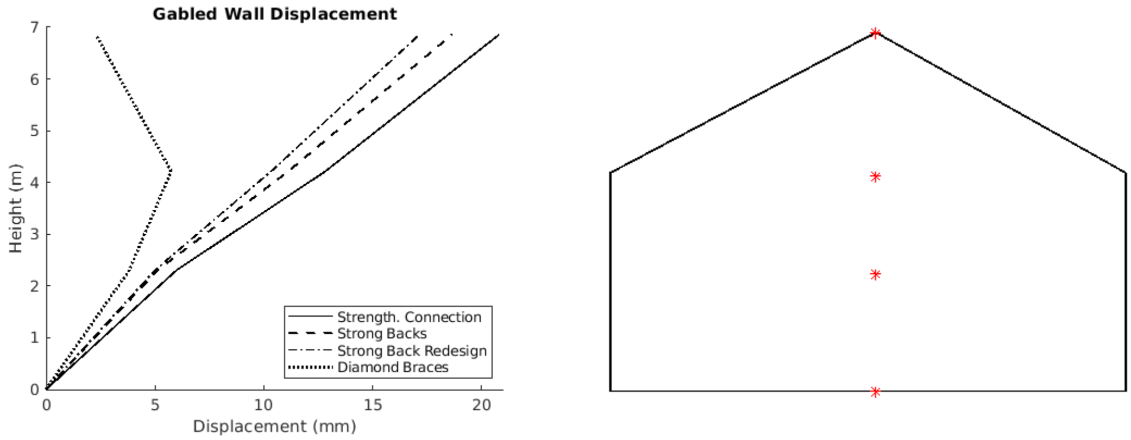

Fig. 10 – Gabled Wall Displacements in Design Level Earthquake

The diamond braces provide significant reductions in displacements at the top of the gabled walls but also along the mid-height of the walls. The out-of-plane dynamic behavior is limited because the large increase in restraint at the top of the wall forces the walls to behave like a simply supported beam rather than a cantilevered beam because the out-of-plane response in the walls is tied back to stiffer portions of the building, specifically the tower and perpendicular walls. The strong backs provide some reduction in displacement though the profile of the displaced wall is nearly identical to the profile of the wall with only strengthened connections. The design of the strong backs was governed by deflection and the members were initially assumed to deflect like a simply supported beam, though the data shown in Figure 10 above shows that the deflection behavior of a cantilever beam is more appropriate. The strong backs were redesigned for deflection according to the deflection behavior of a cantilever and the new design is significantly larger at HSS 14x14x7/8 in. (HSS 355.6x355.6x22.2 mm). The strength the original strong back design provided the wall was sufficient.

6. Conclusions

The goal of this research was to design and analyze retrofit solutions for the unreinforced sandstone masonry Point Sur Lighthouse that would have minimal effect on the historic architectural features of the building, especially the first-order Fresnel lens, while having maximum effect on the out-of-plane dynamic behavior of the unreinforced masonry walls. Other important considerations that factored into the retrofit solution designs were cost and the additional load the retrofit options might place on the lighthouse foundations.

Ultra-low forced vibration testing (UL-FVT) was conducted on the building in order identify and characterize the dynamic response of the building. Evidence of load path deficiencies was found in the minimal dynamic response of locations in the lighthouse on the far side of spanning elements such as roof trusses and the roof diaphragm. The most critical deficiencies were found in the roof diaphragm and the connection to the masonry walls as well as the connection of the trusses to the masonry walls. These deficiencies are in need of retrofit in order to mitigate damage to the building as a whole as well as the Fresnel lens in the event of strong ground shaking. The UL-FVT data was also used to validate the natural frequencies and mode shapes of a finite element model of the lighthouse. The finite element model exhibits similar enough dynamic behavior to the experimental response of the lighthouse that it can also be assumed to accurately predict the effects of any retrofit strengthening schemes should they be implemented. The focus of the finite element model was accurate modeling of the unreinforced masonry walls and tower section because the masonry makes up a majority of the mass of the building. Other considerations were the meshing of the solids and the material properties of the unreinforced masonry. The mesh size and material properties of the solids, which range widely for sandstone masonry, had large effects on the natural frequencies of the building.

All of the retrofit solutions that were considered aimed to improve the out-of-plane deficiencies of the gabled walls and their effects were diverse. It was difficult to quantitatively evaluate the effectiveness of each of the solutions using only the plan view of the mode shapes, however, the elevation view of the mode shapes highlights the differences in the retrofit schemes. Subjecting the lighthouse to a linear static design level earthquake further emphasized the pros and cons of each retrofit solution. The retrofit scheme involving diamond braces in the plane of the roof that tie the gabled walls into the adjacent perpendicular was the only scheme that significantly reduced the out-of-plane deflections of the gabled walls, which was the main focus of the rehabilitation research.

This research will continue with UL-FVT experimentation and finite element modeling when a retrofit solution is implemented in the lighthouse in order to compare the effects of the retrofit solution on the dynamic behavior of the lighthouse with the expected effects exhibited in the finite element model.

7. References

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