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COST-EFFECTIVE IMPLEMENTATION OF NITINOL TO IMPROVE THE SEISMIC PERFORMANCE OF AN UNREINFORCED MASONRY BUILDING

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Abstract. *Congregation Sherith Israel, one of San Francisco's lesser known gems, is an historic unreinforced masonry building with a naturally lighted dome that rises over 100 feet above its centerpiece --- an ornate mural painted vaulted sanctuary that occupies nearly 90 percent of the plan area of the building. Though damaged only modestly by the 1906 earthquake that destroyed many of its unreinforced masonry neighbors, it was subject to a local seismic upgrade ordinance. Various innovative strengthening techniques --- including the first known use in North America of super-elastic nitinol for seismic resistance --- were implemented to supplement the structure's inherent strengths. The design was developed to permit historically significant features to remain virtually undisturbed; construction was completed in 2017.*

The building was required by the municipality to be strengthened to meet seismic safety standards for assembly occupancy. Following years of study, a strengthening plan that was feasible and cost-effective, yet would not harm the historic interiors and exterior, was developed and installed. Construction involved an initial phase of more traditional strengthening in 2010 to maintain occupancy on an interim basis, followed by a second phase of innovative technologies to achieve full compliance.

The strengthening philosophy relied heavily on supplementing, rather than supplanting the existing building's strengths and leveraging its natural dynamic characteristics. The nitinol was designed to provide cost-effective easy-to-install structural "fuses" within an octagonally-configured tension tie system to promote re-centering and control out-of-phase, out-of-plane behavior of the vulnerable gable end walls, parapet and arches that define the main facades. Though the nitinol interconnects all four gable end walls, the octagonal configuration of the system causes no disruption to the domed sanctuary by circumventing the interior dome, and the sanctuary today is in the same condition as before the project. The supplemental seismic work included a fiber reinforced polymer catenary and rocking compression-only pilasters to control displacements without incrementing the demand on its floor-to-wall ties.

1 INTRODUCTION

Tension ties have been employed at least since the Augustan age to supplement the stability of masonry structures. Hoop-type tension rings have been used to resist spreading of masonry vaults and domes since the Middle Ages and through-structure iron ties in direct tension have been in common use to supplement the out-of-plane stability of masonry walls at least since the Renaissance. Such iron ties were often added as retrofit measures after cracking or deformation became concerning but were sometimes installed during original construction [1].

Whether deliberate or not, tension ties in ancient structures participate in the resistance of inertial forces that are generated during earthquake shaking. When they are installed in the upper reaches of masonry walls where local accelerations tend to maximize in any given structure, they can alleviate critical local seismic vulnerabilities, for example, of gabled end walls. Gabled end walls, which are not typically load-bearing and have low gravity-induced compression, are often the first major part of a structure to exhibit severe damage or collapse in earthquakes. However, iron ties are comparatively inflexible relative to the out-of-plane character of masonry walls, meaning that when existing structures are retrofitted with tension ties, local restraints that can be detrimental to the structural response of the masonry can be introduced. Moreover, iron ties are not conducive to promoting re-centering of the masonry; once the iron yields, whatever displacements that caused the yielding are locked-in, and iron is not especially ductile. A structural material with more forgiving properties would therefore be ideal for use in a retrofit tension tie.

Since the architecture of many houses of worship includes gabled end walls, tension ties in these structures are relatively common. Iron tension ties can often be observed transecting the main sanctuary. In addition to the aforementioned potential detrimental interaction, adding ties to a sanctuary that does not have them already can be highly disruptive to the integrity of the space, not to mention to the finishes through which such ties often pass. Means to bypass the sanctuary space without sacrificing the beneficial attributes of tension ties would also therefore be beneficial.

Recently, Sherith Israel, a historic synagogue with a domed sanctuary and gabled end walls in San Francisco, was seismically retrofitted. The centerpiece of that retrofit was an octagonally-configured tension tie system that circumvented the sanctuary space, and the sides of the octagon were connected to the gabled end walls of the structure with nitinol fuses, thus resolving the concerns described above. The system was designed to utilize inexpensive materials and installation methods without compromising performance. The design concept and details are described herein, along with the some of the other strengthening measures implemented in the project, which was completed in 2017.

2 PROJECT CONTEXT

Designed by Albert Pissis, a prominent San Francisco engineer/architect trained at the Ecole de Beaux-Arts in Paris, Sherith Israel was constructed in 1904 and is on the National Register of Historic Places. The exterior shell of the building consists of heavy brick masonry bearing walls that are clad with Colusa sandstone slabs as veneer and rise to roughly 65 feet at the main roof and 75 feet at the gables. At the east, west, and south elevations, these walls are articulated in plan and include large corbelled arch substructures that provide significant out-of-plane stability and support the gable end walls vertically. The building is capped by a steel-framed, zinc-clad, 60-foot diameter, naturally lighted dome and drum that rise to roughly 120 feet above the street and roughly 100 feet above its centerpiece --- an ornate mural painted vaulted sanctuary that occupies nearly 90 percent of the plan area of the building and is large enough to seat nearly 1400 people. [Figures 1 and 2]

Figure 1: Exterior south and west elevations circa 1910
[photo from OpenSFHistory / wnp4.1507.jpg]

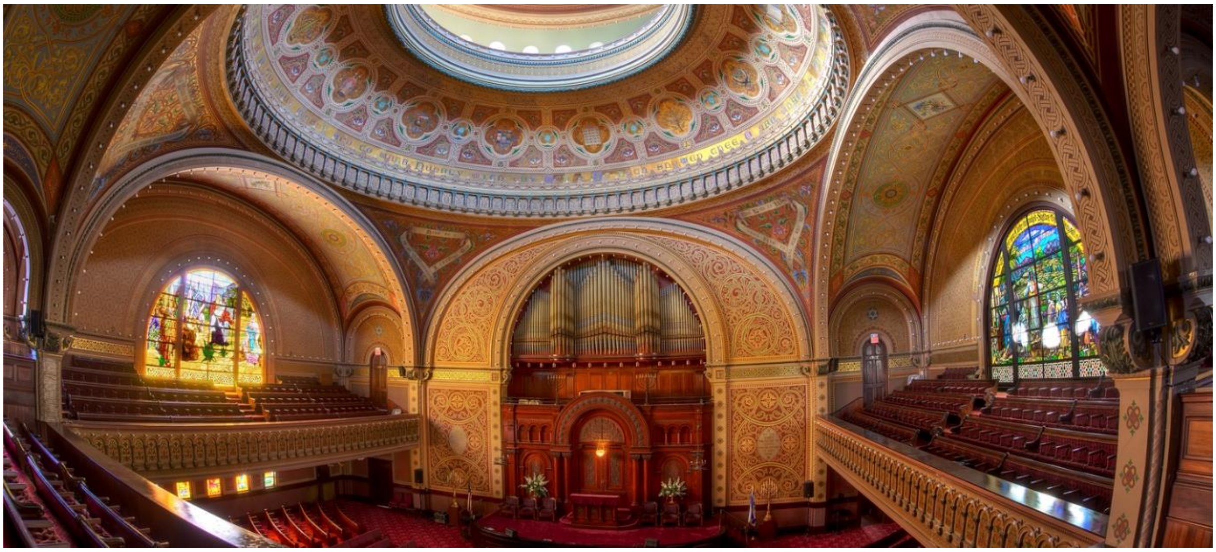

Figure 2: Interior of the sanctuary
[photo by Bruce Schneider Photography, Oakland, California]

The drum and dome are supported on a plan-octagonal system of riveted steel trusses that span to four built-up riveted steel columns; these columns support virtually all of the vertical loads from the wood and steel framing at the roof, the balcony, and the sanctuary levels, except for a roughly 8-foot strip of floor adjacent to the perimeter walls.

A mural painted lath and plaster inner dome that is partially open to permit light from the drum above to enter the sanctuary rises to the elevation of the peak of the gables and is also supported by the octagonal trusses. The interior steel framing engages the masonry shell at only eight locations on each framing level, two on each elevation. [Figures 3 and 4]

Figure 3: Sherith Israel under construction circa 1904
[photo from OpenSFHistory / wnp14.0971.jpg]

The Great San Francisco Earthquake of 1906 exposed the initial failure modes of the building, causing substantial damage to the gable end walls (some of which remains visible), and loss of parapet coping and detachment of the masonry gable end wall from the roof framing on at least one elevation. [Figure 5]

Substantial cracking of interior plaster partitions, ceilings and the inner dome also resulted [2]. Overall though, the seismic response of Sherith Israel was as laudable as the behavior of other Albert Pissis-designed landmark unreinforced masonry structures in San Francisco, all of which survived the earthquake with little structural damage relative to many other buildings in the community, indicating that unreinforced masonry is not inherently high-risk construction if it is well-configured and constructed with high-quality materials.

In the 1980’s, the State of California passed legislation requiring that the seismic risk deriving from unreinforced masonry structures that could not be demonstrated to have certain minimum levels of seismic resistance be mitigated. Effectively, the synagogue’s congregation had a choice: demolish its home or strengthen it to comply.

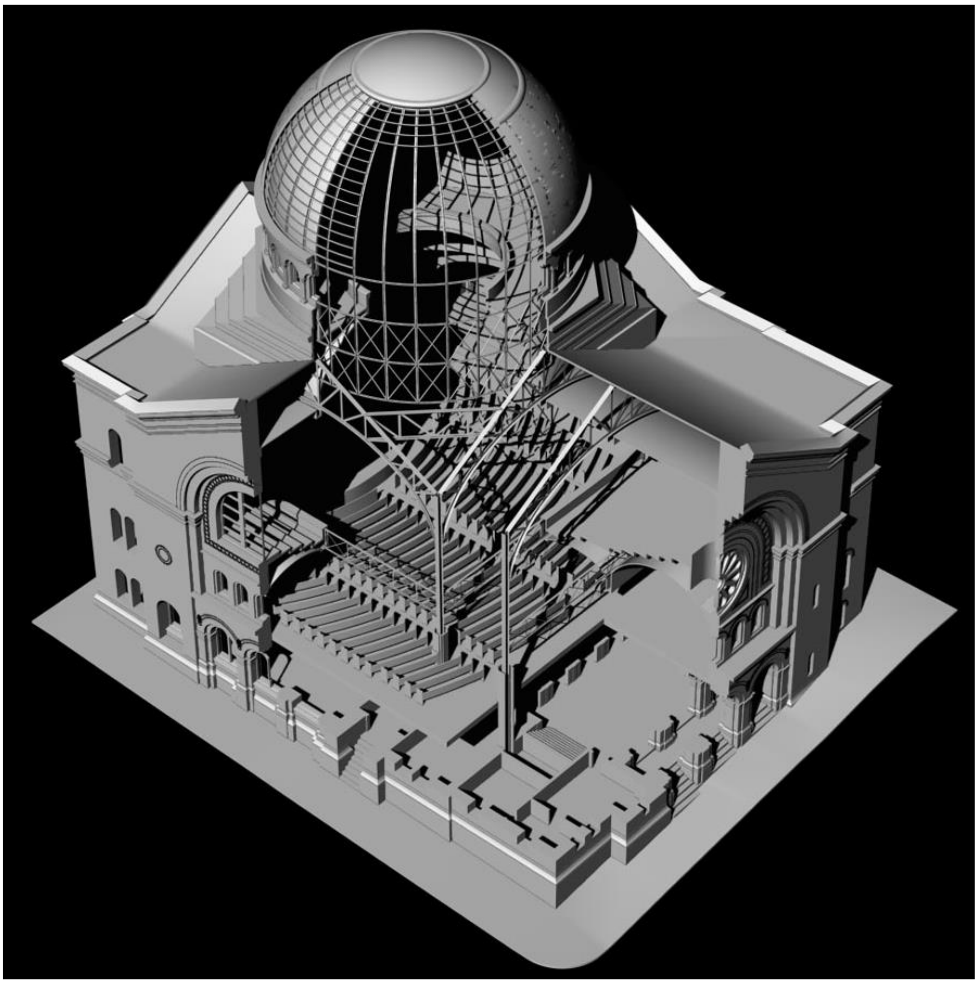

Figure 4: Isometric cut-away model
[rendering by ELS Architecture and Urban Design, Berkeley, California]

3 PROJECT PROGRESSION

The initial stages of the project included substantial documentary research and field investigation to understand the response of the structure to the 1906 shaking and to characterize its constituent construction materials, construction quality and current condition. These efforts will not be described herein beyond noting that aside from localized deterioration from periodic water intrusion during the 20th century, the building was a virtual museum in that many of the manifestations of 1906 ground shaking had gone unrepaired and remained visible in both finished and unfinished portions of the building, which revealed a great deal of empirical data about how this structure responds to ground shaking. The guiding philosophy of the project -- Engineering for Preservation -- has been described elsewhere [3][4].

Figure 5: South cornice undergoing repair after 1906 earthquake
[photo from Bancroft Library, Berkeley, California: https://calisphere.org/item/ark:/13030/hb4m3nb45g/]

3.1 Analysis

Linear dynamic analyses were conducted for various purposes, including to identify primary response modes and dynamic amplification over the building height. These analyses, which were performed using SAP2000 [5], revealed that the structure survived the 1906 earthquake in part due to the dynamic separation between in-plane and out-of-plane response modes of the perimeter masonry walls. These walls contribute roughly 85% of the mass of the entire structure and dominate the building response [6]. This separation results from the flexibility of the diaphragm framing, including the open sanctuary, that permitted a large proportion of the building mass to respond at a fraction of the spectral plateau (i.e., out-of-plane wall behavior). Preservation of this behavior was deemed to be a design priority -- second only to the goal of preserving the historic fabric of the building -- because without dynamic separation between these modes, the in-plane shear strength of the masonry would have needed to be supplemented, which would have itself resulted in widespread disruption of historic materials. The linear dynamic analyses also clearly demonstrated that the dynamic response of the gable end walls is significantly amplified with respect to the input motions and even with respect to the global spectral acceleration. Without explicit consideration of this amplification, the gable end walls would always be vulnerable to earthquake shaking.

Nonlinear adaptive pushover analyses of each of the exterior brick masonry wall elevations were performed using ADINA [7][8]. The analyses improved our understanding of the demands arising from deformation compatibility between the wood floor diaphragms and the plan-articulated masonry walls that is enforced by the diaphragm-to-wall connections necessary to preclude out-of-plane wall failure. The flexible diaphragms permit substantial out-of-plane response of the masonry, meaning that in plan, the masonry walls are subject to behavior somewhat analogous to catenary action and must lengthen to conform to the lengthening of the diaphragm chords. However, because Sherith Israel’s exterior walls are plan-articulated, coincident with lengthening, the walls undergo partial unfolding of the articulations. Understanding this unfolding via nonlinear analysis was judged to be key to designing interventions to maintain the integrity of the masonry during strong shaking. Significant cracking from unfolding during 1906 shaking was still evident in the building attic.

4 PREMISE AND GEOMETRY OF THE TENSION TIE SYSTEM

The design goals for the tension tie system included a variety of considerations, but chief among them were

1. to provide some control on the outward out-of-phase displacement of the gable-end walls and corbelled arch substructures without overly restraining them, i.e. allowing them to deform under inertial loading but limiting the opportunity for the masonry to become unstable;
2. to provide some post-yield re-centering/restorative force such that each cycle of motion would not ratchet the end wall displacement outward, but rather, would encourage the end walls to return to their original position; and
3. to add only relatively marginal cost to the overall seismic improvement project, meaning that the system had to utilize inexpensive components and it had to be readily installable in the attic of the historic synagogue. Sub-requirements for the latter included that all the components of the system had to be light enough to be easily transported to and around the attic, and assembly of the system had to be possible without welding or flame-cutting to avoid risk of fire and without shoring the historic ceilings that also functions as the floor of the attic.

A system composed of steel with nitinol fuses inserted into the load path was deemed to be ideal to accomplish these goals because both the stiffness and the strength of such a system could be readily tuned to the stiffness of the end walls, and because a design was envisioned early on that could use relatively inexpensive nitinol rods and anchoring components. Nitinol, a nickel-titanium alloy, is a superelastic material with heat-activated shape-memory attributes that was developed in the mid-twentieth century.

Until recently, the primary commercial uses of nitinol have been in the optometric and biomedical fields for which nitinol alloys are typically customized and are quite expensive. While structural/seismic uses of nitinol have been explored in university laboratories for decades, until recently, practical applications for nitinol in buildings have been elusive, partly because its hardness makes machining of it both difficult and expensive. After the 1997 Assisi earthquake, nitinol was used to retrofit three ancient structures that were damaged by that earthquake, including the Basilica of San Francisco in Assisi that was the first structure in the world to be retrofitted with a shape-memory alloy device (SMAD) [9][10]**Error! Reference source not found.**. The SMADs deployed on those projects were customized and had to be specially manufactured for their intended use. Since that time, the cost of nitinol -- especially off-the-shelf nitinol product --has become quite affordable considering the great structural advantages it can provide. It is believed that the Sherith Israel project described herein is the first use of nitinol in North America, and the use of nitinol on this project was specifically designed to use simple off-the-shelf products, rather than devices of special manufacture.

The geometry of the tension tie system in Sherith Israel was dictated by the complex geometry of the roof, which consisted of flat areas at the building corners, steep gables at the approximate center of each of the four building elevations, and an inner lath and plaster dome that completely interrupted the line of sight between opposing gabled end walls. The desire to interconnect opposing walls at a common elevation with linear elements was thus at odds with the design goal to not allow new structural elements to intrude through the historic mural painted plaster finishes and across the sanctuary. In this instance, the design goal of preserving the integrity of the sanctuary space won out, and the tension tie system therefore designed to circumvent the inner dome, meaning that opposing walls could not be directly interconnected, but rather were interconnected via an intermediate element.

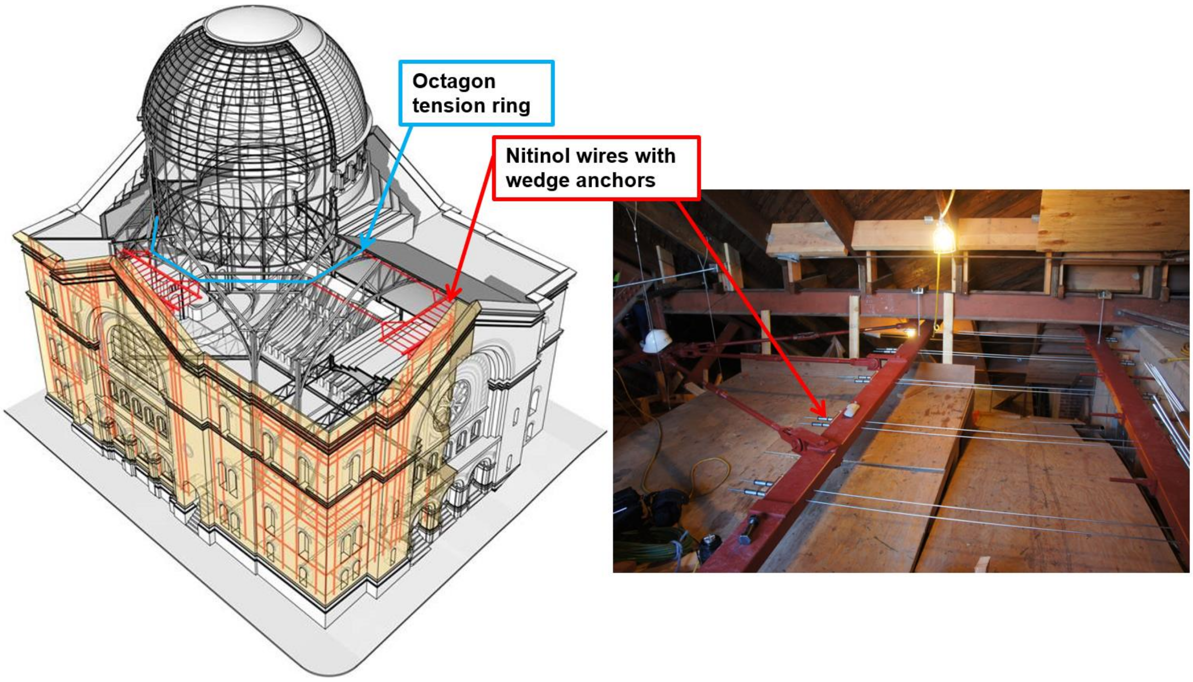

Figure 6: Geometry and installation of the tension tie system
[rendering by ELS Architecture and Urban Design, Berkeley, California]

The intermediate element was an “octagon tension ring” which was comprised of 2-inch diameter Dywidag rods. These rods were selected to balance the requirements of constructability against the desire to make the octagon tension ring as stiff as possible. The ring was located at the single elevation at which any interconnection of the end walls could be accomplished without disrupting the inner dome and still remain within the enclosed attic space; utilizing that elevation necessitated that the interconnecting elements take the form of an octagon that paralleled the existing octagonally-configured truss framing supporting the roof, the drum and the domes. For the tension ring to be installed without penetrating the roof, the new Dywidag rod elements had to be threaded through the only obstacle-free pathways that would allow for interconnection; these were barely large enough for the Dywidags to pass through. Steel plates were sized and shaped to provide “hubs” for interconnecting the Dywidags to each other at each of the eight nodes of the octagon and for connecting other rods that extended toward the exterior walls, along the path of which were inserted the assemblies that held the nitinol fuses [Figure 6].

The nitinol fuses consisted of arrays of 0.190-inch diameter nitinol wire strung roughly 6 feet between hollow square structural steel tubes and anchored on the backside of each tube with spring-loaded wedge anchors that are typically used for anchoring high strength post-tensioning strand. Two spring-loaded wedge anchors were installed at each end of each wire to provide redundancy in the anchorage. One array was installed adjacent to, and anchored to, each of the four exterior walls, with twenty-four nitinol wires installed in each array. The interior side of each array was connected via steel tension rods to its respective nodes on the octagon tension ring. Given their appearance, the arrays were referred to as looms or harps by project personnel. The 6-foot length of the fuses was selected to allow roughly 4 inches of deformation of the wall to occur before the nitinol would begin to experience any permanent set and the cross-sectional area of nitinol in each fuse was selected such that yield of the fuse would occur at a local end wall acceleration of roughly 50 percent of gravity and one inch of outward displacement. The wire diameter was selected to permit wires to be spread more or less uniformly across the width of the gable end walls at the elevation of the tension tie, just above the top of the corbelled arches. Given the properties of the nitinol and the design configuration, outward wall deformations greater than 8 inches should be readily accommodated by the fuses.

5 NITINOL TESTING AND TUNING OF THE FUSES

Nitinol was selected early on in the design process because of its inherent ability to sustain large deformations while remaining elastic, i.e. providing a re-centering force with each cycle of response. Energy dissipation was a benefit but was not explicitly factored into the selection criteria. In general terms, nitinol’s stress-strain relationship includes a linear-elastic range of response to a strain of roughly 1 percent elongation, a superelastic ‘flagging’ response between about 1 percent and 6 percent elongation, followed by a stiffening nonlinear inelastic response to a failure strain typically around 12 percent [Figure 7]. While nitinol has been increasingly used by the optometric and medical communities, those applications are typically highly specialized (e.g., interventional stents), use relatively expensive medical-grade alloys with highly customized and controlled properties, and employ very small-diameter wires that would be inadequate for structural applications.

The superelastic mechanical properties of nitinol can vary widely as a function of the alloying ratios, heat treatment program, and in-service temperature; as such, a robust testing program was developed so that the global behavior of the nitinol fuses could be appropriately tuned for both strength and stiffness over the superelastic range of response and means for reliably anchoring the nitinol could be verified. The diameters and lengths of the nitinol wires were first roughly sized to achieve the target properties using expected nitinol properties for off-the-shelf product; those parameters were then fine tuned based on the results of the testing program, material availability and cost.

The deformation capacity of the fuses was governed by the overall length and the maximum superelastic strain that could be expected from the nitinol wires. The target 4-inch elastic deformation capacity and the typical 6 percent maximum superelastic strain resulted in the need for wires with a gage length of about 67 inches; wires with an overall length of 84 inches were ultimately selected to allow for the doubled wedge anchors at each end. This gage length is expected to yield superelastic behavior, energy dissipation, and a net restoring force to the masonry walls over a deflection range up to 4 inches.

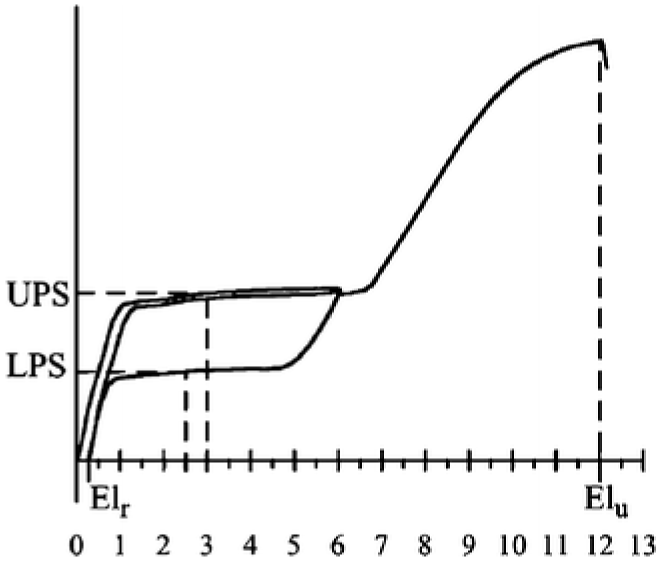

Figure 7: Typical stress-strain behavior of nitinol [11]

The quantity and diameter of nitinol wires that were used in each fuse was selected such that the total force resisted over the superelastic range of response would be roughly equivalent to 50 percent of the weight of the tributary wall area. During the initial design phase, the estimated superelastic upper plateau stress ("UPS" in Figure 7) of 70 ksi --- which is, in actuality, highly dependent on the specific nitinol alloy, the heat treatment process, and the actual ambient temperature --- resulted in the need for twelve ¼-inch-diameter wires at each nitinol fuse.

Testing to confirm and fine tune the preliminary design progressed generally in two phases: an initial feasibility study during the early design phase in 2007, and a later study in 2016 to fine-tune the selection of both diameter and alloy / heat treatment. Both phases of testing included study of means to anchor the nitinol wires, as nitinol is both very hard and difficult to machine. The first round of testing generally included cyclic tension tests of ¼-inch diameter wires to different levels of elongation in the superelastic range of response and at temperature ranges that were expected in the attic of Sherith Israel (45F, 72F, and 90F) [Figure 8]. The results of the testing program confirmed that the length preliminarily chosen for the nitinol wires would provide the anticipated deformation capacity, the diameter and quantity of wires would provide the target restoring force on the perimeter bearing walls, and that wedge anchors would provide a reliable means to transmit the seismic-induced forces into the nitinol. The exact nitinol formulation used in the 2007 testing program was no longer available for the final installation in 2016, so the design using twelve ¼-inch-diameter wires was updated with a design using twenty-four 0.190-inch-diameter wires at each fuse. The 2016 testing program was used to confirm that the actual nitinol wires selected would behave as intended and that the wedge anchors would provide sufficient anchorage at forces and deformations exceeding those expected to occur during an earthquake.

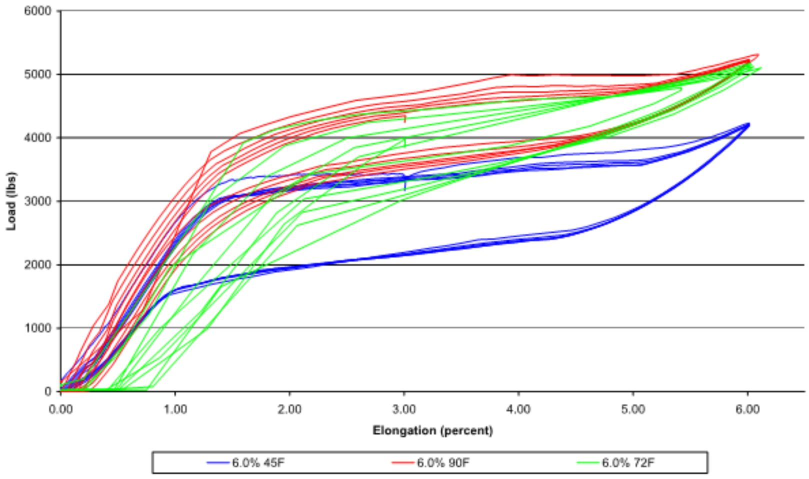

Figure 8: Stress-strain behavior of nitinol wires at a target elongation of 6%

It is worth noting that while the process for selecting and testing of the nitinol wires is described above as being rather neat and linear, the actual selection process was more circuitous. The material that was used during the initial phase of testing in 2007 was no longer available from the original nitinol supplier, so in the later phase of work, an alternate supplier had to be secured; at least three different wedge anchor products were tested and rejected due to inadequate force transfer or slipping of the grips during testing; and the later phase of testing included several different diameters of wire and several different suppliers, as some of the nitinol that was being provided did not achieve the target stress-strain behavior. This selection process served as an important reminder that the use of novel materials or systems requires additional thought and effort prior to implementation.

12 CONCLUSIONS

The historic temple, Congregation Sherith Israel, one of San Francisco’s century-old gems, was constructed largely of unreinforced brick and stone masonry circa 1904 and survived the Great San Francisco Earthquake of 1906 with relatively modest damage compared to other nearby buildings, many of which suffered partial or complete collapse. Nonetheless, the structure was recently required by the municipality to be seismically improved. Out-of-plane behavior of the unreinforced masonry perimeter walls and the gabled ends, which were damaged in the earthquake, was confirmed analytically and by on-site observations of the vestiges of damage from historic earthquakes to be one of the primary vulnerabilities of the structure.

Among various other interventions, a novel, highly cost-effective, tension tie system with nitinol fuses was designed to specifically target this primary vulnerability by interconnecting all four perimeter walls to mitigate the propensity of these walls to move outward, and to introduce re-centering attributes into the system. This was the first known seismic application of this superelastic material in the United States. To avoid disruption to the temple’s ornate mural painted sanctuary, whose inner dome rose to the elevation of the tops of the gable end walls, the tension tie system was designed to circumvent the inner dome and to be fully contained within the attic space using an octagonal tension ring.

Cost-efficiency and installation-efficiency were primary considerations. All components of the tension tie system were selected such that they could be hand-carried and easily installed without the need for any welding or torch work, and without heavy machinery or disruption of any interior finishes. The nitinol fuses were also designed to use “off-the-shelf” material that required no customization and no machining, and to use readily available means for anchoring.

Based on an extensive laboratory testing program, the individual nitinol rods comprising the fuses were sized both in length and diameter to limit --- but not prevent --- outward deflection of the perimeter walls, while at the same time providing a restoring force to help limit residual wall deflections after an earthquake.

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