Seismic Considerations for Guastavino Ceiling, Vault, and Dome Construction

DOUG ROBERTSON

The seismic and program improvements to the Hearst Memorial Mining Building on the University of California, Berkeley campus, will be the first project in which Guastavino construction is deliberately strengthened for improved seismic performance.

Guastavino construction is found predominantly in turn-of-the-century architecture across the eastern United States, with fewer examples as you travel further from New York City, the home of the Guastavino Company. There are very few examples of Guastavino construction in California, where frequent earthquakes pose a risk to this brittle architectural system. Within the San Francisco Bay region there are three known examples of the Guastavinos' work: the San Francisco Stock Exchange, Grace Cathedral in San Francisco, and the Hearst Memorial Mining Building on the University of California, Berkeley campus.

Similar forms of unreinforced masonry vaulted and domed construction are prevalent in older architecture in the United States and abroad. Investigation of some of these buildings and literature searches reveal relatively little about the seismic behavior of this type of construction. Little analysis and testing of vaulted and domed masonry construction has been done, and its performance in past earthquakes has been seldom and poorly documented.

The findings presented in this article are based on investigations and evaluations related to the seismic behavior of various examples of masonry vaulted and domed construction and, in particular, field investigations and testing of the vaulted ceiling in the Hearst Memorial Mining Building (Fig. 1).[1]

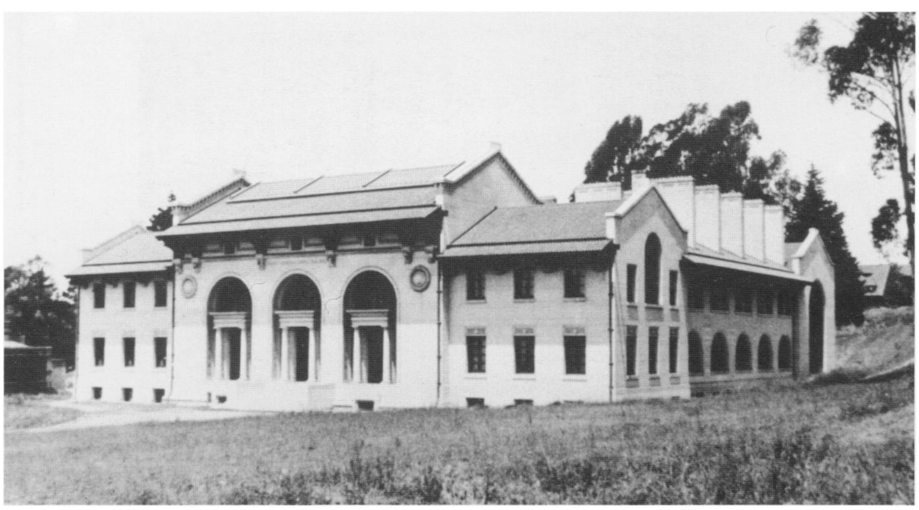

Fig. 1. Hearst Memorial Mining Building. Courtesy of Bancroft Library, University of California, Berkeley.

Historical Perspective

To place the seismic vulnerability of Guastavino construction in perspective, it is useful to first consider the seismic performance of other forms of unreinforced masonry construction in past earthquakes. Following is an overview of the seismic performance of masonry buildings, the behavior of conventional brick vaulted and domed masonry construction, and knowledge of the seismic behavior of Guastavino.

Unreinforced masonry. The majority of earthquake engineers and researchers consider unreinforced masonry buildings to be the most hazardous form of building construction. Over many years, earthquake after earthquake has reaffirmed this view. The poor seismic performance of this type of building is due to many factors, including the brittle nature of the materials, the non-homogeneous manner in which the materials are used, deficient workmanship, and design and detailing that inadequately consider the effects of earthquakes on building construction.

The "gluing" together of masonry pieces with mortar and the strong yet brittle nature of masonry materials make this type of construction subject to potentially sudden and catastrophic failures under dynamic earthquake forces. For this reason, "unreinforced"

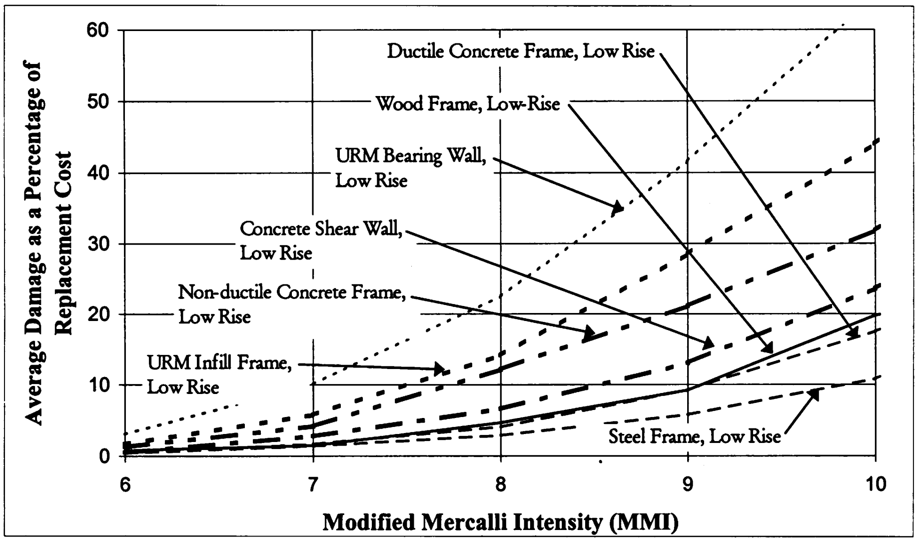

Fig. 2. Damage projections for various building types. Illustration by author.

masonry construction is no longer permitted by modern U.S. building codes in zones of moderate to high seismic risk.

Fig. 2, taken from ATC-13[2], shows the relative performance of masonry buildings compared to other lateral building systems. Performance is measured in terms of damage projections accessed as a percentage of building replacement cost. Performance was considered at different levels of ground-shaking intensity using the Modified Mercalli Intensity (MMI) scale.

These estimates were developed by polling a substantial number of earthquake experts with a systematic methodology. The chart suggests that masonry performs poorly compared to all other materials and systems.

Masonry vaults and domes. Relatively few records exist of past performance of vaulted and domed masonry construction in earthquakes. There are a number of possible reasons for the limited historical record. This type of construction is often found in regions of the world with relatively low seismicity or in third-world countries where earthquake damage is poorly documented. Probably the most important reason is that failure of this type of system is thought, in many instances, to lead to more significant damage or collapse of the unreinforced masonry structure. Damage to the vaulted or domed roof or ceiling construction then becomes difficult to identify, and the interest in its behavior is of little concern compared to the general structural collapse, concern for rescuing potential survivors, and the urgency to rebuild.

One type of system employed by the Guastavinos that is prone to earthquake damage was used extensively throughout many European cities and elsewhere in the world. This system, sometimes referred to as "jack vault," used bricks or clay tile to form repetitive vaults spanning between the bottom flange of regularly spaced steel wide-flange beams (Fig. 3).

There are three primary reasons that this jack vault system is vulnerable to earthquake damage. The most common deficiency is that the walls are not positively connected to these relatively rigid vaulted floor, ceiling, and roof systems. In an earthquake, the walls pull away from the vaults, leading to collapse of either the walls, the vaults, or both. The walls parallel with the steel I-beams are particularly susceptible to this type of damage since, unlike the walls that support the steel beams, there are no beams to provide a nominal tie to the walls. These vaults also often have a shallow radius, which make them prone to earthquake damage when the supporting, unbraced, steel I-beams spread laterally. Lastly, these vaults often support loose, heavy fill materials, such as dirt, ash, or rubble, which in an earthquake add to their inertial forces and their vulnerability.

In his speech at the 1893 World's Fair on "cohesive construction," Rafael Guastavino referred to the Persians as the "fathers of the cohesive mode of construction."[3] Ironically, it is in this region of the world where several examples of failure of this system have been documented. In 1990, widespread damage occurred in a Richter magnitude 7.3 earthquake in northwest Iran that killed an estimated 35,000 to 50,000 people and damaged about 100,000 buildings. Measured peak ground acceleration

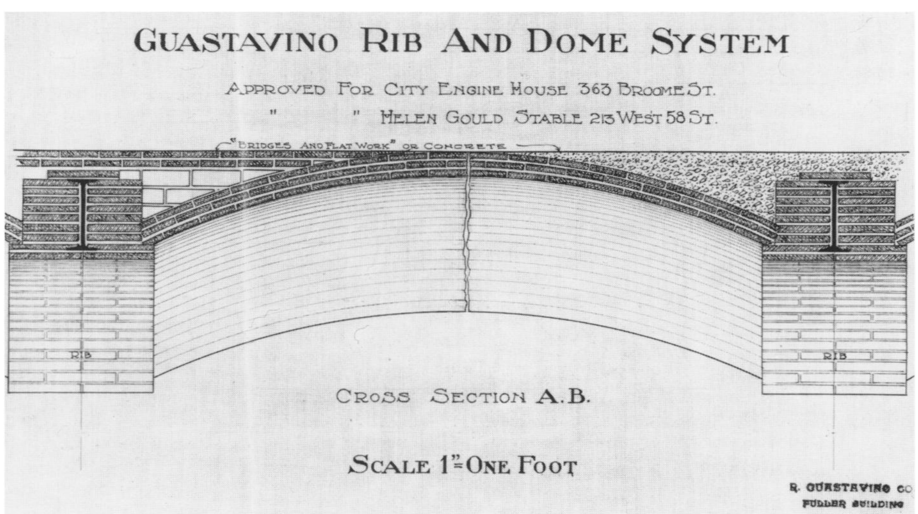

Fig. 3. Jack vault. Courtesy of the Guastavino/Collins Archive, Drawings and Archives, Avery Architectural and Fine Arts Library, Columbia University.

varied from 0.02g to 0.65g in the areas surrounding the epicenter. Many buildings with this type of vaulted masonry/I-beam system collapsed. These collapses were reportedly precipitated in most instances by the walls pulling away from the vaults.[4]

In March 1997 another earthquake in northwest Iran, with a magnitude of 5.5, caused widespread damage, killing 965 people and injuring more than 2,600. None of the recently engineered steel or concrete buildings experienced any noticeable damage. However, many of the buildings with this repetitive brick vaulted roof system collapsed.[5]

The most widely recognized and notable example of earthquake damage of a vaulted or domed structure was the damage to the ceiling in the Basilica of St. Francis of Assisi in September 1997. Accelerometers indicated ground accelerations of 0.2g or greater. Subsequent engineering analysis indicated that the capacity of these vaults would be reached at a lateral acceleration of about 0.2g.[6]

Damage was certainly partly attributable to the increased mass of loose fill above the vaults and also the building's geometry. The ceiling vault collapse was in an area referred to by earthquake engineers as a "reentrant corner," where a significant change in the plan of the building occurs at an inward corner, in this instance, at the transition between the basilica's nave and transept (Fig. 4). This condition is recognized by engineers to be particularly vulnerable to earthquake damage. The failure of these particular pendentives may have been avoided or their performance significantly improved by adding cross building ties at this location. This type of building, with its tall and massive exterior walls, is particularly vulnerable to earthquakes. The lateral forces produced by an earthquake cause the walls to pull away from the vaulted/domed ceilings, leaving the roof and ceiling without support.

Available documentation seems to indicate that all of these examples of vault collapse involved vaults constructed of brick and not clay tile. Brick vaults have been observed to normally include a single course of brick compared to the multi-course technique used in Guastavino construction. These vaults are also often constructed in countries where the quality of construction may not be well controlled. Though these failed vaults may not have been consistent with the quality and construction methods used in Guastavino construction, their failures are still useful in beginning to understand the potential seismic risks and vulnerabilities of vaulted and domed construction.

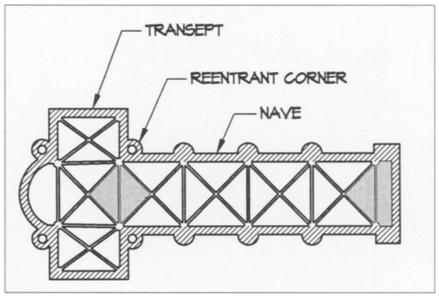

Fig. 4. Plan view of the Basilica of St. Francis of Assisi. (Gray areas indicate collapsed zones.) Illustration by author.

Guastavino construction. There is little information available on the seismic performance of Guastavino construction, yet there are a few indicators that poor performance could occur. At Grace Cathedral in San Francisco, the Guastavino ceilings experienced some damage in the 1989 Loma Prieta earthquake.[7] The cathedral is founded on a rock site and is about 55 miles from the earthquake epicenter. The level of shaking was therefore relatively low, with peak ground acceleration of about 0.15g. Yet, even at this low acceleration, some cracking was reported along the ridge of four pendentives. This cracking, visible from the top, caused fragments of mortar to fall to the floor below. Strengthening using Gunite placed from the backside of the ceiling had been proposed in the 1950s or early 1960s, but this work was never carried out.[8,9]

Another example of Guastavino construction is found in the Hearst Memorial Mining Building. The building site is located roughly 60 miles from the Loma Prieta earthquake epicenter and experienced an estimated acceleration of about 0.1g. The building was completely undamaged except for a couple of fallen Guastavino tiles.

The strength of Guastavino under gravity loading is generally undisputed. George Collins pointed out one important difference between the timbrel vault and the more conventional stone-masonry vault. He noted that the vault "is very thin, consisting of little more than a surface, and derives its rigidity not from massiveness or thickness but rather from its particular geometric form."[10] This statement explains precisely why the Guastavinos' many works have performed so well over the past century. However, it must be emphasized that Guastavino construction derives its substantial strength not just from its vaulted form but also from its geometric orientation relative to the direction of loading. While Guastavino construction has performed admirably over the past century under the forces of gravity, its performance when subjected to the added lateral or vertical loads from an earthquake has not yet been adequately tested.

The Guastavino System

Although masonry construction has proven very vulnerable to earthquake damage, the Guastavino system generally offers a number of advances over more conventional stone and brick vaulted and domed construction. Rafael Guastavino was a master of design as well as construction, often implementing details of construction that added strength and redundancy helpful in reducing the effects of earthquake forces. Photos and drawings indicate that he generally detailed and provided well-anchored and rigid boundary conditions, which in an earthquake help prevent the vaults from spreading laterally and losing support. Although these details were likely implemented to improve the gravity-loading behavior of the vaults and domes, they may also improve their seismic performance.

The Guastavinos also used multiple courses of tile with overlapping mortar joints. Though this adds more mass to the system, the overlapping joints can provide improved strength. In order to achieve Guastavino's "cohesive construction," the bond between courses of tile and mortar is essential, particularly under earthquake loading. Guastavino

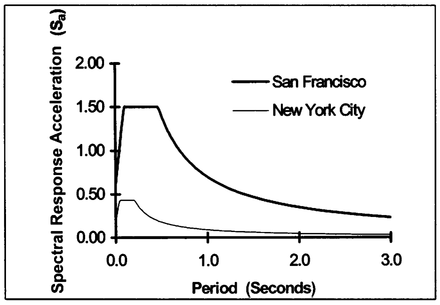

Fig. 5. Earthquake response spectra for New York and San Francisco. Illustration by author.

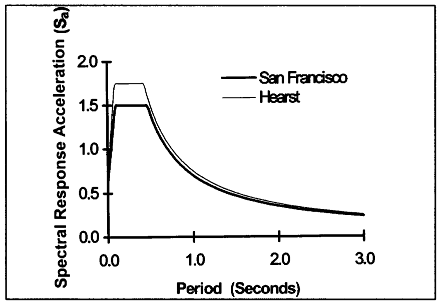

Fig. 6. Earthquake response spectra for San Francisco and Hearst. Illustration by author.

recognized the importance of this bond, noting the "danger of sliding in the horizontal joints, only in case strong cements are not used."[11]

The Guastavinos also frequently added extra tiles to form stiffening ribs at a repetitive spacing or at changes in geometry. Above the vaults they added supplemental steel members to brace vault ribs and boundaries and to provide cross ties to prevent the supports from spreading. They also used cement mortar except at the joints of the face tile, where plaster-of-paris mortar was generally used for quick set time.

The implementation of these details should generally help to improve the seismic behavior of Guastavino construction. However, Guastavino's system remains very brittle and potentially hazardous when subjected to dynamic earthquake forces.

Guastavino Seismic Evaluation

To determine whether seismic strengthening should be implemented as a part of the preservation strategy on a specific project, a number of risk factors should be considered. These include the level of seismic hazard associated with the building site, the capability of the primary building structure to resist seismic forces, the level of force that can be transferred through the building structure to the Guastavino system, the boundary conditions of the Guastavino system, and other specific details of the particular installation.

The seismic hazard at any given building site depends on the region's geotectonics and the site soil characteristics. The sites proximity to active faults, the likely fault rupture mechanism and depth of rupture, and the type and depth of soil all play a part in determining the probable earthquake response.

Some of the buildings that contain Guastavino construction are themselves susceptible to earthquake damage if left unstrengthened. It may make little sense to strengthen the Guastavino construction, only to have the supporting structure severely damaged in an earthquake. However, Guastavino is often found in high-occupancy public spaces, where the life and safety of building occupants may be threatened more by falling tiles then by damage to the primary structure.

The magnitude of earthquake forces imposed on any building system is first dependent on the rigidity of the global building structure. More rigid structures will experience higher accelerations and more flexible structures lower accelerations, due to their period of response. Secondly, the forces imposed on a Guastavino system are dependent on its proximity and lateral rigidity relative to other horizontal roof and floor diaphragm construction. For instance, a rigid concrete roof located directly above the Guastavino system may attract much of the lateral force, reducing the loads on the Guastavino elements, whereas a flexible wood roof diaphragm in the same proximity will take relatively little load compared to the more rigid Guastavino. In cases where the Guastavino system serves as the sole roof or floor diaphragm system, it may provide the only lateral load path for resisting seismic forces.

As in the jack vault system, the boundary conditions of Guastavino construction are generally very important to its seismic performance, not only the boundary connection of the Guastavino itself but also the strength and rigidity of the surrounding support structure.

The one characteristic relevant to all Guastavino work is the vault span-to-thickness ratio. The thickness, which depends on the number of tile and mortar courses, will influence the rigidity and strength of the system. In order to take advantage of this multi-layer shell, it is important that the courses of tile and mortar have sufficient strength and bond to transfer stresses. Failure to maintain this monolithic behavior can lead to significantly greater seismic stresses and deformation, making the system much more susceptible to failure. The specific, and often unique, details of each Guastavino application will lead to variable seismic behavior. Therefore, the Guastavino system and details in each building must be evaluated individually.

Two examples of earthquake response spectra, developed using the NEHRP *Guidelines for the Seismic Rehabilitation of Buildings,*[12] are shown in Fig. 5. One spectrum is applicable to New York City and the other to Northern California for the maximum considered earthquake (MCE) on a Class B rock soil site. Since no reduction factor "R" is permitted by NHERP for unrein-

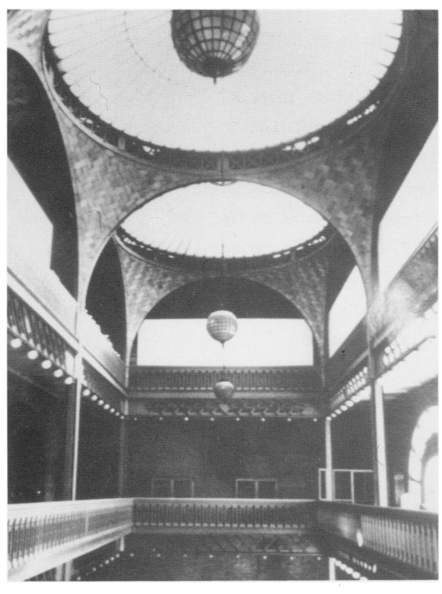

Fig. 7. Hearst Memorial Mining Building Guastavino ceiling. Courtesy of Bancroft Library, University of California, Berkeley.

forced masonry construction such as Guastavino, design forces can be determined directly from these spectra based on building period. Most of the buildings that incorporate the Guastavino system rely on a masonry or concrete shear-wall lateral system, which leads to a relatively short building period. Therefore, in most instances the seismic response of Guastavino falls at the plateau of the spectrum where the acceleration is the greatest.

The plateau of the New York spectrum reaches a peak horizontal ground acceleration of 0.4g or 40% of the force of gravity. Though this does not seem particularly high compared to the California spectrum, which reaches 1.5g, these smaller earthquake forces should not be dismissed too hastily. The Guastavino ceiling in the Grace Cathedral experienced some cracking at accelerations of only about 0.15g in the Loma Prieta earthquake, and partial collapse to the Basilica of St. Francis of Assisi occurred at an estimated acceleration of 0.2g.

Preservation and Seismic Protection

Do preservation and seismic-risk-reduction goals conflict? Many preservation purists, even in California, believe that the often-invasive nature of seismic strengthening is entirely inconsistent with the goals of historic preservation. However, this perspective may be slowly changing. There are no assurances that monumental unreinforced masonry buildings, left unstrengthened, will be safe from damage or destruction when the next major earthquake occurs. One need only look at the 1994 Northridge or the 1995 Kobe earthquake devastation to understand the potential for economic and human loss when such an event occurs.

The Hearst Memorial Mining Building

Background. The Hearst Memorial Mining Building, located on the University of California, Berkeley campus, was designed in the Beaux-Arts style by campus architect John Galen Howard and was dedicated in 1907. Constructed as a mining and mineral engineering building, it now houses the Department

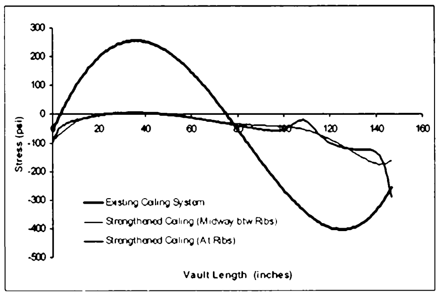

Fig. 8. South side vault: hoop stress at back of face tiles. Courtesy of William Kreysler and Associates.

of Material Science and Mineral Engineering.

In 1978 the first seismic evaluation of campus buildings classified Hearst's seismic safety as "very poor." In recent years, its outdated facilities have also fallen behind its evolving high-tech teaching and research mission. In 1993 preliminary studies were undertaken to consider seismic and program improvements for the building, design was officially begun in 1996, and construction is now underway. The building is to be placed on a system of base isolation bearings, which includes 134 high damping rubber bearings and 24 fluid viscous dampers (12 in each direction), all placed below the existing first-floor level. The base isolation system will greatly reduce the lateral acceleration of the building in an earthquake.

Seismic design criteria. In Fig. 6, the site-specific spectrum for the Hearst site is compared with the spectrum for a more typical northern California rock site (NEHRP soil Class B). The Hearst spectrum has higher accelerations than the typical spectrum because Hearst is considered a "near fault" site. More specifically, the is building within 800 feet of the active Hayward fault.

Without base isolation, the high spectral acceleration represented by the Hearst spectrum plateau would be used for seismic design. Base isolation lengthens the building period, represented by the declining portion of the spectrum, thus reducing the building base shear from about 1.8g at the spectrum plateau to 0.25g at a design period of three seconds. Although isolation reduces the horizontal acceleration, it does not reduce the vertical acceleration associated with proximity to a fault.

The seismic design criteria for the Hearst Guastavino ceiling was thus driven by two primary factors: first, base isolation significantly reduces the lateral acceleration, and second, isolation does nothing to reduce the near-fault vertical acceleration. The design vertical acceleration considered in analyses was 2.5g. These forces were derived from site-specific time history analyses for the building that were used for design of the isolation system.

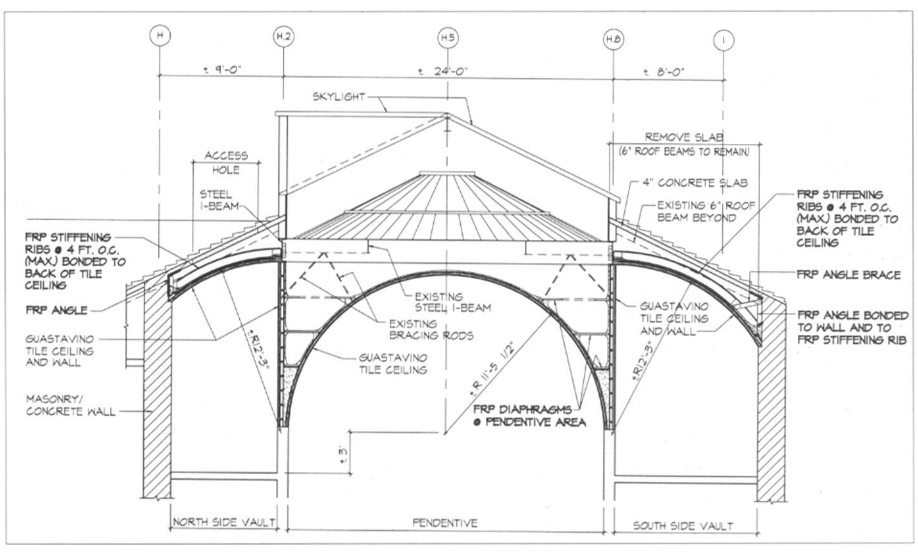

Fig. 9. Section of vaulted ceiling strengthening. Courtesy of William Kreysler and Associates.

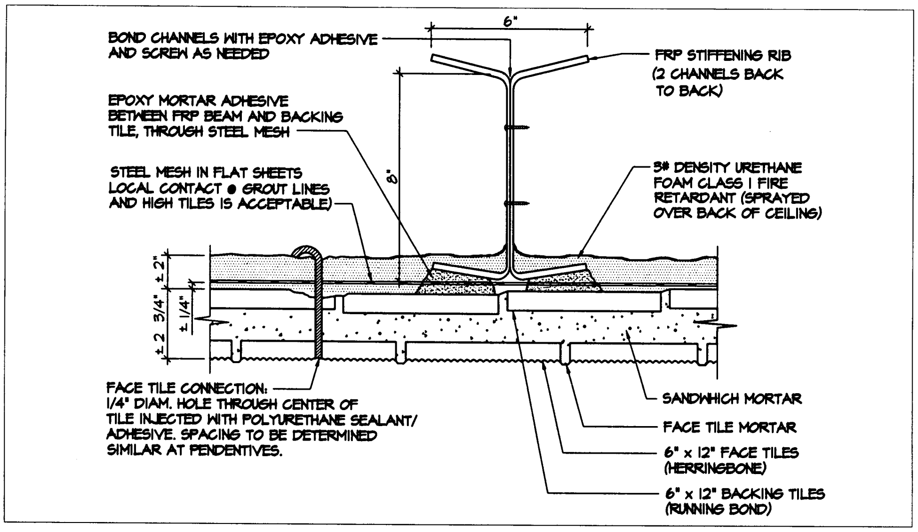

Fig. 10. Detail of cross section at vault stiffening rib. Courtesy of William Kreysler and Associates.

Seismic evaluation. In 1993 the first investigation of the Guastavino ceiling included a sounding study of the area above the galleries (Fig. 7). This technique has become the commonly accepted method for evaluating the adhesion between the face tiles and the backing mortar/tile substrate. A wooden mallet was used to detect a hollow ring sound, which was assumed to indicate marginal or compromised bond between the face tile and the mortar substrate. Several qualifiers regarding the accuracy of the sounding were provided in the testing report.[13] The report indicates that the "Interpretation of the sounds is somewhat subjective " and also points out that "Tiles were generally sounded in only one area (usually near the center of the tile)" and that indications sometimes vary between different parts of the tile. This study indicated only about 5% of the tiles have questionable bond to the substrate. Therefore, it was assumed in early design studies that the bond between layers of tile and mortar was generally sufficient to maintain the system's monolithic behavior under seismic forces.

Last year, a second investigation was carried out to identify boundary conditions, determine material properties, and confirm by more explicit testing techniques the bond strength between the three-layers of Guastavino construction. These layers include a single mortar layer sandwiched between two layers of clay tile, a rough corrugated face tile laid in a herringbone lay-up, and a smooth tile backing layer placed in a running-bond lay-up. As discussed previously, this bond between alternating layers of tile and mortar is essential to transfer lateral stresses, which under normal gravity load are lower and less important to the ceiling stability. To assess the bond strength, direct tension tests were conducted. The testing was accomplished from the backside of the vaults and pendentives after locating face tiles with expected good adhesion by sounding from the underside. Testing was performed by drilling 3-inch-diameter cores from the backside, penetrating only to the back face of the face tile. A steel plate connected to a threaded rod was then bonded to the back of the core. Next, a tension load was applied to the rod to determine the tension capacity of the core and to locate the failure plane. This was considered the least invasive testing method that would provide a measure of actual bond strength without impacting large areas of tile or the appearance of the ceiling.

The results of this testing were unexpected and conflicted with the assumed conditions based on the earlier sounding study and descriptions of the "tenacious" nature of the mortar bond between layers of tile as described by George Collins.[10] The majority of tension tests suggested that the bond between layers was generally negligible. Of the thirty tests, twenty showed negligible bond, while the remaining ten tests had widely varying tension capacities ranging from 13 to 99 psi. The failure plane between tile and mortar appeared random, with about half of the cores failing at the face tile to mortar plane and the other half occurring at the mortar to backing tile interface.

An initial concern about this testing method was that the coring might cause vibration or stresses that might initiate failure, leading to unreliable test results. A similar core sampling approach for testing shear capacity in brick walls was abandoned more than ten years ago for this reason. However, the coring reportedly was very smooth, without vibration, and it is believed that the manner and load at which the cores failed were largely due to the weak bond between layers of tile and mortar. Other evidence appears to support this conclusion. Inspection of the cores following testing generally revealed a clean break at the tile to mortar interface, with no tile or mortar remnants adhered to the opposing half of the core sample. Material testing of the tile and mortar revealed reasonably high strengths, confirming that test failures were not attributable to material failure. Mortar and tile compression strengths ranged from 4,396 to 5,871 psi and 3,017 to 6,552 psi, respectively. At one location the mortar was carefully removed around one tile that had been first sounded to confirm reasonable bond. However, the tile fell from the ceiling under its own weight once the surrounding mortar was removed. Although these results do not provide conclusive evidence, they do cast some doubt on the reliability of sounding for determining the conditions of face tile and backing mortar bond. One can conceive how a tile with a well-filled collar joint and tight edge joints could produce a solid sound, as if well bonded, while actually having no bond whatsoever. This technique also provides no assurances of the condition of bond between subsequent layers of tile and mortar.

The weak bond between tile and mortar courses could be attributable to a number of factors. During construction, the masons may have failed to wet the in-place tile or mortar prior to placing the subsequent course. As the next course was placed, the surface moisture may have been absorbed by the new dry materials, drawings the moisture away from the bond interface and leading to a weakened joint. This is a common failing in masonry construction even today. Another explanation may be that the masons were unfamiliar with the materials and dry climate of California, since most of their prior work had been done in eastern states.

Finite element analysis. Based on the material properties derived from testing and other information gathered from field investigations, finite element analyses were carried out first to consider the potential seismic vulnerabilities of the existing unstrengthened system and then to evaluate various strengthening alternatives. Two basic models were developed for the longest span vault condition. The first included mortar joints replicating the herringbone face tile pattern. With this model, the expected benefits of this pattern were considered and the mortar joint stresses evaluated. This model was first used to evaluate the stresses in the existing system. A second model was then created to evaluate various strengthening alternatives. To simplify the analyses and perform these evaluations more efficiently, this model excluded the mortar joints. Finally, once the strengthening approach was selected, the strengthening elements were added to the original, more detailed model, and the new system was analyzed to verify its effectiveness.

Load testing by Rafael Guastavino himself showed the tremendous load-carrying capability of his system under vertical loading conditions. However, his testing generally included a vault with boundary conditions at a single coincident elevation, whereas many of his projects incorporated much more complex geometry. The original assumption, that the vaulted ceiling in Hearst would behave under vertical loads as one expects of a properly designed arch, producing uniform compression and outward thrust at the base and no tensile stresses, was incorrect. Analysis revealed that under downward vertical loads the

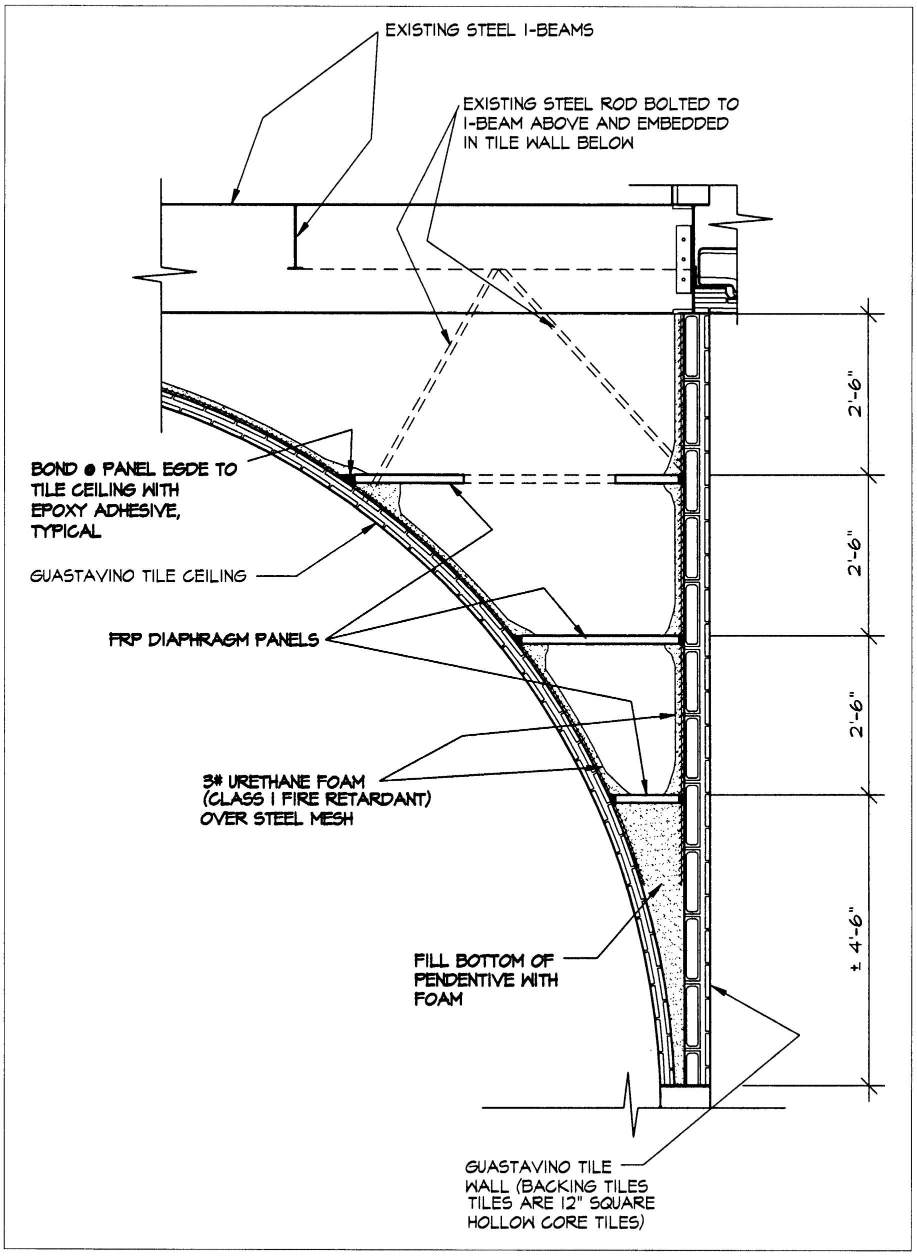

Fig. 11. Section at pedentive. Courtesy of William Kreysler Associates.

vaulted ceiling behaved in a flexural mode, with compression in the face tiles at the upper end and tension at the lower end of the vault consistent with the hoop stresses depicted in Fig. 8. The critical load case considered in analyses included a downward acceleration of 2.5g (seismic plus gravity) plus a lateral load of 0.3g.

Under these loading conditions, analyses indicated that the weak plaster-of-paris face tile mortar joints would be significantly overstressed, rendering it ineffective. Once the face-tile mortar joints were removed from the model, the bond stresses between face tile and backing mortar greatly exceeded its minimal capacity. For subsequent analyses, it was assumed that the face tile would not contribute to the system strength but would be supported by the remaining strengthened system.

The existing ceiling analyses revealed high hoop, through-thickness, and shear stresses in various areas throughout the system. The level of stress would be excessive in most masonry construction but particularly in the Hearst ceiling, because of the very weak bond between tile and mortar courses.

A three-pronged strengthening approach was developed jointly by Rutherford and Chekene, the project structural engineering firm, and William Kreysler and Associates, a firm specializing in the design and fabrication/construction of composite materials. The first step is to reduce the ceiling deformation and thus the global stresses to acceptable levels; second, provide a back-up system to provide support should significant deterioration occur during long duration earthquake shaking; and third, positively anchor the face tiles to prevent individual tiles from falling and to retain the vault curvature (Fig. 9).

To reduce overall deformations and stresses in the vaults, epoxy-fiberglass double-channel ribs will be bonded with epoxy to the back of the vaults at a 4-ft. spacing. Second, a backing system, consisting of rigid steel wire mesh embedded in sprayed-on urethane foam (3 lbs/ft^3), will be placed over the top of vaults. This lightweight and rigid foam and steel mesh system spans between composite ribs and supports the Guastavino system by adhering firmly to the back of the vaults. Lastly, the face tiles will be connected with regularly spaced 1/4-inch-diameter urethane elastomer pins, injected through a hole drilled in the center of tiles. Fig. 10 shows a detailed section of the strengthening system. The spacing of pins remains to be determined, based on the magnitude of stresses in each area. Rather than resisting the expected high hoop and through-thickness stresses, these pins are designed to elongate slightly under inplane stresses, so that high stresses do not fail the connection. Connecting all tiles is considered unnecessary, since the vault curvature is essentially maintained, and the unconnected tiles remain confined by connected tiles. Before a tile could fall, a majority of the surrounding mortar would need to be lost, which is considered unlikely. The urethane pins are injected through the foam backing and over the steel mesh system to provide positive anchorage. The pendentives will be strengthened in a similar manner, except that three fiberglass composite diaphragms, rather than ribs, will be placed horizontally in each pendentive (Fig. 11) to provide the necessary stiffness and strength. Alternative methods for anchoring the face tiles were studied, but the high stresses that would be developed in using these rigid connection methods were found to be untenable.

A comparison between the maximum hoop stresses in the existing Guastavino system and the strengthened Guastavino system is shown in Fig. 8. For the vertical axis, negative values represent compression and positive value tension stresses. The horizontal axis follows the length of the south vault as depicted in Fig. 9 on the right.

Conclusion

Guastavino vaulted and domed construction may eventually prove to provide superior earthquake performance compared with more conventional unreinforced brick and stone masonry vaults and domes. However, considering the rigid and brittle nature of the materials and the prolific use of Guastavino construction in vastly differing applications and conditions, seismic deficiencies must exist in some buildings.

Although the seismic hazards in the eastern U.S. are considerably less than in California, a degree of seismic risk remains for this unique architectural motif. Future renovation projects of buildings that include this important historic resource should not discount the potential loss of the system in an earthquake. The seismic vulnerably of Guastavino work should be given appropriate and due consideration together with other project program and renovation goals. Even though planning for a major earthquake may not be economically feasible on many projects, architects and engineers should seriously consider the impact of smaller earthquakes that occur with greater frequency. This study indicates that some Guastavino systems will perform poorly in a major earthquake. However, we do not know with any certainty how these relatively delicate thin shells will perform in smaller events with accelerations of 0.1, 0.2, or 0.3g, especially as the form and details are unique to each installation.

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Notes

1. Preserving Historic Guastavino Tile Ceilings, Vaults, and Domes," Conference at Columbia University, sponsored by the New York Landmarks Conservancy, February 6, 1999. This article expands on the information presented by the author at the conference.

2. ATC-13 (Applied Technology Council), *Earthquake Damage Evaluation Data for California* (Redwood City, Calif.: 1985).

3. Rafael Guastavino, "Cohesive Construction, Its Past, Its Present, Its Future," *The American Architect and Building News* 41 (922: 26, August 1893): 125-129

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