

# SEISMIC STRENGTHENING OF A 19TH CENTURY BANKING TEMPLE

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#### Abstract

Abandoned by its namesake and left to the pigeons in the 1980s, the Hibernia Bank Building (circa 1892) was used briefly by the San Francisco Police Department as a substation in one of its most colorful neighborhoods: the Tenderloin. The police left in the year 2000 and the "banking temple" was left to sit vacant for another decade. Things seem to be improving for this San Francisco City Landmark, not only is the neighborhood (now coined "mid-Market") slated for improvements, but the new owners of the building have just completed an ambitious renovation including seismic strengthening. WJE's retrofit concept was designed in deference to the structural mechanism of rocking that allowed the unreinforced masonry building to survive the ground shaking associated with the 1906 earthquake relatively unscathed, although the building was later ravaged by the great fire. Although concrete shear walls were added as part of the retrofit, they are placed discretely and surgically in the structure, located away from historic finishes which have survived remarkably intact despite decades of neglect. Other unique components of the retrofit include: 1) center coring of the massive granite and masonry piers to promote rocking and reduce cracking and walking, 2) installation of corrugated sheet steel shear walls as a backup lateral system for a portion load transfer mechanisms. This paper will discuss technical aspects of the retrofit scheme including computer modeling and use of provisions in the California Historic Building Code, but also will demonstrate how treating seismic safety and historic preservation with equal priority can enhance project outcomes.

Keywords: Unreinforced masonry building; Seismic strengthening; Historic structure; Center coring; Rocking piers



# 1. Introduction

In 1992, San Francisco adopted an ordinance requiring owners of unreinforced masonry buildings to abide by a timeline for seismic assessment and strengthening of their structure, if needed. At that time, the Hibernia Bank Building had been abandoned by its namesake and was occupied by the San Francisco Police Department. Inside the giant hall of this banking "temple" built in 1892 cast-iron columns can be seen, and in the attic a network of steel trusses supported the roof. Perhaps this is why for over a hundred years the building had been listed on historic insurance maps as being a "frame building" (Fig. 1) [1]. No one guessed that there may not be steel inside the massive granite and brick piers at the perimeter of the building. And why would they? The building had survived the great earthquake of 1906 unscathed, as is thoroughly documented by photographs of the exterior walls: not behavior characteristic of an unreinforced masonry building.

The era when a building of this scale and prominence being able to economically function as a bank has long passed; the approximately 650-square-meter banking hall, while quite grand, is well beyond the size of most retail banks today. With a projected increase in occupant load, it was required that the structure be brought into compliance with the 2010 California Historical Building Code, and other applicable codes. The entire exterior and large portions of the interior of the Hibernia Bank Building are very historically significant, which was the primary driver in the selection of the means for seismic upgrade of the building and the location for new structural elements. To improve integrity, stability, continuity, and thereby, performance, while preserving the historical integrity of the decorative stone and painted plaster finishes, the seismic upgrade selectively supplements and relies on the substantial seismic resistance of the existing building.

This paper describes the seismic retrofit of the Hibernia Bank Building that has just been recently completed. The technical details of the seismic retrofit that helped preserve its historic fabric are presented, along with the colorful past of this landmark building.



Fig. 1 – Sanborn map of the Hibernia Bank Building (left) and key (right) in 1905 [1].

# 2. History of the Hibernia Bank Building at Jones & McAllister

In 1889, the French-trained architect Albert Pissis and his partner William Moore won a national design competition for the design of a new headquarters for the Hibernia Bank at the corner of Jones, McAllister, & Market streets in San Francisco [2]. The structure was completed in 1892 (Fig. 2) of white granite quarried from Rocklin, California, including colossal full-height fluted granite column shafts extending from base to capital on

the street-side elevations. Each of these massive granite shafts were cut from one stone, not stacked such as the method in the nearby San Francisco City Hall. It was Pissis' first landmark structure in San Francisco after returning from instruction at the École des Beaux-Arts in Paris, France, and was constructed before other Pissis masterpieces such as the Flood Building and the Emporium at Powell and Market streets, and Temple Sherith Israel in Pacific Heights, which all survived the great 1906 earthquake. The interior of the building was dominated by a vast banking hall with a ceiling almost 11 meters above the banking counters; highly detailed painted plaster and stone finishes were provided throughout. Two stories of luxurious offices were located on the McAllister street side, including a private staircase from the rotunda to the offices of Tobin & Tobin, the bank's lawyers.



Fig. 2 – Hibernia Bank Building in 1894 (left) and after the 1906 earthquake (right)

In 1904-1905, a seamless addition to the building that extended the building along McAllister Street enlarged the banking hall and added offices was also designed by Albert Pissis (Fig. 2). The final interior touches to the addition were being completed in January 1906, just three months prior to the April 18, 1906 earthquake. The structure survived the shaking from the earthquake structurally intact, or so it seems, but sustained significant fire damage to the interior and some of the granite exterior during the subsequent conflagration that consumed much of the city. From 1906-1908, the building was repaired to its former glory with new skylights designed by Pissis and executed in San Francisco [2] along with some new features including the construction of "fire proof" hollow clay tile (HCT) partitions throughout and the installation of roll-up steel security shutters at each window.

In the 1930s, the increasing role of women in the workforce and in the employ of Hibernia Bank necessitated an expansion to accommodate segregated women's facilities including a "kitchen, lounge, locker room, and a sundeck." [2] The architect Arthur J. Brown, Jr., an architect known for designing San Francisco landmarks such as the San Francisco City Hall and Coit Tower, designed this "penthouse" addition to accommodate facilities for the new staff.

With the neighborhood in decline, the bank left the building in 1985 turning it over to a San Francisco Police Department substation that operated in the basement. In the year 2000, the police department sold to the building to His Holiness Grandmaster Professor Thomas Lin-Yun, the founder and supreme leader of the contemporary Black Sect of Tantric Buddhism, who had planned on turning the banking temple into a Temple for his religious organization. After nearly a decade of being abandoned, with the Temple project languishing, His Holiness sold the building to a developer who hoped to take advantage of the resurgence in the neighborhood. Luckily, the rough neighborhood of the previous several decades had not affected the interior of the building, which was well protected behind the steel roll-up security shutters installed after the 1907 earthquake (Fig. 3).



Fig. 3 – Interior of the Hibernia Bank Building

## 3. Structural Description

The plan of the building is generally rectangular; with plan dimensions of approximately 40 meters by 38 meters, and includes a rotunda at its main entrance, located at the corner of Jones and McAllister (Fig. 4). In general, the building is an unreinforced stone and clay brick masonry bearing wall building with a semi-rigid diaphragm. The primary vertical elements of the structure are its four massive perimeter unreinforced brick and granite masonry walls and piers, which are constructed predominantly of large granite blocks, full-height fluted granite column shafts, as well as massive interior brick walls that divide the structure and its roof diaphragm into discrete sections. The perimeter walls are an interlaced combination of both clay-brick and granite, with different walls having different proportions of each material; for the typical street elevations, the proportion is approximately 70 percent granite to 30 percent clay-brick.

The executive office portion of the structure along McAllister Street is two stories tall; its second floor is a brick arch and concrete floor system which is supported by steel beams and cast iron columns. The operations and banking vault portion of the structure to the north (in what is referred to herein as the "bookend") also has two stories with an added mezzanine. Its brick arch and concrete second floor and mezzanine, and its concrete roof deck, are supported by steel framing that spans between two closely-spaced massive masonry walls, one of which is the north exterior wall of the structure. Located between the executive offices and the "bookend" and comprising the bulk of the building's plan area is the main single-story banking hall which measures about 38 meters by 21 meters. The main banking hall is finished with decorative painted plaster, polished stone and two large skylights above decorative laylights at the ceiling level.



Sections of the roof are divided into discrete regions by unreinforced stone and clay-brick masonry parapets, and are highly perforated by large skylights and mechanical shaft openings. The roof diaphragm is 4-inches of wire-reinforced concrete spanning in one-direction and supported on a network of bolted and riveted steel trusses. Beneath the above-grade structure is a full basement which daylights on all four perimeter elevations. The basement includes a large number of massive interior and perimeter walls and wall footings. The building is on a mildly sloping site and is supported by brick and concrete foundations. Numerous vaults are distributed throughout the building.



Fig. 4 – First (or ground) floor plan

# 4. Seismicity & Structural Response during the 1906 Earthquake

The primary damage to the Hibernia Bank Building following the 1906 San Francisco earthquake and fire was related to the great fire and was not structural damage from ground shaking. This was similar to other Pissisdesigned landmark structures in San Francisco which survived the earthquake with little structural damage including the Temple Sherith Israel, the Flood Building, and the Emporium. The last two of these reside within one kilometer of the Hibernia Bank Building and are supported on similar soils, defined in generic maps as areas where there is a historic occurrence of liquefaction or conditions that indicate a potential for permanent ground displacements [4]. The geotechnical consultant for the current project classified the bearing material at the Hibernia Bank as 2 meters of poorly graded fills, 20 meters of dune sand, and then many meters of dense clayey sand of the Colma formation, corresponding to a Soil Type D; they also estimated that the potential for permanent ground displacements a total of two centimeters across the site [4].

Today only minor settlement-related damage physically reveals itself in the building today. Similarly, no earthquake damage is observed in the finished spaces of the building, all of which were substantially repaired after the fire. Physical damage from the 1906 earthquake and fire has revealed itself in other locations in the building, however, including a buckled truss member at the northeast corner of the building and buckled steel beams located between the bookend walls; these appear to reflect buckling due to restraint to heat-induced expansion.



A reconnaissance report from the United States Geological Society after the 1906 earthquake mentions damage that occurred at the Hibernia Bank, but it is vague about whether the damage was caused by the earthquake or by the fire:

...the granite fronts, especially around the doors and windows, were badly spalled by fire; other damage to the structure was confined almost entirely to the roof [3].

Another confusing report from after the earthquake comes from the Bank Secretary, who alludes to the fact that the building may have been set alight from the inside?

The flames from the outside did not seem to cause the igniting of the structure. The structural granite just outside of the windows, as ascertained after the fire, was chipped by the heat from the inside. Glass fuses at a temperature of five hundred degrees. The heat must have been that in the bank interior, for we found glass bottles fused with the glass of the windows. [2]

A more likely thesis is that the roof of the structure was damaged from the earthquake, and that raining embers from other burning locales in San Francisco set inflammables ablaze inside the bank. As mentioned previously, the roof diaphragm currently is 3-inches of wire-reinforced concrete, but it is unclear whether this is original or was replaced in the period 1906-1908. Given the level of spalling-related damage seen to the outside of the windows and 1) the lack of charring, or discoloration to the concrete roof diaphragms directly above these locations, and 2) the uniformity of the roof diaphragm in all portions of the building, it seems likely that the entire roof diaphragm was replaced in the period after the earthquake. As the entire building was shown in the 1905 Sanborn map as "fire proof" it can be assumed that the original roof diaphragm was likely of similar construction and not wood-framing.

Damage to the concrete roof diaphragm in the 1906 earthquake was confirmed by benchmarking studies conducted using a SAP2000 v. 15 analysis model of the building as it appeared in 1906; an overall view of the model is shown in Fig. 5. The model was constructed after an extensive round of masonry shear testing that demonstrated the very competent state of the masonry. The model incorporates primarily linear-elastic material properties and elements, but includes nonlinear behavior associated with rocking of the masonry piers in both the in-plane and out-of-plane directions. The rocking capacities were incorporated into the model using nonlinear, elastic-plastic hinges at the bases and, where applicable, at the tops of existing masonry piers expected to undergo rocking during a strong ground shaking event. In cases in which there was a semi-circular full-height fluted granite column shafts, as is typically the case along the Jones St. and McAllister St. elevations, the cross-sectional area of that element was neglected for stiffness purposes but was included for mass and weight calculations.



Fig. 5 – Hibernia model looking at Jones Street elevation (left) and west alley elevation (right)



The original drawings for the 1892 original building and 1905 addition may be in a private collection or may have been lost in the great fire. Dimensions, lengths, and sizes of structural elements of the existing structure are based primarily on a laser-scan of the building supplemented with field measurements. At the roof level, all of the diaphragm slabs are sloping. In the executive office wing and in the bookend, these slopes were explicitly modeled based on field data. On the perimeter of the building, piers are comprised partly of brick masonry and partly of granite masonry. The effective densities (and stiffnesses) of the piers used in the model was based on best-estimates of relative thicknesses from field observations collected visually and data derived from investigation with rotary hammers and ground-penetrating radar. A similar procedure was followed for modeling the shell thickness of existing masonry walls not undergoing rocking.

Simulating the intensity of ground shaking from the 1906 earthquake is possible from contour mapping performed after the earthquake along with more recent research, suggesting that the potential for damage at the Hibernia site was "Moderate/Heavy" as shown in Fig. 6. We analyzed our building model using the response spectrum analysis as it was subjected to a spectra generated from inferred 1906 earthquake ground shaking [5]. Our analysis model showed high stresses in the heavily perforated roof diaphragm. These stresses could certainly have led to local collapse of the original roof diaphragm material.



Fig. 6 - 1906 earthquake intensity contour maps of San Francisco peninsula (left) and downtown (right) [5]

## 6. Objective of Seismic Retrofit

In 2009, a study was conducted prior to our involvement by a local engineering firm. Ignoring the lateral capacities of the existing materials, the firm designed a retrofit that involved the infilling of five two-story exterior windows with shotcrete walls, adding a massive shear wall within the banking hall, installation of a one-meter thick mat foundation (rendering the basement useless), and specifying a gridwork of tube steel in the attic over the main banking hall which would pass through the decorative laylights in dozens of locations and be visible from the floor of the banking hall. Luckily, these concepts were dismissed by the owner who realized that such a plan would never be approved by the Historic Preservation Commission in San Francisco.

When presented with the other firm's conceptual work, the authors advised that such traditional retrofit approaches had no place in the retrofit of this historic structure and that the use of the California Historical Building Code (CHBC) [7, 8], which allows the use of rational methods to determine the capacities of historical materials and lateral systems, should be explored. As many new uses for the building would be categorized into a higher occupancy (or risk) category than its previous use as a bank due to an assembly occupancy, the regulations required the use of an importance factor increasing the seismic demands.



Attention to preservation of historic fabric and maintenance of character defining features was paramount in guiding the retrofit solution. The vast majority of the interior and exterior of the building was rated as "very significant" by the historic structures report [2], which greatly limited the locations where structural interventions could reasonably be made. Fortunately, the dynamic characteristics of the unaltered building that brokered its survival through the 1906 earthquake were testament to the underlying strength of the structure. It was not desired to drastically alter that dynamic behavior, but to leverage it to protect this historic resource during the next large earthquake.

## 7. Strengthening Techniques

While some characteristics of the building, such as rocking piers, were to be leveraged in the strengthening scheme, others were less advantageous and required strengthening, such as the torsional behavior resulting from the long, stiff bookend walls on the north end of the building. Each of the primary strengthening techniques used in the retrofit are described below.

#### 7.1 Center-cored reinforcement

Pioneered in the 1980s [9], center coring of unreinforced masonry walls improves the integrity of unreinforced masonry walls. The increased strength that may also result from center-coring however is not permitted to be relied on in the San Francisco Building Code. At Hibernia, this reinforcement involved the coring of holes, installation of 25 mm diameter high-strength Dywidag rods as reinforcing, and grouting with a high-strength, low modulus polyester resin-based grout that was custom designed to reduced shrinkage and heat build-up during curing. The hybrid walls at Hibernia made center coring more difficult than usual, as coring through granite is substantially more difficult than coring through clay-brick masonry. Center coring was provided in every unreinforced masonry wall at the building, even though calculations show that most masonry piers will rock before shear cracking can occur, and only the very long masonry walls in the structure are shear critical. All of the rods were anchored into bond beams at the roof level, where their ends were fitted with a nut and encased in reinforced concrete, with many rods extending into the original concrete foundation. The spacing of the rods was approximately 1.3 meters on center, with rods also extending down to improve integrity of the masonry above doorways and window openings (Fig. 7). The full height center coring of the pediment piers, located on both street elevations, provides resistance against pier walking, which can be one drawback to pier rocking that otherwise is an effective earthquake survival mechanism.



Fig. 7 – Center coring at Jones Street elevation





In a historic building flat-roofed building like Hibernia, which has a historically-significant façade and century-old interior finishes, the best place to do construction work is not within the building at all, but on the roof. The roof at the Hibernia was well-suited for strengthening interventions, including around the perimeter of the building where granite balustrades could be removed and replaced with simple gantry cranes to facilitate installation of reinforced concrete bond beams. The addition of bond beams to the tops of unreinforced masonry walls is a proven method to strengthen the walls by preventing the propagation of cracks to the free edge of the masonry and act as chords and collectors for the roof diaphragm. The bond beams also served to anchor the tops of the center-cores, the base of the balustrades, and as an anchorage point for roof-to-wall ties. Anchoring the balustrades to the bond beams also allowed for removal of dense unattractive steel parapet bracing, a welcome aesthetic improvement to the roof. (Fig. 8) The bond beams around the perimeter and at interior brick walls are completely hidden from street view since the beams are located where previously there was unreinforced brick masonry. The distribution of base shear along various wall lines is provided for east-west loading in Figure 9. The figure demonstrates the role played by the diaphragm in distributing the inertial forces that originate in the heavy orthogonal walls to the walls parallel to the direction of loading. Also shown in Figure 9 is the displaced shape of the analytical building model (amplified 150 times) showing the deformation of the roof diaphragm.

### 7.3 Strengthening at roof diaphragm

Around the perimeter of most of the discrete diaphragm segments on the roof, and allowing significant movement between the roof diaphragm and the perimeter and interior masonry walls, was a deep gutter (Fig. 8). To provide a connection between the bond beams and the existing concrete roof diaphragm, and to create continuity between adjacent roof segments, a pre-tensioned "tie plate," essentially a 150-mm thick reinforced concrete element cast monolithically with the bond beam, was cast over each roof diaphragm segment along its perimeter and the interface was pre-compressed with regularly spaced 19-mm diameter bolts and square plates. This provided a connection using shear-friction between the roof diaphragm and the bond beams that required no slip to engage.



Fig. 8 – View of roof including gutter (red arrow) and 1980s parapet bracing (left). Surface-mounted, topside FRP roof-strengthening plan (right).



Fig. 9 - EW earthquake direction: distribution of base shear resistance (left), displaced shape (right)

To complete the roof strengthening, a unique FRP strengthening system was devised that allowed the project to save the original roof diaphragm, preventing the exposure of the interior finishes in the main banking hall to weather and dust during retrofit. The FRP effort involved installation of surface-mounted Carbon FRP (CFRP) sheets (Fig. 8) at the top of the slab, and near-surface-mounted CFRP rods around the perimeter and at openings in the diaphragm to provide chord strengthening and crack control. Additional CFRP surface-mounted sheets were provided at the soffit of the roof diaphragm in areas of higher demands. To provide a "basketing" backup system to help prevent chunks of the roof diaphragm from becoming dislodged and falling below, the topside and soffit surface-mounted FRP strengthening system were tied together with a network of FRP splayed anchors, with spacing determined from the level of stress in the analytical model.

### 7.4 Bookend "vertical box girder" & closet shear wall

Another location where the historic structures report did not identify historically significant finishes was in the "bookend" located on the north side of the building where massive parallel masonry walls only twelve feet apart add significantly to the north-south seismic mass which was detrimental to the predicted building performance (Fig. 4). To turn this negative attribute into beneficial behavior, the walls were interconnected with reinforced concrete "web" members such that the entire system resists loads analogous to how a cantilevered box girder resists shear and bending moment in the north-south direction, with the existing massive masonry walls acting as flanges and the assembly acting as a vertical box girder that cantilevers for the basement (Fig. 10). Shear transfer between the new reinforced concrete wall and the existing masonry was accomplished using C-shaped wall sections that expanded the area available for the 45 degree inclined epoxy doweling between the concrete and the masonry. Force is delivered to the new walls via concrete collectors on the roof, while overturning is resisted in the basement by long reinforced concrete outrigger beams that help mobilize weight from the heavy first floor slab. This vertical box girder resists a substantial proportion of the total north-south seismic loading, hence the "bookend" terminology that was adopted for the element.

Another strategically-placed reinforced concrete wall was located in a janitor's closet near the entrance to the building, another innocuous locale that would disturb no historic finishes. Although the dimensions of this wall are limited by its location, it does provide a ductile element on this line of resistance. Similar to the bookend walls, force is delivered primarily at the roof level via collectors and overturning resistance is buffeted by outriggers underneath the ground floor. Outriggers below the main banking hall and at the roof stiffen this wall and improve its rocking behavior.



Fig. 10 - Closeup of bookend showing "box girder"

### 7.5 Corrugated sheet steel shear wall

Due to the presence of very significant historic finishes, wholesale strengthening of the "executive office" wing of the building (Fig. 4) was not possible. One side was supported laterally on rocking masonry piers, while the other side was supported on several cast-iron columns topped with a steel beam that provided little lateral support. In the geometric center of the executive office wing, there exists a wall line, constructed of two wythes of HCT with historic finishes on one side, however adding thickness to this wall was not an option due to the presence of historic metal door frames and a decorative band of floor tile adjacent to the wall, and neither was replacing it.

While providing positive lateral support to the executive office wing was desired, adding stiffness via a reinforced concrete wall would have been incompatible with the rocking pier stiffness that provided its lateral support historically and would have attracted excessive forces that would have required additional strengthening elsewhere along the load path. Moreover, doing reinforced concrete construction within these historically significant spaces put the finishes in those spaces at risk. A strengthening solution was found using a corrugated sheet steel shear wall (CSSSW) [10]. To retain the original wall thickness, one wythe of hollow clay tile was surgically removed from the side of the wall without very historically significant finishes and the CSSSW was designed to fit within that space. The CSSSW in this case was tuned to a desired flexibility using two 2-meter long walls with 22 gauge corrugated steel decking. The CSSSW was constructed using 300-mm thick coldformed steel studs with 300-mm square HSS boundary sections and mid-height strong back. Since the CSSSW walls could be "stick-built", they also provided ease of construction that minimized the potential for damaging the historic finishes in this area and along the path through the banking hall to this area. The relatively low stiffness of the CSSSW means that its contribution to the overall resistance in the east-west direction is minor (Fig. 10). It is intended primarily as a backup system for the executive office floor and to engage in the event of a very large earthquake once the nearby rocking masonry piers have reached their capacities.

### 7.6 Hollow clay tile stabilization

Based on the presence of charred wood that the authors found at the building in the course of this project, it is believed that at least some of the original partition walls of the Hibernia Bank Building were constructed of wood-framing that burned in the great fire of 1906. In the repair effort from 1906-1908, the partition walls throughout the building were replaced with stacked, single-wythe or double-wythe HCT, which was installed for its "fire proof" properties. The behavior of such unreinforced, ungrouted HCT masonry in an earthquake, however, is quite poor, both due to its brittleness in shear and to its large height-to-thickness (h/t) ratio, particularly for single-wythe construction. To satisfy the CHBC and San Francisco Building Code with respect to providing a safe egress from the building, several innovative methods were developed to stabilize existing certain HCT walls located in paths of egress without damaging historic finishes. The HCT stabilization in Hibernia was limited to those walls that potentially affected egress. The three methods are explained below:

1. Along the length of the CSSSW, where a cold-formed steel shear wall being installed, the remaining single-wythe HCT was attached using an 8-mm diameter toggle bolt. The bolt was attached to flat stud blocking placed between studs. The toggle-bolt method was also used at other portions of two-wythe HCT walls where one-wythe could be replaced with steel studs without damaging historic finishes.



- 2. In certain locations on the second floor, the tops of a number of HCT partition walls were visible in the attic, above the plaster ceiling that provided out-of-plane support. To stabilize them, these partitions were intermittently reinforced and grouted. Drilling was first performed where necessary to provide a clear vertical chase for reinforcement, in locations where the HCT cells were not all oriented in the same direction, or where debris or mortar filled the HCT cells. A 2-inch deep hole was also extended into the slab at the bottom of the wall to provide keying action into the floor. Subsequently, reinforcing rods were threaded down through the cells and the cells were filled with zero-bleed grout.
- 3. In locations where neither of the above techniques was appropriate and the finishes allowed, welded wire mesh was embedded in plaster on one side of the wall and was fastened around its perimeter to adjacent construction with small-diameter steel anchors. Anchors intermittently were placed within the field of the mesh to connect it to the HCT.

### 8. Conclusions

Over one hundred years after the Hibernia Bank Building resisted the 1906 earthquake with little structural damage, the structure is undergoing an increase in occupant load, requiring a seismic upgrade. The positive seismic characteristics of the building are maintained and leveraged, including the rocking behavior of the massive stone masonry perimeter piers, and the extant capacity was surgically supplemented. The retrofit designed provides strength and stiffness in locations where benchmarked analytical studies have identified areas that could benefit from improvement, leaving the very significant interior historic finishes and the exterior façade entirely intact. The strengthening employed include several innovative techniques including a corrugated sheet steel shear wall, center coring of unreinforced masonry, and a unique diaphragm strengthening scheme including surface mounted CFRP sheets, near-surface mounted FRP bars, and FRP splayed anchors.

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