HISTORIC CATEGORY

HIBERNIA BANK BUILDING

SAN FRANCISCO, CALIFORNIA

SUBMITTED BY SIKA CORPORATION

Fig. 1: Hibernia Bank after 1906 earthquake

Background

The Hibernia Bank Building, originally built in 1892, is a true testament to old world design in a new age society (Fig. 1 and 2). The building, designed by architect Albert Pissis, would quickly come to capture the hearts of the locals and redefine the landscape of San Francisco. A true survivor, the Hibernia Bank Building was one of very few structures to survive the 1906 earthquake, shortly after a seamless addition was included on the west side of the original building. While it survived the earthquake, the building did not survive unscathed, as fire destroyed the interior of the building. It was one of the first buildings in the fire-affected commercial portion of San Francisco to be re-opened, and depositors were able to access their money five weeks after the earthquake.

Located in the now up and coming Mid-Market District, this historic building was vacated by Hibernia Bank in the 1970s and was used as a substation by the San Francisco Police Department until 2000. It was then purchased by an out-of-town investor and sat empty and unmaintained for over 10 years. Luckily, the building was purchased by a new owner, committed to restoring the building and returning it to functionality.

Building Construction

The main building structure is comprised of bearing walls of massive unreinforced granite

Fig. 3: Tiffany-style skylight over main banking hall

Fig. 2: Hibernia Bank Building

blocks and brick masonry which surround a huge interior banking hall. Riveted steel trusses that span over the banking hall support a thin lightly reinforced concrete roof slab that has large openings for historic stained glass skylights (Fig. 3) that amazingly remain intact to this day.

Because unreinforced masonry (URM) buildings have been identified as a class of buildings that are potentially more vulnerable to earthquake damage, in 1986, California enacted a law that required local governments in higher seismic hazard areas to inventory URM buildings and to establish a URM loss reduction program. Each local government was allowed to tailor its program to their own specifications. In response, San Francisco prepared an inventory and established mandatory retrofit programs. For whatever reason, the Hibernia Bank Building was omitted from this list, so when the building was most recently purchased, the owners took it upon themselves to have the building evaluated for its seismic adequacy and for possible strengthening. Ultimately, it was decided that the building would be upgraded to a higher than minimum level in order to allow greater potential and flexibility for the unknown future use of the space.

Testing and Evaluation

To begin the process of bringing the Hibernia Bank Building up to modern seismic retrofit criteria, an engineer was hired to perform a seismic evaluation of the existing structure. Before this could be performed, the engineering firm had to identify and document the existing structure configuration and capacities of existing materials. To design the most efficient repair approach that would result in the least disruption to the historic façade and interiors, utilizing the existing structure and its capacity to the fullest extent was desired. Since there were no existing structural or architectural drawings of the building to guide even preliminary seismic studies, extensive testing and evaluation were done to further understand the existing conditions and better define the seismic evaluation (Fig. 4). On the concrete slabs, ground penetrating radar was used to determine thickness and reinforcement location. Cores were taken to further evaluate concrete condition and compressive strength.

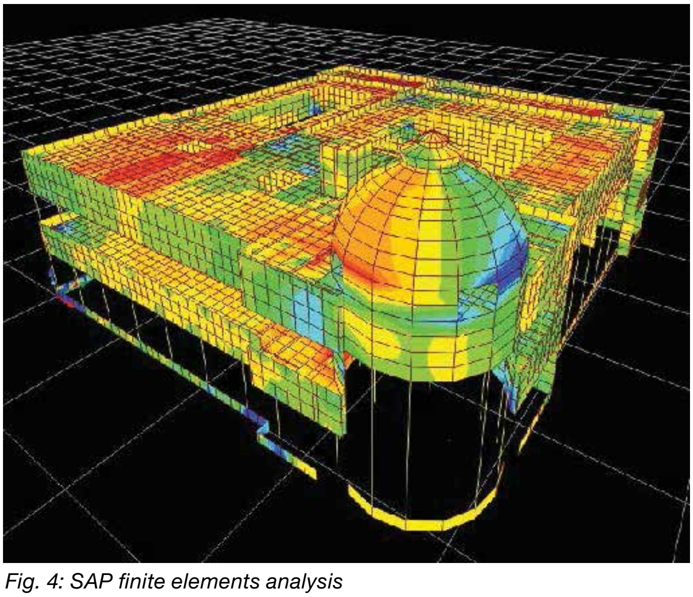

Fig. 4: SAP finite elements analysis

Extensive masonry shear tests were performed on the brick masonry walls. In less visible locations, single bricks were removed from the wythe closest to the interior. The mortar was cleaned out around the void and a hydraulic jack was fitted horizontally into the space. The vertical mortar was then removed from the side of the brick adjacent to the void, leaving only the upper and lower bedding mortar remaining. The jack was then engaged and the force was recorded at the time of first movement or slip of the brick. This procedure helped the engineers quantify the in-plane shear capacity of the masonry walls. As part of the seismic evaluation, it was concluded that many of the masonry piers were not shear critical and only wanted to rock, which is an accepted mode of seismic resistance for unreinforced masonry, and was likely a significant factor for the building surviving the earthquake in 1906.

Based on an understanding of the existing structural capabilities, evaluation and design analyses were performed that would utilize the full capabilities of the structure as it stood. This was critical for the historic preservation of the building, since this meant they would be able to add as few new structure elements as possible to this landmark building and still retrofit to meet applicable retrofit criteria. With so much of the load being transferred into existing members, the aesthetics of the original building would remain intact.

Limited, 3D non-linear analysis using SAP software was also performed at the building. Nonlinear gap elements were incorporated, allowing modeling of uplift/rocking of the piers and walls at their base or at the soil-structure interface.

Restoration Program

Center-Coring of Masonry Walls

Although not needed for in-plane shear strength, extensive center-coring of the existing stone and brick masonry walls was performed (Fig. 5) to provide additional structural integrity and damage reduction for the masonry walls, and to provide keying across potential slip-planes in some of the slender masonry piers. To gain access to the center of these walls, the granite balusters and coping stones at the roof had to be removed, as well as a dense network of structural steel parapet bracing that had been installed on the roof in response to an earlier San Francisco retrofit ordinance. Holes several inches in diameter were drilled vertically down 30-40 ft (9-12 m) through the brick and granite walls to the foundations, and steel rods were dropped into place.

Fig. 5: Center core drilling

In assessing what type of material should be used to fill the center core voids, water-based cementitious grouts were rejected due to the potential to damage historic plaster finishes in the walls, and epoxy grout was rejected as being too stiff to be compatible with the masonry. A shrinkage-compensated, polyester-based grout was selected to fill the voids because the modulus closely matched the existing wall properties and the material was very flowable, allowing for ease of application. The grout mix was tweaked on-site to meet the desired material properties.

A reinforced concrete bond beam that grabbed the top of each center-core was installed over all the masonry walls (Fig. 6) to tie the whole building together at the roof, and the coping stones and balustrades were reinstalled. During re-installation, the steel parapet bracing was omitted, instead securing everything in place with hidden pins and dowels.

Concrete Roof Diaphragm

The most critical design aspect, especially in the case of seismic activity, was the concrete roof diaphragm. The roof spans the entire massive banking hall completely unsupported from the ground with large, domed, stained glass skylights. Early on, the engineers knew that earthquake forces would have to flow around the large open area to transfer loads to the shear-capable walls surrounding the main hall. The load would then be carried down to the foundation by the existing center-core reinforced walls, supplemented by a very limited number of new concrete shear walls. The new concrete walls could not add or detract from any of the historical elements so were placed extremely discretely. In one location, new cold-formed steel shear walls were added, comprised of metal decking on cold formed studs using an approach developed at a local California university. These shear walls are quite possibly the first ones of this kind to be installed in California.

The roof diaphragm had to distribute drag loads to the parallel walls. During the initial evaluation, it was found that the concrete roof slab was only approximately 3.5 in (90 mm) thick across the entire roof and was very lightly reinforced, meaning that the roof deck required substantial strengthening. Adding significant load to the roof deck to accomplish strengthening, which would have then required major strengthening or completely replacing the supporting steel structural system, and which could not have been accomplished without massive disruption to the banking hall finishes, was not a possibility. The only logical choice to reinforce the roof slab while maintaining the skylight openings was fiber reinforced polymer strengthening.

Many design iterations were evaluated but the final design selected was a multi-layer weave of carbon fiber reinforced polymer (CFRP) fabric on the top and bottom of the roof slab, providing increased

Fig. 6: Bond beam reinforcement

Fig. 7: NSM CFRP geometry

shear strength, used in combination with near-surface mounted (NSM) CFRP bars providing tensile capacity (Fig. 7), and through-slab fiber anchors to add integrity. Because the slab was so thin, only very small diameter bars could be installed.

To add even more complexity to an already difficult design, the existing roof diaphragm was not continuous. Not only were there openings around the skylights, but the roof diaphragm consisted of seven different segments, each segment bounded by masonry parapets. The engineering team came up with a novel solution to integrate the seven segments and create continuity of the diaphragm across the roof using shear-friction. On each parapet, the new concrete bond beam that was poured in place included "wings" that projected from the side of the new beam and overlapped the existing roof slab by several feet. All the seismic forces had to be transferred within the overlap regions. Before the bond beams and wings were poured, near-surface CFRP bars were installed both parallel and perpendicular to the bond beam, and a layer of carbon fabric was installed onto the existing slab as a "starter sheet" to later splice with the fabric in the field of the diaphragm (Fig. 8 through 11). Holes for high-strength bolts were drilled through the existing slab, the top of the slab was roughened, the heads of the high-strength bolts supported by rebar, and their threaded shanks dangling down through the existing slab. The wings were then poured over the roughened area and half the starter sheet. To eliminate any slip that would otherwise be needed to mobilize shear-friction at the interface between the wings and the roughened surface of the slab, and maintain intimate contact at these surfaces, the high-strength bolts were pre-tensioned after the concrete cured. This methodology had the additional advantage of clamping the starter sheet tightly between new and existing construc-

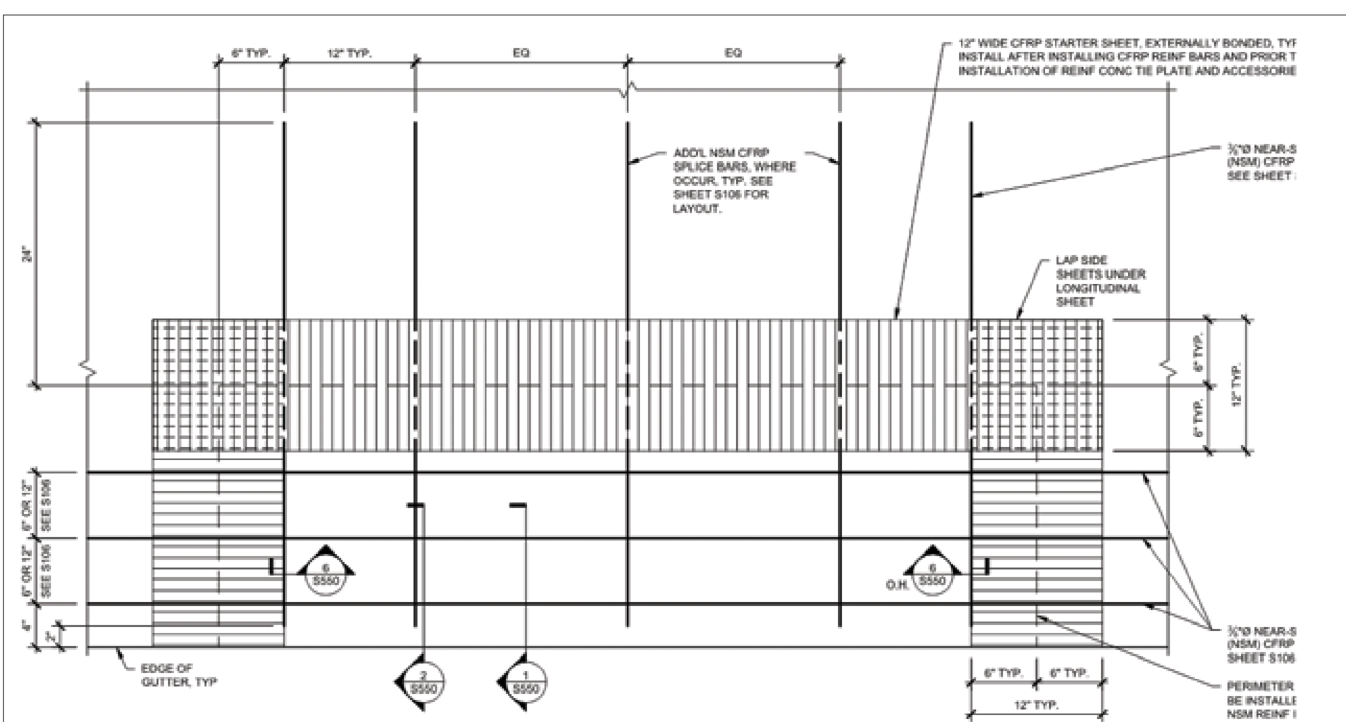

Fig. 8: Tie plate, NSM, CFRP, FRP and externally bonded starter sheet detail

tion, which improves the development of the starter sheet.

Fig. 9: FRP starter sheet application

Fig. 11: Completed concrete roof work

Conclusion

While initial evaluation of the Hibernia Bank Building began as far back as 2009, strengthening construction began in 2014. As of today, construction has been completed and the owners are currently searching for potential clients to lease this incredible space. The project's success is largely due to the over-riding philosophy that governed the design, wherein preservation was elevated to an equal level of importance as the seismic retrofit being designed. The engineering team came up with a plan that addressed the seismic concerns while continuing to rely on the extant seismic resistance of the original structure. The contractor had to use great care to protect in place and surgically install the complex series of reinforcement laid out by the design team, a feat that is truly remarkable considering that the contractor had not been involved with FRP work prior to this job. The structure now complies with the requirements of the City of San Francisco UMB Ordinance while leaving the interior and exterior essentially undisturbed.

Fig. 10: Roof FRP installation

Hibernia Bank Building

OWNER
One Jones Street
San Francisco, CA

PROJECT ENGINEER/DESIGNER
Wiss, Janney, Elstner Associates, Inc.
Emeryville, CA

REPAIR CONTRACTOR
Landmark Construction
San Francisco, CA

MATERIALS SUPPLIER/MANUFACTURER
Sika Corporation
Lyndhurst, NJ