REHABILITATION

Expect the Unexpected

Restoration and rehabilitation projects present numerous challenges to structural engineers, most discovered after construction commences. The completion of a successful project requires an ability to evaluate and solve specific and often unique conditions, throughout both the design and construction phases. The process of course requires engineering/technical knowledge, and also necessitates a good measure of foresight, flexibility, creativity, and a clear understanding of the owner's financial limitations.

The following article describes two restoration projects in which structural engineers met project challenges with a cadre of tools, and lots of ingenuity.

Lenox Hotel Façade Repairs

By Michael Teller, AIA

Background

Boston's Lenox Hotel is located in the Copley Square area, just a short walk from Prudential Center, the Hynes Convention Center and the Symphony Hall. The historic hotel's lobby features a high ceiling, glittering chandeliers and hand carved wood molding.

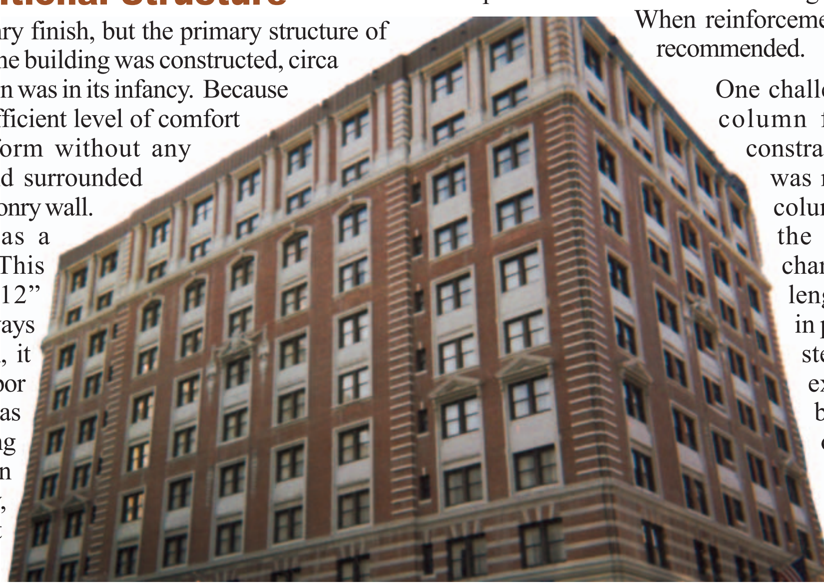
The Lenox was built in 1900 at a cost of $1,100,000. Its exterior of red and white terra cotta bricks and lavishly decorated interior were unmatched in the city of Boston at the turn of the century. Built in a record-breaking eight months by George A. Fuller Company, the hotel is named after the family of Lady Sarah Lenox, wife of King George III, who ruled before and during the American Revolution. Lucias Boomer, original owner of The Lenox, decided on the name to honor the anniversary of over 150 years of peace between England and North America.

When structural engineers with CBI Consulting, Inc., Boston, were called in to this project, the owner wanted to know why bricks had begun falling off the building from the eleventh floor.

CBI assisted the owner of the Hotel in the investigation and analysis of the underlying problems, in the designing of various repair options, and in the administration of the construction. As part of the analysis, a limited investigation protocol was prepared, involving a 150' man lift and a mason to accompany technical staff up to the problem areas of the building. They were able to remove selected areas of masonry to reveal the existing steel structure of the building.

A Transitional Structure

The building has a masonry finish, but the primary structure of the building is steel. When the building was constructed, circa 1900, steel frame construction was in its infancy. Because engineers did not have a sufficient level of comfort that the steel would perform without any assistance, they infilled and surrounded each piece of steel with a masonry wall. This is what is known as a "transitional structure." This solid brick assembly (12" minimum thickness) will always absorb moisture. Although, it then releases that water as vapor as the sun heats the brick or as heat is lost from the building interior. The steel, which is in the middle of the masonry, unfortunately remains moist much of the time.



Corrosion Problems

One hundred years of this process resulted in significant parts of the steel structure rusting away, especially at the corner columns. When CBI accessed the exterior of the building and cut test holes to observe the steel, they found many locations where the steel no longer existed. Examples of rusting and missing steel were so numerous, it was recommended that all the steel on the exterior of the building be exposed for review and repair for life safety reasons.

Three significant structural issues, and several minor structural issues, were in need of resolution. Existing steel column sections were built-up elements utilizing plates, angles and Z-sections riveted together. Beams were rolled channels and I-sections.

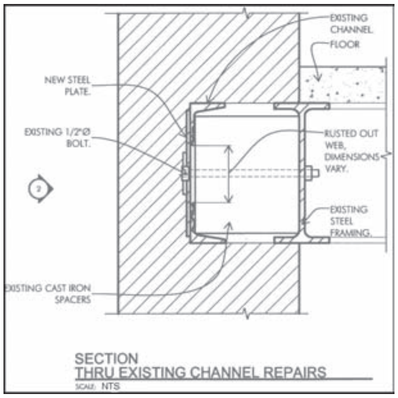
Steel lintel angles riveted to channel sections at each floor line were extremely corroded. In many locations, holes were discovered through the channel webs. In two cases, column deterioration was so severe that the steel cross section had completely disintegrated, and the masonry substrate and veneer was carrying vertical loads. The bid package contained details for reinforcement and full replacement of each type of element. Construction documents also provided many detail variations, to accommodate varying field conditions and budget.

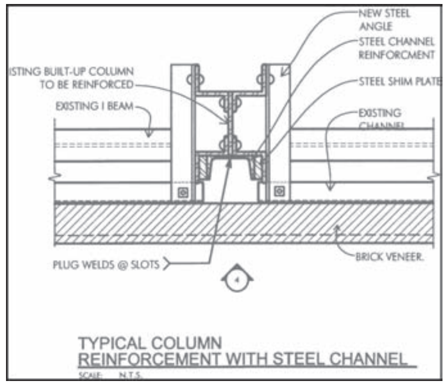
Rehabilitation vs. Replacement

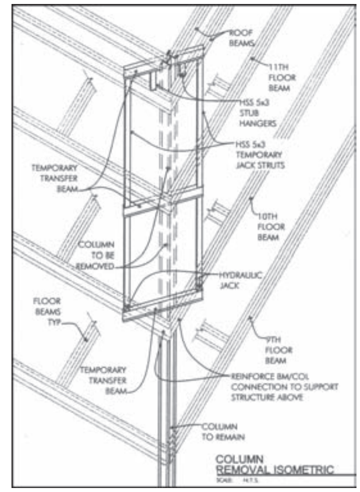
All existing steel was evaluated. When it was determined that the column cross section could be rehabilitated, a series of supplemental steel pieces were added to the existing cleaned cross section as reinforcement. When reinforcement was not feasible, replacement was recommended.

One challenge was in the understanding that a column fully embedded in masonry was constrained. When the surrounding masonry was removed for repair, large sections of column cross section were reduced so that the element's slenderness ratios were changed. Repairs were staged. A given length of column was often rehabilitated in phases, so that the corroded remaining steel would not be exposed to such an extent that local buckling would occur before it could be reinforced. In some of the worst situations, consideration was given to reducing the live load in certain locations to relieve column stresses. At no time, however, was this necessary.

& RESTORATION







During the investigation phase, it was decided to test the process of removing a column and replacing it with a new cross section. The purpose was to test which procedures would be most user-friendly to a contractor, and to establish budget costing while documenting the procedure for subsequent construction and use of bidders. (These sketches demonstrated the types of reinforcements that were added to address a variety of deficiencies).

Accommodating Loads

The columns requiring replacement were at the upper two floors of the 11-story building. One straightforward, but costly, procedure to remove a load from the column area would have been to introduce full height shoring from the foundation level to the beams delivering load to the column to be removed. This would have required large shoring towers on both the interior and exterior of the building. The second approach, and the one ultimately selected, was to find a way to pick up the roof and the floor loads in the area of the column being replaced, and deliver them back in to the column section below the replacement area while the work was underway. To prepare for this, masonry was removed from the beam connection joint locations. With the masonry removed in this area, field measurements were made so that as-built conditions could be documented.

One approach consisted of extending the intersecting beams through the column to be removed. The extended beams would become support points for shoring external to the building (in the area where the column was to be removed) returning back to the support column, two floors down. The second alternative was to reinforce the connection at the return floor so that shoring loads external to the building could be delivered back to the support column by the beams intersecting that column at the support floor. This was the solution chosen.

Because the building corner was under load, and with the knowledge that the interior partitions of the hotel had recently been refinished, it was deemed necessary to preload the existing floor systems so that the structure would not displace when the existing column and masonry was removed. In order to accomplish this, two Enerpac rams were placed beneath the support shoring and the shoring was preloaded to the level of structure dead load. Our rudimentary attempt to load balance, which was felt advisable (as the roof load was less than the 11th floor load), was not entirely successful. The roof structure jumped up approximately ½" when the last portion of the column was cut loose.

Did You Know?
The Lenox Hotel sits on the finish line of the Boston Marathon, the renowned 26.2 mile footrace that occurs annually in April.

Bulging Brick

An additional cause for such widespread deterioration of the steel relates to the quality of the mortar used. The existing mortar was extremely soft, with a very high lime content. The mortar also allowed a great deal of water into the structure, and was not able to resist the rust jacking that occurred on the horizontal steel elements. When steel rusts, it expands to seven times its original thickness. No material constructed by man can resist the resulting forces. As such, the masonry cracked at locations corresponding to the rusting steel. The cracks then allowed additional water into the system, which would freeze in the winter. When water freezes, it expands, causing cracks to expand slightly. As the cycle repeats season after season, the cracks grow and eventually the wall begins to come apart.

During the investigations, the worst areas of bulging brick were found in the spandrel areas between windows. In this location, the brick is corbelled, or stepped out, creating small shelves above the brick onto which water can pond and eventually seep into the system. Making matters worse, the steel support angle was not located within a mortar joint but within the body of a brick course, meaning that the rear of the brick needed to be cut away, leaving a very thin piece of brick. This thin area allowed additional water into the system at these particularly vulnerable locations.

The Façade

Exposing all the steel for evaluation and repair led to a series of decisions involving the existing historic nature of the building, and the reality of having to remove so much of the building façade to expose the structure. Such extensive removal offered the owner an opportunity to change the entire look of the building. CBI calculated that after removing enough of the façade to expose all the steel, less than 30% of the brick and into terracotta would remain. Reinstalling the masonry removed, or matching the original, would have been very costly and much more difficult for the masons to install. This led to an exploration of various other cladding types, including modern systems such as glass curtain wall, EIFS, and precast concrete. Each option was reviewed in terms of its aesthetic, monetary, technical, and historical implications.

The Lenox Hotel is listed on the National Register of Historic Places. CBI and the project team worked very closely with the Boston Landmarks Commission to ensure that the new design satisfied their requests. In the end, the hotel façade was reconstructed to match the original in every detail. In fact, details that had been removed over the years have now been restored. It is believed that a large decorative cornice at the roof was removed in the early 1960's. This cornice was reconstructed as part of the project, using original photographs and documentation.

The existing façade, comprised of brick with terracotta stone surrounds at the windows, was completely removed. The system was then changed to a cavity wall system to improve waterproofing of the structure and to add insulation, improving the thermal performance of the building. (Every aspect of this hotel is addressed with energy conservation in mind.) Along with the new insulation and a full air barrier, high performance windows were selected to replace all existing windows on the exterior of the building. To address sustainability, the existing windows and brick were recycled. The new brick, a dark water struck by local manufacturer Stiles & Hart, matches the original. Mortar samples were analyzed and the original color was duplicated.

For budgetary and scheduling purposes, the original terracotta was replicated in GFRP panels (fiberglass) based on molds created from the original stonework. The original terracotta color was matched from preserved samples found on the building exterior. The GFRP was also designed to hide various cellular antenna installations that have been applied to the building over the years. This consolidation, and camouflage of high tech appendages further, restores the building to its original look.

Out of Plumb

During the demolition, CBI observed that the building was leaning or had settled quite a bit. A professional surveyor, with a laser transit, was brought in to establish the full extent. It was determined that, depending on the corner of the building, it leaned between six and twelve inches inward and/or outward at the corners. This presented a unique challenge to the contractor. The steel structure of the building matched the various angles. The brick therefore had to follow the steel. This meant, however, that the union bricklayers, normally used to building everything plumb and true, had to build on an angle. Each corner being a different angle made it even more of a challenge. Knowing that the window rough openings had to follow the angle of the building, the centerlines of the window openings were used to establish the vertical benchmark.

Unfortunately, the new windows had to be installed plumb within the rough openings, in order to maintain the warranty. The solution was to oversize the rough openings, in order to provide room to shim the windows within the existing opening. Wide aluminum panning on the interior and exterior gave us the opportunity to scribe-to-fit each opening to each custom opening. Because the distance of the lean was spread out over eleven stories, it is not noticeable.

Lessons Learned

Transitional buildings are not consistent from floor to floor, or face to face, in terms of construction composition and dimension. Recommend a larger than usual contingency to the owner for time and budget (and engineering fees…). Allow as much flexibility in the design as possible to handle unforeseen conditions. Advance test repairs are extremely valuable to determine labor costs in advance of bidding. Finally, it is critical to educate the owner to the potential difficulties of a transition building restoration, so that expectations of all parties can be met.

Restoration of Fraser Field Stadium

By Wayne R. Lawson, P.E.

Fraser Field Stadium in Lynn, Massachusetts, is a 4,500 seat, cast in place concrete baseball stadium that was constructed in the early 1940's. The typical bleacher seating level construction consists of a cast in place reinforced concrete riser system supported by cast in place concrete beams and columns. The middle portion of the grandstand is covered by a unique cantilevered cast in place concrete roof system. The geometry of the roof is an arc, thereby varying the span of the roof slab. The roof slab itself varies in thickness from approximately 4 inches at the front, to 8 inches at the back of the stadium. Despite its high visibility and importance to local athletics, little attention was given to maintaining and protecting the exposed concrete from the damaging effects of the environment after the original construction was completed.

In 1998, noticeable deterioration of the facility prompted the City of Lynn, MA, to commission an engineering study to evaluate the condition of the structure. Based on the recommendations of this investigation, $400,000 was expended for the erection of temporary emergency structural steel shoring and foundations to allow the stadium to remain open and maintain public safety. The temporary shoring was installed to support both grandstand and roof elements, and consisted of wide flange steel beams and tubular steel columns.

In 1999, CBI Consulting Inc. was engaged by the City of Lynn to develop construction documents for permanent structural repairs and waterproofing of the stadium. The extent of the concrete deterioration necessitated a variety of construction methods, details and material types to rehabilitate and extend the life of the complex, while adhering to the client's budget restriction. The project also presented other structural engineering challenges that shaped the final scope of the $2,200,000 restoration.

Emergency Shoring

During the 1998 study, the city's consultant analyzed the concrete grandstand riser system by employing a simplified model of individual 4 inch wide by 8 inch deep discreet beams, each reinforced with a single #4 bar top and bottom, as shown on the original drawings. The yield strength, Fy, of the existing reinforcing steel was not listed on the structural drawings. In lieu of removing steel samples and performing laboratory tests to determine the yield strength, the structure was analyzed using yield strengths of 33 ksi, 40ksi and 50 ksi. Reinforcing steel of these strengths were all available and in use during the period that the stadium was constructed. Based on this analysis, the consultant concluded that the live load capacity of the concrete grandstands was only between 40 psf and 70 psf, significantly less than the 100 psf requirement of the Massachusetts Building Code. This finding, coupled with the recognition of the extent and global degree of degradation, lead to the implementation of emergency shoring.

Load Testing

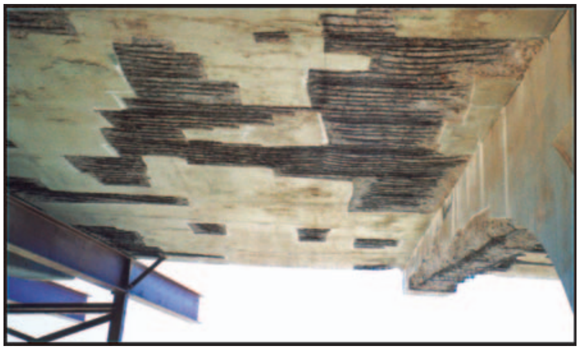
In developing the 1999 repair program, CBI confirmed the results of the previous grandstand gravity load analysis. However, CBI also recognized that the stadium had been successfully utilized for almost 60 years without signs of overstress or failure. It was this historical performance that lead CBI to advise the owner that undertaking a field load test was an expenditure that, if successful, would derive significant cost saving to the project. A series of static load tests were conducted in the summer of 1999 in accordance with the requirements of 780 CMR 1709.0 of the Massachusetts State Building Code and ACI parameters.

Prior to initiation of the load tests, CBI reviewed the original structural drawings and existing site conditions. Test locations were selected based on proximity to an existing expansion joint to minimize the effects of continuity, and to utilize an area with representative concrete deterioration. The temporary steel shoring was left in place in the event of failure during testing; however, the oak wedges immediately between the shoring and the underside of the concrete seating were removed. The test was conducted in two phases, the first phase involved loading a single bay and the second phase involved loading two adjacent bays. By running two tests, both maximum positive and negative conditions could be evaluated.

The loading for the tests was calculated in accordance with ACI. The test load was applied in the form of concrete blocks, 2'0" x 2'6" x 4'0" having an approximate weight of 3000 #/unit. A total of seventeen blocks were applied to the test area in phase I, for an equivalent uniform load of 187.5 psf. A total of 34 blocks were applied over the two test areas in phase 2, also for an equivalent uniform load of 187.5 psf.

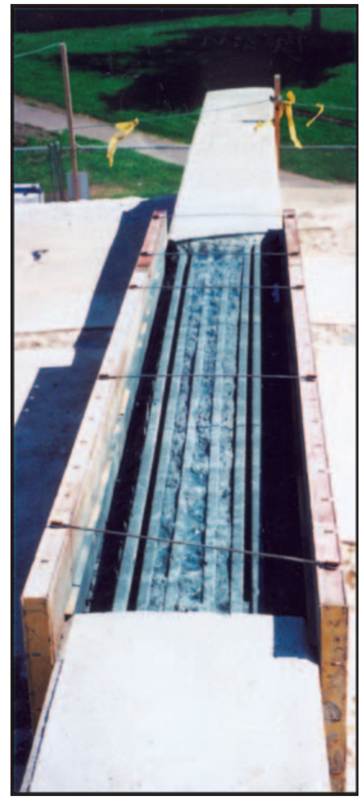
During each of the tests, loads were applied in four increments with deflection measurements taken at midspan and adjacent to supporting beams after each load increment. The deflections were then measured after the load had been in place for a twenty-four hour period. The test load was subsequently removed after completion of the twenty–four hour load period, and in both loading arrangements the structure recovered more than seventy-five percent of the maximum deflection. The load tests results successfully demonstrated that the existing grandstands could safely support a uniform live load of 100 psf. These results eliminated the need for installation of permanent supplemental framing, allowing more of the budget to be expended for other necessary stadium repairs and improvements.

Grandstand Beam Repairs - Underside



Roof Repairs - Slab Underside

Roof Beam Repairs - Top Surface

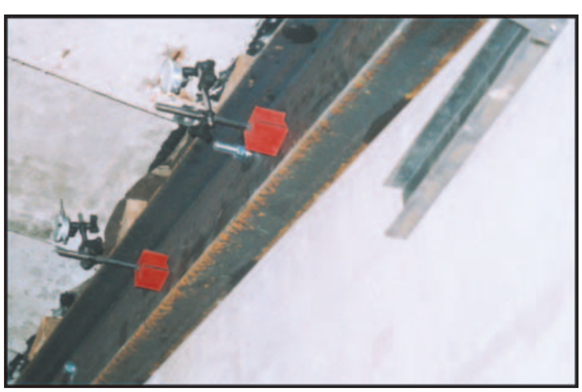


Repairs

The deteriorated concrete at top and underside grandstand surfaces was removed, corroded reinforcement cleaned and supplemented if needed. In some instances large full depth repairs were completed with conventional cast in place concrete, while smaller areas were repaired with site mixed proprietary mortars. Large areas of vertical surfaces were repaired with shot crete.

While preparing repair documents, CBI also discovered an error in the design of the cantilever portion of the grandstand beams under the grandstand roof. The roof framing was detailed in such a way that the column supporting the back span of the roof beam was supported on the cantilever portion of the grandstand beam. The ratio of roof spans was such that a net uplift of the roof column occurred under dead load plus full snow loading on the roof. The upward force on the column resulted in a net positive moment in the grandstand beam cantilever, thereby creating tension stresses in the bottom of the beam. Calculations using structural grade reinforcing steel (Fy = 33 ksi) indicated that this segment of the floor beam was 30% overstressed. In this case, the use of an in situ load test was logistically and economically impractical. Initially, during the design phase of the project, the introduction of supplemental tension reinforcement along the bottom of the existing beams was considered. However, it was decided for structural as well as cost benefit reasons to construct a new concrete site wall to counterattack the uplift, and also enclose the area under the grandstands. While the project was under construction, it became apparent that there was significantly more deterioration along the edge of the roof slab than originally estimated. At this juncture, CBI concluded that removing the outward 10'-0" of slab resulted in sufficient load balancing to eliminate the uplift. The cost for sawcutting and removing this portion of the roof was less than the costs required to salvage it, make the necessary concrete repairs and apply the waterproof coatings. Additional cost savings were also realized by deleting the concrete site wall.

As part of the evaluation/study phase of the project, a series of concrete cores were recovered for laboratory testing. The presence of chloride ions, often due to the use of de-icing salts, indicates a high potential for corrosion of reinforcing which can lead to spalling and cracking of concrete. According to the American Concrete Institute, chloride ion content should not exceed 0.15 percent by weight of cement in the concrete, which translates into about 0.02 percent by weight of concrete. The maximum tested value in Fraser Field concrete was 0.016 percent by weight of concrete, which is within the acceptable limits. Based on these results, CBI concluded that corrosion due to chloride ion concentrations was not a significant factor in the deterioration of the concrete in this structure. These tests also indicated that the concrete compressive strength was 5000 psi and higher.



Deflection Measurement During Load Test

During repairs to the existing concrete roof beams, deposits of silica gel appeared to be present in the demolished concrete. CBI immediately coordinated a program of concrete cores and sampling for laboratory testing and petrographic analysis. The tests indicated that the concrete had acceptable compressive strength (4650 psi minimum). The silica gel deposits were not heavy and the concrete was not extensively cracked, which suggested only moderate ASR. As a result of ASR, the concrete durability was compromised, increasing the rate of leaching and freeze thaw deterioration. Given the budget constraints, complete removal and reconstruction of the roof beams was not a practical or economical solution. The areas of deteriorated concrete were removed to sound concrete, forms erected and new concrete placed. Visible cracks were routed and sealed, and an elastomeric coating was applied to top and underside roof surfaces to mitigate moisture intrusion and subsequent freeze thaw cycling. The condition of the existing beams will be monitored annually, and repairs and recoating made on an as needed basis.

Conclusions

The restoration of Fraser Field Stadium illustrates the challenges that face the engineer. The completion of a successful project requires an ability to evaluate and solve specific and often unique conditions throughout the design and the construction process. These issues require engineering/technical knowledge and experience, with a clear understanding of the owner's financial limitations.

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Project Credits:

Lenox Hotel

SER: CBI Consulting, Inc., Boston, MA
Owner: Saunders Hotel Group, Boston, MA

Fraser Field

SER: CBI Consulting, Inc., Boston, MA
Owner: City of Lynn, Dept. of Public Works