Structural Forensics
investigating structures and their components

The Structural Investigation of a 19th Century Manufacturing Complex

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Structural engineers are often asked to provide input during the conceptual design phase of building projects. During the conceptual design phase of an adaptive reuse project the structural engineer's involvement is more complex as it becomes necessary to address the existing buildings' suitability for the proposed new uses in the absence of existing drawings and in the presence of any existing structural deterioration.

When the project involves a 19th century manufacturing complex with a large number of existing, historic buildings, the structural due diligence associated with the conceptual planning becomes very challenging. Questions that must be answered as a part of this initial effort include: What is the structural condition of the buildings today? How are the structures to be investigated non-destructively? Will non-destructive investigations tell us enough? What structural repairs are immediately required? What uses can be proposed based on the existing structural capacities? What upgrades for code requirements are necessary? Can upgrades be made to increase structural integrity or repair deteriorated areas without compromising historic integrity? And, the list goes on.

One structural engineering firm, Pennoni Associates, Inc. (Pennsylvania), took on this challenge when developers of the complex (located in the Eastern US) requested site/civil, geotechnical, structural and MEP services. The following is an overview of the process and findings associated with the structural effort.

Project Overview

The project involved the visual condition assessment of 22 separate or conjoined buildings ranging in height from one to four stories, not including any basements, for a total gross area of approximately 350,000 square feet. In addition, the investigation included a visual condition assessment of an existing pedestrian bridge between two of the main buildings, a site retaining wall around the west and north sides of another main building, and a series of retaining walls along the bank of an existing creek. The existing structures were erected between the early 1880s and the mid 1960s, with the majority of the buildings having been constructed before the early 1900s. All of the oldest buildings were constructed with wood-framed floors and roofs supported by load-bearing brick walls.

Specific tasks associated with the structural investigation included:

1) A site survey to evaluate the overall condition of the referenced structures and to measure the typical floor and attic framing member sizes, where applicable. The primary purpose of the survey was to identify any readily observable structural deficiencies or conditions that required repair, in order to enable the adaptive reuse of the structures. The condition survey was limited to visual observations made at readily accessible and exposed portions of the structures.
2) A structural analysis of the typical floor and attic framing elements to establish their approximate load-carrying capacities. The analysis was limited to the typical girder and beam framing and support columns, and attic roof trusses as required for the purposes of an adaptive reuse assessment.

Typical Building Elevation.

Condition Assessment

The results of the condition assessment indicated that most of the existing structures were in relatively fair to good condition and appeared to have adequate structural capacity to enable their adaptive reuse as residential, educational, office, retail, light storage or light manufacturing facilities. The existing buildings, however, did require a considerable amount of structural repair work to assure their safe use and continued service life. The total estimated cost to complete the structural repairs recommended by the investigation was approximately $3 million.

Queen Post Truss Spanning Between Gable Truss.

Pennoni identified two of the main buildings as being in such poor condition that existing holes in the roofs, and the entire wall and roof enclosures, needed to be repaired and made weatherproof as soon as possible. This was necessary in order to mitigate any further deterioration so as to maintain the potential for economical repair and renovation in the near future, in lieu of unavoidable partial collapse or demolition.

Another main building's condition, as a result of fire damage, was such that the extent of repair, reconstruction and renovation required would be considerably more than the cost to demolish it and construct a replacement. An adjacent smaller building, although in better condition, was only accessible through the same fire damaged building and served an ancillary role as a loading dock. Therefore, Pennoni recommended that both buildings be demolished and removed.

The instability of a portion of the attic and roof truss framing at another main building posed the potential for the partial collapse of the adjacent exterior gable end masonry wall onto an adjacent street. Pennoni recommended temporary shoring of the affected framing in order to make continued safe access along the adjacent street possible.

The existing site retaining walls located along the creek and around one of the main buildings required a significant amount of repair work. In addition, in the case of the wall at the main building, Pennoni recommended excavation of a portion of the soil retained by the wall in order to stabilize the existing condition.

The existing 120-foot-tall masonry stack associated with a boiler plant building appeared to be in good to fair condition as visually observed from the ground. However, because of the height of this structure, Pennoni recommended an additional and more thorough condition assessment of the entire stack.

Structural Analysis

The following assumptions served as a basis for the structural analysis of the typical framed floors and attics. Pennoni recommended that all analysis assumptions be verified and confirmed through additional material and/or in-situ, non-destructive testing, and limited selective exploratory demolition.

1) During the time period of the construction of the majority of the original timber structures, there were large quantities of Hemlock logged in the local woodlands for use in buildings; therefore, this species of lumber was assumed for analysis. Historical records indicate that Hemlock had allowable stresses of approximately 1,800 psi in bending and approximately 1,200 psi in compression parallel to the grain.
2) The primary timber floor support beams for two of the main buildings were constructed as two identical wood members bolted or sistered together. Butt joints of the individual pieces occurred over interior support columns, and were always offset such that only one beam joint occurred at any one location along the length of the member. For purposes of analysis, these members were assumed to be continuous over all of the interior supports, even though at least one of the members was discontinuous at the butt joint. This approach was based on the rationalization that the butt joints are capable of acting as a 100% negative moment splice of the individual beam, given the following assumptions:
   a) The nature of the butt joint allowed for the transmission of compressive stresses at the bottom of the individual beam, induced by the negative moment at the support columns via the direct contact of the adjacent butt ends of the adjoining wood beams.
   b) Tensile forces at the top of the individual beams induced by the negative moment at the supports were resisted at the butt joint by the spline jointed timber floor planking, which was assumed to be spiked together with at least 60d nails.

Collapsed 1st Floor Framing over Basement.

3) The primary timber floor support beams for another main building were constructed with a shallower timber member stacked on top of a deeper timber member. Because wood laminating was performed during the era when this building was constructed, for purposes of analysis, it was assumed that these two pieces of timber were laminated and therefore act compositely as one member.
4) Using AISC historical data from local mills producing steel beams in the late 1800s and early 1900s, an allowable bending stress of 16,000 psi was assumed for all steel floor beams.

continued on next page

3-Span Continuous Wood Beams.

Because of the lack of safe access, lack of adequate visibility or other similar physical limitations, access to the 1st floor framing from the basement areas below was not possible in any of the main buildings. As a result, the analysis of the existing floor framing only addressed the 2nd through 4th floors. However, it was reasonable to assume that the existing 1st floor framing had at least as much load-carrying capacity as the 2nd floor framing above.

The calculated 2nd, 3rd and 4th floor load-carrying capacities varied from 60 PSF to 160 PSF, 50 PSF to 125 PSF and 50 PSF to 55 PSF, respectively. The calculated attic floor load-carrying capacities varied from 10 PSF to 40 PSF. These calculated values only included the self-weight of the framing members and decking; therefore, the allowable live load capacity would have to be adjusted for any superimposed dead loads associated with new mechanical systems, ceilings, floor finishes or other similar appurtenances.

The result of the analysis indicated that the upper floors typically had less capacity than the lower floors. The reason for this condition can best be understood in the context of the 19th century industrial use specific to the site. When the existing manufacturing facility opened in the early 1880s, it produced only one product line, which involved raw material purchased in large bales. The subsequent processing involved the use of large machines that were located on the first floor of the buildings.

When the facility expanded into the manufacturing of additional products, large equipment was also used for the additional processing. These large and heavy pieces of equipment would not have been located on the upper floors of any of the buildings, which were likely used only for storage. Attic areas were also likely used for storage, or merely for access to rooftop water storage tanks.

Table 1 summarizes the minimum live load capacity specified by the governing building code (IBC 2006), which served as the basis for the adaptive reuse potential recommended by the investigation.

Table 1: Minimum Code Live Load Requirements.

Live Load	Utilization					
	Residential	**Educational Classrooms**	**Office**	**Retail**	**Light Storage Warehouse**	**Light Manufacturing**
Minimum Uniform	40 PSF (Notes 2, 5, 6)	40 PSF (Notes 3, 5, 6)	50 PSF (Notes 3, 5. 6)	100 PSF (Notes 4, 6)	125 PSF	125 PSF

Notes:
1) Based on IBC 2006.
2) 100 PSF at public rooms and corridors.
3) 100 PSF at 1st floor corridors and lobbies; 80 PSF at corridors above 1st floor.
4) 75 PSF at upper floors.
5) Not including 15 PSF partition load required by the Code.
6) Not including a minimum of 5 PSF for suspended ceilings, miscellaneous MEP loads, and floor finishes.

Construction Classification and Fire Resistance Ratings

Fire resistance ratings and sprinkler requirements are listed in Table 601 of IBC 2006, which indicates that Heavy Timber (HT) or Type IV construction has advantages over other non-combustible types of construction. This is because HT construction has greater fire resistance than unprotected structural steel.

IBC 2006 specifies minimum HT dimensions of 8 inches for columns per Section 602.4.1 and 6 inches (width) x 10 inches (depth) for floor framing per Section 602.4.2. In addition, floors must be constructed with splined or tongue-and-grove planks of not less than 3-inch thickness covered with 1-inch tongue-and-grove floor deck laid orthogonally or diagonally to the span of the plank.

The investigation indicated that not all of the columns and floor framing members complied with the minimum size requirements. In addition, floor beams that are sistered together result in a concealed space, which is prohibited by Section 602.4.

There are also prescriptive framing and connectivity requirements for HT construction specified in Section 2304.10. However, due to the limitations of the visual observations made during the site assessment, it was not possible to document the presence of all of the items prescribed in Section 2304.10. Consequently, Pennoni recommended a more thorough investigation in order to determine compliance.

Conclusion

The investigation determined that none of the buildings, in their entirety, qualify as HT or Type IV construction. This will directly impact the requirements for sprinklers and fire ratings of all of the buildings based on the adaptive reuse occupancy.

Currently this project is in the initial design phase for the renovation and adaptive reuse of a portion of the main buildings. Previously recommended emergency repairs have already been implemented. Subsequent phases for the adaptive reuse of the remaining buildings are in the early conceptual design phase. Ultimately, the entire site will be renovated so that it can provide an important economic contribution to the surrounding community.▪

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