Evaluating Historic Structures for Adaptive Re-Use

By Dominick R. Pilla, P.E., C.E., S.E., R.A. and Xiaoli Tong, P.E.

The H. Lawrence & Sons Rope Works complex, located at 221 McKibbin Street at the western edge of North Brooklyn's Industrial Business Zone, has been in continuous use for industrial purposes for more than 160 years. The Greenpoint Manufacturing and Design Center (GMDC), a nonprofit industrial developer of sustainable and viable manufacturing sectors in urban neighborhoods, purchased the property in 2007 and started renovation of the complex utilizing New Markets and Historic Rehabilitation Tax Credits (*Figure 1*).

GMDC selected the team of Oaklander Coogan & Vitto, Architects (OCV Architects) and structural engineer, Dominick R. Pilla Associates (DRPilla), to preserve the historic building and adapt it for modern uses.

Project Description

Built on poor soils with an adjacent subway, the existing nine-building complex was in need of structural remediation and renovation. The oldest three-story building's structure consists of 16-inch thick exterior brick walls, a roof truss system, two timber joist floor systems supported by cast-iron columns/walls, and a slab-on-grade ground floor, with a footprint of approximate 247 by 41 feet and a maximum height of 34 feet. Adjacent on the eastern façade is a two-story brick wall building, which consists of 12-inch thick exterior load-bearing walls, a flat roof, a timber joist floor supported by wide flange steel girders and columns, and slab-on-grade ground floor with a footprint of approximately 43.5 by 64 feet and a maximum height of 26 feet 3 inches. The remaining seven buildings on the site are all one-story flat roof buildings, including a 2,100 square foot new addition. The total building area is approximately 72,000 square feet. The scope of the renovation work consisted of the evaluation of the existing structures, the retrofit of structural deficiencies, and specific upgrades to isolated portions of the buildings to meet the loading requirements for various occupancies ranging from multiple light industrial to artisanal use.

The main challenge facing the structural engineer (DRPilla) was developing a systematic method to evaluate the patchwork of structural elements throughout the complex and evaluating the structural integrity of each for proposed new uses.

Condition Assessment

Lacking construction documents, visual inspection and detailed field measurements were the primary means used to assess the buildings (*Figure 2*). The buildings were examined, and areas of spalling and cracking were noted.

In addition to the anticipated deterioration due to time, large settlement cracks were observed on most of the one-story building exterior walls and at the south-west corner of the three-story building. Test pits and in-situ dynamic cone penetration tests were conducted adjacent to the existing wall footing. The testing revealed the bearing soil layer was mainly fill comprising sand with gravels, crushed concrete, etc. The sample fill had a low bearing capacity of less than 1.0 tsf.

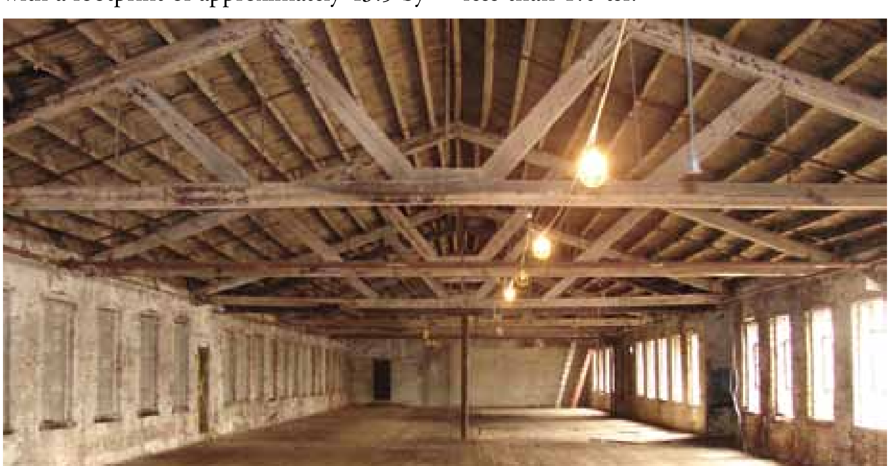

Figure 2: Lacking construction documents, visual inspection and detail field measurement were the primary means used to assess the buildings.

Figure 1: This former rope manufacturing complex now provides 72,000-square feet of light manufacturing and artisan workspace. Courtesy of Mireille Moga.

Since the building had been bearing on the soil for at least 60 years, the 1.0 tsf capacity was not as much of a concern for soil below the footings as was whether the back-fill was fully and uniformly compacted during construction. Differential settlement from non-uniform bearing soil was likely the primary cause of the settlement cracks and leaning interior columns observed throughout. Excavation of the column footings proved the footings were dimensionally adequate. However the footings were rotating at the corners, a condition indicative of improper in-filling and soil compaction when the building complex was initially built.

Most roof purlins presented varying degrees of deformation. Further exploration of the roof profile revealed multiple layers of roof finish material, which resulted from installations completed over time. The significant additional load of multiple roofing layers, as well as long-term exposure to moisture, were determined to be the main cause of the purlins' deformation. The deformations were so large that some purlins were considered to be failures in serviceability according to the current building code.

Building Code Compliance

In order to develop a reasonable structural renovation plan, DRPilla conducted a comparison between historic NYC building codes and the current code.

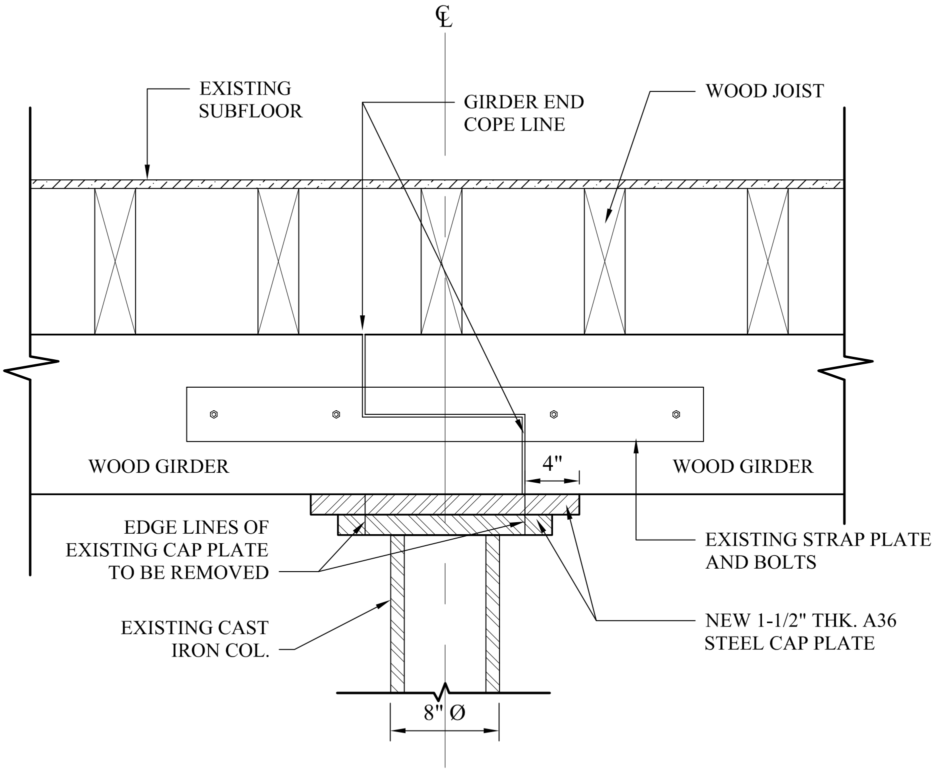

Figure 3: Retrofitted cast-iron column top plate connection.

The first NYC comprehensive building code was enacted in 1850, when construction for the complex began. Complete revisions were performed in 1938 and 1968 respectively. The current NYC building code is the 1968 NYC Building Code, with up-to-date supplements and local laws (Note: the latest is 2008 construction code which is effective July 2008).

The comparison shows that minimum design loads are slightly reduced in the current code for a light factory. For example, the floor live load was reduced from 120 psf to 100 psf in the current code; the roof live load was reduced from 40 psf (3:12 rise or less) to 30 psf in the current code.

Wind loads were generally not considered for a building under 100 feet high until 1968, when at least 20 psf pressure was specified. The fact that the building has persevered through the ages is proof of its resilience to loads imposed by wind, snow and gravity. However, all structural members and connections needed to be examined and verified. Based on the governing seismic code, local law 17/95, the new enlargement was comparatively small and seismic analysis was exempted for the alteration of the existing complex. Only the new addition was required to be designed for seismic resistance.

In all, the existing building structural system was deemed adequate with the decrease of design live loads as well as the roof dead load by removal of existing multiple roof material. The structural members and connections were analyzed and retrofitted in accordance with the applicable design code references.

Roof System

Multiple layers of a variety of roofing materials were removed in order to reduce the roof dead load and quantify the loading capacity of the roof. The roof purlins/joists with large deformations were replaced by sawn timbers of various sizes, depending on the spans.

The existing roof truss is a 41-foot-span timber double howe-type, with all vertical web members made of steel rods (tension ties). The atypical use of steel rods, instead of solid wood members, simplified the construction at the connections between vertical/diagonal web members and the top/bottom chord. At an average spacing of 20 feet, the roof truss analysis results showed gravity loads prevailed in design and all vertical chords (the tension-only rods) would not be in compression, even under the combinations of unbalanced snow load and wind load. The modified double howe roof trusses are capable and stable when compromised chords were repaired.

Floor System

The floors on every level of the main building were evaluated. Similar to the layers of roofing material, layers of flooring material needed to be removed in order to properly evaluate the condition and strength of the over 100-year old timber floors. Removal of the flooring material revealed rotted and fire-damaged areas.

Existing timber girders and joists were analyzed in accordance with the current analysis method. In the absence of the timber specie, typical "old growth" Douglas Fir was assumed, and values of the allowable stresses were obtained from the National Lumber Manufactures Association (now the American Wood Council). The results showed the floor structural members were adequate to resist the design floor load of 120 psf in total after renovation. Joists and girders with severe deterioration and large deformation were replaced by new timber joists and steel beams respectively.

Columns

Columns with severe corrosion and large incline were replaced by new steel columns. Corresponding concrete footings larger than original ones were designed to accommodate the poor bearing fill.

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Some of the existing girders were discovered to be coped with half depth to overlap each other. This connection detail provided for more uniform stress bearing on the top plate because the end reaction forces vary largely due to widely varying adjacent spans.

While this top plate connection detail benefitted the columns by reducing the possibility of eccentric loading at their tops, it impaired the coped girder because of a decrease to its shear capacity. An analysis showed the existing shear resistance at the coped section was only 70% of the required shear force. The existing top plate detail was replaced by a new design (*Figure 3, page 15*), which provided adequate bearing length from the cope line for the coped girder.

Foundation system

The existing spread footings were determined to be adequate. However, helical piles incorporated with grade beams were designed to strengthen the wall footing at the cracking south-west corner of the three-story building (*Figure 4*).The advantage of helical piles is mainly its minor disturbance to the existing building and the subway structure adjacent to the building during installation. The piles prevent further cracking of the existing wall and increased the building's integrity.

Masonry Walls

To retain as much of the existing structure as possible, portions of walls with compromised masonry were disassembled. These sections were re-built and tied back to the stable portions of the walls. Close coordination between the architect and engineer was crucial during this phase.

The previous NYC building code generally specified the minimum wall thickness for varying building height. This design idea is adopted as an empirical provision in the current NYC building code. For a 3-story building, 16-inch thick walls at the lower 2 floors and 12-inch thick walls at the top floor were required in 1920s, while only 12-inch thick walls are required for all three floors by the current code. The area of openings is less than 25% of the gross area on every load bearing wall. Therefore, the existing brick wall construction complies with the empirical provision in the current code. The stability and integrity of walls were not comprised when repaired and repointed.

Sustainable and Viable Historic Preservation Goal Achieved

Renovation of the 221 McKibbin Street complex was completed in 2009. The new complex contains 20 units accommodating various tenant space requirements, ranging from 1,200 to 7,200 square feet of affordable small light manufacturing and artisan workspace. The facility will house more than 100 full-time jobs when fully leased, preserving the industrial heritage of Brooklyn for a new generation of workers. Moreover, an economic analysis on investment and operation shows that the renovated complex is anticipated to create $181 million in total economic impact, and $19.7 million in federal and state fiscal impact. One hundred and one jobs were created during construction and an estimated 136 jobs will be created and maintained during the first 10 years of operation.

The Brooklyn Chamber of Commerce recognized 221 Mckibbin Street Industrial Center with its Historic Preservation Award at the 2009 Building Brooklyn Awards Ceremony held in the Brooklyn Navy Yard on July 15, 2009 (*Figure 5*). DRPilla's structural evaluation and remediation helped developer Greenpoint Manufacturing and Design Center achieve their vision of developing a new sustainable and viable manufacturing facility in an urban neighborhood. As a recipient of the Historic Rehabilitation tax credit, 221 McKibbin Street will be placed on the National Register of Historic Places.▪

Figure 5: 221 McKibbin Street Industrial Center was recognized with an Historic Preservation Award in 2009.

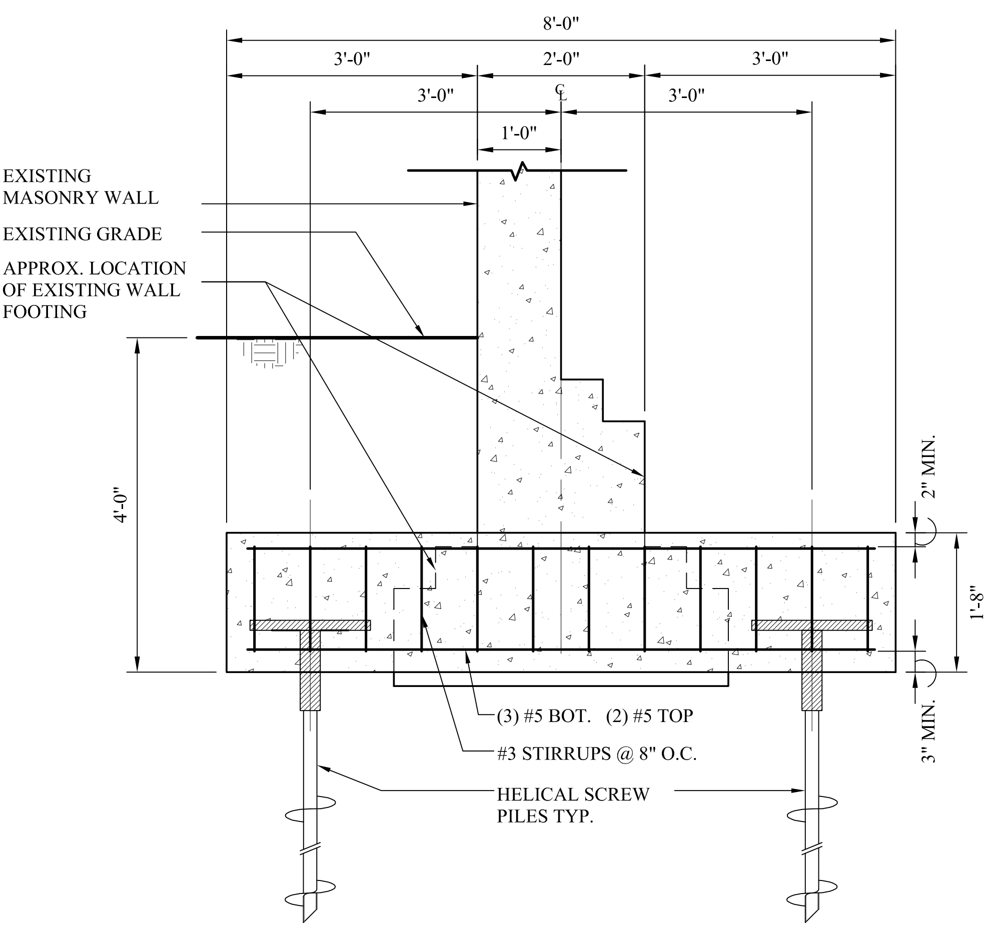

Figure 4: Helical Pile Grade Beam Support.

Dominick R. Pilla, P.E., C.E., S.E., R.A. is the Principal of Dominick R. Pilla Associates, P.C. in Nyack, New York. He is also an Associate Professor at the Bernard and Anne Spitzer School of Architecture at The City College of the City University of New York. He can be reached at ***dominick@drpilla.com****.*

Xiaoli Tong, P.E. is a structural engineer with Dominick R. Pilla Associates, PC in Nyack, New York. He can be reached at ***xiaolit@drpilla.com****.*