Shoring Up the Past NEW YORK CITY Style

By Alan M. Rosa, P.E. and Stephen Lehigh

The design of temporary shoring for existing buildings offers the engineer challenges on multiple levels, especially on vintage structures in New York City when not all the existing conditions can be known. This article presents a project that involved temporary shoring at the second floor of approximately ninety feet of exterior bearing wall and storefront of a depression-era six-story apartment building located on a busy intersection in midtown Manhattan. The building was continuously occupied during shoring operations. The design included an innovative rigid support of an excavation system designed for removal of existing foundation walls, and support of temporary shoring systems. The project, located at the corner of East 63rd Street and 3rd Avenue, is part of the construction of a new Metropolitan Transit Authority (MTA) 2nd Avenue Subway Line project. This building will serve as a new entrance to the 63rd Street/Lexington Avenue Station by way of a newly installed escalator entry.

Proposed Construction

The proposed permanent design creates access to the lower level of the subway station adjacent to the building using an escalator at the northwest corner of the building. The access point exits at street level within the envelope of the existing apartment building. During the construction of the new entrance, the building will receive a new reinforced concrete foundation wall that will replace an existing stone rubble wall. The masonry bearing walls and the existing storefront above the street level will be replaced with a new perimeter steel support frame. The new foundation will include a reinforced concrete slab, at approximately the same elevation of the existing basement, that will ramp down fifteen feet below the basement level for the new escalator.

Existing Building Construction

Typical multi-story residential construction of this era consisted of wood floor framing, masonry bearing walls, and perimeter steel storefront framing. The estimated temporary shoring loads of this project at the second level varied from 8 kips per foot (kips/ft) to approximately 13 kips/ft of wall. The foundation consisted of mortared stone rubble foundation walls, with brick masonry piers at existing column locations within the basement. The interior of the building is supported by steel beams and columns on spread footings.

Support of Excavation

A rigid support was required for both the support of the temporary shoring frames needed upon removal of the existing rubble foundation walls and for the deep excavation system below the basement level. Conventional methods were not possible due the proximity of the street to the building and the vast amount of existing street utilities. The supports of the excavation were required to be installed prior to any demolition, and their sequences were limited to occur after installation and preloading of the temporary support steel.

Figure 1. Elevation of temporary shoring system at north storefront and bearing wall.

A system utilizing eight 5-foot, 6-inch x 5-foot, 6-inch post-tensioned unreinforced concrete piers spaced at 9 feet on centers was developed. Hand excavated pits were advanced using horizontal sheeted timber rings forming a box, similar to conventional underpinning methods. The base of each pier was extended below the bottom of the proposed excavation to an adequate subgrade bearing strata. Once the concrete was poured and cured, the tops of the piers were post-tensioned using self-drilled rock anchors installed at a 1:4 slope to accommodate the proximity of the existing building foundation (*Figure 1*). The steeply sloped tiebacks were also advantageous to avoid the existing street utilities. The anchors were located at the center of the piers in a Polyvinyl Chloride (PVC) sleeve within the pier, and grouted into ledge rock. The embedment into rock ranged from twelve to fifteen feet and each tieback was tested to 133% of the anticipated 240 kip lock-off load, i.e. the design anchor load at the tallest pier. At the northwest corner of the building, the existing adjacent below grade station entrance framing was used for support of the temporary shoring system.

Temporary Shoring

The existing building is classified by the MTA as a fragile structure, defined as the limit of damage allowed is no more than very slight. This is defined as damage that contains fine cracks (up to 1⁄32 inch wide) in the exterior wall façade that are easily treatable and damage that is generally limited to interior wall finishes. The restrictions for the temporary shoring requirements were many, and included stringent tolerances on the maximum and relative movement. A maximum limit of ⅛ inch, with a threshold limit of 1⁄16 inch in both the horizontal and vertical directions, was required.

Of the variety of temporary shoring designs required on this project, including one for removal of a six story steel interior column, the two main systems discussed in this article will be one for the support of the existing perimeter beams over the storefront and another for the support of the existing brick bearing walls.

At Storefronts

Within the limits of the new entry, the existing structure was demolished up to the underside of the existing steel beams that support

Figure 2. Typical A-Frame shoring at storefronts.

the masonry above and span the storefronts. Thus, any temporary shoring system must leave clearance for the installation of new perimeter beams and columns installed directly below the existing second floor perimeter steel framing. In addition, a system was required to allow the installation of a new concrete foundation wall to replace the existing rubble foundation wall. An A-Frame system consisting of compression struts and tension ties was developed (*Figure 2*). The exterior ends of the frames were supported on the post-tensioned concrete piers, and the interior ends of the frame were supported on a steel frame system which in turn was supported on 3-foot by 4-foot concrete piers to a depth of approximately thirty feet below

Figure 3. Elevation of northwest corner with post-tensioned pier supporting the A-Frames.

the street level (*Figure 3*). The tops of the A-Frames were attached directly to the existing steel beams by welding the compression struts to the existing girder (*Figure 4, page 28*). The frames straddled the existing beam ends above the existing columns that were being removed so as to not change the existing support conditions of the beams; that is, one A-Frame at each beam end. The A-Frames were preloaded using hydraulic jacks at each side on the frame. The jacks were supported by channels connected to the main temporary girders just below the tension members of the frames. At other locations, it was possible to jack only from one side of the A-Frame. In these cases, the lateral movement of the frames at the apex due to the one-sided jacking was determined to be negligible. Jacking loads were limited to ninety percent of the calculated dead load plus a small allowance for live load throughout the building. Since each frame was jacked independently, the expectation was that little to no vertical movement would occur due to the restraint provided by the existing brick walls. Thus, it was important that the jacking loads be determined as precisely as possible, and that the A-Frames and supporting system would have a significant amount of extra strength available to confidently remove the existing steel columns supporting the perimeter storefront steel. Monitoring systems were installed to register any movements. As a result, all original building columns were removed successfully without any appreciable movement measured or cracking observed.

At Bearing Walls

Along East 63rd Street, a thirty-eight foot long section of an existing 16-inch thick brick masonry bearing wall was required to be removed to the same elevation as the adjacent storefront (*Figure 5, page 28*). Needle beams spaced at two feet on centers, on average, were utilized. As in the case of the A-Frames, the interior ends of the beams were supported on the steel frames, which in turn were supported on the 3-foot by 4-foot concrete piers. The exterior frames were supported on the post-tensioned concrete piers (*Figure 4*). At the second floor, there were four existing steel beams supported by the existing bearing wall that needed to be shored prior to wall removal. Hung beams adjacent to the wall being removed provided the support of the existing beams and were attached to the bottom flanges of the needle beams. Distribution beams adjacent and parallel to the exterior wall were required at the larger window openings, and at the existing fire escapes, to evenly distribute the wall load to all needle beams (*Figure 6, page 28*). In order to uniformly load the existing wall, jacking methods were not practical given the various frame deflections required to preload the system. It was therefore determined that the best approach was to wedge-shim each needle beam at each end to the required vertical displacement and then to pack it with flat shims to provide uniform bearing support. The required displacement was determined from an estimate of the uniform wall load and the tributary width of an individual needle beam. In order to account for deformations in the piers and the steel columns, and the deflections of the supporting frames, a comprehensive structural analysis was performed on both the interior and exterior frames to determine the exact level of shims required for each needle beam. Shimming could not commence until the entire web space and the top flange was packed with grout so that the masonry bearing area was increased and the masonry

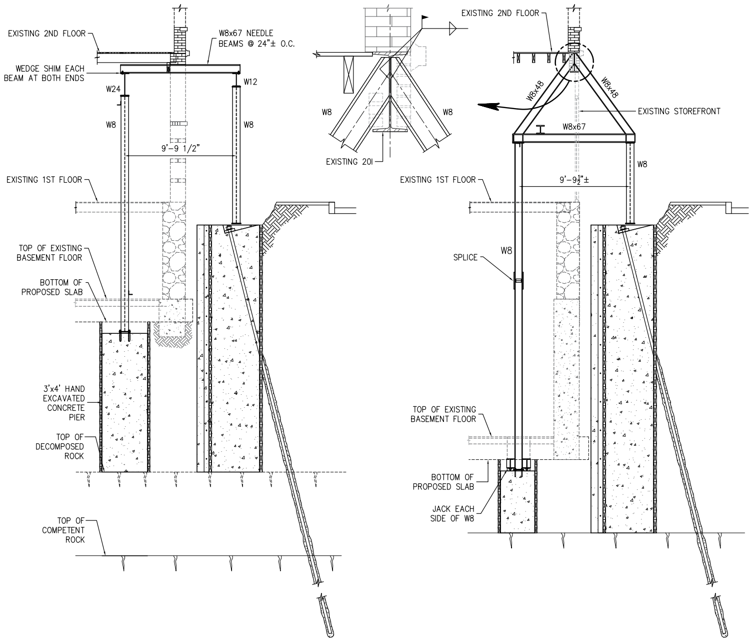

Figure 4. Typical A-Frame and needle beam construction sections at existing storefront and bearing wall.

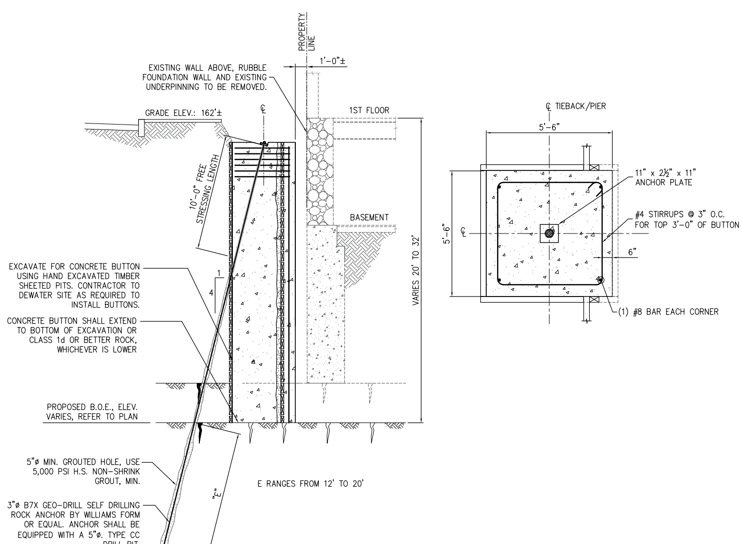

Figure 5. Post-tensioned concrete pier construction.

Figure 6. Needle beam support at existing masonry bearing wall with window distribution beam.

was not overstressed (*Figure 6*). Concrete piers were monitored for settlement during shimming operations, with the understanding that the required design shim thicknesses may need to be adjusted if settlement occurred. The heaviest loaded needle beams were shimmed to an estimated mid-span concentrated load of 24 kips. Once completed, the masonry wall was removed in sections starting directly under the bottom flanges of the needle beams. Removal was completed within two days with no movement or cracking registered.

Monitoring

One of the more important aspects of this temporary support design was the monitoring of the existing building for displacement and rotation. With close monitoring and tight restrictions given for movement and rotation, it allowed adjustments to the design if unintended movements occur. Therefore, the implementation of a sound, well thought out monitoring program was an important design consideration for this project. On this project, multiple prisms and rotational meters were strategically placed at the face of each building elevation. All instruments were measured continuously from a remote location. The tops of the post-tensioned concrete piers were monitored during preloading of the A-Frames and the needle beams, in anticipation of possible settlements and possible horizontal displacements due to relaxation of the post-tensioning anchors. Monitoring will extend to the end of construction.

Conclusion

The design of temporary shoring systems in New York City offers the engineer many challenges given special constraints, existing utilities and unknown conditions. Innovative, yet practical solutions are necessary to achieve the desired result, one that is cost effective and on time. A sound monitoring program is an essential part of the temporary shoring design and construction.▪

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