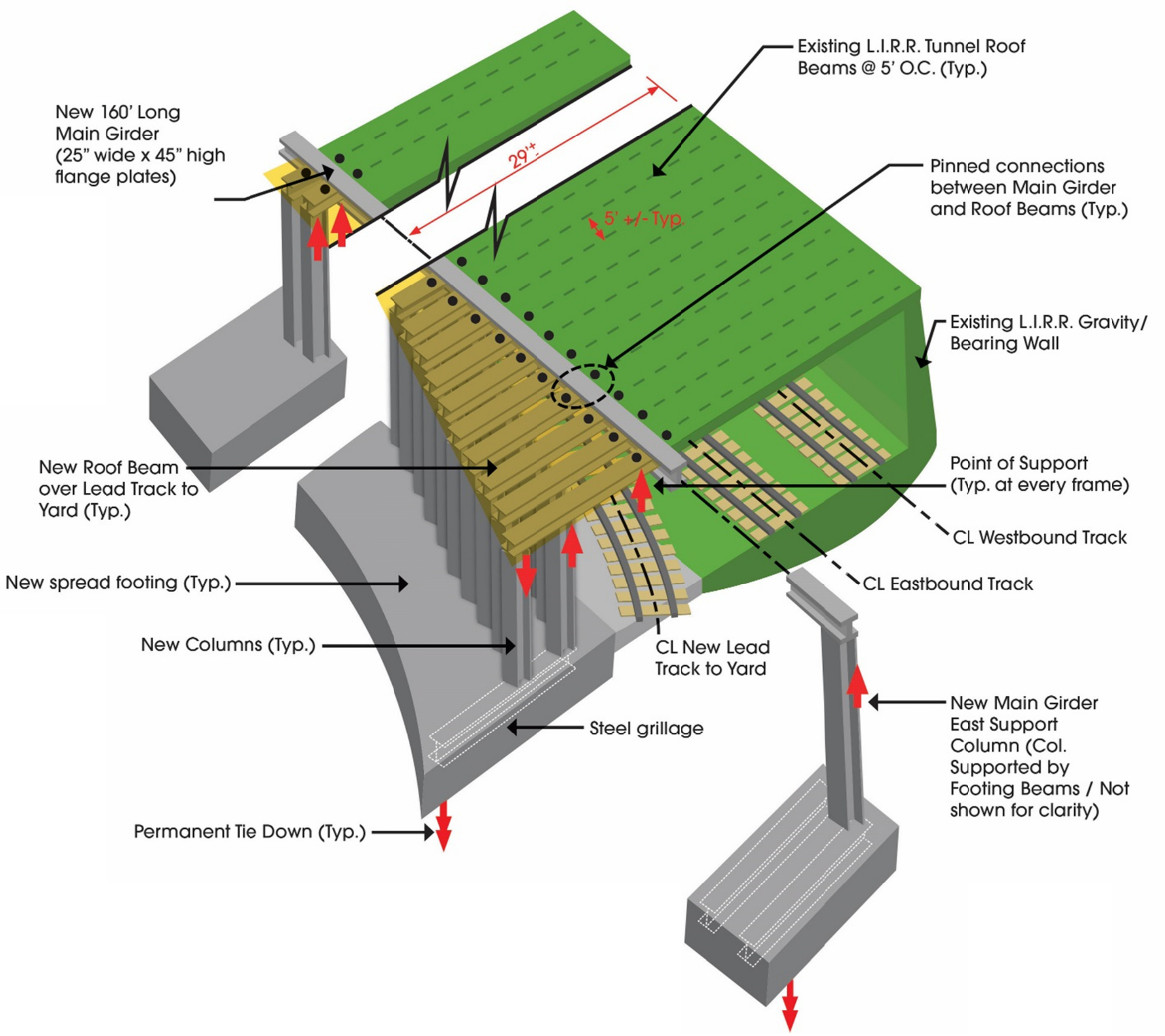

Figure 6. Illustration of the structural system for the center portion of the West Portal.

The columns in this section are also connected to steel grillage beams. The frames are supported on spread footings in this section. Just as in the western section, permanent tie downs were used in a limited number of frames where the uplift and bearing pressure made for an impractical heel length. The east end of the main girder is supported directly by a column which is supported by a series of footing beams and permanent tie downs.

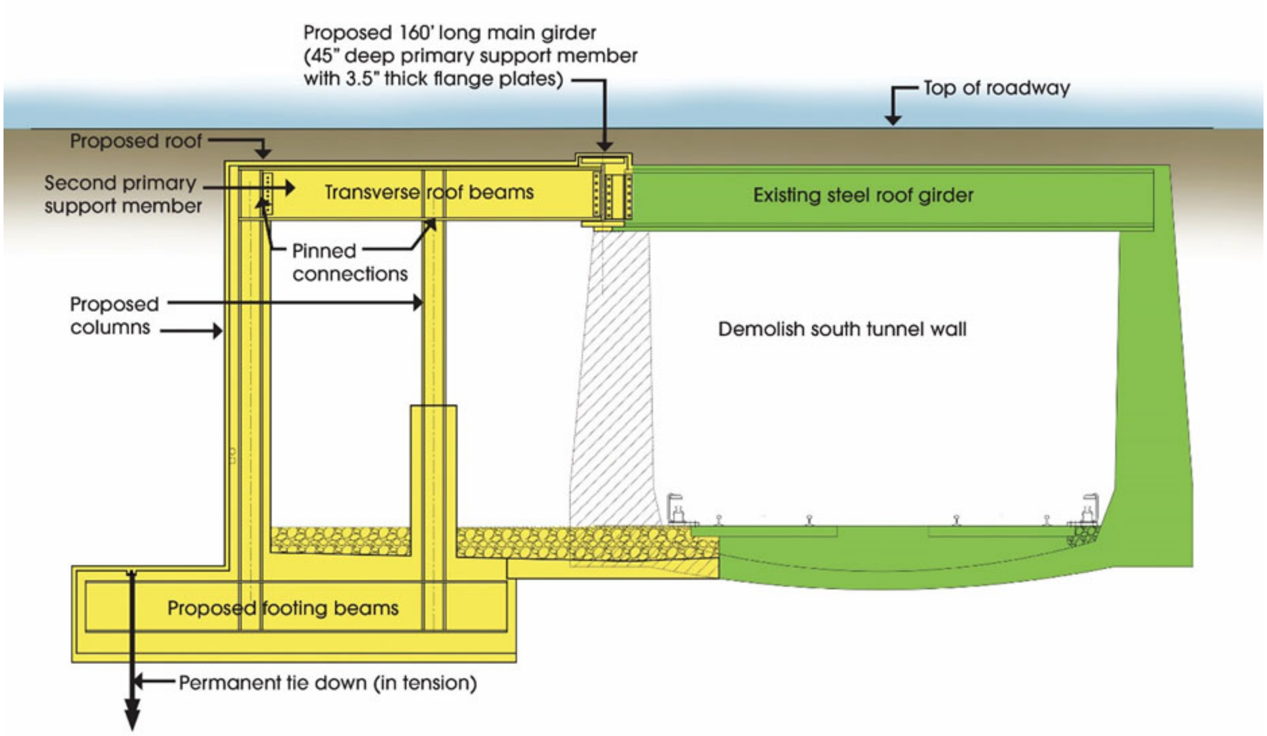

Figure 7. Section showing two column steel frame in center segment of West Portal.

The Central section of framing was modeled with a 3-dimensional model using STAAD Pro software. The proposed main girder, roof beams and columns, and footing beam elements were completely modeled along with existing tunnel framing. This was done so that the stiffness of all proposed and existing elements could be properly captured. This was especially crucial since the main girder was supporting loading in conjunction with the transverse roof beams.

The existing roof beams were modeled to have pin connections on both ends. The proposed roof members were modeled to have a pin connection into the main girder to expedite construction. As previously mentioned, the south column connection is fixed at western frames and pinned at others while the interior column for frames where there is one, has a simple propped connection. Live loads were moved within the model to capture the worst case effects. The footing beams were modeled to be supported by a series of soil springs which were closely coordinated with the soils engineer. See Figure 8 for the schematic of the 3D model which also shows the soil springs.

Results from the model were then analyzed on a frame to frame basis. When bearing pressure would result in an impractical spread footing length, a permanent tie down was introduced to assist in reducing the bearing pressure due to the overturning. All design and engineering analysis of the new and existing tunnel were done consistently with the original tunnel design.

Anticipated dead load deflections of the roof were computed with the 3D model. This deflection was accounted for by the pre-cambering the main girder up to 2.2”. The proposed and existing roof members were checked to ensure the minimum train envelope was accommodated even after dead load deflections.

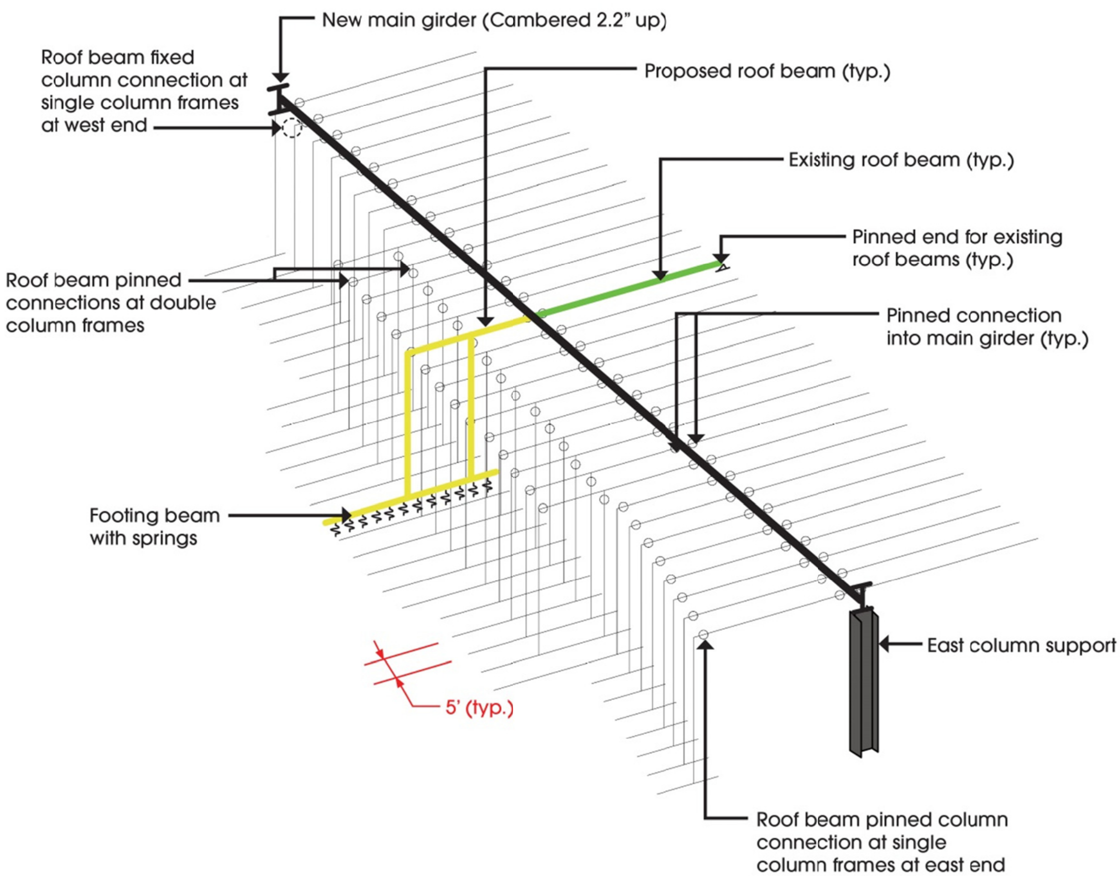

Figure 8. Isometric view of STAAD 3D model.

The last section, which is approximately 115 feet long, is a new tunnel that does not directly connect with the existing tunnel. This section is also composed of steel framing encased in structural concrete but is complicated by the need to construct utility troughs which had to be supported by built up steel girders (see Figure 9). The girders span approximately 64 feet and run parallel with Atlantic Avenue.

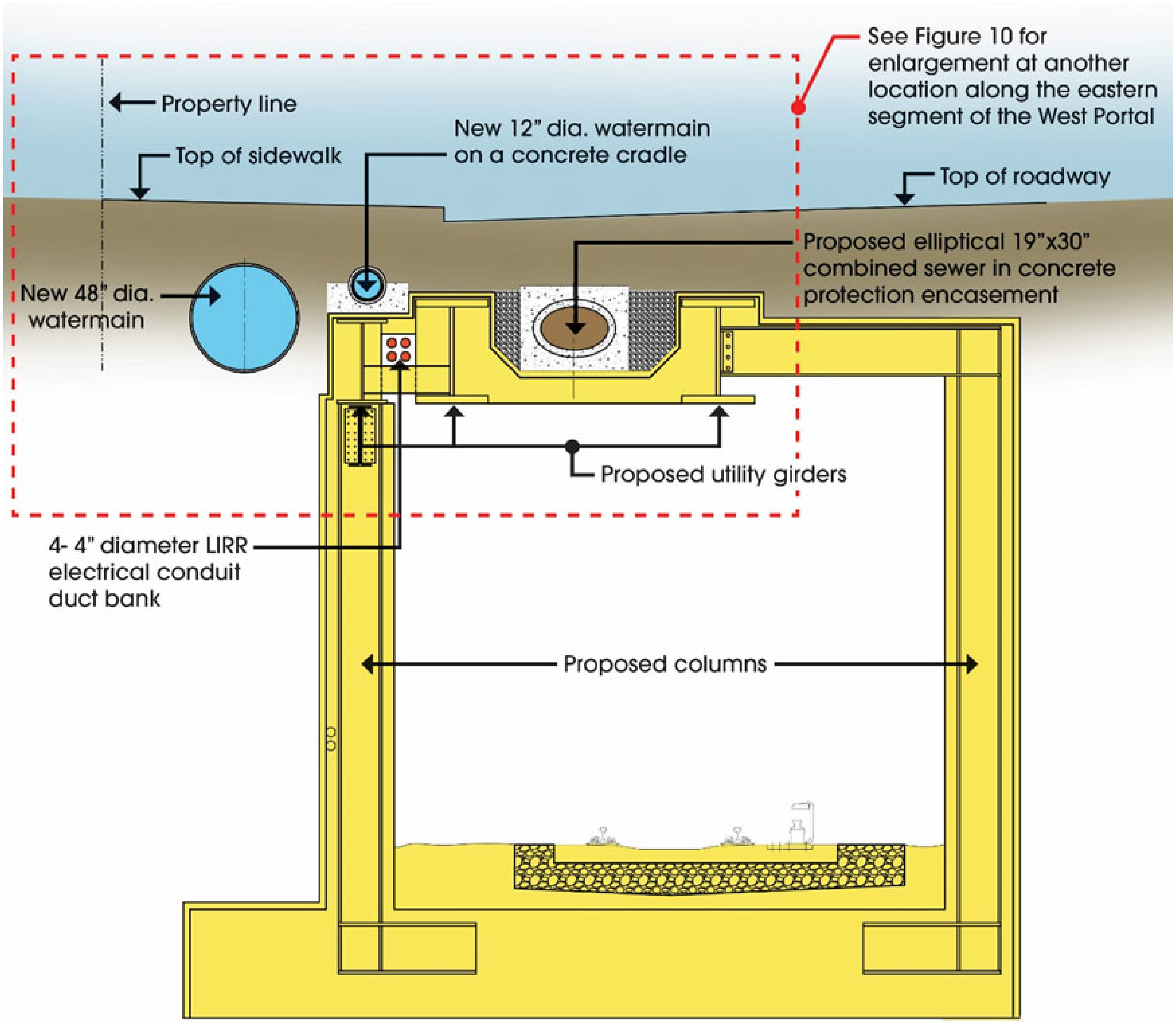

Figure 9. Section through the eastern segment of the West Portal.

OTHER DESIGN CHALLENGES

There were significant modifications to the utilities along Atlantic Avenue. With much of the existing utilities aged beyond their serviceable life, all utilities were replaced and then supported in place. Directly east of the Atlantic and 6th Avenue intersection, the new portal structure had to be specifically designed to support the new utilities through two troughs in the structure (See Figure 10). A new 48" diameter watermain, a 12" diameter watermain, a 19"x30" combined sewer, and four – 4" diameter LIRR electrical conduits pass above the new West Portal tunnel structure. The troughs were designed in a manner that structurally isolates the West Portal structure from these utilities. Obtaining the separation between the structure and utilities all while maintaining proper clearance and protection over the utilities was an important parameter to allow for future utility repairs and replacements without impact to the tunnel structure. This solution had to be heavily coordinated with the LIRR, New York City Department of

Environmental Protection (NYCDEP), Con Edison, Verizon, and New York City Department of Transportation (NYCDOT) and was eventually approved by all parties.

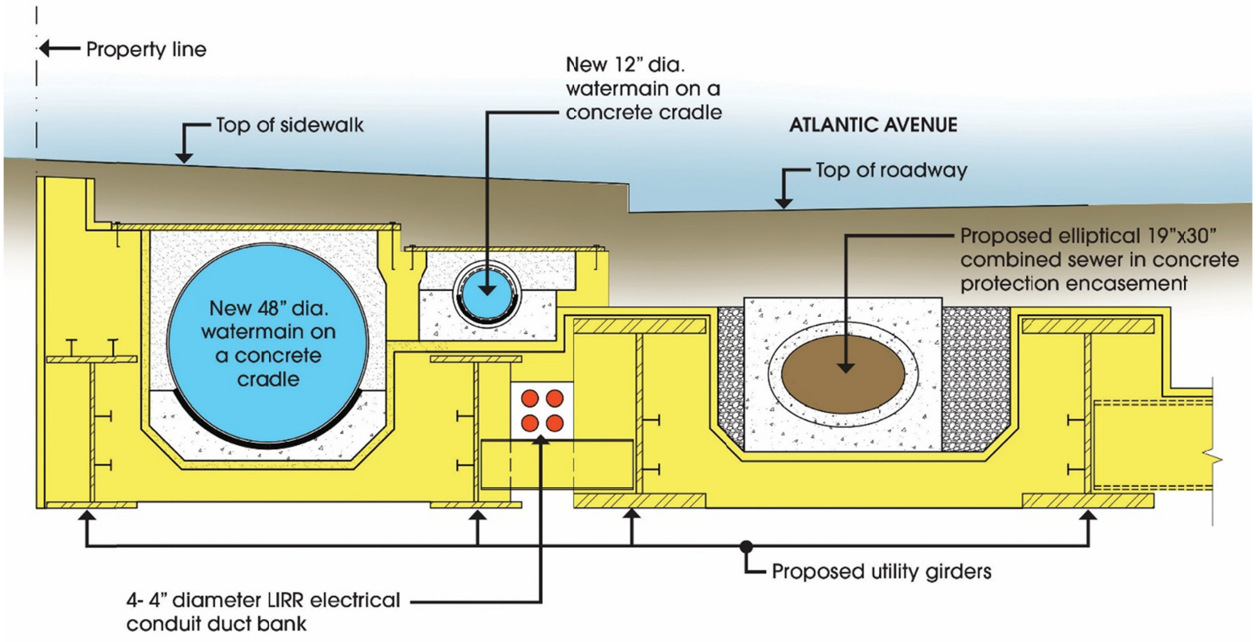

Figure 10. Utilities supported by the West Portal structure.

Obtaining a minimum of 18" cover over the NYCDEP infrastructure was a primary constraint in redesigning the 500 LF of the Atlantic Avenue roadway including an intersection with 6th Avenue. Significant coordination between the top of structure, top of utility, and top of roadway elevation was critical to finalizing the design. Much of the Atlantic Avenue and 6th Avenue intersection had to be raised to accommodate these critical clearances. Raising any street in NYC presents challenges, however, raising the south side of Atlantic Avenue approximately 12" presented a number of challenges including maintaining low points for drainage, reconstruction of a large number of roadways and sidewalks, and the interface of the proposed sidewalk with the existing elevations at property lines and adjacent roadways and intersections.

Although the West Portal Tunnel project physically only impacted one intersection due to the Atlantic Yards and Pacific Park developments, Stantec had to assess the traffic impacts from a number of various construction projects surrounding the project site including 5 new high-rise residential buildings, a new yard substation, and the Barclays Center green roof installation. Stantec had to analyze the impacts from each of these projects and implement various traffic mitigation measures which would be installed at a wider context in the surrounding neighborhoods. See Figure 11 for an illustration of the Maintenance and Protection of Traffic for the West Portal construction work zone.

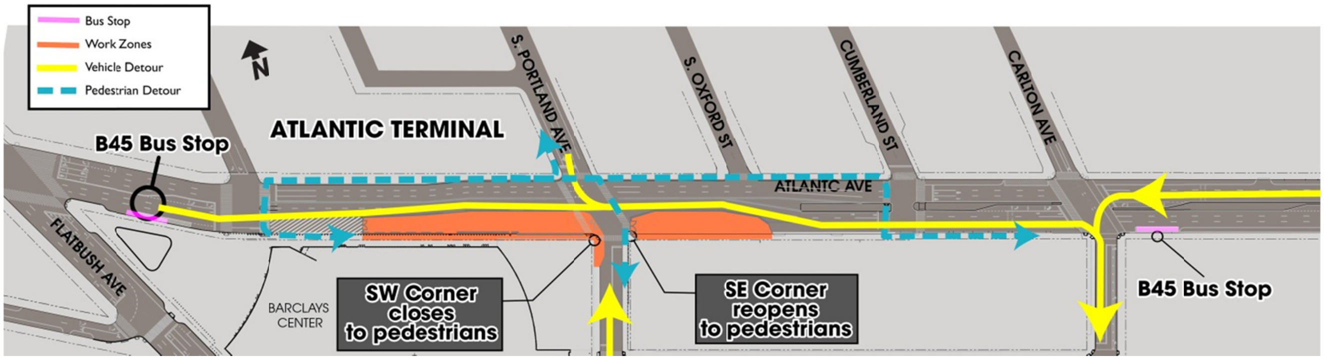

Figure 11. MPT coordination in the vicinity of the West Portal work zone.

CONSTRUCTION PROCEDURES AND DESIGN CHANGES DURING CONSTRUCTION

To facilitate construction of the West Portal structure, soil between the existing tunnel and the property line had to be excavated. A secant pile wall near the south property line was installed to facilitate both construction of the West Portal structure and the proposed work on the LIRR yard. Temporary decking was installed over the excavation area to allow traffic to run in that footprint on Atlantic Avenue. The decking was supported onto the existing roof as well as the already installed secant piles (this temporary decking and secant pile wall system was developed by another consultant). See Figure 12 and Photo 2 for a section showing this system.

Additionally, the transverse roof members at four frames had to be jacked down in order to make the connection between the proposed roof beam and the main girder (this is also shown in Figure 12). On these four frames, the Contractor used a pancake jack below the main girder top flange. These jacks pushed the transverse roof members down to facilitate the connection to the main girder. A side benefit of this jacking was that the roof system received preloading.

The tie downs discussed above were the result of a change that occurred during construction. Tie downs were considered during the design phase but piles were selected instead at the time because the same types of piles were being used in other parts of the project. The original design called for 54 piles which served the function to assist in resisting uplift at various bents. During construction, permanent tie downs were brought back on the table and ultimately chosen instead of the piles. The tie downs are 1 ¾" diameter 150KSI threaded bars. The system chosen involves grout bonded multiple corrosion protection anchors which offer excellent corrosion resistance.

Construction of the West Portal by Posillico-Tully Joint Venture began towards the end of 2014 and is scheduled to be completed by the end of June in 2017.

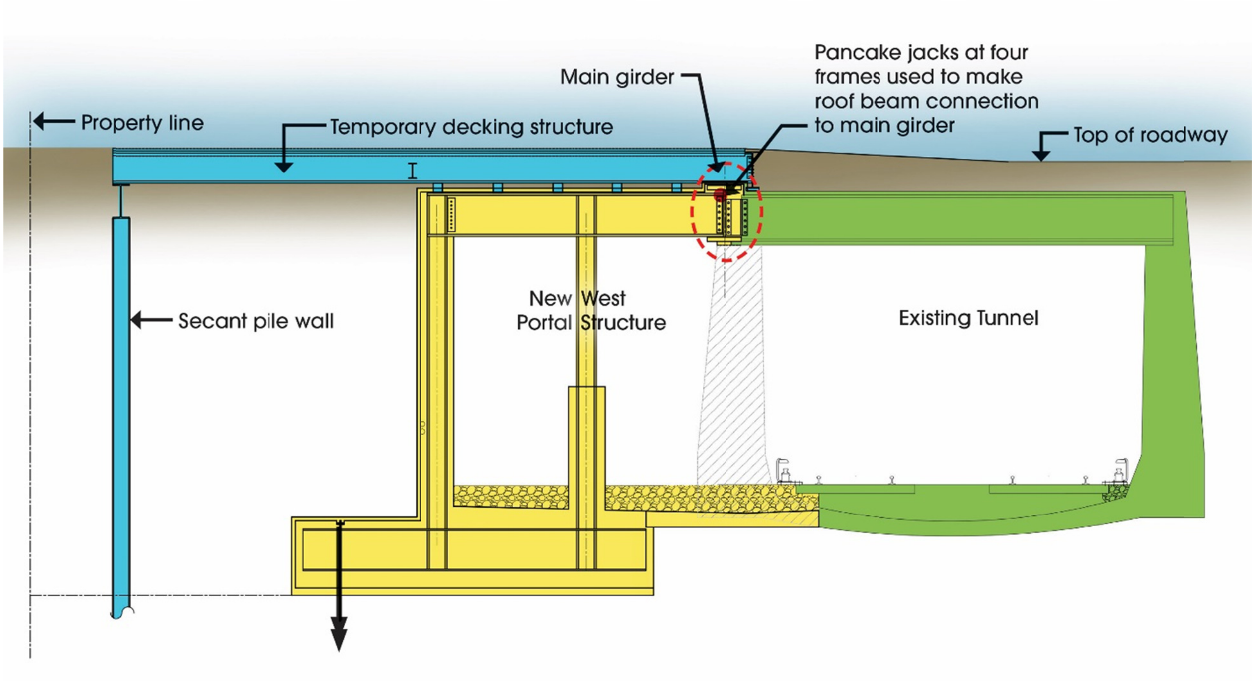

Figure 12. Temporary decking and main girder jacking during construction.

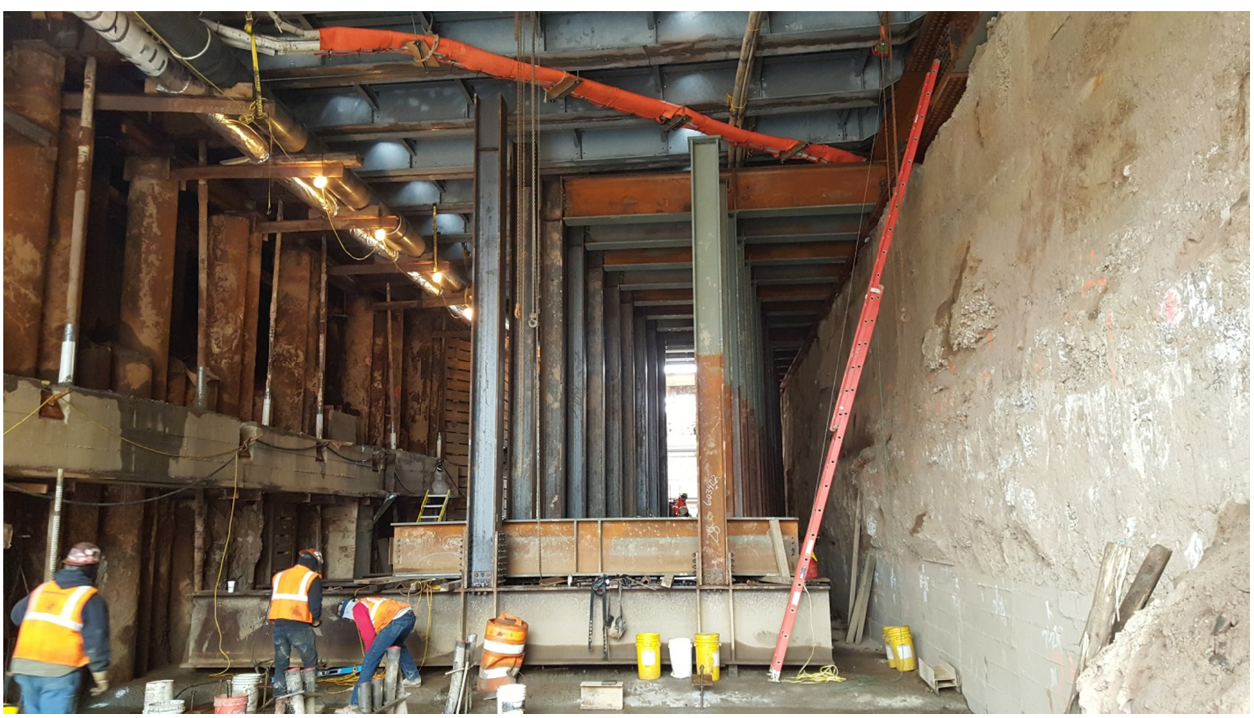

Photo 2 –Photo showing secant pile wall and temporary decking over proposed West Portal structure steel frames and main girder - Courtesy of Stantec Consulting

CONCLUSION

A new West Portal structure was required for a new lead track to / from the reconfigured LIRR Atlantic Yard as part of the overall project that involved the construction of Barclays Center as well as the development of the air rights over the rail yard. With the reconfigured yard, LIRR operations will now see the benefit of added flexibility as they will now have two points of access into the existing 2 track tunnel.

The existing LIRR rail tunnel brings in 25,500 daily passengers (Mass Transit Authority 2010). A traditional approach for a new portal into the yard involved replacing the entire existing tunnel roof and would have resulted in significant impacts to LIRR operations, vehicular / pedestrian traffic on Atlantic Avenue, and the community.

The design team developed an innovative approach by using a structural system that would involve two main load carrying systems. The use of a new 160 foot-long built up steel main girder along with a series of steel transverse frames that have cantilever roof members, each assisting in carrying the roof load of the south end of the existing LIRR tunnel roof framing, allowed for the existing tunnel to remain mostly intact during construction.

To perform the structural design of the West Portal, a complex 3D STAAD model was produced of the new structure. This model took into account the relative stiffness contributions from all the major steel components of the structure. Structural members were sized based on the loading that was captured from the model. Where required, permanent tie downs were used for global stability.

Our innovative solution meant that from the existing tunnel structure, less than 230' of the existing south wall of the LIRR tunnel would have to be demolished to allow for the new West Portal structure connection. Although there were impacts even with this extremely innovative solution, the impacts with a traditional approach would have been orders of magnitude more severe. More of Atlantic Avenue would have been closed off to vehicular and pedestrian traffic for a longer construction period which would also have meant more utility impacts and of course more costs. Additionally, the LIRR would have seen a significant amount of track outages with a more traditional approach. Ultimately the West Portal structure will be completed in the end of June in 2017 and will result in a new structure with significant benefits to LIRR and with minimized impacts to Atlantic Avenue and the community.

REFERENCES

MTA (Mass Transit Authority). (2010). "New LIRR Atlantic Terminal Pavilion Opens to the Public" [Press Release], <http://www.mta.info/press-release/lirr/new-lirr-atlantic-terminal-pavilion-opens-public> (Jan. 5, 2010).

MTA (Mass Transit Authority). (2015). "Average Weekday Subway Ridership", < http://web.mta.info/nyct/facts/ridership/ridership_sub.htm> (2015).

NYSDOT (New York State Department of Transportation). (2014), "Average Daily Traffic, 2014" < http://gis3.dot.ny.gov/html5viewer/?viewer=tdv> (2014)

Rapid Seismic Repair of Severely Damaged Reinforced Concrete Bridge Piers

Ruo-Yang Wu, S.M.ASCE[1]; and Chris P. Pantelides, Ph.D., S.E., M.ASCE[2]

[1]Dept. of Civil and Environmental Engineering, Univ. of Utah, Salt Lake City, UT 84112. E-mail: ruoyang.wu@utah.edu
[2]Dept. of Civil and Environmental Engineering, Univ. of Utah, Salt Lake City, UT 84112. E-mail: c.pantelides@utah.edu

Abstract

A repair technique for severely damaged cast-in-place and precast concrete bridge columns has been developed that utilizes a carbon fiber-reinforced polymer shell and epoxy anchored headed steel bars to relocate the column plastic hinge. This shell encloses the headed steel bars and is filled with non-shrink concrete to a certain height to form a repair "donut". The repair also employs a steel collar with steel studs that is attached to the column to increase the bond between the column and repair concrete. Three columns were repaired using the repair method described. The repair successfully relocated the plastic hinge at a section adjacent to the "donut" and was capable of restoring the load and displacement capacity. The steel collar combined with the "donut" were effective in making the repair system work in a composite manner. The method is rapid and could be classified as an accelerated bridge construction technique.

Keywords: ABAQUS; Bridge; Earthquake; Fiber reinforced polymer composites; Finite Element Analysis; Plastic hinge relocation; Repair.

INTRODUCTION

Based on current seismic bridge design methodology, damage is to occur at the ends of bridge columns to avoid other types of failure. Seismic repair of damaged columns is preferable to replacement; benefits include rapid construction and decreased interruption of regular service in addition to cost-savings (Parks et al. 2016). Significant research efforts have been made to study seismic repair and retrofit of Reinforced Concrete (RC) bridge columns. Many alternatives have been studied for the repair of damaged columns; rapid repair methods include the use of carbon fiber-reinforced polymer (CFRP) shells (Parks et al. 2016) and prestressed steel jackets (Fakharifar et al. 2016). CFRP composites are used because of their high-strength, light weight and resistance to corrosion. A CFRP shell also serves as a stay-in-place concrete form and provides continuous confinement to the column concrete and corrosion protection for the steel reinforcement.

Recently, grouted splice sleeves (GSS) have been studied (Ameli et al. 2015) to be used in moderate to high seismic regions, consistent with accelerated bridge construction (ABC). A practical and successful post-earthquake repair method has been developed to relocate the plastic hinge region for columns with GSS (Parks et al. 2016).

There is little research regarding the repair of severely damaged RC Cast-in-Place (CIP) bridge columns, common in existing bridges; after strong earthquakes, the longitudinal reinforcement may experience severe yielding, leading to buckling or fracture. A rapid repair method utilizing a CFRP shell and epoxy anchored headed steel bars to relocate the column plastic hinge is developed for CIP bridge columns. The CFRP shell encloses the headed steel bars and is filled with non-shrink concrete to a certain height to form a repair "CFRP donut". The method developed in this paper has been designed and applied on two severely damaged CIP specimens, a cap beam-to-column connection and a footing-to-column connection. The repair method is found to be practical and efficient for seismic repair of severely damaged CIP bridges. The repair is relatively fast to construct and implement compared to traditional techniques and thus it could be classified as a rapid repair technique.

EXPERIMENTAL INVESTIGATION OF ORIGINAL SPECIMENS

Original specimens

Three specimens, referred to as CB-CIP-O, F-CIP-O and CB-PC-O, respectively were designed based on current seismic bridge design standards(AASHTO 2011). Notation CB stands for Cap Beam-to-Column connection and *F* represents a Footing-to-Column connection; letter O stands for original and R for repair test. The corresponding repaired specimens are referred as CB-CIP-R, F-CIP-R and CB-PC-R.

The geometry and reinforcement of the original specimens, including a load stub, column and footing is shown in Figure 1. The column is 8.5 ft (2.59 m) high with a 21 in. (533 mm) wide octagonal cross section and an effective column height of 96 in. (2438 mm) measured from the top of the cap beam/footing to the centerline of the load stub, which represents the inflection point of the column. The longitudinal reinforcement consists of six No. 8 (25 mm) bars arranged in a circular pattern. A No. 4 (13 mm) Grade 60 (414 MPa) spiral at a 2.5 in. (64 mm) pitch is provided for transverse column reinforcement. The footing is 6 ft (1.82 m) long, 2 ft (610 mm) deep, and 3 ft (914 mm) wide. The pier cap beam is 9 ft (2.74 m) long, 2 ft (610 mm) deep, and 2 ft (610 mm) wide. The concrete compressive strength measured on the date of testing was 6.7 ksi (46 MPa). The measured yield strength of the longitudinal and transverse reinforcement was 68 ksi (469 MPa) and 63 ksi (434 MPa), respectively. The measured ultimate strength of the longitudinal and transverse reinforcement was 93 ksi (641 MPa) and 103 ksi (710 MPa), respectively.

Original specimen results

The original test results are summarized in Table 1 in terms of maximum lateral load, ultimate drift ratio and failure mode. The failure mode of CB-CIP-O and F-CIP-O was fracture of the two extreme longitudinal bars. The CB-CIP-O and F-CIP-O were tested to drift ratio 10.4% and 9.7%, respectively; CB-PC-O was tested to a 5.9% drift ratio. Figure 2 shows the original column damage where extensive spalling and cracking occurred in the plastic hinge region; flexural cracking extended to 16 in. (406 mm) away from the interface.

Five-level damage states (DS) were used to evaluate damage of the original specimens (Vosooghi and Saiidi 2010). DS-5 is the start of core damage indicating imminent column failure. The original specimens had reached a damage state designation of DS-5 since fracture and buckling of the reinforcing bars had occurred, leading to significant reduction of the lateral load-carrying capacity of the columns. It is difficult to repair structural components with a damage level of DS-5; this requires removal of crushed concrete, and replacement of the buckled and fractured steel bars. The method proposed in this paper is a feasible repair option for severely damaged as-built CIP columns with minimal intervention.