A Method for Underpinning Bridge Foundations & Its Application in the NSC Project in Pittsburgh

The method described herein provides a practical way to underpin bridges in congested urban areas where virtually zero displacement is needed during and after construction.

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The North Shore Connector Light Rail Transit Project, owned by the Port Authority of Allegheny County (PAAC), in Pennsylvania, is a 1.2-mile extension of the existing underground light rail transit (LRT) system from the current terminus in downtown Pittsburgh to the north shore of the Allegheny River. Figure 1 illustrates the extension of the Gateway Line north under the Allegheny River; it also details future expansion possibilities to the north and west. The existing LRT system already serves southern communities of Pittsburgh with a dense network of rail and stations. The current project will provide major transportation improvements to the north shore, which is home to significant new development — including new professional baseball and football venues as well as various commercial expansions (see Figure 2).

As part of the North Shore Connector (NSC) LRT Project, PAAC is constructing underground stations and tunneling using both cut-and-cover construction methods and tunnel boring technologies to extend the system below the Allegheny River to its new terminus just north of Heinz Field. On the north shore, the LRT tunnel alignment extends west across Tony Dorsett Boulevard where the alignment crosses directly below three separate foundations of State Route 65 (SR 0065, also known as Ohio River

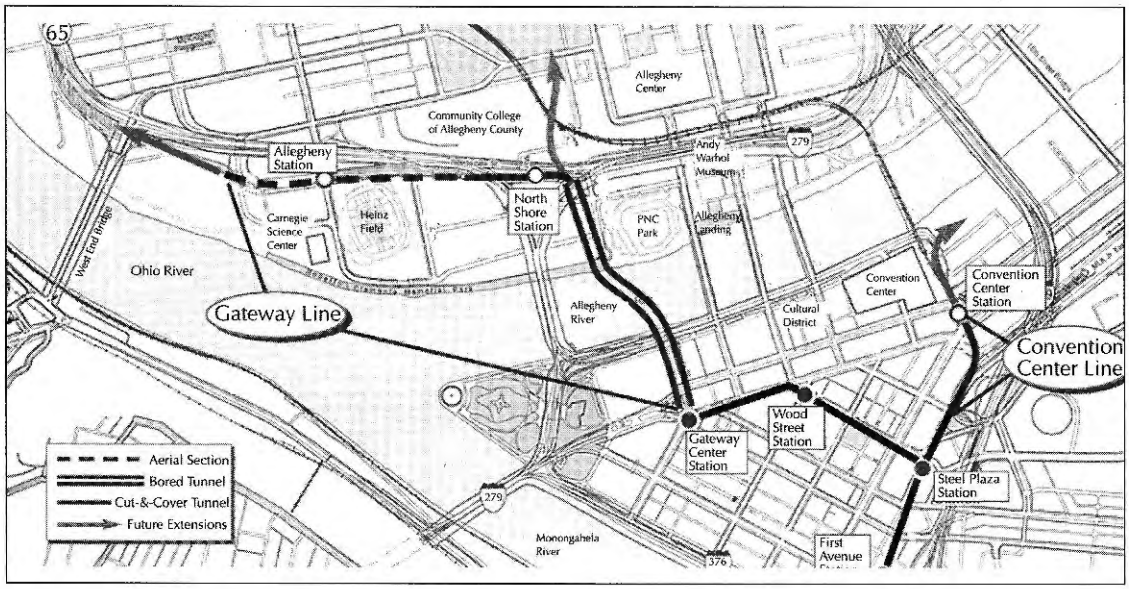

FIGURE 1. North Shore Connector LRT project plan overview.

Boulevard) northbound and southbound approaches to the Fort Duquesne Bridge, requiring the underpinning of the existing SR 0065 foundations.

Contruction on the project began in 2006. The tunnel boring machine started mining the first tunnel under the Allegheny River in January 2008. Work on the first tunnel was

FIGURE 2. Aerial schematic of the LRT north shore alignment.

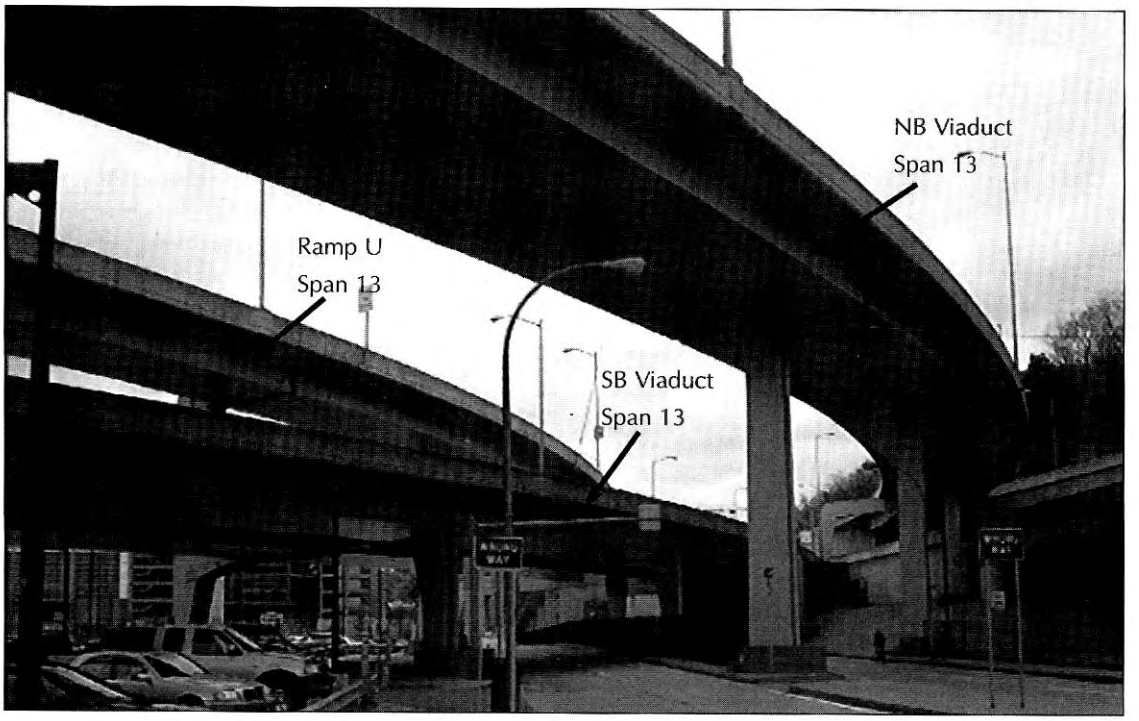

FIGURE 3. Adjacent SR 0065 NB and SB viaducts, with the Bent 13 expansion joint location shown.

finished in July 2008, and the second tunnel was done in early 2009. Current schedule calls for the NSC to be open in 2011.

Existing SR 0065 Viaducts

The SR 0065 viaducts, which are owned and operated by the Pennsylvania Department of Transportation (PennDOT), are a key link between northwest Pittsburgh communities, downtown Pittsburgh and various southern destinations. The viaducts were constructed in the late 1960s and typify the welded steel construction techniques of the era. These unique viaducts, which carry SR 0065 northbound and southbound across the north shore to the Fort Duquesne Bridge, extend a total length of 1,500 feet and reach a maximum height of 45 feet. The two adjacent structures (northbound and southbound) are a series of curved three-span continuous structures consisting of welded steel box girders rigidly framed into steel support bents with welded connections. The northbound (NB) viaduct features a single roadway level carried by a single three-webbed box girder and supported by fixed single-column steel bents. The adjacent southbound (SB) viaduct features two stacked roadway levels supported by pinned two-column steel bents. The lower SB level is carried by two box girders while the upper level (Ramp U) cantilevers over the lower and is carried by a single box girder. Figure 3 shows the relative location and size of the viaducts as well as the cantilevered Ramp U. Each steel bent is supported by concrete pedestals and pile caps founded on bedrock with end-bearing pile foundations.

Problem Definition

Various LRT alignments were investigated, including those that allowed for an unobstructed passage of the tunnel under the viaducts. Such an alignment required an S-curve to weave around the viaduct foundations, which conflicted with the alignment of an anticipated aerial structure just west of the viaducts and the location of an underground station just east of the viaducts (see Figure 1 for the aerial structure and North Shore Station). Subsequently, the current

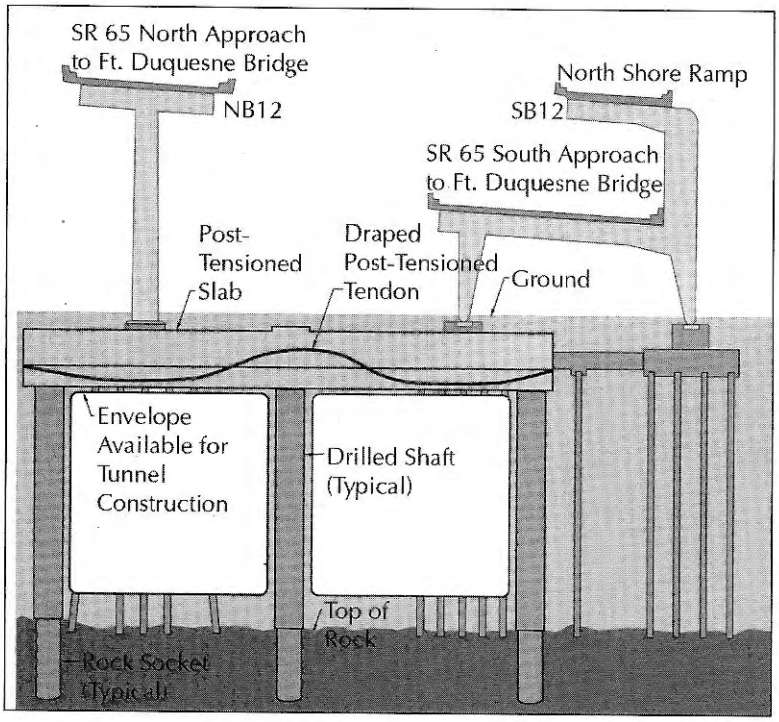

FIGURE 4. Underpinning of SR 0065 — schematic of the Bent 12 underpinning is shown.

LRT alignment crosses directly below one NB and two separate SB viaduct foundations (three total), requiring the removal of steel H-piles that extend about 40 feet to the bedrock below. Figure 4 illustrates the underpinning concept that allows for the tunnel construction.

The alignment skew relative to the viaducts dictated the need for the construction of two individual underpinning systems. The supports requiring underpinning were columns of Bents NB12 and SB12 (Bent 12 underpinning) and Bent SB13 (Bent 13 underpinning). Once completed, the Bent 12 underpinning will carry the pile caps of the bent NB12 and SB12 northeast (NE) column, and Bent 13 underpinning will support the pile cap of the SB13 southwest (SW) column. Figures 5 and 6 illustrate the relationship of the underpinned supports. Bents NB12 and SB12 are middle supports of the two adjacent structures between Bents 10 and 13S (carrying Spans 11 through 13), with a structure length between expansion joints of 318 feet (SR 0065 NB) and 303 feet (SR 0065 SB). Bent 13N is an expansion bent that is the end support of a second three-span continuous structure between Bent 13N and Bent 16 (carrying Spans 14

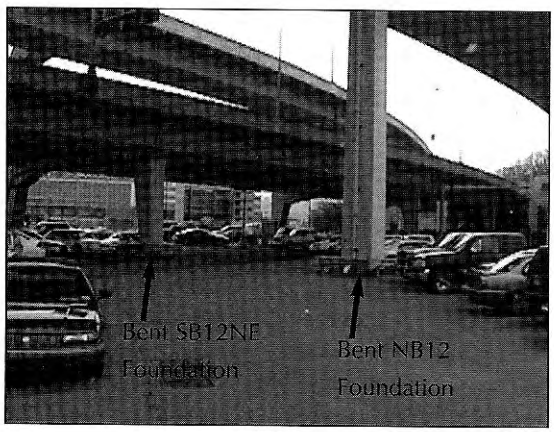

FIGURE 5. SR 0065 supports requiring underpinning — Bent 12 underpinning carries Bent SB12NE and NB12 foundations.

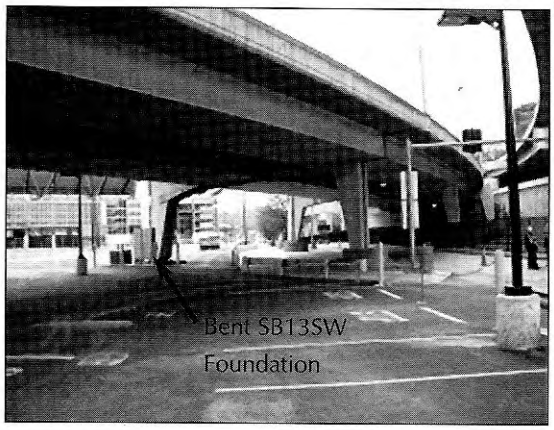

FIGURE 6. SR 0065 supports requiring underpinning – Bent 13 underpinning carries the Bent SB13SW foundation only.

through 16). Figure 3 shows Span 13 and the Bent 13 support location with Spans 14 through 16 in the background. The separate bents of the Bent 13 expansion joint location share a common footing and, therefore, the same pile foundations. Consequently, Bent 13 underpinning will carry the end supports of two successive three-span units of the SR 0065 viaducts.

Solution Evolution

Several alternative underpinning systems were investigated, including an above-grade steel transfer girder (see Figure 7), a sub-grade conventionally reinforced concrete transfer beam, a post-tensioned monolithic concrete slab (see Figure 8), a post-tensioned inverted concrete tub and a post-tensioned inverted concrete T-beam. Two primary project requirements drove the evolution of the solution:

- the underpinning systems must bridge the alignment completely and not bear any load on the tunnels below; and,
- the underpinning systems must control deflections of the SR 0065 viaduct foundations to less than 0.25 inch maximum vertical displacement during construction and 0.125 inch on completion of construction.

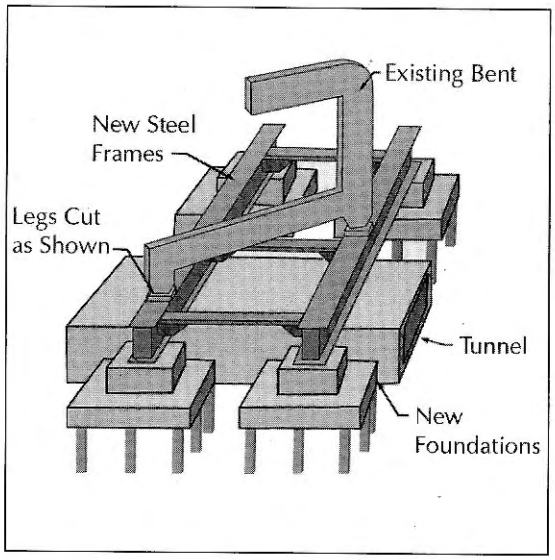

FIGURE 7. Steel transfer girder underpinning alternative.

The steel transfer girder and reinforced concrete transfer beam conceptually provided sufficient capacity to bridge the tunnel alignment while utilizing hydraulic jacks to unload

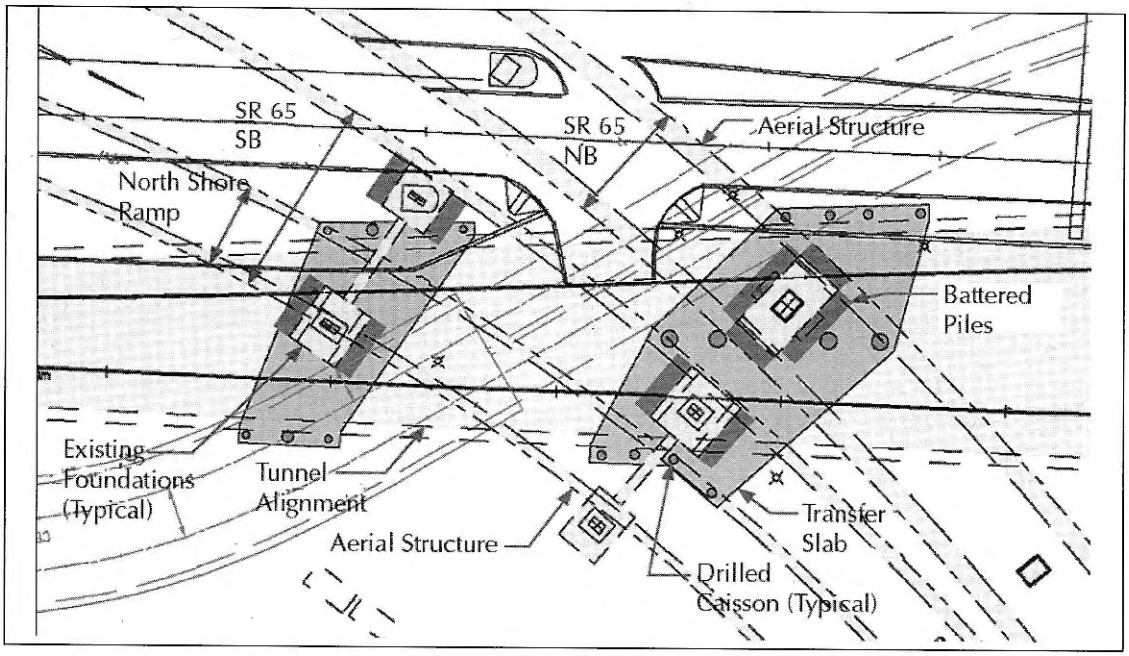

FIGURE 8. Monolithic concrete slab underpinning alternative.

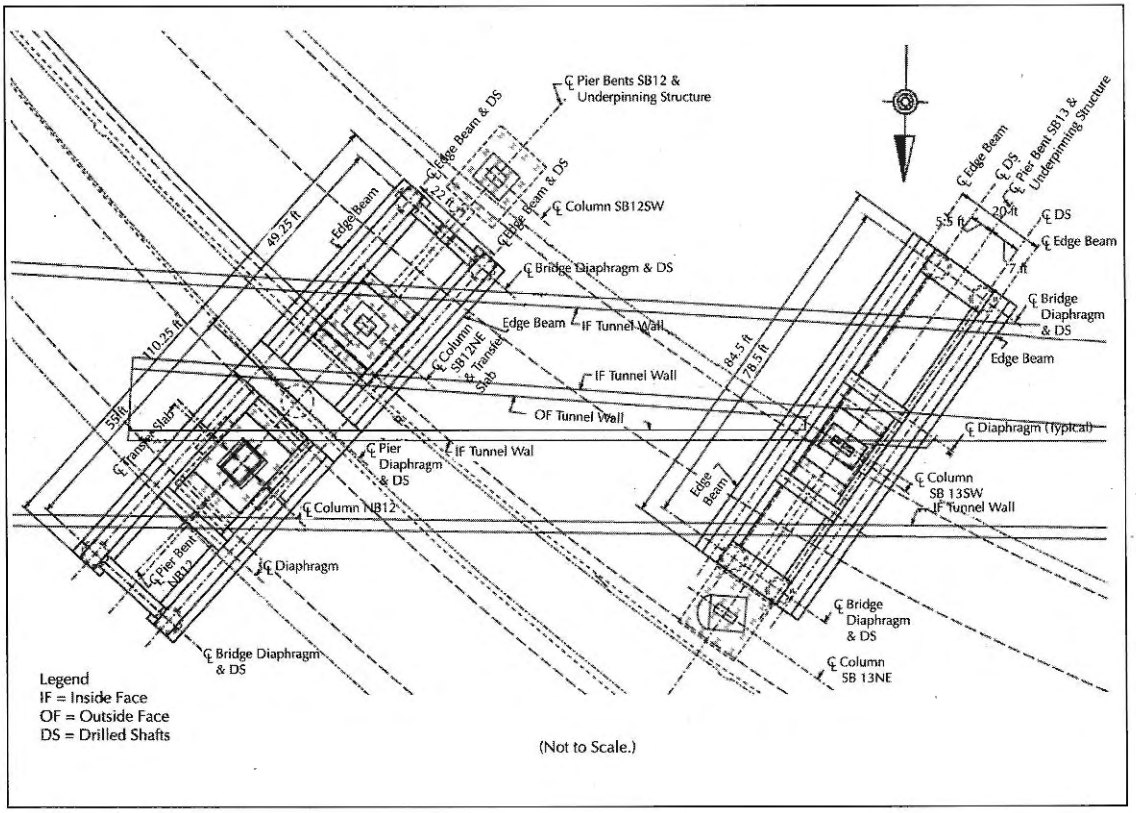

FIGURE 9. Plan view of the underpinning systems.

the foundation piles. These systems, however, were not conducive to providing cambers to counter successive deflections induced by staged load transfer (pile cutting). The steel girder alternative also presented potential corrosion and deterioration concerns. Post-tensioning was deemed a more reliable and compatible means of controlling load transfer and the associated deflections of the viaducts. The post-tensioned monolithic concrete slab consisted of two large post-tensioned slabs supported on a series of drilled shafts spanning the tunnels and supporting the viaduct foundations. The disadvantage of this system was the relative flexibility and complex geometry, which would make deflection control difficult.

The search for a durable alternative with large stiffness resulting in negligible deflections led to the development of a post-tensioned inverted concrete tub section. Conceptually, a 1-foot gap would be provided between the undersides of the viaduct pile caps and the tub section where hydraulic flat jacks would be placed. The envisioned purpose of the jacks was to maintain the pile caps stationary as the viaduct loads were being transferred from the piles to the tub section through the pile cutting process. This alternative provided a good means for controlling deflections and for the transfer of vertical loads; however, its weaknesses became apparent as the lateral loads exerted on the viaduct pile caps were considered. The flat jacks were unable to transfer these loads and temporary struts were needed to transfer footing lateral loads to the underpinning structure as vertical and battered piles were cut, which further complicated this alternative.

Subsequent modifications to the post-tensioned inverted concrete tub section led to the preferred solution. First, the 1-foot gap and the flat jacks were eliminated in favor of direct contact between the tub section and the pile caps in order to allow for the direct transfer of the vertical and lateral loads. Second, it was decided to offset the deflections at each stage

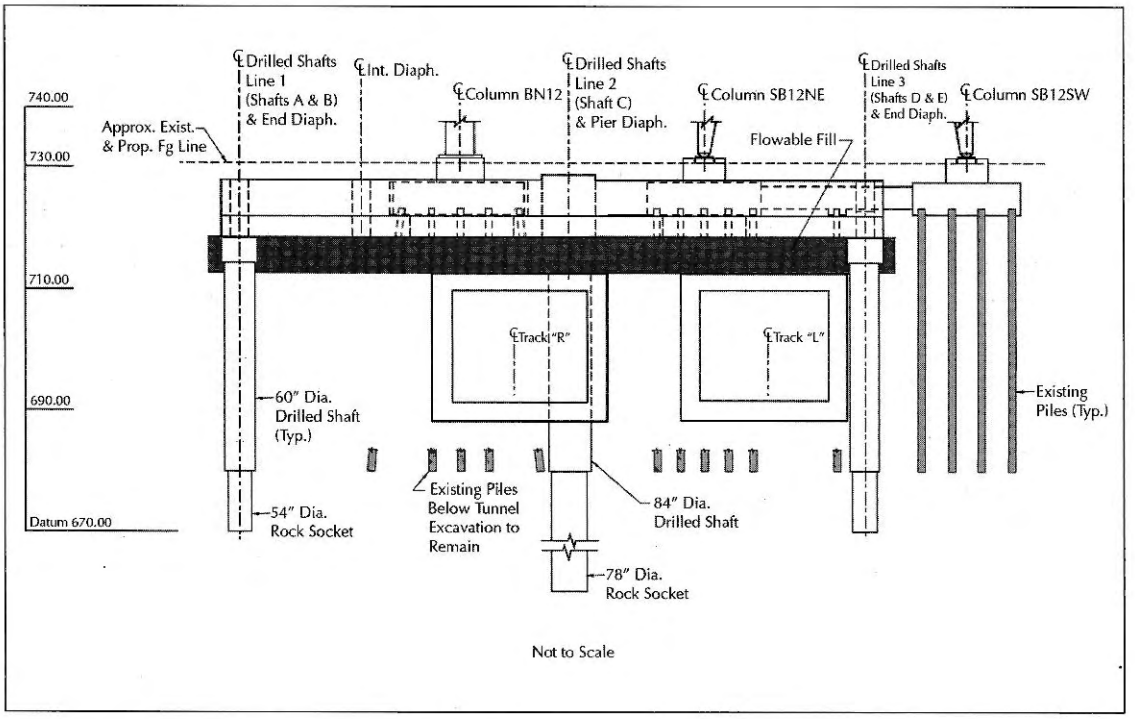

FIGURE 10. Bent 12 underpinning elevation.

of the load transfer using sequential post-tensioning. Third, areas of the tub section not in contact with the pile caps were eliminated in order to save on construction material and to reduce backfill surcharge loads. In effect, the tub section was transformed into two edge beams with slabs (transfer slabs) spanning between the edge beams to support the viaduct pile caps; the drilled shaft foundation concept remained unchanged. This alternative was found to satisfy project requirements of constructability, durability, redundancy and viaduct displacement limitations.

Underpinning System Overview

The proposed underpinning structures were similar and each was composed of concrete transfer slabs, inverted T-beams (edge beams), diaphragms and drilled shafts. Figure 9 shows the underpinning components and the relationship between the tunnel alignment and the viaduct foundations. The transfer slabs were located under the existing pile caps in order to carry the loads upon the removal of the piles. The loads were then transferred from the transfer slabs to the edge beams and from there to the drilled shafts. Diaphragms were provided to control axial rotation and lateral displacement of the edge beams and to distribute lateral loads between the edge beams. The end diaphragms of the Bent 13 underpinning also contributed to the transfer of loads to the drilled shafts while the middle pier diaphragm of the Bent 12 underpinning transferred the loads to the middle drilled shaft support.

Bent 12 underpinning (see Figure 10) was a two-span continuous structure where the edge beams spanned between three rows of drilled shafts. The end supports consisted of two 5-foot-diameter shafts, while the middle support was a 7-foot-diameter mono-shaft located between the left and right track single-cell tunnel box structure below. The Bent 12 underpinning had a total length of 104.25 feet, spanning 55 and 49.25 feet between supports. The underpinning connections were pinned at the two ends and were monolithic (moment connection) at the center support. Bent 13 underpinning (see Figure 11) was a 78.5-foot single-span structure supported on

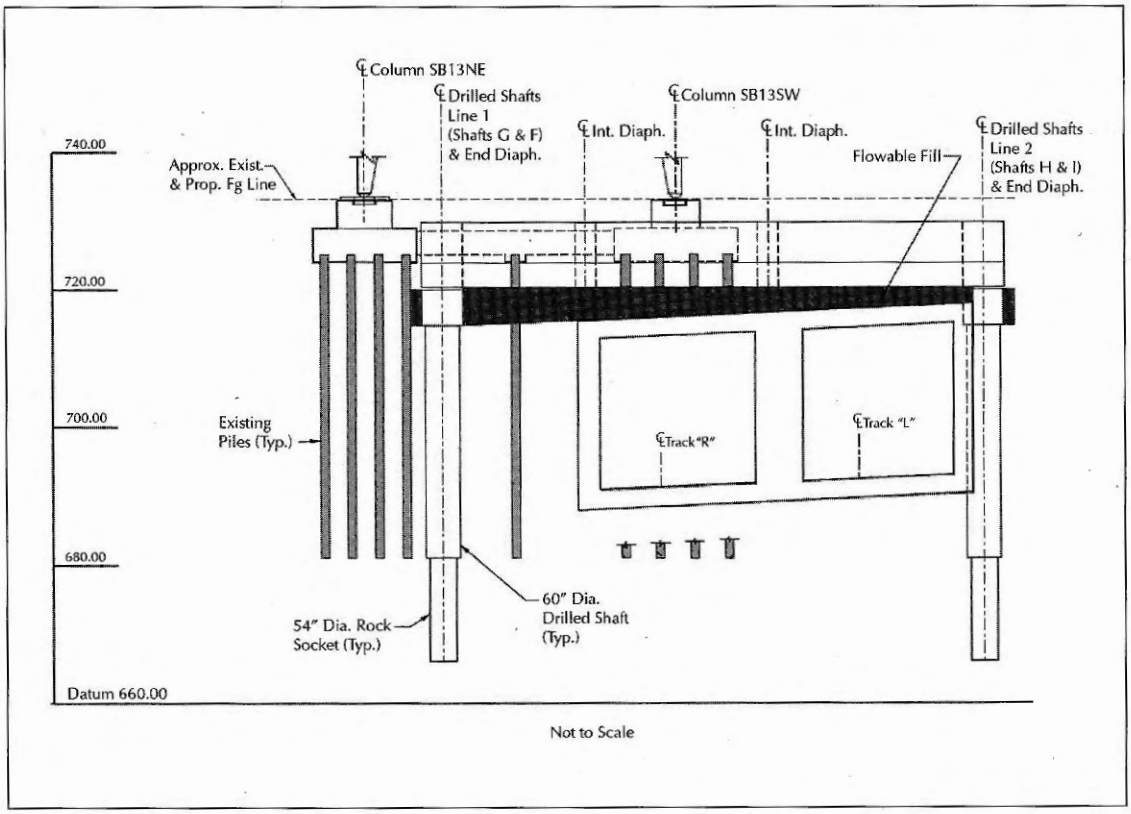

FIGURE 11. Bent 13 underpinning elevation.

two rows of two 5-foot-diameter shafts. The connections were pinned at the two ends. The two underpinning structures had a total of nine drilled shafts, all of which penetrated into bedrock. The 7-foot-diameter shaft was exposed during the excavation and construction of the tunnels and was therefore designed as an unbraced column. The 5-foot-diameter shafts were outside the support of excavation (SOE) and were placed and utilized with limited exposure. The drilled shaft design included the impact of the adjacent SOE lateral movements.

Viaduct Load Development

The viaduct load development commenced by establishing the viaduct structure geometry and the extent of the underpinning impacts on the viaduct structures. A software package was utilized to develop the complex curved geometry based on the available existing structure plans and field observations. The geometry information was then used to develop detailed three-dimensional models of the viaducts.

The viaduct loads and section properties were developed in accordance with the AASHTO *Standard Specifications* (16th edition) and supplemented by PennDOT's *Structures Design Manual (DM-4)*. Structural analysis and design software was employed to generate the foundation loads based on the applicable viaduct loading conditions, including:

- Dead loads;
- Live loads (HS25 and PennDOT's 204 kip permit load [P-82]);
- Thermal forces;
- Wind loads (including wind on live load);
- Braking forces; and,
- Centrifugal forces.

The dead loads of the SR 0065 viaducts were calculated in the structural analysis and design program to the bottom of the columns and combined with the base plate, concrete

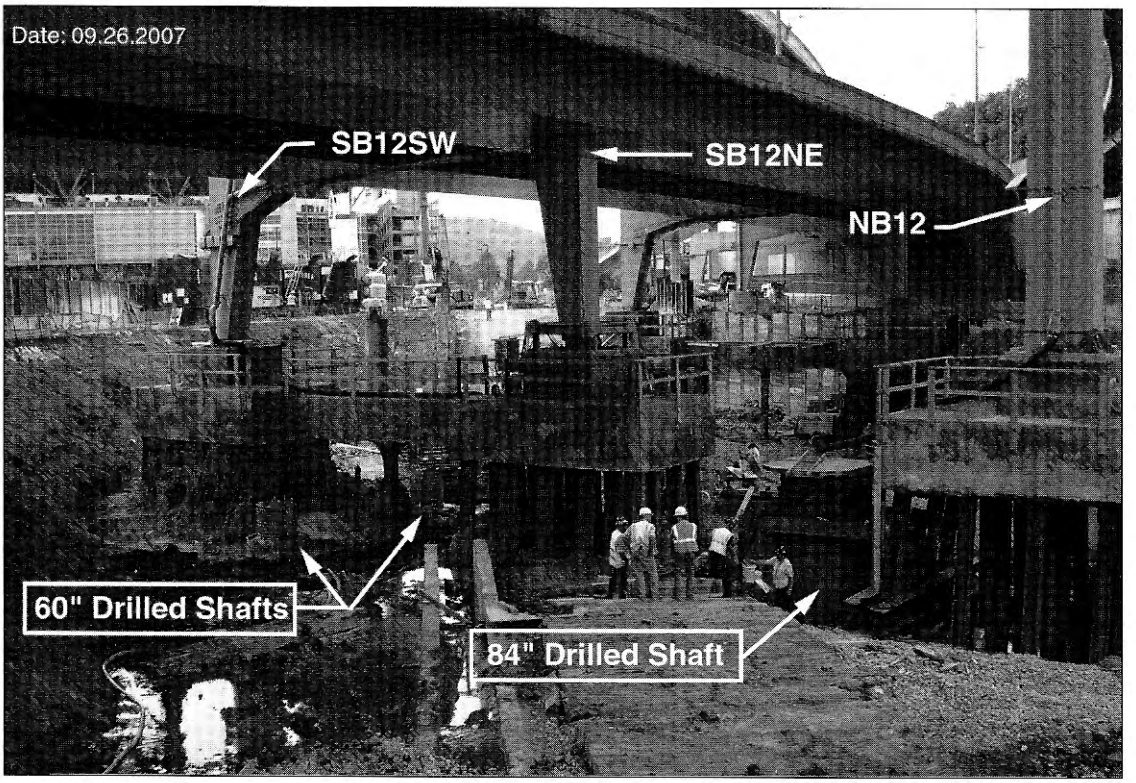

FIGURE 12. Bent 12 underpinning with foundation piles exposed.

pedestal and pile cap loads for application on the underpinning system. Live load influence lines were generated as well in the structural analysis and design program, from which the maximum and minimum live load reactions for each underpinned foundation were determined. Load force effects were determined in all three orthogonal directions and transformed to the plane of the underpinning structures for design application.

Underpinning System Design & Analysis Approach

Construction Sequence. The construction sequences of the two underpinning systems were an integral part of their design and were devised to ensure the safe transfer of the loads to the new foundations and to meet the displacements limits of 0.25 inch during construction and 0.125 inch in the final (permanent) configuration. The general construction sequence for both underpinning structures was similar. The sequence for Bent 12 underpinning, which was somewhat more complex than the Bent 13 underpinning due to the two-span configuration, started with local excavation, which was required to increase the headroom below the viaducts in order to build the drilled shafts and to verify the as-built bottom of pile cap elevations prior to the construction of the drilled shafts. The initial excavation depth was 4.5 feet below the bottom of the pile caps.

With the drilled shafts in place, the pier diaphragm was constructed at the top of the 7-foot-diameter mono-shaft followed by construction of the edge beams (see Figures 10 and 12). Placement of the intermediate and end diaphragms was the final step in the underpinning system assembly. At this stage, excavation proceeded under the edge beams. The intent was to ensure the edge beam's deflection due to self-weight that would occur prior to the load transfer (pile cutting) and prior to jacking the post-tensioning tendons. The excavation then proceeded deeper under the viaducts' pile caps, exposing the underside of the pile caps and the top 7.75 feet of the

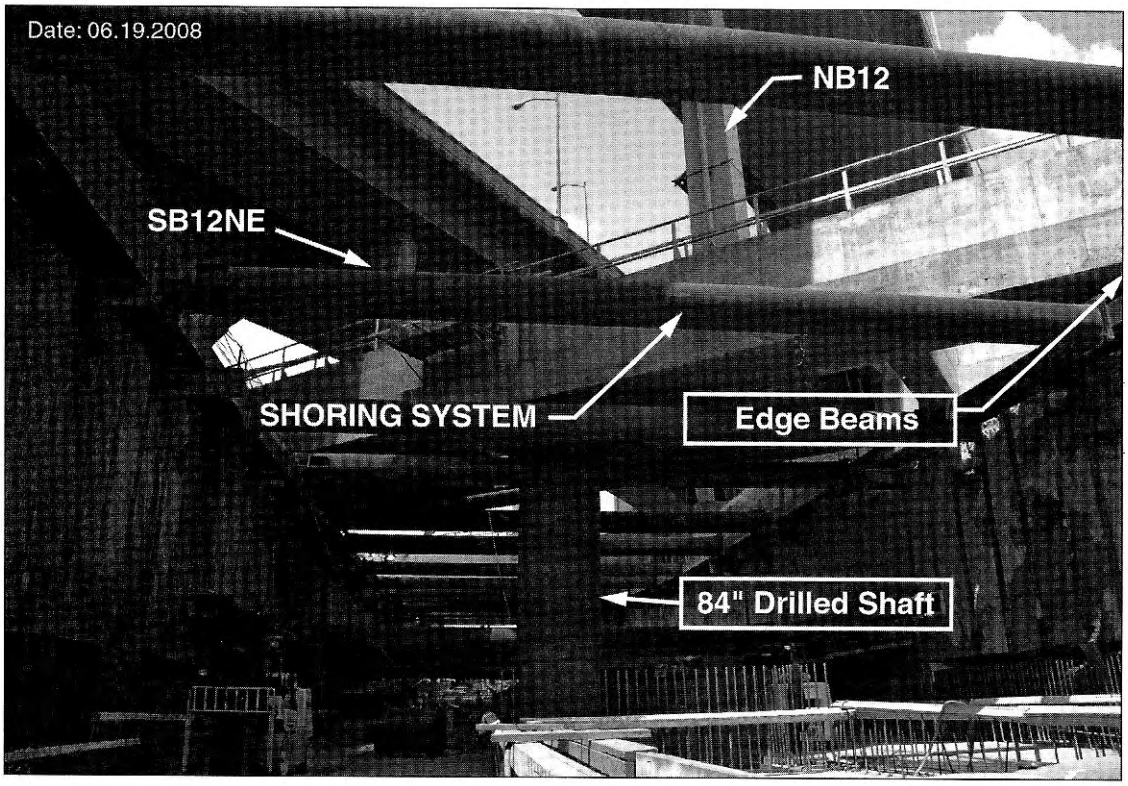

FIGURE 13. Bent 12 underpinning in place and foundation piles removed.

piles. The underside of the pile caps and the top 4 feet of the piles, which were to remain in place and encased in the transfer-slabs, were cleaned by sand blasting. Transfer slabs were constructed so that a 3-inch gap was provided between the top of the transfer slabs and the bottom of pile caps. This gap was subsequently filled with a non-shrink grout to provide full contact and support.

The post-tensioning and pile cutting procedures commenced only after the aforementioned 3-inch gap was grouted and cured so that the axial and lateral pile loads could be transferred to the transfer slabs. Two of the six tendons were jacked in the edge beams, relieving a portion of compression in the piles. Doing so allowed for cutting approximately a third of the piles. The post-tensioning and pile cutting process was repeated two more times until all tendons were jacked, all piles were cut and the loads were fully transferred to the underpinning structures. Throughout this process, displacements at the base plate of the viaduct columns were measured and monitored. The piles were removed in two stages. Segments of piles that were exposed directly below the transfer slab were removed (approximately 3.75 feet of pile) after the load transfer was complete. The remaining parts were removed during tunnel excavation, facilitating cut-and-cover construction and the advancement of the LRT tunnel. On completion of tunnel construction, a flowable backfill was placed under the edge beams extending from the top of tunnels to the bottom of edge beams. The flowable backfill was used as a backfill expedient and not as a structural element. Flowable backfill was also placed between the pile caps and the edge beams. The final stage of construction was the placement of structural backfill to the finished grade and the placement of pavement at the surface. Figure 13 shows the completed Bent 13 underpinning in service with piles removed and tunnel construction in progress.

Design & Analysis of Transfer Slabs & Edge Beams. The design approach for transfer slabs and edge beams was to develop suitably sized

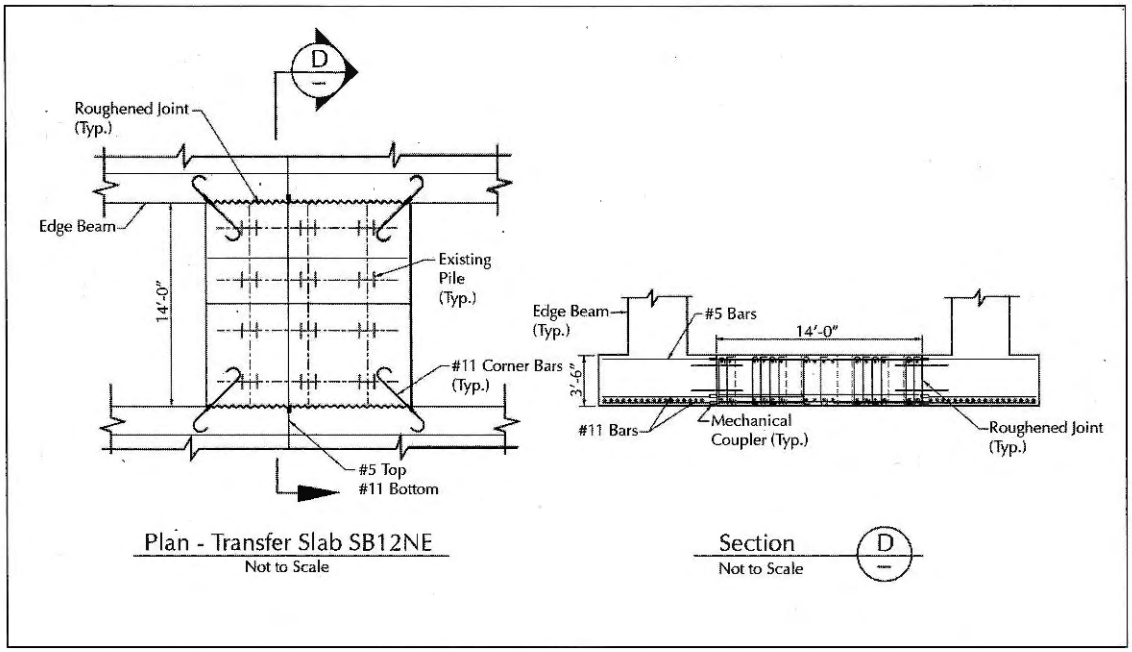

FIGURE 14. Typical underpinning transfer slab supporting the existing SR 0065 viaduct foundation.

and constructible components based on AASHTO and PennDOT loading and design criteria and to control displacements at all stages of construction. The load factor method was used for the design of the transfer slabs and the diaphragms. The post-tensioned components of the edge beams and the intermediate diaphragms were designed using service loads and checked against factored loads. The concrete strength ($f'c$) was 6,000 pounds per square inch for the edge beams and transfer slabs.

The 3.5-foot deep transfer slabs (see Figure 14) were slightly narrower than the pile cap they support and were reinforced concrete structures that transfer loads to the edge beams through shear friction. Sufficient reinforcement was provided to take flexure at the top and bottom of slabs and to transfer shear to the edge beams. A three-dimensional structural anaylsis and design model was used to analyze the transfer slabs by applying a finite element analysis method.

All the diaphragms, with the exception of the Bent 12 underpinning pier diaphragm, were reinforced concrete structures. The pier diaphragm at Bent 12 was post-tensioned.

The edge beams were post-tensioned using six tendons consisting of twelve 0.6-inch-diameter tendons with two contingency ducts provided. The contingency ducts were grouted at the end of construction regardless of use. Figure 15 illustrates the tendon profile across the asymmetric spans of the Bent 12 underpinning. The tendons were jacked in three stages. At each stage, prior to pile cutting operations, two tendons were jacked in the edge beams. The force and profile of the tendons were designed to not only provide the required camber for displacement control but also to ensure that the edge beams remained in compression during all stages of construction. No tension was permitted in the edge beams under either of the following two load combinations:

- prestressing force and permanent dead load; or,
- prestressing force, permanent loads, and live load.

The analysis and design of the edge beams was carried out using a program that was capable of performing a step-by-step time-

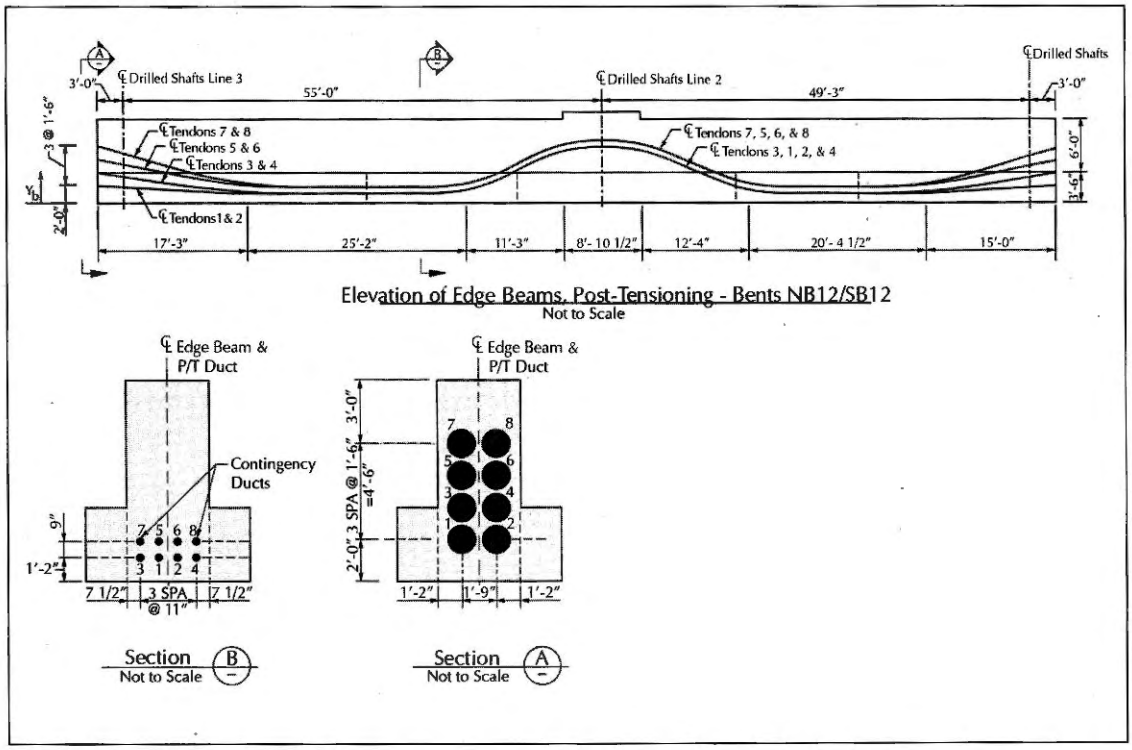

FIGURE 15. Edge-beam tendon profile.

dependent analysis. Time-dependent induced forces such as creep, shrinkage and post-tensioning losses were accounted for at each stage of construction. Creep and shrinkage-induced strains, variation of modulus of elasticity and concrete strength with time were modeled using 1991 CEB-FIP model code, which is widely used in the industry for the design of post-tensioned segmental concrete bridges. Forces, stresses and displacements were calculated at each stage of construction as well as at completion and thirty years after the completion of construction. In addition to time-dependent analysis, a corresponding structural analysis and design model was developed to analyze the transfer slabs, to obtain drilled shaft loads and to adjust the time-dependent model for the three-dimensional effect where required. The flexibility of the drilled shafts was considered and modeled in both the time-dependent and three-dimensional structural analysis and design models.

Prediction of Displacements. Since the SR 0065 viaducts consisted of steel box girders that were framed into steel bents with welded connections, these rigid connections were susceptible to foundation settlements that could induce significant stresses into the superstructure. Therefore, displacement control at each stage of construction and at the completion of construction was imperative. Displacements were monitored at the base plate of the three affected columns. During construction, the net vertical displacements were limited to no more than 0.25 inch and on completion of construction to no more than 0.125 inch.

The displacement calculations were carried out using structural analysis and design as well as time-dependent analyses for the transfer slabs and the edge beams, respectively, with hand calculations for the short- and long-term drilled shaft axial shortenings. The results had to be adjusted carefully for the construction staging effects. Displacements of the underpinning structures occurring prior to load transfer (prior to grouting of the 3-inch gap between the transfer slabs and the pile caps, and post-tensioning) were not relevant

and were taken out from the net displacement. These displacements included instantaneous displacement due to the self weight of the edge beams and diaphragms, the associated elastic shortening of the drilled shafts and rock settlement. Relevant downward displacements were due to the self weight of the viaducts, viaduct live loads, the backfill and surcharge. The displacements components were:

- Edge beam instantaneous and long-term deformations due to creep and shrinkage;
- Transfer slab deformation;
- Drilled shaft axial shortening; and,
- Rock settlement.

All these components were considered in the calculations. The post-tensioning forces resulted in camber, offsetting the downward displacements. The post-tensioning was designed to ensure that the net displacements were kept within the allowable displacement limits at all times.

The calculations showed that the maximum live load deflection at the base plate of the viaducts under the governing live load (PennDOT P-82 permit loading) was 0.0625 inch. The viaducts were to be open to traffic during construction when this maximum displacement occurred. On completion of construction, no live load deflection was expected due to the placement of flowable backfill below the edge beams.

Drilled Shafts. A total of nine drilled shafts supported the two underpinning structures: five for Bent 12 underpinning and four for Bent 13 underpinning. These shafts wre designed to transfer loads to bedrock through side friction only. There were four 60-inch-diameter drilled shafts and one 84-inch-diameter drilled shaft supporting the two-span continuous Bent 12 underpinning. The 84-inch drilled shaft extended 36 feet into rock, with a 78-inch diameter rock socket. The single-span Bent 13 underpinning was supported by four 60-inch-diameter drilled shafts. The concrete strength ($f'c$) of the drilled shafts was 5,000 pounds per square inch.

Two primary types of forces were considered in calculating the displacement at the top of the drilled shafts and applied in design. Type 1 forces include thermal, post-tensioning, creep and shrinkage of the underpinning system. Type 2 forces are those external to the underpinning system including the viaduct loads. Where applicable, forces induced by movement of the adjacent SOE system were also estimated and included in the design of the drilled shafts. Design of the drilled shafts and calculation of lateral displacements required the use of a program for the analysis of the deflection and capacity of piles under lateral loads (COM624P) to verify the underpinning system stability, and to check displacements at the top of shafts using Type 2 service (un-factored) forces. The lateral forces at the top of the drilled shafts were combined using the square-root-of-the-sum-of-the-squares (SRSS) method.

COM624P analysis was used to perform a serviceability check in the drilled shafts by applying the combined Type 1 and Type 2 service forces. Concrete design software (pcaColumn) and the results from COM624P were used to design the drilled shafts and rock sockets. In using pcaColumn, moment magnification of the drilled shafts was considered by assuming the column length to be four times the diameter of the shaft. Using this parameter ensured a length higher than the point of fixity of the drilled shafts, which was in the range of 2.5 to three times the diameter. The boundary conditions used in the pcaColumn analyses were free at the top and fixed at the bottom for the 5-foot-diameter shafts, and restrained at top and fixed at bottom for the 7-foot-diameter shaft.

pcaColumn again was used to design the 7-foot-diameter shaft as an unbraced column, since the shaft was exposed during construction and would behave like a column until it was backfilled. Factored loads obtained from a three-dimensional structural analysis and design model were used for this analysis. The boundary condition used in the pcaColumn analysis was restrained at the top and fixed at the bottom with the true column length applied. The 7-foot-diameter drilled shaft was integrally connected to the pier diaphragm. The point of application of loads to this shaft was located at the centroid of the pier

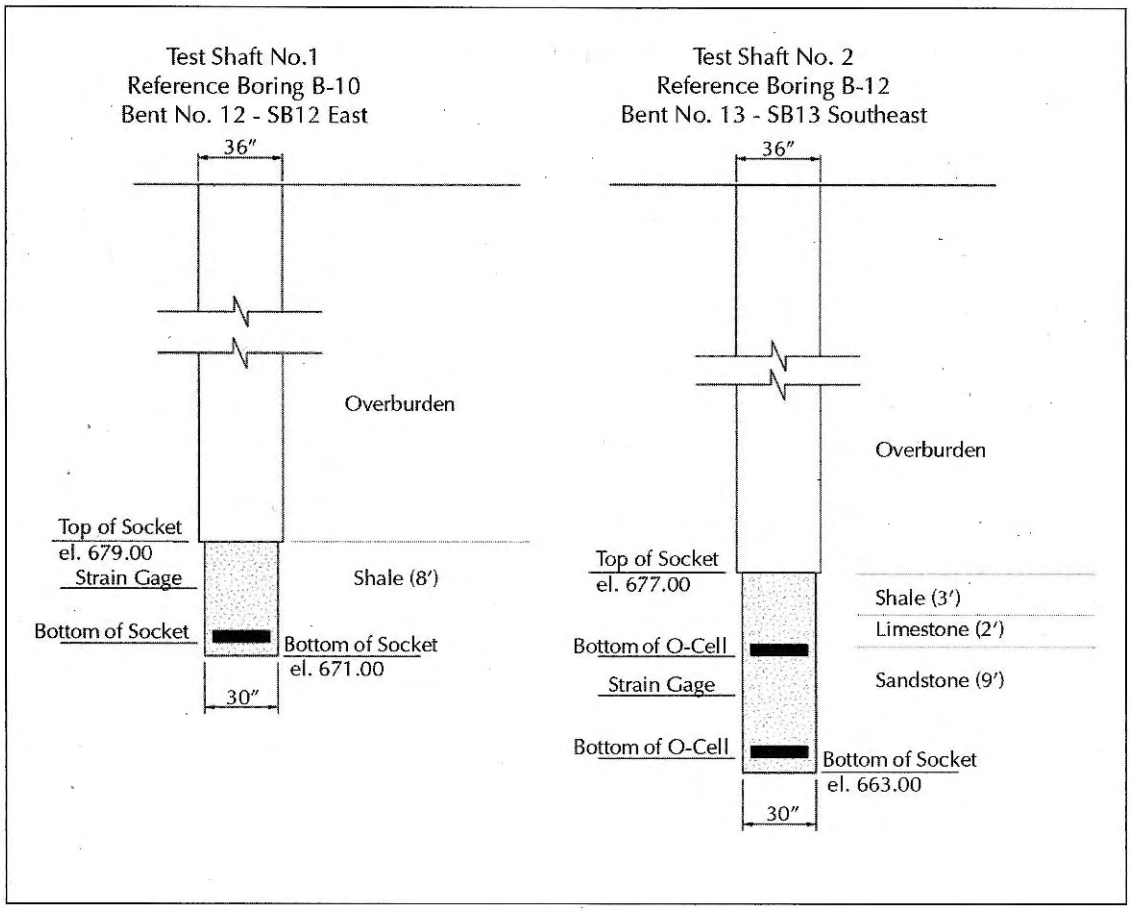

FIGURE 16. Drilled shaft load test schematic.

diaphragm (4 feet above the top of shaft); therefore, the height of the 7-foot-diameter shaft was modeled to this point. The 10- by 14- by 10-foot high pier diaphragm restraining the top of the drilled shaft was modeled in COM624P applying a 4-foot-deep, 10-foot-diameter shaft in determining displacements for the 7-foot-diameter shaft.

Drilled Shaft Load Test. The bedrock in the vicinity of the underpinning consists of layers of shale and sandstone. To verify the bedrock design parameters, two 3-foot-diameter drilled test shafts with 30-inch-diameter rock sockets were used in the proximity of the proposed production shafts and were subjected to the Osterberg Cell (O-Cell) load test. The O-Cell load test was done prior to the construction of the production drilled shafts. The primary goal of the load test was to verify drilled shaft settlement values, rock socket side friction values and end bearing response. The integrity of the production shaft concrete was tested using the cross sonic log (CSL) method. A total of eight CSL tubes were provided: four within the drilled shafts, with another four extending to the rock sockets.

The O-Cell is a sacrificial jack-like-device that can be installed within the drilled shaft, or in this case, within the rock socket as shown in Figure 16. Strain gages were proposed to help determine the contribution of load response from the overburden soils, sections of the rock socket and the end bearing component. The first test shaft was constructed with one O-Cell and one level of strain gage within 8 feet of rock socket. The second test shaft consisted of two O-Cells and one level of strain gages within 14 feet of rock socket. Each of these cells had a 1,250 ton capacity and a diameter of 21 inches. The O-Cell load test was to com-

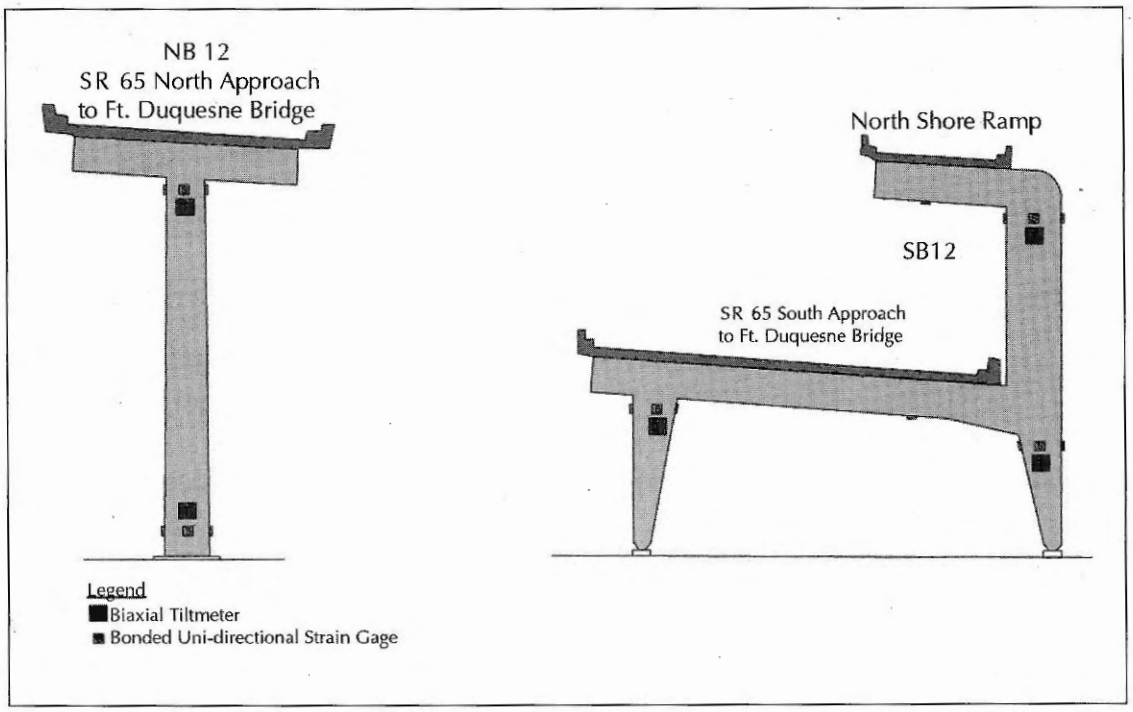

FIGURE 17. General strain gage and tiltmeter locations.

mence after concrete strength of 5,000 pounds per square inch was achieved.

Instrumentation & Monitoring. An instrumentation and monitoring program specific to underpinning operations was proposed to monitor displacements of the underpinning structures and SR 0065 viaducts as well as strain behavior of the viaducts.

The locations on the underpinning structures identified for monitoring included points on top of the edge beams located immediately above the center line of the drilled shaft. At these points, linear variable differential transformers (LVDTs) were installed to monitor and verify rock settlements and drilled shaft axial shortening at specific stages of construction as load on the drilled shafts was increased. This information was used to adjust the post-tensioning force, if needed, to offset the differences between the anticipated and actual rock settlement values. Additional critical locations included the base plates of underpinned columns NB12, SB12NE and SB13SW. Three LVDTs were called for at each location (two separate locations on NB12 and one location on each SB base plate) to monitor potential movements in the orthogonal directions, providing a total of twelve LVDTs for displacement monitoring. A total of eighty-four locations were identified on the bents and superstructure of the viaducts for strain gages in order to monitor potential impacts to the strain behavior of the steel structures. An additional fifty-four tiltmeters were provided throughout Spans 11 through 16 to monitor viaduct movements. The tiltmeters and strains gages were generally located at the top of columns for Bents 11, 12, 13, 14 and 15, with additional strain gages provided at several points on the superstructure and bent cross beams. Figure 17 illustrates general locations of the strain gages and tiltmeters at a typical bent location.

The contract provisions required that the contractor limit the stresses in the structures and the associated deflections during excavation, construction of the new underpinning systems and transfer of the load to the underpinning systems. The movement and stress limits are shown in Table 1.

The contractor was also required to employ independent firms to monitor the stresses in the superstructure during construction and to

TABLE 1.
Project Specification Threshold & Limiting Values

Instrument	Threshold Value*	Limiting Value*
Tiltmeter	0.15°	0.20°
Bonded Strain Gages	±50 microstrain	±100 microstrain
LVDTs	±0.25-inch (uniform vertical displacement)	±0.50-inch (uniform vertical displacement)
	0.25-inch (differential displacement between bent legs)	0.50-inch (differential displacement between bent legs)

Note: *Threshold values and limiting values represent response values for duration of construction.

inspect the viaducts both before and after construction. The special provisions further required that the instrumentation be installed and monitored for thirty days prior to any construction activities in order to establish a set of baseline readings for the viaducts. These data showed the actual stresses recognized by the structure and were used as a check on the stress limits contained in the contract special provisions. The stresses and displacements in the SR 0065 support bents were periodically monitored during construction and continuously monitored during the load transfer. Project specifications dictated the frequency that data must be reported and also specified "threshold" and "limiting values" beyond which action would be required by the contractor to ensure worker safety and to preserve the integrity of the viaducts.

Durability & Redundancy. Measures had been incorporated in the underpinning structures to ensure their durability. These measures included but were not limited to:

- use of epoxy coated reinforcing bars, incorporation of the latest research and technology in grouting post-tensioning tendons;
- provision of 3-inch minimum cover for concrete;
- provision of a water proof membrane at the construction joints, edge beams, and transfer slab; and,
- crack control by maintaining concrete in compression at all times.

The expected useful service life for the underpinning structures is one hundred years, which is based on the fact that by incorporating all the recommended industry measures for post-tensioned structures to accomplish durability, it should attain the targeted life expectancy for post-tensioned bridges. Therefore, it is expected that the underpinning structures will have a much longer useful life than the remaining life of the existing forty-year-old SR 0065 viaducts.

Redundancy was provided through primary load path (post-tensioning) and secondary load path (mild steel reinforcing).

Post tensioning provided the camber required for deflection control and carried the service and ultimate loads imposed on the underpinning by the SR 0065 viaduct structure. The industry standard of care for the post-tensioned structures was maintained by providing multi-tendons in lieu of fewer larger tendons, so that corrosion of one tendon would not be detrimental to the entire structure. The post-tensioning tendons were fully grouted.

Retrofit of SR 0065 Viaducts. As part of the current NSC project, and due to the anticipated permanent underpinning displacements, select box girders of the northbound and southbound viaducts were retrofitted by providing internal transverse bottom flange stiffeners. The channel stiffeners (shown in Figure 18) were installed throughout the negative moment regions about pier Bent SB11 and NB11 to brace the bottom flange in compres-

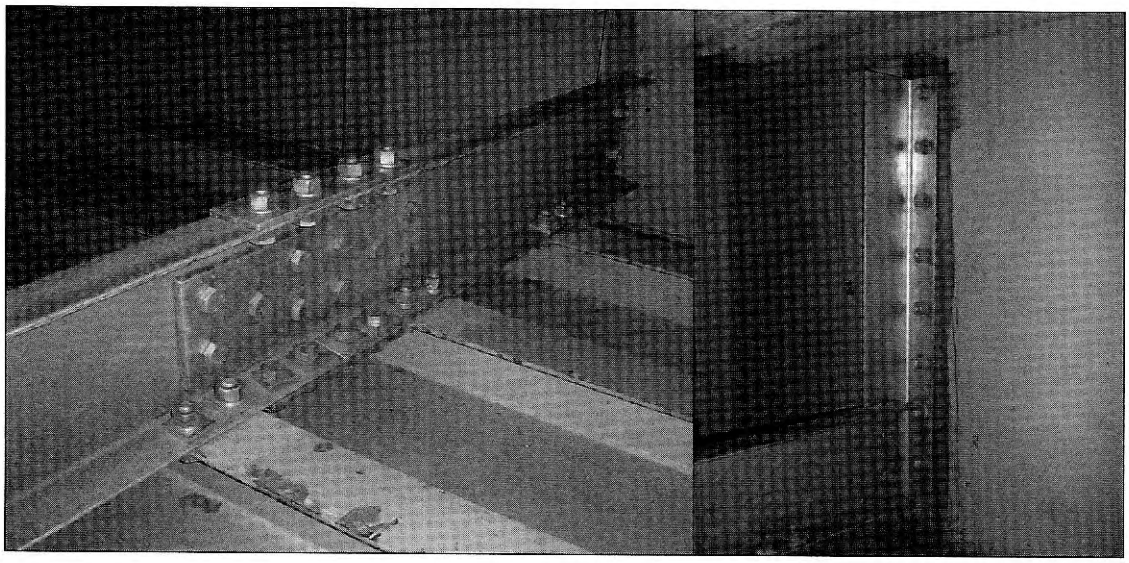

FIGURE 18. SR 0065 viaduct box girder retrofit.

sion and ultimately offset potential displacement effects on the overall live load capacity of the viaducts. The need for, and the extent of, the viaduct retrofits were determined as part of a live load capacity assessment, which demonstrated potential adverse impacts to the capacity of the adjacent negative moment regions about Bent 11 due to anticipated heave at the Bent 12 underpinning location.

The channel sections were field bolted to the existing web and bottom flange stiffeners of the box girders; and in the case of the northbound three-webbed box girder, a connection angle was also installed on the middle web to allow for the connection of the channel stiffener.

Conclusion

As the need for construction of tunnels and major utilities in congested urban areas increases, there will be more instances of conflict between these structures and existing bridge foundations. The underpinning of these foundations would be particularly challenging where bridge superstructure and piers are connected monolithically or where straddle bents are use. To protect these bridges against cracking, differential displacement or tilting, owners of these bridges would demand stringent deflection tolerances as well as durability provisions. The method developed for the NSC project provides a practical way for underpinning bridges particularly where virtually zero displacement is desired.

For this project, the two underpinning structures were constructed successfully and the actual measured displacements were well within the tight tolerances during and after completion of the construction. The durability measures implemented for this project will ensure that the underpinning structures outlast the bridges they support.

NOTES — *Cogo-PC Plus geometry software was utilized to develop the complex curved geometry based on the available existing structure plans and field observations. The geometry information was then used to develop detailed three-dimensional STAAD models of the viaducts. The analysis and design of the edge-beams was carried out using the program BDII by Interactive Design Systems.*

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