TDOT I-240 MemFix4 – CMGC – Memphis, TN

Jake Williams, PE, Alfred Benesch & Company, 615-370-6079, jwilliams@benesch.com
Chris Kelley, PE, Alfred Benesch & Company, 615-370-6079, ckelley@benesch.com
Lia Obaid, PE, Tennessee Department of Transportation, 615-532-7522, lia.obaid@tn.gov
David Paris, PE, Kiewit Infrastructure South Co., 501-399-5269, david.paris@kiewit.com

INTRODUCTION

Project Purpose and Need

Interstate 240 (I-240) forms a loop around the southern boundary of Memphis, Tennessee and serves as a critical east-west connection for travelers and commerce at the convergence of three interstates and crossings of the Mississippi River. Dubbed MemFix4, this project area at the Poplar Avenue interchange has seen its fair share of past projects, with the most recent occurring between 2010 and 2015. This specific past project attempted to widen I-240 to an eight-lane mainline and improve traffic flow through the interchange. However, widening was not fully completed due to hidden conditions revealed during construction. All bridges in the interchange were found to have short driven piles at their abutments and were supported only by spread footings at each pier. When this was discovered during the previous widening project, work halted in areas where there was concern of destabilizing the existing structures.

The Memphis area resides in the influence zone of the New Madrid Fault, which in 1811 and 1812 produced four of the most the powerful earthquakes east of the Rocky Mountains in recorded history. The discovery of insufficient foundations supporting these structures made addressing this issue a top priority. The Tennessee Department of Transportation (TDOT) realized that the next project needed to replace the structures while minimizing impacts to the thousands of vehicular travelers through this interchange and the nearly 20 trains per day on the Norfolk Southern Railroad (NSR) I-240 overpass.

The MemFix4 project replaced the two Poplar Ave interchange bridges, the NSR double track overpass and retrofit a third highway bridge over I-240. The eastbound and westbound Poplar Avenue structures were replaced with two-span steel beam structures using weekend closures of I-240 and modular bridge units. The NSR bridge was replaced utilizing a temporary shoo fly alignment and an innovative bridge slide. The Park Avenue bridge was not replaced but underwent a significant seismic upgrade with the pier foundations being retrofitted with micropiles.

TDOT elected to use the Construction Manager/General Contractor (CMGC) delivery method for the purpose of tackling complex challenges, handling delicate coordination issues, and to utilize innovative techniques such as ABC. In CM/GC, the owner, designer, and the Construction Manager (CM) all share an active role in the design of the project and form a project design task force. The goal of the task force is to tackle the challenges of a complex project by implementing innovations based on input from the CM, accelerating the project schedule, and deciding upon a mutually agreed distribution of risk.

The MemFix4 project was the second CMGC pilot project undertaken by TDOT. Besides coordination with NSR regarding the replacement of the I-240 overpass structure, numerous challenges had to be resolved, including maintaining rail operations, minimizing impacts to roadway travelers, implementing ABC techniques, solving utility conflicts, and meeting an aggressive construction schedule.

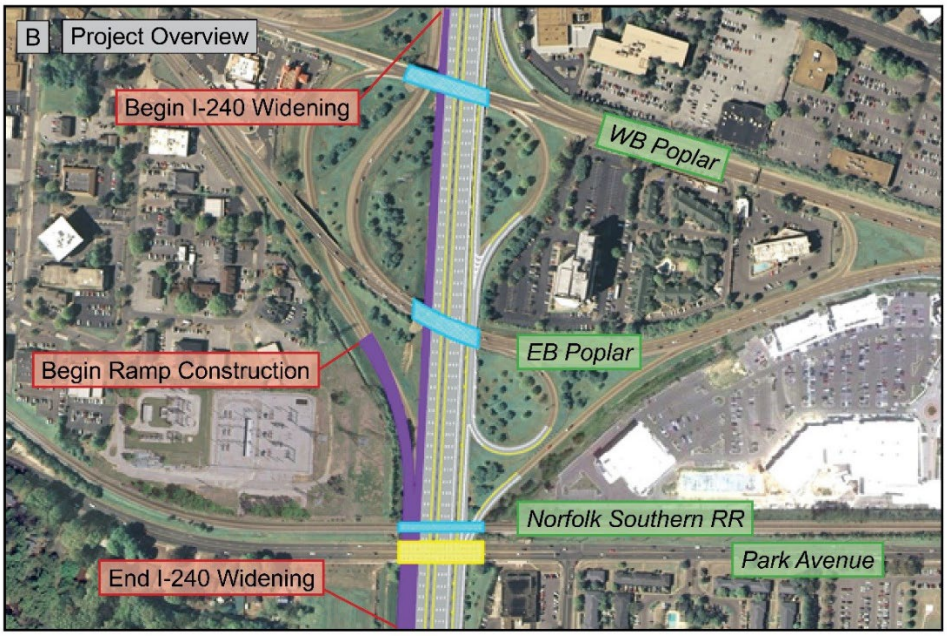

Figure 1 – The I-240 corridor surrounds Memphis, TN. The above shows the specific project limits.

SEISMIC ANALYSIS

A site-specific ground motion analysis was performed to develop a project-specific design acceleration response spectrum (see Figure 2). The project location falls in Seismic Design Category C with a site-specific 1-second period design spectral acceleration (S_{D1}) of .446g, and bridge structures were designed using design strategy Type 1: Ductile Substructures with Essentially Elastic Superstructure in accordance with the AASHTO Guide Specifications for LRFD Seismic Bridge Design (Guide Spec).

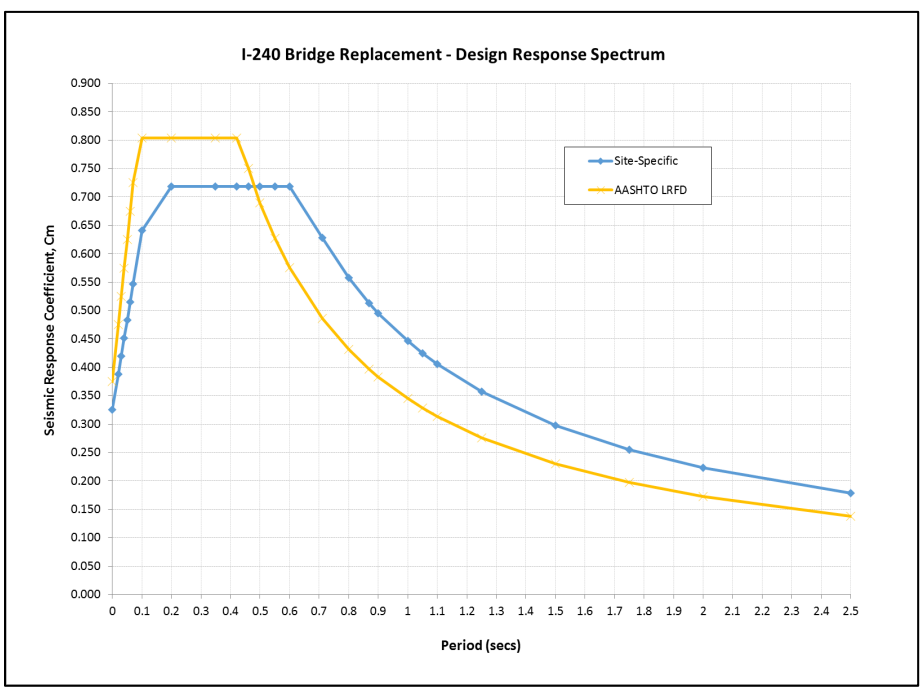

Figure 2 – Design Response Spectrum Comparison

Seismic analysis of the roadway bridge structures utilized LUSAS 3D Finite Element Modeling (FEM) software for multi-modal spectral analysis in combination with LPILE for foundation modeling. The use of FEM allowed for rapid iteration and optimization of the bridge configuration, including bridge super- and substructure properties and foundation stiffness. The ability to precisely control and quickly modify bridge properties was especially well suited to meeting the combined demands of ABC construction and high seismic conditions and allowed the design team to quickly respond to feedback from the Owner and Contractor during the design phase, which is key to the success of the CMGC process.

For the NSR Bridge, seismic design for the temporary shoo fly and permanent structures was based upon provisions listed in Chapter 9 of the American Railway Engineering and Maintenance-of-Way Association (AREMA) publication which details design, assessment, and operational criteria for railway structures. AREMA's seismic risk assessment accounts for multiple factors when developing the structure's Importance Classification. These factors are then used to adjust the return periods for various Performance Criteria Limit States (i.e., Serviceability, Ultimate and Survivability) and subsequently the magnitudes of ground accelerations used in design. Although Memphis is known for sandy soils, liquefaction was determined to be unlikely based upon the subsurface conditions encountered at the site.

The new five-span NSR bridge was classified as a multi-span regular bridge due to uniform pier stiffness, relatively uniform superstructure mass and stiffness, straight alignment, and very minor skew. Therefore, the equivalent lateral force procedure was utilized. The period of the structure, T_m, was computed as 0.185 seconds. Seismic weights were calculated based upon superstructure dead loads and 50% mass participation of the piers. Longitudinal seismic assessment neglected the abutments for Level 1 and 2 ground motions. For Level 3 ground motions, one abutment was assumed to be engaged, with spans in contact with the backwall. The longitudinal stiffness of the abutments was based upon soil passive resistance equaling 2/3H (ksf) without the flexural stiffness of the abutment piles considered. Longitudinal seismic demand was proportioned to each substructure element based on stiffness for Level 1 through Level 3 ground motions. Transverse seismic demands were proportioned to substructure units based upon tributary area due to simple spans.

POPLAR AVENUE BRIDGES

The Poplar Avenue bridges are comprised of steel plate girder superstructures with composite cast-in-place (CIP) concrete deck, supported by CIP concrete piers and abutments. The steel superstructures were designed to be constructed off-site in modular units and lifted into their final position during a single weekend closure per bridge. Substructures for each bridge consist of closed western abutment on micropiles, a single median wall pier on micropiles, and a stub eastern abutment on driven piles. Western abutments and median piers were constructed under traffic, while the eastern abutments were constructed during weekend closures and re-buried until the bridge replacement weekend.

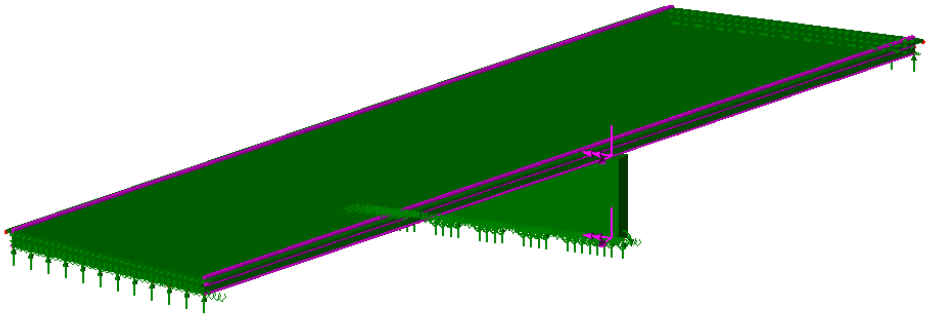

Figure 3 – LUSAS Finite Element Model of WB Poplar, Isometric View

Superstructure Design

Superstructure design was constrained by the need to improve vertical clearance above I-240 with minimal adjustment to the existing Poplar approach profiles. Additionally, both bridges were designed for simple span behavior to eliminate the need establish continuity at the pier, thus minimizing complexity and construction time of weekend bridge replacement. Abutment backwalls are integral with the girders and were poured at the off-site bridge farm and lifted into place with the superstructure units. Typical framing elements were used through most of the superstructure, with additional top flange lateral bracing added on both sides of the pier to ensure lateral load path continuity from bridge deck to pier diaphragms is maintained in the closure pour zone.

Each Poplar Avenue bridge superstructure was built in four modular units at the bridge farm, and then rolled to the project location using Self-Propelled Modular Transporters (SPMTs) while being continuously monitored for excessive deformation during transport to ensure the deck would not be overstressed. The pieces were then lifted into their final position using a tandem pick between a 600 Ton and a 400 Ton crane, with the largest single piece weighing over 1,000 kips with a length of 155 feet. After the superstructure units were placed in their final position, the longitudinal deck joints and closure pour over the pier were completed using Class X Concrete.

Figure 4 – Tandem Pick Between a 600 Ton and a 400 Ton Crane Superstructure Unit Erection

Pier Bearing Design

In addition to serving as the sole link between superstructure and substructure, pier bearing design was primarily driven by two goals:

1. Provide a simple design with low installation difficulty to minimize construction time and risk during weekend closures
2. Fill the vertical gap between existing top of pier cap and proposed bottom of girder, which varies from approximately 8 inches to 18 inches.

To meet these needs, heavy steel pedestal bearings are utilized at the piers. Load is transferred to the substructure through two high strength steel anchors per bearing, which were installed during the weekend closure after demolition of the existing bridge was completed. Bearing pedestals are placed on plain elastomeric pads to accommodate girder end rotations.

Figure 5 – End view of Poplar Avenue Girders showing Bearings

Pier Design

Though the new concrete piers encase the existing median piers, the existing pier structure was largely neglected for the purpose of the new pier design, and the piers were detailed to minimize interaction with the existing structure except at the cap. The new pier wall section was detailed to minimize formwork complexity and maximize speed of construction. The pier wall includes a plastic hinge in the weak axis, which is reinforced with A706 steel to allow the use of a reduced overstrength factor for design of attached elastic elements.

Figure 6 – Poplar Avenue Median Pier During Construction

Micropile Foundation Design

Micropiles were fully designed and detailed in the contract plans. Because the existing median piers were founded on spread footings, micropiles were designed to support all bridge loads in the final condition. Seismic demands governed design in both the axial and lateral direction.

Subsurface conditions in the project area are characterized by dense sands extending to a practically indefinite depth. As a result, axial geotechnical capacity is achieved through soil friction alone. In order to achieve adequate geotechnical resistance, Type B (Pressure Grouted) Micropiles were utilized.

Several casing sizes were investigated for a number of factors. Projected material availability and ability to resist lateral flexure loads were critical criteria for casing selection. Ultimately, 7.625” diameter casings were selected for final design of all structures.

Though micropiles afford the option of aggressive pile batter angles, the design utilized typical 1H:4V pile batter at the piers despite the high lateral demands. This decision was informed by a number of factors, including foundation stiffness effects and a desire to avoid creating a sustained high compression force in the footing. As a result, piles are subject to significant lateral flexural demands in combination with high axial loads in the seismic load case. Pile casing joints within the high flexural demand zones were reinforced as needed.

Micropiles were subject to verification and proof testing at every structure location. Modified load testing schedules were developed to account for LRFD factored design loads and differentiate between testing requirements for Extreme Event loads and Factored service condition loads within a single comprehensive load schedule. All tested piles exceeded their required minimum performance criteria.

Figure 7 – Poplar Avenue Abutment Micropiles

NSR BRIDGE DESIGN

The existing bridge over I-240 was a six-span, double-track, rolled beam bridge constructed by TDOT in 1958 as part of the Memphis Circumferential Interstate Highway project. The bridge had been owned and maintained by TDOT for the last 60 years. The new structure had to meet current seismic design

requirements in accordance with codes developed by AREMA. The proposed replacement structure was a 338-foot-long, five-span, ballasted-deck, double-track, steel deck plate girder bridge.

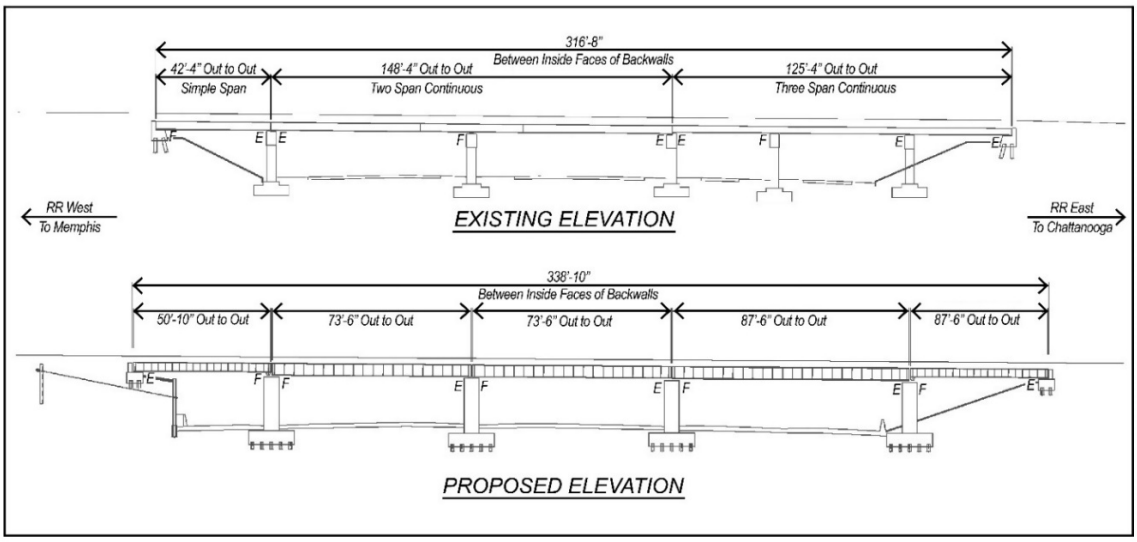

Figure 8 - Existing and proposed bridge elevations

Superstructure Design

Typical NSR deck plate girder structures used on public projects feature a concrete deck utilizing four girders to resist load from each track and additionally a pair of exterior beams to provide a trainman's walking surface integrated into the ballasted deck. However, the high seismic hazard at the project site dictated that the self-weight of the superstructure be minimized to reduce seismic demands on the substructures. To lighten the superstructure, a variance was requested and approved by NSR to use a steel deck plate and eliminate the exterior beams in favor of a discrete trainman's walkway brackets attached to the primary deck plate girders, resulting in a 30% weight reduction versus the typical cross-section.

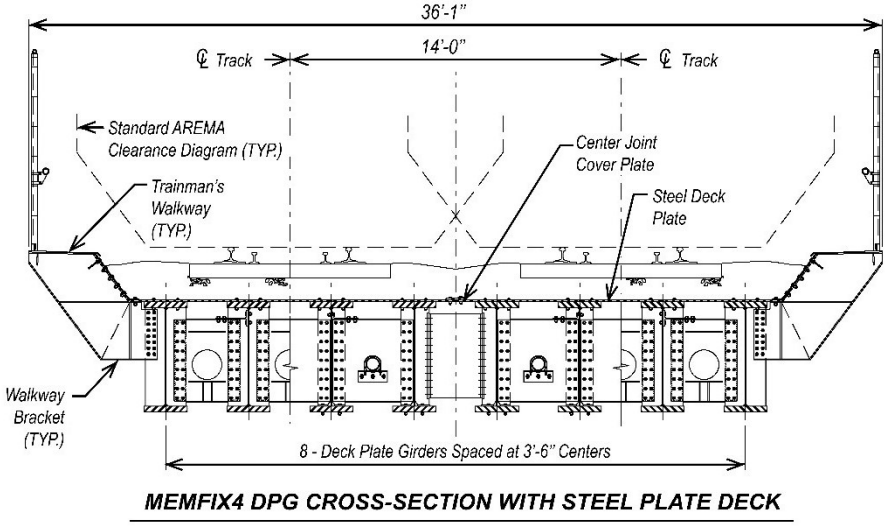

Figure 9 – MemFix4 NSR bridge deck plate girder cross-section

NSR Bridge Slide

The task force explored multiple options to construct the replacement bridge while maintaining rail service and vehicular traffic through the project site. As an additional constraint, Norfolk Southern required that the new bridge be constructed on the existing alignment in order to minimize the introduction of new curves and alignment shifts in this section of rail. Ultimately, it was determined that the use of the same superstructure in both the temporary and permanent locations was the most economical solution. This meant that the permanent superstructure would first be erected on temporary substructures on the adjacent and parallel shoo-fly alignment. During two weekend closures of I-240, the permanent steel superstructure was erected in two beam, modular units which did not require any interruption of train traffic. Once train traffic was shifted to the shoo fly, the existing bridge was demolished making way for the construction of permanent piers and abutments on the original alignment. Once the permanent substructures were in place, a lateral bridge slide of the superstructure from the temporary substructure to the permanent substructure was required.

A scheme for two separate bridge slides was developed by the task force to eliminate the need for a double track outage. Also, in order to minimize the duration of single-track operations, it was decided to move the bridges as fully-intact as possible without removing any dead load due to track and ballast. The design team detailed the superstructure to function as two independent, five-span, ballasted-deck, single-track bridges. These single-track bridges were then moved from the temporary substructure to the permanent substructure one track “half” at a time during full closures of I-240.

Figure 10 – Utilizing a first-of-its-kind center joint allowed the superstructure to be constructed separately and slid one track at a time. Incorporating a custom shoring system allowed the joint to be bolted up, waterproofed, and covered with ballast without impacting the rail.

This approach, which required the lateral repositioning of all five spans simultaneously, necessitated development of detailed special provisions for the slides. These special provisions included a geometry control plan, monitoring surveillance plan, contingency plan, and tolerances for positioning, alignment, elevation, and twist of the bridge.

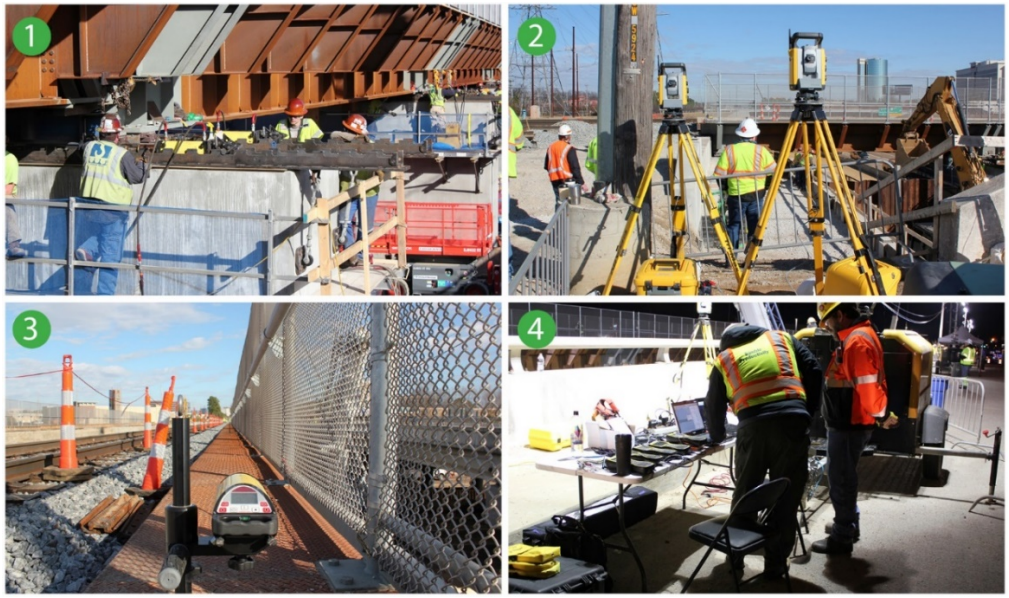

Figure 11 – Careful monitoring during the slide operations

Slide System

In order to perform the lateral slides in a controlled manner and meet the tolerances specified, the GC chose to use a jack and slide system. This procedure used a unified system which included 30 conventional jacks to raise all five spans of each bridge half simultaneously. Once the spans were jacked a few inches, enough to clear the pre-cut bearing anchor rods, they were lowered onto blocking within slide tracks installed along the edges of the temporary and permanent substructures. The slide tracks were made of built-up steel channels with positive key-hole cutouts within the flanges and were anchored to the bridge seats (see Figure 12). The segments of blocking, situated between the flanges of the slide track channels, were made up of steel tube shapes and plates welded together. A low-friction material attached to the bottom steel surface of the blocking, combined with the polished surface of the slide track, reduced the demand on the sliding jacks.

Figure 12 – The slide track at an abutment for the NSR bridge.

Collectively, the design features of the jack-and-slide system allowed the superstructures to be moved laterally in an incremental fashion, allowing the measurement and recording of span movements and ensured that movement to was controlled within allowable tolerances.

The 35'-3" lateral slides of the two 2.2-million-pound bridge halves of the 338-foot bridge superstructure took place over two weekends in February 2019. During these two slides, the interstate was shut down from approximately 10:00 p.m. Friday until about 5:00 a.m. Monday to allow ample time for the GC to mobilize, set up and test the jacking equipment prior to the slides, and then demobilize and clean up the roadway afterward. Additionally, 12-to-16-hour single track outages were granted by NSR.

Figure 13 – The first of two bridge slides; the structure of the siding track is moving into its final location while rail traffic is maintained on the adjacent main track.

The slide of the structure supporting the siding track took place on the morning of February 2nd and returned the siding track to service 16 hours later (see Figure 12). The main track structure slide was performed on the morning of February 9th and returned the main track to service 12 hours later. Both slides were performed within the tolerances specified in the special provision and were completed within the time frames required by TDOT and NSR, a success that can be attributed to the careful planning of the task force and collaboration of all parties involved.