

INNOVATIONS REDUCE COST AND SCHEDULE ON US-24 BRIDGE REHABILITATION OVER LOWER ROUGE RIVER

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ABSTRACT

The Michigan Department of Transportation (MDOT) has undertaken a program to rehabilitate seven bridges along 3.75 miles of US-24 through Dearborn and Dearborn Heights. One of the bridges in Dearborn should have been replaced due to wide, horizontal cracks across its abutment faces and extensive superstructure deterioration. An alternative design approach used a vertical post-tensioning system with an engineered backfill to reuse the bridge's existing concrete abutments instead of replacing them. The project scope was reduced from a bridge replacement to superstructure replacement saving time and money. This technique also reduced construction impacts on the Lower Rouge River and surrounding areas, including the Dearborn Hills Golf Course, and enabled MDOT to advance the overall schedule of their rehabilitation program.

The bridge meets the American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications HS-25 design loading MDOT required at that time. Post-tensioning the walls and using an engineered backfill strengthened the abutment walls and significantly reduced lateral soil pressures.

MDOT seeks context sensitive solutions that enable projects to fit in with their surrounding community and effectively serve transportation needs. Post-tensioning and superstructure replacement reduced the construction schedule to restore this bridge's structural safety and geometric requirements. The surrounding community was well served by limiting impacts on the traveling public, area businesses and an adjacent golf course.

Keywords: Abutment Post-tensioning, Bridge Rehabilitation, Reinforced Backfill, Geogrid, Context Sensitive Issues

INTRODUCTION

Bridge rehabilitation projects come in many varieties. This is clearly demonstrated by MDOT's rehabilitation program for seven bridges along a 3.75-mile stretch of US-24 through Dearborn and Dearborn Heights, Michigan. Bridge rehabilitation approaches ranged from simple epoxy overlays and substructure patching to complete superstructure replacements, pier cap replacements, abutment repairs and deck widening. Innovation was key to rehabilitate one of the bridges known as Structure B01.

Structure B01 is a 60-foot-long, single span, simply supported bridge that carries US-24's four lanes of traffic in each direction over the Lower Rouge River in Dearborn. Three through lanes are included on each side plus ramp lanes for nearby WB US-12. The existing bridge consisted of rolled steel multi-stringers with a concrete deck supported by tall cantilevered abutments. The abutments are founded entirely on timber piling. In addition to spanning the river, the bridge also spans a golf course cart path bridge that parallels the north abutment. This can be seen in Figure 1.



Fig. 1 – Aerial View of B01 Bridge

The original B01 structure was constructed in 1936. A bridge widening occurred in 1957 which increased the roadway width in each direction from 30 to 50 feet. More recent work to the bridge approaches and guardrail occurred in 2003.

PROJECT DESCRIPTION AND INVESTIGATION

Structure B01 is located within a slight horizontal curve of US-24. The existing cross-slope did not meet current AASHTO standards. Although the original scope of work called for deck replacement among other items of work, it became clear during early stages of the project that for cross-slope corrections to occur, work on the top of abutments would require removal of the existing steel girders, temporary storage and re-setting in place. Another major item of work was concrete patching at both abutments. The original scoping report noted horizontal cracking along the faces of both abutments. The horizontal cracking was continuous from one end to the other and measured 1/16 inch on average. Cracking was present some six to eight feet below the bearing seats. This is seen in Figures 2 and 3.

Two other measurements taken during the preliminary phases of the project indicated further deterioration. One was a measurement along the construction joint where the older and newer sections of the abutment wall were joined. Above the cracks, near the top of the wall, the two faces either side of the construction joint were offset by 1 3/4 inches, while below the cracks, the two faces were plumb and flush with one another as shown in Figure 4. The other measurement was taken between the north and south abutment faces and compared to plan dimensions. According to the as-built plans, the clear dimension was 56'-7". The as-measured dimensions were short by 3 1/2" to 5 1/2", depending upon the location along the abutment faces.



Fig. 2 – North Abutment



Fig. 3 – South Abutment

An initial assumption proposed that if the top of the wall was offset by almost two inches and portions of the walls at and below the horizontal cracks were plumb, it seemed logical that the top of the walls may have been pushed back or towards the approaches. Cracks appearing on the outside face supported this assumption. Under true cantilever wall design, walls overloaded from lateral soil loads above what they were designed for would show

tension cracks on the inside face while the outside face would be in compression. Any horizontal cracks along the outside face should close from this compressive force.



Fig. 4 – Differential Wall Movement

To further study the cause of these cracks, other assumptions were proposed. One was that the bearings locked up due to age and rusting and thermal expansion of the steel beams pushed the walls out. However, thermal expansion calculations for these steel beams that are approximately 61 feet long are less than ½ inch. This small horizontal expansion would not be the cause for the almost two inches of measured lateral wall movement.

The next assumption was that the walls are sliding or had previously slid in conjunction with the bearing lock up and subsequent strut action of the steel stringers. The walls sliding towards each other, along with the strut action at the top of the abutments, would have caused the outside faces to go into tension, for which they would not have been designed for as cantilever walls. This assumption seemed consistent with the change in horizontal measurements between the north and south

abutment faces. Further investigation was required to prove the 3 ½- inch to 5 ½-inch differences could have been caused by wall movement.

Full depth concrete core samples were taken from each abutment wall at the location of the cracks to determine the character of the cracks. It was noted that the cracks were either full depth or near full depth, and that they were widest at the front face and minimal to no crack at the back face. This was consistent with a wall bending outwards towards the river, acting in a simple span configuration rather than as a cantilever.

A geotechnical investigation was performed on the soils behind both abutment walls¹. Results showed that the existing soil densities ranged from 109 pcf to 114 pcf, and an equivalent fluid pressure of 38 psf/foot. These values are fairly consistent as typical soil parameters for a drained soil behind abutment walls. These values were used as part of a wall analysis of the existing abutments. Using the WINABUT[®] program with modified loadings specific to MDOT load cases, several models were run of the existing abutments using the measured soil densities and lateral load. The original 1936 abutments as well as the original plus widening were separately analyzed. The outputs were a determination of the walls' structural section capacities as well as their vertical and lateral pile capacities.

It was determined that the back face (soil-side) reinforcement was 13% overstressed. In addition, the pile sliding resistance was 64% less than required, and the vertical pile capacity was 32% less than required. Based on these values, it was concluded that the walls most

likely had slid towards each other due to the deficient pile capacities, and along with the strutting action of the steel stringers, the front face horizontal cracks developed along both walls. The minimum reinforcement provided on these front faces could not resist the tension causing cracks to form.

Two rehabilitation alternatives were offered for consideration in lieu of just a deck replacement. One was a complete bridge replacement and the other was a superstructure replacement with abutment repairs.

COMPARISON OF ALTERNATIVES

In the past, the US-24 bridge's failing abutments would have been deemed unusable and removed. This would have resulted in a bridge replacement project. As part of the study phase for this project, bridge replacement costs were evaluated. A complete bridge replacement with approach work had an estimated cost of \$2.08 million. This would have included removal of the superstructure, removal of substructure concrete, full and partial removal of the existing timber piles, and construction of a new bridge utilizing 33-inch precast, prestressed, concrete spread box beams. While a new bridge could potentially give the longest service life, the cost along with longer disruption to traffic and greater impact to the golf course in having to remove the cart path boardwalk did not make this an attractive alternative. If the existing abutments could be kept through long term repair and superstructure replacement only, costs and time delays could be significantly reduced.

With the type of analyzed deficiency that occurred at the abutments, it was felt that reducing the lateral soil loads and also "re-connecting" the top half of the wall to the bottom half to close up the crack could provide a long term solution to the cracking problem, rather than just patching the crack. The goal was to provide a permanent fix rather than one that would require additional, more costly repairs in the near future.

With a preliminary cost estimate of \$1.32 million, the superstructure replacement with abutment rehabilitation offered a savings of \$760,000. Additional benefits included less disruption to US-24 traffic, and with proper safety, provisions for the golf course cart path to remain open during construction.

DESIGN METHODOLOGY

The proposed bridge replacement was designed to AASHTO HS-25 loading. The precast spread box beams were designed under Load Factor Design using Leap Conspan[®] design software. There are a total of 15, 270K- low relaxation strands in each box beam over two rows (13 in Row 1, 2 in Row 2). The deck slab is a nine-inch-thick reinforced concrete slab

placed over galvanized stay-in-place forms. A minimum one-inch haunch exists over each box beam.

For the abutment design/analysis, an iterative series of runs were done using WINABUT with the same modified loadings specific to MDOT load cases. The iterations were necessary to zero in on the controlling equivalent fluid pressure (EFP) that would result in the existing abutment and pile foundation system safely carrying the loads without distress. The controlling EFP as analyzed by the consultant was determined to be 14.2 pounds per square foot per lineal foot depth. In order to accomplish this, an engineered backfill utilizing alternating layers of HDPE biaxial geogrid and a well graded backfill material of varying thickness was designed for placement behind each abutment wall. The final as-designed backfill had an EFP of 12 psf/ft, some 74 percent less than the existing backfill in place prior to this project. Figure 5 illustrates a typical cross section of the engineered backfill.

The length of the geogrid was designed to extend 16 feet behind the abutment wall with three-foot anchorage laps immediately behind the back face of wall. The engineered fill extends the full width of the abutment walls between the 90 degree return wingwalls.

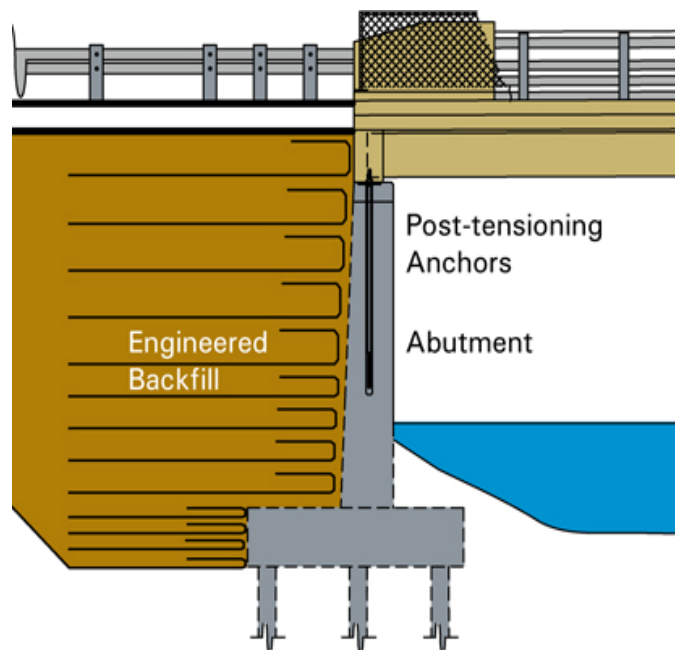


Fig. 5 – Typical Abutment Cross Section

CONSTRUCTION

Figure 6 shows the cross section of the proposed superstructure. Due to the requirement to maintain US-24 traffic, superstructure construction and abutment repair was performed in two stages. The first stage consisted of maintaining one through lane of traffic in each direction on the outside and one lane of exit and entrance lanes for nearby westbound US-12, which crosses over US-24 a few hundred feet south of the project site. The inside portion of the bridge consisting of nine beams and deck were removed. The top three feet of the abutments were partially saw cut and removed to provide a horizontal surface for the installation of the post-tensioned rods into the abutments. The existing reinforcement was left in place where still useable, and new reinforcement was added along with galvanized anodes to protect the existing reinforcement. Temporary sheeting was placed between the active construction stage and the in-use roadway on each side to retain the approach backfill under the active roadway.

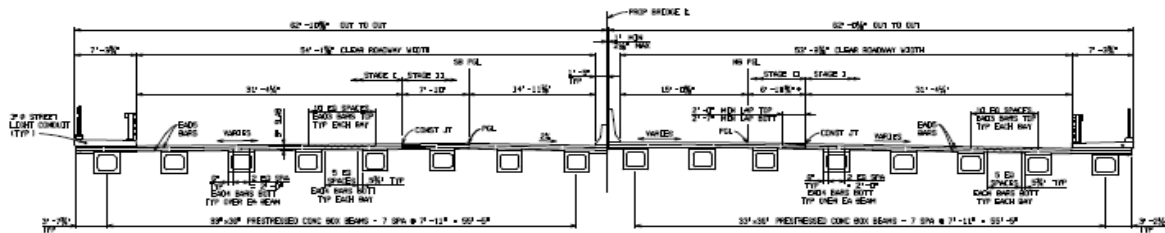


Fig. 6 – Typical Deck Cross Section

The existing backfill in Stage 1 was removed down to the bottom of footing for the entire width and for a length of slightly over the 16 feet required for the geogrid installation. Beyond that point it was excavated out on a one-on-one slope back up to the existing subgrade. Along the bottom of the excavation, a composite drainage mat was placed. This mat consisted of filter fabric on both sides of a polymeric drain core. The mat was run to a new foundation underdrain placed behind the wall that was connected to the existing weep holes equally spaced in walls. This composite drainage mat was installed to help prevent any long term hydrostatic pressure on the back face of the abutment walls. The geogrid was then placed in alternating layers with compacted Class II sand, and brought up to an elevation slightly below top of concrete. In the excavated area beyond the reinforced backfill area, Class II sand alone was placed and compacted.

After completing the backfill operations, three-inch-diameter holes were drilled vertically into the top of the abutment at 7'-11" on center, midway between each proposed box beam which had the same spacing (Figure 7). Figure 8 shows the drilling operations at the north abutment. The holes were drilled a minimum of three feet below the crack line. Grout was placed in the bottom portion of the hole followed by a one-inch-diameter, high strength steel rod with threads top and bottom and a smooth shaft between. The overall length varied depending on the location of the crack, but on average they were approximately 10 feet. The first stage of grout was required to cure for a minimum of 72 hours before the second stage of

grout was placed around the remainder of the rod. Immediately after the second stage grout placement, a bearing plate, trumpet and nut were placed and the rods were tensioned to 30 percent of tensile yield. For a one-inch rod with an area of 0.85 square inches and a yield of 150 ksi, the load was calculated to be slightly over 38 kips per bar. This closed the cracks while allowing enough compressive strength in the existing concrete to carry the vertical load from the superstructure.

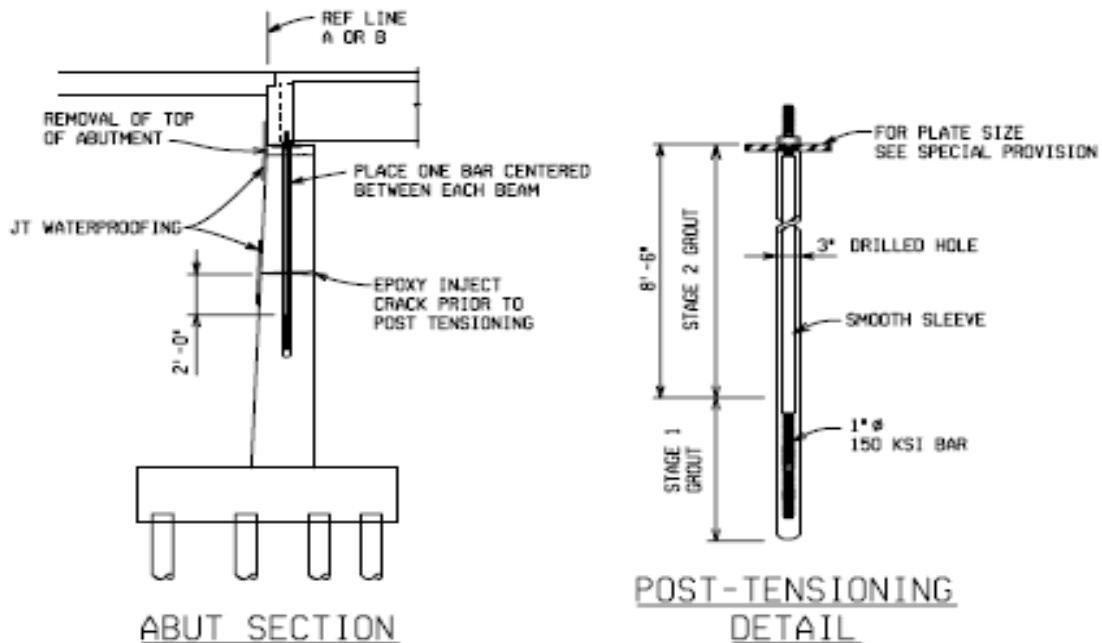


Fig. 7 - Post-Tensioning Details

With the grout set, the tops of the abutments were cast with the new beam seat elevations set. The beams were placed with an expansion bearing at one abutment and a fixed bearing at the other abutment. The remainder of the superstructure was finished and then traffic was shifted onto the interior portion of the bridge (one lane each direction) while the outside portions of the bridge were closed to traffic and reconstructed in a similar fashion to that described above. During construction of the outside portions, traffic to and from the adjacent WB US-12 ramps was detoured while the ramps were closed.

CONTEXT SENSITIVE ISSUES AND SOLUTIONS

Context sensitivity was critical to this bridge rehabilitation solution. The project team proactively evaluated issues right from the start to determine what the best alternative would be to minimize construction impacts.

The first issue was the impact to the Lower Rouge River. This branch is one of three major branches of the Rouge River, a major watershed in southeastern Michigan. The Lower Rouge River flows through approximately a dozen communities before joining with the other two branches some four miles east of the project site. If this had been a bridge replacement project, a major disturbance of the river bottom sediments would have occurred with the installation of a sheet pile cofferdam as well as installation of the foundation piling. These sediments would have been transported downstream and impaired stream quality. The cofferdam would have restricted the width of the river and during storm events with much higher flows the likelihood of sediment scour would have been much greater.

The superstructure replacement and abutment repairs performed resulted in minimal impact to the river. No cofferdams were required, no foundation piling was installed and the small amount of concrete abutment that was removed was caught by a small barge placed in the river adjacent to the work zone. False decking was placed between the existing steel beams during deck removal to catch falling concrete.

The next issue was the impact to the traveling public using US-24. This is a major north-south urban arterial in western Wayne County that carries some 59,000 vehicles per day. To have an extended construction time frame would result in user delays and major traffic back ups especially during rush hour. It was estimated that a bridge replacement could take six to eight months to complete with staged construction. The rehabilitation approach reduced construction time to four months, a savings of between two to four months. Not only did this allow normal traffic flows to resume quicker, it also allowed the contractor to move on to other segments of the overall project sooner.



Fig. 8 - Post-Tensioning Drilling & Roof Construction

construction project, various alternatives were looked at to shuttle golfers across US-24. Alternatives like a signalized pedestrian crosswalk at US-24 to the project paying for shuttle

The third issue was the impact to the Dearborn Hills Golf Course. This public, 18-hole golf course has been in this location for years. The course is laid out on both sides of US-24 within the floodplain of the Lower Rouge River. To traverse from one side of the course to the other side, a timber pathway structure parallels the north abutment wall and is attached to the wall. A bridge replacement would have required removal of the pathway structure. Since the golf course was adamant about not being shut down by the

bus service between the two sides proved either unsafe or too expensive. With the superstructure replacement option, the entire golf course remained open. A temporary roof constructed of cantilever trusses attached to the north abutment wall and covered with a metal roof deck was constructed along the entire pathway structure to protect golfers and golf course workers from debris that might fall from the construction above. Figure 8 shows the roof under construction. There were only small time frames where the path was temporarily closed during the concrete removal or concrete placement. Other than that, it was open for business as usual.

CONCLUSION

While the post-tensioning technique is commonly used in new structures, it is used less frequently on existing structures, and can have very limited applications, especially with existing bridges. More often, the decision is made to remove and replace the affected element. For this project, it would have meant a bridge replacement, including the abutments since the existing had been designed to carry less load, temporary closure of part of the golf course and increased user delays on and adjacent to US-24. By vertically post-tensioning the abutments, only the top few feet of the abutments needed to be removed rather than the entire element. This resulted in a superstructure replacement only, saving the abutments which saved time and money.

The original abutment walls were designed to carry less load than what is required by current State and AASHTO standards. Vertically post-tensioning the walls and replacing the backfill with an engineered backfill, strengthened the abutment walls and significantly reduced the lateral loads. HS-25 loading requirements were achieved while reusing the existing abutments and foundation system.

Impacts to the Rouge River, the adjacent golf course and the traveling public were minimized using a context sensitive approach. The rehabilitated bridge will serve the surrounding community well into the future.

REFERENCES

1. Stoops, J and Wanlass, D, "Report of Geotechnical Investigation, Proposed Bridge Improvements, Telegraph Road over Lower Rouge River, B01 of 82053-JN 58175D, Dearborn, Michigan", G2 Consulting Group, March 7, 2005.