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# Reconstruction of the 7th Street Bridge over the Black River
# Port Huron, Michigan
# Pier Stabilization

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## Abstract

The 7th Street Bridge over the Black River in Port Huron, Michigan is a unique single leaf bascule designed by J.A.L. Waddell in 1931. The bridge, completed in July of 1932, is on the national register of historic places and is the only single leaf bridge remaining in the state of Michigan. The rest pier and the bascule pier are founded on timber piles. During the life of the bridge, the piers have been moving towards each other, necessitating “trimming” of the superstructure and rest pier concrete to allow operation of the leaf. In fact, over the life of the structure, survey results indicate that the piers have moved closer together by some 7 inches. The City of Port Huron contracted with HNTB to provide engineering design services associated with the planning and design for foundation stabilization, superstructure rehabilitation, control house replacement, and drive machinery and electrical system replacement. This paper focuses on the challenges associated with stabilizing the piers.


## Introduction

The 7th Street Bascule Bridge is a 114’-0” Single Leaf Bascule with girders haunched on both ends to give the appearance of a double leaf bascule bridge. The Bascule Pier is located on the South side of the river and the rest pier to the North. The piers are founded on timber piling likely driven to bedrock.

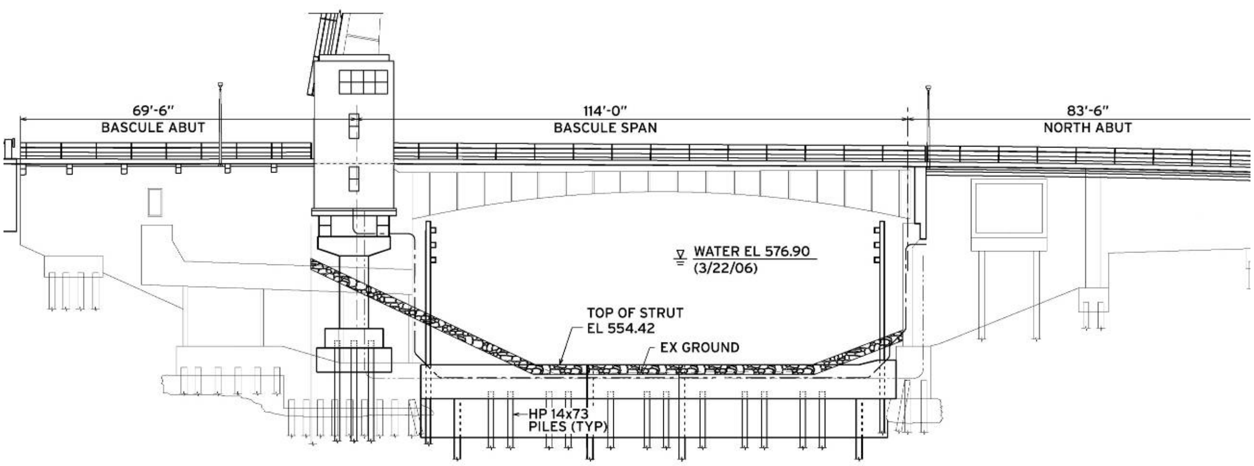



**Figure 1:** *General Elevation*

## Background

Over time the piers of the 7th Street Bridge have moved some 7 inches closer together. Vertical movement has been observed to be minor. During the last 30 years, the ends of the bascule girders have been cut back on a regular basis to allow the bridge to open and close. The North Abutment Concrete was also chipped back to provide clearance as the distance between the piers continued to shrink. Surveys performed from 2002 through 2005 confirmed that pier movement was still occurring and active.

In the spring of 2005, we began a geotechnical investigation and analysis of the bridge. During the investigation, we made the following observations. The offset in the aluminum railing shown in figure 2 offered antidotal evidence that the steep slope to the west of the south abutment is creeping toward the river relative to the bascule pier. At the southeast corner of the bridge, the slope paving shown in figure 3 is buckled and more dramatically shows that the south bank is not stable. But in both cases, it is clear that the banks have moved *more* than the bascule pier. Further, the bascule pier may actually be stable, since the intersection directly behind it has not suffered from any unusual distress. For instance, an old deep brick sewer manhole located at the north end of the intersection was in good condition when last observed by City officials.

**Figure 2:** *Railing at SW corner of bridge*

**Figure 3:** *Slope Paving at SE corner of bridge*

In contrast, the pavement behind the rest pier on the north bank was observed to have settled relative to the pier. Overall, our observations tend to suggest that the north abutment has moved more than the south abutment.

## Exploration and Subsurface Conditions

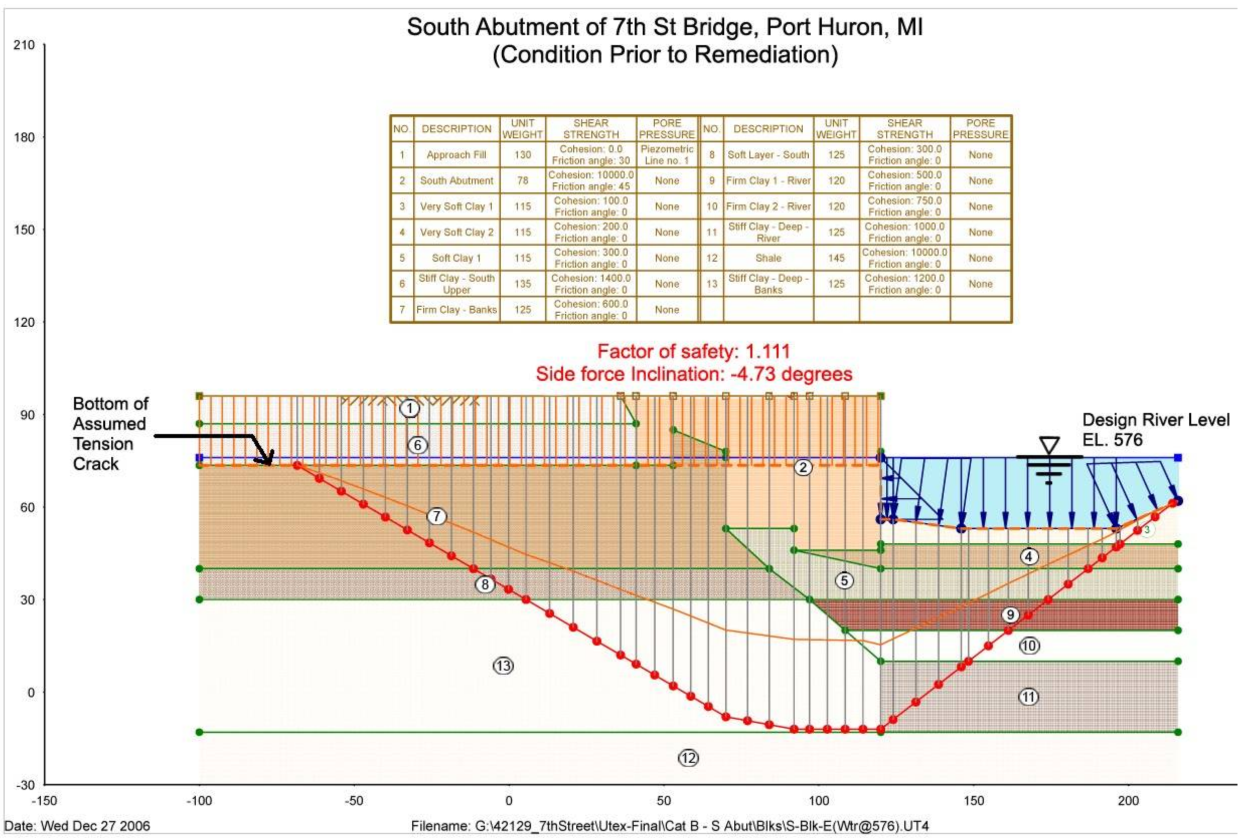
Three exploratory borings were performed: one behind each abutment during preliminary design and one on the north edge of the river during final design. The borings ranged in depth from 100 to 120 feet.

Behind both abutments, top of pavement is about 40 feet higher than the river mudline and 20 feet higher than the river water surface. Below the pavement is a layer of silty sand fill. The fill is about 10 feet thick behind the south pier (bascule pier) and about 26 feet thick behind the north pier (rest pier). Wood, brick and rock debris were encountered in the bottom half of the fill behind the north pier. Underlying the fill is a thick deposit of soft to stiff lean clay with traces of gravel, sand and silt. The lean clay deposit is underlain by hard black shale bedrock (Antrim Shale) encountered 70 feet below the river mudline.

Two generalized soil profiles were developed: one with undrained strength parameters and one with drained strength parameters.

In the undrained strength profile, the thick native clay deposit was assigned a friction angle of zero. The thick clay deposit was divided into several different zones to capture expected differences in undrained shear strength. The undrained strength profiles for the south and north banks are shown in Figures 4 and 5, respectively.

In the drained strength profile, the thick clay deposit was assigned an effective cohesion of zero. Because unusually low strength test results occurred between Elevation 530 and 535, old land slides were suspected. For that reason, soils above Elevation 530 were assigned a residual drained strength friction angle of 17 degrees. Soils below Elevation 530 were assigned an effective friction angle of 27 degrees.




| NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE | NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Approach Fill | 130 | Cohesion: 0.0 Friction angle: 30 | Piezometric Line no. 1 | 8 | Soft Layer - South | 125 | Cohesion: 300.0 Friction angle: 0 | None |
| 2 | South Abutment | 78 | Cohesion: 10000.0 Friction angle: 45 | None | 9 | Firm Clay 1 - River | 120 | Cohesion: 500.0 Friction angle: 0 | None |
| 3 | Very Soft Clay 1 | 115 | Cohesion: 100.0 Friction angle: 0 | None | 10 | Firm Clay 2 - River | 120 | Cohesion: 750.0 Friction angle: 0 | None |
| 4 | Very Soft Clay 2 | 115 | Cohesion: 200.0 Friction angle: 0 | None | 11 | Stiff Clay - Deep - River | 125 | Cohesion: 1000.0 Friction angle: 0 | None |
| 5 | Soft Clay 1 | 115 | Cohesion: 300.0 Friction angle: 0 | None | 12 | Shale | 145 | Cohesion: 10000.0 Friction angle: 0 | None |
| 6 | Stiff Clay - South Upper | 135 | Cohesion: 1400.0 Friction angle: 0 | None | 13 | Stiff Clay - Deep - Banks | 125 | Cohesion: 1200.0 Friction angle: 0 | None |
| 7 | Firm Clay - Banks | 125 | Cohesion: 600.0 Friction angle: 0 | None | | | | | |

Figure 4: *South Abutment – Most Critical Potential Failure Plane*

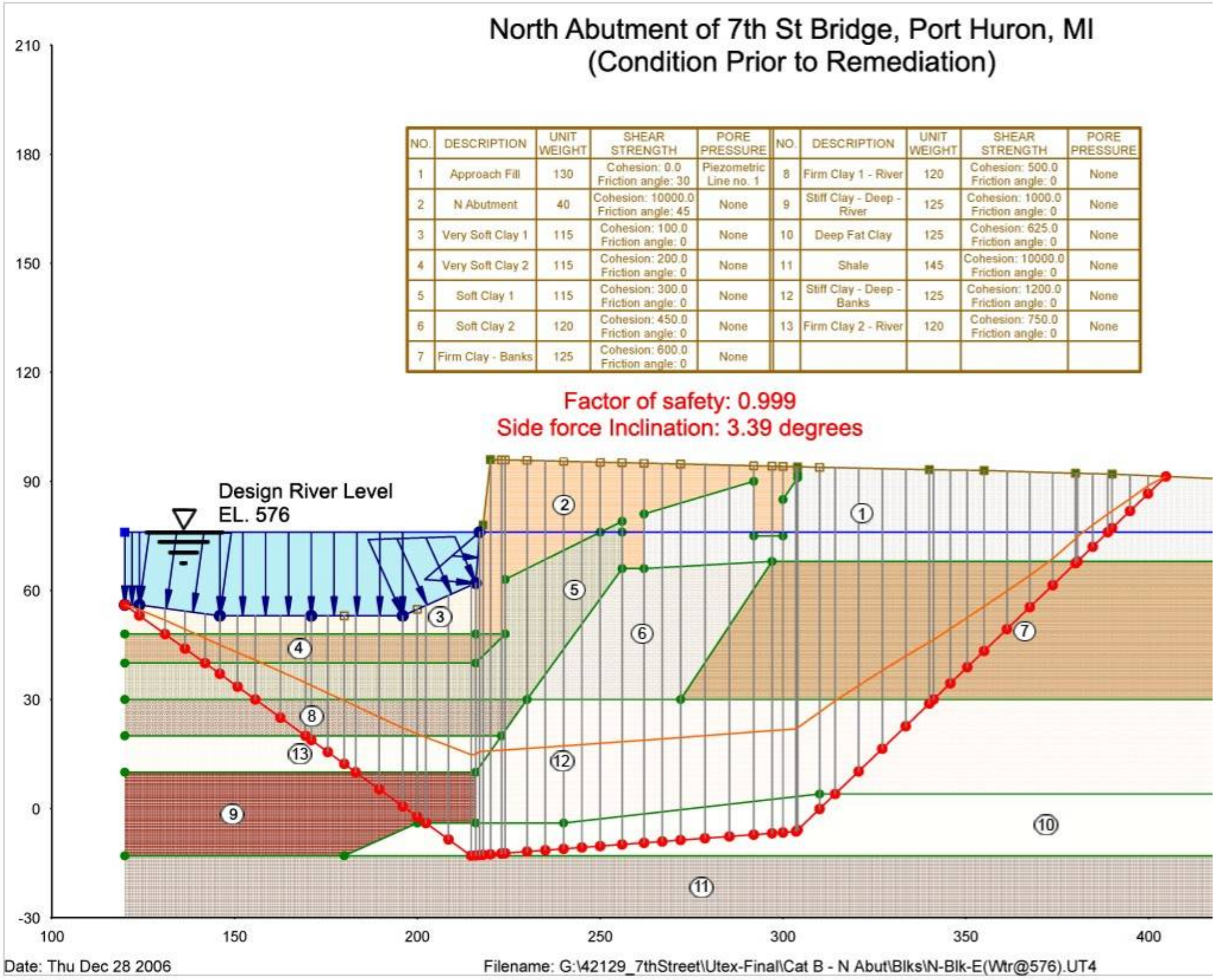



Figure 5: *North Abutment – Most Critical Potential Failure Plane*

## Stability Analysis

Using Spencer's method in the program UTEXAS4, many potential slide planes below both abutments were analyzed to find the most critical slide surfaces. The potential slides of concern to both abutments were separated into two categories: A and B. Category A slides are block slides that do not penetrate the stiff clay encountered below Elevation 530. To move an abutment toward the river, a Category A slide would have to laterally deflect the existing timber piles that extend below the slide. Category B slides are block and circular slides that extend to the soil-bedrock interface. Some existing timber piles might bear on the shale bedrock, but they are not expected to penetrate very far into the shale and were not assumed to resist Category B slides.

When analyzing Category A slides, which pass well above the estimated tips of existing piles, the weight of the abutments was removed from the analysis because their weight was assumed to be fully supported by existing piles bearing below the slide mass. When analyzing Category B slides, which pass near the tips of existing piles, the weight of the abutments was included. Analysis of a Category B block slides appear in Figures 4 and 5.

Despite assigning residual drained strength parameters to all soils above Elevation 530, the total stress analyses with undrained strength parameters proved to be more critical for both categories of slides. Results of the total stress analyses for the bridge prior to being stabilized are summarized in the table below:

*Critical Factors of Safety for Pre-Rehab Condition*

| Abutment | Category A Slides | Category B Slides |
|---|---|---|
| North | 1.0 | 1.0 |
| South | 1.0 | 1.1 |

Though our analysis was intentionally conservative and indicates that either pier could be the source of historic movement, the north (rest) pier appears to be the more likely to have moved. In addition to our analysis and site observations, we discovered by researching the history of Port Huron that the north side of the mouth of the Black River was created by importing fill (see figure 6). This information supports our conclusion that the north pier is the side that is experiencing movement. It was built on fill as opposed to the natural ground on the south side of the bridge.

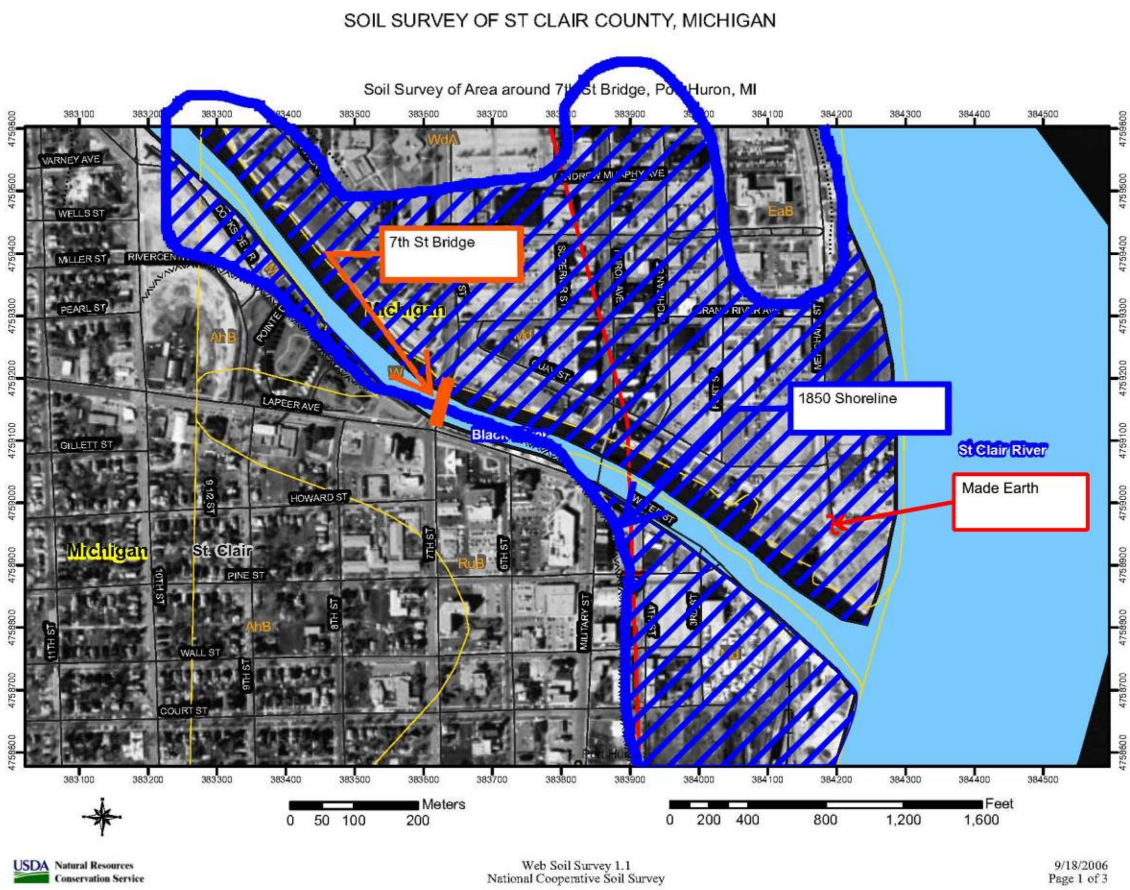



**Figure 6**: *Map of Port Huron Showing Shoreline Circa 1850*

## Concepts Considered to Arrest the Pier Movement

The first course of action was to determine whether replacement or rehabilitation was the best option. From a cost standpoint, rehabilitation was estimated to be in the $10 million range, while replacement was estimated to be more than $20 million. Rehabilitation also afforded the best opportunity to retain the character of this historic bridge and was the option selected by the City. Bids were received on May 4, 2007, and the contract was awarded to Anlaan Corporation for $10.2 Million.

*A. Grouted Soil Anchor Tiebacks*

Consideration was given to installing a tieback system consisting of grouted or helical anchors. This option would have required very long embedment lengths perhaps as deep as 100 feet. This was due to the fact that the soil failure plane was deep seated and the tiebacks would need to extend significantly beyond this plane. These would have been installed in both piers. While the North Pier was more suspect to movement, it could not be ruled out that both piers were moving. Furthermore the North Pier is the Rest Pier and its construction is therefore much lighter. High forces would have been needed to be transferred between the anchors and the North Pier. This would have required large transfer elements inside the pier and potential strengthening of the pier walls to handle the load. The work required in the North Pier alone for this option appeared to be similar in cost to that of total replacement of the rest pier.

*B. Battered Steel Piling and Concrete Transfer Beams inside Piers*

Since the deck of the North Abutment had extensive rehabilitation, consideration was given to driving battered piling inside the piers and using transfer elements to connect to the pier walls. This required a large number of piles, heavy transfer elements and expensive connections to the pier walls. Like option A above the costs were prohibitive and did not address any possible movement in the South Pier.

*C. Battered Steel Piling with Concrete Thrust Blocks in front of Piers*

Transfer blocks on battered piles were studied in depth during the conceptual stages of the project. However, upon more detailed study during final design, a lateral load analysis of the pile group using the program GROUP 6.0 revealed that a transfer block supported by the largest practical number of piles would be too flexible.

*D. Lightweight Fill*

This technique reduces the driving load on the potential slides. It raises the factor of safety against both a rotational and block failure slide. It has the additional benefit of reducing settlement behind the abutment. This option was selected in combination with the Cast-In-Place Struts described in the next section.

*E. Cast-In-Place Concrete Struts below the Mudline between the Piers*

The navigation depth required for the type of vessels that pass through the channel was not extremely deep (around 10 feet) and the desired dredged depth was about 20 feet. Therefore, stabilizing the piers below the mudline was a possibility. Furthermore, during final design it was discovered that a newer movable bridge down river at Military street utilized pre-cast struts running below the mudline. The struts at Military street created a maximum dredged channel depth of about 20 feet at normal river elevations. After further study of boat traffic and backwater calculations, it was determined that this was a feasible solution. Our biggest concern was the feasibility of constructing cast-in-place concrete struts between two existing piers that would require cofferdams de-watered in excess of 30 feet below the water surface.

*F. Pre-Cast Concrete Struts below the Mudline between the Piers*

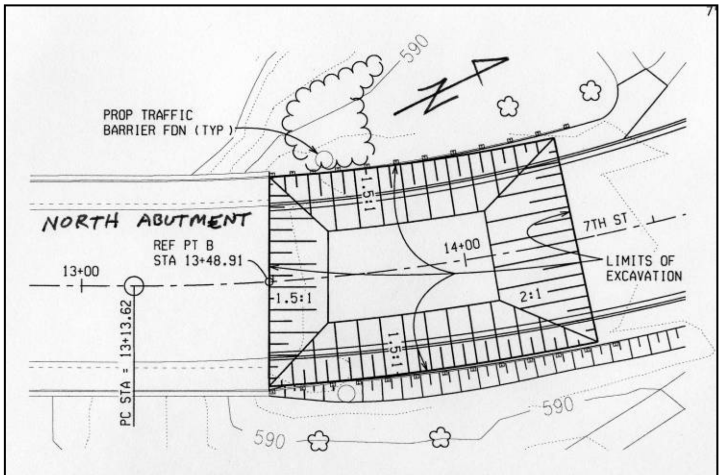
Nearby, at the Military Street Bridge, pre-cast struts were used to brace the piers below the mudline. This strategy was used at Military Street in combination with new piers as a “belt and suspenders” approach to protect against pier movement. This approach was also facilitated by the fact that while constructing the piers they were able to plan ahead for the installation of pre-cast struts.

At the 7th Street Bridge, the river is at a slight skew to the bridge and the forces from shifting soil masses would deliver a force into the stabilizing struts at a slight skew. Therefore the struts needed to be attached to the piers. Furthermore, uplift forces on the struts from the potential passive soil failure wedge also necessitated that the struts have a moment connection at the piers. These two factors precluded the use of pre-cast struts without a positive attachment to the piers. A positive attachment of a strut to the pier required a cofferdam, and since the cofferdam adjacent to the pier is the most difficult part of the strut construction, a cast-in-place option was favored over the pre-cast option.

**Selected Alternative**

The selected option employed the use of two 7’x7’cast-in-place concrete struts to brace the abutments against each other. A follow-up structural analysis found that the north abutment was not structurally able to transmit more than 1650 kips of lateral earth pressure to each CIP concrete strut (for a total of 3300 kips). Therefore a reduction in the driving force was necessary. To do this we combined the cast-in-place concrete strut option with lightweight fill behind the North Abutment. We replaced about 1,000 cubic yards of soil with lightweight fill with a unit weight of 30 psf. We thereby reduced the weight of soil behind the north abutment by some 1,200 tons. In combination with the maximum allowable resistance from the struts, this reduction in soil weight increased the rest pier’s factor of safety on global stability to 1.4.

After consideration of several lightweight fill options, lightweight “foamed” concrete was specified. This product was selected as it can be excavated after placement without special equipment and, unlike EPS blocks, would simply flow to fill the entire excavation, eliminating the need for hand compaction in tight areas. A plan view of the necessary 16-foot deep removal and replacement is shown to the left. In accordance with State policy, the top of the lightweight fill layer was held three feet below the top of pavement to limit the potential for differential icing. To minimize the potential for problems with groundwater intrusion, the bottom of the excavation was set at Elevation 580, 2 feet above the normal elevation of the river. One other concern for lightweight fill was floatation during a flood event. By limiting the depth of the replacement, a suitable factor of safety against uplift was achieved. During construction, some minor groundwater seepage did lead to disturbance and pumping of a portion of the base of the excavation. Disturbed materials were replaced with gravel then the lightweight concrete was placed. It was mixed on-site and quality control testing verified that required densities and strengths were achieved.



PLAN OF EXCAVATION FOR LIGHTWEIGHT FILL

The impact of excavating and dewatering the cofferdams used to construct the CIP struts on the global stability of the abutments was analyzed using detailed models in UTEXAS4. Because planned excavations would undercut the Category A slides, temporary cross-river braces were installed prior to cofferdam construction. The risk of destabilizing a Category B slide below the rest pier was minimized by replacing the embankment with lightweight fill before installing the CIP struts. At the south abutment, the risk of dewatering a cofferdam was unavoidable but was considered acceptable since the reduced factor of safety was not less than 1.1.

Our main concern at this point was constructability of the struts within cofferdams that would need to be sealed up against an existing pier. We discussed the potential construction methods of the struts with three Michigan contractors prior to letting the project out to bid to assess this risk. All three convinced us that, while difficult to construct, it was feasible. Double walled cofferdams, bentonite, secant piling and grouting were all methods discussed to seal the three sided cofferdams to the piers.

## Pier Monitoring

To address risks associated with excavating and dewatering cofferdams in front of the marginally stable abutments, HNTB developed and specified a movement monitoring program for the abutments. Four points on corners of each abutment were surveyed daily by the contractor and biweekly by the engineer's surveyor. To detect movement trends as early as possible, a precision of +- 0.01 feet or better was required. The contractor was responsible for reviewing the survey results daily and for quickly responding to arrest any movements in excess of 0.04 feet. During strut installation, the movement monitoring data was also analyzed by HNTB on a weekly basis. Over the course of all potentially destabilizing construction activities, the bascule abutment moved about 0.03 feet (north) toward the river and the rest pier moved about 0.02 feet (south) toward the river. Although the amount of movement detected was small and was not large enough to trigger a response, the timing of incremental movements was found to correspond to the most adverse river work activities. The temporary cross-river bracing was not monitored by strain gages, so its contribution to limiting movement during construction is not known.

## Construction Challenges

It was anticipated that during construction in the cold weather climate of Michigan that cofferdams would be susceptible to Ice Jams that could case damage to the cofferdams and potential upstream flooding. This combined with the need to keep more than half of the river open to prevent high backwaters; it was decided to build the struts in thirds.

In an effort to remove some of the uncertainly of subsurface conditions and potential obstacles to the construction of the struts; prior to construction of the cofferdams, we required the contractor to probe for debris along sheet pile lines. During probing, concrete rubble as deep as 16 feet was encountered in front of most of the south abutment. It had to be removed to allow installation of sheet piling. To minimize impacts to the marginally stable abutment, HNTB developed a plan for staged removal and replacement of the rubble with sand backfill.

As mentioned earlier the biggest concern during plan development was the ability of the contractor to seal the cofferdams up against the pier to facilitate de-watering. During construction this proved to be the biggest challenge. The sealing of the cofferdam was not difficult for the vertical face from water line down to the existing footing. However, below that level it proved to be extremely difficult. The footing was not built in clean straight lines but rather was jagged and set on top of a tremie that was equally "jagged". Furthermore, some of the existing piling for the fender and wood formwork was still in place. The result was a cofferdam that was very difficult to seal.

Three options were considered when this situation was encountered. A double walled cofferdam that would be grouted between the sheets in the area of the existing footing and tremie was the first option considered. The second option was to use secant piling adjacent to the piers. The third option was to pour a reinforced tremie pour. This final option was decided to be the most cost effective option.

Therefore the cage of steel next to the pier was tied up on a barge and the ends of the re-bar placed through a bulkhead that would be the forming surface for the reinforced tremie pour. The cage and form were lowered down to the proper elevation and braced against the cofferdam sheeting and weighted down so that the bulkhead would not float during the tremie pour. Horizontal ducts were used to block out locations that would be used after de-watering to drill through the reinforced tremie pour to drill and grout re-bar into the existing pier footing to connect the strut to the pier. This method worked extremely well to seal this area of the cofferdam and was employed for all four connections of the struts to the existing pier footings with success.

Figure 7: *Rendering of Concrete Strut showing the staged construction*

After the first section of the struts were built, the strut construction progressed across the river in thirds with a transition cell used to facilitate splicing the re-bar in the strut as shown in Figure 7 above. The sheeting at the interface of the transition did leak significantly between the first and second strut pours. Therefore, the contractor welded studs onto the transition sheets from the second to third strut pours to hold the sheets tight against the previous pour and prevent water from coming in through the interface at the re-bar locations.

## Conclusion

With the addition of the struts and by replacing a portion of the north approach embankment with lightweight fill, the stability of both piers was increased to acceptable levels. Calculated factors of safety for the fully rehabilitated structure are summarized below. The strut construction was successfully completed in April 2008.

Though there were challenges during construction, this method of stabilizing the piers proved to be both an economical and effective solution. The primary challenges were rubble from the previous swing bridge piled against the south abutment and difficulties with sealing 3-sided cofferdams against the existing abutments. If there was a lesson learned, it would be to indicate the use of reinforced tremie pours for the 3-sided cells against the irregularly faced existing piers in the design plans thereby eliminating the difficulty of dewatering the cofferdams.

*Critical Factors of Safety for Rehabilitated Condition*

| Abutment | Category A Slides | Category B Slides |
|---|---|---|
| North | >1.5 | 1.4 |
| South | 1.5 | 1.4 |

## References

HNTB Corporation; January 10, 2007; 7th Street Bridge Rehabilitation; Port Huron, MI; Final Geotechnical Investigation Report

HNTB Corporation; February 16, 2007; 7th Street over the Black River; Port Huron, MI; Final Plans for the Proposed Bridge Rehabilitation