# RISK MITIGATION STRATEGIES FOR CUT AND COVER TUNNELS AT THE MITCHELL INTERCHANGE

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As part of the I-94 North-South Corridor reconstruction program, the Wisconsin Department of Transportation (department) is reconstructing the Mitchell Interchange in Milwaukee, Wisconsin. Located on the south side of Milwaukee near Mitchell International Airport, The Mitchell Interchange interconnects I-94, I-43 and I-894. Reconstruction began in 2009 with the letting of the first advance contract for local crossroad bridge reconstruction. A second advance contract for construction of collector-distributer roads on each side of the I-94 mainline was completed in 2010. The main contract involving reconstruction of the core of the Mitchell Interchange was let in the summer of 2010, and is scheduled to be completed by the end of 2012. The core contract includes construction of three cut and cover tunnels along two system interchange ramps. Construction of the three tunnel structures is on the critical construction path. The contract includes an interim contract completion date that requires the tunnels to be open to traffic by the end of 2011.

A top down method of construction using drilled shaft secant piles for the sidewalls of the cut and cover tunnel structures was selected as the preferred structure type to address subsurface and construction schedule challenges. The department identified in pre bid coordination that no cost reduction incentives would be reviewed or accepted for changes in tunnel wall type. A major component of the tunnel structures requires the installation of 1,500, 4-foot diameter secant piles. This paper presents mitigation strategies to manage construction risks associated with the installation of the secant piles including; prequalification of the secant pile subcontractors, requirement for use of full depth temporary casing, a site specific quality control plan, specifying drilling equipment, and developing a geotechnical baseline report for this critical component of the Mitchell Interchange reconstruction project.

## GENERAL STRUCTURE DESCRIPTION

The three cut and cover tunnel structures are located in the core of the reconstructed Mitchell Interchange and are labeled tunnels 1, 2, and 3 in Figure 1. Tunnels 1 and 2 carry 2 lanes of traffic with a total clear span of 55 feet underneath 8 separate roadways. Tunnel 3 carries three lanes of traffic with a clear span of 65 feet underneath 4 separate roadways. The longitudinal length of Tunnels 1, 2, and 3 are 585 feet, 744 feet, and 650 feet, respectively.

All three tunnel structures consist of two opposing walls constructed in a top down manner from a series of 47.25 inch (1.2 meter) diameter secant pile drilled shafts installed on 3.5-footspacing.

The tunnels sidewalls are designed to resist lateral loads in a cantilever condition for the initial 15 foot excavation of the tunnel interior. The roof is comprised of transverse prestressed I-beams with a cast-in-place topping slab similar to conventional prestressed girder construction. After placement, the roof acts as a transverse bracing strut at the top of the tunnel sidewalls allowing the excavation to continue to a maximum exposed height of approximately 20 feet. Tunnels 1 and 2 also include a reinforced tunnel invert slab which serves as a lower level compression strut as well as the roadway in these tunnels. Tunnel 3, the shallowest of the tunnels did not require such a slab.

The secant pile walls in the tunnel interior are finished with shotcrete and porcelain ceramic tile. The tunnel top slab received a hot mix asphalt rubberized water proofing membrane prior to being backfilled to finished grade above the top of the tunnel. The tunnels are completed and ready for traffic after tunnel interior roadway, emergency lighting, and a firefighting standpipe are placed.

## SUBSURFACE CONDITIONS

The soils in the project area were deposited approximately 13,000 to 14,500 years ago, and are referred to as part of the “Oak Creek” formation. The Oak Creek formation typically consists of a fine-grained silt and clay till with a variable mixture of sand, gravel, cobbles, and boulders. Interbedded lacustrine, outwash, and ice-margin deposits are also common in the Oak Creek formation. The area is underlain by gently dipping sedimentary bedrock which generally consists of dolomite, with occasional shale seams.

Groundwater elevations varied from just below tunnel invert slabs to approximately 20 feet above the three tunnel inverts. The ground water table was monitored for a period of nearly 18 months. To further assess the characteristics of soil transmisivity, dewatering challenges, and risks associated with ground water seepage during construction, a ground water pump test program was conducted.

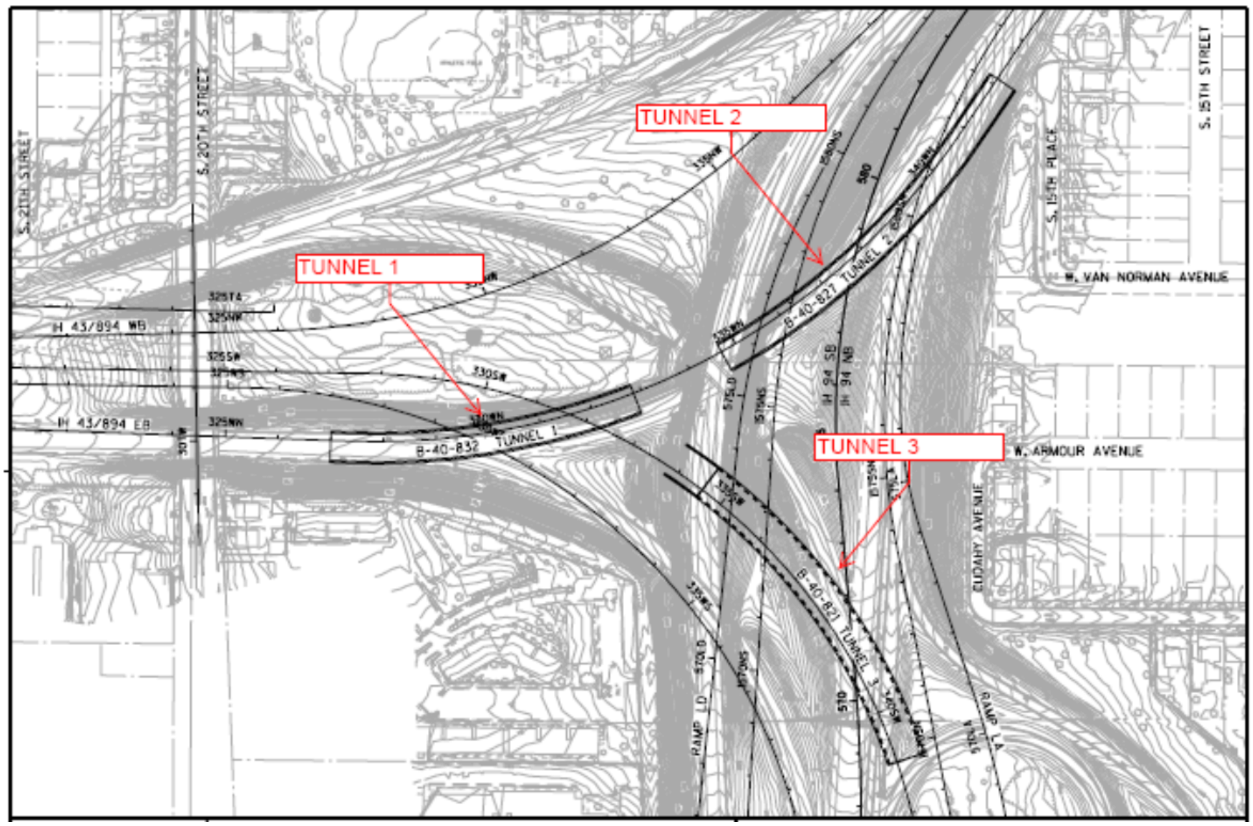


**FIGURE 1 – TUNNEL LAYOUT**

A rigorous subsurface exploration program was considered paramount to mitigate project risk. The program consisted of 38 boreholes along the alignments or in close proximity to the three tunnels structures, piezocones, and a groundwater pump test program to characterize the ground and ground water conditions.

The pump test consisted of nine 4-inch diameter extraction wells spaced at 60 foot increments, six observation wells and ten piezometers. The total amount of water pumped from the extraction wells was nearly 2 million gallons. The pump test resulted in the following major conclusions which were utilized during the design and preconstruction phases:

**Saturated, layered soil seams:** The pump test confirmed the presence of saturated seams and layers of granular material within cohesive units along the tunnel alignment, within the proposed depth of excavation. The extent of the saturated granular seams/layers and interconnectedness between them are erratic. Moreover, dewatering of all these layers/seams through dewatering of the underlying sand and gravel layer would be unrealistic. The dewatering concern resulted in the proposed side wall design to focus on water cut-off type walls.

The hydraulic conductivity coefficients of the different soils encountered at the site was established as a result of the pump test and is summarized in Table 1 below**.**

**Table 1 - Hydraulic Conductivity Summary**

| Soil Layer | Hydraulic Conductivity Coefficients (ft/day) |
|---|---|
| Glacial Till | $2x10^{-5}$ to $5x10^{-3}$ |
| Layers of Fine Sand, Sandy Silt and Glacial Outwash | 0.25 to 17 |
| Lower Glacial Outwash | 55-150 |

## STRUCTURE TYPE SELECTION

Original functional plans developed for the system interchange included construction of 7 skewed bridge structures – including significant volumes of roadway excavation. Design value engineering identified reduced costs associated with construction of three cut and cover tunnels.

During preliminary design, construction techniques and associated structure type alternatives that used either a “bottom-up” or “top-down” construction method were considered for cut and cover tunnel construction.

The “bottom-up” alternative consisted of excavating a trench section and building a conventional cast-in-place (CIP) CIP concrete box structure within the trench prior to backfilling of the trench. For this alternative the sides of the excavation would require temporary excavation support. Three temporary excavation bracing methods were evaluated structure type for this construction type: soil nail walls, sheet pile walls and a soldier pile and lagging walls.

Soil nail excavation bracing was eliminated due to risks associated with groundwater and saturated sand and gravel seams that would be encountered. Sheet pile walls were eliminated due to risks associated with anticipated installation difficulties of driving sheet piles in the presence of boulders, cobbles and hard glacial tills. Thus, the “bottom-up” construction alternative was evaluated assuming solider pile and timber lagging for the required temporary excavation support.

The primary advantage of the bottom-up construction alternative is that it would be possible to apply waterproofing and groundwater collection drains around the outer perimeter of the CIP tunnel structure, making it relatively easy to avoid ground water from seeping into the tunnel interior. The disadvantages of this alternative technique were long construction duration and high initial construction costs. The most significant disadvantage, this alternative required dewatering, which was considered very challenging if not infeasible based on results from the pump test program.

Top-down construction methods consisted of installing either a reinforced concrete slurry wall, secant pile wall, or tangent pile wall that would function as both the excavation bracing and be used for the permanent structural side walls of the tunnels.

The tangent pile wall option was the only “top-down” wall option considered that was not a water-cutoff type wall. Initial construction cost for the tangent pile wall option was slightly less than the secant pile option. However, the risks of potential problems from groundwater seepage entering the excavation during the construction phase and concerns over the long term durability of the completed structure outweighed the modest initial cost savings. Thus, the tangent pile wall option was eliminated from further consideration. An advantage of the slurry trench wall was that it was comprised of larger wall segments which would have less construction joints compared to the secant pile wall option and therefore was likely to be more watertight compared to a secant pile wall. The disadvantages was that construction duration was longer for the reinforced concrete slurry wall and a higher construction cost

The Mitchell Interchange construction staging required that all three cut and cover tunnels to be completed in one year. Secant pile walls were selected as the recommended option for the “top-down” construction alternative based on lower initial construction cost, and shorter duration as compared to slurry trench walls. Constructing the cut and cover structure sidewalls using Secant Pile Drilled Shafts was determined to be the most effective alternative providing the “best value”.

**Underdrain System Required:** An underdrain system was recommended and detailed to be installed under the invert slabs. With an installed underdrain system under the invert slab and the small volume of seepage anticipated, it was not necessary to design the invert slab to resist hydrostatic uplift pressure.

The tip elevation of secant pile walls in the tunnels could generally be terminated at depth of approximately 45 feet to provide adequate ground water cut-off to minimize ground water seepage under the invert slabs.

## SECANT PILE DESIGN ANALYSIS

### Geotechnical Analysis

Since the project required the majority of the interchange remain open during construction, the project was broken up into several intermediate construction phases.

A complete geotechnical analysis was utilized to simulate the different construction phases along with the final condition of the tunnel prior to completion of the structural design as differences in performance were possible between the short term and final conditions.

Intermediate construction phases include temporary conditions where traffic is placed over the top of the tunnels prior to placement of structural slabs at the tunnel invert in Tunnels 1 and 2.

Finite element software Plaxis was used to simulate the construction phases and final conditions of the three tunnels.

A hardening soil model was used to model the soil elements. An undrained analysis was used to analyze each construction stage due to the shorter term durations between each stage. A drained analysis was also performed for a few select cases in order to evaluate the performance of the excavation support system under the long term and worst case condition.

For the final loading condition, a drained analysis was performed using drained soil strength parameters. Consolidation analysis was performed to simulate long-term consolidation allowing complete dissipation of excess pore pressures prior to changing the soil shear strength during the analysis, from undrained to drained parameters. Elastic material properties were used to model the walls and slabs with beam elements that have both axial and bending stiffness.

Analysis of the secant pile walls and CIP top slab included non-linear behavior due to stiffness reduction from concrete cracking. The use of elastic material properties, which may be stiffer than the actual condition, could result in higher forces in the structural members. However, this would result in a relatively conservative structural design.

The horizontal width of the model was selected to be 7 times the excavation width. The bottom dimension of the model was selected approximately 3 times the height of the excavation height. Standard Plaxis boundaries, in which both sides of the model are only allowed to move vertically while the bottom of the model is fixed, were used. These boundaries are considered reasonable for settlement analysis under static load.

### Structural Design

As structural elements, the design of the secant shafts was performed per the AASHTO Standard Specifications LFD design for the maximum force developed from the Plaxis analysis. Additional verification of design forces obtained from Plaxis were performed using the FB-Multiplier computer program.

Secant pile shafts are often designed with an alternating weak-strong shaft arrangement with every other shaft designed as a strong, structurally reinforced element with weak, lean concrete infill (lean) shafts between the structural shafts. This arrangement allows for easier secant pile shaft installation. The structural shafts are installed between the lean shafts of lower strength, which are easier to drill through and maintain alignment.

Early on in the design process it was determined that all shafts needed to be reinforced in order to be able to design the walls without tiebacks, while maintaining reasonable percentage of reinforcement. Adjacent shafts were designated as Sequence 1 and Sequence 2 shafts. Sequence 1 shafts have been installed first and Sequence 2 shafts have been installed in between Sequence 1 shafts after gaining enough strength to prevent damage.

Two reinforcement arrangements were detailed. A rectangular reinforcement cage was used for the Sequence 1 shafts and a round cage for the Sequence 2 shafts. The rectangular cages in the sequence 1 shafts allowed for more effective reinforcement arrangement, while maintaining clearance on the sides of the shafts for the sequence two shafts to be excavated without hitting the reinforcement. Shaft layout is shown in Figure 2. Every sixth shaft required cross hole sonic logging (CSL) tubes for quality assurance testing.

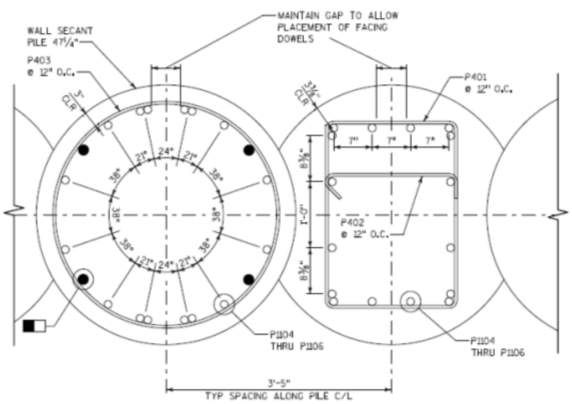


**FIGURE 2 - Typical Secant Pile Layout and Reinforcement**

## RISK MITIGATION

The construction of the core of the Mitchell Interchange is required to be completed in two years. To accomplish this, all three tunnels must be constructed simultaneously in the first year of the core construction contract (2011). The installation of the secant pile shafts were identified as a relatively large risk, with many subsequent elements of the Mitchell interchange dependant on the timely and successful installation of the secant pile shafts. Several mitigation strategies were employed and incorporated into the construction contract to address these risks:

**Subsurface Conditions' Risk:** To mitigate this risk the comprehensive subsurface exploration program described above was conducted conducted. The cost of these investigations were approximately 1.5% of the estimated construction cost of the three cut and cover tunnel structures. The subsurface information obtained was incorporated into the construction contract documents in the form of a geotechnical baseline report that defined an equal baseline subsurface condition for all bidders, mitigating risk for claims associated with differing site conditions.

**Secant Pile Installation Risk:** To mitigate this risk, the specification required that installation of drilled shafts for secant pile wall construction be accomplished using full length temporary casing advanced a minimum of 5-feet ahead of the shaft excavation to the design tip elevation of the shaft. Telescoping temporary casing and uncased boreholes were not permitted. Slurry was permitted in addition to the temporary casing.

To ensure that the full length casing would be reliably installed and extracted subsurface conditions at the site the specification required that a hydraulic casing oscillator be available onsite if needed to accomplish casing installation and extraction.

The contract required the successful contractor to install a non-production trial wall panel at the site, prior to beginning installation of the production shafts. Construction of the trial panel had to be completed in accordance with the contract specifications and the accepted drilled shaft installation plan. Construction of the trial panel was intended to confirm that the contractor's proposed methods, equipment and crews were capable of successfully installing the secant pile drilled shafts at the Mitchell Interchange site. The trial panel was tested using CSL to confirm the integrity of the shafts.

**Risk of water bearing sand and gravel:** Due to the presence of water bearing sand and gravel bearing layers that can be encountered at the bottom of the shafts, it was required to keep a positive head of water inside the casing in order to mitigate this risk.

**Unqualified Contractor's Risk:** To mitigate the potential risk of having inexperienced contractors attempt installation of the secant piles, a prequalification process for contractors or subcontractors who intended to perform the installation of drilled secant shafts for the cut and cover sidewalls was required. Prequalification was based on recent applicable experience on similar projects and having an adequate inventory of the specialized drilling equipment that could be committed to the project. Contractors seeking prequalification had to demonstrate relevant experience on similar projects along with the listing of qualified personnel available and committed for this project.

Four prequalified drilling subcontractors were identified in the contract bid documents and the contract required that only the listed prequalified subcontractors could perform the secant pile work.

**Inspection Risk:** Training on the installation of drilled shafts, was conducted for all on-site inspection staff.

## CONSTRUCTION

The overall risk mitigation strategy was passed from design to construction. The specified construction methods, along with extensive materials information provided in the project documents allowed the contractor to maintain a project schedule despite encountering challenging conditions which were both anticipated and unanticipated.

### Installation Overview

The installations of secant piles were performed in accordance with the prequalification requirements and bid documents. Large top drive drill rigs (Figure 3) with the ability to deliver high torque and crowd were able to install segmental temporary casing in advance of the excavation over the entire drilled shaft length. The lead casings were outfitted with carbide tipped drill teeth allowing the tip of the casing to act like a core barrel. The sectional casing was able advance through obstructions and concrete from previously installed adjacent drilled shafts when installing Sequence Two drilled shafts.

**FIGURE 3 – Top Drive Drill**

Placement of concrete was accomplished by the tremie concrete placement method. Both the tremie pipe and temporary casing remained embedded into the concrete throughout concrete placement.

### Installation Sequence

In order to prevent disturbance of recently installed drilled shafts a minimum center to center spacing of 3.5 times the shaft diameter (D) was maintained. This spacing was increased periodically as dictated by localized subsurface conditions. A sample sequence is shown in Figure 4 where the numbers represent which day a particular shaft was installed.

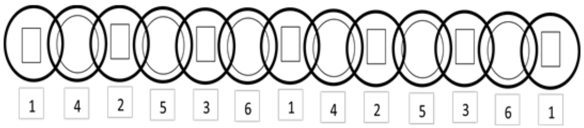


**FIGURE 4 – Sample Installation Sequence**

An area of Tunnel 1 had a large section of lower outwash unit that was encountered towards the bottom of the drilled shafts. Despite efforts to increase the spacing beyond 3.5D there were several occurrences of shafts “communicating”, between each other during concrete placement, which means concrete being placed in one shaft will flow into a nearby shaft being excavated Concrete would displace soil at the shaft tip or sidewalls and migrate at times more than 20’ and appear in the excavation of nearby drilled shafts.

Since a large head of concrete was being maintained within the temporary casing, the concrete level was able to drop until equilibrium was established without movement of groundwater or soil into the fluid concrete column. This communication of concrete occurred on shafts that were CSL tested, the results of which confirmed there was no adverse effect on shaft quality.

Rapid drill fluid losses were also observed in this soil despite advancing the casing ahead of the excavation. At times it was not possible to maintain a fluid head of more than 10 feet within the casing.

While the concrete communication was carefully observed, the prescribed construction method allowed for drilled shafts to be installed into this material without compromising the project schedule.

The behavior of the soils however, was of enough concern to warrant the installation of an Osterberg load cell in order to determine the end bearing and skin friction capacities in this outwash material. The results of the Osterberg load cell confirmed the design assumptions for this material and no modification to the drilled shafts were required. Department staff has offered that Osterberg load cell testing in advance of construction or in conjunction with test panel construction will be a consideration on future projects of similar magnitude.

**<u>Concrete Mix Effect on Construction</u>**

The concrete mix was required to maintain a high slump for an extended period of time to ensure that casing extraction would be possible even during unanticipated durations of concrete placement (Brown and Turner, 2010). Additionally slowed or delayed concrete cure was preferable in order to maintain production rates when installing Sequence 2 shafts.

The drilled shaft specification required daily concrete testing to be performed by the contractor. Concrete cylinder sets were taken once for every 100 CY of concrete placed. Testing of these samples was originally being performed at 7 and 28 day intervals. Over the first 2 months of the project the 7 day strength showed variability of around 2000 psi. Despite the high variability, the low end concrete strength was strong enough to withstand the drilling operation while the higher end strength was still drillable with the equipment utilized on the site.

A concern exists when drilling a Sequence 2 shaft through two Sequence 1 Shafts with uneven strengths as the drill will typically drift toward the weaker shaft. The use of the specified full length casing along with required guide wall alleviated much of the concern of drifting as the temporary casing can be constantly monitored for verticality and adjustments can be made by pushing against the guide wall when necessary. A photo of guide wall is shown in Figure 5.

**FIGURE 5 – Completed Guide wall Section**

After approximately 2 months of working on the project, several consecutive sets of 7 day strength data showed that the concrete was not having an initial strength gain within 7 days.

As detailed above, a slower strength gain of the concrete is ideal to allow for easier installation of the Sequence 2 shafts. However, a certain amount of concrete strength is required in order for the Sequence 1 shafts to maintain their integrity during installation of the Sequence 2 shafts.

A few days prior to seeing the first test data showing low 7 day strengths, the drill operator noticed that the concrete in Sequence 1 shafts appeared not to have set when attempting to start installation of Sequence 2 shafts. An adjustment in spacing and continuance of Sequence 1 shafts was implemented in order to allow for a longer cure time prior to installation of further Sequence 2 shafts.

No changes to the concrete mix coincident with the changes to the concrete strength gain. This change in concrete strength gain did happen during the middle of winter where high temperatures sometimes did not exceed single digits. A review of air temperatures and concrete temperatures at time of placement was completed and showed that the ambient temperature did not appear to be the cause of the slower concrete strength gain.

The concrete mix design was revised several times throughout the project in order to obtain more consistent results. The different mixes used had highly variable short term strengths (3 to 7 days). It was suspected that the variability had to do with the wide range of admixtures used in each mix design. The 28 day strength results were consistently over the specified 28 day strengths.

Although the inconsistent concrete strengths have never been fully diagnosed, it is suspected that variances within control limits for the admixtures, fly ash, cement and aggregate may have been the cause.

Since a change to aggregate or cement sources would be a lengthy event, it was determined that changes to the admixture would be the prudent alternative. Trial batches were conducted on three alternate mix designs where only the admixtures and their proportions were modified.

Concurrent with using the revised mix designs, the testing frequency was doubled and maturity probes were placed in several drilled shafts to give an estimate of the in-situ concrete strength. While the early strength data was still highly variable, the admixture change did produce a mix that was at least strong enough to withstand drilling of Sequence 2 shafts within a maximum of 5 days and typically within 3 days and compressive strength between 300 and 2500 psi. Sample concrete maturity curves are presented in Figure 6.

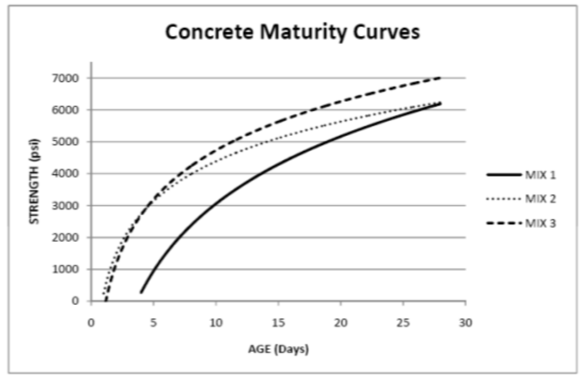


**FIGURE 6 Sample Strength Gain Curves**

The requirement for contractor conducted material testing along with an experienced project staff allowed for a quick identification of the slow setting concrete. Sequencing was able to be modified to allow the concrete to gain strength in order to prevent damage to previously installed drilled shafts.

## CONCLUSION

The following are the major conclusions of the risk mitigation efforts undertaken on the project.

1. Results from a rigorous subsurface exploration program, incorporated into contract documents can be considered an effective risk mitigation tool.
2. Identification of risks and mitigation measures should be developed throughout planning, investigation, design and construction.
3. A concept such as “best value” that consider cost, schedule, and risk is an appropriate tool for large and mega projects.

4. Prequalifying specialty contractors who can demonstrate past successful projects while utilizing state-of-the-art equipment and qualified personnel can offer risk mitigation on projects with both aggressive schedule and high quality requirements.
5. Extensive information passed on in the project documents allowed a well experienced contractor to be properly equipped in order to easily navigate both foreseen and unforeseen challenges encountered during construction.
6. The required quality control plan provided the project staff with up to date information which allowed for adjustments in construction methods along with the ability to implement appropriate mitigation plans. This allowed construction to continue unimpeded when conditions varied from what was anticipated

An informed and collaborative owner, Wisconsin Department of Transportation and Federal Highway Administration, were essential to the success of this risk based design and construction approach, as they correctly understood the value of adopting this strategy and without their support, this was not possible.

## REFERENCE

BROWN and TURNER 2010. Drilled shafts: Construction procedures and LRFD Design Methods. U.S. Department of Transportation Federal Highway Administration Publication No. FHWA-NHI-10-016 FHWA GEC 010, pp. 175-176.