M-222 Slope Stabilization Case History – Geotechnical Lessons Learned from Michigan Department of Transportation Design Build Project

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M-222 SLOPE STABILIZATION CASE HISTORY – GEOTECHNICAL LESSONS LEARNED FROM MICHIGAN DEPARTMENT OF TRANSPORTATION DESIGN BUILD PROJECT

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ABSTRACT

In 2009, the Michigan Department of Transportation (MDOT) became concerned about ongoing slope movements adjacent to a segment of M-222 located on outside bend of the Kalamazoo River in the City of Allegan, Michigan. Over the next couple years, continued river erosion and seasonally wet springs caused 8- to 10-foot high scarps adjacent to M-222, condemnation of a home, and several large block slides into the river. In the early spring of 2011, MDOT secured their first Construction Manager/General Contractor (CMGC) delivery method contract to protect M-222 and repair the slope. Improvements included constructing an up to 26-foot tall retaining wall, re-grading the roughly 70-foot high slope, and armoring the toe of slope. The improvements used were selected based on assessed risks and mobility requirements. Construction of the project began in July of 2011 and was completed in spring of 2012. A history of the slope instability progression using aerial photography, selection and design of the improvements, and resulting construction challenges are discussed. The authors conclusions on geotechnical lessons learned are shared.

INTRODUCTION

In 2009, the Michigan Department of Transportation (MDOT) became concerned about ongoing slope movements located adjacent to a segment of a Michigan state highway, M-222, on an outside bend of the Kalamazoo River in the City of Allegan, Michigan. Translational slides caused an over-steepened M-222 foreslope condition along a portion of the two-lane roadway. The translational slides extended beyond the M-222 66-foot right-of-way onto City of Allegan and private property and then down to the Kalamazoo River. This reach of the Kalamazoo River is designated a Superfund Site which presented challenges with dredging and spoil disposal during the project.

Over the next two years, continued river erosion combined with seasonally wet springs resulted in numerous translational slides, additional scarps adjacent to M-222, cracking and translation of portions of the M-222 shoulder and eastbound travel lane, and condemnation of a home. In the early spring of 2011, MDOT secured their first Construction Manager/General Contractor (CM/GC) delivery method contract to repair and protect M-222 and correct the issues creating the slope movements. MDOT selected the CM/GC delivery mechanism to allow for concurrent design and constructability review, to combine design and construction expertise, and accelerate design and construction. MDOT selected their design and CM/GC teams (project team) to match design with construction strengths needed for this project. The design team was lead by URS Corporation (URS) as the prime consultant and included Soil and Materials Engineers, Inc. (SME) as the geotechnical consultant. The CM/GC was lead by Millbocker & Sons, Inc. (Millbocker) as the prime contractor and included Nicholson Construction Company (Nicholson) as a specialty sub-contractor for constructing the retaining wall and ground anchors.

Shortly after starting design in April of 2011, MDOT assigned the project an emergency status that further accelerated the design phase. This paper focuses on the geotechnical design and construction aspects of the project and presents geotechnical lessons learned.

SLOPE INSTABILITY PROGRESSION

The project site is located on an outside bend of the Kalamazoo River. Figure 1 shows the location of the project site. Scour along the outside bends of rivers has been well documented and results in continued erosion and movement of the river bank. The design team used aerial photographs to review the rate of scour along the river bank. Figure 2 depicts the progression of river scour at the project site over a period extending from 1999 to 2009. The progressive toe cutting caused by river scour resulted in translational slides of the adjacent slope. Similar slides are visible along other outside bends of the Kalamazoo River in the City of Allegan as also shown in Figure 2. The design team determined the design river velocity was approximately 13 feet/second (ft/sec) in the scour zone.

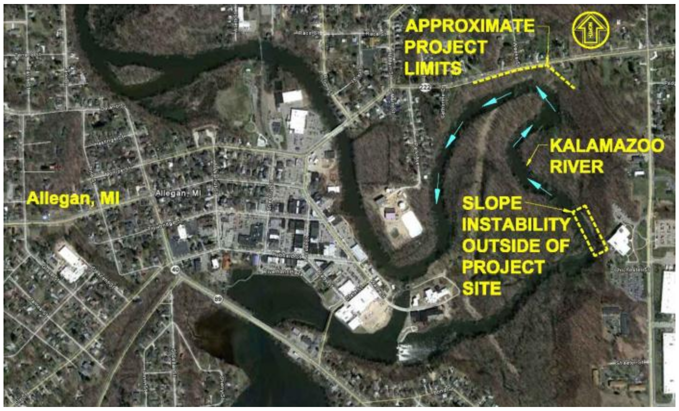

Fig. 1. Site Location Map

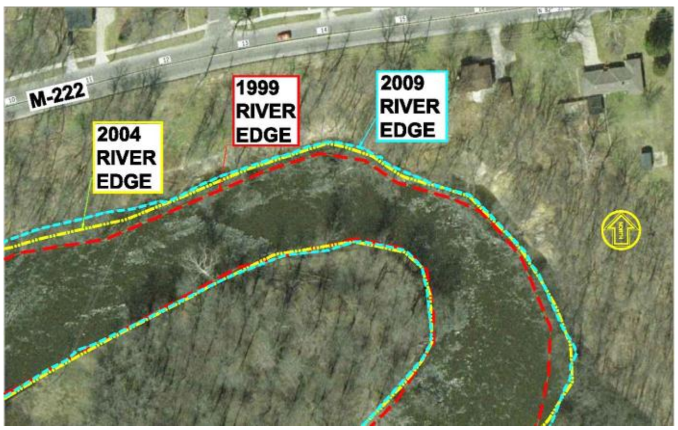

Fig. 2. Aerial View of Scour Progression

In 2009, translational slides resulted in a scarp forming adjacent to M-222 as shown in Figure 3. MDOT periodically observed changes in slope conditions until 2010 when MDOT retained SME to visually observe and photograph existing slope conditions on a weekly basis. MDOT also barricaded off the east-bound M-222 shoulder. Groundwater seepage and soil erosion from piping were observed in areas of the translational slides.

Over the next 12 months while MDOT secured right-of-way and project funding for a larger and longer-term stabilization project, continued river erosion combined with seasonally wet springs resulted in numerous additional and larger translational slides. The translational slide slip surfaces appeared to generate at a maximum vertical depth of about 10 feet below the original (pre-slide) slope face. The larger translational slides resulted in the loss of a portion of the lawn of two residences adjacent to the failures and structural damage to one of the residences. The City of Allegan condemned and then demolished the residence with structural damage.

Fig. 3. Scarp Adjacent to M-222 (2009)

In 2011, heavy rain events lead to higher river and ground water levels, which rapidly accelerated the rate and extent of slope instability and worsened the scarp adjacent to M-222. In May 2011, MDOT declared the M-222 Slope Stabilization project an emergency and closed the entire M-222 roadway through the project site. Figures 4 through 7 illustrate the progression and extent of slope instability.

Fig. 4. Scarp Adjacent to M-222 (2011)

Fig. 5. Example of Translational Slide (2011)

Fig. 6. Translational Slide - Condemned Residence (2011)

Fig. 7. Aerial View of Slope Instability (2011)

SUBSURFACE CONDITIONS

Eleven soil borings were drilled to explore the subsurface conditions. The soil borings were drilled to depths ranging from 5 to 80 feet. The approximate locations of the soil borings are shown in Figure 8.

Routine laboratory testing on the soil samples included visual engineering classification, moisture content determination on clays, and unconfined compressive strength estimated by hand penetrometer tests on clays. Additional laboratory tests included grain size determinations on soil samples from the anticipated river scour zone, a consolidated-undrained (CU) triaxial test with pore water pressure measurements, dry density determinations, Atterberg limit tests and specific gravity tests.

Geotechnical data collected from the test holes was used to develop a generalized soil profile and geotechnical conditions for the project. The generalized soil profile and mean values of select geotechnical index properties are shown in Figure 9 and Table 1.

The generalized geotechnical conditions and soil profile identify one approximately 45-foot thick, silty clay layer. This clay layer represents the average of an upper and lower clay stratum. The upper clay stratum was approximately 25 to 30 feet thick. The lower clay stratum was approximately 15 to 20 feet thick. Mean values of the measured index properties for the upper and lower clay strata are presented in Table 2 for information.

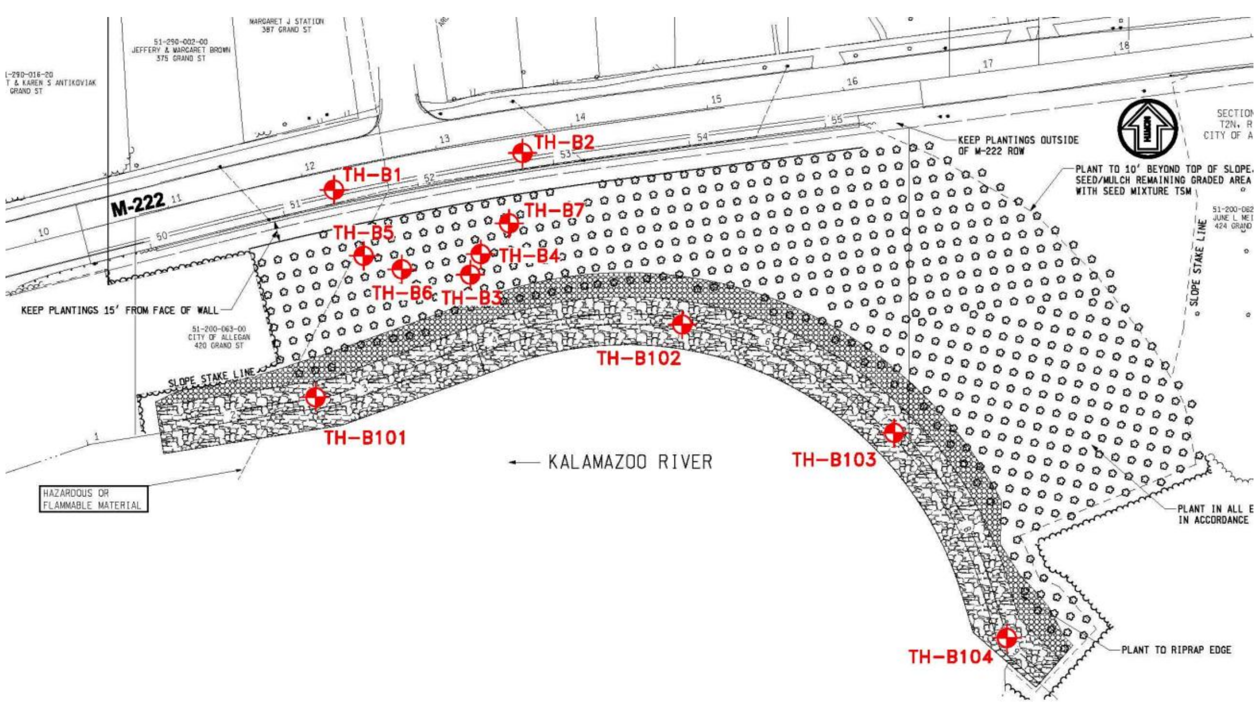

Fig. 8. Soil Boring Location Diagram

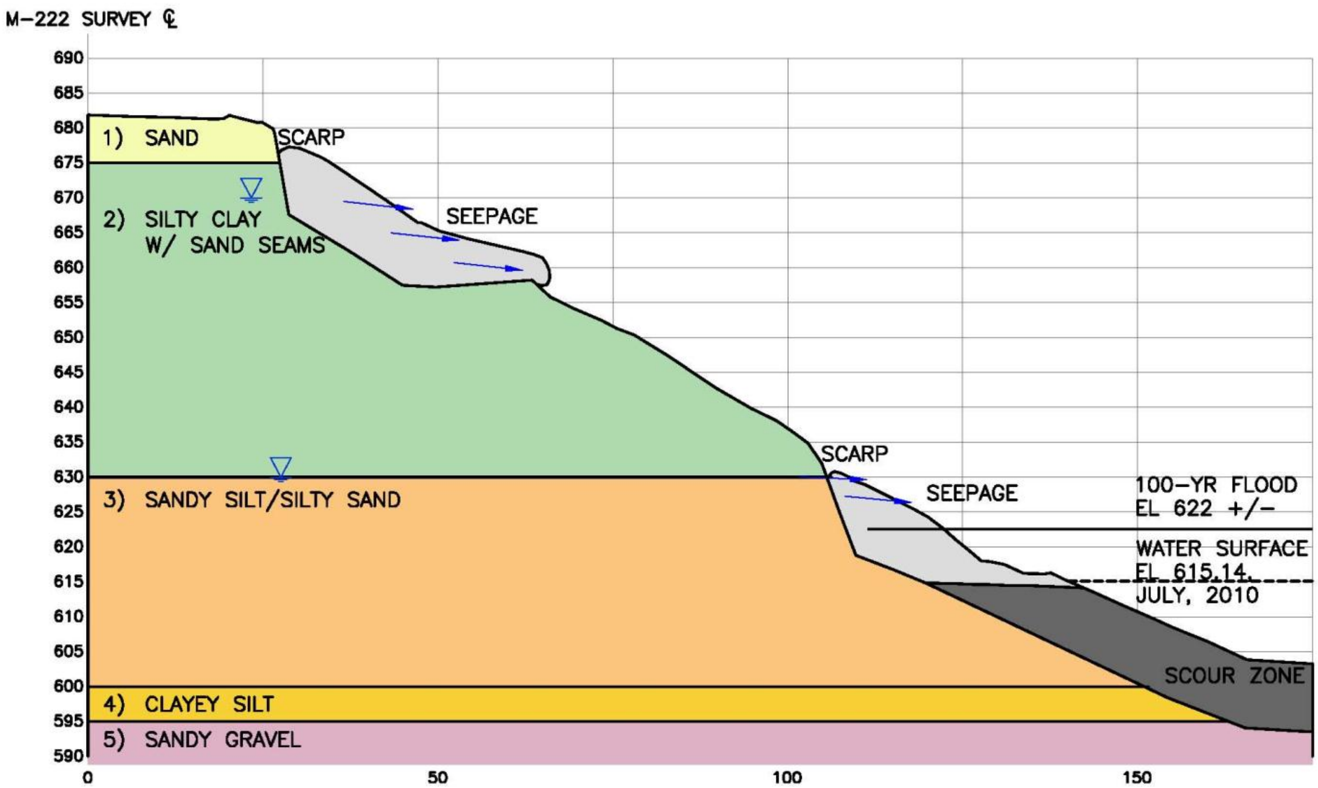

Fig. 9. Generalized Soil Profile

Table 1. Generalized Soil Profile

Strata No.	Soil Description	N-value (bpf)	γ_t (pcf)	ϕ (deg)	c (psf)	ϕ' (deg)	c' (psf)
1	Sand Fill	5	115	30	0	30	0
2	Silty Clay	12	135	0	2,000	31	100
3	Sandy Silt/Silty Sand	22	125	33	0	33	0
4	Clayey Silt	21	125	0	1,500	31	100
5	Sandy Gravel	41	130	38	0	38	0

Table 2. Index Properties of Upper and Lower Clay Stratum

Strata No.	Soil Description	N-value (bpf)	γ_t (pcf)	w (%)	s_u (psf)	LL (%)	PL (%)	PI (%)
2A	Upper Clay	13	134	15	2,500	26	14	12
2B	Lower Clay	10	126	26	2,000	48	19	27

Groundwater level measurements collected from the monitoring wells suggested at least two phreatic surfaces existed within the slope. The upper phreatic surface appeared to be trapped within the clay profile above the level of the Kalamazoo River. The lower phreatic surface appeared to be located within the sands or silts and connected to the Kalamazoo River. A third phreatic surface also exists at the site and is perched near the surface sands located above the clays.

SELECTION OF IMPROVEMENTS

MDOT held a project kick-off meeting, brainstorming workshops, and design workshops with their project team to identify and develop solutions for design and construction challenges. During the kick-off meeting, the project team reviewed the following topics:

- existing right-of-way conditions,
- project limits,
- project schedule,
- project coordination and communication,
- existing utility locations and coordination,
- traffic control requirements,
- required minimum design life of 75-years,
- river hydraulics and geomorphology,
- geotechnical conditions,
- environmental conditions, and
- preliminary stabilization concepts.

Preliminary stabilization concepts required armoring the toe of slope to protect against continued river erosion, controlling groundwater seepage through the slope and stabilizing the failing slope above the toe with slope protection. At the request of the CM/GC, the project limits were expanded to the opposite shore of the river to allow temporary construction access along an existing Consumers Energy easement. This construction access proved critical to accelerate construction by allowing delivery and storage of materials at a location other than along the M-222 right-of-way.

During the first brainstorming session, the project team reviewed existing river and slope geometry conditions relative to the location of M-222, geotechnical conditions, on-going and accelerating slope instability, and construction access limitations to brainstorm stabilization options and confirm right-of-way requirements. Since armoring the toe was essential, brainstorming options focused first on slope stabilization methods and then on toe protection.

Stabilizing the existing slope at its continually changing slope inclination (e.g. by installing soil nails, anchor slabs, etc.) was quickly dismissed due to the high levels of design and construction risk. Reinforced soil slopes were also quickly dismissed for similar reasons. As a result, slope stabilization discussions proceeded to using a wall system to allow grading of the slope to a flatter slope inclination. Terraced wall systems were dismissed due to mid-slope construction challenges. The project team agreed to design and construct a wall at the top of the slope (top wall) adjacent to M-222 to allow grading of the slope to a flatter slope inclination. A mechanically stabilized earth wall system was dismissed due to utility conflicts within the existing M-222 right-of-way. Similarly a soil nail wall was dismissed. The project team agreed that top-down wall construction methods with drilled wall elements meet the project constraints and balanced construction risks. Soldier pile and lagging, tangent pile and secant pile walls were reviewed. Continuous Flight Auger (CFA) pile walls were initially considered but dismissed based on the lack of published and FHWA accepted durability and life-cycle cost studies on permanent CFA walls. MDOT selected the soldier pile and lagging wall with a permanent cast-in-place concrete (CIPC) facing system. Precast concrete lagging was not used since the emergency status of the project did not provide the lead time required for precast products.

Discussions related to armoring the toe of slope focused immediately on hard armor solutions to protect against scour resulting from the design river velocity of approximately 13 ft/sec. An open cell steel sheet piling was initially preferred by the CM/GC, but proved to be cost prohibitive based on the sizes of the cells required to support the 70-foot high slope. Riprap, precast concrete mats and gabion filled baskets were also considered to armor the toe of slope. MDOT selected the riprap (revetment) option despite the construction risks associated with excavation and disposal costs of contaminated sediments from the Kalamazoo River.

MDOT and their project team agreed to stabilize the slope by installing a retaining wall at the top of slope, grading the slope to a flatter slope inclination, and installing revetment at toe of slope as shown in Figure 10.

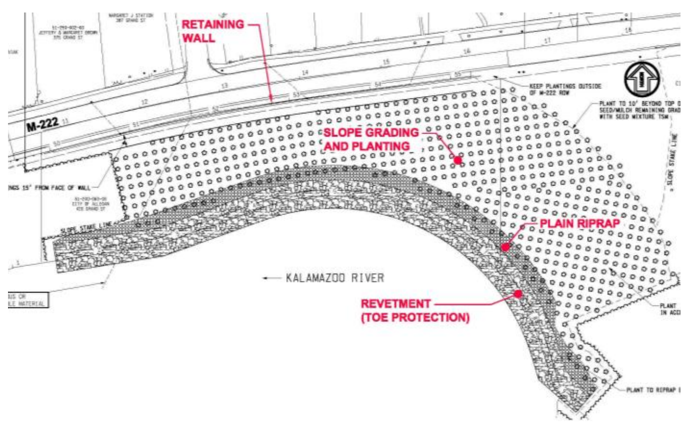

Fig. 10. Slope Stabilization Features

REVETMENT DESIGN

The Michigan Department of Environmental Quality (MDEQ), which is in-part responsible for permitting work performed with the river, generally requires a compensation cut to install revetment and limit backwater increases to 1-foot or less. MDOT was able to successfully negotiate a "previous condition" based on the 1999 river's edge contour shown in Figure 2. This successful negotiation reduced the retained height of the retaining wall at the top of slope required to grade the slope at a flatter slope inclination. MDOT was also successful in negotiating placement of the riprap revetment a maximum of 15 feet beyond the 2011 river's edge. This successful negotiation reduced the amount of compensation cut required. Even with these successful negotiations, the amount of riprap revetment that could be placed below the 100-year flood plain elevation was limited.

URS designed the riprap revetment system following FHWA HEC No. 11, "Design of Riprap Revetment". The riprap revetment was designed to protect against a design maximum scour depth of 10 feet below the 100-yr (1% chance of occurrence) flood elevation of 620.5 feet. An approximate D_{50} size of 2 feet was selected for the quarried limestone riprap revetment. A maximum slope inclination of 1-vertical to 1.5-horizontal (1V:1.5H) was used for the riprap revetment. The resulting average diameter of the riprap revetment required development of a project specific special provision (Heavy Riprap, Special).

The limits of the revetment system were determined based on the requirements of FHWA HEC No. 11 as shown in Figure 11. URS and SME recommended and MDOT selected a launched riprap revetment option to reduce construction risks associated with cost to excavation and disposal of river sediments. The launched revetment geometry was generally based on the requirements of FHWA HEC No. 11 as shown in Figure 12, with one exception. The exception included a modified launching system. The modified launching system was developed to limit the excavation of river sediments and provide a greater launching storage volume above the ordinary water surface as shown in Figure 13.

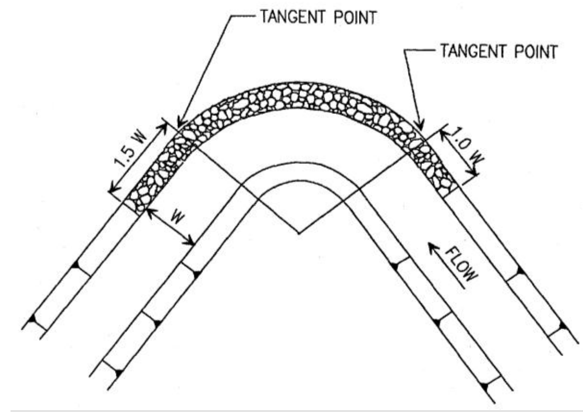

Fig. 11. Revetment Limits (FHWA, 1989)

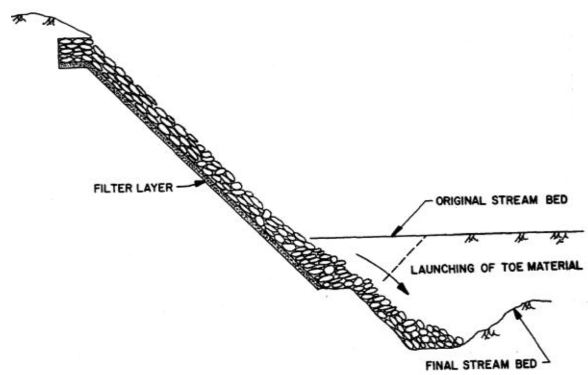

Fig. 12. Launched Revetment System (FHWA, 1989)

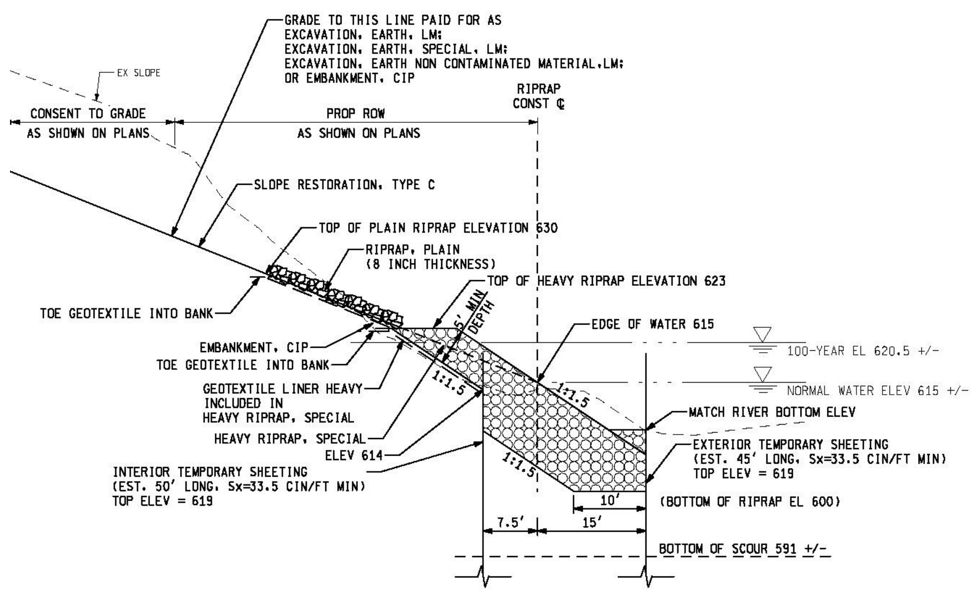

Fig. 13. Launched Revetment Detail

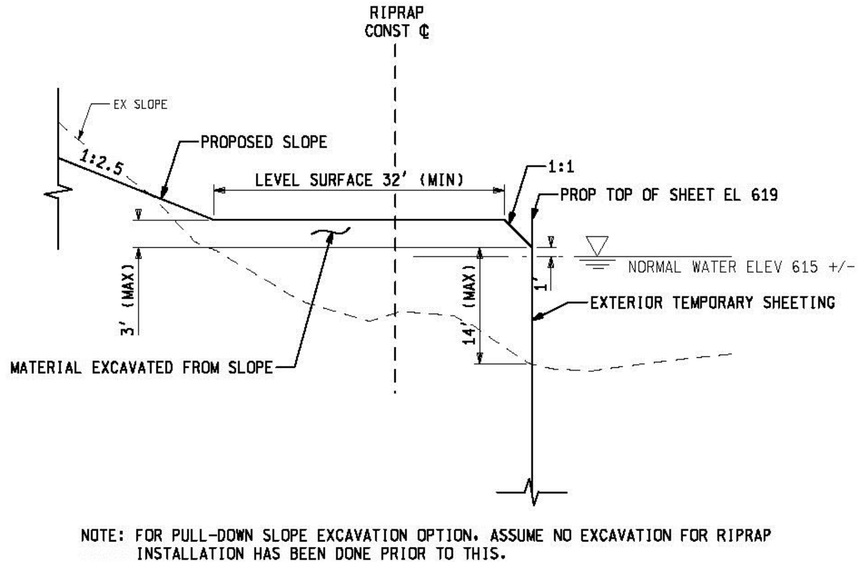

Fig. 14. Stage 1 Exterior Revetment Sheeting

The U.S. Environmental Protection Agency (EPA) required a sediment mitigation plan to install the riprap. The temporary steel sheet piling required to install the riprap revetment served to mitigate sediment transport. The temporary steel sheet piling included an interior and exterior sheeting line as shown in Figure 13. The CM/GC proposed installing steel sheet piling transverse to the interior and exterior sheeting drive lines to create individual cells and allow a staged construction of the riprap revetment. SME designed the temporary sheeting required for the U.S. EPA sedimentation mitigation plan based on the CM/GC requirements. The design of the exterior sheeting included a Stage 1 configuration to allow the CM/GC to remove soil during grading of the slope down to the river's edge as shown in Figure 14.

The revetment design also included MDOT plain riprap above the riprap revetment system. The purpose of the MDOT plain riprap was to prevent erosion of the silty sand and sandy silt (Stratum 3) shown in Figure 9. The design intent was to extend the MDOT plain riprap 2 vertical feet above Stratum 3 shown in Figure 9. As indicated in Figure 13, the construction drawings identified a top elevation for the MDOT plain riprap of 630 feet.

RETAINING WALL DESIGN

SME completed the retaining wall design with the exceptions that URS performed the structural design of the cast-in-place concrete (CIPC) facing and concrete barrier connected to the top of the retaining wall. Long lead time items (soldier piles and ground anchors) were sized early in the design phase to allow for early procurement, fabrication and delivery to meet the accelerated project schedule.

The retaining wall was designed following the 5th Edition of the AASHTO LRFD Bridge Design Specification, with 2010 Interim Revisions (AASHTO LRFD, 2010), and MDOT project specific design requirements. MDOT project specific design requirements for the permanent retaining wall included a design life of 75-years, a design traffic (normal operation) uniform live load surcharge of 360 psf behind the retaining wall, and a design impact force (extreme event) of 45 kips acting against the barrier connected to the top of the retaining wall. In addition, The CM/GC team required the retaining wall to resist a design construction, uniform surcharge live load of 600 psf directly behind the retaining wall.

Non-gravity retaining walls are generally designed based on Strength I, Service I, and (as with this project) Extreme Event II limit states. Both the Strength I and Service I limit states account for load combinations under normal conditions. The Extreme Event II limit state accounted for the MDOT required vehicle impact live load acting on the barrier. The Strength I and Service I limit states were evaluated based on both shorter-term total stress (undrained) soil shear strength and longer-term effective stress (drained) soil shear strength parameters. The Strength I limit state, using effective stress soil shear strength parameters, controlled the design of the retaining wall elements. The Extreme Event II limit state condition was evaluated based on total stress soil shear strength parameters that would result after a sudden vehicle impact and did not control the design of the retaining wall.

The major design steps referenced in Table 6.3.2 of FHWA "LRFD for Highway Bridge Substructures and Earth Retaining Structures – Reference Manual" (FHWA, 2007) were followed to complete the retaining wall design. Figure 15 shows the typical retaining wall section used. The following subsections discuss several of the major design steps followed to complete the retaining wall design.

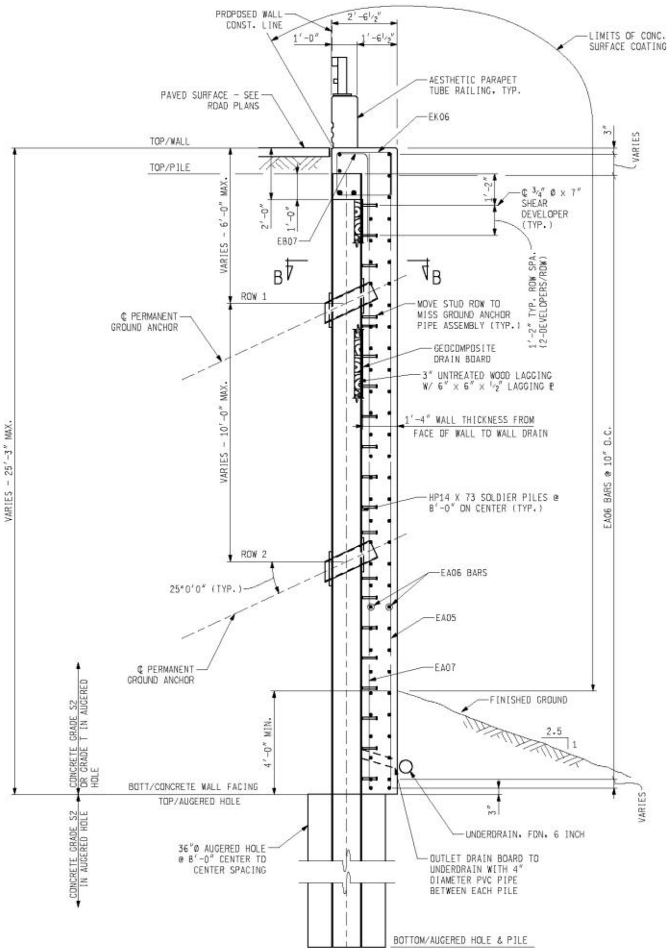

Fig. 15. Typical Retaining Wall Section

Lateral Pressure Distributions

For the predominately clay soils retained, lateral earth pressure diagrams calculated based on effective stress shear strength parameters controlled the design of the retaining wall. AASHTO LRFD (2010) lateral earth pressure diagrams were used to design the three wall types used. The three wall types included a cantilever wall with a maximum design height of 8 feet (Type I Wall), a one-level anchored wall with a maximum design height of 18 feet (Type II Wall), and a two-level anchored wall with a maximum design height of 26 feet (Type III Wall). The design heights of the retaining wall extended to the bottom of the CIPC facing. Since provisions for drainage of water from behind the wall were provided, unbalanced hydrostatic (water) pressures were not included. Lateral pressures resulting from traffic surcharge loads were also included.

Active lateral earth pressure coefficients were calculated for a vertical wall and level ground conditions based on AASHTO LRFD (2010). Passive lateral earth pressure coefficients were calculated for a vertical wall with a sloping ground condition (in front of the wall) at an angle of -21.8 deg (or 2.5H:1V) based on AASHTO LRFD (2010) Figure 3.11-5.4-2. Friction between the wall and retained soils was neglected (i.e. δ assumed to be 0 deg.). For the cantilever wall type, the earth pressure diagram shown in Figure 3.11.5.6-1 AASHTO (2010) was used. For the anchored wall conditions, the apparent earth pressure (AEP) diagrams shown in Figure 3.11.5.7.1-1 AASHTO (2010) were used. Traffic surcharge loads were transferred to a uniform lateral earth pressure based on the active earth pressure coefficient.

Soldier Piles

Nicholson selected a soldier pile spacing of 8 feet on center and a predrilled diameter of 3 feet. A pile spacing equal to 8 feet (approximately 2.66 times the soldier pile predrilled diameter) was used to calculate minimum required soldier pile embedment depths. Minimum required embedment depths of 25 feet, 15 feet and 19 feet were used for the Type I, Type II and Type III walls respectively. The soldier piles developed adequate vertical capacity within the embedment length determined for stability due to gravity and vertical ground anchor loads. HP14x73 AASHTO M270 (Gr. 50) structural steel shapes were specified for each soldier pile. The Type I cantilever wall controlled the soldier pile design for bending. Eight-inch diameter schedule 40 pipe sections were used to construct anchor pockets.

The anchor pocket diameter was selected by the CM/GC based on the ground anchor design discussed later. Nicholson elected to prefabricate the anchor pockets for each ground anchor at a fabrication shop prior to delivering the structural steel shapes to the project site.

Ground Anchors

A ground anchor inclination of 25 degrees down from the horizontal was selected by the CM/GC to avoid potential conflicts with existing utilities within the M-222 right-of-way. For the Type II and Type III walls, factored design loads (FDL) of 134 kips and 139 kips were calculated for the level 1 (upper) and level 2 (lower) ground anchors, respectively. For this fast-tracked project, all ground anchors were specified to provide a FDL of 140 kips. Minimum unbounded lengths of 29 feet and 15 feet were specified for the level 1 and level 2 ground anchors, respectively, to position the bonded zones adequately beyond the potential active zone failure plans behind the wall.

The ground anchors were designed by Nicholson as pressure grouted anchors with a bond diameter of 6 inches. Ground anchor bond lengths were sized using an ultimate (nominal) unit bond stress of 15 psi (2.16 ksf). Since each anchor was at least proof tested, a resistance factor of 1.0 was applied to the nominal unit bond stress. The nominal unit bond stress, selected based on Nicholson's experience, fell within the range of presumptive values for very stiff clay with medium plasticity presented in Table C11.9.4.2.1 AASHTO LRFD (2010) and Table 6.2 from the Post-Tensioning Institute, "Recommendations for Prestressed Rock and Soil Anchors" (PTI, 2004), but approached the calculated mean shear strength of the clay soils within the ground anchor bonded zone. Nicholson planned to post-grout the anchor bond zone as needed to achieve a nominal unit bond stress of 15 psi. ASTM A416 (Gr. 270) high strength steel strands were used as the tensile tendon in the ground anchors. Three strand anchors were selected based on the FDL of 140 kips and a strand factored tensile resistance of 46.87 kips. All ground anchors included Class I Corrosion Protection (for permanent applications) as shown in Figure 5.2c from PTI (2004).

Timber Lagging

Timber lagging was used to temporarily support the earth and surcharge lateral loads until the permanent CIPC concrete facing was constructed and achieved design strength. Three-inch thick by 8-inch wide timber lagging was installed behind the front flanges of the soldier piles, which corresponded to a 7-foot clear span length. The timber lagging thickness was selected using recommended values for competent soils presented in Table 12 from FHWA GEC No. 4 (FHWA, 1999) for SI units and Table 6.63.3b from FHWA "LRFD for Highway Bridge Substructures and Earth Retaining Structures – Reference Manual" (FHWA, 2007) for U.S. units.

Overall Stability

Overall stability of the retaining wall, slope and revetment system was controlled by effective stress parameters. A slope inclination at an angle of 21.8 deg (or 2.5H:1V) was selected to satisfy overall stability requirements. An effective stress shear strength cohesion of 100 psf was used for the silty clay and clayey silt strata. The results of the overall stability review are shown in Figure 16.

Overall stability of the temporary working bench (haul road in front of the retaining wall) was also checked using total stress parameters and a uniform construction surcharge load of 600 psf. The temporary working bench geometry is shown in Figure 17.

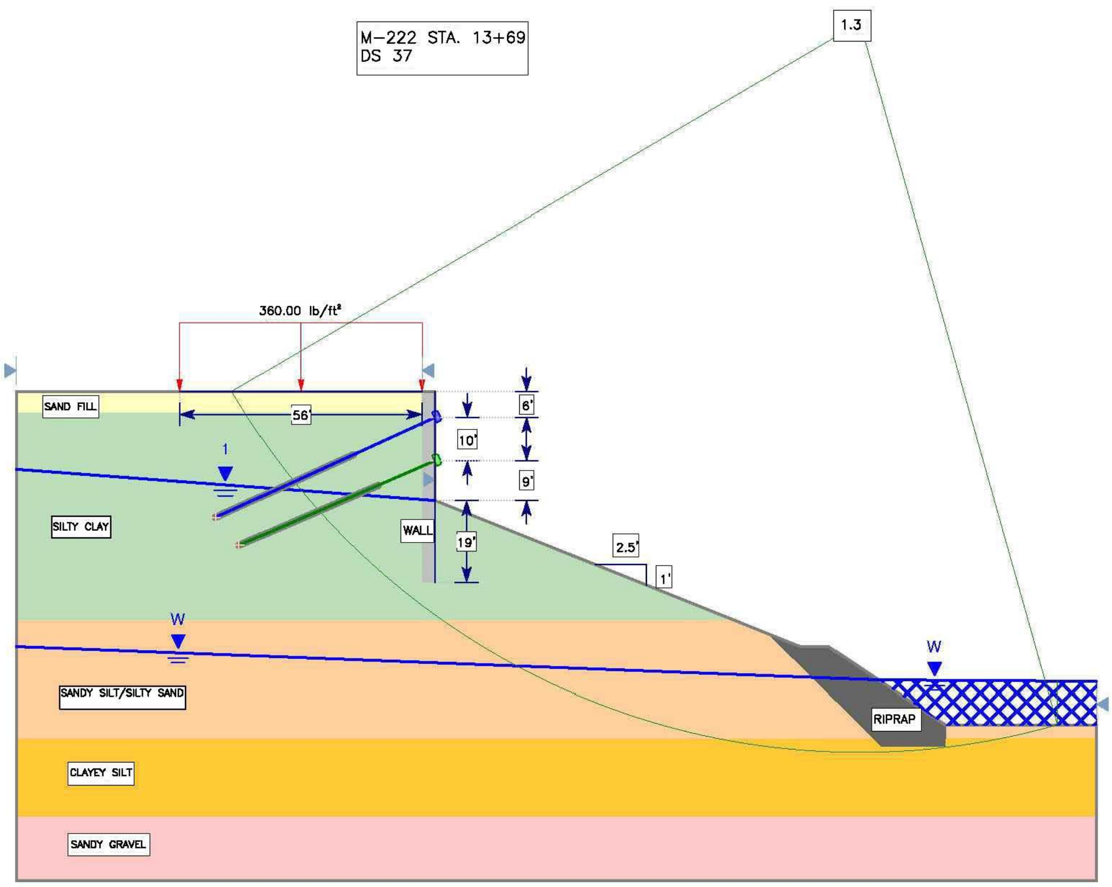

Fig. 16. Overall Stability Results (Effective Stress Parameters)

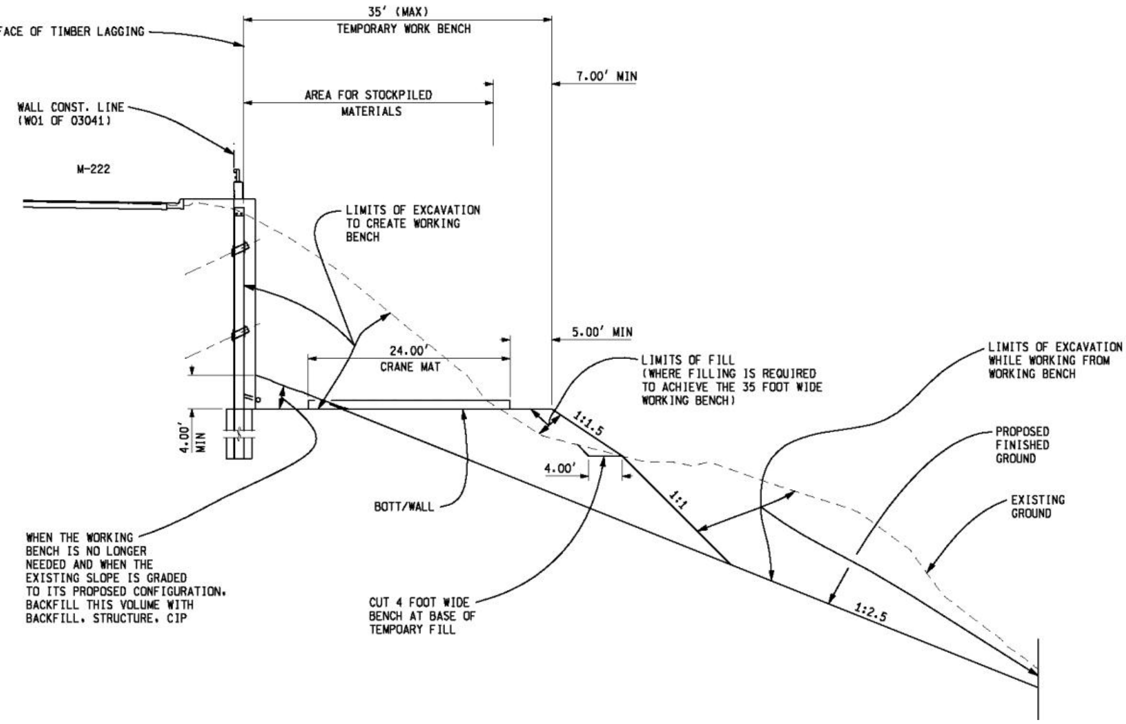

Fig. 17. Temporary Working Bench Detail

WALL CONSTRUCTION

A total of sixty-six (66) permanent soldier piles were installed using traditional drilled shaft installation methods and a 3-foot diameter hole shown in Figure 18. Temporary casing was used in the upper portion of the soldier pile hole due to the sandy soil encountered near the ground surface. Structural ready-mix concrete was placed in each soldier pile hole below the designed CIPC elevation while the remaining height was filled with a controlled low strength material (CLSM).

Fig. 18. Soldier Pile Installation

Timber lagging was installed behind the front flanges of the soldier piles in 5-foot lifts with the exception of the first lift which was limited to 3 to 4-foot lifts in certain areas due to the sandy soil encountered near the ground surface shown in Figure 19. The CLSM from the soldier pile installation was chipped away to expose the front flanges of each soldier pile for lagging installation.

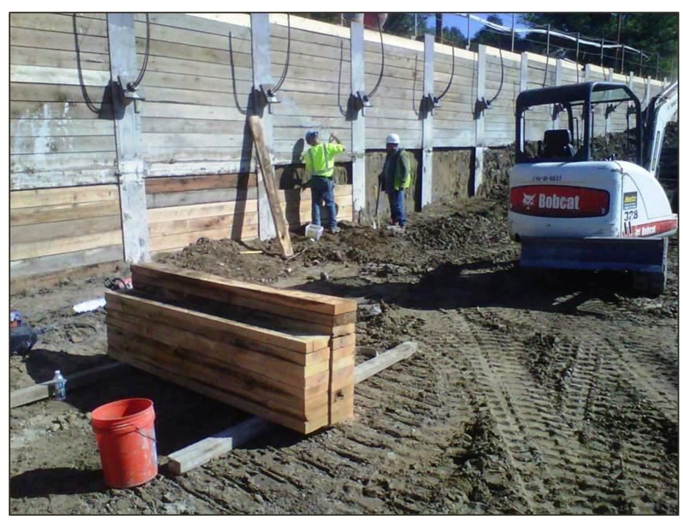

Fig. 19. Wood Lagging Installation

Ninety-two (92) ground anchors were installed using temporary drill casing with air and water as the flushing medium as shown in Figure 20. Anchors were tested up to

FDL increased in specified increments using a hydraulic jack according to the project specifications as shown in Figure 21. Results from the ground anchor stressing showed that the elastic elongation was within the PTI criteria, but the majority of the anchors did not meet the creep criterion without post-grouting.

Fig. 20. Ground Anchor Installation

Fig. 21. Ground Anchor Stressing

One challenge encountered during the ground anchor stressing operation consisted of excessive movement of a few soldier piles near the scarp area while post-tensioning the upper row of ground anchors. Horizontal movement in excess of 1 inch was noted during stressing prior to reaching FDL due to the lack of passive earth pressure provided in the scarp area. The FDL for these select ground anchors was reduced based on case-by-case evaluation when the FDL could not be verified.

The other main challenge from the ground anchor stressing was meeting the creep criterion specified in PTI even with subsequent post-grouting and extended creep tests. Nicholson found through several single strand gun barrel tests that significant creep was coming from the strand itself and not the ground to grout adhesion. This is also known as metallurgical creep, but PTI's creep criterion does not separate metallurgical creep from their criteria. The confirmed metallurgical creep found in the strand anchors confirmed the elastic elongation performance of the ground anchors, which showed that the full bond length was not being utilized during post-tensioning.

Based on the specifications and challenges mentioned above, ground anchor acceptance was determined based on the decision tree shown in Figure 8.5 PTI (2004).

REVETMENT CONSTRUCTION

The authors do not have direct knowledge of the revetment construction or the drainage and cast-in-place concrete facing related to the soldier pile and lagging wall, but the following figures show the progression of the work as these operations occurred.

Fig. 22. Revetment Installation

Fig. 23. Retaining Wall Drainage Installation

Fig. 24. Slope Grading with Temporary Working Bench

Fig. 25. Completed Project (Prior to Bareroot Planting)

GEOTECHNICAL LESSONS LEARNED

This paper presents some of the design and construction challenges encountered during the M-222 Slope Stabilization Project. The authors of this paper submit the following geotechnical lessons learned from this case history:

1. Aerial photographs are a useful tool to review river geomorphology trends and might prove successful in convincing environmental regulators to allow revetment to extend beyond a current water's edge condition to a historically water's edge condition.

2. Studies on the durability and life-cycle cost of permanent CFA walls are needed. Without further studies, it seems unlikely that MDOT will allow the use of permanent CFA walls in transportation projects.

3. Ground anchor drilling means and methods have a significant impact on the ground to grout adhesion values determined through post-tensioning operations.

4. Nicholson determined that metallurgical creep found in the steel strand during the post-tension operations on this project provided a false failure in the creep criterion and should be reviewed during the next revision of the PTI Recommendations for Prestressed Rock and Soil Anchors.

ACKNOWLEDGEMENTS

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