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EXPLORATION, DECONSTRUCTION, AND REPAIR OF A DISTRESSED MSE RETAINING WALL IN SAINT PAUL, MINNESOTA

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ABSTRACT

A 225-ft long, 11-ft high MSE retaining wall was constructed in fall 2008 around the lowest corner of a parking lot at a community college. The wall provided grade separation between the higher parking lot and the green areas below. No geotechnical exploration was performed for the wall, although one had been performed for building additions elsewhere on campus, and density testing was performed periodically during MSE wall construction. The following spring, pavement had subsided up to a foot near two catch basins located several feet behind the retaining wall facing. Cracks in the pavement opened adjacent to the catch basins, allowing water to infiltrate into the wall backfill and thereby circumventing the planned drainage from the parking lot surface into the catch basins. In that area, the retaining wall facing blocks had also settled by several inches. At that point in time, geotechnical consultation was sought, and a subsurface exploration program was performed. The case history discusses the results of the subsurface exploration program, the probable causes of the wall distress and what went wrong, recommendations made for remediation of the wall, observations of a partial deconstruction of approximately half the wall, and reconstruction of the wall.

INTRODUCTION

As part of renovations and additions at Inver Hills Community College, a 225-ft long segmental block mechanically stabilized earth (MSE) retaining wall was constructed in fall 2008 around the lowest corner of a parking lot that was expanded as part of the construction. In spring 2009, settlement of the pavement near catch basins above the wall, as well as of the wall facing itself, was observed. The college's facilities management commissioned a forensic geotechnical investigation to determine the cause of the wall distress and provide recommendations for mitigation.

BACKGROUND

Inver Hills Community College is located in Inver Grove Heights, Minnesota, which is located about 10 miles southeast of downtown Saint Paul, Minnesota. Prior to planned building additions to an art building, the college hired a local geotechnical engineering firm to perform a subsurface exploration program consisting of standard penetration test borings. The borings encountered glacially deposited soils, with silty sand being the predominant soil type. Ground water was not encountered in any of the borings, which were performed at elevations as low as about 20 feet below the eventual bottom elevation of the MSE wall. The report focused on geotechnical recommendations for the building additions, with the only mention of retaining walls being as part of a general recommendation for field density testing of all backfill and fill near structures, including behind retaining walls. Later on in project planning, an expansion of a campus parking lot was added. The parking lot expansion would be toward a lower elevation area (Fig. 1), and the project team decided to utilize a retaining wall to provide grade separation between the higher parking lot and the green area below. The north-south length of the parking lot was to be expanded from a dimension of about 400 feet to about 600 feet (approximate east-west dimension remained at about 500 feet). Proposed grades for the parking lot would direct a drainage area of about 170 feet by 500 feet of the parking lot toward the retaining wall.

The wall's location, geometry, and facing type (segmental blocks) were chosen by the project civil/structural engineering firm. The wall would be 225 feet long, wrapping around the northwest corner of the parking lot (Fig. 2). The maximum height of the wall (not including embedment) was 11 feet, and its top elevation was to be constant (meaning that the bottom of wall elevation would change along its length as it tied into

Fig. 1. "Bird's eye" aerial imagery of site prior to parking lot expansion and MSE wall construction. North is upwards. (Credit for photo to www.bing.com.)

slopes on either end). The wall itself was designed later on during the project as an MSE wall by a different engineer, who was hired by the wall vendor.

The wall was to have a facing batter of about 7 degrees. Six layers of geogrid were specified for the tallest wall section (11 feet), with a maximum geogrid length of 14 feet; the ratio of reinforcement length to wall height was therefore 1.3. Geogrid was to be "sandwiched" between courses of facing blocks, as is common for segmental block-faced MSE walls. Design depth of soil cover over the wall toe was to be 2 feet. The friction angle and unit weight of all wall backfill, foundation soils, and retained soils were assumed to be 28 degrees and 125 pcf. No strength testing of the backfill soil was specified, and clayey soils were explicitly allowed as backfill material, provided the plasticity index was 20 or less. Density testing was specified at the rate of 1 test for every two feet vertically, for every 50 lineal feet of wall, with changes allowed as directed by the project geotechnical engineer; over 20 density tests would be expected for the 225-foot long wall. Compaction levels were to be 98% of Standard Proctor dry density for the wall backfill, and 100% for utility trenches below the wall. Wall drainage was to be achieved by 12 inches of "free draining aggregate" behind the wall facing. The specified range of allowable gradations for the drainage aggregate was quite wide. For instance, maximum allowed particle size was 1 inch, but the portion passing the No. 4 sieve could range from 0 up to 60%. Five percent fines content was allowed. A geosynthetic separator was not shown on the drawings, although the construction notes indicated filter fabric should be placed directly behind the facing blocks.

A catch basin was to be installed in the curb of the parking lot above the wall, to collect storm water runoff from the parking lot (about one-third of which was to be graded toward the MSE wall), and feed it into a 24-inch diameter reinforced concrete pipe (RCP) located eight feet below the wall. The RCP was shown on the wall drawings, as was one of the catch basins. The designer specified that the backfill above the RCP and below the MSE wall must be compacted to at least 100% of Standard Proctor maximum dry density. The plans indicated that the geogrid should be trimmed as needed around the below-grade concrete drop structure below the catch basin, which was to be located 7 feet behind the wall. Design geogrid length at that location was 10 feet, and the wall height was 7 feet. The MSE wall plans did not show a second catch basin a short distance to the east that was indicated on the civil drawings; this second catch basin, which was to have a sump elevation about 7 feet below top of wall (about 12 feet above the base of the adjacent catch basin's drop structure). It is not clear to the author whether or when the wall designer was made aware of the second catch basin.

Construction of the wall occurred in early fall 2008. Density testing was performed periodically during MSE wall construction by the same geotechnical engineering firm that had performed the geotechnical exploration. However, later that fall, the owner elected to change testing firms, and American Engineering Testing, Inc. (AET) was hired to provide construction testing services.

Fig. 2. Overhead aerial imagery of site following parking lot expansion and MSE wall construction at northwest corner of lot. North is upwards. (Credit for photo to www.bing.com.)

Fig. 3. Subsidence and openings in pavement near catch basins located within wall backfill. Settlement of the wall facing blocks (behind chain-link fence) is also apparent.

During the following spring, pavement subsidence on the order of several inches to up to a foot occurred near two catch basins located several feet behind the face of the retaining wall (Fig. 3). Displacements of the pavement were great enough that cracks and openings in the pavement adjacent to the catch basins could allow water to infiltrate into the wall backfill and thereby circumvent the planned drainage from the parking lot surface into the catch basins. In that area, the retaining wall facing blocks also underwent downward movements of several inches to a foot. At that point in time (early April 2009), geotechnical consultation was sought from American Engineering Testing, Inc., at which time the author began his involvement in the project.

SITE OBSERVATIONS

A site visit was performed to observe the condition of the wall, and the following observations were made:

1. A pile of snow was present on the pavement surface above the wall. Melt water from this snow pile was observed to be entering voids in the pavement adjacent to the catch basins.
2. A surface depression was located directly behind the low point of the retaining wall, between the wall facing and the catch basins (Fig. 3). From observing the soils exposed in this depression, it appeared the drainage aggregate might not extend 12 inches behind the facing blocks (as design plans showed). Erosional features below the wall also suggested the wall had been overtopped by runoff.
3. Cracking of the pavement had occurred at a distance of about 20 to 25 feet behind the wall facing both along the north side of the wall and along the west side to about 50 feet south of the northwest corner of the wall.
4. A slight bend in the wall alignment was visible along the west side of the retaining wall, at about 20 feet south from the northwest corner of the wall.
5. Some facing blocks had cracked near the northwest corner of the wall.
6. Fines (i.e. silt and clay) were visible on the "ledges" of the courses of wall facing blocks in the general area where the parking lot pavement was distressed, as well as below the drain tile outlets (Fig. 4). The brown color of the fines would match that of the backfill soil (as encountered by a soil boring discussed later), but not the light tan color of the drainage aggregate.
7. Drainage pipe outlets were installed above the ground surface at the toe of the wall, as much as two feet above design elevation (Fig. 5). The design drawings showed that the drainage pipes should be located at the bottom of the first course, daylighting through the soil cover. This was not the case.
8. Maximum exposed height of the wall was about 10 to 10½ feet, with about 15 courses of the eight-inch tall facing blocks exposed, in addition to one course of the approximately four-inch tall cap blocks. Based on the total design wall height of 11 feet, the soil cover provided was in the range of ½ to 1 foot, which was less than the design soil cover thickness of 2 feet.
9. Cracking of the soil surface was visible at a distance of about 20 feet on either side of the storm sewer near a manhole located to the west below the wall (Fig. 6).

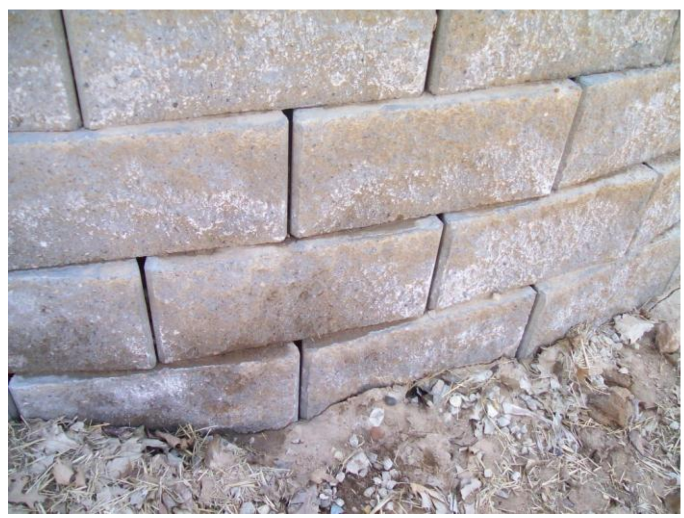

Fig. 4. Silt and clay deposited on and below facing blocks by migration of fines out of backfill and through joints in facing.

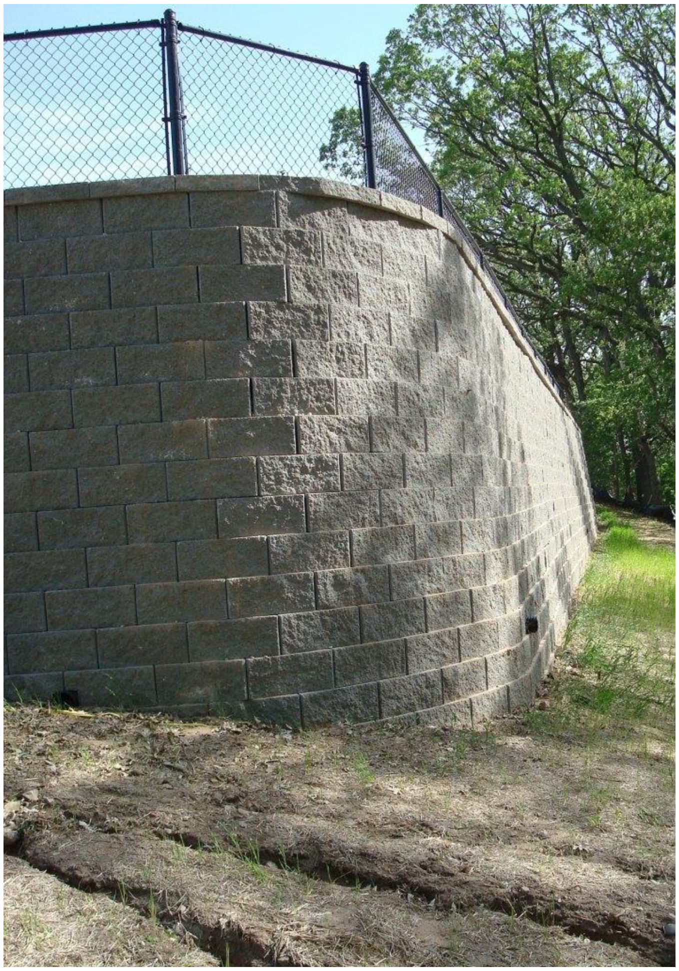

Fig. 5. View looking south from northwest corner of wall, showing drain pipes located above bottom-of-wall, and bend along west line of wall. Surface erosion due to overtopping of wall is present in foreground.

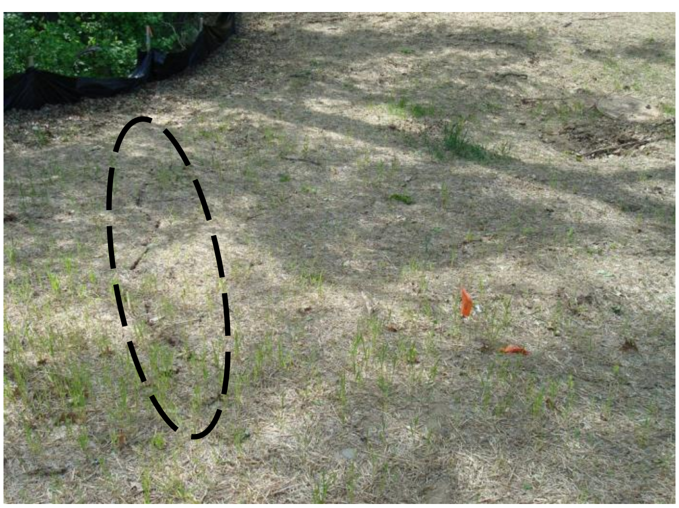

Fig. 6. Surface crack along south (lefthand) edge of backfilled storm sewer trench.

ELEVATION MEASUREMENTS

Measurements of surface elevations of selected points of the wall were made during a subsequent site visit (at the same time as the subsurface exploration discussed in the following section). The pertinent findings from those measurements were:

1. The elevation of the northwest corner of the wall was 0.19 feet below that of the east end (top-of-wall elevation was to be constant according to the design).
2. The low point along the north side of the retaining wall (in front of the catch basins) was almost 7 inches lower than the east end of the wall, and 6 inches of that elevation difference occurred in a horizontal distance of about 6 feet (Fig. 7).
3. The west catch basin was tilted toward the east and an average of 1 inch lower than design elevation, the east catch basin over 4 inches lower than design elevation, and the northwest corner of the parking lot over 8 inches low.

Fig. 7. Subsidence of facing blocks along north side of retaining wall (view is opposite that shown by Fig. 1).

SUBSURFACE EXPLORATION

A subsurface exploration program was performed using both cone penetration test (CPT) soundings and a soil boring sampled continuously to a depth of 21 feet using three-inch diameter, thin-wall (Shelby) tubes. The CPT soundings were advanced to depths of 24 to 40 feet below pavement surface. The soil boring was performed about five feet southwesterly from the catch basin/drop structure (about 10 feet behind the wall facing, just behind the reinforcement). One of the CPT soundings was located about 1.5 feet away from the soil boring, to provide correlation between the CPT data and the soil boring (the CPT sounding was performed first, followed by the soil boring). A second CPT sounding was performed about 10 feet farther back, and the third CPT sounding was performed near the south end of the wall in an area of no visible wall distress. Figure 8 indicates boring and sounding locations relative to the wall and the catch basins.

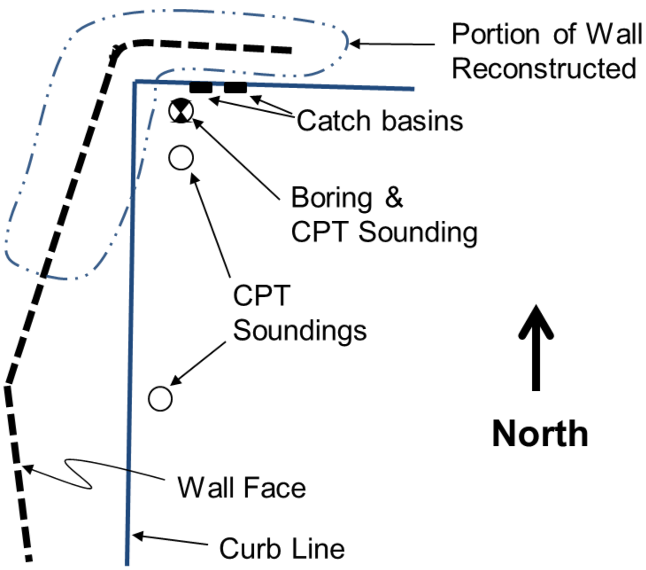

Fig. 8. Approximate soil boring and CPT sounding locations relative to MSE wall plan layout (scale varies).

The soil boring encountered wall backfill consisting of a mixture of silty sand, clayey sand, and sandy lean clay. This was in direct contrast to the sand with silt backfill that had been indicated on the report(s) for all nine field density tests performed on MSE wall backfill during construction by the original testing firm; all tests had been reported as passing.

Three density tests were also performed on leveling pad base aggregate, and 13 tests had been performed on utility trench backfill below the bottom elevation of the wall. All of these tests were reported to have passed. Of note is that the original testing agency reported gradation test results for a sample of retaining wall backfill that had 27% passing the No. 200 sieve (contradicting its classification as sand with silt on the field density test summary reports). Furthermore, the reported maximum dry density for the material was 130.8 pounds per cubic foot (pcf), based on the Modified Proctor test. (This is according to information presented in subsequent meetings, although reports had originally identified it as Standard Proctor maximum dry density.) In either case, this maximum dry density is a high value for sand with silt, based on the author's experience.

Silty sand and clayey sand would be consistent with the predominant native soils on site (glacial till deposits), although one boring log from the original geotechnical exploration had indicated some layers of sand with silt interbedded with the silty sand till. While it was plausible based on the grading plans that this material could have been excavated as borrow material and used as wall backfill, the soil boring and CPT soundings indicated this was not likely the case. Fines content for the recovered soil samples ranged from 36% to 58%.

In-place dry densities were determined from the Shelby tube samples by cutting the Shelby tubes into approximately 8-inch long sections: an upper, a middle, and a lower section. The upper section was not used, in case some of that material had been disturbed during the previous sampling interval or by the drilling process. The moist samples were then weighed, dried in an oven, re-weighed, and then removed from the Shelby tube sections so the tube sections themselves could be weighed. Table 1 shows the results of the density tests. Samples were also combined into two composite samples (one of clayey sand and one of silty sand) to allow Standard Proctor tests to be performed. A maximum dry density of 128.2 pounds per cubic foot was determined for the clayey sand and 129.3 pcf for the silty sand; both had an optimum water content of about 9%. As the third column in Table 1 shows, the percent compaction for materials recovered from the soil boring ranged from 83% to 91%, significantly less than either the 98% specified for wall backfill, or the 100% compaction level specified for utility trench backfill below the wall.

Table 1. In-place Density Test Results for Soil Boring

Depth (ft)	Water Content (%)	Dry Density (pcf)	Compaction Level (%)
6	19	107	83
6.5	14	117	90*
8	13	111	87
8.5	13	117	91
10	13	115	90
10.5	14	114	89
12	14	102	79*
12.5	14	115	89*
14	15	111	87
14.5	15	112	87
16	12	109	85
16.5	13	116	90

Note: Compaction levels based on 129.3 pcf for the asterisked values; all others based on 128.2 pcf.

Because of the high fines content of the backfill soil, and based on the specified gradation for "free draining aggregate" on the plans for the wall, small excavations behind the wall facing were dug using shovels to recover samples of the drainage aggregate. From these limited excavations, it did not appear that a full 12-inches of drainage aggregate was provided behind the wall blocks. Gradation testing was performed on the drainage aggregate, in order to assess whether it was compatible as a filter material for the backfill soils, based on USACE filter criteria (USACE 1993). The drainage aggregate had 80% of its particles between 0.75 and 0.5 inch, with a D_{15} of about 11 mm. From the single

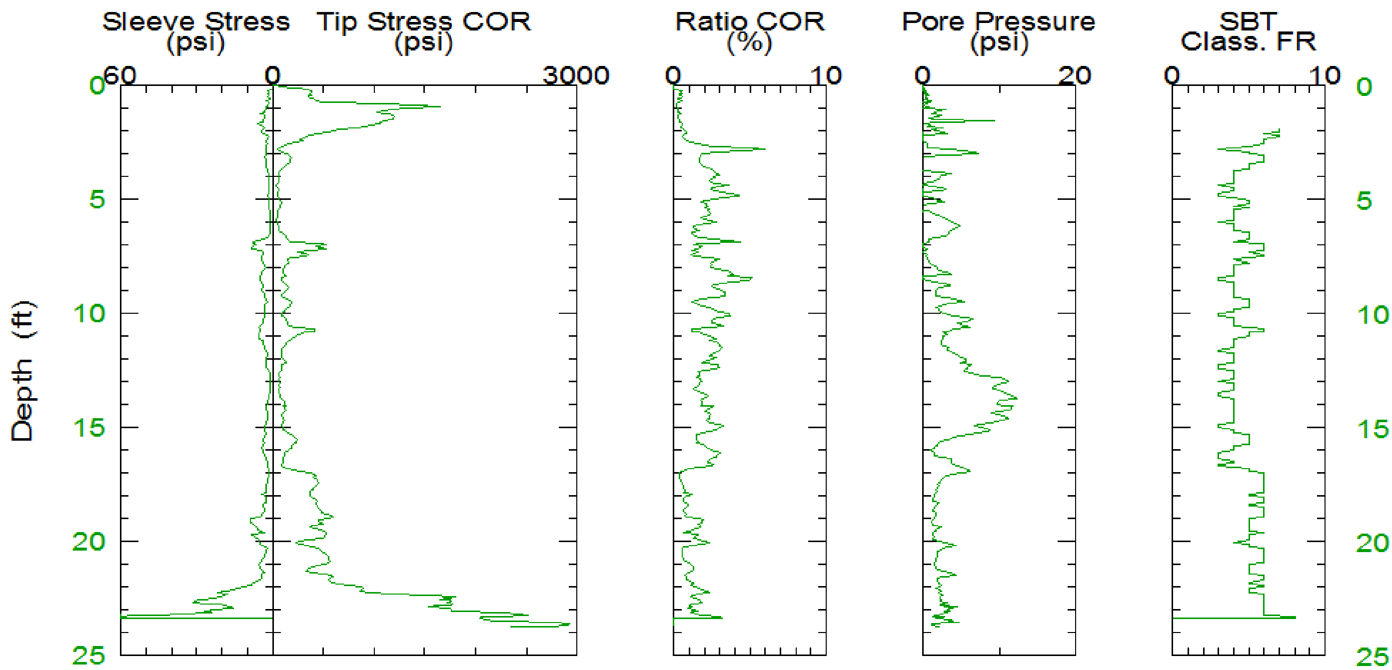

Fig. 9. Results of CPT sounding performed adjacent to soil boring, through wall backfill. Soil Behavior Type (SBT) in the far right column is based on friction ratio (Robertson 1990).

gradation test performed on the backfill soils during construction, the d_{85} for those soils was 0.8 mm (and the gradation testing on samples recovered from the soil boring suggests that value was on the high end).

The necessary D_{15} for the drainage aggregate to meet filter criterion against the backfill soils for d_{85} = 0.8 mm would have been 1.6 mm, and therefore it was not a suitable filter material for the silty to clayey sand backfill. This corroborated the observations of silt deposited on the ledges of the facing blocks for the wall and at the base of the wall. Because the drainage aggregate was relatively uniform in size (and much larger in size than the backfill soil particles), fines could migrate out of the backfill soils when they became saturated and drained into the aggregate.

From Figure 9, it is apparent that the CPT sounding adjacent to the soil boring corroborated that the backfill material was not nearly as competent as the in-situ silty sand glacial till soils located below a depth of about 17 feet. The soil behavior type of the fill soils based on normalized friction ratio (Robertson 1990) was typically Type 4.

The CPT results were used to estimate shear strength parameters, and a global stability analysis was performed. (No global stability analysis had been performed as part of the original wall design, even though grade in front of the wall was sloping downward.) Results showed that a global stability failure was not likely—the computed factor of safety for a circular failure surface encompassing the MSE wall was 1.6. This was the case even assuming that the clayey sand backfill soils of the sewer trench below the wall would behave as a soft to firm cohesive soil with undrained shear strengths in the range of 500 to 1,000 pounds per square foot (psf). Therefore, the wall distress was determined to not be the result of a global stability issue.

PROBABLE CAUSE OF WALL DISTRESS

After having reviewed the available information and the results of the subsurface investigation, it was clear that several factors likely contributed to the wall movements and failure of the pavement near the catch basins:

1. The number of field density tests was less than the wall designer had specified, and discrepancies between reported density test results, soil type, and information from the post-construction soil boring near the catch basin suggest less-than-ideal compaction of the wall backfill occurred in at least some areas.
2. The silty and clayey wall backfill material was highly frost susceptible (an important consideration in Minnesota), and it was not well-draining.
3. The wall drainage system was not constructed as designed, based on the location of the drain pipe outlets and the small amount of drainage aggregate behind the blocks. However, the drainage system design was insufficient for the potential surface water flow toward the wall.

Based on the above, AET concluded that some frost heave of the catch basins likely occurred during the winter. Differential settlements between the two catch basins also likely occurred following construction because the eastern catch basin was founded on an additional 12 feet of fill compared to the adjacent catch basin. These two phenomena either initiated or increased pavement cracking adjacent to the catch basins, allowing surface water to enter the soil along the storm sewer. Settlement of the pavement area (and associated cracking) resulted from:

1. Consolidation of the poorly compacted wall and utility trench backfill under its self-weight and the additional weight due to infiltrated water.
2. Migration of fines out of the backfill material and through the drainage aggregate and joints of the facing blocks due to flowing water.
3. Internal erosion of backfill material along the sewer pipe.

Furthermore, during project meetings subsequent to the forensic geotechnical study of the distressed wall, it was revealed that the second (eastern) catch basin had been added shortly after the wall was constructed to its full height, so as to satisfy a city requirement regarding the storm sewer capacity. The utility subcontractor stated that no disassembly of the wall facing was performed (nor was any apparently required by the project design team), suggesting that the catch basin and sump were installed in a very tight excavation and backfilled without any independent observation or testing. This further called into question what the state of compaction was near the eastern catch basin.

MITIGATION OPTIONS

Based on the visible distress to the wall and the additional problems revealed by the soil boring and CPT soundings, there was some discussion of replacing the MSE wall entirely with a cast-in-place reinforced concrete wall, which could be designed to resist hydrostatic forces (assuming similar drainage issues arose again). The idea of a reinforced concrete wall as a mitigation option may have been due to an understandable perception by some involved parties that an MSE wall was unreliable. However, AET concluded that an MSE wall with a robust internal drainage capability and well-draining backfill would have been unlikely to settle due to water infiltration and would have likely withstood unplanned amounts of storm and melt water entering the backfill.

Therefore, AET recommended that the wall be deconstructed, then reconstructed with improved backfill and drainage in the area of visible distress to the wall and/or the pavement overlying the wall backfill. This area was largely delineated based on surface cracking of the pavement overlying the wall backfill. Of the 225 feet of the wall, about 115 feet at the south end of the west line was left in place. Other recommendations for the portion of the MSE wall to be reconstructed were as follows:

1. A minimum 6-inch thick base of well-graded sand with gravel and silt or well graded gravel with sand and silt should underlie the wall and its reinforcing zone plus three additional feet. The purpose was to provide a moderate permeability, high-strength base to prevent water within the backfill from readily seeping deeper into underlying silty or clayey fill soils.
2. The backfill material in the reinforced zone should be a clean, crushed rock backfill with maximum particle size of 1 inch, not more than 10% passing the No. 4 sieve, and not more than 3% passing the No. 200 sieve.
3. Behind the entire reinforced zone, a minimum 3-foot wide well-graded sand filter should be provided. The sand filter needed to meet filter criterion against the retained silty to clayey sand soils, and likewise with the crushed rock backfill material. Compaction of this material should be 100% of Standard Proctor maximum dry density.
4. Drain tile outlets should daylight at bottom-of-wall elevation.

The crushed rock backfill would have higher strength than likely actually needed (recall that the wall was originally designed based on a friction angle of just 28 degrees), and the same is likely true for the permeability of the backfill. However, a conservative re-design was considered worthwhile to definitively avoid future wall distress, and to provide some measure of drainage for the adjacent portion of the original wall that would remain in place.

The original wall designer incorporated AET's recommendations for wall re-construction into their revised drawings for the re-designed wall, and both the civil/structural firm and AET provided review of the shop drawings. In that sense, AET's "forensic" geotechnical report essentially served as the geotechnical report that should have been done for the wall prior to its original construction.

The gradation finally specified for the wall backfill material was the same as that for "coarse filter aggregate" often specified by the Minnesota Department of Transportation (MnDOT). Similarly, the gradation specification for the sand filter material was the same as for "fine filter aggregate' often specified by MnDOT. This is evidence that the problem of incompatible materials and the solution of compatible graded filters are neither mysterious nor unsolvable, but rather are often ignored in non-transportation projects.

WALL DECONSTRUCTION

Deconstruction of the distressed portion of the MSE wall began in mid-July 2009 and lasted 4 work days. The wall subcontractor salvaged the facing blocks for later re-use when rebuilding the wall (Fig. 10). Wall backfill and geogrid were not suitable for re-use.

Fig. 10. Early stage of MSE wall deconstruction. Note drainage aggregate within blocks, but not extending 12 inches behind facing blocks per design.

At the request of the owner and the civil/structural firm, an engineer from AET was present to document wall deconstruction and observed deviations from project specifications or drawings. There were significant deviations. For instance, lengths of geogrid were measured and compared to design geogrid lengths. Geogrid lengths were generally found to be at or within a few inches of design lengths.

However, three of the four layers of geogrid at one cross-section of the wall, located about 10 feet west of the primary catch basin, were only 10 feet in length rather than the 14 feet design length. The uppermost layer of geogrid remained at 10 feet for a distance of 30 feet farther west. Furthermore, gaps of 10 to 12 inches were observed between adjacent pieces of geogrid in this area; the wall was to have had full coverage. Lastly, geogrid also seemed to be entirely lacking in the area of the second catch basin. Therefore, the reinforcement ratio of the wall within the zone of greatest wall distress was certainly less than the design value of 1.3. This was likely at least a contributing factor to the wall distress, in that shorter geogrid lengths reduce the mass of the reinforced zone, lowering the resistance to lateral earth (or water) pressures. The result would be greater lateral displacements of the wall, which also could have opened cracks in the pavement above the wall, thereby allowing surface water infiltration.

Additional shortcomings of the wall that were observed during deconstruction included:

1. If anything, samples of the backfill soils were typically higher in fines content and more clayey than the soil boring and CPT soundings had indicated. Significant amounts of sandy lean clay were also encountered.
2. The minimum 12 inches of "free draining aggregate" to be placed immediately behind the back of the facing blocks was not observed. Because the "H" shape of the facing blocks required placement of aggregate within the blocks to lock them together, it is possible that the wall subcontractor erroneously believed that this "interlock" aggregate satisfied the design requirement (Fig. 11).
3. No horizontal drain tile line was present (Fig. 11), meaning the drain tile outlets were simply short pieces passing through the wall, but connected to nothing (analogous to weep holes).
4. Seven density tests were taken at different elevations during wall deconstruction. Six tests had compaction levels below 95%, and one at 97% (recall 98% was specified for wall backfill); these confirmed the tube densities of samples recovered from the boring.

Fig. 11. Wall partially deconstructed, with relative lack of drainage aggregate and no horizontal drain tile or geotextile filter fabric observed.

Based on the additional information discovered during wall deconstruction, some additional conclusions can be made regarding the observed wall distress. In particular, the backfill in the reinforced zone can be described as predominantly cohesive. Expected lateral displacements to mobilize active earth pressure can be an order of magnitude higher for clays compared to clean sands (Das 2000). Hence, for an 11-ft wall, where one might expect to develop active conditions in sand after lateral movement of about 0.1% times the height (or about one-eighth inch), the displacement in clay could be on the order of 1% (over 1 inch). This movement could well have occurred during the winter, leading to cracking of the pavement, and infiltration of surface water. Finally, the installed drainage system was entirely inadequate to drain the backfill.

WALL RECONSTRUCTION

Reconstruction of the MSE wall began once a competent excavation bottom was reached—additional overexcavation was performed below the wall following field judgments by AET's on-site engineer that the exposed soils were wet, soft, and had low bearing capacity (Fig. 12), including below the area of greatest settlement shown in Figure 7. Up to four feet of overexcavation was performed, and this was backfilled with clean sand, capped by at least 1 foot of a well-graded crushed limestone aggregate base (Fig. 13).

The three-foot wide well-graded sand filter zone behind the wall backfill separated the crushed rock backfill from the silty and clayey retained soils (Fig. 14). Sieve analysis tests showed a D_{15} for the crushed rock backfill to be 6.9 mm, whereas d_{85} for the sand filter was 2.0 mm—this is a ratio of about 3.5, less than the maximum recommended ratio of 4 to 5 between a sand base soil and a gravel filter.

A total of 37 field density tests were performed on wall backfill, utility trench backfill, and pavement subgrade soils during reconstruction of the wall. Most tests passed; two tests of the sand backfill below the wall base and three tests of the pavement aggregate base did not pass. These required re-working of the material to attain the minimum specified compaction level of 100%.

Fig. 12. Overexcavation was performed below bottom-of-wall elevation, based on field judgment (and field density test results) showing marginal density of in-place fill soils.

No compaction tests were performed on the crushed rock wall backfill, although visual observations allowed judgment of when sufficient compactive effort had been applied. The material type itself greatly facilitated compaction. This is particularly evidenced in Figure 15, for which it is difficult to imagine clayey backfill being well-compacted around the two catch basins.

Fig. 13. Clean sand capped with well-graded crushed limestone aggregate base material across the entire base of the MSE wall.

Fig. 14. The reconstructed MSE wall was backfilled with a well-graded gravel filter material, with a three-foot wide well-graded sand filter zone behind.

Fig. 15. Compacting gravel backfill around catch basin risers. Geotextile-wrapped filtered drain tile is also visible.

CONCLUSIONS

The MSE wall has performed per the owner's expectations in the three years following the partial reconstruction. The author is not aware of any legal action taken as a result of the wall distress; the general contractor apparently made a claim on the wall subcontractor.

Among the lessons learned (or perhaps more accurately, reinforced for the author as a geotechnical engineer) were:

1. MSE walls themselves are remarkably tolerant of movement, but retained structures, utilities, and pavements are often not, and distress to those elements can subsequently adversely impact the wall.
2. Failure of an MSE wall does not necessarily indicate that the wall type was not suitable—the backfill or other materials may have been unsuitable to the demands on the wall.
3. Consideration of filter compatibility of backfill material and drainage aggregate is very important, especially for segmental block walls, even for walls capped with impervious pavements.
4. If a geosynthetic filter is to be used to separate incompatible materials, then it must be shown on the drawings. If it will not be used (for ease of construction), then it is critical that the drainage aggregate be an appropriately graded filter material for the backfill soil.
5. Walls constructed "in fill" are not immune to water and backfill drainage problems.
6. Silty and, in particular, clayey backfill soils can reduce the margin of safety of a wall design due in part to their low permeability, moisture sensitivity, and frost susceptibility. While these soils can be successfully used in MSE wall construction, they require special design considerations including particular attention to drainage of the backfill, and the owner may need to reconsider expectations with respect to settlement or lateral movements. Displacements from clayey backfill soils tend to be greater in magnitude and can more slowly following construction compared to granular backfill.
7. Placing utilities (especially water utilities) behind retaining walls is risky, but choices can be made with respect to wall backfill type and the wall's drainage system to at least partially mitigate those risks.
8. Poor compaction of utility backfill within the wall backfill zone can cause serious problems to the wall.

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