Settlement plates were placed at the center and also half-way between cell center and the upstream face of the cofferdam cells. Data from the diver inspection showed that settlement plates shifted during cell filling into the cell fill material. At these new locations, settlement plate readings were evaluated to monitor amount of movement within the cell fill. It was not possible to identify the difference in movement between center of the cell and the half-way between cell center and the upstream face of cofferdam cells as intended originally. However, readings were able to be evaluated for a comprehensive understanding of movements within the cell.

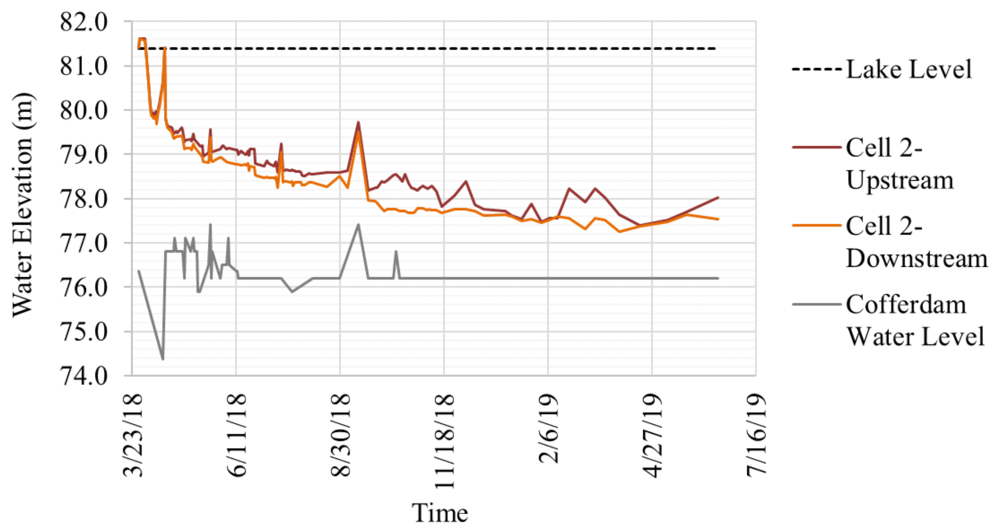

Figure 6: Comparison of Observation Well Readings, Cell 2

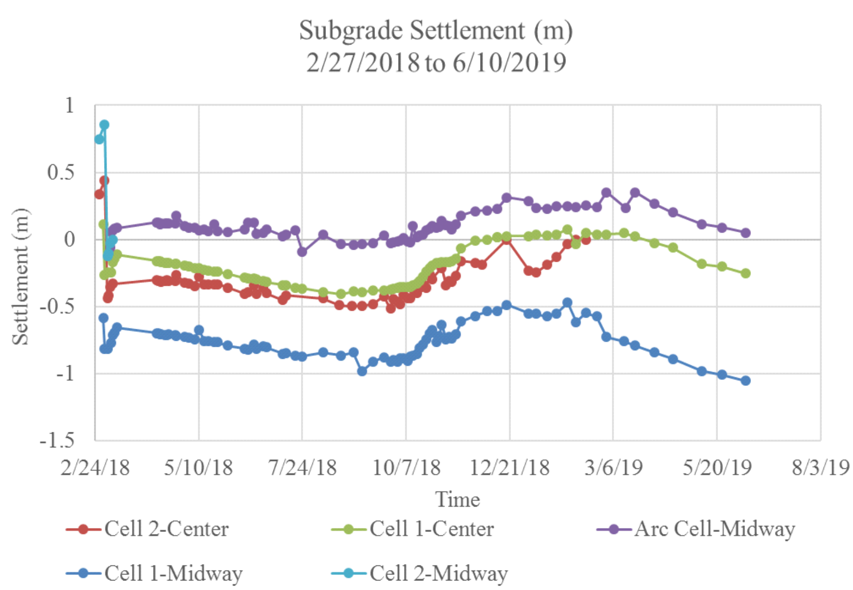

Figure 7: Comparison of Settlement Plate Readings

Settlement plate readings presented in Figure 7 show an immediate settlement up to 90 cm during cell filling followed by heave up to 17 cm during cell dewatering. Two of the settlement plates stopped functioning while the remaining three settlement plates continued to collect data. The settlement plate readings changed steadily at a slow rate as the cell saturation levels reached an equilibrium and consolidation settlement of the foundation soils under the weight of cell fill slowed down. The elevation of fill material at the top of the cofferdam cells was also surveyed to identify if any area settles more than expected and if additional fill would be necessary. Field surveys showed no significant change in cell fill elevation.

Vibrating wire strain gauges were used to monitor bending stresses and interlock tensions. They were installed at multiple elevations on the inside and outside faces of the sheet piles and readings were compared with pre-determined action levels to inform whether any interlocks in the vicinity of the strain gauges were subjected to excessive tensions. While some of the strain gauges produced continuous data, others experienced temporary malfunctioning or provided data with a low signal-to-noise ratio. An example set of strain gauge data (presented in Figure 8) show that channels 2, 3 and 4 had significant noise in data and data in other channels were reasonably clear and steady. Several strain gauges showed sudden spikes or had temporary malfunctioned. The overall goal was to capture the steady changes in strain levels of the sheet piles. Only in one channel did the strain level reach the action level throughout the monitoring program. However, comparing the readings from that gauge with readings from other strain gauges in the vicinity indicated that particular strain gauge might have malfunctioned. As an additional precautionary measure, field surveys were performed in this area to monitor movements during this timeframe.

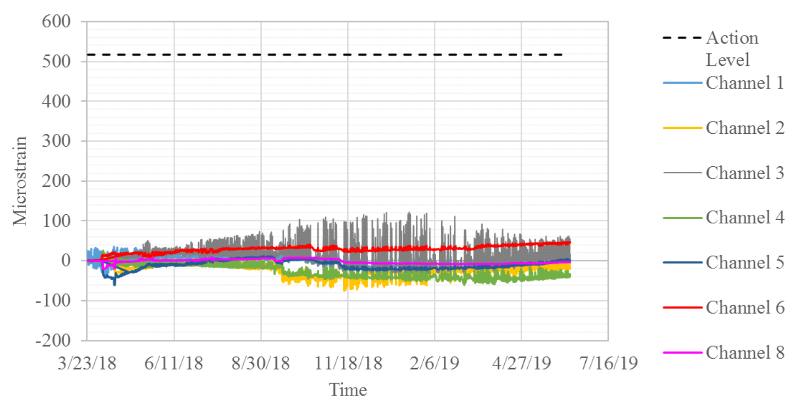

Figure 8: Strain Gauge Data

LESSONS LEARNED

Designing a hybrid cofferdam provided significant cost and time benefits to this project. The instrumentation and monitoring program validated the cofferdam design by providing continuous data to evaluate the performance of the cofferdam system and reduced the risks associated with the spillway re-construction that took place immediately downstream of the cofferdam. Main lessons learned from the instrumentation and monitoring program are as follows:

- The instrumentation and monitoring program showed the majority of the movements (settlements) occurring during the cell filling and dewatering phases. Cofferdam saturation levels reached equilibrium after several weeks. Several pumps were required to dewater the cofferdam area. Throttling of the pumps were required over the course of the project to maintain a dewatered condition, accounting for variations in inflow and rainfall.

- The instrumentation program included redundancy to account for possible instrument failures during or post construction which proved to be valuable following the expected loss of some instrumentation during installation.
- Several instruments required battery replacements or recalibrations during the monitoring period. Symmetrically located instruments helped identify possible malfunctioning instruments.
- Although the alert levels were selected for each instrument, performance of this flexible cellular cofferdam system was best characterized by a comprehensive study of all available data. For example, if a strain gauge was showing a significant increase in strain level, other nearby instruments were checked to identify if this increase was due to a real change in the strain conditions or possible instrument malfunctioning.
- According to the inclinometer readings, the deflections were at their highest values at the end of filling the cofferdam cells and after dewatering of the cofferdam system. The inclinometer readings have been stable after the dewatering period with minor changes in displacement during spillway re-construction.
- Observation well readings showed, even with free draining cell fill materials, that it took several months for the saturation level in each cell to reach equilibrium. During this time period, weep holes continued releasing water and the pumping efforts were coordinated to maintain the water levels within the cofferdam area at a target elevation. It is very important to have backup power and spare pumps on site during the dewatering phase and throughout the entire spillway re-construction project in order to be prepared for storm and other unexpected events.
- A geomembrane bag, positioned on top of the cellular fill material, was used for collecting the water pumped from the cofferdam system before depositing it back into the lake. During one instance, a steady increasing trend of the water level within an observation well indicated that damage had occurred to one of the geomembrane bags which was causing water leakage back into cell. This problem was resolved simply by replacing the damaged geomembrane bag.
- Settlement plates shifted during construction. However, the data were still valuable as they provided information about the overall trend and rate of settlements, and also the differential settlements between different cells.
- Visual observations indicated that the cell fill adjacent to the cofferdam cell sheet piles settled a few centimeters more than the rest of the cell fill material. Since the instrument data at the connections showed no anomaly, this was likely due to reorientation of the cofferdam fill over time as the fill material placed adjacent to the sheet piles is less compacted than the fill material placed away from the sheet piles.
- Although the measured settlements of the settlement plates exceeded the estimated settlement limit of 30 cm, an evaluation of the settlement data along with data from other instruments indicated no significant concerns for the stability of the cofferdam cells and no significant settlement was observed at the top of the cofferdam cell fill during site inspections.
- Field surveys were performed at a location where the vibrating-wire strain gauges stopped functioning. A couple of the vibrating-wire strain gauges were recovered after temporary malfunctioning while others failed and were inoperable for the duration of the construction. Some of the instruments continued to have a signal noise. However, having

redundancy in the monitoring program and symmetrically locating the instruments provided continuous monitoring of the cofferdam.

CONCLUSION

This paper summarizes the design approach, construction challenges and results from the monitoring program of a hybrid cofferdam designed to provide a dry area for a spillway re-construction project. The hybrid cofferdam consists of two circular cells, an arc cell, two cantilever sheet pile walls that connected to the upstream embankment of the dam, and a gravel berm for additional support. The cofferdam cells were filled with free-draining cell fill material placed on highly plastic, overconsolidated, foundation clay soils. To monitor stresses, deformations, and settlements of this flexible cofferdam system, an instrumentation program was developed to monitor the performance of the cofferdam throughout its service life. The design decisions that led to the construction of a cellular cofferdam system, challenges experienced during construction, and results of the instrumentation and monitoring program are presented in this paper. Lessons learned were also discussed.

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The Effects of Poor Design and Construction Workmanship on a Mechanically Stabilized Earth (MSE) Segmental Retaining Wall (SRW) in North Carolina

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ABSTRACT

A two-tiered mechanically stabilized earth (MSE) segmental retaining wall (SRW) system with a maximum height of approximately 90 feet along its curving, concaved section was built at a new residential development near Asheville, North Carolina, in 2011. The two-tier section of the wall was approximately 400 feet in length where the upper wall was nested within the lower wall to account for a topographic depression and the site-specific grading. During the last layers of fill placement of the two-tier portion, substantial bulging, vertical deformations, and cracking along the face blocks of the wall were observed. As a result, the owner of the wall decided to demolish a significant portion of the upper wall. A detailed field investigation was performed, before and during the wall removal, to evaluate the as-built conditions of the MSE/SRW wall and to assess the potential factors that contributed to the observed movement and distress. The field investigation included detailed observations of various components of the wall: 1) backfill; 2) foundation; 3) geogrid reinforcement; 4) facing blocks; and 5) drainage system. This was done by means of 13 hand-dug test pits, 13 sand cone tests, and laboratory analyses of multiple bulk samples. The results of observations, testing, and analyses illustrate how standard design calculations for MSE/SRW walls do not incorporate the influence of some of these five components. In particular, concave sections of such walls as well as serviceability conditions are not very well accounted for by standard design practices. Finally, this paper provides insights into the design and construction practices that contributed to the observable deformations and distress in the wall, the importance of appropriate construction to appropriately match the design assumptions, and the relative effect of various site conditions on the performance of an MSE/SRW wall.

INTRODUCTION

The vast majority of Mechanically Stabilized Earth (MSE) Segmental Retaining Walls (SRWs) that have been constructed meet performance expectations by designers and owners. There are plenty of cases, however, where the performance of an MSE/SRW wall does not meet performance expectations and it may even fail (e.g., Yoo, 2006; Roy, 2008; Look, 2016; Haddad, 2008). In these situations, the MSE/SRW wall may be rebuilt with modifications or replaced with a different retention system.

The authors have investigated several MSE/SRW walls that appeared to have followed core design principals and material standards and yet did not meet performance expectations. These situations appear to fall into one or more of the following categories: 1) risky design; 2) poor detailing; and 3) poor workmanship.

This paper presents a case history of a MSE/SRW wall with a maximum height of approximately 90 feet along its two-tier and concaved section, constructed at a new residential development near Asheville, Buncombe County, North Carolina in 2011. This wall did not meet performance expectations and, therefore, was disassembled. The disassembly afforded the opportunity to document conditions of the wall that produced the undesired outcomes. The subsequent observations and forensic evaluations emphasize the necessity of appropriate quality control and the need for a design engineer to critically evaluate constructability of their selected design and consider design conditions that deviate from standard plane strain assumptions.

CORE DESIGN PRINCIPLES AND CONSTRUCTION GUIDANCE

The current design and construction of MSE/SRW walls are governed by guidelines provided by National Concrete Masonry Association ("NCMA") (NCMA, 2016), Federal Highway Administration ("FHWA") (FHWA, 2009) and the American Association of State Highway and Transportation Officials ("AASHTO") (AASHTO, 2008). In general, federal highway projects follow the FHWA and AASHTO guidelines, whereas the private projects follow NCMA guidelines. Consequently, due to the nature of the subject project being part of a private development, the Third Edition of the Design Manual for Segmental Retaining Walls published by the NCMA was the appropriate guidance for its design and construction.

The following five recommendations by NCMA are considered to be some of the key pertinent design and construction guidelines for the subject MSE/SRW wall:

1) *Backfill:* NCMA suggests that for a wall with height greater than 20 feet, the plasticity index of the backfill should be less than 6. The preferred material should be a cohesionless, free-draining material (with less than 50% passing a number 40 sieve and less than 10% passing a number 200 sieve). The reinforced fill should be compacted to a minimum of 95% of the standard Proctor Density (ASTM D1557). The moisture content of the fill should be within 2% of its optimum moisture content.
2) *Foundation:* NCMA recommends that the foundation soils provide adequate support to the structure without excessive settlement.
3) *Geogrid reinforcement:* NCMA recommends that reinforcement has a National Transportation Product Evaluation Program ("NTPEP") certification for Geosynthetic Reinforcement ("REGEO") evaluation. NCMA states that reinforcement spacing should be less than 24 inches.
4) *Facing blocks:* NCMA recommends that the facing blocks conform, at a minimum, to ASTM C1372.
5) *Drainage system:* A face drain from gravel should be installed. It should be placed a minimum of 12 inches (305 mm) away from the back of the wall unit and a minimum of 24 inches (600 mm) away from the face of the wall. In addition to the above drainage guidelines, NCMA recommends the installation of an internal drain pipe close to the base of the wall.

Each of these design elements are relevant in the subject case history and discussed below.

PROJECT BACKGROUND

Development Description

The Berrington Village development project is located in Asheville, NC. Three hundred multi-family units were planned in thirteen buildings at this development. A small creek, flowing to the west, passes through the site. A total of twenty-five MSE/SRW walls were planned as part of this development to provide relatively level pads for the proposed buildings.

For Buildings 4 and 5, a two-tier MSE/SRW wall was planned as shown in Figure 1. Wall 9A ("Lower Wall") varied in height from 4 to 48 feet and was approximately 400 feet long. Wall 9B ("Upper Wall") varied in height from 4 to 50 feet and was approximately 1,050 feet long. Construction for Wall 9A started approximately on May 19, 2010.

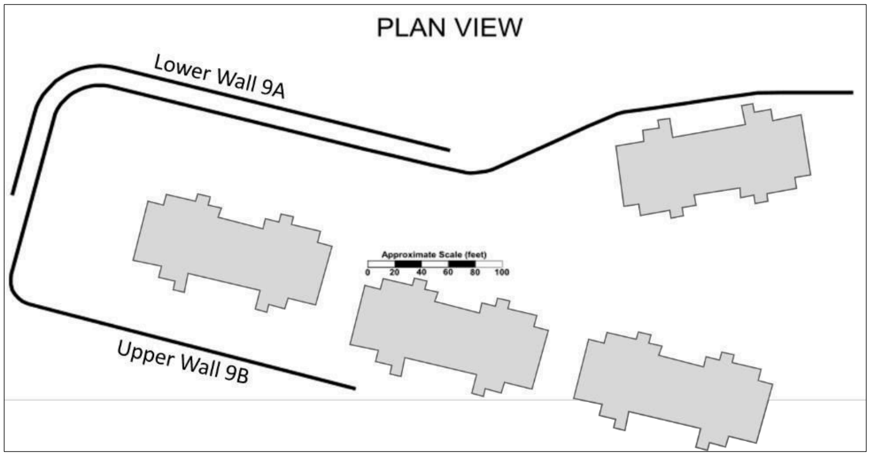

Figure 1. Site schematic layout showing Walls 9A ("Lower Wall") and 9B ("Upper Wall")

Geotechnical and Groundwater Conditions

Based on the review of the design geotechnical report for the subject wall, the geologic materials that formed the foundation for the MSE/SRW walls consist of weathered in-place residual soil overlying bedrock. Depending on the building location, the bedrock was reported to be of igneous, metamorphic, or sedimentary type materials that have been significantly distorted by tectonic movements. The residual soils overlying the bedrock were observed to be predominantly clay.

As part of the design study of specifically Walls 9A and 9B, six soil borings were performed to depths below ground surface of 28.5 feet. Multiple other borings were performed beneath the other proposed walls and buildings. These explorations indicated that weathered rock was as deep as 5 to 10 feet below the natural predevelopment grade. Moist materials were found in the design soil borings, and a groundwater table was not encountered within the depths explored.

Wall Design and Construction Specifications

As indicated in Figure 2, the key design elements captured in the construction drawings and specifications are as follows:

- *Backfill:* Use a maximum particle size of 3 inches installed at a maximum 8-inch lifts compacted to a minimum of 95% standard compaction (ASTM D698). No roots or wood are allowed.
- *Foundation:* A minimum pad thickness of 8 inches of crushed rock, gravel, or compacted sand should be installed.
- *Geogrid Reinforcement:* Geogrid should be taut. A minimum vertical spacing of 3 inches should be maintained between overlapping geogrid. Geogrid shall be rejected if 20% or more of a structural rib has been cut or ripped. The principal direction of weave should be perpendicular to the block wall. Geogrid should be flat and level with 100% of grid coverage in reinforced backfill zone. The geogrids specified were Highland II, IV, and

VIII (currently discontinued). They shall meet the specification requirements published by Ridgerock, the supplier of the segmental retaining wall units and the geosynthetic reinforcement.

- *Facing Blocks:* Blocks should be level. Adhesives are allowed for use with cap units only, not face blocks. No provision for the use of partial blocks.
- *Drainage System:* Drainage pipe should be installed behind the wall, 12 inches above the base. Twelve to 36 inches of minimum free draining granular material should be placed behind wall and the base.

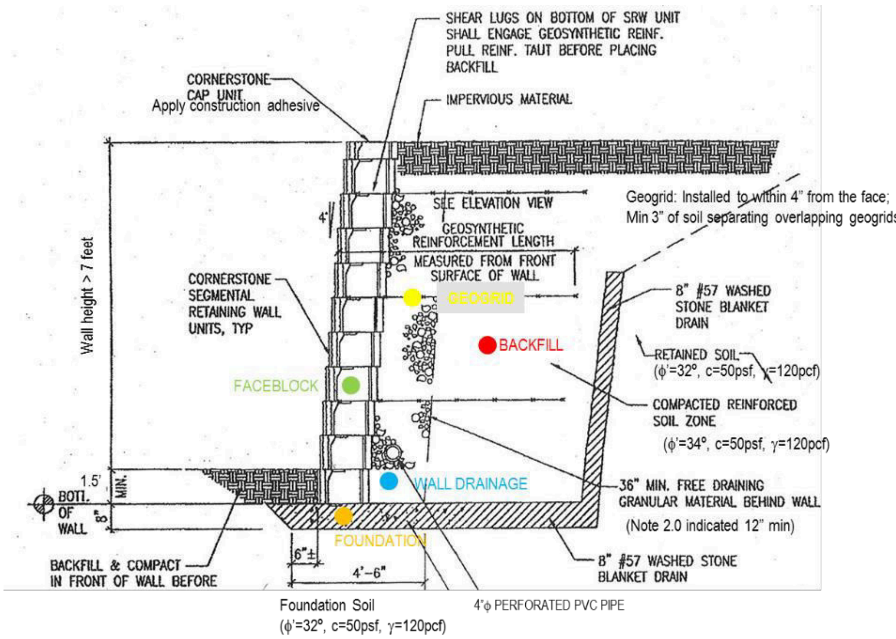

Figure 2. Typical MSE/SRW cross section. Noted points indicate features evaluated during the forensic investigation.

Construction Timeline

The construction of the Lower Wall started on May 2010. Construction of the upper wall commenced on July 2, 2010, following completion of the Lower Wall, as planned. The Upper Wall was completed around August 2010.

Distress Observations

Based on a June 24, 2010 field note, cracks in the facing blocks on the Lower Wall were noticed early in the construction process, before completion of the Lower Wall. The Lower Wall continued to deform during construction of the Upper Wall. In response, crack gages were installed at select locations on the facing of the Lower Wall on July 26, 2010 to monitor movements and assess changes in wall conditions.

Cracking of the concrete facings of the Lower Wall continued to worsen and facing blocks on the Upper Wall also began to crack. Example photos of these cracks are shown in Figures 3a and b. By January 2011, multiple cracks were conspicuous along the facing blocks and the Lower Wall was noticeably bulging as shown in Figure 3. Differential settlement of both walls was visible. In response, Buncombe County building officials issued a stop work order on

January 4, 2011.

In January 2011, differential settlement of the wall was observed. For example on January 20, 2011 a differential settlement of the Upper Wall of approximately one foot between stations 6+50 and 7+25 was observed. Four inclinometers were installed in multiple locations both within and outside the reinforced zone. The results of inclinometer readings showed a continuous accumulation of lateral movements. For example, one inclinometer, in near proximity to the face of the wall, recorded accumulated lateral deflections of 4.5 inches incrementally increasing from May 4, 2011, when the inclinometer was initialized, to August 25, 2011. These movements were increasing with the height of the wall. Between May and August 2011, the maximum inclinometer differential horizontal movement was approximately 4 inches.

In late 2011, the Developer decided that a portion of the Upper Wall would be demolished and the development plan modified to accommodate this site change.

(a) (b)

Figure 3. Cracks in and widened separations between facing blocks: a) Northeast corner showing Upper and Lower Walls (photo date: 09/29/2011); and b) Southwest corner of Lower Wall (photo date: 12/02/2011).

OBSERVATIONS AND MEAUREMENTS

The field investigation during disassembly of the wall included detailed observations and measurements of: 1) backfill; 2) foundation; 3) geogrid reinforcement; 4) facing blocks; and 5) drainage system. These keys areas are highlighted in Figure 2.

Backfill

As part of the authors' field investigation of the backfill zone, 13 test pits and 13 sand cone field density tests were performed and in the process, 60 geogrid samples, 15 bulk soil samples and 4 masonry block samples were collected. Laboratory analysis consisted of moisture content testing (per ASTM D2216) of samples collected in sand cone testing (ASTM D1556) and standard compaction testing (ASTM D698) on collected bulk samples. Table 1 shows a summary of the field dry unit weights and the percentage compaction based on the measured compaction tests. Density measurements showed that the in situ fill density was well below the design specification of 95% relative compaction. Moreover, a significant percentage of fines was found in the samples. Five out of six samples tested were found to have more than 20% fines. In multiple locations, cobbles and boulders were encountered within the backfill (Figure 4). In some cases, boulders as large as 29 inches were observed. Large boulders are known to severely damage the geogrid during construction and cause difficulties with achieving the required backfill compaction. In other locations, roots were observed (e.g., STA 7+80). The conditions of poor compaction and backfill, not in compliance with material specifications, appeared to be the result of poor workmanship and inadequate inspection.

Table 1. Results of sandcone and gradation measurements

Sample	Location	Dry Density (pcf)	Relative Compaction (%)	% Passing No. 200 Sieve
EXC-1	N/A	105	79	-
EXC-2	N/A	111	84	-
EXC-3	N/A	112	84	-
EXC-4	N/A	101	76	-
EXC-5	N/A	119	90	-
EXC-6	5+75 (9B)	113	86	-
EXC-7	5+75 (9B)	132	99	18
EXC-8	4+00 (9B)	127	95	-
EXC-9	5+00 (9B)	121	91	24
EXC-10	7+80 (9B)	114	86	27
EXC-11	7+80 (9B)	116	87	31
EXC-12	2+90 (9A)	111	84	26
EXC-13	6+50 (9B)	113	85	29

*Using a maximum corrected dry density of 132 pcf from Exponent's sample, as tested per ASTM D698.

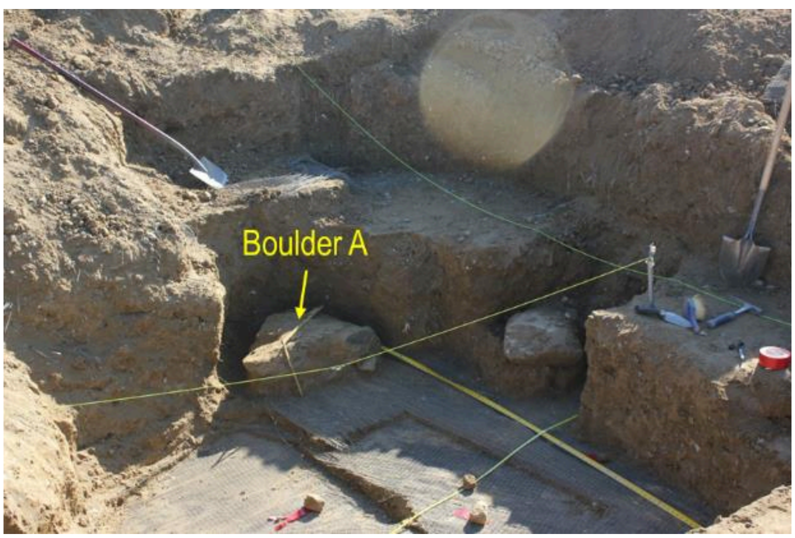

Figure 4. Cobbles and boulders in reinforced zone backfill. "Boulder A" has maximum triangular dimensions on sides of 28"x29"x23" and a height of 10" (photo date: 12/10/2011).

Foundation

The foundation of the Upper and Lower Walls were exposed by excavations and test pits in many locations (Figure 5). It was observed that the leveling crushed stone pad was measured to