Difficult Geologic Conditions Mandate Retaining Wall Redesign

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ABSTRACT

This paper discusses a case history where hard boulders embedded in a residual clay matrix prevented completion of a top-down retaining wall. Observations of soil conditions in excavations during construction allowed an indirect estimate of soil shear strength properties, which were used to analyze the stability of an approximately 27-foot deep excavation with a vertical side. This excavation was necessary to construct a concrete cantilever retaining wall as an alternative to the top-down wall. The design of the cantilever wall within the limited right-of-way is discussed. The construction monitoring program used to verify soil properties and protect workers is also discussed.

INTRODUCTION

This case history involves the widening of the Outer Loop of Interstate Highway 695 (I-695), northwest of Baltimore, Maryland, south of the interchange with I-70. The existing Outer Loop of the Baltimore Beltway was two southbound lanes and the project required widening it to three lanes. It also included construction of several retaining walls due to the topography, which sloped upwards from the edge of the highway in some locations along the alignment. To widen the highway, retaining walls were required for grade separation because the resulting ground surface would have been too steep without them.

One such retaining wall, RW-8, was designed as a top-down cantilever wall in which the contractor was required to drill 3-foot diameter holes with a drilled shaft rig, place a specially fabricated soldier pile inside the hole, fill the hole from the soldier pile tip elevation to the bottom of the pavement with reinforced concrete, excavate the ground in front of the wall to the back flange of the piles, slide pre-cast concrete panels between the flanges of adjoining piles, fill the zone between the native ground and the back flange of the pre-cast panels with drainage material and construct a cast-in-place reinforced concrete facing on the front side (highway side) of the wall.

Retaining wall RW-8 was 688 feet long and required 87 drilled shaft excavations spaced 8 feet apart. The wall height ranges from 3.5 to 20 feet.

Excavation of the drilled shafts became a problem that increased with difficulty as the project progressed, eventually requiring the contractor to develop an alternative approach to complete the project. Obstructions in the ground in the form of boulders hindered the drilled shaft subcontractor in advancing the excavations to the design elevations.

This paper describes the alternative approach taken when the ground conditions prevented the original design from being constructed.

SITE AND GEOLOGIC CONDITIONS

The site topography can be described as rolling. Within the limits of RW-8, the highway grade rises gently to the north, but the ground on the outside of the pavement slopes upward about 20 to 25 feet. Thus, the highway widening required a wall to maintain a manageable slope outside the highway limits. Figure 1 shows a section view of the proposed top-down wall and the ground surface conditions.

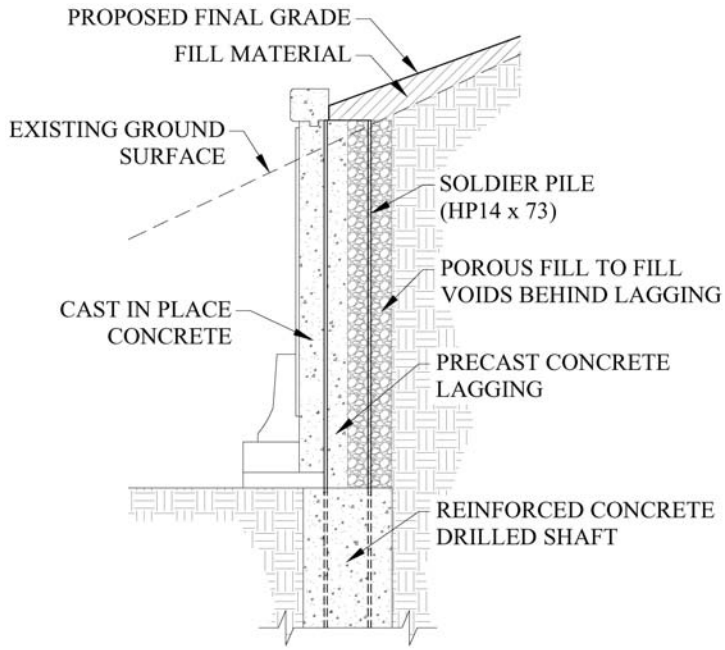

Figure 1. Top-down retaining wall section.

The property line is 18 feet from the front face of the wall and there were no opportunities to obtain a temporary easement for construction of an alternative wall type due to historical properties adjacent to the proposed wall. A fence was located along the right-of-way, separating the highway from an asphalt driveway and parking lot.

The site is located in the Piedmont Region and has residual soils resulting from the chemically weathering of the native bedrock. Bedrock at this site is amphibolite and soil is silty clay.

Nine test borings were drilled near the proposed RW-8 alignment during design. In general, the boring logs show soil above the pavement elevation that consists of stiff to hard silty clay. The boring logs show that the conditions beneath the pavement elevation consist of silty clay with layers of amphilobite. The drillers collected core samples of the amphilobite and the logs note the presence of soil seams in some of the core samples. At some boring locations, rock was penetrated with the split spoon sampler, revealing saprolite, a very weathered rock. A soil sample collected during construction from an excavation had a liquid limit of 96 percent and plastic index of 46. The borings extended to below the tip of the drilled shaft elevations. The top of competent rock elevation varies erratically between borings.

Groundwater levels were measured in the borings at various times following completion, but installation of piezometers was not included in the subsurface exploration program. Water levels in the borings ranged from about 12 feet above to 15 feet below the pavement elevation. These data were insufficient to characterize the groundwater regime. During construction, water accumulated in the base of the retaining wall excavations, but was never observed seeping from the side slopes.

CONSTRUCTION ISSUES

Construction of the wall began in late 2003 with the drilled shaft excavations, but problems emerged from the outset. The drillers encountered hard boulders making progress much slower than anticipated. The caisson contractor attempted several techniques to advance the drilled shaft excavations including augering with different diameter augers, core barrel drilling, cluster drilling with small diameter percussion drills and down-hole hammer method. All proved largely unsuccessful due to boulders embedded in the soil matrix. The augers could not penetrate the boulders and coring through a mixed face (part of the base of the hole was soil and part was rock) at the base of the hole was not successful because the ground stiffness varied with the boulders present, causing the core barrel to wander off alignment.

The contractor's approach was then to advance the auger to refusal and then place a laborer inside the hole with a jack hammer and rock splitter to break and remove the boulders. The contractor then continued augering until refusal was encountered again. Then, the boulder breaking process was repeated. This significantly extended the time required to advance the holes such that by August 2004, excavation for the piles in a 142-foot length of the wall had not yet begun.

ALTERNATIVES ANALYSIS

The contractor considered other options for constructing the 142 feet of the retaining wall including a conventional cast-in-place concrete cantilever wall. However, it was considered that construction of this wall type would require installation of a sheet pile or soldier pile wall along the right-of-way to support the excavation.

Installation of piles was rejected because the boulders embedded in the soil matrix that hindered advancement of the drilled shafts would also prevent piles from being advanced. Additionally, lateral support necessary for the shoring was not feasible. Tiebacks could not be used because a temporary easement was unavailable. Rakers could not be used because they would interfere with the retaining wall construction.

The contractor had successfully completed drilling some excavations for the soldier piles and had placed them in the drilled excavations. It excavated the sloped soil in front of the soldier piles, extending the excavation with an unsupported vertical face to the back flange of the soldier piles, as shown in Figure 2 to allow installation of the concrete lagging between the soldier piles and drainage material behind the lagging. These excavations allowed an indirect measure of the soil undrained shear strength.

In two locations along the RW-8 alignment, the contractor had left excavations between the soldier piles for the lagging (similar to the conditions shown in Figure 2) open for several weeks before installing the lagging. The excavations appeared stable and the ground surface at the top of the excavations showed no indication of lateral movement, such as tension cracks or sloughing. The excavation height was 16.5 feet.

The contractor then excavated a test pit along the retaining wall alignment to observe the soil conditions and check the soil mass for features, such as fissures, slickensides, etc. that cold adversely affect the stability of a vertical excavation slope. The test pit revealed a silty clay matrix mixed with hard amphilobite boulders. The soil matrix was hard. A thumbnail could be indented about 1/16 to 3/16 inches into the soil matrix only with much difficulty. Peck (1974) relate penetration of the soil mass to unconfined compressive strength. The amount of difficulty observed at this soil mass indicates the unconfined compressive strength exceeds 4 tons per square foot (tsf).

Figure 2. Top-down wall before lagging installation.

The test boring logs along RW-8 were reviewed and the least favorable boring had an average Standard Penetration Test N-value of 15. Based on the Terzaghi and Peck correlation appearing in Department of the Navy (1971), the unconfined compressive strength could be inferred as 2 tsf. However, considering the Sowers and Sowers correlation for “clays of high plasticity” (the PI of a sample was 46), the unconfined compressive strength could be 3.7 tsf.

Construction of a cast-in-place concrete cantilever retaining wall would require an excavation along the property line that would be 26.8 feet high. Using the Taylor Chart, Peck (1974), for a vertical slope, the N_c term equals 3.85. Using an estimated unit weight of 125 pounds per cubic foot, and an unconfined compressive strength of 2 tsf, the factor of safety for a 26.8-foot vertical excavation is about 2.4.

The contractor left the test pit open and observed the response of the ground with time. The excavation side slopes were nearly vertical and remained stable for at least five weeks. No secondary features manifested themselves during this period.

The results of these observations and estimates of the soil undrained shear strength allowed a cast-in-place cantilever wall to be considered as an alternative because they indicated that the overburden could be excavated with a vertical side slope.

ALTERNATIVE WALL DESIGN

A conventional cast-in-place cantilever wall was proposed such with the identical facing and architectural elements as the specified top-down wall so that this portion of the wall would not be distinguishable from the highway. Many challenges faced the designers in that the wall height was specified and the alternative wall needed to fit within the restricted space behind and in front of the wall.

The alternative wall was designed following conventional approaches by achieving factors of safety against sliding and overturning specified by the American Association of State Highway and Transportation Officials (1996) and for bearing capacity. Drained soil strength parameters were used for design. The soil mass was assigned an angle of internal friction of 28 degrees, based on the Atterberg limit data. Many iterations were necessary to achieve the specified factors of safety and fit the cantilever wall within the limited space. The width of the toe was limited by the specified location of a pavement drain that needed to be installed in front of the toe. The heel needed to be limited in width to keep the theoretical failure plane within the backfill to the extent possible. This resulted in an unusual configuration having a relatively wide toe and a narrow heel.

The designers elected to include a shear key to assist with sliding resistance and specified a uniformly graded crushed stone for backfill. This material was estimated to have an angle of internal friction of 38 degrees and an estimated unit weight of 100 pounds per cubic foot, Alva (1981). This friction angle placed a majority of the theoretical failure plane within the backfill. The high friction angle and low unit weight also reduced the lateral forces on the wall. Figure 3 shows a typical section of the alternative wall.

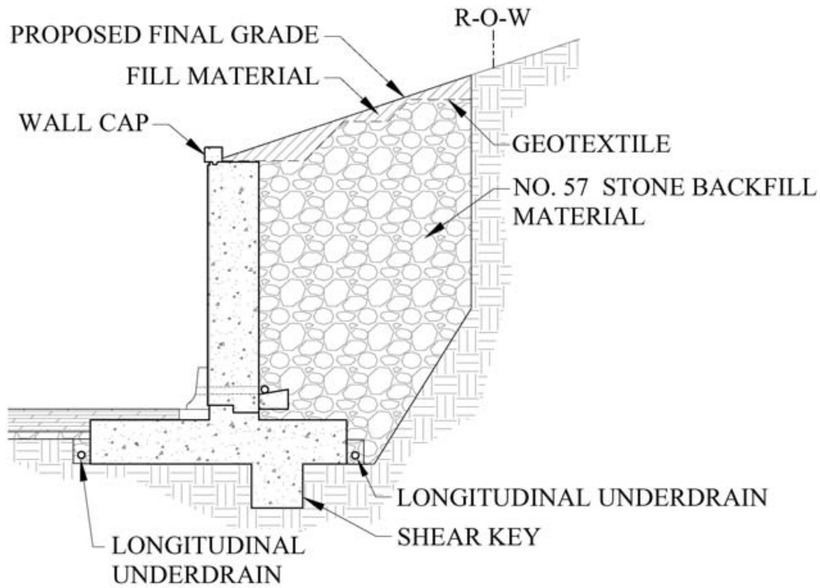

Figure 3. Alternative retaining wall section.

Although the estimates of the soil shear strength indicated that the excavation slopes would remain stable for the duration of construction, the designers elected to divide the length of the cast-in-place wall into four approximately 35-foot sections for construction. The first section was excavated and the wall was built and backfilled before the adjacent wall section excavation was begun to take advantage of any soil arching that might exist between the wall sections to assist in maintaining the vertical slope stable. The construction sequence (see Figure 4) was:

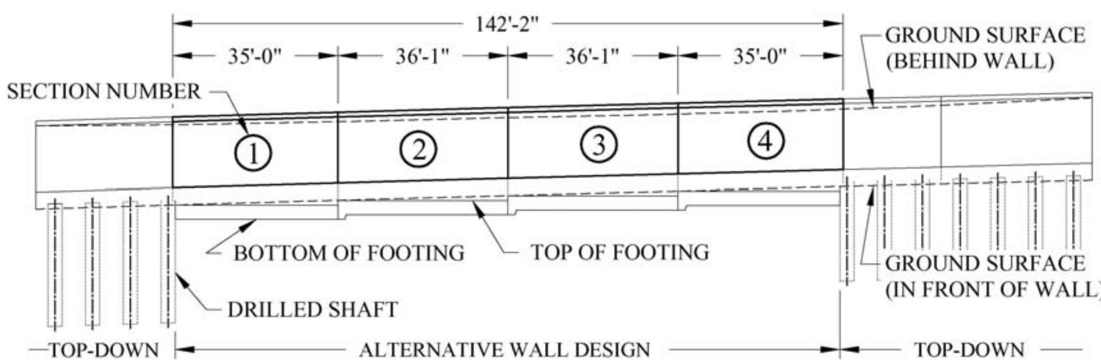

Figure 4. Alternative wall profile.

Excavate and build Section 1 (southernmost section)
Excavate and build Section 3
Excavate and build Section 2
Excavate and build Section 4

The contractor placed a Jersey barrier along the top of the slope, about 7 feet from the crest to prevent traffic from applying a surcharge to the top of the slope.

CONSTRUCTION OBSERVATIONS

Two concerns about this approach existed that led to a construction monitoring program. The primary concern was for the safety of workers building the wall next to the vertical slope because of the potential exposure of workers to falling soil clods and/or cobbles from the excavation face. Another concern existed for a general slope failure into the excavation. This would damage adjacent property and could injure workers and the general public using the facility at the top of the slope. A landslide was considered possible if soil conditions (i.e., soil strength and/or soil composition) or groundwater conditions (e.g., seepage through the excavation side slopes) along the alignment differed from those observed in excavations and test pit. Therefore, a monitoring program was initiated to address these issues.

A series of frequent observations of the ground behind the excavation were initiated. These included measuring the position of six hubs installed in the ground behind the excavation slope and monitoring the width of cracks in the pavement at the top of the slope that had existed before construction. Movement of the hubs or opening of the cracks would alert the contractor that the excavation slope was behaving differently than planned.

Since the excavations were made during the dry fall weather when the soil would dry, slake and fall into the excavation, the condition of the excavation face was inspected twice daily, at the start of work and following lunch. The inspector used a man-basket on a crane to inspect the slope condition and to scale any loose soil clods or rocks from the excavation face.

Additionally, the contractor placed a tarp over the slope during non-working hours to slow drying (and slaking) of soil on the excavation face and to prevent erosion into the excavation. This tarp was removed during work periods so the slope could be inspected.

Figure 5 shows the excavation of Section 1 in progress and the boulders removed from the excavation. This amount of boulders was the cause for the drilling difficulties.

CONCLUSIONS

This case history demonstrates the critical importance of accurately characterizing subsurface conditions. In this case, test pits could have proved useful for identifying the boulder condition. Test borings often do not accurately identify the extent boulders embedded in a soil matrix.

Drilled shaft equipment can successfully excavate through soil with augers and through rock with core barrels (or rock loosening tools), but is ill-suited to excavate soils laden with hard boulders.

Figure 5. Excavation of Section 1.

The solution to the construction problem at this site involved the observational approach. Utilizing observations of soil behavior in excavations and a test pit together with test boring data allowed the concept of an approximately 27-foot high excavation with nearly vertical side slopes to be considered. Once the feasibility of such an excavation was demonstrated, a monitoring program was initiated to check that the estimates made during design were valid. The design estimates proved valid and the alternative wall design was successfully constructed.

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