FIRST APPLICATION OF ARTICULATED CONCRETE BLOCK ARMORING WITH A STABILIZED STONE DRAINAGE LAYER

JEREMY R. YOUNG, PE[1] AND JIM NADEAU[2]

[1]Schnabel Engineering
12301 Research Blvd, Suite 150
Austin, Texas 78759
e-mail: jyoung@schnabel-eng.com

[2]Ferguson Waterworks/ACF Environmental
2831 Cardwell Road
Richmond, Virginia 23234
e-mail: jnadeau@ferguson.com

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Summary. Results from recent hydraulic testing of Articulated Concrete Block (ACB) systems have shown variations in the level of the ACB surface is attributed to the movement of an unconfined stone drainage layer. This movement appears to be exacerbated by thicker drainage layers, higher overtopping depths, and longer test flumes. While the apparent movement did not constitute "threshold of performance" as currently defined by ASTM D7276 and D7277, there was a desire to mitigate movement of the stone drainage layer and maintain conservatism in ACB design and construction. This paper will review the findings of Nadeau and Wedin's paper from Protections 2018, which proposed an ACB system with a stone drainage layer stabilized with geocell to increase ACB performance under steady state and hydraulic jump conditions. A similar ACB system was recently designed and constructed to provide overtopping protection at Hollymead Lake Dam located near Charlottesville, VA. The system was selected for added conservatism to address overtopping depths approaching 1.6m (5.4-ft), hydraulic jump conditions occurring on the downstream slope, and other complex flow conditions due to site geometry. The authors believe this to be the first application of its kind. Design and analysis of the Hollymead Lake Dam ACB will be summarized and include a comparison of the factor of safety with and without the geocell stabilized stone drainage layer. The project was completed in November 2019. Details from construction will also be presented.

1 INTRODUCTION

An ACB system is a matrix of interconnected concrete block units installed to provide an erosion-resistant lining with specific hydraulic characteristics. The connection between the individual ACB units is by geometric interlock and/or cables. The term "articulated" describes the ability of the matrix to confirm to changes in terrain while maintaining geometric interlock. Typical installation of a cable-tied mattress of ACB is shown in Figure 1.

FEMA (2014) suggests that ACBs are appropriate to armor embankments with heights less than about 12 m (39-ft) and for design overtopping depths of less than 1.3 m (4.3-ft), based on experimental test data available at that time, although heavier blocks are currently being developed and tested that will likely increase those limits. ACBs are also typically limited to flow velocities of less than 7.6 m/s (25 ft/s), unless otherwise validated by flume testing performed by the manufacturer.

Figure 1. ACB installation.

ACB systems for embankment overtopping applications must include a properly designed filter system to permit free drainage of seepage exiting the subgrade while preventing loss of subgrade materials. The filter typically consists of a non-woven geotextile placed over a soil subgrade. Typical ACB systems also include a stone drainage layer directly under the ACB to prevent uplift pressures from developing below the ACB during spillway flow. The drainage layer also acts as ballast to hold the geotextile in intimate contact with the subgrade. A geogrid layer is often provided as a confining layer between the stone drainage layer and the ACBs to prevent loss of stone particles through the openings in the ACBs.

By definition, failure of the system occurs when ACBs lose intimate contact with the subgrade. This can occur when high-pressure lift in the subgrade exceeds the sum of the ACB mass and low pressure in the high velocity flow above the ACB surface. This condition will cause the ACB to lift and initiate erosion of the stone drainage layer. Therefore, confinement of the stone drainage layer is critical to the safe performance of an ACB system.

2 EFFECTS OF A STABILIZED STONE DRAINAGE LAYER ON ACB PERFORMANCE

Movement (more than 2.5-inches) of ACB blocks with stone drainage layers was first documented during full scale flume testing in 2013 (Thornton et al., Armortec 40-T Testing Steady State Overtopping Flow Conditions, unpublished proprietary report, 2013); however, the initiation of erosion below the blocks had not been observed. While this did not meet the “threshold for performance” as defined by ASTM D7276 and D7277, the displacement could result in projections in the blocks which would adversely affect overall performance of the ACB system. A proposed solution to correct the issue was developed, tested, and presented at Protections 2018 (Nadeau and Wedin, 2018).

The improvements included confining the stone drainage layer with 3-dimensional transfer platform consisting of geotextile, geo-cell, and geogrid which function as a single entity. The geocell is a matrix of open cells with rigid walls that are designed to be filled with the stone drainfill as shown in Figure 2. ACB is mounted on top of the cellular confinement layer.

Figure 2. Geocell stabilized drainfill.

A detail illustrating the components of typical ACB system with a stabilized stone drainage layer is shown in Figure 3.

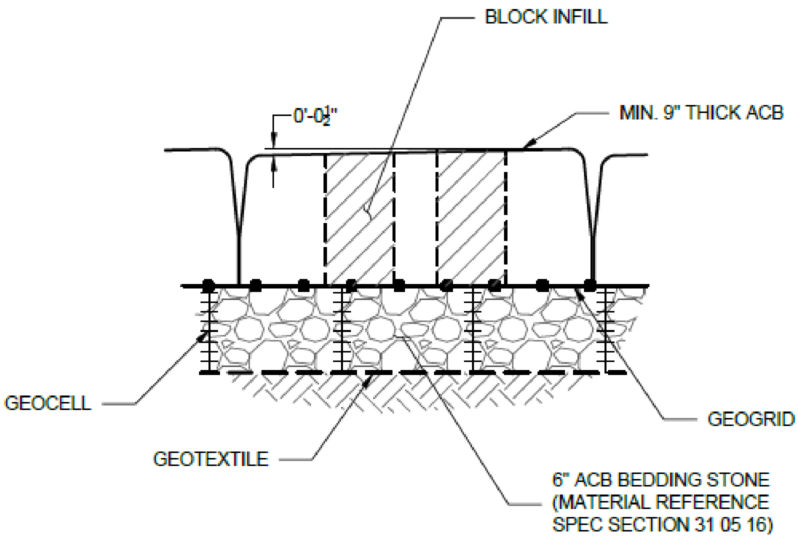

Figure 3. ACB System Detail (flow left to right).

Nadeau and Wedin showed that the stabilized stone drainage layer produced negligible ACB block movement, which leads to reliable and significant increases in ACB performance under overtopping flow and hydraulic jump conditions. Nadeau and Wedin also speculated that ACB systems with a stabilized stone drainage layer may also increase the potential the range of applications for complex site conditions.

3 HOLLYMEAD LAKE DAM CASE STUDY

3.1 Introduction

The hydraulic conditions and site configuration for the Hollymead Lake Dam Spillway Improvement Project would not have led to an adequate factors of safety or hydraulic jump stability for ACB just a few years ago; however, due to updated full scale flume testing and development of an ACB system with a stabilized stone drainage layer, the use of ACB was found to be a viable option to update spillway capacity in terms of performance, aesthetics, constructability, and economics. The following sections present a summary of the design and construction of the ACB overtopping project at Hollymead Lake Dam, which was believed to the first use ACB with a stabilized stone drainage layer.

3.2 Site Description

Hollymead Lake Dam is located on a tributary to Powell Creek in a residential area, about eleven kilometers (seven miles) north of Charlottesville in Albemarle County, Virginia. The dam was originally constructed in 1974 and is currently owned by Albemarle County (County). The watershed is a combination of suburbs, woods, and pasture, with a drainage area of about 3.67 km^3 (1.42 square miles (mi^2)). The dam is an earthen embankment approximately 13 m (43-ft) high at its maximum section, 90 m (300-ft) long. A local road (Timberwood Parkway) serves as the crest of the dam. The dam impounds about 287,000 m^3 (226 acre-ft) of water at normal pool. The principal spillway consists of a concrete riser with a 64 cm (24-inch) diameter reinforced concrete pipe (RCP) with a series of manholes drops before transitioning to a 1.4 m (54-inch) diameter RCP outlet conduit. The dam also has a 1.2 m (48-inch) diameter RCP set above normal pool which serves as an auxiliary spillway. The auxiliary spillway conduit includes a series of manhole drops before connecting to the 1.4 m (54-inch) diameter outlet conduit which discharges into a small plunge pool. Critical project elevations are summarized in Table 1.

Table 1. Critical project elevations.

Feature	Elevation	
	Feet	Meters
Top of Dam (non-level)	EL 440.2 – 448.3	EL 134.2 – 136.6
Principal Spillway Crest (Normal Pool)	EL 426.4	EL 128.8
Auxiliary Spillway Invert (at inlet)	EL 427.1	EL 130.2

4 DAM SAFETY EVALUATIONS

4.1 2013 Dam Breach Analysis

In 2013, Schnabel Engineering (Schnabel) performed a dam breach analysis and inundation mapping for Hollymead Lake Dam using one-dimensional unsteady flow routing (full Saint Venant equations) in the U.S. Army Corps of Engineers (USACE) HEC-RAS software, and recommended that the hazard potential classification be changed from significant to high due to downstream development. The results from the 2013 study also indicated that the dam had inadequate discharge capacity to safely pass the spillway design flood (SDF) required by Virginia Department of Conservation and Recreation (DCR) dam safety regulations. The SDF required for existing high hazard impounding structures is 90 percent of the Probable Maximum

Precipitation (PMP), unless a lesser flood can be justified through an Incremental Damage Analysis. The County selected the full PMP as the SDF since the incremental cost to upgrade spillway capacity was not considered significant.

4.2 2015 Evaluation of Upgrading Alternatives

In 2015, Schnabel evaluated various upgrading alternatives to safely pass a flood resulting from the PMP:

- Alternative 1 – Raise top of dam
- Alternative 2a – Armor embankment with articulated concrete blocks (ACB)
- Alternative 2b – Armor embankment with roller compacted concrete (RCC)
- Alternative 3a – Raise top of dam and construct a box-inlet drop spillway with box culvert chute, and stilling basin
- Alternative 3b – Raise top of dam and construct a labyrinth spillway, chute, and stilling basin with a vehicle bridge over the spillway

4.3 2016 Dam Rehabilitation Design

The County ultimately elected to armor the embankment with ACBs (Alternative 2a) to prevent failure of the dam during an overtopping event, which is considered an acceptable and common means of passing the SDF by DCR. The rehabilitation design generally consisted of the following components:

- Armoring the downstream slope and toe with ACB (covered with topsoil and permanent turf). The crest of the dam (Timberwood Parkway) was not armored with ACB.
- The ACB apron at the downstream toe of the embankment slope was designed to contain the hydraulic jump resulting from 50 percent of the Probable Maximum Flood (0.5 PMF). For larger overtopping events, damage to the ACBs at the toe could occur and may require replacement or repairs; however, the embankment will remain protected.
- The upstream edge of the overtopping protection includes an ACB turn-down covered with mass concrete. The mass concrete was intended to stabilize the crest ACBs while providing protection near existing guide rail posts, which could potentially create turbulent flow conditions at the crest edge. Damage to the asphalt road and guardrails could be expected during an overtopping event; however, the embankment will remain protected.
- The downstream edge of the overtopping protection also includes an ACB turn-down to protect against scour that may occur downstream of the apron. Additionally, the turned down ACBs will allow the drain fill beneath the ACB blocks to outlet unimpeded.
- Construction of cast-in-place concrete cantilevered retaining walls on the downstream edge of the crest to confine overtopping flows to the armored section. The walls include shallow spread footings buried below frost depth.
- Replacing sections of the existing principal and auxiliary spillway conduits that exhibited cracking and open joints. Remaining conduit sections were repaired with a cure-in-place pipe lining and joint repairs. A filter diaphragm was also installed around each conduit.
- Replacing the existing principal spillway riser with a new concrete riser.

Details illustrating the design concept are shown in Figure 4, Figure 5, and Figure 6.

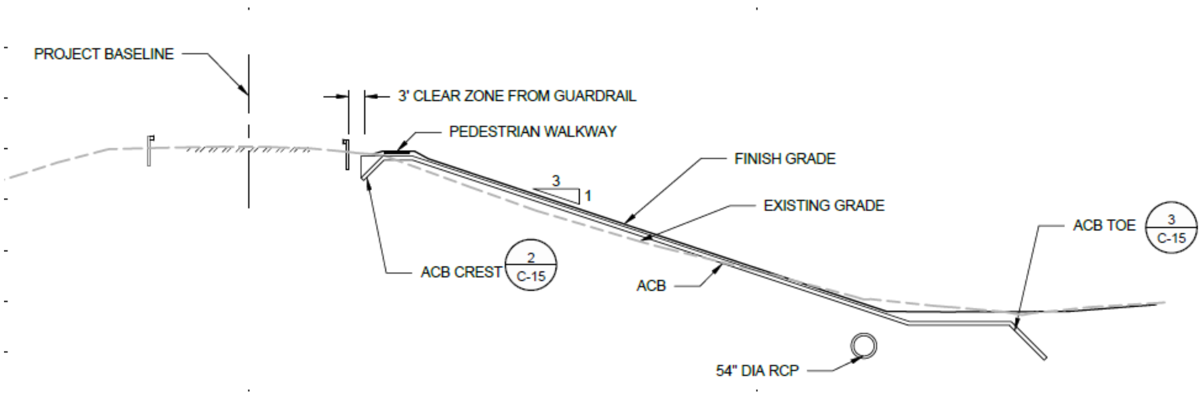

Figure 4. Typical embankment section.

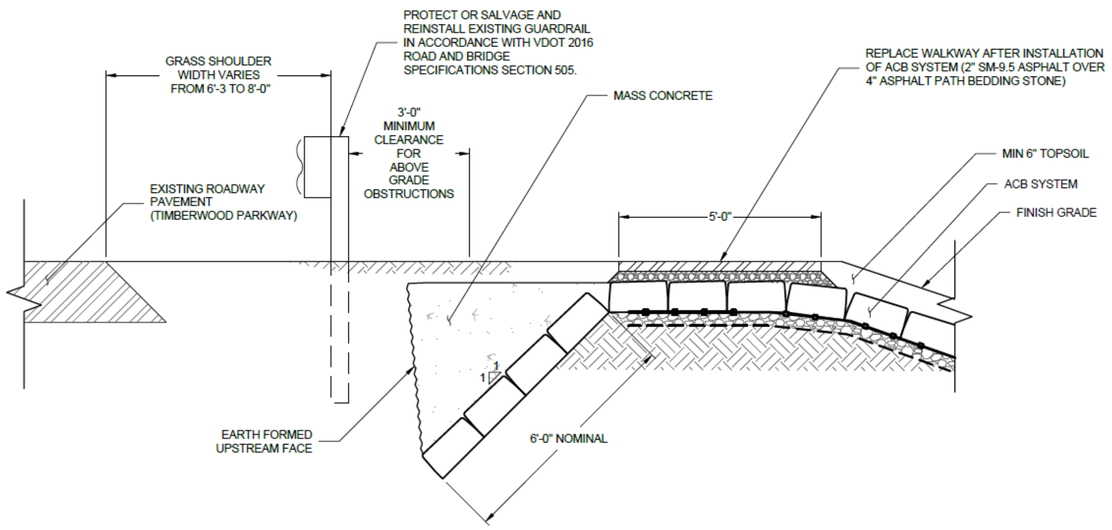

Figure 5. ACB crest detail (flow left to right).

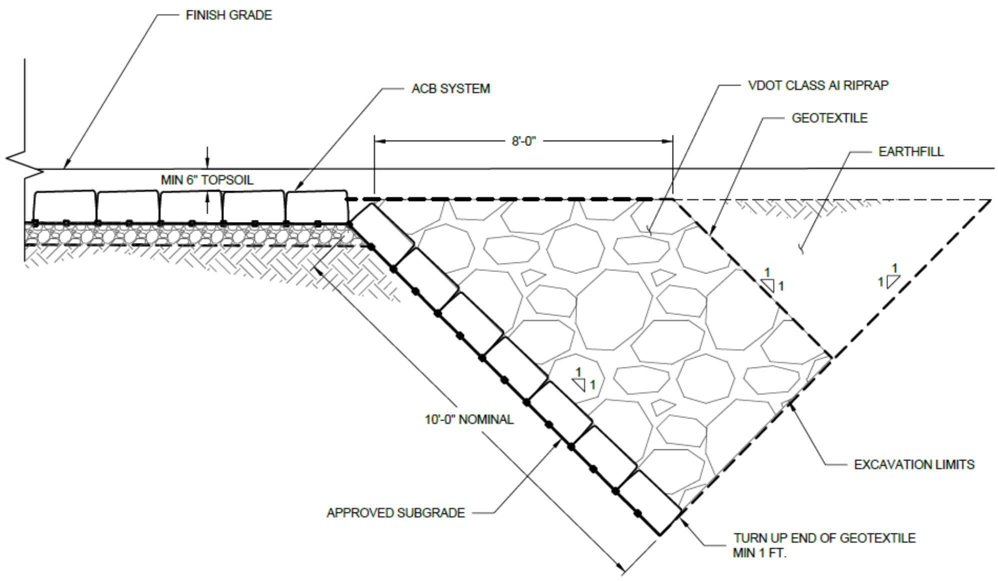

Figure 6. ACB toe detail (flow left to right).

5 ACB DESIGN

5.1 Hydrologic and Hydraulic Analysis

Rainfall-runoff computations and reservoir routing were performed for existing conditions using the USACE HEC-HMS software for several storm events, including the 24-hr PMF which is approximately 81 cm (32-inches) of precipitation. The existing conditions models considered a non-level top of dam, as well as the principal and auxiliary spillways. The proposed conditions were modeled essentially the same, although the non-level crest was truncated to account for the proposed crest walls to confine flow to the ACB armored section. The results of the HEC-HMS modeling for existing and proposed conditions are summarized in Table 2 and Table 3.

Table 2. HEC-HMS results – Existing conditions.

Storm Event	Peak Inflow m^3/s (cfs)	Peak Outflow m^3/s (cfs)	Peak Stage EL Meters (Feet)
2-Year	5.9 (210)	1.2 (41)	130.3 (427.5)
10-Year	18.8 (663)	3.1 (110)	130.2 (430.5)
50-Year	33.1 (1170)	5.1 (180)	132.3 (434.0)
100-Year	40.2 (1420)	5.9 (210)	132.8 (435.8)
500-Year	57.5 (2030)	7.6 (270)	134.2 (440.4)*
PMF	202 (7140)	195 (6900)	135.7 (445.1)*

* Embankment Overtops (low point in Crest is EL 134.2 m (440.2 ft)).

Table 3. HEC-HMS results – Proposed conditions.

Storm Event	Peak Inflow m^3/s (cfs)	Peak Outflow m^3/s (cfs)	Peak Stage EL Meters (Feet)
2-Year	5.9 (210)	1.0 (37)	130.3 (427.6)
10-Year	18.8 (663)	3.1 (110)	131.2 (430.5)
50-Year	33.1 (1170)	5.4 (190)	132.3 (434.0)
100-Year	40.2 (1420)	5.9 (210)	132.8 (435.8)
500-Year	57.5 (2030)	7.6 (270)	134.2 (440.4)*
PMF	202 (7140)	193 (6800)	135.8 (445.6)*

* Embankment Overtops (low point in Crest is EL 134.2 m (440.2)).

The results indicate that the hydraulics of the proposed spillways closely match the existing hydraulics. Overtopping of the embankment is expected for events approaching the modeled 500-year storm or approximately 23 percent of the PMF. The maximum overtopping depth at the crown of the road increases from 1.5 m (4.9 ft) for existing conditions to 1.6 m (5.4 ft) for proposed conditions. The estimated duration of overtopping for the PMF is about 8 hours for proposed conditions. The crest walls were designed to tie out to EL 135.9 m (446.0 ft) to contain overtopping to the armored sections of the embankment and allow for nominal freeboard.

5.2 ACB Layout, Stability, and Apron Sizing

The armored embankment was designed to have a subtle trapezoidal shape in cross-section. This shape was selected to generally match the crest road (Timberwood Parkway) grades that form the top of dam and to create a relatively uniform flow area. The side slopes of the armored embankment were designed to contain the computed PMF flow depth along the downstream slope. The ACB apron layout (length and side elevations for containment) were based on hydraulic jump calculations discussed below.

The ACBs were designed to meet stability criteria for the PMF loading condition. Two methods of evaluating the stability of the ACB system were initially considered: the National Concrete Masonry Associations (NCMA) Method outlined in *Design Manual for Articulating Concrete Block (ACB) Revetment Systems* (NCMA, 2010) and the CSU method developed by Amanda Cox, PhD, of Colorado State University outlined in *Moment Stability Analysis Method for Determining Safety Factors for Articulated Concrete Blocks* (Cox, 2010). The NCMA method represented the state of practice for estimating ACB stability at the time of design, while the CSU Method was generally considered to be more robust and selected by the design team.

The CSU method more accurately considers lift and drag in the factor of safety equations, while the NCMA method does not consider any effects due to flow velocity, and boundary shear stress. The CSU Method has been since incorporated into Part 628 of the NRCS National Engineering Handbook, Chapter 54 – ACB Armored Spillways (NRCS, 2019), and is considered the current state of the practice. Factors of safety are calculated by using a moment stability analysis coupled with the computation of hydrodynamic forces including boundary shear stress and flow velocity. The required hydraulic inputs include shear stress, flow velocity, and bed slope, as well as physical characteristics of the proposed ACB block (i.e., dimensions and weight). The CSU Method also examines factors of safety for three possible rotations of ACB on the channel slope and one rotation for the channel bed. The lowest factor of safety controls design.

Hydraulic parameters used in the ACB stability analysis were obtained from a Computational Fluid Dynamic (CFD) model developed to evaluate complex overtopping flow conditions on the crest, downstream slope, and toe. The CFD model was a sectional model of the embankment and included upstream and downstream boundary conditions defined by the peak reservoir elevation computed in HEC-HMS and the tailwater elevations computed in HEC-RAS as part of the 2013 dam breach analysis, respectively. The hydraulic parameters used in the stability analysis are summarized in Table 4.

Table 4. Hydraulic parameters for stability analysis.

Storm Event	Shear Stress kPa (lb/ft^2)	Velocity m/s (ft/s)	Side Slope	Bed Slope
PMF	0.48 (10)	9 (30)	50H:1V	3H:1V

Various sizes of readily available standard ACB blocks were analyzed. As recommended in the NCMA Design Manual, "a target factor of safety of 1.3-1.5 is adequate for applications in which design hydraulics and site geometry are clearly understood, such as dam overtopping or spillway applications." Considering the complex flow conditions at the site due to the non-level top of dam, guardrails, and trapezoidal armored section, Schnabel selected a minimum FS of 2.0 for this project. Note that Part 628 of the NRCS National Engineering Handbook, Chapter 54 now has updated guidance for minimum recommended factors of safety. The block selected for design was the *Shoreblock EPEC SD-900 OCT*, a 23 cm (9.0-inch) thick open celled tapered block with a geocell stabilized based course, which was considered one of the most robust ACB systems available. The computed factors of safety for the proposed ACB with and without a geocell stabilized stone drainage layer is summarized in Table 5 for a comparison.

Table 5. Computed ACB stability factors of safety.

ACB	Stone Drainage Layer	Factors of Safety (FS)
Shoreblock EPEC SD-900 OCT	Stabilized	2.16
Shoreblock SD-900 OCT	Non-Stabilized	1.67

The proposed ACB armored apron at the toe of the embankment was sized so that the hydraulic jump occurs on the apron for the 0.5 PMF. Due to site constraints (existing 1.4 m (54-inch) diameter pipe), the lowest elevation that the apron could be set was approximately EL 124 m (406 ft). Hydraulic jump calculations were performed in general accordance with U.S. Army Corps of Engineers *Hydraulic Design Criteria Vol. 1 Section 124 – Spillway Stilling Basins* (USACE, 1977). The hydraulic jump length was computed based on hydraulic parameters on the embankment slope (flow depth, velocity, and Froude number) obtained from the CFD model, while the tailwater was based on the HEC-RAS model. The calculations indicated that approximately 6.1 m (20 ft) of apron is required to contain the hydraulic jump. The results of the CFD model appeared to generally match the computed hydraulic jump length. The elevation of the lateral tie-outs of the apron was set based on the computed hydraulic jump height for the 0.5 PMF.

It should be noted that overtopping of the road and embankment is anticipated to cause some damage, including erosion of topsoil overlying the ACBs and damage to the asphalt road and guardrails. For larger overtopping events, there may also be damage to the ACBs at the toe of the dam that would require replacement or repairs; however, the embankment is anticipated to remain protected, which is the primary objective for the project.

5.3 Geotextile Design

Schnabel performed an analysis to assess compatibility of proposed geotextiles used as a separation/drainage layer between the ACB system and base soil at Hollymead Lake Dam. The base soil was assumed to be represented by soil samples collected during a 2017 subsurface exploration. These soils were classified as silt with sand (ML) and lean clay with sand (CL), but because the following conditions outlined in *Design Manual of ACB Revetment Systems* (NCMA 2010) were met, a geotextile with an apparent opening size (AOS) of less than the #70 sieve (0.210mm) was required.

> *If the required AOS is smaller than that of available geotextiles, then a granular transition layer is required, even if the base soil is not clay. However, this requirement can be waived if the base soil exhibits the following conditions for hydraulic conductivity K, plasticity index PI, and undrained shear strength c: $Ks<1 \times 10^{-7}$, PI>15, c>10 kPa. Under these soil conditions there is sufficient cohesion to prevent soil loss through the geotextile. A geotextile with an AOS (Apparent Opening Size) less than a #70 sieve can be used with soils meeting these conditions*

6 MISCELLANEOUS DESIGN AND CONSTRUCTION CONSIDERATIONS

The following design and construction issues were also considered by the project team:

- **Control of Water** - Hollymead Lake was drawn down to the existing principal spillway conduit invert of EL 128.1 m (420.3 ft) throughout construction which significantly reduced seepage pressures and the need for an extensive excavation dewatering system on the downstream slope. Water was initially diverted through the existing auxiliary spillway conduit. After the work on the principal spillway riser and conduit was complete, the principal spillway conduit was used for diversion.

- **Maintenance of Traffic** – Partial closure of Timberwood Parkway was required to facilitate construction of the project. It was important to the surrounding community er to maintain traffic on Timberwood Parkway throughout construction (i.e., no road closure/ detour). One lane was closed during construction, but this complicated site access for the contractor, deliveries of construction materials, and ACB installation via crane.
- **Manholes** – The three existing manholes on the downstream slope modified slightly to gradually tie-in to the armoring to prevent turbulent flows due to abrupt grade.

7 CONSTRUCTION

Construction was completed between December 2018 and November 2019. The project was awarded to the lowest bidder for a lump sum contract in the amount of $2.33 mill. USD. Construction was completed with no change orders Selected photographs from construction are shown below in Figure 7 to Figure 9.

Figure 7. ACB drainfill placement.

Figure 8. ACB armoring.

Figure 9. Finished embankment covered with topsoil and seed.

8 SUMMARY AND CONCLUSION

Movement (more than 2.5-inches) of ACB blocks with stone drainage layers was first documented during full scale flume testing in 2013. While this did not meet the "threshold for performance" as defined by ASTM D7276 and D7277, the displacement could result in projections in the blocks which would adversely affect overall performance of the ACB system. A proposed solution to correct the issue was developed, tested, and presented at Protections 2018, which included confining the stone drainage layer with 3-dimensional transfer platform consisting of geotextile, geo-cell, and geogrid. The stabilized stone drainage layer produced negligible ACB block movement during flume testing, which leads to reliable and significant increases in ACB performance under overtopping flow and hydraulic jump conditions.

Based on the recommendations presented at Protections 2018, Schnabel Engineering evaluated ACB systems with and without a stabilized stone drainage layer for a dam rehabilitation project located near Charlottesville, Virginia. The results indicated that adequate factors of safety were achieved with the stabilized stone drainage layer, while ACBs systems with non-stabilized stone drainage layers did not comply with the required design criteria. The block selected for design was the *Shoreblock EPEC SD-900 OCT*, a 22.9 cm (9-inch) thick open celled tapered block with a geocell stabilized based course, which was considered one of the most robust ACB systems available. The system was selected for added conservatism to address overtopping depths approaching 1.6m (5.4-ft), hydraulic jump conditions occurring on the downstream slope, and other complex flow conditions due to site geometry. Construction at Hollymead Lake Dam was completed between December 2018 and November 2019, and is believe this to be the first application of its kind.

Following the construction of Hollymead Lake Dam, there have been several other ACB projects designed and constructed with a stabilized stone drainage layer, leveraging the system's robustness, and expanding the range of ACB applications.

Table 6. Selected ACB projects with stabilized stone drainage layers.

Project Name	Location	Year Completed
Hollymead Lake Dam	VA	2019
Richfield Dam		2020
Cabo San Lucas Drainage Channel	MX	Unknown
USACE Herbert Hoover Dike S-288		2021
Oxoboxo Dam		2021
USACE Herbert Hoover Dike		2022
NRCS Kintz Creek Dam	PA	Est. 2023
380 Town Center Dam	TX	Est. 2023
Brookside Detention Ponds	TX	Est. 2023
Lake Matoka Dam		Est. 2023
Hardy Dam	MI	Est. 2023
Anderson Dam		Est. 2023
Wheaton Branch Dam		Est. 2023
Puerto Nuevo River	PR	Est. 2024
NRCS Brush Creek Site 9	WV	Est. 2024
Fordville Dam		Est. 2024
Matejeck Dam		Est. 2025

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