Assessment of the stability of a century-old water supply dam in north-central Pennsylvania

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ABSTRACT: The Bradford No.3 Dam, located a few miles west of the City of Bradford, Pennsylvania, is a 47-foot high earth embankment that was constructed as a water supply impoundment in the late Nineteenth Century and is still used for that purpose today. Although the dam has served its purpose admirably over the past hundred years, its stability had not been formally evaluated nor had the potential for overtopping. This prompted a detailed assessment of the dam pursuant to upgrading the structure to meet state regulatory requirements. The consequent drilling and testing program to establish the types and properties of the embankment and foundation soils revealed soft zones within the embankment that were evidenced by Standard Penetration Test N-values near zero, accompanied by settlement of the drilling tools under their own weight. Difficulties experienced in procuring and testing representative "undisturbed" embankment samples prompted a program of dilatometer and borehole shear testing to more reliably define and characterize the soils. These in-place tests contributed greatly to a rational assessment of the stability of the dam embankment and to the design of cost-effective rehabilitation measures that are expected to extend the life of the dam for decades to come.

1. INTRODUCTION

The Bradford No.3 Dam has for over one hundred years impounded the flows of Marilla Brook to form the Marilla Reservoir, a twenty-acre lake that supplies water to the City of Bradford and provides recreational opportunities for fishing, canoeing, and hiking. The lake is located approximately two miles west of the City of Bradford in McKean County, situated in the Allegheny National Forest Region of north-central Pennsylvania, just south of the New York state line.

Owned and operated by the Bradford City Water Authority, the dam is a diaphragm-earth embankment structure that impounds approximately 500-acre-feet of water at normal pool (Figure 1). It was constructed in 1898-99 by a local contractor and placed in service in 1900. The Pennsylvania Division of Dam Safety classifies the dam as a "B-1", High Hazard "1" structure, the B-1 classification pertaining to dams that are 40-feet or more in height and the High Hazard "1" classification to structures whose sudden failure could result in substantial loss of life and excessive economic losses.

The dam has performed commendably over its first century of service. Maintenance has so far involved relatively minor issues, such as im-

Figure 1 Bradford No.3 Dam at Spillway

proving drainage in wet areas immediately downstream of the toe of dam, locally resetting stone on the upstream face of the dam, replacing wood planks in the spillway apron, and the like. Even so, no documentation concerning the stability of the dam was known to exist, and, given the age of the structure, none may ever have existed. Also, the spillway was undersized by today's standards, creating the possibility according to recent projections that the embankment might someday be overtopped (although so far that has never happened). These circumstances, along with recent attention to a wet zone and a localized surficial slip on the downstream face of the embankment prompted the Bradford City Water Authority, at the request of the Pennsylvania Division of Dam Safety, to assess the stability of the dam and to design and implement rehabilitation measures to bring the dam into compliance with current standards of the Commonwealth of Pennsylvania. The first writers' firm was contracted by the Authority to perform the assessment, design rehabilitation measures, and prepare the necessary technical specifications and drawings.

2. BACKGROUND ON THE DAM

The Bradford No.3 Dam is a 47-foot high, 770-foot long diaphragm-earth embankment, whose embankment faces slope at 2.H:1V (downstream) and 2.5H:1V (upstream), with the topmost six feet of the upstream face steepening to 1.5H:1V.

The spillway is a 58.6-foot wide stone masonry weir located near the left abutment, with a crest elevation approximately six feet below the top of the earth embankment. The principal outlet works consist of a 16-inch cast iron water supply line and a 20-inch diameter cast iron drawdown, or discharge, pipe. Control valves are located in a small building at the downstream toe of the dam.

An 1898 drawing provides the only information available concerning the internal structure of the dam. It indicates that the dam embankment was built of soil derived from a borrow area at the upstream end of the reservoir and, according to 19th Century boring logs, was founded on alluvial deposits of gravel, sand, and clay. "Selected" soil of specification no longer known was used to construct the core of the dam as well as the 8 to 20 foot thick wedge of soil forming the lower half of the upstream face. The core is 60-feet wide at foundation level narrowing to 12 feet at the top of the dam. Within the core is a stone masonry diaphragm (a two to six foot thick wall, narrowing to the top, and constructed of sandstone blocks and Portland cement mortar) that is located along the longitudinal centerline of the dam. It extends to within six feet of the top of the dam embankment and to a depth of nine feet below the original ground surface in an 8 to 12 foot wide trench at the base of the dam. Undifferentiated earth fill was used to construct the shell of the dam. Specifications for fill placement and compaction are unknown.

The upstream face of the dam is armored with tabular slabs of sandstone that have been laid side-by-side, edge-to-edge on the sloping embankment face. Rip rap is reportedly present on the upstream face of the dam at the toe but is out of view below pool level.

The 1898 drawing shows no drainage blanket beneath the embankment downstream of the core.

3. INITIAL SUBSURFACE INVESTIGATION

To achieve a better understanding of site conditions, a traditional subsurface investigation was undertaken that consisted of:

a) Drilling a series of test borings through the embankment and into the foundation, terminating in the alluvial deposits 15 to 25 feet below the base of the embankment, and including borings at the top, at mid-slope, and at the downstream toe of the dam arranged along an uphill-downhill line through the highest embankment section.

b) Conducting Standard Penetration Tests [ASTM D-1586] on a continuous basis along with pocket penetrometer tests on any soils that exhibited cohesive characteristics.

c) Collecting "undisturbed" Shelby tube samples of soil for laboratory testing.

d) Installing piezometers in the test borings.

The drilling investigation was conducted in the summer of 2003 while the reservoir was at normal pool. Drilling began with a boring (B-3-1) at the top of the dam, five feet downstream of the masonry cutoff wall. As the boring was advanced, SPT values at or near zero were recorded at certain depths, accompanied by the drill tools settling under their own weight. The initial boring was terminated at 20 foot depth, 50 feet above the target elevation, while plans for further drilling were reevaluated in light of the soft soil conditions and possible implications concerning embankment stability.

Drilling subsequently resumed with a second boring (B-3-1A) being advanced from the top of the dam near the first boring. The second boring was, in effect, an extension of the first and was augered without sampling to the bottom elevation of the first boring and then advanced with continuous SPTs through the remainder of the embankment and into the foundation soils. The embankment soils encountered in B-3-1A were similar to those in B-3-1 – certain intervals being soft to very soft. Soils encountered in this boring near the base of the dam just downstream of the masonry core wall were characterized by the field geologist as "mud."

Additional borings were drilled on the downstream face of the dam by securing the drill rig by cable to a second rig positioned at the top of the dam as a deadman. This drilling revealed embankment soils that were generally similar to those of the top-of–dam borings, although less frequently as soft.

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4. MATERIAL PROPERTIES

Laboratory tests conducted on a series of SPT and Shelby tube samples showed the embankment soils to include clay with sand [CL], clayey sand [SC-SM], silty sand with gravel [SM], and silty gravel with sand [GM]. Distinctions between certain embankment zones shown on the 1898 drawing were somewhat blurred, however, there being no apparent difference between the “selected material” of the central core and the undifferentiated material forming the rest of the embankment.

A representative profile of Standard Penetration Test N-values is presented in Figure 2. The N-value is defined as the number of blows required to drive a standard split barrel sampler a distance of 12 inches into the soil using a 140-pound hammer dropping through a height of 30 inches. Corresponding pocket penetrometer values are also presented in Figure 2. The penetrometer values provide a rough estimate of Qu, the unconfined compressive strength, and in turn the undrained shear strength Su of a cohesive soil.

Profiles of Su estimates from the top-of-dam borings showed a predominance of soft to very soft material (Su $\leq$ 0.5 ksf). Mid-slope and toe borings showed a greater proportion of soils of medium consistency (0.5ksf $\leq$ Su $\leq$ 1ksf). On the basis of the penetrometer tests, the mean value of Su was found to be 0.59 ksf, and the median value, 0.5 ksf.

Laboratory direct shear tests conducted on Shelby tube samples of soil yielded effective friction angles of 33 to 37 degrees and effective cohesion values of 0.476 ksf to 1.34 ksf. These values were regarded as suspiciously high, but were the only results available from the “undisturbed” samples that were collected.

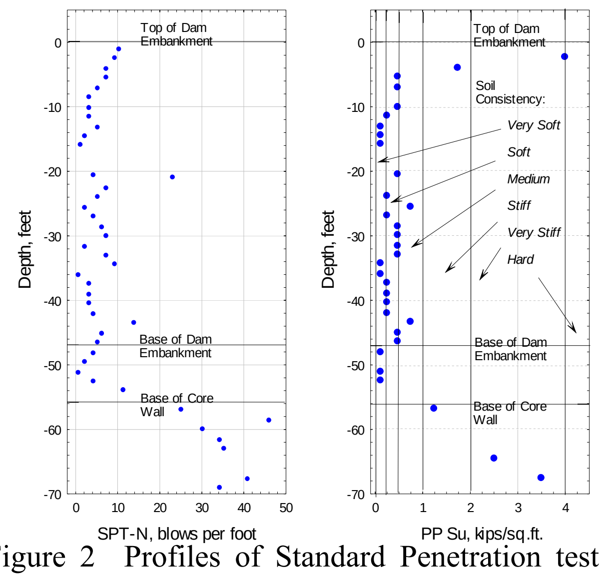

Figure 2 Profiles of Standard Penetration test N-values (left) and Pocket Penetrometer Estimates of undrained Shear Strength (right) for Top-of-Dam Test Borings B-3-1 and B-3-1A (combined)

5. SEEPAGE ANALYSES

The normal pool of Marilla Reservoir is approximately six feet below the top of the dam embankment, and the tail water is downhill of the embankment toe.

A wet zone observed on the downstream face of the dam during the site investigation – an area where surficial slippage was also noted - was recognized as an outbreak of seepage from the reservoir that extended approximately 30 feet upslope from the toe at the mid-section of the embankment and tapered off towards each abutment. Its presence was consistent with no drainage blanket being located beneath the embankment downstream of the core and was expected on the basis of piezometer readings and seepage analyses performed using Seep/W software (Geo-Slope International, Inc).

6. STABILITY ANALYSES

The Pennsylvania Division of Dam Safety requires a factor of safety of no less than 1.5 against failure of the downstream dam face (as does the Corps of Engineers (2003)) for the case of long-term steady state seepage at normal pool.

The Corps of Engineers (2003) has commented on the challenges of assessing the stability of existing dams: “There is danger in relying too heavily on slope stability analyses for existing dams. Appropriate emphasis must be placed on the often difficult task of establishing the true nature of the behavior of the dam through field investigations and research into the historical design, construction records, and observed performance of the embankment. In many instances monitoring and evaluation of instrumentation are the keys to a meaningful assessment of stability. Nevertheless, stability analyses are essential for evaluating remedial measures that involve changes in dam cross sections.”

The Bradford No.3 Dam analysis was to provide, in addition to an assessment of existing embankment stability, a baseline for: 1) designing stabilization measures, such as a buttress, in the event that the factor of safety of the existing embankment was unsatisfactory, and 2) determining how far the reservoir pool must be lowered on an interim basis to achieve an acceptable factor of safety while stabilization measures were being designed. Emptying the reservoir in its entirety was to be avoided given its function as a water supply and fish habitat.

The effective stress stability analysis subsequently performed was based on the laboratory-determined effective strength parameters and the pore pressures determined from a steady state seepage analysis. The

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Morgenstern-Price method, implemented with Slope/W software (Geo-Slope International, Inc.), indicated a factor of safety of 1.84 of the downstream face of the existing embankment under steady state seepage/normal pool conditions. This was suspected to be a serious overestimate of the factor of safety and to reflect the difficulties of procuring representative samples of the soft to very soft embankment soils, transporting them, and testing them in the laboratory.

For comparison, a total stress analysis of the existing embankment was also performed, with the core and flanking soils being assigned undrained shear strengths between 0.5 ksf and 0.59 ksf, as had been estimated from pocket penetrometer tests. This analysis yielded a factor of safety of between 1 and 1.2 for the downstream face of the dam. Considering the steepness of the downstream face, the undesirable seepage condition on the face, and known low strength zones within the embankment, the results of the total stress analysis were considered more plausible than those of the effective stress analysis.

It was concluded that: 1) the traditional subsurface investigation, which had involved Shelby tube sampling and laboratory testing along with Standard Penetration and Pocket Penetrometer Tests, as are customary for projects of this type and size in this region of the United States, had yielded unreliable and/or contradictory estimates of embankment shear strength and factors of safety, and 2) a supplementary field investigation involving more sophisticated in-place testing was required to reliably determine the strength parameters essential for the effective stress stability analyses, which were needed to assess interim drawdown requirements and to design long term stabilization measures.

7. SUPPLEMENTARY INVESTIGATION

The supplementary field investigation included two dilatometer soundings and three borehole shear tests conducted from the top of the dam near where the first test borings had been drilled. All of the in-place tests were performed at the direction of the writers by In-Situ Soil Testing using downhole equipment temporarily mounted on the drilling contractor's rig, which served as a reaction platform.

7.1 *Dilatometer Soundings*

The flat dilatometer is a steel blade having a thin circular expandable steel membrane mounted on one face. The blade is advanced vertically into the ground by means of push rods, which transfer the thrust from the insertion rig to the blade. (The hydraulic system of a drill rig was used in this case to push the blade, Figure 3). The blade is connected to a control unit on the ground surface by a pneumatic-electrical tube. At regular depth intervals (generally every 8 inches) penetration is stopped and the membrane is inflated by use of compressed gas. Two pressure readings are taken at each depth:

P_o = pressure required to just begin to move the membrane against the soil ("lift-off" pressure)

P_i = pressure required to move the center of the membrane 1.1 mm against the soil.

This process provides an essentially continuous profile of soil properties with depth.

The dilatometer soundings, through correlations such as presented by Marchetti (1980) and ISSMGE

Figure 3 Dilatometer Test in progress at the top of the dam

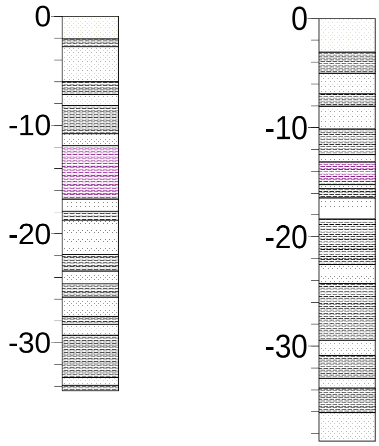

Figure 4 Soil Layering Delineated by Top-of-Dam Dilatometer Soundings D-3-1 (left) and D-3-2 (right). Light-toned layers are cohesionless soils; dark-toned layers are cohesive soils. Vertical axis is in feet.

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(2001), differentiated cohesionless layers from cohesive, quantified the effective friction angles of the cohesionless layers, and quantified the undrained shear strengths of the cohesive layers. Of particular significance are the following points:

- Soil Layers. Cohesive layers within the dam embankment alternate with cohesive layers. The cohesive layers were found to range from 0.5 feet to 5 feet in thickness, and to average 2 feet. The cohesionless layers were found to range from 0.35 to 3.2 feet in thickness, and to average 1.6 feet (Figure 4). No correlation of layers is evident between soundings. The layering is thought to reflect the construction methods used a century ago when horse and mule-drawn equipment was used to place the embankment fill (Figure 5), and rudimentary pavements of cohesionless soils were alternated with soft, low permeability cohesive soils to enable the construction equipment to cross the embankment without bogging down. The dilatometer soundings indicate that the embankment consists of approximately 56 percent cohesive soils and 44 percent cohesionless soils.

Figure 5 Bradford Dam No.3 (circa 1898) under construction

- Effective Friction Angle of Cohesionless Soil Layers. The drained friction angle was found to range between 26 and 42 degrees, with a mean value of approximately 34 degrees (Figure 6). These values are based on the correlation of Marchetti presented in ISSMGE (2001):

$$N_{safe,DMT} = 28^{o} + 14.6^{o}\ log K_D - 2.1^{o} log^2 K_D,$$

where K_D is the horizontal stress index.

- Undrained Shear Strength of Cohesive Layers. Su values of the cohesive layers ranged from 0.15 ksf to 0.7 ksf. The mean value was approximately 0.28 ksf (Figure 7). These values are based on the correlation of Marchetti (1980):

$$S_u = 0.22\ \Phi\rho_{vo}\ (0.5K_D)^{1.25},$$

where $\Phi\rho_{vo}$ is the vertical effective stress prior to blade insertion and K_D is as above.

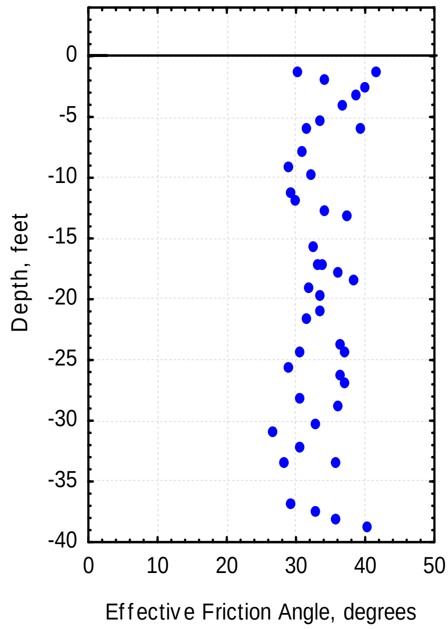

Figure 6 Profile of Effective (Drained) Friction Angle Values for the Cohesionless Soil Layers as determined from dilatometer soundings

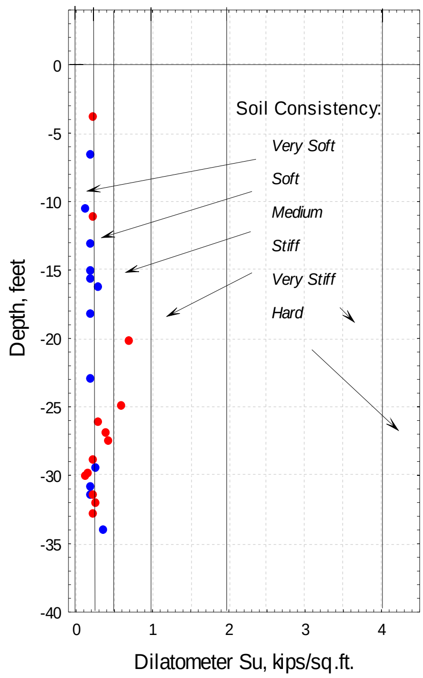

Figure 7 Profile of Undrained Shear Strength Values for the Cohesive layers as determined from dilatometer soundings

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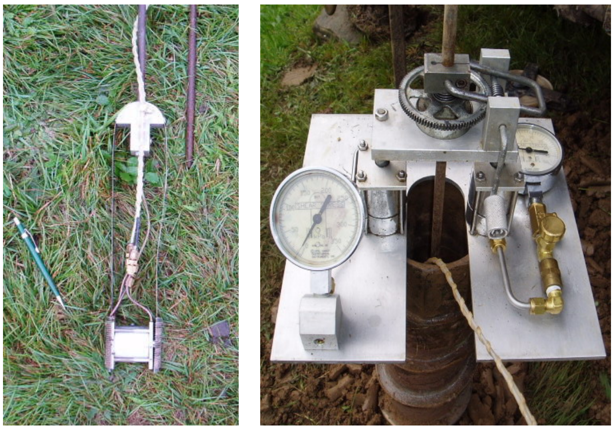


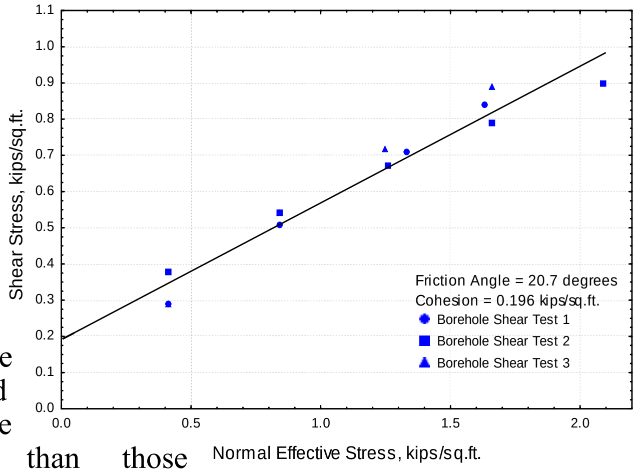
Figure 8 Borehole Shear Device – downhole component (left); components at collar of boring (right)

7.2 *Borehole Shear Testing*

The borehole shear test is essentially a direct shear test that is performed downhole to determine the effective strength parameters of a cohesive soil (Handy, 2002). The borehole is augered by conventional means to a depth approximately 18 inches above the test interval. A Shelby tube is then pushed through the test interval to create a smooth side wall. Upon extraction of the tube, the cylindrically-shaped borehole shear device is lowered to the test depth and a normal stress is applied to the borehole sidewall by two opposed, hydraulically-activated platens (Figure 8). The soil is allowed to consolidate under the normal stress and is then sheared by pulling the expanded BST device axially upward to at a sufficiently slow rate to limit the development of excess pore pressure within the soil. The BST is performed in a stepwise manner at each test depth, so as to define a Mohr envelope from the shear stress values at slippage at progressively higher levels of effective normal stress.

Of particular significance are the following points:

- All three borehole shear tests, which were performed at depths of 10 ft., 15 ft., and 30 ft. below the top of dam and within intervals identified by dilatometer testing to be cohesive, yielded similar results.
- These three tests yielded values of effective friction angle between 17.2 and 25.2 degrees and cohesion between 0.122 and 0.269 ksf. Taken together, these tests suggest an effective friction angle of 20.7 degrees and an effective cohesion of 0.196 ksf to be representative of the cohesive soils (Figure 9).



The BST-derived strength parameter values are considered far more plausible than those obtained from the laboratory direct shear tests on so-called "undisturbed" Shelby tube samples, which are suspected to have been disturbed or to reflect the presence of granular soils that may inadvertently have been incorporated into the samples or to be otherwise non-representative.

Figure 9 Mohr Envelope developed from Borehole Shear Testing conducted in cohesive soil intervals within top-of-dam borings

7.3 *Refined Estimate of Effective Strength Parameters and Revised Factor of Safety*

For the purposes of further stability analyses, a value of 25 degrees was assigned to the effective friction angle and 0.110 ksf to the effective cohesion of the soils that comprise the dam embankment. These values represent a weighted average of the effective strength parameters of the cohesionless and cohesive soil layers interpreted from the dilatometer and borehole shear tests.

Using these values, the factor of safety of the downstream face of the existing dam was computed to be 1.05 for the steady state seepage/normal pool condition. The refined stability analysis provided a basis for the interim drawdown strategy implemented while the rehabilitation measures were being designed as well as a basis for the design of stabilization measures, which include a downstream buttress, an internal drainage system and overtopping protection consisting of roller compacted concrete.

8. CONCLUSIONS

The field and laboratory methods commonly employed in investigations of slope stability in northern Appalachia – soil borings with Standard Penetration and Pocket Penetrometer Tests, Shelby tube sampling, and laboratory testing – produced results that were contradictory and/or unreliable in the case of the Bradford No.3 Dam.

A subsequent, more refined program of field investigation that included dilatometer and borehole

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shear testing played an invaluable role in characterizing the soils that compose the dam embankment – in differentiating cohesionless soil layers from cohesive and in quantifying their respective effective strength parameters. This enabled the stability of the existing embankment to be evaluated in a manner that could confidently be used as a basis for the design of stabilization measures.

Neither the dilatometer nor the borehole shear test is in common use in projects of this type and size in the northern Appalachian Region. Their future use is to be recommended when customary methods of soil characterization prove inadequate.

9. ACKNOWLEDGEMENTS

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