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On the Cover

The Corps of Engineers' Beltzville Lake in East-Central Pennsylvania. In this south-facing photo, from the bottom to the top, features include the project office, the emergency spillway, the 4,560-foot long embankment, the intake tower, and a series of ridges of the Appalachian front. Beltzville Lake is on Pohopoco Creek, which drains into the Lehigh River. The Lehigh River's water gap through Blue Mountain can be seen in the background of the photo. (Photo by Anthony S. Bley.)

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- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

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GREEN LANE GETS GREEN LIGHT: PARAMETER SENSITIVITY IN GRAVITY DAM STABILITY ANALYSIS[1]

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ABSTRACT

This paper includes a case history of the evaluations of Green Lane Dam, a 103-ft high concrete gravity dam constructed in the mid 1950s. In 2000, the Pennsylvania Department of Environmental Protection (PADEP), Division of Dam Safety expressed concerns regarding the dam's ability to safely pass the Probable Maximum Flood (PMF) based on current meteorologic and hydrologic criteria. Initial stability analyses of the structure indicated that the dam did not meet stability criteria under the increased PMF loading condition and would require strengthening. In 2004, Aqua Pennsylvania engaged Schnabel Engineering to provide subsurface investigations and preparation of design documents for a rock-anchoring program. The subsurface exploration, laboratory testing, detailed hydraulic analyses, and a thorough review of construction documents and photos indicated that the following factors significantly contributed to stability of the structure:

- The unit weight of the concrete was higher than expected
- Tailwater levels during the PMF were higher than assumed in previous analyses
- The dam was "keyed" into the rock foundation

Stability analyses incorporating these factors and considerations showed that the dam meets Army Corps of Engineers' criteria and that stabilization is not required.

This paper focuses on the results of the field investigations and stability analyses, illustrates the significance of parameter selection in gravity dam stability analysis, and presents a dam safety story with a happy ending.

Green Lane Dam

[1] A more comprehensive paper with the same title and subject was published in the ASDSO 2005 Annual Conference Proceedings.
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PROJECT OVERVIEW AND HISTORY

Green Lane Dam is a 103 ft high concrete gravity dam impounding a 13,400 acre-ft reservoir located in Montgomery County, Pennsylvania, about 40 miles northwest of Philadelphia. The dam was designed in the early 1950s by Albright and Friel for the owner, Philadelphia Suburban Water Company (now Aqua Pennsylvania) and constructed between 1955 and 1957. The dam has a total length of 786 ft with a spillway length of 424 ft. The drainage area to the dam is 70.1 square miles.

Previous Studies

Several studies were performed that incorporated stability analyses for the Green Lane Dam. In 1976, prior to the Army Corps of Engineers' Phase I Inspection, an evaluation was performed and found that "Green Lane Dam has adequate sliding resistance under very conservative design loading conditions." In the late 1970s the federal government instituted the Phase I Inspection program, which included evaluations of non federal high and significant hazard dams. These evaluations included historical file reviews, visual inspections, hydrologic, hydraulic, and stability analyses, and an overall assessment of the safety of the dam and recommendations for future investigations and analyses. The Phase I Inspection of Green Lane Dam concluded that the spillway is capable of passing about 76 percent of the PMF without overtopping the non-overflow sections and the stability analysis showed development of tension for the PMF condition and a factor of safety of less than 3 for the PMF condition; however, further investigations of stability were not considered warranted.

Since the Phase I Inspections, estimates the Probable Maximum Precipitation (PMP) were increased. In 2002, PADEP and Aqua Pennsylvania agreed to a PMF with a computed a peak inflow to the reservoir of about 127,000 cfs, resulting in overtopping of the non-overflow sections by about two feet. This provided the basis for a preliminary stability analysis (by others) performed using Bureau of Reclamation (Reclamation) methods and criteria and showed a factor of safety of 1.4 for the spillway section and 0.9 to 1.3 for the non overflow sections. For Reclamation criteria, the recommended minimum factor of safety for the "unusual" PMF condition is 2.0. It should be noted that PADEP allows the use of any "generally accepted" criteria for gravity dam stability analysis; however, the use of Army Corps' of Engineers methods and criteria are recommended. Based on the results of the analysis, stabilization with post-tensioned anchors was recommended, following a detailed geotechnical investigation and revised stability analyses based on the results of the investigation.

INVESTIGATIONS AND SIGNIFICANT FINDINGS

The engineering evaluation of Green Lane Dam included a historic file review, subsurface explorations and laboratory testing, hydraulic analysis, and gravity dam stability analyses.

Historic File Review

The files of Aqua Pennsylvania and PADEP contained significant information regarding the design and construction of Green Lane Dam, including as-built drawings, construction photos, hydraulic model studies, and other engineering calculations. A review of these documents revealed information that impacted the stability evaluation.

As-built drawings and construction photos indicated that significant rock excavation (15 to 30 ft) was performed (especially in the non-overflow sections) to construct the dam. The downstream excavation slope was near vertical, and concrete was cast directly against the rock for a minimum height of 5 ft. The remaining area between the concrete gravity section and the rock that remained was backfilled with aggregate fill (see Figure 1). Because of this geometry, sliding could not occur without mobilizing a significant rock wedge (shear through the bedrock – see next section for description of the bedrock).

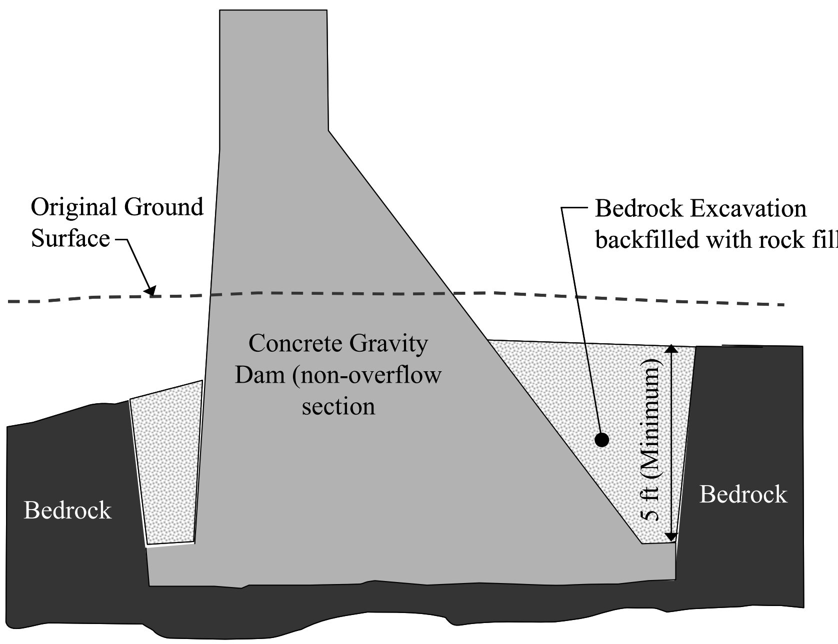


Figure 1. Typical Section – Non Overflow Portions

From the drawings and photos, it was also found that the roller bucket energy dissipater in the spillway section has a minimum thickness of concrete of about 5 ft and is anchored to the foundation with steel bars drilled into the bedrock, providing supplemental sliding resistance.

As part of the original design, Philadelphia Suburban Water Company performed model studies of the roller bucket energy dissipater to estimate tailwater at the dam. Generally, well-designed roller buckets result in less "wash-out" of the tailwater than hydraulic jump energy dissipaters. The hydraulic evaluation of the roller bucket and development of effective tailwater depths is discussed in greater detail later in this paper.

Subsurface Investigations and Laboratory Testing

A detailed subsurface investigation is critical to the design of strengthening measures for a concrete gravity dam. For the Green Lane project, four core borings were advanced from the crest of the non-overflow and spillway sections through the concrete dam and into the foundation bedrock. The concrete of the dam was found to be good quality and most of the horizontal construction joints were unidentifiable by visual inspection, indicating good bond at these internal joints. The rock at the concrete/bedrock interface is highly fractured, indicating little or no shear (cohesion) resistance at the interface. The rock cored is generally slightly weathered, hard, gray shale or Hornfel with near vertical joints generally perpendicular to the bedding. Based on Rock Mass Rating Measurements (RMR), the rock is generally fair and locally good near the bottom of some of the borings. The bedrock decreases in fracturing with depth from highly fractured to moderately fractured. The bedrock encountered beneath the spillway sections was generally higher quality than that encountered in the borings in the non-overflow sections.

Laboratory testing included unit weight and compressive strength tests of the concrete and rock samples obtained. Eleven concrete samples totaling about 17 ft of NQ (1.875-inch diamer) core were tested, having unit weights ranging from 150 to 165 pcf. The computed length-weighted average dry unit weight was 157 pcf with a standard deviation of 3.5 pcf. The saturated surface dry unit weight was generally about 2 to 3 pcf higher than the dry weight. Typically, the unit weight of good quality mass concrete is between 145 pcf to 155 pcf. The relatively high unit weight of the Green Lane Dam concrete, resulting from the granitic aggregate with high specific gravity, has a significant impact on the stability of the structure.

Hydraulic Analysis

The hydrologic analyses for development of the PMF at Green Lane Dam had been performed and accepted by PADEP, Division of Dam Safety. The results indicated that the non-overflow sections of the dam would overtop by about 2 ft for nearly four hours.

For the stability analyses, ratios of the PMF were also computed. The computed 0.2 PMF peak flow corresponds approximately to the 500-year storm at Green Lane Dam as reported in the Montgomery County Flood Insurance Study (FIS).

The tailwater depth at the dam is an important loading for surcharge pool conditions at this gravity dam. Hydraulic analyses were performed to estimate the tailwater at Green

Lane Dam for various events up to and including the PMF for use in the stability evaluation. The HEC-RAS computer program was used to model flow in Perkiomen Creek between Green Lane Dam and Hill Road, located about 1,200 ft downstream. For the stability evaluation of the spillway sections, tailwater as computed in the HEC-RAS analysis was adjusted to reflect effective tailwater against the dam, as influenced by high velocity flow through the spillway and roller bucket. Model studies of the roller bucket were conducted as part of the original design. In addition, the Army Corps of Engineers' EM 1110-2-1603, Hydraulic Design of Spillways (1990) provides guidance related to roller bucket energy dissipaters. The tailwater computed in the HEC-RAS analysis was used to compute effective tailwater depth at the roller bucket. Application of the methods presented in EM 1110-2-1603 yielded very similar results to those found in the original dam design documents. For flows up to the 0.4 PMF, the original design data was used, and for greater flows (beyond the limits of original model study), the methods of EM 1110-2-1603 were applied.

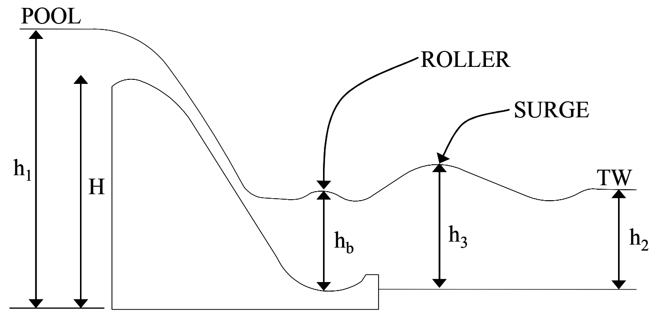


Figure 2. Roller Bucket Definition Sketch
(based on EM 1110-2-1603)

Hydraulic jump energy dissipators typically reduce the tailwater to a much greater extent (down to about 60 percent of the depth in the downstream channel) than roller buckets. The effective computed tailwater for the Green Lane Dam roller bucket is to about 85 to 90 percent of the downstream tailwater depth. This additional tailwater at the dam translates to increased stability of the structure.

GRAVITY DAM STABILITY ANALYSIS

Stability analyses for Green Lane Dam were performed by means of spreadsheet program developed for the analysis of gravity dams.

Methodology

For gravity dams, PADEP Division of Dam Safety recommends application of the methodology contained in the Army Corps of Engineers' EM 1110-2-2200, *Gravity Dam Design* (1995), which states the following basic stability requirements for all loading conditions for a gravity dam:

- That it be safe against overturning at any horizontal or near horizontal plane within the structure, at the base, or at a plane below the base.
- That it be safe against sliding on any horizontal or near horizontal plane within the structure, at the base, or on any rock seam in the foundation.

- That the allowable unit stresses in the concrete or in the foundation material shall not be exceeded.

The gravity method of analysis requires that the resultant of all forces acting on the dam lie within the middle one-third of the base to avoid tensile stresses in the heel of the dam (rigid body analysis). When the resultant lies outside of the middle one-third of the base, tensile stresses develop along the base of the dam. The criteria contained in EM 1110-2-2200 require that the resultant lie within the middle third for usual loading conditions, the middle half for unusual loading conditions, and within the base for extreme loading conditions.

The guidelines of the Bureau of Reclamation (Reclamation) and the Federal Energy Regulatory Commission (FERC) suggest that when tensile stresses form at the base of the dam, the analysis should assume that a crack develops propagating from the heel of the dam. Full uplift is then assumed to act on the cracked section of the base (except under seismic loading), and the analysis is revised to reflect this modified uplift distribution, and with shear strength acting only along the uncracked portion of the base. EM-1110-2-2200 does not include recommendations for performing cracked section analyses; however, a more recent Corps manual, EM 1110-2-2100, *Stability Analysis of Concrete Structures* (2005) includes provisions for development of a cracked base. At the time of the evaluation of Green Lane Dam, this manual was still an engineering circular (EC 1110-2-6058).

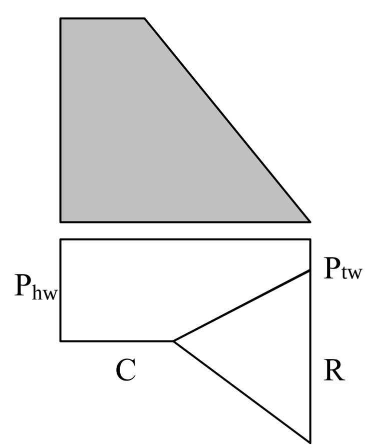
While most agency guidance suggests an iterative method of analysis when tension is computed at the heel of a gravity dam, the crack length and reaction pressure at the toe of the dam can be computed explicitly using the following equations (Campbell, 1989).

$$C = B - \frac{3\left(M' - \frac{B^2 P_{hw}}{2}\right)}{\left(V' - BP_{hw}\right)} \qquad R_t = \frac{2\left(V' - BP_{hw}\right)^2}{3\left(M' - \frac{B^2 P_{hw}}{2}\right)} + \left(P_{hw} - P_{tw}\right)$$

<u>Where:</u>

P_{hw}:	Headwater pressure
P_{tw}:	Tailwater pressure
R_t:	Reaction at toe
B:	Base length
C:	Crack length
V':	Net vertical forces applied to the foundation*
M':	Net moments applied to the foundation*

* excludes uplift



For Green Lane Dam, cracked-section analyses were performed if tensile stresses were calculated at the heel of the dam. It is common to accommodate a nominal tensile strength for internal (concrete-concrete) lift surfaces within the dam (partial sections), and to limit cracked-section analysis to conditions where tensile stresses exceed tensile strength. For Green Lane Dam, cracked-section analyses were performed whenever any tensile stress was indicated.

Loading Conditions and Material Properties

Stability computations were performed for the spillway and the non-overflow sections, with structure geometry obtained from as-built drawings. Partial section stability analyses (along horizontal joints within the dam) were also performed. All analyses were developed for normal pool, ice, and earthquake conditions, and for the full PMF and ratios thereof. The loading condition type (i.e., Usual, Unusual, and Extreme) and associated factor of safety were obtained from EM 1110-2-2200. EM 1110-2-2200 does not specifically identify the break between unusual and extreme flooding conditions; however, the PMF is considered an extreme loading condition, and the standard project flood (SPF) is considered an unusual loading condition. Army Corps of Engineers' documents vary on the definition of the SPF, but it is typically on the order of the 100-year to 300-year storm. For this analysis, 20 percent of the PMF, which corresponds to about a 500-year flood, was considered an unusual loading condition and greater floods were considered to be an extreme loading condition. For comparison, Army Corps of Engineers' documentation considers the operating basis earthquake to be an unusual loading condition, and this event corresponds to a 290-year event for a structure with a design life of 200 years.

For earthquake conditions, both the maximum credible earthquake (MCE) and the operating basis earthquake (OBE) were evaluated. Peak acceleration values (as a fraction of gravity) were estimated from maps provided in the USGS National Hazard Mapping Project (2002). Hydrodynamic forces for the MCE and OBE were assessed using the Westergaard Formula.

Unit weights for concrete and rock of 158 and 172 pcf, respectively, were determined by the laboratory testing of recovered samples, while other material properties were estimated by correlation to empirical data for similar materials.

A friction angle of 45° was used for the dam/foundation contact and internal joints in the concrete. Shear strength (cohesion) at the base of the dam (concrete/rock interface) was conservatively assumed to be zero. There is clearly some level of shear strength at the interface; however, the magnitude of this shear strength is difficult to estimate and the geometry of the cores obtained with a bonded interface (2 out of 4 were bonded) did not permit testing to obtain an estimate of shear strength. Shear strength along horizontal concrete joints for partial sections was conservatively estimated to be 25 psi, based on the compressive strength of the concrete (about 6,000 psi, based on laboratory testing) and the condition of the joints as observed in the subsurface exploration. Typically, shear strength of concrete is considered to be equal to twice the square root of the compressive strength, or about 155 psi. Strength along joints can conservatively be assumed to be about 80 percent of the strength of the parent material and a factor of safety of five was applied to this shear strength.

Because there is flow over the non-overflow sections for the PMF condition, it was assumed that the downstream backfill soils would be completely removed by scour for all loading conditions.

Although measurements taken during the subsurface investigations indicate that the grout curtain reduces uplift across the foundation, uplift was assumed to vary linearly from full headwater at the heel to full tailwater at the toe. Within the body of the dam (partial sections), EM 1110-2-2200 recommends that uplift be "assumed to vary linearly from 50 percent of the maximum headwater at the upstream face to 50 percent of tailwater, or zero, as the case may be, at the downstream face." For these analyses, a more conservative approach was used. For normal pool conditions, uplift was assumed to vary linearly from the normal pool depth at the upstream face to the estimated tailwater depth at the downstream face. For flood conditions, uplift was assumed to vary linearly from 50 percent of the difference between maximum and normal headwater depth at the upstream face to 50 percent of the difference between maximum and normal tailwater depth.

As indicated earlier in this paper, a significant amount of rock was removed to construct the dam and the concrete was placed against a wall of bedrock having a minimum height of about 20 ft in the non-overflow sections. To consider this condition, a passive wedge analysis was performed to estimate the resisting force available in the rock downstream of the dam. As previously stated, the spillway's roller bucket energy dissipater is anchored into the foundation bedrock. The shear resistance of these steel anchors was computed and included in the analysis as a passive force. Additional passive pressure from the bedrock against which the bucket was cast was not considered.

Results

The computed sliding factors of safety for selected loading conditions with (condition 1) and without (condition 2) consideration of the passive forces from the bedrock and shear strength from the roller bucket anchors are presented in Table 1 along with the minimum required criteria.

Table 1. Stability Analysis Results

Loading Condition and Type	Minimum Required FS	Computed FS (Condition 1 / 2)	
		Spillway Section	Non-overflow Section
Normal Pool (Usual)	2.0	2.3 / 2.6	2.8 / 3.5
OBE Earthquake (Unusual)	1.7	2.0 / 2.3	2.5 / 3.1
MCE Earthquake (Extreme)	1.3	1.4 / 1.5	1.7 / 2.1
Flood – 0.2 PMF (Unusual)	1.7	2.0 / 2.3	2.2 / 2.8
Flood – 0.4 PMF (Extreme)	1.3	2.0 / 2.2	1.9 / 2.4
Flood – PMF (Extreme)	1.3	1.9 / 2.1	1.2* / 1.7*

* Cracked section; resultant within middle ½ of base.

For comparison, the previous analysis, which utilized lower concrete unit weight, less tailwater, and no passive resistance, resulted in a computed factor of safety of 1.4 for the spillway section and less than 1.0 for the non-overflow section.

These results indicate that the dam meets the Army Corps of Engineers' stability criteria for all conditions except the PMF if passive resistance from the bedrock is not considered. Conservatively incorporating this additional resistance increases the factors of safety to meet the Army Corps of Engineers Stability Criteria. The analysis was submitted to and accepted by PADEP Division of Dam Safety.

Parameter Sensitivity

To evaluate the impact of various parameters on the results of the gravity dam stability analysis, a parametric study was performed. The primary differences between the previous analysis (showing the dam to be unstable) and the new analysis are the concrete unit weight, the tailwater elevation, and the resistance from the downstream bedrock and anchors in the roller bucket. The effects of incorporating the bedrock and anchors into the analysis can be seen in the increase in the safety factors shown in Table 1.

The unit weight of the concrete was higher than originally expected. Typically, the unit weight of good quality concrete ranges from 145 to 155 pcf. The unit weight of 158 pcf at Green Lane Dam was developed from laboratory testing on samples obtained during the subsurface exploration. To illustrate the effects of concrete unit weight on the results, stability analyses were performed for the spillway section for the PMF condition using a range of unit weights. Other parameters were kept constant. Results indicated that the safety factor varies linearly from about 1.6 to nearly 1.9 within a reasonable range of concrete unit weight values of 145 to 160 pcf.

Varying tailwater over a reasonable range of depths from about 15 ft to about 50 ft (measured from invert of roller bucket) results in a safety factor ranging from 1.5 to nearly 2.0 for the PMF condition.

Analyses were also performed varying both tailwater and concrete unit weight, showing that within a reasonable range of estimated tailwater depth and concrete unit weight, the computed safety factor could range between about 1.3 and 2.0.

SUMMARY AND CONCLUSIONS

For nearly 5 years, the regulators, owner, and engineers involved in the Green Lane Dam project believed that the dam would need significant rehabilitation to meet dam safety criteria. Aqua Pennsylvania engaged an engineer to prepare a design for this rehabilitation. Through a thorough subsurface exploration and engineering evaluation, it was demonstrated that the dam meets the gravity dam stability criteria required by the State of Pennsylvania (based on Corps of Engineers methodology) under all applicable loading conditions. Because the dam did not require upgrading (originally estimated at $1M or more), Aqua Pennsylvania could use the savings to expedite upgrading of it's other high hazard dams, creating a win-win situation for the owner and PADEP Division of Dam Safety.

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