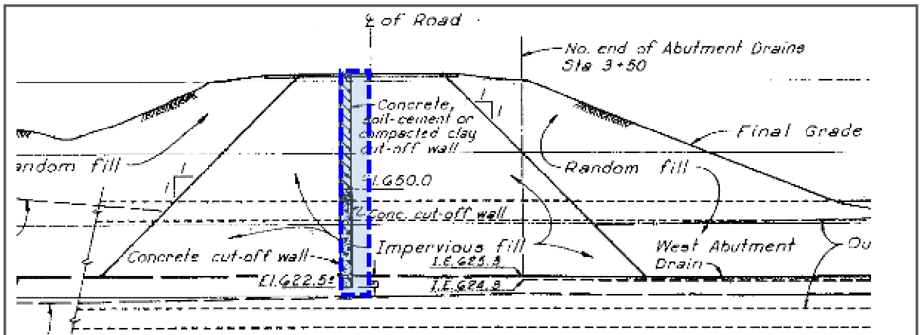
Marshy Area and Seepage Collector Drain

Seepage has been documented in the area between the Service Spillway and the left end of the Spillway Levee since at least the 1940s. The seepage continued at higher quantities after construction of the Side Channel Spillway. This seepage has developed a perennially marshy area beside the left training wall of the Service Spillway (**Figure 9**).



(1) Construct an impervious core from the present embankment to the left abutment as shown on Plates 1, 2, and 3.

(2) Place impervious backfill upstream of the core with a width of approximately 10 ft adjacent to the excavated slopes up to the top of the upper packsand (about El 640 to 650). The remainder of backfill upstream of the core and adjacent to the spillway structure can be random material, as impervious as practical to obtain.

(3) Install a cutoff wall through the upper packsand from the existing spillway up to a point 63.5 ft from the centerline of the spillway conduit as shown on Plates 1, 2, and 3.

(4) Place drainage blankets up the face of the excavation slopes from the upper shale at about El 625 up to El 635 as shown on Plates 1, 2, and 3.

Figure 8: Proposed modifications for cutoff wall (1970)

Figure 9: Marshy area on left side of Service Spillway basin

The 1969 final grading plan shows a trapezoidal flat area left of the left training wall at Elev. 626. The flat area is about 95 feet long, matching the stilling basin wall. Considerable fill was placed at some time, raising the grade at the northeast corner of the flat area to about Elev. 631 and flattening the slopes somewhat north and east of the flat area. The north 40 feet of the previously flat area is perennially wet. Water seeps to the ground surface and runs over the top of the training wall into the stilling basin. The wet area is generally un-trafficable to mowing equipment. The wet area extends north up the slope another 10 to 20 feet. Green vegetation extends northwest along the chute wall about 70 feet from the corner and extends northeast up the groin about 40 feet. From the air, the green area is Y-shaped.

A drainage ditch may have been excavated behind the wall to intercept the seepage prior to 1949, but sometime later a perforated pipe drain (similar to a "French" drain) was installed a few feet east of the stilling basin wall that discharged onto low ground near the downstream end of the wall. No drawings of the drain have been located. The drain is a 4-inch diameter PVC pipe with ¾-inch perforations and surrounded by limestone gravel mixed with clay. It is located about 4 feet back of the training wall and 2 feet below the ground surface. It is underlain by shale (likely Shale C). Flow has been measured as about 2 gallons per minute. Seepage over the wall and from drains and cracks in the east stilling basin wall and the east chute training wall is estimated as another 2 gallons per minute.

2012 Grouting Program

A grouting program was conducted in 2012 to address seepage areas identified during a comprehensive study of the reservoir in the early 2010s. As part of a larger grouting program on both sides of the spillway, the left side was intended to tighten up embankment materials down to the bedrock (generally described as a sandstone). The grouting procedure included injecting a polyurethane, permeation-type grout through clay, sand and gravel and terminating one or two feet into the bedrock. The grout curtain was constructed just downstream of the embankment centerline between the spillways by Hayward Baker. After a period of 18 months, it became apparent that the seepage persisted because the grouting program addressed the Overburden A stratum but only the top of the Packsand B stratum. An in-depth investigation conducted by Freese and Nichols, Inc. found that seepage was concentrated within the Packsand B stratum that is present below the grout curtain.

CURRENT STUDY (2014 - 2019)

Field Exploration

A multi-phase investigation was performed at the site from 2014 through 2019 (**Figure 10**). The study included two borings advanced with traditional drill rigs, three hand auger borings, ten borings advanced with a sonic drilling rig, and two test pits. Select sonic borings were directionally drilled with continuous casings at angles from the upstream side of the crest to avoid disturbing the steep downstream embankment slope.

Test Pits and Boil Development

Two test pits were excavated in 2014 using a tractor-mounted backhoe. Test Pit 1 was excavated in the slope immediately north of the north end of the east wall of the stilling basin and roughly 25 feet east of the wall. The trench was two feet wide, about 17 feet long, and had a maximum depth of about 4-1/2 feet. It encountered clay over clean, fine-grained, white sand over fine-grained, white sandstone (Packsand B). Water entered the pit rapidly, and the sides of the trench began to slough. To minimize instability, the trench was backfilled promptly after the data collection. With the surface clay layer disrupted, the seepage developed at the surface, and a small sand boil developed. An inverted filter consisting of a mound of concrete sand covered by a mound of concrete gravel was constructed to filter the boil and control internal erosion. Water continues to flow across the marsh and over the stilling basin wall at this location.

Test Pit 2 was excavated about 56 feet north of the south end of the east wall of the stilling basin to locate and examine the perforated pipe drain. The pit uncovered the drainpipe, a white, 4-inch diameter PVC pipe with 3/4-inch-diameter perforations. It was surrounded by limestone gravel with significant clay. There was a mat of small roots around the pipe. The pipe sat on a surface of shale (likely Shale C) and there was no evidence of filter sand or geotextile filter material.

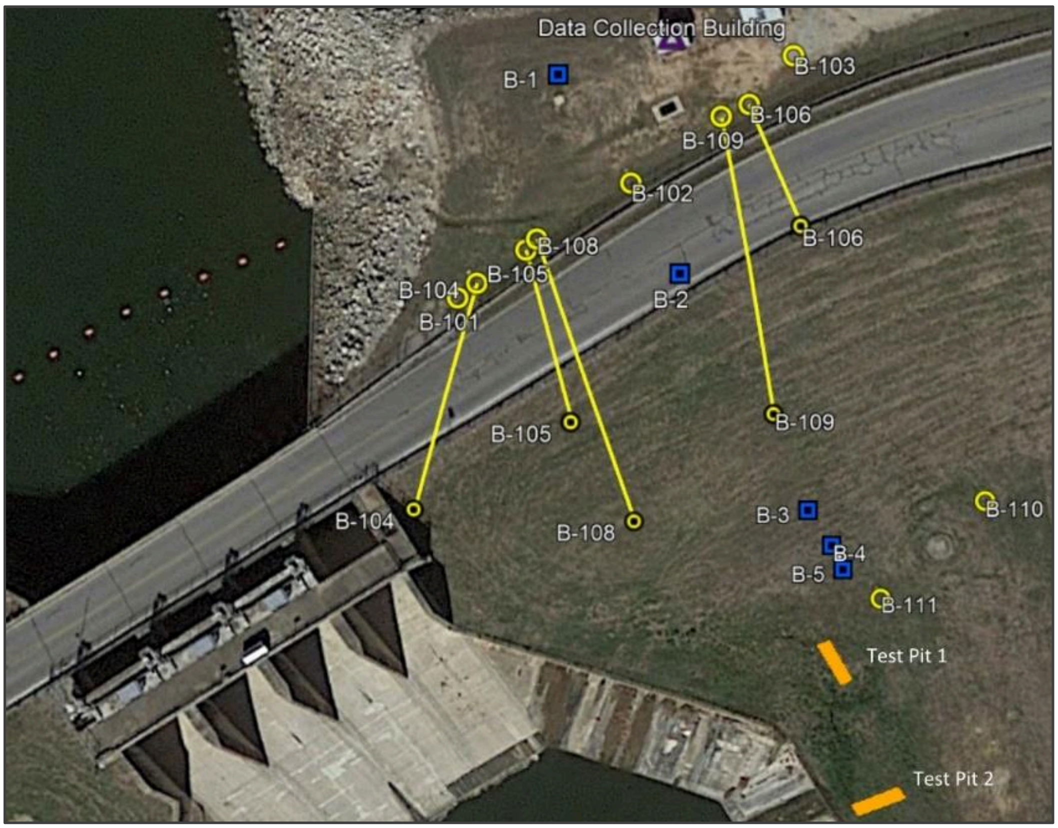


Figure 10: Plan of borings and test pits

Piezometer Instrumentation and Data Collection

The installations also included piezometer instrumentation within the embankment fill, Packsand B, Shale C, and Packsand D. Both grouted and open-riser instrumentation was used. In all cases, vibrating wire transducer instruments were installed to allow for automated data collection. Periodic readings were collected manually until these instruments were hard wired into the Tarrant Regional Water District automated data acquisition system (ADAS) via cabling leading to new multiplexers installed within the District's existing data collection building.

Figure 11: View of angled advancement of Borehole 103

SUMMARY OF FINDINGS

During the original dam construction, excavation to final grades to the left of the stilling basin likely exposed Packsand B. Westward flow of groundwater from the left abutment would have been blocked by the spillway, so seepage may have developed in the area now occupied by the marsh even before the lake contained water, and very likely would have developed shortly after first filling. The marsh area may have been cut down to create a drainage ditch to divert the seepage around the end of the training wall.

The review of the record data suggests that there was some uncertainty or possibly some misinterpretation of the subsurface stratigraphy during the original construction, as evidenced by the use of the term "lime rock" to describe the bedrock. The original investigators may have been influenced by the numerous limey outcrops that are present in this area and along the Trinity River but is likely also due in part to the level of local geologic knowledge at that time. Regardless, is seems that the amount of underseepage through the foundation bedrock was not fully appreciated during the original construction. But it is clear that the study of the Side Channel Spillway correctly identified the issue.

During construction of the Side Channel Spillway, water was discharging from Packsand B into this excavation at 60 GPM from the west bank and 40 GPM from the east bank. Most of this flow was concentrated in piping holes eroded in the packsand. Charles Mansur predicted that the seepage through the packsand would increase when the cofferdam around the north end of the SCS intake structure was removed and the intake channel was excavated, and recommended construction of an excavated concrete cutoff between the two spillways and the construction of filtered drains on each side of the box culvert to relieve the water seeping through Packsand B. A clay core in the culvert backfill was already in the plans. The impervious backfill and the drains were constructed, but the concrete cutoff apparently was not. It is not clear why this cutoff was omitted but it could be due to uncertainty with the installation, as the technology for excavated cutoffs was in the early stages of development at that time. Drilled secant pile cutoffs were not uncommon but their use may have been cost prohibitive.

An attempt was made in 2012 to address this issue by grouting the lower, more permeable zones along the base of the dam embankment, but this program did not grout the lower and permeable Packsand B stratum.

The Paluxy sandstone and overlying sand have displayed a vulnerability to piping several times, usually during construction operations. Packsand B is particularly susceptible to internal erosion piping based on the record data for the construction of the Side Channel Spillway. These conditions present the potential for internal erosion and adds credibility to internal erosion failure modes and associated risks. The credibility of internal erosion failure mechanisms depends on the type of material subject to erosion. The sandstone in the Paluxy formation is generally fine-grained. The more cemented portions of the sandstone are not considered erodible, but the less cemented and soil-like portions are documented as erodible. The cemented portions of the Paluxy formation or the overlying clays in Overburden A could form a roof when less cemented portions are eroded. These conditions indicate that the internal erosion failure mode for Packsand B is credible. Based on this finding, action was recommended to address the seepage.

RECOMMENDATIONS

Seepage Cutoff

Because of potential instabilities associated with the groundwater regime, it is desirable to provide a seepage cutoff wall through Packsand B and into Shale C to improve dam safety and improve site maintenance and operations. The alignment of this cutoff was very similar to that proposed by Mansur in 1970, proving that sound engineering principles rarely change.

A variety of geotechnical-based technologies were considered for the recommended cutoff. The following list provides a summary of these technologies along with associated benefits and disadvantages:

- Injection Grout Curtain: This technology involves the injection of cementitious or other grout types to fill voids in the stratigraphy matrix and reduce seepage flows within the targeted zone. Although used successfully used on other parts of this project, is not recommended because the packsand material will be difficult to grout as it requires filling of the soil matrix voids as well as macro-void features. This creates a high probability of seepage windows after construction.
- Secant Pile Wall: This system involves the construction of non-reinforced, lean concrete augered piles/piers to create a low-permeability, full-height seepage barrier. Although this provides an essentially continuous, full-height barrier with a high degree of certainty of closure, it has a longer construction schedule, is more expensive, and will involve the use of heavy equipment. It may also be more disruptive of local traffic.
- Cutter Soil Mix (CSM) Wall: The system is a “mix-in-place” technology that utilizes two sets of counter rotating, vertically mounted cutter wheels that are used with grout mixing to create low-permeability, overlapping panels to create a full-height barrier. This provides an essentially continuous, full-height barrier with a high degree of certainty of closure but has a longer construction schedule, is relatively expensive, will involve the

use of heavy equipment, and presents conflicts with the existing instrumentation and utilities.

- Slurry Trench Wall: This cutoff is typically constructed using a long-reach excavator and bentonite slurry to create a deep trench that is then backfilled with an engineered mixture of soil, bentonite and cement to create a low-permeability, full-height seepage barrier. In tight areas such as this, a crane operated clam shell bucket excavator may prove more practical. This has space limitations due to the size of the equipment involved in construction and creates potential embankment stability considerations during construction.
- DeWind One-pass Trench Wall: The technology is proprietary to DeWind and is similar to a slurry trench wall, except that the soil excavation and placement of the engineered mixture occurs in a continuous operation with a supported vertical excavation. It is an efficient operation by a heavy and large machine. This technology has difficulties with potential alignment adjustments due to the space limitations on the western end of the cutoff. It also presents conflicts with the existing instrumentation and utilities.
- Jet Grout Column Wall: This method creates overlapping columns of grouted soil using a grouting monitor attached to the end of a drill stem. This jet grouting process introduces the risk of potentially hydrofracturing the adjacent areas of the Spillway Levee embankment and could introduce additional defects in the embankment and/or foundation.
- Relief Well Network: Although not a cutoff technology, a relief well system collects seepage through a network of closely spaced and filtered well points. They are typically passive in nature and require the use of collection piping and controlled outfall points. Wells will significantly increase annual maintenance activities and have significant surface obstructions.

Selection of an appropriate alternative will depend on a variety of factors, including cost, equipment size/accessibility, construction duration, certainty of cutoff closure at completion, adverse construction stability risks, obstruction of crest road traffic during construction, construction traffic impacts, etc. Because access on the crest road and upstream side of the embankment is limited by both space and existing infrastructure, equipment size and accessibility were the main consideration during selection of the cutoff technology. Based on cost, accessibility, the reduction of spoils, and equipment flexibility, the Cutter Soil Mixer (CSM) option was recommended.

A special localized grouting treatment is planned around the left abutment of the Service Spillway to address seepage that could be travelling along the interface of the abutment wall and the backfill. This is also necessary due to the geometry of the training wall counterforts (refer to **Figure 2**), which are expected to obstruct installation of the CSM panels at the far right end of the cutoff.

The final design is currently underway and project construction is expected to begin during the first quarter of 2021. The total construction duration is estimated at 4 months.

CLOSING

The variable geology at this site has presented adverse conditions during this facility's life, with persistent seepage being the primary challenge. This challenge was present during the original construction but was not insurmountable because the reservoir head was not present.

However, construction of the morning glory spillway was abandoned in the mid-1960s due to uncontrollable seepage, and aggressive dewatering was required for the successful Side Channel Spillway construction in the 1970s. A concrete cutoff between the spillways was recommended but was never constructed, for reasons that are not clear. Attempts have been made to control the persistent seepage in the intervening decades but have met with limited or temporary success. It is clear that the original intent is a suitable solution, and the design of a cutter soil mixer (CSM) cutoff wall is currently underway and planned for construction in the first quarter of 2021.

* * *

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A Comprehensive Assessment of a Dynamic Compaction Operation Using Pre- and Post-Compaction Shear Wave Velocity Measurements and Attenuation Studies

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ABSTRACT

In a comprehensive study, both the compaction-induced vibrations and extent of ground improvement were investigated at a dynamic compaction site. The site of interest was to be compacted by a 16-t (32 kips) weight. First, an attenuation study was conducted to assist in controlling ground vibration from the weight drops. The attenuation study consisted of 20 pilot drops from different heights ranging from 20 to 50 ft. The vibrations were recorded by 20 three-component geophones deployed linearly from 50 to 400 ft from the drop locations. After establishing the site-specific equation of attenuation, weight drop heights were established for different zones based upon the distance to off-site structures. Off-site structures were monitored at select locations during the production phase of the compaction to ensure vibrations were maintained below the established limit. Additionally, pre- and post-compaction measurements of the shear-wave velocity (V_s) were performed using the multichannel analysis of surface waves (MASW) method. MASW is a non-destructive seismic method that measures the dispersion of surface waves (Rayleigh or Love to estimate subsurface stiffness). MASW results indicated that the V_s of the uppermost 8 ft of subsurface increased by up to 300 ft/s, about 60% of its pre-compaction velocity. The next 6 ft exhibited negligible changes in velocity. This agreed reasonably well with the estimated depth of influence for this operation.

INTRODUCTION

Dynamic compaction is a ground improvement technique that uses weight drops to densify loose soils. Two topics are of interest when dynamic compaction is performed. One is the level of vibrations caused by a weight drop. The compaction operation must be executed in a manner not to create vibrations that could damage nearby structures or utilities. This concern is often addressed by establishing vibration criteria to limit the vibrations. The second topic of interest is the amount of ground improvement and the corresponding depth of influence. This can be estimated using nondestructive testing methods utilizing seismic waves. Surface wave testing methods such as Multichannel Analysis of Surface Waves (MASW) offer the potential to determine shear wave velocity (V_s) profiles that can be used as a direct proxy to the rigidity of a soil/rock formation. MASW estimates the subsurface stiffness by measuring the dispersion of surface waves. In a layered medium, surface waves exhibit a dispersive behavior meaning that different frequency components of an input signal travel at different velocities as they propagate through the domain. The characteristic dispersion behavior at a given location (i.e., the dispersion curve) is subsequently used in an inversion process that generates a subsurface V_s model that best matches the measured dispersion behavior. One of the earliest studies that employed MASW, after its formal introduction in 1999, to assess ground improvement was Park and Miller (2003). In the following years, several studies employed MASW in a similar manner to assess the ground improvement by performing before- and after-treatment testing (e.g., Burke and Schofield 2008; Waddell et al. 2010; Karray et al. 2010; Bitri et al. 2013; Donohue et al. 2013; Jafri et al. 2017; Lu et al. 2018; Mahvelati et al. 2020). Expectedly, a common observation in almost all such studies was that the shallow parts of subsurface exhibited larger post-compaction V_s, where the threshold for "shallow" depends on the site geology and compaction equipment. However, as it will be discussed later, despite an intuitive expectation, several of these studies reported some reduction in V_s after dynamic compaction not in the shallow deposits but at greater depths (e.g., Park and Miller 2003; Bitri et al. 2013). A common explanation for this observation was that many compaction techniques, including dynamic compaction and blasting, destructs the original cohesive bonding between soil particles that had been strengthened throughout many decades (Park and Miller 2003; Mahvelati et al. 2020) and therefore, an immediate post-compaction measurement would only reveal such stiffness reductions.

This study summarizes the results from a comprehensive study at a deep dynamic compaction site where vibration criteria for adjacent structures were determined, an attenuation study was conducted, and surface wave testing was performed. The purpose of the attenuation study was to determine the site-specific vibration attenuation rate and utilize this attenuation rate to determine if any changes should be made to the drop height in compaction areas in close proximity to neighboring structures to ensure compliance with the established vibration criteria. Once the rate of attenuation was defined, the vibration levels at a certain distance and a given energy can be predicted with a high degree of confidence. In addition to the attenuation study, surface wave testing was carried out using the Multichannel Analysis of Surface Waves (MASW) method (Park et al. 1999; Xia et al. 1999) prior to and after the dynamic compaction to evaluate the depth and amount of compaction.

SITE LOCATION

The testing location is a construction site operated by Premier Design and Build Group in Phillipsburg, NJ (Figure 1-a). The map shows the approximate area to be compacted, MASW spread, and nearby railroad tracks. Previous geotechnical subsurface explorations, all conducted within the areas of proposed construction and not between the site and nearby structures of concern, have demonstrated that the top 2-5 feet of fill material is underlain by yellow to brown silty clays varying from very soft to hard in consistency with traces of fine gravel. Weathered and fractured dolomite bedrock was typically encountered at 20 feet and deeper. The structures of concern are the railroad tracks, and the structures to the North and Southwest of the site. The dynamic compaction company would like to get as close to the areas of concern without causing vibration-induced damages.

For the attenuation study, a total of 20 three-component seismometers recorded ground vibrations resulting from the drop of the 16-ton weight. All the seismographs used have a dynamic range up to 10.0 in/sec, and a sampling rate of 1024 samples/second/channel. One linear array consisting of the 20 seismometers was deployed running from the impact area south/southwest of the site. A total of 20 drops were completed for this investigation. The 16-ton weight was dropped from various heights staring from 20 feet and going up to 50 feet at 10 feet intervals and 5 drops from each height (i.e., 4×5=20 total drops). The 4 to-be-compacted spots (one for each height) were arranged on a 2×2 grid, about 10 feet apart center-to-center. The instruments were programmed to record vibration levels that exceeded a threshold level of 0.03 in/sec. The record length was set at 3 seconds. The tri-axial geophone systems were buried approximately 6 to 8 inches in the ground and utilized three 3-inch spikes to ensure proper coupling.

(a)

(b) **(c)**

Figure 1. (a) Site map: red line is the MASW spread, green is the attenuation line, yellow box is the approximate area to be compacted, and blue line is the nearby railroad tracks; (b) and (c) 16-ton weight drop.

ATTENUATION STUDY

A site-specific velocity attenuation equation can be developed by applying statistical analysis techniques to the vibration data pairs (Peak Particle Velocity vs. Scaled Distance). Scaled distance (*SD*), as referred to in this study, equals the distance from the impact point to the recording location (in ft.) divided by the square root of the energy which is the multiplication of the weight by the height of drop ($E=WH$ in ft.-tons). The relationship between peak particle velocity and scaled distance can be obtained by determining the "best fitting" line to the plotted data. The mean (best fit) curve of the plotted data represents the best approximation of the amplitude of the vibrations from an energy source at a given scaled distance. Statistically, one half of all vibrations recorded at a given scaled distance should exceed the mean amplitude, the