REAL-TIME SEEPAGE RISK REDUCTION MEASURE
DESIGN AND IMPLEMENTATION AT ZOAR LEVEE

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ABSTRACT

Zoar Levee is located along the Tuscarawas River in the Dover Dam pool of Ohio and provides protection to the historic Village of Zoar. It was constructed on a foundation consisting of pervious bedrock and glacial outwash during 1937, prior to the advent of modern stability analysis and design approaches. During March 2008, the Muskingum River Basin experienced considerable flooding, and Zoar Levee was loaded with a 52-year pool by Easter Sunday. Seepage through rock and soil foundation materials early in the event progressed into conditions which posed a threat to the integrity of the southeast portion (pump station reach) of the levee. Under the 52-year flood frequency event, uncontrolled seepage occurred over a 300 by 550 foot downstream area and discharged an estimated 5000 gallons per minute (gpm). Within this area, numerous boils having openings up to 1.5 feet in diameter were present and produced up to 300 gpm each and moved a significant amount of foundation soils. Due to the intolerable level of risk posed by these conditions, immediate action was taken to construct a seepage berm as an emergency risk reduction measure. The situation’s urgency coupled with a lack of historic subsurface data forced adaptive design methods to be employed in the field to stabilize the levee and reduce associated risk to tolerable levels. Within 36 hours of initiating design and construction, problematic conditions were stabilized, and within 150 hours, over 37,000 tons of granular material had been placed, and the pump station reach of the levee was provided with a level of protection compatible with the Dover Dam interim operating (90-year event) pool. Design of a permanent solution to further increase the level of protection for Zoar Levee to its design pool is underway. This paper discusses observed seepage deficiencies and geotechnical aspects of the levee’s pump station reach, and the March 2008 real-time risk reduction measure design and implementation.

INTRODUCTION

Zoar Levee provides flood protection to the historic Village of Zoar during high pools occurring on the Tuscarawas River. It was constructed in 1937 as an appurtenant structure to Dover Dam, which is located 4.7 miles downstream. Dover Dam normally operates as an open river dam, but retains a pool during flood events to mitigate damage in the Muskingum River Basin. The basin covers approximately 20% of the State of Ohio, and within it, the Tuscarawas River is one of the major tributaries of the Muskingum River, and the drainage basin of Dover Dam is approximately 1,400 square miles.

Zoar Levee was originally constructed to elevation 919.0 feet (three feet above the Dover Dam spillway crest), but was raised to its current elevation of 928.6 feet during 1950. Presently, the levee has a crest length of 3,900 feet and a maximum height of 45 feet, measured from the riverward toe to the top of crest. For discussion purposes, the levee is divided into two major subreaches (Figure 1A): the ball field reach, and the pump station reach. The pump station...
Figure 1. Northward (A) and southward (B) aerial 3D views of Zoar Levee with the Tuscarawas River at normal stage. The entire levee is shown in (A) while the pump station reach is framed in (B).
reach (see close-up view in Figure 1B) is the focus of this paper; the reaches are separated by State Route 212 which crosses over the levee. The pool of record (907.4 feet; 75-year event) for Zoar Levee occurred during January 2005. During the record event, pervasive area seepage in both project reaches, and boils in the pump station reach were closely monitored. During March 2008, Zoar Levee was loaded with a 904.6 feet pool (52-year event), and seepage conditions similar to those experienced during 2005 were observed early in the event, but at considerably lower pool elevations than they occurred at in 2005. As the 2008 event progressed, seepage conditions became much more adverse in the pump station reach than observed during 2005. With conditions continuing to deteriorate, and the structural integrity of the levee threatened, immediate action was taken to design and construct a seepage berm in order to reduce the imposed intolerable level of risk. This paper focuses on seepage instability during the March 2008 flood, and the real-time emergency remedial work that was performed to mitigate observed instability and raise the pump station reach to a level of protection (pool elevation of 909.0; 90-year event) compatible with the current Dover Dam interim operating pool.

Geology and Subsurface Conditions

The Zoar region has not been covered by glaciers, but glacial ice advanced to the immediate north of the project area. Landform characteristics of the area have been produced primarily through depositional and erosional action of running water. The terrain is generally rugged and hilly, with flat areas primarily occurring within floodplains and on flat-topped terrace features produced by glacial outwash deposition. Outwash and lake deposits occurred in the floodplains and other low lying areas that existed below the glacial margin; these deposits generally consist of gradational layers of clay, silt, sand and gravel. Throughout the Muskingum Basin, these (pervious) sands and gravels are good groundwater resources.

Zoar Levee is separated into two reaches having distinct subsurface conditions. The ball field reach is typified by glacial and alluvial deposits extending to depths up to 140 feet below the base of the levee. The pump station reach is characterized by glacial and alluvial deposits of variable thickness, situated on a localized bedrock high that extends upward to the base of the levee; this area was “high ground” prior to the raising of the levee in 1950. Figure 2 shows a topographic map for the pump station reach; note the ground surface in the vicinity of piezometers CD-07-117, CD-07-121, CD-07-122, and D-94-16, which intercepts the side slope of the levee near the crest. Figure 3 contains generalized cross sections showing the actual subsurface conditions encountered in the borings acquired in this reach.

Boring CD-07-121 was acquired along the levee crest (Figure 2), and encountered levee embankment material comprised of lean clay extending from the ground surface to the top of bedrock at elevation 919.0. The upper 6.6 feet of bedrock at this location consists of highly weathered shale. Underlying the shale (elevation 912.4), highly weathered sandstone with a fracture zone and stained joints extends to a depth of 33.8 feet (elevation 895.3) with one 3.5 feet thick shale stratum between elevations 910.8 and 907.2. Underlying the sandstone, interbedded shale and sandstone was encountered and extended to an elevation of 884.2. Several vertical fractures were observed in the interbedded shale and sandstone. Limestone was encountered underlying the sandstone (elevation 882.7) with the lower horizon at a depth of 50.7 feet. (elevation 878.4). The driller’s log indicated a 0.6 feet void encountered at the contact between the interbedded shale/sandstone and the limestone. Generally, the limestone was un-weathered and hard with no solution cavities observed in the obtained core. Other area borings (Figures 2 and 3) indicated the presence of fine grained soil generally overlying more pervious sand and gravel deposits. Alluvial surficial clay blanket was four feet thick at boring CD-07-118 and thickened in the direction of boring D-07-125; at boring CD-07-122 such blanket was non-existent. Borings CD-07-117 and CD-07-122 encountered bedrock at elevations
Figure 2. Topographic base map for the pump station reach of Zoar Levee. Boring and piezometer locations (heavy black text) are shown along with transects A-A and B-B which indicate locations of the geologic cross-sections shown in Figure 3.
Figure 3. Geologic cross-sections along transects A-A and B-B shown in Figure 2. Maximum piezometric elevations recorded during the March 2008 flood event (Figure 4) are plotted on the sections.

ranging from 892.3 to 896.9, respectively; the bedrock conditions were relatively uniform in these borings, with shale encountered at the bedrock surface extending to an elevation of approximately 888.0 to 889.0. Limestone was encountered underlying the shale and ranged from three to four feet in thickness. The driller's logs indicated a 0.2 feet void at the limestone and overlying shale contact (CD-07-122), and loss of water at the limestone and underlying shale contact (CD-07-117). Shale underlying the limestone in these borings extended to boring termination at an elevation of approximately 880.0. Piezometers were installed in the borings shown in Figure 2 except for CD-07-115; piezometric data acquired during March 2008 are shown in Figure 4. The tips of piezometers CD-07-117, CD-07-121, CD-07-122, D-07-125, and D-94-16 were positioned in fractured bedrock zones at the approximate limestone elevation, whereas the tips of the other three piezometers (D-07-116, CD-07-118, CD-07-119) were positioned in sand and gravel deposits.
Figure 4. Dover Dam pool elevation data and piezometric data acquired for the pump station reach of Zoar Levee (Figure 2) during the March 2008 flood event.

MARCH 2008 FLOOD EVENT

During the first three weeks of March 2008, the drainage basin which affects Zoar Levee was subjected to steady precipitation (rain and snow) which fell on already saturated ground. A classified blizzard contributed about 14 inches of snowfall during the month’s second week. According to Monthly Water Inventory Reports for Ohio (published by the Ohio Department of Natural Resources), the Zoar region received the first and fifth largest amounts of precipitation for the months of February and March respectively in the state’s 126 years of record. As shown by the Dover Dam pool elevation data plotted in Figure 4, steady regional precipitation resulted in the pool against Zoar Levee rising from an elevation of 895.6 (6-year event) on 10 March to an elevation of 904.6 (52-year event) on 23 March. The levee conditions were monitored on a continual basis by the Huntington District’s geotechnical personnel throughout the March 2008 flood. As seepage conditions at the levee progressed during the event, monitoring consisted of intense 24-hour surveillance.

Progression of Seepage Instability

During inspection on 10 March, several small boils and channelized area seepage flow developing in the pump station reach were monitored (Figure 5A). The boils had been sand-bagged a few days earlier as a precautionary measure against losing foundation soils. The lateral extent and maximum elevation of the seepage emergence front were also marked as a reference to allow the gauging of anticipated seepage progression. By 18 March, with the pool
Figure 5. Photographs of March 2008 seepage conditions in the pump station reach of Zoar Levee; pool elevations for dates shown in the photographs are contained in Figure 4. Early in the event, boils were sand-bagged and channelized flow of seepage developed (A). As the pool elevation increased during the next two weeks the number of boils, the size of boils, and the total quantity of seepage across the area increased significantly (B to D). Boils repeatedly expanded beyond and collapsed sandbag rings, and coalesced at multiple project locations (D) into large unstable regions. Several boils had openings up to 1.5 feet in diameter (E), and produced up to 300 gpm each and moved a significant amount of foundation soils. In F, sand bag removal from boils is shown as the seepage berm advances down slope.
elevation having risen to an elevation above 900.0, the quantity and extent of emerging seepage and the size of the boils had substantially increased (Figure 5B). On 22 March with the pool elevation above 904.0, sand-bagged boils had enlarged to diameters of over one foot, and substantial amounts of foundation soil were being deposited around their openings. Individual boils were estimated to be producing approximately 300 gallons per minute (gpm) with the entire pump station reach estimated to be yielding approximately 5,000 gpm of seepage. Sand bag rings were beginning to fail due to material piping out from beneath them, and in attempt to minimize occurring collapses, all sand bag rings were fortified to four or five bags wide at their base. As the seepage was monitored, it was evident that the exit elevation (Figure 5C) was continuing to increase, and that new boils were developing at a rapid rate. These observations were consistent with increasing piezometric trends; by this date, piezometric data were being acquired and analyzed at a 6-hour interval.

On 22 March, many smaller boils began coalescing to create additional larger boils capable of moving more foundation material faster, and large unstable regions were developing. A photograph taken on March 24 (Figure 5D), although perhaps showing slightly more developed conditions than were observed on 22 March, illustrates such conditions. Figure 5E shows a close up view of a large sand-bagged boil; when this boil was probed its base was found to be at a depth approximately four feet, and it appeared to continue backwards laterally as an open pipe in the direction of the levee. As subsequently discussed, all sand bags were removed from boils prior to being covered by seepage berm material (Figure 5F). On 22 March, the seepage situation had reached a point where it was no longer manageable through sand bags; when they were stacked high enough to stop material movement at a particular boil location, another boil adjacent to the ring would immediately develop as a result. With erosion continuing, the next increment of conditions deterioration that the authors expected to possibly see would have been the development of one or more very large erosional features encompassing regions of coalesced boils and propagating rapidly backwards towards the levee. Field interpretations of draft boring logs (piezometers had only recently been installed in the pump station reach) and piezometric data acquired thus far in the event indicated seepage through bedrock was responsible for observed conditions. Based on the authors’ experience with pervious glacial outwash materials in the region, as well as acquired piezometric data, it was determined probable that seepage under the levee in these materials was a contributing factor. Potential failure modes envisioned included levee undermining due to backward pipe propagation (through outwash), and/or levee collapse due to embankment erosion caused by large unrestricted discharge through rock (note that no signs of embankment erosion were observed during the March 2008 flood event). Based on the above geotechnical considerations, and the fact that hydraulic predictions on 22 March indicated that the Dover pool would exceed, and then not decrease below its current elevation (>904.0) for at least another week, the decision was made in the late evening to reduce the imposed risk to a tolerable level through emergency seepage berm design and construction.

**Emergency Contractor Mobilization**

After notifying the Contractor (TAB Construction) to proceed late in the evening of 22 March, equipment was immediately mobilized to Zoar Levee. The District holds Indefinite Delivery Indefinite Quantity (IDIQ) contracts with several regional Contractors; TAB Construction is located in close proximity to Zoar Levee, was qualified to perform the work, and was therefore selected as the Contractor. Flatbeds with dozers arrived at approximately 1 am on 23 March (Easter morning), and by 7 am approximately 20 tractor trailers loaded with stone had arrived. During the 2005 flood event, an attempt was made to construct a small filter over boils in the pump station reach but was abandoned due to soft ground conditions; the areas of concern were not traversable by tracked equipment. Therefore, it was decided that an access road would be built during the 2008 flood event in order to ensure material could be effectively
delivered to areas of concern. Anticipating that construction would be very difficult given the soft ground conditions associated with seepage areas, it was decided that low ground pressure equipment (e.g. small dozers - Caterpillar D-4) would be utilized. Additionally, it was specified that a minimum three foot thick layer of granular berm material would be maintained at all times during construction in heavy seepage areas. The decision to not use filter fabric in association with the emergency seepage berm construction was made in order to avoid the future possibility of bridging over potential voids and not being able to see surface expression of material loss.

SEEPAGE BERM DESIGN AND CONTRUCTION

Shown in Figure 6A is an aerial photograph of Zoar Levee taken on 23 March which includes both the ball field and pump station reaches of the levee. In this photograph, the Dover pool is at its event crest elevation of 904.6 (Figure 4) and area seeps in the ball field reach of the levee are indicated. Shown in Figure 6B is an aerial photograph which frames the pump station reach and notes the regions of area seepage and large boils; the total length of the area seepage (along levee centerline) was approximately 550 feet. Also visible in Figure 6B is the initiation of construction of the access road to the pump station reach. Approximately 24-inches of dense graded aggregate (approximately 2,400 total tons) were utilized with geotextile reinforcement/separation fabric to construct the access road and offloading facility/staging area. An average of 24 tons of aggregate per truck was delivered to the site throughout the duration of construction. The access road was completed by 6 pm on 23 March, such that berm material could then begin being placed on the land side of the levee where seepage was occurring.

Material Requirements

An early decision required during seepage berm design was the specification of material type so that sufficient quantities could begin being delivered. As discussed earlier, several borings were recently taken in the vicinity of the boils, however, only field logs of the boreholes were available since the obtained samples were in the laboratory undergoing index testing for proper classification. The objective in the area of the boils was to choose a material that would have a high enough conductivity to handle the large flows exiting the boils and area seepage (Figure 7A) while being able to filter soils that were observed in and around the boils. As shown on the cross sections in Figure 3 and as described previously, the area of the boils consisted of sand and gravel deposits exposed near the ground surface. Figure 7B shows a close up of one of the large boils after the sand bag ring had been removed; note the presence of foundation gravels. For these reasons, and considering constructability on the extremely poor foundation conditions (Figure 7C), #8 (pea-sized) river gravel was chosen for the material to construct the pump station reach seepage berm. In other proximate areas (e.g. where relatively-impervious surficial clays existed landward of the boils), the objective was to add resisting weight to minimize the potential for erosion initiation due to surficial blanket uplift. As shown in Figure 7D, the selected berm material performed well once placed under near maximum event loading, as it resulted in no apparent head loss, no further foundation soil loss, and a uniform distribution of discharging seepage.

Remedial Design and Implementation

At the start of the emergency response, the initial remedial goal was to stabilize the areas of seepage instability for the predicted Dover pool crest elevation of 904.6 and event duration. As mentioned above, hydraulic predictions prior to 23 March indicated the pool would remain above elevation 904.0 for at least another week, however, in effort to reduce the magnitude and duration of loading on the levee, the District made regional hydraulic operation adjustments shortly after construction initiation which reduced the pool elevation quicker than originally
Figure 6. Aerial photographs of Zoar Levee (A) and the pump station reach of Zoar Levee (B) when the Dover Dam pool was at elevation 904.6 on Easter Sunday. Areas where large seepage quantities and extensive pin boils occurred are apparent (as standing land side water regions) along the ball field side of the levee (A). Construction of the access road to allow pump station reach seepage berm construction is evident, and the extents of heavy seepage and large boils (shown in other figures) are indicated (B).
Figure 7. Photographs illustrating some of the issues relevant to the selection of seepage berm material (#8 pea gravel). The total area seepage was approximately 5000 gpm (A), and borings in the immediate vicinity of boils and visual observations (B) indicated the presence of poorly graded gravels in the near-surface. Given large discharge, adequate placement potential of sand was questionable, and allowing discharge to continue without restriction was important. Even while maintaining a minimum 3 foot thick work platform during berm construction, a dozer became buried up in gravel sloughing due to emerging seepage (C). The berm was observed to perform well under near maximum event loading, resulting in no apparent head loss, no further foundation soil loss, and a uniform distribution of discharging seepage (D).

predicted (Figure 4). At this time, the District also developed and was ready to implement a contingency plan if necessary, which would have consisted of installing a flood fighting structure (e.g. sand bag closure) across low elevation regions along the pump station reach rim ridge and back flooding the pump station reach in order to equalize head on either side of the levee with minimal impacts to the community. After the access road was completed through an intense around the clock construction schedule, seepage berm construction began at approximately 6 pm on 23 March. Berm material was continuously delivered to the site throughout the night, and off-road trucks transported the material across the levee to near the areas of concern. As the sand bags were removed from the boils on the morning of 24 March, just prior to berm material placement over them (Figure 5F), the discharge and amount of foundation soils exiting from the boils increased considerably. As the first and second waves of berm material were pushed over the boils, it was quickly removed by the exiting flows, and the upslope piles of berm material
began sloughing towards the boils (Figure 7C); this caused one of the dozers to get stuck and require winching out by the other. With the third and subsequent pushes of berm material over the boils, the boils started to become adequately covered with stone, and seepage began emerging uniformly from the face of the newly constructed berm (Figure 7D). By the late morning of 24 March, the initial design goal of stabilizing seepage conditions was accomplished as a minimum 3 foot thickness of berm existed over a 550 foot by 100 foot area containing the heaviest seepage and boils.

As emergency work was progressing during the event, a second remedial goal was developed on 24 March by the District: the seepage berm was to be constructed such that the pump station reach of the levee would have a level of protection for seepage instability compatible with the Dover Dam interim operating (90-year) pool. Designing to this level of protection would serve as an interim (prior to completion of a permanent solution) risk reduction measure, and would provide a tolerable level of risk for the pump station reach of the levee up to a Dover pool elevation of 909.0. The design criteria selected by geotechnical personnel in the field for this phase of remedial construction included ensuring that for a pool elevation of 909.0, the projected piezometric surface would be below the top of berm in the area of large boils, a (effective stress) factor of safety (FS) against foundation erosion initiation of greater than or equal to (>=) 2.0 would exist everywhere along the levee toe (i.e. between the pump station and CD-07-117 and D-07-125), and a (effective stress) FS against foundation erosion initiation of >= 1.1 would exist everywhere along the landward extent of the seepage berm. Designing to a minimum FS of 1.1 at the berm toe was determined to be reasonable for several reasons including: this was an interim risk reduction measure and a permanent remedial solution is currently being designed, if the pool reaches an elevation of 909.0 geotechnical personnel will be performing constant area surveillance, access to the pump station reach now exists should future emergency measures need to be quickly implemented, and the reach could be back-flooded (by raising the pump station tail race) to help counteract any erosion initiation which may occur; at least 10 feet of water could be impounded without impact of structures or people in this reach.

Given the need to immediately accomplish the above-described second remedial goal, the lack of subsurface and historic piezometric data (this was the first significant pool since the piezometers were installed in 2007), and the relatively limited spread of piezometers in the pump station reach, adaptive design methods had to be employed in the field. Typical extensive office-based analyses utilizing lab data, spreadsheets, and modeling were not possible. Note that field design methods and decisions have been re-evaluated subsequent to the March 2008 event using hindsight and all data acquired to date, and are still considered as valid and summarized herein. Designed berm thickness and extents were based in the field on evaluations of available borings and the piezometric data acquired through 24 March. A number of typical factors influenced the piezometric responses observed in Figure 4, including: the piezometer location, tip elevation, tip sensing zone/material, and the pool elevation. It is clear based on readings that the bedrock and outwash deposits were each communicating well with the pool; an immediate rise in bedrock piezometric elevations with pool occurred, while outwash deposits exhibited a one to two day lag time in their piezometric response. It is also clear from the data (as well as from the boils and area seepage) that minimal head loss was occurring through these materials. Also influencing piezometric responses were non-typical factors which included berm construction and the development of a probable secondary seepage entrance (to the south and southeast of the levee) as the pool elevation increased and filled a normally dry tributary channel. Effects of berm construction on the piezometric response is particularly evident by observing the noticeable rises in reading elevations for CD-07-117, CD-07-121, CD-07-122, and D-94-16 (bedrock piezometers located in the area of initial thick berm placement) which occurred subsequent to the peak pool elevation.
To facilitate an improved understanding of head distribution across the pump station reach so that required berm thickness and extent design could be best determined a maximum piezometric head distribution map was constructed (Figure 8). This map contains interpreted contours based on maximum piezometric elevations (Figure 4) recorded in rock and soil units during the event. The map is also based on field measurements and observations, and was verified by subsequent analyses of aerial photographs taken during the event; these approaches consisted of gauging excess head amounts in the vicinity of boils, and mapping seepage emergence and standing water elevations versus location. Certain tradeoffs regarding groundwater theory were made in constructing this map (e.g. in the combination of head values from different subsurface units, and in the mapping all flow as horizontal without vertical gradients), however, the approach was deemed reasonable given the situational constraints under which design had to occur, and it provided practical benefits. For instance, it is clear from Figure 8 that head distribution in the reach is strongly governed by flow through the bedrock high located near piezometers CD-07-117, CD-07-121, CD-07-122, and D-94-16. The head distribution is also seen to be affected by the natural high ground in the eastern part of the reach, by the (pump station controlled) tail race elevation, and by seepage coming under the levee through outwash deposits between the pump station and CD-07-117. Using the groundwater head values and ground surface elevations on this map, excess head values were able to be approximated at locations where piezometers did not exist. Developing this capability was important given that the berm characteristics had to be defined between piezometers having wide spacing in some areas (e.g. landward of the large boils). By using this map, projecting subsurface data from available borings, and then projecting maximum piezometric elevations resulting from the 904.6 pool elevation to the design 909.0 pool elevation, FS values against foundation erosion initiation were able to be calculated, and the seepage berm thickness and extent requirements were able to be designed.

To illustrate the design process, a description of analyses in the location of certain large boils (Figure 6) is presented. The top of ground elevation at this analysis location was 895.5 and the effective tail water (selected as the immediate landward standing water elevation in a drainage ditch) was 890.0. Based on the immediate upslope seepage emergence line (Figure 5C) elevation and approximate gauging at the boils, the total head (elevation plus pressure heads; velocity head likely negligible) at this location resulting from the 904.6 pool loading was interpreted as 898.0. This resulted in a calculated net head remaining (NHR) value of 54.8% \[ \text{NHR} = 1 - \frac{(\text{pool elevation} - \text{piezometric head})}{(\text{pool elevation} - \text{effective tail water elevation})} \], which was then used to project the total head at this location under the interim design pool loading. With the total head projected to be 900.4 under the 909.0 pool, the design required the berm top to be a minimum of 1.5 feet above 900.4; this requirement was met during construction with the final as-built top of berm elevation verified to be 902.5 at this location. As another example of the design process, a description of analyses at piezometer CD-07-118 is presented. The top of ground at this location is 889.0, and the total head resulting from the 904.6 pool loading was 892.4 (3.4 feet of excess head was measured here using a riser extension on the flush mount piezometer). In boring CD-07-118 a four foot thick alluvial clay blanket was encountered, and a buoyant unit weight of 57.6 pounds per cubic foot (pcf) for the blanket was assumed. By conservatively assuming the excess head value measured at the piezometer tip (located at a depth of 15.0 feet in the outwash) existed at the blanket base (elevation 885.0), an effective stress uplift FS value of 1.09 (equivalent to 1.04 total stress uplift FS) was calculated for this location under the 904.6 pool loading. For an effective tail water elevation of 889.0 (given regional emerging seepage/standing water at this elevation) the calculated NHR value was 21.8% at piezometer CD-07-118. Using this NHR value, the total head was projected to be 893.4 under the 909.0 pool, and this resulted in an effective stress uplift FS value of 0.85 (equivalent to 0.92 total stress uplift FS). Based on these results, the seepage berm toe was located further landward of CD-07-118, and the calculated FS value at the piezometer was greater than 1.1 for the design (909.0 pool) loading. Numerous calculations
Figure 8. Maximum piezometric head distribution map for the pump station reach of Zoar Levee during the March 2008 flood event. Interpreted contours are based on piezometric data (Figure 4) acquired in rock and soil units, field measurements and observations, and analyses of aerial photographs taken during the flood. The position of the seepage berm perimeter which was constructed (as shown in subsequent figures) relative to head distribution contours is indicated by the heavy black line. See text for discussion.
such as those described above were performed in order to define the seepage berm thickness and extent requirements across the pump station reach, and surveying during and after construction ensured that they were met.

In order to meet the second remedial goal of providing the pump station reach with a level of protection compatible with the Dover Dam interim operating pool (El. 909.0), the seepage berm was expanded to the limits and thicknesses shown on Figures 9 and 10, respectively (using the design approach described above). The final shape of the seepage berm as it appears in plan view was determined based upon calculations made during the design process. The berm is seen to extend beyond piezometer D-07-125 along its eastern edge due to the low rate of head dissipation with distance measured in bedrock. Along the north edge of the berm, its toe configuration is non-uniform due to interception of pre-existing topographical variations (i.e. the berm was not always required where it tied into relatively high ground). As construction progressed, the Contractor was placing approximately 6,000 tons of material per day utilizing two 30 ton off-road trucks, two dozers, a loader, and an excavator. Figures 11A to 11F contain photographs illustrating the entire construction sequence from start to finish. Figure 11D shows an aerial view of the on-going construction on 26 March; at this time the total construction effort was approximately 60 percent complete. By 28 March, the berm was approximately 90 percent complete as shown in Figure 11E. Since the seepage areas of most concern had been covered with the seepage berm by 28 March, the Contractor was directed to back off to day shifts in an effort to conserve resources. By the morning of 29 March, and 150 hours after the initiation of design and construction efforts, approximately 37,000 tons of material had been placed and construction of the seepage berm was essentially completed. A two foot thick layer of #2 sized stone was utilized as a filter at the out slope of the berm to keep material from rolling and to allow the water to escape as shown in Figure 11F. In Figure 12, panoramic views of the completed seepage berm along with the labeled final dimensions, and an isometric topographical map illustrating the layout are shown.

CONCLUSIONS

The March 2008 flood event at Zoar Levee initiated seepage instability manifested through numerous boils producing up to 300 gpm each and a large area seep that threatened the integrity of the embankment. The seepage instability posed an intolerable risk and progressed to a level that could no longer be reasonably controlled with sand bag rings. At this point, a Contractor was mobilized to the site and construction of emergency seepage remediation measures commenced. Meanwhile, a design was being developed not only to mitigate the observed instability, but to provide a level of protection for the pump station reach of Zoar Levee which would be compatible with the current interim operating pool (elevation 909.0) for Dover Dam. Limited subsurface information and piezometric data were available for this particular reach of the levee. A seepage berm was designed in the field based on the available field logs, observed conditions, and piezometric data obtained during this event. A net head remaining concept along with effective stress seepage FS equations were utilized with piezometric data in order to project the head distribution and FS values that could be anticipated for the interim design pool. This was done to determine the aerial extents and thickness of the berm that would ensure a factor of safety against initiation of erosion of the foundation material greater than 2.0 and 1.1 at the toe of the levee and the toe of the seepage berm, respectively at a 909.0 pool. Additionally, the design ensured that the projected piezometric surface at the locations of boils would be below the top of the berm under the design loading.

There are potential failure modes that cannot be ruled out with this interim remedy in place given the low head loss through the bedrock high located in the foundation coupled with the lack of subsurface investigation in this reach. Currently, supplemental subsurface investigation plans are being developed as part of the on-going Major Rehabilitation Study for Zoar Levee,
Figure 9. Topographic base map with emergency access (road and staging) area and seepage berm (emergency blanket) with as-built top elevations shown. Piezometer and boring locations (heavy black text) are shown along with transects A-A and B-B, which indicate the locations of the post-construction cross-sections in Figure 10. The maximum March 2008 pool and piezometric elevations (blue text) which were contained in Figure 8 are also shown here for illustrative purposes. See text for discussion.
and investigation results will be used to develop a permanent remedy for both the pump station and ball field reaches of the project that fully considers all potential failure modes. As a result of the performance observed during the March 2008 flood event, the USACE Senior Oversight Group has recommended that Zoar Levee be reclassified as a Dam Safety Action Class I (from a Class II ranking). This means that the conditions at Zoar Levee are urgent and compelling with a need for immediate intervention and that the structure was observed to be critically near failure during normal operations. As the Major Rehabilitation Study progresses, interim (prior to permanent fix implementation) risk reduction measures will continue to be implemented to help reduce present risk, and the Huntington District will remain at a heightened state of alert with intense monitoring and surveillance during future loadings prior to implementing the final seepage remediation.
Figure 11. Photographs showing seepage berm construction sequence. Remedial design and construction initiated around midnight on Easter Sunday, and road access to the area of concern had been established by Easter evening (A). Within 36 hours, areas of heavy seepage and boils were stabilized (B). Construction proceeded rapidly over the next several days (C, D, and E), and within 150 hours of design and construction initiation, over 37,000 tons of granular material was placed, and the pump station reach of the levee was provided with a level of protection compatible with the Dover Dam interim operating pool. A post-construction view of the downstream seepage berm toe (faced with #2 stone) is shown in F.
Figure 12. Eastward (A) and northeastward (B) photographic views of the seepage berm taken several weeks after construction was completed. A southwestward isometric view of the completed seepage berm is shown (C); the 904.6 foot pool and 886.0 foot tailrace extents are shown for illustrative purposes.
Potential Failure Modes

The discussion provided above described the piping initiation and continuation that was observed at Zoar Levee. During the assessment and analysis of the observed conditions, limited subsurface and piezometric data were available to characterize and identify the possible entrance and path for the observed emerging seepage. It was apparent that piping of the foundation material had initiated and appeared to be working backwards toward the levee, threatening to undermine the foundation and possibly result in collapse of the levee and flooding the Village of Zoar. The constructed seepage berm interrupted foundation erosion continuation in that it acted as a filter for the observed surficial soils in the area of the boils while allowing the seepage to freely exit. Further landward in the flatter area, the berm material is relied upon solely for additional vertical stress to increase the factor of safety against uplift of the natural blanket. The unknowns at this point in time are: the exact locations at which the foundation is connected to Dover pool and the media extent through which seepage is traveling directly beneath the embankment with less than adequate head loss. Given that the embankment is founded on bedrock in the area of the bedrock high, it is possible that levee material could be eroded through joints in the rock until it is eventually undermined. The aforementioned considerations are intended to explain that while the observed initiation and continuation of piping was interrupted by the seepage berm designed to withstand a pool up to elevation 909.0, that no visual evidence existed of erosion of embankment material into the rock foundation. Therefore, the emergency remedial measures implemented during this event are solely intended as a treatment for the observed seepage conditions and are not intended to have a measureable impact on reducing the threat of alternate potential failure modes.

Expedited Design and Construction

The assessment, design, and construction of the March 2008 seepage remediation at Zoar Levee occurred within a very short time frame. This effort was successful in large part due to the awareness and preparedness of the Huntington District for this situation. The immediate threat was mitigated with 36 hours of the notification mobilizing the Contractor, and in total, the seepage berm consisted of approximately 37,000 tons of #8 aggregate which was placed within 150 hours. The newly constructed seepage berm was observed to perform as designed throughout the March 2008 event.

Flood Response Considerations

A major key to the successful outcome during the March 2008 flood event at Zoar Levee was readiness. Standard procedure at the Huntington District requires that Emergency Action Plans (EAP’s) are established for every project; however, due to the heightened awareness following the region’s 2005 flood, several additional measures were taken that proved beneficial during the 2008 flood fight. An emergency exercise had been conducted within the District that simulated a flood event in the Muskingum Basin. During this exercise, hydraulic models simulated pools and decisions were made regarding gate operations and maximum allowable pool levels given the simulated level of risk at each of the dependent projects. Substantial quantities of sand and sand bags had also been stockpiled at projects. As the March 2008 flood event was developing, construction IDIQ Contractors were placed on standby so that immediate response would be possible, if required. Several local quarries were also placed on an emergency contact list, and specified gradations were established to ensure that sufficient quantities materials needed for flood fighting construction efforts would be available if needed.

From the authors’ experience, some general (and perhaps intuitive, but often overlooked) recommendations are provided for consideration by those who may be involved in the
monitoring of dams and levees; benefits were realized during the Zoar Levee flood fight as a result of adherence to them. When preparing for deployment at the onset of or during an event, it is wise to envision the possibility of conditions developing to the point where decisions regarding potential immediate implementation of risk reduction measures are necessary. Weather patterns are often unstable, and the peak pool elevation which was expected at the time of deployment could be lower than the actual pool elevation which the project experiences before the end of the event. As demonstrated by Zoar Levee, a project which has performed to a certain level at a past pool elevation may perform at a lower level during a subsequent similar pool elevation due to many factors. It should not be envisioned upon deployment that since a project has seen the projected pool before and no problems occurred that problems could not develop currently. It is imperative to deploy with all available subsurface information along with relevant reports and past instrumentation data for a project in order to be able to make accurate real-time assessments of performance while in the field. Critical field analyses and risk reduction measure design could be delayed during a flood fight if simple but necessary tools such as a piezometer water level indicator, graph paper, calculator, and scale are not readily available. Having cell phone coverage in the project area, as well as emergency contact information for all relevant District personnel and elements is important. Established means for downloading and transmitting photographs back to the District office while in the field is also valuable, as this can improve the ability of office personnel to better visualize on the ground conditions, and can assist in decision making and effectively mobilizing a Contractor.

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