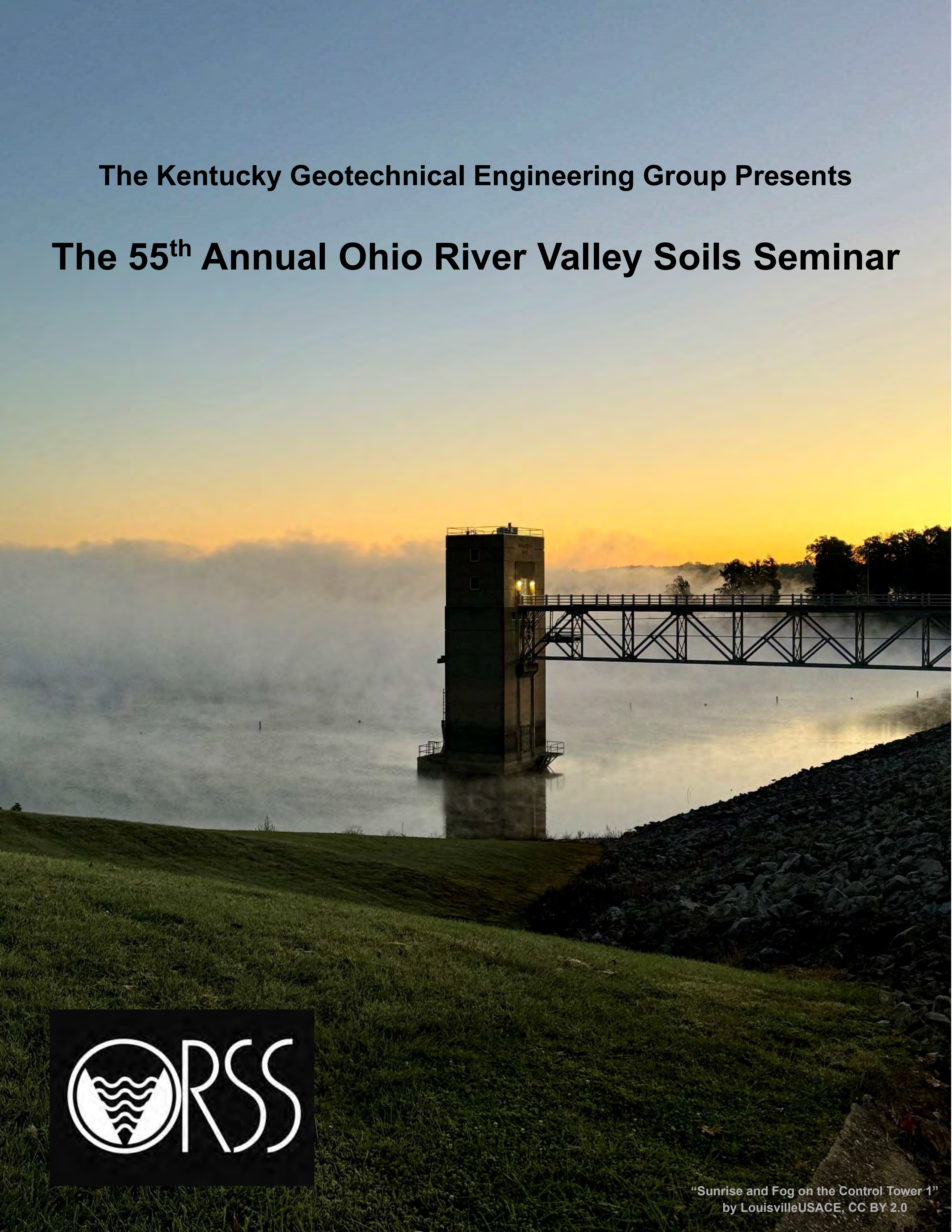
The Kentucky Geotechnical Engineering Group Presents

The 55th Annual Ohio River Valley Soils Seminar



“Sunrise and Fog on the Control Tower 1”
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55th ANNUAL OHIO RIVER VALLEY SOILS SEMINAR
NOVEMBER 12, 2025

The Campbell House | 1375 S Broadway, Lexington, KY 40504

06:00 AM - 07:00 AM: Exhibitor Move-In / Registration / Setup

07:00 AM - 07:55 AM: Registration / Breakfast

07:55 AM - 08:00 AM: Welcome Remarks

08:00 AM - 08:50 AM: Levee Case Histories: Failures and Incidents Prior to Overtopping (Matthew S. Whelan, PE / USACE Louisville District)

08:55 AM - 09:45 AM: Why Did This New Bridge Pier Move? (W. Allen Marr, PhD, PE / Geocomp and Jamal Nusairat, PhD, PE / E. L. Robinson Engineering)

09:45 AM - 10:05 AM: Break

10:05 AM – 10:55 AM: Effect of Rainfall-Induced Soil Saturation on Formation of Runoff and Slope Failure (Mir Ali Hosseini / Department of Civil and Environmental Engineering, University of Louisville)

11:00 AM – 11:50 AM: From Failure to Repair – A Success Story for the City of Marion, Kentucky (Ben Webster, PE, PMP / Schnabel Engineering)

11:50 AM - 01:00 PM: Lunch

12:45 PM - 01:35 PM: Keynote: Challenges in Addressing Karst on Civil Works (Hugo Aparacio, PE)

01:40 PM - 02:30 PM: Louisville MSD's Waterway Protection Tunnel (2017-2022): An Exploration of Louisville, KY's Subsurface (R. M. True / Black & Veatch and Jacob Mathis, PE / Louisville MSD)

02:30 PM - 02:45 PM: Afternoon Break

02:45 PM – 03:35 PM: Geotechnical Assessment of Embankment Dams Under Extreme Rainfall Events: A study of Slope Stability and Seepage Behavior (Luis Felipe Salome Simon / Department of Civil Engineering, University of Kentucky)

03:40 PM - 04:30 PM: Cellular Cofferdam Designs For Adverse Foundation Conditions (Daniel Lund, PE / USACE Louisville District)

04:30 PM: Concluding Remarks and Door Prize Drawing

ORVSS LV

Presentation 1

Levee Case Histories: Failures and Incidents Prior to Overtopping

Matthew S. Whelan, P.E.

USACE Risk Management Center

US Army Corps of Engineers®
Risk Management Center

Levee Case Histories: Failures and Incidents prior to Overtopping

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Abstract for ORVSS 2025:

Between 2008 and 2013, flood events in western Indiana and eastern Illinois caused significant distress to numerous levee systems, leading to multiple incidents that were required flood fighting as well as some embankment failures. This paper investigates these events, focusing on case histories with distress due to internal erosion. Five case histories are examined: three instances of backward erosion piping, one case of concentrated leak erosion, and one failure due to internal migration. The typical mechanisms and event trees associated with these internal erosion failure modes are reviewed, and the contributing factors leading to the incidents in each case history are discussed. The paper also provides general statistics on levee failure modes to contextualize the findings.

1 Introduction

The analysis of past incidents and failures is a critical tool in understanding the mechanisms that can lead to levee or dam failures. By comparing the conditions of past events with those observed in other projects or future designs, engineers and scientists can enhance their ability to design and assess risk more effectively. Familiarity with case histories is widely regarded as an essential aspect of improving the safety and resilience of water infrastructure. This paper examines a series of internal erosion incidents and breaches that occurred within the U.S. Army Corps of Engineers (USACE) Louisville District during flood events from 2008 to 2013, as outlined in Table 1. These incidents have not been publicly documented in previous literature, and the intent of this paper is to contribute to the growing body of case history data that informs current and future risk management practices.

Table 1. Levee Internal Erosion Incidents and Breaches

Levee System	Flood Event	Incident/Breach	Nearest USGS Gauge
McGinnis Levee	June 2008	Breach	Edwardsport, IN
Russell Allison Levee	June 2008	Breach	Vincennes, IN
Mason J Niblack Levee	June 2008	Incident	Vincennes, IN
Levee Unit 8	March 2011	Breach	Edwardsport, IN
Russell Allison Levee	April 2013	Incident	Vincennes, IN

Three of the five processes of internal erosion are likely to have led to the incidents of Table 1. These five processes, defined in *Best Practices in Dam and Levee Safety Risk Analysis* are*;*

1. Concentrated Leak Erosion
2. Backward Erosion Piping
3. Suffusion / Suffosion
4. Soil Contact Erosion
5. Internal Migration

Incidents of concentrated leak erosion, backward erosion piping, and internal migration are discussed in the case studies of this paper.

Two of the breaches and one of the incidents occurred during the June 2008 flood. The June 2008 flood in central Indiana was triggered by an intense rainfall event that overwhelmed flood risk reduction systems throughout the region. Record levels were recorded at numerous gauges, and many levees were overtopped. At least eight levee segments participating in the USACE Public Law (PL) 84-99 Rehabilitation Program failed due to overtopping.

Figure 1 illustrates the fourteen-day rainfall totals for June 2008. The counties of Vigo, Clay, Spencer, Morgan, and Johnson experienced the highest rainfall volumes, exceeding 15 inches.

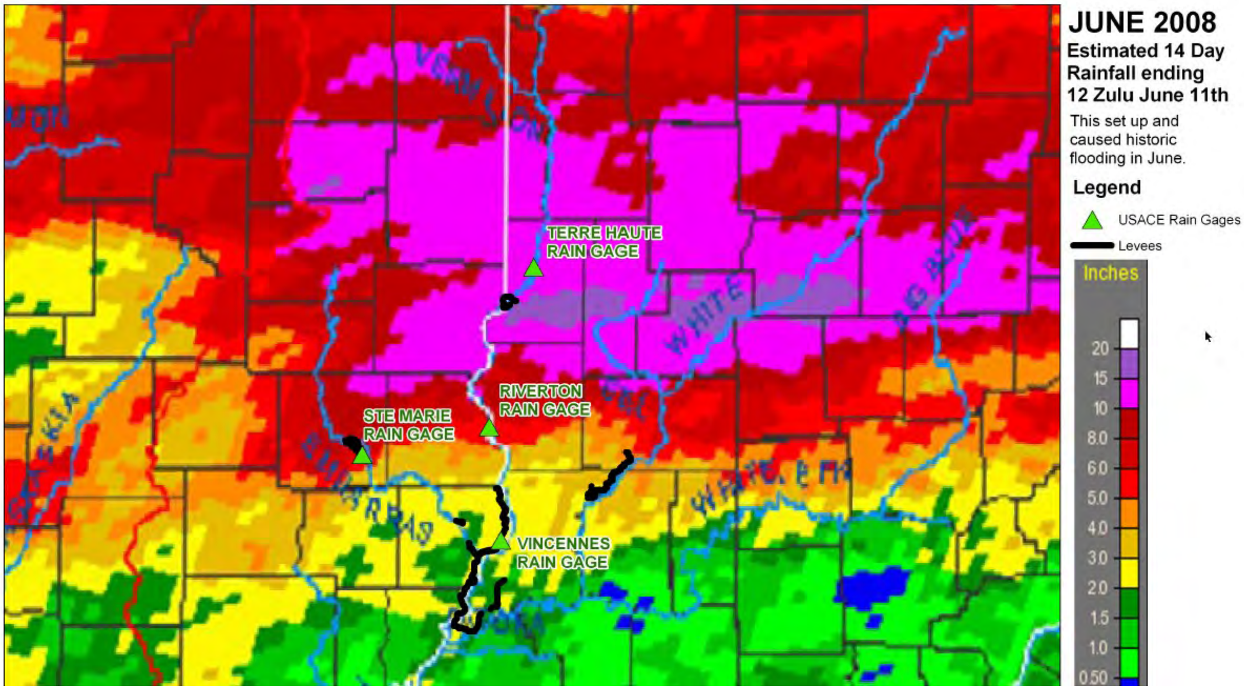

Figure 1. Fourteen-day Rainfall Totals of Central Indiana/Illinois in June 2008 (NWS).

The primary storm responsible for the severe rainfall moved from west to east across central Indiana between Saturday, June 6, and Sunday, June 7, as shown in Figure 2. Most of the rain fell between 6 PM on June 6 and noon on June 7 (USGS, 2008). While the official Indiana state record for 24-hour rainfall remains 10.5 inches, set in Princeton, IN (Gibson County) in 1905, an unofficial gauge in Edinburgh, IN (Johnson County), recorded 10.7 inches during this event.

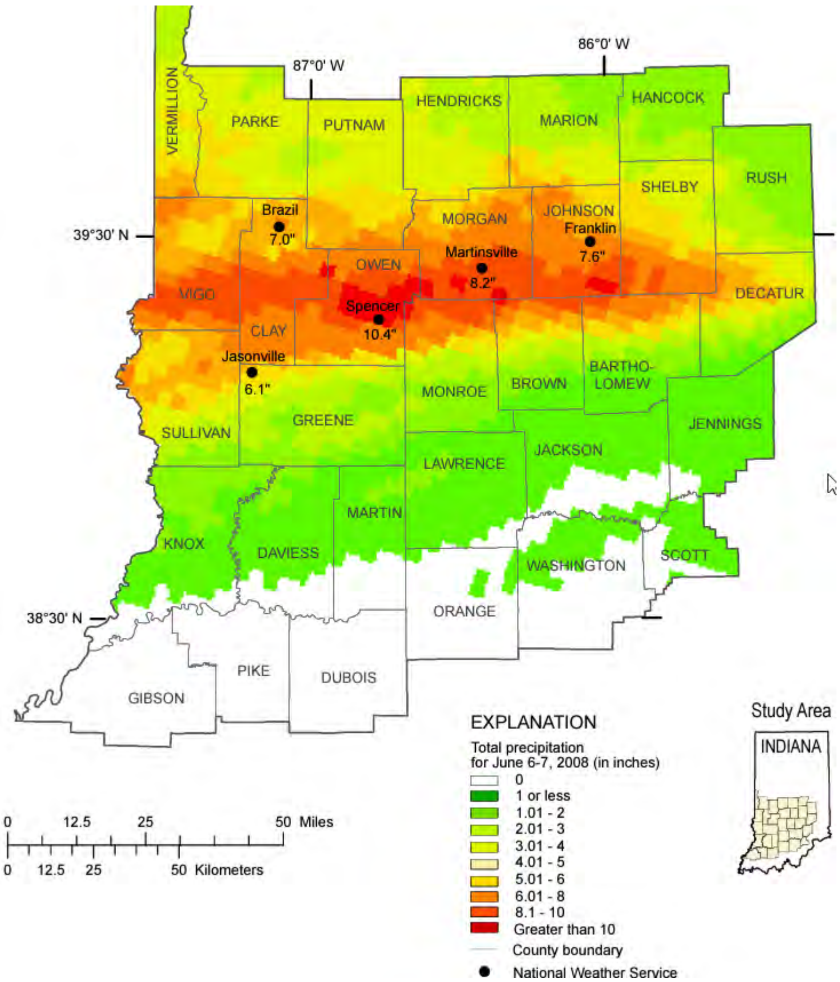

Figure 2. Total Precipitation for June 6-7 2008 (USGS 2008)

The 2008 flood resulted in record water levels at several gauges across western Indiana and eastern Illinois, including the White River at Edwardsport, IN, the East Fork White River at Columbus, IN, and the Embarras River at Lawrenceville, IL. Besides overtopping failures, some levee breaches occurred through other mechanisms, and flood-fighting efforts likely prevented even more failures. Projects damaged in the 2008 event are shown in Figure 3.

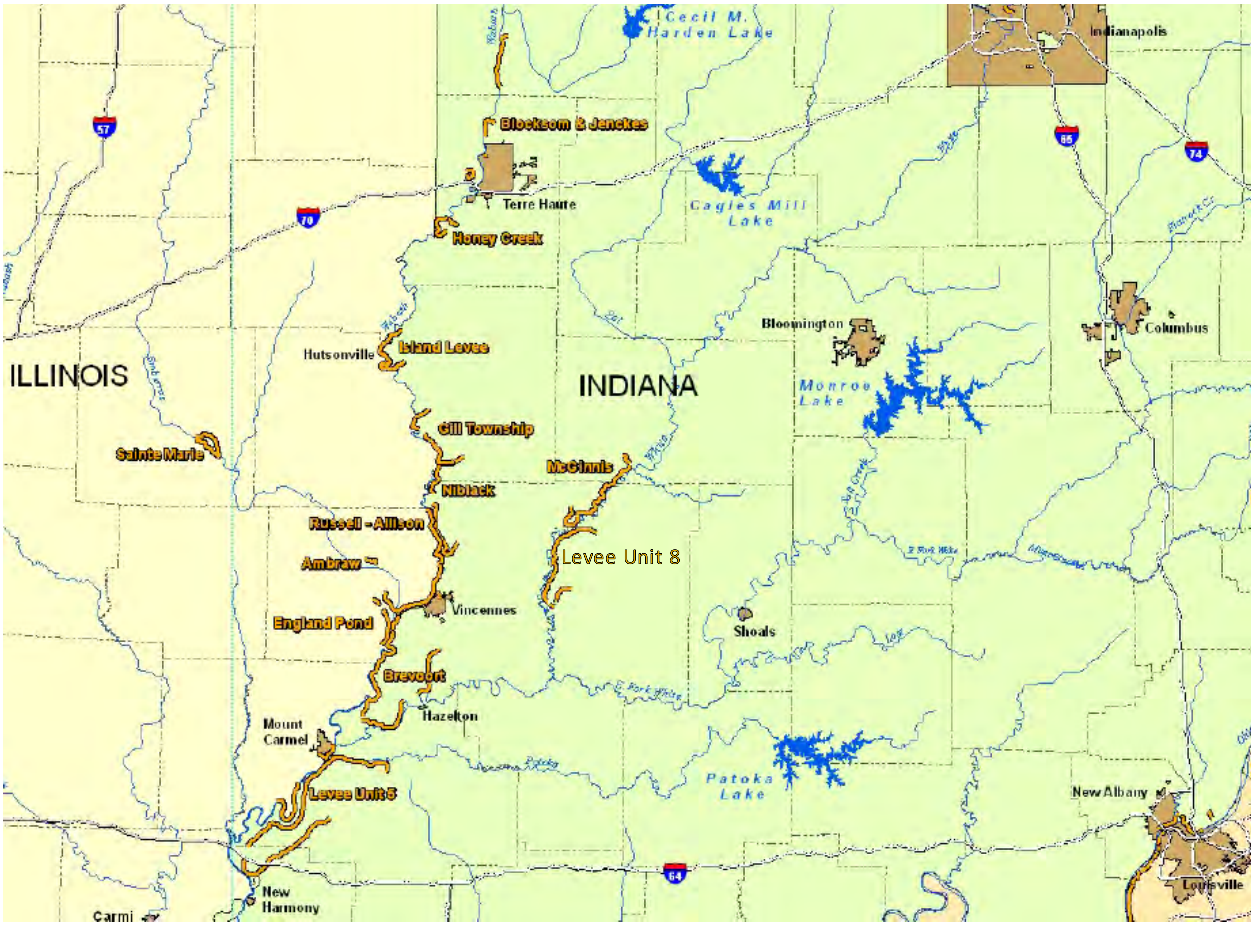

Figure 3. Projects Damaged in the June 2008 Flood

Subsequent flooding events in 2011 and 2013 further strained the region's flood defenses. The 2013 flood particularly impacted the Wabash River levee systems, often surpassing the 2008 water levels. During these later events, further levee incidents and breaches were observed.

2 Levee Incidents and Failures

2.1 McGinnis Levee 2008

The McGinnis Levee is located on the White River near Edwardsport, IN and experienced three breaches during the 2008 flood. Two overtopping breaches occurred at low spots in the embankment. A third breach toward the downstream end of the project (Figure 4) failed directly over a concrete culvert. Following the flood event, USACE representatives met with the local sponsor at the site to observe the damage. Photos of the breach are seen in Figure 5, Figure 6, Figure 7, and Figure 11.

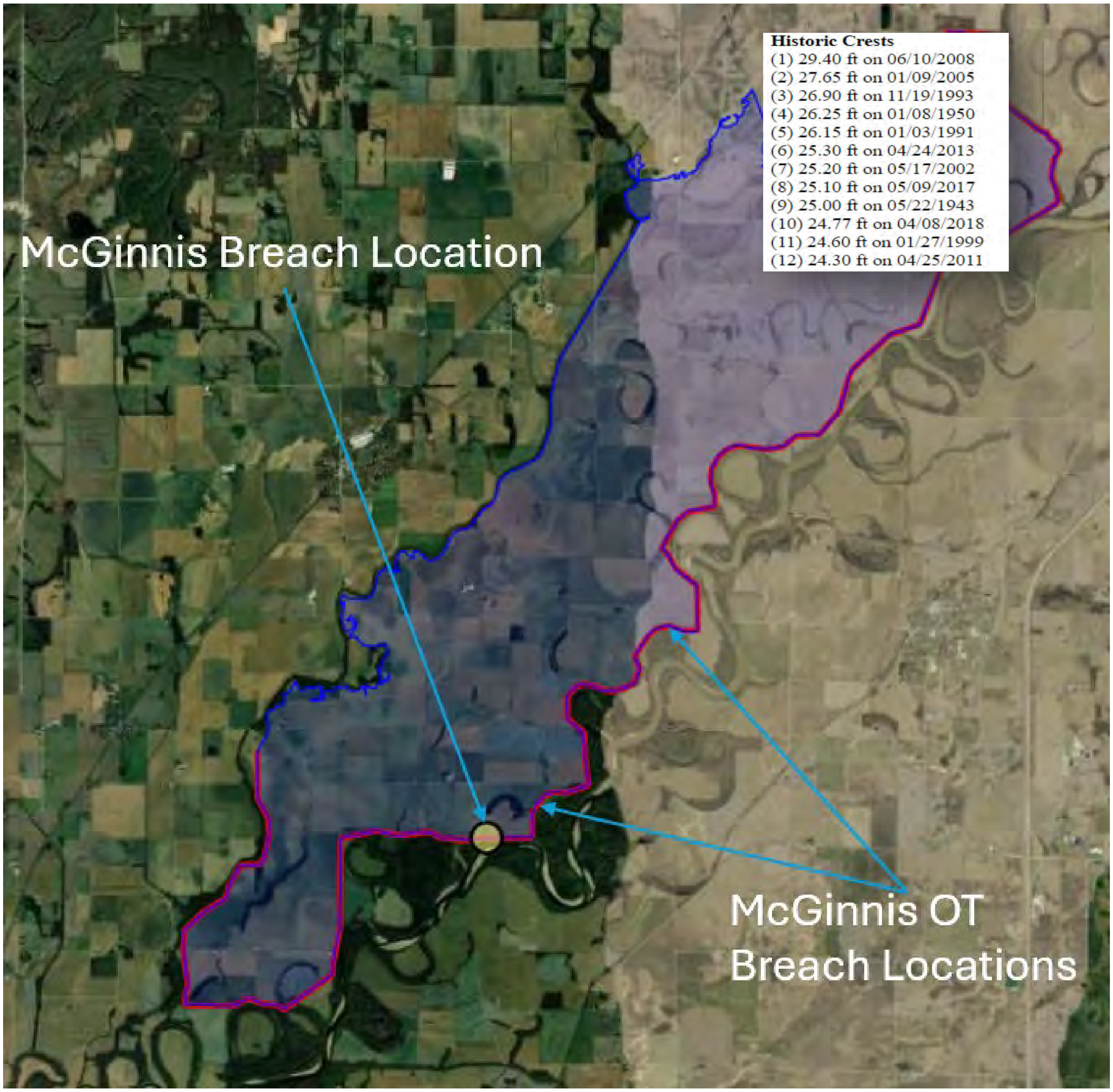

Figure 4. McGinnis Levee 2008 Breach Locations (USDA 6/2008 Aerial Imagery) and Edwardsport Gauge Historic Crests

Figure 5. McGinnis Levee Failure looking toward White River

Upon investigating the third breach, USACE representatives noted the levee to the left and right surrounding the breach did not appear to have been subjected to overtopping erosion or other distress. The culvert, exposed by the breach (see Figure 5), showed a large crack upstream of the levee centerline, visible just above the waterline. On the other side of the culvert, the team discovered a concrete placement adjacent the culvert as evidence of a prior repair.

Figure 6. McGinnis Levee Failure with Crack in Concrete Culvert

It was theorized the cracked culvert may have led to the breach due to the internal erosion mechanism of internal migration (stoping). This potential failure mode (PFM) is a result of a flaw or open defect in the culvert, the existence of a downward gradient into the void, and loss of material into the void until a collapse of the levee crest occurs manifested as a sinkhole. Depending on the sinkhole location, overtopping can occur or lead to sloughing, erosion, and other instability. A schematic representation of the hypothesized stope development above the known crack is presented in Figure 7.

Figure 7. McGinnis Levee Failure and theorized Stope over Flaw in Culvert

Further evidence of an internal migration PFM is the documented performance of a similar culvert upstream of the breach location during the 2005 flood event, as illustrated in Figure 8. While not the same structure, this culvert exhibits a similar rectangular conduit design, potentially indicating systemic vulnerabilities to cracking within this type of structure or its placement and backfilling. The construction dates of these concrete culverts are unknown.

Figure 8. McGinnis 2005 Levee Collapse at Different Culvert Location

The progression of internal migration at this location is depicted in Figure 9.

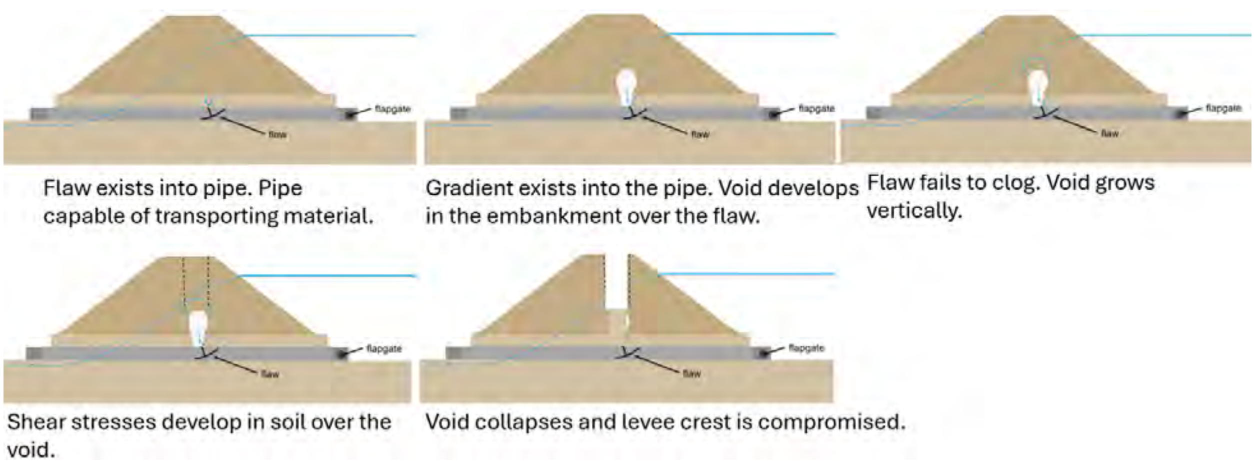

Figure 9. Internal Migration Development over a Flaw in a Conduit

USACE employs event tree analysis (ETA) to assess the probability of failure from various PFMs. An event tree is a graphical representation of the sequential events leading to failure and uncontrolled release of impounded water. Ideally, event trees are constructed sequentially from left to right, commencing with an initiating event and progressing through all necessary subsequent events in their chronological order.

At each node in the event tree, a conditional probability is assigned, representing the probability of the event occurring given the preceding event has occurred. The product of these conditional probabilities yields the overall system response probability for a specific failure path. Figure 10 is a generic event tree for the internal migration PFM. In the case of the failure observed at the McGinnis Levee, all nodes within the event tree would be assigned probability of one, indicating the occurrence of all prerequisite events leading to failure.

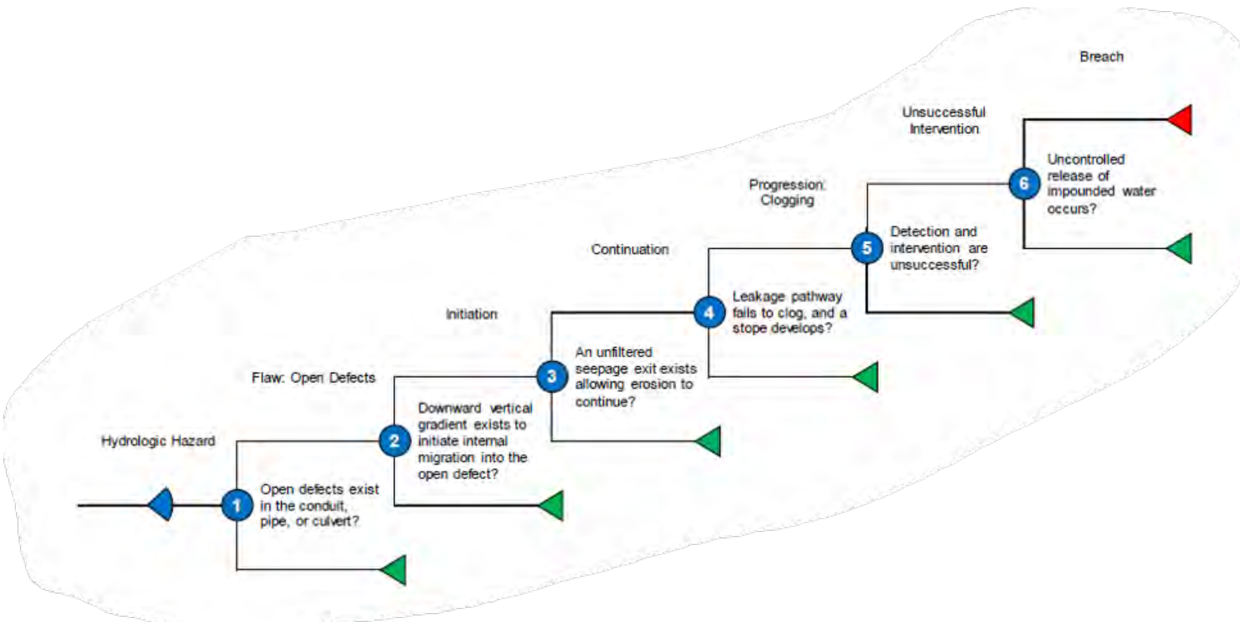

Figure 10. Generic Event Tree for Internal Migration into Conduit/Pipe

Based on the known crack location upstream of the levee centerline, the levee crest was unlikely to have been completely compromised by a developing stope; however, internal migration may have contributed to crest settlement in the area, potentially creating a localized low point in the embankment leading to subsequent overtopping. The crack in the conduit was not further investigated by USACE as, the local sponsor elected to have repairs performed independently.

While overtopping cannot be excluded as the primary breach mechanism at this location, its probability is considered comparable to that of internal migration. Prior internal migration may have contributed to crest settlement in the area, potentially creating a localized low point in the embankment for overtopping. Concentrated leak erosion along the culvert is considered less probable based on the observed integrity of the embankment surrounding the downstream headwall, as depicted in Figure 11. For concentrated leak erosion to occur, a continuous flaw or preferential path must extend from the riverside to the landside. The intact material observed on the landside adjacent to the conduit is evidence against a continuous flaw.

Figure 11. McGinnis Levee Failure Landside Headwall

2.2 Russell Allison Levee 2008

The Russell Allison Levee, located downstream from Vincennes, IN, experienced a breach on the morning of June 10th. The local sponsor reportedly conducted a levee inspection approximately 2 to 3 hours prior to the breach being detected. While overtopping was occurring at locations downstream of this site, the levee at the breach location did not overtop. Following initial detection of the breach from a distance, aerial photography captured by a local pilot (Figure 12) revealed a relatively narrow breach width, estimated to be between 20 and 30 feet. The breach subsequently expanded to an approximate width of 300 feet. Post-breach inspection revealed that the upper clay blanket foundation remained largely intact, with no significant scour hole formation (Figure 13).

Figure 12. Russell Allison Levee Breach

While the precise cause of this breach carries uncertainty, available evidence suggests that the most plausible PFM was the internal erosion mechanism of concentrated leak erosion (CLE). A notable observation supporting this hypothesis was the documented presence of animal burrows in this area during past inspections. Further evidence of animal burrowing was observed upstream of the breach location during the subsequent repair efforts. Although the sponsor levee inspection conducted earlier that morning did not reveal any indications of distress, given the time of day, there was likely low daylight during this inspection.

Figure 13. Russell Allison Breach Post-Inspection Photo

A theorized sequence that could have led to the breach by concentrated leak erosion is shown in Figure 14.

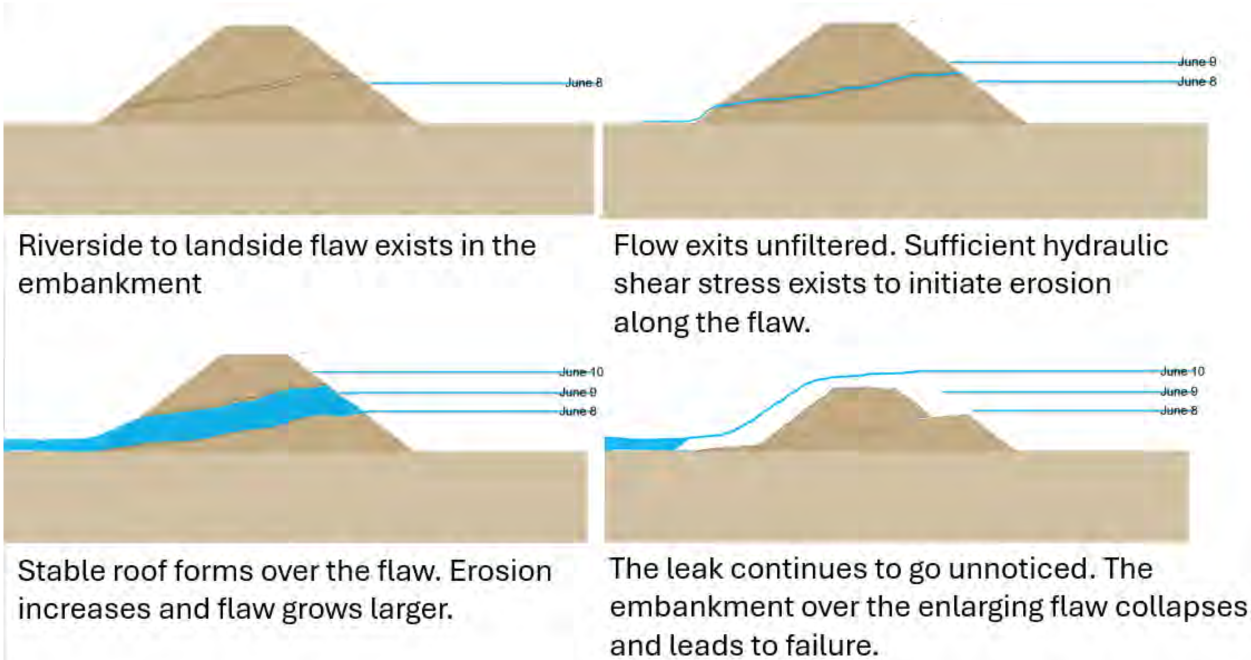

Figure 14. Concentrated Leak Erosion Sequence

A generic event tree depicting the same sequence shows the nodes for CLE to lead to failure in Figure 15.

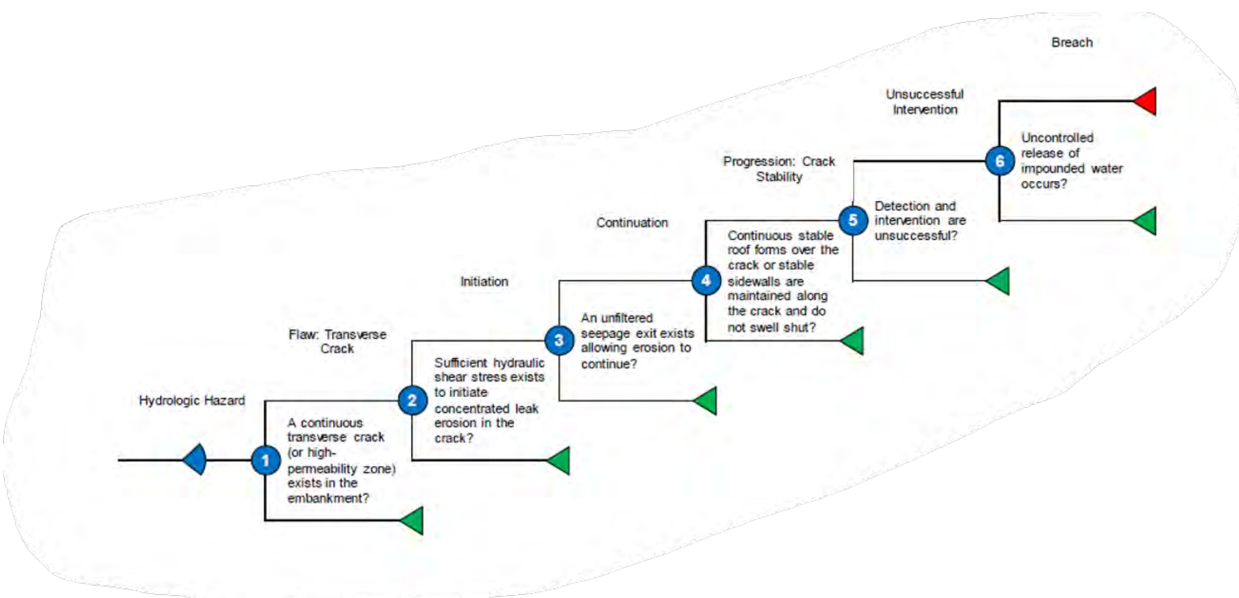

Figure 15. Concentrated Leak Erosion Event Tree (homogeneous embankment)

The breach at the Russell Allison Levee exacerbated flooding within the leveed area. Due to its upstream location relative to the overtopping breach locations, the breach resulted in both increased flood depths and a larger inundated surface area within the protected zone. Figure 16 is a photograph taken from a vantage point just over the bridge into Illinois from Vincennes, with the Russell Allison levee alignment extending to the left (downstream) and right (upstream) of the photographer's position.

Figure 16. Photo of Illinois Flooding within the Russell Allison Leveed Area

2.3 Mason J. Niblack Levee 2008

The Niblack Levee, a federally constructed levee system, is located across the Wabash River from the Russell Allison Levee. The primary area of distress on the Niblack Levee is delineated by the yellow box in Figure 17. As part of the federal project, selective raising and enlargement of the levee embankment were undertaken, including a segment situated within the aforementioned area of distress. The design details of the embankment and the associated relief drain are illustrated in Figure 18.

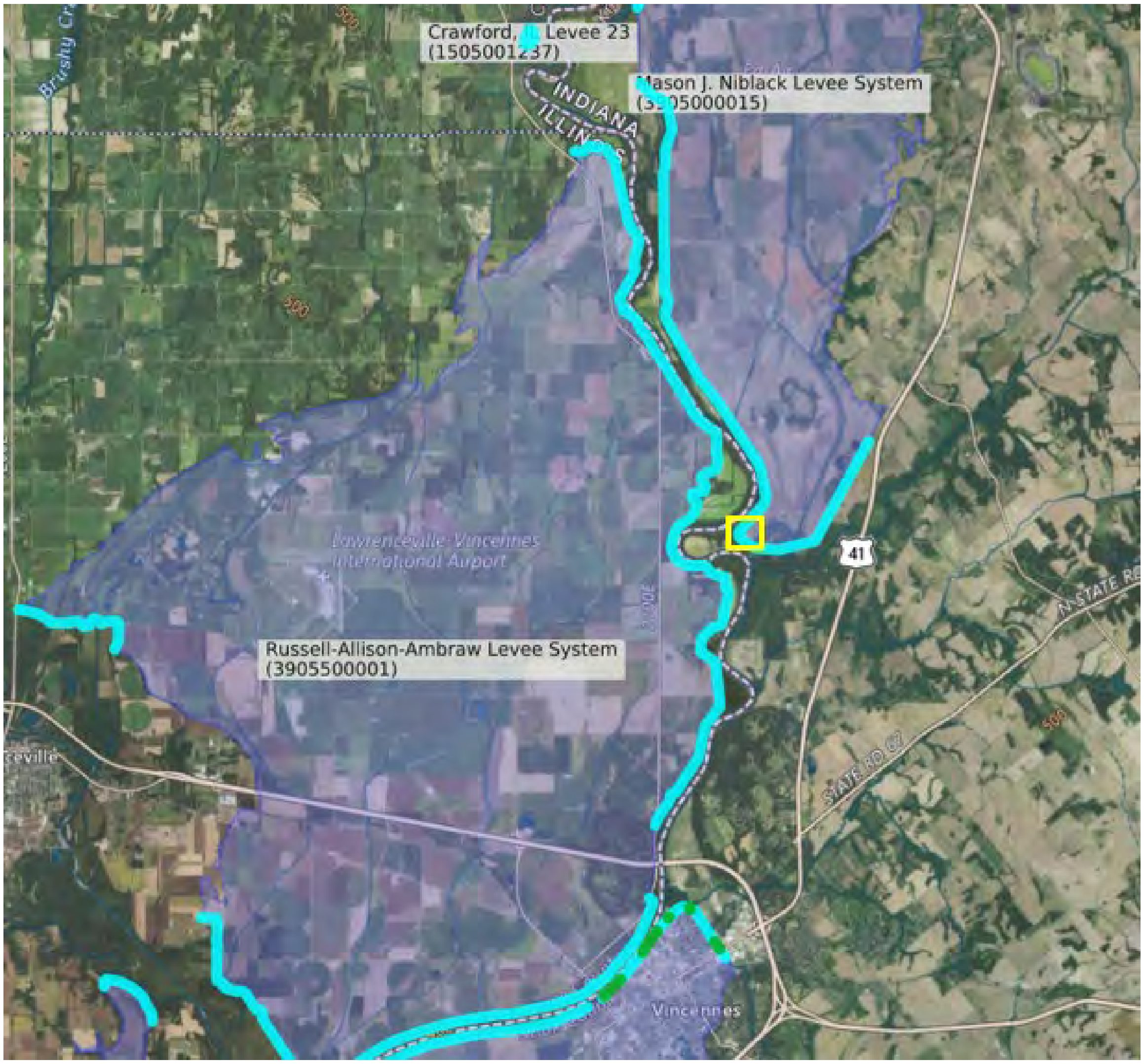

Figure 17. Niblack Levee System Distress Location

Figure 18. Niblack Aerial Image 2008 and Area with Relief Drain

Figure 18 aerial imagery shows several sand boils of notable size and discharge volume that were observed during the post-flood site visit. Contrary to initial expectations, the area incorporating the relief drain did not exhibit a demonstrably lower level of distress compared to the area on the opposite side of the bend. Representative photographs of the observed sand boils are presented in Figure 19 and Figure 20.

Figure 19. Large Boil During Flood Event at Landside Toe

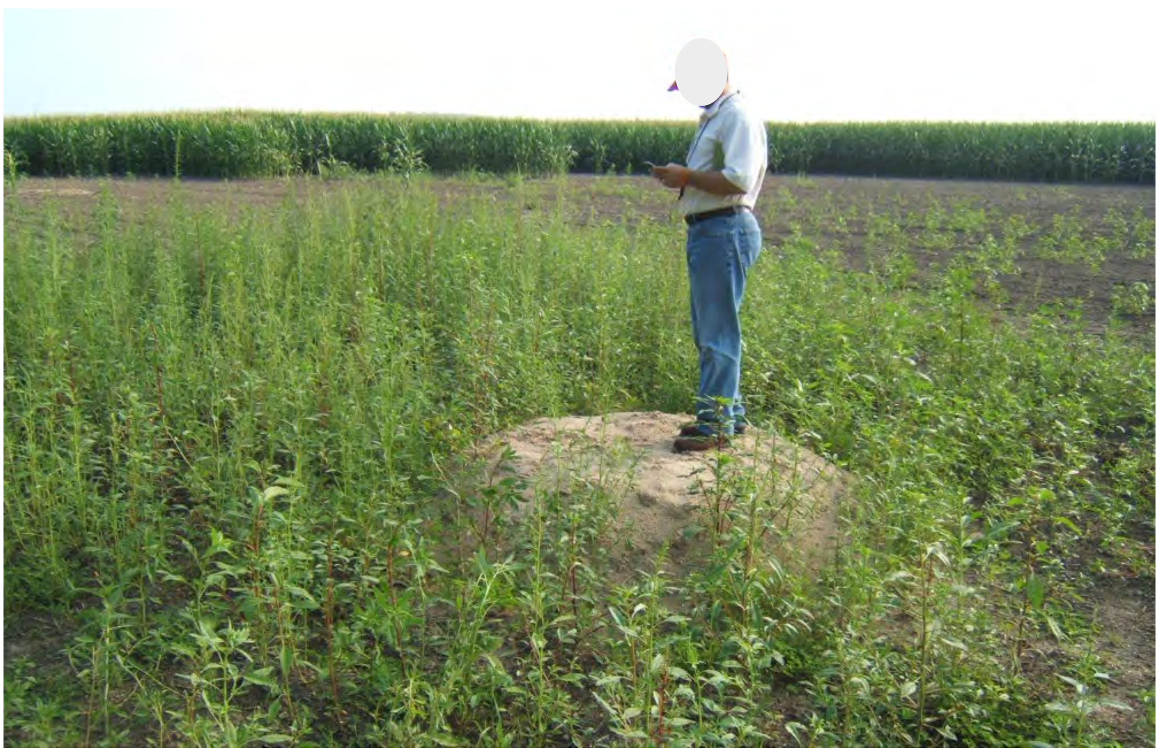

Figure 20. Typical Boil at Niblack 2008

The primary concern associated with sand boils is the potential for the internal erosion mechanism of backward erosion piping (BEP). Sand boils located directly at the levee toe, such as the one illustrated in Figure 19, are of particular concern to flood-fighting personnel because of their potentially shortened seepage path and high horizontal hydraulic gradient. With increasing horizontal hydraulic gradient, the likelihood of backward erosion piping progression, leading to a complete hydraulic connection between the boil and the river, increases. This connection can then result in rapid widening of the pipe and subsequent collapse and levee failure. A schematic representation of the BEP progression leading to failure is presented in Figure 21.

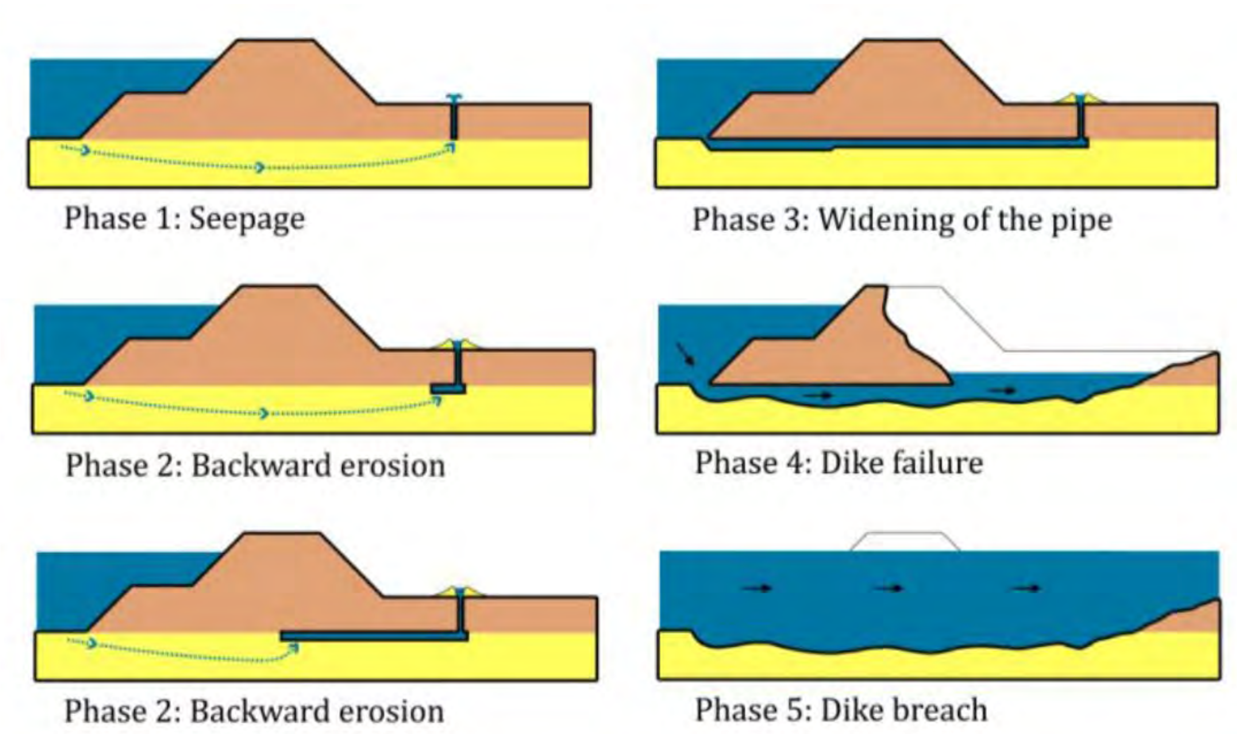

Figure 21. Process of Backward Erosion Piping (van Beek et. al 2011)

In response to the distress from the 2008 flood event, USACE constructed a sand berm on the landside toe of the levee in 2009. This berm was 150 feet wide and approximately 5 feet thick. The berm proved effective in suppressing sand boils near the levee toe during the 2011 and 2013 flood events. However, significant sand boils did develop beyond the landside toe of the berm. An example of such a boil from the 2016 flood is shown in Figure 24, and multiple large boils are visible in aerial imagery captured after the 2013 flood, as presented in Figure 22.

Figure 22. Niblack Sand Berm and Boils Beyond from the 2013 Flood.

The location of seepage within the inside bend of the levee alignment is likely related to three-dimensional flow effects. Hydrostatic loading from two directions within the bend results in increased uplift pressures and, consequently, increased flow rates from the observed sand boils. A similar failure prior to overtopping was documented in 1943 at a geometrically comparable location on the Brevoort Levee, located 17 miles downstream of the Niblack Levee.

Compounding the potential for seepage at this location, the river channel exhibits a tendency to meander and shorten its course by cutting across the bend just west of the levee. The resulting flow across the peninsula adjacent to the levee, as illustrated in Figure 23, induces erosion, potentially exposing foundation sands and creating a preferential seepage entry point near the levee toe.

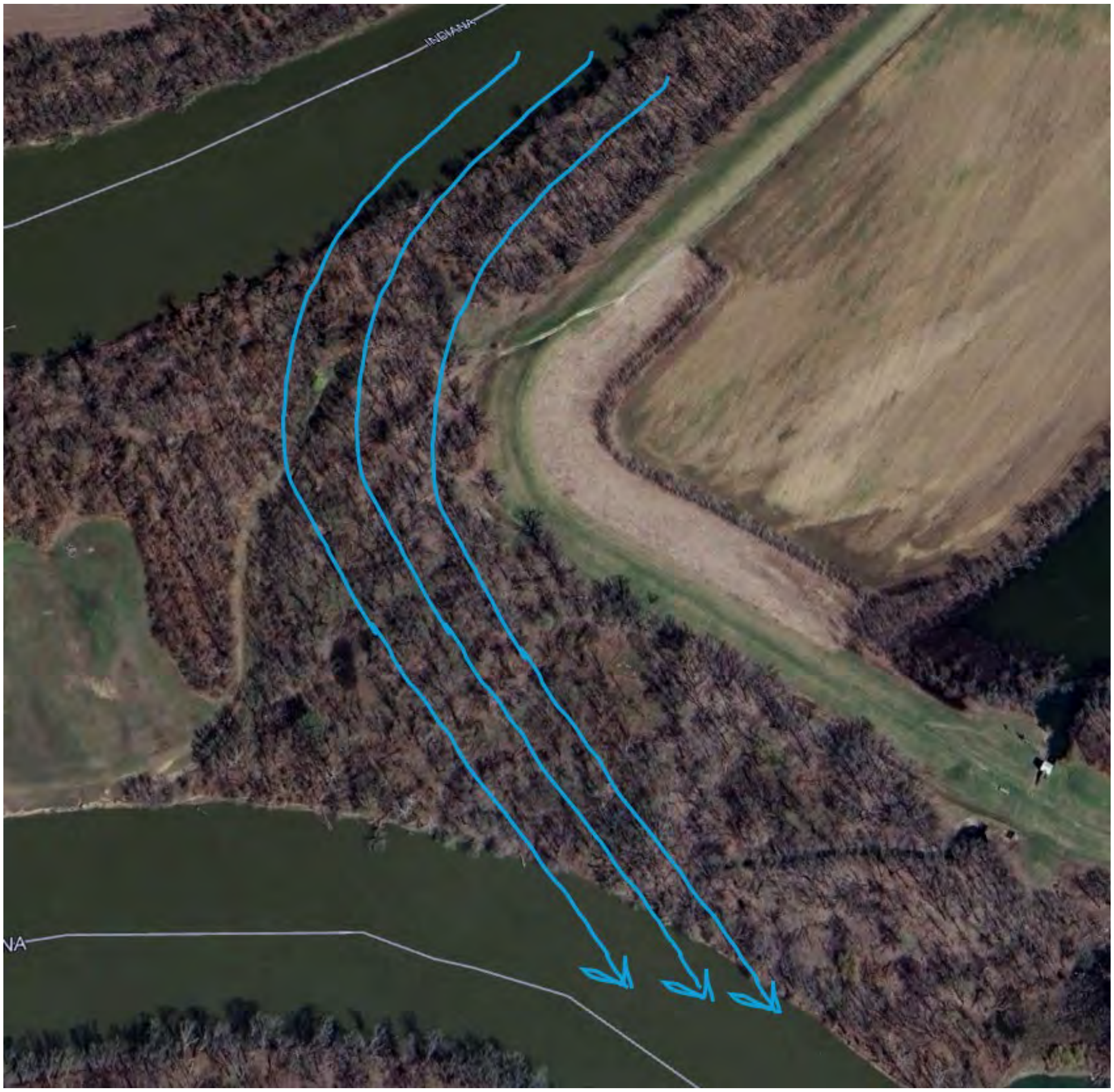

Figure 23. High Water Flow Path Adjacent Levee

The question of whether sand boils located beyond the landside berm pose a significant risk has been frequently raised. Measurements indicate that the closest boils are situated approximately 300 feet from the riverside levee toe. Given a maximum levee height estimated at 14 feet, the estimated horizontal hydraulic gradient for a top-of-levee loading condition is approximately 14 feet / 300 feet, or just under 0.05. Documented case histories of BEP progressing at such a low hydraulic gradient are considered rare. Forthcoming revisions to USACE Engineer Manual (EM) 1110-2-1913 *Design and Construction of Levees* will include berm length design criteria based on a target horizontal hydraulic gradient of 0.05. Therefore, backward erosion piping progressing to levee failure is not expected absent substantial lateral erosion of the riverside levee toe.

Figure 24. Photo of Toe of Niblack Sand Berm and Large Boil from 2016 Flood

2.4 Levee Unit 8 – 2011

On the morning of Friday, March 4, 2011, a breach occurred at Levee Unit 8, a federally constructed levee located on the White River near Edwardsport and Washington, IN (Figure 25). A photograph of the breach is presented in Figure 26. As depicted in this photograph, the levee was only moderately loaded at the time of the failure, estimated at a maximum of 25% to 50% loaded based on the landside levee toe elevation. The stage elevation during this event was several feet lower than those recorded during previous high-water events (Figure 25).

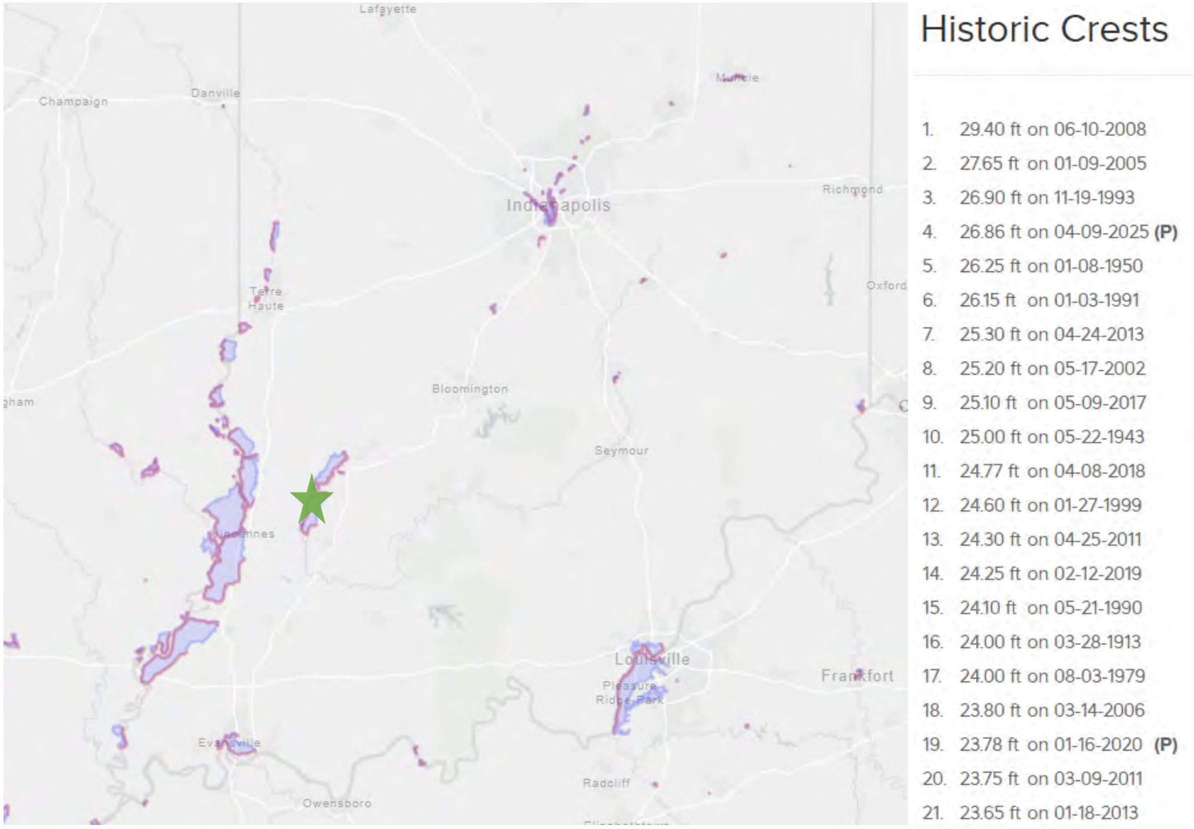

Figure 25. Levee Unit 8 Location and Edwardsport, IN Historic Crests

Figure 26. Levee Unit 8 Breach March 4th 2008 Looking Downstream

The previous day, a USACE flood-fight team was on site and traversed the eventual breach location. Observations of the area are shown in Figure 27, Figure 28, and Figure 29.

Figure 27. Levee Unit 8 with Sloughing in Landside Ditch

Figure 28. Boiling and Material Discharge within Landside Ditch

A drainage ditch situated landside of the levee embankment, likely just outside the levee easement, was exhibiting sloughing along its edges as show in Figure 27. Seepage was observed within the sidewalls of the ditch, primarily manifesting as horizontal flow along both sides of the ditch and active boiling with cloudy discharge was noted (Figure 28) within the ditch in several locations. While these observations were concerning, they were not dissimilar to conditions observed by USACE personnel during prior flood events and did not, at the time, indicate an imminent failure. Given projections of a limited further rise in river stage, a recommendation was conveyed to the sponsoring agency to attempt to increase water levels within the ditch by ceasing pump operations at the downstream discharge point of the ditch and constructing a temporary dam to impound water within the ditch. The sponsoring agency subsequently installed a berm across the ditch near the pump station, but this may not have increased the water depth upstream where the breach occurred. During the March 3rd site visit, the river stage was at the riverside levee toe, resulting in an estimated hydraulic head differential of only a few feet in relation to the ditch bottom. The river stage continued to rise and crested with approximately 5 feet of freeboard remaining. Figure 29 presents a photograph taken from the levee crest, likely in close proximity to the eventual 2011 breach location. A previous breach repair from the 2008 flood event is shown just upstream.

Figure 29. Breach location of Levee Unit 8 in Foreground and 2008 Repair in Background

In response to the failure, the local sponsor hired an engineering firm to design a repair. The firm drilled two borings, N-1 and N-2, on either side of the breach through the crest as shown in Figure 30 and Figure 31.

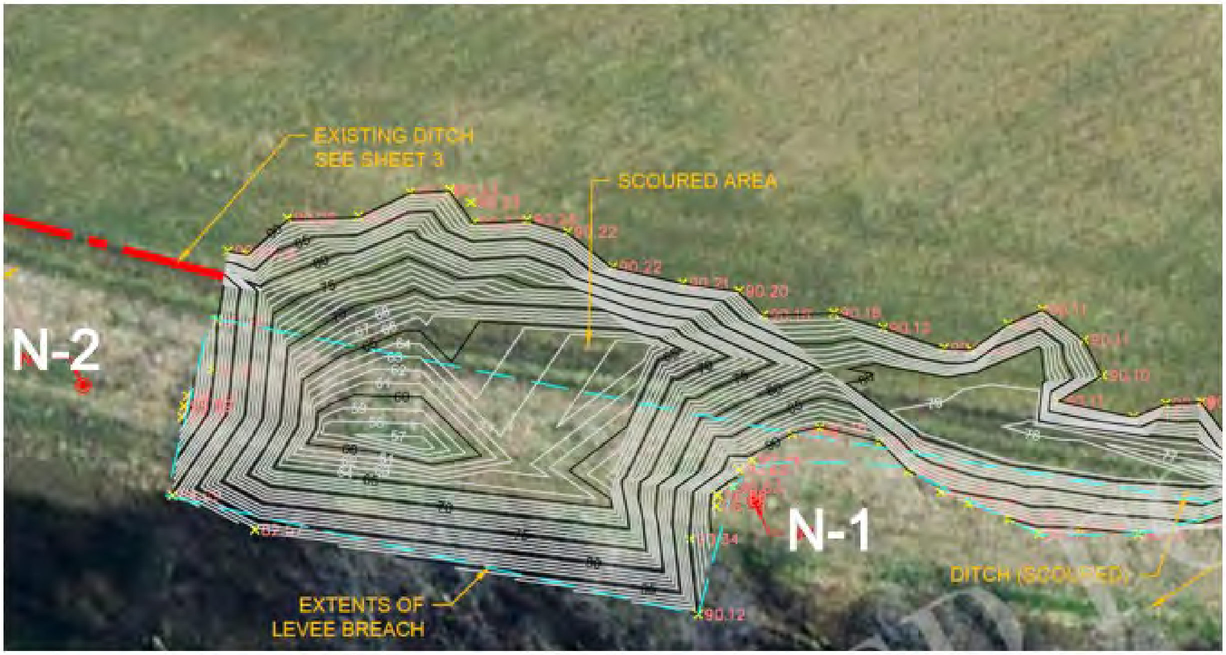

Figure 30. Levee Unit 8 Boring Location Plan

N-1

No.	Rec %	Blow Counts	Depth ft	DESCRIPTION/CLASSIFICATION and REMARKS	q_p tsf	q_u tsf	γ_d pcf	W %	LL %	PL %	PI %
SS-1	65	2-1-2-3		**TOPSOIL** **CRUSHED STONE**, (fill)	0.5			24.6	26	22	4
SS-2	55	0-0-1-2		**CL-ML**, **SILTY CLAY**, trace sand, medium to very soft, brown, with roots near 9' (fill to 11')	0.5			24.6			
SS-3	65	0-1-1-1	5		0.25	0.36	96.9	26.5 26.5			
SS-4	100	0-0-2-2			<0.25 0.5	0.25	93.1	28.6 30.2			
SS-5	90	1-2-3-2			0.25 1.0			29.0 21.4			
SS-6	100	0-0-2-2	10		<0.25 0.75	0.40	92.1	28.6 26.0			
SS-7	100	1-1-3-2			0.5 0.25	0.35 0.46	96.8 93.3	25.8 28.9			
SS-8	100	0-2-3-3	15	**CL**, **LEAN CLAY**, trace sand, very soft to soft, brown	<0.25 0.25	0.72 0.64	89.0 78.4	32.5 39.9			
SS-9	80	1-1-2-1			0.25 <0.25	0.58	90.6	32.2 30.6	32	20	12
SS-10	95	0-0-0-1		**CL-ML**, **SILTY CLAY**, trace gravel, soft to very soft, brown, with 6" medium sand seam near 23.3'	<0.25 0.25			31.7 30.1	25	21	4
SS-11	100	1-1-1-0	20		0.25			30.2			
SS-12	100	0-0-2-3			<0.25			26.2			
SS-13	80	2-3-4-5	25	**SP-SM**, **FINE SAND**, loose, wet, brown	<0.25			26.5			
SS-14	100	2-3-2-2		**SP**, **FINE TO MEDIUM SAND**, trace gravel, very loose to loose, wet, brown to gray below 26.8'							
SS-15	70	2-3-3-3	30	Continued Next Page							

N-2

No.	Rec %	Blow Counts	Depth ft	DESCRIPTION/CLASSIFICATION and REMARKS	q_p tsf	q_u tsf	γ_d pcf	W %	LL %	PL %	PI %
SS-1	50	1-3-4-3		**TOPSOIL** **CL-ML**, **SILTY CLAY**, little sand, stiff, moist, brown, (fill)	1.25			28.7			
SS-2	40	2-2-2-2		**CL**, **LEAN CLAY**, trace sand, stiff to medium, brown, (fill)	1.0			31.7			
SS-3	50	1-2-2-3	5		1.0			35.8			
SS-4	80	1-2-2-2			0.75	0.64	88.1	31.6 34.5			
SS-5	90	1-2-2-2			0.5	0.49	85.4	36.4 36.4	36	19	17
SS-6	90	0-1-2-2	10	**CL**, **LEAN CLAY**, soft to medium, moist, brown, with very moist silty clay seams near 11.8', 12.4' and 13.9' (fill)	0.25	0.39	82.2	36.1 28.5			
SS-7	100	0-1-2-2			0.25 0.5			32.2 29.9	35	19	16
SS-8	95	0-0-0-0	15	**ML**, **SILT**, little sand, very soft, brown	<0.25			32.7	19	18	1
SS-9	90	1-2-4-3		**SP-SM**, **FINE SAND**, loose, wet, brown	0.25	0.27	83.3	35.5			
SS-10	55	3-4-3-5		**SP-SM**, **FINE SAND**, loose, wet, brown							
SS-11	60	4-4-3-5	20								
SS-12	50	2-4-5-5									
SS-13	60	3-3-6-5	25								

Figure 31. Borings N-1 and N-2

Boring N-1 and N-2 both encountered approximately 11 feet of clay fill (the levee embankment), then clay overburden. Both borings were potentially drilled through the inspection trench as seen in Figure 32. N-2, closer to the 2008 breach, encountered SP-SM fine sand at 16 feet in depth (about 5 feet below the natural ground surface) where N-1 encountered SP-SM fine sand closer to 24 feet in depth. The fine sand encountered below the very loose silt in N-2 had a coefficient of uniformity of 2.2 in gradation testing with a fines content of 5.7%. A team of USACE Engineers visited the site in October 2011.

Following the breach, additional high water occurred in April and May 2011, and the breach had been further eroded from water flowing in/out of the leveed area during these events. The original as built cross section is shown in Figure 32.

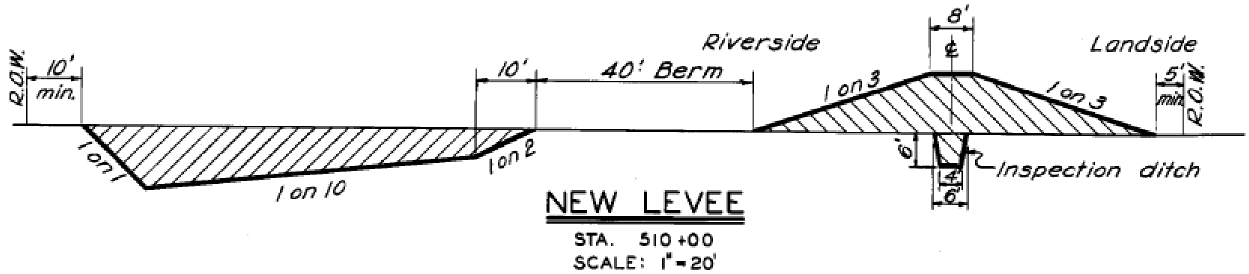

Figure 32. Typical as-built Cross Section

During the inspection, the presence of two significant depressions were documented in the riverside borrow area. Regrettably, no high-quality photographs of these features were acquired during the field visit. However, Figure 33 shows one such depression in the background, along with its approximate location in the adjacent aerial imagery. These depressions were subsequently hypothesized to represent potential seepage entry points resulting from the progression and subsequent collapse of a backward erosion pipe.

Figure 33. Location of Riverside Holes in Borrow Pit

The failure at Levee Unit 8 is hypothesized to have been caused by the internal erosion mechanism of BEP. Geotechnical investigations at boring location N-2 revealed the presence of fine sand with a low coefficient of uniformity, which would likely have been situated near the base of the ditch. Under these conditions, the development of a sand boil or concentrated seepage through the ditch bottom would have resulted in a horizontal hydraulic gradient (H/L) of approximately 0.06 to 0.08. This range is based on the estimated seepage entry point, variations in the ditch elevation, water level within the ditch, and the headwater elevation. Figure 34 presents a schematic representation of the cross-section used in the post-failure seepage modeling.

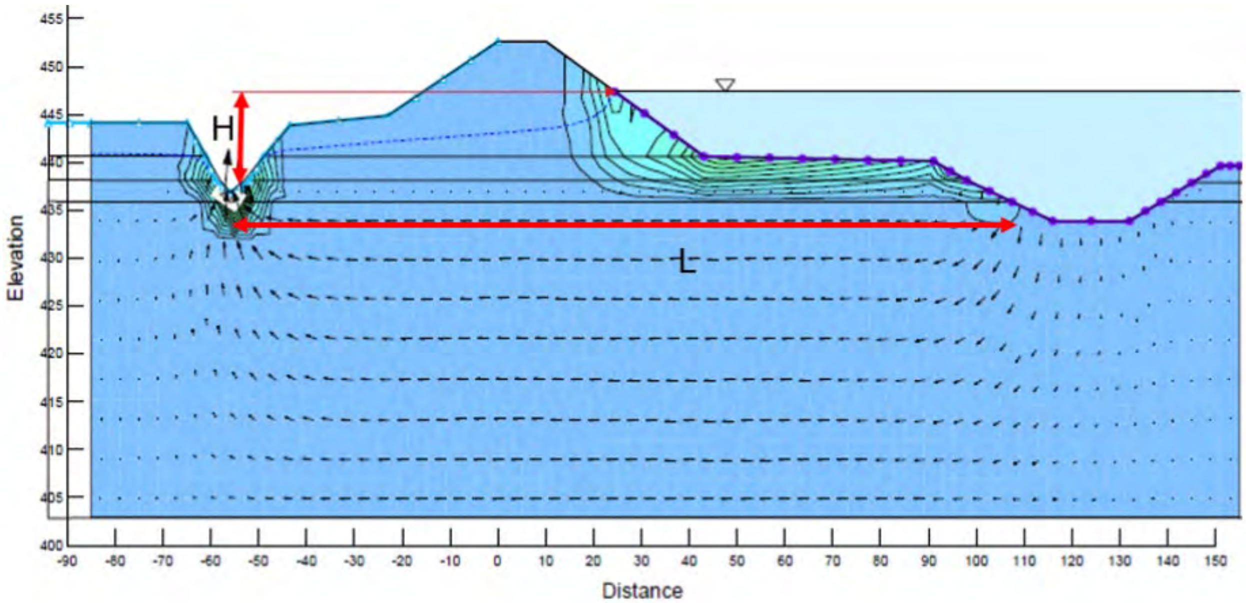

Figure 34. Seepage Model Cross Section

Although a significant sand boil was not observed in the area of the breach during the inspection conducted the day prior to the failure, the river stage was rising at the time of the inspection, and it is plausible that a sand boil initiated and rapidly developed following the inspection.

After the 2011 breach, the cause of the adjacent 2008 breach was reexamined. While the 2008 flood event was the flood of record for this levee system and photographic documentation confirms overtopping failure at two locations downstream of this location, this particular site may not have breached from overtopping. Aerial imagery from 2008 (Figure 35) reveals the presence of the ditch at this location, with subsequent reconnaissance of earlier aerial imagery suggesting the ditch was excavated in 2007. Notably, the 2005 flood event, the second highest on record after 2008, did not result in levee failure.

Given the possibility that the 2008 breach may also have been initiated by BEP, it is plausible that pre-existing vulnerabilities were present at the 2011 breach location. The 2008 event subjected the levee to a hydraulic loading approximately 5 feet higher than that experienced during the 2011 event. A remnant piping pathway may have existed beneath the embankment and reactivated erosion in 2011.

Figure 35. Levee Unit 8 2008 and 2011 Breaches

Analogous to the Niblack Levee, potential three-dimensional hydraulic loading effects may have influenced conditions at this site. The failure location is within a levee segment characterized by two inside bends, as illustrated in Figure 36. Consequently, localized increases in uplift pressures and flow may have contributed to the observed distress at these breach locations.

Figure 36. Levee Alignment at Breach Locations

Following the 2011 failure, repairs included filling the ditch along the levee and replacing the embankment. Following the backfilling of the landside ditch, there have been no reported incidents during subsequent flood events, including those exceeding the March 2011 water levels. These observations suggest that the presence of the ditch was a significant contributing factor to the 2011 failure and may have played a role in the 2008 failure.

2.5 Russell Allison 2013

The Russell Allison levee again experienced severe distress in 2013 at a new location shown in Figure 37. The levee alignment at this distress location exhibits a complex geometry characterized by the presence of two setback levees. There are no records that indicate incidents at this location during the 1943 or 1950 flood events; however, the presence of two large scour features on the riverside suggests a pattern of levee breaches followed by the construction of the setback levees on the landside of the resulting scour. The 2013 distress was located on the upstream setback embankment.

The 2013 Wabash River flood event was the third highest on record at this location, with stage elevations marginally exceeding those observed during the 2008 flood (approximately 0.2 feet) when sand boils were also documented at this location.

Figure 37. Russell Allision 2013 Distress and Historic Crests

In 2013, the local sponsor discovered a large sand boil had developed and was discharging a high volume of sand. Immediate flood-fighting measures were initiated, involving the construction of a sandbag ring around the boil to increase the landside hydraulic head and thereby reduce the overall hydraulic gradient. This intervention resulted in a reduction in the boil's discharge rate, with the sandbag ring accumulating transported sediment. Shortly thereafter, a second sand boil exhibiting similar characteristics emerged. The local sponsor implemented the same sandbag ring construction technique, again achieving a reduction in the discharge rate. Subsequently, a third sand boil, located further from the levee, began discharging sediment at a high rate, transporting a considerable volume of material. The first and second boils, located near the landside levee toe, are shown in Figure 38.

Figure 38. First and Second Boil During Flood

The third sand boil was also encircled with sandbags, and it also continued to discharge sediment. Rather abruptly, the discharge from the third boil stopped. The local sponsor characterized this cessation to the author as an abrupt and unexpected loss of flow. The local sponsor anticipated the formation of additional boils, but no further activity was observed. A post-flood photograph illustrating the three boils is shown in Figure 39. As can be seen in the background, several smaller sand boils were also present in the area.

Following the flood event, a collapse was identified in the riverside levee embankment slope, aligning directly with the location of the large boils observed during the flood. A photograph of the collapse, taken from the riverside levee toe, is shown in Figure 40.

Figure 39. Three Large Sand Boils at Russell Allison 2013

Figure 40. Riverside Slope Subsidence

The observed subsidence in the riverside slope and the abrupt cessation of flow from the third sand boil during the peak of the flood event presented unexpected phenomena. The characteristics of the boils, including their size and discharge rates, were consistent with those observed in other case histories where BEP was confirmed and, in some instances, led to levee breaches. In this particular scenario, the sudden stoppage or reduction in flow may be attributable to a roof collapse over the developing pipe beneath the levee embankment. A schematic representation of this potential mechanism is shown in Figure 41.

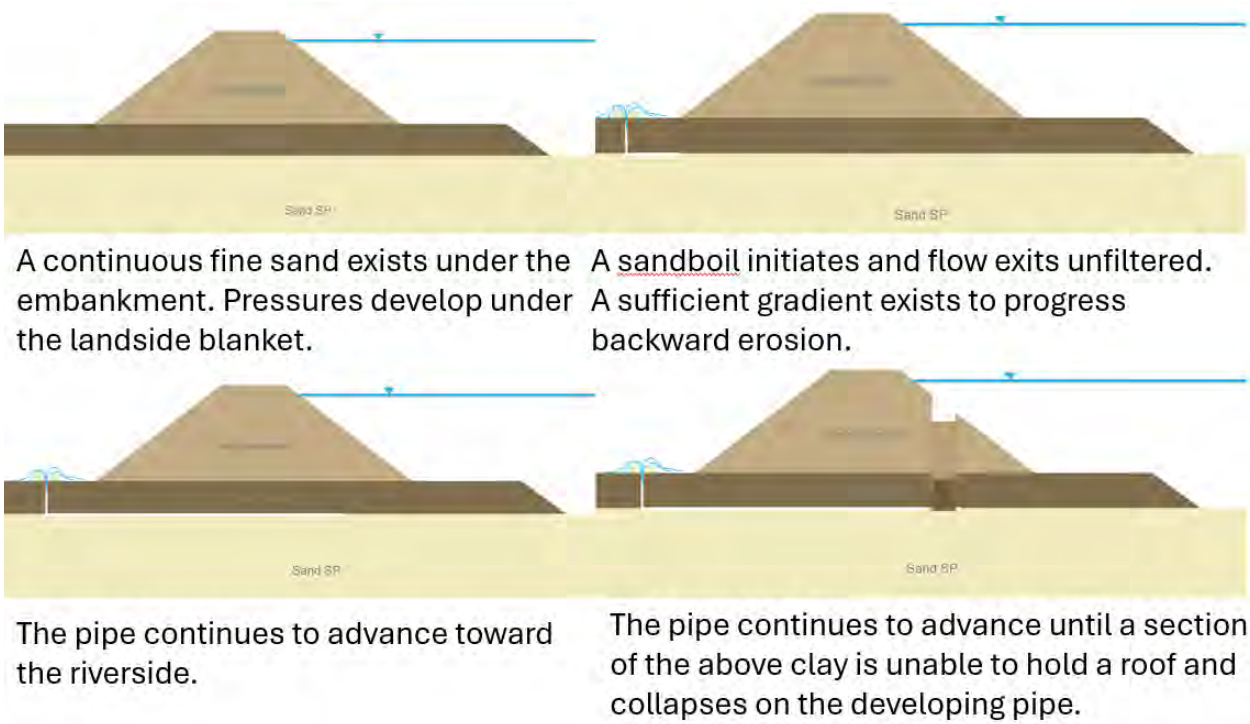

Figure 41. Sequence of Potential Roof Collapse

During the summer of 2013, the local sponsor initiated repairs to the area affected by the sand boils. The repair procedure involved excavating the boils down to the underlying sand layer and compacting the existing clay blanket back into the excavation. A photograph of one of the boils during excavation is shown in Figure 42, which the sponsor reported was representative of the general condition of the other boils. The excavated boils exhibited vertical throats of similar dimensions, approximately 6 inches in diameter, and all were found to be filled with sand. Notably, no voids or other anomalies were encountered beneath the boils during the excavation process. The riverside slope subsidence was also excavated; however, the sponsor reported that this excavation likewise revealed no apparent voids or other anomalous features. Representatives from USACE were not present during the riverside excavation.

Figure 42. Boil Excavation Following Flood

A generic event tree for BEP includes a node for continuous stable roof for BEP to progress. Empirical evidence indicates that roof collapse can, in certain circumstances, arrest BEP progression, thus preventing levee failure. This phenomenon is explicitly represented in Node 4 of the generic event tree shown in Figure 43, where roof collapse is a pathway to non-failure.

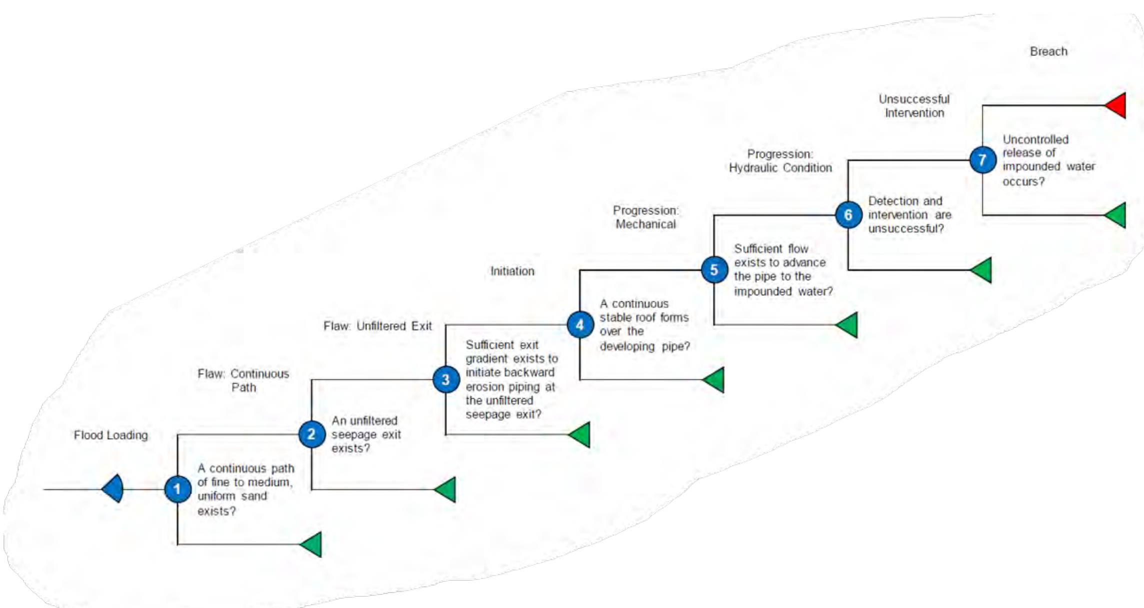

Figure 43. Generic Event Tree for Backward Erosion Piping

A perplexing aspect of this case is the observation of a likely roof collapse despite the presence of a clay blanket, which is typically considered conducive to roof stability over a developing pipe. Figure 44 presents a summary of various material roof types and their associated probabilities of roof formation from *Best Practices in Dam and Levee Safety Risk Analysis*. For a moderate-plasticity clay, similar to that observed in Figure 42, the estimated probability of holding a roof would typically exceed 90 percent.

Table D-6-3.—Probability of Holding a Roof (adapted from Fell et al. 2008)

USCS Soil Classification	Fines Content, FC (percent)	Plasticity of Fines	Moisture Condition	Probability of Holding a Roof (P_{PR})
Clays, sandy clays (CL, CH, CL-CH)	FC ≥ 50	Plastic	Moist or Saturated	0.9+
Silts (ML, MH)	FC ≥ 50	Plastic or Non-Plastic	Moist or Saturated	0.9+
Clayey sands, gravelly clays (SC, GC)	15 ≤ FC < 50	Plastic	Moist or Saturated	0.9+
Silty sands, silty gravels, silty sandy gravel (SM, GM)	15 ≤ FC < 50	Non-Plastic	Moist Saturated	0.7 to 0.9+ 0.5 to 0.9+
Granular soils with some cohesive fines (SP-SC, SW-SC, GP-GC, GW-GC)	5 ≤ FC < 15	Plastic	Moist Saturated	0.5 to 0.9+ 0.2 to 0.5
Granular soils with some non-plastic fines (SP-SM, SW-SM, GP-GM, GW-GM)	5 ≤ FC < 15	Non-Plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils (SP, SW, GP, GW)	FC < 5	Plastic	Moist or Saturated	0.001 to 0.01
		Non-Plastic	Moist or Saturated	0.0001

Figure 44. USACE Best Practices Probability of Holding a Roof

Following the flood, the local sponsor asked for guidance on the repair and USACE suggested a sand berm similar to that installed at Niblack. A berm was installed, but not to the suggested width. Figure 45 illustrates the geographic proximity of the BEP concerns at the Russell Allison Levee and the Niblack Levee, which are located approximately opposite one another across the river channel. The Russell Allison 2013 area of distress warrants continued monitoring during future high-water events to facilitate timely flood-fighting response and potential intervention.

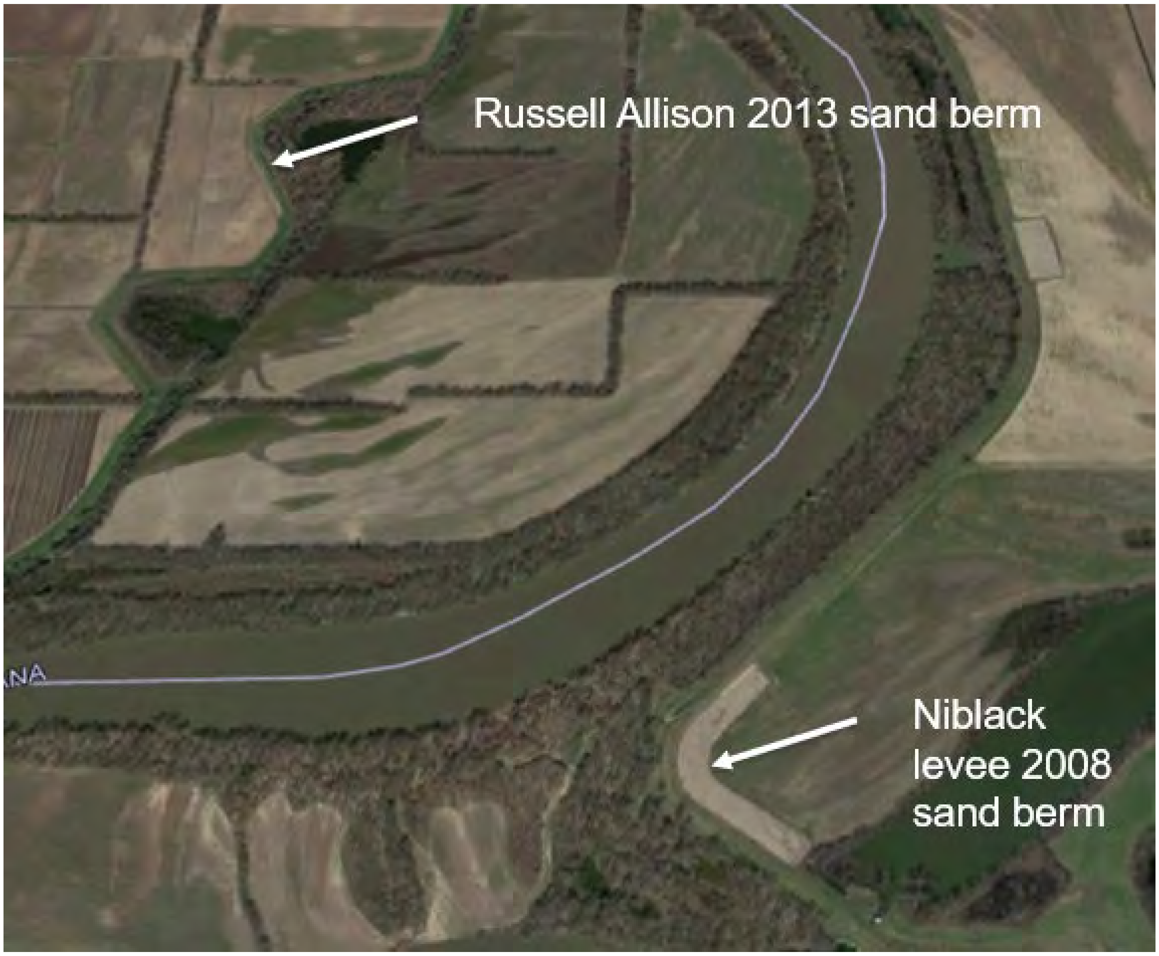

Figure 45. Russell Allison and Niblack Proximity

3 Statistics and Conclusion

3.1 Statistics

The USACE Risk Management Center is gathering statistics on levee incidents and failures across the nation. As of September 2025, data has been gathered on approximately 800 levee segments, and there have been a total of 218 individual segment breaches prior to overtopping recorded as follows;

- Embankment (riverine) erosion – 74
- Embankment through seepage IE – 46
- Embankment foundation seepage IE – 43
- Closure structure – 27
- Undetermined – 10
- Large culvert gate opening – 9
- Embankment instability – 4
- FW instability – 3
- FW underseepage – 2

Embankment foundation seepage is primarily associated with backward erosion piping, and embankment through seepage is primarily associated with concentrated leak erosion. It is noteworthy that a considerable number of levee failures have been documented, given the relatively limited review to date encompassing approximately 800 levee segments. This data includes multiple breaches from the same levee segment and both federally and non-federally constructed levee systems. The rate of failure is higher for non-federally constructed levee systems such as Russell Allison and McGinnis.

As of April 2025, over 5,700 incidents were documented for the 720 levee segments reviewed at that time. Approximately 2,200 of these incidents were classified as significant, indicating the occurrence of a breach, the necessity of an intensive flood-fighting effort to avert a breach, major interior flooding, or repair costs exceeding $1 million.

3.2 Conclusion

This review of levee failure case histories reveals some insights into the vulnerabilities of flood risk reduction systems. While diverse in their specific circumstances, these cases underscore the recurring significance of a limited set of factors. The disproportionate number of significant BEP incidents occurring in locations with three-dimensional hydraulic loading effects suggests a need for revised design methodologies and targeted monitoring strategies in these geometrically complex areas. Given the findings presented herein, three-dimensional hydraulic loading effects may have also contributed to other notable levee failures involving BEP, such as the L-575 Iowa breach in 2008 and the Bois Brule Levee breach in Missouri in 1993. The observed tendency for incidents and failures to reoccur at the same geographic locations further highlights the importance of thoroughly investigating and remediating sites with a history of levee distress. To enhance the resilience of levee systems and minimize future flood risks, further research is needed to quantify the relationship between geometric factors, hydraulic loading, and BEP initiation and progression. Ultimately, a holistic approach that integrates improved engineering practices, effective monitoring, and proactive risk management is paramount to ensuring the long-term integrity and reliability of our nation's levee infrastructure.

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Notes

ORVSS LV

Presentation 2

Why Did This New Bridge Pier Move?

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Geocomp

Jamal Nusairat, Ph.D., P.E.

E.L. Robinson Engineering

Ohio River Valley Soils Seminar 55

Kentucky Geotechnical Group
Lexington, Kentucky

Unexpected movement of new bridge pier

November 12, 2025

Why did the Bridge Pier Move?

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Contents

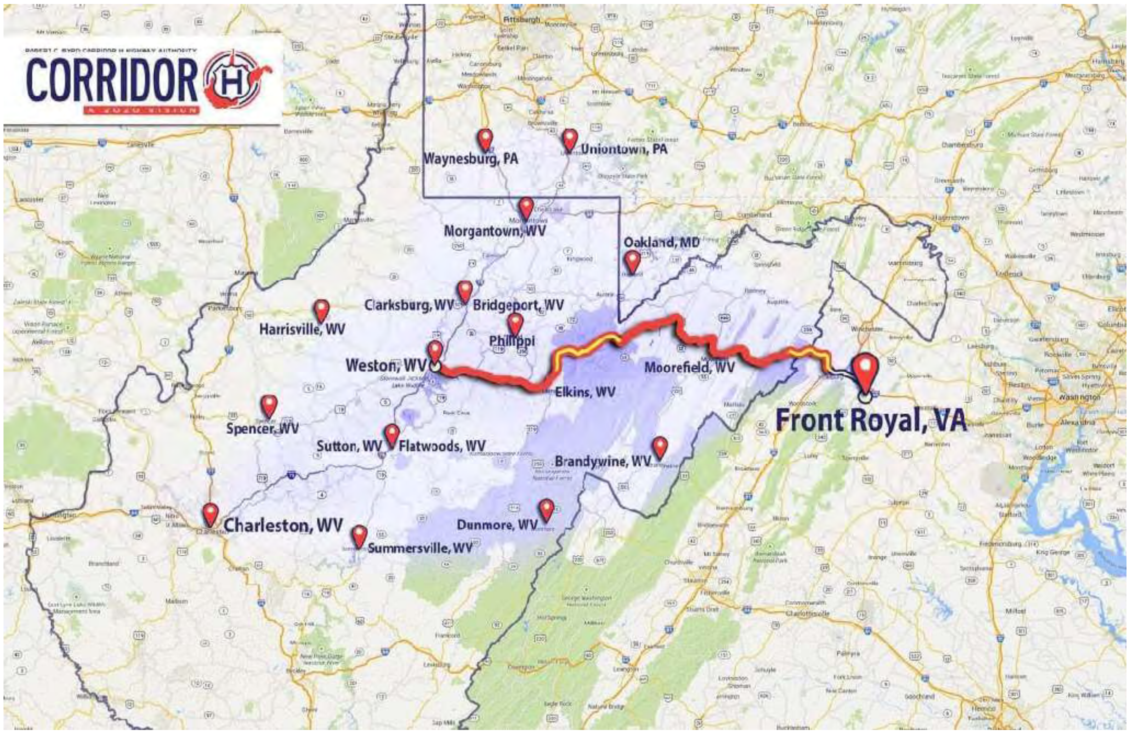
Overview

This presentation focuses on the unexpected displacement of a mountain side during the construction of a main bridge for a new four-lane section of Corridor H: Kerens to US 219 Connector in Randolph and Tucker Counties, West Virginia.

From surveying, the position of the Pier 1 appeared to be shifted from its designed position by up to 12 inches in the downhill direction and to possibly continuing to move.

This presentation covers how the cause of the problem was determined and what remedial scheme was chosen to stabilize the mountain side. Results from the remediation are described. Recommendations are provided to help avoid similar occurrences in future construction in mountainous areas.

Corridor H Project



- ✓ Corridor H is 130-mile, four-lane highway in West Virginia, part of the Appalachian Development Highway System, designed to connect I-79 at Weston with I-81 near Front Royal, Virginia.
- ✓ Bridge 5 is located near Parsons, WV in Monongahela National Forest

Project Team

Owner

State of West Virginia, Department of Highways

Design-Build Team

Contractor: Kokosing Construction Company

Lead Designer: E. L. Robinson Engineering Co.

Lead Geotech Engineer: Jamal Nusairat.

Lead Bridge Engineer: Faheem Ahmad

Geotechnical Consultants: W. Allen Marr, Jerry DiMaggio

The Contract

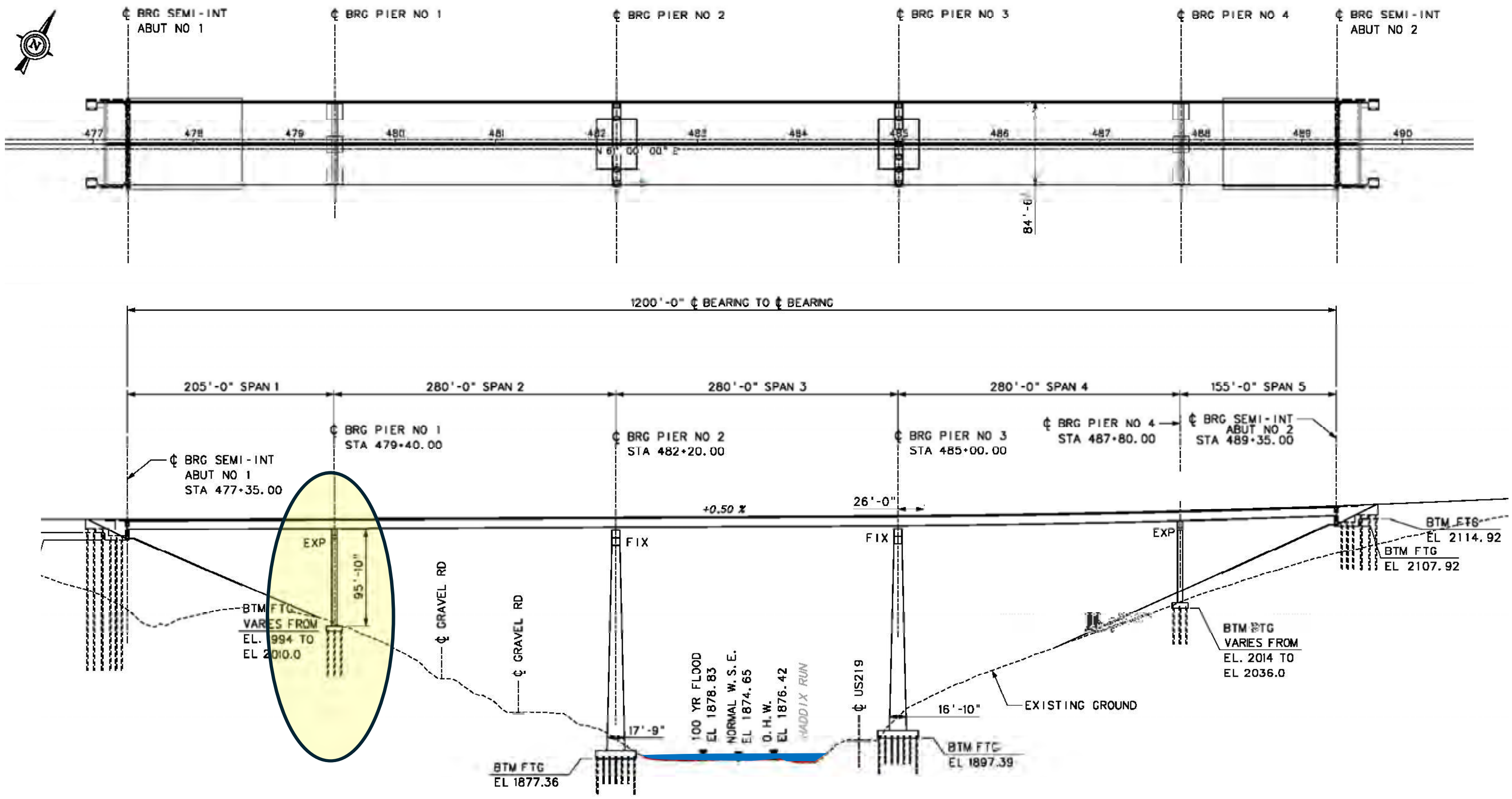
- DOH awarded the Project to the Design-Build Team of Kokosing and ELR at a contract price of $209,700,000, as reflected in the contract signed by Kokosing and DOH on December 21, 2015. Anticipated completion date of October 1, 2019.

- Currently revised to $250,200,000 and the final completion date is December 31, 2025.

Bridge 5 – Photo taken in August 2019

Bridge 5 over US 219 & Haddix Run

General Plan & Elevation

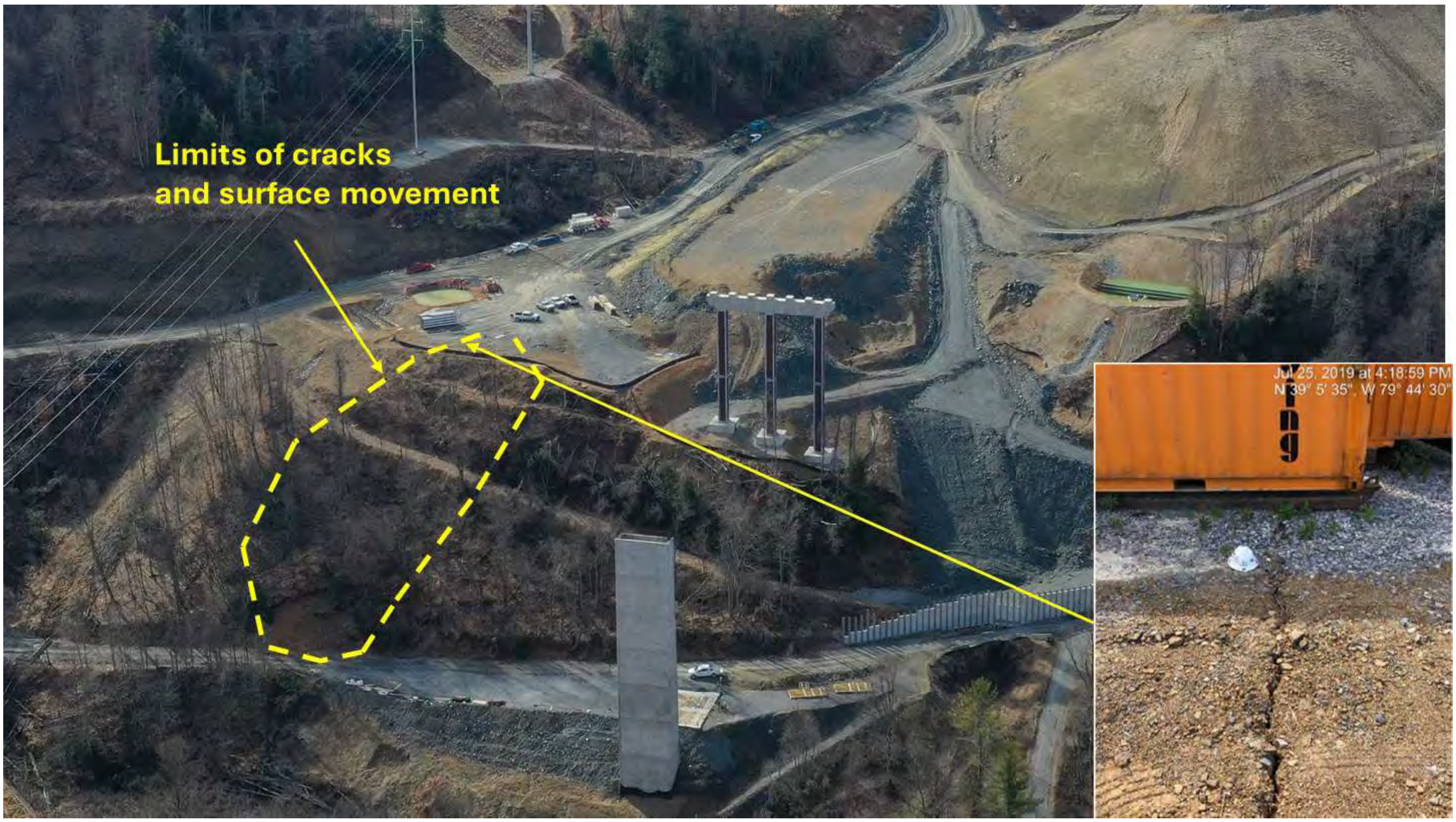


The Event

- On or about July 22, 2019, the KCC survey team found inconsistencies in the survey data for the position of Bridge 5 Pier #1. The data showed the pier might be out of alignment.
- A crack appeared on the ground surface uphill of the pier.
- Further surveying verified the foundation piles about 1 foot out of horizontal position.
- KCC's lead surveyor was returning from vacation at the time so there was uncertainty about the accuracy of this finding. KCC proceeded to check its results on July 24 and 25. On July 25, 2019, the KCC Construction Project Manager observed a xx ft long crack in the ground surface near the mechanic's shed.
    - Was it a construction error?
    - Was it a surveying error?
    - Had the slope moved?

Surface Cracks Near Mechanic's Yard

Indicating shallow slide in the overburden soils or something else?



July 22, 2019 KCC found inconsistencies in position of Pier #1 of 12 inches
~2 ½ inches displacement to NE between Aug 16, 2019 and Jan 15, 2020

Inclinometer Installation

Initially – installed 5 inclinometers in borings to determine where ground movement was occurring at what depth

Measured Horizontal movement - Inclinometer B-1

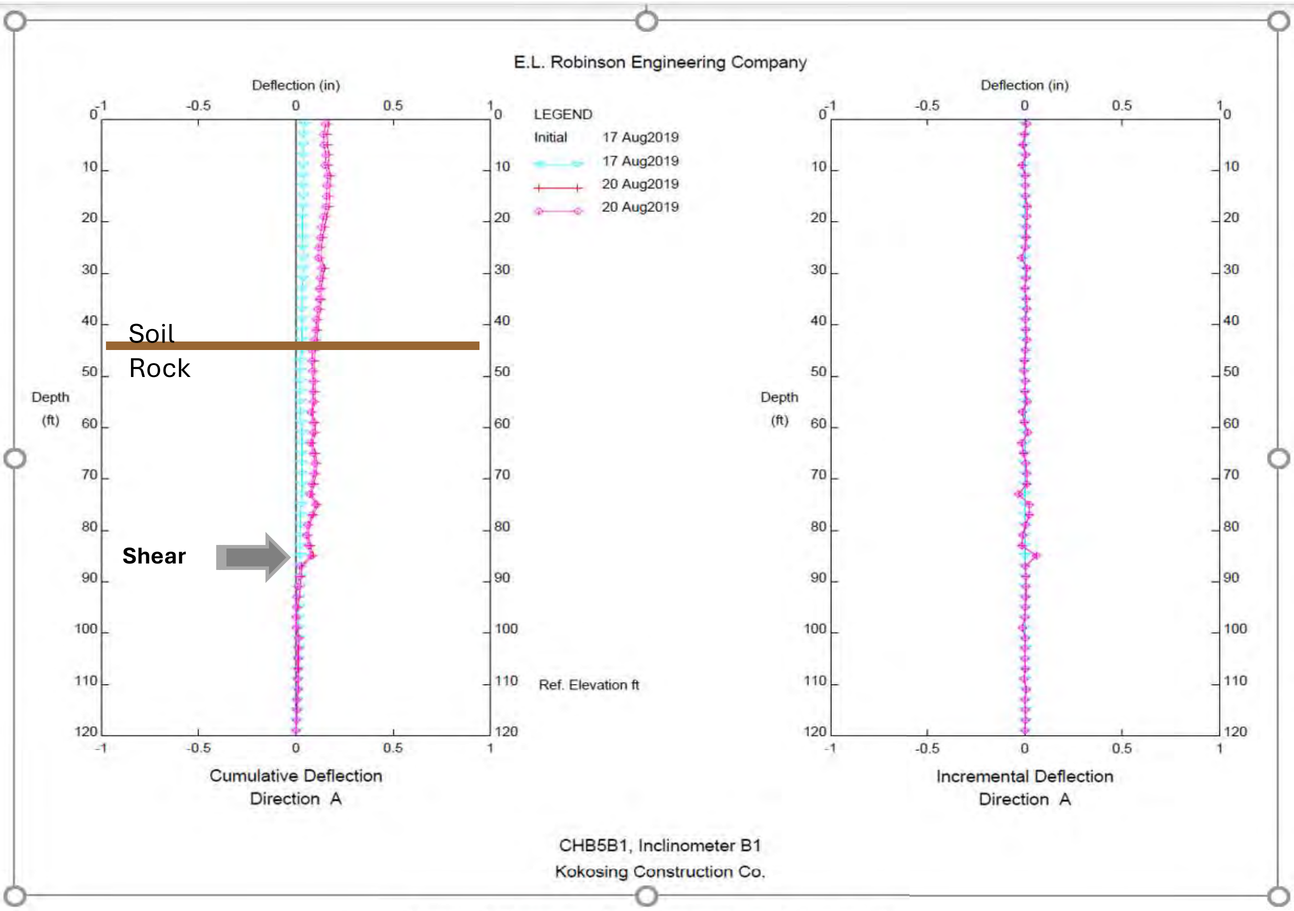


Figure 4: Early readings from inclinometer B1 A-A' direction

Measured Horizontal movement - Inclinometer B-2

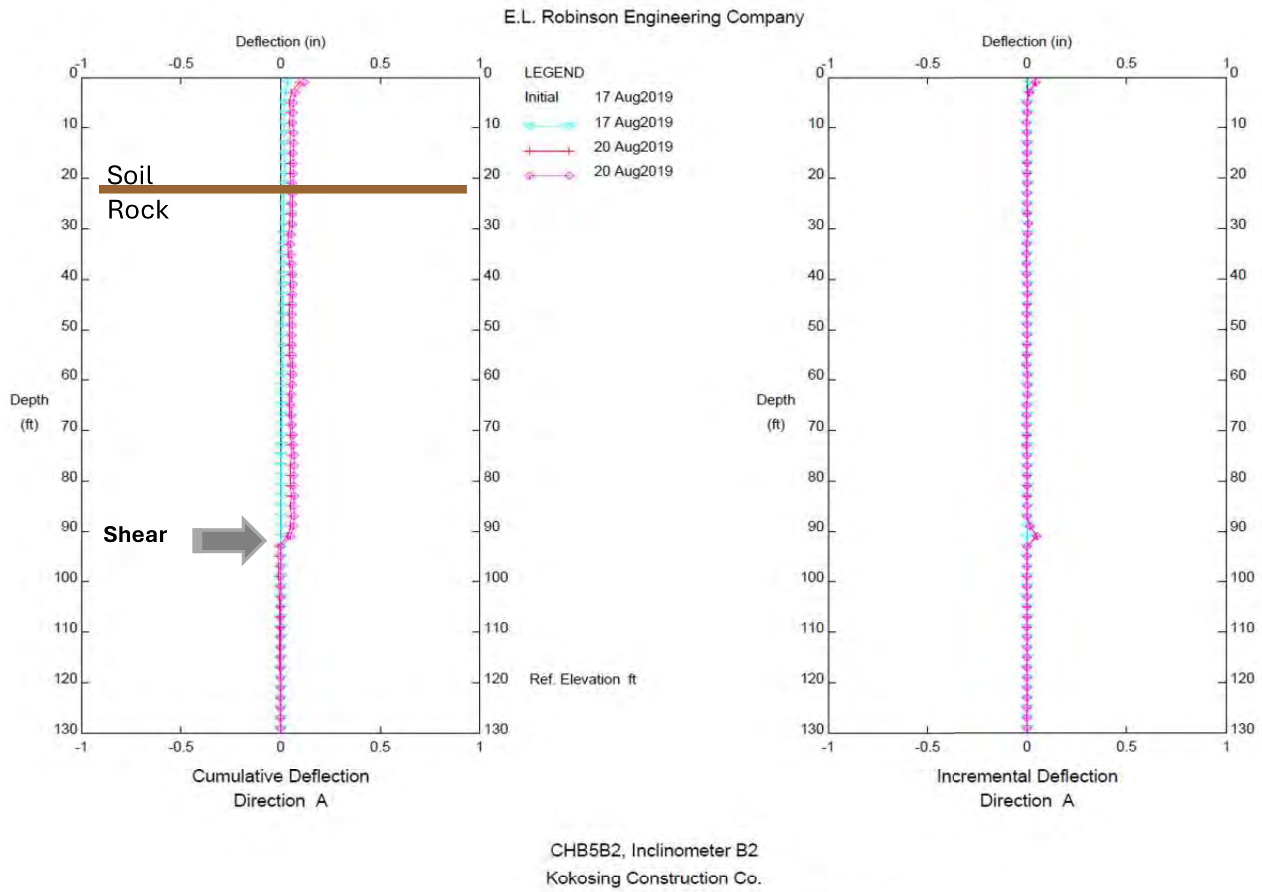
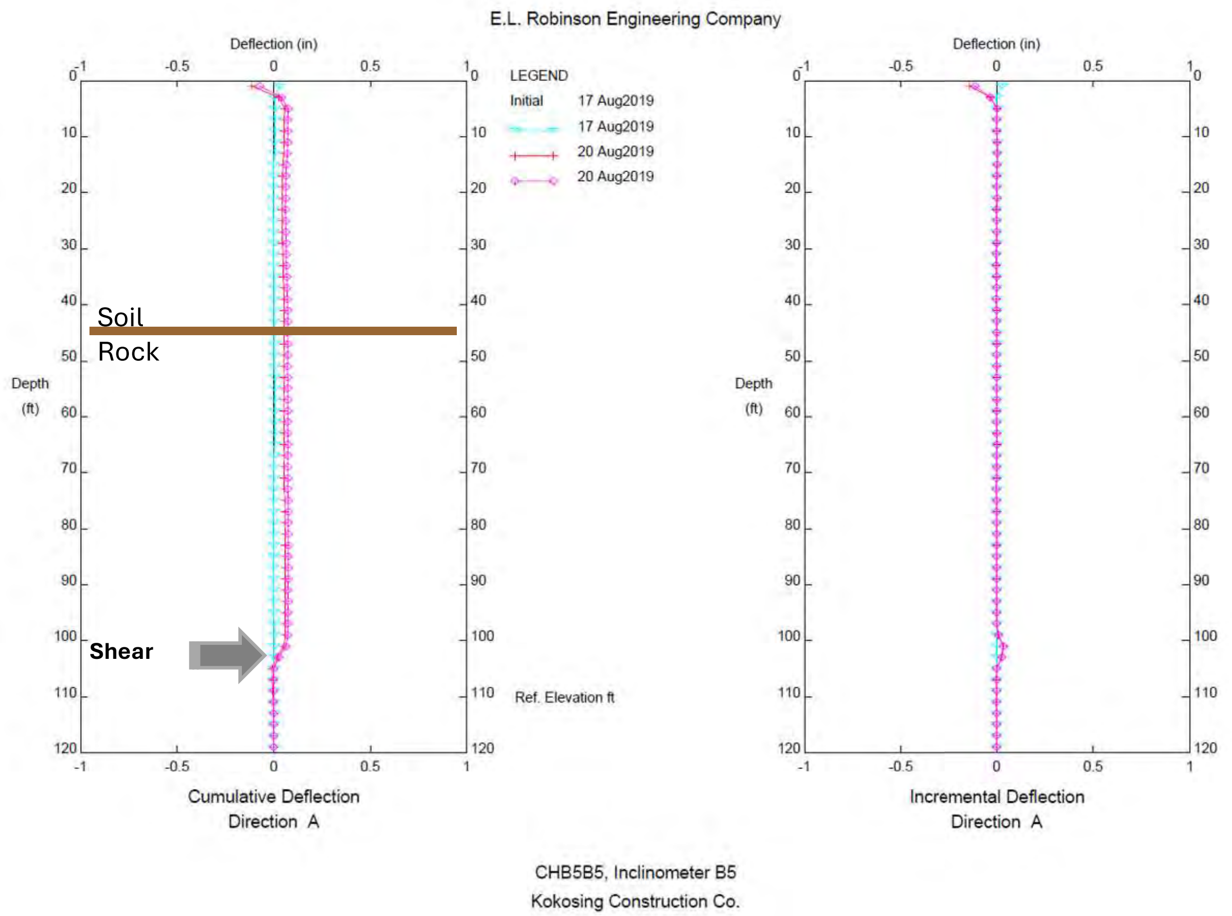


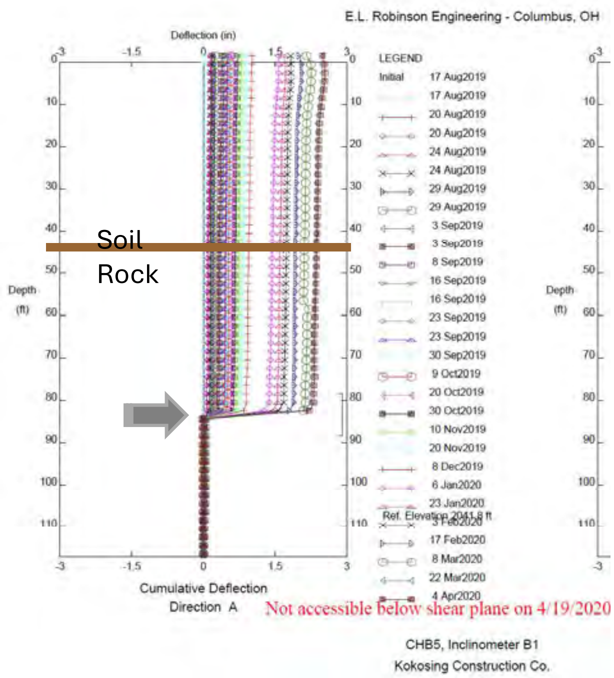
Figure 5: Early readings from inclinometer B2 A-A' direction

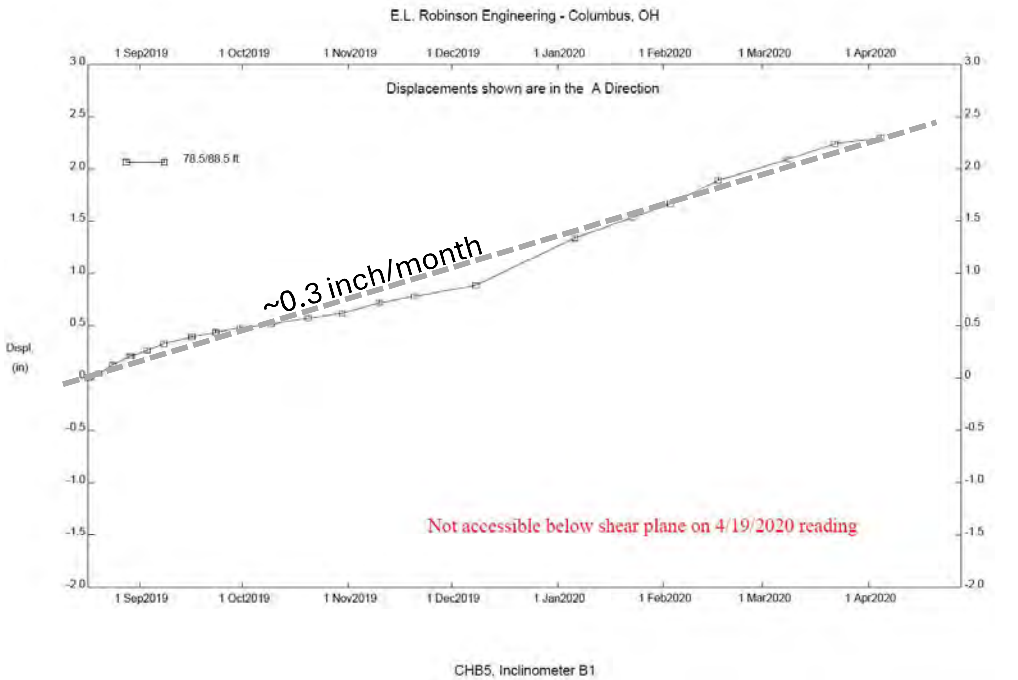
Measured Horizontal movement - Inclinometer B-5



August 20 possibility of movement on distinct sliding plane.
Confirmed by readings on Aug 24 and 29.

Inclinometer B-1 down slope movement in NE direction





Assessment of cause – findings in borings

Boring B-5 showing soil then rock and locations of weak zones in rock

Figure 33 B-5 80.2-95.2 feet

Figure 34 B-5 95.2-105.2 feet

Figure 35 B-5 Shear Zone 105.2-110.2 feet

Steps in Assessment of Cause

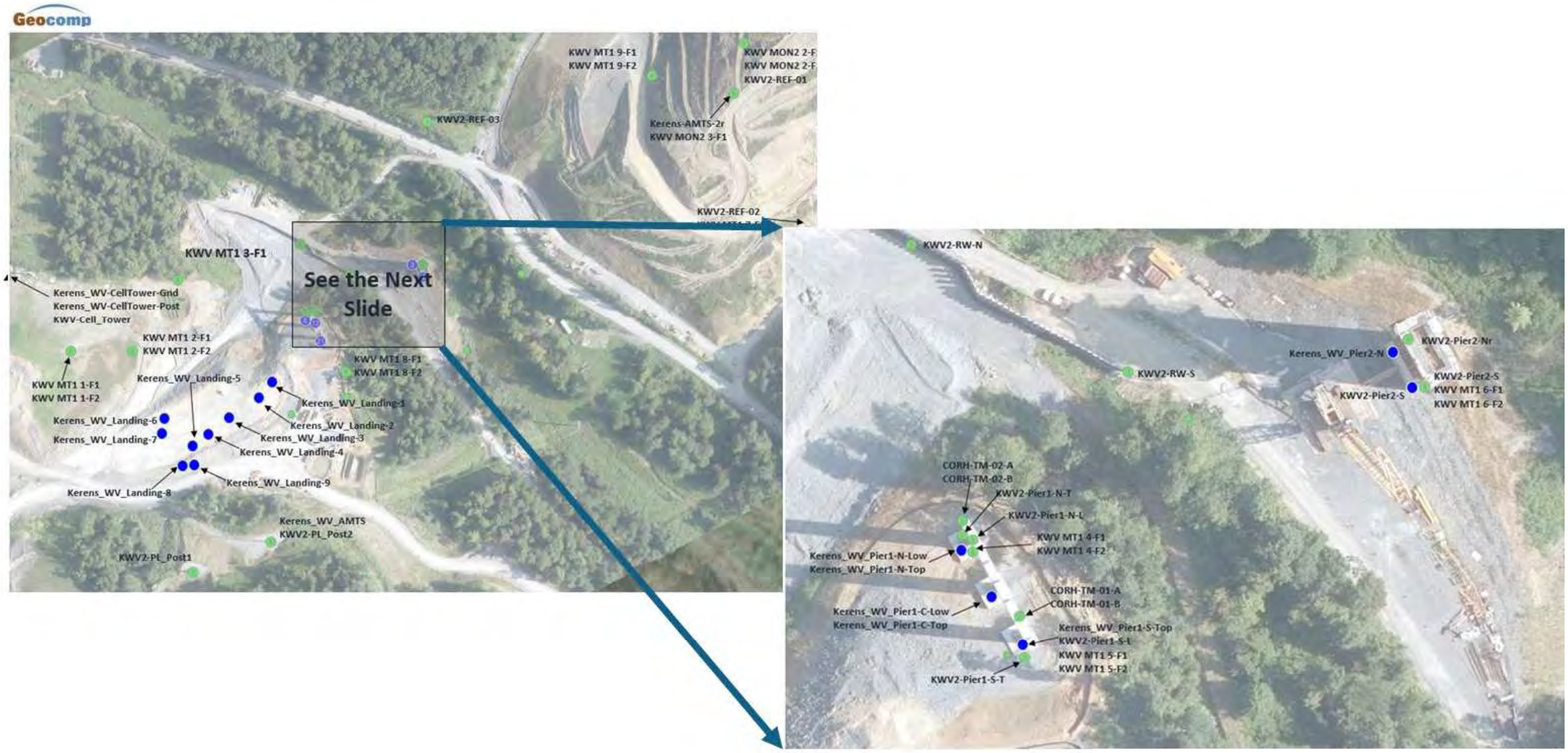
- Reviewed design
- Checked documentation of the work activities
- Checked the surveys
- Made analyses of the "as-is" conditions to determine what could explain the movement
- Made additional borings with careful coring and sampling to identify any potential slip planes in soil and rock
- Installed instrumentation to monitor surface settlement, horizontal movements with depth and internal pore water pressures.

Measured surface displacements with AMTS

Photo 4: RTS #2 used to measure displacements of prisms above ground

Photo 5: Typical prism used to monitor ground surface deformations

Displacement of surface monuments monitored each day



Additional Inclinometer Installation

Installed 15 additional inclinometers with piezometers in borings to determine lateral extent of ground movement and measure pore pressures

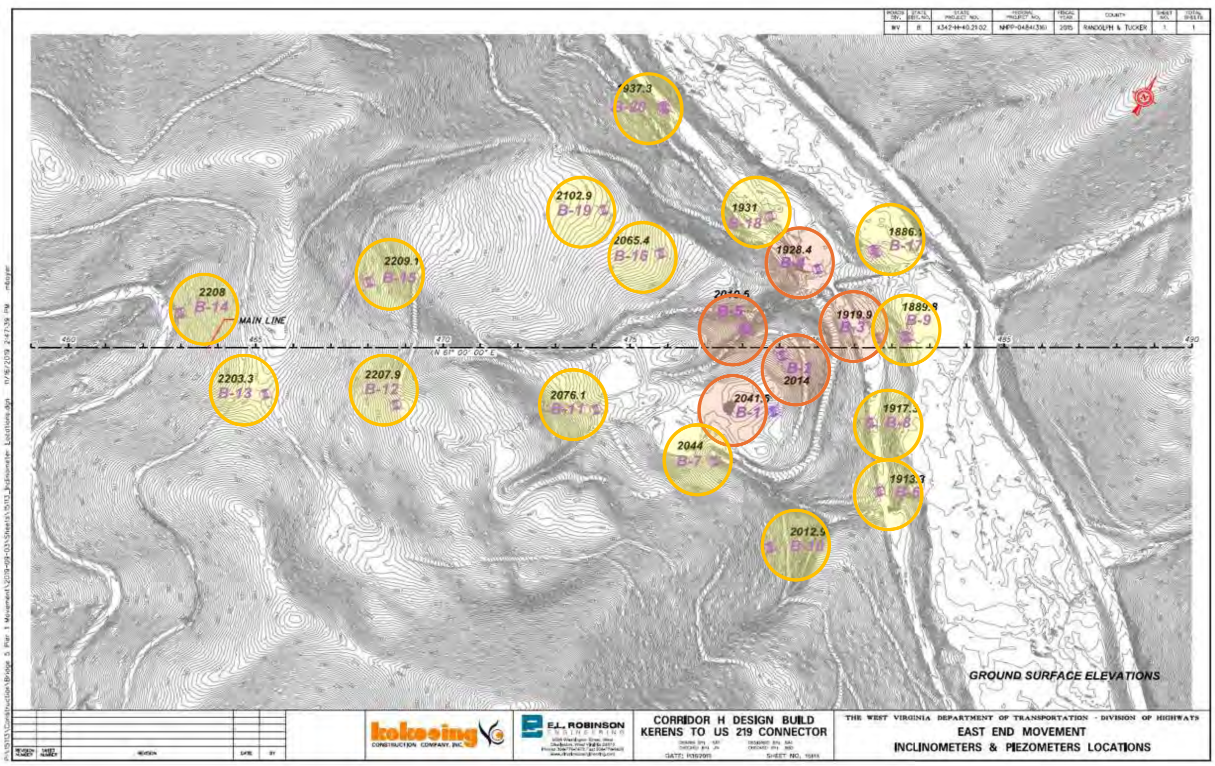
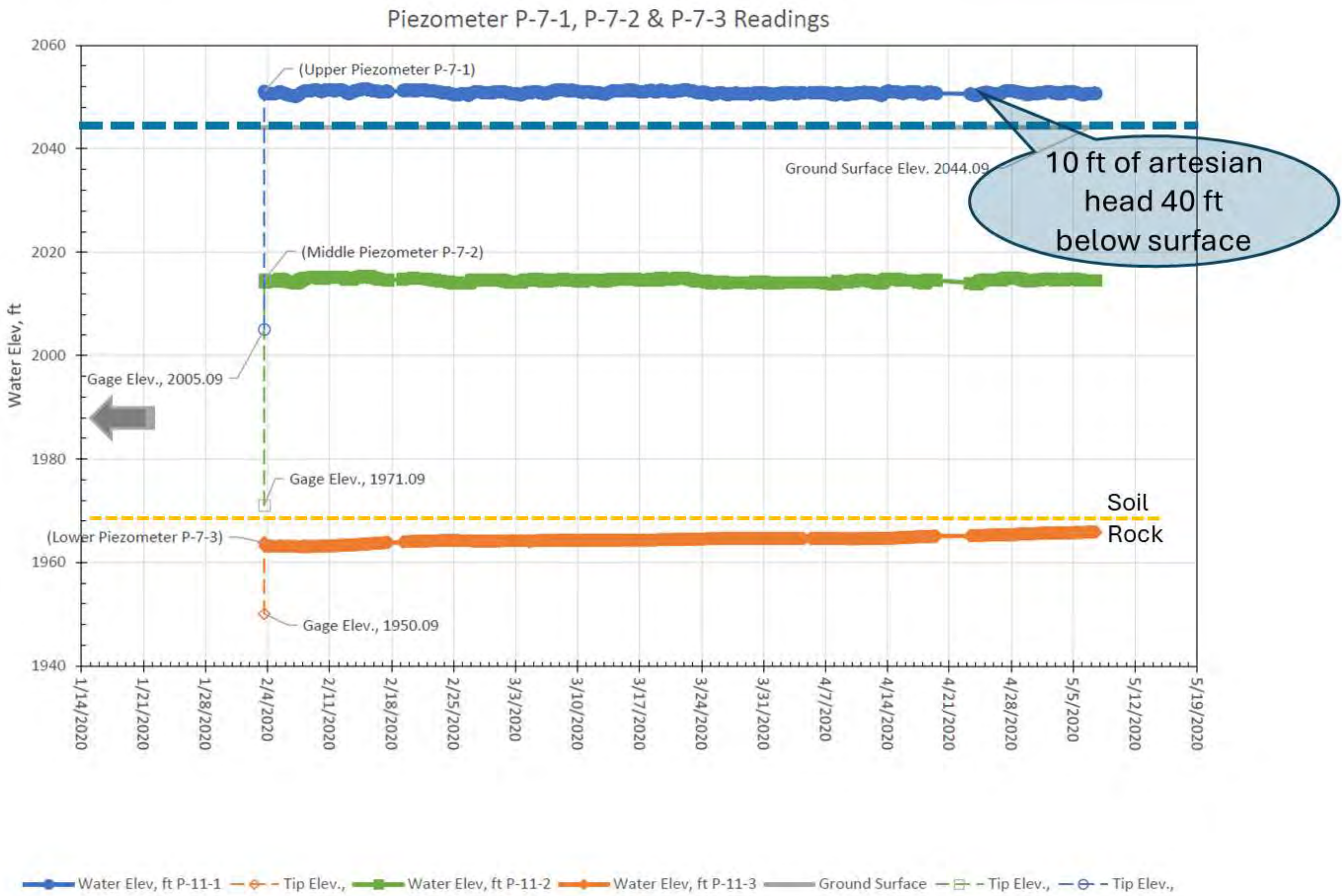
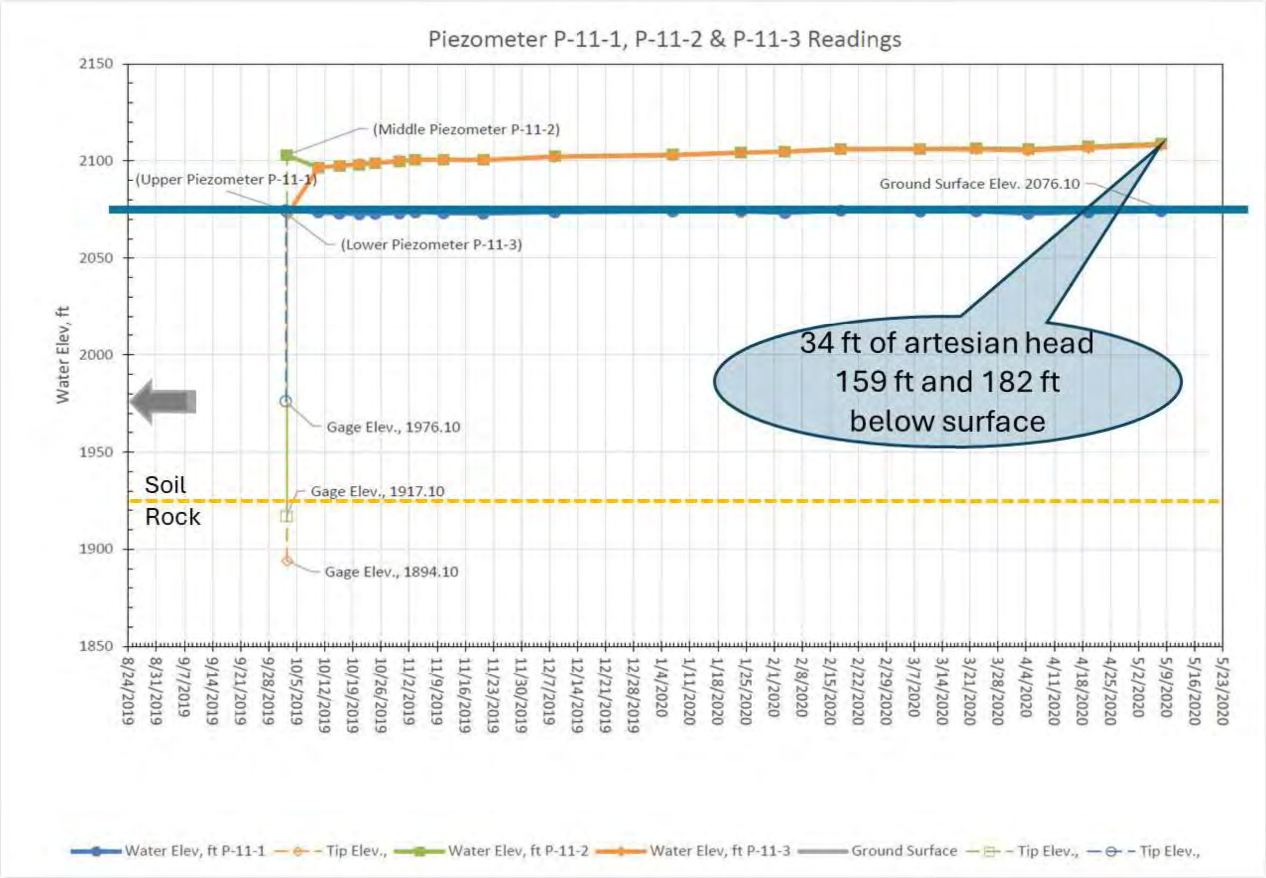


Figure 9: Plan for Bridge 5 site showing locations of all twenty inclinometers

Excessively Elevated Water Pressures in Soil at B-7



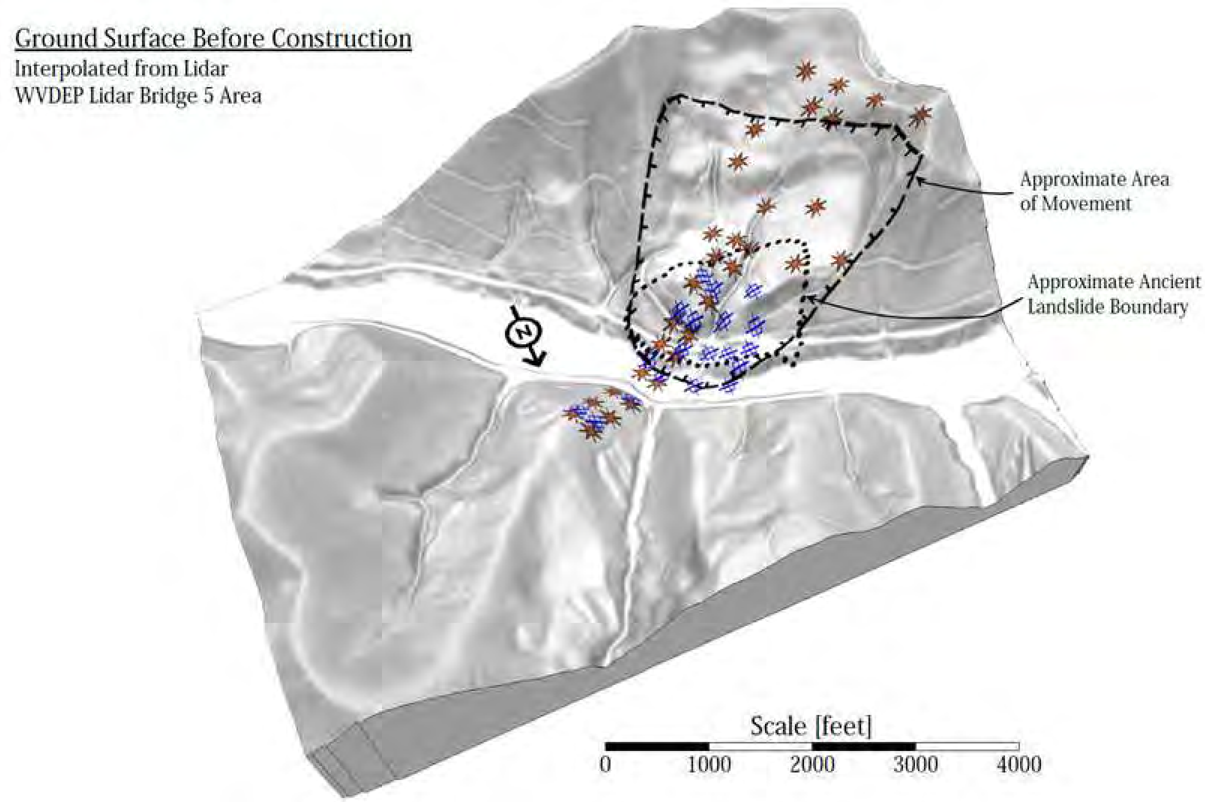
Elevated water pressure in the rock at B-11



Visual proof of elevated heads on the mountainside at B-12 and B-15

Drillers Note:
90’ to 100’ below grade: water flowing at 100 to 120 GPM
Normal flow in the region where encountered ~ 10 – 15 GPM
Not reported in the borings provided in the Geotechnical Data Report

In total 43 borings available to DBT in subject area



- Scope of site investigation by Designer and DBT
- Nothing found to indicate a pre-existing slip plane deep within the bedrock.

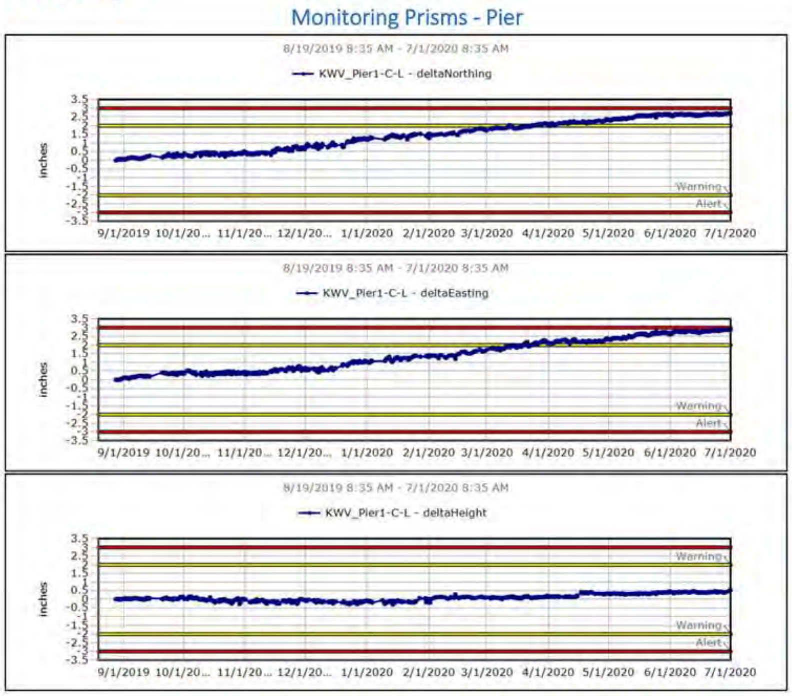
Rock information near slip surface from GDR and DBT borings

Investigation	Boring	Depth to Rock, ft	Length of coring, ft	Approximate Elevation of the Slip Plane^2 , ft	Material Description
RFP	KP1B-208	35	100	none	Siltstone at BOH 135 ft. RQD=91,63,82
	KP1B-209	41	10	none	Siltstone at BOH 50.3 ft. RQD 62,54
	KP1B-210	25.8	65	2250	At 2250 ft reddish orange to brown silty SAND and gravel (colluvium?) N=23. (Rock at 2240 ft -Siltstone. Soil infilling at ~2235.)
	KP1B-211	40	46.5	Below BOH	ended in Siltstone HCSI=2-3; RQD=77,100
	KP1B-212	95.1	10	2015	At 2015 ft greenish gray to dark brown SANDY GRAVEL: trace silty clay, gravel, decomposed rock. (colluvium?) N=39,30
	KP1B-213	126	71	1955 or 1977	"Silty Gravel:rock fragments, colluvium". N=32,27,52
	KP1B-214	no rock	19	1960	Transition zone at 1960.8 ft from CLAY: some gravel to GRAVEL with cobbles. (colluvium?) N>50
	KP1B-215	60	20	Below BOH at 60 ft	ended in Siltstone HCSI=3; RQD=78,73,50
	KP1B-216	78	29	Below BOH at 105 ft	ended in Shale HCSI=2; RQD=93,70,94
	KP1B-217	21	30	Below BOH at 51 ft	ended in Siltstone HCSI=2; RQD=85,25,95
	KP1B-218	75	20	Below BOH at 95 ft	ended in Siltstone HCSI=3; RQD=78,36,90
	KP1B-219	75	30	1900 or 1950	Silty Sand or Sandy Gravel-Colluvium. N=40,24,27,13,60,38,23,27
	KP1B-220	65.7	20	1900	Sandy Clay some gravel (colluvium?). N=30,25,19
	KP1B-221	36	30.5	1875	Silty Gravel (colluvium?). N=46
	KP1B-222	51.5	30.5	1875	Boulders and cobbles with clay (in river N.A.)
	KP1B-223	6.1	15	1875	"gray and brown SILT : trace gravel" (in River N.A.)
DBT	SB5-05	15	11	1875	Silty Clay (shallow with N=7)
	SB5-06	22.5	15	1875 or higher	weathered Sandstone. RDQ=0

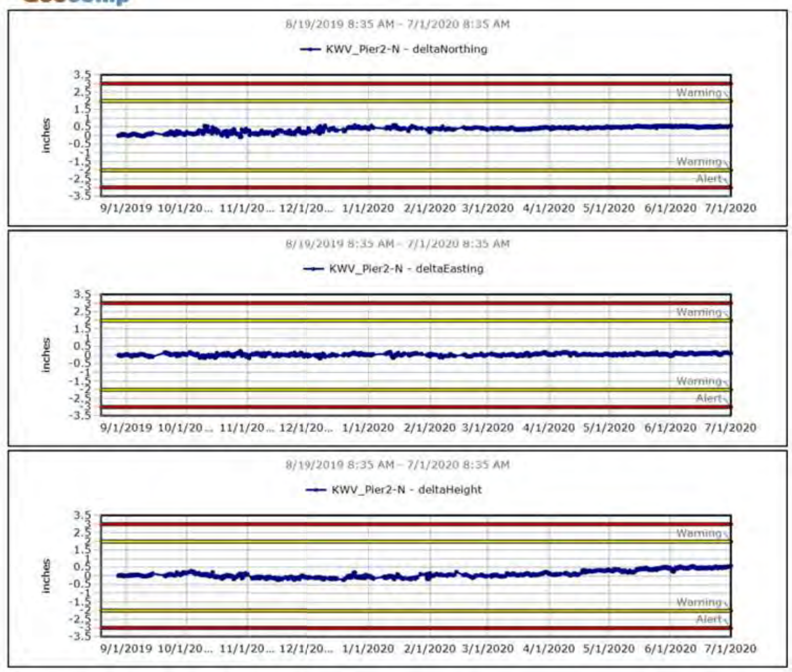
-no slickensides identified; no pre-existing slip plane identified;
-no evidence of a weak sliding plane in colluvium from an ancient landslide

Measured Deformation of Piers

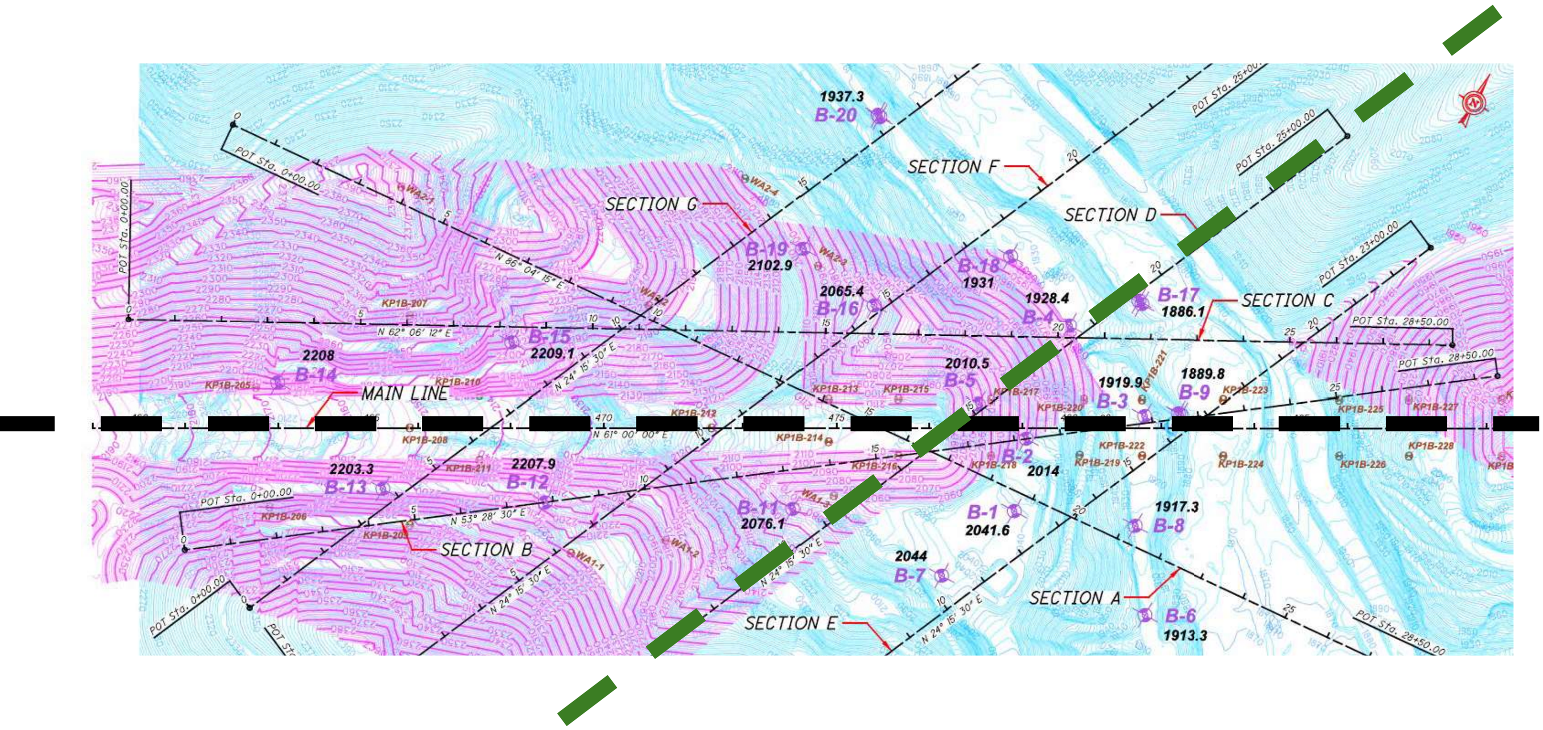
Pier #1



Pier #2

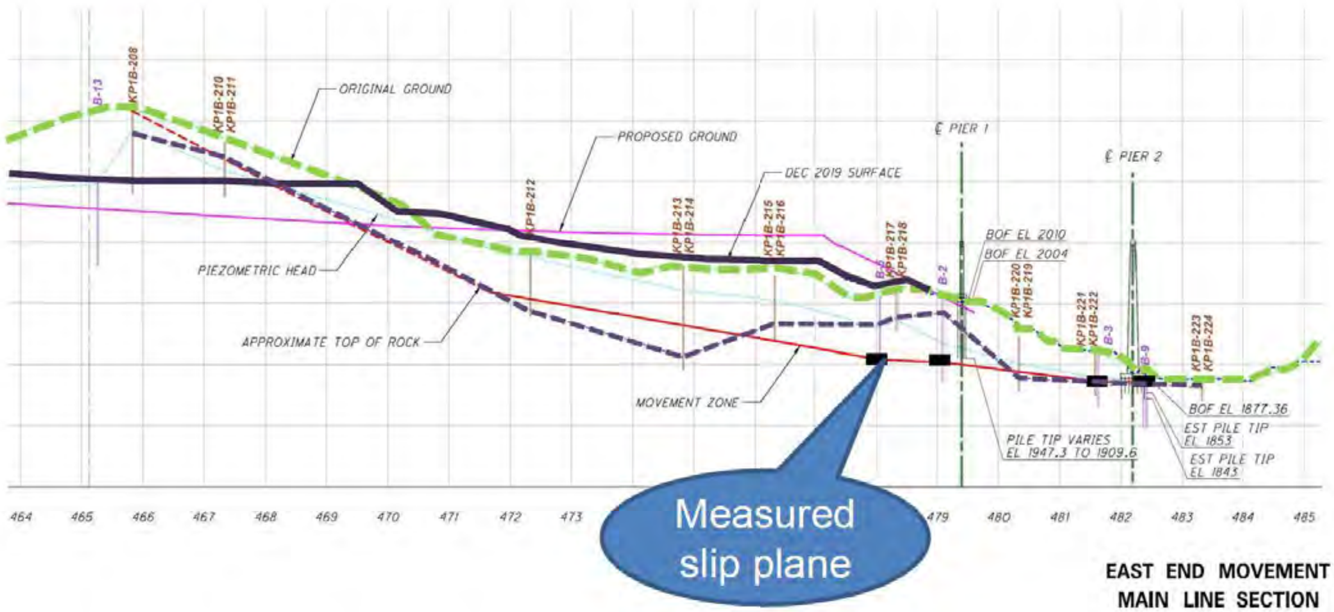


Back analysis of stability



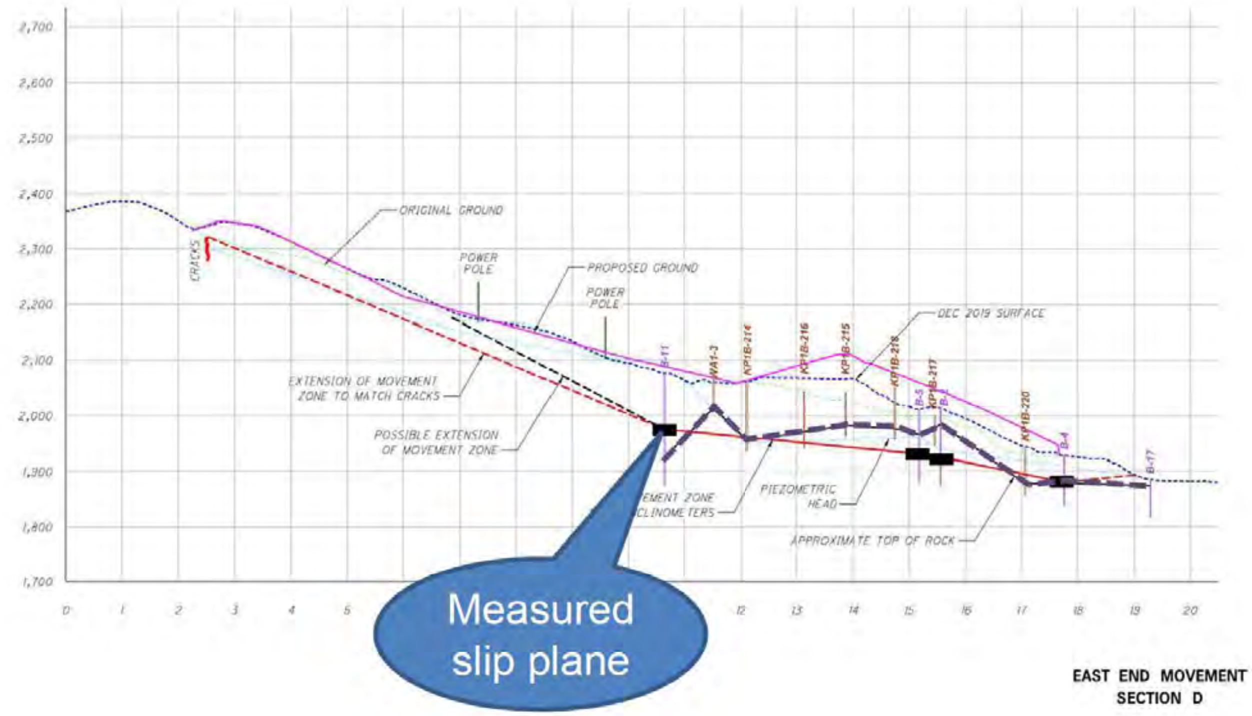
Ground surface contours are as existed July 2019

Back analysis of stability - main line section



Sliding in soil and rock – friction angle on sliding plane = 15° from back analysis

Back analysis of stability of Section D



Sliding in soil and rock – friction angle on sliding plane = 15° from back analysis
Design used WVDOH presumptive values of c'=7200 psf, ϕ'=22° for rock.

Depth of sliding below top of rock – sliding went deep below the top of bedrock

Table 1

Boring	Ground Surface Elevation, ft	Elevation of top of rock, ft	Elevation of Sliding Surface, ft	Depth of sliding below ground surface, ft	Depth of sliding below top of rock, ft
B1	2042	1998	1959	83	39
B2	2014	1984	1924	90	60
B3	1920	1865	1872	48	-7
B4	1928	1879	1876	52	3
B5	2011	1986	1912	99	74
B5A	2010	1965	1912	98	53

Table 2

Boring	Ground Surface Elevation, ft	Elevation of top of rock, ft	Elevation of Sliding Surface, ft	Depth of sliding below ground surface, ft	Depth of sliding below top of rock, ft
B6	1910	1890	none	none	none
B7	2044	1969	1988	56	in soil
B8	1917	1877	1908	9	in soil
B9	1890	1869	1869	21	0
B10	2013	1983	none	none	none
B11	2076	1924	1976	100	in soil
B12	2208	2188	2200**	8	in soil**
B13	2203	2198	none	none	none
B14	2209	2199	2140	69	59
B15	2209	2161	2117	92	44
B16	2065	2005	1977	88	28
B17	1887	1874	1870	17	4
B18	1930	1872	1883	47	in soil
B19	2103	2016	2028	75	in soil
B20	1937	1900	none	none	none
** also movement in rock elev 2050 - 2110					

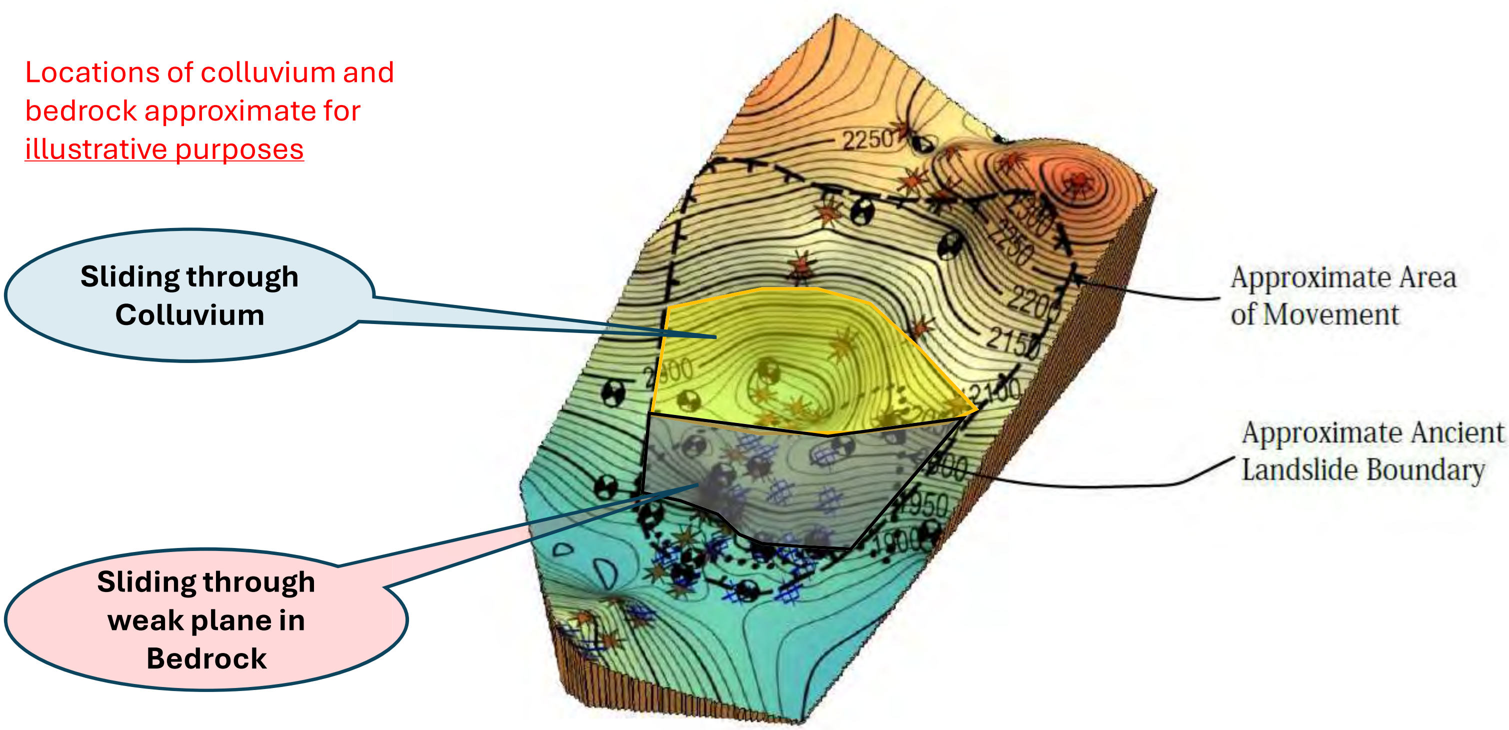
In design calculations critical slide surface was through overburden.

But measured sliding is occurring:

- on or within bedrock in upper part of slide
- through overburden in middle
- through weak plane in bedrock in lower part of slide
- through soil at base of slide

Bedrock surface from all borings and surface outcrops

Locations of colluvium and bedrock approximate for <u>illustrative purposes</u>

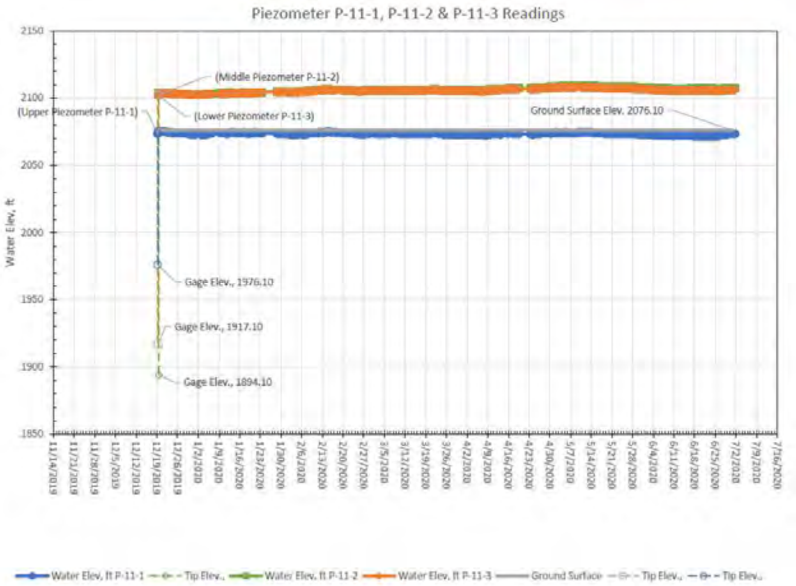


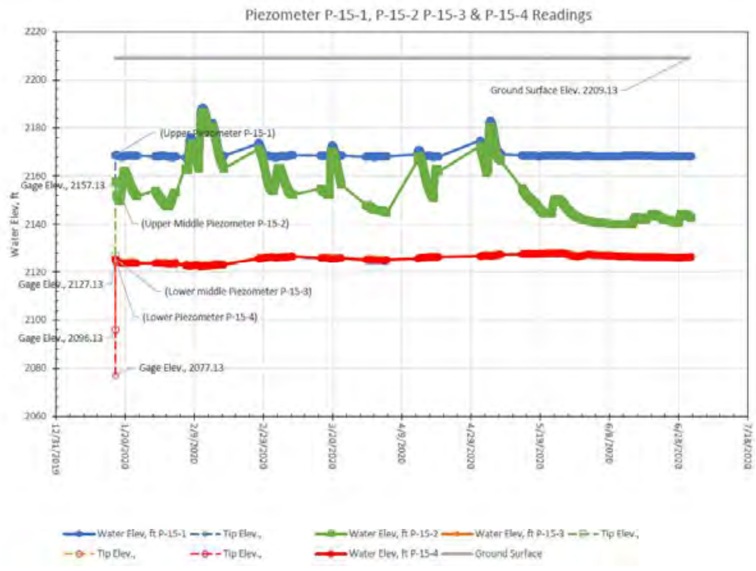
Current landslide is occurring through a weak plane in bedrock, a wholly unexpected event.

Pore Pressure

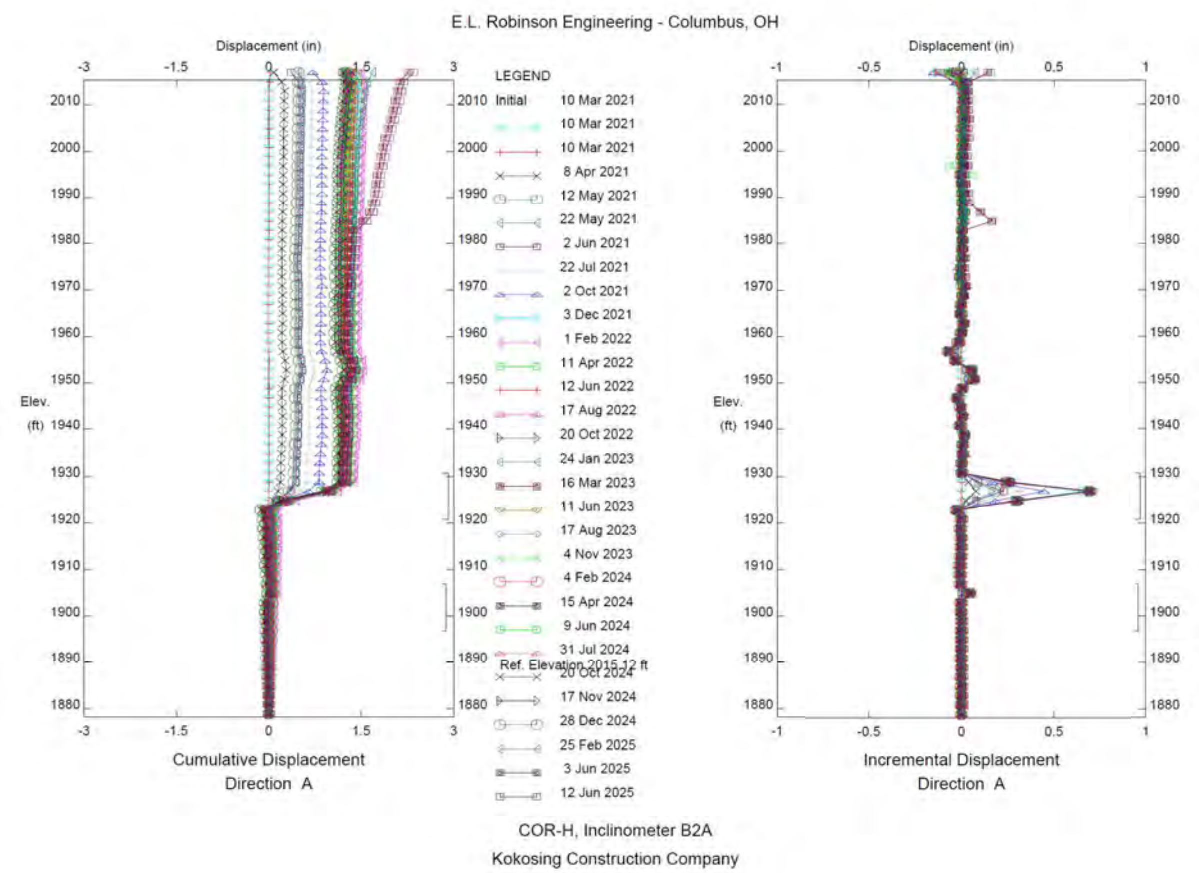
Elevated pore water pressures, even artesian conditions, in soil and rock much higher than information would indicate and differing from those ordinarily encountered and generally recognized as inherent in the work . .

Expected from the available information – hydrostatic with depth with groundwater generally deeper than 30 ft Encountered from measurements – locations on mountainside with elevated pore water pressures, even artesian conditions, that lowered stability compared to what was reasonably anticipated by Designer and QAM



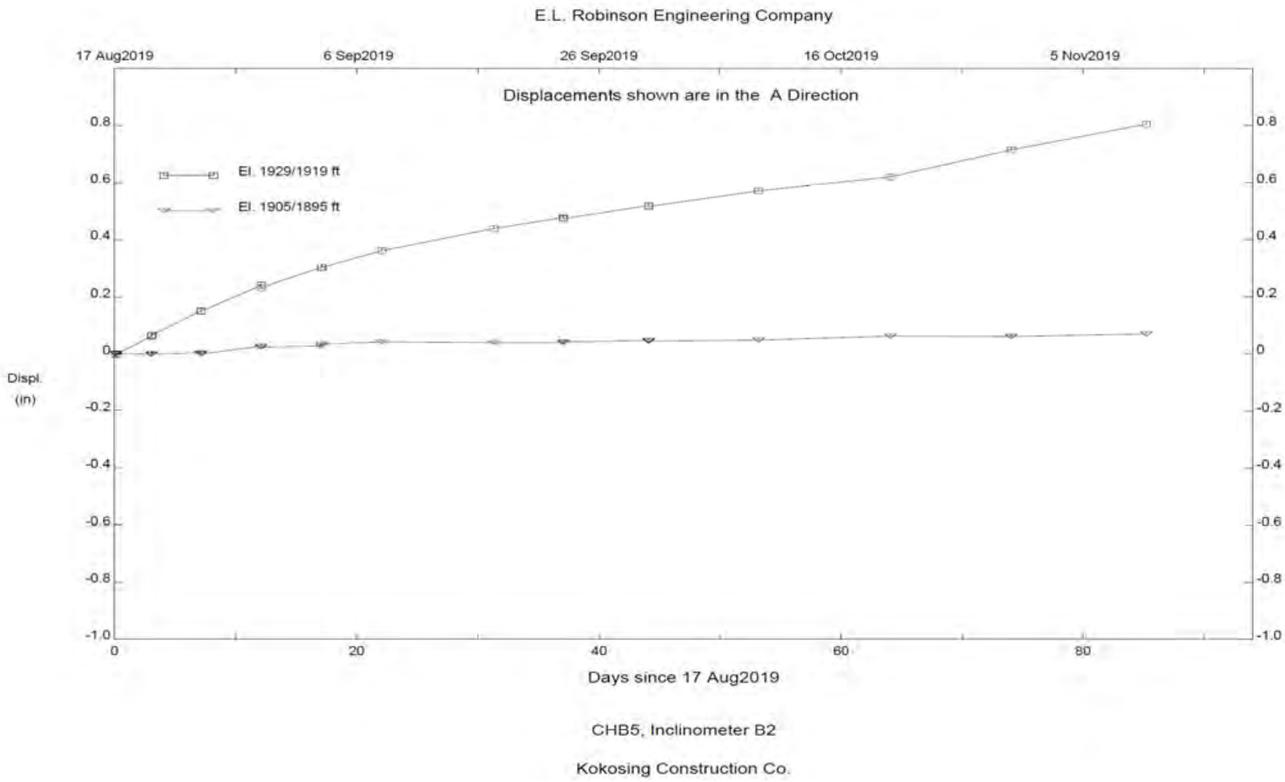


Inclinometer Readings



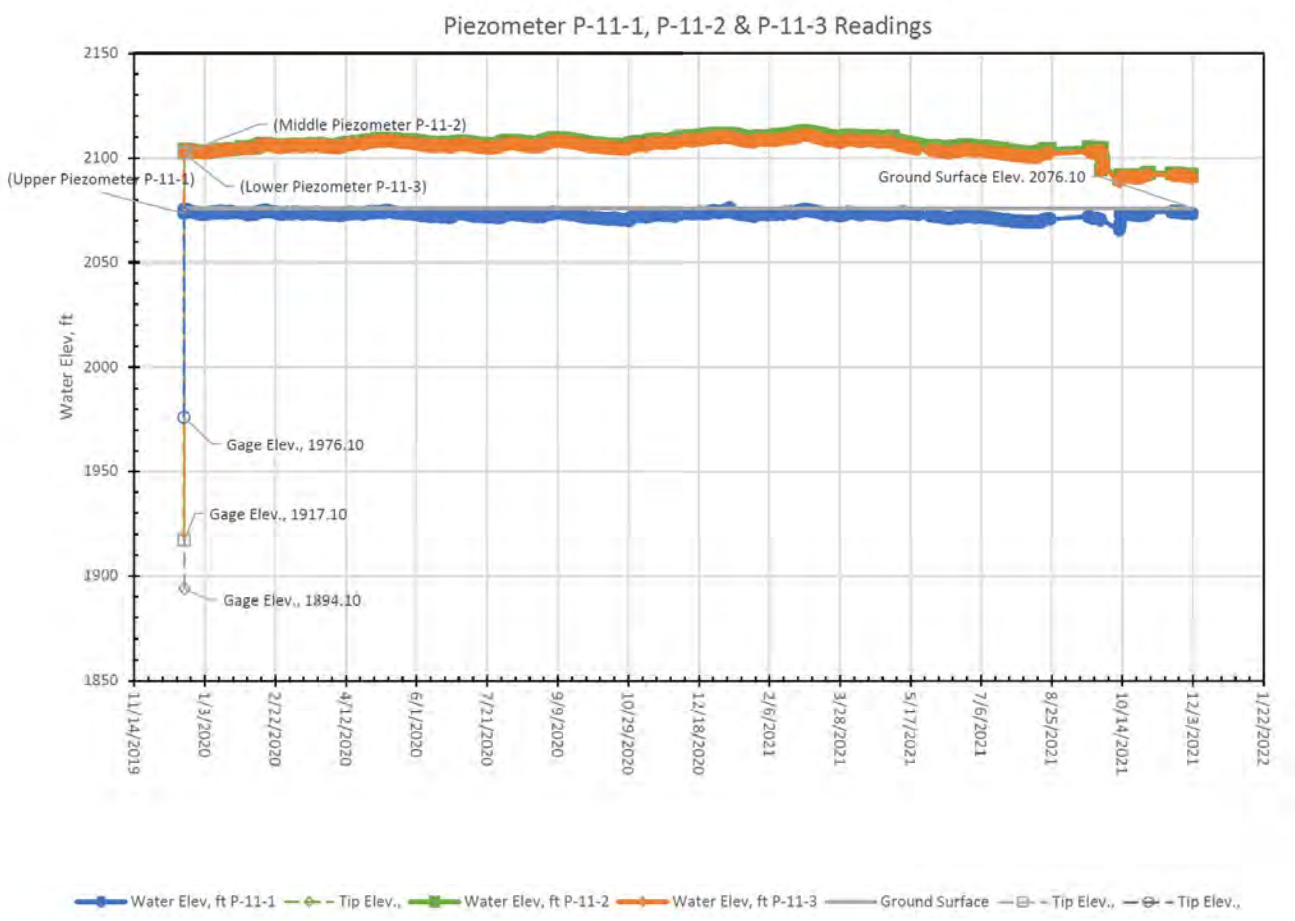
Assessment of cause – initial results from inclinometers

- Early data showing zone of movement in the rock
- Graph of horizontal movement at select depths with time

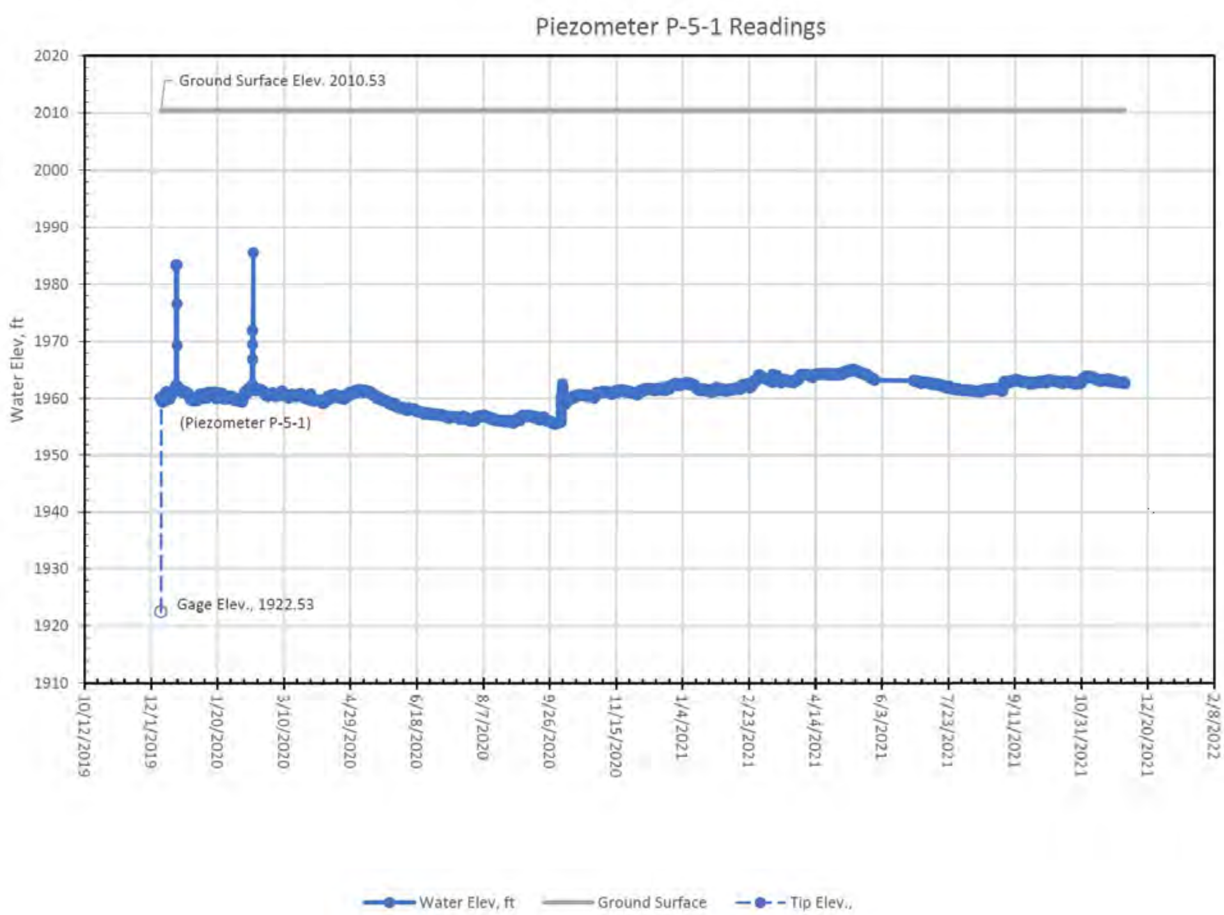


Additional Inclinometers

Piezometer Readings



Piezometer Elevation before Drainage



Considered Options for Remediation

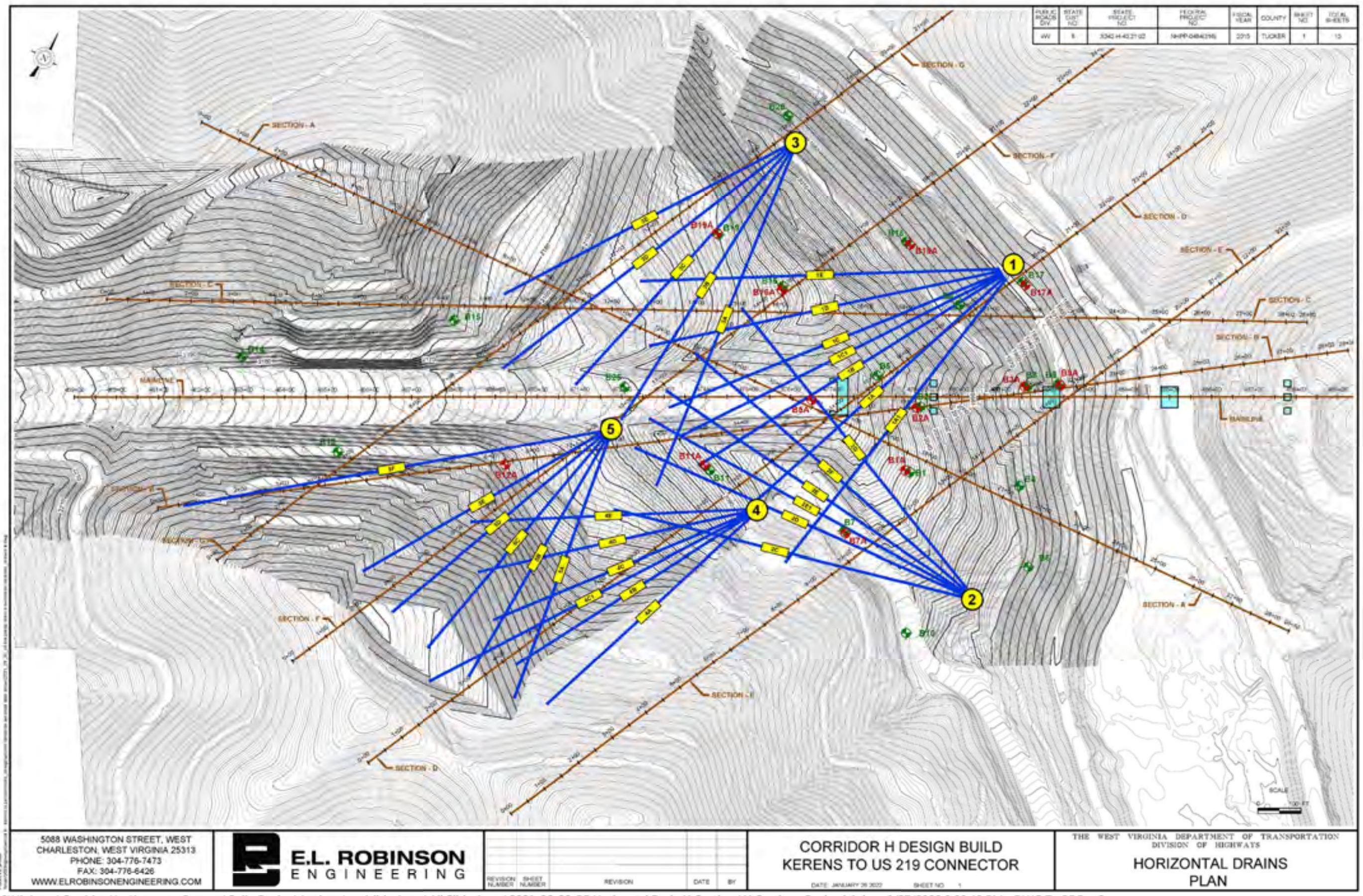
- Unload top of slide area
- Buttress toe of slide area
- Dewater the slope
- Ground anchors
- Grout to strengthen weak rocks
- Add retaining walls
- Construct drilled shafts

Chosen Remediation and its Design

- Horizontal drains up to 1000 ft long to remain permanent
- Remove materials from upper parts of the moving mass
- Place soil and rock as a toe buttress
- Monitor pore pressures and deformations to prove effectiveness of the remediation
- Line ditches to reduce surface water infiltration

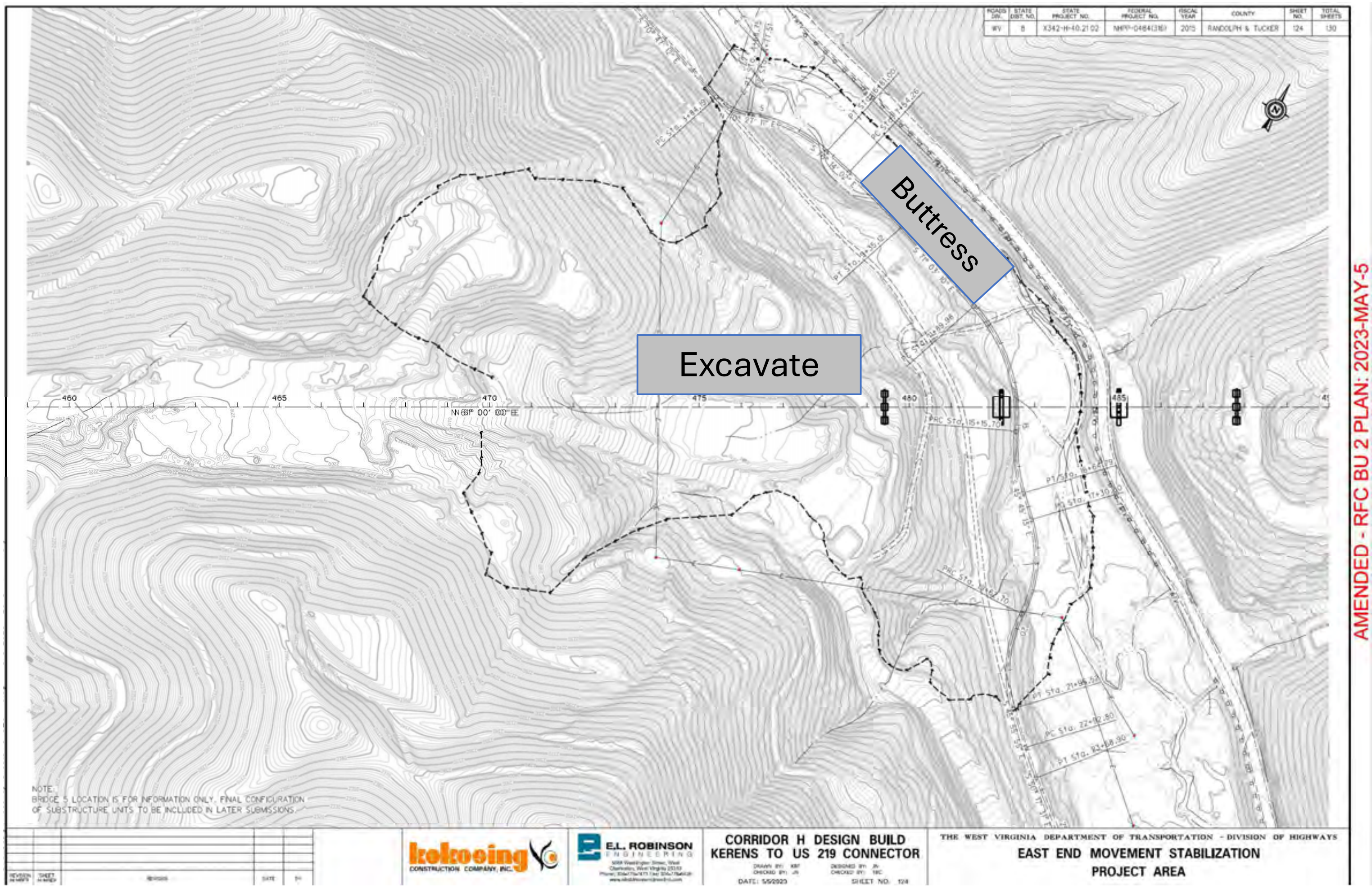
Horizontal Drain Layout

Horizontal Drain Layout

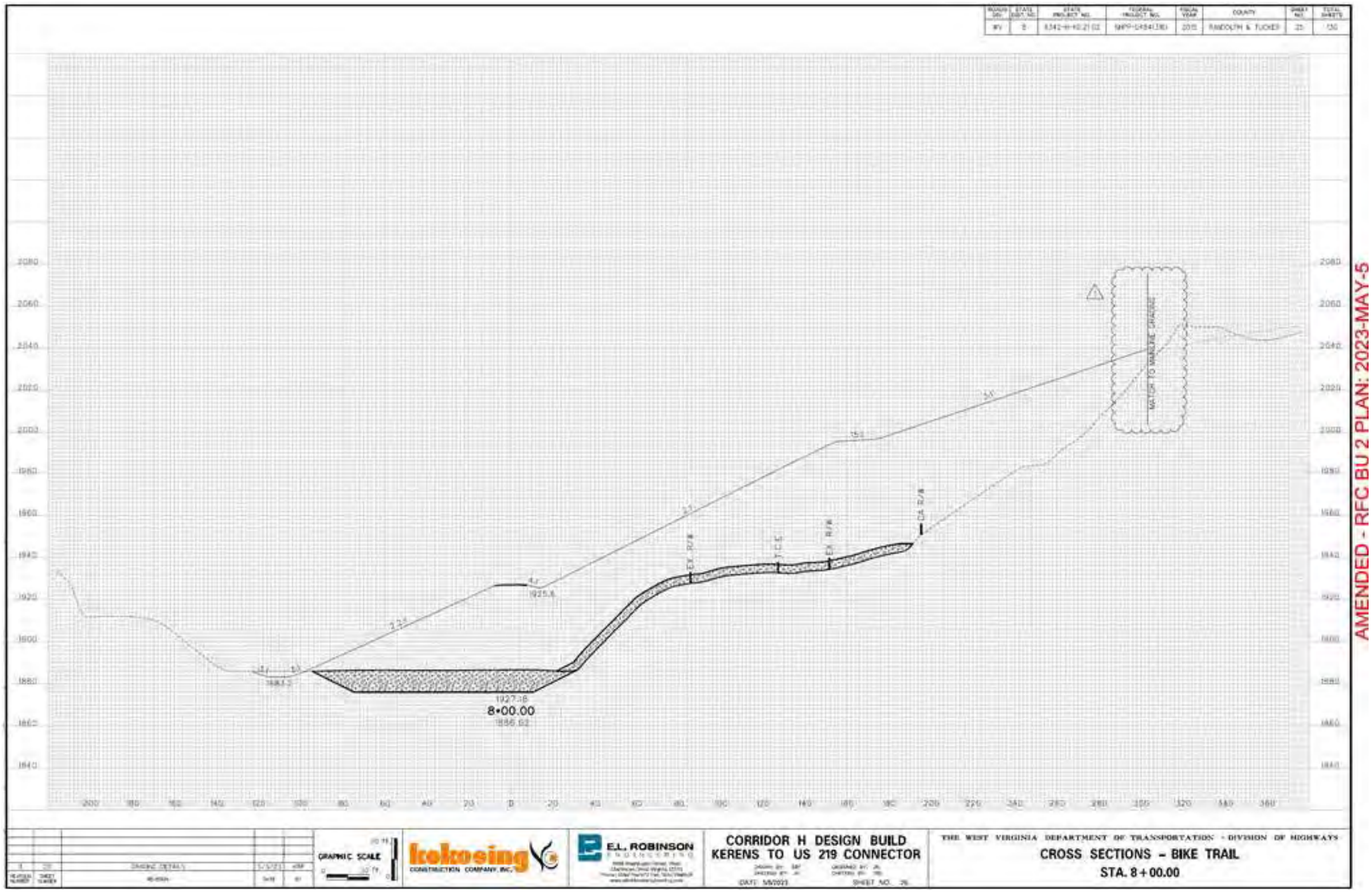


\\main\(09)Highway\Corridor H - Kerens to Parsons\Soils_Design\Ancient Landslide Areas\All Slide Areas\2021_08_30_B5 Horizontal Drain X-Sections\X-Sections_Revised 4.dwg, 1/27/2022 3:09:48 PM, _DWG To PDF.pc3

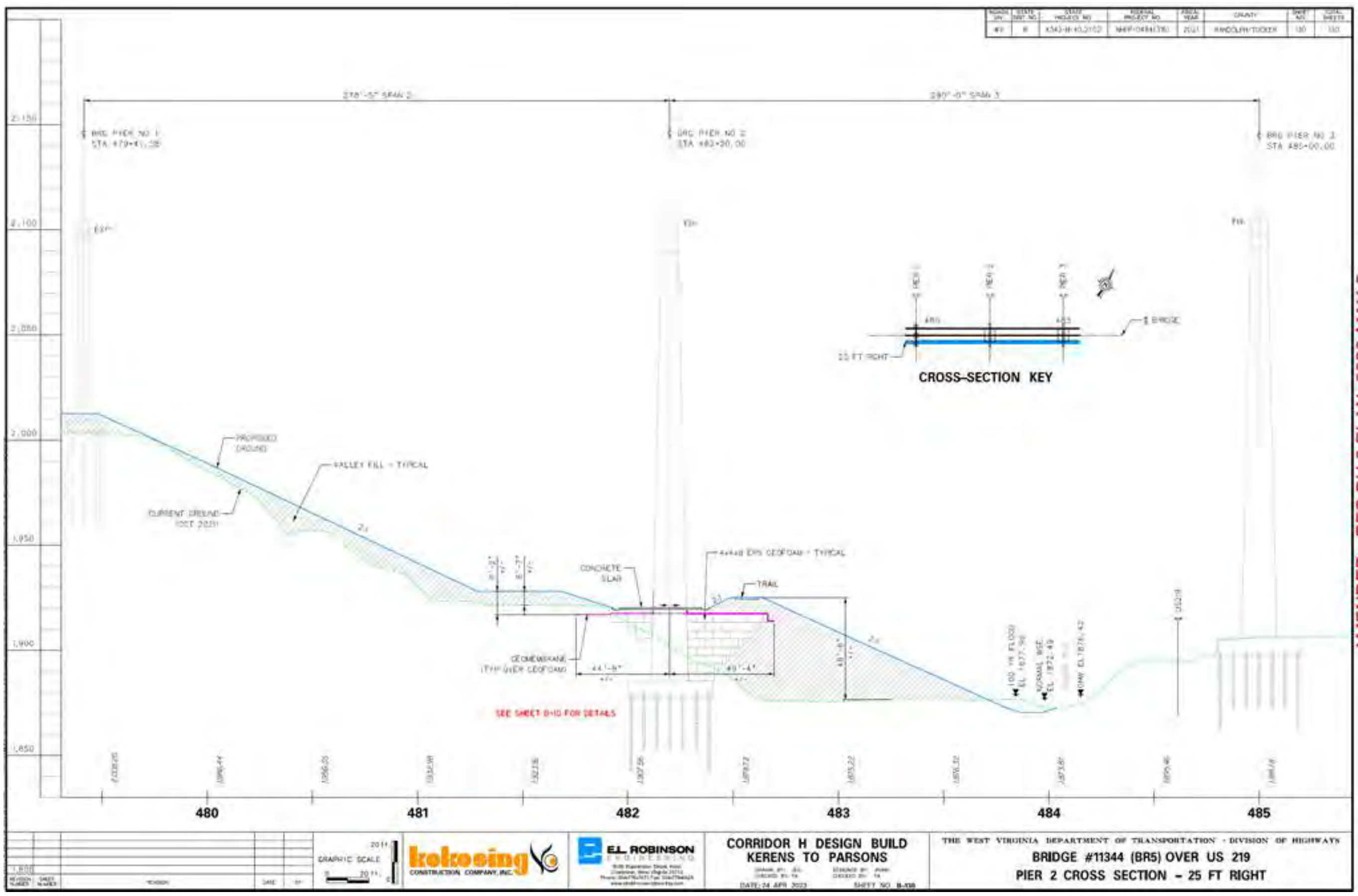
Remediation design – earthwork



Remediation Design - Earthwork

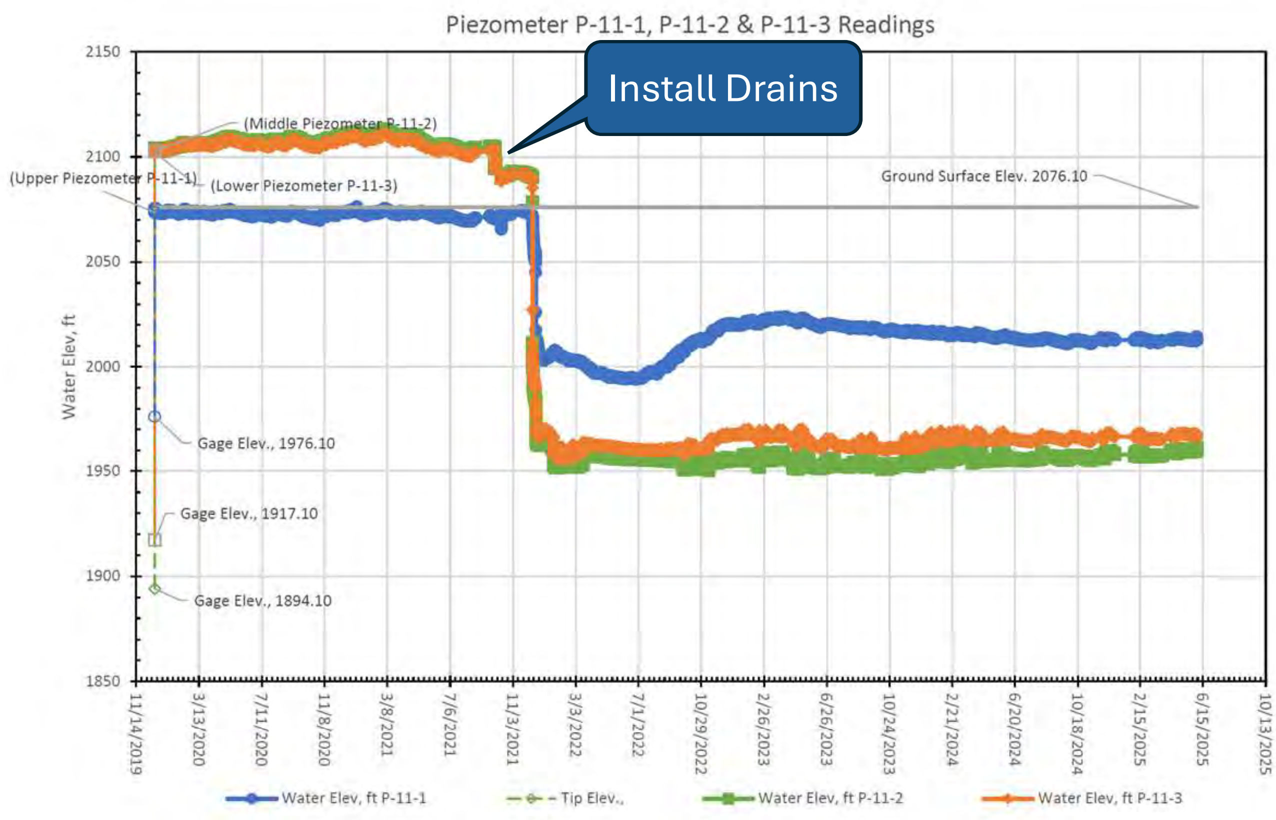


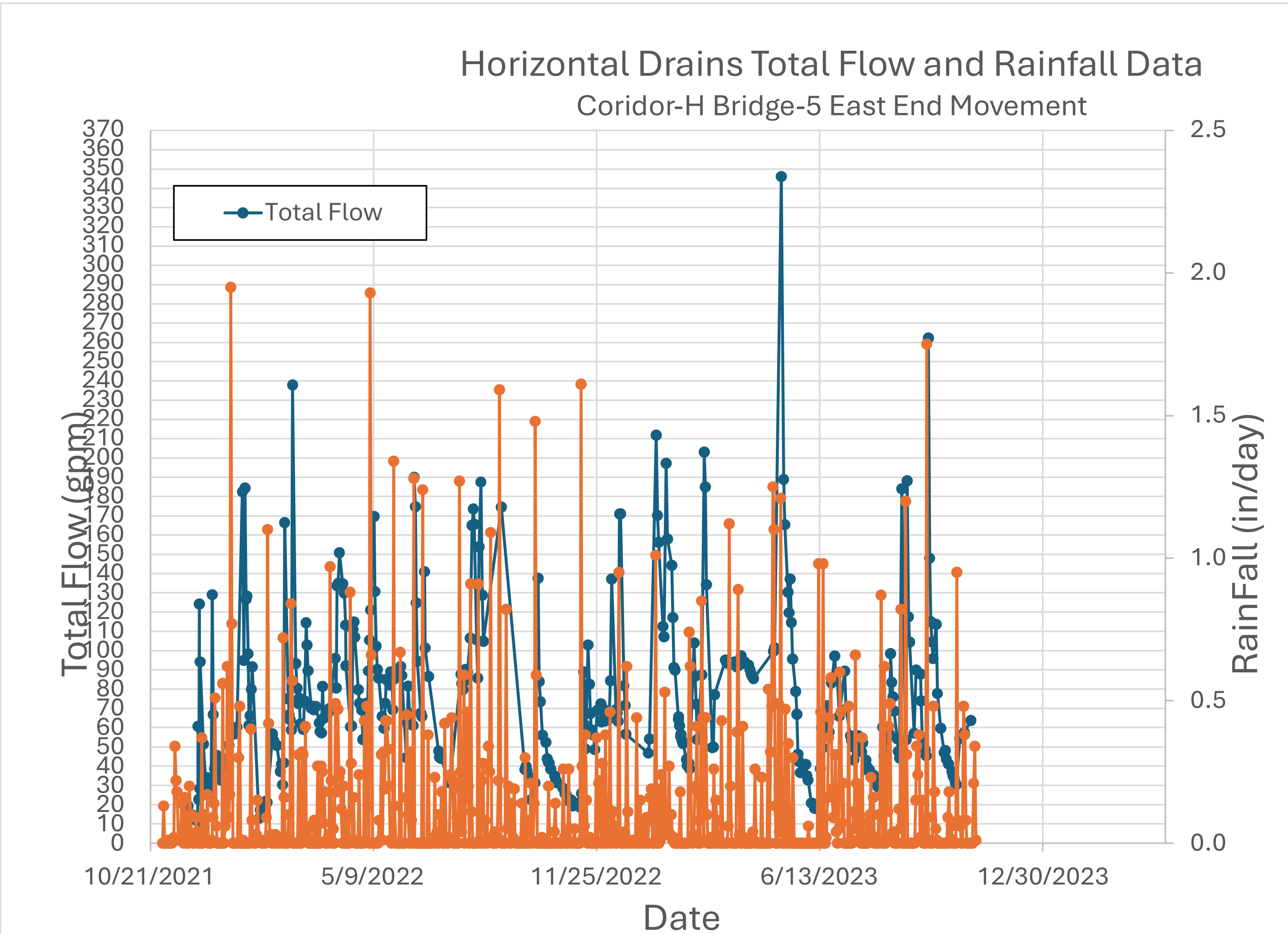
Remediation Design - Earthwork



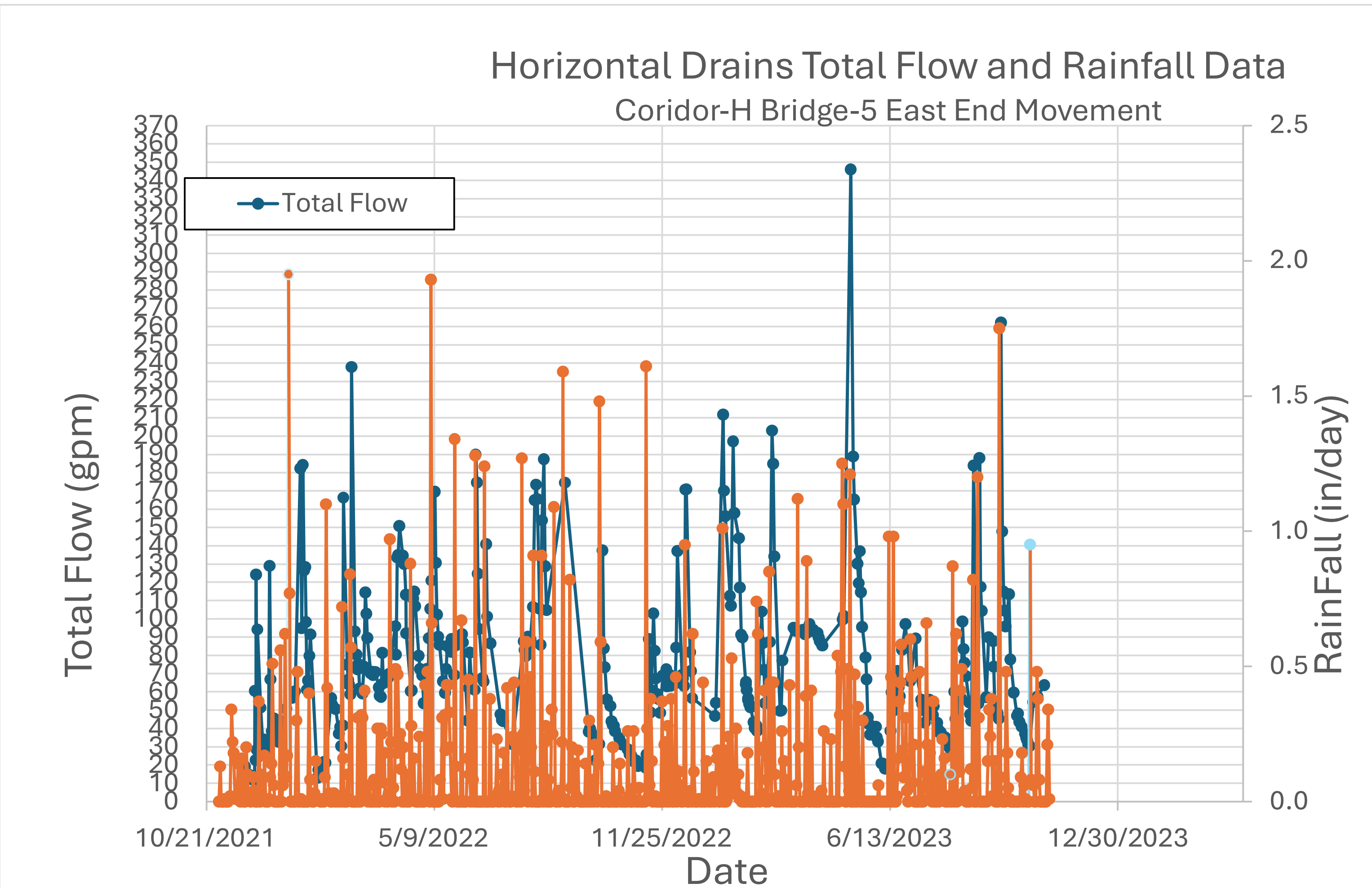
Horizontal Drains Working

Piezometer Readings



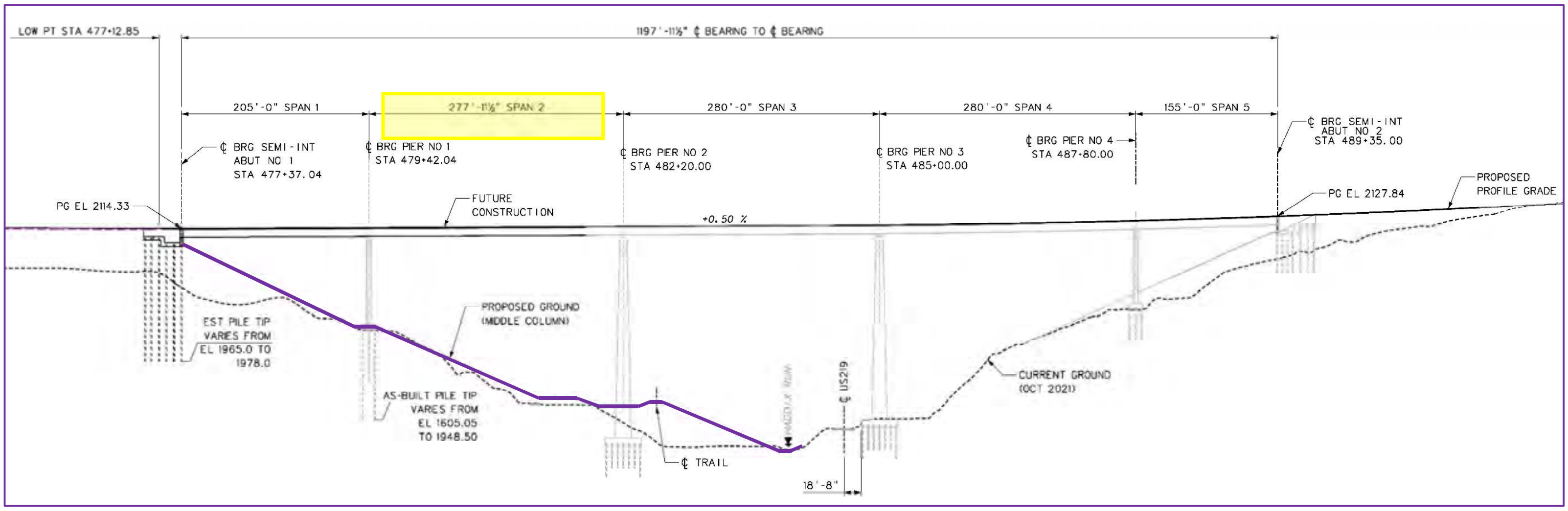

Horizontal Drains Total Flow and Rainfall Data
Coridor-H Bridge-5 East End Movement
Total Flow
Total Flow (gpm)
RainFall (in/day)
Date
10/21/2021
5/9/2022
11/25/2022
6/13/2023
12/30/2023

Total Flow from Drains and Rainfall

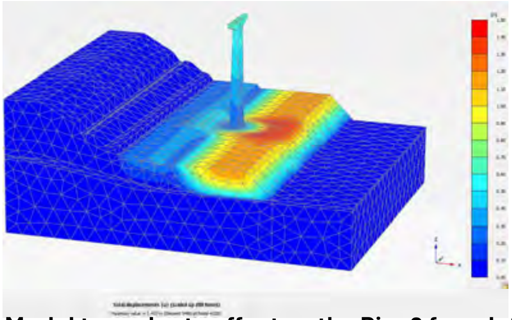


Changes to Bridge 5

- Abutment and Pier Grading for Buttress Fill
- Pier 2 Foundation Changes Due to Grading. Use of Lightweight (GEOFOAM)
- Span 2 Length Modification. Shortened Span 2 by approximately 2 ft
- Girder Modifications

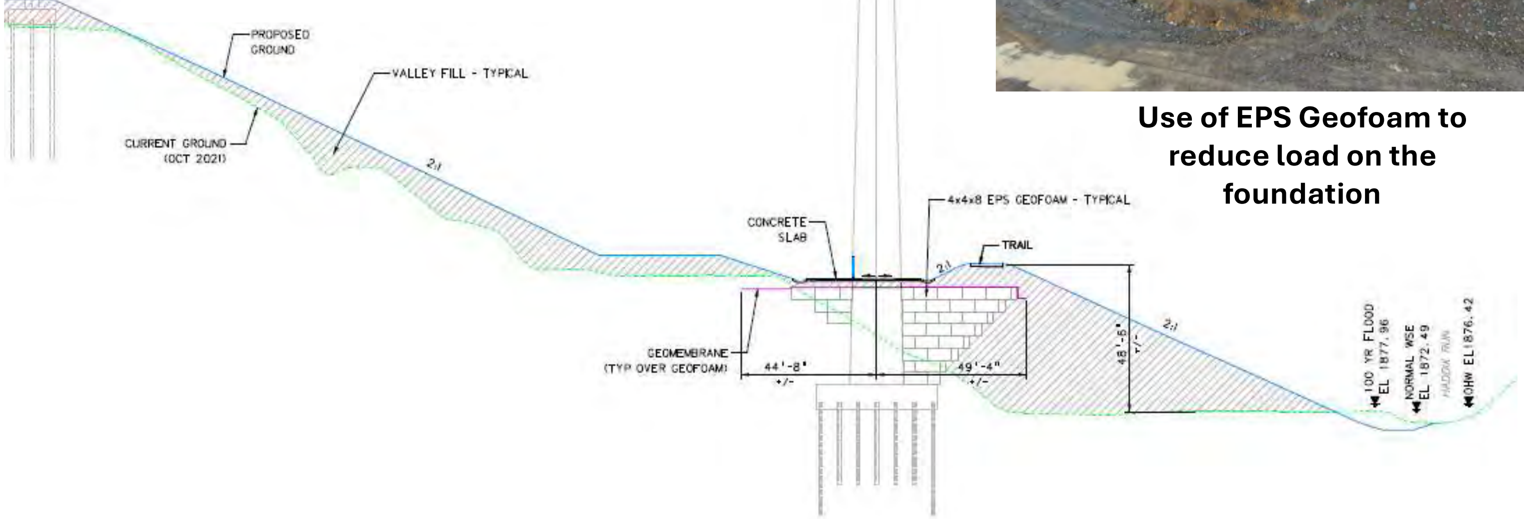


Pier 2 Changes

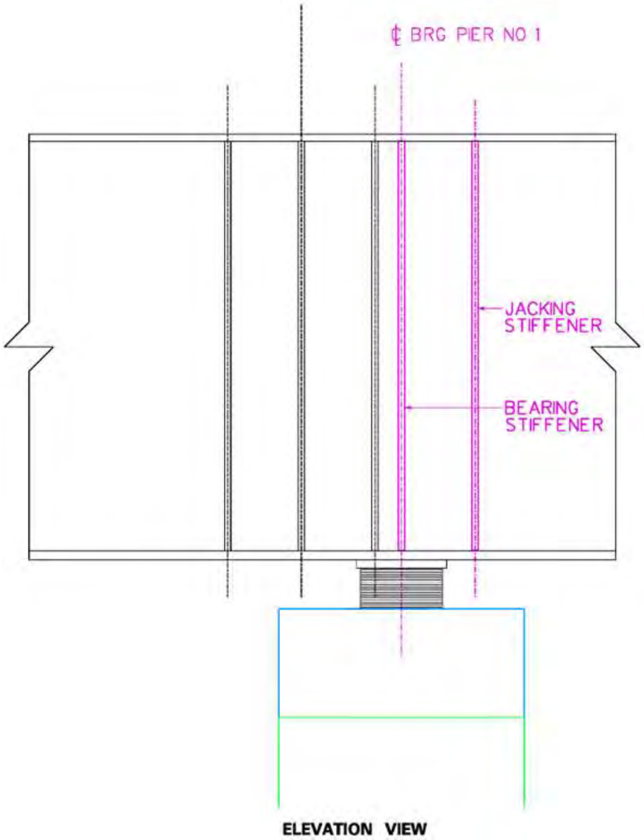


Plaxis 3D Model to evaluate effect on the Pier 2 foundation

Use of EPS Geofoam to reduce load on the foundation



Girder Changes at Pier 1



Added new Jacking and Bearing Stiffeners to the fabricated girders

Conclusions

- Unexpected movements of the slope were caused by unexpected subsurface conditions:
    - Pre-existing slip plane deep within the rock that reduced shear strength on that plane
    - Artesian pore pressures acting on that failure plane
    - Monitoring showed rapid response of the pore pressures to surface water inflows.
- Dewatering stopped further slope movement almost immediately.
- Unloading top of slope and buttressing toe provided additional stabilization for ensuring long term stability.
- Real-time performance monitoring helped inform remediation decisions (response of movements to pore water pressures).
- Performance monitoring proved that the remediation stopped further movements of the slope.

Lessons to apply

- Beware of possibility of weak planes in rock from historical events
- Where slope instability may be a possible failure mode, measure pore pressures in controlling strata with piezometers over time during site investigation to know what to design for.
- When there is uncertainty about the possible performance of the design, measure performance to update predictions of the future performance.
- Include redundancy in the monitoring program because unexpected things happen.
- Dewatering to control high pore pressures can be an effective remedial measure to stabilize an unstable slope.
- Undetected rock defects and high pore water pressures can threaten a project's success through added cost and time to complete.

Acknowledgements

Stephen T. Rumbaugh, PE
Secretary of Transportation
West Virginia Department of Transportation
Division of Highways

James T. Collins, PE
Chief Engineer of Special Programs
West Virginia Department of Transportation
Division of Highways

John Householder
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Heather Woods
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E. L. Robinson Engineering Co.

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Sr. Geotechnical Engineer
Geocomp Inc.

Matt Ham
Instrumentation Data Jockey
Geocomp Inc.

Notes

ORVSS LV

Presentation 3

Effect of Rainfall-Induced Soil Saturation on Formation of Runoff and Slope Failure

Mir Ali Hosseini

Department of Civil and Environmental Engineering,

University of Louisville

Effect of Rainfall-Induced Soil Saturation on Formation of Runoff and Slope Failure

Mir Ali Hosseini[1], Fereydoun Najafian Jazi[2], Md Rojimul Hussain[3], Omid Ghasemi-Fare[4], Thomas D. Rockaway[5]

Abstract

Significant amounts of precipitation remain a primary concern for researchers because of their impact on many natural hazards, such as flooding and slope failure, which can cause extensive damage to infrastructure and houses, along with severe risks to human health. A deeper understanding of the mechanisms that lead to rainfall-induced failures is essential for improving early warning systems, designing effective mitigation measures, and ensuring the long-term resilience of both natural and engineered slopes. Therefore, many studies have been devoted to investigating different factors that contribute to rainfall-induced disasters. Geological and site characteristics play an important role in forming runoff and providing a medium that allows portions of water to infiltrate the shallow, unsaturated part of the subsurface during rainfall, thereby causing the layer to become saturated. The occurrence of runoff and slope failure depends greatly on the soil conditions, as it transitions from unsaturated to saturated with precipitation. This study includes two different models to investigate the formation of a saturated soil layer and its effect on runoff and slope failure.

During rainfall events, the porous structure of the shallow subsurface soil enables some of the rainwater to seep into the pore spaces of the unsaturated soil, reducing surface water buildup. However, run-off is unavoidable if the rainfall continues until the unsaturated zone becomes completely saturated. Therefore, it is important to determine the time needed for unsaturated subsurface soils to reach full saturation during heavy rainfall. This matter is investigated by developing a fully coupled thermo-hydraulic model using a commercially available finite element software, COMSOL Multiphysics. The model solves the mass conservation equation for liquid water and water vapor, while incorporating phase changes between liquid and gaseous water using a non-equilibrium phase change model. Additionally, it considers the advective flow of liquid water and the diffusive movement of water vapor. The thermal energy conservation equation, which accounts for heat transfer through conduction and convection, is coupled with the mass conservation equations while also considering the latent heat of vaporization. Climate conditions

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are taken into account through a complex model that considers the interactions between soil and atmosphere at the surface. This includes factors such as evaporation and condensation, heat exchange, and particularly, rainfall infiltration. The developed model is validated by simulating a non-isothermal experiment from the literature. Several rainfall rates and soil permeabilities are used in the validated model to study how saturation time changes under different conditions. Results are interpreted in terms of temperature and moisture content variations in the unsaturated layer, as well as saturation time under different scenarios.

Additionally, a physical model is constructed in a transparent soil box to investigate the influence of rainfall on the stability of slopes. The model is equipped with various sensors and a camera to monitor the soil moisture content, run-off, pore water pressure, and the displacements resulting from failure. Artificial rainfall with a uniform intensity of 75 mm/h was applied to the model to represent heavy rainfall at a laboratory scale. Two tests were conducted with different soil configurations: 1) a mixture of 95% sand and 5% clay (kaolinite) with a dry density of 1.4 g/cm^3 and an inclination of 45 degrees, and 2) pure sand with a dry density of 1.6 g/cm^3 and an inclination of 30 degrees. The model was prepared using the wet tamping method by mixing dry soil with a density of 1.4 g/cm^3 for test 1 and 1.6g/cm^3 for test 2 with 5% water and constructing it layer by layer, each with a 5.08 cm (2 inch) thickness. Both tests showed that rainfall infiltration quickly raised the water content, eventually causing the slope to become saturated. However, slope failure occured only in Test 2, about 18 to 20 minutes after the rain started. The failure began upslope and involved only the shallow layers. The soil flow continued until it reached a stable condition, resulting in an accumulation of soil at the toe of the slope. In contrast, despite having a steeper slope and longer rainfall duration, the mixed-soil configuration in Test 1 remained stable. Pore water pressure trends supported these findings. The higher saturation zone led to a loss of shear strength in a broader soil mass in Test 2, triggering failure. On the other hand, adding kaolinite to Test 1 helped maintain matric suction and limited the development of positive pore pressure, thereby enhancing slope stability.

Key Words: Rainfall, Flood, Slope Stability, Runoff, Physical Model, Numerical Model

Notes

ORVSS LV

Presentation 4

From Failure to Repair – A Success Story for the City of Marion, Kentucky

Ben Webster, P.E., P.M.P.

Schnabel Engineering

From Failure to Repair

A Success Story for the City of Marion, Kentucky[1]

ABSTRACT

Lake George Dam is a 28-foot-high, 700-foot-long earth fill dam that impounds a tributary to Crooked Creek and forms the secondary (and largest) source of raw water storage for the City of Marion (City) Water District, which serves approximately 17,000 people.

The dam was originally constructed in 1954. A dam safety incident occurred in April 2022 which involved a sinkhole on the downstream face of the dam and seepage boils near the toe. During the incident, the left abutment was intentionally breached to lower the water level and reduce the likelihood of catastrophic release of the reservoir. A state of emergency was declared, and the national guard provided drinking water to the City in the months following the incident.

Due to deteriorating conditions throughout 2022 and into 2023, and the suspected collapse of a 12-inch cast iron pipe (CIP) running along the base of the embankment, a fast-tracked design for Interim Risk Reduction Measures (IRRM) was completed in late 2023.

Construction of the IRRM was completed in December 2024, and the City is currently working to secure a new long-term water supply approach while operating Lake George at a lowered lake level during the interim period.

This is a story of success highlighting how local and state government entities collaborated during and after a dam safety incident to arrive at an interim solution to maintain water storage and provide time for identification and implementation of a long-term water supply strategy.

BACKGROUND AND SITE HISTORY

Lake George Dam (also referred to as Marion City Dam) was designed by Paul M. Jones and Associates (PMJ), Civil and Mining Engineers in 1954. The dam impounds a tributary to Crooked Creek to form Lake George, which serves as the City's largest raw water storage reservoir (operating in tandem with Old City Lake immediately downstream). The lake also serves as a valued recreational amenity for residents of the City and Crittenden County. Available original design and construction information is limited to excerpts from the original design plans. Figure 1 shows a plan and cross-section view of the dam along an outlet pipe

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system alignment. The original drawing excerpts also include details related to the outlet system, and cross-sections with associated estimated construction quantities. Other design information in support of the original 1954 design plans was not found.

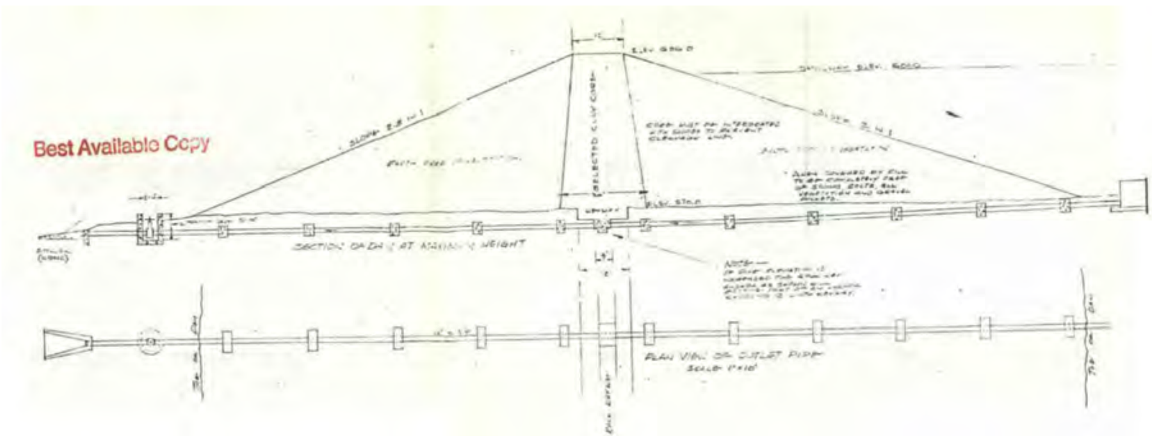

Figure 1. Plan and Cross-Section from 1954 Original Design Drawings

Available records indicate a total of 19 inspections of the dam were completed from 1979 to 2022. The first recorded inspection was completed by G. Reynolds Watkins/ATEC Associates in June of 1979 on behalf of the U.S. Army Corps of Engineers, Louisville District and summarized in a Phase I Inspection Report in accordance with the National Program of Inspection of Non-Federal Dams. Except for the original PMJ design plans, no information between original construction in 1954 to the 1979 inspection was found. Key findings from the 1979 inspection include localized seeps near the downstream toe, shallow sloughing of the downstream slope, insufficient lake drawdown capabilities, and inadequate spillway capacity. The Phase I Inspection Report recommended a Phase II study to evaluate slope stability of the dam under steady state seepage based on a geotechnical investigation. Other recommendations included assessing potential vulnerability of the dam to seismic events, implementing corrective measures to control seepage along the toe, assessing feasibility of installing a principal spillway and adequate drawdown facilities, and assessing feasibility of enlarging the emergency spillway to pass the required spillway design flood (the flood resulting from half of the Probable Maximum Precipitation).

The remaining 18 inspections between 1979 and 2022 were conducted by the Kentucky Division of Water (DOW) Dam Safety Section. Using the initial 1979 report as a baseline, the overall dam characteristics were routinely monitored for noticeable changes in structure and function. Except for two inspection reports in 1995 and 2010, all the inspection reports indicated the presence of wet ground or seepage near or at the downstream toe, which demonstrates documented recurring observations spanning back to at least 1979. Records also indicate that chemical testing was performed on two water samples obtained from the lake and dam in 1981. One sample was taken from the source of downstream seepage, and another was taken from the lake for comparison. The chemical analysis results varied greatly in composition such that the environmental consultant believed the water was coming from different sources. Later correspondence from DOW interpreted these findings to mean the seepage within the wet area was not coming from the reservoir itself, but rather an underground source. No further information was found on the testing or associated conclusions.

The original design drawings show a 12-inch CIP installed at the base of the embankment with a submerged inlet, downstream valve, and outlet end discharging into the downstream creek. However, the 1979 Phase 1 Inspection Report showed a vertical perforated riser pipe extension at the intake location, and a pipeline connected to the outlet to route water directly to the treatment plant approximately one half of a mile downstream. It is assumed modifications to add the perforated riser and downstream water supply pipeline were performed between original construction and 1979. A conceptualized cross-section of the dam along the outlet pipe system alignment (from the 1979 Phase 1 Inspection Report) is included as Figure 2.

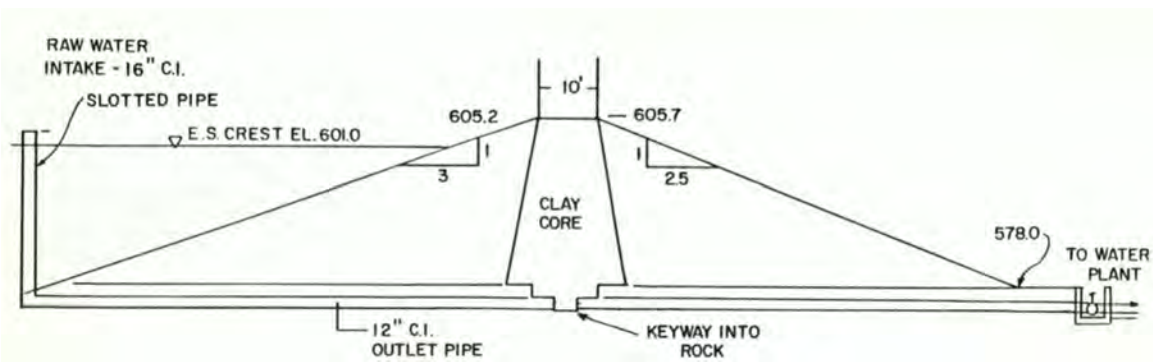

Figure 2. Cross-Section along Outlet Pipe from 1979 Phase 1 Inspection Report

A set of design drawings by Carter Dixon Partners Engineers Inc. (CDP) dated February 2006 indicate that modifications were made around that time to replace the downstream water supply pipeline running to the plant. As part of that project, new connections, valves, pipes, and a pigging port were installed near the downstream toe of the dam. Raw water from the original 12-inch CIP running along the base of the dam was rerouted through a series of new valves and pipes.

While not documented on the 2006 drawings, the intake structure on the 12-inch CIP was also reportedly modified around this time. The previously mentioned vertical perforated pipe was replaced with a swiveling intake pipe attached to a floating platform within the lake.

It is important to note that throughout the history of the project, the 12-inch CIP remained charged under head pressure from the reservoir.

APRIL 2022 DAM SAFETY INCIDENT

Although seepage near the downstream toe had been observed for decades, it appeared to remain largely consistent until April 2022. On April 26, 2022, a local fisherman reported leaking conditions near the toe of the dam to City officials. On the morning of April 27, 2022, the City reported an apparent dam failure in progress to DOW. DOW engineers mobilized later that day to assess site conditions. Seepage boils were observed at the ground surface near the downstream toe. A defect in the 12-inch CIP was suspected to be a contributing cause of the leakage. DOW recommended that the intake associated with the 12-inch CIP be raised above pool level to hydraulically disconnect the pipe from the reservoir. The City also advised City personnel to monitor the boils closely for any changes in flow or color and to alert DOW of any changes.

On April 29, 2022, the seepage became more turbid (see Figure 3), followed by development of a sinkhole mid-slope along the downstream face (see Figure 4). The sinkhole was above and near the alignment of the 12-inch CIP. Emergency pumping operations were performed to draw the reservoir level down as the sinkhole continued to increase in size (see Figure 5). Drawdown efforts made little impact to the reservoir level, and the City initiated a breach of the dam at the left abutment in late afternoon on April 30, 2022 (see Figure 6). Detailed documentation of the events leading up to the controlled breach are included in DOW Certification of Inspection (COI) reports dated April 27 to May 4, 2022.

Figure 3. Turbid Boils and Seepage near Toe of Dam

Figure 4. Sinkhole Immediately after Initial Development

Figure 5. Enlarged Sinkhole after 24 Hours

Figure 6. Breach Channel at Left Abutment

On May 4, 2022, the City performed an excavation adjacent to the sinkhole on the downstream face of the dam. The excavation was made with an excavator and measured approximately 100 feet by 100 feet in plan view dimension, with a maximum depth of approximately 15 feet. The excavation was reportedly performed to expose the 12-inch cast-iron pipe and/or further breach the dam.

A summary of the April/May 2022 incident, as recorded by DOW, is tabulated below in Table 1.

Table 1. Approximate Timeline of Sinkhole Development and Breach

Date	Time (CDT)	Recorded Observations and Actions
April 27, 2022	9:26 AM	DOW received initial reports of a potential dam failure
	4:00 PM	DOW staff arrived on site to assess
April 29, 2022	12:30 PM	Seepage along downstream toe became more turbid
	7:15 PM	Development of sinkhole mid-slope of downstream face
	8:38 PM	Sinkhole developed to 6 feet in diameter
	11:50 PM	Reservoir drawdown efforts initiated
April 30, 2022	11:45 AM	Recorded reservoir drop of 3 inches
		Sinkhole expanded to approximately 15 feet x 10 feet in plan
	1:00 PM	Drawdown efforts ceased
	3:47 PM	Controlled breach initiated at left abutment
	4:45 PM	DOW staff departed site
May 4, 2022	NA	Excavation by City along downstream dam face (adjacent to sinkhole)

Following the incident, DOW issued a Directive Letter on May 9, 2022, and a Notice of Violation (NOV) on May 24, 2022. Two violations were cited in the NOV, as follows.

- The City of Marion has failed to comply with KRS 151.250 by failing to apply for and receive a permit, issued by the Energy and Environment Cabinet, before commencing the excavation of the downstream slope.
- The City of Marion has failed to comply with 401 KAR 4:030 Section 10 by failing to conduct and submit to the Energy and Environment Cabinet the required geotechnical study and slope stability analysis for this structure.

Figure 7 is an aerial image from June 2022 showing the breach at the left abutment and the excavation of the downstream slope near the location of the sinkhole. Note that the reservoir level at the time was well below the breach invert elevation, presumably due to continued draining of water through the 12-inch CIP and evaporation due to dry hydrologic conditions following the initial Dam Safety incident.

Figure 7. Post Failure Aerial from June 2022

POST-INCIDENT STUDIES AND EFFORTS

On June 18, 2022, Governor Andy Beshear declared a state of emergency via executive order due to the water shortage that resulted from the April 2022 dam safety incident. In the months that followed, the National Guard hauled raw water to the downstream reservoir (Old City Lake) and provided bottled drinking water to City residents. Early engineering efforts related to assessment and repair planning were performed in late 2022 by Bacon Farmer Workman and consisted of geotechnical exploration and conceptual repair design. In March 2023, Schnabel was retained by the City to perform an assessment of existing dam conditions, which were largely unchanged from that shown in Figure 7, except that the Lake George Reservoir level had reached the invert elevation of the left abutment breach and was flowing through the channel. As a function of the higher lake level, water was observed flowing from the excavated area within the downstream slope, as shown in Figure 8.

Figure 8. Seepage in Excavated Area (March 2023)

In April 2023, the City reported that the seepage flowrate had noticeably increased as compared to recent observations, and the lake level had dropped 3-5 feet over the preceding few days. In an attempt to reduce or stop flow into the 12-inch CIP, the City contracted Marine Solutions, Inc. (MSI) to install a steel plate over the submerged open intake. The plate was installed over the intake by lowering it with supports from a jonboat and relying on suction to adhere it to the intake flange. Upon installation, it was held over the intake by the water pressure head pushing it against the intake flange (i.e., it was not fixed in place by bolting, welding, or other means). The City indicated that flow into the pipe was reduced by an estimated 95% as a result of the plate installation.

With the pipe now largely dewatered, a camera inspection was performed to observe the interior of the 12-inch CIP On May 18, 2023. The inspection revealed a crack within the sidewall of the pipe at a location corresponding closely with the sinkhole. Apparent root growth was observed within the crack, indicating that it had likely existed for many years. A screenshot showing a portion of the crack is included as Figure 9.

Figure 9. Crack in 12-inch CIP Interior Sidewall

Based on the review of available historical information, and the studies and efforts in the year following the dam safety incident, it is theorized that the seepage boils and sinkhole development in April 2022 were likely associated with gradual seepage of water from the pressurized 12-inch CIP over time. The water slowly eroded the embankment material from around the pipe and mobilized it downstream, likely along the exterior of the conduit. Over time, the internal erosion led to stoping (i.e., upward progressive collapse of material into a subterranean void) within the embankment and resulted in sinkhole development. The persistent seepage near the toe of the dam that had been observed for decades was likely a sign of the internal erosion process that occurred over decades.

INTERIM RISK REDUCTION MEASURES PROJECT

Due to progressing deterioration of the dam, and continued concern for dam safety and water supply security, the City elected to move forward with a fast-tracked Interim Risk Reduction Measures (IRRM) project. From June 2023 to September 2023, Schnabel developed an IRRM design consisting of the following key elements.

- Construction of a rock berm within the lake to provide water detainment to mitigate against catastrophic release of the reservoir associated with failure of the dam at the compromised area.
- Backfilling the compromised area within the middle of the embankment with filter layers and riprap and installing a sand trench filter at the embankment toe.
- Fully grouting and abandoning the damaged 12-inch CIP.
- Installation of a dual siphon system. A 20-inch (OD) self-priming siphon to serve as principal pool control to maintain normal pool at a level 10-feet below the former normal pool. An additional 6-inch (OD) siphon to be used in times of low flow to convey water to the downstream creek, and ultimately to Old City Lake for withdrawal at the City's water treatment plant.
- Armoring and raising the controlling elevation of the left abutment breach and downstream channel. The controlling elevation of the spillway, which acts as an auxiliary spillway under the IRRM design, is set at the 25-year storm recurrence interval.

MSI began implementing the IRRM project in December 2023, under funding from Kentucky Emergency Management. Due to wet weather conditions and heavy construction traffic on an otherwise light-duty access road, early efforts consisted of significant road improvements to accommodate construction loading. Construction of the major project elements started in earest in March 2024 with the following general sequence of construction.

1. Construction of the Rock Berm
2. Abandonment of the 12-inch CIP via fully grouting the interior of the pipe under in-the-wet conditions.
3. Backfilling the excavated section in the downstream slope, and installing a trench filter along the downstream toe of the dam.
4. Installation of the siphon system, including an outlet control structure, plunge pool, thrust blocks, concrete cradle, and a diaphragm filter.
5. Raising the left abutment breach controlling elevation with grouted riprap, and lining the outlet channel with riprap.

Substantial Completion was achieved on November 1, 2024, and Final Completion was achieveed on December 23, 2024. Select photos of the completed project are shown in Figures 10 and 11.

Figure 10. Completed IRRM Project Looking Downstream

Figure 11. Completed IRRM Project Looking Upstream

CONCLUSION AND PATH FORWARD

Dam engineering best practices have progressed significantly since the Lake George Dam was designed and constructed. Modern-day dam design would not include several features that were characteristic of dam building in the 1950s, such as cast-iron pipes within the embankment, anti-seep collars, and unfiltered seepage exits. Lake George Dam also had a long history of persistent seepage at the toe of the dam. This case study highlights the importance of investigation into observed dam safety issues, especially those relating to internal erosion. Internal erosion failure modes often take years or decades to progress to failure, but intervention and correction can be achieved if defects are effectively investigated and diagnosed early.

The Lake George Dam incident also highlights the success of state and local partners working together to keep human health and safety paramount. Collaboration between government officials and other stakeholders led to successful implementation of the IRRM, which now allows the City time to plan for a long-term water supply approach.

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Notes

ORVSS LV

Presentation 5, Keynote

Challenges in Addressing Karst on Civil Works

Hugo Aparacio, P.E.

Stantec

CHALLENGES IN ADDRESSING KARST IN CIVIL WORKS

Hugo Aparacio, PE

Addressing the impact of karst features on existing or proposed civil work projects remains challenging due to the complex, variable and unpredictable nature of these features. The decision to build on top or even near karst features is driven primarily by the cost of the measures needed to eliminate the risk of their potential impact on civil works or, in a few cases, by constraints related to the project site location. While advances in methods to explore karst conditions have progressed slowly, methods to construct foundations or remediate existing foundations affected by karst conditions have improved significantly. Examples of projects involving karst conditions of different complexity are cited and used to describe approaches and methods implemented to address the impact of karst.

CHALLENGES IN ADDRESSING KARST IN CIVIL WORKS

PRESENTATION OUTLINE

- A Karst Definition
- Karst Presence in Kentucky
- Karst Presence in Fayette County, KY
- Learning to Address the Impact of Karst on Civil Projects
  - Structural Support
  - Groundwater Protection
- Local City Ordinance
- Methods to Explore Karst Features

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PRESENTATION OUTLINE

- Typical Treatment of Common Karst Features in Central Kentucky for Non-Buildable Case
- University of Kentucky W.T. Young Library
- Crystal Springs Cave
- Wolf Creek Dam
- Boone Dam
- To Build or not on Top of Karst Features
- Summary

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A KARST DEFINITION

- Karst is a term applied to features encountered in terrain underlain by limestone or dolomite, where the rock has dissolved partially after coming in contact with acidic water.
- The dissolved rock results in sinkholes, sinking streams, closed ground depressions, caves, crevices and springs.
- All these features form over millions of years and are shaped erratically.

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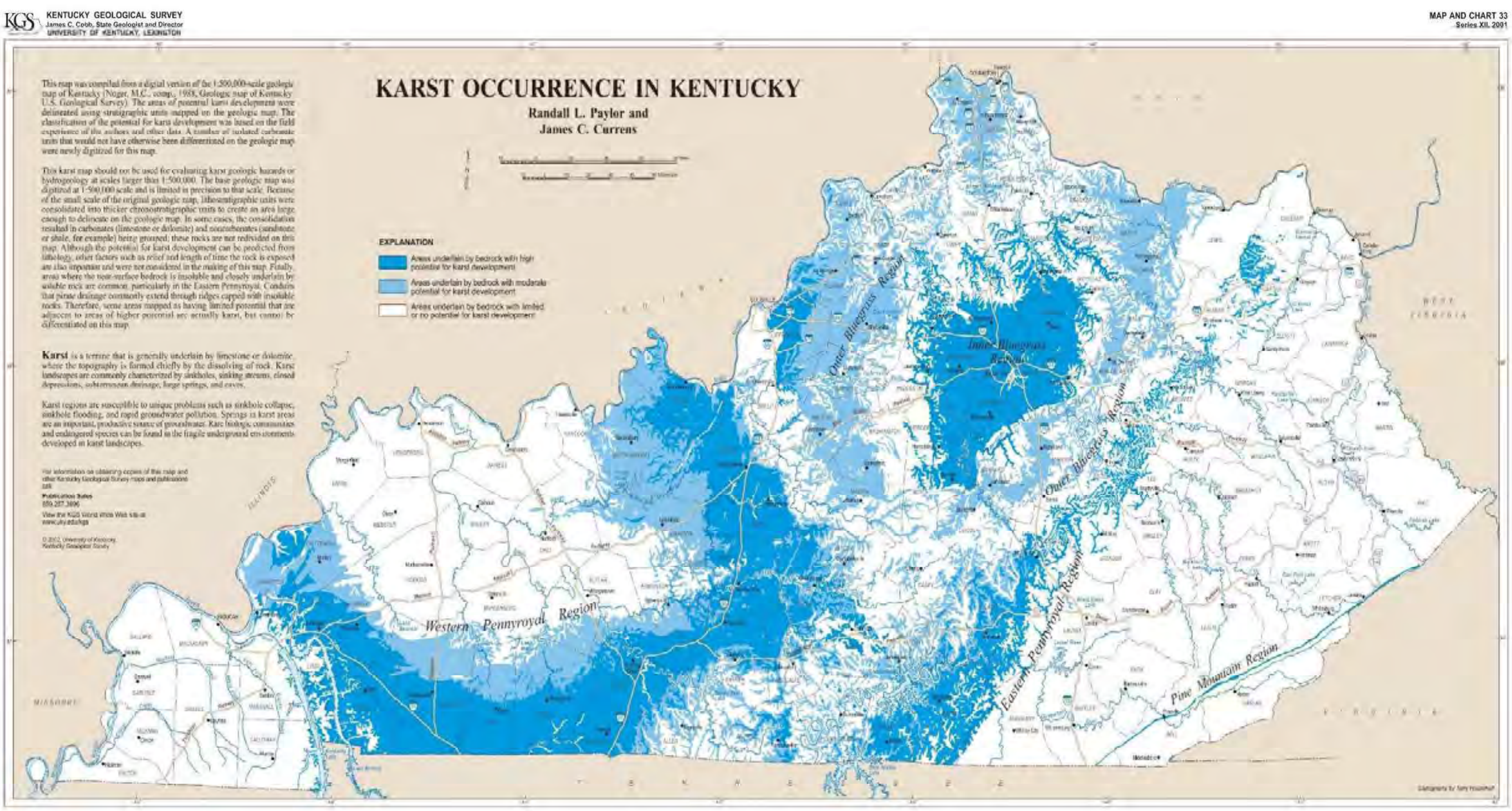
KARST PRESENCE IN KENTUCKY



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KARST PRESENCE IN KENTUCKY

Karst Occurrence in Kentucky

- The map shows areas of high, moderate and limited (or no) potential for Karst development.
- The Inner Bluegrass (**Lexington**) and Western Pennyroyal (**Mammoth Cave**) Regions are shown as high potential, while **Wolf Creek Dam** appears in a region of moderate potential.
- Regardless of what the map shows, the actual impact of the Karst presence is entirely dependent on the site geology and the type of Civil Work of the project.
- The varying extent and uncertain nature of Karst features present significant challenges in interpreting and addressing their impact on Civil Works.

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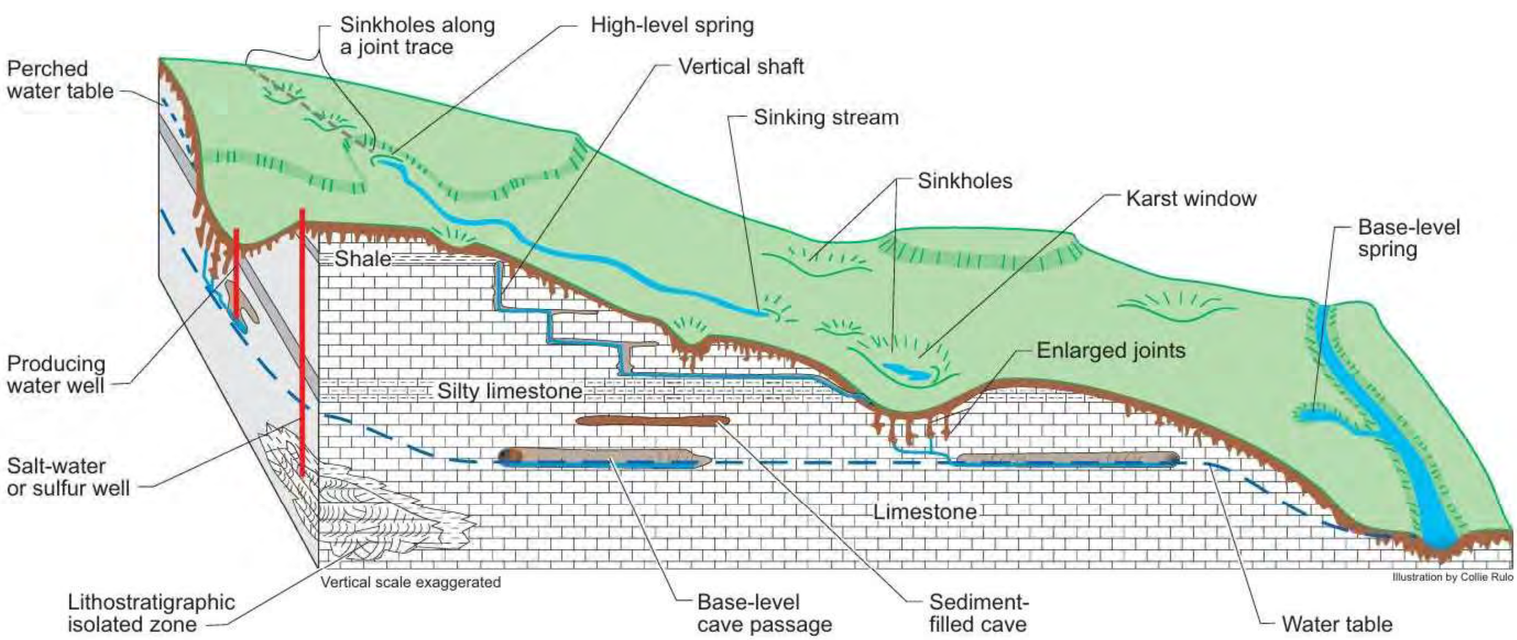
KARST PRESENCE IN THE BLUEGRASS REGION (3)



KARST PRESENCE IN THE BLUEGRASS REGION

- Karst Occurrence in the Central Kentucky Area
- The block diagram shows all the karst features encountered in the Bluegrass Region. It is missing the fact that there are also open caves in the region, as discussed by an example presented later.
- Most of the karst features encountered in the Fayette County area consist of open sinkholes, ground closed depressions and springs.
- Sinkhole can be found on flat ground or near the center of ground depressions. Closed depressions are usually dry but can retain surface water during periods of heavy rain. Springs don’t dry often.
- The top of bedrock is relatively shallow, very seldom reaching 10 or more feet below the ground surface. As discussed next, shallow bedrock surface facilitates both the exploration and treatment of karst features.

LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS (4)

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LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS

Structural Support

- On my first project involving karst features I was asked to review a small flat area on a proposed subdivision where the top of rock was at the ground surface. There were two 15-foot (+/-) long parallel cracks, no more 5 inches wide at the top. The depth of the crack voids, measured using tape measure, was no more than 15 inches.
- With the purpose of exploring the extent of the voids, we advanced 3 rock core borings next to the cracks to a depth of 15 feet each. We encountered no voids and had 100 percent rock core recovery.

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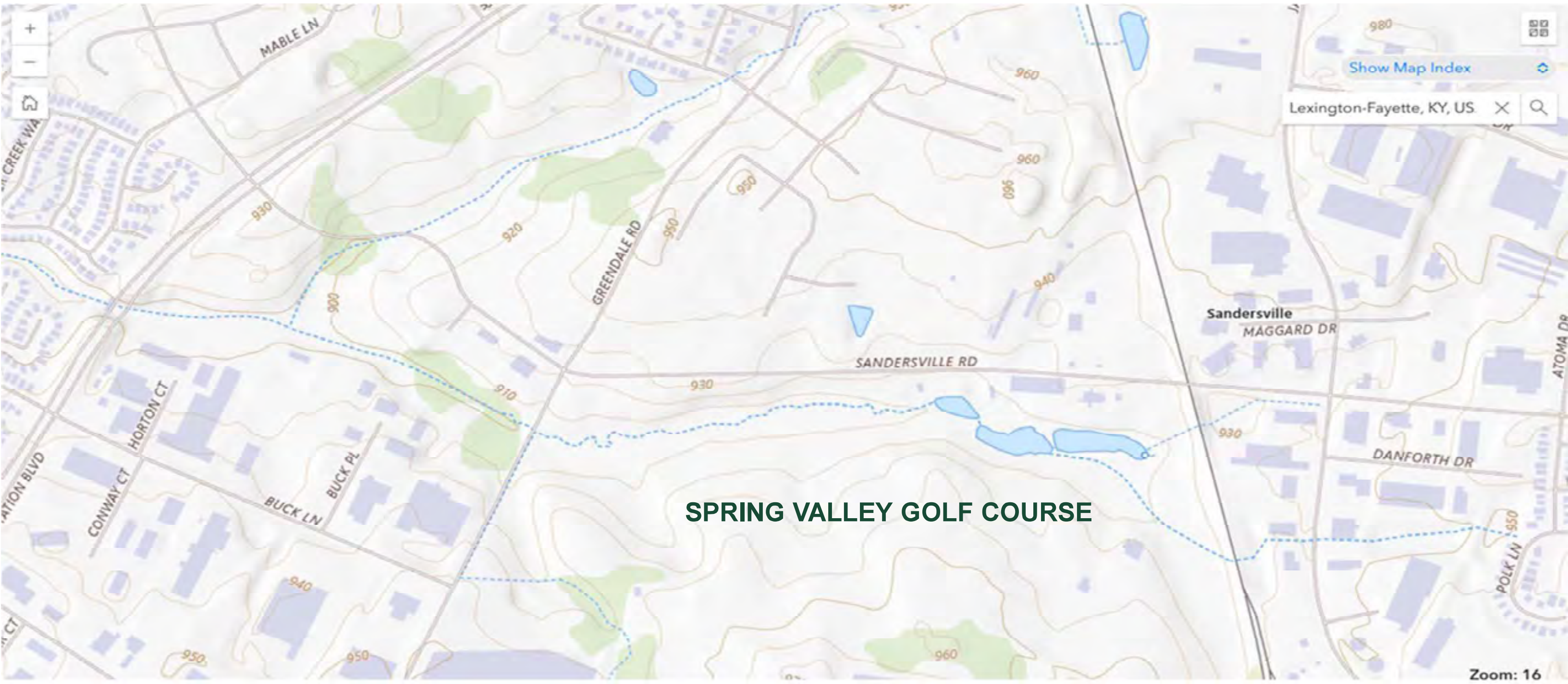
LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS

Karst Aquifer Protection

- On my next project involving karst related issues I was asked to review conditions of a sinking stream near a golf course (Spring Valley Golf Club – now closed). The stream left the course, entered a road culvert and drained into swampy area where it sunk. This sinking stream reappeared as a spring a few hundred feet away at a roadside ditch.
- Also learned that several springs within the golf curse fed two large ponds that formed part of the stream. The ponds were used to water the course year-round.
- Later, a water main was constructed within the golf course traversing parallel to the stream, intercepting the spring flow. The pipe trenches extended 3 to 5 feet below the top of rock and intercepted karst voids.
- We recommended measures be added to the backfill of the trenches to ensure that groundwater continue feed the springs.
- Around this time, we learned that the city of Georgetown got its drinking water from Royal Springs, which is fed by sinkholes and other karst features.

LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS



LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS

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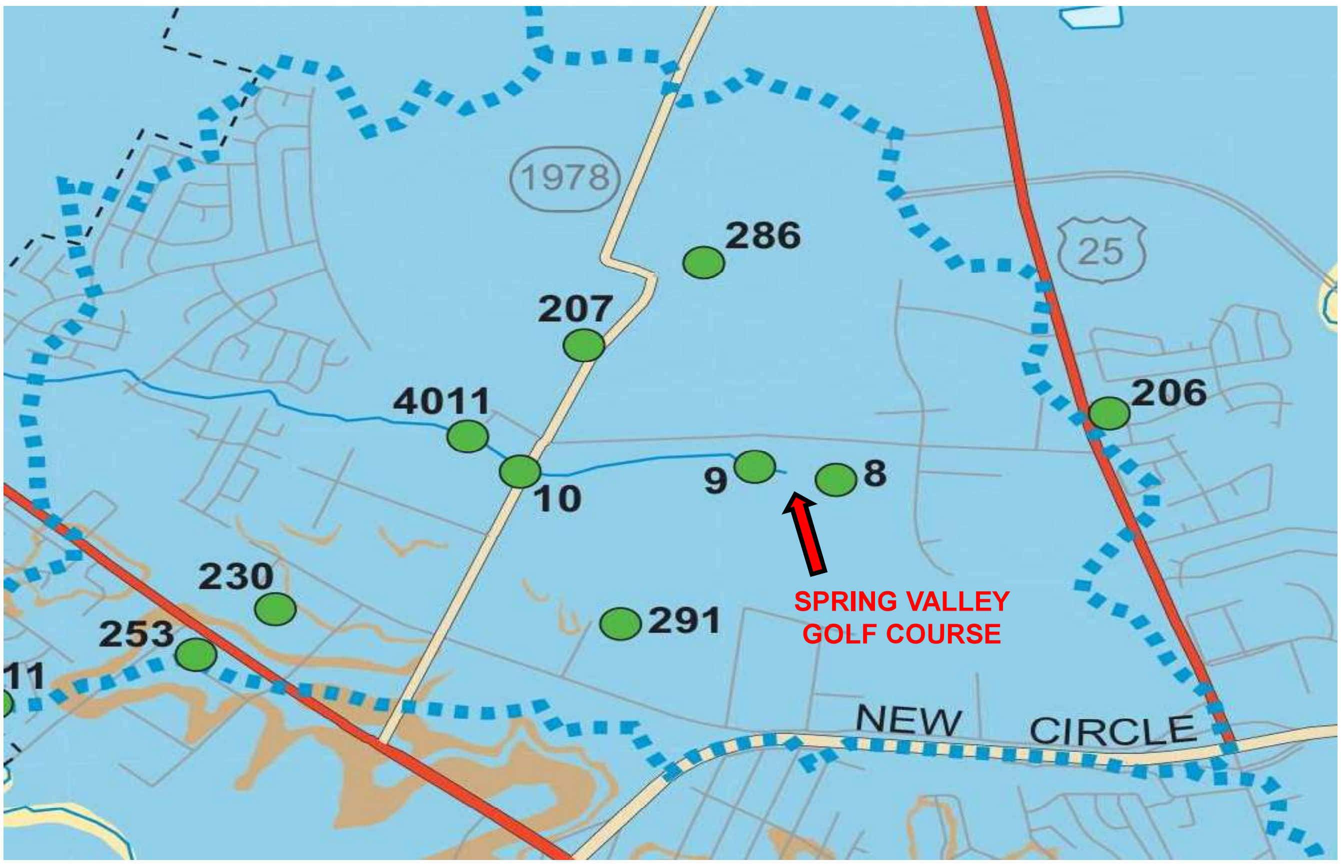
LEARNING TO ADDRESS THE IMPACT OF KARST ON CIVIL PROJECTS



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LFUCG SINKHOLE ORDINANCE
Adopted: 1985

Definition of a Sinkhole

- Any closed depression formed by removal (typical underground) of water, surficial soil, rock, or other material.

Definition of Karst

- A topography formed over limestone, dolomite, or gypsum and characterized by sinkholes, caves, and underground drainage.
- LFUCG Plan Requirements
- Each sinkhole feature within the development shall be shown.
- Sinkhole-Related Non-Buildable Areas Shall Be Constructed Based On
- Site topography, geology, soil conditions, site history and stormwater analysis (No buildings, parking areas, or other structures)
- Division of Engineering may require recommendations from a consulting engineer and hydrologist.

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LFUCG SINKHOLE ORDINANCE
Adopted: 1985 (Continued)

Development in Sinkhole Drainage Areas

- Development may occur provided surface drainage is directed away from the sinkholes.
- Sinkholes can be used to meet the required open/green space requirements.

Sinkhole Surface Drainage Analyses

Alternative 1

- Sinkhole can be used for surface runoff provided that any increase in the quantity of surface runoff will not increase the potential for flooding

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LFUCG SINKHOLE ORDINANCE
Adopted: 1985 (Continued)

Sinkhole Surface Drainage Analyses (Cont'd)

Alternative 2

- Sinkholes can be used for surface drainage provided the following conditions are met
  1. Runoff completely retained/detained in retention/detention basin.
  2. Divert enough of the sinkhole drainage such that the original total quantity of the runoff into the sinkhole does not increase.
  3. Outlet flow and quantity of the sinkhole shall not be greater than that which existed before development

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LFUCG SINKHOLE ORDINANCE

Adopted: 1985 (Continued)

Filling in Sinkhole and Drainage Areas

- Development may involve some filling. However, no building with soil-bearing foundations shall be permitted.

Required Plan Notes

- No guarantee that future sinkhole problems will not occur
- Non-Buildable areas are unsuitable for buildings, parking areas and any other structures
- Sinkholes or restricted fill areas are unsuitable for soil bearing foundation systems and any approved structures must be founded on solid rock
- All basement or fire floor elevations shall be 1 foot above the 100 year six-hour storm.

METHODS TO EXPLORE KARST FEATURES

- There are no effective methods to explore the subsurface conditions of karst features
- In cases where top of bedrock is relatively shallow, the characteristics of karst features at or above the top of bedrock can be explored using mechanical excavation methods. Examples of the use of these methods are provided below.
- Geophysical methods such as electrical resistivity, electromagnetism, magnetic survey and induced polarization have limited use in exploring karst features. The epikarst (shallow cavities, conduits, and enlarged fractures) can be mapped using Ground-Penetrating Radar (GPR) to a certain extent. GPR can be used to determine the presence of shallow voids.
- Drilling techniques such as rock coring can be used to obtain complementary information. An example is provided later

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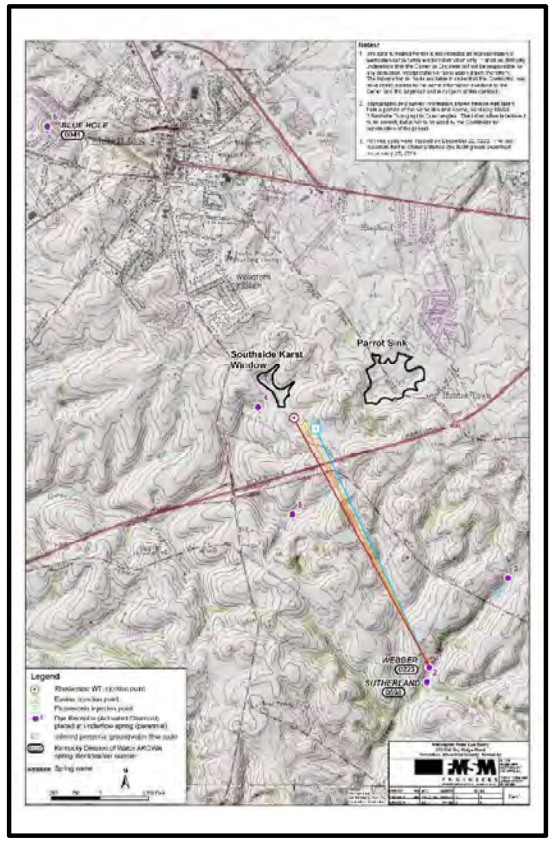
EXPLORATION OF COMMON KARST FEATURES IN CENTRAL KENTUCKY FOR NON-BILDABLE CASE

WATER PURGING AND DYE TRACING/TESTING



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TYPICAL TREATMENT OF COMMON KARST FEATURES IN CENTRAL KENTUCKY FOR NON-BUILDABLE CASE

The methods described in the next slides to treat sinkhole were selected based on the following assumptions:

- The sinkhole area (sinkhole throat or sinkhole depression) is to be stabilized.
- Water must continue flowing into the bedrock opening after treatment

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TYPICAL TREATMENT OF COMMON KARST FEATURES IN CENTRAL KENTUCKY FOR NON-BUILDABLE CASE

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TYPICAL TREATMENT OF COMMON KARST FEATURES IN CENTRAL KENTUCKY FOR NON-BUILDABLE CASE

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TYPICAL TREATMENT OF COMMON KARST FEATURES IN CENTRAL KENTUCKY FOR NON-BUILDABLE CASE [9]

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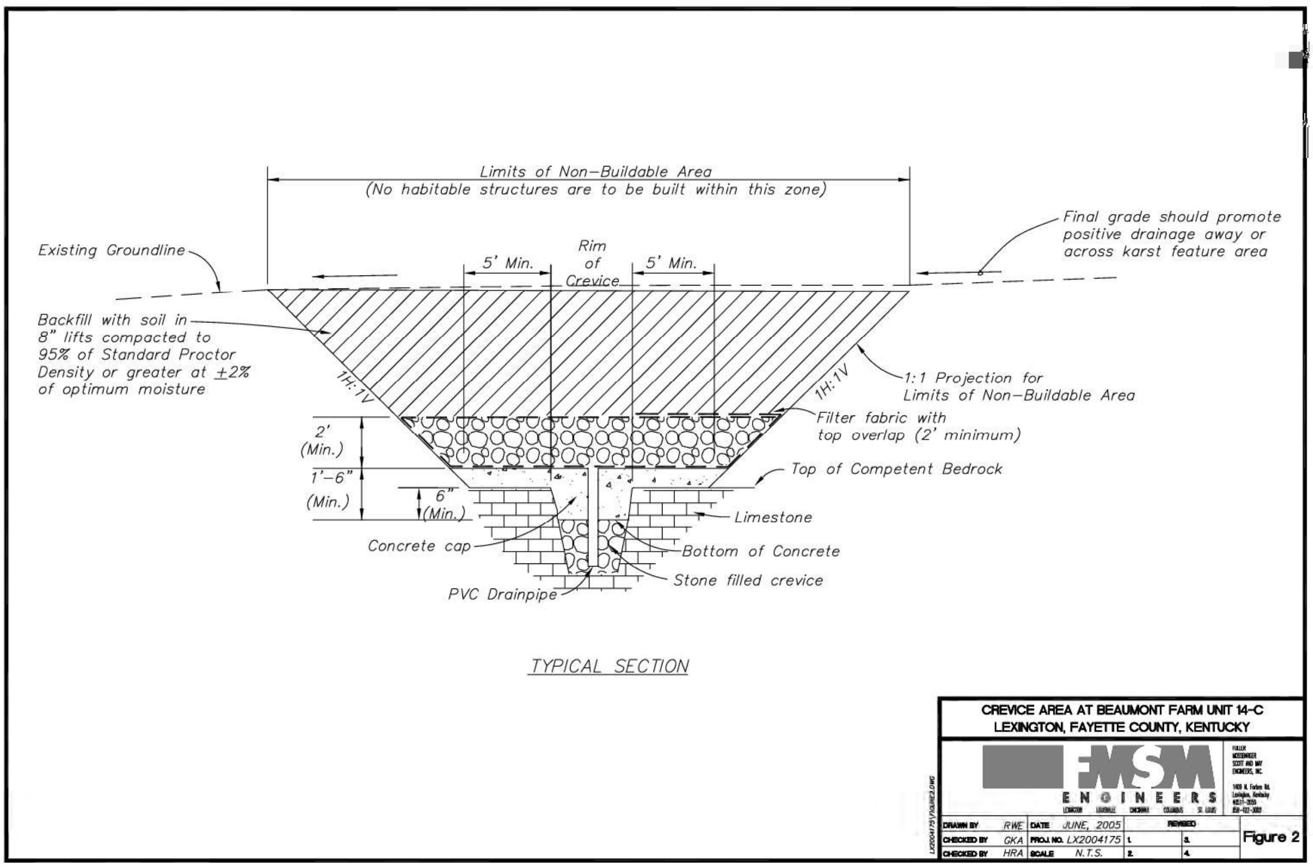
SINKHOLE AREA TREATMENT (TYPICAL SECTION) (9)



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UNIVERSITY OF KENTUCKY W.T. YOUNG LIBRARY

- This was a project to replace the Margaret I. King Library, the main university library within the main campus. Construction started in 1994, and the library opened in 1998.
- Central site locations within campus for such a large facility were scarce or non-existent. The university chose an elevated site located between two large sinkhole depressions.
- One of these depressions flooded during significant rain events, but the sinkhole throats were not visible on the surface.
- FMSM performed a geotechnical exploration, found up to 30-foot-thick voids and other minor solution features within the upper zone of the bedrock, as well as competent bedrock between and below the voids.
- FMSM recommended the use of caissons for the foundation system. Over 200 caissons were installed for this purpose.
- The recommendations included extending each caisson to a certain depth interval of competent bedrock, as well as casing the concrete caissons where voids were encountered.

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W.T. YOUNG LIBRARY

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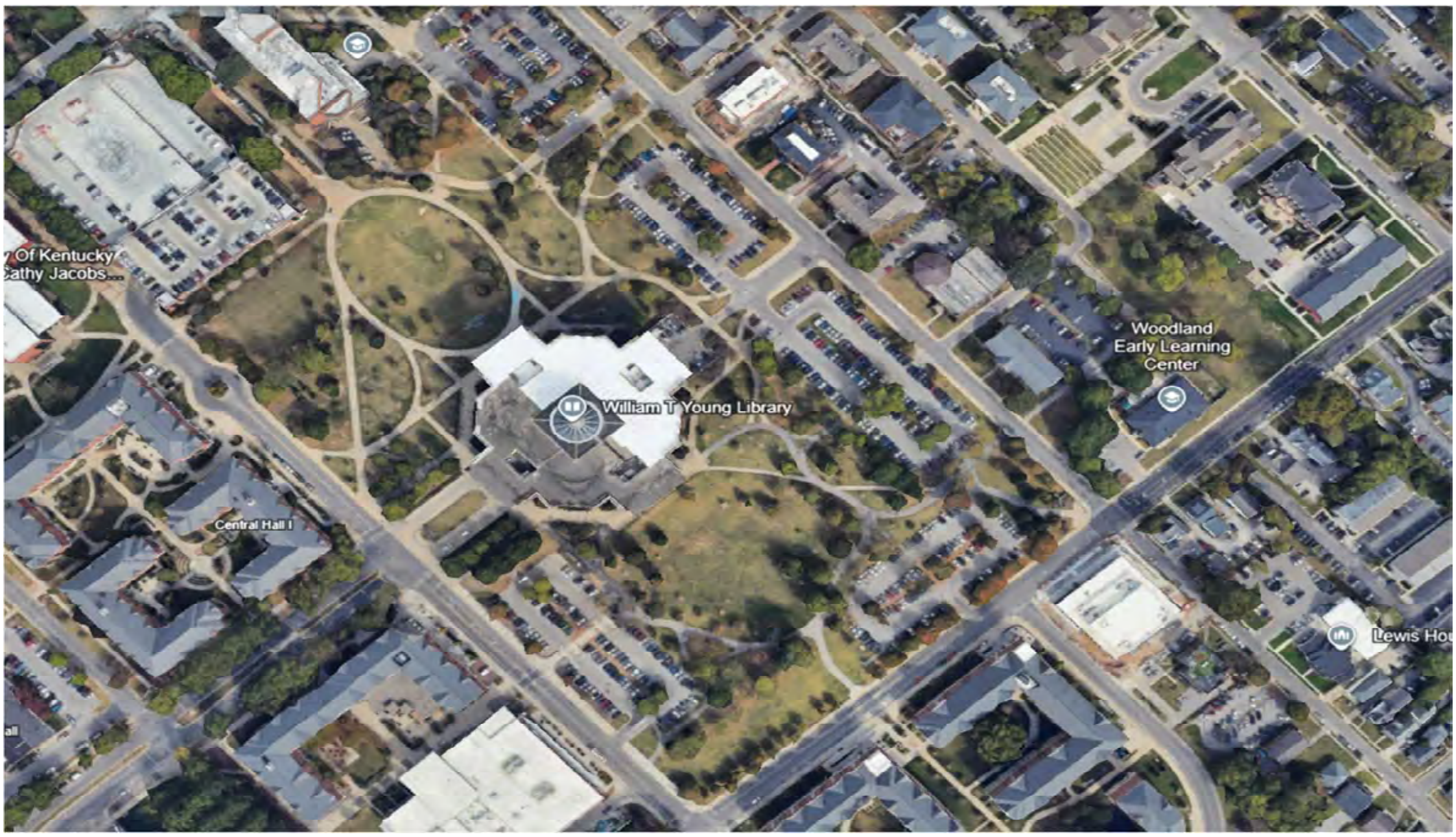

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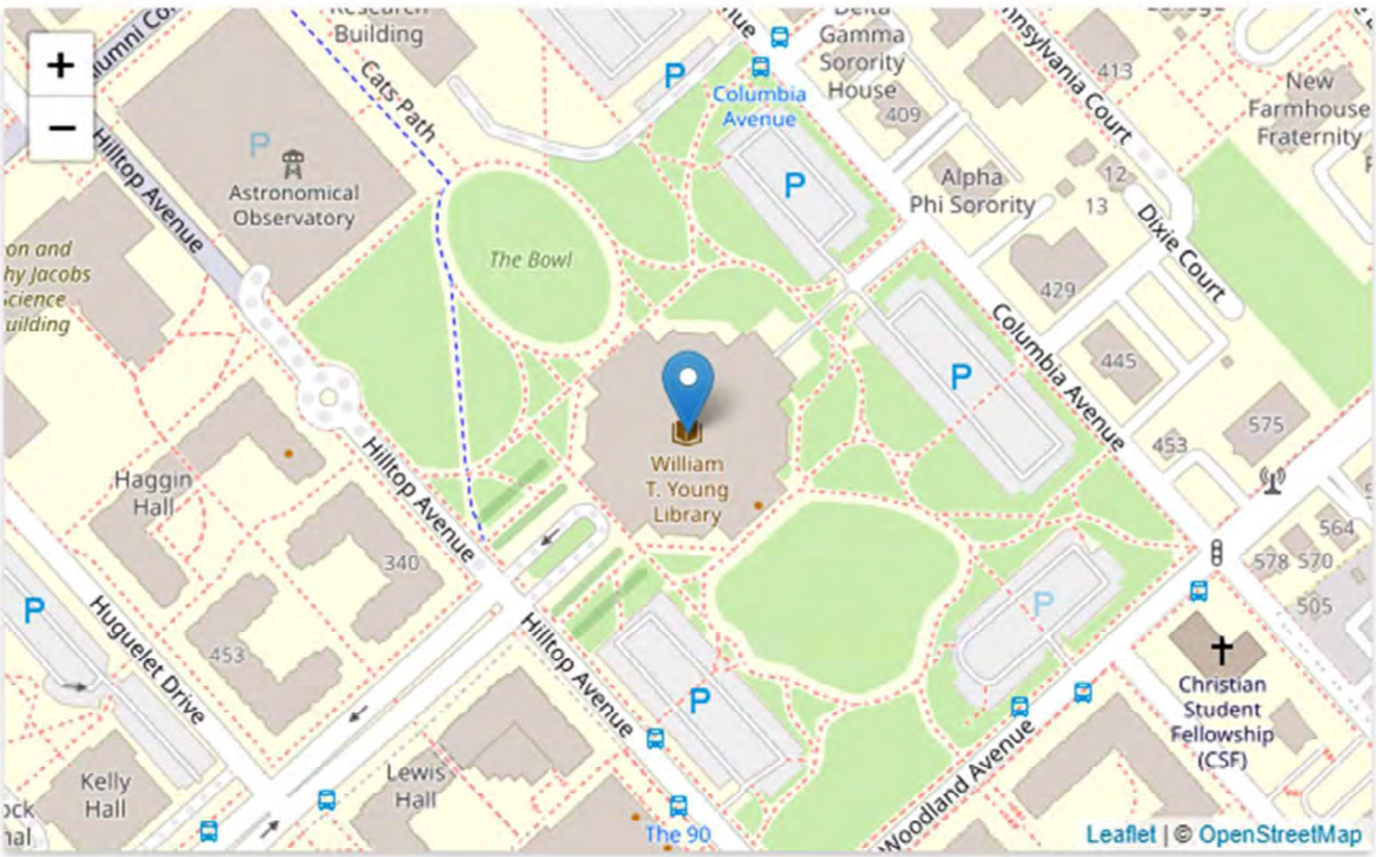
W.T. YOUNG LIBRARY



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CRYSTAL CAVE

- Crystal Cave, the second largest known cave in Fayette County, is located on the west end of the Beaumont Subdivision, next to Man-O-War BLVD.
- The developer planned to extend the subdivision all the way toward Man-O-War Blvd.
- FMSM performed several exploratory tasks outside and inside the cave and developed a recommended non-buildable area in conformance with the City ordinance.

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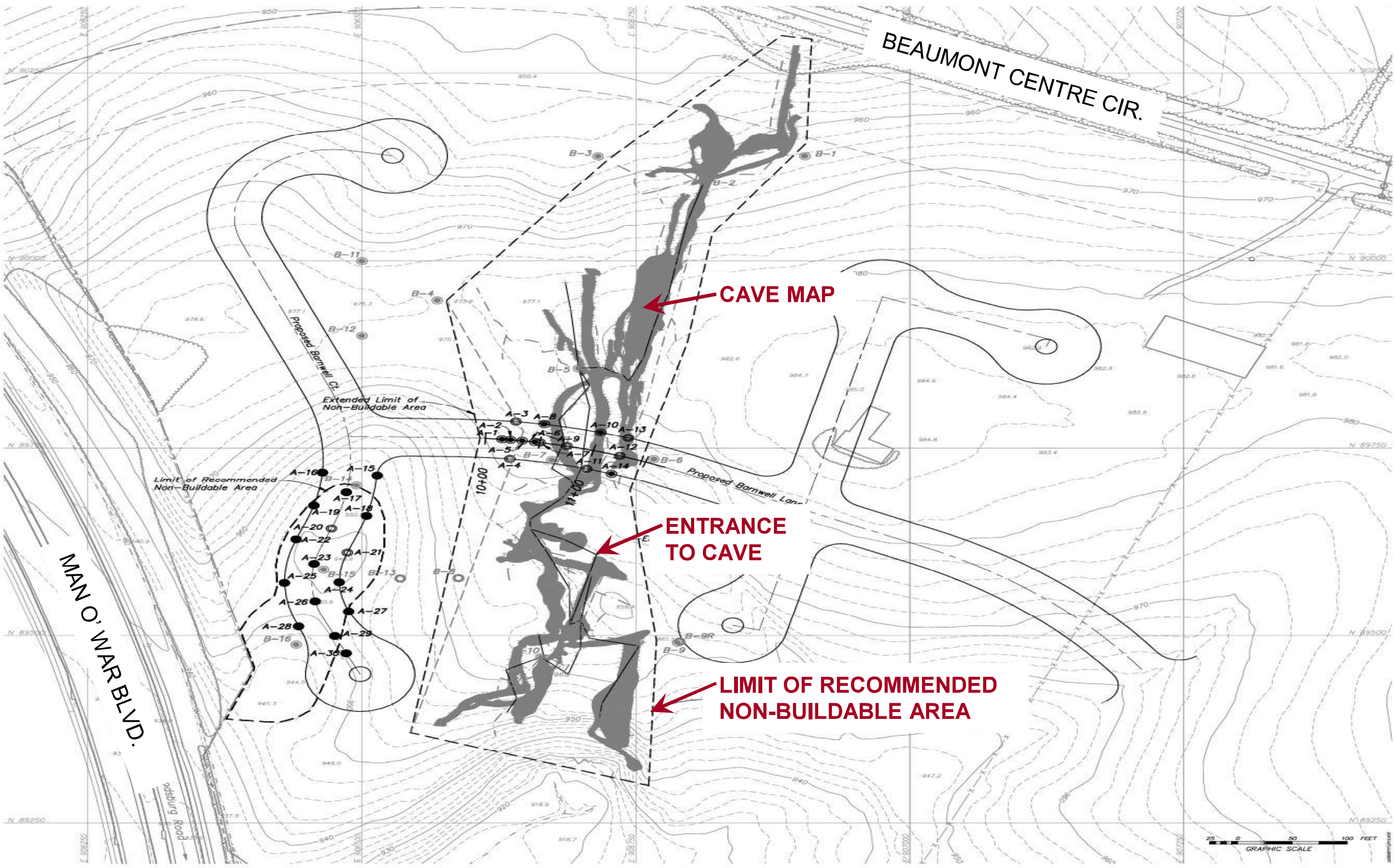
CRYSTAL CAVE



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WOLF CREEK DAM

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WOLF CREEK DAM

- Wolf Creek Dam is located in south central Kentucky and impounds Lake Cumberland. Construction of the dam started in 1941. Due to delays brought about by World War II, construction of the dam was only completed in 1952.
- The dam is comprised of a 1,796-foot-long concrete gravity section (which houses a powerhouse with 6 turbines) and an adjoining 3,940-foot-long earth embankment.
- According to the profile presented on the next slide shows, most of the dam is founded on the Leipers formation and the Catheys formation. These formations are carved with solution features that affect the foundation of Wolf Creek Dam.
- The embankment section included a core trench under the upstream section of the embankment, rather than centered along the axis of the dam.

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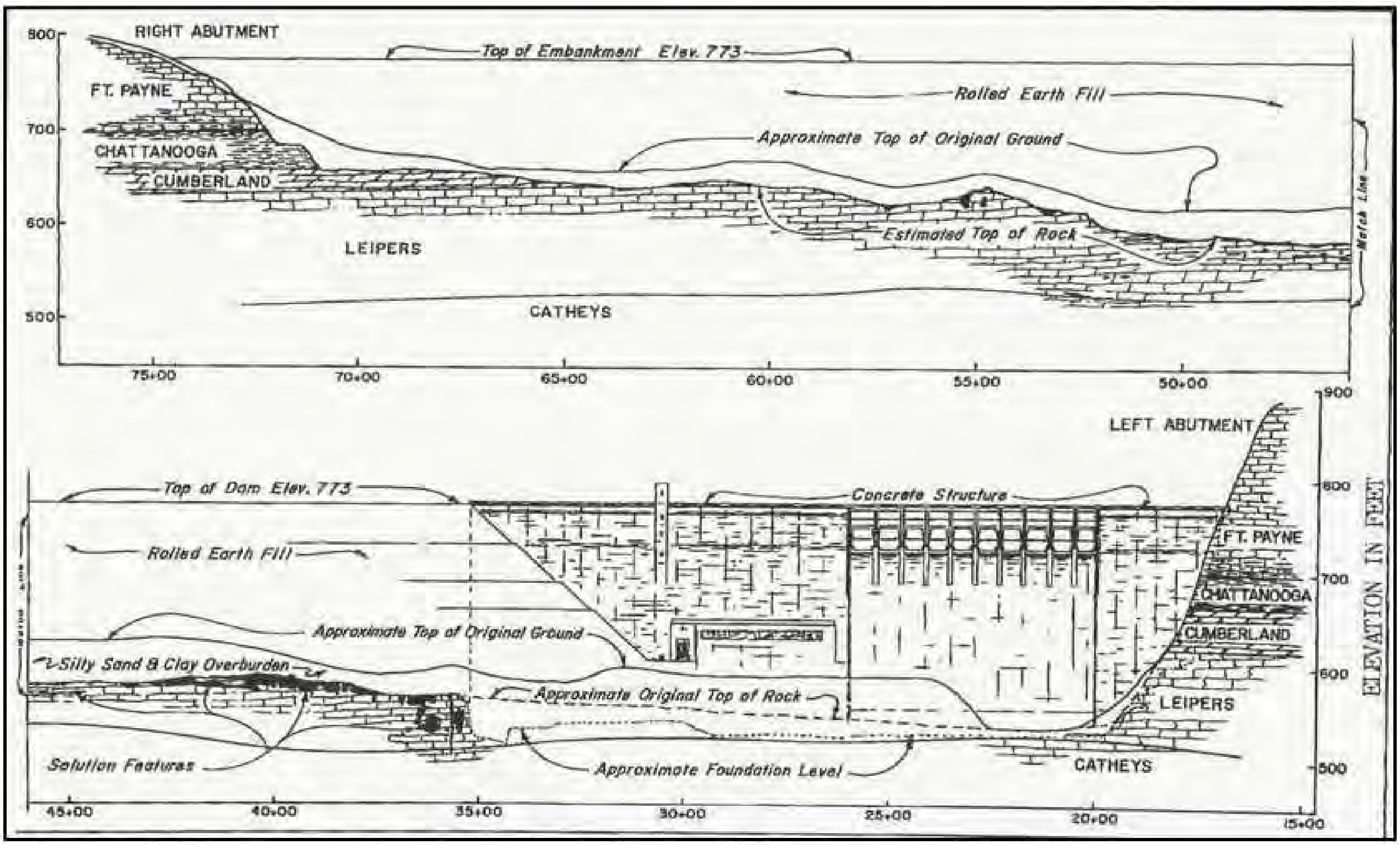
WOLF CREEK DAM
PROFILE



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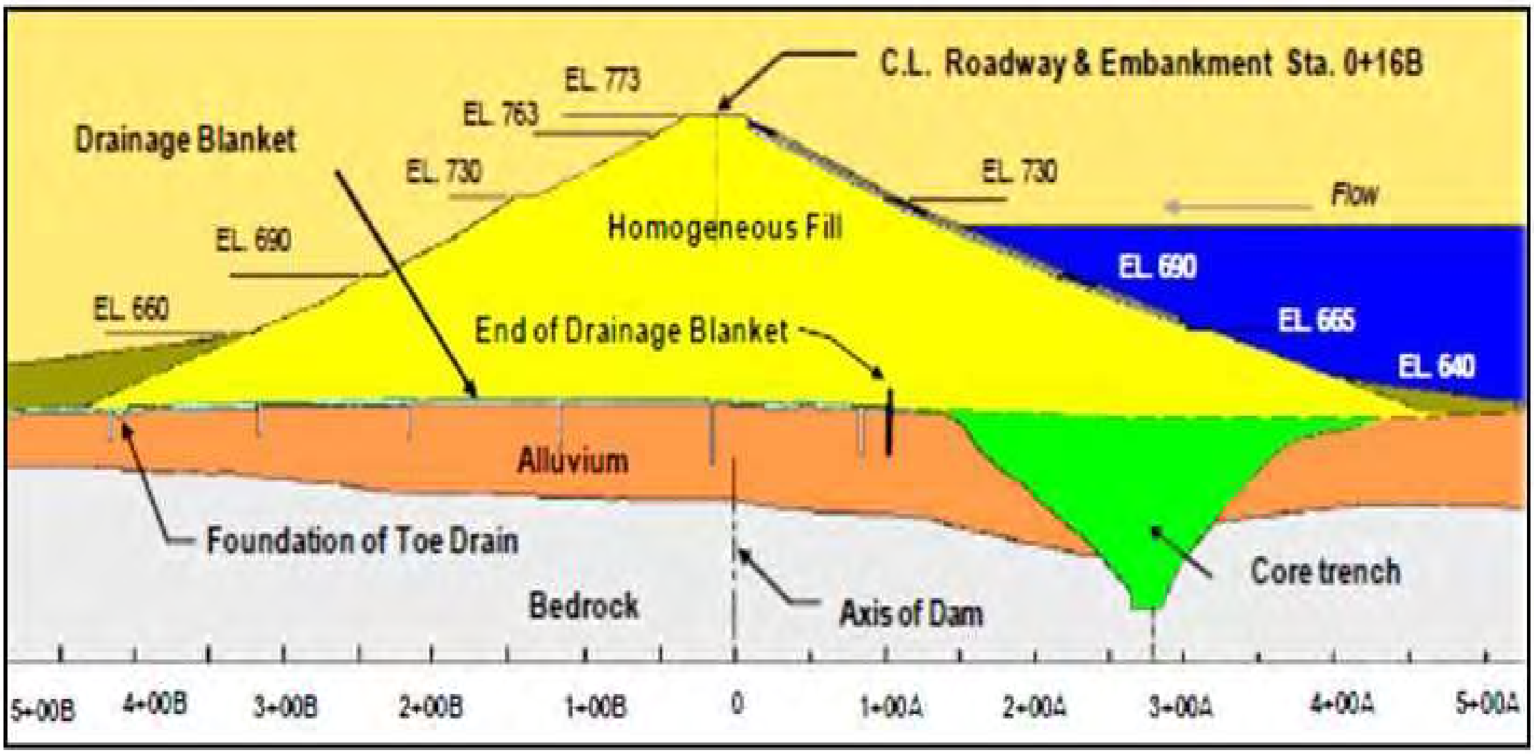
WOLF CREEK DAM
TYPICAL CROSS SECTION



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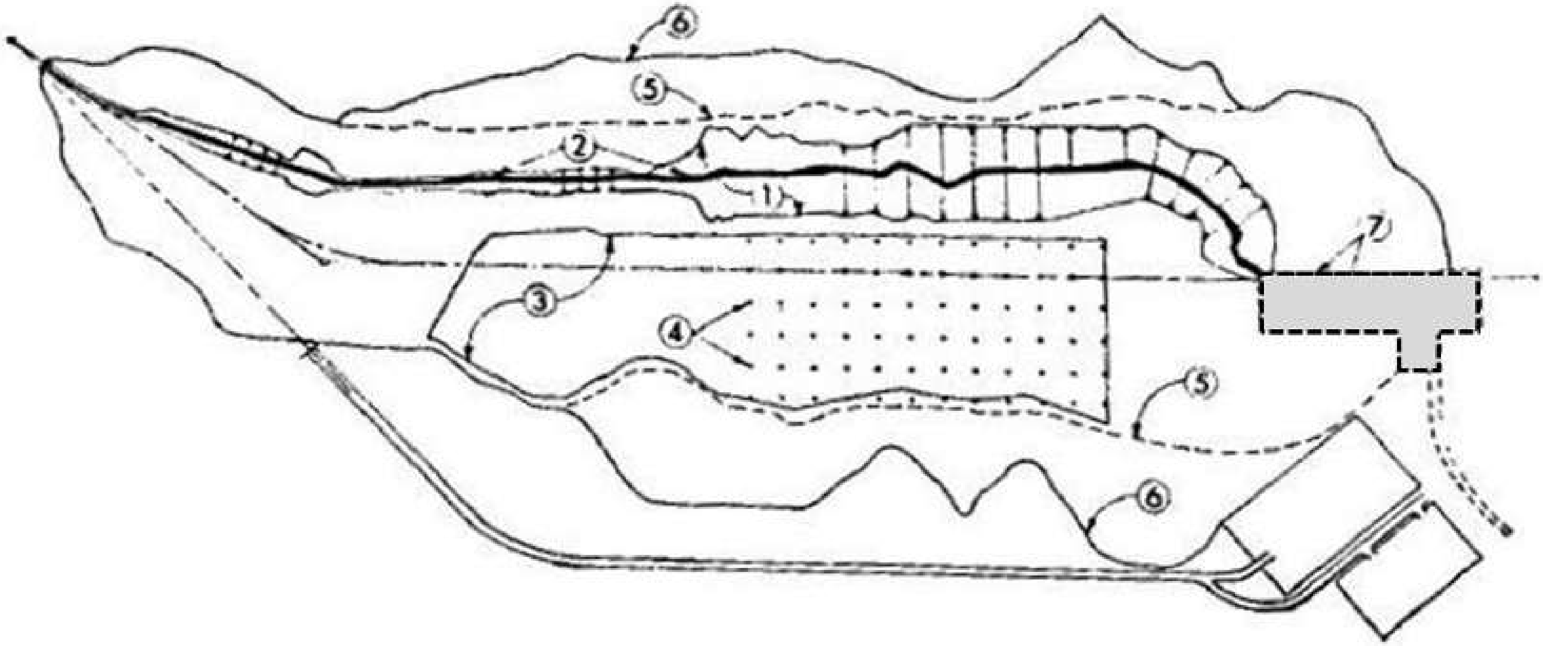
WOLF CREEK DAM
PLAN VIEW OF EMBANKMENT SECTION

ORIGINAL CONSTRUCTION



WOLF CREEK DAM
CUTOFF TRENCH TREATMENT

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WOLF CREEK DAM
CUTOFF TRENCH TREATMENT

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WOLF CREEK DAM
CUTOFF TRENCH TREATMENT

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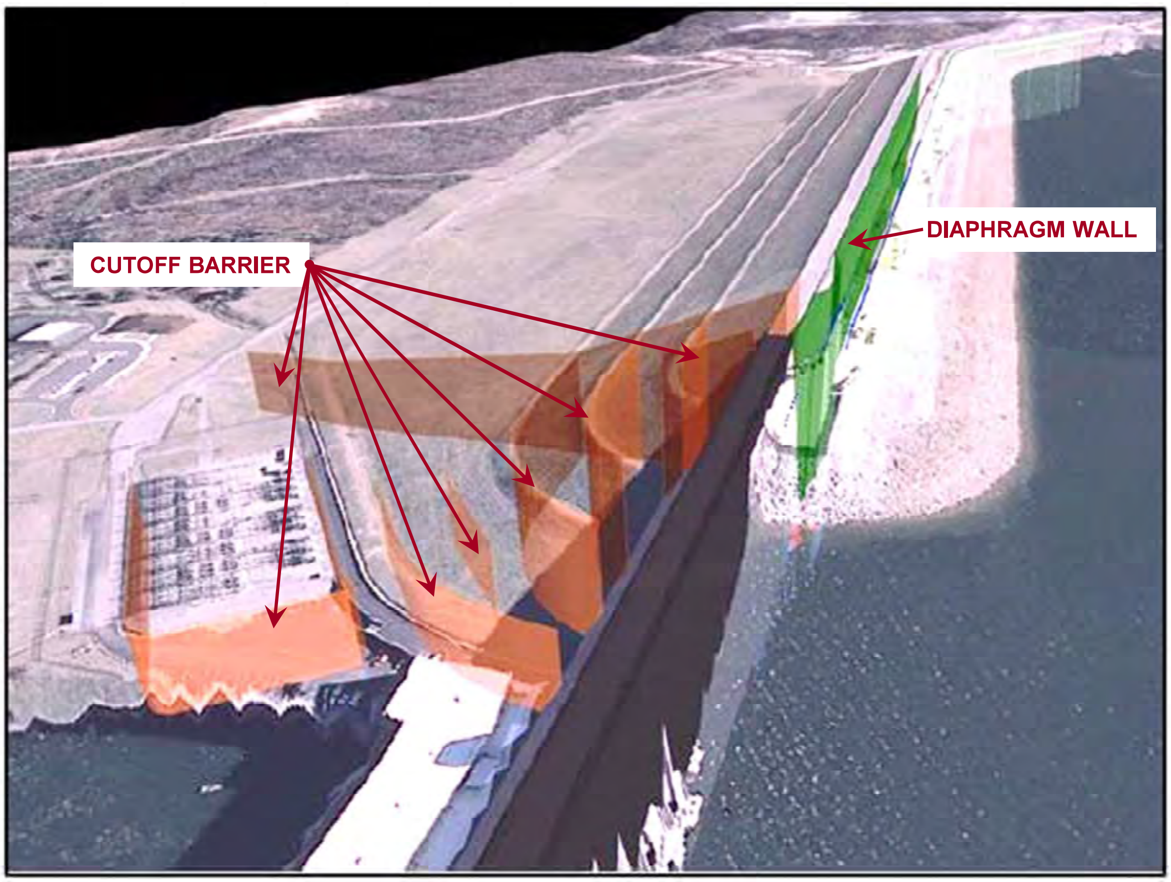
WOLF CREEK DAM
SIGNS OF DISTRESS 1962-1971

- Signs of distress due to excessive internal erosion (piping) and uncontrolled seepage were observed at various times between 1962 and 1971. They consisted of wet areas along the downstream toe of the embankment, sinkholes, muddy water exiting to the tailrace, and depressions on the upstream face of the dam.
- Corrective actions consisting of placing grout to fill voids and lower the high hydrostatic pressures along karst features located adjacent to the concrete structure, and cutoff barriers along the crest of the east embankment and along the riverside of the tunnel and switchyard to provide long-term protection against a "piping" failure of the dam. This work completed in 1979.
- However, the diaphragm wall was shortened in length and depth. The wall extended about 5 feet into the Catheys formation in two short intervals and stopped short of the karstified Catheys-Leipert along the rest of the wall.

WOLF CREEK DAM
1970's REMEDIATION MEASURES



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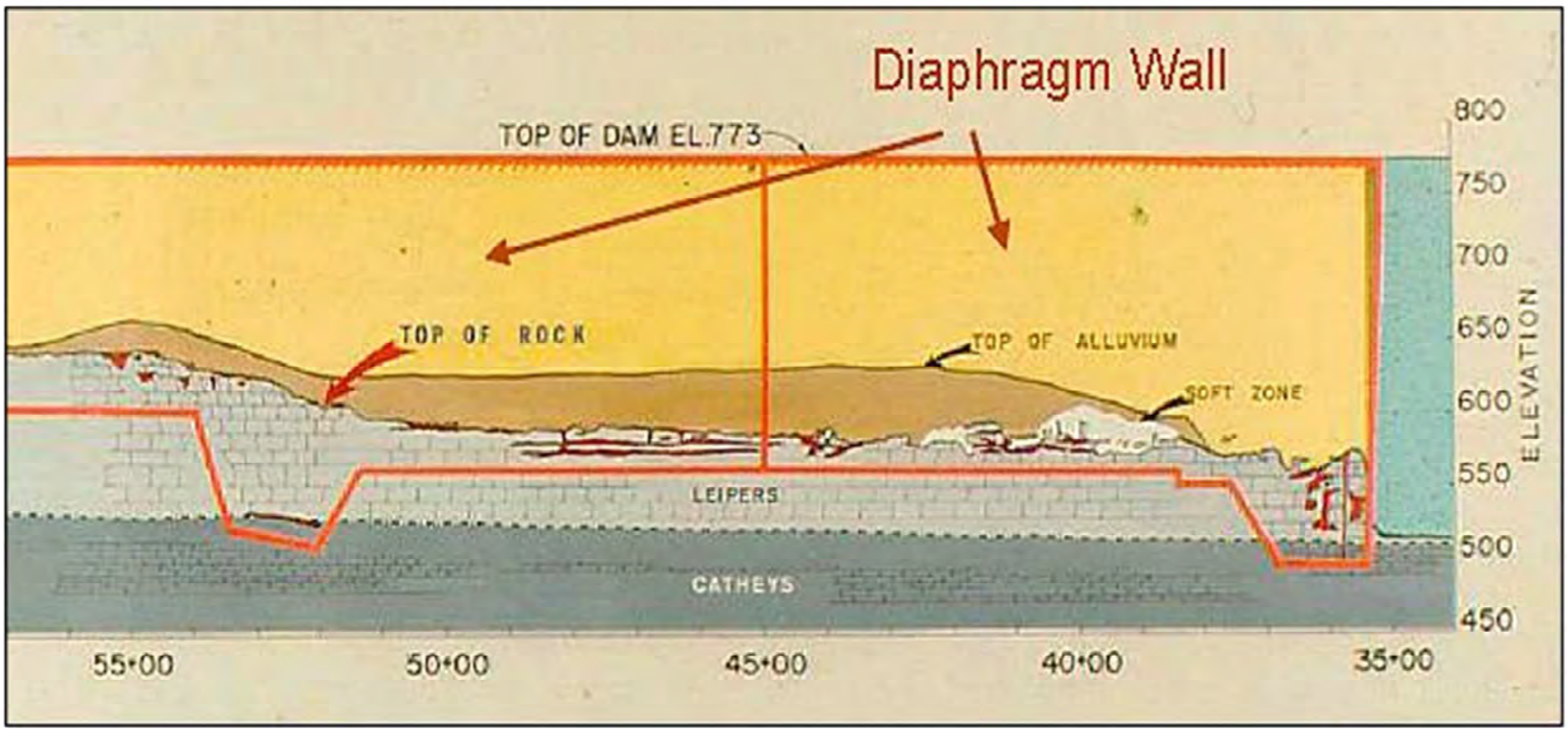
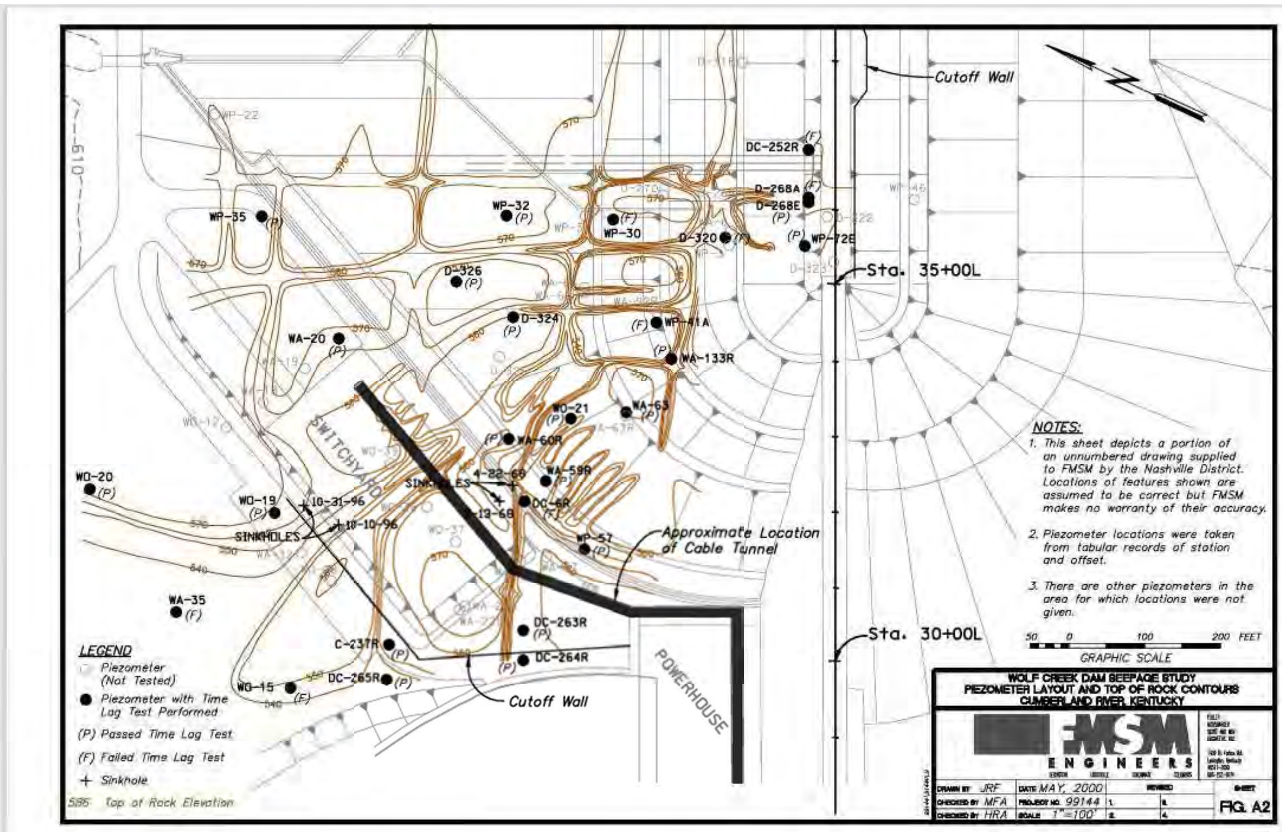
WOLF CREEK DAM
1970's REMEDIATION MEASURES



WOLF CREEK DAM 2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION



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WOLF CREEK DAM
2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION

- In 2000 FMSM was hired to investigate seepage entering the cable tunnel that extends from the east side of powerhouse to the switchyard.
- The purpose of the investigation was to determine the source of the water seeping into the tunnel by reviewing historical data, obtaining subsurface information and installing piezometers near an active seep area of the tunnel. The investigation included surveying portions of the earth embankment of the dam and testing most of the existing piezometers in this area. Historical data and new readings of piezometers installed near the top of rock were used to plot piezometric contours on a plan view of the site.

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WOLF CREEK DAM
2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION

- Readings obtained in August 2000 in some piezometers near the crest of the dam and the interface of the earth embankment and the concrete gravity dam show a 5- to 10-foot increase in elevation over readings taken in August 1984. Conversely, piezometric elevation contours drawn on a plan view of the site show a depression in the piezometric surface that extends from this interface area of the dam (downstream of the cutoff wall) to the toe of the embankment area immediately above the cable tunnel and switchyard. The depression in the piezometric level probably indicates that the underlying karst bedrock system still serves as an active drain for seepage through the dam and under the embankment cutoff wall, which can cause internal erosion (piping) of the fine-grained soil material from the base of the dam.

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WOLF CREEK DAM
2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION

- Uncontrolled seepage and undetected escape of soil material from the foundation drainage system could threaten the integrity of the structure. In addition, the piezometric elevation contour plot shows approximate flow lines that converge toward the toe of embankment area directly above the cable tunnel, which is also an area of extensive karst activity and where sinkholes have developed in the past. Piezometers located along the toe of the embankment, the switchyard, and cable tunnel show a 4- to 6-foot decrease in piezometric level since April 1999. This probably indicates increased seepage through or under the powerhouse cutoff wall.

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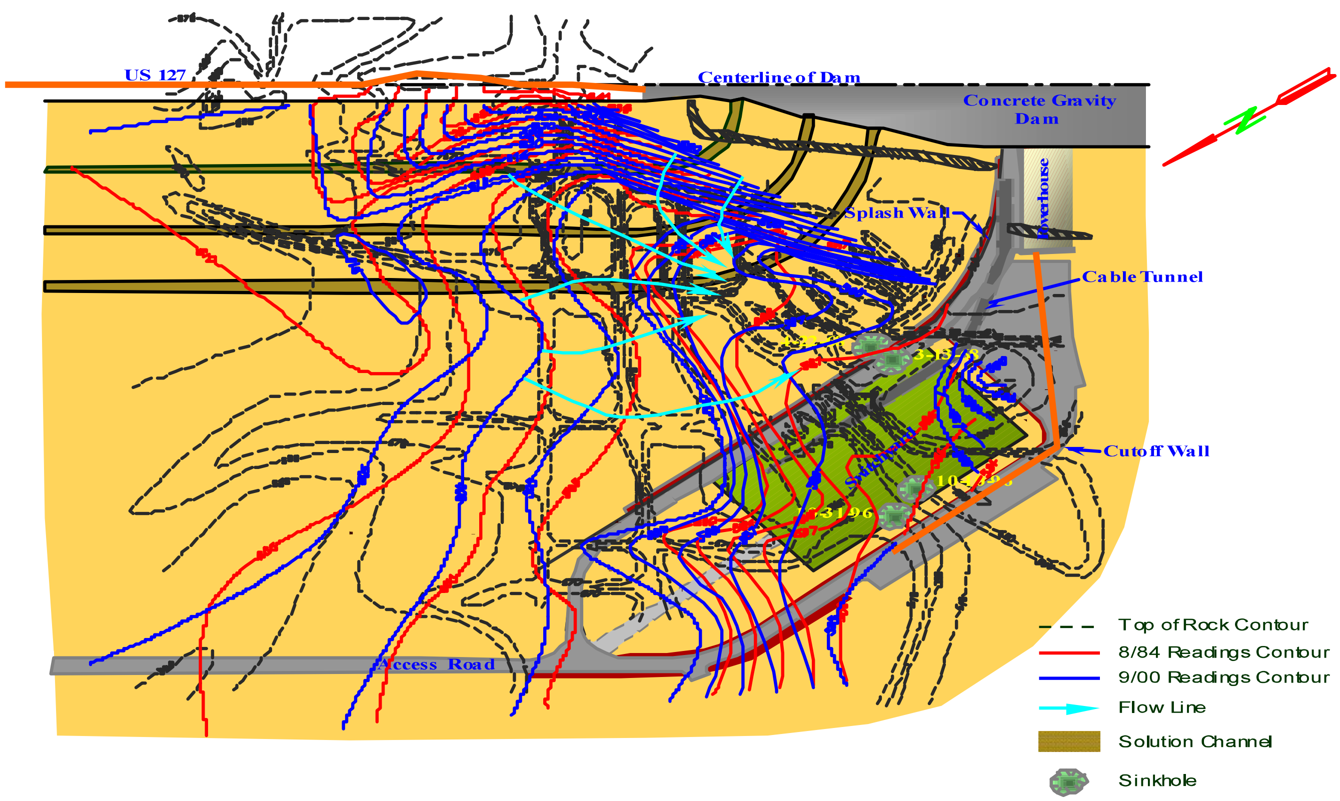
WOLF CREEK DAM 2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION (18)



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WOLF CREEK DAM 2000-2001 CABLE TUNNEL SEEPAGE INVESTIGATION

- In view of the changes in piezometric conditions observed during this study, it was recommended that a comprehensive investigation of the effectiveness of the cutoff walls after 20 years of service be performed. The investigation should include the installation of new piezometers and exploratory drilling in several areas to investigate the anomalies encountered during this study.
- Various signs of distress observed before the seepage investigation, additional subsequent broader investigations, and reviews by different panels of consultants led to the implementation of emergency measures and the subsequent construction of a new diaphragm wall.
- The construction of the new diaphragm wall is well documented. It was completed on March 6, 2013. The work completed at Wolf Creek Dam in 2013 at an estimated cost of $594 million is considered the most complex dam foundation remediation project of any dam in the world.

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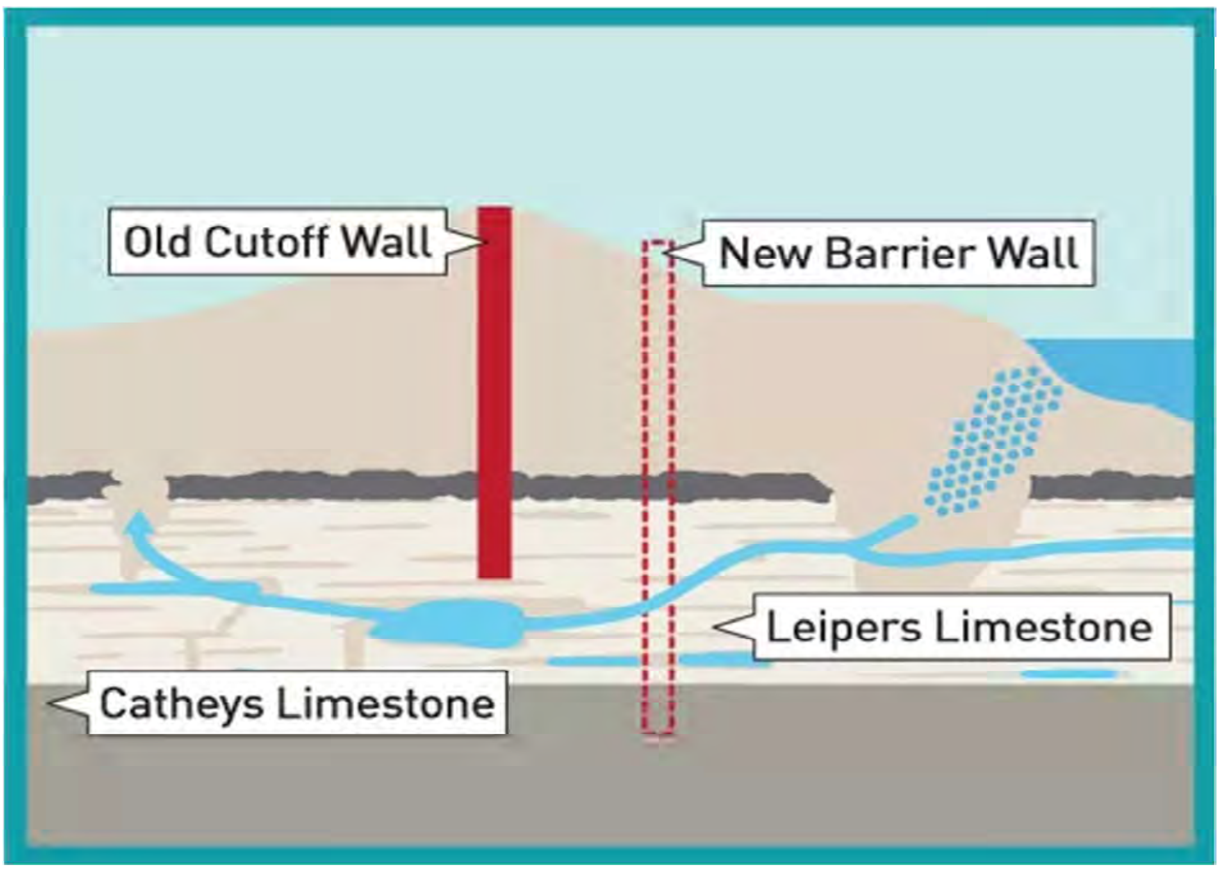
WOLF CREEK DAM
OLD AND NEW BARRIER WALLS



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WOLF CREEK DAM COSTS

- Construction completed in 1952 at a cost of **$80.4 million**
- 1975-1979 Remediation work – Diaphragm Wall **$96 million**
- 2008-2013 Remediation work – Barrier Wall **$594 million**

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TVA BOONE DAM

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TVA BOONE DAM

- Boone Dam is in northeastern Tennessee, on the South Fork of the Holston River. Construction of the dam started in August 1950 and was completed in December 1952.
- The dam is 160 feet high and has a total length of 1532 feet. The main section of the dam is concrete structure, while the north half (right abutment) of the dam consists of a 750-foot-long earth-and-fill structure.
- The dam is found on a karstic limestone formation. The epikarst zone of the limestone includes pinnacled rock protruding into the overlying soil overburden.
- The construction of the embankment section included a cutoff trench under the upstream side of the embankment, a grout curtain along the centerline of the trench and a drainage blanket under portions of the downstream side of the embankment.

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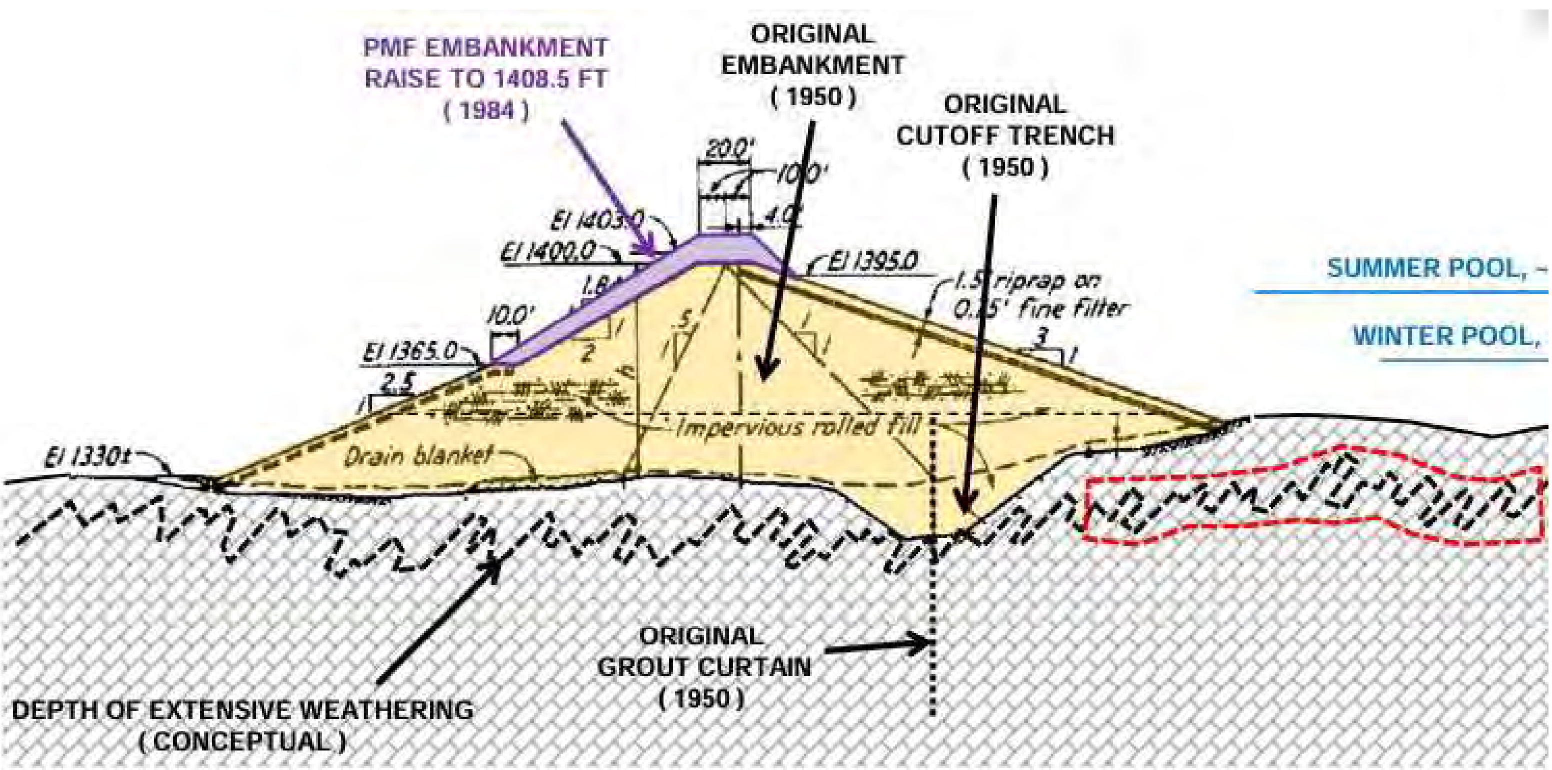
TVA BOONE DAM



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TVA BOONE DAM

TVA BOONE DAM

- In October 2014, a sinkhole was discovered near the base of the embankment at Boone Dam, and water and sediment was found seeping into tailrace. Although sinkhole occurrence is not uncommon in the region, the locations of the sinkhole and the muddy discharge were indicators of potential internal erosion within the dam.

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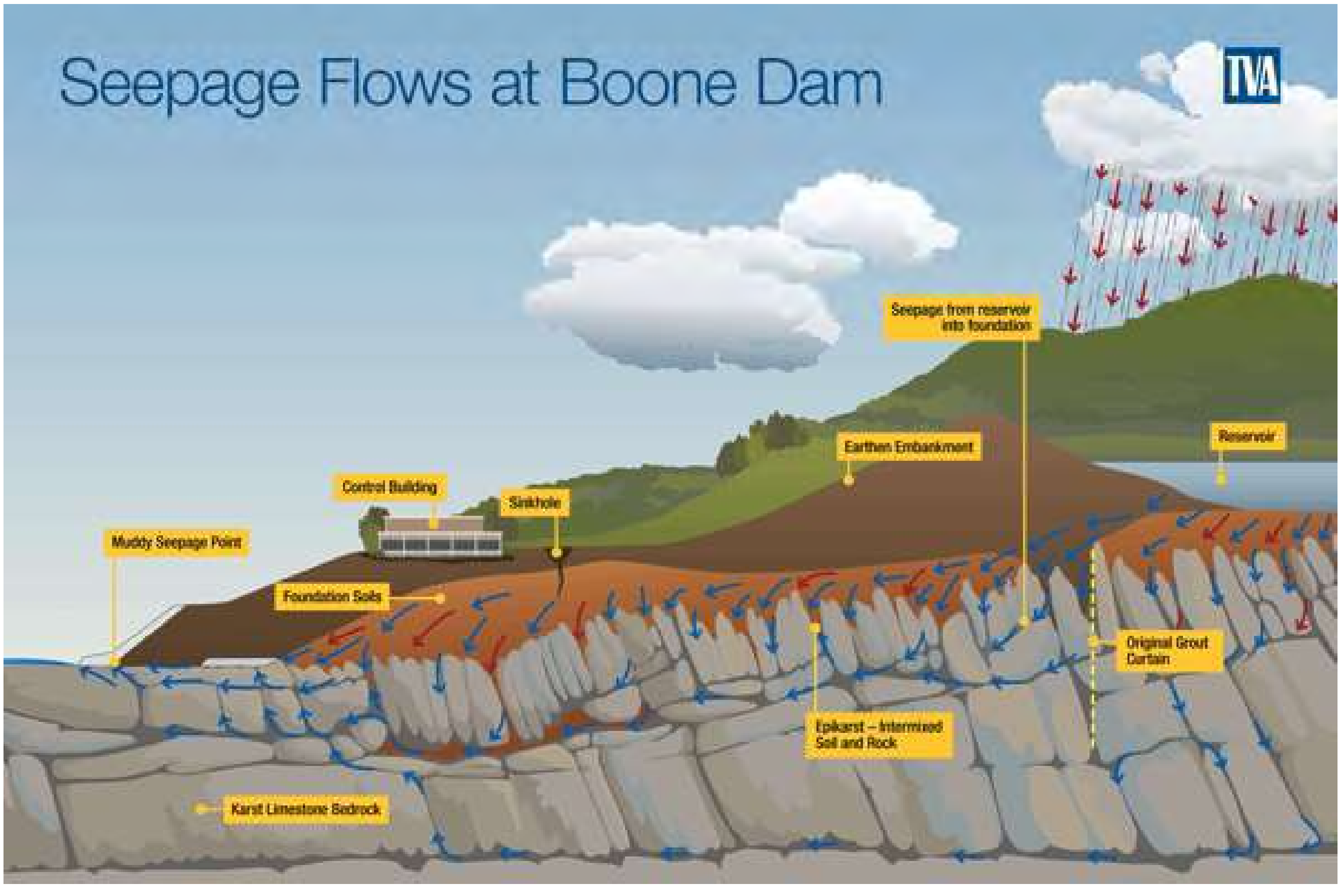
TVA BOONE DAM

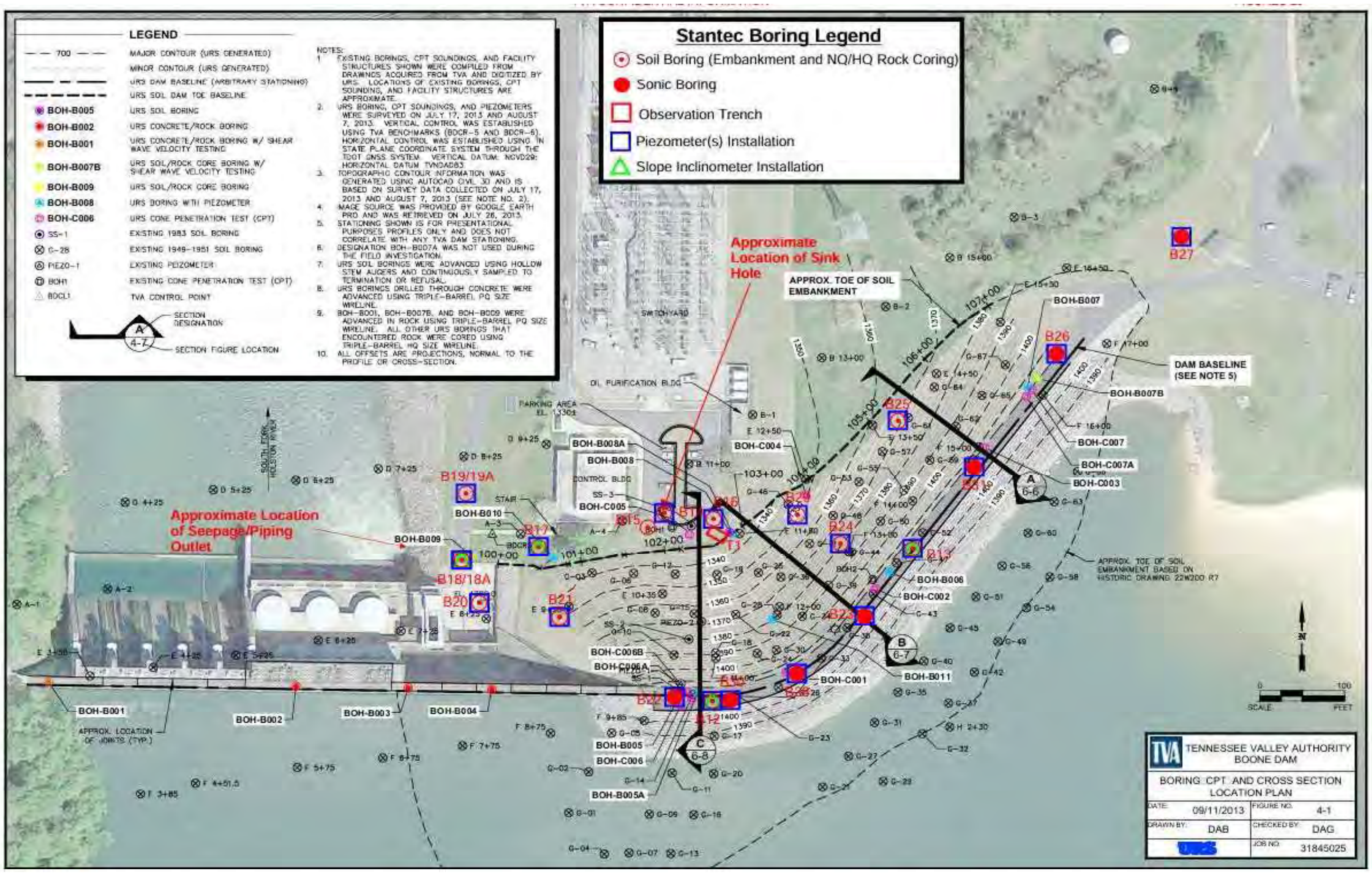


TVA BOONE DAM

The October 2014 events triggered a series of immediate actions:

- Thorough inspections of all surface dam components
- Reviews of historical records
- Risk reduction measures including installation of monitoring devices and graded filter in the tailrace
- Geotechnical, hydrogeologic and geophysical studies including the installation of different surface and subsurface monitoring devices, and dye injection and tracing.

TVA BOONE DAM



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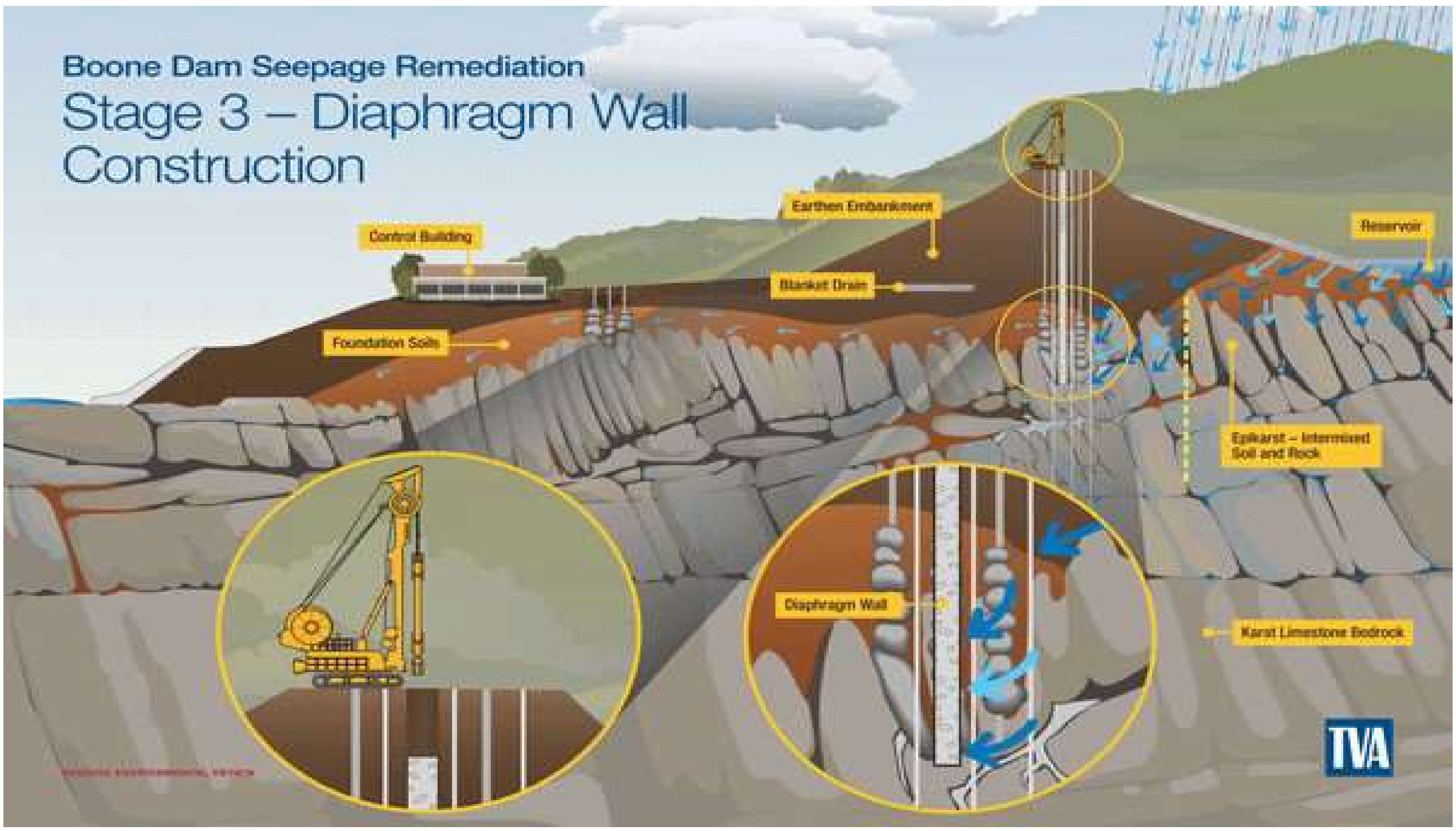
TVA BOONE DAM

- After considering several remediation methods, and holding decision-making process and internal-challenge sessions, the construction of a composite (grout curtain and diaphragm wall) seepage barrier was selected.
- The remediation work started with a exploratory drilling and grouting program. The grouting was performed in phases and included monitoring movement within the embankment exerted by the grout injection.
- The grouting was followed by the construction of the concrete diaphragm wall through the dam and epikarst terminating in the underlying bedrock
- The remediation measures included the construction of berms on the downstream and upstream sides of the earth embankment

Stantec

TVA BOONE DAM



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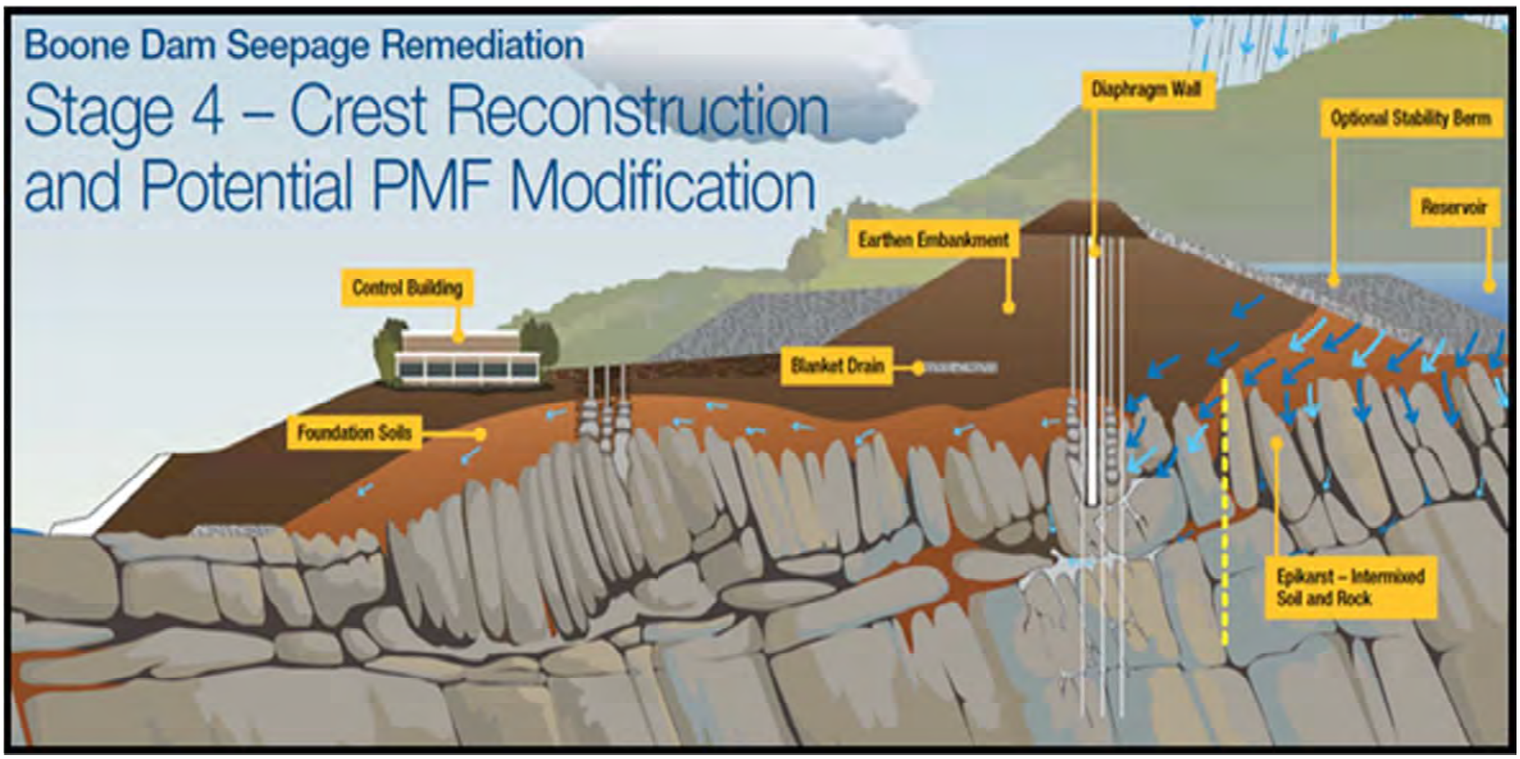

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TVA BOONE DAM COSTS

- Construction completed in 1952 at a cost of **$27.7 million**
- 2015-2022 Remediation work – approx. **$326 million**

Stantec

Notes

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Presentation 6

Louisville MSD's Waterway Protection Tunnel (2017-2022): An Exploration of Louisville, KY's Subsurface

R. M. True

Black & Veatch

Jacob Mathis, P.E.

Louisville MSD

Louisville MSD's Waterway Protection Tunnel (2017-2022): An Exploration of Louisville, KY's Subsurface

R. M. True, Civil Engineer | Black & Veatch
Jacob Mathis, P.E., Project Manager | Louisville MSD

In June 2022, Louisville and Jefferson County Metropolitan Sewer District (MSD) completed a Combined Sewer Overflow (CSO) conveyance and storage tunnel system located along the Ohio River in downtown Louisville, Kentucky. Louisville MSD's Waterway Protection Tunnel (WPT) is a 4.0-mile long, 20-foot diameter, 200-foot deep CSO storage and conveyance tunnel with a storage capacity of 55 million gallons (MG). During wet-weather events, a total of 25 CSOs are diverted to the tunnel for storage and conveyance. Captured CSO flows downstream to the Rowan Pump Station, and ultimately ends up at a treatment center before discharging clean water to the river. The WPT is designed to prevent 439 million gallons of CSO from discharging to public waterways like the Ohio River in a typical year. For the WPT, Black & Veatch served as the Engineer of Record with a joint-venture between J. F. Shea Company and Traylor Bros., Inc. (Shea-Traylor Joint-Venture) serving as Contractor.

Black & Veatch completed the design of the WPT (2.5-miles in length, with four drop shafts) in under eight months. The work included phased deep-rock geotechnical investigations, alternatives analyses, and detailed design. Black & Veatch also provided construction management and inspection services. In August 2018, while tunnel construction was underway for the original 2.5-mile tunnel, MSD requested Black & Veatch explore extending the WPT 1.5-miles to increase CSO storage capacity. Among other major WPT design modifications that provided MSD with additional 11 MG of CSO storage and operational flexibility, Black & Veatch executed an accelerated subsurface geotechnical investigation and detailed design in under four months to maintain the Consent Decree schedule.

In total the phased deep-bedrock geotechnical investigation for the complete WPT included the following:

- 26 deep bedrock geotechnical borings ranging from 115.3' to 255.6' bgs (totaling over 1-mile of total drilled footage and approximately 3,400-LF of total rock core) which included pneumatic packer water pressure testing and rock sampling analyses to influence design parameters and characterize tunnel bore constructability;
- Exploration of Ordovician, Silurian, and Devonian bedrock through 10 bedrock lithologies including: Louisville Limestone, Waldron Shale, Laurel Dolomite, Osgood Formation, Brassfield Formation, Drakes Formation (Saluda, Bardstown, Rowland), Bull Fork Formation, and Grant Lake Limestone;
- 25 Gas Vent Borings to dissipate natural gas along tunnel alignment within the tunnel bore area prior to TBM arrival which included geophysical borehole logging and monitoring;
- Five (5) shallow soil borings; AND
- Five (5) environmental borings

In September 2020, after 20 months of excavating through four miles and 650,000 tons of rock, the tunnel boring machine (TBM) "holed-through" the wall of the TBM retrieval shaft. Through geologically complex ground, the TBM safely and efficiently excavated the WPT due in part to Black & Veatch's comprehensive geotechnical investigations. As the first deep bedrock tunnel in downtown Louisville, the design and safe construction of the WPT represents a significant geotechnical and construction engineering achievement.

The WPT was completed in May 2022, months prior to the Consent Decree deadline. In the last three years months of operation, the WPT captured over 1 billion gallons of CSO that would have otherwise polluted Louisville's waterways.

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Presentation 7

Geotechnical Assessment of Embankment Dams Under Extreme Rainfall Events: A Study of Slope Stability and Seepage Behavior

Luis Felipe Salome Simon

Department of Civil Engineering,

University of Kentucky

Ohio River Valley Soils Seminar

Geotechnical Assessment of Embankment Dams Under Extreme Rainfall Events: A study of Slope Stability and Seepage Behavior

by

Luis Felipe Salome Simon

Prof. Michael E. Kalinski, Ph.D., P.E.

Abstract

Climate change has emerged as a significant challenge to civil engineering practice, with the increasing frequency and intensity of extreme rainfall events imposing unprecedented hydrologic demands on aging infrastructure. In the United States, earthen embankment dams are particularly susceptible to these evolving climate stressors, facing elevated risks of overtopping, slope instability, surface erosion, and internal erosion (piping). This research and literature review investigates documented cases where intense precipitation has directly contributed to partial or complete dam failure. Representative examples include the Oroville Dam spillway incident (2017), the Edenvale and Sanford Dam failures (2020), and the historic Kelly Barnes Dam disaster (1977), each illustrating the vulnerability of embankment dams under extreme hydrologic loading. The review synthesizes key failure mechanisms, including overtopping-induced surface erosion, pore pressure build-up leading to slope instability, and seepage-driven internal erosion. Additionally, this work examines modern analytical approaches integrating hydrologic modeling, seepage analysis, and slope stability assessment to evaluate dam resilience under projected climate extremes. The findings underscore the critical need to update probable maximum flood (PMF) estimates, modernize spillway capacity, enhance real-time monitoring, and implement adaptive risk management strategies to protect downstream communities and maintain the integrity of critical water infrastructure in the face of intensifying climate impacts.

1. Introduction

Approximately 80% of the nation's 90,000 dams are registered in the National Inventory of Dams. (USACE, 2020). Many of these dams were designed and constructed before modern hydrological data and without consideration for the intensification of extreme rainfall driven by climate change. (Miller et al., 2018; FEMA, 2018).

There has been an increase in high intensity rainfall events that has posed significant threats to these aging structures. During the fourth national climate assessment, research and reporting documents showed that intensity of heavy rainfall events has increased markedly across the midwest, northeast and southwest United States, with some regions experiencing over a 40% rise in the heaviest 1% of precipitation events since the mid-20th century. (Kunkel et al., 2013). This directly challenged the reliability of older design assumptions, especially for small and intermediate earth dams with limited spillway capacity and outdated flood design standards.

2. Incidents And Climate Driven Dam Failures Through Time

Kelly Barnes Dam (Georgia, 1977)

In this case study, 7 inches of rainfall occurred within 24 hours, which led to the overtopping failure of the dam. This privately owned earth embankment dam resulted in 39 fatalities and extensive damage downstream. (FEMA, 1978). The failure was caused by inadequate spillway capacity and internal erosion exacerbated by piping.

Edenville and Sanford Dam Failures (Michigan, 2020)

Dam failures occurred during multiple day events of rainfall that produced 7 inches of rain corresponding to a 500-year flood in some locations (ASDSO, 2020). The privately-owned Edenville Dam failed due to overtopping and internal erosion, followed by sequential failure of the downfield Sanford Dam. There were more than 10,000 residents that were evacuated and the event exposed regulatory gaps and the hazards posed by undermaintained, high hazard structures.

Lake Delhi Dam (Iowa, 2010)

The Lake Delhi Dam suffered a short duration and high intensity rainfall which overwhelmed the spillway. This led to overtopping and rapid embankment erosion followed by full breach of the dam (FEMA, 2010). The event illustrates how embankment dams with undersized or deteriorating spillways remain highly vulnerable.

Localized Failures and Emergency Incidents

Multiple small earthen dams particularly farm ponds, municipal water supply reservoirs, and private recreational lakes have failed during or narrowly avoided failure following record breaking storms that have happened over the summer in the Midwest, Appalachia, and southeast in the past 20 years (Ehansi and Mahdavi 2022). It can me said that incidents do not result in total failure but necessitate emergency drawdowns and costly repairs.

3. Failure under Extreme Rainfall – Mechanisms

It can be concluded that overtopping is highlighted as the leading cause of earthen dam failures worldwide (FEMA, 2015). The primary mechanisms is aggravated by extreme precipitation events, which trigger the following failure mechanisms.

Surface Erosion from Overtopping

When dealing with inflows of exceeded spillway discharge capacity, water flows over the crest and downstream face. This quickly erodes the soil and protective vegetation. Progressive head cutting can promptly lead to a breach of the dam. Rapidly removing protective soil layers and removal the vegetation serves to accelerate the process. Once the protective layer of the soil is lost, the embankment materials that are usually made of soils that erode, are directly exposed to the erosive action of water. This starts a progressive head cutting of the material where the erosion moves upstream towards the reservoir. If not taken care of quickly this procedure can quickly deepen and widen the breach channel, leading to dam failure. As water flow rates accelerates, erosion rates also accelerate.

Slope Instability

High intensity rainfall infiltrates the embankment and foundation soils, elevating pore water pressures and reducing effective stress and shear strength as summarized in Table 1 (Duncan et al., 2014). This can trigger shallow and deep-seated rotational failures. The risk is amplified if the embankment was built with low plasticity or poorly compacted soils. High intensity rainfall can significantly compromise the structural integrity of embankment dams through a sequence of geotechnical processes such as rainfall infiltration and pore water pressure rise. This happens when rainfall infiltrates into embankment foundation soils and it elevates pore water pressures with the soil matrix. As the pore water pressure increases due to infiltration the effective stress decreases, which reduces the soils' ability to resist shear stress by reducing its shear strength.

Table 1. Summary of effect of excessive rainfall on susceptibility to dam failure.

Mechanism	Rainfall Effect	Geotechnical Consequence
Pore water pressure	Increases due to infiltration	Decreases effective stress and shear strength
Effective stress	Reduces pore water pressure	Lower soil strength more and more prone to failure
Type of Failure	Shallow or deep seated depending on soil	Depends on embankment materials and compaction
Coupled Response	Rainfall plus water rapid water level rise	Accelerated instability due to transient pore water peaks.

Reduced effective stress dramatically lowers shear resistance along potential slipping surfaces. Depending on the embankments soil characteristics and compaction quality, this can lead to shallow rotational slides. This risk is higher in embankments constructed with low plasticity clays and or poorly compacted fill, where shear strength is already marginal.

Modern research emphasizes the coupled nature of rainfall infiltration and mechanical response, where seepage alters stress states and instigates deformation or failure. A modeling study illustrates that faster reservoirs level rise under coupling with rainfall raises pore water pressures faster and accelerates slope instability (Fig. 1).

Internal Erosion and Seepage (Piping)

Prolonged high reservoir levels create greater hydraulic gradients. This can cause internal erosion along the flow paths. This involves poorly compacted fill zones, cracked cores or animal burrows (ICOLD, 2016). Without designed filters and drains, fine particles can be moved and washed out, creating subsurface voids that lead to a collapse of a dam.

Prolonged high reservoir levels increase hydraulic head, intensifying hydraulic gradients. When these gradients exceed the critical hydraulic gradient, effective stress can approach zero and internal erosion processes can be triggered.

Erosion and Failure Modes

Concentrated leak erosion starts at cracks or flaws and leads to a focus pathway. The cracks may result from poor compaction, desiccation, settlements and animal burrows. Erosion starts at the downstream exit point and works its way back creating subsurface tunnels and propagates to failure. Piping is when finer particles pass through voids in a coarser matrix, leading to progressive internal removal of material. Then soil contact erosion flow along the contact zones between coarse and fine soils erodes the finer soil, forming cavities that can collapse and escalate into voids and sinkholes.

Prolonged high reservoir levels significantly elevate pore pressures and hydraulic gradients, enhancing the risk of internal erosion, especially through flawed compaction, animal activity, filter deficiencies, and adverse geometry. To guard against catastrophic dam failure, meticulous design of filters and drains, regular inspection, and monitoring of seepage is essential.

Instability of Toe Embankment

Rapid drawdown operations may be initiated during a flood to mitigate against dam failure. However, rapid drawdown can destabilize the upstream slope of a dam by triggering localized slumping (Schweiger et al., 2019). Rapid drawdown during flood operations can create a dangerous imbalance between fast lowered reservoir levels and slow pore pressure dissipation in embankment dams. This condition reduces slope stability and can cause slope failures on the upstream side of the dam.

Risks are influenced by soil type, permeability, drawdown rate, and drainage design. Reducing hazards, dams require gentle slopes, effective filters and drains, controlled drawdown schedules. If rapid drawdown is inevitable, reinforcement measures such as berms, buttressing and seepage control can be employed.

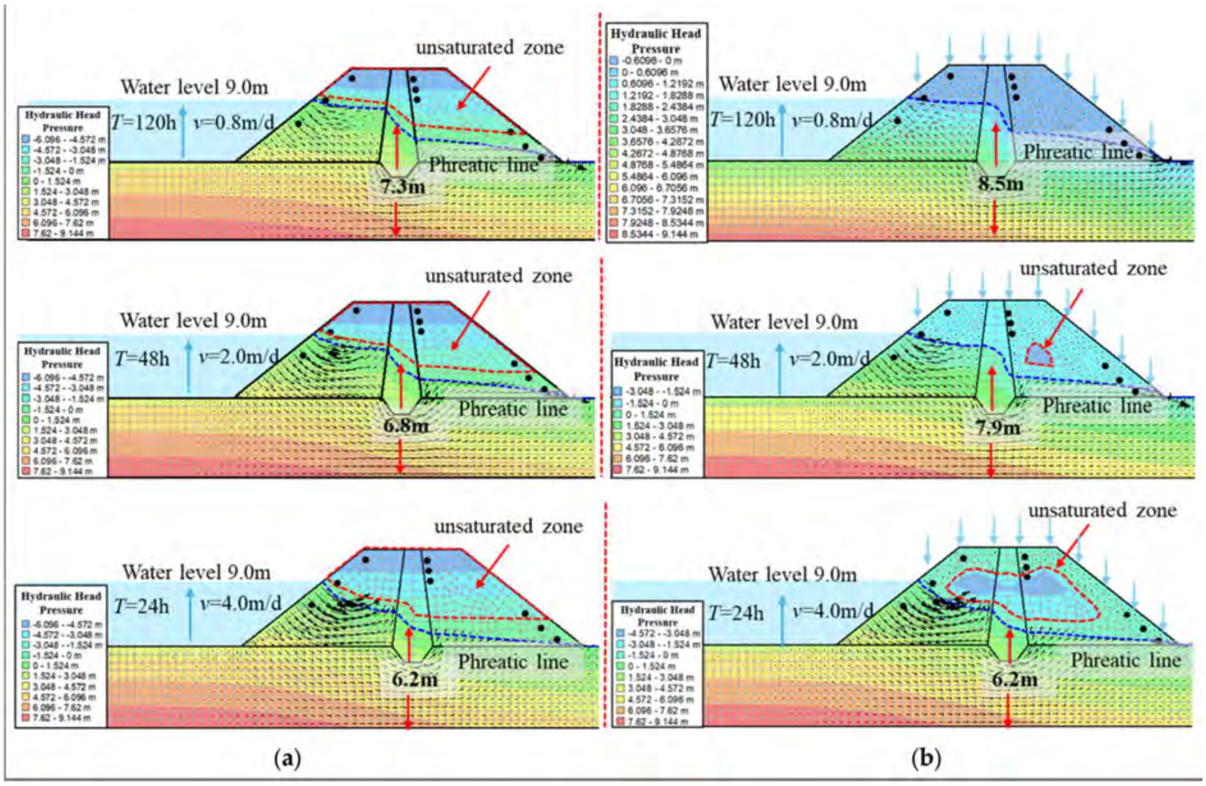

Figure 1. Seepage and stability analysis results for an earth-rock dam under varying rainfall and reservoir level conditions. Adapted from Sun, Q., Yang, Q., Zhang, L., Zhou, C., & Zhan, J. (2024). *Numerical Analysis of Seepage and Stability in Earth-Rock Dams Considering Rainfall and Reservoir Level Changes.* **Water**, 16(23), 3340. MDPI. https://doi.org/10.3390/w16233340

4. Analytical and Modeling Approaches

There is a need to employ observation and modeling analyses to predict the response of earth dams to excessive precipitation events. Below are some of the techniques that can be employed.

Hydrologic Modeling

Design storms increasingly rely on the NOAA Atlas 14 data and regional climate projections to estimate probable maximum precipitation and probable maximum flood values that are more representative of future extremes in the post-climate change world (USACE, 2014; Zhang et al. 2021). Modern dam design increasingly uses NOAA Atlas 14 precipitation data combined with the regional climate projections to capture the greater intensity and frequency of the extreme rainfall events. Older models are based only on historical records, while this newer approach gives more accurate estimates of probable maximum precipitation and probable maximum flood discharges, underscoring the need for updated hydrometeorological methods and probabilistic approaches to reduce overtopping and downstream flooding risks.

Seepage Analysis

SEEP/W, PLAXIS and FLAC3D are examples of numerical tools to model infiltration, transient pore pressure response, and flow nets through heterogeneous embankments (Fell et al., 2005). The models can assess filter and drainage performance under elevated levels of reservoir levels. Numerical tools such as SEEP/W, PLAXIS and FLAC3D are essential for stimulating seepage, transient pore pressure response and flow net methods.

These programs incorporate anisotropic hydraulic conductivity, nonlinear soil water curves, and time dependent boundary conditions, enabling more realistic predictions under rapid drawdown or rising reservoir levels. SEEP/W is commonly used to calculate exit gradients and assess piping risk, while PLAXIS couples hydraulic and mechanical behavior to capture deformation, hydraulic fracturing, and stress redistribution. They provide a rigorous framework for evaluating filter and drainage performance. Identifying zones vulnerable to internal erosion, the practitioner can use this information to design remedial measures for drains, relief walls and cutoff walls in line with modern dam safety standards.

Slope Stability Analysis

Factors of safety against slope failure can be calculated using limit equilibrium and finite element analyses under saturated conditions, dynamic loads and drawdown scenarios. Sensitivity studies can test how different intensities in rainfall affect failure probabilities (Duncan and Wright, 2005). The factor of safety below unity indicates failure, while values of 1 to 1.3 represent marginal stability depending on the dam’s hazard classification and regulation criteria.

There are two approaches that are used: Limit Equilibrium methods and Finite element analyses (FEA/FDM). Limit equilibrium methods discretize the potential mass into slices and evaluate the equilibrium conditions using assumptions about the inter slice forces and pore water pressure distributions. The methods most used are the Janbu, simplified Bishop, Spencer and

Morgenstern-Price methods. Limit equilibrium methods are most used in engineering practice because of their low computation cost and ease of implementation in software such as SLOPE/W that are met with well-established regulatory acceptance. The Limit equilibrium methods require prior assumptions about slip surface geometry and do not inherently capture progressive failure mechanisms.

Rapid drawdown scenarios following high inflow events, may quickly lower reservoirs levels to prevent overtopping. Rapid drawdown reduces the stabilizing hydrostatic pressure on the upstream slope while pore pressures remain elevated, potentially triggering instability. This scenario is often modeled by assigning time dependent pore pressure distributions within the slope and evaluating transient changes in factor of safety.

By systematically altering these parameters, engineers can estimate the probability of failure under a range of future rainfall scenarios. This framework based on probability is increasingly recommended for dam safety evaluations under climate change conditions. (USACE, 2019; Zhang et al., 2021).

Sensitivity Studies and Climate-Driven Rainfall

To address uncertainty from future climate projections, parametric sensitivity is used, and this involves varying rainfall intensity and duration to simulate 50- and 100-year events. Infiltration rates based on soil hydraulic conductivity, reservoir rise rates affecting phreatic surface position and soil shear strength parameters due to variability or degradation are also varied in the form of a parametric study.

Probabilistic Risk Assessment

In current years, probabilistic methodologies have increasingly been integrated into dam safety evaluations to better capture the inherent uncertainties of associated climate change impacts, hydrologic loading, and structural performance. Traditional deterministic dam breach analysis is based on single design flood assumptions. This often fails to reflect the wide spectrum of possible future events. Agencies such as the U.S. Army Corps of Engineers (USACE, 2019) now are integrating climate adjusted hydrologic frequency analyses intro probabilistic risk assessment frameworks, allowing for multiple scenarios to be modeled with variable precipitation intensities, storm durations, and spatial distributions informed by downscaled climate projections.

This involves combining hazard curves that include different probabilities of hydrologic and seismic loadings with fragility functions that represent the conditional probability of failure given a load. These fragility functions can be developed from empirical dam performance data and physical modeling and numerical simulations. This has uncertainties from climate models and allows catalysts to propagate not only hydrologic variability but also structural response into dam failure simulations.

The results of probabilistic risk assessments are typically expressed as failure probabilities that combine the likelihood of failure with the magnitude of downstream consequences such as potential loss of life and economic damages and environmental impacts. This allows emergency action plans to be updated for a range of plausible scenarios.

Agencies are moving towards adaptive risk informed decision making where public risk analysis outputs feed into cost benefit analyses for design retrofits, spillway expansions and enhanced monitoring systems or controlled drawdown protocols. This approach allows for prioritization of upgrades not only based on current risk levels, but on project risk evolution over decades of climate change exposure. In this manner public risk analysis serves as a decision to guide incremental resilience investments while keeping operational flexibility under uncertainty.

5. Management and Policy Implications

Re-evaluation of Design floods

Many dams were designed for return periods that no longer match updated rainfall frequency analyses. Re assessment of probable maximum flood is critical in order to re-calculate flood magnitude versus return period to allow for more accurate risk assessment. (FEMA, 2018).

Spillway Capacity Upgrades

To safely convey extreme flow volumes, aging and undersized spillways need to be retrofitted (ASDSO,2020). This is done to meet updated hydrologic design criteria, including climate adjusted probable maximum flood estimates. Many existing spillways were design based on historical data that underestimate current peak flows, creating overtopping and downstream erosion risks. Retrofit measures may include channel enlargements and auxiliary spillways, fuse plugs, or reinforced chutes and are typically evaluated using advanced unsteady flow and overtopping simulations to ensure sufficient discharge capacity and compliance with modern dam safety standards.

Vegetation and Animal Control

Inspection and maintenance are essential in preventing burrowing animals and root intrusion that can start the internal erosion of the structure. Regular inspection and maintenance are critical for identifying internal erosion in embankment dams caused by burrowing animals and root intrusion. Here, early identification is extremely important. Left undetected, these internal erosional features can compromise the integrity of the structure, create preferential flow paths that concentrate seepage, increase local hydraulic gradients and initiate backward erosion piping, potentially leading to progressive internal failure if left unattended.

Inspections should include systematic surveys of the dam crest, slopes and downstream toe for evidence of burrows, vegetation encroachment, and surface cracking, while maintenance may involve filling or grouting burrows, controlling vegetation and removing deep rooted plants that could penetrate filter or drainage layers. Coupled with monitoring of seepage rates and pore pressures, these measures reduce risk of internal erosion by ensuring the health and long-term stability and safety of the embankment.

Real Time Monitoring and Early Warning

Instrumentation for pore water pressure, seepage, and reservoir levels, coupled with community emergency action plans is critical for high hazard dams (USACE, 2019). For high hazard dams, continuous instrumentation and monitoring are essential to detect early signs of instability and reduce failure risk. Piezometers, seepage weirs and reservoir level sensors provide real time data on pore water pressures and seepage and hydraulic loading allowing engineers to identify abnormal responses to rainfall or reservoir fluctuations. (USACE, 2019). These systems must be integrated with community-based action plans that have well-developed warning protocols, including inundation mapping and coordinated evacuation procedures ensuring both technical risk and public safety readiness.

Integration with Climate Adaption Frameworks

Watershed resilience strategies are being implemented, especially by agencies that increasingly recognize dams as part of broader watershed resilience, such as watershed resilience strategies, emphasizing green infrastructure, controlled releases and emergency drawdown capacity (USACE,2019). Modern dam management increasingly integrates watershed resilience strategies recognizing dams as critical components of broader hydrologic and ecological systems (USACE,2019). These strategies emphasize green infrastructure, such as wetlands and floodplain restoration to attenuate peak flows alongside controlled reservoir releases and enhanced emergency drawdown capacity to manage extreme events safely. By combining structural upgrades with watershed scale interventions, agencies aim to reduce flood risk and improve ecosystems services and enhance the adaptive capacity of both communities and infrastructure that is under the changing climate and land use conditions.

6. Conclusion

This report underscores that earth embankment dams that are increasingly exposed to hydrologic loads that are above their original design assumptions due to climate change. Compounded risks of overtopping, slope failure and piping demand require integrated engineering solutions that combine updated rainfall estimates, in-depth geotechnical modeling, improved spillway design, proactive maintenance and community preparedness to safeguard lives and property. This research highlights an urgent need for collaboration among engineers in the field, policy makers, and local organizations and communities to ensure dam infrastructure remains resilient under intensifying climate extremes.

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Notes

ORVSS LV

Presentation 8

Cellular Cofferdam Designs For Adverse Foundation Conditions

Daniel Lund, P.E.

USACE Louisville District

US Army Corps of Engineers®
Louisville District

Cofferdams for Civil Works Projects: Sliding Failure Case Histories, and Designing for Challenging Foundation Conditions

Daniel Lund, P.E., USACE Louisville District

Abstract: The inland navigation system of the United States is an economic boon to the nation allowing cost-effective transportation of high-volume bulk goods that would otherwise encumber rail and highway networks. The U.S. Army Corps of Engineers owns, operates, and maintains the navigation locks that are integral to the reliability of the inland waterways system. As new locks are built to increase capacity or replace aging infrastructure, cellular cofferdams are often employed as a means to facilitate a dewatered site for construction. This presentation will provide case histories of cofferdam failures at Civil Works projects in the Ohio River basin and provide examples of modern cofferdam designs that mitigated similar foundation concerns.

Several cofferdam sliding failures have occurred at civil works project within the Ohio River basin due to weak planes beneath the structure. At Louisville & Portland Canal in 1915 an original canal wall used as part of a cofferdam failed on natural seams below the foundation; important factors in the failure included overestimating resistance to sliding, construction activities adversely impacting stability, and inadequate attention to signs of distress prior to failure. At Wheeler Lock and Dam in 1961 an original lock wall used as part of a cofferdam failed on a weak clay seam beneath the base of the monolith; important factors in the failure included missing a thin clay seam in the investigations for both the original and new lock, and construction activities adversely affecting stability. At Cannelton Lock and Dam in 1967 a cellular cofferdam suffered a progressive sliding failure on a bedding plane at the base of a clayey shale; important factors in the failure included overestimating sliding resistance at the base of the shale, construction activities altering the stress regime at the site, and inadequate attention to signs of distress prior to failure. At Uniontown (now JT Myers) Lock and Dam in 1971 a cellular cofferdam failed due to deep-seated sliding on a coal seam with underclay; important factors in the failure included overestimating sliding resistance of a coal underclay, and failure to recognize faults reducing the size of passive wedges. As a result of these and other failures, the Army Corps of Engineers has designed all major cofferdams on USACE projects since the 1970s.

Modern cofferdam designs for USACE Civil Works projects have applied lessons learned from past failures and addressed similar deep-seated stability concerns in several ways. Stabilization methods have included micropiles installed through sheet pile cells to address solution features, and post-tensioned anchors installed through existing lock wall monoliths to address shear zones and clay seams, and pipe piles installed through sheet pile cells to transfer lateral load to sound rock

beneath weak layers. This presentation will discuss the cofferdam design for Montgomery Lock, a lock replacement project currently under construction on the Ohio River in western Pennsylvania. The instability mechanism of concern for the Montgomery cofferdam is deep-seated sliding on a weak layer of coal typically found 5 feet below the top of rock. To increase stability of the cofferdam a system of 24-in x 1-in steel pipe piles will be placed through the bottom of sheet pile cofferdam cells, socketed and grouted into rock, will be used to transfer lateral load from the cofferdam into the competent rock below the weak coal layer.

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COFFERDAMS FOR CIVIL WORKS PROJECTS:
SLIDING FAILURE CASE HISTORIES, AND DESIGNING FOR CHALLENGING FOUNDATION CONDITIONS

ORVSS LV: Civil Works

Daniel Lund, P.E.
Geotechnical Engineer
USACE - Louisville District

12 November 2025

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US INLAND WATERWAYS SYSTEM

- ➢ 12,000 miles of navigable waters
- ➢ Nearly 500 million tons per year
- ➢ Over $150 billion per year
- ➢ 192 navigation lock projects

Fuel-Taxed Inland Waterways Systems

Source: U.S. Army Corps of Engineers

← 2025 ASCE Infrastructure Report Card

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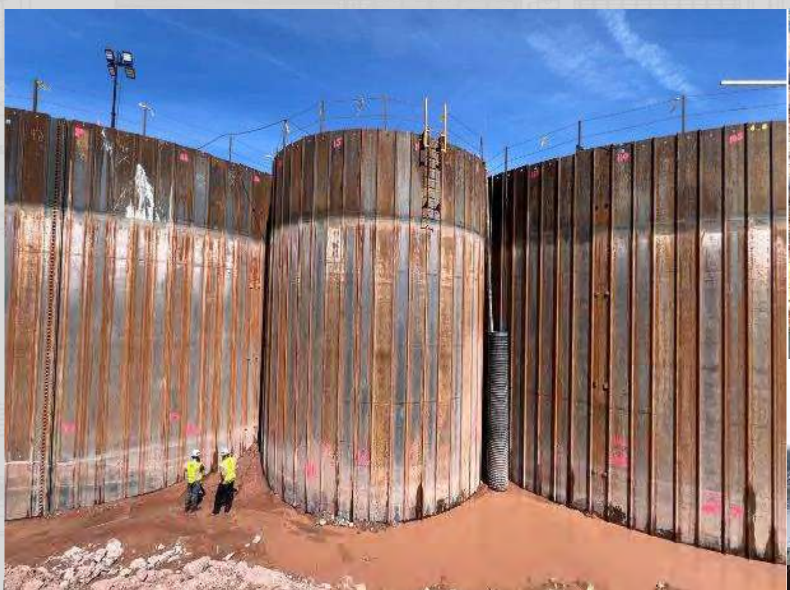

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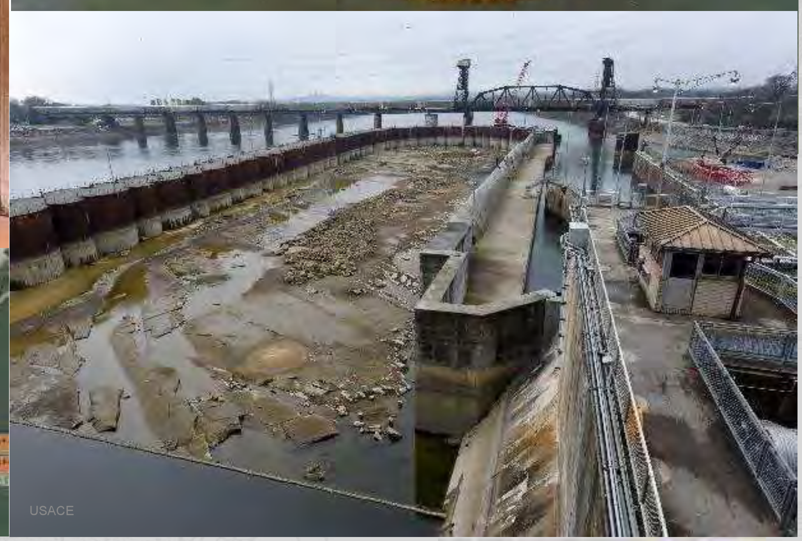
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AGENDA

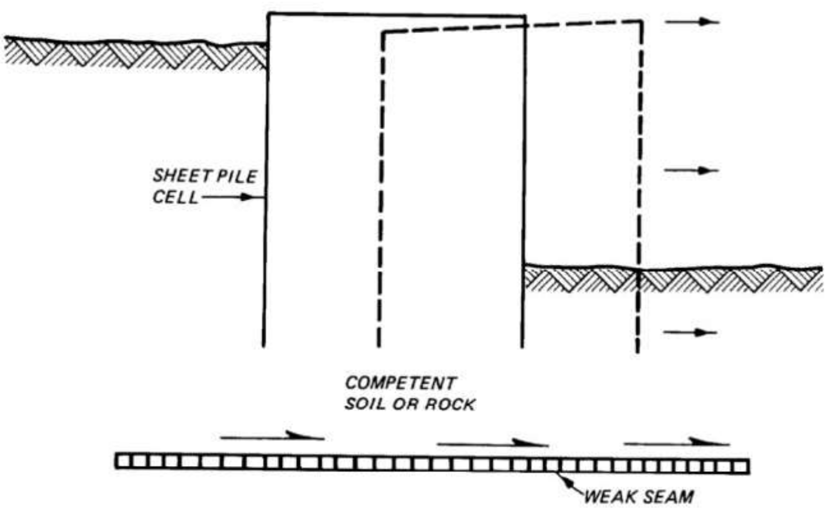
1) Case Histories – Ohio River Basin Cofferdam Failures

2) Common Causes / Lessons Learned

3) Case Study of Montgomery Lock Cofferdam Design

CASE HISTORIES OF COFFERDAM FAILURES

1) Louisville & Portland Canal
2) Wheeler Lock & Dam
3) Cannelton Lock & Dam
4) Uniontown Lock & Dam



USACE EM 1100-2-2503

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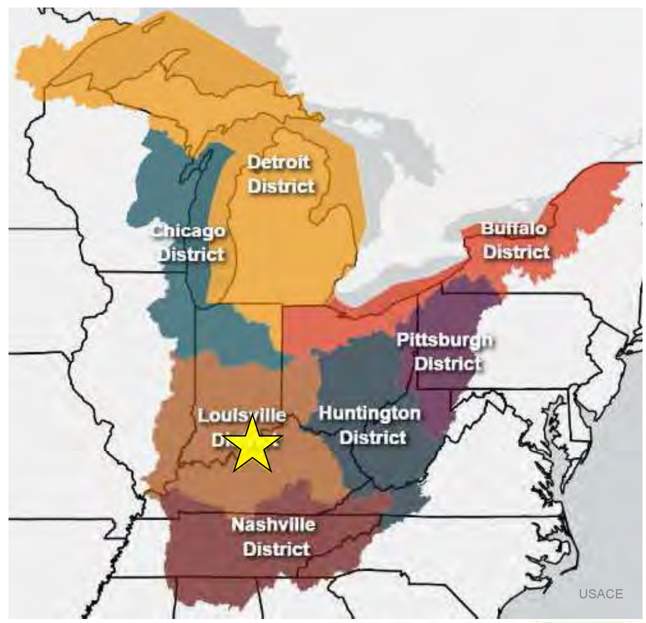
LOUISVILLE & PORTLAND CANAL

- 1826: 64 feet wide
- 1870s: widened to 87 feet
- 1915: widened to 200 feet



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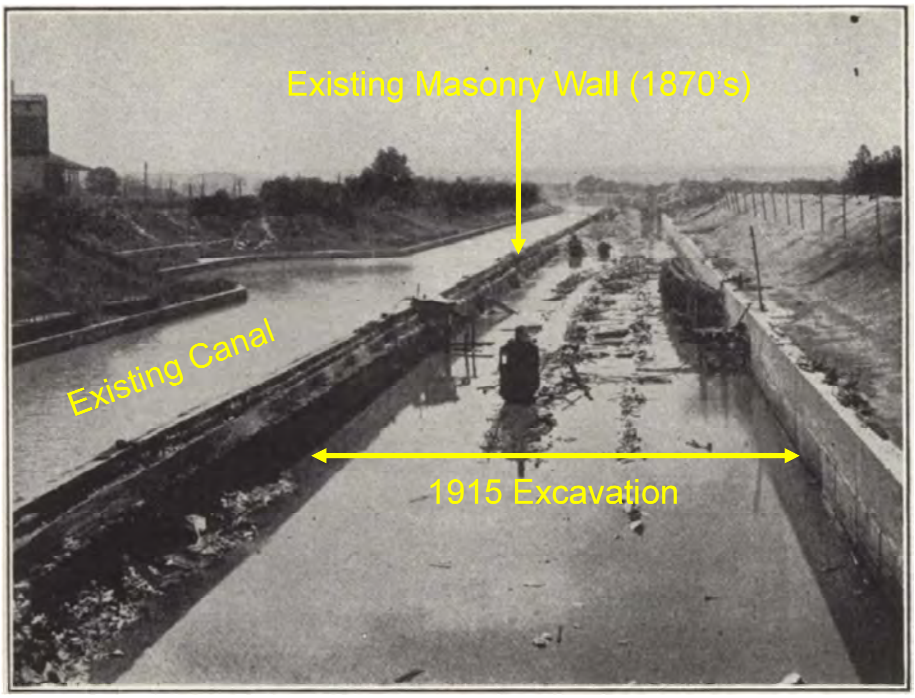
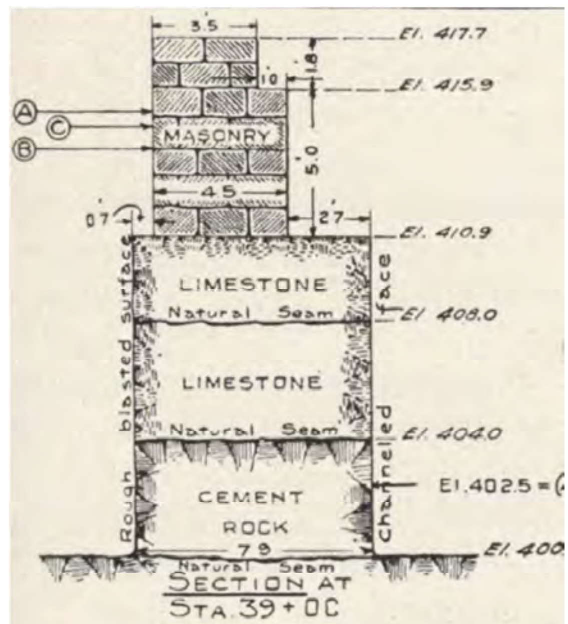
L & P CANAL – 1915 EXPANSION





Oakes 1916

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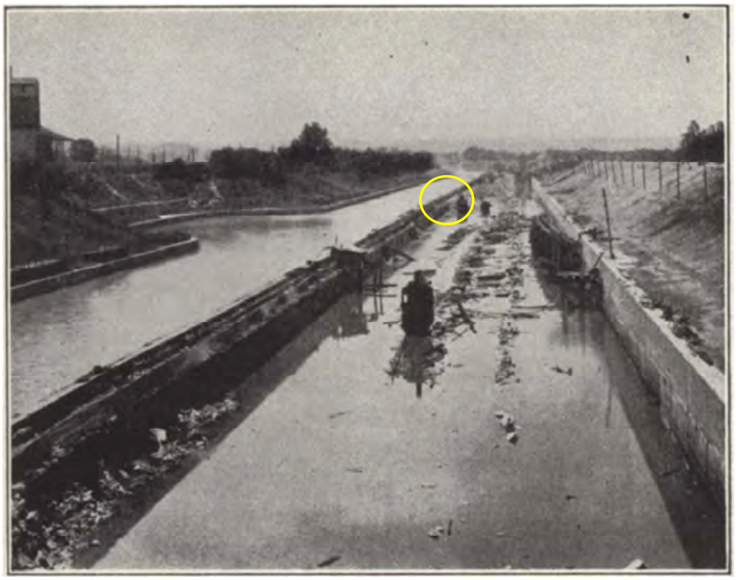
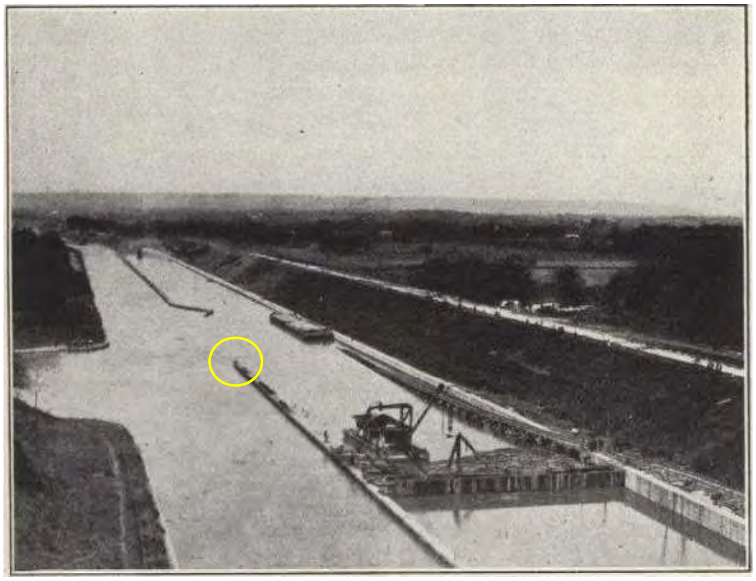
L & P CANAL – COFFERDAM FAILURE





- Failure occurred 2 June 1961
- 1 person was killed

Oakes 1916

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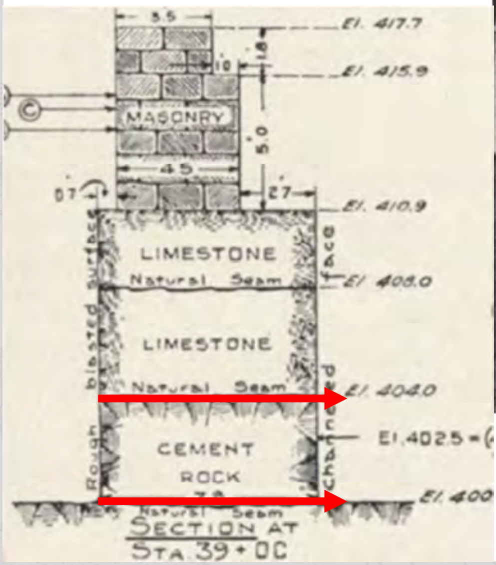
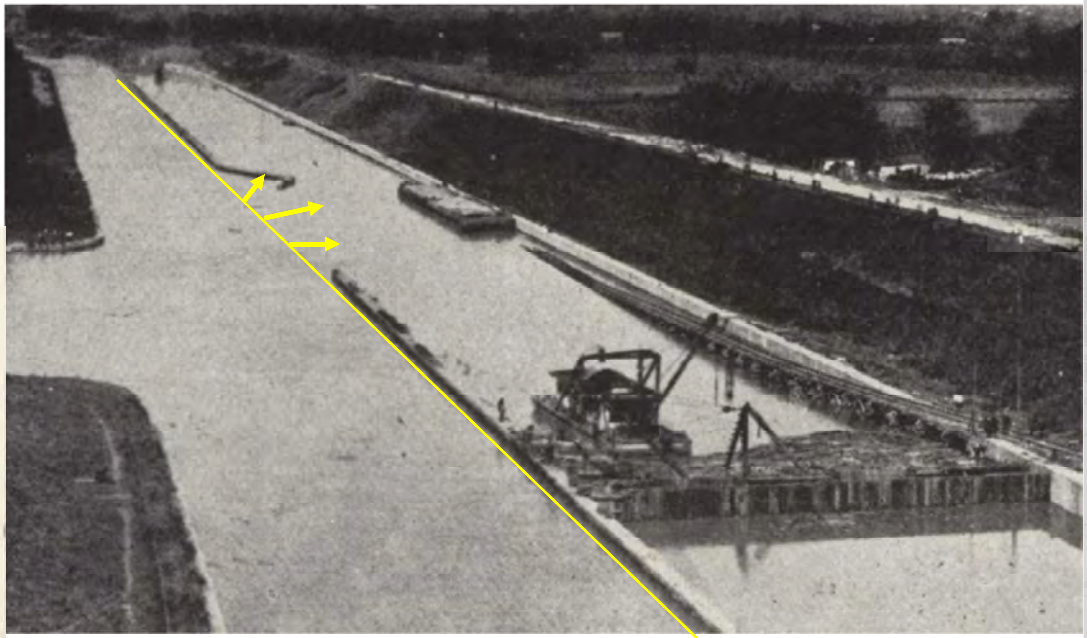
L & P CANAL – COFFERDAM FAILURE

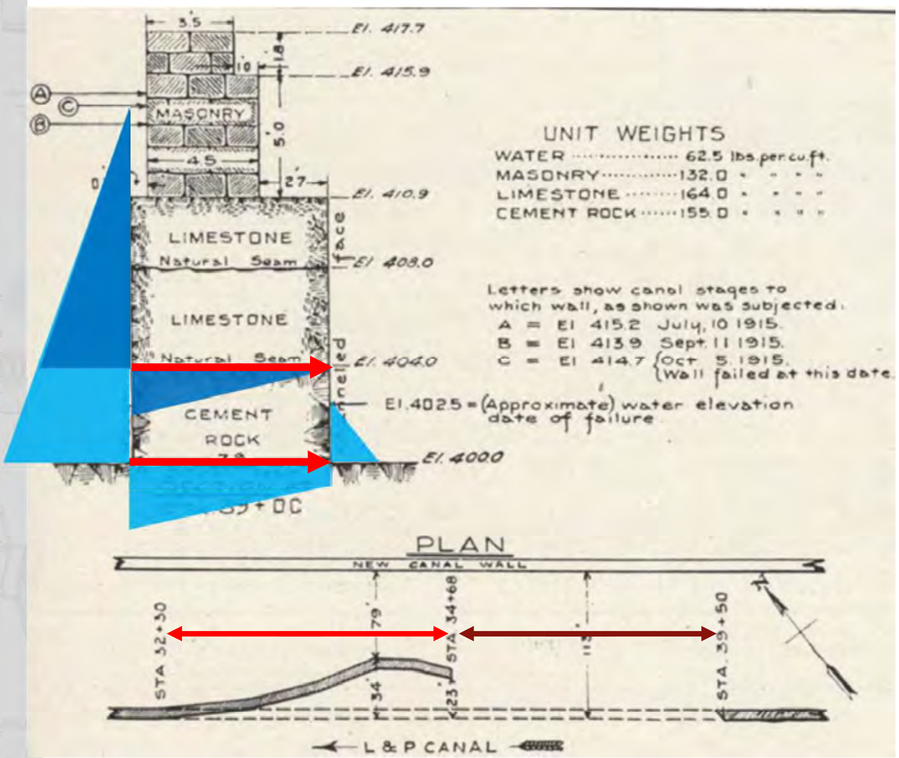
- Slid 34 feet
- Two sliding planes

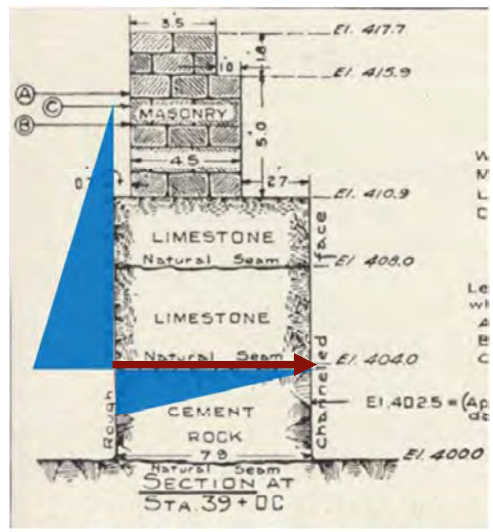




Oakes 1916

L & P CANAL – FAILURE DETAILS





Oakes 1916

L & P CANAL FAILURE – IMPORTANT FACTORS

1) Investigations
- Some borings identified clay seam at EL 404 but this was not represented in phi angle used in analysis.

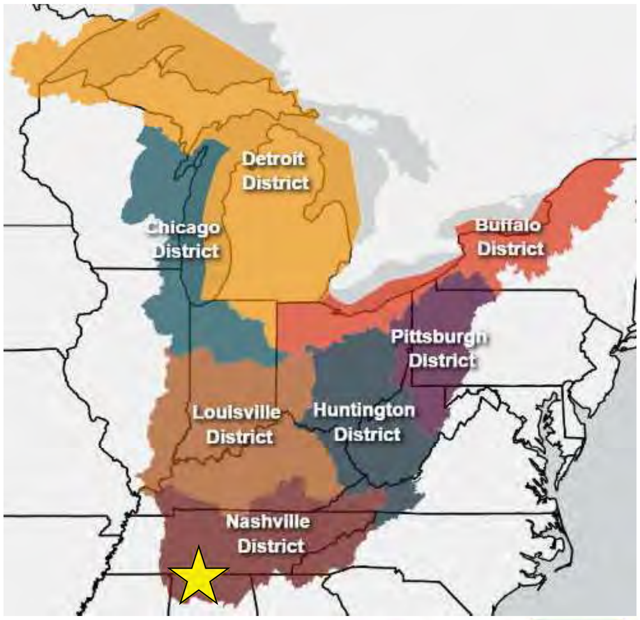
2) Construction activity
- Blasting may have damaged rock ledge leading to post-peak rock strength.
- Attempts to stop seepage from exiting on dry side would have increased uplift pressure.

3) Instrumentation and monitoring
- Missed significance of baseline shift

WHEELER LOCK & DAM

- 1934: Original Lock Constructed
- 1961: New Land Chamber Begins

TVA 1961



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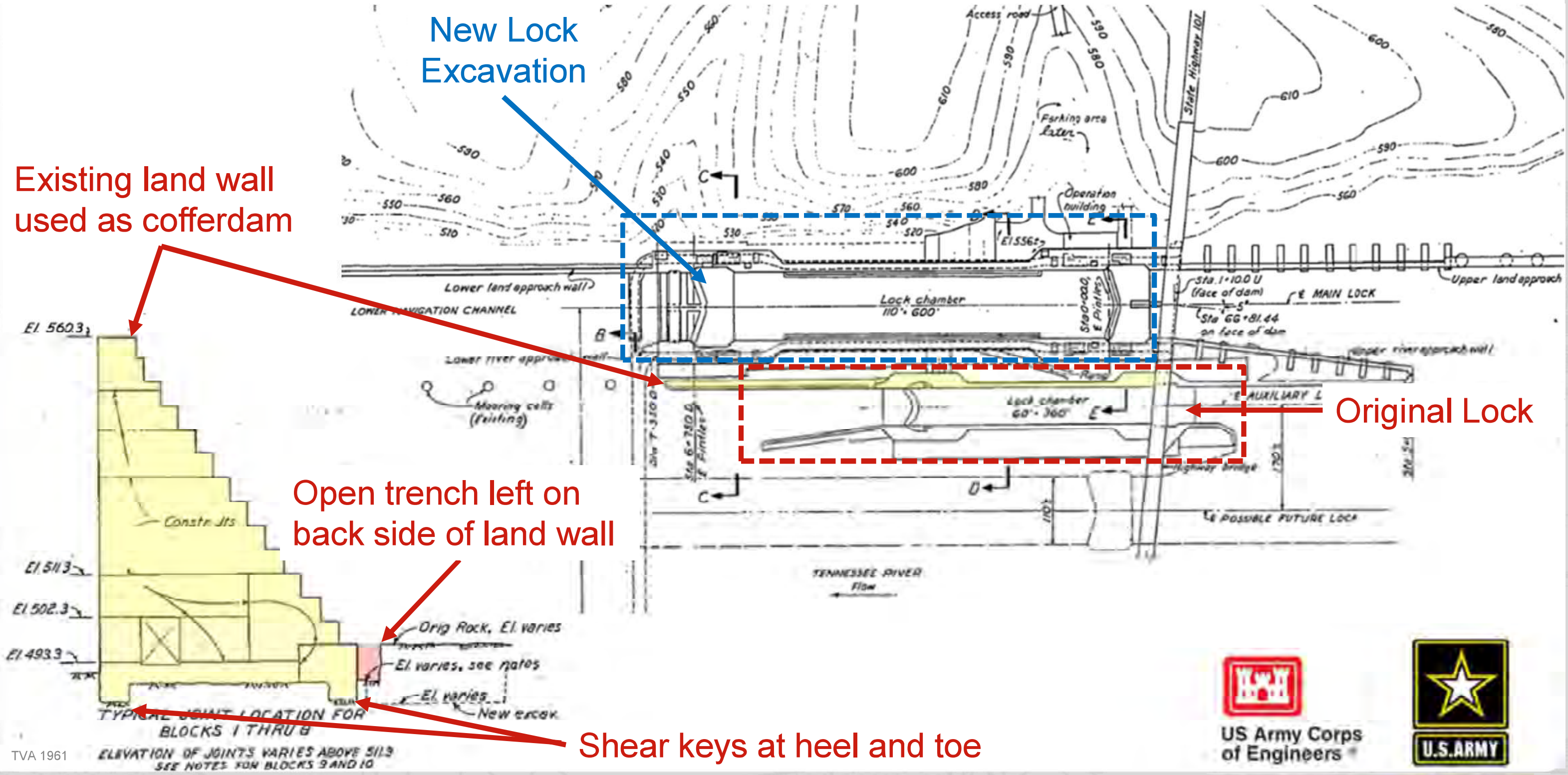
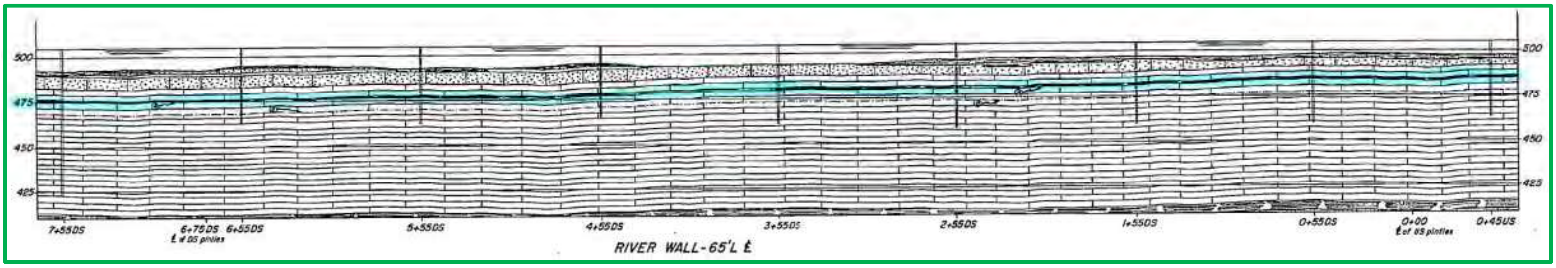
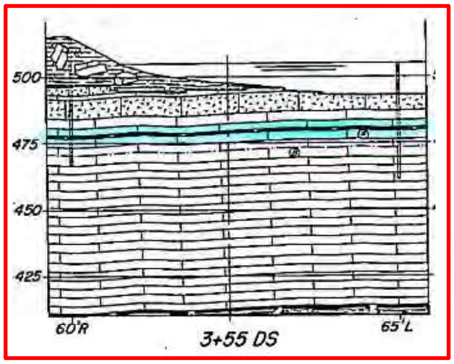
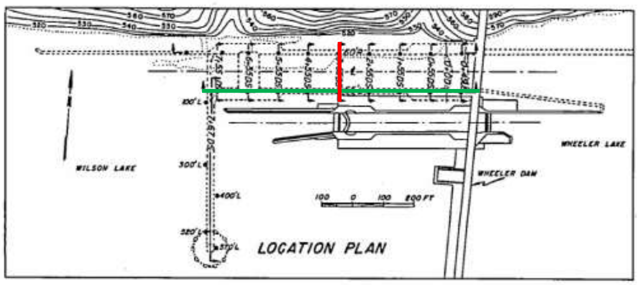
WHEELER LOCK & DAM – 2ND LOCK CONSTRUCTION



WHEELER LOCK & DAM GEOLOGY







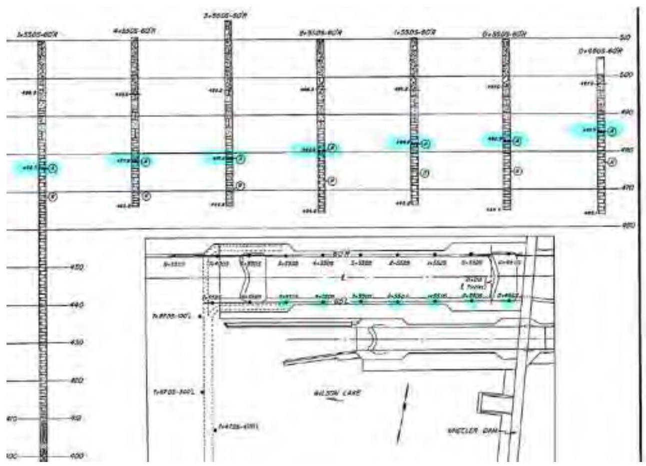
TVA 1961

WHEELER LOCK FOUNDATION EXPLORATION

CONCLUSIONS

Exploratory drilling in the area proposed for the new lock at the Wheeler project has shown that no serious foundation problems should be encountered in the construction of the lock. Cores showed the rock to be sound and unweathered throughout. Inasmuch as the foundation rock is

During the construction of Wheeler Dam the final cleanup of the foundation blocks was deferred until just prior to pouring concrete. It was found that over most of the foundation if final cleanup was done too soon the exposed rock surfaces would slake and crack from alternate expansion and contraction. The only strata which is expected to disintegrate markedly upon exposure to air is key bed "A" (Exhibits 1 and 2). This thin - 0.5' average thickness - horizon was the only one to show any signs of disintegration in the cores. The other cores recovered from the exploratory holes



WHEELER LOCK FAILURE

- Failure occurred 2 June 1961
- 2 people were killed

TVA 1961

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WHEELER LOCK FAILURE INVESTIGATION

Foundation inspection in mid-June provided no clue to the reason for the unusual weakness of the shale band and it was decided that structural tests would be essential to learn the reason for its inadequate strength. The first step in this program was laboratory testing. The results showed strengths much above that needed to have withstood failure, but the techniques available for cutting the test samples from the foundation could not obtain a short section near the base of the shale band. It was therefore decided to attempt structural tests on the rock in place.

While steps were being taken for field tests of the strength of the foundation, rock excavation to levels below the shale band was going on in areas of the lock walls and in a 36-inch pilot hole needed to initiate excavation for the main lock discharge structure. Early in September some of these areas were pumped dry and an engineer inspecting a rock face which exposed the shale band found the thin seam of plastic clay. Steps were taken immediately to determine the extent of this clay. New 36-inch calyx holes, noted in the consultants' report, were a part of this investigation. These foundation checks, combined with facts already available, established that:

(1) The plastic clay seam existed essentially throughout the lock area.

(2) The clay seam was so nearly in a plane and so nearly horizontal that it could easily lubricate and be a sliding surface for the rock immediately above and below.

(3) Under the walls of the old lock the clay seam was at an elevation which made it particularly critical during excavation for the new lock.

This information established the reason for the failure. Accordingly,

TVA 1961

- Failure on 2 June
- Mid-June, determined sliding within shale band
- September, thin clay seam found at base of shale

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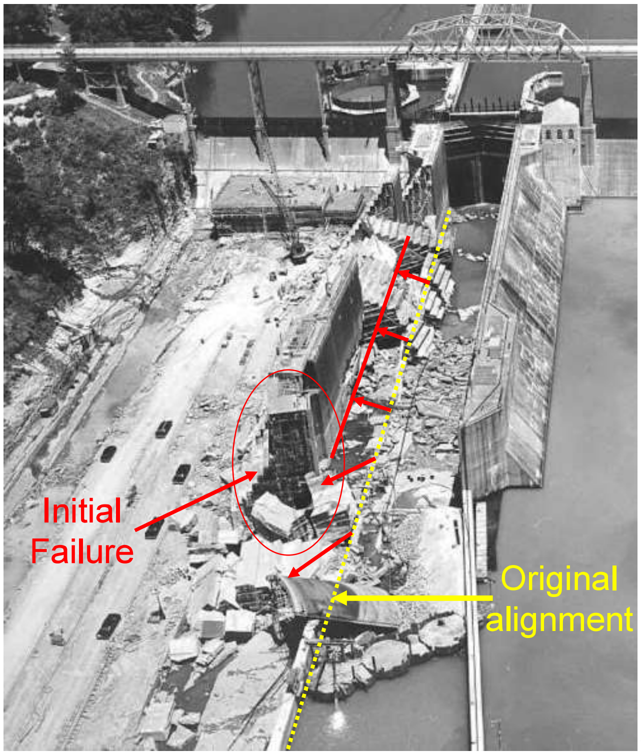
WHEELER LOCK FAILURE REPORT

No record of the clay seam has been found in the reports on the original lock construction. In the new investigations before construction no evidence of the clay seam showed up in the cores of any of the NX holes and the visual inspection of the faces of the calyx holes likewise failed to reveal its existence. The clay seam was too thin to show up in the NX holes. It was not detected in the calyx holes because it has the same color as the contiguous shale and both the shale and the seam were recessed by the grinding action of the chilled shot used as the cutting medium. Accordingly, the stability of the existing land wall was evaluated on the assumption that the cores from the drill holes correctly reflected the condition of the underlying foundation, and the steps taken in constructing the new lock were predicated on the stability of the old lock wall as thus evaluated.

TVA 1961

WHEELER LOCK FAILURE DETAILS



TVA 1961

- Slid on clay seam 1/16” to 3/8” thick at base of shale
- Failure initiated at downstream pintle block
- Downstream guide wall monoliths followed almost immediately
- Failure then propagated rapidly upstream

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WHEELER LOCK FAILURE

TVA 1961

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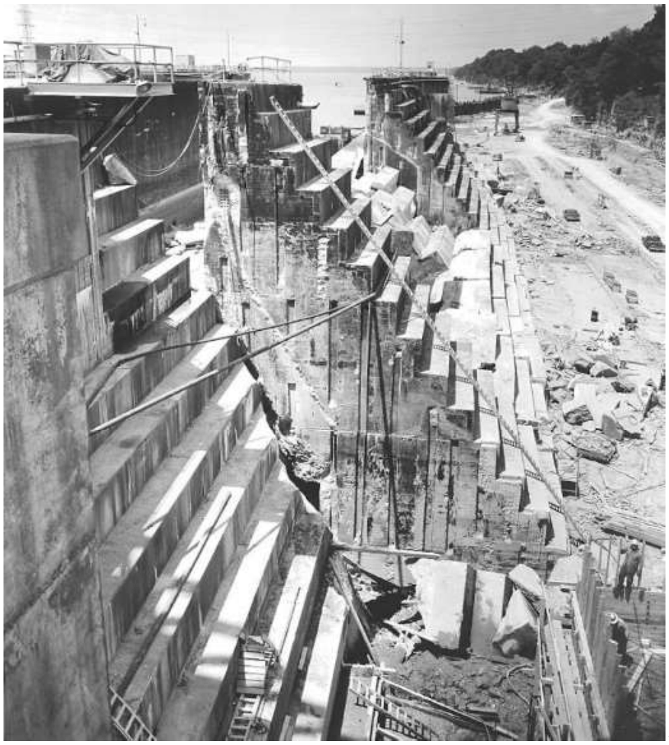
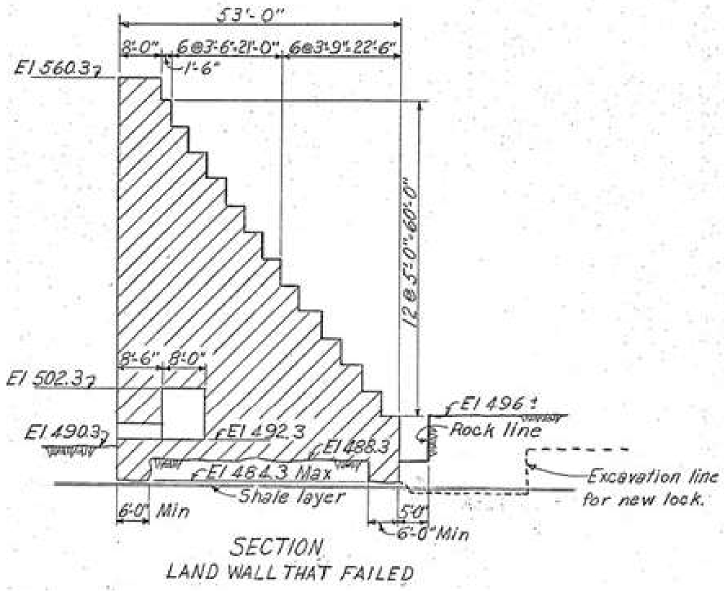
WHEELER LOCK FAILURE



TVA 1961



WHEELER LOCK FAILURE – IMPORTANT FACTORS

1) Investigations
- Missed clay seam at base of shale leading to overestimated sliding resistance

2) Construction Activity
- Excavation behind wall daylighted clay seam and removed passive wedge
- Possible blasting damage

CANNELTON LOCK & DAM

- 1963: Lock construction begins
- 1965: Dam construction begins
- 1966: Locks operational

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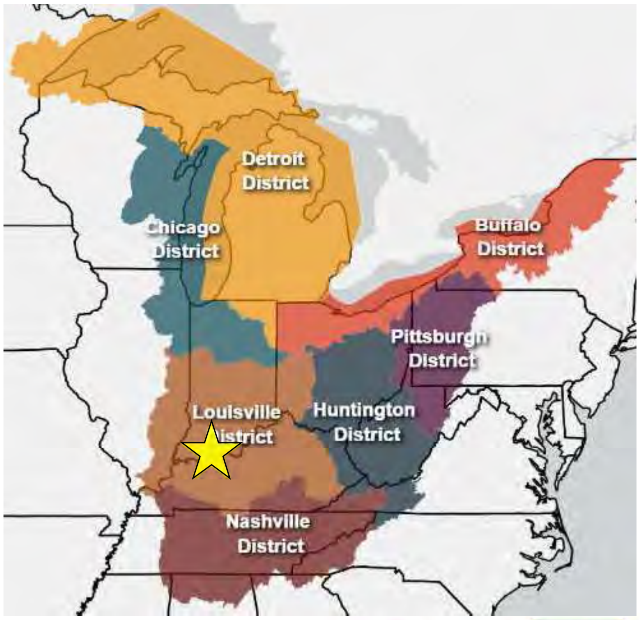

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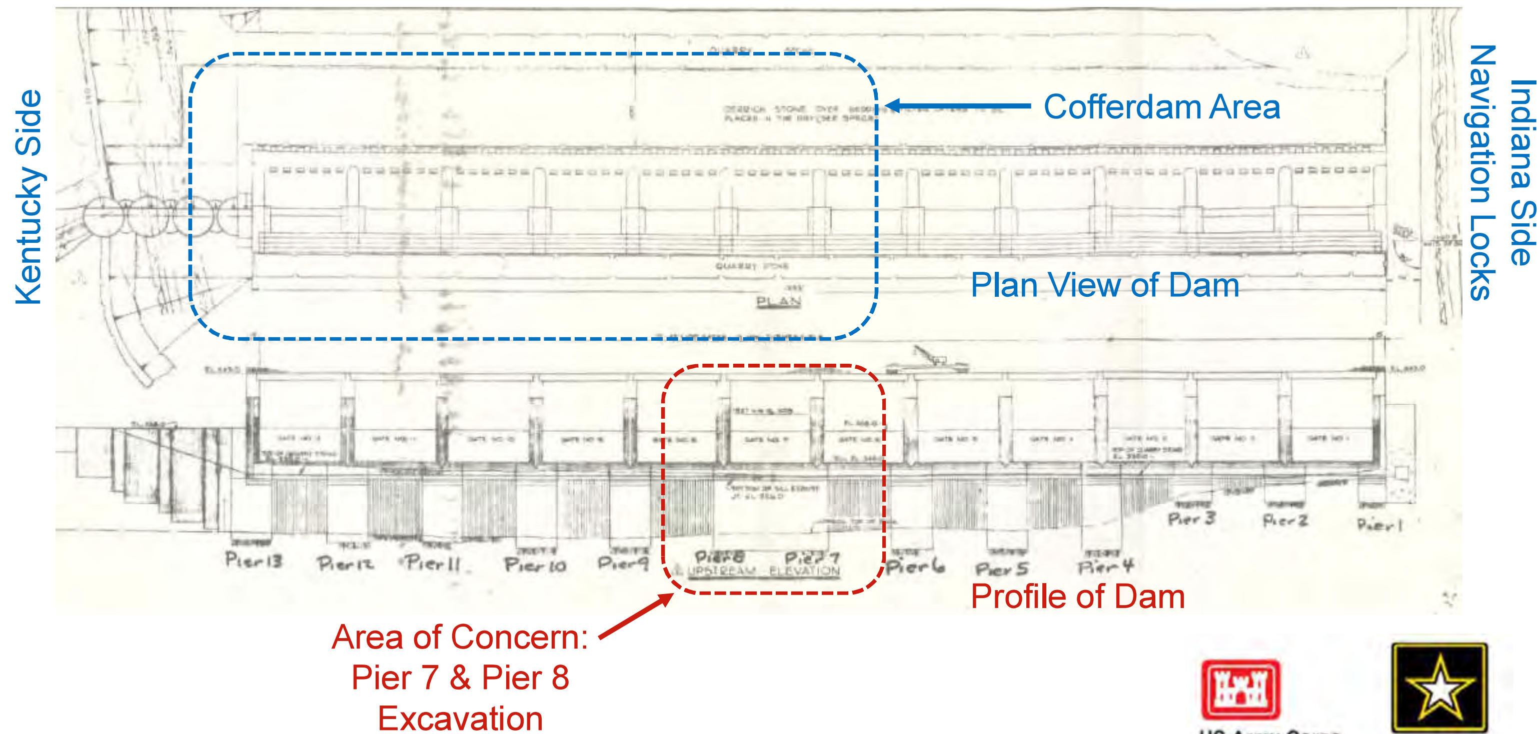
CANNELTON DAM



Fetzer 1975

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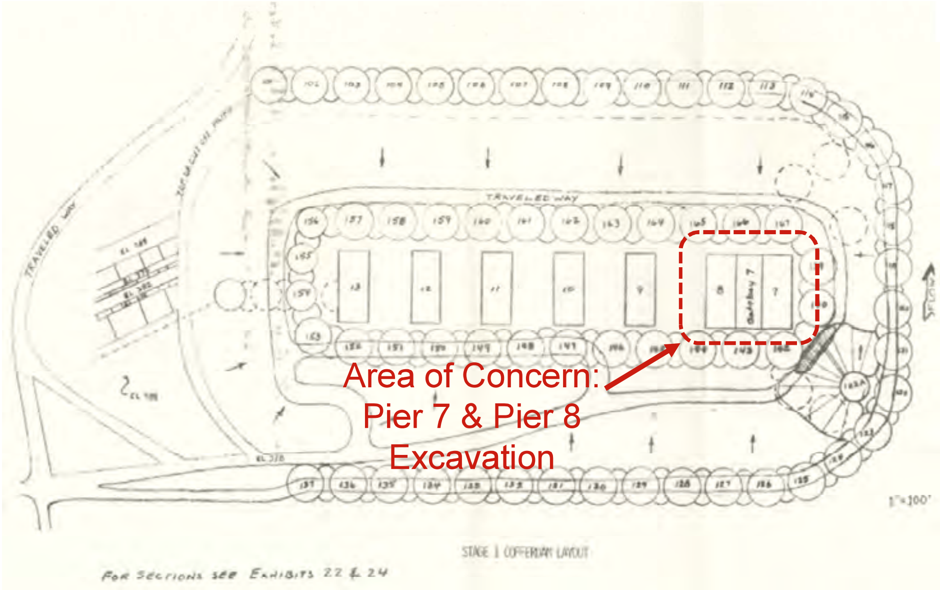
CANNELTON STAGE I COFFERDAM

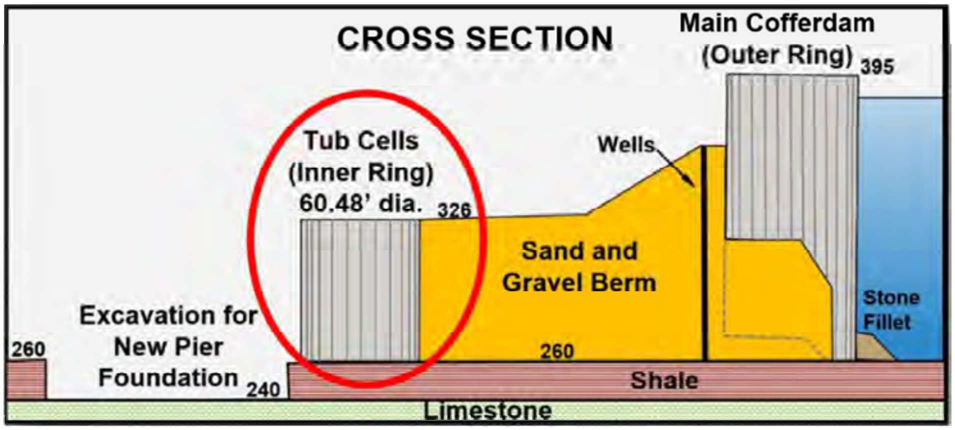
- Double-ringed cofferdam
- Outer ring – main cofferdam
- Inner ring – shorter "tub" cells
- Stability berm between rings



Fetzer 1975

CANNELTON COFFERDAM SECTION

- Inner ring retained stability berm
- Toe of inner ring very close to excavation face



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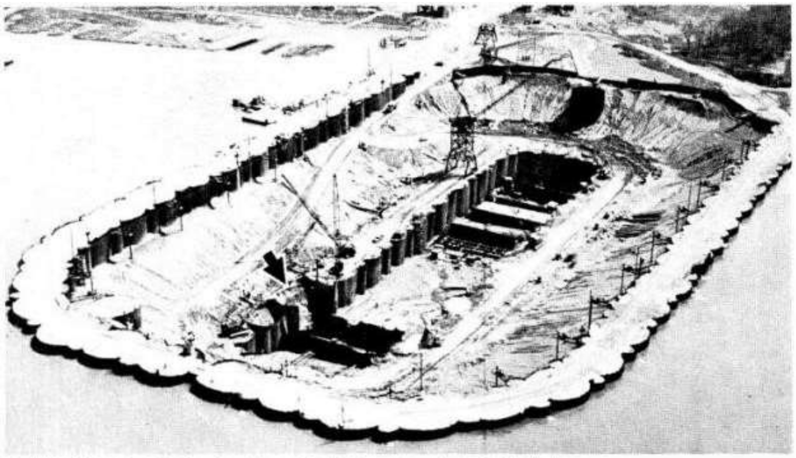
CANNELTON COFFERDAM FAILURE

FAILURE AFTERMATH (arrow). Flooding and sand dumping stabilized cells.



COFFERDAM in the Ohio River is a 20-acre, 160-ft hole; arrow marks ruptured cell, slippage area.

- Failure occurred 1 Nov 1967
- No fatalities (crew on break)

Fetzer 1975

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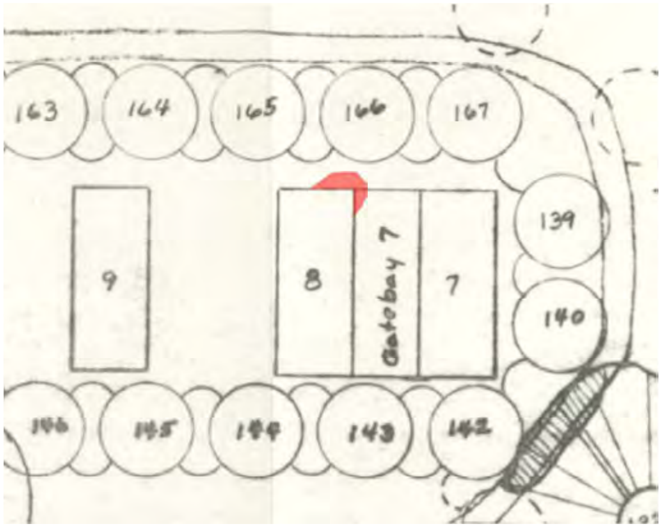
CANNELTON COFFERDAM FAILURE TIMELINE

- 12-17 OCT: Pier 8 blasting & excavation
- 18 OCT: Pier 8 excavation 2-3' from grade, Shale at DS IN corner sloughing in, lots of leakage thru shale at DS end.
- 19 OCT: KY side of Pier 7 pre-split. Spalling continues at upper 4 ft around Pier 8 excavation.
- 20 OCT: Most of Pier 7 pre-split. Sloughing in DS end of Pier 8 continues.



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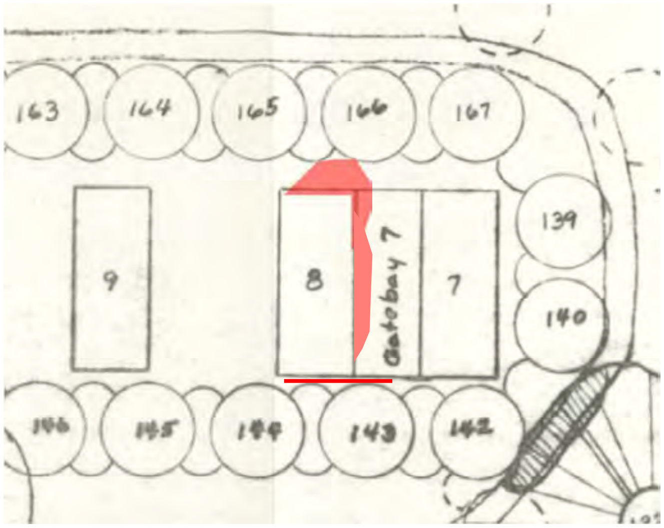
CANNELTON COFFERDAM FAILURE TIMELINE

- 21 OCT
  - Sloughing reaches toe of cell 166
  - Cracks observed in unexcavated area between Pier 7 & 8, and toe of cell 143
  - Shale between Pier 7 & 8 moves laterally 2-3 feet into Pier 8 excavation (along previously observed cracks)
  - Crack at toe of cell 143 extends to arc 143/144 and opens to width of 2-4”
- 22 OCT: Pier 8 excavation flooded to stop caving of sides
- 24 OCT: Crack at toe of cell 143 widens, depth increases to 6-8’



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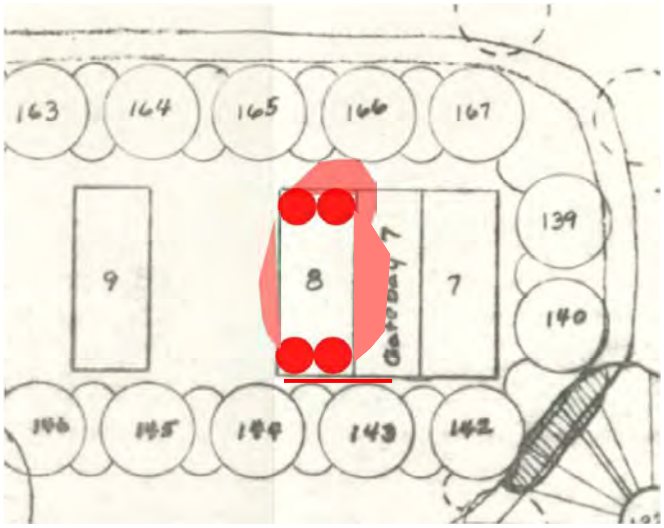
CANNELTON COFFERDAM FAILURE TIMELINE

- 25-27 OCT: 20-ft diameter cells placed in both ends of Pier 8 excavation
- 27 OCT: Sheet piling driven along Pier 7 perimeter pre-split line. Spalling visible along KY wall of Pier 8.
- 29-30 OCT: Water pumped out of Pier 8. Sloughing continues along KY side of Pier 8 and blocks falling out of IN side.
- 31 OCT: Large block of shale falls from IN side covering an end loader. IN wall sloped back to stop sloughing of shale blocks. Spalling around Pier 8 continues



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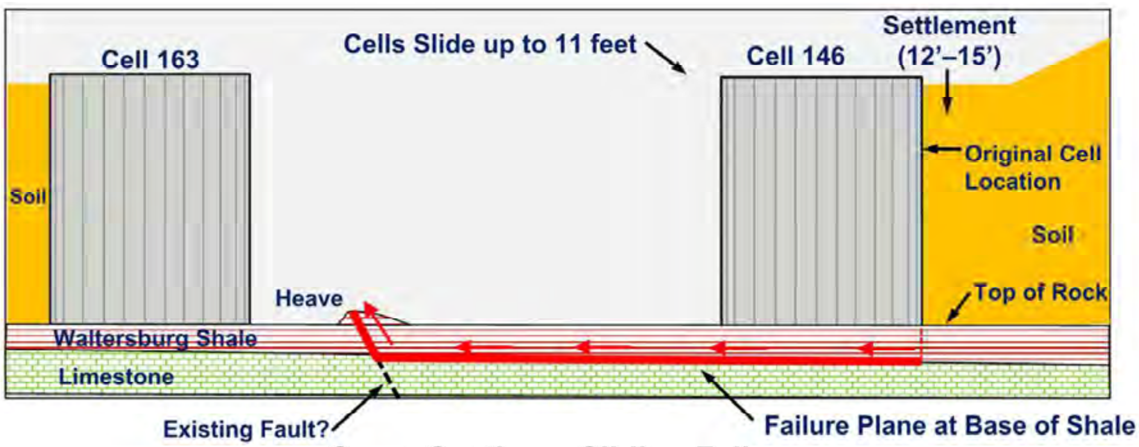
CANNELTON FAILURE CROSS SECTION



Cross Section – Sliding Failure

- 1 NOV: Cells 144-149 slide up to 11 feet DS. Cell 143 fails suddenly by rupturing along T-pile connection to adjacent arc. Subsidence in berm. Upheaved zone in excavation

Nield 2019

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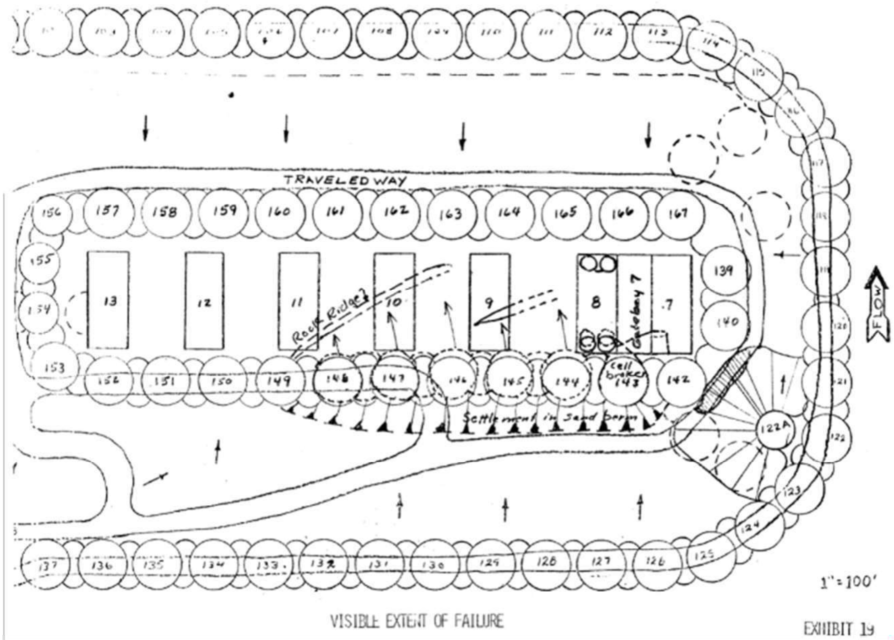
CANNELTON FAILURE PLAN VIEW



Fetzer 1975

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CANNELTON FAILURE – IMPORTANT FACTORS

1) Design assumptions
- Overestimated sliding resistance at base of shale

2) Construction activity
- Altered stress regime from high vertical / low horizontal stresses to low vertical / high horizontal stresses due to excavation, dewatering, and imposition of loads from cofferdam.

3) Instrumentation and monitoring
- Signs of distress for at least 10 days prior to failure

UNIONTOWN DAM LOCKS AND DAM

- 1965: Lock Construction Begins
- 1969: Locks Operational
- 1970: Dam Construction Begins
- 1977: Dam Operational

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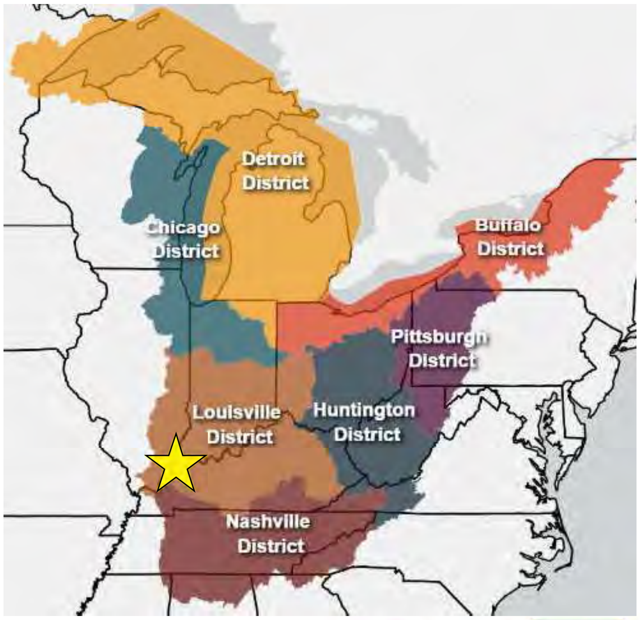

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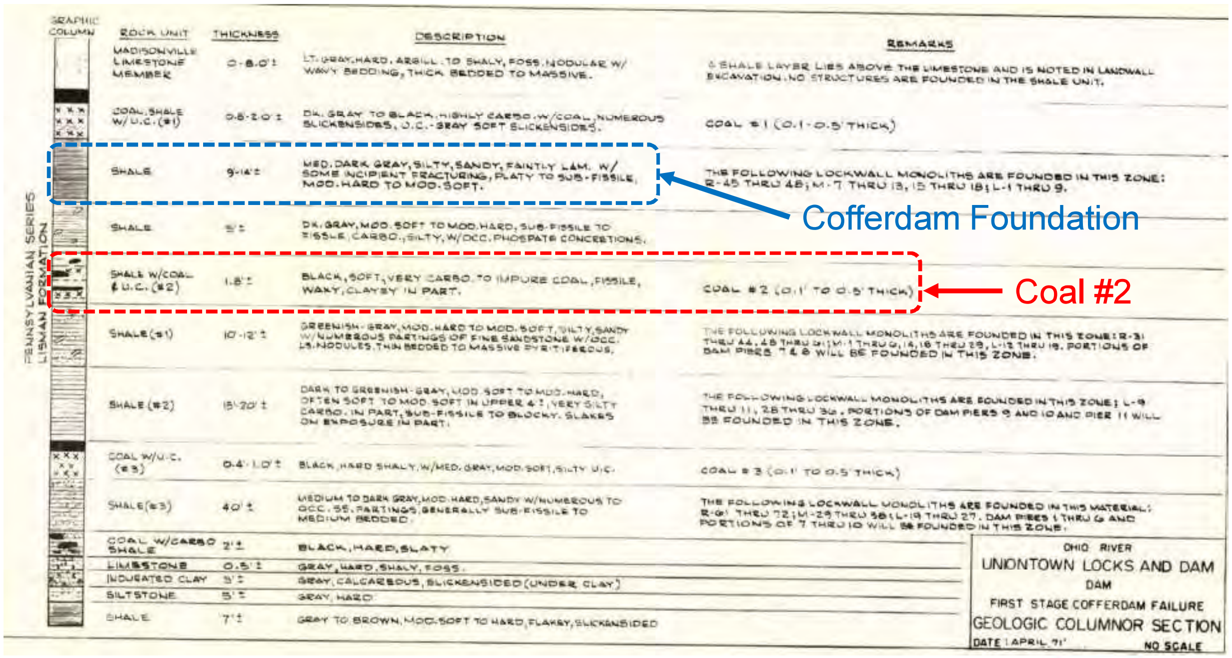
UNIONTOWN DAM GEOLOGIC COLUMN



GRAPHIC COLUMN	ROCK UNIT	THICKNESS	DESCRIPTION	REMARKS
	MADISONVILLE LIMESTONE MEMBER	0-8.0'±	LT. GRAY, HARD, ARGILL. TO SHALY, FOSS. NODULAR W/ WAVY BEDDING, THICK BEDDED TO MASSIVE.	A SHALE LAYER LIES ABOVE THE LIMESTONE AND IS NOTED IN LANDWALL EXCAVATION. NO STRUCTURES ARE FOUNDED IN THE SHALE UNIT.
	COAL, SHALE W/ U.C. (#1)	0.5-2.0'±	DK. GRAY TO BLACK, HIGHLY CARBO. W/COAL, NUMEROUS SLICKENSIDES, U.C. - GRAY SOFT SLICKENSIDES.	COAL #1 (0.1-0.5' THICK)
	SHALE	9-14'±	MED. DARK GRAY, SILTY, SANDY, FAINTLY LAM. W/ SOME INCIPIENT FRACTURING, PLATY TO SUB-FISSILE, MOD. HARD TO MOD. SOFT.	THE FOLLOWING LOCKWALL MONOLITHS ARE FOUNDED IN THIS ZONE: R-45 THRU 48; M-7 THRU 13, 15 THRU 18; L-1 THRU 9.
	SHALE	5'±	DK. GRAY, MOD. SOFT TO MOD. HARD, SUB-FISSILE TO FISSLE, CARBO., SILTY, W/OCC. PHOSPATE CONCRETIONS.	
	SHALE W/COAL & U.C. (#2)	1.8'±	BLACK, SOFT, VERY CARBO. TO IMPURE COAL, FISSILE, WAXY, CLAYEY IN PART.	COAL #2 (0.1' TO 0.5' THICK)
	SHALE (#1)	10'-12'±	GREENISH-GRAY, MOD. HARD TO MOD. SOFT, SILTY, SANDY W/NUMEROUS PARTINGS OF FINE SANDSTONE W/OCC. LS. NODULES. THIN BEDDED TO MASSIVE PYRITIFEROUS.	THE FOLLOWING LOCKWALL MONOLITHS ARE FOUNDED IN THIS ZONE: R-31 THRU 44, 48 THRU 61; M-1 THRU 6, 14, 16 THRU 28, L-12 THRU 18. PORTIONS OF DAM PIERS 7 & 8 WILL BE FOUNDED IN THIS ZONE.
	SHALE (#2)	15'-20'±	DARK TO GREENISH-GRAY, MOD. SOFT TO MOD. HARD, OFTEN SOFT TO MOD. SOFT IN UPPER 4'±, VERY SILTY CARBO. IN PART, SUB-FISSILE TO BLOCKY. SLAKES ON EXPOSURE IN PART.	THE FOLLOWING LOCKWALL MONOLITHS ARE FOUNDED IN THIS ZONE: L-9 THRU 11, 28 THRU 36. PORTIONS OF DAM PIERS 9 AND 10 AND PIER 11 WILL BE FOUNDED IN THIS ZONE.
	COAL W/U.C. (#3)	0.4'-1.0'±	BLACK, HARD SHALY, W/MED. GRAY, MOD. SOFT, SILTY U.C.	COAL #3 (0.1' TO 0.5' THICK)
	SHALE (#3)	40'±	MEDIUM TO DARK GRAY, MOD. HARD, SANDY W/NUMEROUS TO OCC. SS. PARTINGS, GENERALLY SUB-FISSILE TO MEDIUM BEDDED.	THE FOLLOWING LOCKWALL MONOLITHS ARE FOUNDED IN THIS MATERIAL: R-61 THRU 72; M-29 THRU 58; L-19 THRU 27. DAM PIERS 1 THRU 6 AND PORTIONS OF 7 THRU 10 WILL BE FOUNDED IN THIS ZONE.
	COAL W/CARBO SHALE	2'±	BLACK, HARD, SLATY	
	LIMESTONE	0.5'±	GRAY, HARD, SHALY, FOSS.	
	INDURATED CLAY	3'±	GRAY, CALCAREOUS, SLICKENSIDED (UNDER CLAY)	
	SILTSTONE	5'±	GRAY, HARD	
	SHALE	7'±	GRAY TO BROWN, MOD. SOFT TO HARD, FLAKEY, SLICKENSIDED	

OHIO RIVER
UNIONTOWN LOCKS AND DAM
DAM
FIRST STAGE COFFERDAM FAILURE
GEOLOGIC COLUMNOR SECTION
DATE: APRIL 71' NO SCALE

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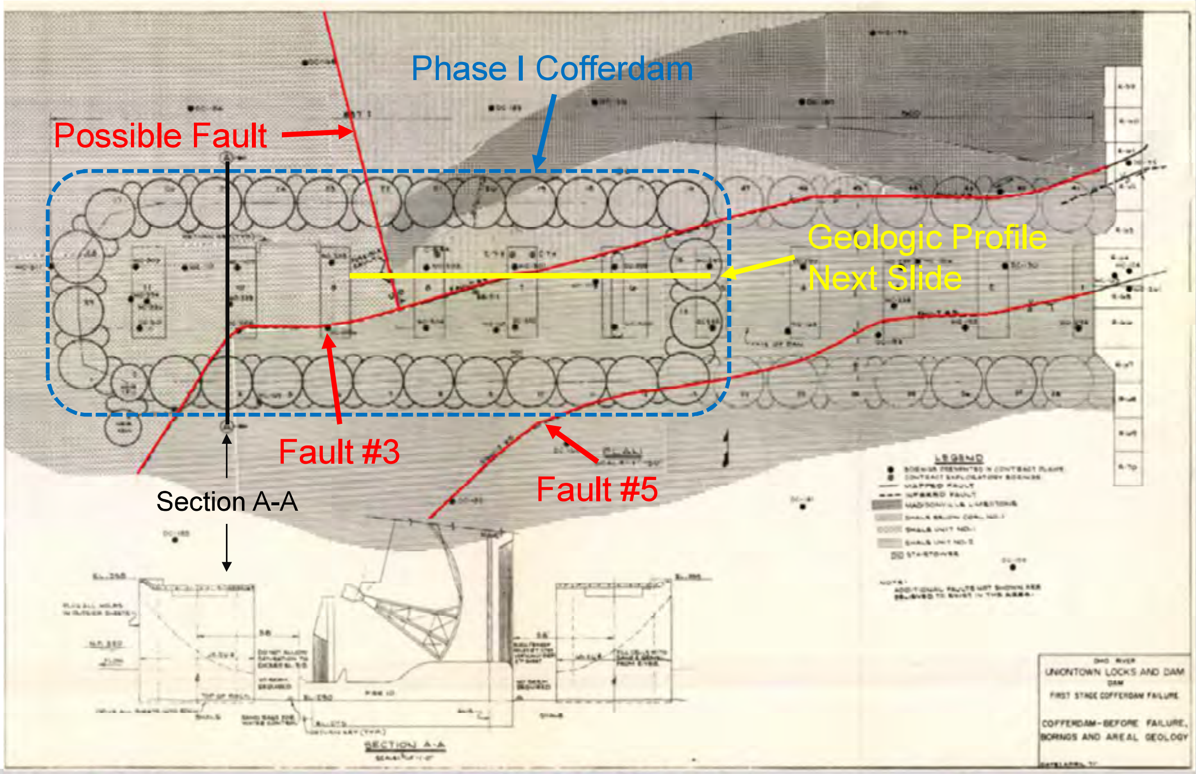
UNIONTOWN DAM PHASE I COFFERDAM

- Cofferdam for Dam Piers
- Shale Foundation
- Heavily faulted area



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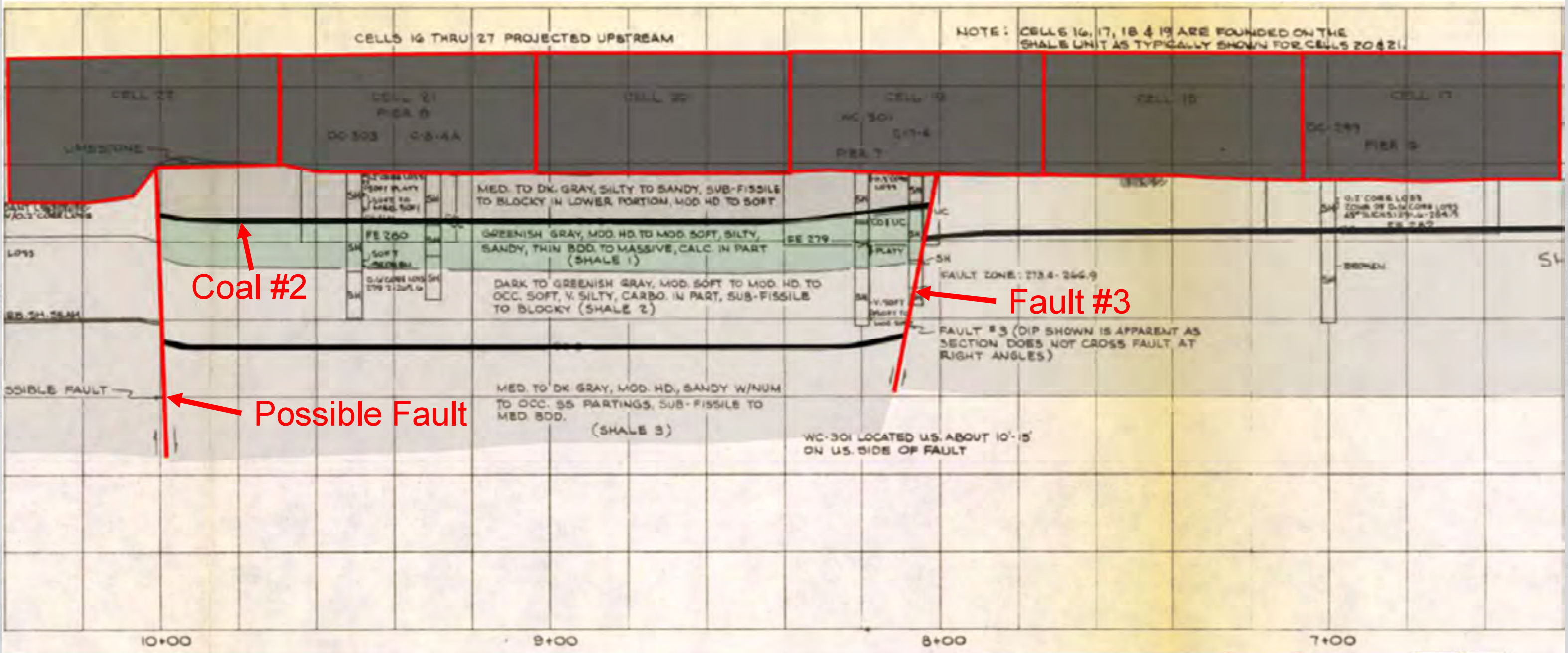
UNIONTOWN COFFERDAM GEOLOGIC PROFILE



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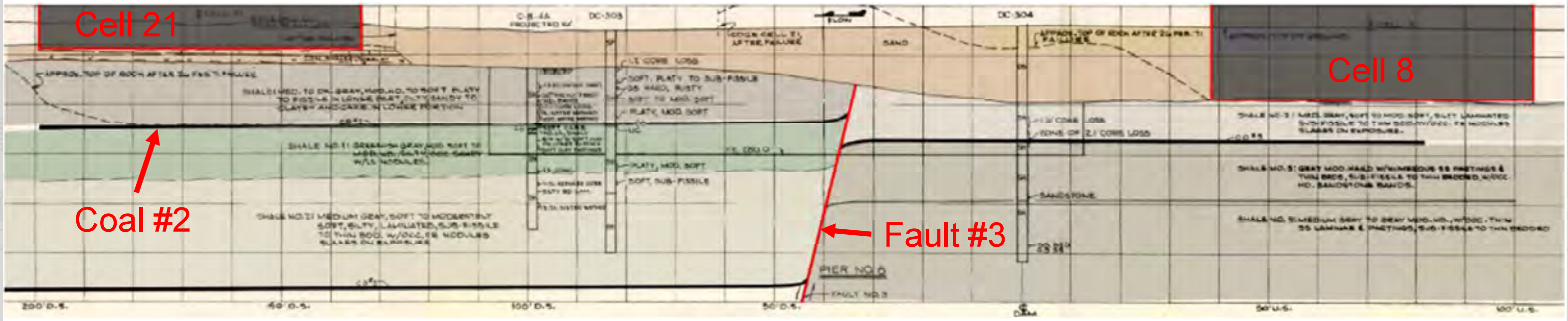
UNIONTOWN COFFERDAM GEOLOGIC SECTION



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UNIONTOWN DAM CONSTRUCTION – 1971

- 16 JAN Cofferdam Complete
- 16 FEB Cofferdam Unwatered

Slide Photograph No. 7, 25 Feb 71 - Cofferdam interior from Cell #15 looking toward Ky. Core drill on C-8-4, Air Track drilling gatebay 6 pre-split holes.

Slide Photograph No. 1, 24 Feb 71 - Aerial View of dewatered Uniontown Cofferdam before failure, from upstream Ky. corner.

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UNIONTOWN DAM – COFFERDAM FAILURE

- Failure occurred on 26 FEB 1971
- Cofferdam dewatered for 10 days

Slide Photograph No. 10, 26 Feb 71 - View of Cells #20 and #21 with connector arc, taken from concrete mixer barge, viewed toward Ky. Note aggregate barge inside cofferdam.

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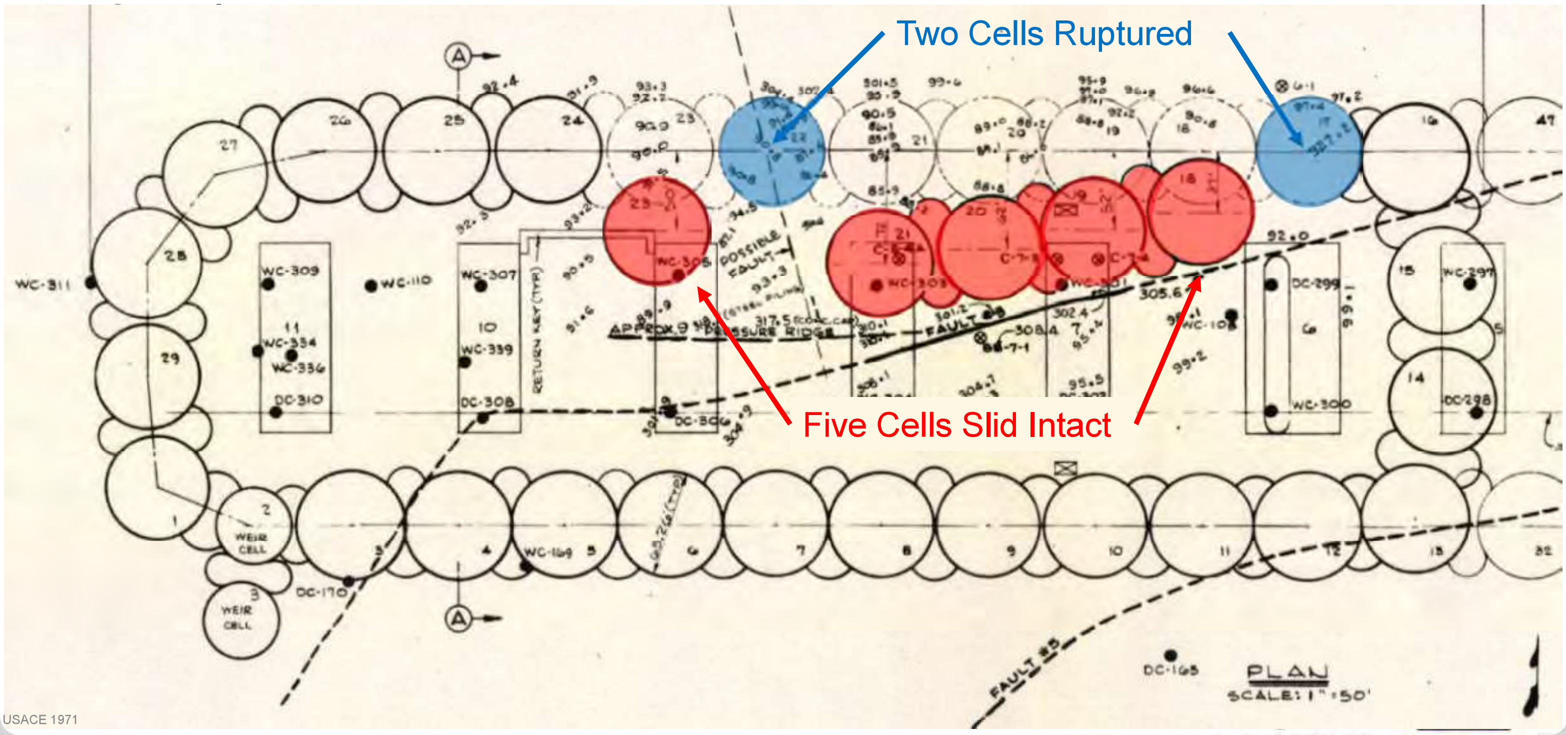
UNIONTOWN COFFERDAM SLIDING FAILURE



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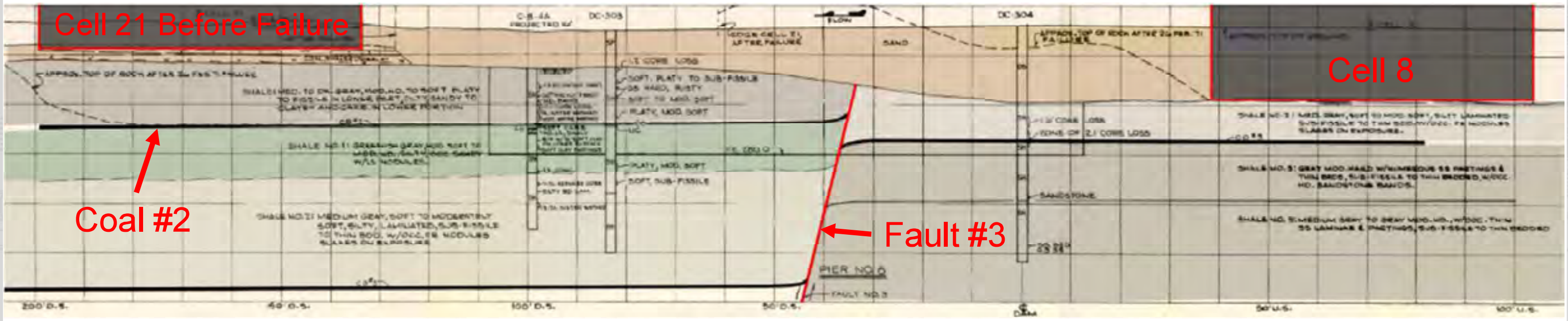
UNIONTOWN GEOLOGIC SECTION PRE-FAILURE



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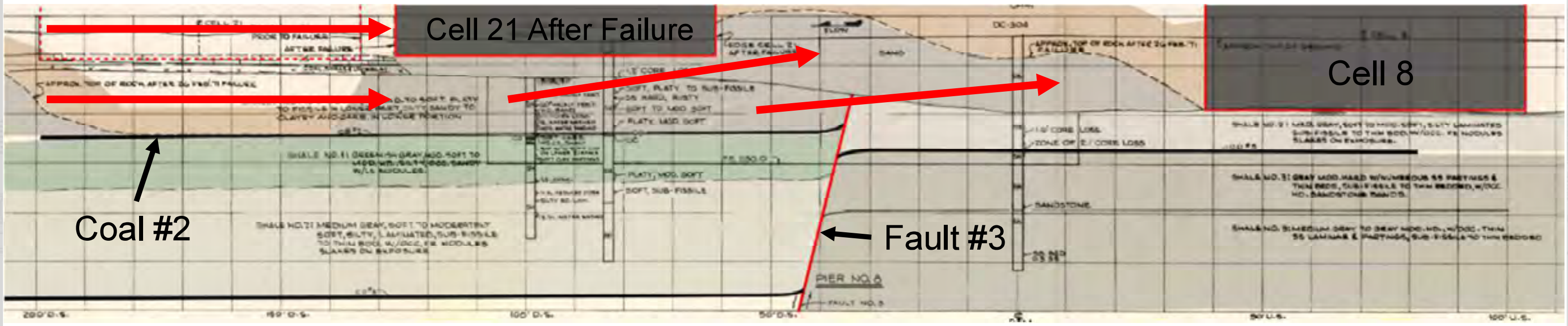
UNIONTOWN GEOLOGIC SECTION POST-FAILURE



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UNIONTOWN FAILURE – IMPORTANT FACTORS

1) Design assumptions
   - Overestimated sliding resistance of coal/underclay
   - Faults may have reduced size of passive wedge

2) Instrumentation and monitoring
   - Alert personnel and rapid response during failure likely saved lives

AGENDA

1) Case Histories – Ohio River Basin Cofferdam Failures

2) Lessons Learned

3) Case Study of Montgomery Lock Cofferdam Design

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LESSONS LEARNED

1) Investigations. Identify continuous weak seams; impacts of faults and/or joints

2) Design assumptions. Uplift considerations; appropriate sliding parameters.

3) Construction activity. How can construction affect stability?

4) Instrumentation and monitoring.

5) Owning the risk – USACE has designed all major cofferdams at Corps projects since the 1970s.

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AGENDA

1) Case Histories – Ohio River Basin Cofferdam Failures

2) Lessons Learned

3) Case Study of Montgomery Lock Cofferdam Design

MONTGOMERY LOCK

- 1932: Construction begins
- 1936: Locks operational
- 2024: Contract awarded for new river chamber

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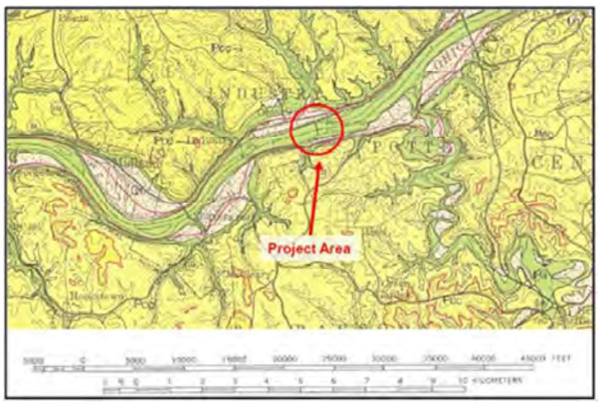
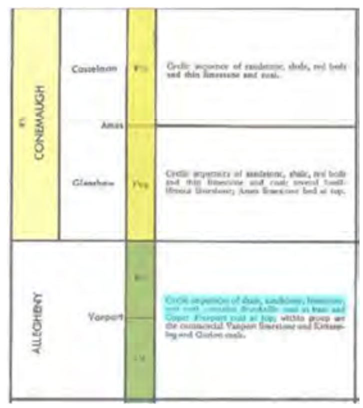
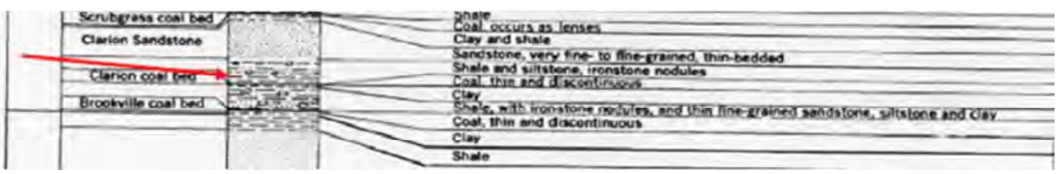
MONTGOMERY LOCK





River Bed
at Site
El. ~ 650



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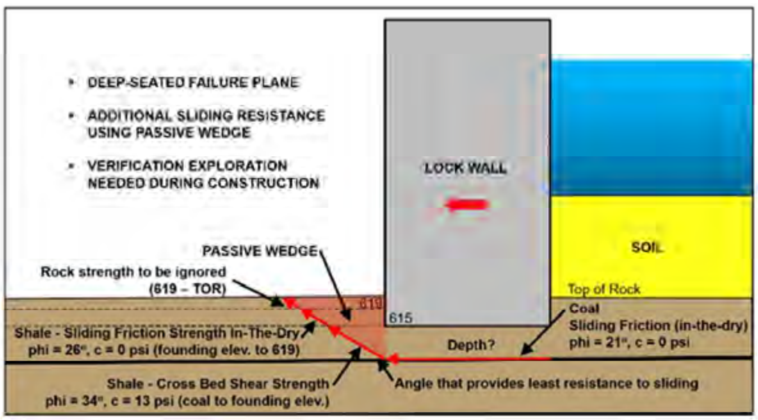
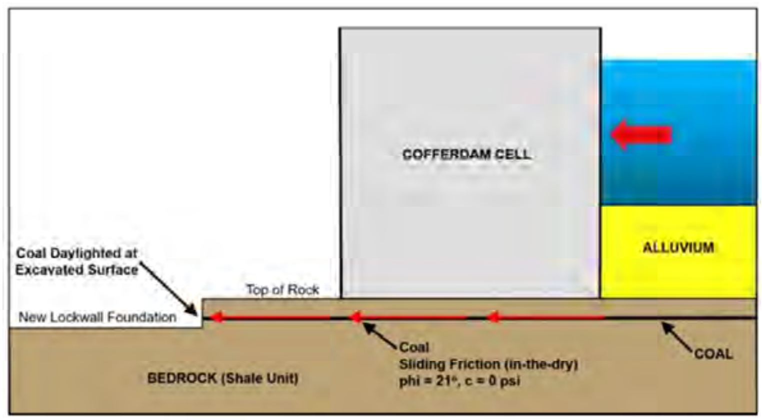
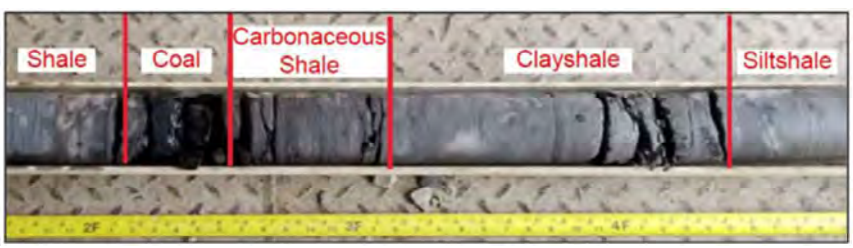
MONTGOMERY LOCK







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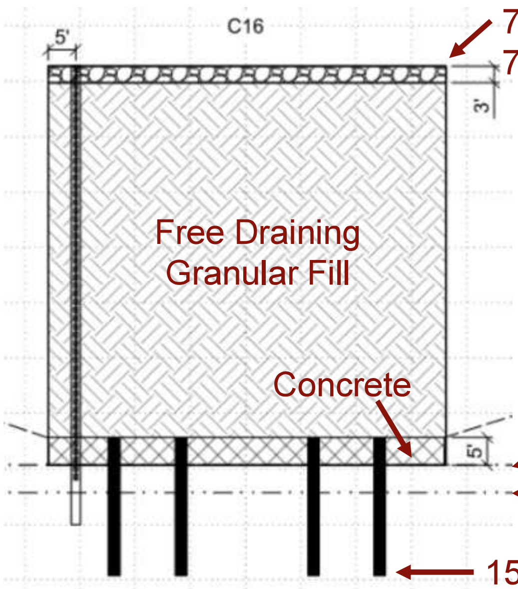
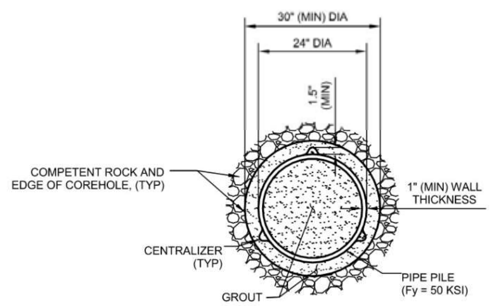
MONTGOMERY LOCK COFFERDAM DESIGN



Pipe Piles
24-in x 1-in
ASTM A252 – 50 ksi



SECTION A - A

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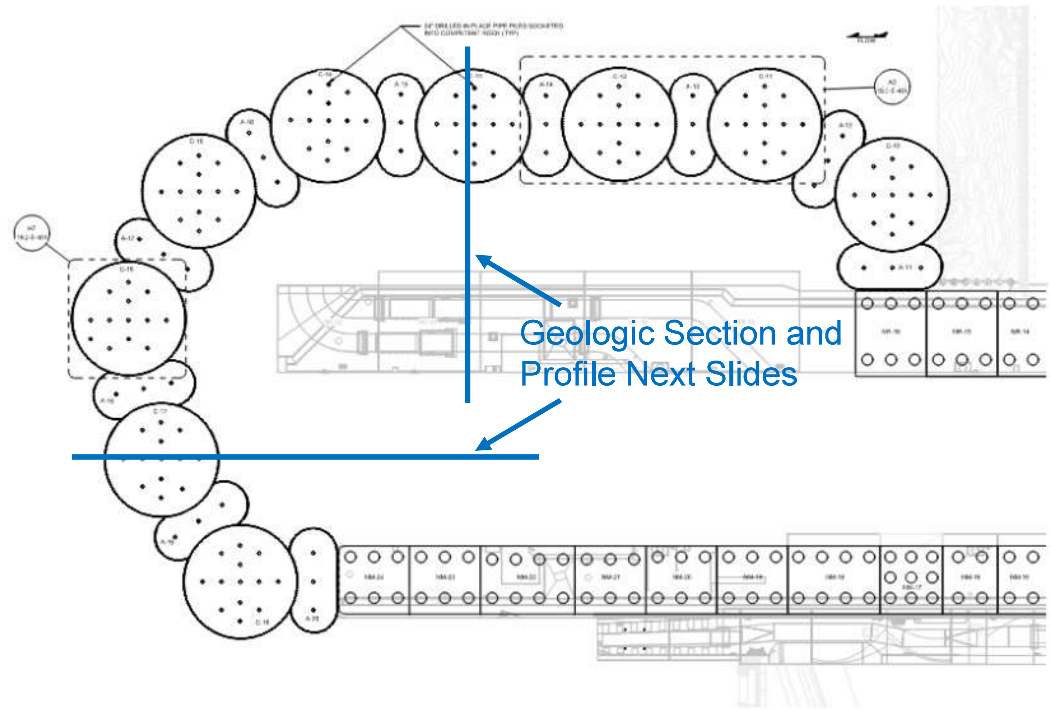
MONTGOMERY LOCK – DOWNSTREAM COFFERDAM



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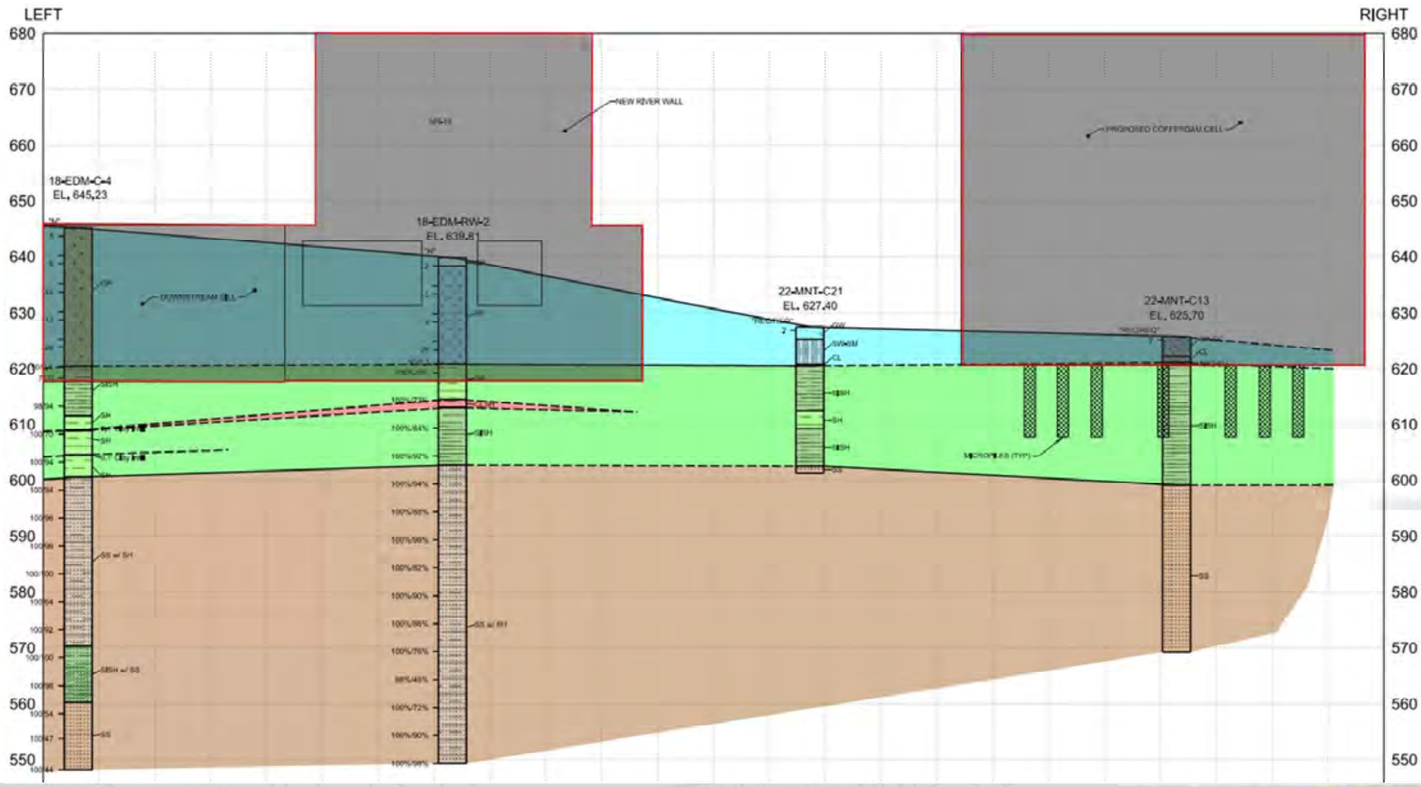
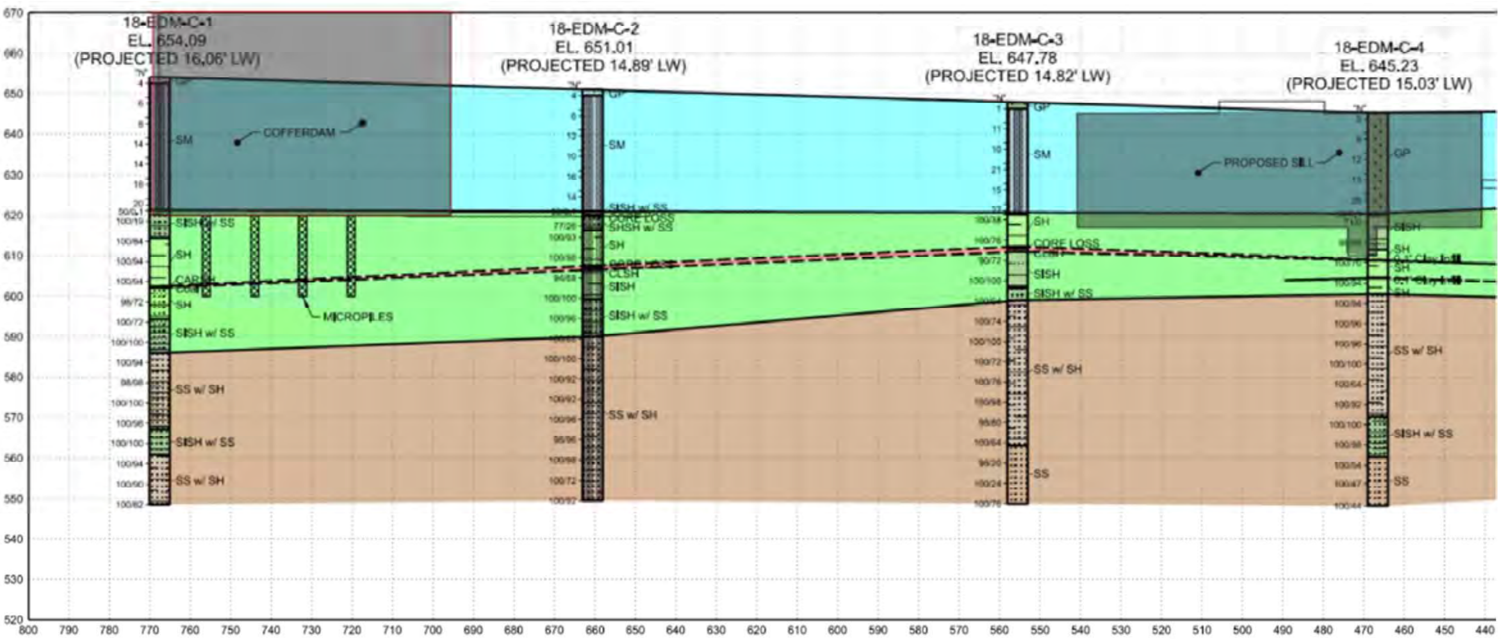
MONTGOMERY COFFERDAM GEOLOGIC SECTION



MONTGOMERY COFFERDAM GEOLOGIC PROFILE



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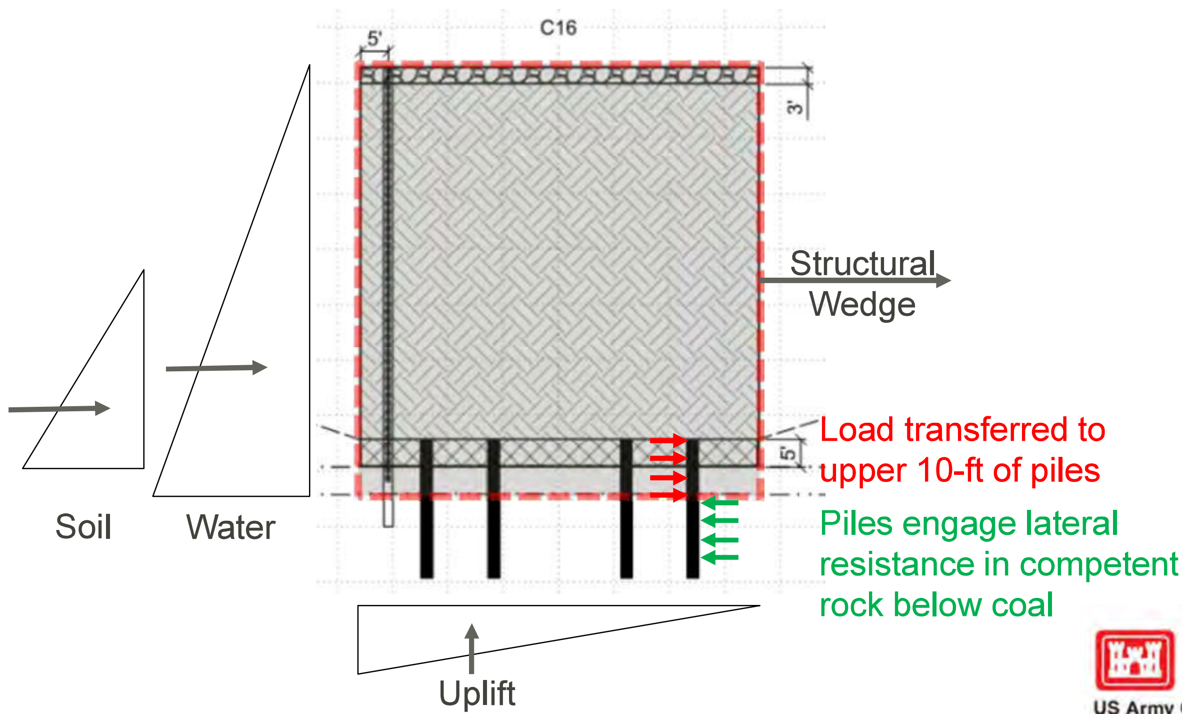
MONTGOMERY COFFERDAM SECTION



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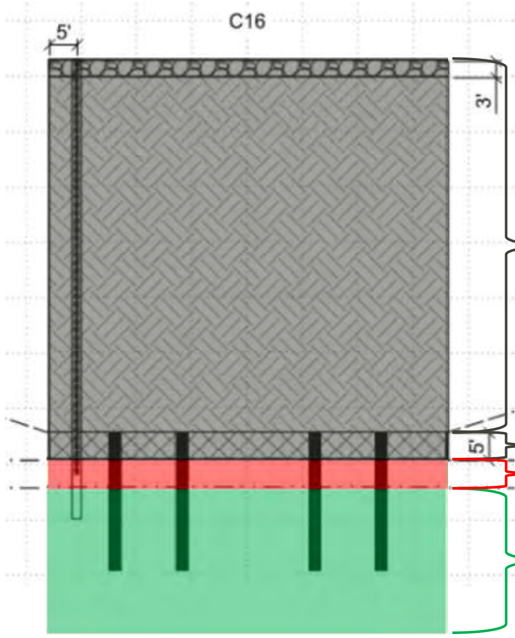
MONTGOMERY COFFERDAM LPILE ANALYSIS



LPILE Assumptions:

- Fixed-head at bottom of concrete
- No strength above sliding plane since those materials slide w/ structural wedge

Layer 1: Cell Fill (no impact on LPILE analysis)

Layer 2: Concrete (no impact on LPILE analysis)

Layer 3: Weathered & Jointed Rock
(no strength in LPILE analysis – structural wedge)

Layer 4: Competent Rock
(model as Strong Rock w/ design parameters informed by lab testing)

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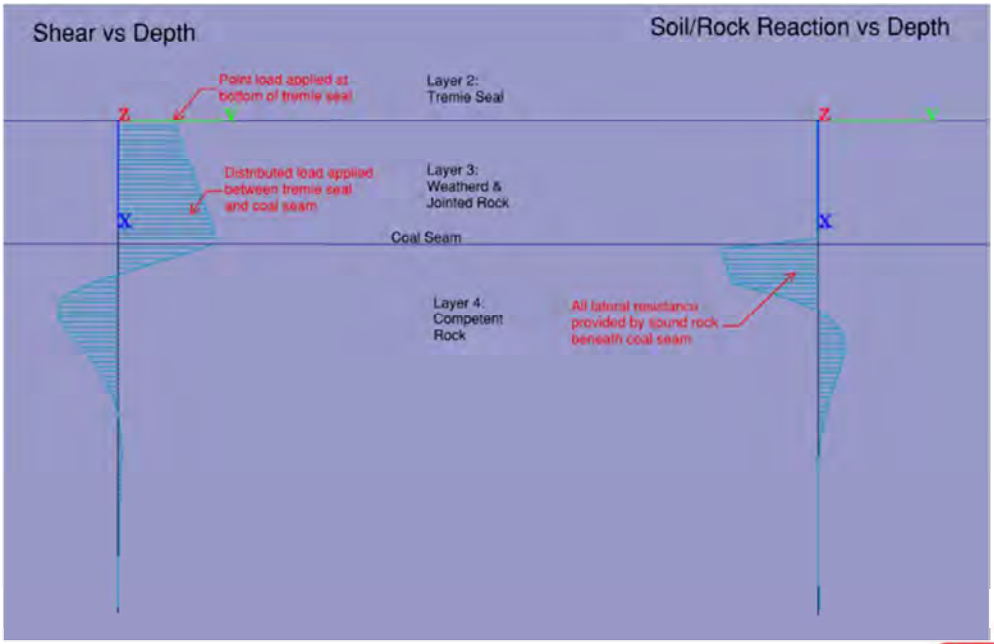
MONTGOMERY COFFERDAM LPILE ANALYSIS



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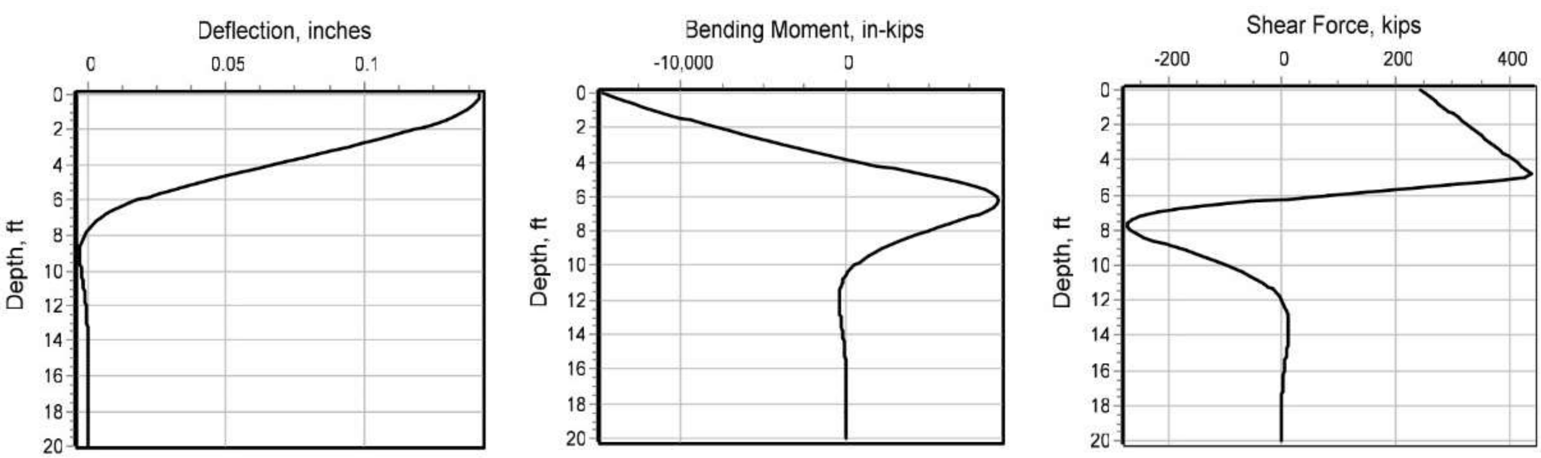
MONTGOMERY COFFERDAM LPILE RESULTS



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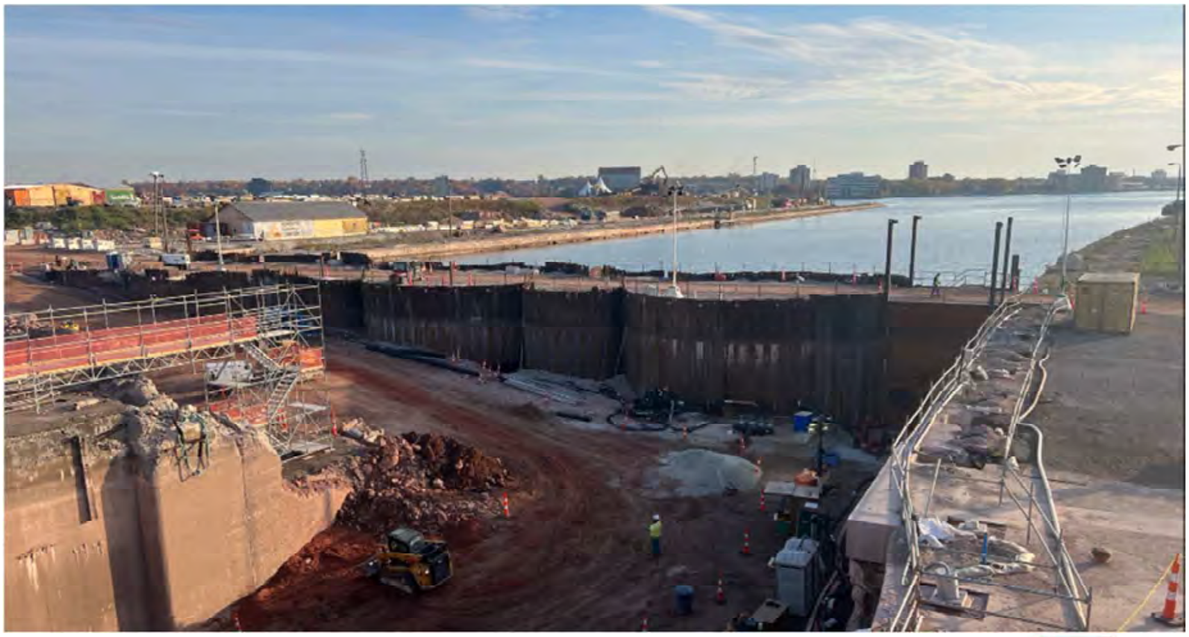
QUESTIONS???



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Notes