INDEPENDENT FORENSIC TEAM

FINAL REPORT

INVESTIGATION OF FAILURES OF EDENVILLE AND SANFORD DAMS

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Investigation of Failures of Edenville and Sanford Dams

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Summary

The physical mechanism of the May 19, 2020 failure of Edenville Dam was static liquefaction (sudden loss of soil strength) in a section of the embankment, which resulted in instability failure of the downstream slope and then breach of the reservoir through the dam. The static liquefaction occurred when Wixom Lake reached a level that was about 3 feet higher than the previous high level which occurred in 1929. When Edenville Dam failed, the resulting downstream flooding caused overtopping failure of Sanford Dam.

The Wixom Lake level at the time of the failure was about 5.5 feet higher than the normal lake level, and about 1 to 1.5 feet below the crest of the dam. The unusually high lake level, relative to the previous 1929 high level, was caused by an unusual and unfortunate combination of factors related to where and when the rain fell, and unusually impervious ground conditions which greatly increased the runoff from the rain, resulting in 100- to 200-year flooding being produced by a 25- to 50-year rainfall.

During the May 2020 event, the Edenville Dam spillway gates were opened only about 7 feet, because of concerns for operator safety and possible damage to the gates from use of the mechanism needed to lift the gates further. This amount of gate opening limited the flow through the spillways, because the gates were not lifted high enough to clear the flow of water over the spillway concrete crests. If the Edenville Dam gates had been fully opened, the IFT estimates that the maximum lake level would have been lowered by about 1 foot relative to the level at the time of failure. If, in addition to fully opening the Edenville gates, the 2019-2020 winter Wixom Lake drawdown had also been continued through May, it is estimated that the maximum lake level would have been lowered by about 1.4 feet relative to the level at the time of failure. Since the lake would have stayed at a relatively high level for several hours if the instability failure had not occurred when it did, it is uncertain whether these amounts of lake lowering would have prevented the failure.

With respect to the human judgments, decisions, actions, and inactions during the project history leading up to the May 2020 event, the dam failures were foreseeable and preventable. Edenville Dam was constructed in the 1920s in a manner which significantly deviated from the design plans and construction specifications, and this resulted in the embankments being constructed with sections of very loose to loose sands which created the fundamental physical condition required for static liquefaction. The embankments were also constructed in some locations, including the failure location, with steep downstream slopes that did not meet modern requirements for stability factors of safety. During the nine decades after construction, particularly once the dam came under Federal Energy Regulatory Commission (FERC) regulation about three decades before the failure, embankment stability analyses were performed for two locations where there were specific stability or seepage concerns, but these analyses were not sufficient in scope to evaluate the stability of the entire 6,000-foot length of the Edenville embankments. Therefore, the stability deficiency of the embankment section that failed was never recognized. If this vulnerability had been recognized, it could have been remediated by slope flattening or buttressing, which the dam owners would have been able to afford and which had already been done at other locations. This remediation would likely have prevented the embankment failure.

The potential for the Edenville Dam watershed to generate unusually high runoff and rise in the lake level was also foreseeable, because it was preceded by documented past dam failures in and near the watershed due to unusually high runoff. The main reason for not recognizing this potential for unusually high runoff was that, rather than carefully considering a range of possible floods and their associated risks, the engineers involved in the project were focused on regulatory requirements of FERC and then EGLE, which required that the dam be able to pass extreme floods, resulting from extreme rainfall, without overtopping.

None of the three dam owners were able to make significant progress towards meeting those regulatory requirements, because the revenue from generating power at the dams was insufficient to fund a spillway capacity upgrade which would have cost several million dollars. If, many years before the May 2020 failure, the dams had become publicly owned or a public-private partnership had been established, sufficient funds would have been available to upgrade the spillway capacity to pass an extreme flood, and therefore the rise of the lake in May 2020 would have been limited and the failure would almost certainly have been prevented. However, the embankment would have remained vulnerable to instability failure during future extreme floods if the embankment section that failed was not modified to increase stability.

On Tuesday, May 19, 2020, the Edenville and Sanford Dams, located in central Michigan, failed. In August 2020, the Federal Energy Regulatory Commission (FERC) engaged a five-member independent forensic team (IFT) to investigate the failures and the physical and human factors that contributed to them. This report presents the results of the IFT's work.

The Edenville and Sanford Dams were two of four dams in Michigan owned at the time of the failures by Boyce Hydro, and located in series along the Tittabawassee River. The other two Boyce Hydro dams were Secord Dam and Smallwood Dam. All four dams were built in the 1920s. Secord is the most upstream of the four dams, followed further downstream by Smallwood, Edenville, and, finally, Sanford. All four of the dams included earthfill embankments, gated concrete spillways, and powerhouses. Sanford Dam also included a fuse plug auxiliary spillway, and Smallwood Dam included an earth-lined auxiliary spillway. At the time of the failures, Secord, Smallwood, and Sanford Dams were active hydroelectric facilities, and Edenville Dam's powerhouse was inactive because its FERC license had been revoked in September 2018.

Overview of Factors Contributing to the Edenville Dam Failure

The May 2020 failure of Edenville Dam was a result of interactions of numerous physical and human factors, beginning with the design and construction of the project in the 1920s and continuing throughout the life of the project until the failure. During the nearly 100 years the project was in place prior to failure, incorrect conclusions were drawn regarding the stability of the Edenville Dam embankments and the capacity of the spillways. The potential for a non-extreme rainfall event to result in the lake rising by several feet to near the embankment crest was not recognized, and judgments and decisions were made that eventually contributed to the failure or to not preventing the failure.

The extent to which the numerous contributing factors combined and aligned to result in the failure primarily reflects both deficiencies in the construction of Edenville Dam and deficiencies in subsequent industry practices during the history of the project. The failure also secondarily involves an unfortunate combination of factors related to the variability of the dam along its length, the variations in the seepage behavior of the dam, the embankment stability analyses that were and were not performed, the hydrologic characteristics of the May 2020 storm event, and the timing of that storm event relative to planned upgrades to the Edenville gate hoist systems and spillways.

The IFT understands the natural desire to place "blame" for the failure. However, the IFT found that the failure cannot reasonably be attributed to any one individual, group, or organization. Instead, it was the overall system for financing, designing, constructing, operating, evaluating, and upgrading the four dams, involving many parties during the nearly 100 years of project history, which fell short in ensuring a safe dam at the Edenville site. All of the parties associated with the dams can be seen as having been acting "rationally" relative to their respective incentives, disincentives, responsibilities, and constraints. However, collectively, they were operating within a system that had conflicting interests and goals, resulting in the system having non-cooperative relationships. The net result was the failure of Edenville and Sanford Dams, which was a negative outcome for *all* of the parties.

Chronology of the Edenville Dam Failure

The dam failures occurred near the end of a three-day rain event. On Saturday, May 16, 2020, three days before the failures, the levels of the lakes impounded by the four Boyce Hydro dams were all slightly below normal operating lake elevations. On that day, AccuWeather was predicting heavy rain in the coming days. Beginning Sunday, May 17 and extending through Tuesday, May 19, the watersheds upstream of the four dams received significant, but not extreme, rainfall. Rainfall totals for those three days at Secord, Smallwood, Edenville, and Sanford Dams were 5.90 inches, 3.69 inches, 3.76 inches, and 2.95 inches, respectively. The vast majority of the total rainfall for each dam occurred on Monday, May 18 – about 96 percent at Secord, 100 percent at Smallwood, 82 percent at Edenville, and 95 percent at Sanford. In fact, most of the rainfall occurred in an 18-hour period from 5:00 a.m. through 11:00 p.m. on May 18 when the average rainfall intensity across the basin was nearly constant at about 0.22 inches/hour.

At the end of the day (11:59 p.m.) on Sunday, May 17, the levels of all four lakes were slightly below the lower limits prescribed in the normal summer operating rules, which were for the lake levels to be within the range of +0.3 feet to -0.4 feet of normal lake level. Around daylight on Monday, May 18, the lake levels at Secord, Smallwood, and Edenville Dams reportedly began to rise. The lake level at Sanford Dam reportedly began rising at about mid-day on Monday, May 18.

Spillway gates were opened at all four dams throughout the day on Monday, May 18, beginning at 7:00 a.m. By about 3:30 p.m. that afternoon, all gates at Secord, Smallwood, and Edenville Dams were open. There were no further gate operations at these three dams before the failure of Edenville Dam. Gate operations at Sanford Dam continued until about 8:00 p.m. on May 18, after which there were no further gate operations at any of the Boyce Hydro dams before the failure of Edenville Dam. The gates at Secord, Smallwood, and Sanford Dams were lifted sufficiently to allow free flow over the concrete spillway crests for lake levels up to the embankment crests. For reasons discussed below, the gates at Edenville Dam were lifted only about 7 feet, which was not sufficient to allow free flow over the crest for high Wixom Lake levels. Rather, at high lake levels the 7-foot gate openings resulted in orifice flow between the concrete spillway crests and the bottoms of the gates, which reduced the amount of spillway discharge as compared to what would have been available under free weir flow.

The lake levels continued to rise throughout the afternoon and evening of Monday, May 18 and the morning of Tuesday, May 19. The lake level at Secord Dam peaked in the early afternoon of Tuesday, May 19, and the lake level at Smallwood is believed to have peaked in the late afternoon that day.

At about 1:00 a.m. on Tuesday, May 19, Wixom Lake, the lake impounded by the Edenville Dam, reached the previous pool of record – the highest lake level previously recorded – which is about 2.5 feet above normal pool level and about 4 feet below the Edenville Dam embankment crest elevation. Wixom Lake continued to rise throughout that day until the time of the failure, when the lake level is estimated to have been about 5.5 feet above normal lake level, which is about 3 feet higher than the previous pool of record, and 1 to 1.5 feet below the nominal embankment crest elevation.

The forensic investigation was aided substantially by a video of the failure recorded by local resident Mr. Lynn Coleman and referred to in this report as “the dam failure video.” Based on that video, photographs, and eyewitness accounts, a downstream section of the Edenville dam failed suddenly at about 5:35 p.m. on Tuesday, May 19. The failure section was 40 to 80 feet wide, measured along the crest, and was located in the Edenville left[1] embankment, between the Edenville gated spillway and the left (east) abutment. The downstream embankment section failed in less than 10 seconds. An upstream remnant remained standing for 10 to 20 seconds, before it gave way and the embankment was fully breached. The breach enlarged over the next few hours, releasing the water stored in Wixom Lake.

[1] In this report, the terms “left” and “right” are used from the perspective of someone looking downstream in the direction water is flowing. This is commonly accepted terminology in dam engineering.

Physical Mechanism of the Edenville Dam Failure

The IFT closely examined three potential primary failure mechanisms for Edenville Dam: overtopping, internal erosion, and instability.

The IFT is confident that the embankment did not overtop. Although the reservoir level at the time of the failure was approaching the crest, especially at a depression in the crest at the location of the failure which developed about 35 minutes before the failure, there is no evidence of water flowing across the crest and no observed evidence of water on the downstream face of the dam until less than 4 minutes, and likely just seconds, before the failure.

Internal erosion was judged to not be plausible as the primary mechanism of failure. The observed physical characteristics of the failure are not consistent with an internal erosion failure mode:

- No seepage exiting the ground surface was detected; in fact, no water was detected on the downstream ground surface until just before failure.
- No turbid water discharge was detected.
- No evidence of a developing open pipe, sinkhole, or progressive sloughing was observed.
- The kinetics of the failure, in particular the global acceleration and velocity of the failure mass, are not consistent with historical observations of internal erosion failures.

In the IFT's opinion, the most plausible principal mechanism for the failure of Edenville Dam, supported by strong evidence, is static liquefaction (flow) instability of saturated, loose sands in the downstream section of the embankment. Static liquefaction is discussed further in this report, but, briefly, it occurs when the mobilized shear strength in a saturated, loose sand decreases rapidly – a sudden loss of soil strength – to values significantly less than the applied static shear stresses resulting in a force imbalance that creates accelerations and velocities.

Static liquefaction has been receiving increasing attention in recent years in the tailings (mine waste) dam arena because of several recent tailings dam failures. This failure mechanism has been rare, but not unprecedented, for water storage dams, and water storage dam engineers have not typically considered it. Geotechnical engineers have generally assumed that, under loading conditions other than earthquakes, water will be able to flow in and out of sands and their strength will be defined by the drained shear strength, regardless of the density of the sand – i.e., there will be no dramatic strength reduction as occurs in static liquefaction.

The conclusion regarding static liquefaction at Edenville Dam is supported by (a) the accelerations and velocities of the failing soil mass evident in the dam failure video, (b) strong evidence of loose, uniform fine sand in the embankment, (c) strength loss behavior exhibited in laboratory tests on loose specimens of uniform sand collected from the breach remnant, and (d) a reasonably close match of a simplified kinetic analysis with the characteristics of the failure shown in the dam failure video. Although there is uncertainty concerning the exact trigger or triggers that led to the static liquefaction failure, there are several phenomena that are plausible triggers, either individually or in some combination, as explained in the report. The record lake level at the time of the failure and the duration of high water levels almost certainly contributed to the static liquefaction instability failure.

The IFT also considered the possibility that the instability was caused by a rise in phreatic surface within the embankment, decreasing effective stresses and drained strengths in the sand, and lowering the conventional stability factor of safety below 1.0. However, a failure of this type is not consistent with the kinetics of the observed behavior. The stress-strain behavior of loose sand in drained loading does not show a dramatic strength loss. Without dramatic strength loss, it is not possible to create the force imbalance needed to generate the observed accelerations and velocities during the failure. That failure scenario would instead typically be characterized by a rise in pore water pressure dropping stability the factor of safety below 1.0 and causing enough deformation to restore stability, followed by further rise in pore water pressure causing further slumping, with this progressive process continuing and ultimately leading to enough deformation to cause failure by overtopping or internal erosion. The observed characteristics of the failure are not consistent with this mechanism, and therefore the IFT judged this mechanism to be implausible as the primary mechanism of the failure.

Physical Mechanism of the Sanford Dam Failure

The physics of the Sanford Dam failure are very clear. The failure was the result of embankment overtopping. The breach outflows from Wixom Lake after the failure of Edenville Dam caused the water level in Sanford Lake to rise more quickly than could be accommodated by the spillways at Sanford Dam. As the lake level rose above the crest of the fuse plug spillway at about 7:19 p.m. on May 19, the fuse plug began to erode, however, photographic evidence suggests that erosion did not progress through the crest of the fuse plug until about the time of initial embankment overtopping at about 7:46 p.m. Once the crest of the fuse plug began to erode there was an increase in release capacity at Sanford Dam, but the combined release capacity of the fuse plug spillway and the gated spillway was not sufficient to prevent continued rise of the Sanford Lake level, ultimately leading to an overtopping breach failure which initiated at about 8:20 p.m., almost 3 hours after the failure of Edenville Dam. Given the failure of Edenville Dam, the failure of Sanford Dam was not unexpected. Regulators and engineers understood that, if a breach occurred at Edenville Dam, Sanford Dam would almost certainly be overtopped and fail.

The IFT’s investigation focused principally on the failure of Edenville Dam, because if Edenville Dam had not failed it is likely that Sanford Dam would not have failed.

Geotechnical Factors

The IFT found that the design of Edenville Dam was not substantially inadequate if the dam had been built according to the design plans and construction specifications. However, the actual construction of the dam deviated substantially from the design plans and construction specifications, apparently with the knowledge of the dam designer, and as a result, the safety margins of the dam were below average even compared to dams of similar type and size built from the 1910s through the 1930s. There are several possible reasons for the deviations from the design plans and construction specifications, including limited availability of suitable construction equipment, limited availability of suitable materials in borrow sources, insufficient staffing for construction and construction inspection, high costs or schedule pressures, and/or a desire to complete the construction ahead of schedule in order to start generating power and revenue sooner.

The primary geotechnical factor contributing to the failure was the very loose to loose sands present in the embankment as a result of the initial dam construction. Had the embankment been constructed in compacted layers in accordance with the original construction specification, the sands in the embankment would not have been loose and the embankment instability failure would almost certainly not have occurred.

Another contributing geotechnical factor was the steep downstream slope of the Edenville east embankment, with an average inclination of 1.8 horizontal to 1 vertical. This steep slope produced high static shear stresses in the embankment that left very little margin against the stress conditions that triggered static liquefaction. Hence, only relatively small changes in pore water pressure and/or shear stresses were needed to trigger the failure.

The lack of a comprehensive geotechnical evaluation of the Edenville Dam embankments was a third contributing factor. Subsurface investigations of the embankments at Edenville Dam before 2020 were relatively limited for the approximately 6,000 feet length of embankments. However, the available investigations consistently showed the presence of very loose to loose sands in the embankments. Prior to review of construction photos during a Potential Failure Mode Analysis (PFMA) completed in 2005, it was believed that the embankment materials had been compacted, despite the evidence to the contrary provided by prior test borings. Stability analyses for the Edenville Dam had been completed for only two cross-sections and none had been completed for the Edenville east embankment.

The IFT's analysis indicates that, if stability analysis had been completed for the Edenville east embankment where the failure occurred, with its steep downstream slope, that embankment section would have been identified as having stability factors of safety significantly less than recommended minimum values. This would likely have resulted in a recommendation for slope flattening or a buttress. Since similar slope modifications had been constructed at other sections of the Edenville Dam embankments, it is likely they would also have been constructed at the Edenville east embankment. Such embankment modifications would have very likely prevented the static liquefaction failure by reducing the static shear stresses.

Hydrologic and Hydraulic Factors

Two significant hydrologic and hydraulic factors were (1) the unusual characteristics of the May 2020 rainfall event and the response of the watershed to this rainfall and (2) a lack of understanding of the hydrologic risk of high lake levels at or near the embankment crest.

Rainfall Event Characteristics: The spatial and temporal characteristics of the May 2020 rainfall combined with the watershed conditions produced an unusually high runoff. The heaviest rainfall during the event occurred in the northern and eastern areas of the watershed, increasing the inflow to Wixom Lake. Almost all the rainfall was concentrated in an 18-hour period with an intensity of about 0.22 inches/hour. This resulted in a concentration of the volume of the runoff in a relatively short time period.

The IFT's analysis of the May 2020 event indicates that a relatively high percentage of the total rainfall (about 35 percent) was converted into runoff, resulting in a record inflow into Wixom Lake. In comparison, analysis of the "great flood" of 1986, which occurred in September 1986, indicates that only about 12 percent of the total rainfall for that event was converted into runoff. Therefore, despite the rainfall in the September 1986 event being significantly higher than that in May 2020, the runoff was much higher in May 2020. The IFT believes that the difference is attributable to high ground saturation and partially frozen ground in parts of the watershed in May 2020, especially in the forest swamps and wetlands. These conditions were not present at the time of the September 1986 event.

The combination of the characteristics of the rainfall and the watershed conditions in May 2020 resulted in an estimated 25- to 50-year rainfall event producing an estimated 100- to 200- year flood. The main reason why this potential for unusually high runoff was not recognized by the engineers involved in the project was that, rather than carefully considering a range of possible floods and their associated risks, these engineers were focused on regulatory requirements of FERC and then EGLE, which required that the dam be able to pass extreme floods, resulting from extreme rainfall, without overtopping.

Risk of High Lake Levels: As early as 1978, it was recognized that the spillways at Edenville Dam could not pass the probable maximum flood (PMF). This conclusion was repeatedly confirmed during the period of FERC regulation by analyses completed between 1991 and 2013. Based on interviews, it appears that those involved with the project during the period of FERC regulation thought that the spillway capacity was about 50 percent PMF and the likelihood of a lake level exceeding the embankment crest elevation was perceived to be very low. Although the 2005 Potential Failure Modes Analysis (PFMA) report includes a statement that the Edenville Dam spillway capacity was sufficient to "safely pass a flood event roughly equal to a 200-year flood," this finding was categorized as making overtopping "less likely" rather than "more likely," and therefore even a spillway capacity in the 200-year flood range apparently was perceived by the PFMA group as reflecting a low overtopping risk.

The analyses of spillway capacity completed before the failure were all based on an assumption that the spillway gates could be opened sufficiently to allow free flow over the concrete spillway crests, but this was not a valid assumption, except perhaps between 2015 and 2019. Consequently, the actual spillway capacity was less than assumed in the analyses.

The original gate hoist system was configured in a way that limited the gates to an opening of no more than about 7 feet, while at least 10 feet of opening was needed to allow free flow over the concrete crests. Between 2012 and 2015, Boyce Hydro fabricated an A-frame system to supplement the original gate hoist system and demonstrated that the A-frame system could physically lift the gates to more than 10 feet. In June 2019, a gate operation test resulted in a recommendation that the A-frame system not be used because of concern for safety of the operators and possible damage to the gates. As a result, during the May 2020 event, the gates at Edenville Dam were opened only about 7 feet, with the original hoist system and sparing use of the A-frame system. Plans were in process for modifying the gate hoist systems at Edenville Dam in late 2020 to allow for gate opening to 10 feet or more, but, unfortunately, the May 2020 event occurred before the hoist systems were modified.

Gate openings of 7 feet reduce the estimated spillway discharge by about 27 percent relative to the estimated discharge with 10-foot openings, for the Wixom Lake level at the time of failure. Had the gates been opened 10 feet during the May 2020 event, the IFT's analysis indicates that the peak lake level would have been about 1 foot lower than the lake level at the time of the failure. Because of uncertainty regarding the triggering of static liquefaction, the IFT cannot conclude that this reduction in lake level would have prevented the failure, but it might have.

With regard to the risk of a Wixom Lake level exceeding the Edenville Dam embankment crest elevation, an analysis after the failure (Ayres 2021) indicated that with gates opened to about 9 feet, as opposed to about 7 feet during the May 2020 event, the Edenville Dam spillways could have passed a 200-year flood (an annual exceedance probability of 0.005) with about 0.1 feet of freeboard to the embankment crest elevation. Consequently, it appears that the annual risk of a flood resulting in a lake level above the Edenville Dam embankment crest elevation was greater than 0.005 given the gate hoist limitations. This is in contrast with a lower probability that was perceived for the 50 percent PMF, but in agreement with the statement in the 2005 PFMA report. Had the higher risk of embankment overtopping been understood, it is possible that there would have been a greater urgency to modify the dam to increase the spillway capacity, even if not initially to full PMF capacity.

Interactions of Involved Parties

During the era of FERC regulation of the four Boyce Hydro dams, the interactions of the various parties involved with the four hydroelectric projects contributed to the circumstances that resulted in the May 2020 failures. These parties included the three dam owners, the dam owner's engineering consultants, FERC, EGLE, FLTF, FLTF's engineering consultants, Consumers Energy, Gladwin and Midland Counties, the lakefront property owners, and lake users who did not own lakefront properties. As discussed in detail in Section 7.2.3 of this report, all of the parties had their own goals, and the goals of these parties were at times in conflict with each other.

Some of the more significant interactions that contributed to the unfortunate May 2020 dam failures were:

- FERC was trying to compel the dam owners to complete spillway upgrades at Edenville Dam to accommodate the PMF, while the owners claimed to have insufficient financial resources to support the upgrades. None of FERC's enforcement options would help this situation, and, in fact, they all would make the situation worse by reducing the owners' revenue. The action ultimately taken by FERC, license revocation, did not reduce the risks presented by Edenville Dam.
- The counties, the lakefront residents, and other lake users benefited from the presence of the dams and lakes in the form of increased property tax revenues, increased property values, recreational opportunities, and increased commerce in the local economy without needing to contribute to paying for costs of maintaining and upgrading the dams. With the planned sale of the dams to FLTF, as the delegated authority of the counties, plans were in place for local financial resources to be applied to support dam safety upgrades, but the record high Wixom Lake water level and the resultant dam failures most unfortunately occurred before this plan could be implemented.
- Throughout the tenure of its ownership, Boyce Hydro had difficulty in negotiating higher power purchase rates from Consumers Energy. Higher power purchase rates may have provided sufficient financial resources to allow Boyce Hydro to at least partially comply with the spillway capacity increases being required by FERC.

These interactions, along with others described in this report, contributed to setting the stage for the May 2020 dam failures, which was a bad outcome for all of the parties involved with the dams, and even more so to property owners downstream of the dams who only indirectly benefitted from the dams and lakes but experienced the brunt of the direct effects of the dam failures.

Emergency Response

The failures of the two dams significantly increased the downstream property damage above that which would have occurred from the natural flood alone, but fortunately there were no reported fatalities or serious injuries. This result is attributable to emergency management of the event. Evacuations ordered by local authorities were effective at protecting public health and preventing loss of life because of a prudent, proactive decision by the Midland County emergency manager (EM).

Evacuations were initiated in the late hours of May 18 and early morning hours of May 19 because the Midland County EM was not comfortable with the reports she was receiving from the dam operator concerning conditions at the dam. The EM also noted that in the overnight hours she had access to a large group of volunteer firefighters to implement the evacuations, many of whom would not be available during daylight working hours.

This evacuation decision did not strictly follow the guidance in the emergency action plan (EAP) for Edenville Dam, and the guidance in the EAP regarding evacuation was not consistent throughout the document. One interpretation of the EAP guidance would have resulted in the initiation of evacuations being delayed until Edenville Dam failed. In this case, because of the rapidity of the embankment failure, had evacuations not been initiated before the actual failure, it is entirely possible that lives would have been lost.

The evacuations were reportedly well organized and orderly. The EMs attributed this, at least in part, to an EAP exercise conducted in 2019 and evacuations plans developed as a result of that exercise.

Lessons to be Learned

From this investigation, the IFT identified several lessons to be learned. These lessons are listed briefly below, and further reasoning supporting the lessons is discussed in Section 8.

Static Liquefaction: Static liquefaction instability failure should be considered as a potential failure mode (PFM) for water storage or flood management dams when saturated or potentially saturated, loose or very loose sands, silty sands or nonplastic silts are present in the embankment or foundation of the dam. The challenge for water dam engineers now is to develop procedures and protocols to screen and evaluate static liquefaction potential and determine when risk reduction actions to address this PFM are appropriate. Developing these procedures and protocols should leverage the work that has been done by tailings dam practitioners.

Comprehensive Reviews: Repeating a lesson to be learned from the Oroville Dam spillway incident forensic investigation (France et al. 2018), physical inspections, while a necessary part of a dam safety program, are not sufficient by themselves to identify risks and manage safety. Dam safety evaluations need to include periodic comprehensive reviews of original design and construction, performance, operations, analyses of record, maintenance, and repairs. These evaluations need to be an independent review, unbiased by previous conclusions by others. They should include a review that analyses of record adequately reflect the expected performance of the dam under a range of loading conditions. In response to the Oroville incident, FERC has promulgated rules that took effect in April 2022 to include comprehensive reviews in its Part 12D inspection and evaluation process.

However, FERC-regulated dams are only a small percentage of dams in the United States. Over 90 percent of the dams in the country are regulated by state authorities, most of which still rely heavily on periodic physical inspections with no requirements for comprehensive reviews. The IFT suggests that comprehensive reviews should be periodically performed for high-hazard dams which are state-regulated.

Inadequate Financial Resources: In the case of Edenville Dam, the Boyce Hydro projects did not provide sufficient revenue to support a $5 million to $10 million PMF spillway upgrade project. More broadly, the Association of State Dam Safety Officials (ASDSO) estimates that needed rehabilitation/upgrade of the more than 88,000 non-federal dams in the United States would cost over $75 billion, including about $24 billion dollars for more than 15,000 non-federal high hazard potential dams (ASDSO 2022). Many of those rehabilitation/upgrade needs are not being met because of owners' lack of financial resources. Although some state and federal financial assistance programs exist, the available financial resources are only a small fraction of the need.

If progress is to be made on addressing the identified dam safety rehabilitation needs and protecting the public in the United States, more financial support to owners will be needed from federal, state, and local levels of government.

Imbalance between Benefits and Financial Responsibility: For these four dams and their associated lakes, substantial financial and other benefits from the projects, in the form of county tax revenues, increased property values, and recreational opportunities, accrued to the counties and local residents. Yet the dam owner was expected to pay all costs related to the project, including dam safety upgrades. This imbalance was in the process of being addressed at the time of the failures, through the planned sale of the projects to Four Lake Tasks Force (FLTF), acting as the delegated authority of the counties. Most unfortunately, the unusual combination of rainfall and basin conditions in May 2020 created the record Wixom Lake level that triggered the Edenville Dam embankment failure, before the culmination of the sale and the completion of a PMF upgrade.

More broadly, where such imbalances between benefits and financial responsibility exist, sales of dams to local public entities should be considered along with more creative solutions such as public-private partnerships.

Dam Regulatory Enforcement Tools: FERC's dam safety regulatory enforcement tools do not include the authority to order a breach of a dam except in the case of an emergency. In the case of Edenville Dam, had FERC been able to order a breach of the dam, the action would likely have met with strong resistance from the counties and local residents. However, the prospect of this scenario may have led to development of a mechanism for the counties and the residents to acquire the dams or contribute to the spillway upgrade. If this had happened sooner, it is possible that a spillway upgrade would have been constructed before the May 2020 event, and the failures very likely would not have happened in May 2020. Michigan Department of Environment, Great Lakes, and Energy (EGLE) had the authority to order a breach, and, if necessary, implement it. At the time of the failure, EGLE had been the regulator of Edenville Dam for only a short time, and the agency was still evaluating the dam and had not developed a basis for considering a breach.

Again, more broadly, the IFT suggests that all dam safety regulatory agencies, including FERC, should have the regulatory authority to order a dam breach if dam safety risks are judged to be unacceptable and an owner does not have the financial resources to reduce the risks or does not comply with a directive to reduce the risks. Further, the IFT suggests that regulatory agencies should have the authority to breach the dam if the owner does not comply with a breach order. The regulatory agencies will also need access to funding to breach dams when necessary.

Hydrologic Risk: The risk of overtopping the embankments at Edenville Dam was not well understood. Had the actual hydrologic risk at Edenville Dam been better understood, there may have been greater urgency assigned to the need for some degree of increased spillway capacity.

For dams that do not meet regulatory spillway capacity requirements, the urgency of the need for spillway capacity upgrade should be based on quantitative analysis of risks, and consideration should be given to staged spillway capacity improvements/upgrades as interim risk reduction measures.

For multiple dams in series in the same watershed, flood operating plans should be developed to model the dams and associated lakes as a system, with operating guidance developed for a variety of storm and flooding scenarios, including both the inflow design flood (IDF) and more frequent storms.

Rainfall Return Periods versus Flood Return Periods: The May 2020 rainfall event is estimated to have been a 25- to 50-year event, but it produced an estimated 100- to 200-year flood event because of an unusual combination of rainfall characteristics and watershed conditions.

In estimating hydrologic risks for less than extreme events, the potential for unfavorable basin conditions and spatial and temporal rainfall distributions should be considered. In this regard, some watersheds are more seasonally influenced than others. For watersheds in northern climates, such as Michigan, the potential for frozen ground or frost leading to saturated ground conditions in the spring should be considered.

Emergency Action Plans: Overall, the emergency response to the potential and then actual failures of Edenville and Sanford Dams must be considered very successful, because there were no fatalities or serious injuries. The success of the emergency response can be principally attributed to a prudent, proactive, and cautious decision by the Midland County emergency manager (EM) to initiate early evacuations. This decision did not strictly follow the guidance in the Edenville Dam EAP. The guidance in the EAP was inconsistent and contradictory and could have led to a delay in evacuations until Edenville Dam failed suddenly. If evacuations had been delayed until after failure occurred, it is entirely possible that lives would have been lost.

EAPs should provide consistent guidance to decision-makers regarding when evacuation is warranted, and should allow for judgment, so that evacuations can be ordered when the risk of failure is judged to be sufficiently high, rather than waiting for failure to initiate. In addition, an EAP should not be viewed as complete until an EAP exercise has been completed.

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Acronyms and Abbreviations

c’	drained cohesion, cohesion intercept
%	percent
ϕ’	friction angle
°F	degrees Fahrenheit
ac-ft	acre-feet
AEP	annual exceedance probability
ASCE	American Society of Civil Engineers
ASDSO	Association of State Dam Safety Officials
ASTM	ASTM International
AWA	Applied Weather Associates
Ayres Associates	Ayres
Barr	Barr Enginerring
bgs	below ground surface
bpf	blows per foot
Blystra	A. R. Blystra & Associates
BOC	Board of Consultants
Boyce Hydro	Boyce Hydro, LLC; Boyce Hydro Power, LLC
CEII	Critical Energy/Electric Infrastructure Information
cfs	cubic feet per second
CID	isotropically consolidated, drained triaxial shear
CIU’	isotropically consolidated, undrained triaxial shear with pore pressure
cm/s	centimeter per second
Consumers	Consumers Energy, Consuer Power Company
CSIR	Consultant’s Safety Inspection Report
C_V	coefficient of consolidation
DSI	dam safety inspection
EAP	emergency action plan
EDT	Eastern Daylight Time
EGLE	Michigan Department of Environment, Great Lakes, and Energy
El.	Elevation
EM	Emergency Manager
EPRI	Electric Power Research Institute
FC	fines content
FERC	Federal Energy Regulatory Commission
FLO	Four Lakes Operations
FLTF	Four Lakes Task Force
FPC	Federal Power Commission
FS	factor(s) of safety
ft/mi	foot per mile

ft/sec	feet per second
GEI	GEI Consultants Inc.
GIS	geographic information system
H:V	horizontal:vertical
HDPE	high-density polyethylene
HEC	Hydrologic Engineering Center
HEC-HMS	Hydrologic Engineering Center-Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
HEC-SSP	Hydrologic Engineering Center-Statistical Software Package
HMR	Hydrometeorological Report
IC	Independent Consultant
IDF	Inflow Design Flood
IFT	Independent Forensic Team
IPPC	Independent Power Producers Coalition
kh	horizontal permeability
kv	vertical permeability
ksf	kips per square foot
LANDSAT	land remote-sensing satellite
LiDAR	light detection and ranging
M-30	Michigan Highway 30
Mill Road	Mill Road Engineering
MIRECS	Michigan Renewable Energy Certification System
MPEA	Multiple Project Environmental Assessment
MSL	mean sea level
m_V	coefficient of volume compressibility
NAVD88	North American Vertical Datum of 1988
NEXRAD	Next Generation Weather Radar
NGVD29	National Geodetic Vertical Datum of 1929
NLCD	National Land Cover Database
NOAA	National Oceanic Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWL	normal water level
NWS	National Weather Service
pcf	pounds per cubic foot
PFM	potential failure mode
PFMA	Potential Failure Mode Analysis
Phase I Report	National Dam Safety Program Inspection Report
PI	plasticity index
PMF	probable maximum flood
PMP	probable maximum precipitation
PMS	probable maximum storm

PPA	power purchase agreement
PPP	public-private partnership
psf	pounds per square foot
psi	pounds per square inch
PURPA	Public Utilities Regulatory Policies Act
PVC	polyvinyl chloride
REC	Renewable Energy Certificate
Reclamation	U.S. Department of the Interioir, Bureau of Reclamation
SAD	Special Assessment District
SLPA	Sanford Lake Preservation Association
SME	Soils and Materials Engineers, Inc.
SPAS	Storm Precipitation Analysis System
SPT	standard penetration test
SSURGO	Soil Survey Geographic Database
Sta.	Station
State	State of Michigan
STATSGO	State Soil Geographic Database
STID	Supporting Technical Information Document
UNET	Unsteady Flow Network Model
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
Wolverine	Wolverine Power Corporation

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1. Introduction

At approximately 5:35 p.m. EDT on May 19, 2020, a section of Edenville Dam located on the Tittabawassee River in central Michigan failed suddenly during a rainfall event that resulted in a record-high water level within the impounded Wixom Lake. The outflow from the Edenville Dam failure flowed into Sanford Lake, and almost 3 hours after the failure of Edenville Dam, an overtopping failure of the Sanford Dam embankment initiated. The combined outflows from the failures of Edenville Dam and Sanford Dam and runoff from the natural watershed inundated downstream areas of Midland County and Saginaw County. At the time of the failures, Edenville Dam and Sanford Dam were two of four dams located in series along the Tittabawassee River, owned by companies controlled by Boyce Hydro Power, LLC and Boyce Hydro, LLC, all of which were owned by three W.D. Boyce trusts and all collectively referred to in this report as "Boyce Hydro."

The four dams are, in sequence from upstream to downstream, Secord Dam, Smallwood Dam, Edenville Dam, and Sanford Dam. At the time of the May 2020 failures, all of the Boyce Hydro dams[2] except Edenville Dam were active hydroelectric facilities under the regulation of the Federal Energy Regulatory Commission (FERC). Edenville Dam was also a FERC-regulated hydroelectric facility previously, but FERC revoked the Edenville Dam license in September 2018. At the time of the failures, Edenville Dam was regulated by the Michigan Department of Environment, Great Lakes, and Energy (EGLE). In December 2020, months after the failures, ownership of the four dams was deeded to Midland County and Gladwin County through a settlement agreement with Boyce Hydro, with Four Lakes Task Force (FLTF) functioning as the counties' delegated authority. The ownership details of the four dams are discussed in more detail in Section 2.

Following the failures, an Independent Forensic Team (IFT) was formed to study the causes of the failures and provide lessons to be learned to prevent future similar failures. This report presents the results of the IFT investigation.

1.1 Team Formation and Authorization

After the failures of Edenville Dam and Sanford Dam, FERC and EGLE, in coordination, issued directives for Boyce Hydro to engage a fully independent forensic investigation team to develop findings and opinions on the causes of the failures. Per FERC, the team was to consist of dam safety experts well versed in the following disciplines: Hydraulics and Hydrology, Geotechnical Engineering, Structural Engineering, Reservoir Operations, Emergency Action Planning, and Organizational/Human Factors. The team members were not to have worked on any of the Boyce Hydro projects in the past.

Boyce Hydro selected the proposed team members, who were subsequently approved by FERC and accepted by EGLE in June 2020. The start of the investigation was delayed due to a series of contract and logistical discussions between the IFT members and Boyce Hydro, during which time Boyce Hydro filed for bankruptcy on July 31, 2020. FERC then proceeded to contract with the IFT members to allow the investigation to commence. The IFT members and their roles in the investigation are as follows:

- John W. France, PE, D.GE, D.WRE, JWF Consulting LLC – Team Leader, Geotechnical Investigation and Analysis, and Emergency Response Evaluation

[2] Reference to the "Projects" or "Boyce Hydro dams" throughout this report refers to structures associated with the Secord Dam, Smallwood Dam, Edenville Dam, and Sanford Dam, and reflects the ownership at the time of the May 2020 failures.

- Irfan A. Alvi, PE, Alvi Associates – Human Factors Investigation and Analysis, Emergency Response Evaluation, Hydrologic Investigation and Analysis, and Geotechnical Analysis
- Art Miller, PE, PhD, AECOM – Hydrologic and Hydraulic Investigation and Analysis, and Reservoir Operations Evaluation
- Jennifer L. Williams, PE, AECOM – Geotechnical Investigation and Analysis
- Steve Higinbotham, PE, Independent Consultant – Hydraulic Structures Evaluation

Resumes for all IFT members are provided in Appendix H of this report. Contracts for all team members were issued by July 31, 2020, and the IFT's work commenced at that time. The IFT members were assisted throughout the investigation by several supporting engineers, principally:

- Harry Donaghy, PE, AECOM – Data Management and Geotechnical Investigation and Analysis
- Hayden LoSasso, PE, AECOM – Hydrologic and Hydraulic Analysis
- Ian Selock, PE, Alvi Associates – Human Factors Investigation and Hydrologic Investigation
- Caroline Bibb, PE, Alvi Associates – Human Factors Investigation and Hydrologic Investigation
- Michael Itzla, PE, Alvi Associates – Wave Force Analysis

Although the IFT members' contracts were with FERC, the IFT carried out its work independently, and its efforts were not directed or controlled by FERC, EGLE, Boyce Hydro, or FLTF. The IFT also completed its work independently of all other organizations and agencies, including regulatory bodies, government agencies, consulting firms, water user groups, and professional societies, including the Association of State Dam Safety Officials (ASDSO), United States Society on Dams (USSD), and American Society of Civil Engineers (ASCE). Neither the IFT members nor their home organizations were involved with the design, construction, inspection, operation, maintenance, or investigations of the Boyce Hydro structures prior to the failures. The IFT members were not involved with any of the emergency response or long-term repairs following the failures.

FERC, EGLE, Boyce Hydro, and FLTF supported and cooperated with the IFT by providing data from their files, assisting with arranging meetings and interviews with personnel, and facilitating completion of on-site investigative work requested by the IFT. Other than fact-checking of some sections, this report was not reviewed by FERC, EGLE, Boyce Hydro, FLTF, or any other agency, organization, or individual at any point prior to its finalization.

1.2 Purpose of Investigation

The IFT performed a thorough review of available information, conducted interviews, and performed engineering analyses to develop findings and opinions on the chain of physical conditions, judgments, decisions, actions, and inactions that, on May 19, 2020, led to the failures of the Edenville Dam and Sanford Dam in Michigan, and why opportunities for intervention in this chain of events were not realized. The purposes of the investigation were to evaluate physical and human factors that contributed to the failures and to identify lessons to be learned by the industry, emergency management agencies, and the public to prevent future similar failures.

1.3 Focus and Limitations of Investigation

The IFT's efforts were focused on Edenville Dam and Sanford Dam. The two Boyce Hydro dams upstream of Edenville along the Tittabawassee River, Secord Dam and Smallwood Dam, were investigated only to the degree of evaluating how their operations and performance influenced the failures of Edenville Dam and Sanford Dam. Operations and outflows from Chapel Dam and Beaverton Dam, located upstream of Edenville Dam along the Tobacco River watershed, and Lake Lancer Dam, located on the Sugar River upstream of Smallwood Dam, all owned by others, were also considered to the degree that they may have affected the ultimate outcome at Edenville Dam.

The IFT considered the emergency management of the incident by local emergency managers, as well as by Boyce Hydro, FERC and EGLE. Emergency management was considered to the degree that it affected decisions regarding activation of the Emergency Action Plan, warning communication, and evacuation. Details regarding public response to and logistics of the emergency response were beyond the scope of the IFT's mission.

A peer review of the EGLE dam safety program was conducted by an ASDSO peer review team (ASDSO 2020) at the request of EGLE. The objective of the ASDSO peer review was to perform an evaluation of the EGLE dam safety program's mission, objectives, policies, procedures, and other factors, including the competence of EGLE's dam safety program relevant to the generally accepted standards of practice for dam safety engineering and management. In addition, at the request of the Michigan governor, EGLE also convened a task force to investigate the events leading up to the May 2020 dam failures and to recommend policy, legislative, budgetary, and enforcement reforms through a review of the statutory structure, budget, and program design of the EGLE dam safety program and review of the adequacy of Michigan's dam safety standards (EGLE 2021). The IFT did not perform a separate general review of EGLE's dam safety program, but rather reviewed the findings of the peer review and task force studies and considered the influence of EGLE's decisions and actions in order to reach conclusions relating to factors contributing to the May 2020 failures.

Similarly, a general review of the FERC dam safety program was outside the scope of the IFT's investigation, but the IFT did consider the influence of FERC's decisions and actions in relation to factors contributing to the May 2020 failures. The IFT believes that FERC would benefit from having an external peer review performed, similar to the peer review which ASDSO performed for the EGLE dam safety program, if such a review has not recently been completed.

The IFT based the opinions and findings presented in this report in large part on information that was made available to the IFT during the investigation. The IFT attempted to cast a broad net to obtain pertinent information, including press releases asking for input from the public. However, the IFT cannot be certain that all relevant information was discovered and compiled. During interviews and other communications, the IFT became aware of some items of pertinent information that had not been originally provided and requested copies of that information. It must be acknowledged that some items of relevant information that could have influenced the IFT's opinions and findings could come to light after this report is issued.

To maintain independence from the owner and regulators, a draft of this report was not provided to any party before finalization, although, as noted above, some sections of the report were provided to FERC, EGLE, Boyce Hydro, and FLTF for fact-checking. The IFT endeavored to verify eyewitness accounts and information obtained in interviews, as well as its interpretation of documented information through corroborating evidence and discussion with multiple parties. An Interim Report (France et al. 2021) was issued on September 13, 2021, which primarily discussed the event chronology and physical factors.

After the Interim Report was released, comments on the Interim Report were solicited from FERC, EGLE, Boyce Hydro and FLTF. Unsolicited comments were also received from other individuals. All comments were considered in the completion of this final report.

1.4 Investigation Methodology

The IFT endeavored to complete as thorough an investigation and evaluation as practical to develop evidence for its findings and opinions. The IFT's work included:

- A thorough and critical review of initial documents provided by Boyce Hydro, FERC, and EGLE. Based on this data review, additional documents were requested and subsequently provided to the IFT as they became available. Documents reviewed included:
  - Documents related to original design and construction of the project. Available documents were limited and consisted primarily of excerpts of specifications and construction reports, a limited number of drawings, and construction photographs. Design or as-built drawings showing the internal configuration of the embankments of Edenville and Sanford Dams were not available. Exhibit figures drawn post-construction were discovered; however, the source of the information used to develop these figures is not known.
  - Records of inspections, maintenance, and evaluations of the projects by various entities including previous owners, Boyce Hydro, FERC, Independent Consultants as part of FERC's Part 12D process, and EGLE, including:
    - Potential Failure Mode Analyses (PFMAs) and FERC Part 12D reports completed for the project.
    - Records of maintenance and surveillance monitoring activities, repairs, and modifications to the Projects with a focus on the Edenville Dam and Sanford Dam embankments and spillways.
    - Records of evaluations of issues related to the embankments and spillway gates.
    - Records of spillway operations.
    - Written logs, photographs, videos, and other documentation of the May 2020 flood event.
    - Results of investigations and evaluations completed by EGLE, FLTF, and their consultants after the May 2020 incident.
    - Governance, guidance, and procedural documents for applicable dam safety, operation, and maintenance organizations associated with the Projects.
- A visit to the site by two members of the IFT, namely Ms. Jennifer Williams and Dr. Art Miller, on September 24 and 25, 2020. Due to public health and safety guidelines related to travel during the COVID-19 pandemic, not all team members were able to travel to the site. During the site visit, Ms. Williams and Dr. Miller accomplished the following:
  - Performed visits to all four Boyce Hydro dam sites as well as to Beaverton Dam and Chapel Dam.
  - Conducted interviews with Boyce Hydro operators, EGLE personnel, and FERC personnel.
  - Inspected the breach sites of both Edenville Dam and Sanford Dam.

    - Performed a preliminary review of documents located at the Boyce Hydro office at Edenville Dam.

- Meetings and interviews with individuals involved in various aspects of the Boyce Hydro dams or the May 2020 flood event, or otherwise in a position to provide information relevant to the investigation. Individuals interviewed included current and past Boyce Hydro employees, EGLE employees, FERC employees, representatives of FLTF, local residents, eyewitnesses, and individuals associated with previous inspections and analyses of Edenville Dam. In total, more than 25 individuals were interviewed, primarily by virtual meetings. Some individuals were interviewed more than once. Most interviews were in-depth and lasted more than an hour.
- Hydrologic and hydraulic analyses performed to simulate the May 2020 rainfall/runoff event for the Sanford watershed. The model included the four Boyce Hydro dams, Lake Lancer Dam on the Sugar River, and the two dams on the Tobacco River (Chappel Dam and Beaverton Dam). The model was also used to evaluate the potential effects of hypothetical alternative operational scenarios for the four Boyce Hydro dams.
- Field geotechnical investigations by IFT member Ms. Jennifer Williams with the assistance of Mr. Harry Donaghy from December 9 through 11, 2020. The field investigation consisted of the following activities:
    - Faces of each breach side slope were cleaned of loose soil and debris and the exposed materials were mapped.
    - Bulk (disturbed) soil samples of various materials were collected.
    - Thin-walled tube samples were collected from each breach face.
    - In situ density tests were performed using the sand cone method.
    - Limited review of documents located in the Boyce Hydro office at the Edenville Dam site was also performed during this site visit.
- Laboratory testing on the samples collected in the field by the IFT.
- Several geotechnical analyses relating to failure kinetics, seepage, and stability.
- Public requests for information related to the March 2020 event, with an independent email box established to contact the IFT. The public requests were made with the assistance of ASDSO. Numerous emails were received, and several individuals who contacted the IFT were interviewed.
- Over 50 IFT working sessions to collectively discuss and evaluate factual information and develop opinions. Due to the COVID-19 pandemic, these working sessions were all conducted virtually.
- Personal discussions with experts regarding static liquefaction and its applicability to the Edenville Dam failure.
- Personal discussions with experts regarding seasonal watershed characteristics.
- Preparation of an Interim Report (France et. al 2021) issued on September 13, 2021, to publicly share the IFT's findings as of that date. The interim report focused on the physical mechanisms contributing to the failures.

The IFT work culminated in the preparation of this final report of the IFT's findings and opinions. Evaluations of judgments, decisions, actions, and inactions for the various stages of the project (predesign, design, construction, and operations and maintenance) considered the states of practice applicable to the various time periods involved. Through an iterative process, the IFT applied a mix of inductive and deductive reasoning to the assembled information, the resulting evidence and arguments of which are provided in this report and the appendices.

1.5 Report Organization

This report is organized into nine sections and eight appendices. The main report provides sufficient information regarding the failures to follow the physics of what happened and why the failures happened in terms of both physical and human factors.[3] The findings and opinions in the main report are supported by discussions and information in the appendices. In this report, the IFT provides (1) an introduction to the scope and approach of the investigation, (2) brief descriptions of the four Boyce Hydro dams, (3) a discussion of the chronology of the failures, (4) the IFT's findings concerning the physical mechanisms of the failures, (5) an evaluation of the flood event and the effect of operations, (6) an evaluation of the effectiveness of the emergency response actions, (7) the IFT's findings concerning the human factors contributing to the failures, (8) a summary of lessons to be learned, and (9) a list of references used throughout the course of the investigation.

[3] In this report, the term "human factors" is intended to extend beyond individual factors to include organizational and industry factors.

2. Background

This section of the report provides background information on the Boyce Hydro projects, including a brief history of project ownership, descriptions of the four dams, and a discussion of the spillway gate hoist systems at the dams.

The background information was developed in large part from the Supporting Technical Information Documents (STIDs) and the five-year Part 12D Consultant's Safety Inspection Reports (CSIRs) for each project required by the Federal Energy Regulatory Commission (FERC), supplemented by additional information from other sources.

2.1 Brief History of Boyce Hydro Projects

A brief history of the Boyce Hydro projects is provided in this section. For further details on the project history, see Section 7.1 and Appendices A, B, C, and D.

The Edenville and Sanford Dams, located in series along the Tittabawassee River, were two of four dams in Michigan that were operated at the time of the failures by Boyce Hydro. The other two Boyce Hydro dams are Secord Dam and Smallwood Dam. All four dams were built between 1923 and 1925 and are located in Gladwin and Midland Counties in central Michigan. At the time of the 2020 failures, all of the Boyce Hydro dams, except Edenville Dam, were active hydroelectric facilities under the regulation of FERC. Edenville Dam was also a FERC-regulated hydroelectric facility previously, but FERC revoked the Edenville Dam license in September 2018 (FERC 2018f). At the time of the failures, Edenville Dam was regulated by EGLE. In its revocation order for Edenville Dam, FERC cited a number of asserted non-compliances on the part of Boyce Hydro, but the principal reason cited for the revocation was failure to increase the spillway capacity at the dam to accommodate the probable maximum flood (PMF), as required by FERC guidelines. A summary of the key features of the four dams is provided in Table 2-1 below.

Table 2-1: Key Features of the Boyce Hydro Projects

Dam	Rated Capacity (megawatts)	Maximum Height (feet)	Total Length (feet)	Number of Spillway Gates	Lake Area at Normal Lake Level (acres)	Lake Storage at Normal Lake Level (acre-feet)
Secord	1.2	56	2,000	2	920	15,000
Smallwood	1.2	38	1,010	2	400	6,000
Edenville	4.8	52	6,130	6	2,270	36,000
Sanford	3.6	36	1,580	6	1,550	15,000

Under FERC guidelines, all four Boyce Hydro dams were classified as high-hazard dams. The criteria for a high-hazard classification is that there is potential for loss of life in the event of a dam failure. The hazard classification does not identify the physical condition or potential for failure of the dam. Edenville Dam was similarly classified by EGLE after its FERC license was revoked.

Secord Dam and Smallwood Dam are located on the Tittabawassee River upstream of Edenville Dam, and Sanford Dam is located on the Tittabawassee River downstream of Edenville Dam. Edenville Dam was constructed across both the Tittabawassee River and the Tobacco River, a tributary to the

Tittabawassee, just upstream of their confluence. Lancer Lake Dam, owned by others, is on the Sugar River, which is another tributary of the Tittabawassee River, and the confluence is below Secord Dam and just above Smallwood Dam. Two other dams, Beaverton Dam and Chappel Dam, are owned by others and are located on the Tobacco River, upstream of Edenville Dam. The locations of all seven dams are shown in Figure 2.[4]

All four dams operated by Boyce Hydro were designed by Holland, Ackerman and Holland of Ann Arbor, Michigan. The primary purpose for constructing the dams was to generate electrical power. The normal operating lake elevation is primarily set to have enough head to provide efficient power production. During normal (non-flood) operations, the lake fluctuations were typically less than 1 foot.

The operating rules for all of the lakes were to maintain lake levels between +0.3 foot and -0.4 foot of the normal lake level, except during flood operations, when lake levels could be higher, or during winter drawdown operations. Winter drawdown could begin after December 15 and was to be completed by January 15. During winter operations, the lakes were not to be lowered more than 3 feet below the normal lake level, and the daily fluctuation in lake level was not to exceed 0.7 foot. For Edenville Dam, the minimum winter lake level was Elevation (El.) 672.8[5], which is 3 feet below the normal lake level (El. 675.8 feet). The lakes were to be returned to the normal lake levels prior to the surface temperatures of the lakes reaching 39 degrees Fahrenheit (°F). These operating rules were established for all four lakes under FERC regulation and were set for all four lakes and re-established for Wixom Lake and Edenville Dam after the revocation of the Edenville hydropower license by the State of Michigan in the Circuit Court for the County of Midland on May 28, 2019 (Michigan 2019).

The four dams were originally constructed by the Wolverine Power Company, which was founded by Frank Wixom in 1923. The Wolverine Power Company entered bankruptcy in the early 1930s, and in May 1934, sold all of its assets and properties to Edenville Power Company, which was owned by Frank Wixom. Edenville Power Company was then immediately renamed Wolverine Power *Corporation* (Wolverine). Several decades later, New World Power Corporation, a holding entity focused on renewable energy, was formed in 1993 and by May 1994 it had acquired all shares of Wolverine.

In 2003, Wolverine defaulted on a loan from Synex Energy Resources, Ltd., a Vancouver-based engineering and consulting company. Synex Energy Resources foreclosed on the four dams; acquired the deeds on all land, equipment, and offices; and created a holding company named Synex Michigan, LLC, a Synex Energy Resources subsidiary.

In 2006, W. D. Boyce Trusts purchased the real estate consisting of the four dams and related property, and separately acquired 100 percent of the LLC membership interests of Synex Michigan. In 2007, Synex Michigan was renamed Boyce Hydro Power, LLC. Under this ownership, the W. D. Boyce Trusts owned multiple companies associated with the dam, which are collectively referred to in this report as “Boyce Hydro.” A more detailed accounting of project ownership history is provided in Section 7.1.1.

In July 2018, the Sanford Lake Preservation Association (SLPA), which would later become known as the Four Lakes Task Force (FLTF), and Boyce Hydro signed a letter of intent (Boyce Hydro 2018g) for SLPA to purchase the four Boyce Hydro projects. The letter of intent was followed by a tentative agreement between FLTF and Boyce Hydro in April 2019 (Boyce Hydro 2019d), a purchase agreement

[4] The distances shown in the table in Figure 2 are based on streamline data for the Tittabawassee River (USGS 2020).
[5] Elevations cited in this report are in feet, based on the National Geodetic Vertical Datum of 1929 (NGVD29). Original design drawings and some other references are based on a plant datum that is 5.8 feet higher than NGVD29. For example, the normal pool level for Wixom Lake is El. 675.8 feet NGVD29 and El. 670.0 feet plant datum.

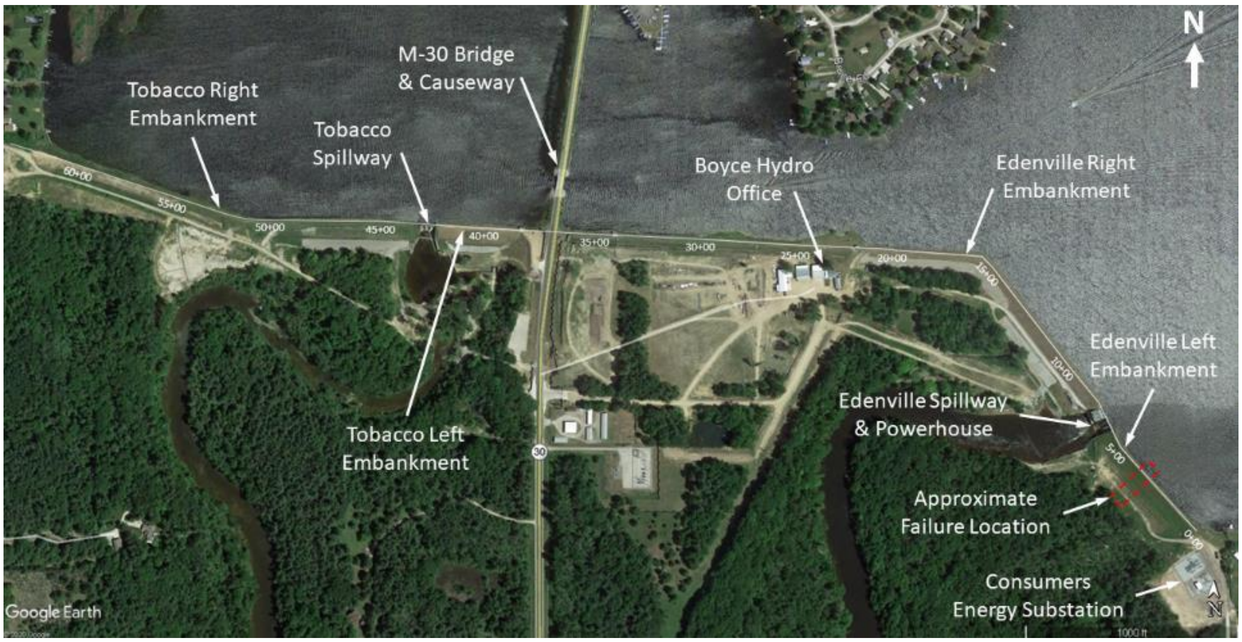
between the same parties in December 2019 (Boyce Hydro 2019e). An amendment to the purchase agreement in May 2020 (Boyce Hydro 2020d) set a closing date of June 1, 2020. However, on May 20, 2020, Edenville Dam and Sanford Dam failed and the purchase agreement was subsequently nullified, and Gladwin and Midland Counties then proceeded to take ownership of all four dams via condemnation. (See Section 7.1.1).

Figure 2-1: Dam Locations in Central Michigan

2.2 Edenville Dam

The Edenville Hydroelectric Project (Edenville Dam) was operated as a power generation facility until its license was revoked by FERC in 2018. At the time of the 2020 failure, the facility consisted of four earthfill embankments, two gated concrete spillways, and a powerhouse—all constructed across the Tittabawassee River and Tobacco River in Michigan and totaling more than 6,000 feet in length, as shown in Figure 2. The reservoir impounded by the facility was known as Wixom Lake, which had a reported area of approximately 2,270 acres[6] at a normal lake level of El. 675.8.

A causeway on Michigan Highway 30 (M-30) effectively divided the lake, with the east side impounding water from the Tittabawassee River and the west side impounding water from the Tobacco River. A causeway bridge opening on the M-30 hydraulically connected the two sides of the lake—the Tittabawassee, or Edenville,[7] side and the Tobacco side. The design crest elevation for all four embankments is El. 682.8.



Source of aerial image: Google Earth

Figure 2-2: Edenville Dam Configuration

Edenville Side – The Edenville side included two embankment sections, a spillway, and the powerhouse.

The Edenville left embankment extended from the left[8] (east) abutment to the Edenville spillway, a length of approximately 625 feet. The maximum height of this embankment was about 52 feet immediately adjacent to the spillway and about 32 feet further to the left. The failure occurred within a 30- to 32-foot tall section of this embankment, as noted on Figure 2. The Edenville left embankment appeared to have been constructed using sand fill in the lower section of the embankment downstream of the centerline; clay fill in the lower section of the embankment upstream of the centerline; and a soil mixture of silty, clayey sand in the upper section of the embankment. However, the internal cross section is not consistent along the full length of this embankment and appeared to transition to a predominantly clay fill, with sand fill located only near the outer embankment slopes in the portion of the embankment near the spillway. The available information on the composition of the Edenville left embankment is not definitive and in some ways contradictory, so there is uncertainty regarding the cross section of the embankment and how

6 The reservoir surface area shown is based on 2017 light detection and ranging (LiDAR) data. Other project documents report several different values for surface areas at normal lake level.

7 In different documents, the eastern side of the facility impounding the Tittabawassee River is referred to as either Tittabawassee or Edenville. In this report, Edenville is used.

8 In this report, the terms "left" and "right" are used from the perspective of someone looking downstream in the direction water is flowing. This is commonly accepted terminology in dam engineering.

it varied along the length of the embankment. See Appendices A and E for further discussion of the embankment cross section.

The Edenville spillway and the powerhouse are a single, combined structure. Based on the original plans, the Edenville spillway included two Tainter (radial) gates (Gate No. 1 and Gate No. 2), which are each 20 feet wide by 9 feet 6 inches high, and one Tainter (radial) gate (Gate No. 3), which is 23 feet 7 inches wide by 9 feet 6 inches high. The gate sills are at El. 667.8, 8 feet below the normal lake level. Gate No. 1 is located adjacent to the powerhouse. The powerhouse, which has been inactive since revocation of the FERC license, contains two vertical-shaft generating units with a rated capacity of 2.4 megawatts each, for a total of 4.8 megawatts. According to the original drawings, the spillway structure also includes a low-level gated sluiceway (located in Bay No. 1), which was reportedly not operable for decades prior to the dam failure. The Edenville spillway structure is 68.6 feet wide, and the powerhouse is 50.6 feet wide, for a total structure width of 119.2 feet.

The Edenville right embankment extends from the powerhouse to the M-30 in a dogleg pattern, for an embankment length of about 2,900 feet. The maximum height of this embankment is about 50 feet immediately adjacent to the powerhouse and about 40 feet further to the right. The Edenville right embankment appeared to have been constructed as a relatively homogenous cross section consisting predominantly of sand fill, with areas of higher fines content near the base of the embankment and a clay blanket constructed along the upper portion of the upstream slope extending from the crest down to an unknown elevation. See Appendices A and E for further discussion of the embankment cross section.

According to the original design drawings, the upstream and downstream slopes of the embankments are nominally 2.5H:1V (horizontal:vertical) and 2H:1V, respectively. However, downstream slopes have been flattened and berms have been added in some locations, and survey data show that the downstream slope is steeper than 2H:1V in some locations. In addition, survey data after the flood event indicate that the upstream slope was steeper than 2.5H:1V in some locations.

Tobacco Side – The Tobacco side includes two embankment sections and a spillway. The Tobacco left embankment extends from the M-30 to the Tobacco spillway, a length of approximately 520 feet. The maximum height of this embankment is about 47 feet adjacent to the spillway and about 45 feet further to the left.

Based on the original plans, the Tobacco spillway included two Tainter (radial) gates (Gate No. 1 and Gate No. 3), which are each 23 feet 7 inches wide by 9 feet 6 inches high, and one Tainter (radial) gate (Gate No. 2 in the center bay), which is 20 feet wide by 9 feet 6 inches high. Gate No. 1 is located in the left (east) bay. The gate sills are at El. 667.8, 8 feet below the normal lake level. As on the Edenville side, according to the original drawings, the spillway structure also included a low-level gated sluiceway (in the center bay), which was reportedly not operable for decades prior to the failure. The Tobacco spillway structure is 72.2 feet wide.

The Tobacco right embankment extends from the Tobacco spillway to the right (west) abutment, in a slight dogleg pattern for an embankment length of about 1,895 feet. The maximum height of this embankment is about 47 feet adjacent to the spillway and about 32 feet further to the right. Based on limited available information, the Tobacco embankments appeared to have been constructed as a relatively homogenous cross section similar to the Edenville right embankment.

Again, as on the Edenville side, according to the original design drawings, the upstream and downstream slopes of the embankments are nominally 2.5H:1V and 2H:1V, respectively. However, downstream slopes have been flattened and berms have been added in some locations, and survey data show that the downstream slope is steeper than 2H:1V in some locations and the upstream slope is steeper than 2.5H:1V in some locations.

2.3 Sanford Dam

At the time of the failure, Sanford Dam consisted of three embankments, a fuse plug spillway, a gated spillway, and a powerhouse, as shown in Figure 2. Sanford Lake had a surface area of about 1,550 acres[9] at a normal pool level of El. 630.8.

Source of aerial image: Google Earth

Figure 2-3: Sanford Dam Configuration

The Sanford left embankment extended from the left abutment to the powerhouse, a distance of about 160 feet. The maximum height of this embankment section was about 34 feet.

The powerhouse and gated spillway are a combined structure. The Sanford Dam gated spillway contains six Tainter (radial) gates. Gate No. 1 and Gate No. 6 are each 25 feet 4 inches wide and 10 feet high, and the remaining four gates (Gates 2 through 5) are each 22 feet wide and 10 feet high. Spillway Bay No. 5 included a low-level gated sluiceway, which was reportedly not operable for decades prior to the failure. The gate sills are at El. 622.3, 8.5 feet below the normal lake level. The powerhouse contains three vertical-shaft generating units with a rated capacity originally of 1.1 megawatts each, for a total of 3.3 megawatts, and upgraded in 2015 and 2016 to 1.2 megawatts each, for a total of 3.6 megawatts.

The Sanford Dam center embankment extended from the gated spillway to the fuse plug spillway in a dogleg pattern for a total distance of about 300 feet. The maximum height of this embankment section was about 34 feet, based on interpretation of test boring data.

The fuse plug spillway was 190 feet wide. The fuse plug spillway was designed in accordance with Bureau of Reclamation criteria (Reclamation 1985) and constructed in 2002. The design crest of the fuse plug was El. 634.8 feet. The design crest of the concrete slab at the bottom of the fuse plug was El. 631.8

9 The reservoir surface area shown is based on 2017 LiDAR data. Other project documents report several different values for surface areas at normal lake level.

feet (1 foot above the normal water surface elevation of 630.8 feet), and the design crest of the embankment was El. 636.8 feet. For a more detailed description of the fuse plug design and operational characteristics, see Section 3.3 and Appendix A-4.2.

The Sanford Dam right embankment extended from the fuse plug spillway to the right abutment, a distance of about 710 feet. The maximum height of this embankment section was about 36 feet.

2.4 Secord and Smallwood Dams

Secord Dam and Smallwood Dam did not fail and were not the focus of this investigation; hence, detailed descriptions of these two dams are not presented here, but are provided in Appendix A. Similar to Edenville Dam and Sanford Dam, Secord Dam and Smallwood Dam were both composed of embankments, gated concrete spillways, and powerhouses.

Secord Dam has two embankments with a maximum height of 56 feet, and a combined spillway and powerhouse structure. The spillway includes two Tainter (radial) gates: Gate No. 1 is 20 feet 6 inches wide by 10 feet high and Gate No. 2 is 23 feet 7 inches wide by 10 feet high. Gate No. 1 is adjacent to the powerhouse, which contains a single generating unit with a rated capacity of 1.2 megawatts.

As originally constructed, Smallwood Dam included two embankments with a maximum height of 36 feet, and a combined spillway and powerhouse structure. The spillway includes two Tainter (radial) gates, each 23 feet 5 inches wide by 10 feet 6 inches high. Gate No. 1 is adjacent to the powerhouse, which contains a single generating unit with a rated capacity of 1.2 megawatts.

Smallwood Dam was modified in 1999, 2001, and 2016-2017. In 1999, sheet pile walls were constructed along the upstream edge of the dam crest from the right abutment to the powerhouse and from the spillway for a distance of 350 feet to the left (north) in the left embankment and then downstream to the river channel. With the top of sheet pile wall at El. 715.7 and the top of the left embankment at El. 710, much of the left embankment was effectively converted into an auxiliary[10] spillway. In 2001, the area of the embankment between the sheet piles and the left abutment was regraded lower to El. 708.7 to enhance the flood-routing capability of the auxiliary spillway. The auxiliary spillway configuration was further modified in 2016-2017 by raising the embankment crest to El. 712, from the sheet pile wall for a distance of 260 feet to the left. This last modification was made to improve erosion protection near the sheet pile wall. The combination of all of the modifications effectively created a two-level auxiliary spillway: a 540-foot length at El. 708.7 and a 260-foot length at El. 712.

2.5 Spillway Gate Hoists

The original spillway gate hoist systems were essentially the same at all spillways at all four dams. Each spillway was equipped with electric hoists mounted on a cart that could be moved on rails from gate to gate, as shown in Figure 2. The electric hoist connected to a lifting bridal, as shown in Figure 2.

This configuration created a physical limitation on the gate opening height. Chains attached to angles at the bottom corners of each gate were connected to a yoke at a higher elevation near the center of each gate. The yoke was attached to the lifting chain that travels through an opening on the hoist rail frame. During the gate lifting process, the yoke on the chain lifting bridal catches on the underside of the hoist rail frame after about 6 to 7 feet of opening (depending on the specific dimensional characteristics of each spillway). With this limitation, the gates could not be lifted clear of the spillway flow with the lake

[10] In industry practice, both the terms auxiliary spillway and emergency spillway have been used for this type of spillway. For the purposes of this report, the term auxiliary spillway is used.

surface a small amount (1 to 2 feet) higher than the normal lake elevations. Hence, flow through the gates during large flood releases would likely be orifice flow.

Figure 2-4: Electric Hoist at Edenville

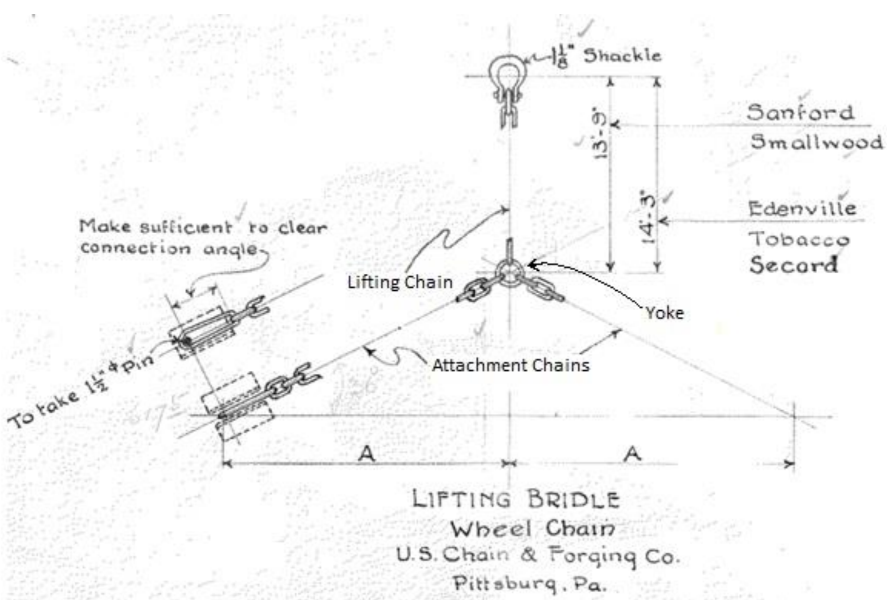


Figure 2-5: Configuration of Original Gate Hoist Lifting Bridal

The limitations on the gate hoists were noted in the report of gate opening tests completed on June 16, 1996 (Wolverine 1996), which indicated that the Tittabawassee spillway gates were opened to about 8 feet and the Tobacco spillway gates were open to between 5.5 and 6.5 feet. A letter from FERC to Wolverine Power, dated April 20, 1998 (FERC 1998a), stated: “Some of the gates at your projects were not fully opened in the last five years. Please ensure that this is accomplished in the spring of this year.” The 2001 CSIR report for Sanford Dam (Boyce Hydro 2002a) also stated: “The existing hoist system cannot lift the gates completely out of the water.”

Although it appears that there were some communications between the project owners (Wolverine, Synex, and Boyce Hydro) and FERC regarding the gate openings from 1998 to 2012, efforts to increase the gate opening capability were not made until 2012. A letter from Boyce Hydro to FERC, dated May 29, 2012 (Boyce Hydro 2012b), stated:

> “We have conducted tests on most of the gates and have found they can be opened from 8 to 9 feet under current conditions. The current hoisting operation is limited by the design of the chain lifting bridle that is fastened to the bottom of the gate…. The limitation that exists is when the yoke at the top of the bridle strikes the underside of the hoist rail frame. This limits the lift of the gate to between 8 and 9 feet depending on the geometry at each site.”

The gate opening height was later corrected to “6 to 7 feet” in a letter from Boyce Hydro to FERC, dated October 29, 2012 (Boyce Hydro 2012c).

Boyce Hydro proposed a plan to be able to open all gates to a height of at least 10 feet by fabricating a supplemental lifting system consisting of a portable steel A-frame, shown in Figure 2 and Figure 2, and secondary “safety” chains that connected to the existing connection points on the gate, shown in Figure 2 and Figure 2. Modifications were performed at each of the dams to accommodate the A-frame system, but it was not until 2015 (Boyce Hydro 2015c) when the lifting capabilities were increased at all four dams. The A-frame system was implemented at Secord Dam, Smallwood Dam, and Edenville Dam starting in 2015. The system at Sanford Dam was also modified in 2015 so that the gates could be fully opened with electric hoist systems without needing to use an A-frame system (Boyce Hydro 2015d).

Figure 2-6: Spillway Gate A-Frame Hoisting System (Boyce Hydro 2012b)

Figure 2-7: A-Frame Hoisting System in Place at Tobacco Spillway

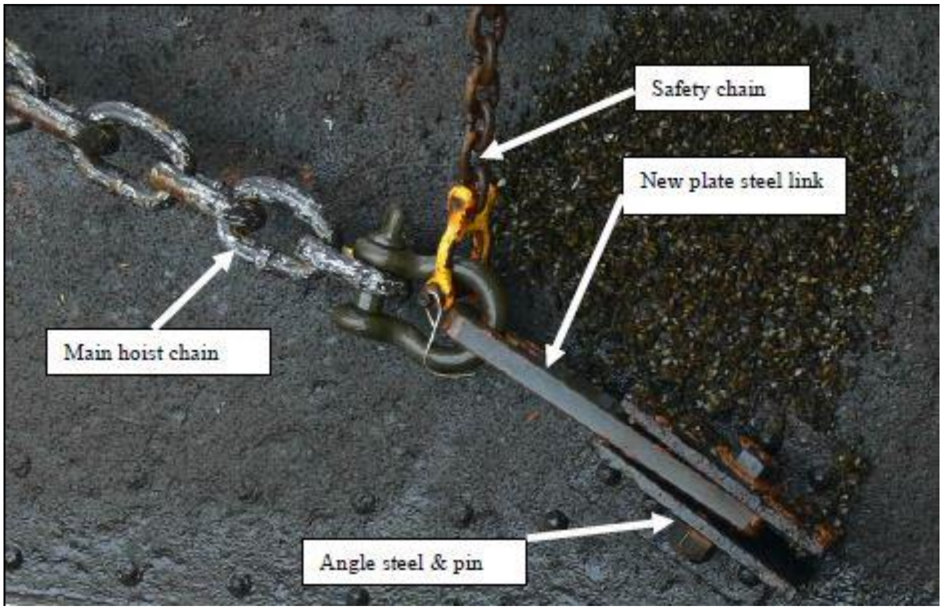


Figure 2-8: A-Frame Connection to Gate (Spicer 2019)

Figure 2-9: A-Frame Induced Bending at Connection to Gate (Spicer 2019)

A letter from Boyce Hydro to FERC, dated August 31, 2015 (Boyce Hydro 2015d), indicated the following gate opening capabilities:

- Four of six gates at Sanford Dam tested to a lift of 11.3 feet; one of the gates tested to a lift of 11.2 feet; and the sixth gate (adjacent to the powerhouse) tested to a lift of only 8.5 feet because of a restriction caused by a window frame in the powerhouse.
- Five of the six gates at Edenville Dam (Tobacco and Edenville spillways) could be lifted 11.2 feet with the A-frame system; Gate 1 at the Edenville spillway (adjacent to the powerhouse) could be lifted only 8 feet with the A-frame because of a restriction from bulging concrete, which has since fallen off; all six gates could be opened 7.5 feet without the A-frame.
- The two gates at Smallwood Dam could be lifted 9.5 feet and 11 feet with the A-frame and only 7 feet without the A-frame.
- The two gates at Secord Dam could be lifted 11.2 and 11.5 feet with the A-frame and only 8 feet without the A-frame.

The gate lifting mechanisms at Smallwood Dam and Secord Dam were subsequently modified using hydraulic hoist systems so that the gates at those dams could be fully opened[11] without the A-frame system. According to discussions with Boyce Hydro personnel, these modifications occurred in 2017 for Smallwood Dam and in 2019 for Secord Dam. Hence, after 2019, Edenville Dam was the only one of the four dams where the gates could not be opened fully without the A-frame system.

[11] In this report, "fully open" with respect to the gates means that the bottom of a gate could be lifted vertically to a distance equal to the gate height.

A gate operation test was completed at Edenville Dam on June 14, 2019. The results were documented in a report from the Spicer Group (Spicer) to EGLE, dated September 18, 2019 (Spicer 2019). That report included the following statements:

- "The maximum gate opening for the hoist system is 6 feet. The gates cannot be lifted higher due to the limitations of the cart height above the bridge deck and the center yoke not allowing further travel."
- "… the safety chains [used in the A-frame system] appear to induce a bending moment on the upper steel angle ..."
- "The engineers present were concerned that attempting to open the gates to the height required by FERC (approximately 14 feet) or required by EGLE (approximately 12 feet) was unsafe, especially in light of the observed bending moment being induced by the safety chain arrangement."
- "The current engineering opinion, based on observation of the gate tests, is the gates at Edenville should only be operated with the original hoist mechanisms until they can be replaced with electric hoists. Using the portable A-frames and the manual lever hoist is cumbersome and requires too much time to operate under emergency conditions. Most importantly, though, the A-frames require that a minimum of three operators be at each site. Those operators would be exposed to unsafe conditions, which is unacceptable."

From discussions with the parties involved, the IFT understands that plans were being developed to modify the Edenville Dam gate hoists late in 2020 to allow for full opening, but at the time of the May 17 through May 19 flood, the engineering guidance was that use of the A-frames was unsafe. During the May 2020 event, the gates were opened to about 7 feet with limited use of the A-frames (see Section 3.1).

3. Chronology of the Failures

3.1 Precipitation, Lake Levels, and Gate and Powerplant Operations

On Saturday, May 16, 2020, 3 days before the failure, the levels of the four lakes[12] impounded by the Secord, Smallwood, Edenville, and Sanford Dams (the four Boyce Hydro dams) were all slightly below the normal operating lake elevations. On that day, AccuWeather was forecasting significant rainfall in the coming days.

Table 3 lists daily rainfall totals at the four Boyce Hydro dams from May 1 through May 19, 2020. As the table shows, the total rainfall amounts for May 17 through May 19, 2020, were 5.90 inches, 3.69 inches, 3.76 inches, and 2.95 inches for Secord Dam, Smallwood Dam, Edenville Dam, and Sanford Dam, respectively. At all four dams, the vast majority of the total rainfall occurred on Monday, May 18, 2020 – about 96 percent at Secord, 100 percent at Smallwood, 82 percent at Edenville, and 95 percent at Sanford. For additional information on the May 2020 rainfall event, see Section 5.2.

Table 3-1: Rainfall at the Four Boyce Hydro Dams from May 1 through May 19, 2020

Daily Rainfall Totals, inches				
Date	**Sanford**	**Edenville**	**Smallwood**	**Secord**
5/1	0	0	0.03	0
5/2	0	0	0.03	0.03
5/3	0	0	0	0
5/4	0	0	0	0
5/5	0	0	0	0
5/6	0	0	0	0
5/7	0	0	0	0
5/8	0	0	0	0
5/9	0	0	0	0
5/10	0	0	0	0
5/11	0.03	0	0	0.06
5/12	0	0	0	0
5/13	0	0	0	0
5/14	0.27	0.17	0.14	0.13
5/15	0.73	0.75	0.40	0.83
5/16	0	0	0	0
5/17	0.2	0.46	0	0
5/18	2.79	3.08	3.69	5.67
5/19	0.16	0.32	0	0.23

[12] Lake levels are based on data from reservoir level recorders provided by Boyce Hydro for times when recorded data are available. The recorded data are supplemented with eyewitness accounts and results of the IFT's hydrologic/hydraulic model for times when recorded data are not available.

Daily Rainfall Totals, inches				
Date	**Sanford**	**Edenville**	**Smallwood**	**Secord**
Total 5/17 through 5/19	2.95	3.76	3.69	5.90
Total for May 1 through May 19	**4.18**	**4.78**	**4.29**	**6.95**

Some minor adjustments were made to the gates at Edenville Dam and Sanford Dam on Saturday and Sunday, May 16 and May 17, 2020. After the FERC license for Edenville was revoked in 2018, electricity could not be generated and water could not be released through the powerhouse, and the low-level sluiceways had apparently been inoperable for decades. The spillway gates were the only facilities available to control the lake level at Edenville Dam within the operating rules, and the Edenville Dam gates were therefore operated more frequently, both in closing and opening, than gates at the other three dams. At the end of the day (11:59 p.m.) on Sunday, May 17, the levels of all four lakes were slightly below the lower limits established in their operating rules.

During the early morning hours overnight on Monday, May 18, the lake levels at all four dams either remained constant or dropped, remaining at levels below the normal operating levels. However, in the predawn hours, the lake levels at Secord, Smallwood, and Edenville Dams began to rise. The lake level at Sanford Dam was reported to begin to rise at about midday on May 18.

Gates were reportedly opened at all four dams throughout the day on Monday, May 18, 2020, beginning at 7:00 a.m. By about 3:30 p.m. that afternoon, all gates at Secord, Smallwood, and Edenville Dams were open. There were reportedly no further gate operations at these three dams before the failure of Edenville Dam. Gate operations at Sanford Dam continued until about 8:00 p.m. on May 18, after which there were reportedly no further gate operations at any of the Boyce Hydro dams before the failure of Edenville Dam.

The IFT had two sources of information for the gate opening heights at the various spillways at the Boyce Hydro dams: (1) handwritten logs attached to the Boyce Supplemental Incident Report submitted to FERC (Boyce Hydro 2020b), and (2) information obtained from independent discussions with Boyce Hydro operators who were on-site on May 18 and May 19. The reported openings from the two sources are summarized in Table 3-2.

Table 3-2: Reported Gate Openings from Two Sources

Dam / Spillway	Gate Number	Reported Gate Opening as of 8:00 P.M. on May 18, 2020	
		Handwritten Logs	**Operator Interviews**
Secord	1	6 feet	9 feet
	2	7 feet	10 feet
Smallwood	1	9 feet	10 feet
	2	9 feet	10 feet
Edenville / Edenville Spillway	1	7 feet	7 feet
	2	7 feet	7 feet
	3	7 feet	7 feet

Dam / Spillway	Gate Number	Reported Gate Opening as of 8:00 P.M. on May 18, 2020	
		Handwritten Logs	Operator Interviews
Edenville / Tobacco Spillway	1	7 feet	7 feet
	2	7 feet	7 feet
	3	7 feet	7 feet
Sanford	1	7 feet	9 feet
	2	7 feet	10 feet
	3	7 feet	10 feet
	4	7 feet	10 feet
	5	7 feet	10 feet
	6	7 feet	10 feet

All three operators reported that the gates at Edenville Dam were lifted with a combination of the original chain hoist system and sparing use of the supplemental A-frame system. The A-frame system was used sparingly because of the recommendations resulting from the 2019 gate tests (Spicer 2019), which raised concern for personnel safety and potential damage to the gates with use of the A-frames.

All three operators reported that the gates at the three other dams were lifted using the new hoist systems that had been installed since 2012 – individual electric hoists for Sanford Dam and individual hydraulic hoists for Secord Dam and Smallwood Dam.

After careful consideration of the two sources of information on gate openings, the IFT judged the operators' reports to be more reliable for the following reasons:

- The three operators were questioned separately, and all three reported the same gate openings.
- With the new hoists at Secord, Smallwood, and Sanford Dams, there was nothing that prevented safely opening the gates fully, and the operators would have had no reason to limit openings while they were trying to pass the flood flows.

Figures in Appendix F1 (F1-27, F1-28, and F1-29) show the gate openings for Sanford and Edenville dams after the failure. With respect to effects on the resulting water level in Wixom Lake during the event, the gate openings at Sanford Dam would have no impact, since Sanford Dam is downstream of Edenville Dam. The gate openings at Secord Dam and Sanford Dam would affect the rate at which floodwater was passed through these dams and the resulting rise in Wixom Lake. The discrepancies in gate openings at Smallwood Dam from the two sources is relatively small – only about 1 foot at each gate. The discrepancies at Secord Dam are larger – about 3 feet at each gate. The gate opening information for Edenville Dam is the same for the two sources.

At the beginning of the event, power generation operations and releases were normal at Secord, Smallwood, and Sanford Dams. As noted above, there were no power generation operations or releases through the powerhouse at Edenville Dam because the hydropower license had been revoked. On May 19, the day of the failure, individuals at the site discussed the possibility of releasing water through the turbines to help limit the rise of the lake, but it was concluded that attempting to run the turbines without power delivery was unsafe, due to risk of catastrophic equipment failure which would pose a hazard to personnel, and therefore no powerhouse operation was attempted.

Power generation operations and releases at Secord Dam continued throughout the event. However, power generation operations and releases were stopped at Smallwood Dam on Monday, May 18, at 9:00 p.m., and at Sanford Dam on Tuesday, May 19, at 1:00 a.m. in accordance with operating practices during floods to protect the generating units, as is typical at most hydropower plants during large floods.

The lake levels continued to rise at all four dams throughout Monday, May 18, and the morning of Tuesday, May 19. Table 3 summarizes the lake levels at the end of the day (11:59 p.m.) on Monday, May 18; at 6:00 a.m. on Tuesday, May 19; and at noon on Tuesday, May 19. Most of the rise in lake levels at Secord, Smallwood, and Edenville Dams began in the afternoon and evening of Monday, May 18, and the gates at those three dams had been opened by the afternoon of that day. Most of the lake level rise at Sanford Dam began in the evening of May 18, and the gates at Sanford Dam had been opened by the evening of that day.

Table 3-3: Reported and Estimated Lake Levels at 11:59 P.M. May 18 (Monday), 6:00 A.M. May 19 (Tuesday), and Noon May 19 (Tuesday), 2020

Dam	Lake Levels		
	11:59 p.m. May 18	6:00 a.m. May 19	Noon May 19
Secord	El. 752.3 1.5 feet above normal lake level	El. 753.0 2.2 feet above normal lake level	El. 753.2, 2.5 feet above normal lake
Smallwood	El. 708.0, 3.2 feet above normal lake level	El. 709.2, 4.4 feet above normal lake level	Estimated to be 710.3, 5.5 feet above normal lake level
Edenville	El. 678.0, 2.2 feet above normal lake level	El. 679.4, 3.6 feet above normal lake level	Not available
Sanford	El. 630.9, 0.1 feet above normal lake level	El. 631.7, 0.9 feet above normal lake level	El. 632.3 feet, 1.5 feet above normal lake level

The water level at Secord Dam peaked at about El. 753.5 feet, about 4.3 feet below the embankment crest at 1:00 p.m. on Tuesday, May 19, about 4 hours and 35 minutes before the failure of Edenville Dam. At Smallwood Dam, the water level was El. 709.2 feet at about 6:00 a.m. on May 19, the time of the last recorded water level at Smallwood Dam. The lake level at Smallwood Dam is believed to have continued to rise throughout the day, cresting at about 710.3 feet in the late afternoon. The water levels at Edenville and Sanford Dams after noon on May 19 are discussed below.

3.2 Edenville Dam Failure

At midnight on Sunday, May 17, Wixom Lake, the lake impounded by the Edenville Dam, was at about El. 675.4, about 0.4 foot below the normal lake level, at the lower limit of the authorized summer operating range.[13] The lake level began to rise at about 5:00 a.m. on Monday, May 18, and continued to rise until the time of the Edenville Dam failure about 36.5 hours later, at about 5:35 p.m. on Tuesday, May 19, see Figure F1-30 in Appendix F1.

Spillway gates at both the Tobacco and Edenville spillways at Edenville Dam were operated between 7:00 a.m. and 3:30 p.m. on Monday, May 18. At 3:30 p.m. that day, all six gates, three at the Tobacco spillway and three at the Edenville spillway, had been opened as described above and remained at those openings until the time of the failure.

[13] A Boyce Hydro incident report submitted to FERC indicates that, at the end of the day on May 17, Wixom Lake was 0.44 foot below normal pool level, or slightly below the lower limit of the authorized range.

At about 1:00 p.m. on Monday, May 18, Wixom Lake reached the normal lake level, El. 675.8. The next day, Tuesday, May 19, at about 1:00 a.m., the lake reached the previous pool of record,[14] El. 678.3. According to the Boyce Hydro incident report to FERC, at 3:30 a.m. that day, the lake was at least 3 feet above the normal lake level. Wixom Lake continued to rise throughout the day on May 19, until the time of the failure. Measurements of Wixom Lake water levels are not available for the afternoon of May 19. Based on eyewitness accounts and evaluation of photographs, the IFT estimates that the water level of the lake at the time of the failure was in the range of about El. 681 to El. 681.5. This lake level is about 3 feet higher than the previous pool of record, about 5.5 feet higher than the normal lake level, and about 1 foot to 1.5 feet below the estimated pre-failure crest elevations of the Edenville left embankment.

The first report of significant damage at Edenville Dam was at daybreak on Tuesday, May 19, when two of the operators reported erosion and sloughing near the top of the upstream slope of the east end of the Edenville Dam embankments. The Chief Operator and the Assistant Chief Operator, both Boyce Hydro employees, traveled to Edenville Dam. They were joined at the site later that morning by individuals from EGLE, Fisher Contracting, and Spicer. The Chief Operator was also in contact with Boyce Hydro ownership in Nevada that morning. A Boyce Hydro incident report submitted to FERC states, “The engineers and contractor met at the dam between 10:30–11:00 a.m. They all walked the length of the dam and noted no leakage or damage to the embankment on the downstream slopes.” The report further states, “It was agreed that Fisher Contracting would attempt to mitigate the erosion of the embankment by placing turbidity barriers, sandbags, and geocloth on the reservoir face of the dam....” Information collected by the IFT in interviews confirms that Fisher Contracting spent the rest of the day deploying erosion control measures on the upstream face of the Edenville Dam embankments. At the time of the Edenville Dam failure, crews were reportedly working in the vicinity of the Boyce Hydro office, to the right of the Edenville spillway and the powerhouse.

Early in the afternoon, a large depression was observed on the downstream slope of the Tobacco left embankment, adjacent to the Tobacco spillway. This depression is believed to have been caused by erosion from currents in the spillway discharge.

As the lake continued to rise throughout the day, the parties were concerned about both the erosion on the upstream slope and the possibility that the lake would rise high enough to overtop the embankments. The possibility of attempting a controlled breach of Edenville Dam was being considered as the lake continued to rise in the afternoon, with the likely controlled breach location at the west (right) end of the Tobacco embankments, where the height of the embankment was small. At the time of the dam failure, the controlled breach was still being considered, but the decision had been deferred because the rate of rise of the lake had slowed from an estimated 2 inches per hour before 3:00 p.m. to an estimated 1 to 1.5 inches per hour after 3:00 p.m.

Because the crews were not deploying erosion control measures on the Edenville left embankment (at the east end), limited attention was paid to that section of embankment, which was the location of the failure. The IFT was not able to find any record of close-up examination of the Edenville left embankment during the afternoon of Tuesday, May 19. In interviews, the IFT heard several eyewitness accounts of observations of the Edenville left embankment made from the lakeshore upstream and from the Consumers substation downstream before the failure, during the failure, and after the failure. The IFT was

[14] “Pool of record” is the highest known lake level in the history of the project. Based on available information, the previous pool of record at Wixom Lake was El. 678.3, 2.5 feet above normal pool level. The previous pool of record occurred on April 5, 1929.

also able to collect a number of still photography and video images of the Edenville left embankment, which were provided by the eyewitnesses.

Other than the upstream erosion, the IFT was not able to find any reports of distress in the Edenville left embankment on Tuesday, May 19, prior to about 5:00 p.m. Eyewitnesses reported observing a depression or subsidence in a section of the Edenville left embankment crest at about 5:00 p.m., about a half hour before the failure, and this observation is supported by photographic evidence, as shown in Figure 3 and Figure 3. Another photograph taken at 2:52 p.m. that afternoon does not show a depression in the crest (Figure 3). The photographs in Figure 3 through Figure 3 were taken by local residents who were observing the lake and the dam from the upstream left (east) bank of the lake.

After observing the depression in the crest, several residents walked to the Consumers substation located downstream of the Edenville left embankment. The photograph of the downstream side of the embankment in Figure 3 was taken at 5:31 p.m. that day. The photograph shows the depression in the crest, but otherwise does not show any significant signs of distress in the embankment. Figure 3 shows a lighter-colored area near the toe of the embankment at the ultimate failure location. The IFT considered the possibility that this light color might indicate seepage, but after reviewing other photographs and talking with eyewitnesses, the IFT concluded that the light color did not appear to be seepage. Historical photographs showed significant variability of color on the face of the embankment. However, because Figure 3 is a distant view of the embankment, the possibility that the color difference represents seepage cannot be ruled out.

Figure 3 shows a view of the depression in the crest from upstream at 5:33 p.m. The photograph in Figure 3 also shows the erosion that had occurred on the upstream slope of the embankment.

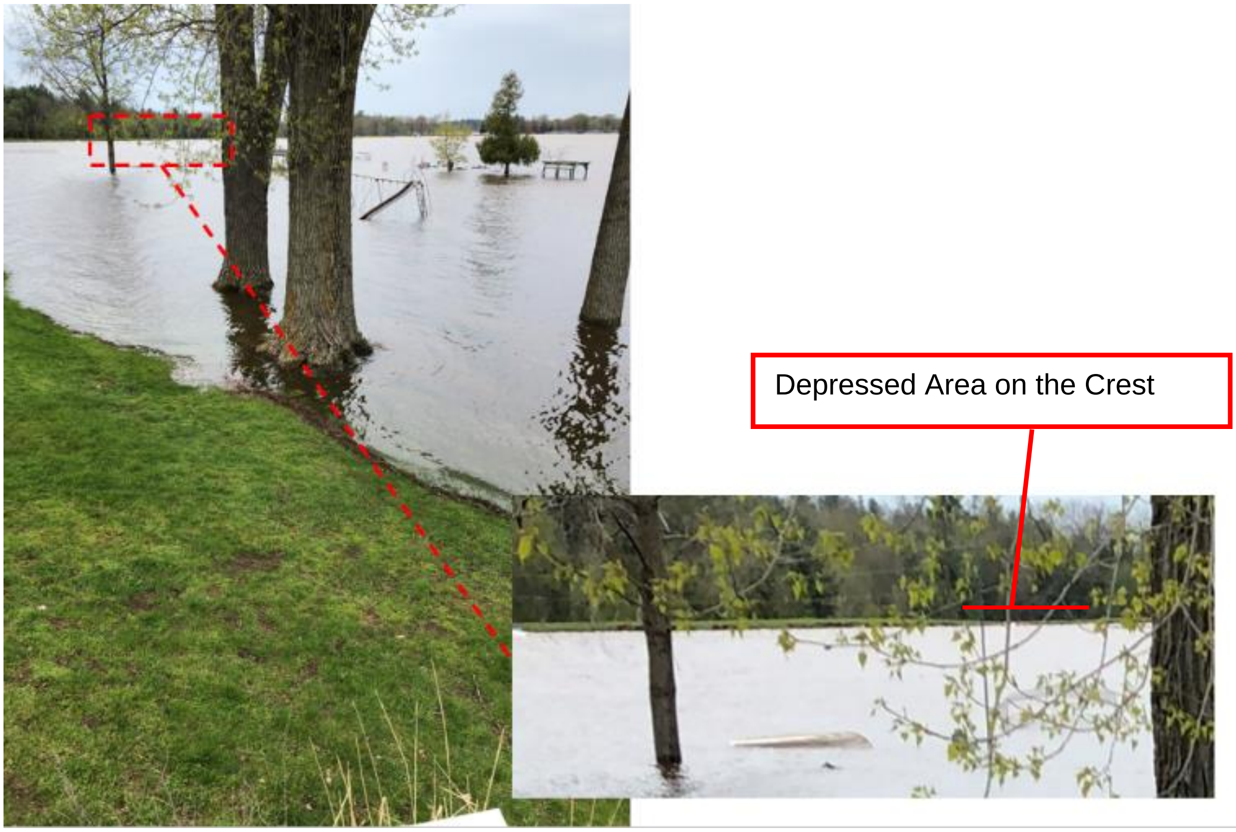


Figure 3-1: Photograph Showing Depression in the Crest of the Edenville Left Embankment at 5:03 P.M., Tuesday, May 19, 2020 (photo courtesy of local resident)

Figure 3-2: Photograph Showing Depression in the Crest of the Edenville Left Embankment at 5:20 P.M., Tuesday, May 19, 2020

Figure 3-3: Photograph Showing the Crest of the Edenville Left Embankment at 2:52 P.M., Tuesday, May 19, 2020

Figure 3-4: Photograph Showing the Downstream Side of the Edenville Left Embankment at 5:31 P.M., Tuesday, May 19, 2020

Figure 3-5: Enlarged Photograph Showing the Upstream Side of the Edenville Left Embankment at 5:31 P.M., Tuesday, May 19, 2020

Shortly after the photographs shown in Figure 3 and Figure 3 were taken, the eyewitnesses reported observing a stream of water flowing down the downstream face of the dam, which prompted one of the eyewitnesses, local resident Mr. Lynn Coleman, to begin a video recording on his phone at 5:35 p.m. His video is referred to in this report as "the dam failure video." Several still images from the video are shown in Figure 3 through Figure 3:

- Figure 3 is at the beginning of the video (0 seconds, 5:35 p.m.) and shows the depression in the crest and the stream of water on the downstream face, but no other obvious signs of distress.
- Figure 3, at 4 seconds, shows a slight bulge in the downstream toe and possibly an increase in the depression at the crest.
- Figure 3, at 6 seconds, shows a significant increase in the toe bulge and the depression in the crest.
- Figure 3 through Figure 3 show the progression of the embankment failure over the subsequent 8 seconds, from 7 seconds to 15 seconds in the video, concluding with a large mass of soil deposited downstream of the toe of the dam.
- Figure 3, at 28 seconds, shows increasing water flow through the failed embankment location.
- By 36 seconds, Figure 3, a breach through the embankment has formed.

From the dam failure video, it can be concluded that the failure of the embankment occurred in no more than about 10 seconds and the breach developed between about 10 and 20 seconds later. It is also noted that the failure of the embankment was initially limited to a length estimated to be between 40 and 80 feet along the axis of the dam.

Over the next several hours the breach widened, and Wixom Lake was drained. Four images of the widening of the breach are shown in Figure 3 through Figure 3.

Figure 3-6: Enlarged Still Image from Dam Failure Video at 0 Seconds

Figure 3-7: Enlarged Still Image from Dam Failure Video at 4 Seconds

Figure 3-8: Enlarged Still Image from Dam Failure Video at 6 Seconds

Figure 3-9: Enlarged Still Image from Dam Failure Video at 7 Seconds

Figure 3-10: Enlarged Still Image from Dam Failure Video at 8 Seconds

Figure 3-11: Enlarged Still Image from Dam Failure Video at 10 Seconds

Figure 3-12: Enlarged Still Image from Dam Failure Video at 15 Seconds

Figure 3-13: Enlarged Still Image from Dam Failure Video at 28 Seconds

Figure 3-14: Enlarged Still Image from Dam Failure Video at 36 Seconds

Figure 3-15: Edenville Left Embankment Breach at 6:00 P.M. on Tuesday, May 19, 2020

Figure 3-16: Edenville Left Embankment Breach at 6:24 P.M. on Tuesday, May 19, 2020

Figure 3-17: Edenville Left Embankment Breach at 7:13 P.M. on Tuesday, May 19, 2020

Figure 3-18: Edenville Left Embankment Breach at 9:04 P.M. on Tuesday, May 19, 2020

3.3 Sanford Dam Failure

The outflow from the Edenville Dam failure flowed into Sanford Lake. At about 7:19 p.m. on Tuesday, May 19, the lake level at Sanford Dam reached El. 634.8, the crest of the fuse plug spillway. Water began to flow over the fuse plug spillway, but the lake level still continued to rise. At about 7:46 p.m., the lake level at Sanford Dam reached El. 636.8, the low spot on the embankment crest, and continued to rise. Shortly afterward, an overtopping failure of Sanford Dam occurred and a large portion of the right embankment eroded and released the stored reservoir water. The failure flood from Edenville Dam plus the contents of Sanford Lake were released, inundating the areas downstream. A view of the failed Sanford Dam is shown in Figure 3.

Given the failure of Edenville Dam, the failure of Sanford Dam was not unexpected. As noted in the Boyce Hydro incident report (Boyce Hydro 2020a), "It was always understood by the regulators and engineers involved that should a breach occur at Edenville, Sanford would necessarily fail."

During the 27-minutes between initial flow over the fuse plug spillway and overtopping of the embankments, significant erosion and head cutting occurred along the downstream slope of the fuse plug, but the crest remained largely intact, as shown in Figure 3 through Figure 3. It is not clear when the crest of the fuse plug completely eroded through, but it appears to have been after overtopping of the embankments occurred (see Figure 3 and Figure 3). Once complete erosion of the fuse plug had occurred, there was a significant increase in reservoir release capacity (around 6,000 cfs for a reservoir level at the dam crest), but that capacity combined with the gated spillway capacity was not nearly enough to accommodate the inflow resulting from the Edenville Dam breach. Partial breach of the right embankment began around 8:20 p.m., as shown in Figure 3.

As discussed in Appendix A, the erosion of the fuse plug was likely slowed by the relatively flat downstream slope (4H:1V), the relatively wide fuse plug crest (8 feet), the lack of pilot channels in the crest, possible compaction of the fuse plug crest from vehicle traffic, and vegetation on the fuse plug crest and downstream slope. However, the discharge from the Edenville Dam breach was significantly greater than the combined capacity of both spillways at Sanford Dam. It is the IFT's opinion that the outcome of the May 2020 event would have been essentially the same even if the fuse plug had eroded more quickly.

Figure 3-19: Sanford Dam Failure, Viewed from Upstream (photo courtesy of EGLE)

Figure 3-20: Erosion along Toe of Fuse Plug Prior to Overtopping (photo courtesy of EGLE)

Figure 3-21: Initial Overtopping of Fuse Plug at about 7:19 P.M. (photo courtesy of EGLE)

Figure 3-22: Erosion of Fuse Plug Embankment and Head Cutting at about 7:25 P.M. (photo courtesy of EGLE)

Source: Michigan Drone Services

Figure 3-23: Still Image from Drone Video Approximately 7:46 P.M., Just Before the Initial Embankment Overtopping

Source: Michigan Drone Services

Figure 3-24: Still Image from Drone Video at Approximately 8:11 P.M. (Note almost complete washout of fuse plug and entire embankment being overtopped.)


Source: Michigan Drone Services

Figure 3-25: Still Image from Drone Video at Approximately 8:20 P.M. with Breach Beginning along Sections of Right Embankment.

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4. Physical Mechanisms of the Embankment Failures

4.1 Edenville Dam Failure

After consideration of a range of possible explanations for the physics of the failure of Edenville Dam on Tuesday, May 19, 2020, it is the IFT's opinion that a static liquefaction (flow) instability failure of the downstream slope of the Edenville left embankment is the most plausible primary physical mechanism for the failure, which occurred over a 40- to 80-foot length of the embankment. The IFT believes that the high lake level on May 19, about 3 feet higher than the previous pool of record, contributed to the instability failure but water did not overtop the Edenville embankments. Although there is uncertainty concerning the exact trigger or triggers of the static liquefaction failure, there are several phenomena that are plausible triggers, either individually or in some combination. After the downstream slope failure, the remnant upstream embankment section briefly held back the reservoir for about 10 to 20 seconds before the remnant embankment began to give way and the stored reservoir water was released through the breach.

In the discussion below, the IFT describes its consideration of potential failure mechanisms and the reasons for the conclusion that a static liquefaction instability failure is the most plausible mechanism. In evaluating potential failure mechanisms, the IFT weighed information collected from videos and still photographs taken prior to and during the failure and from interviews with eyewitnesses. The IFT also considered data from previous subsurface investigations and analyses; subsurface investigations performed after the failure; and soil mapping, testing, and seepage and stability analyses performed by the IFT. Table 4- lists a number of physical attributes and observations identified in the investigation that were considered in the evaluation of the failure mechanism. The confidence in the physical attributes and observations varied, as noted in the table. The highest confidence was assigned to direct videographic or photographic evidence, and the lowest confidence was assigned to eyewitness reports of a single eyewitness, which were not corroborated by other eyewitnesses or videographic or photographic evidence.

Table 4-1: Edenville Dam Failure Physical Attributes and Observations

Physical Attribute or Observation	Comments on Confidence
Less than 4 minutes, likely a matter of only seconds, before the failure, water was observed running down the face; no water exiting the face was observed before that time, though a lesser amount of seepage, which was not visually detectable from a distance, cannot be ruled out.	Strong confidence for water running down the face before failure. • The dam failure video clearly shows water running down the face just before the failure. • Eyewitnesses report that water exited the face just before the failure. Reasonably strong confidence for lack of substantial flow of water exiting the face earlier. • The videographer reports that the observation of water exiting the face was the reason that he started the cell phone video. • A still photograph at 5:31 p.m., 4 minutes before failure, does not show water exiting the face. This photo at 5:31 p.m. is from a distance (from the Consumers Energy substation), so it may not show lesser amounts of seepage exiting the face.

Physical Attribute or Observation	Comments on Confidence
Failure of the downstream section of the embankment occurred in about 10 seconds.	Strong confidence. • The dam failure video captures the failure of the downstream section.
The upstream embankment remnant failed, releasing the reservoir, about 10 to 20 seconds after the failure of the downstream section of the embankment.	Strong confidence. • The dam failure video captures the release of the reservoir.
During the failure, jets of water emanated from the failure mass.	Strong confidence. • The dam failure video clearly shows jets of water.
Lake level was at a historic high at the time of the failure, about 3 feet higher than the previous pool of record.	Reasonably strong confidence. • At the time of the failure, the lake level was estimated to be about 1 foot to 1.5 feet below the level of the unsettled embankment crest, based on eyewitnesses and photographs. This equates to a lake level between El. 681 and El. 682 based on a nominal embankment crest of El. 682.5 to El. 683 from previous surveys. • Available records indicate that the historic high-water level in the lake was El. 678.3, which occurred on April 5, 1929. This elevation is about 2.5 feet above normal lake level and about 4 feet below the embankment crest. • The available lake level records may be incomplete.
A section of the embankment crest in the failure location settled at least several inches, about 30 to 35 minutes before the embankment failed.	Reasonably strong confidence. • A photograph (at distance) in mid-afternoon (2:52 p.m.) shows a level crest. • Photos (at distance) at 5:03 p.m. and 5:20 p.m. show a lowered section of the crest in the area of the ultimate failure. • Eyewitnesses report that the depression in the crest occurred suddenly at about the time of the 5:03 p.m. photograph; observation of the depression is the reason the eyewitnesses walked to the downstream side of the dam, where they arrived minutes before the failure.
During the failure, velocities of the failure mass (both total and horizontal) reached 5 meters per second (16.4 feet per second) and displacements (both total and horizontal) exceeded 25 meters (82 feet).	Reasonably strong confidence. • Based on American Society of Civil Engineers (ASCE) team pixel-tracing analysis of the failure video (ASCE 2021)
The embankment failure was confined to the embankment and did not extend into the foundation.	Reasonably strong confidence. • The mass seen moving in the dam failure video clearly appears to be confined to the embankment, and does not extend into the foundation. • In the ASCE pixel tracing, Point P5 at the toe of the embankment did not show movement.
Cracks occurred in the settled section of the crest before the failure.	Moderate confidence. • Photos from both upstream and downstream appear to show a longitudinal crack, but the photos are from a distance. • Some eyewitnesses report observing cracks, possibly including transverse cracks.

Physical Attribute or Observation	Comments on Confidence
A circular flow of water in the lake (“whirlpool”) was suggested near the upstream face of the dam, based on the movement of floating debris.	Moderately weak confidence. • This was reported by only one eyewitness but was described in detail regarding the diameter and rotational rate of the flow. • Other eyewitnesses did not report this observation, even when directly asked, although this may be difficult to recognize.
Water exiting the downstream face of the dam at about two-thirds height jetted out horizontally about 10 to 20 feet prior to failure.	Weak confidence. • This was reported by only one eyewitness. • Other eyewitnesses did not report this observation, even when directly asked. • This is only possible if there was an open defect present (continuous upstream to downstream), which does not appear to be plausible.

4.1.1 Failure Mechanisms

Potential primary failure mechanisms considered for the failure of Edenville Dam were grouped into the following general categories:

- Embankment overtopping
- Internal erosion of the embankment or foundation
- Embankment instability

Each potential mechanism is discussed below.

4.1.1.1 Embankment Overtopping

Embankment overtopping during floods is a common cause of embankment dam failures and might well have been considered a likely cause of the Edenville Dam failure if not for the video graphic, photographic, and eyewitness evidence, particularly the dam failure video. Although the lake level at the time of the failure was approaching the crest, especially the depressed crest at the location of the failure, there is no evidence of water flowing across the crest and no evidence of water on the downstream face of the dam until less than 4 minutes, and probably just seconds, before the failure. While it appears that there is a small amount of water flowing over the depressed section of the crest at the beginning of the failure video, the video does not show any evidence of erosion of the downstream face of the embankment before the failure, as would be expected in an overtopping-induced failure. Overall, the characteristics of the failure in the video are not consistent with an overtopping failure. In addition, there are other possible explanations for the water that appeared on the downstream face moments before failure, as discussed further below. Consequently, the IFT concluded that overtopping was not a plausible mechanism for the failure of Edenville Dam.

4.1.1.2 Internal Erosion

The IFT looked closely at internal erosion as a potential mechanism for the failure because internal erosion is also a common cause of embankment dam failures, and internal erosion failures often occur when historic reservoir pool levels are exceeded. However, there are several factors that argue against internal erosion being the primary failure mechanism.

Past inspections have not indicated any significant seepage on the downstream face or on the ground surface at the downstream toe of the embankment at the failure location (only minor wet or soft spots had been noted at isolated locations of the Edenville left embankment). Until a stream of water flowing down the face of the dam was observed just before the failure, eyewitness accounts and photographic evidence do not indicate visually detectable seepage exiting at the ground surface on the dam or at the downstream toe prior to the failure.

There may have been increased seepage through the foundation drains immediately before the failure, which could not be detected in the dam failure video or by eyewitnesses. Historically, water has been observed flowing from foundation drain pipes that discharge into a ditch downstream of the Edenville left embankment, and there are historical reports of intermittent sediment in these drain outfalls. However, as explained below, the IFT does not believe that it is plausible that there was sufficient undetected seepage and soil transport through the foundation drains in the minutes and hours before the failure to produce a failure by an internal erosion mechanism.

There are no design or construction documents, nor any physical evidence, that indicate that an engineered filter was constructed within the embankment. However, construction specifications indicate that an attempt may have been made to create a bi-zoned embankment, with fine-grained relatively impervious soil placed upstream of the embankment centerline and more pervious fill placed downstream of the centerline. Available soil gradation analyses from historical explorations and from post-failure investigations indicate that the materials in the dam and its foundation consist of sands, silty and clayey sands, and sandy clays and silts. No pockets of gravel or significant amounts of gravel particles were encountered. Based on gradation analyses, the sands would function as filters for the finer soils. Without the presence of coarse gravel pockets or zones, the free face of the embankment and foundation or an unfiltered foundation drain pipe are the most probable seepage exits that would allow any eroding sand, silt, or clay to be removed. Open joints or cracks in the foundation drain pipe would not allow a significant volume of material to be eroded rapidly. With no signs of significant seepage or eroding material on the dam face, the available information does not support the sudden breach being principally caused by internal erosion. Based on mapping of the remnant breach faces, it is uncertain whether the attempt at zoning described above occurred consistently along the embankment, or even at all. Regardless of whether the embankment was bi-zoned or homogeneous, the conclusion would be the same: the lack of observed seepage or embankment distress prior to the sudden failure does not support internal erosion as the primary failure mechanism.

It is possible that animal burrows or deteriorating remnants of wooden trestles or railroad ties were present within the embankment. Such features could lead to localized internal erosion or collapse within the embankment, but not to a full internal erosion failure mode without breakout of seepage on the downstream slope or at the toe to initiate backward erosion piping. As noted above, significant seepage breaking out on the surface has not been reported historically for the Edenville left embankment.

The embankment is founded on a soil foundation consisting of a sand layer overlying glacial till (hardpan) that is at least 20 to 40 feet thick. Top-of-rock is believed to be deep at the site. The shallow foundation sand layer is medium dense to dense with varying amounts of silt and would have some erosion resistance under the estimated seepage gradients. Seepage through this layer would most likely daylight near the embankment toe, where eroding material would have been visible. There are no indications of other materials or other features in the foundation that would serve as likely pathways for internal erosion, particularly without the observation of seepage breakout near the toe.

Finally, the observed physical characteristics of the failure are not generally consistent with an internal erosion failure mode:

- No seepage exiting the ground surface was detected; in fact, no water was detected on the downstream ground surface until just before failure.
- No turbid water discharge was detected.
- No evidence near the time of failure of a developing open pipe, sinkhole, or progressive sloughing that might indicate global backward erosion piping was occurring.
- The kinetics of the failure, in particular the global acceleration and velocity of the failure mass, are not consistent with historical observations of internal erosion failures.

In the IFT's opinion, internal erosion may have contributed to the depression in the crest that was observed about 35 minutes before the failure, and possibly changes in the phreatic surface and pore water pressures, but it does not explain the primary physics of the failure.

4.1.1.3 Embankment Instability

In the IFT's opinion, the video of the failure shows a rapid instability failure of a significant portion of the downstream section of the dam, followed shortly after by a collapse of the upstream remnant of the embankment. Particularly striking characteristics of the instability failure are the apparent acceleration and velocity of the failure mass, which is indicative of a flow instability failure.

Such failures can only occur with soils that have a brittle, strain-weakening[15] stress-strain curve in which the mobilized strength drops substantially with relatively little strain. This behavior is illustrated by the solid line in Figure 4, which shows the stress-strain curve for a saturated, loose sand in *undrained* shear – shear without drainage of the pore water from the soil mass. As shown in Figure 4, initially the shear stress increases from an initial static shear stress as the sample is strained (deformed) until it reaches a peak value (peak shear strength), after which the shear strength drops quickly with further straining to a much lower strength (steady-state or residual strength), which is lower than the initial static shear stress. For saturated, loose sands, the peak strength can occur at a small strain (e.g., 1 percent or less), and the large decline to residual strength can occur with relatively little additional strain.

For comparison, the dashed line in Figure 4 illustrates ductile stress-strain behavior in which there is little or no reduction in shear strength with further straining after the peak strength is reached, which is typical of a saturated, loose sand in *drained* shear.

For brittle, strain-weakening stress-strain behavior, if the steady-state or residual strength is much less than the static shear stress, the rapid and substantial strength decrease can cause sufficient force imbalance to create acceleration and velocity, as observed in the Edenville Dam failure. The large imbalance between applied forces (stresses) and resisting forces (strengths) causes acceleration of the soil mass, according to the familiar physics equation: Force = Mass x Acceleration.

[15] In the geotechnical literature this stress strain behavior has most often been referred to as "strain-softening." In a comment on the IFT's interim report, Dr. Kaare Hoeg, former director-general of the Norwegian Geotechnical Institute, suggested that "strain-weakening" is a better term, because the behavior is more related to reduction in strength than to reduction in stiffness. The IFT agreed and adopted strain-weakening for this report.

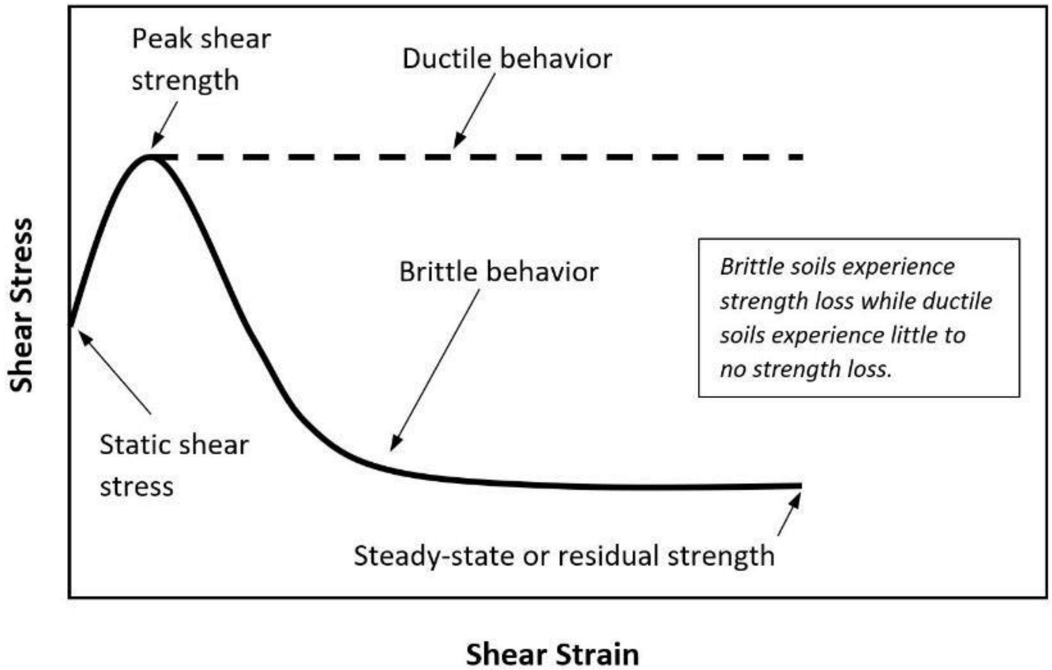


Figure 4-1: Brittle, Strain-Weakening Stress-Strain Curve

Static Liquefaction

In the IFT’s opinion, the Edenville Dam failure is consistent with a static liquefaction failure of loose, saturated sands in the downstream section of the dam, which led to embankment instability. Static liquefaction is a phenomenon in which saturated, loose sand tends to lose strength and collapse rapidly under sustained (static) shear loading, generating high pore water pressure in the soil mass and very low strength. The stress-strain behavior is brittle, and the low residual strength is much lower than the static shear stresses, creating a large force imbalance, acceleration and velocity, and flow of the soil mass.

Static liquefaction has been receiving increasing attention in recent years in the tailings dam arena because of a number of tailings dam failures, including the 2019 Feijão Dam I tailings dam failure near Brumadinho, Brazil, which resulted in more than 250 fatalities (Robertson et. al. 2019). Although earthquake-triggered liquefaction has been a concern of water storage dam engineers since the failure of the Lower San Fernando Dam in 1973, the potential for liquefaction has not been widely considered for non-seismic loading—loads that are applied slowly enough that it has been assumed that drainage of the soil would occur. Because earthquake loads are applied very quickly, it is easy to understand why sand might behave in an undrained manner under this loading condition; the loading occurs more quickly than water can flow in or out of the voids in the soil, even for relatively high-permeability sands. However, it has generally been assumed by geotechnical engineers that under most other loading conditions, water will be able to flow in and out of sands readily and they will behave drained during shear, with the strength defined by the drained shear strength, regardless of the density of the sand. Further, reported static liquefaction failures of water storage dams have been rare, though not unprecedented.

Research, principally by tailings dam practitioners, has shown that collapse behavior in saturated, loose sands can be triggered at stress states below those at the drained friction failure envelope, which is normally taken to indicate the available strength of the soil if the loading is not seismic. Figure 4 shows a stress path for an undrained triaxial shear test of the saturated, loose sand in Figure 4. The stress path is a plot of the relationship between the applied shear stress and confining stress (portrayed in the figure as mean effective stress) as the soil is sheared. The stress path starts at the point labeled “Static shear stress” in Figure 4, which corresponds to the point of zero shear strain in Figure 4, indicating the initial undeformed state of the soil. As the soil is strained (deformed), the stress path moves to the point labeled “Peak strength (collapse point)” in Figure 4, which corresponds to the point labeled “Peak shear strength” in Figure 4. Finally, the stress path continues to the point labeled “Steady-state or residual strength” in Figure 4, which corresponds to the point labeled the same in Figure 4. Stress path movement toward the left indicates development of increasing pore water pressure in the soil. From Figure 4, it is seen that some increase in pore water pressure develops as the shear stress increases to the peak strength, but this is followed by a large increase in pore water pressure, reducing the strength to the steady-state or residual strength.

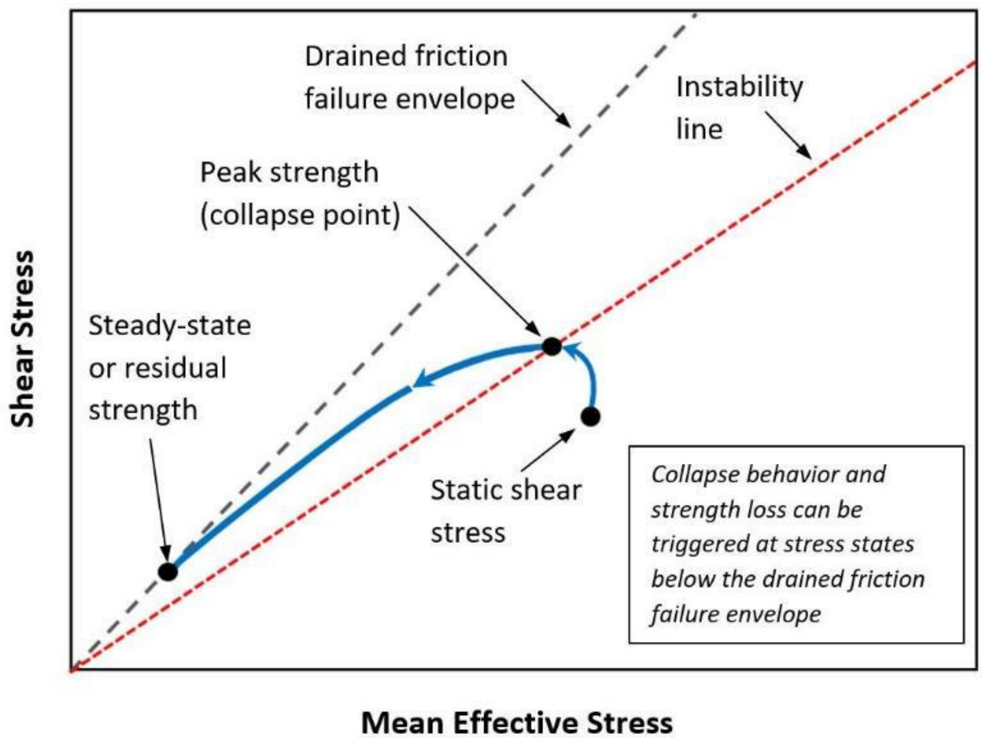


Figure 4-2: Initiation of Collapse Behavior at Stress State below the Drained Friction Failure Envelope

If the test illustrated in Figure 4 is conducted using “load control,” that is with the load producing the shear stress applied with dead weights, a collapse behavior is observed. As weights are slowly added, there is a small amount of strain and some increase in pore water pressure as the shear stress increases to peak strength. After the peak strength is reached, the pore water pressure and the strain both increase rapidly and the sample effectively collapses because the increase in pore water pressure dramatically decreases the strength. The same behavior occurs in the field in a flow slide failure of saturated, loose sand. Because of this observed behavior, the line of undrained peak strengths shown on Figure 4 has been called the collapse line or instability line. Because the collapse or instability line is significantly below the drained friction failure envelope, as shown in Figure 4, the shear stress required to reach the collapse or instability line is less than that at the drained friction failure envelope. Hence, in a stability analysis, the factor of safety against reaching the instability line would be less than that against reaching the drained friction failure envelope.

Research has also shown that the instability line, where collapse is triggered, can be reached by (1) an increase in pore water pressure (decrease in effective stress), (2) an increase in shear stress, or (3) a combination of the two, as shown in Figure 4. Path 1, the orange path in Figure 4, shows that an increase in pore water pressure with no change in the applied static shear stress can move the stress path to the left until it intersects the collapse or instability line, at which point pore water pressure increases rapidly, strength decreases dramatically, and collapse/instability results. This behavior is significant, because it indicates that even a gradual rise in pore water pressures in an embankment that includes saturated, loose sands can trigger the collapse behavior, without any significant changes in shear stress and at a stress state below the drained friction failure envelope.

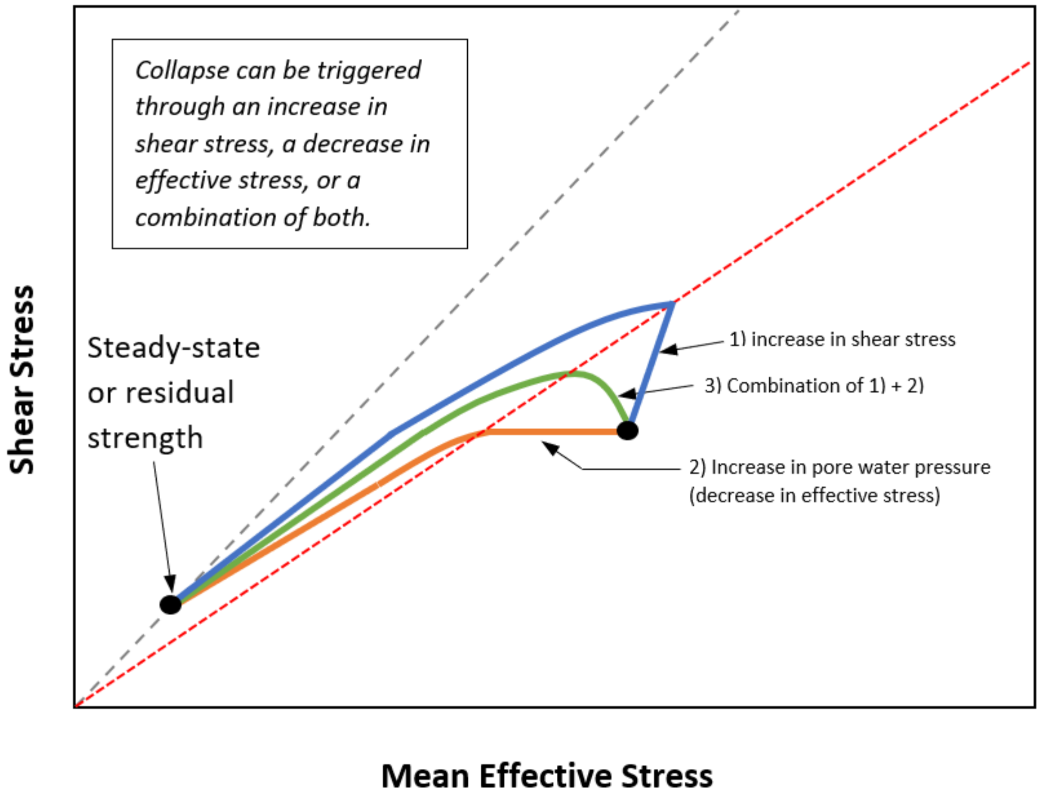


Figure 4-3: Conceptual Stress Paths to Reach the Instability Line

Evidence for Static Liquefaction at Edenville Dam

There is substantial evidence that the Edenville left embankment may have contained saturated, low-density (loose) sands or silty sands that were potentially subject to static liquefaction. Numerous borings in the Edenville Dam embankments, including the Edenville left embankment, encountered uniform fine sands with low blow counts, indicating they were loose. Loose, uniform fine sands were found in the lower part of the downstream section of the Edenville left embankment remnant at the left side of the breach. Although information concerning the original construction methods for the embankments is contradictory, it is reasonable to conclude that some of the embankment materials were dumped with little or no compaction.

Laboratory tests on specimens from a bulk uniform fine sand sample collected from the left breach face show dramatic brittle, strain-weakening collapse behavior when the sand is loose. Figure 4 shows the results of undrained triaxial strength tests on three specimens of the sand compacted to 30 percent relative density, to represent a loose state, and consolidated to different confining pressures. The left side of the figure shows stress-strain curves for the three specimens. All three curves show brittle stress-strain behavior like that shown in Figure 4. The right side of Figure 4 shows stress paths for the three specimens. All three specimens show peak undrained shear strengths at stress states well below the drained frictional envelope of 31 degrees, followed by large increases in pore water pressure and dramatic decreases in strength, similar to the behavior illustrated in Figure 4. Consequently, these test results show that a sand sample taken from the left embankment, when it is loose, exhibits the stress-strain characteristics necessary for static liquefaction.

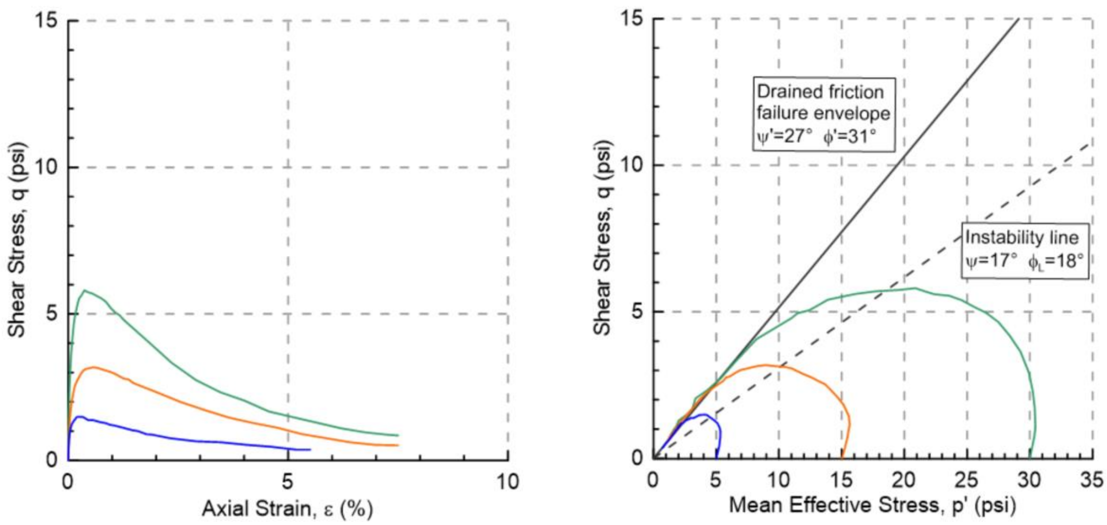


Figure 4-4: Laboratory Undrained Triaxial Test Results for Edenville Dam Uniform Fine Sand Sample at 30 Percent Relative Density[16]

The IFT also completed a kinetic analysis of a possible static liquefaction failure scenario for the Edenville left embankment, using the simplified procedure originally proposed by Davis et al. (1988). This procedure consists of calculating accelerations, velocities, and deformations for a potential failure mass within the embankment. A travel path for the center of gravity of the mass is assumed, and the initial acceleration is based on the force imbalance created by the reduction of strength to the steady-state or residual strength. The changes in accelerations, velocities, and deformations are calculated incrementally in time steps. The results are shown in Figure 4 as plots of deformation, velocity, and acceleration versus time. An ASCE investigation team completed pixel tracing analysis on the failure video and concluded that the failure mass reached a velocity of about 5 meters per second, or 16.4 feet per second (ft/sec) (ASCE 2021), which is reasonably close to the 15.8 ft/sec resulting from the simplified kinetic analysis. In addition, the displacement time of about 7.9 seconds from the kinetic analysis is similar to the failure time interpreted from the dam failure video. Consequently, the results of the kinetic analysis support the plausibility of a static liquefaction failure.

[16] ψ′ is the angle of the failure envelope in the q-p′ plot, and ϕ′ is the corresponding effective stress friction angle.

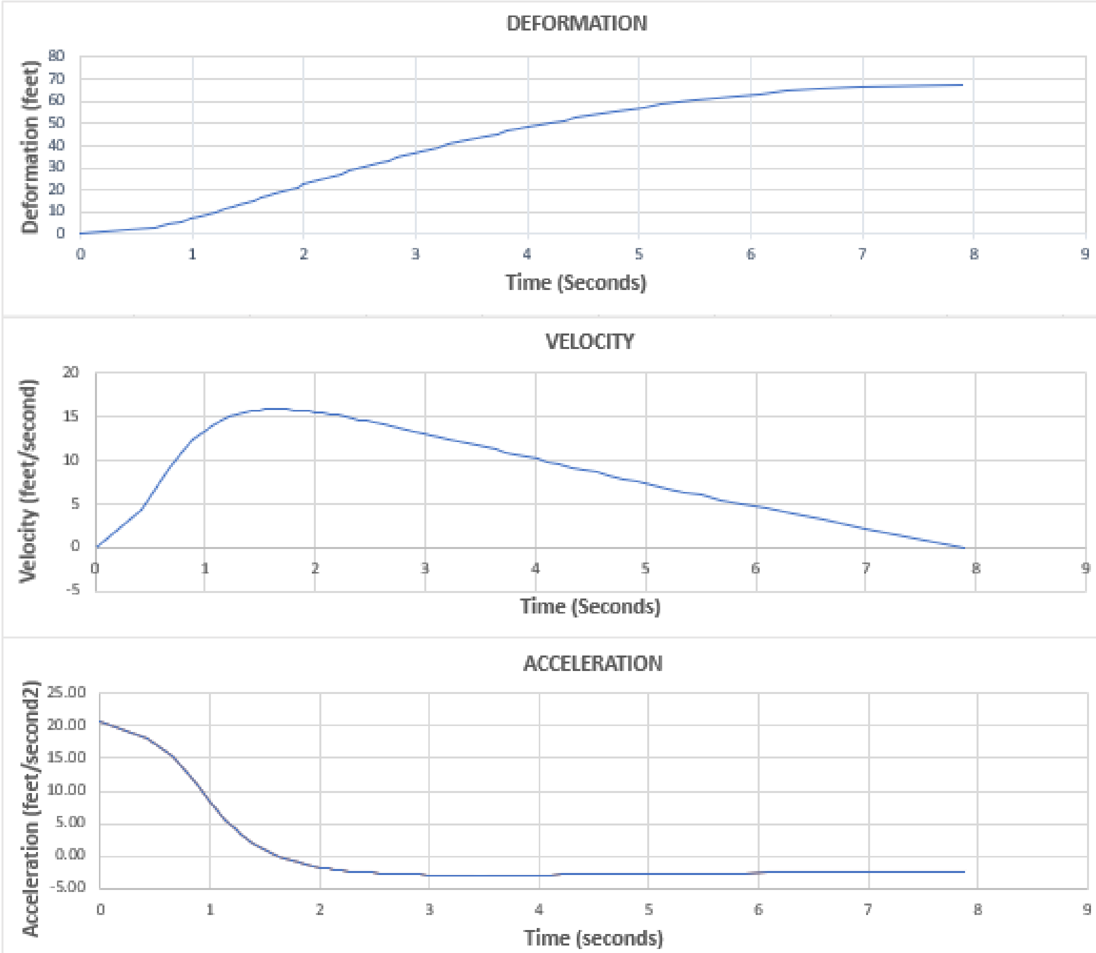


Figure 4-5: Results of Kinetic Calculations for Possible Static Liquefaction Failure of Edenville Left Embankment

Based on (1) the likelihood of loose, uniform fine sands and silty sands in the embankment, (2) the dramatic collapse behavior exhibited in laboratory tests on loose specimens of uniform sand collected from the breach remnant, and (3) the reasonably close match of a simplified kinetic analysis with the characteristics of the failure shown in the dam failure video, the IFT believes that static liquefaction failure of the downstream section of the Edenville left embankment is the most plausible explanation for the physics of the failure.

Trigger for Static Liquefaction

It is less clear exactly what triggered the static liquefaction, but there are several plausible triggers. As noted above, static liquefaction can be triggered by increases in pore water pressures, increases in shear stresses, or a combination of the two.

The relatively steep downstream embankment slope at the failure location (1.7H:1V to 2H:1V) resulted in relatively high initial shear stress ratios, indicating that small increases in pore water pressure and/or shear stresses could increase the stress ratio to intersect the instability line. This is supported by a stress analysis described in Appendix F2, which indicates that the shear stress ratios of the saturated, loose sands within the lower downstream embankment section were likely near the instability line for lake levels near the normal lake level. In addition, stability analyses presented in Appendix F2 indicate that, for friction angles equivalent to the estimated instability line, the stability factor of safety under normal lake level conditions would be only slightly above 1.0. Under these conditions, small increases in pore pressures and/or shear stresses could trigger static liquefaction, and stability analyses in Appendix F2 indicate a factor of safety of much less than 1.0 for liquefied strength, which would be sufficient to cause flow instability.

As the lake level rose on Monday and Tuesday, May 18 and May 19, 2020, there were several possible sources of increased pore water pressure in sand located in the downstream section of the dam. The upper sand foundation layer provides a source for increased pore water pressure in the embankment and possibly elevation of the phreatic surface in the downstream section, particularly if the upstream section of the dam was composed of lower permeability fill. The historically high lake level could have led to historically high pore water pressures and phreatic surface levels from this source. The rise in the pore pressures, which may elevate the phreatic surface, could also increase the volume of saturated sand susceptible to collapse. A rise in the phreatic level, saturating previously unsaturated soils, could also have the effect of reducing negative pore pressures (suction) and therefore shear strength, within the unsaturated zone near the phreatic surface.

The lake level at the time of the failure is believed to have been about 3 feet higher than the previous pool of record. This represents an external hydraulic load on the embankment about 10 percent higher than had been previously experienced, which could have produced historically high shear stresses in the embankment. The higher lake level also exposed a 3-foot steep high section of the upstream embankment face to the reservoir for the first time – a first filling for that portion of the embankment. Within this 3-foot horizon, there could have been pervious soils that contributed to elevated phreatic levels in the embankment. There is also some evidence that cracks developed in the embankment near the crest in conjunction with the crest subsidence that occurred about 35 minutes before failure. Flow through these cracks, if they developed, could also have contributed to elevated phreatic levels and pore water pressures.

Other potential triggers, listed below, were considered, but ultimately judged to have had no or very limited influence on triggering for the reasons noted:

- Wave action causing increases in shear stress due to dynamic impact forces was judged to not be a significant factor because the wave forces and resulting dynamic stresses in the embankment would have been relatively small and wave action had diminished by the time of the failure.
- Saturation due to rainfall was judged to not be a significant factor because infiltration is believed to have been limited in depth because of the steep, grass-covered embankment slopes and the limited duration and total depth of the rainfall.
- Elevated pore pressure due to frozen ground blocking natural drainage of the embankment was judged to not be a significant factor because the presence of frozen ground on May 19, 2020, was judged to be unlikely for the south-facing embankment slope.
- Differential settlement leading to low stress zones was judged to not be a significant factor because there are no known embankment or foundation features that would lead to significant differential settlement.

Consideration of Conventional Instability

The IFT considered the possibility that the instability was primarily caused by a rise in pore pressure, decreasing effective stresses and drained strengths in the sand and lowering the factor of safety below 1.0.

However, a failure of this type is not consistent with the kinetics of the observed behavior. Drained triaxial strength tests were completed on specimens from the bulk sample used for the undrained strength tests presented in Figure 4. These specimens were also prepared at 30 percent relative density and consolidated to three different confining stresses. The results are shown in Figure 4. The stress-strain curves, on the left side of Figure 4, do not exhibit brittle, strain-weakening behavior, but rather show ductile behavior, with little or no decrease in strength from the peak strength. Without brittle, strain-weakening behavior, it is not possible to create the force imbalance needed to generate the observed accelerations and velocities during the failure. No evidence was found indicating that any other materials in this embankment would have brittle, strain-weakening stress-strain behavior.

Therefore, if the soils behaved in a drained manner, with ductile stress-strain behavior, it would be expected that the instability failure would be progressive—a rise in pore water pressure dropping the factor of safety below 1.0 and causing enough deformation to restore stability, followed by further rise in pore water pressure causing further slumping, with this process continuing and ultimately leading to enough deformation to cause failure through overtopping or internal erosion. The observed characteristics of the failure are not consistent with this mechanism, and the IFT judged this mechanism to be implausible as the primary cause of the failure.

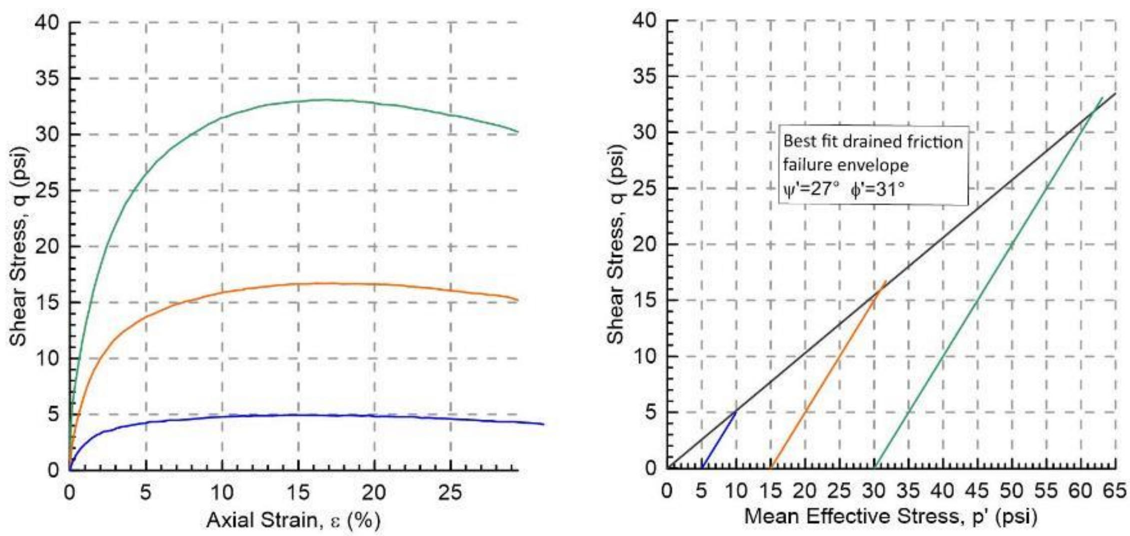


Figure 4-6: Laboratory Drained Triaxial Test Results for Edenville Dam Uniform Fine Sand Sample at 30 Percent Relative Density

4.1.2 Cause of Crest Subsidence

The specific reasons for the crest subsidence are not entirely clear, although several plausible possibilities exist.

In the IFT's opinion, the most plausible cause of the subsidence is an initial liquefaction of a limited amount of soil in the embankment. The extent of the liquefied area would not have been sufficient to cause the full flow failure, but enough to cause an initial slump. One piece of evidence that argues against this interpretation is the lack of obvious bulging at the toe of the downstream slope associated with the crest subsidence. However, for the limited crest subsidence, the bulge could have been small, and the IFT did not have any photographs of the downstream slope before and after the occurrence of the subsidence, only photographs after.

The subsidence could have also been caused by the wetting-induced collapse of previously unwetted loose sand within the upper part of the embankment, or collapse of voids in the upper part of the embankment from animal burrows or rotted wooden members (utility poles or railroad ties from construction). It is also possible that there were one or more voids primarily within the lower part of the embankment from a prior history of internal erosion into the underdrains, and these voids may have collapsed. The potential for internal erosion into the drains is considered credible, given the typical nature of the gravel-covered, open-joint clay tile drains used in that era of construction. The gravel would have sufficient void size and the clay tile pipes would have sufficiently large openings to allow migration of soil particles. The relatively uniform nature of the downstream sand fill indicates that it may be erodible under relatively low seepage gradients. The drain inspections in 2012 indicated some sediment in the drain pipes, and there had been historic reports of some limited sediment from the drains.

The main evidence against the alternative explanations for the subsidence presented in the previous paragraph is that the subsidence occurred only at the ultimate failure location and was fairly uniform in magnitude across the entire failure area. The potential defects noted in the previous paragraph would have been present in other locations in the Edenville Dam embankments. With about 6,000 feet of embankment length, it does not appear plausible that subsidence would occur at only one location if the cause was one or more of the conditions noted in the previous paragraph. It appears more plausible that the subsidence was associated in some way with the ultimate instability failure mechanism.

It is also noted that the subsidence could have contributed to the development of the ultimate flow instability failure. The photographic evidence and eyewitness accounts suggest that cracks developed in the embankment after the subsidence. Cracks would have allowed water into the embankment, would have reduced the available total shear resistance along potential failure surfaces, and may also have resulted in some shock stresses within the embankment, all of which may have contributed to more widespread propagation of static liquefaction.

4.1.3 Reasons for the Location of the Failure

One question that arose during the IFT's investigation is: Why was the failure limited to a 40- to 80-foot length of the embankment? Possible contributing factors include:

- Clay tile underdrains beneath the embankment were missing for much of the failure section, as described further below.
- The failure section may have included the loosest sands and silty sands within the embankment.
- This section of the dam had a steep downstream slope and high static shear stresses.

A 2012 investigation of the drains did not identify drains beneath the center of the failure location, as shown in Figure 4, suggesting that the drains at that location either were never installed or subsequently had become buried and ineffective. Since the remainder of the drains along the Edenville left embankment have been observed to consistently flow, they appear to provide some degree of drainage for the embankment. The lack of drains in the center of the failure location could have resulted in higher pore pressures at that location than elsewhere along the embankment.

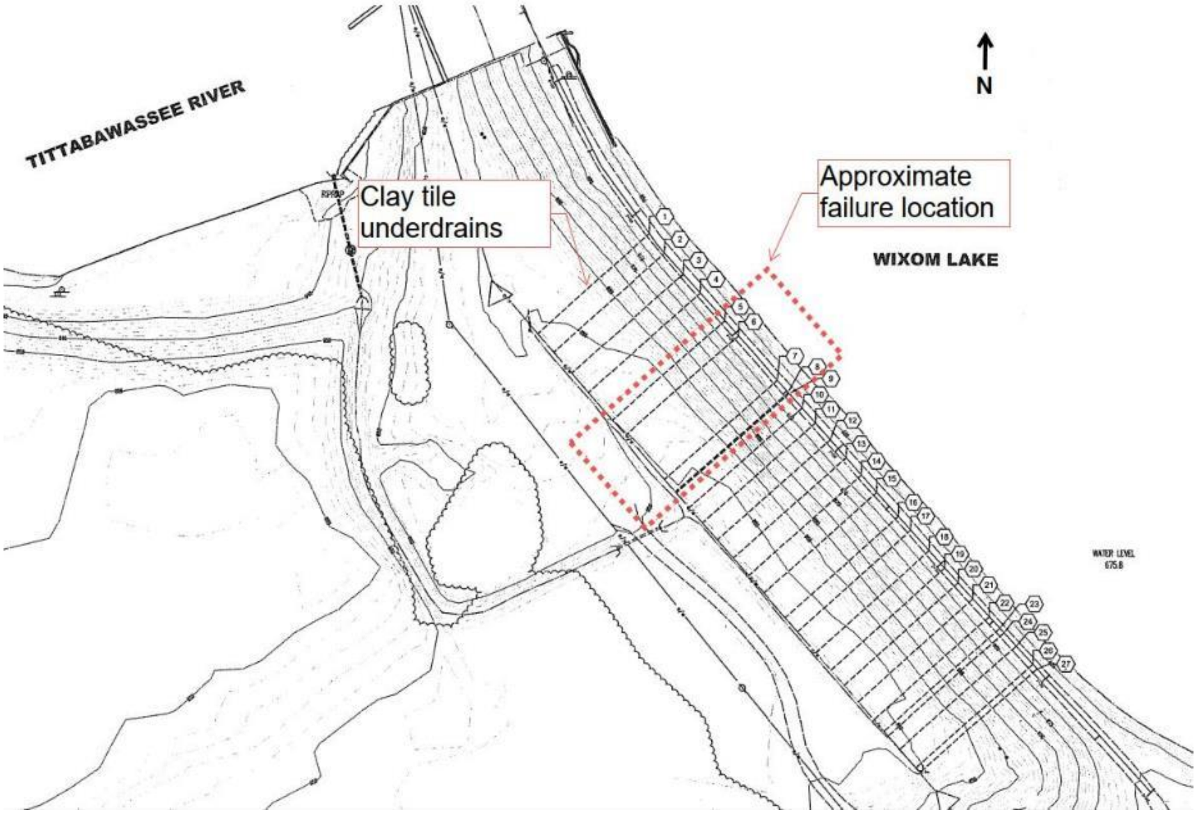


Figure 4-7: Plan from 2012 Underdrain Survey Showing Lack of Underdrains in the Failure Location

4.2 Sanford Dam Failure

The physics of the Sanford Dam failure are very clear. The failure was the result of embankment overtopping. The breach outflows from Wixom Lake after the failure of Edenville Dam caused the water in Sanford Lake to rise more quickly than could be accommodated by the spillways at Sanford Dam. Based on eyewitness accounts, videos, and photos, about 105 minutes after the initial failure of Edenville Dam, the water in Sanford Lake rose to the crest of the fuse plug spillway. As the lake continued to rise, the fuse plug eroded, but the additional discharge capacity provided by the eroding fuse plug spillway was not sufficient to prevent the lake level from exceeding the embankment dam crest elevation, which occurred at a little more than 2 hours after the failure of Edenville Dam. As water flowed over the Sanford Dam embankment crest and down the downstream slope, the embankment eroded, creating a breach of the embankment. Section 3.3 and Appendix A provide more detailed discussion of the fuse plug performance at Sanford Dam.

The IFT did not perform a detailed hydrologic and hydraulic analysis to determine whether Sanford Dam would have overtopped and failed if Edenville Dam had *not* failed. However, based on comparison of the inflows into Wixom Lake, the projected outflows from Edenville Dam if it had not failed, and the combined spillway capacity at Sanford Dam, it appears likely that Sanford Dam would not have failed if Edenville Dam had not failed.

5. Evaluation of the Flood Event

5.1 Historic Operations and High Lake Levels

A FERC license was issued for the Edenville project in 1998, which among other things established the operating rules for Wixom Lake. The FERC license requirements for the Edenville project established a normal operating lake level of El. 675.8. The FERC license allowed for limited peaking power operations but restricted fluctuations in the lake level to 0.4 foot below to 0.3 foot above the normal lake elevation under normal operating conditions. The license allowed for a winter drawdown in which the lake could be lowered to a minimum elevation of 672.8 feet (3 feet below normal lake level) and operated so that the daily fluctuation in lake level did not exceed 0.7 foot. Winter drawdown was not to begin until December 15, and the normal lake level was to be restored prior to the lake surface water temperature reaching 39˚F. During floods, the spillway gates would be operated to limit the rise in the lake level and return the lake level to the specified normal range as soon as possible (FERC 1998a).

Prior to the FERC regulation, the facility could be operated in a less restrictive peak power mode, which could result in the lake level fluctuating over a wider range to store and release water.

Following the revocation of the FERC license for the Edenville project in September 2018, FERC lake level requirements no longer governed the lake operations. In May 2019, Gladwin and Midland Counties, through a court order (State of Michigan 2019), established lake level requirements under Part 307 of the Michigan code. The lake level requirements set for Wixom Lake in the court order were the same as the earlier FERC requirements.

Available lake level records indicate that over a period of almost a century, Wixom Lake rarely rose significantly above the FERC-established normal lake level of El. 675.8, even prior to FERC regulation, as shown in Figure 5. In fact, there were only 4 years when the lake rose 1.4 feet or more above El. 675.8 – in 1928, 1929, and 1945, and during the failure in 2020. In 2014, the lake level rose by 1.36 feet, just short of 1.4 feet. There were 22 years when the lake rose more than 1 foot and less than 1.4 feet; however, all of those years except one (2014) were before the implementation of the FERC lake level requirements. As can be seen in Figure 5 maximum yearly lake levels were generally slightly higher before 1975 than since that time, likely reflecting in part the less restrictive lake operations.

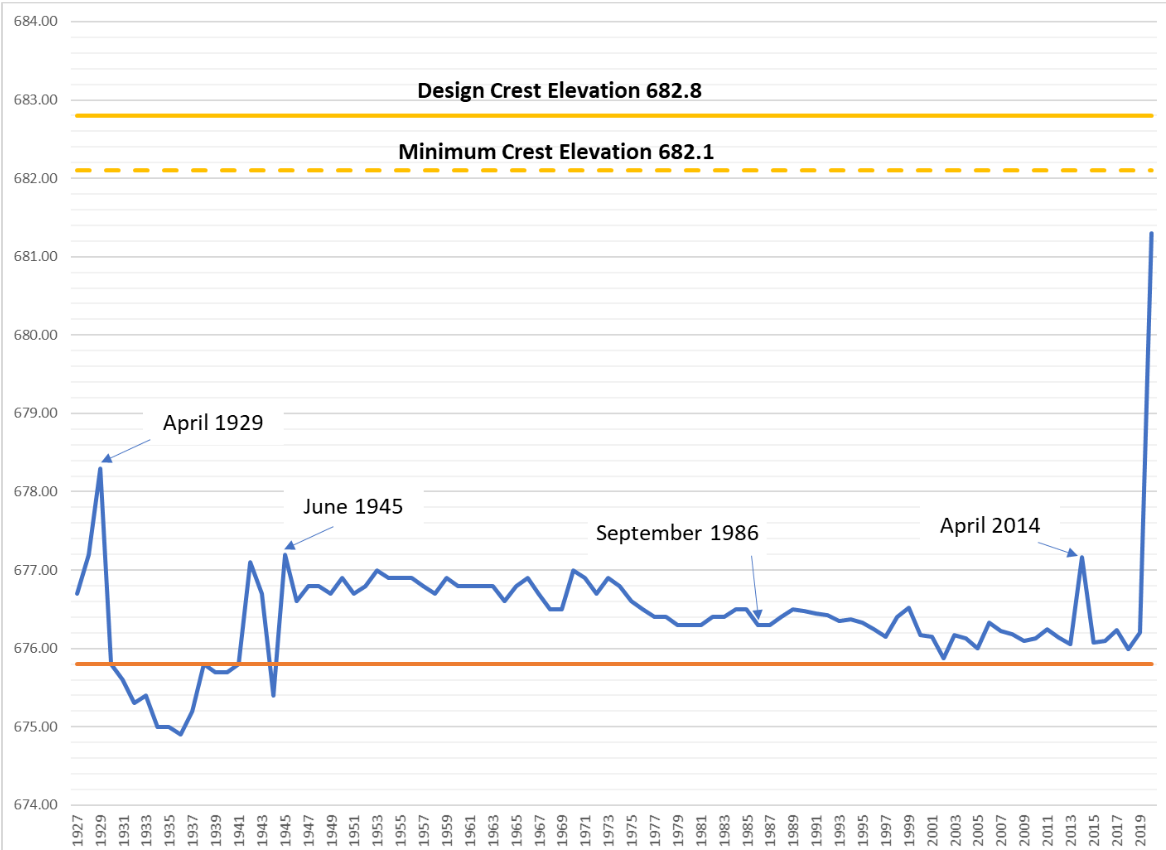


Figure 5-1: Annual Maximum Wixom Lake Levels 1927 through 2020

The IFT is not certain that the available information on the Wixom Lake levels is entirely complete, and some discrepancies were found in the available data. When discrepancies were found, the information from the original handwritten logs was used. According to our interpretation of the available information, the pool of record, prior to May 2020, occurred in April 1929 at 2.5 feet above the normal lake level (El. 678.3). A more detailed discussion of historic and recent lake operations is provided in Appendix F1.

5.2 Evaluation of May 2020 Flood Event

The rainfall event that occurred on May 17 through 19, 2020, was significant but not extreme. As discussed further in Appendix F1, for 11 subbasins upstream of Sanford Dam, the total 42-hour rainfall for this May 2020 event ranged from 3.57 to 5.36 inches, with a weighted average of 4.29 inches across the entire watershed. By comparison, in a 2021 study (AWA 2021), Applied Weather Associates (AWA) estimated the probable maximum precipitation (PMP) for the Edenville Dam basin to be 17.2 inches over 72 hours, with about 85 percent of the precipitation (14.4 inches) occurring during the middle 24 hours of the event. Therefore, the May 2020 rainfall was only a fraction of the PMP rainfall.

It is reasonable then to ask why the May 17 through 19 storm resulted in a record-high Wixom Lake level, 3 feet higher than the previous high lake level that occurred in 1929. To address this question, the IFT completed two tasks:

- An evaluation of seven selected historical storms, including the May 2020 storm

- A comparison of the May 2020 storm and the September 1986 storm, which is locally viewed as having resulted in a "great flood" in the area

Appendix F1 provides detailed discussions of these evaluations, and a summary of the results is presented here.

5.2.1 General Comparison of Past Storm Events

The maximum lake levels and the rainfall/duration data for the seven selected storms are listed in Table 5. The seven selected storms include the five highest Wixom Lake levels in the available record plus two other relatively high rainfall events that did not result in high Wixom Lake levels.

Table 5-1: Seven Selected Rainfall Events with Maximum Lake Levels, Including the May 2020 Event

Maximum Lake Level		Depth above Normal Lake Level	Rainfall/Duration
Date	Elevation [(1)]		
June 26, 1928	677.2	+ 1.4 feet	2.92 inches/2 days [2]
April 6, 1929	678.3	+ 2.5 feet	4.46 inches/3 days
June 2, 1945	677.2	+ 1.4 feet	3.59 inches/2 days
September 12, 1986	676.3	+ 0.5 feet	6.83 inches/3 days
April 13, 2014	677.2	+ 1.4 feet	4.53 inches/2 days
June 23, 2017	676.2	+ 0.4 feet	5.04 inches/4 days
May 19, 2020	681.3	+ 5.5 feet	4.30 inches/2 days

(1) Maximum lake levels based on original operations logs.
(2) Precipitation data in the table are from the NOAA Gladwin Station, except for the June 1928 event. There was no precipitation data recorded at the NOAA Gladwin Station in 1928 – rainfall data for that storm are from the West Branch MI NOAA Weather Station.

The data presented in Table 5 show that for precipitation values in this range, the Wixom Lake level does not correlate directly with precipitation. The two highest total rainfall events in the table, the September 1986 and June 2017 events, resulted in lake levels no more than 0.5 foot above normal, while the May 2020 event, the event with the third lowest total rainfall, resulted in the highest lake level.

The five highest historical lake levels occurred in the months of April, May, and June, with the two highest lake levels occurring in April and May during the latter part of the cold season, when there is potential for rain to occur on ground that is partially saturated and/or frozen, and therefore has potential for a reduced infiltration rate and reduced infiltration capacity.

The Tittabawassee River flood of April 5 to 15, 1929 is an example of runoff from rapid melting of ice and snow aggravated by ground that is frozen and impervious (State of Michigan House of Representatives 1932).

A more recent storm event that resulted in the Wraco Lodge Lake dam failure on April 15, 2014 was also the result of the rain event occurring on partially frozen ground. Approximately 4 inches of rainfall occurred in 2 days, which would be around a 10- to 25-year return period[17] (NOAA 2022b) and resulted

[17] Return period is the theoretical period between occurrences and the inverse of the Annual Exceedance Probability (AEP). AEP is defined as the probability that a given rainfall total accumulated over a given duration will be exceeded in any one year; i.e., a 100-year return period would have an AEP of 0.01.

in an estimated at least 200-year flood on the Muskegon River near Evart, Michigan, at the USGS stream gage at Evart, MI (USGS 2022).

A detailed review of the conditions when these seven events occurred, as discussed in Appendix F1, suggests that both storm characteristics (spatial and temporal) and watershed characteristics substantially affected the amount of the precipitation that was converted to runoff into Wixom Lake, with the result that total rainfall, runoff, and the Wixom Lake level are not directly correlated.

5.2.2 Model for May 2020 Event

The IFT developed a hydrologic and hydraulic model to simulate the May 2020 rainfall and flood event. The model is discussed in more detail in Appendix F1. The total drainage area contributing to the Sanford watershed is approximately 978 square miles. The Sanford watershed consists of the area that contributes to all four lakes (Secord, Smallwood, Wixom Lake, and Sanford) with a basin outlet at Sanford Dam. A map of the watershed is shown in Figure 5.

The IFT contracted with AWA to characterize the magnitude, temporal details, and spatial details of the May 17 through 19, 2020 storm. The rainfall data provided was input into the hydrologic model in the form of temporal rainfall distributions.

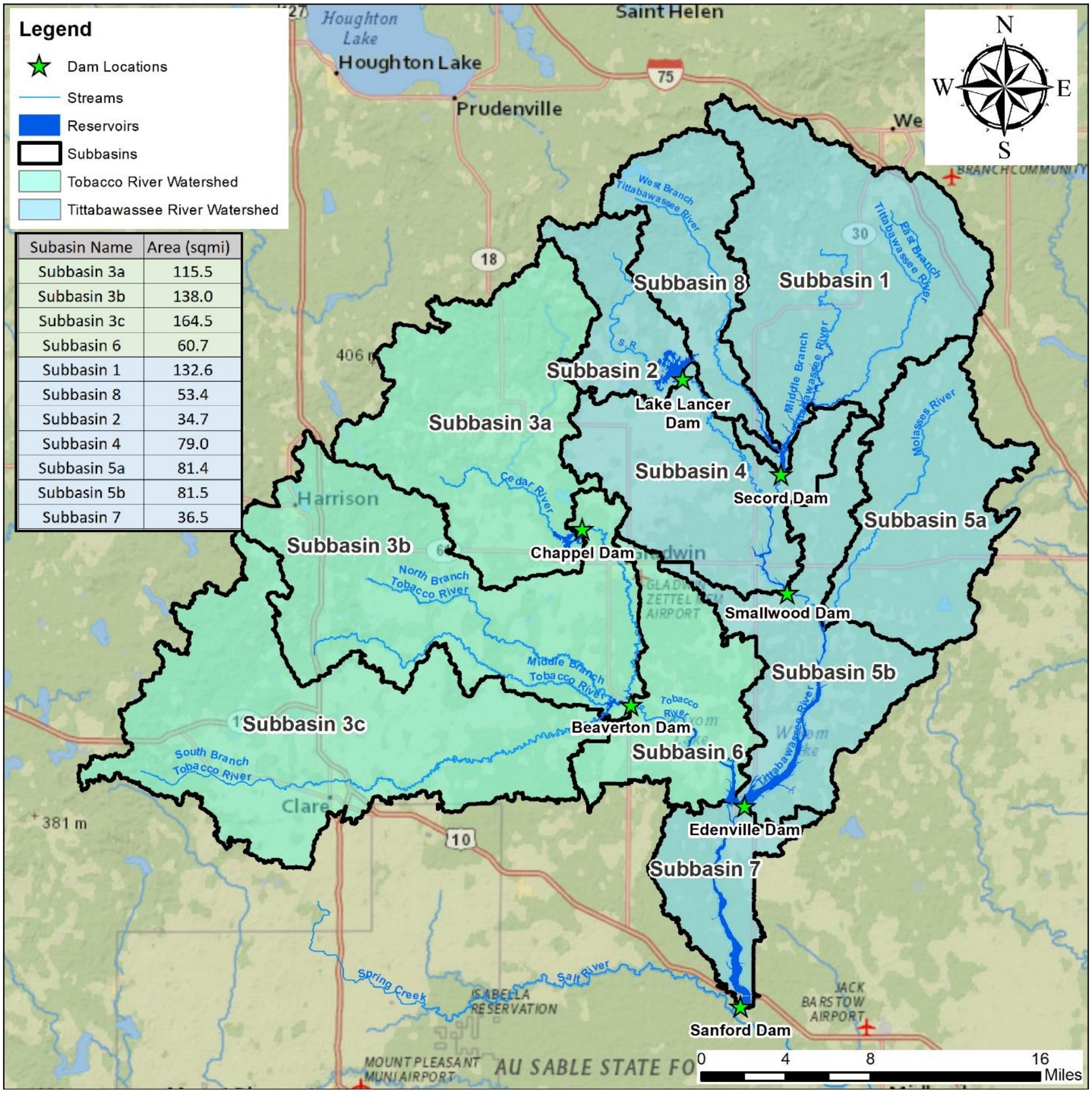
The heaviest rainfall during the May 17 through 19, 2020 event, more than 8 inches, actually occurred *outside* of the Sanford Dam watershed (the entire drainage basin upstream of Sanford Dam), 10 to 50 miles east of the basin. The distribution of rainfall within the watershed is shown in Figure 5. The heaviest rainfall occurred in the northeastern section of the watershed, upstream of Secord Dam and Smallwood Dam. The plot of cumulative rainfall versus time in Figure 5 shows that, although the total rainfall varied among the 11 subbasins included in the model, all of the subbasins experienced a period of about 18 hours of high-intensity rainfall at about 0.22 inch/hour during the middle of the storm.

The hydrologic parameters of the model were initially developed using detailed characteristics of the watershed and were later adjusted in model calibration. The model was calibrated against the measured and estimated lake levels during the May 2020 event. Documented spillway operations and gate openings of the May 2020 event were used in the model to simulate spillway outflows. The hydrologic parameters were adjusted iteratively to simulate the basin runoff that resulted in model lake levels that approximately matched the documented lake level recordings. A plot of the recorded lake level data of Wixom Lake and the modeled lake level data are shown in Figure 5.

Based on the calibrated model, the peak inflow into Wixom Lake during the May 2020 event was estimated to be about 24,500 cfs. The hydrologic and hydraulic model developed is believed to generally approximate the May 2020 runoff, lake levels, and outflows based on the provided rainfall data, spillway gate operations, and lake level data. However, it should be noted that the accuracy of the model is limited by the modeling assumptions incorporated in the computer software used for the modeling, the resolution and accuracy of the available rainfall data, the resolution and accuracy of the land characteristics data available for the watershed, and the data available for calibrating the model.

The return period for the rainfall that occurred during the May 2020 event was estimated to be approximately 25 to 50 years for the Sanford Watershed and the 0.22 inch/hour intensity for 18 hours is also estimated to have a 25- to 50-year return period based on NOAA Atlas 14 (NOAA 2022b). Areas in the northeastern part of the watershed experienced greater rainfall totals with a return period approaching a 100-year event for some areas. By comparison, based on flood frequency analyses recently completed by Ayres (2021) using the USACE, Hydrologic Engineering Center’s (HEC) Statistical Software

Package, HEC-SSP (USACE 2022), an inflow of 24,500 cfs to Wixom Lake corresponds to an estimated return period on the order of 100 to 200 years. These results clearly illustrate that the return period for a rainfall event and the return period for the resulting flood can be different for rainfall events in the range of the seven selected storms, with the amount of runoff being highly dependent on the amount of rainfall that infiltrates the ground.



Subasin Name	Area (sqmi)
Subbasin 3a	115.5
Subbasin 3b	138.0
Subbasin 3c	164.5
Subbasin 6	60.7
Subbasin 1	132.6
Subbasin 8	53.4
Subbasin 2	34.7
Subbasin 4	79.0
Subbasin 5a	81.4
Subbasin 5b	81.5
Subbasin 7	36.5

Figure 5-2: Sanford Dam Watershed

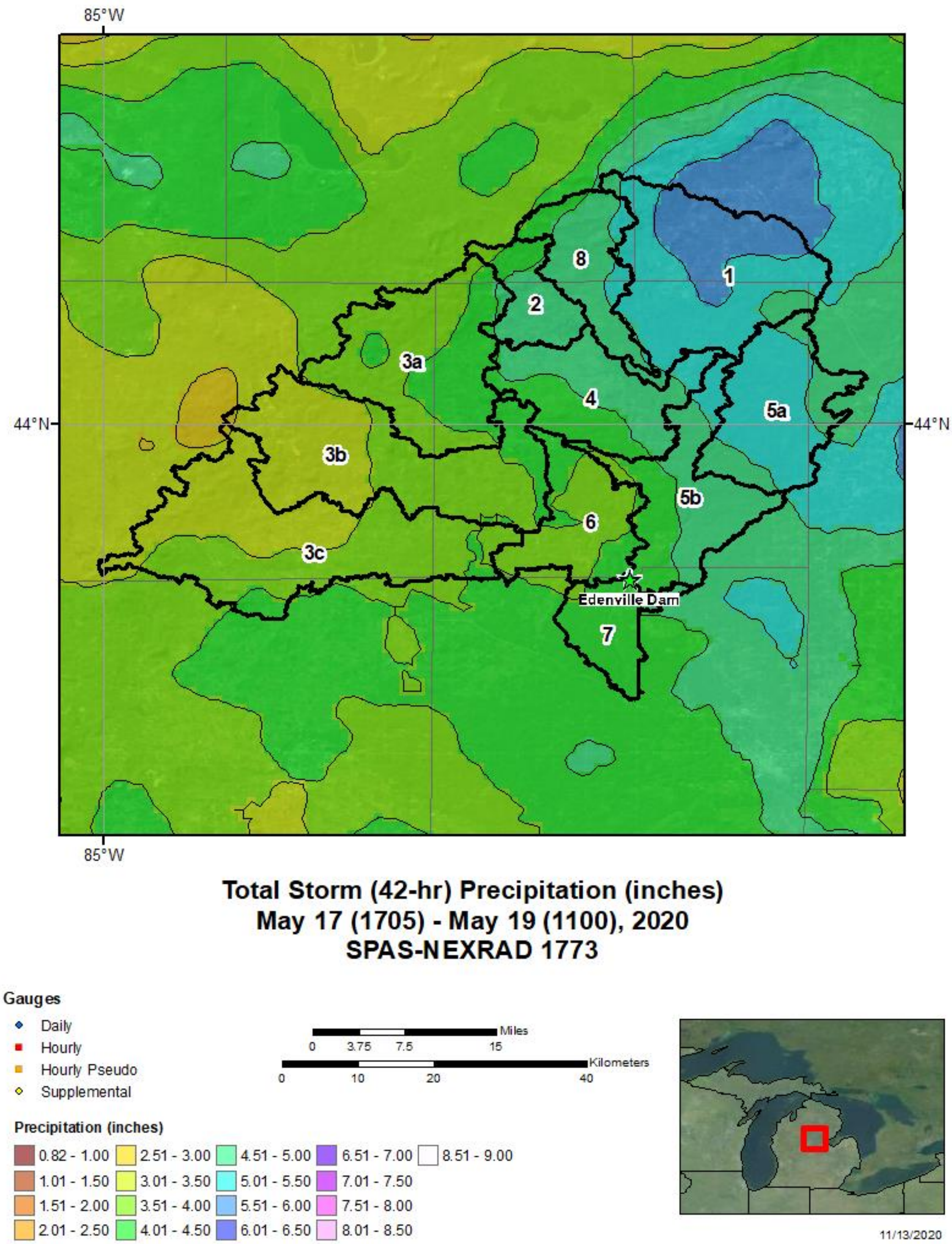


Figure 5-3: Total Storm (42-Hour) Precipitation for May 17 through 19, 2020, for the Sanford Watershed

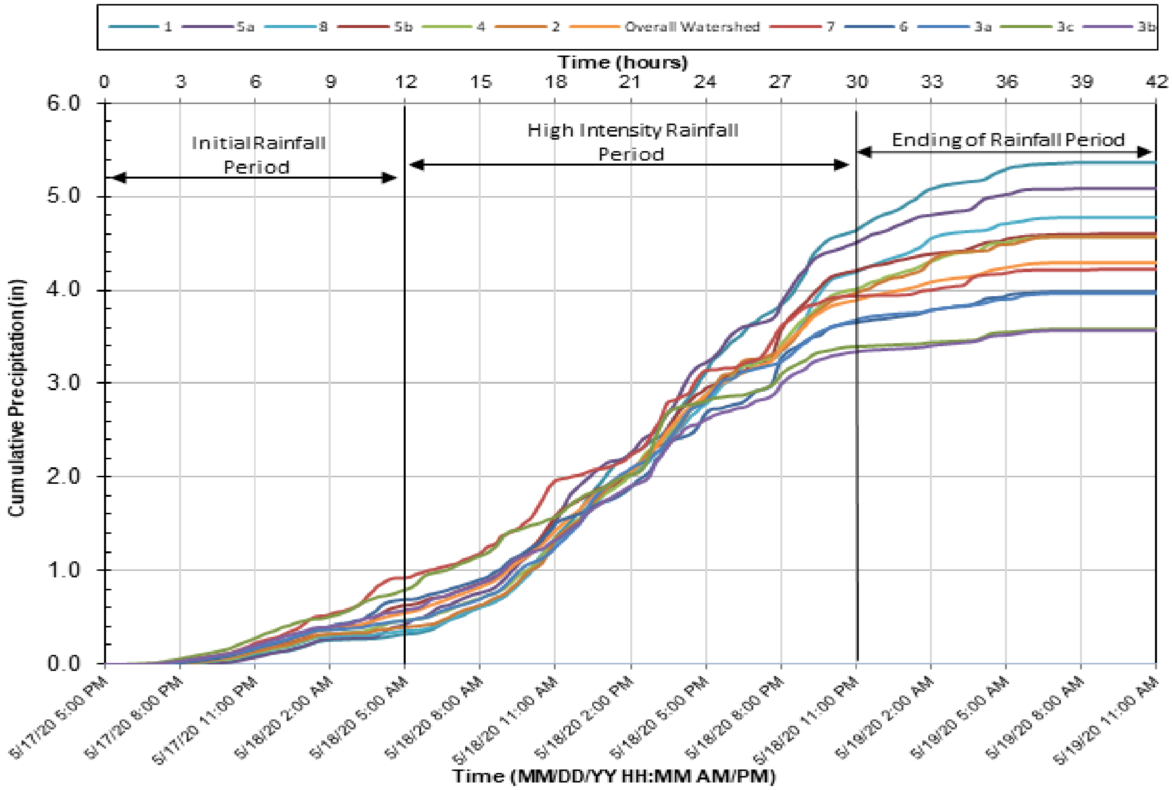


Figure 5-4: 42-Hour May 2020 Storm Cumulative Subbasin Precipitation

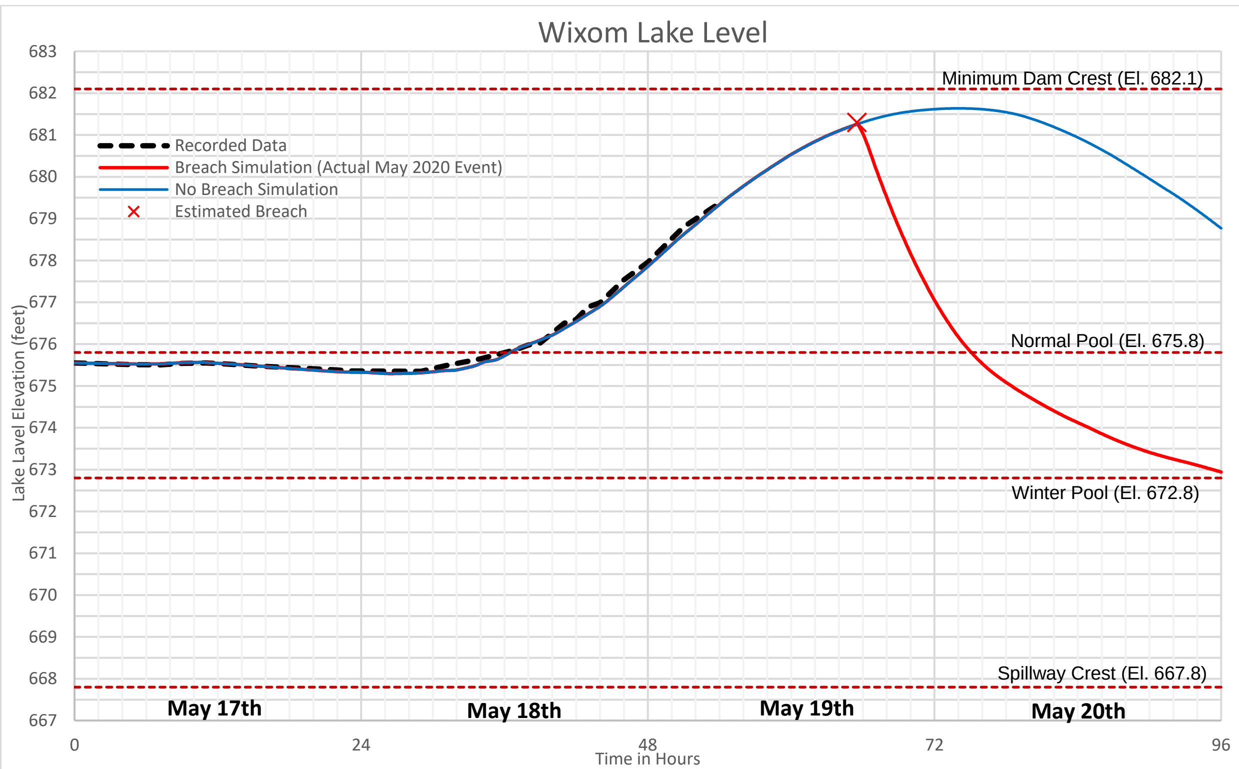


Figure 5-5: Wixom Recorded and Simulated Lake Level Data

5.2.3 Comparison of May 2020 and September 1986 Events

To further investigate the relationships between rainfall, runoff, and Wixom Lake levels, a more detailed comparison was made of the May 2020 event (about 4.3 inches of rain over 2 days and a maximum lake level 5.5 feet above normal) and the September 1986 event (about 6.8 inches of rainfall over 3 days and a maximum lake level 0.5 foot above normal).

As with the May 2020 storm, the IFT contracted with AWA to characterize the magnitude, temporal distribution, and spatial distribution of the September 9 through 13, 1986 storm. The spatial distribution of total rainfall is shown in Figure 5, and cumulative rainfall versus time plot is shown in Figure 5 for the September 1986 storm.

These figures illustrate some differences when the September 1986 storm is compared to the May 2020 storm. For the September 1986 storm the highest rainfall totals were in the central and southern portions of the basin, whereas in the May 2020 storm the highest rainfall totals were in the northern and eastern portions. The rainfall intensities in the September 1986 storm were 0.16 inch per hour or less compared to rainfall intensities of about 0.22 inch per hour in the May 2020 storm.

As discussed in Appendix F1, there were also differences in watershed characteristics between the May 2020 and September 1986 events. Prior to the May 2020 event (May 14 and 15), there was approximately an inch of rainfall adding to the antecedent moisture conditions, saturation of wetlands, and possible areas of partially frozen ground, especially in the conifer swamp areas of the watershed. None of these conditions was present during the September 1986 event.

In a simulation, the spatial and temporal rainfall distribution of the September 1986 storm was applied to the calibrated model for the May 2020 storm. This simulation provides an approximate answer to the hypothetical question of what would have happened if the September 1986 rainfall had occurred on the basin along with the characteristics that were apparently present on May 17 through 19, 2020.

From the outflow hydrograph for the 1986 storm from the Edenville plant logs, it is estimated that the total basin weighted average rainfall of about 7.1 inches resulted in only 0.83 inch of excess runoff and a peak outflow from Wixom Lake of about 9,300 cfs (excess runoff is the amount of the total rainfall that is converted into surface runoff into the streams and lakes). That excess runoff value indicates that only about 12 percent of the total rainfall was converted to runoff during the September 1986 storm.

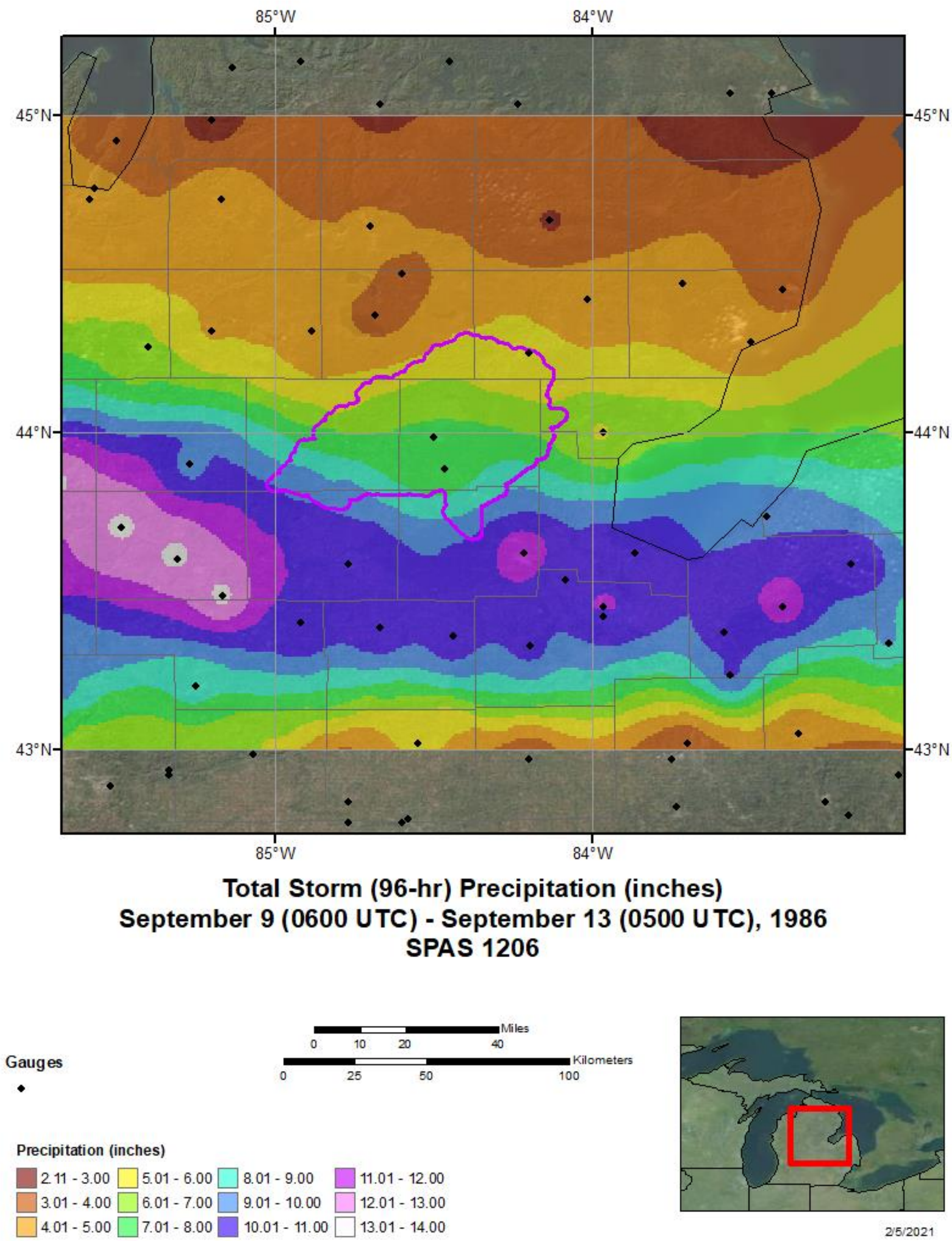


Figure 5-6: Total Storm (96-Hour) Precipitation for September 9 through 13, 1986, in Central Michigan

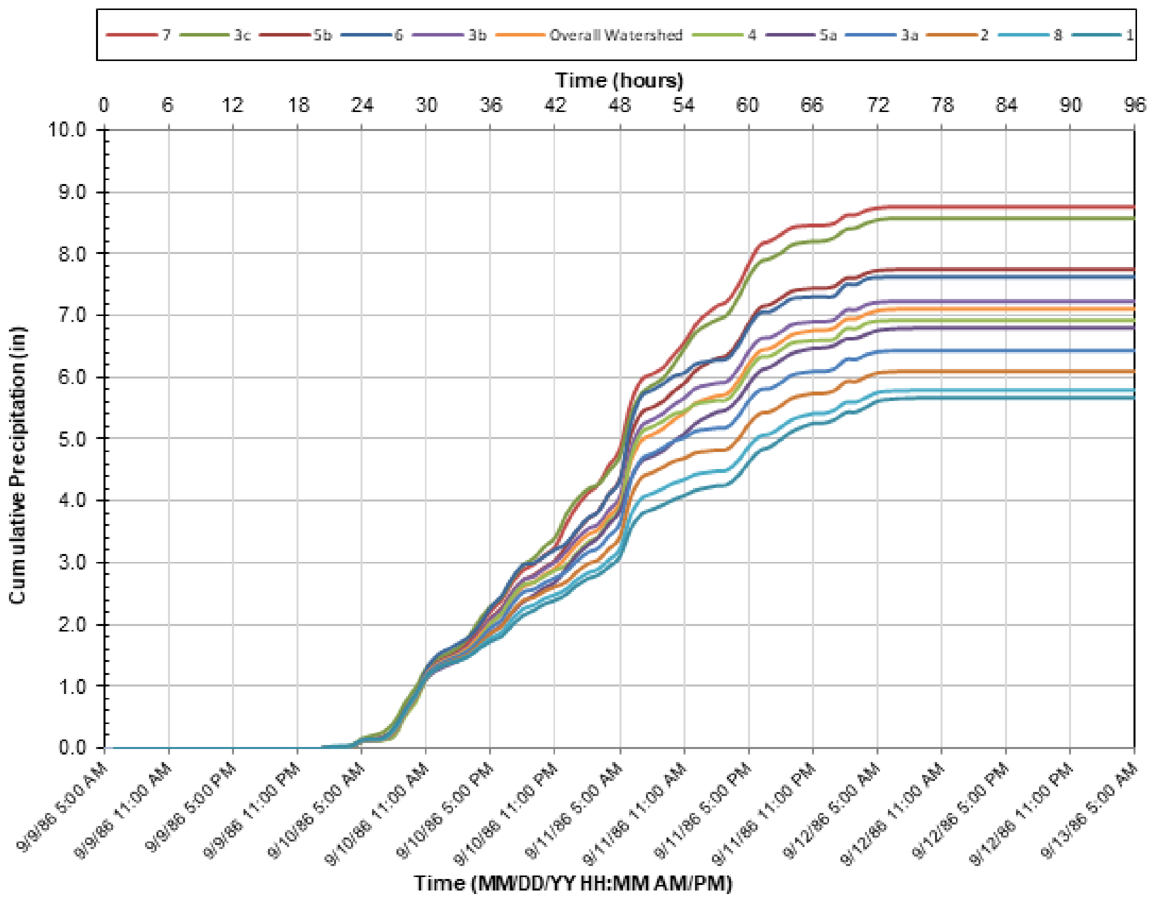


Figure 5-7: 96-Hour September 1986 Storm Cumulative Subbasin Precipitation

By comparison, the results of the model simulation indicate that if the September 1986 rainfall had occurred with the May 2020 basin conditions, the excess runoff would have been approximately 2.82 inches, more than *three times* greater than the excess runoff that actually occurred in September 1986. A simplified hydrologic model was used to compare differences in the excess runoff. The simulations resulted in a peak inflow into Wixom Lake of about 31,000 cfs during the September 1986 storm and of about 21,000 cfs during the May 2020 storm.

These results again indicate that watershed conditions at the time of a storm can dramatically impact the amount of excess runoff during the event.

5.2.4 Flood Event Conclusions

In answer to the question posed at the beginning of Section 5.2, the combination of the storm pattern (spatial and temporal rainfall distribution) and ground conditions (saturated and/or frozen ground/frost) that existed during the May 2020 event produced a relatively high excess runoff percentage (about 35 percent), a record inflow into Wixom Lake, and a record high lake level, even though the total rainfall was not a record event. Antecedent moisture conditions (e.g., elevated groundwater levels and ground saturation) in the undeveloped and wetland areas of the watershed, along with areas of partially frozen ground at or near the ground surface, likely resulted in reduced infiltration and percolation into the ground and led to a higher percentage of the rainfall being converted to runoff, which flowed into the river and lake systems.

In addition to the significant runoff that entered the reservoir, the available spillway capacity of Edenville during the May 2020 event was limited because the gates were judged to not be able to be safely fully opened, which reduced the available outflow and thus contributed to the high lake level. A hydrologic analysis showed that, with the gates opened to about 9 feet, the spillway could pass an approximately 200-year event with a lake level of 682.0 feet (0.1-foot freeboard) (Ayres 2021).

5.2.5 Hypothetical Alternative Scenarios

With the calibrated model, the IFT considered several hypothetical alternative scenarios (“counterfactuals”) related to hydrologic and hydraulic aspects of operations. Appendix F1 and Section G-3.7 of Appendix G provide more detailed discussions of counterfactual scenarios.

As discussed earlier in this report, releases through the Edenville powerhouse were considered to be not available after the FERC license was revoked, and there were limitations to the height that the spillway gates at Edenville Dam could be opened. The IFT learned that some parties suggested that the Wixom Lake level should have been pre-lowered before the storm, and the question has also been raised as to whether the Edenville embankments would have overtopped if the instability failure had not occurred.

To address these and other questions about how various factors may have changed the outcome in May 2020, and possibly have prevented the failure, numerous scenarios were considered, including a spillway upgrade option that could pass the PMF which was proposed by Mill Road Engineering (Mill Road) in 2012 but not constructed. Two different scenarios were considered for the duration of powerhouse operations, due to ambiguity in the Edenville Dam emergency action plan (EAP) with regard to what the powerhouse operations should be during a flood (see Section 6).

Wixom Lake level at the time of the instability failure is estimated to have been El. 681.3. The baseline actual case was that the Edenville Dam gates were open about 7 feet, the lake was at approximately normal level before the May 2020 flood, and the powerhouse was not operated. The results for the analyzed counterfactual scenarios in comparison to the baseline actual case can be briefly summarized as follows:

- *No Instability Failure.* If the dam had not failed due to embankment instability, the estimated maximum lake level under the operational scenario that occurred in May 2020 (about 7-foot gate openings and no powerhouse operations) is only about 0.3 feet higher than the level at the time of failure, at El. 681.6. This would have been below the dam crest, even in the area of crest subsidence in the Edenville left (east) embankment, and therefore the embankment would not have been overtopped.
- *Partial Duration of Powerhouse Operation.* If the full discharge capacity of the powerhouse had been available until the lake level was 3 feet above the normal lake level, and the embankment had not failed, the resulting estimated maximum lake level is approximately equal to the lake level at the time of failure, at El. 681.3.
- *Full Duration of Powerhouse Operation.* If the full discharge capacity of the powerhouse had been available to pass flows throughout the event and the embankment had not failed, the estimated maximum lake level is El. 680.5, about 0.8 feet lower than the estimated maximum lake level at the time of failure.
- *Gates Fully Open*. If all six of the gates at the Edenville and Tobacco spillways had been opened to a height of at least 10 feet, the resulting estimated maximum lake level is about 1.1 feet lower than the lake level at the time of failure, at El. 680.2.

- *Gates Fully Open and Partial Duration of Powerhouse Operation.* If the full discharge capacity of the powerhouse had been available until the lake level was 3 feet above the normal lake level, and all six spillway gates at the Tobacco and Edenville spillways had been opened to a height of 10 feet, the resulting estimated maximum lake level is about 1.4 feet lower than the lake level at the time of failure, at El. 679.9.
- *Gates Fully Open and Full Duration of Powerhouse Operation.* If the full discharge capacity of the powerhouse had been available to pass flows throughout the event, and all six spillway gates at the Tobacco and Edenville spillways had been opened to a height of 10 feet, the estimated maximum lake level is El. 679.5, about 1.8 feet lower than the lake level at the time of failure.
- *Pre-Lowering Wixom Lake with May 2020 Operations.* Three different pre-lowering scenarios were considered for the case of Edenville and Tobacco spillway gates open 7 feet (the estimated opening during the May 2020 event): (1) pre-lowering Wixom Lake to winter lake level (El. 672.8), (2) pre-lowering the lakes at all four Boyce Hydro dams to winter lake levels (3 feet below normal lake levels), and (3) pre-lowering Wixom Lake to run-of-the-river operations over the concrete spillway crests (estimated to be El. 670).

For the first case, the estimated maximum lake level is 0.1 feet higher than the lake level at the time of failure, assuming no embankment failure. For the other two scenarios, the reduction in the estimated maximum Wixom Lake level is estimated to be 0.2 feet relative to the lake level at the time of failure, at El. 681.1.

- *Gates Fully Open and Pre-Lowering Wixom Lake to Run-of-the-River.* As an interim risk reduction measure, if pre-lowering to run-of-the-river over the spillway crest had occurred, and all six spillway gates at the Tobacco and Edenville spillways had been opened to a height of 10 feet, the resulting estimated maximum lake level is about 1.4 feet lower than the lake level at the time of failure, at El. 679.9.
- *Upgraded PMF Spillway*. If the modifications proposed in 2012 for upgrading the Tobacco and Edenville spillways to accommodate the PMF (Tainter Gate Design Report and Calculations, Edenville Project [Mill Road 2012]) had been constructed and gates had been operated properly, the resulting maximum lake level would have been at or below the normal operating level.

For the run-of-the-river pre-lowered lake level cases described above, the estimated initial lake level at El. 670 is 2.2 feet above the concrete spillway crest elevation. In run-of-the-river mode, all flow through the river system must pass over the concrete spillway crest. El. 670 was selected to represent the Wixom Lake level that would be needed to pass the river flow, based on the experience during deep reservoir lowering in the winters of 2018-2019 and 2019-2020; see Appendix F1 for a discussion of lake levels during the deep drawdowns.

The purpose of completing the hypothetical scenario analyses was to provide information to assess whether different conditions or operations during the May 2020 event would have resulted in lower peak Wixom Lake levels to a degree that might have prevented the failure. This assessment is complicated by the lack of definitive knowledge of the conditions that triggered static liquefaction at Edenville Dam. As discussed in Section 4.1.1.3, there are very small differences in stress conditions between those that trigger static liquefaction and those that do not, and the failure occurs rapidly when the trigger conditions are reached. Although the IFT is confident that the record high lake level on May 19, 2020 contributed to triggering static liquefaction, it is not definitively known how much lower the peak lake level would have needed to be to prevent triggering. A lake level somewhat lower than that at the time of the failure, but sustained for a sufficient length of time, might have still triggered the failure.

Of the hypothetical scenarios considered, only the upgraded PMF spillway scenario would have clearly prevented the failure in May 2020, because it would have resulted in essentially no rise in lake level. Even for this scenario, some future flood approaching the PMF – a very low-probability event – would still have triggered the failure if the same geotechnical conditions existed.

Partial-duration operation of the powerhouse with the Edenville Dam spillway gate openings limited to 7 feet would have resulted in essentially no effect on peak lake level and very likely would have still resulted in the embankment failure. The pre-lowering alternatives with the gate openings limited to 7 feet would have resulted in less than 0.2 feet difference in peak lake level, which the IFT believes is unlikely to have prevented the failures.

Full-duration operation of the powerhouse with the Edenville Dam spillway gate openings limited to 7 feet would have resulted in a modest reduction in peak lake level of about 0.8 feet, which may or may not have prevented the failure.

The remaining hypothetical scenarios, all of which assumed full opening of the spillway gates to 10 feet or more, resulted in peak lake level reductions between 1.1 and 1.8 feet. As the peak lake level reduction increases, the likelihood that the failure would have been prevented increases. However, for the range of peak lake level reductions of 1.1 to 1.8 feet, the IFT cannot be confident that failure would have been prevented, and hence judges for these scenarios that the instability failure may or may not have happened.

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6. Evaluation of Emergency Response and Emergency Action Plans

The primary emergency management goal is to protect human lives and provide for public safety. In that regard, the emergency response to the failures of Edenville and Sanford Dams must be considered very successful. On May 18 and 19, 2020, about 11,000 people were evacuated without any reported fatalities or serious injuries before or after Edenville and Sanford Dams failed.

According to the 2020 major disaster declaration request by the State (Michigan 2020), the damage across a five-county area was estimated at roughly $190 million in losses for residents. Over 3,000 homes were affected, and there was $55 million in damage to public infrastructure; however, protection of property is a secondary goal of emergency management. It should also be noted that not all of the property damage is attributable to the two dam failures. Widespread flooding unrelated to the dam failures was reported in the area. Although the dam failures undoubtedly increased the property damage in Gladwin, Midland, and Saginaw Counties, to the IFT's knowledge, the property damage specifically attributable to the dam failures has not been quantified.

Emergency Decision-Making and Response

All four Boyce Hydro dams had written emergency action plans (EAPs), and copies of the EAPs were available at the dam sites, as required by FERC. The latest versions of the EAPs at the time of the failures were prepared in June 2018 (Boyce Hydro 2018a, 2018b, 2018c, and 2018d). However, the decision-making and emergency actions during the May 17 through 19 flood did not strictly follow the guidance in the EAPs.

The IFT interviewed the Emergency Managers (EMs) from Gladwin and Midland Counties, as well as others, regarding the emergency response to the flooding and the dam failures. Based on the interviews, the IFT understands that the EMs from the counties were coordinating response throughout the event.

Late in the day on May 18, as lake levels were rising at all four Boyce Hydro dams, the Midland County EM initiated the process that led to initial evacuations late that night and in the early morning hours of May 19. This evacuation decision was made after consultation with a Boyce Hydro operator regarding the status of the dam and a discussion with representatives of Midland County government. Based on discussions with the Midland County EM, the IFT understands that the Boyce Hydro operator was not able to confidently assure the EM that the dams would continue to perform well. The EM was uncomfortable with this situation and was concerned with the possibility of having to initiate evacuations in the early morning overnight hours if conditions deteriorated.

No engineer had input in discussions with the Midland County EM with regard to the decision to evacuate, and EGLE engineers were not even aware of the situation at the dam until hours after the decision had been made to evacuate. Since the decision to evacuate was timely and effective, the IFT does not believe that, in this case, input to the EM from engineers would have improved that decision. It is even possible that input from engineers could have resulted in delaying the decision to evacuate, possibly until the dam failed. However, it should not be inferred from this counterfactual scenario that engineering input is generally not needed or desired during dam emergency situations. On the contrary, the IFT believes that, as a general practice, it is desirable that a dam owner's qualified engineer or engineering consultant be available to advise the dam owner, EMs, and regulators when a potential or actual emergency situation develops at a dam.

The decision to begin evacuations was made late in the evening on May 18, and evacuations began shortly thereafter. The Midland County EM indicated that the decision to begin evacuations at that time was also based in part on availability of evacuation resources. The community surrounding the dams is rural and depends on volunteer firefighters to implement evacuations. The resources available to implement evacuations outside of daytime working hours are estimated to be eight or more times greater than those during daytime working hours. Further evacuations were ordered after Edenville Dam failed at about 5:35 p.m. on May 19.

The evacuations were reportedly well organized and orderly. In discussions with the IFT, the local EMs credit the successful evacuations, at least in part, to an EAP exercise that was conducted in 2019. The exercise allowed the EMs to understand the magnitude of flooding from potential dam failures and to develop evacuation plans with local first responders. From the exercises, public safety officials also realized that they would need to make evacuation decisions sooner than described in the EAPs to allow time for safe evacuations.

Discussions at the Edenville Dam Site

On May 19, the day of the failure, after evacuations had already begun, numerous individuals representing Boyce Hydro, FLTF, Spicer, Midland County, EGLE, and others had gathered at the Edenville Dam site to observe the lake level and the condition of the dam, and to discuss options to limit the rise of the lake level, limit erosion on the upstream slope of the dam, and prevent overtopping of the dam.

Boyce Hydro did not have an engineering consultant available to come to the site or participate in discussions related to the status of the dam. FLTF had engineers from its consultant, Spicer, on-site at the dam the day of the failure, and they provided input in discussions. However, since FLTF was not yet the legal dam owner, these engineers did not make recommendations or decisions as an owner’s representative.

Similarly, two engineers from EGLE were on-site on May 19, and they participated in discussions, but they also were not authorized to make recommendations or decisions as an owner’s representative. EGLE had authority to order that actions be taken to prevent dam failure, but the embankment instability failure was unforeseen by everyone at the site. The primary concern was with potential overtopping of the dam, and the lake levels were being closely monitored. The lake level was still more than 1 foot below the dam crest and the rate of rise was slowing shortly before the embankment instability failure occurred. Therefore, EGLE had not reached the point of ordering that action be taken to prevent failure of the dam, beyond the actions that were already being taken to address erosion at the upstream slope of the dam. Like EGLE, FERC would also have had authority to order a controlled breach if they had been the regulator, however FERC was no longer the regulator for the Edenville project after the license was revoked in 2018.

Emergency Action Plans (EAPs)

The emergency management decision process would have been somewhat different if the Edenville Dam EAP had been strictly followed. The Edenville Dam EAP (Boyce 2018a) establishes three general response conditions on page 9 of the document:

- Alert. This warning is issued when all gates are open and the reservoir pond level is rising above the high-water level (+0.3’). This condition indicates that the Operators do not have a means of controlling the water level of the impoundment until the level recedes below the high level.

- Condition A (Failure is imminent or has occurred). This emergency condition is issued once *failure has occurred, is occurring or it is obvious that failure is about to occur*. This would mean *a portion of the dam has collapsed or the embankment has been overtopped*. Emergency agencies will immediately activate all calling and evacuation procedures. [*Emphasis added.*]
- Condition B (Potential failure situation is developing). This emergency condition is issued when *a dam failure is likely, but not assured*. During this time the Licensee and Emergency Service Agencies further assess the situation and try to mitigate the conditions that may lead to dam failure. It is recommended that Emergency agencies be activated; however, *initiation of evacuation plans is not recommended*. The Licensee will provide frequent updates of the conditions to the agencies and *when failure or overtopping is a certainty, the status will be upgraded to Condition A*. [*Emphasis added.*]

Different descriptions of Conditions A and B, which are more narrowly defined, are provided on page 11 of the same document:

- Condition B is when either the reservoir level has risen more than 3 feet above the full pond level and is still rising, or when a serious leak has been detected in the embankment.
- Condition A is when *either the embankment has been overtopped, or the serious leak has caused the embankment to begin eroding*. [*Emphasis added.*]

An earlier section of the EAP includes the following statement regarding actions during Condition B:

- At such time as the problem is determined *to be more serious than at first thought, or is worsening, immediately declare a Condition A*. [*Emphasis added.*]

In general, the guidance in the EAP is inconsistent and ambiguous. For example:

1 The EAP indicates that Condition A is not reached until (1) “the embankment has been overtopped or the serious leak has caused the embankment to begin eroding;” (2) “failure has occurred, is occurring or it is obvious that failure is about to occur;” or (3) “a portion of the dam has collapsed or the embankment has been overtopped.” On the other hand, the document indicates that the condition can be elevated from B to A when “the problem is determined to be more serious than at first thought or is worsening.”

2 The statements for the general description of Condition B that “failure is likely, but not assured” and “initiation of evacuation plans is not recommended” reflect contradictory assessments of the situation, with the latter statement actively discouraging initiation of evacuation until Condition A is reached, when “failure has occurred, is occurring or it is obvious that failure is about to occur.”

3 Though the powerhouse was not operational in May 2020 due to the revocation of the FERC license in 2018, the EAP gives inconsistent guidance regarding operation of the turbines during a flood event.

   On page 10 of the EAP, it is implied that the turbines should remain running to help control the lake level, although it is not clear whether it is intended that the turbines be operated at Edenville Dam or Sanford Dam: “If a Condition A or B has been declared, the Licensee shall continue to take any steps that may lead to alleviating the emergency, including lowering the water level with the spillway gates, ensuring that downstream dams are able to pass any increased water flows *by operating turbines* and spillways, and by effecting temporary repairs.” [*Emphasis added.*]

> On page 11, the EAP appears to indicate that the turbines should be shut down once Condition B has been reached, although it is not clear whether it is intended that the turbines be shut down at Edenville Dam or Sanford Dam: "If a Condition A or B has been declared two Operators will be dispatched to the Sanford site to raise all gates to their maximum height. *The turbines will be shut down* and the high voltage knife switches opened." [*Emphasis added.*]
>
> On page 12, the EAP indicates that the turbines should not be shut down until Condition A is "anticipated," leaving the operators to judge how far beyond Condition B, and how close to Condition A, the situation needs to be before shutting down the turbines: "Severe floods may lead to overtopping of the dam despite the operation of the spillway gates to their maximum opening. In this instance there is little more that the operator can do other than keep a close watch on the dam and report on any changed conditions that may occur. *The turbines should be shut off if a Condition A is anticipated* and the main disconnect or knife switch opened to remove the plant from the grid." [*Emphasis added.*]

More broadly, when looking at the EAPs for all four Boyce Hydro dams and considering the dams to comprise a system of four dams in series, the four EAPs link the operation of each dam to the other dams to some extent, but the guidance related to these operations is inconsistent across the four EAPs.

All of these inconsistencies and ambiguities in the EAPs create a dilemma for the operators, and the operators have to draw largely on their own judgment when making decisions, which is generally what they did during the May 2020 event.

In addition, it should be noted that the EAPs for Secord Dam (Boyce Hydro 2018c) and Smallwood Dam (Boyce Hydro 2018d) appear to assume that failure of either of those dams would not result in overtopping failure of Edenville Dam: "the Edenville Reservoir will absorb *most* of the flow from a dam failure at Secord and so areas downstream of Edenville should not be severely impacted" and "the Edenville Reservoir will absorb *much* of the flow from a dam failure at Smallwood and so areas downstream of Edenville should not be severely impacted." [*Emphasis added.*] In making these statements, the EAPs do not make a distinction between "sunny day" failure versus failure during a storm. In the opinion of the IFT, it is plausible that, if a flood wave resulting from failure of Secord Dam or Smallwood Dam arrived at Wixom Lake when that lake was already high due to inflows from a storm (as was the case in May 2020), then Edenville Dam could be overtopped and fail. The IFT was told that the scenario considered during the 2019 EAP exercise assumed that Smallwood Dam fails, and, contrary to the EAPs, further assumed that Edenville Dam would fail as a result of the Smallwood Dam failure. It is fortunate that the scenario of Edenville Dam failing was considered during that exercise.

Event Sequence

Based on the Edenville EAP and the recorded water levels at Wixom Lake, an Alert condition was reached at Edenville Dam sometime during the day on May 18, and Condition B was reached sometime between midnight and 6:00 a.m. on May 19. The Edenville Dam EAP indicates that Midland County Dispatch and Gladwin County Central Dispatch are to be notified for the Alert Condition as well as for Conditions A and B. For Conditions A and B, several other organizations, including EGLE and FERC, are to be notified, although the FERC notification was no longer applicable for Edenville Dam on May 19, 2020, because of the prior license revocation. The Edenville Dam EAP does not include Saginaw County Central Dispatch in the notification chart, but Saginaw County Emergency Management is listed as one of the holders of a copy of the EAP (Boyce 2018a).

The supplemental incident report (Boyce Hydro 2020b) indicates that the operator declared a Condition B at Edenville at 3:30 a.m. on May 19, when Wixom Lake had risen to 3 feet above the normal lake level. The report further indicates that the operator notified “the two Emergency Management departments.” The Midland County EM confirms that notification of Condition B was received at 4:35 a.m. on May 19 and that notification of the breach of Edenville Dam was received after the embankment failure occurred. No specific notification of an Alert condition was received, but the Midland County EM was in contact with the dam operator during the evening of May 18, and gate operations at the dams were reported to county emergency dispatch centers.

From these discussions, it can be seen that evacuations started while Edenville Dam was still in the Alert condition but approaching Condition B, which is earlier than would have been indicated by the EAP. Had the decision to evacuate not been made at that time, the IFT believes that it is likely that the decision to evacuate would have been made by late morning on May 19, which would have still allowed sufficient time for evacuations before the failure of Edenville Dam at 5:35 p.m.

In the daylight morning hours of May 19, Wixom Lake was still rising, erosion had occurred on the upstream slope near the crest, and serious concerns existed about possible embankment overtopping (to the point of discussions of a possible controlled breach). Given those conditions, the IFT believes that authorities would have judged the likelihood of failure to be high enough to warrant evacuation. However, the EAP guidance could be interpreted to mean that evacuations would not have been issued until the embankment failure occurred at 5:35 p.m. Had that choice been taken, the evacuation may not have been nearly as successful, and fatalities may have resulted.

The Midland County EM also indicated that evacuations could have been more complicated, if they had been delayed until late morning or afternoon on May 19. The EM reported that some residents on Wixom Lake in Midland County who were part of the initial evacuation try to return to their homes around 6:00 a.m. on May 19 to find many of the homes inaccessible because of water levels in and around the structures. Delaying the evacuations of these families until May 19 would have resulted in some of the families awakening to more than a foot of water in their homes, which would have made evacuation less safe.

Summary

In summary, the emergency management of the event and the resulting evacuations were effective at protecting public health and preventing loss of life. The evacuations occurred early, because of a prudent, proactive decision by the Midland County EM. This event reinforced the importance of EAP exercises, in that the EAP exercise completed in 2019 resulted in the local EMs developing an understanding of the significance of possible dam failures and developing appropriate evacuation plans.

Although the IFT believes that evacuations likely would have been ordered no later than midday on May 19, had they not been ordered the night before, it is possible that the inconsistencies in the EAP could have led to a delay in evacuations. This points out the need for EAP guidance to be consistent and to allow for judgment, so that evacuations can be ordered when the risk of failure is judged to be sufficiently high, rather than waiting for failure to initiate evacuation. In this case, because of the rapidity of the embankment failure, had evacuations not been initiated before the actual failure, it is entirely possible that lives would have been lost if evacuation had been delayed until after the failure.

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7. Contributions of Human Factors to the Edenville Dam Failure

From a physical standpoint, the failure of Edenville Dam was fundamentally an embankment instability failure that was triggered by a lake level that was historically high, but not high enough to result in overtopping of the dam. More fundamentally, the failure was due to a long history of interactions of physical and human factors, going back to the original design and construction of the dam. Section 7.1 provides a history of events that are relevant to human factors, Section 7.2 presents the IFT's findings regarding the primary human factors that contributed to the failure, and Appendix G describes the human factors framework, methodology, and analysis.

7.1 Relevant Project History

7.1.1 Project Ownership

As noted in Section 2.1, Wolverine Power Company was founded by Frank Wixom in 1923, and Wolverine Power Company and Consumers Power Company (Consumers) signed a power purchase agreement (PPA) for a term of 99 years (Wolverine 1923) at the same time that Wolverine Power Company filed a deed for land purchased from the Riverdale Farm Company for the four future dams and lake bottoms. As specified in the PPA, Wolverine Power Company was required to build the dams and sell all power to Consumers, and Consumers would have limited authority to direct operations, as described in more detail in Section 7.1.8.

The Wolverine Power Company went into bankruptcy in the early 1930s, and in May 1934, Wolverine Power Company sold all of its assets and properties to Edenville Power Company, which was owned by Frank Wixom, and then Edenville Power Company was immediately renamed Wolverine Power Corporation (Wolverine). Around 1948, there was discussion between Wolverine and Dow Chemical about selling the assets and properties to Dow Chemical; however, the sale never moved forward. Several decades later, New World Power Corporation, a holding entity focused on renewable energy, was formed in 1993 and by May 1994 had acquired all shares of Wolverine.

In 2003, Wolverine defaulted on a loan from Synex Energy Resources, Ltd., a Vancouver-based engineering and consulting company. Synex Energy Resources foreclosed on the four dams; acquired the deeds on all land, equipment, and offices; and created a holding company called Synex Michigan, LLC (some documents refer to Synex Wolverine, LLC), a Synex Energy Resources subsidiary.

After only 3 years of Synex Michigan ownership, in 2006 W.D. Boyce Trusts purchased real estate consisting of the four dams and related property, and separately acquired 100 percent of the LLC membership interests of Synex Michigan. In 2007, Synex Michigan was renamed Boyce Hydro Power, LLC. The full ownership structure for the entities owned by three W.D. Boyce Trusts is illustrated in Figure 7-1 (Boyce Hydro 2020e). While Boyce Hydro Power, LLC, held the FERC licenses and the contracts with Consumers[18], Boyce Hydro, LLC, was responsible for operations and management. In this report, all of the entities related to the dams and owned by the Boyce trusts are collectively referred to as "Boyce Hydro."

[18] Consumers Power Company became Consumers Energy in 1997. In this report both entities are referred to as "Consumers."

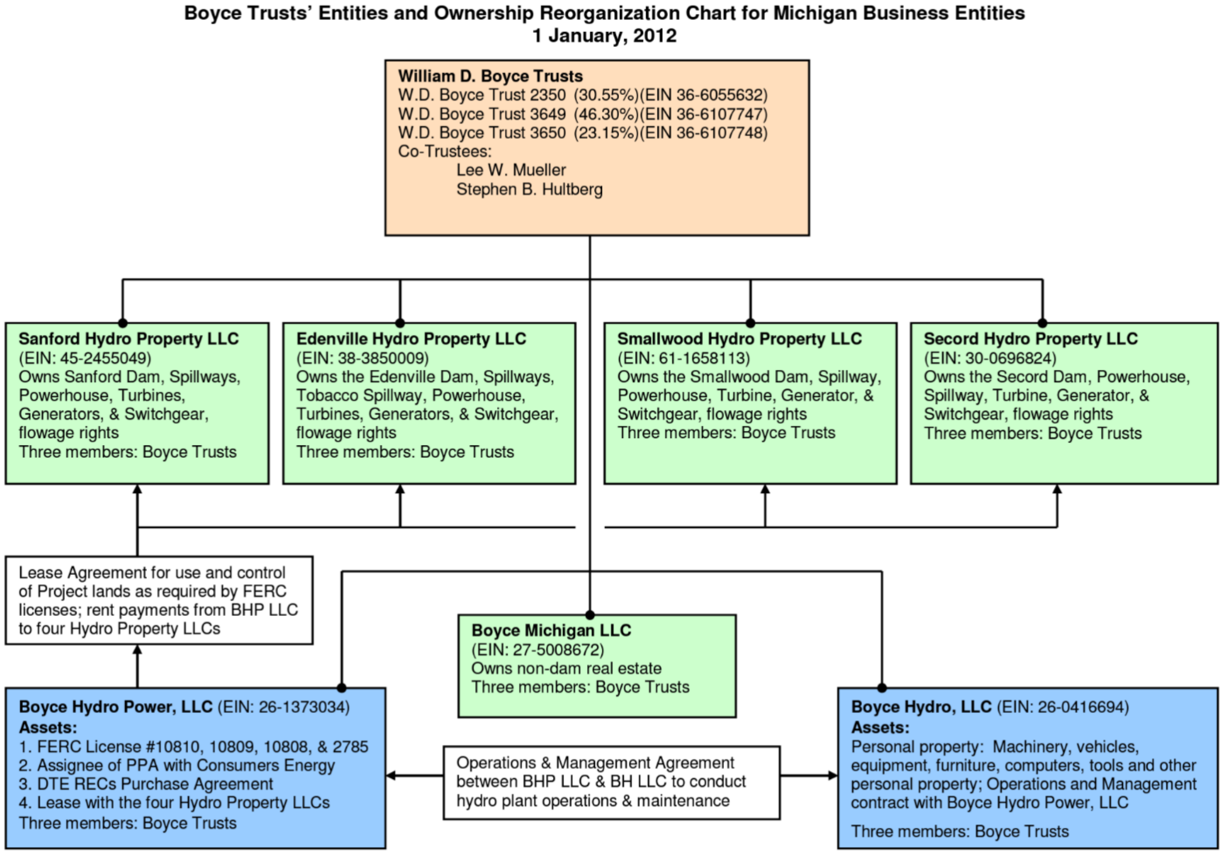

Figure 7-1: Boyce Hydro Ownership Structure (Boyce Hydro 2020e)

The Sanford Lake Preservation Association (SLPA) was formed as a 501(c)(3) organization in 2011 in an effort to facilitate restoration of the Sanford Lake level, which had been lowered by an emergency permit in 2010 for repairs. Around 2012, the SLPA began discussions with Boyce Hydro about finding a future buyer for the dams, including a community-owned model. The SLPA and other lake associations, working with Gladwin County, eventually formed FLTF through a memorandum of understanding (MOU) (FLTF 2018) in July 2018 to address the sustainability of all the lakes; this original version of the FLTF formed by the MOU was a task force and not a legal entity. In July 2018, the SLPA and Boyce Hydro signed a letter of intent (Boyce Hydro 2018g) for the SLPA to purchase the four hydro projects. Two months later, in September 2018, FERC revoked the license for Edenville Dam. In November 2018, SLPA changed its charter to include all four lakes and created a "DBA" name: Four Lakes Task Force. The SLPA was renamed the Four Lakes Task Force in September 2019, replacing the previously formed task force of the same name.

In April 2019, FLTF and Boyce Hydro signed a tentative agreement (Boyce Hydro 2019d) for FLTF to purchase the four dams and lakes. This agreement included provisions that Boyce Hydro would continue to operate the dams until the sale was complete, which was anticipated to be sometime in 2022. In August 2019, Midland and Gladwin Counties and the FLTF entered into a three-party agreement (FLTF 2019c) formally establishing FLTF as the counties' legal delegated authority for acquiring, managing, repairing, and maintaining the four dams and lakes, including maintaining the legal lake levels established by court order in May 2019 (State of Michigan 2019); Boyce Hydro was not a party to this agreement. A Special Assessment District (SAD) was also established, which consisted of properties with lake frontage or easements that could be required to contribute special assessment funds to finance actions related to the dams and lakes.

Once FERC revoked the license for the Edenville project, EGLE became the regulator for Edenville Dam. Under Part 315 and Part 307 of the Michigan Natural Resources and Environmental Protection Act, the *functional* “ownership” of the dam from EGLE’s perspective became less clear. Part 315, Section 31504(5) states that "owner" means a person who owns, leases, controls, operates, maintains, manages, or proposes to construct a dam. According to EGLE, both Boyce Hydro and FLTF would have met this definition after the legal lake level order and delegation of authority to FLTF. Although EGLE was working directly with FLTF and its consultant on issues related to Edenville Dam, since Boyce Hydro still legally owned the dam, EGLE required that Boyce Hydro provide concurrence for any permits issued and sign those permits because EGLE cannot typically authorize anyone other than the landowner to take action on their property.

In addition, Part 307, Section 30701(e) states that “... 'delegated authority' means the county drain commissioner or any other person designated by the county board to perform duties required under this part.” Further, Section 30702(3) states that “If a court-determined normal level is established pursuant to this part, the delegated authority of the county or counties in which the lake is located shall maintain that normal level” and Section 30722 states that:

> “(1) The delegated authority of a county shall cause an inspection to be made of each dam on an inland lake within the county which has a normal level established under this part or under any previous act governing lake levels. The inspection shall be conducted by a licensed professional engineer. The inspection shall take place every third year from the date of completion of a new dam or every third year from the determination of a normal level for an existing dam. An inspection report shall be submitted promptly to the department in the form and manner the department prescribes.
>
> (2) If a report discloses a need for repairs or a change in condition of the dam that relates to the dam's safety or danger to natural resources, the department shall conduct an inspection to confirm the report. If the report is confirmed and the public safety or natural resources are endangered by the risk of failure of the dam, the department may require the county either to repair or to replace the dam. Plans and specifications for the repairs or replacement shall be prepared by a licensed professional engineer under the direction of the delegated authority. The plans and specifications shall be approved by the department before construction begins. The department shall review and approve or reject the plans and specifications within 30 days after they are received by the department. If the plans and specifications are rejected, the department shall propose changes in the plans and specifications that would result in their approval by the department. If the dam is in imminent danger of failure, the department may order an immediate lowering of the lake level until necessary repair or replacement is complete.
>
> (3) A person failing to comply with this section, or falsely representing dam conditions, is guilty of misconduct in office.”

In short, according to EGLE, the delegated authority assumes many of the “ownership” responsibilities when they are given authority over a dam under Part 307, even if they do not legally own the dam. In the opinion of the IFT, the situation of having two organizations *functionally* (not legally) acting in a dam owner’s role did not generally result in significant gaps with respect to engineering studies and decisions under normal dam operating conditions; however, it did have implications for decision-making during the May 2020 event (see Section 6).

In December 2019, Boyce Hydro and FLTF signed a purchase agreement (Boyce Hydro 2019e) that would have required FLTF to deposit funds and Boyce Hydro to deposit transaction documents to an escrow agent. The date of the first installment payment was to be January 2020, which was delayed to June 2020 due to litigation between EGLE and Boyce Hydro over winter drawdown and alleged impacts to mussels. Per this purchase agreement, after closing of the first installment, Boyce Hydro would remain the legal owner of the dams, party to the Consumers PPA, and the FERC licensee (for Sanford, Smallwood, and Secord Dams) through the closing date, scheduled for 2 years after the first payment, while FLTF would remain the delegated authority for Gladwin and Midland Counties.

With this process, FTLF would purchase the dams and related assets with funding from the sale of bonds; however, these bonds could not be issued until the SAD had been established. FLTF had received about $5 million in state grant funding for studies, capital improvements, and acquisition costs to bring the dams into compliance with FERC requirements. Until the sale was completed, the plan was that Boyce Hydro would continue operating the dams and retain all revenue from the sale of electricity and Renewable Energy Certificates (RECs), under contract with Four Lakes Operations (FLO), an operating company created by FLTF. After the sale was completed, the plan was that FLO would transfer the operating contract to a new operator and Boyce Hydro would no longer have any involvement with the dams. For the time which Boyce Hydro would continue operation of the dams, FLTF would assume responsibility for directing seasonal reservoir levels, any reporting associated with FERC-mandated environmental regulation at the Sanford, Smallwood, and Secord dams, and for all engineering, design, permitting, and construction of dam-related repairs or improvement projects. Boyce Hydro’s obligation was to provide daily operations at the dams, general maintenance of and minor repairs to the facilities, and to communicate with and report on operation activities to FLTF through FLO. On May 9, 2020, less than two weeks before the dam failures, Boyce Hydro, FLTF, and FLO completed an amendment (Boyce Hydro 2020d) to the purchase agreement, which set the closing date for June 1, 2020.

However, on May 19, 2020, Edenville and Sanford Dams failed, and FLTF notified Boyce Hydro that the sale would not proceed because the conditions of the closing could not be met. The purchase agreement was therefore nullified. Gladwin and Midland Counties then proceeded to take ownership of all four dams after the failures via their condemnation authority under Section 30708 of Part 307.

7.1.2 Design and Construction

The four dams were designed by Holland, Ackerman & Holland and were constructed between 1923 and 1925. The agreement between Wolverine and Consumers, discussed in Section 7.1.1 above, specified that construction was to begin on or about June 1, 1923, and be completed by December 31, 1925, a period of 2.5 years. There was a financial disincentive for late completion in the form of penalty payments from Wolverine to Consumers related to both interest and depreciation associated with the cost of the transmission lines and related equipment that was constructed or installed by Consumers.

The IFT found excerpts of construction specifications entitled “Agreement and Specifications for the Construction of the Earth Embankments Central Section of the Edenville Plant” (Holland 1924a) and “Agreement and Specifications for the Construction of the Earth Embankments – South East End of the Edenville Plant” (Holland 1924b) that appeared to contain the same language and were presumably intended to be used for construction of those sections of the Edenville Dam project. Regarding the materials to be used in the embankment, the construction specification stated that:

> “The materials of the earth fill shall be free from vegetable and other perishable matter and in the upstream part of the earth fill from all stones measuring six inches or more in

> their greatest dimension. Upstream of the center line of the earth fill the material shall be selected of clay, gravel and loam mixed *so as to make as impervious an earth fill as possible*, while that of the downstream side of the earth fill shall be free from clay so far as the same may be obtained in the borrow pits and *be pervious*. The earth of the different parts of the earth fill shall be taken from the borrow site over such areas as directed by the Engineer *to obtain the best available materials for the earth fills*." [*Emphasis added.*]

This section of the specification indicates a clear intention to have distinct upstream and downstream zones in the embankment, and it has long been generally understood in the dam industry that an "impervious" upstream zone is intended to reduce seepage through an embankment dam.

Regarding the placement of these materials, the construction specification stated that:

> "All materials of the earth fills shall be spread and harrowed in horizontal layers and compacted before any layer exceeds nine inches in thickness. Compacting shall be done with a grooved roller weighing not less than one ton per lineal foot of roller tread, or such equivalent of the roller as will meet the approval of the Engineer; *the intention being to insure a thorough compacting of the whole earth fill*. If the materials as deposited in the earth fill cohere such as to form lumps not easily compacted, such materials after spreading shall be thoroughly disked in addition to harrowing to insure the pulverization of such lumps. Unless the materials are sufficiently moist when spread, each layer shall be wetted to secure the desired compacting. When the location does not admit of rolling or other similar means of compacting, all layers shall be compacted with heavy rammers to the satisfaction of the Engineer. Care must be taken in placing the earth fill not to break the tile drains hereinbefore described. *No work shall be done when the earth fill or ground is frozen, except under the express direction of the Engineer*." [*Emphasis added.*]

While this section of the specification does not indicate a specific degree of compaction, the intention of "thorough compacting of the whole earth fill" is clear. The lack of a specific compaction requirement is likely inherent to the time at which the dams were constructed, as the practice of soil mechanics, now geotechnical engineering, was in its infancy; for instance, the now widely used Proctor compaction test method was not in use until the early 1930s. This section of the construction specification also demonstrates recognition that construction of the embankment could be compromised when soil is frozen, and therefore generally prohibits embankment construction during such periods.

The construction specification also stated the following regarding the geometry of the embankments:

> "The top of the earth fill during construction shall be at all times maintained at substantially uniform elevation, with the sides one to two feet higher than the center, except that the Engineer may allow certain sections of the earth fill to be carried forward in advance of the remainder. In constructing the earth fill the Contractor shall allow five percent of the height of the same above the original ground line for settling. *The slopes of the earth fill up and downstream shall be that shown and continuous throughout the length of the earth fill. The slopes may be made flatter at the discretion of the Engineer*. The top of the earth fill shall be of the width shown and at the elevation shown on the plans, plus the addition hereinbefore named to provide for settlement. The exposed downstream surface of the earth fill shall be smoothed for seeding." [*Emphasis added*]

The design plans indicate that the downstream slope was to be 2H:1V and the upstream slope was to be 2.5H:1V. Surveys indicate that the actual slopes were steeper than these design slopes in some locations, including the failure section.

The construction specification indicated the following regarding the sources of material to be used to construct the embankments:

> "*The borrow for the earth fills may preferably be made from pits adjacent but not nearer than a line parallel to and fifty feet from the upstream toe of the earth fill*. The slope of the borrow pit on the site adjacent to the earth fill shall not exceed one foot vertical to three feet horizontal. *The borrow for the earth fills may also be made from pits downstream of the earth fill when approved by the Engineer, and when so made shall not be nearer than fifty feet from the downstream toe of the earth fill*, and with the side of the borrow pit adjacent to the earth fill at a slope no steeper than one foot vertical to three feet horizontal. Such borrow pits must be drained to the river and left so not to present in the opinion of the Engineer an unsightly appearance." [*Emphasis added*]

Considering that the total length of the Edenville embankments is about 6,000 feet, it could be expected that borrowing material along the length of the embankments would increase the potential for variability in the materials placed in the embankments, and therefore there is increased potential for some sections of the embankment to contain materials that generally did not meet the requirements of the construction specification.

Regarding the installation of drain tile, the construction specifications stated that:

> "The Contractor shall place tile drains in the foundations of the earth fill and other places and shown on the plans or directed by the Engineer. Drainage will be provided by tile drains located in the downstream half of the earth fill as shown. After site of the earth fill has been prepared as hereinbefore provided under "STRIPPING" and where necessary sufficient earth fill has been made to substantially level the site to the original ground surface, trenches about 18 inches wide and 12 inches deep shall be made to receive the tile drains. Spoil from these trenches shall be spread on the earth fill. The tile shall then be carefully laid in these trenches on a layer of gravel at a grade of not less than 1 foot in 100 feet, and with an opening of about ¾ of an inches left between the spigot end of one tile and the seat of the socket of the next adjacent tile. The trench shall then be back filled with coarse gravel to the level of the site. The tile shall terminate at the upstream edge of a shallow ditch which will be constructed under these Specifications about 10 feet downstream of the toe of the earth fill ..." and "... The tile will be standard quality vitrified sewer crock with bell and spigot ends. Only whole tile without cracks will be used in the work. Should any obstructions be found in any drain it shall be promptly removed, and should any damaged drain tile be discovered before the completion of the work under these Specifications, they shall be removed and replaced at the expense of the Contractor with new pieces, in a manner satisfactory to the Engineer. Particular care must be taken in making the earth fill as hereinafter provided, that the drain tile will not be damaged or broken."

The IFT found construction records that indicate that the design firm also provided construction-phase services. During the construction phase of the project, a series of communications between H. K. Holland (from the design firm) and C. E. Bottum (resident engineer, apparently also from the design firm) were particularly informative. Excerpts from construction memos are provided in Appendix B. These communications discuss Edenville embankment construction issues related to the installation of drains, seepage, "sluffing" of slopes, frost, and embankment settlement and cracking. Most of these issues pertain specifically to the sections of the embankment north and west of the powerhouse (not the Edenville left embankment), and a few pertain specifically to the Edenville left (east) embankment, which included the location where the May 2020 failure occurred. The following are excerpts from these communications, which span a period of about 6 months during the winter of 1924/1925 into spring 1925, presented in chronological order:

A memorandum dated December 10, 1924, from Wolverine to the design firm indicates the following:

> "... the main embankment at Edenville North of the power house was becoming saturated at the toe at a point about midway between the power house and the first bench and for a distance of about 100 feet. This is the seepage that we were expecting when we notified Mr. Bick that it would be necessary to place a clay layer on the face of the sand embankments. The most satisfactory remedy is to place tile drains about even with the ground surface on 15 ft. centers, extending into the bank at least 10 feet. These will carry off the water and leave the sand dry to retain the saturated material inside of the fill. The reason the moisture does not get to the tile drains is as we discussed previously, *which is that the first 2 or 3 feet of embankment was loam running quite heavily to clay taken from the borrow pit on the flats upstream of the embankment*. This material is rather impervious and the water in place of penetrating this layer to get into the drains comes out along the top, so that the additional drains should be placed in the sand, *and not covered up with this clay layer as was done previously*. That is, place the drains on top of the layer, rather than into it." *[Emphasis added.]*

Although this communication is not addressing the Edenville left embankment, it indicates that a sufficient arrangement of functioning drains may not have been included in the original design of the dam, that additional drains may have been added during construction as a means to address seepage problems, and that there appear to have been problems with improper installation of drains, which may have diminished or prevented their functioning.

This memorandum also indicated, in reference to the passage above:

> "While this seepage is serious, it is not at its present stage dangerous. *There would be considerably more danger from the clay fill south of the retaining wall with a seepage similar to this one, as the clay does not settle down and fill voids as the sand will do. I believe at the present time the point that gives us the most concern is the narrow fill south of the Retaining wall at Edenville*." [*Emphasis added.*]

This passage indicates that the seepage occurring north (northwest) of the powerhouse was not occurring south (southeast) of the powerhouse, which includes the section of embankment that eventually failed in 2020. The absence of seepage south and east of the powerhouse and reference in this and a subsequent memorandum to clay in the embankment south and east of the powerhouse and sand embankments north and west of the powerhouse support the possibility that the section south and east of the powerhouse (the Edenville left embankment) may have been constructed of different material than other sections of the embankment.

A memorandum dated January 20, 1925, from Wolverine to the design firm indicates the following:

> "... the Sanford plant was on at noon, and was glad to receive your telephone call saying that the Edenville plant was also connected to the Consumers system. We are leaving it to you to get in touch with Mr. Conrad and arrange for operating and for delivering energy to the Consumer Power system. *This should be done as quickly as possible so that the plant will begin returning a revenue.*" [*Emphasis added.*]

Given that the agreement between Wolverine and Consumers required that the project be completed by December 31, 1925, it appears that construction was well ahead of this schedule, and this may have been driven by a desire to generate power and revenue "as quickly as possible." This memorandum also indicated that "it is essential that the Edenville pond on the Tittabawassee River reaches the crest before the spring floods so that we will have an opportunity to repair the seepage spots before we have higher floods." The locations of these seepage spots are not indicated in this memorandum, although it can be inferred from the various memoranda that these locations are typically north and west of the powerhouse. It is apparent that there was a significant concern about seepage worsening in those locations when the lake reached a higher level.

In a memorandum dated January 30, 1925, from the resident engineer to the design firm, several issues are noted, namely that there may be issues with installing a clay layer on the upstream face north and west of the powerhouse, that material for embankment construction was starting to be in short supply, and that the embankment south and east of the powerhouse had remained dry. Also, there were concerns about the potential for excessive seepage north and west of the powerhouse due to inadequate tile drains once frost thawed in the spring and spring flooding occurred:

> "It has been the intention to place clay on the upstream face of the Dam on the section North of the P.H. after the other work is completed. I do not believe it is going to be feasible to place this clay without pulling the pond and besides *we are going to need all the dirt in the present borrow pits to finish the dam proper*. It would seem to me that some method could better be used where the work could be done on the downstream side and I believe we should consider such methods. I am not sure that the tile drains are going to handle the situation at the time of the spring flood and at which time the frost will be coming out of the downstream toe." [*Emphasis added.*]

In this memorandum, it was also noted that considerable frost was present along the downstream slope and presenting concerns: "There is about 8" of frost on the downstream side of the fill and since the downstream slope is greater than the natural slope of the sand when wet the material underneath the frost takes the flatter slope and leaves the frost standing up and hence leaves a void between the frost and the wet dirt. This void extends back for a distance of 6 or 8 ft. in places we have picked out."

This appears to reflect a violation of the construction specification requirement that work not take place when the ground is frozen, and also indicates a concern that the downstream slope was unstable if it was not adequately drained. A discussion of potential remedies for the issue was not found, and therefore it is unclear what corrective action, if any, was taken. This memorandum also stated that *"on the South side of the Power house the downstream toe shows no signs of being wet to date."* [*Emphasis added.*] This indicates that the embankment south and east of the powerhouse, which includes the section that failed in 2020, was not experiencing the same level of seepage as the "sand embankments" to the north and west of the powerhouse.

A memorandum dated February 2, 1925, from the design firm to the resident engineer, and referencing the "East central section" of the Edenville embankment, cautioned that inspection during the installation of each drain line was imperative when excavating in wet sand:

> "It has been our experience that when the drains are carefully laid surrounded with gravel and laid for a distance of 10 to 15 feet inside of the line of the downstream toe of the embankment that this has dried up the moisture above the tile drains in the embankments 30 to 50 feet high. But where the drains have not been carefully laid it has been hard to tell any difference from the amount of sluffing, and to get drains properly laid it is necessary for someone to inspect each line of tile as they are being placed, as when excavating in wet sand the sand runs in so fast that the workmen will lay the tile in almost any position so as to say they have laid them, and then cover the tile up. We have found that *in sand embankments such as the one at Edenville*, it is necessary to lay the tile on a

> board to lay them anywhere near straight and to keep them straight, and that the tile must be laid as near the bottom of the sand on top of the impervious layer as possible. The tile should then be covered with coarse gravel so as to act as a screen to keep the tile from being filled with sand. The tile should be carried 10 to 15 feet downstream of the toe and be buried to at least 12" deep with fill if they are above the level of the natural ground. Where tile are carefully laid and inspected we have had excellent results. *Where they have been poorly laid and not inspected we have had uniformly poor results.* We do not know of any method that will work better or be more satisfactory than tile in the downstream toe ...." [*Emphasis added.*]

Three days later, in a memorandum dated February 5, 1925, the resident engineer acknowledged a lack of construction oversight:

> "... As for the inspection: Both Mr. Crew and myself are giving as much time to the Edenville embankment as we can take from other part of the work, and the men have been shown the proper method of laying the tile and in most cases are getting the tile in pretty good. However, *since it has been impossible for either Mr. Crew or myself to be on this part of the work at all times*, the tile were being laid, and hence not able to inspect every tile, there have been some which were not laid deep enough and perhaps did not have enough gravel around them as you suggested. These in some cases have plugged up and we have had them relaid. The downstream toe is very wet yet in places up to about elevation 642." [*Emphasis added.*]

These communications indicate that the Edenville embankments north and west of the powerhouse generally consisted of sandy materials, that tile drains were considered to be effective in draining those embankments if installed correctly, that resources to inspect the installation of the tile drains were limited, and that some drains were installed incorrectly to the extent that they were rendered ineffective. For the section of the embankment south and east of the powerhouse where the failure section was located, from these communications it is not clear what the extent of installed tile drains was or what difficulties may have been encountered in properly installing drains in that section of the embankment.

A memorandum dated February 5, 1925, from the resident engineer to the design firm indicates:

> "... As to the clay on the upstream face: This of course would seal the embankment to a large extent but the question in my mind is how to get the clay where we want it and where to get it from. We could probably buy more land on the East end of the dam and haul it over the spillway with the dinkies. But, would *dumping off the top of the fill* place it where we need it most? We might place it with barges or perhaps we could rig up a cable and car outfit, a long boom clam shell or some other way. There are a good many ways possible, but most, it seems, *would be so expensive* that I thought perhaps it would be cheaper to work from the downstream side. I believe we should get some plan under way for the clay if this is to be put on at once, and *hence my deep concern.*" [*Emphasis added.*]

Although the location for the proposed clay material is not indicated in the excerpt above, the preceding and following chain of correspondence suggests it was to be placed north and west of the powerhouse. This communication indicates that there was a "deep concern," which was presumably related to seepage, that sufficient clay material was not readily available, that modifications to address seepage on the upstream side were hampered by the lake level, and that cost was a major consideration and potentially viewed as prohibitive. In addition, there is reference to dumping material, but there is no reference to compacting it.

A memorandum dated March 6, 1925, from the resident engineer to the design firm indicates that "... We are going to place clay on the Edenville embankment on the section North of the power house to the high bank – clay to be placed approximately 2 feet thick from the top of embankment to the water line." This appears to confirm that a clay layer was installed on the upstream slope north and west of the powerhouse, but only between the embankment crest and the water level. Based on construction photos, it appears that the water level was relatively high at that time.

A memorandum dated April 16, 1925, from the resident engineer to the design firm stated that "The Edenville embankment on the East end is settling and cracking as you expected and the tile do not run very freely. However the downstream toe is dry and we are watching every movement." This memorandum apparently pertains to the section where the failure section was located. The referenced settlement and cracking of the embankment suggests that the material was placed with little or no compaction. The reference to cracking and a dry toe may also be a result of higher fines content within this section of the embankment. There was also possibly a problem with the installation of the tile drains, which resulted in their not draining the embankment effectively. However, the lack of flow out of the drains may also be a result of lower permeability material. It appears that significant seepage was not observed at the downstream slope in this section during construction, and this is consistent with the limited seepage observed in this location over the next 90+ years until the failure in May 2020. The lack of seepage in this location suggests that some portion of the embankment was constructed with a significant fraction of clay and/or silt materials and/or the drains were ineffective.

A memorandum dated May 13, 1925, from the design engineer to the resident engineer describes a concerning observation at the Secord embankment based upon its similarity to issues experienced at Edenville:

> "After seeing the Secord embankment on May fourth I have been a little bit concerned about the method of placing the earth on the East side, that being placed by Mr. Burton. That fill is getting very much the same condition that the *east side of Edenville* was, especially due to *dumping* the wheel scrapers off of the high embankment *so that the earth gets no packing* and the carrying up of a very small portion of the embankment 10 to 15 feet higher than the remainder of the embankment which will cause very bad settling, and in this clay material will produce cracks that will make this embankment of a *considerable hazard*." [*Emphasis added.*]

This communication refers to both Secord Dam and Edenville Dam, and appears to indicate that clay was used in some portions of the Edenville embankments, and that rather than constructing the embankment in compacted layers, the material was instead dumped with little or no compaction, which was resulting in settlement and cracking that were causing serious concerns. Specific reference is made to the "east side of Edenville," which may include the location where the May 2020 failure occurred.

Numerous construction photographs indicate that men, horses, tractors, steam shovels, and steam engines were used for the earthmoving and construction operations. None of the photographs appears to show any compaction equipment or compaction operations, such as the grooved rollers or heavy rammers that the 1924 construction specification indicated should be used. It appears that the east section of the embankment was constructed by spreading from a stockpile (downstream) and dumping from rail cars (upstream) without use of a trestle, whereas trestles appear to have been used for construction of the Tobacco embankments. In addition, some of the construction photos show a color variation from upstream to downstream in the Edenville left embankment. There also is a strong indication that the contractor used material from two different borrow sources for the Edenville left embankment, one for the downstream portion that appeared to come from a downstream borrow area (a light-colored material that appeared granular based on the angle of repose of the stockpile) and another for the upstream portion that seemed to come from an area east of the dam. Selected construction photos are provided in Appendix D.

The rudimentary nature of the construction equipment available to the contractor in the early 1920s would be expected to influence the quality of work achieved, due to the hardship presented by performing work by hand that is now performed with powerful modern equipment. In addition, it appears that some construction was done during the winter when the soil was frozen, which would have further exacerbated the difficulties involved in performing work by hand.

The communications between the designer and resident engineer, and the construction photographs are also consistent with the available boring logs and field observations by the IFT and others. Again, this indicates that, contrary to the requirements of the 1924 construction specification, there was apparently little or no compaction of the embankment materials performed, which resulted in most of the embankment material being placed in a loose state. As noted above, it is also apparent that the fill materials used varied along the alignment with indications that upstream and downstream zoning may have been implemented in the embankment south and east of the powerhouse (the Edenville left embankment), while the embankments north and west of the powerhouse (the Edenville right embankment and the Tobacco embankments) were predominantly sand with some clay lower in the embankment and a partial upstream clay blanket at higher elevations of the embankment. In addition, as noted above, while the construction contract documents specified nominal design slopes of 2H:1V on the downstream side and 2.5H:1V on the upstream side, surveys indicate that the actual slopes were steeper than these design slopes in some locations. In the failure section, the upper part of the downstream slope was about 1.6H:1V and the average slope from crest to toe was about 1.8H:1V.

These apparent major deviations from the original design plans and construction specifications may have possibly been due to limited availability of suitable construction equipment, limited availability of suitable materials in borrow sources, insufficient staffing for construction and construction inspection, high costs or schedule pressures, and/or a desire to complete the construction ahead of schedule in order to start generating power and revenue sooner. Regardless of the reason(s) for the apparent deviations from the construction plans and specifications, the end result was an as-built dam that was below average in its safety margins compared to current industry practices, and even compared to other dams built in Michigan during the 1910s through 1930s, based on information provided by FERC and EGLE.

7.1.3 FERC Licensing

Original Project Licensing (1976 to 2006)

As noted in Section 2.1 and elsewhere in this report, the Tittabawassee River was declared a navigable waterway in 1976, requiring the four hydro projects to be federally licensed by FERC. After years of studies, the first license was granted for Sanford Dam in 1987 (FERC 1987), and in 1998 individual licenses were granted for the other three dams (Secord, Smallwood, and Edenville) and the license for Sanford Dam was renewed (FERC 1998b, 1998c, 1998d, 1998e). The license term for each dam was 30 years and was thus set to expire in 2028. The original FERC licenses were granted to Wolverine and were subsequently transferred to Synex Michigan in 2003 and then to Boyce Hydro in 2006.

Prior to 2006, during the period of ownership by Wolverine and then Synex, a series of communications between the owner and FERC indicate that it was recognized by both parties that the spillway capacity at Edenville Dam was deficient relative to the FERC PMF requirement. In 2005, FERC required Synex to convene a Board of Consultants (BOC) to oversee investigation, design, and construction of the auxiliary spillways proposed for the Edenville project. The same BOC was retained after the later transfer of ownership to Boyce Hydro. Written coordination between the owners and FERC indicates that there were several meetings conducted to discuss the spillway capacity deficiency, and Wolverine and Synex performed studies to determine the magnitude of deficiency and develop options for increasing the spillway capacity (studies of options to increase the spillway capacity, which were performed by Boyce Hydro, are described in Section 7.1.6). FERC issued noncompliance letters to both Wolverine and Synex, which were primarily related to the spillway capacity deficiency; however, FERC did not order cessation of power generation, lake level restrictions, or license revocation, nor did FERC threaten to do so.

Possible License Surrender (2013)

Between May and September 2013, Boyce Hydro submitted an Outline of License Surrender (Boyce Hydro 2013) to FERC due to claimed lack of available funds for spillway improvements to meet the PMF spillway capacity requirement. In the outline, Boyce referenced discussions with FERC to lower Wixom Lake by opening the gates and allowing run-of-the-river flow through the spillways, thus lowering the lake level by about 6 to 8 feet to slightly above the level of the spillway crests, to provide a safety buffer for passing floods. Another option considered for interim risk reduction was to reactivate the original sluiceway butterfly valves, although it was noted that this could require months of work.

After discussions with FERC, Boyce Hydro withdrew the license surrender request and FERC continued working with Boyce Hydro and its BOC on alternative approaches for increasing spillway capacity.

Compliance Order (2017)

On June 15, 2017, FERC issued a Compliance Order (FERC 2017a) citing noncompliance for alleged: (1) failure to increase the capacity of the spillways to enable them to pass the PMF; (2) performing unauthorized dam repairs; (3) performing unauthorized earthmoving activities; (4) failure to file an adequate Public Safety Plan; (5) unduly restricting public access to project facilities and failure to construct approved recreation facilities; (6) failure to acquire and document all necessary project property rights; and (7) failure to comply with FERC's 1999 Order approving Boyce Hydro's Water Quality Monitoring Plan.

The compliance order stated that "the Commission's primary concern had been the licensee's longstanding failure to address the project's inadequate spillway capacity" and required Boyce Hydro to provide plans, specifications, reports, and other information.

Order to Cease Generation (2017 and 2018)

On November 20, 2017, FERC issued an Order to Cease Generation (FERC 2017c) for alleged failure to comply with the preceding compliance order, in addition to failure to comply with additional directives.

Boyce Hydro ceased generation and on December 1, 2017, (FERC 2017d) filed an emergency motion to stay the order. Then, on December 20, 2017, Boyce Hydro filed a request for rehearing (FERC 2017e), and on December 28, 2017, filed a supplement to its motion to stay and request for rehearing (FERC 2017f).

On January 5, 2018, FERC issued an Order on Stay (FERC 2018a), temporarily staying the cease generation order until March 1, 2018, based on a request from Boyce Hydro asserting that its inability to generate power, and therefore to release water through the turbines in the powerhouse, would impact project and personnel safety during extremely cold winter weather, during which times the project's spillway gates were prone to freeze in place and the available methods to keep the gates operable were considered to be hazardous to operators. Boyce also asserted that, without allowing water to flow through the turbines during these extreme cold weather periods, there was no safe method for controlling reservoir levels.

On January 19, 2018, FERC granted a rehearing (appeal) of the order to cease generation (FERC 2018b). On February 2, 2018, Boyce Hydro filed a request for rehearing (FERC 2018c) on the January 5, 2018 Order on Stay, and on February 7, 2018, the D.C. Circuit granted Boyce Hydro's motion for a stay, in part, and stayed the portion of the Order to Cease Generation that required Boyce Hydro to cease generation.

Edenville License Revocation and Appeals (2018 and 2019)

On February 15, 2018, FERC reversed its position and issued an Order Denying Rehearing (FERC 2018d) of the order to cease generation, and also separately issued an Order Proposing Revocation of License (FERC 2018e) for the Edenville project. Following its issuance, motions to intervene were filed by multiple stakeholders, which were subsequently granted in April.

Based on interviews conducted by the IFT, lake associations in the area expressed concerns that the revocation would compromise recreational opportunities, and the Wixom Lake Association and the Sanford Lake Association requested that FERC delay its decision on the license revocation until November 2018 so that the lake associations could investigate the possibility of transitioning the project ownership to a public or nonprofit corporation. On March 3, 2018, the Sanford Lake Preservation Association (SLPA), Sanford Lake Association, and Wixom Lake Association (WLA) met with EGLE, DNR, and staff members of elected officials to discuss the situation. This coordination eventually led to the formation of the FLTF in July 2018, as discussed in Section 7.1.1.

Boyce Hydro issued a Motion of Boyce Hydro Power, LLC for Withdrawal of Order Proposing Revocation of License (Boyce Hydro 2018h), dated March 16, 2018, requesting that FERC withdraw the revocation order, citing three justifications: (1) it had made progress in funding the construction of the Tobacco River auxiliary spillway; (2) it had made all reasonable efforts to comply with the 2017 compliance order's requirements; and (3) revoking the license would be self-defeating in that it would remove the funding mechanism and therefore the potential for constructing spillway upgrades.

Nevertheless, on September 10, 2018, FERC issued an Order Revoking License (FERC 2018f) for the Edenville project, to become effective September 25, 2018, citing alleged (a) failure to increase the project's spillway capacity and (b) failure to comply with other license conditions identified in the 2017 compliance order. The revocation order required Boyce Hydro to cease generation at the Edenville project. EGLE was notified of the license revocation and on September 25, 2018, became the new regulator by default under the Part 315 dam safety statute of the Michigan Natural Resources and Environmental Protection Act.

Based on interviews conducted by the IFT, Boyce Hydro contacted EGLE on September 13 and 14, 2018, urging a site meeting the following week, and EGLE responded by indicating that EGLE would not have regulatory oversight of the dam until September 25. Boyce Hydro expressed concerns with termination of flow through the powerhouse and deterioration of the gated spillways at the dam, and indicated that their engineering consultant had advised against passing flow continuously through the gated spillways (as opposed to regulating non-flood flows through the powerhouse). Boyce Hydro requested that EGLE perform an assessment of the condition of the spillways during the next week and report its findings back to FERC. EGLE responded to Boyce Hydro by requesting the latest dam safety inspection reports and photos of the areas of the spillway that caused concern. On September 28, 2018, EGLE issued a FERC Critical Energy/Electric Infrastructure Information (CEII) Freedom of Information Act request for several technical documents related to the Edenville project, in an attempt to understand the history and deficiencies of the dam while it was under FERC regulation. Representatives of EGLE met on-site with Boyce Hydro personnel on October 4, 2018, and subsequent to the meeting, EGLE concluded that they did not observe any critical deficiencies with the spillway structures that would indicate they were not structurally sound enough to pass flow.

On October 18, 2018, FERC issued an order denying stay (appeal) of the Edenville license revocation (FERC 2018g). According to information provided to the IFT, on November 1, 2018, while FLTF was in the process of arranging to purchase the dams, a Michigan Congressman and a member of FLTF took part in a call with FERC, requesting support of additional time to manage the ownership transition from Boyce.

FLTF subsequently filed to obtain a FERC preliminary hydro license to generate hydroelectric power from the Edenville project in June 2019. On June 20, 2019, FERC issued a final denial of rehearing of the Edenville license revocation (FERC 2019).

The Secord Dam, Smallwood Dam, and Sanford Dam licenses remained active, with Boyce Hydro as the licensee, until May 27, 2021.

7.1.4 Edenville Dam Inspections and Dam Safety Reviews

This section describes the regular dam inspections and FERC-mandated Part 12D dam safety reviews that were performed for Edenville Dam, starting around 1987 when the FERC license for Sanford Dam was issued. The IFT was not able to find significant documentation related to inspection and dam safety review activities performed for Edenville Dam prior to 1987. This section also summarizes inspections and studies performed by EGLE, FLTF, and FLTF consultants for Edenville Dam prior to the dam failure.

7.1.4.1 FERC Dam Safety Inspections

FERC first performed a dam safety inspection (DSI) of the Edenville project in September 1986 prior to the license application, which occurred soon after in November 1986. Subsequent to the initial application, FERC inspections performed prior to licensure occurred in 1987, 1989, 1991, 1993, 1995, and 1997, and in August 1998; the license was issued in October 1998. Once licensed, regular inspections of the project were performed at least annually by FERC. Additional special inspections were performed to observe construction activities or operations under certain conditions. This section summarizes the general observations and conclusions regarding the physical condition of the embankments as documented in the FERC DSI reports. Spillway gate testing and spillway capacity are discussed in Section 2.5 and Appendix C.

The inspection reports for the Edenville and Tobacco embankment sections prior to 1995 were not reviewed; however, project documents reference a wet and soft area noted by FERC during the 1987 inspection, which prompted a recommendation to perform a stability analysis for this section. Observations documented in the FERC DSI reports are generally consistent from 1995 through 2002. The embankment sections were judged to be in satisfactory to excellent condition. Specific observations regarding the physical condition of the embankments noted from 1995 through 2002 are summarized below.

- The 1997 FERC DSI included a statement that lateral drains were placed about 2 feet apart in the Tobacco left embankment “since a wet area developed in the slope” (it is unclear whether they were referring to a wet area observed during original construction). This is the only inspection report from 1995 through 2002 that mentions indications of wet or previously wet areas on the downstream slope of any of the Edenville Dam embankments.

- The 2000 FERC DSI noted that some of the lateral drains (unspecified location) were not producing flow. Otherwise, the lateral drains during inspections in this time frame were noted to be functioning. The lateral drain discharge was noted to be clear. No sediment was observed in the collection ditch for the lateral drains or in the series of seven weirs.
- On May 9, 2003, a special inspection was performed by FERC due to small sinkholes and a minor slough that occurred in the Edenville right embankment immediately adjacent to the right powerhouse wall. Soil fines were observed exiting from the drains and two sinkholes had developed. A similar condition was noted at the Tobacco left embankment, resulting in a small sinkhole on the mid-slope of the downstream embankment face near the spillway wall. By July 2003, the two areas of sinkholes were reported to be dry.

Between 2004 and 2018, the embankments were judged by FERC to be in good condition overall. Very little seepage was observed at the Edenville left embankment section in comparison to the condition of the other embankments: “the lack of seepage was obvious from the health and type of vegetation on the downstream slope as well as the lack of flow in the lateral drainage ditch” (2006-2018 DSI reports). The left groin of the Edenville left embankment was suspected to be a potential seepage area due to vegetation overgrowth between 2013 and 2016. In 2016, the area was mowed and found to be dry.

Figure B-5 in Appendix B shows key observations as documented in the DSI reports.

7.1.4.2 Independent Consultant (Part 12D) Dam Safety Reviews and PFMAs

Part 12D of FERC’s regulations requires an Independent Consultant (IC) to perform a dam safety review of all water-retaining features on a five-year frequency. This FERC-mandated requirement applies to any constructed project that FERC determines to be jurisdictional, regardless of whether a license has yet been issued. Six Part 12D dam safety reviews were completed for Edenville Dam in 1991, 1994, 2000, 2005, 2010, and 2015, the first two of which were completed prior to project licensure.

The scope of the Part 12D dam safety reviews consisted of review of available design and construction documents to gain an understanding of the structures; a site inspection to evaluate the physical condition of the facility; a review of investigations and analyses (and in some cases performance of analyses), including stability of the embankments and concrete structures and PMF and spillway capacity analyses; a review of surveillance and instrumentation monitoring data; and a review of operations.

Starting with the 2005 Part 12D, the scope of the dam safety reviews was expanded to include performing a PFMA to hypothesize and evaluate the potential mechanisms that could lead to uncontrolled release of reservoir water by failure of any of the water-retaining structures (e.g., embankments, concrete structures, spillway gates).

Below is a summary of observations and findings related primarily to the condition of the embankment sections as presented in the IC’s CSIRs. Text in italics in this section indicates inferences by the IFT. Figure B-5 in Appendix B summarizes the locations of the key observations from the field inspections.

Reviews and recommendations related to geotechnical evaluations and hydrologic/hydraulic evaluations performed in conjunction with, or in response to, Part 12D reviews are described in Sections 7.1.5 and 7.1.6, respectively, and are not repeated here in Section 7.1.4.

Findings and recommendations related to the physical inspections and structural evaluations of the spillway and powerhouse structures performed in conjunction with, or in response to, the Part 12D reviews are not described in this forensic report because the structural integrity of these features was not found to significantly influence the failure of Edenville Dam.

1991 Part 12D – A. R. Blystra & Associates (Blystra 1991)

Based on samples collected in the downstream slope, the embankments were described as consisting of poorly graded sand and constructed in compacted lifts based on available specifications. An electrical resistivity survey was completed during the field inspection, and it indicated "that the embankments appear to be quite uniform." Specific locations of the resistivity survey soundings were not reported. A crest survey was performed as part of the field investigation and it identified several areas below the design crest elevation. Two areas of seepage were noted, one along the downstream toe of the Edenville left embankment near the spillway and the other near the downstream right training wall of the powerhouse. Seepage was described as "very minor" and clear. A small deposit of sand was observed at the end of a lateral drain located about 300 feet left of the Edenville spillway structure.

Active embankment instrumentation data consisted of seven seepage weirs (four downstream of the Tobacco embankments and three downstream of the Edenville right embankments), 14 piezometers within the Tobacco right embankment, and two piezometers installed in 1991 in the Edenville right embankment, the latter of which had no data yet reported. Seepage weir data had only been collected for about a year and were not plotted; however, it was stated that the seepage quantity correlated with reservoir (lake) level. It was stated that no trends regarding the piezometers could be drawn because the reservoir level was not always recorded at the same time as piezometer readings.

The IC recommended repairs to surface erosion areas observed at the embankment section located to the left of the Tobacco spillway structure. The IC also recommended verifying in field the upstream embankment slope and updating analyses as necessary. The reservoir drawdown rate was recommended to be limited to 1 inch per hour until stability analyses were updated. Low areas identified as part of the crest survey were recommended to be raised to the design crest elevation.

Slope stability analysis was performed, which is described below in Section 7.1.5.

1994 Part 12D -- Mead & Hunt, Inc (Mead & Hunt 1994c, 1995a)

Similar to the 1991 Part12D, the embankments were described as consisting of poorly graded sand, based on samples collected in the downstream slope, and as constructed in compacted lifts based on available specifications. A crest survey was performed as part of the inspection, and the results indicated that the embankment crest had not settled and that low spots documented in the 1991 Part 12D crest had apparently been backfilled. The upstream slope at the left abutment of the Edenville left embankment showed signs of sloughing into the reservoir. The toe drains and collection systems were noted to be operating properly. Seepage collected (*understood to be within the collection ditch)* from the Edenville right embankment section between the powerhouse and the Edenville office contained a "moderate" amount of fine sands. An animal burrow was observed within the Edenville right embankment.

Active embankment instrumentation consisted of 14 embankment piezometers (12 within the Tobacco right embankment and two in the Edenville right embankment) and seven seepage weirs (four downstream of the Tobacco embankment and three downstream of the Edenville embankments). In addition, seven drain outfalls in the Tobacco embankment were manually measured with a bottle and stopwatch. It was stated that no increasing trends in data were observed for the piezometers or the weirs. Considerable fluctuation in the amount of seepage measured at the weirs and horizontal drains was noted, but was judged not to be excessive. *From the IFT's review of the instrumentation data, both piezometer and seepage data appear to fluctuate with reservoir level, with some instrument response more pronounced than others.* Survey movement monuments were installed, as those previously installed in 1991 could not be located.

The IC recommended constructing sediment traps at each of the toe drains to determine the origination of the sands and to monitor the rate of movement, repairing the slough on left upstream slope, and repairing rodent burrows as a part of routine maintenance. The IC also recommended initiating a boring program to verify embankment soil strength parameters used in analysis (see Section 7.1.5). It does not appear that the recommended boring program or sediments traps were implemented.

2000 Part 12D -- Mead & Hunt, Inc (Mead & Hunt 2000)

Similar to the 1991 Part12D, the embankments were described as consisting of poorly graded sand, based on samples collected in the downstream slope, and as constructed in compacted lifts based on available specifications.

Sediment deposits were observed in several of the lateral drains of the Tobacco embankments. At other locations, sediment deposits were observed in the collector ditch at the discharge location of the lateral drain. A "soft muddy" area was observed at the toe of the Edenville left embankment about 50 feet left of the Edenville spillway. The soft, muddy area was not observed during the previous Part 12D inspections. No flowing water was observed; however, fine clay-like deposits were noted in the area. It was postulated that the fine clay-like deposits could be fine-grained material migrating out of the embankment.

Embankment instrumentation consisted of 14 embankment piezometers (12 within the Tobacco right embankment and two in the Edenville right embankment), eight seepage weirs (four downstream of Tobacco embankment and four downstream of Edenville embankments), and crest movement monuments established in 1994. In addition, seven drain outfalls in the Tobacco embankment were manually measured with a bottle and stopwatch. It was stated that piezometer and seepage measurements fluctuated seasonally with lake level and no increasing or decreasing trends were observed. Crest surveys were stated to be consistent with the previous measurements in 1994.

Settlement observed along the right downstream Tobacco spillway wall was recommended to be filled to restore the slope of the embankment. Voids observed in the riprap on the upstream slope were recommended to be repaired. Cleaning of the lateral drains exiting the Tobacco embankments, which were observed to have sediment, was recommended, and installation of additional weirs was recommended to facilitate monitoring seepage and transport of fines. The IC also recommended extending the seepage collection ditch located along the toe of the Edenville left embankment to the tailrace and installing additional weirs to facilitate the monitoring of seepage and transport of fines.

There was no mention of the boring program recommended in the 1994 Part 12D CSIR nor the issues raised by FERC in response to the 1991 Part 12D CSIR related to confirming material properties along the length of the embankment, as discussed in Section 7.1.5.

2005 Part 12D -- Mead & Hunt, Inc (Mead & Hunt 2005a)

The embankments were described as consisting of poorly graded sand, but reference to compacted lifts was not included in the description. The lack of compaction was listed as a Major Finding and Understanding of the 2005 PFMA and appeared to result from the associated more detailed review of the construction photos.

Edenville Embankment Observations – Animal burrows were observed along the upstream slope of both the left and right embankments. Two relatively large areas of standing water were observed near the downstream toe of the right embankment between the bend in the embankment alignment and the Edenville office. A soft spot at the downstream toe was observed approximately 400 feet left of the Edenville spillway structure. The weir that collects seepage from the Edenville left embankment was dry because the seepage emanating from the lateral drains and into the collection ditch appeared to be bypassing the weir. The sheet pile wall along the left downstream spillway apron was observed to be leaning outward, and a gap between the wall and soil was noted. On both sides of the Edenville spillway structure, significant gaps were noted between the upstream concrete walls and the sheet pile cutoff walls. It appeared timbers had been used to fill the gap between the sheet pile cutoff wall and the spillway wall. The rotting timbers were leading to the formation of the gap, and seepage appeared to be entering the embankments at these locations. This seepage source and gap created by the deteriorating timbers was listed as a Major Finding and Understanding of the 2005 initial PFMA.

Tobacco Embankment Observations – Significant erosion and rutting of the embankment fill was observed behind the downstream abutment walls on both the left and right side of the Tobacco spillway structure. Significant slumping and erosion were noted on the downstream slope between the left abutment wall and the area of slope repair (*interpreted as the slope repair from 2003 sinkhole and slumping*). The inclined concrete scour protection slab on the upstream slope of the Tobacco right embankment was undermined, causing portions of the slab to crack and collapse. A relatively large wet area was noted at the toe of the Tobacco right embankment adjacent to the Tobacco spillway structure.

Embankment instrumentation consisted of 14 embankment piezometers (12 within the Tobacco right embankment and two in the Edenville right embankment), eight seepage weirs (four downstream of the Tobacco embankment and four downstream of the Edenville embankments), and crest movement monuments. Outflow from the seven drain outfalls in the Tobacco embankment were no longer measured, following the installation of filter berms along the downstream toe. There was no discussion regarding the instrumentation data trends.

The IC recommended that animal burrows, areas of sparse riprap, the gaps between the upstream concrete abutment walls and the upstream sheet pile cutoff walls, and the voids behind the upstream cutoff wall be repaired. In addition, monitoring of the soft spot at the toe of the Edenville left embankment, the areas of standing water at the toe of the Edenville right embankment, and the sheet pile leaning outward to the left and downstream of the Edenville spillway structure was recommended. The IC also recommended repair of the surface erosion and rutting of the Tobacco embankment and the upstream concrete scour protection slab. Staking and monitoring of the wet area at the downstream toe of the Tobacco right embankment adjacent to the spillway structure was recommended, and a dive inspection of the powerhouse turbine discharge bay was recommended for inspection for erosion. These recommendations were subsequently reported to have been completed according to the 2010 Part 12D CSIR.

2010 Part 12D – Mill Road Engineering (Mill Road 2011b)

The embankments were described as consisting of poorly graded sand.

Edenville Embankment Observations – The Edenville right embankment was dry, with no indications of seepage. Animal burrows were observed along the right embankment. The Edenville left embankment was found to be in good condition. The left abutment contact at the location of previously reported seepage was inspected, and no signs of seepage were noted. The Edenville left embankment downstream toe and collection ditch were inspected, and it was found that the outlets of some of the lateral drains were submerged below standing water in the ditch.

Tobacco Embankment Observations –A sinkhole was observed behind the right training wall of the spillway near the downstream toe. It appeared that the material eroded through an open joint in the wall. Some areas of woody vegetation were observed to be growing on the upstream slope. No signs of sediment transport, piping, springs, or seeps were observed during the inspection.

Embankment instrumentation consisted of 16 embankment piezometers (12 within the Tobacco right embankment, two within the Tobacco left embankment, and two in the Edenville right embankment), eight seepage weirs (three downstream of the Tobacco embankments and five downstream of the Edenville embankments), and crest movement monuments. According to the IC, the piezometer and weir data showed seasonal variation within historical ranges and did not depict dam safety concerns. No specific discussion related to magnitude or time frame of instrument response to reservoir level was noted.

Recommendations related to the embankments included lowering the collection ditch invert along the Edenville left embankment to allow free discharge from the lateral drains, which would facilitate inspection for sediment transport through the drains. A crest survey was recommended to identify low areas along the embankments. Removal of the woody vegetation observed on the upstream slope and repair of animal burrows were recommended. Completion of the Tobacco reverse filter toe drain berm was recommended so that the toe drain program at the project would be completed. These embankment recommendations were subsequently reported to have been completed according to the 2015 Part 12D CSIR.

2015 Part 12D -- Purkeypile Consulting, LLC (Purkeypile 2016a)

The embankments were described as consisting of poorly graded sand. At the time of the 2015 Part 12D review, an area for a proposed auxiliary spillway from Station (Sta.) 51+00 to Sta. 54+00 had been cleared and grading had started.

In general, the embankments were described as being in good to excellent condition. No slumps, slides, cracks, animal burrows, wet or soft areas, or other unusual conditions were noted. Some “oozing” seepage was noted along the Edenville left downstream abutment groin area near an old concrete structure (the old pump house structure). The drainage ditch downstream of the Edenville left embankment was noted to be functioning properly, with the drain pipes freely discharging into the collector trench. No sand or fines were noted in the drainage ditch or discharging from the drain pipes. Only minor, clear seepage was observed to be emanating from the drain pipes of the Edenville left embankment.

Embankment instrumentation consisted of 18 embankment piezometers (14 within the Tobacco right embankment, two within the Tobacco left embankment, and two in the Edenville right embankment), six seepage weirs and two pipes (three weirs and the two measured pipes are downstream of the Edenville embankments, and three seepage weirs are downstream of the Tobacco embankments), and crest movement monuments. There was no discussion regarding the instrumentation data trends.

Recommendations related to the embankments included replacing the open seepage weirs with closed-boxed weirs for improved monitoring of sediment and armoring the downstream left bank adjacent to the Edenville spillway. The IC also recommended installation of a “weighted filter berm” from Sta. 29+00 to Sta. 35+00 (Edenville right embankment) and from Sta. 51+00 to Sta. 60+00 (Tobacco right embankment); the latter segment could be reduced to Sta. 54+00 to Sta. 60+00 if the auxiliary spillway were to be constructed where planned between Sta. 51+00 and Sta. 54+00. It appears this recommendation was a result of the 2015 PFMA review, which stated that these were the two reaches of the embankment that did not have a new drain filter. It is unclear why the Edenville left embankment was not included in this recommendation. The IC recommended regrading the crest to a uniform design elevation, but that this could be delayed and completed in conjunction with future spillway upgrades. These recommendations had not been completed by the time of the May 2020 failure.

Potential Failure Mode Analyses (PFMAs)

The initial FERC-mandated PFMA was performed in conjunction with the 2005 Part 12D review. The initial PFMA was performed jointly by the IC and representatives from FERC and Synex Wolverine, with a total of four individuals on the core team (two from FERC, the IC, and one owner's representative) and three individuals who were observers (one from FERC, one consultant who was the note-taker, and one plant operator). On the core team, the facilitator was an experienced FERC civil engineer with a geotechnical specialization, the other FERC representative was a junior civil engineer, the IC was an experienced civil engineer with a structural specialization, and the owner's representative was a civil engineer with experience in the hydropower industry. The IFT did not find any information indicating that anyone on the core team or among the observers was an experienced engineer with a hydrology and hydraulics specialization. In interviews with the IFT, the core team members had mixed opinions regarding whether they had sufficient time to review documents and complete the PFMA, and they also had mixed opinions regarding whether the team had sufficiently probing discussion and debate to thoroughly and critically evaluate PFMs. In general, the atmosphere among the PFMA group was described as "polite," with everyone treated as an equal and no one being discouraged to participate in discussions.

In accordance with FERC guidelines (FERC 2003/2017b), the group brainstormed ways in which the dam could fail and categorized the identified potential failure modes (PFMs) into one of the four categories described in the FERC guidelines:

- Category I (Highlighted) - Those potential failure modes of greatest significance considering need for awareness, potential for occurrence, magnitude of consequence and likelihood of adverse response (physical possibility is evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.
- Category II (Considered but not Highlighted) - These are judged to be of lesser significance and likelihood. Note that even though these potential failure modes are considered less significant than Category I they are all also described and included with reasons for and against the occurrence of the potential failure mode. The reason for the lesser significance is noted and summarized in the documentation report or notes.
- Category III (More Information or Analysis Needed) - More Information or Analyses are needed in order to classify these potential failure modes; to some degree lacked information to allow a confident judgment of significance and thus a dam safety investigative action or analyses can be recommended. Because action is required before resolution the need for this action may also be highlighted.
- Category IV (Ruled Out) - Potential failure modes may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the potential failure mode, or the potential failure mode is clearly so remote a possibility as to be non-credible or not reasonable to postulate.

It should be noted that the definitions for these four categories have some inconsistencies and ambiguities which can make it difficult to categorize a PFM:

- In comparing Categories I and II, the boundary between "greatest significance" and "lesser significance" is unclear.
- Categories I and II involve judgment of both "significance" and "likelihood," rather than only considering likelihood. This implies that risk is being considered, however, in practice, the PFMA is not explicitly a risk analysis framework with which consequences of dam failure are evaluated in detail.
- With Category IV, the threshold for judging a PFM to be "so remote a possibility as to be non-credible," "not reasonable to postulate," or such that "the physical possibility does not exist" should, in principle, be so high that a failure mode should rarely be placed in Category IV. Moreover, there is an inconsistency between a PFM being postulated and evaluated, and then concluding that a PFM is "not reasonable to postulate." However, in practice, it has not been uncommon to categorize PFMs in Category IV; this may reflect overconfidence bias on the part of groups participating in PFMAs.
- Describing Category IV as "ruled out" can lead to the categorization of a PFM never being reevaluated in future PFM reviews, even if the condition of the dam has changed, new information becomes available, or a new PFMA team would have a different expertise set which could result in a different evaluation and categorization of a PFM.

For Edenville Dam, during the initial PFMA in 2005, fifteen PFMs were identified and categorized, as summarized in Table 7-1. Three of the 15 PFMs were categorized as Category I, seven as Category II, two as Category III, and three as Category IV.

The three PFMs that were categorized as Category I (Highlighted) were PFM 2 (embankment overtopping during a flood), PFM 4 (internal erosion into drain pipes or along foundation), and PFM 9 (structural failure of the concrete spillway). Recommendations found in the 2005 Part 12D CSIR related to reducing the risk of these highlighted PFMs consisted of recommended repairs of deteriorated concrete and recommendations related to monitoring of seepage and piezometers.

For PFM 2, the spillway capacity was described as being sufficient to "safely pass a flood event roughly equal to a 200-year flood." This finding was categorized as making overtopping "less likely" rather than "more likely," and therefore this spillway capacity apparently was perceived by the PFMA group as reflecting a low overtopping risk, even though a 200-year flood would have about a 5 percent probability of occurring in a 10-year period and about a 10 percent probability of occurring in a 20-year period. The judgment of overtopping risk may have been different if the PFMA team had included a hydrology and hydraulics expert. Despite this perception of a low overtopping risk, because the spillways could not pass the PMF flood, this PFM was categorized in Category I. It was noted that the licensee was currently working with FERC to address spillway capacity concerns, though there was no specific recommendation related to this PFM in the 2005 Part 12D CSIR.

PFM 15 considered embankment instability due to a lake level "higher than the historical levels," which approximately describes the failure mode which occurred on May 19, 2020, though it does not specifically describe a static liquefaction failure mode and phreatic levels were not necessarily significantly increased on May 19. This PFM was assigned to Category IV based on stability analyses previously performed by others, that indicated adequate factors of safety under a high lake level. This is understood by the IFT to refer to the 1991 embankment stability analyses included in the 2005 STID, which considered a lake level up to El. 682 (minimum embankment crest) and a homogenous embankment with a drained strength of 34.5 degrees friction angle. That analysis indicated a factor of safety of 1.8 (see Section 7.1.5 and 7.2 for more discussion of stability analyses). The IFT did not find any evidence that the PFMA team attempted to scrutinize the assumptions and methodology of the 1991 embankment stability analyses, nor whether these analyses were sufficient in scope to represent the entire length of the Edenville Dam embankments. By placing this PFM in Category IV, it was indeed "ruled out" in the sense that it was never re-evaluated during the future PFMA reviews in 2010 and 2015 – once it was "off the radar," it stayed off the radar (see Table 7-1).

As a result of the initial 2005 PFMA, presumably due to a more thorough review of the construction photos, it was realized that the embankments were placed by dumped-fill techniques and not compacted in lifts as indicated by the available construction specifications and as believed during previous Part 12D dam safety reviews. Based on this finding, PFM 5 related to seismic-induced liquefaction was assigned to Category III and a liquefaction analysis of the embankments was recommended, which was later completed in 2010 (see Section 7.1.5). However, the possibility that the 1991 embankment stability analyses may be based on unrepresentative assumptions regarding the soil density and strength, and may not represent the critical embankment cross section, was apparently not considered when selecting a Category IV ("Ruled out") categorization of PFM 15 – instability under high lake levels.

In addition to PFM 15 approximately describing the failure mode which occurred, it is noteworthy that the mechanism of the actual failure mode also potentially involved some elements of three other PFMs:

- Although the dam was not overtopped, not fully opening the spillway gates did increase the lake level by about 1 foot (PFM 3) (see Section 5.2.5).
- It is possible that ineffective functioning (or absence in one location) of toe drains contributed to an increased phreatic surface and/or increased pore water pressures, which in turn could have contributed to the instability failure of the downstream slope (PFM 10) (see Section 4.1.3).
- Significant waves combined with high pool levels caused erosion of the upstream slope on May 19 (PFM 11). While this did not directly lead to failure of the embankment as a result of the reservoir washing over the embankment crest, it is possible that the erosion affected seepage and pore water pressures in the embankment, and thereby contributed to triggering static liquefaction.

The fact that the mechanism of the actual failure mode may have involved a blend of elements from several identified PFMs, as well as elements which were not described in any of the identified PFMs (e.g., static liquefaction), illustrates the challenges involved in postulating a set of PFMs which is sufficient to reasonably capture all the ways in which a dam might fail.

A review of PFMs was conducted as part of each subsequent five-year Part 12D dam safety review (2010 and 2015). It is unclear whether the 2010 PFMA review session was completed jointly with FERC and the licensee. However, the 2010 IC provided comments for each of the credible 2005 PFMs regarding completed or planned work and made recommendations for revised categorizations in some instances. Boyce Hydro submitted an updated STID with revised PFM categorizations in September 2011. Table 7-1 provides a summary of the revised categorizations and the supporting comments from the 2010 IC for each credible PFM. Recommendations in the 2010 Part 12D CSIR that related to the PFMA findings included completing the Tobacco reverse filter toe drain berm, completing various repairs to the concrete structures, and completing the plan for a new spillway to increase capacity and improve gate operations.

Another PFMA review session was conducted in a joint session with the IC and representatives from FERC and Boyce Hydro as part of the 2015 Part 12D. The team appeared to have reviewed the PFMs as documented in the 2005 PFMA report and to have recommended revised categorizations, which in some cases were different from the updated categorizations documented in the 2011 STID. Table 7-1 provides a summary of the recommended revised categorizations and the supporting comments for each credible PFM. An update to the STID formalizing the PFM recategorizations was not completed prior to the license revocation in 2018. Recommendations in the 2015 Part 12D CSIR that related to the PFMA findings included completing a spillway rehabilitation program to increase spillway capacity, completing a weighted filter along sections of the Edenville right embankment (Sta. 29+00 to Sta. 35+00) and Tobacco right embankment (Sta. 51+00 to Sta. 60+00), raising low areas of the dam embankment crest, and performing miscellaneous concrete repairs.

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Table 7-1: Evolution of Potential Failure Modes from 2005 PFMA to 2015 PFMA Review

PFM #	2005 PFM Description (Mead & Hunt 2005)	2005 Cat. (Note 1)	2010 PFM Notes *Italicized text are summaries of IC comments related to the PFM categorization*	2011 Cat. (Note 2)	2015 PFM Notes (Purkeypile 2016b,c) (Note 3) *Italicized text are summaries of IC comments related to the PFM categorization*	2015 Cat.
1	The timber piles that provide a seal between the steel sheetpiling and the concrete spillway walls deteriorate leading to a rise in the phreatic surface within the embankment. Elevated phreatic surfaces result in a sloughing failure on the downstream slope, loss of embankment material, and the eventual failure of the sheetpile cutoff wall resulting in an uncontrolled release of the reservoir.	II	*Recommended to reclassify as IV based on a concrete grouting program of the area performed in 2007 judged to have eliminated the risk of this PFM.*	IV	*Grouting work was judged by the IC to have satisfactorily addressed this PFM; however, at the recommendation of FERC this PFM remained classified as Category II.*	II
2	The embankment is overtopped during an extreme flood event. This could result in significant erosion of the downstream slope leading to failure of the embankment and an uncontrolled release of the reservoir.	I	*It was noted by the IC that a plan for a new spillway is in progress, and once completed the PFM would be reclassified as IV.*	I	*Defined the threshold for this PFM as an event that exceeds approximately 28,500 cfs discharge (includes 2,000 cfs turbine flow), which is a flood event that is approximately equivalent to 44% of the PMF.* *It was noted by the IC that a plan for a new spillway, including larger automated spillway gates and an auxiliary spillway, is in progress, and once completed the PFM would be reclassified as IV.*	I
3	Failure to operate the spillway gates properly results in overtopping of the embankment. This could result in significant erosion of the downstream slope leading to failure of the embankment and an uncontrolled release of the reservoir.	II	*It was noted that the plan for a new spillway includes remotely operated gates and individual hoists, and once completed, would address this PFM.*	II	*It was noted that the plan for a new spillway includes remotely operated gates and individual hoists once completed, would address this PFM.*	II
4	Seepage through the embankment enters into a tile drain pipe or seeps along the embankment's foundation resulting in piping and internal erosion. This could lead to a developing pipe or sinkholes connecting to the reservoir. Increased seepage over time could result in a blowout of the embankment and an uncontrolled release of the reservoir.	I	*A reverse filter toe drain was reported to have been installed from the powerhouse to the bend in the Edenville right embankment, from M-30 to the Tobacco spillway, and was in of being installed from the Tobacco spillway to approximately the center of the Tobacco right embankment. It was judged that completing the Tobacco* "toe would eliminate this PFM from concern" *as the remaining sections of the embankment were stated to have* "limited dam elevation for which the phreatic driving head is limiting or the dam has incorporated a portion of natural soil in which there are no toe drains and the section is no[t] subject to failure by piping."	I	*It was noted that* "weighted filter toe drain installation for all main sections of the embankment have been completed as of the date of this CSIR. The remaining sections of the dam that do not have new drains have low embankment heights with either little or no static head driving the seepage through the raised embankment sections. Their original drains appear to be working properly. Therefore, there is little risk associated with those sections of the embankment. It is recommended that PFM 4 remain classified Category I until the toe drains are completed."	I
5	An MCE seismic event occurs resulting in liquefaction of the embankment material and an uncontrolled release of the reservoir.	III	*Recommended reclassifying as IV based on 2010 seismic liquefaction analyses indicating the dam is stable during a seismic event.*	IV	*A liquefaction analysis was completed in 2010 based on soil borings. It was judged that the analysis determined the embankment is not subject liquefaction during earthquakes. It was recommended that PFM 5 be reclassified as Category IV*	IV
6	A sliding or overturning failure of one of the concrete structures results in an uncontrolled release of the reservoir.	II	*Recommended reclassifying as IV based on sliding stability analyses (2011) meeting FERC guidelines.*	IV	*Sliding stability analyses for all structures have indicated the structures have factors of safety under all studied loading conditions that exceed the minimum allowable factors of safety for sliding stability required in the FERC guidelines. It was recommended that PFM 6 be reclassified as Category IV.*	IV
7	Undermining of the spillway aprons is allowed to develop and eventually results in the formation of a pipe underneath the structure and an uncontrolled release of the reservoir.	II	*No recommendation for reclassification. Recommended continued dive inspections.*	II	*No recommendation for reclassification. Recommended continued dive inspections and additional inspections after flood events.*	II
8	During periods of high flows, tailwater levels increase and eddying currents erode the downstream toe of the embankment in the vicinity of the abutment walls. This could lead to an eventual slope failure and an uncontrolled release of the reservoir.	II	*No recommendation for reclassification. It was noted that the planned new spillway and the in-progress construction of the toe drain berms adjacent to the (Tobacco) spillway will help manage the risk of this PFM.*	II	*No recommendation for reclassification. It was noted that the planned new spillway will help manage the risk of this PFM. It was recommended that inspections and repair of the toe area be performed and that the recommended rehabilitation plans for the spillway be approved.*	II
9	Significant deterioration of the concrete that comprises the spillway ogee railway or retaining walls leads to a structural failure of the spillway, resulting in an uncontrolled release of the reservoir.	I	*No recommendation for reclassification. It was noted that the plan for a new spillway was judged to provide the concrete rehabilitation to address this PFM.*	I	*This PFM was recommended to be divided between the ogee rollway and the retaining walls. Both were recommended as Category I, with a comment that the spillway rehabilitation project would address these PFMs.* Concrete Deterioration of the Ogee Spillway: Significant deterioration of the concrete that comprises the spillway ogee rollway leads to a structural failure of the spillway resulting in an uncontrolled release of the reservoir.	I
					Concrete Deterioration of the Ogee Spillway: Significant deterioration of the retaining walls leads to a structural failure of the spillway resulting in an uncontrolled release of the reservoir.	I

PFM #	2005 PFM Description (Mead & Hunt 2005)	2005 Cat. (Note 1)	2010 PFM Notes *Italicized text are summaries of IC comments related to the PFM categorization*	2011 Cat. (Note 2)	2015 PFM Notes (Purkeypile 2016b,c) (Note 3) *Italicized text are summaries of IC comments related to the PFM categorization*	2015 Cat.
10	Clogging of toe drains leads to higher-than-normal phreatic surface levels within the embankments resulting in failure of the downstream slope and the eventual uncontrolled release of the reservoir.	II	*It was stated that the reverse filter toe drain installation has eliminated this risk and continued function of the clay tile drain system is not required. This PFM was recommended to be reclassified as Category IV once the [toe drain] work is complete.*	II	*It was stated that* "weighted filter toe drain installation has eliminated possible high phreatic water levels along the embankment toe by intercepting seepage. The massive filter provides sufficient additional collection capability to prevent clogging of the filter. The design of the reverse filter protects the dam from this failure mode. Completion of the weighted filter toe drains for the remainder of the embankment is recommended. Once this work is completed, this PFM should be reclassified to Category III."	II
11	Significant waves combined with high pool levels cause erosion of the upstream slope. This allows the reservoir to wash up and over the embankment crest, leading to a failure of the embankment and an uncontrolled release of the reservoir.	II	*It was recommended this PFM be reclassified as IV based on wave runup analyses performed in April 2011 and the presence and annual maintenance of the upstream riprap.*	IV	*No recommendation for reclassification from the 2005 Category II. However, it was noted that wind wave erosion is not a significant issue with this project, and the review of the PMF wind condition provides the understanding that wind setdown will occur at the embankment. Sufficient riprap exists along the embankment and is inspected every year.*	II
12	Failure of one of the spillway gates on either structure results in rapid drawdown of the upstream pool, leading to failure of the upstream slope of the earth embankment and an uncontrolled release of the reservoir.	IV	*No comment provided*	IV	*No comment provided*	IV
13	Erosion in the floor of the turbine discharge bay could lead to significant undermining of the powerhouse. Over time, a pipe could develop under the powerhouse connecting the upstream reservoir with tailwater. Significant flows through this pipe could result in failure of the powerhouse and an uncontrolled release of the reservoir.	III	*It was recommended this PFM be reclassified as IV based on the estimated velocity judged to be significantly below that which would cause concern for erosion.* *The updated PFM categorizations as documented in the STID (Boyce Hydro 2011) did not propose revising the categorization of this PFM.*	III	*No recommendation for reclassification from the 2005 Category III. However, it was stated that by design, the discharge velocity is reduced to the point where the exit velocity of the draft tube is significantly below the velocity where the discharge can cause erosion.*	III
14	Debris or ice buildup beneath the M-30 Bridge creates a restriction of flow between the drainage basins. This coupled with a significant flood event over one of the basins could result in flows in excess of the spillway capacity, overtopping and failure of the earth embankment, and an uncontrolled release of the reservoir.	IV	*No comment provided*	IV	*No comment provided*	IV
15	Reservoir levels reaching higher than historical levels results in increased phreatic surface levels within the earth embankments leading to a slope failure and an uncontrolled release of the reservoir.	IV	*No comment provided*	IV	*No comment provided*	IV

CAT. = Category
FERC = Federal Energy Regulatory Commission
M-30 = Michigan Highway 30
MCE = maximum considered earthquake
PFM = potential failure mode
STID = Supporting Technical Information Document
Notes:
1. In accordance with FERC guidelines (FERC 2017b) Category I = Highlighted, Category II = Considered but not Highlighted, Category III = More Information Needed, and Category IV = Ruled Out.
2. The revised final 2010 Part 12D report (Mill Road 2011b) included comments regarding suggested recategorization of PFMs. An update to the STID with revised PFM categorizations was submitted in September 2011 (Boyce Hydro 2011) concurrent with the submittal of the revised final 2010 Part 12D report.
3. The 2015 PFMA Review reviewed the PFMs as categorized and documented in the 2005 STID (Mead & Hunt 2005b).

7.1.4.3 EGLE Dam Inspections and Evaluations

Prior to the May 2020 failure, EGLE served as the dam safety regulator for Edenville Dam for a period of less than 2 years, starting on September 25, 2018. During this period, EGLE performed an inspection of the dam and questioned the hydrologic and hydraulic evaluations of the dam, as described below. It should be noted that during this period, Secord Dam, Smallwood Dam, and Sanford Dam were still regulated by FERC, with no "co-regulation" by EGLE, which resulted in a situation where there were four dams that were hydraulically linked in series, but regulated by two different dam safety regulators whose information sharing with each other was limited.

On October 4, 2018, EGLE performed a cursory inspection of the dam, which judged the dam as being in "fair" structural condition. No slope stability concerns were noted. Boyce Hydro provided EGLE with a copy of the 2015 Part 12D CSIR during this inspection. The EGLE engineers observed that Wixom Lake was drawn down approximately 4 feet below the normal lake level for that time of year and that no permits had been issued for drawdown of the impoundment. FERC had ordered Boyce Hydro to inspect tailrace/spillway aprons slabs at Smallwood Dam and Secord Dam, and to complete the inspection at Smallwood Dam, Wixom Lake needed to be drawn down. FERC's intention was that any drawdown would be temporary and normal lake levels would be restored after the tailrace/spillway apron inspections were completed.

When EGLE became the dam safety regulator for Edenville Dam, EGLE had very limited information about the dam and had difficulty in obtaining information from FERC due to CEII restrictions. Boyce Hydro filled this gap by providing information to EGLE over the next few months, and after reviewing this information, EGLE requested specific information from Boyce Hydro related to EGLE's concerns about the spillway capacity, particularly the spillway capacity rating curve developed by Boyce Hydro's consultant, Purkeypile Consulting, LLC (Purkeypile).

In January 2019, Purkeypile had prepared a Technical Memorandum (Purkeypile 2019) addressed to Boyce Hydro, but also distributed to EGLE, related to hydraulic analysis of the spillway adequacy of the Edenville Dam, in view of the recent license revocation and the shift from FERC to EGLE regulation. The memo concluded that the spillway capacity was adequate to meet the EGLE requirement of "half PMF." EGLE reviewed the memo and submitted questions in March 2019, which were later reviewed and addressed by Spicer in September 2019, as discussed in Section 7.1.4.4.

It appears that the Purkeypile analysis assumed that flow through the spillways would always be unrestricted flow over the concrete spillway weirs; it also assumed a constant weir coefficient of 3.95, even though the restriction of the gates to being capable of opening only about 6 to 7 feet would result in restricted orifice flow for higher lake levels. It also appears that there was some confusion among the engineering consultants and EGLE regarding whether the hydraulic analysis should be based on half of the PMF discharge versus the "half PMF," which EGLE defines as the discharge resulting from half of the PMP storm. In the opinion of the IFT, EGLE's use of the term "half PMF" to describe the discharge resulting from half of the PMP storm can be misleading and should be revised to a more accurate term, such as "half PMP flood."

In January and February 2020, using gate opening geometry provided by Spicer, EGLE performed its own hydraulic analysis to calculate the spillway rating curve. EGLE's analysis concluded that Edenville Dam could not pass the "half PMF," using the maximum gate openings actually achieved during the June 2019 gate opening tests, which had used A-frames to extend the openings about 1 foot beyond the opening that could be achieved using the electric hoists alone (see Section 2.5). It can be presumed that

EGLE would have pressed Boyce Hydro and FLTF to increase the spillway capacity to meet EGLE's "half PMF" requirement, but the dam failed just a few months later in May 2020.

7.1.4.4 FLTF/Spicer Dam Inspections and Evaluations

According to FLTF, FLTF and their engineering consultant, Spicer, began performing "due diligence" reviews of the dams and available records in August 2019. This effort included observing gate operations, inspecting hydropower equipment at all four projects, visually observing all dam components, and mapping properties and flowage rights.

On September 18, 2019, FLTF submitted inspection findings to EGLE and noted that Edenville Dam could not be operated to meet the EGLE requirements to pass the "half PMF" without repairs and improvements. As noted in Section 7.1.4.3, it was during this time that FLTF, Spicer, and EGLE were coordinating with one another to determine the spillway capacity at Edenville Dam, which EGLE eventually confirmed in January 2020 to be insufficient to pass the state-required "half PMF."

In the fall of 2019, Spicer began topographic surveys at all four dams, and by February 2020, preliminary existing condition plans were completed based on the survey. The surveyed features included, but were not limited to, embankment cross sections, subsurface drain system locations, crest elevations, concrete features, and property boundaries. According to Spicer, the purpose of this effort was to capture the existing condition of the facilities in anticipation of future repairs.

Also in the fall of 2019, FLTF contracted with Ayres Associates to perform a PMF study, and the report was published in May 2020. The spillway rating curves were derived from the GEI Consultants Inc. (GEI) April 2020 Technical Memorandum. All of the dams were computed to overtop during the PMF (Ayres 2020).

During February 2020, FLTF contracted with Fisher Contracting to perform concrete repairs to Edenville Dam. Repair locations included the upstream piers and wingwalls on both the Tobacco and Tittabawassee spillways, and repairs consisted of removing and replacing deteriorated concrete. In addition, new lifting connections were installed on the existing gates to safely lift the gates to their maximum height once new hoist systems were installed. New hoists were ordered in April 2020, and were planned to be received and installed in the fall 2020, but the failure occurred in May 2020 before the hoists could be installed.

Also in February 2020, FLTF contracted with GEI to perform design for further dam repairs and improvements. GEI was primarily tasked with designing spillway capacity upgrades for Secord Dam, but according to Spicer, GEI was expected to be involved with design work for the other three dams as well.

In June 2020, FLTF submitted the final Part 315 Inspection Report, which had been prepared by Spicer based upon inspections performed between June 2019 and March 2020 (see Section 7.1.5). The report identified the dams to be in fair to poor condition and recommended Phase I repairs consisting of concrete and embankment repairs as well as Phase II work, which included significant improvements, including upgrading the spillway capacity. The report noted that further detailed studies, engineering, and permitting were needed to implement the repairs.

7.1.5 Geotechnical Investigations and Analyses

The geotechnical investigations performed at Edenville Dam are summarized below and described in further detail in Appendix B. The geotechnical investigations described in this report began in 1987, and the IFT did not find documentation of any significant geotechnical investigations of the Edenville embankments prior to 1987. Since the Edenville Dam failure involved static liquefaction, and that failure mode had been considered for Secord Dam in 2001 and 2005, the static liquefaction analysis for Secord Dam is also described in this section.

1987 Geotechnical Investigation for Tobacco Right Embankment

In 1987, Wolverine retained Soils and Materials Engineers, Inc. (SME) to perform an investigation at one location (about Sta. 48+00) of the Tobacco right embankment where seepage and a soft area were observed on the downstream slope. This was one of the taller sections of the Edenville embankments, with a height of about 40 feet. The investigation report (SME 1987) included test borings, laboratory testing, installation of groundwater monitoring wells, and slope stability analysis. The factors of safety for stability of the downstream slope were calculated to be 1.4 for the normal water level (El. 675.8) and 1.3 for a high reservoir water level (El. 677.3, which is only 1.5 feet above the normal lake level).

For the stability analyses, strength parameters for the embankment and foundation soils were estimated to be zero cohesion and friction angles ranging from 32 to 35 degrees, based on triaxial tests on reconstituted samples and SME's judgment. Raw, uncorrected standard penetration test (SPT) blow counts in the embankment soils ranged from 5 to 14 bpf, and those in the foundation sand layer ranged from 16 to 44 bpf. The strength of the foundation glacial till (hardpan) was estimated to be cohesion equal to 4.5 ksf with zero friction angle. Phreatic surfaces within the embankment were based on water levels measured in the monitoring wells.

SME cited a factor of safety (FS) of 1.5 as being "commonly accepted by the geotechnical engineering profession as an acceptable level for embankment stability under normal pool conditions" (SME 1987), which is true and which is also a FERC requirement. SME went on to state that "While the calculated FS for the normal and high water levels are not dangerously low, we believe it may be prudent to take some positive steps toward increasing the FS of the embankment in the area of Station 48+00" (SME 1987). SME recommended a "weighted filter" embankment overlay in this location as the "most cost effective alternative." The recommended overlay was ultimately constructed, but not completed until 2012. The reason for the long delay in implementation of this recommendation is not known. The overlay, referred to as a toe filter drain, consisted of a downstream two-stage filter overlay, resleeving existing foundation drains, and adding a perforated pipe toe drain.

No investigation for other locations of the embankment was recommended in the 1987 work.

1991 Slope Stability Analysis

In 1991, the initial Part 12D Consultants Safety Inspection Report (CSIR) was prepared for Wolverine by A. R. Blystra & Associates (Blystra). This report included slope stability analyses completed for Sta. 10+00 in the Edenville right embankment, a section of embankment with a height of about 40 feet (Blystra 1991).

No boring logs were included in the report; however, the report indicates that soil strength parameters were based on direct shear test on two reconstituted specimens from soil samples taken from two locations, one from the Tobacco embankments and one from the Edenville embankments. The strength parameters from these two tests were zero cohesion and friction angles of 34.3 and 34.6 degrees. The soils tested were classified as poorly graded sand with little or no fines. The phreatic surfaces in the slope stability analyses were based on water level readings in two groundwater monitoring wells. The friction angle used in the slope stability analyses was 34.5 degrees.

For the downstream slope, the calculated factors of safety were 2.12 for the normal water level (El. 675.8), 1.8 for a high water level (at the embankment crest), and 1.76 for seismic loading with normal water level. These factors of safety were judged to be satisfactory. Rapid drawdown analyses were completed for the upstream slope, but they are not pertinent to this forensic investigation.

The potential for seismic liquefaction of loose, saturated sands was considered. However, the embankments were stated to have been constructed using compacted layers of sand with no evidence of hydraulically placed fill, which would indicate a medium-dense consistency. Therefore, the embankments were judged to be safe against liquefaction.

In an August 6, 1993 letter to Wolverine (FERC 1993) recognizing the significance of the approximately 6,000-foot length of the Edenville embankments, FERC raised questions about the Blystra slope stability analysis and stated that:

> "*Because the embankments are so long*, the consultant should provide exhibits that show the location of all known borings taken in the embankments and the boring logs. *Additional borings will be necessary in the Tobacco spillway and in the sections of the embankments that have not been included in the previous soil-boring programs. These additional borings are necessary, unless the existing borings fully support the assumptions made regarding the strength parameters of the foundation and embankment soils*. The consultant should provide drill logs and test results that support the soil properties shown in the addendum. Also, he should provide an exhibit showing the location of those piezometers whose data is included in the analyses. All stability analyses must be performed based on the known phreatic surface and soil parameters.
>
> You are asked to address the above items in an addendum to be submitted by December 31, 1993. The data and analyses submitted to date are fragmented and incomplete. Therefore, the addendum must be a "stand-alone" document that consists of all the necessary studies, analyses and data outlined in the FERC guidelines for consultant's safety inspection reports." [*Emphasis added*.]

In a Supplemental Information Report dated December 1993 (Blystra 1993) submitted by Blystra as an addendum to their 1991 CSIR, Blystra responded with the following statement, referencing the Edenville construction specifications (see Section 7.1.2 and Appendix B):

> "EARTH EMBANKMENTS
>
> FERC has questioned whether or not the same materials were used in the earth embankments because of the total length of the embankments. Appendix E is the construction specification for the earth embankments. *The same material was used on all embankment construction*. As indicated in References 1 and 2, embankment soils were obtained during installation of piezometers. Soil properties for stability analysis were based on these assigned properties." [*Emphasis added*]

It appears that the boring locations, boring logs, test results, and piezometer locations requested in the FERC letter were not provided. It should be noted that the construction specification does not state or imply that "the same material was used on all embankment construction," but rather states that the material should be zoned in an upstream versus downstream manner, and the noted sources of borrow for the material could be expected to result in variability along the length of the embankments:

> "Upstream of the center line of the earth fill the material shall be selected of clay, gravel and loam mixed so as to make as impervious an earth fill as possible, while that of the downstream side of the earth fill shall be free from clay so far as the same may be obtained in the borrow pits and be pervious. The earth of the different parts of the earth

> fill shall be taken from the borrow site over such areas as directed by the Engineer to obtain the best available materials for the earth fills.
>
> The borrow for the earth fills may preferably be made from pits adjacent but not nearer than a line parallel to and fifty feet from the upstream toe of the earth fill ... The borrow for the earth fills may also be made from pits downstream of the earth fill when approved by the Engineer, and when so made shall not be nearer than fifty feet from the downstream toe of the earth fill ….”

In a March 3, 1994 letter to Wolverine (FERC 1994), FERC responded with the following statement, noting that the requested data were not provided to FERC, and FERC again questioned the basis of the soil strength parameters used in the stability analysis:

> “Your consultant submitted a copy of the specifications for the embankments when the project was being constructed (Appendix E of the addendum). *While this implies embankment homogeneity and shows the soils were placed and compacted, it does not substantiate assumptions made regarding the soil strength parameters*. Available boring logs, soil testing data and plan views of the bore hole locations have not been provided. The plan view showing the location of the piezometers whose data are included in the CSIR, has not been provided.” [*Emphasis added.*]

The IFT did not find any documentation indicating that Wolverine and Blystra responded to FERC’s 1994 letter and provided the information requested by FERC, nor that FERC ever followed up on its requests for data and justification for the soil parameters assumed in the slope stability analysis.

1994 Part 12D Report

The 1994 Part 12D CSIR was prepared for Wolverine by Mead & Hunt (Mead & Hunt 1994c). This report made the following statement, which recognized the 1993 and 1994 communications between FERC and Wolverine and Blystra. This statement recommended that a soil boring program be completed in the summer of 1995 and confirming that Mead & Hunt was not able to locate the boring logs and lab test data that had been requested by FERC:

> “Embankments. The 1991 Initial Consultant’s Safety Inspection Report indicates that direct shear testing and gradation analyses were performed on a soil sample retrieved from the downstream slope of the embankment. One sample was taken from the Tobacco embankment and one sample was taken from the Tittabawassee embankment. Results yielded a poorly graded sand with little or no fines and an internal angle of friction of 34.6 degrees for the Tobacco embankment and 34.3 degrees for the Tittabawassee embankment. The internal angle of friction used in the slope stability analysis is 34.5 degrees. The friction angle used for the foundation is 35.5 degrees. The dry unit weight used in the stability analysis is 115 pcf for the embankment and 117 pcf for the foundation. The foundation material was assumed to have an internal angle of friction of 34.2 degrees and a dry unit weight of 117 pcf. The 1991 Consultant’s Safety Inspection Report indicates that these valued [sic] are based on the results of testing conducted on a sample taken from the embankment and foundation. *A copy of the embankment soil boring log or laboratory test report could not be located for review*.
>
> In their March 3, 1994 letter, the FERC asks for additional information regarding available boring logs, soil testing data, and plan views of bore hole locations. This data could not be located for review. *It is recommended that the licensee initiate a boring program on the earth embankments to verify the soil parameters used in the slope stability analysis. The boring program should include enough borings to evaluate the*

> *homogeneity of the embankment material. Soil borings should be completed during the summer of 1995*." [*Emphasis added.*]

The IFT did not find any evidence that the recommended boring program was ever completed and could not find an explanation for why it was not completed.

2000 Part 12D Report

The 2000 Part 12D CSIR was prepared for Synex by Mead & Hunt (Mead & Hunt 2000). The report makes the following statement, repeated almost verbatim from the 1994 Part 12D CSIR:

> "Earthen Structures
>
> Direct shear testing and gradation analyses were performed on a soil sample retrieved from the downstream slope of the embankment for the 1991 safety inspection report. One sample was taken from the Tobacco embankment and one sample was taken from the Tittabawassee embankment. Results yielded a poorly graded sand with little or no fines and an internal angle of friction of 34.6 degrees for the Tobacco embankment and 34.3 degrees for the Tittabawassee embankment. The internal angle of friction used in the slope stability analysis is 34.5 degrees. The friction angle used for the foundation is 35.5 degrees. The dry unit weight used in the stability analysis is 115 pcf for the embankment and 117 pcf for the foundation. The foundation material was assumed to have an internal angle of friction of 34.2 degrees and a dry unit weight of 117 pcf. The 1991 inspection report indicates that these values are based on the results of testing conducted on a sample taken from the embankment and foundation. *A copy of the embankment soil boring log or laboratory test report could not be located for review*." [*Emphasis added*]

This statement again confirms that Mead & Hunt was not able to locate the boring logs and lab test data previously requested by FERC. It is noteworthy that this 2000 Part 12D CSIR makes no reference to the boring program recommended in the 1994 Part 12D CSIR, which was to have been completed in the summer of 1995 but apparently was not, despite the fact that the same consultant had prepared both the 1994 and 2000 Part 12D CSIRs. The IFT was not able to find an explanation for why the recommendation to complete a boring program was not carried forward into the 2000 Part 12D CSIR.

2003 Geotechnical Investigation

In 2003, a significant increase in seepage and some accumulation of fines coming from the drains of the Edenville right embankment and Tobacco left embankment adjacent to the powerhouse were observed. Two small sinkholes and some surface sloughing were also observed, adjacent to the retaining wall downstream of the Edenville powerhouse.

Wolverine retained McDowell and Associates to perform an investigation at the Edenville right and Tobacco left embankments where seepage was observed (McDowell 2003). The investigation included test borings, laboratory testing, installation of groundwater monitoring wells, and dye tests.

The investigation found that the embankment fill typically consisted of poorly graded fine sand fill with little fines content and blow counts ranging from 4 to 15 bpf. The dye test was performed in the groundwater monitoring wells adjacent to the Edenville powerhouse. No dye was observed exiting the downstream side of the embankment. Twenty days later, dye was observed in the wells, apparently not having been flushed out from the groundwater flow.

In 2004, a "toe filter drain" was installed along the Edenville right embankment along the location of the observed seepage. The project consisted of a downstream two-stage filter overlay, resleeving the existing underdrains, and adding a perforated pipe toe drain. The IFT found no record of stability analysis being done in conjunction with this modification.

2005 Part 12D Report

The 2005 Part 12D CSIR was prepared for Synex by Mead & Hunt (Mead & Hunt 2005a) and referenced the stability analysis documented in the 1991 Part 12D CSIR in which adequate factors of safety for the downstream slope had been calculated. No reference was made in this Part 12D CSIR to the subsurface exploration program recommended in the 1994 Part 12D CSIR, nor was any recommendation made to conduct such a program or additional slope stability analyses for any location of the dam despite the fact that the same consultant had prepared the 1994, 2000, and 2005 Part 12D CSIRs. The IFT was not able to find an explanation for why the 1994 recommendation to complete a boring program was not carried forward into the 2005 Part 12D CSIR.

2007-2009 Embankment Stability Analysis

From 2007 to 2009, embankment stability analyses were performed by Mill Road Engineering for Boyce Hydro and reviewed by the BOC. The first submission was dated August 5, 2007, and the final, fifth revision was dated February 28, 2009 (Mill Road 2009a). The stability analyses were completed to support the design of a toe filter drain overlay at Sta. 48+00 in the Tobacco right embankment, the same cross-section analyzed by SME in 1987. The embankment geometry, phreatic surfaces, and strength parameters were based on the SME 1987 analysis.

The proposed toe filter drain installation consisted of a downstream two-stage filter overlay, re-sleeving the existing underdrains, and adding a perforated pipe toe drain. Stability analyses were completed for normal water level (El. 675.8) and a high water level at the embankment crest (El. 683). The resulting calculated factors of safety were 1.53 and 1.40 for normal and high water levels, respectively, for the case of the toe filter drain in place. These factors of safety met FERC requirements. Stability analyses were also completed for the upstream slope under rapid drawdown, but they are not pertinent to this forensic investigation. The toe drain filter at Sta. 48+00 was not completed until 2012.

It should be noted that FERC reviewed the submissions of the stability analysis and apparently performed its own in-house stability analysis, concluding at one point that one of Mill Road Engineering's stability analyses was overly conservative and not consistent with the stability analyses previously performed by other consultants. In a letter to Boyce Hydro dated September 26, 2007 (FERC 2007), FERC made the following comments under the heading "Embankment Stability Analysis:"

> "Please provide a reanalysis of the embankment stability. Verify input geometry. It is unclear why the Factors of Safety are much lower than previous stability analyses. Previous Factors of Safety for Flood Case were at 1.80 as provided in the STID. An independent stability check by FERC staff also resulted in much higher Factors of Safety than the submitted analysis."

It should be noted that the BOC also reviewed the submissions of the stability analysis and performed its own stability analysis. In the subsequent Report by the Special Board of Consultants (BOC 2009), the BOC concluded that:

> "...even under the worst possible assumption of a fully saturated embankment (inoperable drains) during the PMF (full pond) the factor of safety would be higher than 1.1, which in our opinion, provides adequate safety for an extreme loading of a very short duration. Furthermore, the slip surface under these conditions is relatively shallow and will not affect the overall stability of the dam."

2010 Seismic Liquefaction Analysis

The 2005 Part 12D CSIR included a recommendation to perform a seismic liquefaction triggering analysis. In 2010, two borings were drilled in the Edenville left embankment to support a seismic liquefaction evaluation by Mill Road Engineering for Boyce Hydro (Mill Road 2010). One boring was completed at the crest and the other at the toe, along a cross section very near the ultimate failure location in May 2020. The crest boring indicated raw, uncorrected blow counts ranging from 2 to 10 bpf in the embankment and 16 to 23 bpf in the foundation sand. Lab testing included index tests and unconfined strength tests. The embankment was judged not to be subject to earthquake-induced liquefaction due to the low energy of an estimated earthquake event for the site.

2010 Part 12D Report

The 2010 Part 12D CSIR (Mill Road 2011b) was prepared for Boyce Hydro by Mill Road Engineering and referenced the stability analysis performed in the 1991 Part 12D CSIR in which adequate factors of safety for the downstream slope had been calculated. No reference was made in this Part 12D CSIR to the subsurface exploration program recommended in the 1994 Part 12D CSIR, nor was any recommendation made to conduct such a program or further slope stability analyses for any location of the dam.

2015 Part 12D Report

The 2015 Part 12D CSIR was prepared for Boyce Hydro by Purkeypile (Purkeypile 2016a) and referenced the stability analysis documented in the 1991 Part 12D CSIR in which adequate factors of safety for the downstream slope had been calculated. No reference was made in this Part 12D CSIR to the subsurface exploration program recommended in the 1994 Part 12D CSIR, nor was any recommendation made to conduct such a program or further slope stability analyses for any location of the dam.

Borings for Possible Spillway Locations

In 2005, 2013, 2015, and 2018, explorations were completed in support of evaluations of possible spillway locations (McDowell 2005a/b, 2013, 2015; Gomez and Sullivan 2018). A total of 16 borings were drilled in these four campaigns, including six borings from the crests of the Edenville right and left embankments. According to the reports, where the embankment and foundation soils were sampled, the raw, uncorrected blow counts ranged from 2 to 23 bpf in the embankments, and blow counts ranging from 16 to 31 bpf were typical in the foundation sand.

2020 Spicer Inspection Report

As a consultant to FLTF, Spicer performed inspections of Edenville Dam from June 2019 through March 2020 (Spicer 2020). Spicer did not perform any slope stability analysis for the Edenville embankments, and reportedly did not have a geotechnical engineer on staff or as a subconsultant for this work. Spicer made the following statement in their report:

> "The embankments on the Edenville and Tobacco side extend from the dam to the natural high ground on either side. The minimum crest elevation is approximately 682.1 feet at the Tittabawassee embankment and the Tobacco embankment. The upstream and downstream slopes of to [sic] 2.5H:1V and 2H:1V, respectively, are *steeper in some areas*. The earthen embankment does not have a core wall but is *constructed of relatively impermeable glacial till deposits* with a seepage tile system for internal drainage. Adjacent to each spillway, a steel sheet piling core-wall exists.

An embankment stability analysis is recommended." [*Emphasis added.*]

It is noteworthy that Spicer recognized the presence of steeper slopes and the need for additional slope stability analysis, and it is possible that if and when such a stability analysis had been done, a deficiency in the downstream slope stability may have been identified. It is also noteworthy that Spicer formed the impression that the embankments were "constructed of relatively impermeable glacial till deposits," whereas the information in previous geotechnical investigations and CSIRs after 2005 generally indicated that the embankments were constructed at least in significant part with sandy soils with "little or no fines."

Static (Flow) Liquefaction Analysis for Secord Dam

Detailed review of geotechnical investigations for Secord, Smallwood, and Sanford Dams was not in the scope of the IFT's investigation. However, based on a cursory review of the associated documents, the IFT discovered that the potential for static (flow) liquefaction was identified for the Secord Dam embankment as part of the 2001 Part 12D dam safety review performed by Barr Engineering (Barr 2001). The Barr team that performed the 2001 Part 12D for Secord was different from the teams for the Part 12D reviews of the other Boyce Hydro dams performed in a similar time frame. To the IFT's knowledge, Barr was never involved with Edenville Dam.

Two sets of borings were available to the 2001 Part 12D review team for Secord Dam: a series of hollow stem auger borings drilled in 1996 within 10 to 12 feet to the right of the spillway abutment wall, and two borings drilled in 2001 using hollow stem augers (surcharged with a bentonite slurry) located slightly further offset from the spillway/powerhouse structure, at approximately 80 feet to the right of the spillway and 50 feet to the left of the powerhouse. The material encountered in the 2001 borings consisted of poorly graded sands underlain by clayey loam that extended to the foundation. The 1996 borings, performed immediately adjacent to the right spillway abutment wall encountered different soil conditions than those encountered in the 2001 borings. The 1996 borings showed the presence of very loose, saturated, poorly graded sands with intermittent layers of lean clay and silt that extended to the foundation.

The 2001 Secord Dam Part 12D team performed an analysis to evaluate the susceptibility of the embankment soils to both seismic (cyclic) and static (flow) liquefaction, using the 1996 and 2001 borings. The evaluation of flow liquefaction considered five conditions that are required for flow liquefaction to occur:

- Condition 1: Embankment and foundation soils must be susceptible to strain-weakening.
- Condition 2: In situ shear stresses must be greater than the minimum undrained shear strength.
- Condition 3: A trigger must exist to cause undrained shear.
- Condition 4: A sufficient volume of strain-weakening material must exist.
- Condition 5: Site geometry must form a kinematically permissible mechanism.

Condition 1 was evaluated based on procedures proposed by Fear and Robertson (1995), which recommends an empirical boundary line between dilative and contractive behavior based on corrected SPT blow counts. Based on this analysis, the materials encountered in the 1996 borings were judged susceptible to flow liquefaction behavior. Condition 2 requires a detailed analysis, which was not performed as part of the 2001 Part 12D, but instead it was assumed that it could occur. It was also assumed that Condition 3 could occur. Boring data were judged to support the plausibility of Conditions 4 and 5. Therefore, a post-liquefaction stability analysis was performed, resulting in a calculated factor of safety of 0.67. It was judged that the loose soils in the area immediately adjacent to the concrete structures may have been due either to seepage or difficulty in compaction. Therefore, it was postulated that a similar condition may have been present immediately adjacent to the left powerhouse abutment wall. The 2001 Part 12D team recommended that stabilization measures be implemented in the area to the right of the spillway and that investigations be performed in the area to the left of the powerhouse.

In response to this recommendation, additional hollow stem auger borings using a bentonite slurry were performed in 2005 in the area to the left and right of the spillway/powerhouse structure. The 2005 borings encountered poorly graded sands overlying clays, but the sands did not extend to the foundation. As part of the 2005 Part 12D dam safety review performed by A. Rieli & Associates, an analysis was performed to evaluate the potential for seismic (cyclic) liquefaction using blow count data from the 2005 borings for the depths investigated. It was concluded that the soils were not susceptible to cyclic liquefaction (A. Rieli & Associates 2006). Although the 2005 Part 12D report also stated that the materials encountered in the 2005 borings were not susceptible to flow liquefaction, the IFT was unable to locate any analyses performed to support that conclusion.

The recommendation to stabilize the embankment against a flow liquefaction failure was not carried forward in the 2005 Part 12D CSIR. This appeared to be based on the results of the cyclic liquefaction analysis and the judgment that the 1996 blow count data were erroneously low. It was stated that the 1996 borings did not use a bentonite slurry and that could have resulted in erroneously low blow counts below the water table. Although this may be true, blow counts in saturated, poorly graded sands measured in the 2005 borings in some instances were lower than blow counts measured in the 1996 borings, which were judged by Barr Engineering (Barr) to indicate a susceptibility to flow liquefaction. Therefore, such cases of the 2005 blow count data would have also indicated a susceptibility to flow liquefaction, had the data been analyzed in a similar manner. In addition, the materials identified as being susceptible to seismic liquefaction using the 1996 borings were below the depths investigated in the 2005 borings, and therefore a valid comparison could not be made.

7.1.6 Edenville Dam Hydrologic and Hydraulic Analyses

This section summarizes previous hydrologic and hydraulic analyses performed for Edenville Dam, as well as concepts *that were studied for upgrading the spillway capacity. Further information on these analyses and studies is provided in Appendix C.*

7.1.6.1 Spillway Capacity Design and PMF Studies

The IFT found only one gate discharge curve developed by Holland, Ackerman and Holland, dated 1926, which correlates lake level, gate opening, and discharge at the Edenville and Tobacco spillways. Weir flow was assumed with the gates opened to 10 feet (lake level at 677.8) higher than the sill elevation of 667.8 feet. The maximum flow for a gate shown on the discharge rating curves is 2,800 cfs, and there is no estimate of higher flows. No other evidence was found by the IFT related to spillway capacity or hydrologic and hydraulic analyses incorporated into the original design of the project, and it is not clear how the spillways were originally sized with respect to hydraulic capacity.

The National Dam Safety Program Inspection Report (Phase I Report) for Edenville Dam was completed in 1978 by Commonwealth Associates (1978). The Probable Maximum Storm (PMS) during the maximum 24-hour period was provided by the Michigan Department of Natural Resources and was estimated to be about 22 inches of rainfall, producing a PMF of 125,500 cfs. The total spillway capacity of the dam was estimated to be about 29,000 cfs for both the Edenville and Tobacco spillways. This was assuming that all gates were fully open and that the powerhouse turbines were not operating. The peak outflow was estimated to be 124,500 cfs, with the earth embankments overtopped by approximately 2.68 feet and with the assumption that the dam would not fail. In the opinion of the IFT, this is a questionable assumption.

The initial Part 12D CSIR was prepared by Blystra and submitted in 1991, and this report indicated that the PMF inflow was estimated to be 56,200 cfs (less than half of the 1978 estimate). The spillway capacity with the lake level at the top of the dam was estimated to be 22,950 cfs, or 41 percent of the PMF. The antecedent moisture condition assumed that the soils in the watershed had relatively high infiltration rates (low runoff potential) (Blystra 1991).

Based on the results of the PMF and a dam breach analysis that was performed, it was concluded that the spillway capacity should be increased. However, a new PMP study was being conducted for Michigan and Wisconsin by the Electric Power Research Institute (EPRI) and a private consultant. It was believed that the study would impact the PMF, and therefore the results of the new PMP study should be considered before any spillway modification was considered.

After initiation of FERC regulation, the various PMF studies described below were undertaken to comply with FERC regulations. The results of these analyses indicate that it was known as early as 1991 that the spillway capacity was not sufficient to pass the PMF, a conclusion consistent with that from the Phase I report in 1978. Thus, the spillway capacity was considered to be deficient relative to FERC requirements.

A PMF study was performed in 1994 by Mead & Hunt (Mead & Hunt 1994a) incorporating the new PMP study by EPRI. It was estimated that the PMF inflow was 74,400 cfs. It also reported that the spillway capacity was estimated to be 23,650 cfs without overtopping the embankment or 32,800 cfs when the headwater lake level reached the nominal crest elevation, which assumed some overtopping flow over the low points of the dam crest. The spillway discharge calculations do not appear to have considered any restriction on the height of the gate openings.

In 2008, 2009, and 2011, a PMF reanalysis was performed by Mill Road Engineering that estimated the PMF inflow to be 61,790 cfs (Mill Road 2011a). The report also concluded that the spillway capacity was insufficient to pass the PMF. Mill Road used a Hydrologic Engineering Center-River Analysis System (HEC-RAS) model to estimate the gate discharges. The HEC-RAS model provides two forms of the Tainter gate equation. There is no mention in this report of a limitation of the gate openings which would have reduced the spillway capacity. Mill Road Engineering modeled the reservoir segments on the Tobacco River and the Tittabawassee River, using an unsteady flow model to account for the interconnection of the two rivers via the M-30 bridge. Even though the PMF was less than that obtained by Mead & Hunt, the openings provided by the spillway gate system were concluded to be insufficient to pass the flood without overtopping the dam.

In 2013, Ayres Associates performed a study to determine if the Inflow Design Flood (IDF) for Edenville could be less than the PMF that Mill Road Engineering had calculated. They performed a dam breach analysis to evaluate the incremental damage downstream and concluded that the IDF is the full PMF (Ayres 2013a); in other words, the IDF could not be reduced below the PMF without resulting in excessive downstream incremental damage if the dam were to fail. This analysis used PMF inflow hydrographs taken from the model previously developed by Mill Road Engineering. River reaches upstream of Wixom Lake were truncated so that the model's upstream boundaries were 5.5 miles upstream of the dam on the Tobacco River and 10.3 miles upstream of the dam on the Tittabawassee River. PMF hydrographs at the truncated boundary conditions were taken from Mill Road Engineering's PMF model.

The results indicated that the dam would be overtopped during the PMF. PMF outflows were estimated to be 41,900 cfs at the Edenville side and 25,900 cfs at the Tobacco side, including both spillway flows and embankment overtopping flows (without failure).

In April 2020, just a month before the dam failure, GEI completed a memorandum that presented results of new spillway discharge rating calculations developed for the Tainter gate spillways and overflow sections at the Secord, Smallwood, Edenville and Sanford Projects (GEI 2020). Prior to the GEI study, supporting calculations for spillway discharge rating curves presented in the STID were not available for some of the dams, and the rating curves appeared to be inconsistent with recent spillway surveys and maximum gate opening tests. FLTF had requested that GEI review the available hydraulic information and develop new spillway discharge rating curves for each of the four projects. The GEI study accounted for both weir and gate orifice flow, variable discharge coefficients, and effective weir length. This appears to be the first spillway rating curve for the Edenville project that accounted for pier and abutment flow contractions. The total zero-freeboard discharge capacity at Edenville Dam was estimated to be 20,670 cfs at the minimum dam crest elevation of 682.1, assuming the gates could be opened to openings between 8.9 and 9.6 feet, except for one of the Tobacco spillways gates, which was estimated to be opened to only 4.5 feet.

In May 2020, Ayres Associates performed a PMF study (2020) using spillway geometry and discharge data provided by GEI (2020) and estimated the PMF inflow to be 80,900 cfs. The report was dated May 15, 2020, just a few days before the Edenville Dam failure. Wixom Lake is bifurcated by the M-30 causeway. This study assumed a free exchange of flows and equalized lake levels between the Tobacco River and the Tittabawassee River sides, a condition that was demonstrated in an earlier study by Spicer. Contrary to the actual gate openings during the May 2020 storm event (see Section 3.1), which were about 7 feet, all of the spillway gates were estimated to be opened to heights between 8.9 and 9.6 feet, except for one of the Tobacco spillways gates, which was estimated to be opened to only 4.5 feet.

In 2021, after the failures, Ayres Associates prepared a Design Flood Hydrologic Analysis which estimated the PMF inflow to be 113,400 cfs, which is much higher than the PMF estimates in the studies from 1991 to 2020 (Ayres 2021). This study incorporated a new site-specific PMP study for the Tittabawassee River (AWA 2021). The results of the PMP study were highly influenced by the May 17 through 19, 2020 precipitation event that resulted in a 100- to 200-year runoff event from a 25- to 50-year rainfall event, as discussed in Section 5, Evaluation of the Flood Event, and Appendix F1.

7.1.6.2 Spillway Capacity Upgrade Studies

In 1995, Wolverine began a chain of correspondence with FERC regarding the spillway capacity at Edenville Dam. Wolverine requested that spillway capacity upgrades be delayed in order to allow them to further assess whether prior PMF studies were accurate and whether a new study should be performed to avoid unnecessary spillway capacity upgrades. FERC allowed an extension, but in a letter to Wolverine dated January 4, 1999, stated that “Modifications to increase spillway capacities at all of your projects remain the primary concern. Work is expected to begin on your Smallwood project in 1999” (FERC 1999). In a subsequent letter dated April 14, 2000, FERC stated that “You must complete all modifications to the Smallwood dam prior to the end of 2000 as scheduled” and “You should begin work on spillway capacity modifications at your Sanford dam in 2001 with completion no later than the end of 2002,” and also “Work should begin in 2003 on Secord and 2005 on Edenville unless conditions necessitate a more accelerated schedule, or your rainfall data suggests otherwise.” (FERC 2000).

Beginning around 2001, Wolverine began to contend that they were unable to finance the necessary spillway capacity upgrades due to insufficient revenue, and construction (including construction at Edenville Dam) was deferred. Despite FERC's directives, this deferral continued throughout the rest of Wolverine's tenure as the dam owner, into and throughout Synex's subsequent ownership, and again into Boyce Hydro's ownership.

Boyce Hydro first proposed Edenville spillway modifications to FERC around 2009 and continued evaluations of numerous plans for spillway capacity upgrades, including modifications to the gated spillways and consideration of an open-weir or labyrinth auxiliary spillway, until and during the license revocation process. The proposed spillway modifications are discussed in more detail in Appendix C. Four of the more significant proposals are described below.

1 *PMF Spillway Upgrade.* A spillway upgrade concept to achieve PMF capacity was developed from about 2008 to 2013 and involved modifying the existing concrete spillways at Edenville Dam. This included potential modification of both the Edenville and Tobacco gated spillways. Mill Road made their final recommendation that gate modifications be made to the Edenville spillway and the Tobacco spillway by lowering the existing gate sill elevation by 13 feet, from 667.8 to 654.8, for all six gates. The total new spillway capacity was to be in excess of 64,000 cfs, including the powerhouse running at full capacity. The estimated cost of construction was in the range of $5 million to $10 million. Due to lack of funding and financing, this project did not move forward to final design and construction.

2 *Open-Weir Auxiliary Spillway*. A spillway upgrade concept was finalized in 2016 and proposed as an interim risk reduction measure. This concept consisted of an open-weir auxiliary spillway with an approximate capacity of 5,600 cfs. A preliminary rough estimate of the construction cost for this option was about $1.6 million. The BOC indicated that the concept would be a satisfactory interim measure to reduce risk of overtopping dam failure, provided that other dam modifications would subsequently be made to try to increase the total capacity to be able to pass the PMF.

3 *Modified Open-Weir Auxiliary Spillway*. The open-weir auxiliary spillway concept described above was further developed by Gomez and Sullivan Engineers (Gomez and Sullivan), with studies and design continuing through March 2018. Gomez and Sullivan had completed soil borings and were developing plans to increase the capacity of the spillways. The supporting design report was approximately 75 percent complete. As with the previously developed auxiliary spillway concept, the auxiliary spillway was to have a capacity of 5,600 cfs, bringing the combined spillway capacity to an estimated 34,000 cfs. A temporary construction emergency action plan, quality control and inspection, and a water management report that had been prepared by Boyce Hydro were all about 80 percent completed.

4 *Labyrinth Auxiliary Spillway*. Another spillway concept upgrade concept which was studied by Gomez and Sullivan in 2017 and 2018, as an alternative to the open-weir auxiliary spillway concept, involved a labyrinth auxiliary spillway with a capacity of 12,000 cfs for an estimated total combined spillway capacity of 40,000 cfs. Drawings of the spillway were developed, and some analyses were performed by Boyce Hydro and Gomez and Sullivan.

Like the previous dam owners, Boyce Hydro stated that the spillway upgrades needed to achieve full PMF capacity were cost prohibitive, but partial upgrades to the spillway capacity could accomplished if Boyce Hydro could negotiate a higher rate with Consumers for selling power, comparable to the average rates being paid to other hydropower producers by Consumers, and Boyce Hydro was making significant efforts to negotiate higher rates. In addition to significant engineering efforts by Boyce Hydro to develop spillway upgrade proposals, construction in the form of limited grading downstream of the Tobacco right embankment was initiated for one of the spillway capacity upgrade proposals but was not taken to completion. Boyce Hydro also stated that disputes with EGLE (then known as DEQ, the Department of Environmental Quality) and the Michigan Department of Natural Resources, starting in 2015, led to litigation and had a significant impact on Boyce Hydro's ability to finalize the proposed open-weir and labyrinth spillway designs from 2016 to 2018, due to difficulty in obtaining permits and depletion of Boyce Hydro financial resources that were being accumulated to fund a first auxiliary spillway project.

In 2018, FERC revoked the license for Edenville Dam, and, thereafter, the dam was regulated by the State of Michigan through EGLE. Under EGLE regulation, the spillway capacity must meet or exceed the “half PMF,” which is defined by Michigan dam safety regulators as the flood resulting from half of the PMP, rather than half of the PMF hydrograph. For Edenville Dam, the "half PMF” results in an inflow which is about 40 percent of the full PMF, not numerically one-half of the PMF inflow. It is noteworthy that Michigan is one of only a few states which uses the “half PMF” as the IDF for high hazard dams. By comparison, most states require the full PMF for high-hazard dams, as does FERC (FEMA 2012).

In January 2019, Boyce Hydro provided EGLE with a memorandum prepared by Purkeypile Consulting, LLC which stated that the dam “...is capable of safely passing the 50 percent PMF with approximately 0.7 feet of freeboard” and that the dam was therefore in compliance with state requirements (Purkeypile 2019). EGLE indicated concerns with the analysis, which used the same spillway rating curve (the spillway rating curve indicates the spillway capacity as a function of the lake elevation) as the 2015 CSIR and did not consider orifice flow or wave set-up/run up (the 1994 Mead & Hunt study had concluded that wave runup was negligible during a 72-hour PMF event, since the high winds would occur before the peak inflow would be reached). Throughout the period from January to April of 2019, Boyce, FLTF, EGLE, and the engineering consultants continued to coordinate evaluation of Edenville Dam’s spillway capacity.

According to discussions with EGLE, on February 4, 2020, EGLE provided FLTF’s consultant with a revised spillway rating curve for the dam, which was based on the gate testing results and geometries that EGLE had received from Spicer. EGLE confirmed the rating curve with FLTF’s consultant and concluded definitively that the dam lacked spillway capacity to convey the design flood (the “half PMF”). EGLE requested that this revised hydraulic information be included in the forthcoming Part 315 dam safety inspection report, which was being finalized by FLTF’s consultant at the time of the failure.

Gate opening limitations also impacted the spillway discharge rating curves. In 2012, the existing hoist system for all four Boyce dams could only hoist the gates 6 to 7 feet. Since the gates needed to preferably be able to be raised an additional 4 feet or more, so that the gates would have clearance above high water during a significant flood condition, an A-frame system was designed to manually lift the gates an additional 4 feet (see Section 2.5). Later, the A-frames were replaced by an electric hoist system at Sanford Dam and hydraulic hoist systems at Secord Dam and Smallwood Dam. Sometime in late 2020, it was planned to have a hydraulic gate hoist system installed at Edenville Dam, but the dam failure occurred before the new hoist system was installed.

In 2019, based on observations during gate tests, it was determined that the maximum gate opening at Edenville Dam for the existing hoist system was about 6 feet, because it was judged that the gates at Edenville Dam should be operated only with the original hoist mechanisms until the new hydraulic gate hoist systems were installed. The A-frame and manual lever hoist were cumbersome to use and required too much time to operate under emergency conditions, and were judged to present a significant safety hazard to the operators and a possibility of damaging the gates.

Since the gates at Edenville Dam could not be fully opened (above the water flowing through the spillways at high lake levels), none of the spillway rating curves used in the previous studies were representative of the actual spillway capacity. Although it was known that the spillways at Edenville Dam could not pass inflow from the PMF without overtopping the embankments, it was not known what return period flood (e.g., 100-year, 500-year) would raise the lake to the embankment crest. Hence the risk of embankment overtopping was unknown. From the Ayres report on the Design Flood Hydrologic Analyses (2021), the 200-year flood, i.e., an annual exceedance probability (AEP) of 0.005, has a peak inflow of 25,400 cfs with a freeboard of 0.1 foot (lake level 682.0 foot) and peak spillway outflow of 20,400 cfs based on the assumption that the gates would be open about 9 feet. This result provides an estimate of the return period of the flood that would overtop the dam, and the analysis used spillway rating curves derived from calculations provided by GEI (2020). Since the gate openings at Edenville Dam used in GEI's analysis were greater than the actual gate openings during the May 2020 flood, the return period for the threshold flood that would raise the lake level to the embankment crest low point was actually less than 200 years (AEP greater than 0.005).

7.1.7 Edenville Dam Embankment Modifications

For Edenville Dam, significant modifications were performed at the dam which pertained to sheet piling and embankment overlays and drains, and these modifications are summarized below. Lists of modifications for each of the four embankments are provided in Appendix A of this report, and Appendix B provides additional information related to modifications to the Edenville Dam embankments, including Figure B-6, which shows the locations of various modifications and the corresponding dam stationing.

Sheet Piling between 1934 and 1936, sheet pile cutoff walls were installed on both sides of the Tittabawassee and Tobacco spillway structures and between Sta. 48+00 and Sta. 55+00. In 2008, the sheet pile wall caps were repaired. In 2015, sheet piling was placed along the downstream edge of the concrete apron (paving slabs) of the spillway stilling basin.

Embankment Overlays and Drains

In 2004, embankment overlays were constructed on the downstream slope along a section of the Edenville right embankment (~Sta. 10+00 to Sta. 13+50) and a section of the Tobacco left embankment (~Sta. 39+00 to Sta. 42+00). In 2007 and 2008, an embankment overlay was constructed along another section of the Edenville right embankment from ~Sta.13+50 to Sta. 21+00.

These overlays were generally configured as weighted filters, which both (a) flattened and buttressed the downstream slopes to increase slope stability and (b) provided a means to prevent internal erosion. In addition to the embankment overlays, the original clay tile underdrains at the locations of the overlays were resleeved with polyvinyl chloride (PVC) pipe for a portion of the length, connected to perforated PVC toe drain pipes installed in the collection ditch, and surrounded by gravel.

In 2009, a portion of the Tobacco right embankment barrel drain system, located at the downstream toe, was replaced to improve drainage. The system was replaced with new steel manholes placed on a bed of crushed stone. Perforated PVC pipes were placed between manholes, and the existing clay tile underdrains were extended into crushed stone with PVC pipe.

In 2011 and 2012, another embankment overlay was constructed on the downstream slope of the Tobacco right embankment from ~Sta. 44+00 to Sta. 49+00.

In 2012, the barrel drain system, located at the left downstream groin of the Tobacco left embankment, was replaced to improve drainage.

In 2014, the remaining portion of the Tobacco right embankment barrel drain system was replaced. Additional buttress fill was placed on the upper portion of the Tobacco right embankment downstream slope. Fill was placed from the top of the embankment overlay to the crest, from ~ Sta. 44+00 to Sta. 49+00.

The Edenville left embankment, left of the Edenville spillway in the area where the failure section was located, did not have any overlays constructed, nor did the IFT find a documented recommendation from an engineer to do so. Sometime between 2010 and 2012, the Edenville left embankment toe drain collection ditch was moved approximately 25 to 45 feet downstream from the original position and the clay tile drains were extended with PVC pipes. The IFT did not find any documentation of this work.

In total, as of May 2020, approximately 30 percent of the total length of the Edenville Dam embankments had an overlay constructed on the downstream slope between 2004 and 2014. Some of these overlay locations were constructed in order to address documented seepage and/or slope stability concerns. For other locations where overlays were constructed, the IFT did not find documentation that gave the reasons for constructing these overlays; however, slope stability analyses were performed for only two locations of the dam (see Section 7.1.5 and Appendix B) and therefore, as suggested to the IFT in interviews, overlays in most or all locations were likely constructed primarily to address seepage concerns.

7.1.8 Power Generation and Revenues

Power generation at the four projects began in 1924 and 1925, pursuant to the original agreement between Wolverine and Consumers dated 1923 and discussed in Section 7.1.1. The agreement was subsequently supplemented and amended nine times, generally to establish new rates or a new rate structure (in 1955, 1963, 1965, 1970, 1975, 1980, 1986, 2007, and 2014). The following are descriptions of several of these rates and rate structures:

- The original agreement specified a rate of $0.00563 per kilowatt-hour delivered to Consumers, which correlates to about $0.0833 in 2020 dollars.
- The second supplement and amendment, in 1963, included a new rate structure with a capacity charge of $1.00 per kilowatt "nameplate capacity" per month ($8.46 in 2020 dollars), and an energy charge of $0.0045 per kilowatt-hour on-peak ($0.038 in 2020 dollars) and $0.002 per kilowatt-hour off-peak ($0.0169 in 2020 dollars). "Nameplate capacity" is also known as "rated capacity" and "installed capacity" and is determined by the manufacturer of the generator. It refers to the sustained capacity of the hydroelectric facility under full load, based on various characteristics including flow and head. The total nameplate capacity of the four projects was 10,500 kilowatts (10.5 megawatts), and as long as all generators were operational (not down for maintenance), the capacity charge would be paid. The energy charge is based on the actual kilowatt-hours produced, which would be affected by changes in operational efficiency (which relates to reservoir level and powerhouse release). In general, the establishment of higher on-peak rates encourages "peaking operations," which typically maximize generation during peak power usage times in order to maximize profits.
- The capacity charge was eliminated in the sixth supplement and amendment in 1980.

- The seventh supplement and amendment, in 1986, restored the capacity charge, but changed the structure such that it was also based on the energy delivered and would include on-peak and off-peak rates, and also modified the energy charge such that it included a calculation based on expenses associated with operations as opposed to a flat rate.
- The eighth supplement and amendment, in 2007 (after Boyce Hydro acquired the projects), retained the capacity charge from the previous supplement and amendment, but reverted the energy charge structure to simple flat rates for on-peak and off-peak generation. The capacity charge was $0.0091 per kilowatt-hour on-peak and $0.0077 per kilowatt-hour off-peak, and an energy charge of $0.0512 per kilowatt-hour on-peak and $0.0217 per kilowatt-hour off-peak. Combining the capacity and energy charges yields on-peak and off-peak rates of $0.0603 and $0.0294 per kilowatt-hour, respectively, or $0.0753 and $0.0367 per kilowatt-hour adjusted to 2020 dollars.
- The ninth and final supplement and amendment, in 2014, included a capacity charge of $0.0233 per kilowatt-hour on-peak and $0.0197 per kilowatt-hour off-peak, and an energy charge of $0.0467 per kilowatt-hour on-peak and $0.0366 per kilowatt-hour off-peak. Combining the capacity and energy charges yields on-peak and off-peak rates of $0.070 and $0.0563 per kilowatt-hour, respectively, or $0.0765 and $0.0615 per kilowatt-hour adjusted to 2020 dollars.

The original agreement between Wolverine and Consumers specified that all power generated at the four hydroelectric projects be sold to Consumers. In 1978, the federal Public Utilities Regulatory Policies Act (PURPA) was enacted. PURPA created a market for power from non-utility power producers by forcing utilities to buy power from other more efficient producers if that cost was less than the utility's own "avoided cost" rate ("avoided cost" is the additional costs an electric utility would incur if it generated the required power itself).

In 2008, Michigan's Clean, Renewable, and Efficient Energy Act (Public Act 295) was enacted, which required utilities to generate 10 percent of retail electricity from renewable sources by 2015. This act set up the Michigan Renewable Energy Certification System (MIRECS). From 2009 to 2015, in addition to selling power to Consumers, Boyce Hydro sold RECs to Detroit Edison; the contract expired in 2015 and Detroit Edison declined to renew (one REC is equal to one megawatt-hour of renewable energy). From 2017 to 2020, in addition to selling power to Consumers Energy, Boyce Hydro sold RECs through a broker to various purchasers, though at substantially lower rates than those received while under contract with Detroit Edison.

In 2016, the Michigan Public Service Commission Staff formed a Technical Advisory Committee and subsequently published a report titled "Report on the Continued Appropriateness of the Commission's Implementation of PURPA" (Michigan Public Service Commission 2016) to research and review the "avoided cost" that drives the rates at which power is purchased. Appendix B of that report provided useful information for comparing the rates received by Boyce Hydro with those of other energy producers in Michigan, including each producer's total production in megawatt-hours during 2014, the energy rate and capacity rate at which Consumers purchased energy, an administrative charge that Consumers charged some of the producers, and the resulting total payment in dollars per megawatt-hour that each producer received in 2014. Comparing only the hydroelectric producers, which included Boyce Hydro and 12 others, the IFT calculated that the overall rates received by Boyce Hydro were about 10 percent lower than the average of the other producers. An increase of 10 percent in revenue would have amounted to roughly $200,000 of additional revenue per year for Boyce Hydro. As another comparison, Boyce Hydro provided the IFT with an analysis in which Boyce Hydro calculated that, if Boyce Hydro had been paid the same rates as the owner of Beaverton Dam from 2007 to 2019, the additional revenue during that period for Boyce Hydro would have been about $6 million, which averages to about $470,000 of additional revenue per year during those 13 years.

Financial documents spanning the period from 2007 to 2020 were furnished to the IFT by Boyce Hydro. It was not within the scope of the IFT investigation to perform an accounting audit of this financial information, and the IFT cannot attest to accuracy of the information provided. However, the information provided by Boyce Hydro indicates that the revenue generated by the four projects averaged roughly $2 million per year over the course of those 14 years, and the net income averaged about $380,000 per year after subtracting expenses and single-year improvements and equipment. The net income reduced to about $260,000 per year after subtracting principal payments on financed multi-year improvements and equipment, and further reduced to about $68,000 per year after subtracting the costs of the unfinanced portion of multi-year improvements and equipment. This net income varied year to year, and in some years, Boyce Hydro operated at a net loss. The Edenville project was the largest of the production facilities and accounted for between 50 percent and 60 percent of the revenue from the four Boyce Hydro projects; therefore, revocation of the Edenville license and inability to generate power at Edenville Dam essentially ensured that the four projects would operate at a net loss.

Boyce Hydro stated to the IFT that the rates previously negotiated with Consumers in 2007 were not sufficient to support comprehensive spillway upgrades meeting the FERC PMF requirement. Boyce Hydro further stated that, had it successfully been able to negotiate higher rates from Consumers or had it received rates comparable to the average of other independent power producers in Michigan, the additional revenue that would have been generated over time (roughly $200,000 per year) would have been sufficient to obtain a commercial loan to fund capital improvements such as increasing the spillway capacity at Edenville Dam, which would have partially closed the gap in meeting the FERC PMF requirement.

In 2019, the Independent Power Producers Coalition (IPPC), which Boyce Hydro was a member of at the time, successfully concluded an effort to obtain new rates from Consumers, which resulted in new PPAs between Consumers and each of the IPPC members. Boyce Hydro signed a new PPA on March 5, 2020, and returned it to Consumers for final signature; however, it was never executed by Consumers. Information provided by Boyce Hydro indicates that the new rate under the IPPC-negotiated contract no longer included an on-peak and off-peak component, and instead included an energy rate and a fixed capacity rate, the net effect of which resulted in a starting rate of about $0.070 per kilowatt-hour.

7.2 Human Factors Findings

The May 2020 failure of Edenville Dam was a result of interactions of numerous physical and human factors, beginning with the design and construction of the project in the 1920s and continuing throughout the life of the project until the failure. During the nearly 100 years the project was in place prior to failure, incorrect conclusions were drawn regarding the stability of the Edenville embankments and the capacity of the spillways. The potential for a non-extreme rainfall event to result in the lake rising by several feet to near the embankment crest was not recognized, and judgments and decisions were made that eventually contributed to the failure or to not preventing the failure.

The extent to which the numerous contributing factors combined and aligned to result in the failure primarily reflects both deficiencies in the construction of Edenville Dam and deficiencies in subsequent industry practices during the history of the project. The failure also secondarily involves an unfortunate combination of factors related to the variability of the dam along its length, the variations in the seepage behavior of the dam, the embankment stability analyses that were and were not performed, the hydrologic characteristics of the May 2020 storm event, and the timing of that storm event relative to planned upgrades to the Edenville gate hoist systems and spillways (see Appendix G, Section G-3.7 for details).

The IFT understands the natural desire to place "blame" for the failure. However, the IFT found that the failure cannot reasonably be attributed to any one individual, group, or organization. Instead, it was the overall system for financing, designing, constructing, operating, evaluating, and upgrading the four dams, involving many parties during the nearly 100 years of project history, which fell short in ensuring a safe dam at the Edenville site. All of the parties associated with the dams can be seen as having been acting "rationally" relative to their respective incentives, disincentives, responsibilities, and constraints. However, collectively, they were operating within a system that had conflicting interests and goals, resulting in the system having non-cooperative relationships. The net result was the failure of Edenville and Sanford Dams, which was a negative outcome for *all* of the parties.

Section 7.2 summarizes the IFT's findings regarding the primary human factors that contributed to the failures. Appendix G provides general background on what the term "human factors" involves, describes the human factors framework that was applied during the IFT's investigation of the Edenville Dam and Sanford Dam failures, describes the methodology the IFT used for investigating the human factors aspects of the failures, and provides an analysis of the human factors contributing to the failures.

7.2.1 Dam Design, Construction, Geotechnical Investigations, and Safety Reviews

7.2.1.1 Dam Design and Construction

The IFT found that the design of Edenville Dam was not substantially inadequate if the dam had been built according to the design plans and construction specifications. However, the actual construction of the dam deviated substantially from the design plans and construction specifications, apparently with the knowledge of the dam designer, and as a result, the safety margins of the dam were below average even compared to dams of similar type and size built from the 1910s through the 1930s.

This manifested in the lack of a consistent upstream impervious zone in the embankment fill, lack of compaction of the fill (and resulting low density of the soil), and downstream slopes that were steeper than the 2H:1V design slopes in some locations (including a slope of about 1.6H:1V in the upper part of the failure section and an average slope of about 1.8H:1V at that section), and possibly compromised functioning of some of the embankment drains.

There are several possible reasons for the deviations from the design plans and construction specifications, including limited availability of suitable construction equipment, limited availability of suitable materials in borrow sources, insufficient staffing for construction and construction inspection, high costs or schedule pressures, and/or a desire to complete the construction ahead of schedule in order to start generating power and revenue sooner.

These deviations substantially compromised the safety margins of the embankment stability with respect to both conventional and static liquefaction stability failure modes. These inadequate stability safety margins physically set the stage for the dam failure when the lake rose to about 5.5 feet higher than the normal lake level in May 2020, which was about 3 feet above the previous record high lake level in 1929. When the lake rose to this level in May 2020, static liquefaction was triggered, causing a rapid liquefaction flow instability failure. The instability failure was unforeseen, but not unforeseeable.

7.2.1.2 Geotechnical Investigations and Analyses

Static liquefaction, the mechanism for failure of Edenville Dam, was not considered in any of the geotechnical analyses completed for the dam. This is not surprising, because this PFM has not commonly been considered for hydroelectric dams, or other water storage dams or flood management dams. In fact, references and guidance documents for dam design and analysis indicate that drained strengths should be used for sands and silty sands for all load conditions except rapidly applied loads, such as earthquakes. This guidance implicitly excludes consideration of static liquefaction. All previous stability analyses for Edenville Dam were based on drained strength parameters, consistent with the published guidance. In the past decade or so, practitioners in tailings (mine waste) dams field have begun to recognize static liquefaction as a credible PFM, but that recognition has not transferred to common practice in water dams.

When evaluating the stability of the embankments over a period of about three decades, starting with the FERC regulation of the dam, the engineers involved with the dam, including both consultants and regulators, collectively did not fully account for the available information related to the as-built dam construction and the potential for variability along the approximately 6,000-foot length of the embankments. This was despite the fact that FERC and one of the consultants raised concerns about these issues in 1993 and 1994 respectively, that construction photos were found in which none of the photos showed evidence that the embankment fill had been compacted, and that low blow counts of less than 10 bpf had been observed in many soil borings.

The embankment stability was analyzed at only two cross-sectional locations, using somewhat unconservative soil parameters relative to the low blow counts, and at least one of these analyses resulted in calculated factors of safety that were less than the FERC-specified minimums. The two locations where embankment stability was analyzed were among the tallest locations along the embankments, but the average downstream slopes in these locations were not as steep as the average slope in the failure section. Seepage was a concern at the cross-sections that were analyzed, and there was a tendency on the part of the engineers to assume that stability was only a concern in locations where significant seepage was observed. The location of the embankment which failed happened to be a location where limited seepage was observed during the history of project, possibly because there were differences in how this section of the embankment had been constructed as compared to the other Edenville Dam embankments.

From 2004 to 2014, embankment overlays were generally placed in seepage locations and locations where the embankment stability factors of safety were calculated to be inadequate. However, the further step of checking the embankment stability at other cross-section locations was not taken. In this regard, the IFT found that if embankment stability had been analyzed for the failure section, assuming the same somewhat unconservative soil parameters as used in the previous analyses at other locations of the dam, the factors of safety for the downstream slope would not have met FERC requirements. Therefore, embankment stability analysis for the failure section would likely have led to remedial actions, such as construction of an overlay for the downstream slope, which would have increased the factors of safety for conventional instability failure. Although such analyses would not have been based on the actual failure mechanism that occurred in May 2020, they may have resulted in action that would have prevented the failure. The flatter slopes or buttresses that would have been needed to increase conventional stability factors of safety to meet FERC guidelines would have reduced the static shear stress ratios in the loose sands in the embankment sufficiently to have likely prevented the May 2020 failure.

As discussed in Section 7.1.5, surprisingly, static liquefaction *was* checked for Secord Dam around 2001 by Barr (2001), who was a consultant for FERC Part 12D dam safety reviews for Secord Dam, Smallwood Dam, and Sanford Dams Dam, but not for Edenville Dam. The Barr team performing the Part 12D dam safety review for Secord was a different team from the one that performed the other Part 12D reviews. Barr concluded that static liquefaction was a concern at Secord Dam and recommended that remedial action be taken. The particular engineer at Barr who identified the static liquefaction failure mode for Secord Dam happened to be a recent PhD graduate of the University of Illinois Urbana-Champaign, and he was aware of this failure mode because research on this topic had been recently done at that university (his PhD thesis advisor was an expert on this topic).

If that same consultant team had performed an embankment stability analysis for Edenville Dam, it appears likely that the potential for static liquefaction at Edenville Dam would have been identified, and it is possible that remedial action would have been taken and the failure prevented if this failure mode had been evaluated along the full embankment alignment. However, it is also noteworthy that, as part of the 2005 Part 12D review, A. Rieli and Associates evaluated Barr's concern regarding the potential for static liquefaction at Secord Dam and argued that Barr's analysis was inaccurate, that the soils at Secord Dam were not susceptible to static liquefaction, and that therefore no remedial action was necessary to prevent a static liquefaction failure at that dam (A. Rieli and Associates 2006). Had a similar scenario played out for Edenville Dam, it is possible that the static liquefaction concern would have been identified, yet not remedied because of conflicting engineering opinions.

7.2.1.3 Dam Safety Reviews

The intent of the FERC Part 12D process is to perform a comprehensive "deep dive" review of a dam, focusing on safety, including review of design, construction, analyses, operations, and performance monitoring. Six Part 12D reviews were completed from 1991 to 2015 for Edenville Dam. As has often been the case in the dam industry with Part 12D reviews, the intent of a comprehensive review was not met by these six Part 12D reviews. Instead, each Part 12D review tended to rely upon and defer to the findings in the prior reviews and engineering analyses without a detailed assessment of their appropriateness, and focused on updating the last review to reflect any observed changed conditions.

A key example of this deference was the reliance on the embankment stability analysis factors of safety calculated in the original 1991 Part 12D CSIR report, even after FERC questioned the validity of some of the assumptions on which that analysis was based. The consultant who prepared the second Part 12D report in 1994 noted the questions raised by FERC and recommended that a subsurface boring program be undertaken in the summer of 1995, but neither the subsequent Part 12D consultants nor FERC itself followed up on those questions, and the recommended boring program was never completed.

Similarly, the FERC process for PFMA was intended to provide another means to evaluate dam risks and provide a framework in which to facilitate comprehensive review of data. A PFMA was performed for Edenville Dam in 2005 and updated in 2010 and 2015. Like the Part 12D reviews, the PFMAs in some aspects also did not accomplish the intent of a comprehensive review, mainly because they relied on the findings of previous engineering analyses performed by others, particularly the 1991 embankment stability analysis in the first Part 12D report, and because PFMs which previously "ruled out" were truly ruled out in the sense that they were not re-examined during the subsequent PFMA reviews. The 2005 PFMA also resulted in a questionable judgment that a spillway capacity sufficient to "safely pass a flood event roughly equal to a 200-year flood" indicated that overtopping was "less likely" rather than "more likely." While it was recognized that the spillway capacity needed to be increased in order to meet the FERC PMF requirement, this judgment of the overtopping risk may have contributed to the view that spillway capacity increase was not needed on an urgent basis. A formal risk analysis may have led to a different conclusion regarding how urgently the spillway capacity needed to be increased.

These gaps between intent and practice in FERC Part 12D and PFMA reviews were identified in the forensic investigation report for the 2017 Oroville Dam spillway incidents (France et al. 2018), and FERC has recently addressed this gap by revising its guidelines to require that comprehensive reviews be performed for all high-hazard dams regulated by FERC (FERC 2021) and that these reviews include a review of previous analyses.

There is no such requirement to periodically perform comprehensive reviews, nor is there a process analogous to the FERC Part 12D and PFMA processes, for most state-regulated dams in the United States. In the opinion of the IFT, lack of periodic comprehensive reviews of state-regulated high-hazard dams in the United States represents a gap in dam safety regulatory practice.

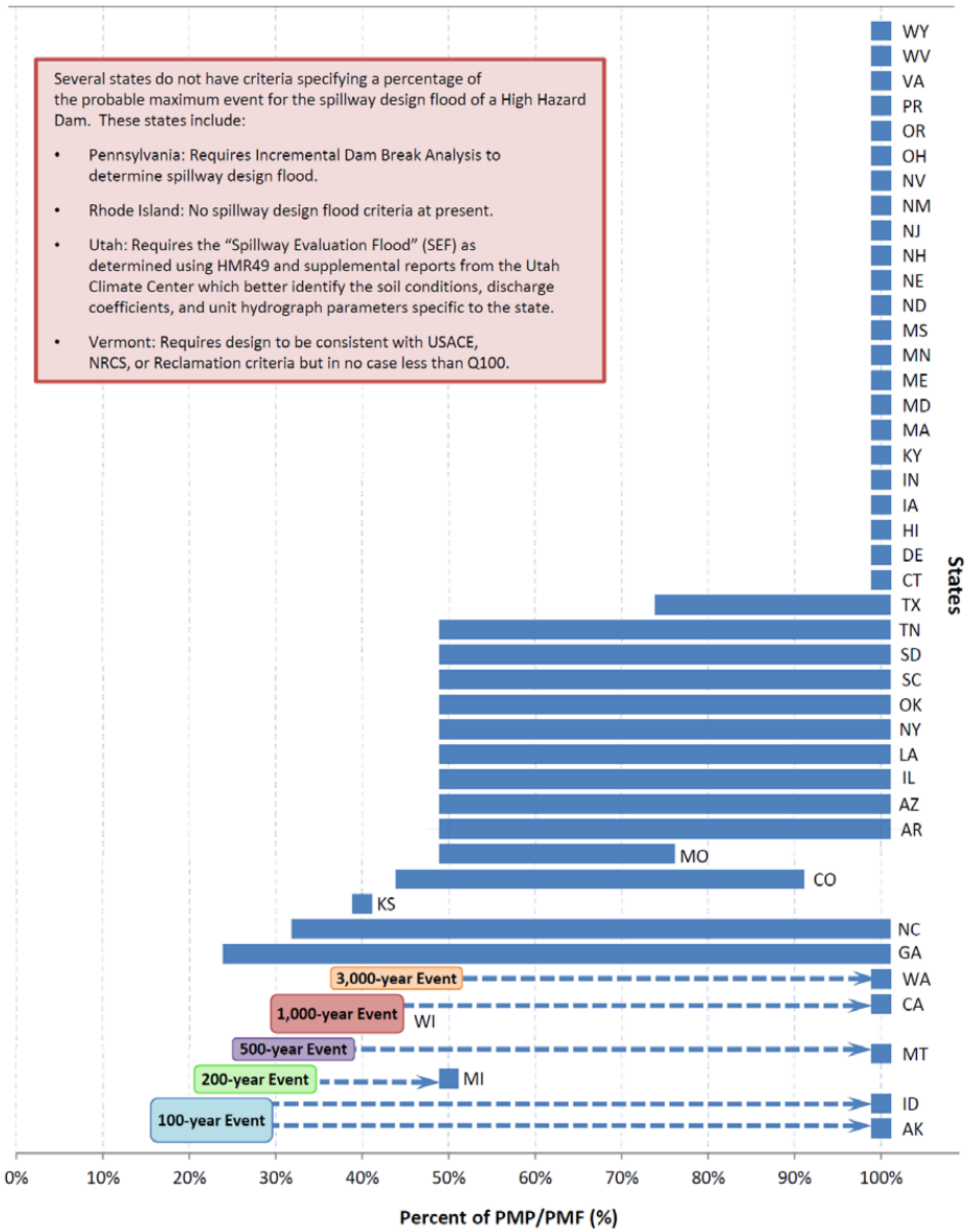
7.2.2 Hydrology, Hydraulics, and Lake Levels for Edenville Dam

7.2.2.1 PMF Studies and Inadequacy of Calculated Spillway Capacity

From 1978 to 2020, numerous hydrologic/hydraulic analyses were performed by multiple engineering consultants to estimate the PMF and the spillway capacity for Edenville Dam. These calculated PMF estimates ranged widely from 56,200 cfs to 125,500 cfs, due to varying assumptions with regard to the rainfall amounts and positioning of the PMP and the watershed characteristics (unit hydrograph parameters and the hydrologic losses).

Passing the PMF without overtopping the dam was the FERC spillway capacity requirement for Edenville Dam, since Edenville Dam is a high-hazard dam. However, once the FERC license was revoked in 2018 and EGLE became the regulator, the spillway capacity requirement became the “half PMF” discharge, which EGLE defines as the flood discharge resulting from half of the PMP. For Edenville Dam, the EGLE requirement results in a peak inflow that is about 40 percent of the PMF discharge.

It is noteworthy that, if Edenville Dam had been less than 40 feet in height, the EGLE spillway capacity requirement would have been the 200-year flood, which corresponds to about 80 percent of the “half PMF,” based on the estimation that the “half PMF” is approximately 40 percent of the PMF and using the PMF computed by Mill Road (Mill Road 2011a). Edenville Dam was only moderately taller than the 40-foot threshold, with a maximum height of 52 feet, and that maximum height occurred only adjacent to the two spillways. Based on this height consideration, the EGLE spillway capacity requirement for Secord Dam would have been the “half PMF,” whereas the EGLE requirement for Smallwood Dam and Sanford Dam would have been the 200-year flood, though the EGLE requirements did not apply to Sanford Dam, Smallwood Dam, and Secord Dam at the time of the May 2020 failure because they were still subject to the FERC full PMF requirement. Comparatively, the EGLE spillway capacity requirements are generally on the low side compared to most U.S. states’ requirements, as shown in Figure 7-2 (FEMA 2012). Several states have updated their guidelines since this figure was developed in 2012, but the relative comparison for Michigan is still judged to be valid.



Note: For probability events (such as the 100-year flood), the corresponding percentage of the probable maximum event varies significantly in different areas of the country. The plotted location of probability events do not, therefore, represent the corresponding PMP/PMF percentage.

Figure 7-2: Range of Spillway Design Flood Criteria for New High Hazard Dams (FEMA 2012)

The calculated spillway capacities were all much lower than the various calculated PMF flood discharges, and therefore it was recognized by all parties that the spillway capacity was deficient relative to the FERC PMF requirement. Prior to EGLE regulating Edenville Dam, it was generally believed that the spillway capacity was approximately 50 percent of the PMF.

Despite these spillway capacity deficiencies relative to the regulatory requirements, based on the following factors, the IFT does not believe that any of the parties associated with the dam perceived the risk of overtopping to be high enough to require urgent upgrading of the spillway capacity:

1 The lake had risen by more than 1.5 feet above the normal lake level only once (2.5 feet in 1929) over more than nine decades of operation, with a rise of only 0.5 foot during the “great flood” of 1986.

2 The wide variation in spillway design requirements of FERC, EGLE, and other states created ambiguity regarding what spillway capacity should be considered truly deficient from a safety standpoint. The FERC PMF requirement may have been viewed as being overly conservative, as it typically corresponds to a return period of many thousands or even millions of years, and therefore the annual probability of a PMF event occurring is considered to be extremely low.

3 The estimates for the magnitude of the PMF varied over a wide range, which created uncertainty regarding the magnitude of the PMF.

4 The EGLE “half PMF” requirement appeared to be close to being met.

5 The 2005 PFMA team concluded that the spillway capacity was sufficient to “safely pass a flood event roughly equal to a 200-year flood,” and this finding was categorized as making overtopping “less likely” rather than “more likely.” Therefore, even a spillway capacity in the 200-year flood range apparently was perceived by the PFMA team as reflecting a low overtopping risk.

Consistent with all of the above, the spillway capacity was described as being “robust” by one of the regulators in an interview with the IFT.

Considering all of this, if some form of formal risk analysis had been done, accounting for the historical and seasonal behavior of the watershed, the potential for a non-extreme storm to result in an unusually high lake level and possibly overtopping the embankment may have been recognized. In its review of a 2016 design report prepared by Boyce Hydro, the BOC recommended that a risk analysis be done, but the IFT did not find any documentation of discussions or a decision related to this recommendation (see Appendix C).

7.2.2.2 Inflows into Wixom Lake

While the mechanism of the dam failure was fundamentally an embankment instability failure, a key contributing factor to the Edenville Dam failure was that Wixom Lake reached a level about 3 feet higher than the previous pool of record, leaving about 1 to 1.5 feet of freeboard to the crest of the embankment in locations where the dam crest had not settled. As noted above, the previous record high lake level was 2.5 feet above the normal lake level, which occurred in 1929, a few years after construction was complete. The high lake level in 1929 may not have come as a surprise at the time, since the 1928 flood in the previous year resulted in failure of both the Chappel Dam and the Schulz Dams on the Cedar River (Turner 2011), which added to the flow entering Wixom Lake. A report prepared by the Chief of Engineers, U.S. Army Corps of Engineers (USACE) in 1932 (State of Michigan 1932) noted that multiple dam failures had previously occurred in the watershed.

As noted above, after 1929, for over nine decades of operating history, the lake never rose more than 1.5 feet above the normal lake level until it rose by about 5.5 feet in May 2020. By comparison, even the “great flood” of 1986 resulted in the lake rising by only 0.5 foot. Given the long history of relatively limited rise in the lake levels and the non-extreme amount of rain in the days before the May 2020 failure (about a 25- to 50-year rainfall event), the amount that the lake rose on May 19 came as a surprise to Boyce Hydro, EGLE, and FERC, especially since, the day prior to the failure (May 18), all gates in the system had been opened to the maximum height they were going to be opened.

The apparent reason for the unprecedented rise of the lake on May 19, 2020, was a set of “perfect storm” conditions, where more than a week of overnight temperatures below freezing (at or near record lows) was followed by warming, then 2 days of moderate rain, which occurred several days before the failure, and then finally heavy rain the day before the failure. The IFT believes that the period of cold temperatures resulted in partial freezing of the ground near the surface in parts of the watershed (especially forested swamps and wetlands), which reduced the capacity of the ground to allow infiltration of rain. As a result, the 2 days of moderate rain saturated the ground above the frozen zone, and then an unusually high percentage of the heavy rain the day before the failure became surface runoff, resulting in a large inflow into the lake and the outlier extent of rise in the lake.

While the amount of rise of the lake on May 19 was unforeseen, it was not entirely unforeseeable. While the IFT found that the potential for non-extreme storms to produce unusually high runoff during the cold season (until about the end of May) due to ground freezing and saturation in the Edenville Dam watershed was not recognized by the dam owners, their engineering consultants, FERC, or EGLE, there were several warning signs that such an event could occur:

- Historically, higher lake levels usually occurred in the winter and spring, even though there were sometimes greater rainfalls at other times of the year.
- Reports from USACE in 1932 (State of Michigan House of Representatives 1932) and Schrouder et. al. (2009) described high runoffs due to ground freezing in the watershed.
- In April 2014, there was an overtopping failure of Wraco Lodge Lake Dam, which is located just outside the Edenville Dam watershed. EGLE investigated this failure and attributed the high runoff mainly to frozen ground. The runoff was estimated to have a return period of at least 200 years from a 25- to 50-year rainfall.

If the engineers involved with Edenville Dam over the past few decades had done a historical review and “connected the dots” on these warning signs related to the behavior of this watershed, they might have recognized that during the cold season, there was potential for a non-extreme rainfall to produce a disproportionately large runoff that could result in sufficient rise of the lake to overtop the dam, as nearly occurred in May 2020.

The IFT believes that this recognition did not occur mainly for three reasons. First, many engineers involved in the project, particularly engineers whose offices were not in or near Michigan, may not have viewed April and especially May as part of the cold season for this area of Michigan.

Second, relatively few engineers in the dam industry had significant experience in modeling the behavior of watersheds in cold regions under conditions that could result in partially frozen ground and consequent ground surface saturation and high runoff. Instead, even in cold regions, most engineers were focused on predicting the PMF flood, which generally has an extreme amount of rainfall (often tens of inches), which typically does not occur during the cold season and therefore the potential effects of frozen ground do not usually come into play when estimating the PMF.

Third, for Edenville Dam, there was a focus on whether the FERC PMF requirement and later the EGLE "half PMF" requirement were met, both of which corresponded to floods much larger than the May 2020 flood, which was approximately a 100- to 200-year event with respect to runoff. As is often the case in the dam industry, meeting the regulatory requirements appears to have been viewed as a "pass/fail" situation, resulting in some lack of attention to the *amount* by which the regulatory requirements were not met and the safety risks that entailed.

In the case of Edenville Dam, given the available spillway capacity, if more attention had been paid to gauging the level of risk, specifically the overtopping risk, that may have led to a careful study of the watershed behavior across the seasons of the year for a range of rain events. Such a study, in turn, may have led to a recognition of the potential for non-extreme rain to produce high runoff, and therefore there might have been a greater sense of urgency to at least partially increase the spillway capacity. In this regard, it is plausible that if a partial spillway capacity upgrade had been constructed before May 2020, the resulting reduction in the lake level rise might have been sufficient to prevent the dam embankment instability failure in May 2020. However, the inherent risk of an embankment instability failure under some future high lake level, albeit for a rarer storm, would have remained unless the Edenville left (east) embankment was modified.

7.2.2.3 Effects of Gate Openings

The *actual* deficiency in spillway capacity relative to both the FERC and EGLE requirements was significantly larger than was calculated, because the gate openings assumed in the spillway capacity analyses were larger than the openings to which the dam operators believed the gates could be safety opened in May 2020 with limited use of A-frames, while avoiding excessive risk of damage to the gates or injury to the operators. During the May 2020 flood event, the gates were in fact opened to about 7 feet. Since the gates at Edenville could not be opened to the 10-foot fully open height, none of the spillway rating curves used in the various hydraulic analyses were representative of the available spillway capacity for the full range of lake levels.

The IFT found that the spillway capacity for Edenville Dam, assuming gate openings of about 7 feet, was about 18,100 cfs at the minimum dam crest elevation (682.1 foot), which is considerably less than the EGLE "half PMF" discharge for all of the previous PMF estimates. By comparison, in the Ayres 2021 report on the Design Flood Hydrologic Analyses (Ayres 2021), the 200-year flood, i.e., an AEP of 0.005, was calculated to have a peak inflow of 25,400 cfs and an outflow of 20,400 cfs with a freeboard of 0.1 foot (lake level 682.0 feet) to the dam crest. However, this analysis was based on assumed gate openings of about 9 feet used in the spillway rating curves derived from calculations provided by GEI (GEI 2020). Specifically for the May 2020 flood, the IFT calculated that, if it had been possible to open the gates to at least about 10 feet, the maximum lake level on May 19 would have been lowered by about 1 foot, a lake level at which the failure may or may not have occurred.

Modifications to the gate hoist system that would have enabled safely opening the gates at least 10 feet were planned for late 2020, at a cost of several hundred thousand dollars, and it is unfortunate that the May 2020 flood occurred before these modifications to the gate hoist system were completed. Modifications to the gate hoist systems at Secord, Smallwood, and Sanford Dams had already been completed, and the IFT found that similar modifications at Edenville Dam had likely been deferred because there was an intent to substantially modify the spillways. The spillway modifications would have resulted in the modified gate hoist systems eventually being discarded, resulting in a "waste" of hundreds of thousands of dollars of investment to address what was perceived as a very low risk of overtopping.

If and when this spillway modification had been completed, which would likely have been within several years after FLTF took legal ownership of the dams, it is likely that the spillway capacity and the ability to pass a range of floods would have been greatly increased, resulting in a lower lake level rise during floods. Therefore, it is unfortunate that an unusual event like the May 2020 flood occurred before the spillway capacity was increased.

7.2.2.4 Effects of FERC Actions

The situation of dam spillway capacity not meeting regulatory requirements, and the dam owner not having sufficient funds to fully meet the regulatory requirements, presents a challenge for both the dam owner and the regulator. For Edenville Dam, when the FERC license was revoked in 2018 and EGLE became the regulator, agreements were already being established for sale of the dams to FLTF. FLTF would have had sufficient funds to upgrade the spillway capacity to meet EGLE (or FERC) requirements from grants and collection of annual fees from the lakefront property owners through the SAD. However, the spillway improvements would likely not have been constructed for several years, so the unfortunate events of May 2020 would likely still have occurred.

For the three decades under FERC regulation, the IFT concluded that none of the three dam owners appeared to have had sufficient funds to upgrade the spillway capacity to meet FERC requirements. As a regulator during this period, FERC had discretion to formally evaluate the financial capacity of the dam owners, however FERC's position and practice was that the mandate to enforce its safety regulations was not dependent on a licensee's financial capacity, and licensees were expected to comply regardless of the costs. FERC typically evaluated the financial capacity of owners during initial licensing and during subsequent license transfer processes, but not typically during the term of a license.

The following is the IFT's analysis of FERC's options to exert pressure on the dam owners to at least partially upgrade the spillway capacity while still maintaining a license:

- *Impose fines, which would accumulate over time* – This option would have further reduced the funds available to the dam owners to upgrade spillway capacity and therefore would not have been a good option to exercise. Additionally, FERC did not have discretion on how to use the funds created via fines, and thus they could not simply be applied toward addressing compliance issues.
- *Order lowering of Wixom Lake to the spillway crest* – If the spillway gates had been kept open, the lake level would have been lowered to about 6 to 8 feet below the normal lake level. This option, by itself, would have had a limited benefit with respect to reducing the chance of overtopping the dam. If this option, by itself, had been exercised, the peak lake level would have been about 0.2 foot lower than the level at the time of the May 2020 failure. This option would have also impacted the ability to generate power, and therefore would have reduced the funds available to the dam owners to upgrade spillway capacity. This option would also have impacted recreational use of the lake as well as property values, and possibly have caused adverse environmental impacts. Considering these factors, this would not have been a good option to exercise. Consistent with this, the IFT was told that FERC did evaluate this option and judged it to not be a good option to order the dam owner to undertake.
- *Order lowering of Wixom Lake to drain the lake* – If the lake had been fully drained, this would have had a significant impact with respect to reducing the chance of overtopping the dam. However, the existing low-level sluiceways were inoperable. It would have been necessary to refurbish the sluiceways or construct new low-level outlets, either of which would have been costly and likely required draining the lake for construction. In addition, this option would have

eliminated power generation at Edenville Dam, which was the source of about half of the total revenue from the four dams. Therefore, this option would have greatly reduced the funds available to the dam owners to upgrade the spillway capacity. This option would have also eliminated recreational use of the lake, substantially impacted property values, and possibly have caused adverse environmental impacts.

While this option would have provided some benefit with respect to reducing overtopping risk, the IFT believes that this option would have been highly opposed by the local communities, the lake associations, EGLE, and the dam owners, and therefore was not a viable option. Moreover, it was FERC policy that a lake would only be ordered to be drained under emergency conditions, not as an interim or permanent risk reduction measure, and therefore it would be up to the dam owner to propose draining a lake as a risk reduction measure.

- *Order ceasing generation of power* – Again, due to the inability to generate power, this option would have eliminated about half of the total revenue from the four dams. Therefore, this option would have greatly reduced the funds available to the dam owners to upgrade the spillway capacity. This option would have also eliminated the ability to augment the spillway capacity by releasing water through the powerhouse; the powerhouse provides about 2,000 cfs of maximum flow, which is about 5 to 10 percent of the spillway capacity. FERC had exercised this option of a cease generation order with Boyce Hydro in 2017, but the order was "stayed" (cancelled) in 2018 after Boyce Hydro explained that controlling release of water solely with the spillway gates created operational difficulties in the winter when the gates were subject to icing. Considering these factors, this was not a good option to exercise.
- *Order controlled breach of the dam under emergency conditions* – This option would drain the lake and alleviate the risk of dam failure. However, this option only applied under emergency conditions, not as a risk reduction measure under normal non-emergency conditions. The IFT's understanding is that emergency conditions never occurred at Edenville Dam while FERC was the regulator.

Since the above options were not viable or good options for the reasons noted, FERC's "last resort" was to revoke the Edenville license, which it finally did in 2018, after three decades of being unable to get any of the three dam owners to upgrade the spillway capacity to meet the PMF requirement, though Boyce Hydro did study numerous options to increase spillway capacity and had initiated construction for one of them. The IFT was told that only about 1 percent of the dams under FERC regulation have a spillway capacity that does not meet FERC's deterministic requirements, and prior to the revocation of the Edenville Dam license, FERC had never before revoked a license primarily because of inadequate spillway capacity.

It turned out that the license revocation had a net negative effect from a dam safety standpoint because it had the same effects as ordering the dam owner to cease generating power, namely elimination of about half of the revenue from the four dams and loss of the ability to release up to 2,000 cfs of water through the powerhouse. The IFT found that if it had been possible to release water through the powerhouse during the May 2020 event, the lake level would have been lowered by up to about 0.8 foot, which may or may not have prevented the embankment instability failure. Moreover, by revoking the license, the spillway capacity requirement was reduced by more than 50 percent when the dam reverted to EGLE regulation with a spillway requirement of "half PMF," and therefore the incentive to meet the industry "gold standard" of being able to pass the PMF flood was greatly reduced. It is important to note that this change in deterministic regulatory requirement did not significantly change the physical hydrologic risk of the dam, since it did not result in any changes in the inflow to the lake, nor any changes in the spillway capacity, though the possibility of releasing water through the powerhouse was eliminated under EGLE regulation.

With the benefit of hindsight, and considering that agreements were already being established to sell the dams to FLTF before the license was revoked, the IFT believes the best course of action for FERC would have been to not revoke the license and instead continue to work with both Boyce Hydro and FLTF to perform an overtopping risk analysis and develop an acceptable plan and schedule to upgrade the spillway capacity to meet the PMF requirement. This approach would have enabled Boyce Hydro and FLTF to continue generating power and revenue, which would have increased the financial capacity to fund the eventual spillway capacity upgrade. It would have also enabled release of water through the powerhouse during the May 2020 event, which, as noted above, may or may not have prevented the dam failure during that event.

It is noteworthy that the IFT was told in interviews that in the months leading up to the license revocation, FLTF was not able to have discussions about the planned ownership transition with FERC and present its arguments for why the license should not be revoked because it was FERC’s policy to have communications only with the licensee, which was Boyce Hydro. This is in sharp contrast to EGLE’s policy, when they became the regulator, of viewing Boyce Hydro and FLTF as both functioning in an owner role, even if only Boyce Hydro was the legal owner. Therefore, EGLE was willing to communicate with both parties, although EGLE’s communications regarding technical issues were primarily with FLTF and its consultants, rather than with Boyce Hydro.

Finally, decommissioning a dam (removal or controlled permanent breach) is an option to reduce dam safety risks posed by a dam that does not meet regulatory requirements when the required upgrades are infeasible or otherwise not preferred. The option to impose a permanent breach of the dam under non-emergency conditions was available to EGLE, but was not available to FERC. Under FERC regulation, the owner would need to propose decommissioning as its preferred alternative to rehabilitation.

7.2.2.5 Effects of EGLE Actions

As noted in Section 7.1.4.3, EGLE was the dam safety regulator for Edenville Dam for less than 2 years before the dam failed. Once EGLE became the dam safety regulator in September 2018, EGLE reviewed records related to the dam and by early 2020 had concluded that the dam did not have sufficient spillway capacity to meet the EGLE “half PMF” requirement. EGLE would have presumably pressed Boyce Hydro and FLTF to upgrade the spillway capacity, which would have likely taken at least a few years to design and construct, but the dam failed a few months later in May 2020.

Given that the high lake level on May 19, 2020 was a physical contributor to the embankment instability failure, it is noteworthy that a regulator in most dam safety programs would not typically raise questions about embankment stability unless there were specific physical observations suggesting problems, such as shallow slumping, cracking, or new or increased seepage at a downstream slope. In the case of Edenville Dam, the dam had already gone through a PFMA, several geotechnical investigations, and six FERC Part 12D inspections and evaluations, the most recent being in 2015. The conclusions by the engineering consultants and FERC were that there were no significant embankment stability concerns which had not already been addressed by embankment overlays and drain modifications. Therefore, in the opinion of the IFT, even if all the recommendations in ASDSO’s peer review of the EGLE dam safety program (ASDSO 2020) had been implemented before 2020, that would not likely have led to subsequent decisions or actions by EGLE that would have resulted in preventing the May 2020 failure.

During the winters of 2018-2019 and 2019-2020, Wixom Lake had been lowered about 6 feet below the normal lake level by keeping the spillway gates open, and there were disputes about the rationale and impacts of doing this among Boyce Hydro, FLTF, and EGLE. As discussed in Section 5 and Appendix F1, the IFT found that, if the lake had been kept lower by this amount until the May 2020 flood occurred, the effect on the lake level on May 19, by itself, would very likely have been too small to prevent the dam failure. Therefore, the IFT did not investigate the issues related to these winter drawdowns in depth.

7.2.2.6 Effect of Smallwood Dam Modifications

Decisions regarding modifications to the four dams had to be prioritized among the dams, given that they had one owner and were in a series on the same river. One such decision was that modifications were made to Smallwood Dam, initially in 1999, and then again in 2001 and 2016-2017, as described in Section 2.4. The combination of all the modifications effectively created a two-level auxiliary spillway at Smallwood Dam. Water flowed through this auxiliary spillway during the May 2020 event and the dam did not overtop.

Since Smallwood Dam did not fail during the May 2020 event, the IFT did not perform detailed hydrologic and hydraulic analyses for this dam to evaluate "what if" scenarios. However, the IFT believes that the creation of the auxiliary spillway at Smallwood Dam may have prevented overtopping failure of that dam during the May 2020 flood. A failure of Smallwood Dam would have possibly caused an overtopping failure of the downstream Edenville and Sanford Dams (this scenario was considered during the 2019 EAP exercise), possibly with some loss of life and certainly with increased property damage. Therefore, the decision to modify Smallwood Dam appears to have been prudent and may have prevented the failure consequences from being worse than they were.

7.2.3 Project Ownership, Financing, and Relationships

The situation with these four dams was one where multiple parties were interacting with each other and each party was pursuing its own goals, but the decisions of each party were influenced by what it thought the other parties would do and what the other parties actually did. These parties included the three dam owners, the dam owners' engineering consultants, FERC, EGLE, FLTF, FLTF's engineering consultants, Consumers, Gladwin and Midland Counties, the lakefront property owners, lake users who did not own lakefront properties, and the property owners downstream of the lakes.

These parties had diverse and conflicting goals, the relationships among the parties were a mix of cooperative and non-cooperative relationships (largely because of the conflicting goals), the parties had diverse obligations and abilities, the information available to the parties varied widely (there was no structure that enabled pooling of information, and there were restrictions in sharing information for security reasons), and the parties were exposed to different risk profiles. A summary of this situation, along with the inferred "rational" preferences and courses of action for each party, is presented in Table 7-2 below.

Table 7-2: Project Parties and Circumstances

Party	Benefits from Project	Risks from Project	Inferred "Rational" Preferences and Courses of Action
Dam owners	Relatively small profit margin	Potential for annual financial losses if project costs increase or revenues decrease; large financial loss and liability, and likely bankruptcy if dam failure occurs	• Make minimum financial investments needed to make dam failure very unlikely, maintain FERC licenses and ability to continue generating power, and meet EGLE environmental regulatory requirements as needed to avoid penalties • Solicit funding from other project beneficiaries to help pay for project costs, including upgrading the spillway capacity • Negotiate a higher rate from Consumers to increase project revenue and profits • Sell the dams for a reasonable profit, or at least not at a significant loss, if a suitable buyer can be found
Dam owner's engineering consultants	Engineering fees	Potential liability and reputational loss if dam failure or environmental damage occurs	• Make recommendations for managing project risks without imposing excessive costs on the dam owner, which could result in the dam owner discontinuing use of the consultant's services • Limit engineering scope and fees to what the dam owner will accept • Perform engineering services efficiently, which may include relying on previous work or analyses performed by other consultants when applicable and where risk of doing so is deemed reasonable
FERC	None, beyond a general mission to support production of power in the United States	Potential liability and reputational loss if dam failure occurs	• Enforce FERC regulations and avoid the scenario of a dam failure while the dam is under FERC regulation: first give the dam owners a reasonable amount of time to meet FERC regulatory requirements; then exert increasing pressure to meet the requirements with threats of various orders; finally, carry out cease generation orders, financial penalties and/or ordering lake level restriction, and, as a last resort, license revocation
EGLE dam safety division	None	Potential liability and reputational loss if dam failure occurs	• Enforce EGLE dam safety regulations and avoid the scenario of a dam failure while the dam is under EGLE dam safety regulation: first give the dam owners a reasonable amount of time to meet EGLE regulatory requirements; then exert increasing pressure to meet the requirements; finally, as a last resort, have the dam breached or removed by the State of Michigan and attempt to recover the cost of doing so from the dam owner through legal action
EGLE environmental division	Preservation of environmental benefits associated with the presence of the lakes	Potential liability and reputational loss if environmental damage occurs due to dam failure or lowering of the lakes	• Enforce EGLE environmental regulations

Party	Benefits from Project	Risks from Project	Inferred "Rational" Preferences and Courses of Action
FLTF	Preservation of the lakes and the associated recreational, aesthetic, and property value benefits to lakefront property owners and lake users	Liability, and property value and reputational loss if dam failure occurs	• Purchase the dams at the lowest possible cost • Make improvements to the dams at the minimum cost needed to meet regulatory requirements and reasonably address dam safety concerns so that the annual costs passed on to property owners are minimized • Ensure that lake levels are maintained so that property owners derive the benefits of the lakes
FLTF's engineering consultants	Engineering fees	Potential liability and reputational loss if dam failure or environmental damage occurs	• Make recommendations for managing project risks without imposing excessive costs on FLTF, which could result in FLTF discontinuing use of the consultant's services • Limit engineering scope and fees to what FLTF will accept • Perform engineering services efficiently, which may include relying on previous work or analyses performed by other consultants when applicable and where risk of doing so is deemed reasonable
Gladwin and Midland Counties	Increased property taxes collected due to the presence of the lakes	Reduction in property taxes collected if lakes are lost due to dam failure or draining the lakes	• Continue to collect increased property taxes made possible by the presence of the lakes, without helping to pay for the costs associated with the dams
Consumers Energy (Consumers)	Profits from sale of energy purchased from the dam owners	Loss of potential profit if the dams do not generate power due to discontinuing power generation or dam failure	• Negotiate the lowest possible energy rates to be paid to the dam owners to maximize the profit on sale of energy generated by the dam
Lakefront property owners	Increased property values, recreational value of the lakes, and aesthetic value of the lakes	Loss of lake benefits if the lakes are lost due to dam failure or draining the lakes	• Continue to derive the substantial benefits provided by the lakes, without helping to pay for the costs associated with the dams • Report unusual activity or concerns related to the dams to authorities
Lake users who do not own lakefront properties	Recreational value of the lakes	Loss of recreational value if the lakes are lost	• Continue to derive the recreational benefits provided by the lakes without helping to pay for the costs associated with the dams
Property owners downstream of the lakes in the breach inundation zone	Access to recreation on the lakes and economic benefits from the presence of the lakes	Likely property damage or destruction and potential loss of life if dam failure occurs	• Rely upon the dam owners and regulators to take actions that make the likelihood of dam failure "as low as reasonably practicable"

EGLE = Michigan Department of Environment, Great Lakes, and Energy
FERC = Federal Energy Regulatory Commission
FLTF = Four Lakes Task Force

Comparing the inferred "rational" preferences and courses of action for each party with what each party actually did during the history of the project (see Section 7.1), it is not surprising that each party did generally behave "rationally" – in other words, all of the parties behaved pretty much as they should have been expected to behave from the perspective of their own circumstances and goals, and what they expected the other parties to do.

Weighing the overall aggregate benefits (nonspecific to any of the parties) from the project (e.g., recreational value, aesthetics, increased property values, increased property taxes collected, power generation, revenue) with the project costs (e.g., annual operating and maintenance costs, costs for upgrades such as a spillway capacity increase), the project benefits far outweighed the costs, and therefore the existence of the project was justified and removal of the dams would not seem reasonable.

In this regard, it is noteworthy that, on behalf of the counties and the lakefront property owners, FLTF was willing to invest more than $200 million to restore the lakes after the dams failed, even if there was no revenue from power generation. This indicates the true value of the dams to the counties and the lakefront property owners due to recreational, aesthetic, property value, and property tax benefits. By comparison, the cost to operate, maintain, and upgrade the dams before the dam failures was roughly $2 million in annual operating and maintenance costs for all four dams, and the estimated cost to upgrade the Edenville Dam spillway to pass the PMF was about $5 million to $10 million. These are relatively low costs compared to the true benefits provided by the dams.

Moreover, *before* the dam failures, the FLTF's agreed price to purchase the dams and related properties from Boyce Hydro was about $16 million, and FLTF's total budget to buy the properties and bring the dams fully into compliance with regulatory requirements was about $40 million. This is only a small fraction of the more than $200 million FLTF was willing to spend *after* the failures to achieve the same benefits as were expected before the failures. FLTF had the financial capacity to make such a large investment because, in addition to obtaining grant funds, it could obtain funds from the lakefront property owners via annual fees through the Special Assessment District (SAD). In contrast, the financial capacity of Boyce Hydro and the prior dam owners was limited to the revenue they could obtain by selling power to Consumers at the negotiated rates, and this revenue was not sufficient to fund major safety investments, such as upgrading the spillway capacity at Edenville Dam to meet the FERC PMF requirement.

The problem with the structure of the situation in which the parties were engaged is that it resulted in a substantial amount of noncooperation. Viewed from the perspectives of the parties:

- When the dams were privately owned, the counties, lakefront property owners, and other lake users were not compelled to help pay for the costs of the dams. They had no incentive to do so if they could get substantial financial, recreational, and aesthetic benefits from the dams "for free," and they had no information indicating that the dams were at substantial risk of failure due to insufficient safety investments (although the public was generally aware that there was an issue with spillway capacity).
- From the point of view of the private dam owners, the profit margins from selling power generated by the dams were relatively small, even if expenses were limited to normal annual operating and maintenance costs. If a major safety investment was to be made, such as spending $5 million to $10 million to upgrade the Edenville Dam spillway capacity to fully meet the FERC PMF requirement, there would likely have been an annual financial loss on the project for many years, and it may not have been possible to secure a loan to fund such an upgrade.

The dam owners *would* have had an incentive to invest in major safety upgrades, even if that entailed some annual financial loss, if it was believed that doing so would prevent dam failure. But none of the engineering consultants and neither regulator indicated that the risk of such a failure was believed to be high enough to require major safety upgrades on an urgent basis (beyond the safety upgrades that had already been performed). Communications among the parties indicate that the desire to upgrade the spillway capacity was driven mainly by the need to meet formalized regulatory requirements, not by a perceived high risk of dam failure. And again, it must be emphasized that the physical mechanism of the Edenville Dam failure in May 2020 was fundamentally an unforeseen embankment instability failure, not an overtopping failure resulting from inadequate spillway capacity.

- Consumers had no incentive to pay a higher rate to the dam owners than it had to, since paying a lower rate to the dam owners would increase its profit margin when it would resell that power. In addition, Consumers had no reason to expect that paying a lower rate to the dam owners would contribute to a dam failure nor that such a dam failure would have significant adverse consequences for Consumers (other than loss of its ability to purchase power from the dam owners, which was only a very small percentage of the utility's energy portfolio). Moreover, as a large utility company that was essentially the only customer for the relatively small amount of power generated by the dams, Consumers may have had more bargaining power than the dam owners in negotiating the rate that would be paid for that power. The result was that Boyce Hydro was paid a rate that was below average compared to other small hydro power producers.
- The engineering consultants had an incentive to make safety recommendations to the dam owners that would address obvious safety risks. However, the engineering consultants also had an incentive (motivational bias) to not make overly conservative recommendations that would pressure the dam owners to make costly safety upgrades. Such recommendations could displease the dam owners and motivate them to find other consultants for engineering services and recommendations. This structure of incentives may have contributed to engineering studies that were less thorough and less conservative than they ideally might have been.
- FERC derived no significant benefit from the project. Its mandate and obligation was to enforce its regulations, with the expectation that doing so would implicitly reduce the risk of dam failure to an acceptable level. However, FERC had limited options to enforce its regulations, and *none* of them were good options. When FERC finally revoked the Edenville Dam license as a "last resort," after about three decades in which the three dam owners did not upgrade the spillway capacity, their action could be viewed as somewhat increasing the risk of dam failure rather than decreasing it, for at least some duration, since pressure on the dam owner to eventually increase the spillway capacity to be able to pass the PMF was removed, funding for spillway capacity upgrades was diminished due to the loss of ability to generate power, and ability to release water through the powerhouse was lost. Although FERC had the authority to review the dam owner's financial position to inform its own decision-making, FERC's position and practice was that the licensee's financial position had no bearing on the requirement to comply with safety regulations, and FERC did not have any authority or ability to assist the dam owner with improving its financial position.
- EGLE derived no significant benefit from the project, but had mandates and obligations to enforce its regulations related to both dam safety and environmental protection, and there was potential for these two sets of considerations to be in conflict. For example, lowering the lakes could improve dam safety and dam operator safety, but it could also potentially result in environmental impacts.

EGLE had no authority to review the dam owner's financial position in order to inform its own decision-making, and EGLE did not have authority or ability to assist the dam owner with improving its financial position.

- The parties most at risk from the project, while deriving little or no benefit from the project, were the downstream property owners who did in fact experience devastating property losses of about $200+ million during the May 2020 flood event; a significant portion of this property damage was likely attributable to the dam failures. Fortunately, there was no loss of life or serious injury. The downstream property owners had limited information about the dams and limited ability to influence decisions being made in relation to the dams. They were therefore in a position where they simply had to rely on other parties to take actions to keep the risks posed by the dams at an acceptably low level. In effect, the downstream property owners were "innocent bystanders" who ultimately became "innocent victims" when the dams failed.

The net result of the structure of the situation in which the parties were engaged was that the parties made decisions that were individually "rational," yet the set of decisions taken collectively over the history of project ultimately contributed to the failure of the dams, which was a bad outcome for *all* of the parties:

- As the dam owner at the time of the failures, Boyce Hydro lost its investment in the dams, had to file for bankruptcy, and faced numerous lawsuits.
- FLTF, on behalf of the counties and local lakefront property owners, had to spend much more money to rebuild the failed dams and restore the lakes than the costs that would have been incurred if the dams had not failed and safety improvements had been made to prevent the dam failures.
- Lakefront property owners lost the lakes for a substantial period of time, their property values were likely reduced during that time, and they faced the prospect of greatly increased annual fees through the SAD as a result of the dam failures.
- The engineering consultants involved in the project potentially faced liability and reputational damage.
- FERC and EGLE potentially faced potential liability and reputational damage.
- While the IFT did not evaluate this impact, the counties would be expected to collect lower property taxes from lakefront property owners until the lakes were restored.
- Consumers lost the power supplied by the dams and, therefore, the potential for profit on reselling that power.
- The downstream property owners experienced devastating and costly property losses during the May 2020 flood, a significant portion of which were attributable to the dam failures, and it is the IFT's understanding that a significant portion of these losses were not covered by property insurance.

Weighing all of these considerations, the structure of the situation in which the parties were engaged can be considered to have been a significant contributing factor to the dam failures and the resulting adverse consequences for all of the parties. The situation *should* have been structured so that the parties were generally compelled to participate in cooperative relationships, with minimal non-cooperative relationships. While there are various methods to achieve "fair" cost allocation in a cooperative relationship, a straightforward "solution" is that each party should contribute to paying for project costs roughly in proportion to the share of benefits it receives from the project.

Since the vast majority of the benefit provided by these four dams went to the counties and the lakefront property owners, and since the revenue from power generation was a comparatively very small benefit, a good solution would have been for the dams to be owned by the counties, managed and operated by a delegated authority of the counties, and paid for by lakefront property owners through a SAD. This was precisely the solution that was planned to be implemented before the dams failed and even before the Edenville Dam license was revoked. The solution of transferring the dam ownership to the counties was apparently set in motion when the lakefront property owners and lake associations had concerns about the lake levels not being maintained, and thus the potential loss of the "free" benefits that they had been receiving for several decades. This solution – the counties taking ownership of the dams, with FLTF acting as their delegated authority – was actually implemented after the dams failed.

While this solution is a reasonable post-failure solution, a better solution would have been for the dams to have been purchased by the counties decades ago from the private dam owner (Wolverine) when it was first determined that the spillway capacity at Edenville Dam was inadequate relative to the FERC PMF requirement and Wolverine was unable to fund a spillway capacity upgrade to meet that requirement. Through a SAD, the counties would have had the financial means to outbid private parties and purchase the dams. If the dams had become publicly owned at that time, in addition to upgrading the spillway capacity, more in-depth engineering studies may have been done, which may have revealed the low embankment stability factors of safety in the Edenville left (east) embankment where the failure occurred. That finding may have led to remedial actions, such as downstream slope overlays covering the full length of Edenville Dam. Overlays would have increased the factors of safety for a conventional instability failure mode to acceptable levels and would likely also have coincidentally prevented an embankment static liquefaction flow failure even if the lake level reached the dam crest.

Another solution for changing the situation in which the parties were engaged would have been to establish a public-private partnership (PPP) in which the dams would remain privately owned, the lakefront property owners would contribute their "fair share" in paying for the operating, maintenance, and safety upgrade costs associated with the dams, and the dam owner would have obligations with respect to transparency and proper use of the funds contributed by the lakefront property owners. The arrangement of a PPP would have accomplished essentially the same results as public ownership of the dams. However, the structure of the situation in which the parties were already engaged – which was a result of the history of the project – gave an incentive to the dam owner to proceed to establish a PPP, whereas the incentive for the lakefront property owners and counties was to continue to derive the benefits "for free" without entering into a PPP arrangement. There was no external party, governmental or otherwise, that had the authority and "span of control" to force a PPP arrangement to be established, and therefore a PPP arrangement was never established. The parties were trapped in their existing relationships, which unfortunately were largely non-cooperative, until the counties, acting through their delegated authority, finally had an incentive to take ownership of the dams on behalf of the lakefront property owners.

8. Lessons to be Learned

8.1 Static Liquefaction

Static liquefaction instability failure should be considered as a potential failure mode (PFM) for water storage or flood management dams (referred to in this discussion as water dams) when saturated or potentially saturated, loose or very loose sands, silty sands or nonplastic silts are present in the embankment or foundation of the dam.

Historically, this PFM has not typically been considered for water dams. Texts and guidance documents have indicated that static stability analysis of sands, silty sands, and nonplastic silts for water dams should be based on drained strengths, regardless of the relative densities of the soils. The potential for dramatically reduced undrained strength and liquefaction instability failure in water dams has been considered for sands, silty sands, and nonplastic silts only for rapid loadings, such as those that occur during earthquakes. The failure of Edenville Dam has demonstrated that static liquefaction instability failure can occur in water dams with saturated, loose sands, silty sands, and nonplastic silts without rapid loading.

While static liquefaction failure in water dams is not unprecedented, reported cases have been rare. Examples include the north dike of Wachusett Dam in 1907, Calaveras Dam in 1918, and Fort Peck Dam in 1938. It is important to note that in the case of Edenville Dam, without the dam failure video recorded by a local citizen, the evidence for static liquefaction would have been far less conclusive. It is therefore possible that some embankment failures that have occurred during floods were actually static liquefaction failures, but they were attributed to overtopping or other causes or mechanisms because of the lack of visual evidence. Static liquefaction should be considered an important PFM because, although its occurrence appears to be rare, it can be very dangerous because of the speed of the failure. The Edenville Dam failure video demonstrated that the breach of the embankment developed in less than 40 seconds, with no obvious warning signs of distress until about 35 minutes before the failure.

In recent decades, static liquefaction instability failure has received increasing attention in mine waste or tailings dam practice, because of notable failures such as the Feijão Dam I tailings dam failure near Brumadinho, Brazil in 2019. Tailings dam practitioners have been developing methods to address static liquefaction. One of the challenges they have encountered is the lack of clarity of the conditions that have triggered the phenomenon in the known cases.

The challenge for water dam engineers now is to develop procedures and protocols to screen and evaluate static liquefaction potential and determine when risk reduction actions to address this PFM are appropriate. Developing these procedures and protocols should leverage the work that has been done by tailings dam practitioners.

8.2 Comprehensive Reviews

Among the lessons to be learned from the independent forensic investigation of the 2017 Oroville Dam spillway incident (France at al. 2018) were the following:

- Physical inspections, while a necessary part of a dam safety program, are not sufficient by themselves to identify risks and manage safety.

- Periodic comprehensive reviews of original design and construction, performance, maintenance, and repairs are needed for all features of dam projects.

The failure of Edenville Dam again highlights these two lessons to be learned.

Limitations of Part 12D Inspections and Evaluations

Physical inspections of Edenville Dam have been completed annually by FERC since the projects were licensed by the agency. In addition, independent consultant FERC Part 12D inspections and reviews were completed for Edenville Dam on six occasions, in 1991, 1994, 2000, 2005, 2010, and 2015. None of these inspections and reviews identified serious concerns with the Edenville Dam left (east) embankment where the failure occurred. Some concerns with seepage and sloughing were identified in other sections of the embankment, leading in some cases to flattening of downstream slopes, construction of downstream berms, and installation of filters and drains. No such concerns were identified for the embankment section that ultimately failed.

The requirements and guidance for the FERC five-year Part 12D inspection and review process in effect during the time that the Edenville Dam inspections were completed appear to intend completion of a comprehensive review of a dam. However, in practice this has not always been accomplished. Quite often, the Part 12D reviews have accepted the results of earlier analyses and evaluations with cursory or limited review, and “deep dives” into the historical records of design, construction, and performance have not been completed.

The IFT notes that FERC has begun to address this shortcoming based on the findings of the Oroville forensic investigation. The agency has adopted new rules modifying the Part 12D five-year inspection and review process to require comprehensive reviews and semi-quantitative risk analyses every ten years, with more limited inspections and reviews in the intermediate five-year milestones. These new rules took effect in April 2022.

The lack of detailed comprehensive reviews impacted Edenville Dam with respect to both geotechnical and hydrologic/hydraulic considerations.

Geotechnical Evaluations for Edenville Dam

The IFT did not find evidence that a comprehensive geotechnical review of the Edenville Dam embankments was ever completed. The geotechnical investigations that were completed were of limited extent and focused on specific locations and observations. Most notably, two of the investigations and associated stability analyses were focused on the taller sections of the Tobacco and Edenville embankments, where seepage or slope sloughing had been observed. These investigations resulted in conclusions that the static stability factors of safety for these sections were less than the minimum values required by FERC. Ultimately, these specific locations were modified, with the installation of filters and drains and with slope flattening or downstream berms, to increase the factors of safety to above the required minimum values. However, after this was done, the factor of safety was not checked for other sections of the embankment, including the section of the Edenville Dam left embankment that ultimately failed, which was the tallest remaining section with the originally constructed steep slopes (2H:1V or steeper).

As discussed in Appendix F2, the IFT completed static stability analyses for the embankment section that failed by simply using the same cross-section characteristics (scaled to the geometry of the failure section) and soil parameters previously used in the stability analyses for the taller sections of the Tobacco and Edenville embankments. This analysis indicated factors of safety lower than the FERC required minimum values. Had this analysis been completed by the consultants, it would likely have led to either further investigation of that section or directly to modification of that section. The IFT is confident that further investigation would have confirmed the low factors of safety. Modification of the section to improve the factors of safety, such as slope flattening or a berm, would have reduced the static stress ratios in the dam, which likely would have prevented the failure that occurred on May 19, 2020. This effort would not have identified the static liquefaction mechanism that ultimately occurred, but nonetheless would likely have resulted in modifications that would have prevented development of the failure mechanism.

The example of the stability analyses for the Edenville Dam also challenges the validity of two common practices in embankment dam stability analyses: (1) a focus on the maximum height section of the embankment and (2) a presumption that lack of seepage can be taken as a proxy for adequate stability. The maximum height section is not always the most critical. Variability in embankment and foundation conditions (geometry, strength, pore pressures) needs to be considered when performing site investigations and analyses, particularly for long embankments, for which there is limited design and construction information. Also, as illustrated in this case, embankment sections with no history of serious seepage concerns can still fail by instability.

In a broader sense, the geotechnical investigations and evaluations did not consider the totality of the embankments. The inconsistency between the specifications indicating a two-zone compacted fill embankment and the very low blow counts found in the test borings was not recognized, nor was the apparent poor quality control or deviation from the design plans and construction specifications during construction. These factors were all documented in the available project information, but were not assimilated into a comprehensive geotechnical assessment. Considering the nearly 6,000 feet total length of embankments, the number of test borings completed was small, and they were clustered at just a few locations. Considering the uncertainties with the Edenville Dam embankments, a much more comprehensive investigation would have been appropriate. The IFT understands the concerns in the industry regarding limiting intrusive explorations in embankment dams, particularly in light of the fact that intrusive investigations can cause problems such as hydraulic fracturing. However, in circumstances like those with Edenville Dam, the need to resolve uncertainties is high and protocols can be used to limit the risks of explorations.

Hydrologic/Hydraulic Evaluations for Edenville Dam

The hydrologic and hydraulic evaluations for Edenville Dam also illustrate a lack of comprehensive evaluation. The spillway rating curves that were developed and used in all of Part 12D inspection efforts through 2015 were based on weir equations, which are further based on the assumption that the Tainter gates can be lifted high enough that they do not interfere with the flow even when the lake level is high.

Yet, as early as 1996, it was known that the gate hoist mechanisms did not allow the gates to be lifted that high. Boyce Hydro began efforts to supplement the gate openings at the dams in 2012, but these efforts were not completed until 2015, when the supplemental A-frame system was operational. Consequently, the analyses through this period over-estimated the spillway capacity and the magnitude of flood that could be passed by the spillways.

It should also be noted that use of the supplemental A-frame system was cumbersome and time consuming, and ultimately, in 2019, the A-frame system was judged dangerous to operating personnel and potentially damaging to the gates, so it was recommended by engineering consultants that it not be used. The lack of consistency between the spillway rating curves and the actual gate operations is an example of "dots not being connected," and this likely would not have happened if comprehensive evaluation of flood routing capacity for the dam had been performed.

Comprehensive Reviews

As noted above, after the Oroville Dam spillway incident, FERC recognized the problem with the lack of comprehensive reviews and is making changes to address the issue. However, over 90 percent of the more than 90,000 dams in the United States are under the jurisdiction of state dam safety regulations and are not impacted by FERC regulations. Although some states, such as Colorado, New Mexico, Hawaii, and California, have begun to include more comprehensive reviews and dam safety risk considerations into their programs, most states still rely heavily on periodic physical inspections with no requirements for comprehensive reviews.

In large part, this is attributed to limited resources being allocated to the state dam safety programs and limited resources of dam owners. However, it is reasonable to ask the question that, even with limited resources, how can those resources best benefit dam safety? Is it through frequent periodic physical inspections that cannot identify hidden latent defects, or through less frequent physical inspections combined with comprehensive reviews of all available data for the dam? The IFT believes that the latter is the better option.

The IFT also suggests that owners should consider comprehensive reviews, even if they are not required by the dam safety regulator. The liability for a dam failure ultimately lies mainly with the owner. To manage its risk, an owner should consider doing or requiring periodic comprehensive reviews.

Repeating what was stated in the Oroville Dam spillway incident forensic investigation report (France et al. 2018), comprehensive reviews should compare the various features of the project with the current state of practice to answer the following questions:

- Is the feature consistent with current design and construction practice?
- If there are variations from current practice, do they compromise the structure and present a risk of failure or unsatisfactory performance?
- If there is not enough information available to make those judgments, is the potential risk sufficient to justify further study or evaluation?

The comprehensive reviews should be:

- Thorough, taking advantage of all available information.
- Critical and independent, rather than relying largely on the findings of past reviews.
- Completed by people with appropriate technical expertise, experience, and qualifications to cover all aspects of design, construction, maintenance, repair, and failure modes of the assets under consideration.

8.3 Dam Safety Rehabilitation/Upgrade Financing

The Edenville Dam failure points out two problems with dam safety rehabilitation/upgrade financing:

- Inadequate financial resources.
- Mismatches between benefits and financial responsibility.

Inadequate Financial Resources

The inability of Edenville Dam to safely pass the PMF inflow had been known as early as the Phase 1 inspection report in 1978 (Commonwealth Associates 1978), when the project was owned by Wolverine. During the time that the project was under FERC regulation, the dam owners (Wolverine, Synex, and Boyce Hydro) were directed by FERC to design and construct spillway upgrades to accommodate the PMF. Although possible spillway upgrade alternatives were considered, the owners all contended, the IFT believes correctly, that the hydroelectric projects (all four of the Boyce Hydro dams) did not provide sufficient financial resources to construct the PMF upgrades at Edenville Dam.

During Boyce Hydro's ownership of the projects, the Edenville PMF spillway upgrade was estimated to cost $5 million to $10 million. During the Boyce Hydro period of ownership, the average gross revenue from the projects was on average about $2 million per year, which produced an average net income of between $68,000 and $380,000 per year, depending on adjustments for multi-year improvements and equipment, and in some years the projects operated at a financial loss. This income was not sufficient to secure a commercial loan for the PMF spillway capacity upgrade construction. The income for the project was limited by the power purchase contract with Consumers Energy, and there was no practical way for the owners to increase the income without a change to the contract. FERC's revocation of the Edenville license exacerbated the financial resources problem because the inability to generate power at Edenville Dam reduced the projects' gross income by about 50 percent, resulting in the projects operating at a net financial loss.

During the period of FERC regulation, the owners did fund some dam safety improvements at the projects. At Edenville Dam, embankment stability and seepage overlays were constructed, and spillway concrete repairs were completed. The fuse plug spillway and a filter overlay were constructed at Sanford Dam, and the auxiliary spillway at Smallwood Dam was originally constructed and subsequently modified twice. The gate hoist systems at Secord Dam, Smallwood Dam, and Sanford Dam were upgraded.

An A-frame system for opening the gates more than possible with the original hoists was developed for Edenville Dam, but it was subsequently judged unsafe in 2019 (Spicer Group 2019) and was deployed in only a limited way in the May 2020 event. Since it was believed between 2015 and 2019 that the A-frame system was workable, upgrades to the gate hoists at Edenville Dam were deferred until there was resolution of the plan for a PMF spillway upgrade. After the conclusion was reached in 2019 that the A-frame system was unsafe, it was planned to upgrade the Edenville Dam gate hoists in late 2020, but unfortunately the May 2020 event occurred before the gate hoist improvements could be completed. As noted earlier in this report, had the gates been able to be fully opened during the May 2020 event, it is estimated that the maximum Wixom Lake water level would have been about 1 foot lower than the water level at the time of the Edenville Dam failure, which may (or may not) have prevented the failure.

The lack of financial resources for dam safety rehabilitation/upgrade at Edenville Dam is representative of a broader dam safety problem in the United States. ASDSO estimates that needed rehabilitation/upgrade of the more than 88,000 nonfederal dams in the United States would cost over $75 billion, including about $24 billion dollars for more than 15,000 nonfederal high hazard potential dams (ASDSO 2022). Many of those rehabilitation/upgrade needs are not being met because of owners' lack of financial resources.

According to responses to an ASDSO data call, 22 of the 49 state dam safety programs (Alabama does not have a state dam safety program) report that their states have some form of low-interest loan or grant programs to assist dam owners with rehabilitation. The other 27 states do not report any dam owner financial assistance programs, and the programs in the 22 states that have them vary widely in scope and are far from sufficient to meet the needs.

Despite many years of advocacy by ASDSO and ASCE, no federal funding for dam safety rehabilitation/upgrade of non-federal dams was available until recent years. A federal High Hazard Potential Dam Rehabilitation grant program was finally authorized in 2016 (US Code citation 33USC Chapter 9 Subchapter VII Section 467f-2), but not funded until federal fiscal year 2019. After a few years of limited funding of $10 million to $12 million per year (against authorizations of up to $60 million per year), the program was funded late last year for $585 million dollars, of which $75 million must be used for dam removals, as part of the Infrastructure Investment and Jobs Act (PL 117-58 H.R. 3684). The same act included $118 million dollars for Natural Resources Conservation Service (NRCS) Small Watershed Rehabilitation grants; $64 million for a new USACE program for low-interest loans for dam repair (sufficient seed funds for over $900 million in loans); approximately $800 million for dam removal projects; and approximately $800 million for dam safety, environmental, and electric grid upgrades for hydropower dams.

Although the recent legislation represents a significant increase in dam safety rehabilitation funding, it represents only a small fraction of the need identified by ASDSO. It should also be noted that the federal High Hazard Potential Dam Rehabilitation grant program excludes licensed hydropower dams with installed capacity greater than 1.5 megawatts. Edenville Dam (while it was FERC licensed) and Sanford Dam with nameplate rated capacities of 4.8 megawatts and 3.6 megawatts, respectively, would have been excluded from the program, while Secord Dam and Smallwood Dam with rated capacities of 1.2 megawatts each would have potentially been eligible.

If progress is to be made on addressing the identified dam safety rehabilitation needs and protecting the public in the United States, more financial support to owners will be needed from the federal, state, and local levels of government.

Imbalance between benefits and financial responsibility

For the Boyce Hydro projects, including Edenville Dam, the dam owner was expected to pay for all costs related to the projects. Yet, the counties and local residents realized, at no cost, substantial financial and other benefits from the projects in the form of county tax revenues, increased property values, and recreational opportunities, and these benefits were much larger in value than the profit resulting from generating and selling power.

In the case of the Boyce Hydro projects, this imbalance in benefits and financial responsibility was in the process of being addressed at the time of the failures, through the planned sale of the projects to FLTF, acting as the delegated authority of the counties. Most unfortunately, the unusual combination of rainfall and basin conditions in May 2020 created the record Wixom Lake level that triggered the Edenville Dam embankment failure, before the culmination of the sale and the completion of a PMF upgrade that could have been completed with funding provided by the Special Assessment District.

As for the financial resources issue, the imbalance of benefits and financial responsibility for the Boyce Hydro projects is not unique, but rather likely exists for many other projects across the country. Where such imbalances exist, sales of dams to local public entities should be considered along with more creative solutions such as public-private partnerships.

8.4 Dam Safety Regulatory Enforcement Tools

During the years that FERC was unsuccessfully directing the owners of Edenville Dam to upgrade the spillway capacity at the dam to accommodate the PMF, the agency's enforcement options were limited. Specifically, enforcement options available to FERC included:

- Fines
- Cease generation orders
- License revocation
- Restriction of reservoir level

Given that the dam owners had insufficient financial resources to support the required spillway upgrade, none of these options would improve the safety of Edenville Dam.

Fines would have further diminished the owners' financial resources. A cease generation order would have eliminated the income from hydroelectric generation at Edenville (about half of the total generation revenue from the four projects) but would have retained FERC regulation of the Edenville project.

Revocation of the license, the choice ultimately made by FERC, eliminated the income from hydroelectric generation at Edenville and transferred the project to EGLE's state regulatory control.

Although a reservoir restriction might be thought to provide significant improvement with respect to hydrologic risk, this often is not the case. The IFT's analyses show that a Wixom Lake restriction to run-of-the-river operations over the concrete spillway crest would have resulted in a peak lake level only about 0.2 feet lower than the level at the time of the May 2020 failure. Restriction to lake levels lower than the spillway crest would have required restoring the low-level sluice gates to operation or constructing a new low-level outlet structure.

Except for emergencies, FERC did not have legal authority to order a breach of Edenville Dam, which would have reduced the downstream risks posed by the project. In the case of Edenville Dam, had FERC been able to order a breach of the dam, the action would likely have met with strong resistance from the counties and local residents. However, the prospect of this scenario may have led to development of a mechanism for the counties and the residents to acquire the dams or contribute to paying for a spillway upgrade. If this had happened sooner, it is possible that a spillway upgrade would have been constructed before the May 2020 event, and the failures very likely would not have happened in May 2020.

The IFT was not able to obtain information regarding which state dam safety programs have the authority specifically to order a dam breach or physically breach the dam if the owner does not or cannot comply with a breach order. However, in response to an ASDSO questionnaire to dam safety programs in 49 states and Puerto Rico, 49 programs answered "yes" to the first two questions below and 50 answered "yes" to the third question:

- Authority to order repairs of a dam or modifications to a dam's operation to assure the dam's safety.
- Authority to take such corrective action as required to carry out the purpose of the statute.
- Authority to take emergency action.

Again, these questions do not specifically address dam breach as an enforcement action. The IFT is aware of some state programs (e.g., Pennsylvania) that have the authority to order and, if necessary, implement a breach for dam safety reasons, and have done so in some cases. EGLE also has the authority to order a dam breach if there are serious dam safety deficiencies. At the time of the May 2020 dam failures, EGLE had been the regulator for Edenville Dam for less than 2 years and was still working with FLTF to evaluate the spillway capacity at Edenville Dam, and had not yet identified any issues that would lead them to order a dam breach.

The IFT suggests that all dam safety regulatory agencies, including FERC, should have the regulatory authority to order a dam breach if dam safety risks are judged to be unacceptable and an owner does not have the financial resources to reduce the risks or refuses to comply with a directive to reduce the risks. Further, the IFT suggests that regulatory agencies should have the authority to breach the dam if the owner does not comply with a breach order. The regulatory agencies will also need access to funding to breach dams when necessary.

8.5 Hydrologic Risk

The IFT found that the risk of embankment overtopping at Edenville Dam was not well understood. Although the 2005 PFMA includes a statement that the spillway capacity at Edenville Dam is capable of safely passing a flood event roughly equal to a 200-year flood, the basis for this statement is not known. The IFT did not find any subsequent mention of this conclusion or other estimates of a flood return period that would result in embankment overtopping. Instead, the spillway capacity was simply compared to deterministic standards – the PMF under FERC regulation and "half PMF" under EGLE regulation. Based on information from interviews, it is the IFT's understanding that those involved with Edenville Dam typically perceived the spillway capacity to be close to or more than 50 percent PMF, which was thought to be a rare event.

In addition, under FERC regulation, the spillway discharge capacity was typically estimated based on free discharge over the concrete spillway crests (weir flow), which implicitly assumes that the gates are lifted free of the flow. As discussed in Section 2.5, prior to about 2015, none of the gates at the four Boyce projects could be lifted free of the flow for lake levels approaching the crests of the embankments. Subsequently, gate hoists were installed at Sanford Dam, Smallwood Dam, and Secord Dam to provide the ability to lift the gates free of the flow.

At Edenville Dam, an A-frame system was developed between 2013 and 2015, which was originally believed to allow the gates to be lifted free of the flow. However, based on gate testing performed in 2019, it was judged that use of the A-frame system was unsafe for the operators and could potentially damage the gates. Therefore, at the time of the failure in May 2020, the gate opening capability at Edenville Dam was believed to be limited to about 6 to 7 feet. Had limitations on the gate openings at Edenville Dam been considered, the estimated spillway discharge capacity would have been found to be lower than calculated in the various analyses completed prior to the 2020 failure.

As noted above, during most of the period of FERC regulation, it was generally believed that the spillway capacity at Edenville Dam was close to 50 percent of the PMF, a flood which was perceived to have a low probability of occurrence. Hence, the urgency of hydrologic risk reduction actions was not judged to be high, and this judgment was supported to some degree by the nearly 100 years of project performance with few high reservoir levels and none more than 2.5 feet above the normal lake level. As noted above, these judgments were based in part on inaccurate spillway capacity estimates.

More recent analyses completed by Ayres (Ayres 2021) indicated that a flood with an estimated 200-year return period (annual exceedance probability equal to 0.005) would cause a Wixom Lake level slightly below the embankment crest elevation, assuming that the gates could be lifted about 9 feet. If this analysis had assumed that gates could be lifted only 6 to 7 feet, the return period would have been calculated to be less than 200 years, which reflects a relatively high likelihood of overtopping for a high-hazard dam. This analysis seems to confirm the information in the 2005 PFMA, which apparently was not acknowledged or not understood to represent a relatively urgent issue in subsequent evaluations. Had this risk of embankment overtopping been recognized, there may have been a greater urgency assigned to the need for gate operation improvements and spillway upgrades, including staged spillway upgrades to incrementally increase spillway capacity and reduce risks.

Considering all of this, for dams that do not meet regulatory spillway capacity requirements, the IFT believes that the urgency of the need for spillway capacity upgrade should be based on quantitative analysis of risks, and consideration should be given to staged spillway capacity improvements/upgrades as interim risk reduction measures.

For the four Boyce Hydro dams, another aspect of hydrologic risk is the way in which the dams and lakes interact as a system during floods. With regard to coordination of operations between the four dams, there was no formal operating procedure for the four dams other than to maintain the lake levels for all the dams within a 0.7-foot range, from 0.3 feet above the normal lake level to 0.4 feet below the normal lake level. For a run-of-the-river system, this would require the flow that is being released from an upstream dam to be accommodated by opening the spillway gates at the downstream dam to maintain the lake level within the range of the normal lake elevation. However, for many flood situations this may not be possible and it was understood that the lake level could temporarily rise above the normal operating range.

Other than these operating guidelines, the IFT found that, with four dams in series on the Tittabawassee River, and no FERC or EGLE requirement that dams in series be operated as hydraulically interacting components of a single system, there was no formal analytical approach to managing the four dams as a system with respect to dam operations for safely passing floods, prioritizing spillway capacity upgrades to the dams, and recognizing how the interactions of the dams affect hydrologic risks. Although modeling dams in series is required by FERC for determining the IDF and the hazard classification, the FERC guidelines for operating plans do not have specific requirements for considering multiple dams as a system during flood operations.

The IFT believes that the operating plan for a system of dams should be based on system modeling and analysis, and should enable management of hydrologic risk by providing clear guidance for dam operators on how to operate the facilities as a system for a variety of storm and flooding scenarios, including both the inflow design flood (IDF) and more frequent storms.

8.6 Rainfall Return Periods versus Flood Return Periods

Contrary to what many may assume, the return period and annual exceedance probability of flooding do not always match the return period and annual exceedance probability of the precipitation causing the flooding. This type of divergence is clearly demonstrated by the May 2020 event in the Sanford Dam watershed, which includes Edenville Dam. The IFT's analyses indicate that, although the average precipitation across the watershed had an estimated return period of about 25 to 50 years (annual exceedance probability of about 0.04 to 0.02), the resulting inflow to Wixom Lake had an estimated return period of 100 to 200 years (an annual exceedance probability less than 0.01).

This large difference was caused by a combination of weather and ground conditions in the basin (antecedent moisture conditions, frozen ground or frost in some areas, and saturated ground conditions) and spatial and temporal characteristics of the precipitation. While this divergence is not typically significant for extreme floods like the PMF, it is potentially significant for more frequent floods like the one that occurred in May 2020 in the watershed. This can be particularly important when spillway capacity is limited relative to rarer floods (e.g., PMF), and thus more frequent precipitation events can produce floods approaching the spillway capacity.

In estimating hydrologic risks for less than extreme events, the potential for unfavorable basin conditions and spatial and temporal storm distributions should be considered. In this regard, some watersheds are more seasonally influenced than others. For watersheds in northern climes, such as Michigan, the potential for frozen ground or frost leading to saturated ground conditions in the spring should be considered.

8.7 Emergency Action Plans

Overall, the emergency response to the potential and then actual failures of Edenville and Sanford Dams must be considered very successful. On May 18 and 19, 2020, about 11,000 people were evacuated without any reported fatalities or serious injuries before or after Edenville and Sanford Dams failed. The emergency response achieved the primary goal of emergency management, which is to protect human lives and provide for public safety.

The success of the emergency response can be principally attributed to a prudent, proactive, and cautious decision by the Midland County emergency manager (EM) to initiate evacuations in the late hours of May 18, with evacuations occurring mostly in the early morning hours of May 19.

This decision did not exactly follow the guidance in the Edenville Dam emergency action plan (EAP). The guidance in the EAP was not consistent throughout the document, and generally indicated later evacuations, potentially as late as when Edenville Dam failed. Because of the rapidity of the embankment failure, had evacuations not been initiated before the actual failure, it is entirely possible that lives would have been lost.

The evacuations were issued early because the Midland County EM was not comfortable with the reports she was receiving from the dam operator concerning conditions at the dam, who was honestly expressing significant uncertainty about how high the lake would rise in the coming hours and the potential for the dam to fail. The EM also noted that in the overnight hours she had access to a large group of volunteer firefighters to implement the evacuations, many of whom would not be available during daylight working hours the next day.

The evacuations were reportedly well organized and orderly. The EM attributed this, at least in part, to an EAP exercise conducted in 2019 and evacuation plans developed as a result of that exercise. The EM also noted that the local communities were generally trusting of government, and therefore took the evacuation notices seriously, rather than viewing them as “false alarms.”

A lesson to be learned from this emergency response is that EAPs should provide consistent guidance to decision-makers regarding when evacuation is warranted, and should allow for judgment, so that evacuations can be ordered when the risk of failure is judged to be sufficiently high, rather than waiting for failure to initiate. In addition, an EAP should not be viewed as complete until an EAP exercise has been completed.

9. References

A. Rieli & Associates, LLC. 2006 (January). *2005 Consultant's Safety Inspection, Secord Hydroelectric Project, FERC Project No. 10809*

American Society of Civil Engineers. 2021. "Edenville and Sanford Dam Failures Field Reconnaissance Report." *Geotechnical Special Publication No. 327*. Prepared by Embankments, Dams, and Slopes Committee of the Geo- Institute. Edited by D. Pradel and A. Lobbestael.

Association of State Dam Safety Officials (ASDSO). 2020 (September). ASDSO Peer Review Report of Michigan Department of Environment, Great Lakes and Energy, Dam Safety Program. https://www.michigan.gov/documents/egle/egle-edenville-MIPeerReviewReport_703962_7.pdf

———. 2022 (March). The Cost of Rehabilitating Our Nation's Dams. https://damsafety-prod.s3.amazonaws.com/s3fs-public/files/Cost%20of%20Rehab%20Report-2022%20FINAL.pdf

Applied Weather Associates (AWA). 2021 (June). Site-Specific Probable Maximum Precipitation Study for Tittabawassee River Basin, Michigan, Final Report. Prepared for Four Lake Task Force in coordination with Spicer Group, Ayres Associates, GEI Consultants.

Ayres Associates (Ayres). 2013a (November). *Inflow Design Flood Determination – Edenville Hydroelectric Project*. Prepared by Ayres Associates.

———. 2013b (November). HEC-RAS model "PMF-IDF-to-Sanford." Prepared for *Inflow Design Flood Determination – Edenville Hydroelectric Project.* Prepared by Ayres Associates.

———. 2015. (February 17) P-10808 – Final Flow Calculations. Letter from Boyce Hydro Power, LLC to FERC. February 2015.

———. 2020 (May 15). Probable Maximum Flood Determination, Tittabawassee River Hydroelectric Projects, Secord (P-10809), Smallwood (P-10810), Sanford (P-2785), Edenville (unlicensed). Prepared by Ayres Associates.

———. 2021 (July). *Design Flood Hydrologic Analysis*: *Secord, Smallwood, Edenville, and Sanford Dams, Gladwin and Midland Counties, Michigan*. Prepared for Four Lakes Task Force, Midland, MI. Available: https://www.four-lakes-taskforce-mi.com/uploads/1/2/3/1/123199575/ayres_design_flood_hydrologic_analyses_july_2021.pdf.

Barr Engineering Company (Barr). 2001 (October). *2001 Independent Consultant's Safety Inspection Report, Secord Hydroelectric Project FERC License No. 10809*.

Blystra & Associates (Blystra). 1991 (March). *Initial Consultant's Safety Inspection Report, Edenville Hydroelectric Project*.

______. 1993 (December 29). *Supplemental Information Report, Edenville Hydroelectric Project.*

Board of Consultants (BOC). 2009 (February 5). *Edenville Hydroelectric Project, FERC No. 10808, Report by the Special Board of Consultants, Submitted to Boyce Hydro.*

Boyce Hydro Power, LLC. (Boyce Hydro). 2002a (February). 2001 CSIR for Sanford Dam, P-2785.

______. 2006 (December). *Supporting Technical Information document, Smallwood Project, FERC P-10810*.

———. 2011 (September). Updated CSIR and STID for Edenville Dam, P-10808.

———. 2012a (May 10). *Final Construction Report for Toe Filter Installation West of M-30 at Tobacco Embankment, Edenville Hydro Project.*

———. 2012b (May 29). P-2785, P-10808, P-10809, P-10810 – Gate Hoists. Letter from Boyce Hydro Power, LLC to FERC.

———. 2012c (October 29). P-2785, P-10808, P-10809, P-10810 – Gate Hoists. Letter from Boyce Hydro Power, LLC to FERC.

———. 2013 (May 20). Mueller, Lee. Letter to Kimberly D. Bose, Secretary, FERC. "Edenville Dam - Michigan, Outline of License Surrender," May 20, 2013.

———. 2014a (October 16). Letter to the FERC, RE: Project No. 10808- Edenville Dam- "Barrel Drain" Improvements.

———. 2014b (December). *Summary of Findings, Embankment Underdrain Investigations.*

———. 2015a (April). *Supporting Technical Information Document, Edenville Hydroelectric Project, P-10808.*

———. 2015b (April 13). Submission of Embankment Underdrain Investigations, Sanford – P-2785, Edenville – P-10808, Secord – P-10809, Smallwood – P-10810.

———. 2015c (January 12). P-2785, P-10808, P-10809, P-10810 – Spillway Gate Tests. Letter from Boyce Hydro Power, LLC to FERC.

———. 2015d (August 31). P-2785, P-10808, P-10809, P-10810 – Spillway Gate Tests. Letter from Boyce Hydro Power, LLC to FERC.

———. 2016a (December). *Supporting Technical Information Document, Secord Hydroelectric Project, P-10809.*

———. 2018a (June). *Emergency Action Plan, Edenville Dam Project No. 10808*.

———. 2018b (June). *Emergency Action Plan, Sanford Dam Project No. 2785*.

———. 2018c (June). *Emergency Action Plan, Secord Dam Project No 10809*.

———. 2018d (June). *Emergency Action Plan, Smallwood Dam Project No 10810*.

———. 2018e (December 31). *STID and PFMA revisions per 2017 6th Part 12D CSIR, Sanford Hydroelectric Project P-2785*.

———. 2018f (December). *Updated STID Smallwood Hydroelectric, Project P-10810*. December 2018.

———. 2018g (July 30). *Letter of Intent.* LOI between SLPA and W.D. Boyce Trusts. July 30, 2018.

———. 2018h. *Motion of Boyce Hydro Power, LLC for Withdrawal of Order Proposing Revocation of License.*

———. 2019a (December 9). *Annual Emergency Action Plan Status Report, Sanford Project (P-2785), Smallwood Project (P-10810), Secord Project (P-10809).*

———. 2019b (December 6). Annual Spillway Gate Operation Certificates Sanford (P-2785), Smallwood (P-10810), and Secord (P-10808). Transmittal letter to FERC and certificates.

———. 2019c (April 15). *Confidential Correspondence Disclosing Terms and Conditions for Boyce Hydro, LLC Participation in Raising the Wixom Reservoir Pond Levels to Former FERC License Elevations*. From Boyce Hydro Power, LLC to David Kepler II, Sanford Lake Preservation Association, Four LakesTask Force.

———. 2019d (April 16). *Letter of Understanding Regarding Acquisition of Lake Level Assets*. Letter from FLTF to Boyce Hydro Power, LLC. April 16, 2019.

———. 2019e (August 3). *Declaration of Lee W. Mueller in Support of First-Day Motions and Applications*. Boyce Hydro bankruptcy filing. August 3, 2020.

———. 2020a (July 1). *Part 12.10 Incident Report - Flood event of May 18, 2020 dam flood and breach incidents*.

———. 2020b (September 14). *Part 1. Supplemental Report to 12.10 Incident Report for May 19, 2020 Flood and Dam Breach*.

———. 2020c (June). *Dam Safety Program for FERC-Licensed Facilities – P-2785, Sanford, P-10809, Secord, P-10810, Smallwood – Revision 7*.

———. 2020d. *First Amendment to Purchase Agreement Between Boyce, Four Lakes Task Force and Four Lakes Operations Company, Inc.* Amendment to Agreement between Boyce, FLTF and FLO.

———. 2020e. *First Amendment to Purchase Agreement Between Boyce, Four Lakes Task Force and Four Lakes Operations Company, Inc.* Amendment to Agreement between Boyce, FLTF and FLO.

Bureau of Reclamation (Reclamation). 1985 (December). *Hydraulic Model Studies of Fuse Plug Embankments, REC-ERC-85-7*. U.S. Department of the Interior, Bureau of Reclamation.

Cedar River. 2011. Cedar River Watershed Management Plan, University Outreach, University of Michigan-Flint.

Christie Engineering. 2002 (September 13). Final Design Submission to FERC for Sanford Fuse Plug Spillway.

———. 2003 (January). *Final Construction Report for Sanford Fuse Plug Spillway*.

Commonwealth Associates. 1978 (September). Tittabawassee River Basin, Gladwin and Midland Counties, Michigan, National Dam Safety Program Inspection Reports, USACE Detroit District, Detroit Michigan, Edenville Dam. Prepared for Michigan Department of Natural Resources.

Davis, A.P., G. Castro, and S.J. Poulos. 1988. “Strengths backfigured from liquefaction case histories.” *Proc., 2nd Int. Conf. on Case Histories in Geotechnical Engineering.*, St. Louis, Vol. IV, 1693-1701.

Dolinar, E. 2020. High Flows from Secord Dam in Gladwin County Flash Flooding, New Alert on Fox News, Dolinar Meteorologist.

Fear, C. E., and Robertson, P. K. 1995. ‘‘Estimating the undrained strength of sand: A theoretical framework,’’ Canadian Geotechnical Journal, 32(4), 859–870.

Federal Energy Regulatory Commission (FERC). 1987 (December 1). *Order Issuing License, Project No. 2785-001. Issued by the Federal Energy Regulatory Commission.*

______. 1993 (August 6). In reply refer to: D2SI-OHL-CH Project No. 10808. Letter from FERC to Wolverine Power Corporation. August 6, 1993.

______. 1994 (March 3). In reply refer to: D2SI-OHL-CH Project Nos. 10808, 10809 and 10810. Letter from FERC to Wolverine Power Corporation. March 3, 1994.

______. 1998a (April 20). In reply refer to: P-10808, NATDAM No. MI00549. Letter from FERC to Wolverine Power Company. April 20, 1998.

______. 1998b (October 16). *Order Issuing Minor License, Project No. 10809-000. Issued by the Federal Energy Regulatory Commission.*

______. 1998c (October 16). *Order Issuing Minor License, Project No. 10810-000. Issued by the Federal Energy Regulatory Commission.*

______. 1998d (October 16). *Order Issuing Original License, Project No. 10808-000. Issued by the Federal Energy Regulatory Commission.*

______. 1998e (October 16). *Order On Rehearing And Amending License Order, Project Nos. 2875-002, 008 and 009. Issued by the Federal Energy Regulatory Commission.*

______. 1999 (January 4). In reply refer to: P-2785, P-10808, P-10809 and P-10810. Letter from FERC to Wolverine Power Corporation. January 4, 1999.

______. 2000 (April 14). In reply refer to: P-2785, P-10808, P-10809, P-10810. Letter from FERC to Wolverine Power Corporation. April 14, 2000.

______. 2007 (September 26). In reply refer to: P-10808. Letter from FERC to Mr. Lee Mueller. September 26, 2007.

———. 2017a (February 15). *Compliance Order, Project No. 10808-053, -044, -047, -055.* Issued by the Federal Energy Regulatory Commission.

———. 2017b (May 8). *Dam Safety Performance Monitoring Program.* Chapter 14 of the Engineering Guidelines for the Evaluation of Hydropower Projects, original publication April 11, 2003, revised July 1, 2005; January 3, 2017 and May 8, 2017.

———. 2017c (November 20). *Order to Cease Generation, Project No. 10808-053, -047.* Issued by the Federal Energy Regulatory Commission.

———. 2017d (December 1). *Emergency Motion for Stay Pending Rehearing, Request for Approval of Plan to Fund the Tobacco Auxiliary Spillway, and Request for Extensions of time, Project No. 10808.* Issued on behalf of Boyce Hydro Power LLC.

———. 2017e (December 20). *Request for Rehearing of Boyce Hydro Power, LLC, Project No. 10808-047, 10808-053.* Issued on behalf of Boyce Hydro Power, LLC.

———. 2017f (December 28). *Supplement to Emergency Motion for Stay and to Request for Rehearing of Boyce Hydro Power, LLC, Project No. 10808-056, 10808-057.* Issued on behalf of Boyce Hydro Power, LLC.

———. 2018a (January 5). *Order On Stay, Project No. 10808-057.* Issued by the Federal Energy Regulatory Commission.

———. 2018b (January 19). *Order Granting Rehearing for Further Consideration, Project No. 10808-057.* Issued by the Federal Energy Regulatory Commission.

———. 2018c (February 2). *Request for Rehearing of Boyce Hydro Power, LLC, Project No. 10808-057.* Issued by the Federal Energy Regulatory commission.

———. 2018d (February 15). *Order Denying Rehearing, Project No. 10808-056.* Issued by the Federal Energy Regulatory commission.

———. 2018e (February 15). *Order Proposing Revocation of License, Project No. 10808-058.* Issued by the Federal Energy Regulatory commission.

———. 2018f (September 10). *Order Revoking License, Project No. 10808-058*. Issued by the Federal Energy Regulatory Commission.

———. 2018g (October 18). *Order Denying Stay, Project No. 10808-062, 10808-063.*. Issued by the Federal Energy Regulatory commission.

______. 2019 (June 20). *Order Denying Reconsideration and Dismissing Application, Project No. 10808-066*. Issued by the Federal Energy Regulatory Commission.

———. 2020a (June 18). *Special Dam Safety Inspection Report No. 2, Sanford Hydroelectric Project, Project No. 02785-MI*. Prepared by the Federal Energy Regulatory Commission, Office of Energy Projects, Division of Dam Safety and Inspections, Chicago Regional Office. Reported filed June 18, 2020 for inspection on June 9, 2020.

———. 2020b (June 18). *Special Dam Safety Inspection Report No. 2, Secord Hydroelectric Project, Project No. 10809-MI*. Prepared by the Federal Energy Regulatory Commission, Office of Energy Projects, Division of Dam Safety and Inspections, Chicago Regional Office. Reported filed June 18, 2020 for inspection on June 10, 2020.

———. 2020c (June 18). *Special Dam Safety Inspection Report No. 2, Smallwood Hydroelectric Project, Project No. 10810-MI*. Prepared by the Federal Energy Regulatory Commission, Office of Energy Projects, Division of Dam Safety and Inspections, Chicago Regional Office. Reported filed June 18, 2020 for inspection on June 10, 2020.

———. 2021 (December 16). *Part 12D Program.* Chapter 16 of the Engineering Guidelines for the Evaluation of Hydropower Projects.

Federal Emergency Management Agency (FEMA). 2012. *Summary of Existing Guidelines to Hydrologic Safety of Dams.* FEMA P-919, July 2012.

Four Lakes Task Force (FLTF).2018. *Memorandum of Understanding Between Lake Associations for Wixom Lake and Sanford Lake (Gladwin and Midland Counties)*. Memorandum Between SLPA, SLA, and WLA.

______.2019a (April 16). *Letter of Understanding Regarding Acquisition of Lake Level Assets*. From Four Lakes Task Force to Mr. Lee W. Mueller. April 16, 2019.

———.2019b (December 2). *Extension of April 15, 2019 Letter Agreement; Operations of Edenville Dam and Maintenance of Normal Levels of Wixom Lake*. From Four Lakes Task Force to Mr. Lee W. Mueller.

———.2019c (August 21, 2019). *Agreement, Midland and Gladwin Counties and Four Lakes Task Force.* Agreement between Midland and Gladwin Counties and FLTF. August 21, 2019.

France, J.W., I.A. Alvi, P.A. Dickson, H.T. Falvey, S.J. Rigbey, and J. Trojanowski. 2018 (January 5). Independent Forensic Team Report, Oroville Dam Spillway Incident.

France, J.W., I.A. Alvi, J.L. Williams, A. Miller, and S. Higinbotham. 2021 (September 13). Independent Forensic Team Interim Report, Investigation of Failures of Edenville and Sanford Dams.

Gomez and Sullivan Engineers (Gomez and Sullivan). 2018 (January 12). *Subsurface Investigation, Work Plan, Edenville Hydroelectric Project, FERC Project No. 10808.*

Google Earth. 2022. Available: https://earth.google.com/web/.

Holland, Ackerman, & Holland (Holland). 1924a. Agreement and Specification of the Earth Embankments – Central Section of the Edenville Plant for J. N. Bick General Contractor.

______.1924b. Agreement and Specification of the Earth Embankments – South East End of the Edenville Plant for J. N. Bick General Contractor.

McDowell and Associates (McDowell). 2003 (June 6). *Soils Exploration, Edenville Dam and Spillway*.

———. 2005a (July 18). *Soils Investigation, Edenville Hydroelectric Project, Auxiliary Spillway Construction, Edenville and Meridian Roads.*

———. 2005b (October 21). *Soils Exploration, Edenville and Secord Dams.*

______. 2013 (November 9). Soil Borings for Edenville Temporary Spillway, Letter from McDowell & Associates to Boyce Hydro, LLC containing soil borings and laboratory testing results. November 9, 2013.

______. 2015. Soils borings prepared by McDowell & Associates for Wixom Lake Emergency Spillway and dated December 29 and 30, 2014 and January 15 and 16, 2015.

Mead & Hunt. 1994a (August). *Probable Maximum Flood Study, Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects*. Prepared by Mead and Hunt.

———. 1994b (August). *Inflow Design Flood, Edenville Hydroelectric Project*.

______. 1994c (December). *Report on Inspection, Edenville Hydroelectric Project*.

———. 1995a (February) *Supplement to December 1994 Report on Inspection, Edenville Hydroelectric* Project.

———. 1995b (December). *Review of Probable Maximum Flood Calculations, Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects*.

———. 1996 (March). *Probable Maximum Flood Restudy – Phase 1, Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects*.

———. 2000 (June). *2000 Consultant's Safety Inspection Report, Edenville Hydroelectric* Project.

———. 2005a (December). 2005 *Consultant's Safety Inspection Report (final), Edenville Hydroelectric Project, FERC Project No. 10808*.

———. 2005b (December). *Supporting Technical Information Document (final), Edenville Hydroelectric Project, FERC Project No. 10808*.

Michigan Department of Environment, Great Lakes, and Energy (EGLE). 2019 (November 25). Application Denial, Submission Number: HNS-X-17S-6B1RG; Gladwin County. Issued by EGLE.

———. 2020 (April 9). *Permit No.: WRP02178 v.1*. Issued by State of Michigan Department of Environment, Great Lakes and Energy (EGLE). 2020.

———. 2021 (February). Michigan Dam Safety Task Force Report. https://www.michigan.gov/documents/egle/2021-02-25-MI-Dam-Safety-Task-Force-Report-to-Governor-Whitmer_717510_7.pdf

Michigan Public Service Commission. 2016 (April 8). *PURPA Technical Advisory Committee, Report on the Continued Appropriateness of the Commission's Implementation of PURPA.*

Mill Road Engineering (Mill Road). 2006a (August). Sanford Project, LP-2785, Potential Failure Mode Analysis.

———. 2006b (August). Smallwood Project LP-10810 Potential Failure Mode Analysis.

———. 2007 (December). *Supporting Technical Information Document, Sanford Project, FERC Project No. 2785-MI*.

______. 2008 (October). *Probable Maximum Flood Reanalysis Edenville Hydroelectric Project*. FERC Project No. 10808.

______. 2009a (February). *Probable Maximum Flood Reanalysis Edenville Hydroelectric Project*. FERC Project No. 10808.

———. 2009b (Revised February 28). *Embankment Stability Analysis, Toe Drain Addition, Cross Section Modification and Safety Factor, Edenville Project.*

———. 2010 (December 29). *Liquefaction Analysis, Edenville Hydroelectric Project*.

______. 2011a (April 11). *Probable Maximum Flood Reanalysis, Edenville Hydroelectric Project.* FERC Project No. 10808

———. 2011b (Revised September 9). *2010 Consultant's Safety Inspection Report, Edenville Hydroelectric Project*

———. 2011c (December). *2011 Consultant's Safety Inspection Report, Smallwood Project*

———. 2012 (March 7). *Tainter Gate Design Report and Calculations, Edenville Project, Edenville Hydroelectric Project, FERC Project No. 10808*.

National Oceanic and Atmospheric Administration (NOAA). 2022b. NOAA Atlas 14 Point Precipitation Frequency Estimates. Available: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html

Purkeypile Consulting, LLC (Purkeypile). 2016a (March). *2015 Consultant's Safety Inspection Report, Edenville Hydroelectric Project, FERC Project No. 10808.*

———. 2016b (March 18). *2015 Potential Failure Modes Analysis, PFMA Supplement, Completed as Part of 2015 CSIR, Edenville Hydroelectric Project, FERC Project No. 10808*.

———. 2016c (December 31). *2016 Consultant's Safety Inspection Report Secord Hydroelectric Project FERC Project No. 10809*.

———. 2016d (December). *Updated Supporting Technical Information Document, Secord Hydroelectric Project, FERC Project No. 10809, NATDAM ID MI-00547*.

———. 2016e (December 31). *2016 Consultant's Safety Inspection Report Smallwood Hydroelectric Project FERC Project No. 10810*.

———. 2016f (December 31). *2016 Potential Failure Modes Analysis, PFMA Supplement, Completed as Part of 2016 CSIR, Smallwood Hydroelectric Project, FERC Project No. 10810*

———. 2017 (December 29). *2017 Consultant's Safety Inspection Report, Sanford Hydroelectric Project, FERC Project No. 2785*.

———. 2019 (January 4). *Technical Memorandum, Hydraulic Analysis of Edenville Project: Determination of Hydraulic Adequacy of Existing Dam and Radial Gate Spillways.* Memorandum to Mr. Lee W. Mueller.

A. Rieli and Associates, LLC (Rieli). 2006 (February). Supporting Technical Information Document, Secord Hydroelectric Project, FERC Project No. 10809, NATDAM ID MI-00547.

Robertson, P. K., L. de Melo, D. J. Williams, and G.W. Wilson. 2019 (December 12). *Report of the Expert Panel on the Technical Causes of the Failure of Feijão Dam I*. Available: https://bdrb1investigationstacc.z15.web.core.windows.net/assets/Feijao-Dam-I-Expert-Panel-Report-ENG.pdf.

Schrouder, K. S., Lockwood, R. N. and Baker, J. P. 2009. *Tittabawassee River Assessment.* State of Michigan Department of Natural Resources. September 2009.

Sigma Engineering Ltd. (Sigma). 2004 (July). *Design Report for Edenville Dam Auxiliary Spillway*.

Soil and Materials Engineers, Inc. (SME). 1987 (April 30). *Geotechnical Evaluation Report, Edenville Dam – Station 48+00.*

Spicer Group. 2019 (September 18). *Edenville Dam Gate Tests*. Memorandum to Luke Trumble, Michigan Department of Environment, Great Lakes, and Energy.

______. 2020. *Edenville Dam Inspection Report.*

State of Michigan. 2019 (May 28). *Order Setting Normal Lake Levels for Sanford Lake, Wixom Lake, Smallwood Lake and Secord Lake and Confirming the Four Lakes Special Assessment District Boundaries*. Issued by the State of Michigan in the Circuit Court for the County of Midland. Available: https://www.thesafetyequipmentstore.com/miva/graphics/00000001/4-Lakes-Order-Engwis.pdf.

State of Michigan House of Representatives. 1932. House of Representatives Document No. 273 Tittabawassee and Chippewa Rivers, MI, 72D Congress.

Synex Wolverine, LLC (Synex). 2005 (January 8). *Final Construction Report, Edenville Hydroelectric Project Toe Filter Drain Installation.*

United State Army Corps of Engineers (USACE). 2022 (January) *Hydrologic Engineering Center, HEC-SSP User's Manual Version 2.2.* Available: https://www.hec.usace.army.mil/software/hec-ssp/.

U.S. Geological Survey (USGS). 2016. National Land Cover Database (NLCD) 2016.

______. 2020 (September 7). National Hydrography Dataset (NHD), Hydrological Unit (HU) 8 - 04080201.

______. 2022. USGS Stream Gage Station 01421500 Muskegon River, MI.

Weather Bureau. 1955 (December). *Rainfall Intensity-Duration-Frequency Curves for Selected Stations in the United States, Alaska, Hawaiian Islands, and Puerto Rico.* U.S. Department of Commerce, Weather Bureau, Technical Paper No. 25.

Whitmer, Gretchen. 2020 (June 15). Letter from the Honorable Gretchen Whitmer, Governor of Michigan, to the Honorable Donald J. Trump, President of the United States, through Mr. James K. Joseph, Regional Administrator, FEMA Region V regarding request for major disaster declaration.

Wolverine Power Corporation (Wolverine). 1923 (May 31). *Agreement between Wolverine Power Company and Consumers Power Company.* Agreement between Wolverine Power Company and Consumers Power Company. May 31, 1923.

______. 1952 (May 22). *Agreement.* Agreement between Wolverine Power Corporation and The Dow Chemical Company. May 22, 1952.

______. 1997 (January 8). Annual Gate Operation Certifications – FERC Project No. 10808, Tittabawassee – Edenville Dam. Gate test date: June 18, 1996. Filed with FERC: January 8, 1997.

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Appendix A: Project Description

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At the time of the failures of the Edenville Dam and Sanford Dam in May 2020, they were two of four dams in Michigan operated by Boyce Hydro and located in series along the Tittabawassee River; the other two Boyce Hydro dams are Secord Dam and Smallwood Dam. All four dams were built between 1923 and 1925 and are located in Gladwin and Midland Counties in central Michigan. At the time of the failures, all of the Boyce Hydro dams except Edenville Dam were active hydroelectric facilities under the regulation of the Federal Energy Regulatory Commission (FERC). The Edenville Dam had also been a FERC-regulated hydroelectric facility, but FERC revoked the dam's license in September 2018 (FERC 2018). At the time of the failures, Edenville Dam was regulated by the State of Michigan Department of Environment, Great Lakes, and Energy (EGLE).

Secord Dam and Smallwood Dam are located on the Tittabawassee River, upstream of Edenville Dam, and Sanford Dam is located on the Tittabawassee River downstream of Edenville Dam. Edenville Dam was constructed across both the Tittabawassee River and the Tobacco River, a tributary to the Tittabawassee, just upstream of their confluence.

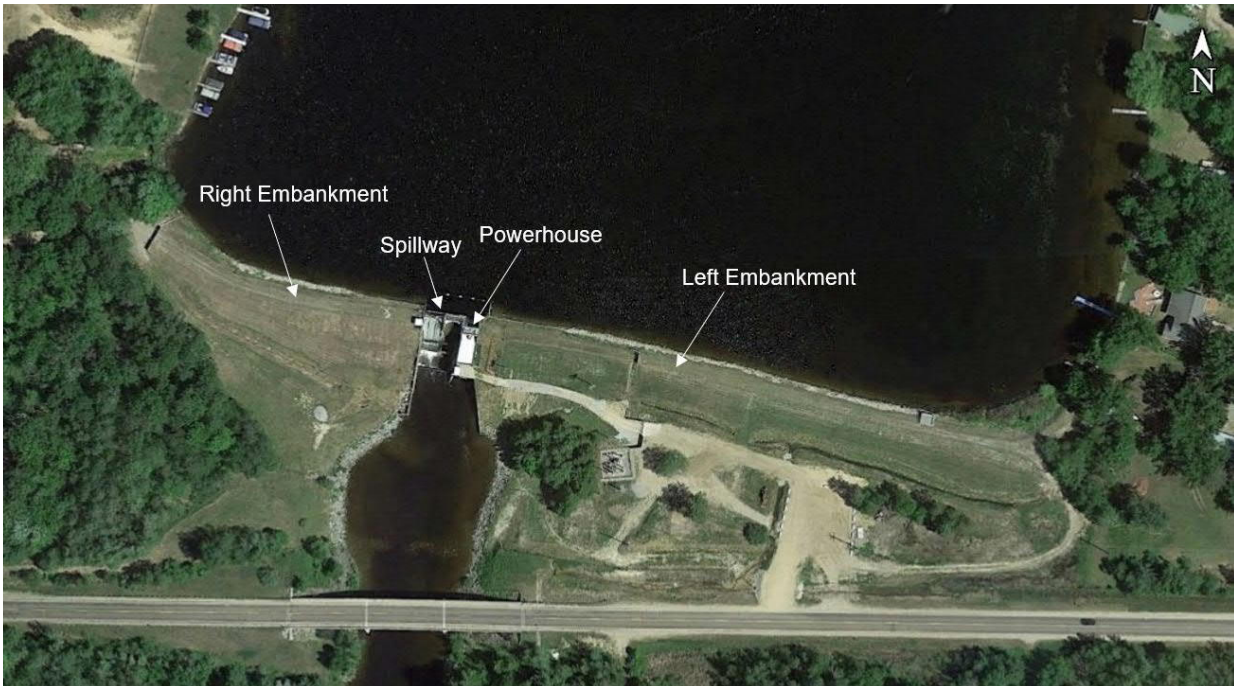
All four dams operated by Boyce Hydro were designed by Holland, Ackerman and Holland of Ann Arbor, Michigan. The primary purpose for constructing the dams was to provide sufficient operating reservoir heads for generating electrical power. The normal operating pool at each dam was primarily used to allow discharge of the run of river flows at a rate that will provide efficient power production. During normal (non-flood) operations, the reservoir fluctuations were less than a foot in total range in accordance with the FERC license requirements. The reservoirs provided recreational opportunities as a secondary benefit. Some recreational benefits were later formalized in the FERC license agreements.

Each of the dams, from upstream to downstream, is described in this appendix.

A-1 Secord Dam

Secord Dam is the farthest upstream project facility located on the Tittabawassee River (41 miles upstream from Midland, Michigan, and 7.7 miles upstream from Smallwood Dam[1]). The dam is composed of two embankments and a combined spillway and powerhouse structure, as shown in Figure A-1. The left embankment is about 1,400 feet long and extends from the left side of the powerhouse to the natural high ground on the left abutment. The combined width of the powerhouse and spillway is about 65.5 feet. The right embankment extends from the right side of the spillway to the right abutment, for a length of about 600 feet. The data provided here and in the following sections are primarily extracted from the Supporting Technical Information Document (STID) for Secord Dam (Boyce Hydro 2016) unless otherwise noted.

[1] These distances are taken from the table on Figure 2-1 in the main report and are based on streamline data for the Tittabawassee River. (USGS 2020).



Source of aerial image: Google Earth

Figure A-1: Aerial View of Secord Dam

A-1.1 Embankments

The embankments were constructed over native soils consisting of "clay with interspersed layer of silty sand with some gravel overlying a hardpan unit." The embankments were reportedly constructed of clay and poorly graded sand. The maximum height of the embankments is 56 feet in the natural river channel next to the powerhouse and spillway structure. The design crest width is 8 feet, and the design upstream and downstream slopes are nominally 2.5H:1V and 2H:1V, respectively. Riprap was placed on the upstream slope, starting 4 feet below normal pool and extending to 2 feet above normal pool. Transverse foundation drains, consisting of open-joint clay tile pipe, are spaced on approximately 15 to 20-foot centers below the downstream shell of the embankment. Each drain extends from approximately the centerline of the dam downstream to a toe drain trench that parallels the embankment. The upstream ends of the drains are connected with a longitudinal clay tile pipe drain. The clay tile pipes were covered with gravel.

In 1938, a steel sheet pile cutoff wall was installed through the dam embankments on the side of the spillway and powerhouse walls to reduce seepage through the deepest sections of both embankments. There is an 82-foot-long section of sheet pile driven into hardpan extending from the spillway right wall along the alignment of the right embankment, and there is a 96-foot-long section of sheet pile driven into hardpan extending from the powerhouse left wall along the alignment of the left embankment. The tops of the sheet pile walls have been capped with concrete. Based on inspection photos, the sheet pile wall was constructed upstream from and lower than the dam crest. Exact dimensions and elevations were not found by the Independent Forensic Team (IFT).

A-1.2 Powerhouse

The right side of the powerhouse is connected to the left side of the spillway. The powerhouse consists of a reinforced concrete substructure and brick superstructure. The powerhouse contains a single generating unit with a rated capacity of 1.2 megawatts. The maximum discharge capacity through the turbine is 450 cfs with a head of 48 feet. The turbine is a Francis, Type S, with a rating of 2100 hp at 200 rpm. The 1,500-KVA, 2,400-volt, 3-phase, 60-cycle, generator operates at 200 rpm with a direct-connected 25 kW, 125-volt exciter. The powerhouse normally operates with very little fluctuation in the reservoir level.

A-1.3 Concrete Gated Spillway

The spillway consists of a multiple-arched, reinforced concrete ogee crest structure with a reinforced concrete apron and stilling pool. Flow through the spillway is controlled by two Tainter (radial) gates. The left gate (Gate No. 1) is 20 feet 6 inches wide and 10 feet high, and the right gate (Gate No. 2) is 23 feet 7 inches wide and 10 feet high. Gate No. 1 is adjacent to the powerhouse. The gates were originally operated by a single electric chain hoist system mounted on a cart that traveled on rails between the gates. The gates could not be fully opened[2] by the original chain hoist system, so in 2019 (based on information provided during an interview with one of the dam operators), the electric hoist system was replaced with hydraulically operated cable hoists for each gate. With the new hoist system the gates can be fully opened. See Section 2.4 in the main report for more details of the gate operations.

A-1.4 Operations and Maintenance

A-1.4.1 Normal Spillway Gate Operation and Maintenance Procedures

The 2016 updated STID for Secord Dam (Boyce Hydro 2016) provides the following gate and hoist system maintenance schedule:

- Monthly – Visual inspection of all gates
- Annually – Visual inspection of hoist components, including roller chains
- Annually – Test operation (partial opening) of each gate
- Every 5 years – Test operation of all gates in fully open position

A-1.4.2 Normal Reservoir Operating Rules

As with the other Boyce dams, the operating rules for Secord Lake were to maintain lake levels between +0.3 foot and -0.4 foot of the normal pool level (El. 750.8),[3] except during flood operations, when lake levels could be higher, or during winter drawdown operations. Winter drawdown could begin after December 15 and had to be completed by January 15. During winter operations, the minimum lake level was Elevation (El.) 747.8, 3 feet below normal pool level, and the daily fluctuation in lake level was not to exceed 0.7 foot. The lake was to be returned to normal pool level before the surface temperature of the lake reached 39 degrees Fahrenheit (°F).

A-1.5 Pertinent Project Data

- Dam Crest El. 757.8
- Spillway Concrete Crest El. 742.8

[2] The phrase “fully open” as used in this report refers to the ability to open the gate to the full height of the gate (e.g., 10 feet) above the spillway crest.

[3] Elevations cited in this report are in feet based on National Geodetic Vertical Datum of 1929 (NGVD29) unless otherwise indicated. NGVD29 = Plant Datum + 5.8 feet.

• Normal Reservoir Level	El. 750.8
• Normal Tailwater Level	El. 704.6
• Drainage Area (using a geographic information system [GIS])	186 sq. mi.
• Reservoir Volume	15,000 ac-ft
• Reservoir Surface Area[4]	920 acres

A-1.6 History of Modifications/Repairs[5]

- 1938 – Installation of sheet piles adjacent to the spillway and powerhouse to control seepage.
- 1948 – Replacement of turbine and generator due to fire.
- 1950 – Repair of spillway rollway slabs.
- Mid-1950s – Area downstream of the stilling basin was riprapped.
- 1966 – Right exterior spillway wall repairs.
- 1990 – Installation of piezometers below the powerhouse to verify uplift assumptions.
- 1991 – Repair of upstream base of the spillway "barrel" (left bay).
- Mid-1990s - Area downstream of the stilling basin was riprapped.
- 1996 – Repair of right abutment spillway wall (right bay). Consisted of anchoring a concrete overlay to the spillway wall to the existing counterforts.
- Date unknown – Installation of polyvinyl chloride seepage pipes through the left powerhouse wall.
- 2005 – Raising of low areas along the dam crest to the original dam crest elevation (El. 757.8).
- 2008 – Sheet pile cutoff wall repairs at connection to the concrete walls of the spillway and powerhouse.
- 2012 – Toe berm added to the far left downstream slope to provide stability and reduce the potential for transport of fines through the embankment.
- 2013 – Repair of spillway concrete rollway slabs by adding a reinforced concrete overlay.
- 2013 – Placement of additional riprap in locations along the upstream side of the embankment.
- 2014 – Scour repair at the connection of spillway and powerhouse tailrace slabs and the east corner of the spillway tailrace slab to reduce the potential for undermining of the slabs.
- 2019 –Individual hydraulically operated gate hoists replaced the single chain hoist and cart system.

[4] The reservoir surface area shown is based on 2016 Light Detection and Ranging (LiDAR) data. (USGS 2016a, Gladwin County). Other project documents may report different values for surface areas at normal pool.
[5] Much of the history of modifications/repairs for each dam is as described in various project documents; in some cases the descriptions in these documents use terminology that is not consistent with more typical industry terminology.

A-2 Smallwood Dam

Smallwood Dam is located 7.7 miles downstream from Secord Dam on the Tittabawassee River, about 13.5 miles upstream of Edenville Dam and about 34 miles upstream of Midland Michigan.[6] The dam consists of two embankments and a combined gated spillway and powerhouse structure, as shown in Figure A-2. The left embankment is approximately 800 feet long between the left abutment and the left side of the spillway. The combined spillway and powerhouse structure is about 83 feet wide. The right embankment, between the right side of the powerhouse and right abutment, is about 125 feet long. The spillway is a reinforced concrete structure with two Tainter (radial) gates. The powerhouse consists of a reinforced concrete substructure and brick superstructure that is connected to the right side of the spillway. The data provided here and in the following sections are primarily extracted from the STID for Smallwood Dam (Boyce Hydro 2018), unless otherwise noted.

Source of aerial image: Google Earth

Figure A-2: Aerial View of Smallwood Dam

A-2.1 Embankments

The earth embankments were constructed with the native sand, silt, and clay materials located on-site, and had an initial crest level of about El. 710. Foundation treatment consisted of removing topsoil prior to placing and compacting fill. The maximum height of the embankments is about 38 feet in the vicinity of the spillway and powerhouse. The design crest width is 8 feet, and the design upstream and downstream slopes are nominally 2H:1V and 2.5H:1V, respectively. Riprap was placed on the upstream slope, starting 4 feet below normal pool, and extending to 2 feet above normal pool (see Figure A-4 below), which is typical for all four dams). Transverse foundation drains, consisting of open-joint clay tile pipe, are spaced on approximately 15- to 20-foot centers below the downstream shell of the embankment. Each drain

[6] These distances are taken from the table on Figure 2-1 in the main report and are based on streamline data for the Tittabawassee River. (USGS 2020).

extends from approximately the centerline of the dam downstream to a toe drain trench that parallels the embankment. The upstream ends of the drains are connected with a longitudinal clay tile pipe drain. The clay tile pipes were covered with gravel.

Given the fact that the dam could not accommodate the probable maximum flood (PMF) without overtopping the embankments, a sheet pile wall was constructed on the upstream edge of the dam crest in 1999. The sheet piles were driven about 2 feet into foundation hardpan material. The top of the sheet pile wall was completed to El. 715.7, which effectively raises the dam crest by 5.7 feet and reportedly provides about 1 foot of freeboard during the estimated PMF. The sheet pile wall extended along the upstream face of the dam from the right abutment to the power plant and from the spillway for a distance of 350 feet to the left (north). At that point, the sheet pile wall turned downstream and then back into the river channel downstream from the spillway stilling basin. Following installation of the sheet pile wall, the phreatic surface within the embankment was reported to have dropped significantly. The location of the sheet pile wall is shown on Figure A-2.

In 2001, additional work was performed to improve the expected performance of the dam during extreme floods. The crest of the left embankment between the end of the sheet pile wall and left abutment was regraded to El. 708.7 to serve as an emergency overflow section during extreme flood events. In 2016/2017, the crest of the left embankment was modified again. For a distance of 260 feet to the left of the sheet piles the embankment crest was raised to El. 712. This modification effectively created a two-level emergency spillway and increased the return period of the flood event that would first result in flows directly in contact with the sheet pile wall at the dam crest. The 2016/2017 modifications were developed to address a then recently identified potential failure mode (PFM) concerning erosion and undermining of a section of the sheet pile wall extending from the dam crest down to the river. This could lead to erosion along the toe of the embankment within the enclosure of the sheet pile wall. The modification also included adding a 20-foot-wide strip of riprap in a toe trench along the sheet pile wall from the dam crest to the entrance into the channel.

The extent of the sheet pile into the left abutment was intended to stop where the depth of the abutment was much shallower. The height of the left embankment in the area of the emergency spillway section is reported to be about 5 to 10 feet above natural ground, before finally tapering to zero as natural ground rises near the abutment.

A-2.2 Powerhouse

The powerhouse is connected to the right side of the spillway and consists of a reinforced concrete substructure and brick superstructure. The powerhouse contains a single generating unit with a rated capacity of 1.2 megawatts. The maximum discharge capacity through the turbine is 702 cfs with a head of 28 feet. The turbine is an Allis Chalmers 76-inch diameter Type NX, with a capacity of 1.2 MW. The generator is an Allis Chalmers unit rated at 1500 KVA, 2300 volt, 3-phase, 60 cycle. The exciter rating is 36kW at 125 volts. The powerhouse normally operates with very little fluctuation in the reservoir level.

A-2.3 Concrete Gated Spillway and Auxiliary Spillway

The concrete spillway structure consists of a multiple-arched, reinforced concrete ogee crest structure with a reinforced concrete apron and stilling pool. Flows through the spillway are controlled by two Tainter (radial) gates. Both gates are 23 feet 5 inches wide and 10 feet 6 inches high. The gates are referenced as Gate No. 1 (adjacent to the powerhouse) and Gate No. 2. The gates were originally operated by a single electric hoist mounted on a cart that traveled on rails between the gates. The gates could not be fully opened by the original chain hoist system, so a hydraulically driven cable hoist system was installed in 2017 (based on information provided during an interview with one of the operators) which allows for fully opening the gates. See Section 2.4 of the main report for more information on the gate operations.

As discussed above, the left embankment was initially constructed to the dam crest elevation (El. 710) but has subsequently been modified by installation of a sheet pile wall and regrading to create a two-level auxiliary spillway: a 540-foot length at El. 708.7 and a 260-foot length at El. 712.

A-2.4 Operations and Maintenance

A-2.4.1 Normal Spillway Gate Operation and Maintenance Procedures

The 2018 updated STID for Smallwood Dam (Boyce Hydro 2018) indicates the following gate and hoist system maintenance schedule:

- Monthly – Visual inspection of all gates
- Annually – Visual inspection of hoist components including roller chains
- Annually – Test operation of each gate
- Every 5 years – Test operation of all gates in fully open position

A-2.4.2 Normal Reservoir Operating Rules

As with the other Boyce dams, the operating rules for Smallwood Dam were to maintain lake levels between +0.3 foot and -0.4 foot of the normal pool level (El. 704.8), except during flood operations, when lake levels could be higher, or during winter drawdown operations. Winter drawdown could begin after December 15 and had to be completed by January 15. During winter operations, the minimum lake level was El. 701.8, 3 feet below normal pool level, and the daily fluctuation in lake level was not to exceed 0.7 foot. The lake was to be returned to normal pool level prior to the surface temperature of the lake reaching 39°F.

A-2.5 Pertinent Project Data

• Dam Crest	El. 710
• Top of Sheet Pile Cutoff Wall	El. 715.7
• Gated Spillway Crest	El. 694.8
• Auxiliary Spillway Crest	El. 708.7; El. 712 left segment
• Normal Reservoir Level	El. 704.8
• Normal Tailwater Elevation	El. 676.8
• Drainage Area (using GIS)	306 sq. mi.
• Reservoir Capacity	6,000 ac-ft
• Reservoir Surface Area[7]	400 acres

[7] The reservoir surface area shown is based on 2016 LiDAR data. (USGS 2016a, Gladwin County). Other project documents may report different values for surface areas at normal pool.

A-2.6 History of Modifications

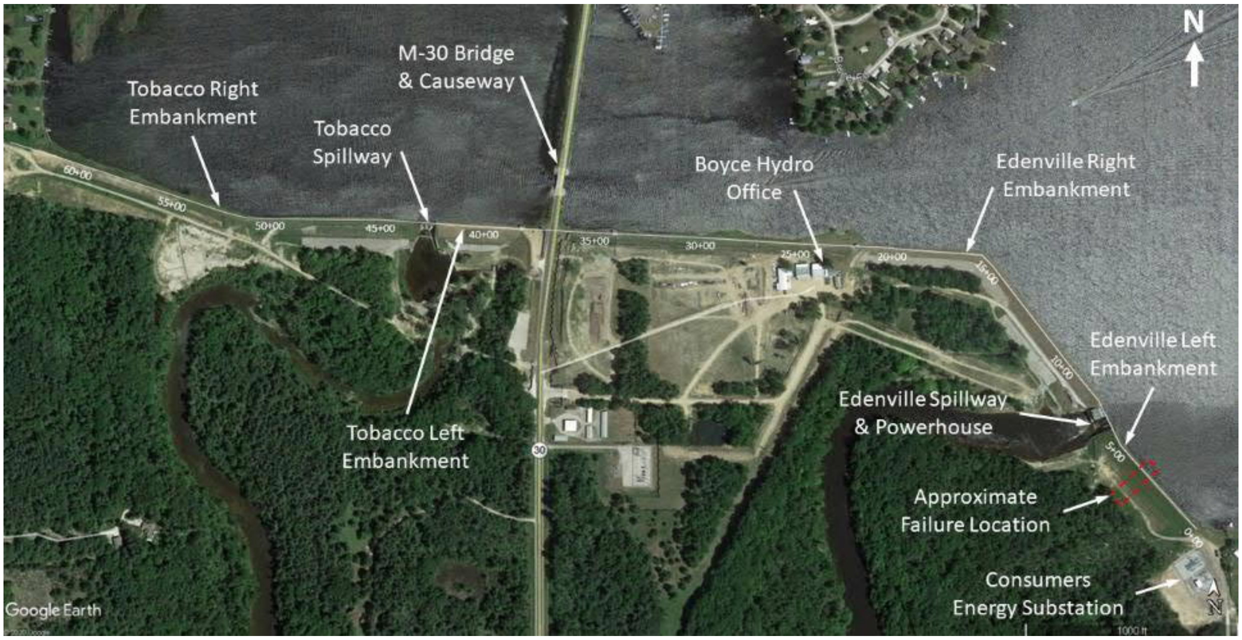
- 1927 – Additional material placed on the left downstream embankment.
- 1986 – Turbine unit overhauled. New headgates installed. Spillway gates rehabilitated. Spillway retaining walls repaired. Spillway slab rebuilt.
- 1993 – Water level monitors installed. Riprap added to downstream toe of dam. New controls and excitation installed. New bulkhead gate and log boom installed.
- 1994 – Generator rewound. Seepage measurement weir installed.
- 1999 – Sheet pile wall installed to effectively create an emergency spillway.
- 2001 – Section of left embankment crest (left of sheet pile wall) regraded to El. 708.7.
- 2001 – Turbine rehabilitated.
- 2011 – Scour below downstream powerhouse slab repaired.
- 2016 – 260-foot-long section of left embankment crest raised to El. 712 and riprap added to reduce the potential for erosion along the sheet pile wall extending down the embankment and back to the river
- 2017 – Individual hydraulically operated gate hoists replaced the single chain hoist and cart system.

A-3 Edenville Dam

At the time of the failure, the facility consisted of four earthfill embankments, two concrete spillways, and a powerhouse, all constructed across the Tittabawassee and Tobacco Rivers (about a half mile upstream from their original confluence), as shown in Figure A-3. Edenville Dam is located about 13.5 miles downstream from Smallwood Dam on the Tittabawassee River, about 11.5 miles upstream from Sanford Dam on the Tittabawassee River, and about 9 miles downstream from Beaverton Dam on the Tobacco River.[8] The data provided here and in the following sections are primarily extracted from the STID for Edenville Dam (Boyce Hydro 2015a) unless otherwise noted.

A causeway on Michigan Highway 30 (M-30) effectively divided the lake, with the east side impounding water from the Tittabawassee River and the west side impounding water from the Tobacco River. A causeway bridge opening on the M-30 hydraulically connected the two sides of the lake: the Tittabawassee side (also known as the Edenville side) and the Tobacco side.

[8] These distances are taken from the table on Figure 2-1 in the main report and are based on streamline data for the Tittabawassee River (USGS 2020).



Source of aerial image: Google Earth

Figure A-3: Aerial View of Edenville Dam

A-3.1 Embankments

A-3.1.1 Edenville Side

The Edenville side included two embankment sections, a spillway, and the powerhouse. The Edenville left embankment extended from the left (east) abutment to the Edenville spillway, a length of approximately 625 feet. The maximum height of this embankment was about 52 feet immediately adjacent to the spillway and about 32 feet further to the left.

The Edenville spillway and the powerhouse comprise a single, combined structure. The Edenville spillway structure is 68.6 feet wide, and the powerhouse is 50.6 feet wide, for a total structure width of 119.2 feet.

The Edenville right embankment extends from the powerhouse to the M-30 in a dogleg pattern, for an embankment length of about 2,900 feet. The maximum height of this embankment is about 50 feet immediately adjacent to the powerhouse and about 40 feet further to the right.

According to the original design drawings, the design crest level is El. 682.8, the design crest width of the embankments is 8 feet, and the upstream and downstream slopes of the embankments are nominally 2.5H:1V and 2H:1V, respectively. However, downstream slopes were flattened, and berms were added in some locations, and survey data show that the downstream slope is steeper than 2H:1V in some locations. In addition, survey data after the flood event indicate that the upstream slope was steeper than 2.5H:1V in some locations. An original design drawing (see Figure A-4) shows the limits of riprap placed on the upstream slope as starting 4 feet below normal pool and extending to 2 feet above normal pool. Transverse foundation drains, comprising open-joint clay tile pipe, are spaced on approximately 15 to 20-foot centers below the downstream shell of the embankment. Each drain extends from approximately the center line of the dam to a toe drain trench that parallels the embankment. The upstream ends of the drains are connected with a longitudinal clay tile pipe drain. The clay tile pipes were covered with gravel. In some locations, the downstream ends of the clay tile pipes have been covered by construction of flattened slopes or berms.

A-3.1.2 Tobacco Side

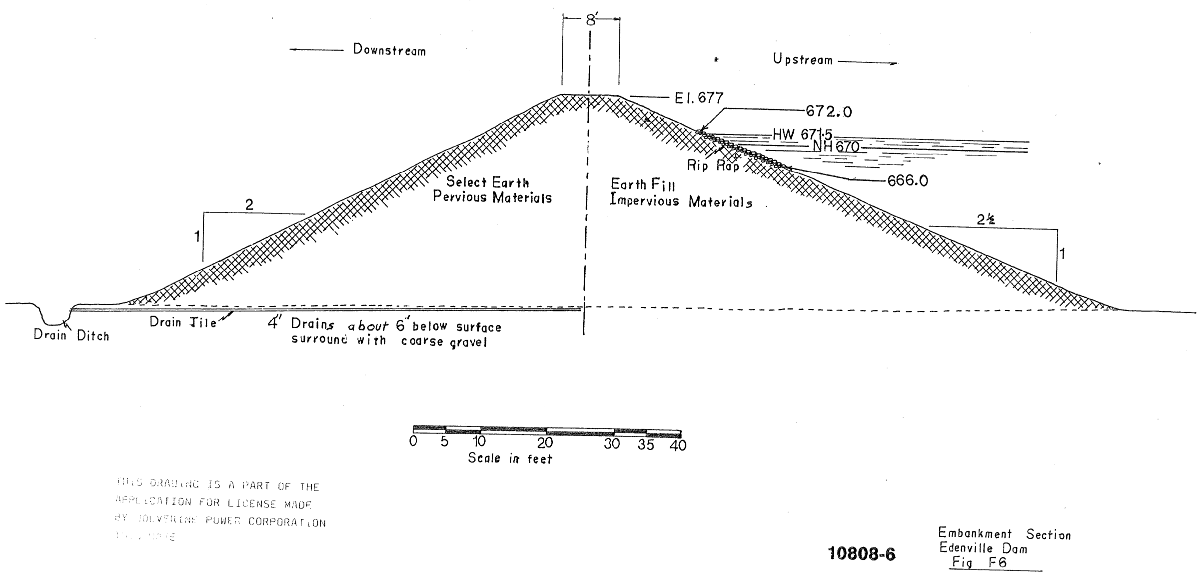
The Tobacco side includes two embankment sections and a spillway. The Tobacco left embankment extends from the M-30 crossing to the Tobacco Spillway, a length of approximately 520 feet. The maximum height of this embankment is about 47 feet in the vicinity of the spillway. The Tobacco spillway structure is 72.2 feet wide.

The Tobacco right embankment extends from the Tobacco spillway to the right (west) abutment, for an embankment length of about 1,895 feet. The maximum height of this embankment is about 47 feet adjacent to the spillway and about 32 feet further to the right.

Again, as on the Edenville side, according to the original design drawings, the design crest level is El. 682.8, the design crest width of the embankments is 8 feet and the upstream and downstream slopes of the embankments are nominally 2.5H:1V and 2H:1V, respectively. However, downstream slopes have been flattened and berms have been added in some locations, and survey data show that the downstream slope is steeper than 2H:1V in some locations. According to a design drawing (see Figure A-4), riprap was placed on the upstream slope, starting 4 feet below normal pool, and extending to 2 feet above normal pool. Transverse foundation drains, comprising open-joint clay tile pipe, are spaced on approximately 20-foot centers below the downstream shell of the embankment. Each drain extends from approximately the center line of the dam to a toe drain trench that parallels the embankment. The upstream ends of the drains are connected with a longitudinal clay tile pipe drain. The clay tile pipes were covered with gravel. In some locations, the downstream ends of the clay tile pipes have been covered by construction of flattened slopes or berms.

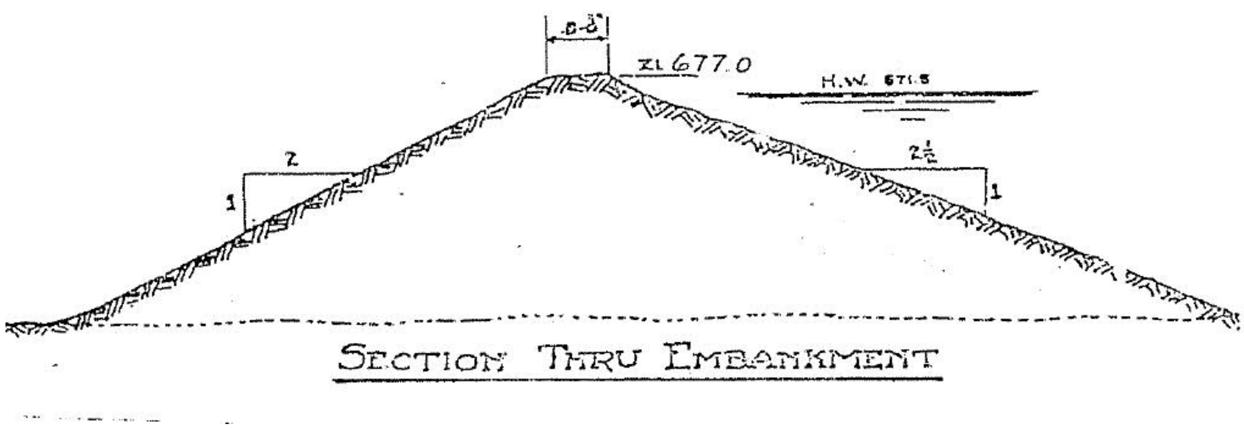
A-3.1.3 Embankment Cross Section

There is uncertainty regarding the internal cross section of the embankment due to limited and contradictory documentation of original construction. No as-built drawings of the embankments were found in the available documents. Original construction specifications (Holland, Ackerman, & Holland 1924a and 1924b*)* and renderings of the embankment in license exhibits (see Figure A-4) indicate a two-zoned embankment, in which material upstream of the embankment centerline would comprise “impervious” fill and material downstream of the embankment centerline would comprise pervious fill. However, drawings in other project documents do not reflect zoning within the embankment as shown in Figure A-5. The cross section shown in Figure A-5 was taken from an early (1923) design drawing that also shows a plan view alignment of the Edenville embankments that does not match the as-built alignment, indicating the design may have evolved between 1923 and when construction started in 1924.



Note: Drawing elevations are in Plant Datum (NGVD29 = Plant Datum + 5.8')

Figure A-4: Rendering of Edenville Dam Cross Section from Exhibit F License Figure (Wolverine n.d.)



Note: Drawing elevations are in Plant Datum (NGVD29 = Plant Datum + 5.8')
Source: Holland, Ackerman, & Holland 1923.

Figure A-5: Edenville Dam Cross Section from 1923 Design Drawing

Based on photos and memorandums from original construction (included in Appendix D), surface mapping of the breach face performed by IFT members (included in Appendix E), and subsurface drilling performed prior to and after the failure (summarized in Appendix B), the following understandings and uncertainties were developed by the IFT regarding the cross section of the Edenville embankments, with a focus on the Edenville left embankment where the May 2020 failure occurred:

- Based on construction specifications and construction memorandums, the embankments were constructed with sand, silt, and clay materials borrowed on site. The clay borrow is described in some documents as being taken from the river “flats” upstream of the embankment. Clay borrow deposits were also referenced as being located along the east side of the Tittabawassee River.

- An available excerpt from the original design specifications (Holland, Ackerman & Holland 1924a and 1924b) for this project indicated that the fill was to be placed and compacted in layers (see Appendix D). Construction photographs (see Appendix D) do not show compaction equipment, and some photographs show fill being dumped from rail carts (Edenville side) and from trestles (Tobacco side). Test borings completed since the 1980s indicate low blow counts in the embankment soils. Consequently, it appears likely that the embankment soils were not consistently compacted, if compacted at all.
- Based on construction photos (see Appendix D), there is evidence that suggests some effort at embankment zoning may have been employed in the construction of the Edenville left embankment. This includes evidence of color distinctions between material hauled from east of the embankment in rail carts being generally of darker color, and material stockpiled from excavations downstream of the embankment being of lighter color. The darker material was most typically visible being placed in the upstream section of the left embankment and the lighter color material was most typically visible being placed in the downstream section of the left embankment. The upstream and downstream sections of the dam were also visibly different at different time periods during the fill placement.
- Borings completed within the upstream slope of the embankments were not available to improve the understanding of upstream and downstream zoning. However, borings performed along the crest encountered areas of both clay and sand fill within the Edenville left embankment, while they predominantly encountered sand fill in the Edenville right and Tobacco embankments. A limited number of borings completed within the downstream slope of the Edenville right and Tobacco embankments also encountered sand fill.
- Correspondence memorandums between a representative from the designer (H. K. Holland) and the on-site resident engineer during construction refer to sand embankments north of the Edenville powerhouse, and clay fill located within the embankment south of the powerhouse. However, specific zones or areas of clay fill were not referenced in the correspondence.
- IFT members performed a site investigation which included trenching and cleaning the left and right faces of the remnant embankment at the breach location. This investigation is described in Appendix E. Findings indicate a predominantly clay fill with sandy soil along the outer slopes in the embankment section near the spillway and what appeared to be some zoning within the embankment section near the left abutment, which consisted of clay soil upstream of centerline in the lower section, sand fill downstream of centerline in the lower section, and a soil mixture (silty, clayey, sand) in the upper embankment section.
- An excavation completed through the Edenville right embankment as part of rehabilitation design efforts following the failure encountered a relatively homogenous sand embankment (GEI 2022)

Based on the above observations, the IFT postulates that the embankments north of the powerhouse, including the Edenville right and Tobacco left and right embankments, are likely to be relatively homogenous, comprised of sand fill with fines contents (silt and sand) typically less than 20 percent. Lower segments of these embankments were reportedly constructed with a loam fill that may have had a higher clay content. A portion of these embankments was modified during construction to include a 2-foot-thick clay blanket along the upper portion of the upstream slope. The exact location and elevation range of this clay blanket was not mentioned in the correspondence. The Edenville left embankment, where the failure occurred, appears to have some semblance of zoning, as described in more detail in Appendix E.

In 1934 through 1936, steel sheet pile wall sections were constructed on each side of the combined spillway/powerhouse on the Edenville side, and on each side of the Tobacco side spillway. In addition, another section of steel sheet pile wall was constructed about 900 feet west of the Tobacco spillway. All these segments of sheet pile walls were installed along the upstream faces of the embankments, and then covered with a concrete cap. The intent of the sheet pile walls was to control seepage in these locations.

A-3.1.4 Embankment Foundations

Based on test borings (McDowell 2005; Mill Road Engineering 2010; GEI 2022), the embankment foundations consist of a layer of uniform, medium dense to dense sand over a very stiff clay hardpan (Glacial Till) unit. The depth to rock is unknown.

A-3.2 Powerhouse

The powerhouse is located immediately adjacent to the right side of the Edenville spillway and consists of a reinforced concrete substructure and concrete and brick superstructure. The powerhouse contains two vertical shaft generating units with a rated capacity of 2.4 megawatts each, for a total of 4.8 megawatts. The maximum discharge capacity through each turbine is 986 cfs with a head of 44.5 feet. The turbines are identical 73-inch Francis-type Allis Chalmers turbines with a rating of 3,000 horsepower (HP) at 138 revolutions per minute (rpm) and a head of 45 feet. The turbines are connected to twin 3,000 kilovolt-ampere, 2,300-volt, 3-phase, 60-cycle Allis Chalmers generators. The generators operate at 138 rpm with directly connected 37 kilowatt, 125-volt exciters (Boyce Hydro 2000). The powerhouse normally operated with very little fluctuation in the reservoir level.

A-3.3 Concrete Gated Spillways

As previously stated, there are two spillways for the Edenville Dam, one on the Edenville side and one on the Tobacco side. Each spillway consists of a multiple-arched, reinforced concrete ogee structure with a reinforced concrete apron and stilling pool. The gates were originally designed to be operated by a single electric chain hoist mounted on a cart that traveled on rails between the gates. The gates could only be opened 6 to 7 feet with the original chain hoist systems. A portable A-frame system was developed by Boyce Hydro between 2012 and 2015 to extend the gate travel to the fully open position. During a gate operations test conducted in 2019, the A-frame system was judged to be unsafe for the operators and potentially damaging to the gate connection brackets, which could lead to a gate failure. See Section 2.4 of the main report for more details on the gate operations.

A-3.3.1 Edenville Spillway

The Edenville spillway is located adjacent to and on the left (east) side of the powerhouse. The spillway includes three Tainter (radial) gates. Two of the gates (Gate No.1 and Gate No. 2) are 20 feet wide by 9 feet 6 inches high, and one gate (Gate No. 3) is 23 feet 7 inches wide by 9 feet 6 inches high. Gate No. 1 is located adjacent to the powerhouse. The gate sills are at El. 667.8, 8 feet below normal pool level (El. 675.8). Low-level gated sluiceways were constructed through the rollover section of the spillway bay adjacent to the powerhouse. However, the sluiceways were not operational for decades prior to the failure.

A-3.3.2 Tobacco Spillway

The Tobacco spillway is located approximately 520 feet west of the M-30. The reinforced concrete spillway contains three Tainter (radial) gates. Two of the gates (Gate No.1 and Gate No. 3) are 23 feet 7 inches wide and 9 feet 6 inches high, and one gate (Gate No. 2) is 20 feet wide and 9 feet 6 inches high. Gate No. 1 is located in the left (east) bay. The gate sills are at El. 667.8, 8 feet below normal pool level. As with the Edenville Side spillway, low-level gated sluiceways were constructed through the rollover section of the center spillway bay but were inoperable for decades prior to the failure.

A-3.4 Operations and Maintenance

A-3.4.1 Normal Spillway Gate Operations and Maintenance Procedures

According to project documents, the spillway gates and hoist systems at Edenville Dam were maintained in accordance with the following schedule, as stated in the updated 2016 STID for Secord Dam (Boyce Hydro 2016):

- Monthly – Visual inspection of all gates
- Annually – Visual inspection of hoist components including chains
- Annually – Test operation of each gate
- Every 5 years – Test operation of all gates in fully open position

A-3.4.2 Normal Reservoir Operating Rules

As with the other Boyce Dams, the operating rules for Wixom Lake, the reservoir impounded by Edenville Dam, were to maintain lake levels between +0.3 foot and -0.4 foot of the normal pool level (El. 675.8) except during flood operations, when lake levels could be higher, or during winter drawdown operations. Winter drawdown could begin after December 15 and had to be completed by January 15. During winter operations, the minimum lake level was El. 672.8, 3 feet below normal pool level, and the daily fluctuation in lake level was not to exceed 0.7 foot. The lake was to be returned to normal pool level before the surface temperature of the lake reached 39°F. Minimum flows are released into the Tobacco River at a rate of 66 cfs between April 1 and September 30, and at a rate of 40 cfs between October 1 and March 31.

A-3.5 Pertinent Project Data

• Dam Crest	El. 682.8
• Spillway Crest	El. 667.8
• Normal Reservoir Level	El. 675.8
• Normal Tailwater Level	El. 627.4
• Drainage Area (using GIS)	932 sq. mi.
• Reservoir Volume	36,000 ac/ft.

- Reservoir Surface Area [9] 2,270 acres

A-3.6 History of Modifications

- 1924-1925 – Original Construction
- 1934-1936 – Sheet pile cutoff walls installed on both sides of Tittabawassee and Tobacco spillway structures and in the embankment about 900 feet west of the Tobacco spillway.
- 1947 – Repairs made to Tobacco spillway concrete apron.
- 1991 – Installation of Tittabawassee spillway piezometers.
- 2004 – Toe filter drains constructed along a 340-foot-long section of the right Edenville embankment and a 300-foot-long section of the left Tobacco embankment.
- 2004 – Left training wall repaired downstream of the Tittabawassee spillway.
- 2004 – Concrete deterioration of the Tobacco spillway piers repaired.
- 2005 – Reverse toe filter drain constructed on a section of the downstream slope of the right abutment of the Edenville embankment.
- 2007 – Extension of reverse toe filter drain constructed in 2005.
- 2008 – Edenville sheet pile cutoffs extended.
- 2009 – Improvements to the “barrel drain” drainage system along the toe of the right abutment about 450 feet right of the Tobacco spillway.
- 2009 – Toe drains, filter overlays, and clay drain re-sleeve at Edenville right embankment (Station 15 to 20).
- 2010 – Edenville spillway retaining wall repair and barrel drain improvements.
- 2011 – Concrete bags placed along the downstream end of the tailrace and spillway aprons to address erosion and undermining that was noted during an underwater inspection following a significant flow event.
- 2011 to 2012 – Toe drain extension and installation west of the M-30 along a section of the Tobacco side embankment.
- 2012 – Inspection and cleaning of foundation clay tile pipe drains.
- 2014 – Replacement of the remaining barrel drain system that was started in 2009.
- 2014 – Scour (undermining) along the spillway apron, which was noted during an underwater inspection of the downstream side of the spillway and tailrace aprons, repaired by installing concrete mix bags and anchoring them into the foundation.
- 2015 – Completion of the barrel drain system replacement on the downstream side of the Tobacco side embankment.

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[9] The reservoir surface area shown is based on 2016 LiDAR data. (USGS 2016b, Midland County). Other project documents may report different values for surface areas at normal pool.

- 2012 to 2015 – Fabrication of A-frame system for lifting spillway gates.
- 2015 – Replacement of two of the downstream apron slabs of the Tittabawassee spillway, construction of overlay of powerhouse training wall, and placement of sheet piling at downstream edge of paving slabs.
- 2016 – Tittabawassee spillway apron slab repaired.
- 2020 – Repairs to deteriorated concrete on wing walls and pier noses and to Edenville and Tobacco spillways completed; new gate lifting lugs installed in preparation for new automatic gate operators (to replace the A-frame system).

Locations of the key embankment modifications are shown on Figure B-6 included in Appendix B.

A-4 Sanford Dam

At the time of the failure, Sanford Dam consisted of three embankments, a fuse plug spillway, a gated spillway, and a powerhouse, as shown in Figure A-6. Sanford Dam is located 11.5 miles downstream from Edenville Dam on the Tittabawassee River, and about 10.5 miles upstream of Midland Michigan.[10] The left embankment is approximately 160 feet long between the left abutment and the left side of the powerhouse. The maximum height of this embankment section was estimated to be about 34 feet, based on interpretation of test boring data. The combined spillway and powerhouse are about 219 feet wide. The center embankment extends in a dogleg pattern from the right side of the gated spillway to the fuse plug spillway and is about 300 feet long. This section of the embankment is estimated to be about 34 feet high near the spillway. The fuse plug spillway is located between the center and right embankments with a crest length of 190 feet. The right embankment, between the right side of the fuse plug spillway and the right abutment, is about 710 feet long. The maximum height of this embankment section was estimated to be about 36 feet based on interpretation of test boring data. The data provided here and in the following sections are primarily extracted from the STID for Sanford Dam (Boyce Hydro 2017b), unless otherwise noted.

[10] These distances are taken from the Table on Figure 2-1 in the main report and are based on streamline data for the Tittabawassee River. (USGS 2020)

Source of aerial image: Google Earth

Figure A-6: Aerial View of Sanford Dam

A-4.1 Embankments

The earth embankments were constructed with native soil materials borrowed on-site and have a design crest level at El. 636.8 feet. Foundation treatment consisted of removing topsoil prior to placing and compacting fill. The dam foundation consists of clay with interspersed layers of silty sand with some gravel overlying a very stiff clay hardpan unit. The upstream and downstream design slopes are nominally 2.5H:1V and 2H:1V, respectively, with an original design crest width of 8 feet. Riprap was placed on the upstream slope, starting 4 feet below normal pool and extending to 2 feet above normal pool. See Figure A-4 above, which is typical for all four dams. Transverse foundation drains, consisting of open-joint clay tile pipe, are spaced on approximately 20-foot centers below the downstream shell of the embankment. Each drain extends from approximately the centerline of the dam to a toe drain trench that parallels the embankment. The upstream ends of the drains are connected with a longitudinal clay tile pipe drain. The clay tile pipes were covered with gravel.

Due to the inability of the dam to accommodate the inflow design flood (IDF) without overtopping the embankments, an auxiliary fuse plug spillway was constructed in 2002. The fuse plug spillway was constructed over a section of previously existing embankment, between what became designated as the right and center embankments.

A-4.2 Concrete Gated Spillway and Fuse Plug Spillway

Sanford Dam includes two spillway structures: a concrete gated spillway and a fuse plug spillway.

The gated concrete spillway consists of a multiple-arched, reinforced concrete ogee structure with a reinforced concrete apron and stilling pool. Flow through this spillway is controlled by six Tainter (radial) gates: Gate No. 1 and Gate No. 6 are 25 feet 4 inches wide and 10 feet high, and the remaining four gates (Gate No. 2 through Gate No. 5) are 22 feet wide and 10 feet high. The gate sills are at El. 622.3, 8.5 feet below normal pool level (El. 630.8). The gates were originally operated by a single electric chain hoist mounted on a cart that traveled on rails between the gates. The gates could not be fully opened by the original chain hoist systems. See Section 2.4 in the main report for more details of the gate operations. In 2013, the chain hoist system was replaced by an electric steel cable hoist system on Gate No. 2, Gate No. 3, and Gate No. 4. In 2015, the chain hoist system on the remaining gates (Gate No. 1, Gate No. 5, and Gate No. 6) were replaced with electric steel cable hoists (Boyce Hydro 2015b). With the new hoist system, the gates can be opened to their full height. Spillway Bay No. 5 included a sluiceway consisting of two slide gates installed in the normal rollway section of the spillway to allow release of reservoir water below the spillway crest elevation. However, the slide gates have reportedly not been used for decades and are rusted in the closed position and inoperable.

In 2002, an auxiliary spillway fuse plug was constructed through a section of the right embankment to supplement spillway flows to allow for passage of the IDF, which was judged to be approximately 49 percent of the PMF based on an incremental damage assessment. The fuse plug spillway section is 190 feet long, with a fuse plug embankment crest at El. 634.8 and a concrete crest elevation of 631.8 (1 foot above the normal reservoir level); the fuse plug design capacity is reported to be 5,100 cfs. The design for this fuse plug included a height between top of fuse plug and non-erodible surface (concrete sill) of only 3 feet, presumably to minimize the loss of reservoir water after operation and washout of the fuse plug. The design of the fuse plug components was in accordance with Bureau of Reclamation criteria (Reclamation 1985). To compensate for the limited operable height, the length of the fuse plug was relatively long to provide the discharge capacity required. The limited operable height may have led the designers to exclude the use of pilot channels, which are normally included in fuse plug designs to accelerate the breach of the fuse plug by facilitating lateral erosion. The fuse plug pilot channel is usually constructed of more erodible material, which initiates the process of lateral erosion more quickly. Lateral erosion is the preferred method of removing the fuse plug because it occurs at a predictable rate and controls the rate of rise in discharge. The width of the fuse plug crest was approximately 8 feet, and the downstream slope was relatively flat (4H:1V). The wider crest and relatively flat downstream slopes will tend to slow down the erosion process, especially since the erosion is dependent on eroding from the top down.

Other factors that affected the erosion process at the Sanford fuse plug spillway were the possible compaction of the embankment along the crest and vegetation along the downstream slope of the fuse plug. The IFT noted that, in pre-failure photographs, ramps had been constructed from the dam crest down to the fuse plug crest. Interviews revealed that light vehicular traffic was allowed to cross over the top of the fuse plug crest, which could have compacted the soil at the crest, making it more erosion resistant. In addition, the 2017 inspection report indicated some light vegetation growth in the erodible fill. Both factors would slow the rate of erosion of the fuse plug.

The factors noted above may have slowed the overall rate of erosion of the fuse plug, but the discharge from the Edenville Dam breach was significantly greater than the combined capacity of both spillways at Sanford Dam. It is the IFT’s opinion that the outcome of the May 2020 event would have been essentially the same even if the fuse plug had eroded more quickly.

A-4.3 Operations and Maintenance

A-4.3.1 Normal Spillway Gate Operation and Maintenance Procedures

According to project documents, the spillway gates and hoist systems at Sanford Dam were maintained in accordance with the following schedule, as stated in the STID for Sanford Dam (Boyce Hydro 2017b):

- Monthly – Visual inspection of all gates
- Annually – Visual inspection of hoist components, including cables and chains

- Annually – Test operation of each gate
- Every 5 years – Test operation of all gates in fully open position

A-4.3.2 Normal Reservoir Operating Rules

As with the other Boyce Dams, the operating rules for Sanford Dam were to maintain lake levels between +0.3 foot and -0.4 foot of the normal pool level, except during flood operations, when lake levels could be higher, or during winter drawdown operations. Winter drawdown could begin after December 15 and had to be completed by January 15. During winter operations, the minimum lake level was El. 627.8, 3 feet below normal pool level, and the daily fluctuation in lake level was not to exceed 0.7 foot. The lake was to be returned to normal pool level before the surface temperature of the lake reached 39°F.

A-4.4 Powerhouse

The powerhouse consists of a reinforced concrete substructure and concrete and brick superstructure. The original design of the powerhouse included three vertical shaft generating units with a rated capacity of 1.1 megawatts each, for a total of 3.3 megawatts. The units were upgraded in 2015 and 2016 to 1.2 megawatts each, for a total of 3.6 megawatts. A new turbine was installed in Bay No. 1 in 2016 to satisfy minimum downstream flow requirements (Boyce 2017a). The three original Allis Chalmers turbines were 76-inch-diameter Type NX, with a rating of 1,800 HP at 225 rpm and a discharge of 727 cfs at a head of 28 feet. The 1,375 KVA, 2,300-volt, 3-phase, 60-cycle Allis Chalmers generators operate at 225 rpm with a direct connected 125-volt exciter. The powerhouse normally operates with very little fluctuation in the reservoir level.

A-4.5 Pertinent Project Data

• Dam Crest	El. 636.8
• Gated Spillway Concrete Crest	El. 622.3
• Fuse Plug Embankment Crest	El. 634.8
• Fuse Plug Concrete Crest	El. 631.8
• Normal Reservoir Level	El. 630.8
• Normal Tailwater Elevation	El. 603.0
• Maximum Gated Spillway Discharge	23,600 cfs
• Maximum Fuse Plug Discharge	5,100 cfs
• Drainage Area (using GIS)	1140 sq. mi.
• Reservoir Volume	15,000 ac-ft
• Reservoir Surface Area[11]	1,550 ac.

[11] The reservoir surface area shown is based on 2016 LiDAR data. (USGS 2016b, Midland County). Other project documents may report different values for surface areas at normal pool.

A-4.6 History of Modifications

- 1948 – Construction of cement bag retaining wall on toe of right (now center) embankment.
- 1950s – Extension of south training wall downstream of powerhouse.
- 1983 – Walkway upstream of powerhouse replaced.
- 1985 – Spillway gate pins and piers repaired.
- 1991 – Riprap placed downstream from Spillway Bay No. 1 for erosion protection.
- 1992 – Additional riprap for erosion protection placed. Dam drainage improvements installed. Rewound generator. Stopped leak in ogee and covered exposed rebar with hydraulic cement.
- 1992 – Installed canoe portage on right abutment.
- 1993 – Installed water level monitors. Riprap protection installed along 300 feet of right embankment. New controls and excitation installed. Bulkhead gate repaired. New log boom floats installed.
- Date Unknown – Anchors and tie back systems for retaining walls of the spillway and powerhouse installed.
- 2002 – Auxiliary fuse plug spillway constructed to supplement spillway flows and satisfy IDF requirement.
- 2011 – Reverse filter blanket (toe drain) installed to the right of the fuse plug.
- 2011 – Toe drains installed behind the concrete sack armor between the gated concrete spillway and fuse plug.
- 2011 – Underwater inspection of spillway slab undermining.
- 2013 – Underwater inspection for spillway and powerhouse slab undermining.
- 2013 – Electric chain hoists replaced with electric cable hoists for three spillway gates: Gate No. 2, Gate No. 3, and Gate No. 4.
- 2014 – Turbine Unit No. 3 upgraded.
- 2015 – Electric chain hoists replaced with electric cable hoists for three spillway gates: Gate No. 1, Gate No. 5, and Gate No. 6. This included strengthening the mounting beams for the two larger gates (Gate No. 1 and Gate No. 6).
- 2015 – Left abutment widened and armored. Placement of concrete overlay for south training walls of powerhouse intake and tailrace.
- Unknown date – A new turbine installed at Bay No. 1 to provide minimum release from the reservoir.
- 2017 – Removal of damaged sack concrete wall on center downstream bank above the north training wall of the spillway.

A-5 References

Boyce Hydro Power, LLC (Boyce Hydro). 2000 (June). Consultant's Safety Inspection Report, Part 12, *Edenville Hydroelectric Project, P-10808.*

______. 2015a (April). *Supporting Technical Information Document, Edenville Hydroelectric Project, P-10808.*

______. 2015b (August 31). P-2785, P-10808, P-10809, P-10810 – Spillway Gate Tests. Letter from Boyce Hydro Power, LLC to FERC.

______. 2016 (December). *Supporting Technical Information Document, Secord Hydroelectric Project, P-10809.*

______. 2017a (December). *Consultant's Safety Inspection Report, Sanford Hydroelectric Project, P-2785.*

______. 2017b (December). *Supporting Technical Information Document, Sanford Hydroelectric Project, P-2785.*

———. 2018 (December). *Updated STID Smallwood Hydroelectric, Project P-10810.*

Bureau of Reclamation (Reclamation). 1985 (December). *Hydraulic Model Studies of Fuse Plug Embankments, REC-ERC-85-7*. U.S. Department of the Interior, Bureau of Reclamation.

Federal Energy Regulatory Commission (FERC). 2018 (September 10). Order Revoking License, Project No. 10808-058. Issued by the Federal Energy Regulatory Commission.

GEI Consultants (GEI). 2022 (April). *Geotechnical Data Report, Edenville Dam.* Prepared for Four Lakes Task Force (FLTF).

Google Earth. 2022. Available: https://earth.google.com/web/.

Holland, Ackerman, & Holland. 1923 (March 19). Edenville Plant Plan and Sections. Drawing 7606. Included as Figure F1(10808-1) of FERC License Exhibit F for Project P-108080.

Holland, Ackerman, & Holland. 1924a. Agreement and Specification for the Construction of the Earth Embankments – Central Section of the Edenville Plant for J. N. Bick General Contractor.

Holland, Ackerman, & Holland. 1924b. Agreement and Specification for the Construction of the Earth Embankments – East End of the Edenville Plant for J. N. Bick General Contractor.

McDowell & Associates (McDowell). 2005 (July 18). *Soils Investigation, Edenville Hydroelectric Project, Auxiliary Spillway Construction, Edenville and Meridian Roads.*

Mill Road Engineering. 2010 (December 29). *Liquefaction Analysis, Edenville Hydroelectric Project.*

Holland, Ackerman, & Holland. 1923 (March 19). Edenville Plant Plan and Sections. Drawing 7606. Included as Figure F1(10808-1) of FERC License Exhibit F for Project P-108080.

U.S. Geological Survey (USGS). 2016a. Gladwin County One Meter LiDAR Data. Available: https://apps.nationalmap.gov/downloader/#/.

———. 2016b. Midland County One Meter LiDAR Data. Available: https://apps.nationalmap.gov/downloader/#/.

———. 2020. National Hydrography Dataset (NHD), Hydrological Unit (HU) 8 - 04080201. September 7.

Wolverine Power Corporation (Wolverine). n.d. Figure F6 (10808-6) of FERC License Exhibit F for Project P-108080

Appendix B: Geotechnical Data of Edenville Dam

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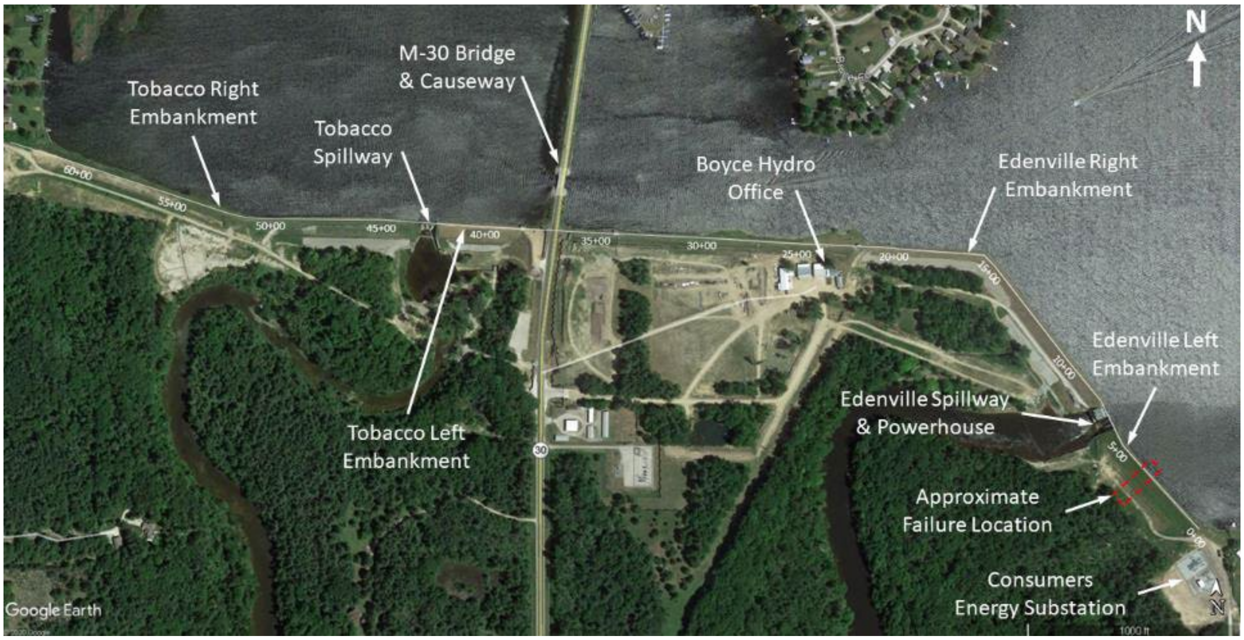
B-1 Introduction

At the time of the May 2020 failure, Edenville Dam consisted of four earthfill embankments (the Edenville left, Edenville right, Tobacco left, and Tobacco right embankments) as shown in Figure B-1, two gated concrete spillways, and a powerhouse—all constructed across the Tittabawassee and Tobacco Rivers in Michigan and totaling more than 6,000 feet in length.



Source of aerial image: Google Earth

Figure B-1: Edenville Dam Configuration

B-2 Data Review

The following geotechnical data were reviewed during the Independent Forensic Team (IFT) investigation:

- Borings (Holland, Ackerman & Holland 1936, cited in Mead and Hunt 2005; SME 1987; McDowell 2003; McDowell 2005a and 2005b; Mill Road Engineering 2010; McDowell 2013; McDowell 2015; McDowell 2018; Somat Engineering 2020)
- Laboratory data (SME 1987; McDowell 2003; McDowell 2005a and 2005b; Mill Road Engineering 2010; Somat Engineering 2020)
- Geotechnical analyses (SME 1987; Blystra 1991; Mill Road Engineering 2009, 2010)
- Underdrain Investigation (Boyce Hydro 2014)
- Construction era photos 1924-1925 provided by Boyce Hydro
- Construction memos 1924-1925 (Boyce Hydro 1924-1925, cited in Mead and Hunt 2005)

Figure B-2 shows the overall project area with callouts for the various historical geotechnical evaluations. A total of 39 borings were performed along the Edenville alignment during the life of the project (17 were performed at or near the downstream toe, and 22 were performed through the crest or downstream slope).

Figure B-3 and Figure B-4 show the locations of the geotechnical investigations. The purpose, description, pertinent findings, and follow-on actions for each evaluation are summarized in Table B-1.

Figure B-5 shows the overall project area with callouts to the various inspection observations and recommendations from regular inspections and Part 12D Consultant Safety Inspection Reports. The figure does not include all findings from previous inspections, but highlights observations related to the geotechnical performance of the embankments.

Figure B-6 shows the overall project area with callouts to the various dam modifications, including filter toe drains (embankment overlays), barrel drain replacement, and sheet pile installation and repair. Figure B-6 does not show all known previous dam modifications (as listed in Appendix A); however, the figure does indicate the dam modifications that are significant to the geotechnical aspects of the IFT investigation.

Attachment B-1 of this appendix includes the report from the 2012 underdrain investigation (Boyce Hydro 2014).

The following historic information was also reviewed by the IFT and is included in Appendix D.

- Construction photos.
- Excerpts from construction memos (the full set of construction memos was not available).
- Excerpts from original construction specifications (full specifications were not available).
- A selected historical embankment drawing, which shows a different alignment than the one that was actually constructed.

B-3 Key Takeaways

The following are the key findings of the IFT's review of the historical geotechnical evaluations:

A. The embankment was constructed on native sands of varying thickness overlying a glacial till hardpan.

- In general, the soil overlying the glacial till consisted of a medium dense to dense sand. The native sand was found to be denser than the overlying embankment fill.

B. The embankment fill in the Edenville left embankment was different from the fill in the other embankments along the dam alignment. The Edenville left embankment contained zones of both sand fill and clay fill, whereas the other embankments along the dam alignment consisted predominantly of sand fill.

- A number of unconfined compression tests were completed on samples from the Edenville left embankment fill, while the samples from borings in other areas of the Edenville embankments were not tested for unconfined compression. This suggests that some of the embankment fill sampled in the Edenville left embankment was finer-grained and more cohesive than that in the other embankments.
- Available gradation data from the historical geotechnical evaluations shows that some of the fill in the Edenville left embankment had higher fines contents than the other areas of the Edenville dam embankments.

- The construction memos, provided in Appendix D, reference the "clay fill South of the retaining wall" in contrast with the "sand embankments" north of the powerhouse. The memos describe problems with seepage and sloughing on the north embankments, while the performance of the embankment "on the South side of the powerhouse" is described as showing "no signs of being wet" and no issues with sloughing are noted. The contrast in seepage performance between the Edenville left embankment and the Edenville right embankment is believed to be due to the higher clay content in some of the fill in the Edenville left embankment. The memos also discuss an effort to bring clay from the east end of the dam to the Edenville right embankment by hauling it over the spillway. This implies that a greater source of clay borrow existed on the east end of the dam (left abutment), which logistically explains why the Edenville left embankment may include more clay content material.
- In some of the construction photos, provided in Appendix D, there is a notable color difference between the fill placed in the upstream and downstream portions of the Edenville left embankment. The fill appears to be darker in the upstream portion of the dam. It is conceivable that the darker fill indicates a different material and/or a higher moisture content. The photos also show a large stockpile of lighter-colored material excavated from downstream of the powerhouse and placed in the downstream area of the embankment (presumably for spreading in the downstream portion of the embankment), while rail cars hauling material from the left (east) side of the river appeared to be dumping mostly darker-colored fill. The majority of the photos show the rail alignment along the upstream portion of the Edenville left embankment (with one exception). As the fill is shown at higher elevations, the large downstream stockpile is no longer present and the rail cars appear closer to the centerline of the embankment. It is possible that this illustrates some effort to accomplish the upstream/downstream zoning described in the original specifications (provided in Appendix D) along this embankment, particularly for the lower portions of the Edenville left embankment. Similar color variation and material placement methods were not observed in construction photos of the Edenville right embankment and the two Tobacco embankments.

C. The embankment fill throughout the alignment was found to have relatively low blow counts in both the sand and clay portions, indicating a lack of compaction.

- Table B-1 describes some of the raw blow count data encountered across the Edenville embankments. The majority of the blow counts in the embankment fill were found to be less than 10 blows per foot.
- No compaction equipment was observed in any of the construction photos included in Appendix D. Lack of compaction effort would explain the low blow counts encountered in the borings.

D. Stability analysis was limited to two cross sections of the embankment, and no analysis was completed for the Edenville left embankment (the segment containing the failure section). In addition, the Edenville left embankment was the tallest section remaining without a toe filter drain.

- The 1987 slope stability analyses were completed for Station (Sta.) 48+00 in the Tobacco right embankment. The report included analyses for the embankment as it was and for the embankment with a downstream weighted filter. The analyses calculated factors of safety (FS) below recommended minimum values without the weighted filter.

- The 1991 slope stability analyses were completed for Sta. 10+00 in the Edenville right embankment. The analyses calculated FS values above recommended minimum values, with the exception of the rapid drawdown analysis.
- The 2007-2009 slope stability analyses were completed for Sta. 48+00 in the Tobacco right embankment. The analyses were completed for the embankment with the addition of a toe filter drain. The analyses calculated FS values above recommended minimum values.
- The two cross sections analyzed in 1987 and 1991, and from 2007 to 2009 were the tallest sections of the embankment, which were each about 40 feet tall. The failure section (near Sta. 3+50) was the next tallest embankment section, with a height of about 30 feet, and had a steeper average downstream slope than Sta. 48+00 and Sta. 10+00.

E. Underdrains appear to be missing or blocked in the Edenville left embankment beneath the central portion of the breach location. Some of the underdrains surrounding the breach location were found to be broken or blocked.

- The underdrain investigation completed in 2012 for the Edenville left embankment (included in Attachment B-1) shows the typical drain spacing to be about 15 to 20 feet, while near the failure location, there is a 38-foot spacing between identified drains.
- The drain survey of the Edenville left embankment (provided in Attachment B-1) shows that some of the drains within the estimated breach location were found to be broken or blocked. Drain 5 was found to be broken at 37 feet in from the outfall, and Drain 7 was only accessible for the first 30 feet of drain line.
- The construction memos describe how the drains beneath the Edenville left embankment “do not run very freely.” However, later inspection reports, following decades of operation, indicate that the drains along this section did flow regularly.

B-4 References

A. R. Blystra & Associates (Blystra). 1991 (March). *Initial Consultant's Safety Inspection Report, Edenville Hydroelectric Project*.

Boyce Hydro Power, LLC (Boyce Hydro). 1924-1925. Construction Memos. Cited in Mead and Hunt 2005.

———. 2013 (November 29). *PMF Alterations Design Plan. Additional Evaluations Edenville P-10808, November 2013 Submittal*. Second Submittal (First Submittal September 26, 2013). Prepared by Christie Engineering.

———. 2014 (December). *Summary of Findings, Embankment Underdrain Investigations.*

———. 2015. *Supporting Technical Information Document, Edenville Hydroelectric Project, P-10808.*

Christie, Frank O., PE. General Manager, Wolverine Power Cooperative. June 16, 2003—letter to Peggy Ann Harding, PE, Regional Director of FERC, regarding Edenville Project No. 10808.

Gomez & Sullivan. 2018 (January 12). *Subsurface Investigation, Work Plan, Edenville Hydroelectric Project, FERC Project No. 10808.*

Google Earth. 2022. Available: https://earth.google.com/web/.

Holland, Ackerman & Holland. 1936. Sheet Pile Wall and Borings. Cited in Mead and Hunt 2005.

Mead and Hunt, Inc. (Mead and Hunt). 2005 (December). *Supporting Technical Information Document (final), Edenville Hydroelectric Project, FERC Project No. 10808.* [CD of document was provided in Boyce Hydro 2015.]

McDowell and Associates (McDowell). 2003 (June 6). *Soils Exploration, Edenville Dam and Spillway*.

———. 2005a (October 21). *Soils Exploration, Edenville and Secord Dams.*

———. 2005b (July 18). *Soils Investigation, Edenville Hydroelectric Project, Auxiliary Spillway Construction, Edenville and Meridian Roads.*

———. 2013. Soil Borings for Edenville Temporary Spillway. Cited in Boyce Hydro 2013.

———. 2015. Soils Investigation, Wixom Lake Emergency Spillway, Edenville Hydroelectric Project. Cited in Gomez and Sullivan 2018.

———. 2018. Soils Investigation, Tobacco Auxiliary Spillway, Edenville Hydroelectric Project. Cited in Somat 2020.

Mill Road Engineering. 2009. *Embankment Stability Analysis, Toe Drain Addition, Cross Section Modification and Safety Factor, 5th Revision (February).* Cited in Boyce Hydro 2015.

———. 2010. *Liquefaction Analysis, Edenville Hydroelectric Project.* Cited in Boyce Hydro 2015.

Soil and Materials Engineers, Inc. (SME). 1987 (April 30). *Geotechnical Evaluation Report, Edenville Dam – Station 48+00.*

Somat Engineering Inc. (Somat). 2020. *Geotechnical Data Report, Edenville Dam Repair*.

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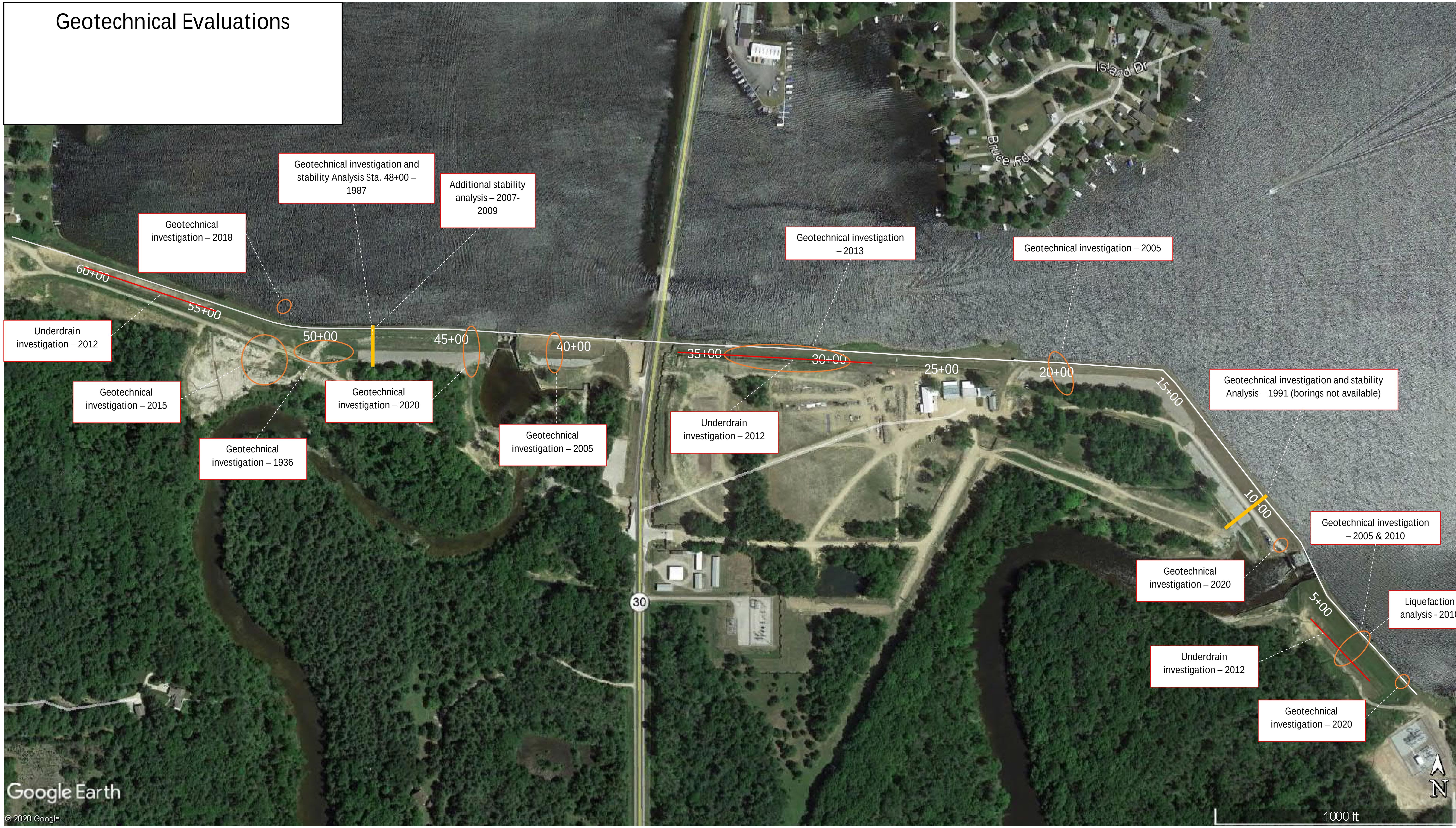

Figure B-2: Geotechnical Evaluations

Figure B-3: Edenville Embankment – Historic Investigations

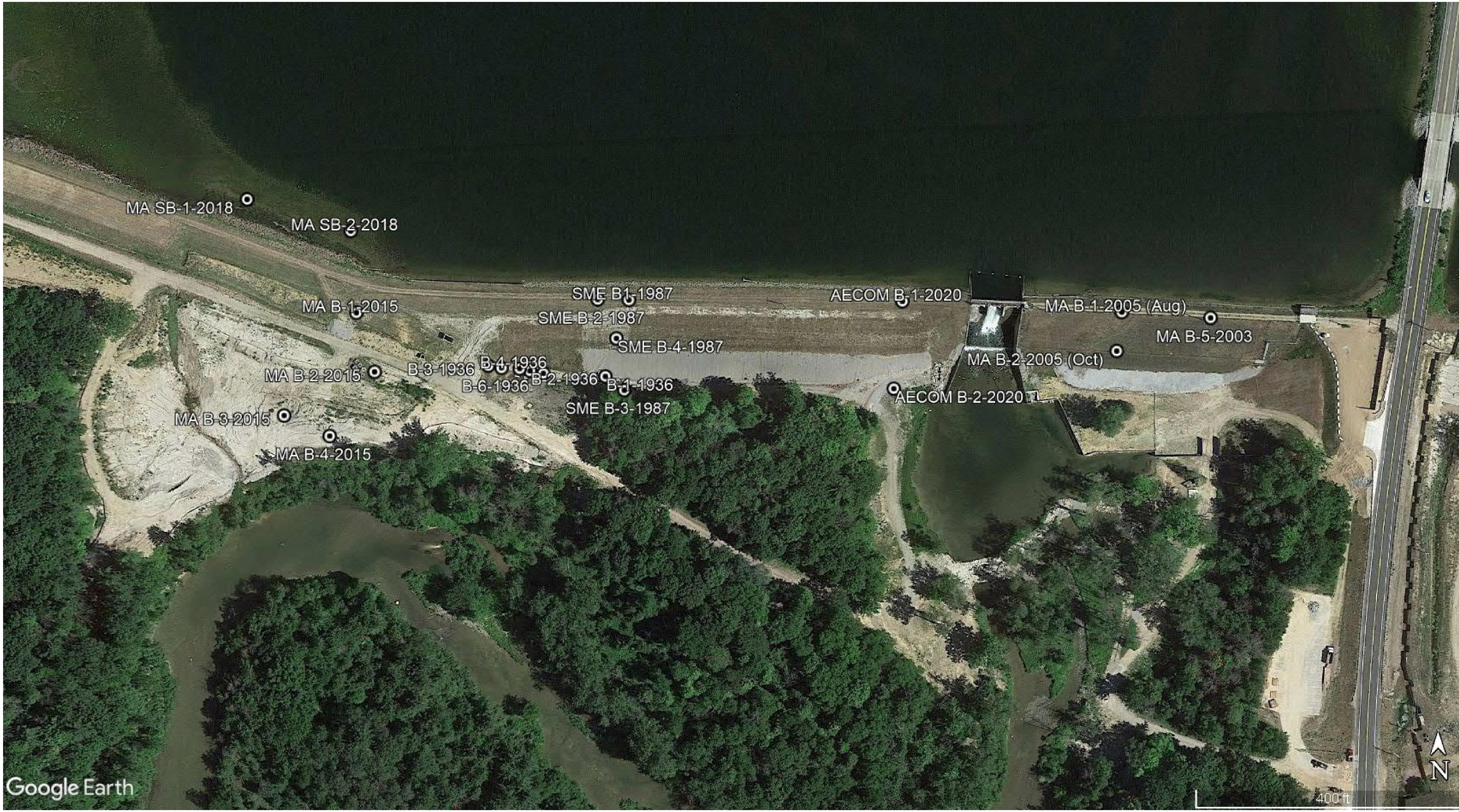

Figure B-4: Tobacco Embankment – Historic Investigations

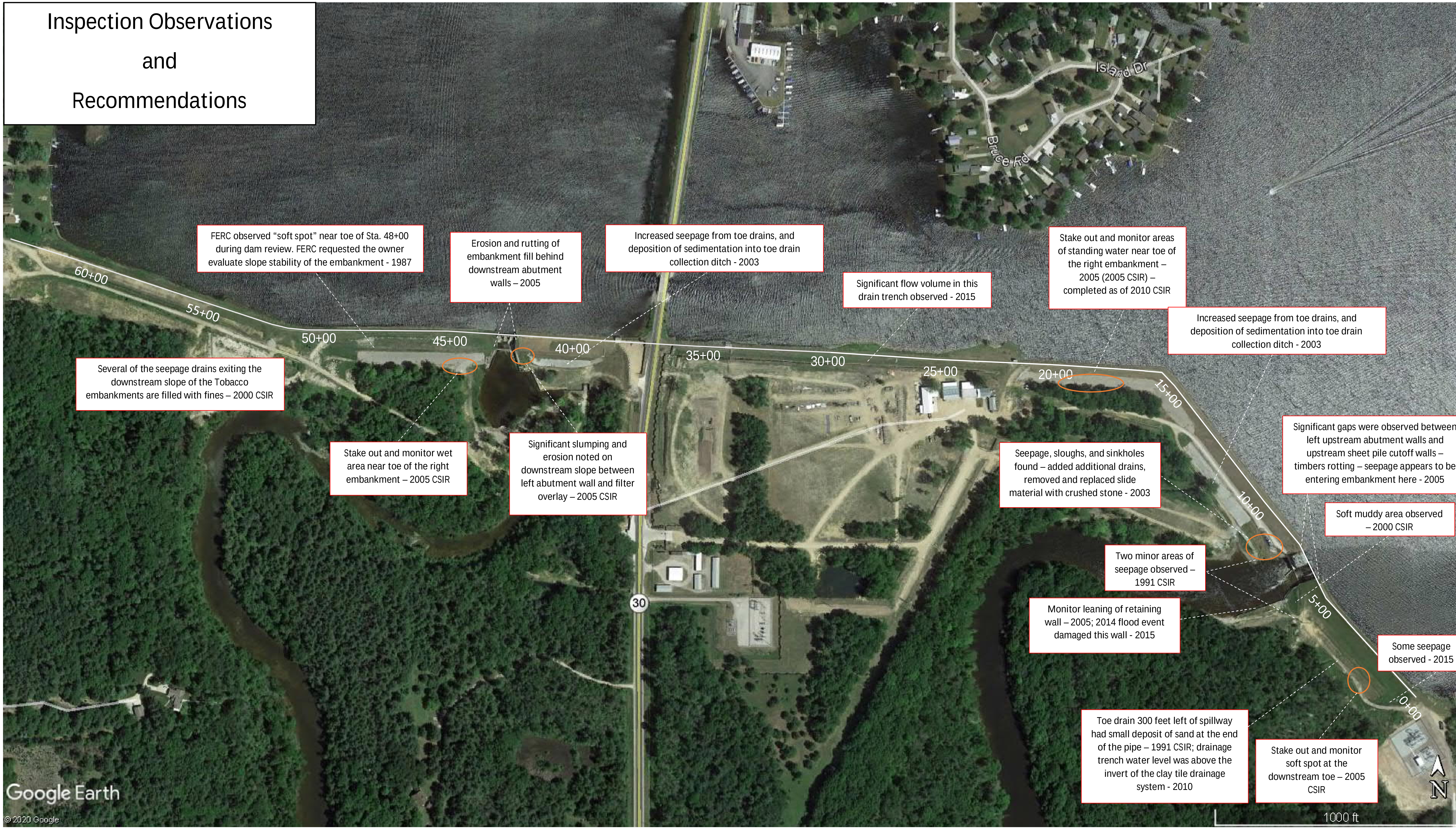

Figure B-5: Inspection Observations and Recommendations

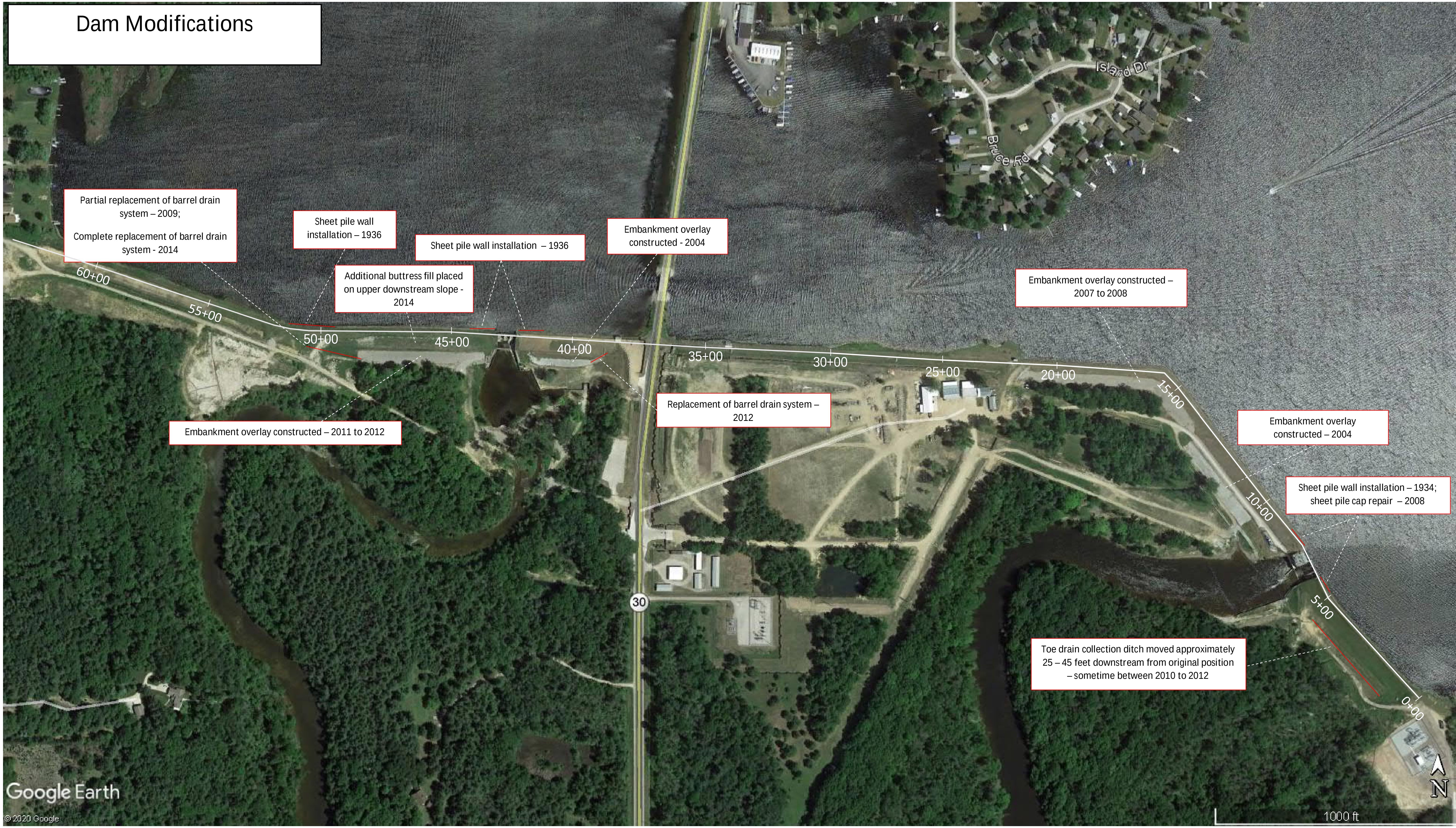

Figure B-6: Dam Modifications

Table B-1: Summary of Evaluations

Date	Project Location	Purpose of Evaluation	Description of Evaluation	Key Findings	Follow-on Actions
1936 (Holland, Ackerman & Holland 1936)	Tobacco Right Embankment	• Investigate the area of seepage at the downstream toe between Station (Sta.) 50+00 and Sta. 48+00 to support the design of a sheet pile cutoff wall.	• The investigation included six borings performed at the downstream toe between Sta. 50+00 and Sta. 48+00. • No boring logs were discovered during the course of the Independent Forensic Team's (IFT's) investigation.	• The investigation identified a deep zone of native soil overlaying Glacial Till between Sta. 50+00 and Sta. 48+00. The native material was relatively deeper than surrounding areas.	• A sheet pile cutoff wall was constructed between Sta. 50+00 and Sta. 48+00. The cutoff wall was constructed through the embankment and driven into Glacial Till.
1987 (SME 1987)	Tobacco Right Embankment Sta. 48+00	• Investigate the area of observed seepage and soft ground on the downstream slope of the embankment at Sta. 48+00 (stated in Geotechnical Investigation Report). • Install groundwater monitoring wells. • Perform a slope stability analysis at Sta. 48+00.	• The investigation included four borings and one test pit in the vicinity of Sta. 48+00. Two borings were taken from the crest, one boring was taken from mid-downstream slope, and one boring was taken at the downstream toe. • Three groundwater observation wells were installed. One well was installed near the crest, one at the mid-downstream slope, and one near the downstream toe. • Standard Penetration Testing (SPT) field tests and pocket penetrometer tests were performed. Pocket penetrometers were performed in the Glacial Till. • Disturbed and undisturbed samples were obtained and laboratory testing was completed, which included index testing and consolidated isotropic undrained (CIU) triaxial tests. • Slope stability analyses were performed using material characterization from shear tests results and a phreatic surface based on groundwater observation wells. • Two stability loading conditions were analyzed, including steady-state at Normal Water Level (NWL) El. 675.8 and High-Water Level (HWL) El. 677.3, using effective stress methods.	• The test pit, near the toe of the slope, encountered groundwater at 1 – 2 feet below-grade. • The borings taken from the crest (Borings 1 and 2) found 38-40 feet of loose to medium dense fine to medium sand fill and silty sand fill with clay lumps [SPT blow counts of N = 5-14 blows per foot (bpf), fines content (FC) = 5.5-54%], overlying ~4-6 feet of medium dense to dense, possibly native, silty fine to medium sand with trace gravel and organics [N = 16-44 bpf, FC = 5-36%]. Glacial Till was encountered at ~44 feet below ground surface (bgs). • The material properties used in the slope stability analysis were based on triaxial data from reconstituted samples "but were conservatively adjusted to account for potential variations in soil strength": o Fine to medium sand fill 35-deg o Mixture of silt and sand fill 32-deg o Native silty sand 34-deg o Glacial Till C = 4.5-ksf • The slope stability analysis for the NWL loading condition resulted in a Factor of Safety (FS) = 1.4. • The slope stability analysis for the HWL loading condition resulted in a FS = 1.3.	• The geotechnical report provided recommendations for remedial action, including construction of a downstream toe filter drain. • In 2011 – 2012, the toe filter drain was completed for the Tobacco right embankment from ~ Sta. 44+00 to Sta. 49+00. • The toe filter drain installation consisted of installing a downstream two-stage filter overlay, re-sleeving the existing underdrains, and adding a perforated pipe toe drain.
1991 (Blystra 1991)	Edenville Right Embankment	• Install two groundwater monitoring wells in the Edenville right embankment to measure the phreatic surface. This work was reported in the 1991 Consultant's Safety Inspection Report (CSIR). • Perform a slope stability analysis at Sta. 10+00.	• One groundwater monitoring well was installed at the mid-downstream slope bench and one at the downstream toe. • As part of the CSIR, soil samples from the embankment and from the embankment foundation were tested in the laboratory. A separate laboratory report was not prepared. • No boring logs were found during the course of the IFT's investigation. The 1991 CSIR describes the soil samples collected: one sample was from the Tobacco embankment (downstream slope) and one sample was from the Edenville embankment (downstream slope). Samples were classified as SP with little to no fines. • Slope stability analyses were performed using strength parameters from direct shear results and phreatic surfaces from groundwater observation. • Five stability loading conditions were analyzed, including steady-state at Normal Water Level (NWL) El. 675.8 and High-Water Level (HWL) at embankment crest elevation using effective stress methods; pseudo-static loading; and rapid drawdown from NWL and from HWL.	• Direct shear tests resulted in effective stress friction angles of 34.6 and 34.3 degrees. The reconstituted dry unit weight of the sand was 115 pcf. • The strength parameters used in the slope stability analysis were taken from the direct shear test results. • The slope stability analysis for the NWL loading condition for the downstream slope resulted in a FS = 2.12. • The slope stability analysis for the HWL loading condition for the downstream slope resulted in a FS = 1.80. • The slope stability analysis for the seismic loading condition for the downstream slope resulted in a FS = 1.76. • The slope stability analysis for the rapid drawdown loading condition from NWL for the upstream slope resulted in a FS = 0.95 for a 2.5:1 slope. • The slope stability analysis for the rapid drawdown loading condition from HWL for the upstream slope resulted in a FS = 0.93 for a 2.5:1 slope. • The slope stability analysis for the rapid drawdown loading condition from NWL for the upstream slope resulted in a FS = 1.12 for a 3:1 slope. • The slope stability analysis for the rapid drawdown loading condition from HWL for the upstream slope resulted in a FS = 1.12 for a 3:1 slope.	• No follow-on actions were identified.
March 2003 (McDowell 2003)	Edenville Right and Tobacco Left Embankments	• Investigate the Edenville right embankment, to the right of the powerhouse, and the Tobacco left embankment. • Observations that initiated this investigation: o Significant increase in seepage from the drains of the Edenville right embankment and the Tobacco left embankment adjacent to the spillway/powerhouse structure	• Investigation included five borings and the installation of three groundwater monitoring wells. • Borings 1 and 2 were performed at the crest of the Edenville right embankment, and Borings 3 and 4 were taken at the roadway midway on the downstream slope. Boring 5 was taken on the crest of the Tobacco left embankment. • Borings 1, 2, and 5 were advanced to a depth of about 40 feet bgs. Borings 3 and 4 were advanced to a depth of 25 feet bgs.	• The entire depth of Boring 1 (40.5 ft) was reported to consist of fine sand fill [N = 7-15 bpf, FC = 2.3 – 2.5%]. Groundwater was encountered at 18 ft bgs during drilling. • Boring 2 had 38 ft of fine sand fill [N = 7-13 bpf, FC = 1.7 – 2.7%] overlying 2.5 feet of possibly native gray fine sand [N = 22 bpf]. Groundwater was encountered at 17.5 ft bgs during drilling. • Boring 3 had 13 ft of fine silty sand fill [N =4-6 bpf; FC=22.2] overlying 8 ft of possibly native gray sand and sand with silt and clay [N = 9-26 bpf; FC = 20.5-20.7%] overlaying 2 ft of cobbles	• A toe filter drain was completed at the Edenville right embankment from ~ Sta. 10+00 to Sta. 13+50, and at the Tobacco left embankment from ~ Sta. 39+00 to Sta. 42+00 in 2004. • A toe filter drain was completed at the Edenville right embankment from ~ Sta. 13+50 to Sta. 21+00 from 2007 to 2008.

Date	Project Location	Purpose of Evaluation	Description of Evaluation	Key Findings	Follow-on Actions
		○ Some seepage observed exiting the downstream face near the spillway/powerhouse structure ○ Some accumulation of fines from the drains ○ Two small sink holes on the downstream face ○ A surface slough adjacent to the retaining wall downstream of the Edenville powerhouse	• The monitoring wells were installed adjacent to the powerhouse on the crest and set approximately 50 feet bgs. • SPT field tests were performed. Disturbed samples were obtained, and laboratory testing included grain size analyses. • Dye testing was performed in the monitoring wells to attempt to trace seepage through the embankment. • Soil samples were taken from the toe drain collection ditches where soil was observed to accumulate. Grain size analyses were performed on these samples.	with sand. Glacial Till was encountered at 23 ft bgs. Groundwater was encountered at 7 ft bgs during drilling. • Boring 4 had 13 ft of fine to medium sand [N =7-15 bpf; FC = 11.8%] overlying 5 ft of possibly native gray sand [N = 11;FC = 21.8%] overlaying 2 ft of gray clay. Glacial Till was encountered at 20 ft bgs. Groundwater was encountered at 3.5 ft bgs during drilling. • The entire depth of Boring 5 (40.5 ft) was reported to consist of fine sand fill [N = 5-10 bpf, FC = 1.5-7.0%]. Groundwater was encountered at 17.5 ft bgs during drilling. • The soil in the toe drain collection ditches was found to consist of poorly graded fine sand with little silt [FC = 1.3-13.1%]. • No dye was observed from the downstream side of the embankment. Twenty days later, dye was observed to be present in the wells apparently not having been flushed out from groundwater flow.	• The toe filter drain installations consisted of installing a downstream two-stage filter overlay, re-sleeving the existing underdrains, and adding a perforated pipe toe drain.
July 2005 (McDowell 2005b)	Edenville Left and Right Embankments	• Investigate possible locations for construction of auxiliary spillway (interpreted from geotechnical investigation report subject).	• The investigation included four borings at the Edenville left and right embankments: Borings 1 and 2 were at the Edenville left embankment, and Borings 3 and 4 were at the Edenville right embankment. • Borings 1 and 4 were taken at the crests of the embankments, while Borings 2 and 3 were taken at the downstream toes of the embankments. • Field tests included SPT and pocket penetrometers. • Undisturbed and disturbed samples were obtained and laboratory tests included grain size analysis, moisture content and unit weight, and unconfined compression tests. • Total unit weights in embankment fill were based on "Undisturbed Liner" samples.	• Boring 1, from the crest of the left embankment, encountered 14 feet of silty fine sand and silt and sand [N = 2-7 bpf, FC = 12-45%] overlying 17 feet of silty clay with wet sand layers [N = 2-10 bpf, unconfined compression tests UC = 605 – 1760 psf], overlying possible native silty fine sand and gravelly sand with silt [N = 19 – 22 bpf, FC = 18-20%]. Glacial Till was encountered at 38.5 feet bgs. Groundwater was encountered at 10 feet at time of drilling and 39 feet after completion.	• No follow-on actions were identified.
August – October 2005 (McDowell 2005a)	Tobacco Left Embankment	• Investigate Tobacco left embankment and install groundwater monitoring wells (interpreted from geotechnical investigation report subject).	• The investigation included two borings at the Tobacco left embankment. • Boring 1 was performed on the crest, and Boring 2 was performed midway on the downstream slope. • Two groundwater monitoring wells were installed at each boring location. • Field tests included SPT and pocket penetrometers. • Undisturbed and disturbed samples were obtained and laboratory tests included: moisture content and unit weight, and unconfined compression tests. • Total unit weights in embankment fill were based on "Undisturbed Liner" samples.	• Boring 1 encountered 47.5 feet of fine to medium sand with traces of silt and gravel [N = 5-22 bpf] overlying 3.5 feet of possibly native fine sand with trace silt and occasional stones and cobbles [N = 18-28 bpf]. Glacial Till was encountered at 51 feet bgs. Groundwater depth encountered was 11 feet at time of drilling and 12 feet after completion.	• No follow-on actions were identified.
August 2007 (Initial Analysis) February 2009 (5th Revision) (Mill Road Engineering 2009)	Tobacco Right Embankment	• Perform slope stability analysis at Sta. 48+00, including toe filter drain design. • Provide basis for design of the toe filter drain.	• A slope stability analysis was completed for the embankment section at Sta. 48+00, including the design layout of the toe filter drain. • Section geometry was based on the modification design and stratigraphy from the 1987 geotechnical evaluation. • Material properties were based on the results of the 1987 geotechnical evaluation. • Pore pressure conditions were based on the Sta. 48+00 groundwater monitoring wells. • Four stability loading conditions were analyzed, including steady-state at Normal Water Level (NWL) El. 675.8 and High-Water Level (HWL) El. 683.0 (embankment crest) using effective stress methods; and rapid drawdown from HWL and from NWL.	• The material properties used in the slope stability analysis were taken from the 1987 stability analysis with the addition of the toe filter drain: ○ Fine to medium sand fill 35-deg ○ Mixture of silt and sand fill 32-deg ○ Native silty sand 34-deg ○ Glacial Till C = 4.5-ksf ○ Overlay 37.5-deg • The slope stability analysis for the NWL loading condition resulted in a Factor of Safety (FS) = 1.53. • The slope stability analysis for the HWL loading condition resulted in a Factor of Safety (FS) = 1.40. • The slope stability analysis for the rapid drawdown loading condition resulted in a Factor of Safety (FS) = 1.29 from HWL and 1.3 from NWL.	• The toe drain filter overlay project at the Tobacco right embankment, was completed in the period from 2011 to 2012. • The toe filter drain installation consisted of installing a downstream two-stage filter overlay, re-sleeving the existing underdrains, and adding a perforated pipe toe drain.

Date	Project Location	Purpose of Evaluation	Description of Evaluation	Key Findings	Follow-on Actions
December 2010 (Mill Road Engineering 2010)	Edenville Left Embankment	• Perform a seismic liquefaction triggering assessment of the embankment and foundation materials. • This analysis was completed in response to a FERC information request.	• The evaluation included two borings at the left Edenville embankment. • Boring 1 was performed at the crest and Boring 2 was performed at the toe of the embankment. • Undisturbed and disturbed samples were obtained and laboratory tests included: grain size analysis, moisture content and unit weight, and unconfined compression tests. • Total unit weights in embankment fill were based on “Undisturbed Liner” samples. • An SPT-based seismic liquefaction triggering assessment was completed. The 2005 borings were also evaluated for triggering.	• Boring 2 consisted of 13 feet of silt with sand and sandy silt [N = 4-7 bpf, FC = 22.8-95.8%] overlying 21 feet of sandy clay and clayey sand [N = 2-10 bpf, F C= 11.1-81.5%, UC = 750-3000 psf] overlying 2.5 feet of sand [N = 16-23 bpf, FC = 35%]. Glacial Till was encountered at 37.5 feet bgs. Groundwater was encountered at 10.5 feet bgs during drilling. • The results of the study indicated that the dam is not subject to earthquake-induced liquefaction due to the combination of soil characteristics and the very low energy of an earthquake event for this location.	• No follow-on actions were identified.
2012 (Boyce Hydro 2014)	Edenville Left Embankment, Portions of Edenville Right Embankment, and Portions of Tobacco Right Embankment	• Investigate and clean out embankment underdrains.	• Underdrain system was documented at each embankment where drains were accessible. Drains were not accessible in areas where the embankment was previously modified with toe filter drains. • Drain pipes were flushed out with a low-pressure cleaning hose and nozzle. • Hose was pushed up the pipe as far as it would go, and distance was recorded. • CCTV camera inspections were completed where possible. • Locations of drains were recorded and plotted on drawings.	• The underdrains were found to generally extend to the centerline of the embankment, except for some drains that were blocked. This distance was about 100 feet for the Edenville left embankment. • CCTV results found that some tiles were broken and collapsed.	• No follow-on actions were identified.
October 2013 (McDowell 2013)	Edenville Right Embankment	• Investigate proposed auxiliary spillway location to support design (Probable Maximum Flood Alterations Design Plan).	• The investigation included six borings performed at the Edenville right embankment between Sta. 28+00 and Sta. 34+00. • Borings 2, 4, 5, and 6 were completed on the crest. • Borings 1 and 3 were completed at the downstream toe. • Field tests included SPT and pocket penetrometers. • Undisturbed and disturbed samples were obtained, and laboratory tests included grain size analysis, moisture contents and unit weights, and unconfined compression tests. • Total unit weights in embankment fill were based on “Undisturbed Liner” samples.	• Borings 2, 4, 5, 6 consisted of ~ 10 - 20 feet of medium to fine sand [N = 5-9 bpf, FC = 2-17%] overlying ~5-15 feet of possibly native fine sand [N = 16-31 bpf, FC = 1-7%]. Glacial Till was encountered at 25-27 feet bgs. Groundwater was encountered at 9-11 feet bgs during drilling.	• Auxiliary spillway design progressed, but the auxiliary spillway was never constructed.
January 2015 (McDowell 2015)	Tobacco Right Embankment Downstream Area	• Investigate proposed auxiliary spillway location to support design.	• Investigation included four borings performed downstream of the Tobacco right embankment near Sta. 55+00.	• Borings were drilled up to 59 feet below ground surface • All four of the Borings encountered between 10 to 30 feet of stiff to extremely stiff sandy clay Glacial Till. • The Glacial Till was underlain by a very compact sand stratum that was not fully penetrated by the Borings. • Only Boring 3 encountered an extremely compact sandy silt below the sand stratum at 48 feet bgs.	• No follow-on actions were identified.
February 2018 (McDowell 2018)	Tobacco Right Embankment	• Investigate proposed auxiliary spillway location to support design.	• The investigation included two borings performed at the Tobacco right embankment around Sta. 53+00. • The borings were completed upstream of the embankment crest on the frozen reservoir.	• Boring depth ranged from 11 to 16 ft. • The borings found 7 to 7.5 feet of possibly native fine to medium sand with silt overlaying sandy silt and Glacial Till.	• No follow-on actions were identified.
October 2020 (Somat 2020)	Edenville Left (abutment) and Right Embankments and Tobacco Right Embankment	• Investigate the dam after the failure for development of emergency modifications.	• The investigation included four borings performed at Edenville Dam post-failure. • Boring 4 was completed at the Edenville left embankment crest near the abutment contact at ~Sta. 1+00. • Boring 3A was completed at the mid-downstream bench of the Edenville right embankment near the powerhouse at ~Sta. 8+50. • Boring 2 was completed at the downstream toe of the Tobacco right embankment at ~Sta. 43+50. • Boring 1 was completed at the crest of the Tobacco right embankment at ~Sta. 43+50. • Laboratory tests included: grain size analysis, moisture contents, Atterberg limits, direct shear tests, unconfined	• Boring 4 encountered 16 feet of highly variable fill, including silty sand, clayey sand, silt, and lean clay [N = 3-6 bpf; FC = 13-99%] overlying 12.5 feet of possibly native stiff lean to fat clay [N = 8-13 bpf ; PI = 26-35; UC =1540-5020 psf] overlying 5 feet of possibly native fine to medium poorly graded sand [N = 39] overlying 2.5 feet of possibly native stiff lean clay [N=14; PI = 7; UC = 3680 psf]. Glacial Till was encountered 36 feet bgs. Groundwater depth encountered at 29.5 feet bgs during drilling. • A suite of three direct shear tests were completed on a composite sample from split spoon samples SS3-SS9 from Boring 1, which ranged from 7.5 to 22.5 feet bgs (clean sand fill). The results of the direct shear test showed contractive behavior with an ultimate shear strength of c = 383 psf and ϕ = 25.9 deg at dry densities ranging from 94.5 to 97.1 pcf.	• Not Applicable.

Date	Project Location	Purpose of Evaluation	Description of Evaluation	Key Findings	Follow-on Actions
			compression tests, standard proctor tests, and constant head permeability tests.	• A suite of three direct shear tests were also completed on a composite sample from split spoon samples SS10-SS15 from Boring 1, which ranged from 25 to 50 feet bgs (clean sand fill and possibly native sand). The results of the direct shear test showed contractive behavior with an ultimate shear strength of c = 402 psf and ϕ = 25.1 deg at dry densities ranging from 91.6 to 93.4 pcf. • Two constant head permeability tests were completed on the composite samples from Boring 1. The results ranged from 1.44E-02 cm/s to 2.25E-02 cm/s at dry densities of 90.1 and 88.0 pcf, respectively.	

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Attachment B-1: Underdrain Investigation

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EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF POWERHOUSE
DATE SURVEYED: SEPTEMBER 7-12, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY-FT.	LENGTH BY WATER JET,FT.	OBSTRUCTION OF CAMERA
0	4" BLACK PVC	37	106	
1	4" BLACK PVC	25	100	CAMERA ACCESS TO 50' ONLY-BAD SHIFT
2	4" BLACK PVC	32	106	LARGE ROCK IN LINE AT 49'
3	4" BLACK PVC	31	95	CAMERA ACCESS TO 80' ONLY
4	4" BLACK PVC	32		BROKEN AT 37'
5	4" BLACK PVC	37	100	CAMERA ACCESS TO 5' ONLY-LARGE ROCK
6	4" BLACK PVC	30		CAMERA ACCESS TO 30'-ROCK IN LINE
7	4" BLACK PVC	31	83	
8	4" BLACK PVC	31	100	CAMERA ACCESS TO 75'-ROCK IN LINE
9	4" BLACK PVC	30	100	SHIFT AND ROCKS AT 85'
10	4" BLACK PVC	36	80	
11	4" BLACK PVC	25	95	BAD SHIFT AT 54'
12	4" BLACK PVC	20	100	CAMERA ACCESS TO 35'-SHIFT
13	4" BLACK PVC	25	100	BAD SHIFT AT 61'
14	4" BLACK PVC	25	95	CAMERA ACCESS ONLY TO 68'
15	4" BLACK PVC	27	45	BROKEN
16	4" BLACK PVC		90	BAD SHIFT AT 12'
17	4" BLACK PVC	31	90-95	BROKEN AT 70'
18	4" BLACK PVC	31	95	CAMERA ACCESS TO 88'
19	4" BLACK PVC	29	95	CAMERA ACCESS TO 70'
20	4" BLACK PVC	29	80	SHIFT AT 34'
21	4" BLACK PVC	27	67.5	BROKEN TILE AT 67.5'
22	4" BLACK PVC	26	47	BROKEN AT 47'
23	4" BLACK PVC	26	70.5	
24	4" BLACK PVC	30	38	BIG ROCK AT 38' NO CAMERA OR JET PAST
25	4" BLACK PVC	29	95	SHIFT AT 50'
26	4" BLACK PVC	35	99'3"	

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF M-30
DATE SURVEYED: SEPTEMBER 20-25, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY-FT.	LENGTH BY WATER JET,FT.	OBSTRUCTION OF CAMERA
1	4" CLAY TILE		33	
2	4" PVC	9'6"	41	
3	4" PVC	8'6"	36	
4	4" PVC	11	36	
5	4" PVC	12	33	
6	4" PVC	10	30	
7	4" PVC	8	31	
8	4" PVC	10	23	
9	4" PVC	3	4'5"	LINE IS BROKEN 4'5" FROM DITCH
10	4" CLAY TILE		15'2	
11	4" CLAY TILE			BROKEN TILE AT 6.5'
12	4" CLAY TILE		28	
13	4" CLAY TILE			CAMERA TO 27'10" ONLY-LARGE ROCK
14	4" PVC	7	34	PINCHED AT DITCHLINE
15	4" PVC	7	34	
16	4" CLAY TILE		34.5	
17	4" PVC	7	37'8"	
18	4" PVC	10	26	
19	4" CLAY TILE		25'7"	
20	4" PVC	7	37	
21	4" PVC	11	37'8"	
22	4" PVC	9	37'7"	
23	4" PVC	8	33	
24	4" PVC	9	40	
25	4" PVC	15	35.5	
26	4" PVC	9	37.5	
27	4" PVC	12	38	
28	4" PVC	13	38	
29	4" PVC	15	38	
30	4" PVC			LINE IS CAPPED AT 3'
31	4" PVC	10	35	
32	4" PVC	5	32.9	
33	4" PVC	17	38	
34	4" PVC	16	39'9"	
35	4" PVC	7	39'3"	
36	4" PVC	12	36'7"	
37	4" PVC	7	41'4"	
38	4" PVC	6	41	
39	4" PVC	7	18	CAMERA TO 18'-APPEARS BROKEN @ 18'
40	4" PVC	4	37.5	
41	4" PVC	18	36	
42	4" PVC	4	34'8"	
43	4" PVC	5	37	

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF M-30 (CONTINUED)
DATE SURVEYED: SEPTEMBER 20-25, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY-FT.	LENGTH BY WATER JET,FT.	OBSTRUCTION OF CAMERA
44	4" PVC		9.5	
45	4" PVC	13	37'8"	
46	4" PVC	6	34'9"	
47	4" PVC	9	35'2"	
48	4" PVC	3	25.5	HOLE IN BOTTOM OF LINE AT 20'

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: WEST SIDE OF M-30 – TOBACCO SIDE
DATE SURVEYED: SEPTEMBER 26-27, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY-FT.	LENGTH BY WATER JET,FT.	OBSTRUCTION OF CAMERA
1	4" PVC	2	33'2"	
2	4" PVC	10	33'3"	
3	4" PVC	10	33	
4	4" PVC		30	CAMERA TO 1.5' ONLY- BAD SHIFT
5	4" PVC	3	32'8"	
6	4" PVC	2	32	
7	4" PVC	7	31	CAMERA TO 7' ONLY- SHIFT
8	4" PVC	8	32	
9	4" PVC	7	32	
10	4" PVC	8	34'3"	
11	4" PVC	11	31.5	
12	4" PVC	7	28'7"	
13	4" PVC	4	29	
14	4" CLAY TILE		25.5	
15	4" CLAY TILE		24'3"	SEPERATION 1.5' INTO LINE
16	4" PVC	3	29	
17	4" CLAY TILE		26	
18	4" PVC	5	30'4"	
19	4" PVC	7	31'1"	
20	4" CLAY TILE			LINE IS BROKEN 2' INTO LINE
21	4" PVC	5	32'7"	
22	4" CLAY TILE			LINE IS BROKEN 2' INTO LINE
23	4" PVC	5	33.5	
24	4" CLAY TILE		33	
25	4" PVC	3	31	
26	4" PVC	5	32'2"	
27	4" PVC	6	30	SHIFT AT 6'
28	4" PVC	7	29.5	
29	4" PVC	6	27	
30	4" PVC	4	26	
31	4" PVC	3	26.5	
32	4" PVC	5	25	
33	4" PVC	5	28	
34	4" PVC	7	28.5	
35	4" PVC	7	27'7"	
36	4" PVC	7	27'4"	
37	4" PVC	7	27'7"	
38	4" PVC	7	26	

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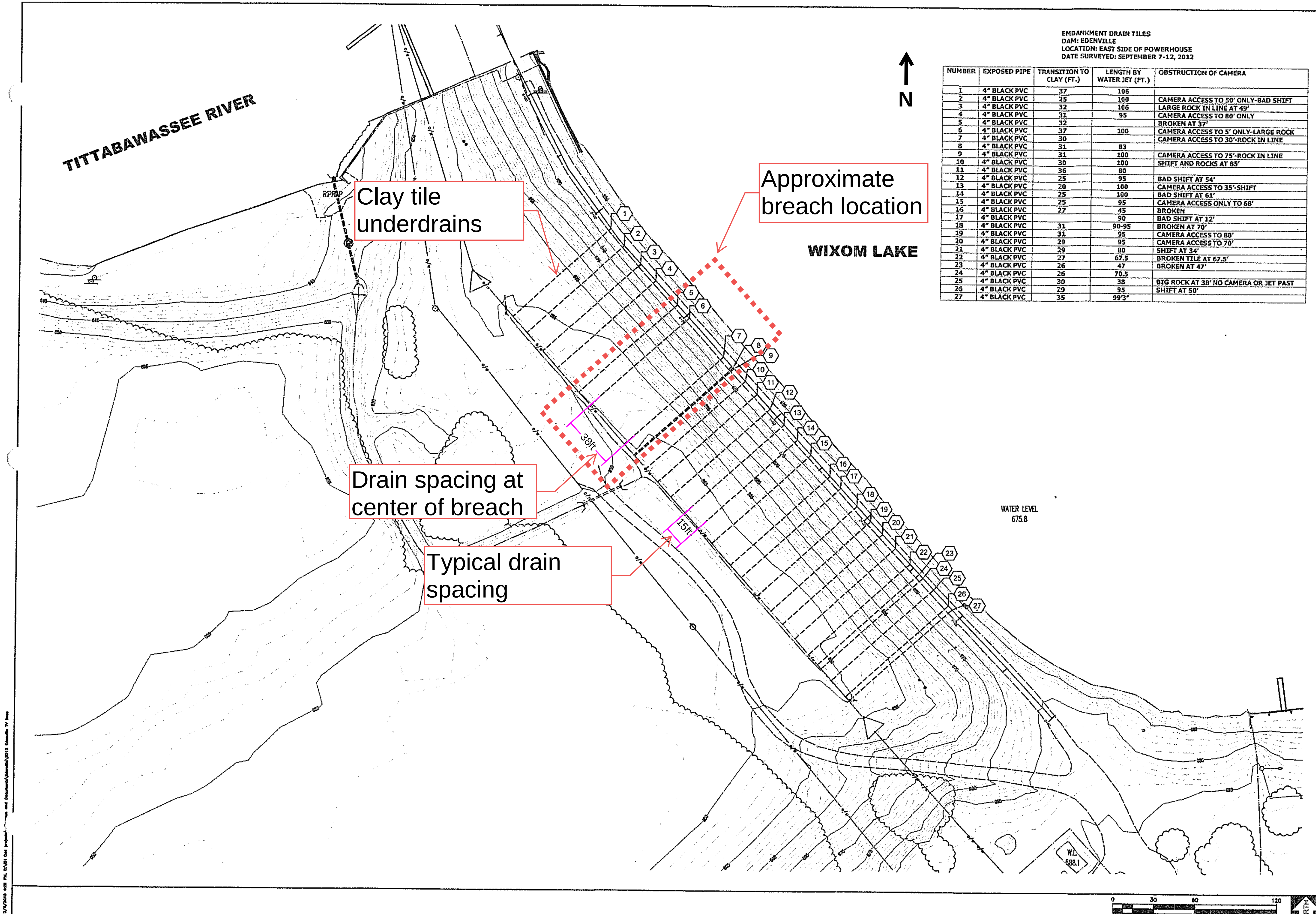

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF POWERHOUSE
DATE SURVEYED: SEPTEMBER 7-12, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY (FT.)	LENGTH BY WATER JET (FT.)	OBSTRUCTION OF CAMERA
1	4" BLACK PVC	37	106	
2	4" BLACK PVC	25	100	CAMERA ACCESS TO 50' ONLY-BAD SHIFT
3	4" BLACK PVC	32	106	LARGE ROCK IN LINE AT 49'
4	4" BLACK PVC	31	95	CAMERA ACCESS TO 80' ONLY
5	4" BLACK PVC	32		BROKEN AT 37'
6	4" BLACK PVC	37	100	CAMERA ACCESS TO 5' ONLY-LARGE ROCK
7	4" BLACK PVC	30		CAMERA ACCESS TO 30'-ROCK IN LINE
8	4" BLACK PVC	31	83	
9	4" BLACK PVC	31	100	CAMERA ACCESS TO 75'-ROCK IN LINE
10	4" BLACK PVC	30	100	SHIFT AND ROCKS AT 85'
11	4" BLACK PVC	36	80	
12	4" BLACK PVC	25	95	BAD SHIFT AT 54'
13	4" BLACK PVC	20	100	CAMERA ACCESS TO 35'-SHIFT
14	4" BLACK PVC	25	100	BAD SHIFT AT 61'
15	4" BLACK PVC	25	95	CAMERA ACCESS ONLY TO 68'
16	4" BLACK PVC	27	45	BROKEN
17	4" BLACK PVC		90	BAD SHIFT AT 12'
18	4" BLACK PVC	31	90-95	BROKEN AT 70'
19	4" BLACK PVC	31	95	CAMERA ACCESS TO 88'
20	4" BLACK PVC	29	95	CAMERA ACCESS TO 70'
21	4" BLACK PVC	29	80	SHIFT AT 34'
22	4" BLACK PVC	27	67.5	BROKEN TILE AT 67.5'
23	4" BLACK PVC	26	47	BROKEN AT 47'
24	4" BLACK PVC	26	70.5	
25	4" BLACK PVC	30	38	BIG ROCK AT 38' NO CAMERA OR JET PAST
26	4" BLACK PVC	29	95	SHIFT AT 50'
27	4" BLACK PVC	35	99'3"	

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DESCRIPTION		
0	09/30/2014	MDF
DRAWING ORIGINATION		

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DRAWING TITLE

FERC PROJECT No. 10808
EDENVILLE DAM

EDENVILLE
EAST OF
TITTABAWASSEE RIVER

DRAWING LINE INVENTORY

FILE	2012 Edenville TV lines
SCALE	AS NOTED
DRAWN	MDF
CHECKED	FOC
APPROVED	
REV. DATE	2/6/2015

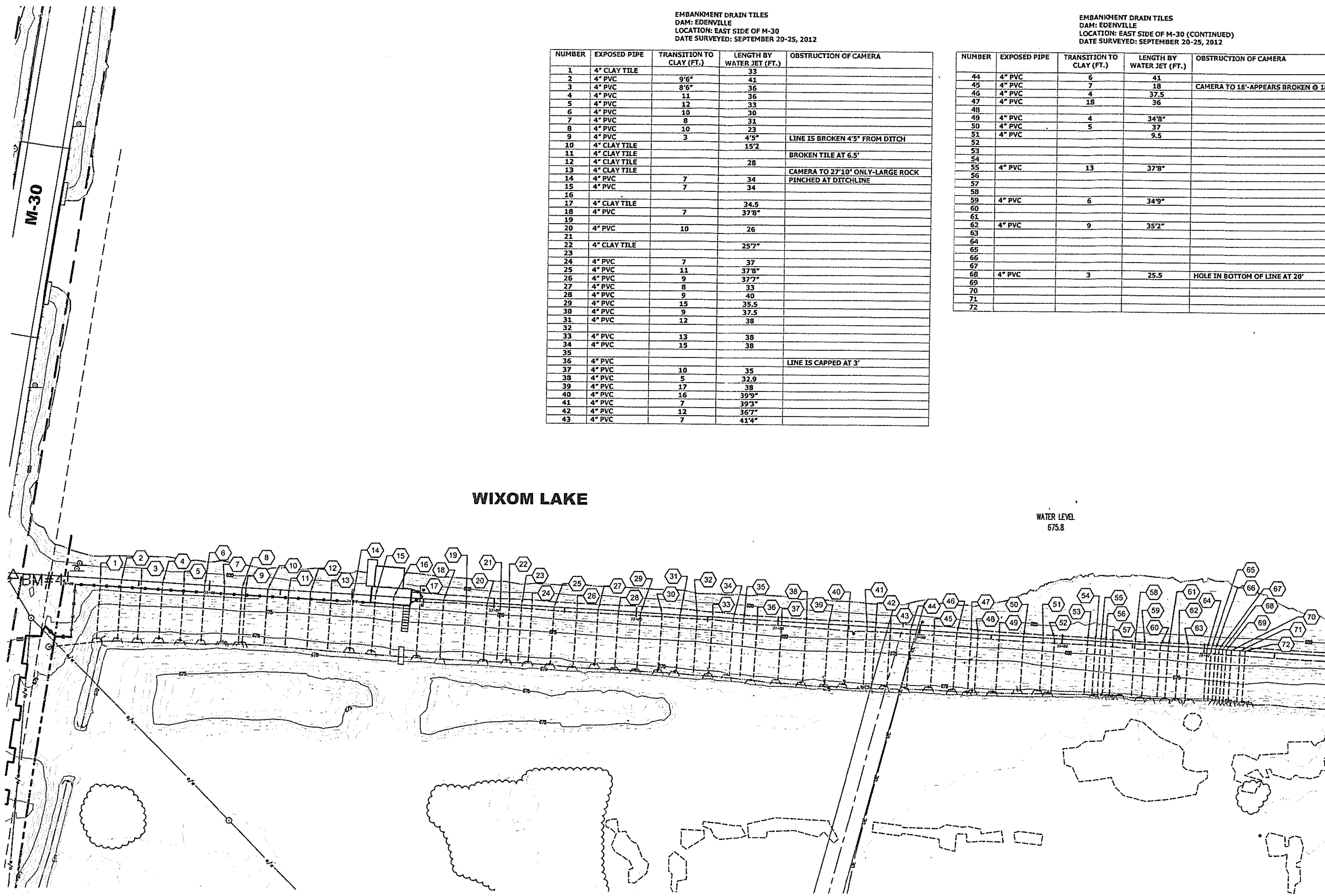
DRAWING NO.
SITE #112, PROJECT # 09
0.3.00

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF M-30
DATE SURVEYED: SEPTEMBER 20-25, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY (FT.)	LENGTH BY WATER JET (FT.)	OBSTRUCTION OF CAMERA
1	4" CLAY TILE		33	
2	4" PVC	9'6"	41	
3	4" PVC	8'6"	36	
4	4" PVC	11	36	
5	4" PVC	12	33	
6	4" PVC	10	30	
7	4" PVC	8	31	
8	4" PVC	10	23	
9	4" PVC	3	4'5"	LINE IS BROKEN 4'5" FROM DITCH
10	4" CLAY TILE		15'2	
11	4" CLAY TILE			BROKEN TILE AT 6.5'
12	4" CLAY TILE		28	
13	4" CLAY TILE			CAMERA TO 27'10" ONLY-LARGE ROCK
14	4" PVC	7	34	PINCHED AT DITCHLINE
15	4" PVC	7	34	
16				
17	4" CLAY TILE		34.5	
18	4" PVC	7	37'8"	
19				
20	4" PVC	10	26	
21				
22	4" CLAY TILE		25'7"	
23				
24	4" PVC	7	37	
25	4" PVC	11	37'8"	
26	4" PVC	9	37'7"	
27	4" PVC	8	33	
28	4" PVC	9	40	
29	4" PVC	15	35.5	
30	4" PVC	9	37.5	
31	4" PVC	12	38	
32				
33	4" PVC	13	38	
34	4" PVC	15	38	
35				
36	4" PVC			LINE IS CAPPED AT 3'
37	4" PVC	10	35	
38	4" PVC	5	32.9	
39	4" PVC	17	38	
40	4" PVC	16	39'9"	
41	4" PVC	7	39'3"	
42	4" PVC	12	36'7"	
43	4" PVC	7	41'4"	

EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: EAST SIDE OF M-30 (CONTINUED)
DATE SURVEYED: SEPTEMBER 20-25, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY (FT.)	LENGTH BY WATER JET (FT.)	OBSTRUCTION OF CAMERA
44	4" PVC	6	41	
45	4" PVC	7	18	CAMERA TO 18'-APPEARS BROKEN @ 18'
46	4" PVC	4	37.5	
47	4" PVC	18	36	
48				
49	4" PVC	4	34'8"	
50	4" PVC	5	37	
51	4" PVC		9.5	
52				
53				
54				
55	4" PVC	13	37'8"	
56				
57				
58				
59	4" PVC	6	34'9"	
60				
61				
62	4" PVC	9	35'2"	
63				
64				
65				
66				
67				
68	4" PVC	3	25.5	HOLE IN BOTTOM OF LINE AT 20'
69				
70				
71				
72				



0 30 60 120
SCALE: 1" = 30'
NORTH

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DRAWING TITLE

FERC PROJECT
No. 10808
EDENVILLE DAM

EDENVILLE
EAST OF M-30

DRAIN LINE INVENTORY

FILE	2012 Edenville TV lines
SCALE	AS NOTED
DRAWN	MDF
CHECKED	FOC
APPROVED	
REV. DATE	2/6/2015

DRAWING NO.
SITE #112, PROJECT # 09
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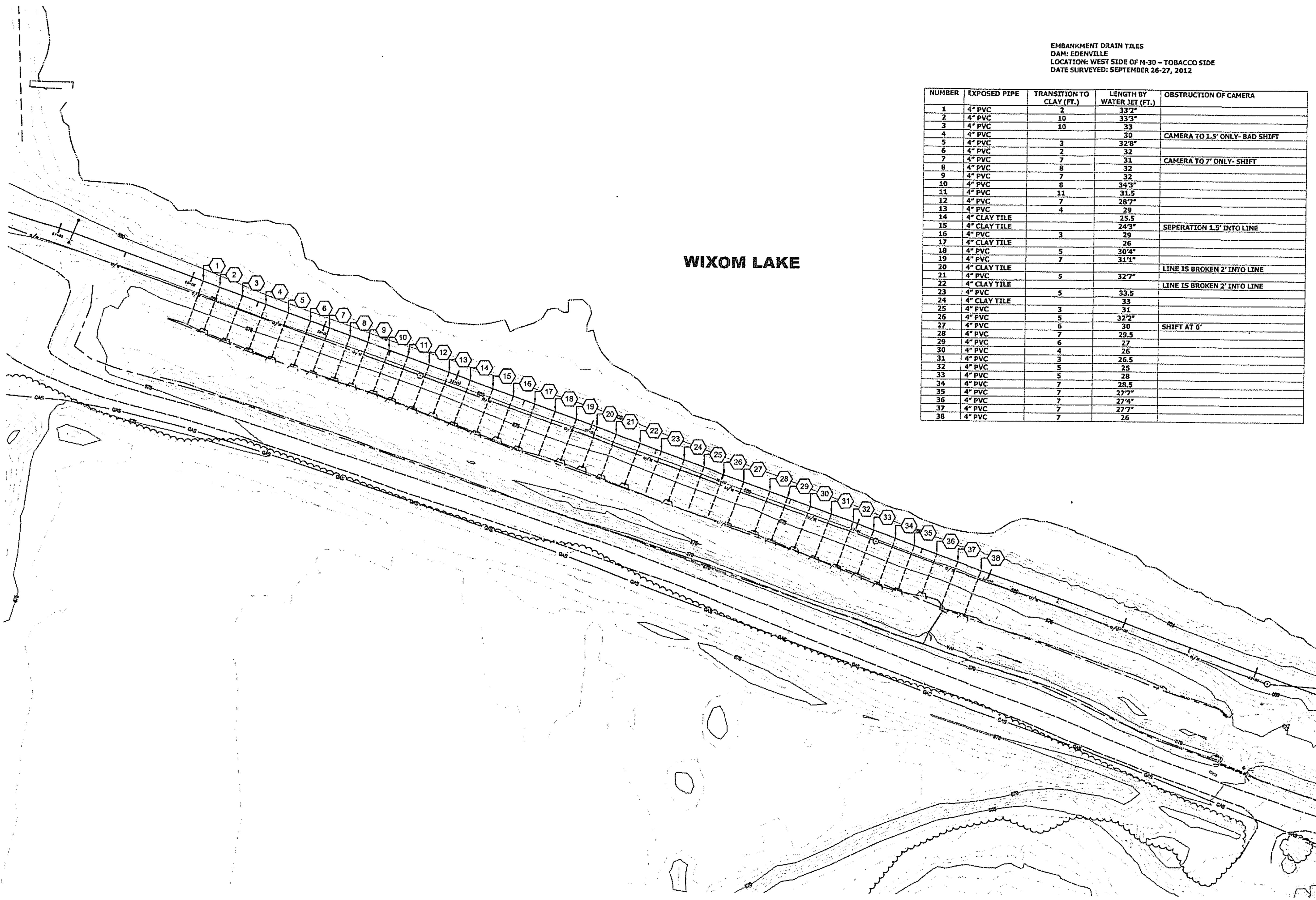
EMBANKMENT DRAIN TILES
DAM: EDENVILLE
LOCATION: WEST SIDE OF M-30 – TOBACCO SIDE
DATE SURVEYED: SEPTEMBER 26-27, 2012

NUMBER	EXPOSED PIPE	TRANSITION TO CLAY (FT.)	LENGTH BY WATER JET (FT.)	OBSTRUCTION OF CAMERA
1	4" PVC	2	33'2"	
2	4" PVC	10	33'3"	
3	4" PVC	10	33	
4	4" PVC		30	CAMERA TO 1.5' ONLY- BAD SHIFT
5	4" PVC	3	32'8"	
6	4" PVC	2	32	
7	4" PVC	7	31	CAMERA TO 7' ONLY- SHIFT
8	4" PVC	8	32	
9	4" PVC	7	32	
10	4" PVC	8	34'3"	
11	4" PVC	11	31.5	
12	4" PVC	7	28'7"	
13	4" PVC	4	29	
14	4" CLAY TILE		25.5	
15	4" CLAY TILE		24'3"	SEPERATION 1.5' INTO LINE
16	4" PVC	3	29	
17	4" CLAY TILE		26	
18	4" PVC	5	30'4"	
19	4" PVC	7	31'1"	
20	4" CLAY TILE			LINE IS BROKEN 2' INTO LINE
21	4" PVC	5	32'7"	
22	4" CLAY TILE			LINE IS BROKEN 2' INTO LINE
23	4" PVC	5	33.5	
24	4" CLAY TILE		33	
25	4" PVC	3	31	
26	4" PVC	5	32'2"	
27	4" PVC	6	30	SHIFT AT 6'
28	4" PVC	7	29.5	
29	4" PVC	6	27	
30	4" PVC	4	26	
31	4" PVC	3	26.5	
32	4" PVC	5	25	
33	4" PVC	5	28	
34	4" PVC	7	28.5	
35	4" PVC	7	27'7"	
36	4" PVC	7	27'4"	
37	4" PVC	7	27'7"	
38	4" PVC	7	26	



0 30 60 120
SCALE: 1"= 30'
NORTH

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FERC PROJECT
No. 10808
EDENVILLE DAM

EDENVILLE
WEST OF
TOBACCO RIVER

DRAIN LINE INVENTORY

FILE	2012 Edenville TV lines
SCALE	AS NOTED
DRAWN	MDF
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Appendix C: Historical Hydrologic & Hydraulic Evaluations

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C-1 Historical Hydrology

The hydrologic criteria used to originally design the spillways for the four Boyce Hydro dams are not known. The Independent Forensic Team (IFT) was not able to locate any documentation of design calculations from the 1920s designs of these dams other than several discharge rating curves. The curves for the Edenville and Tobacco spillways only computed the discharge to elevation 677.8 feet or 10 feet above the sill elevation of the concrete crests.

The Hydrometeorological Section of the U.S. Weather Bureau was organized in 1937 to determine limiting amounts of precipitation. Showalter and Solot published one of the early papers on the computation of maximum possible precipitation, later to be known as the probable maximum precipitation (PMP) (Showalter and Solot 1942). The National Weather Service (NWS) published estimates for the PMP for the eastern portion of the U.S. east of the 105th meridian (NWS 1978). In 1982, the NWS published criteria for developing the spatial and temporal precipitation distribution characteristics for the Probable Maximum Storm (PMS), later to be called the probable maximum flood (PMF) [U.S. Army Corps of Engineers {USACE} 1982]. The PMS was intended to be used in conjunction with a rainfall/runoff simulation model to calculate the PMF.

The USACE Hydrologic Engineering Center (HEC) developed the first widely adopted rainfall/runoff simulation model, HEC-1, in 1973 (USACE 1973). The HEC-1 model was replaced with the HEC-Hydrologic Modeling System (HEC-HMS) model in 1998 and these two models were utilized for all hydrologic modeling for the four Boyce Hydro dams in the Sanford watershed. Most of the modeling studies dealt with calculating the PMF for dam safety.

In 1978, Commonwealth Associates Inc. completed the USACE National Dam Safety Program Inspection Reports for Sanford, Edenville, Smallwood, and Secord Dams (Commonwealth Associates 1978a, 1978b, 1978c, 1978d). However, there was little technical information relating to the hydrologic modeling presented in those reports. The HEC-1 computer program was utilized in conjunction with the Snyder Unit hydrograph to determine the PMF for the four dams. The PMP values were most likely determined using the 1956 NWS Hydrometeorological Report (HMR) 33 (NWS 1956), since it would have been the most recent HMR available at that time.

The more recent hydrological studies that have been completed include the 1994 PMF Tittabawassee River Michigan study (Mead & Hunt 1994) and the 1995 Addendum No. 1 to that report (Mead & Hunt 1995); the PMF Reanalysis Edenville Hydroelectric Project (Mill Road Engineering [Mill Road] 2008, 2009, 2011); and the three studies by Ayres Associates (Ayres): (1) the Inflow Design Determination Edenville Hydroelectric Project (Ayres 2013); (2) the 2020 PMF Determination Tittabawassee River Hydroelectric Project (Ayres 2020); and (3) the Design Flood Hydrologic Analyses for the Sanford, Edenville, Smallwood, and Secord dams (Ayres 2021). Note that the third Ayres study was completed after the failures that occurred in May 2020.

C-1.1 Hydrologic Processes

The following sections are summarized from the HEC-1 and HEC-HMS reference manuals (USACE 1998, USACE 2020).

The USACE models were designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a subdivision of the basin. A component may represent a surface runoff entity, a stream channel, or a reservoir (dam). Representation of a component requires a set of parameters that describe the particular characteristics of the component and mathematical relations that describe the physical processes. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.

Subdividing the basin into a number of subbasins determines the number and types of stream network components to be used in the model. Two factors impact the basin subdivision: the study purpose and the hydrometeorological variability throughout the basin. First, the study purpose defines the areas of interest in the basin, and hence, the points where subbasin boundaries should occur (e.g., at the Edenville Dam). Second, the variability of the hydrometeorological processes and basin characteristics impacts the number and location of subbasins. Each subbasin is intended to represent an area of the watershed, which, on average, has similar hydraulic/hydrologic properties. The 1994 Mead & Hunt study divided the watershed into eight subbasins (Mead & Hunt 1994). The later studies subdivided the watershed into 11 subbasins in addition to separate reservoir components to capture the quick rise in the lake levels at the beginning of the rainfall event. (Mill Road 2008, Ayres 2013) The subbasin components are then linked to the channel or reservoir routing components to represent the connectivity of the river basin.

C-1.2 Hydrologic Steps

The methodologies presented in the 1994 Mead & Hunt report were essentially utilized in the future studies conducted by Mill Road and Ayres. The steps involved in the Mead & Hunt and the other studies are described below.

Delineation of the Watershed Boundaries

The watershed boundaries in the 1978 study were delineated using U.S. Geological Survey (USGS) topographic maps, and, in the later studies, the Environmental Systems Research Institute (ESRI)'s ArcGIS software provided a simple method for watershed delineation with initial resolution varying: 30 meter (m), 10 m, and 1 m resolution for the most recent studies. With the increased accuracy in the ArcGIS resolution, the watershed areas have been corrected and vary slightly from report to report (ESRI 2020)

Determine the Soil Characteristics of the Subbasins

"The soil databases have evolved from the State Soil Geographic (STATSGO) Database (U.S. Department of Agriculture [USDA] 1995) to Soil Survey Geographic (SSURGO) Database (USDA 2020) to the Natural Resources Conservation Service (NRCS, formerly Soil Conservation Service) (USDA 2022) for the U.S. The STATSGO data set is a digital general soil association map developed by the National Cooperative Soil Survey and distributed by the NRCS of the U.S. Department of Agriculture. It consists of a broad-based inventory of soils and non-soil areas that occur in a repeatable pattern on the landscape and that can be cartographically mapped. The soil maps for STATSGO are compiled by generalizing more detailed soil survey maps. Where more detailed soil survey maps are not available, data on geology, topography, vegetation, and climate are assembled, together with Land Remote-Sensing Satellite (LANDSAT) images. Soils of like areas are studied, and the probable classification and extent of the soils are determined. The mapping scale for STATSGO is 1:250,000" (USDA 1995).

The SSURGO database provides the most detailed level of information and contains information about soils as collected by the National Cooperative Soil Survey for the past 100 years (USDA 2020). The information can be displayed in tables or as maps and is available for most areas in the U.S. The information was gathered by soil scientists walking over the land and observing the soil. In addition, many soil samples were collected and analyzed in laboratories during these observations. The maps outline areas called map units. The map units describe soils and other components that have unique properties, interpretations, and productivity. The information was collected at scales ranging from 1:12,000 to 1:63,360. The mapping is intended for natural resource planning and management by landowners, but it is also suitable for engineering applications on a watershed scale. Some knowledge of soils data and map scale is necessary to avoid misinterpreting the information. (USDA 2020)

The NRCS provides soil data and information produced by the National Cooperative Soil Survey and maintained online as the single authoritative source of soil survey information (USDA 2022).

Determination of PMP

Prior to 1950, hydrologic design standards for dams were based mainly on judgment and experience. In 1964, the American Society of Civil Engineers (ASCE) Task Force on Spillway Design Floods classified dams into three categories:

Class 1: Dams where failure cannot be tolerated

Class 2: Dams where failure would result in serious economic loss

Class 3: Dams where failure would result in minor damage

"For large major structures (Class 1) that would be subject to possible failure if the selected capacity were exceeded, there would be a few instances, if any, where anything less than the provision for the Probable Maximum Flood can be justified" (Snyder 1964).

The estimated PMFs for the Sanford, Edenville, Smallwood, and Secord Dams have changed with the various studies as shown in Table C-3. The PMP methodology has improved, along with the additional rainfall data being collected over the past 100 years. The NWS has produced several HMRs for estimating the PMP over the U.S. east of the 105th meridian: HMR 23 in 1947; HMR 33 in 1953; and HMR 51/52 issued in 1978 and revised in 1982. In addition to the NWS studies, there have been two other studies conducted: the first, by the Electric Power Research Institute (EPRI) in 1993, *Probable Maximum Precipitation Study for Michigan and Wisconsin* (EPRI 1993); and the second, by Applied Weather Associates (AWA) in 2021, a site-specific study for the Tittabawassee River Basin prepared for Four Lake Task Force in coordination with Spicer Group, Ayres, and GEI Consultants Inc (GEI). The AWA methodology utilizes the Storm Precipitation Analysis System (SPAS), which is a state-of-the-science hydrometeorological tool used to characterize the magnitude, temporal, and spatial details of a precipitation event. SPAS utilizes real-time rain gauge observations, optimized Next Generation Weather Radar (NEXRAD) data, and climatological "basemap" approach to produce gridded rainfall at a spatial resolution of 1/3-square mile and a temporal resolution of 5 minutes. (NWS 1947, NWS 1956, NWS 1982, EPRI 1993, AWA 2021).

As the estimates of the PMP have improved, so have the hydrologic rainfall/runoff models advanced with improved computer software technology. The geographic information system (GIS) creates, manages, analyzes, and maps data such as USDA soils data and USGS land-use data that can be coupled to interact with the new computer software models (ESRI 2020).

Hydrologic Losses

The subbasin land surface runoff component is used to represent the movement of water over the land surface and in stream channels. The input to this component is a precipitation hyetograph. Precipitation excess is computed by subtracting infiltration and detention losses based on a soil water infiltration rate function. The rainfall and infiltration are assumed to be uniform over the subbasin for each computational time step. All of the PMF hydrologic studies that have been performed since 1994 have utilized the initial and uniform loss rate function. The initial loss rate is in units of depth (e.g., inches), and the constant loss rate function is in units of depth/hour (e.g., inches/hour). All rainfall is lost until the volume of initial loss is satisfied; after the initial loss is satisfied, rainfall is lost at the constant rate (inches/hour).

Unit Hydrograph

The unit hydrograph technique transforms (convolutes) the excess precipitation to a runoff hydrograph at the most downstream point in the subbasin. The Clark method (Clark 1945) has been used in all of the PMF studies since 1994 and requires three parameters to calculate a unit hydrograph: TC, the time of concentration for the basin; R, a storage coefficient, and a time-area curve. A time-area curve defines the cumulative area of the watershed contributing runoff to the subbasin outlet as a function of time (expressed as a proportion of TC). The ordinates of the time-area curve are converted to volume of runoff per second for unit excess and interpolated to the given time interval. The resulting translation hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin; the resulting unit hydrograph for instantaneous excess is then averaged to produce the hydrograph for unit excess occurring in the given time interval.

Spillway Hydraulics

Spillway capacity was generally estimated assuming free flow through the spillways using the weir equation with the gates being raised high enough to avoid orifice flow. (Mead & Hunt 1994).

Channel and Reservoir Routing

A river routing component is used to represent flood wave movement in a river channel. The input to the component is an upstream hydrograph resulting from one or more contributions of subbasin runoff. The hydrograph is routed to a downstream point based on the characteristics of the channel. There are a number of techniques available to route the runoff hydrograph. The preferred HEC-1 model is the Modified Puls method that utilizes the conservation equation in which inflow minus outflow through the river reach equals the change in the volume of storage within the river reach (USACE 1978).

The HEC-HMS model has a number of hydrologic routing methods, none of which can account for backwater effects; however, the Muskingum-Cunge is the preferred method for most situations. The Muskingum-Cunge routing technique can be used to route an upstream hydrograph through a main channel (USACE 1978). The channel routing technique is a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: (1) the parameters of the model are physically based; (2) the method has been shown to compare favorably against full unsteady flow equations over a wide range of flow situations; and (3) the solution is independent of the user-specified computation interval. The major limitations of the Muskingum-Cunge application are: (1) it cannot account for backwater effects; and (2) the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through very flat slopes (i.e., channel slopes less than 1 foot per mile [ft/mi]).

If necessary, the HEC-HMS model can be used in conjunction with the HEC-RAS hydraulic model to route the unsteady flow through the channel. The HEC-RAS model can route the flows using the dynamic wave equations for one- and two-dimensional flow (USACE 2021).

Use of the reservoir component is similar to that of the river routing component. The reservoir component can be used to represent the storage-outflow characteristics of a reservoir. The reservoir component functions by receiving upstream inflows and routing these inflows through a reservoir using storage routing methods. The reservoir routing can either be calculated with the hydrologic models (HEC-1 or HEC-HMS) or using the HEC-RAS model (USACE 1978, USACE 2021).

The initial software used for the hydrologic modeling was the HEC-1 Flood Hydrograph Package. This was later replaced by the HEC-HMS, which is designed to simulate the complete hydrologic processes of dendritic watershed system. The more recent advancement in the hydrologic/hydraulic modeling capability has been the incorporation of the HEC-RAS to encompass HEC-HMS (USACE 1978, USACE 2020, USACE 2021).

C-2 Hydrologic and Hydraulic Studies

1926

The IFT found a gate discharge curve developed by Holland, Ackerman, and Holland, dated 1926, that correlated pond level, gate opening, and discharge. However, criteria for sizing the spillways during the original designs was not determined by the IFT, and it is not clear how the spillway was sized regarding hydraulic capacity.

1978, September

National Dam Safety Phase I Program Inspection Reports for Sanford, Edenville, Smallwood, and Secord.

PMP values that were reported in the Phase I Inspection Reports were most likely determined using HMR 33, which was published in 1956 (NWS 1956, Commonwealth Associates. 1978a, 1978b, 1978c, 1978d).

The spillway capacity of the dam was estimated to be about 29,000 cubic feet per second (cfs) with 15 feet of water flowing over the top of the spillway structure; this was assuming that all gates are open, and the powerhouse turbines are not in operation. From the Phase I Edenville study, it was estimated that the peak outflow would be about 124,500 cfs and the earth embankments of the dam would be overtopped by approximately 2.68 feet with the assumption that the dam would not fail (Commonwealth Associates 1978c).

1991, March

Initial Consultant’s Safety Inspection Reports for: Sanford, Edenville, Smallwood, and Secord, A. R. Blystra & Associates, Ltd. (Blystra 1991)

The initial Part 12D Consultant’s Safety Inspection Report (CSIR) was prepared by A. R. Blystra & Associates and submitted in 1991 (Blystra 1991). This report indicated that the PMF inflow was estimated to be 56,200 cfs (less than half of the 1978 estimate shown above). The spillway capacity with the lake level at the top of the dam was estimated to be 22,950 cfs, or 41 percent of the PMF. The antecedent moisture condition assumed that the soils in the watershed had relatively high infiltration rates (low runoff potential).

1994, August

Probable Maximum Flood Study Tittabawassee River Basin, Michigan (Mead & Hunt 1994)

The 1994 Probable Maximum Flood Study Tittabawassee River Basin by Mead & Hunt was the first comprehensive hydrologic model for the Sanford watershed performed for the Federal Energy Regulatory Commission (FERC). The loss rate function and the unit hydrograph parameters were calibrated to stream gauge data for the South Branch Tobacco River and to four floods that had occurred on a similar gauged watershed of the Rifle River in Ogemaw County, Michigan. The Clark unit hydrograph parameters and the loss rate methodology were utilized or modified in all subsequent hydrologic studies by Mill Road and Ayres (Mill Road 2008, 2009, 2011, Ayres 2013).

The study was conducted to estimate the PMFs for the Secord, Smallwood, Edenville, and Sanford hydroelectric projects on the Tittabawassee River in Michigan. The *Probable Maximum Precipitation Study for Wisconsin and Michigan, Volumes 1 and 2* (EPRI 1993) was used to estimate the PMP values. This was in lieu of utilizing HMR 51. The EPRI study was the first major PMP study in the U.S. performed by an agency other than the NWS.

The USACE HEC-1 watershed model was used to generate the predicted flood hydrographs for each subbasin. The USACE Unsteady NETwork model (UNET) was used to combine and route the subbasin flood hydrograph downstream to the Sanford Dam, generating PMF hydrographs at each subbasin. Dynamic routing was used in reservoirs with outflow controlled by rating curves that were developed assuming all gates to be fully opened to allow free flow over the concrete spillway crests. The PMF inflow for the Edenville Dam was estimated to be 74,400 cfs, while the spillway capacity was 32,800 cfs, which included some overtopping flow over the low points of the dam crest.

1995, February

***Addendum No. 1* to the August 1994 PMF Study (Mead & Hunt 1995)**

This study was undertaken to confirm an IDF for Sanford Dam that was less than the PMF value. The IDF was computed to be about 49 percent of the PMF event. The IDF was based on “incremental damage consequence” criteria.

1995, December

Review of Probable Maximum Flood Calculations Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects (Mead & Hunt 1995)

The 1994 Mead & Hunt report was accepted by FERC. Based on observed operations during flood events, Wolverine Power Corporation requested Mead & Hunt to review and analyze operations during flood events to determine if a restudy of the PMF might be warranted. The objective of the study was to use available storm operation records to assess the ability of the developed watershed model to replicate runoff and time sequences of the reservoir data.

The conclusion of this study was that a reevaluation would require a literature search and extensive stream gauge analysis to evaluate relationships between the unit hydrograph parameters and watershed characteristics. Moving forward, a two-phase approach was suggested. Phase I consisted of performing the literature search to determine if appropriate and sufficient data existed. The result of Phase I would determine if proceeding with a detailed restudy (Phase II) was warranted.

Neither the analysis of a 1986 rainfall event nor a general review of hydrologic parameters derived for the existing PMF model indicated whether a restudy of unit hydrograph parameters would likely yield an overall positive or negative change in project PMFs: therefore, no further studies were performed.

2008, October

Probable Maximum Flood Reanalysis Edenville Hydroelectric FERC Project No. 10808 (Mill Road 2008)

Mill Road reviewed the data used in the 1994 Mead & Hunt PMF Study and concluded that the inflows from the contributing river segments from the Tobacco and the Tittabawassee Rivers were not correctly routed through the basin because the two rivers peak at significantly different times. Based on the conclusion that the river peaks would not be additive, a new study was undertaken.

This study used the Mead & Hunt rainfall excess and unit hydrographs developed in the 1994 study. The storm event was routed using the HEC-RAS computer model instead of the UNET USACE routing model utilized by Mead & Hunt. Using HEC-RAS provided dynamic river routing (unsteady flow routing) to model the interaction between the Tobacco River and the Tittabawassee Rivers via the Highway M-30 bridge-causeway. The M-30 causeway separates the reservoirs and when the river inflow peaks are separated, the abilities of both the Edenville and Tobacco gate sets allow both gate sets to discharge flood flows from each peak as they occur. The model indicated the benefit of modeling the two river basins separately.

The 2008 HEC-RAS model produced smaller total inflow than the Mead & Hunt study, and even though the maximum inflow peak was lower than the Mead & Hunt value, the existing gate capacity in the Edenville Dam was found insufficient to pass the flood without the dam overtopping. Mill Road initially looked at several solutions to increase the spillway capacity and pass the flood without the dam overtopping:

1. Utilize a stanchion structure solution comprising a structural slab support on piers spaced regularly across an opening. The opening is a broad crested weir with a side wall supporting the earthfill beyond the structure. There would be two structures: one on the Edenville side would be 121 feet wide with a sill elevation of 667.8, the same as the sill elevation of the gates, and one on the Tobacco side would be 36 feet wide with a sill elevation of 667.8. The peak inflow equals 65,911 cfs, with outflow of 64,885 cfs.
2. Lower the existing gate sill at the Edenville gated spillway to elevation (El.) 656.0 with no change to the existing width of the three Edenville gates. No other modifications would be necessary.
3. Install a 56-foot-wide overflow structure with a sill elevation of 676.0 in the Tobacco Dam on high ground west of the existing Tainter gates and install 101-foot-long fuse plug on the Edenville embankment with a crest elevation of 678 and sill elevation of 665.5. No additional changes would be required. Each structure will discharge into a riprap-protected swale, then into the river adjacent to these structures.
4. Install a 600-foot-long fuse plug on the Edenville side (Tittabawassee River side). The fuse plug will have a sill elevation of 676.5 feet and a crest elevation of 681.
5. Install a sheet pile opening in the Edenville crest, 250 feet wide, with an invert elevation of 667.8 feet protected by riprap, extending downstream between two sheet pile side walls and buttress

embankment, ending in a fuse plug, 350 feet long, with a crest elevation of 681 and a sill elevation of 681.

Each of the above solutions were modeled in HEC-RAS to confirm the final lake level and that the flow capacity was adequate to pass the PMF.

2009, February

Probable Maximum Flood Reanalysis Edenville Hydroelectric FERC Project No. 10808 (Mill Road 2009)

In 2009, Mill Road modified their initial reanalysis and determined that the least cost solution based on the comparative conceptual estimates is to depress the sill elevation of the Edenville and Tobacco gates to El. 655.8 and El. 657.8, respectively. The solution would have been accomplished incrementally such that the six gates did not need to be taken out of service as a group to accomplish this spillway modification. Construction was planned based on using a cofferdam, which would have been located on the existing pier and sloping ridge beam found at each pier location of each dam. A summary of the analyzed spillway alternatives is presented in Table C-1.

Table C-1: Summary of 2009 Mill Road Engineering Alternatives

Alternative	Peak Reservoir Elevation (feet)	Combined Peak Inflow (cfs)	Peak Outflow (cfs)
Stanchion Structure Option – Sill El. 667.8	Tobacco 682.9 Tittabawassee 682.8	60,923	60,923
600 ft long fuse plug at Edenville – Sill El. 676.8	Tobacco 682.75 Tittabawassee 682.73	65,911	64,885
Gate Solution, depress Edenville Gate Crest to El. 655.8 and depress the Tobacco Gate Cress El. 657.8	Tobacco 679.82 Tittabawassee 682.25	60,514	59,951
56 feet Broad Crested weir at Tobacco – Sill El. 676.0, 101 fee Edenville Fuse Plug, Fuse Plug El. 665.5	Tobacco 682.66 Tittabawassee 682.74	62,968	62,655
A sheet pile opening in the Edenville Crest just east of M-30 – Sill El. 671.0, Stanchion outlet control structure downstream	Tobacco 682.42 Tittabawassee 682.38	65,077	64,895

2011, April

Probable Maximum Flood Reanalysis Edenville Hydroelectric FERC Project No. 10808 (Mill Road 2011)

In 2011, Mill Road made their final recommendation that gate modifications be made to the Edenville spillway and the Tobacco spillway by lowering the existing gate sill elevation from 667.8 to 654.8 for all six structures. The existing Tainter gates would be replaced to provide the necessary structural strength for the new design head. The existing gate hoist chains and gate hoists were not able to achieve the full open gate position required and modeled in this study. Therefore, it was recommended that dual cable gate hoists be installed at the Edenville and Tobacco spillways to allow the gates to be opened to their fully open position. These spillway modifications were never constructed.

2012, March

Tainter Gate Design Report and Calculations, Edenville Project, Edenville Hydroelectric Project, FERC Project No. 10808 (Mill Road 2011)

Mill Road provided a replacement Tainter gate design for the Edenville and Tobacco spillways based on the previously PMF study and recommended gate modification studies. The proposed spillway modifications included the lowering of the spillway crests to El. 654.8 feet. The design width of each spillway bay was approximately two feet narrower than the existing bay widths. The proposed Tainter gate design would have stood 21 feet tall from the spillway crest to the normal pool elevation. The top of gate elevation was proposed at 677.37 feet. The final PMF reservoir elevation on the Tobacco Ponds was 680.26, 2.74 feet below the dam crest. The Edenville final pond elevation was 682.17, 0.83 feet below the dam crest.

During the years of the ongoing PMF studies, Boyce Hydro had chosen to rehabilitate the existing spillways and at the same time to lower the spillway crest from the current elevation of 667.8 to 654.8 feet mean sea level (MSL). A design to accomplish that goal was completed and submitted to FERC. As result of several joint meetings with FERC and the Board of Consultant (BOC) that was comprised of two qualified experts to review the design; the design was vetted and was accepted as the method satisfying the PMF criteria for the Edenville Project, P-10808 in November 2012. Once the design was approved, FERC requested a schedule for execution. Construction was scheduled for June 2013. However, prior to the time scheduled for construction to start, Boyce Hydro notified FERC that it did not have the finances to begin construction as previously anticipated. FERC scheduled a meeting in Washington D.C at FERC's office on August 9, 2013, to discuss this and requested that Boyce Hydro, LLC meet with the BOC, FERC, and its staff to discuss alternatives to resolve the PMF concern. The end result was that these spillway modifications were never constructed.

2013

Inflow Design Flood Determination, Edenville Hydroelectric Project, FERC Project No. 10808. (Ayres Associates 2013)

A dam failure analysis of the Edenville Dam was performed to determine the IDF for the project under FERC dam safety guidelines. To complete the analysis, Ayres modified and merged two existing HEC-RAS models of the Tittabawassee and Tobacco Rivers. This analysis used inflow hydrographs taken from the model previously developed by Mill Road. The updated model included the two separate project spillways, the causeway bifurcating the Edenville impoundment, the confluence of the Tobacco and Tittabawassee Rivers below the project, and the Tittabawassee River from the Tobacco-Tittabawassee confluence to Sanford Dam.

River reaches upstream of Wixom Lake were truncated so that the model's upstream boundaries were 5.5 miles upstream of the dam on the Tobacco River and 10.3 miles upstream of the dam on the Tittabawassee River. PMF hydrographs at the truncated boundary conditions were taken from Mill Road's PMF model.

The analysis showed that a failure of the embankment at either the Tobacco or the Edenville dam structures during the PMF event would cause a flood wave of up to 11 feet in height at various inhabited structures between Edenville and Sanford. Therefore, the IDF for the project was determined to be the PMF.

2013, November

PMF Alterations Design Plan Additional Evaluations, Edenville P-10808 (Christie Engineering 2013)

This study evaluated the effects of drawing down Secord, Smallwood, and Wixom Lakes. The Edenville Dam project PMF studies had been ongoing for a number of years. The original owners of the project, Wolverine Power Company, began PMF studies in 1994.

The purpose of the study was to determine the impact on the PMF analysis of pre-drawing the Secord, Smallwood, and Edenville reservoirs down to the Tainter gate spillway crests. The results showed that the pre-dawn reservoir provides a small benefit to the total PMF flow by reducing the peak flow by less than 2,000 cfs and delaying the peak by one hour.

The spillway capacity was estimated to be a combined 30,000 cfs at the top of dam (El. 682.8), using the weir equation with a discharge coefficient of 3.95, a head of 15 feet, and a crest length of 63.5 feet and 67 feet for the Tittabawassee and Tobacco River gates, respectively.

Further evaluations of the pre-lowering showed the delay of the arrival of the peak flood wave of 6 hours. The peak flood elevation and flow would not be changed. Therefore, based on the magnitude of the PMF flow and early warnings (4 to 6 days), there was not believed to be a significant benefit from attempting to lower the reservoirs in advance of a possible PMF condition.

This study also looked at interim mitigation and permanent solutions for the spillway deficiency at Edenville for passing the PMF.

Interim Solutions:

Option 1: Flat slab spillway with end walls, a sheet pile cutoff wall on the upstream end, and a top of slab elevation of 1 foot above the normal lake level. To accommodate the full flow, the Tobacco side would have required a spillway length of 330 feet, and the Tittabawassee a spillway length of 372 feet.

Option 2: Flat slab spillway with end walls, a sheet pile cutoff wall, and a top of slab elevation of 4 feet below normal pool. Steel flap gates, or boards, would have been located near the reservoir end of the slab and extended about 2 feet above normal pool. To accommodate the full flow, the tobacco side would have required a spillway length of 133 feet, and the Tittabawassee a spillway length of 150 feet.

Option 3: A labyrinth weir with the top of weir 4 feet below the top of dam, or 3 feet above normal pool. Overall lengths of 267 feet and 303 feet were respectively estimated for the Tobacco and Tittabawassee sides of the dam.

Proposal: The interim proposal would have been to construct one-third of Option 1 on both the Tittabawassee and Tobacco sides of the reservoir. This would have provided approximately 10,600 cfs of additional spillway capacity at a water level at the top of dam.

Permanent Solution:

Option 1: If the existing spillways could be repaired at minimum cost, the final solution may be to continue with the auxiliary spillway Option 1 described above until the desired capacity was reached.

Option 2: Assuming the spillways were in need of extensive concrete renovation, the spillways would be reconstructed as needed to replace and repair the concrete. The reconstruction would have included demolishing the existing spillway slabs and reconstruction of a new solid spillway 6 to 8 feet lower than the existing crest. New gates would have been installed that were 6 to 8 feet higher than the existing gates. The proposed modified main spillways would be smaller and less expensive than the 2012 proposal because there would be additional capacity for the interim auxiliary spillway constructed to pass 10,600 cfs.

2016

Design Report for Edenville Hydroelectric Project Tobacco River Auxiliary Spillway FERC No. 10808 (Boyce Hydro 2016)

This auxiliary spillway was to provide an additional 5,600 cfs of flow capacity when the reservoir is at the top of the dam, El. 682.8. This would increase the total capacity of the spillways to 34,000 cfs (existing capacity of 28,400 cfs + 5,600 cfs). The spillway was designed as a broad crested weir, where the upstream end of the slab would be set 2 feet above the normal reservoir level with an initial level section of 8 feet.

The BOC reviewed the documents and drawings sent by the Boyce Hydro engineer and commented in an April 22, 2016, letter to Boyce Hydro that “the design assumptions and calculations were reasonable and adequate.” The BOC suggested that a formal risk analysis could provide valuable information as to what the acceptable design flood should be. The flood frequency curve that was presented in the flood frequency report yielded a return period for the PMF of 62,000 cfs of approximately 10,900,000 years (an annual exceedance probability [AEP] of $9x10^{-8}$). The combined spillway capacity after construction of the auxiliary spillway of 34,000 cfs would have an estimated return period of 10,000 years or an AEP of 0.0001.

2018, March

Edenville Hydro Project FERC Project No. 10808 Tobacco Auxiliary Spillway. Project Status Report, March 16, 2018. (Gomez and Sullivan 2018)

The report titled Edenville Hydro Project FERC Project #10808 Tobacco Auxiliary Spillway summarizes work activities completed by Gomez and Sullivan through March 15, 2018. Gomez and Sullivan completed soil boring tests and were developing plans to increase the capacity of the spillway. The supporting design report was approximately 75 percent complete. The auxiliary spillway was to have a capacity of 5,600 cfs, bringing the combined spillway capacity to about 34,000 cfs. Temporary construction emergency action plan, quality control and inspection, and water management reports that had been prepared by Boyce Hydro were all about 80 percent completed.

At the same time, Gomez and Sullivan were also investigating a labyrinth spillway that would increase the capacity by 12,000 cfs for a total combined flow of 40,000 cfs. Gomez and Sullivan were then tasked by Boyce Hydro to support the design efforts for the proposed labyrinth spillway.

2019, June

Gate Tests – Edenville Project, Technical Memorandum (Purkeypile Consulting LLC 2019)

Purkeypile Consulting (Purkeypile) provided a memorandum to Boyce Hydro supporting the evaluation of the discharge capacity of the six existing radial gates for the Edenville Hydroelectric

Project. All six gates were opened to the maximum height considered to be safe. The results of the test were that the maximum openings of the Tittabawassee gates were 9.49, 8.98, and 9.55 for Gates 1, 2, and 3, respectively. The maximum openings of the Tobacco gates were 9.85, 8.93, and 8.89 feet for Gates 1, 2, and 3, respectively.

Purkeypile's opinion was that the six spillway gates for the Edenville Hydroelectric Project should only be operated with the original hoist mechanisms until they can be replaced with electric hoists and that using the portable A-frames and the manual level hoist is cumbersome and requires too much time to operate under emergency conditions.

It was also recommended that electric gate hoists be installed to lift the existing radial gates at the Edenville Project as soon as practicable.

2020, April

Draft Discharge Rating Curves (Secord, Smallwood, Edenville and Sanford Projects) Four Lakes Task Force (FLTF) (GEI Consultants Inc. 2020)

In April 2020, GEI completed a memorandum that presented results of new spillway discharge rating curves developed for the Tainter gate spillway and overflow sections located at the Secord, Smallwood, Edenville and Sanford Projects. The supporting calculations for spillway discharge rating curves presented in the Standard Technical Information Documents (STID) were not available, and the rating curves appeared to be inconsistent with recent spillway surveys and maximum gate opening tests. The FLTF had requested that GEI review the available hydraulic information and develop new spillway discharge rating curves for each of the four projects.

The hydraulic computations accounted for both free flow over the weirs and radial gate flow when the upstream water surface was greater than or equal to 1.25 times the gate opening height. The spillway rating curves included variable discharge coefficients in relation to energy head on the crest and changes in effective length of the spillways as a result of the piers and abutments.

The total zero-freeboard discharge capacity at Edenville Dam was estimated to be 20,700 cfs at the minimum dam crest elevation of 682.1.

2020, May

Probable Maximum Flood Determination, Tittabawassee River Hydroelectric Projects, Secord, Smallwood, Sanford, Edenville (Ayres Associates 2020)

PMF and spillway capacity analyses completed for the projects between 1994 and 2017 indicated that the spillway capacities at Secord and Edenville Dams were less than the required PMF discharges, while the capacity at Smallwood Dam exceeded the PMF. This study was undertaken on behalf of the FLTF, which was preparing to acquire the dams. The objective of the study was to re-evaluate the PMF at all four Tittabawassee River facilities using improved precipitation, streamflow, and watershed data, and reflecting updated FERC guidelines. To accomplish this, a HEC-HMS model was constructed for the upstream basin boundary to Sanford Dam. Changes in the 2020 HMS model relative to the previous HEC-1 model included delineation of basin and subbasin boundaries using the National Elevation Dataset; aggregation into a single HMS model instead of the 1994 HEC-1/UNET combination; more detailed subbasin division; use of SSURGO soil data to estimate loss potential distributions for each subbasin; and calibration of the model using NEXRAD precipitation records, streamflow data from USGS gauges and the lake levels for the four dams.

The calculated PMF peak inflows to the Secord, Smallwood, Edenville, and Sanford reservoirs were 29,400 cfs, 41,200 cfs, 80,900 cfs, and 80,600 cfs, respectively. These represent increases of 10 percent, 0.5 percent, 30 percent, and 7 percent, respectively, over the previously accepted values of the Mead & Hunt (Secord, Smallwood, and Sanford). The increases can be attributed primarily to a decrease in simulated hydrologic loss rates in 2020. The disproportionately larger increase at Edenville Dam is also attributed to: (1) a more critical storm position over the watershed than was previously evaluated; and (2) determined that there was not a significant offset between the Tobacco and the Tittabawassee flows shown by the Mill Road study Ayres (2020) (Mill Road 2011).

2021, July

Design Flood Hydrologic Analyses, Secord, Smallwood, Edenville, and Sanford Dams, Gladwin and Midland Counties, MI (Ayres Associates 2021)

Note: This study was completed after the failures of Edenville and Sanford Dams.

FLTF contracted with Ayres for hydrologic analyses to support the selection of spillway design floods for the Secord, Smallwood, Edenville, and Sanford Dams on the Tittabawassee River in Michigan. At the time of writing this forensic report, the FLTF, a delegated authority of Gladwin and Midland Counties, Michigan, is in the process of having the dams redesigned and reconstructed following the failures in May 2020, and the selection of appropriate design floods is a cornerstone of that effort.

Design flood hydrographs developed in this study included the PMF; the “half PMF”, which is defined by Michigan dam safety regulators as the flood resulting from half of the PMP; and floods generated from precipitation events having AEPs of 0.01, 0.005, 0.002, 0.001, and 0.0002.

The USACE HEC-HMS model was used to generate flood hydrographs at various locations throughout the watershed. The HEC-HMS model was calibrated to four observed events using precipitation time series developed by AWA. This resulted in four sets of calibrated unit hydrograph and loss parameters. These were weighted for use in the PMF and “half-PMF” model based on flood magnitude and calibration quality. For the exceedance probability flood model, the calibrated parameters were weighted based on calibration quality and correspondence between the exceedance probability of the rainfall event and the exceedance probability of the modeled flood. The adopted model was tested against a fifth flood and accepted with no further modifications.

Spillway rating curves used for this study were derived from calculations provided by GEI prior to the May 2020 flood (GEI 2020). The Smallwood, Edenville, and Sanford inflows were based on the pre-failure 2020 storage-discharge relationships at upstream dams. The resulting Edenville PMF inflow increased from 80,900 cfs in the 2020 study to 113,400 cfs for this study. The increase was mostly attributed to the model being calibrated to the four storms, but in particular to the May 19, 2020, rainfall event.

C-2.1 Spillway Capacity Summary

A summary of the estimated spillway capacities at the Edenville Dam from 1991 to 2020 is given in Table C-2.

Table C-2: Estimated Spillway Capacity at Edenville Dam – Various Studies and Documents from 1991 through 2020

1991 Inspection Report (Blystra 1991)	
Edenville spillway capacity (at top of dam)	22,950 cfs
1994 PMF Study (1994 Mead & Hunt)	
Edenville spillway capacity (at nominal top-of-dike elevation)	23,650 cfs[1]
2013 Inflow Design Flood Determination Study (Ayres 2013)	
Edenville spillway capacity (at top of dam 683.0 feet)	32,000 cfs[2]
2013 PMF Alterations Design Plan (Christie Engineering 2013)	
Edenville spillway capacity (at top of dam)	30,000 cfs
2015 CSIR (Purkeypile 2016a)	
Edenville spillway capacity (at minimum dam crest elevation of 682.1 feet)	26,487 cfs
Edenville spillway capacity (at design dam crest elevation of 682.8 feet)	28,338 cfs
2020 Discharge Rating Curve Study (GEI 2020) (Note: Adopted for 2020, 2021 Ayres studies)	
Edenville spillway capacity (at minimum dam crest elevation of 682.1 feet)	20,670 cfs
Edenville spillway capacity (at El. 683.0 feet)	21,210 cfs

[1] Total 1994 capacity documented as 32,800 cfs, which included 9,150 cfs of overtopping flow.

[2] Includes flow through a small powerhouse sluice.

C-2.2 PMF Summary

A summary of the estimated peak PMF values for the four dams from 1978 to 2021 is shown in Table C-3.

Table C-3: Estimated Probable Maximum Flood Peak Inflows – Sanford, Edenville, Smallwood, and Secord Dams – Various Studies from 1978 through 2021

1978 Phase I Inspection Report (Commonwealth Assoc. 1978)	
Secord inflow	PMF Not Available
Smallwood inflow	PMF 73,700 cfs
Edenville inflow	PMF 125,500 cfs
Sanford inflow	PMF 129,000 cfs
1991 Inspection Report (Blystra 1991)	
Secord inflow	PMF 20,600 cfs
Smallwood inflow	PMF 18,600 cfs
Edenville inflow	PMF 56,200 cfs
Sanford inflow	PMF 56,100 cfs
1994 PMF Study (Mead & Hunt 1994)	
Secord inflow	PMF 27,200 cfs
Smallwood inflow	PMF 41,000 cfs
Edenville inflow	PMF 74,400 cfs
Sanford inflow	PMF 75,500 cfs

2008 and 2009 PMF Reanalysis Studies (Mill Road 2008, 2009)	
Edenville inflow (varies across solutions analyzed)	PMF 61,000 – 66,000 cfs
2011 PMF Reanalysis Study (Mill Road 2011)	
Edenville inflow	PMF 61,900 cfs
2013 Inflow Design Flood Determination Study (Ayres 2013)	
Edenville – Tittabawassee River side peak inflow	PMF 52,900 cfs
Edenville – Tobacco River side peak inflow	PMF 24,600 cfs
Edenville – Estimated combined peak inflow	PMF 67,400 cfs
2020 PMF Determination Study (Ayres 2020)	
Secord inflow	PMF 29,400 cfs
Smallwood inflow	PMF 41,200 cfs
Edenville inflow	PMF 80,900 cfs
Sanford inflow	PMF 80,600 cfs
2021 Design Flood Hydrologic Analyses (Ayres 2021)	
Secord inflow	PMF 29,200 cfs
Smallwood inflow	PMF 48,200 cfs
Edenville inflow	PMF 113,400 cfs
Sanford inflow	PMF 117,200 cfs

C-3 References

Applied Weather Associates (AWA). 2021. *Site-Specific Probable Maximum Precipitation Study for Tittabawassee River Basin, Michigan: Final Report.*

A.R. Blystra & Associates (Blystra). 1991 (March). *Initial Consultant's Safety Inspection Report, Edenville Hydroelectric Project*.

Ayres Associates (Ayres). 2013. *Inflow Design Flood Determination, Edenville Hydroelectric Project*. FERC Project No. 10808.

———. 2020 (May). *Probable Maximum Flood Determination, Tittabawassee River Hydroelectric Projects, Secord, Smallwood, Sanford, and Edenville Dams*. Prepared for Spicer Group, Inc. and Four Lakes Task Force. Available: https://www.four-lakes-taskforce-mi.com/uploads/1/2/3/1/123199575/ttbw_pmf_final_report-may2020.pdf.

———. 2021 (July). *Design Flood Hydrologic Analyses, Secord, Smallwood, Edenville, and Sanford Dams*.

Boyce Hydro Power, LLC (Boyce Hydro 2016) *Design Report for Edenville Hydroelectric Project Tobacco River Auxiliary Spillway,* FERC Project No. 10808, January 2016).

Clark, C.O. 1945 (January). "Storage and the Unit Hydrograph." *Transactions of the American Society of Civil Engineers* 110:1419-1446.

Christie Engineering. 2013 (November 29). PMF Alterations Design Plan Additional Evaluations, Edenville P-10808.

Commonwealth Associates. 1978a (September). Tittabawassee River Basin, Gladwin and Midland Counties, Michigan, National Dam Safety Program Inspection Reports, USACE Detroit District, Detroit Michigan, Secord Dam. Prepared for Michigan Department of Natural Resources.

———. 1978b (September). Tittabawassee River Basin, Gladwin and Midland Counties, Michigan, National Dam Safety Program Inspection Reports, USACE Detroit District, Detroit Michigan, Smallwood Dam. Prepared for Michigan Department of Natural Resources.

———. 1978c (September). Tittabawassee River Basin, Gladwin and Midland Counties, Michigan, National Dam Safety Program Inspection Reports, USACE Detroit District, Detroit Michigan, Edenville Dam. Prepared for Michigan Department of Natural Resources.

———. 1978d (September). Tittabawassee River Basin, Gladwin and Midland Counties, Michigan, National Dam Safety Program Inspection Reports, USACE Detroit District, Detroit Michigan, Sanford Dam. Prepared for Michigan Department of Natural Resources.

Electric Power Research Institute (EPRI). (1993). Probable Maximum Precipitation Study for Wisconsin and Michigan: Volume 1 and Volume 2. EPRI TR-101554.

Environmental Systems Research Institute (ESRI). 2020. ArcGIS software is available from ESRI.

Geotechnical Engineering Inc. (GEI). 2020 (April 27). *Discharge Rating Curves (Secord, Smallwood, Edenville and Sanford Projects* for Four Lakes Task Force, Midland, MI.

Gomez and Sullivan Engineers (Gomez and Sullivan). 2018 (March 16). Edenville Hydro Project FERC Project No. 10808 Tobacco Auxiliary Spillway. Project Status Report.

Mead & Hunt. 1994 (August). “Probable Maximum Flood Study, Tittabawassee river, Michigan, Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects,” FERC Project Nos. 10809, 10810, 10808, 2781.

———. 1995 (December). Review of Probable Maximum Flood Calculations Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects.

Mill Road Engineering (Mill Road). 2008 (October). *Probable Maximum Flood Reanalysis Edenville Hydroelectric Project*. FERC Project No. 10808. October 16, 2008.

______. 2009 (February). *Probable Maximum Flood Reanalysis Edenville Hydroelectric Project*. FERC Project No. 10808.

_______.2011 (April). *Probable Maximum Flood Reanalysis Edenville Hydroelectric Project*. FERC Project No. 10808.

_______.2012 (March). *Tainter Gate Design Report and Calculations, Edenville Project, Edenville Hydroelectric Project*, FERC Project No. 10808

Natural Resources Conservation Service (NRCS). 2022. Web Soil Survey. U.S. Department of Agriculture, Natural Resources Conservation Service. Available: http://websoilsurvey.nrcs.usda.gov/.

National Weather Service (NWS). 1947 (June). Generalized Estimates of Maximum Possible Precipitation over the U.S. East of the 105th Meridian, for Areas of 10, 200, and 500 Square Miles. Hydrometeorological Report No. 23. Available: https://www.weather.gov/media/owp/oh/hdsc/docs/HMR23.pdf.

———. 1956 (April). *Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours*. Hydrometeorological Report No. 33. Available: https://www.weather.gov/media/owp/oh/hdsc/docs/HMR33.pdf.

———. 1978. *Probable Maximum Precipitation Estimates, U.S. East of the 105th Meridian*. Hydrometeorological Report No. 51. Available: https://www.weather.gov/media/owp/oh/hdsc/docs/HMR51.pdf.

______. 1982. *HMR52 Probable Maximum Storm (Eastern U.S.)*. March 1984 (Revised April 1987). Available: https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HMR52_UsersManual_(CPD-46).pdf

Purkeypile Consulting, LLC (Purkeypile). 2016a (March). *2015 Consultant's Safety Inspection Report, Edenville Hydroelectric Project, FERC Project No. 10808*.

Purkeypile Consulting, LLC (Purkeypile). 2016b (March 18). *2015 Potential Failure Modes Analysis, PFMA Supplement, Completed as Part of 2015 CSIR, Edenville Hydroelectric Project, FERC Project No. 10808*.

_______. 2019. Edenville Project, Technical Memorandum submitted to Boyce Hydro, LLC in support of an evaluation of the discharge capacity of the six existing radial gates for the Edenville Hydroelectric Project (Purkeypile Consulting, LLC 2019)

Showalter, A. K., and S. B. Solot, 1942: Computation of maximum possible precipitation. Trans. Amer. Geophysical. Union, 23 (part 2), 258-274. 4. (1942)

Snyder, F.F. (May 1964), "Hydrology of Spillway Design: Large Structures - Adequate Data," Proceedings of the American Society of Civil Engineers, Journal of the Hydraulics Division, New York, N.Y.

U.S. Army Corps of Engineers (USACE). 1973. *HEC-1 Flood Hydrograph Package: User's Manual.* Available: https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-1_UsersManual_(CPD-1a).pdf.

______. 1978. *HEC-1 Flood Hydrograph Package: Updated last Version of User's Manual.* Available https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-1_UsersManual_(CPD-1a).pdf.

______. 2020 (April). HEC-HMS, *Hydrologic Modeling Systems*, Hydrologic Engineering Center, Reference Manual Available: https://www.hec.usace.army.mil/confluence/hmesdocs/hmstrm

______. 2021. *River Analysis System* (HEC-RAS), Hydrologic Engineering Center, Reference Manual. Available: https://www.hec.usace.army.mil/software/hec-ras/documentation.aspx

U.S. Department of Agriculture (USDA). 1995. State Soil Geographic (STATSGO) Data Base for the Conterminous U.S.. Available: https://water.usgs.gov/lookup/getspatial?ussoils.

———. 2020. Soil Survey Staff, Natural Resources Conservation Service, U.S. Department of Agriculture. Soil Survey Geographic (SSURGO) Database. Available: https://sdmdataaccess.sc.egov.usda.gov.

_______. 2022. Natural Resources Conservation Service, United State Department of Agricultural. Web Soil Data. Available: https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

U.S. Department of Interior Bureau of Reclamation (USBR) 1987. Design of Small Dams (Third Edition 1987). Available: https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf

Appendix D: Construction Drawings, Specifications, Photos and Memos

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Attachment D-1: Construction Photos

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Fill placed near the foundation level of Edenville east (left) embankment. Note stockpile of material in background in downstream portion of the embankment. (April 1924)

Fill placed near the foundation level of Edenville east (left) embankment. Note stockpile of material in background in downstream portion of the embankment and what appears to be linear alignments of foundation drains. (May 1924)

Fill placed in lower portion of Edenville east (left) embankment. Note material in background being stockpiled with clam-shell bucket in downstream portion of the embankment. Foundation drain pipes are visible along the downstream toe. Note the apparent color variation between upstream and downstream areas of the embankment. (May 1924)

Fill placed in lower portion of Edenville east (left) embankment. Note material in background being stockpiled in downstream portion of the embankment. Note in this photo the rail car from the east is placing fill in the downstream area and the apparent color varation in that area. (July 1924)

Fill placed in lower portion of Edenville east (left) embankment. Note material in background stockpiled in downstream portion of the embankment. Note the downstream area of the embankment at a slightly higher elevation than the upstream. Aspen poles used for construction utilities are visible [represenative of an aspen pole observed in the embankment fill of the remnant right face of the embankment breach] (August 1924)

Fill placed in lower portion of Edenville east (left) embankment. Note material in background stockpiled in downstream portion of the embankment. Note the apparent color variation between upstream and downstream areas of the embankment. (no date)

Fill placed in Edenville east (left) embankment. Note the apparent color variation between upstream and downstream areas of the embankment. (October 1924)

Fill placed in Edenville east (left) embankment. Note material previously stockpiled in the background is no longer visbly present. Note cold-weather fill placement on snow-covered ground. Note rising lake level in comparison to earlier photo. (November 1924)

Fill placed in Edenville east (left) embankment near top-out elevation. Note what appears to be a slightly steeper slope near the top elevation. Note lake level in right background of photo. Note water along downstream toe (later construction memos [April 1925] will note that the downstream toe of this embankment is dry). (February 1925)

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Attachment D-2: Construction Memos

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WOLVERINE POWER COMPANY Ann Arbor, Michigan

General Jan. 20, 1925

H. K. Holland C. E. Bottum, Res. Engr.

We received Ellis' wire today saying that the Sanford plant was on at noon, and was glad to receive your telephone call saying that the Edenville plant was also connected to the Consumers system. We are leaving it to you to get in touch with Mr. Conrad and arrange for operating and for delivering energy to the Consumers Power system. This should be done as quickly as possible so that the plant will begin returning a revenue. The watthour meters which are on the bottom of the switchboards should be read at both the Sanford and Edenville plants, reading all of the meters, including the meters on the Consumers Power Company panels at Edenville. This should be done immediately so that we will know what the meters read when connection is made to the Consumers system.

Confirming our conversation over the 'phone, we do not want to pull our heads any further at either Edenville or Sanford and deliver only the power necessary each day so that the pond will still keep rising. It is essential that the Edenville pond on the Tittabawassee River reaches the crest before Spring floods so that we will have an opportunity to repair the seepage spots before we have higher floods. The Sanford plant should not be run to furnish water for the bearings. It will be cheaper to install the gasoline pump

Exhibit 3-2

HOLLAND, ACKERMAN & HOLLAND
Consulting Engineers

Inter-Office Correspondence

106
53 V **Exhibit 3-3** Arbor ago

Job WOLVERINE POWER CO. Place Sanford, Michigan.

Subject EDENVILLE EMBANKMENT. Date Jan. 30, 1925.

From C. E. Bottum. To H. K. Holland.

On Jan. 29th the Edenville pond was at Elev. 658.2 which is the highest it has been to date. The pond up until the time we started operating ranged around 656 to 655. The Embankment first showed signs of "sluffing" on Dec. 10, 1924 on which date the pond was at El. 655.35. On Dec. 8th and 9th the pond had been at 655.6 which was the highest up until it showed signs of sluffing on Dec. 10th At the first signs of sluffing tile drains were put in, in addition to the drains already in the fill. These drains were put in at right angles to the center line of the Dam from the downstream toe running back into the fill as far as it was possible to place them. These seemed to dry up the seapage considerably and stopped the sluffing. Additional dirt was placed on the downstream toe to carry out the toe farther in the place where the sluffing had been the worst. The Embankment from the Power house to Sta. 22 was wet on the downstream toe and in the worst places additional tile were placed. Since this time the Embankment has been in about the same condition up until we started raising the head for operating purposes. As the head comes up the fill of course becomes more wet and as this has occured additional tile have been placed which seem to help but do not entirely dry up the downstream toe. There is about 8" of frost on the downstream side of the fill and since the downstream slope is greater than the natural slope of the sand when wet the material underneath the frost takes the flatter slope and leaves the frost standing up and hence leaves a void between the frost and the wet dirt. This void extends back for a distance of 6 or 8 ft. in places we have picked out.

On the South side of he Power House the downstream toe shows no signs of being wet to date.

It has been the intention to place clay on the upstream face of the Dam on the section North of the P.H. after the other work is completed. I do not believe it is going to be feasible to place this clay without pulling the pond and besides we are going to need all the dirt in the present borrow pits to finish the Dam proper. It would seem to me that some method could better be used where the work could be done on the downstream side and I believe we should consider some such method. I am not sure that the tile drains are going to handle the situation at the time of the spring flood and at which time the frost will be coming out of the downstream toe.

C.E.Bottum.

WOLVERINE POWER COMPANY Ann Arbor, Michigan

Edenville Embankment and General Feb. 2, 1925

H. K. Holland C. E. Bottum, Res. Engr.

Your letters of January 30th and 31st received. We note in your description of the seepage in the East central section of the Edenville embankment, that as the head increases the seepage and sluffing increases. You say that you are not sure that the tile drains will take care of the sluffing, especially when the frost starts going out of the ground and out of the downstream side of the embankment. It has been our experience that when the drains are carefully laid surrounded with gravel and laid for a distance of 10 to 15 feet inside of the line of the downstream tow of the embankment that this has dried up the moisture above the tile drains in the embankments 30 to 50 feet high. But where the drains have not been carefully laid it has been hard to tell any difference from the amount of sluffing, and to get drains properly laid it is necessary for someone to inspect each line of tile as they are being placed, as when excavating in wet sand the sand runs in so fast that the workmen will lay the tile in almost any position so as to say they have laid them, and then cover the tile up. We have found that in sand embankments such as the one at Edenville, it is necessary to lay the tile on a board to lay them anywhere near straight and to keep them straight, and that the tile must be laid as near the bottom of the sand on top of the impervious layer as possible. The tile should then be covered with coarse gravel so as to act as a screeen to keep the tile from being filled with sand. The tile should be carried 10 to 15 feet downstream of the toe and burried to at least 12" deep with fill if they are above the level of the natural ground. Where tile are carefully laid and inspected we have had excellent results. Where they have been poorly laid and not inspected we have uniformly poor results. We do not know of any method that will work better or be more satisfactory than tile in the downstream toe, nor do we know of a better solution to seal the upstream slope of the embankment than by the use of a fine silty material such as we would have with clay. If you have any suggestions we certainly would be glad to discuss them, as we are anxious to get this work done and completed as quickly as possible and to have the seepage taken care of in the most satisfactory manner.

~~We note at Smallwood that the ice is thawed in the hydraucone and that you have cut around the edges before operating the turbine. We believe it will be necessary to remove all of the large cakes of ice from the hydraucone before start-~~

HOLLAND, ACKERMAN & HOLLAND
Consulting Engineers

Inter-Office Correspondence

106 E. Liberty St., Ann Arbor
53 W. Jackson St., Chicago

Job WOLVERINE POWER COMPANY Place Midland, Michigan

Subject Edenville Embankment Date Feb. 5, 1925

From C. E. Bottum To H. K. Holland

It was not my intention, in my letter of January 30th, to criticise the methods we are using to prevent sluffing of the Edenville embankment. I believe thoroughly in the tile drains and know they have worked in the past and do not feel that there is any better methos is, as you say, they are put in properly.

As for the inspection: Both Mr. Crew and myself are giving as much time to the Edenville embankment as we can take from the other parts of the work, and the men have been shown the proper method of laying the tile and in most cases are getting the tile in pretty good. However, since it has been impossible for either Mr. Crew or myself to be on this part of the work at all times, the tile were being laid, and hence not able to inspect every tile, there have been some which were not laid deep enough and perhaps did not have enough gravel around them as you suggested. These in some cases have plugged up and we have had them relaid. The downstream toe is very wet yet in places up to about elevation 642. I am enclosing a sketch of a cross section at about Sta. 10 where we put in a drain yesterday which shows the approximate line of saturation.

We are laying the tile in between the lines of original tile so that we have a row of tile now about every 8 feet. Most of them are running pretty well. The bank is so wet however that they found it necessary to drive sheeting today and five men worked all day getting one row of tile in.

What I meant in my letter of January 30th was, that if the tile alone did not dry up the embankment sufficiently, it might be necessary to place either rock on the downstream toe which in my opinion would hold in the dirt to a large extent and still give drainage, or else extend the downstream toe so as to further reduce the velocity. I wished to picture the conditions to you so that you would be considering some auxilliary method if you thought necessary.

As to the clay on the upstream face: This of course would seal the embankment to a large extent but the question in my mind is how to get the clay where we want it and where to get it from. We could probably buy more land on the East

Exhibit 3-5

end of the dam and haul it over the spillway with the dinkies. But, would dumping off the top of the fill place it where we need it most ? We might place it with barges or perhaps we could rig up a cable and car outfit, a long boom clam shell or some other way. There are a good many ways possible, but most, it seems, would be so expensive that I thought perhaps it would be cheaper to work from the downstream side. I believe we should get some plan under way for the clay if this is to be put on at once, and hence my deep concern. If Mr. Bick and you meet on the job next week it will give us an opportunity to decide on this.

C. E. BOTTUM.

WOLVERINE POWER COMPANY Ann Arbor, Michigan
General Feb. 9, 1925
H. K. Holland C. E. Bottum, Res. Engr.

After talking with you this morning I called Mr. Bick, and he said he would be unable to go North this week because of directors' meeting on Wednesday, but was planning on coming up next Sunday and being there the first part of the week. Rather than come up the last of this week and return again the first of next, I will wait so as to be there while Mr. Bick is there. He tells me that Trapp and Thompson will both be through within the next week or ten days. From the photographs it looks as if Trapp's embankment was not out to full width on the upstream side, about the center of the embankment.

Confirming our conversation this morning. We will compute the Secord quantities in the office and need the original cross section notes, the elevation of the top of the hard-pan, and the elevation at finished grade. Also, whatever changes have been made in the field that will affect the concrete quantities.

Your suggestion of placing the drain tile at the elevation of the berm in the sand embankment North of the power house at Edenville before the water is raised higher is a good suggestion. These should be placed 15 ft. centers and carried into the embankment at least 10 feet, beginning at the power

Exhibit 3-6

WOLVERINE POWER COMPANY

Ann Arbor, Michigan

Feb. 11, 1925

H. K. Holland

C. E. Bottum, Res. Engr.

From the elevations of the ponds phoned to me this morning it looks as if the Smallwood pond would start flowing over the concrete crest sometime during the day today. As soon as the water starts over the crest then the sluice gate should be closed and the units started up and dried out.

The concrete covers for the temporary openings in the Tobacco spillway should be closed now so that any sand that is washed from the embankment will deposit on top of the doors which will help to make them tighter. We estimate that the closing of these doors will not raise the water surface of the Tobacco River enough to affect the elevation of the water at the Little Cedar River, and we believe it to be advantageous to close these openings before the river rises to a higher stage. The closing of these doors will also prevent the scouring of the sand at the toes of the embankment where they extend into the river channel at the upstream ends.

An inspection should be made each day of the Smallwood embankment on both sides of the river. Also look for any seepage that might come through or under the concrete work. An inspection should also be made of the Sanford embankments.

H.K.H.

P.S.

Enclosed find copy of bill from A-C. Mfg. Co. Please note if this material has been received and if so, O.K. and return.

H.K.H.

Exhibit 3-7

WOLVERINE POWER COMPANY — Ann Arbor, Michigan

General — March 6, 1925

H. K. Holland — C. E. Bottum, Res. Engr.

Confirming our conversation this afternoon. We are going to place clay on the Edenville embankment on the section North of the power house to the high bank - the clay to be placed approximately 2 feet thick from the top of the embankment to the water line. This should be placed as soon as possible.

Confirming our conversation of Monday, March 2nd, The Sanford pond can be raised 18", which is to elevation 623, and the Tobacco pond was to be held at an elevation about 1 foot below the road at the Little Cedar bridge. You said today that the Tobacco pond was rising, and I so informed Mr. Wixom who was here, and he will take active steps to have this highway abandoned. However, this will take 10 days as legal notices have to be posted. Meanwhile we should keep the road open if possible.

Mr. Ellis will be at Smallwood Monday to assist in phasing out the generator and placing it on the line.

Mr. Bick ~~Left~~ let the Hunter bridge erection and concrete floor to the Benjamin Douglas Company for a lump sum of $1800; ~~we~~ Bick to furnish all the extra rivets that we now have on the work.

H.K.H.

Exhibit 3-8

WOLVERINE POWER COMPANY

Ann Arbor, Michigan

General

April 16, 1925

C. E. Bottum

H. K. Holland

I acknowledge receipt of your letter of April 15th confirming Mrs. Hollnd's phone call, and note the failure of French Landing dam.

We are following the directions in your letter and will take every possible precaution at allthree dams. The Smallwood embankment looks pretty good. The dirt in the bottom of the air shaft is wet however, and one would sink clear to the hips in it, it being necessary to use planks in the bottom of the shaft to walk on. We will put stone in the bottom of this shaft as we did on the Tobacco.

The Edenville embankment on the East end is settling and cracking as you expected and the tile do not run very freely. However the downstream toe is dry and we are watching every movement. We find a settlement under the fish chute and are repairing before the water reaches a higher stage. The frost is not all out of this section yet.

The fill North of the power house looks in good condition and Trapp is coming fine with the clay which I believe will be a big help to this section.

The Vogel embankment East of the spillway has to be constantly repaired though it has dried up considerably. The fill West of the spillway in the old Tobacco channel as far has caused us no trouble and looks good. The springs on the West end are about the same.

Exhibit 3-9

WOLVERINE POWER COMPANY

Secord Embankment

H. K. Holland

Ann Arbor, Michigan

May 13, 1925

C. E. Bottum, Res. Engr.

Enclosed find your expense check, No. 5980, and check No. 5978 to H. Crampton. Enclosed also find copy of letter received from the Westinghouse Company which gives the delivery of the disconnecting switches for Secord.

After seeing the Secord embankment on May 4th I have been a little bit concerned about the method of placing the earth on the East side, that being placed by Mr. Burton. That fill is getting in very much the same condition that the East side of Edenville was, especially due to dumping the wheel scrapers off of the high embankment so that the earth gets no packing and the carrying up of a very small portion of the embankment 10 to 15 feet higher than the remainder of the embankment which will cause very bad settling, and in this clay material will produce cracks that will make this embankment of a considerable hazard. To correct this the downstream portion of the embankment should be carried up so that the upstream section slopes to the downstream section so that the embankment can be placed in thin layers of not to exceed 9" or 10", and these layers packed by teaming over them when the next layer is placed. I see no reason why this cannot be done for all of this embankment excepting the toe that extends upstream of the concrete work, which can not be placed until after the temporary

Exhibit 3-10

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Attachment D-3: Construction Specifications

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Office Copy

O. Vogel

AGREEMENT
and
SPECIFICATIONS
for the
C O N S T R U C T I O N
of the
EARTH EMBANKMENTS - CENTRAL SECTION
of the
EDENVILLE PLANT
for
J. N. BICK, GEN'L CONTRACTOR

HOLLAND, ACKERMAN & HOLLAND
CONSULTING ENGINEERS
ANN ARBOR -- CHICAGO
1924

6 inches. The spoil from such stripping shall be placed closely upstream of the site of the work or other place approved by the Engineer. Stripping of the borrow pits where required is included under this classification.

Under Class "B" Stripping it is intended to strip and remove vegetable containing soil and muck where occurring, from the site of the earth fill or other places, and when and where required by the Engineer to excavate cut-off trenches within the site of the earth fills and at other places where the spoil for the same is unsuitable or otherwise unavailable for use in the earth fills. The spoil from Class "B" Stripping shall be placed in the earth fill when in the opinion of the Engineer the materials thereof are suitable therefor; otherwise, the materials shall be spoiled closely upstream of the earth fill, or in other places approved by the Engineer.

TILE DRAINS - Item 3

ART. 5. WORK TO BE DONE: The Contractor shall place tile drains in the foundations of the earth fill and other places as shown on the plans or directed by the Engineer.

Drainage will be provided by tile drains located [illegible] the downstream half of the earth fill as shown. After [illegible] site of the earth fill has been prepared as hereinbefore [illegible]vided under "STRIPPING" and where necessary sufficient earth fill has been made to substantially level the site to the original ground surface, trenches about 18 inches wide and 12 inches deep shall be made to receive the tile drains. Spoil from these trenches shall be spread on the earth fill. The tile shall then be carefully laid in these trenches on a layer of gravel at a grade of not less than 1 foot in 100 feet, and with an opening of about 3/4 of an inch left between the spigot end of one tile and the seat of the socket of the next adjacent tile. The trench shall then be back filled with coarse gravel to the level of the site. The tile shall terminate at the upstream edge of a shallow ditch which will be constructed under these Specifications about 10 feet downstream of the toe of the earth fill.

The compensation under this item shall include furnishing and placing coarse gravel around the tile drains as above described.

The drainage tile used under this item will be delivered to the Contractor by the Principal, f.o.b. cars Sanford, Michigan. The Contractor shall receive the tile, unload, transport and lay the same as herein provided. The tile will be a standard quality vitrified sewer crock with bell and spigot ends. Only whole tile without cracks will be used in the work.

Should any obstruction be found in any drain it shall be promptly removed, and should any damaged drain tile be discovered before the completion of the work under these Specifications, they shall be removed and replaced at the expense of the Contractor with new pieces, in a manner satisfactory to the Engineer.

Particular care must be taken in making the earth fill as hereinafter provided, that the drain tile will not be damaged or broken.

The length of drain tile for compensation shall be the lineal feet measured along the center line of the tile drain after laying.

EARTH FILL - Item 4

ART. 6 WORK TO BE DONE: The Contractor shall construct earth fills in the locations named in the Agreement and to the lines and grades shown on the plans or directed by the Engineer.

After the site of the earth fill has been prepared satisfactory to the Engineer as hereinbefore provided under "STRIPPING" and immediately before beginning the earth fill, if required by the Engineer suitable connection of the earth fill to the natural ground shall be made by plowing the surface of the site upstream of the center line of the earth fill to a depth of at least 5 inches in furrows, one to every 3 feet in width of the same, the furrows to run parallel to the center line of the earth fill.

The materials of the earth fill shall be free from vegetable and other perishable matter and in the upstream part of the earth fill from all stones measuring 6 inches or more in their greatest dimension. Upstream of the center line of the earth fill the materials shall be selected of clay, gravel and loam mixed so as to make as impervious an earth fill as possible, while that of the downstream side of the earth fill shall be free from clay so far as the same may be obtained in the borrow pits and be pervious. The earth of the different parts of the earth fill shall be taken from the borrow pits over such areas as directed by the Engineer to obtain the best available materials for the earth fills.

All materials of the earth fills shall be spread and harrowed in horizontal layers and compacted before any layer exceeds 9 inches in thickness. Compacting shall be done with a grooved roller weighing not less than one ton per lineal foot of roller tread, or such equivalent of the roller as will meet the approval of the Engineer; the intention being to insure a thorough compacting of the whole earth fill. If the materials as deposited in the earth fill cohere such as to form lumps not easily compacted, such materials after spreading shall be thoroughly disked in addition to harrowing to insure the pulverization of such lumps. Unless the materials are sufficiently moist when spread, each layer shall

3

[illegible] & HOLLAND
CONSULTING ENGINEERS
[illegible] ST.
ANN ARBOR • MICHIGAN

[illegible] McKinney

[illegible]

EARTH [illegible] EAST END

[illegible]

J. N. [illegible], CONTRACTOR

HOLLAND, ACKERMAN & HOLLAND
CONSULTING ENGINEERS
ANN ARBOR — CHICAGO

192[illegible]

6 inches. The spoil from such stripping shall be placed closely upstream of the site of the work or other place approved by the Engineer. Stripping of the borrow pits where required is included under this classification.

Under Class "B" Stripping it is intended to strip and remove vegetable containing soil and muck where occurring, from the site of the earth fill or other places, and when and where required by the Engineer to excavate cut-off trenches within the site of the earth fills and at other places where the spoil for the same is unsuitable or otherwise unavailable for use in the earth fills. The spoil from Class "B" Stripping shall be placed in the earth fill when in the opinion of the Engineer the materials thereof are suitable therefor; otherwise, the materials shall be spoiled closely upstream of the earth fill, or in other places approved by the Engineer.

TILE DRAINS - Item 3

ART. 5. WORK TO BE DONE: The Contractor shall place tile drains in the foundations of the earth fill and other places as shown on the plans or directed by the Engineer.

Drainage will be provided by tile drains located in the downstream half of the earth fill as shown. After the site of the earth fill has been prepared as hereinbefore provided under "STRIPPING" and where necessary sufficient earth fill has been made to substantially level the site to the original ground surface, trenches about 18 inches wide and 12 inches deep shall be made to receive the tile drains. Spoil from these trenches shall be spread on the earth fill. The tile shall then be carefully laid in these trenches on a layer of gravel at a grade of not less than 1 foot in 100 feet, and with an opening of about 3/4 of an inch left between the spigot end of one tile and the seat of the socket of the next adjacent tile. The trench shall then be back filled with coarse gravel to the level of the site. The tile shall terminate at the upstream edge of a shallow ditch which will be constructed under these Specifications about 10 feet downstream of the toe of the earth fill.

The compensation under this item shall include furnishing and placing coarse gravel around the tile drains as above described.

The drainage tile used under this item will be delivered to the Contractor by the Principal, f.o.b. cars Sanford, Michigan. The Contractor shall receive the tile, unload, transport and lay the same as herein provided. The tile will be a standard quality vitrified sewer crock with bell and spigot ends. Only whole tile without cracks will be used in the work.

Should any obstruction be found in any drain it shall be promptly removed, and should any damaged drain tile be discovered before the completion of the work under these Specifications, they shall be removed and replaced at the expense of the Contractor with new pieces, in a manner satisfactory to the Engineer.

Particular care must be taken in making the earth fill as hereinafter provided, that the drain tile will not be damaged or broken.

The length of drain tile for compensation shall be the lineal feet measured along the center line of the tile drain after laying.

EARTH FILL - Item 4

ART. 6. WORK TO BE DONE: The Contractor shall construct earth fills in the locations named in the Agreement and to the lines and grades shown on the plans or directed by the Engineer.

After the site of the earth fill has been prepared satisfactory to the Engineer as hereinbefore provided under "STRIPPING" and immediately before beginning the earth fill, if required by the Engineer suitable connection of the earth fill to the natural ground shall be made by plowing the surface of the site upstream of the center line of the earth fill to a depth of at least 5 inches in furrows, one to every 3 feet in width of the same, the furrows to run parallel to the center line of the earth fill.

The materials of the earth fill shall be free from vegetable and other perishable matter and in the upstream part of the earth fill from all stones measuring 6 inches or more in their greatest dimension. Upstream of the center line of the earth fill the materials shall be selected of clay, gravel and loam mixed so as to make as impervious an earth fill as possible, while that of the downstream side of the earth fill shall be free from clay so far as the same may be obtained in the borrow pits and be pervious. The earth of the different parts of the earth fill shall be taken from the borrow pits over such areas as directed by the Engineer to obtain the best available materials for the earth fills.

All materials of the earth fills shall be spread and harrowed in horizontal layers and compacted before any layer exceeds 9 inches in thickness. Compacting shall be done with a grooved roller weighing not less than one ton per lineal foot of roller tread, or such equivalent of the roller as will meet the ap_roval of the Engineer; the intention being to insure a thorough compacting of the whole earth fill. If the materials as deposited in the earth fill cohere such as to form lumps not easily compacted, such materials after spreading shall be thoroughly disked in addition to harrowing to insure the pulverization of such lumps. Unless the materials are sufficiently moist when spread, each layer

Attachment D-4: Selected Historical Embankment Drawings

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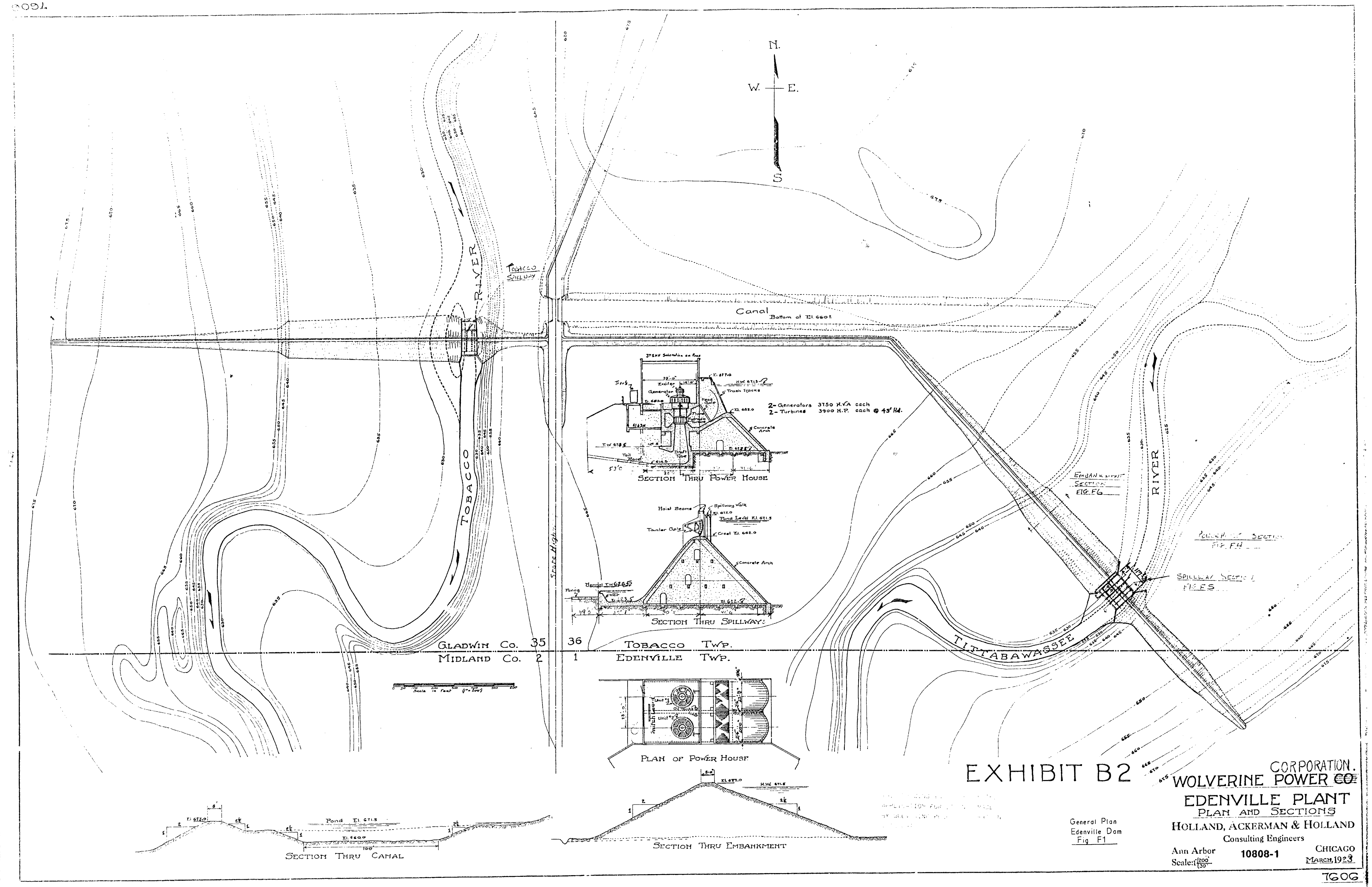
N.
W.
E.
S.
Tobacco Spillway
Tobacco River
Canal
Bottom of El. 660±
Section Thru Power House
2 - Generators 3750 K.V.A. each
2 - Turbines 3900 H.P. each @ 45' Hd.
Section Thru Spillway
Spillway Walk
Tainter Gate
Hoist Beams
Concrete Arch
State Highway
Gladwin Co. 35
36 Tobacco Twp.
Midland Co. 2
1 Edenville Twp.
Tittabawassee River
Embankment Section Fig. F6
Powerhouse Section Fig. F4
Spillway Section Fig. F5
Plan of Power House
Unit #1
Unit #2
Pond El. 671.5
Section Thru Canal
Section Thru Embankment
EXHIBIT B2
WOLVERINE POWER CORPORATION.
EDENVILLE PLANT
PLAN AND SECTIONS
HOLLAND, ACKERMAN & HOLLAND
Consulting Engineers
Ann Arbor
CHICAGO
Scale: 1"=200' / 1"=30'
10808-1
March, 1923
General Plan
Edenville Dam
Fig. F1
7606

Appendix E: Forensic Team Field and Laboratory Investigations

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E-1 Introduction

In September 2020, members of the Independent Forensic Team (IFT) conducted an initial site visit to the Edenville, Sanford, Smallwood, and Secord Dams. Following the initial site visit, members of the IFT completed a field investigation of the Edenville Dam left embankment breach in December 2020. The purpose of the investigation was to obtain additional information on the embankment cross-section geometry and foundation composition of the Edenville left embankment based on the left and right breach faces. The activities included logging the embankment cross sections at the left and right breach faces and collecting samples. The soils were classified by visual-manual procedures in general accordance with the American Society for Testing and Materials (ASTM), Standard D2488 (ASTM 2018a). In 2021, laboratory testing was performed on the samples obtained during the field investigation to evaluate index and engineering properties. The soil samples were classified based on laboratory index properties in general accordance with ASTM D2487 (2020b).

Figure E-1 shows the post-failure breach of the Edenville left embankment and depicts the general conditions at the time of the field investigation.

Figure E-1: View of Edenville Left Embankment, Post Failure (photo courtesy of EGLE)

The downstream embankment section of the left breach remnant, from the embankment centerline to the downstream toe, was generally visible, with loose slough present at the base of the remnant embankment. Slough and debris covered the upstream embankment section. A clay tile drain pipe was exposed at the downstream toe. The left abutment (natural foundation) appeared to be exposed near the upstream toe. Waste fill, which included lumber debris, appeared to have been placed upstream of the embankment section on the abutment/foundation contact. Remnants of an old intake and waterline structure, which originally connected to a downstream pump house structure, were exposed near the upstream toe. An aerial image of the left breach remnant is shown in Figure E-2.

Figure E-2: Aerial View of Left Breach Remnant (photo courtesy of EGLE)

The downstream embankment section of the right breach remnant, from the centerline to the downstream toe, was generally visible and intact, and some loose slough could be seen on the face and at the base of the embankment. Upstream of the centerline, the embankment had slumped/deformed toward the upstream side during the May 2020 event due to the failure of the upstream sheet pile cutoff wall. The upstream toe was submerged and not visible. An aspen pole, likely a temporary utility pole from the original construction, was embedded and protruding from the face of the embankment. An 8-inch corrugated high-density polyethylene (HDPE) pipe, likely an extension from the original clay tile drain pipes, was exposed near the downstream toe. An aerial view of the right breach remnant is shown in Figure E-3.

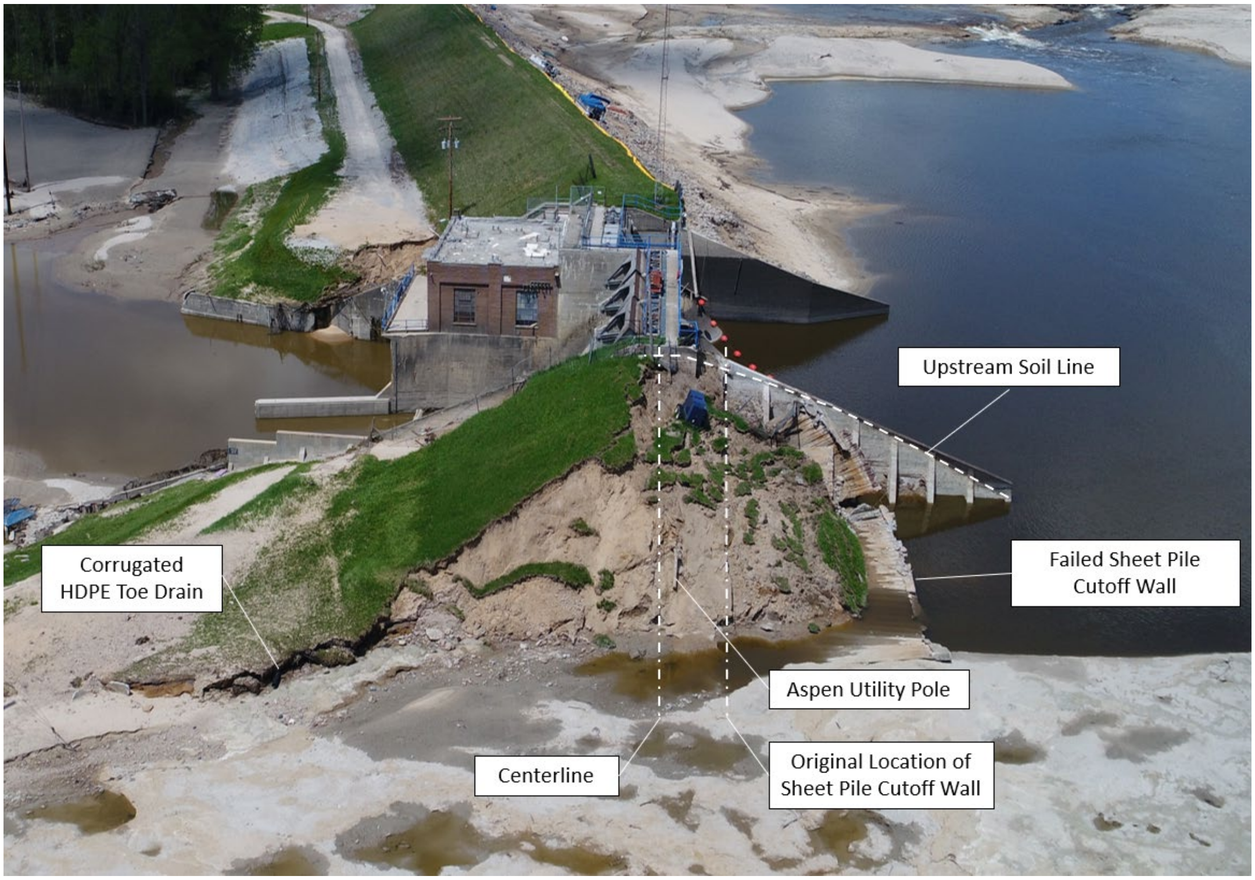

Figure E-3: Aerial View of Right Breach Remnant (photo courtesy of EGLE)

E-2 Methodology

E-2.1 Mapping

Fisher Contracting Company was contracted through the Michigan Department of Environment, Great Lakes, and Energy (EGLE) to assist the IFT in the field investigation. A Hyundai HX 480L Excavator was used to remove loose fill and other debris to expose the face of each breach remnant, as shown in Figure E-4 and Figure E-5.

Figure E-4: Excavator Removing Loose Fill and Debris from the Face of the Left Breach Remnant

Figure E-5: Excavator Removing Loose Fill from the Face of the Right Breach Remnant, near the Crest

The exposed cross sections at the left and right breach face were logged through sketches, photographs, and field classification of materials. The material designations within the embankment section were further refined through the results of subsequent laboratory testing.

E-2.2 Sampling

A total of 23 grab samples (12 from the left breach face, 11 from the right breach face) were collected at discrete zones observed in the embankment and foundation materials for the purpose of index testing to support material characterization. A total of 11 bulk samples (six from the left breach face, five from the right breach face) were collected for the purpose of testing for engineering properties. In addition, a total of 12 thin-walled Shelby tube samples (five from the left breach face, seven from the right breach face) were obtained for index testing as further described below. These samples were not used for testing of engineering properties due to the potential for sample disturbance resulting from the non-standard method of sample collection, as discussed below. Survey shots were taken at Shelby tube and bulk sample locations and other pertinent locations.

E-2.3 In Situ Density

Three in situ sand cone density tests were performed within the left remnant, following ASTM D1556 (2016a). Figure E-6 shows a sand cone density test performed at the left embankment remnant.

Figure E-6: Sand Cone Density Test Performed at the Left Embankment Remnant

In situ densities were also estimated using the thin-walled tube samples. Sample collection consisted of pushing a thin-walled Shelby tube sampler of known volume into the embankment, using the excavator bucket; extracting the sampler by slowly pulling with an attached strap, using the excavator; removing any loose material; and measuring the intact sample length. The sample was then shipped to the laboratory where a total sample weight and moisture content was measured. Figure E-7 shows a Shelby tube sample being taken.

Figure E-7: Shelby Tube Collection from Left Breach Remnant

E-3 Field Investigation Results

A set of photo exhibits documenting the field investigation is attached to this appendix as Attachment E-1. Descriptions of the exhibits are as follows:

E-3.1 Left Breach Remnant

- Exhibit E-1 shows a view of the left breach remnant during the September 2020 site visit. The glacial till is visible at the base of the embankment and across the new river channel. The glacial till generally consisted of a light gray sandy lean clay to a silty sand with gravel and occasional cobbles (Unified Soil Classification System [USCS] classification CL, SM).
- Exhibit E-2 shows the embankment fill variability and layering at the crest of the left breach remnant. The source of the hole visible near the crest is uncertain and may be a result of previous sampling by others or animal activity. The cause of the cracking visible near the embankment crest is also uncertain. It is postulated that the cracking occurred during or after the breach, as there were no observations of cracking exposed at the crest in earlier inspections. Some layers exhibited higher moisture content and appeared wetter than others.
- Exhibit E-3 shows a clay tile drain located at the downstream toe of the left breach remnant.
- Exhibit E-4 shows the excavator clearing a bench in the downstream slope of the left breach remnant for sampling. Slough is shown at the base of the embankment. Slough and debris are shown covering the upstream embankment section. A 3-foot-thick gray native sand layer with organic matter is visible at the base of the embankment, and generally consisted of a dense sand with silt and some clay with organic matter (USCS classification SP-SM, SM).

- Exhibit E-5 shows a view of the entire left breach remnant, with approximate material boundaries identified by the IFT.
- The upper 15 feet of the embankment, near the crest, consists of a silty sand fill, and is shown on Exhibits E-6 to E-7. The fill was found to be quite heterogeneous, with visible pockets of silts and sands as well as some clay nodules. In general, the fill consisted of light brown, moist, loose, fine to medium silty sand fill with occasional clay nodules (USCS classification SM). There was no apparent difference in the upstream versus downstream fill.
- The upstream section of the lower part of the embankment consists of a clayey fill underlain by a native clayey foundation material under the upstream slope, as shown in Exhibits E-8 to E-9. An approximately 3-foot-long layer of charcoal was observed, interpreted to delineate the fill (above) from the native abutment (below). The clayey fill generally consisted of brown and gray, moist, stiff, sandy clay with occasional gravel (USCS classification CL). The native clay abutment foundation generally consisted of gray, moist, stiff, lean clay (USCS classification CL). The clayey fill extended slightly downstream of the centerline, as shown in Exhibit E-5.
- The downstream section of the lower part of the embankment consists of a clayey sand fill, near mid-embankment height, which transitions to a clean sand fill with clay lenses and nodules near the downstream toe and base of the embankment, as shown in Exhibits E-10 to E-12. The clean sand fill generally consisted of light brown, moist, fine to medium sand with little silt (USCS classification SP-SM) and appeared to be loose to compact. This material extends upstream of the centerline, below the clayey fill in the lower portion of the embankment, as shown on Exhibit E-5. Exhibits E-13 to E-15 show the contact between the clayey fill and clean sand fill about 20 to 25 feet below the crest.
- A native sand layer was observed at the base of the embankment, which appeared to extend from upstream to downstream. The native sand foundation generally consisted of dark brown and gray, wet, medium dense to dense, sand with little silt (USCS classification SP-SM) and is shown in Exhibit E-16.

E-3.2 Right Breach Remnant

- Exhibit E-17 shows the right breach remnant, observed during the September 2020 site visit.
- Exhibit E-18 shows a view of the mapped right breach remnant with approximate material boundaries identified by the IFT. The right breach face consisted primarily of clay fill with a 5- to 8-foot-thick layer of sand fill on the outer slopes and crest. The clayey fill generally consisted of brown and gray, moist, stiff, sandy clay with occasional gravel (USCS classification CL) and is shown in Exhibit E-19. The silty sand fill generally consisted of a light brown, moist, loose, silty sand (USCS classification SM).
- Exhibits E-20 to E-23 show the sloping contact between the clayey fill and the overlying silty sand fill, which was observed to be about 5 feet thick. At the crest, the contact was observed to be generally horizontal, about 8 feet below the crest.
- Exhibit E-23 shows a horizontal seam of charcoal under the lower portion of the downstream slope, marking the foundation contact. Native sand foundation was observed below the charcoal seam and generally consisted of a light brown, moist, medium dense, silty sand (USCS classification SM).

- Near the centerline, the clayey fill extended to the glacial till hard pan foundation (cutting off the sand foundation layer), about 5 feet deeper than the interpreted foundation contact near the downstream toe. A pocket of metal and other construction debris was observed above the glacial till, marking the foundation contact. The glacial till generally consisted of a gray sandy lean clay to a silty sand with gravel (Unified Soil Classification System [USCS] classification CL, SM).

E-4 Laboratory Investigation Results

Following the completion of the field investigation, the samples were sent to TerraSense LLC. in New Jersey to perform index and engineering property tests.

Based on the findings of the field investigation, it is apparent that the internal cross section was not consistent along the length of the Edenville left embankment. The IFT reviewed the logs of the borings completed in 2005 and 2010 near the breach location, which included layers of silty sand and silt, clay, native sand, and glacial till (hardpan). The IFT found the left breach remnant to be similar to the borings. The boring logs did not appear consistent with the mapped right breach remnant, which was primarily clay fill. Therefore, the IFT focused the laboratory investigation on the samples from the left breach remnant. The locations of samples obtained from the left breach remnant are shown in Figure E-8.

Table E-1 summarizes the testing performed on samples from the Edenville left embankment left breach face.

Table E-1: Soil Testing Program

Soil Testing	Number
Moisture Content – ASTM D2216 (2019a)	17
Atterberg Limits – ASTM D4318 (2018b)	8
Grain Size – ASTM D6913 (2017)	13
Specific Gravity – ASTM D854 (2016c)	1
Hydrometer – ASTM D7928 (2021)	7
Maximum and Minimum Index Density – ASTM D4253 (2019b) & D4254 (2016b)	1
Triaxial Shear – Isotropically Consolidated, Undrained with Pore Pressure Measurements (CIU') – ASTM D4767 (2020c)	3 (Points)
Triaxial Shear – Consolidated, Drained (CID) – ASTM D7181 (2020a)	3 (Points)

E-4.1 Grab Sample Index Test Results

Table E-2 is a summary of index property results from the grab samples.

Table E-2: Summary of Index Property Results from Grab Samples

Sample No.	Moisture Content (%)	Liquid Limit	Plasticity Index	Sieve Minus No. 200 (%)	Hydrometer Minus 2 μm (%)	USCS Classification[1]	Notes
G-1	20.2	25	8	83.3	16	CL	-
G-2	13.7	NT[2]	NT	28.8	NT	SM	-
G-3	9.4	NT	NT	16.6	NT	SM	Discrete sample of brown sand within silty sand fill

Sample No.	Moisture Content (%)	Liquid Limit	Plasticity Index	Sieve Minus No. 200 (%)	Hydrometer Minus 2 μm (%)	USCS Classification[1]	Notes
G-4	13.3	16	2	31.1	9	SM	Composite sample of silty sand fill
G-5	15.8	19	7	55.8	NT	CL-ML	Sample above charcoal – possibly fill
G-6	24.2	35	21	94.5	41	CL	Sample below charcoal – possibly native
G-7	12.1	21	10	39	13	SC	-
G-8	16.1	22	11	61	23	CL	-
G-9	7.1	NT	NT	5.3	NT	SP-SM	-
G-10	7.7	NT	NT	8.6	NT	SP-SM	-
G-11	18.8	27	16	63	27	CL	Discrete sample of clay seam within clean sand fill
G-12	20.0		NP	11.3	4	SP-SM	-

[1] USCS (Unified Soil Classification System) symbol is based on visual observation and sieve and Atterberg limits reported.
[2] NT=Not Tested.

The grab sample gradation results are included in Figure E-9.

E-4.2 Moisture-Density Results

Table E-3 is a summary of the moisture content and density results from Shelby tubes and sand cone density tests.

Table E-3: Summary of Moisture Content – Density Results from Sand Cone Density Tests and Shelby Tube Samples

Sample No. [1]	Embankment Zone[2]	Moisture Content (%)	Dry Unit Weight (pcf) [3,4]	Total Unit Weight (pcf)
SC-1	Clean Sand Fill	8.3	110.2	119.3
SC-2	Silty Sand Fill	10.6	125.1	138.4
SC-3	Silty Sand Fill	11.7	109.4	122.2
S-1	Silty Sand Fill	-	-	123.6
S-2	Silty/Clayey Sand Fill	-	-	112.3
S-3	Silty/Clayey Sand Fill	-	-	124.2
S-4	Silty Sand Fill	5.4	95.6[3]	100.8
S-5	Clay Lens in Sand Fill	-	-	123.8

[1] SC=Sand Cone. Dry unit weights were estimated in general accordance with ASTM D1556 (2016a).
[2] Embankment zoning designation is based on the visual mapping shown in Exhibit E-5 and Figure E-8.
[3] The moisture content values for Shelby tube samples S-1, S-2, S-3, and S-5 were not obtained; therefore, dry unit weights could not be calculated.
[4] Dry unit weight was not measured directly. It was estimated by measuring the moist mass of the sample collected within the Shelby tube based on the estimated sample volume and subtracting the moisture content.

E-4.3 Engineering Property Results

A composite sample was developed from the bulk samples obtained in the clean sand fill (Samples B-1, B-2, B-3, and B-4). The bulk samples were mixed together and any clay nodules that were retained on the No. 4 sieve were removed from the composite sample.

Table E-4 is a summary of the index properties and minimum and maximum density results.

Table E-4: Clean Sand Fill Composite Sample – Index Properties and Relative Density Results

D_{50} (mm)	Fines Content (%)	Cc	Cu	USCS Symbol	Minimum Dry Unit Weight (pcf)	Maximum Dry Unit Weight (pcf)	Specific Gravity
0.2	9.9	1.4	2.9	SP-SM	89.8	112.1	2.63

The composite sample gradation is shown in Figure E-9.

A series of three isotropically consolidated, undrained triaxial shear with pore pressure measurement (CIU’) tests were completed to evaluate the undrained stress-strain behavior of the clean sand fill at a loose, saturated state. The samples were reconstituted to a target relative density of 30 percent. The target relative density was selected to evaluate the behavior of the sand in a loose state. The CIU’ specimens were consolidated to effective consolidation stresses of 5, 15, and 30 pounds per square inch (psi). Table E-5 is a summary of the CIU’ test specimen parameters.

Table E-5: CIU' Test Specimen Summary

Point Name	Effective Consolidation Stress (psi)	Dry Density after Consolidation, γ_{dc} (pcf)	Relative Density (%)
A	5	94.9	27
B	15	99.1	47
C	30	96.3	34

% = percent
CIU’ = isotropically consolidated, undrained triaxial shear with pore pressure measurement
pcf = pounds per cubic foot
psi = pounds per square inch

The results of the CIU’ tests, which are presented in Figure E-10, showed contractive and brittle behavior of the clean sand fill at loose relative density. The left side of Figure E-10 shows stress-strain curves for the three specimens. All three curves show brittle, strain-weakening behavior (see the description in Section 4.1.3 of the main report for a discussion of brittle, strain-weakening behavior). The right side of Figure E-10 shows stress paths for the three specimens. All three specimens show peak undrained shear strengths at stress states well below the drained frictional envelope of 31 degrees, followed by large increases in pore water pressure and dramatic decreases in strength. The collapse of the specimens at failure was found to be so fast that the instrumentation (load cell, pore pressure transducer) could not keep up with the rapid deformation and had difficulty capturing all the data at increasing strains.

A series of consolidated drained triaxial strength (CID) tests (CID) were also completed on the composite sample. These specimens were also prepared at 30 percent relative density and consolidated to three different confining stresses of 5, 15, and 30 psi. Table E-6 is a summary of the CID test specimen parameters.

Table E-6: CID Test Specimen Summary

Point Name	Effective Consolidation Stress (psi)	Dry Density after Consolidation, γ_{dc} (pcf)	Relative Density (%)
A	5	95.1	28
B	15	98.8	46
C	30	98.7	45

The results of the CID tests are provided in Figure E-11. The stress-strain curves, on the left side of Figure E-11, do not exhibit brittle, strain-weakening behavior, but rather show ductile behavior, with little or no decrease in strength from the peak strength. The drained stress paths are shown on the right side of Figure E-11, with a linear increase in shear stress from the consolidated state to the failure line. The drained friction angle from these three tests is 31 degrees.

E-5 Key Takeaways

The key takeaways from the IFT's field and laboratory investigation are described below.

The embankment composition and internal zoning varied significantly from the left to the right end of the Edenville left embankment.

- The left breach remnant appeared to consist of three main fill groups: silty sand fill at the crest section, clayey fill at the upstream lower section, and sand fill (transitioning from clayey sand to clean sand) at the downstream lower section.
- The right breach remnant was found to consist mainly of clayey fill with a 5- to 8-foot-thick silty sand layer along the outer slopes and crest (e.g., a sand "cap").
- The left breach remnant was founded on medium dense native sand that extended from upstream to downstream.
- The sand foundation layer appeared to have been excavated within the center section of the right breach remnant, where fill was founded directly on glacial till.

The left breach embankment section appears to have some semblance of upstream to downstream zoning, with impervious fill on the upstream side and pervious fill on the downstream side, as stated in the original specifications (see Appendix B).

- There is a notable color difference between the upstream and downstream fill from about 20 feet below the crest of the dam to the base of the dam. The upstream fill appears to be darker than the downstream fill.
- The field classification and laboratory index properties identified clayey fill (USCS classification CL and SC) in the lower upstream section of the embankment. Clean sand fill with less than 10 percent silty fines was found near the downstream toe of the left embankment remnant.

The CIU' test results from the clean sand fill of the left breach remnant exhibited contractive, brittle, strain-weakening behavior. This behavior is characteristic of material susceptible to static liquefaction.

E-6 References

American Society for Testing and Materials (ASTM). 2016a (December). *D1556/D1556M-15e1, Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method.* ASTM International.

______. 2016b (December). *D4254-16, Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density.* ASTM International.

______. 2016c (December). *D854-14, Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer.* ASTM International.

______. 2017 (February). *D6913/D6913M-17, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.* ASTM International.

______. 2018a (March). *D2488-17e1, Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)*. ASTM International.

______. 2018b (April). *D4318-17e1, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.* ASTM International.

______. 2019a (March). *D2216-19, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.* ASTM International.

______. 2019b (December). *D4253-16e1, Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table.* ASTM International.

______. 2020a (February). *D7181-20, Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils.* ASTM International.

______. 2020b (April). *D2487-17, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).* ASTM International.

______. 2020c (April). *D4767-11, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.* ASTM International.

______. 2021 (June). *D7928-21e1, Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis.* ASTM International.

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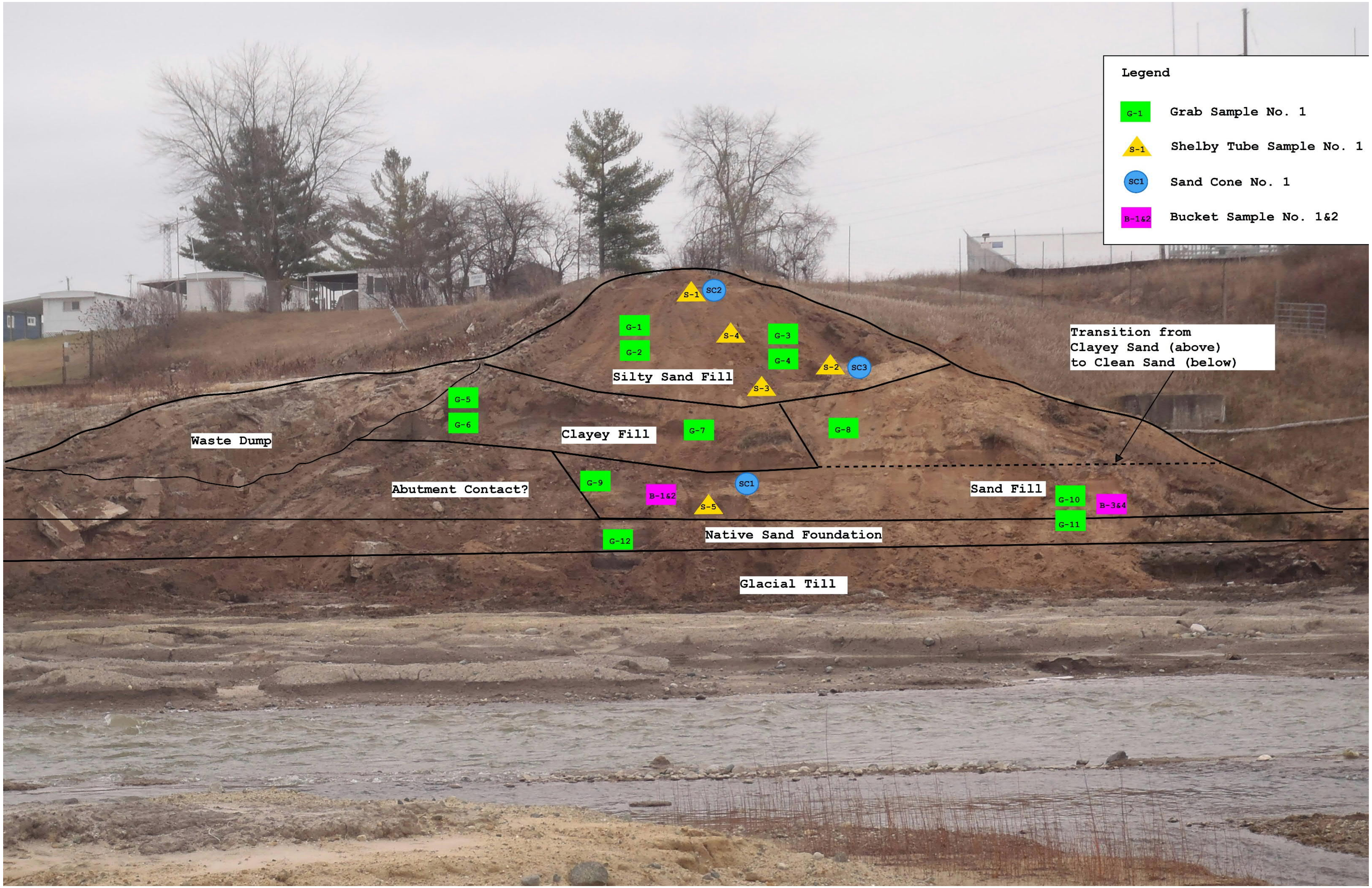

Figure E-8: Left Breach Remnant Sample Location

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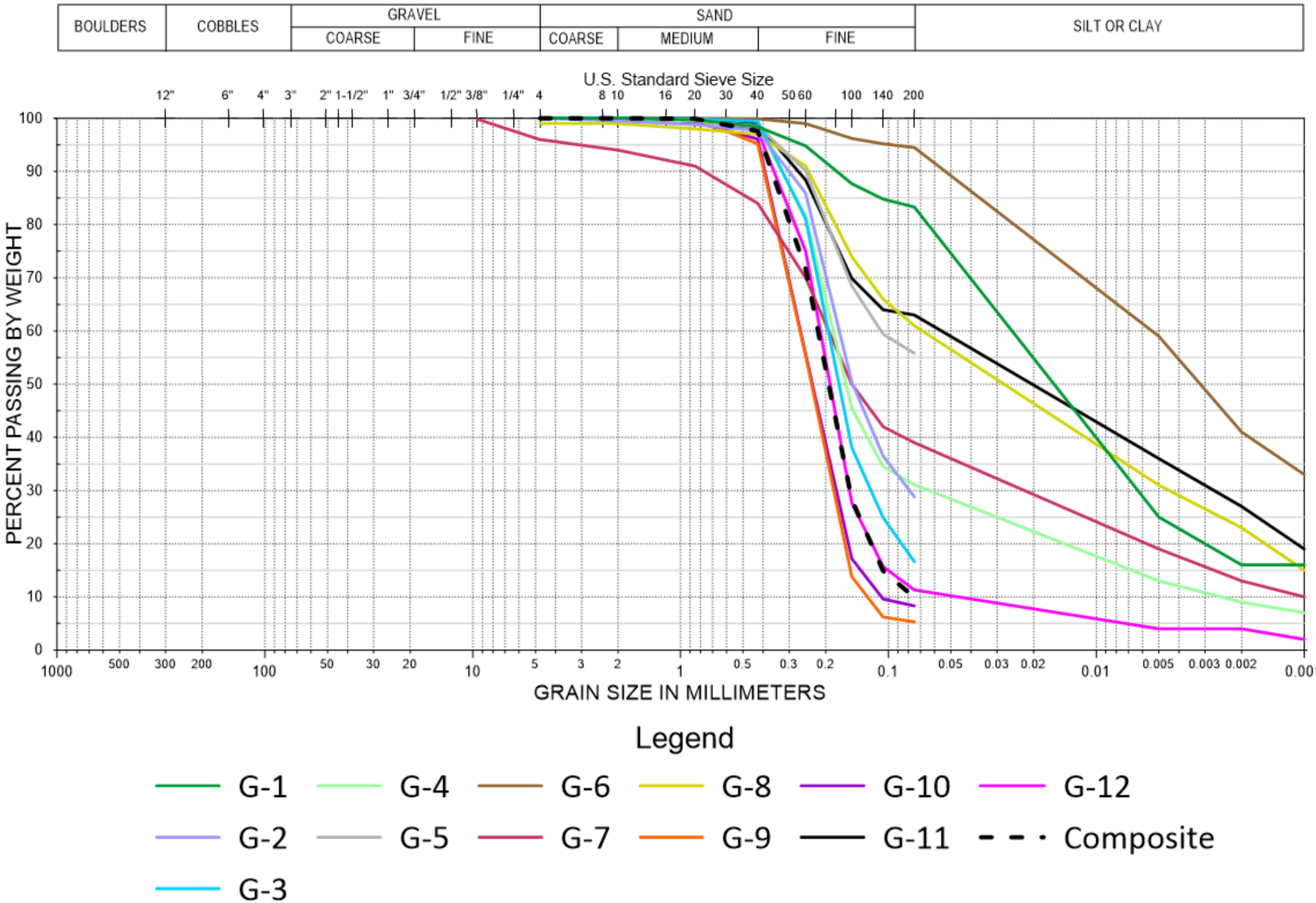
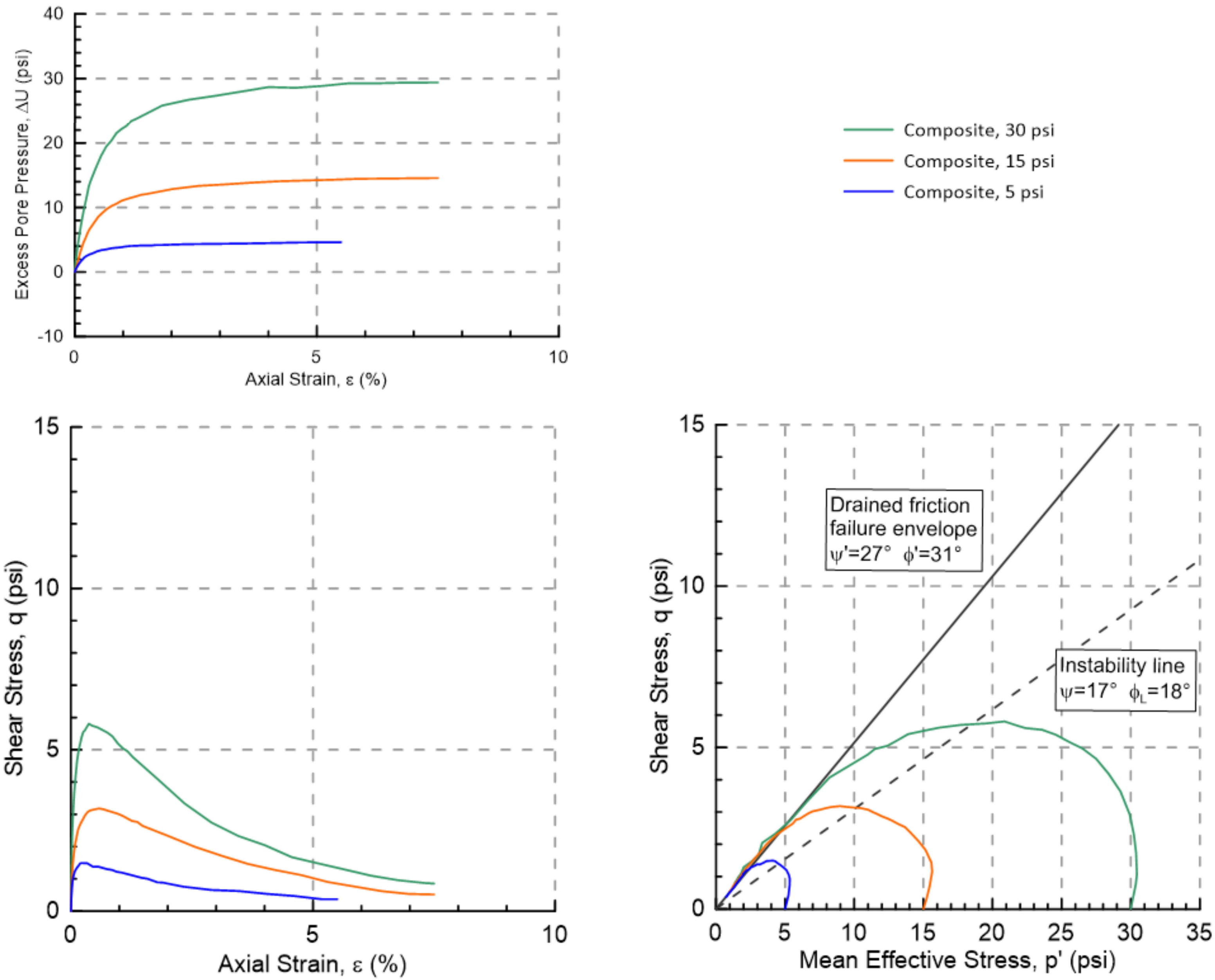
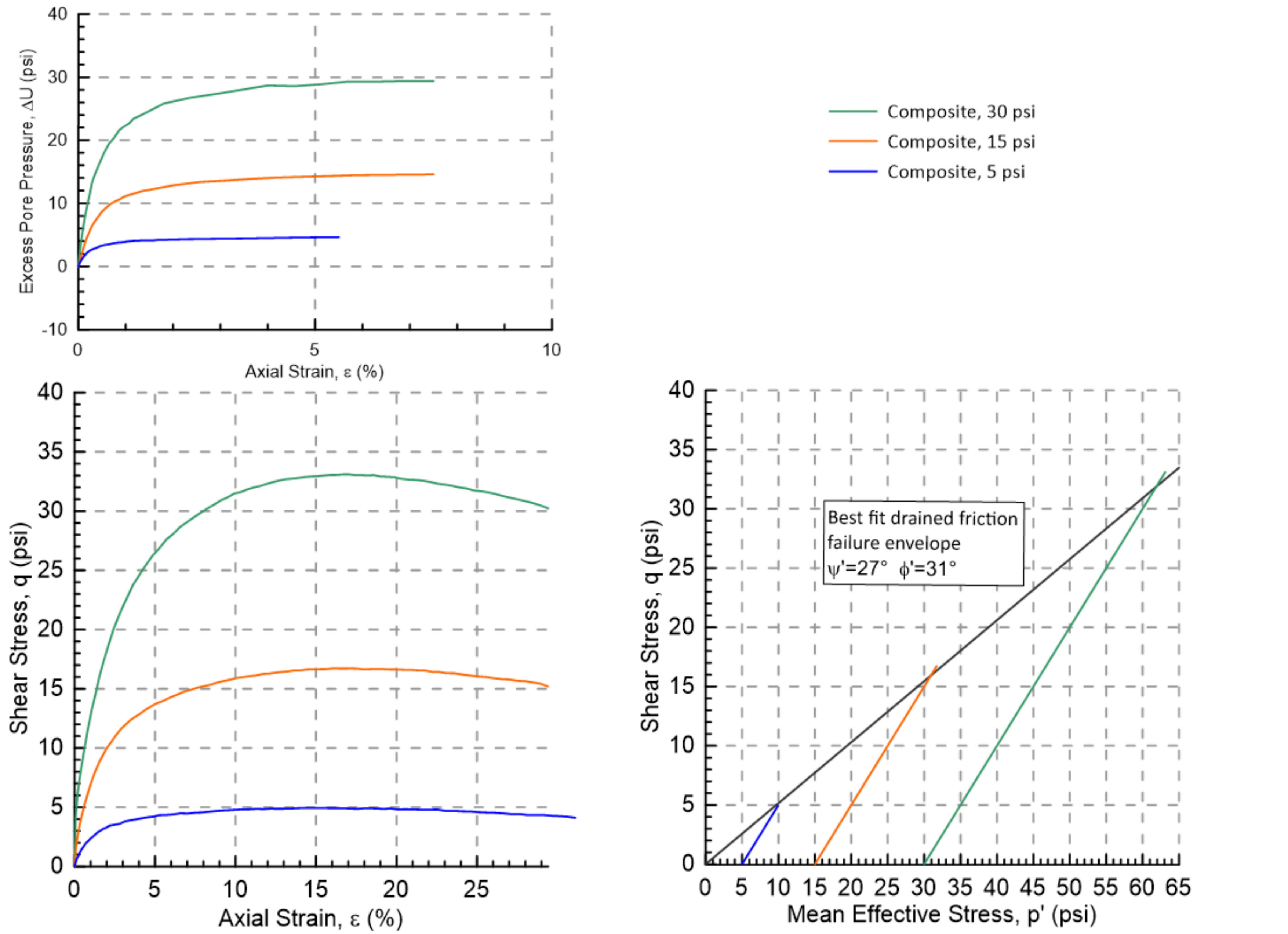


Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	**Grain Size Analysis Results**	Figure E-9



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	CIU' Test Results	Figure E-10



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	**CID Test Results**	Figure E-11

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Attachment E-1: Forensic Team Field and Laboratory Investigations

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Exhibit E-1

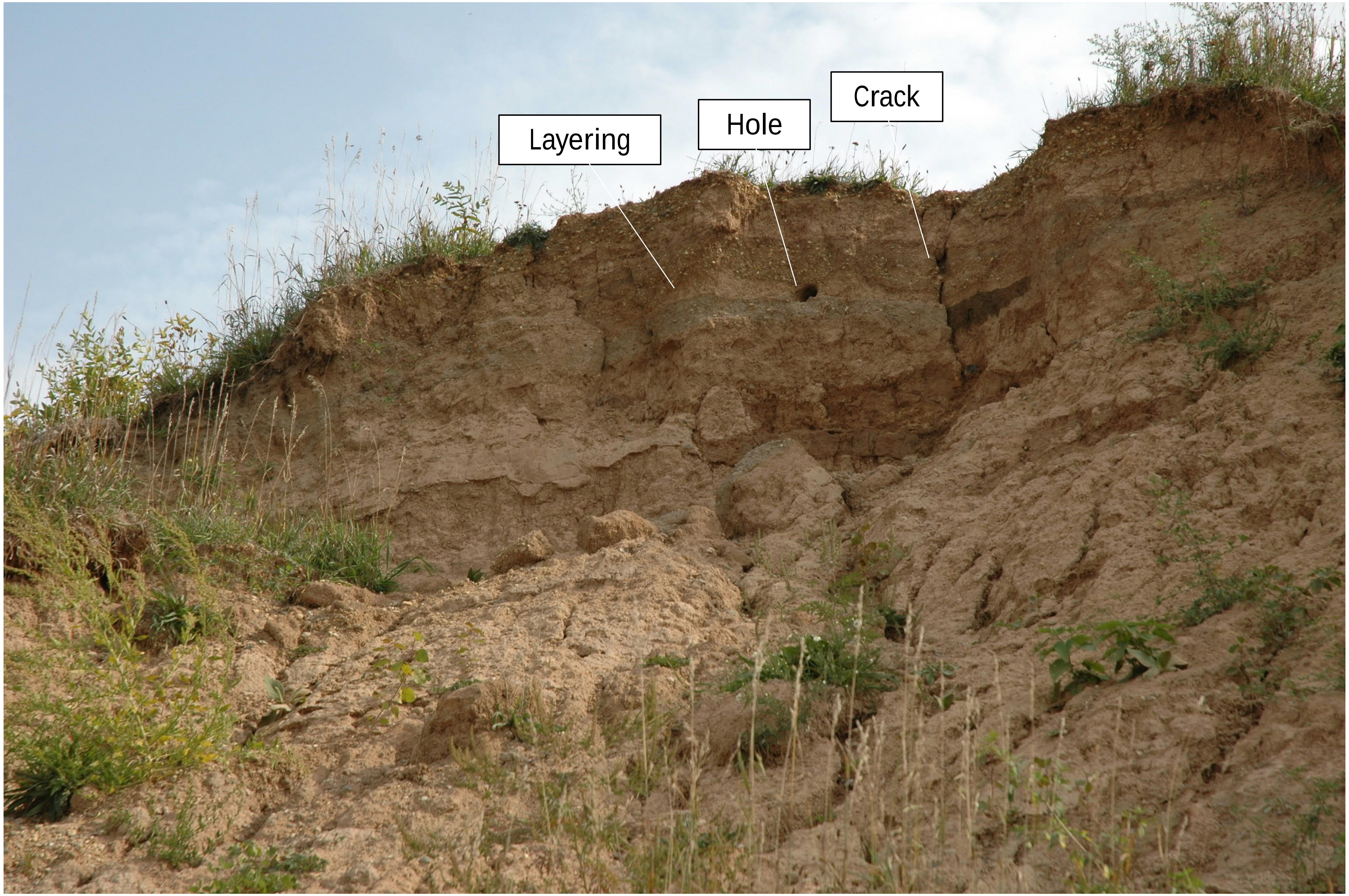


Exhibit E-2


Native Sand Foundation
with Organic Matter
Clay Tile Drain
Glacial Till

Exhibit E-3


Slough and Debris
Glacial Till
Slough
Native Sand Foundation
w/ Organic Matter

Exhibit E-4


Silty Sand Fill
Transition from
Clayey Sand (above)
to Clean Sand (below)
Clayey Fill
Waste Dump
Abutment Contact
Native Clay
Sand Fill
Native Sand Foundation
Native Sand Foundation w/ Organic Matter
Glacial Till
Exhibit E-5

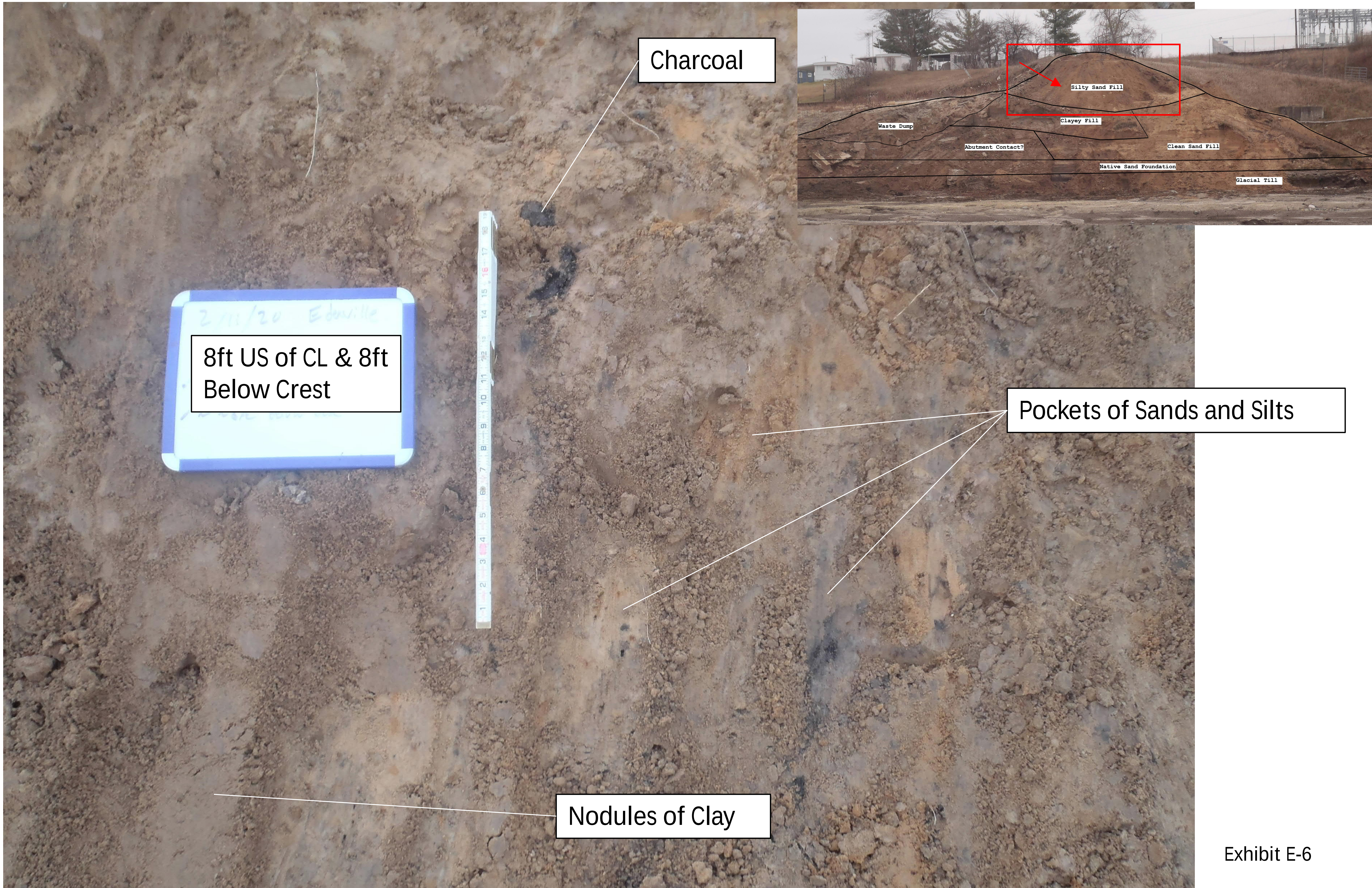

Charcoal
8ft US of CL & 8ft Below Crest
Pockets of Sands and Silts
Nodules of Clay
Silty Sand Fill
Clayey Fill
Waste Dump
Abutment Contact?
Clean Sand Fill
Native Sand Foundation
Glacial Till

Exhibit E-6

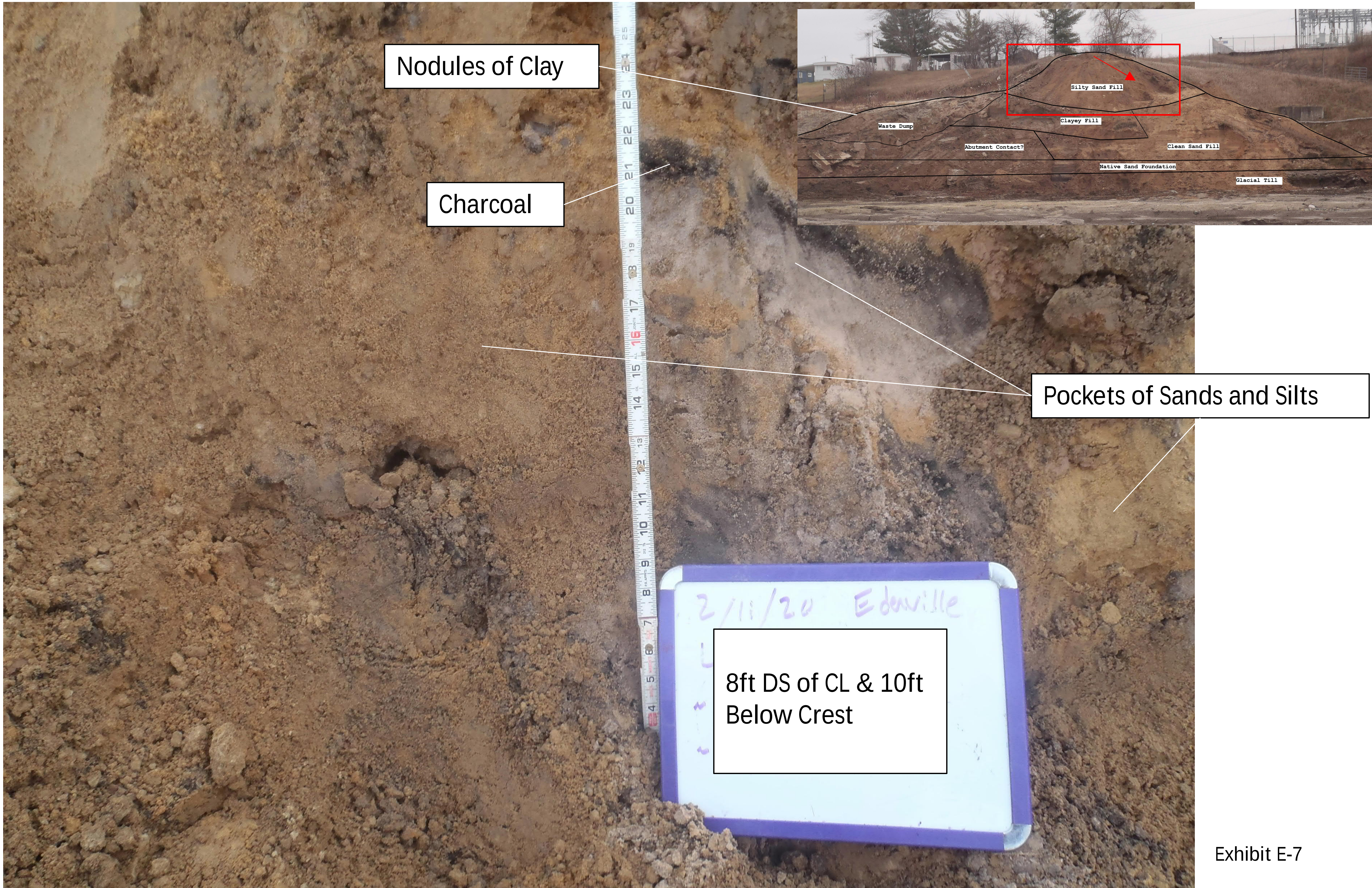

Nodules of Clay
Charcoal
Pockets of Sands and Silts
2/11/20 Edenville
8ft DS of CL & 10ft Below Crest
Silty Sand Fill
Clayey Fill
Waste Dump
Abutment Contact?
Clean Sand Fill
Native Sand Foundation
Glacial Till

Exhibit E-7

15ft US of CL
20ft Below Crest

Exhibit E-8


Clayey Fill
Charcoal
Native Clay
15ft US of CL
20ft Below Crest

Exhibit E-9

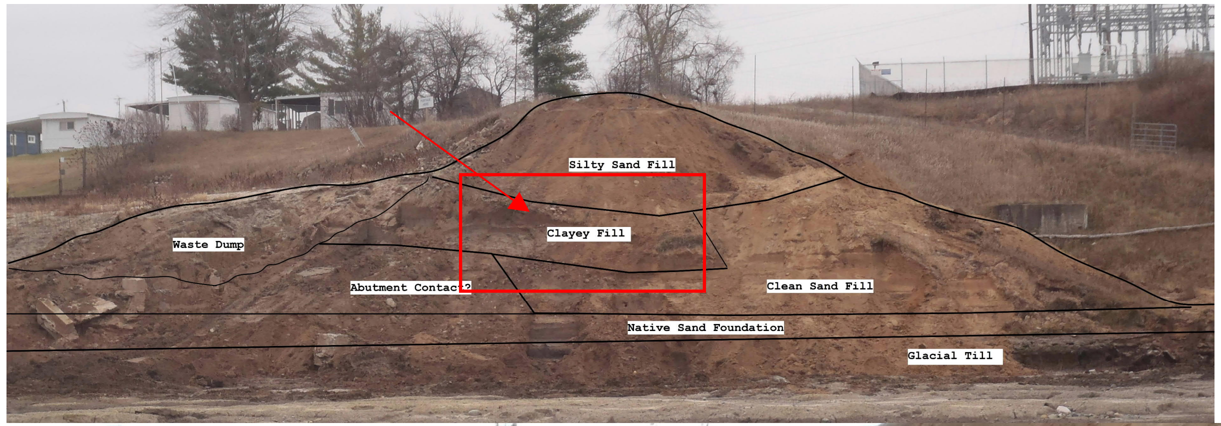

Exhibit E-11
Exhibit E-12
Silty Sand Fill
Clayey Fill
Waste Dump
Abutment Contact?
Clean Sand Fill
Native Sand Foundation
Glacial Till

Exhibit E-10

Exhibit E-11

Exhibit E-12


10ft US of CL
20ft Below Crest
Clayey Fill


Silty Sand Fill
Clayey Fill
Waste Dump
Abutment Contact
Clean Sand Fill
Native Sand Foundation
Glacial Till


Clayey Fill / Sand Contact

Exhibit E-13

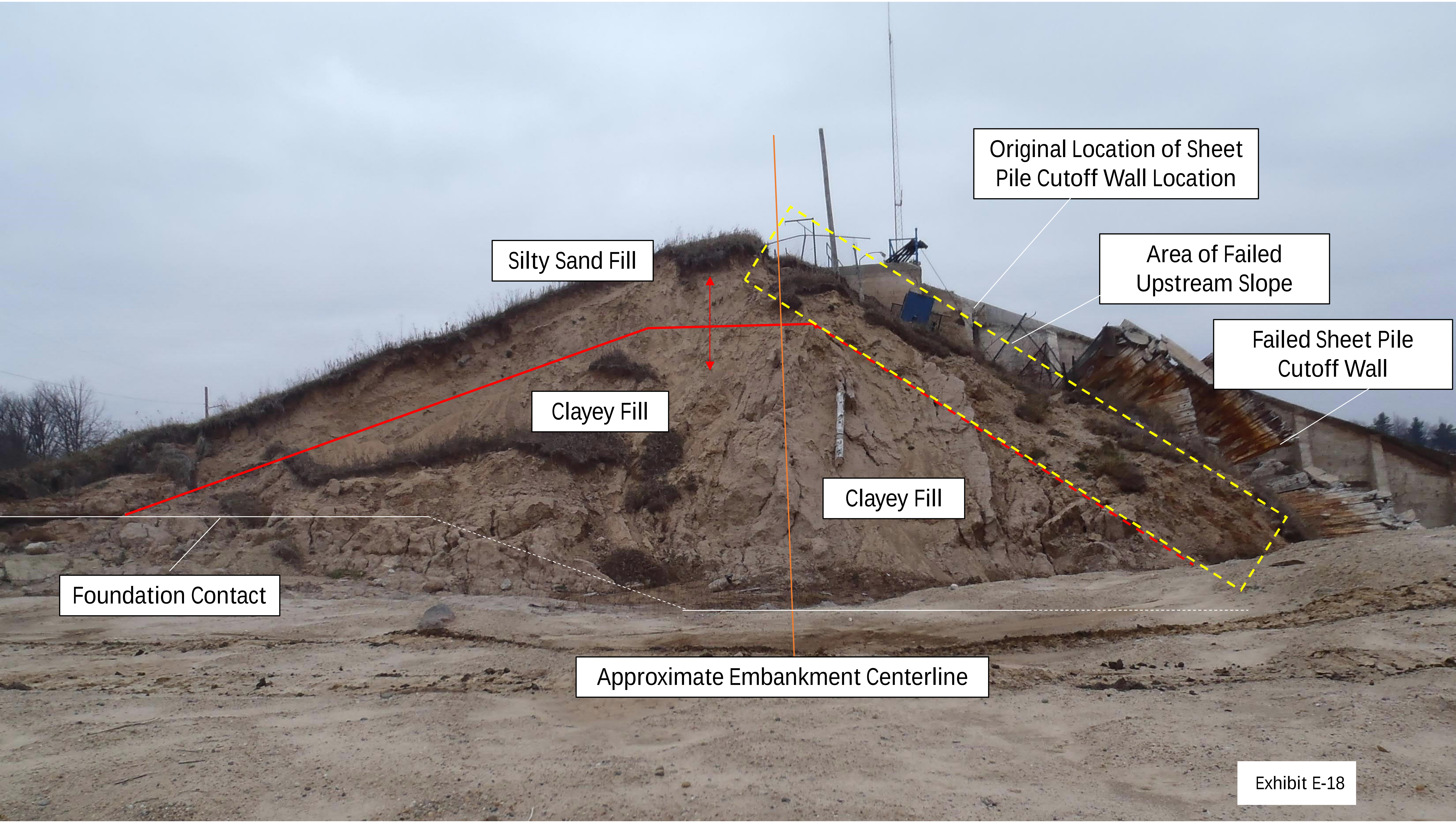

Silty Sand Fill
Clayey Fill
Waste Dump
Abutment Contact?
Clean Sand Fill
Native Sand Foundation
Glacial Till
Exhibit E-15
Clay and Sand contact
Clean Sand Fill

Exhibit E-14

Exhibit E-15

Exhibit E-16

Exhibit E-17


Original Location of Sheet Pile Cutoff Wall Location
Area of Failed Upstream Slope
Failed Sheet Pile Cutoff Wall
Silty Sand Fill
Clayey Fill
Clayey Fill
Foundation Contact
Approximate Embankment Centerline
Exhibit E-18

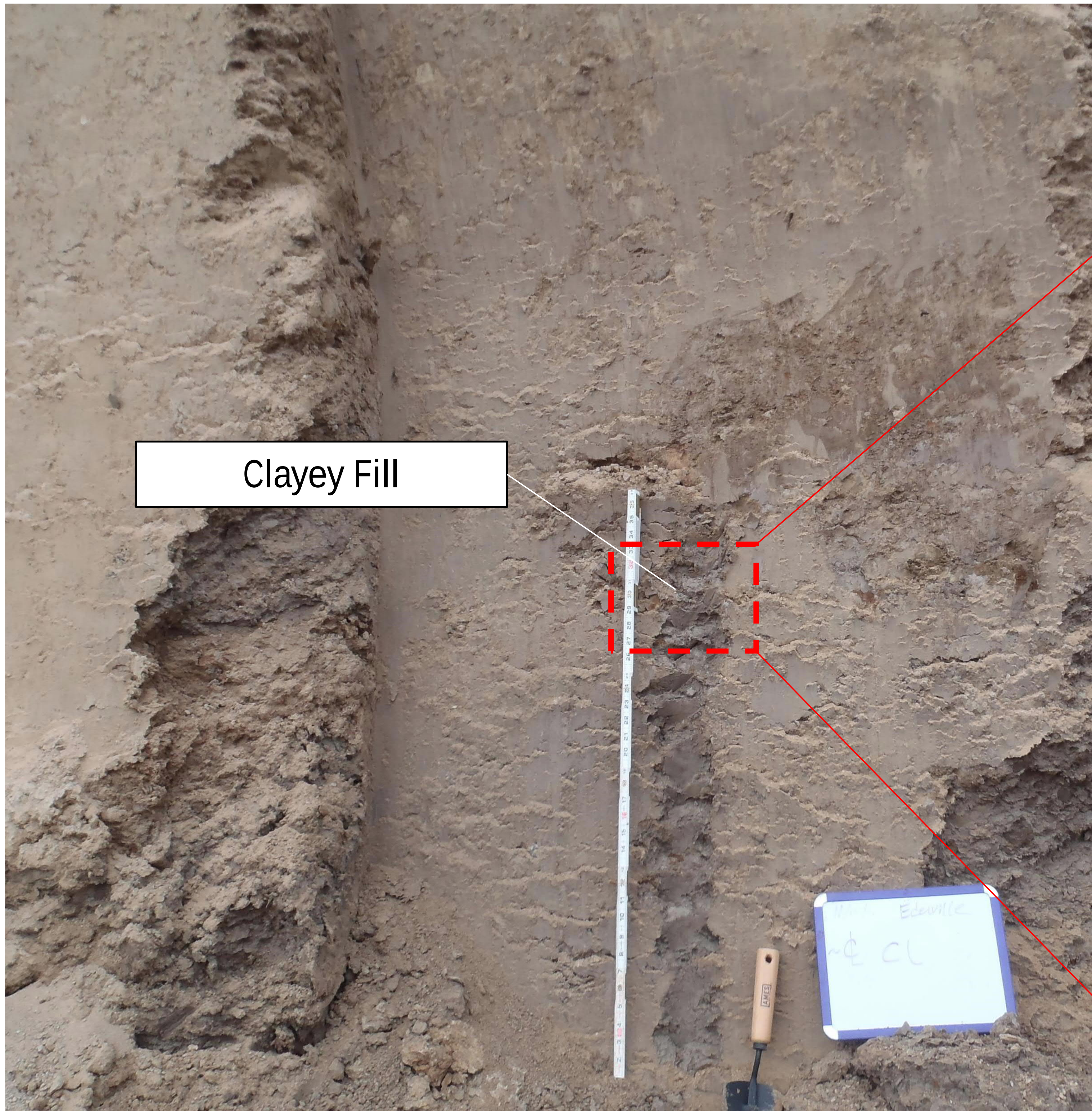


Exhibit E-19


Silty Sand Fill
Clayey Fill
HYUNDAI
Exhibit E-20

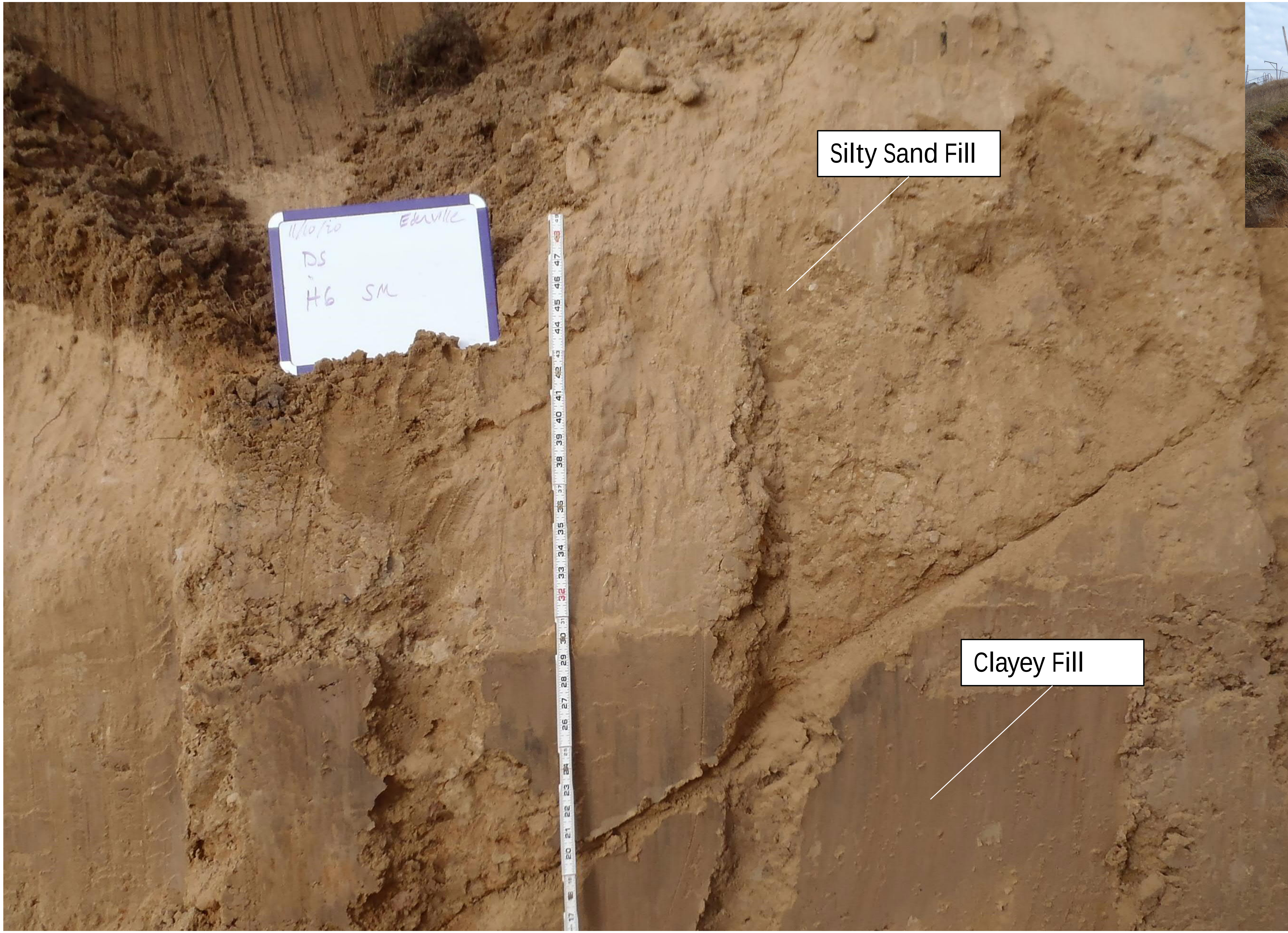


Exhibit E-21

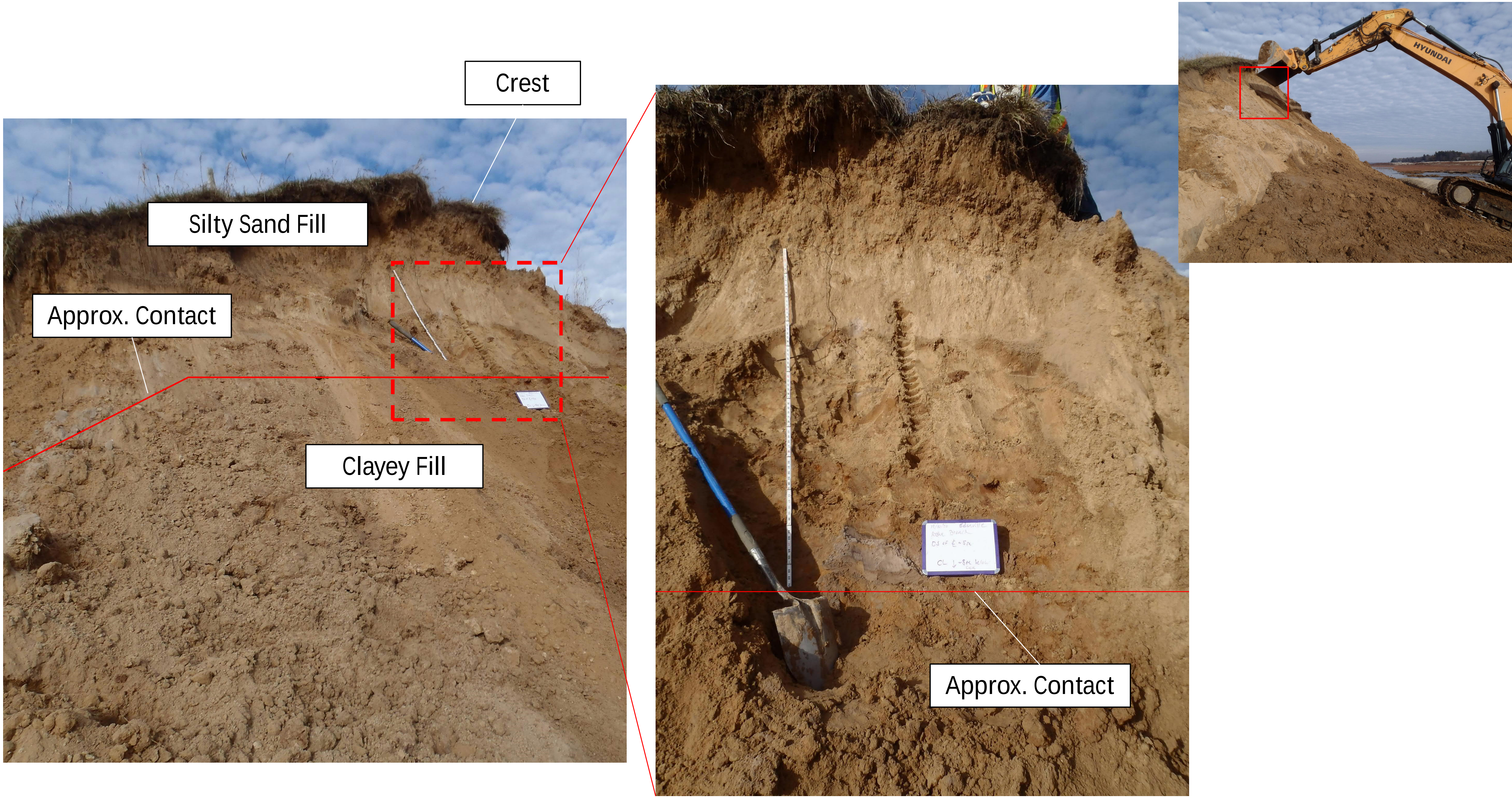

Crest
Silty Sand Fill
Approx. Contact
Clayey Fill
Approx. Contact
HYUNDAI

Exhibit E-22

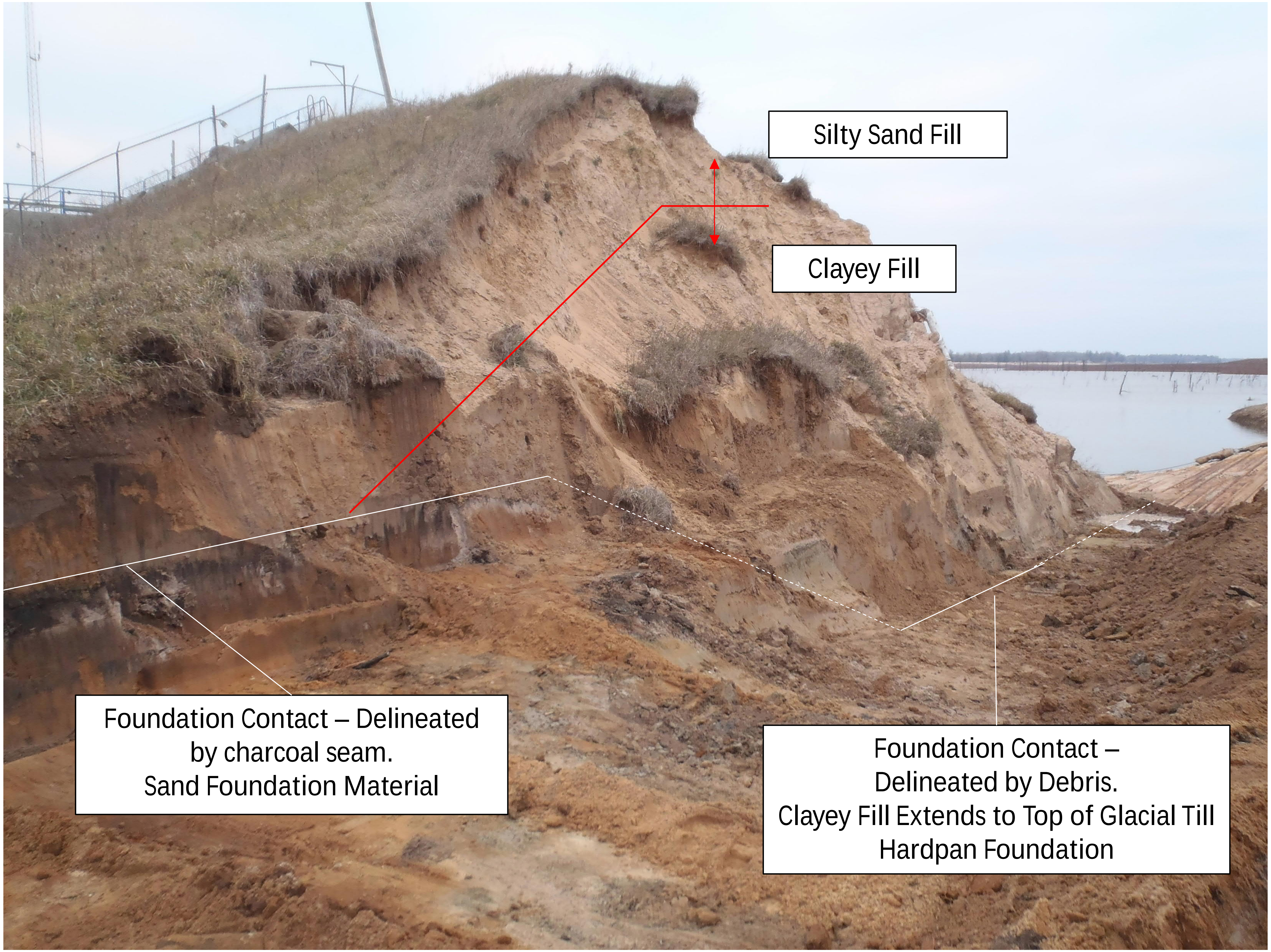

Silty Sand Fill
Clayey Fill
Foundation Contact – Delineated by charcoal seam. Sand Foundation Material
Foundation Contact – Delineated by Debris. Clayey Fill Extends to Top of Glacial Till Hardpan Foundation

Exhibit E-23

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Appendix F: Forensic Team Technical Analyses

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F1: Forensic Team Analysis - Hydrologic and Hydrologic Analyses

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The Independent Forensic Team (IFT) performed a series of hydrologic and hydraulic analyses to better understand the environmental and operational impacts that led to the lake levels observed in Wixom Lake during the May 2020 flood. The IFT performed a thorough review of available records regarding past flood events and related lake operations to inform the analyses. This appendix presents the results of the data review and the IFT's hydrologic and hydraulic analyses.

F1-1 History of Edenville Reservoir Operations

F1-1.1 Long-Term Operations History

In February 1976 the Federal Power Commission (FPC), later replaced by the Federal Energy Regulatory Commission (FERC), declared that the Tittabawassee River is a navigable waterway of the United States and that Wolverine Power, then the owner of the facilities, shall make application for licenses for the Sanford, Edenville, Smallwood, and Secord developments pursuant to the Federal Power Act. Wolverine filed its initial FERC license application for Sanford Dam in 1983, and on December 1, 1987, a single license was granted for the Sanford Project (FERC Project Number P-2785) based on a Single Project Environmental Assessment.

In 1989, Wolverine filed FERC license applications for the Edenville, Smallwood, and Secord developments. Following notice of application, motions were filed by multiple parties to intervene in the proceedings, including statements that large fluctuations in levels adversely affect boaters and lakefront residences and to request that any license issued limit such fluctuations. Upon review of the license applications for the other three facilities, FERC determined that all four developments were hydraulically connected and in 1989 initiated a Multiple Project Environmental Assessment (MPEA) for the four facilities, which was completed in August 1998. In October 1998, 30-year FERC licenses were granted to the other three developments, Edenville, Smallwood, and Secord (Project Numbers P-10808, P-10810, and P-80809, respectively), and the license for Sanford was reissued. The FERC licenses were transferred from Wolverine to Synex Michigan in 2004. In 2006, W.D. Boyce Trusts purchased Synex, and in 2007 Synex Michigan, LLC was renamed Boyce Hydro Power, LLC (Boyce Hydro) and the licenses were transferred to Boyce Hydro.

The FERC licenses established the operating rules for the reservoirs impounded by the four projects based on the MPEA, as described in Section 2.1 of this report. For Edenville, these requirements were:

> "Within sixty days of the installation of reservoir level gages required by Article 404 the Licensee shall operate the Edenville Project so that the project reservoir elevation does not fluctuate more than 0.4 foot below or 0.3 foot above the normal pool elevation of 675.8 feet National Geodetic Vertical Datum (NGVD) except during the winter drawdown. The Licensee shall begin the winter drawdown after December 15 and shall complete the winter drawdown by January 15 of each year. The Licensee shall complete the refill of the reservoir, thus ending the winter drawdown period, prior to the surface water temperature of the reservoir reaching 39°F. During the winter drawdown, the Licensee shall operate the Edenville Project so that the reservoir level does not fall below 672.8 feet NGVD, and so that the daily fluctuation in reservoir elevation does not exceed 0.7 foot. (FERC 1998b)."

FERC requirements for lake levels for all four dams allowed for limited peaking power operations, in which water could be discharged through the turbines preferentially during peak power demand times and not during off-peak hours as long as the lake levels stayed within the 0.7-foot limit established by the permit. During floods, the spillway gates would be operated to limit the rise in the lake level and return the lake level to the specified normal range as soon as possible.

Prior to FERC regulation, the facilities could be operated in a less restrictive peak power mode in which the owner could adjust operations to maximize electricity production when there was high demand. This, in turn, could result in the lake levels fluctuating over a wider range to store and release water.

The use of the facilities to help manage downstream flooding has been considered at various times throughout the project history. However, the primary purpose of the dam operations has consistently been to generate power, which requires different operating procedures than those for flood management facilities. The storage provided by the facilities was found to be inadequate to effectively manage downstream flooding, and any flood mitigation provided by dam operations was viewed as a secondary benefit (State of Michigan House of Representatives 1932). In 1998, FERC reported that "The U.S. Army Corps of Engineers investigated the flood hazard reduction potential of Wolverine's projects and found it to be negligible. Local residents are concerned that elimination of the late-winter drawdowns would reduce the projects' flood protection benefits. The flood control benefits of the late-winter drawdown are negligible in true flood situations. However, some benefits are provided, mostly to shoreline residents, by minor reduction in the extent and magnitude of spring runoff flows" (FERC 1998a). These conclusions were confirmed by the IFT's hydrologic and hydraulic analyses, discussed later in this appendix, which showed that pre-lowering of Wixom Lake before the May 17 through 19, 2020, event would have had very little effect on the ultimate lake level during the storm.

Despite the apparently less stringent lake level limitations prior to FERC licensing, available records indicate that Wixom Lake (the reservoir impounded by Edenville Dam) rarely rose significantly above the FERC established normal pool level of Elevation (El.) 675.8, as shown in Figure F1-1. In fact, there were only 4 years when the reservoir rose 1.4 feet or more above El. 675.8 – in 1928, 1929, and 1945, and during the failure in 2020. In 2014, the lake level rose by 1.36 feet, just short of 1.4 feet. There were 22 years when the reservoir rose to greater than 1 foot and less than 1.4 feet; however, all of those years except one (2014) were before the implementation of the FERC lake level requirements. As can be seen in Figure F1-1, maximum yearly lake levels were higher before 1975 than since that time, likely reflecting in part the less restrictive peak power operations in earlier years.

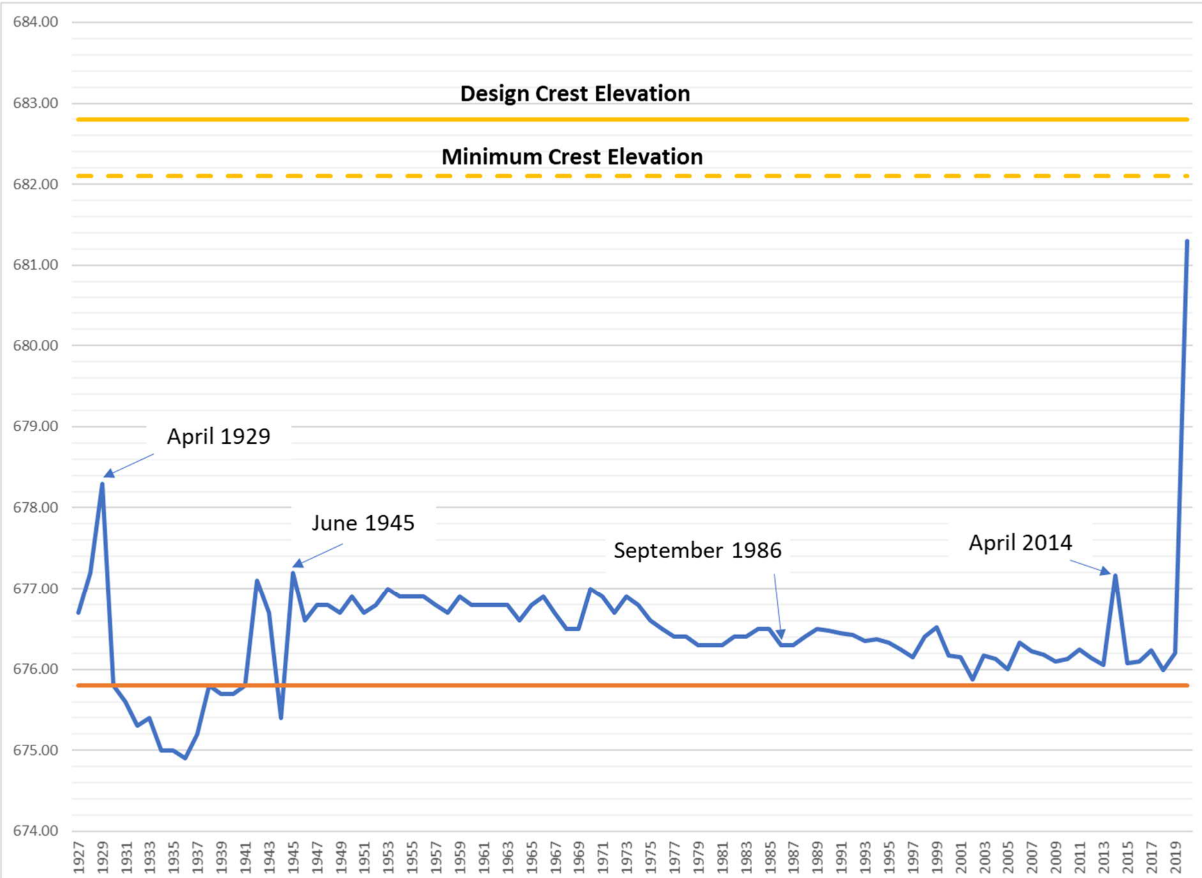


Figure F1-1: Annual Maximum Wixom Lake Levels from 1927 through 2020

A few comments regarding the historic Wixom Lake data are appropriate. The IFT is not certain that the available information on Wixom Lake levels is entirely complete. The information available to the IFT consisted of a spreadsheet with the highest annual lake levels and some handwritten records regarding lake levels. Boyce Hydro reported to the IFT that the spreadsheet was created after Boyce Hydro became the project owner. The spreadsheet was compiled from available data by part-time Boyce Hydro employees. When the IFT compared spreadsheet entries with the handwritten records, some discrepancies were found. For example, the spreadsheet indicated a maximum water level of El. 678.8 for 1930. When those handwritten records were reviewed, it was found that the lake levels for the referenced date were actually below the normal pool level when the reservoir was being operated in a drawdown condition over winter. The actual maximum water level for 1930 was estimated to be at normal pool level. Discrepancies of lesser magnitudes were found for 1927, 1928, and 1929. When discrepancies were found, the information from the handwritten logs was used. It should also be noted that the 2015 Consultant's Safety Inspection Report for Edenville Dam (Purkeypile 2016) states that the flood of record occurred on June 3, 1945, with a Wixom Lake level 2.4 feet above the normal pool level. According to the IFT's interpretation of the available information, the pool of record prior to May 2020 actually occurred in April 1929 at 2.5 feet above normal pool level, and the 1945 maximum pool level was only 1.4 feet above the normal pool level.

F1-1.2 Recent Operations History

After FERC revoked the license for Edenville Dam in September 2018, FERC lake level requirements no longer governed the lake operations. No state legal lake level requirements under Part 307 of the Michigan code were in place for Wixom Lake until Gladwin and Midland Counties, through a court order, (State of Michigan 2019) established lake level requirements in May 2019. The lake level requirements set for Wixom Lake in the court order were the same as the earlier FERC requirements. Therefore, the 2018-2019 drawdown occurred during the time period when there were no FERC or Michigan Part 307 lake level requirements in place. Moreover, the 2019-2020 drawdown occurred after the counties and the court had used the Part 307 process to reimpose requirements that were the same as the FERC lake level requirements, and Boyce Hydro and the Four Lakes Task Force (FLTF) had subsequently entered into an agreement to follow the Part 307 lake level requirements (FLTF 2019c).

On September 20, 2018, Boyce Hydro began to draw down Wixom Lake to perform safety-related “focused spillway and gate assessments” on Smallwood Dam, as directed by FERC. The drawdown for this activity lowered the water level of Wixom Lake to about 4.6 feet below the normal pool elevation. The FERC directive (Nims 2018) for this activity called for refilling the lake to normal water level after the spillway and gate assessments were completed, which was in October 2019. However, with Edenville Dam no longer FERC-licensed, Boyce Hydro opened the gates at Edenville Dam and allowed the lake to operate “run-of-river,” with the lake level controlled by flow over the spillway concrete sill at El. 667.8. Michigan Department of Environment, Great Lakes, and Energy (EGLE) personnel have indicated to the IFT that although specific Part 307 lake level requirements were not in place, the State’s position is that Wixom Lake was still subject to Part 301 of the state statutes, which requires a permit for increase or diminishment of an inland lake or stream. EGLE personnel have told the IFT that to their recollection, there was no communication from Boyce Hydro to EGLE before the gates were opened. Through the 2018-2019 winter season, Wixom Lake remained below El. 670, except for four instances of higher water levels caused by precipitation-related events. The maximum drawdown of 7.1 feet occurred on January 22, 2019. The lake was restored to its normal pool level, El. 675.8, on May 12, 2019. The 2018-2019 winter drawdown lake levels are shown in Figure F1-2. EGLE and the Michigan Department of Natural Resources have alleged that the 2018-2019 Wixom Lake drawdown resulted in the mortality of 100,000 or more freshwater mussels, which the State asserts would have cost hundreds of millions of dollars to replace. The alleged environmental damage ultimately led to the filing of a lawsuit by the State against Boyce Hydro on April 30, 2020.

On April 16, 2019, a Letter of Agreement (FLTF 2019a) was executed by FLTF and Boyce Trusts indicating the intention of FLTF to buy Sanford, Edenville, Smallwood, and Secord Dams. Later in 2019, FLTF, in conjunction with Boyce, applied to EGLE for a permit for an 8-foot early winter drawdown of Wixom Lake. FLTF retained consultants to prepare and submit a permit request, naming Boyce Hydro as the applicant, to lower the lake earlier than December 15. The reason presented in support of the request for the earlier and deeper drawdown was concern over problems with ice buildup on the spillway gates. Given that the powerhouse could not pass water and generate electricity because of the loss of the FERC license, flows had to be conveyed through the spillways to maintain lake levels within the court-ordered operating range. The winter ice buildup on the spillway gates could endanger operators when they are adjusting gate positions and could endanger the gates themselves.

In conjunction with the drawdown request, FLTF proposed to provide due diligence, having biologists and engineers conduct an organized survey of the lake bottom and shallow waters for stranded fish and native freshwater mussels. Stranded fish and mussels would be relocated to the nearest area of lake deep enough to help ensure that they would survive through the winter until the lake water temperature reached 39 degrees Fahrenheit (°F), after which the lake would be restored to normal pool level, El. 675.8, later in the spring, in accordance with the normal spring refill requirements.

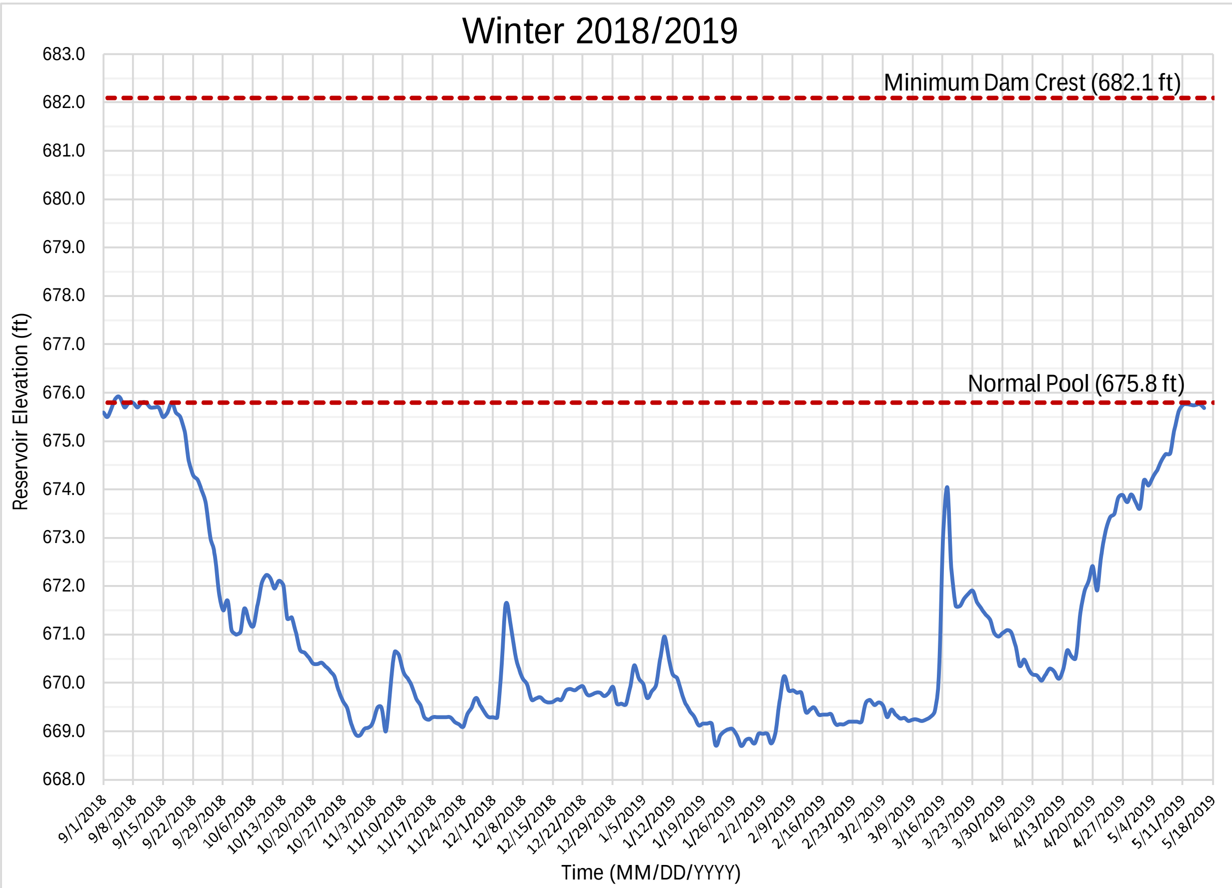


Figure F1-2: Wixom Lake Levels – September 18, 2018, through May 12, 2019

In discussions with the IFT, EGLE indicated that it concluded from discussions with other dam operators in Michigan that the spillway gates at Edenville Dam could be safely operated through the winter if measures were taken to limit ice development.

While the permit request was pending, Boyce Hydro, at the direction of FLTF (as indicated by email correspondence), began lowering the lake level in early November 2019, 1.5 months earlier than the December 15 date in the county and court-issued lake level requirements. On November 25, 2019, EGLE issued a denial of the request (FLTF 2019b), principally citing adverse natural resources and environmental effects, but also suggesting that EGLE believed there were alternatives for addressing the ice concern.

A maximum drawdown of 6.43 feet occurred on December 27, 2019. The 2019-2020 winter drawdown lake levels are shown in Figure F1-3.

In March 2020, FLTF filed a permit application with EGLE to refill the Edenville reservoir, naming Boyce Hydro as the applicant, and on April 9, 2020, EGLE issued Permit No. WRPO21788 v.1 (EGLE 2020), which directed Boyce Hydro to “conduct refill activities at the Edenville Facility to raise the

Wixom Lake water level to normal summer pool elevation of 675.8 (North American Vertical Datum of 1988) during the spring 2020." On May 4, 2020, the lake level reached 676.0 feet, 0.2 foot above normal pool level.

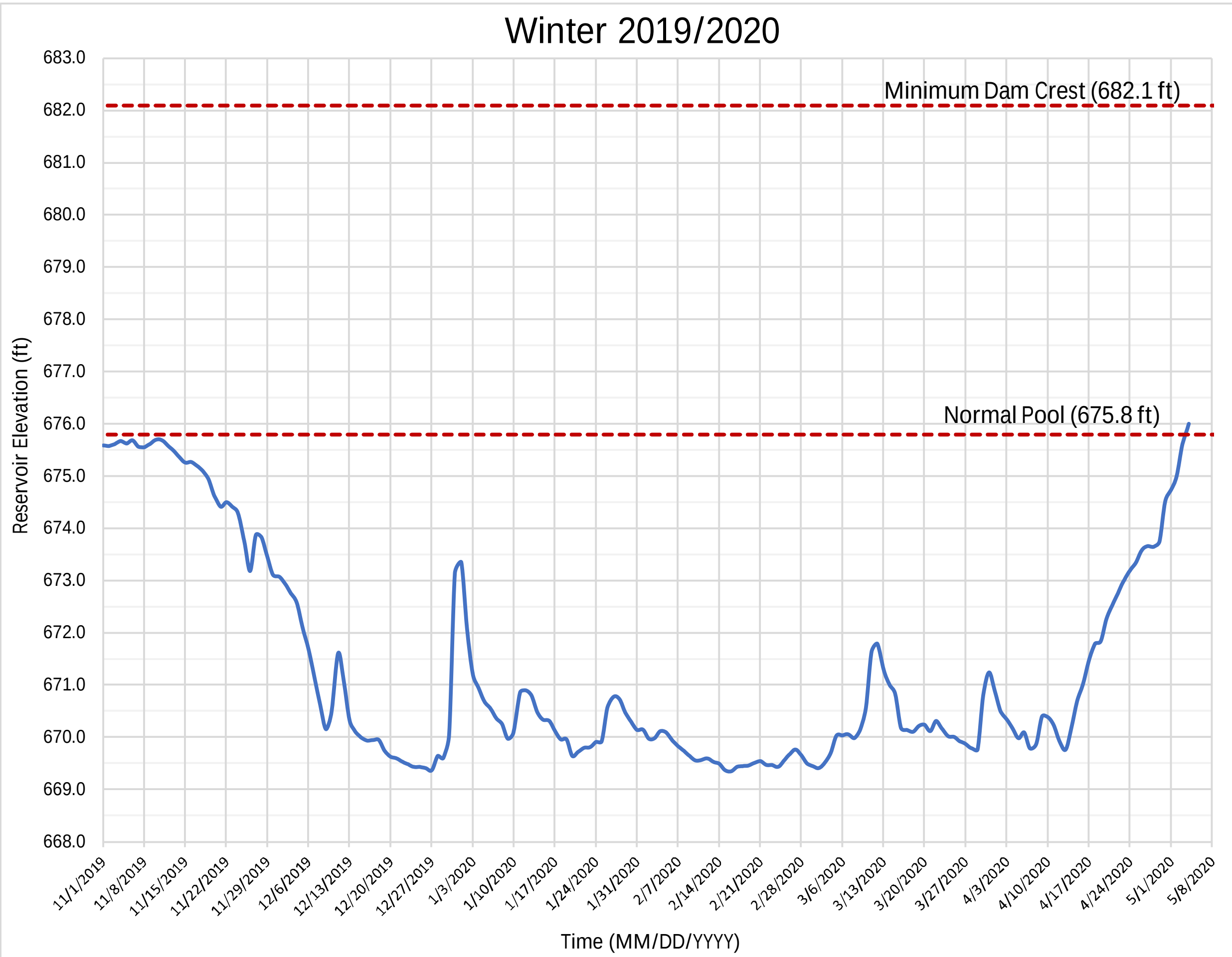


Figure F1-3: Wixom Lake Levels - November 1, 2019, through May 4, 2020

F1-2 Historical Rainfall Events

In assessing the May 17 through 19, 2020 event, it is reasonable to ask if larger rainfall events have occurred in the watershed since the construction of the dams in the mid-1920s. A search of the rainfall records of the National Oceanic Atmospheric Administration (NOAA) Gladwin, MI weather station was used to identify other significant rainfall events that have occurred over the past 100 years. The Gladwin weather station's monthly rainfall data from 1927 through 2020 was used to select the months with the highest rainfall. Those months were then investigated, and a list compiled of the highest rainfall events (rainfall/duration), and that list was further narrowed to 20 storm events.

The list of 20 storm events was further reduced by examining the operational logs to identify how the gate operations and powerhouse generation impacted the lake levels. Storm events were eliminated: (1) if the lake levels remained within the normal operating pool criteria (0.4 foot below or 0.3 foot above), or (2) when the lake level may have risen above normal operating pool criteria, but the gates were not opened to lower the lake level and maintain normal pool. The maximum lake level rise was about 1.0 foot for some of the events that were eliminated. The powerhouse for all these events was operating at 100 percent capacity, with no or minimal gate openings (no spillway flow). Typically, the power generation was between 7:00 a.m. and 9:00 p.m., depending on the stream flow.

Combining the list of significant rainfall events with the maximum lake levels (see Figure F1-1 and the discussion in Section F1-1.1), a short list of six of the most consequential historical storms plus the May 2020 event (seven events in total) was selected for further evaluation and is summarized in Table F1-1. The highest lake levels recorded over the past nearly 100 years were for 1928, 1929, 1945, 2014, and 2020 storm events. By a large margin, the May 2020 event resulted in the maximum historical lake level at Wixom Lake, approximately El. 681.3 feet, 5.5 feet above normal pool level and about 3 feet higher than the previous highest lake level in 1929, even though the event did not have the highest total rainfall. The 2017 and 1986 rainfall events were selected because they were the two largest runoff events recorded at the U.S. Geological Survey (USGS) stream gage station in Midland, MI, and had significant rainfall. The 1928 storm event was selected because it had a lake level of +1.4 feet above normal pool level, even though it was not one of the higher rainfall totals.

There are clear differences in the rainfall depths versus the maximum lake levels for the seven storm events. The two largest riverine flows recorded at the USGS stream gage station in Midland, MI, were 39,100 cfs and 38,700 cfs for the June 2017 and September 1986 rainfall events, respectively. These two events recorded the two largest rainfall totals of the short-listed storms. It is interesting to note that the Wixom Lake levels for those two storms were the two *lowest* lake elevations (+0.5 foot and +0.4 foot above normal pool level) of the seven storm events listed in Table F1-1. This indicates that total rainfall, by itself, is not a good predictor of lake levels in this watershed. All of the events listed in Table F1-1, except for the May 2020 event, are described in more detail in this subsection. The May 2020 event is described in Section F1-4. Air temperatures identified in the following sections are based on data recorded at the Gladwin, MI weather station. Air temperatures in much of the Edenville watershed may have been colder than the recorded temperatures at the Gladwin Weather Station, especially during the winter and spring, since much of the watershed consists of forested areas, whereas the Gladwin, MI weather station is in an open area in the town of Gladwin.

Table F1-1: Seven Notable Rainfall Events with Maximum Pool Levels, including the May 2020 Event

Maximum Lake Level		Depth above Normal Pool	Rainfall/Duration
Date	Elevation (1)		
June 26, 1928	677.2	+ 1.4 feet	2.92 inches/2 day (2)
April 6, 1929	678.3	+ 2.5 feet	4.46 inches/3 day
June 2, 1945	677.2	+ 1.4 feet	3.59 inches/2 day
September 12, 1986	676.3	+ 0.5 feet	6.83 inches/3 day
April 13, 2014	677.2	+ 1.4 feet	4.53 inches/2 day
June 23, 2017	676.2	+ 0.4 feet	5.04 inches/4 day
May 19, 2020	681.3	+ 5.5 feet	4.30 inches/2 day

(1) Maximum lake levels based on original operations logs.
(2) Precipitation data in the table are from the NOAA Gladwin, MI weather station, except for the June 1928 event. No precipitation data were recorded at the NOAA Gladwin, MI station in 1928; rainfall data for that storm are from the NOAA West Branch, MI weather station.

F1-2.1 June 25 and 26, 1928 Storm Event

The West Branch, MI weather station was used to estimate the rainfall for the June 25 and 26, 1928 storm event, because the Gladwin weather station was inoperable at that time. The West Branch weather station is approximately 30 miles north of Gladwin, and, therefore, would provide a reasonable estimate of the weather for the basin.

The total amount of rain on June 25 and 26 was 2.92 inches. For the 7 days prior to June 25 a total of about 1.98 inches of rain, which added to the antecedent moisture, was recorded. The flood of 1928 caused the failure of both the Chappel Dam and the Schulz Dams on the Cedar River (Little Forks Conservancy 2011), which added to the flow entering Wixom Lake.

Figure F1-4 shows the lake levels for the June 1928 event as obtained from Boyce Hydro record log information. The level of Wixom Lake began to rise above normal pool at midnight on June 25 and returned to normal pool at 3:00 a.m. on June 28. By noon on June 26, all six gates at both spillways were opened to 7.5 feet and the lake level had reached El. 676.8. At 4:00 p.m. on June 26, the powerhouse was at 100 percent discharge capacity, with the lake at El. 676.8. Five of the six gates were open 7.5 feet, and Gate 3 at the Tobacco spillway was open 4.5 feet. The maximum lake level was 677.2 feet (1.4 feet above normal pool) at 6:00 p.m. on June 26.

Temperature data are shown in Figure F1-5 for April through June 1928. There were no freezing temperatures for an extended period in the 6 weeks before the event.

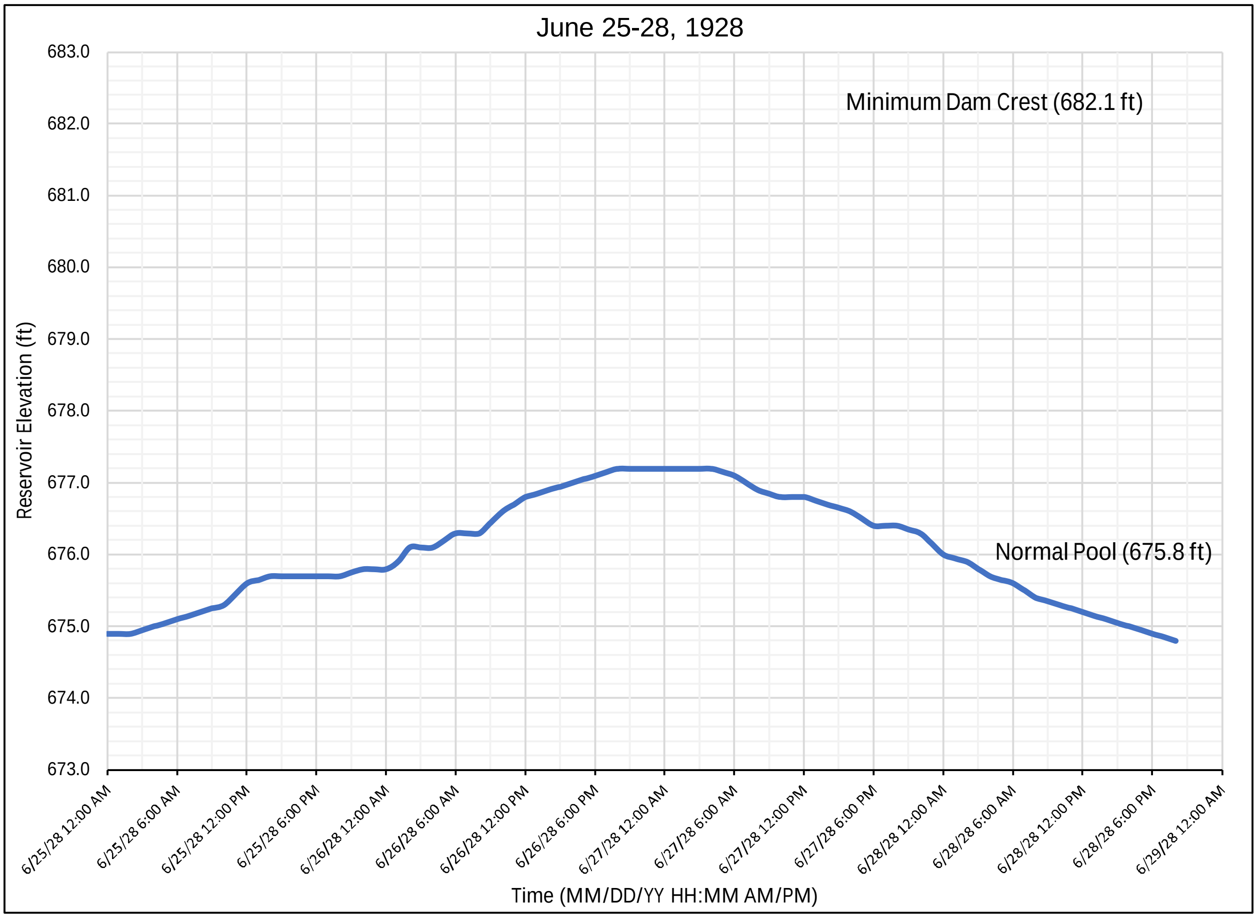
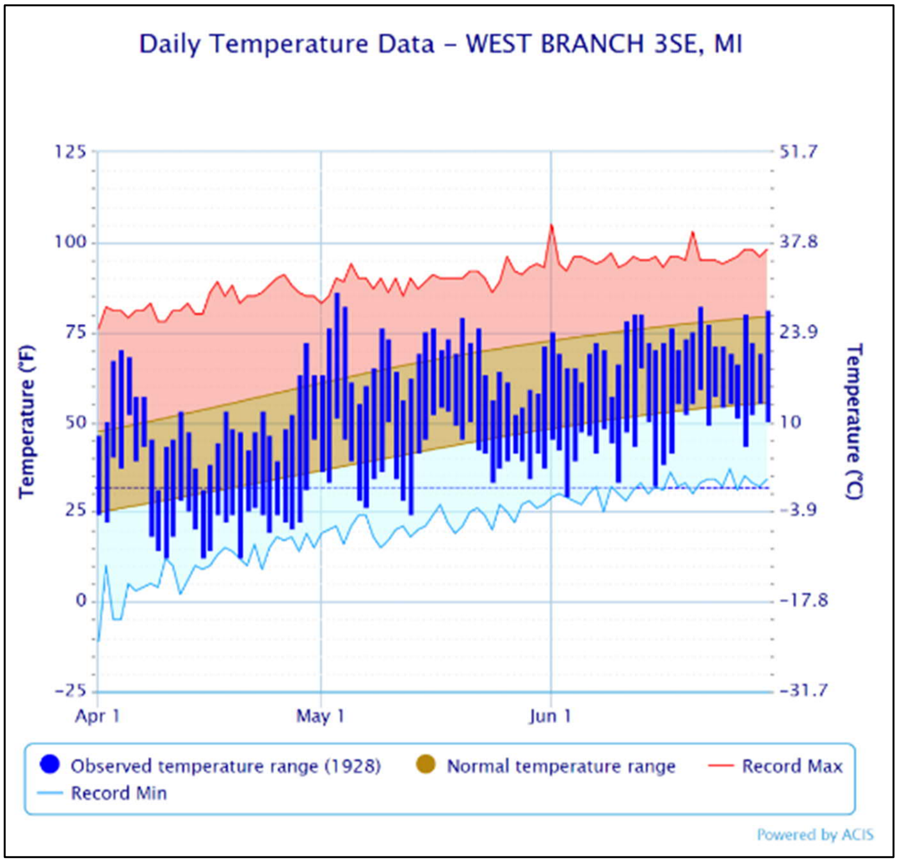


Figure F1-4: Wixom Lake Levels from June 25 through 28, 1928



Source: NOAA 2022a

Figure F1-5: Temperature Data for April through June 1928

F1-2.2 April 5 through 9, 1929 Storm Event

"The flood of April 5-15, 1929, on the Tittabawassee River was due to excess rainfall preceded by a glaze storm and accompanied and increased by the failure of three small power dams in the headwater" (State of Michigan House of Representatives 1932). The Gladwin weather station recorded 4.46 inches of rain in 3 days, with 2.70 inches occurring on April 5, no rain on April 6, and 1.76 inches on April 7. The West Branch weather station recorded about an inch of rain prior to the April 5 storm event.

Wixom Lake began to rise above normal pool at 5:00 a.m. on April 5 and reached El. 676.2 at 6:00 a.m. April 5, with all six gates open to 7.5 feet. At 7:00 a.m., the powerhouse was at 100 percent capacity. The maximum lake level of El. 678.3 feet was recorded on April 6 at 2:00 a.m. Figure F1-6 is a time history of Wixom Lake levels for the April 1929 event as obtained from Boyce Hydro record log information. The lake level stayed above normal pool for approximately 6 days. The maximum lake level was 2.5 feet above normal pool level, the pool of record, until the May 2020 event. Additional precipitation totaling 0.69 inch occurred on April 9 and 10, 1929.

The average temperature at the Gladwin weather station for the preceding month of March was 34.3°F, with the minimum daily temperature averaging around 20.4°F. The minimum temperature dropped for the last day in March and the first couple of days in April to below 25°F. On April 3 the minimum temperature rose to 40°F and remained at or above that temperature for the remainder of the month. Figure F1-7 shows the temperature data for the April event.

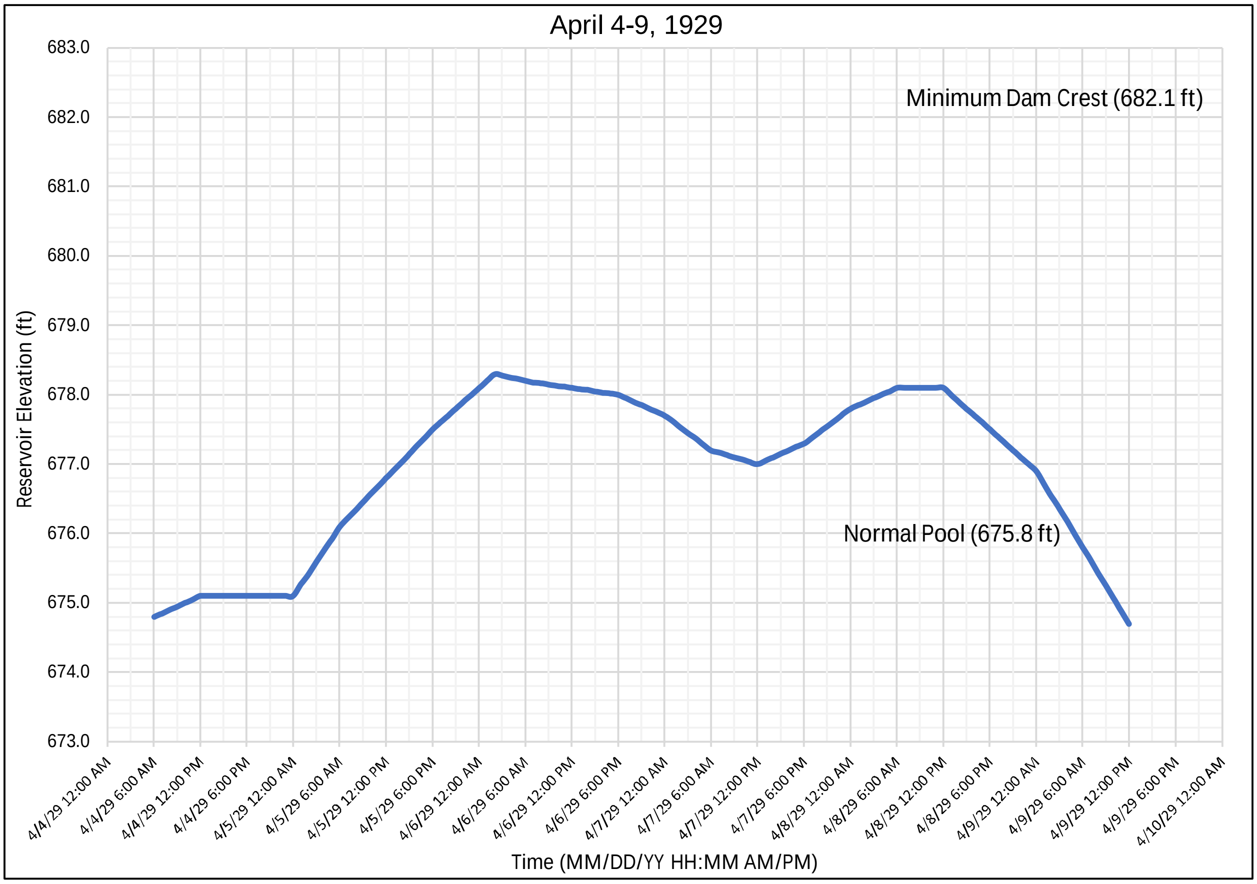
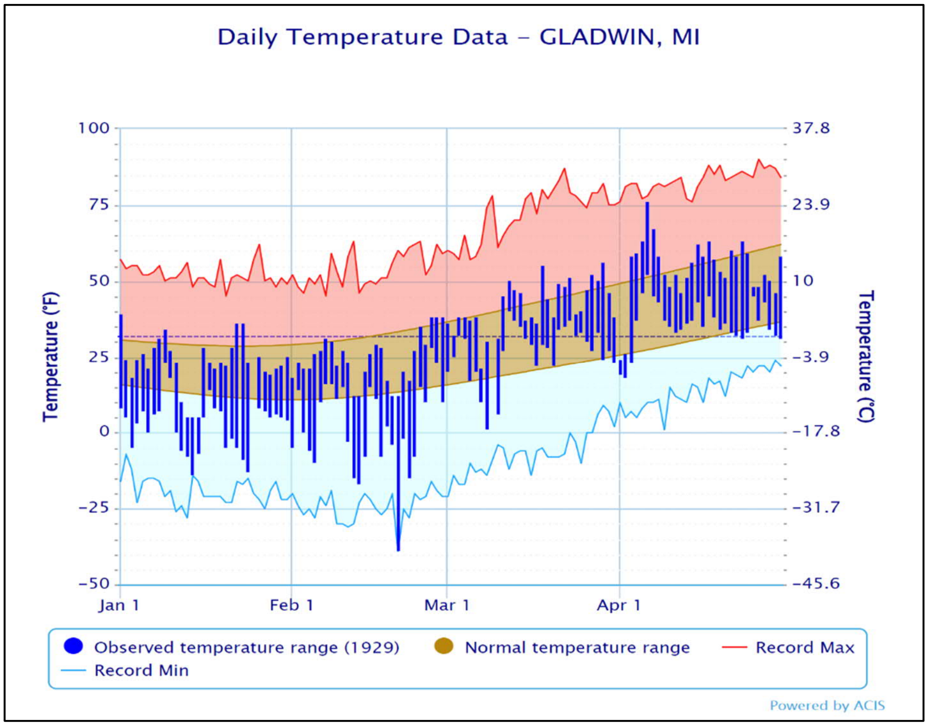


Figure F1-6: Wixom Lake Levels from April 4 through 9, 1929



Source: NOAA 2022a

Figure F1-7: Temperature Data for January through April 1929

F1-2.3 June 1 and 2, 1945 Storm Event

The Gladwin weather station recorded 3.05 and 0.54 inches of precipitation on June 1 and 2, 1945, for a total of 3.59 inches for 2 days. The station recorded 2.17 inches of rain from May 25 to May 29, contributing to antecedent moisture conditions.

Figure F1-8 shows that Wixom Lake started rising at 11:00 a.m. June 2 and returned to normal pool level at 10:00 a.m. as obtained from Boyce Hydro record log information. June 4. The maximum lake level was El. 677.2, (1.4 feet above normal pool) at 1:00 a.m. on June 3, with all six gates open to 7 feet. The powerhouse was at 100 percent flow capacity at 7:00 a.m. on May 28 and remained at full capacity until 7:00 a.m. on June 9.

Figure F1-9 shows the temperature data from April through June 1945. There were no extended periods of freezing temperatures for 2 months before the event.

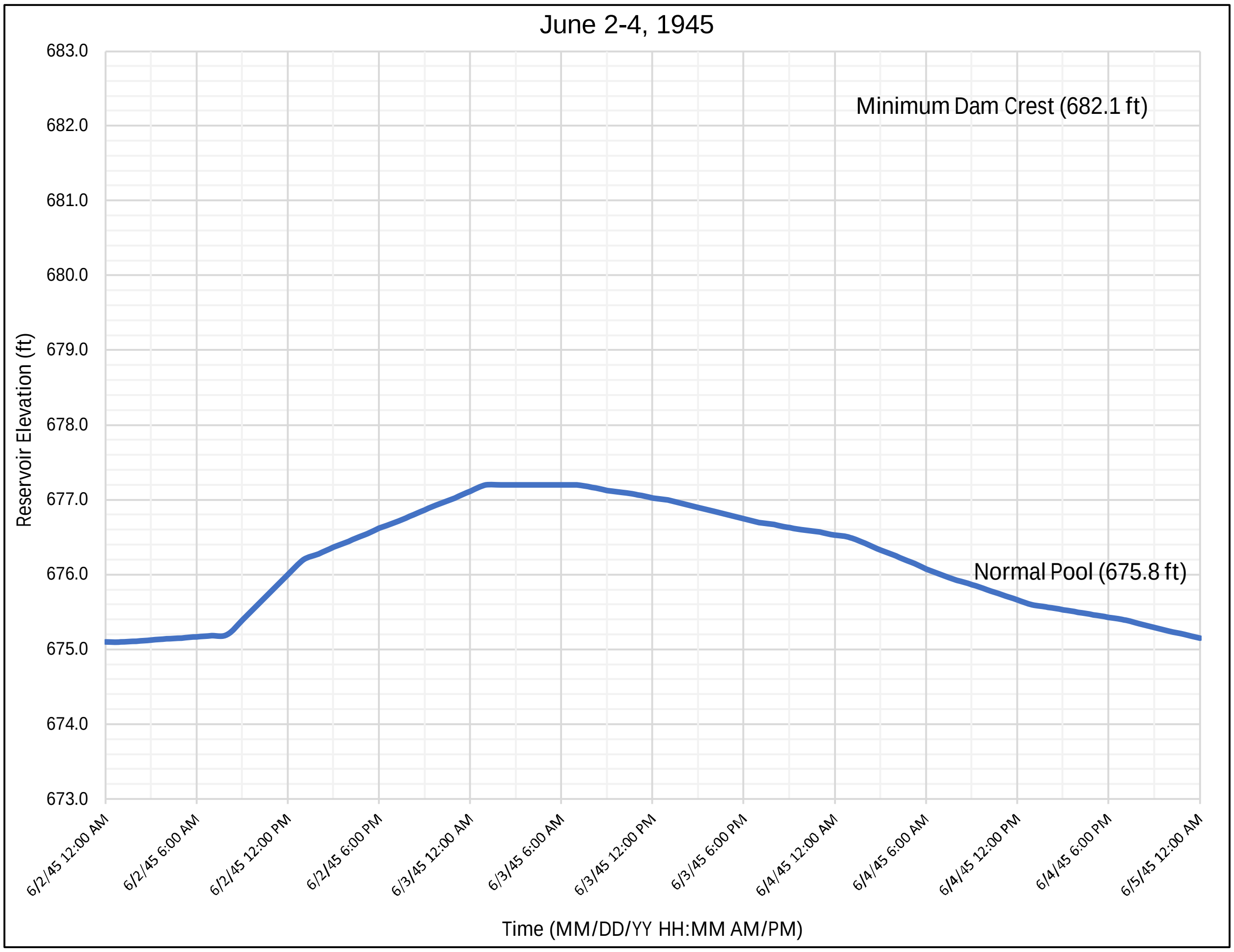
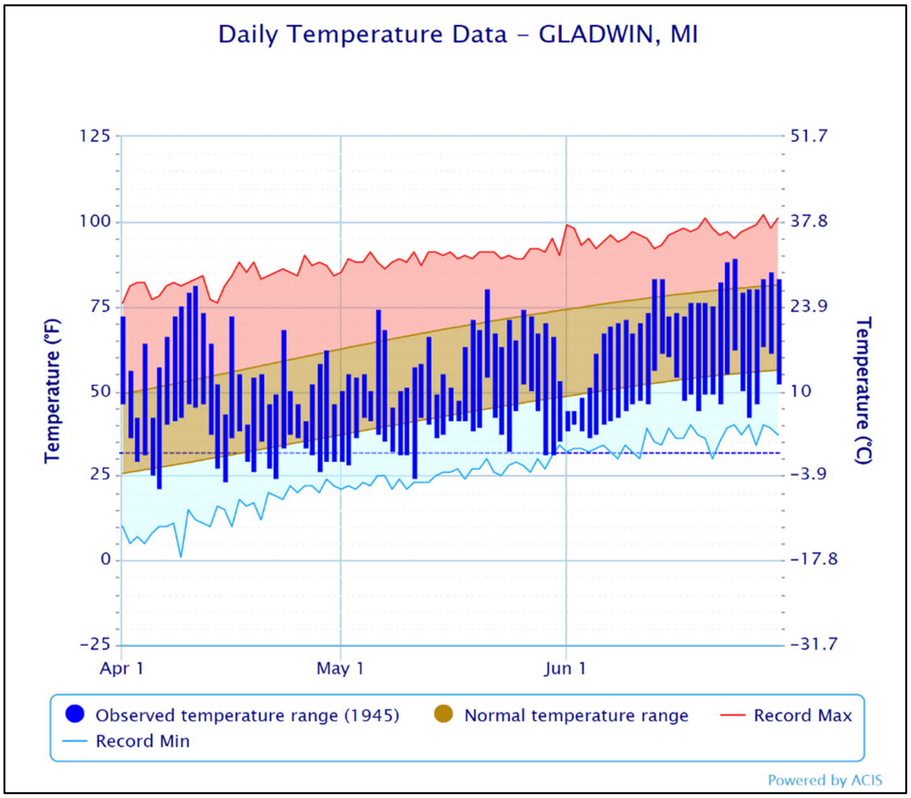


Figure F1-8: Wixom Lake Levels from June 2 through 4, 1945



Source: NOAA 2022a

Figure F1-9: Temperature Data for April through June 1945

F1-2.4 September 9 through 13, 1986 Storm Event

The September 1986 rainfall event was the largest total rainfall event of the seven storms listed in Table F1-1 and was the second highest flood elevation recorded at the USGS Midland, MI stream gage station. The Gladwin weather station recorded 6.83 inches of rain in 3 days (September 9, 10, and 11), with 4.3 inches occurring on September 11. There was 0.12 inch of rainfall in the 2 weeks prior to the event.

Figure F1-10 shows the Wixom Lake levels for the September 1986 event as obtained from Boyce Hydro record log information. The lake levels were primarily controlled by the flows through the powerhouse, so there was no need to significantly open the gates (Gate No. 1 was opened to 5 feet and the other gates were all closed). The Edenville powerhouse was at 100 percent discharge capacity on September 11 at 1:00 a.m., when the lake level was at El. 675.0. The maximum lake level at El. 676.3 was reached at 6:00 p.m. on September 14, with the powerhouse operating at 100 percent capacity.

Later in September, about 1.41 inches of rain were recorded at Edenville Dam, and the lake reached El. 676.3 a second time for the month. The ground had likely become saturated from the precipitation of the earlier storm, and the groundwater table was possibly elevated in much of the watershed. The three gates at the Edenville spillway were opened to 7 feet; Gates 1 and 2 at the Tobacco spillway were opened to 7 and 5 feet, respectively; and the powerhouse was operated at 100 percent capacity to limit the lake rise during the later storm.

Figure F1-11 shows the temperature data from July through September 1986. There were no freezing temperatures for an extended period of time before the event.

The September 9 through 13, 1986 event was one of the larger rainfall events that has occurred in the watershed, and yet the Wixom Lake level for the 1986 storm was only +0.5 foot above normal pool level. As discussed in Section F2-8, further analysis was performed for the September 1986 rainfall event, which was also compared with the May 2020 event.

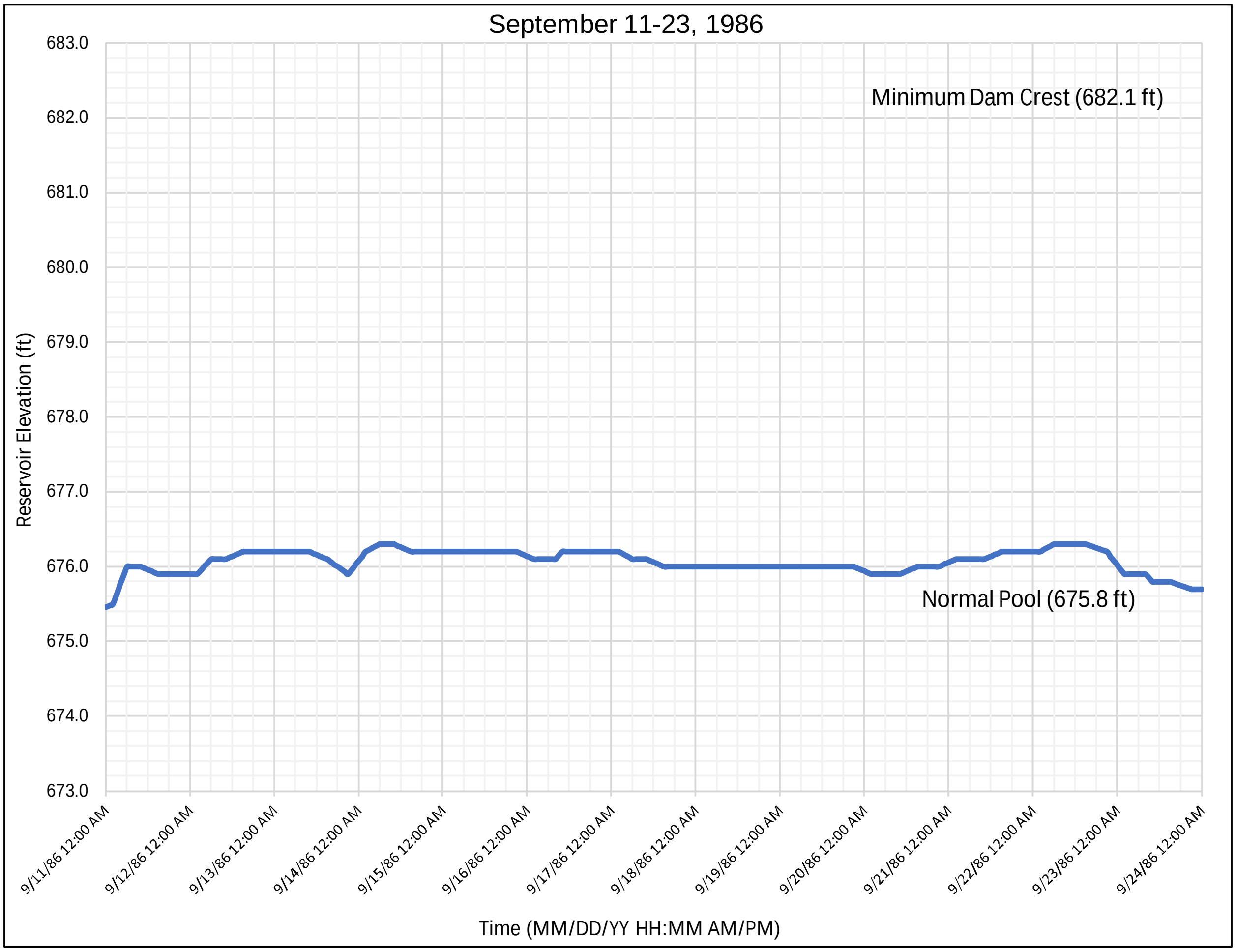
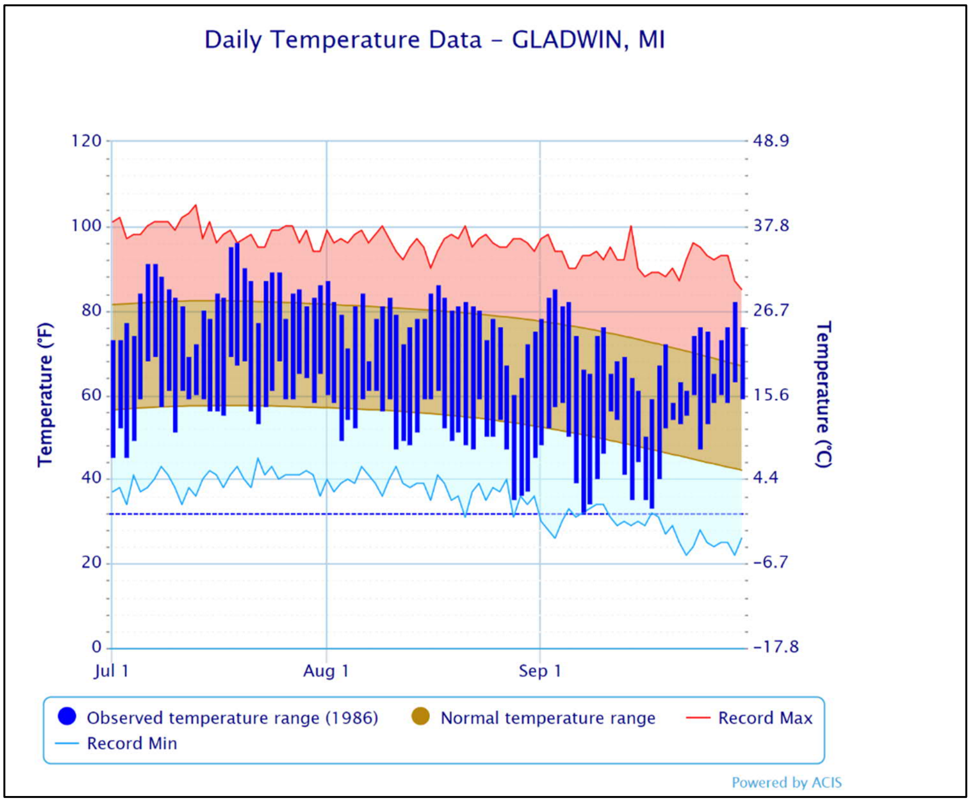


Figure F1-10: Wixom Lake Levels September 11 through 23, 1986



Source: NOAA 2022a

Figure F1-11: Temperature Data for July through September 1986

F1-2.5 April 13 through 15, 2014 Storm Event

The Gladwin weather station recorded 2.78, 0.35 and 0.40 inches of precipitation on April 13, 14, and 15, 2014, respectively, for a total rainfall of 4.53 inches, with most of the rainfall occurring in 48 hours. The antecedent rainfall was 0.05 inch in the previous 7 days, and there was no record of a recent snowfall or snow on the ground. The West Branch weather station recorded 3.42 inches in the same 3 days.

Wixom Lake began to rise above normal pool level on April 13 at 2:00 a.m. and returned to normal pool on April 17 at 11:00 p.m. The maximum lake level was El. 677.2 (1.4 feet above normal pool) on April 14 at 4:00 p.m. All gates were opened to 7 feet at 7:00 a.m. on April 14, and the powerhouse was operating at 100 percent capacity. The time history of Wixom Lake levels for this event is shown in Figure F1-12 as obtained from Boyce Hydro record log information.

Temperature data in Figure F1-13 show the average temperatures at Gladwin station were well below 30°F up until March 27, averaging 18°F. For the next 10 days, the average temperatures increased to 34.5°F, and on April 6 the average temperatures were over 40°F and remained so for the rest of the month, although overnight temperatures were near or below freezing on some days.

It was noted that this event resulted in the failure of Wraco Lodge Dam, which is located just outside of and northwest of the watershed.

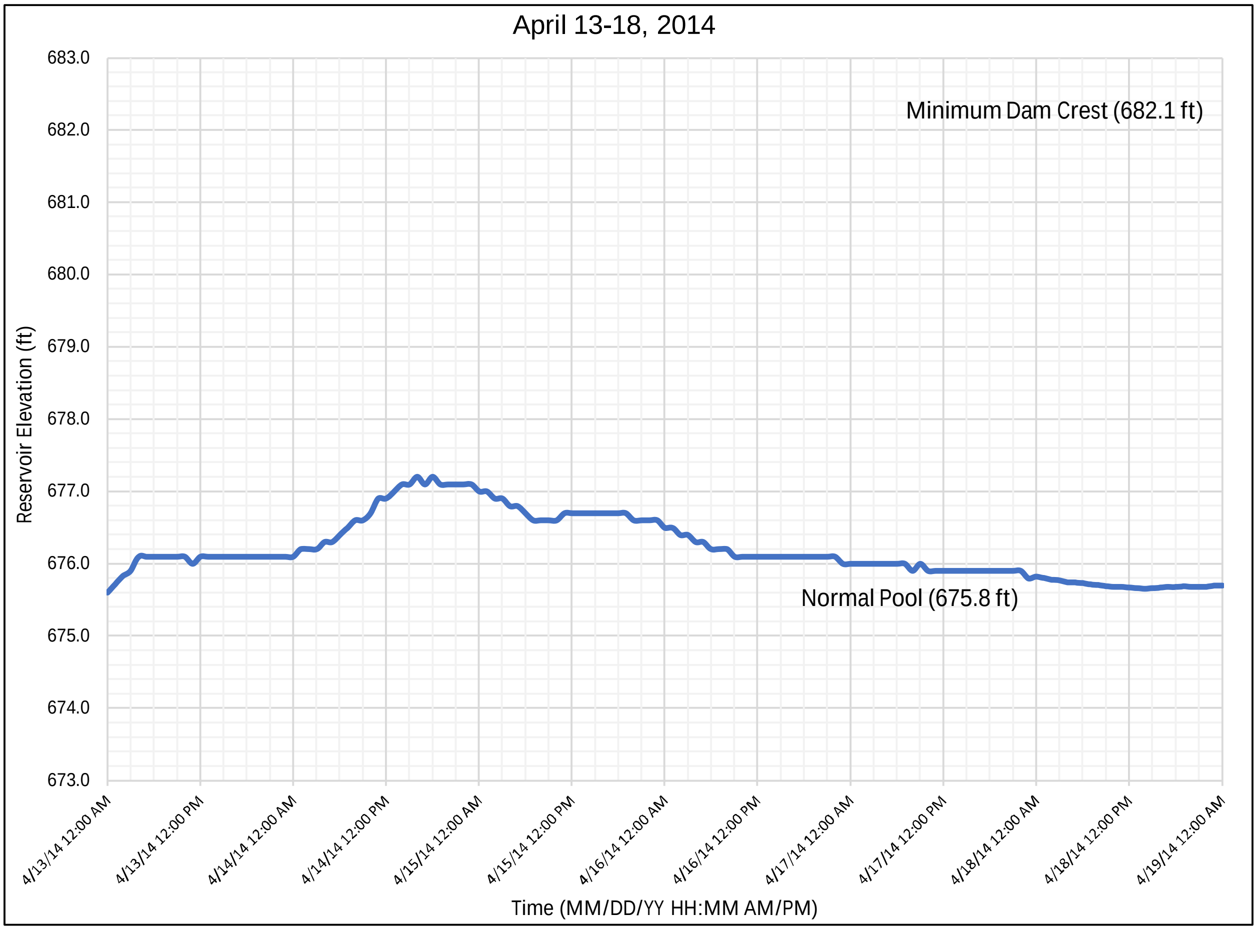
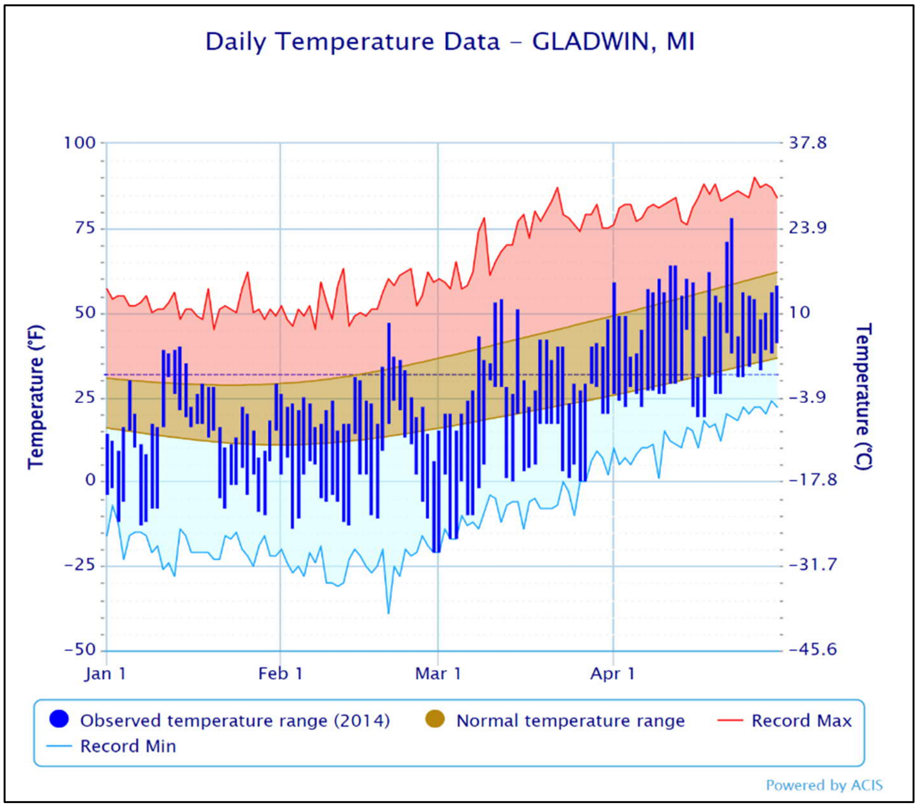


Figure F1-12: Wixom Lake Levels from April 13 through 18, 2014



Source: NOAA 2022a

Figure F1-13: Temperature Data for January through April 2014

F1-2.6 June 22 through 25, 2017 Storm Event

The 4-day precipitation for this 2017 rainfall event recorded at the four Boyce Hydro dams is shown in Table F1-2. The average 4-day rainfall for the basin was 5.03 inches, with the heaviest rainfall occurring downstream of Edenville. This was an approximate 20-year return period event, using the NOAA Atlas 14 (NOAA 2022b). During this event, the Gladwin weather station recorded 5.04 inches over a 6-day period. From June 1 through June 14, there was little rainfall; however, for the 5 days preceding the event 2.44 inches of rain was recorded at the Gladwin weather station. Since most of the rainfall occurred in the first 2 days, the precipitation frequency would be a little higher, perhaps a 25-year return interval.

The maximum lake level shown in Figure F1-14 was reached on June 23 at 10:00 a.m. as obtained from Boyce Hydro record log information, with the powerhouse operating at 59 percent capacity for both units and with none of the gates opened. The dam operations were such that the lake levels were limited to just slightly higher than normal operating guidelines (676.2 – 675.8 = 0.4 foot above normal pool).

The temperature data are shown in Figure F1-15. There were no freezing temperatures for an extended period of time before the event.

Table F1-2: Rainfall Recorded at the Four Dams for the June 2017 Storm Event

Date	Sanford Rainfall	Edenville Rainfall	Smallwood Rainfall	Secord Rainfall
6/22/2017	1.36	1.17	0.73	0.23
6/23/2017	5.60	3.72	2.21	2.93
6/24/2017	0.14	0.79	0.43	0.38
6/25/2017	0.20	0.06	0.05	0.11
Total	**7.30**	**5.74**	**3.42**	**3.65**

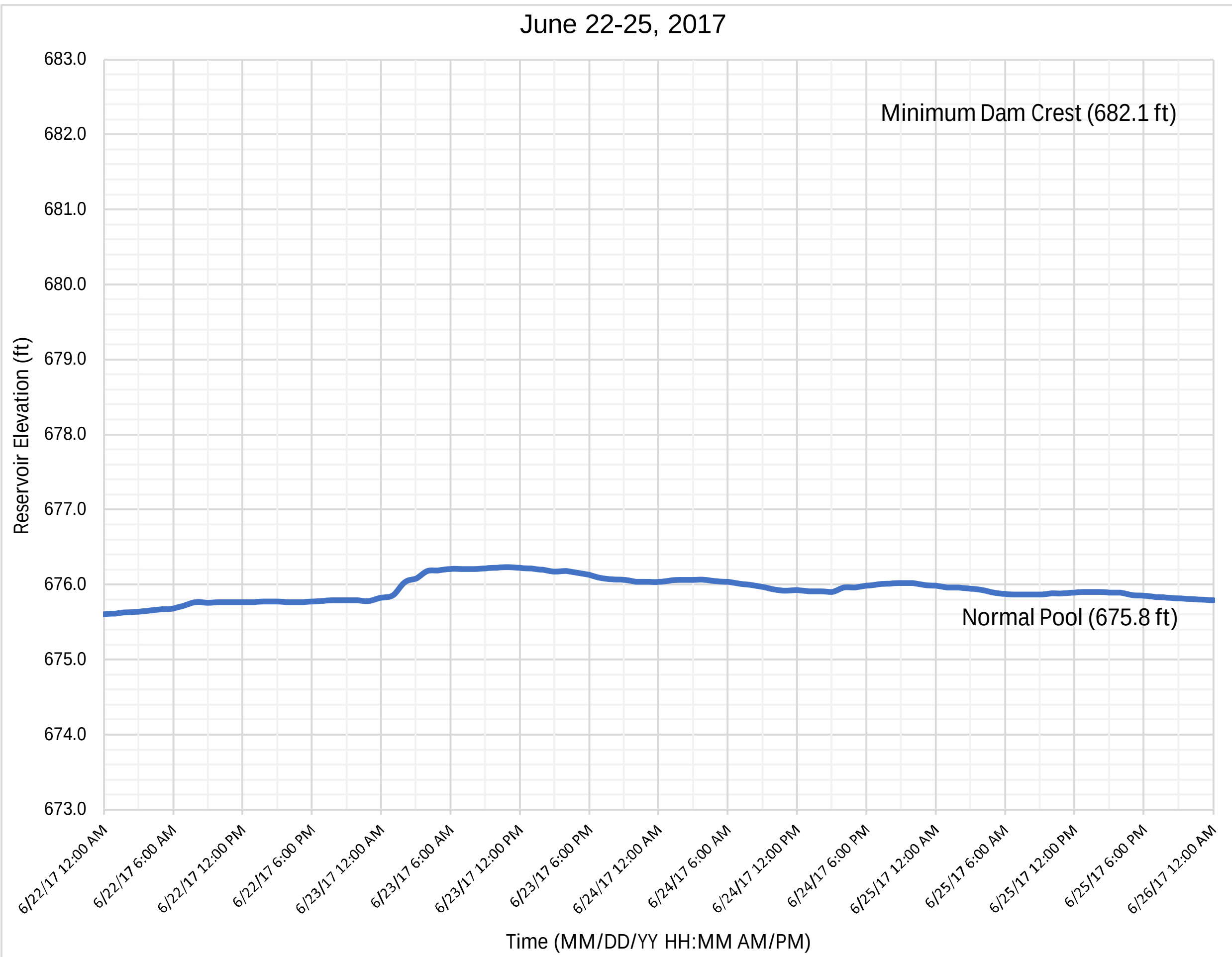
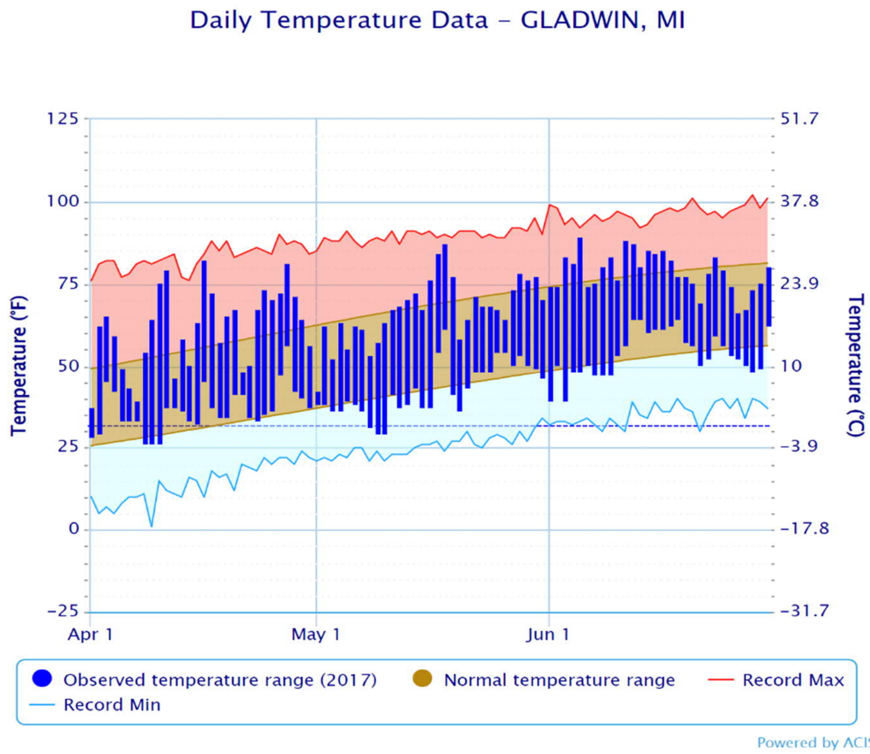


Figure F1-14: Wixom Lake Levels from June 22 through 25, 2017



Source: NOAA 2022a

Figure F1-15: Temperature Data for April through June 2017

F1-2.7 Summary of the Six Short-Listed Historical Storm Events

Historically, most of the basin precipitation occurred during the months of April through October, and the heaviest historical rainfall events occurred in May and June. The four storm events of June 1928, June 1945, September 1986, and June 2017 were characterized by precipitation occurring with some antecedent moisture conditions (rainfall/runoff processes).

The two largest of these events (in total rainfall), June 2017 and September 1986, have estimated 20- and 100-year return periods, respectively, associated with the rainfall/duration using NOAA Atlas 14 (NOAA 2022b). These events occurred outside of the cold season, which ends at the end of May, and the antecedent ground conditions for both these storms were apparently dry, resulting in reduced runoff volume. These two events had the lowest lake levels of the six storms listed in Table F1-3.

The June 1945 event has an estimated 15-year return period and represents a typical storm event in which the initial rainfall raises the antecedent moisture condition of the soil and is followed by more intense rainfall, producing slightly elevated runoff. There were no precipitation records for the June 1928 storm from the Gladwin or Midland weather stations. However, a National Weather Service (NWS) press release stated that there was a long period of relatively steady rainfall, which saturated the ground, followed by a heavy storm event. The 1945 rainfall was typical of a storm event in which the initial rainfall raises the antecedent moisture condition of the soil and then is followed by more intense rainfall and runoff.

The station at Secord, in Gladwin County, reported a 24-hour precipitation of 4.08 inches for the June 1928 storm event (State of Michigan House of Representatives 1932). This storm event would have an approximate return period of 25 years for precipitation; however, two dams on Cedar River, the Chappel and Schulz Dams, both failed, adding to the flow into Wixom Lake.

The operations of the dams during these four events were such that the rises in Wixom Lake levels above normal pool level were 1.4 feet for the June 1928 and June 1945 events, 0.5 foot for the September 1986 event, and 0.4 foot for the June 2017 event.

It is likely that the two storm events in April 1929 and April 2014 both had a portion of the watershed with frozen ground near or below the ground surface, especially in forested areas, including forested swamps. Frozen ground acts as an impermeable zone that produces changes in infiltration, percolation, and runoff, increasing the percentage of the rainfall that becomes runoff. According to the records (State of Michigan House of Representatives 1932), in the April 1929 storm event, heavy rainfall occurred following an ice storm from March 31 through April 1, and the combination of the rainfall and ice storm contributed to a destructive flood in the Tittabawassee Basin on April 6, 1929. Although there was an increase in warmer temperatures 1 week before the April 2014 storm event's rainfall began, it is still likely that the April 2014 event had some frozen ground; it also had the most rainfall of the two April events. The rises in Wixom Lake levels above normal pool level for these two events were 2.5 feet in April 1929 and 1.4 feet in April 2014. Table F1-3 provides a brief summary of the six storm events.

The runoff amounts for the two April storms were influenced by the colder temperatures, which created frozen ground or frost conditions resulting in higher runoff due to impermeable ground conditions. The rise in lake level for the June 26 storm was partially due to the failure of the two upstream dams and the above average rainfall. The June 1945 storm event had 3.59 inches of rain in 2 days preceded by 2.17 inches of antecedent moisture, making the ground conditions more saturated and thus increasing the runoff. Even though the September 1986 rainfall event had 6.83 inches of rain in 3 days, there was only 0.12 inch of rainfall in the 2 weeks before the event; as a result, ground conditions were relatively dry, resulting in higher soil infiltration rates. The highest 4-day rainfall amounts for the June 2017 event occurred in the southern portion of the watershed, and Sanford and Edenville maximum 1-day recorded rainfalls were 5.60 inches and 3.72 inches, respectively. With minimal antecedent moisture and the highest rainfall amounts in the southern portion of the watershed, Edenville did not experience severe flood conditions.

Table F1-3: Brief Summary of the Six Storm Events

Storm Event	Rainfall/Duration	Type of Event	Maximum Depth Above Normal Pool in Feet
June 26, 1928	2.92 inches (in.)/2 days	Rainfall/Runoff + 2 Upstream Dam Failures	+1.4
April 6, 1929	4.46 in./3 days	Winter/Frozen Ground	+2.5
June 2, 1945	3.59 in./2 days	Rainfall/Runoff	+1.4
September 14, 1986	6.83 in./3 days	Rainfall/Runoff	+0.5
April 13, 2014	4.53 in./3 days	Potential Combination	+1.4
June 23, 2017	5.04 in./4 days	Rainfall/Runoff	+0.4

F1-3 Watershed Description

F1-3.1 Topography

The total drainage area contributing to the Sanford watershed is approximately 978 square miles, with a centroid located at 44.0023° N, 84.4952° W. The Sanford watershed consists of the area that contributes to all four lakes (Secord, Smallwood, Wixom Lake, and Sanford) with a basin outlet at Sanford Dam.

An updated watershed delineation was performed as part of this analysis, using USGS 1-meter terrain data. The watershed was divided into 11 subbasins, based on major drainage and reservoir locations, with drainage areas in the subbasins ranging from 35 to 164 square miles. The subbasin delineation and numbering used in recent hydrologic analyses (Ayres 2020) was adopted for this study for comparison and consistency purposes. A map of the watershed is shown in Figure F1-16.

Elevations within the watershed range from about 1,574 feet in the headwaters to 577 feet near the mouth, and the average elevation is approximately 725 feet. The topography of the eastern and southeastern part of the basin is relatively flat, with little relief. The western and northern portions of the watershed have hilly and rolling terrain.

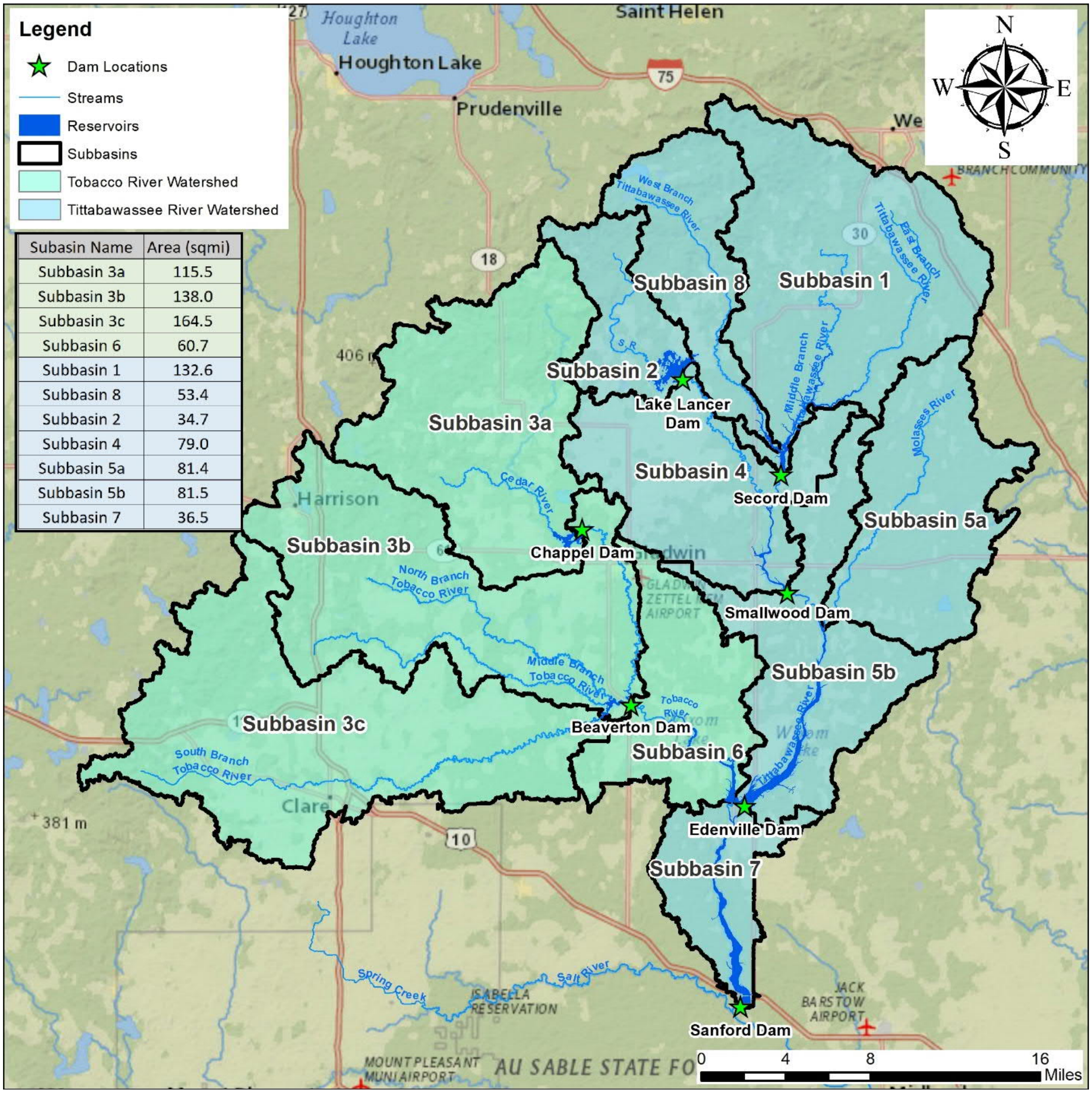


Figure F1-16: Sanford Watershed

F1-3.2 Land Cover Characteristics

Land cover data for the Four Lakes watershed was obtained from the National Land Cover Database (USGS 2016c). A map of the land cover in the Four Lakes watershed is shown in Figure F1-17 Most of the cultivated crops and deciduous forest areas are in the western and central portion of the Sanford watershed, primarily in the Tobacco River watershed. Woody wetlands comprise 30.5 percent of the watershed, and the majority of these wetlands are located in the eastern portion of the watershed, in the Tittabawassee River watershed.

The eastern half of the Sanford watershed contains many interconnected lakes and wetlands that will potentially produce a larger portion of the total basin runoff. Much of the forested land contains mixed conifer and hardwood swamps.

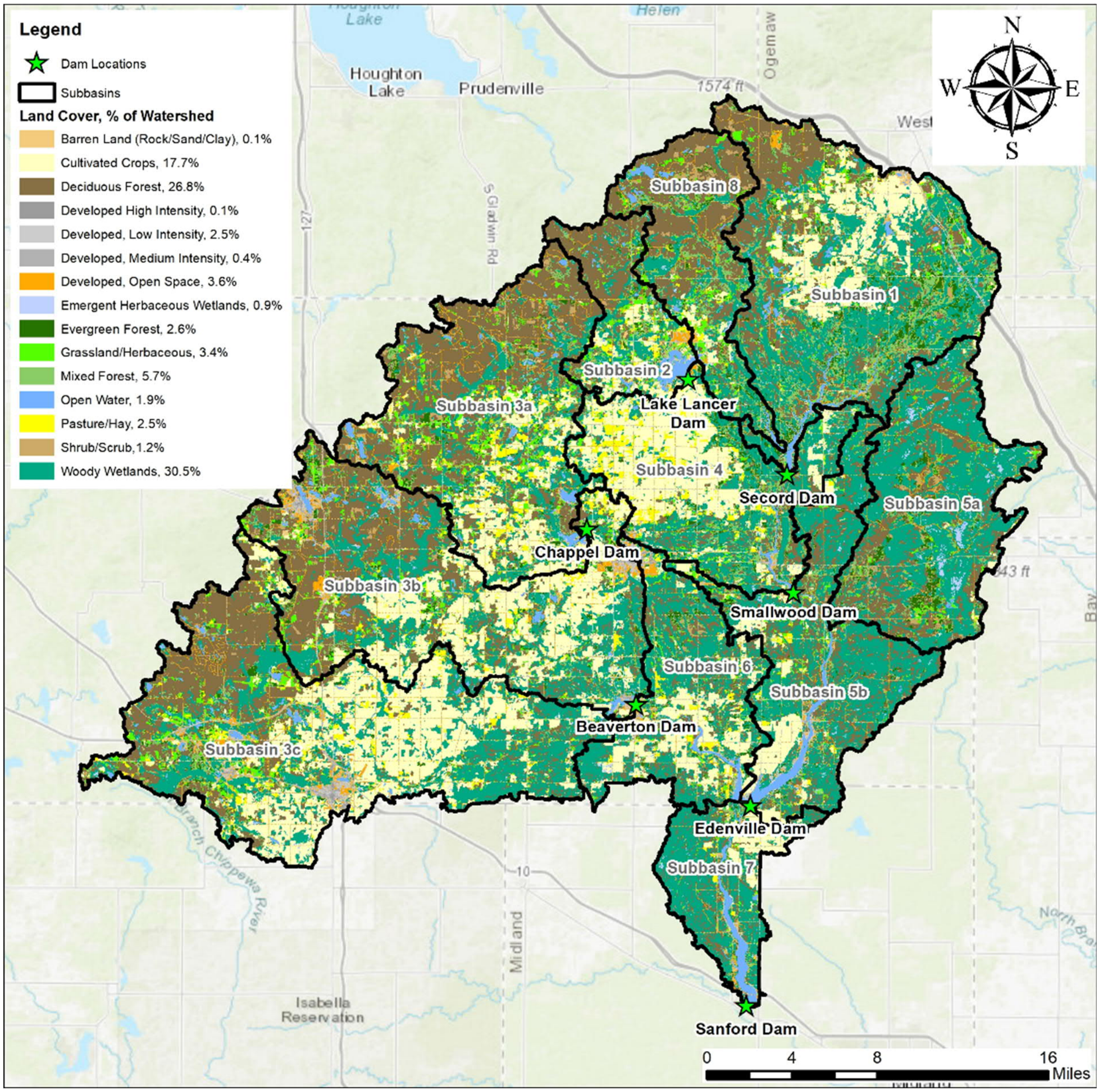


Figure F1-17: Land Cover for the Sanford Watershed

F1-3.3 Soil Characteristics

Soil data were obtained from the Natural Resources Conservation Service (NRCS) Web Soil Survey for the Sanford watershed (NRCS 2017). Hydrologic soil groups were assigned to each soil type based on the dominant hydrologic soil group condition provided in the soil area metadata. A map of the hydrologic soil group classifications for the Sanford watershed is shown in Figure F1-18. Infiltration rates are higher for hydrologic soil group A and less for hydrologic soil group D. An approximate range of loss rates for the different soil groups is summarized in Table F1-4 (NRCS 2021).

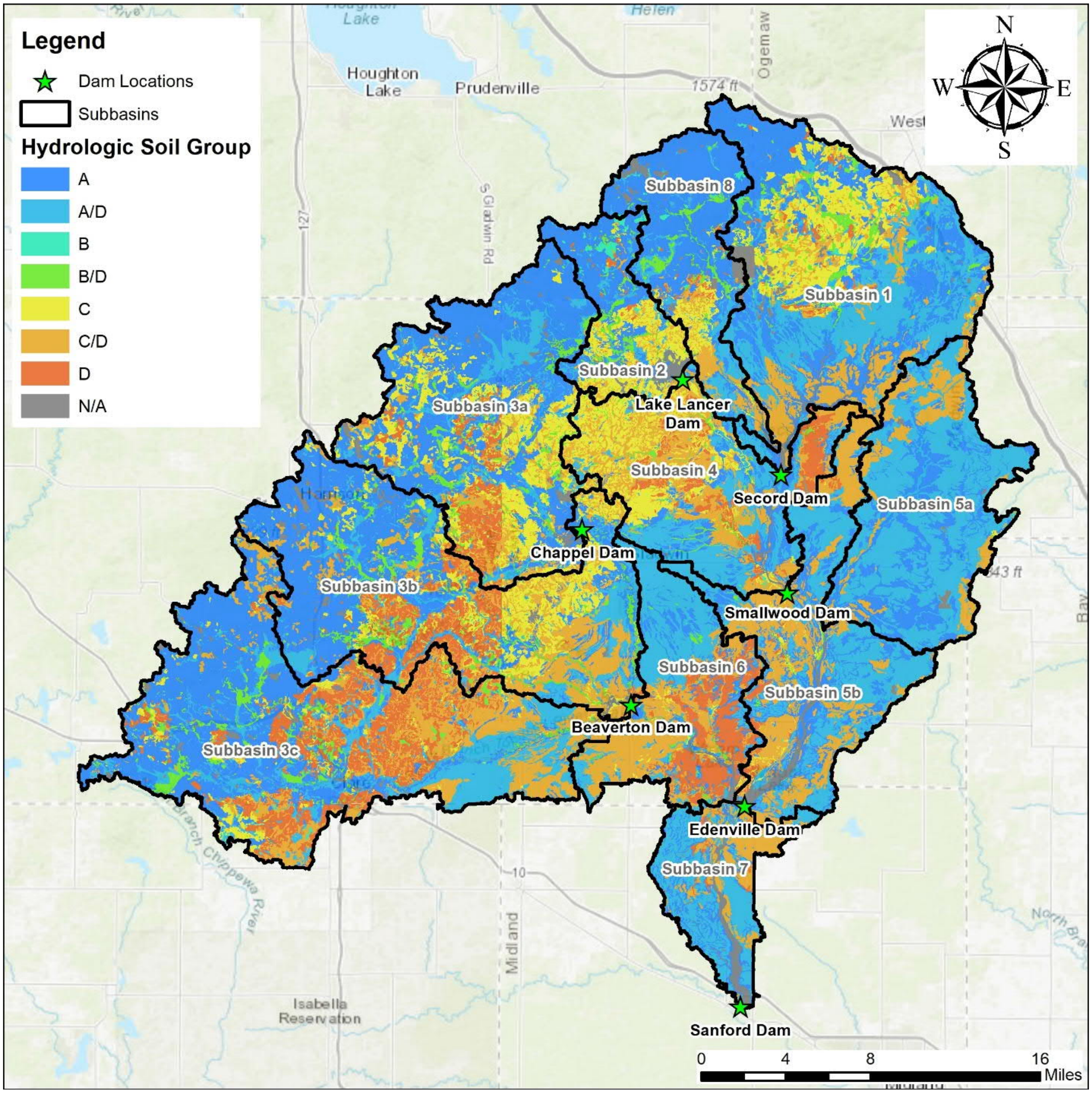


Figure F1-18: Hydrologic Soil Group Map for the Sanford Watershed

Table F1-4: Range of Loss Rates for Soil Group Classification

Soil Group	Description	Range of Loss Rates (in/hr)
A	Deep sand, deep loess, aggregated silts	0.30-0.45
B	Shallow loess, sandy loam	0.15-0.30
C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay	0.05-0.15
D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils	0.00-0.05

F1-4 May 17 through 19, 2020 Rainfall Event

On May 17 through May 19, 2020, the Sanford watershed experienced significant rainfall that eventually contributed to the failure of the Edenville Dam on May 19 and the subsequent failure of Sanford Dam. The characteristics of that rainfall event are described below.

F1-4.1 Precipitation Distribution of May 2020 Event

The IFT contracted with Applied Weather Associates (AWA) to characterize the magnitude, temporal details, and spatial details of the May 17 through 19, 2020 storm event, using their Storm Precipitation Analysis hydrometeorological tool. The tool uses real-time rain gauge observations, optimized Next Generation Weather Radar (NEXRAD) data, and a climatological “base map” approach to produce gridded rainfall at a spatial resolution of one-third square mile and a temporal resolution of 5 minutes. The results of the AWA analyses depicting the spatial variability in total rainfall of the 42-hour May 17 through 19 rainfall event are shown in Figure F1-19. The greatest precipitation, more than 8 inches, occurred 10 to 50 miles east of the Sanford watershed.

In developing the hydrological model for the event, the Sanford watershed was divided into 11 subbasins. The spatial rainfall distribution in the watershed is shown in Figure F1-20. The approximate 42-hour rainfall totals for each subbasin are summarized in Table F1-5. The watershed average total rainfall was approximately 4.29 inches. However, higher total rainfall amounts occurred in the northeastern portion of the watershed, with an average of 4.87 inches contributing to the Tittabawassee River; some locations in the watershed received more than 5 inches of total rain. The Tobacco River basin received an average total rainfall of 3.73 inches.

All 11 subbasins had similar temporal distributions, as shown in Figure F1-21. The temporal distribution can be described as having three phases: the first 12 hours had relatively little rainfall, the next 18 hours had a nearly uniform sustained rainfall intensity of 0.22 inch per hour, and the last 12 hours showed precipitation tapering off.

The observed rainfall from May 17 through May 19, 2020, is estimated to be a 25- to 50-year return period event overall for the Sanford watershed. However, portions of the northeastern section of the watershed that experienced more rainfall had an estimated rainfall return period of about a 100-year rainfall event, based on NOAA Atlas 14; conversely, other portions of the watershed had less than a 25-

to 50-year rainfall. As noted above, most of the precipitation occurred in 18 hours and had a sustained rainfall intensity of 0.22 inch per hour. According to the NWS, a rainfall intensity of 0.22 inch per hour for a duration of 18 hours would be a 25- to 50-year return period (NOAA 2022b).

The return period for rainfall does not necessarily correspond to the flood frequency of runoff (i.e., a 100-year rainfall event does not necessarily correspond to a 100-year runoff event). A precipitation frequency estimate is based on the depth of precipitation at a specific location for a specific duration, which has a certain probability of being equaled or exceeded in any given year. Frequency of flow is based on the magnitude of the flow caused by the rainfall event, which can be different from the frequency of the rainfall event as a result of the percentage of the rainfall that is converted to runoff.

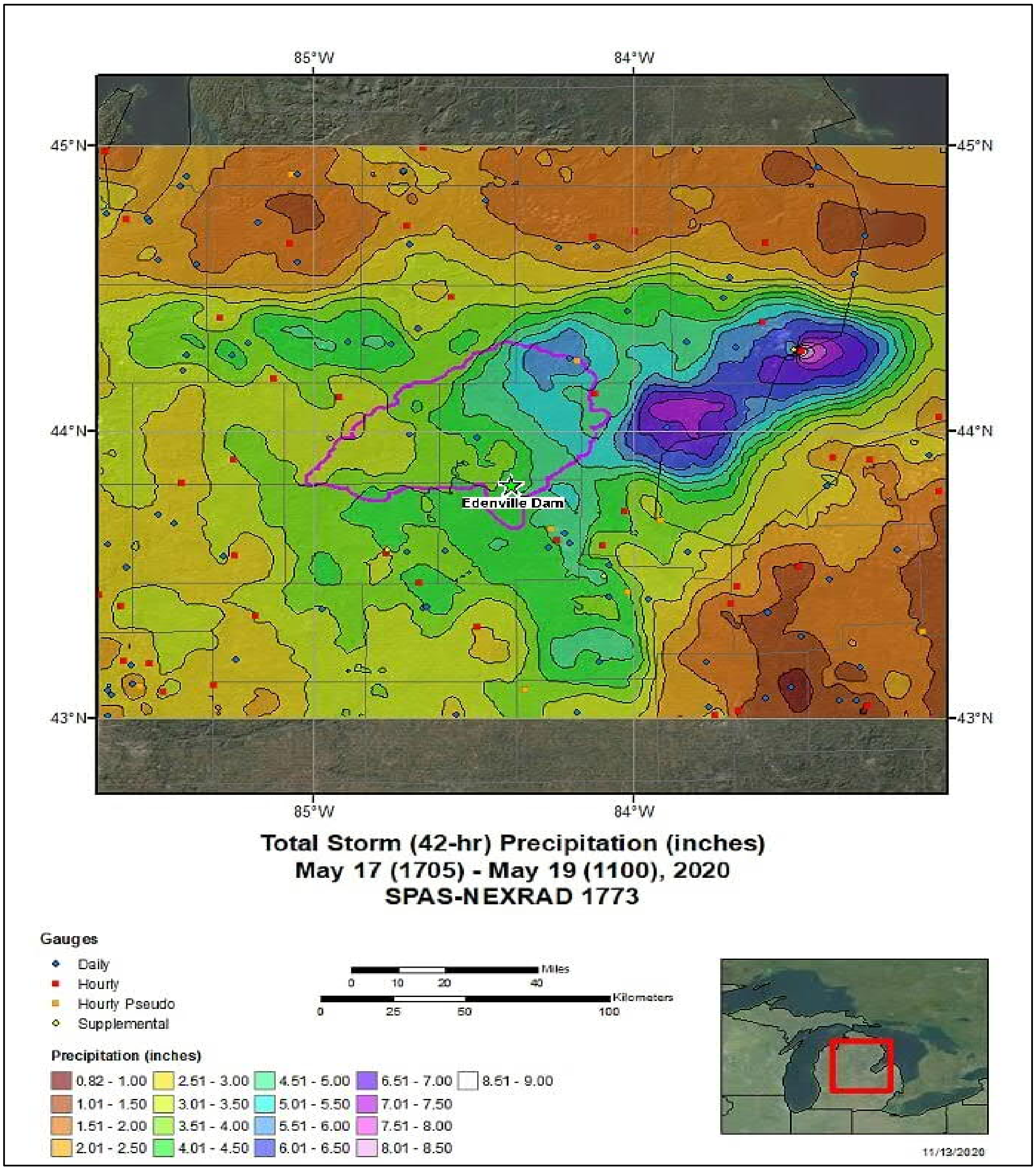


Figure F1-19: Total Storm (42-Hour) Precipitation for May 17 through 19, 2020, in Central Michigan

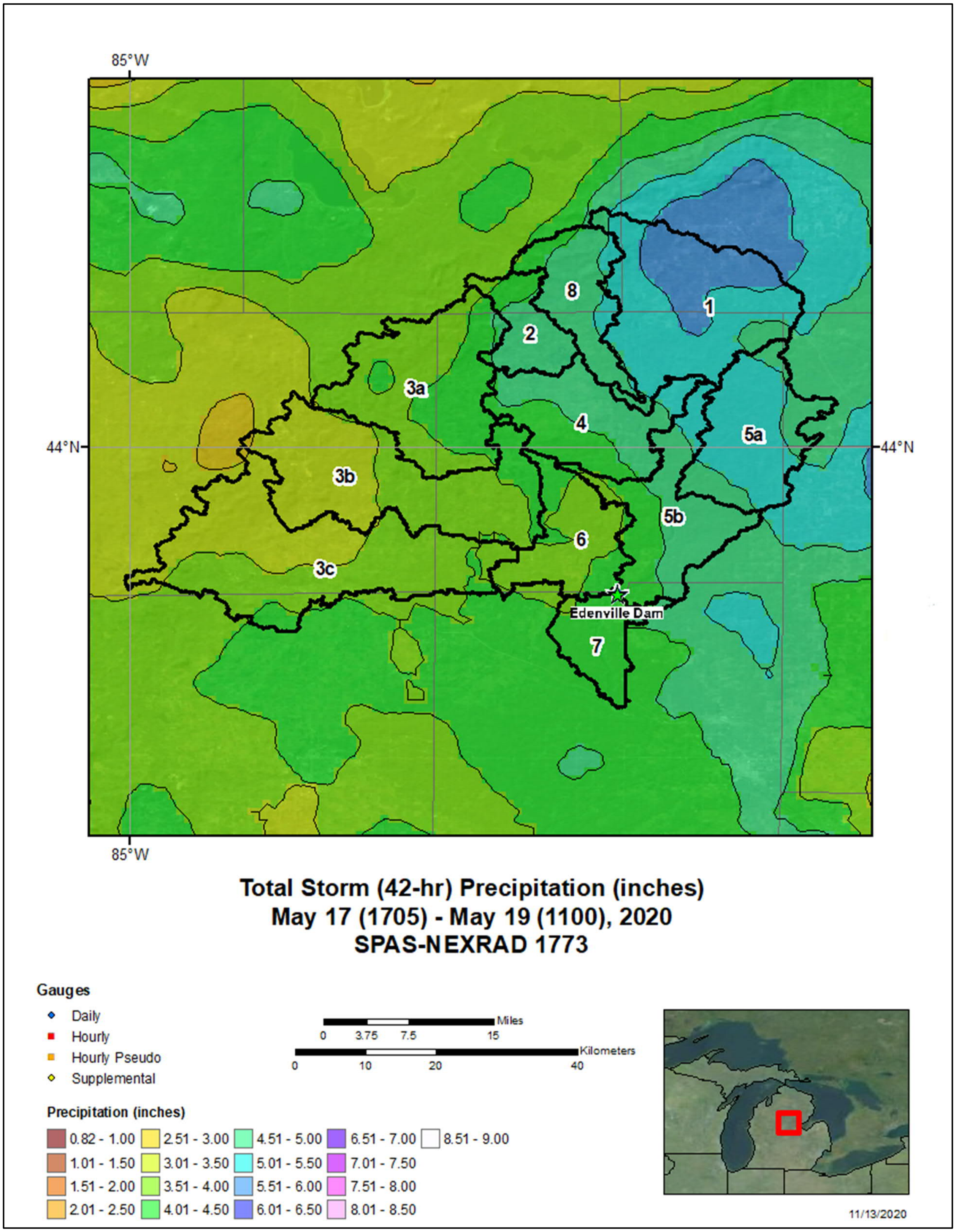


Figure F1-20: Total Storm (42-Hour) Precipitation for May 17 through 19, 2020, for the Sanford Watershed

Table F1-5: Subbasin Total Rainfall Amounts for the May 17 through 19, 2020 Storm Event

Subbasin	Inches of Rainfall in 42-Hours
Subbasin 1	5.36
Subbasin 8	4.78
Subbasin 2	4.57
Subbasin 4	4.58
Subbasin 5a	5.10
Subbasin 5b	4.60
Subbasin 6	3.98
Subbasin 3a	3.96
Subbasin 3b	3.57
Subbasin 3c	3.59
Subbasin 7	4.23
Overall Watershed Weighted Average	4.29

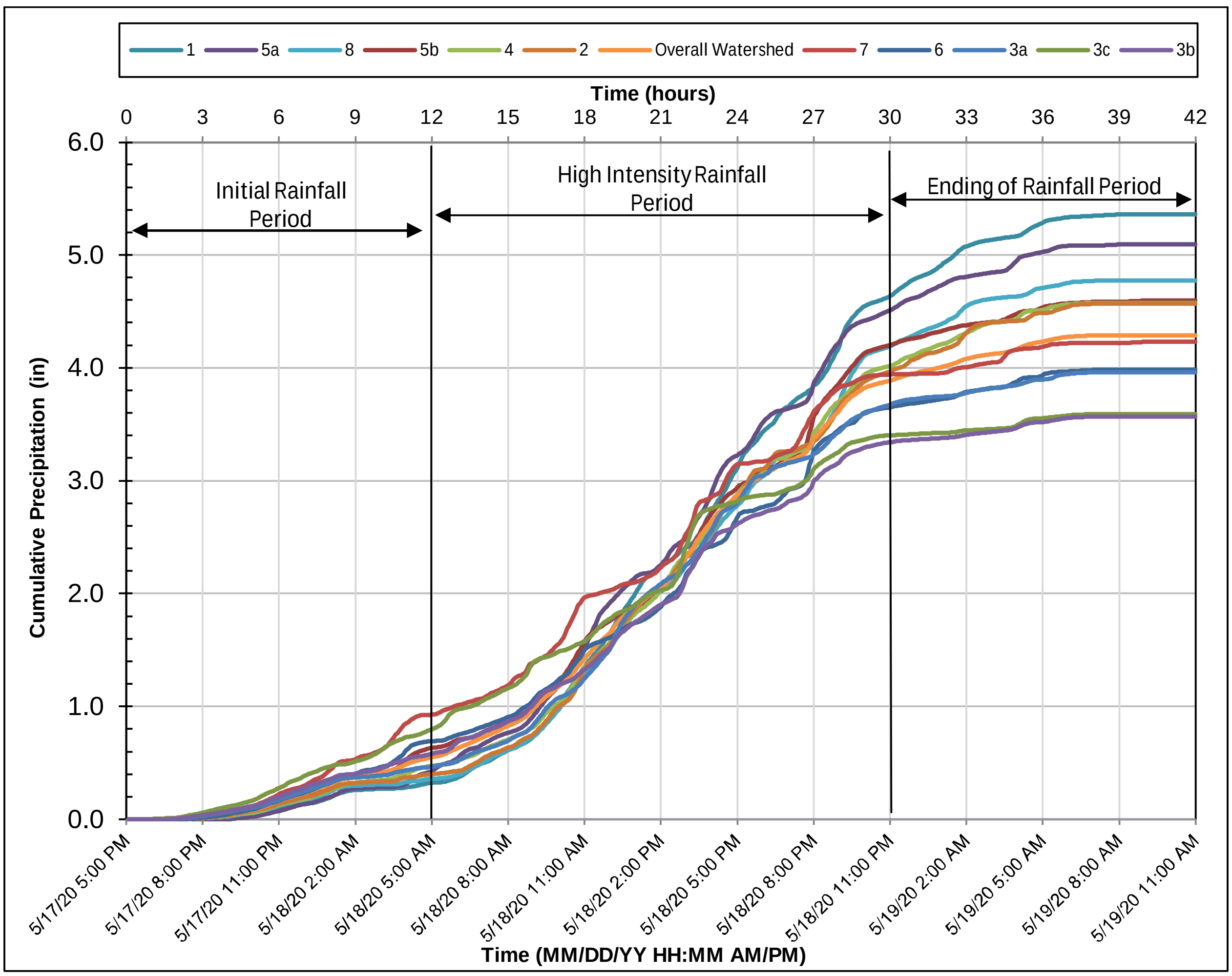


Figure F1-21: 42-Hour May 2020 Storm Cumulative Subbasin Precipitation

F1-4.2 Antecedent Conditions Prior to May 2020 Event

In addition to the precipitation quantity, rainfall intensity, spatial distribution, and temporal distribution, antecedent ground conditions also impact the amount of runoff that will occur from a rainfall event. Prior to the May 17 through 19 storm, there were two rainfall events that added to the antecedent moisture. The first occurred from April 28 through April 30, and the second occurred from May 14 through May 15. The total rainfall amounts recorded at precipitation gages at the four dams and at the NWS) Gladwin Michigan weather station for these two storms is provided in Table F1-6 along with the recorded rainfall for the May 17 through 19 storm event. As a result of these two prior rainfall events, the watershed had sufficient antecedent rainfall to potentially produce fairly wet antecedent ground conditions before the May 17 through 19 event.

Table F1-6: Precipitation for the Rainfall Events Preceding the Edenville Dam Failure

Location	April 28, 2020	April 29, 2020	April 30, 2020	Total 3 Day Rainfall
Sanford Dam	0.06	1.50	0.23	1.76
Edenville Dam	0.08	1.50	0.17	1.75
Smallwood Dam	0.11	0.50	0.35	0.96
Secord Dam	0.10	1.52	0.27	1.89
Gladwin, MI	0.12	1.14	0.70	1.96
Location	**May 14, 2020**	**May 15, 2020**	**May 16, 2020**	**Total 3 Day Rainfall**
Sanford Dam	0.27	0.73	0.00	1.00
Edenville Dam	0.17	0.75	0.00	0.92
Smallwood Dam	0.14	0.40	0.00	0.54
Secord Dam	0.13	0.83	0.00	0.96
Gladwin, MI	0.02	0.65	0.25	0.92
Location	**May 17, 2020**	**May 18, 2020**	**May 19,2020**	**Total 3 Day Rainfall**
Sanford Dam	0.20	2.79	0.16	3.15
Edenville Dam	0.46	3.08	0.32	3.86
Smallwood Dam	0.00	3.69	0.00	3.69
Secord Dam	0.00	5.67	0.23	5.90
Gladwin, MI	0.00	1.93	3.71	5.64

The NWS further characterized the May 17 through 19, 2020 rainfall event as follows (NWS 2020):

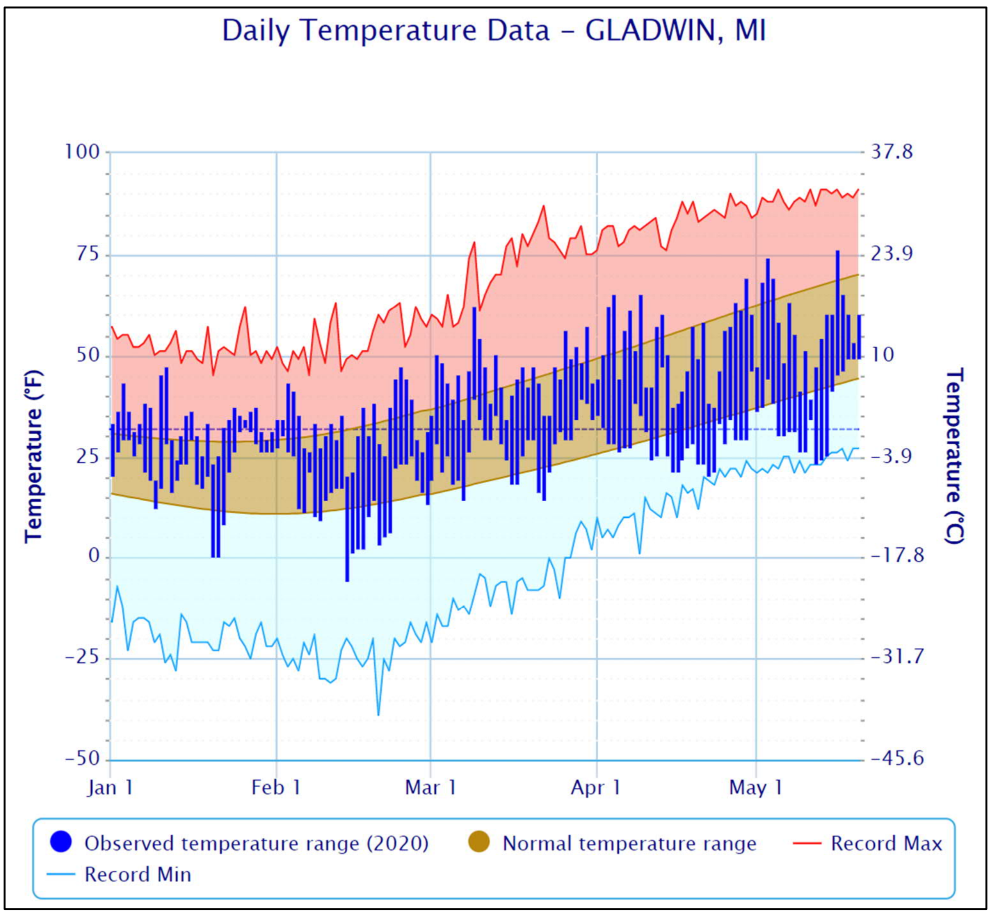
- There was a “stalled low-pressure system and frontal boundary across the southern Great Lakes region that brought record rainfall to southeastern Michigan beginning the morning of May 17th and continuing into the afternoon hours of May 19th.”
- “The stalled low-pressure system that brought 2 to 4+ inches of rain to the region exhibited a few unique meteorological qualities that made it a historical rather than normative event…. This pattern was further enhanced by the slow northeast progression of the early-season Tropical Storm Arthur just off the eastern seaboard.”
- “Despite a lack of extraordinarily high rainfall rates, simple persistence of steady rain resulted in significant rainfall totals.”

F1-4.3 Season and Farming Practices

The Tittabawassee River watershed experienced large temperature differentials through the 2020 “water year” prior to the May 2020 event (shown in Figure F1-22. This had the potential to result in ground freezing and thawing conditions that could impact runoff. Specifically, if the soil was even partially frozen, the soil infiltration could be greatly reduced and therefore the runoff would be increased.

Table F1-7 shows the maximum, minimum (evening), and average temperatures for the Gladwin weather station from April 24 through May 19, 2020.

From May 5 to 14, 2020, the daily minimum temperatures at the Gladwin weather station were near or at record lows, and below freezing for several days. In shaded areas like conifer forests and swamps, the temperatures could be expected to be lower than the recorded temperatures at the Gladwin weather station. In addition, shading of the forest floor can reduce the rate of frost thaw occurring within the soil. These influences could have resulted in either the persistence of ground frost within the soil profile or some surface ground freezing, either of which could have been sufficient to perch rainfall infiltration and produce a saturated condition at the surface that would increase the runoff. Temperatures after May 14 increased to above 32 degrees for the rest of the month.



Source: NOAA 2022a

Figure F1-22: Gladwin Daily Temperatures before the May 2020 Event

Table F1-7: Temperatures Recorded at Gladwin Weather Station from April 24 through May 19, 2020

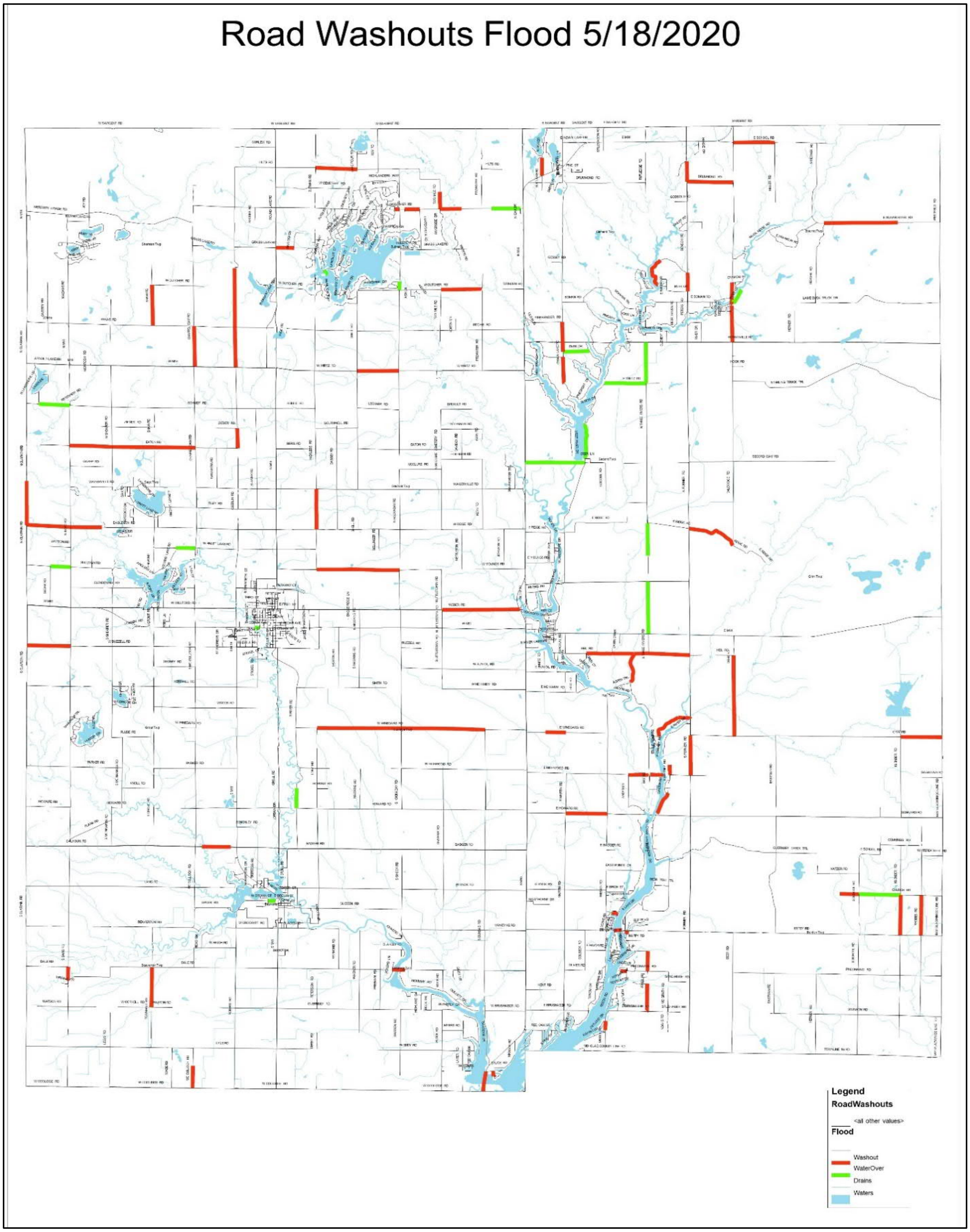
Date	Temperature		
	Maximum	Minimum	Average
4/24/20	46	33	39.5
4/25/20	56	28	42.0
4/26/20	57	34	45.5
4/27/20	63	29	46.0
4/28/20	61	29	45.0
4/29/20	69	29	49.0
4/30/20	60	46	53.0
5/1/20	47	36	41.5
5/2/20	68	37	52.5
5/3/20	74	44	59.0
5/4/20	69	38	53.5
5/5/20	58	30	44.0
5/6/20	48	30	39.0
5/7/20	63	31	47.0
5/8/20	55	31	43.0
5/9/20	41	26	33.5
5/10/20	51	26	38.5
5/11/20	39	34	36.2
5/12/20	47	33	35.0
5/13/20	54	24	39.0
5/14/20	60	25	42.5
5/15/20	60	41	50.5
5/16/20	76	45	60.5
5/17/20	65	46	55.5
5/1820	60	49	54.5
5/19/20	53	49	51.0

In addition to seasonal weather, seasonal agricultural activities could have affected the response of the watershed, especially given that about 20 percent of the Sanford watershed is composed of agricultural land use. Although a number of agricultural activities are occurring in the spring months, one noteworthy factor in the context of hydrology is field preparation and tillage, the latter in particular being widely understood to increase runoff. Depending upon how much of the agricultural land was tilled at the time of the event, it could conceivably have resulted in higher runoff than would have been experienced in the absence of tillage. However, the actual magnitude of this effect was not determined. Moreover, the trend in farming is toward generally more sustainable agriculture, which includes reduced or more conservative tillage operations.

F1-4.4 Flooding Upstream of Edenville During the May 2020 Event

The rainfall and corresponding flooding event had significant impacts in the watershed on roads and structures upstream of the Edenville Dam. On May 18, 2020, heavy rain as well as rising floodwaters made numerous roads in mid-Michigan impassable. Flooding was widespread throughout Gladwin County prior to the May 19 dam failure. Figure F1-23 shows the widespread roadway flooding in Gladwin County. The NWS Gaylord office issued a Flash Flood Warning for south-central Gladwin County along the Tittabawassee River below Secord Dam on May 18 (Dolinar 2020).

Figure F1-24 shows the Sugar River at the M-30 highway bridge just upstream of Smallwood Lake experiencing high flows and inundating trees. Figure F1-25 shows floodwaters overtopping a roadway upstream of Secord Dam on May 18, 2020.



Courtesy Mr. Robert North, the Gladwin County Emergency Management Director

Figure F1-23: Gladwin County Roads Flooded on May 18, 2020

Figure F1-24: Sugar River at M-30 Highway Bridge on May 18, 2020

Figure F1-25: Local Roadway Upstream of Secord Dam on May 18, 2020

F1-5 Model of the May 2020 Event

F1-5.1 Hydrologic and Hydraulic Model Development

Hydrologic and hydraulic models were created using the U.S. Army Corps of Engineers (USACE), Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS) software programs to simulate the May 17 through 19, 2020 storm event and lake levels (USACE 2016, 2019). Rainfall data were input into a HEC-HMS model based on the May 2020 temporal rainfall distributions provided by AWA. Infiltration and unit hydrograph data for the watershed subbasins were initially calculated based on available topographic (USGS 2016a, 2016b), land cover (USGS 2016c), and soil data (NRCS 2021). Subbasin outflow hydrographs from the HEC-HMS model were input into a one-dimensional/two-dimensional (1D/2D) HEC-RAS model to simulate the time-series of the reservoir gate operations and to simulate reservoir inflows, outflows, and water levels. The reservoirs were modeled in HEC-RAS as storage areas using level pool routing. Elevation-storage information was estimated for the reservoirs, using available light detection and ranging (LiDAR) data (USGS 2016a, 2016b) that captures elevation data for approximately normal pool and above. Some extrapolation was performed to estimate elevation-storage values below normal pool.

The dams were modeled as storage area connections, and the spillways were modeled using the radial gate function. Modeling the gates, using the radial gate function, allows the HEC-RAS model to simulate the spillway function as either weir flow or gate flows based on the reservoir elevation and gate openings through the simulations. The spillway and gate parameters used in the model simulations were obtained from *Draft Discharge Rating Curves (Secord, Smallwood, Edenville and Sanford Projects) Four Lakes Task Force (FLTF)* (GEI 2020). The spillway gate openings for the May 17 through 19, 2020 storm event were input into the HEC-RAS model as a time-series based on the spillway gate operations provided by Boyce Hydro through handwritten logs and interviews.

F1-5.2 Hydrologic and Hydraulic Model Calibration

The hydrologic model (HEC-HMS) set up for the watershed was calibrated to simulate the watershed conditions for the May 17 through 19, 2020 storm event (i.e., calibrated the unit hydrograph and the hydrologic loss parameters). The HEC-HMS model is a lumped parameter model that can simulate the hydrologic processes of a dendritic watershed system. The hydrologic model was iteratively calibrated by adjusting infiltration, impervious area, and unit hydrograph parameters so as to get reservoir water levels in the hydraulic model to approximately match the documented reservoir water level recordings.

The model calibration was performed by starting with the most upstream watershed, in this case the Secord watershed, and adjusting the watershed parameters based on matching the lake levels recorded at the Secord Dam. The flow was then routed to Smallwood Dam through the Tittabawassee River. The lake levels, with an estimated peak lake level of El. 710.3, and the timing of the peak lake level at Smallwood were used to adjust the watershed parameters that were contributing to the Smallwood Dam inflow. The flow to Smallwood Dam included the flow from Lake Lancer on the Sugar River. Source: Harrison via Storyful 2020

Figure F1-26 shows flow going over the auxiliary spillway that was used to check flow depth at the spillway, and gate openings were also used to aid the calibration process. Outflow from Smallwood Dam then gets modeled as inflow into Edenville Dam. Figure F1-27 shows the Tobacco spillway on May 21, 2020.

Source: Harrison via Storyful 2020

Figure F1-26: Smallwood Dam Showing Flow over the Auxiliary Spillway

Photograph courtesy of EGLE.

Figure F1-27: Tobacco Spillway – May 21, 2020

Figure F1-28 and Figure F1-29 show the gate openings at the Edenville and Sanford Dams, respectively. The photographs were taken by the IFT team during a site visit in September 2020.

Figure F1-28: Edenville Spillway – September 2020

Figure F1-29: Sanford Spillway – September 2020

For the Tobacco River watershed, the basins that contribute runoff to the Beaverton and Chappel reservoirs were estimated from the hydrograph that was recorded at the USGS stream gage on the Tobacco River at Beaverton below Beaverton Dam.

To estimate the inflow to Wixom Lake, the flow from the USGS stream gage at Beaverton was routed through the Tobacco River and added to the routed flow through the Tittabawassee River from the Smallwood Dam; this flow was also added to the flow from other contributing subbasins to Wixom Lake. Therefore, the existing watershed parameters that produce the lake levels depict the watershed at that time. If portions of the watershed had frozen ground, the parameters would have been adjusted to obtain the lake levels that were recorded given the frozen ground. The model was calibrated such that the watershed parameters present specifically during the May 2020 storm event are captured by the model, and all characteristics that influenced the response of the watershed are therefore factored into the calibration. Figure F1-30 shows the Wixom Lake levels for the May event as obtained from Boyce Hydro record log information and estimated beyond available log information. The peak inflow to Wixom Lake was estimated to be 24,500 cfs and the estimated lake elevation was 681.3 feet at the time of failure.

However, it should be noted that the accuracy of the model is limited by the modeling assumptions incorporated in the computer software used for the modeling, the resolution and accuracy of the available rainfall data, the resolution and accuracy of the land characteristics data available for the watershed, and the data available for calibrating the model.

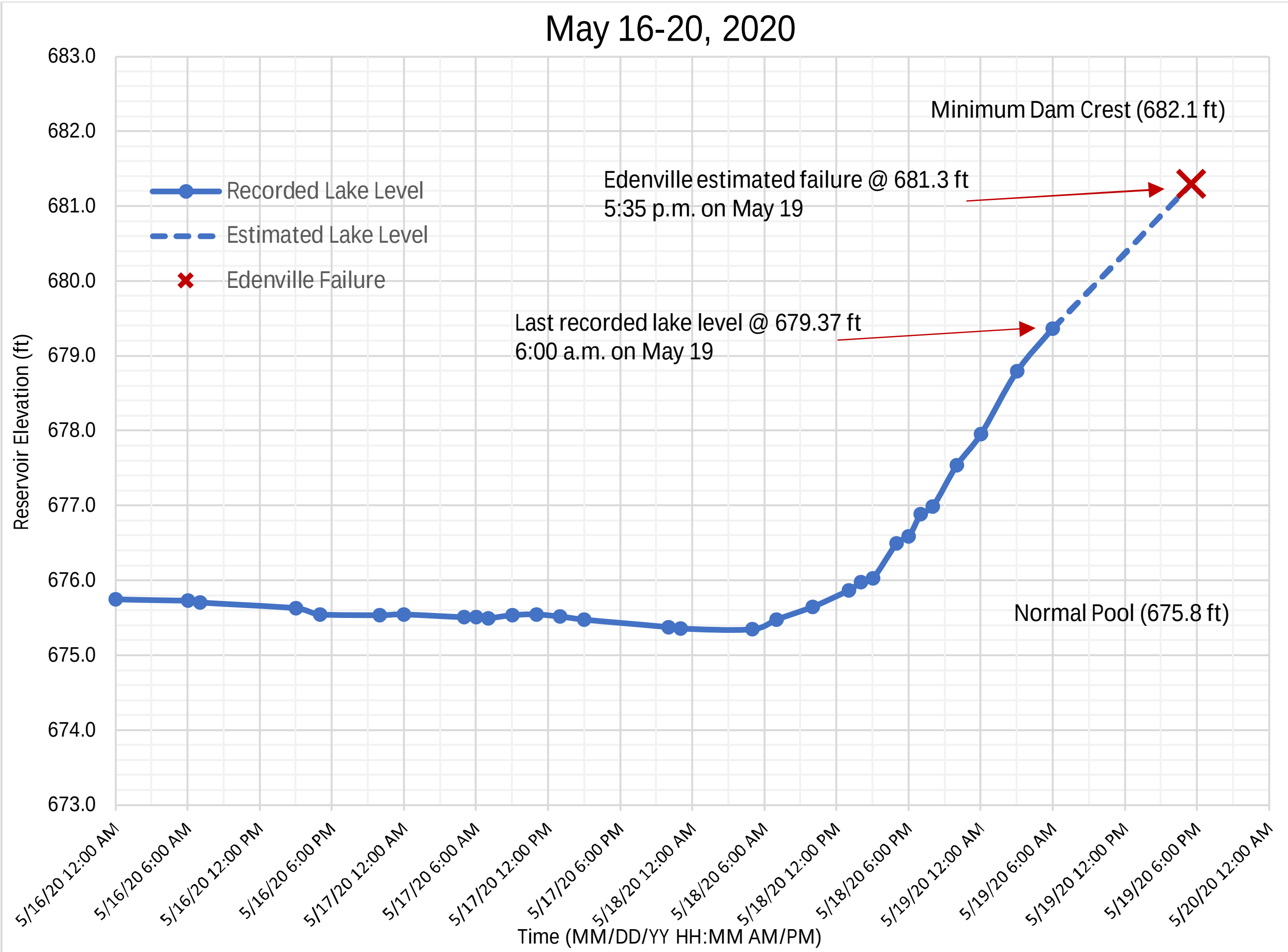


Figure F1-30: Wixom Lake Levels from May 16 to 19, 2020

F1-5.3 Comparison of May 2020 and September 1986 Flood Events

F1-5.3.1 September 9 through 13, 1986 Rainfall Event

Even though the Wixom Lake level for the 1986 storm was only +0.5 foot above normal pool, the 1986 event was one of the larger rainfall events that has occurred in the watershed. To understand why the flooding impacts were significantly greater during the May 2020 storm compared to the September 1986 storm, the 1986 rainfall event underwent further analysis.

A previous study (Mead & Hunt 1994) had stated:

> "Antecedent Moisture conditions (saturated or dry) can have a strong influence on runoff. The degree of saturation would be expected to be especially important in watersheds such as the upper Tittabawassee, where wetlands are common. In dry conditions such as those that preceded September 9 through 13, 1986 storm, wetlands can reduce runoff volumes by more or less permanently storing precipitation. However, once this storage capacity is filled, wetlands can act as impervious surfaces, returning almost all subsequent rainfall as runoff."

AWA was contracted by the IFT to characterize the magnitude, temporal details, and spatial details of the 1986 storm to further evaluate the event. Figure F1-31 shows the results of the analyses depicting the spatial variability of the 96-hour precipitation totals. Figure F1-32 shows the temporal variation of the rainfall for the 11 subbasins. The maximum rainfall intensities for the September 1986 event varied across the subbasins from about 0.08 inch per hour to 0.16 inch per hour, compared to the 0.22 inch per hour across all subbasins for the May 2020 event. NEXRAD radar data were not available for the 1986 storm event, and therefore AWA used real-time rain gage observations and a climatological "base map" approach to produce the gridded rainfall amounts.

Table F1-8 shows the September 1986 rainfall values for the 11 subbasins. The basin weighted average is 7.06 inches for the event. This would be about a 100-year return interval for rainfall, which is considerably larger than the May 2020 rainfall event. In general, the lightest rains occurred in the northeastern portion of the basin—Subbasins 1, 8, 2, 4, and 5a.

The September 1986 rains were triggered by a nearly stationary front that stretched east-west across Central Lower Michigan. There was only 0.04 inch of rain in the 10 days prior to the event, so there was little antecedent moisture. The heaviest rainfall, 11.78 inches in 2 days, occurred outside the Sanford watershed, south of Sanford in Midland, MI. The spatial distribution of the rainfall shown in Figure F1-31 also indicates the heaviest rainfall occurred south of the Sanford watershed, resulting in significant flooding in Midland MI.

From September 22 through 24, 1986, an additional 1.41 inches of rain was recorded at Edenville Dam, and the lake reached El. 676.3 for a second time that month. The ground in much of the watershed had likely become saturated from the earlier storm.

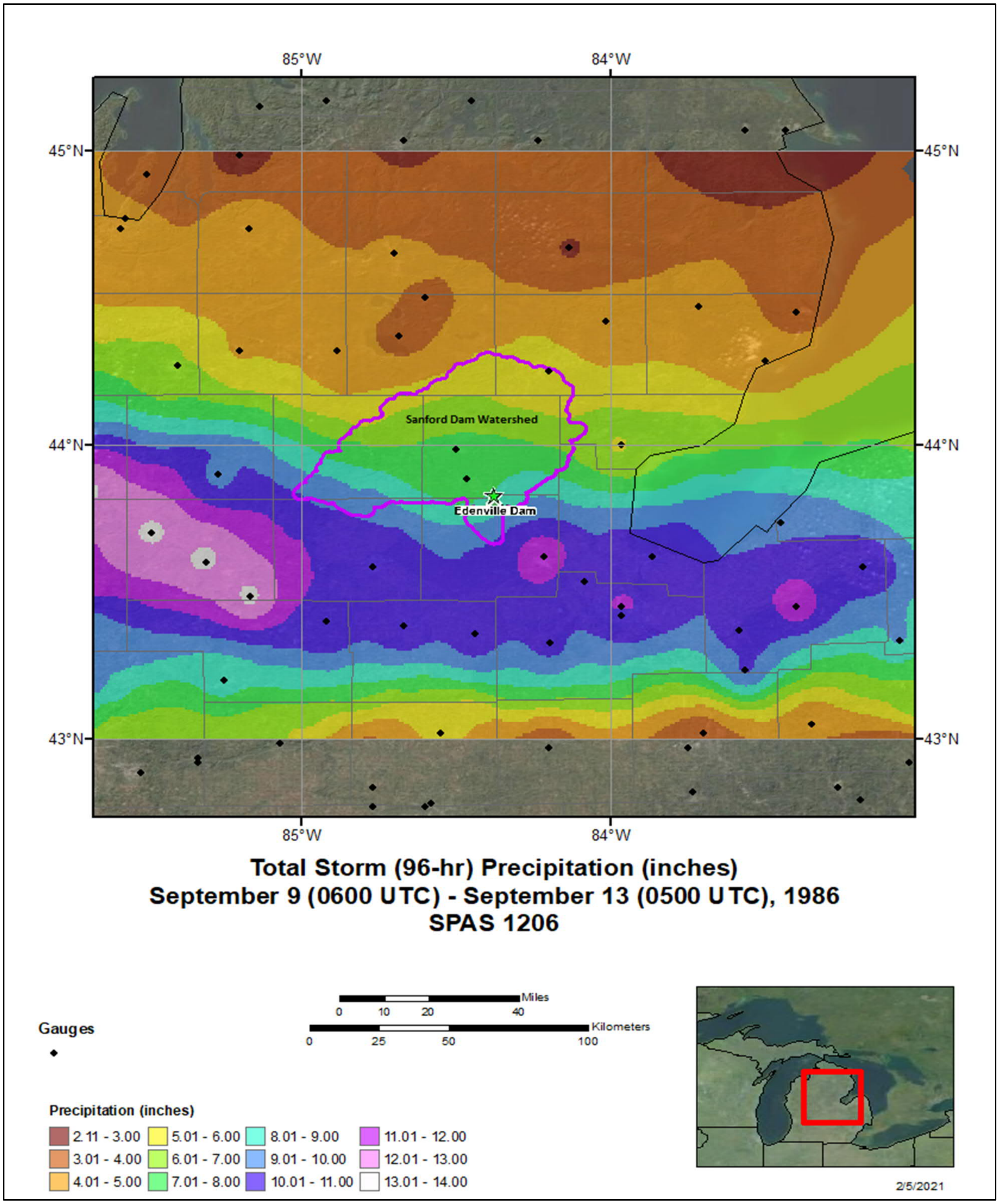


Figure F1-31: Total Storm (96-Hour) Precipitation for September 9 through 13, 1986, in Central Michigan

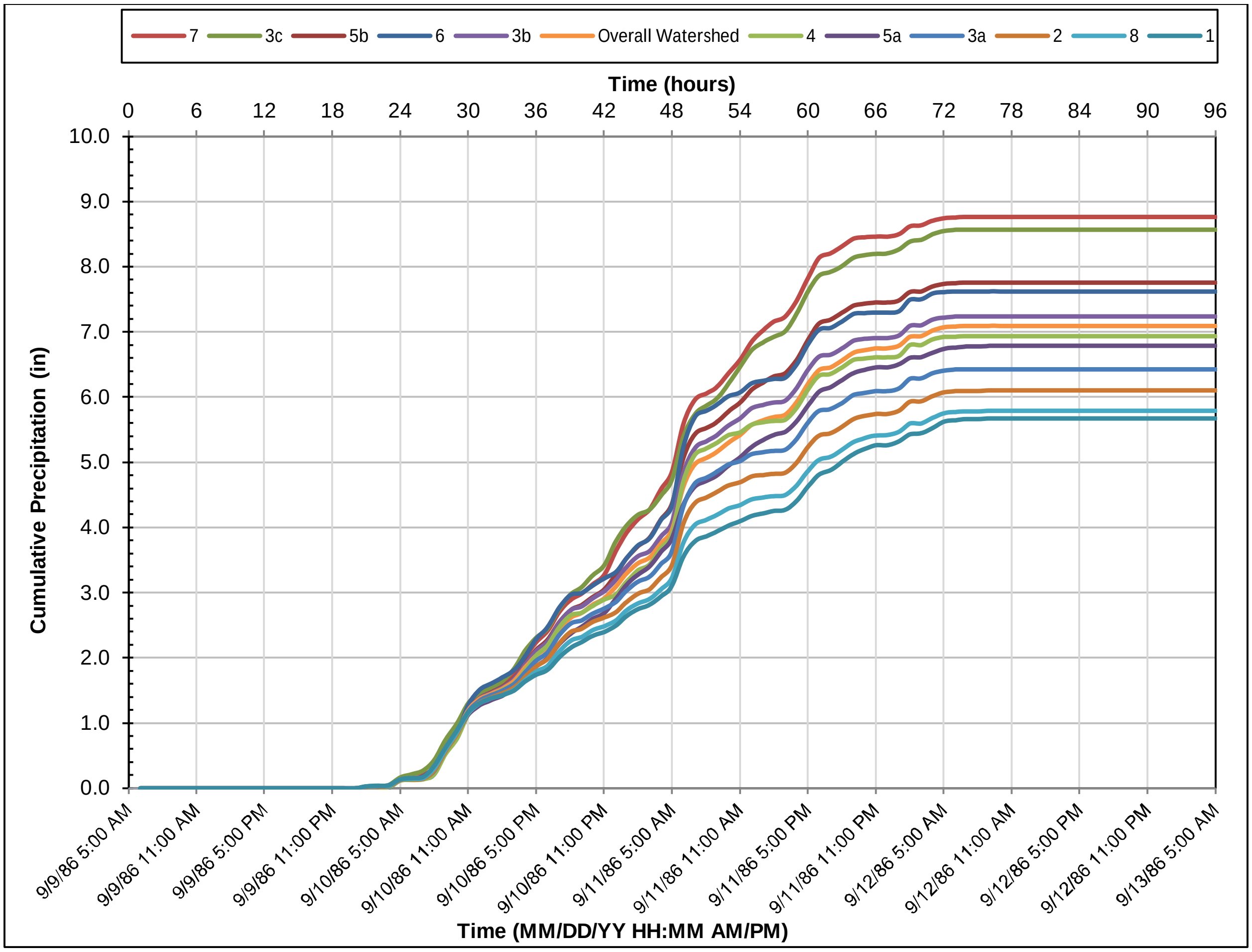


Figure F1-32: 96-Hour September 1986 Storm Cumulative Subbasin Precipitation for Sanford Watershed

Table F1-8: Subbasin Rainfall Amounts for September 9 through 13, 1986

Subbasin	September 9 through 13, 1986 Inches of Rainfall in 96 Hours	Drainage Area in Square Miles
Subbasin 1	5.68	132.6
Subbasin 8	5.79	53.4
Subbasin 2	6.10	34.7
Subbasin 4	6.93	79.0
Subbasin 5a	6.79	81.4
Subbasin 5b	7.76	81.5
Subbasin 6	7.63	60.7
Subbasin 3a	6.43	115.5
Subbasin 3b	7.24	138.0
Subbasin 3c	8.58	164.5
Subbasin 7	8.77	36.5
Overall Basin Weighted Average	7.06	977.9

F1-5.3.2 Comparison of May 2020 and September 1986 Flood Events

The total subbasin rainfall amounts for the May 2020 event compared to those for the September 1986 event are shown in Table F1-9. However, it should be noted that the May 2020 rainfall occurred over a shorter duration of time (42 hours) compared to the duration of the September 1986 storm (96 hours).

If the 1986 rainfall event had counterfactually occurred with the basin characteristics at the time of the May 2020 event, the excess runoff would have been much greater than it was in 1986. The excess runoff for that scenario was estimated by applying the spatial and temporal distributions of the 1986 event to the calibrated watershed model of the May 2020 event. A model calibration reflects both the spatiotemporal rainfall distribution and the watershed characteristics, and it is not possible to calibrate the model specifically for the 1986 storm on the May 2020 watershed (since that event did not occur). However, the IFT believes that applying the 1986 rainfall to the model calibrated for the May 2020 event should provide a reasonable approximation of what the runoff would have been for that 1986 rainfall.

Based on this approach, the discharge, precipitation, loss volume, and excess volume of precipitation are shown in Table F1-10 for each subbasin. Using the loss rate function described in Appendix C, the loss volume is the initial and constant loss rates over the duration of the rainfall event. The excess loss is the precipitation minus the loss volume and is defined as the excess rainfall that contributes to the runoff. For the May 2020 storm event, the excess runoff was computed to be 1.70 inches; for the 1986 storm event modeled with the May 2020 watershed characteristics, the weighted excess runoff was 2.82 inches, which is almost two times as much excess volume of runoff.

This estimate is based on the excess runoff from the subbasins and does not predict the peak flow that would have occurred at the reservoir due to the unknowns in the how the gates would have been operated. Table F1-10 provides the runoff results for the 11 subbasins. The conclusion is that the watershed characteristics for the May 2020 event were significantly different from those in September 1986, and had the May 2020 basin characteristics existed for the September 1986 rainfall, the runoff would have been much higher than the runoff that occurred in September 1986.

Also, from the outflow hydrograph for the 1986 storm event (from the Edenville plant logs), the excess runoff was estimated to be 0.83 inch with a peak outflow of 9,336 cfs. By comparison, the results of the model simulation indicate that if the September 1986 rainfall event occurred with the May 2020 basin conditions, the excess runoff of 2.82 inches would have been about three times greater than what occurred in September 1986.

Table F1-9: Subbasin Total Rainfall Amounts for the May 2020 and September 1986 Storm Events

Subbasin	May 2020 Inches of Rainfall in 42 Hours	September 1986 Inches of Rainfall in 96 hours
Subbasin 1	5.36	5.68
Subbasin 8	4.78	5.79
Subbasin 2	4.57	6.10
Subbasin 4	4.58	6.93
Subbasin 5a	5.10	6.79
Subbasin 5b	4.60	7.76
Subbasin 6	3.98	7.63
Subbasin 3a	3.96	6.43
Subbasin 3b	3.57	7.24

Subbasin	May 2020 Inches of Rainfall in 42 Hours	September 1986 Inches of Rainfall in 96 hours
Subbasin 3c	3.59	8.58
Subbasin 7	4.23	8.77
Overall, Basin Weighted Average	4.29	7.06

Table F1-10: May 2020 and September 1986 Rainfalls Applied to the May 2020 Watershed Conditions

Parameter	May 2020 Precipitation Event	September 1986 Precipitation Applied to May 2020 Watershed Conditions
Subbasin 1		
Discharge	5,580 cfs	2,540 cfs
Precipitation Volume	5.36 inches	5.68 inches
Loss Volume	2.53 inches	3.85 inches
Excess Volume	2.84 inches	1.83 inches
Subbasin 2		
Discharge	1,210 cfs	1,100 cfs
Precipitation Volume	4.57 inches	6.10 inches
Loss Volume	2.80 inches	4.17 inches
Excess Volume	1.77 inches	1.94 inches
Subbasin 3a		
Discharge	2,970 cfs	3,840 cfs
Precipitation Volume	3.96 inches	6.43 inches
Loss Volume	2.89 inches	4.67 inches
Excess Volume	1.07 inches	1.76 inches
Subbasin 3b		
Discharge	2,200 cfs	5,200 cfs
Precipitation Volume	3.57 inches	7.24 inches
Loss Volume	2.43 inches	3.96 inches
Excess Volume	1.14 inches	3.28 inches
Subbasin 3c		
Discharge	2,370 cfs	7,140 cfs
Precipitation Volume	3.59 inches	8.58 inches
Loss Volume	2.43 inches	4.36 inches
Excess Volume	1.16 inches	4.21 inches
Subbasin 4		
Discharge	3,830 cfs	5,240 cfs

Parameter	May 2020 Precipitation Event	September 1986 Precipitation Applied to May 2020 Watershed Conditions
Precipitation Volume	4.54 inches	6.94 inches
Loss Volume	2.72 inches	3.86 inches
Excess Volume	1.82 inches	3.08 inches
Subbasin #5a		
Discharge	2,840 cfs	1,940 cfs
Precipitation Volume	5.10 inches	6.79 inches
Loss Volume	3.18 inches	5.18 inches
Excess Volume	1.92 inches	1.61 inches
Subbasin #5b		
Discharge	3,010 cfs	3,910 cfs
Precipitation Volume	4.58 inches	7.76 inches
Loss Volume	2.77 inches	4.61 inches
Excess Volume	1.81 inches	3.16 inches
Subbasin #6	1,390 cfs	2,830 cfs
Discharge	3.98 inches	7.63 inches
Precipitation Volume	2.55 inches	3.90 inches
Loss Volume	1.43 inches	3.73 inches
Excess Volume		
Subbasin 7		
Discharge	960 cfs	1,880 cfs
Precipitation Volume	4.24 inches	8.77 inches
Loss Volume	2.80 inches	5.15 inches
Excess Volume	1.44 inches	3.61 inches
Subbasin 8		
Discharge	1,550 cfs	1,220 cfs
Precipitation Volume	4.75 inches	5.79 inches
Loss Volume	2.45 inches	3.72 inches
Excess Volume	2.30 inches	2.08 inches
Total Weighted Excess Volume in inches of Runoff	1.66 inches	2.82 inches

To further investigate the watershed characteristics during the May 2020 event, a simplified model was developed to estimate the inflows into Wixom Lake for the May 2020 and September 1986 rainfall events, using the watershed conditions (characteristics) of May 2020. Figure F1-33 shows the results of the two-computer simulation runs. These simplified models do not model variable gate operations and assume the gates of all modeled reservoirs were fully open throughout the simulations. The models show a general comparison of the two rainfall events over the May 2020 watershed conditions, but do not represent the actual May 2020 or September 1986 water levels or outflows. The May 2020 storm event resulted in a peak inflow to Wixom Lake of 21,500 cfs, while the September 1986 rainfall event applied to the May 2020 watershed conditions resulted in a simulated peak inflow to Wixom Lake of approximately 31,000 cfs, assuming all gate operations were the same for both events. This analysis indicates that the watershed conditions during the May 2020 event led to a significantly higher peak inflow compared to the watershed conditions during the September 1986 event, given the two distinct spatial and temporal rainfall distributions.

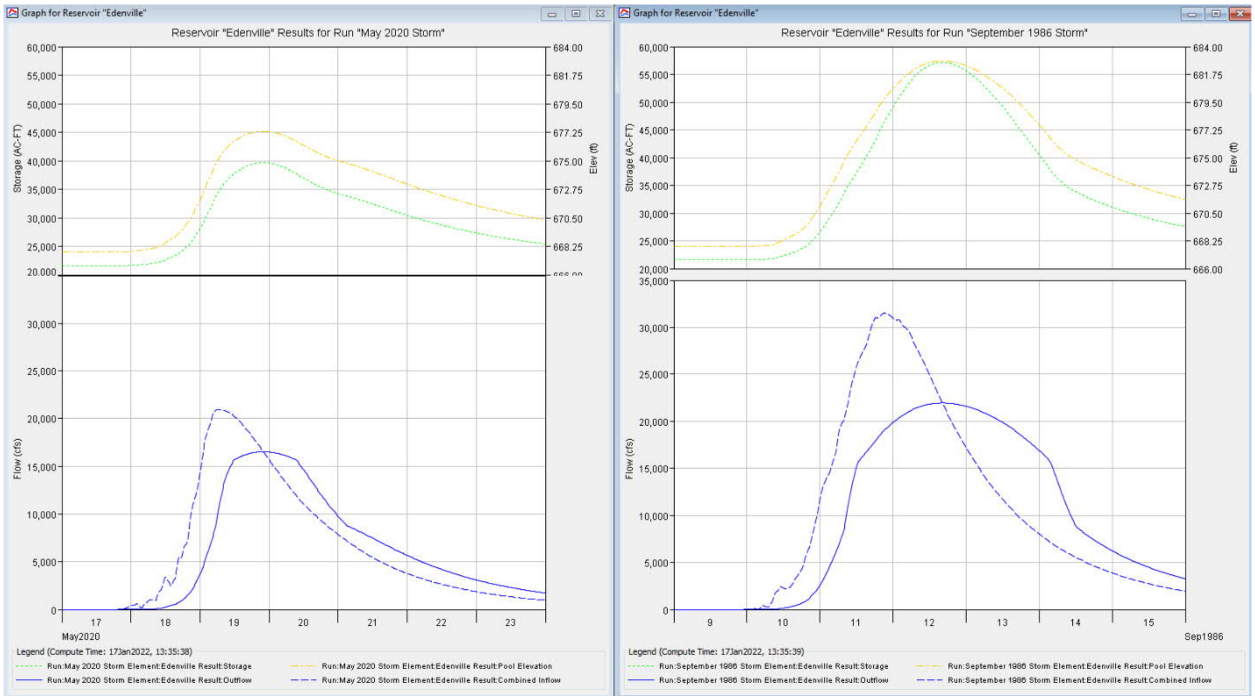


Figure F1-33: Comparison of Peak Flow for May 2020 and September 1986 Rainfall Events on the Basin with May 2020 Characteristics

F1-5.4 Models of Hypothetical Scenarios

A series of hypothetical operational scenarios were evaluated using the calibrated hydrologic and hydraulic model developed to reproduce the May 2020 storm event. These scenarios were evaluated to assess the sensitivity of lake levels to various operational parameters and evaluate potential counterfactual "what if" scenarios. The results of the various scenarios considered are summarized in Table F1-11 and Figure F1-34. It should be noted that the accuracy of the model is limited by the modeling assumptions incorporated in the computer software used for the modeling, the resolution and accuracy of the available rainfall data, the resolution and accuracy of the land characteristics data available for the watershed, and the data available for calibrating the model.

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Table F1-11: Simulated Peak Reservoir Elevation of Wixom Lake

Scenario	Assumed Opening Height for Edenville and Tobacco Gates	Starting Lake Level of Wixom Lake	Powerhouse (1)	Model Peak Reservoir Elevation (feet)	Notes
Actual	7 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Not Operated	681.3	Simulates the dam breach on May 19, 2020, at 5:30 p.m. at approximate reservoir elevation of 681.3 feet.
1	7 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Not Operated	681.6	Assumes the actual May 2020 operations, but assuming no breach occurs to estimate the maximum lake level if the dam did not fail.
2a	7 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Operated (Shut down once lake is 3 feet above normal pool)	681.3	Assumes powerhouse flows were included for Edenville Dam (approximately 2,000 cfs) once all gates were opened on May 18 at 2:30 p.m. The powerhouse flows remained included until the Wixom Lake level reached 3 feet above normal pool (678.8 feet) at which time it was assumed they would have been shut off and were no longer included in the model simulation.
2b	7 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Operated (Remained operating throughout simulation)	680.5	Assumes powerhouse flows were included for Edenville Dam (approximately 2,000 cfs) once all gates were opened on May 18 at 2:30 p.m. The powerhouse flows remained on throughout the remaining simulation through the peak lake level.
3	10 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Not Operated	680.2	Assumes all Edenville Gates (both Tobacco and Edenville Spillways) to have been fully opened to maximum height of 10 feet on May 18 at 2:30 p.m.
4a	10 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Operated (Shut down once lake is 3 feet above normal pool)	679.9	Assumes all Edenville Gates (both Tobacco and Edenville Spillways) to have been fully opened to maximum height of 10 feet, and that powerhouse flows were included for Edenville Dam (approximately 2,000 cfs) throughout the simulation when all gates were fully opened on May 18 at 2:30 p.m. The powerhouse flows remained included until the Wixom Lake level reached 3 feet above normal pool (678.8 feet) at which time it was assumed they would have been shut off and were no longer included in the model simulation.
4b	10 feet	~ Normal Pool (2) (El. 675.6)	Powerhouse Operated (remained operating throughout simulation)	679.5	Assumes all Edenville Gates (both Tobacco and Edenville Spillways) to have been fully opened to maximum height of 10 feet, and that powerhouse flows were included for Edenville Dam (approximately 2,000 cfs) throughout the simulation when all gates were fully opened on May 18 at 2:30 p.m. The powerhouse flows remained on throughout the remaining simulation through the peak lake level.
5	7 feet	Winter Pool (El. 672.8)	Powerhouse Not Operated	681.4	Assumes Wixom Lake starting at winter pool level (El. 672.8) with May 2020 gate operations for all reservoirs.
6	7 feet	All Reservoirs Starting at Winter Pool	Powerhouse Not Operated	681.1	Assumes all Boyce reservoirs starting at winter pool level (3 feet below normal pools) with May 2020 gate operations for all reservoirs.
7	7 feet	2018/2019 Drawdown Level (El. 670)	Powerhouse Not Operated	681.1	Assumes Wixom Lake starting at run-of-river lake level (estimated as El. 670.0) with Edenville and Tobacco Spillway Gates open 7 feet throughout the event, and May 2020 gate operations for other reservoirs.
8	10 feet	2018/2019 Drawdown (El. 670)	Powerhouse Not Operated	679.9	Assumes Wixom Lake starting at run-of-river pool level (estimated as El. 670.0) with Edenville and Tobacco Spillway Gates open approximately 10 feet throughout the event, and May 2020 gate operations for other reservoirs.
9	21 feet (3)	~ Normal Pool (2) (El. 675.6)	Powerhouse Not Operated	675.6	Models the proposed Edenville Tainter Gate Design for Tittabawassee and Tobacco River Gates. The 2012 proposed gate design (Mill Road Engineering 2012) included a spillway crest at 654.8 feet (13 feet lower than existing) and 22.6-foot-high spillway gates. Modeled assuming proposed Edenville gates had been open sufficiently to allow weir flow at the time of their maximum opening during the May 2020 storm event (by May 18 at 2:30 p.m.). See Note 3.

Notes:

(1) Edenville has two turbines that were able to pass approximately 1,000 cfs per turbine or a combined flow of 2,000 cfs when they were operating at 100 percent capacity. This postulates the condition in which the powerhouse would have been able to operate at 100 percent capacity either because the license had not been revoked or the powerhouse had been modified to allow full discharge once power generation had ceased.

(2) The starting lake level of Wixom Lake for the May 2020 lake level scenarios was 675.6 feet (0.2 feet below normal pool). This value was obtained from the recorded Wixom Lake level data on May 17, 2020 at 12:00 AM, which is the selected starting time of the model simulations.

(3) Scenario 9 models the hypothetical condition in which the spillways were upgraded in accordance with the 2012 proposed design (Mill Road Engineering 2012). In this modeling scenario the gates are assumed to have been opened sufficiently to maintain weir flow.

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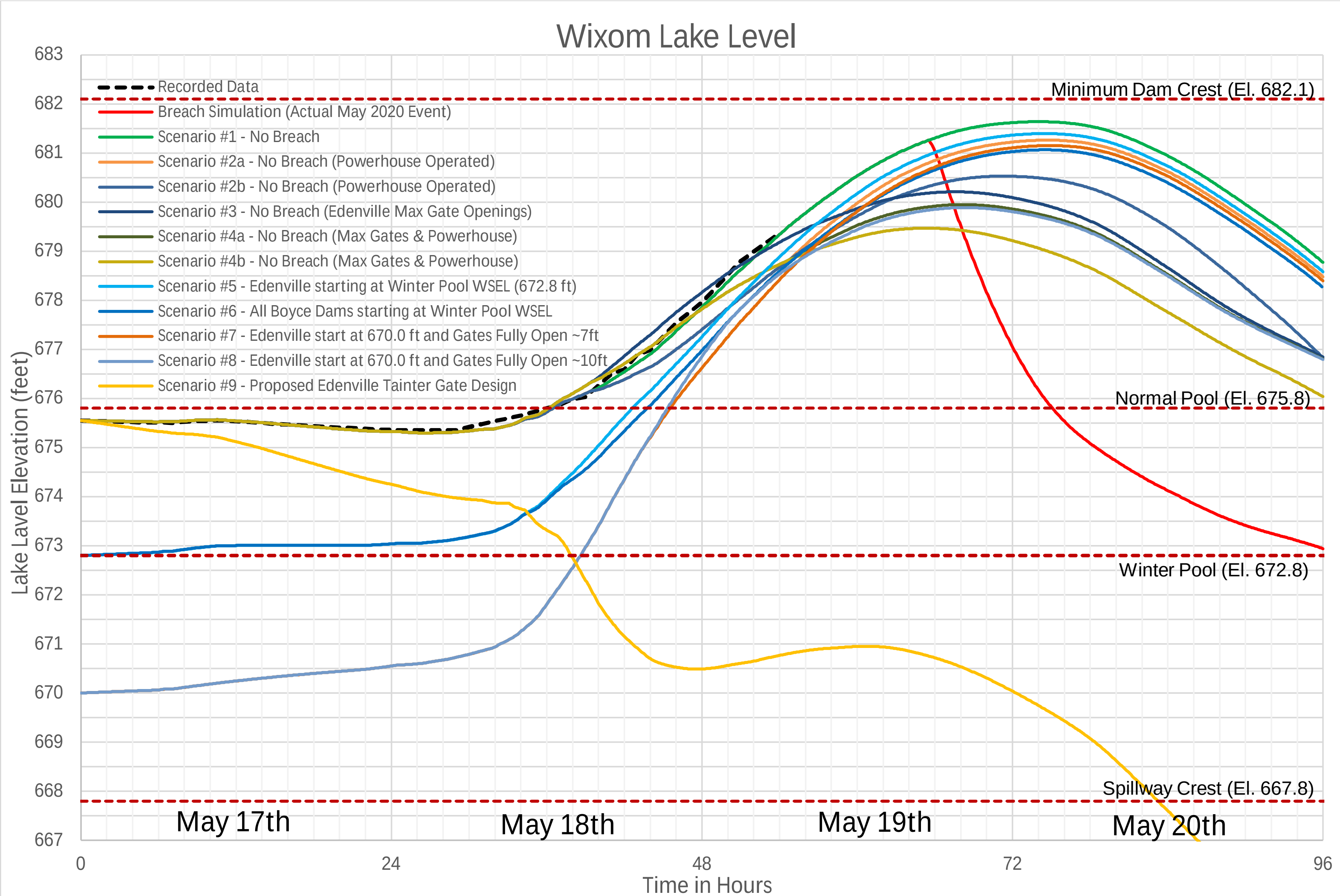

Figure F1-34: Model Simulation Results of Hypothetical Operational Scenarios for May 2020 Event

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F1-6 Summary of Factors Impacting the May 2020 Flooding Event

Factors that influenced the Wixom Lake Level during May 17 through 19, 2020 storm event are discussed below.

F1-6.1 Spatial Rainfall Distribution

Approximately 4.3 inches of rainfall occurred on average over the watershed from May 17 through May 19. However, the spatial distribution indicates that more rainfall occurred in the northeastern portion of the watershed, which produces inflow to Wixom Lake. The Tittabawassee River and the Tobacco River received 4.87 and 3.73 inches of rain, respectively. The highest rainfall totals occurred over the highest concentration of wetlands in the eastern portion of the watershed.

F1-6.2 Temporal Rainfall Distribution

All 11 subbasins show similar temporal rainfall distributions (Figure F1-32). Most of the precipitation occurred in 18 hours and had a sustained rainfall intensity of 0.22 inch per hour. According to the NWS, a rainfall intensity of 0.22 inch per hour for a duration of 18 hours would be a 25- to 50-year return period rainfall event antecedent moisture (NOAA 2022b).

Antecedent moisture and/or groundwater levels were above average, especially in the wetlands in the northeastern portion of the watershed. About 2 inches of rain occurred in the last 3 days of April 2020, and 1 inch occurred the day before the event. This would increase the antecedent moisture conditions prior to the initiation of the storm event. As a result, the basin could be expected to have had fairly wet antecedent moisture conditions, which would result in more runoff to the reservoirs.

F1-6.3 Seasonal Variation of the Watershed

The eastern half of the basin contains many lakes and wetlands that will result in a larger portion of the rainfall being converted to runoff if there is frozen ground or a groundwater level near or above the ground surface. Much of the forested land contains mixed conifer and hardwood swamps, which tend to have lower air temperatures and a greater tendency to have frozen ground than open unshaded areas; this leads to less infiltration and higher runoff in the forested lands. Figure F1-35 shows the landcover of the watershed compared with the May 2020 precipitation distribution to highlight how the most significant rainfall occurred in the wooded wetlands portion of the watershed.

For 1.5 weeks prior to May 17, the Gladwin NWS station recorded minimum (evening) temperatures below freezing (see Figure F1-36). Temperatures in the forested and wetland areas would be several degrees lower (colder) than the recorded temperatures at the NWS station. The cooler temperatures could result in frost conditions and reduce the permeability of the ground due to perching the infiltration above a zone of partially frozen ground. This would particularly affect the northeastern portion of the watershed, which could result in higher runoff.

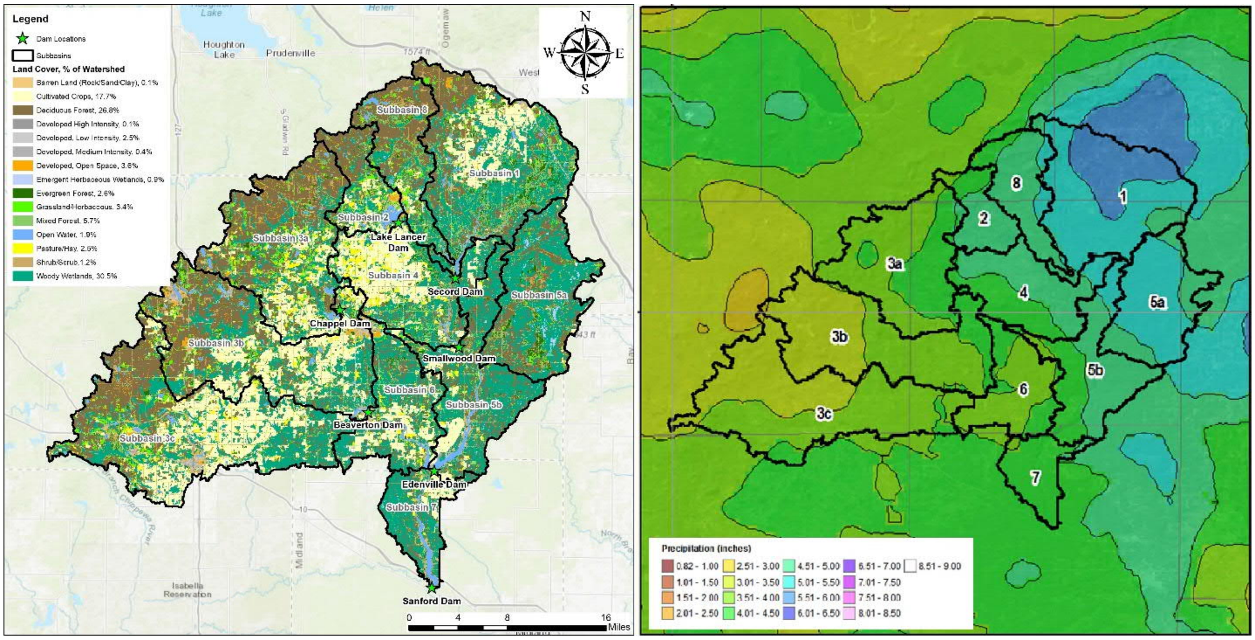

Figure F1-35: Land Cover vs. May 2020 Rainfall Comparison Figure

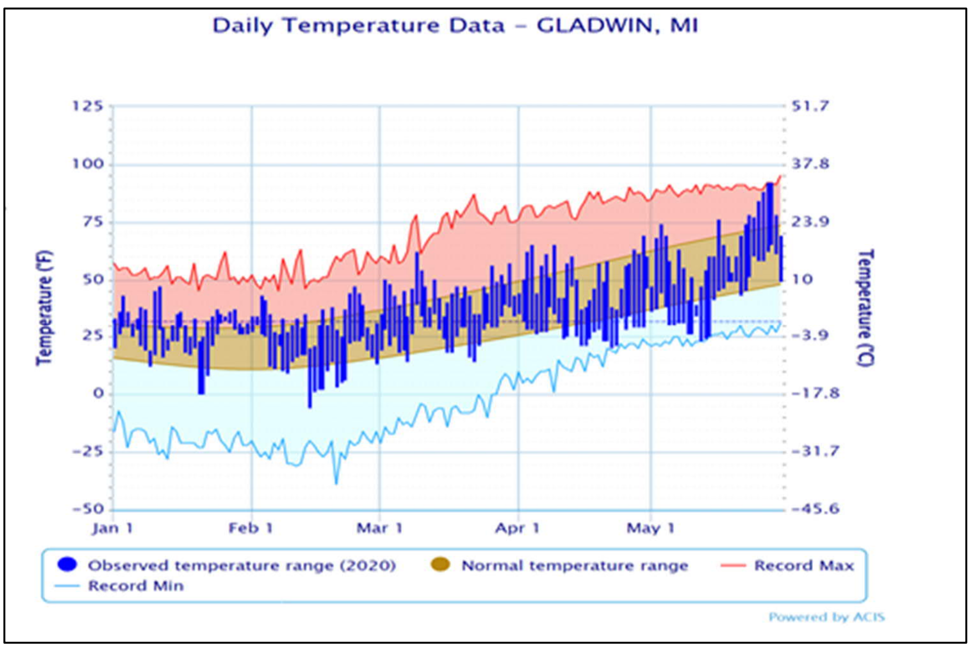

Figure F1-36: Temperature Data for January through May 2020

Agricultural activities taking place during the spring (April/May) season (such as tilling and potential rolling) contribute to higher runoff soil conditions than the agricultural activities taking place at other times of the year, particularly in the summer and fall, when there would be more vegetation and the root systems would support improved infiltration. The IFT did not evaluate changes in agricultural practices in this watershed during the one-century history of the project; however, it is possible that changes in agricultural practices contributed to increased runoff in May 2020. This possibility may warrant further investigation.

F1-6.4 Powerhouse Releases

During the May 2020 event, no water was released through the powerhouse because the Edenville license had been revoked and, on May 18 and 19, the view of the parties on-site was that it was not physically safe to allow water to flow through the turbines without generating power. It is estimated that releasing water through the powerhouse by itself would have lowered the peak Wixom Lake level in the May 2020 event by about 0.8 foot below the lake level at the time of the failure.

F1-6.5 Gate Openings

During the May 2020 event, the gates at Edenville Dams were opened to only about 7 feet. If the gates were fully opened to 10 or more feet, it is estimated that the peak Wixom Lake level would have been about 1.1 feet lower than the lake level at the time of the failure, at an elevation of 680.2 or 4.4 feet above normal pool.

F1-6.6 Summary

The May 17 through 19, 2020 rainfall event was a significant but not an extreme rainfall event (an estimated 25- to 50-year return interval rainfall event), not of the magnitude of a probable maximum flood (PMF) or even a ½ PMF. Reviewing past storm events that occurred in the Sanford watershed, the May 2020 event resulted in one of the largest inflows into Wixom Lake. The maximum Wixom Lake level during the event was approximately 1.3 feet below the crest of the dam when the embankment failed. It is estimated that the reservoir would not have overtopped the dam if the embankment failure had not occurred.

The IFT hydrologic model estimated the peak inflow to Wixom Lake to be 24,500 cfs on May 19, 2020, at the time of failure. Ayres Associates performed flood frequency analysis (Ayres 2015) using the USACE, HEC Statistical Software Package (HEC-SSP) (USACE 2022). In a later study, Ayres (2021) updated those values with additional data, and an inflow of 24,500 cfs would be approximately a 100- to 200-year return period flow. In the 2021 study, Ayres estimated that a 200-year return period (annual exceedance probability = 0.005) would have a peak inflow of 25,400 cfs and would have a freeboard of 0.1 foot with assumed gate openings of 8.9 to 9.6 feet.

F1-7 References

Ayres Associates.2015 (February 17). P-10808 – Final Flow Calculations. Letter from Boyce Hydro Power, LLC, to FERC. February 17, 2015.

———. 2020 (May 15). *Probable Maximum Flood Determination, Tittabawassee River Hydroelectric Projects: Secord (P-10809), Smallwood (P-10810), Sanford (P-2785), Edenville (unlicensed).* Available: https://www.four-lakes-taskforce-mi.com/uploads/1/2/3/1/123199575/ttbw_pmf_final_report-may2020.pdf.

———. 2021 (July). *Design Flood Hydrologic Analysis*: *Secord, Smallwood, Edenville, and Sanford Dams, Gladwin and Midland Counties, Michigan*. Prepared for Four Lakes Task Force, Midland, MI. Available: https://www.four-lakes-taskforce-mi.com/uploads/1/2/3/1/123199575/ayres_design_flood_hydrologic_analyses_july_2021.pdf.

Dolinar, E. 2020. High Flows from Secord Dam in Gladwin County Flash Flooding, New Alert on Fox News, Dolinar Meteorologist.

EGLE. 2019. Application Denial, Submission Number: HNS-X-17S-6B1RG; Gladwin County. Issued by EGLE

_____. 2020. *Permit No.: WRP02178 v.1*. Issued by State of Michigan Department of Environment, Great Lakes and Energy (EGLE). April 9, 2020.

Federal Energy Regulatory Commission (FERC). 1998a (April 20). In reply refer to: P-10808, NATDAM No. MI00549. Letter from FERC to Wolverine Power Company. April 20, 1998.

______. 1998b. (October 16). Unofficial FERC-Generated PDF of 19981020-3011. Issued by FERC OSEC 10/16/1998 Docket#: P-10808-000, Article 404.

Four Lakes Task Force (FLTF). 2019a (December 2). *Extension of April 15, 2019 Letter Agreement; Operations of Edenville Dam and Maintenance of Normal Levels of Wixom Lake*. From Four Lakes Task Force to Mr. Lee W. Mueller.

———. 2019b (April 16). *Letter of Understanding Regarding Acquisition of Lake Level Assets*. From Four Lakes Task Force to Mr. Lee W. Mueller.

———. 2019c. Agreement between Midland and Gladwin Counties and Four Lakes Task Force. (December 2). *Extension of April 15, 2019 Letter Agreement; Operations of Edenville Dam and Maintenance of Normal Levels of Wixom Lake*. From Four Lakes Task Force to Mr. Lee W. Mueller.

GEI Consultants Inc. (GEI). 2020 (April 27). *Draft Discharge Rating Curves (Secord, Smallwood, Edenville and Sanford Projects) Four Lakes Task Force (FLTF).* Draft report courtesy of FLTF.

Harrison, Robert J via Storyful. 2020 (May 20). *Smallwood Lake Dam Overflows After Edenville Dam Failure*.

Little Forks Conservancy. 2011 (August). *Cedar River Watershed Management Plan.* Update for Little Forks Conservancy, Midland, MI, through a grant from the Michigan Department of Natural Resources and Environment. Tracking Number 2009-0036. Original plan was created in August

2001 by Michael D. Turner, Gladwin Conservation District. Available: http://www.gladwincd.org/uploads/1/2/3/3/12338134/updated_cedar_river_watershed_management_plan.pdf.

Mead & Hunt. 1994 (August). "Probable Maximum Flood Study, Tittabawassee river, Michigan, Secord, Smallwood, Edenville, and Sanford Hydroelectric Projects," FERC Project Nos. 10809, 10810, 10808, 2781.

Michigan Department of Environment, Great Lakes, and Energy (EGLE). 2019 (November 25). Application Denial, Submission Number: HNS-X-17S-6B1RG; Gladwin County. Issued by EGLE

Mill Road Engineering. 2012 (March 20). *Tainter Gate Design Report and Calculations, Edenville Project*.

National Oceanic Atmospheric Administration (NOAA). 2022a. NOAA Online Weather Data. Available: https://www.weather.gov/wrh/Climate?wfo=apx.

_______. (NOAA). 2022b. NOAA Atlas 14 Point Precipitation Frequency Estimates. Available: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html.

National Weather Service. 2020. NWS Historic Flooding May 17-20, 2020. Available: https://www.weather.gov/dtx/HistoricFlooding-May-17-20-2020

Natural Resources Conservation Service (NRCS). 2017. Natural Resources Conservation Service Web Based Soil Survey. Available: https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

———. 2021. National Engineering Handbook Part 630 Hydrology. Letter from Secretary of War Transmitting Report. Available: https://www.nrcs.usda.gov.

Nims, Tereasa. 2018 (September 18). FERC Approves Lake Drawdowns; Wixom Remains in Danger. *Midland Daily News*. Available: https://www.ourmidland.com/news/article/FERC-approves-nbsp-lake-drawdowns-Wixom-13236523.php.

Purkeypile Consulting, LLC (Purkeypile). 2016 (March). *2015 Consultant's Safety Inspection Report, Edenville Hydroelectric Project, FERC Project No. 10808*.

State of Michigan. 2019 (May 28). *Order Setting Normal Lake Levels for Sanford Lake, Wixom Lake, Smallwood Lake and Secord Lake and Confirming the Four Lakes Special Assessment District Boundaries*. Issued by the State of Michigan in the Circuit Court for the County of Midland. Available: https://www.thesafetyequipmentstore.com/miva/graphics/00000001/4-Lakes-Order-Engwis.pdf.

State of Michigan House of Representatives. 1932. House of Representatives Document No. 273 Tittabawassee and Chippewa Rivers, MI, 72D Congress.

U.S. Geological Survey (USGS). 2016a. Gladwin, County One Meter LiDAR Data. Available: https://apps.nationalmap.gov/downloader/#/

______. (USGS). 2016b. Midland, County One Meter LiDAR Data. Available https://apps.nationalmap.gov/downloader/#/

______. (USGS). 2016c. National Land Cover Database (NLCD) 2016. Available: https://www.mrlc.gov/national-land-cover-database-nlcd-2016.

U.S. Army Corps of Engineers (USACE). 2022 (January). Hydrologic Engineering Center, HEC-SSP User's Manual version 2.2. Available: https://www.hec.usace.army.mil/software/hec-ssp/.

______. (USACE). 2019. Hydrologic Engineering Center, River Analysis System (HEC-RAS). Version 5.0.7.

______. (USACE). 2016. Hydrologic Engineering Center, Hydrologic Modeling System (HEC-HMS). Version 4.2, Build: 1668.

F2: Forensic Team Analysis - Geotechnical Analyses

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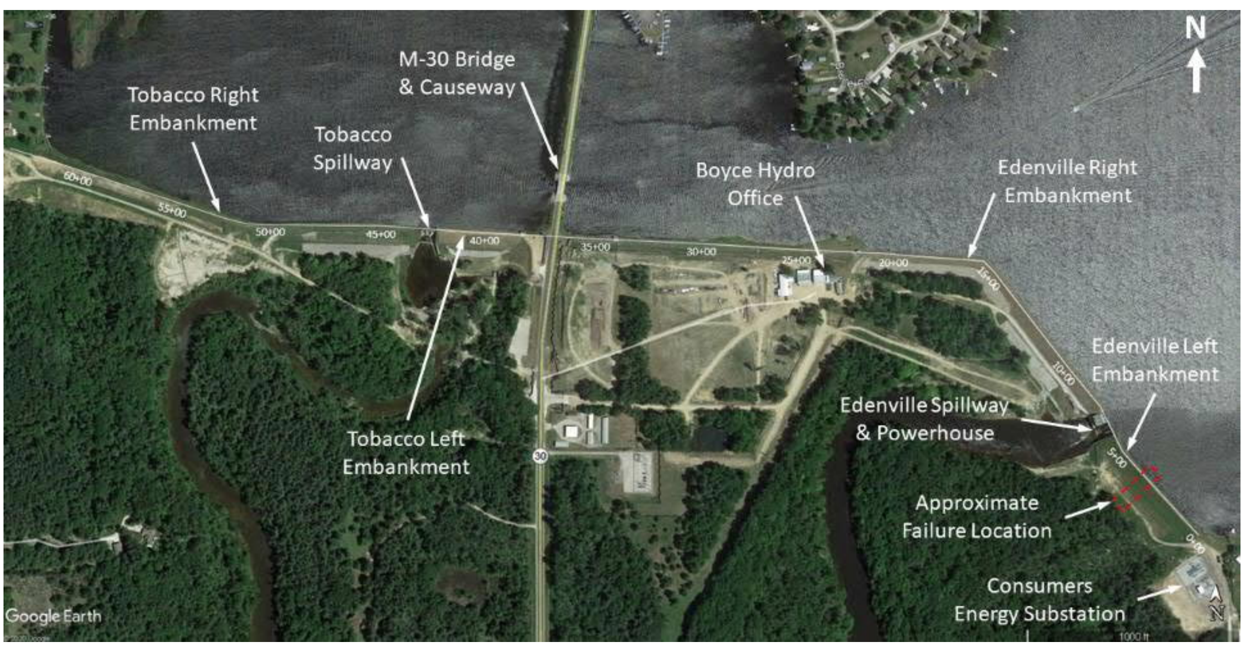
F2-1 Objective

The Independent Forensic Team (IFT) performed the following geotechnical analyses to assist in the development of the findings of the forensic investigation:

1. Steady-state slope stability analysis for the Edenville left embankment at the postulated failure section (Station [Sta.] 3+50), using material zoning and properties similar to the 1987 analysis of the Tobacco right embankment section (Sta. 48+00)
2. Steady-state seepage analysis of Sta. 3+50, using section geometry and stratigraphy based on available geotechnical data and the IFT field and laboratory investigation
3. Steady-state slope stability analysis of Sta. 3+50, using pore pressures from the steady-state seepage analysis, estimated peak drained strengths, estimated angle of instability (instability line), and liquefied strength based on available geotechnical data and the IFT field and laboratory investigation
4. Stress state analysis of Sta. 3+50, using pore pressures from the steady-state seepage analysis and approximate isotropic elastic properties
5. Kinetic analysis of Sta. 3+50 at failure

F2-2 Section Development

At the time of the May 2020 failure, Edenville Dam consisted of four earthfill embankments, two gated concrete spillways, and a powerhouse—all constructed across the Tittabawassee and Tobacco Rivers in Michigan and totaling more than 6,000 feet in length, as shown in Figure F2-1.



Source of aerial image: Google Earth

Figure F2-1: Edenville Dam Configuration

Figure F2-1 identifies the four embankments: Edenville left, Edenville right, Tobacco left, and Tobacco right embankments. The May 2020 failure occurred at the Edenville left embankment.

Embankment geometry representing the initial breach section of the Edenville left embankment was evaluated for each of the analyses described in this appendix. The breach location was estimated to be at or near Sta. 3+50, based on drone video and cell phone video taken during and soon after the initial failure. Landmarks observed in the videos were matched against project survey to determine the stationing of the initial breach.

The external geometry for Sta. 3+50 was obtained from 2018 U.S. Geological Survey (USGS) LiDAR data. The upstream slope was projected from the reservoir surface to the elevation (El.) of the downstream toe (El. 651) at a slope of 2.5H:1V based on design drawings. The external geometry at Sta. 3+50 was compared to the geometry at Sta. 48+00 (1987 analysis) and Sta. 10+00 (1991 analysis), as shown in Figure F2-2. The results show that Sta. 48+00 and Sta. 10+00 are the taller sections of the dam, with a height of about 40 feet from crest to downstream toe, while Sta. 3+50 is shorter at about 32 feet tall. However, Sta. 3+50 has an overall steeper downstream slope (average of approximately 1.8H:1V) than the other sections.

Information on the internal section geometry for the Edenville left embankment is limited. The available geotechnical data in the vicinity of the failure section includes two crest borings and two toe borings (McDowell 2005b; Mill Road Engineering 2010); one crest boring, located near the left abutment (Somat 2020); construction specifications, memos and photos; and the IFT field and laboratory investigation. The borings are described in Appendix B, the specifications and photos are presented in Appendix D, and the IFT field and laboratory investigation is described in Appendix E.

For the IFT's analyses, a zoned embankment cross section was developed based on the findings from the IFT's field investigation; however, there is significant uncertainty concerning the actual cross section at the failure location, as discussed in Appendix A and the main report. The analyses completed using this cross section should not be viewed as detailed analyses of the actual cross section that existed, but rather as analyses of a postulated cross section that demonstrate the plausibility of a static liquefaction instability failure and a relatively low factor of safety (but greater than 1.0) for a conventional limit equilibrium instability failure.

The postulated Sta. 3+50 cross section consisted of five embankment and foundation layers and zones:

- Silty Sand Fill
- Clayey Fill
- Clean Sand Fill
- Native Sand Foundation
- Glacial Till

The analysis section used in the IFT geotechnical models, including material zoning is shown in Figure F2-3. The embankment was modeled with a crest width of 8 feet, upstream slope of 2.5H:1V, and a downstream slope that varied along the height based on survey data, with an average of about 1.8H:1V. The structural height of embankment fill is 32 feet. The postulated embankment model consists of 13 feet of silty sand fill overlaying 19 feet of clayey fill on the upstream portion and clean sand fill on the downstream portion. The division between the upstream clayey fill and downstream clean sand fill is located near the centerline of the embankment and at a slope of 0.5H:1V towards the downstream. Based on mapping of the remnant breach face by the IFT (see Appendix E), the material being termed clean sand fill contained nodules and lenses of clay, but the behavior of the fill was judged to be controlled by the clean sand matrix material. The embankment fill overlies 6.5 feet of native sand foundation, which overlies glacial till. The glacial till extends to the bottom of the model. The model extends 60 feet upstream from the upstream toe and 67 feet downstream from the downstream toe.

F2-3 General Methodology

Two-dimensional steady-state seepage analyses were performed using the SEEP/W computer program, Version 11.1.3 (GEOSLOPE 2021). SEEP/W is a finite element software package that can be used to simulate the flow and pore water pressure distribution within porous media. The program simulates both saturated and unsaturated flow of water and is suited for analyzing flow of water through the embankment and foundation materials.

Two-dimensional limit equilibrium slope stability analyses were performed using the Slope/W computer program, Version 11.1.3 (GEOSLOPE 2021b) to calculate the factor of safety against slope instability. Spencer's method of slices was used in conjunction with circular and noncircular automated search routines to identify critical shear failure surfaces. The iterative method used in the program involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied.

The methodologies and assumptions specific to each analysis are described further in the following sections.

F2-4 Slope Stability Analysis with 1987 Parameters

F2-4.1 Purpose and Methodology

A slope stability analysis was performed for the Edenville left embankment at the postulated failure section (Sta. 3+50), using material zoning and properties similar to the 1987 analysis of the Tobacco right embankment section (Sta. 48+00) under a normal pool condition. The purpose of this analysis was to evaluate whether application to Sta. 3+50 of assumptions and parameters similar to those used in 1987 to analyze Sta. 48+00 would have indicated a relatively low factor of safety at Sta. 3+50, potentially indicating a concern for instability at this section on par with other embankment sections for which stabilization methods were recommended.

F2-4.2 Material Properties

The cross-section model used in the 1987 analysis of Sta. 48+00 included four material types, which are summarized in Table F2-1 below:

Table F2-1: Summary of Material Properties Used in the 1987 Slope Stability Analysis

Material	Total Unit Weight, γ_t (pcf) [1]	Effective Cohesion, c' (ksf)[2]	Effective Friction Angle, ϕ' (degrees)
Fine to Medium Sand Fill	120	0	35
Silt and Sand Fill	105	0	32
Natural Sand	120	0	34
Sandy Clay Hardpan	130	4.5	0

Source: SME 1987
Notes:
[1] The total unit weights are in units of pounds per cubic foot (pcf). The values used in the 1987 slope stability analysis were not listed in the report. These values were applied to the Sta. 3+50 section based on 1987 laboratory data.
[2] Units are in kips per square foot (ksf).

The zoning of the embankment materials was applied to Sta. 3+50, using depths proportional to the height of the embankment. For example, the total height of the embankment at Sta. 48+00 (from crest to native sand) is 40 feet, and the total height of the embankment at Sta. 3+50 is 32 feet. Thus, the depth of fine to medium sand fill in the Sta. 3+50 section was taken as 20 feet, in contrast to the 26-foot depth in the Sta. 48+00 section.

F2-4.3 Pore Pressure

A piezometric line was used to define the pore pressures in the 1987 slope stability analysis. The piezometric level was based on groundwater monitoring wells installed during a geotechnical investigation at Sta. 48+00. The piezometric line from the 1987 analysis is defined by three straight line segments through the embankment: from normal water level (NWL) to a point on the centerline of the embankment, 26 feet below the dam crest, then surfacing on the downstream face, 30 feet below the dam crest elevation, and extending along the downstream embankment face to the downstream toe.

Past inspections have not indicated any significant seepage on the downstream face or on the ground surface at the downstream toe of the embankment at the failure location (only minor wet or soft spots had been noted at isolated locations on the Edenville left embankment). Therefore, a straight line piezometric surface was applied to the Sta. 3+50 section, from the NWL to the downstream toe. This is meant to represent a reasonably conservative, simplified approach to estimating a phreatic surface in the absence of piezometric data.

F2-4.4 Results

The results of the 1987 slope stability analysis comparison are shown in Figure F2-4. Results indicate a factor of safety against slope instability of approximately 1.0 along a shear surface that extends from the embankment crest to the embankment toe.

F2-5 Steady-State Seepage Analysis

F2-5.1 Methodology and Purpose

A steady-state seepage analysis was performed for the Edenville left embankment for the failure location (Sta. 3+50) with the postulated embankment cross section and a normal lake level, El. 675.8 feet (National Geodetic Vertical Datum of 1929 [NGVD29]). This analysis was performed to estimate the steady-state phreatic surface, identify pore pressures that could reasonably occur during normal operating conditions, and support initial pore-pressure assumptions to be used in stability analyses.

F2-5.2 Material Properties

Material properties needed for the seepage analysis included saturated horizontal hydraulic conductivity (k_h) and the anisotropy ratio (i.e., the ratio of horizontal to vertical permeability). Laboratory test results, including constant head permeability (ASTM D 2434), one-dimensional consolidation (ASTM D 2435), and grain size analysis (ASTM D 6913), were used to estimate the materials' hydraulic properties. When laboratory data were unavailable, engineering judgment was used to select hydraulic conductivity values.

The embankment materials were assigned a saturated/unsaturated material model. The saturated/unsaturated hydraulic conductivity curves were estimated based on volumetric water content curves pre-programed into Seep/W. The native sand and glacial till were assigned a saturated-only material model.

The embankment materials were assigned an anisotropy ratio (kh/kv) of 4, and foundation materials were assigned an anisotropy ratio of 10. The values of anisotropy were based on typical values used in practice, and it was observed that they did not significantly affect the resulting steady-state phreatic surface.

F2-5.2.1 Clean Sand Fill

Somat (2020) performed two constant head permeability tests (ASTM D 2434) on composite clean sand fill samples from the Tobacco right embankment. The results ranged from 1.44×10^{-2} cm/s to 2.25×10^{-2} cm/s at dry densities of 90.1 and 88.0 pcf, respectively.

The IFT field and laboratory investigation indicated that the grain size distribution for the clean sand fill was similar to the composite sample tested by Somat (2020). Therefore, an upper-bound permeability of the sand fill was taken to be 1×10^{-2} cm/s.

In comparison, the estimated permeability of the clean sand fill from the IFT investigation, using Hazen's formula (Fell et al. 2015) (Eq. F2-1) was 5×10^{-3} cm/s. Therefore, a lower-bound permeability of the clean sand fill was taken to be 1×10^{-3} cm/s.

$$k\ (\text{cm/s}) = CD_{10}^2 \qquad \text{Eq. F2-1}$$

Where: k = the hydraulic conductivity in cm/s,
D_{10} = the grain size at 10% passing the grain size distribution in millimeters, and
C = a constant ranging from 0.8 to 1.2, taken here as 1.

F2-5.2.2 Clayey Fill

Somat (2020) performed a one-dimensional consolidation test on a clay sample obtained from Boring 4 on the Edenville left embankment, near the abutment contact. The coefficient of volume compressibility (m_V) and coefficient of consolidation (C_V) were estimated for the 1,000 to 2,000 psf increment (within the vertical effective stress range in the embankment), and then the permeability was computed using Eq F2-2:

$$k = C_V . m_V . \Upsilon_w \qquad \text{Eq. F2-2}$$

Where: k = the hydraulic conductivity in cm/s,
m_V = the coefficient of volume compressibility in centimeters squared per pound
C_V = the coefficient of consolidation in centimeters squared per second, and
Υ_w = the unit weight of water in pounds per cubic centimeter (pcf).

The permeability of the clay sample was found to be $4x10^{-7}$ cm/s. However, the clay sample is considered to be part of the native abutment. The IFT investigation found the clay fill in the embankment to consist of clayey sand and lean clay (Unified Soil Classification System [USCS]: SC, CL) with little compaction. Therefore, the clay fill was estimated to have an upper bound permeability of $1x10^{-5}$ cm/s and a lower-bound permeability of $1x10^{-7}$ cm/s.

F2-5.2.3 Silty Sand Fill

The silty sand fill in the upper portion of the Edenville left embankment was found to predominantly consist of loose silty sand with low plasticity. The Kozeny-Carmen relationship (Eq. F2-3) (Carrier 2003) was used to estimate the permeability of the silty sand embankment fill, using grain size data from the IFT investigation.

$$k = 1.99x10^4 \left[\frac{100\%}{\sum \left[\frac{f_i}{D_{li}^{0.404} x D_{si}^{0.595}} \right]} \right]^2 x \frac{1}{SF^2} x \frac{e^3}{1+e} \qquad \text{Eq. F2-3}$$

Where: k = the hydraulic conductivity in cm/s,
f_i = fraction of particles between two adjacent sieve sizes
D_{li} = the particle size of the coarser sieve in cm
D_{si} = the particle size of the finer sieve in cm
SF = the shape factor (taken as 7 here)
e = void ratio (taken as 0.75 here)

The estimated permeability was $7x10^{-5}$ cm/s. Therefore, the IFT estimated the permeability of the silty sand fill to be $1x10^{-4}$ cm/s.

F2-5.2.4 Native Sand Foundation

The native sand foundation below the Edenville left embankment was found to be relatively similar to the clean sand fill. However, the IFT found the native sand to generally have a higher non-plastic fines content and higher densities than the clean sand fill. Therefore, the IFT estimated the permeability of the native sand foundation to be similar to the lower bound of the clean sand fill at $1x10^{-3}$ cm/s.

F2-5.2.5 Glacial Till

The glacial till, which consists of stiff to hard sandy clay, was considered to be the base of the seepage model and therefore was assigned a low permeability (relative to the embankment and native sand foundation) of $1x10^{-8}$ cm/s.

F2-5.3 Boundary Conditions

A total head boundary condition of 675.8 feet was applied to the upstream face, which represents the normal operating reservoir surface elevation. The downstream face of the embankment and downstream ground surface were assigned a potential seepage face boundary condition. A total head boundary condition of 649 feet was applied to the downstream vertical face of the model. No flow boundaries were assigned to the upstream vertical face and the bottom surface of the model.

F2-5.4 Calibration

Data for calibrating the seepage model was limited to the water levels encountered during drilling, which were not well documented. Based on interpretation of the limited data on the boring logs for test holes performed in 2005 and 2010 near Sta. 3+50, it was assumed that water was encountered at 10 feet below the crest and 2 feet below the toe. The locations of the borings performed at the toe are not clear but were considered to be performed downstream of the toe drain collection ditch.

Table F2-2 is a summary of the calibrated material properties.

Table F2-2: Calibrated Material Properties for Steady-State Seepage Analysis

Material	Saturated Horizontal Hydraulic Conductivity, k_h (cm/s)	Anisotropy Ratio, k_h/k_v
Silty Sand Fill	$1x10^{-4}$	4
Clean Sand Fill	$1x10^{-2}$	4
Clayey Fill	$5x10^{-6}$	4
Native Sand	$1x10^{-3}$	10
Glacial Till	$1x10^{-8}$	10

F2-5.5 Results

The results of the analysis in terms of phreatic surface and flow vectors are shown in Figure F2-5. The phreatic surface is the upper line of saturation through the embankment and foundation. The flow vectors show the direction and relative magnitude of seepage through the embankment and foundation.

The slightly concave phreatic surface shape in the clayey fill is due to the high contrast in permeability and sharp material zone boundaries. It is more likely that the phreatic surface would enter the clean sand fill at the contact with the silty sand fill.

F2-6 Steady-State Slope Stability Analysis

F2-6.1 Methodology and Purpose

A suite of steady-state slope stability analyses was performed for the Edenville left embankment at the postulated failure section (Sta. 3+50).

The analysis calculated the factor of safety against slope instability for the Edenville left embankment, assuming three different shear strength assignments.

- Normal pool steady-state seepage with peak drained strengths for all embankment and foundation materials
- Normal pool steady-state seepage with an estimated angle of instability assigned to the saturated clean sand fill and peak drained strengths assigned for all other materials
- External flood load coupled with normal pool steady-state seepage pore pressures with liquefied strength assigned to the saturated clean sand fill and peak drained strengths assigned for all other materials

The purpose of the first analysis is to represent the safety margin against conventional slope instability under normal operating conditions. The purpose of the second analysis is to evaluate the factor of safety against potential triggering of static liquefaction within the loose clean sand soils. The purpose of the third analysis is to evaluate the factor of safety against failure after the liquefied sands reach residual strength.

Several slip surface definitions were used in Slope/W to evaluate the stability of the embankment and foundation, including searches with specification of entry-exit and blocks, and fully specified slip surfaces.

F2-6.2 Material Properties

Material properties needed for the slope stability analysis included peak drained (effective) friction angle (ϕ'), drained (effective) cohesion (c'), angle of instability (ϕ_L), and total unit weight (γ_t). Laboratory testing results, including isotropically consolidated, undrained triaxial shear with pore pressure (CIU') (ASTM D 4767), isotropically consolidated, drained triaxial shear (CID) (ASTM D 7181), and direct shear (ASTM D 3080) were used to estimate shear strength properties for materials with such data available. Standard penetration test (SPT) results from geotechnical investigations and laboratory index properties, including grain size analysis (ASTM D 6913) and Atterberg limits (ASTM D 4318), were also used to estimate shear strength properties, using empirical relationships. Total unit weights were developed from relatively undisturbed liner samples obtained during previous geotechnical investigations and from the IFT field and laboratory investigation. When laboratory or in situ field data were unavailable, engineering judgment based on experience with similar soils was used to select shear strength and total unit weight values.

F2-6.2.1 Clean Sand Fill

The IFT obtained bulk samples of the clean sand fill within the downstream section of the left embankment breach remnant. Samples consisting of similar soils were mixed into a composite sample and remolded to a relative density of 30 percent. Triaxial shear tests (CIU') were performed on the remolded samples, consolidated to 5, 15, and 30 psi, as described in Appendix E. A linear triaxial shear strength envelope was developed to represent the angle of instability based on the peak deviator failure criteria. The results supported an angle of instability (ϕ_L) of 18 degrees for 30 percent relative density, with some variation between results. The results of the laboratory testing are included in Appendix E. Variation in relative density would also be expected to have existed in the saturated clean sand fill. Lade (1993) presents a summary of ϕ_L values for various sands and found that for relative densities less than 50 percent, the range of ϕ_L was about 15 to 25 degrees. Therefore, 18 to 22 degrees is considered a reasonable range of instability angles.

The liquefied strength of the clean sand fill was estimated based on guidance provided by Idriss and Boulanger (2008). The residual shear strength ratio (S_r/σ'_{vc}) was estimated to range from 0.08 to 0.16 based on the equivalent clean sand blow counts, estimated from the Edenville left embankment investigations in the silty sand fill (McDowell 2005b; Mill Road Engineering 2010; Somat 2020). A residual shear strength ratio of 0.1 was assigned to the liquefied clean sand fill. The liquefied strength was applied as an equivalent friction angle (ϕ_{liq}) of 5.7 degrees.

A three-point CID envelope was also developed by remolding and consolidating the clean sand fill to the same conditions described for the CIU' tests. A peak drained friction angle and cohesion were defined by a linear envelope based on peak deviator failure criteria. The failure envelope was estimated to have an effective friction angle, ϕ', of 31 degrees and an effective cohesion intercept, c', of 0 psf.

SME (1987) performed a suite of CIU' tests on two samples of silty sand embankment fill. Table F2-3 summarizes the soil properties, testing parameters, and results.

Table F2-3: CIU' Test Results Performed by SME (1987).

CIU' Suite	USCS Classification and Fines Content	Consolidation Stress (psi)	Dry Unit Weight (pcf)	Mohr-Coulomb Effective Stress Strength Parameters
Boring 1 Sample 7	SM, 18%	10 20 30	119.1 119.8 119.8	ϕ'=36º, c'=0 psi
Boring 3 Sample 3	SM, 15%	10 20 30	105.3 104.9 104.4	ϕ'=34º, c'=4 psi

c'= drained cohesion
% = percent
ϕ' = friction angle
CIU' = undrained triaxial shear with pore pressure
pcf = pounds per cubic foot
psi = pounds per square inch
SME = Soils and Materials Engineers, Inc.
USCS = Unified Soil Classification System

Blystra (1991) describes one soil sample obtained from the downstream slope of the Tobacco embankment and one from the downstream slope of the Edenville embankment. Samples were classified as poorly graded sand (SP) with little to no fines. Direct shear tests resulted in peak drained friction angles of 34.6 and 34.3 degrees. The reconstituted dry unit weight of the sand was 115 pcf.

Somat (2020) performed a suite of three direct shear tests on a composite sample from split spoon samples SS3-SS9 from Boring 1, which ranged from 7.5 to 22.5 feet below ground surface (bgs) within clean sand fill (based on grain size analyses with fines content less than 4 percent). The results of the direct shear test showed contractive behavior with an ultimate shear strength of c' = 383 psf and ϕ'= 25.9 degrees at dry densities ranging from 94.5 to 97.1 pcf.

Three direct shear tests were also completed by Somat (2020) on a composite sample from split spoon samples SS10-SS15 from Boring 1, which ranged from 25 to 50 feet bgs. The material is considered clean sand fill based on the grain size analysis results with fines content less than 4 percent. The results of the direct shear test showed contractive behavior with an ultimate shear strength of c' = 402 psf and ϕ'= 25.1 degrees at dry densities ranging from 91.6 to 93.4 pcf.

In comparison, the peak drained friction angle estimated from SPT blow counts within sand fill (McGregor and Duncan 1998) resulted in a range of 30 to 33 degrees.

After comparing the various shear strength estimates, the IFT selected a peak drained friction angle and the angle of instability for the clean sand fill of 31 degrees and 18 to 22 degrees, respectively, each with a zero-cohesion intercept (c'=0). The selected liquefied strength was taken as an equivalent friction angle of 5.7 degrees.

Total unit weights obtained from relatively undisturbed liner samples taken from the Edenville right, Tobacco left, and Tobacco right embankment investigations (McDowell 2005a, 2005b; Mill Road Engineering 2010; SME 1987) generally ranged from 105 to 120 pcf. Sand cone tests obtained from the IFT field and laboratory investigation yielded total unit weights of 119 to 122 pcf. The selected total unit weight for the clean sand fill was taken as 110 pcf.

F2-6.2.2 Clayey Fill

The IFT investigation obtained several grab samples of the clayey fill in the upstream portion of the left embankment breach remnant. Grain size analyses and Atterberg limit tests were performed on the material with the results indicating fines content (FC) ranging from 39 to 61 percent, plasticity index (PI) ranging from 7 to 11, and USCS classification of CL-ML, CL, and SC. One sample, which was believed to be in native abutment material, resulted in FC of 95 percent, PI of 21, and USCS classification of CL.

Somat (2020) also obtained several samples of clayey fill and native clay abutment from Boring B-4, located on the crest of the left remnant embankment section near the left abutment, and performed grain size analyses, Atterberg limits, and one three-point suite of triaxial shear testing (CIU’). Although many of the clay samples obtained during the Somat (2020) investigation are considered to be in native clay, the information was still considered in assigning shear strength properties.

Correlations presented in Duncan et al. (1989) were used to estimate the peak drained friction angle from the plasticity index test results. A range of 35 to 38 degrees was estimated for the IFT samples obtained in clayey fill, and 31 degrees for the sample obtained in native clay abutment. A range of 28 to 38 degrees was estimated for the Somat (2020) samples.

The triaxial shear test by Somat (2020) was performed on relatively undisturbed samples from the native clay abutment with dry densities of 100.2, 101.0, and 98.2 pcf at consolidation stresses of 12.3, 28.2, and 22.3 psi, respectively. The peak effective stress strength was defined by a linear envelope with an effective friction angle, ϕ’, of 26.9 degrees and an effective cohesion intercept, c’, of 38 psf.

After comparing the various shear strength estimates, the IFT selected a peak drained strength of ϕ’=28 degrees and c’=0 psf.

Total unit weights, obtained from relatively undisturbed liner samples taken from the Edenville left embankment investigations (McDowell 2005b; Mill Road Engineering 2010), generally ranged from 107 to 132 pcf. Total unit weights based on dry unit weight correlations from SPT blow counts (McGregor and Duncan 1998) and moisture content tests resulted in a range of 107 to 131 pcf. The selected total unit weight for the clayey fill was taken as 115 pcf.

F2-6.2.3 Silty Sand Fill

The IFT investigation obtained several grab (bag) samples of the silty sand fill in the upper portion of the left embankment breach remnant. Grain size analyses and Atterberg limit tests were performed on the material, with results indicating FC from 29 to 31 degrees; only one Atterberg limit test was performed and resulted in a PI of 2 and a USCS classification of SM.

The field investigations of the Edenville left embankment (McDowell 2005b; Mill Road Engineering 2010; Somat 2020) found the upper portion of the embankment to be generally cohesionless, with FC ranging from 13 to 99 percent and USCS classifications of ML and SM.

The peak drained friction angle estimated from SPT blowcounts (McGregor and Duncan 1998) ranged from 30 to 33 degrees. Therefore, the IFT selected a peak drained strength of ϕ’=30 degrees and c’=0 psf.

Total unit weights obtained from relatively undisturbed liner samples taken from the Edenville left embankment investigations (McDowell 2005b; Mill Road Engineering 2010; Somat, 2020), generally ranged from 107 to 125 pcf. Total unit weights based on dry unit weight correlations from SPT blowcounts (McGregor and Duncan 1998) and moisture content tests resulted in a range from 93 to 128 pcf. Shelby tube samples obtained from the IFT field and laboratory investigation yielded total unit weight estimates of 112 to 124 pcf. The selected total unit weight for the silty sand fill was taken as 110 pcf.

F2-6.2.4 Native Sand Foundation

The IFT investigation obtained one grab sample of the native sand at the bottom of the left embankment breach remnant. A grain size analysis and Atterberg limit test were performed on the material. The tests resulted in FC of 11 percent, a non-plastic material, and USCS classification of SP-SM.

The field investigations of the Edenville left embankment (McDowell 2005b; Mill Road Engineering 2010) found the native sand to be generally cohesionless with FC of 18 to 35 percent and a USCS classification of SM. The SPT blowcounts also showed that the native sand was denser than embankment fill.

The peak drained friction angle estimated from SPT blowcounts (McGregor and Duncan 1998) ranged from 32 to 34 degrees.

The results of the CIU' tests performed by SME (1987) on the native foundation sand, summarized in Table F2-3, were considered to be similar in gradation and density to the native sand observed below the Edenville left embankment.

After comparing the various shear strength estimates, the IFT selected a peak drained strength of $\phi'=34$ degrees and $c'=0$ psf for the native foundation sand.

Total unit weights, obtained from relatively undisturbed liner samples taken from the Edenville left embankment investigations (McDowell 2005b; Mill Road Engineering 2010), generally ranged from 107 to 117 pcf. Total unit weights based on dry unit weight correlations from SPT blow counts (McGregor and Duncan 1998) and moisture content tests resulted in a range of 121 to 136 pcf. The selected total unit weight for the native sand foundation was taken as 125 pcf.

F2-6.2.5 Glacial Till

The high blow counts and cohesive nature indicate that the glacial till can be considered the base of the stability model, with relative strengths much greater than the overlying native sand and embankment fills. Therefore, the glacial till was assigned an "Impenetrable" strength in Slope/W, meaning that no potential slip surface was able to pass through the material.

Total unit weights, obtained from relatively undisturbed liner samples taken from the Edenville left and right embankment investigations (McDowell 2005a, 2005b; Mill Road Engineering 2010), generally ranged from 124 to 144 pcf. The selected total unit weight for the glacial till was taken as 130 pcf.

The material properties used in the slope stability analyses are summarized in Table F2-4.

Table F2-4: Summary of Sta. 3+50 Material Properties for Steady-State Slope Stability Analysis.

Material	Total Unit Weight (pcf)	Peak Drained Strength		Angle of Instability, ϕ_L (deg)	Liquefied Strength, ϕ_{Liq} (deg)
		Effective Cohesion, c' (psf)	Effective Friction Angle, ϕ' (deg)		
Clean Sand Fill	110	0	31	18	5.7
Clayey Fill	115	0	28	-	-
Silty Sand Fill	110	0	30	-	-
Native Sand Foundation	125	0	34	-	-
Glacial Till	130	-	-	-	-

F2-6.3 Pore Pressure

Pore pressures were assigned via a piezometric surface based on the results of the steady-state seepage analysis described above. The slightly concave phreatic surface shape in the clayey fill shown in Figure F2-5 is due to the high contrast in permeability and sharp material zone boundaries. It is more likely that the phreatic surface would enter the clean sand fill at the contact with the silty sand fill. Therefore, the piezometric line shown in Fig F2-6 was used to represent the steady-state seepage conditions for the stability analyses.

For the liquefied case, the pore pressures were kept the same as the steady-state condition and a surcharge load was placed on the upstream slope to represent the flood pool during the May 2020 event. Although the pore pressures likely increased during the flood event, it is difficult to determine, with accuracy, the change in pore pressures due to limited data on the cross section, piezometric levels prior to the flood, and uncertainty in the accuracy of a transient seepage model.

F2-6.4 Results

The results of the analyses are shown in Figures F2-6 through F2-9. The analyses resulted in a factor of safety against slope instability of 1.3, using peak drained strengths, representing the estimated safety margin under normal operating conditions for the postulated cross section. The analysis resulted in a factor of safety of 1.0 (computationally slightly above 1.0) when using an angle of instability of 18 degrees for the saturated clean sand fill (Figure F2-7) and a factor of safety of 1.1 when using an angle of instability of 22 deg (Figure F2-8), representing the condition when the loose, saturated sands reach a stress state near the instability line.

The analysis resulted in a factor of safety of 0.6 when using the liquefied strength for the saturated clean sand fill (Figure F2-9), representing the condition when the loose, saturated sands collapse and reach residual strength. The average mobilized shear stress along the entire slip surface was calculated to be 198 psf.

F2-7 Stress State Analysis

F2-7.1 Methodology and Purpose

A stress state analysis was performed for the Edenville left embankment for the failure location (Sta. 3+50) with the postulated embankment cross section and a normal lake level, El. 675.8 feet (NGVD29). This analysis was performed to estimate the in situ effective stresses and shear stresses that could reasonably have existed under normal pool conditions prior to the May 2020 flood event.

The stress state analysis was performed using the SIGMA/W computer program, Version 11.1.3 (GEOSLOPE 2021c). Sigma/W is a finite element software package that can be used to compute stress-deformation analyses of earth structures.

The stress state analysis was performed using the *In-Situ* analysis type in SIGMA/W. The In-Situ analysis establishes initial stresses as a result of the self-weight of soil using the *Gravity Activation* method. Initial stresses computed by the program include total and effective stresses as well as shear stresses.

F2-7.2 Material Properties

Material properties needed for the stress state analysis included total unit weight and the coefficient of earth pressure at rest (K_0) (i.e., the ratio of horizontal to vertical effective stresses). Total unit weights were developed as discussed in Section F2-7, Stress State Analysis.

The coefficient of earth pressure at rest (K_0) for the embankment and foundation materials was taken as a uniform value of 0.6 for this simplified analysis. The actual K_0 likely varies throughout the embankment and is difficult to estimate with confidence. However, this simplified analysis was intended to represent an estimate for embankment soils to evaluate the plausibility of conditions that may have existed in the embankment prior to failure.

F2-7.3 Pore Pressure

The pore pressures from the steady-state seepage analysis were directly imported into the SIGMA/W model.

F2-7.4 Boundary Conditions

A hydrostatic pressure boundary condition of 675.8 feet was applied to the upstream face, which represents the normal operating lake elevation. A fixed x (force/displacement) boundary condition was assigned to the upstream and downstream vertical faces of the model. A fixed x/y (force/displacement) boundary condition was assigned to the bottom surface of the model.

F2-7.5 Results

The results of the analysis are presented in Figure F2-10. The results show the contours of stress ratio (q/p') calculated in the embankment and foundation. The stress ratio is defined as the deviatoric stress (q) divided by the mean effective stress (p'). The high stress ratios on the upstream slope and at the downstream toe are due to the low confining stresses and high shear stresses induced by the sloping ground and reservoir load.

The critical slip surface from the liquefied strength case (described in Section F2-6, Steady-State Slope Stability Analysis) is shown in Figure F2-10. The mean effective stresses (p') and deviatoric stresses (q) along the slip surface are plotted in Figure F2-11. The figure also shows the estimated drained failure envelope of 31 degrees and estimated range of instability lines of 18 and 22 degrees. The majority of initial stresses along the slip surface fall within the range of the instability lines. Some stresses at the toe are shown to be above the drained friction envelope; however, the excess stresses would shed to adjacent elements (i.e., stress redistribution). The results indicate that the relatively small changes in pore pressure and/or shear stresses could cause the stress ratio to intersect or exceed the instability line.

F2-8 Kinetic Analysis

F2-8.1 Methodology and Purpose

The IFT also completed a kinetic analysis of the possible static liquefaction failure scenario for the Edenville left embankment, using the simplified procedure originally proposed by Davis et al. (1988). This procedure consisted of calculating accelerations, velocities, and deformations for a potential failure mass within the embankment. A travel path for the center of gravity of the mass is assumed, and the initial acceleration is based on the force imbalance created by the reduction of strength to the steady-state or residual strength. The changes in accelerations, velocities, and deformations are calculated incrementally in time steps.

The pre-failure slip surface geometry at Sta. 3+50 was taken as the critical slip surface from the liquefied strength case slope stability analysis. The initial center of gravity for the potential sliding mass was obtained in AutoCAD. A hyperbola was adopted to represent the travel path for the center of gravity of the sliding mass from initial position to final position. The final position of the center of gravity of the sliding mass was estimated, based on the dam failure video, to be about 30 feet from the downstream toe of the embankment.

The net force on the sliding mass is given by the equation:

$$F=(W \sin \theta) - (S_m L) \qquad \text{Eq. F2-4}$$

Where: F = the net force on the sliding mass
W = the weight of the sliding mass
S_m = the mobilized shear strength
L = the length of the failure surface
θ = the slope of the tangent to the curve of the hyperbola

The weight of the sliding mass was taken as the soil unit weight (110 pcf) multiplied by the area of the sliding mass, obtained from AutoCAD, and multiplied by a unit width (1 foot). Due to the low phreatic surface level estimated from the steady-state seepage analysis, the saturated zone within the failure mass was found to be about 12 percent of the total. Thus, a buoyant unit weight was applied to 12 percent of the failure mass. The initial length of the failure surface was also obtained from AutoCAD. Although the length of the failure surface is sometimes assumed to stretch during the deformation, the length of the failure surface was assumed to be constant in the IFT's calculations. Estimation of the length of the final mass after failure indicated an increase in length of no more than about 10 percent compared to the initial length. The geometry used in the kinetic analysis is shown in Figure F2-12.

The slope of the tangent to the curve of the hyperbola was calculated incrementally for each time step based on the change in position of the center of mass along the assumed travel path. The mobilized shear strength along the failure surface was assumed to reduce by 50 percent over the course of the failure due to hydroplaning (i.e., slide material riding on a layer of water). The value of the initial mobilized shear strength was revised until a solution of the kinetic model was obtained. The 50 percent reduction over the course of the failure was used by Olson et al (2000). Analyses were also completed for reductions of 0 percent, 25 percent, and 75 percent, with results that were similar to those for 50 percent.

The acceleration, velocity, and displacement kinetics were obtained numerically by incremental spreadsheet calculations. The acceleration was calculated for each incremental time step by dividing the net force by the mass of the slide. The velocity was calculated by integrating the acceleration with time by multiplying the average acceleration for each time step by the initial velocity at the start of each time step. The displacement was calculated for each incremental change of the center of gravity position along the travel path. The displacement, velocity, and acceleration were plotted and the mobilized shear strength was revised to obtain a reasonable fit with the observed displacement, velocity, acceleration, and duration, observed from the dam failure video and the American Society of Civil Engineers (ASCE) pixel tracking results (ASCE 2021).

The purpose of this analysis was not to provide a precise analysis of the kinetics of the failure, but rather to demonstrate that the observed kinetics of the failure mass are consistent with a flow liquefaction type failure.

F2-8.2 Results

The kinetic analysis resulted in an estimated initial mobilized shear strength of 129 psf. This value resulted in the kinematics shown in Figure F2-13 as plots of deformation, velocity, and acceleration versus time. An ASCE investigation team completed pixel tracing analysis of the failure video and concluded that the failure mass reached a velocity of about 5 meters per second or 16.4 feet per second (ft/sec), which is close to the 15.8 ft/sec resulting from the simplified kinetic analysis. In addition, the displacement time of about 7.9 seconds from the kinetic analysis is similar to the failure time interpreted from the dam failure video.

F2-9 Key Takeaways

The key takeaways from the IFT's geotechnical analyses are described below.

- The analysis of the embankment geometry at Sta. 3+50, using proportional material zoning and the same material properties used in the 1987 slope stability analysis of Sta. 48+00, indicated a factor of safety equal to about 1.0 for steady-state (normal operating) conditions, which is much lower than the Federal Energy Regulatory Commission's (FERC's) required minimum value of 1.5. This result is also lower than the factor of safety calculated for Sta. 48+00 due to an overall steeper average downstream slope at Sta. 3+50. This result shows that the Edenville left embankment may have been more critical with respect to stability than the embankment sections that were analyzed. Had an analysis similar to this been done, it would likely have led to either further investigation of that section or to a recommended stabilizing modification of that section.
- The results of the steady-state seepage analysis show that the majority of seepage occurs through the native (foundation) sand layer, flowing below the clayey fill and into the downstream clean sand fill. Some seepage occurs through the silty sand upper portion of the embankment and into the clean sand fill. For the postulated cross section, the phreatic level in the clean sand fill is estimated to be only a few feet above the native sand layer due to the high permeability of the clean sand fill. The model indicates a phreatic surface that breaks out at the downstream toe about 1 foot above the native sand layer. This condition was not readily observed or documented in the field; however, the model did not account for the effects of the underdrain system, which would likely maintain a phreatic surface slightly below the ground surface at the downstream toe. The slightly concave phreatic surface shape in the clayey fill is due to the high contrast in permeability and sharp material zone boundaries. It is more likely that the phreatic surface would enter the clean sand fill at the contact with the silty sand fill.
- The results of the stress state analysis show that the initial stress state of the postulated embankment was close to a postulated instability line under normal pool conditions, and small increases in pore pressures and/or shear stress could lead to a rapid transition to residual strengths and collapse of the loose sands. The flood event in May 2020 could have plausibly provided an increase in both shear stress and pore pressure, which led to the failure.
- The use of peak drained strengths estimated for the materials near Sta. 3+50 resulted in a factor of safety of 1.3 for the postulated cross section, lower than the FERC's required minimum value, indicating the Edenville left embankment had a marginal safety factor under steady-state seepage conditions at normal water level El. 675.8 feet (normal operating conditions). A similar factor of safety resulted in recommendations to implement embankment modifications to improve the stability at other sections of the dam (e.g., Sta. 48+00).
- The factor of safety was approximately equal to 1.0 under normal operating conditions, when the saturated clean sand was assigned a strength equal to the estimated angle of instability. This analysis shows that if the stress state in the saturated loose sands were to approach the instability line, then the embankment would be on the verge of failure.

- The factor of safety was significantly less than 1.0 under normal operating conditions when the saturated clean sand was assigned a liquefied strength. This shows that if the sand were to liquefy, then the large imbalance in driving to resisting forces would most certainly result in failure.
- The kinetic analysis provides strong evidence that areas of the potential slip surface would have to be experiencing a flow-type (liquefaction) failure to produce the kinetics observed during failure.

F2-10 References

American Society of Civil Engineers. 2021. "Edenville and Sanford Dam Failures Field Reconnaissance Report." *Geotechnical Special Publication No. 327*. Prepared by Embankments, Dams, and Slopes Committee of the Geo-Institute. Edited by D. Pradel and A. Lobbestael.

A. R. Blystra & Associates (Blystra). 1991 (March). *Initial Consultant's Safety Inspection Report, Edenville Hydroelectric Project*.

American Society for Testing and Materials (ASTM). 2017. *D6913/D6913M-17, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.* ASTM International.

______. 2018 (April). *D4318-17e1, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. ASTM International.

______. 2020 (February). *D7181-20, Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils.* ASTM International.

______. 2020 (April). *D2435/2435M-11(2020), Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.* ASTM International.

______. 2020 (April). *D3080/d3080M-11 (Withdrawn 2020), Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.* ASTM International.

______. 2020 (April). *D4767-11, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.* ASTM International.

______. 2022 (April). *D2434-22, Standard Test Methods for Measurement of Hydraulic Conductivity of Coarse-Grained Soils.* ASTM International.

Carrier III, W. 2003. *Goodbye, Hazen; Hello, Kozeny-Carman.* Canadian Geotechnical Journal. The Canadian Geotechnical Society.

Davis, A.P., G. Castro, and S.J. Poulos. 1988. *Strengths backfigured from liquefaction case histories.* Proc., 2nd Int. Conf. on Case Histories in Geotechnical Engineering., St. Louis, Vol. IV, 1693-1701.

Duncan, J. M., R. C. Horz, and T. L. Yang. 1989 (August). *Shear Strength Correlations for Geotechnical Engineering*. Report of a study performed by the Virginia Tech Center for Geotechnical Practice and Research. CGPR #4. Page 99. Available: https://www.scribd.com/doc/246803736/CGPR-4-Shear-Strength-Correlations-for-Geotechnical-Engineering.

Fell, R., J. P. Macgregor, D. H. Stapledon, G. Bell, and M. Foster. 2015. *Geotechnical Engineering of Dams (2nd ed.)*. CRC Press/Balkema, Leiden.

Idriss, I. M., and R. W. Boulanger. 2008. *Soil Liquefaction During Earthquakes, MNO – 12.* Earthquake Engineering Research Institute. California.

GEOSLOPE International Ltd. (GEOSLOPE). 2021. Geostudio, SEEP/W computer program, Version 11.1.3 (2021 R2). Seequent Limited.

———. 2021b. Geostudio, SLOPE/W computer program, Version 11.1.3 (2021 R2). Seequent Limited.

———. 2021c. Geostudio, SIGMA/W computer program, Version 11.1.3 (2021 R2). Seequent Limited.

Google Earth. 2022. Available: https://earth.google.com/web/.

Lade, P.V. 1993. *Initiation of Static Instability in the Submarine Nerlerk Berm.* Canadian Geotechnical Journal 30:895–904.

McDowell and Associates (McDowell). 2005a (October 21). *Soils Exploration, Edenville and Secord Dams.*

———. 2005b (July 18). *Soils Investigation, Edenville Hydroelectric Project, Auxiliary Spillway Construction, Edenville and Meridian Roads.*

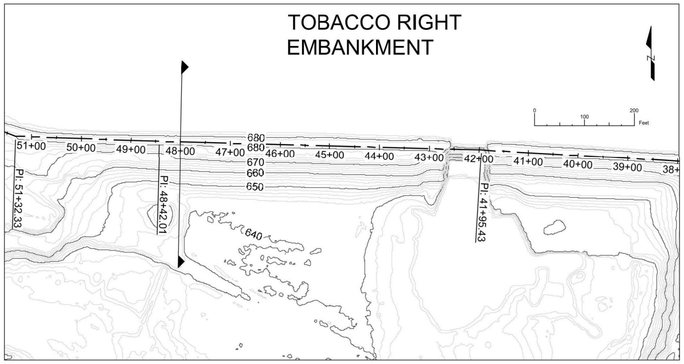
McGregor, J. A., and J.M. Duncan. 1998 (October). *Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice*. Report of a study performed by the Virginia Tech Center for Geotechnical Practice and Research. CGPR #12. Page 64, Table 21. Available: https://cgpr.cee.vt.edu/cgpr-publications-search/.

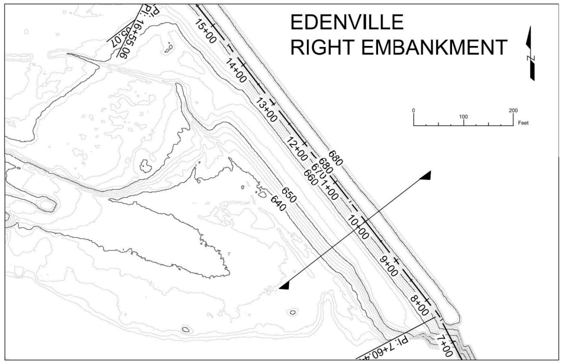
Mill Road Engineering. 2010. *Liquefaction Analysis, Edenville Hydroelectric Project.*

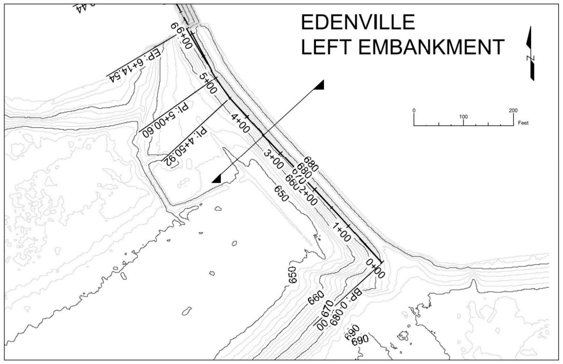
Olson, S.M., Castro, G., Stark, T.D. 2000 (December). *1907 Static Liquefaction flow failure of the north dike of Wachusett Dam.* Journal of Geotechnical and Geoenvironmental Engineering., ASCE, Vol. 126(12), pp. 1184-1193.

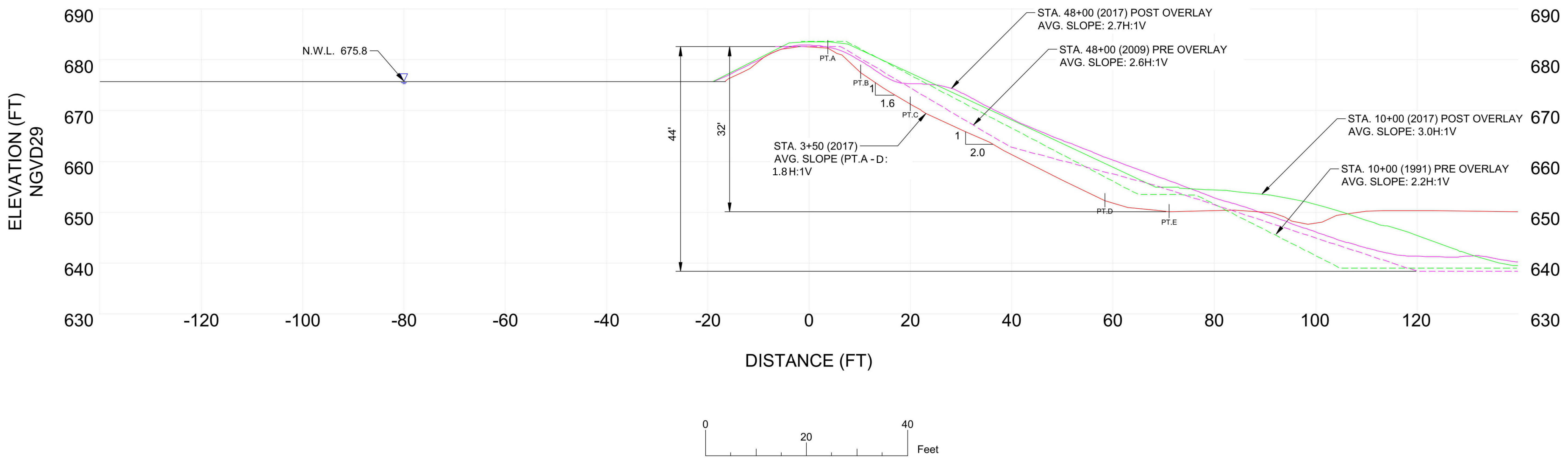
Soil and Materials Engineers, Inc. (SME). 1987 (April 30). *Geotechnical Evaluation Report, Edenville Dam – Station 48+00.*

Somat Engineering Inc. (Somat). 2020. *Geotechnical Data Report, Edenville Dam Repair.*



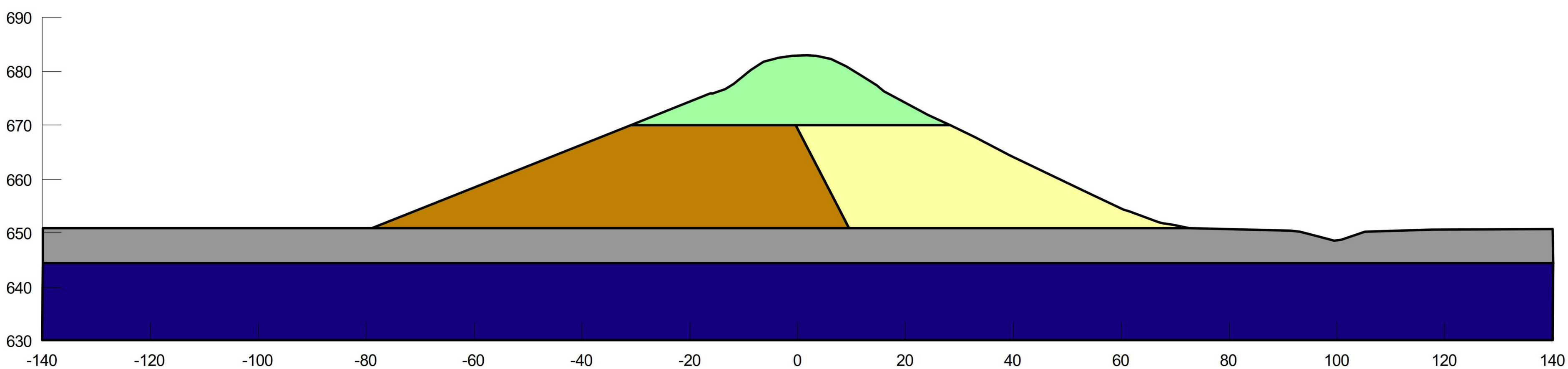






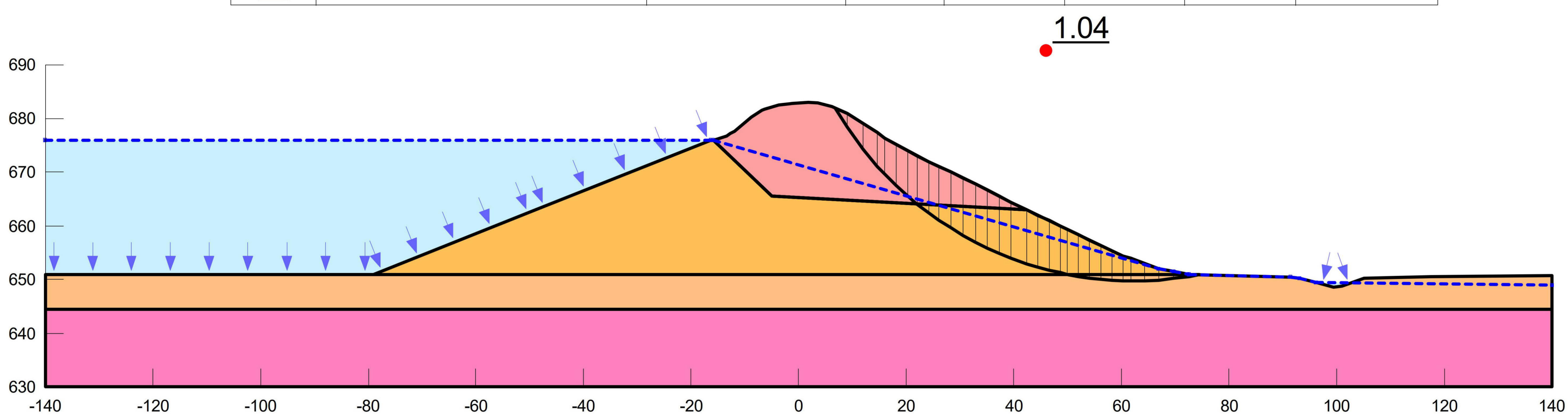
Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Section Geometry Comparison	Figure F2-2

Color	Name
	Silty Sand Fill
	Clean Sand Fill
	Clayey Fill
	Native Sand Foundation
	Glacial Till



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	IFT Analysis Geometry	Figure F2-3

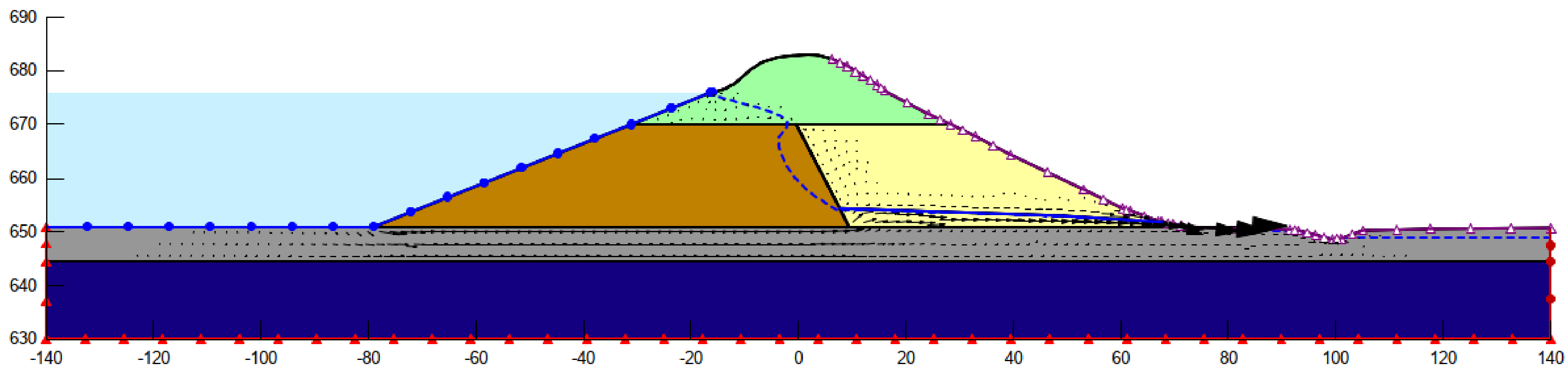
Color	Name	Material Model	Unit Weight (pcf)	Cohesion (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	1987 - Fine to Medium Sand Fill	Mohr-Coulomb	120		0	35	1
	1987 - Natural Sand	Mohr-Coulomb	120		0	34	1
	1987 - Sandy Clay Hardpan	Undrained (Phi=0)	130	4,500			1
	1987 - Silt and Sand Fill	Mohr-Coulomb	105		0	32	1



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	1987 Slope Stability Comparison Analysis Results	Figure F2-4

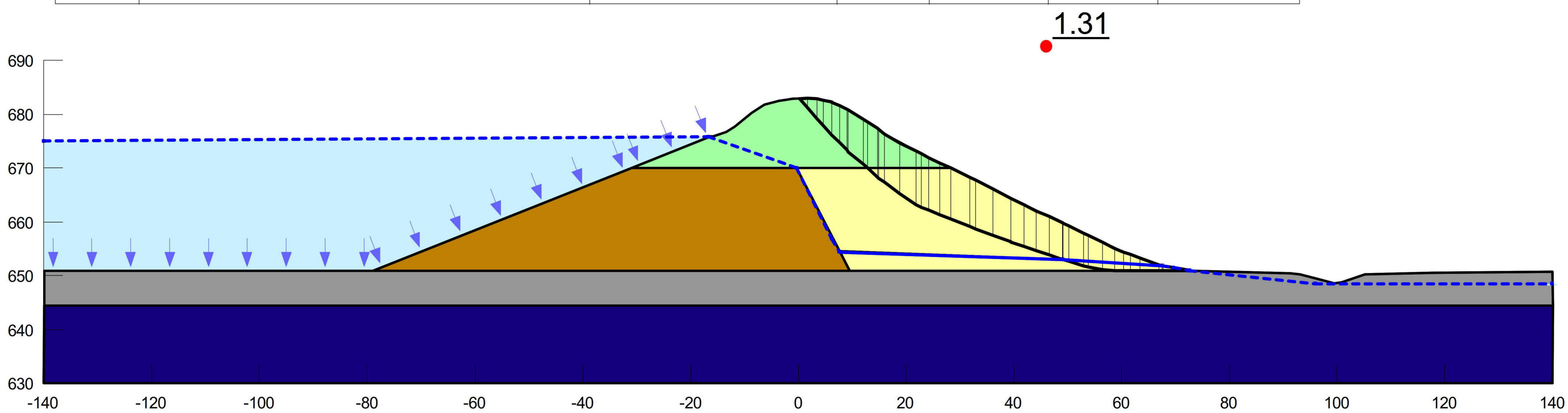
Color	Name	Sat Kx (cm/sec)	K-Function	Ky'/Kx' Ratio
	Silty Sand Fill		Silty Sand k C2 kh 1x10-4	0.25
	Clean Sand Fill		Clean Sand k C4 kh 1x10-2	0.25
	Clayey Fill		Clay kh 5x10-6	0.25
	Native Sand Foundation	0.001		0.1
	Glacial Till	1e-08		0.1

Color	Name	Category	Kind	Parameters
	Downstream GWT	Hydraulic	Water Total Head	649 ft
	Drainage	Hydraulic	Water Rate	0 ft³/sec
	No Flow	Hydraulic	Water Rate	0 ft³/sec
	Total Head NWL	Hydraulic	Water Total Head	675.8 ft



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Steady-State Seepage Analysis Results	Figure F2-5

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Silty Sand Fill	Mohr-Coulomb	110	0	30	1
	Clean Sand Fill	Mohr-Coulomb	110	0	31	1
	Clayey Fill	Mohr-Coulomb	115	0	28	1
	Native Sand Foundation	Mohr-Coulomb	125	0	34	1
	Glacial Till	Bedrock (Impenetrable)				1



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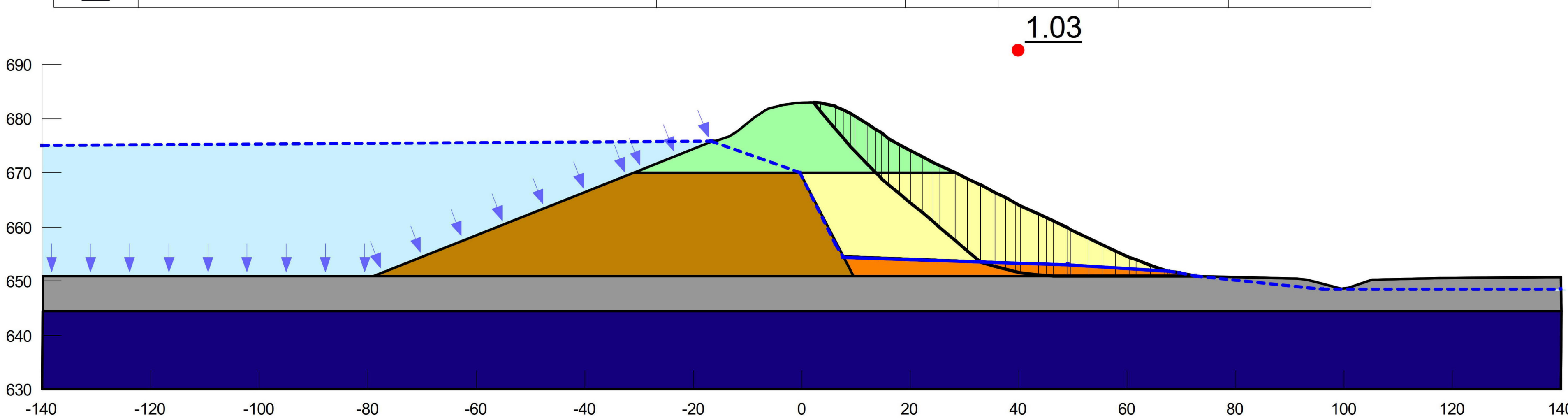
Steady-State Slope Stability Analysis Results

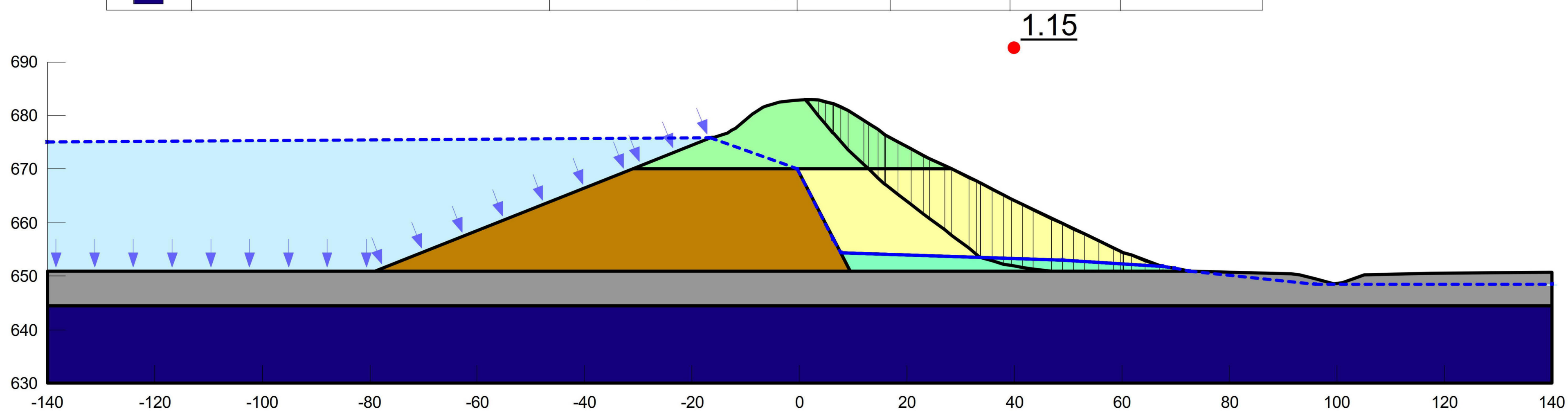
Figure F2-6

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Silty Sand Fill	Mohr-Coulomb	110	0	30	1
	Saturated Clean Sand Fill	Mohr-Coulomb	110	0	18	1
	Clean Sand Fill	Mohr-Coulomb	110	0	31	1
	Clayey Fill	Mohr-Coulomb	115	0	28	1
	Native Sand Foundation	Mohr-Coulomb	125	0	34	1
	Glacial Till	Bedrock (Impenetrable)				1



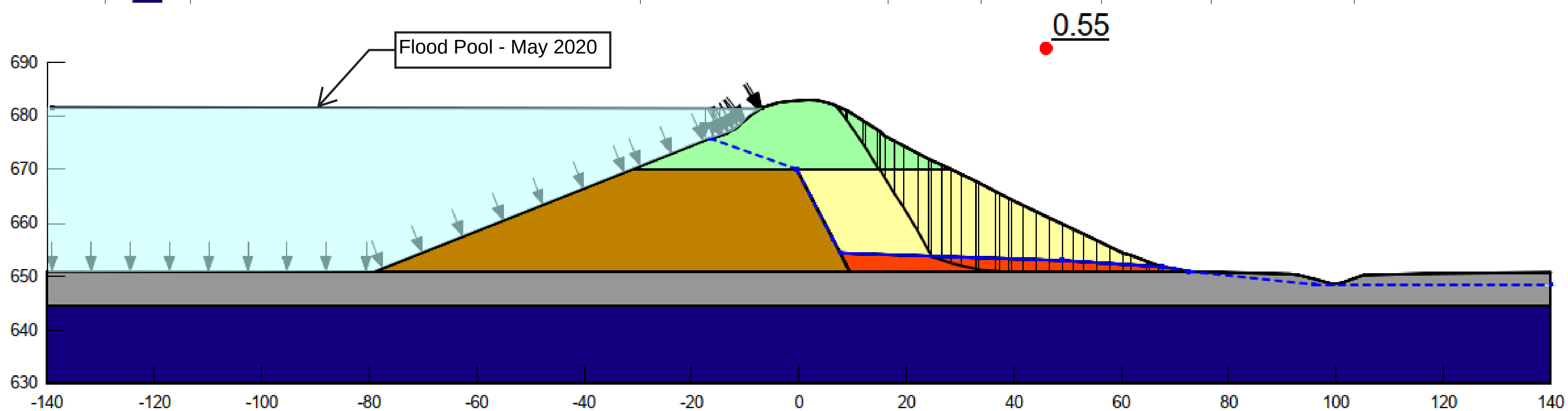
Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Steady-State Slope Stability Analysis Results 18-Degree Angle of Instability	Figure F2-7

Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Silty Sand Fill	Mohr-Coulomb	110	0	30	1
	Saturated Clean Sand Fill	Mohr-Coulomb	110	0	22	1
	Clean Sand Fill	Mohr-Coulomb	110	0	31	1
	Clayey Fill	Mohr-Coulomb	115	0	28	1
	Native Sand Foundation	Mohr-Coulomb	125	0	34	1
	Glacial Till	Bedrock (Impenetrable)				1



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Steady-State Slope Stability Analysis Results 22-Degree Angle of Instability	Figure F2-8

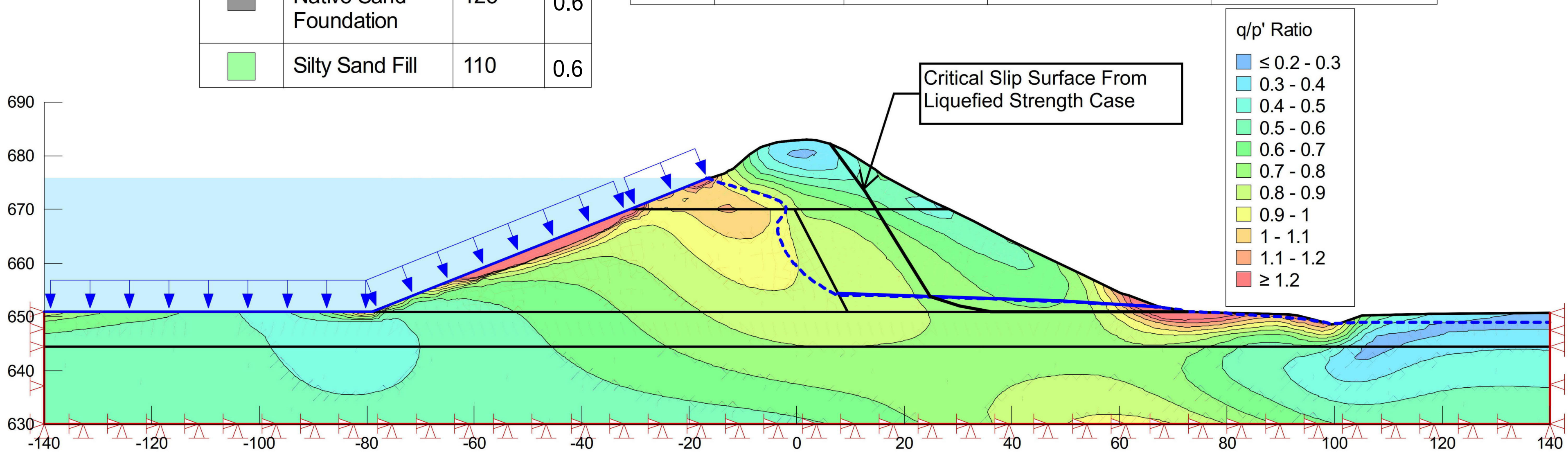
Color	Name	Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)	Piezometric Line
	Silty Sand Fill	Mohr-Coulomb	110	0	30	1
	Clean Sand Fill	Mohr-Coulomb	110	0	31	1
	Saturated Clean Sand Fill	Mohr-Coulomb	110	0	5.7	1
	Clayey Fill	Mohr-Coulomb	115	0	28	1
	Native Sand Foundation	Mohr-Coulomb	125	0	34	1
	Glacial Till	Bedrock (Impenetrable)				1



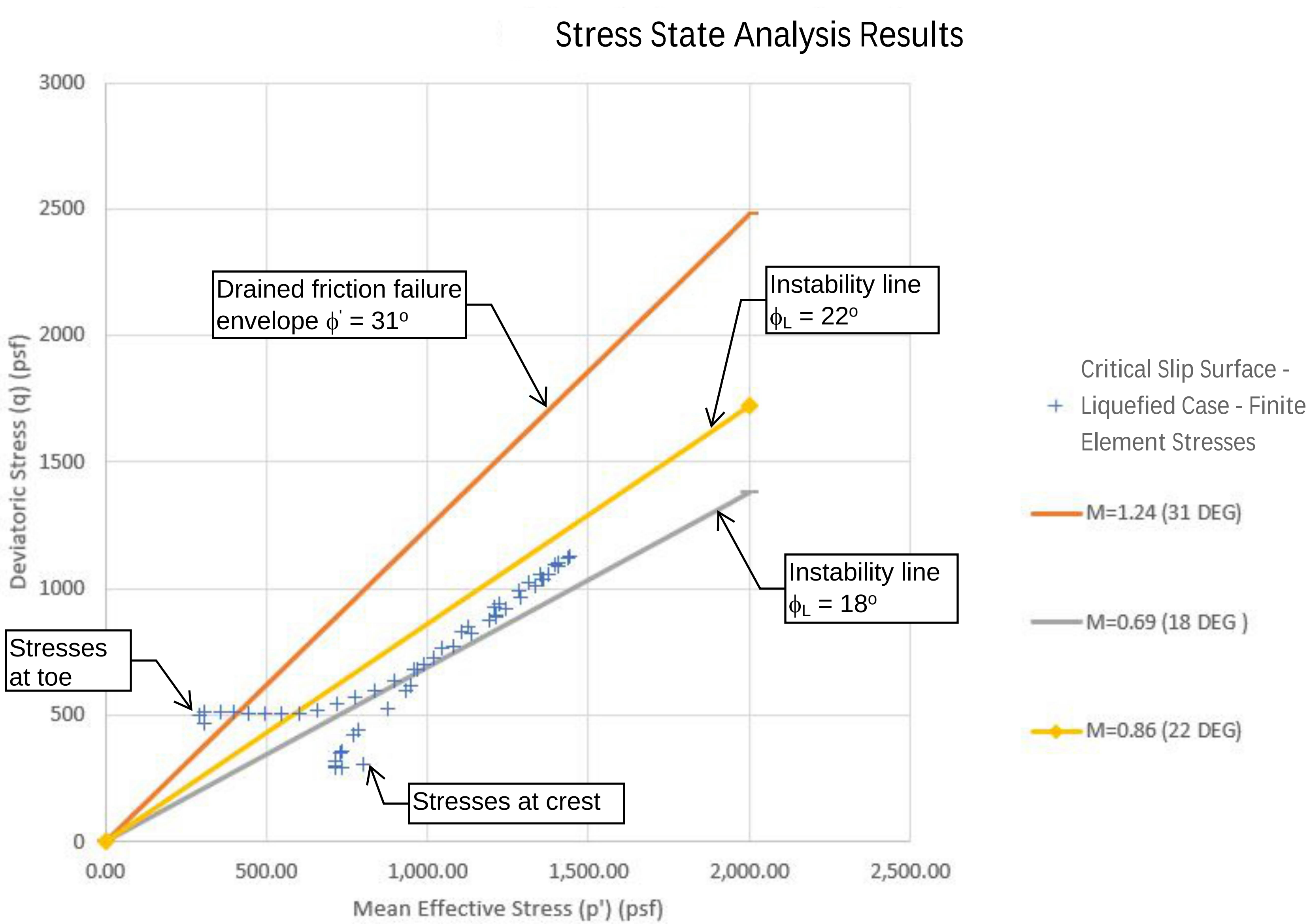
Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Steady-State Slope Stability Analysis Results Liquefied Strength	Figure F2-9

Color	Name	Unit Weight (pcf)	K0
	Clayey Fill	115	0.6
	Clean Sand Fill	110	0.6
	Glacial Till	130	0.6
	Native Sand Foundation	125	0.6
	Silty Sand Fill	110	0.6

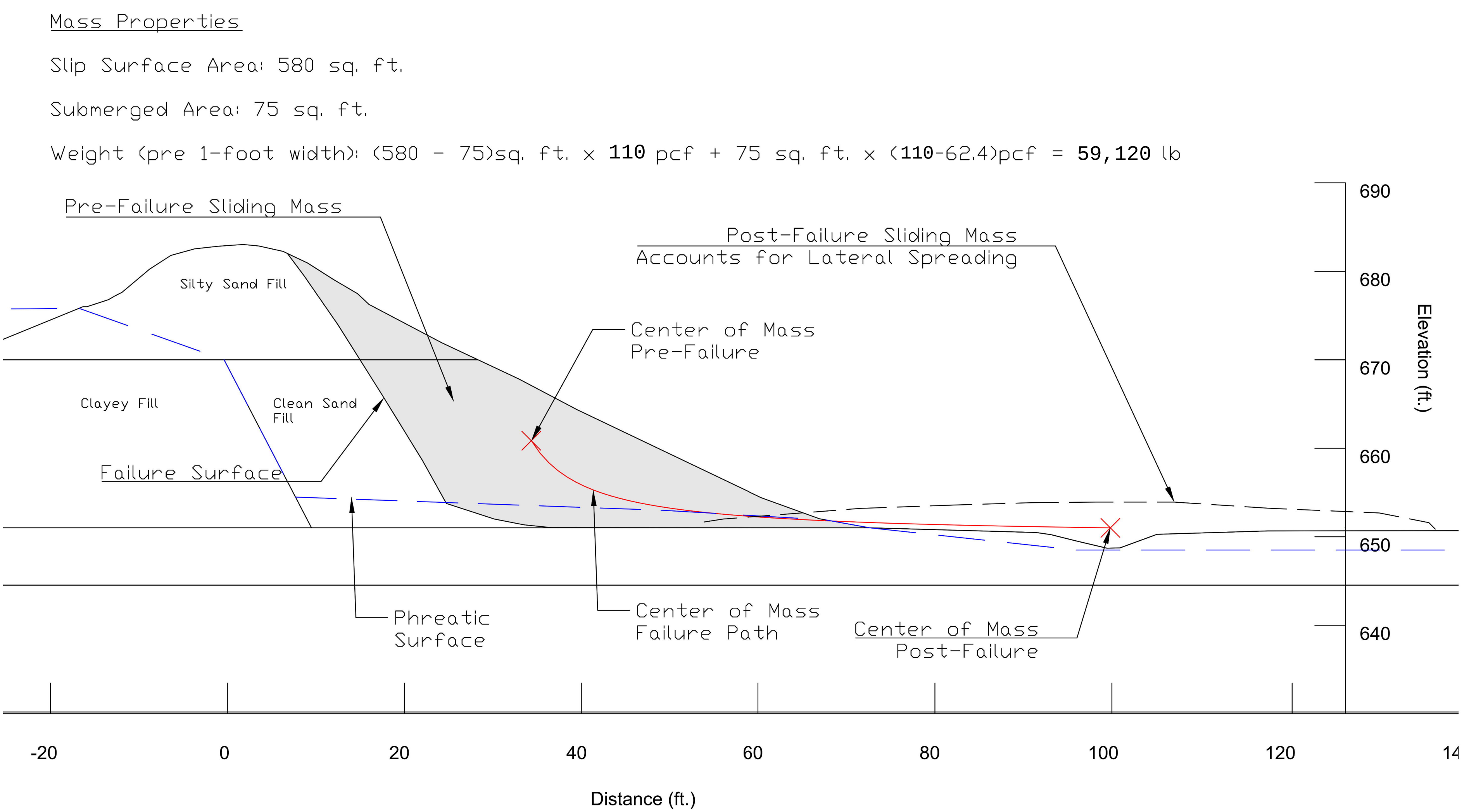
Color	Name	Category	Kind	Parameters
	Fixed X	Stress/Strain	Force/Displacement	X-Displacement: 0 ft
	Fixed X/Y	Stress/Strain	Force/Displacement	X-Displacement: 0 ft Y-Displacement: 0 ft
	Total Head Boundary NWL	Stress/Strain	Hydrostatic Pressure	Elevation: 675.8 ft Unit Weight per Unit Depth: 62.430189 pcf



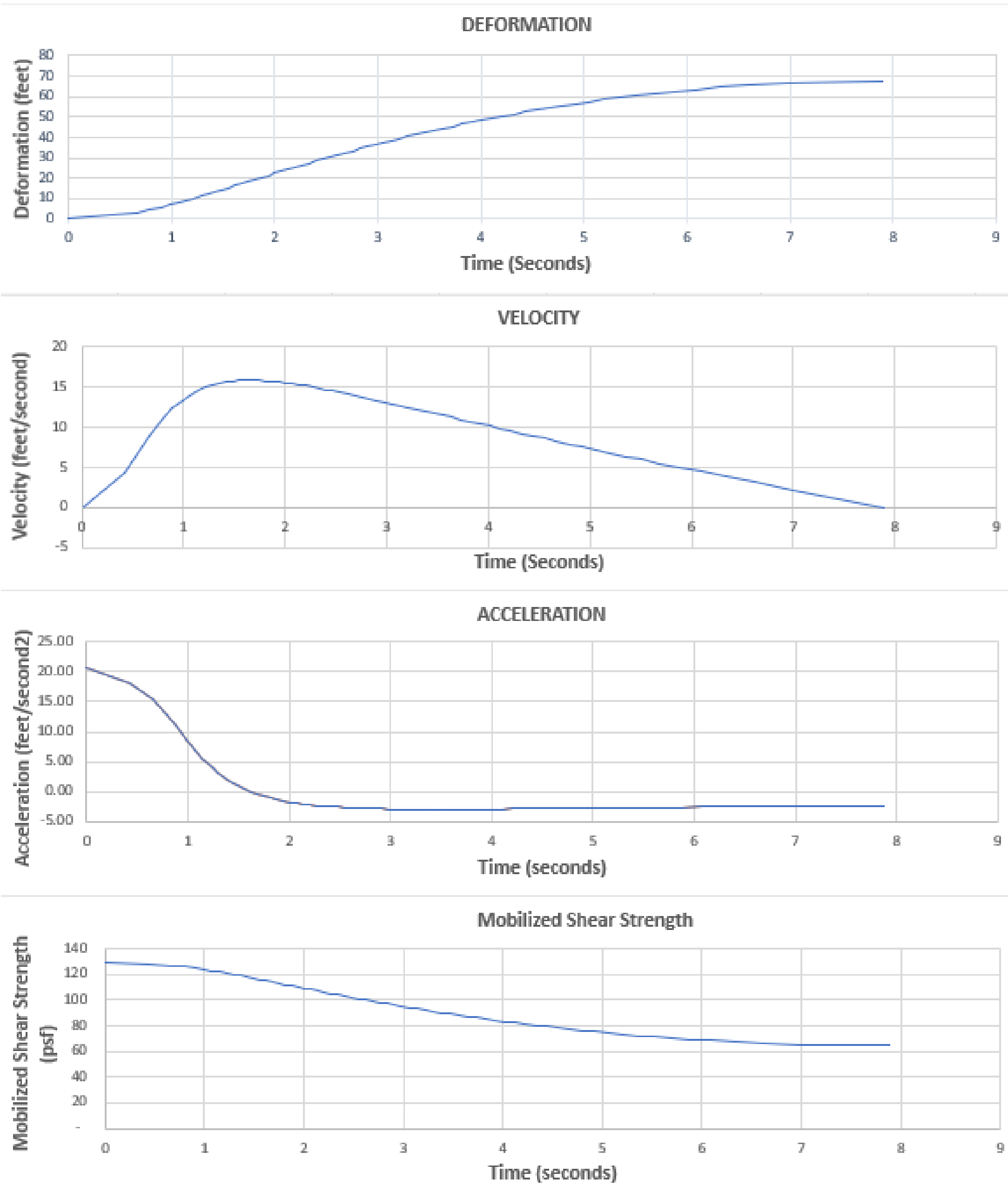
Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Stress State Analysis Results	Figure F2-10



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	**Stress State Analysis** **q/p' Plot**	Figure F2-11



Independent Forensic Team Final Report Investigation of Failures of Edenville and Sanford Dams	Kinetic Analysis Geometry	Figure F2-12



Total Duration (s)	
7.9	
Total Displacement (ft)	
67.4	
Max Velocity (ft/s)	**(m/s)**
15.80	4.88
Max Acceleration (ft/s^2)	**(m/s^2)**
20.59	6.36

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Kinetic Analysis Results

Figure
F2-13

Appendix G: Forensic Team Analysis - Human Factors

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This appendix provides general background on what the term "human factors" involves, describes the methodology the Independent Forensic Team (IFT) used for investigating human factors aspects of the failure, describes the human factors framework that was applied during the IFT's investigation of the Edenville Dam failure, and provides an analysis of the human factors contributing to the failure, including the influence of "luck" and a "game theory" perspective on the failure. Further details regarding human factors findings, relevant project history, and supporting references for statements made in this appendix are provided in the main report.

G-1 Background On Human Factors

The field of "human factors" considers how and why systems meet or do not meet performance expectations, with an emphasis on understanding and prevention of incidents and failures. The systems considered in human factors work typically include both human and physical aspects and are sometimes referred to as "sociotechnical" systems.

In the case of Edenville Dam, the broader sociotechnical system includes Edenville Dam and its lake and watershed; the other three Boyce Hydro dams (Secord, Smallwood, and Sanford Dams) and lakes on the Tittabawassee River; the other three dams (Beaverton, Chappel, and Lake Lancer Dams) and lakes in the Edenville Dam watershed, which are on tributaries to the Tittabawassee River; the dam designer and construction contractor; all three dam owners (Wolverine, Synex, and Boyce Hydro); Four Lakes Task Force (FLTF) (and its predecessor organizations); the engineering consultants to the dam owners and FLTF; the Michigan Department of Environment, Great Lakes, and Energy (EGLE); the Federal Energy Regulatory Commission (FERC); the lakefront and downstream property owners; Gladwin and Midland Counties; Consumers Energy (Consumers); the dam industry; the power industry in Michigan; and the local and state socioeconomic and political environment in Michigan.

Since the range of human factors that may be considered spans scales of individuals, groups, organizations, industries, and the broader social, economic, and political context, the field of human factors involves the application of social science and philosophy and draws on knowledge from fields such as psychology, social psychology, sociology, cultural anthropology, management, economics, political science, game theory, and history. At the same time, because human factors approaches are often applied to physical systems, such as dams, specialist knowledge of these physical systems is also necessary. As a result, human factors is a highly interdisciplinary field.

The field of human factors has evolved and grown during the past few decades, and a variety of frameworks have been developed. These frameworks generally have overlapping aspects, but with some variety in their assumptions and the aspects they emphasize. Therefore, each framework has particular strengths and limitations. The literature on human factors is very extensive, and the references for *General Human Factors* in this appendix are a selected sample of the literature, which describes many human factors frameworks.

Human factors approaches have been extensively applied in fields such as aviation, nuclear power, chemical processing, and health care. The application of human factors approaches specifically to civil infrastructure, particularly dam safety, is more recent, with most of this work having occurred during the past decade. The references for *Human Factors in Civil and Dam Engineering* in this appendix describe some of this work, with an emphasis on applications to dams.

G-2 Human Factors Investigation Methodology

The scope of human factors that the IFT considered in its investigation included the judgments, decisions, actions, inactions, influencing situational factors, and interactions of the following parties: Wolverine, Synex, Boyce Hydro, FLTF, FERC, EGLE, engineering consultants to the dam owners and FLTF, Consumers, Gladwin and Midland Counties, the lakefront property owners, and the broader dam engineering and safety community and industry in the United States.

In-depth investigation of FERC, EGLE, or their dam safety programs was not considered to be within the scope of the IFT's investigation. However, the role of these regulators and the associated regulatory framework were considered with respect to contributing factors to the Edenville Dam failure.

With regard to these various parties, the IFT applied the human factors framework described in Section G-3 to formulate questions such as those listed below, most of which can be followed with the question "If not, why not?"

- Was the Edenville Dam original design and construction sufficiently conservative, customized to the site, and based on generally accepted best practices of the era for design and construction?
- Did the parties involved in Edenville Dam design, construction, operations, inspection, maintenance, repairs, five-year reviews, and PFMAs have sufficient technical expertise for the tasks they undertook? Did they recognize the limitations of their expertise? Were there any warning signs of the dam failure and, if so, why were they missed?
- How much priority was given, both on paper and in practice, to dam safety versus non-safety goals, such as generating power, controlling costs, meeting schedules, maintaining lake levels, and protecting the environment? Did the dam owners generally have sufficient budgets and staffing to support the goal of dam safety? Did the individuals with key roles related to dam safety have decision-making authority commensurate with their responsibilities?
- Were there any noteworthy cognitive biases, such as anchoring bias and confirmation bias, that contributed to the incident?
- Did system complexity contribute to the dam failure modes not being identified and prevented?
- Was the information management of the dam owners and regulators adequate?
- How well did the dam owners coordinate with their engineering consultants and the regulators?

Based on these types of questions, investigative hypotheses were progressively formulated, tested, and modified through an iterative process. The information that was gathered was generally treated as confidential with respect to sources. This information was gathered through a combination of the following processes:

- **Public requests for information:** Public requests for information, with an independent email box established to contact the IFT, were publicized through the media and the Association of State Dam Safety Officials (ASDSO). Numerous emails were received, and several individuals who contacted the IFT were interviewed.
- **Document review:** Boyce Hydro, FLTF, FERC, and EGLE provided the IFT with access to a large number of documents, totaling many thousands of pages. Documents considered relevant to the Edenville Dam and Sanford Dam failures were reviewed by the IFT.

- **Interviews:** Interviews were conducted with individuals involved in various aspects of the dam and the emergency response, or who were otherwise in a position to provide information relevant to the investigation. Individuals interviewed included current and former Boyce Hydro employees, FLTF members, FERC employees, EGLE employees, several engineering consultants, and private citizens who were eyewitnesses. A few parties, particularly some of the engineering consultants, either declined or did not respond to requests for interviews. In total, more than 25 individuals were interviewed via videoconferencing, a few individuals were interviewed in person, and some individuals were interviewed more than once. Most interviews were in-depth and lasted more than an hour, and in some cases much longer than an hour. The IFT found that interviewees were generally candid and thoughtful in their responses.

In total, the IFT collectively spent thousands of hours on its investigation. A substantial portion of this time was focused on human factors aspects of the investigation, in which all five IFT team members participated. While investigation of human factors necessarily involves a degree of subjectivity associated with the backgrounds and perspectives that investigators bring to bear, the IFT notes that its team members are relatively diverse in these respects.

G-3 Human Factors Analysis

This section describes the human factors framework used for this IFT investigation. The framework, which is a synthesis of various perspectives, was previously developed by one of the IFT team members (Alvi) specifically for dam safety and has been refined during the past decade based on extensive literature review, review of dozens of case studies of past incidents and failures of dams and other systems, and application to several prior dam failure investigations, including investigations of the 2017 spillway incident at Oroville Dam in California and the 2019 failure of Spencer Dam in Nebraska (Alvi 2013a, 2015a, 2015b, 2015c, 2018, 2020; Myers et al. 2015; Alvi et al. 2016; France et al. 2018; Baker et al. 2020).

G-3.1 Key Observations and Assumptions

The human factors framework used for this IFT investigation is based on the following observations regarding past failures of dams and other systems, all of which apply to Edenville Dam:

- Failures are typically preceded by interactions of physical and human factors that begin years or decades prior to the failure (Pidgeon and O'Leary 2000). For Edenville Dam, these interactions began with the original design and construction of the dam in the 1920s.
- The interactions among physical and human factors are often not simple and linear. Instead, they may be complex and involve nonlinear relationships, feedback loops, causes having multiple effects, effects having multiple causes, and a lack of distinct "root causes" or dominant contributing factors (Dorner 1997; Strauch 2002; Dekker 2006, 2011; Qureshi 2008). Edenville Dam fits this pattern and can be considered to be a multi-causal failure, although the stage was set for the failure when the dam was not constructed in accordance with the design plans and construction specifications in the 1920s.
- Interactions among physical and human factors usually generate "warning signs" that are not recognized, or not sufficiently acted upon, prior to failures (Weick and Sutcliffe 2015). The area of Michigan within and near the Edenville watershed had generated warning signs of the potential for unusually high runoff during the cold season (until about the end of May) by causing higher

lake levels and several dam failures in the past, typically during the cold season, as documented in historical lake levels, two watershed studies, and dam failure investigations, but the dots were not connected as needed to recognize these lake levels and dam failures as being warning signs. The low soil blow counts and steep slopes in some locations of the downstream section of Edenville Dam, including the failure section, were also missed as warning signs of potential embankment instability.

- Physical processes deterministically follow physical laws, with no possibility of physical "mistakes." Therefore, failures – in the sense of human intentions not being fulfilled – are fundamentally due to human factors, as a result of human efforts individually and collectively "falling short" in various ways. A story of why a failure happened, therefore, cannot be complete without reference to contributing human factors (Pidgeon and O'Leary 2000). This IFT forensic report aims to tell this story for Edenville Dam.
- A natural tendency is for systems to move towards disorder and failure, similar to the concept of increasing entropy (disorder) in physics. Therefore, systems such as dams are typically not inherently "safe" (Perrow 1999), and continual human effort is needed to maintain order and prevent failure (Reason 1997, 2008; Leveson 2011). This collective human effort evidently fell short for Edenville Dam.
- Systems such as dam systems, including the people involved in designing, building, operating, and managing them, tend to conservatively have numerous "barriers" that must be overcome for failures to occur (Reason 1997; Hollnagel 2004). This generally makes failures unlikely and results in low overall failure rates. However, when dealing with a large collection of systems, such as the approximately 90,000 dams in the United States, it can be expected that "unlikely" failures will sometimes occur due to physical and human factors "lining up" in an adverse way that overcomes all barriers (Reason 1997). In this regard, the failure of Edenville Dam can be considered to have involved some elements of unfortunate ("bad") luck, which are described in Section G-3.7 below.

With these observations in mind, the propensity towards failure can be viewed as being determined by the balance of human factors that contribute to failure ("demand") versus those that contribute to safety ("capacity"). Thus, applying a standard engineering metaphor, failure results when the human factors demand on the system exceeds the human factors capacity, and this was the case for Edenville Dam.

G-3.2 Primary Drivers of Failure

The human factors contributing to safety "demand," and therefore the potential for failure, can generally be placed into three categories of primary drivers of failure, all three of which applied to Edenville Dam:

- **Pressure from non-safety goals** (Dekker 2011) includes pressures to deliver water, generate power and revenue, reduce cost, increase profit, maintain property values, collect property taxes, meet schedules, protect the environment, provide recreational benefits, build and maintain relationships, achieve personal goals, and achieve political goals.

  As described in Section G-3.8 below, for Edenville Dam, there were strong pressures to generate power and revenue, reduce costs and increase profit, maintain property values, collect property taxes, protect the environment, and provide recreational benefits. Some of these diverse goals were in competition with each other. The value of the benefits provided by the lake created by Edenville Dam, as well as the additional property taxes collected by the counties due to the presence of the lake, far exceeded the hydropower revenue generated by the dam and also exceeded the costs of maintaining and upgrading the dam, and therefore the existence of the dam could be economically justified even if it did not generate any power.

However, prior to the formation of the Special Assessment District (SAD) described in Sections 7.1.1 and 7.1.8 of the main report, which required ownership of the dam and lake by the counties rather than a private owner, there was no mechanism, such as a public-private partnership (PPP) (Devernay 2009), that enabled the public and the counties to contribute to paying the costs of the dam while the dam was privately owned. Had such a mechanism existed, there could have been ample funding to upgrade the spillway capacity of the dam, and if that had been done before the May 2020 storm, the rise in the lake on May 18 and 19 could have been kept to a low level, thereby preventing the unforeseen instability failure of the embankment for a storm and runoff of a magnitude similar to the May 2020 storm.

It should also be noted that Edenville Dam was owned and operated as part of a group of four dams; therefore, decisions had to be made regarding how to allocate spending across the four dams from the limited funds derived from selling power from the four dams. This revenue depended on the flow of the Tittabawassee River, which varied from year to year and seasonally within each year, and also depended on the rate the dam owner was paid for power by Consumers, which was below average compared to what Consumers paid to other power producers (see Section 7.1.8). This revenue averaged roughly $2 million per year for all four dams combined when all four dams were generating power. Once the FERC license for Edenville Dam was revoked in 2018, the total revenue was reduced roughly by half. This essentially eliminated funds available for safety improvements such as spillway capacity upgrades and even compromised the ability to pay for routine maintenance and repairs, and therefore Boyce Hydro was effectively forced to sell all four dams and lakes.

- **Human fallibility and limitations** are associated with misperception, faulty memory, ambiguity and vagueness in the use of language, incompleteness of information, lack of knowledge, lack of expertise, unreliability of intuition, inaccuracy of models, cognitive biases operating subconsciously at the individual level and group level, use of heuristic shortcuts, emotions, and fatigue (Plough 1993; Gilovich et al. 2002; Kiser 2010; Kahneman 2011; Alvi 2013b, 2015c, 2020; Sunstein and Hastie 2015).

  In the case of Edenville Dam, while the IFT cannot "read the minds" of the people who were involved in the project over nearly a century of its history, we can confidently conclude that there were examples of misperception (e.g., not perceiving the potential problem with steepness of some embankment slopes), incompleteness of information and lack of knowledge (e.g., regarding the materials used in the embankment and their lack of compaction, and the resulting potential for static liquefaction), unreliability of intuition (e.g., not recognizing the potential for the Edenville watershed to produce unusually high runoff during the cold season), inaccuracy of models (e.g., somewhat unconservative soil parameters used in slope stability analyses and inaccurate gate opening assumptions made in spillway capacity analyses), cognitive biases (e.g., anchoring bias to anchor to soil parameters and factor of safety findings from previous embankment stability analyses), and misuse of heuristic shortcuts (e.g., if embankment seepage indicates potential slope instability, then wrongly inferring that *lack* of embankment seepage indicates adequate embankment stability – fallacy of "denying the antecedent").

- **Complexity** results from multiple system components having interactions that may involve nonlinearities, feedback loops, and network effects. Such interactions can result in large effects from small causes, including "tipping points" when thresholds are reached, and they make complex systems difficult to model, predict, and control (Dorner 1997; Dekker 2011; Mitchell 2011). Complexity generally exacerbates the effects of human fallibility and limitations.

  The broader Edenville Dam "sociotechnical system" was certainly complex and involved a large watershed subject to ground freezing and saturation during the cold season, multiple dams, and many stakeholders. In addition, the dam failure occurred when the lake level reached a threshold (about 5.5 feet above the normal lake level) which had never been approached before, resulting in a sudden embankment instability failure that was unforeseen. In this regard, it is important to note that, if the watershed soils had been somewhat more impervious on May 18 and 19 due to colder preceding temperatures and more extensive frozen ground, the runoff into the lake could have been sufficient to overtop the dam and cause dam failure, even if an embankment instability failure had not occurred. While this "counterfactual" scenario did not occur and would not have occurred, the fact that it nearly occurred – a "near miss" situation – indicates that the dam was susceptible to *both* geotechnical instability and hydrologic overtopping failure modes. This is discussed further in Section G-3.7 below.

G-3.3 Human Error

The primary drivers of failure lead to various types of "human errors," which can include categories such as "slips" (actions committed inadvertently), "lapses" (inadvertent inactions), and "mistakes" (intended actions with unintended outcomes, due to errors in thinking) (Reason 1990; Senders and Moray 1991). In the context of dam safety, mistakes are the most common type of human errors that contribute to failures, and this was the case with Edenville Dam, primarily with regard to engineering evaluations of embankment stability, spillway capacity, and the potential for the Edenville watershed to produce unusually high runoff from a non-extreme rainfall event (the May 2020 storm produced about 100- to 200-year runoff from about a 25- to 50-year rainfall event).

"Violations" are also sometimes classified as a category of human errors, and involve situations in which there is deliberate noncompliance with rules and procedures, usually because the rules or procedures are viewed as unworkable in practice (Reason 1990). While one may wish to claim that all of the owners of Edenville Dam were in violation of FERC regulatory requirements, particularly with regard to spillway capacity, it should be noted that the spillway capacity requirement of EGLE was only about half that of the FERC requirement and *none* of the parties involved with Edenville Dam viewed the Edenville spillway capacity as being deficient to the extent of posing an excessively high risk of overtopping failure. As a result, while FERC did press all three dam owners over a period of three decades to upgrade the spillway capacity in order to meet their probable maximum flood (PMF) regulatory requirement, none of the engineering consultants, FERC, nor any of the dam owners perceived a high level of urgency to complete this spillway capacity upgrade from the standpoint of risk reduction. This was likely influenced by the fact that the spillway capacity was overestimated, the normal lake level allowed about 7 feet of "freeboard" rise in the lake before the dam would overtop, and in the nearly one-century history of the dam, the lake had only risen a maximum of about 2.5 feet above the normal lake level, in 1929. Over the next nine decades, the lake never rose more than 1.5 feet above the normal lake level (it rose only 0.5 foot during the "Great 1986 flood") until it rose about 5.5 feet in May 2020.

In general, with all categories of human errors, judgments regarding what constitutes "error" are usually made in retrospect and are subject to the pitfalls of hindsight bias and fundamental attribution bias

(Dekker 2006). Care must therefore be taken in forensic investigations to avoid readily assigning "blame" (Shaver 1985). Instead, investigators must put themselves in the shoes of the people whose judgments, decisions, and actions are being evaluated, recognizing that they faced pressures from their situational contexts, were inherently fallible and limited, and did not have the benefit of clear foresight when they made their decisions and took their actions (Dekker 2006). It must also be recognized that factors beyond the prediction and control of an individual or group can sometimes result in generally "good" decisions and actions having undesirable outcomes, and generally "bad" decisions and actions sometimes having desirable outcomes; the role of "luck" cannot be entirely eliminated, as discussed further in Section G-3.7 below.

Based on these considerations, identification of "human errors" is not a sufficient end point for a forensic investigation, and assigning blame to individuals is often unreasonable and counterproductive (Dekker 2006). Instead, identified human errors should be treated as prompts to investigate further and delve deeper to understand the situational and contextual factors that contributed to those human errors (Woods and Cook 2002; Dekker 2005; Dekker 2006; Woods et al. 2010). That is the approach the IFT endeavored to take for this investigation.

G-3.4 Risk Management

With the above caveats regarding "human errors" in mind, human errors and the underlying primary drivers of failure noted in Section G-3.2 often lead to inadequate risk management. There are three specific types of inadequacy in risk management due to human errors (Alvi 2015c):

- **Ignorance** involves being insufficiently aware of risks. This may be due to aspects of human fallibility and limitations such as misperception, faulty memory, lack of information, inaccurate information, lack of knowledge and expertise, unreliable intuition, and cognitive biases such as confirmation bias. Complexity can also contribute to ignorance.
- **Complacency** involves being sufficiently aware of risks, but being overly risk tolerant. This may be due to aspects of human fallibility and limitations such as fatigue, emotions, indifference, and optimism bias ("it won't happen to me"). Pressure from non-safety goals can also contribute to complacency.
- **Overconfidence** involves being sufficiently aware of risks, but overestimating ability to deal with them. This may be due to aspects of human fallibility and limitations such as inherent overconfidence bias, which results in overestimating knowledge, capabilities, and performance (Alvi 2020).

The Edenville Dam failure clearly involved ignorance, given that the risks of an unusually high lake level and embankment instability were not recognized despite there being some warning signs, which were missed (see Sections G-3.1 and G-3.6). More broadly, the dam industry as a whole, with rare exceptions, was generally ignorant of the potential for static liquefaction failures to occur in water storage dams.

The IFT believes that there may have been complacency and overconfidence during the construction of dam, as evidenced by the major deviations from the design plans and construction specifications, which resulted in a lack of consistently using relatively impervious materials in the upstream half of the embankment, lack of adequately compacting the embankments, and relatively steep embankment slopes.

The IFT found very limited documentation related to management of the dam after construction in the 1920s until about 1990. From about 1990 onwards, once FERC regulation began, the IFT did not find evidence of substantial complacency or overconfidence. Instead, the IFT believes that the parties generally made reasonable efforts to address whatever significant risks they identified, as evidenced by the spillway modifications at Smallwood Dam and Sanford Dam; the gate hoist system upgrades at Secord, Smallwood, and Sanford Dams; and the placement of overlays in several locations of the downstream slope of Edenville Dam and the other dams. However, it is noteworthy that unconservative slopes and somewhat unconservative soil parameters were assumed in slope stability analyses and inaccurate gate opening assumptions were made in spillway capacity analyses, and all of these assumptions contributed to underestimating safety risks and therefore contributed to ignorance in risk management.

G-3.5 Safety Culture

Counterbalancing the drivers of failure described in Sections G-3.2 through G-3.4, the human factors contributing to system capacity for safety generally emanate from what is referred to as "safety culture" (Pantakar et al. 2001; Alvi 2015c). While this term is sometimes interpreted as applying specifically to worker safety and prevention of injuries on the job, the concept of safety culture is much more general and refers to the safety of any system, including dams.

The general idea of safety culture is that individuals at all levels of an organization place a high value on safety, which leads to a humble and vigilant attitude with respect to preventing failure (Weick and Sutcliffe 2015; Alvi 2015c). For such a safety culture to be developed and maintained in an organization, the senior leadership of the organization must visibly give priority to safety, including allocating the resources and accepting the tradeoffs needed to achieve safety.

While there appears to have been substantially inadequate safety culture during the original design and construction of Edenville Dam, the IFT did not find evidence that there was a major gap in the safety culture of any of the organizations subsequently involved in the dam from about 1990 onwards. However, there were certainly opportunities to improve the safety culture in many of these organizations – as is often the case in the dam industry – and such improvements could have led to actions that would have prevented the dam failure, as described in Section G-3.6 below.

G-3.6 Best Practices

Experience in dam safety shows that strong safety cultures naturally lead to implementation of numerous "best practices" for dam safety risk management, with the understanding that these best practices evolve as the industry learns and improves (Alvi 2015c). As a corollary, dam incidents and failures are typically preceded by long-term cumulative *neglect* of numerous accepted best practices. These best practices can be organized into two categories: general design and construction features of dam projects, and general organizational and professional practices.

Best practices for general design and construction features of dam projects include the following (Alvi 2015c, 2020):

- **Specific design and construction best practices:** Generally accepted best practices for specific aspects of design and construction should be identified and applied. This apparently did not happen during the construction of Edenville Dam.
- **Design conservatism:** Designs should be sufficiently conservative and provide factors of safety commensurate with uncertainties and risks. To the extent possible, designs should also preferably provide physical redundancy, robustness, and resilience, as well as identify failure modes that generate warning signs.

The IFT believes that the Edenville Dam design would have been sufficiently conservative to prevent an embankment stability failure if the dam had been built according to the design plans and construction specifications. However, because there were apparently major deviations from the design plans and construction specifications during construction, the as-built Edenville Dam did not have adequate slope stability safety margins.

- **Design customization:** Designs should be customized to suit features of project sites. This involves "scenario planning" during design to be ready to handle situations that may potentially be encountered during construction, testing during construction to verify that design assumptions and intent are met, and design adaptation during construction to address observed conditions.

  Edenville Dam was located in a relatively wide and flat river valley. The dam had relatively good foundation conditions and therefore did not appear to require any special design adaptations during construction, other than possibly providing a cutoff through the foundation sand layer. However, it appears that construction inspection, testing, and surveying were either not done or not enforced in a manner sufficient to prevent major construction deviations from the design plans and construction specification, and/or there were intentional decisions to deviate from the design plans and construction specifications.

- **Budget and schedule contingencies:** Provisions should be made for accommodating reasonable contingencies when establishing design and construction budgets and schedules. In the case of Edenville Dam, it appears that construction was accelerated and completed about a year ahead of schedule, possibly to decrease construction costs and/or expedite generation of power and revenue. This accelerated schedule may have contributed decisions and actions which resulted in lack of impervious materials in the upstream half of the embankment, poor functioning of some drains, lack of compaction of the embankment, and steeper than specified embankment slopes in some locations.

Best practices for general organizational and professional practices, which encompass all project phases and tasks, include the following (Alvi 2015c, 2020):

- **Resources and resilience:** Sufficient budget and staffing resources should be provided, so that systems and people are not stretched to their limits, thereby increasing error and failure rates (Dekker 2011). The organization should also be resilient, in the sense of having sufficient internal diversity and adaptive capability to provide a broad and flexible repertoire of possible responses to cope with the potential challenges faced by the organization (Hollnagel et al. 2006).

  The IFT believes that budget and staffing resources during the original design and construction of Edenville Dam, including potential lack of qualified and experienced construction workers, may have been inadequate, as evidenced by apparent "cutting corners" during construction. The IFT also believes that FERC may have been able to do more timely and thorough reviews of submissions made by the Edenville Dam owners if FERC had employed more in-house dam engineering staff, especially in the 1993 to 2000 timeframe (see Section 7.1.5).

- **Humility, learning, and expertise:** Individuals and organizations should humbly recognize the limitations of their knowledge and skills, engage in continuing education and training, learn from study of past incidents and failures, and collaboratively draw on expertise wherever it may be found, rather than simply deferring to authority based on position in a hierarchy (Weick and Sutcliffe 2015).

While the potential for greatly increased runoff in cold regions due to frozen and saturated ground during cold seasons was not widely recognized in the dam industry, previous dam failures within and near the Edenville watershed due to frozen and saturated ground could have led to a higher sense of urgency to improve the spillway capacity at Edenville Dam, if those dam failures had been known to the engineers and regulators involved with Edenville Dam.

- **Cognitive diversity:** Teams should have cognitively diverse membership to bring in diversity of perspectives, education, training, experience, information, knowledge, models, skills, problem-solving methods, and heuristics (Gilovich et al. 2002; Page 2008; Alvi 2015c, 2020). With effective team leadership, structure, and group dynamics, cognitively diverse teams can avoid problems such as groupthink, and can outperform more homogeneous teams of the "best" people.

  The engineering consultants involved with Edenville Dam apparently did not collectively have sufficient knowledge and experience with cold regions hydrology to recognize the potential for the Edenville watershed to produce unusually high runoff during the cold season, although one consultant did recognize that wetlands were generally prone to increased runoff if the ground surface became saturated, and EGLE was aware of a dam failure just outside the Edenville Dam watershed which was attributed to high runoff due to frozen ground (see Section 5 and Appendix F1). The engineering consultants also did not recognize that an insufficient extent of soil sampling and testing, somewhat unconservative soil parameters, and an insufficient number of analysis cross-sections were used in the embankment stability analyses.

  In addition, while the engineering consultant who prepared the Consultant Safety Inspection Report (CSIR) for Secord Dam in 2001 recognized the potential for static liquefaction at that dam and recommended remedial action, the engineering consultants for Edenville Dam did not recognize the potential for static liquefaction. This does not necessarily reflect a shortcoming on the part of the engineering consultants involved with Edenville Dam, but rather reflects a general lack of knowledge and attention to static liquefaction in the dam industry. The engineering consultant who recognized the potential for static liquefaction at Secord Dam could be considered to be well "ahead of the curve" and therefore substantially surpassed industry best practices.

- **Decision-making authority:** Decision-making authority should be commensurate with responsibilities and expertise, rather than this authority being contravened by organizational structure (Weick and Sutcliffe 2015). This is particularly the case for safety personnel, who should be selected for their positions based on having relevant experience, vigilance, caution, humility, inquisitiveness, skepticism, discipline, meticulousness, communication ability, and assertiveness.

  The IFT did not generally find that there were problems with decision-making authority in relation to Edenville Dam, except that, on May 18 and 19, 2020, neither Boyce Hydro nor FLTF had an engineer on site, acting a dam owner's representative, who was willing and able to provide recommendations related to dam operations, monitoring of the dam, measures for prevention of dam failure, and the need for emergency action plan activation. Engineers from EGLE tried to fill this gap somewhat on May 19, but they did not have the authority to act on behalf of the dam owner. Had such an engineer been on site, different decisions that would have prevented the failure would not necessarily have been made, but if the circumstances on May 18 and 19 had been marginally different with respect to what was physically happening with the runoff into the lake, the lake levels, and gate operations, it is conceivable that decisions could have been made in some of these "counterfactual" scenarios that would have made the difference between the dam failing versus not failing (see Section 6 and Section G-3.7).

- **System modeling:** Appropriate system models should be developed, with a full range of potential failure modes identified and emergency action plans developed accordingly. For actively operated systems such as hydropower dams, these failure modes should include operational failure modes, and it may be appropriate to explicitly account for interactions of physical and human factors in the system models. Where models are implemented through software, the software should be carefully developed, validated, and used (Woods et al. 2010).

  The IFT found that, with four dams in series on the Tittabawassee River, and no FERC or EGLE requirement that dams in series be modeled as hydraulically interacting components of a single system, there was no formal analytical approach to managing the four dams generally as a system with respect to dam operations for safely passing floods, prioritizing upgrades to the dams, and recognizing how the interactions of the dams affected risks. Instead, these system-level considerations were accounted for on an informal basis (see Section 6).

- **Checklists:** Checklists should be used to reduce the incidence of human errors, especially for tasks which are relatively recurrent, such as inspections (Gawande 2010). Checklists should be customized for each situation, clear and unambiguous, focused on items that are important but prone to being missed, prepared at a level of detail appropriate for the time available for using the checklist, and regularly updated based on experience. Recognizing that checklists are most effective for prevention of slips, lapses, and violations but somewhat less effective for prevention of mistakes (see Section G-3.3), checklists should be used to supplement, not replace, situation-specific attentive observation and critical thinking.

  The IFT did not find evidence that checklists were generally used for Edenville Dam. While it is not dam industry practice to extensively use checklists for analyses, it is possible that use of appropriate checklists for tasks such as hydrologic analysis and embankment stability analysis, in addition to inspections, could have led to recognition of warning signs of the potential for embankment instability and high lake levels, and therefore could have led to actions that would have prevented the dam failure.

- **Information management:** Information management should involve thorough, well-organized, and readily accessed documentation; open and collaborative information sharing within and across organizations; and discussion/analysis that is not dismissive of dissenting voices. This will enable surfacing and synthesis of fragmentary information to help "connect the dots" and better understand system behavior (Catino 2013; Weick and Sutcliffe 2015).

  Once the Edenville Dam FERC license was revoked and EGLE became the regulator, FERC Critical Energy/Electric Infrastructure Information (CEII) restrictions made it difficult for EGLE to obtain information about the dam from FERC. The IFT did not reach a conclusion regarding the adequacy of the information management systems of Wolverine, Synex, FERC, or EGLE; however, the IFT did find that the information related to Edenville Dam was relatively well organized in the Boyce Hydro filing system, and Boyce Hydro was able to promptly provide information requested by the IFT.

- **Warning signs:** There should be vigilant monitoring to detect "warning signs" that a system is headed toward failure, while there is still a "window of recovery" available (Weick and Sutcliffe, 2015). This monitoring should be conducted at regular intervals, after unusual events, and also during apparent "quiet periods." Once potential warning signs are detected, there should be prompt and appropriate investigative follow-up, verification of that follow-up, thorough documentation of observations and findings so that emerging patterns can be discerned and evaluated, and prompt implementation of any needed remedial actions. As a heuristic to help judge whether a potential warning sign warrants action, "simulated hindsight" can be used: fast-forward into the future, imagine that failure has occurred, and ask whether ignoring the potential warning was justifiable; if not, take the potential warning sign seriously.

As noted above, for Edenville Dam, the area of Michigan within and near the Edenville watershed had generated warning signs of the potential for unusually high runoff during the cold season. These warning signs included higher lake levels typically occurring during the cold season, and several dam failures due to high runoff, as documented in historical lake levels, two watershed studies, and at least one dam failure investigation. However, these lake levels and dam failures were not recognized as warning signs.

The steep slopes in some locations of the downstream slope of Edenville Dam, including the failure section, were also a missed warning sign of potential embankment instability.

From a positive standpoint, it should be noted that special inspections of the dam *were* often performed after significant storm and runoff events.

- **Standards:** High professional, ethical, legal, and regulatory standards should be maintained – especially when lives are at stake. While the IFT did not identify any obvious lapses in existing standards being met for Edenville Dam, the IFT believes that the industry standards were themselves inadequate and need to be expanded to better address cold regions hydrology, comprehensiveness of geotechnical evaluations including embankment stability evaluation, and consideration of the potential for static liquefaction.

In summary, organizations that have the capacity to handle demands on safety from various drivers of failure have a strong safety culture and diligently implement numerous best practices. Such organizations are mindful, cautious, humble, oriented towards learning and improving, resiliently adaptive, and they maintain high professional and ethical standards. They vigilantly search for and promptly address warning signs before problems grow too large, and they make effective use of available information, expertise, resources, and management tools to properly balance safety against other organizational goals.

The IFT believes that the parties involved in the original design and construction of Edenville Dam collectively fell short of meeting these ideals. While the parties subsequently involved with the dam *after* construction, at least from about 1990 onwards, may not have fallen far short of these ideals, the failure may have been prevented if some of them had come closer to meeting these ideals.

G-3.7 "Luck"

If we can fully predict and control the behavior of a system, such as a dam – in other words, there is no uncertainty about the system – we will always get the outcomes we want. In such an ideal case, there is no "risk" or "luck" associated with the system. However, in the real world, and certainly in the design, construction, operation, and management of dams, there is *always* some uncertainty about the physical characteristics of dams, the loads they may be subjected to, and their behavior in response to loads.

To deal with this uncertainty and produce an acceptably low failure rate for dams, various heuristics are used in engineering practice, such as standards and best practices for design and construction features of dams, general conservatism in design (including redundancy, robustness, and resilience), factors of safety to quantify safety margins, and, more recently, risk analysis using qualitative, semi-quantitative, and quantitative methods. Because they deal with human uncertainty, which is inherently subjective, all of these heuristics involve elements of subjective human judgment, regardless of the degree of quantification involved.

A corollary of uncertainty is *risk* – the prospect, but not certainty, of undesirable outcomes – and risk can never be reduced to zero if there is any degree of uncertainty. This is evident when even the most rigorous quantitative risk analysis is done for dams, where the probabilities of various possible events and net probabilities of failure are generally always assumed to be greater than zero. It follows that, even when risk is judged to be relatively low, there is still the *possibility* of failure, and if failure actually occurs in a case where the risk was judged to be relatively low, there is an element of surprise and the failure could potentially be considered to have involved a degree of unfavorable ("bad") luck. Just as uncertainty and risk can be reduced, but not eliminated, the prospect of "bad luck" cannot be eliminated either.

An important distinction to be made is that risk is generally attributed prospectively to an uncertain future, whereas luck is generally attributed in hindsight to past events, and therefore risk analysis can be applied in design, construction, operation, and management of a system, whereas "luck analysis" can be applied in forensic investigation. The two types of analysis are fundamentally very similar, because they both imagine a set of possible histories of a system and make judgments about the likelihood of each history. Luck analysis is somewhat more straightforward than risk analysis because, in luck analysis, a particular history has already been realized and the analysis considers possible deviations from that particular history. By contrast, in risk analysis, the possible histories are all "open" and there is always the question of whether the set of possible histories being considered is sufficiently complete.

Bad luck is often judged to be a factor in a failure when it is found, again in hindsight, that the physical circumstances of the situation "lined up" in an unusual and seemingly unlikely way, analogous to a "perfect storm," and the failure is judged to have not been reasonably preventable. Such an alignment of circumstances would generally be unforeseen, and possibly unforeseeable, due to limitations of human knowledge and expertise. However, when dealing with a large class of systems, such as the approximately 90,000 dams in the United States, it can be expected that failures will sometimes occur, just as about 35,000 auto accident fatalities collectively occur in the United States each year, despite the fatality rate being only about 1 fatality per 100 million miles driven.

Bad luck will often be judged to be a factor particularly in dam failures because, for dams which are judged to have relatively high risk, remedial action is typically taken to bring the perceived risk down to what is judged to be a low risk, and study of the dams that have actually failed historically shows that they were typically judged to be low risk (probability of failure much closer to 0 than 1) prior to the failures (as was the case with Edenville Dam).

While the terms "luck," "bad luck," and "good luck" are in common usage in the English language, and therefore everyone has intuitive understandings of the meanings of these terms, careful analysis of the meanings of these terms is helpful in order to better understand how they might be applied in the context of dam safety. During the past few decades, analytic philosophers, and sometimes psychologists, have undertaken just such an analysis of the concept of luck, and have thereby generated a substantial literature on this topic (e.g., see references in this appendix for *Luck*). In this appendix, we apply the main ideas in this literature to the Edenville Dam failure.

It has been argued that luck is not *objectively* real, but rather, like probability and risk, the attribution of luck is fundamentally *subjective,* relative to human knowledge, capabilities, and biases (Hales 2016, 2020). From this standpoint, the concept of luck is not really one unified concept, and instead it can be subdivided into three relatively distinct aspects that are not necessarily mutually exclusive. When there has been a bad outcome, such as a dam failure, a judgment of whether there has been bad luck can be made relative to each of these three aspects:

- *Likelihood and Predictability:* Can the event be viewed as having been highly unlikely relative to normal human experience and expectations, and therefore not likely to have been predicted?
- *Control:* Could the event have not been readily prevented? In other words, did people lack control of the outcome?
- *Sensitivity to "What If" Counterfactuals:* Would the event not have occurred, or would the consequences have been less, if the circumstances had been slightly different? In other words, would the bad outcome have been avoided if the world had been a "counterfactual" world, which was not quite the same as the actual world? Are there scenarios for what "almost happened" that would have led to a "near miss" rather than a bad outcome?

These three sets of questions are also often asked prospectively when doing risk analysis, and so, as described above, luck and risk can be viewed as being past versus future "mirrors" of each other. With respect to luck, an affirmative answer to one or more of those questions is usually viewed as justifying an attribution of bad luck to the event. From this standpoint, while the Edenville Dam failure *cannot and should not be attributed solely or primarily to bad luck*, it can be judged to have involved several elements of bad (and good) luck as contributing factors, as described below.

Unusually Unstable Embankment Failure Section

As described in this report, the construction of the Edenville Dam embankments deviated substantially from the design plans and construction specifications, and these major deviations set the stage for the embankment instability failure in May 2020. These deviations could have been prevented by making different choices during construction, and it could have been foreseen that these deviations would increase the potential for embankment instability. In other words, counterfactually, if the embankments had been constructed in accordance with the design plans and construction specifications, it can be expected that instability failure would not have occurred in May 2020. Therefore, the instability of the embankments cannot reasonably be attributed simply to bad luck – the instability was both predictable and controllable.

However, it is noteworthy that the Edenville Dam embankments totaled about 6,000 feet in length, and it was only a failure section about 40 to 80 feet long that failed in May 2020 when the lake rose to a record high level. This indicates that, despite the dam generally not being constructed in accordance with the design plans and construction specifications, it was a section of the embankment which represented only about 1 percent of the length of the dam that had a particular and relatively unlikely combination of physical characteristics sufficient to result in failure of that section when the lake rose. The IFT was not able to conclusively determine what that particular combination of characteristics was, but it would be expected that if, counterfactually, the failure section had characteristics similar to the rest of the dam, it may not have failed in May 2020 and the dam may not have breached. Therefore, it could be argued that there was an element of bad luck in the failure section having the particular characteristics that it had.

However, again counterfactually, if the failure section had not been susceptible to failure and had not failed, the lake would have continued to rise and would have stayed at a relatively high level for many hours before gradually dropping; thus it is possible that a different section of the embankment would have eventually had an instability failure. Therefore, if the failure section did not have the particular characteristics that it did, it is still possible that the dam would have eventually breached due to instability failure at a different location. This reinforces the point that the fundamental problem was with how the dam was constructed in general.

A related consideration is that, unlike many other locations of the downstream slope of the dam, the failure section did *not* exhibit significant seepage emerging at the downstream slope. Lack of such seepage is generally considered to be a favorable performance indicator for an embankment dam. However, in the particular case of Edenville Dam, in locations where significant seepage *had* been observed at the downstream slope, embankment overlays had been constructed, and it could be expected that an overlay would have been placed at the failure section if, counterfactually, seepage had been observed there. If such an overlay had been placed at the failure section, the static liquefaction failure may have been prevented if the overlay provided enough stabilizing weight at the toe, since there is a physical relationship between shear stress ratios and static liquefaction failure modes. Therefore, it could be considered bad luck that significant seepage did not occur, or was not observed, at the downstream slope of the failure section.

Looking further at human factors, the IFT found that there was sufficient information available to the engineers involved with Edenville Dam to identify the failure section as having inadequate factors of safety for a conventional instability failure mode. If these deficient factors of safety had been remedied by placing a downstream slope overlay, as was done at other locations of the dam, again, the static liquefaction failure would very likely have been prevented, because the flatter slopes would have reduced the shear stress ratios. Therefore, it was not simply a matter of bad luck that the embankment instability in the failure section was not identified – the relatively low margins of safety for instability could have been predicted and action could have been taken to increase the margin of safety, which could have prevented the static liquefaction failure.

A consultant who was involved in embankment stability analysis for Secord Dam in 2001, Barr Engineering, *did* recognize the potential of that dam to have a static liquefaction failure and recommended that remedial action be taken (see Section 7.1.5). It is highly unusual, and therefore could be considered good luck, that this failure mode was evaluated, since engineers in the dam industry were not generally considering the potential for static liquefaction in 2001, nor even in 2020. The particular engineer at Barr who identified the static liquefaction failure mode happened to be a recent PhD graduate of the University of Illinois Urbana-Champaign, and he was aware of this failure mode because research on this topic had been recently done at that university (his PhD thesis advisor was an expert on this topic). Counterfactually, if that same consultant team had performed an embankment stability analysis for Edenville Dam, it appears likely that the potential for static liquefaction at Edenville Dam would have been identified, remedial action may have been taken, and the failure may have been prevented. This sequence of events would have been good luck, and therefore it could be considered bad luck that the Secord Dam consultant team did not happen to be retained to perform an embankment stability analysis for Edenville Dam.

However, to extend this counterfactual further, in 2005, A. Rieli and Associates evaluated Barr's concern regarding the potential for static liquefaction at Secord Dam, and argued that Barr's analysis was inaccurate and that the soils at Secord Dam were not susceptible to static liquefaction, and that therefore no remedial action was necessary to prevent a static liquefaction failure at that dam. As a result, it appears that no remedial action was taken at Secord Dam. Had a similar scenario played out for Edenville Dam, it is possible that the static liquefaction concern could have been identified (good luck), yet not remedied because of conflicting engineering opinions (bad luck).

Unusually High Runoff into the Lake

The particular combination of physical circumstances that resulted in high runoff into the lake from the watershed in May 2020 was highly unusual. The Edenville watershed contained large areas of forested swamps, and these types of land areas are more prone to ground freezing, and staying frozen, than other types of land areas, partly because of shading of the ground surface. The air temperatures in the watershed in May typically average in the 50s °F, but in 2020, in the 2 weeks prior to the failure (from May 5 to 14), there were 10 days of unusually cold overnight temperatures, generally below freezing and at or near record lows. These conditions were likely sufficient to produce some ground freezing in the watershed, especially in the forested swamps. This cold period was followed by warming above freezing and then moderate rain on May 15, which likely saturated the ground above underlying frozen ground beneath the surface and in wetlands. This ground saturation over a large area in the watershed set the stage for a subsequent large rainfall event to produce unusually high runoff, and indeed, just 3 days later on May 18, an average of almost 4 inches of rain fell in the watershed in just 1 day, with an even higher rainfall in the eastern and northeastern parts of the watershed where the forested swamps happened to be more prevalent.

The unusually high runoff caused the lake to rise by about 5.5 feet, which is about 3 feet higher than it had ever risen before during nearly a century of operation of the dam. The highest known lake level before May 2020 was about 2.5 feet above the normal level about 90 years earlier, in 1929, and since then the lake had never risen more than 1.5 feet above the normal level until May 2020. Even during the "great flood" of 1986, the lake rose only about 0.5 foot. From this history of lake levels, it is evident that the combination of physical circumstances in May 2020 was highly unusual and highly unlikely, and therefore there was an element of bad luck that such a high runoff occurred.

However, as described in Sections G-3.1 and G-3.6 of this appendix, there *were* warning signs, which, if they had been recognized as warning signs, could have led to identification of the potential for high runoff into the lake during the cold season. If this potential for high runoff had been recognized, nothing could have been done to prevent the high runoff – there was a lack of control – and therefore the high runoff itself could be considered to involve bad luck. However, the rise in the *lake level* could have been less if the spillway capacity had been substantially greater, and that might have prevented the failure. However, in this regard, it should be noted that a primary purpose of the spillways is to prevent overtopping of the dam, and the available spillway capacity in May 2020 was sufficient that the dam did not overtop, and would not have overtopped if the embankment stability failure had not occurred. Therefore, lack of spillway capacity, by itself, cannot be considered to be a "root cause" nor "sufficient cause" of the failure.

In considering the counterfactuals of what might have happened if the physical circumstances in the watershed had been slightly different in May 2020, it is likely that the runoff into the lake, and therefore the rise in the lake, would have been much less if there had been less forested swamp and wetlands in the watershed, if the rain had been less in the forested swamp areas of the watershed, if the temperatures during the cold period had been a few degrees warmer, if the moderate rain on May 15 had not occurred or been less, if the rain on May 18 had been less, and/or if the rain had been less in the forest swamp areas. It could be viewed as bad luck that none of these conditions that would have reduced the runoff occurred, and that instead they all "lined up" to produce the unusually high runoff.

On the other hand, if one or more of these conditions had been worse with respect to *increasing* the runoff, it is entirely plausible that the runoff into the lake would have been sufficient to overtop the dam, which would have resulted in dam failure even if the embankment stability failure had not occurred. Moreover, the consequences of an overtopping failure could have been worse than those of the actual failure which occurred in May 2020, because an overtopping failure might have resulted in simultaneously breaching the dam in more than one location along the approximately 6,000-foot length of the embankments, resulting in an increase in breach discharge and downstream flooding. Therefore, there was an element of *good* luck that the failure consequences were not more severe than they actually were.

The primary remedy to ensure that that dam did not overtop due to a large runoff event was to greatly increase the spillway capacity to meet the FERC probable maximum flood (PMF) requirement, or at least the EGLE requirement (which was about 40 percent of the FERC requirement), but the level of need and urgency for this spillway capacity increase was not recognized by the engineers involved in the Edenville project because the spillway capacity was overestimated, the potential for a non-extreme storm to produce an unusually high runoff during the cold season was not recognized, and it was generally believed by all parties that the chance of overtopping of the dam was very low.

Instead of, or in addition to, increasing the spillway capacity, an alternative option to reduce the chance of overtopping was to construct an operable low-level outlet and fully lower the lake (the existing low-level sluiceway was inoperable). This lowering could have been combined with excavating into the embankment to create a temporary auxiliary spillway at a selected location, such as towards the west end, where an intentional breach was being contemplated by the parties who were at the site during the afternoon of May 19, hours before the failure. However, again, these options were not under serious consideration before May 2020 because, again, the potential for a non-extreme storm to produce an unusually high runoff during the cold season was not recognized, and for several other reasons (see Section 7.2.2). Therefore, it was not simply bad luck that these options were not pursued.

Prior to May 2020, the idea of operating the dam with the gates kept open and the lake lowered about 6 to 8 feet to "run of river" level had been considered. After the FERC license was revoked and water could no longer be released through the powerhouse, keeping the gates open would have avoided the need to continually de-ice the gates in the winter in order to operate them, and one might guess that lowering the lake by about 6 to 8 feet could have also provided some benefit with respect to keeping the lake level lower and reducing the chance of overtopping. However, the IFT evaluated this counterfactual scenario and found that, for the May 2020 event, pre-lowering the lake by this amount before the storm, by itself, would have made a small difference in the peak lake level (because the storage volume of the lake between the normal level and the spillway crests was a small portion of the storm runoff volume) (see Section 5.2.5). Therefore, with respect to the dam failure, it is unlikely that there was any significant bad or good luck associated with the decision to bring the lake back up to the normal level a few weeks before the dam failure, after it had been lowered during the winter of 2019-2020.

The IFT estimated that, counterfactually, if it had been possible to release water through the powerhouse during the May 2020 event, the peak lake level on May 19 would have been up to about 0.8 foot lower, relative to the level of the lake at the time of failure, and this amount of lowering of the lake may or may not have prevented the embankment instability failure (see Section 5.2.5). The fact that water was not released through the powerhouse is not itself a matter of luck, because it was a conscious decision to revoke the license and the effect of releasing water through the powerhouse was predictable. Therefore, *if* lowering the lake by 0.8 feet *would* have prevented the failure, the fact that the lake was not lowered by releasing water through the powerhouse does not reflect bad luck.

Timing of the Storm in May 2020

Secord, Smallwood, Edenville, and Sanford Dams were located in series along the Tittabawassee River and were originally built for the purpose of generating power and revenue. The four dams were always owned by a single owner at any given time, and therefore the finances of the dams were tied together in the sense that the collective revenue from all four dams could be spent on operations, maintenance, upgrades, and other expenses at each of the four dams at the owner's discretion.

The original owner, Wolverine, owned the dams for almost eight decades until Synex became the owner in 2003, possibly because the Wolverine's revenue generated by the dams was not sufficient to cover the costs of meeting the FERC regulatory requirements, which had come into effect in recent years, and so Wolverine was unable to repay a loan from Synex. After only three years of ownership, Synex sold the dams to Boyce Hydro in 2006, possibly also because of financial difficulties. Boyce Hydro also faced financial difficulties, in part because the energy rate paid to Boyce Hydro by Consumers was below average compared to other hydropower producers (see Section 7.1.8), and therefore Boyce Hydro began discussions to sell the dams to Gladwin and Midland Counties or others in 2012. Eventually, an agreement to sell the dams and related properties to these counties, via their delegated authority (eventually to become known as the Four Lakes Task Force, FLTF) was reached and various documents related to the sale of the dams were signed in 2018 through 2020 (see Section 7.1.1).

Based on results of gate testing at Edenville Dam in 2019, FLTF had decided to upgrade the gate hoist systems at Edenville Dam, with the work to be completed in late 2020. This upgrade, which was to be similar to the upgrades already made at the other three dams, would have enabled the gates to be lifted at least 10 feet, instead of the approximately 7 feet they were lifted during the May 2020 event, and higher lifting of the gates would have lowered the peak lake level by an estimated 1.1 to 1.8 foot relative to the lake level at the time of failure (depending on whether the full gate opening was combined with other actions), and this amount of lowering of the lake may or may not have prevented the failure (see Section 5.2.5). Counterfactually, *if* lowering the lake by 1.1 to 1.8 feet *would* have prevented the failure, and either (a) the Edenville gate hoist system had been upgraded before May 2020, just months before that upgrade was planned to be done, or (b) the storm had occurred a year later, then this different timing of events would have prevented the embankment instability failure. Therefore, given that the May 2020 storm was somewhat of an outlier in producing a runoff that was much higher than any other storm in the watershed since the mid-1920s, the timing of that storm less than a year before the gate hoist system was expected to be upgraded can be viewed as potentially reflecting bad luck.

FLTF had also planned to significantly increase the spillway capacity to at least be sufficient to meet EGLE's "half PMF" requirement, but possibly not the FERC full PMF requirement. Counterfactually, if this spillway capacity increase had been completed before the May 2020 storm or if that storm had occurred a few years in the future after the spillway capacity had been upgraded, the lake level could likely have been kept several feet lower, and therefore the embankment instability failure almost certainly would have been prevented in May 2020. Since the spillway capacity increase was planned by FLTF and likely would have been completed within a few years after 2020, the timing of the storm in May 2020, rather than a few years later, after the spillway capacity had been increased, can be viewed as reflecting bad luck. However, it should be noted that the embankment would have remained vulnerable to instability failure during future extreme floods if that vulnerability had not been identified and remediated.

Summary

The Edenville Dam failure can be judged to have had several elements of bad luck that either contributed to the failure or at least resulted in a missed opportunity to prevent the failure. Based on a three-part analysis of "luck," these elements of bad luck are related to unlikely and unexpected events occurring, an inability to fully control the outcomes of events, and the fact that failure might not have occurred if various circumstances had counterfactually been somewhat different. These elements of bad luck are related to the variability of the dam along its length, the variability in seepage behavior of the dam, the embankment stability analyses that were and were not performed, the hydrologic characteristics of the May 2020 storm event, and the timing of that storm event relative to planned upgrades to the Edenville gate hoist systems and spillways. Analysis of the role of luck in this failure is useful with respect to understanding why the failure occurred, what could have been done to prevent it, and the lessons to be learned to help prevent similar failures in the future.

However, it must be emphasized that the Edenville Dam failure cannot simply be attributed to bad luck alone. There were numerous human judgments, decisions, actions, and inactions related to the dam that shaped the physical history of the dam in a way that ultimately set the stage for the failure of the dam in May 2020. If different judgments and decisions had been made or different actions taken, the failure could undoubtedly have been prevented in May 2020.

Broadly, the Edenville Dam failure must be understood in terms of the complex one-century history of the overall system for financing, designing, constructing, operating, evaluating, and upgrading the four dams, which involved many parties, as described in Section 7.1 of the main report. This history resulted in inadequate physical safety margins for the dam, which then enabled “bad luck” to be among the contributors to the failure of the dam. If the physical safety margins had been adequate, the failure would not have occurred and “luck” would not have been relevant to the dam because the margins of safety would have allowed for some degree of “bad luck,” and therefore the failure would not have occurred.

G-3.8 “Game Theory” Perspective

“Game theory” is an analytical framework, first developed in the 1940s, for modeling situations where multiple parties are interacting with each other and each pursuing their own goals, but the decisions of each party are influenced by what they think other parties will do and what the other parties actually do (Dixit and Nalebuff 1991; Binmore 2007; Pastine et al. 2017; Ross 2021; Wikipedia 2022). A set of circumstances where parties have this kind of relationship is called a “game,” since games such as chess and football have this kind of structure. However, this kind of structure is also found throughout all domains of human interactions, such as interpersonal relationships, business, economics, law, politics, foreign policy, and military strategy. Game theory has also been applied in areas such as biology and ecology (McNamara and Leimar 2020) as well as specifically in water resources engineering (Dinar et al. 1992; Zara et al. 2006; Madani 2010; Dinar and Hogarth 2015; Hui et al. 2015, Madani et al. 2015; Liu et al. 2022).

If a situation is structured such that the parties are competing with each other to achieve their goals (e.g., one party has to lose in order for another party to win, as in a tennis match), that situation is described as a “non-cooperative game” or “competitive game.” If the situation is instead structured such that all of the parties must cooperate with each other in order to achieve their goals (e.g., a group of people who need to pool their money to buy something that none of them can individually afford to buy), that situation is described as a “cooperative game.” Actual games in the real world can have a mix of cooperative and non-cooperative relationships, which can change over time.

What each party would “rationally” do in order to pursue its own goals is determined by the particular situation and structure of a game with respect to who the parties are, their relationships with each other, what each party is pressured or obligated to do, what each party is able to do, the information available to each party and related uncertainties and risks for each party, and the particular goals of each party.

For the relatively complex situation of Edenville Dam and the associated Secord, Smallwood, and Sanford Dams, there were numerous parties interacting with each other; the parties had diverse and conflicting goals; the relationships among the parties were a mix of cooperative and non-cooperative relationships (largely because of the conflicting goals); the parties had diverse obligations and abilities; the information available to the parties varied widely (there was no structure that enabled pooling of information, and there were restrictions in sharing information for security reasons); and the parties were exposed to different risk profiles. This situation and the inferred "rational" preferences and courses of action for each party are summarized in Table G-1 below.

Table G-1: Project Parties and Circumstances

Party	Benefits from Project	Risks from Project	Inferred "Rational" Preferences and Courses of Action
Dam owners	Relatively small profit margin	Potential for annual financial losses if project costs increase or revenues decrease; large financial loss and liability, and likely bankruptcy if dam failure occurs	• Make minimum financial investments needed to make dam failure very unlikely, maintain FERC licenses and ability to continue generating power, and meet EGLE environmental regulatory requirements as needed to avoid penalties • Solicit funding from other project beneficiaries to help pay for project costs, including upgrading the spillway capacity • Negotiate a higher rate from Consumers to increase project revenue and profits • Sell the dams for a reasonable profit, or at least not at a significant loss, if a suitable buyer can be found
Dam owner's engineering consultants	Engineering fees	Potential liability and reputational loss if dam failure or environmental damage occurs	• Make recommendations for managing project risks without imposing excessive costs on the dam owner, which could result in the dam owner discontinuing use of the consultant's services • Limit engineering scope and fees to what the dam owner will accept • Perform engineering services efficiently, which may include relying on previous work or analyses performed by other consultants when applicable and where risk of doing so is deemed reasonable
FERC	None, beyond a general mission to support production of power in the United States	Potential liability and reputational loss if dam failure occurs	• Enforce FERC regulations and avoid the scenario of a dam failure while the dam is under FERC regulation: first give the dam owners a reasonable amount of time to meet FERC regulatory requirements; then exert increasing pressure to meet the requirements with threats of various orders; finally, carry out cease generation orders, financial penalties and/or ordering lake level restriction, and, as a last resort, license revocation
EGLE dam safety division	None	Potential liability and reputational loss if dam failure occurs	• Enforce EGLE dam safety regulations and avoid the scenario of a dam failure while the dam is under EGLE dam safety regulation: first give the dam owners a reasonable amount of time to meet EGLE regulatory requirements; then exert increasing pressure to meet the requirements; finally, as a last resort, have the dam breached or removed by the State of Michigan and attempt to recover the cost of doing so from the dam owner through legal action

Party	Benefits from Project	Risks from Project	Inferred “Rational” Preferences and Courses of Action
EGLE environmental division	Preservation of environmental benefits associated with the presence of the lakes	Potential liability and reputational loss if environmental damage occurs due to dam failure or lowering of the lakes	• Enforce EGLE environmental regulations
FLTF	Preservation of the lakes and the associated recreational, aesthetic, and property value benefits to lakefront property owners and lake users	Liability, and property value and reputational loss if dam failure occurs	• Purchase the dams at the lowest possible cost • Make improvements to the dams at the minimum cost needed to meet regulatory requirements and reasonably address dam safety concerns so that the annual costs passed on to property owners are minimized • Ensure that lake levels are maintained so that property owners derive the benefits of the lakes
FLTF’s engineering consultants	Engineering fees	Potential liability and reputational loss if dam failure or environmental damage occurs	• Make recommendations for managing project risks without imposing excessive costs on FLTF, which could result in FLTF discontinuing use of the consultant’s services • Limit engineering scope and fees to what FLTF will accept • Perform engineering services efficiently, which may include relying on previous work or analyses performed by other consultants when applicable and where risk of doing so is deemed reasonable
Gladwin and Midland Counties	Increased property taxes collected due to the presence of the lakes	Reduction in property taxes collected if lakes are lost due to dam failure or draining the lakes	• Continue to collect increased property taxes made possible by the presence of the lakes, without helping to pay for the costs associated with the dams
Consumers Energy (Consumers)	Profits from sale of energy purchased from the dam owners	Loss of potential profit if the dams do not generate power due to discontinuing power generation or dam failure	• Negotiate the lowest possible energy rates to be paid to the dam owners to maximize the profit on sale of energy generated by the dam
Lakefront property owners	Increased property values, recreational value of the lakes, and aesthetic value of the lakes	Loss of lake benefits if the lakes are lost due to dam failure or draining the lakes	• Continue to derive the substantial benefits provided by the lakes, without helping to pay for the costs associated with the dams • Report unusual activity or concerns related to the dams to authorities
Lake users who do not own lakefront properties	Recreational value of the lakes	Loss of recreational value if the lakes are lost	• Continue to derive the recreational benefits provided by the lakes without helping to pay for the costs associated with the dams
Property owners downstream of the lakes in the breach inundation zone	Access to recreation on the lakes and economic benefits from the presence of the lakes	Likely property damage or destruction and potential loss of life if dam failure occurs	• Rely upon the dam owners and regulators to take actions that make the likelihood of dam failure “as low as reasonably practicable”

EGLE = Michigan Department of Environment, Great Lakes, and Energy
FERC = Federal Energy Regulatory Commission
FLTF = Four Lakes Task Force

Comparing the inferred "rational" preferences and courses of action for each party with what each party actually did during the history of the project (see Section 7.1), it is not surprising that each party did generally behave "rationally" from a game theory perspective – in other words, all of the parties behaved pretty much as they should have been expected to behave from the perspective of their own circumstances and goals, and what they expected the other parties to do.

Weighing the overall aggregate benefits (nonspecific to any of the parties) from the project (e.g., recreational value, aesthetics, increased property values, increased property taxes collected, power generation, revenue) with the project costs (e.g., annual operating and maintenance costs, costs for upgrades such as a spillway capacity increase), the project benefits far outweighed the costs, and therefore the existence of the project was justified and removal of the dams would not seem reasonable.

In this regard, it is noteworthy that, on behalf of the counties and the lakefront property owners, FLTF was willing to invest more than $200 million to restore the lakes after the dams failed, even if there was no revenue from power generation. This indicates the true value of the dams to the counties and the lakefront property owners due to recreational, aesthetic, property value, and property tax benefits. By comparison, the cost to operate, maintain, and upgrade the dams before the dam failures was roughly $2 million in annual operating and maintenance costs for all four dams, and the estimated cost to upgrade the Edenville Dam spillway to pass the PMF was about $5 million to $10 million. These are relatively low costs compared to the true benefits provided by the dams.

Moreover, *before* the dam failures, the FLTF's agreed price to purchase the dams and related properties from Boyce Hydro was about $16 million, and FLTF's total budget to buy the properties and bring the dams fully into compliance with regulatory requirements was about $40 million. This is only a small fraction of the more than $200 million FLTF was willing to spend *after* the failures to achieve the same benefits as were expected before the failures. FLTF had the financial capacity to make such a large investment because, in addition to obtaining grant funds, it could obtain funds from the lakefront property owners via annual fees through the Special Assessment District (SAD). In contrast, the financial capacity of Boyce Hydro and the prior dam owners was limited to the revenue they could obtain by selling power to Consumers at the negotiated rates, and this revenue was not sufficient to fund major safety investments, such as upgrading the spillway capacity at Edenville Dam to meet the FERC PMF requirement.

The problem with the structure of the situation (game) in which the parties were engaged is that it resulted in a substantial amount of noncooperation. Viewed from the perspectives of the parties:

- When the dams were privately owned, the counties, lakefront property owners, and other lake users were not compelled to help pay for the costs of the dams. They had no incentive to do so if they could get substantial financial, recreational, and aesthetic benefits from the dams "for free," and they had no information indicating that the dams were at substantial risk of failure due to insufficient safety investments (although the public was generally aware that there was an issue with spillway capacity).
- From the point of view of the private dam owners, the profit margins from selling power generated by the dams were relatively small, even if expenses were limited to normal annual operating and maintenance costs. If a major safety investment was to be made, such as spending $5 million to $10 million to upgrade the Edenville Dam spillway capacity to fully meet the FERC PMF requirement, there would likely have been an annual financial loss on the project for many years, and it may not have been possible to secure a loan to fund such an upgrade.

The dam owners *would* have had an incentive to invest in major safety upgrades, even if that entailed some annual financial loss, if it was believed that doing so would prevent dam failure. But none of the engineering consultants and neither regulator indicated that the risk of such a failure was believed to be high enough to require major safety upgrades on an urgent basis (beyond the safety upgrades that had already been performed). Communications among the parties indicate that the desire to upgrade the spillway capacity was driven mainly by the need to meet formalized regulatory requirements, not by a perceived high risk of dam failure. And again, it must be emphasized that the physical mechanism of the Edenville Dam failure in May 2020 was fundamentally an unforeseen embankment instability failure, not an overtopping failure resulting from inadequate spillway capacity.

- Consumers had no incentive to pay a higher rate to the dam owners than it had to, since paying a lower rate to the dam owners would increase its profit margin when it would resell that power. In addition, Consumers had no reason to expect that paying a lower rate to the dam owners would contribute to a dam failure nor that such a dam failure would have significant adverse consequences for Consumers (other than loss of its ability to purchase power from the dam owners, which was only a very small percentage of the utility's energy portfolio). Moreover, as a large utility company that was essentially the only customer for the relatively small amount of power generated by the dams, Consumers may have had more bargaining power than the dam owners in negotiating the rate that would be paid for that power. The result was that Boyce Hydro was paid a rate that was below average compared to other small hydro power producers.
- The engineering consultants were "socially captive" (Johnston et al. 1996; Schmidt 2010) and had an incentive to make safety recommendations to the dam owners that would address obvious safety risks. However, the engineering consultants also had an incentive (motivational bias) to not make overly conservative recommendations that would pressure the dam owners to make costly safety upgrades. Such recommendations could displease the dam owners and motivate them to find other consultants for engineering services and recommendations. This structure of incentives may have contributed to engineering studies that were less thorough and less conservative than they ideally might have been.
- FERC derived no significant benefit from the project. Its mandate and obligation was to enforce its regulations, with the expectation that doing so would implicitly reduce the risk of dam failure to an acceptable level. However, FERC had limited options to enforce its regulations, and *none* of them were good options. When FERC finally revoked the Edenville Dam license as a "last resort," after about three decades in which the three dam owners did not upgrade the spillway capacity, their action could be viewed as somewhat increasing the risk of dam failure rather than decreasing it, for at least some duration, since pressure on the dam owner to eventually increase the spillway capacity to be able to pass the PMF was removed, funding for spillway capacity upgrades was diminished due to the loss of ability to generate power, and ability to release water through the powerhouse was lost. Although FERC had the authority to review the dam owner's financial position to inform its own decision-making, FERC's position and practice was that the licensee's financial position had no bearing on the requirement to comply with safety regulations, and FERC did not have any authority or ability to assist the dam owner with improving its financial position.
- EGLE derived no significant benefit from the project, but had mandates and obligations to enforce its regulations related to both dam safety and environmental protection, and there was

potential for these two sets of considerations to be in conflict. For example, lowering the lakes could improve dam safety and dam operator safety, but it could also potentially result in environmental impacts. EGLE had no authority to review the dam owner's financial position in order to inform its own decision-making, and EGLE did not have authority or ability to assist the dam owner with improving its financial position.

- The parties most at risk from the project, while deriving little or no benefit from the project, were the downstream property owners who did in fact experience devastating property losses of about $200+ million during the May 2020 flood event; a significant portion of this property damage was likely attributable to the dam failures. Fortunately, there was no loss of life or serious injury. The downstream property owners had limited information about the dams and limited ability to influence decisions being made in relation to the dams. They were therefore in a position where they simply had to rely on other parties to take actions to keep the risks posed by the dams at an acceptably low level. In effect, the downstream property owners were "innocent bystanders" who ultimately became "innocent victims" when the dams failed.

The net result of the structure of the situation (game) in which the parties were engaged was that the parties made decisions that were individually "rational," yet the set of decisions taken collectively over the history of project ultimately contributed to the failure of the dams, which was a bad outcome for *all* of the parties:

- As the dam owner at the time of the failures, Boyce Hydro lost its investment in the dams, had to file for bankruptcy, and faced numerous lawsuits.
- FLTF, on behalf of the counties and local lakefront property owners, had to spend much more money to rebuild the failed dams and restore the lakes than the costs that would have been incurred if the dams had not failed and safety improvements had been made to prevent the dam failures.
- Lakefront property owners lost the lakes for a substantial period of time, their property values were likely reduced during that time, and they faced the prospect of greatly increased annual fees through the SAD as a result of the dam failures.
- The engineering consultants involved in the project potentially faced liability and reputational damage.
- FERC and EGLE potentially faced potential liability and reputational damage.
- While the IFT did not evaluate this impact, the counties would be expected to collect lower property taxes from lakefront property owners until the lakes were restored.
- Consumers lost the power supplied by the dams and, therefore, the potential for profit on reselling that power.
- The downstream property owners experienced devastating and costly property losses during the May 2020 flood, a significant portion of which were attributable to the dam failures, and it is the IFT's understanding that a significant portion of these losses were not covered by property insurance.

Weighing all of these considerations, the structure of the situation (game) in which the parties were engaged can be considered to have been a significant contributing factor to the dam failures and the resulting adverse consequences for all of the parties. Viewed from the perspective of game theory, the situation *should* have been structured so that the parties were generally compelled to participate in a cooperative game, with minimal non-cooperative relationships. While there are various methods in game theory for "fair" cost allocation in a cooperative game, a straightforward "solution" is that each party should contribute to paying for project costs roughly in proportion to the share of benefits it receives from the project.

Since the vast majority of the benefit provided by these four dams went to the counties and the lakefront property owners, and since the revenue from power generation was a comparatively very small benefit, a good solution would have been for the dams to be owned by the counties, managed and operated by a delegated authority of the counties, and paid for by lakefront property owners through a SAD. This was precisely the solution that was planned to be implemented before the dams failed and even before the Edenville Dam license was revoked. The solution of transferring the dam ownership to the counties was apparently set in motion when the lakefront property owners and lake associations had concerns about the lake levels not being maintained, and thus the potential loss of the "free" benefits that they had been receiving for several decades. This solution – the counties taking ownership of the dams, with FLTF acting as their delegated authority – was actually implemented after the dams failed.

While this solution is a reasonable post-failure solution, a better solution would have been for the dams to have been purchased by the counties decades ago from the private dam owner (Wolverine) when it was first determined that the spillway capacity at Edenville Dam was inadequate relative to the FERC PMF requirement and Wolverine was unable to fund a spillway capacity upgrade to meet that requirement. Through a SAD, the counties would have had the financial means to outbid private parties and purchase the dams. If the dams had become publicly owned at that time, in addition to upgrading the spillway capacity, more in-depth engineering studies may have been done, which may have revealed the low embankment stability factors of safety in the Edenville left (east) embankment where the failure occurred. That finding may have led to remedial actions, such as downstream slope overlays covering the full length of Edenville Dam. Overlays would have increased the factors of safety for a conventional instability failure mode to acceptable levels and would likely also have coincidentally prevented an embankment static liquefaction flow failure even if the lake level reached the dam crest.

Another solution for changing the "game" in which the parties were engaged would have been to establish a public-private partnership (PPP) (Devernay 2009) in which the dams would remain privately owned, the lakefront property owners would contribute their "fair share" in paying for the operating, maintenance, and safety upgrade costs associated with the dams, and the dam owner would have obligations with respect to transparency and proper use of the funds contributed by the lakefront property owners. The arrangement of a PPP would have accomplished essentially the same results as public ownership of the dams. However, the structure of the game in which the parties were already engaged – which was a result of the history of the project – gave an incentive to the dam owner to proceed to establish a PPP, whereas the incentive for the lakefront property owners and counties was to continue to derive the benefits "for free" without entering into a PPP arrangement. There was no external party, governmental or otherwise, that had the authority and "span of control" to force a PPP arrangement to be established, and therefore a PPP arrangement was never established. The parties were trapped in their existing game, which unfortunately was largely non-cooperative, until the counties, acting through their delegated authority, finally had an incentive to take ownership of the dams on behalf of the lakefront property owners and thereby "change the game."

G-4 Summary

Going all the way back to the design and construction of Edenville Dam in the 1920s, the failure was preceded by decades of interactions and effects of human and physical factors, many of which were inadequate or detrimental. The overall set of interactions was complex, and eventually conditions unfortunately lined up in a way that resulted in failure of the dam.

There was apparently a significant degree of complacency and possibly overconfidence during the construction of the dam, which resulted in the dam's susceptibility to static liquefaction and embankment stability safety margins that were generally below average compared to other embankment dams of similar height that were built in Michigan during the 1910s through the 1930s. This may have been due to a general lack of quality control during construction, a desire to reduce construction costs, and/or a desire to complete the construction ahead of schedule in order to start generating power and revenue. During the nine decades after construction, some warning signs of the potential for embankment instability and high lake levels were missed, and many barriers that were intended to provide "checks and balances" were overcome to eventually produce the dam failure. This indicates a long-term inadequacy of general industry practices to recognize and address the deficiencies and warning signs that preceded the failure. The incident cannot reasonably be "blamed" on any one individual, group, or organization.

The inadequacies in dam safety risk management that contributed to the failure primarily involved ignorance about the existence of the risks associated with the dam, which was mainly due to inadequate industry practices for comprehensive geotechnical evaluation (including evaluation of the potential for static liquefaction) and for evaluating cold-season runoff characteristics in watersheds for rainfall on ground subject to freezing, particularly in cases where a dam has a spillway capacity that is much less than the PMF or the regulatory requirement. The spillway capacity itself was also overestimated due to inaccurate modeling assumptions. The FERC Part 12D process, while intended to provide thorough safety reviews of dams, in practice did not result in the type of comprehensive review that would likely have resulted in identifying the low embankment stability factors of safety, which in turn may have led to modifications that would have reduced the potential for static liquefaction. After the 2017 Oroville Dam spillway incident, FERC recognized this gap in the Part 12D process and revised its process, effective 2022, to require periodic comprehensive reviews. The lack of comprehensive reviews still remains an issue for most state-regulated dams.

An additional factor that contributed to the Edenville Dam failure was limited financial resources. Since the start of FERC regulation, the power generation revenue of the dam owners was inadequate to fund an upgrade of the spillway capacity of the dam to fully meet the FERC PMF requirement. The FERC license revocation in 2018 substantially decreased the dam owner's revenues. The primary beneficiaries of the dam were not actually the dam owners, but rather the counties, which collected additional property taxes because of the lakes, and the lakefront property owners. However, prior to the formation of a delegated authority for the counties, there was no arrangement in place that resulted in the counties and the lakefront property owners contributing to paying for the costs of the dam. FERC did not have the authority to order a breach of the dam, under non-emergency conditions. If a breach order had been available to FERC, such an order may have "changed the game" and promoted either public purchase or a public-private partnership (PPP) because a permanent dam breach would have resulted in a loss for all parties.

Overall, in terms of human factors, the safety "demands" that contributed to the dam failure were significant, while the systemic "capacity" to meet those demands and maintain dam safety was lacking in several areas. Almost a century after the design and construction of Edenville Dam, this systemic imbalance in human factors had set the stage for the dam failure to occur in May 2020.

G-5 References

The references below for this appendix are divided into several subsections, organized by topic. Within several of these subsections, the references are organized in chronological order (rather than alphabetical order by author) in order to give a sense of the historical development of the literature on that subtopic.

General Human Factors (Chronological Order)

Reason, J. 1990. *Human Error*, Cambridge University Press.

Senders, J. and Moray, N. 1991. *Human Error: Cause, Prediction, and Reduction*, Lawrence Erlbaum Associates.

Reason, J. 1997. *Managing the Risks of Organizational Accidents*, Ashgate Publishing Company.

Rasmussen, J. 1997. "Risk management in a dynamic society: a modelling problem," Safety Science, 27 (2-3), 183-213.

Dorner, D. 1997. *The Logic of Failure: Recognizing and Avoiding Error in Complex Situations*, Basic Books.

Perrow, C. 1999. *Normal Accidents: Living with High-Risk Technologies*, Princeton University Press.

Pidgeon, N. and O'Leary, M. 2000. "Man-made disasters: why technology and organizations (sometimes) fail," Safety Science, 34 (1–3), 15–30.

Patankar, M., Brown, J., Sabin, E., and Bigda-Peyton, T. 2001. *Safety Culture*, Ashgate Publishing Company.

Strauch, B. 2002. *Investigating Human Error: Incidents, Accidents, and Complex Systems*, Ashgate Publishing Company.

Woods, D. and Cook, R. 2002. "Nine Steps to Move Forward from Error," Cognition, Technology & Work, 4, 137–144.

Hollnagel, E. 2004. *Barriers and Accident Prevention*, Ashgate Publishing Company.

Hollnagel, E., Woods, D. and Leveson, N., Eds. 2006. *Resilience Engineering: Concepts and Precepts*, Ashgate Publishing Company.

Dekker, S. 2005. *Ten Questions about Human Error*, CRC Press.

Dekker, S. 2006. *The Field Guide to Understanding Human Error*, Ashgate Publishing Company.

Reason, J. 2008. *The Human Contribution: Unsafe Acts, Accidents, and Heroic Recoveries*, Ashgate Publishing Company.

Qureshi, Z. 2008. *A Review of Accident Modelling Approaches for Complex Critical Sociotechnical Systems*, Report No. DSTO-TR-2094, Australian Defence Science and Technology Organisation.

Woods, D., Dekker, S., Cook, R., Johannesen, L., and Sarter, N. 2010. *Behind Human Error*, 2nd Ed., Ashgate Publishing Company.

Rosness, R., Grøtan, T., Guttormsen, G., Herrera, I., Steiro, T., Størseth, F., Tinmannsvik, R., and Wærø, I., 2010. "Organisational Accidents and Resilient Organisations: Six Perspectives, Revision 2," No. Sintef A 17034, SINTEF Technology and Society, Trondheim.

Salmon, P., Stanton, N., Lenne, M., Jenkins, D., Rafferty, L., and Walker, G. 2011. *Human Factors Methods and Accident Analysis*, Ashgate Publishing Company.

Dekker, S. 2011. *Drift into Failure: From Hunting Broken Components to Understanding Complex Systems,* Ashgate Publishing Company.

Leveson, N. 2011. *Engineering a Safer World: Systems Thinking Applied to Safety*, MIT Press.

Conklin, T. 2012. *Pre-Accident Investigations: An Introduction to Organizational Safety*, Ashgate Publishing Company.

Catino, M. 2013. *Organizational Myopia: Problems of Rationality and Foresight in Organizations*, Cambridge University Press.

Hollnagel, E. 2014. *Safety-I and Safety-II: The Past and Future of Safety Management*, Ashgate Publishing Company.

Weick, K. and Sutcliffe, K. 2015. *Managing the Unexpected: Resilient Performance in an Age of Uncertainty*, 3rd Ed., Jossey-Bass.

Human Factors in Civil and Dam Engineering (Chronological Order)

Peck, R. 1973. "Influence of Nontechnical Factors on the Quality of Embankment Dams," in *Embankment-Dam Engineering: Casagrande Volume*, Wiley, 201-208.

Melchers, R., 1977. "Organizational Factors in the Failure of Civil Engineering Projects," *Transactions of Institution of Engineers Australia*, GE 1, 48-53.

Nowak, A., Ed. 1986. *Modeling Human Error in Structural Design and Construction*, ASCE.

Sowers, G. 1993. "Human Factors in Civil and Geotechnical Engineering Failures," *ASCE Journal of Geotechnical Engineering*, 119:2, 238–256.

Bea, R. 2006. "Reliability and Human Factors in Geotechnical Engineering," *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 132:5, 631–643.

Norstedt, U., Rollenhagen, C., and Eveneus, P. 2008. "Considering Human Factors in Dam Safety," HRW-Hydro Review Worldwide. (G-32) Regan, P. 2010) "Dams as Systems – A Holistic Approach to Dam Safety," 2010 USSD Conference Proceedings, 1307-1340.

Hill, C. 2012. "Organizational Response to Failure," 2012 USSD Conference Proceedings, 89-102.

Lord, D. 2012. "The Taum Sauk Dam Failure Was Preventable – How Do We Prevent the Next Operational Dam Failure?" 2012 USSD Conference Proceedings, 139-154.

Alvi, I., 2013a. "Human Factors in Dam Failures," 2013 ASDSO Dam Safety Conference Proceedings.

de Rubertis, K. and van Donkelaar, C. 2013. "Look Both Ways," 2013 USSD Conference Proceedings, 601-620.

Bryan, C. and de Rubertis, K. 2014. "Examining the Role of Human Error in Dam Incidents and Failures," *USSD Newsletter*, 164, 42-49.

Mattox, A., Higman, B., Coil, D., and McKittrick, E. 2014. "Big Dams & Bad Choices: Two Case Studies in Human Factors and Dam Failure," published online at: http://www.groundtruthtrekking.org/Issues/OtherIssues/dam-failure-human-factors-cases-TetonVajont.html

Myers, D., Ferguson, K., Alvi, I., and Baker, M. 2015. "Reexamination and Lessons Learned from the 2004 Failure of Big Bay Dam, Mississippi," *ASDSO Journal of Dam Safety*, 13:2, 9-18.

Alvi, I. 2015a. "Human Factors in Dam Failure & Safety, Case Study: Ka Loko Dam Failure," 2015 ASDSO Northeast Regional Conference Keynote Address.

Alvi, I. 2015b. "Failure of Sella Zerbino Secondary Dam in Molare, Italy," 2015 ASDSO Dam Safety Conference Proceedings.

Alvi, 2015c. "Human Factors in Dam Failure and Safety," ASDSO November 2015 webinar.

Ascila, R., Baecher, G., Hartford, D., Komey, A., Patev, R., and Zielinski, P. 2015. "Systems analysis of dam safety at operating facilities," 2015 USSD Conference Proceedings.

Hartford, D., Baecher, G., Zielinski, P., Patev, R., Ascila, R., and Rytters, K. 2016. *Operational Safety of Dams and Reservoirs: Understanding the reliability of flow-control systems*, ICE Publishing.

Alvi, I., Richards, G., and Baker, M. 2016. "10th Anniversary of Ka Loko Dam Failure, Hawaii," 2016 ASDSO Dam Safety Conference presentation.

France, J., Alvi, I., Dickson, P., Falvey, H., Rigbey, S., and Trojanowski, J. 2018. "Independent Forensic Team Report – Oroville Dam Spillway Incident."

Alvi, I. 2018. "Human Factors in the Oroville Dam Spillway Incident," ASDSO August 2018 webinar.

Alvi, I. 2020. "Judgment & Decision-Making in Dam Engineering," ASDSO November 2020 webinar.

Baker, M., Ettema, R., Teal, M., and Trojanowski, J. 2020. "Spencer Dam Failure Investigation Report," ASDSO.

Luck (Chronological Order)

Rescher, N. 1995. *Luck: The Brilliant Randomness of Everyday Life*, Farrar Straus & Giroux.

Fredriksen, S. 2005. "Luck, Risk, and Blame," *Journal of Medicine and Philosophy*, 30:5, 535-553.

Pritchard, D. and Whittington, L. 2015. *The Philosophy of Luck*, Wiley-Blackwell.

Pritchard, D. 2015. "Risk," *Metaphilosophy,* 46:3, 436-461.

Broncano-Berrocal, Fernando. 2015. "Luck as risk and the lack of control account of luck," *Metaphilosophy* 46: 1–25.

Hales, S. 2016. "Why Every Theory of Luck is Wrong," *NOUS,* 50:3, 490–508.

Fernando Broncano-Berrocal. 2016. "Luck," The Internet Encyclopedia of Philosophy, https://iep.utm.edu/luck/.

Church, I. and Hartman, R. (eds). 2019. *The Routledge Handbook of the Philosophy and Psychology of Luck*, Routledge.

Hales, S. 2020. *The Myth of Luck: Philosophy, Fate, and Fortune*, Bloomsbury.

Game Theory (By Author)

Binmore, K. 2007. *Game Theory: A Very Short Introduction*, Oxford University Press.

Dinar, A., Ratner, A., and Yaron, D. 1992. "Evaluating cooperative game theory in water resources," *Theory and Decision,* (32), 1–20.

Dinar, A. and Hogarth, M. 2015. "Game Theory and Water Resources: Critical Review of its Contributions, Progress and Remaining Challenges," *Foundations and Trends in Microeconomics,* Vol. 11, Nos. 1–2, 1–139.

Dixit, A., and Nalebuff, B. 1991. *Thinking Strategically*, Norton.

Hui, R., Lund, J.R. and Madani, K. 2015. "Game theory and risk-based leveed river system planning with noncooperation," *Water Resour. Res.*, 51.

Johnston, S., Lee, A., McGregor, H. 1996. "Engineering as Captive Discourse," *PHIL & TECH,* 1:3-4 Spring 1996. https://scholar.lib.vt.edu/ejournals/SPT/v1n3n4/pdf/johnsto.pdf

Liu, B., Yao, K., Wang, F., Chi, X., and Gong, Y. 2022. "Benefit Sharing in Hydropower Development: A Model Using Game Theory and Cost–Benefit Analysis," *Water*, 14, 1208.

Madani, K. 2010. "Game theory and water resources," *Journal of Hydrology*, 381, 225-238.

Madani, K., Darch, G., Parra, F., and Workman, M. 2015. "Using Game Theory to Address Modern Resource Management Problems," Grantham Institute Briefing Note No 2, Imperial College London.

McNamara, J. and Leimar, O. 2020. *Game theory in biology: concepts and frontiers*, Oxford University Press.

Pastine, I., Pastine, T., and Humberstone, T. 2017. *Introducing Game Theory: A Graphic Guide*, Icon Books.

Ross, D. 2021. "Game Theory," The Stanford Encyclopedia of Philosophy (Fall 2021 Edition), Edward N. Zalta (ed.), https://plato.stanford.edu/archives/fall2021/entries/game-theory.

Schmidt, J. 2010. "The Social Captivity of Engineering," *STRUCTURE* magazine, May 2010. https://www.structuremag.org/wp-content/uploads/2014/08/C-Infocus-Schmidt-May101.pdf

Wikipedia 2022. "Game theory." https://en.wikipedia.org/wiki/Game_theory

Zara, S., Dinar, A., and Patrone, F., 2006. "Cooperative game theory and its application to natural, environmental, and water resource issues: Application to Natural and Environmental Resources," World Bank Policy Research Working Paper 4073, WPS4072.

Other Topics Related to Human Factors (By Author)

Alvi, I. 2013b. "Engineers Need to Get Real, But Can't: The Role of Models," ASCE Structures Congress 2013, 916-927.

Devernay, J-M. 2009. "Finance: Combining Public and Private Funds: the PPP Approach," Hydro Review. https://www.hydroreview.com/world-regions/finance-combining-public-and-private-funds-the-ppp-approach/#gref

Gawande, A. 2010. *The Checklist Manifesto: How to Get Things Right*, Henry Holt and Company.

Gilovich, T., Griffin, D., and Kahneman, D., Eds. 2002. *Heuristics and Biases: The Psychology of Intuitive Judgment*, Cambridge University Press.

Kahneman, D. 2011. *Thinking, Fast and Slow*, Farrar, Straus and Giroux.

Kiser, R. 2010. *Beyond Right and Wrong: The Power of Effective Decision Making for Attorneys and Clients*, Springer Verlag.

Mitchell, M. 2011. *Complexity: A Guided Tour*, Oxford University Press.

Page, S. 2008. *The Difference: How the Power of Diversity Creates Better Groups, Firms, Schools, and Societies*, Princeton University Press.

Plous, S. 1993. *The Psychology of Judgment and Decision Making*, McGraw-Hill.

Shaver, K. 1985. *The Attribution of Blame: Causality, Responsibility, and Blameworthiness*, Springer Verlag.

Sunstein, C. and Hastie, R. 2015. *Wiser: Getting Beyond Groupthink to Make Groups Smarter*, Harvard Business Review Press.

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Appendix H: Resumes of Independent Forensic Team Members

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John W. France, PE, D. GE, D.WRE
JWF Consulting LLC

Areas of Expertise

Dam Safety
Dam & Levee Engineering
Civil - Geotechnical Engineering
Water Resources
Seepage/Slope Stability Analyses

Education

MS, Civil Engineering, Cornell University, 1976
BS, Civil Engineering, Cornell University, 1972

Licenses/Registrations

Professional Engineer, Colorado
Professional Engineer, Oklahoma
Professional Engineer, Massachusetts
Professional Engineer, North Carolina

Years of Experience

With JWF Consulting: 4 years

With AECOM: 25
With Other Firms: 16

Professional Associations

American Society of Civil Engineers
US Society on Dams
Association of State Sam Safety Officials
Colorado Water Congress
American Council of Engineering Companies in Colorado

Summary

Mr. France has more than 45 years of experience in engineering consulting and design. Most of Mr. France's technical work for the past 35 years has focused on dams and water retention structures. This experience includes dam safety inspections and analyses, risk analyses, detailed geotechnical and geological field and laboratory investigations, hazard classification, seepage and static stability analyses and evaluations, seismic stability/seismic deformation analyses, conceptual and final designs of new structures, rehabilitation of existing structures, and consultation during construction.

He has served on Consultant Review Boards for the U.S. Bureau of Reclamation for several dam safety rehabilitation projects and on a three-member Independent Panel that provided annual reviews of Reclamation's dam safety programs. He has served on two Advisory Boards for BC Hydro for dam safety studies and modifications of five of its dams. For the USACE, he has served on Senior Technical Advisory Panels for Wolf Creek Dam, Center Hill Dam, Isabella Dam, Success Dam, and Martis Creek Dam; provided Quality Control and Consistency Reviews for risk analyses for Herbert Hoover Dikes, Howard Hanson Dam, East Branch Dam, Lewisville Dam, Barker Dam, Brookville Dam, and Mill Creek Storage Dam; served on a risk analysis expert elicitation teams for Herbert Hoover Dikes, Success Dam, and Lewisville Dam; and served on an independent external review panel for modification designs for Isabella Dam.

He is an active member of the U.S. Society on Dams (USSD) and the Association of State Dam Safety Officials (ASDSO). He is a past Vice president and Ex-Officio Member of the Board of Directors for USSD, and he is a past chairman of the Affiliate Member Advisory Committee for ASDSO. Mr. France also served for six years as the private sector member on the National Dam Safety Review Board. In 2010, he was the recipient of the prestigious President's Award from ASDSO for his contributions to dam safety.

Experience

Mosul Dam, Iraq, U.S. Army Corps of Engineers. Facilitated a semi-quantitative risk analysis for Mosul Dam in support of the USACE's role as engineer of record for the re-initiation of foundation at this critical dam in Iraq. The three-week-long effort considered the full range of potential failure modes for this dam founded on rock strata with soluble gypsum layers. Also provided senior technical review and advice for the grouting program.

Spillway Incident Forensic Investigation Team, Oroville Dam, CA. Leader of a six-member team charged with investigating the February 2017 Oroville Dam spillway incident, which consisted of failure of a concrete spillway chute and erosion of a natural hillside downstream of the emergency spillway crest structure, ultimately resulting in the temporary evacuation of almost 190,000 downstream residents. The team is charged with developing opinions on causes of the incident, considering both physical factors and human factors.

Review of Dam Safety Risk Analysis Practices. Chair of a five-person team tasked with reviewing the dam safety risk analysis practices of the U.S. Army Corps of Engineers; the U.S. Department of the Interior, Bureau of Reclamation; and the Federal Energy Regulatory Commission. This effort was undertaken in response to a directive from the U.S. Congress, after the Oroville Dam spillway incident.

Herbert Hoover Dike, FL, U.S. Army Corps of Engineers. Served as an SME and estimator on a team that completed a potential failure mode analysis and detailed quantitative dam safety risk assessment for Herbert Hoover Dike, FL, which is a DSAC 1 (highest risk) facility in the USACE's inventory of dams. Potential dam safety concerns for this 150 mile long embankment structure centered on seepage and internal erosion potential failure modes and overtopping and overwash failure modes. This was a four year long risk analysis effort.

Independent Expert Panel, Isabella Dam, CA, U.S. Army Corps of Engineers. Served on a team for an independent expert panel review of design and construction of dam safety modifications for Isabella Dam, CA, which is a DSAC 1 facility. The modifications are being designed to address seismic stability and spillway capacity concerns for this existing facility.

Teller Dam, CO. Facilitator for semi-quantitative risk analyses for existing conditions and risk reduction alternatives. Senior technical reviewer for risk reduction alternatives.

USACE, Technical Advisory Panel, Wolf Creek Dam, Kentucky. Served as chairman of a Technical Advisory Panel reviewing design and construction of major dam safety modifications for Wolf Creek Dam, which is a Dam Safety Action Class (DSAC) 1 facility – the class of highest dam safety concern for the Corps of Engineers. The modifications were completed to address seepage concerns in the karstic foundation of the embankment section of the dam. The solution included foundation grouting and a deep concrete diaphragm seepage barrier wall of unprecedented proportions.

USACE, Technical Advisory Panel, Center Hill Dam, Tennessee. Served as chairman of a Technical Advisory Panel reviewing design and construction of major dam safety modifications for Center Hill Dam, which is a DSAC 1 facility. The modifications were completed to address seepage concerns in the karstic foundation of the embankment section of the dam. The solution included foundation grouting and a deep concrete diaphragm seepage barrier wall.

USACE, Technical Advisory Panel, Success Dam, California. Served on a five-member Technical Advisory Panel reviewing design and construction of major dam safety modifications for Success Dam, which is a DSAC 2 facility.

Technical Advisory Panel, Bolivar Dam, OH, U.S. Army Corps of Engineers. Served as member of a Technical Advisory Panel reviewing construction of a seepage barrier wall for Bolivar Dam, which is a DSAC 2 facility.

Board of Consultants, Chilhowee Dam, TN. Serving on a Board of Consultants for seepage related investigations and modifications of Chilhowee Dam, TN, and existing Federal Energy Regulatory Commission (FERC)-regulated hydropower dam in Tennessee.

Senior Review Board, Chimney Hollow Dam, CO. Member of a senior technical review board for design and construction of a new, 400-ft high, asphalt core rockfill dam to be constructed in Colorado.

United States Bureau of Reclamation, Consultant Review Board (CRB),Mormon Island Auxiliary Dam and Other Embankment Dams Associated With the Folsom Project, California. Member of Consultant Review Boards which provided senior technical review of dam safety evaluations, dam modification designs, and construction for one of the embankment dams that impound Folsom Lake, California. The principal dam safety issues are embankment and foundation seepage and piping, seismic stability concerns and inadequate spillway capacity. Modifications may include a large fuse plug spillway. To date, the work has involved a detailed review of Reclamation's risk analyses and conceptual designs of modifications.

United States Bureau of Reclamation, Consultant Review Board, Clear Lake Dam, California. Member of a two-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for an embankment dam located in northern, California. The principal dam safety issues were embankment and foundation seepage and piping concerns. The embankment dam was replaced with a new roller compacted concrete dam.

Toker Dam, Eritrea, East Africa. Project manager for design and construction of a new, 210-foot-high RCC gravity dam, in Eritrea. The design included preparation of complete plans and specifications for solicitation of tenders from international construction firms. Dam construction was completed in the summer of 1999, at a cost of about $20 million.

WSSC, T. Howard Duckett Dam Safety Analysis, Laurel, Maryland. Technical Reviewer for comprehensive safety analysis and alternatives evaluation for a 135- foot-high slab & buttress (Ambursen Dam) dam. Project includes detailed stability evaluations for the concrete dam and seven. 16-foot-high, 20-foot-wide taintor gates, probable maximum flood (PMF) estimate, seismic analysis, rock erodability evaluation, and development of rehabilitation alternatives.

United States Bureau of Reclamation, Consultant Review Board, Lauro Dam, California. Serving on a three-person CRB providing senior technical review of dam safety evaluations and dam modification designs for an embankment dam in California. The principal dam safety issue is stability and deformation during an earthquake.

United States Bureau of Reclamation, Consultant Review Board, Horsetooth Dam, Colorado. Member of a three-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for four large embankment dams located near Fort Collins, Colorado. The principal dam safety issues were seepage-related, including solutioning of limestone and gypsum foundation rock in the left abutment of one of the dams.

United States Bureau of Reclamation, Consultant Review Board, Keechelus Dam, Washington. Member of a three-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for an embankment dam located near Cle Elum, Washington. The principal dam safety issues were embankment and foundation seepage and piping concerns.

United States Bureau of Reclamation, Consultant Review Board, Wasco Dam, Oregon. Served as a single reviewer providing senior technical review of dam safety evaluations and dam modification designs for an embankment dam in Oregon. The principal dam safety issues were embankment and foundation seepage and piping concerns.

United States Bureau of Reclamation, Consultant Review Board, Red Willow and Norton Dams, Nebraska. Served as a single reviewer providing senior technical review of dam safety evaluations and dam modification designs for two embankment dams in Nebraska. The principal dam safety issues were embankment and foundation seepage and piping concerns.

Advisory Board Member, BC Hydro, Vancouver Island, Canada. Serving on an Advisory Board for review of BC Hydro's planned dam safety modifications of Strathcona, Ladore, and John Hart Dam.

Advisory Board Member, BC Hydro, Canada. Serving on an Advisory Board for review of BC Hydro's planned dam safety modifications of Ruskin and Blind Slough Dams.

United States Bureau of Reclamation, Member Dam Safety Independent Review Panel, Various. For five years, served as one of three members of an expert panel charged with providing an annual review of Reclamation's Dam Safety Program. The panel met twice per year, for one week at each meeting, to review Reclamation's dam safety program and provide findings and recommendations, as judged appropriate by the panel. A principal focus of the panel's activities was a detailed use of Reclamation's application of risk analysis and risk-based dam safety decision making.

United States Bureau of Reclamation, Comprehensive Facility Reviews, Prosser Creek, Norman Dam, and Whiskeytown Dam, Canada and California. Served as a senior engineer responsible for detailed, six-year comprehensive facility reviews, including updated risk analyses, for these three Reclamation dams. (2006, 2007, 2009 respectively)

AQUA Ohio, McKelvey Dam Risk Analysis, Ohio. Facilitated the risk analysis workshop for this 80-foot-high, thin-arch concrete dam in Youngstown, OH. Potential failure modes for flooding events were evaluated and probability of failure was estimated.

United States Fish and Wildlife Service, Rush Dam Risk Analysis, Oklahoma. Facilitated a hydrologic and seismic risk analysis workshop for this 65-foot-high concrete gravity dam in southwest Oklahoma.

Standley Lake Dam, Colorado. Served as an expert witness concerning interpretation of Colorado's Dam Safety Rules as they applied to recently constructed improvements to Standley Lake Dam.

United States Bureau of Reclamation, Seismic Risk Analysis, Utah. Principal- in-charge and facilitator for detailed seismic risk analysis for Echo Dam.

United States Bureau of Reclamation, Risk Analysis Team Member, Washington and Oregon. Served as a technical team member for detailed risk analyses completed by Reclamation for Cle Elum Dam, Conconully Dam, and Salmon Lake Dam, all located in Washington State, and Wickiup Dam, Oregon.

W.W. Wheeler and Associates and the City of Golden, Guanella Dam, Colorado. Geotechnical project manager for design and construction of a new water storage dam located near Empire, Colorado. Design and construction were accomplished on a 13-month, fast-track approach to address drought.

United States Fish and Wildlife Service, New Construction, Elmer Thomas, Oklahoma. Project Manager. Managed field investigations and conceptual and final designs of dam safety actions for an existing 97-foot-high earthfill/rockfill dam. Completed final design of a new 113-foot-high RCC replacement dam.

United States Bureau of Reclamation, Final Design and Construction Phase Consultation, Rye Patch Dam, Nevada. Project Manager. Provided final design and construction phase consultation for remediation of foundation liquefaction potential for this 75-foot-high existing earthfill dam. The successful construction bid for the project was $3.8 million, compared to the engineer's estimate of $5.5 million, as prepared by USBR's cost estimators. This project was a rare example of the USBR entrusting design of remediation of one of its dams to a consultant, and the successful construction of the project.

Proposed New Dams, Colorado. Project manager for preliminary design for two large new storage dams: a 380-foot-high structure located near Loveland, and a 150-foot-high structure located near Granby.

Rocky Pen Run Reservoir, Dam Design, Stafford County, Virginia. Senior technical review for a 130-foot-high embankment dam, gated spillway and inlet/outlet works, 20-foot-high saddle dike, 60 MGD river intake, pumping station, and pipeline.

Instructor, Seepage Analysis for Embankment Dams. Instructor for a two-day course for ASDSO, presented in 2014, 2016, 2017, 2018, and 2019 (twice).

Instructor, Slope Stability Analysis for Embankment Dams. Instructor for a three-day course for ASDSO presented in 2014 and 2016.

Instructor, Emergency Action Plans for Dams and Levees. Instructor for a course that has been presented thirteen different times across the United States.

Instructor, Embankment Dam Design. Instructor for a four-day course for the U.S. Army Corps of Engineers.

Instructor, Dam Seepage Rehabilitation. Instructor for a three-day course for the U.S. Army Corps of Engineers.

Instructor, Semi-Quantitative Risk Analysis (SQRA). Instructor for a three-day course for USSD, presented in 2019 and scheduled for presentation in 2020.

Publications and Presentations. Numerous publications and presentations for such organizations as ASDSO, USSD, and ASCE.

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Irfan A. Alvi, P.E.
President & Chief Engineer
ialvi@alviassociates.com

RESUME

Summary of Experience

Mr. Alvi has 33 years of multidisciplinary experience in structural, water resources, geotechnical, and transportation engineering for dams, hydraulic structures, and other infrastructure.

Mr. Alvi has completed many hundreds of projects involving inspection, materials testing, forensic investigation, studies, remedial design, and new design. Many of these projects have involved providing innovative solutions to meet challenging situations, with the result that many of his projects have received design awards.

Mr. Alvi has nationally-recognized expertise in dam engineering. He served as technical leader for Alvi Associates' Prettyboy Dam project, which received the *2010 National Rehabilitation Project of the Year Award* from the Association of State Dam Safety Officials (ASDSO), which is among the most prestigious awards attainable in the dam engineering profession, as well as three other awards in 2011, including an ASCE/MD Outstanding Civil Engineering Achievement Project award, ACEC/MD Engineering Excellence Outstanding Project Award, and ESB Outstanding Engineering Achievement Award. Mr. Alvi also received the prestigious *2018 ASDSO President's Award.*

More generally, Mr. Alvi's experience with dam projects has involved diverse technical aspects including inspection, materials testing, forensic investigation, potential failure mode analysis, hydrologic and hydraulic analysis, reservoir routing and spillway capacity analysis, spillway erosion analysis, dam breach modeling and inundation mapping, stream geomorphic study and restoration design, fish passage design, seepage and stability analysis, three-dimensional structural analysis, remedial design, design of new concrete and embankment dams, evaluation and design for dam removal, and construction management.

Mr. Alvi is internationally recognized as a pioneer and leader in the role of human factors in dam failure and safety, including organizational and industry aspects. He has served on the ASDSO Dam Failures and Incidents Committee (DFIC) since 2010, leading the committee's work on human factors, making numerous presentations on human factors at ASDSO conferences (including a keynote address), publishing several peer-reviewed papers, and at the request of ASDSO, in 2015, he presented a two-hour webinar (link) on human factors in dam failure and safety. In 2017-2018, he served as a human factors expert on the forensic team which investigated the spillway failures at Oroville Dam, and he presented a two-hour webinar (link) on this investigation for ASDSO in 2018. In 2019, he served as a human factors technical advisor for the investigation of the failure of Spencer Dam in Nebraska. In 2020, he expanded his work on human factors by presenting an ASDSO webinar on judgment and decision-making in dam engineering. Most recently, in 2020-2022, he has served as a human factors expert on the forensic team which is investigating the failures of Edenville and Sanford dams in Michigan.

Education

B.S. (Honors), 1989, Civil Engineering
University of Maryland

More than 70 credits of post-graduate coursework in engineering, risk analysis, geology, physics, chemistry, biology, and other topics

Professional Registrations

1994, PE, MD, 20775
1999, PE, VA, 033361
2012, PE, DE, 18065
2015, PE, WV, 021527
2016, PE, PA, 084842

Relevant Areas of Experience

Human Factors Investigation
Forensic Investigation
New Dam Design
Dam Rehabilitation Design
Hydraulic Structures
Dam Inspection
Materials Testing
Reservoir Routing Analysis
Open Channel Hydraulics
Erosion Analysis
Seepage and Stability Analysis
Structural Engineering
Geotechnical Engineering
Risk Analysis
Dam Removal Study
Fluvial Geomorphology
Construction Management

Examples of Dam Study and Design Projects

Prettyboy Dam in Baltimore, Maryland. Lead Engineer for aspects related to the gatehouse of this large high-hazard concrete gravity dam which is a key component in the water supply system for the City of Baltimore. Performed review of extensive records related the dam's construction and history (including previous crack monitoring and investigations), abovewater and underwater inspection using an ROV, concrete coring and testing, forensic investigation of structural cracking using three-dimensional structural analysis (accounting for creep effects related to structure/foundation interaction) and an innovative causes/effects matrix model, and gatehouse stability analysis considering a wide range of potential failure surfaces.

Based on the findings of this investigation and analysis, performed remedial design for a $6 million post-tensioned anchorage system installed underwater in water depths up to more than 100 feet and consisting of 38 anchors drilled up to 70 feet into the dam (the first system of this type in the world). Also performed contractor prequalification, and extensive construction-phase services including development and evaluation of a preproduction anchor testing program. This 15-year project received four major design awards, as noted above.

Greenbrier Dam in Washington County, Maryland. Mr. Alvi is serving as a Senior Technical Advisor and reviewer for forensic investigation and remedial design for a high-hazard earthen embankment dam, 63 feet high and built in 1965 for flood control and recreation. The dam has experienced seepage and possibly piping problems over the past few decades, with these problems accelerating in the past two years. The dam is currently being closely monitored, with the lake level being restricted. Mr. Alvi's role includes hydrologic, hydraulic, geotechnical, structural, inspection, O&M, and EAP aspects.

Mill Pond Dam in Cecil County, Maryland. As a Senior Engineer, participated in alternatives studies and preliminary design for dam reconstruction to address breach in 1999 of an embankment dam dating to circa 1837. Alternatives included elements such as a new twin-cell box culvert outlet structure with a multi-stepped weir and a fish ladder, reconstruction of the failed embankment, embankment widening to allow a wider roadway, roadway reconstruction, a new sheet pile wall, riprap slope protection, and measures to control seepage, piping, and erosion within the new and re-used portions of the embankment dam.

Seneca Crossing Dam in Montgomery County, Maryland. Lead Engineer for design for a new concrete gravity dam flanked by embankment dams at each abutment. The concrete gravity dam was selected in order to minimize the dam footprint, and thus reduce the impact to wetlands. Due to adverse subsurface conditions involving highly compressible and permeable materials, an innovative design founding the dam on steel piles was developed and a sheet pile cutoff wall extending 18 feet deep was designed for seepage and uplift control. This design is estimated to have reduced construction costs by at least 40% relative to a conventional concrete dam.

Examples of Dam Forensic and Human Factors Investigations

Edenville and Sanford Dams in Michigan. These two hydropower embankment dams were built in the 1920s. In 2020, Edenville Dam experienced a sudden stability failure when the reservoir rose, without overtopping of the dam. The resulting breach caused overtopping and failure of the downstream Sanford Dam, and the resulting flooding caused more than $200 million dollars in property damages. Fortunately, activation of emergency action plans, which resulted in evacuation of about 11,000, prevented loss of life. Mr. Alvi served on the forensic investigation team and participating in all aspect of the investigation, including hydrologic, geotechnical, operational, and emergency response aspects, while also leading the team's investigation of human factors.

Oroville Dam in California. This embankment dam is 770 feet high and is the tallest dam in the United States. In 2017, both the service spillway and emergency spillway of the dam experienced severe damage, resulting in evacuation of 188,000 people and a recovery cost of $1.1 billion.

An ASDSO/USSD task force conducted an international search to select six members of a forensic team to investigate the spillway failures. Mr. Alvi was selected to serve on the team as a human factors expert, while also participating in other aspects of the investigation related to structural, geotechnical, geologic, hydraulic, and erosion aspects. Human factors were given the same attention as physical factors in this investigation, and Mr. Alvi spent over 1,000 hours on this investigation. The forensic team completed its investigation in 9 months, and produced two interim memoranda as well as a 584-page final report (link). Mr. Alvi also presented a detailed review of the human factors findings of the investigation in a two-hour webinar for ASDSO (link).

The forensic team found that the service spillway failed by uplift of the chute slab. This uplift resulted from water injection through cracks and joints in the slab, likely driven by stagnation pressure. Contributing factors were an inadequate chute slab design, poor foundation conditions, and inadequate repairs. The unlined emergency spillway experienced erosion which was much more rapid and extensive than anticipated, due to the in-situ material being much erodible than the assumption of non-erodible rock. The human factors contributing to the incident were numerous, spanning the entire half-century history of the project and involving shortcomings in the practices of the owner, its regulators, its consultants, and the dam engineering and safety industry in the United States. Based on these human factors findings, the forensic team identified several lessons to be learned for the dam industry.

Spencer Dam in Nebraska. This run-of-river hydropower embankment dam has a gated concrete spillway structure and was built in the 1960s. In 2019, flooding and major ice run caused overtopping and breach of the embankment, as well as destruction of the spillway and powerhouse, resulting in downstream flooding which caused a fatality. Mr. Alvi served as a human factors technical advisor for the forensic investigation of this failure. Human factors contributing to the failure included lack of industry expertise and guidelines related to designing dams to withstand ice runs, inadequate documentation of the dam history (the dam had previously been damaged and failed due to ice runs), and development downstream of the dam without a revision of the dam hazard classification from significant to high hazard.

Prettyboy Dam in Baltimore, Maryland. This high-hazard concrete gravity dam is founded on micaceous schist, and is 150 feet high and 700 feet long.

By 1978, extensive cracking was observed in the gatehouse and the adjacent main body of the dam, along with substantial water leakage into the gatehouse stairwell. To respond to this concern, continuing until 1994, six investigations of the cracking had been performed by five previous consultants, but with inconclusive and/or inconsistent findings.

This lead to the forensic investigation for which Mr. Alvi served as Lead Engineer. This involved forensic structural/geotechnical investigation of the gatehouse cracking, eventually discerning that the cracks clustered into eight distinct groups, and likewise discerning three distinct general causes of the cracking, with each cause contributing in varying degrees to each crack group. In other words, a “cause-effect matrix” was developed, thus transcending the usual assumption of a simple one-to-one influence of cause to effect. The three identified causes of the cracking were vertical flexure of the dam, differential settlement between the gatehouse and main body of the dam, and deformation from the reactions of the bridge spans adjacent to the gatehouse. The hypothesized causal matrix was quantitatively validated by analyses of stresses and deformations of the dam, gatehouse, and bedrock, and the resulting predictions were found to fit the observed cracking remarkably well.

More broadly, preceding the design phase, the project involved a comprehensive multi-phase dam investigation involving many tasks: exhaustive review and summary of all available records, abovewater inspection, underwater inspection using divers and a remote-operated vehicle (ROV), precise mapping of defects throughout the exterior of the dam as well as inside the gatehouse, crack monitoring during gate testing operations, concrete coring and testing, analyses and evaluations, and preparation of a 300-page study report with recommendations.

The findings of the investigation were presented in the report for the client, a peer-reviewed paper in the ASDSO *Journal of Dam Safety* (link), and a presentation at the ASDSO national conference.

Big Bay Dam in Mississippi. This embankment dam was over 50 feet high and 2000 feet long, and failed in 2004, resulting in damage or destruction of more 100 structures. As Lead Human Factors Investigator, performed a comprehensive investigation of the failure, including review of many hundreds of pages of documents, including plans, calculations, construction records, deposition transcripts, engineering reports, etc.

Focusing on the human factors aspect of the failure, Mr. Alvi identified the roles of the engineer, owner, state regulatory agency, maintenance personnel, and inspectors, as well as the complex interaction of human factors and physical factors during the two decades from the design until the failure. Findings of the investigation were presented in a peer-reviewed paper in the ASDSO *Journal of Dam Safety*, a dedicated ‘soapbox’ session at the ASDSO national conference, an ASCE invited speaker presentation (link), and Mr. Alvi’s 2015 webinar for ASDSO.

Sella Zerbino Secondary Dam in Italy. The Sella Zerbino secondary dam was a concrete gravity dam about 46 feet high and 360 feet long. In 1935, a decade after construction, the dam failed catastrophically, resulting in at least 111 fatalities. Starting with the planning of the project, a series of human and physical factors interacted and compounded, until a 1000-year storm was the final physical trigger for the failure. Additional physical factors included lack of a spillway for the secondary dam, instability and erodibility of the foundation rock at the secondary dam, and grossly inadequate discharge capacity for the reservoir, which was exacerbated by clogging of spillways and outlets.

The human factors contributing to the failure included hasty design and construction of the secondary dam after a late decision to raise the height of the main dam, inadequate geologic investigation and missed warning signs related to the foundation of the secondary dam, and lack of rainfall data to adequately design spillways and outlets. Focusing on human factors, Mr. Alvi performed an extensive literature review, mapped out the role of physical factors, and contributed new insights into the failure by identifying the role of human factors in the failure, using the framework pioneered by Mr. Alvi. Findings of this investigation were presented in a peer-reviewed 2015 paper (link) and presented at an ASDSO national conference.

St. Francis Dam in California. This arched concrete gravity dam near Los Angeles was nearly 200 feet high, and failed in 1928, about four years after construction began and a day after fully filling the reservoir for the first time, resulting in a flood which extended more than 50 miles and resulted in at least 400 fatalities, along with millions of dollars of property damage. The failure is considered by many to be the worst US civil engineering disaster of the 20th century.

Focusing on human factors, Mr. Alvi performed a comprehensive investigation of the failure, including review of hundreds of pages of documents, including plans, engineering analyses, other investigations, etc., and identified the roles of the chief engineer, other engineers working under the chief engineer, City of Los Angeles, and local citizens who reported warning signs, as well as the complex interaction of human factors and physical factors during the years preceding the failure. Findings of the investigation were presented in a peer-reviewed 2013 paper (link) and a presentation at the ASDSO national conference.

Ka Loko Dam in Hawaii. This embankment dam was 42 feet high and 770 feet long, and failed in 2006, resulting in flood depths of 10 to 30 feet, seven fatalities, extensive property and environmental damage, a criminal sentence for the owner, and a civil settlement of many millions of dollars.

Mr. Alvi performed a comprehensive investigation of the failure, including extensive literature review of many hundreds of pages of documents, including plans, calculations, engineering reports, other investigations, news reports, etc., and identified the roles of the owner, Corps of Engineers, a trust which owned a portion of the reservoir, the County and Mayor, state regulatory agency, federal regulatory agencies, maintenance personnel, and inspectors, as well as the complex interaction of human factors and physical factors during the century preceding the failure. Findings of the investigation were presented at an ASDSO national conference, a keynote address at an ASDSO conference (link), and Mr. Alvi's 2015 webinar for ASDSO.

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Arthur C. Miller, Ph.D, PE, PH, D.WRE

Science Practice Leader; Hydrology & Hydraulics

Areas of Expertise
- Hydrology
- Hydraulics
- Floodplain Delineation
- Dam Safety
- Bridge Scour
- River Mechanics
- Sediment Transport

Years of Experience
- With AECOM: 16 Years
- With Other Firms: 34 Years

Education
- PhD/ Hydraulics and Water Resources/1972/ Colorado State University
- MS/Hydraulics and Water Resources/1967/Colorado State University
- BS/Civil Engineering/1965/ University of Massachusetts

Registration/Certification
- Professional Engineer: PA, MD, NC, VA, CO
- Professional Land Surveyor: PA
- Professional Hydrologist
- Certified Instructor-PA Dept. of Transportation
- Certified Instructor-National Highway Institute #1080

Overview

Dr. Miller has over 45 years of experience in water resources performing research, consulting and publishing in hydrology, hydraulics, floodplain delineation, dam safety, bridge scour, river mechanics, sediment transport and on impacts of Climate Change. He teaches courses both nationally and internationally on topics ranging from fundamental hydraulics, open channel flow, to hydrologic processes.

He has extensive experience with numerous hydrologic and hydraulic models and has taught courses for FERC dealing with many of these models. Additionally, he has taught H&H courses for FEMA, on dam safety for over twenty years. He is currently contracted by the Association of State Dam Safety Officials (ASDSO), as Developer and Instructor of HEC-RAS, Advanced HEC-RAS, and HEC-RAS 2D, HMS, and GeoHMS for Dam Safety. Dr. Miller is a certified instructor by The National Highway Institute and by The Pennsylvania Department of Transportation.

He spent a year as a Visiting Scientist with U.S. Army Corps of Engineers Waterways Experiment Station. He served as Chair of the Energy Power Research Institute (EPRI) Task Committee on Standards for PMP for Dam Safety Analysis, served on the ASCE Task Committee on Spillway Design Criteria for the Hydrologic Safety of Dams; established and helped write their National Guidelines for the Hydrologic Safety of Dams and served as Chair of a Task Committee for FERC that established their Federal Guidelines for Dam Safety.

Dr. Miller received several professional engineering awards for his expertise in water resources. In 2006, he received from The Department of The Army - Certificate of Appreciation for Patriotic Civilian Service for service and outstanding achievement while serving on the Interagency Performance Evaluation Task Force Consequences Team (IPET) in the wake of Hurricane Katrina. In 2007 he was the first recipient of the Terry L. Hampton Medal recognizing an individual making significant contributions to the dam engineering community in the fields of hydrology and hydraulics, given by the Association of State Dam Safety Officials (ASDSO).

RECENT PROJECT EXPERIENCE

Hawaii DLNR – Hawaii PMP Study (June 2021-Present) H&H Expert. Review and assess technical areas and meteorological decisions related to PMP development and hydrologic applications. Provide input into storm selection, storm analysis results, storm adjustments, transposition limits, PMP calculations, reasonableness of PMP depths, PMP spatial patterns, PMP temporal patterns, documentation review and hydrologic applications.

New Jersey Department of Environmental Protection (2020-Present) H&H Expert. AECOM is developing a site specific probable maximum precipitation (PMP) with Applied Weather Associates.

AECOM will also be developing the temporal rainfall distribution to determine the Probable Maximum Flood (PMF).

Pennsylvania PMP Panel - Expert Hydrologist (22016-2019) H&H Expert, Pennsylvania Department of Environmental Protection (DEP), Bureau of Waterways Engineering and Wetlands. This review panel was to oversee the development of probable maximum precipitation (PMP) studies for the purpose of updating PMP values for the entire state of Pennsylvania.

Dam Safety & Floodplain Management Panel - Expert Hydrologist (2013-2016) H&H Expert, Virginia Department of Conservation & Recreation. The Virginia House and Senate passed and the Governor signed two bills requiring a study to update statewide PMP values for the state of Virginia, the panel was established to provide expertise and to oversee the project.

Folsom Dam Water Control Manual Update (June 2017) – H&H Expert, U.S. Army Corps of Engineers, Sacramento, CA District, Independent External Peer Review (IPER). Reviewed and provided expertise for the hydraulics & hydrology of the Folsom Dam Water Control Manual Update.

Little Calumet River Flood Risk Management Project –(2017)- H&H Expert, U.S. Army Corps of Engineers, Chicago, IN District, Independent External Peer Review (IPER). Reviewed and provided expertise for the hydraulics & hydrology of the Little Calumet River Flood Risk Management Project, Engineering Documentation Report.

Fargo Morehead Metropolitan Area Flood Risk Management Project - H&H Expert, Fargo-Moorhead Metro Flood Diversion Authority, Fargo, ND (2016-2017). This comprehensive Project consisted of an embankment (dam) with upstream water staging/storage, a downstream diversion channel, flood protection along the Red River of the North, in Fargo and Moorhead, and associated infrastructure and mitigation projects. The purpose of the project was to provide permanent certifiable flood risk reduction for the protected area.

Barataria Sediment Diversion Project (BA 153), (March 2017)-H&H Expert, State of Louisiana and USACE, Baton Rouge, LA. Coastal Protection and Restoration Authority. Provided expertise on the numerical modeling for the design of an intake structure to divert flow from the Mississippi River with Sediment-to-Water (SWR) ratio of 1 or greater at the maximum flow capacity of 75,000 cfs. Modeling was also conducted to assist in the design of the conveyance channel and outfall transition to efficiently transport sediment to the Barataria Basin.

Safety Assurance Review (SAR) for the Isabella Lake Dam Safety Modification Project, H&H Expert, USACE Sacramento, Isabella, CA. (Completed 2016). The objective of this project was to assess, analyze, interpret, and evaluate design and engineering criteria through a process known as Type II Independent External Peer Review (IEPR) Safety Assurance Review (SAR) for the Isabella Lake Dam Safety Modification Project.

Ohio Dams Inflow Design Flood Study Project, H&H Expert, USACE Huntingdon District for Dover, Bolivar and Mohawk Dams (2015). Reviewed the HEC-HMS and HEC-ResSim models being run for the Dover, Bolivar and Mohawk Dams. The review included the wind-wave run-up and the PMP calculations.

Selecting and Accommodating the Inflow Design Floods for Dams, Technical Advisor/H&H Expert, Federal Emergency Management Agency (FEMA), Department of Homeland Security. 2010 – 2013. The project was to develop and publish a guidance document for the evaluation of the risk-based hydrologic safety of dams, including guidelines for determining the Inflow Design Flood (IDF) for new and existing dams. The Guidance Document provides a tool to assist state dam safety programs in evaluating the adequacy of their current hydrologic guidelines. The effort included an in-depth review of past hydrologic design practices and guidelines as well as documentation of the state of the practice for evaluating the hydrologic safety of dams, including a review of hydrologic guidelines currently used in each state and federal agency that regulates dams. The summation of this initial research has been published by FEMA in a report titled Summary of Existing Guidelines for Hydrologic Safety of Dams (FEMA 2012).

Downstream Effects Resulting from the Operation of the Pompton Lake Dam Floodgates, Technical Advisor/H&H Expert, State of New Jersey, Department of Environmental Protection (2011 – 2012). The Governor's Passaic River Basin Flood Advisory Commission charged New Jersey Department of

Environmental Protection (NJDEP) to evaluate the operational impacts of the Pompton Lake Dam Floodgate Facility. NJDEP retained AECOM to conduct a study on the impact of gate operations during flood events. The floodgate facility provides an unconventional approach to flood mitigation and only provides flood reduction benefits to upstream residents. The USACE HEC-RAS Unsteady Flow model was used to determine downstream water surface elevations during various flood events. The HEC-RAS model incorporated the existing rules for controlling the floodgate openings and operation. An existing HEC-HMS model was used to develop inflow hydrographs for input into the unsteady flow HEC-RAS model

Flood Insurance Studies - Review of the statistical validity of the modeling efforts, H&H Expert. FEMA Region IV contracted Watershed IV Alliance to conduct Flood Insurance Studies in the five counties surrounding Lake Okeechobee, Florida (2012). Analysis of work being done by the USACE on the Herbert Hoover Dike, the structure surrounding the lake. To leverage existing studies, reduce duplicative efforts, and minimize the chance of conflicting model results, FEMA would like to use current USACE dam break modeling of the system for flood mapping purposes. This project reviewed the statistical analysis of 1% chance of failure of the levee system for Lake Okeechobee.

Quality Control/Quality Assurance Expert for FERC, Ameren Hydroelectric Company, St. Louis MO and Paul Rizzo & Associates (2007 – 2009). Review of the Probable Maximum Precipitation (PMP) and Probable Maximum Flood (PMF) for the design flood for the Bagnell Dam and Harry S. Truman Dams, to assure the continued operation of a reliable power source in the region while minimizing downstream erosion and protecting water quality.

Impact of Climate Change on the NFIP and Improving Coastal Flood Plain Mapping, FEMA HQ Nationwide. (2008 – 2013), Technical Advisor. This study involved examining the impacts of climate change on the NFIP by assessing existing research, data, and reports on climate change. This study was to quantify how the 100-year flood (the flood having a 1% chance of exceedance) may change, based on climate model projections through the year 2100.

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Jennifer Williams, PE

Geotechnical Engineer

Areas of Expertise

Dams and Dam Safety

Risk Assessment

Geotechnical Engineering

Years of Experience

With AECOM: 25 Years

With Other Firms: 0 Years

Education

MS/ Civil Engineering/2002/ University of Colorado

BS/Civil Engineering/1997/Colorado School of Mines

Registration/Certification

Professional Engineer: CO, OK, TX, WY, OR, WA, HI, ID

Professional Affiliations

Association of State Dam Safety Officials

U.S. Society of Dams

American Society of Civil Engineers

Overview

Ms. Williams is a licensed Professional Engineer with 25 years of experience managing and leading geotechnical engineering projects. The majority of her professional career has focused on dam safety. She has worked on over 40 embankment and concrete dams including inspection; risk assessment; field investigations and characterization; seepage, stability and deformation analyses; feasibility and alternatives studies; final design drawings and contract documents; and construction inspection observation.

Her design engineering experience includes new dams and rehabilitation design of existing embankment dams for seepage, stability and hydrologic deficiencies. She has been the lead designer on over 10 dams and containment facilities in the last 15 years. Ms. Williams' field and construction experience includes preparing and managing the field investigations for earthfill dams; preparation of construction specifications and drawings; and engineer in charge of construction of rehabilitation measures. Her dam safety inspection experience includes serving as the Senior Engineer/Independent Consultant for over 20 dam safety inspections of state and federal regulated dams.

RECENT PROJECT EXPERIENCE

Ms. Williams served as the FERC-approved Part 12D Independent Consultant for the below dams. The scope of work included a field inspection of the dam and appurtenant facilities, facilitating a PFMA review session, and comprehensive review of the Supporting Technical Information Document (STID) and all associated technical analyses, operation/maintenance plans, and inspection and monitoring plans:

Piercefield Dam, Brookfield (2016)

Upper Newton Falls Dam, Brookfield (2016)

Allen Falls Dam, Brookfield (2015)

South Colton, Five Falls, and Rainbow Dams, Brookfield (2014)

Yale Dam, PacifiCorp (2017, 2022)

Swift Dam PacifiCorp (2017, 2022)

Oneida Dam, PacifiCorp (2014)

Lemolo No. 1, Toketee, Soda Springs and Prospect Dams, PacifiCorp (2014)

Toketee, PacifiCorp (2019)

Ashton Dam, PacifiCorp (2016, 2021)

Alcona Dam, Consumers Energy (2021)

Electric Lake Dam, PacifiCorp (2019); Performed Part 12d dam safety review following FERC Guidelines for state-regulated dam.

Bridger Surge Pond Dam, PacifiCorp (2015 and 2020); Performed Part 12d dam safety review following FERC Guidelines for state-regulated dam.

RECENT DAM ANALYSIS AND DESIGN PROJECTS

Northern Colorado Water Conservancy District, Glade Dam Design, Colorado. AECOM's Project manager and lead geotechnical engineer. AECOM, under subcontract with Black & Veatch, is performing the geotechnical field investigations, borrow study, analyses, and design of the new Glade Dam and its appurtenant hydraulic structures. The dam will be a 280-foot-high zoned earth dam impounding 170,000 acre-feet of off-channel water storage. (2015-present)

Upper Brushy Creek WDIC, Final Design of Dam 101, Texas. Engineer of Record for the design of the new flood control structure, which included an earthen embankment dam, and principal and auxiliary spillways. (present)

PacifiCorp, Seismic Rehabilitation for Yale Saddle Dam, Washington. Project Manager and senior geotechnical engineer for the investigation, analyses, development and evaluation of remediation to address seismic stability of zoned earthfill dam founded on potentially liquefiable soils. (2020-present)

Bureau of Reclamation, Echo Dam Corrective Action Study and Final Design, Utah. Project Manager and geotechnical engineer for a Seismic Risk Analysis Issue Evaluation, Corrective Action Alternatives Study, and Final Design for remedial works. Key project components included liquefaction analysis, strength characterization, post-seismic stability analyses, deformation analyses, risk analysis using the PrecisionTree and @Risk software programs, alternatives study for seismic upgrades to the embankment and gated spillway crest structure. Embankment alternatives evaluated included deep excavation and replacement, dynamic compaction, and jet grouting of the liquefiable downstream foundation materials. (2009 –2012)

Bureau of Reclamation, Mormon Island Auxiliary Dam, Appraisal Design, California. Project Manager and geotechnical engineer for the seismic evaluation and alternatives study for remediating liquefiable foundation materials. Key project components included strength characterization, post-seismic stability analyses, design of deep secant excavation shoring system, constructability review including participating in and documenting a Construction Risk Analysis Workshop and developing a construction cost estimate. (2009)

PacifiCorp, Rehabilitation of Ashton Dam, Idaho. Project Manager and geotechnical design engineer for the alternatives analysis, final rehabilitation design, and construction oversight services for a zoned earth and rock embankment dam. The scope included an alternatives analysis, which consisted of evaluating alternatives using technical, constructability, cost, and risk considerations. The risk analysis included estimating baseline risks and risk-reduction for rehabilitation alternatives. Final design services include geotechnical design analyses, construction level drawings, specifications, bid schedule and cost estimate. The rehabilitation design included a diversion tunnel through the basalt abutment with a slide gate control structure, excavation of the upstream portion of the zoned earth embankment, and reconstruction of the embankment including a rockfill core, three-stage filter, low-permeability silt core zone, crack stopper and upstream rock buttress. (2008 – 2013)

Natural Resources Conservation Service (NRCS), Various Sites, Rehabilitation Final Design, Oklahoma. Project Manager and Sr. Responsible Engineer for the final rehabilitation design for four earth embankment dams (Upper Clear Boggy 32, Sallisaw 18M, Cottonwood 17, Cottonwood 16) . Design services included geotechnical design analyses, construction level drawings, specifications, bid schedule and cost estimate to complete the rehabilitation to upgrade the dam to meet minimum high hazard federal and state dam safety requirements. Rehabilitation included a dam raises, new principal spillway intakes and conduits, widening earthcut auxiliary spillways, and seepage collection systems.(2006-2009)

RISK ANALYSIS EXPERIENCE

U.S. Forest Service, Nugget Creek Dam, SQRA, IRRMP and O&M Plan, Alaska. Facilitator for performing a semi-quantitative risk analysis (SQRA) and developing an interim risk reduction measures plan (IRRMP) and Operations & Maintenance (O&M) Plan based on the risk results. The SQRA was performed in general accordance with USACE/Reclamation Best Practices and the IRRMP was developed in general accordance with USACE Policy and Procedures (2021).

Screening-Level Risk Assessments, New Mexico Dam Safety Bureau of the State Engineer's Office (SEO), (2019). Facilitator and geotechnical subject matter expert for the risk prioritization program of New Mexico's state-regulated high and significant hazard dams. Ms. William's role consisted of developing and presenting an owner's outreach presentation on the use of risk-informed prioritization, developing SLRA templates, facilitating one-week SLRA workshops and/or serving as subject matter expect for the risk estimates. The scope for 2020 includes performing three one-week workshops and evaluating up to 20 of their high and significant hazard dams. (2020)

Screening Level Risk Analysis, and Phase 1 Dam Safety Reviews, Hawaii Department of Land and Natural Resources. Ms. Williams served as facilitator for a two-day training and pilot study workshop in 2018 to perform a screening level risk assessment for four dams within the state's dam safety branch jurisdiction. Ms. Williams served as senior engineer for Phase 1 inspections and facilitator for SLRA workshops on four dams In 2020 and eight dams in 2021. The SLRA methodology was done in general accordance with FERC Risk-Informed Decision Making (RIDM) guidelines. (2018, 2020, 2022)

SQRAs for Comprehensive Dam Safety Evaluations, Colorado Dam Safety Branco of the State Engineer's Office, Colorado. Project Manager and SQRA subject matter expert for the Comprehensive Dam Safety Evaluations (CDSEs) of six dams. Scope of work included a review and summary of all available information, developing a PFM brainstorm and screening list, conducting an SQRA, and preparation of a CDSE report for each dam. In 2016, Ms. Williams, together with Mr. John France developed and presented a 2-day SQRA training course for Colorado SEO staff and assisted their team in developing the CESE programmatic documents including a library of PFMs descriptions. (2016, 2019)

Consumers Energy, Alcona Hydroelectric Project, RIDM Level III Semi-Quantitative Risk Analysis. Ms. Williams is serving as a subject matter expert (SME) and risk estimator for one of the first FERC RIDM pilot studies. The project consists of data review, site inspection, conducting a Level III SQRA, and eventually conducting a Level IV Quantitative Risk Analysis (QRA) to address primarily flood- and seismic-related PFMs for the Alcona dam. The first Level III SQRA working session was completed in September 2017. Current work consists of performing studies and analyses to support a Level IV QRA (2017-ongoing).

Denver Water, Ralston Dam Spillway SQRA and Modification Design, Golden, CO. Facilitator and Geotechnical Subject Matter Expert for a SQRA for the existing concrete lined service spillway. The spillway was approximately 500 feet in length and trapezoidal in shape. The results of the SQRA recommended replacing portions of the spillway chute to reduce dam safety and operational risks. (2018)

United States Army Corps of Engineers, Herbert Hoover Dike Risk Analysis, Florida. Project Manager, Risk Estimator, and report reviewer serving on the team responsible for performing a detailed risk analysis for the 143-mile dike around Lake Okeechobee. Risk analyses were performed in a series of 15 one-week long risk workshops using the expert elicitation process. Scope included identification and screening of Potential Failure Modes and performing quantitative risk analysis on all viable Failure Modes. Risk analyses were completed for existing (baseline) conditions as well as risk reduction for various rehabilitation alternatives following U.S. Corps of Engineers risk guidelines. (2011 – 2014)

BF Sisk Dam Corrective Action Study, Reclamation, Santa Nella, California. Project manager, Precision Tree and @Risk operator, and report reviewer for this risk analyses of baseline (existing conditions) and rehabilitation alternatives for seismic upgrades of the 380-ft high earth embankment that impounds over 2M acre-feet. Scope also included development of constructability report. (2011 – 2013)

PROFESSIONAL SOCIETIES

Association of State Dam Safety Officials (ASDSO): Ms. Williams was awarded the 2015 ASDSO West Regional Award of Merit for her contributions to dam safety in the region. She serves on the ASDSO Advisory Committee and Dam Design and Construction Technical Issues Committee. She has published papers and delivered presentations at numerous ASDSO National conferences. In addition, she is an instructor for ASDSO webinars and seminars listed below.

Presenter, Filters and Drainage Systems for Embankment Dams: Presenter for the ASDSO webinar (2020).

Instructor, Evaluation and Rehabilitation of Embankment Dams for Seepage Concerns: Instructor for a one-day workshop for ASDSO (2019)

Instructor, On-Site Response Guidance for Seepage and Internal Erosion Incidents: Instructor for a half-day workshop for ASDSO (2018)

Instructor, Internal Erosion of Dams: Instructor for a 2-day course for ASDSO sponsored by State of Wyoming and Department of Homeland Security National Dam Safety Program. (2017)

Instructor, Definition and Evaluation of Internal Erosion Potential Failure Modes: Instructor for a one-day workshop for ASDSO (2016).

Instructor, Seepage Analysis for Embankment Dams: Instructor for a 2.5-day course for ASDSO. (2014, 2016, 2018, 2019, 2020, 2021, 2022)

Presenter, Seepage Rehabilitation: Presenter for the ASDSO webinar (2015).

AREAS OF EXPERTISE

- Concrete Dams
- Outlet Works
- Spillways
- Water Conveyance Structures
- Decommissioning of Dams

EDUCATION

Post Graduate Work in Structural and Hydraulic Engineering, University of Colorado

BS, Civil Engineering, Colorado State University, 1971

PROFESSIONAL HISTORY

Hydraulic Structures Engineer Consultant, 1998-Present
Bureau of Reclamation, Technical Specialist and Supervisor/Manager - Waterways and Concrete, Dams Group, 1976-1997

U.S. Geological Survey, Hydraulic Engineer, 1971-1975

REGISTRATIONS/ CERTIFICATIONS

Registered Professional Engineer: State of Colorado No. 13303

PROFESSIONAL SUMMARY

Mr. Higinbotham has more than 40 years of experience in the design of hydraulic structures for large dams.

For the Bureau of Reclamation, Mr. Higinbotham prepared final designs and specifications for spillways and outlet works for numerous dams.

As supervisor/manager within the Waterways and Concrete Dams Group of Reclamation, responsibilities included:

- Plan, direct, and support the work associated with safety of dams' studies (including Risk Analyses), appraisal and feasibility level designs, modification designs for existing facilities, new appurtenant structure and concrete dam designs, and special studies and analyses to address problems and concerns.
- Inspect new sites and existing facilities, develop field investigation programs, discuss design and construction issues, and coordinate activities between offices.
- Provide a technical review of work within the Waterways and Concrete Dams Group to ensure technical quality and consistency with current standards and policies.

Participated on interagency teams to evaluate and develop concepts for the removal of dams and hydroelectric facilities for environmental considerations. Projects included two concrete dams on the Elwha River near Port Angeles, Washington; Savage Rapids Dam near Grants Pass, Oregon; and the lower four Snake River dams in southeastern Washington, and the Klamath River dams near Klamath Falls, Oregon.

While with Reclamation, contributed to the development of several standards and manuals including: the development of criteria for designing low level outlet works; the development of criteria for using and designing fuse plugs in auxiliary spillways; the development of Reclamation guidelines for the examination frequency of normally inundated portions of outlet works; and the development of Reclamation guidelines for controlling seepage and preventing piping of earthfill along conduits.

RELEVANT EXPERIENCE

- **McPhee Dam and Great Cut Dike, Delores Project, Colorado**
  This project involved the construction of a 250 foot and 200 foot high embankment dams and included a two radial gated spillway with a capacity of 33,000 cfs. A cut and cover outlet works was designed for the lower dam,

AFFILIATIONS

American Society of Civil Engineers, 1968-1996. Member of the Hydraulic Structures Committee, served as Chairman for one year.

REFERENCES

Ernie Hall
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Waterways and Concrete Dams
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and a 20-foot diameter tunnel outlet works was designed for the larger dam. A shaft and footbridge was provided for access to the larger outlet works.

- **Stewart Mountain Dam, Salt River Project, Arizona**. This project included seismic stabilization of the existing arch dam, addition of a new right abutment radial gated spillway, and installing a bulkhead (underwater) for enlarging the existing outlet works/penstock.
- **Theodore Roosevelt Dam, Salt River Project, Arizona.** This project involved raising the existing cyclopean concrete arch dam 77 feet, replacing existing spillways with new top seal radial gated spillways through the thrust blocks on each abutment (design capacity of 150,000 cfs), and constructing a new tunnel outlet works/penstock through the left abutment. The outlet works included a lake tap at 150 feet in the reservoir. The project included a physical hydraulic model study of the new spillways, and prototype tests of bond strength at the interface of the new concrete and existing dam.
- **Upper Pony Creek Dam, Coos Bay North Bend Water Board, Oregon**
This project involved the construction of a new 100-foot high embankment dam with a low-level outlet works along the right abutment of the dam. The outlet works included a modified baffled energy dissipation structure.
- **Soldier Canyon Dam Outlet Works, Colorado Big Thompson Project - Colorado**
This project involved final designs for installing a steel liner inside an existing tunnel outlet works, and backfill grouting around the perimeter of the liner. The liner was required to accommodate converting the tunnel from a free flow conveyance to a pressurized conveyance.
- **Upper San Joaquin River Basin Storage Investigations, California.**
This work included appraisal level designs of new RCC gravity dams, arch dams, and concrete faced rockfill dams that ranged in size from 300 to 600 feet high. The appurtenant structures for each dam included spillways (design capacity of 150,000 cfs), construction diversion tunnels (30-foot diameter), and combined outlets works/penstock, and powerplant. Additional alternatives evaluated included raising the existing Friant Dam, which is a 300-foot high concrete gravity dam.
- **Elwha River Restoration Project, Washington.**
Develop dam removal concepts for two major concrete dams with hydroelectric facilities. Prepare final structural

designs for water quality mitigation facilities, which included the city of Port Angeles water treatment plant, and an intake structure, pump station, and fish screen structure for a new river diversion facility with a capacity of 180 cfs.

- **Deer Flat Dam Outlet Works, Idaho.**
  Prepared hydraulic and structural designs for a replacement outlet works, and prepared structural designs and details for modifications to an existing outlet works.
- **Carter Lake Dam No. 1 Outlet Works**
  Prepared hydraulic and structural designs for a new tunnel outlet works through the left abutment of the existing dam, including a selective level intake tower, meter structure, and valve structure.
- **Rocky Pen Dam (New Dam)**
  Participated in the design of the labyrinth spillway, outlet works with a multilevel intake tower, and closure details for the temporary diversion structure.
- **West Silver Basin Dam (New Dam)**
  This is a new 130-foot-high asphalt core rockfill dam with a spillway, reservoir inlet structure, and outlet works.
- **Glade Reservoir Project (New Dam)**
  Developing layouts for a tunnel low level inlet/outlet works near the left abutment, a new high-level outlet works on the right abutment, and a spillway on the right abutment. The high-level outlet works includes selective level withdraw capability.
- **Chimney Hollow Dam Consultant Review Board**
  Participating as a member on the review board for the new Chimney Hollow Dam. Performing reviews of the hydraulic structure designs and analyses, including a tunnel inlet/outlet works, spillway, and water conveyance structures.

Mr. Higinbotham has served on the following **Consultant Review Boards** for the Bureau of Reclamation:

- Glendo Dam Corrective Action Study, including the review of associated risk analyses.
- A. R. Bowman Dam Corrective Action Study, including the review of associated risk analyses.
- Echo Dam Corrective Action Study and Seismic Issue Evaluation, including the review of associated risk analyses.
- Deer Creek Dam Spillway Modification Designs
- Scofield Dam Spillway Modification Designs and Construction

- Wheatfields Dam Out Works Replacement Designs
- Stampede Dam Raise and Spillway Modification, including the review of associated risk analysis. Included a construction site visit.
- Bull Lake Dam Corrective Action Study and final designs for Spillway Modifications.
- Altus Dam Corrective Action Study and final designs for hydrologic deficiencies. Included a construction site visit.
- Hyrum Dam Corrective Action Study and final designs to address existing spillway deficiencies.
- Steinaker Dam Corrective Action Study and final designs to address an upstream slope failure and the upstream extension of the outlet works.

Mr. Higinbotham has served on the following QCC (Quality Control and Consistency Review) panels for the USCOE Baseline Risk Assessment Studies:

- Bluestone Dam – Concrete Gravity Dam with overtopping issues in West Virginia.
- Clearwater Dam – Embankment Dam with karstic foundation issues in Missouri.
- Mohawk Dam – Embankment Dam with potential failure modes related to piping through the embankment and spillway outlet channel erosion head cutting up to control weir.
- Lewisville Dam QCC - Embankment Dam with potential failure mode related to dam overtopping due to insufficient spillway capacity for new PMF.
- Isabella Lake Dam Safety Modification Design Reviews. Modifications include a dam raise, new curved labyrinth spillway, and upgrades to the existing service spillway and outlet works. Currently serving on the IEPR Panel for construction phase of the DSMP.
- Cherry Creek Dam - Dam Safety Modification Study. Embankment Dam with potential failure mode related to dam overtopping due to insufficient spillway capacity for new PMF.

PUBLICATIONS

Mr. Higinbotham participated in the preparation of the ASCE publication “Guidelines for the Retirement of Dams and Hydroelectric Facilities,” and also authored and co-authored several publications related to hydraulic and structural designs and modifications of hydraulic structures for dams, and overtopping protection of dams.

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