

INVESTIGATION REPORT FOR COON CREEK WATERSHED STRUCTURES 21, 23, AND 29,  
AND WEST FORK KICKAPOO WATERSHED STRUCTURES MLSNA AND 1  
MONROE AND VERNON COUNTIES, WISCONSIN

U.S. DEPARTMENT OF AGRICULTURE  
NATURAL RESOURCES CONSERVATION SERVICE

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**Projects:** Watershed Protection and Flood Prevention Program, Coon Creek Watershed and West Fork Kickapoo Watershed

**Locations and Site Numbers:**

West Fork Kickapoo\* Watershed Protection and Flood Prevention Project, Vernon County, Wisconsin:

Site Number	Local Name	Section	Township	Range	Latitude	Longitude
<b>1</b>	Jersey Valley	6	14N	3W	43.6891°	-90.7989°
<b>Mlsna</b>	Mlsna	13	14N	4W	43.7122°	-90.7524°

Coon Creek\* Watershed Protection and Flood Prevention Project, Monroe County, Wisconsin:

Site Number	Local Name	Section	Township	Range	Latitude	Longitude
<b>21</b>	Luckasson	34	15N	4W	43.7314°	-90.8420°
<b>23</b>	Bilhovde	27	15N	4W	43.7430°	-90.8416°
<b>29</b>	Korn	19	15N	4W	43.7674°	-90.8979°

\* Watershed names are abbreviated as WFK (West Fork Kickapoo) and CC (Coon Creek) for brevity in this report.

**Appropriation:** Public Law 83-566

**General Description of Problem or Deficiency:** There was a complete failure of the five dams listed in this report. Late on August 27 a storm system entered southwest Wisconsin that included Monroe and Vernon Counties. Two PL-566 watersheds located in these counties, West Fork Kickapoo and Coon Creek, had structures impacted by the storms. Intense rainfall began to fall in this area around 9:00 pm and continued until approximately 4:00 am August 28, an approximate six to seven-hour event. The projected recurrence intervals, based on NEXRAD, ranged from a low of about 300 years for WFK Mlsna to 650 years for CC-23. These rainfall amounts exceeded the principal spillway design storms reported on the as-built drawings. All three Coon Creek structures were designed with a principal spillway point rainfall of 4.6 inches. This corresponds closest to the interpreted value on the 100-Year 6-Hour Rainfall chart published in Technical Paper 40 (May 1961). WFK-1 was designed with a principal spillway point rainfall of 3.9 inches which corresponds closest to the 50-Year, 6-Hour Rainfall chart. The recorded rainfall also exceeded the freeboard storms of the Coon Creek sites. The WFK Mlsna as-built drawing does not have a hydraulic sheet so there was no basis for comparison at this site. The top of dam elevation at WFK-1 was set according to a freeboard hydrograph applying a six-hour distribution with a point rainfall of 10.9 inches. The estimated six-hour rainfall for WFK-1 was 7.3 inches. This exceeded the auxiliary spillway design rainfall of 6.2 inches. See Tables 3 and 4 of the Hydrology section for a report of recorded rainfall and design rainfall.

Four of the five dams overtopped during this event. The one exception is WFK-1 which saw significant auxiliary spillway flow only. The total depth of the auxiliary spillway channel was 8.5 feet and the estimated maximum flow depth during the event was four feet. There was auxiliary spillway flow at the other four dams as well. The breach of WFK-1 and WFK Mlsna occurred in the auxiliary spillway area. CC-21, CC-23 and CC-29 breached on the end opposite the auxiliary spillway.

Auxiliary spillway flow resulted from this event at ten other dams in these two watersheds. Of those, five overtopped. None of these ten dams breached and damage ranged from minor to significant erosion in the groins and/or auxiliary spillway. None of Wisconsin's (89) PL-566 structures had ever overtopped prior to this event, except for the WFK Pilot Klinkner structure that overtopped several times due to a design error.

According to witness reports, the breaches occurred in the early morning of August 28, perhaps between 2:00 and 3:30 am. Once the vegetative cover failed, each breach progressed very rapidly leading to a sudden release of flood water at high head. The breach process was not only rapid, but complete. Erosion of the breach channel at each site extended into the foundation bedrock and the resulting breach channel was lower than the upstream sediment pool. (see Attachment E, Investigation Survey Drawings and Photos).

These sudden failures led to a large breach wave at each site with sufficient velocity and energy to cause significant damage and flooding downstream. Physical evidence of the size of breach waves can be found at the sites and in the breach inundation areas. On several sites the breach wave was high enough to overtop the lower half of the auxiliary spillway dike. Boulders were carried hundreds of feet downstream. Debris fields consisting largely of sand and rock covered wide areas of the valley floor and for thousands of feet downstream. Downstream roads were flooded with damage to culverts and bridges. Over 2,000 feet downstream of CC-23, an unoccupied ranch-style home was moved off its foundation. Fortunately, no fatalities were reported as a result of the breaches.

**Authority:** In accordance with the National Engineering Manual (NEM), Part 504, Subpart A – Problems and Deficiencies, an investigation committee was appointed by a letter of appointment dated September 12, 2018, from Angela L. Biggs, State Conservationist. The committee membership was proposed by John Ramsden, State Conservation Engineer by letter dated September 11, 2018 and received concurrence from Noller Herbert, CED, Washington, DC, on September 11, 2018.

**Composition of Committee:**

Mark McCurdy, Asst. SCE, Des Moines, IA, Chair  
Karl Visser, Hydraulic Engineer, NDCSMC, Fort Worth, TX  
Tim Weisbrod, Geologist, WI/MN, St. Paul, MN  
Matt Blohowiak, Civil Engineer, Altoona, WI  
Mike Dreischmeier, Area Engineer, Richland Center, WI

Committee Activities:

The committee first met at the USDA Service Center in Viroqua, Wisconsin, on September 18, 2018 for a briefing from Wisconsin NRCS and Vernon County staff. The committee also used this time to plan activities for the remainder of the week. The entire committee was present along with the following individuals:

Scott Mueller, Asst. SCE, Madison, WI  
Janet Vosberg, Agricultural Engineer, Richland Center, WI  
Glen Lorenz, Civil Engineer (Retired NRCS), Viroqua, WI  
Mark Erickson, Resource Conservationist, Vernon County Land & Water Conservation Department, Viroqua, WI

Scott Mueller, Mike Dreischmeier and Mark Erickson gave a brief history of the events leading up to and including the rainfall event which began the evening of August 27 and continued into August 28. According to their account, a breach of each of the five dams in question occurred in the early morning hours of August 28 during heavy rains which induced significant flooding in Vernon and Monroe Counties.

All five dams had annual inspections and maintenance was up to date. There were no known repair issues related to the area of each dam where the breach occurred. According to witness accounts and physical data at each site, the breach process at each dam was relatively quick. While there is no onsite data to show exactly how fast each failure occurred, it is known that they all occurred sometime within the storm event which lasted about six hours.

Five other dams overtopped during this event but none of them failed. Erosion damage was observed in the downstream groins and auxiliary spillways which will require repair work, but these dams are not part of this investigation.

There were local weather stations, but Mark Erickson said these have proven unreliable in the past and that they should not be trusted. NEXRAD data is the best data available for the investigation.

All the documentation available for each dam was provided to the committee. This included as-built drawings, a more recent detailed geologic analysis of WFK-1 that was part of a repair contract, and aerial photos of each dam post failure.

The breach area was surveyed at each site. LiDAR was used to show the pre-event topography as well. Janet Vosberg agreed to take the lead on the site surveys. A total station laser scanner was used to scan the sides of the breach channels for a detailed survey of the bedrock. Janet also used GPS equipment where satellite coverage was available and where satellite coverage was not available, she used total station equipment.

Mike Dreischmeier gathered local information from news articles and eyewitness accounts to piece together a timeline of events to the degree possible.

The committee visited all five sites on September 18<sup>th</sup> and 19<sup>th</sup>. One PL-566 site, WFK 3, and one pilot watershed site, Klinkner Pilot, that were in proximity to WFK-1 and MIsna, were included on the tour. WFK 3 overtopped but did not fail. The sites were visited in the following order:

September 18:

WFK-1 (Jersey Valley)

WFK MIsna (Pilot)

WFK 3 (not part of the investigation)

Klinkner Pilot (not part of the investigation)

September 19:

CC-21

CC-29

CC-23

On September 20, the committee met at the Viroqua Field Office to review the as-built drawings and available geology. It was agreed that the two most important areas to focus on were geology and hydrology. And the focus of the investigation should be:

- A. Were design criteria met in the geologic investigations and the hydrologic design?
- B. What does the available information, the as-built drawings, the actual rainfall information, and new geologic data reveal about the cause of failure at each site?

**Investigation:**

The following is a timeline of events leading up to and following the observation of the problem:

- Construction completion dates:

Table 1 shows the construction drawing approval and construction completed dates from the as-built drawings:

Table 1

Site	Date Approved	Date Completed
WFK Mlsna	May 15, 1954	August 21, 1956
CC-21	September 1, 1961	June 21, 1963
CC-23	June 1, 1959	July 12, 1969
CC-29	June 1, 1959	July 12, 1969
WFK-1	February 25, 1965	July 1971

- Active springs in both abutments at West Fork Kickapoo 1 (Jersey Valley) have caused concern since it was constructed and filled in the mid 1960's. The gate was closed on the structure in August 1969. The pool filled to elevation 1063.5 by November 1969 where it hovered for several years before finally reaching its design elevation of 1076.5 in March 1972. In October 1970, a drainage system consisting of two right abutment drains was installed on the right downstream side of the dam. The left upstream abutment blanket installed at the time of construction was found sloughing into the lake in the late 1990's and was repaired in 2000. In 2004 the right abutment drain installed in 1970, which was no longer functioning, was replaced. In March 2005, there was a manure runoff event that caused a fish kill in the lake. Effluent was observed in the discharge of seepage water in the downstream left abutment indicating a direct flow path through the left abutment from the lake. In April 2005, the Wisconsin DNR required that the seepage through the left abutment be addressed. A watershed rehabilitation plan, based on a High Hazard potential of the dam, was completed by NRCS in January 2008. The County chose not to pursue any alternatives outlined in the rehabilitation plan. Instead, the County elected to implement an alternative that met State of Wisconsin High Hazard criteria using a private engineer. AECOM designed a grout curtain system and it was installed in 2009/2010. After the grouting, the gate was closed and the lake refilled.
- Sponsors have been completing inspection on an annual basis. At the time of failure all Operation and Maintenance was up to date and the sites were reported to be in good condition. None of the inspection reports indicate serious problems. One maintenance item reported on some of the sites was the encroachment of brush into the groins. This brush was cleared out of the groins of these dams before the breach events. Since the flood event, lack of adequate vegetative cover on the downstream groins has been

identified as a material weakness. At the time of the inspections this appeared to be only a minor issue. The 2015 dam Safety Inspection Report of WFK-1, completed by Ayres Associates, acknowledged seepage from the toe drains and bedrock from the abutment on the right side downstream of the dam. Note that the report stated that the seepage flow rate was consistent with past observations made in 2011 and 2013, and that “overall seepage through the embankment had decreased following construction of the grout curtain.” Copies of recent inspection reports are available with the investigation support data. Photos from those reports show the vegetative stand was generally excellent. The vegetation on WFK Mlsna was fair to good. These photos may be found in Attachment D, O&M Inspection Photos.

- Local Rainfall and Breach Accounts: August 27-August 28, 2018

A neighbor downstream of Coon Creek 21, Rick Vaught, reported on the events of early Monday morning, August 28<sup>th</sup>:

*“We were woken when the shed outside our bedroom window toppled, that was at 02:30. The breach of the dam happened about 02:15. We saw the highest levels, which was under the corner of our cabin, about 02:30. We still had power on at that time but it went off soon after. The power from our house is underground to the Helgerson’s house. The (water) level went down approximately 6 feet by 03:30 so I believe that was the time of the surge. The power faulted when the downstream poles were taken out.*

*The amount of rain would be a guess, but I did have two large outdoor-type cooking pots outside on the grass. Both were full, with the bigger being 14” in depth.*

*The seven inches on the second storm (Tuesday afternoon August 28) was from a rain gauge that did not overflow.”*

Jeff Mlsna of the Vernon County Town of Clinton Volunteer Fire Department was quoted in the Crawford County Independent (Thursday September 6, 2018):

*“We’d been out all evening (August 27) responding to emergency calls and repairing driveways. We were just heading back in about 1:30 am (August 28) when it started to pour again, and didn’t let up until about 4:30 am. All those driveways we’d repaired were all washed out again by then.”*

Mike Dreischmeier concludes:

*All the breaches in the two counties (Vernon and Monroe) happened well before 6 a.m., or first light. No one reported seeing the breaches occur. I think that it is likely that all (five dam breaches) occurred between 2am and 3:30am (Tuesday, August 28, 2018).*

From these reports, the critical precipitation that caused the five dams in Monroe and Vernon Counties to breach is the rain that fell on the evening of Monday, August 27<sup>th</sup> and early morning Tuesday, August 28<sup>th</sup>.

- Rainfall and Breach Chronology: August 27-September 3, 2018

The rainfall event that breached the five dams in this report occurred overnight during the evening of Monday, August 27 through early morning Tuesday, August 28, 2018. Mike Dreischmeier, Richland Center NRCS engineer, surveyed the damaged sites in Vernon County (Jersey Valley and Mlsna) during the day August 28<sup>th</sup>. The Mlsna flood pool was completely drained and was below the principal spillway inlet. The Jersey Valley dam had breached and was draining below the level of the principal spillway. Dan Gunderson, Sparta NRCS Civil Engineering Technician and Bob Micheel, Monroe County Conservationist, visited the three Coon Creek breached dams (21, 23, 29) on August 28<sup>th</sup>. All three of the Coon Creek dams had breached and drained below the principal spillway inlet elevation.

Additional rain fell during the afternoon of August 28<sup>th</sup>. This rain put additional flow through the breached embankments, slightly widening the breach openings.

A storm on Labor Day hit one week later – Monday, September 3, 2018. Again, more storm runoff widened the existing breach openings even more.

- On September 11, 2018, the State Conservation Engineer, John Ramsden, made a formal request to Noller Herbert, Director of Conservation Engineering Division at NHQ, for concurrence of the investigation committee as listed above. Concurrence was granted the same day.

### Site Inspections

#### West Fork Kickapoo 1:

Late in the morning of September 18 the committee traveled to WFK-1. A small construction crew was on site. The Sponsors had hired a contractor to clean up the breach area and shape the embankment side of the breach channel to a stable slope (see Attachment B, Photos 1 and 2).

The committee was informed that this site did not overtop. The runoff event caused auxiliary spillway flow about four feet deep in the level section (see Attachment B, Photo A1). The auxiliary spillway is located in natural ground on the right end of the dam (see Attachment C, WFK-1 As-Builts, Sheets 5A and 6 of 21). It was not uniformly graded to the flood plain. Rather, it was graded out to natural ground on the right abutment above the flood plain leaving a steeper flow path downstream of the auxiliary spillway outlet. This is not shown on the auxiliary spillway profile in the construction drawings. But it was observed from the LiDAR data and can be seen on Sheet 5 (Profile – C/L Auxiliary Spillway) near station 5+60, in Attachment E.

The breach of the dam developed in the auxiliary spillway and eroded down to bedrock in the foundation. The embankment from approximately station 7+10 and right is completely gone. The embankment left of this station remained intact with no visible signs of damage. The overall condition of the remaining dam appeared to be in good condition and well maintained. The committee spent most of its time inspecting the foundation area and abutment of the breach channel (see Attachment B for site photos and Attachment E for the investigation survey drawings).

Pressure grouting in 2009 and 2010 formed a curtain wall across the entire valley from abutment to abutment (see Attachment B, Photo 13). The committee observed evidence of the pressure grouting in the exposed foundation and abutment bedrock (see Attachment B, Photos 4 and 5). Scott Mueller explained to the committee that sustained seepage through the left abutment downstream of the dam had been observed prior to the grouting. After completion of the grouting, seepage from the left abutment downstream stopped.

Remnants of the grout holes were observed in the breach channel bottom and abutment slope (see Attachment B, Photos 6 and 7). Some joints were filled with grout, but not all. The grout curtain was not as effective in cutting off seepage on the right side as it was on the left side. But, as noted in the O&M inspection report summary, observed seepage from the drains on the right side decreased following construction of the grout curtain. The committee observed that the grouted bedrock did not collapse as far into the abutment and retained more of its structure at the exposed surface (see Attachment B, Photo 8).

Test holes from a geologic investigation completed in 2005 showed bedrock from 11 feet below the outside edge of the auxiliary spillway channel, to 32 feet at the inside edge. The survey and geology show that the breach channel eroded down to the bedrock. As the breach progressed, the erosion moved laterally into the embankment (see Breach Zone on Sheet 3 in Attachment E).

The size and volume of eroded material, and the extent to which it was carried downstream gives evidence of how rapid and complete the breach was. The downstream valley in view from the top of dam is mostly covered with material from the breach (see Attachment B, Photo 9). The breach transported material over 1,000 feet downstream. A post-breach survey of the breach channel estimated eroded material volume at 56,000 cubic yards.

West Fork Kickapoo Mlsna:

The committee traveled on to WFK Mlsna in the afternoon. This site had been grazed and appeared to be slightly over-grazed (see Attachment B, Photo 14). However, there doesn't appear to have been extensive damage by the cattle, though there is a worn path on the upstream berm of the dam.

There was overtopping flow at this site, but outside of the breach there was no significant damage to the embankment. A scour hole did develop at the downstream toe of the left groin and some of the vegetation was starting to erode from the lower slope of the groin. The scour hole was about three feet deep and had eroded into bedrock (see Attachment B, Photos 15 and 16).

The breach formed in the right abutment of this site in the location of the auxiliary spillway. As with WFK-1, the auxiliary spillway was constructed in natural ground (see Attachment C, WFK Mlsna As-Builts, Sheets 4 of 16 and 2 of 15).

The breach on this site was a complete breach eroding down to bedrock and leaving a headcut into the upstream sediment pool (see Attachment B, Photo 17). The breach also eroded into the bedrock in the abutment (see Attachment B, Photos 18, 19, 20, 21, 22 and 23). The breach appears to have moved laterally into the embankment once it eroded down to the case-hardened rock (see Attachment E, WFK Mlsna, Sheet 2, Cross Section).

Evidence of a sudden breach on this site is similar to WFK-1 with an extensive debris field downstream consisting of fragments of bedrock material (see Attachment B, Photo 24). There was also extensive damage to a town road in the breach inundation area (see Attachment B, Photo 25). A post-breach survey of the breach channel estimated the of eroded material volume at 18,000 cubic yards.

#### West Fork Kickapoo 3:

The committee then traveled to WFK 3. While this site was not part of the investigation, it was an opportunity for the committee to see a site that overtopped without breaching. Except for a shallow scour hole at the very end of the auxiliary spillway (see Attachment B, Photo 26 and Photo 27), there was remarkably little damage (see Attachment B, Photo 28). Note that a field fence had been constructed immediately downstream of the end of the auxiliary spillway and that there was a defined drop in elevation from the auxiliary spillway side to the field side of the fence (see Attachment B, Photo 27). This sharp change in gradient appears to be the impetus for developing the scour hole. A much smaller area of scour was noted at the downstream toe of the right groin; a function of the overtopping flow (see Attachment B, Photo 30).

No damage to the downstream side of the embankment was observed. The vegetative cover appeared to be intact showing no sign that vegetative shear stress was exceeded (see Attachment B, Photo 29). The groin on the left end of the embankment, however, was damaged significantly (see Attachment B, Photos 31, 32 and 33). A fence had been constructed down the groin and there is a defined change in vegetative cover with grass on the dam side and timber on the opposite side of the fence. The vegetation evidently failed on the timber side first, on the lower slope of the groin, and then migrated upstream crossing under the fence and onto the dam side of the groin. The as-built plans show that sod was placed in all the groins with a channel shape similar to a parabola (see Attachment C, WFK 3 Sheet 18 of 18). While the right groin has been maintained, the left groin has not. The fence and encroachment of the timber have changed the flow characteristics and erosion resistance of the left groin. This observation holds significant implications for the stability of the vegetation in the groins of all the dams in this area.

#### Klinkner Pilot:

The committee visited Klinkner Pilot at the end of the day on September 18. This site is not part of the investigation and does not offer any insights relevant to the investigation.

#### Coon Creek 21:

On September 19, the committee traveled to Coon Creek 21, the largest of the Coon Creek sites in terms of drainage area. The vegetative cover on the dam is uniform and vigorous (see Attachment B, Photo 35). There were fresh signs of cattle on the dam, but it did not appear grazed so it's possible the breach knocked down a fence which allowed cattle on the site. Overall maintenance appeared to be in good condition.

The dam overtopped but the auxiliary spillway did not fail (see Attachment B, Photos 36 and 37). The auxiliary spillway vegetation did fail in several areas, but a breach of the auxiliary spillway did not appear imminent. The largest area of erosion was on the right side working its way up from the outlet end, about 1/3 of the channel length (see Attachment B, Photos 34 and 38). The auxiliary spillway dike also incurred some erosion, but this appears to have been

caused by tree branches that scraped away the sod when the dike was overtopped during the breach of the dam.

The breach at this site occurred at the left end of the dam. The as-built plans show the dam was constructed with a crown of 0.9 ft. The LiDAR data confirms that there was little settlement as the crown did not settle to the design settled top of dam (see Attachment E, Sheet 13). Therefore, the dam overtopped at the left end before the center of the embankment. Comparing the plan view of combined LiDAR and post-breach contours, it is evident that the breach eroded down through embankment material and then straight into bedrock (see Attachment E, Sheet 12, Plan View, and Sheet 13, Cross Section). After eroding through bedrock under the left end of the embankment, the breach then eroded left into the abutment bedrock (see Attachment B, Photo 34). However, the breach did not move laterally into the deepest part of the embankment, where the principal spillway is located. The erodible nature of the underlying non-cohesive material, and the seepage forces within the valley relief fractures of the bedrock provided less resistance to erosion than the cohesive surface material in the embankment (see Attachment B, Photos 39 and 40). The detailed failure mechanisms are covered in detail in the Geology section.

The WFK 3 left abutment provides clues why the breach eroded the bedrock rather than the embankment. The left downstream groin of WFK 3 eroded the bedrock where there was little vegetative cover. Then the headcut migrated up the groin into the soft, jointed bedrock. Like CC-21, WFK 3 had a crown with the ends sloping down to the settled top of dam (see Attachment C, CC-21, Sheet 1 of 3 and WFK 3, Sheet 1 of 4). The center of the embankment on both sites overtopped without damage. WFK 3 was probably in the early stages of a failure similar to CC-21 where the failure started in the left downstream groin. Note that the right groin, which also saw overtopping flow, did not incur any erosion (see Attachment B, Photo 41).

As with the other sites, it was a complete breach that cut down below the sediment pool behind the dam. Evidence of a sudden breach through the abutment bedrock can be seen in the extensive debris field downstream which consists of fragments of bedrock material (see Attachment B, Photo 42). A post-breach survey of the breach estimated the eroded material volume at 28,000 cubic yards.

#### Coon Creek 29:

The committee traveled next to CC-29 in the afternoon. The vegetative cover on this site was excellent and overall maintenance appeared to be good (see Attachment B, Photo 43). The dam overtopped on this site and there was significant damage to the auxiliary spillway (see Attachment B, Photo 44). The auxiliary spillway is a ramped spillway with an 80 ft. wide bottom. The extent of the erosion channel, at the time of the breach, extended well over half the bottom width with a three to four-foot-deep headcut. The headcut advanced to within about 30 feet of the level section of the auxiliary spillway (see Attachment B, Photos 45 through 47). The extensive damage to the auxiliary spillway suggests that it was close to breaching at the time the breach occurred at the left end of the dam.

Coon Creek 29 was crowned during construction in similar fashion to CC-21. The crown was more pronounced on this site with a design difference between constructed height and settled height of 2.5 feet. The LiDAR data shows that the crown may have settled some, but was still very pronounced (see Attachment E, CC-29, Sheet 22, Cross Section). The top of dam crown shunted the initial overtopping flows to the embankment ends and down the groins. Only once

the ends were overtopped would the higher center of the embankment overtop, just as CC-21. The left end of CC-29 must have breached first, thereby preventing the right side from breaching (see Attachment B, Photos 48 through 52).

The breach extended below the sediment pool upstream (see Attachment B, Photo 53). The extent of erosion into the abutment was very significant (see Attachment E, CC-29, Sheet 21, Plan View and Sheet 22, Cross Section and Attachment B, Photo 44). The debris field in the breach inundation area of this site was extensive (see Attachment B, Photo 54). A post-breach survey estimated the eroded material volume at 17,000 cubic yards. The failure conditions at CC-29 are similar to those at CC-21. Refer to the narrative on CC-21 and the Geology section for more detail.

#### Coon Creek 23:

The committee traveled last to CC-23. The vegetative cover on the dam appears to be in excellent condition (see Attachment B, Photos 55 and 56). Overall maintenance appeared to be in good condition. The breach wave shifted an unoccupied ranch house off its foundation and washed out a small bridge on a town road just downstream of the house. The house is located approximately 0.4 mile downstream of the dam (see Attachment B, Photos 57 through 59).

The dam overtopped on this site. The auxiliary spillway, a ramp spillway on the right end of the dam with a 30 ft. wide bottom, did not fail but it sustained significant erosion damage (see Attachment B, Photos 60 through 63). Two large headcuts formed in the lower 2/3 of the spillway. The depth of erosion was well into the bedrock. A smaller headcut, about 18 inches deep, advanced up the exit channel and close to the control section.

Coon Creek 23 was designed with a crowned top of dam, about 1.75 feet. However, the LiDAR data shows that the crown may have settled out as the top of dam appears to be nearly level (see Attachment E, CC-23, Sheet 17, Cross Section). If so, the overtopping flow would have occurred across the entire length of dam at approximately the same depth. There was no observed erosion on the downstream side of the main embankment (see Attachment B, Photos 55 and 64). While flow may have been evenly distributed over the dam, the breach of the left groin appears to have been similar to the failures at CC-21 and CC-29. There was no failure of vegetation in the right groin which is all compacted earthfall (see Attachment B, Photo 65)

Generally, the valleys in which dams are constructed have sloping abutments. As a result, the dam embankment is wider at the top and is narrower down near the stream channel.

Overtopping flow starts at the top of the dam, which is the widest part of the abutment. As the overtopping flow descends the backslope of the dam, the valley and embankment narrow and effectively squeeze the overtopping flow together. The groins are the intersection of the embankment backslope and valley slopes, where the squeezing occurs. Therefore, the unit discharge in the groin areas is greater than the unit discharge of the overtopping flows in the center of the embankment. In addition, the groin flow may also receive surface runoff from the valley slopes downstream of the embankment.

The breach eroded below the sediment pool upstream (see Attachment B, Photo 66). The erosion significantly extended into the abutment (see Attachment E, CC-23, Sheet 16, Plan View and Sheet 17, Cross Section, and Photos 67 through 69). A headcut approximately six feet deep eroded the bedrock at the bottom of the breach channel (see Attachment B, Photos 71 and 72). The exposed bedrock at this site revealed the widest vertical valley relief fractures

of any of the sites (see Attachment B, Photos 71 through 73). The debris field in the breach inundation area of this site was extensive (see Attachment B, Photo 70). A post-breach survey estimates the eroded material volume at 15,000 cubic yards. The failure conditions at CC-23 are similar to CC-21. Refer to the narrative on CC-21 and the Geology section for more detail.

### Site Surveys

The committee determined that site surveys were needed to accurately define the extent of the breaches. Surveys also provided estimated high-water elevations and volume estimates of the material removed by the breaches. The following is a summary of the engineering surveys conducted on all five sites.

Table 2 - Summary of Survey Parameters

Structure	Date(s) surveyed	Surveyed By*	Vertical Datum Used On-site	Horizontal Datum Used On-site	Benchmark	As-built Elev	NAVD 88 Elevation	Difference AsBuilt - NAVD88
WFK #1 (Jersey Valley)	9/21/18 and 10/17/18	Janet Vosberg Stephanie Schultz Tim Weisbrod Steve Grady (Trimble)	As-built	Vernon County, USFT, Geoid 12A	Crest of the SE corner of the principal spillway concrete inlet riser	1076.5	1075.85	0.65
Mlsna	10/4/2018	Janet Vosberg Stephanie Schultz	As-built	Vernon County, USFT, Geoid 12A	Crest of the SW corner of the principal spillway concrete inlet riser on top of the metal rack bolted to the inlet	1145.0	1135.25	9.75
CC #21	10/12/2018	Janet Vosberg Tim Weisbrod	NAVD 88 obtained with GPS	Monroe County, USFT, Geoid 12A	Crest of the NW corner of the principal spillway concrete inlet riser	983.2	1083.08	-99.88
CC #23	10/16/2018	Janet Vosberg John Vosberg	NAVD 88 obtained with GPS	Monroe County, USFT, Geoid 12A	Crest of the SE corner of the principal spillway concrete inlet riser	1103.0	1103.0	0
CC #29	10/11/2018	Janet Vosberg Stephanie Schultz	As-built	Arbitrary (10,000, 10,000)	Crest of the SW corner of the principal spillway concrete inlet riser	1064.0	N/A	Based on LIDAR contour data, the difference is negligible

\*All sites surveyed by NRCS staff

Engineering surveys were completed during September and October 2018. Minor stabilization work had been completed on WFK-1 prior to the survey so the site could be safely accessed. Therefore, the WFK-1 drawings in Attachment E reflect the shape of the end of the embankment after grading instead of the shape immediately after the breach. No repair work was done on any of the structures prior to survey.

The outer slope and channel bottom of each breach at the five structures consists of exposed bedrock within the abutments. The purpose of the surveys was to develop existing topography for each structure, determine high water elevations if possible, and provide detailed horizontal and vertical information of the exposed bedrock for geologic analysis. Total Station Laser Scanning (TSLs) technology was utilized to capture the bedrock detail (see Attachment E, Investigation Survey Drawings and Photos).

Tim Weisbrod conducted a subsurface investigation of some of the sites that had high water during the same rain event using Electrical Resistivity Imaging (ERI). The purpose of the investigation was to determine the extent of jointing in the bedrock and to determine if seepage forces could have played a significant role in the failures. One of the dams investigated was summarized in the text found below.

## As-built Design Elements

WFK MIsna and WFK-1 were designed as Low hazard dams with a planned service life of 50 years. WFK MIsna has exceeded its planned service life and WFK-1 will reach its planned service life in 2019. Both are currently classified as High hazard. These are rolled earthfill dams with zoned embankments (see Attachment C, WFK MIsna, Sheet 3 of 15 and WFK-1, Sheet 1 of 4). WFK MIsna has a core and a single outer shell. WFK-1 has a core of fine-grained material up to the auxiliary spillway crest, an inner shell of ML and SM material, a middle shell of silty and clean gravels and small rock, and an outer shell of floodplain borrow with some rock greater than 9 inches. According to the as-built plan, the core trench extends to the bedrock in the foundation of both sites.

The auxiliary spillways on both sites were designed and constructed into natural ground.

There is no record of the hydrology used in the design of WFK MIsna. WFK-1 was designed with "B" storm distribution and six-hour storm durations. Details of the design rainfalls are included in the Hydrology section of this report.

The design of WFK-1 did attempt to address seepage through the foundation. A request was made to do pressure grouting. This request was denied as documented in the WFK1, Geologic Investigation Report dated December 2005. A six-foot thick clay liner was installed in the bottom of the pool and extended up the abutments to elevation 1089.0 as part of the original construction. The blanket starts at the upstream toe of the dam and ends approximately 300 feet upstream. However, seepage developed through the bedrock over time. As a result, a spring developed downstream of the dam on the left side. In 2009, construction of a pressure grouted curtain wall began. The curtain wall extended approximately 25 feet into the bedrock across the valley floor and deeper into the abutments (see Attachment B, Photo 13). In January 2010, the contractor, AECOM, reported that the grout takes under the dam and right abutment were larger than anticipated. They surmised that the large grout takes during backfilling of casing through the embankment soils suggested possible development of voids at the embankment/bedrock contact. However, they concluded that the grout program was working as the outer rows were confining and there was grout refusal in the middle row. This conclusion was supported by the observation that the spring coming from the left abutment downstream had stopped. Piezometers were installed to allow monitoring of the phreatic line and piezometer pressures during and after re-filling of the reservoir.

Coon Creek Sites 21, 23 and 29 were all designed as Low hazard dams with a planned service life of 50 years. The hazard classification of all three sites is still Low hazard and all have exceeded their planned service life. These are rolled earthfill dams with zoned embankments. The plans specify Class B2 or B3 earthfill in the core but don't define those classifications. It is believed these were defined in the construction specifications, which are not available. The textural classes of the soils to be used for earthfill were not defined on the plans either. Notes were used in several places indicating "Core of select material" (see Attachment C, CC-29, Sheet 19 of 36), or "Use most impervious material in core" (see Attachment C, CC-21, Drawing 3-E-46088-C, Sheet 1).

The as-built plan copies are of poor quality, but no test holes extend to bedrock in the foundation area on centerline of any of the three dams. Erosion in the breach channels removed soil in the top of the foundation and abutment areas leaving bedrock exposed on all three sites. The plans don't indicate whether the bottoms of the core trenches contacted bedrock or not.

All three sites were designed with foundation drains in the downstream zone of the embankments, including a blanket drain at each end of the foundation drain. These blanket drains were laid on the abutments and extended above the valley floor.

The auxiliary spillways were designed and constructed as ramp spillways, though there may have been a minor amount of cut in natural ground in sections of the outside channel bottom of CC-21 and CC-23. The as-built plans show a minor cut slope in several sections along the length of each auxiliary spillway. Erosion of the auxiliary spillways of CC-21 and CC-23 was more pronounced on the outside of the channel bottom, though the erosion of CC-21 was relatively minor. On CC-23 the erosion was severe as it moved down through the soil profile into the bedrock. The two pronounced scour holes also moved laterally into the bedrock, indicating bedrock material weakness.

Hydraulic sheets for the three as-builts indicate the hydrologic designs used six-hour duration storms. Storm Distribution Curve B was clearly indicated on CC-21 but not the other two sites. It appears to be a reasonable assumption that they were also designed with the same distribution as this would have been consistent with other designs of the period. Details of the design rainfalls are included in the Hydrology section of this report.

## Hydrology

The following is a summary of the hydrologic analysis completed for this report. The complete report may be found in Attachment F, Hydrology, 'Detailed Hydrology Report'.

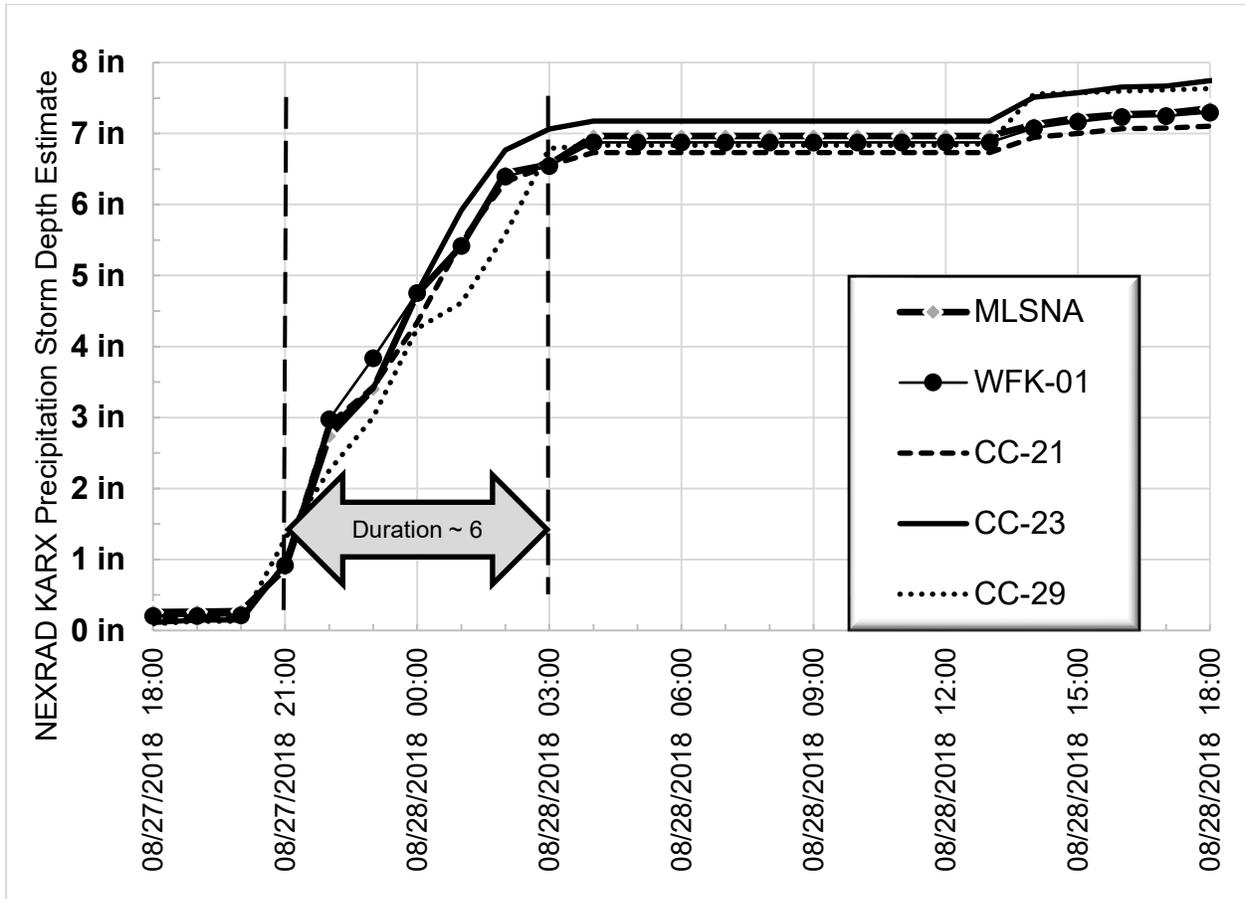


Figure 1 Storm Rainfall from NEXRAD Data KARX-LaCrosse, WI

Table 3 shows the expected return frequency for two different reported amounts, the National Weather Service (NWS) QPE 24-hour estimate and the NWS NEXRAD radar estimate. It is not clear what time is covered in the NWS QPE 24-hour storm rainfall—it is unknown if the rainfall on the evening of August 27 is included or if rainfall on the afternoon is included. The critical rainfall that caused the breach of the five dams occurred on the evening of August 27 and early morning of August 28. The breaches were confirmed by NRCS and County personnel during daytime site visits on August 28. Additional rain fell August 28 after the site visits.

Table 3 - Expected Return Frequency of Storm Rainfall

County	Watershed Name	Local Name	Ending Date of Storm	QPE* 24-hr		NEXRAD 6-hr	
				QPE* Storm Rainfall (Inches)	Return Freq (Years)	6-hr Storm Rainfall	Return Freq (Years)
Monroe	Coon Creek 21	Luckasson	2018-08-28	10.4 in	500-yr	7.1 in	400-yr
Monroe	Coon Creek 23	Bilhovde	2018-08-28	11.0 in	600-yr	7.8 in	400-yr
Monroe	Coon Creek 29	Korn	2018-08-28	10.4 in	500-yr	7.6 in	400-yr
Vernon	West Fork Kickapoo 1	Jersey Valley	2018-08-28	9.4 in	300-yr	7.3 in	400-yr
Vernon	Mlsna	Mlsna	2018-08-28	9.4 in	300-yr	7.4 in	400-yr

QPE\* - quantified precipitation estimate

Regardless of which data is used, the rainfall return frequency is in the 300-year to 600-year interval.

Table 4 lists the freeboard design area rainfall information gleaned from the original as-built drawings. Sites are listed from largest to smallest drainage area. The design rainfall depth was not available for WFK Mlsna.

Table 4 - Freeboard Design Area Rainfall Depth and Duration (sorted by drainage area)

Dam	Local Name	County	Freeboard Design Area Rainfall Depth	Freeboard Design Rainfall Duration	Drainage Area
West Fork Kickapoo 1	Jersey Valley	Vernon	10.9 inches	6 hours	8.06 sq mi
Coon Creek 21	Luckasson	Monroe	5.4 inches	6 hours	3.16 sq mi
Coon Creek 29	Korn	Monroe	5.37 inches	6 hours	2.88 sq mi
West Fork Kickapoo	Mlsna	Vernon	NA	NA	1.48 sq mi
Coon Creek 23	Bilhovde	Monroe	5.50 inches	6 hours	1.42 sq mi

## Geology

General Geology of the sites:

The geology at these sites is dominated by Cambrian and Ordovician aged bedrock deposited in the Hollandale Embayment. The rocks of the Hollandale Embayment have weathered over time creating deeply incised valleys and a series of plateaus, with the most erosion resistant rock forming the top of the plateau and the weaker, more erosive units forming steep hillsides.

The sites impacted by this storm event are in the Prairie du Chien plateau (Figure 2). The youngest unit on these sites is the Ordovician aged Oneota Formation- the stratigraphically lowest formation in the Prairie du Chien Group. The Oneota formation is a chemically weatherable karst forming unit that varies from a dolostone to a silty dolostone, with some weakly cemented sand lenses present. Since the Oneota has greater structural strength than the surrounding units, it forms the top of the plateau in this area. As a result, the Oneota is isolated to the hills above the sites, although large boulders of Oneota dolomite may have been transported downslope to the sites as colluvium (Figures 2 & 3).

There are three Cambrian aged Formations found within the effective stratigraphy of these sites

– the Jordan, the St. Lawrence, and the Lone Rock Formation of the Tunnel City Group, also known as the Franconia Formation (Figures 2 & 3). The primary bedrock unit forming the steep wooded hillsides on the abutments of the structures is the Jordan Sandstone. It is comprised of two distinct lithofacies: the Van Osier Member, which is the upper one, is fine to coarse grained quartz sandstone; and the Norwalk Member, which is the lower one, is a very fine-grained feldspathic sandstone (Mossler, 2008). Based on other project sites in the area and outcrops on site, the Jordan Sandstone is often soft, poorly cemented, and friable, especially near the surface where it is easily weathered into a soft sand (typically a non-plastic sand).

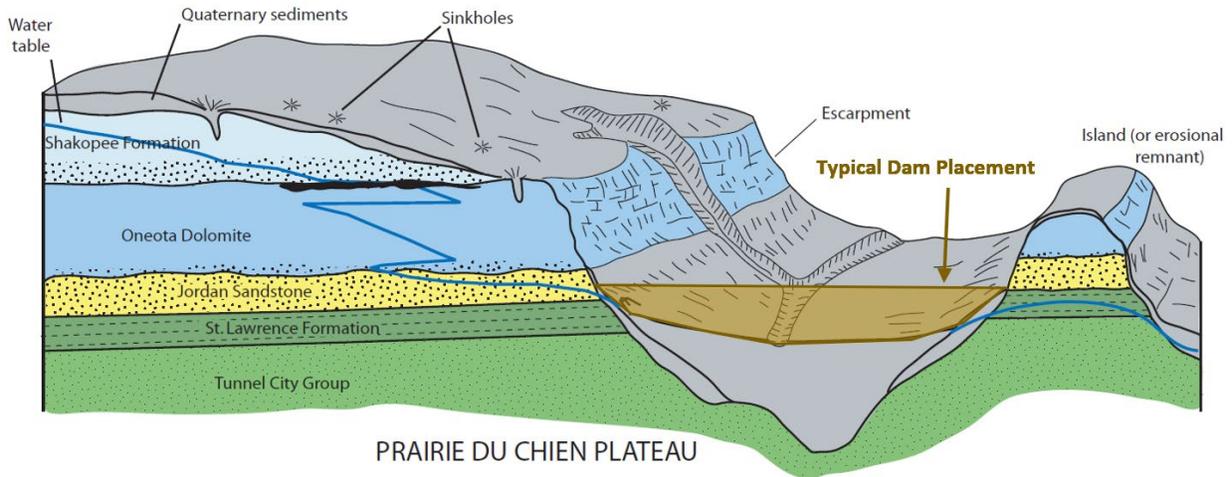


Figure 2- Block Diagram showing the general geologic setting of the Prairie Du Chien Plateau and the PL-566 structures breached in the August 2018 flood event. The typical dam placement is shown in brown. Note that this is just for illustrative purposes. The dam could be slightly higher or lower depending on how high up the valley it is. Also note that even without the dams in place the regional water table naturally discharges out of the hill at the intersection of the St. Lawrence (on top of Black Earth Member aquiclude) and the Jordan formation and discharges into the colluvium and into the alluvium below the dams.

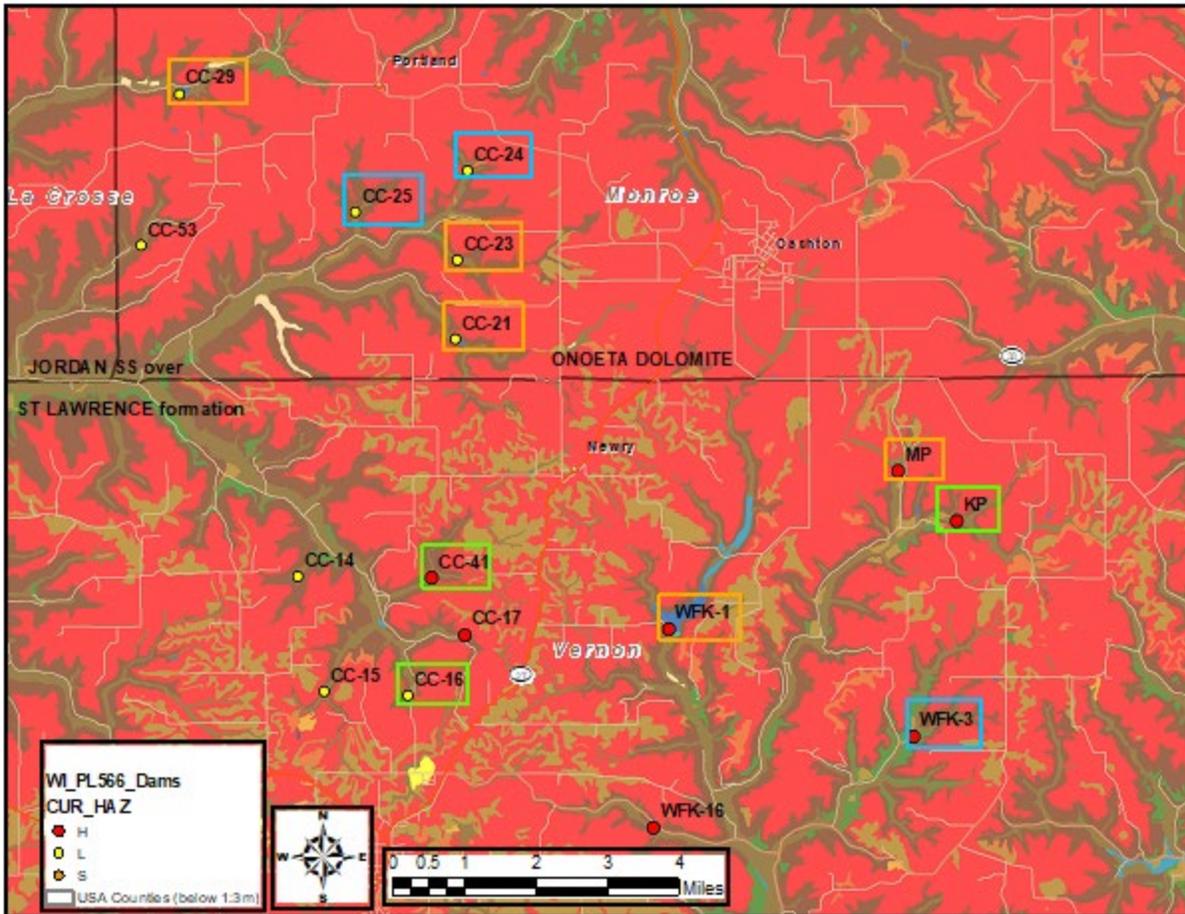


Figure 3- Map of PL-566 Structures in the location with the highest rainfall totals during the August 28, 2018 rain event. This figure shows the five dams that breached (orange box), three sites that overtopped but did not breach (blue box), and three sites that had flow in the auxiliary spillway (green box). The upland areas draining to the sites are in the Oneota Dolomite (red areas). It has been estimated that 86 of the 89 PL-566 structures in Wisconsin are in the Jordan, St. Lawrence, or Tunnel City Formations (brown areas). MP is MIsna Pilot and KP is Klinkner Pilot.

The field estimated material strength parameters of the Jordan Sandstone typically vary from 0.45- 4.5 Material Strength (Ms) making it a very soft to soft rock (NEH 628 Ch. 52, Table 52-4). When significant fracturing occurs in this unit, or pressurized water has access to the weaker cemented portions causing internal erosion of the rock unit, the rock mass stability and resulting Kh value will be lower. The Jordan Sandstone is considered an aquifer and usually has a regional water table with adequate quantity for residential wells.

Just below the Jordan is the St. Lawrence formation. The St. Lawrence has two distinct lithofacies. The Lodi Member is the upper portion and is typically a light gray to yellowish gray thinly bedded siltstone. Due to the thin bedding and large number of fractures, the Lodi Member tends to be highly permeable. The field estimated material strength parameters of the Lodi Siltstone typically vary from 1-12.5 Ms making it a soft to moderately soft rock (NEH 628 Ch. 52 Table 52-4). The thin, relatively flat lying bedding and corresponding small block size make this unit prone to plucking erosion in a spillway. When combined with the high permeability, the result is a low rock mass stability.

The Black Earth Member is the lower portion of the St. Lawrence Formation. It is generally a light olive-gray to yellowish-gray, thickly bedded glauconitic dolostone. The field estimated material strength parameters of the Black Earth Member typically vary from 12.5-50 Ms making it a moderately hard rock (NEH 628 Ch. 52 Table 52-4). Not only is the Black Earth member the hardest rock unit in the effective stratigraphy of the dams (as shown by WFK1 erosion stopping in this unit) but it also has the lowest permeability and porosity making it a partial aquiclude that typically has springs or seeps associated with the top of the formation.

The Tunnel City Group is primarily composed of a light olive-gray to greenish-gray very fine-grained glauconitic sandstone (Mossler, 2008). The field estimated material strength parameters of the Tunnel City Sandstone typically vary from 4.5-12.5 Ms making it a soft rock (NEH 628 Ch. 52, Table 52-4). The Tunnel City is considered an aquifer and usually has a regional water table with adequate quantity for residential wells.

#### Interpreted Geologic Conditions:

The goal of this section of the report is to analyze the data collected on site using a geologic framework to get a better understanding of the potential failure modes that may have been present. The geologic conditions have been interpreted based on guidance from NEH Part 628 Ch. 52- Field Procedures Guide for the Headcut Erodibility Index and the FEMA document named P-1032: Evaluation and Monitoring of Internal Erosion.

Material Strength, Permeability, and Regional Groundwater Flow are important geologic conditions that likely caused adverse effects at the dams. As shown in Figure 4, all five PL-566 structures that have breached in Wisconsin were either in the soft, weakly cemented Jordan Sandstone or the high permeability Lodi Member of the St. Lawrence formation. Experience in this area shows that the Jordan Sandstone weathers to a loose highly permeable non-plastic sand (SM or SP). The previously mentioned materials were expected to be present in the breached zone based on observations on site and as-built interpretations summarized on the drawings in Attachment E.

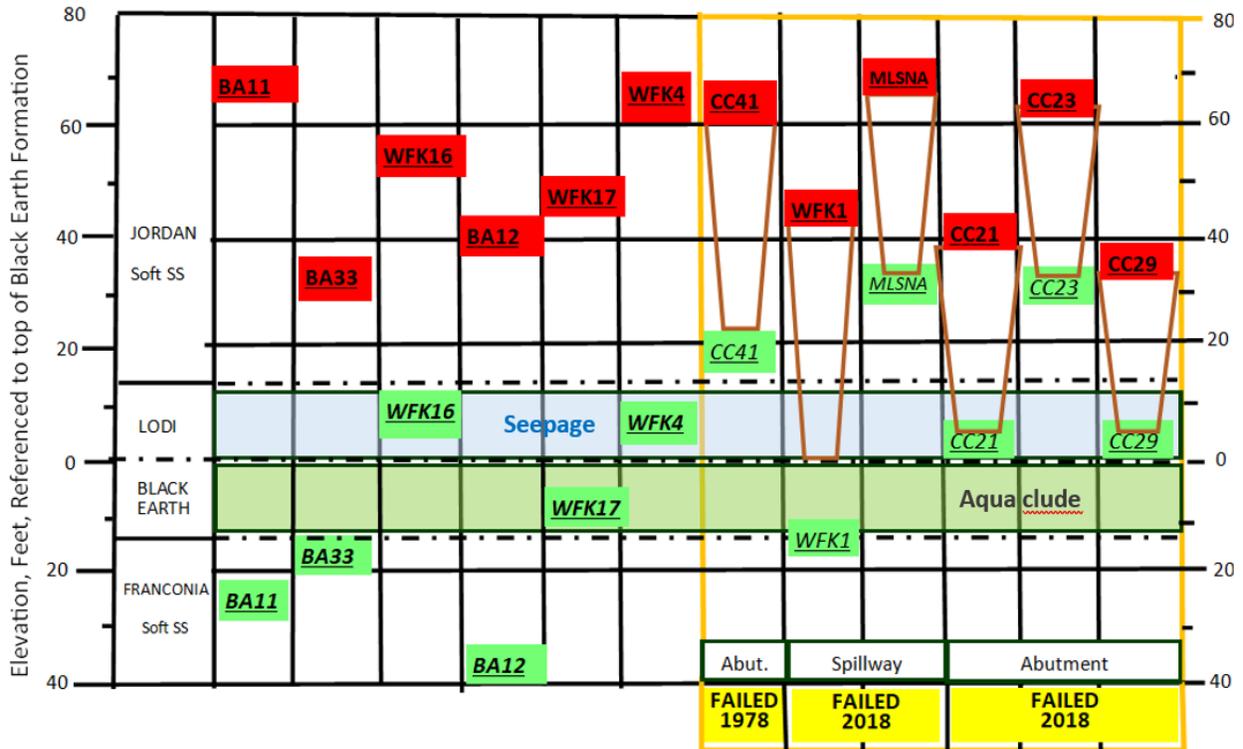


Figure 4- Effective Stratigraphy of selected PL-566 dams in WI. Top of dam elevation shown in red and bottom of core trench elevation shown in green. Brown polygons have been drawn to show the relative elevation of breaches.

As shown in Figures 2 & 4, the Black Earth Member of the St. Lawrence formation is a known Regional Aquiclude. This means that whether the dams were built in those locations or not, the local water table would have already been elevated. And so there was a high likelihood of water seepage and springs already occurring at that stratigraphic position.

In addition to the naturally elevated regional water table that was present prior to the installation of the structures, there was a high potential for pool water to seep into the abutments when the pool areas filled during rain events. This is because highly permeable sediments in the valley are covered by a thin (2-10 foot thick) windblown loess cap of Lean Clay (CL) or Silt (ML) material. Repeated filling and draining from flood events over the 50-60 year history of the structures may have started flow paths that progressed close to, or into, the permeable rock making it easier for water to flow into and through the bedrock during the August 2018 event.

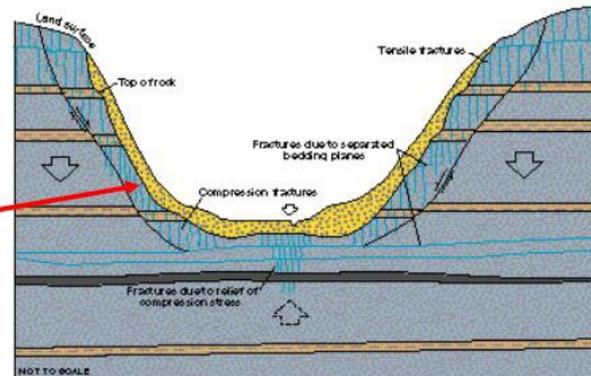
In addition to the geologic conditions of the earth materials listed above; landscape formation and other geologic factors have created secondary fracturing in the rocks that go parallel to the valley. Valley relief fracturing is a relatively common occurrence in brittle flat lying rocks that have been subjected to extensive erosion and downcutting during the formation of a mature landscape (Figure 5). Examples of valley relief fracturing were observed in the CC-21, CC-23, WFK-1 breach areas during the site visits and in the after-failure photos of the CC-41 abutment foundation.

**Valley Relief Fractures in the CC-23 2018 Abutment failure**



**Figure 92.** Stress relief occurs when compressional stress is partly removed by erosion of overlying rocks; this results in the formation of predictable fracture patterns in valleys. The fractures are generally horizontal under valley floors and vertical along valley walls. The interconnected fracture system greatly increases the permeability of the rocks, especially in the valleys.

EXPLANATION	
	Colluvium and alluvium
	Sandstone
	Shale
	Coal seam
	Compressional stress— Proportional to size of arrow
	Resultant stress
	Plane of slumping— Arrows show relative movement

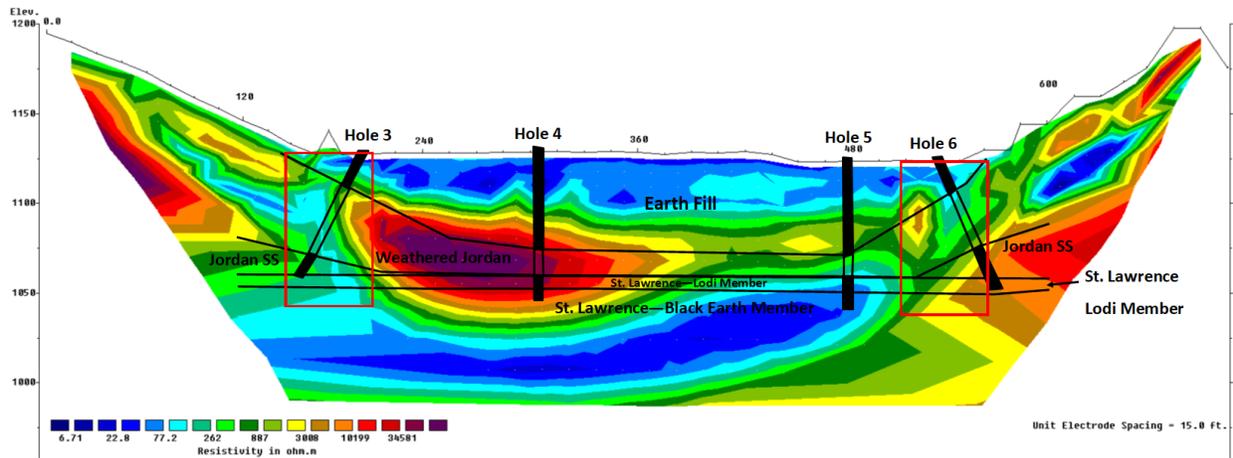


Modified from: Wyrick, G.G., and Borchers, J.W., 1981, Hydrologic effects of stress-relief fracturing in an Appalachian valley: U.S. Geological Survey Water-Supply Paper 2177, 51 p.

Figure 5 provides an explanation of how valley stress relief fractures form, including a photo of an interpreted valley relief fracture in the wall of the CC-23 abutment. These fractures run parallel to the valley flow direction which make them a likely bypass for the structure.

Since valley stress relief fractures can be wide (up to 6 inches) and run parallel with the valley, they offer direct pathways for water from the pool area to bypass the impoundment structure during a large rain event. Once water has filled these pathways within the rock, the system of pathways becomes pressurized if the water is not free to drain downward through the fracture system, or out the abutment. This pressurized system has the potential to push out a block of rock releasing water from within the hillside. This problem becomes exacerbated by the high regional water table. And the typically very thin layer of soil over the bedrock on these sites is problematic because it presents little resistance to the seepage forces within the rock formation.

Unfortunately, it is difficult to see valley relief fracturing from the surface because of the soil layer. Evidence of valley relief fractures have been found on other PL-566 sites in southwest Wisconsin by use of rock coring and Electrical Resistivity Imaging (ERI). Figure 6 is an example of the use of geophysics technology for investigating a PL-566 structure in southwest Wisconsin. Ideally, the Geophysics Technology, such as Electrical Resistivity Imaging (ERI), Electromagnetics (EM), or Seismic Refraction, is completed first to identify vertical low conductivity or soft anomalies as outlined with red rectangles in Figure 6. If needed, a rock core boring can be used after the geophysics investigation to verify the anomaly.



CC-33 ERI Line 1 DIP-DIP—48 electrode—15 foot spacing.

Figure 6 illustrates an Electrical Resistivity Imaging (ERI) line and rock borings of a PL-566 site with highly weathered fracture zones on the two abutments. These were identified based on the presence of particularly soft and weathered sandstone in locations of low resistivity anomalies (red boxes). The low resistivity may indicate that water is moving through the fracture.

### Records, Reports, and Documents Reviewed

The design folders and remaining as-built documents are not available. The flood event described in this report was one of a series of storms causing flooding in southern Wisconsin. During this flood, the Wisconsin State Office storage area was flooded. The design and as-built documents were submerged and saturated. NRCS Wisconsin has contracted with a vendor to put all the flooded documents through a freeze-dry process to recover them. Unfortunately, it is unknown if this process will result in recovery of any documents. At the time of this report, those documents have not been returned by the vendor.

Following a review of the criteria in place during the design of these dams, Engineering Memorandum 3, “Design Notes and Standard Criteria for Design of Retarding Dams”, 1954, was the applicable design criteria when WFK Mlsna was designed. EM 3 (1954) has two alternatives for flood routing covering multiple conditions. But because there is no design report and because there are no hydraulic sheets in the as-builts for WFK Mlsna, it is not possible to evaluate the hydrology applied to this site.

The foundation investigation for WFK Mlsna similarly lacks documentation. EM 3 (1954), “Foundation and Embankment Investigations”, pg. 7, states, “Other things being equal, if failure of a particular proposed dam would cause significant damage to people or property the foundation investigations should be very thorough and complete; whereas if no hazard exists other than the loss of the dam itself, the requirements for a foundation investigation could be much less rigid. A knowledge of the geology of the site coupled with past experiences at similar sites will minimize, in some cases at least, the extent of required subsurface explorations.”

The next bullet item in EM 3 (1954) states, “At least five test holes or pits should be dug to a depth equal to three-fourths the height of the dam or more or to a continuous firm foundation of rock or highly consolidated material for each earth dam being designed.”

Five test holes are shown on centerline of dam of WFK Mlsna (see Attachment C, Sheet 2 of 15) and they all appear to have been drilled to bedrock. This structure was designed as low

hazard. The committee has inferred that the scope of geology was based on the hazard class of the dam and that it met the minimum requirements of EM 3 (1954).

EM 3 (Rv. 1957) did not change any criteria. It provided an updated definition of the dams falling under EM 3. The only structure which does not fall under EM 3 is CC-23. National Engineering Handbook (NEH) Part 9, Earth Dams and Dugouts was developed for low hazard dams with a height-storage product under 3,000. CC-23 was designed as a low hazard dam with a height storage product of approximately 2450. Therefore, NEH Part 9 has been used to evaluate the geology and hydrology of CC-23.

In NEH Part 9, Subject 2, Surveys and Investigations, Soil Mechanics Investigations, Foundation and Abutment Investigations states "Foundation and abutment investigations are made to determine their adequacy." The following are pertinent paragraphs under this heading:

- (1) To insure stable support for the structure under all conditions of loading:
  - c. The foundation and/or abutments must be relatively insoluble and resistant to erosion to prevent the eroding or washing away of the material from upstream, downstream, or under the dam.
- (2) To ensure that the movement of water through the foundation and/or abutment will not cause:
  - a. Piping and scour under the dam,
  - b. Excessive loss of water stored behind the dam,
  - c. The formation of excessive uplift pressure.

A general knowledge of the bedrock material in the abutments should have demonstrated the Jordan Sandstone is not resistant to erosion. General knowledge of the wide valley fractures should have been an indication (2) a, b, and c were all possibilities.

The section titled "Extent of Investigations. Extent of investigations of foundation and abutment and borrow pit areas:" may be found on page 9-10 of NEH Part 9. The following are pertinent paragraphs in this section:

- (1) Enough test holes must be put down to establish the extent and continuity of various strata...
- (4) At all sites, a thorough investigation of the abutments is necessary to indicate the presence or absence of hazardous geologic conditions or unstable soil conditions, both above and in the abutment.

It appears from the as-built drawings that the test holes at Sta 20+29 were taken to represent the geologic conditions on centerline of dam at Sta 29+00, almost 900 feet downstream of the test holes. These test holes are too far from the centerline of dam to be representative of the foundation and abutment materials. Based on the as-built drawings, the geologic investigation for this site failed to meet several minimum criteria set forth in Subject 2. Surveys and Investigations.

Subject 3, Design of Earth Dams and Dugouts, covers minimum required capacity of the principal spillway for a floodwater retarding dam with a product under 3,000 – this applies to CC-23. There is no minimum design storm frequency for the principal spillway listed in this section. Rather, the required capacity is dependent on one or more of the following five factors: benefits that accrue due to reduction of discharge; needs to satisfy downstream water use; damages resulting from storage and prolonged outflow; effect of significant runoff from two or more consecutive storm events during the time required to discharge the detention storage; and capacity of the downstream channel. The evaluation of these factors was likely documented in the planning support data or the design folder. Neither was available for this investigation so the committee could not evaluate that analysis. However, the principal spillway storm frequency documented on the as-built drawings shows that the principal spillway storm exceeded the emergency spillway storm as outlined on page 9-27 of NEH Part 9.

The “Emergency Spillways” section of Subject 3 states that, “The emergency spillway should be designed to handle, as a minimum, the 25-year peak flood flow.” According to Sheet 1 of 9, Drawing No. 3-E-45670-H, the design principal spillway rainfall duration - frequency for CC-23 was the 6 hour – 25 year storm, or 4.6 inches. Therefore, the auxiliary spillway design exceeds the minimum required for CC-23. The freeboard design rainfall recorded on the as-built plan is 5.79 inches. According to the work plan, this is the 100 year storm

Based on approval and design dates on the as-built drawings of the other three dams, WFK-1, CC-21, and CC-29, the applicable design criteria appear to come from Engineering Memorandum (EM) SCS-27 dated March 14, 1958. Minimum criteria for setting the auxiliary spillway elevation may be found on page 4 under Paragraph I.2. The text states, “The Principal Spillway Hydrograph shall represent a flood event that will not be equaled or exceeded, on the average, more often than once in 25 years for class (a) structures, 50 years for class (b) structures, and 100 years for class (c) structures.” Since all three dams were designed as low hazard, the minimum required storm frequency was 25 years. The principal spillway flood routing sheet in the as-built drawings for CC-21 used the 6 hour – 100 year storm. The storm frequency isn’t documented for CC-29, but the same design rainfall was used. These two dams are in proximity to one another so it’s logical that CC-29 was also designed with a 100-year principal spillway storm.

WFK-1 as-builts don’t document the principal spillway storm frequency but it does document that the hydrograph coordinates were taken from Hydrology Guide 3-21 and that the 6 hour storm was used. The third paragraph on Page 4 of EM SCS-27 states, “Minimum runoff volumes for the hydrographs shall be determined using either a frequency analysis of streamflow data with an adequate length of record, or by using a 6-hour rainfall of the required frequency and the associated runoff computed by the method of the Hydrology Guide Section 3.10 with antecedent moisture condition II. The hydrographs shall be constructed using the methods of the Hydrology Guide, Sections 3.16 or 3.21.” This documentation supports that the principal spillway routing was done in accordance with EM SCS-27.

The minimum design capacity of the auxiliary spillway channel and for establishing the minimum freeboard is found under Paragraph I.1 on pages 2 and 3. For Class (a) dams EM SCS-27 has the following requirements. “The spillway shall be proportioned so it will pass the Emergency Spillway Hydrograph computed by the method in Section 3.21 of the Hydrology Guide using moisture condition II and 0.5 of the 6-hour rainfall shown by Figures 3.21-1, -2 or -3 at the safe velocity determined for the site.” And for the freeboard, “The minimum capacity of the emergency spillway shall be such that it will pass the Freeboard Hydrograph computed by the method in Section 3.21 of the Hydrology Guide using moisture condition II and 0.75 of the 6-

hour rainfall shown by Figures 3.21-1, -2 and -3 with the water surface in the reservoir at or below the elevation of the settled height of the dam.”

The CC-21 as-builts do not have the hydraulic data sheet for the “Flood Routing Emergency Spillway” so there’s no evaluation of the emergency spillway routing. The “Flood Routing Freeboard” sheet, Sheet 2 of 7, does show that the 6-hour storm, moisture condition II, and 0.75 of the 6-hour rainfall were all used for setting the freeboard elevation.

The CC-29 as-builts also lack the hydraulic data sheet for the emergency spillway. Although Sheet 6 of 9 is titled “Flood Routing Emergency (Freeboard) Spillway”, it does not have the emergency spillway rainfall data or routing. This sheet was used exclusively for the freeboard flood routing. The 6-hour storm was used for the freeboard storm, but the frequency isn’t documented on this sheet. It is significantly higher than the principal spillway storm and the factor of 0.75 was applied to the rainfall. Though the documentation is not clear or conclusive, it appears EM SCS-27 was probably adhered to on CC-29.

The WFK-1 as-builts provide good documentation of the hydrology. Flood routing sheets 2 and 3 indicate that hydrograph coordinates were taken from Section 3.21 of the Hydrology Guide using moisture condition II and the 6-hour rainfall. The factor of 0.5 was used for the emergency spillway flood routing and the factor of 0.75 was used for the freeboard flood routing. All the documentation is consistent with EM SCS-27.

## **Evaluation:**

### Summary of Possible Causes

- Weaknesses in the foundation geology at these five sites were identified as the probable cause for how quickly the breaches occurred and the extent to which they breached.
  - On West Fork Kickapoo Sites 1 and Mlsna the breach is attributed to the weak material located in the auxiliary spillways and abutments. Seepage and internal erosion are considered likely key factors in initiating the surface erosion while the speed and severity of erosion is attributed more to the weak materials.
  - On Coon Creek 21, 23 and 29 the breach is attributed to the combination of the characteristics of the bedrock. Valley relief fractures and the high permeability Lodi Member of the St. Lawrence formation facilitate the movement of water within the bedrock which leads to hydrostatic pressures and water flow. The soft, weakly cemented Jordan Sandstone weathers to a loose sand and is highly erodible. And the low permeability and porosity of harder members of the formation prevent drainage of water creating an elevated aquifer. The thin layer of soil over the bedrock is also associated with the speed and severity of the breaches on these sites.
- The committee could not rule out the possibility that any one of these dams may have breached even if the designs had addressed the weaknesses in the foundations. The storm event that occurred from the evening of August 27 through the early morning of August 28 generated runoff that exceeded the design hydraulic capacity of four of the five dams. On the fifth dam, WFK-1, the volume and rate of runoff exceeded stable channel flow in the auxiliary spillway. This event provided the hydraulic conditions that exposed the latent weaknesses inherent to the geology in the two watersheds. But the committee would not have expected such a rapid and complete breach to have occurred from this rainfall event alone. Because the committee could not rule out the possibility

that any one of these dams may have failed during passage of this event, extreme runoff is given equal weight to the foundation materials as the probable cause of failure- but not the speed or degree to which failure occurred.

## Evaluation of Data

### Hydrology:

Jersey Valley has the largest drainage area (> 8 square miles) and largest design rainfall depth, nearly 11 inches. This dam was the only breached dam that did not overtop. There are two possible explanations why Jersey Valley did not overtop:

1. As indicated by the NWS NEXRAD 6-hour data, the storm rainfall depth was less than the design rainfall depth (11 inches). If this was the case, there was insufficient storm runoff to overtop the dam.
2. The auxiliary spillway breached before the reservoir level was high enough to overtop the embankment. If the storm runoff greatly exceeded the original design runoff, the rapid breach erosion in the auxiliary spillway channel, and/or the right abutment, provided enough increased discharge capacity to prevent the dam from overtopping.

Since the drainage area at Jersey Valley was significantly larger than the other dams, it would have taken longer for the peak storm runoff to reach the Jersey Valley reservoir and begin to overtop the dam. But the breach occurred in a short span of time and so the committee could not draw a clear conclusion as to whether the August storm rainfall depth was less than or greater than the design rainfall depth based solely that it didn't overtop.

Since Jersey Valley did not overtop, insufficient hydrologic capacity did not cause the breach. Instead, it appears that high volumes of storm runoff in the reservoir were sufficient to cause rapid erosion of the auxiliary spillway and nearby right abutment before the water surface in the reservoir could reach top of dam. A model of the storm in SITES resulted in a breach of the auxiliary spillway with a water surface elevation higher than the water surface elevation at the time of the actual breach. This indicates that the erosion either proceeded at a much faster rate than modeled, or the erosion had an internal component beyond the simple surface erosion/headcut model in SITES.

The Coon Creek dams were designed for 6- to 7-inch rainfall depths. Flattened vegetation on the backslopes of the embankments indicates overtopping. Since these dams overtopped, the storm rainfall depth was probably greater than the original design rainfall depth.

The Mlsna dam has the second smallest drainage area (1.48 square miles), so the design rainfall depth would likely have been slightly smaller or equal to the three Coon Creek designs. The Mlsna dam overtopped, so it is likely that the storm rainfall depth was greater than the design rainfall depth.

The design rainfall duration is listed as 6 hours, which is the appropriate design storm duration criteria listed in Engineering Memo 3 (issued 1956) and Engineering Memo 27 (originally issued 1965, supplements released through 1976). The dams were built in the 1950s and 1960s and have performed well, without any hydrologic/hydraulic capacity issues for the last 50 years.

There is no evidence that the five dams breached because the hydrologic design criteria were insufficient. Rather, the highly unusual August 2018 storm rainfall was greater than the original design rainfall for the four dams that overtopped.

Potential Geologic Failure Mechanisms and Risks:

According to FEMA Interagency Publication P-1032 – Evaluation and Monitoring of Internal Erosion (<https://www.fema.gov/media-library/assets/documents/107639>), the two most likely failure mechanisms of earthen dams have been overtopping and piping caused by internal erosion (Table 5). Since dam overtopping failures commonly destroy any evidence of piping during the surface erosion and breach of the structure, it is hard to know if one of these was the dominant cause of the failures. The five structures that failed in Wisconsin were not the only structures that overtopped during this rain event. There were likely other conditions that led these structures to fail when the others did not.

Table 5 – Summary of Failure Modes \*

Mode of failure	Number of cases		Percent failures (where known)	
	All failures	Failures in operation	All failures	Failures in operation
Overtopping	46	40	35.9	34.2
Spillway/gate	16	15	12.5	12.8
<b>Subtotal</b>	<b>62</b>	<b>55</b>	<b>48.4</b>	<b>47.0</b>
Piping through embankment	39	38	30.5	32.5
Piping through foundation	19	18	14.8	15.4
Piping from embankment into foundation	2	2	1.6	1.7
<b>Subtotal</b>	<b>59</b>	<b>57</b>	<b>46.1</b>	<b>48.7</b>
Slides	7	5	5.5	4.3
Earthquake/liquefaction	2	2	1.6	1.7
Unknown	8	7		
<b>Total number of failures</b>	<b>136</b>	<b>124</b>		

\* Table copied from Chapter 2 of “Evaluation and Monitoring of Seepage and Internal Erosion”, FEMA P-1032/May 2015. Note that there are minor errors in the piping subtotals and the totals. These errors are in the source document.

A combination of the potential weaknesses (both geologic conditions and other types) listed previously may have led to one or more of the following failure mechanisms: overtopping causing headward advancing gully erosion which eventually initiated the breach (NEH 628 Ch. 52); seepage and internal erosion causing concentrated leak erosion (FEMA P-1032 section 3.2.1), or internal erosion causing backward erosion piping (FEMA P-1032 section 3.2.2).

Headward ‘classic’ gully erosion is common on the landscape and occurs when surface flows begin scouring earth material at a location with concentrated flow, an abrupt change in slope,

and/or disturbed vegetation. As mentioned earlier in the report, other sites that overtopped or had spillway flows during this event began developing smaller surface gullies on the spillways or on the downstream abutment groin area. The two sites that failed in the auxiliary spillway (WFK-1 and MLSNA) appeared to have the longest duration of flows and may have failed from surface erosion of the weak material with some help from seepage and internal erosion to start the process (as seen after the BA12 spillway erosion in 2008). Although this is impossible to validate in the five breached structures because most of the evidence is gone.

Concentrated leak erosion can occur when preferential flow paths (such as vertical valley relief fractures) receive relatively high velocity water which subsequently erodes the walls of the open crack (Paraphrased from section FEMA P-1032 section 3.2.1 and 3.2.1.8). Observations of eroded cracks and large volumes of sand have led to the theory that this occurred at CC-23, CC-21, CC-41 (1978 failure), and likely CC-29 (although there is less evidence because there is sediment and water covering the bottom of the breach). This process may have started with seepage through the soil while the pool area filled to levels not seen before in these dams. The record hydraulic head and duration may have been sufficient for the seepage to penetrate through the soil and into the cracks in the rock. But it's also possible that this connection through the soil to the rock slowly developed over the life of the dams.

Other possibilities are that this seepage path occurred due to: the thin layer of soil; cracks in the soil; cracks in the earth fill created by differential settlement (FEMA P-1032 section 3.2.1.1 and Figure 3-1a), or a combination of these factors. The main theory is that once the seepage water reached the interconnected valley relief fractures, the hydrostatic pressure within this confined fracture increased until the overlying topsoil failed. Then blocks of bedrock were pushed out of the bedrock face creating a discharge point for the seepage water and initiated the erosion process at the downstream toe of the dam. Concentrated leak erosion accelerated internal erosion of the upstream soil leading to an increase in seepage velocity. This increase in flow velocity could have easily eroded the sand grains out of the vertical fractures. A positive cut off slurry trench and/or a clay blanket may be the best way to fill and stabilize the valley relief fractures that have the potential of causing concentrated leak erosion during large rain events.

Backward erosion piping is subsurface erosion that initiates at an exit point and progresses upstream in a similar way that gully heads progress (paraphrased from section FEMA P-1032 section 3.2.2 and Figure 3-6). Over time and multiple rain events, seepage water slowly moves through more discrete weaknesses in the foundation and brings particles of soil or weathered rock with it. On these sites the weathered sand, Jordan Sandstone, or fractured rock that is not as interconnected as the valley relief fractures, yet still permeable like the Lodi Member of the St. Lawrence Formation, may all be susceptible to piping. This erosion requires layers susceptible to piping covered with stiffer soils (like the CL material that is present on these sites) that can form a roof to conceal the erosion from the surface. This erosion would also require that the toe of the abutment is not filtered, or it has a non-functioning toe drain. As a result of the geologic conditions, adding toe drains to the abutments of these sites will be helpful for all 86 PL-566 structures in southwest Wisconsin. The failure of WFK-1 shows that pressure grouting alone may not be sufficient.

The three sites that failed at the abutment (CC-21, CC-23 and CC-29) all had erosion in the spillways from surface water flowing over the spillways before the abutment breached. These surface erosion areas stopped in either very soft to very stiff Lean Clay (CL) with a material strength (Ms) from 0.02 to 0.45 averaging 0.05 to 0.1. These soil material strength values are low and difficult to replicate even in the soft rock units found on these sites (Jordan had the lowest Ms of 0.45) without the additional geologic conditions such as weathered loose sand,

fracturing, high permeability, or high regional water table to reduce the strength and start erosion via an additional failure mechanism such as internal erosion. Yet the breaches at CC-21, CC-23 and CC-29 clearly showed that the abutment failed before the spillways. This indicates that the geologic conditions listed above have a significantly adverse influence on the material strength of the abutments in order to make them weaker than the soil on the spillways.

This theory is further supported by evidence from the CC-41 breach in 1978. The abutment failed on this site even though the dam did not overtop. Photos of the breach area showed vertical valley relief fractures in the floor of the breach. And while the subsurface seepage on other sites, such as Bad Axe 12 (BA-12) and West Fork Kickapoo site 17 (WFK-17), did not result in a breach, there was damage to these structures as well. All this evidence strongly suggests that the internal erosion failure mechanisms of concentrated leak erosion and backward erosion piping may have had an equal, if not greater role in the failures at the abutments than surface erosion.

In summary, regardless of the exact failure mechanisms, the largest risk to the integrity of these dams is the quality of the rock and the weaknesses (both geologic and non-geologic) in the abutments. Note that this would apply to all the dams in these rock units. Of specific concern is the seepage potential through highly connected fractures that likely induce horizontal flow. This seepage potential is primarily due to the depth of the water table and the layered bedrock containing a partially confining layer (Black Earth Member) sandwiched between two highly permeable fractured bedrock units (Jordan Sandstone / Lodi Member and the Tunnel City Member). When valley relief fracturing is present these rock formations are even more prone to lateral water movement and potential seepage at the downstream edge of the auxiliary spillway or abutment. Other flood control structures in similar geologic conditions in Wisconsin have had an increased risk of abutment failure due to water seepage causing landslides (WFK-17), seepage induced gully head erosion (BA-12) and in one other case breach of the structure (CC-41). The best way to treat these geologic conditions and prevent these potential failure mechanisms is by cutting off the subsurface water (using abutment drains, positive cut-offs, or clay blankets as determined by a design engineer) while protecting spillways that are built directly on the rock abutments using a concrete spillway with toe drains underneath.

### Design Documentation

None of the original design documents were available to the committee. As-built records containing the original design folders may have been part of documents flooded during September storm events. Files that were salvaged have been sent to a vendor for recovery but at the time of this investigation and report, no documents relevant to these five sites have been returned to NRCS.

The as-built drawings were the only source that had any design data. On some sites this included test holes showing foundation and borrow materials as well as depth to bedrock, borrow source materials and hydrologic data. The as-built drawings for WFK Mlsna did not have hydrologic data.

## Construction Documentation

As with design data, the only construction documentation available to the committee were copies of the as-built drawings. Information helpful to the investigation included the as-built elevations, core trench depth, and embankment materials.

Additional construction documentation may be recovered after completion of this report. If information comes to light that alters any of the findings, this report shall be amended.

## **Conclusions:**

### Summary of Conclusions:

- The committee has concluded that weaknesses in the foundation geology and extreme runoff during the rainfall event of August 27 and 28 are the probable causes for the dam failures. The committee has concluded that weaknesses in the foundation geology led to very rapid breaches that eroded all the way down to and through bedrock.
- The failure of the Coon Creek sites occurred in the groins opposite the auxiliary spillways. The mode of failure appears to be a combination of weak materials and poor bedrock conditions with seepage forces playing an instrumental role in the type and speed of the breach. The failure of the West Fork Kickapoo sites occurred in the auxiliary spillways. While there were likely similar forces at work in the two watersheds, it appears erosion of the surface materials in the auxiliary spillway played a more significant role in West Fork Kickapoo. The committee arrived at this conclusion because the valley relief fractures were not as extensive at these sites and there more erosion occurred in the embankments of the West Fork Kickapoo sites. On WFK-1 a significant amount of the valley relief fractures had been successfully pressure grouted. On WFK MIsna there just wasn't the presence of fractures at the exposed bedrock surface. The case-hardened sandstone withstood the breach flow. See the Geology section for a detailed evaluation of the foundation materials and potential failure mechanisms.
- WFK MIsna is the only site without any hydrologic data in the as-builts. The documentation is clear on the other four sites and demonstrates that the design followed applicable criteria whether specified by NEH Part 9 or EM SCS-27. Although the hydrology on WFK MIsna could not be positively confirmed, the committee has concluded that it is reasonable to believe EM SCS-27 was followed on WFK MIsna and therefore there is no deficiency related to the hydrology on any of the five sites.
- The storm event that occurred the night of August 27th and 28th exceeded the design capacity of at least four of the five dams. WFK-1 being the only dam that did not overtop may have received flood volume in excess of the design. However, the auxiliary spillway breached, and this may have occurred prior to passing of the maximum flood volume so the committee could not make a conclusive determination on this site.
- The applicable criteria at the time of design of CC-23 required geologic investigation of the foundation and abutments. The as-built drawings show the only test holes were almost 900 feet upstream of centerline of dam and not within the foundation of the dam. However, the geology does not appreciably change between locations and is very similar to the other two Coon Creek sites. It appears reasonable to assert that even if the geology had been properly investigated in the foundation area of CC-23, there would

have been no significant difference in the design approach. Therefore, the committee has concluded that while NEH Part 9 was not followed in the design of CC-23, this fact did not lead to a design deficiency.

### **Recommendations:**

The committee offers the following recommendations for consideration:

1. Conduct a Planning Study to develop and evaluate alternatives for each dam and the entire watershed. This study may include:
  - a. Complete an assessment of current resource concerns, future flood control benefits and costs to aid Sponsors in evaluating what course of action best meets their needs.
  - b. Decommissioning by removal of the dam, stabilizing the site and completing stream restoration.
  - c. A redesign or relocation of the dam and all its components to current standards and specifications. Measures to effectively treat the foundation and abutments will be a necessary component.
2. Design considerations for dams that will be repaired or replaced:
  - a. Investigation:
    - i. Complete additional geologic investigation of the abutments to determine direction and extent of sandstone formations and jointing.
    - ii. Coon Creek 41 had a similar breach without overtopping. Complete an inspection and assessment of Coon Creek 41. Review the failure report and repair design. Use the lessons learned from this failure and repair to guide the repair design.
  - b. Seepage Control:
    - i. Develop a pressure grouting plan or a slurry trench plan to cutoff upstream to downstream seepage flow and to prevent the buildup of hydrostatic pressure at the end of the dam.
    - ii. Blanket valley walls up to the top of dam elevation with compacted earthfill. Geologic investigation and seepage analysis are required to determine upstream extent of blankets.
    - iii. Construct the downstream groins with a clay liner to increase the head loss of abutment seepage and redirect ground water discharge downstream of the dam.
    - iv. Construct drains in the downstream groins to provide a stable outlet for seepage through the abutments.
  - c. Auxiliary Spillways
    - i. Design the dams without a vegetated auxiliary spillway.

- ii. Provide a structural auxiliary spillway to replace the vegetated auxiliary spillway.
    - iii. Design the dams with a ramped spillway located away from the abutments graded all the way to the valley floor.
  - d. Protect the downstream groins by raising the ends of the dams to provide overtopping sheet flow across the entire width of the dam excluding the groin areas.
3. For all watershed dams located in this geologic formation:
- a. Use Geophysics techniques such as Electrical Resistivity Imaging (ERI), Electrical Magnetic Induction (EMI), or Seismic techniques to assess potential for failure of the abutments.
  - b. Prioritize sites for the geophysics analysis by using slope mapping in ArcMap to identify sites that have significant steep hillslopes near the dam that may indicate the pool areas are shallow to bedrock or have bedrock outcrops that could increase the risk of seepage.
  - c. Inspect the downstream groins for signs of erosion or material weakness and consider implementing measures listed below that protect the groins from overtopping or erosion from sidehill runoff.
    - i. Review the vegetation of the groins and assure a good stand of grass exists on the embankment and the abutment side of each groin. Where timber encroaches on the abutment side of the groin, clear and/or grub trees to provide better growing conditions for grass on the abutment side of the groin.
    - ii. Raise the ends of the dams to prevent concentrated overtopping flow down the groins.
    - iii. Build up the groins with earthfill to keep overtopping flow off the soils that are shallow to bedrock on the wooded slopes.
  - d. Review Emergency Action Plans to make sure contact information and actions planned are up to date. Review the protocols with appropriate personnel.

**List of Attachments:**

- A. Letters of Appointment
- B. Photographs
- C. Selected As-built Drawings
- D. O&M Inspection Photos
- E. Investigation Survey Drawings and Photos
- F. Hydrology
- G. References

Report submitted by:

**MARK MCCURDY**

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Date: 2019.10.03 10:41:10 -05'00'

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Mark McCurdy, Asst. SCE, Des Moines, IA

**KARL VISSER**

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Date: 2019.10.07 10:22:26  
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Karl Visser, Hydraulic Engineer, NDCSMC, Fort Worth, TX

**TIMOTHY WEISBROD**

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WEISBROD  
Date: 2019.10.04 12:12:14 -05'00'

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Tim Weisbrod, Geologist, WI/MN, St. Paul, MN

**MATTHEW  
BLOHOWIAK**

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BLOHOWIAK  
Date: 2019.10.04 09:11:45 -05'00'

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Matt Blohowiak, Civil Engineer, Altoona, WI

**MICHEL  
DREISCHMEIER**

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Mike Dreischmeier, Area Engineer, Richland Center, WI

Attachment A  
Letters of Appointment

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SUBJECT: ENG – Dam Failure Investigation Committee Appointment

Date: September 12, 2018

TO: Mark McCurdy, Assistant SCE, NRCS, Des Moines, IA  
Karl Visser, Hydraulic Engineer, NRCS, NDCSMC, Fort Worth, TX  
Tim Weisbrod, Geologist, NRCS, WI/MN, St. Paul, MN  
Matt Blohowiak, Civil Engineer, NRCS, Altoona, WI  
Mike Dreischmeier, Area Engineer, NRCS, Richland Center, WI

File Code: 210

The recent historically intense rainfall in portions of Wisconsin caused the failure of several PL-566 dams. Overtopping with groin erosion, auxiliary spillway damage, and five breaches, resulted. At least one is a Class VII due to the High Hazard Rating.

Per NEM 504.3, the State Conservationist and the Director, Conservation Engineering Division, determined the investigation committee membership. This letter documents your appointment to the committee. Mark McCurdy will serve as the committee chair.

Members have indicated their availability for the week of September 17<sup>th</sup> to conduct site visits. An expeditious analysis and draft report would be helpful in discussing repair options and potential NRCS financial assistance with the sponsors.

Scott Mueller, Assistant State Conservation Engineer, will be your contact for any assistance needed by the committee. He may be contacted at 608-662-4422 Ext. 265 or [scott.mueller@wi.usda.gov](mailto:scott.mueller@wi.usda.gov).

Your assistance in completing this investigation is greatly appreciated.

Sincerely,

ANGELA L. BIGGS  
State Conservationist

cc:

Noller Herbert, Director CED, NRCS, Washington D.C.  
Steve Durgin, National Design Engineer, NRCS, Washington D.C.  
Johnny Green, Director NDCSMC, NRCS, Fort Worth, TX  
Christian Osborn, SCE, NRCS, Des Moines, IA  
Dave Jones, SCE, NRCS, St. Paul, MN  
John Ramsden, SCE, NRCS, Madison, WI  
Mark Kulig, ASTC-FO, NRCS, Richland Center, WI  
Mary King, Area Engineer, NRCS, Altoona, WI



SUBJECT: ENG – Dam Failure Investigation Committee

Date: September 11, 2018

TO: Noller Herbert  
Director Conservation Engineering Division  
NRCS  
Washington, D.C.

File Code: 210

The recent historic rainfall in portions of Wisconsin caused the failure of several PL-566 dams. Overtopping with groin erosion, auxiliary spillway damage, and five breaches, resulted. At least one is a Class VII due to the High Hazard Rating.

Per NEM 504.3, you and the State Conservationist must determine the investigation committee membership. We are proposing the following members:

Mark McCurdy, Asst. SCE, Des Moines, IA  
Karl Visser, Hydraulic Engineer, NDCSMC, Fort Worth, TX  
Tim Weisbrod, Geologist, WI/MN, St. Paul, MN  
Matt Blohowiak, Civil Engineer, Altoona, WI  
Mike Drieshmeier, Area Engineer, Richland Center, WI

These individuals represent the disciplines and experience needed for the investigations. Potential members have indicated an availability for the week of September 17<sup>th</sup>.

We are requesting your concurrence of the team members.

JOHN R. RAMSDEN, P.E.  
State Conservation Engineer

**NOLLER  
HERBERT**

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NOLLER HERBERT

Date: 2018.09.11  
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cc:

Angela Biggs, State Conservationist, NRCS, Madison, Wisconsin  
Steve Durgin National Design Engineer, NRCS, Washington D.C.

Attachment B  
Photographs

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Photo A1  
Aerial View of WFK 1 following breach on August 28, 2018

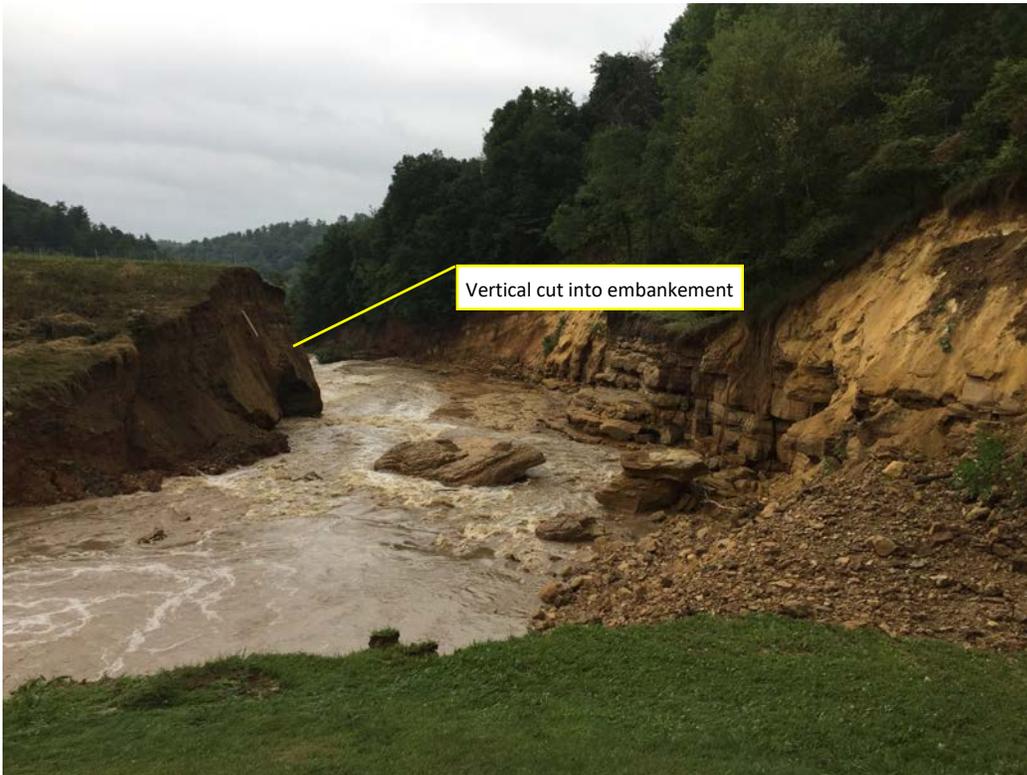


Photo 1  
View of Breach Channel from Upstream  
(WFK 1)



Photo 2  
View of Breach Channel from Upstream  
(WFK 1)



Photo 3  
Exposed right abutment showing jointed bedrock  
(WFK 1)



Photo 4

Close up of exposed right abutment near centerline of dam. Grouted fractures in the breach channel bottom and large voids of eroded sandstone in the abutment face.  
(WFK 1)



Photo 5

Vertical fractures in exposed bedrock. Note white and gray material is from 2009/2010 pressure grouting.  
(WFK 1)



Photo 6

PVC and grout from 2009/2010 pressure grouting – right abutment near centerline dam. Note large voids in bedrock left by eroded sandstone.

(WFK 1)



Photo 7

Remnant PVC and grout from 2009/2010 pressure grouting.

(WFK 1)



Photo 8

Breach channel from centerline top of dam. Note difference in stability of bedrock in curtain wall versus bedrock upstream and downstream.  
(WFK 1)



Photo 9

View of downstream breach inundation area from top of dam.  
(WFK 1)



Photo 10  
View of top of dam from right end near top of cut slope.  
(WFK 1)



Photo 11  
View of upstream pool from beach area.  
(WFK 1)



Photo 12  
View of breach channel and left end of dam from downstream of dam.  
(WFK 1)

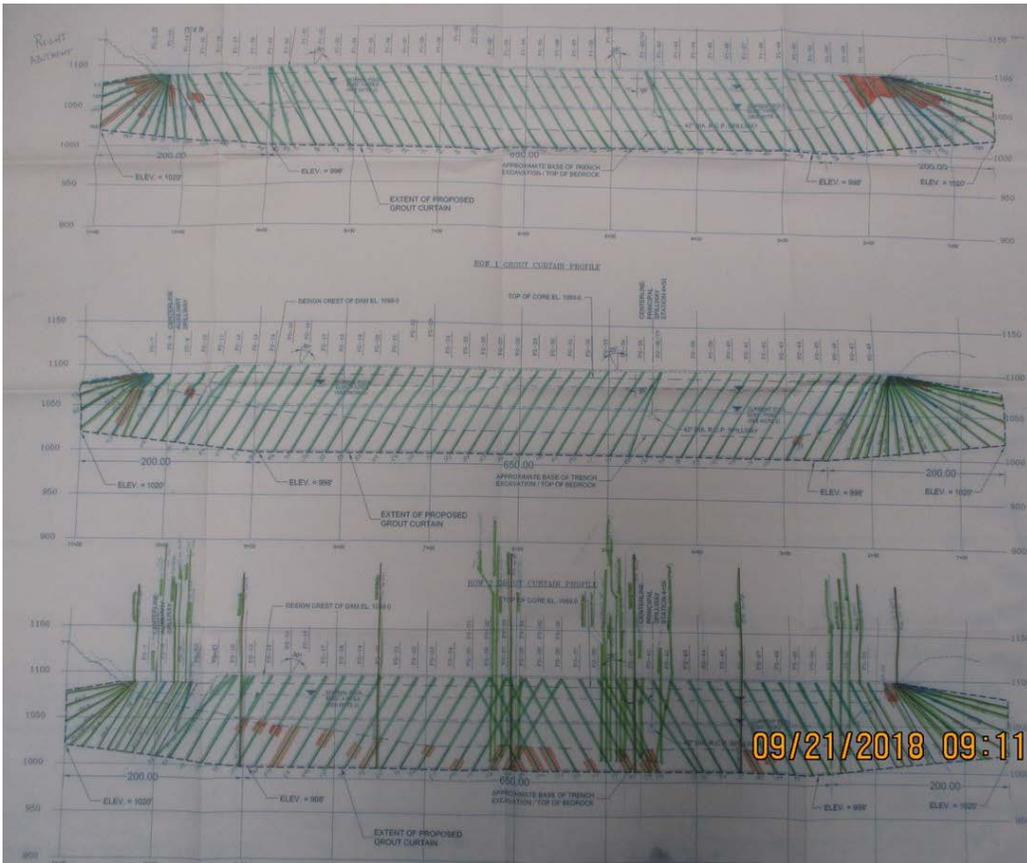


Photo 13  
Drilling plan for 2009/2010 contract to install pressure grouted curtain wall.  
(WFK 1)



Photo 14

View of the embankment from the left end of dam with breach in background.  
Note short vegetation due to grazing.  
(WFK MLSNA)



Photo 15

View of scour hole at the toe of the left downstream groin from the top of dam.  
(WFK MLSNA)



Photo 16  
Scour hole at toe of left downstream groin.  
(WFK MLSNA)



Photo 17  
View of breach channel and upstream headcut from breach channel near centerline of dam.  
(WFK MLSNA)



Photo 18

View of exposed bedrock in breach channel (right abutment) from top of dam.  
(WFK MLSNA)



Photo 19

View of downstream section of breach channel showing right abutment bedrock from top of  
dam.  
(WFK MLSNA)



Photo 20

View looking upstream of exposed bedrock in the breach channel (right abutment) from the downstream end of the breach channel.  
(WFK MLSNA)



Photo 21

Close up of bedrock in breach channel.  
(WFK MLSNA)



Photo 22

View of right end of dam in breach channel from the downstream end of the breach channel.  
(WFK MLSNA)



Photo 23

View of dam from abutment to abutment from downstream of dam.  
(WFK MLSNA)



Photo 24  
View of breach inundation area from top of dam.  
(WFK MLSNA)



Photo 25  
Damaged road in breach inundation area.  
(WFK MLSNA)



Photo 26

View of scour hole at downstream end of auxiliary spillway from downstream side.  
(WFK 3)



Photo 27

View of scour hole at downstream end of auxiliary spillway from upstream side.  
(WFK 3)



Photo 28  
Exit channel of auxiliary spillway.  
(WFK 3)



Photo 29  
Downstream side of embankment. Vegetative cover did not fail during over-topping.  
(WFK 3)



Photo 30  
View of right groin from top of dam.  
(WFK 3)



Photo 31  
Erosion in left groin.  
(WFK 3)



Photo 32  
View of erosion in left groin from top of dam.  
(WFK 3)



Photo 33  
Erosion in left groin.  
(WFK 3)

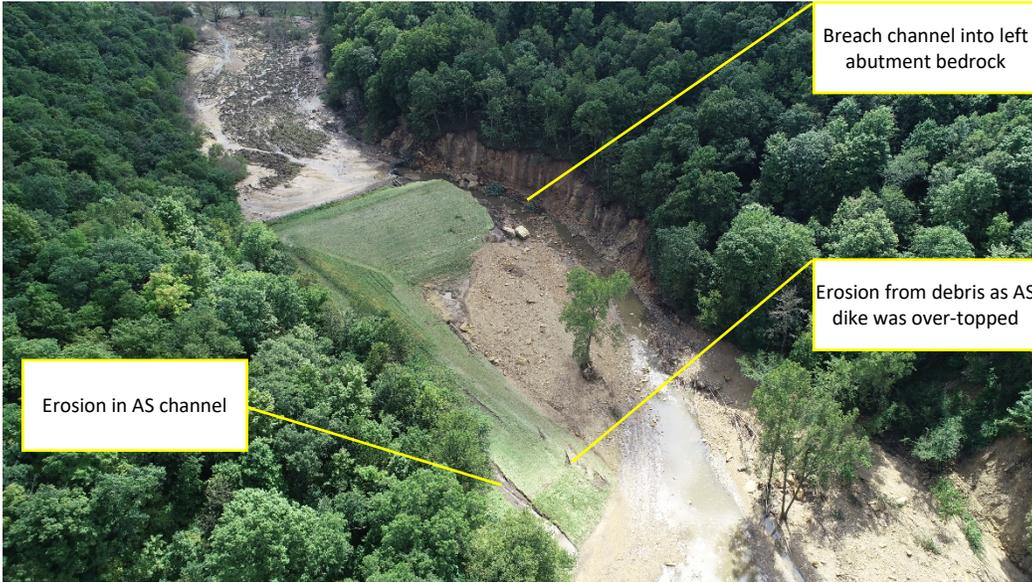


Photo 34  
Aerial View  
(CC 21)



Photo 35  
View of vegetation on downstream berm and embankment.  
(CC 21)



Photo 36  
View of dam, auxiliary spillway and debris field from downstream end of auxiliary spillway.  
(CC 21)



Photo 37  
View of auxiliary spillway exit channel from level section  
(CC 21)



Photo 38

View of auxiliary spillway exit channel from downstream end of auxiliary spillway.  
(CC 21)



Photo 39

View of exposed bedrock in breach channel from near upstream toe of dam.  
(CC 21)



Photo 40  
View of exposed bedrock in breach channel from near upstream toe of dam.  
(CC 21)



Photo 41  
View of right downstream groin from top of dam.  
(CC 21)



Photo 42  
View downstream of dam of outwash from left abutment.  
(CC 21)



Photo 43  
View of downstream embankment and auxiliary spillway dike vegetation from top of dam.  
(CC 29)

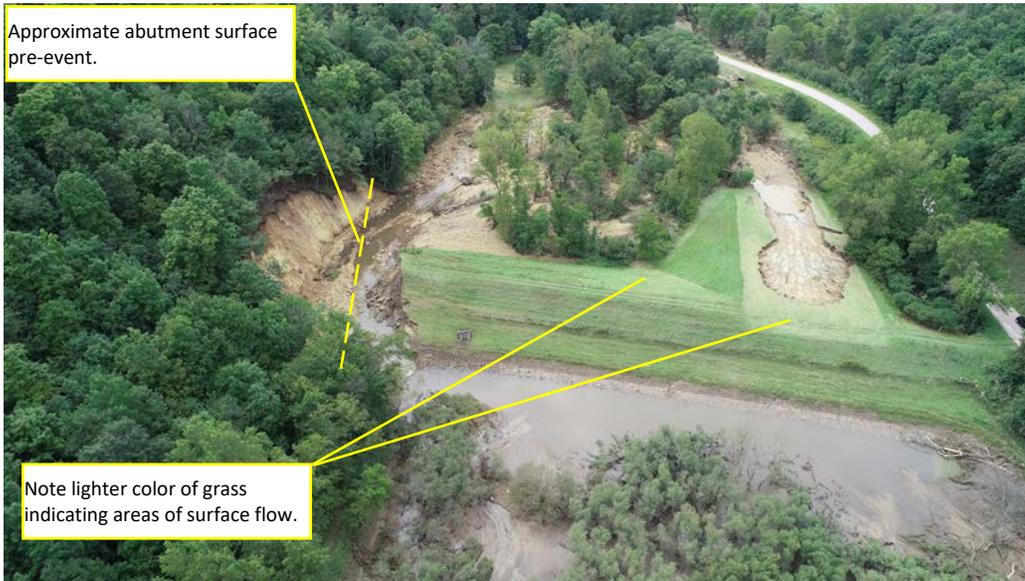


Photo 44  
Aerial photo of dam and auxiliary spillway from upstream of dam.  
(CC 29)



Photo 45  
View of auxiliary spillway exit channel from level section.  
(CC 29)



Approximate location of Control Section..

Upstream extent of AS head cut..

Photo 46

View of upstream end of auxiliary spillway from the right end of top of dam.  
(CC 29)



Photo 47

View of auxiliary spillway head cut from left side of channel.  
(CC 29)



Photo 48

View of top of dam and left abutment, post-breach, from right end of dam.  
(CC 29)



Photo 49

View of breach channel from downstream of the dam. Note photo taken the morning of the event.  
(CC 29)



Photo 50

View of jointed and eroded bedrock in left abutment,  
downstream end of breach channel, from top of dam.  
(CC 29)



Photo 51

Close up view of left abutment bedrock in outlet end of breach channel.  
(CC 29)



Photo 52

View of exposed end of embankment in breach channel. (CC 29)



Photo 53

View of head cut into sediment pool from upstream toe of dam. (CC 29)



Photo 54  
View of debris in breach inundation area from top of dam.  
(CC 29)



Photo 55  
Aerial view of structure site.  
(CC 23)



Photo 56  
View of top of embankment from top left end of dam.  
(CC 23)



Photo 57  
View of ranch house and bridge washout from county road.  
(CC 23)



Photo 58  
View from downstream county road of damaged house in breach inundation area.  
(CC 23)



Photo 59  
Remnants of county bridge downstream of house.  
(CC 23)



Photo 60

View of auxiliary spillway exit channel from downstream end. Note two large headcuts and one small headcut near control section.

(CC 23)



Photo 61

View of auxiliary spillway exit channel from downstream end.

(CC 23)



Photo 62

View of auxiliary spillway exit channel from near control section.  
(CC 23)



Photo 63

Close up of full depth of bottom scour hole in auxiliary spillway exit channel.  
(CC 23)



Photo 64

View of downstream side of embankment from outlet of auxiliary spillway exit channel.  
(CC 23)



Photo 65  
View of right groin from top of dam.  
(CC 23)



Photo 66  
Head cut into upstream sediment pool due to breach of dam.  
(CC 23)



Photo 67  
View of breach channel from downstream of dam.  
(CC 23)



Photo 68  
View of breach channel from upstream toe of dam.  
(CC 23)



Photo 69  
View of exposed bedrock in left abutment – breach channel.  
(CC 23)



Photo 70  
View of debris in breach inundation area downstream of dam.  
(CC 23)



Photo 71  
View of head cut into bedrock of breach channel.  
(CC 23)



Photo 72  
Close up view of head cut into bedrock of breach channel.  
(CC 23)

Ee Photo 74 for closeup view.



Photo 73

View of vertical valley relief fractures in exposed bedrock of breach channel.  
(CC 23)



Photo 74  
Close up view of vertical valley relief fracture.  
(CC 23)



Photo 75

View of vertical relief fractures from downstream of head cut in bedrock of breach channel  
(CC 23)

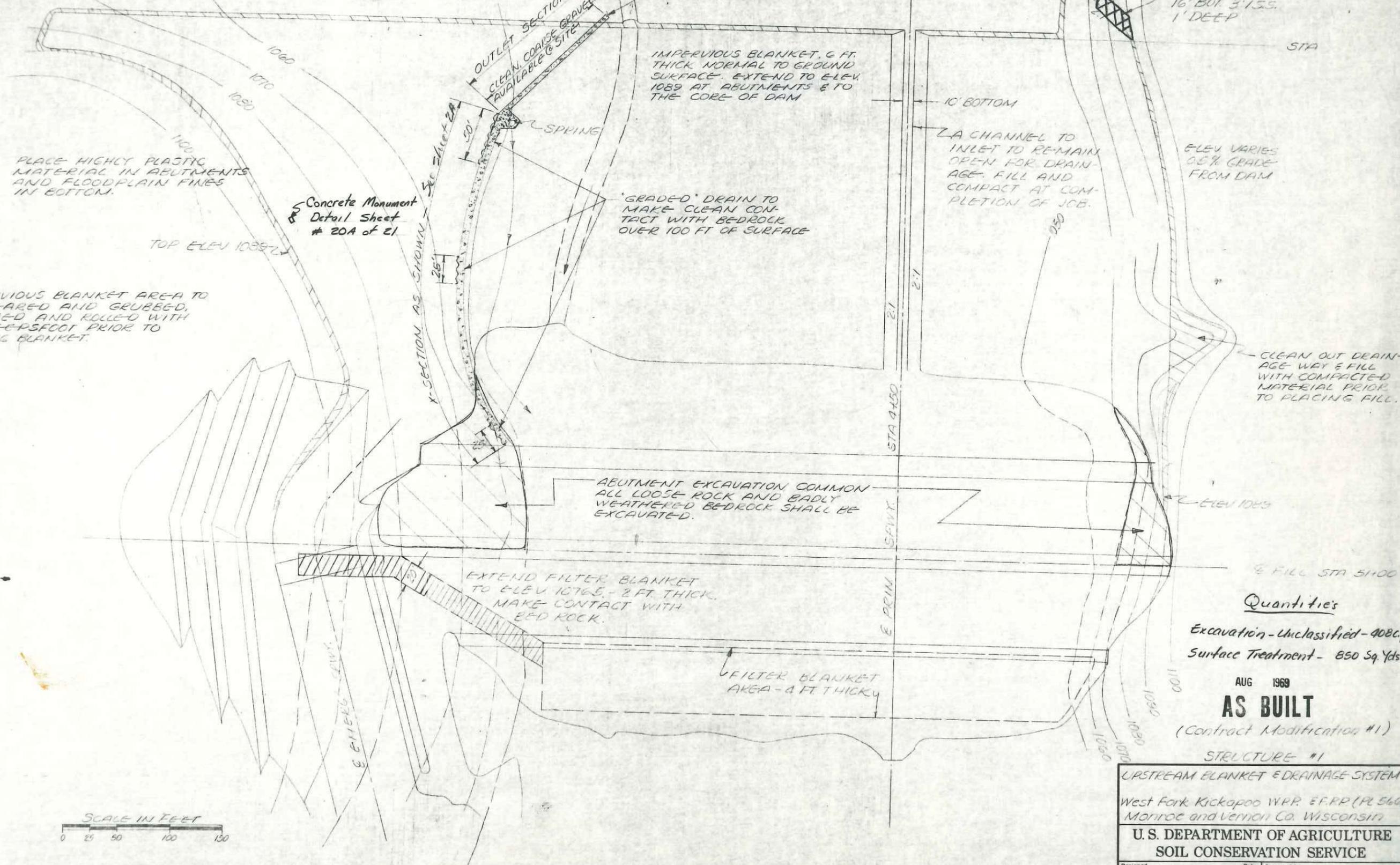
Attachment C  
Selected As-Built Drawings

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NOTE: SEE SHEET 2A FOR X-SECTIONS OF GRADED DRAIN AND OUTLET

OUTLET ABOVE BLANKET. DUMP 20 CY. GRAVEL OVER EXPOSED END AFTER COMPLETION OF BLANKET

SOD CHUTE 16' BOT 3' 155. 1' DEEP



PLACE HIGHLY PLASTIC MATERIAL IN ABUTMENTS AND FLOODPLAIN FINES IN BOTTOM.

Concrete Monument # 20A of 21

GRADED DRAIN TO MAKE CLEAN CONTACT WITH BEDROCK OVER 100 FT OF SURFACE

LA CHANNEL TO INLET TO REMAIN OPEN FOR DRAINAGE. FILL AND COMPACT AT COMPLETION OF JOB.

ELEV VARIES 0.5% GRADE FROM DAM

NOTE: IMPERVIOUS BLANKET AREA TO BE CLEARED AND GRUBBED, STRIPPED AND ROLLED WITH A SHEEPSFOOT PRIOR TO PLACING BLANKET.

ABUTMENT EXCAVATION COMMON ALL LOOSE ROCK AND BADLY WEATHERED BEDROCK SHALL BE EXCAVATED.

EXTEND FILTER BLANKET TO ELEV 10765. - 2 FT THICK. MAKE CONTACT WITH BED ROCK.

FILTER BLANKET AREA - 4 FT THICK

CLEAN OUT DRAINAGE WAY & FILL WITH COMPACTED MATERIAL PRIOR TO PLACING FILL.

Quantities

Excavation - Unclassified - 408cy.  
Surface Treatment - 850 Sq. Yds.

AUG 1969

**AS BUILT**

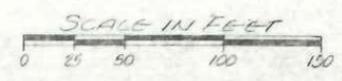
(Contract Modification #1)

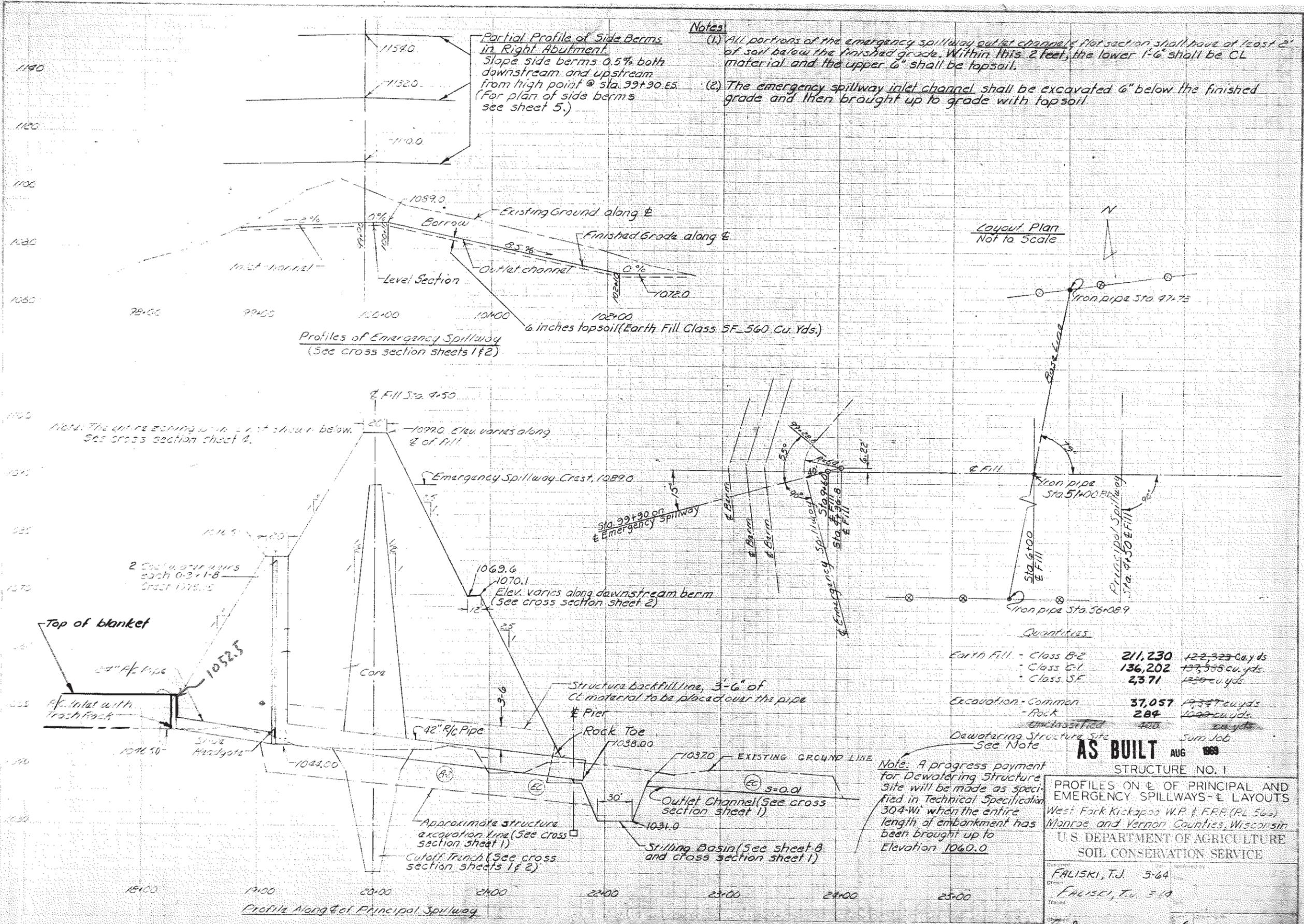
STRUCTURE #1

UPSTREAM BLANKET & DRAINAGE SYSTEM  
West Fork Kickapoo WRP. ET. PP. (PL 566)  
Monroe and Vernon Co. Wisconsin

U. S. DEPARTMENT OF AGRICULTURE  
SOIL CONSERVATION SERVICE

Designed <i>R. Eintzler</i>	Date 6-67	Approved by	Title
Drawn			
Traced <i>G. Lorene</i>	Date 6-67		Title
Checked <i>C. E. Dennis</i>	Date 6/2/67	Sheet No. 5A of 21	Drawing No. 5E-21299 P





- Notes**
- (1) All portions of the emergency spillway outlet channel flat section shall have at least 2' of soil below the finished grade. Within this 2 feet, the lower 1'-6" shall be CL material and the upper 6" shall be topsoil.
  - (2) The emergency spillway inlet channel shall be excavated 6" below the finished grade and then brought up to grade with topsoil.

*LAYOUT PLAN*  
Not to Scale

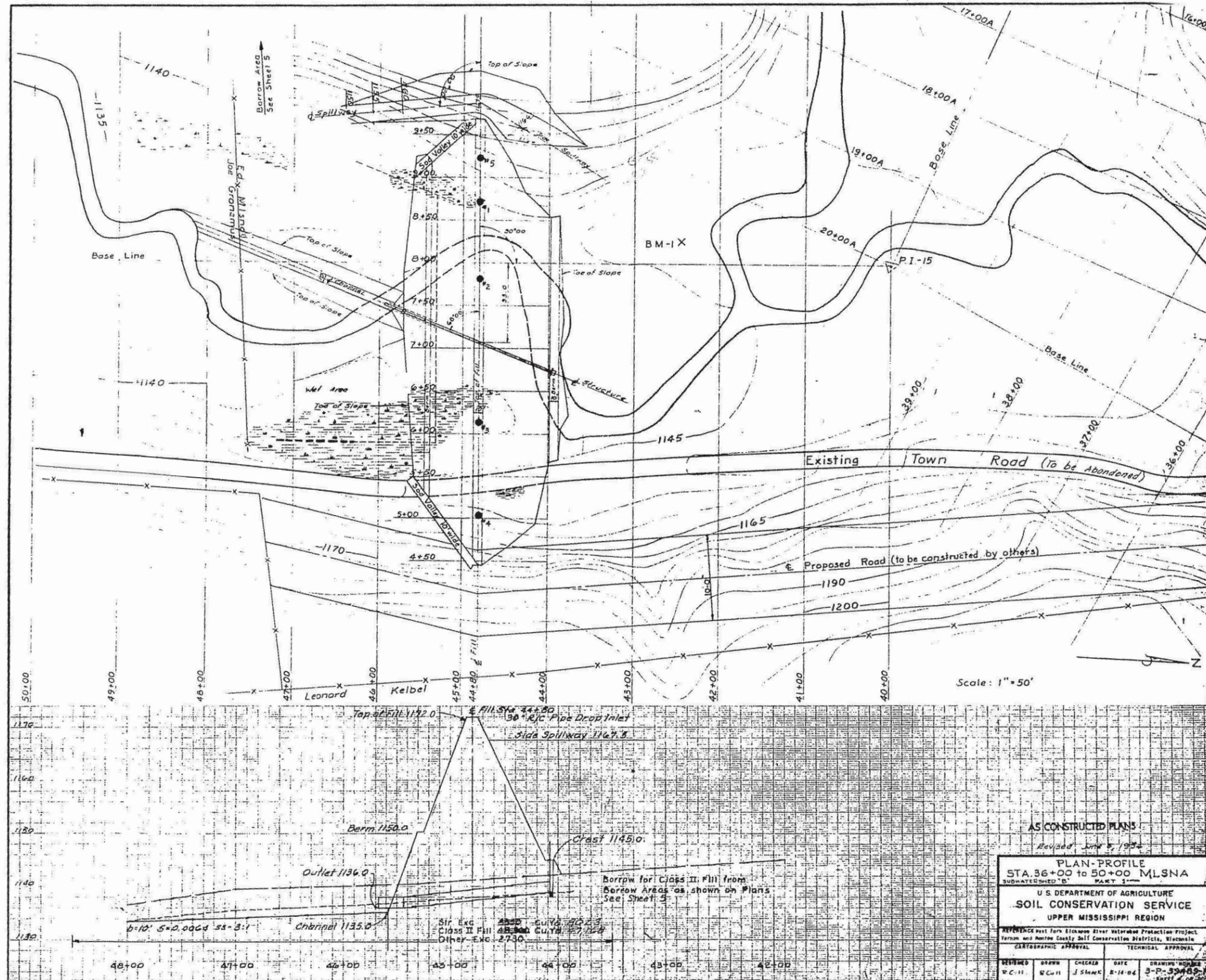
*Quantities*

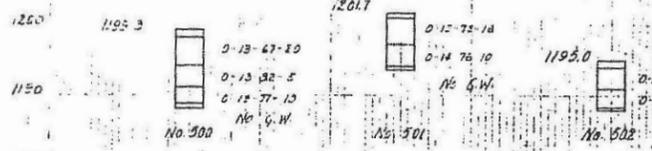
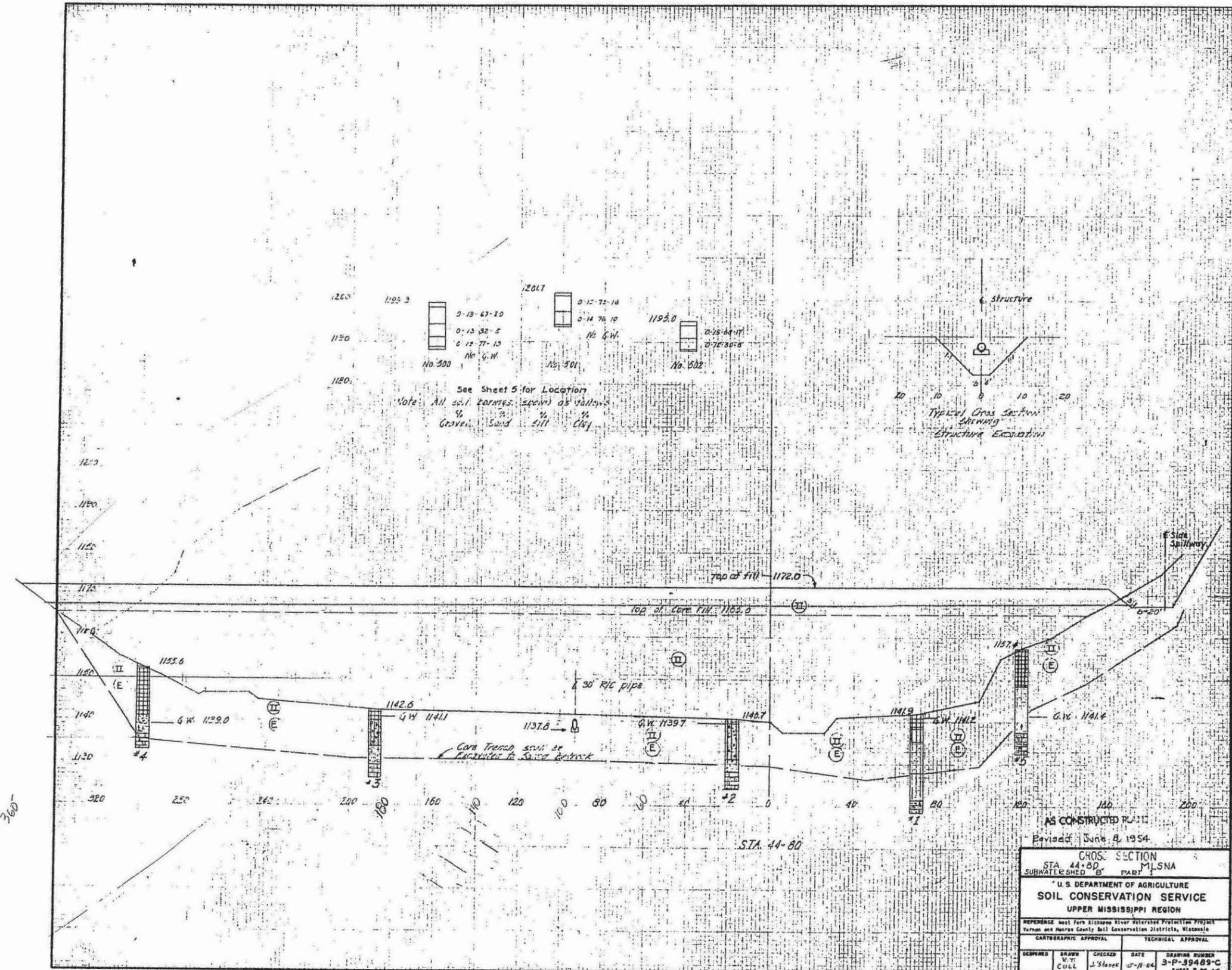
Earth Fill - Class B-2	211,230	122,523 cu. yds.
- Class C-1	136,202	137,558 cu. yds.
- Class SF	2,371	1,350 cu. yds.
Excavation - Common	37,057	19,347 cu. yds.
- Rock	284	1,000 cu. yds.
- Unclassified	400	cu. yds.
Dewatering Structure Site		Sum Job
- See Note		

*Note:* A progress payment for Dewatering Structure Site will be made as specified in Technical Specification 304-WI when the entire length of embankment has been brought up to Elevation 1060.0

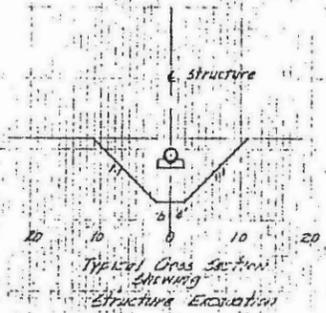
**AS BUILT** AUG 1969  
STRUCTURE NO. 1  
PROFILES ON E OF PRINCIPAL AND EMERGENCY SPILLWAYS - E LAYOUTS  
West Fork Kickapoo W.P. & F.F.F. (R.L. 560)  
Monroe and Vernon Counties, Wisconsin  
U.S. DEPARTMENT OF AGRICULTURE  
SOIL CONSERVATION SERVICE

Date: FALISKI, T.J. 3-64  
Drawn: FALISKI, T.J. 3-64  
Title: *AS BUILT*  
1 of 21 5-E-21,299-P

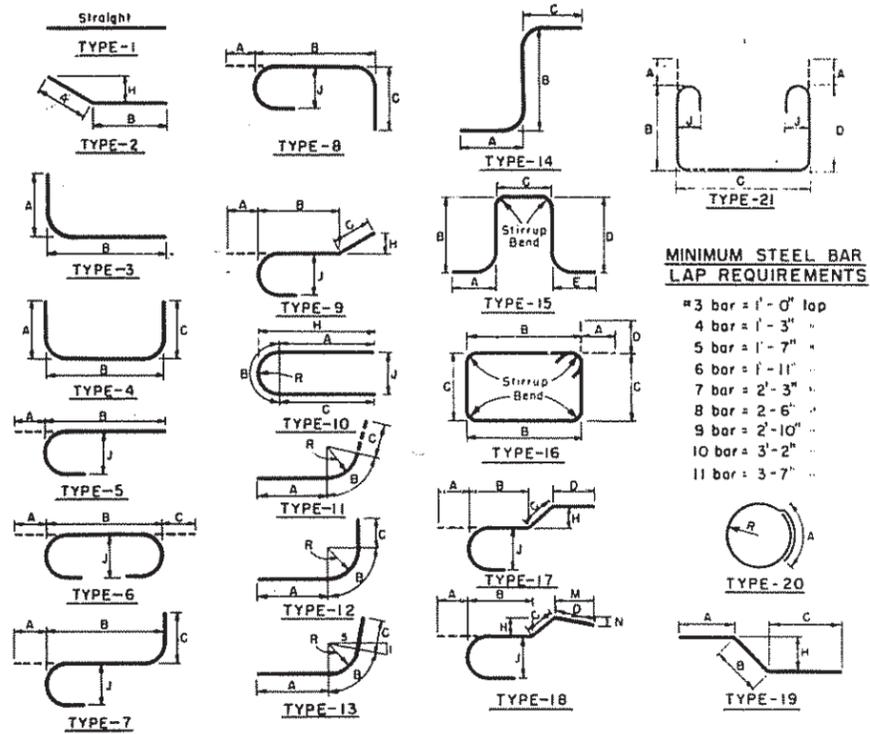




See Sheet 5 for Location  
 Note: All soil samples shown as follows:  
 % Gravel, % Sand, % Silt, % Clay



AS CONSTRUCTED PLAN				
Revised June 8, 1954				
CROSS SECTION		STA 44+80 M.S.N.A.		
SUBWATER SHED B		PART		
U.S. DEPARTMENT OF AGRICULTURE				
SOIL CONSERVATION SERVICE				
UPPER MISSISSIPPI REGION				
REFERENCE West Fork Arkansas River Watershed Protection Project Yarman and Monroe County Soil Conservation Districts, Missouri				
GEOGRAPHIC APPROVAL		TECHNICAL APPROVAL		
DRAWN W.T. CULL	CHECKED J. Slapak	DATE 5-11-54	DRAWING NUMBER 3-5-39489-C SHEET 2 OF 15	

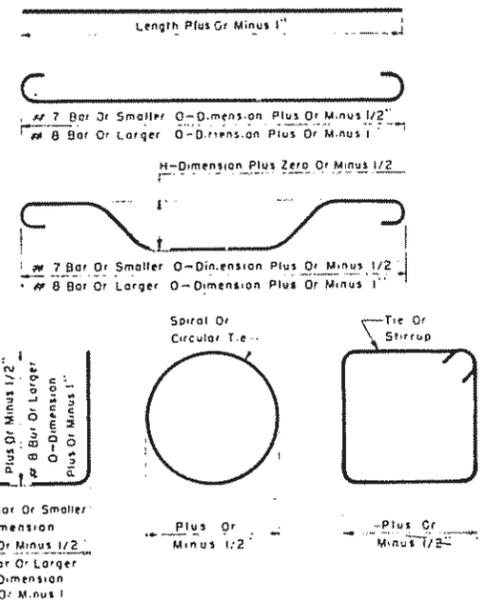


**MINIMUM STEEL BAR LAP REQUIREMENTS**

- #3 bar = 1'-0" lap
- 4 bar = 1'-3"
- 5 bar = 1'-7"
- 6 bar = 1'-11"
- 7 bar = 2'-3"
- 8 bar = 2'-6"
- 9 bar = 2'-10"
- 10 bar = 3'-2"
- 11 bar = 3'-7"

NOTE: All bends not otherwise specified shall conform to the proposed standard A C I code, recommended sizes. All dimensions are measured from outside to outside of bar, except where otherwise specified. It shall be the responsibility of the fabricator to assign a code number to bars for each structure and parts thereof.

**BAR TYPE DETAILS**  
Revised 1-22-54



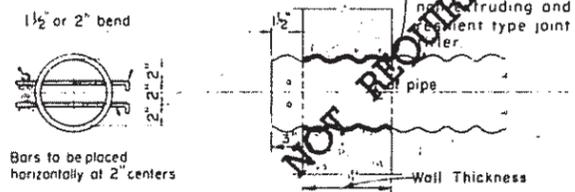
NOTE: Tolerances for outside to outside dimensions of bar types not shown shall conform to the tolerances of similar bar types and specified bar sizes as given in Manual of Standard Practice for Detailing Reinforced Concrete Structures A C I 315.

**TYPICAL PERMISSIBLE TOLERANCES**  
9-22-59

BILL OF MATERIAL			
Pipe Diam	Bar Size	Quantity	Length
6"	#3 or #4	2	1'-0"
8"	#3 or #4	3	1'-3"
10"	#3 or #4	4	1'-6"

All holes in pipe to receive bars shall be 9/16" dia

Drill 1/8" hole 1" from end of bars to receive nail, wire or cotter pin

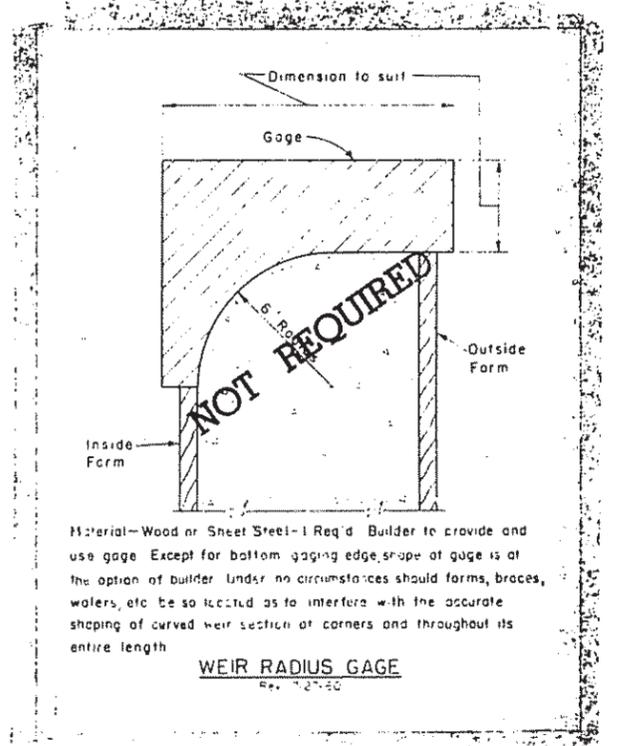


Bars to be placed horizontally at 2" centers

*Used Flap Gate Rat Guard*

**DETAILS OF ANIMAL GUARD**

Scale 1/2" = 1'-0"

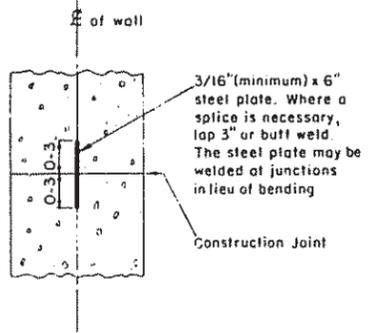


Material—Wood or Sheet Steel—1 Req'd. Builder to provide and use gage. Except for bottom gaging edge, shape of gage is at the option of builder. Under no circumstances should forms, braces, walers, etc. be so located as to interfere with the accurate shaping of curved weir section at corners and throughout its entire length.

**WEIR RADIUS GAGE**  
Rev. 7-27-50

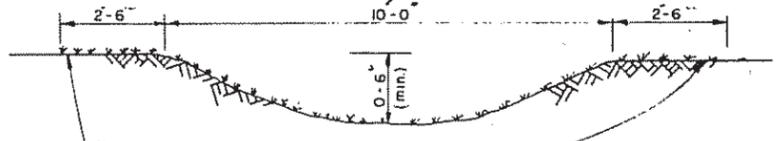
Note: Metal plate shall be free from dirt, oil, grease, paint, mill scale, loose or thick rust, or other coating which might destroy or reduce its bond with concrete.

Metal plate and installation to be included with unit price for concrete.



**DETAIL OF METAL PLATE TYPE CONSTRUCTION JOINT**

Scale 1/2" = 1'-0" Rev 9-2-59



Note: These extensions spaced as described in 520.5 Technical Specification 520

**DETAIL OF SOD GUTTER**

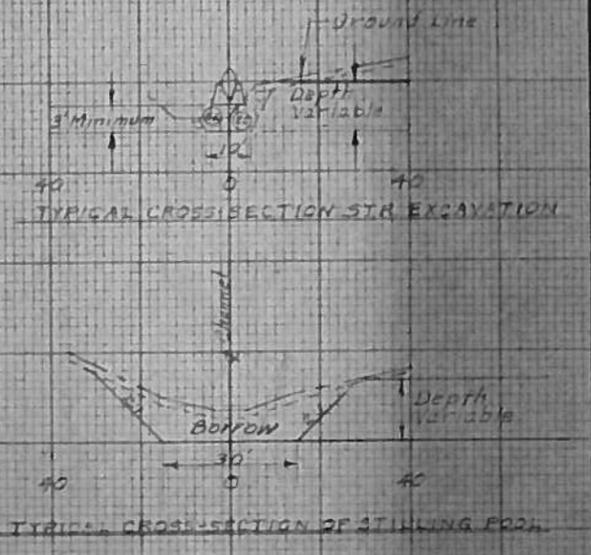
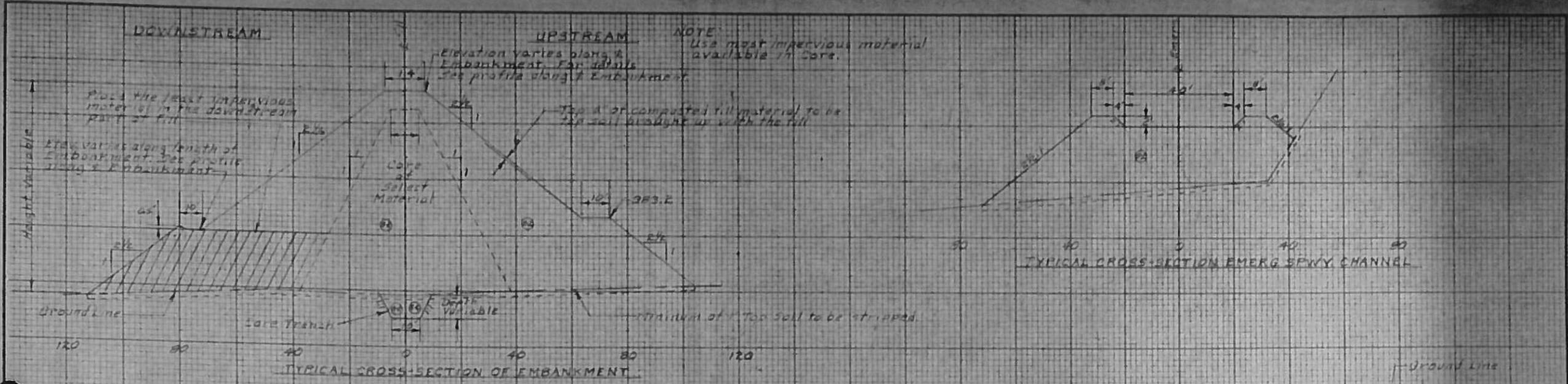
**STRUCTURES 3 AND 12**

**STANDARD DETAILS**

W. Fork Kickapoo W.P. and F.P.P. (PL 566) Monroe and Vernon Counties, Wis.

U. S. DEPARTMENT OF AGRICULTURE  
SOIL CONSERVATION SERVICE

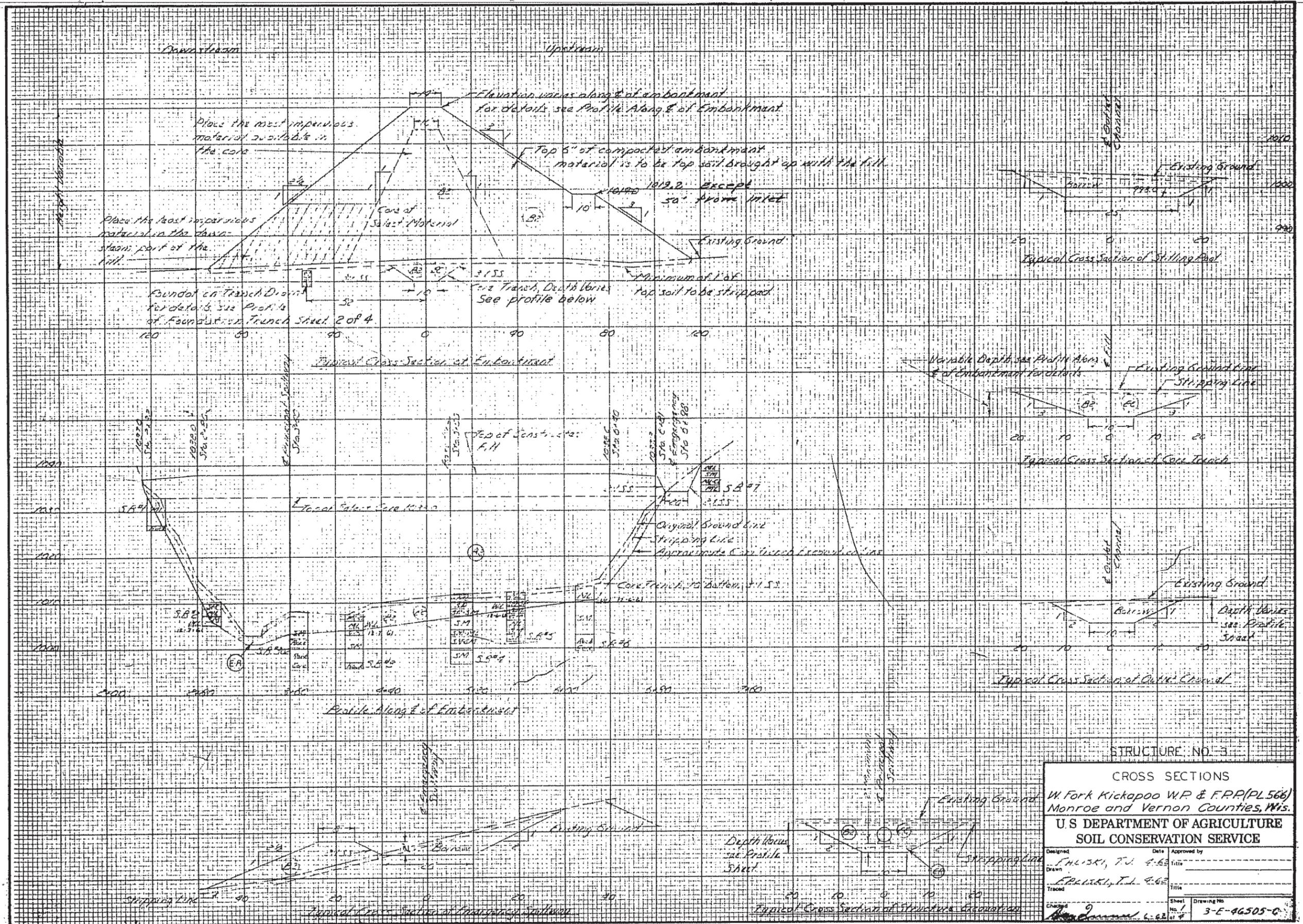
Designed.....	Date.....	Approved By.....
Drawn.....		Title.....
Traced.....		Scale.....
Checked.....		Sheet No. 18
		Drawing No. 3-E-46505-P



**AS BUILT**

STRUCTURE NO 21  
 PROFILE ALONG E EMBANKMENT  
 AND TYPICAL CROSS SECTIONS  
 Coon Creek Watershed Protection and  
 Flood Prevention Project, Monroe Co., Wis.  
 U. S. DEPARTMENT OF AGRICULTURE  
 SOIL CONSERVATION SERVICE

Designed	Date	Approved by
Drawn	12-69	Title
Checked	12-69	Title
Checked	0	Sheet
1-61	1	3-E-46088-C

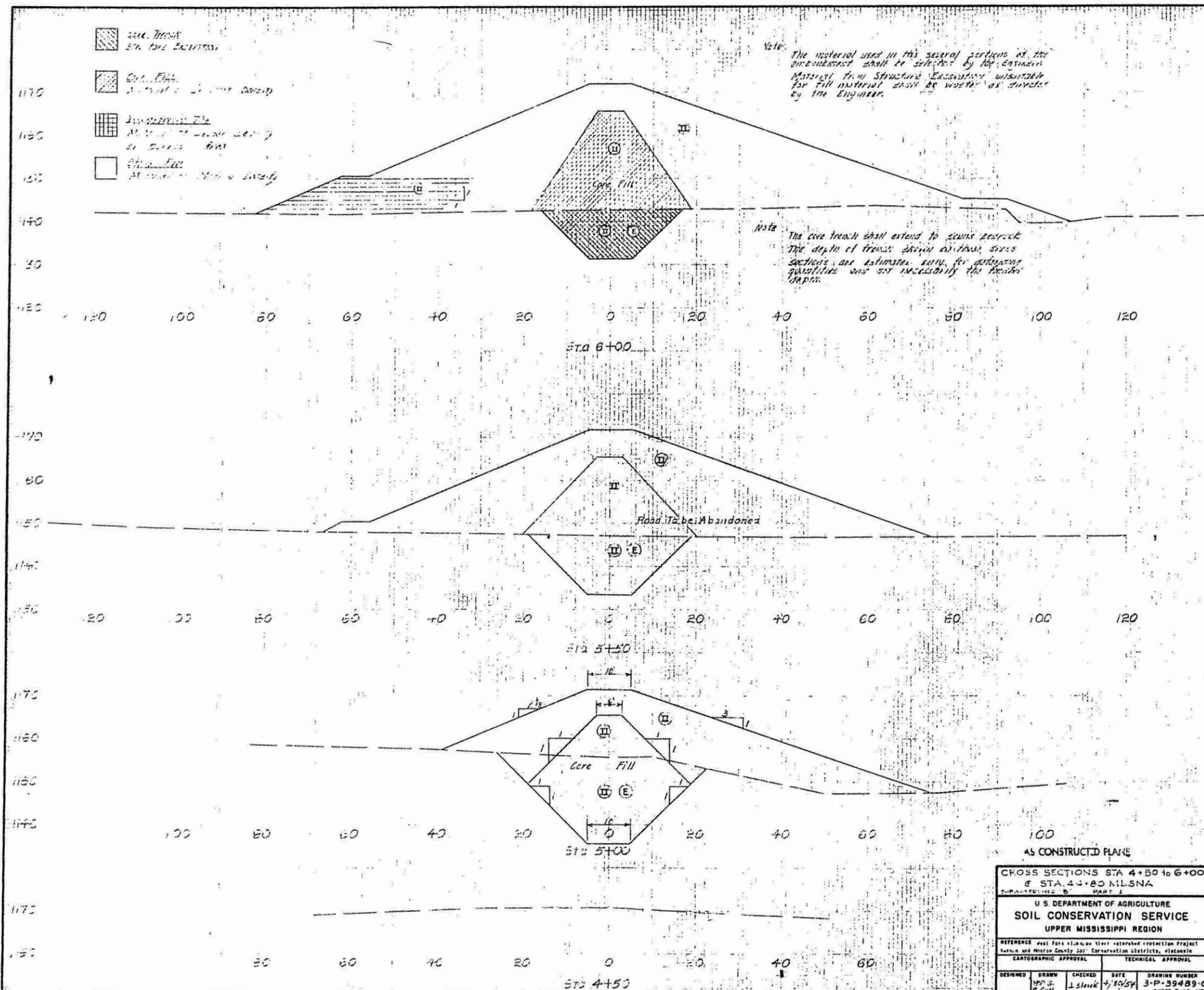


STRUCTURE NO. 3

**CROSS SECTIONS**  
 W. Fork Kickapoo W.P. & F.P.P. (PL 566)  
 Monroe and Vernon Counties, Wis.

**U.S. DEPARTMENT OF AGRICULTURE**  
**SOIL CONSERVATION SERVICE**

Designed F. L. ISKI, T. J. 4-62	Date 4-62
Drawn F. L. ISKI, T. J. 4-62	Approved by Title
Traced	Title
Checked J. J. [Signature]	Sheet No. 1 of 4
	Drawing No. 3-E-46505-C



45 CONSTRUCTED PLANS

CROSS SECTIONS STA 4+50 to 6+00  
 @ STA. 4+50 NLSNA

U.S. DEPARTMENT OF AGRICULTURE  
 SOIL CONSERVATION SERVICE  
 UPPER MISSISSIPPI REGION

REFERENCE: Soil Erosion Control Manual, 1954, Chapter 10, Section 10.1, 10.2, 10.3, 10.4, 10.5, 10.6, 10.7, 10.8, 10.9, 10.10, 10.11, 10.12, 10.13, 10.14, 10.15, 10.16, 10.17, 10.18, 10.19, 10.20, 10.21, 10.22, 10.23, 10.24, 10.25, 10.26, 10.27, 10.28, 10.29, 10.30, 10.31, 10.32, 10.33, 10.34, 10.35, 10.36, 10.37, 10.38, 10.39, 10.40, 10.41, 10.42, 10.43, 10.44, 10.45, 10.46, 10.47, 10.48, 10.49, 10.50, 10.51, 10.52, 10.53, 10.54, 10.55, 10.56, 10.57, 10.58, 10.59, 10.60, 10.61, 10.62, 10.63, 10.64, 10.65, 10.66, 10.67, 10.68, 10.69, 10.70, 10.71, 10.72, 10.73, 10.74, 10.75, 10.76, 10.77, 10.78, 10.79, 10.80, 10.81, 10.82, 10.83, 10.84, 10.85, 10.86, 10.87, 10.88, 10.89, 10.90, 10.91, 10.92, 10.93, 10.94, 10.95, 10.96, 10.97, 10.98, 10.99, 10.100.

DESIGNED		DRAWN		CHECKED		DATE		DRAWING NUMBER	
J. S. R. Cull		J. S. R. Cull		J. S. R. Cull		4/29/54		3-P-39489-C	
CARTOGRAPHIC APPROVAL					TECHNICAL APPROVAL				

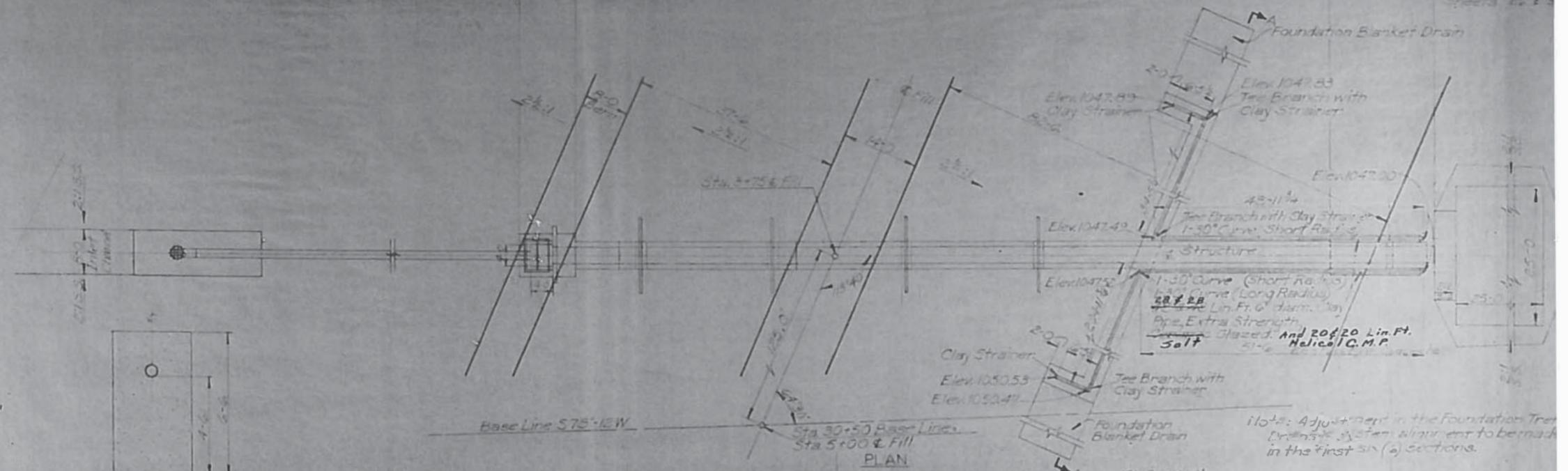
SHEET 9 OF 15



For Details See Sheets 32 & 33

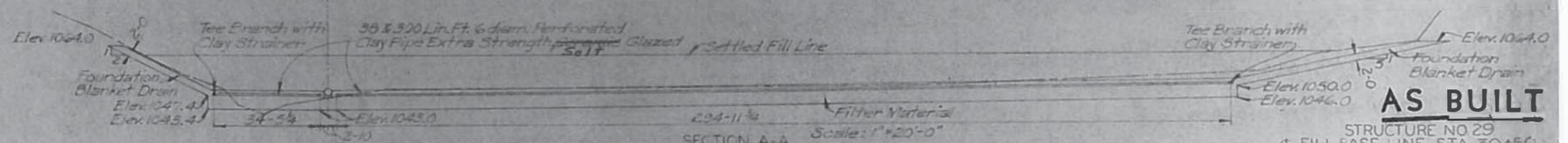
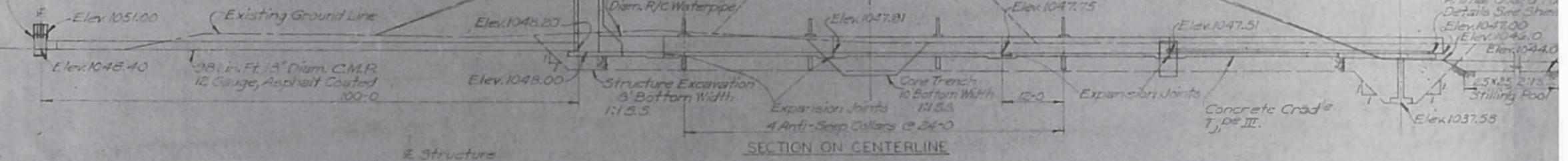
For Details See Sheets 29 & 30

For Details See Sheets 34 & 35



CROSS SECTION OF FOUNDATION TRENCH DRAIN FOR 6\"/>

Inlet Assembly For Details See Sheet 35



**AS BUILT**

STRUCTURE NO 29 & FILL BASE LINE STA 30+50

R/C DROP, INLET 36\"/>

GENERAL LAYOUT

Soon Creek Watershed Protection & Flood Prevention Project (PL 566) WISCONSIN

U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE

Designed	Date	Approved by
Drawn	File	
Checked	File	
Trace	File	
Issue	File	
Revised	File	

QUANTITIES	
Concrete - Class B, Air Entrained	57.3 Cu.Yds
Steel Bar Reinforcement	226.2 Pounds
Pipe Conduit - Reinforced Concrete Water Pipe, 36\"/>	16.0 Lin.Ft.
Structure Drainage - 24\"/>	4.1 Lin.Ft.
" - 18\"/>	98.9 Lin.Ft.
" - 6\"/>	342.3 Lin.Ft.
" - 6\"/>	60.5 Lin.Ft.
" - Filter Material - Pit Run Sand and Gravel	358.0 Cu.Yds
Trash Racks and Guard Rails	40.0 Lin.Ft.

Structure Drainage - Helical C.M.P. - 6\"/>

Scale: 1\"/>

Attachment D  
O&M Inspection Photos

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Coon Creek 21 (Luckason)  
Monroe County  
5/18/16



0036305181607  
Embankment frontslope



0036305181608  
Embankment backslope



0036305181611  
Auxiliary spillway crest looking downstream



0036305181612  
Auxiliary spillway crest looking upstream

Coon Creek 23 (Bilhovde)  
Monroe County  
5/18/16



0036105181607  
Embankment frontslope



0036105181608  
Embankment backslope



0036105181611  
Auxiliary spillway crest looking downstream



0036105181612  
Auxiliary spillway crest looking upstream

Coon Creek 29 (Korn)  
Monroe County  
4/26/12



0035804262012\_10  
Embankment Backslope



0035804262012\_08  
Embankment Frontslope



0035804262012\_01  
Auxiliary Spillway Exit Channel



0035804262012\_19  
Auxiliary Spillway Inlet Channel

West Fork Kickapoo MIsna  
Vernon County  
5/03/18



00393050318\_11  
Embankment Frontslope



00393050318\_12  
Embankment Backslope



00393050318\_07  
Auxiliary spillway looking downstream



00393050318\_08  
Auxiliary spillway looking upstream

West Fork Kickapoo 1  
Vernon County  
7/22/15



Embankment Frontslope



Embankment Backslope



Auxiliary Spillway Looking Downstream



Seepage from Right Abutment Drains

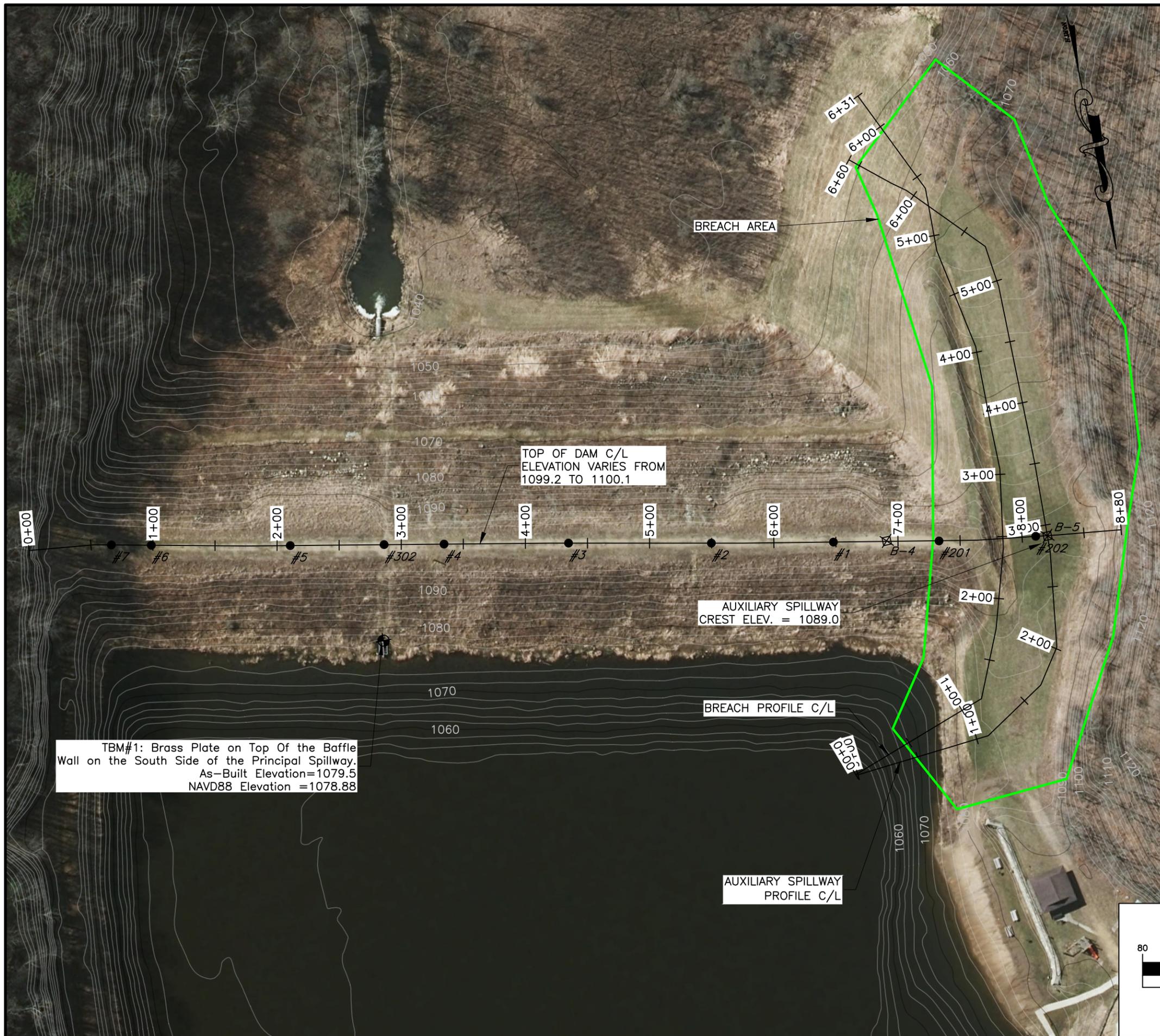


Seepage from Right Abutment Bedrock

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Attachment E  
Investigation Survey Drawings and Photos

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TBM#1: Brass Plate on Top Of the Baffle Wall on the South Side of the Principal Spillway. As-Built Elevation=1079.5 NAVD88 Elevation =1078.88

TOP OF DAM C/L ELEVATION VARIES FROM 1099.2 TO 1100.1

AUXILIARY SPILLWAY CREST ELEV. = 1089.0

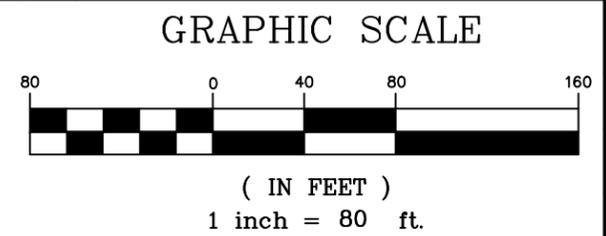
BREACH PROFILE C/L

AUXILIARY SPILLWAY PROFILE C/L



- LEGEND**
- LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
  - TYPICAL SOIL BORING LOCATION & NUMBER #1 (Approximate Location)
  - TYPICAL ROCK CORE BORING LOCATION & NUMBER B-5 (Approximate Location)

**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

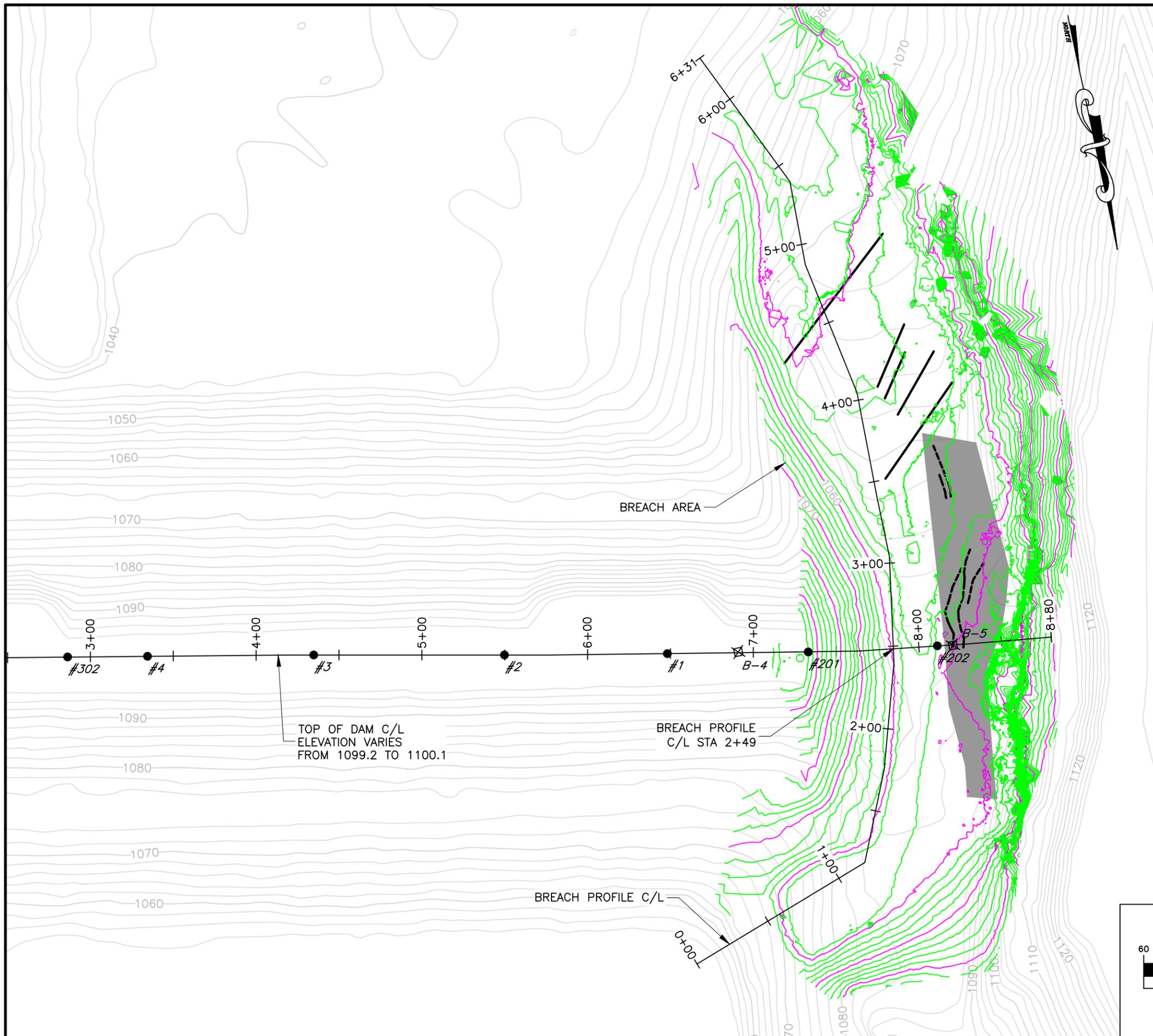


Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

**WEST FORK #1 - JERSEY VALLEY  
 PLAN VIEW - OVERALL**  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON

United States Department of Agriculture  
**Natural Resources Conservation Service**

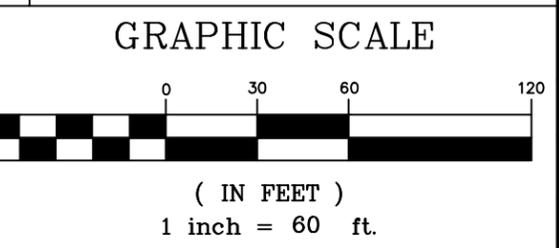
File Name	WI-006
Date	08/14
Sheet	1 of 24



**LEGEND**

- 1060 — LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- 1070 — POST-BREACH GROUND CONTOURS
- #1 — TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)
- ⊠ #B-5 — TYPICAL ROCK CORE BORING LOCATION & NUMBER (Approximate Location)
- — ESTIMATED VERTICAL FRACTURES EXTENDING INTO THE ABUTMENT (Most Prevalent Fractures Shown Based on Post-Breach Exposed Abutment)
- - - ESTIMATED VERTICAL VALLEY RELIEF FRACTURES (Most Prevalent Fractures Shown Based on Post-Breach Exposed Abutment)
- — ESTIMATED ZONE OF VERTICAL VALLEY RELIEF FRACTURES. (Based on Post-Breach Exposed Abutment)

**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 VERTICAL VALLEY RELIEF FRACTURES ARE GENERALLY PARALLEL TO THE VALLEY. VERTICAL FRACTURES ARE GENERALLY NOT PARALLEL TO THE VALLEY AND EXTEND INTO THE BREACH ABUTMENT.  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT



Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

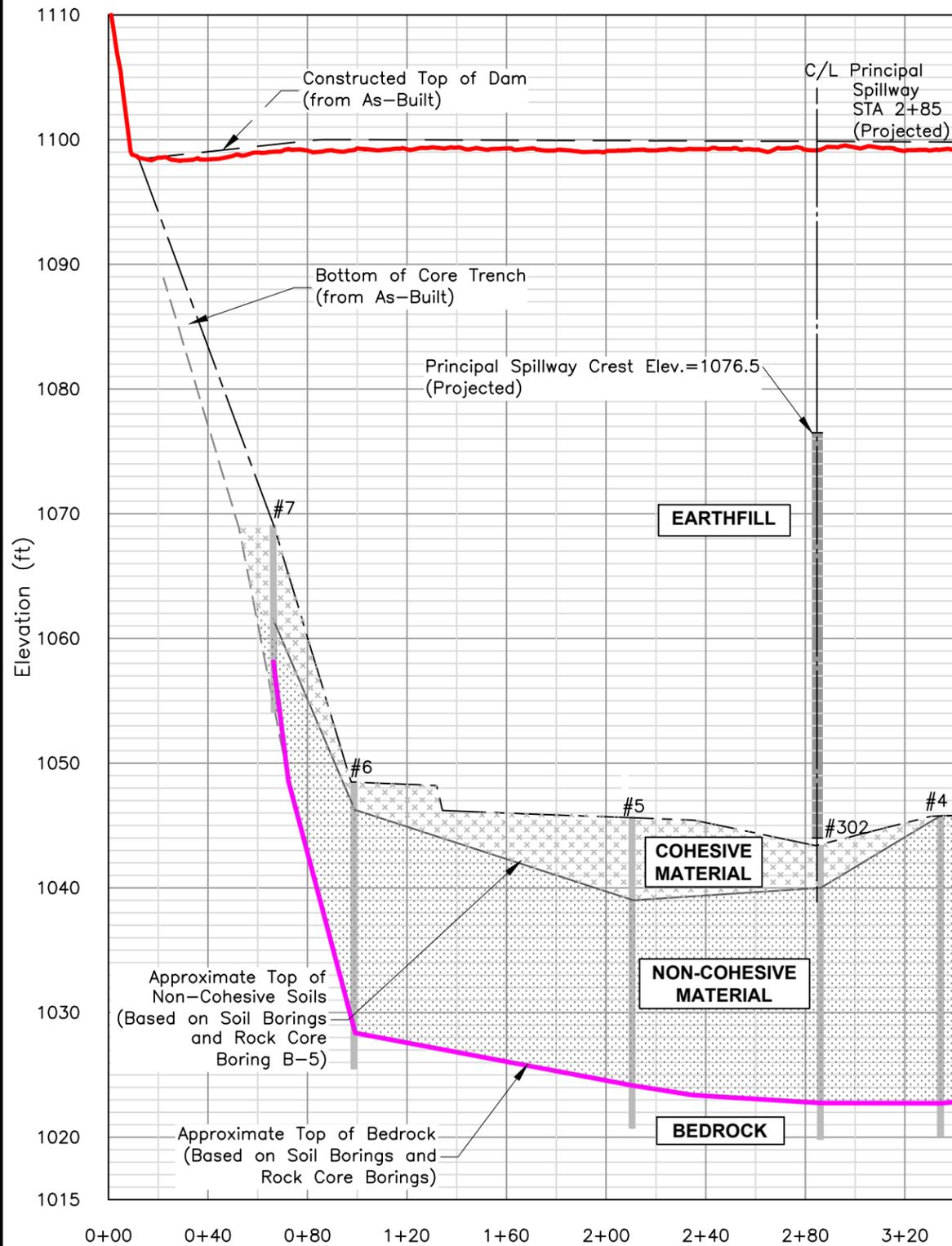
**WEST FORK #1 - JERSEY VALLEY PLAN VIEW  
 BREACH AREA**

OWNER: VERNON COUNTY  
 COUNTY: VERNON

United States  
 Department of  
 Agriculture  
**USDA**  
 Natural Resources  
 Conservation Service

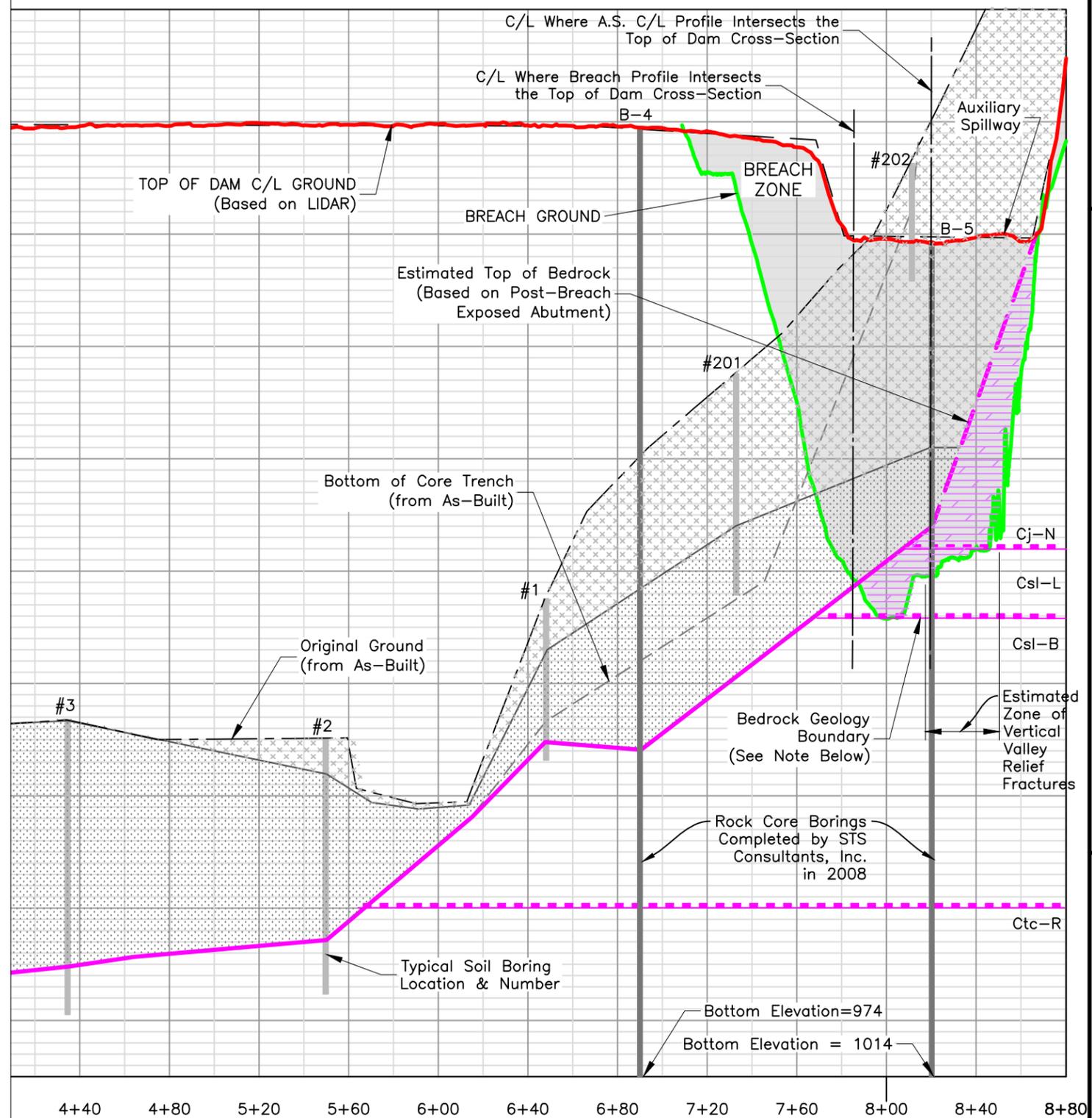
File Name	WI-006
Date	08/14
Sheet 2 of 24	

CROSS-SECTION - TOP OF DAM CENTERLINE STA 0+00 TO 3+40 @ STA 2+49  
OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM)



5:1 VERTICAL EXAGGERATION

CROSS-SECTION - TOP OF DAM CENTERLINE STA 4+07 TO 8+80 @ STA 2+49  
OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM)



Station (ft)

**NOTES:**

ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

**BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)**

Cj-N : Norwalk Member of the Jordan Sandstone (Case Hardened)

Csl-L: Lodi Member of the St. Lawrence Siltstone (Highly Fractured)

Csl-B: Black Earth Member of the St. Lawrence Dolostone (Aquiclude)

Ctc-R: Reno member of the Tunnel City Formation (Glaucconitic Sandstone)

Date 5/2019  
Designed JEV  
Drawn  
Checked  
Approved

WEST FORK #1 - JERSEY VALLEY  
TOP OF DAM C/L CROSS SECTION

OWNER: VERNON COUNTY  
COUNTY: VERNON



File Name

WI-006

Date

08/14

Sheet 3 of 24

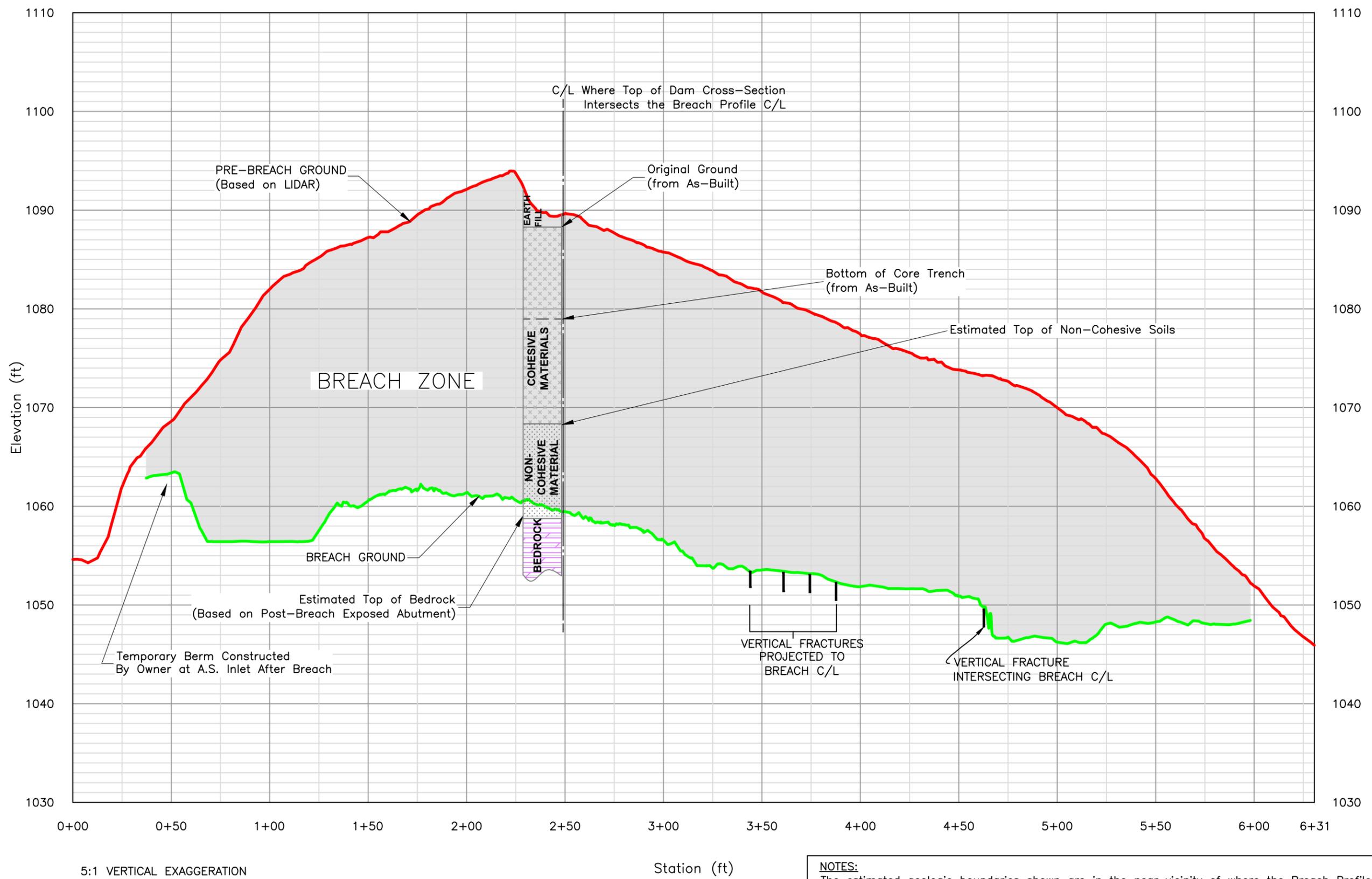
Date 5/2019  
 Designed JEV  
 Drawn  
 Checked  
 Approved

WEST FORK #1 - JERSEY VALLEY  
 PROFILE - C/L OF BREACH  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON



File Name WI-006  
 Date 08/14  
 Sheet 4 of 24

PROFILE CENTERLINE OF BREACH (AS-BUILT DATUM)

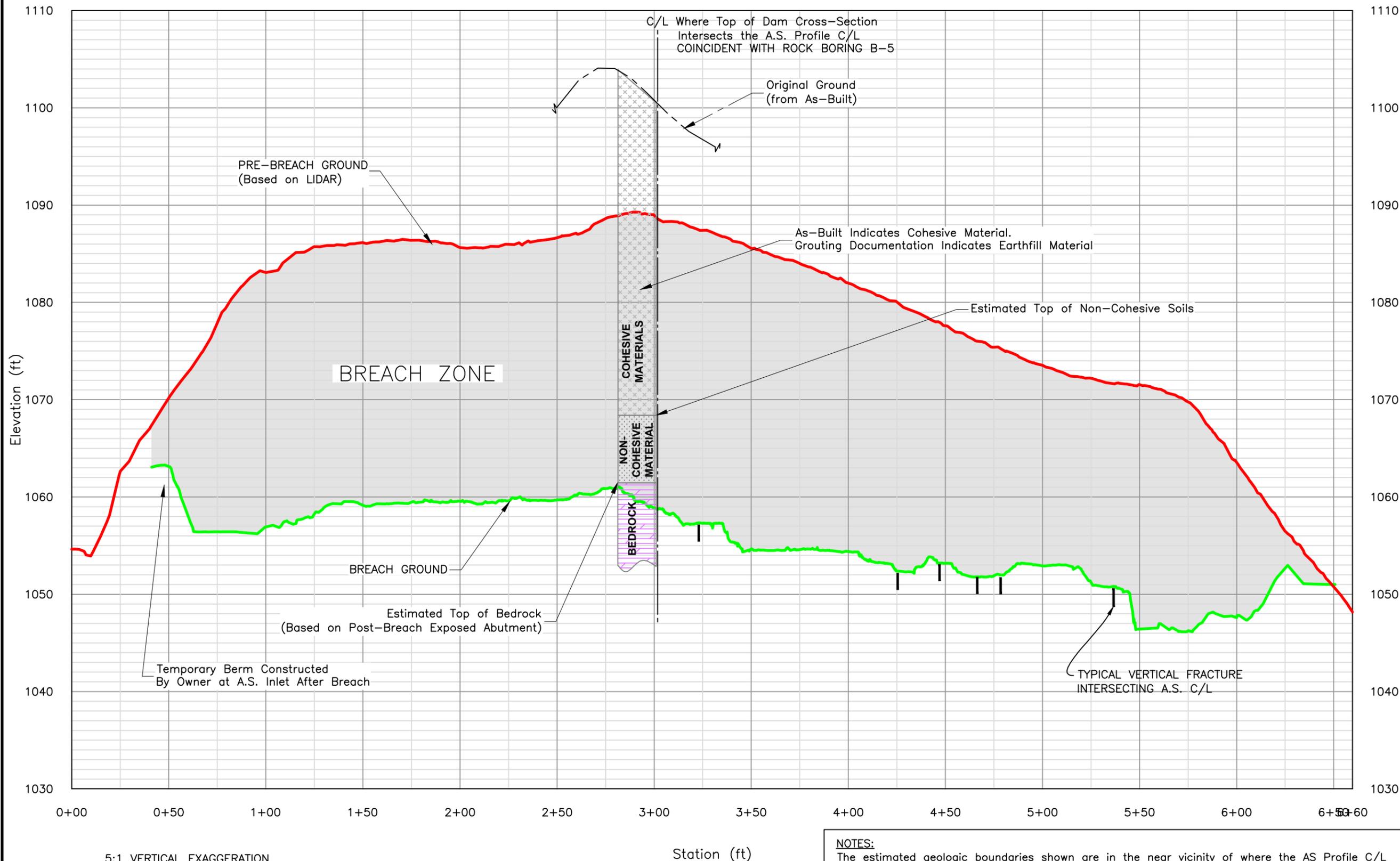


5:1 VERTICAL EXAGGERATION

Station (ft)

**NOTES:**  
 The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.  
 Vertical Extent of Vertical Fractures is Unknown

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS-BUILT DATUM)



5:1 VERTICAL EXAGGERATION

Station (ft)

**NOTES:**  
 The estimated geologic boundaries shown are in the near vicinity of where the AS Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.  
 Vertical Extent of Vertical Fractures is Unknown

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

WEST FORK #1 - JERSEY VALLEY  
 PROFILE - C/L OF AUXILIARY SPILLWAY  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON



File Name	WI-006
Date	08/14
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**KNAPP VALLEY ROAD**

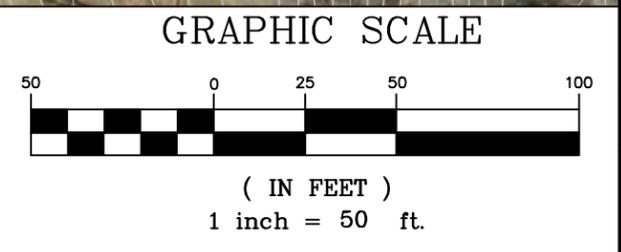
TOP OF DAM C/L  
ELEVATION VARIES FROM  
1170.5 to 1171.4

AUXILIARY SPILLWAY  
CREST ELEV. = 1167.5

TBM#1: Crest of the SW Corner  
of the Principal Spillway on Top  
of Metal Rack Bolted to Principal Spillway.  
As-Built Elevation=1145.0  
NAVD88 Elevation =1135.26

**NOTES:**  
ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
  
ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

**LEGEND**  
 1060 — LIDAR GROUND CONTOUR  
(Approximate Pre-Breach Condition)  
 ● #1 TYPICAL SOIL BORING LOCATION & NUMBER  
(Approximate Location)

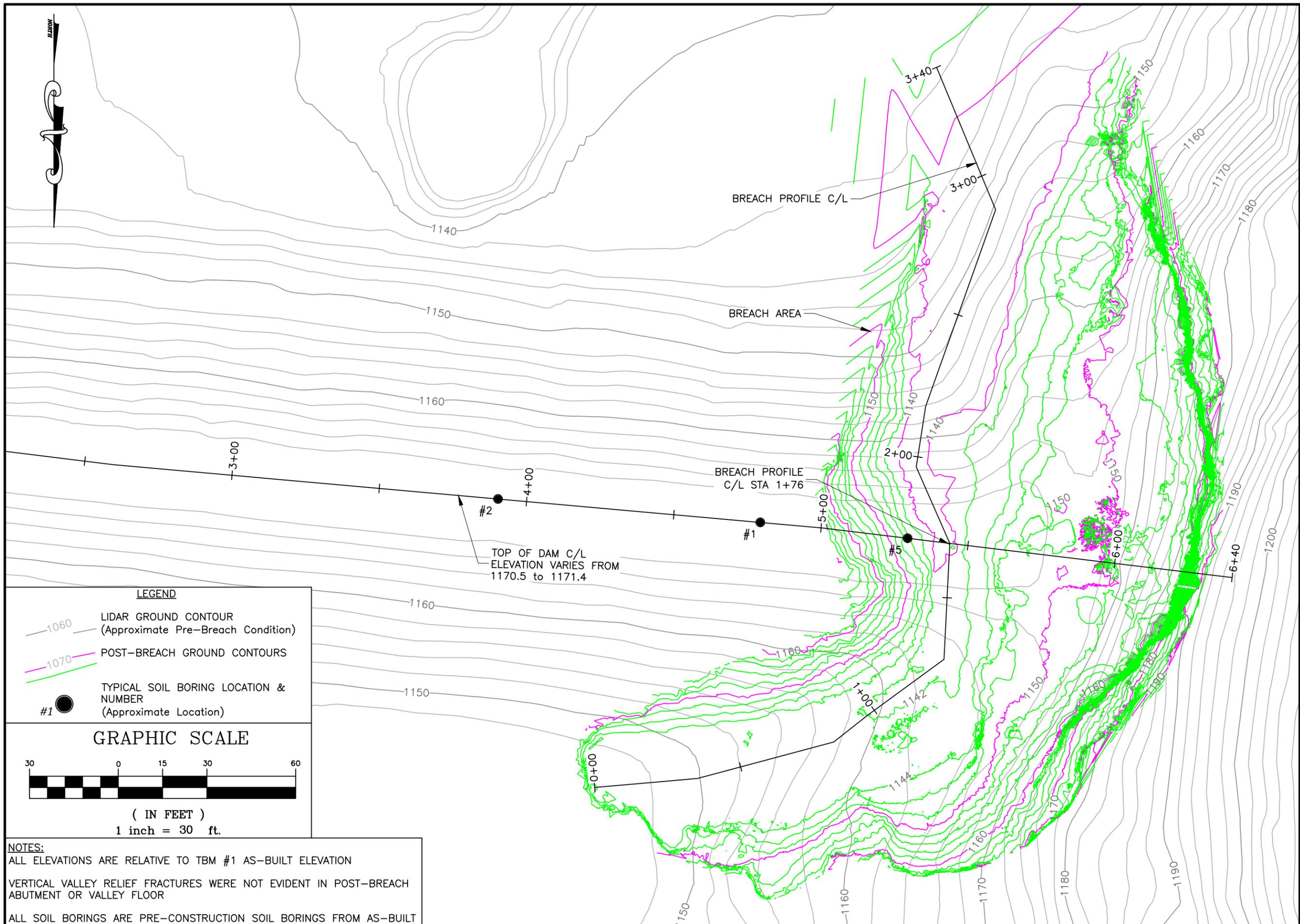


Designed	JEV	Date	4/2019
Drawn			
Checked			
Approved			

**MLSNA PLAN VIEW - OVERALL**  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON

United States  
Department of  
Agriculture  
**Natural Resources  
Conservation Service**

File Name  
WI-006  
Date  
08/14  
Sheet 6 of 24



**LEGEND**

-  LIDAR GROUND CONTOUR  
(Approximate Pre-Breach Condition)
-  POST-BREACH GROUND CONTOURS
-  TYPICAL SOIL BORING LOCATION & NUMBER  
(Approximate Location)

**GRAPHIC SCALE**



( IN FEET )  
1 inch = 30 ft.

**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 VERTICAL VALLEY RELIEF FRACTURES WERE NOT EVIDENT IN POST-BREACH ABUTMENT OR VALLEY FLOOR  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

Designed	JEV	Date	4/2019
Drawn			
Checked			
Approved			

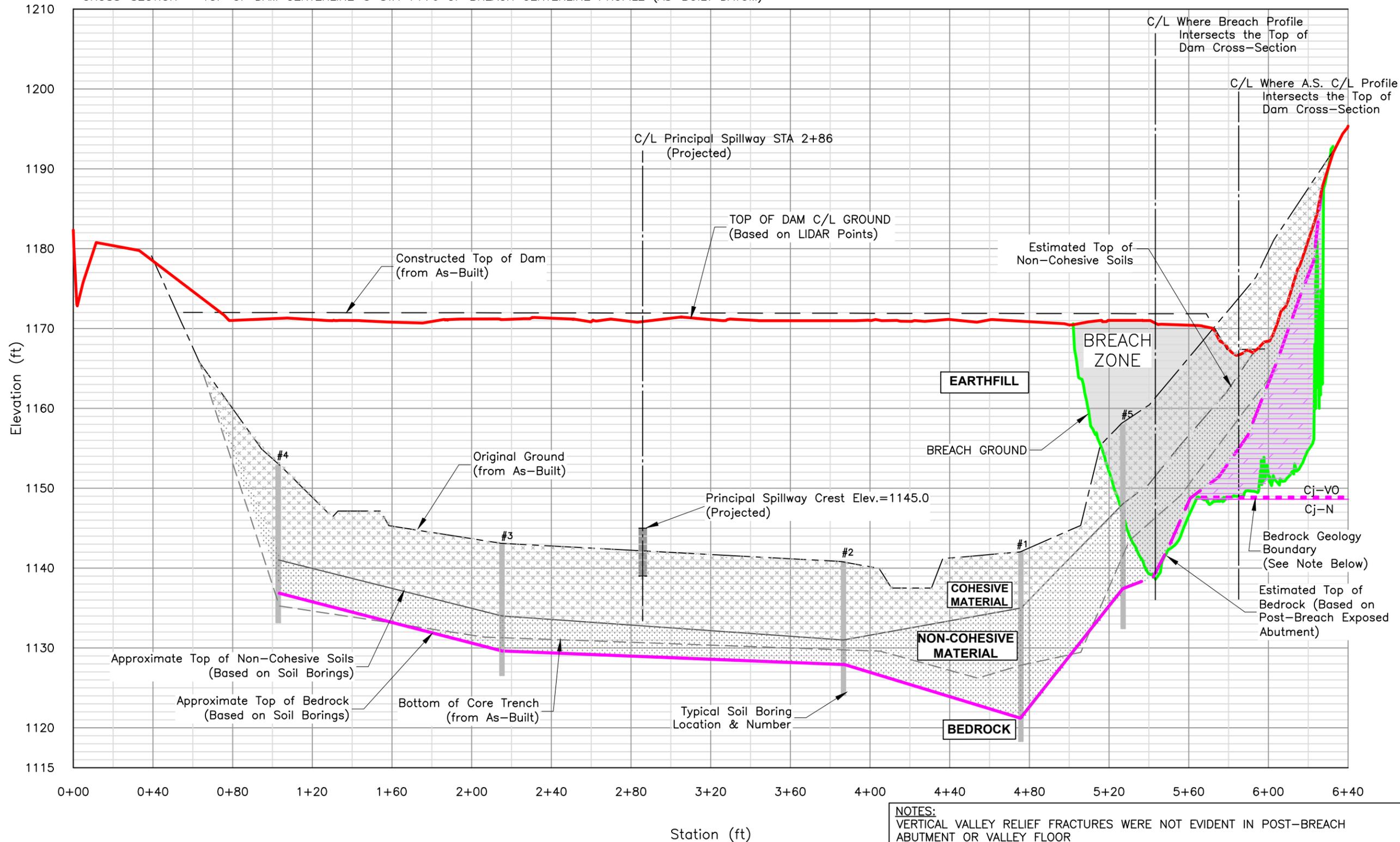
**MLSNA PLAN VIEW - BREACH AREA**

OWNER: VERNON COUNTY  
 COUNTY: VERNON



File Name	WI-006
Date	08/14
Sheet	7 of 24

CROSS-SECTION - TOP OF DAM CENTERLINE @ STA 1+76 OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

**NOTES:**  
 VERTICAL VALLEY RELIEF FRACTURES WERE NOT EVIDENT IN POST-BREACH ABUTMENT OR VALLEY FLOOR  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT  
BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)  
 Cj-V0: Van Oser Member of the Jordan Sandstone (Soft, Poorly Cemented)  
 Cj-N: Norwalk Member of the Jordan Sandstone (Case Hardened)

Date	4/2019
Designed	JEV
Drawn	
Checked	
Approved	

MLSNA  
**TOP OF DAM C/L CROSS-SECTION**  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON

United States Department of Agriculture  
**USDA**  
 Natural Resources Conservation Service

File Name	WI-006
Date	08/14
Sheet	8 of 24

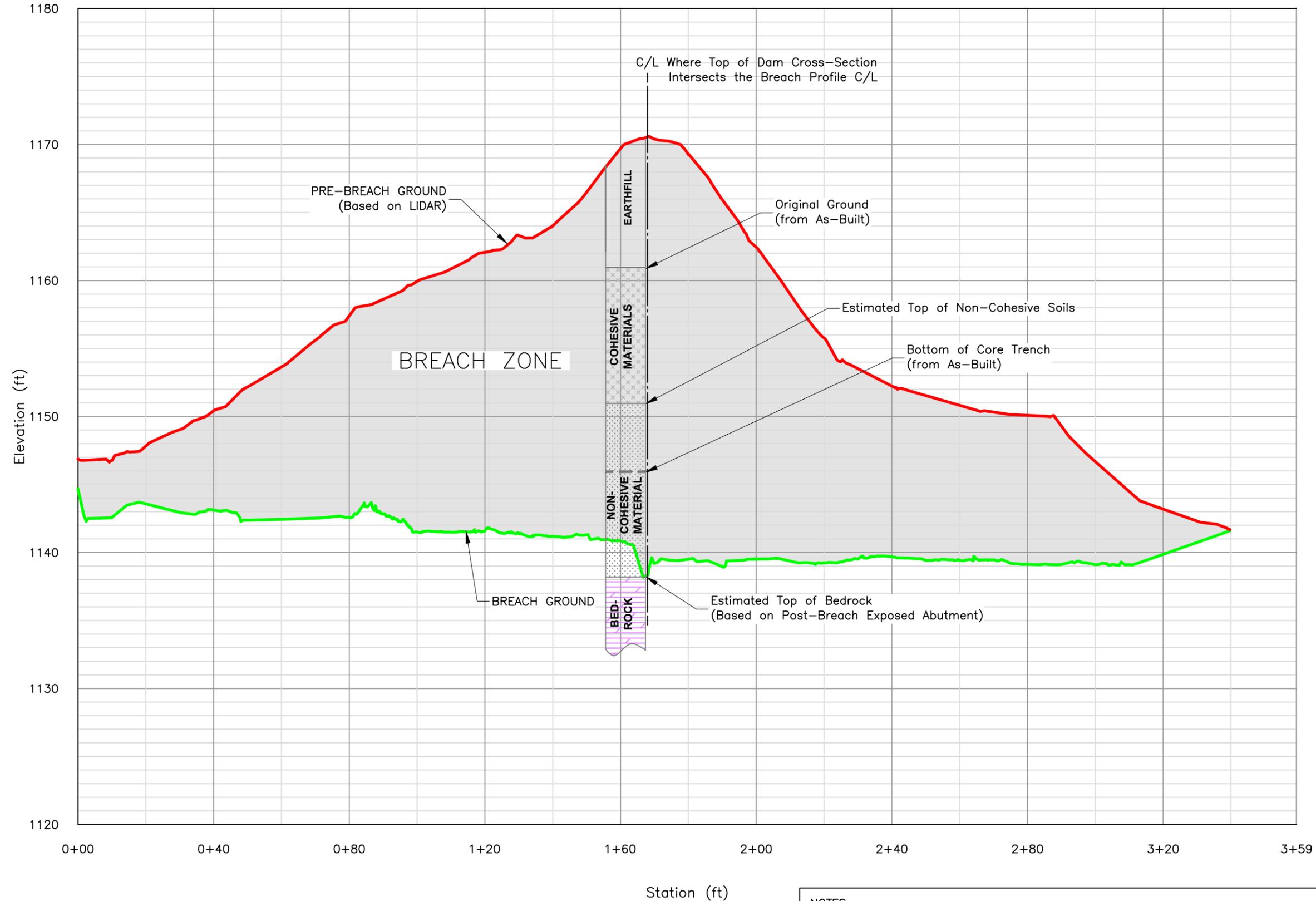
Date 4/2019  
 Designed JEV  
 Drawn  
 Checked  
 Approved

MLSNA  
 PROFILE - C/L OF BREACH  
 OWNER: VERNON COUNTY  
 COUNTY: VERNON



File Name WI-006  
 Date 08/14  
 Sheet 9 of 24

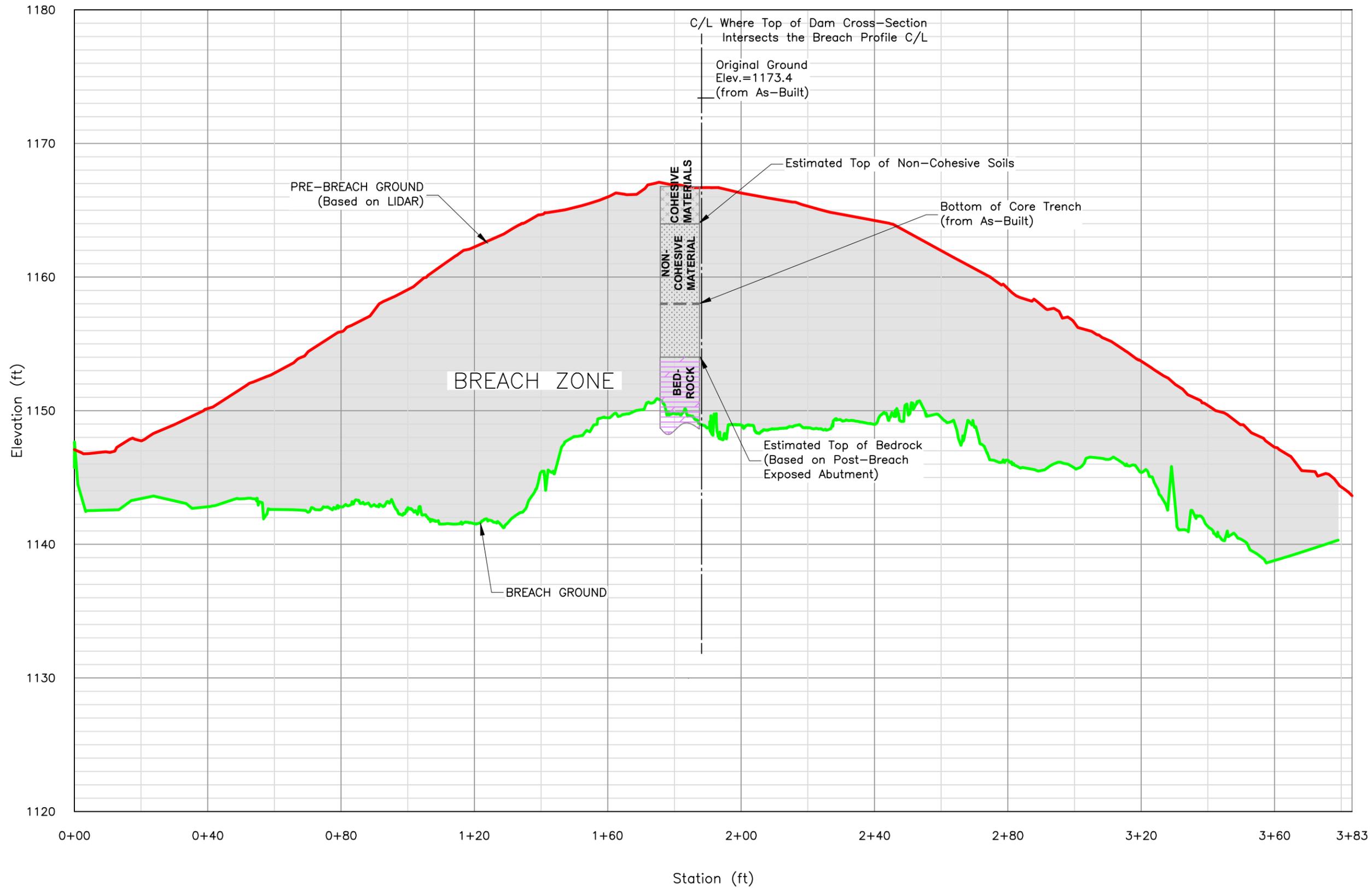
PROFILE CENTERLINE OF BREACH (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

**NOTES:**  
 The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

**NOTES:**  
The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.

Date 4/2019  
Designed JEV  
Drawn  
Checked  
Approved

MLSNA  
OWNER: VERNON COUNTY  
COUNTY: VERNON  
PROFILE - C/L OF AUXILIARY SPILLWAY



File Name WI-006  
Date 08/14  
Sheet 10 of 24



TBM#1: Crest of the NW Corner of the Principal Spillway.  
As-Built Elevation=983.2  
NAVD88 Elevation =1083.08

TOP OF DAM C/L  
ELEVATION VARIES FROM  
1005.8 to 1006.2

AUXILIARY SPILLWAY  
CREST ELEVATION = 1002.0



N.T.S.

**LEGEND**

- LIDAR GROUND CONTOUR  
(Approximate Pre-Breach Condition)
- #1 TYPICAL SOIL BORING LOCATION & NUMBER  
(Approximate Location)

**GRAPHIC SCALE**



( IN FEET )  
1 inch = 50 ft.

**NOTES:**  
ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION

ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

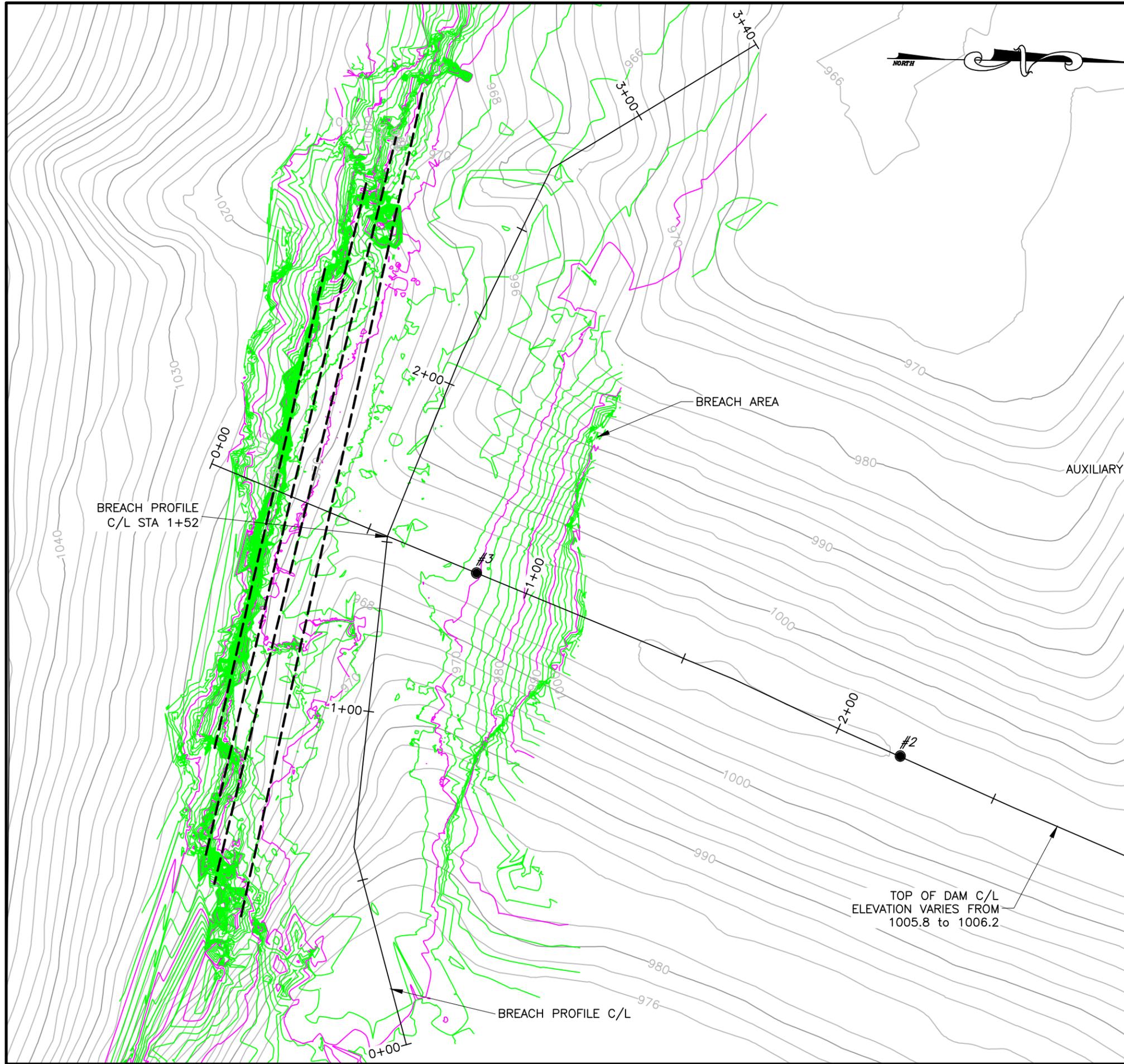
Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

**COON CREEK #21 PLAN VIEW - OVERALL**

OWNER: MONROE COUNTY  
COUNTY: MONROE



File Name	WI-006
Date	08/14
Sheet	11 of 24



**LEGEND**

- LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- POST-BREACH GROUND CONTOUR
- #1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)
- ESTIMATED VERTICAL VALLEY RELIEF FRACTURE LOCATION (Based on Post-Breach Exposed Abutment)

**GRAPHIC SCALE**

( IN FEET )  
1 inch = 30 ft.

**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

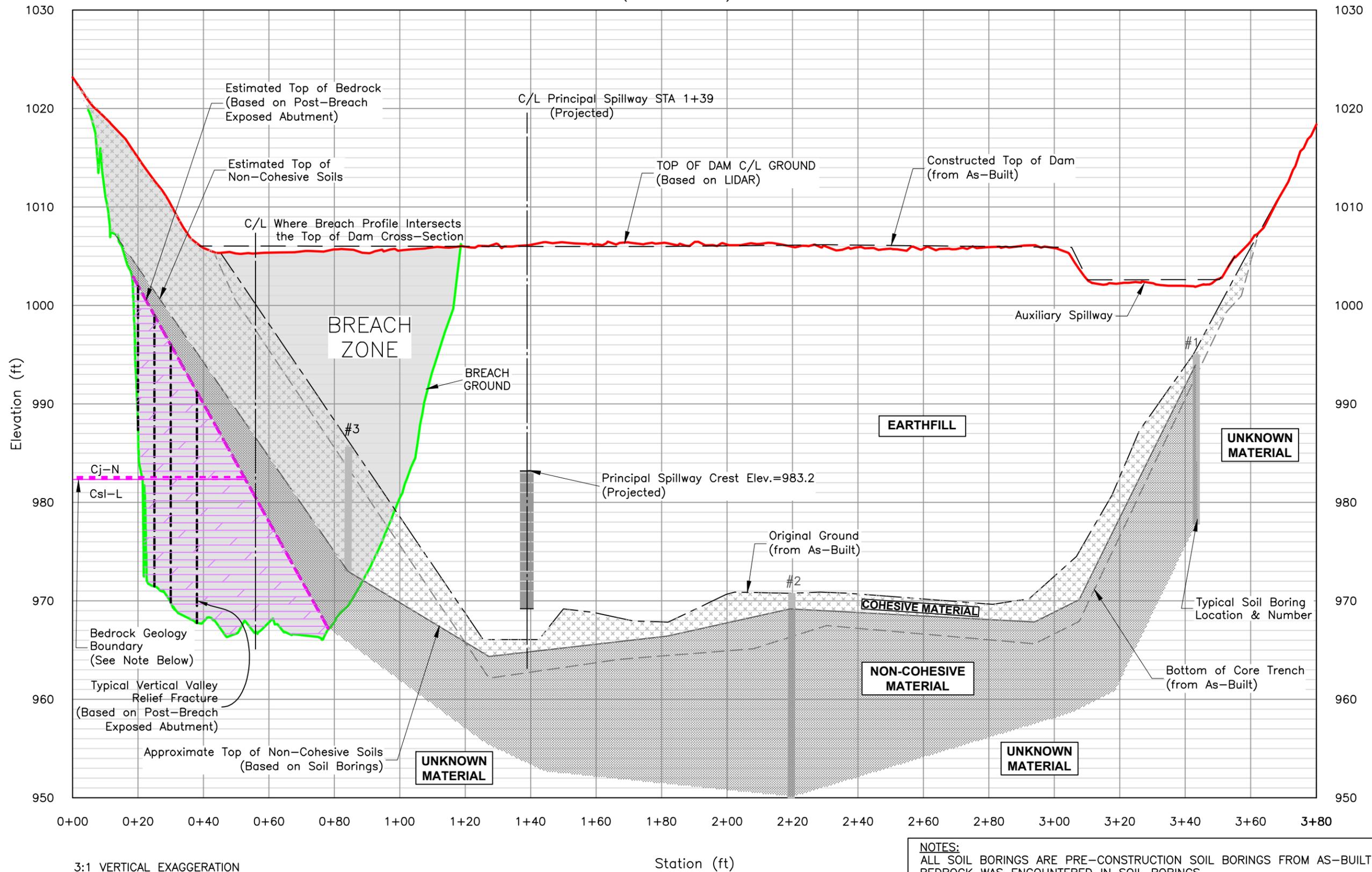
Date: 5/2019  
 Designed: JEV  
 Drawn: \_\_\_\_\_  
 Checked: \_\_\_\_\_  
 Approved: \_\_\_\_\_

**COON CREEK #21 PLAN VIEW  
 BREACH AREA**  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name: WI-006  
 Date: 08/14  
 Sheet 12 of 24

CROSS-SECTION TOP OF DAM CENTERLINE - STA 1+52 OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM)



3:1 VERTICAL EXAGGERATION

Station (ft)

**NOTES:**

ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT. NO BEDROCK WAS ENCOUNTERED IN SOIL BORINGS

**BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)**

Cj-N : Norwalk Member of the Jordan Sandstone (Case Hardened)  
 Csl-L: Lodi Member of the St. Lawrence Siltstone (Highly Fractured)

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

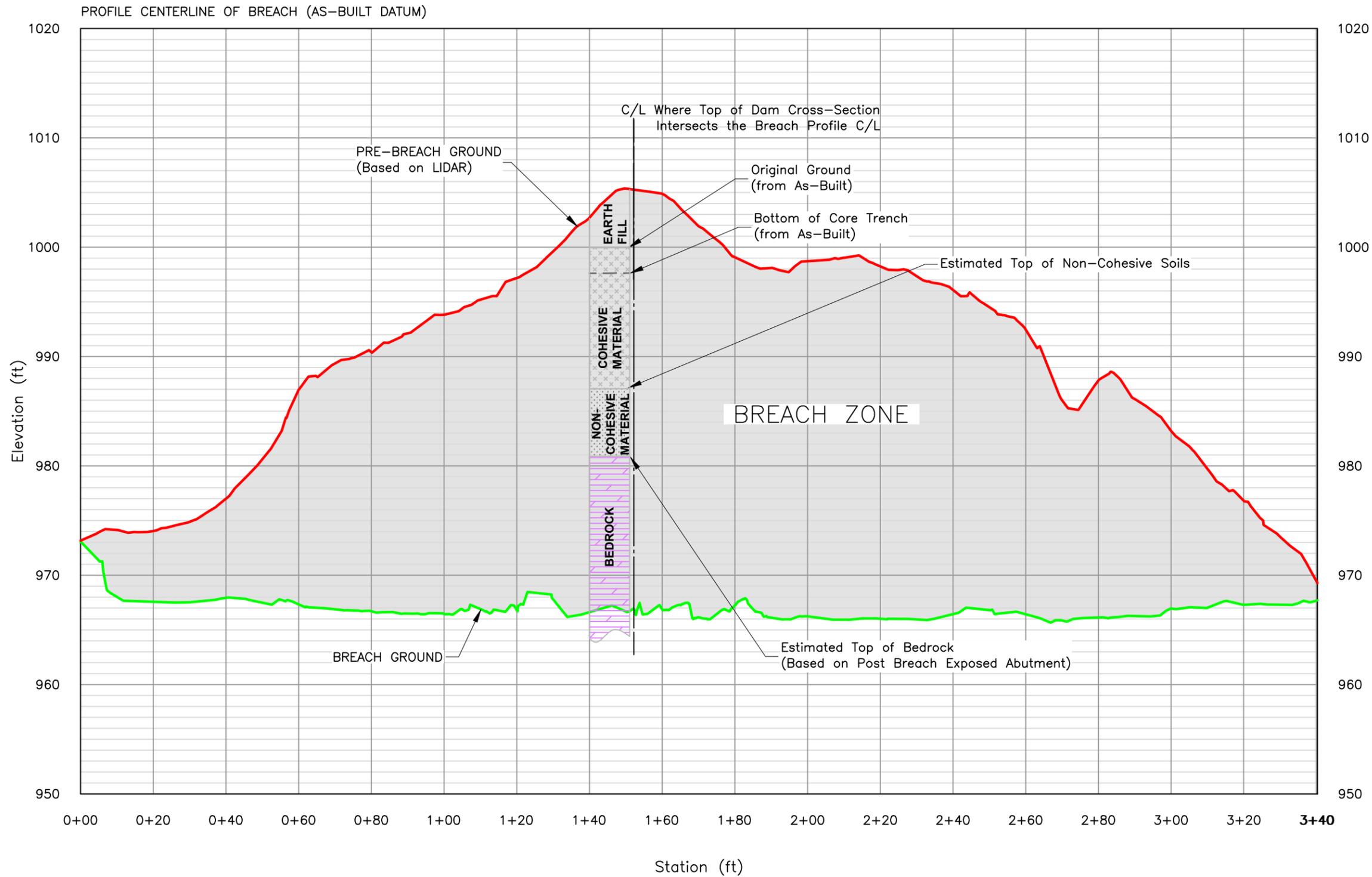
COON CREEK #21  
 TOP OF DAM C/L CROSS-SECTION  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name  
 WI-006

Date  
 08/14

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3:1 VERTICAL EXAGGERATION

**NOTES:**

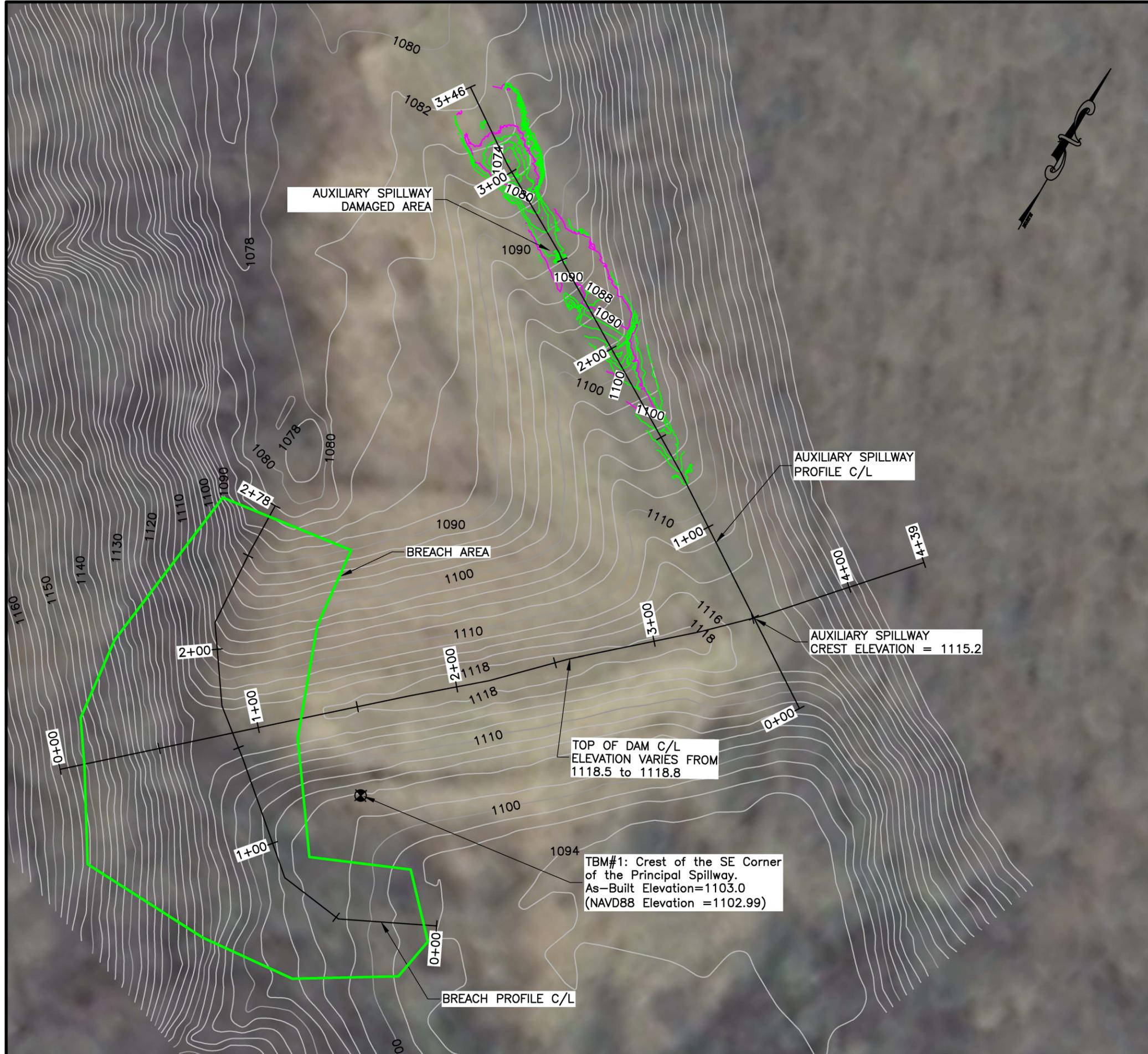
The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.

Designed	JEV	Date	5/2019
Drawn			
Checked			
Approved			

COON CREEK #21  
 PROFILE-C/L OF BREACH  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



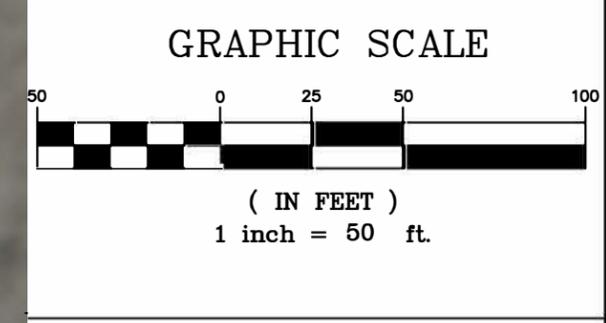
File Name	WI-006
Date	08/14
Sheet	14 of 24



**LEGEND**

— 1060 — LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)

— 1070 — POST-BREACH GROUND CONTOURS



**NOTES:**  
SOIL BORING INFORMATION AT DAM SITE IS NOT AVAILABLE

ALL ELEVATIONS BASED ON NAVD88 ELEVATION. AS-BUILT DATUM VARIES FROM NAVD88 DATUM BY +0.01'.

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

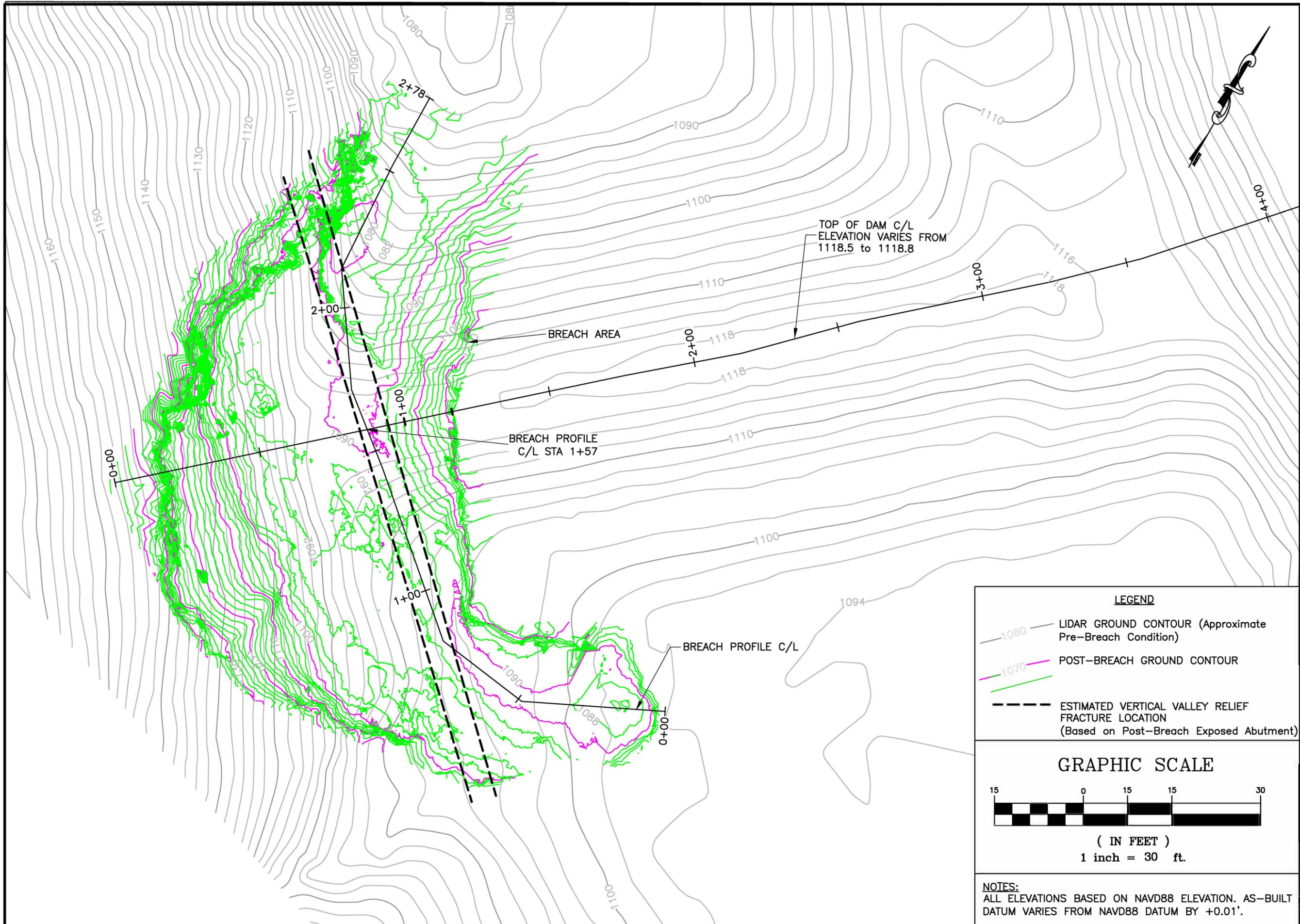
**COON CREEK #23 PLAN VIEW - OVERALL**

OWNER: MONROE COUNTY  
COUNTY: MONROE

United States Department of Agriculture  
**USDA**  
Natural Resources Conservation Service

File Name: WI-006  
Date: 08/14

Sheet 15 of 24



Date: 5/2019  
 Designed: JEV  
 Drawn: \_\_\_\_\_  
 Checked: \_\_\_\_\_  
 Approved: \_\_\_\_\_

COON CREEK #23 PLAN VIEW - BREACH AREA

OWNER: MONROE COUNTY  
 COUNTY: MONROE

**LEGEND**

- LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- POST-BREACH GROUND CONTOUR
- ESTIMATED VERTICAL VALLEY RELIEF FRACTURE LOCATION (Based on Post-Breach Exposed Abutment)

**GRAPHIC SCALE**

( IN FEET )  
1 inch = 30 ft.

**NOTES:**  
ALL ELEVATIONS BASED ON NAVD88 ELEVATION. AS-BUILT DATUM VARIES FROM NAVD88 DATUM BY +0.01'.

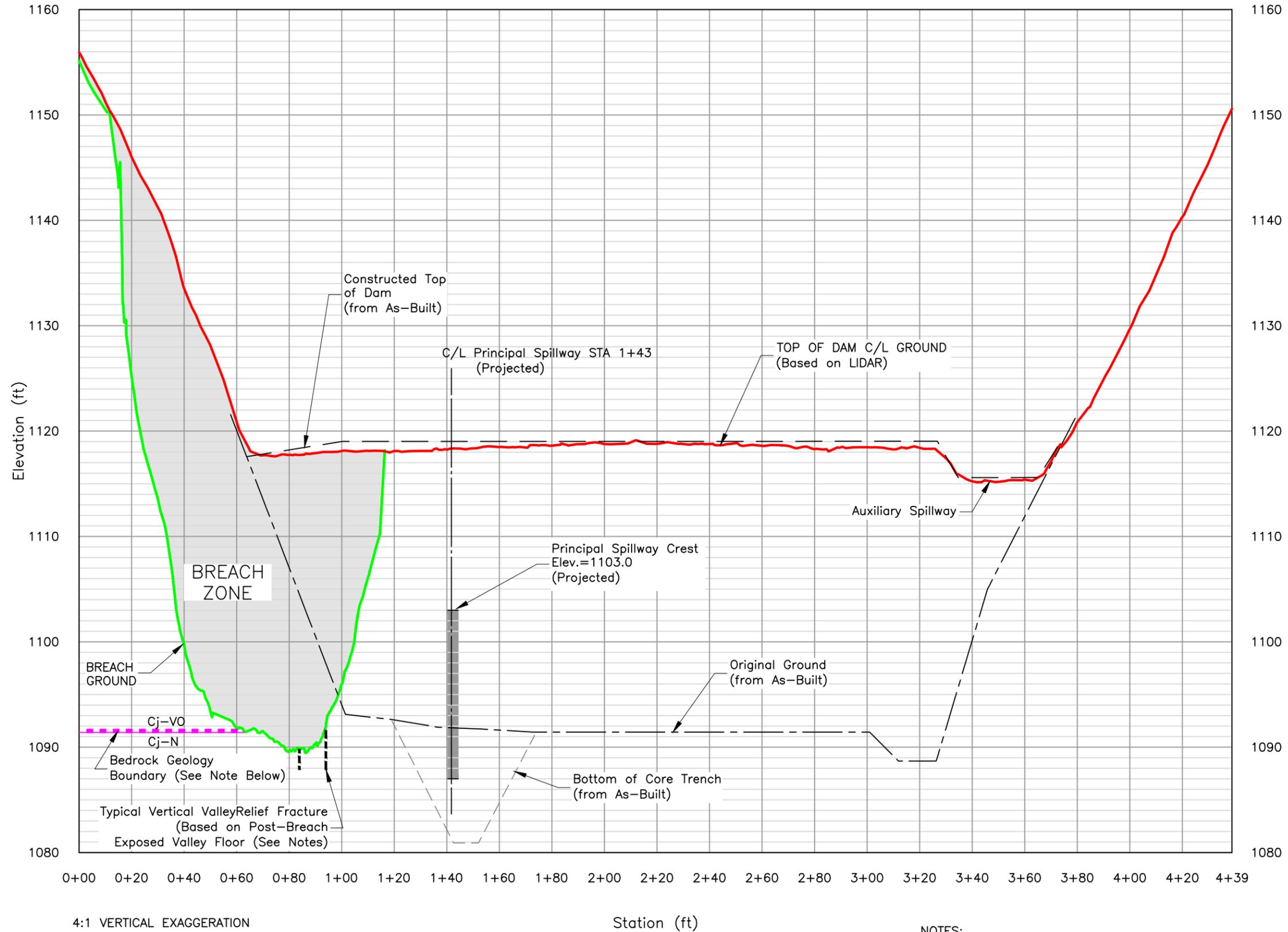
United States  
 Department of  
 Agriculture  
  
 Natural Resources  
 Conservation Service

File Name: WI-006

Date: 08/14

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Sheet of

CROSS-SECTION TOP OF DAM CENTERLINE - STA 1+57 OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM).



4:1 VERTICAL EXAGGERATION

Station (ft)

NOTES:

SOIL BORING INFORMATION AT DAM SITE IS NOT AVAILABLE

VERTICAL EXTENT OF VERTICAL VALLEY RELIEF FRACTURES IS UNKNOWN.

BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)

Cj-VO: Van Oser Member of the Jordan Sandstone (Soft, Poorly Cemented)

Cj-N : Norwalk Member of the Jordan Sandstone (Case Hardened)

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

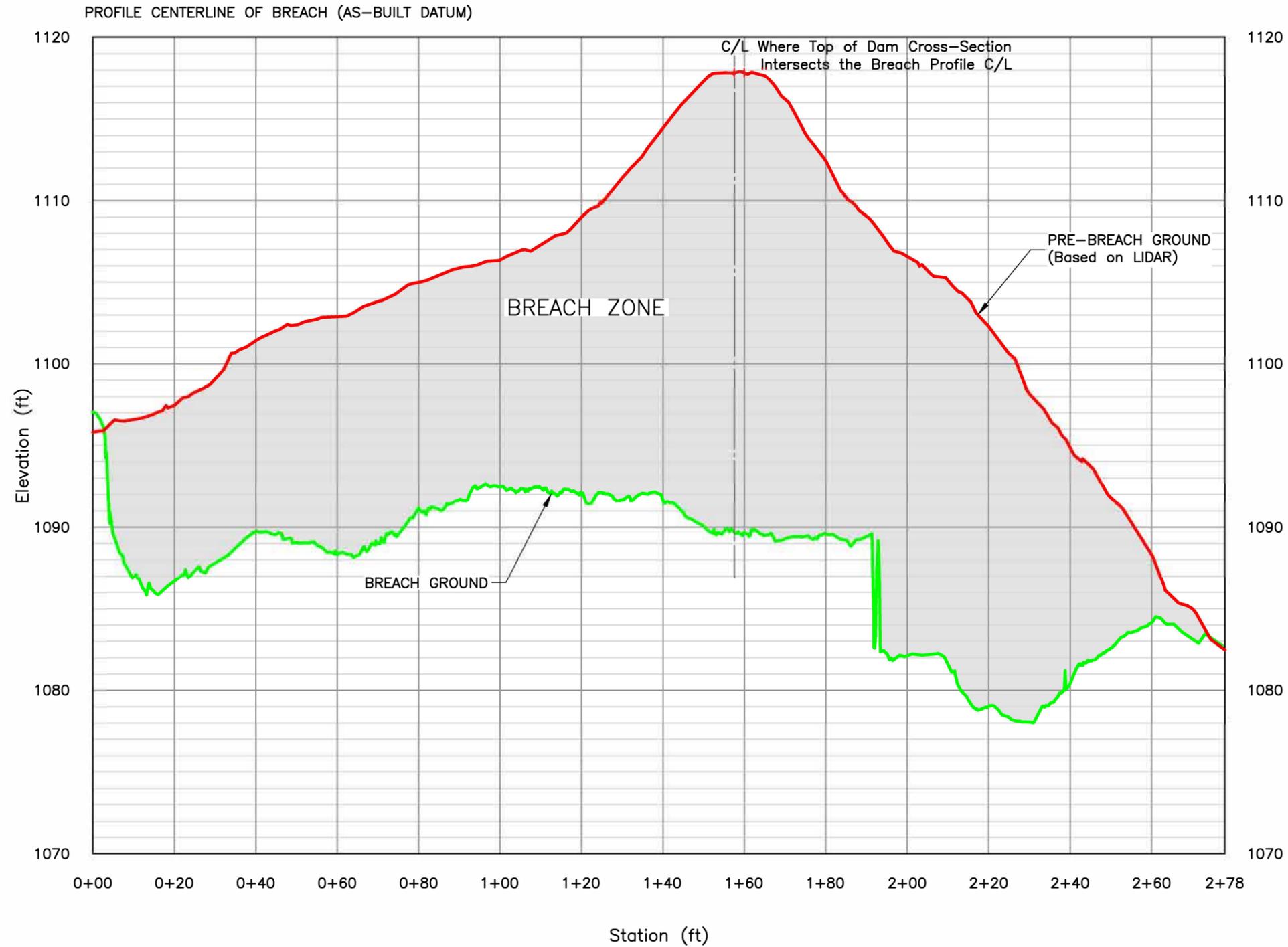
COON CREEK #23  
 TOP OF DAM C/L CROSS-SECTION  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name  
 WI-006

Date  
 08/14

Sheet 17 of 24



4:1 VERTICAL EXAGGERATION

NOTES:  
SOIL BORING INFORMATION AT DAM SITE IS NOT AVAILABLE

Designed	JEV	Date	5/2019
Drawn			
Checked			
Approved			

COON CREEK #23  
 PROFILE - C/L OF BREACH  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE

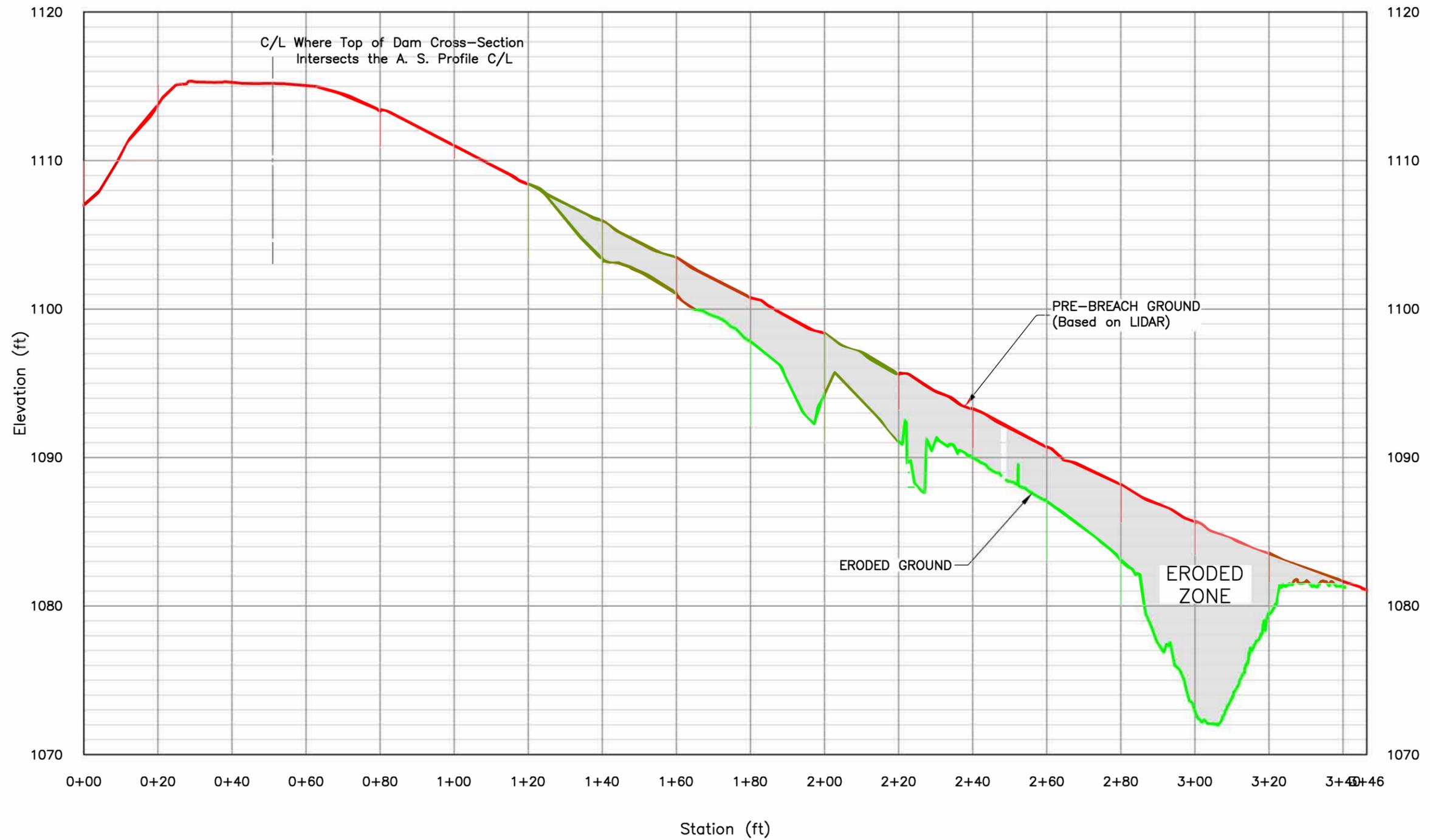


File Name  
WI-006

Date  
08/14

18 24  
Sheet of

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS-BUILT DATUM)



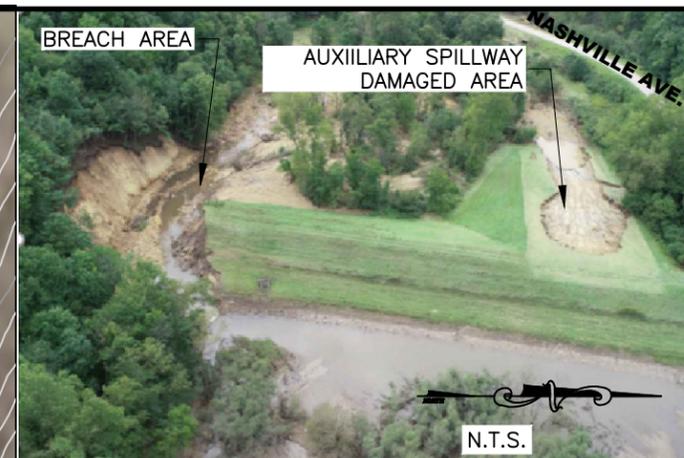
4:1 VERTICAL EXAGGERATION

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

COON CREEK #23  
 PROFILE - C/L OF AUXILIARY SPILLWAY (A.S.)  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



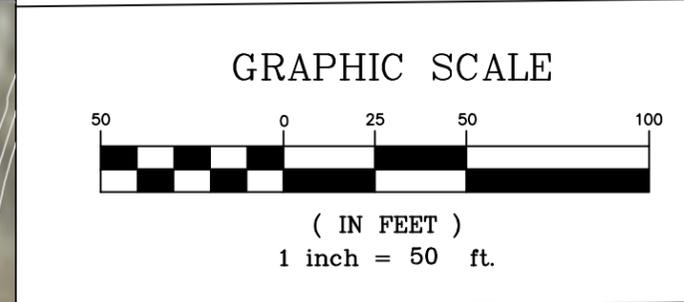
File Name	WI-006
Date	08/14
Sheet	19 of 24



Date	4/2019
Designed	JEV
Drawn	
Checked	
Approved	

**LEGEND**

- LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- POST-BREACH GROUND CONTOUR
- TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)



**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT  
 NO NAVD88 SITE ELEVATION DATA AVAILABLE

TBM#1: Crest of the SW Corner of the Principal Spillway.  
As-Built Elevation=1064.00

TOP OF DAM C/L  
ELEVATION VARIES FROM 1080.4 to 1080.9

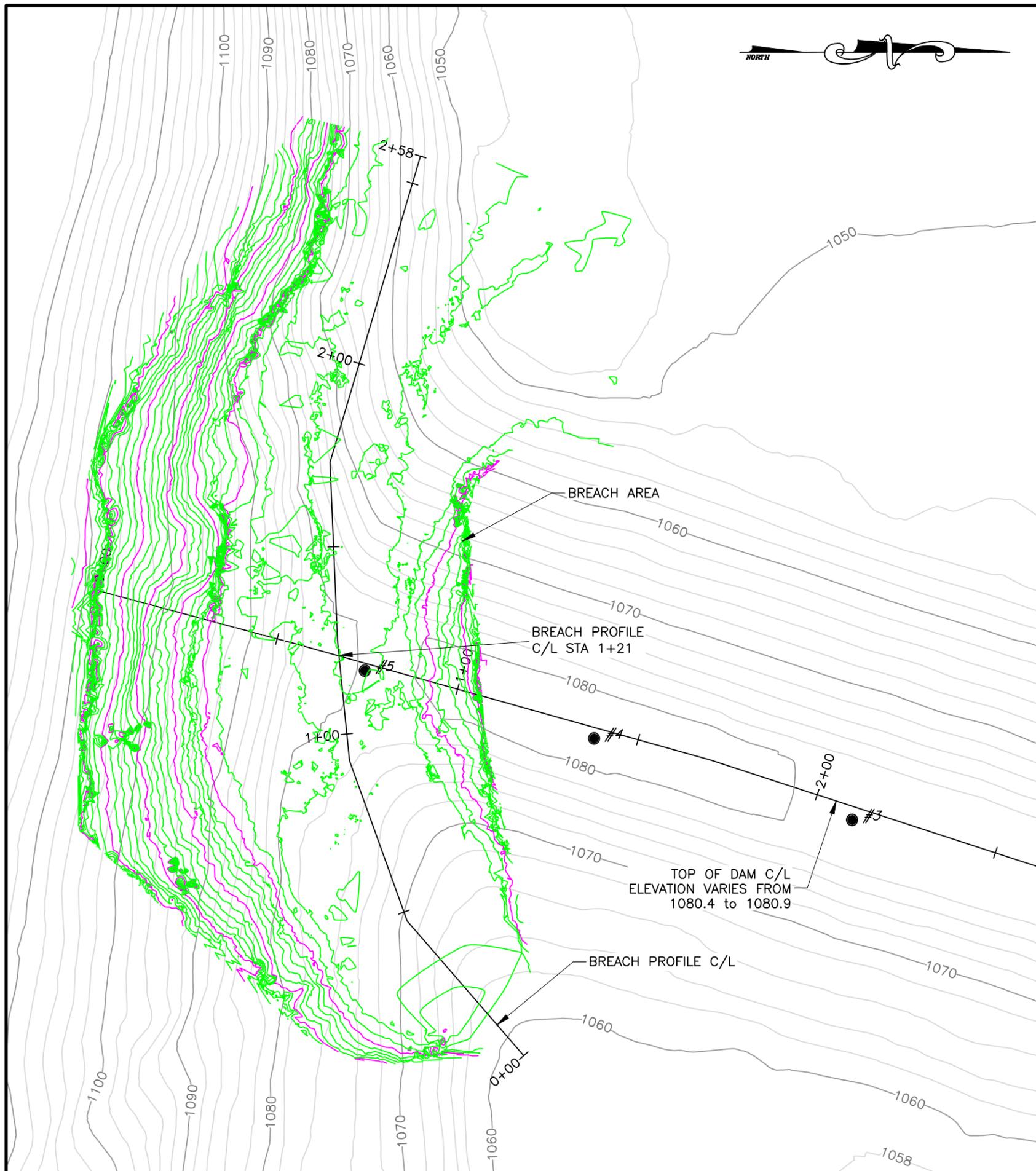
AUXILIARY SPILLWAY  
CREST ELEVATION = 1077.0

**COON CREEK #29 PLAN VIEW - OVERALL**

OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name	WI-006
Date	08/14
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**LEGEND**

- 1060 — LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- 1070 — POST-BREACH GROUND CONTOUR
- #1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)

**GRAPHIC SCALE**

( IN FEET )  
1 inch = 30 ft.

**NOTES:**  
 ALL ELEVATIONS ARE RELATIVE TO TBM #1 AS-BUILT ELEVATION  
 VERTICAL VALLEY RELIEF FRACTURES WERE NOT EVIDENT IN POST-BREACH ABUTMENT OR VALLEY FLOOR  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT

Date	4/2019
Designed	JEV
Drawn	
Checked	
Approved	

**CO CREEK #29 PLAN VIEW - BREACH AREA**

OWNER: MONROE COUNTY  
 COUNTY: MONROE

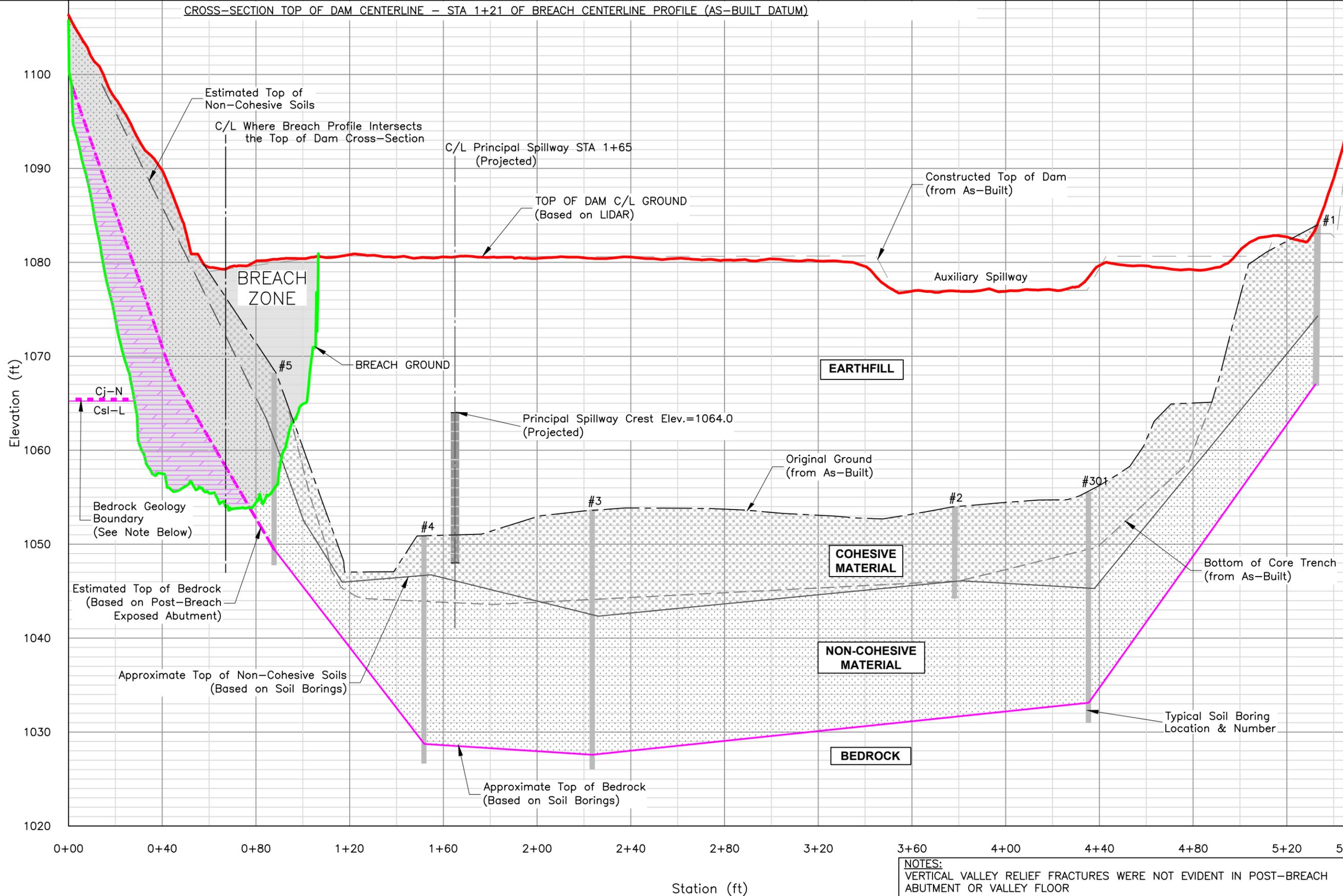
United States  
 Department of  
 Agriculture  
**USDA**  
 Natural Resources  
 Conservation Service

File Name  
 WI-006

Date  
 08/14

Sheet 21 of 24

CROSS-SECTION TOP OF DAM CENTERLINE - STA 1+21 OF BREACH CENTERLINE PROFILE (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

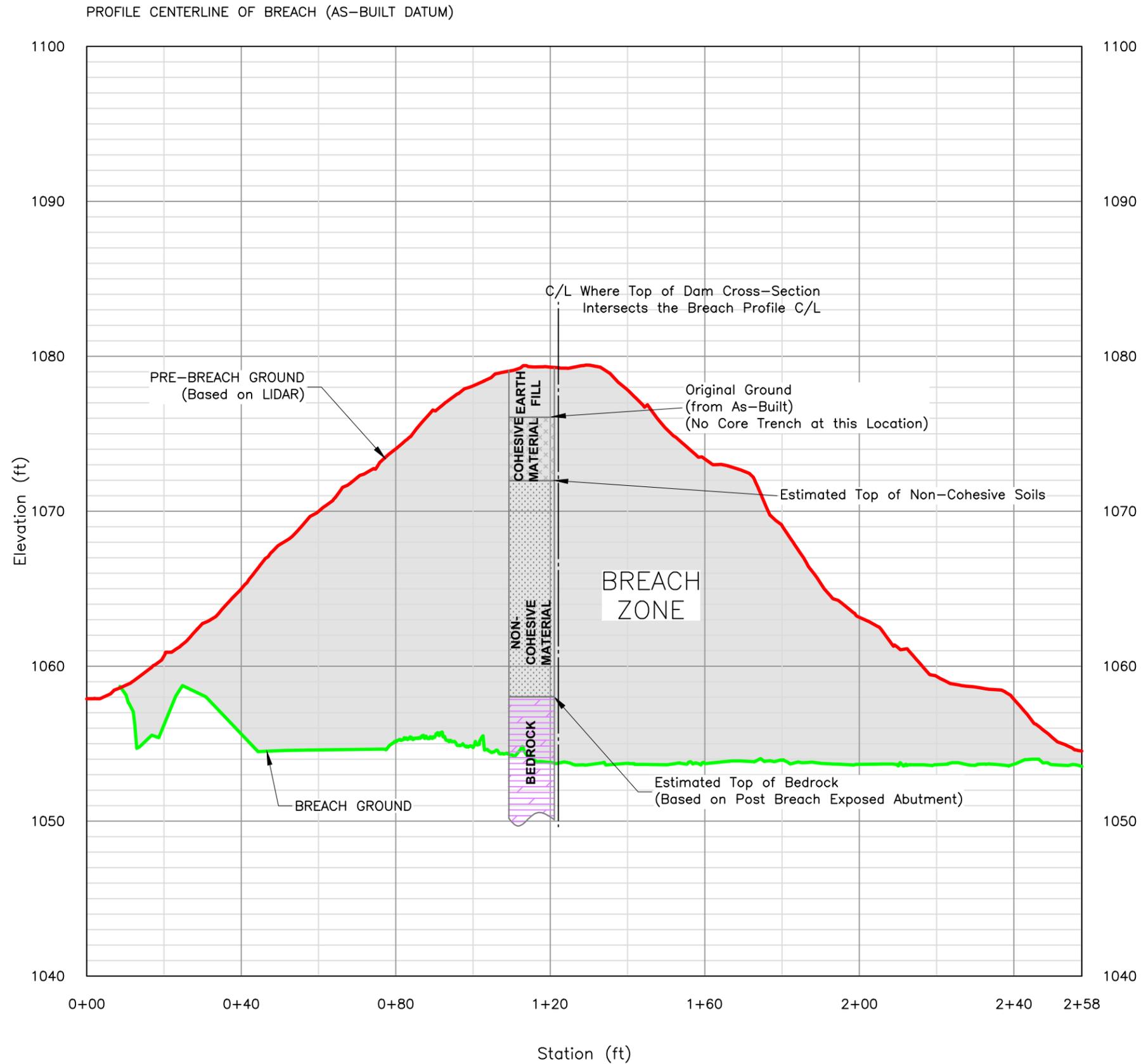
**NOTES:**  
 VERTICAL VALLEY RELIEF FRACTURES WERE NOT EVIDENT IN POST-BREACH ABUTMENT OR VALLEY FLOOR  
 ALL SOIL BORINGS ARE PRE-CONSTRUCTION SOIL BORINGS FROM AS-BUILT  
BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)  
 Cj-N : Norwalk Member of the Jordan Sandstone (Case Hardened)  
 Csl-L: Lodi Member of the St. Lawrence Siltstone (Highly Fractured)

Date	4/2019
Designed	JEV
Drawn	
Checked	
Approved	

COON CREEK #29  
 TOP OF DAM C/L CROSS-SECTION  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE

United States Department of Agriculture  
**USDA**  
 Natural Resources Conservation Service

File Name	WI-006
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4:1 VERTICAL EXAGGERATION

**NOTES:**

The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.

Designed	JEV	Date	4/2019
Drawn			
Checked			
Approved			

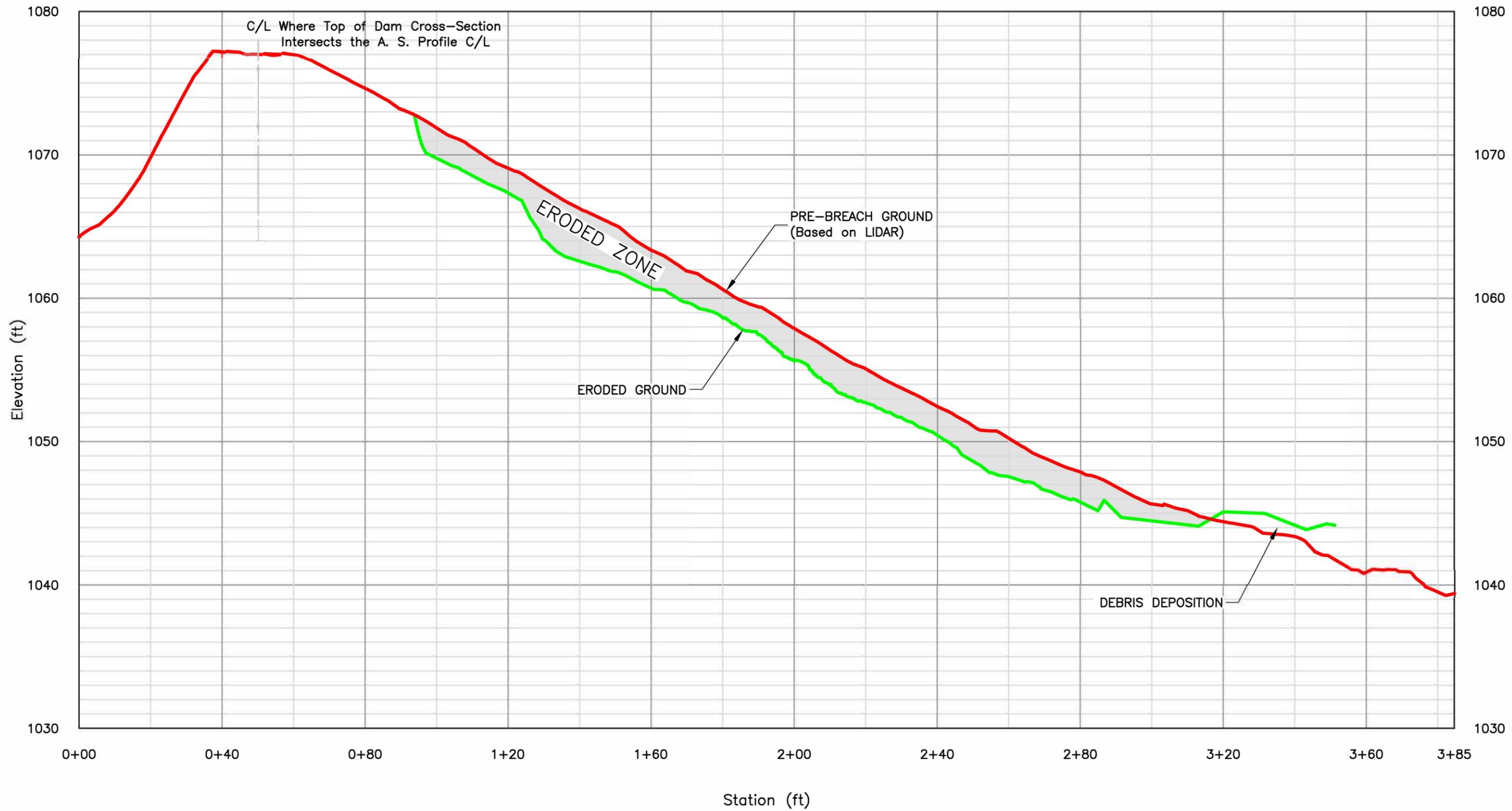
COON CREEK #29  
 PROFILE-C/L OF BREACH  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name  
 WI-006

Date  
 08/14

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

Date	4/2019
Designed	JEV
Drawn	
Checked	
Approved	

COON CREEK #29  
 PROFILE - C/L OF AUXILIARY SPILLWAY (A.S.)  
 OWNER: MONROE COUNTY  
 COUNTY: MONROE



File Name	WI-006
Date	08/14
Sheet	24 of 24

## Investigation Survey Photos



Photo 1  
Typical exposure of bedrock in breach channel.  
(WFK 1)



Photo 2  
Trimble SX10 TSLs 3D Scanner



Photo 3

Typical SX10 setup for “fine” 3D scan of exposed abutment bedrock in the breach channel.  
(CC 23)

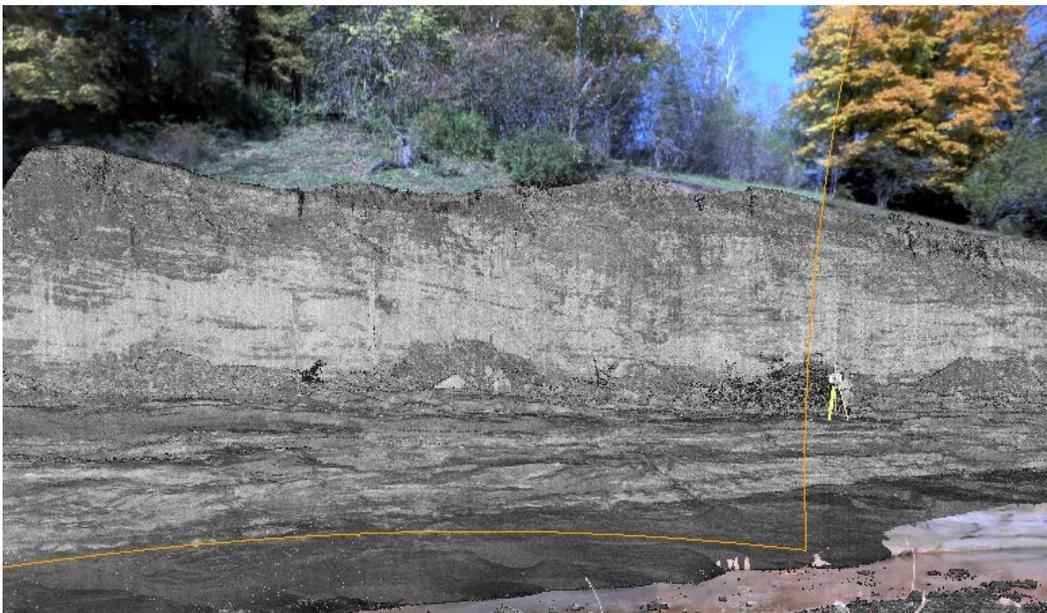


Photo 4

Example of ‘fine’ point cloud density along the exposed bedrock in breach channel. Point separation is approximately 0.15’.



Photo 5

3D Point Cloud data with colorization using TBC software



Photo 6

TBC point cloud data overlain on geologic formations.

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Attachment F  
Hydrology

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**August 27-28, 2018 Hydrology  
Vernon and Monroe Counties, WI**

**June 20, 2019**

A. Rainfall and Breach Chronology: August 27-September 3, 2018

The rainfall event that breached the five dams in this report occurred overnight during the evening of Monday, August 27 through early morning Tuesday, August 28, 2018. Mike Dreischmeier, Richland Center NRCS engineer, surveyed the damaged sites in Vernon County (Jersey Valley and Mlsna) during the day August 28<sup>th</sup>. The Mlsna flood pool was completely drained and was below the principal spillway inlet. The Jersey Valley dam had breached and was draining below the level of the principal spillway. Dan Gunderson, Sparta NRCS Civil Engineering Technician and Bob Micheel, Monroe County, visited the three Coon Creek breached dams (21, 23, 29) on August 28<sup>th</sup>. All three of the Coon Creek dams had breached and had already drained below the principal spillway inlet elevation.

Additional rain fell during the afternoon of August 28<sup>th</sup>. This rain put additional flow through the breached embankments, widening the breach openings.

Finally, the Labor Day storm hit one week later – Monday, September 3, 2018. Again, more storm runoff widened the existing breach openings even more.

## B. Local Rainfall and Breach Accounts

A neighbor downstream of Coon Creek 21, Rick Vaught, reported on the events on the early morning of Monday, August 28<sup>th</sup>:

*“We were woken when the shed outside our bedroom window toppled, that was at 02:30. The breach of the dam happened about 02:15. We saw the highest levels, which was under the corner of our cabin, about 02:30. We still had power on at that time but it went off soon after. The power from our house is underground to the Helgerson’s house. The (water) level went down approximately 6 feet by 03:30 so I believe that was the time of the surge. The power faulted when the downstream poles were taken out.*

*The amount of rain would be a guess, but I did have two large outdoor-type cooking pots outside on the grass. Both were full, with the bigger being 14” in depth.*

*The seven inches on the second storm (Tuesday afternoon August 28) was from a rain gauge that did not overflow.”*

Jeff Mlsna of the Vernon County Town of Clinton Volunteer Fire Department was quoted in the Crawford County Independent (Thursday September 6, 2018):

*“We’d been out all evening (August 27) responding to emergency calls and repairing driveway. We were just heading back in about 1:30 am (August 28) when it started to pour again, and didn’t let up until about 4:30 am. All those driveways we’d repaired were all washed out again by then.”*

Mike Dreischmeier concludes:

*All of the breaches in the two counties (Vernon and Monroe) happened well before 6AM as it got light out. No one reported seeing the breaches occur. I think that it likely that all (five dam breaches) occurred between 2am and 3:30am (Tuesday, August 28, 2018).*

From these reports, the critical precipitation that caused the five dams in Monroe and Vernon Counties to breach is the overnight rainfall on the evening of Monday, August 27<sup>th</sup> and early morning Tuesday, August 28<sup>th</sup>.

## C. Public Rainfall Reports

### 1) National Weather Service (NWS) or NWS-derived

The National Weather Service has a webpage dedicated to the details of the storm event:

<https://www.weather.gov/arx/aug2818>

The NWS overview comments from that webpage are displayed in Figure 1.

#### Overview

Several rounds of severe weather impacted the area from Monday August 27th through Tuesday August 28th.

Major flash flooding occurred Monday night into Tuesday morning and additional flash flooding occurred Tuesday afternoon and evening. Record flooding occurred along portions of the Kickapoo River.

A line of thunderstorms produced wind damage and an isolated tornado Monday afternoon and evening, with the severe threat quickly transitioning over to flash flooding Monday night into Tuesday morning. A storm produced large hail over portions of northeast Iowa Tuesday morning, then more severe thunderstorms developed Tuesday afternoon producing wind damage and additional flooding.

Numerous communities in southwest Wisconsin experienced record flooding. This included evacuations and rescues during the peak of the flooding.

Figure 1 <https://www.weather.gov/arx/aug2818> (accessed 10-10-2018)

NWS radar loops show the timing of the storm, as shown in Figure 2 **Error! Reference source not found.** The significant rain started around 8 pm Monday, August 27<sup>th</sup> and ended by 4 am Tuesday, August 28<sup>th</sup>. The NWS warned of flooding along Coon Creek with estimated rainfall of 5 to 12 inches.

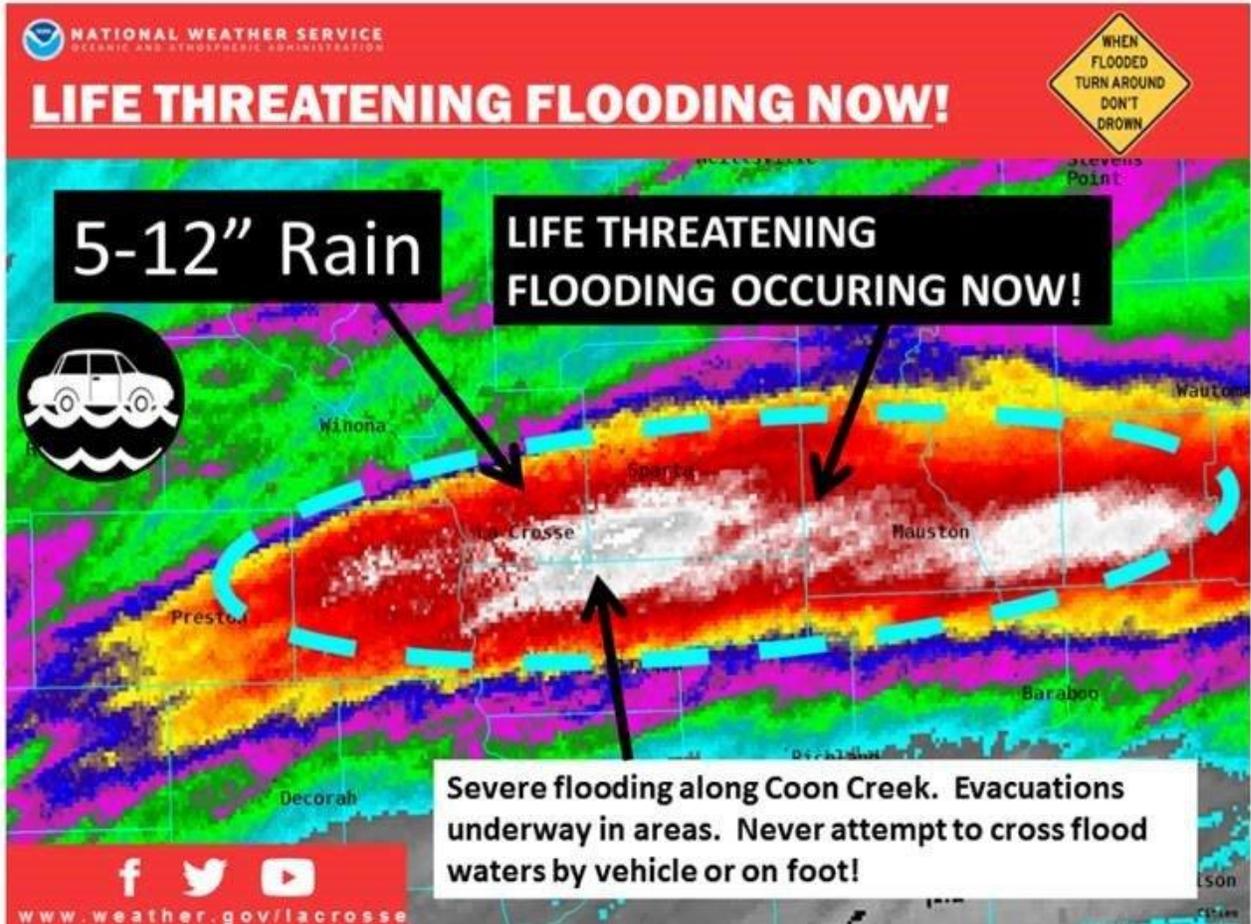


Figure 2 NWS Rainfall Graphic warning of Coon Creek flooding (Rainfall duration unspecified) Published Tues Aug 28th

Figure 3 shows the NWS rainfall text report for Monday night August 27-28, 2019 in Monroe and Vernon Counties. Figure 4 shows the NWS rainfall text report for Tuesday afternoon and night August 28, 2019 for Monroe and Vernon Counties, WI. Figure 5 is the NWS storm summary August 26-29, 2019 for the three-state area of IA-MN-WI. Figure 6 lists the NWS wettest day records that were set by the August 2019 storm.

Rain Reports for Monroe and Vernon Counties, WI

PUBLIC INFORMATION STATEMENT  
NATIONAL WEATHER SERVICE LA CROSSE WI  
1120 AM CDT TUE AUG 28 2018

...RAINFALL REPORTS FROM MONDAY NIGHT AUGUST 27-28...

LOCATION	AMOUNT	TIME/DATE	LAT/LON
...WISCONSIN...			
...MONROE COUNTY...			
MELVINA	8.66 IN	0700 AM 08/28	43.81N/90.76W
3 WNW KENDALL	8.58 IN	0700 AM 08/28	43.81N/90.44W
CASHTON	7.25 IN	1119 PM 08/27	43.74N/90.78W
TOMAH RANGER STATION 2	5.06 IN	0800 AM 08/28	43.97N/90.47W
SPARTA	4.22 IN	0800 AM 08/28	43.94N/90.82W
WARRENS 4WSW	3.02 IN	0700 AM 08/28	44.10N/90.59W
4 ENE SPARTA	2.24 IN	1234 AM 08/28	43.96N/90.74W
2 E CATARACT	0.92 IN	0708 AM 08/28	44.09N/90.79W
CATARACT	0.75 IN	1204 AM 08/28	44.09N/90.84W
...VERNON COUNTY...			
WESTBY 3ENE	9.98 IN	0700 AM 08/28	43.67N/90.81W
2 NE GENOA	7.75 IN	0729 AM 08/28	43.59N/91.21W
STODDARD	7.42 IN	0700 AM 08/28	43.66N/91.22W
HILLSBORO WSW	7.00 IN	0700 AM 08/28	43.65N/90.35W
HILLSBORO 2SW	6.64 IN	0700 AM 08/28	43.63N/90.38W
LA FARGE	4.70 IN	0834 AM 08/28	43.58N/90.64W
5 E VIROQUA	4.53 IN	1137 PM 08/27	43.58N/90.77W
1 NW VALLEY	4.50 IN	0834 AM 08/28	43.65N/90.56W
VIROQUA 0.8 ESE	4.29 IN	0700 AM 08/28	43.55N/90.87W
GENOA DAM 8	4.09 IN	0600 AM 08/28	43.57N/91.23W
2 NNW VIROQUA	3.55 IN	1225 AM 08/28	43.58N/90.90W
3 WNW VIOLA	2.45 IN	0730 AM 08/28	43.53N/90.74W
READSTOWN	2.27 IN	0500 AM 08/28	43.45N/90.76W
ONTARIO	1.22 IN	0200 AM 08/28	43.72N/90.59W

OBSERVATIONS ARE COLLECTED FROM A VARIETY OF SOURCES WITH VARYING EQUIPMENT AND EXPOSURES. WE THANK ALL VOLUNTEER WEATHER OBSERVERS FOR THEIR DEDICATION. NOT ALL DATA LISTED ARE CONSIDERED OFFICIAL.

*Figure 3 NWS rainfall report for Monday night August 27-28, 2019 in Monroe & Vernon Counties, WI*

# Rainfall Reports from Tuesday Morning-Wednesday Morning

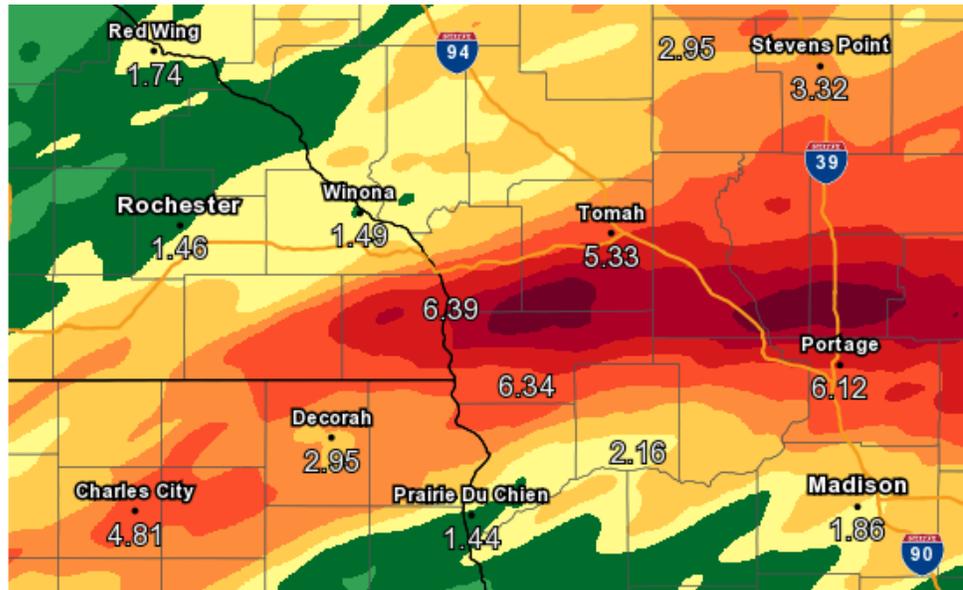
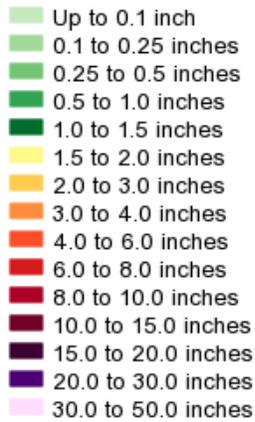
...RAINFALL REPORTS FROM TUESDAY AFTERNOON/NIGHT AUGUST 28 2018...

LOCATION	AMOUNT	TIME/DATE	LAT/LON
...WISCONSIN...			
...MONROE COUNTY...			
CASHTON 3NNW	2.14 IN	0733 AM 08/29	43.79N/90.80W
SPARTA	1.30 IN	0800 AM 08/29	43.94N/90.82W
SPARTA/FORT MCCOY AIRPORT	1.23 IN	0655 AM 08/29	43.96N/90.74W
...VERNON COUNTY...			
HILLSBORO 2SW	4.14 IN	0700 AM 08/29	43.63N/90.38W
3 WNW VIOLA	4.12 IN	0728 AM 08/29	43.53N/90.74W
3 WNW LA FARGE	4.10 IN	1159 PM 08/28	43.59N/90.69W
HILLSBORO WSW	3.81 IN	0700 AM 08/29	43.65N/90.35W
VIROQUA 0.8 ESE	3.10 IN	0700 AM 08/29	43.55N/90.87W
VIROQUA MUNICIPAL AIRPORT	2.19 IN	0655 AM 08/29	43.58N/90.90W
2 NE GENOA	1.80 IN	0836 AM 08/29	43.59N/91.21W
WESTBY 3ENE	1.77 IN	0700 AM 08/29	43.67N/90.81W
STODDARD	1.48 IN	0700 AM 08/29	43.66N/91.22W
GENOA DAM 8	1.40 IN	0600 AM 08/29	43.57N/91.23W

OBSERVATIONS ARE COLLECTED FROM A VARIETY OF SOURCES WITH VARYING EQUIPMENT AND EXPOSURES. WE THANK ALL VOLUNTEER WEATHER OBSERVERS FOR THEIR DEDICATION. NOT ALL DATA LISTED ARE CONSIDERED OFFICIAL.

*Figure 4 NWS rainfall report for Tuesday afternoon/night August 28, 2019 in Monroe & Vernon Counties, WI*

# Total Rainfall August 26-29, 2018



Graphic Created  
August 30th, 2018  
11:12 AM CDT

LOCATION AMOUNT

...WISCONSIN...

...MONROE COUNTY...

CASHTON 3NNW	12.86 IN
CASHTON 4.8N	11.01 IN
WILTON 4.2E	8.58 IN
SPARTA	5.52 IN
TOMAH RANGER STATION	5.06 IN
SPARTA/FORT MCCOY AIRPORT	4.92 IN
2 E CATARACT	1.44 IN

...VERNON COUNTY...

WESTBY 3ENE	12.03 IN
HILLSBORO 2SW	11.16 IN
HILLSBORO WSW	10.81 IN
ONTARIO 3E	9.99 IN
LA FARGE	9.75 IN
STODDARD 5NNE	9.72 IN
STODDARD	8.90 IN
GENOA DAM 8	5.72 IN
3 WNW VIOLA	4.12 IN
3 WNW LA FARGE	4.10 IN
READSTOWN	3.32 IN

OBSERVATIONS ARE COLLECTED FROM A VARIETY OF SOURCES WITH VARYING EQUIPMENT AND EXPOSURES. WE THANK ALL VOLUNTEER WEATHER OBSERVERS FOR THEIR DEDICATION. NOT ALL DATA LISTED ARE CONSIDERED OFFICIAL.

Figure 5 NWS storm report August 26-29, 2019 for IA-MN-WI

## New Wettest Day Records\*\*\*

Location	County (State)	Rainfall Aug 27-28, 2018	Previous Record	Records Began
Caledonia	Houston (MN)	8.10"	6.60" – July 20-21, 1951	1892
Cashton 3NNW	Monroe (WI)	10.52" *	6.15" – July 19-20, 2017 **	1949
Hillsboro 2SW	Vernon (WI)	6.64"	5.61" – September 21-22, 2016	2013
Hokah 4 NW	Houston (MN)	4.95"	3.88" - July 19-20, 2017	2006
Mabel	Fillmore (MN)	5.53"	5.22" – June 9-10, 2018	2013
Mauston 1SE	Juneau (WI)	8.14"	5.22" – July 14-15, 2010	1905
Ontario 3E	Vernon (WI)	9.66"	6.10" – June 7-8, 2008	1975
Tomah Ranger Station 2	Monroe (WI)	5.06"	4.07" – June 2-3, 2002	1936
Westby 3ENE	Vernon (WI)	9.98"	7.17" – August 27-28, 2018	1956

\* This was the 3<sup>rd</sup> highest daily rainfall in the La Crosse Hydrologic Service Area. Only Hokah WWTP (15.10" Minnesota State Record – August 18-19, 2007 - Houston County, MN) and Altura 5W (11.45" – August 18-19, 2007 – Winona County) were higher.

\*\* Threaded with Cashton cooperative record to extend the records for the area from 2013 back to 1949.

\*\*\* All of these cooperative observing sites have a reporting period from 7 AM the previous day to 7 AM the current day.

*Figure 6 NWS wettest day records set by the Aug 2019 storm*

Figure 7 is a NWS 48-hour duration rainfall graphic. Figure 8 is a Channel 8 48-hour duration rainfall graphic, which was likely derived from NWS information.

### August 27-28, 2018 Rainfall Event

Several rounds of heavy rain impacted the area from Monday evening (August 27<sup>th</sup>) through Tuesday evening (August 28<sup>th</sup>). Anywhere from 2 to nearly 13" of rain fell across Adams, Juneau, La Crosse, Monroe, & Vernon counties in Wisconsin and Houston County in southeast Minnesota. **The highest 2-day rain total was 12.66" near Cashton, WI (10.52" of this rain fell on Monday night).** The 24-hour rainfall record for Wisconsin is 11.72" on June 24, 1946 (Mellen 4NE).

Location	County (State)	7 AM Mon - 7 AM Tue	7 AM Tue - 7 AM Wed	2-Day Total
Cashton 3NNW (COOP)	Monroe (WI)	10.52"	2.14"	12.66"
Wisconsin Dells 1.7 NNW (CoCoRaHS)	Juneau (WI)	8.30"	4.14"	12.44"
Westby 3ENE (COOP)	Vernon (WI)	9.98"	1.77"	11.75"
Mauston 1SE (COOP)	Juneau (WI)	8.14"	3.05"	11.22"
Cashton 4.8N (CoCoRaHS)	Monroe (WI)	8.66"	2.35"	11.01"
Hillsboro (U COOP)	Vernon (WI)	7.00"	3.81"	10.81"
St Joseph 3W (NWS)	La Crosse (WI)	9.12"	1.91"	11.03"
Hillsboro 2SW (COOP)	Vernon (WI)	6.64"	4.14"	10.78"
Grand Marsh 1W (CoCoRaHS)	Adams (WI)	7.78"	2.79"	10.57"
La Crosse 5.4SE (CoCoRaHS)	La Crosse (WI)	7.95"	1.86"	9.81"
Ontario 3E (COOP)	Vernon (WI)	9.66"	M	9.66"
Grand Marsh 1.9SSW (CoCoRaHS)	Adams (WI)	6.30"	2.89"	9.19"
Hokah (MN Gage)	Houston (MN)	7.60"	1.30"	8.90"
Oxford 4W (CoCoRaHS)	Adams (WI)	6.44"	2.46"	8.90"
Stoddard (U COOP)	Vernon (WI)	7.42"	1.48"	8.90"
Wildcat Landing (MN Gage)	Houston (MN)	8.63"	M	8.63"
Wilton 4.2E (CoCoRaHS)	Monroe (WI)	8.58"	M	8.58"
La Crosse (NWS Office)	La Crosse (WI)	5.11"	2.37"	7.48"
Irish Hill (Former NWS Employee)	La Crosse (WI)	6.71"	1.52"	8.23"
Caledonia (MN Gage)	Houston (MN)	7.20"	M	7.20"
Viroqua Municipal Airport (AWOS)	Vernon (WI)	4.53"	2.19"	6.72"
La Crescent 1NNW (U Coop)	Houston (MN)	4.40"	1.72"	6.12"
Sparta (Coop)	Monroe (WI)	4.22"	1.30"	5.52"
Yucatan (MN Gage)	Houston (MN)	5.52"	M	5.52"
West Salem (U COOP)	La Crosse (WI)	3.94"	1.03"	4.97"
Sparta/Fort Mc Coy Airport (AWOS)	Monroe (WI)	3.69"	1.22"	4.91"
La Crosse Regional Airport (ASOS)	La Crosse (WI)	3.13"	1.18"	4.31"

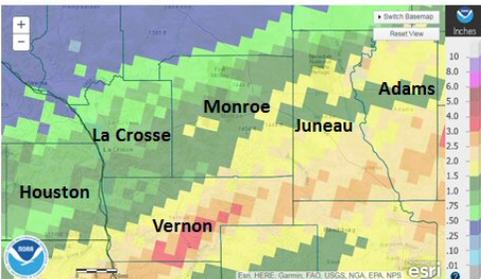
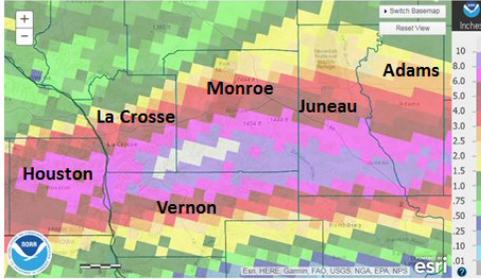


Figure 7 NWS 48-hour duration rainfall graphic



Figure 8 48-hour duration rainfall graphic from Channel 8 La Crosse, WI

## 2) Community Collaborative Rain, Hail & Snow Network (CoCoRaHS)

Figure 9 shows the CoCoRaHS 24-hour rainfall reports for Monroe County, WI ending 7:00 am August 28, 2018. Figure 10 shows the CoCoRaHS 24-hour rainfall reports for Vernon County, WI ending 7:00 am August 28, 2018.

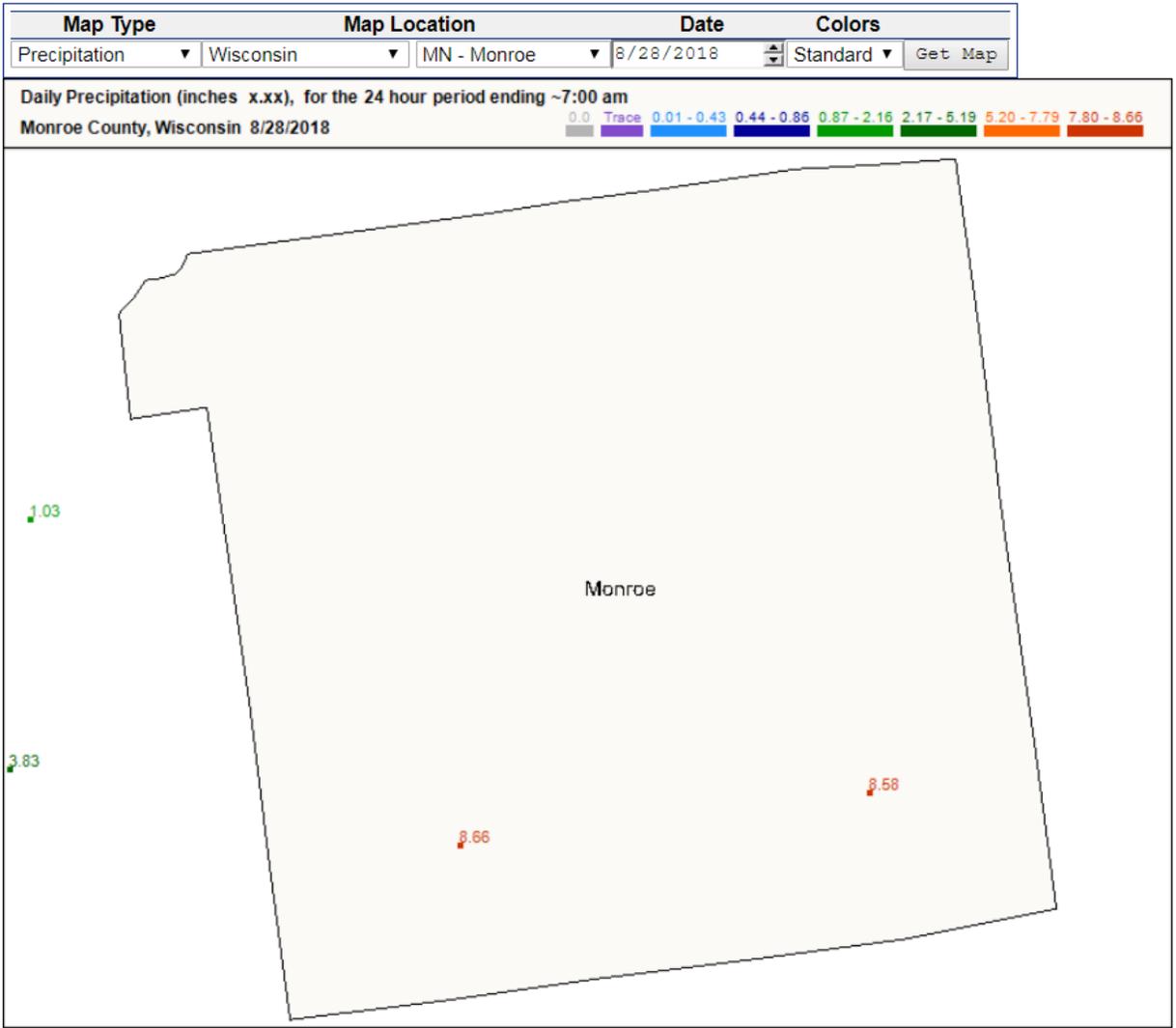


Figure 9 CoCoRaHS Monroe County, WI 24-hour rainfall ending 7 am August 28, 2018

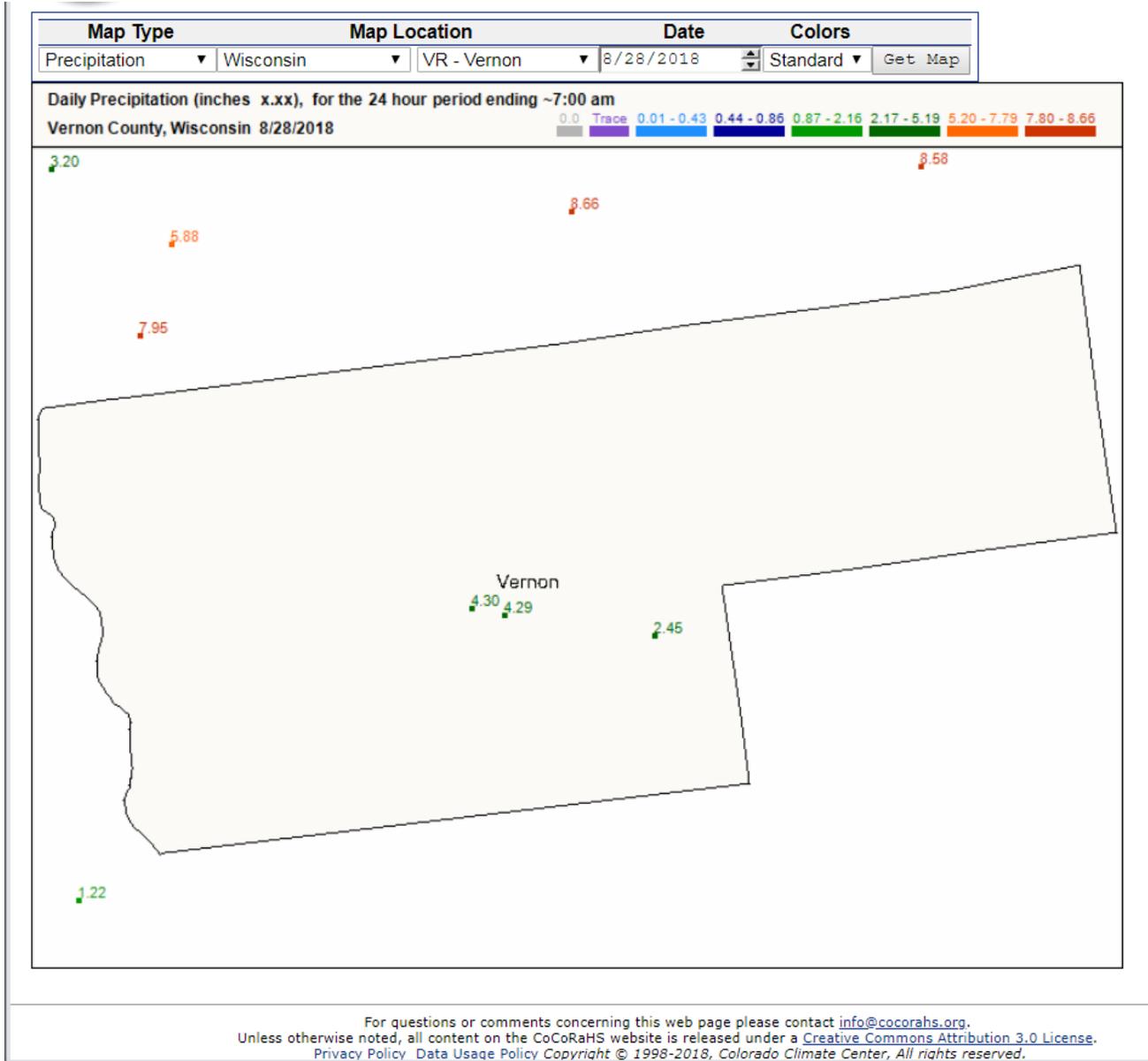


Figure 10 CoCoRaHS Vernon County, WI 24-hour rainfall ending 7 am August 28, 2018

### 3) NEXRAD Radar

Figure 11 shows the NEXRAD radar precipitation estimate from NEXRAD station KARX in LaCrosse, WI for the 24-hour period August 27 18:00 through August 28 18:00. The storm duration is approximately 6 hours, or 7 hours.

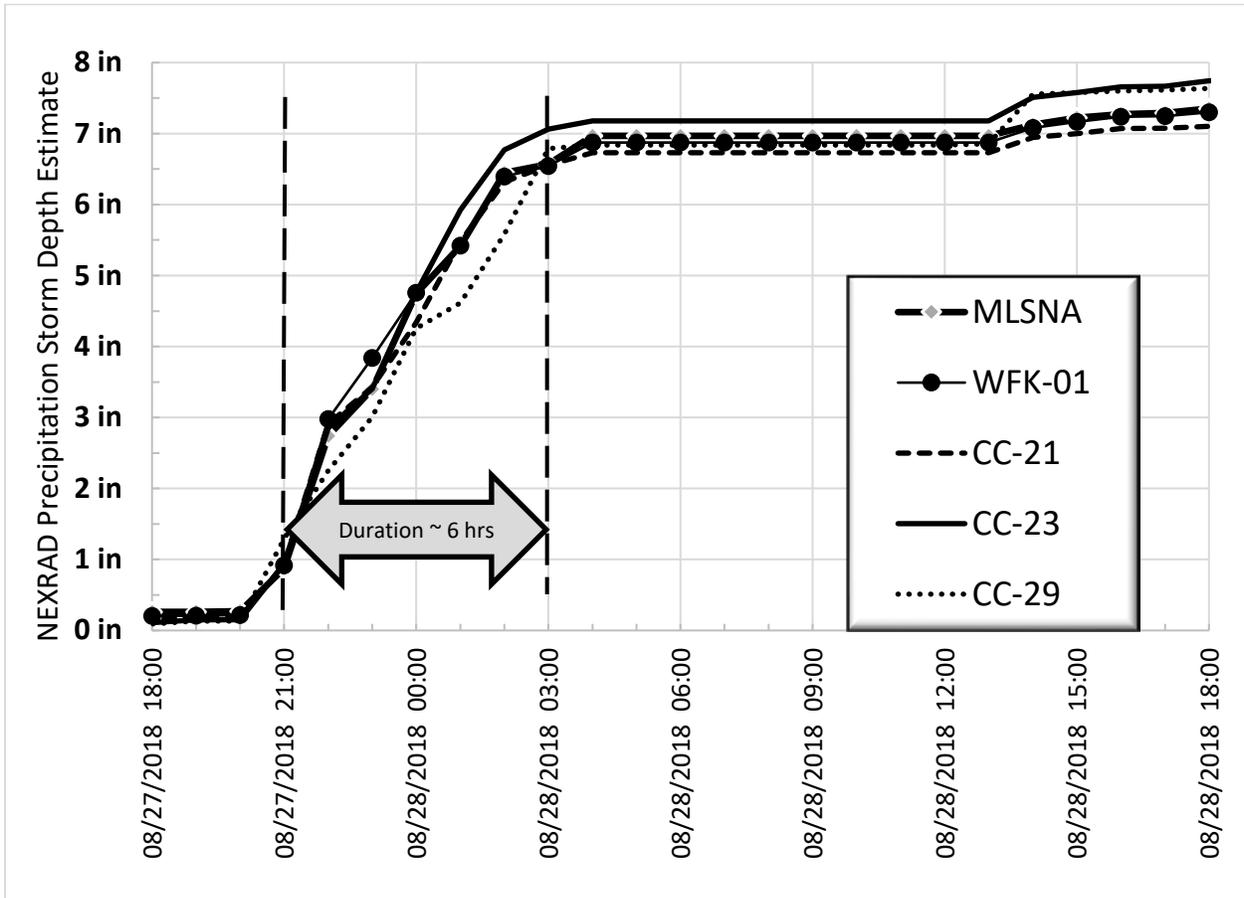


Figure 11 Storm Duration from NEXRAD Data KARX-LaCrosse, WI

4) Rainfall Expected Frequency Estimates – National Weather Service NOAA Atlas 14 for Viroqua (Vernon County) and Cashton (Monroe County)

Figure 12 plots the NOAA Atlas 14 point precipitation frequency estimates for Viroqua, WI in graphical form. Figure 13 is the same estimate presented in tabular form. NOAA Atlas 14 values were created prior to the August 2018 storm event.

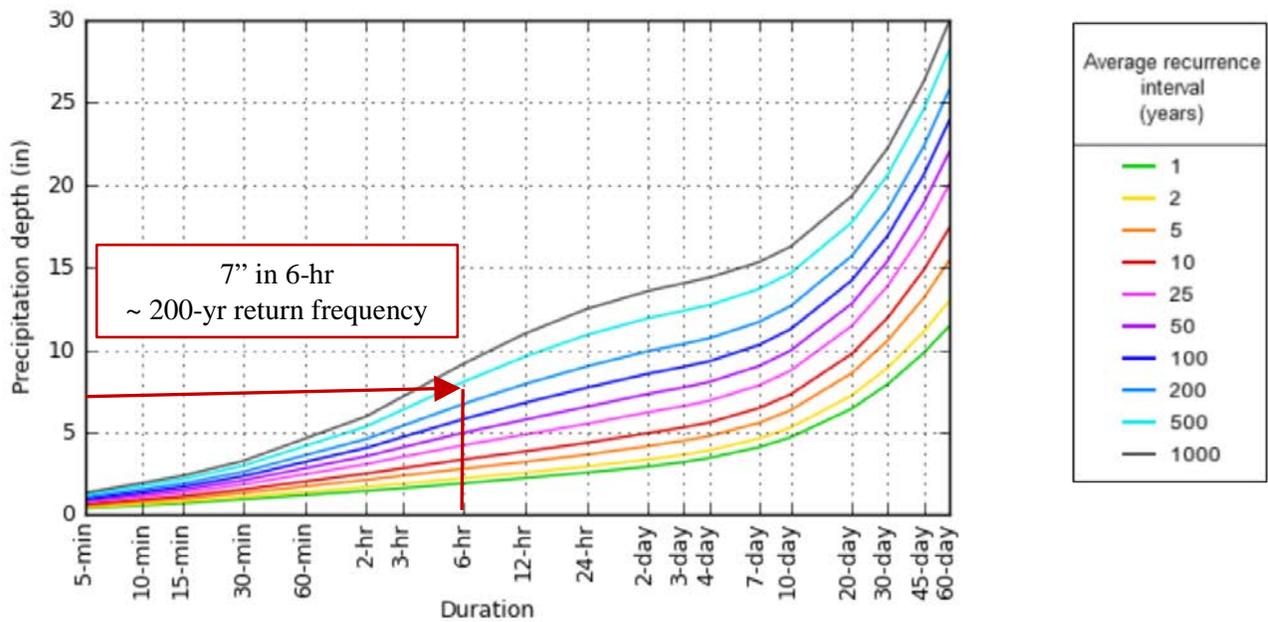
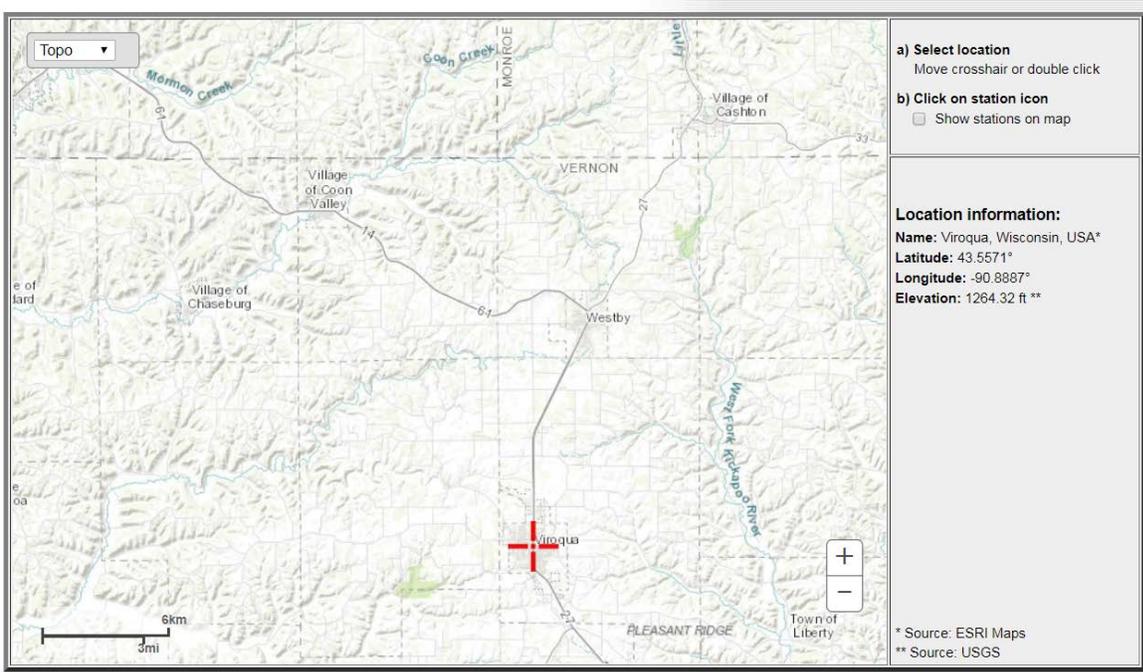


Figure 12 NOAA Atlas 14 point precipitation frequency estimates for Viroqua, WI graphical data

## PF tabular – Viroqua, WI (NOAA Atlas 14)

**PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)<sup>1</sup>**

Duration	Average recurrence interval (years)						
	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr	1,000-yr
<b>1-hr</b>	<b>2.01</b> (1.57-2.55)	<b>2.45</b> (1.87-3.24)	<b>2.81</b> (2.09-3.77)	<b>3.19</b> (2.30-4.39)	<b>3.59</b> (2.49-5.10)	<b>4.16</b> (2.77-6.07)	<b>4.61</b> (2.99-6.81)
<b>2-hr</b>	<b>2.49</b> (1.97-3.12)	<b>3.06</b> (2.36-4.03)	<b>3.53</b> (2.66-4.71)	<b>4.04</b> (2.94-5.53)	<b>4.58</b> (3.20-6.46)	<b>5.35</b> (3.60-7.77)	<b>5.96</b> (3.90-8.76)
<b>3-hr</b>	<b>2.81</b> (2.24-3.50)	<b>3.49</b> (2.72-4.60)	<b>4.07</b> (3.09-5.42)	<b>4.70</b> (3.45-6.43)	<b>5.38</b> (3.79-7.58)	<b>6.35</b> (4.30-9.21)	<b>7.14</b> (4.69-10.4)
<b>6-hr</b>	<b>3.34</b> (2.69-4.12)	<b>4.21</b> (3.33-5.53)	<b>4.97</b> (3.82-6.59)	<b>5.80</b> (4.31-7.91)	<b>6.72</b> (4.79-9.44)	<b>8.06</b> (5.52-11.6)	<b>9.16</b> (6.07-13.3)
<b>12-hr</b>	<b>3.83</b> (3.12-4.68)	<b>4.85</b> (3.90-6.35)	<b>5.76</b> (4.50-7.61)	<b>6.78</b> (5.10-9.20)	<b>7.92</b> (5.71-11.1)	<b>9.59</b> (6.63-13.8)	<b>11.0</b> (7.33-15.9)
<b>24-hr</b>	<b>4.35</b> (3.59-5.26)	<b>5.50</b> (4.48-7.14)	<b>6.54</b> (5.16-8.56)	<b>7.69</b> (5.85-10.4)	<b>8.99</b> (6.54-12.5)	<b>10.9</b> (7.60-15.6)	<b>12.5</b> (8.40-17.9)
<b>2-day</b>	<b>4.94</b> (4.13-5.93)	<b>6.21</b> (5.10-7.95)	<b>7.32</b> (5.83-9.47)	<b>8.55</b> (6.56-11.4)	<b>9.91</b> (7.27-13.6)	<b>11.9</b> (8.38-16.9)	<b>13.6</b> (9.21-19.3)
<b>3-day</b>	<b>5.29</b> (4.46-6.31)	<b>6.59</b> (5.44-8.37)	<b>7.72</b> (6.18-9.93)	<b>8.97</b> (6.92-11.9)	<b>10.4</b> (7.64-14.2)	<b>12.4</b> (8.74-17.4)	<b>14.0</b> (9.58-19.9)
<b>4-day</b>	<b>5.59</b> (4.74-6.64)	<b>6.90</b> (5.73-8.72)	<b>8.05</b> (6.47-10.3)	<b>9.30</b> (7.21-12.3)	<b>10.7</b> (7.92-14.6)	<b>12.7</b> (9.01-17.9)	<b>14.4</b> (9.85-20.4)
<b>7-day</b>	<b>6.46</b> (5.53-7.61)	<b>7.83</b> (6.54-9.76)	<b>9.01</b> (7.30-11.4)	<b>10.3</b> (8.02-13.4)	<b>11.7</b> (8.70-15.7)	<b>13.7</b> (9.76-19.1)	<b>15.3</b> (10.6-21.6)
<b>10-day</b>	<b>7.28</b> (6.28-8.54)	<b>8.73</b> (7.31-10.8)	<b>9.94</b> (8.09-12.5)	<b>11.2</b> (8.81-14.6)	<b>12.6</b> (9.46-16.9)	<b>14.6</b> (10.5-20.3)	<b>16.3</b> (11.3-22.8)

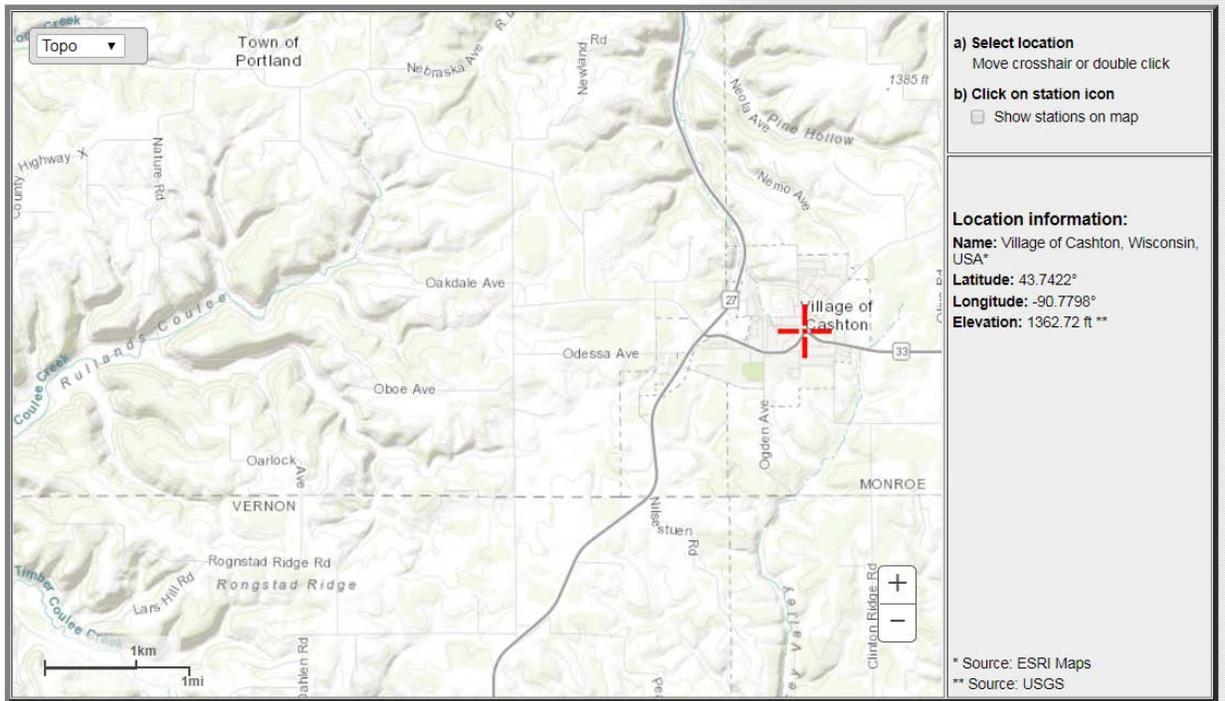
<sup>1</sup> Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

*Figure 13 NOAA Atlas 14 point precipitation frequency estimates for Viroqua, WI tabular data*

Figure 14 plots the NOAA Atlas 14 point precipitation frequency estimates for Cashton, WI in graphical form. Figure 15 is the same estimates presented in tabular form. NOAA Atlas 14 values were created prior to the August 2018 storm event.



PDS-based depth-duration-frequency (DDF) curves  
 Latitude: 43.7422°, Longitude: -90.7798°

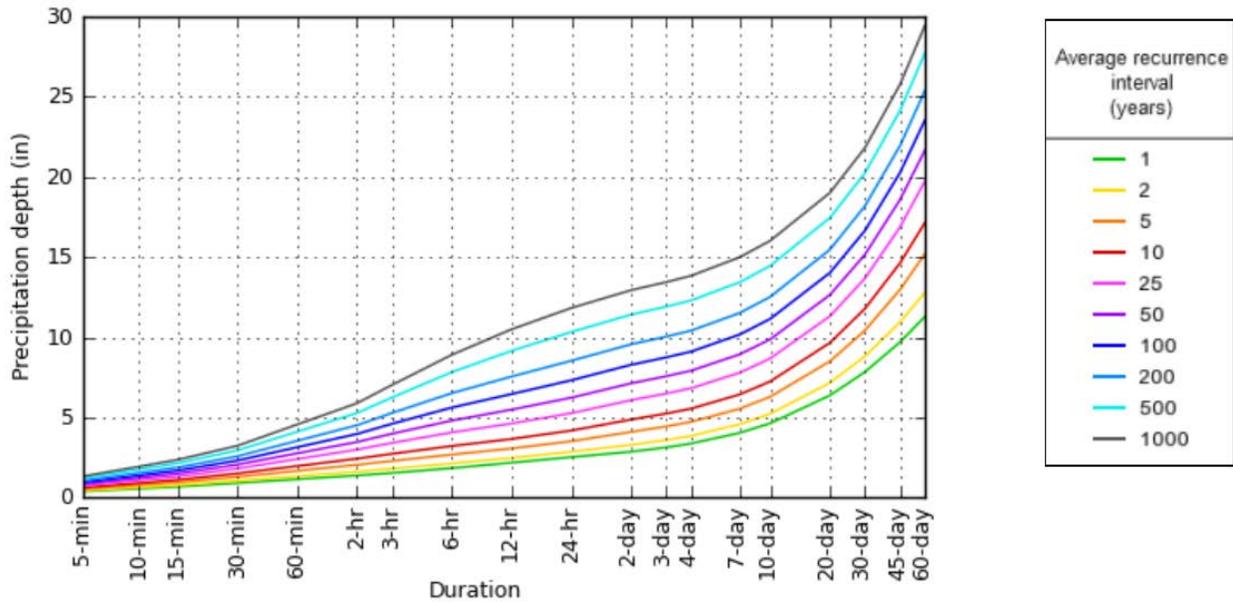


Figure 14 NOAA Atlas 14 point precipitation frequency estimates for Cashton, WI graphical data

## PF Tabular – Cashton, WI (NOAA Atlas 14)

### PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)<sup>1</sup>

Duration	Average recurrence interval (years)						
	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr	1,000-yr
1-hr	<b>1.96</b> (1.51-2.49)	<b>2.40</b> (1.81-3.17)	<b>2.76</b> (2.04-3.69)	<b>3.14</b> (2.26-4.29)	<b>3.54</b> (2.47-4.96)	<b>4.11</b> (2.78-5.90)	<b>4.56</b> (3.01-6.62)
2-hr	<b>2.42</b> (1.88-3.05)	<b>2.99</b> (2.28-3.94)	<b>3.46</b> (2.58-4.61)	<b>3.97</b> (2.88-5.40)	<b>4.51</b> (3.17-6.29)	<b>5.27</b> (3.59-7.54)	<b>5.88</b> (3.91-8.49)
3-hr	<b>2.73</b> (2.13-3.43)	<b>3.41</b> (2.62-4.49)	<b>3.98</b> (2.99-5.29)	<b>4.60</b> (3.36-6.26)	<b>5.28</b> (3.73-7.35)	<b>6.24</b> (4.28-8.91)	<b>7.02</b> (4.69-10.1)
<b>6-hr</b>	<b>3.22</b> (2.53-4.01)	<b>4.06</b> (3.17-5.36)	<b>4.80</b> (3.65-6.37)	<b>5.61</b> (4.15-7.62)	<b>6.51</b> (4.66-9.06)	<b>7.82</b> (5.41-11.1)	<b>8.90</b> (5.98-12.7)
12-hr	<b>3.65</b> (2.90-4.52)	<b>4.62</b> (3.66-6.08)	<b>5.49</b> (4.22-7.26)	<b>6.46</b> (4.83-8.73)	<b>7.55</b> (5.45-10.5)	<b>9.15</b> (6.39-13.0)	<b>10.5</b> (7.10-14.9)
24-hr	<b>4.18</b> (3.36-5.14)	<b>5.26</b> (4.20-6.88)	<b>6.23</b> (4.84-8.19)	<b>7.32</b> (5.51-9.83)	<b>8.54</b> (6.21-11.7)	<b>10.3</b> (7.26-14.6)	<b>11.8</b> (8.05-16.7)
2-day	<b>4.87</b> (3.95-5.94)	<b>6.08</b> (4.87-7.83)	<b>7.13</b> (5.57-9.26)	<b>8.28</b> (6.28-11.0)	<b>9.56</b> (6.99-13.0)	<b>11.4</b> (8.06-15.9)	<b>12.9</b> (8.87-18.1)
3-day	<b>5.24</b> (4.28-6.37)	<b>6.49</b> (5.22-8.30)	<b>7.57</b> (5.94-9.77)	<b>8.74</b> (6.65-11.5)	<b>10.0</b> (7.37-13.6)	<b>11.9</b> (8.44-16.5)	<b>13.4</b> (9.25-18.7)
4-day	<b>5.55</b> (4.55-6.72)	<b>6.82</b> (5.50-8.69)	<b>7.91</b> (6.23-10.2)	<b>9.11</b> (6.95-12.0)	<b>10.4</b> (7.66-14.0)	<b>12.3</b> (8.73-17.0)	<b>13.8</b> (9.55-19.2)
7-day	<b>6.42</b> (5.31-7.74)	<b>7.78</b> (6.31-9.81)	<b>8.93</b> (7.07-11.4)	<b>10.2</b> (7.80-13.3)	<b>11.5</b> (8.51-15.4)	<b>13.4</b> (9.58-18.4)	<b>15.0</b> (10.4-20.7)
10-day	<b>7.24</b> (6.02-8.70)	<b>8.68</b> (7.05-10.9)	<b>9.87</b> (7.84-12.5)	<b>11.1</b> (8.58-14.4)	<b>12.5</b> (9.28-16.7)	<b>14.5</b> (10.3-19.7)	<b>16.0</b> (11.2-22.1)

<sup>1</sup> Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

*Figure 15 NOAA Atlas 14 point precipitation frequency estimates for Cashton, WI tabular data*

#### D. Estimated precipitation return frequency interval

Figure 16 shows the expected return frequency for two different reported amounts, the NWS quantitative precipitation estimation 24-hour estimate and the NWS NEXRAD radar estimate. It is not clear what time is covered in the NWS QPE 24-hour storm rainfall—it is unknown if the rainfall on the evening of August 27 is included or if rainfall on the afternoon is included. The critical rainfall that cause the breach of the five dams occurred on the evening of August 27 and early morning of August 28. The breaches were confirmed by personnel site visits during the day of August 28. After the site visits, additional rain fell that day.

Wikipedia (accessed 6-20-2019) describes Quantitative precipitation estimation or QPE as a method of approximating the amount of precipitation that has fallen at a location or across a region. Maps of the estimated amount of precipitation to have fallen over a certain area and time span are compiled using several different data sources including manual and automatic field observations and radar and satellite data. This process is undertaken every day across the United States at Weather Forecast Offices (WFOs) run by the National Weather Service (NWS).

County	Watershed Name	Local Name	Ending Date of Storm	QPE* 24-hr		NEXRAD 6-hr	
				QPE* Storm Rainfall (Inches)	Return Freq (Years)	6-hr Storm Rainfall	Return Freq (Years)
Monroe	Coon Creek 21	Luckasson	2018-08-28	10.4 in	500-yr	7.1 in	400-yr
Monroe	Coon Creek 23	Bilhovde	2018-08-28	11.0 in	600-yr	7.8 in	400-yr
Monroe	Coon Creek 29	Korn	2018-08-28	10.4 in	500-yr	7.6 in	400-yr
Vernon	West Fork Kickapoo 1	Jersey Valley	2018-08-28	9.4 in	300-yr	7.3 in	400-yr
Vernon	Mlsna	Mlsna	2018-08-28	9.4 in	300-yr	7.4 in	400-yr
QPE* - quantified precipitation estimate							

*Figure 16 Expected return frequency of storm rainfall*

Regardless of which data is used, the rainfall return frequency falls within the 300-year to 600-year interval.

## E. Design Hydrology Criteria

Figure 17 lists the design rainfall information gleaned from the original as-built drawings, arranged by drainage area. The design rainfall depth was not available for the two smallest dams (Mlsna and Bilhovde).

<b>Dam</b>	<b>Local Name</b>	<b>County</b>	<b>Design Rainfall Depth</b>	<b>Design Rainfall Duration</b>	<b>Drainage Area</b>
West Fork Kickapoo 1	Jersey Valley	Vernon	10.9 inches	6 hours	8.06 sq mi
Coon Creek 21	Luckasson	Monroe	5.8 inches	6 hours	3.16 sq mi
Coon Creek 29	Korn	Monroe	7.17 inches	6 hours	2.88 sq mi
West Fork Kickapoo	Mlsna	Vernon	NA	NA	1.48 sq mi
Coon Creek 23	Bilhovde	Monroe	NA	NA	1.42 sq mi

*Figure 17 Design rainfall depth and duration, sorted by drainage area*

Jersey Valley has the largest drainage area (> 8 square miles) and largest freeboard design rainfall depth, nearly 11 inches. This dam was the only breached dam that did not overtop. There are two possible explanations why Jersey Valley did not overtop:

1. The storm rainfall depth was less than the design rainfall depth (11 inches). If this was the case, there was insufficient storm runoff to overtop the dam.
2. The auxiliary spillway breached before the reservoir level was high enough to overtop the embankment. In this case, rapid breach erosion in the auxiliary spillway channel and/or right abutment provided increased discharge capacity to prevent the dam from overtopping due to storm runoff that may have greatly exceeded the original design runoff.

The breaches for all the dams occurred in a short time span since all the breaches occurred during the night, before anyone could visit the dams after daylight on Tuesday, August 28<sup>th</sup>. Since the drainage area at Jersey Valley was larger than the other dams, it would take longer for storm runoff to reach the Jersey Valley reservoir and begin to overtop the dam. Solely because the Jersey Valley dam did not overtop, one cannot conclude the August storm rainfall depth was less than or greater than the design rainfall depth.

The Luckasson and Korn dams were designed for 6- to 7-inch rainfall depths. Flattened vegetation on the backslopes of the embankments indicates overtopping. Since these dams overtopped, the storm rainfall depth was probably greater than the original design rainfall depth.

Mlsna and Bilhovde dams have the smallest drainage area (< 2 square miles), so the design rainfall depth would likely be slightly smaller or equal than Luckasson and Korn designs. Mlsna and Bilhovde dams overtopped, so it is likely that the storm rainfall depth was greater than the design rainfall depth.

The design rainfall duration is listed as 6 hours, which is the appropriate design storm duration criteria listed in Engineering Memo 3 (issued 1956) and Engineering Memo 27 (originally issued 1965, supplements released through 1976). The dams were built in the 1950s and 1960s and have performed well, without any hydrologic/hydraulic capacity issues for the last 50 years.

There is no evidence that the five dams breached because the hydrologic design criteria was insufficient. Rather, the highly unusual August 2018 storm rainfall was greater than the original design rainfall for the four dams that overtopped.

Since Jersey Valley did not overtop, insufficient hydrologic capacity did not cause the breach. Instead, it appears that high volumes of storm runoff in the reservoir were sufficient to cause rapid erosion of the auxiliary spillway and nearby right abutment before the water surface in the reservoir could reach top of dam.

Attachment G  
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