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

June 26, 2019

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Table of Contents

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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
1	Jersey Valley	6	14N	3W	43.6891°	-90.7989°
Mlsna	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
21	Luckasson	34	15N	4W	43.7314°	-90.8420°
23	Bilhovde	27	15N	4W	43.7430°	-90.8416°
29	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 at site WFK Mlsna to 650 years at CC-23. These rainfall amounts greatly exceeded the principal spillway design storms which had a recurrence interval of 100 years, according to the Work Plan and as-built drawings. 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 WKF-1 was set according to a freeboard hydrograph applying a six-hour distribution with a point rainfall of 10.9 inches. The estimated rainfall at WFK-1 was 9.44 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

<u>Committee Activities:</u>

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 18th and 19th. One PL-566 site, WFK 3, and one pilot watershed site, Klinkner Pilot, that were in proximity to WFK-1 and Mlsna, 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 Mlsna (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 in light of the site observations. 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. Was 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:

Scope.

The scope of this investigation is as prescribed in the National Engineering Manual, Part 504.4. The committee was charged with investigating PL-566 dams in Wisconsin that failed during historic rainfall. Overtopping with groin erosion and auxiliary spillway damage due to material weakness resulted in the breach of five dams.

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, brush in 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 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 28th:

  > *"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 27th and early morning Tuesday, August 28th.

- 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 28th. 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 28th. 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 28th. 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.

<u>Site Inspections</u>

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. And 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 and followed that surface until it reached bedrock at the bottom. 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 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 estimate 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	Arbritrary (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 Mlsna and WFK-1 were designed as Low hazard dams with a planned service life of 50 years. WFK Mlsna 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 earthfall dams with zoned embankments (see Attachment C, WFK Mlsna, Sheet 3 of 15 and WFK-1, Sheet 1 of 4). WFK Mlsna 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 Mlsna. 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.

<u>Hydrology</u>

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 shows the radar precipitation estimate from NEXRAD radar station KARX in LaCrosse, WI for the 24-hour period August 27 18:00 through August 28 18:00, which is the time period of the rainfall causing the five breaches. The storm duration is approximately 6 or 7 hours. Additional precipitation estimates from various sources can be found in the Detailed Hydrology Report as an attachment.

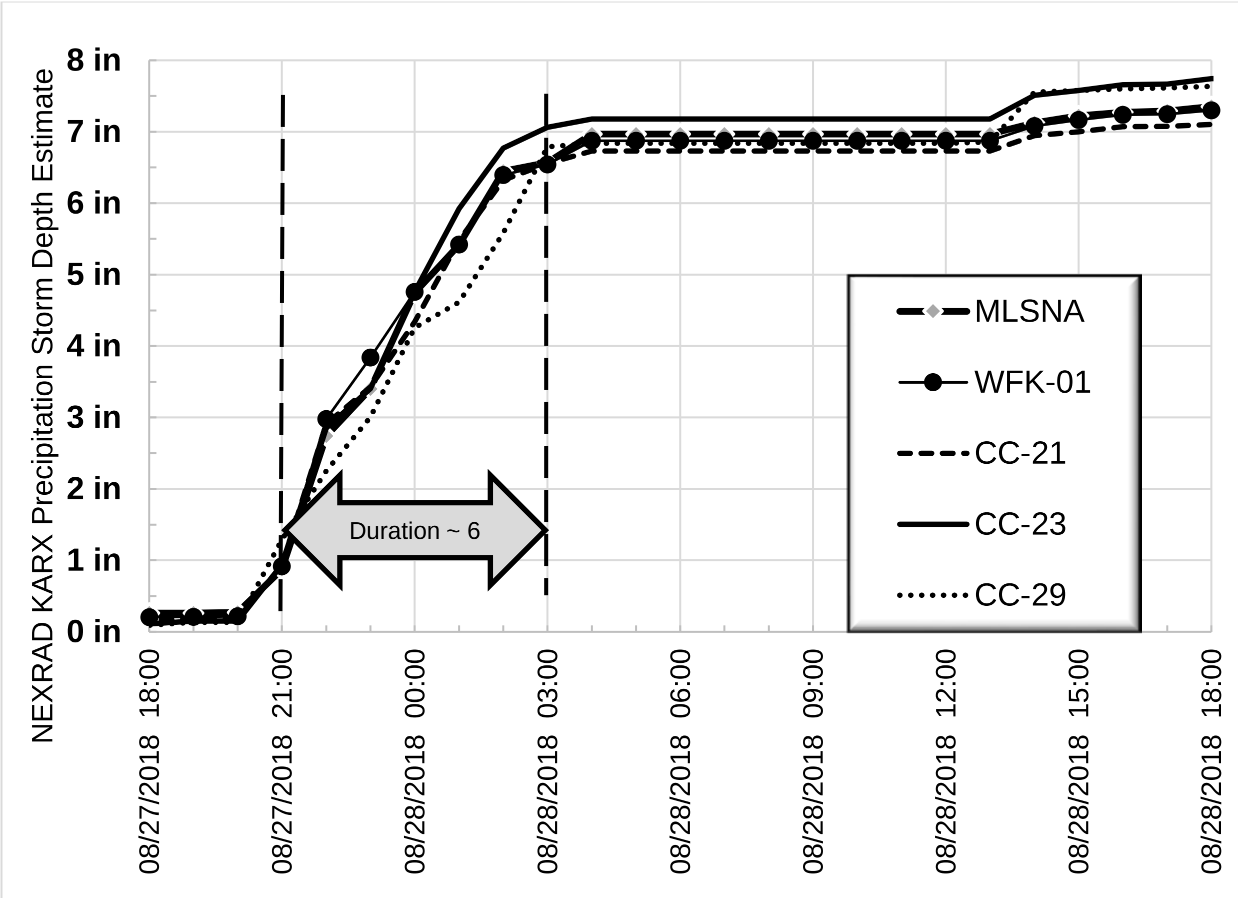


Figure 1 Storm Rainfall from NEXRAD Data KARX-LaCrosse, WI

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

				QPE* 24-hr		NEXRAD 6-hr	
County	**Watershed Name**	**Local Name**	**Ending Date of Storm**	**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, it is clear that the rainfall return frequency is in the 300-year to 600-year interval.

Table 4 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 Mlsna dam.

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

Dam	Local Name	County	Design Rainfall Depth	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	5.8 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	5.79 inches	6 hours	1.42 sq mi

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 <u>before</u> the reservoir level was high enough to overtop the embankment. If the case was that 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.

The Luckasson, Korn, and Bilhovde 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 be 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 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. A model of the storm in SITES resulted in a breach of the auxiliary spillway with a water surface elevation approximately five feet 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.

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 Ordivician 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. Due to the fact that 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).

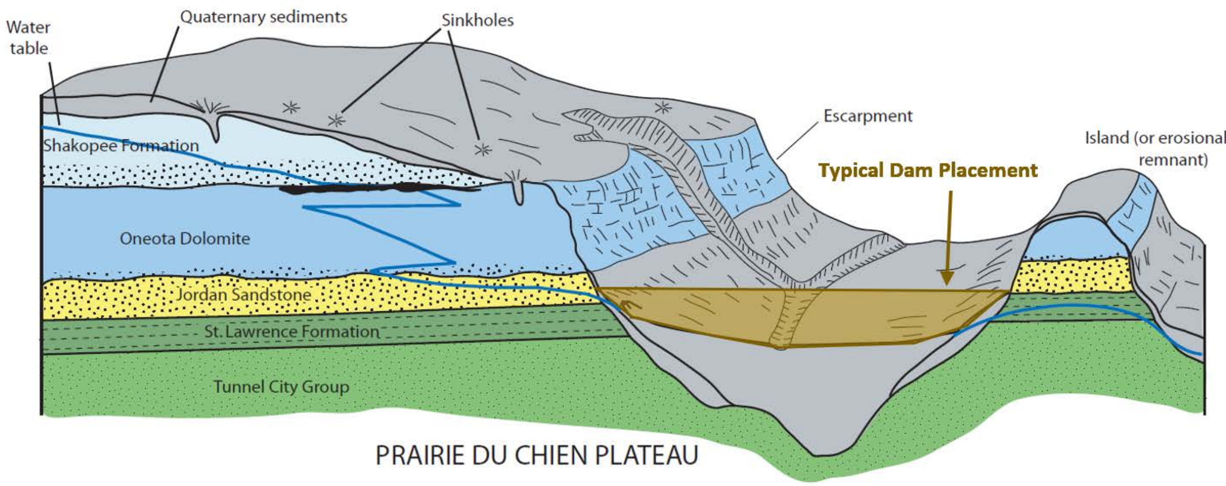


Figure 2- Block Diagram showing the general geologic setting of the Prairie Du Chien Plateau and the PL-566 structures breached in the August 2018 flood event. The typical dam placement is shown in brown. Note that this is just for illustrative purposes. The dam could be slightly higher or lower depending 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.

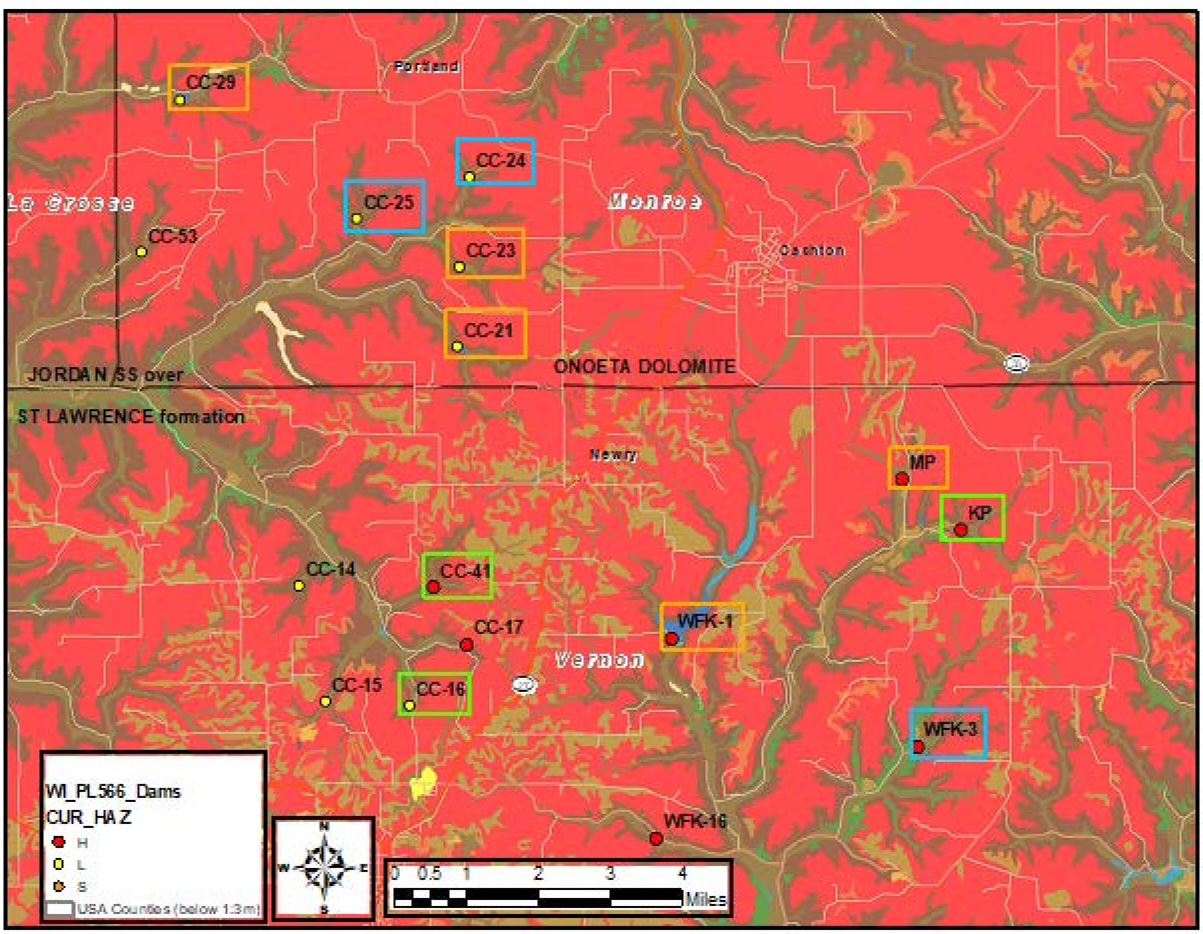


Figure 3- Map of PL-566 Structures in the location with the highest rainfall totals during the August 28, 2018 rain event. This figure shows the five dams that breached (orange box), three sites that overtopped but did not breach

(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 Mlsna 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.

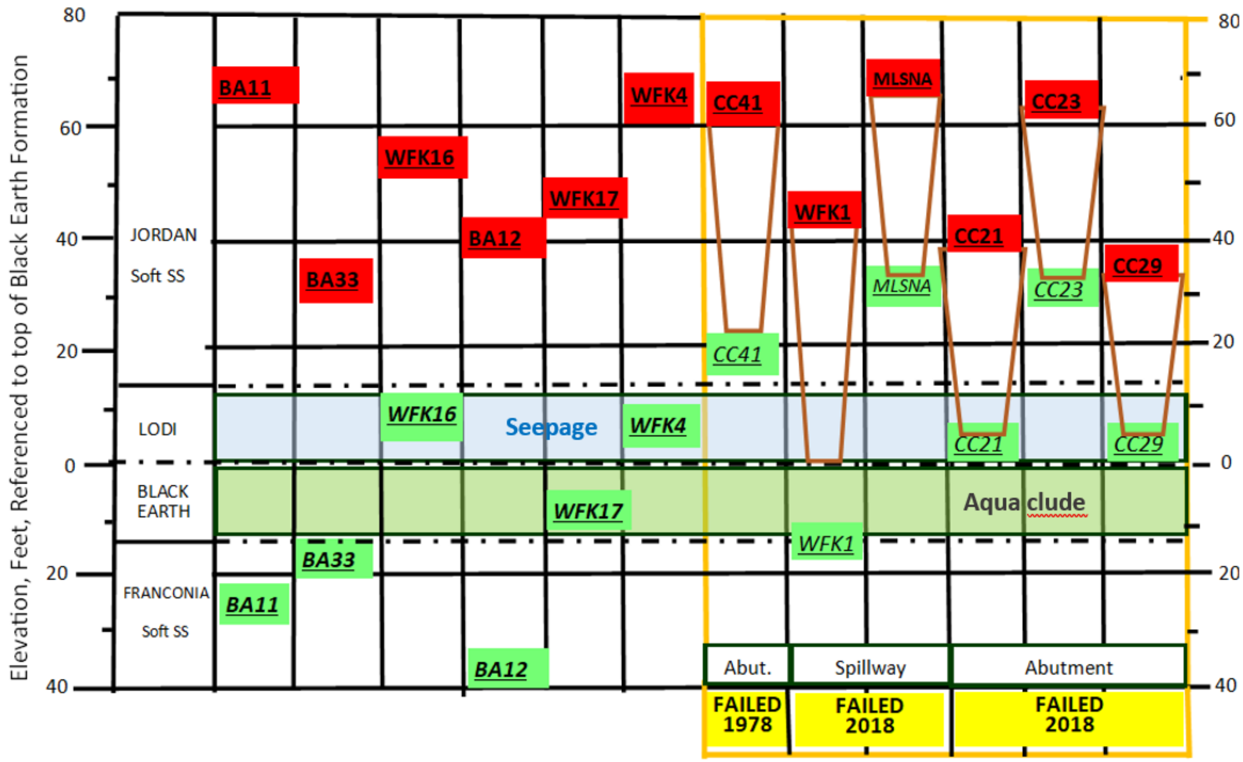


Figure 4- Effective Stratigraphy of selected PL-566 dams in WI. Top of dam elevation shown in red and bottom of core trench elevation shown in green. Brown polygons have been drawn to show the relative elevation of breaches.

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.

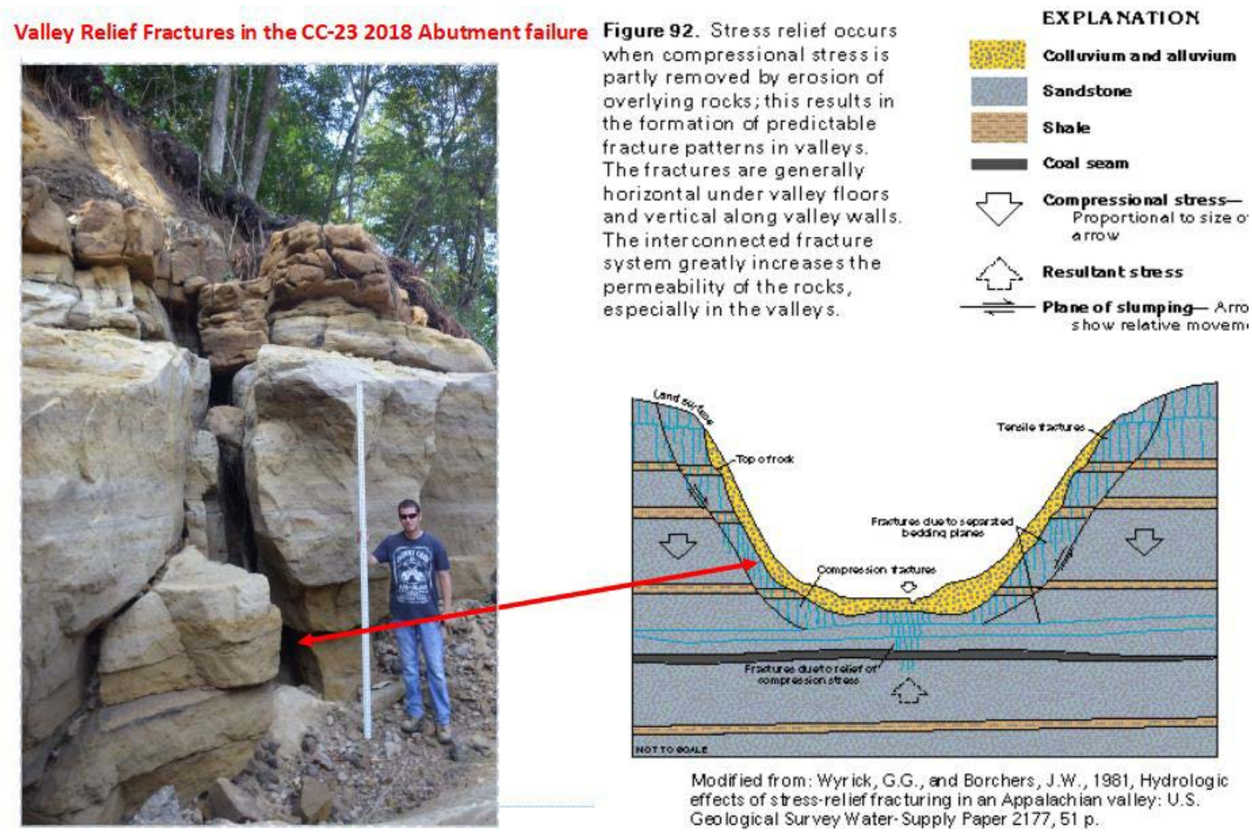
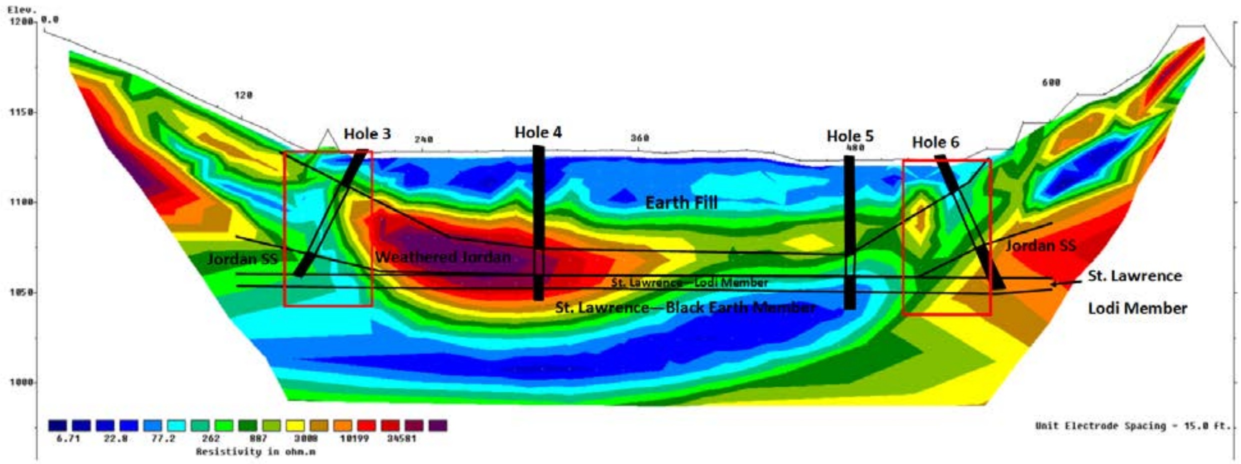


Figure 5 provides an explanation of how valley stress relief fractures form, including a photo of an interpreted valley relief fracture in the wall of the CC-23 abutment. These fractures run parallel to the valley flow direction which make them a likely bypass for the structure.

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.

Potential Geologic Failure Mechanisms and likely risks to other PL-566 dams:

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 was 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
Subtotal	**62**	**55**	**48.4**	**47.0**
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
Subtotal	**59**	**57**	**46.1**	**48.7**
Slides	7	5	5.5	4.3
Earthquake/liquefaction	2	2	1.6	1.7
Unknown	8	7		
Total number of failures	**136**	**124**		

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

A combination of the potential flaws (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 and seepage velocity increased. 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 flaws 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 flaws (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.

<u>Records, Reports, and Documents Reviewed</u>

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:

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:

<u>Summary of Conclusions</u>:

- The applicable criteria at the time of design of Coon Creek 23 required geologic investigation of the foundation and abutments. However, 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. Criteria also called for geologic investigation to ensure that movement of water through the foundation and/or abutment would not cause: piping and scour under the dam; excessive loss of water stored behind the dam; or the formation of excessive uplift pressure. Due to the fractures and differentially cemented sandstone, all these conditions were present in the abutments. Based on these facts, the committee has concluded that there was a deficiency in the geologic investigation of Coon Creek 23.
- 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 prevalent at these sites. On WFK-1 a significant amount of the valley relief fractures had been successfully pressure grouted. On WFK Mlsna there just wasn't the presence of fractures at the exposed bedrock surface. The case-hardened sandstone withstood the breach flow. The fact that four of the five dams were in service 50-60 years without failure, or even major O&M issues, provides practical evidence of the quality of design and construction. See the Geology section for a detailed evaluation of the foundation materials and potential failure mechanisms.
- WFK Mlsna 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 Mlsna could not be positively confirmed, the committee has concluded that it is reasonable to believe EM SCS-27 was followed on WFK Mlsna 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.

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. Use slope mapping in ArcMap as a screening tool for prioritizing and selecting sites for the Geophysics analysis.

   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 provide overtopping sheet flow and to prevent concentration of overtopping flow down the groins.

      iii. Build up the groins with earthfill to make flatter gutters and keep flow off the fragile timber soils.

   d. Review Emergency Action Plans to make sure contact information and actions planned are up to date. Review the protocols with appropriate personnel.

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 Digitally signed by MARK MCCURDY
Date: 2019.07.03 16:10:09 -05'00'

Mark McCurdy, Asst. SCE, Des Moines, IA

KARL VISSER Digitally signed by KARL VISSER
Date: 2019.07.09 08:10:53
-05'00'

Karl Visser, Hydraulic Engineer, NDCSMC, Fort Worth, TX

TIMOTHY WEISBROD Digitally signed by TIMOTHY WEISBROD
Date: 2019.07.08 16:30:35 -05'00'

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

MATTHEW BLOHOWIAK Digitally signed by MATTHEW BLOHOWIAK
Date: 2019.07.08 08:14:01 -05'00'

Matt Blohowiak, Civil Engineer, Altoona, WI

MICHEL DREISCHMEIER Digitally signed by MICHEL DREISCHMEIER
Date: 2019.07.08 15:43:20 -05'00'

Mike Dreischmeier, Area Engineer, Richland Center, WI

MARK MCCURDY Digitally signed by MARK MCCURDY
Date: 2019.07.15 10:19:51 -05'00'

Revisions by Mark A. McCurdy, July 15, 2019

Attachment A

Letters of Appointment

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USDA

United States Department of Agriculture

SUBJECT: ENG – Dam Failure Investigation Committee Appointment Date: September 12, 2018

TO: Mark McCurdy, Assistant SCE, NRCS, Des Moines, IA File Code: 210
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

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 17th 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.

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

USDA

United States Department of Agriculture

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 17th.

We are requesting your concurrence of the team members.

John R. Ramsden

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

NOLLER HERBERT

Digitally signed by NOLLER HERBERT
Date: 2018.09.11 10:34:48 -04'00'

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 verses 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)

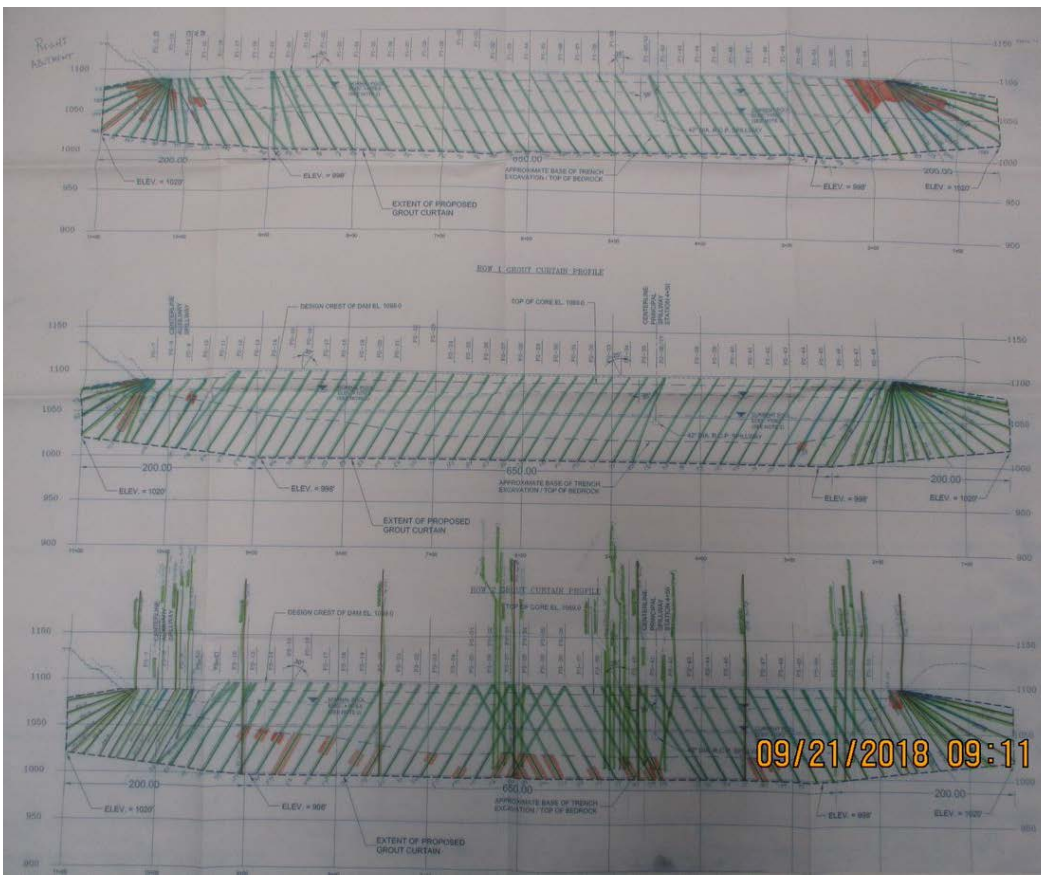

Photo 13
Drilling plan for 2009/2010 contract to install pressure grouted curtain wall.
(WFK 1)

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)

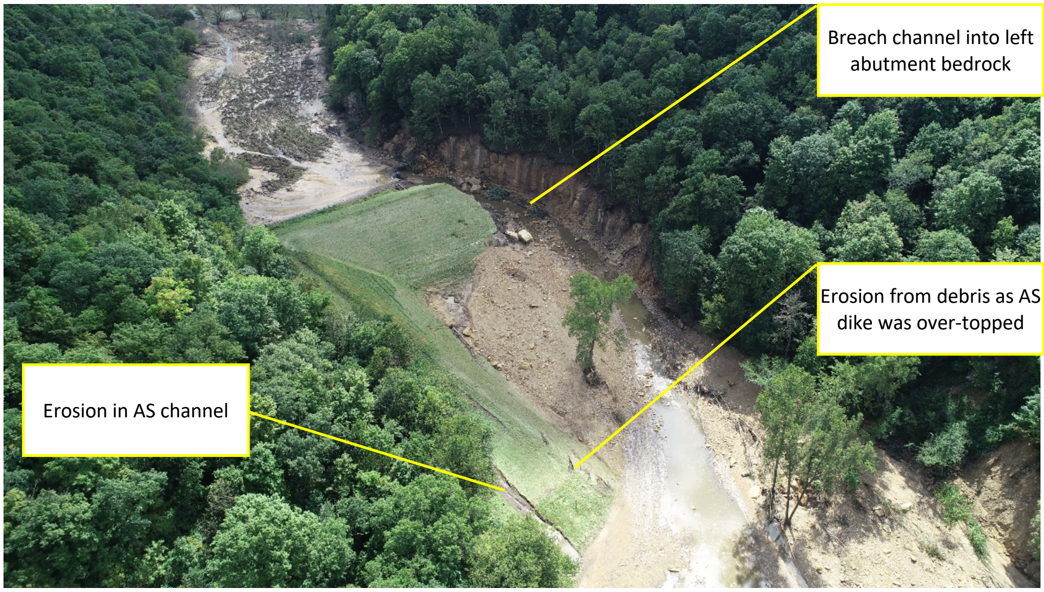

Photo 34
Aerial View
(CC 21)


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)

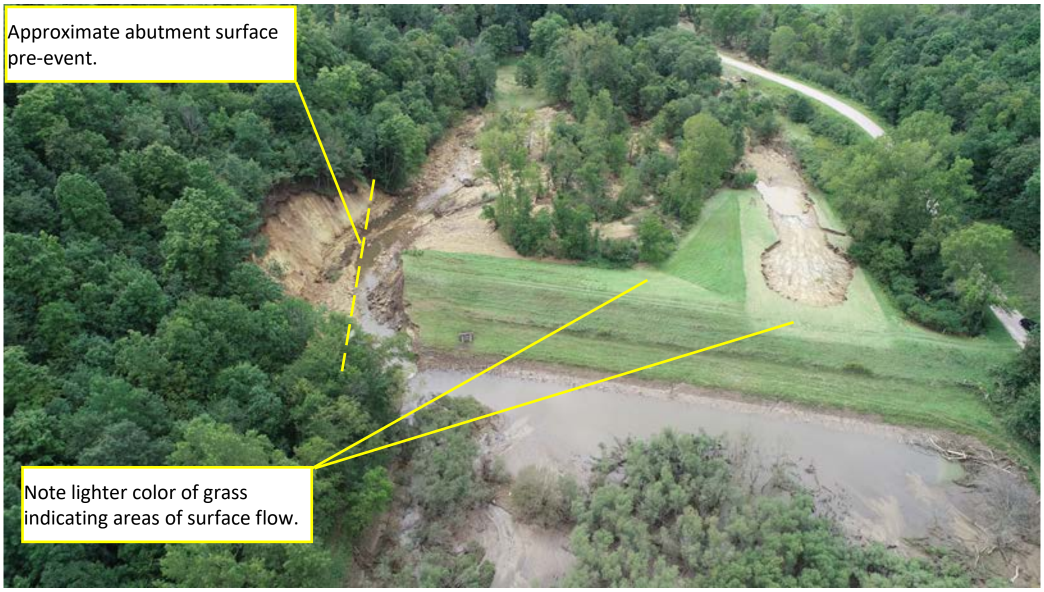

Photo 44
Aerial photo of dam and auxiliary spillway from upstream of dam.
(CC 29)

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

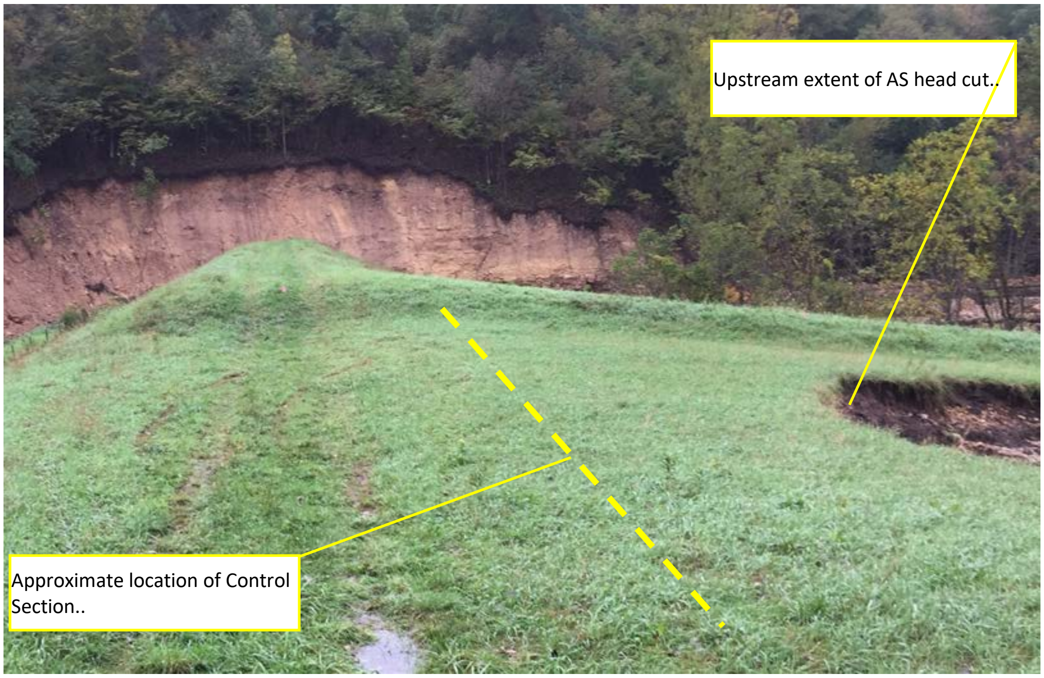

Photo 46
View of upstream end of auxiliary spillway from the right end of top of dam.
(CC 29)


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)

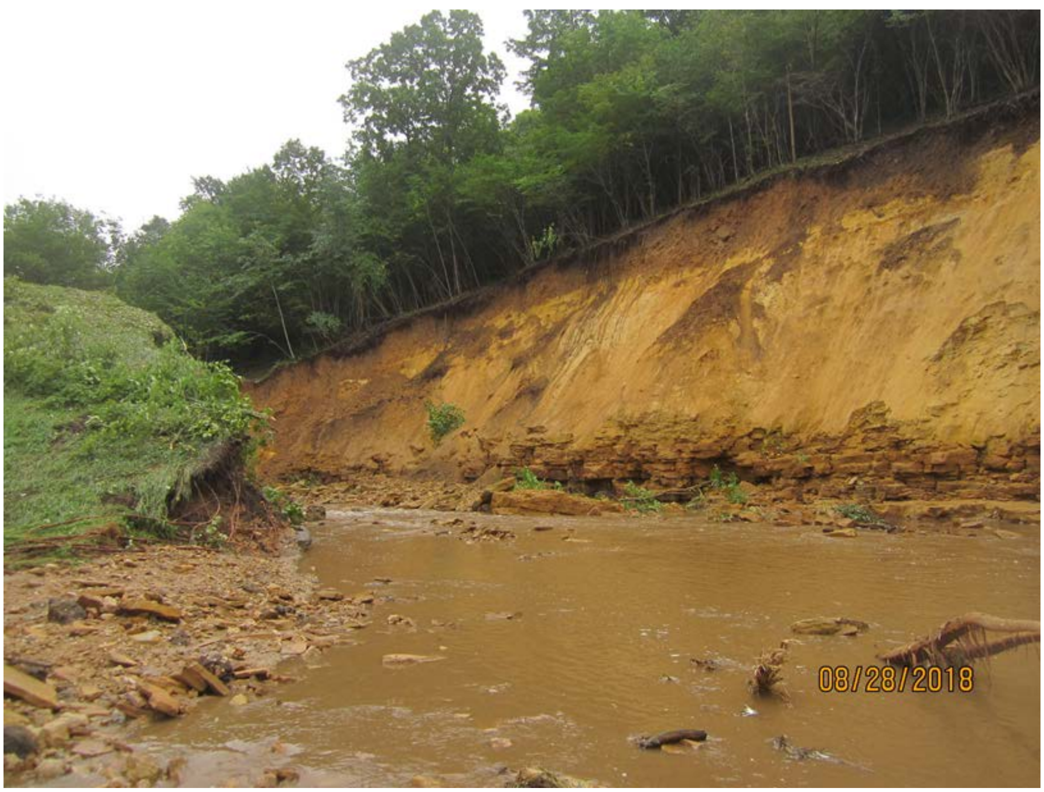

Photo 49
View of breach channel from downstream of the dam. Note photo taken the morning of the event.
(CC 29)

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)

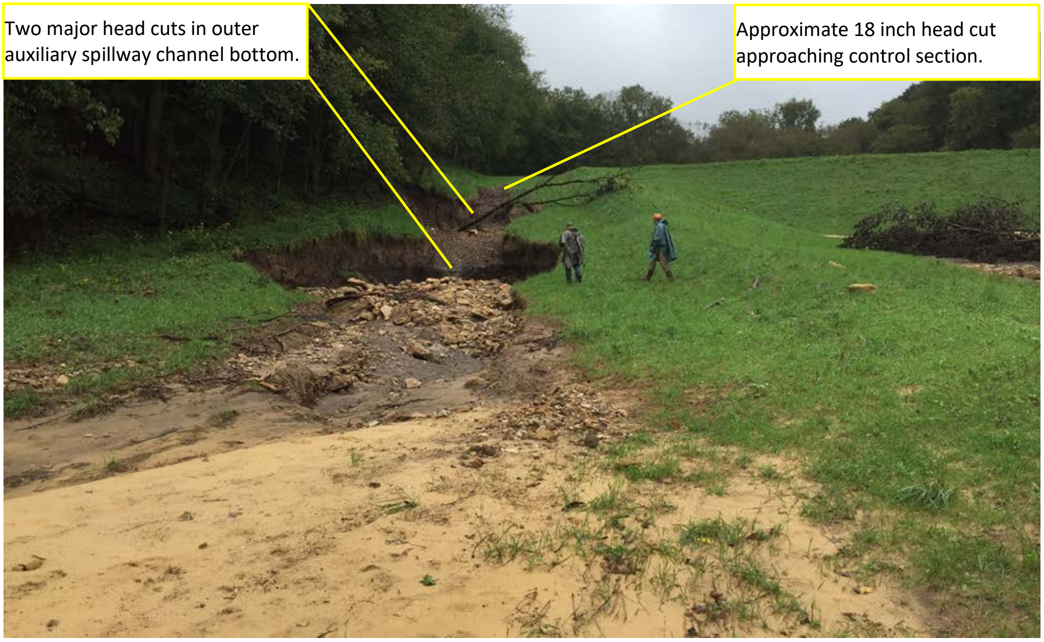

Photo 60
View of auxiliary spillway exit channel from downstream end. Note two large headcuts and one small headcut near control section.
(CC 23)

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)

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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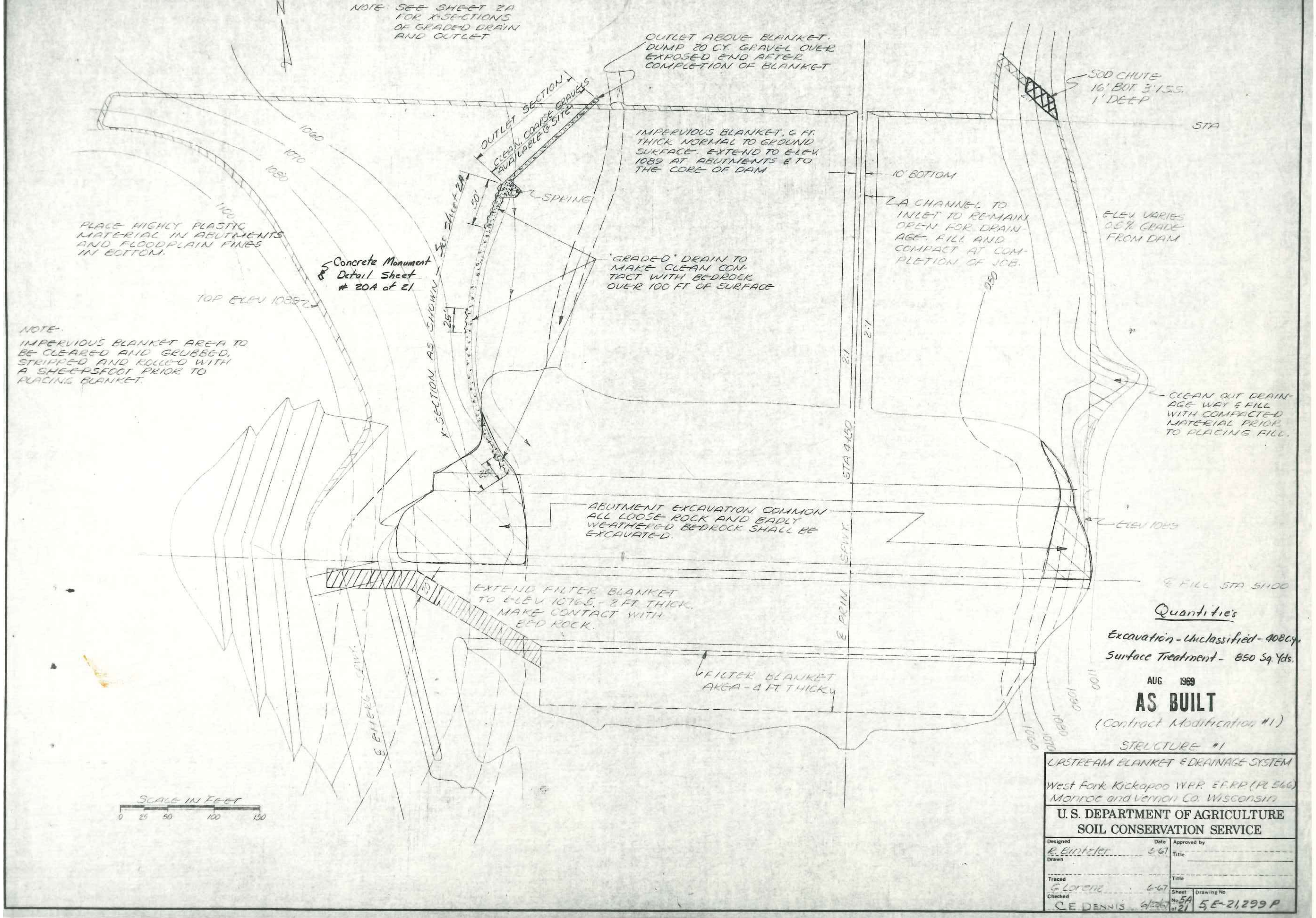
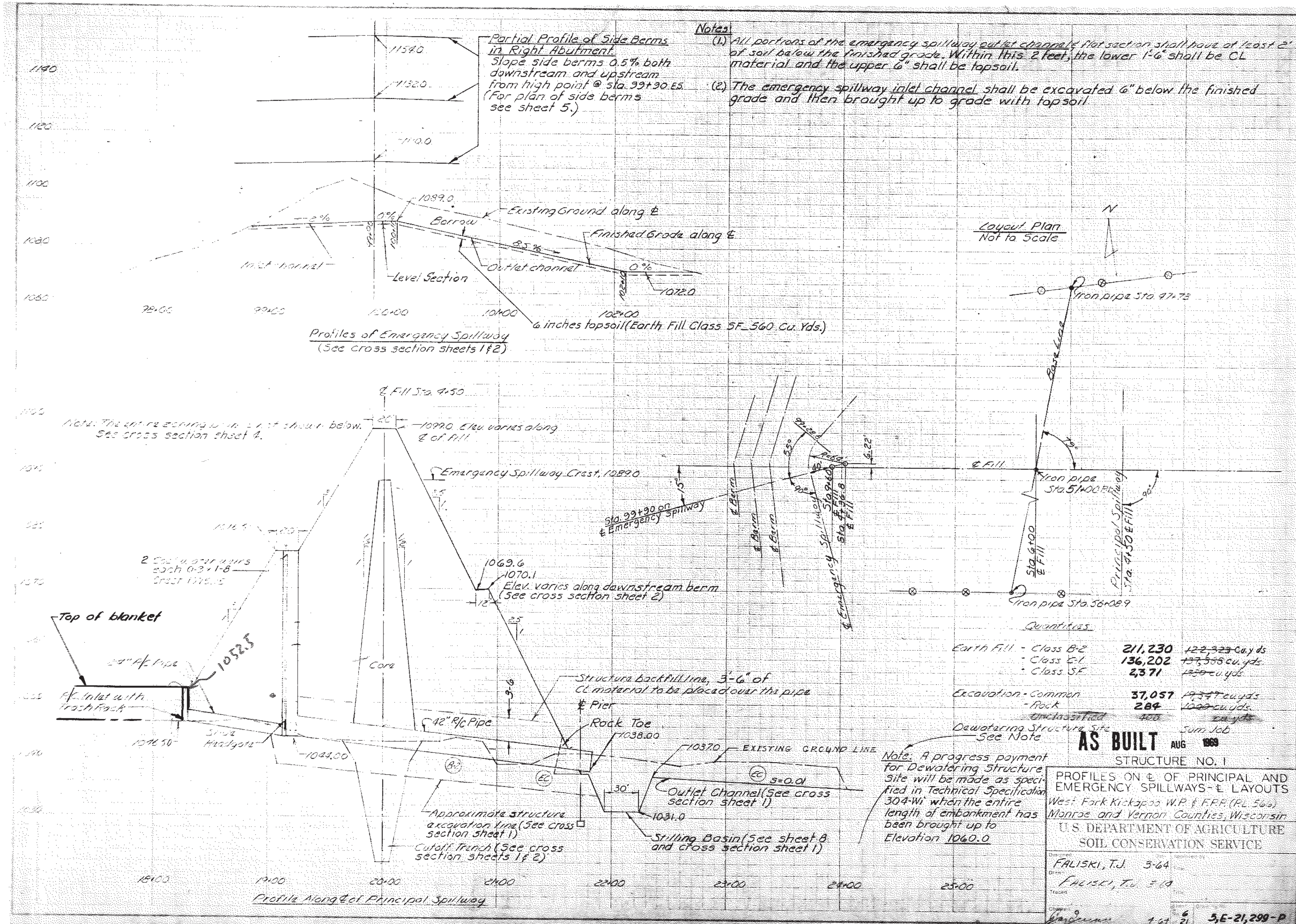


N
NOTE: SEE SHEET 2A FOR X-SECTIONS OF GRADED DRAIN AND OUTLET
OUTLET ABOVE BLANKET. DUMP 20 C.Y. GRAVEL OVER EXPOSED END AFTER COMPLETION OF BLANKET
SOD CHUTE 16' BOT. 3:1 S.S. 1' DEEP
STA
OUTLET SECTION
CLEAN, COARSE GRAVELS AVAILABLE @ SITE
IMPERVIOUS BLANKET, 6 FT. THICK NORMAL TO GROUND SURFACE. EXTEND TO ELEV. 1089 AT ABUTMENTS & TO THE CORE OF DAM
10' BOTTOM
1060
1070
1080
1100
SPRING
50'
2A CHANNEL TO INLET TO REMAIN OPEN FOR DRAINAGE. FILL AND COMPACT AT COMPLETION OF JOB.
ELEV. VARIES 0.5% GRADE FROM DAM
PLACE HIGHLY PLASTIC MATERIAL IN ABUTMENTS AND FLOODPLAIN FINES IN BOTTOM.
Concrete Monument Detail Sheet # 20A of 21
X-SECTION AS SHOWN - See Sheet 2A
"GRADED" DRAIN TO MAKE CLEAN CONTACT WITH BEDROCK OVER 100 FT OF SURFACE
TOP ELEV 1089.2
NOTE: IMPERVIOUS BLANKET AREA TO BE CLEARED AND GRUBBED, STRIPPED AND ROLLED WITH A SHEEPSFOOT PRIOR TO PLACING BLANKET.
25'
2:1
2:1
1050
CLEAN OUT DRAINAGE WAY & FILL WITH COMPACTED MATERIAL PRIOR TO PLACING FILL.
STA 4+50
25'
ABUTMENT EXCAVATION COMMON ALL LOOSE ROCK AND BADLY WEATHERED BEDROCK SHALL BE EXCAVATED.
ELEV 1083
℄ PRIN. SPWY.
℄ FILL STA 5+00
EXTEND FILTER BLANKET TO ELEV. 1076.5 - 2 FT. THICK. MAKE CONTACT WITH BED ROCK.
℄ EMERG. SPWY.
FILTER BLANKET AREA - 4 FT. THICK
Quantities
Excavation - Unclassified - 408 c.y.
Surface Treatment - 850 Sq. Yds.
AUG 1969
AS BUILT
(Contract Modification #1)
STRUCTURE #1
1100
1090
1080
1070
1060
SCALE IN FEET
0 25 50 100 150
UPSTREAM BLANKET & DRAINAGE SYSTEM
West Fork Kickapoo W.P.P. & F.R.P. (PL 566)
Monroe and Vernon Co. Wisconsin
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed R. Bintzler Date 5-67
Drawn
Traced G. Lorenz 6-67
Checked C.E. Dennis 6/26/67
Approved by
Title
Title
Sheet No. 5A of 21
Drawing No 5,E-21,299 P
Form SCS 313 (November 1955)



Notes:

(1) All portions of the emergency spillway outlet channel's 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.

Quantities

Earth Fill	- Class B-2	211,230	~~122,323~~ Cu.yds
	- Class C-1	136,202	~~137,300~~ cu.yds.
	- Class SF	2,371	~~1350~~ cu.yds.
Excavation	- Common	37,057	~~19,347~~ cu.yds
	- Rock	284	~~1000~~ cu.yds.
	~~Unclassified~~	~~400~~	~~cu.yds~~
Dewatering Structure Site See Note			Sum Job

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 ℄ OF PRINCIPAL AND EMERGENCY SPILLWAYS - ℄ LAYOUTS

West Fork Kickapoo W.P. & F.P.P. (P.L. 566)

Monroe and Vernon Counties, Wisconsin

U.S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

Designed FALISKI, T.J. 3-64

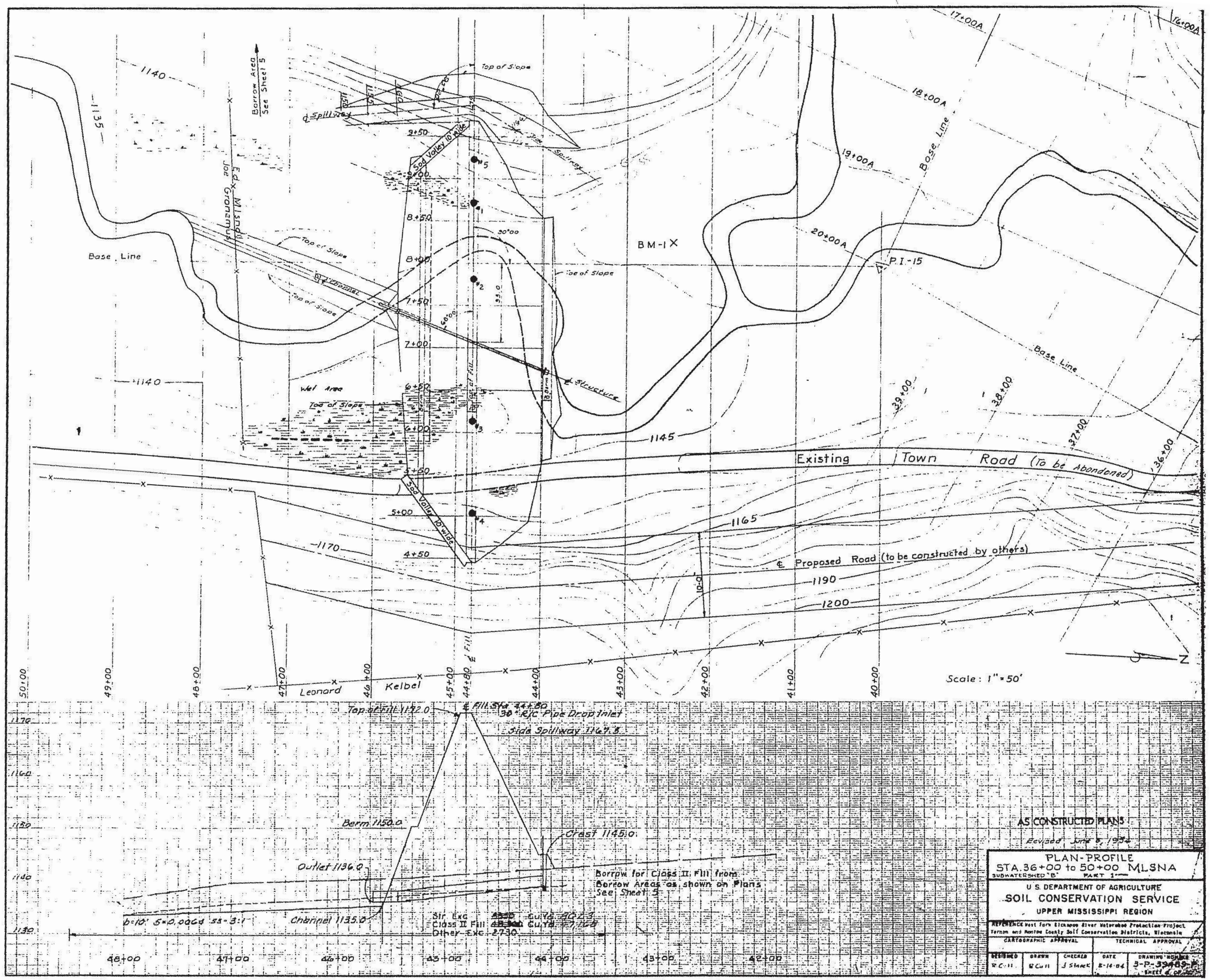
Drawn FALISKI, T.J. 3-64

Traced

Checked 4-64

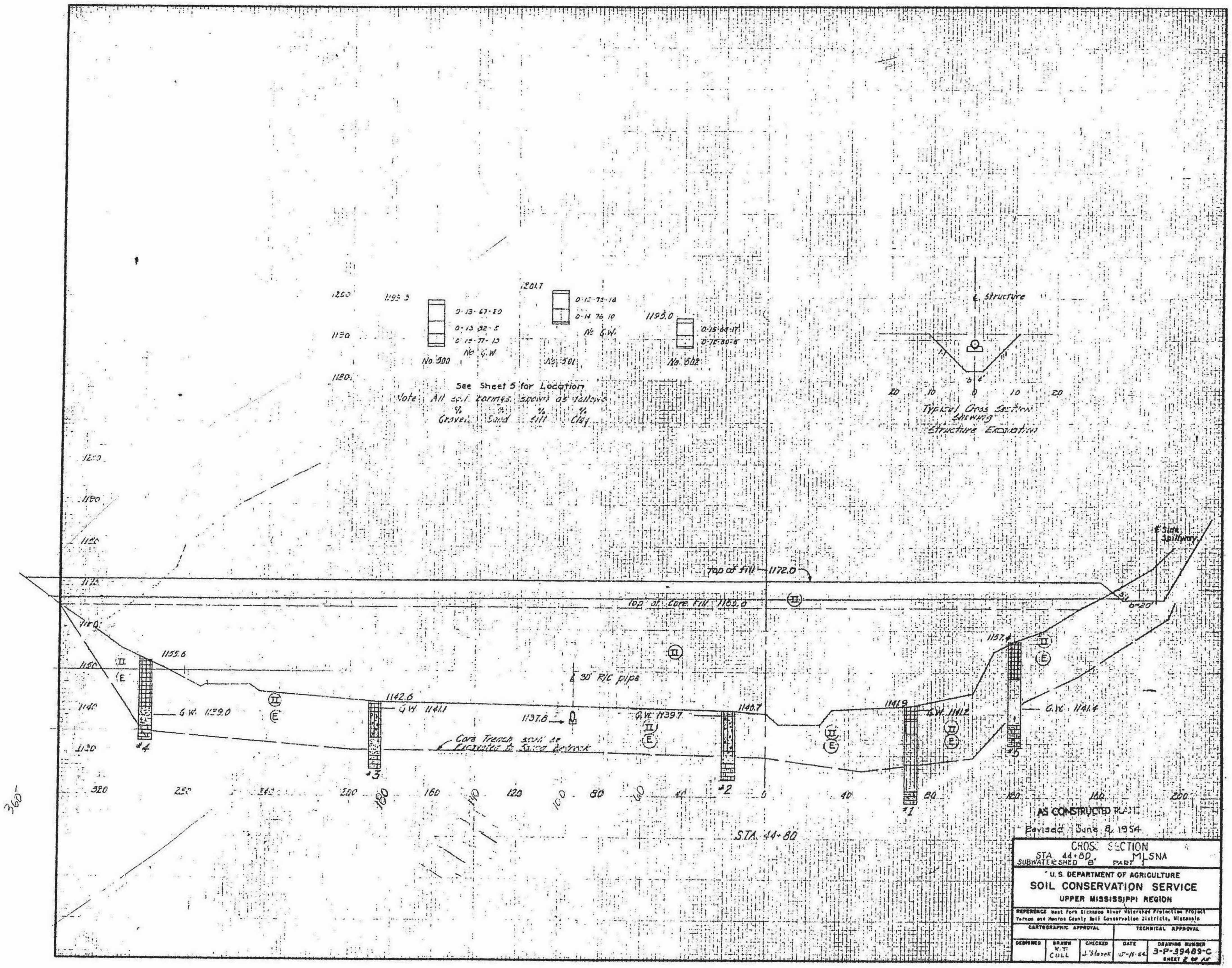
6/21

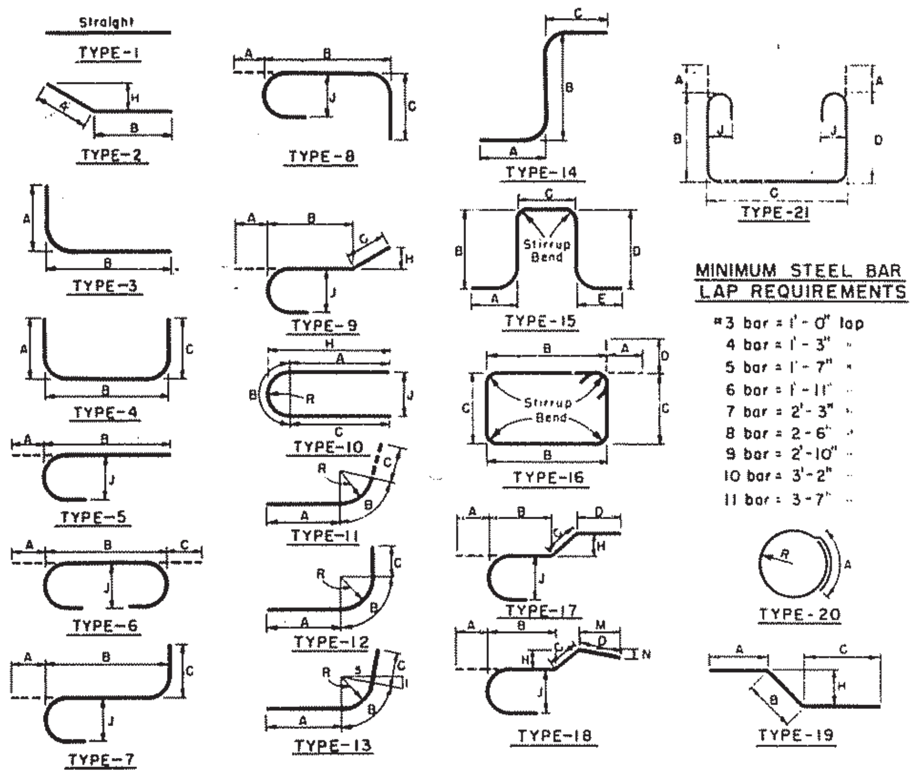
5,E-21,299-P



Top of Slope
Spillway
Sod Valley 10' wide
Base Line
BM-1 X
P.I.-15
Toe of Slope
Wet Area
℄ Structure
Existing Town Road (to be Abandoned)
℄ Proposed Road (to be constructed by others)
1140
1145
1165
1170
1190
1200
Leonard Kelbel
Joe Gronemus
Borrow Area See Sheet 5
Scale: 1" = 50'
Top of Fill 1172.0
℄ Fill Sta. 44+80
30" R/C Pipe Drop Inlet
Side Spillway 1167.3
Berm 1150.0
Crest 1145.0
Outlet 1136.0
Channel 1135.0
Borrow for Class II Fill from Borrow Areas as shown on Plans See Sheet 5
AS CONSTRUCTED PLANS
PLAN-PROFILE
STA. 36+00 to 50+00 MLSNA
SUBWATERSHED "B" PART 1
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
UPPER MISSISSIPPI REGION
REFERENCE West Fork Kickapoo River Watershed Protection Project
Vernon and Monroe County Soil Conservation Districts, Wisconsin
CARTOGRAPHIC APPROVAL
TECHNICAL APPROVAL
DESIGNED
DRAWN
CHECKED
J Staack
DATE
DRAWING NUMBER

Sheet 4 of 16


See Sheet 5 for Location
No. 500
No. 501
No. 502
No G.W.
Top of Fill 1172.0
Top of Core Fill 1162.0
30" R/C pipe
Core Trench
Typical Cross Section Showing Structure Excavation
℄ Structure
E Side Spillway
STA. 44+80
AS CONSTRUCTED PLANS
Revised June 8, 1954
CROSS SECTION
STA. 44+80
SUBWATERSHED "B" PART 1
MLSNA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
UPPER MISSISSIPPI REGION
CARTOGRAPHIC APPROVAL
TECHNICAL APPROVAL
DESIGNED
DRAWN
CHECKED
DATE
DRAWING NUMBER
3-P-39489-C



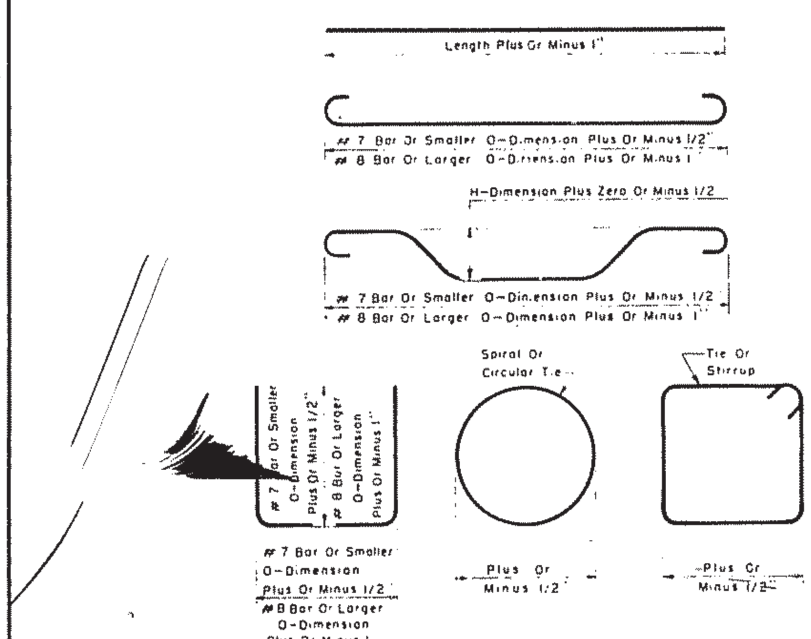
MINIMUM STEEL BAR LAP REQUIREMENTS

#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

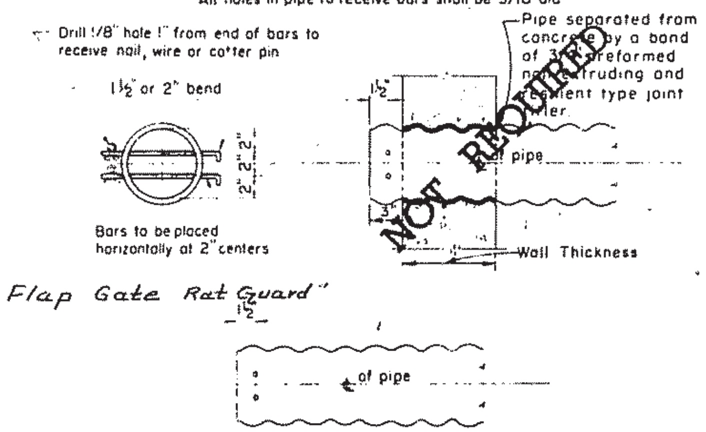
TYPICAL PERMISSIBLE TOLERANCES

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"

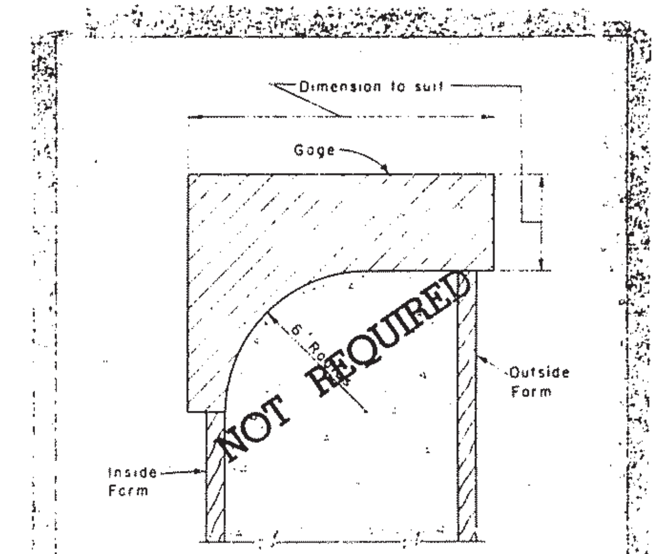
All holes in pipe to receive bars shall be 9/16" dia



"Used Flap Gate Rat Guard"

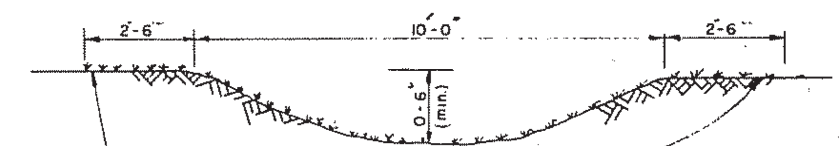
DETAILS OF ANIMAL GUARD

Scale 1½" = 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

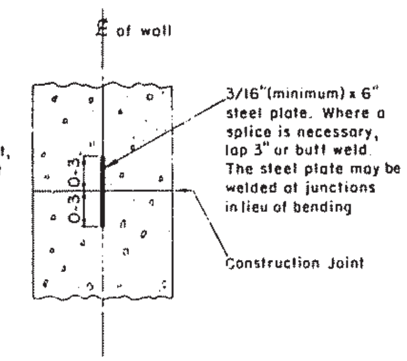


Note:
These extensions spaced as described in 520.5 Technical Specification 520

DETAIL OF SOD GUTTER

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 1½"=1'-0" Rev 9-2-59

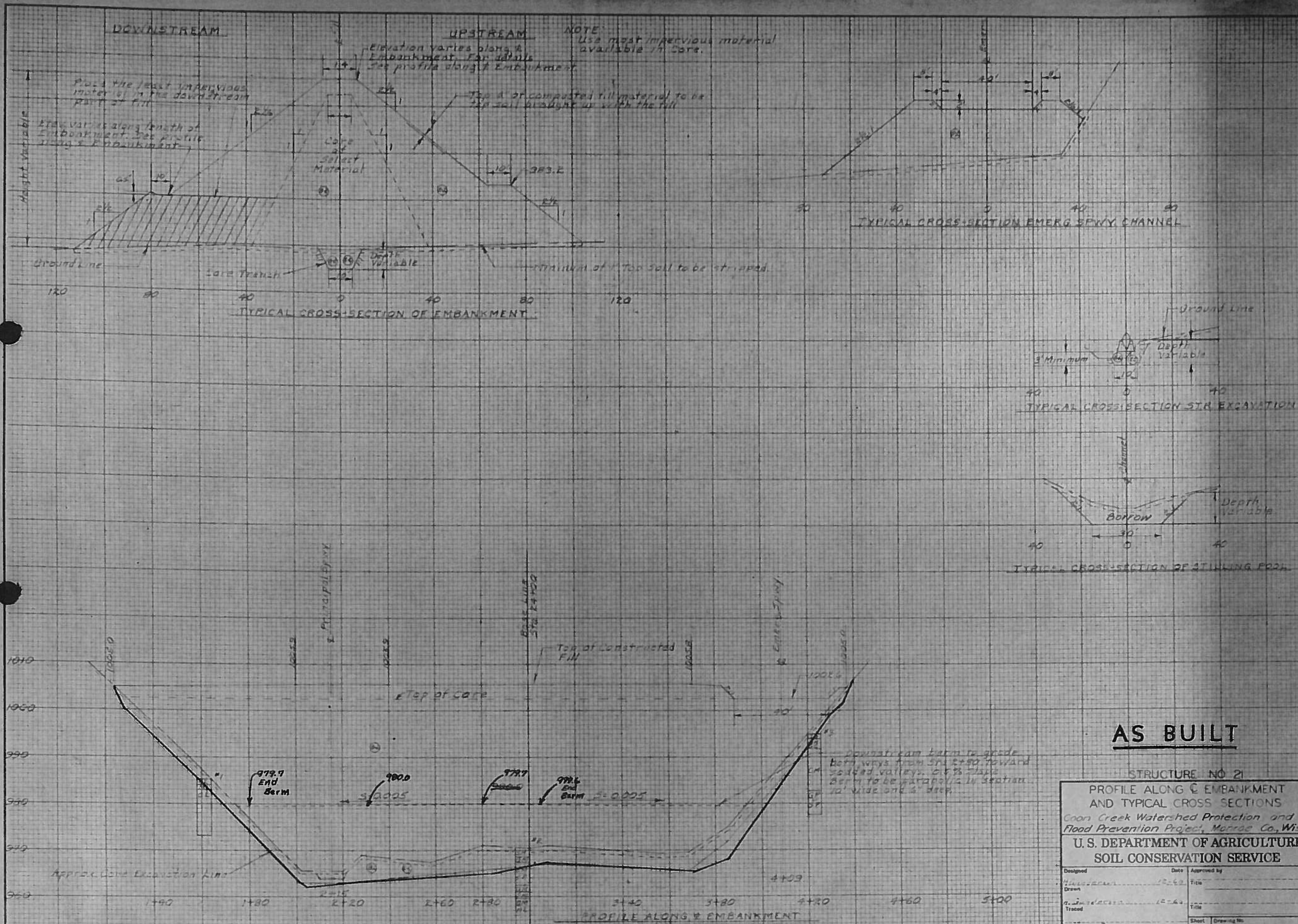
STRUCTURES 3 AND 12

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		Title	
Checked		Sheet No. 18 of 18	Drawing No. 3-E-46505-P



AS BUILT

STRUCTURE NO 21

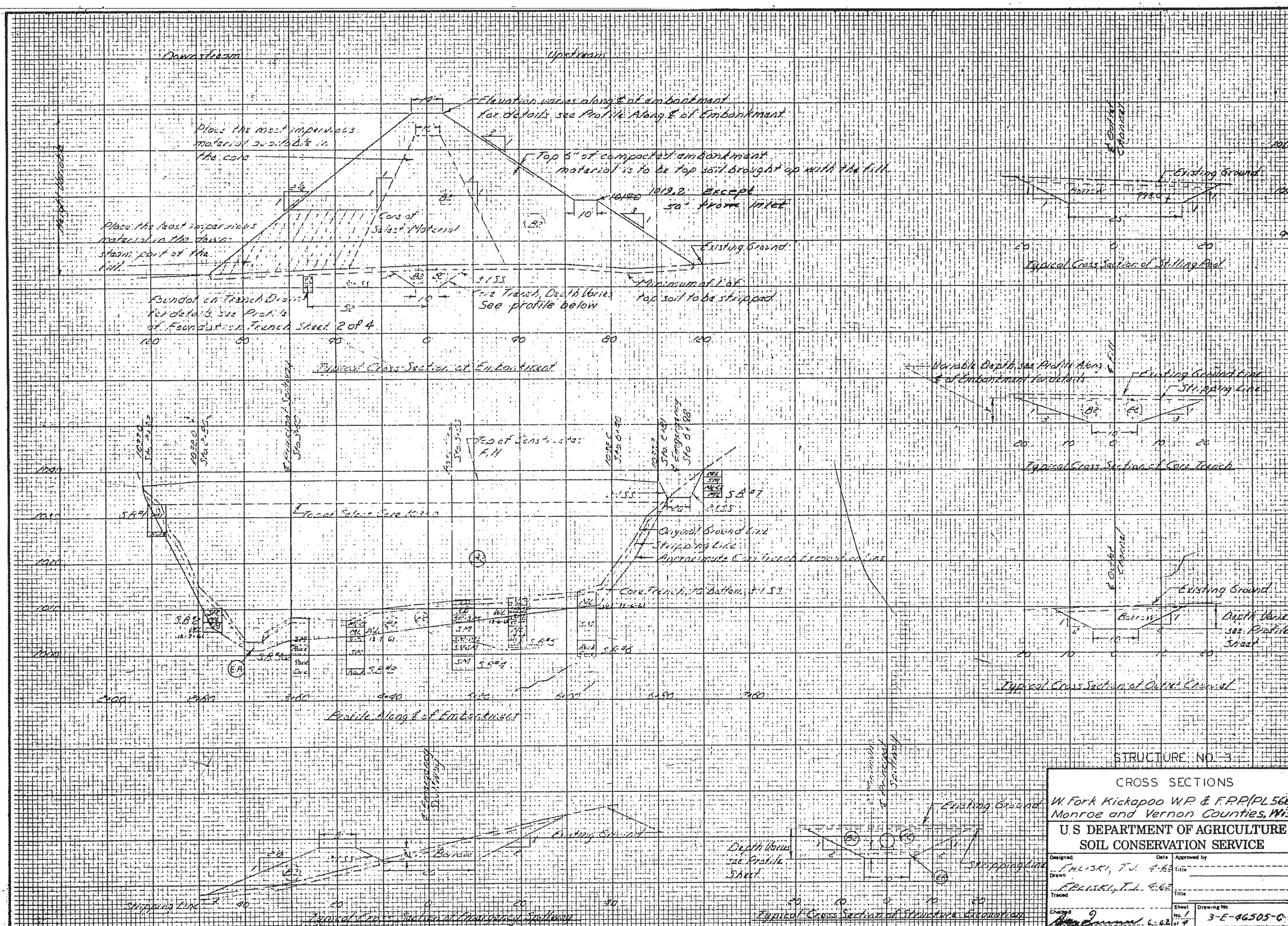
PROFILE ALONG ¢ EMBANKMENT AND TYPICAL CROSS SECTIONS

Coon Creek Watershed Protection and Flood Prevention Project, Monroe Co., Wis.

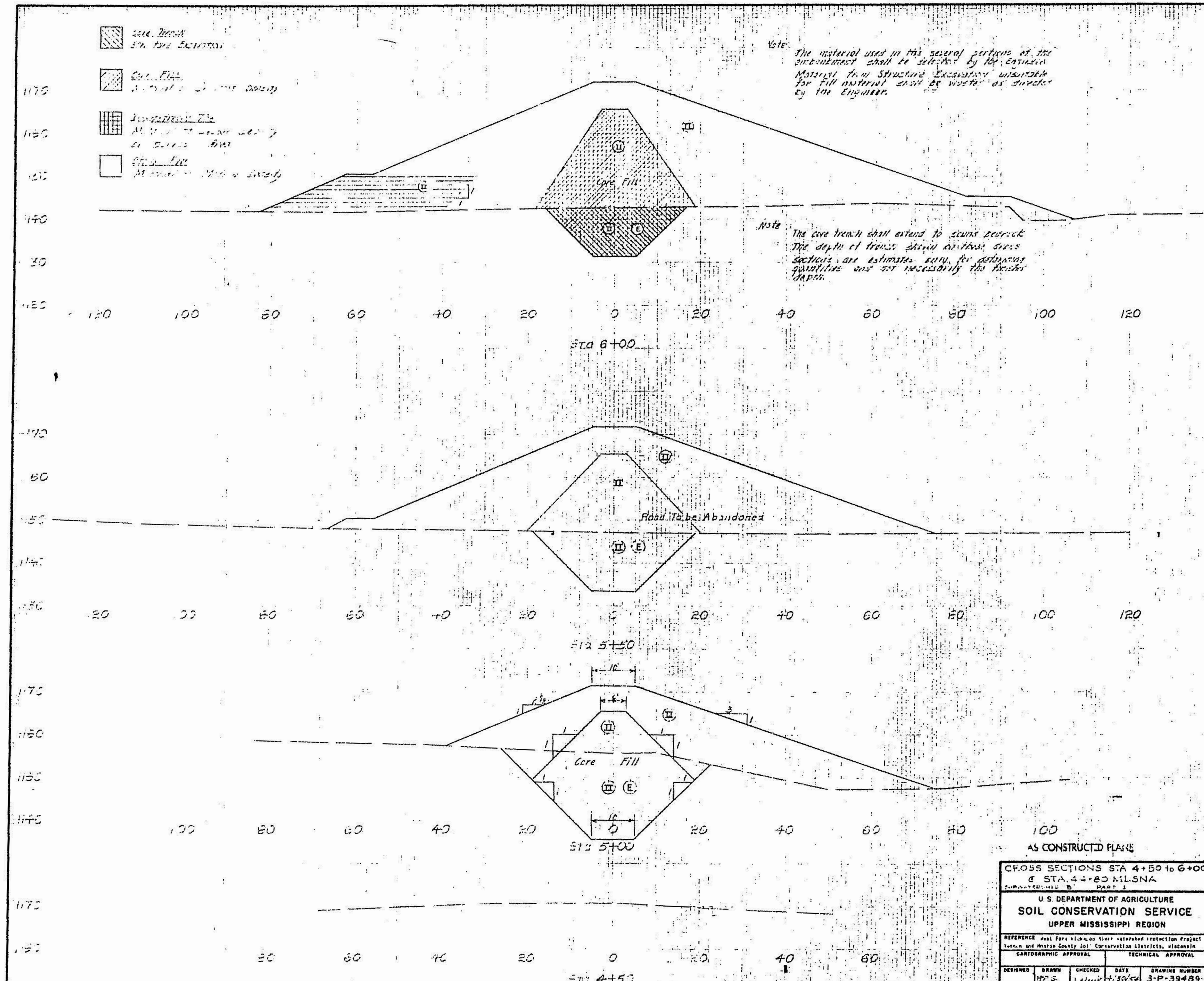
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

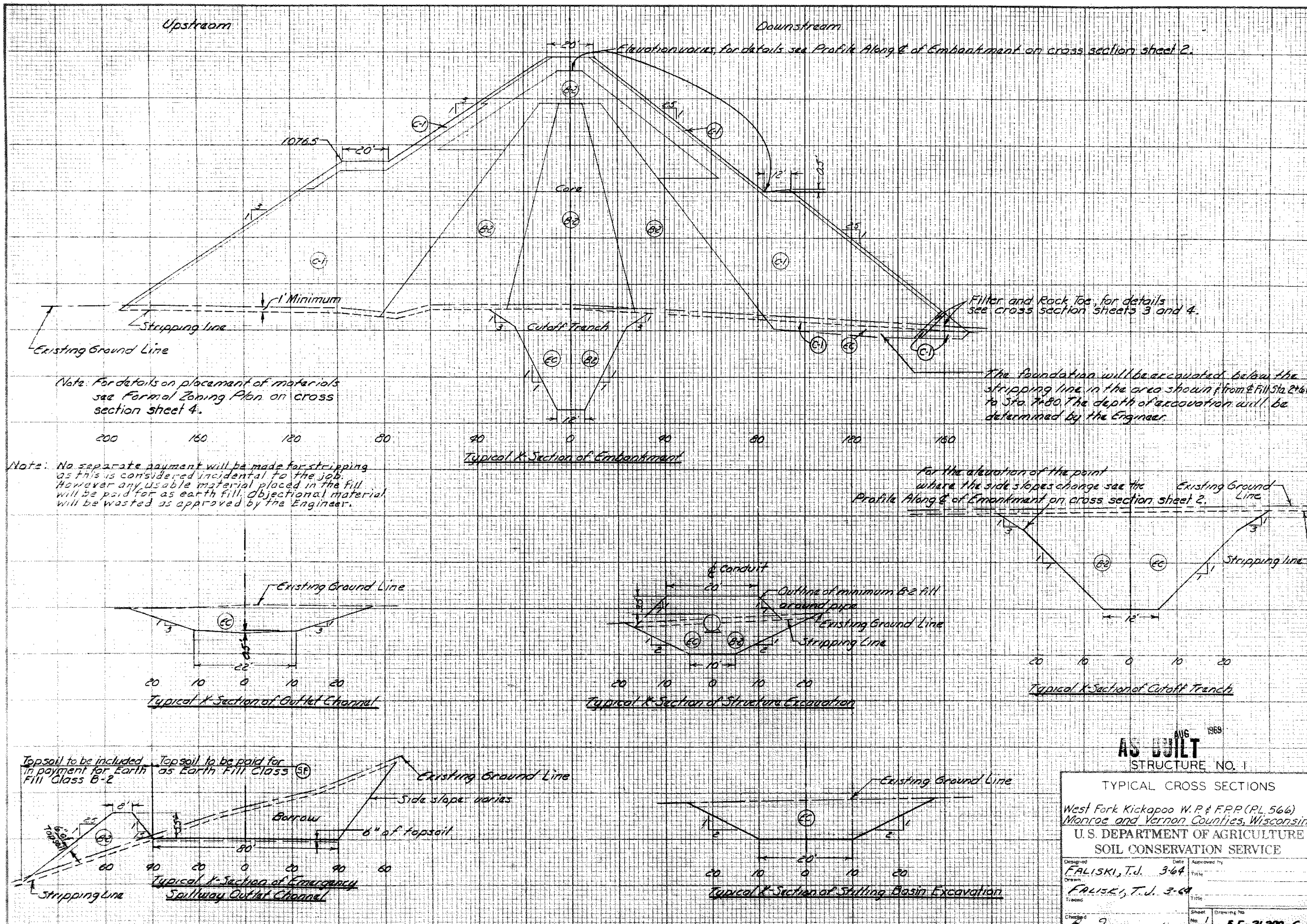
Checked 1-61
Sheet No. 1 of 3
Drawing No. 3-E-46088-C

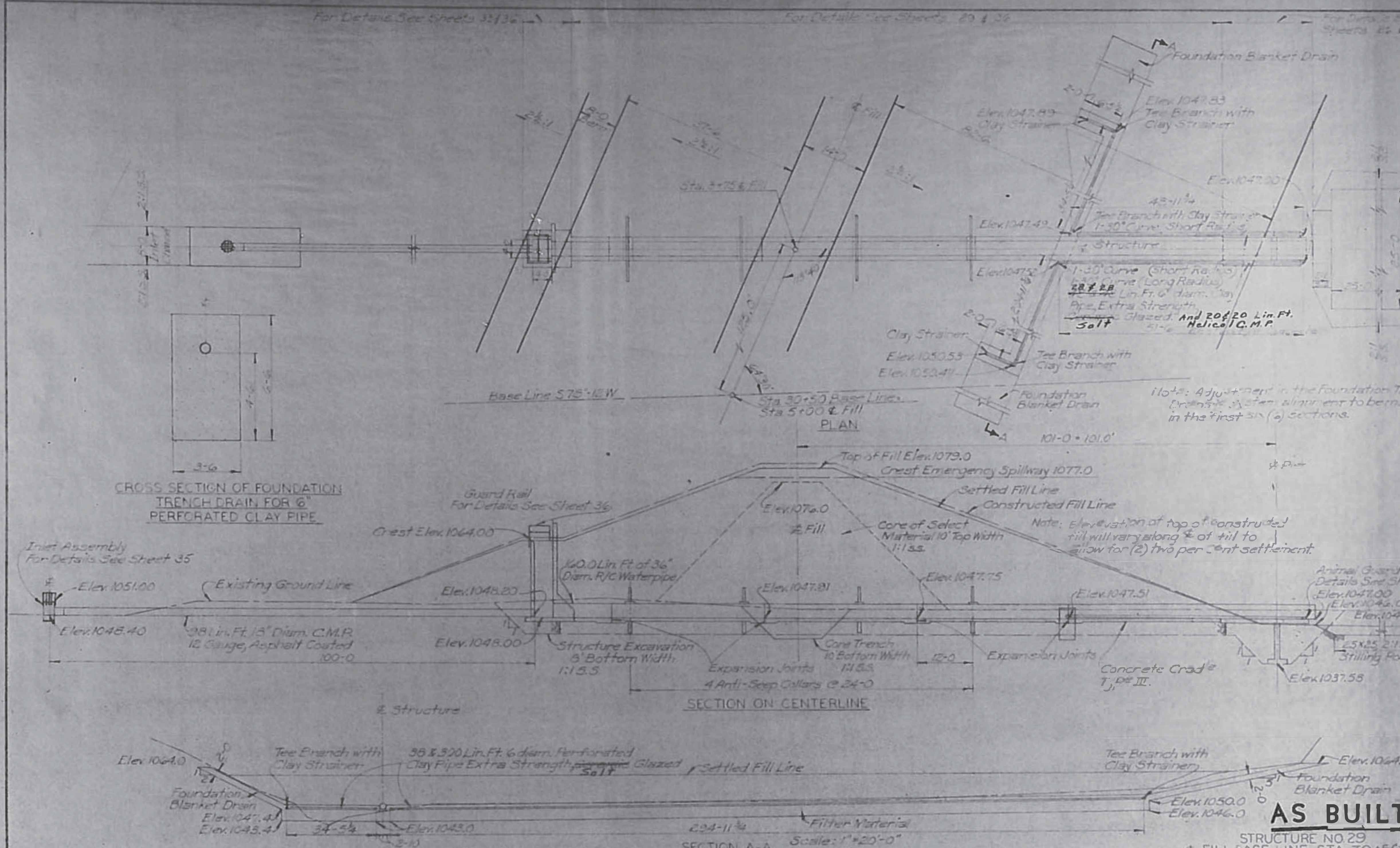
Form SCS-315 November 1955



Downstream
Upstream
Elevation varies along ℄ of embankment for details see Profile Along ℄ of Embankment
Place the most impervious material available in the core
Top 6" of compacted embankment material is to be top soil brought up with the fill
1019.2 Except 50' from inlet
Core of Select Material
Place the least impervious material in the downstream part of the fill
Existing Ground
Core Trench, Depth Varies See profile below
Minimum of 1' of top soil to be stripped
Foundation Trench Drain for details see Profile of Foundation Trench Sheet 2 of 4
Typical Cross Section of Embankment
Typical Cross Section of Stilling Pool
Typical Cross Section of Core Trench
Original Ground Line
Stripping Line
Core Trench, 15' Bottom, 3:1 S.S.
Profile Along ℄ of Embankment
Typical Cross Section of Outlet Channel
Depth Varies see Profile Sheet
Typical Cross Section of Emergency Spillway
Typical Cross Section of Structure Excavation
STRUCTURE NO. 3
CROSS SECTIONS
W. Fork Kickapoo W.P. & F.P.P.(PL566)
Monroe and Vernon Counties, Wis.
U.S DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Sheet No. 1 of 4
Drawing No. 3-E-46505-C


Core Trench
Core Fill
Note:
The material used in the several portions of the embankment shall be selected by the Engineer. Material from Structure Excavation unsuitable for fill material shall be wasted as directed by the Engineer.
Note
The core trench shall extend to sound bedrock. The depth of trench shown on these cross sections are estimates only for estimating quantities and not necessarily the finished depth.
Sta 6+00
Road To be Abandoned
Sta 5+50
Core Fill
Sta 5+00
Sta 4+50
AS CONSTRUCTED PLANS
CROSS SECTIONS STA 4+50 to 6+00
℄ STA. 4+80 MLSNA
PART 1
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
UPPER MISSISSIPPI REGION
REFERENCE
CARTOGRAPHIC APPROVAL
TECHNICAL APPROVAL
DESIGNED
DRAWN
CHECKED
DATE
4/30/54
DRAWING NUMBER
3-P-39489-C
SHEET 5 OF 15


Upstream
Downstream
Elevation varies, for details see Profile Along ℄ of Embankment on cross section sheet 2.
1076.5
20'
Core
Filter and Rock Toe, for details see cross section sheets 3 and 4.
1' Minimum
Stripping line
Existing Ground Line
Cutoff Trench
Note: For details on placement of materials see Formal Zoning Plan on cross section sheet 4.
The foundation will be excavated below the stripping line in the area shown from ℄ Fill Sta. 2+60 to Sta. 7+80. The depth of excavation will be determined by the Engineer.
200 160 120 80 40 0 40 80 120 160
Typical X-Section of Embankment
Note: No separate payment will be made for stripping as this is considered incidental to the job. However any usable material placed in the fill will be paid for as earth fill. Objectional material will be wasted as approved by the Engineer.
For the elevation of the point where the side slopes change see the Profile Along ℄ of Embankment on cross section sheet 2.
Existing Ground Line
Stripping line
20 10 0 10 20
Typical X-Section of Cutoff Trench
Existing Ground Line
22'
20 10 0 10 20
Typical X-Section of Outlet Channel
℄ Conduit
Outline of minimum B-2 fill around pipe
Existing Ground Line
Stripping Line
20 10 0 10 20
Typical X-Section of Structure Excavation
Topsoil to be included in payment for Earth Fill Class B-2
Topsoil to be paid for as Earth Fill Class
Existing Ground Line
Side slope varies
Borrow
6" of topsoil
80'
Stripping Line
Typical X-Section of Emergency Spillway Outlet Channel
Existing Ground Line
20'
20 10 0 10 20
Typical X-Section of Stilling Basin Excavation
AS BUILT AUG 1969
STRUCTURE NO. 1
TYPICAL CROSS SECTIONS
West Fork Kickapoo W.P. & F.P.P. (P.L. 566)
Monroe and Vernon Counties, Wisconsin
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed FALISKI, T.J. Date 3-64
Drawn FALISKI, T.J. 3-64
Traced
Checked 4-64
Approved by
Title
Title
Sheet No. 1 of 4
Drawing No. 5,E-21,299-C



QUANTITIES

Concrete - Class B, Air Entrained	57.3 ~~58.66~~ Cu. Yds.
Steel Bar Reinforcement	2262 ~~2400.0~~ Pounds
Pipe Conduit - Reinforced Concrete Water Pipe, 36" Diameter	161.0 ~~160.6~~ Lin. Ft.
Structure Drainage - 24" to 18" Diam. Corrugated Metal Pipe Vertical Inlet Assembly, 12 Gage, Asphalt Coated	4.1 Lin. Ft.
" " - 18" Diameter Corrugated Metal Pipe, 12 Gage, Asphalt Coated	98.9 Lin. Ft.
" " - 6" Diameter, Perforated Clay Pipe, Extra Strength, ~~Ceramic~~ Salt Glazed	342.3 Lin. Ft.
" " - 6" Diameter Clay Pipe, Extra Strength, ~~Ceramic~~ Salt Glazed	60.5 ~~160.5~~ Lin. Ft.
" " - Filter Material - Pit Run Sand and Gravel	358.0 ~~374~~ Cu. Yds.
Trash Racks and Guard Rails	Part Job
Structure Drainage - Helical C.M.P. - 6" Dia.	40.0 Lin. Ft.

Scale: ⅛" = 1'-0" Unless Otherwise Shown

AS BUILT

STRUCTURE NO. 29
℄ FILL BASE LINE STA. 30+50

R/C DROP INLET 36" DIA. R/C WATER PI[PE]
GENERAL LAYOUT
Coon Creek Watershed Protection & Flood Prevention Project (P.L. 566) Wis.

U.S. DEPARTMENT OF AGRICULTUR[E]
SOIL CONSERVATION SERVICE

Designed	Approved by
Drawn R. C. Heckman 5-14-59	Title
Traced J. J. Haeselich 5-20-59	Title
Checked D. Ralston 6-59	Sheet No. 19 of 36 — Drawing No. 3-E-45670

Form SCS-313

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 Mlsna
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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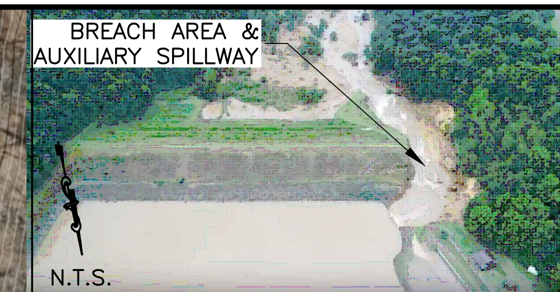


LEGEND

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

#1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)

B-5 TYPICAL ROCK CORE 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

GRAPHIC SCALE

80 0 40 80 160

(IN FEET)
1 inch = 80 ft.

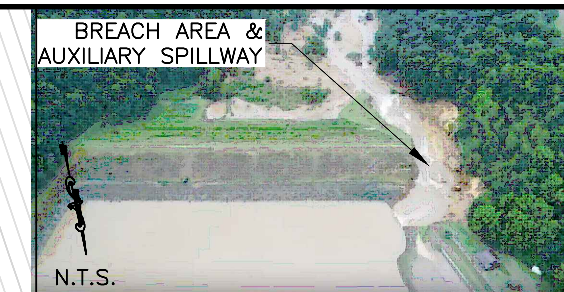
Designed	JEV
Drawn	
Checked	
Approved	
Date	5/2019

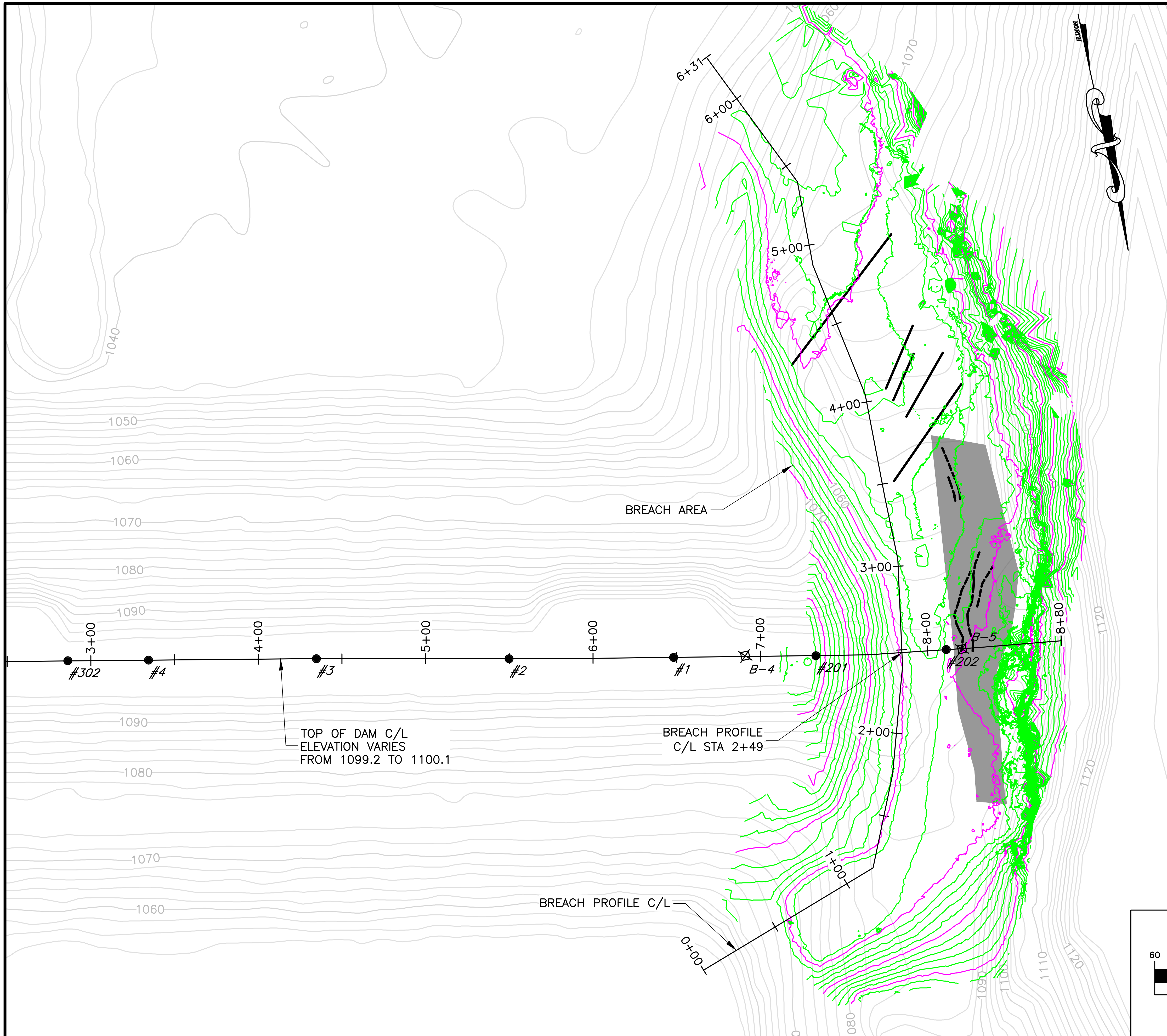
WEST FORK #1 - JERSEY VALLEY
PLAN VIEW - OVERALL

OWNER: VERNON COUNTY
COUNTY: VERNON

United States Department of Agriculture
USDA
Natural Resources Conservation Service

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





LEGEND

- LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
- POST-BREACH GROUND CONTOURS
- TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location) — #1
- TYPICAL ROCK CORE BORING LOCATION & NUMBER (Approximate Location) — #B-5
- 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

GRAPHIC SCALE

60 0 30 60 120

(IN FEET)
1 inch = 60 ft.

Date	5/2019
Designed	JEV
Drawn	
Checked	
Approved	

WEST FORK #1 - JERSEY VALLEY PLAN VIEW BREACH AREA

OWNER: VERNON COUNTY

COUNTY: VERNON

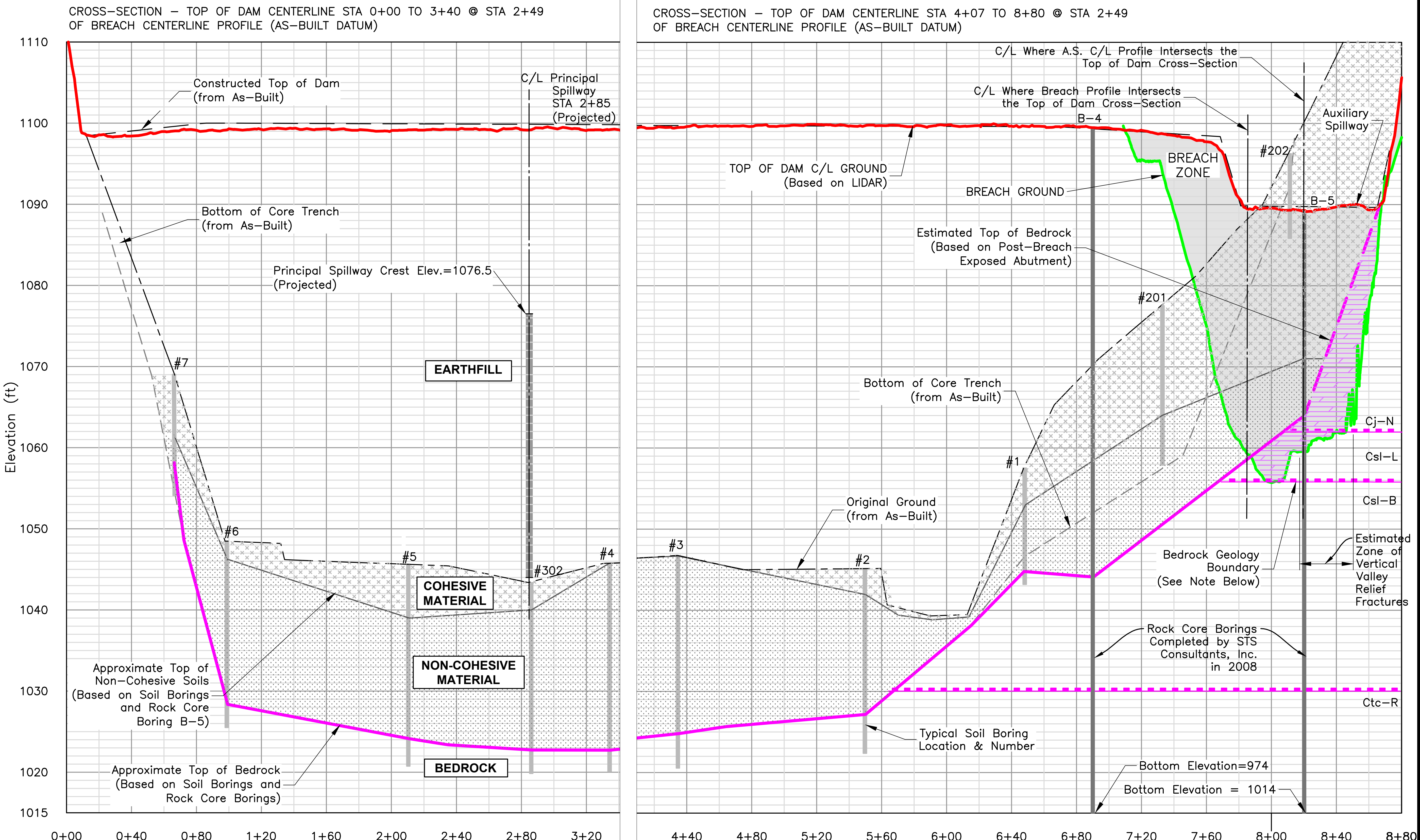
United States Department of Agriculture

USDA Natural Resources Conservation Service

File Name WI-006

Date 08/14

Sheet 2 of 24



5:1 VERTICAL EXAGGERATION

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 (Glauconitic Sandstone)

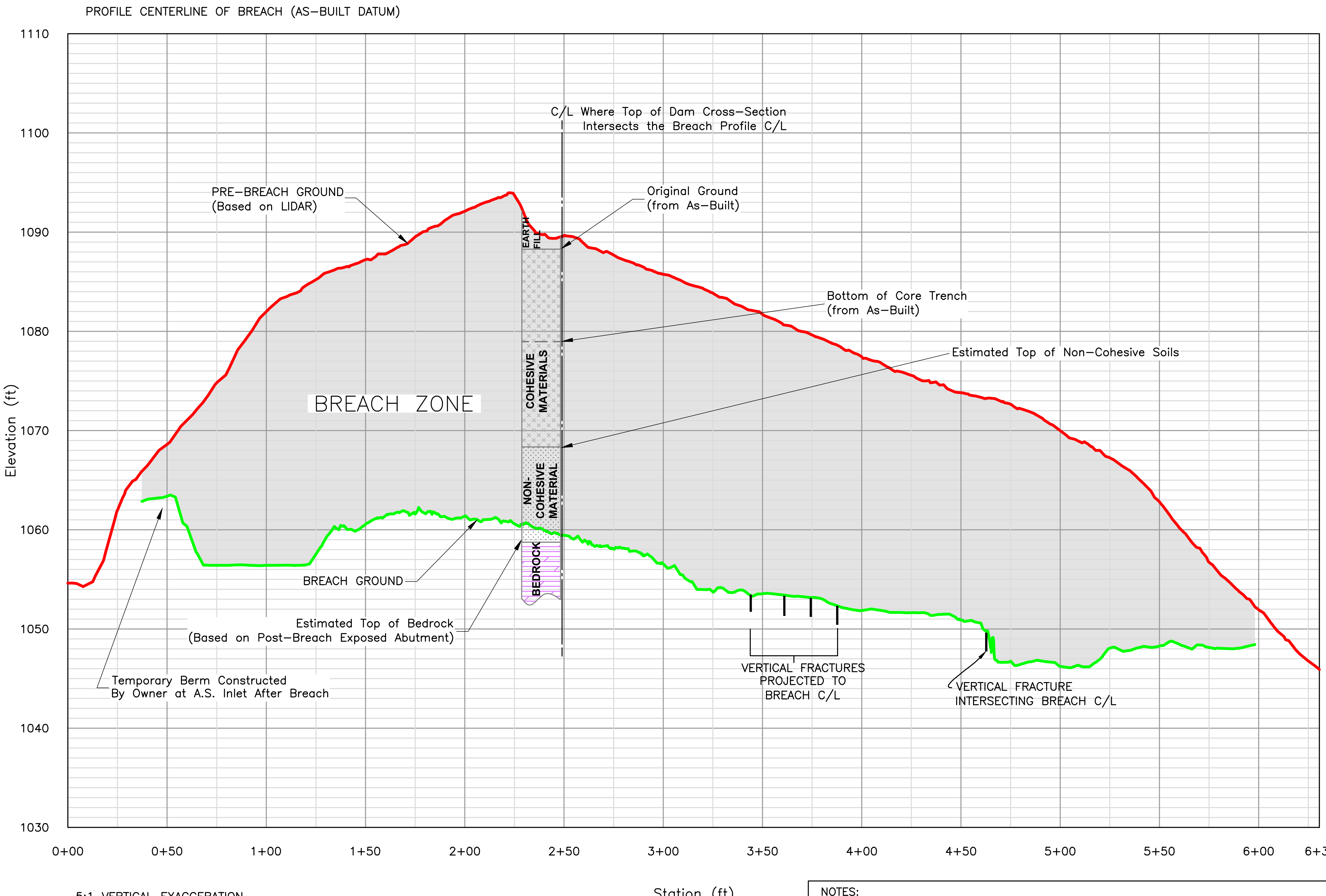
Date 5/2019
Designed JEV
Drawn
Checked
Approved

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

OWNER: VERNON COUNTY
COUNTY: VERNON

United States Department of Agriculture
Natural Resources Conservation Service

File Name WI-006
Date 08/14
Sheet 3 of 24



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

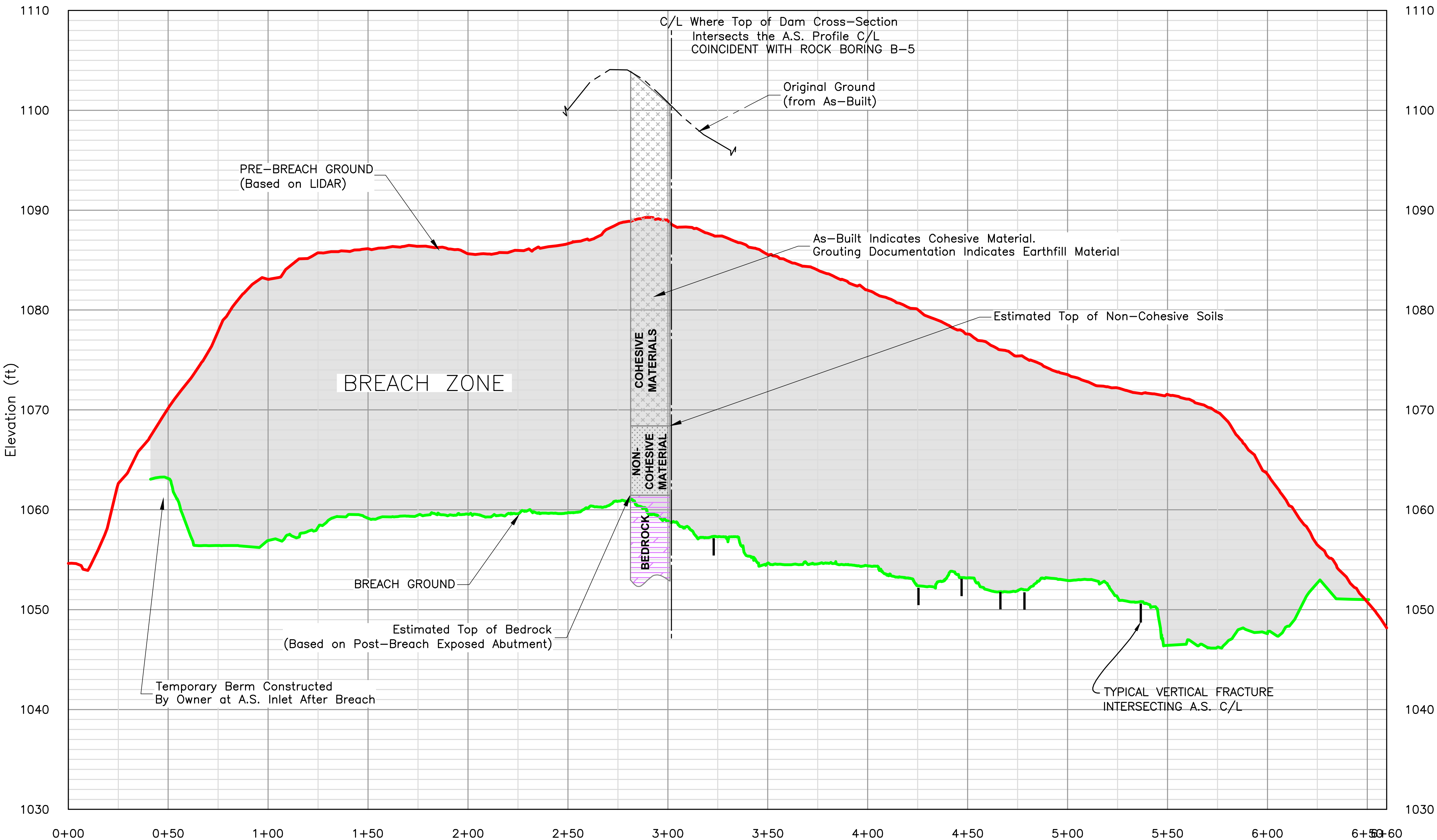
Designed JEV
Date 5/2019
Drawn
Checked
Approved

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

USDA
United States Department of Agriculture
Natural Resources Conservation Service

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

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS–BUILT DATUM)



5:1 VERTICAL EXAGGERATION

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.

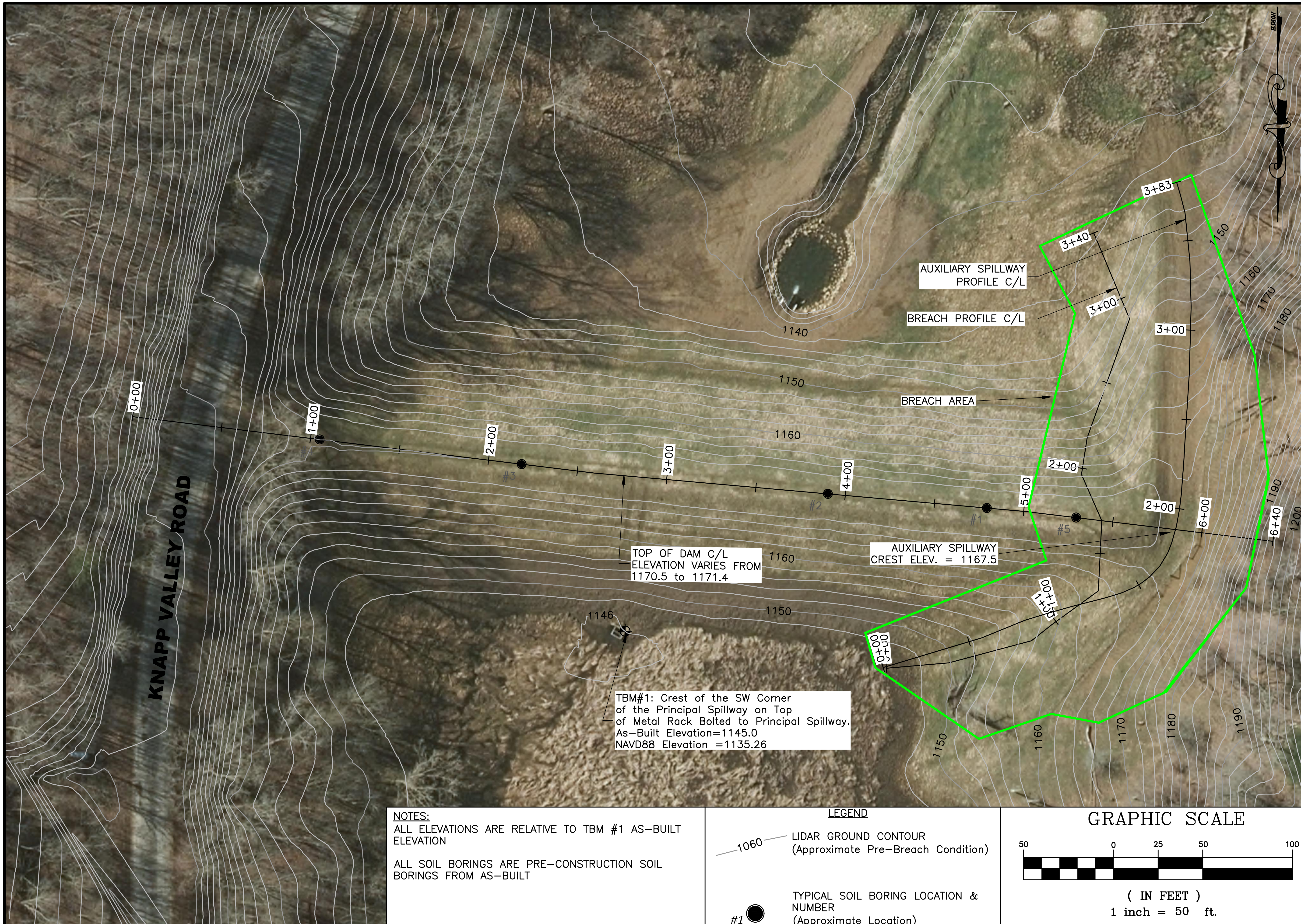
Vertical Extent of Vertical Fractures is Unknown

Date 5/2019
Designed JEV
Drawn
Checked
Approved

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

United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 5 of 24



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)

GRAPHIC SCALE

50 0 25 50 100

(IN FEET)

1 inch = 50 ft.

Date 4/2019

Designed JEV

Drawn

Checked

Approved

MLSNA PLAN VIEW – OVERALL

OWNER: VERNON COUNTY

COUNTY: VERNON

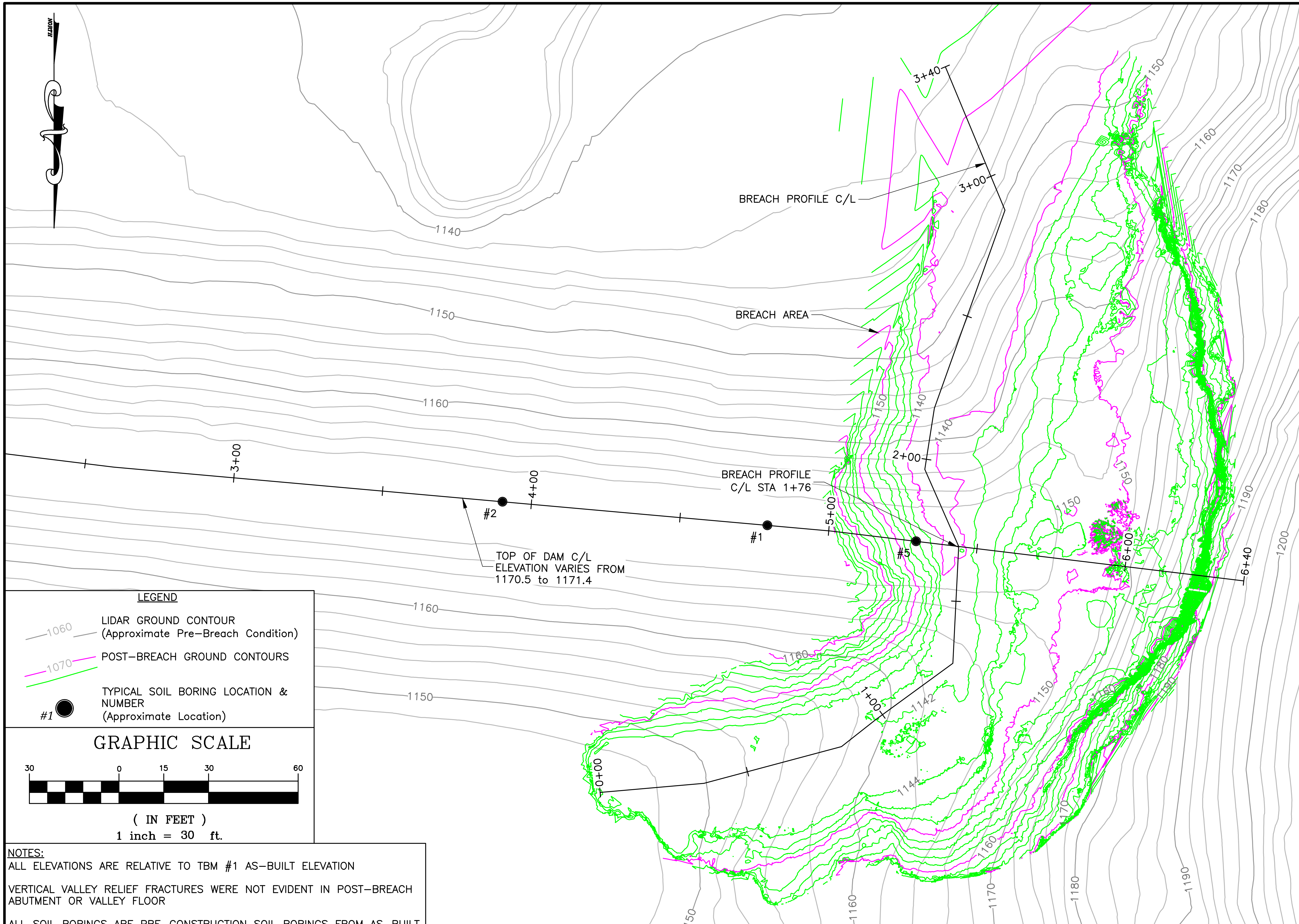
United States Department of Agriculture

USDA Natural Resources Conservation Service

File Name WI-006

Date 08/14

Sheet 6 of 24



LEGEND

1060 LIDAR GROUND CONTOUR (Approximate Pre–Breach Condition)

1070 POST–BREACH GROUND CONTOURS

#1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)

GRAPHIC SCALE

30 0 15 30 60

(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
Drawn	
Checked	
Approved	
Date	4/2019

MLSNA PLAN VIEW – BREACH AREA

OWNER: VERNON COUNTY

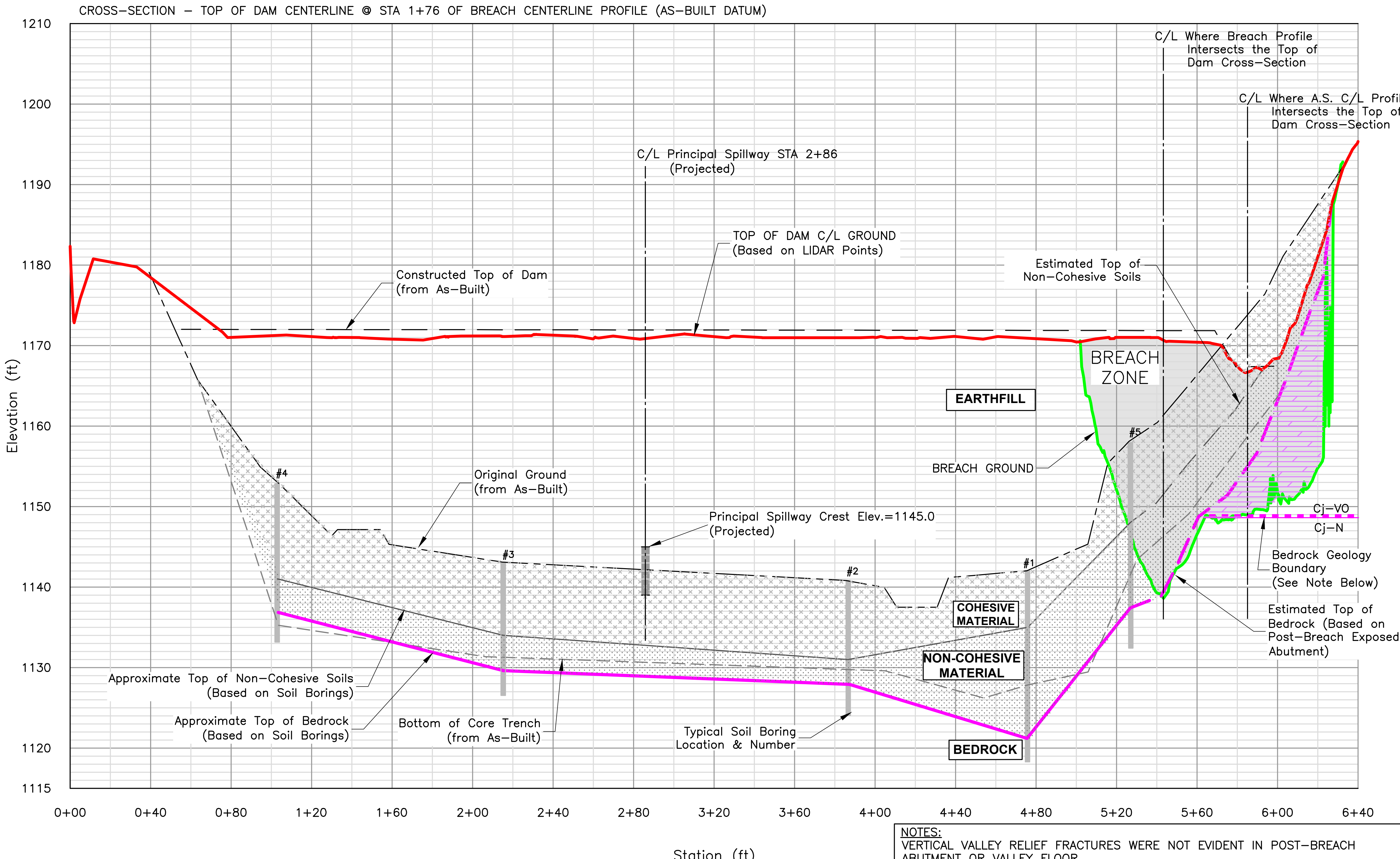
COUNTY: VERNON

United States Department of Agriculture
USDA
Natural Resources Conservation Service

File Name
WI–006

Date
08/14

Sheet 7 of 24



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

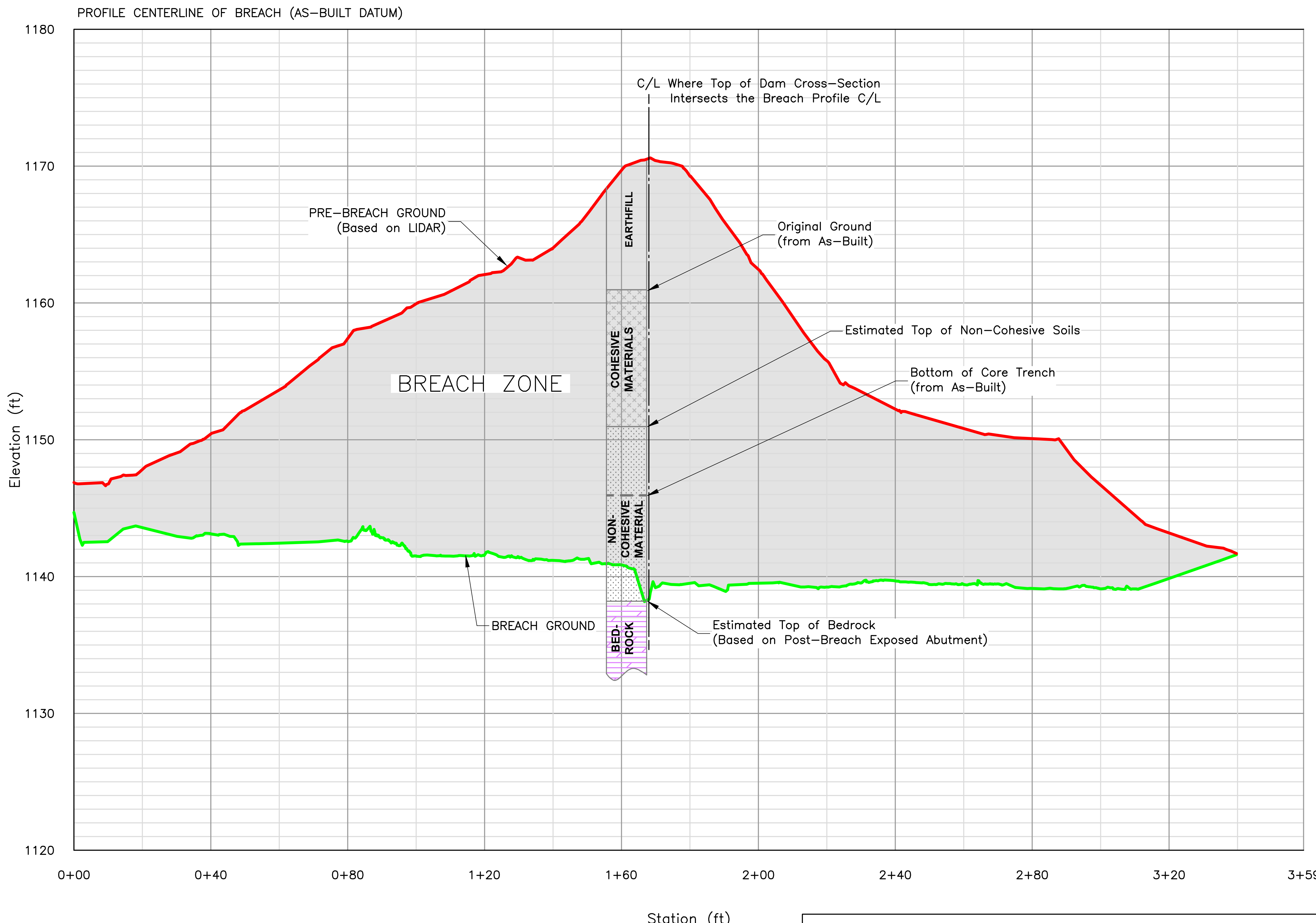
BEDROCK GEOLOGY (Based on Post Breach Exposed Abutment)
Cj–VO: Van Oser Member of the Jordan Sandstone (Soft, Poorly Cemented)
Cj–N: Norwalk Member of the Jordan Sandstone (Case Hardened)

Date 4/2019
Designed JEV
Drawn
Checked
Approved

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

USDA United States Department of Agriculture
Natural Resources Conservation Service

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



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.

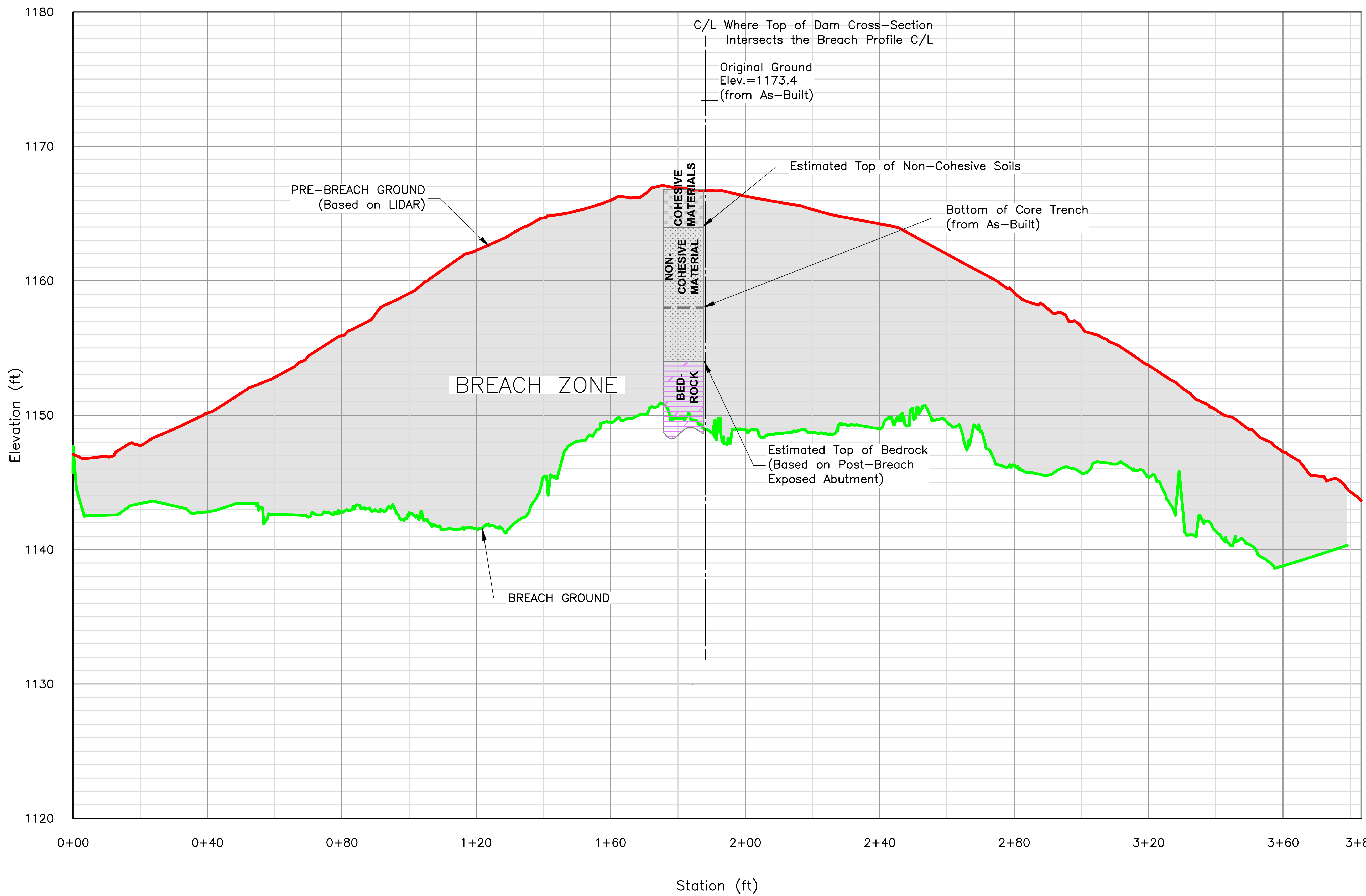
Designed JEV Date 4/2019
Drawn
Checked
Approved

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

United States Department of Agriculture
Natural Resources Conservation Service

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

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
PROFILE – C/L OF AUXILIARY SPILLWAY
OWNER: VERNON COUNTY
COUNTY: VERNON

United States Department of Agriculture
USDA
Natural Resources Conservation Service

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

N.T.S.

LEGEND

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

● #1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)

GRAPHIC SCALE

50 0 25 50 100

(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

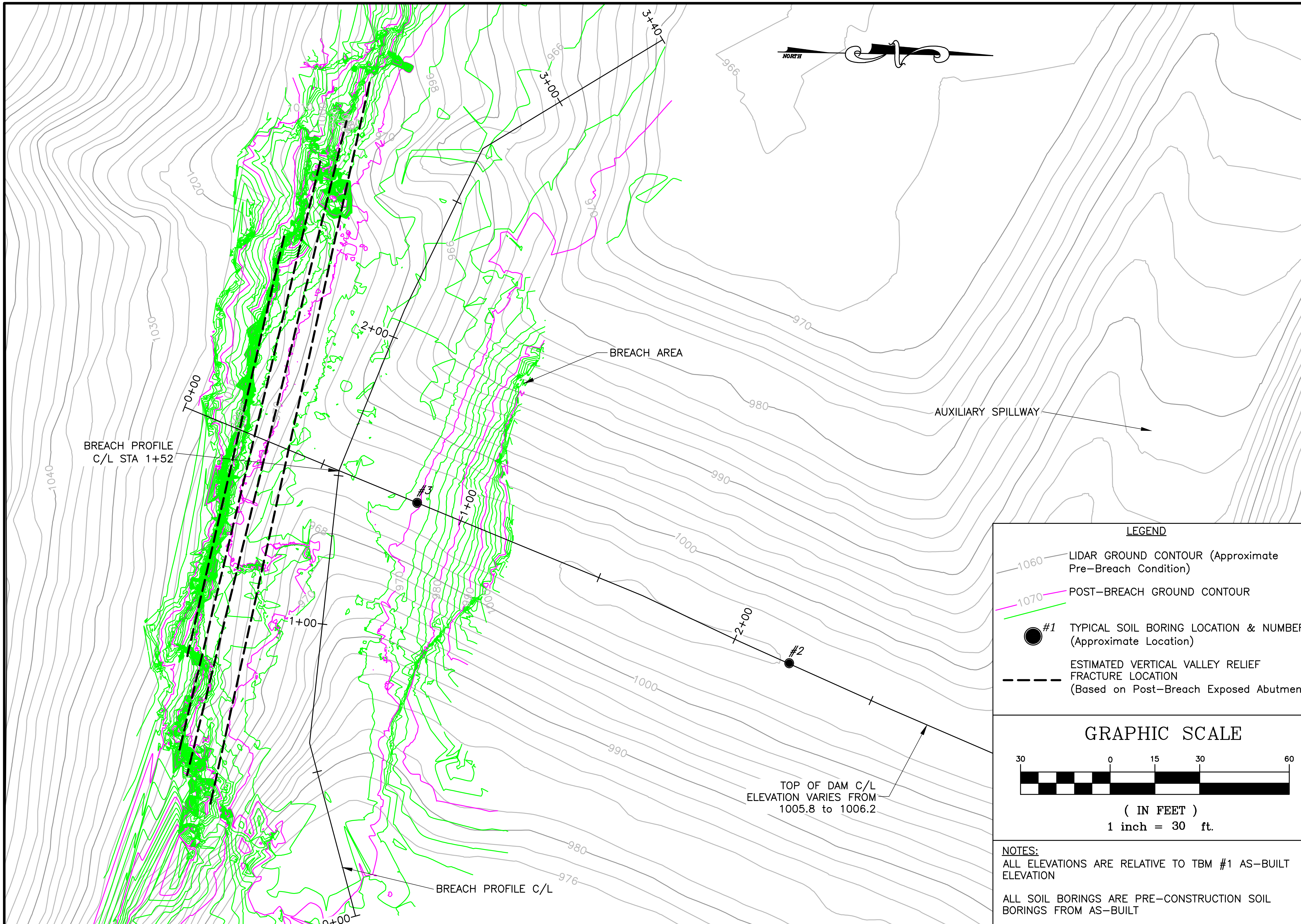
Date 5/2019
Designed JEV
Drawn
Checked
Approved

COON CREEK #21 PLAN VIEW - OVERALL

OWNER: MONROE COUNTY
COUNTY: MONROE

United States Department of Agriculture
USDA Natural Resources Conservation Service

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



LEGEND

1060	LIDAR GROUND CONTOUR (Approximate Pre-Breach Condition)
1070	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

30 0 15 30 60

(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

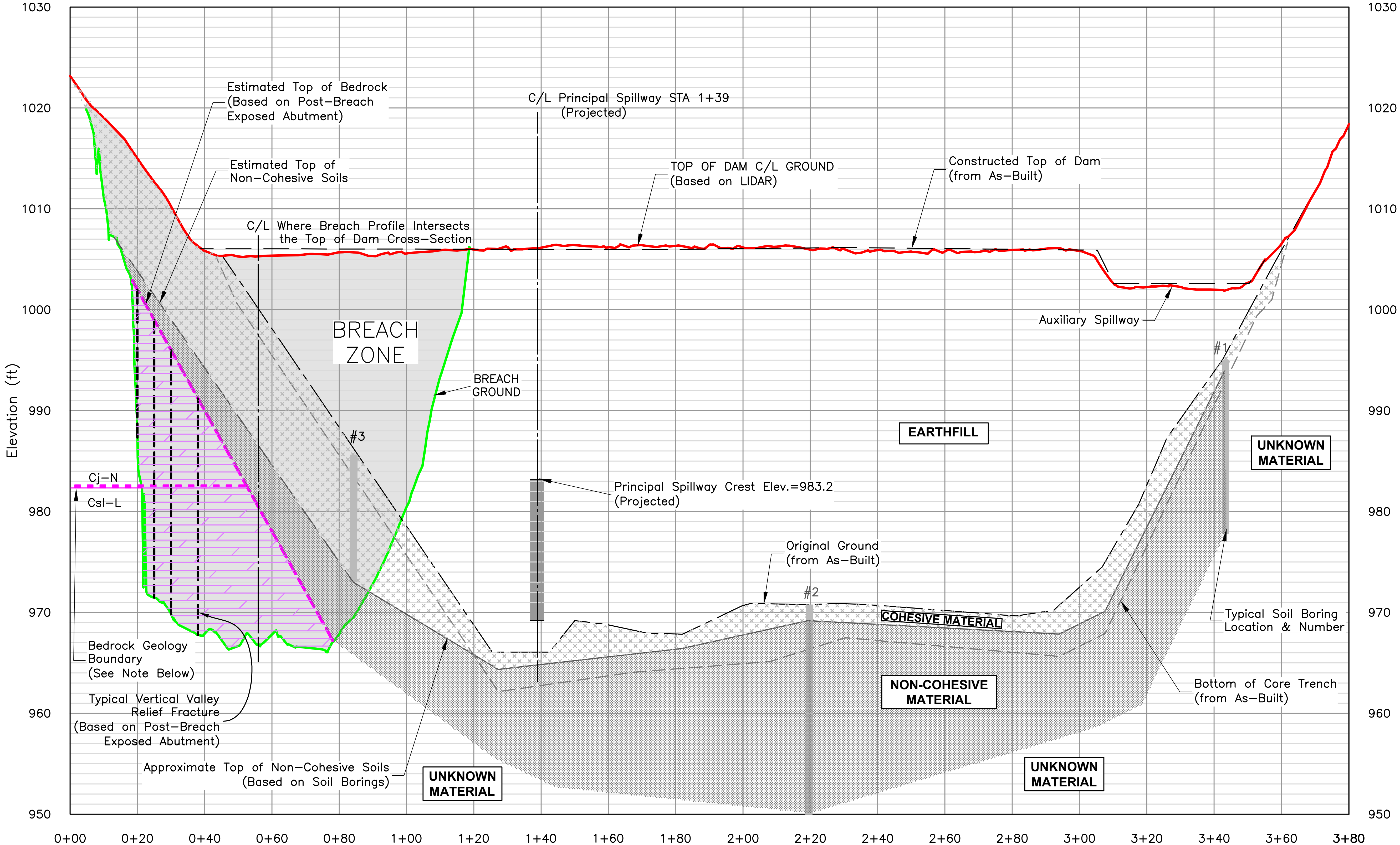
COON CREEK #21 PLAN VIEW BREACH AREA

OWNER: MONROE COUNTY
COUNTY: MONROE

United States Department of Agriculture
USDA
Natural Resources Conservation Service

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

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)

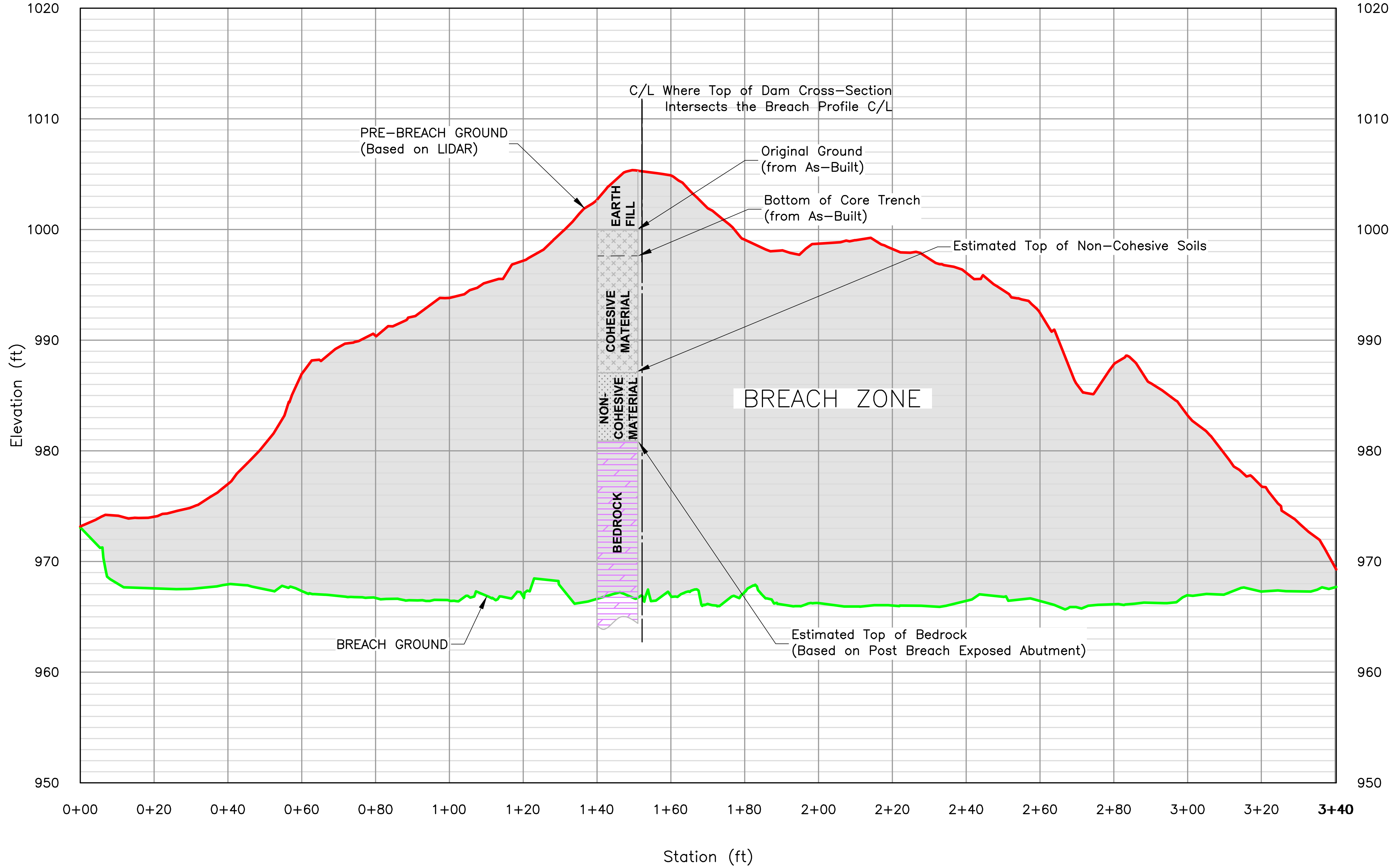
Date 5/2019
Designed JEV
Drawn
Checked
Approved

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

United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 13 of 24

PROFILE CENTERLINE OF BREACH (AS–BUILT DATUM)



3:1 VERTICAL EXAGGERATION

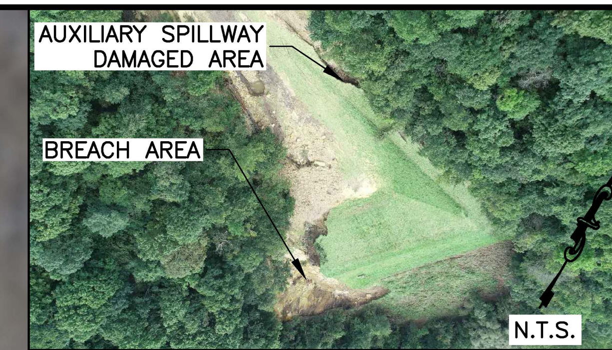
NOTES:
The estimated geologic boundaries shown are in the near vicinity of where the Breach Profile C/L intersects the Top of Dam C/L and cannot be extrapolated to the full extent of the profile.

Date 5/2019
Designed JEV
Drawn
Checked
Approved

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

United States Department of Agriculture
Natural Resources Conservation Service

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

GRAPHIC SCALE

50 0 25 50 100

(IN FEET)
1 inch = 50 ft.

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'.

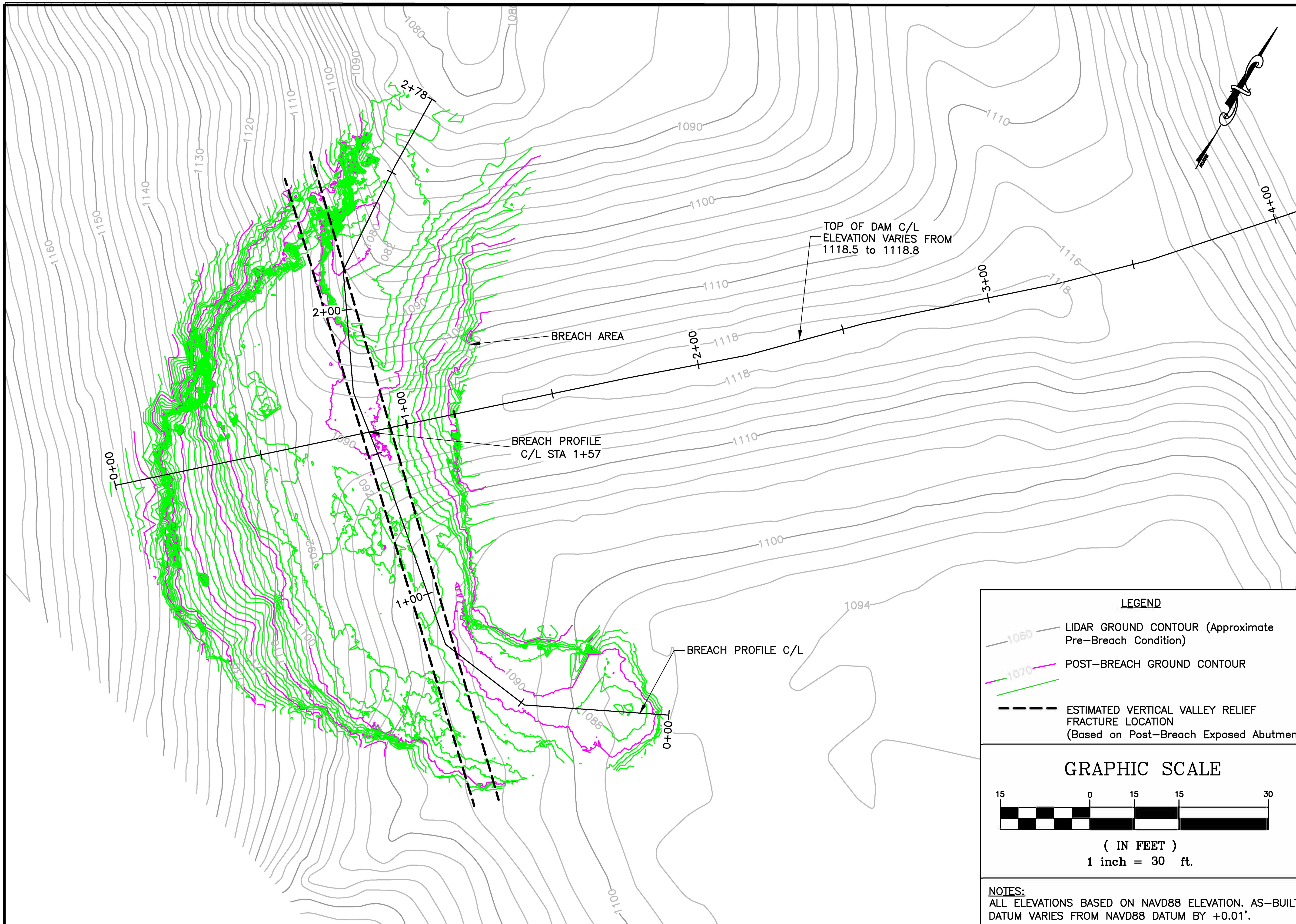
Date 5/2019
Designed JEV
Drawn
Checked
Approved

COON CREEK #23 PLAN VIEW - OVERALL

OWNER: MONROE COUNTY
COUNTY: MONROE

USDA
United States Department of Agriculture
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

USDA United States Department of Agriculture
Natural Resources Conservation Service

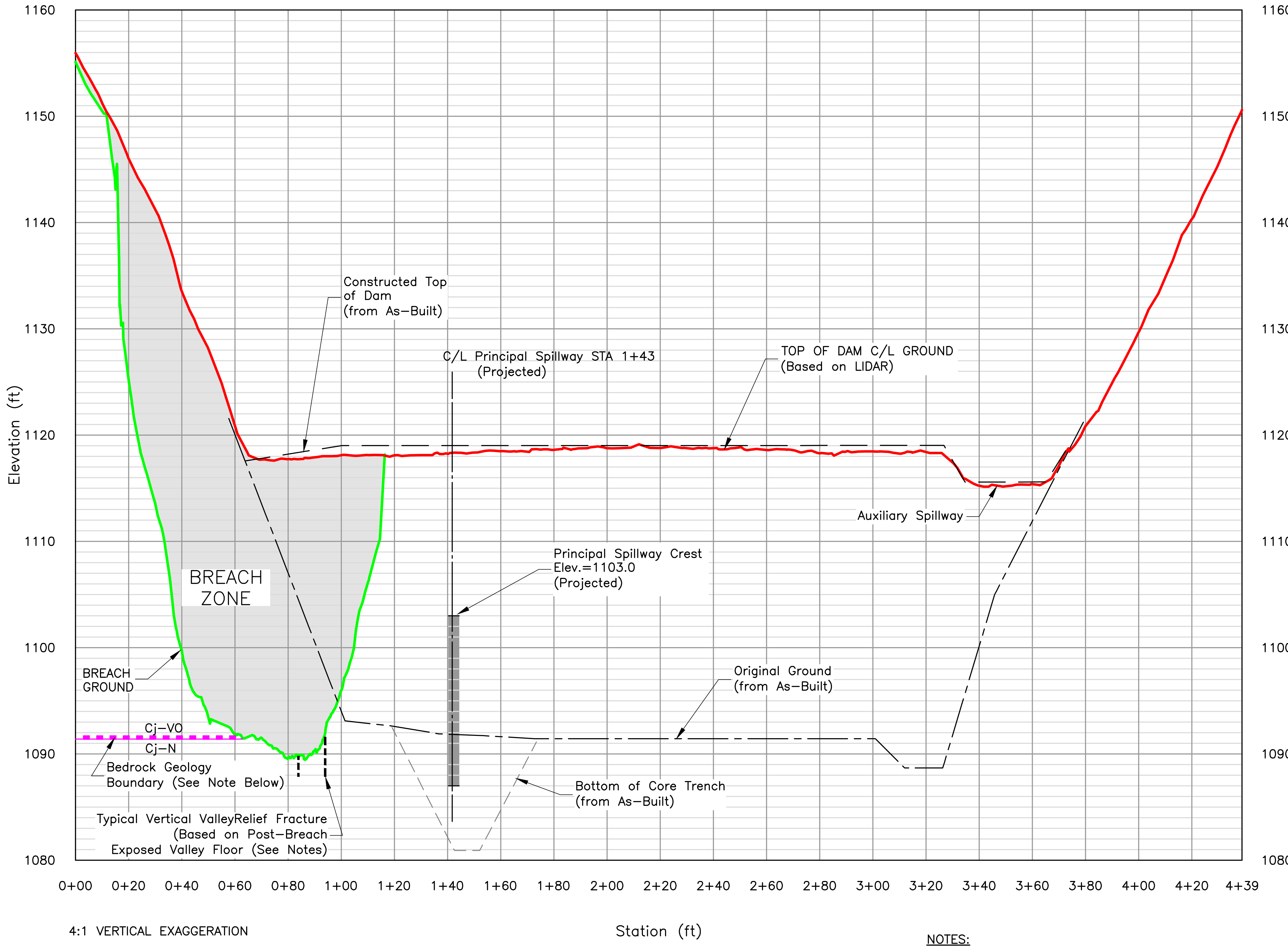
File Name WI–006

Date 08/14

Sheet 16 of 24

16 24

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



4:1 VERTICAL EXAGGERATION

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

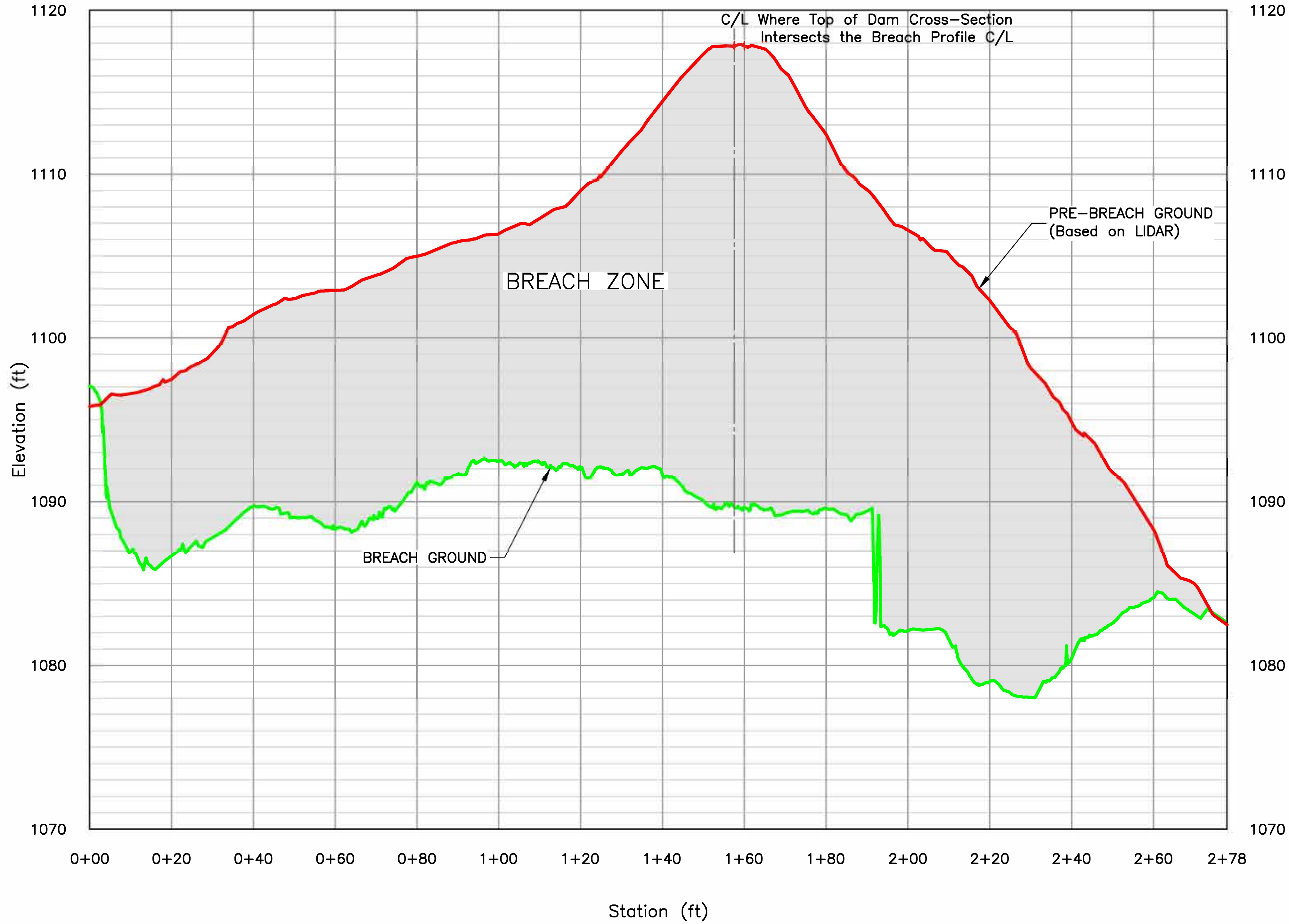
Designed JEV
Drawn
Checked
Approved

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

United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 17 of 24

PROFILE CENTERLINE OF BREACH (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

NOTES:
SOIL BORING INFORMATION AT DAM SITE IS NOT AVAILABLE

Date 5/2019

Designed JEV

Drawn

Checked

Approved

COON CREEK #23
PROFILE – C/L OF BREACH

OWNER: MONROE COUNTY

COUNTY: MONROE

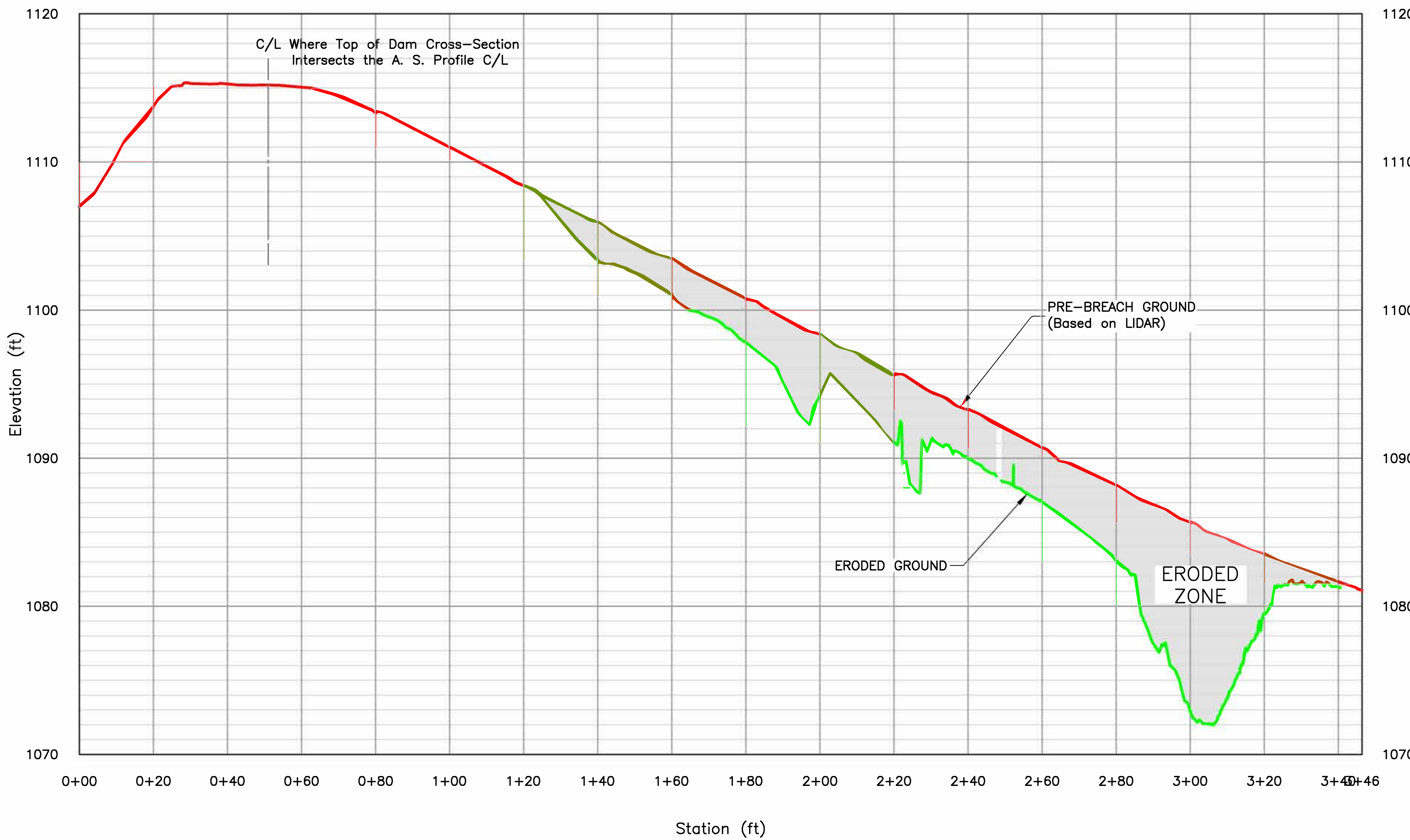
USDA
United States Department of Agriculture
Natural Resources Conservation Service

File Name WI-006

Date 08/14

Sheet 18 of 24

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

USDA United States Department of Agriculture

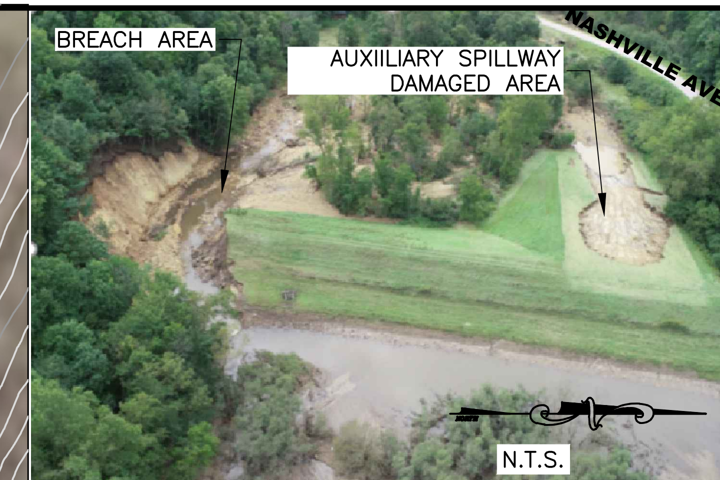
Natural Resources Conservation Service

File Name WI-006

Date 08/14

Sheet 19 of 24

19 24



LEGEND

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

GRAPHIC SCALE

50 0 25 50 100

(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

NO NAVD88 SITE ELEVATION DATA AVAILABLE

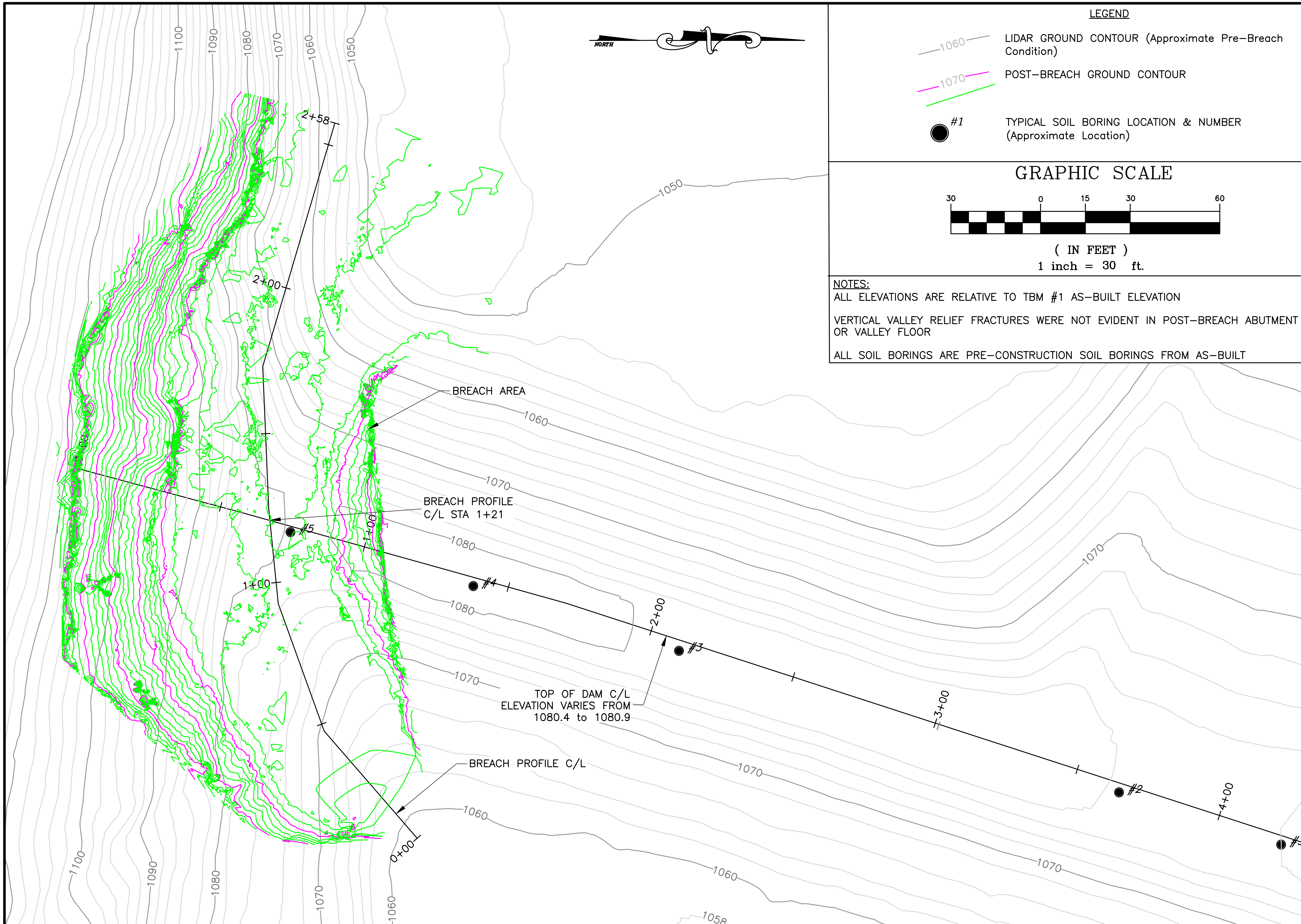
Date 4/2019
Designed JEV
Drawn
Checked
Approved

COON CREEK #29 PLAN VIEW - OVERALL

OWNER: MONROE COUNTY
COUNTY: MONROE

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 20 of 24



LEGEND

1060 LIDAR GROUND CONTOUR (Approximate Pre–Breach Condition)

1070 POST–BREACH GROUND CONTOUR

#1 TYPICAL SOIL BORING LOCATION & NUMBER (Approximate Location)

GRAPHIC SCALE

30 0 15 30 60

(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

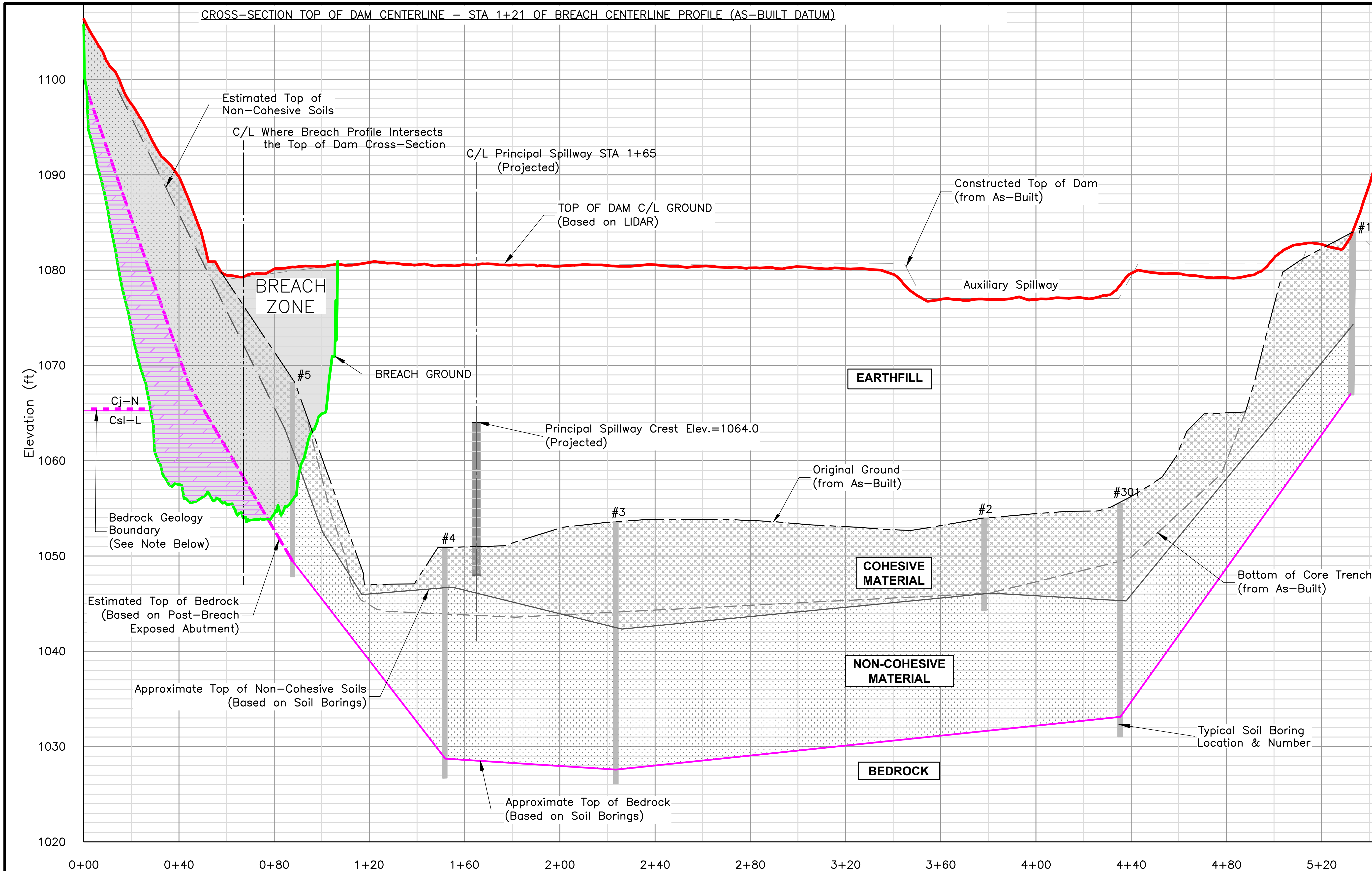
Designed	JEV	Date 4/2019
Drawn		
Checked		
Approved		

CO[illegible] CREEK #29 PLAN VIEW - BREACH AREA

OWNER: MONROE COUNTY
COUNTY: MONROE

USDA
United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 21 of 24



Station (ft)

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

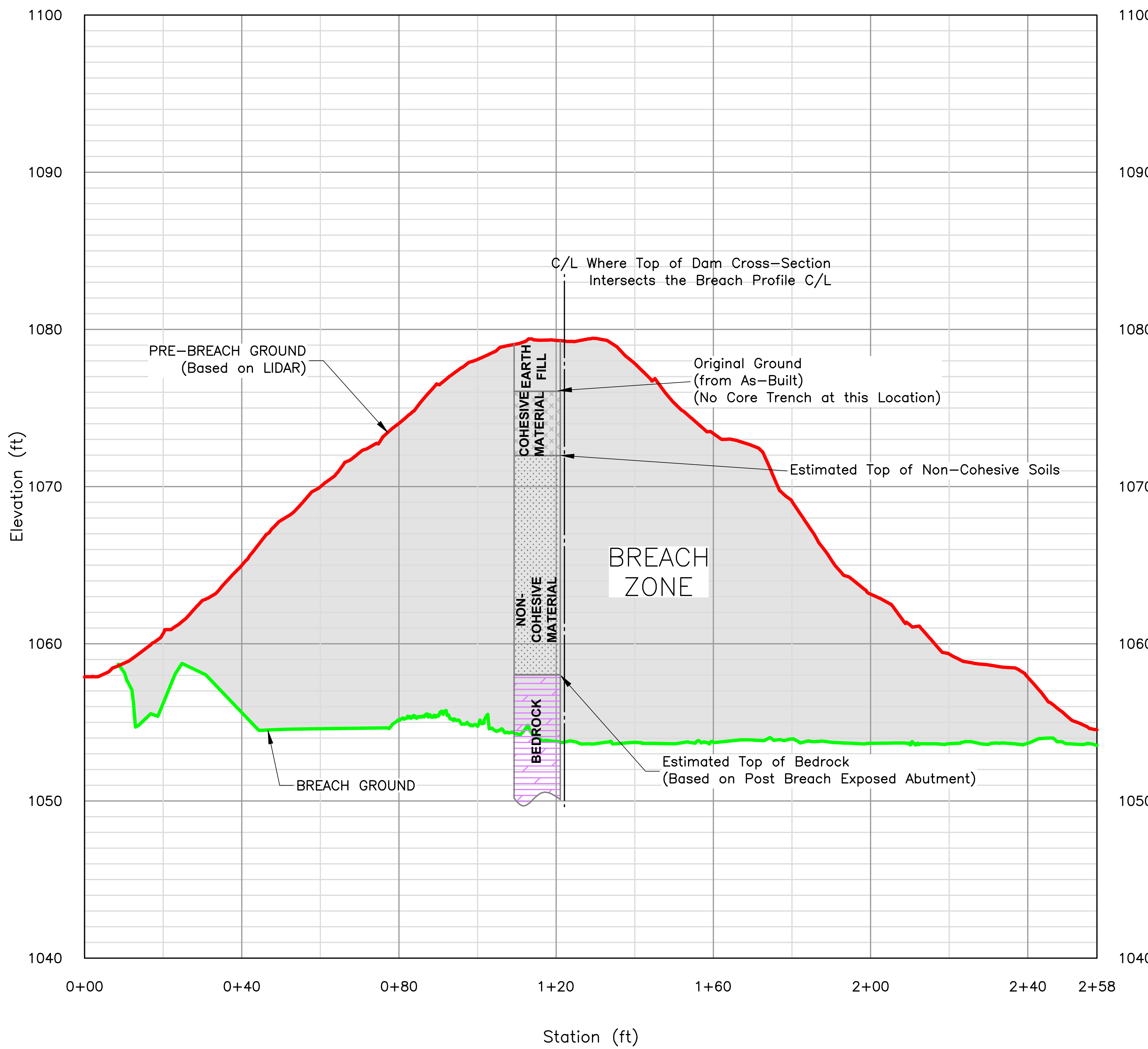
COON CREEK #29
TOP OF DAM C/L CROSS–SECTION

OWNER: MONROE COUNTY
COUNTY: MONROE

United States Department of Agriculture
Natural Resources Conservation Service

File Name
WI–006
Date
08/14
Sheet 22 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.

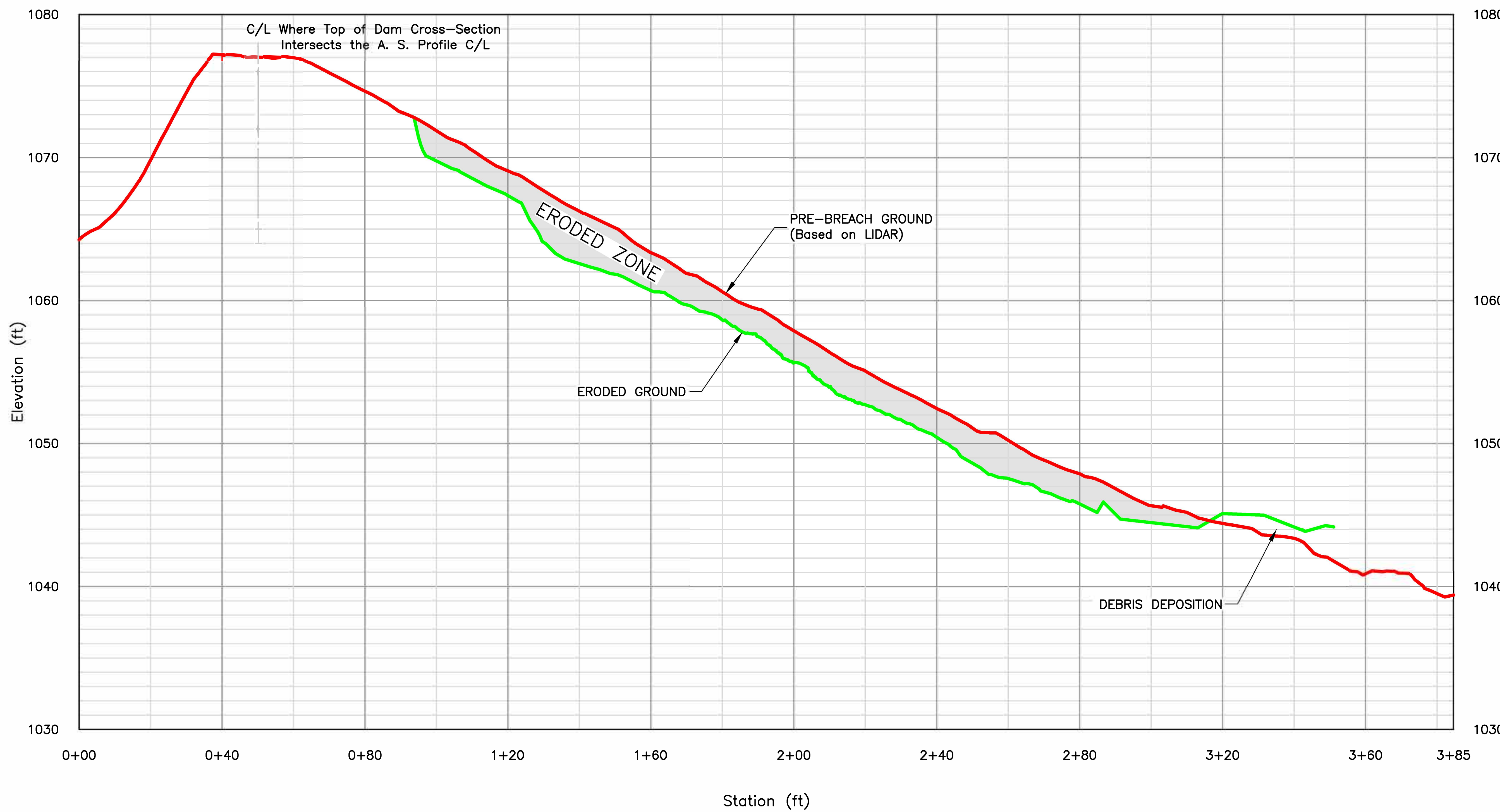
Designed JEV Date 4/2019
Drawn
Checked
Approved

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

USDA United States Department of Agriculture
Natural Resources Conservation Service

File Name WI–006
Date 08/14
Sheet 23 of 24

PROFILE CENTERLINE OF AUXILIARY SPILLWAY (AS-BUILT DATUM)



4:1 VERTICAL EXAGGERATION

Designed JEV
Drawn
Checked
Approved
Date 4/2019

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

United States Department of Agriculture
Natural Resources Conservation Service

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)

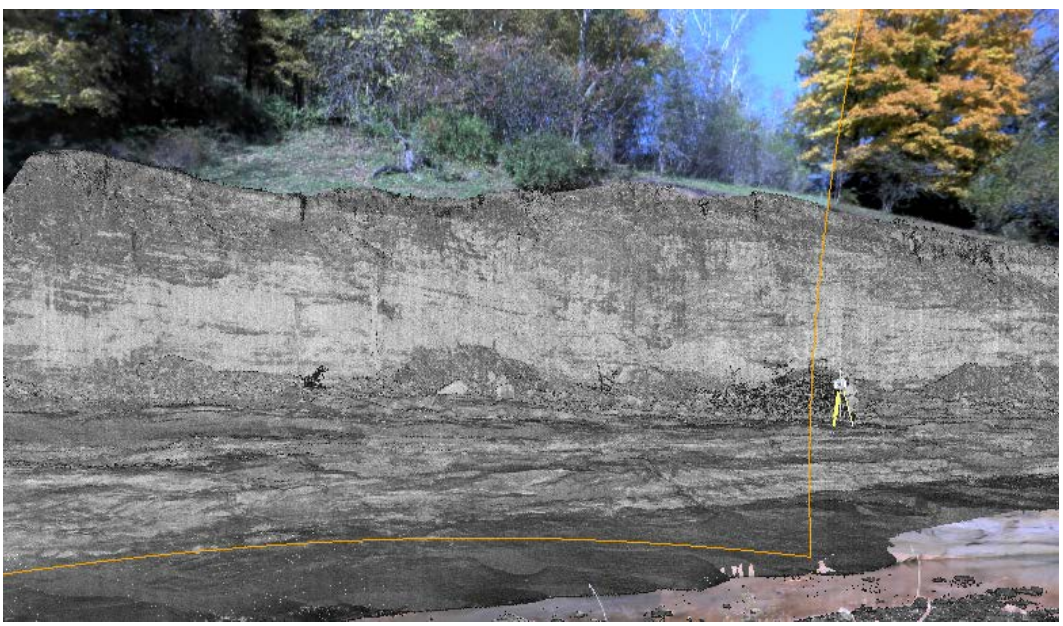


Photo 4
Example of ‘fine’ point cloud density along the exposed bedrock in breach channel. Point separation is approximately 0.15’.

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 28th. 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 28th. 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 28th. 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 28th:

> *"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 27th and early morning Tuesday, August 28th.

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 27th and ended by 4 am Tuesday, August 28th. The NWS warned of flooding along Coon Creek with estimated rainfall of 5 to 12 inches.

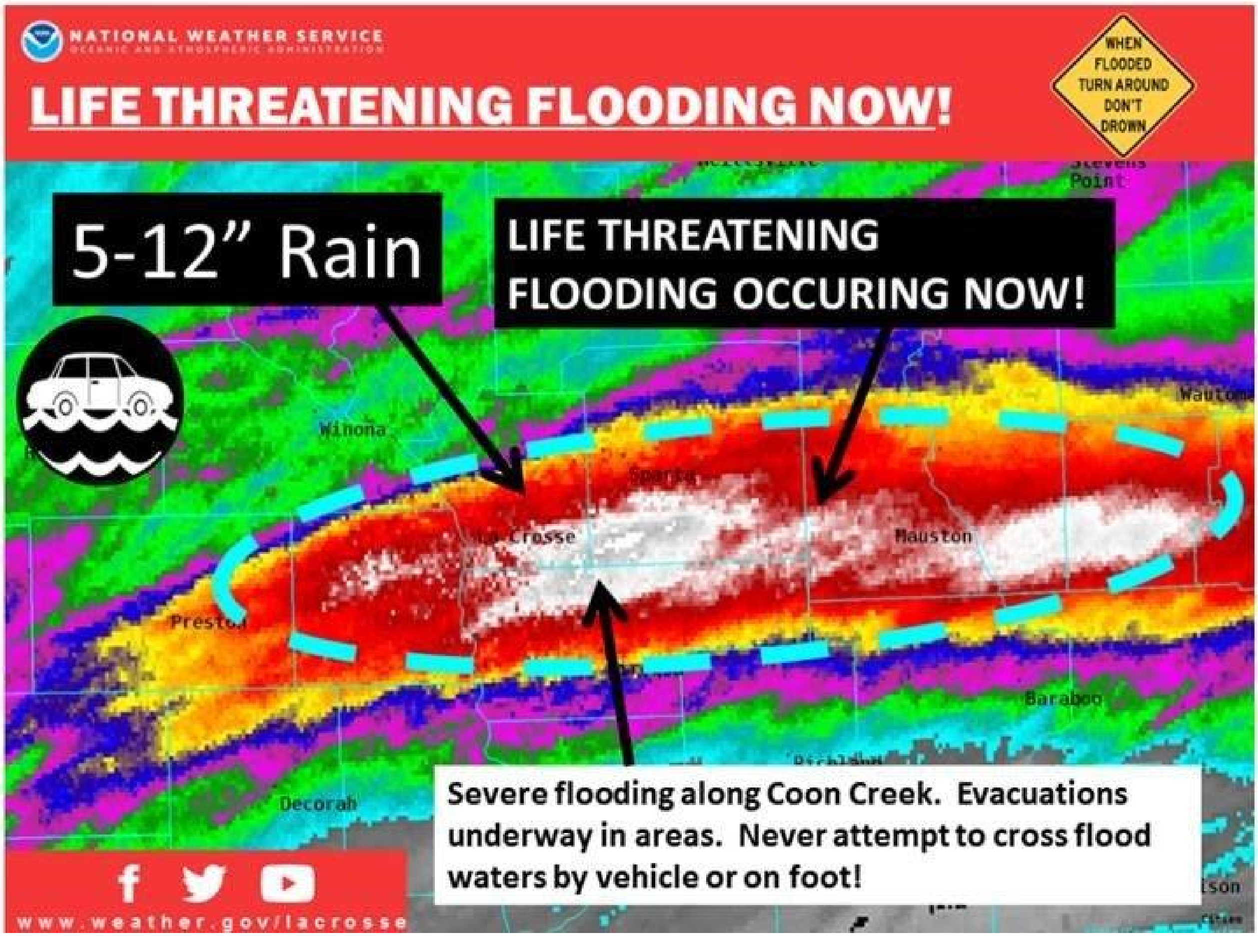


Figure 2 NWS Rainfall Graphic warning of Coon Creek flooding (Rainfall duration unspecified) Published Tues Aug 28th

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

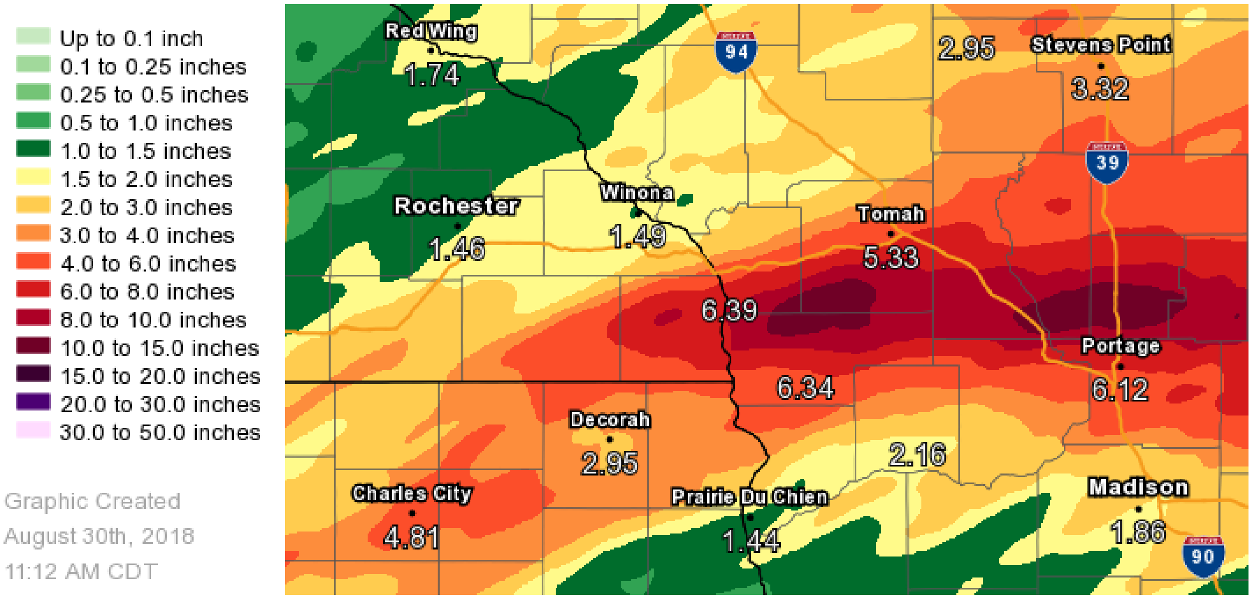
...RAINFALL REPORTS FROM TUESDAY AFTERNOON/NIGHT AUGUST 28 2018...

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




```
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.
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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 3rd 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.**

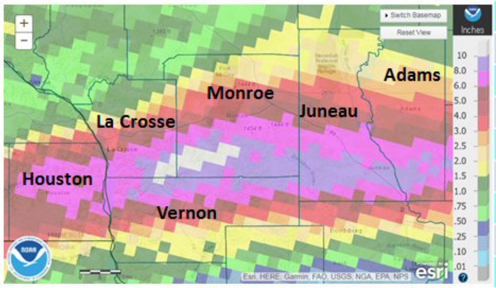
****** All of these cooperative observing sites have a reporting period from 7 AM the previous day to 7 AM the current day.**

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

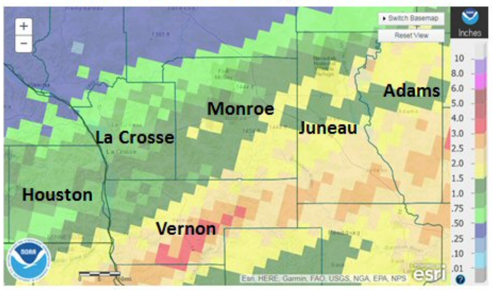
Figure 7 is a NWS 48-hour duration rainfall graphic. Figure 8 is a Channel 8 48-hour duration rainfall graphic, which was likely derived from NWS information.

August 27-28, 2018 Rainfall Event

Several rounds of heavy rain impacted the area from Monday evening (August 27th) through Tuesday evening (August 28th). 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).



Rainfall from 7 AM Monday to 7 AM Tuesday



Rainfall from 7 AM Tuesday to 7 AM Wednesday

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

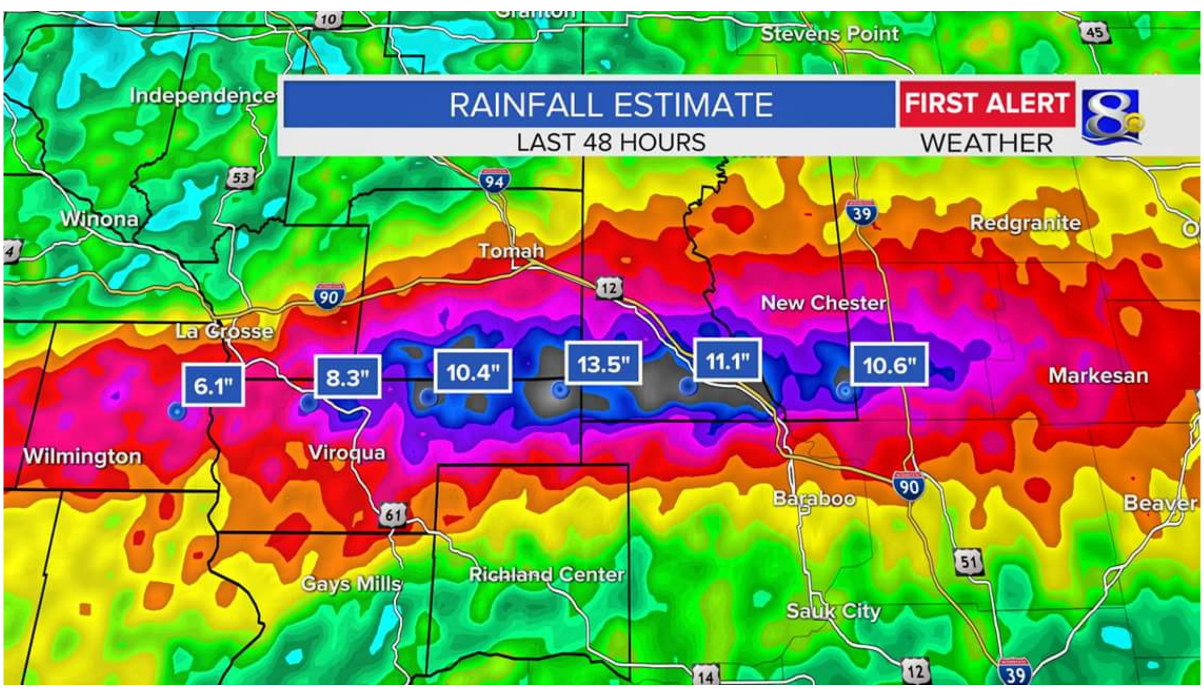


Figure 8 48-hour duration rainfall graphic from Channel 8 La Crosse, WI

2) Community Collaborative Rain, Hail & Snow Network (CoCoRHS)

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.

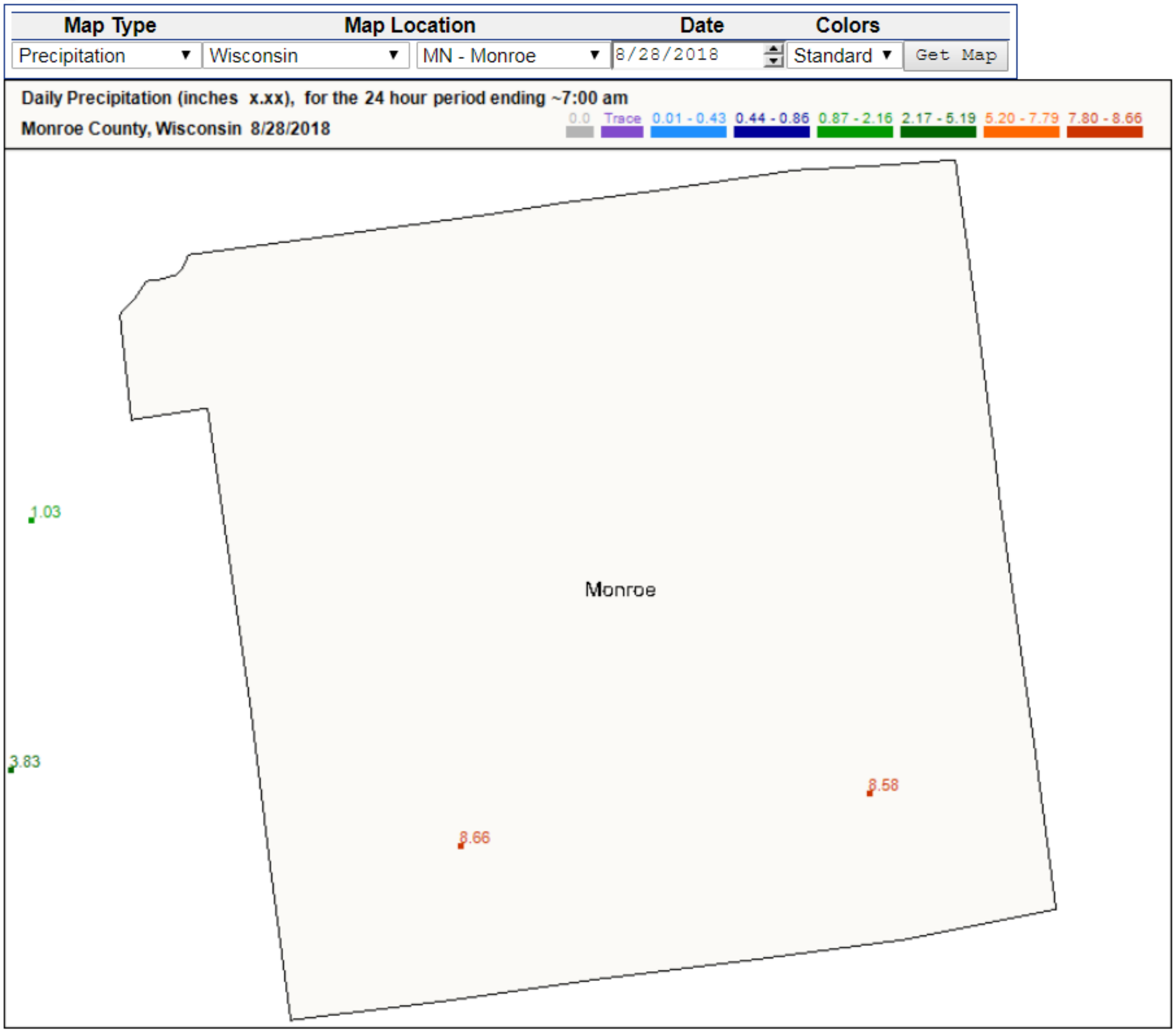


Figure 9 CoCoRaHS Monroe County, WI 24-hour rainfall ending 7 am August 28, 2018

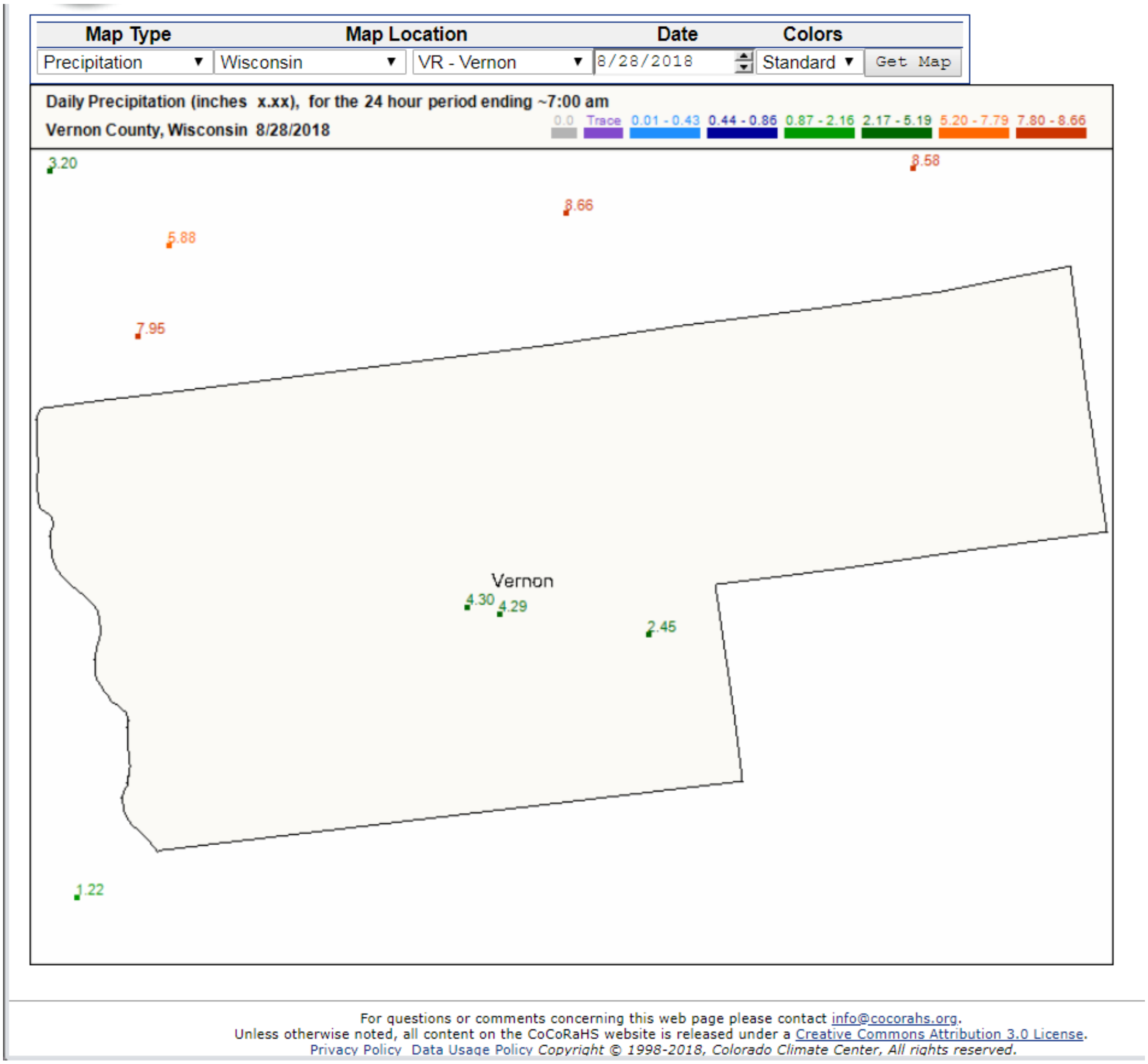


Figure 10 CoCoRaHS Vernon County, WI 24-hour rainfall ending 7 am August 28, 2018

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.

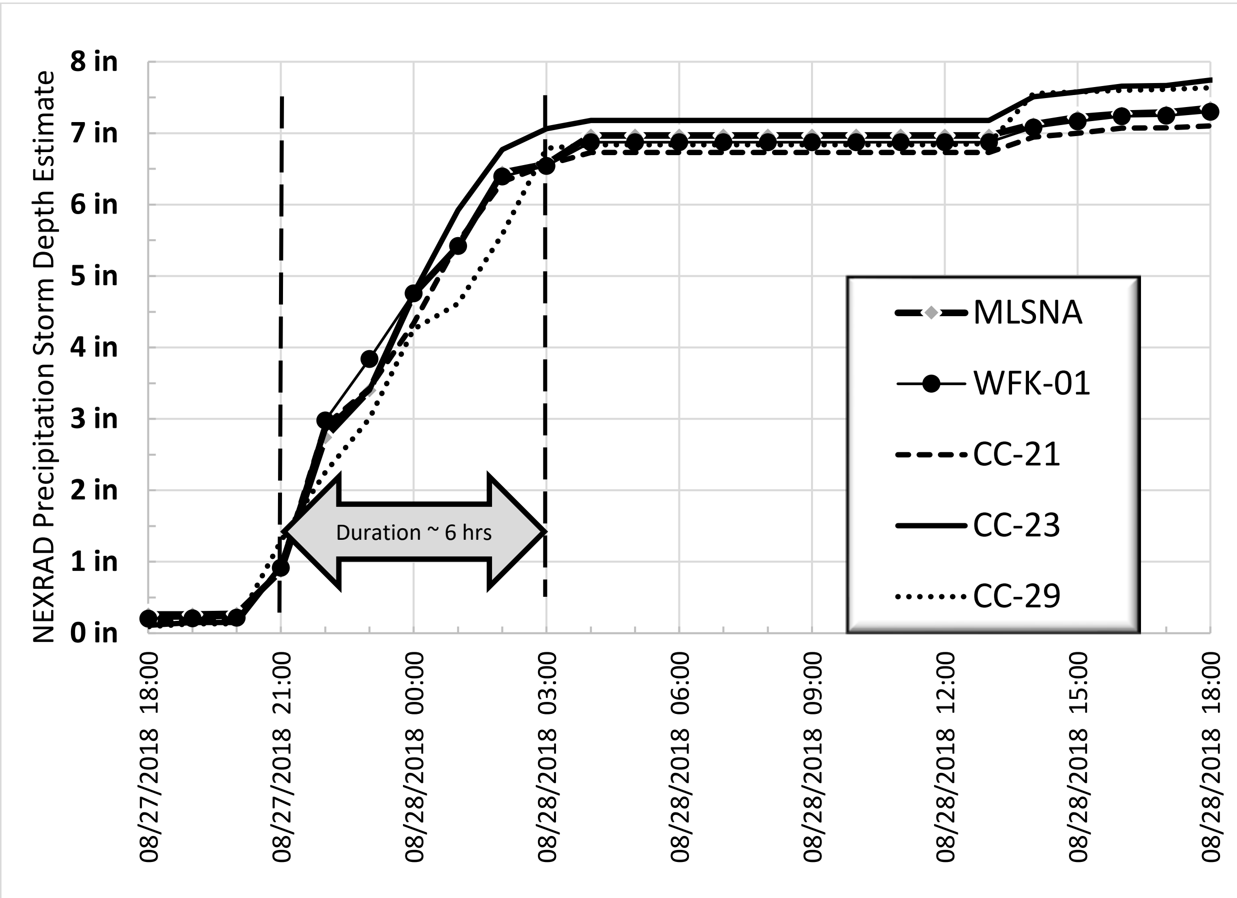


Figure 11 Storm Duration from NEXRAD Data KARX-LaCrosse, WI

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.

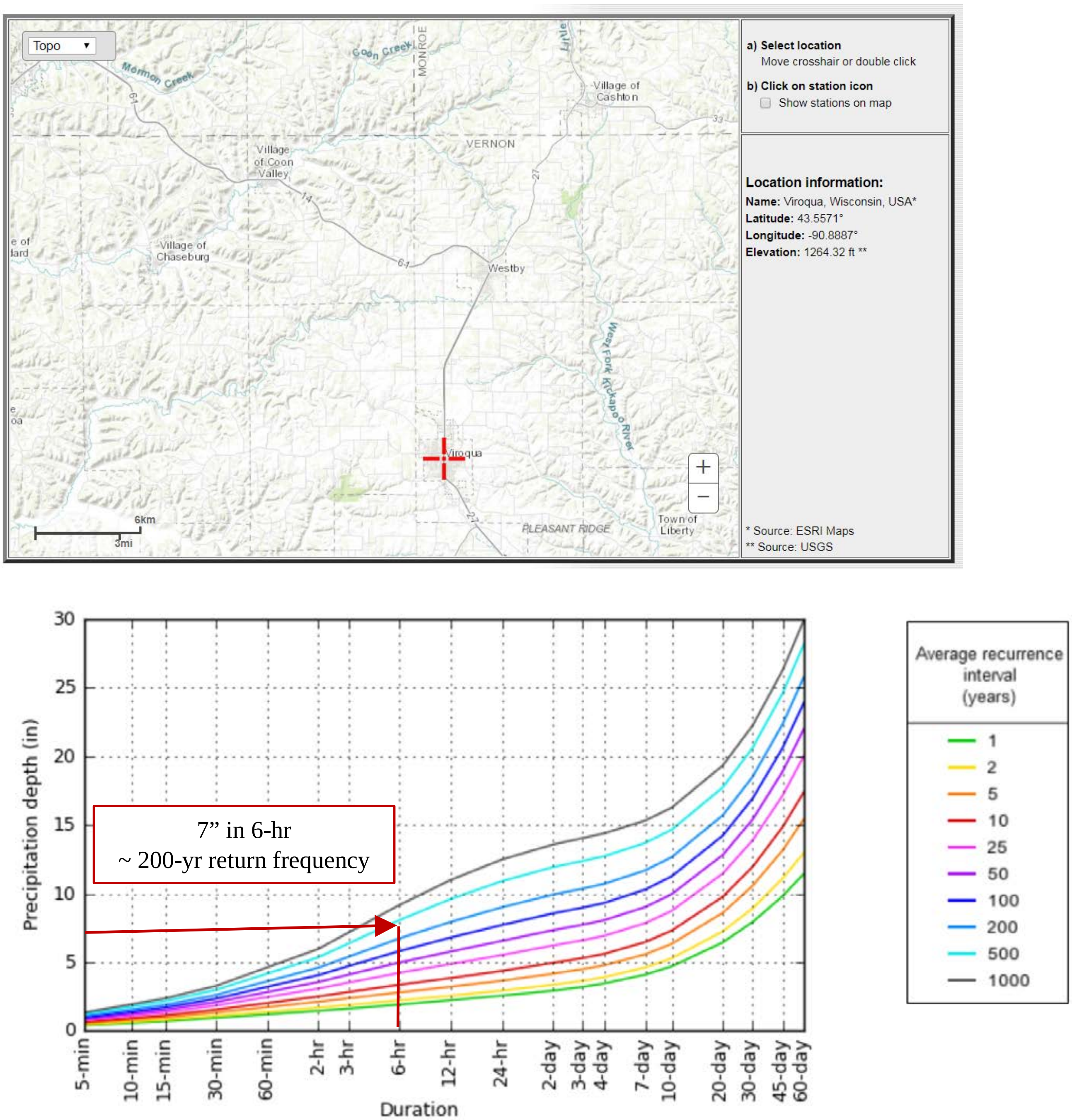


Figure 12 NOAA Atlas 14 point precipitation frequency estimates for Viroqua, WI graphical data

PF tabular – Viroqua, WI (NOAA Atlas 14)							
PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]							
Duration	Average recurrence interval (years)						
	10-yr	25-yr	50-yr	100-yr	200-yr	500-yr	1,000-yr
1-hr	**2.01** (1.57-2.55)	**2.45** (1.87-3.24)	**2.81** (2.09-3.77)	**3.19** (2.30-4.39)	**3.59** (2.49-5.10)	**4.16** (2.77-6.07)	**4.61** (2.99-6.81)
2-hr	**2.49** (1.97-3.12)	**3.06** (2.36-4.03)	**3.53** (2.66-4.71)	**4.04** (2.94-5.53)	**4.58** (3.20-6.46)	**5.35** (3.60-7.77)	**5.96** (3.90-8.76)
3-hr	**2.81** (2.24-3.50)	**3.49** (2.72-4.60)	**4.07** (3.09-5.42)	**4.70** (3.45-6.43)	**5.38** (3.79-7.58)	**6.35** (4.30-9.21)	**7.14** (4.69-10.4)
6-hr	**3.34** (2.69-4.12)	**4.21** (3.33-5.53)	**4.97** (3.82-6.59)	**5.80** (4.31-7.91)	**6.72** (4.79-9.44)	**8.06** (5.52-11.6)	**9.16** (6.07-13.3)
12-hr	**3.83** (3.12-4.68)	**4.85** (3.90-6.35)	**5.76** (4.50-7.61)	**6.78** (5.10-9.20)	**7.92** (5.71-11.1)	**9.59** (6.63-13.8)	**11.0** (7.33-15.9)
24-hr	**4.35** (3.59-5.26)	**5.50** (4.48-7.14)	**6.54** (5.16-8.56)	**7.69** (5.85-10.4)	**8.99** (6.54-12.5)	**10.9** (7.60-15.6)	**12.5** (8.40-17.9)
2-day	**4.94** (4.13-5.93)	**6.21** (5.10-7.95)	**7.32** (5.83-9.47)	**8.55** (6.56-11.4)	**9.91** (7.27-13.6)	**11.9** (8.38-16.9)	**13.6** (9.21-19.3)
3-day	**5.29** (4.46-6.31)	**6.59** (5.44-8.37)	**7.72** (6.18-9.93)	**8.97** (6.92-11.9)	**10.4** (7.64-14.2)	**12.4** (8.74-17.4)	**14.0** (9.58-19.9)
4-day	**5.59** (4.74-6.64)	**6.90** (5.73-8.72)	**8.05** (6.47-10.3)	**9.30** (7.21-12.3)	**10.7** (7.92-14.6)	**12.7** (9.01-17.9)	**14.4** (9.85-20.4)
7-day	**6.46** (5.53-7.61)	**7.83** (6.54-9.76)	**9.01** (7.30-11.4)	**10.3** (8.02-13.4)	**11.7** (8.70-15.7)	**13.7** (9.76-19.1)	**15.3** (10.6-21.6)
10-day	**7.28** (6.28-8.54)	**8.73** (7.31-10.8)	**9.94** (8.09-12.5)	**11.2** (8.81-14.6)	**12.6** (9.46-16.9)	**14.6** (10.5-20.3)	**16.3** (11.3-22.8)

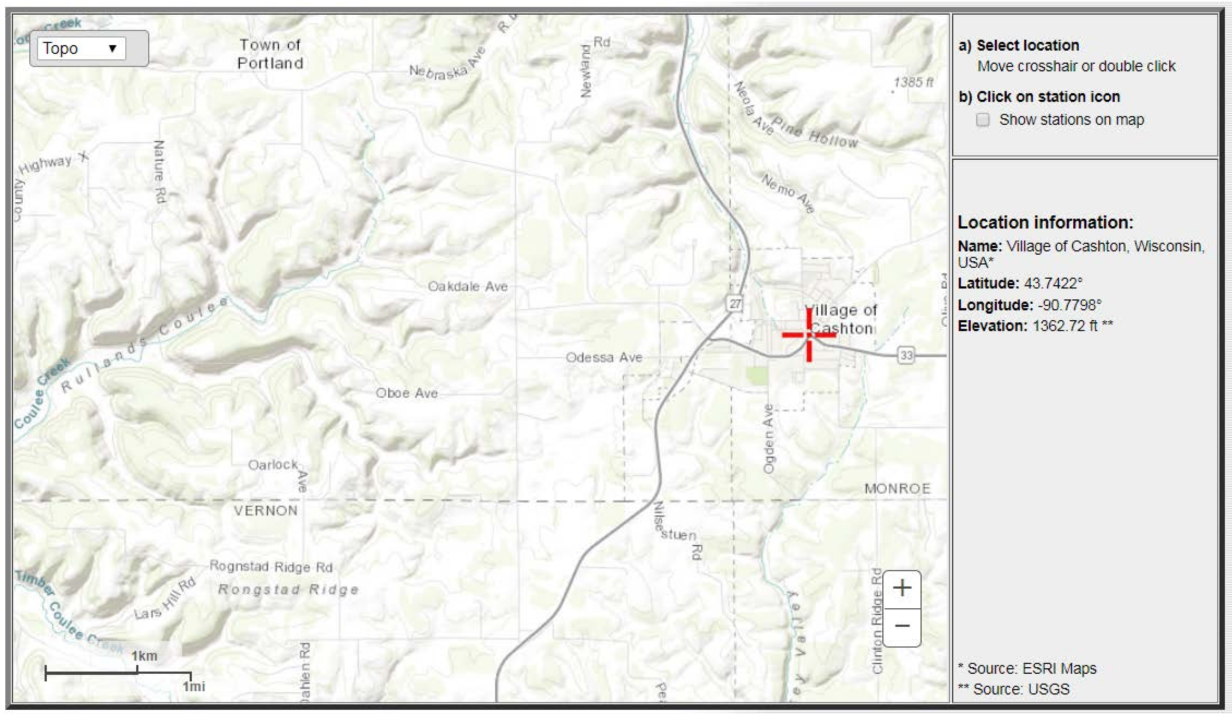
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

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.



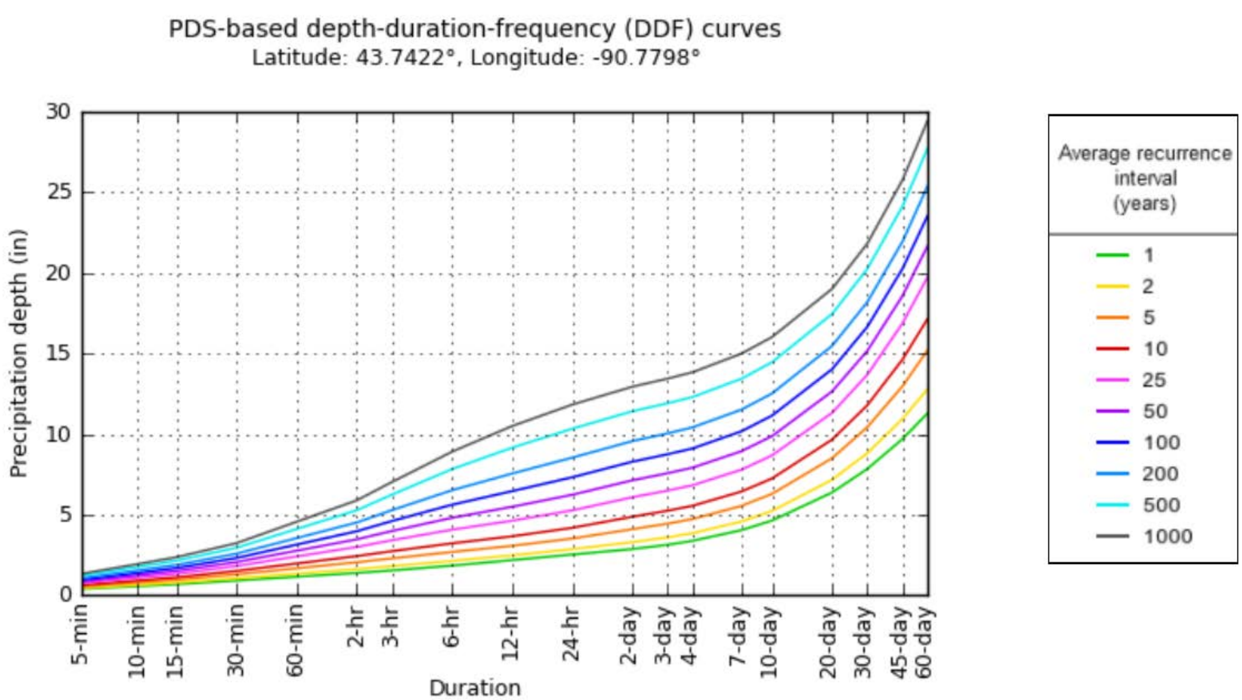


Figure 14 NOAA Atlas 14 point precipitation frequency estimates for Cashton, WI graphical data

PF Tabular – Cashton, WI (NOAA Atlas 14)

PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

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

[1] 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).

				QPE* 24-hr		NEXRAD 6-hr	
County	**Watershed Name**	**Local Name**	**Ending Date of Storm**	**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).

Dam	Local Name	County	Design Rainfall Depth	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	5.8 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 28th. 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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