CASE HISTORY-BASED VALIDATION OF NUMERICAL SIMULATIONS OF BACKWARD EROSION PIPING USING THE BOIS BRULE LEVEE BREACH

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ABSTRACT

Backward erosion piping (BEP) is a major risk factor for both dams and levees. A significant amount of work has been performed to examine the likelihood of initiation of BEP through examination of critical gradients for sand boil formation and the development of semi-empirical approaches to assess the likelihood of erosion continuing and progressing to a breach. Recently, numerical analyses have been developed to model the BEP process and these tools offer a means to incorporate more complex geometries and soil conditions than can be assessed within the semi-empirical approaches. These simulation methods have been primarily applied to laboratory tests and physical models with uniform properties, but there is a need to validate these models using case histories and explore how variability in stratigraphy and properties influences the BEP process. This study will apply numerical approaches for simulating BEP to the case history of the Bois Brule levee breach that occurred during flooding in July 1993. This paper first describes the Bois Brule levee and observations during flooding. Numerical simulations of piping progression are used to explore the effect of reasonable variations in properties and stratigraphy on the likelihood of failure and the sensitivity of the results to the river level at the time of failure. The paper highlights challenges in modeling BEP and provides guidance on which factors have the largest impact of the results.

INTRODUCTION

Levees are a critical component of a flood control system and levee breaches can have catastrophic consequences for communities. Levee breaches can be caused by many factors, but some of the most common are erosion due to overtopping, internal erosion within the body or foundation of the levee, and slope instability (ISSMGE TC201 2022). Backward erosion piping (BEP) is one common internal erosion failure mode considered for levees. BEP occurs when soil particles are progressively eroded through an unfiltered exit (e.g., a sand boil at the toe of a levee) leading to the formation of a pipe or void that extends towards the upstream (ICOLD 2015). This condition is most common when a sandy soil is overlain by a cohesive layer that can serve as a "roof" for the pipe. As the

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pipe enlarges, the levee may breach due to a loss of crest height and localized overtopping. The BEP process can be prevented through properly designed filters that stop eroded particles from leaving the foundation or halted through successful interventions, such as reducing the gradient through either raising the water level at the downstream exit or lowering the upstream head.

The potential for BEP to develop at a site is commonly evaluated using semi-empirical methods, such as Schmertmann (2000) and Sellmeijer et al. (2011). These methods have been widely used in practice, but are limited to relatively simple 2D geometries and can be unconservative for cases with 3D flow conditions, like flow exiting at a single hole rather than in a ditch or a slope face (for details see Robbins 2022). A number of numerical models for BEP have been developed and Robbins (2022) provides a comprehensive review of these studies. Challenges in using these models in practice include a lack of validation against large-scale experiments and/or case histories, input parameters that are difficult to define using standard laboratory tests, and computational challenges due to both lack of public availability of the code and computational burdens. While numerical modeling has gained wide acceptance for many geotechnical problems, numerical modeling of BEP is not commonly performed. A lack of validation is likely one of the primary barriers to these models being more widely adopted.

There have been many levee failures due to BEP, including in the Netherlands, the United States, Japan, Hungary, and China (Van Beek, Bezuijen, and Sellmeijer 2013; Vorogushyn, Merz, and Apel 2009; Takizawa, Horikoshi, and Takahashi 2018), but relatively few have detailed enough information to serve as case histories for model validation. One exception is the Bois Brule levee breach, which occurred along the Mississippi River between St. Louis and Cape Girardeau, MO during flooding in July 1993. For this location, pre-breach characterization data is available from relief well design documents. The geologic conditions in this location are typical for the middle Mississippi River with relatively thin confining blankets underlain by thick, pervious aquifers. Despite the clear failure mechanism and available characterization data, relatively few studies have been published on the Bois Brule breach and the authors are not aware of any previous analyses that examined the potential for BEP at Bois Brule.

This study seeks to provide a case history-based validation of one of the numerical tools (D-Geo Flow) used to assess progression of BEP through the analysis of the Bois Brule levee breach. The paper first describes the Bois Brule levee breach followed by a brief description of the D-Geo Flow program (Van der Meij 2023). The analysis results show that the software does predict complete BEP progression at Bois Brule for the baseline properties, but the results are sensitive to both the input properties and the assumed water level. Some implications of the findings of this study for future analyses of BEP progression are also discussed.

BOIS BRULE LEVEE BREACH

The Bois Brule levee breach was one of multiple breaches that occurred during the July 1993 floods along the middle Mississippi River. This flood event is often referred to as the Great Flood of 1993 and the flooding resulted in at least 50 deaths and billions of dollars in damages across nine states (Changnon 2021). Locally, breaches occurred in both the Kaskaskia and Bois Brule levee districts (locations shown in Figure 1), which are both located along the Mississippi River between St. Louis, MO and Cape Girardeau, MO. The river level at the Chester Bridge gauge (located between Kaskaskia Island and Bois Brule) reached a record high during this flood event (Figure 2), which was approximately 6 feet higher than the previous high from the 1973 flood (shown in Figure 2 for comparison). This level has not been exceeded since, although river levels in 2019 were close to those from 1993. Note that the breach location was repaired with a setback levee and new relief wells, so the conditions in 2019 were not the same as 1993 and the 2019 flood is not used in this study.

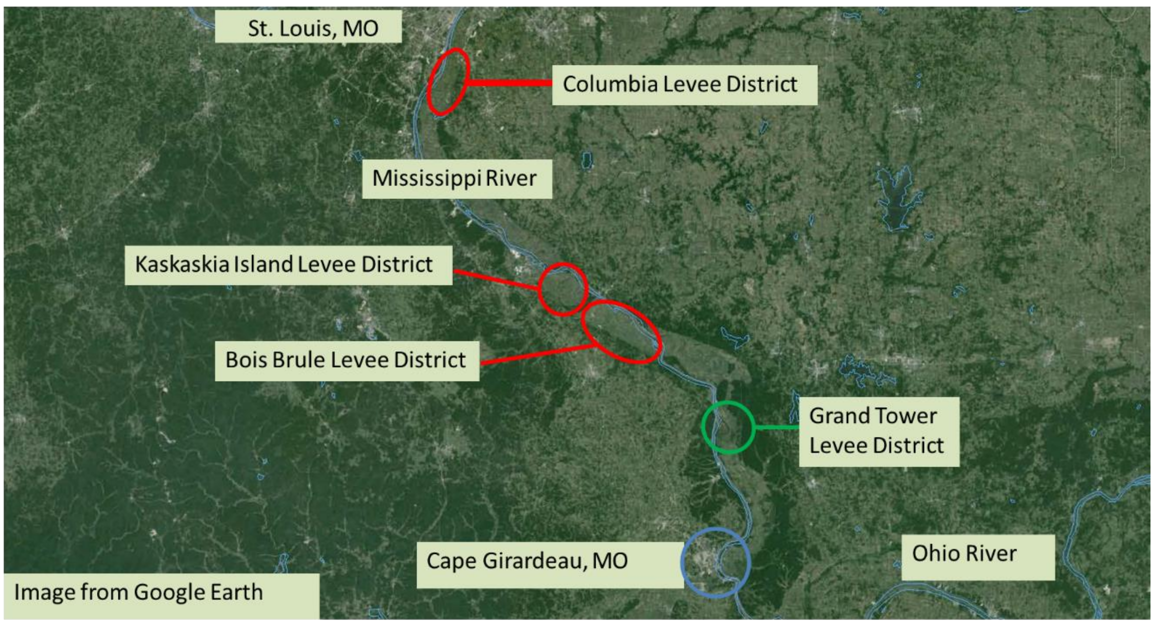

Figure 1. Map of the middle Mississippi River showing the location of the Bois Brule levee district along with nearby levee districts.

The Bois Brule breach occurred around 2:45 am on Sunday, July 25, 1993. Two US Army Corps of Engineers (USACE) employees were patrolling the levee at the time the breach occurred and heard water flowing that sounded “like a freight train” (Allen 1993). While the workers tried to shine a spotlight on the area of the levee where the sound was coming from, the breach occurred with “a big boom and water started gushing” (Allen 1993). Both USACE employees survived, but one was pulled into the breach along with his truck. The USACE employee was able to escape the truck and make it back to the levee, while the truck was later found approximately one mile from the breach location. The breach left a large scour hole evident in post-flood images (Figure 3).

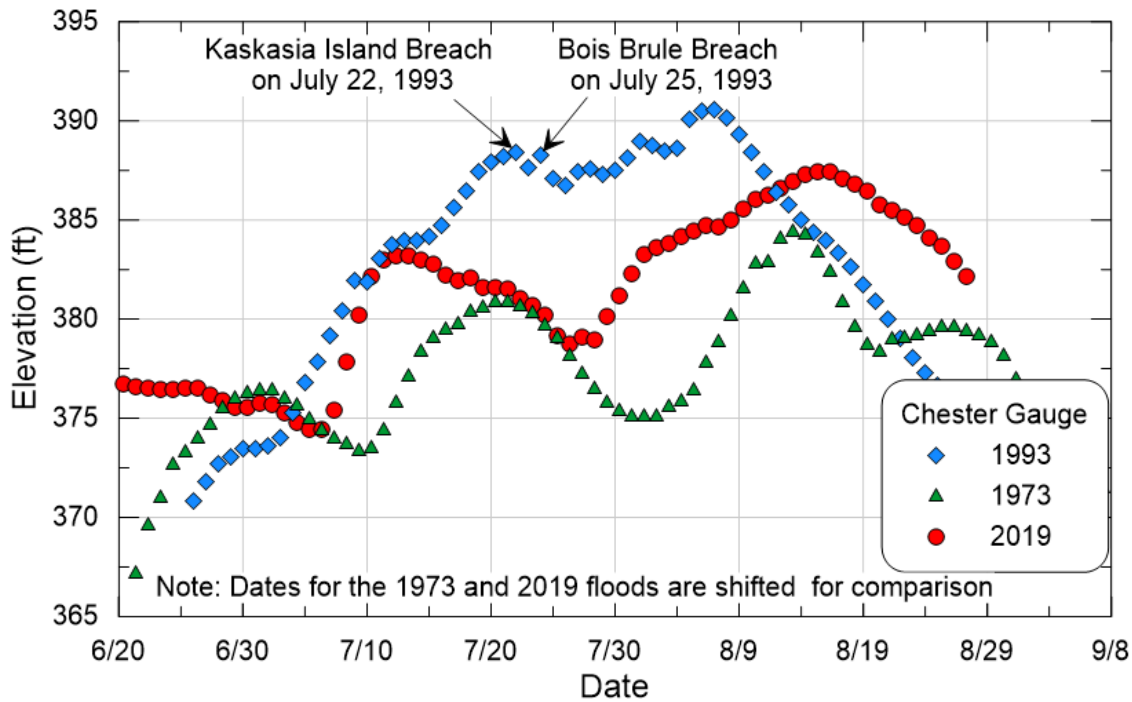

Figure 2. Daily river elevations measured at the Chester Bridge gauge for the 2019, 1993, and 1973 floods. The x-axis labels apply to the 1993 flood while the readings for the 1973 and 2019 floods have been shifted in time for comparison.

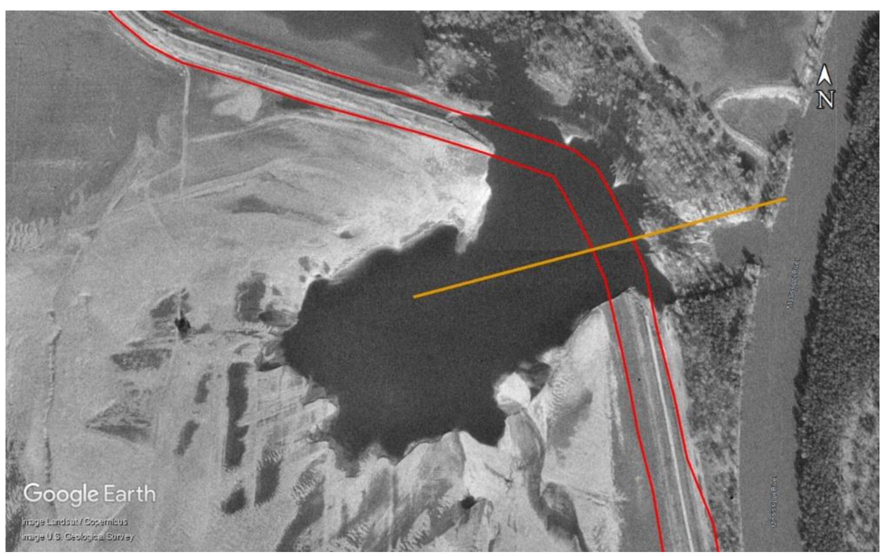

Figure 3. Post-flood aerial images (taken from Google Earth) of the Bois Brule breach showing the large scour hole that was left after the breach. The original levee boundaries are shown in red and the orange line shows the analysis cross-section used for this study.

The levee in the location where the breach occurred was constructed in the 1940s using fine-grained material taken from riverside borrow pits. Levees were typically 20 ft tall with 3:1 side slopes (Figure 4). The foundation consists of relatively thin fine-grained confining blanket (~10 ft) underlain by erodible fine sands and non-plastic silts. The deeper foundation consists of a thick (~100 ft), highly pervious aquifer which becomes coarser (and therefore more permeable) with depth. This is commonly referred to as a fining upward sequence. The levees in Bois Brule were modified in later years to add a landside berm and relief wells. The relief wells were located at the toe of the berm and the wells in place at the time of the breach were 8” I.D. wood-stave wells. The relief wells at the breach location were replaced as part of the breach repairs, but many wood-stave wells are still in use in this area. A simplified cross-section of the levee (~Station 900+00) is shown in Figure 4 based on data used in the relief well design (WES 1956). The chute was assumed to have a direct connection with the middle sand layer and the location is consistent with the relief well design assumptions (400 – 600 feet) and the mapped location of the chute in 1993 (Figure 3).

NUMERICAL SIMULATION APPROACH

The cross-section shown in Figure 4 was used as the basis for the numerical model of the levee cross-section. The riverside edge of the model was located 872 feet from the centerline of the levee to capture the location of the closest river channel (Figure 3). The landside edge of the model was located 1600 feet from the centerline of the levee to eliminate any influence of the landside boundary condition on the model results. The coarse-grained foundation was separated into three layers based on the relief well design and soil classification data from borings collected before and after the breach. In reality, the foundation has a fining upward sequence, which does not likely have distinct layers. The layered profile shown in Figure 4 represents a simplification of the true geologic conditions and the effect of the foundation layering will be examined later.

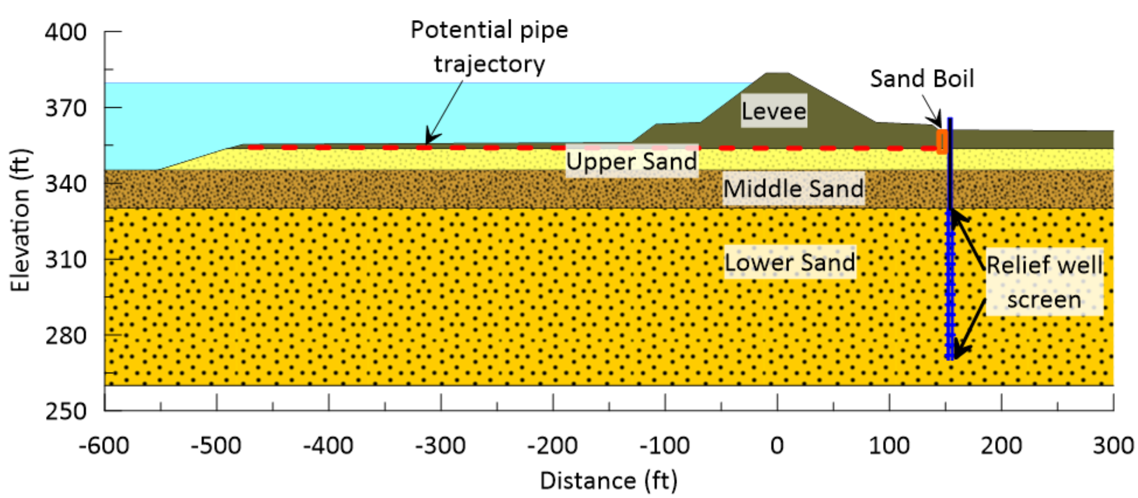

Figure 4. Simplified cross-section at the Bois Brule breach location. The levee cross-section and relief well screen interval are based on the data used for the relief well design (WES 1956). The potential trajectory of the pipe formed during BEP progression analysis is also shown as a red dashed line.

Selection of Material Properties

The two properties needed for the analyses are the hydraulic conductivity of the various layers and the D_{70} (grain diameter corresponding to 70% passing) of the material where the pipe will progress. These data were collected from the 1956 Alton to Gale underseepage study (WES 1956), which covered the Bois Brule levee and included both field and laboratory tests for hydraulic conductivity (k_h) and sieve and hydrometer analysis results. The D_{70} for the sample from the upper sand (352 feet) is approximately 0.2 mm, which will be used in the BEP analyses (Sellmeijer et al. 2011). The relief well designers (WES 1956) estimated the hydraulic conductivity of the "pervious stratum" (designated as lower sand in Figure 4) and the blanket at the breach location and these values are shown in Table 1. For the other layers, it was necessary to use the available data to estimate reasonable hydraulic conductivity values. For the upper and middle sand layers, the D_{10} of the material was used to estimate the hydraulic conductivity using the field correlation proposed by WES (1956) based on data from the Alton to Gale study. The levee hydraulic conductivity was selected based on the laboratory results on silt samples (likely borrowed material for the levee) from WES (1956). All of the selected hydraulic conductivity values are shown in Table 1.

Table 1. Hydraulic conductivity values and D_{70} values for the Bois Brule levee based on the data collected during the design of the relief wells (WES 1956). Note that a single sample was tested for each of the upper and middle sand layers, while the D_{70} for the lower sand is the median value from eight samples.

Layer	**k_h**		**D_{70}**
	cm/s	**ft/day**	**mm**
Levee	0.00004	0.11	-
Blanket	0.0003	0.85	-
Upper sand	0.033	93.2	0.21
Middle sand	0.055	155.5	0.48
Lower sand	0.12	340.2	0.78

Simulation Method

The simulations for this study were performed with the software D-Geo Flow (van der Meij 2023). D-Geo Flow uses the Kratos Multiphysics platform (Ferrándiz et al. 2023) to perform the finite element calculations. Triangular elements are used to represent the soil domain, while 1D "pipe" elements (interface elements) are used to define the location where progression may occur. The user must select this location based on the assumed initiation point (i.e., sand boil location). The trajectory must be horizontal. For this study, the pipe was assumed to initiate below the toe of the landside berm and to progress towards the upstream at the boundary between the blanket and upper sand layer (Figure 4). Once the potential pipe location is assigned, the program solves for the head distribution assuming there is no pipe. The pipe elements are then checked for stability against erosion based on the gradient across the element and the modified Sellmeijer equations (Sellmeijer et al. 2011). If the gradient exceeds that needed for progression to occur, that pipe element is assumed to be eroded and the next element is checked for stability. The height of the pipe is treated as an unknown in the equation and is used to find an equilibrium condition between the flow rate through the pipe and the apparent permeability, which is directly related to the assumed height of the pipe. The solution is checked until all elements are stable or the pipe reaches the selected upstream boundary. Details on this calculation approach are described in the D-Geo Flow manual.

For this study, the mesh size varied with a maximum element size of 5 feet near the base of the model and 3 feet near the pipe location. The ground surface on the riverside was defined as a constant head boundary to represent the flood loading and it was assumed that the ground water table was at the ground surface starting at the toe of the landside berm. The sides and bottom of the model were considered to be no flow boundaries. The relief well was modeled with an internal constant head condition along the screen interval. The relief well design assumed that the outflow would be 22 feet below the crest of the levee (elevation 361.5), but this value cannot be used for the total head at the screen elevation as it does not include potential head loss in the well. For the simulations, the potential head loss in the well was estimated using EM 1110-2-1914 (USACE 1992). Flow measurements were collected from the relief wells located close to the breach five days before the failure and showed flows of 445 - 665 gpm. Based on these measurements and the charts published in EM 1110-2-1914, the total head loss in the well was estimated to be between 1.5 and 3 feet. Using the elevation at the top of the relief well and the calculated head loss, the total head along the screen is assumed to be between 363 feet and 364.5 feet. These values will be considered in the simulations.

The current implementation of D-Geo Flow can only consider horizontal pipes and so it is necessary to define the head conditions at both the upstream and downstream edges of the pipe trajectory. In this study, the pipe was assumed to initiate at a sand boil at the toe of the landside berm (location shown in Figure 4). To select the head value for this location, it is necessary to assume a gradient across the sand boil. One assumption would be that there are hydrostatic conditions in the sand boil (i.e., the total head would be constant at the top and bottom of the blanket), but this assumption would be incompatible with the upward flow that is necessary for progression to occur. Instead it is common to assume a gradient within the sand boil, such that the total head at the bottom of the blanket is higher than the total head at the top of the blanket leading to upward flow. For this study, this gradient was assumed to be 0.3 as recommended by the model developers, which is likely conservative compared to measurements on actual sand boils (Bezuijen et al. 2019).

SIMULATION RESULTS

A set of baseline simulations (using the properties in Table 1) was performed with the two calculated head loss values for the relief well (Figure 5). The simulation with 1.5 feet of head loss in the relief well showed that piping would not be expected to progress more than 3 feet beyond the initiation point. If this head loss is increased to 3 feet, the pipe would be expected to progress for the entire length (636 feet) consistent with observations. This result shows that pipe progression is very sensitive to the relief well conditions. For both scenarios, the relief well flow was computed by extracting the nodal flow rates for each node along the relief well screen and multiplying the total flow rate by the influence length of 175 feet (the spacing between wells). The simulations with the smaller head loss (1.5 feet) showed a flow rate of 681 gpm, which was larger than any of the measurements. The simulations with the larger head loss (3 feet) showed a flow rate of 532 gpm, which is within the measured values of 445 - 665 gpm and demonstrates that the simulations with the larger head loss value are more consistent with observations. There are a number of factors that can influence this agreement between the model and measurements, including the assumed stratigraphy and properties, assumptions of steady-state flow, and the inherent limitations of the 2D model.

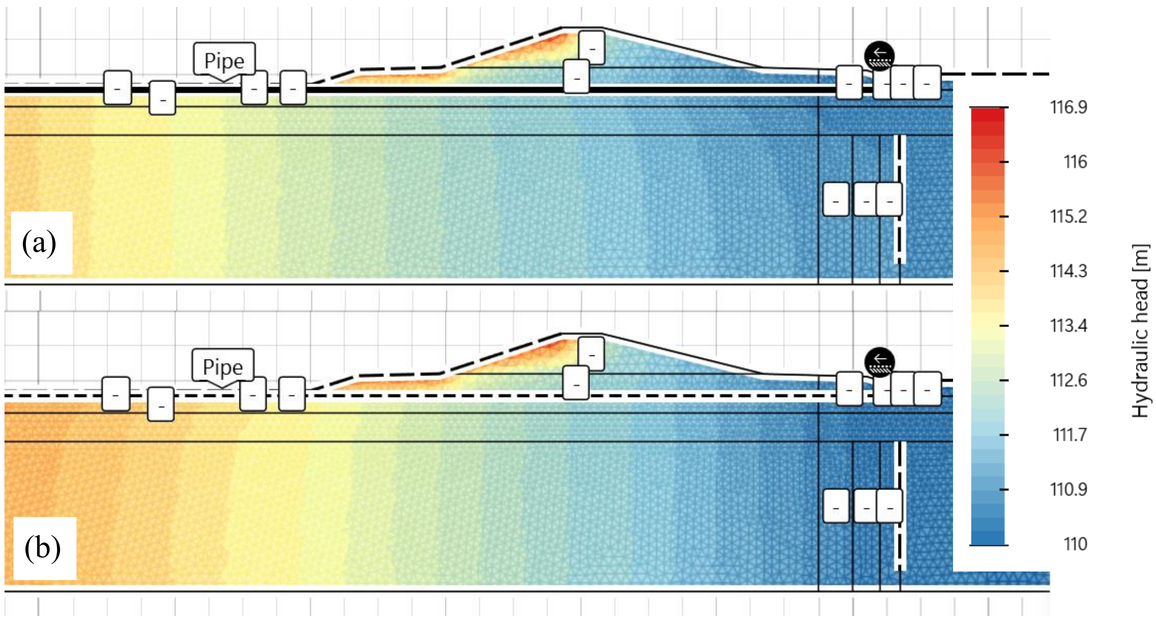

Figure 5. Hydraulic head distribution from the simulations of the Bois Brule levee considering (a) 3 feet and (b) 1.5 feet of of head loss in the relief well. The pipe progression is shown as a solid black line.

<u>Effect of River Level</u>

The effect of river level on the likelihood of piping was examined by repeating the baseline simulation described above with the riverside head set to the level of the 1973 flood, which was the previous high water level before the 1993 breach. There were no breaches in the area during the 1973 flood, although sand boils were observed within the Bois Brule area. The better performance during the 1973 flood is not surprising as the river level was significantly lower and the duration of the high water was shorter Figure 2). The simulation considered the higher head loss (3 feet) in the relief well as this was more consistent with the observations from the 1993 flood. Under this scenario, the pipe was predicted to progress, but would stabilize at 24 feet, which is under the seepage berm. It is not known if BEP initiated and stopped at the breach location during the 1973 flood, but the presence of sand boils in the area at least suggests gradients were high enough to initiate erosion.

Effect of Stratigraphy

Many alluvial deposits exhibit a fining upward sequence whereby the grain sizes become finer towards the top of the unit. In the middle Mississippi River basin, this is a common pattern with very coarse high permeability materials being located near the bottom of the aquifer and finer, siltier sands being located at the top. For examining uplift gradients (as is commonly done to assess initiation of BEP), it may be conservative to ignore this geomorphologic trait and treat the entire aquifer as a coarse, highly permeable material to minimize head loss in the aquifer and increase heads under the blanket. For analyzing progression of BEP, assuming a coarser aquifer is not conservative as coarser soils will be more resistant to erosion and therefore will reduce the likelihood of BEP.

The effect of using a fining upward sequence was examined by repeating the baseline simulations with a uniform foundation representative of the lower sand (coarsest unit). The samples in this unit show the D_{70} ranges from 0.23 mm to 12.4 mm with a median value of 0.78 mm. Using the median D_{70}, the pipe was predicted to progress less than 6 feet under 1993 flood level (Figure 6). If the D_{70} is decreased to 0.48 mm (D_{70} of the middle sand) while keeping k_h the same, then the pipe was predicted to progress less than 20 feet. These results highlight the importance of considering the fining upward sequence as the combination of finer material at the top of the layer with higher hydraulic conductivity material below is particularly susceptible to BEP progression.

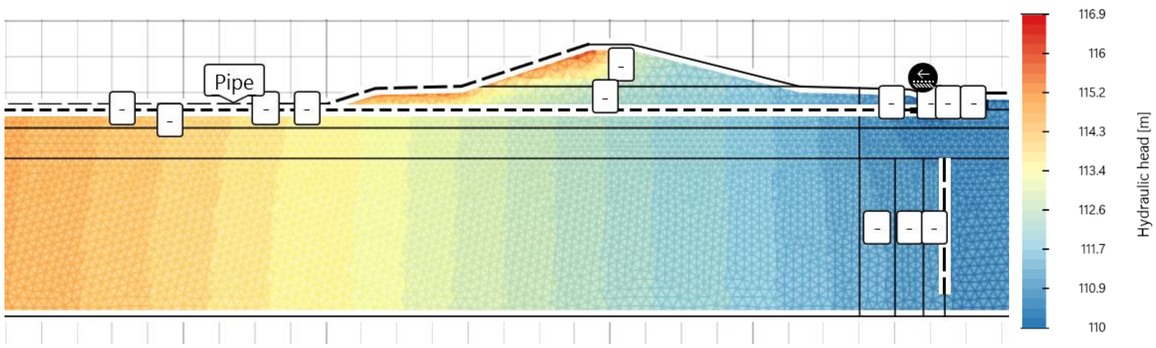

Figure 6. Hydraulic head distribution from the simulations of the Bois Brule levee showing the pipe (solid black line) has progressed only 5 feet when considering the entire foundation as a coarse sand with a D_{70} of 0.78 mm.

The previous discussion has focused on erosion resistance, but changing the stratigraphy also affects the pattern of heads and therefore gradients within the analyses, as can be seen by comparing Figures 5 and 6. Assuming the entire layer is coarse (Figure 6) leads to less head loss in the foundation and therefore higher gradients near the exit. This makes the progression of BEP more likely as this process is driven by local gradients at the pipe tip and not overall average gradients, which are determined by the boundary conditions.

CONCLUSION

The Bois Brule levee breach during the middle Mississippi floods of 1993 serves as an important case history of BEP leading to failure of a levee. The levee at this location was built on top of a thick aquifer with a fining upward stratigraphy and a relatively thin fine-grained blanket. Relief wells and a landside berm were placed at the breach location, but these were not sufficient to prevent BEP from progressing to a breach. Several studies have provided characterization data of the levee and foundation at the Bois Brule breach, which make this a good case history for examining BEP analysis methods.

This study used the numerical software D-Geo Flow to simulate progression of BEP at Bois Brule. This software uses the approach presented by Sellmeijer et al. (2011) to determine how far a pipe is expected to progress under steady-state seepage. The analyses showed that BEP progression would be a concern at Bois Brule given the flood loading from the 1993 event. Simulations for the 1973 flood event do not show significant BEP progression, which is consistent with the good performance of the levee during the 1973 floods.

The analysis results are sensitive to the selection of stratigraphy and soil properties and the assumption regarding head loss in the relief well. Fining upward sequences are particularly susceptible to BEP due to the smaller grain size near the top of the layer and the higher flow rates from deeper layers. The analyses show that ignoring the geomorphology and assuming the entire layer is coarse will lead to higher exit gradients, but the coarser soil also has more erosion resistance and very little progression is observed. Other important factors include the location of the hole in the riverside blanket, the head loss in the sand boil, and the use of horizontal layering. Additionally, this study has only considered two-dimensional analyses despite the true problem being three-dimensional. Analyses with more advanced three-dimensional like those proposed by (Robbins 2022) are planned for future studies.

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