JOURNAL OF COASTAL AND HYDRAULIC STRUCTURES

Review and rebuttal of the paper

Exploring the effect of foreshores on dike breach development via a mid-scale experiment

Van den Berg et al.

Editor handling the paper: Miguel Esteban

The reviewers remain anonymous.

Reviewer B:

This paper presents an experimental study on the effects of foreshores on dike breach development, conducted in a large laboratory facility. The innovation aspects of the study are clear, particularly in addressing the gap in understanding how foreshores influence breach dynamics. The experimental approach provides valuable empirical data that can enhance the accuracy of dike breach models, contributing to better flood risk assessments. The problem statement is concise and focused, highlighting the study's relevance and innovation. The methodology section provides a clear and logical description of the experimental setup, procedures, and analysis, providing coherence and transparency. The presentation of implications of the findings strengthens the relevance of the study to flood risk management and dike breach modeling. This is an overall valuable contribution to the field.

Recommendation: Accept Submission

We thank the reviewer for their acceptance of the manuscript after addressing their initial comments. Their comments significantly improved the structure of the article. (Furthermore, we hope they can also find themselves in the now included changes (restructure) based on the feedback from Reviewer C.)

Reviewer C:

This paper presents results from what appears to be a unique and useful experiment. The authors measure in detail the overflow erosion process of dikes with and without foreshores. The major comment I have is that is it written as a narrative, and not as a scientific report. Its format/style should be changed so that the main body of the report contains only essential information about the background, methods used, results obtained, and significance of these results. The rest of the information provided (narratives of the experiments, for example) should be moved to an appendix. This would make the paper shorter, more direct, and more readable.

Thank you for the review for the kind words. We have carefully considered your comments. Our response is outlined below.

It is not clear whether a waterproofing material was applied anywhere on the upstream slope or dike core. If not, then the phreatic line could reach the face of the protected side, and further instability via pore pressures. This needs to be discussed. It seems like if erosion is the primary mechanism to be considered, then saturation of the sand should be prevented, so that we know it is only the scour process occurring, not macroinstability.

Indeed, this is something we did not address in the article. We agree that the phreatic line could cause macro instability of the slopes. During the test procedure we were aware of this, which is why tests were started immediately once the desired water level was reached. During filling of the basin, no macro instability of the outer or inner slope was observed. Also, once the breaching process has started, water levels equalled quickly.

We have added this in the description of the test procedure, now included in the Appendices (Lines 635-637).

The article is long. It would be useful to rearrange it into a main body with the main methods and results, and an appendix with details such as the descriptions of each test and narratives of how tests were conducted.

In hindsight we understand where this comment comes from. In the Methods/Setup (chapter 2), a lot of information is not primary to understanding the results. With this in mind, we moved a lot of this secondary information to the Appendices. Chapter 3 (Results) remained largely unchanged. We believe much of this information is important to the main body. Only section 3.1 was shortened, where a large paragraph was moved to the Appendices. This made the main body of the article 2.5 pages shorter. More details about the restructuring can be found on the next page.

How were the contours of Fig 10 measured? With the laser scanner? This needs to be specified.

These were indeed obtained from the laser scanner. Thank you for addressing this missing detail. This is added to the figure caption (now Figure 9) and Appendices (Lines 618-619).

Table 1. Why does the fines fraction plus the sands fraction add to 101% for Dutch river clay?

That was a typo, thank you for the sharp eye! We checked the data, and the correct fine fraction was 36%. This is now corrected.

Recommendation: Resubmit for Review

Additionally, there are some textual changes (grammar and clarification). All changes can be found on the next pages.

Restructure of the manuscript

Here we discuss the changes to the structure of the manuscript per (sub)section.

Section		Changes
0.	Abstract	Minor textual changes
1.	Introduction	
2.	Experimental setup	
2.1	Test basin and model dike	Renamed to: General test setup - Details about the test basin moved to Appendices (including Figure 2) - Details about model dike moved to Appendices - Added sentence referring to the Appendix, including the moved sections (2.2 and 2.3) → see Lines 89-90.
2.2	Pump system	Moved to Appendices
2.3	Dike and foreshore sediment characteristics	Moved to Appendices
2.4	Data collection and post-processing	Post-processing moved to Appendices - Added sentence referring to the Appendix → see Line 92-93.
2.5	Test series and procedure	Test procedure moved to Appendices - Added sentence referring the above to Appendices → see Line 111. - Information regarding the test series has retained in the main body of the manuscript. We believe this is valuable information for the reader to interpret the results.
3.	Results	
3.1	General breach development	- Details about the development of stage 1 and 2 have been moved to the Appendix. Reference added to text → see Line 117.
3.2	Stage duration	
3.3	Breach width growth	
3.4	Final breach shape	Minor textual changes

3.5	Specific breach discharge	Minor textual changes
3.6	Estimating the discharge coefficients	
3.7	Comparison with similar breaches	Minor textual changes
4.	Discussion	
4.1	Evaluation of the experimental setup and test series	Minor textual changes
5.	Conclusions	

<u>Graphical representation of the changes is found below.</u>

Green = Added
Yellow = Moved to Appendices
Red = Removed
Blue = Textual change

Abstract:

Coastal and fluvial defences […] with and without a foreshore. We tested two types of foreshores, an erodible sand and low-erodible clay layer, acting as proxies for a sandy beach and unvegetated tidal marsh. Because dike breach flow closely resembles weir flow, the standard weir equation applies, which is also frequently used in breach discharge models. The observed foreshore effects are qualitatively evaluated using this weir equation. Depending on foreshore stability, we find that foreshores affect breach hydrodynamics which alters the weir shape, leading to reduced breach width growth and ultimately limits the specific discharge.	Coastal and fluvial defences […] with and without a foreshore. Two types of foreshores were tested, an erodible sand as a proxy for a beach/sandy wetland and a low-erodible clay layer as a proxy for an unvegetated tidal marsh. Because dike breach flow closely resembles weir flow, the standard weir equation applies, which is also frequently used in breach discharge models. The observed foreshore effects are qualitatively evaluated using this weir equation. Depending on foreshore stability, we find that foreshores affect breach hydrodynamics which alters the weir shape, leading to reduced breach width growth. In our experiment a foreshore reduced the final breach width by 10-20%. Also, we find that the presence of a foreshore has a limiting effect on the specific breach discharge.

Introduction:

Line 14:	It is evident that more overtopping events increase dike failure probability and thus flood risk.	Evidently, more overtopping events increase dike failure probability and thus flood risk.
Line 22:	Unfortunately, coastal wetlands have largely been lost worldwide due to land reclamation (poldering) (Scott et al., 2014) and coastal squeeze (Pontee, 2013).	Unfortunately, coastal wetlands are under severe pressure due to land reclamation (poldering) (Scott et al., 2014) and coastal squeeze (Pontee, 2013).
Line 31:	Generally, flood risk assessment focuses mainly on failure probability of the flood defence. A logical practice, because flood prevention is much preferred over flood mitigation from a social economic perspective, especially in vulnerable low-lying (polder) areas.	Generally, flood risk assessment focuses mainly on failure probability of the flood defence. A logical practice, because flood prevention is much preferred over flood damage mitigation from a social economic perspective, especially in vulnerable low-lying (polder) areas.
Line 41:	According to Visser (1998, 1999) the dike breaching process, given an initial damage at the crest causing overflow, can be split into five stages (Figure 1):	According to Visser (1998, 1999) the dike breaching process, given an initial damage at the crest causing overflow (t_0), can be split into five stages (Figure 1):

Line 55:	In the study described in this paper we performed multiple dike breach tests to investigate how a foreshore affects dike breach development.	In this study we performed multiple dike breach tests to investigate how a foreshore affects dike breach development.
Figure 1 caption	Breach stage n is the time interval between tn−1 and tn, where at the end of stage 5 (t6) breach growth stops.	Breach stage n is the time interval between tn−1 and tn, where at the end of stage 5 (t5) breach growth stops and at t6 flow stops.

Section 2.1 (General test setup):

The series of dike breach tests was done at Delft University of Technology, at the Flood Proof Holland (FPH) facility of VPdelta+ (Figure 2). This facility consists of five basins used to test mobile barriers and one (the largest) to test dikes. In the centre, two water storage basins are situated. A single gravity pipe connects the water storage basins to a basin. A manual valve for each pipe regulates the flow between two basins. There is a cabin for shelter and measurements, two freight containers for material storage, and a temporary depot to store the sediment. In this study the largest basin was used, located in the northeast corner. This basin is 21 m long, 16.5 m wide and 1.6 m deep, see Figure 3A. The sides of the basin consist of wooden retaining walls on the north and south ends and roughly 1:3 sloped embankments on the west and east ends. The bottom of the basin adjacent to the retaining walls is made of concrete slabs, and in between (the center) made of a thick clay layer. The concrete slabs extend 3 m up the embankment while the rest of the embankment is grass. Machinery can access the basin through a 1:9 sloped, 3 m wide, concrete slab road situated in the northwest corner of the basin. In the northeast corner, a brick wall is located from other research which could not be (re)moved. Behind the brick wall a small pump is installed which can be used to empty the basin. The gravity pipe to fill the basin is located in the southwest corner.	The series of dike breach tests was done at Delft University of Technology, at the Flood Proof Holland (FPH) facility of VPdelta+ A 3D representation of the test basin is shown in Figure 2A. This basin is 21 m long, 16.5 m wide and 1.6 m deep. Due to the relatively small size of the basin compared to the expected breach discharge, a pump system was installed to pump downstream water back upstream.

The model dike (Figure 3B) was constructed from three 0.5-0.6 m thick sand layers using a small (5 tonne) excavator. Each layer was compacted using a plate compactor. Once finished, the dike slopes were compacted by pressing the flat end of the excavator bucket onto the slope. The outer (upstream side) slope was 1:2.5. The inner slope (downstream side) was 1:3 at first, but changed to 1:2.5 to increase the downstream basin volume and to prolong tests. Foreshores were built after the dike was completed and consisted of two compacted layers. For the sand foreshore the plate compactor was used, for the clay foreshore the excavator was driven over the top layer for additional compaction.	The model dike (Figure 2B) was constructed from three 0.5-0.6 m thick sand layers using a small (5 tonne) excavator. Each layer was compacted using a plate compactor. Once finished, the dike slopes were compacted by pressing the flat end of the excavator bucket onto the slope. Foreshores were built after the dike was completed and consisted of two compacted layers. For the sand foreshore the plate compactor was used, for the clay foreshore the excavator was driven over the top layer for additional compaction.
	The sediments used in the experiment are dredged North Sea sand and Dutch river clay. The North Sea sand was delivered in two batches (one for the dike body, one for the sandy foreshore) from different sources in the North Sea, the river clay was used only for the foreshore.
	For details about the test basin and pump system, as well as sediment properties and characteristics, we refer to the Appendix A.

<u>Section 2.4 (Data collection):</u>

A schematised top view of the equipment setup is shown in Figure 5A. Two pressure sensors were installed on each sides of the dike to measure upstream and downstream water levels using OSSI-010-003C-03 and Van Essen CTD-Diver DI271, respectively. A fifth pressure sensor (Van Essen CTD-Diver DI271) was used as a barometer to compensate for local air pressure fluctuations. The upstream sensors initially had a 2 Hz measurement frequency, but was increased to 10 Hz after the first test due to waves in the basin once the pump system was operational. The downstream sensors and barometer had a 1 Hz burst frequency, averaging 10	Here, only the data collection done during the experiment is outlined. Post-processing is discussed in the Appendix B. A schematised top view of the equipment setup is shown in 5A. Two pressure sensors were installed on each side of the dike to measure upstream (OSSI-010-003C-03) and downstream (Van Essen CTD-Diver DI271) water levels. A fifth pressure sensor (Van Essen CTD-Diver DI271) was used as a barometer to compensate for local air pressure fluctuations.

measurements and thus had a 0.1 Hz measurement frequency. In post-processing, the upstream pressures were averaged to match the 0.1 Hz frequency of the other sensors. Then, the measured pressures were corrected for atmospheric pressure and their location with respect to the basin bottom. Missing data due to emerged sensors were estimated from camera footage or linear interpolation (see Figure 13).	
Four cameras (GoPro Hero10, 1080p@60fps) were used to record the breach growth from the front, back and top (facing upstream and downstream attached to an overhead metal frame). A 1x1m red grid was spray painted on the inner slope of the dike to record breach growth using the front facing camera. The breach width was first obtained from the camera footage pixel distance at equal intervals to the pressure sensors (0.1 Hz), then converted to meters using the 1x1m grid. Inaccuracies in pixel distance measurements was corrected by ensuring a monotonically increasing breach width.	Four cameras (GoPro Hero10, 1080p@60fps) were used to record the breach growth from the front, back and top (facing upstream and downstream attached to an overhead metal frame). A 1x1m red grid was spray painted on the inner slope of the dike to measure the breach width when post-processing the footage from the front facing camera.
A 3D laser scanner (Leica ScanStation P40) was used to scan the entire basin at three different positions, before and after a test. Unfortunately, only scans could be done from the tests where a foreshore was present due to scanner availability. Using CloudCompare V2.12.4, the three pointclouds were aligned using three reference markers that were placed inside and around the basin. Then, the pointclouds were merged and only the basin area was retained. Lastly, the pointcloud was subsampled with a minimum point distance of 1.5 mm to improve workability. From the scans before a test, basin volumes were obtained. By overlaying the scans after a test, the eroded sediment volume could be computed. From the measured basin volumes at different heights (step size=0.1 m), the basin storage area A as function of the water level (A(h)) was computed (Figure 14).	A 3D laser scanner (Leica ScanStation P40) was used to scan the entire basin at three different positions, before and after a test. From overlaying the scans, breach shape and eroded volumes were determined. Unfortunately, only scans could be done from the tests where a foreshore was present due to scanner availability.
	For details about the post-processing steps of the data, we refer to the Appendices.

Section 3.1 (General breach development):

For all tests, […] , flow along the inner slope initiated stage 1. During stage 1, the inner slope steepened and a staircase profile developed (Figure 6A). […] The downstream water level rose quickly and above the lowering breach bottom, causing a submerged weir during stage 3 (Figure 5D)	For all tests, […], flow along the inner slope initiated stage 1. During stage 1, the inner slope steepened and a staircase profile with pools developed where sediment transport capacity (STC) was reached (Figure 4A, see also Appendix D). […] The downstream water level rose quickly and, due to the small downstream basin area, above the lowering breach bottom, causing a submerged weir during stage 3 (Figure 4D)
The process that led to this is as follows: first, sediment pick-up by the flow along the slope increases the sediment concentration, and simultaneously increases the slope angle (steepness). At some point along the slope, before reaching the toe of the slope, the sediment concentration in the flow is maximum, reaching sediment transport capacity (STC). Once STC is achieved, sediment pick-up (erosion) and deposition (sedimentation) rates are equal. Downstream of this location the original slope angle is retained. Consequently, as upstream the slope angle continuously increases, the flow encounters a step and a pool is formed where sedimentation occurs (lowering sediment concentration). The pool overflow can then again pick-up sediment along the slope and another step is created until the flow reaches the toe of the slope. Throughout stage 1 the number of steps decreased as on average the slope steepened, and remained mostly constant in stage 2 (Figure 6B).	
During stage 2, the observed water surface profile was nearly parallel to the initial dike inner slope. This implies that the slope did not steepen much past its initial slope in stage 1 (nor 2), contrary to the prediction by Visser (1998). The cause may be insufficient STC (staircase profile) to visually steepen the slope. This is supported by the observation that in stage 3 the number of steps	During stage 2 (Figure 5B), the observed water surface profile was nearly parallel to the initial dike inner slope. This implies that the slope did not steepen much past its initial slope in stage 1 (nor 2), contrary to the prediction by Visser (1998).

decreased to none and steeper water surface profiles were observed (Figure 6C).	
In accordance with Visser (1998) the breach bottom quickly lowered in stage 3, resulting in rapid increase of flow velocity and discharge. The downstream water level rose quickly and above the lowering breach bottom, causing a submerged weir during stage 3 (Figure 6D). The breach bottom continued to decrease, but never eroded to the basin bottom. Thus, stage 4 was skipped and stage 5 was reached (Figure 6E). Stage 5 was short and the test ended when the water levels equalled (Figure 6F). Afterwards, the breach side slopes remained unstable and collapsed (slumping), further increasing the breach width. This increase in breach width and changes to the breach shape were only taken into account in the analysis of the final breach shape (Section 3.4). During stages 1 and 2, eroded sediment was deposited near the toe of the inner slope, creating a sill (see Figure 5B, submerged from C onwards). This sill can be eroded in stage 3 by increased flow velocity and discharge (sufficient STC). Although this was briefly observed, the quickly increasing downstream water level and limited basin area prevented this mechanism to continue. The small excavator mitigated this somewhat but its reach and capacity were insufficient to have a noticeable affect. The presence of the sill near the toe of the inner slope limited the development of a scour hole in the breach (Section 3.4).	In accordance with Visser (1998) the breach bottom quickly lowered in stage 3 (Figure 5C), resulting in a rapid increase of flow velocity and breach discharge. The downstream water level rose quickly and above the lowering breach bottom, causing a submerged weir during stage 3 (Figure 5D). The breach bottom continued to decrease, but never eroded to the basin bottom. Thus, stage 4 was skipped and stage 5 was reached (Figure 5E). Stage 5 was short and the test ended when the water levels equalled (Figure 5F). Afterwards, the breach side slopes remained unstable and collapsed (slumping), further increasing the breach width. This increase in breach width and changes to the breach shape were only taken into account in the analysis of the final breach shape (Section 3.4). During stages 1 and 2, eroded sediment was deposited near the toe of the inner slope, creating a sill (see Figure 5B, submerged from C onwards). This sill can be eroded in stage 3 by increased flow velocity and discharge (sufficient STC). Although this was briefly observed, the quickly increasing downstream water level and limited basin area prevented this mechanism to continue. A small excavator was used to mitigate this somewhat but its reach and capacity were insufficient to have a noticeable effect. The presence of the sill near the toe of the inner slope limited the development of a scour hole in the breach (Section 3.4).
Despite the similarities in the breach development, differences in the duration of the same stage between some tests were observed. These differences could sometimes be related to the foreshore (Section 3.2). For the remainder of the results, the effect of the foreshore on the breach development is evaluated using the weir equation (Equation 1). Changes in hydrodynamics alter the erosion processes in the breach, leading to changes	Despite the similarities in the breach development, differences in the duration of the same stage between some tests were observed. These differences could sometimes be related to the foreshore (Section 3.2). For the remainder of the results, the effect of the foreshore on the breach development is evaluated using the weir equation (Equation 1). Changes in hydrodynamics alter the erosion processes in the breach, leading

in the breach shape (width B, Section 3.3 and discharge coefficient Cd, Section 3.4 & 3.6). Ultimately, all changes affect the breach specific discharge (q, Section 3.5). Lastly, we compare the observed weir shape with historic breaches in Section 3.7.	to changes in the breach shape (width B, Section 3.3 and discharge coefficient Cd, Section 3.4 & 3.6). Ultimately, all changes affect the breach specific discharge (q, Section 3.5). Lastly, we compare the observed weir shape with historic breaches in Section 3.7.

Figure 4 caption:

Example of breach development during a test (here: R2, time (t) is test specific). […]	Example of breach development during a test (here: test R2, time (t) is test specific). […]

Section 3.4 (Final breach shape):

A longer stage 3 duration […] reveals foreshore effects on this weir shape.	
Visser (1998) defines the breach cross-section as a trapezoid. The initial breach shape (incision at test start) was trapezoidal, but during the tests the breach side slopes were almost vertical (80-90°), making the trapezoidal shape almost rectangular. The trapezoidal shape was again observed after the tests (Figure 10C), after breach side slope slumping had occurred while the water in the basin was drained after a test. The angle of the breach side slopes below the equilibrium waterlevel (h=100-110 cm, see Appendices, Figure 14) was measured at 25-30°, which is less than the dry internal friction angle of the sediment. On-site, quicksand like behaviour under cyclic loading of the sediment in the breach was observed, releasing a lot of pore water and compacting. […]	Visser (1998) defines the breach cross-section as a trapezoid. The initial breach shape (incision at test start) was trapezoidal, but during the tests almost rectangular because the breach side slopes were almost vertical (80-90°). The trapezoidal shape was again observed after the tests (Figure 9C), after breach side slope slumping had occurred while the water in the basin was drained after a test. The angle of the breach side slopes below the equilibrium waterlevel (h=100-110 cm, see Appendices, Figure 14) was measured at 25-30°, which is less than the dry internal friction angle of the sediment. […]
Regarding the weir shape of breaches with a foreshore, most noticeable is the weir edge (Figure 10A). A straight and elliptical weir edge is observed for respective clay and sand foreshores, owing to a stable (low-erodible) clay layer and erodible sand layer. Further, the contour lines near the foreshore edge indicate a steeper slope for the clay than the sandy foreshore. This is better observed from the breach profiles in Figure 10B. The downstream weir slope for the sand and clay foreshore tests is roughly 15° and 20°, respectively. Similarly to the reference tests, the weir edge of the sand foreshore was rounded, following local streamlines, while an abrupt (headcut like) edge for the tests with a clay foreshores was observed. Also, for the clay foreshore tests the breach has a stronger hourglass shape compared to the sandy foreshore tests (Figure 10A), likely as a result from more horizontal flow contraction.	Regarding the weir shape of breaches with a foreshore, most noticeable is the weir edge (Figure 9A). A straight and elliptical weir edge is observed for respective clay and sand foreshores, owing to a stable (low-erodible) clay layer and erodible sand layer. Further, the contour lines near the foreshore edge indicate a steeper slope for the clay than the sandy foreshore (see also Figure 9B). The downstream weir slope for the sand and clay foreshore tests is roughly 15° and 20°, respectively. Similarly to the reference tests, the weir edge of the sand foreshore was rounded, following local streamlines, while an abrupt (headcut like) edge for the tests with a clay foreshore was observed. Also, for the clay foreshore tests the breach has a stronger hourglass shape compared to the sandy foreshore tests (Figure 9A), likely as a result of more horizontal flow contraction.

Section 3.5 (Specific breach discharge):

To conclude, with just a qualitative analysis it is revealed that foreshore can limit the specific breach discharge, which is caused by the foreshore becoming the weir instead of the dropping breach bottom. Essentially, this mechanism results in a limitation of h0 once the breach bottom drops below the foreshore level. A weak trend may be observed in the comparison between test R3 and the foreshore tests. If the foreshore stability or foreshore level had the largest impact in reducing the specific discharge between the foreshore tests is difficult to determine. Nonetheless, it is likely that for longer test duration and equal initial foreshore level, a clay foreshore is more effective due to its low erodibility.	To conclude, with just a qualitative analysis it is revealed that a foreshore can limit the specific breach discharge, which is caused by the foreshore becoming the weir instead of the dropping breach bottom. Essentially, this mechanism results in a limitation of h0 once the breach bottom drops below the foreshore level. A weak trend may be observed in the comparison between test R3 and the foreshore tests. If the foreshore stability or foreshore level had the largest impact in reducing the specific discharge between the foreshore tests is difficult to determine. Nonetheless, it is likely that for longer test duration and equal initial foreshore level, a clay foreshore is more effective due to its lower erodibility.

Section 3.7 (Comparison with similar breaches):

Section title: Comparison with similar breaches	Section title: Comparison with historic breach data
The observed weir shapes for the sandy and clay foreshores tests (Section 3.4) are also found in historic breaches from the 1953 flood disaster (Watersnoodramp) in the Netherlands (Rijkswaterstaat and KNMI, 1961).	The observed weir shapes for the sandy and clay foreshores tests (Section 3.4) are also found in historic breaches from the 1953 North Sea Flood disaster (Watersnoodramp) in southwestern Netherlands (Rijkswaterstaat and KNMI, 1961).

Figure 12 caption:

Breach cross-sections from the 1953 flood disaster (Watersnoodramp) in the Netherlands […]	Breach cross-sections from the 1953 North Sea Flood disaster (Watersnoodramp) in southwestern Netherlands

Section 4.1 (Evaluation of the experimental setup and test series):

The model dike was 1.5-1.9 m high and the foreshore layer thickness between 0.8 and 1.0 m. With a scale of 1:3-1:4 this translates to respectively dike crest and foreshore levels between 6-8 m and 3-4 m above basin bottom.	The model dike was 1.5-1.9 m high and the foreshore layer thickness between 0.8 and 1.0 m. With a scale of 1:3-1:4 this translates to respectively dike crest and foreshore levels between 6-8 m and 3-4 m above the dike toe (here: basin bottom).
The model dike in this study had no clay (cover) layer unlike a full-scale dike. It was assumed that the clay layer was removed in the breach initiation process, as observed in the 1953 storm flood (Rijkswaterstaat and KNMI, 1961).	The model dike in this study had no clay (cover) layer unlike a full-scale dike. It was assumed that the clay layer was removed in the breach initiation process, as observed in the 1953 North Sea Flood in southwestern Netherlands (Rijkswaterstaat and KNMI, 1961).
To mitigate large water level change rates […] Lastly, basins with a large area relative to breach circumvent the necessity of a pump system, which is both financially and operationally beneficial.	To mitigate large water level change rates […] Lastly, basins with a sufficiently large area circumvent the necessity of a pump system, which is both financially and operationally beneficial.