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Flood-Side Wave Erosion of Earthen Levees: Present State of Knowledge and Assessment of Armoring Necessity

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Abstract: This report is a compilation of facts and information that summarizes the present state of knowledge related to wave attack on the flood side of earthen levees. Particular emphasis was placed on the need for providing flood-side armoring (beyond the protection afforded by grass) for the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS). The report includes: (1) a summary of observations from Hurricane Katrina; (2) an extensive overview of large-scale experiments conducted in Europe, (3) a critical examination of proposed methodologies for predicting wave-induced damage on flood-side grass and bare-clay slopes, (4) an analysis of wave-induced erosion expected to occur on the flood side during hypothetical storms approximating the 100-yr and the 500-year events, (5) and a comprehensive list of conclusions and associated caveats. The erosion estimation methodologies for wave-induced erosion discussed in this report were applied to three sets of hypothetical extreme storm parameters to assess the need for providing wave erosion protection (i.e., armoring) on grass-covered and bare-clay flood-side slopes.

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Executive Summary

This report is a compilation of facts and information that summarizes the present state of knowledge related to wave attack on the flood side of earthen levees. Particular emphasis was placed on the need for providing flood-side armoring (beyond the protection afforded by grass) for the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS). The report includes an analysis of wave-induced erosion expected to occur on the flood side during hypothetical storms approximating the 100-yr and the 500-year events. Erosion was estimated for nine specific reaches of the HSDRRS using the 500-year wave and surge level estimates.

Available documentation of Hurricane Katrina damage gave little evidence of levee failure that could be attributed to wave-induced damage of the flood-side slope. The majority assessment by the technical experts involved in preparing the reports was that earthen levee failures were caused primarily by wave and/or surge overtopping of levees. One report contended that MRGO levees constructed of hydraulically-placed sandy soil failed because of flood-side wave-induced erosion. However, for the levees that were completely destroyed, there was no forensic evidence to verify the claim that wave damage to the flood side of the levee was the sole cause of breaching. Many sections of levee that were constructed of hydraulically-placed fill, which were in proximity to completely failed sections, suffered only minor or no flood-side damage. Hurricane Katrina field evidence suggests that levees constructed of strong cohesive soils can withstand severe hurricane wave loading on the flood-side slope without catastrophic damage, even without any armoring beyond a well-maintained grass covering.

Most of the information about wave attack on levees and on the resiliency and erosion rates for grass covers and bare-clay levee slopes was derived from full-scale laboratory tests conducted in The Netherlands and Germany. These tests provided a number of conclusions based on observations and measurements. In addition, the limited data were used by Dutch researchers to develop several equations and design methods for estimating maximum wave-induced erosion depth for bare clay and grass-covered clay slopes as a function of incident wave height, peak storm duration, and soil conditions.

The erosion estimation methodologies for wave-induced erosion discussed in this report were applied to three sets of hypothetical extreme storm parameters to assess the need for providing wave erosion protection (i.e., armoring) on grass-covered and bare-clay flood-side slopes. Numerous conclusions are given based on the calculations, along with several caveats pointing out short-comings in the analytical methodologies.

The most important conclusions were those directed toward whether or not armoring is needed to protect flood-side levee slopes against wave attack. These conclusions are listed below in their entirety.

- It was concluded with a reasonable degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 1.5 m (4.9 ft), (c) the average wave overtopping discharge is less than 2.8 ft³/s per ft (assumes no floodwalls atop the levee), and (d) peak surge and wave duration is less than 10 hrs.
- It was concluded with less degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 2.44 m (8.0 ft), (c) soil structure¹ has not developed to depths greater than 3.0 ft, (d) the average wave overtopping discharge is less than 2.9 ft³/s per ft (without floodwalls), and (e) peak surge and wave duration is less than 10 hrs..
- It was concluded that the HSDRRS levees can possibly withstand wave conditions greater than $H_{m0} = 2.44$ m (8.0 ft) without flood-side armoring because of the following reasons:
 - The maximum erosion depths for grass-covered slopes were found using equations in which the erosion depth is directly proportional to wave height squared. This proportionality is probably accurate for wave heights less than about 1.5 m (4.9 ft); but for waves greater than 2 m (6.6 ft) the erosion estimates for grass cover layers become unrealistically large compared to similar estimates for bare clay which relate erosion depth to wave height. Furthermore, there are no measurements to validate the wave-height-squared proportionality for high waves.

¹ Soil structure is a slow, but time-dependent process whereby smaller soil particles are removed from the clay, leaving a network of solid particles and voids. A soil with structure is more easily eroded.

- The 10-hr duration of peak storm surge and extreme wave conditions is inordinately long compared to peak surge durations documented for Hurricane Katrina. Peak surges lasting this long are not very likely to occur for hurricanes. Even with this long duration, the estimated erosion for unstructured clay did not threaten the levee crown except where the freeboard was very small and flooding by wave overtopping would be problematic.
 - Erosion of structured soil was the worst case, but soil structure will be slow to occur to significant depths on well-compacted levees that are usually well above water level. However, better understanding of how soil structure develops on HSDRRS levees is needed.
 - Erosion was less when the duration of the peak surge level and wave conditions was similar to the durations recorded for Hurricane Katrina. Durations of peak storm conditions shorter than 10 hrs is a realistic expectation.
 - Surge levels that allow average wave overtopping discharges above 3.0 ft³/s per ft become problematic for landward-side slopes, and this rate of overtopping will cause significant flooding.
 - The vertical distance between levee crown and flood-side slope toe would have to be greater than 20 ft to maintain a suitable freeboard and still have waves break directly on the slope. For many levees of the HSDRRS, larger waves will break on the flood-side berm; and this will decrease the erosive power of waves larger than 8 ft.
- It was concluded that existing levees constructed of hydraulically-placed sandy soils will need to be reconstructed with clay or be armored to prevent damage. A viable alternative may be to provide a thick cover layer of stiff clay over marginal soil similar to European dikes that are intended to withstand storms with longer durations than hurricanes.
 - It was concluded that armoring of the flood-side slope of an earthen levee can only be justified if the landward-side slope is also armored in cases where the earthen levees do not have floodwalls on the crown. In other words, before damage on the flood-side slope could become critical, there is reasonable expectation that the unarmored landward-

side slope will have already sustained severe damage that could lead to potential breaching.

- It was concluded that the present-day design methodologies for estimating levee flood-side wave-induced erosion damage are not sufficient to remove all uncertainty from the above conclusions. Furthermore, there is little likelihood that these methods will be improved and verified for larger wave heights anytime in the near future.

Preface

This technical report describes and summarizes much of the information that is known about the effects of waves breaking on grass-covered and bare clay dike and levees slopes. Included is an overview and assessment of engineering methods available to estimate maximum depth of erosion due to wave action on the flood-side levee slope. The study was conducted by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), Vicksburg, MS, for the U.S. Army Engineer District, New Orleans (MVN), and Task Force Hope (TFH). The purpose of this technical report was to provide TFH with information needed to help in the evaluation of when and where armoring of the flood-side levee slopes might be needed, and to determine if additional engineering analyses are needed. A review draft of this report was submitted to TFH on 31 December 2009. The second draft incorporating comments and suggestions by the Internal Technical Review was completed during the period 1 - 24 March 2010, and the third draft with final comments incorporated was completed on 6 April 2010.

Dean Arnold, Task Force Hope, MVN, was the point of contact for the sponsoring New Orleans District, and he provided study oversight and arranged for independent technical review. Grateful appreciation is expressed to Marieke de Visser (presently at Arcadis, Netherlands), who granted permission to use thirteen original figures that appeared in her Master's thesis from Delft University of Technology, and to Professor Hocine Oumeraci (Technical University of Braunschweig), who granted permission to use two photographs from the Large Wave Flume in Hannover, Germany.

The technical report was written by Dr. Steven A. Hughes, Navigation Division (HN), CHL, during the period September 2009 through December 2009 under the direct supervision of Jackie S. Pettway, Chief, Harbors, Entrances, and Structures Branch, Navigation Division, CHL. Administrative supervision was provided by Dr. William D. Martin, Director, CHL, and Dr. M. Rose Kress, Chief, Navigation Division, CHL.

COL Gary E. Johnston was Commander and Executive Director of ERDC. Dr. Jeffery P. Holland was Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic feet per second per ft	0.0929	cubic meters per second per meter
cubic feet per second per ft	92.90	liters per second per meter
feet	0.3048	meters
inches	0.0254	meters
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
yards	0.9144	meters

1 Purpose

The Independent External Peer Review (IEPR) of the *MVN Design Guidelines* and Task Force Hope (TFH) *Armoring Manual* questioned the lack of design guidance or recommendations related to protecting the flood-side slopes of earthen levees. As a consequence, TFH requested the U.S. Army Engineering Research and Development Center (ERDC) to examine the need for armoring of levee flood-side slopes.

This report is an investigation and compilation of facts and information based on existing peer-reviewed literature, Corps documents, and publications available from private industry and foreign entities. The report summarizes the present state of knowledge related to wave attack on the flood side of earthen levees, and it is intended to address the IEPR criticism. Particular emphasis was placed on the need for providing flood-side protection (beyond the protection afforded by grass) for the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS) during wave attack on the flood side for a storm approximating to the 100-yr event and for a storm similar to the 500-year event.

The following questions have been addressed by this report.

- What is the present state-of-knowledge related to flood-side slope damage and failure?
- What design guidance has been proposed elsewhere, and what is the range of design guidance applicability?
- Under what conditions might flood-side armoring beyond grass be necessary?
- What are the qualitative uncertainties associated with the methodologies discussed in this report?
- How are the HSDRRS levees different than the levees and dikes for which the available design guidance is intended?

No consideration is given in this report to potential damage that might occur because of strong lateral water flows on the flood-side slope. There is a possibility of high, slope-parallel velocities during storm events. Higher velocities might occur where a channel or some other type of inlet/outlet directs storm surge flow along the face of the levee. If such a lateral flow were to occur simultaneously with breaking waves, there is a possibility that erosion potential might be increased. The current could also alter the breaking wave characteristics.

Unfortunately, all of the full-scale testing that examined breaking wave erosion potential on levees and dikes is only for waves approaching the levee head-on. The effect of concurrent lateral flows has never considered in the testing.

2 Hurricane Katrina Observations

Earthen Levee Damage

The performance of the New Orleans system of earthen levees, floodwalls, floodgates, and pump stations has been thoroughly documented by a number of published reports. Most predominant are the following references:

- Seed et al. (2005), “Preliminary Report on the Performance of the New Orleans Levee System in Hurricane Katrina on August 28, 2005”
- Interagency Performance Evaluation Task Force (IPET) (2006), “IPET Report 2: Performance evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Report 2”
- Seed et al. (2006), “Investigation of the Performance of the New Orleans Flood Protection Systems - Vol I”
- External Review Panel (ERP) (2007), “The New Orleans Hurricane Protection System: What Went Wrong and Why”
- Interagency Performance Evaluation Task Force (IPET) (2007c), “Final Report of the Interagency Performance Evaluation Task Force: Volume V - The Performance - Levees and Floodwalls.”
- Interagency Performance Evaluation Task Force (IPET) (2009), “Final Report of the Interagency Performance Evaluation Task Force: Volume I - Executive Summary and Overview.”

These documents were examined for any descriptions or forensic evidence of earthen levee flood-side damage that could be attributed to wave attack during Hurricane Katrina.

In their preliminary report Seed et al. (2005) made no mention of earthen levee damage due to erosion of the flood-side slope. They did acknowledge that most failures were a result of the levees being overtopped by storm surge, waves, or a combination of both.

Most of the levee and floodwall failures were caused by overtopping, as the storm surge rose over the tops of the levees and/or their floodwalls and produced erosion that subsequently led to failures and breaches. (Seed et al., 2005)

Seed et al. (2005) noted the good performance of the Lake Pontchartrain levees of the Orleans East Bank Protected Area where field observations suggested that many of the levees along this reach had little to no overtopping. The relatively successful performance of the Lake Pontchartrain levees was credited to good construction, good cohesive soil, and rip-rap protection on the lake-side slopes.

These levees were well-constructed earthen embankments, constructed using apparently cohesive soils, and they generally had good erosion protection on their outboard faces (generally consisting of large stone rip-rap.) These lakefront levees performed well, and despite some evidence of minor wave overtopping at a few locations, these lakefront levees safely withstood the storm with only minor evidence of any erosion at the crests and back faces evident after the storm had passed. (Seed et al. 2005)

Seed et al.'s (2005) statement implies that riprap protection on the Lake Pontchartrain levees was wide-spread and partially responsible for the good performance of the levees during Hurricane Katrina. According to the final version of the IPET Volume III report describing the design of the Hurricane Protection System levees (IPET 2007a), the following comment was made multiple times in reference to design of the levee reaches fronting Lake Pontchartrain.

Due to the short duration of hurricane floods, the resistant nature of the clayey soils, and the limited conditions for wave generation, no erosion protection was considered. (IPET 2007a)

However, armoring was provided at transitions between earthen levee and other structures such as canal entrances. Portions of the lake foreshore have also been armored with riprap at elevations where waves can routinely impact the shoreline, and a seawall extends from the 17th Street Canal to the University. But generally, there is no flood-side armoring at higher elevations (especially near the levee crown) along most of the lake-side levees. This conflicts with Seed et al.'s (2005) implication that the Lake Pontchartrain levees were successful, in part, because of rip-rap on the flood-side slope.

The IPET Report 2 (IPET 2006) did not cite any specific instances where significant levee damage and breaching was caused primarily by wave

attack on the flood side of an earthen levee. (However, at transitions between earthen levees and vertical structures there was potential for wave damage.) The IPET also noted that earthen levee performance at above-design-level conditions varied greatly, and the report suggested logical reasons for the variation, including the possibility of erosion caused by wave attack on the flood-side slope.

The performance of levees varied significantly throughout the New Orleans area. In some areas the levees performed well in spite of the fact that they were overtopped. While in other areas the levees were completely washed away after being overtopped. Several possible factors could explain the differences in performance. One would be the type of material that was used to construct the levees. Another could be the direct wave action on the levees. The degree of dependence of overtopping versus wave action on the scour and erosion of the levees is yet to be determined and will be addressed in the high resolution analysis [of] the hydrodynamic environment experienced by the structures in the confined canals and channels. This task will examine the type of material used in construction of the levee versus the surge height and wave height to investigate their interdependence. (IPET 2006)

In a comprehensive follow-on report, Seed et al. (2006) described two earthen levee failure modes based on erosion of the flood-side levee soil. The first was loss of erodible soil by high-velocity lateral flows such as occurs at river or channel banks. They noted that embankments that are continuously exposed to high flows are often armored to reduce soil loss.

Levees that are exposed to chronic water flow, such as river levees, are generally designed and constructed with armoring or erosion protection to minimize scour-induced surface erosion. In general, well-compacted levees constructed of high-plasticity clays are much more resistant to surface erosion than uncompacted cohesionless soils (e.g. clean sands) and silty sands. Surface protection such as rip-rap, concrete pads, soil-cement reinforcement, and select vegetation coverings are typical methods used to protect levee faces from surface erosion. (Seed et al. 2006)

The second described failure mode was direct wave impact on the flood-side slope.

Wave-induced erosion consists of run-up (sloshing up and down of water as a result of staggered wave arrival) and mini-jetting when the crest of the waves breaks on the levee face. Levees that are anticipated to be impacted by waves are generally designed with armoring to prevent damage from wave impacts. (Seed et al. 2006)

No direct citations were provided by Seed et al. (2006) to support the statement that levees subject to wave attack “...are generally designed with armoring to prevent damage from wave impacts.” Present erosion prediction and design guidance for earthen levees subjected to flood-side wave attack are examined in a later section of this report.

Seed et al. (2006) contended that much of the earthen levee failures witnessed along the Mississippi River Gulf Outlet (MRGO) and the Gulf Intercoastal Waterway (GIWW) was the result of direct wave action on the flood-side levee slope at surge levels lower than the levee crown elevations. Referring to the southeast corner of New Orleans East, they stated:

High water marks, as determined by IPET (2006) using numerical simulations, suggest that water levels at this location reached a maximum Elevation of approximately +16 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +19 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team. (Seed et al. 2006).

Subsequent work completed by IPET indicated that pre-Katrina earthen levee crown elevations along the New Orleans East Back Levee (NOE BL) averaged between +16 and +17 ft NAVD88-2004.65 (IPET 2007c, Appendix 18).

The average pre-storm elevation of the NOE BL was 16.0 ft (NVGD88, 2004.65) to the east of the pump station [15] and 17.0 ft (NVGD88, 2004.65) to the west of the pump station. (IPET 2007c)

At these crown elevations, the New Orleans East Back Levees would have been heavily overtopped before the water reached the peak storm surge elevation .

The levees adjacent to Lake Borgne were exposed to large wind-generated waves propagating from deeper water, and these levees were probably subjected to the most critical hydrodynamic loading. The damage was so severe that, in some case, the levees were almost completely annihilated. Seed et al. (2006) disagreed with the IPET conclusion that only overtopping-induced erosion was responsible for the loss.

The IPET studies have ascribed this massive erosion principally to overtopping, but it is the view of this investigation that considerable erosion also occurred due to wave action prior to full overtopping, and that through-levee seepage and underseepage may also have played a role at some locations. (Seed et al. 2006)

Seed et al. (2006) included a table showing those levee reaches they believed suffered significant flood-side erosion that led to breaching. They also speculated that forces associated with the breaking waves impacting the MRGO levee may have been sufficient to induce liquefaction in the relatively weak foundation materials.

The levees that suffered catastrophic damage were constructed of fill material (sometimes including sand and sand/shell mixtures) obtained from construction of the MRGO and GIWW, and the erodibility properties of these soils varied considerably. The levees flood sides were protected only by grass. Some levee sections along the same reaches, also constructed of hydraulically-placed fill, suffered minor or no flood-side damage.

Seed et al. (2006) speculated that some levee breaches may have been caused by a notching of the crown caused by flood-side erosion of weak soils by waves before the surge elevation reached the crown height and before wave overtopping occurred. However, forensic evidence does not provide definitive proof of this hypothesis because once a breach was formed, whether by wave attack or by overtopping, the most erodible levees were rapidly destroyed leaving no evidence of the initial breaching mechanism.

Seed et al. (2006) acknowledged that adjacent unprotected levees constructed of better quality clay suffered little damage under similar hydraulic loading despite that fact that erosion protection was not installed.

Interestingly, adjacent levee sections along these same frontages, although also overtopped, performed well; suffering relatively minor erosion and continuing to provide protection as the storm surge subsided after the period of overtopping during the relatively short-lived peak of the storm surge. These better-performing sections were levees comprised of compacted, clayey soils; soils known to have far higher intrinsic resistance to erosion. (Seed et al. 2006)

This finding, based on field evidence, suggests that levees constructed of strong cohesive soils can withstand severe wave loading on the flood-side slope without catastrophic damage, even without any armoring beyond a well-maintained grass covering.

The External Review Panel of the American Society of Civil Engineers summarized the Hurricane Katrina disaster in a report that provided a chronological description of events, discussed some of the decisions leading to the levee system design and maintenance, and provided a list of recommendations for moving forward (ERP 2007). The ERP listed two fundamental causes for levee failure: (1) collapse of concrete flood walls, and (2) *overtopping, where water poured over the tops of the levees and floodwalls and eroded the structures away*. Nowhere in the ERP report was there any mention of wave erosion of earthen levees from the seaward side being a failure mechanism responsible for levee breaching.

One of the four major recommendations made by the ERP to correct deficiencies in the HSDRRS was the following.

Make the levees functional even if overtopped. During Hurricane Katrina, water rushing over the levees severely damaged and compromised their integrity. Overtopping of levees due to hurricanes is inevitable. To prevent damage, the levees need to be armored by resurfacing them with protective non-erodible materials. (ERP 2007)

The final versions of the IPET reports continued to discount any earthen levee failures due primarily to wave action on the flood side.

No levee breaches occurred without overtopping. The degree of erosion and breaching of overtopped levees was directly related to the character of the in-place levee materials and the severity of the surge and wave action. Hydraulically filled levees with higher silt and sand

content in the embankment material that were subjected to high overtopping surge and wave action suffered the most severe damage. Rolled clay levees performed well, even when overtopped. (IPET 2007a).

There was no evidence of systemic breaching caused by erosion on face or water sides of the levees exposed to surge and wave action. The water velocities on the face side were only one-third of those experienced at the crest and back or protected side of the levees. The levees largely performed as designed, withstanding the surge and waves until overtopping, at which time they became highly vulnerable to erosion and breaching, especially those constructed by hydraulic fill. (IPET 2009)

Example Photographs of Earthen Levee Erosion

Figure 1 illustrates erosion damage caused by direct wave impact on the flood-side slope when the water level was less than the levee crown elevation. This photograph is one of the more extreme examples of flood-side erosion on the levees adjacent to the Mississippi River Gulf Outlet (MRGO) facing Lake Borgne. The more typical evidence of wave damage was loss of grass and superficial erosion. The erosion in Figure 1 is seen to be non-uniform along the levee, and each of the erosion zones has a characteristic almost vertical face nearest the crown.

The more common damage mechanism is soil erosion caused by either (1) intermittent wave overtopping when the still water level is lower than the levee crown (positive freeboard), (2) steady storm surge overflow when the still water elevation is above the levee crown elevation (negative freeboard), or (3) a combination of both. Water that flows down the landward-side slope forms a “headcut,” and erosion progresses up the slope (see Figure 2). In time, the crown may be eroded, and breaching could occur.

Hurricane Katrina Storm Surge Hydrographs

In simplest terms the magnitude of earthen levee erosion on the flood side due to wave action is a function of the erosion power of the waves and the duration over which the waves act on the levee. For steady overflow conditions the power of the flow is typically represented by either the flow velocity or the shear stress exerted by the flowing water on the slope.

Accepted relationships for tolerable steady overtopping flow on grass slopes show a strong dependence on flow duration, particularly over the first several hours (Hewlett et al. 1987). Therefore, it is reasonable to conclude that duration is also a critical parameter for the extent of wave erosion that might occur on the flood side of earthen levees.

During tropical storms (i.e., hurricanes) the storm surge elevation varies in time as the hurricane moves into and out of a region. Consequently, the maximum (or peak) storm surge elevation is experienced at a particular location for a short time period relative to the total storm duration. The duration of peak storm surge elevation is mostly a function of the forward speed of the hurricane. Assuming that Hurricane Katrina is representative of Gulf coast hurricanes, an indication of peak storm surge duration can be obtained by examining water level hydrographs developed for Hurricane Katrina.



Figure 1. Wave damage on flood side of MRGO levee facing Lake Borgne.



Figure 2. Wave overtopping or surge overflow damage on landward side of MRGO levee.

Figures 3 - 6 are Hurricane Katrina hydrographs taken from the final IPET Report, Volume IV (IPET 2007b). Some of the hydrographs came from measured gauge data, and others were reconstructed based on measured data, supplemented with other water level observations such as sequential photographs of water levels at different times.

Water level fluctuations were measured with instrumentation during the build-up stage of the storm at a number of sites throughout the study region; however, few instruments operated throughout the storm. Most of them failed prior to the peak. Consequently, there are little measured data that capture peak conditions. In a few cases, photographs and other visual observations were utilized to provide information about the temporal variation of water level to supplement the recorded hydrographs. (IPET 2007b)

Figure 3 is a water level hydrograph measured at Southwest Pass where the Mississippi River joins the Gulf of Mexico. The peak water level was about 7.5 ft (2.3 m), but this peak lasted just a brief period of time. The duration over which the water level exceeded 6 ft (1.8 m) was approximately 5 or 6 hrs. Thus, the water was at or above 80 percent of the peak surge for a total of about 6 hrs at most.

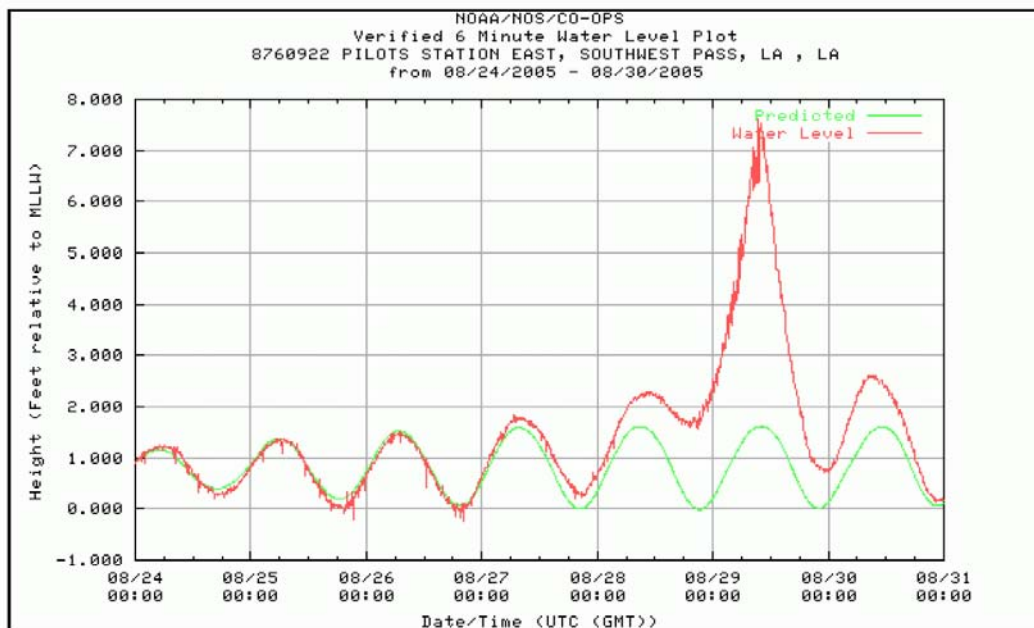


Figure 3. Measured storm surge hydrograph at Southwest Pass.

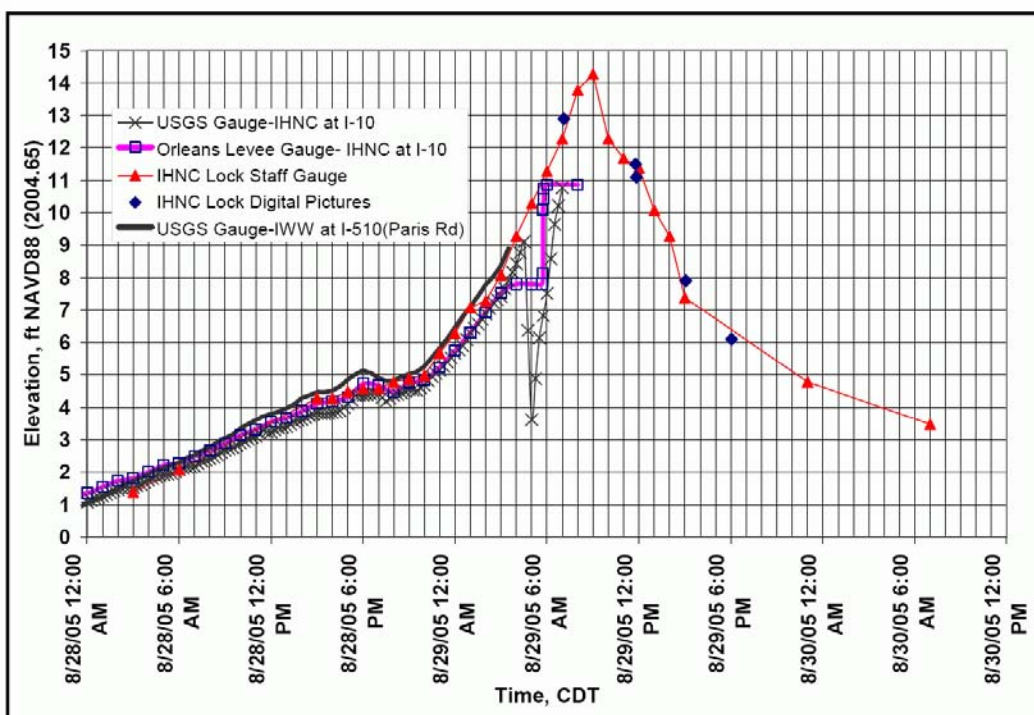


Figure 4. Measured storm surge hydrograph at IHNC lock.

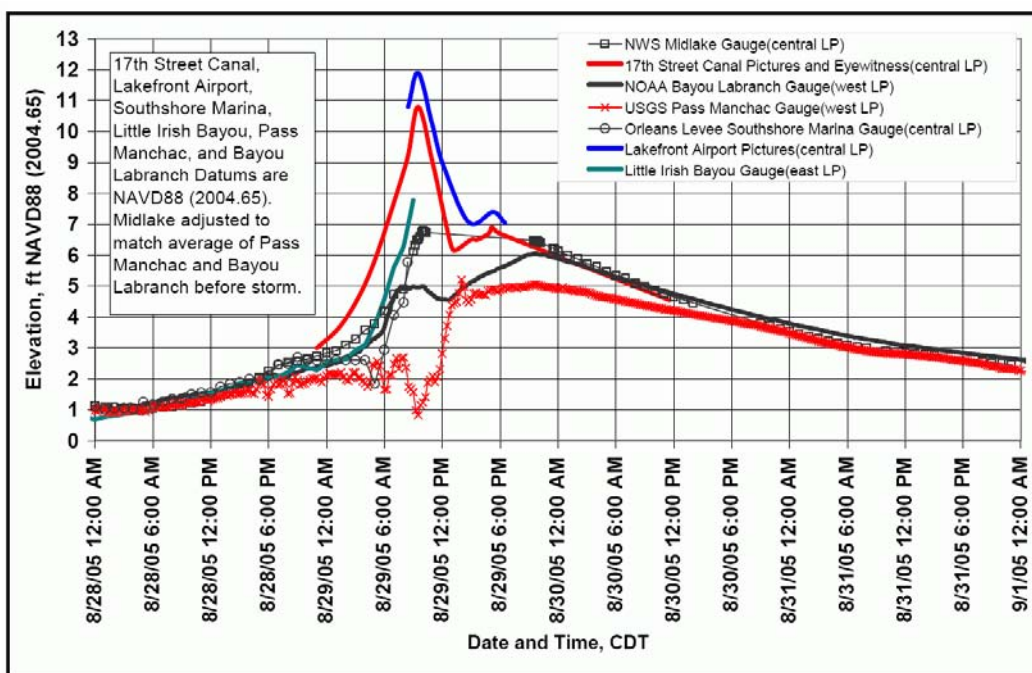


Figure 5. Measured and reconstructed storm surge hydrographs for Lake Pontchartrain.

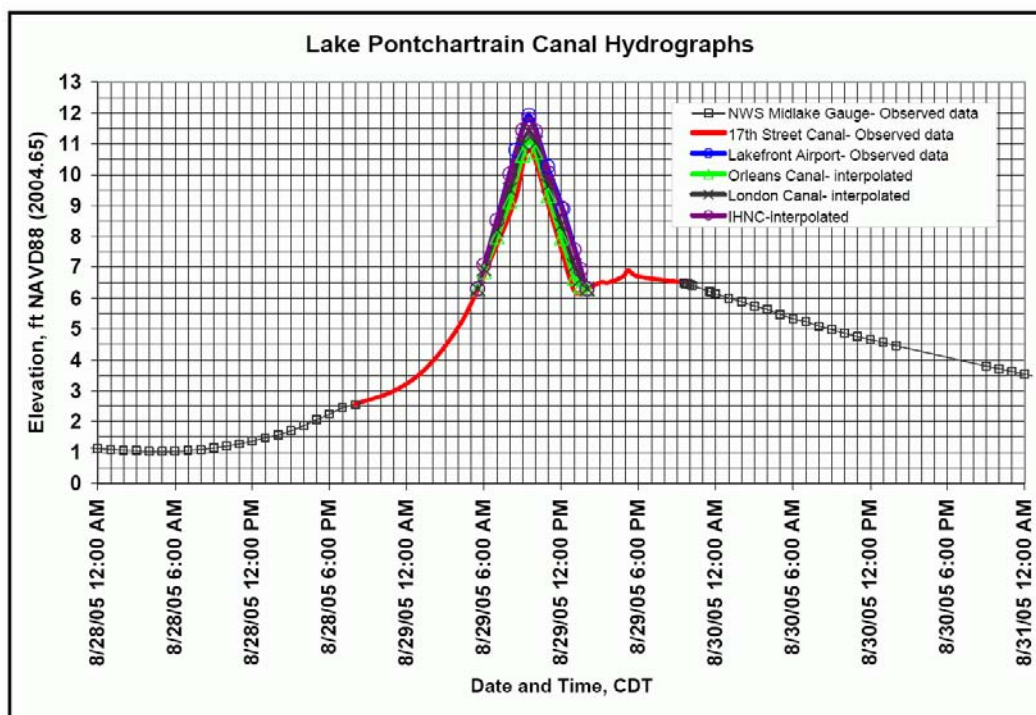


Figure 6. Reconstructed storm surge hydrographs at Lake Pontchartrain Canals.

Figure 4 is the measured storm surge hydrograph recorded by a staff gauge located at the Inner Harbor Navigation Canal (IHNC) lock during Hurricane Katrina. The peak storm surge elevation was slightly over 14 ft (4.3 m), but the peak lasted a short time. The hydrograph indicates that the storm surge exceeded 11 ft (3.4 m) for about 6 hours, and that it exceeded 12 ft (3.7 m) for a total of about 3 hrs. The 11-ft and 12-ft surge elevations are approximately 79 percent and 86 percent of the peak surge, respectively.

Figures 5 and 6 show storm surge hydrographs for Lake Pontchartrain that were reconstructed based on observations at the various locations. Therefore, the curves reflect the interpretation of team doing the reconstructions. Nevertheless, the hydrographs indicate that the duration over which storm surge level for Lake Pontchartrain exceeded 80 percent of the peak surge elevation was between 3 and 4 hours during Hurricane Katrina.

Based on Figures 3 - 6 the storm surge for Hurricane Katrina exceeded 80 percent of the peak surge elevation between 3 and 6 hours duration. Similar analyses could be performed for other Gulf and Atlantic hurricanes where data exist.

3 Grass Cover Layers on European Dikes

Grass coverings are used extensively in Europe as protection on both river and sea dikes, and European researchers have produced most of the available technical information about the performance of grass covers when subjected to direct wave attack. A key resource document summarizing the European research is the technical report, “Erosion Resistance of Grassland as Dike Covering (TAW 1997). Much of the information summarized in this section was taken from this Dutch technical report.

Typical Dutch Storm Parameters Compared to Those of the HSDRRS

Before delving into the details of grass cover layers and dike construction in Europe (primarily The Netherlands), it is beneficial to examine the typical design storm parameters for Dutch dikes. The Netherlands borders on the North Sea, and the design storms are extra-tropical events in which the duration of high surge levels can be considerably longer than the relatively short peak storm surge durations associated with hurricanes. Also, the maximum surge levels of extra-tropical storms are usually lower than hurricanes storm surges, and certainly lower than surges generated by tropical storms the size of Hurricane Katrina. Of course, this is a generalization that does not include the effects of coastal bathymetry on surge elevation.

Van der Meer (2007) summarized the wave conditions that were targeted for simulation in the Dutch Wave Overtopping Simulator (van der Meer, et al. 2006, 2008). The purpose of the Overtopping Simulator is to test the performance of specific dikes subjected to various levels of wave overtopping. Van der Meer described wave conditions representative of the 5-year safety assessment at four locations in the Dutch dike system. The significant wave heights varied between $H_{m0} = 1.75$ m and 3.2 m (5.7 ft and 10.5 ft), and the peak spectral wave period varied between $T_p = 4.4$ s and 8.4 s. The average values of $H_{m0} = 2.0$ m (6.6 ft) and $T_p = 5.7$ s were selected for in-situ testing of dikes using the Overtopping Simulator testing.

For comparison, the following are wave parameters for several reaches of the HSDRRS based on the 1-in-500-year design event. These wave conditions are based on the mean of the 0.2-percent probability of occurrence.

- New Orleans East Back Levee and St. Bernard: $H_{m0} = 2.1 - 2.7$ m (6.8 - 8.8 ft) and $T_p = 7.0 - 8.5$ s.
- Jefferson Lakefront: $H_{m0} = 1.5$ m (5.0 ft) and $T_p = 9.0$ s.
- St. Charles Parish East Bank: $H_{m0} = 0.8 - 0.9$ m (2.6 - 3.0 ft) and $T_p = 4.1 - 5.6$ s.

For much of the HSDRRS storm waves are depth-limited, meaning any waves larger than the depth-limited wave will break before reaching the levee. This results in less runup and less erosive force on the flood-side slope. In comparison to the Dutch 5-year safety assessment, the HSDRRS 500-year design significant waves heights are similar; but the peak periods expected for the HSDSSR are longer.

Grass-Only Cover Layers in Europe

In The Netherlands many of the sea dikes (dikes exposed to significant wave attack) have a relatively steep seaward slope (typically 1:4) with an armored revetment below the design storm surge level and a grass cover above the surge level. The armored revetment protects the dike soil from direct wave attack during storms, and the grass layer resists the forces created by wave runup and rundown. An example of a sea dike is shown in Figure 7.

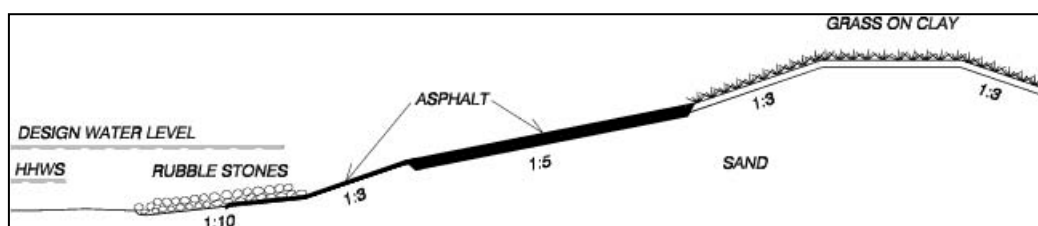


Figure 7. Typical sea dike cross section (from Coastal Engineering Manual).

Grass is used as dike cover extensively for multiple reasons including cost, harvesting of hay, grazing for livestock, and because it is more visually pleasing than hard coverings. However, dikes in The Netherlands play a water defensive role above all else; other functions can only be considered if the required erosion resistance of the covering is sufficiently durable. Safety against water is the primary goal, for river, sea and lake dikes (TAW 1997).

In addition to partially armored sea dikes, there are European examples of “green dikes” that are protected only by a high-quality grass layer. Green dikes can be found in Denmark and northern Germany (de Visser 2007). De Visser noted that typical sea dikes protected only by grass have a milder

flood-side slope (1:6 to 1:8), and they usually have an extensive berm or foreshore up to 400 m (1,300 ft) in width fronting the dike. The transition between the berm and dike slope is smooth, and the toe of the flood-side slope is usually dry except during storm events (on average 20 per year). The presence of the foreshore or berm reduces (through wave breaking) the wave height that will impact directly on the dike flood-side slope.

In the absence of a foreshore or berm, rubble protection is provided on the dike flood-side slope up to the elevation of mean high water (a tidal datum), but not to elevations that correspond to higher storm water levels. The rubble is intended to prevent erosion due to persistent wave action at lower, more frequent, water levels. De Visser (2007) noted that green dike construction is allowed if the wave height [assumed to be the significant wave height, H_s] was below 1.6 m (5.3 ft). With a milder seaward-side slope, green dikes can have a lower crown elevation because the milder slope reduces wave runup; and with a wide, low footprint they are more suited for locations where the foundation soils are weaker (de Visser 2007). Grass-only dikes are also constructed along rivers with steeper slopes (1:3) where the wind-generated waves are small, and the design wave is usually ship-generated.

European Grass Dike Cross Section

The construction cross section of a typical European dike differs from that of the post-Katrina levees, and this difference should be kept in mind when evaluating European results for applicability to the New Orleans HSDRRS levees. Rather than constructing with a solid clay cross section, the Europeans dikes often have a sand core making up the bulk of the cross section. On the flood side, this highly erodible core is protected (typically) by a 1-m-thick (3.3-ft-thick) layer of clay as shown in Figure 8.

The clay layer should be composed of stiffer, more erosion-resistant, clay subsoil, covered by a top soil consisting of clay with a higher sand percentage (maximum 50%) that promotes fast grass growth and a denser root system. The achievement of a close grass mat and thick root mass penetration through the sod layer depends on the type of management used and, less importantly, on the properties of the soil (TAW 1997).

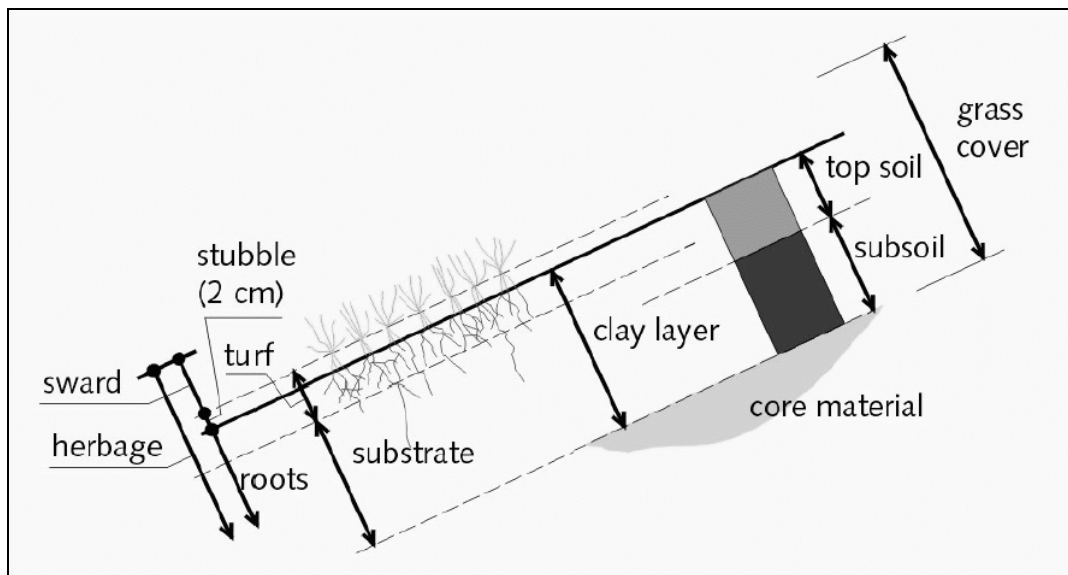


Figure 8. Structure of a typical dike cover layer in The Netherlands (from TAW 1997; original source Rijkswaterstaat).

The topsoil layer thickness is typically 15 to 35 cm (6 to 14 inches) thick. The very top layer (1 to 35 mm thick) of an established grass cover consists of loose soil and decayed plant material that is easily eroded. Beneath the top layer is a layer of loose turf characterized by dense root growth. This layer varies in thickness between 5 and 50 mm (0.2 and 2.0 inch), and it can be slowly eroded by wave action. Farther down in the topsoil layer, the soil is more compacted and the root density is decreased. The thickness of this third layer varies between 5 and 15 cm (2 and 6 inches), and erosion of this layer occurs slowly after lengthy wave action. It was pointed out in TAW (1997) that the above-described vertical profile can vary considerably over short horizontal distances on the dike, giving rise to spatially inhomogeneous grass cover. In other words the progression of erosion will never be uniform, and bare spots will develop piecemeal during an erosion event.

Early Dutch laboratory experiments of water run-off revealed that grass cover layers can have high erosion resistance, and the structure of the root system has greater importance than the thickness of the grass stems and blades above the ground. Part of the reason grass layers are effective in combating soil erosion was given by de Visser (2007).

The roots are responsible for the development of a soil structure of aggregates and cracks in the soil, but they also support the development of cementing substances. Chemical processes in the vicinity of the roots develop into cementing substances which stick the

soil particles so to stay together. This results in an elastic network which provides a strong and flexible layer which can deform without cracking. The erosion resistance of a grass cover is to a large extent based on this. Because of the flexibility the cover is able to resist wave impacts and the root system prevents washing away of the soil particles. The development of a good quality grass cover with a well developed root system takes several seasons, on average 4 years.
(de Visser 2007)

On levees where grass is planted in a uniform, highly erosion-resistant clay with low sand content, the root system might not develop as thickly as it does in sandier clays. Consequently, the grass layer flexibility may not be as effective in absorbing wave impacts, and a grass cover layer with a low root density has less resistance to erosion.

Dike Clay Specification in The Netherlands

Figure 9 shows clay parameters for three categories of clay specified by The Netherlands. The lower portion of the clay layer uses the category 1 (erosion-resistant) clay. The upper portion of the clay layer can use less erosion resistant clay such as the category 2 (moderately erosion-resistant) with greater sand content to promote grass root penetration. Local clay can be placed as the top soil layer if it has appropriate soil properties, otherwise the clay must be brought in from an approved remote site. In addition to providing good conditions for establishing a healthy root system, the top soil layer also helps to decrease loss of moisture from the underlying, stiffer clay layer so the erosion-resistant clay maintains its strength.

Requirement	Erosion-resistance category		
	1 Erosion-resistant	2 Moderately erosion-resistant	3 Little erosion-resistance
Flow limit (w_l)	> 45%	< 45%	< $0.73 \cdot (w_l - 20)\%$
Plasticity index (I_p)	and > $0.73 \cdot (w_l - 20)$	and > 18	and < 18
Sand content	and < 40%	and < 40%	and > 40%
Organic material content	< 5%	< 5%	< 5%
Salt content	< 4g/l	< 4g/l	< 4g/l
Chalk content	< 25%	< 25%	< 25%
Water content for working			
Top layer	$I_c \geq 0.75$	$I_c \geq 0.75$	$I_c \geq 0.75$
Core	$I_c \geq 0.60$	$I_c \geq 0.60$	$I_c \geq 0.60$
Extreme colouring from excavation or drying	Not allowed	Not allowed	Not allowed
Strong smell	Not allowed	Not allowed	Not allowed

Figure 9. Dutch classification of clays for use in dikes (from de Visser; originally from TAW 1996).

Clay specifications for the New Orleans HSDRRS have similar characteristics to the Dutch category 1 and 2 erosion-resistant clays. Clay used in the HSDRRS may be either *lean clay* (CL) that is inorganic clay of low to medium plasticity with a liquid limit less than 50 percent, or *fat clay* (CH) that is inorganic clay of high plasticity with a liquid limit greater than 50 percent. Both clay types must have a plasticity index at or above 10. The percentage of sand content must be less than 35 percent by weight, and the soil must be blended to meet the ASTM D 2487 definitions of CL and CH clays (e.g., no pockets of concentrated sand). Organic content of the clay must be less than 9 percent by weight. A summary of the HSDRRS clay specifications is shown in Table 1.

Table 1. Clay specification for HSDRRS levees.

Property	Lean Clay (CL)	Fat Clay (CF)
Liquid Limit	Less than 50%	Greater than 50%
Plasticity	Low (greater than 10%)	High (greater than 10%)
Sand Content	Less than 35% by weight	Less than 35% by weight
Organic Content	Less than 9 % by weight	Less than 9% by weight

The Dutch experience with existing grass mats demonstrating good erosion resistance, indicates that (moderately) erosion-resistant clay (categories 1 or 2 in Figure 9) is sufficient for the top soil.

A thick network of roots does not develop any faster or better in a more sandy clay, such as that classified as category 3. The erosion resistance of the sod is better, with good management, than that of good erosion-resistant clay. (TAW 1997)

4 Flood-Side Erosion Resistance of Grass

Most of the exciting information related to erosion resistance of grass cover layers subjected to wave attack comes from three full-scale tests conducted in The Netherlands and one test conducted in Germany. These tests used actual sod containing mature grass that was harvested and transported to the testing facilities. Brief summaries of the full-scale tests documented in the literature are given in the following sub-sections. More detailed information is given in de Visser (2007) and TAW (1997). Following the summaries are the conclusions and design rules-of-thumb that have been suggested based on test results.

Delta Flume 1983

De Visser (2007) summarized the experiments originally documented in Dutch by Burger (1984). Experiments were performed in the large Delta Flume of Delft Hydraulics (now Deltares) in 1983. The dike had a flood-side slope of 1:8, and the slope was protected in the vicinity of the wave attack zone by a 1-m-thick (3.3-ft-thick) erosion-resistant clay layer and a 0.5-m-thick (1.6-ft-thick) sod layer. The erosion-resistant clay, compacted in thin layers, consisted of a high amount of small particles, but the consistency and density were low. De Visser (2007) stated that this clay was beneath the present standards for category 1 clay. The upper sod layer was described as very sandy (average 45% sand), and this allowed the grass root system to develop well. The experiments were also described by Seijffert and Philipse (1990), and they stated that the sod was about 10 years old when it was harvested from a sea dike. Therefore, the grass root system was mature.

The irregular waves used in the experiment broke on the grass slope as plunging breakers. However, the plunging jet often impacted a layer of water that remained on the gentle slope as rundown from the previous wave. Therefore, the impact loading on the grass cover was often less severe than would occur with plunging breakers on steeper slopes.

De Visser (2007) described three irregular wave experiments that were conducted at different still water elevations to facilitate waves attacking a portion of the grass cover that had not been affected by earlier tests. The first experiment was conducted at a still water elevation of +2.8 m (9 ft)

above the flume horizontal bottom. Waves having significant wave height of $H_s = 1.03$ m (3.3 ft) and peak spectral period of $T_p = 5.2$ s were run for a total of 18 hours. The upper layer of grass and sandy clay was eroded in the vicinity of the still water line, but the underlying erosion-resistant clay was left intact.

The second experiment used a time-varying suite of wave heights, wave periods, and water levels meant to simulate an actual design storm event in Friesland. Water levels varied between 2.4 m and 5.5 m (7.9 ft and 18.0 ft), significant wave heights varied between 0.65 m and 1.85 m (2.1 ft and 6.1 ft), and the peak wave period varied between 5.0 and 5.9 sec. The experiment continued for 29 hours. Note that wave heights could have been depth limited during a portion of the simulated hydrograph. Observed maximum erosion was between 1/2 cm and 1 cm (0.2 inch and 0.4 inch).

The clay erosion developed in the zone of long lasting moderate waves as well in the zone of heavy waves, occurring during a short period. In both cases the erosion was developed in the zone of 0.5 - 1.0 m under the water level. This clay erosion occurred quite quickly and the roots of the grass cover were still present afterwards. The roots formed a kind of felt layer which prevent the clay from washing away. (de Visser 2007)

The third experiment was conducted at a still water level of +5.0 m (+16.4 ft). The larger irregular waves had $H_s = 1.57$ m (5.2 ft) and $T_p = 5.26$ s. Before the test, holes representing damage or animal burrows were dug into the levee surface at elevations 0.5 m and 1.0 m (1.6 ft and 3.3 ft) below the still water line. The initial holes measured 0.5 m by 0.2 m (2 ft by 0.7 ft), and they had a depth of 7 cm (2.8 inch). Damage developed in the holes just below the still water level after about 5.5 hours. The grass around the holes was undermined by erosion of the sandy clay. Without support, the suspended grass and root system was torn away by the breaking waves. The holes in the grass cover expanded in area and depth over time until the experiment was terminated after a total duration of 8 hours. Erosion profiles are shown on Figure 10. Note that the majority of erosion developed at elevations lower than the still water level for the experiments. This aspect is discussed in the next subsection.

Whereas the upper layer of grass sod was eroded, the underlying erosion-resistant clay was not eroded. Two other holes were placed in the runup zone, but these holes did not suffer any additional damage.

An important aspect to remember about this test is that the still water level was held constant for the entire experiment. This would represent a storm event with the surge remaining at the peak level for an extended time period which is seldom the case for hurricane events (see Section 2).

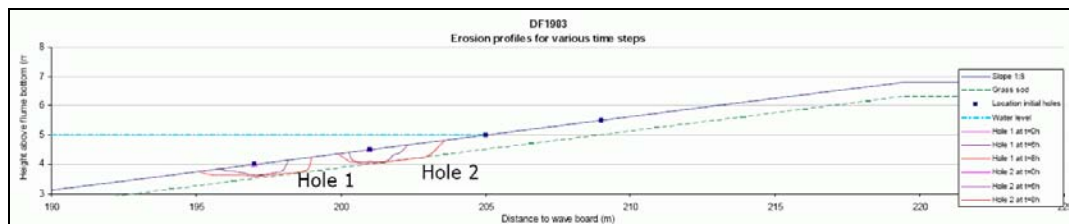


Figure 10. Erosion profiles from the Delft 1983 experiments (from de Visser 2007).

Delta Flume 1992

This test series was described by Smith, et al. (1994), and Verheij and Meijer (1994 in Dutch), and was summarized by de Visser (2007). The flood-side of the dike was constructed in the large Delta Flume with a slope of 1:4, approximately the same as flood-side slopes used in the HSDRRS design. The clay and grass cover layer of the model dike was constructed from large blocks of actual grass cover and underlying clay layer that were extracted from the existing Friesland Wadden sea dike. Each block measured 2.5 m by 2.5 m (8.2 ft by 8.2 ft) and was 1 m (3.3 ft) thick. Thus, the block thickness included the grass and the grass root system in a sandier top soil, and the more erosion-resistant clay below. The sod pieces were placed in the middle 2.5-m (8.2 ft) width of the flume with concrete slopes on both sides to complete the 5-m (16.4 ft) flume width. There was an asphalt covered surface from the toe of the slope up to the +2 m (+6.6 ft) elevation. The tested grass mat was judged to have poor erosion resistance because it had been moderately fertilized and relatively strongly grazed. However, a detailed inspection revealed the vegetation was in good condition, and the grass cover was dense (Smith et al. 1994).

The first five tests used relatively small regular and irregular waves to measure flow and pressure parameters without damaging the grass cover. The sixth test was conducted with the water level at +4.8 m (15.8 ft) using irregular waves having $H_s = 1.35$ m (4.4 ft) and $T_p = 4.7$ s. Visual erosion

was noted after 9 hours at the region of maximum wave impact 1 m (3.3 ft) below the still water level. The initial wave-eroded hole with diameter of 0.75 m (2.5 ft) and depth of 12 cm (5 inch) grew over the next two hours to a diameter of 1.0 m (3.3 ft) with a depth of 15 cm (6 inch). This hole was then repaired so no further damage could occur, and the test continued on for a total duration of 17 hours. By this time a second hole had appeared at a location 0.5 m (1.6 ft) below the still water line. This second hole had a diameter of 0.8 m (2.6 ft) and a depth of 11 cm (4 inch).

The seventh experiment was conducted with smaller waves at a lower water elevation where the grass cover was undamaged by the previous experiments. The still water elevation was 3.5 m (11.5 ft), and irregular waves had $H_s = 0.75$ m (2.5 ft) and $T_p = 3.4$ s. The purpose of this test was to determine the influence of wave height on the erosion rate. The test was terminated after 20 hours with no serious erosion problems noted on the slope.

From these experiments, Smith et al. (1994) observed that maximum erosion occurred in the wave impact zone around the still water level. Because this is a region where slope-parallel flow velocities (up and down the slope) are not large, they concluded that wave impact is an important factor along with any flow-induced shear stress. Based on these results, Smith et al. (1994) defined the three erosion zones shown in Figure 11.

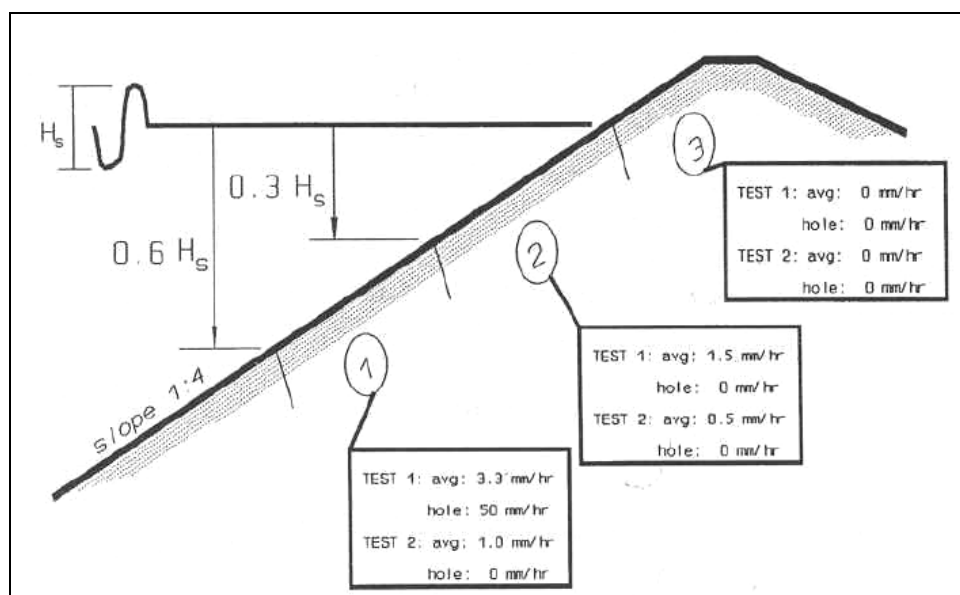


Figure 11. Erosion zones for wave-induced damage on a grass cover layer (Smith et al. 1994).

- Zone 1 is defined at a depth between $0.3 H_s$ and $0.6 H_s$ below still water level. This zone had the most erosion with the highest average erosion rate (see Figure 11). This is the zone in which holes developed in the grass cover.
- Zone 2 lies between still water level and a depth of $0.3 H_s$. The average erosion rate was half of the Zone 1 erosion rate. No holes developed in this zone.
- Zone 3 is the wave runup region above still water level. Very little erosion was observed in this zone.

Smith et al. (1994) commented that these test results clearly demonstrated the strength of this particular grass cover was due to the root system that was between 5 and 10 cm (2 and 4 inch) thick. Verheij and Meijer (1994) also noted that high elasticity and well-established roots restricted the erosion. After completion of the grass cover tests, additional tests were performed on just the clay layer. These tests are described in the following section of this report titled *Flood-Side Erosion Resistance of Clay*.

Schelde Basin 1994

Several different types of river dike grass cover (in spring condition) that had been managed differently were harvested from the upper Large Rivers area of The Netherlands and placed on a 1:3 slope in the Schelde Basin at Delft Hydraulics. The tests were conducted with relatively small irregular waves having $H_s = 0.3$ m (1.0 ft) and $T_p = 2.5$ s in a water depth of just 0.8 m (2.6 ft). These waves might be typical of wind waves generated over a short fetch in a river. All the grass cover samples were installed in a row, so they all received the same waves over the 60-hour duration of the experiment.

The type and amount of erosion were determined by measurements and by visual inspection, and the composition and structure of the vegetation and soil were determined (TAW 1997). Observed erosion was only a few centimeters in the sod with good, dense root systems. Erosion as much as 10 cm (4 inch) was seen where the grass root systems were less dense. Holes up to 20 cm (8 inch) deep developed quickly in places where the roots were sparse.

The damage to the grass and soil occurred mainly in the zone in which the waves were breaking. The maximum erosion rate in the wave impact zone was 0.3 mm/hr (0.01 inch/hr) for grass with a good sod quality and 2.3 mm/hr (0.09 inch/hr) for a bad sod quality (de Visser 2007). In the

wave run-up zone, some erosion also occurred, and the plant cover was damaged.

The amount of erosion was dominated by the quality of the root system and not by the type of soil in the grass cover. The greatest erosion was observed for grass cover with a good erosion-resistant clay, but with a poor root system due to rough mowing (TAW 1997).

It was concluded by TAW (1997) that...

...a good rooting system in the sod is decisive in determining erosion resistance, whereas the erosion-resistance category of the clay is no longer of importance. These findings agree with field observations of 0.4 m (1.3 ft) waves along a sea dike in Zeeland-Vlaanders, in which damage of several decimetres deep was seen in a very poorly developed sod in 48 hours, despite a moderate to good erosion-resistant soil. (TAW 1997).

German Large Wave Flume 2008

Giesenhainer and Oumeraci (2008) described full-scale laboratory experiments conducted in the German Large Wave Flume (GWK) in Hannover. A dike cross section was installed in the flume that was typical of dikes found on the North Sea coast in the German Bight, in The Netherlands, and in Denmark. The dike has a flood-side slope of 1:4 (approximately equal to typical slopes used in HSDRRS design), and seaward of the dike toe was a 1:40 sloped foreshore extending for 40 m (131 ft). The dike was constructed with a sand core and a cover layer consisting of erosion-resistant clay that was 0.6 m (2 ft) thick and a grass cover that was 0.2 m (8 inches) thick.

The winter grass cover layer was harvested from a dike in Ribe, Denmark, in blocks that measured 2.35 m by 1.25 m (7.7 ft by 4.1 ft). The blocks were placed in the flume on top of the compacted clay layer in a “brick” pattern to minimize the length of joints down the slope. The grass had a high root density near the surface, and the topsoil was porous and elastic in moist conditions. Geisenhainer and Oumeraci (2008) noted that good erosion resistance of a grass cover layer occurs when the grass coverage is higher than about 70% to 85%. The root system typically has 65% of the grass roots located in the upper 6 cm (2.4 inch) of the soil and 20% of the roots between 6 cm and 15 cm (2.4 inch and 5.9 inch) from the surface.

The wave impact tests were conducted with the still water level at 3.7 m (12.1 ft) which gave a water depth of 2.7 m (8.9 ft) relative to the toe of the 1:4 slope. The irregular waves for the tests had $H_s = 0.9$ m (3.0 ft) and $T_p = 5.0$ s. Impact pressures, flow velocity and runup were included in the measurement program. Testing continued over a time span of several weeks, although this does not imply that waves were run continuously all day, every day.

Geisenhainer and Oumeraci (2008) described damage to the grass cover layer resulting from a single wave impact that removed a region of the top soil as shown in Figure 12. Note that the wave action had degraded the above-soil grass, but the grass cover layer remained largely intact.

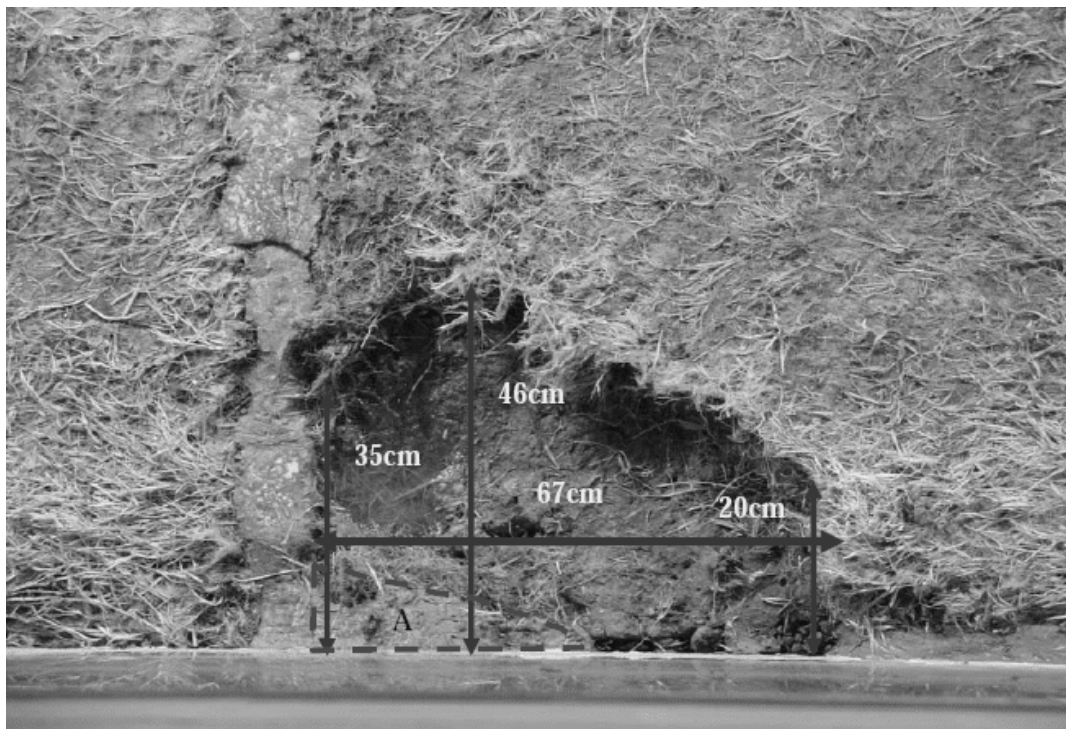


Figure 12. Wave impact damage from German 2008 test (from Geisenhainer 2008).

Over the cumulative duration of the test, several different types of damage to the grass cover were noted, and details are to be provided in a report that was still in preparation as of March 2010. Photographs taken at the end of the test indicated that the grass cover layer was (for the most part) intact with areas in the breaker zone where grass was removed, but most of the root system was still in place. Figure 13 shows some of the more severe damage.



Figure 13. Damage to the cover layer in the breaker zone (from Geisenhainer 2008).

Subsequent tests in this project included raising the still water elevation and subjecting the dike to overtopping and eventual breaching brought on by overtopping and headcutting on the landward side of the dike. During these tests, the upper section of the dike flood side withstood wave action of longer period waves having $H_s = 1.0$ m (3.3 ft) and $T_p = 10.0$ s. Throughout the remainder of the tests, the flood-side grass cover was damaged, but not to the point that large holes developed. Figure 14 is a photograph showing the grass cover on the flood-side slope during the breaching tests.

Conclusions, Caveats, and Concerns

TAW (1997) and de Visser (2007) formulated a number of conclusions and recommendations based on the above-described full-scale tests of wave attack on grass cover layers. These conclusions and recommendations are summarized below.



Figure 14. Flood-side grass cover during overtopping tests.

Grass cover characteristics

The grass cover layer has three characteristics that help protect the clay layer from erosion: (1) the flexibility and springiness of the grass cover helps absorb the high wave impact pressures that would otherwise lead to damage initiation; (2) the root network helps retain soil particles from erosion by flowing water in the runup and rundown zones; and (3) grass stems and blades above the soil that help shield the soil particles from the force of flowing water (TAW 1997).

- Good grass cover grows best in a not-so-cohesive clay layer, so the topsoil layer should be clay with a higher sand percentage so the grass can develop a thicker root system. Beneath the topsoil the clay should be stiffer and more erosion-resistant to help prevent severe erosion if the grass cover is removed entirely (de Visser 2007).
- Grass management is very important to assure a strong grass cover. After sowing on bare ground, the grass mat is at good strength after three to five years (TAW 1997).
- Without a strong root system, the top layer with the grass can erode easily, and the strength of the underlying stiffer clay must slow down the erosion process.

Wave loading

- Erosion of the grass cover layer is most likely to occur in the wave breaking zone just beneath the still water level where the highest wave impacts occur. Whereas single wave impacts might cause damage, the cumulative damage caused by many breaking waves could lead to failure of the grass cover. Therefore, duration of the storm waves at a given surge level is an important factor. (Note: damage is defined as a change in physical condition that does not result in loss of functionality. Failure is defined as damage that leads to loss of functionality. For this situation, failure is loss of erosion protection by the grass cover layer.)
- Erosion due to wave runup on grass cover layers in the zone above the still water level is much less than in the breaking zone. Most experiments noted little to no damage in the runup zone. The rate of erosion is at least four times slower in the runup zone than in the breaking zone (TAW 1997).
- The wave impact loading on a grass cover layer decreases as the flood-side slope decreases. On milder slopes the plunging wave jet often impacts the water from the previous wave rundown rather than on a “dry” slope, and this cushions the impact somewhat.

Wave resistance

- Grass is capable of withstanding considerable wave loads when used as a dike covering, and average erosion rates are on the order of only a few millimeters per hour. However, isolated damage pockets due to weaker grass cover layer can develop and expand at a higher rate.
- Sea and lake dikes show no damage after waves of $H_s = 0.75$ m (2.5 ft) where the grass cover layer is a closed grass mat with a high root density. This limit may be higher for dikes with milder flood-side slopes (TAW 1997).
- There is no grassland management method yet formulated that will cope with wave heights greater than 0.75 m (2.5 ft) without the waves causing some damage (TAW 1997).
- Very good grass mats with underlying erosion-resistant clay can resist waves up to $H_s = 1.0$ m (3.3 ft) on a flood-side slopes of 1:3 to 1:4 with no serious damage after more than one day. The damage-free period for waves of slightly more than 1.0 m was shorter, but still long enough to cope with a high water storm flood (TAW 1997).
- Waves breaking against sea and lake dikes can reach heights of more than $H_s = 1.5$ m (4.9 ft). Based on known information, it should not be

expected that a good grass mat (on flood-side slopes of 1:3 to 1:4) can resist breaking waves of this height for a sufficiently long time to survive the duration of a heavy storm surge (TAW 1997). Therefore, the strength of the underlying clay becomes critical for higher wave conditions.

- The guidelines for green dikes suggest that flood-side slopes of 1:6 and milder can tolerate wave heights up to $H_s = 1.6$ m (5.3 ft) provided there is ample foreshore to reduce the highest possible wave.

Hole development

- Isolated holes can develop in the grass cover in areas where the root structure is less dense, and a hole that is only 7 cm (2.8 inch) deep can trigger additional erosion that will further weaken the grass cover.
- Continued enlargement of a hole in the grass cover depends on two factors: (1) the amount of wave loading being applied at the location of the hole; and (2) the strength of the grass root system adjacent to the hole.

The available full-scale laboratory tests support the contention that well-developed grass cover layers are fairly resistant to direct wave attack on slopes of 1:4 up to about $H_s = 1.0$ m (3.3 ft). Even at higher wave heights, grass cover damage is slow to develop, and what damage does occur appears not to be severe provided good erosion-resistant clay is beneath the grass cover and the storm durations are shorter than (for example) three days. However, the above results are contingent on two important factors that must be considered when transferring this knowledge to the levees of the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS).

- The importance of the grass cover layer having a dense root system was emphasized in the European literature. In Europe the topsoil has greater sand content than the underlying stiff clay to promote dense root system. The HSDRRS levee specifications (Section 31 24 00.00 12 - Embankments and Section 32 92 19.04 12 - Turf Establishment and Maintenance) do not call for topsoil, and instead the entire levee cross section is built from stiff clay with a low percentage of sand. Consequently, while the stiff clay is more erosion-resistant than sandy topsoil, the root system might not develop with sufficient density to afford the level of protection observed in the European tests. Recent HSDRRS specifications allow addition of “strippings” (grass and organic material removed from the grass cover layer prior to a lift) along with grass seeds

- and fertilizer when populating a newly-constructed levee surface. The addition of strippings is expected to promote more rapid establishment of the root system. The performance difference between the European grass cover layers and the HSDRRS grass cover is unknown.
- Most of the European tests had wave periods in the $T_p = 4 - 5$ s range. New Orleans levees are expected to resist waves with longer wave periods. The 500-year design level predicts wave periods up to 9 s, and longer periods were seen during Hurricane Katrina. These longer periods might result in wave impact loads that are greater than those of the European tests if the slope and wave height are such that the waves break directly on the slope as plunging breakers. However, note that overtopping experiments in the German tests used 10-s waves, and the grass cover layer did not suffer severe damage.

5 Flood-Side Erosion Resistance of Clay

For very severe storms it is conceivable that wave impacts might erode large sections of the flood-side grass cover and root system, exposing the bare clay to the forces of impacting waves and strong flow velocities. Also, it is possible that a hurricane could strike a newly-constructed levee before grass has time to establish a good root system. Therefore, it is important to examine what is presently known about the erosion resistance of exposed clay under wave attack.

Erosion resistance of clay has been shown to depend significantly on the “soil structure” of the clay layer. In this section soil structure in clay is examined, followed by descriptions of results from three full-scale experimental programs conducted in the Dutch Delta Flume between 1984 and 1992. A summary of the key findings concludes this section.

Soil Structure

Soil structure describes the arrangement of the solid parts of the soil and of the pore space located between them. When a soil becomes structured, the bulk characteristics, such as permeability and resistance to erosion of the soil structure differ from the characteristics of the soil particles that make up the structure. Unstructured soils have a minimum of void space typical of well compacted clays, and the erosion resistance of unstructured soils is better than structured soils. Soil structure forms over time in clay soil due to smaller soil or vegetable particles being displaced and voids appearing. Consequently, structured soils are more easily eroded.

Soil structure is a somewhat qualitative assessment determined through visual inspection of such characteristics as shape and cohesiveness of soil aggregates and clods. According to the United States Geological Survey (USGS) web site (<http://wwwrcamnl.wr.usgs.gov/uzf/theory.struc.html>):

Because the unsaturated hydraulic properties are fundamentally quantitative, to theoretically relate them to soil structures requires the development of concepts and techniques that quantify soil structure.

Soil structure develops due to multiple causes including animal activity, expansion and contraction, and chemical processes. One measure of soil

structure would be the decrease in volume attained by compacting a soil with structure, but this might not tell us anything about increased erodibility. In short, there does not appear to be a rigorous method for quantifying soil structure, and instead it seems to be a relative description and subjective classification.

De Visser (2007) analyzed and summarized the key aspects of soil structure using multiple information sources including Dutch reports on full-scale experiments in the Delta Flume with bare clay and with grass covers, reports on the residual strength of 11 actual dikes in Zeeland, and the Dutch report “Technical Report Clay for Dikes” published in 1996 (TAW 1996). In a nutshell, de Visser described the essence of soil structure in dikes as follows:

Clay above the zone which is frequently under water has a soil structure as a result of regular changes in water content in this unsaturated zone and a result of environmental circumstances. If the clay is not kept in [a] sufficiently moist condition, atmosphere, flora and fauna affect the integrity of the clay and shrinkage and swelling results in a soil consisting of aggregates and blocks, a so-called soil structure. This soil structure development usually decreases with depth. The aggregates still have a partly mutual coherence. The aggregates are divided by cracks in different sizes, sometimes too small to see with the naked eye. This soil structure in the unsaturated zone limits the resistance of clay soil against loading by high waves. (de Visser 2007).

Accurate prediction of soil structure development in time is hindered by understanding of the complex interaction of all the factors that influence the development, including: soil-water interaction, atmospheric conditions, compaction, cover layer, thermal stresses, soil composition, and even the effect of animal and human activity.

Development of soil structure is most likely to occur on dike or levee slopes that experience cyclic changes in water content that cause alternating shrinkage and swelling. At locations that are always above the high water elevation of the adjacent body of water (except during extreme storms), soil structure develops due to rain water entering shrinkage cracks which causes the soil to swell. This process helps develop the aggregates and soil blocks of different sizes that make up a soil structure, and the resulting soil condition

has less cohesion than unstructured soil. The amount of soil structure decreases with depth. For levees adjacent to tidal waters, the portion of the slope within the inter-tidal zone will develop a soil structure of limited depth because the clay remains moist most of the time. Other key points on soil structure discussed by de Visser (2007) are itemized below.

- Given enough time, a well-developed soil structure will exist to some extent in all clay layers, and this determines the erosion resistance of the clay.
- In the upper 0.1 - 0.3 m (4 - 12 inch) of the unsaturated clay profile, soil structure will be completely developed within 1 to 3 years in a temperate climate. Well-compacted clay will become structured to depths between 0.3 m (12 inch) and 0.5 m (20 inch) after a few years (undefined by de Visser) and increase to depths of 1.6 m to 1.8 m (5.3 ft to 5.9 ft) after 50 years.
- In grass cover layers, the soil structure develops in a few years, but it is strong because the small aggregates of clay are held together by the grass root system.
- Observations indicate that a recognizable soil structure develops under a stone revetment after five years, and after 10 to 15 years the soil structure depth can be 0.8 m (2.6 ft) deep.
- Greater variation in air temperature promotes more shrinkage that, in turn, contributes to soil structure development.
- The INFRAM (2003) design method assumes soil structure develops with depth at a rate of 10 mm/yr (0.4 inch/year).
- Compacted clay will continue to retain high erosion resistance and low permeability if the clay is kept continuously moist.
- Animal burrows or human activities provide a means for surface water to penetrate the clay layer rapidly, and this will increase the rate of soil structure development. This may also increase vegetation growth that causes soil structuring while at the same time providing soil strengthening if the root system is dense.
- Soil structure development is less in clays with lower sand percentages, which are already fairly erosion resistant. Thus, building levees with heavy clay and good compaction throughout provides a layer with good residual strength over a longer time.

Dutch research has identified three categories of clay condition:

- unstructured clay that has a minimum of voids and high erosion resistance;
- structured clay that has less strength and is more easily eroded; and
- moderately structured clay that serves as a transitional phase between the unstructured and structured states.

De Visser (2007) used the quantitative data of TAW Klei (1996) and Kruse and Nieuwenhuis (2000) to propose an empirical relationship for soil structure depth development as a function of time. This relationship is shown in Figure 15 along with the few available data.

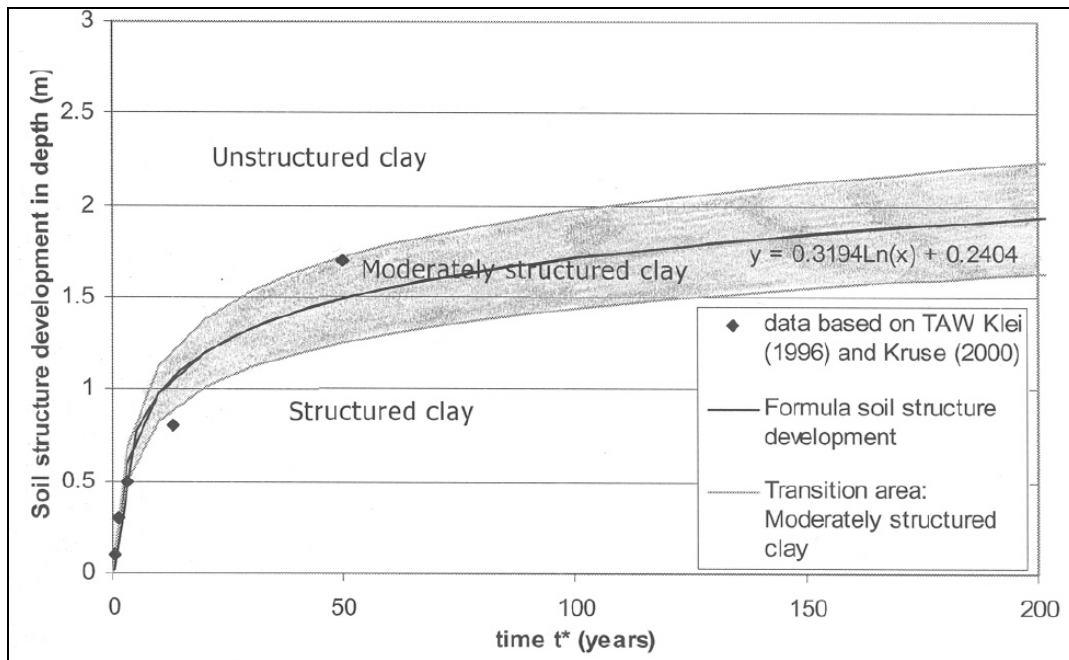


Figure 15. Soil structure development in clay as a function of time (from de Visser 2007).

The solid line is the boundary between structured and unstructured clay, and the shaded area represents the zone of moderately structured clay. The solid line is given by the following dimensionally non-homogeneous equation (de Visser 2007)

$$d_{\text{structure}} = 0.32 \ln(t^*) + 0.24 \quad (1)$$

where $d_{\text{structure}}$ is the depth of soil structure development in meters and t^* is time in years since construction in years (known or assumed).

The form of Equation (1) has no physical basis, and the available data used to construct the curve are limited. In particular, note that the data extend no farther than 50 years, whereas the plot on Figure 16 was drawn to 200 years. Several other factors influencing the rate of soil structure development were not included due to lack of data. For example, clay type and sand percentage are certainly important to soil structure development.

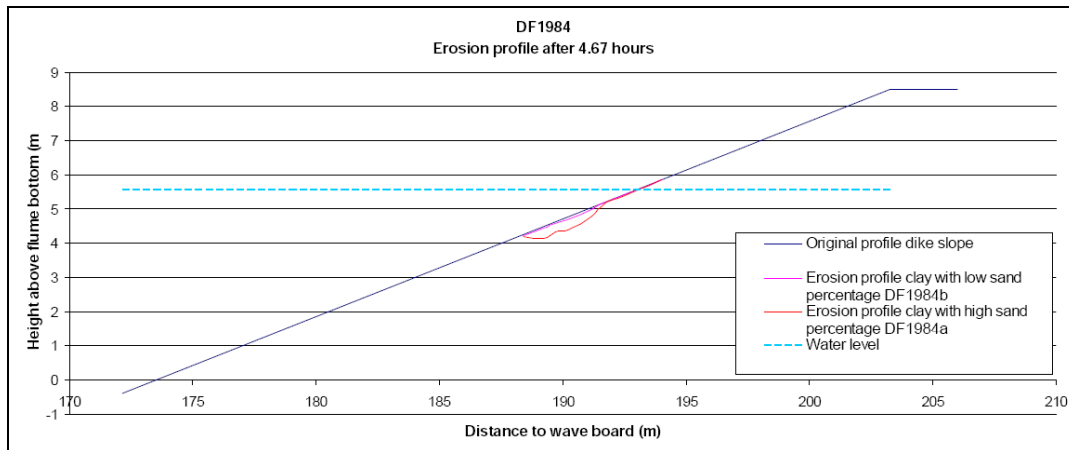


Figure 16. Clay erosion profiles from 1984 Delta Flume tests (from de Visser 2007).

De Visser (2007) summarized results from three full-scale experimental programs conducted in the Dutch Delta Flume between 1984 and 1992. These tests are described in chronological order in the following sections.

Delta Flume 1984

The primary purpose of these experiments was to test the stability of brick revetments protecting an underlying 0.8-m-thick (2.6-ft-thick) clay layer that was installed on a 1-on-3.5 slope in three compacted lifts. Two types of clay were tested: one with a low sand percentage, and one with a high sand percentage. Both clays had high water content; and because of the compaction, the soil was described as unstructured.

The first experiment documented damage that occurred when the armoring layer was left in place with a few exposed areas representing previous damage. Ultimately, many of the bricks were washed away, and the clay sustained some erosion damage. However, the amount of erosion was minor, and there were many scattered bricks left on the slope that may have provided some reduced level of protection.

Of greater interest was the second set of experiments with the revetment removed and the bare clay directly exposed to wave attack. These experiments used regular waves having a wave height of $H = 1.05$ m (3.4 ft) and wave period of $T = 12$ s. The longer wave periods on the slope produced surging waves that are less damaging than plunging breakers, but erosion still occurred in the wave breaking zone beneath the still water level.

The slope constructed of clay with low sand content experienced only minor erosion over the 4.4-hour experiment duration, even when artificial notches were created in an attempt to promote erosion. Greater erosion was seen for the slope constructed of clay with a higher sand percentage. Initially, little erosion occurred for the first 2.4 hours of wave action; but then a small hole started to expand until it reached an erosion depth of 0.4 m (1.3 ft) after 4.4 hours (near end of the experiment). Final erosion profiles for these experiments are shown in Figure 16.

The erosion profile for the clay with low sand percentage differs very little from the original profile. Erosion for the slope with higher sand percentage can be seen in Figure 16. Erosion occurred below the still water level. An erosion depth of 0.4 m (1.3 ft) is half of the installed clay layer, and this would be a concern for dikes having a sand core. However, this depth of erosion would be considered minor damage for a levee constructed entirely of good quality clay. De Visser (2007) also noted that notches placed on the slope above the still water level did not induce any erosion.

Delta Flume 1992 (with stone cover)

This set of unique experiments examined the stability of clay layers that had developed beneath a stone revetment on actual dikes. Two types of in-situ clay were harvested in blocks having lateral dimensions of 2.45 m (8 ft) and a thickness of 0.8 m (2.6 ft). One clay (Kruiningen) was classified as erosion category 1 (see Figure 9) and the other (Perkpolder) was erosion category 2. The top soil in the upper 10 - 20 cm (4 - 8 inch) was more erodible than the underlying clay. The most notable aspect of these dike samples is that the clay had definite soil structure that included sand and silt inclusions in the upper portion (Kruiningen clay) and some sand lenses (Perkpolder clay).

The Delta flume was divided in half lengthwise so both clay types could be tested simultaneously. After initial testing with a cover of revetment stone, the revetment was removed, and the bare soil was subjected to irregular

waves at two different water levels. The wave conditions at the higher water level of 5 m (16.4 ft) were $H_{mo} = 1.47$ m (4.8 ft) and $T_p = 4.9$ s. A hole formed in the category 1 clay just above the still water level after only 10 min. This hole expanded to a diameter of 0.35 m (1.2 ft) with depth also of 0.35 m (1.2 ft) in just 15 min. The erosion was described by de Visser (2007) as starting in the zone of wave breaking and extending slowly upward. No significant erosion occurred in the runup zone.

Blocks of the category 1 clay up to 15 cm (6 inch) in diameter were broken loose and rounded by the wave action. The sides and upslope portion of the erosion hole were steep to vertical. After 2 hours of testing, the category 1 clay was completely eroded through the entire 0.8-m (2.6-ft) thickness. The category 2 clay faired somewhat better because it had a less developed soil structure; and after 2 hours the erosion in the breaking zone was about 0.6 m (2 ft) deep. Erosion profiles are shown in Figure 17. The category 2 clay is shown on the left-hand plot, and the category 1 clay is shown on the right.

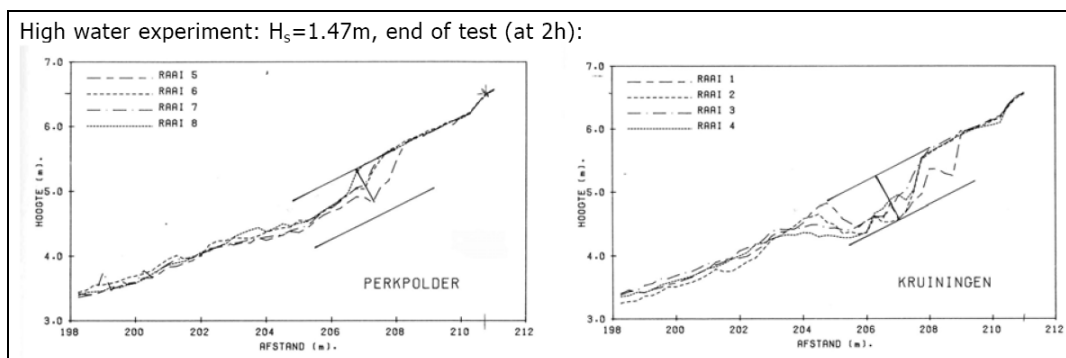


Figure 17. Clay erosion profiles from 1992 Delta Flume high water tests (de Visser 2007).

The water level in the flume was lowered to 3.5 m (11.5 ft) for the second test so the waves would impact on a portion of the profile that was relatively unscathed by the first test. Wave parameters for this test were $H_{mo} = 1.0$ m (3.3 ft) and $T_p = 4.2$ s. The profiles eroded in the same manner as seen in the high-water test with most of the erosion occurring beneath the still water level, but at a slower rate due to decreased wave height. Once again, the category 2 clay exhibited less erosion because it had less soil structure than the category 1 clay that would normally be considered less erodible if well compacted. The erosion profiles after 3.6 hours of testing are shown in Figure 18. The category 2 clay is shown in the left-hand plot with a maximum erosion depth of 0.45 m (1.5 ft), and the category 1 clay is on the right with maximum erosion of 0.7 m (2.3 ft).

Failure of the category 2 clay occurred after 15 hours when the 0.8-m (2.6-ft) thickness was completely eroded.

Table 2 gives the average depth of erosion rates for the two types of clay as presented in de Visser (2007). The category 1 clay having more soil structure eroded about twice as fast as the less structured category 2 clay. The depth erosion rate doubled when the wave height was increased by 50%. In all cases the region of erosion was confined to the wave breaking zone.

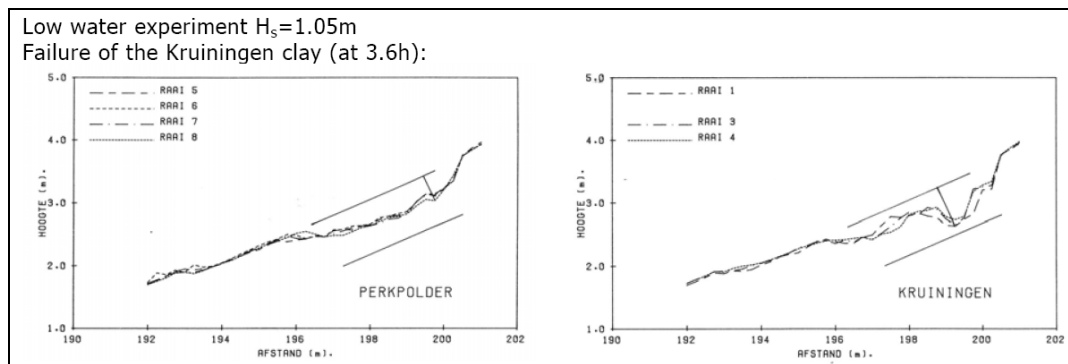


Figure 18. Clay erosion profiles from 1992 Delta Flume low water tests (de Visser 2007).

Table 2. Average depth of erosion rates estimated from Delta Flume 1992 experiments.

Test Series	H_{mo}	T_p	Average Erosion Rate	
			Category 2 Clay	Category 1 Clay
High water level	1.47 m (4.8 ft)	4.9 s	30 cm/hr (12 in/hr)	70 cm/hr (28 in/hr)
Low water level	1.00 m (3.3 ft)	4.2 s	13 cm/hr (5 in/hr)	23 cm/hr (9 in/hr)

Delta Flume 1992 (with grass cover)

Tests to examine the residual clay strength of grass-covered slopes were conducted in the Delta Flume following the tests of grass cover that were described in the previous section of this report titled *Flood-Side Erosion Resistance of Grass*. The dike had a flood-side slope of 1-on-4, similar to the HSDRRS design; and the installed grass was harvested from an existing sea dike as described previously. Smith et al. (1994) stated that the grass and topsoil in the upper 10-20 cm (4-8 inch) had been eroded away in the previous tests, so most of the root system was gone and only the clay sub-layer remained. However, there could have been some root structure helping to strengthen the clay.

Wave conditions were the same as used for the grass-cover tests, i.e., $H_{mo} = 1.35\text{ m}$ (4.4 ft) and $T_p = 4.7\text{ s}$. Water depth was 4.8 m (15.7 ft). After

4 hours of testing, an underwater survey revealed an erosion hole with depth of 0.4 m (1.3 ft) just below the still water level. However, this hole was situated along the edge of the flume, and it was thought to be a sidewall effect similar to what might be seen at a levee transition. At locations away from the flume sidewall, the maximum erosion depth after 5 hours of wave attack was only 0.25 m (0.8 ft). The average depth erosion rate was given by Smith, et al. as 5 cm/hr (2 inch/hr).

Summary of Clay Erosion Rates

De Visser (2007) plotted the maximum erosion depth as a function of time for all the Delta Flume tests of flood-side wave-induced erosion, including both grass covers and bare clay. These plots are shown on Figure 19. An explanation of the plot legend in Figure 19 is given Table 3.

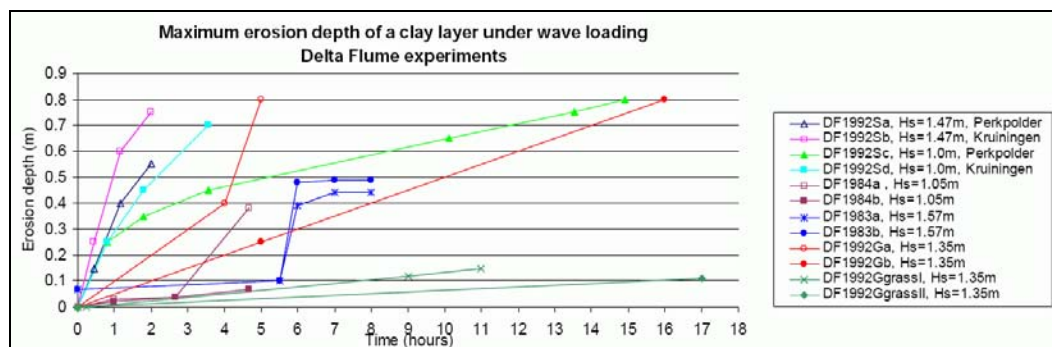


Figure 19. Clay depth erosion rate from all Delta Flume tests (from de Visser 2007).

Table 3. Explanation of Figure 19 legends.

Legend	Description
DF1992Sa	Delta Flume 1992 (with stone cover), high water, category 2 clay
DF1992Sb	Delta Flume 1992 (with stone cover), high water, category 1 clay
DF1992Sc	Delta Flume 1992 (with stone cover), low water, category 2 clay
DF1992Sd	Delta Flume 1992 (with stone cover), low water, category 1 clay
DF1984a	Delta Flume 1984 tests (bricks removed), high sand percentage
DF1984b	Delta Flume 1984 tests (bricks removed), low sand percentage
DF1983a	Delta Flume 1983 tests of grass cover, init. hole at 1 m below SWL
DF1983b	Delta Flume 1983 tests of grass cover, init. hole at 0.5 m below SWL
DF1992Ga	Delta Flume 1992 tests (grass removed), erosion at sidewall
DF1992Gb	Delta Flume 1992 tests (grass removed), erosion at center
DF1992GgrassI	Delta Flume 1992 tests of grass cover, 11-hr duration
DF1992GgrassII	Delta Flume 1992 tests of grass cover, continuation of test to 17 hrs

The depth erosion rates for grass covers are significantly lower than for bare clay, which is to be expected because the sod layer has to be completely eroded before the waves can act on the underlying clay. The differences in the curves for bare clay can be attributed primarily to differences in the clay (sand percentage and soil structure), and to a lesser extent on the significant wave height. Several of the curves show what look like linear trends in time that would be tempting to extrapolate to longer durations. However, this would most likely be ill-advised because the clay erosion may reach some sort of equilibrium configuration after sufficient time similar to sand dune erosion.

The Dutch research focuses on clay layers up to 1 m (3.3 ft) thick that protect a sand core. Therefore, none of the tests provides information for erosion depths greater than about 0.8 m (2.6 ft). Levees composed of solid clay of reasonable quality that have been constructed in compacted lifts should be able to sustain erosion depths well above 1 m (3.3 ft) without putting the levee at risk.

Conclusions, Caveats, and Concerns

Results from three full-scale experimental test series conducted in the Delta Flume in The Netherlands provide several insights into the resiliency of clay levee (dike) flood-side slopes when subjected to direct attack by large breaking waves. Some of the conclusions are listed below.

Clay type

- Even though we expect that clay having a lower sand percentage should have higher erosion resistance, the experimental evidence was not sufficient to make a clear distinction. Other factors appear to be more influential. (Note that very sandy soils were not included in any of the tests, and such soils are expected to be easily eroded.)
- Higher erosion rates were seen for unstructured soils with a higher sand percentage when compared to unstructured soils with low sand percentage under the same forcing conditions.
- Clay type is not the major influencing factor on the erosion rate of clay under wave loading.

Soil structure

- Soil structure is a major factor influencing erosion rates under wave attack.
- Structured clay is more easily eroded than unstructured clay.
- A possible categorization for soil structure is “structured clay, moderately structured clay, and unstructured clay” (de Visser 2007).
- Structured clays with low sand percentage are more easily eroded than moderately structured clays with high sand percentage.
- Within each subdivision of soil structure, the influence of clay type can be seen, i.e., clays with lower sand percentage are more erosion resistant.
- Unstructured clay (compacted) having a high sand percentage may erode in clumps that break loose due to wave impacts.

Wave condition

- Generally, depth erosion rate increases with wave height. However, wave period and flood-side slope are also important because all three parameters determine whether the wave breaks in plunging mode. Plunging breakers are thought to cause greater clay erosion than other breaker types.
- Erosion of structured clays increases with increasing wave height.
- Initial erosion of moderately structured clays appears to depend more on the percentage of sand than on increasing wave height. Eventually, wave height becomes more influential once an erosion hole is formed.

Clay erosion

- Bare clay has substantial erosion resistance provided the clay has limited soil structure and the sand percentage is not high.
- The poorest performing structured clay experienced a maximum erosion depth of 0.75 m (2.5 ft) after 2 hrs of waves having a significant wave height of 1.5 m (4.9 ft).
- The best performing soil was unstructured compacted clay (Delta Flume 1984). The maximum erosion depth for the clay with high sand percentage was 0.4 m (1.3 ft) after 4.4 hrs of regular waves having a wave height of 1.05 m (3.4 ft). Similar compacted clay with low sand percentage was barely eroded. Note, however, that wave periods for these experiments were 11 s, and the waves surged on the flood-side slope rather than breaking as plunging waves.

Levees of the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS) that have been constructed with a cross section composed entirely of well-compacted, high-quality clay will be highly resistant to wave induced erosion caused by plunging waves having a significant wave height up to 1.5 m (4.9 ft). However, several uncertainties remain that cannot be resolved by the Dutch experimental results.

- Dutch research interest was limited to 1-m-thick clay cover layers because that is what they use to protect the dike sand cores. As a consequence, erosion during the experiments never progressed past an erosion depth of 0.8 m (2.6 ft). We do not know how the erosion rates might change if erosion is allowed to continue beyond that depth. Most likely the rate of erosion will decrease with erosion depth; but until that trend is established, a conservative approach would be to do straight-line extrapolation of the erosion rates.
- Soil structure was shown to be important. An understanding is needed about how soil structure develops over time in the climate and environment specific to the HSDRRS levee system, and how that might differ from soil structure development in The Netherlands. At the same time, well maintained grass cover should develop denser root systems as a result of increased soil structure, and this needs to be taken into account.
- There is no research that examines erosion resistance of weak soils such as sandy silts or hydraulically-placed levee soils. Whereas upgrades to the HSDRRS will identify and rehabilitate (or armor) any flood-side slopes constructed of poor soil, these levees will remain at risk until such measures are completed.

6 Available Design Methodologies

Design guidance and methodologies are available for estimating erosion rates due to wave attack on flood-side grass-covered slopes and bare clay slopes. These methods are based almost exclusively on limited Dutch full-scale flume tests described in previous sections of this report, and most researchers are in agreement that the methods are in need of substantial improvement. This section reviews two closely-related empirical relationships for estimating maximum erosion depth for grass-covered levee slopes, and four methods for estimating the maximum erosion depth of bare clay.

The wave heights used to establish the prediction techniques are from the laboratory experiments, and thus, they represent wave heights near the levee in water depths not exceeding about 5 m (about 16 ft). In other words, the wave heights used in the prediction methods should be determined for water depths near the toe of the flood-side slope. For levees with extended flood-side berms, this may mean the waves could become depth-limited before reaching the flood-side slope.

Wave Erosion of Grass-Covered Clay Levee Slopes

Seijffert and Verheji (1998)

Seijffert and Verheij (1998) proposed a simple empirical formula for estimating time-dependent erosion depth (perpendicular to the levee slope) as a function of significant wave height, storm duration, and quality of the grass cover. Using the results from the Dutch full-scale test that are described in this report in the section titled *Flood-Side Erosion Resistance of Grass* they noted the following: (a) waves up to 0.5 m (1.6 ft) caused no damage to grass covers; (b) waves in the range 0.5 - 1.5 m (1.6 - 4.9 ft) with a duration between 6 and 24 hours generally did not cause severe damage; and (c) waves greater than 1.5 m (4.9 ft) will likely cause severe erosion, but experimental data are not available.

As previously stated, the Dutch full-scale flume tests incorporated relatively robust grass layers consisting of clay with a higher sand percentage to promote a dense grass root system. The Dutch researchers indicated that grass quality and root density were more important to

stability than soil type; but on the other hand, none of the tested soils were sand or silty sand.

Seijffert and Verheij (1998) hypothesized that the rate of erosion depth would increase with the square of significant wave height (H_{m0}), and they presented the following empirical formula for erosion depth

$$d = \frac{3,600}{\gamma C_E} t_E \left(\frac{H_{m0}}{10} \right)^2_{\max} \quad (2)$$

where

d = maximum erosion depth of grass layer [m]

γ = safety coefficient greater than 1 [-]

$t_{\max}$ = duration of waves [hr]

C_E = grass cover quality factor [$\text{m}^{-1}\text{s}^{-1}$]

Seijffert and Verheij gave the following ranges for the grass cover quality factor, C_E , but they did not provide much information on how to categorize a particular cover layer at one of these three quality levels.

grass of good quality : $C_E = 0.5 - 1.5 \left(10^{-6} \right) \left[\text{m}^{-1}\text{s}^{-1} \right]$

grass of average quality : $C_E = 1.5 - 2.5 \left(10^{-6} \right) \left[\text{m}^{-1}\text{s}^{-1} \right]$

grass of poor quality : $C_E = 2.5 - 3.5 \left(10^{-6} \right) \left[\text{m}^{-1}\text{s}^{-1} \right]$

Application of Equation (2) requires strict attention to the variable dimensional units. The numerical coefficient (3,600) in the equation has units of sec/hr, and this allows the duration to be specified in units of hours. Otherwise, the equation can be used with any consistent set of units. SI units were used originally as defined above. If Customary English units are used, then d and H_{m0} must be specified in units of ft, and C_E must be converted to units of $\text{ft}^{-1} \text{sec}^{-1}$.

TAW (2004)

A slightly improved version of Equation (2) was given in TAW (2004) as reported by Doorn (2007) that attempted to account for peak wave period and flood-side dike slope. Wave erosion depth was given by the equation

$$d = \frac{3,600}{\gamma C_E} \left(\frac{4}{3} t_r \tan \beta \right)^2_{\max} \quad (3)$$

where

$$H\delta H_{m0} \quad (4)$$

and

$$\delta = 0.5 \frac{(T_p)^{1/2}}{(H_{m0})^{1/4}} \quad (5)$$

The coefficient δ has to be dimensionless; therefore Equation (5) is a dimensionally inhomogeneous equation that is only valid when peak wave period is given in seconds and significant wave height is specified in meters. Hence, the numerical coefficient 0.5 must have units of $(\text{m})^{1/4} (\text{s})^{-1/2}$.

Equation (5) can be converted for use with Customary English units, i.e.,

$$\delta = 0.5 \left(\frac{\text{m}^{1/4}}{\text{s}^{1/2}} \right) \frac{(T_p)^{1/2}}{(H_{m0})^{1/4}} \cdot \left(\frac{1 \text{ ft}}{0.3048 \text{ m}} \right)^{1/4} = 0.673 \left(\frac{\text{ft}^{1/4}}{\text{s}^{1/2}} \right) \frac{(T_p)^{1/2}}{(H_{m0})^{1/4}} \quad (6)$$

Doorn (2007) recommended a safety factor of $\gamma = 2$, and he presented another dimensionally inhomogeneous equation for estimating the peak period, T_p , as

$$T_p = 4 \sqrt{H_{m0}} \quad (7)$$

with the wave height specified in meters. Equation (7) implies that a simple approximation was used to represent the joint probability of wave height and peak period for irregular seas, and it may be better to use the value of wave period determined from the wave analyses.

It is important to stress that Eqns. (2) and (3) are based on limited full-scale test results in which the maximum significant wave height was $H_{m0} = 1.6 \text{ m}$ (5.2 ft) and peak wave periods were in the 4- to 5-sec range. Applying these formulas to wave conditions with higher wave heights and longer wave periods should be done with caution because the reliability of the estimates will decrease with extrapolation away from the tested parameters.

Wave Erosion of Bare Clay Levee Slopes

INFRAM (2003) Design Method

A Dutch engineering firm, INFRAM, developed a design method that is the basis for the current design manual used by Projectbureau Zeeweringen. The development of the method assumed that category 1 bare clay exists in the intermediate and lower layers beneath stone revetments in a region 0.5 m (1.6 ft) below and 1.0 m (3.3 ft) above mean high water (MHW). Above that range, the dike is assumed to have a good-quality grass cover.

The design method is based on previous studies of clay layer residual strength, including the full-scale 1992 Delta Flume tests under stone covers. Erosion due to wave runup and rundown was considered to be minor compared to erosion caused by breaking wave impact. The residual strength of clay layers contained in earlier publications was linearly extrapolated from 11 hours to 35 hours without supporting data. This extrapolation could be considered conservative because the clay is expected to become more unstructured with depth, so the erosion rate should taper off in time instead of continuing to increase linearly. It was also assumed that erosion rates will be higher at locations where the clay does not stay dry most of the time because of the formation of soil structure. The rate of soil structure formation was assumed to be 10 mm (0.4 inch) of depth per year, and this rate needs to be factored into the dike safety evaluation. Finally, the design method is only intended to be applied for waves with significant wave heights less than 2 m (6.6 ft).

The INFRAM design method, shown on Figure 20, consists of three lines representing erosion rates for different significant wave heights. Note that the initial clay thickness is at about 1.2 m (3.9 ft), and the design curves provide an estimate of additional clay thickness needed so that after erosion there will be at least 1.2 m of clay thickness left as reserve strength. After an initial period of no erosion, each erosion rate line has a slope that is given in Table 4. Erosion due to flow during wave run-up and run-down was assumed to have only minor influence on maximum erosion depth compared to erosion by wave impacts.

A key uncertainty of the INFRAM design method is the validity of the erosion curves beyond 10 hours. Logically, the linear extrapolation should be conservative, but no data support this contention. The design method is also not applicable for wave heights greater than 2 m (6.6 ft), and the

effect of longer wave periods in the range of 8 - 14 s is unknown. As noted, INFRAM (2003) recommended a minimum clay layer thickness of 1.2 m (3.9 ft), and they noted the uncertainties in this method. It was suggested that the erosion rate may decrease for flood-side slopes milder than 1:4 (de Visser 2007).

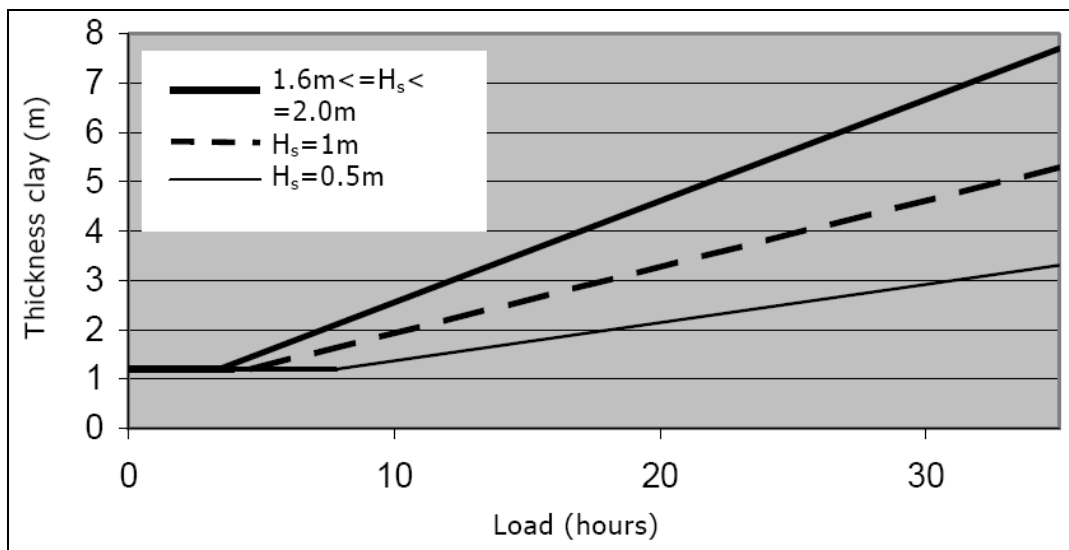


Figure 20. INFRAM (2003) design method (from de Visser 2007).

Table 4. Slopes of the erosion rates given in Figure 20.

H_{mo}	Slope of Erosion Rate
1.6 - 2.0 m (5.3 - 6.6 ft)	0.21
1.0 m (3.3 ft)	0.14
0.5 m (1.6 ft)	0.08

TAW VTV (2004)

Results from the Delta Flume 1992 experiments were used to develop values of residual clay layer strength for clay under stone (after the stone revetment is destroyed). The result was published in Dutch and referred to as “Voorschrift Toetsen op Veilgheld” or VTV (TAW VTV 2004). De Visser (2007) said the VTV guidance was conservative, and it is intended for use in the five-year testing program for Dutch water defenses. Therefore, the VTV gives safe estimates for determining the total revetment strength. Note that this guidance pertains to Dutch sea dikes armored with stone revetments within the tidal zone. In other words, a portion of the slope is submerged during part of each tide cycle.

Table 5 shows the time in hours that different thickness of the clay layer are expected to survive under significant wave heights up to 1.5 m (4.9 ft). The VTV guidance stated that residual clay strength should be considered zero for significant wave heights greater than 2 m (6.6 ft).

Table 5. Residual clay layer strength in hours (from TAW VTV 2004 and de Visser 2007).

Erosion Resistance	Clay Thickness (m)	H_{mo} (m)				H_{mo} (m)			
		0.2	0.5	1.0	1.5	0.2	0.5	1.0	1.5
		Elev. below MHW+1 m				Elev. above MHW+1.0 m			
Low	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.7	2.0	1.5	1.5	1.0	2.0	1.5	1.5	1.0
	1.0	3.5	3.0	3.0	2.0	3.0	3.0	3.0	2.0
	1.2	5.0	4.5	4.5	3.0	4.5	4.5	4.5	3.0
High + Low	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.7	4.0	3.0	2.0	1.5	3.5	2.5	1.5	1.0
	1.0	7.5	6.0	4.0	3.0	6.5	5.0	3.0	2.0
	1.2	11.0	9.0	6.0	4.5	9.5	7.5	4.5	3.0

There are two sets of numbers corresponding to elevations above and below a reference elevation that is 1 m (3.3 ft) above the MHW level. De Visser did not explain what was meant by an erosion resistance termed “High+Low.” Because the row for clay thickness of 0.4 m (1.3 ft) are the same for both erosion resistance categories, it is assumed that “High+Low” refers to a clay layer with a structured upper portion with unstructured clay beneath. However, this is only an assumption. The TAW VTV guidance is not used in this analysis presented in Chapter 7 of this report, so no further effort was made to define “High + Low.”

WL|Delft Hydraulics (2006) formula for residual strength of clay

The VTV guidance for residual strength of clay beneath stone revetments was considered too conservative, and it did not account for different clay conditions. Researchers in The Netherlands (WL|Delft Hydraulics 2006) reanalyzed results from the full-scale experiments of bare clay (Delta Flume 1984 and Delta Flume 1992), and they proposed an empirical formula for estimating the erosion depth of bare clay. This formula might be useful for analyzing erosion of the clay if overlying armor had been removed by wave action, or if the levee slope was subjected to storm waves before the grass cover was established or armoring could be placed. The

Dutch experiments included both structured clay (Delta Flume 1992) and unstructured clay (Delta Flume 1984), and the developed empirical formula contains a coefficient to differentiate between the two types.

Maximum depth of erosion perpendicular to the levee or dike slope during wave action was given by

$$d_e = C_c \cdot H_{m0} [-6.9 + \ln(t_s)] \quad (8)$$

where

d_e = maximum erosion depth [m]

H_{m0} = significant wave height [m]

t_s = duration of waves [s]

C_c = clay type factor [-]

In Equation (8), note that erosion depth is directly proportional to wave height and that the erosion rate is logarithmic in time. Recommended values for the clay-type coefficient are given in Table 6.

Table 6. Recommended values for C_c in Equation (8).

Experiment	Clay Condition	Clay Type	C_c
DF1992S	Structured	Category 2 (less structured)	0.20
		Category 1 (more structured)	0.26
DF1984	Unstructured	Higher percent sand	0.018 - 0.032
		Lower percent sand	0.014 - 0.025

Equation (8) is dimensionally inhomogeneous because time MUST be entered in seconds, so care must be taken when applying this equation. However, there is dimensional consistency between erosion depth and wave height, so Customary English length units could be used for these two parameters.

De Visser (2007)

After an incredibly thorough review of past research and present knowledge on the resistance of clay dikes to wave erosion on the flood-side slope, de Visser (2007) proposed a semi-quantitative model for estimating maximum erosion depth for clay slopes. She schematized the time rate of

wave-induced erosion of a grass-covered clay dike into four distinct time zones as shown on Figure 21.

Zone I is the time needed for the waves to erode the grass-cover layer and expose the underlying clay. This erosion rate is relatively slow for good-quality grass cover. Depending on the age of the dike and other factors, the upper clay layer beneath the grass cover may have a good deal of soil structure. This layer will erode at a fast rate as shown in Zone II of Figure 21. Eventually, the erosion proceeds through the structured clay to a layer of moderately structured clay (Zone III), and the erosion rate decreases. Finally, if the waves continue at the same level of intensity; the deeper unstructured clay is exposed, and the erosion rate in Zone IV slows almost to the same rate as seen initially for grass. Of course, many related factors determine the exact nature and slopes of the generalized curve shown in Figure 21.

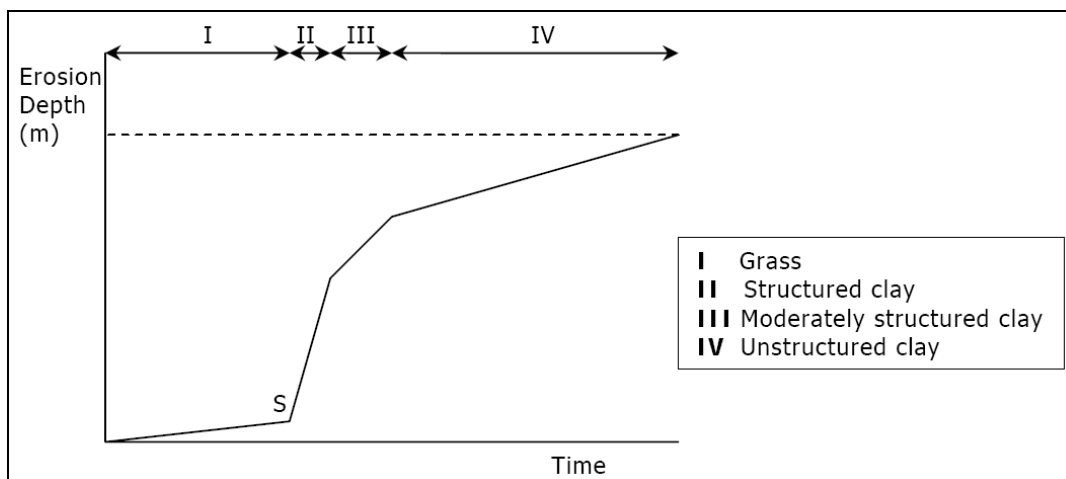


Figure 21. Schematized erosion process of a grass-covered clay dike (from de Visser 2007).

De Visser (2007) reanalyzed the Delta Flume full-scale data to determine reasonable slopes for Zones II, III, and IV. A representative example of de Visser's model is shown in Figure 22. (Important: Figure 22 cannot be used directly for design estimation!) The shaded region brackets the observed gradients of the erosion rates for soil conditions representing the three zones with the top of the gray region representing the highest wave heights in the tests. Note that de Visser did not include the Zone I erosion of the grass cover layer in her model, so the erosion depth is taken relative to the underside of the grass cover layer. The solid line in Figure 22 represents the average erosion gradient for each zone, and the gradient values are listed in Table 7.

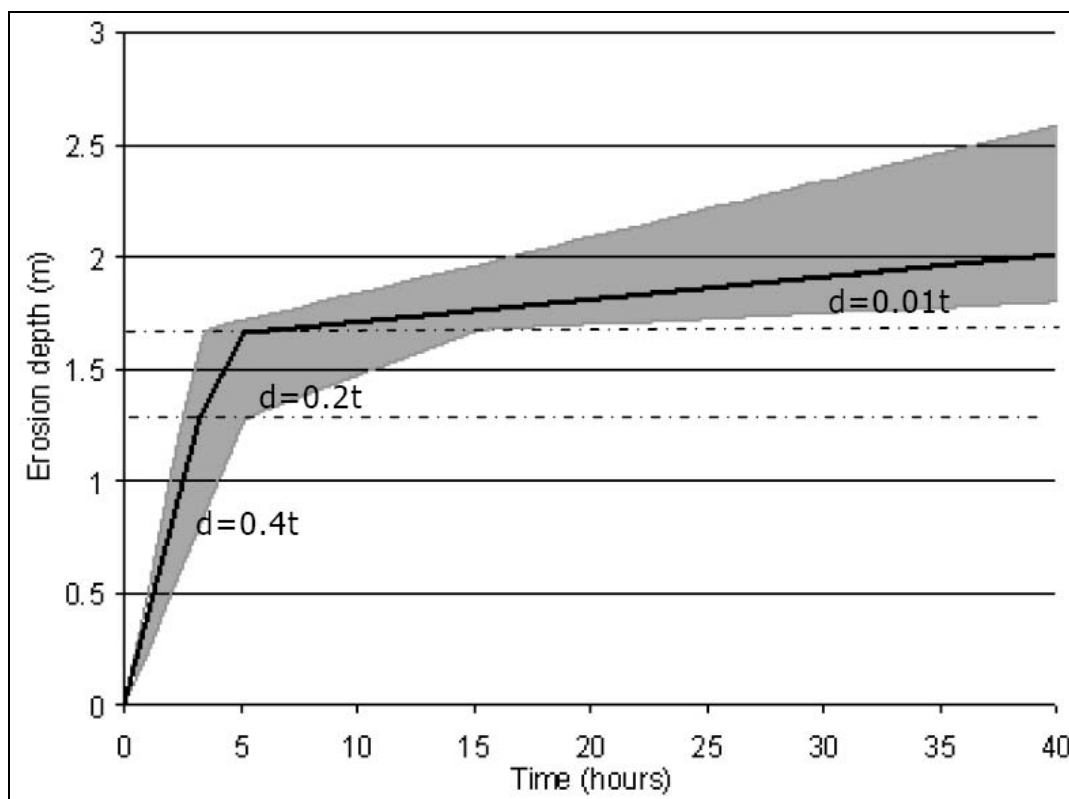


Figure 22. De Visser's semi-quantitative model for clay dike erosion (from de Visser 2007).

Table 7. Erosion gradients for de Visser's semi-quantitative model.

Clay Condition	Zone	Gradient (or Slope)
Structured	II	0.4 m/hr
Moderately structured	III	0.2 m/hr
Unstructured	IV	0.01 m/hr

The equations in Figure 22 for Zones III and IV give the maximum erosion depth from the beginning of that zone and not from initiation of erosion at time $t = 0$. Transition from one zone to the next (location of dashed horizontal lines in Figure 22) depends on the soil structure profile for the site-specific clay layer. An estimate of the soil structure development can be made using Figure 15 and the associated Equation (1), but it would be better to base the estimate on actual site borings. Because of the coupling of de Visser's model with the soil condition, the particular curve shown in Figure 22 is only valid for a soil condition profile that is structured to a depth of about 1.3 m (4.3 ft), moderately structured to a depth of about 1.7 m (5.6 ft), and unstructured beyond that depth.

De Visser (2007) stressed the following points about this semi-quantitative erosion depth model:

- The model is based on full-scale experimental results for significant wave heights ranging between 1.0 - 1.5 m (3.3 - 4.9 ft). Wave periods were about 4 - 6 s. The upper boundary of the gray area represents the 1.5-m (4.9-ft) waves. De Visser stated, “...it is advised against extrapolation of the semi-quantitative model for higher wave heights without further research.”
- The model is intended to give reasonable estimates of actual erosion under the range of wave conditions and soil condition. If the model is eventually adapted for design use, an appropriate factor of safety must be included.
- Attempts to further quantify the model by including wave parameters and breaker type (i.e., flood-side slope) were not successful due to lack of data needed to establish the relationships.

De Visser (2007) also provided approximate geometry of the erosion hole on a slope based on her analysis of the erosion development in all of the full-scale Delta Flume experiments. The idealized profile is reproduced in Figure 23. The dashed lines represent growth of the erosion hole in time.

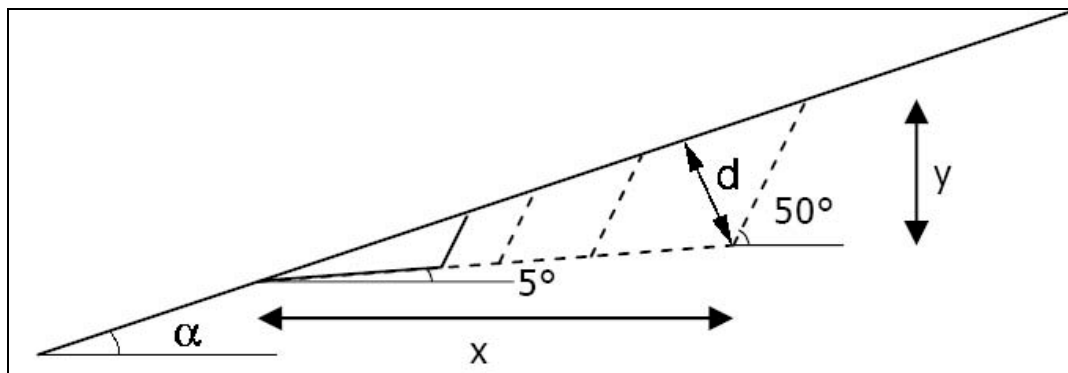


Figure 23. Schematized erosion profile of a grass-covered clay dike (from de Visser 2007).

The erosion hole geometry angles of 5 degs and 50 degs relative to horizontal are averages from the full-scale Delta Flume experiments. The horizontal length, x , and the vertical length, y , can be found geometrically in terms of maximum erosion depth perpendicular to the slope, d , and levee slope angle, α , i.e.,

$$x = \frac{d \cos(5^\circ)}{\sin(\alpha - 5^\circ)} \quad (9)$$

$$y = \frac{x[\tan \alpha - \tan(5^\circ)]}{\left(1 - \frac{\tan \alpha}{\tan(50^\circ)}\right)} \quad (10)$$

The positioning of de Visser's erosion profile with respect to the still water line was not specified. However, the full-scale tests indicated that most of the erosion occurs below the still water level as noted in Figure 11. For the purpose of this present investigation, it has been assumed that the erosion profile will be located such that the transition from the 5-deg slope to 50-deg slope is at the elevation of the still water level. In reality this transition point will probably be slightly lower than the SWL elevation assumed here.

Comparison of Clay Erosion Prediction Methods

De Visser (2007) compared her semi-quantitative clay erosion depth model to the WL|Delft Hydraulics (2006) empirical model given by Equation (8). This comparison from de Visser's thesis is reproduced in Figure 24. Four erosion prediction curves are shown for the empirical method for significant wave heights of $H_{m0} = 1.0$ m and 1.5 m (3.3 ft and 4.9 ft) and soil condition (structured or unstructured). An averaged value of the coefficient C_c was used in the empirical method. The average of the semi-quantitative model of de Visser is shown as the heavy line with the shaded area bracketing the higher and lower significant wave heights. It appears that de Visser assumed the clay profile consisted of structured clay to a depth of about 1.2 m (3.9 ft), followed by a 0.4-m-thick (1.3 ft-thick) layer of moderately structured clay before reaching unstructured clay at a depth of about 1.6 m (5.3 ft).

The empirical method predicts a lower rate of erosion for unstructured clay and a significantly higher erosion rate for structured clay when compared to the semi-quantitative model of de Visser. It is important to remember that any predictions with a duration greater than 11 hours is extrapolation of the full-scale test results into durations for which there is no validation.

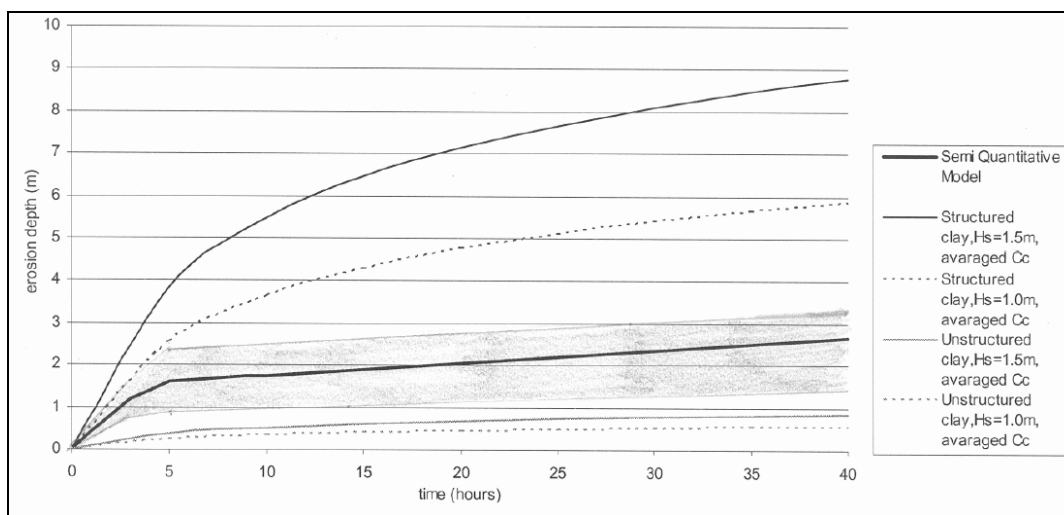


Figure 24. Comparison of semi-quantitative and empirical clay erosion depth prediction (from de Visser 2007).

A comparison of de Visser's semi-quantitative model to the INFRAM design method is shown in Figure 25 (reproduced from de Visser 2007). The de Visser model uses the same assumed soil condition profile as the previous comparison. Three curves are shown for the INFRAM design method corresponding to significant wave heights of $H_{m0} = 0.5$ m, 1.0 m, and 1.6 - 2.0 m (1.6 ft, 3.3 ft, and 5.3 - 6.6 ft).

The two prediction methods show similar estimates in the time span of about 3 hr - 10 hr, but for longer durations, the INFRAM method estimates much greater erosion depth due to the linear increase with time. De Visser (2007) noted that the INFRAM design method should contain an adequate safety margin, whereas her semi-quantitative model attempts to predict actual erosion depth. Thus, there is a concern that the INFRAM design method is not conservative for the time frames associated with many hurricanes.

Conclusions, Caveats, and Concerns

The examination of available maximum erosion depth prediction methods for grass-covered and bare-clay levee slopes exposed to direct wave action led to the following conclusions.

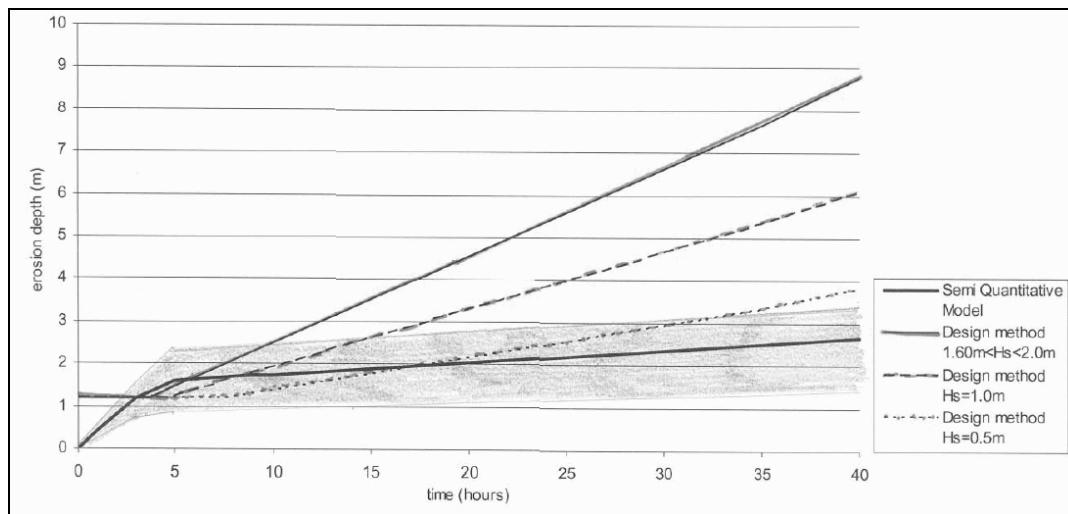


Figure 25. Comparison of semi-quantitative and INFRAM clay erosion depth prediction (from de Visser 2007).

Erosion of grass-covered slopes

- Available laboratory experiments at full-scale provided the following rule-of-thumb guidance: (a) waves up to 0.5 m (1.6 ft) caused no damage to grass covers; (b) waves in the range 0.5 - 1.5 m (1.6 - 4.9 ft) with a duration between 6 and 24 hours generally did not cause severe damage; and (c) waves greater than 1.5 m (4.9 ft) will likely cause severe erosion, but experimental data are not available to confirm this assertion.
- The full-scale tests were with good-quality grass having dense root systems. Grass quality and root density are considered more important to erosion resistance than clay type.
- Two similar equations are available for estimating maximum wave-induced erosion depth for grass cover layers over good-quality clay. One method includes peak wave period and levee slope, whereas the earlier method does not include wave period and levee slope explicitly.
- Results from either equation should be quite similar because they are based on the same laboratory experiments.
- The equations were developed from laboratory tests with significant wave heights up to $H_{m0} = 1.6$ m (5.2 ft) and peak wave periods in the range of $T_p = 4$ s to 5 s. Extrapolation to larger wave heights entails some degree of risk. However, this risk is mitigated somewhat by the fact that maximum erosion depth is assumed proportional to the square of the wave height.

- Time is a variable in the erosion prediction equations, and experiment duration varied between 8 and 29 hours. Therefore, typical hurricane wave and surge duration will fall well within this range.
- The recommended factor of safety to use in these equations is $\gamma = 2$.
- The equations include a grass-quality factor to compensate for less-than-optimum grass covers.
- Maximum erosion rates predicted for grass covers is less than predicted for bare clay, particularly for durations less than 5 hrs (see Figure 19 in the previous section).

Erosion of bare clay slopes

- Four methods for estimating maximum wave-induced erosion depth of bare clay slopes were examined. All four methods are based on the same full-scale laboratory tests.
- The tested wave conditions included significant wave heights up to $H_{m0} = 1.5$ m (4.9 ft). Peak wave periods up to $T_p = 12$ s were tested.
- Tests included clays with significant soil structure and unstructured clays having differing sand percentages.
- The greatest estimated erosion depths are found using the INFRAM design method. This method is presently used by the Dutch, and it has built-in conservatism. Thus, estimates are greater than what would actually be expected.
- The INFRAM design method provides erosion depth as a function of wave load duration for a range of wave heights. Erosion rates are linear in time even though the clay will become unstructured at greater depths, and this should cause the erosion rate to decrease.
- The WL|Delft equation for estimating maximum erosion depth assumes eroded depth is directly proportional to significant wave height, but the rate of erosion decreases in time logarithmically. This decrease in erosion rate better represents the reduced soil structure with clay layer depth. A coefficient that varies between 0.014 and 0.26 accounts for soil structure.
- De Visser's method for estimating maximum erosion depth applies different erosion rates depending on the soil structure encountered during the erosion process. Erosion rates vary between 0.4 m/hr (1.3 ft/hr) for structured clay and 1 cm/hr (0.4 inch/hr) for unstructured clay.
- De Visser's model contains no safety factor, and she advised against extrapolating the model to wave heights greater than $H_{m0} = 1.5$ m (4.9 ft).

- Comparisons between three erosion estimation methods revealed that De Visser's model predicts the least erosion when the methods are extrapolated well beyond the 11-hr duration that corresponds to the limit of the data.
- Attempts to include the influence of clay type, breaker type, and wave characteristics were not conclusive.

Below are several aspects and uncertainties that should be considered when applying the erosion depth estimation methods to the levees of the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS).

- The maximum erosion depth estimation procedures presented in this section, despite the limitations, are the best available for estimating wave impacts on grass-covered and bare levee slopes. These methods should be considered reliable when applied within the parameter ranges and soil conditions of the experimental data.
- The guidance is supported by laboratory test data up to significant wave heights of 1.5 m (4.9 ft) and peak wave periods up to about 6 sec. The design methods can be extrapolated to greater wave heights, but there are no measurements or tests to support this extrapolation.
- There will probably never be any validation of the design methods well beyond a 2-m wave height because no present-day facility exists that is capable of making waves that large. The only validation for extreme conditions would come from successful measurements during a hurricane on an actual levee. Probability of measurement success during hurricanes is exceedingly low.
- Soil structure plays an important role in the erosion rate. Newly constructed levees will have very little soil structure, but soil structure will develop over time because of continual wetting and drying (shrinking and swelling) of the clay. Therefore, older levees will erode at faster rates. On the other hand, the grass root density improves with age (up to a point), and this adds to the initial slope resiliency.

The design methods described in this section are applied to hypothetical cases in the following section to assess the need for providing wave erosion protection beyond that protection offered by well-established grass-covered slopes.

7 Is Flood-Side Armoring Needed?

The design methodologies presented in the preceding section were applied to three sets of hypothetical storm parameters to assess the need for providing wave erosion protection beyond well-established grass-covered flood-side slopes. The methodologies were also applied for the 500-year design parameters at nine specific reaches of the HSDRRS.

Assumptions

The following assumptions have been applied to all the hypothetical cases examined in this section.

- The levee cross section consists of a 1-on-4 linear slope on the flood-side and a horizontal crown having a width of 10 ft (3 m).
- The toe of the flood-side slope is at an elevation sufficiently lower than the crown elevation to allow incident waves to break directly on the slope rather than on the berm seaward of the toe.
- Levee soil is good-quality clay initially installed according to Task Force Hope and the Hurricane Protection Office specifications. This includes layered installation with each lift compacted to 90 percent of maximum density at optimal moisture content. Development of soil structure over time is considered.
- Existing levees constructed of sandy soils or hydraulically-placed materials are excluded from the analyses.
- The levee crown and landward-side slopes can tolerate massive overtopping discharges without damage. This implies armoring of the landward-side levee slopes if the average wave overtopping exceeds the acceptable limit for the 100-year design event.
- There are no T-walls, I-walls, or other type of floodwall situated on the levee crown to prevent wave overtopping.

Case 1 - Limit of the Experimental Data

The first case examined used wave conditions representing the maximums generated during the Dutch full-scale experiments on which the design methodologies are based. Estimations for these conditions will have a higher degree of reliability than estimates using parameters outside the

ranges used in the full-scale laboratory experiments. Incident wave conditions and water level parameters are given in Table 8.

Table 8. Parameters for Case 1.

Parameter	English Units	Metric Units	Comments
H_{mo} (wave height)	4.9 ft	1.5 m	
T_p (peak period)	6 s	6 s	
I_r (Iribbaren #)	1.53	1.53	Plunging breakers
R_c (freeboard)	7.6 ft	2.3 m	
q_w (wave overtopping)	0.1 ft ³ /s per ft	9.3 l/s per m	100-year overtopping criterion
T_{mar} (peak duration)	10 hours	10 hours	Peak surge elevation duration

The significant wave height for this example is the same as the 500-year design wave height being used for the Jefferson Lakefront, and it exceeds the 500-year design wave height for St. Charles Parish East Bank by about 75 percent. The example peak wave period is 3 s less than the Jefferson Lakefront design wave period, and it is about the same as the design wave period for St. Charles Parish East Bank.

The freeboard of 7.6 ft (2.3 m) was calculated using the TAW (2002) overtopping equations to be the freeboard needed for this wave condition to produce an average wave overtopping discharge of 0.1 ft³/s per ft, which is the criterion used for the 100-year design event. Reducing the freeboard would result in greater overtopping discharge (assuming no T-walls on the levee crown).

The value of $T_{max} = 10$ hr is the duration of the maximum storm surge elevation. In other words, it is assumed that the water elevation rises over time until it reaches the elevation corresponding to the 7.6-ft freeboard, and then it remains at this elevation for 10 hours before lowering. As evidenced in Section 2, which overviewed the Hurricane Katrina storm surge, this peak duration is inordinately long.

Grass-covered flood-side slope (Case 1)

The grass cover is assumed to have been established for several growing seasons so the root system has the density expected for mature grass nurtured according to the HSDRRS specifications. Maximum erosion depth as a function of wave height, wave period, grass quality and storm

Because of the long peak surge duration applied in this example, the estimated maximum erosion depths are substantially greater than the erosion depths produced in the full-scale experiments. Furthermore, the depths are well below what could be considered to be the sod layer. Thus, the sod layer would be eroded through, and the same erosion rate is being applied to the underlying clay layer. Whether or not the above estimates are valid when the erosion depth exceeds the upper sod layer is unknown because the full-scale experiments did not erode to these depths.

Using the approximate erosion profile developed by de Visser (2007) for clay erosion, the extent of damage for grass of average and grass of poor quality is shown (drawn undistorted to scale) on Figure 26. The solid blue horizontal line is the still water level that gives a freeboard of 7.6 ft. Two profiles with corresponding maximum erosion depths are shown at this elevation representing average-quality and poor-quality grass.

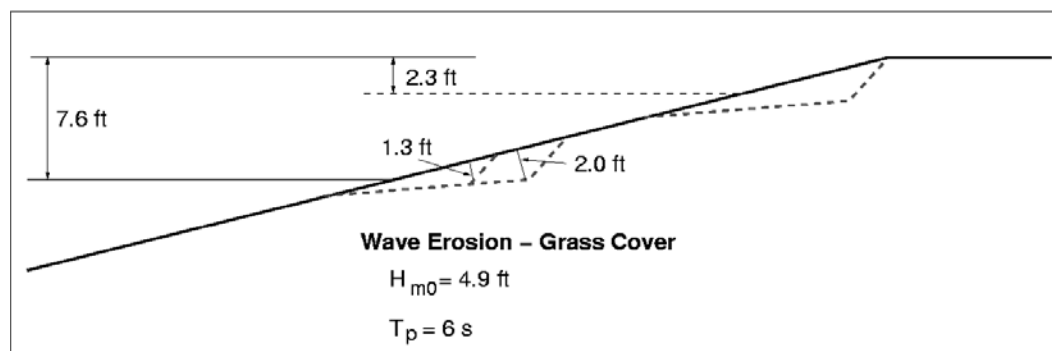


Figure 26. Estimated wave erosion of grass cover for Case 1.

If the larger of the two profiles is shifted up the flood-side slope until it just begins to encroach on the levee crown, the still water level (shown by the dashed horizontal line) would give a freeboard of only 2.3 ft. At this water level the average wave overtopping discharge would be about $2.8 \text{ ft}^3/\text{s}$ per ft, assuming the wave parameters are the same. Thus, in order for wave erosion of poor-quality grass to begin threatening the levee crown at this wave intensity, the landward-side slope (armored or not) must withstand a substantial overtopping rate.

Newly-constructed bare clay flood-side slope (Case 1)

There is a distinct possibility that a damaging hurricane could strike a newly-constructed clay levee before the grass cover layer has had sufficient time to establish a strong root system. The situation where young grass does not contribute to erosion protection might span up to a year after

construction. For analysis purposes it can be assumed the recently-placed clay is still unstructured for all but one or two inches at the surface.

The WL|Delft Hydraulics (2006) formula (Equation 8) estimates maximum erosion depth as a function significant wave height, storm duration, and clay soil structure. The clay specifications for HSDRRS restrict sand content to less than 35 percent. Table 6 recommends an average clay type factor of about $C_c = 0.02$ for unstructured clay with low sand percentage for this method. Substituting this value, along with wave height and duration, into Equation (8) yields

$$\begin{aligned} d_e &= C_c \cdot H_{m0} [-6.9 + \ln(t_s)] \\ &= 0.02 \cdot (1.5 \text{ m}) [-6.9 + \ln(36,000 \text{ s})] = 0.11 \text{ m} = 0.35 \text{ ft} \end{aligned}$$

De Visser's (2007) semi-quantitative method provided a mean erosion rate of 0.01 m/hr for unstructured clay (see Table 7). However, the upper boundary of the shaded area on Figure 22 is supposed to represent the highest wave height of 1.5 m (4.9 ft). An estimate of the erosion in 10 hrs can be extracted from Figure 22 by looking at the difference in the upper boundary of the shaded area between the times of 5 hrs and 15 hrs (10-hr duration). This difference is approximately

$$d_{\text{Visser}} = 0.25 \text{ m} = 0.82 \text{ ft}$$

The INFRAM design method is intended for analysis of structured clay layers that develop beneath stone or block revetments. Consequently, it is not appropriate for application to unstructured bare clay.

De Visser's approximate erosion profile for clay erosion is sketched to scale on Figure 27 for the de Visser's method estimate (the larger of the two estimates). The solid horizontal line is the still water level that provides a freeboard of 7.6 ft. If the water level was raised so that the up-slope erosion extent of the profile just reached the flood-side edge of the levee crown, the corresponding average wave overtopping discharge would be over 4.0 ft³/s per ft for the de Visser estimate. The displaced profile is shown on Figure 27 with the dashed horizontal line indicating the SWL necessary to have erosion beginning at the crown.

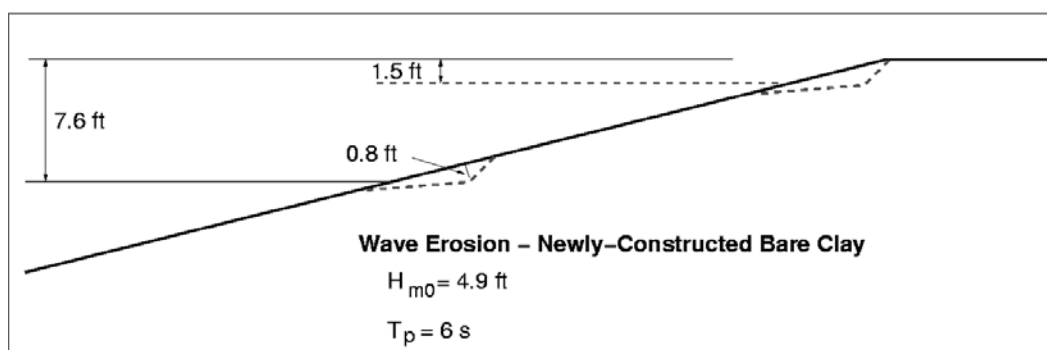


Figure 27. Estimated wave erosion of newly-constructed bare clay for Case 1.

Structured bare clay flood-side slope (Case 1)

Finally, it was assumed that over time a soil structure has developed in the clay layer. This is usually associated with clay that is overlain with a revetment, and the soil undergoes periodic wetting and drying that causes soil structure to develop over time. Weather conditions in New Orleans seem favorable for creating soil structure because of the large temperature variations and the swings between heavy rainfall and dry periods. It is not precisely clear under what circumstances bare clay with soil structure would be exposed to wave action on the flood-side slopes of the HSDSSR because we normally would expect a grass cover layer. Nevertheless, this situation can be examined using the available design methods. For this example assume soil structure has been developing for 10 years.

An estimate of maximum erosion depth using the WL|Delft Hydraulics (2006) formula, given by Equation (8), requires an appropriate value for the clay type factor. From Table 6 a value of $C_c = 0.23$ was selected as an average for structured clay. Substituting this value, along with wave height and duration, into Equation (8) yields

$$\begin{aligned} d_e &= C_c \cdot H_{m0} [-6.9 + \ln(t_s)] \\ &= 0.23 \cdot (1.5 \text{ m}) [-6.9 + \ln(36,000 \text{ s})] = 1.24 \text{ m} = 4.06 \text{ ft} \end{aligned}$$

The de Visser (2007) method accounts for the time of soil structure development, and an approximation of the depth of soil structure was made using Equation (1), i.e.,

$$d_{\text{structure}} = 0.32 \ln(t^*) + 0.24 = 0.32 \ln(10 \text{ yr}) + 0.24 = 0.98 \text{ m} = 3.2 \text{ ft}$$

The time, t_1 , to erode through the structured layer is estimated using the erosion rate for structured soils of 0.4 m/hr as shown on Table 7. Thus,

$$t_1 = \frac{0.98 \text{ m}}{0.4 \text{ m/hr}} = 2.45 \text{ hr}$$

Neglecting the minor band of moderately structured clay, the remaining 7.55 hr of the storm duration would be spent eroding the unstructured clay underneath. Rather than using the erosion rate of 0.01 m/hr shown in Table 7 for unstructured clay, a higher rate was used that corresponds to the slope of the upper portion of the gray envelope shown on Figure 22. This upper bound represents the 1.5-m wave height. This faster erosion rate was estimated to be 0.025 m/hr. So the total maximum erosion depth determined using the de Visser method was

$$d_{\text{Visser}} = 0.98 \text{ m} + (7.55 \text{ hr})(0.025 \text{ m/hr}) = 1.17 \text{ m} = 3.8 \text{ ft}$$

The overly-conservative INFRAM design method, shown on Figure 20, estimates the necessary clay thickness as the maximum erosion depth added on to an initial clay thickness of about 1.2 m (3.9 ft). Using the heavy solid line on Figure 20 for wave heights between 1.6 and 2.0 m, the maximum erosion depth after 10 hours was estimated to be

$$d_{\text{INFRAM}} = 2.5 \text{ m} - 1.2 \text{ m} = 1.3 \text{ m} = 4.3 \text{ ft}$$

It is important to keep in mind that the INFRAM design method is supposed to be purposely conservative.

De Visser's approximate erosion profile for clay erosion is sketched to scale on Figure 28 for the lowest erosion depth estimate using the WL|Delft empirical equation and the largest estimate using the INFRAM design method. The estimate using de Visser's method was between these two. The solid horizontal line is the still water level that provides a freeboard of 7.6 ft. If the water level was raised so that the up-slope erosion extent of the INFRAM estimate profile just reached the flood-side edge of the levee crown, the corresponding average wave overtopping discharge would be about 0.3 ft³/s per ft. The same vertical shift of the WL|Delft erosion profile would give a still water level that would give an average wave overtopping discharge of 1.0 ft³/s per ft.

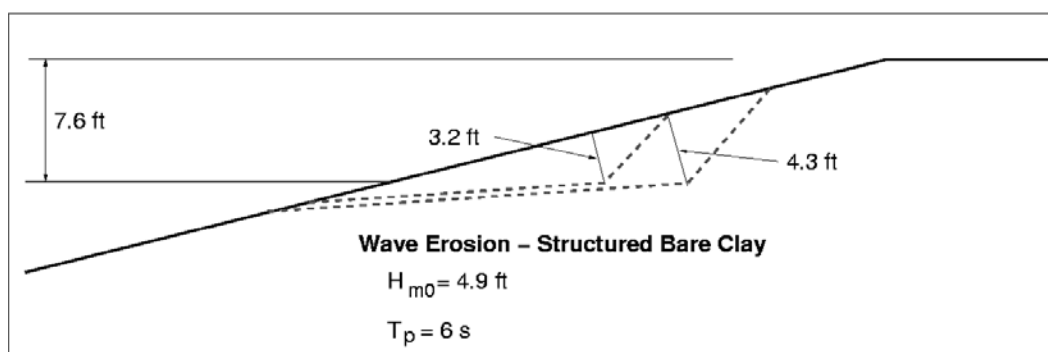


Figure 28. Estimated wave erosion of structured bare clay for Case 1.

The structured bare clay example has produced the most severe erosion estimates. However, there are not many situations where structured bare clay would exist on the HSDRRS. One possibility might be where a portion of the flood-side slope had been covered for 10 years with an articulated concrete mat, and that mat was removed either by wave action or by work crews preparing for a lift to increase levee elevation.

Case 1 discussion and caveats

Case 1 used wave conditions matching the maximum of those used in the full-scale tests that are the basis for the design methodologies. These conditions also approximate the 500-year design level for Jefferson Lakefront, and greatly exceed the 500-year design level for St. Charles Parish East Bank. Thus, the estimated erosion depths shown in Figures 26, 27, and 28 should be reasonably reliable. There are a few caveats that might add some uncertainty to these estimates. First, the grass root density achievable on the HSDRRS levee system may not be as dense as the root systems in the Dutch tests. However, this is countered somewhat by dense, unstructured, highly-compacted clay at the surface that will not be as easily eroded. It is assumed that the growth of the erosion hole on grass slopes is hindered by the strength of the adjacent sod at the edges of the hole. The vertical positioning of the assumed erosion profile on the slope may not be correct, but any error would be biased toward having the profile too high in this example.

The estimates for grass-covered slopes and for unstructured bare clay slopes show that wave-induced erosion results in damage that is no threat to the levee integrity at the water level associated with the 100-year design allowable wave overtopping criterion. In fact, before the predicted erosion would even start to damage the levee crown, the storm surge would have to rise to a level that results in wave overtopping rates of about 2.5 ft³/s per ft.

At this overtopping level, the landward-side slope may need to have robust armoring to avoid failure by wave overtopping erosion. Present experience with the Dutch Wave Overtopping Simulator indicates damage can occur to grass-only slopes at lesser wave overtopping rates, but further testing is needed to determine whether wave overtopping rates of 2.5 ft³/s will cause damage to the HSDRRS unarmored landward-side slopes.

Erosion estimates for structured clay indicate that the damage would not jeopardize the levee crown at the still water level associated with the 100-year design allowable wave overtopping discharge. However, the water level does not have to rise too much farther before the damage would reach the levee crown. Whether or not this might represent a threat to the HSDRRS levees depends on whether situations might arise where structured bare clay would be exposed to hurricane wave conditions.

Finally, the selected 10-hr storm duration is not the entire duration of the storm, but instead it is the time the still water level and corresponding wave intensity are at maximum levels. As discussed in Section 2 for Hurricane Katrina, this is long peak duration for a storm; and most hurricanes would have shorter peak surge duration. The surge elevation for Hurricane Katrina exceeded 80 percent of the peak elevation for approximately 6 hrs. The effect of a varying storm surge level with shorter peak duration would be scouring of a shallower hole that extended from a lower elevation. Such scour would have no impact on levee crown erosion potential.

Case 2 - Extreme Wave and Overtopping Condition

The second case examined extreme wave conditions representing the 500-year design condition for New Orleans East Back Levee and St. Bernard (see Section 3) at a still water elevation that would result in a large average wave overtopping discharge (see Table 9). This overtopping condition most likely would require that landward-side slopes be armored to resist damage. Unarmored landward-side slopes might fail and potentially lead to breaching under this overtopping rate if peak storm surge duration is long. (Presently, it is not possible to determine precisely the conditions under which landward-side beaching might occur.) Erosion estimates on the flood-side slope for these extreme conditions will have a higher degree of uncertainty because the design methodologies are being extrapolated and applied outside the range of the full-scale laboratory data on which they are based.

Table 9. Parameters for Case 2.

Parameter	English Units	Metric Units	Comments
H_{m0} (wave height)	8.0 ft	2.44 m	
T_p (peak period)	12 s	12 s	
I_r (Iribbaren #)	2.4	2.4	Plunging breakers
R_c (freeboard)	7.6 ft	2.3 m	
q_w (wave overtopping)	2.9 ft ³ /s per ft	270. l/s per m	Beyond 500-year criterion
T_{max} (peak duration)	10 hours	10 hours	Peak surge elevation duration

The peak wave period for this example is near the limit of plunging breakers. Waves with longer periods will begin to break as collapsing or surging breakers, and these would be less damaging to grass and clay slopes. The same freeboard of 7.6 ft (2.3 m) calculated for Case 1 was used for Case 2. This is an arbitrary freeboard selection. Increasing or decreasing the freeboard would result in a downward or upward shift, respectively, of the estimated erosion profile. The value of average wave overtopping discharge listed in Table 9 was calculated using the TAW (2002) overtopping equations for the selected freeboard and wave conditions and assuming no T-wall on the levee crown. A decrease in freeboard would result in greater wave overtopping.

Grass-covered flood-side slope (Case 2)

The grass cover is assumed to have been established for several growing seasons so the root system has the density expected for mature grass nurtured according to the HSDRRS specifications. Using the TAW (2004) empirical equation (Equation 3) in metric units, the wave height adjustment factor, δ , was determined using Equation (5), i.e.,

$$\delta = 0.5 \frac{(T_p)^{1/2}}{(H_{m0})^{1/4}} = 0.5 \frac{\text{m}^{1/4}}{\text{s}^{1/2}} \frac{(12\text{s})^{1/2}}{(2.44\text{ m})^{1/4}} = 1.39$$

The adjusted wave height was calculated using Equation (4)

$$H\delta H_{m0} = 1.39 (2.44\text{ m}) = 3.39\text{ m}$$

Using the recommended safety coefficient of $\gamma = 2$, and setting the grass quality factor, C_E , to the mean value for the range given for each of the

three grass qualities; the maximum erosion depths were calculated using Equation (3) as follows.

$$d = 3,600 \frac{s}{hr} (2) \left[\frac{1}{4} t_r \tan \theta \right]_{\max}^2$$

Grass of good quality:

$$d = 3,600 \frac{s}{hr} (2) \left[1.0(10^{-6}) m^{-1} s^{-1} \right] \left[4(3.39 m) (1/4) \right]^2 (10 hr) = 0.83 m = 2.7 ft$$

Grass of average quality:

$$d = 3,600 \frac{s}{hr} (2) \left[2.0(10^{-6}) m^{-1} s^{-1} \right] \left[4(3.39 m) (1/4) \right]^2 (10 hr) = 1.66 m = 5.4 ft$$

Grass of poor quality:

$$d = 3,600 \frac{s}{hr} (2) \left[3.0(10^{-6}) m^{-1} s^{-1} \right] \left[4(3.39 m) (1/4) \right]^2 (10 hr) = 2.48 m = 8.1 ft$$

As noted for Case 1, the maximum erosion depths estimated for Case 2 are much deeper than the sod layer. Thus, the equations are being applied well outside the range of the full-scale experimental results. Consequently, there is substantial uncertainty as to the veracity of these estimates.

The erosion profiles for good-quality and average-quality grass are shown drawn to scale on Figure 29. The solid blue horizontal line is the still water level that gives a freeboard of 7.6 ft. The profile for poor-quality grass is not shown because it will erode a portion of the levee crown, and that would be considered failure.

Erosion estimates for $H_{m0} = 8$ ft are significantly greater than the estimates for $H_{m0} = 4.9$ ft because the TAW (2004) erosion equation assumes erosion depth is proportional to the square of the wave height. Nevertheless, this extreme condition is estimated to be fatal only when the grass cover is of poor quality. In addition, peak storm surges of 10 hrs in duration would be considered rare for a storm of this size. A peak storm

surge lasting 5 hrs would create an erosion profile for poor-quality grass cover somewhere between the two profiles shown on Figure 29.

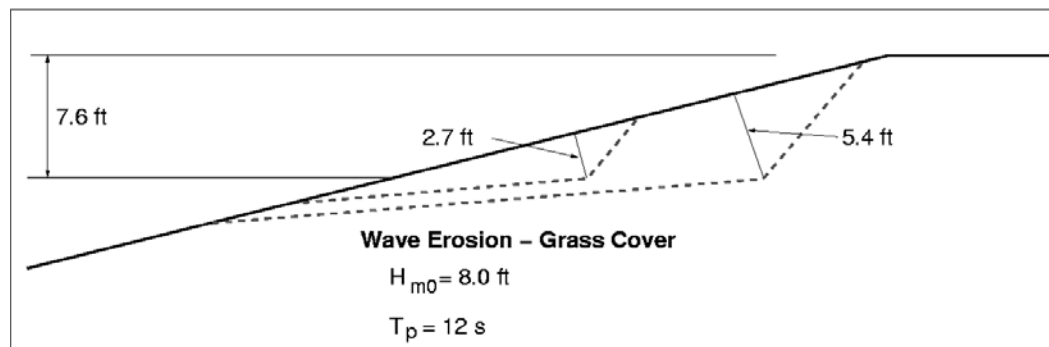


Figure 29. Estimated wave erosion of grass cover for Case 2.

Newly-constructed bare clay flood-side slope (Case 2)

The WL|Delft Hydraulics (2006) formula given by Equation (8) was applied for this extreme wave condition using the recommended average value of clay type factor of about $C_c = 0.02$ for unstructured clay with low sand percentage (see Table 6). Substituting this value, along with wave height and duration, into Equation (8) yields

$$\begin{aligned} d_e &= C_c \cdot H_{m0} [-6.9 + \ln(t_s)] \\ &= 0.02 \cdot (2.44 \text{ m}) [-6.9 + \ln(36,000 \text{ s})] = 0.17 \text{ m} = 0.58 \text{ ft} \end{aligned}$$

De Visser's (2007) analysis provided a mean erosion rate of 0.01 m/hr for unstructured clay (see Table 7). However, the upper boundary of the shaded area on Figure 22 is supposed to represent the highest wave height of 1.5 m (4.9 ft). The slope of this upper boundary gives an erosion rate of about 0.025 m/hr. If we assume the erosion rate is directly proportional to significant wave height, it might be reasonable to scale the erosion rate by the ratio of wave heights, i.e., $(2.44 \text{ m} / 1.5 \text{ m} = 1.62)$ for this case. So the erosion rate would then become about 0.04 m/hr, and the maximum erosion depth after 10 hr would be.

$$d_{\text{Visser}} = 0.04 \text{ m / hr} (10 \text{ hr}) = 0.4 \text{ m} = 1.3 \text{ ft}$$

The approximate erosion profile for clay erosion is sketched to scale on Figure 30 for the de Visser's method estimate (the larger of the two estimates). The solid horizontal line is the still water level that provides a freeboard of 7.6 ft. The estimated erosion is very minor and represents no

threat at all to levee integrity. The most immediate question is why the maximum erosion depth is so much less than the depth predicted for a grass-covered slope that in theory should be stronger. The reason for this disparity is the fact that the unstructured clay design methods assume erosion depth is proportional to wave height whereas the grass-cover estimation method has erosion depth proportional to the wave height squared. The dilemma about which is correct cannot be answered for waves heights greater than 1.5 m (4.9 ft) because no data exist to prove which is correct. A conservative approach would be to use the estimates for grass cover when evaluating levee safety.

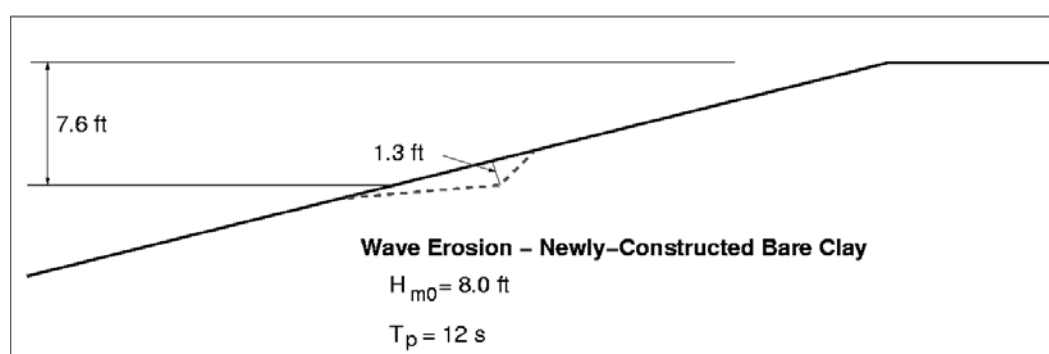


Figure 30. Estimated wave erosion of newly-constructed bare clay for Case 2.

Structured bare clay flood-side slope (Case 2)

For this example assume, once again, soil structure has been developing for 10 years prior to arrival of the extreme storm. From Table 6 a value of $C_c = 0.23$ was selected as an average for structured clay, the same as assumed for Case 1. Substituting this value, along with wave height and duration, into the WL|Delft Hydraulics formula (Equation 8) yields

$$d_e = C_c \cdot H_{m0} [-6.9 + \ln(t_s)]$$

$$= 0.23 \cdot (2.44 \text{ m}) [-6.9 + \ln(36,000 \text{ s})] = 2.0 \text{ m} = 6.6 \text{ ft}$$

This estimated erosion depth is about twice the 3.2-ft depth of soil structure estimated for 10 years (see Case 1 example). Therefore, the maximum depth estimate should be considered excessive because the erosion rate for structured soil is also applied to unstructured soil at depths beyond 3.2 ft.

The de Visser (2007) method accounts for the time of soil structure development; and from the Case 1 example, the depth over which soil

structure develops was estimated to be 0.98 m (3.2 ft). Assuming the rate of structured soil erosion for the larger wave height is approximately twice the 0.4 m/hr rate given in Table 7; the structured soil should be eroded in about

$$t_1 = \frac{0.98 \text{ m}}{0.8 \text{ m/hr}} = 1.2 \text{ hr}$$

Neglecting the minor band of moderately structured clay, the remaining 8.8 hr of the storm duration is spent eroding the unstructured clay. The erosion rate for unstructured clay was estimated to be 0.04 m/hr for this extreme wave height (see unstructured bare clay example for Case 1), so the total maximum erosion depth determined using the de Visser method is

$$d_{\text{Visser}} = 0.98 \text{ m} + (8.8 \text{ hr})(0.04 \text{ m/hr}) = 1.35 \text{ m} = 4.4 \text{ ft}$$

The overly-conservative INFRAM design method, shown on Figure 20, estimates the necessary clay thickness as the maximum erosion depth added on to an initial clay thickness of about 1.2 m (3.9 ft). First, assume the heavy solid line on Figure 20 for wave heights between 1.6 and 2.0 m represents $H_{m0} = 1.6 \text{ m}$. This line has a slope of 0.21 m/hr (see Table 4). If erosion depth is proportional to wave height, then the slope of a line representing $H_{m0} = 2.44 \text{ m}$ would give an erosion rate of

$$e_{\text{INFRAM}} = (0.21 \text{ m/hr}) \left(\frac{2.44 \text{ m}}{1.6 \text{ m}} \right) = 0.32 \text{ m/hr}$$

The maximum erosion depth after 10 hours is estimated to be

$$d_{\text{INFRAM}} = 0.32 \text{ m/h} (10 \text{ hr}) = 3.2 \text{ m} = 10.5 \text{ ft}$$

It is important to keep in mind that the INFRAM design method is purposely conservative, and the soil structure exists only in the first 3.2 ft of eroded depth. The remaining clay would be unstructured, and it would erode at a much slower rate than used in the INFRAM design method.

Figure 31 shows the empirical erosion profile plotted to scale for the WL|Delft empirical equation and for de Visser's method as applied to structured soil. The INFRAM design method is not plotted because the erosion estimated by this method would breach the levee crown. As seen in

the figure, damage from the de Visser method (which is the most reliable estimate) does not encroach on the levee crown, whereas the WL|Delft equation estimate ended up eroding approximately 4.5 ft of the horizontal crown.

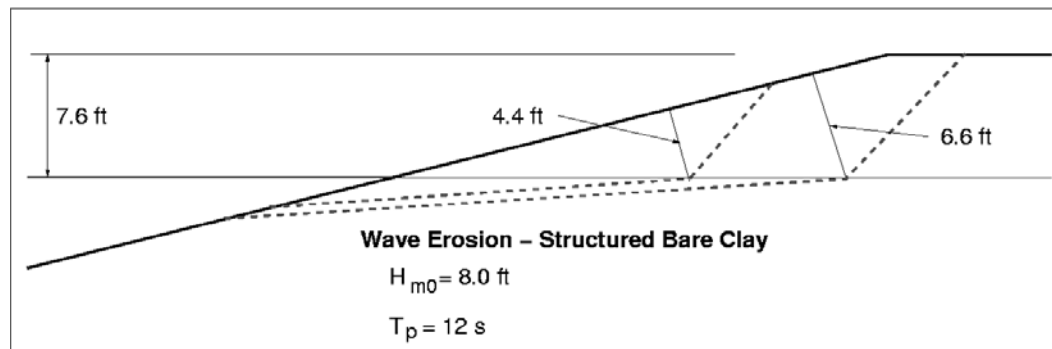


Figure 31. Estimated wave erosion of structured bare clay for Case 2.

As in Case 1, the estimated erosion for this extreme set of wave conditions was worst for structured clay which may, in fact, not be an issue with the levees of the HSDRRS unless reaches of levee with a substantial depth of soil structure are anticipated.

Case 2 discussion and caveats

Case 2 used extreme wave conditions well outside the range of waves that were used to establish the various erosion estimation methodologies. Thus, the estimated erosion depths plotted in Figures 29, 30, and 31 have substantially more uncertainty than the estimates of Case 1.

In addition to the caveats listed for Case 1 (grass root density, contribution of grass cover adjacent to erosion holes, and vertical positioning of the erosion profile) there were a few assumptions that add greater uncertainty to these estimates. The much longer wave period is only included in the grass-cover estimation equation, but its effect has not been substantiated. Wave period does not factor in any of the bare clay estimation techniques. (Period does contribute significantly to the estimate of overtopping rates.) Extrapolation of the methods for bare-clay erosion assumed that erosion rates were proportional to wave height instead of the square of the wave height.

Damage estimates for grass-covered slopes were considerably greater than those calculated for unstructured bare-clay slopes. The reason for this disparity is that the grass-cover equation assumes erosion depth is

proportional to wave height squared. Whether or not this is a correct assumption cannot be determined with the present data and understanding. Nevertheless, for these extreme conditions the levee crown was never threatened for the unstructured bare case example using the WL|Delft Hydraulics (2006) method; and the only instance of predicted crown damage was for poor-quality grass.

Two of the three erosion estimates for structured bare clay indicated that the crown would suffer damage at the selected still water level. However, the de Visser (2007) method did not yield crown damage, and this method should be considered the most reliable, even though it was extrapolated beyond wave heights of 1.5 m (4.9 ft) against de Visser's advice. As discussed for Case 1, it still is not clear whether or not bare structured clay slopes would be exposed on the levees of the HSDRRS. The two methods that produced crown damage implicitly assumed soil structure extending to improbable depths.

The selected 10-hr duration of peak storm surge is much longer than expected for major Hurricanes as evidenced by the peak storm surge durations experienced during Hurricane Katrina. If the peak surge duration is reduced to 5 hrs, none of the estimation methods would produce an erosion profile that would reach the levee crown for the Case 2 conditions.

Finally, the overtopping rate for the Case 2 examples suggests that the landward-side slope may need armoring beyond a grass cover layer to avoid failure (assuming there are no T-walls on the levee crown). Furthermore, the water depth necessary for 8-ft waves to break directly on the slope would be about 13 ft. Adding the 7.6-ft freeboard means that the vertical distance between the crown and the toe of the flood-side slope would have to be at least 20 ft for this scenario to occur. If this differential is less than 20 ft, waves will break on the flood-side berm, and the resulting erosion will be substantially less. As noted in the section describing the 500-year wave conditions for the HSDRRS, most of the waves are depth-limited, and the maximum significant wave height is about 8.8 ft (2.7 m) on the New Orleans East Bank levee and the St. Bernard levees.

Case 3 - Time-Varying Wave and Overtopping Condition

The third case is a more realistic hypothetical hurricane simulation with time-varying wave and surge level parameters approximating the 500-year design condition for the section of the MRGO - Lake Borgne levee referred

to as SB16. Table 10 gives the storm parameters associated with the peak of the storm. The average wave overtopping rate was estimated using the equations for wave overtopping recommended in TAW (2002).

Table 10. Peak storm parameters for Case 3.

Parameter	English Units	Metric Units	Comments
H_{mo} (wave height)	8.4 ft	2.6 m	Maximum of the storm
T_p (peak period)	10.6 s	10.6 s	Maximum of the storm
I_r (Iribbaren #)	2.07	2.07	Plunging breakers
h_{crown} (crown elev.)	26.5 ft	8.1 m	
h_{surge} (surge elev.)	21.3 ft	6.5 m	
R_c (freeboard)	5.2 ft	1.6 m	$h_{crown} - h_{surge}$ (minimum)
q_w (wave overtopping)	6.6 ft ³ /s per ft	0.61 m ³ /s per m	Beyond 500-year criterion
T_{max} (peak duration)	4 hours	4 hours	Peak surge elevation duration

The overtopping discharge at the peak of the storm is extreme the calculation neglects any T-walls that are planned for the levee crown, and without T-walls it most likely would require that landward-side slopes be armored to resist damage. Unarmored landward-side slopes might fail and potentially lead to breaching under this overtopping rate. Perhaps more problematic is the flooding caused by this overtopping rate. With 4-hr duration, the overtopping water volume would be 95,040 ft³ per ft of levee; and this would fill an area extending a distance of three miles from the landward-side toe of the levee to an average depth of 6 ft.

For this hypothetical case the storm surge, significant wave height, and spectral peak wave period were assumed to vary in time as happens during hurricanes. The storm surge hydrographs from Hurricane Katrina presented in Section 2 were examined, and continuous rise and fall of surge level was approximated as a series of discrete steps occurring over similar times as illustrated in the upper plot of Figure 32. Significant wave and peak spectral wave period were also assumed to vary stepwise in time as shown in the lower plots of Figure 32. The parameter values at step increments 1 - 5 are given in Table 11. After Step 5 the hurricane is abating and any additional erosion as the surge level drops will not further endanger the levee. The rightmost column of Table 11 shows that wave overtopping does not begin until Step 3 when the freeboard is 11.5 ft (3.5 m).

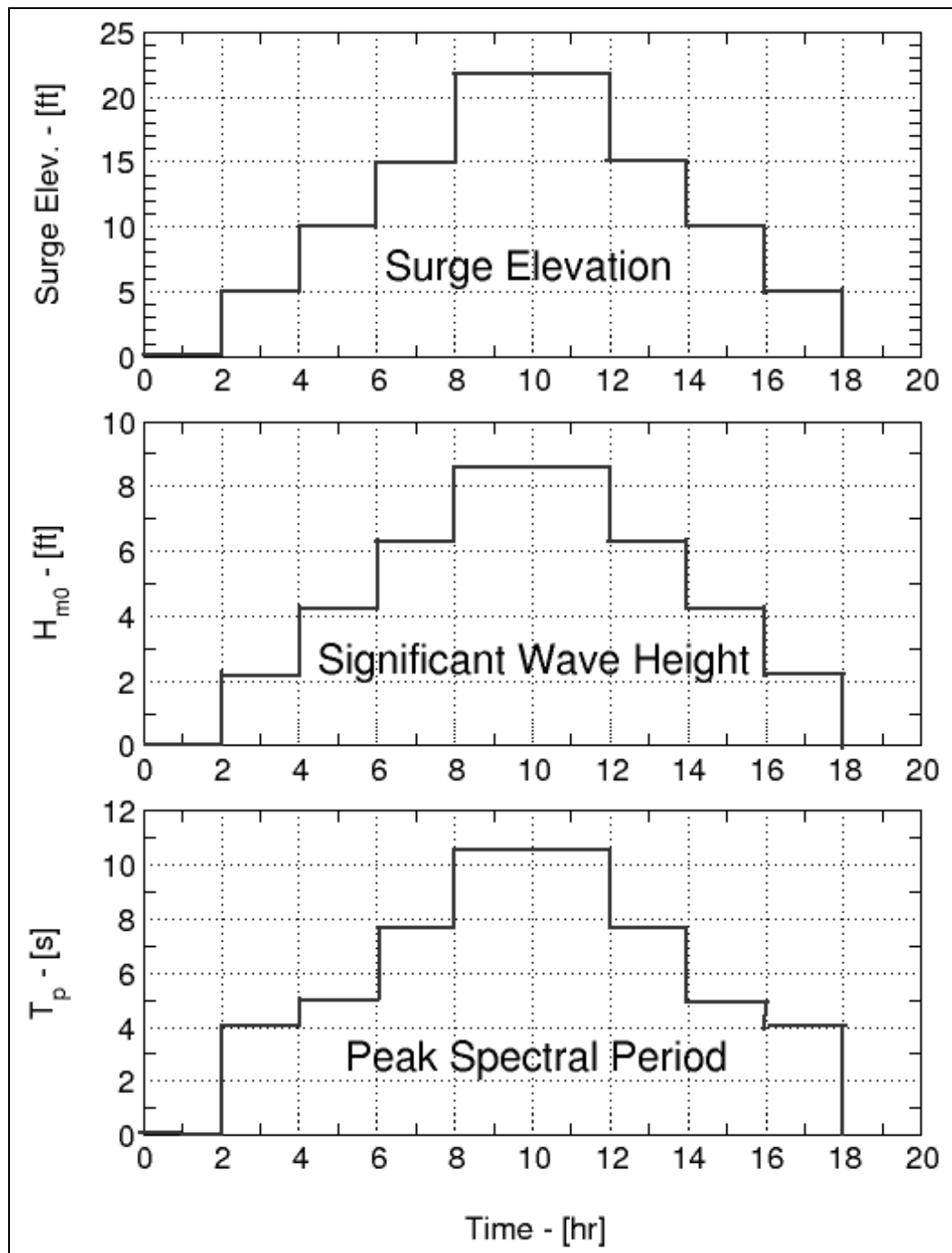


Figure 32. Time-varying parameters for Case 3.

Table 11. Time-varying storm parameters for Case 3.

Step	Wave Ht.	Wave Per.	Surge	Freeboard	Duration	Iribarren	Discharge
	H_{mo} (ft)	T_p (sec)	h_{surge} (ft)	R_c (ft)	T_{step} (hr)	$I_r \#$	q_w (ft ³ /s/ft)
1	2.1	4.0	5.0	21.5	2.0	1.56	0.00
2	4.2	5.0	10.0	16.5	2.0	1.38	0.00
3	6.3	7.8	15.0	11.5	2.0	1.76	0.14
4	8.4	10.6	21.3	5.2	4.0	2.01	6.66
5	6.3	7.8	15.0	11.5	2.0	1.76	0.14

As in Case 2, erosion estimates on the flood-side slope for the highest wave conditions will have a higher degree of uncertainty because the design methodologies are being extrapolated and applied outside the range of the full-scale laboratory data on which they are based. The peak of the storm lasts 4 hrs, whereas the other steps have 2-hr durations.

Grass-covered flood-side slope (Case 3)

The grass cover is assumed to have been established for several growing seasons so the root system has the density expected for mature grass nurtured according to the HSDRRS specifications. Using the TAW (2004) empirical equations in metric units for each step, the wave height adjustment factor, δ , was determined using Equation (5); the adjusted wave height was calculated using Equation (4); and the maximum erosion depth was estimated using Equation (3). The recommended factor of safety ($\gamma = 2$) was used, and the grass quality factor, C_E , was taken to be the mean value for the three grass designations (Good, Average, and Poor) the same as for Cases 1 and 2.

Calculated maximum erosion depths at each step for the three grass categories are shown in Table 12. The maximum erosion depth for each category of grass occurs during the 4-hr duration at the peak of the hurricane.

Figure 33 shows de Visser's (2007) erosion profile geometry for the poor-quality grass prediction located on the levee profile at the minimum freeboard elevation of 5.2 ft (1.6 m) below the levee crown. The still water level intersects the profile at the apex of the profile slope transition. Even for the worst case, the erosion profile does not threaten the levee crown.

Table 12. Grass-cover erosion depths for Case 3.

Step	Freeboard	Duration	Maximum Erosion Depth - d (ft)		
	R_c (ft)	T_{step} (hr)	Good Grass	Average Grass	Poor Grass
1	21.5	2.0	0.02	0.05	0.07
2	16.5	2.0	0.09	0.17	0.26
3	11.5	2.0	0.25	0.49	0.74
4	5.2	4.0	1.03	2.05	3.08
5	11.5	2.0	0.25	0.49	0.74

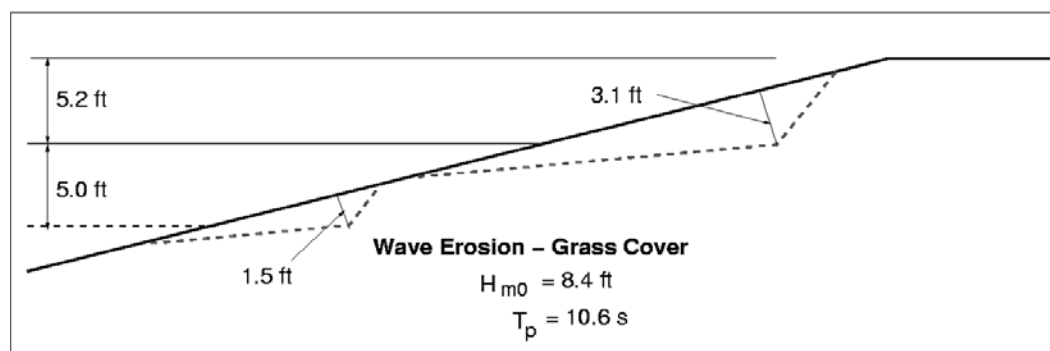


Figure 33. Estimated wave erosion of grass cover for Case 3.

The erosion profile sketched farther down the slope shows the combined erosion of Step 3 during rising surge and Step 5 during falling surge. In reality, the surge level is a continuous rise, and the actual erosion profile will have a continuous erosion profile linking the two profiles shown on Figure 33. However, the overall depth of the continuous erosion profile will be less than the depths shown for the profiles calculated with the water level constant for 2- or 4-hr durations.

Unstructured and structured bare clay flood-side slope (Case 3)

The same erosion prediction methodologies described for unstructured and structured bare clay levees subjected to wave action have been applied to the stepwise hypothetical storm parameters for Case 3. Each of the formulas or methods used the same assumptions given for the previous cases, and the same assumed soil structure development was used for applying de Visser's (2007) semi-quantitative method. The calculation results are summarized in Table 13.

Table 13. Bare clay erosion depths for Case 3.

Step	Freeboard	Duration	Unstructured Clay - d		Structured Clay - d		
	R_c (ft)	T_{step} (hr)	WL Delft (ft)	de Visser (ft)	WL Delft (ft)	de Visser (ft)	INFRAM (ft)
1	21.5	2.0	0.08	0.07	0.96	2.62	1.64
2	16.5	2.0	0.17	0.14	1.91	2.62	1.64
3	11.5	2.0	0.25	0.21	2.87	2.62	1.65
4	5.2	4.0	0.45	0.56	5.17	3.43	4.41
5	11.5	2.0	0.25	0.21	2.87	2.62	1.65

For unstructured bare clay, representing newly-constructed levees, both the WL|Delft Hydraulics (2006) method given by Equation (8), and de Visser's (2007) method produced nearly identical results. The maximum erosion depth is around 6 inch, and there is no threat to the levee crown.

The structured soil example assumes soil structure exists to a depth of 3.2 ft (1.0 m). This fact is only pertinent to the de Visser method, and her method applies a slower erosion rate once the depth of erosion equals the depth of soil structure. The WL|Delft Hydraulics and the INFRAM methods, on the other hand, implicitly assume the soil is structured to the maximum erosion depth, even if the erosion depth exceeds the stated depth of soil structure. Therefore, the WL|Delft Hydraulics maximum erosion estimate of 5.2 ft (1.6 m) and the INFRAM estimate of 4.4 ft (1.3 m) can be considered too large because the rate of erosion will decrease as the erosion depth surpasses 3.2 ft (0.98 m). Figure 34 shows de Visser's erosion profile for the WL|Delft Hydraulics estimate (5.2 ft), and the de Visser estimate (3.4 ft). The WL|Delft Hydraulics erosion profile is encroaching on the levee crown, whereas the de Visser erosion profile does not encroach.

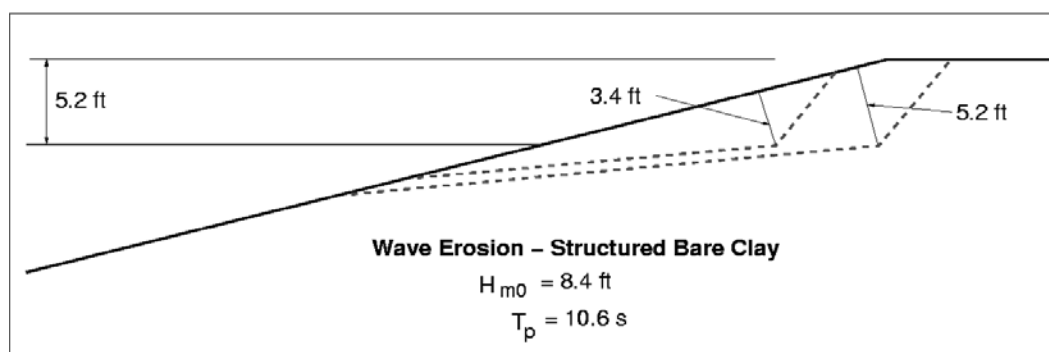


Figure 34. Estimated wave erosion of structured clay for Case 3.

Case 3 discussion and caveats

Case 3 applied a more realistic hypothetical hurricane scenario for the most exposed portion of the MRGO - Lake Borgne levee. At the maximum surge level, the projected wave overtopping quantities are massive, and the landward-side slope would most likely require armoring of some type. The storm hydrograph was approximated as a series of discrete steps having 2-hr durations. The rate and total rise of water level were selected to mimic the measured surge rises recorded during Hurricane Katrina. Wave height and period were varied in a similar manner. The only part of the calculations that exceeded the range of the experimental data used to develop the various formulations was at the peak of the storm. The same caveats listed for Cases 1 and 2 (grass root density, contribution of grass cover adjacent to erosion holes, vertical positioning of the erosion profile, longer wave period, and extrapolation of erosion rates) also apply to the Case 3 erosion estimates.

Damage estimates for grass-covered slopes were considerably greater than those calculated for unstructured bare-clay slopes. The reason for this disparity is that the grass-cover equation assumes erosion depth is proportional to wave height squared. Whether or not this is a correct assumption cannot be determined with the present data and understanding. Nevertheless, for this hypothetical hurricane the levee crown was not threatened when the flood-side slope is covered with poor-quality grass, or when the slope consisted of bare, unstructured clay.

When a flood-side levee slope consists of clay having soil structure to a depth of 3.2 ft (1.0 m), the most reliable prediction method of de Visser (2007) did not predict erosion that would threaten the levee crown. However, the other two methods predicted greater erosion depths, with the INFRAM method projecting damage to the levee crown. The two methods that predicted greater erosion depths implicitly assumed soil structure extending to depths greater than specified in the example.

The de Visser (2007) method should be considered the most reliable, even though it was extrapolated beyond wave heights of 1.5 m (4.9 ft) against de Visser's advice. As discussed for Cases 1 and 2, it still is not clear whether or not bare structured clay slopes would be exposed on the levees of the HSDRRS.

Application to Specific HSDRRS Reaches

It was suggested during initial review of this report, that the flood-side erosion methodologies described in this report be applied to the HSDRRS. Performing these calculations for all reaches of the HSDRRS is well beyond the scope of this report. However, estimates of maximum erosion depths caused by waves breaking on grass-covered and bare clay levee slopes were made for nine of the more vulnerable reaches of the HSDRRS.

Table 14 presents parameters associated with the 500-year design event at the nine HSDRRS reaches. Parameters included in the table are the significant wave height, the peak spectral wave period, the earthen levee crown elevation, and the peak surge elevation. The freeboard was calculated as the difference in the levee crown and surge elevations. It is important to keep in mind that this freeboard is to the top of the earthen levee, and it does not include any additional floodwall elevation arising from construction of a T-wall atop the levee. Likewise, the average wave overtopping discharge shown in Table 14 that was estimated at each reach using the TAW (2002) guidance does not include the effect of any flood-wall structures. These overtopping rates illustrate what would occur in the absence of additional floodwalls atop the earthen levees. (Note that reach SB16 is the reach selected for Case 3 above.)

For each HSDRRS reach listed in Table 14, maximum erosion depths were estimated for (1) grass-covered slopes using the TAW (2004) empirical equations; (2) unstructured bare clay slopes using the WL|Delft Hydraulics (2006) formula and the de Visser (2007) design method; and (3) structured bare clay slopes using the WL|Delft Hydraulics (2006) formula, the de Visser (2007) design method, and the INFRAM (2003) method. The equations were applied in the same manner as Cases 1 - 3 using the same assumptions and empirical coefficients.

Results of the erosion depth calculations are shown in Table 15 for a storm in which the peak surge duration was an improbable 10 hrs. Table 16 contains results for a more reasonable 4-hr peak storm surge duration. In both tables, calculated maximum erosion depths that resulted in an erosion profile encroaching on the levee crown are shown in boldface font.

Table 14. Parameters for 500-year design storm at specific HSDRRS reaches.

Reach	H_{mo} (ft)	H_{mo} (m)	T_p (s)	Crown Elev. (ft)	Surge Elev. (ft)	Freeboard (ft)	Overtopping Discharge (ft ³ /s per ft)
SB11 - Lake Borngé/MRGO	8.84	2.69	8.9	29.0	22.1	6.9	3.49
SB12 - Lake Borngé/MRGO	8.44	2.57	6.9	27.5	21.1	6.4	1.61
SB13 - Lake Borngé/MRGO	8.08	2.46	14.3	26.5	20.2	6.3	4.34
SB15 - Lake Borngé/MRGO	7.96	2.43	14.4	26.5	19.9	6.6	3.79
SB16 - Lake Borngé/MRGO	8.40	2.56	10.6	26.5	21.3	5.2	6.65
SB17 - Lake Borngé/MRGO	7.20	2.19	9.9	26.5	22.1	4.4	5.38
WB01 - Lake Cataouatche	3.10	0.95	7.0	11.5	9.0	2.5	0.97
SC02 - St. Charles	3.40	1.04	5.6	14.5	13.8	0.7	4.23
NE12 - New Orleans East	8.36	2.55	8.5	27.0	20.9	6.1	3.49

Table 15. Estimated maximum erosion depths (ft) for 10-hr peak storm duration.

Reach	Grass Cover Layer			Bare Unstructured Clay		Bare Structured Clay		
	Good	Average	Poor	WL Delft	de Visser	WL Delft	de Visser	INFRAM
SB11	2.33	4.65	6.98	0.63	1.47	7.30	4.33	11.60
SB12	1.69	3.38	5.07	0.61	1.41	6.97	4.28	11.08
SB13	3.26	6.53	9.79	0.58	1.35	6.67	4.23	10.61
SB15	3.22	6.43	9.65	0.57	1.33	6.57	4.22	10.45
SB16	2.55	5.11	7.66	0.60	1.40	6.94	4.27	11.03
SB17	1.90	3.80	5.70	0.52	1.20	5.95	4.12	9.45
WB01	0.38	0.76	1.14	0.22	0.52	2.56	3.61	4.07
SC02	0.35	0.70	1.05	0.24	0.57	2.81	3.64	4.46
NE12	2.04	4.08	6.13	0.60	1.39	6.91	4.27	10.97

Table 16. Estimated maximum erosion depths (ft) for 4-hr peak storm duration.

Reach	Grass Cover Layer			Bare Unstructured Clay		Bare Structured Clay		
	Good	Average	Poor	WL Delft	de Visser	WL Delft	de Visser	INFRAM
SB11	0.93	1.86	2.79	0.47	0.59	5.44	3.44	4.64
SB12	0.68	1.35	2.03	0.45	0.56	5.19	3.43	4.43
SB13	1.31	2.61	3.92	0.43	0.54	4.97	3.42	4.24
SB15	1.29	2.57	3.86	0.43	0.53	4.90	3.42	4.18
SB16	1.02	2.04	3.07	0.45	0.56	5.17	3.43	4.41
SB17	0.76	1.52	2.28	0.39	0.48	4.43	3.40	3.78
WB01	0.15	0.30	0.46	0.17	0.21	1.91	3.30	1.63
SC02	0.14	0.28	0.42	0.18	0.23	2.09	3.30	1.79
NE12	0.82	1.63	2.45	0.45	0.56	5.14	3.43	4.39

As in the previous examples, peak surge duration and freeboard are the critical parameters that determine whether or not erosion of the levee crown might occur.

For a 10-hr peak surge duration, the “good” quality grass provides adequate slope protection, but the “average” and “poor” quality grasses might not protect the levee in some reaches. Bare, unstructured clay is sufficiently robust in all cases. Results are mixed for bare, structured clay slopes. The WL|Delft Hydraulics and the INFRAM method predict crown erosion for all selected reaches, whereas the more realistic de Visser methodology shows a problem only for reaches WB01 and SCo2.

Note that erosion estimates for poor-quality grass using the TAW (2004) method are greater than estimates for structured bare clay using the WL|Delft Hydraulics methods. We would expect bare structured clay to erode more than a slope with a poor grass cover, but the calculations are contrary to this logic. The most probable explanation for this difference is the dependence of the TAW formula for grass-covered slopes on the square of the wave height.

The calculations for peak surge with a 4-hr duration mostly showed no problem with the erosion profile encroaching on the levee crown except for reaches WB01 and SCo2. Reaches SB16 and 17 might be problematic if the

WL|Delft Hydraulics method for bare, structured clay is accepted as being more accurate than the other two methods.

Summary and Conclusions

Existing methodologies for estimating the maximum depth of wave-induced erosion on the flood-side slope of earthen levees were applied for three hypothetical sets of incident wave conditions. The purpose of these example calculations was to evaluate whether or not additional armoring of the flood-side slope would be necessary to prevent breaching of the levee under these hypothetical conditions.

Assumptions

The following assumptions were applied for both hypothetical examples.

- The levee cross section consists of a 1-on-4 linear slope on the flood-side and a horizontal crown having a width of 10 ft (3 m).
- No depth-limited wave breaking occurs on the fronting berm.
- Levee soil is good-quality clay, and soil structure will develop over time.
- Existing levees constructed of sandy soils or hydraulically-placed materials are excluded from consideration.
- The levee crown and landward-side slopes can tolerate massive overtopping discharges without damage (i.e., armored crown and landward-side slopes).
- There are no T-walls, I-walls, or other type of floodwall situated on the levee crown to prevent wave overtopping.

Case 1 results

For incident wave conditions with significant wave heights up to 1.5 m (4.9 ft) and peak spectral wave periods up to 6 s, the following results were obtained for a peak storm surge elevation lasting 10 hrs.

- Grass-covered slopes will suffer some damage, and the maximum erosion depth was estimated to be 2.0 ft for grass of poor quality. At a surge elevation that produces average wave overtopping discharge of 0.1 ft³/s per ft, this maximum estimated erosion depth creates no threat of levee breaching. In order for initiation of damage to begin on the levee crown, the surge level would have to increase substantially;

and the resulting average wave overtopping discharge would be $2.8 \text{ ft}^3/\text{s}$ per ft. Overtopping rates of this magnitude for long durations (10 hours for this example) are potentially catastrophic in terms of flooding and severe damage to the landward-side slopes could occur.

- Newly-constructed bare clay slopes would have unstructured clay. The estimate of maximum erosion depth for bare clay was 0.8 ft, and this represented no threat to the levee other than minor damage to the slope. The surge level would need to rise to an elevation that produces $4.0 \text{ ft}^3/\text{s}$ per ft before the levee crown would begin to see damage.
- In the less likely case that soil structure had developed over a span of ten years, and the flood-side slope had no grass cover; the most reliable estimation procedure (de Visser's method) resulted in a maximum erosion depth of 3.8 ft. The more conservative INFRAM method estimated erosion to be 4.3 ft, but this estimate assumed that the depth of soil structure exceeded the estimate of 3.2 ft. Nevertheless, the erosion did not threaten the levee crown or represent a breaching hazard with the surge level at the 100-yr design level that results in average wave overtopping of $0.1 \text{ ft}^3/\text{s}$ per ft. The INFRAM estimate would start to erode the levee crown if the surge elevation increased a relatively small amount.

The above results are judged to be reliable because the various erosion estimation methodologies were applied inside the range of full-scale experiment parameters used to establish the guidance.

Case 2 results

For incident wave conditions with significant wave heights up to 2.44 m (8.0 ft) and peak spectral wave periods up to 12 s, the following results were obtained for a peak storm surge elevation lasting 10 hrs. This condition represents waves associated with the 500-yr design level for New Orleans East Back Levee and St. Bernard, and the surge level would result in an average wave overtopping discharge of $2.9 \text{ ft}^3/\text{s}$ per ft.

- Grass-covered slopes will suffer fairly extensive damage, and the maximum erosion depth was estimated to be 8.14 ft for grass of poor quality. At the surge elevation associated with $2.9 \text{ ft}^3/\text{s}$ per ft overtopping discharge, the levee crown would start to experience damage due to flood-side erosion. However, the estimated maximum

erosion for average and good grass covers would not be enough to result in erosion of the crown.

- Newly-constructed bare clay slopes would have unstructured clay. The estimate of maximum erosion depth was 1.3 ft, and this represented no threat to the levee other than minor damage to the slope. This estimate was significantly less than for a grass-covered slope because the bare-clay estimate assumes erosion depth is proportional to wave height instead of wave-height-squared as it is for the grass-cover estimation equations.
- For structured clay, the most reliable estimate using de Visser's method did not result in damage that threatened the levee crown, but the other two methods did show damage to the levee crown. It was noted that soil structure development would have to far exceed the estimated depth of 3.2 ft for crown damage to occur. In other words, the two methods that showed crown damage assumed the clay was more easily eroded than unstructured clay that would exist at these depths in the HSDRRS levees.

The Case 2 results are judged to be less reliable than the Case 1 results because the various erosion estimate methodologies were extrapolated and applied well outside the range of full-scale experiment parameters used to establish the guidance. The great difference between the estimates for grass-covered slope and bare-clay slopes is due entirely to the question of whether erosion depth is proportional to wave height or to wave-height-squared. This question cannot be answered with existing or future full-scale laboratory data unless huge, new wave flumes are built. The answer can only come from documented field evidence after actual severe storms.

Case 3 results

A more realistic hypothetical example used time-varying storm surge with time-varying wave conditions to represent a hurricane with parameters of the 500-year design event. These storm parameters were applied at a levee reach that is expected to have minimum freeboard at the peak of the storm. The largest significant wave height was 2.6 m (8.4 ft) and longest peak spectral wave period was 10.6 s. The following results were obtained for a peak storm surge elevation lasting 4 hrs, during which time the surge level would result in an average wave overtopping discharge of 6.6 ft³/s per ft.

- Grass-covered slopes will suffer some erosion damage, and the maximum predicted erosion depth for poor soil was estimated to be 3.1 ft (0.9 m). This erosion depth at the location on the flood-side slope associated with the maximum surge level did not threaten the levee crown. At the peak surge elevation, the 6.6 ft³/s per ft overtopping discharge would probably start to erode the landward-side slope.
- Maximum erosion depth of 0.56 ft (0.2 m) was predicted for newly-constructed bare clay slopes that have unstructured clay. This erosion represented no threat to the levee other than minor damage to the slope.
- For structured clay, the most reliable estimate using de Visser's method predicted maximum erosion depth of 3.4 ft (1.0 m) at the peak of the storm. This erosion did not threaten the levee crown at the maximum surge level. The other two methods did show start of damage to the levee crown. It was noted that soil structure development would have to exceed the estimated depth of 3.2 ft for crown damage to occur. In other words, the two methods that showed crown damage assumed the clay was more easily eroded than unstructured clay that would exist at these depths in the HSDRRS levees.

The Case 3 results are judged to be less reliable than the Case 1 results because at the peak of the storm the various erosion estimate methodologies were extrapolated and applied well outside the range of full-scale experiment parameters used to establish the guidance. However, the time-varying nature of hurricanes means that the maximum conditions persist for durations of several hours rather than the 10-hr duration assumed in Case 2. Therefore, the Case 3 example provides a more rational prediction of the flood-side erosion that might be expected for the 500-year hurricane event.

Application of all erosion depth prediction methodologies to nine specific reaches of the HSDRRS indicated that the erosion profile could encroach on the levee crown at some reaches if the peak storm surge had a 10-hr duration and the grass cover was "poor," or the slope consisted of bare, unstructured clay. Similar calculations using a peak storm surge duration of 4-hrs indicated potential levee crown erosion on only two of the nine reaches.

Conclusions

From these calculated erosion results the following conclusions are offered:

- It is concluded with a reasonable degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 1.5 m (4.9 ft), (c) the average wave overtopping discharge is less than 2.8 ft³/s per ft (assumes no floodwalls atop the levee), and (d) peak surge and wave duration is less than 10 hrs.
- It is concluded with less degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 2.44 m (8.0 ft), (c) soil structure has not developed to depths greater than 3.0 ft, (d) the average wave overtopping discharge is less than 2.9 ft³/s per ft (without floodwalls), and (e) peak surge and wave duration is less than 10 hrs.
- It is concluded that the HSDRRS levees can possibly withstand wave conditions greater than given in conclusion number 2 without flood-side armoring because of the following reasons:
 - o Erosion depths estimated using the equations for grass cover layer are most likely unrealistically large because (a) the maximum erosion depths far exceed the limit of what could be considered as the sod layer, and (b) erosion depth is assumed to be directly proportional to wave height squared in the grass cover erosion equations. The equations were developed for a wave height range of 0.75 to 1.6 m (2.5 to 5.2 ft). In this range the difference between wave height and wave height squared is not too great, and the researchers obviously achieved a better fit to the data using wave height squared. However, applying the equations at greater wave heights leads to excessive erosion rates. Full-scale laboratory data are not available to confirm or refute whether grass cover erosion rates are proportional to wave height or wave height squared.
 - o The 10-hr duration of peak storm surge and extreme wave conditions is inordinately long and not very likely to occur. Even with this long duration, the estimated erosion for unstructured clay did not threaten the levee crown except where the freeboard is very small.

- Erosion of structured soil was the worst case, but soil structure is not likely to occur to significant depths on well-compacted levees that are usually well above water level.
 - Erosion was less when the duration of the peak surge level and wave conditions was similar to the durations recorded for Hurricane Katrina. Durations of peak storm conditions shorter than 10 hrs is a realistic expectation.
 - Surge levels that allow average wave overtopping discharges greater than 3.0 ft³/s per ft are more problematic for landward-side slopes and for flooding potential. (This assumes floodwalls are not present on the levee crown.)
 - The vertical distance between levee crown and flood-side slope toe would have to be greater than 20 ft to maintain the 7.6-ft freeboard and still have waves break directly on the slope. In most cases larger waves will break on the flood-side berm, and this will decrease the erosive power of waves larger than 8 ft.
- It is concluded that existing levees constructed of hydraulically-placed sandy soils will probably need to be reconstructed with clay or be armored to prevent damage. One possibility is placing a thick clay layer of at least 1-m (3.3-ft) thickness over the existing levee similar to European practice
 - It is concluded that armoring of the flood-side slope of an earthen levee can only be justified if the landward-side slope is also armored in cases where the earthen levees do not have floodwalls on the crown. In other words, before damage on the flood-side slope could become critical, the unarmored landward-side slope will have already sustained severe damage that would lead to potential breaching.
 - It is concluded that the present-day design methodologies for estimating levee flood-side wave-induced erosion damage are not sufficient to remove all uncertainty from the above conclusions. Furthermore, there is little likelihood that these methods will be improved and verified for larger wave heights anytime in the near future.

Notes

Any influence on the erosion process caused by longer wave periods associated with open-water hurricane exposure has not been well addressed in the available design methodologies. However, longer wave periods are directly proportional to wave runup and overtopping, and this mandates greater levee crown freeboard in order to achieve the allowable wave overtopping discharge criterion. As freeboard increases, the location of wave-induced erosion on the flood side is farther down the slope and less likely to infringe on the crown.

Waves breaking on the flood-side slope will cause some damage to the grass cover and underlying clay. Damage must be repaired as soon as practical. No techniques have been developed to estimate additional damage from multiple storms when there has not been sufficient time to repair the levee between storms.

8 Summary and Conclusions

This final section summarizes the available information related to the wave-induced erosion of grass covers and underlying clay soil on the flood-side slope of earthen levees. Based on the information and calculations documented in this investigation, conclusions about the need for flood-side armoring are given and justified to the extent possible. Generally, this section gathers and condenses the summaries and conclusions previously given in individual sections of this report

Summary

The purpose of this report was to present and summarize the state of knowledge related to wave attack on the flood side of earthen levees. This information was then used to evaluate the resiliency of grass cover layers and to assess the need for providing additional flood-side slope protection beyond that afforded by grass.

Hurricane Katrina observations

Key technical reports documenting Hurricane Katrina and resulting damage to the levees of the New Orleans levee system were examined for evidence of levee failure that could be attributed to wave-induced damage of the flood-side slope. The majority assessment by the technical experts involved in preparing the reports was that earthen levee failures could be attributed primarily to erosion of the landward-side slope by overtopping waves, surge overflow, or a combination of both. Erosion by overtopping at levee transitions was also cited as a major cause of levee damage or failure.

However, Seed et al. (2006) contended that some of the catastrophic failures of the levees along the Mississippi Gulf River Outlet (MRGO) were the result of wave-induced erosion of the flood-side slope at water elevations below the levee crown elevation. They noted that the failed levees were constructed of hydraulically-placed fill material containing large percentages of sand and shell fragments. For easily eroded sandy soils, flood-side erosion could have contributed to the demise of the levee; but at the surge level needed for the flood-side erosion notch to start eroding the levee crown, wave overtopping would start to occur, and thus accelerate the breaching process by rapid erosion of the landward-side

slope. Unfortunately, these levees were completely destroyed so there is no forensic evidence to verify the claim that wave damage to the flood side of the levee was the sole cause of breaching.

Seed et al. (2006) acknowledged that adjacent grass-covered levees constructed of better clay soil withstood the same wave conditions without breaching. This field evidence suggests that levees constructed of strong cohesive soils can withstand severe hurricane wave loading on the flood-side slope without catastrophic damage, even without any armoring beyond a well-maintained grass covering.

Grass cover layers on European dikes

The cross section of a typical European dike differs from the cross section being specified for the New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS). Many European dikes have an inner core of highly-eroded sand that is protected by 1-m-thick (3.3-ft-thick) layer of stiff clay. The grass cover layer thickness varies between 6 and 14 inches (15 and 36 cm), and it is composed of topsoil with a higher sand percentage. The topsoil promotes rapid growth of a dense root system that has been shown to be primarily responsible for erosion resistance when subjected to breaking waves.

Specifications for the levees of the HSDRRS call for compacted stiff clay throughout the entire cross section with grass planted on the surface of the clay. The grass root system will probably not be as dense as found on mature European dikes; but on the other hand, the surface of the levee would be expected to have greater erosion resistance and durability than the European top soil once the grass is largely eroded away. European specifications for allowable sand content in clay are similar to the specifications being used for the HSDRRS.

Flood-side erosion resistance of grass

Most of the existing information related to erosion resistance of grass cover layers subjected to wave attack comes from three full-scale tests conducted in The Netherlands and one full-scale test conducted in Germany. These tests used actual sod containing mature grass that was harvested and transported to the testing facilities.

The typical European grass cover layer has three characteristics that help protect the underlying clay layer from erosion: (1) the flexibility and springiness of the grass cover helps absorb the high breaking wave impact pressures that would otherwise lead to damage initiation; (2) the root network helps retain soil particles from erosion by flowing water in the runup and rundown zones; and (3) grass stems and blades above the soil that help shield the soil particles from the force of flowing water. The following is a list of the more important conclusions stemming from European research into grass cover layers.

- Good grass cover grows best in a not-so-cohesive clay layer. Thus, the topsoil layer should be clay with a higher sand percentage so the grass can develop a thicker root system. After sowing on bare ground, the grass mat is at good strength after three to five years.
- Erosion of the grass cover layer is most likely to occur in the wave breaking zone just beneath the still water level where the highest wave impacts occur. Erosion due to wave runup on grass cover layers in the zone above the still water level is much less than in the breaking zone. The wave impact loading on a grass cover layer decreases as the flood-side slope decreases.
- Sea and lake dikes show no damage after waves of $H_s = 0.75$ m (2.5 ft) after 20 hrs where the grass cover layer is a closed grass mat with a high root density. This limit may be higher for dikes with milder flood-side slopes.
- Very good grass mats with underlying erosion-resistant clay can resist waves up to $H_s = 1.0$ m (3.3 ft) on a flood-side slopes of 1:3 to 1:4 with no serious damage after more than 24 hours. The damage-free period for waves of slightly more than 1.0 m was shorter, but still long enough to cope with a high water storm flood. (Duration at the peak storm surge level is an important factor.)
- The guidelines for green (unarmored) dikes suggest that flood-side slopes of 1:6 and milder can tolerate wave heights up to $H_s = 1.6$ m (5.3 ft) provided there is ample foreshore to reduce the highest possible wave.

Flood-side erosion resistance of clay

For very severe storms it is conceivable that wave impacts might erode large sections of the flood-side grass cover and root system, exposing the bare clay to the forces of impacting waves and strong flow velocities. Another possibility is that a hurricane could hit a newly-constructed levee

before grass can be established. Results from three full-scale experimental test series conducted in the Delta Flume in The Netherlands provided several insights into the resiliency of clay levee (dike) flood-side slopes when subjected to direct attack by large breaking waves. The more important conclusions are listed below.

- Soil structure is a major factor influencing erosion rates under wave attack with clay type and sand content having lesser importance. Structured clay is more easily eroded than unstructured clay, and structured clays with low sand percentage are more easily eroded than moderately structured clays with high sand percentage. Unstructured clay (compacted) having a high sand percentage may erode in clumps that break loose due to wave impacts.
- Generally, erosion rate perpendicular to the levee slope increases with wave height. However, wave period and flood-side slope are also important because all three parameters determine whether the wave breaks in plunging mode. Plunging breakers are thought to cause greater clay erosion than other breaker types because of higher wave impact pressures that impact the levee surface at nearly perpendicular angles of attack, compared to more benign water velocities that move parallel to the levee slope surface (i.e., runup and rundown).
- Bare clay has substantial erosion resistance provided the clay has limited soil structure and the sand percentage is not high.
- The poorest performing structured clay experienced a maximum erosion depth of 0.75 m (2.5 ft) after 2 hrs of waves having a wave height of 1.5 m (4.9 ft).
- The best performing soil was unstructured compacted clay. The maximum erosion depth for the unstructured clay with high sand percentage was 0.4 m (1.3 ft) after 4.4 hrs of regular waves having a wave height of 1.05 m (3.4 ft). Similar compacted clay with low sand percentage was barely eroded. (Note, however, that wave periods for these experiments were 11 s, and the waves surged on the flood-side slope rather than breaking as plunging breakers.)

Available design methodologies

Design guidance and methodologies are available for estimating erosion rates due to wave attack on flood-side grass-covered slopes and bare clay slopes. These methods are based almost exclusively on limited Dutch full-scale flume tests, and many researchers are in agreement that the methods are in need of substantial improvement. An examination was made of two

closely-related empirical relationships for estimating maximum erosion depth for grass-covered levee slopes and of four methods for estimating the maximum erosion depth of bare clay. This investigation led to the following conclusions.

Grass-covered slopes

- Available laboratory experiments at full-scale provided the following rule-of-thumb guidance: (a) waves up to 0.5 m (1.6 ft) caused no damage to grass covers, and (b) waves in the range 0.5 - 1.5 m (1.6 - 4.9 ft) with duration between 6 and 24 hours generally did not cause severe damage. Waves greater than 1.5 m (4.9 ft) will likely cause severe erosion, but experimental data are not available to support this conjecture.
- Two similar equations are available for estimating maximum wave-induced erosion depth for grass cover layers over good-quality clay. The equations were developed from laboratory tests with significant wave heights up to $H_{m0} = 1.6$ m (5.2 ft) and peak wave periods in the range of $T_p = 4$ s to 5 s. Time is a variable in the erosion prediction equations, and experiment duration varied between 8 and 29 hours.
- The recommended factor of safety to use in these equations is $\gamma = 2$, and the equations include a grass-quality factor to compensate for less-than-optimum grass covers.

Bare clay slopes

- Four methods for estimating maximum wave-induced erosion depth of bare clay slopes were examined. All four methods are based on the same full-scale laboratory test series. The tested wave conditions included significant wave heights up to $H_{m0} = 1.5$ m (4.9 ft). Peak wave periods up to $T_p = 12$ s were tested. Tests included clays with significant soil structure and unstructured clays having differing sand percentages.
- The greatest estimated erosion depths are found using the INFRAM design method. This method is presently used by the Dutch, and it has built-in conservatism. Thus, erosion estimates are greater than what would actually be expected.
- The INFRAM design method provides erosion depth as a function of wave load duration for a range of wave heights. Erosion rates are linear in time even though the clay will become unstructured at greater depths, and this should cause the erosion rate to decrease.

- The WL|Delft Hydraulics equation for estimating maximum erosion depth assumes eroded depth is directly proportional to significant wave height, but the rate of erosion decreases in time logarithmically. A coefficient accounts for soil structure.
- De Visser's method for estimating maximum erosion depth applies different erosion rates depending on the soil structure encountered at different depths during the erosion process. De Visser's model contains no safety factor, and she advised against extrapolating the model to wave heights greater than $H_{m0} = 1.5$ m (4.9 ft).
- Comparisons between three erosion estimation methods revealed that De Visser's model predicts the least erosion when the methods are extrapolated well beyond the 11-hr duration that corresponds to the limit of the full-scale data.

Hypothetical cases

The erosion estimation methodologies for wave-induced erosion discussed in this report were applied to three sets of hypothetical storm parameters to assess the need for providing wave erosion protection (i.e., armoring) on grass-covered flood-side slopes. The purpose of these example calculations was to evaluate whether or not additional armoring of the flood-side slope would be necessary to prevent breaching of the levee under these hypothetical conditions. It was assumed that the levee cross section was constructed of good-quality clay with a 1-on-4 linear flood-side slope, and waves would break directly on the slope in plunging mode.

Case 1 Results (limit of experimental data)

For incident wave conditions with significant wave heights up to 1.5 m (4.9 ft) and peak spectral wave periods up to 6 s, the following results were obtained for a peak storm surge elevation with 10-hr duration.

- Grass-covered slopes will suffer some damage, and the maximum erosion depth was estimated to be 2.0 ft for grass of poor quality. This maximum estimated erosion depth creates no threat of levee breaching.
- The estimate of maximum erosion depth for newly-constructed unstructured bare clay was 0.8 ft, and this represented no threat to the levee other than minor damage to the slope.
- In the less likely case that soil structure had developed to a depth of 3.2 ft over a span of ten years, and the flood-side slope had no grass cover; the most reliable estimation procedure (de Visser's method)

resulted in a maximum erosion depth of 3.8 ft. The more conservative INFRAM method estimated erosion to be 4.3 ft. Neither estimate represented a breaching hazard with the wave and surge condition that corresponds to the 100-year criterion for acceptable wave overtopping.

The above results were judged to be reliable because the various erosion estimate methodologies were applied within the range of full-scale experiment parameters used to establish the guidance.

Case 2 Results (500-yr design condition)

For incident wave conditions with significant wave heights up to 2.44 m (8.0 ft) and peak spectral wave periods up to 12 s, the following results were obtained for a peak storm surge elevation with 10-hr duration. This condition represents waves associated with the 500-yr design level for New Orleans East Back Levee and St. Bernard, and the surge level specified in the example would result in an average wave overtopping discharge equaling 2.9 ft³/s per ft.

- Grass-covered slopes will suffer fairly extensive damage, and the maximum erosion depth was estimated to be 8.1 ft for grass of poor quality. This erosion depth is well beyond the depths seen in the full-scale laboratory experiments. At the surge elevation associated with 2.9 ft³/s per ft overtopping discharge, the levee crown would start to experience damage due to flood-side wave erosion along with expected overtopping damage on the landward-side slope. However, the estimated maximum erosion for average and good grass covers would not be enough to result in erosion of the crown at this surge level.
- The estimate of maximum erosion depth for newly-constructed bare clay was 1.3 ft, and this represented no threat to the levee other than minor damage to the slope.
- For structured clay, the most reliable estimate using de Visser's method did not result in damage that threatened the levee crown, but the other two methods did show damage to the levee crown. It was noted that soil structure development would have to far exceed the estimated depth of 3.2 ft for levee crown damage to occur as estimated. In other words, the two methods showing crown damage assumed the clay was structured over the full erosion depth, and thus, it was more easily eroded than unstructured clay.

The Case 2 results were judged to be less reliable than the Case 1 results because the various erosion estimate methodologies were extrapolated and applied well outside the range of full-scale experiment parameters used to establish the guidance. The difference between the estimates for grass-covered slope and bare-clay slopes is due entirely to the question of whether erosion depth is proportional to wave height (assumed for clay methods) or to wave-height-squared (assumed for grass-cover methods). This question cannot be answered with existing or future full-scale experiment data unless a huge, new wave flume is built. The answer can only come from documented field evidence after actual severe storms.

Case 3 Results (Time-varying 500-yr design condition)

For time-varying storm surge and time-varying incident wave conditions with significant wave heights up to 2.6 m (8.4 ft) and peak spectral wave periods up to 10.6 s, the following results were obtained for a peak storm surge elevation with 4-hr duration. This condition represents waves associated with the 500-yr design level for MRGO - Lake Borgne levee reach SB16, and the surge level specified in the example would result in an average wave overtopping discharge equaling 6.6 ft³/s per ft.

- Grass-covered slopes will suffer some erosion damage, and the maximum predicted erosion depth for poor soil was estimated to be 3.1 ft (0.9 m). This erosion depth at the location on the flood-side slope associated with the maximum surge level did not threaten the levee crown. At the peak surge elevation, the 6.6 ft³/s per ft overtopping discharge would probably start to erode the landward-side slope.
- Maximum erosion depth of 0.56 ft (0.2 m) was predicted for newly-constructed bare clay slopes that have unstructured clay. This erosion represented no threat to the levee other than minor damage to the slope.
- For structured clay, the most reliable estimate using de Visser's method predicted maximum erosion depth of 3.4 ft (1.0 m) at the peak of the storm. This erosion did not threaten the levee crown at the maximum surge level. The other two methods did show start of damage to the levee crown. It was noted that soil structure development would have to exceed the estimated depth of 3.2 ft for crown damage to occur. In other words, the two methods that showed crown damage assumed the clay was more easily eroded than unstructured clay that would exist at these depths in the HSDRRS levees.

The Case 3 results are judged to be less reliable than the Case 1 results because at the peak of the storm the various erosion estimate methodologies were extrapolated and applied well outside the range of full-scale experiment parameters used to establish the guidance. However, the time-varying nature of hurricanes means that the maximum conditions persist for durations of several hours rather than the 10-hr duration assumed in Case 2. Therefore, the Case 3 example provides a more rational prediction of the flood-side erosion that might be expected for the 500-year hurricane event.

Application of all erosion depth prediction methodologies to nine specific reaches of the HSDRRS indicated that the erosion profile could encroach on the levee crown at some reaches if the peak storm surge had a 10-hr duration and the grass cover was “poor,” or the slope consisted of bare, unstructured clay. Similar calculations using a peak storm surge duration of 4-hrs indicated potential levee crown erosion on only two of the nine reaches.

Caveats and Uncertainties

The above-described results and conclusions from European literature and full-scale laboratory experiments are contingent on several important factors and assumptions that must be considered when transferring this knowledge to the levees of the New Orleans *Hurricane & Storm Damage Risk Reduction System* (HSDRRS).

- The importance of grass cover layers having dense root systems was emphasized in the European literature. In Europe the topsoil has greater sand content than the underlying stiff clay to promote dense root system. The HSDRRS levee specifications do not call for topsoil, and instead the entire levee cross section is built from stiff clay with a low percentage of sand. Consequently, while the stiff clay is more erosion-resistant than sandy topsoil, the root system might not develop with sufficient density to afford the level of protection observed in the European tests. The performance difference between these two types of grass cover layers is unknown.
- Most of the European tests had wave periods in the $T_p = 4 - 5$ s range. New Orleans levees are expected to resist waves with longer wave periods up to perhaps 8 - 15 s. These longer periods might result in wave impact loads that are greater than those of the European tests if the slope and wave height are such that the waves break as plunging

- breakers directly on the slope. (However, the overtopping experiments in the German tests used 10-s waves during testing that spanned several days, and the flood-side grass cover layer did not suffer severe damage.)
- Dutch research interest was limited to 1-m-thick clay cover layers because that is what they use to protect the dike sand cores. As a consequence, erosion during the experiments never progressed past an erosion depth of about 0.8 m (2.6 ft). We do not know how the erosion rates might have differed if erosion had been allowed to continue beyond that depth. It is likely that the rate of erosion will decrease with erosion depth; but until that trend is established, a conservative approach would be to do straight-line extrapolation of the erosion rates greater depths.
 - Soil structure was shown to be important. An understanding is needed about how soil structure develops over time in the climate and environment specific to the HSDRRS levee system, and how that might differ from soil structure development in The Netherlands.
 - There is no existing research that examines weak soils such as sandy silts or hydraulically-placed sand fill, other than conventional beach erosion methodologies. Whereas upgrades to the HSDRRS will identify and rehabilitate (or armor) any flood-side slopes constructed of poor soil, these levees will remain at risk until such measures are completed.
 - The maximum erosion depth estimation procedures reviewed in this report, despite the limitations, are the best available for estimating wave impacts on grass-covered and bare levee slopes. The guidance is supported by laboratory test data up to significant wave heights of 1.5 m (4.9 ft) and peak wave periods up to about 6 sec. These methods should be considered reliable when applied within the parameter ranges and soil conditions of the experimental data. The design methods can be extrapolated to greater wave heights, but there are no measurements or tests to support this extrapolation. There will never be any validation of the design methods well beyond a 2-m (6.6 ft) significant wave height because no experimental facility exists capable of making waves that large. The only validation for extreme conditions would have to come from successful measurements during a hurricane on an actual levee. Probability of measurement success during hurricanes is exceedingly low.

Conclusions

General conclusions

- Field evidence from Hurricane Katrina suggests that levees constructed of strong cohesive soils can withstand severe hurricane wave loading on the flood-side slope without catastrophic damage, even without any armoring beyond a well-maintained grass cover.
- The available full-scale laboratory tests support the contention that well-developed grass cover layers are fairly resistant to direct wave attack on slopes of 1:4 up to about $H_s = 1.0$ m (3.3 ft). Even at higher wave heights, grass cover damage is slow to develop, and what damage does occur appears not to be severe, provided good erosion-resistant clay lays beneath the grass cover and storm durations are limited to less than 10 hrs.
- Unstructured bare clay slopes, such as would exist on newly-constructed levees, can resist direct wave attack on slopes of 1:4 up to about $H_s = 1.6$ m (4.9 ft) for sustained durations. If soil structure develops over time, erosion resistance will decrease.

Conclusions regarding the need for armoring

Based on the calculations for three hypothetical examples performed in Section 7, the following conclusions were drawn:

- It was concluded with a reasonable degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 1.5 m (4.9 ft), (c) the average wave overtopping discharge is less than 2.8 ft³/s per ft (assume no floodwalls atop the levee), and (d) peak surge and wave duration is less than 10 hrs.
- It was concluded with less degree of certainty that flood-side armoring is not required anywhere on the HSDRRS where (a) the earthen levees are constructed of good-quality clay, (b) significant wave height is not expected to exceed 2.44 m (8.0 ft), (c) soil structure has not developed to depths greater than 3.0 ft, (d) the average wave overtopping discharge is less than 2.9 ft³/s per ft (without floodwalls), and (e) peak surge and wave duration is less than 10 hrs..
- It was concluded that the HSDRRS levees can possibly withstand wave conditions greater than $H_{m0} = 2.44$ m (8.0 ft) without flood-side armoring because of the following reasons:

- The maximum erosion depths for grass-covered slopes were found using equations in which the erosion depth is directly proportional to wave height squared. This proportionality is probably accurate for wave heights less than about 1.5 m (4.9 ft); but for waves greater than 2 m (6.6 ft) the erosion estimates for grass cover layers become unrealistically large compared to similar estimates for bare clay which relate erosion depth to wave height. Furthermore, there are no measurements to validate the wave-height-squared proportionality for high waves.
- The 10-hr duration of peak storm surge and extreme wave conditions is inordinately long and not very likely to occur for hurricanes. Even with this long duration, the estimated erosion for unstructured clay did not threaten the levee crown except where the freeboard was very small and flooding by wave overtopping would be problematic.
- Erosion of structured soil was the worst case, but soil structure will be slow to occur to significant depths on well-compacted levees that are usually well above water level. However, better understanding of how soil structure develops on HSDRRS levees is needed.
- Erosion was less when the duration of the peak surge level and wave conditions was similar to the durations recorded for Hurricane Katrina. Durations of peak storm conditions shorter than 10 hrs is a realistic expectation.
- Surge levels that allow average wave overtopping discharges above $3.0 \text{ ft}^3/\text{s}$ per ft become problematic for landward-side slopes, and this rate of overtopping will cause significant flooding.
- The vertical distance between levee crown and flood-side slope toe would have to be greater than 20 ft to maintain a suitable freeboard and still have waves break directly on the slope. For many levees of the HSDRRS, larger waves will break on the flood-side berm; and this will decrease the erosive power of waves larger than 8 ft.
- It was concluded that existing levees constructed of hydraulically-placed sandy soils will need to be reconstructed with clay, be armored with a thick clay layer, or be armored with some other alternative to prevent damage.

- It was concluded that armoring of the flood-side slope of an earthen levee can only be justified if the landward-side slope is also armored in cases where the earthen levees do not have floodwalls on the crown. In other words, before damage on the flood-side slope could become critical, there is reasonable expectation that the unarmored landward-side slope will have already sustained severe damage that could lead to potential breaching.
- It was concluded that the present-day design methodologies for estimating levee flood-side wave-induced erosion damage are not sufficient to remove all uncertainty from the above conclusions. Furthermore, there is little likelihood that these methods will be improved and verified for larger wave heights anytime in the near future.

References

- Burger, A. M. 1984. *Strength of the outer slope of a green dike during a super-storm flood*. Technical Report M1980 [in Dutch], Delft Hydraulics, The Netherlands.
- de Visser, M. M. 2007. *A clay layer as a revetment for sea dikes: The behavior of clay under wave loading*. Masters thesis, Delft University of Technology, The Netherlands, 247 p.
- Doorn, N. 2007. Task 4 - Understanding and predicting failure modes: Activity 4.1. Failure modes for revetments," FLOODsite Project Report, February 2007.
- ERP. 2007. *The New Orleans Hurricane Protection System: What went wrong and why*. Hurricane Katrina External Review Panel (ERP), American Society of Civil Engineers, 94 p.
- Geisenhainer, P. and H. Oumeraci. 2008. *Sea dike breach initiation and development: Large scale experiments in GWK*. FLOODsite TR-06-08-12, Germany.
- Hewlett, H. W. M, L. A. Boorman, and M. E. Bramley. 1987. *Design of reinforced grass waterways*. CIRIA Report 116 , Construction and Industry Research and Information Association, London.
- INFRAM. 2003. *Dijkbekledingen van klei, Integrale ontwerpmethod*. [in Dutch] INFRAM, i515.
- IPET. 2006. *Performance evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*. Performance Evaluation Status and Interim Results: Report 2 of a series, Interagency Performance Evaluation Task Force (IPET). Washington, D.C.: U.S. Army Corps of Engineers.
- IPET. 2007a. *Final Report of the Interagency Performance Evaluation Task Force: Volume III - The Hurricane Protection System*. Washington, D.C.: U.S. Army Corps of Engineers.
- IPET. 2007b. *Final Report of the Interagency Performance Evaluation Task Force: Volume IV - The Storm*. Washington, D.C.: U.S. Army Corps of Engineers.
- IPET. 2007c. *Final Report of the Interagency Performance Evaluation Task Force: Volume V - The Performance - Levees and Floodwalls*. Washington, D.C.: U.S. Army Corps of Engineers.
- IPET. 2009. *Final Report of the Interagency Performance Evaluation Task Force: Volume I - Executive Summary and Overview*. Washington, D.C.: U.S. Army Corps of Engineers.
- Kruse, G. A. M., and J. D. Nieuwenhuis. 2000. *Impact of weathering on erosion resistance of cohesive soil*. 8th International IAEG Congress.

- Seed, et al. 2005. *Preliminary Report on the Performance of the New Orleans Levee System in Hurricane Katrina on August 28, 2005*. UCB/CITRIS - 05/01. Funded by the National Science Foundation, American Society of Civil Engineers, and the Center for Information Technology Research in the Service of Society.
- Seed, et al. 2006. *Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina on August 29, 2005 - Volume I: Main Text and Executive Summary*. Funded by the National Science Foundation and the Center for Information Technology Research in the Service of Society.
- Seijffert, J. W., and L. Philipse. 1990. "Resistance of grassmat to wave attack," *Proceedings of the 22th International Conference Coastal Engineering*, American Society of Civil Engineers, Vol 2, pp. 1662-1674.
- Seijffert, J. W., and H. Verheij. 1998. "Grass mats and reinforcement measures," In: *Dikes and revetments*. Pilarczyk, K.W. ed. Design, maintenance and safety assessment, A. A. Balkema, Rotterdam.
- Smith, G. M., J. W. Seijffert, and J. W. van der Meer. 1994. "Erosion and overtopping of a grass dike: Large scale model tests," *Proceedings of the 24th International Conference Coastal Engineering*, American Society of Civil Engineers, (3)2639-2652.
- TAW. 1996. "Clay for dikes," Technical Report [in Dutch], Delft, The Netherlands: Technical Advisory Committee on Water Retaining Structures.
- TAW. 1997. "Erosion resistance of grassland as dike covering," Technical Report. Delft, The Netherlands: Technical Advisory Committee for Flood Defence in The Netherlands.
- TAW. 2002. Technical Report - Wave run-up and wave overtopping at dikes. Delft, The Netherlands: Technical Advisory Committee for Flood Defence in the Netherlands.
- TAW. 2004. *De veiligheid van de primaire waterkeringen in Nederland. Voorschrift Toetsen op Veiligheid voor de tweede toetsronde 2001 – 2006*. [in Dutch]. Delft, The Netherlands: Technical Advisory Committee for Flood Defence in The Netherlands.
- TAW Klei. 1996. *Technisch rapport Klei voor dijken*. [in Dutch] Delft, The Netherlands: Technical Advisory Committee for Flood Defence in The Netherlands.
- TAW VTV. 2004. *De veiligheid van de primaire waterkeringen in Nederland. Voorschrift Toetsen op Veiligheid voor de tweede toetsronde 2001 - 2006 (VTV)*. [in Dutch]. Delft, The Netherlands: Technical Advisory Committee for Flood Defence in The Netherlands.
- Van der Meer, J. W., W. Snijders, and E. Regeling. 2006. "The wave overtopping simulator," *Proceedings of the 30th International Conference Coastal Engineering*, American Society of Civil Engineers, (5)4654-4666.
- Van der Meer, J. W. 2007. *Design, construction, calibration and use of the wave overtopping simulator*. Report version 3.3 of Project 04i103 on behalf of ComCoast and Rijkswaterstaat.

- Van der Meer, J. W., G. J. Steendam, G. de Raat, and P. Bernardini. 2008. "Further developments on the wave overtopping simulator, *Proceedings of the 31st International Conference Coastal Engineering*, American Society of Civil Engineers, (4)2957-2969.
- Verheij, H. J., and D. G. Meijer. 1994. *Grass dikes - analysis of measurements from large scale tests*. Draft Technical Report Q1584 [in Dutch], Delft Hydraulics, The Netherlands.
- WL|Delft Hydraulics. 2006. Kennisleemtes Steenbekledingen, Kennisontwikkeling t.b.v. STEENTOETS2006 (in Dutch). WL| Delft Hydraulics, H4846.

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14. ABSTRACT This report is a compilation of facts and information that summarizes the present state of knowledge related to wave attack on the flood side of earthen levees. Particular emphasis was placed on the need for providing flood-side armoring (beyond the protection afforded by grass) for the New Orleans <i>Hurricane & Storm Damage Risk Reduction System</i> (HSDRRS). The report includes: (1) a summary of observations from Hurricane Katrina; (2) an extensive overview of large-scale experiments conducted in Europe, (3) a critical examination of proposed methodologies for predicting wave-induced damage on flood-side grass and bare-clay slopes, (4) an analysis of wave-induced erosion expected to occur on the flood side during hypothetical storms approximating the 100-yr and the 500-year events, (5) and a comprehensive list of conclusions and associated caveats. The erosion estimation methodologies for wave-induced erosion discussed in this report were applied to three sets of hypothetical extreme storm parameters to assess the need for providing wave erosion protection (i.e., armoring) on grass-covered and bare-clay flood-side slopes.					
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