

Physical modelling analysis of the New Orleans levee breaches

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As part of the Interagency Performance Evaluation Task Force (IPET) investigation into levee breaches in New Orleans following Hurricane Katrina, centrifuge modelling was undertaken of representative levee cross-sections on the 17th Street, Orleans and London Avenue Canals. Two mechanisms were observed leading to breaching of the levee in the models, both of which stemmed from a water-filled crack that formed in front of the flood wall. Depending on the foundation conditions and geometry of the levee and flood wall, the crack led either to a rotation of the flood wall landwards, with uplift and sliding on the top of the sand towards the landward toe of the levee, or to a translational (sliding) failure in the clay layer commencing from the bottom or toe of the flood wall. In the Orleans models no breach ensued, although it was clear these sections were close to failure. The centrifuge model tests identified, at an early stage in the IPET investigation, the importance of the 'gap' mechanism affecting the stability of the flood walls, and confirmed that levee geometry and flood wall depth of penetration, together with the underlying soil profile, were critical to the performance of the system under flood loading.

Notation

D	depth of swampy marsh
D_{50}	mean grain size
$e_{\min}, e_{\max}$	minimum and maximum void ratios
FoS	factor of safety
G	shear modulus
H	depth of water
h	depth below ground, passive side
L	length of shear surface
$1/N$	model scale
q	surcharge
S_u	undrained strength
z	depth below ground, active side
γ	unit weight
γ_w	unit weight of water
σ_h	horizontal stress
σ_v	vertical stress
σ'_v	effective vertical stress
ϕ'	limiting angle of shearing resistance

1. Introduction

The work presented in this paper forms part of an intense performance evaluation of the New Orleans and Southeast

Louisiana hurricane protection system during Hurricane Katrina. It was conducted by the Interagency Performance Evaluation Task Force (IPET, a group of government, academic and private sector scientists and engineers) from shortly after Katrina struck through the start of the next hurricane season (June 2006). Information and documentation related to the IPET efforts can be accessed on a public website, <https://IPET.wes.army.mil>.

The work presented in this paper specifically addresses the physical modelling efforts, through use of a centrifuge, that were conducted to understand and support the overall analysis of the hurricane protection system.

Using a centrifuge is a well-established technique for scale modelling of geotechnical phenomena and many facilities, large and small, exist around the world. Specific details related to the application of centrifuge modelling to the New Orleans levees are given in the IPET reports accessible from the above-mentioned website, and will not be repeated here, although a general description is provided to assist the discussion of the results. The centrifuge models were carried out at the Engineer Research & Development Center (ERDC), Vicksburg, MS, and at Rensselaer Polytechnic Institute (RPI), Troy, NY, with technical support from

engineers at GeoDelft in the Netherlands, and from consultants Steedman & Associates Ltd in the UK. Their support for this work is acknowledged.

The scale models of the levees relied on information existing prior to the hurricane and additional information collected after the hurricane. The pre-existing information consisted primarily of documents related to design and construction of the levees, boring data and geologic mapping studies (Kolb and Saucier, 1982; Eustis Engineering Company, 1984; US Army Corps of Engineers, 1989, 1990; Dunbar *et al.*, 1994, 1995). The information collected after the hurricane consists of standard penetration testing with sampling, laboratory testing, cone penetration testing, surface seismic, borehole seismic, field surveys, light detection and ranging (LIDAR) surveys, and multi-beam sonar.

2. Principles of centrifuge modelling

The use of a centrifuge is necessary to correct the distortion introduced in a scale model of a geotechnical field problem constructed ‘on the laboratory floor’. As the behaviour of soil is stress dependent, it is crucial in studying the performance of a geotechnical system such as the New Orleans levees to ensure that the correct stresses are applied to each element in the levee and foundation. This is difficult to achieve in a scale model under Earth’s gravity alone, as the weight of the model is not sufficient to bring the soil to the correct state. In a centrifuge, however, a model such as that shown in Figure 1 at a scale of $1/N$ can be subjected to a steady acceleration field equivalent to N times Earth’s gravity. In this state, the same stress conditions as exist in the field can be effectively reproduced at all points in the model. An event such as rising water leading to the onset of foundation failure in a levee system can then be modelled in a highly realistic manner. Detailed information can be collected during the model test, including measurements of water pressure in the ground at different locations, the movement of the flood wall and ground surface, and video imagery of the sequence of events. The history of centrifuge modelling and explanation of scaling principles is available in the literature: see, for example, Schofield (1980) or Taylor (1995).

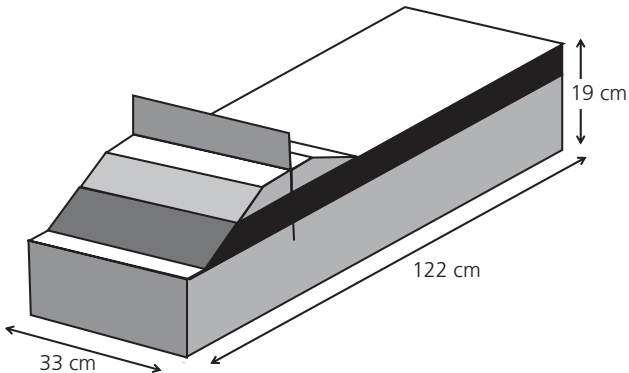


Figure 1. Typical geometry of scale model test (canal on left-hand side)

3. Summary of model tests

Models of the levee system for sections of the 17th Street Canal, London Avenue Canal and Orleans Canal are presented in this paper. The models represent the failed section of the levee for the 17th Street Canal, the two failed sections of the levee for the London Avenue Canal (termed London North and London South) and two sections of the levee for the Orleans Canal (termed Orleans North and Orleans South) where there were no failures. Results will be presented for the centrifuge modelling conducted at both ERDC and RPI. All the models are $1/50$ th scales of the actual levee, and were therefore tested under an increased acceleration field of 50 times the Earth’s normal gravity to maintain proper scaling relationships. Through the rest of this paper, all dimensions will be given in prototype scale. Table 1 summarises the models completed for this investigation.

Details and in-depth discussion regarding the specifics of each model can be found in IPET (2006). This section will present a general discussion of each of the five models: London North, London South, 17th Street, Orleans North and Orleans South.

A sketch cross-section through the London North model is shown in Figure 2a. In equivalent field dimensions, the model consisted of a 7 m thick layer of fine sand, a 1.5 m thick layer of swampy

Levee profile	Research centre	Foundation	Mechanism
London North	ERDC	Swampy marsh over sand	Rotational failure of wall
London North	RPI	Swampy marsh over sand	Rotational failure of wall
London South	ERDC	Swampy marsh over sand	Rotational failure of wall
London South	RPI	Swampy marsh over sand	Rotational failure of wall
17th Street	RPI	Swampy marsh over clay	Translational failure in clay layer
17th Street	ERDC	Swampy marsh over clay	Translational failure in clay layer
Orleans South	RPI	Swampy marsh over sand	No failure
Orleans North	ERDC	Swampy marsh over clay	No failure

Table 1. Model tests completed for IPET

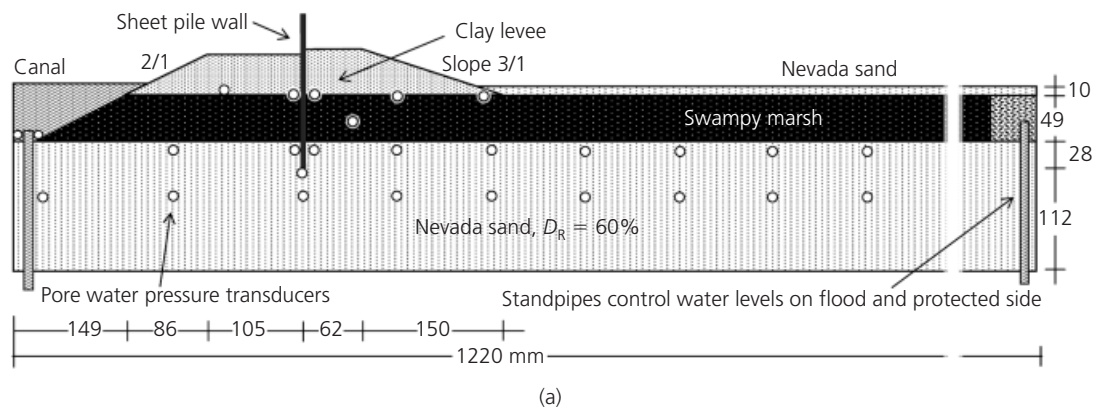


Figure 2. London North model: (a) cross-section (model units);
(b) model

marsh and a 0.5 m thick layer of fine sand on the protected side. In this figure the protected side is to the right and the canal side to the left. The levee, sheet pile and pore pressure transducers are also shown. Not shown in the figure is the laser displacement instrumentation mounted to record movements of the wall during loading. Additionally, a photograph of the completed model is seen in Figure 2b. Notice the black markers in the foundation sand layer and the white markers in the swampy marsh layer, used to track movements.

A sketch and photograph of the London South model are shown in Figures 3a and 3b. The model is almost the same as the London North model, with two notable exceptions. First, the levee has a different geometry, not being lower on the canal side than the land side. Second, the depth of sheet wall penetration into the foundation fine sand layer for this model is approximately twice what it was for the London North model (2.2 m for London South compared with 1.3 m for London North).

A sketch and photograph of the 17th Street model are shown in Figures 4a and 4b. This model has 1.5 m fine sand, 5 m of clay,

2.5 m of swampy marsh being thickest outside the levee footprint and 0.5 m of fine sand on the protected side. The fine sand on the protected side in all of the models represents the thin layer of overburden that exists in the field. This model is different from the London Avenue models in that the foundation has a large clay layer just under the swampy marsh as opposed to a large fine sand layer. Also, the difference in height of the levee from the protected to canal side of the levee is much more dramatic. The sheet pile extends just into the top of the foundation clay layer approximately 0.65 m.

A sketch and photograph of the Orleans North model are shown in Figures 5a and 5b. This model has similarities to the 17th Street model, the exception being the much thicker layer of swampy marsh in the foundation. Notice that the levee on the protected side is markedly larger than in any of the other models. In this area of the Orleans levee the sheet pile extends very deep into the swampy marsh layer.

A sketch and photograph of the Orleans South model are shown in Figures 6a and 6b. This model is similar to those for London

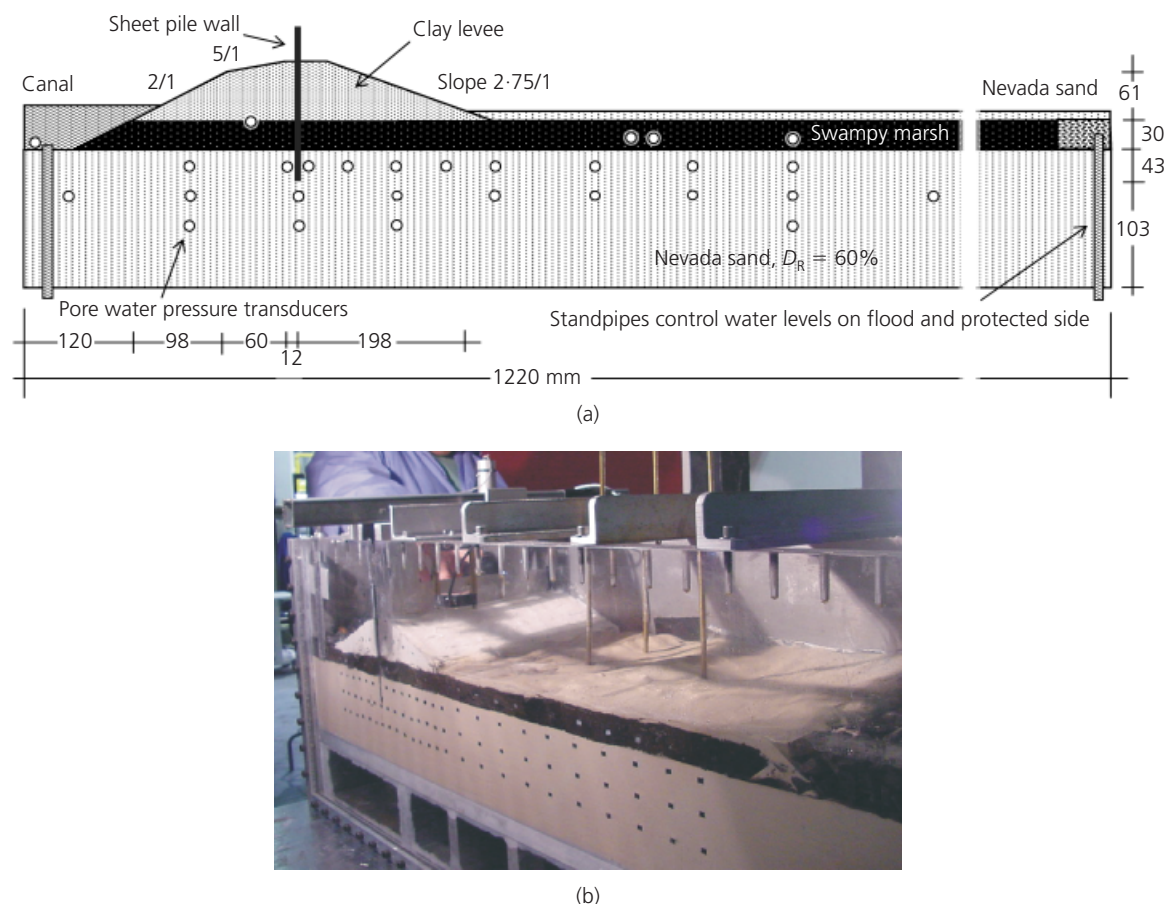


Figure 3. London South model: (a) cross-section (model units);
(b) model

Avenue, with two main differences: the levee is larger than in the London Avenue models, and the sheet pile does not extend through the swampy marsh layer into the fine sand layer.

The model tests generated large volumes of data for analysis. It was evident that in all cases where failure of the levees occurred, the failure was presaged by the opening of a water-filled crack on the canal side of the levee, which allowed full hydrostatic pressure to act on the flood wall over its entire submerged height. The flood water in the centrifuge models was raised steadily, without wave effects, and it was observed that once a crack had opened, then for the London Avenue and 17th Street canal models a failure soon followed, either by rotation of the flood wall (London Avenue) and uplift (sliding) on the top of the sand layer, or by translational failure (sliding) in the clay stratum (17th Street), originating at the bottom (toe) of the sheet pile wall.

In this paper these data are used to illustrate the mechanisms of failure observed in the models, and to draw conclusions concerning the onset of failure in the field. Analysis of the mechanisms of failure provides insights into the factor of safety (FoS) as the water levels in the canals rise.

4. Material properties

The levee and foundation in all of the models were constructed using three materials: Nevada sand to represent the natural sand, kaolin to represent the soft lacustrine clay and levee clay, and natural material from the field to represent the swampy marsh deposits. Nevada sand is well characterised as a result of its widespread use by different research laboratories. It has a specific gravity of 2.67 and maximum and minimum dry densities of 17.33 kN/m^3 and 13.87 kN/m^3 respectively at corresponding minimum and maximum void ratios of $e_{\min} = 0.511$ and $e_{\max} = 0.887$. The permeability of Nevada sand in a medium dense state is around $5.6 \times 10^{-5} \text{ m/s}$, and its mean grain size (D_{50}) is 0.15 mm. The material selected to model the soft, normally consolidated 'lacustrine' clay stratum and the stiffer clay that forms the levees was a speswhite kaolin clay, widely used in geotechnical research laboratories. Kaolin is a coarse-grained clay with a specific gravity of 2.58. For normally consolidated or lightly overconsolidated clays the ratio of undrained strength S_u to the effective overburden stress, S_u/σ'_v , has been shown empirically to lie in the range 0.2–0.5, and for kaolin clay a value of 0.22 is typical. For overconsolidated clays this ratio depends on the overconsolidation ratio, and is larger.

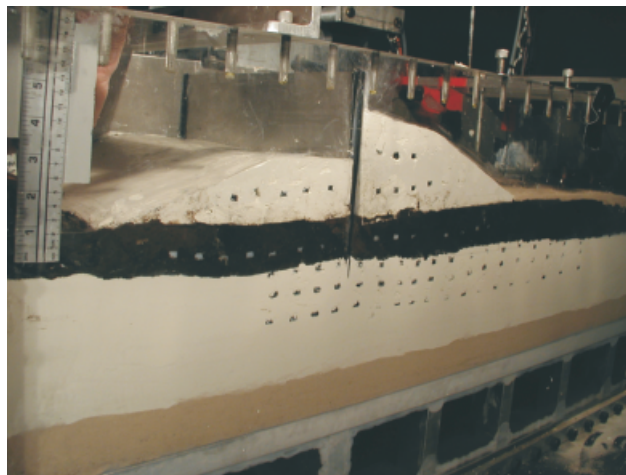
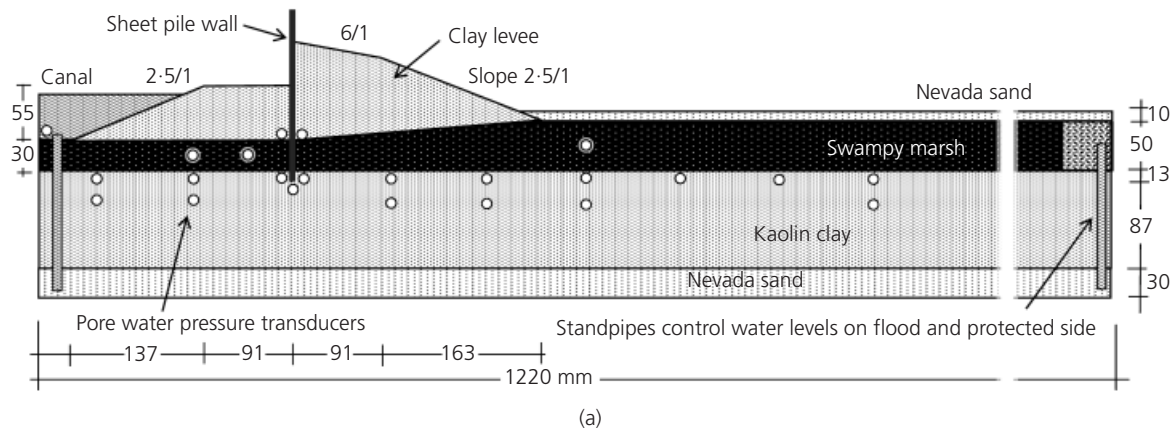


Figure 4. 17th Street model: (a) cross-section (model units);
(b) model

The saturated unit weight of kaolin at a moisture content of around 43% is 17.1 kN/m^3 .

The model tests used bulk samples of the natural material from the field to represent the swampy marsh stratum in the foundation of all the levee models. Undisturbed samples taken from borings provided laboratory samples from which compression tests, moisture content and unit weights were determined. The saturated unit weight of the swampy marsh was taken as 12.6 kN/m^3 . Site investigation reports show that the unit weight of the swampy marsh is variable, from a low of around 10.2 kN/m^3 to a high of around 12.6 kN/m^3 . The stiffness of the swampy marsh varies, but consolidated undrained direct simple shear laboratory tests on samples taken from the 17th Street breach area suggest that a value of shear modulus $G/\sigma'_v \approx 10$, where σ'_v is the initial vertical effective stress, may be appropriate for strains up to 2–3% (IPET, 2006).

In the field, the concrete flood wall is supported by a steel sheet pile wall that passes through the levee, terminating at a variable

elevation. In the model tests, this wall element was scaled using an aluminium or steel plate, with a bending stiffness comparable to the bending stiffness of the sheet pile wall. A solid plate was used in the model tests, as there was no evidence from the walls in the field that water leakage through the sheet piles was significant in the early stages of wall movement.

Further details of each of these materials, how they were used and placed in the models and the techniques for controlling their strength and stiffness with depth have been published (IPET, 2006).

5. Establishing initial conditions

All centrifuge models completed for this study fall broadly in one of two categories: those with a clay substratum (e.g. 17th Street), and those with a sand layer beneath the swampy marsh (e.g. London Avenue). In each case it was necessary to confirm that the correct initial conditions had been established prior to the flood loading.

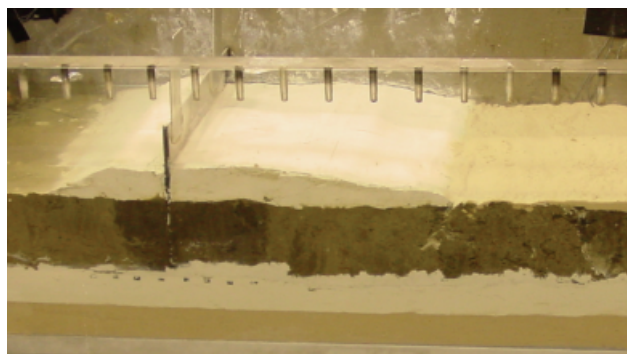
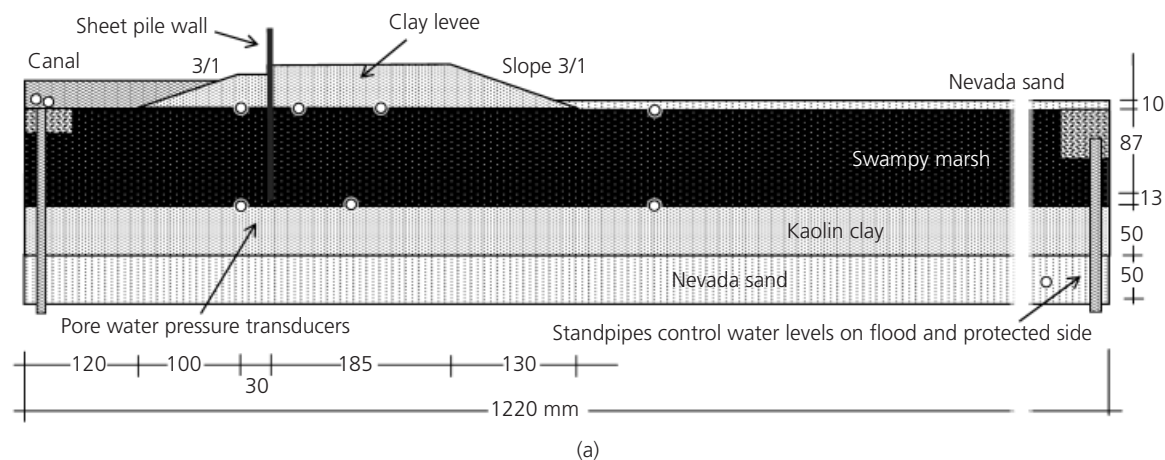


Figure 5. Orleans North model: (a) cross-section (model units);
(b) model

In the sand models, seepage under the levee was driven by a differential head between the water in the canal and the ground-water table on the landward side. Over time, in the field, a steady-state ‘flow net’ will develop governing the excess water pressure or head at different locations under the levee. Figure 7 shows a steady-state flow net, calculated using the software program SEEP-W (Geo-Slope, 2004), compared with piezometric head levels (blue bars) measured by pore pressure transducers at different locations under the levee at approximately the same depth in the sand. The early stages of water rise in the model tests in all cases were characterised by rising water pressures in the soil layers beneath the levee. This was most apparent for the London Avenue models, where the deep sand layer underlying the swampy marsh was connected hydraulically to the canal, and showed an immediate response as the canal water level changed.

In the sand layer, the increase of water pressure (head) was largest nearest to the canal and reduced with distance from the canal, landwards. This is due to head loss in the sand layer as seepage through the sand layer developed from the flood side towards the land side drain, and is seen clearly in Figure 7.

The second category of centrifuge model was the clay models,

simulating the ground profile below the 17th Street canal levees. Establishing initial conditions in the swampy marsh (peat) and clay layers required confirmation that these layers had reached a stress state comparable to that in the field. This was achieved by a process of consolidation in the centrifuge over time, using self-weight and surcharging, to bring the clay forming the levee, the foundation substratum and the shallow swampy marsh deposits into their required condition. The initial pore pressure distribution in the clay was therefore established under a combination of applied total stress and the hydraulic regime.

In field conditions with a clay substratum layer, the flooding of the canal is too rapid a process to permit drained behaviour of the clay to occur, and hence the process may be considered as an undrained event. In the model, kaolin was used as a clay substratum, partly because it is easy to mix and to form a uniform clay layer, and partly because it is relatively permeable and can be consolidated in flight in the centrifuge in a reasonable timeframe. Even so, consolidation of the clay layer in the model under a gravel surcharge (later removed) typically took over 19 h with one-dimensional vertical drainage. At field scale this would be equivalent to 25 000 h, or around 2.9 years.

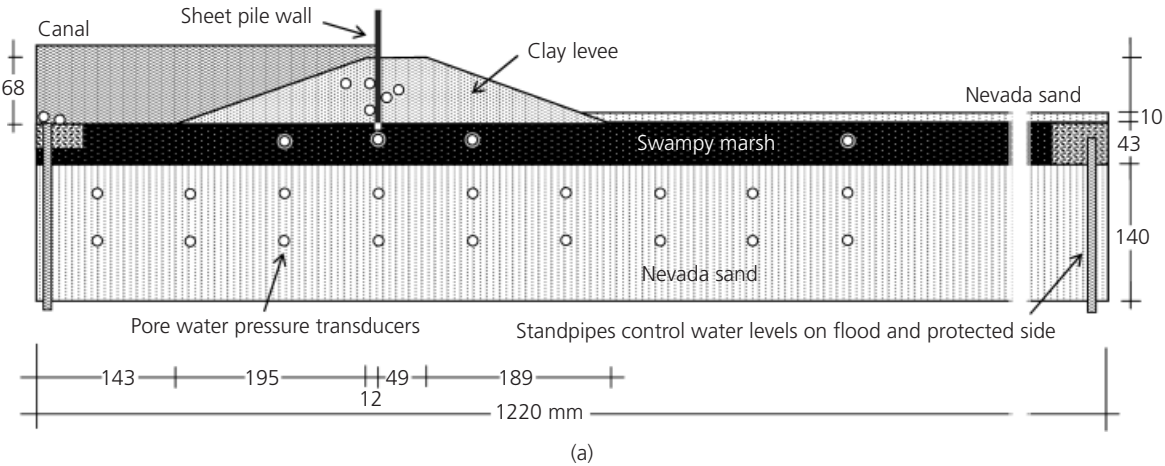


Figure 6. Orleans South model: (a) cross-section (model units);
(b) model

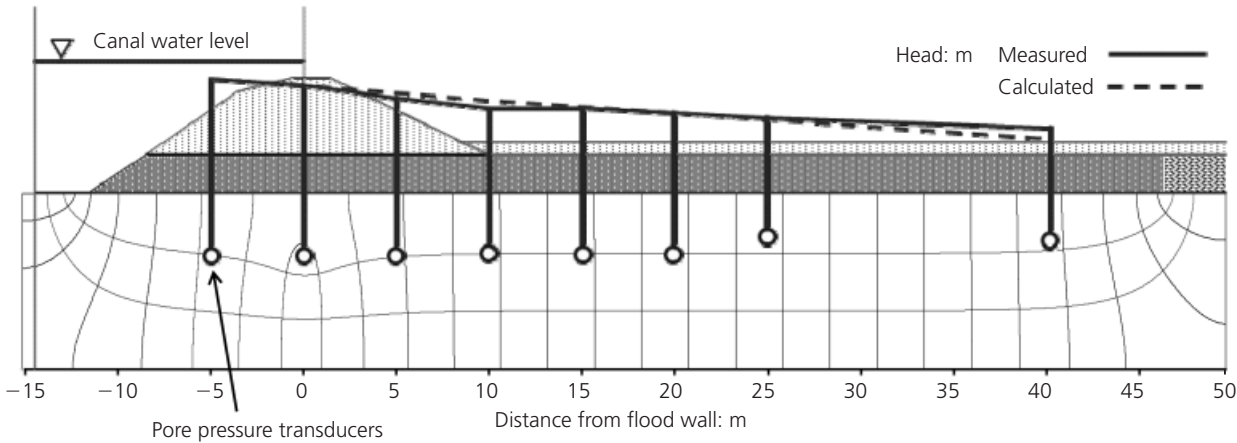


Figure 7. London South model, flow net and piezometric levels
at different locations, typical canal level

In the clay model experiments, with the swampy marsh layer and clay levee in place, canal water levels were raised at a rate of around 1 m in 20 h (field scale), or around 30 s in the model (Figure 8). As the surcharge was water, this stress increment would not provoke consolidation in the clay below the canal side of the levee. Below the levee itself, and towards the landward side of the levee, the long drainage path meant that the response of the clay to this relatively rapid rate of canal level rise was also effectively undrained.

In summary, therefore, as water levels in the canal rise, pore water pressures in the clay also rise owing to the increase in total stress (the weight of the water in the canal) and not to seepage, as in the sand foundation. Effective stresses remain the same until drainage of the excess pore pressure leads to a new effective stress profile. In practice there was no time for drainage to take place, and the models reflect this behaviour, exactly as in the field conditions.

The timescale used in the figures in this paper is an interpretation of the model timeline at field scale, based on principles of diffusion scaling. The resulting timeline at field scale was designed to be of the same order as the timeline of the storm, although in fact it is not necessary for the model timeline to correspond exactly to the actual timeline, as the pore water pressure response is effectively undrained.

Figure 8 shows the rise in pore water pressure at four locations in the clay layer beneath the levee on the canal side as the canal water level is raised. The exact correspondence confirms that the clay layer is responding in an undrained manner, with the increment of total stress (due to the weight of water in the canal) exactly matched by the increment of pore water pressure, regardless of depth in the foundation, which means that the

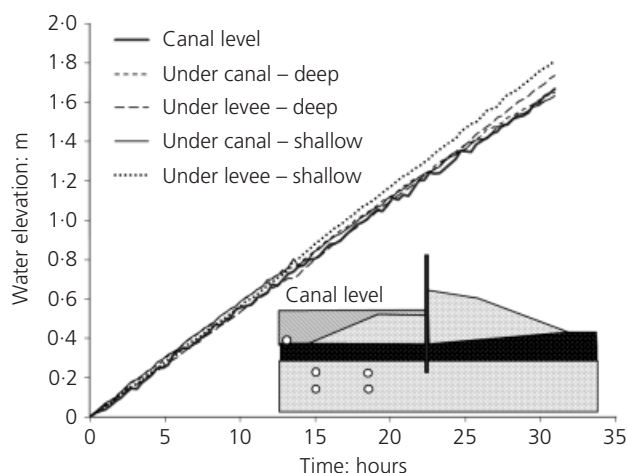


Figure 8. Water level in canal compared with incremental pore water pressure rise in clay layer beneath levee (field scale units)

strength of the underlying clay is fully described by its undrained shear strength. (As the canal level rises higher, the additional load on the canal side of the levee will also alter the elastic stress distribution in the foundation beneath, and this adds a slight additional increment to the transducers under the levee shoulder.)

6. Formation of a water-filled crack

In all the centrifuge model tests on the London Avenue, 17th Street and Orleans levee profiles, with both sand and clay foundations, a crack or gap was observed to form on the flooded side of the flood wall once water had overtopped the canal side crest of the levee and the wall rotated slightly landwards. The gap permitted a full hydrostatic head of water to develop to the bottom of the crack, which for the London South and North profiles provided a hydraulic connection between the underlying sand layer and the canal above the centre of the levee. In the case of the 17th Street profile, where the wall penetrated through the swampy marsh into the clay stratum beneath, the crack opened to near the toe of the wall. In the case of the Orleans South profile, where the wall did not penetrate through the swampy marsh layer, the swampy marsh continued to provide a hydraulic barrier to prevent high water pressures reaching the underlying sand. In the case of the Orleans North profile, the clay layer in the foundation was so thick that it both prevented formation of high water pressures and acted to constrain movement of the wall.

The formation of the crack followed the rise in water level on the canal side. As the water level rose against the levee, the flood wall did not experience any direct increase in lateral load, although there is clearly an increase in lateral load on the levee. Once the water reached the flood wall, however, a hydrostatic force started to build up on the wall, pushing it landwards. All of the flood walls in this study were unpropped (as in the field), and therefore the rising water on the flood wall is reacted by a small rotation of the wall, resisted by the embedment of the sheet pile wall, and the passive resistance of the levee material on the landward side of the flood wall. In this condition an unpropped wall is susceptible to the opening of a tension crack down the canal side of the flood wall, which immediately becomes water filled.

The theoretical depth, z , of a dry tension crack for an unpropped smooth wall retaining a cohesive clay material with undrained strength S_u (Figure 9) is simply derived, following Rankine, from

$$1. \quad z = \frac{2S_u - q}{\gamma}$$

where q is a surcharge, and γ is the unit weight of the clay. A similar depth of crack could occur through desiccation cracking. Where water is able to fill the crack, and the clay responds in an undrained manner, the water is able partially to support the clay, and the theoretical depth of the crack increases according to

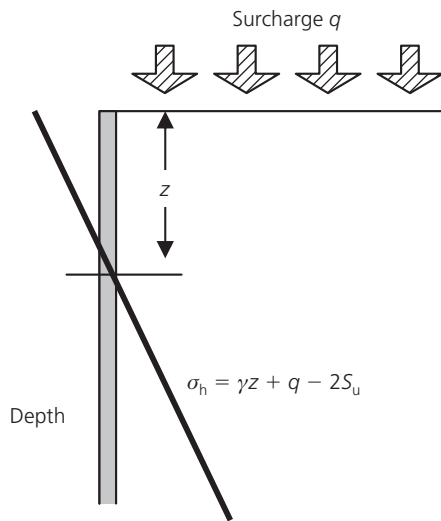


Figure 9. Depth z of dry crack behind unpropped smooth retaining wall; limiting active Rankine stress state

$$2. \quad z = \frac{2S_u - q}{\gamma - \gamma_w}$$

where γ_w is the unit weight of water.

The analysis above is relevant to unpropped walls supporting soil in an undrained condition adjacent to an open excavation. This problem was studied in depth by Bolton and Powrie in the 1980s, using centrifuge model tests to verify the analytical approach (Bolton and Powrie, 1987).

For a wall with support on the passive side, however, the

theoretical depth of the crack will be less, provided that the strain required to mobilise the passive (or propping) resistance is small. A rigid 'mechanical' prop would control the opening of a crack kinematically. In the absence of such a prop, then the stiffness of soil, mobilising passive resistance, may also control the opening of the water-filled crack.

Figure 10 shows the canal side and land side conditions similar to those in the models carried out for this study. Water fills a gap between the wall and the clay on the 'active' side. The depth of the water-filled crack in this active condition (without support) is unchanged from the simple equation above, as the increase in total vertical stress is equal to the increase in hydrostatic pressure from the water in the gap. If limiting passive resistance can be mobilised on the landward side of the wall with negligible landward movement, then the theoretical depth of the crack (in almost all cases) will be controlled by the passive propping force that is mobilised to resist the hydrostatic force on the opposite side of the wall. The depth of the crack, z , below the crest of the levee on the canal side (if any) would then be given by Equation 3, where H is the depth of water above the ground surface on the active side, and h is the depth below ground on the passive side.

$$3. \quad \gamma_w(z + H) = \lambda h + 2S_u$$

However, this stress-balance analysis takes no account of the actual movement required to mobilise passive resistance from the landward side of the levee, and the approach is clearly unsafe for common geometries or soil conditions; it does not predict that cracks would form in the case of the 17th Street or London Avenue levees, as they clearly did. In practice, the geometry of the landward shoulder of the levees and the low shear stiffness of the swampy marsh beneath, mean that signifi-

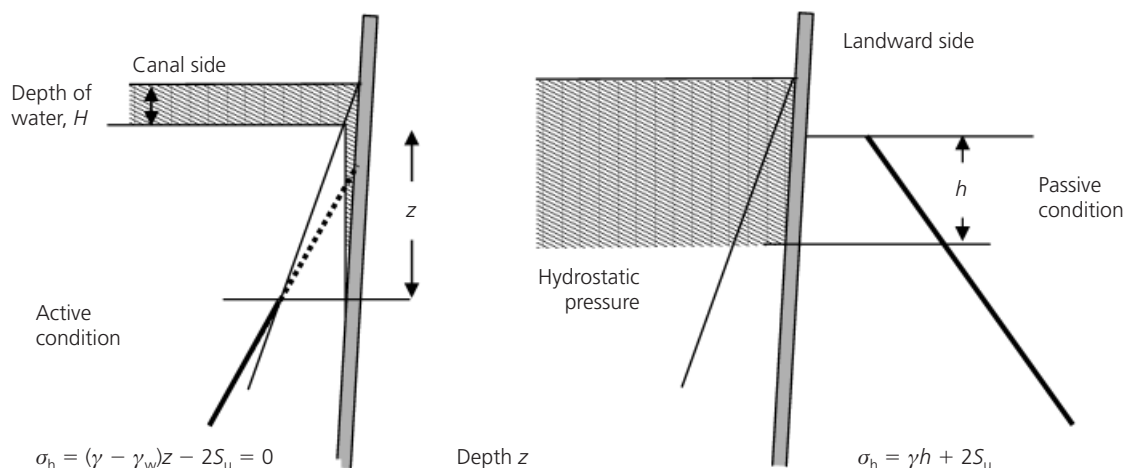


Figure 10. Limiting active and passive Rankine stress states on either side of a stiff wall with water-filled crack

cant movement of the passive block on the landward side is required to mobilise any resistance to the crack formation.

Rotation of the flood wall under the additional hydrostatic load (and moment) caused by the water rising up the exposed face therefore takes place regardless of the theoretical benefit (in terms of reduced crack depth) of the passive resistance of the upper soil layers on the landward side, opening a gap on the canal side of the wall as if there were no soil behind it at all. In reality, the passive soil block is only capable of resisting the lateral load by movement, and any movement landward of the wall will open a crack that is immediately water filled. As the crack or gap opens, full hydrostatic conditions exist (for the levee profiles considered) either to the base of the wall (where the wall does not reach an underlying sand layer), or to the top of the sand layer (where it does). Design to resist rising water levels on the basis of a force balance alone will be inherently unsafe.

The evidence for the formation of the crack is clearly seen in the video imagery from the model tests, a snapshot from the video shown in Figure 11, and also in the record of the pore pressure transducers located near the toe of the flood wall (Figure 12), which showed a rapid rise in water pressure to the full canal water level as the wall moved landwards slightly. The same sequence of crack formation, wall stabilisation and then wall failure was observed in all four of the models performed of the London Avenue canal levee.

In the case of levees on clay foundations, a similar phenomenon was observed. Figure 13 shows the rotation of the wall followed



Figure 11. Early stage of crack formation in London South model

by a brief period of stabilisation before the main failure ensued. Model tests carried out on cross-sections representative of the Orleans canal showed very small to no movements of the wall at flood levels up to Katrina. The wall and levee remained stable, owing to a combination of much larger levee geometry and more stable foundation conditions. When the water levels were allowed to rise beyond the Katrina flood level (done in the model simply to explore the potential for crack formation), movement of the wall was observed, indicative of crack formation. However, the flood wall remained in place, and did not show the same unstable response to high flood levels as the levee sections on 17th Street and London Avenue. This stability is attributable to the levee geometry and foundation conditions.

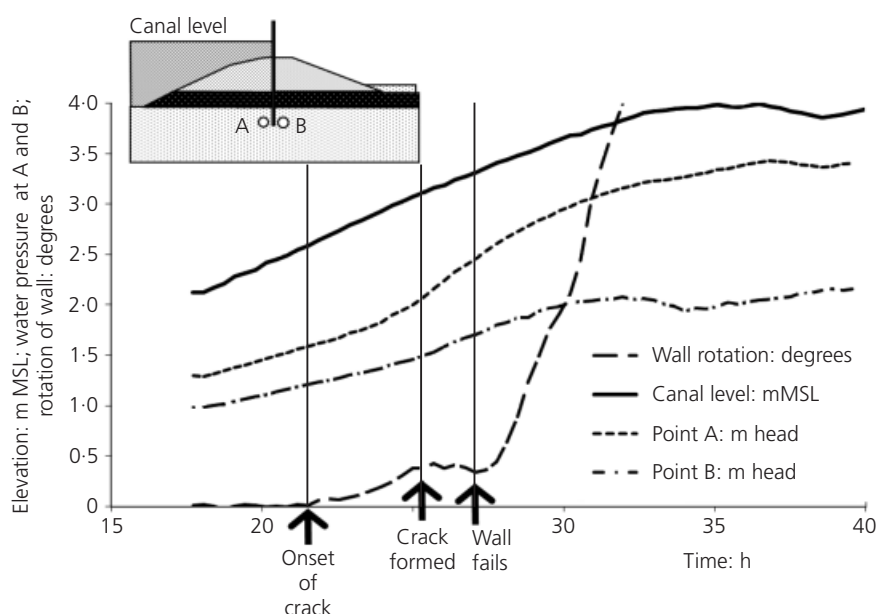


Figure 12. Comparison between canal level, water pressure at base of wall and wall rotation, London South model

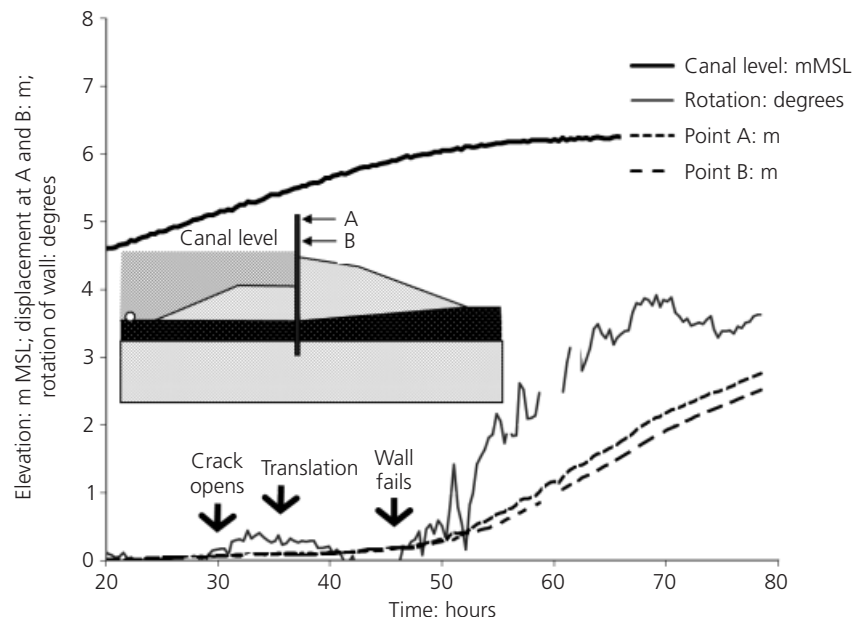


Figure 13. Rotation and translation evidence of crack formation prior to failure, 17th Street model

7. Rotation of the wall

In the model tests where the toe of the sheet pile wall penetrated into the sand and was restrained from lateral movement, the opening of the crack on the canal side of the flood wall was followed by a rotation of the wall landwards. Instruments on the wall recorded the horizontal movement, from which it may be shown that the wall rotated, consistent with the video imagery of the failure. In all of the model tests with sand foundations (London South and London North) that showed gross movements indicative of wall failure, the rotational movement was accompanied by a translational sliding of the landward part of the levee with the underlying swampy marsh, on top of the deeper sand layer.

Piezometric levels indicating the pore water pressure in the sand foundation show that the vertical stress on the foundation at the onset of failure was near zero towards the toe of the levee on the landward side.

Figure 14 shows rotation of the wall in a London South model, and Figure 15 shows a sequence of video images of the rotational movement in progress. A similar sequence was found for all of the London Avenue models.

The rotation of the wall is linked to the reduction in effective confining stress in the sand layer beneath the landward shoulder of the levee. Once the crack is fully opened, pore pressure transducers in the foundation sand layer show that the excess pore pressure in the sand rises to reflect the increase in pressure at the toe of the wall. The increase is seen to reduce linearly with

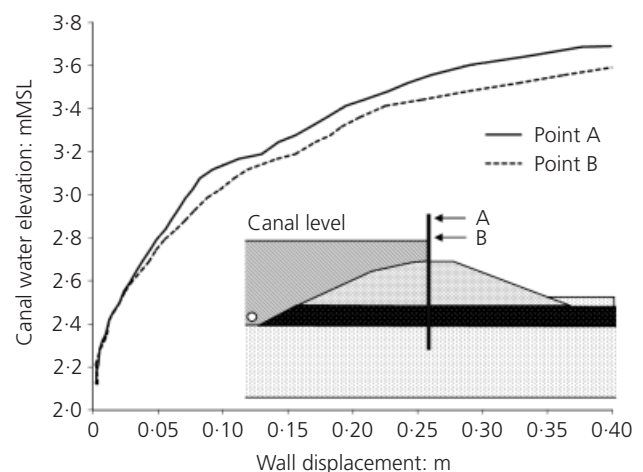


Figure 14. Rotation of wall as canal level rises followed by failure, London South model

distance from the toe of the wall. The increasing water pressure in the sand layer reduces the vertical effective stress under the swampy marsh.

The reduction in vertical effective stress has two effects: the first is to increase the likelihood of uplift of the swampy marsh (as the increasing water pressure in the foundation balances the weight of the levee and swampy marsh layer above), and the second is to reduce the stiffness of the sand

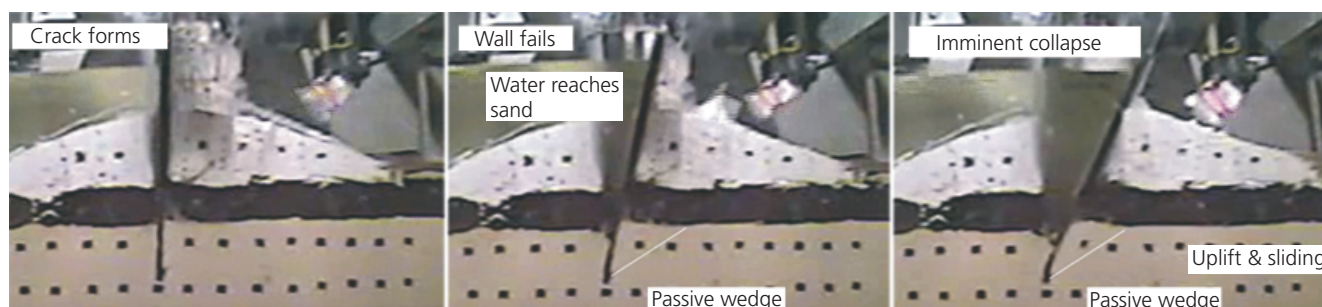


Figure 15. Wall failure follows crack formation, London South model

surrounding the toe of the sheet pile wall, providing passive resistance.

The mechanism of failure in the sand models (London South, London North) was analysed as follows. Piezometric levels under the levee on the landward side rise, reducing the vertical effective stress on the interface between the swampy marsh and the sand as the canal level rises (Figure 16). Rotation of the wall under the hydrostatic load on the exposed section of flood wall above the crest is resisted by the embedment of the wall in the sand layer, the elastic stiffness of the swampy marsh layer, and the clay levee (landward side). The measured piezometric heads for the London South model shown in Figure 16 (shown by the vertical bars) clearly show a downward trend from the wall to the landward toe (see Figure 7 for a typical flow net). As the crack opens and the wall rotates landwards, the piezometric head increases downstream of the wall, roughly proportional to distance, up to around 5 m from the flood wall. Beyond this, additional pore pressure rise is minor, perhaps as a result of drainage, and the fact that the piezometric head is already very high compared with the vertical effective stress.

Observations of the extent of wall rotation prior to failure show that the wall reached around 0.5° prior to a brief period of stabilisation, before the rising piezometric levels under the levee, coupled with the rising canal level, triggered the failure of the wall. This is a relatively small rotation of the embedded toe of the wall prior to failure, and suggests that there is a sudden loss of support elsewhere in the wall–soil system.

Figure 17 shows an equilibrium stress analysis for the wall at the onset of failure used for the analysis of the London Avenue models. Hydrostatic pressure from the canal reaches the base of the swampy marsh. Below this level, an active stress condition exists to the base of the wall (for simplicity). Differences in the hydrostatic pressure on either side of the toe of the wall in the sand lead to a small resultant ‘unbalanced’ force landwards. On the passive side, full passive pressure, taking into account the increased piezometric head, is assumed to be mobilised in the sand, and a limiting shear stress is mobilised along the underside of the swampy marsh beyond the passive zone, where the vertical effective stress is greater than zero. Forces acting on the swampy marsh are discussed further below.

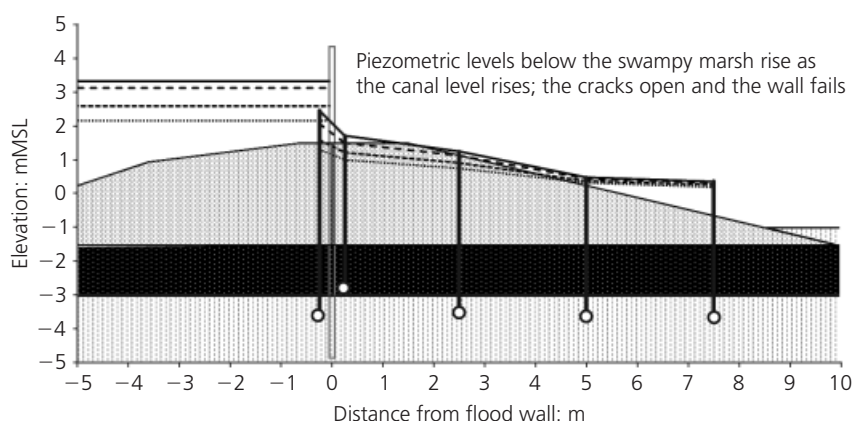


Figure 16. Piezometric levels immediately under swampy marsh show increase under levee as wall rotates, London South

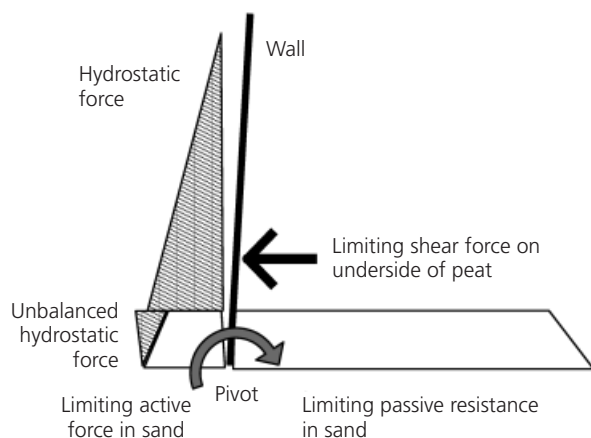


Figure 17. Equilibrium stress analysis for collapse of wall

Figure 18 shows a superposition of three images at different stages of the levee failure, with two marker points highlighted in the passive zone and in the swampy marsh near to the levee toe. It is clear that the swampy marsh is moving laterally, while a wedge-shaped passive zone develops in the sand, showing both a slip surface extending from near the toe of the wall and some heave at the top of the sand layer, below the levee crest.

Figure 19 shows diagrammatically the forces acting on the swampy marsh towards the landward side of the levee and beyond the passive wedge in the sand layer at the toe of the wall. The compressibility of the swampy marsh landward of the levee means that no significant passive resistance from this layer can be mobilised in the 'crumple zone'. Near the toe of the levee, water pressures in the sand layer are sufficiently high that the vertical effective stress between the swampy marsh and the sand layer is near zero, and hence the shear stress that may be carried across this interface in this area is



Figure 18. Overlay of video images of rotational failure of London South model wall

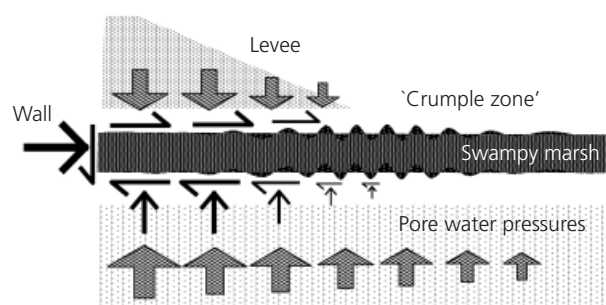


Figure 19. Diagram of forces acting on swampy marsh towards toe of levee

also near zero. Forces acting on the levee from the wall must be transferred through the swampy marsh to the sand below, and therefore the maximum horizontal force that may be applied from the wall is controlled by the maximum shear force that can be mobilised on the underside of the swampy marsh layer.

Figure 16 showed the piezometric levels under the swampy marsh at different canal levels. Figure 20 shows the reduction in vertical effective stress under the swampy marsh as the wall starts to fail (line 'B' indicates conditions at the onset of failure). Near the wall, vertical effective stresses are slightly lower on the canal side of the levee, compared with the landward side. Towards the toe of the levee, the vertical effective stress falls to near zero.

The passive and active forces in the sand were calculated on the basis of a limiting angle of shearing resistance of 33° , considered appropriate to the degree of strain that the wall had undergone. The limiting shear force on the underside of the swampy marsh was calculated on the basis of the average vertical effective stress acting over discrete lengths of the interface, outside the passive zone and where the effective stress was greater than zero. The unbalanced hydrostatic force was computed from the measured water pressures in the sand on either side of the wall (this force is due to seepage effects). Saturated unit weights of the levee clay, swampy marsh and sand were taken as 17.3 kN/m^3 , 12.6 kN/m^3 and 18.9 kN/m^3 respectively, appropriate to the unit weights of kaolin and Nevada sand.

The results of the calculation for two different models were highly consistent, showing that this mechanism of failure was in a limiting condition at the instant of failure, based on the measured data from the model tests. Figure 21 shows the FoS calculated for the two models, at four different stages of normal (typical) canal level, crack opening, crack formed and the onset of wall failure.

8. A two-stage failure mechanism

The analysis is consistent with the observation from all the model tests that the formation of the crack and the readjustment of the

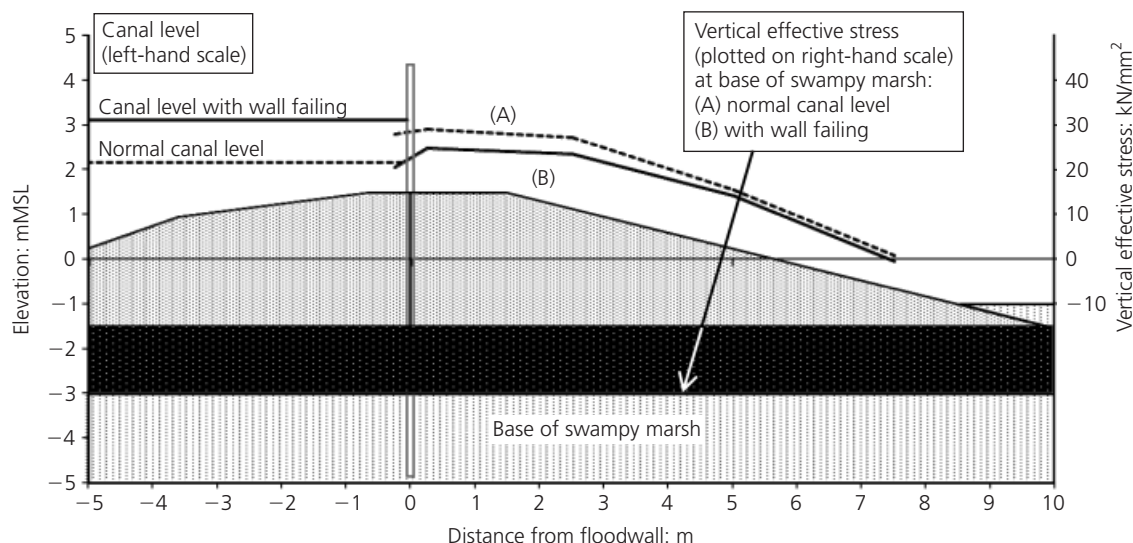


Figure 20. Reduction of vertical effective stress on landward side of flood wall at onset of failure

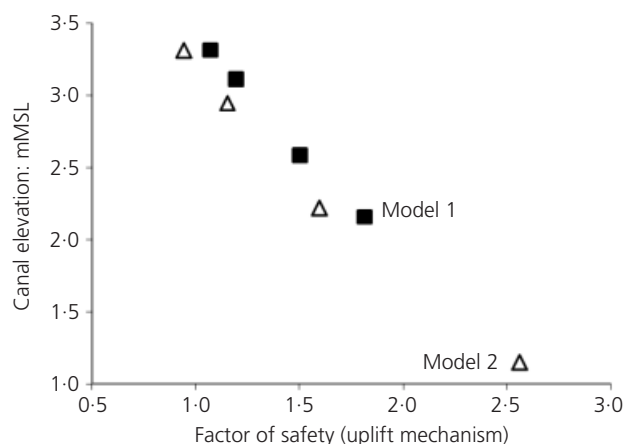


Figure 21. Factor of safety for observed failure mechanism (uplift and rotation) for two London South models

wall to the new hydrostatic loading condition, with increased moment and lateral force, constitute a separate event from the failure that follows. In both models analysed here, the factor of safety remains above 1 at the completion of crack formation, and falls to 1 at the onset of failure.

In both sand and clay foundations the opening of the water-filled crack effectively cuts the flood protection levee system in two, losing the benefit of the canal-side half of the levee altogether. The hydrostatic head on the exposed face of the flood wall and the overturning moment impose a severe load on the landward portion of the levee and the embedded toe of the flood wall. The models show that the walls sustain this

condition for a brief period before the second stage of the failure develops, as the piezometric head beneath the swampy marsh reduces the vertical effective stress beneath the landward toe, and eventually forces a sliding mechanism to be triggered. The extent of rotation or sliding in the collapse stage will depend on the depth of the embedment, and on whether this is in sand or clay.

The rotation and uplift mechanism of failure observed in all the sand foundation models (London South and London North) is sensitive to the unit weight of the swampy marsh. In the analysis of the model tests presented above, the unit weight for this material was selected appropriate to the samples used in the experiments. Reducing the unit weight of the swampy marsh by 13% (from 12.6 kN/m³ to 11.0 kN/m³) reduces the FoS by around 0.1.

Because of the significance of the formation of the crack on both the depth of sand available to provide passive resistance and the increase in piezometric head, the model tests show that shallow penetrations into the sand layer will have a particularly low FoS. Following the analysis above, varying the depth of the bottom of the sheet pile wall by ± 0.3 m changes the FoS by ± 0.15 .

The high pore water pressures under the toe of the levee on the landward side and under the swampy marsh led to water being ejected from the ground at the toe of the levee. This may be deduced from the piezometric heads near the toe of the levee, and may be seen in the video images of the landward toe. Figure 22 shows a view of the levee and flood wall as seen from the 'backyard' of houses behind the levee, showing 'black' water emerging from the toe of the levee as the flood wall rotates landwards.

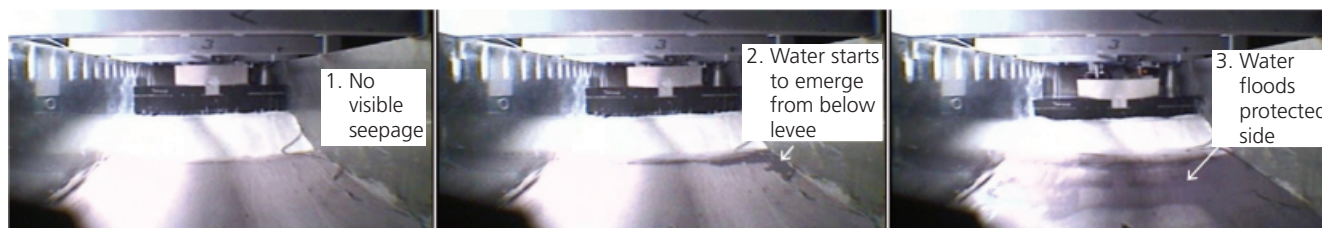


Figure 22. Water emerging from toe of levee as failure progresses, RPI London South model

9. Translation of the wall

In all the scale models where the toe of the sheet pile wall terminated in the clay layer, then following the opening of the water-filled crack a translational failure occurred through the clay, starting at the toe of the wall and progressing landwards. This was clearly seen in the instruments recording movement of the wall and in the video imagery from each of the models, and will be discussed in detail below.

The observation of crack formation in the 17th Street levee models is very similar to that described for the London Avenue models, so that description will not be repeated. Figure 23, from one of the 17th Street models, shows an early rotation that briefly stabilises before the main failure commences (scale time 28 to 36 h). Video imagery of the phase between the initial opening of the crack and the onset of the main failure shows active slumping behind the wall, which may explain the increase in rotation between scale time 37 and 42 h.

The sliding surface developed near the top of the clay layer and progressed landwards until it was outside the levee, before it

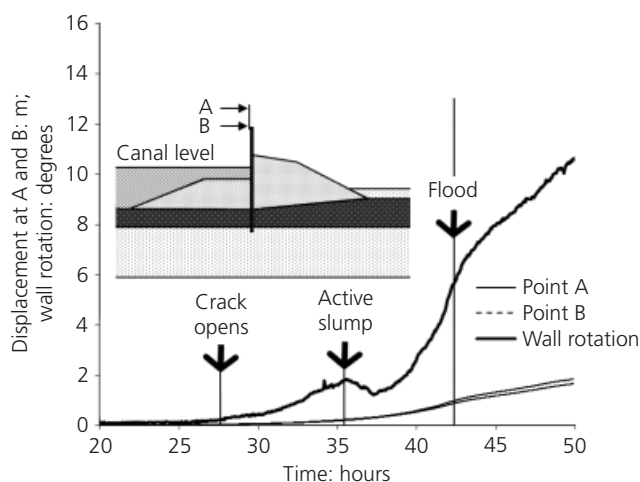


Figure 23. Crack formation prior to wall failure, 17th Street model

turned upwards to exit through the swampy marsh layer. This mechanism is similar to the observations in the field from 17th Street. The progression of the slip surface landwards can be identified from the pore pressure records, which show a progressively delayed reaction landwards to the translational movement.

Comparing the wall rotation with the pore water pressure measurements at locations in the clay on the canal side of the wall, at the toe of the wall and the canal level (expressed as head in metres of water) with the wall rotation, gives further insight into the mechanism of failure (Figure 24). The crack formation phase is marked by a small rotation, up to 2° movement, following which the rotation briefly stabilises. As noted above, active slumping starts towards the end of the crack formation phase, perhaps as early as scale time 30, indicated by the change of slope and reduction in excess head near the toe of the wall on the canal side. During this period, the pore pressures beneath the levee on the canal side reflect the increase in total stress, but show no evidence of the wall movement.

The reduction in excess head near the toe of the wall continues as the wall ceases to rotate but continues to translate (between scale times 36 and 37.5) before rotation starts again, leading to a sharp transition in performance around scale time 42 as the wall fails. At this moment, the canal level drops as a result of the rapid lateral movement, the water pressure at the bottom of the wall changes slope again and rapidly rises (to reach and exceed the canal level), and the rate of rotation decreases slightly as the main translational movement commences.

The sliding failure of the model wall was arrested before a full breach took place to capture the final state of the 17th Street model (Figure 25). Despite the large lateral movement (notice the row of black markers at the top of the foundation clay layer just beneath the swampy marsh layer), there is minimal heave of the swampy marsh layer on the landward side.

All the clay foundation models failed with a similar mechanism under the flood loading, commencing with a small rotation and crack formation, followed by further rotation and large translation associated with a shear plane that formed from the toe of the

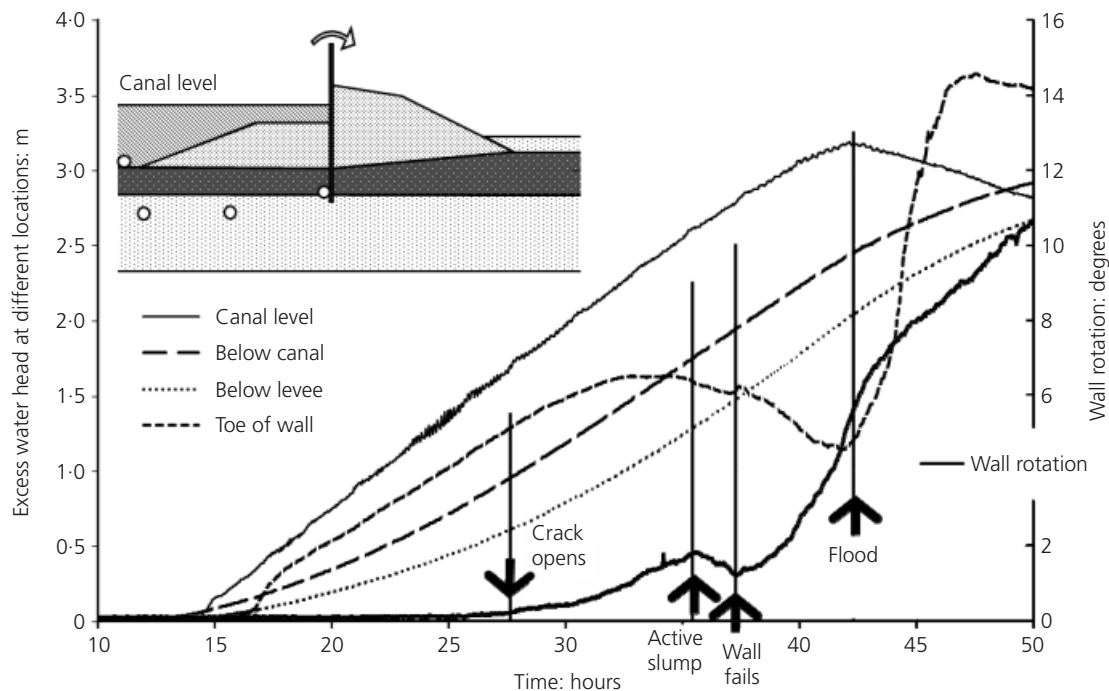


Figure 24. Comparison between wall rotation and excess head at onset of failure, 17th Street model

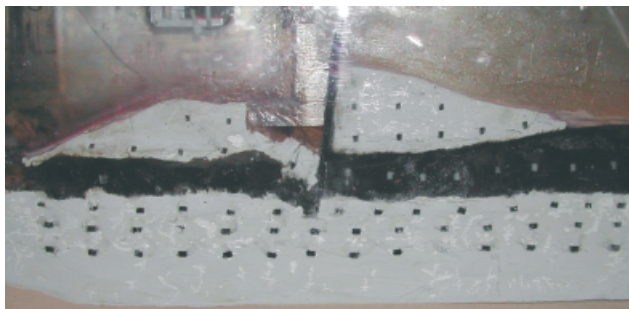


Figure 25. Final state of 17th Street model, showing lateral translation on sliding surface starting at toe of wall and progressing landwards

sheet pile wall and progressed landwards through the top of the clay layer. This was consistent with the field observations.

Other observations relevant to the site conditions on 17th Street were also noted from the centrifuge model tests. The very soft, normally consolidated clay beneath the levee experienced significant settlements as the weight of the levee was increased to full height. The swampy marsh layer also tended to be compressed below the levee.

In one model of the 17th Street levee, the weight of the levee on

the landward side was sufficient to provoke bearing failure in the clay foundation, with the formation of inclined slip planes, prior to the flood condition. This is a situation analogous in the field to raising the height of the levee too rapidly, without allowing time for the high excess pore pressures in the foundation to dissipate and the strength of the clay to develop. On the surface, the only evidence is increased settlement. At depth, however, local slip surfaces form in the soft clay and swampy marsh

10. No-failure condition

Scale model tests were also carried out of levee sections on the Orleans Avenue canal, which did not fail, as part of a validation process. The cross-section chosen to represent the southern portion of the Orleans Avenue levees is similar to the cross-section at London Avenue, except that the levee is wider and higher, and the penetration of the flood wall is less (the toe of the wall is at the base of the levee/top of the swampy marsh layer).

As with the other levee sections, the instruments beneath the model levee responded to the rising water in the canal by increased excess water pressures in the ground, reducing linearly from a maximum near the canal to a minimum to the landward side of the levee (Figure 26). At the water level experienced in Orleans Avenue canal, the flood wall did not show any movement to indicate the opening of a crack (Figure 27).

The piezometric levels below the swampy marsh in the Orleans South model showed a declining level from the canal side to the

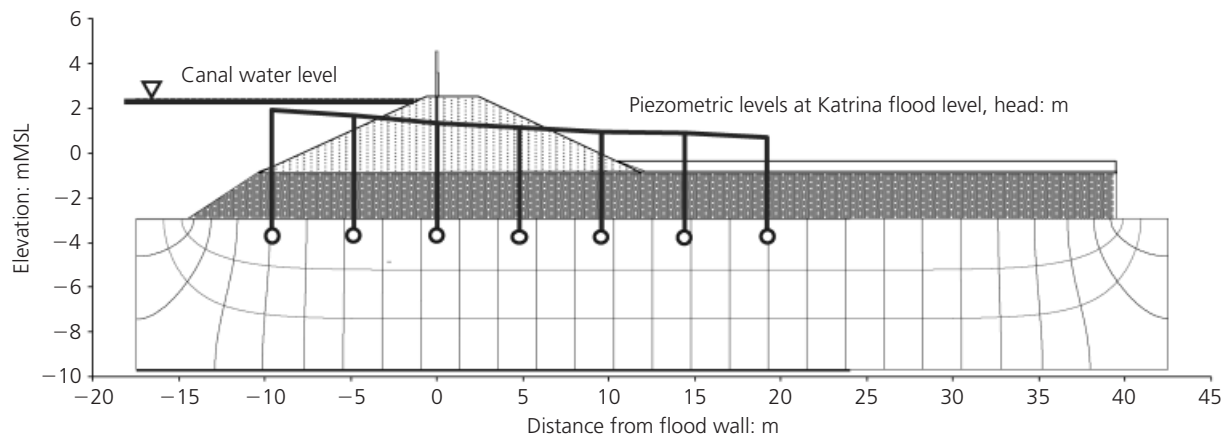


Figure 26. Seepage flow net for Orleans Avenue South with piezometric levels under swampy marsh at Katrina flood level

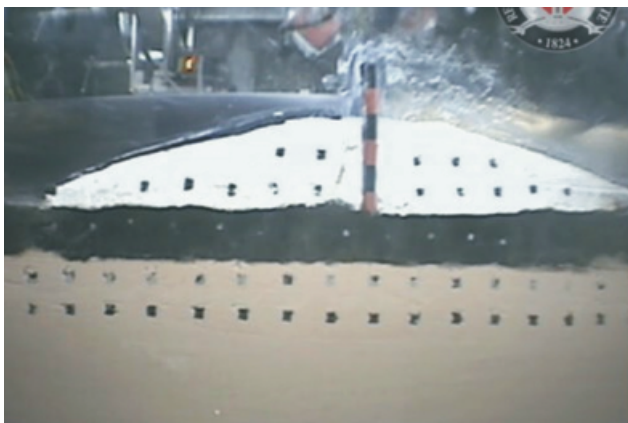


Figure 27. RPI Orleans South model at Katrina flood level

protected side of the levee (Figure 28). The additional height and width of the Orleans South levee, seen overplotted in the figure for both Orleans South and London South, significantly increase its capacity to resist the flood levels, and no evidence was seen of a crack opening through the swampy marsh and providing a hydraulic connection to the sand below. The London South piezometric levels, shown in the figure for comparison together with its cross-section (aligned at the elevation of the top of the sand layer), show a more critical condition in terms of uplift under the landward side of the levee. Both models, however, show high piezometric levels near the toe of the levee on the landward side, sufficient to lead to uplift of the toe of the levee and the underlying swampy marsh.

As the water level was increased above the Katrina flood level in the Orleans South model (done simply to explore the potential for

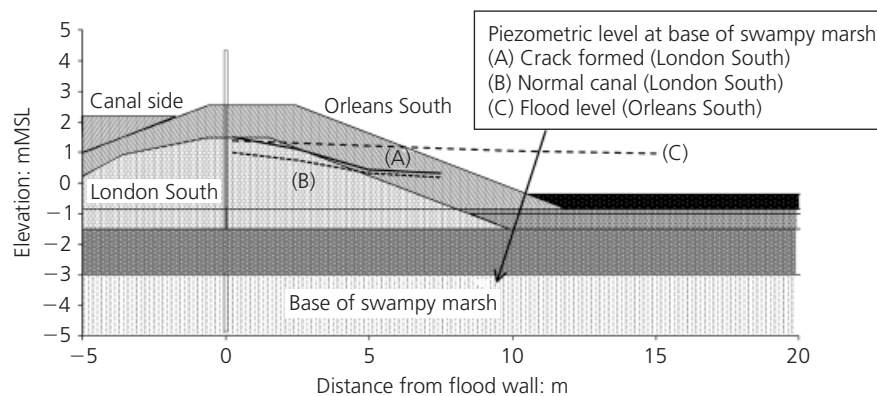


Figure 28. Comparison of piezometric levels in Orleans South (Katrina flood) and London South, with base of the swampy marsh assumed to be at a common elevation

crack formation), a crack was seen to open to the toe of the flood wall, associated with small movements of the flood wall landwards, as shown in Figure 29. Unlike the other models, this crack did not develop into a full failure condition; no unstable movement of the flood wall was observed, despite the water level reaching the top of the flood wall. Translational movements were observed to be controlled by compression of the swampy marsh layer. Relative movement between the top of the sand and the swampy marsh was apparent, in a similar manner to the uplift mechanism in the London Avenue models.

Comparison between the geometry of Orleans and London Avenues shows that the vertical effective stress in the sand foundation below the levee was considerably higher under the Orleans levee than under the London Avenue levees. The capacity of the flood wall to rotate is reduced by the height and weight of the landward section of the Orleans levee. Figure 30 shows the vertical effective stress on the underside of the swampy marsh layer measured in the Orleans South and, for comparison, the London South models. Although the Orleans levee section is higher than London South, the piezometric levels near the landward toe are sufficiently high that a zero vertical effective stress condition is created near the landward toe of the Orleans South levee in both the Katrina flood and full flood conditions (maximum water height, no crack).

Examining the stability of the Orleans South model shows that the formation of the crack during the raising of the water level to the top of the wall was accompanied by lateral sliding or translation of the swampy marsh layer. The landward movement was small but noticeable, and appeared uniform under the landward side of the levee, extending close to the canal side of the wall. A shearing zone appears to dip steeply downwards from the canal side toe of the wall landwards.

The stability of the landward section of the Orleans South levee may be analysed using the simple mechanism shown in Figure 31. Video imagery shows sliding of the swampy marsh on the top of the sand layer. The sliding surfaces through the swampy marsh

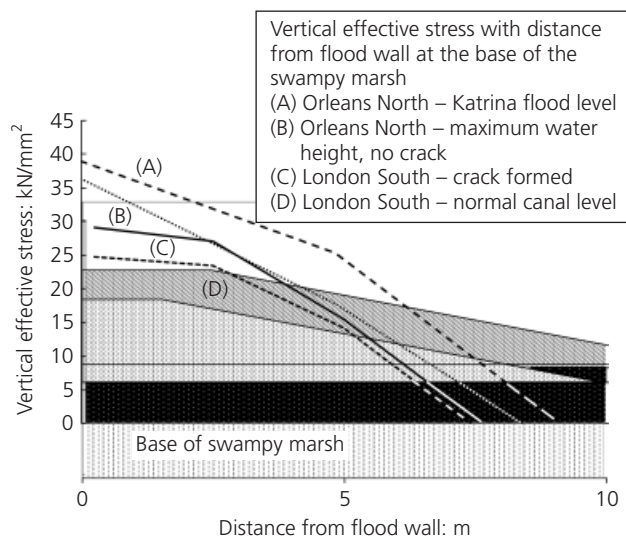


Figure 30. Vertical effective stress comparison at the base of the swampy marsh between Orleans South and London South

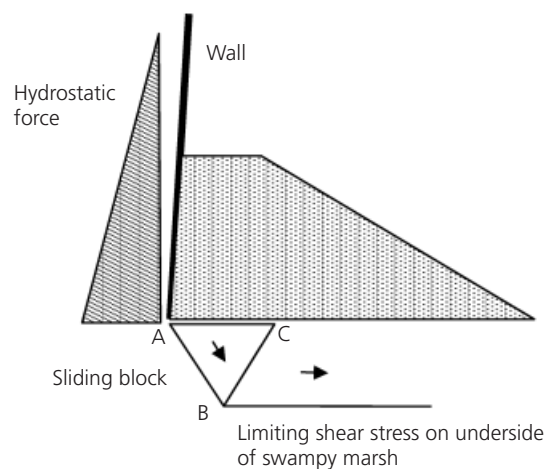


Figure 31. Possible upper-bound mechanism for Orleans South

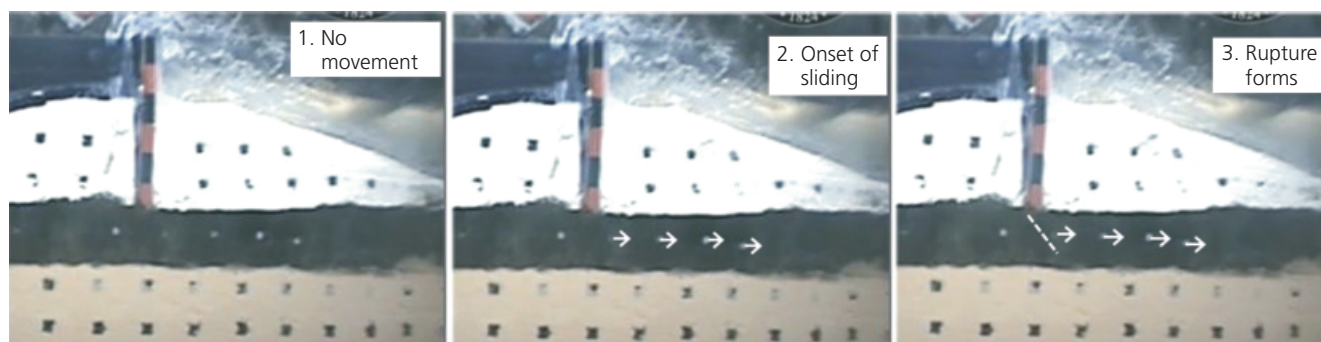


Figure 29. Sliding movement of swampy marsh as crack forms, Orleans South model

forming a triangular sliding block ABC are chosen to lie at angles of 60° for simplicity. Assuming the levee block is free to move laterally with the swampy marsh, then work is done only on the lower surface of the swampy marsh, and on the internal sliding surfaces AB, BC and CA.

For infinitesimal lateral movements, a work balance gives an expression for the limiting height of water H above the toe of the wall as a function of the depth of swampy marsh, D , its undrained shear strength S_u , the limiting shear stress mobilised on the top of the sand layer, $\sigma'_v \tan \phi'$, the vertical stress on the landward side of the flood wall, σ_v , and the length of the shear surface along the top of the sand layer offering significant shear resistance, L ,

$$4. \quad \sigma_v D + \gamma_w H^2 = \frac{6S_u D}{\sqrt{3}} + \sigma'_v \tan \phi' L$$

Solving this equation for H , the critical depth of water in the crack, and using the actual measured excess pore water pressures in the Orleans South model, suggests that the Orleans South model was close to failure at full flood level (not Katrina flood level). Using the value of undrained shear strength adopted for the slope stability calculations for the swampy marsh ($S_u = 19.2 \text{ kN/m}^2$) and a limiting angle of shearing resistance of 33° in the sand, an FoS may be calculated of 1.50 under Katrina flood conditions and 1.07 under full flood conditions (flood to the top of the wall). This is consistent with the observations of small but continuing displacements of the wall under full flood conditions.

Additional scale model tests were carried out for a section of the northern portion of Orleans Avenue canal that did not fail, again as part of a validation process. The cross-section chosen to represent the northern portion of the Orleans Avenue levees is similar to the cross-section at 17th Street, except that the levee is wider and higher, the foundation swampy marsh layer is much thicker with a higher fines content (5 m thick for Orleans as opposed to 2 m thick for 17th Street), and the penetration of the flood wall is larger (extending from the top of the levee through almost the entire thickness of the swampy marsh layer). The results for this model are very similar to those discussed above for the Orleans South model. At Katrina flood levels there was no movement of the wall and no crack formation, and the wall remained perfectly stable. As was done for the Orleans South model, the water level was increased to the top of the wall to explore the potential for crack formation and wall movement. There was no movement of the wall in this model, owing to the increased size of the levee, the depth of penetration of the sheet wall and the constraining effects of the foundation material.

11. Conclusions

(a) Centrifuge model test data of the mechanisms that led to the failure of the 17th Street and London South levees and the near-failure of the Orleans Avenue levee show that this was a two-stage process, with the initial stage being the formation

of a water-filled crack on the canal side of the flood wall. The profile or geometry of the levee, flood wall and foundation was found to be critical to the development of the shallow failure mechanisms that followed.

- (b) Detailed examination of the instrumental data and mechanisms of failure has shown how the levees and flood walls responded to the rising water levels, starting with the development of a water-filled crack in front of the wall and leading to ultimate failure of the levee by rotation or sliding. Pre-existing water pressures in the ground were seen to be key to the failure in all the sand foundation profiles.
- (c) The levees showed a two-stage response in all cases. The first stage comprised the formation of a water-filled crack on the canal side of the flood wall, effectively cutting the levee in two, following which there was a rapid readjustment of the levee and its foundation to the new loading conditions.
- (d) In the second stage, one of three conditions was observed to follow. In the Orleans South model, at water levels higher than the Katrina flood level and sufficiently high to lead to the formation of a crack to the toe of the wall, the wall remained stable, although with creeping movements landwards, indicative of a near-failure condition. The failure mode that was seen to be developing in the Orleans model closely resembles London South. Stability analysis confirmed that the condition of the flood wall was near failure.
- (e) In the London Avenue and 17th Street models the second stage led to rapid failure and breaching. In the London Avenue case, the formation of the crack and resulting hydraulic connection between the canal and the sand layer was followed by an upsurge in the water pressure below the landward section of the levee, leading to an increasing rotation of the flood wall. This was an unstable event, which, once initiated, led to a rapid collapse as the increasing water pressures under the swampy marsh reduced the shear resistance on the sliding surface along the top of the sand to a near-zero condition.
- (f) In the 17th Street models, the formation of the crack was followed in the second stage by the immediate development of a slip movement at the toe of the wall, and an increasing rate of landward movement. The absence of a time delay between these observations was notable. Once triggered, this mode of failure was also unstable. As the initial slip surface extended landwards towards the toe of the levee, the weaker clay further from the centreline of the levee was less and less able to resist the driving forces acting on the levee block.
- (g) The low shear stiffness of the swampy marsh and large passive strain (movement) required in the landward levee block to mobilise passive resistance controlled the mechanism of the crack formation in all models.
- (h) The model studies have important implications for the design of flood protection systems worldwide, showing that design approaches based on force equilibrium require very careful consideration of the effects of soil–wall interaction, and particularly the consequences of wall movement, however small, under ultimate load conditions.

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