Evaluation of I-wall in New Orleans with calibrated soil parameters

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Abstract

This study utilized the field monitoring results from a full scale loading test in London Ave. Canal in New Orleans, LA to calibrate/back-calculate soil moduli. Calibrated soil moduli were used to accurately predict the behavior of the I-wall in London Ave. Canal, so that effective retrofitting techniques can be designed.

From this study, it turned out that the moduli of the field soils are mostly higher (1 to 12 times) than predicted magnitudes from laboratory tests or empirical relations; the gap formation can increase the stability of the levee in some cases by relieving the stresses: that is contradicting to the general belief, but conforms to the measured data; and quantitatively confirmed that the strength reduction in the marsh layer is one of major failure mechanisms.

Keywords: flood protection system, I-Wall, numerical analysis, Hurricane Katrina, levee

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Introduction

The failures of I-walls during hurricane Katrina were the major causes for many breaches of the flood protection systems in New Orleans. After Katrina many research papers have been published regarding the failure mechanisms of the flood wall system such as gap formation behind the flood walls, instability of the flood wall system due to the shearing of the weak foundation soil, the development of high uplift pressure in the sand layer, reduction of the strength of foundation soil due to the under seepage induced pore pressure, and scouring on the front side of the flood wall (IPET 2007, Seed et al. 2008, Brandon et al. 2008, Dunbar and Britsch III 2008, Duncan et al. 2008, Sills et al. 2008). To counteract those failure mechanisms and to enhance the performance of the flood protection system, some retrofitting techniques might be the improved floodwall sections to counteract the overturning; the use of expansive plugs to restrict gap formation; use of composite sheet piles to increase the stiffness; and the protection on the front side of the flood walls against erosion. The numerical analysis is particularly important for the reliable analysis of those counter measures.

For the numerical analysis, appropriate numerical approach and accurate soil parameters are essential. So the study presented in this paper had two objectives. The first objective of the study was the development and verification of the numerical approach to capture the realistic behavior of flood wall system due to water load. The numerical approach included the development of two dimensional geometric model of the flood wall and soil system incorporating the elasto- plastic material behavior of the soils and soil/wall interfaces. The simulation procedure was also accompanied with the algorithm to model the occurrence of gap and the prediction of its depth compatible with the deformation of the soil and wall system. At the same time the numerical procedure also included the reduction of the shear strength of the weak marsh soil susceptible to the high pore pressure during high flood level. The verification of the numerical technique was conducted by comparing its results with those of the full scale load test done in the London Avenue Canal of New Orleans.

The second objective of the study was to calibrate the soil parameters that will be used for further numerical analysis. Though there has been a wide variability of the soil strength parameters in New Orleans region, especially for marsh and the levee fill (Duncan et al. 2008), the IPET strength model which is used to provide the representative soil strength for levee and marsh of that region and which is based on the strength parameters obtained by actual shear strength test of the soil samples from the borings (this model is based on the samples from 17th Street canal) can be used for the two dimensional numerical analysis in London Avenue Canal as well . The inclusion of the variation of the soil strength in the analysis is beyond the scope of two dimensional analyses since it can only be captured by three dimensional analyses .So a major concern regarding the soil parameters in the two dimensional numerical analysis is the prediction of stiffness parameters. In most of the cases, there is a direct correlation of undrained shear strength with the stiffness of the soil. Duncan and Buchignani 1976 have proposed a relationship of these parameters with the function of over consolidation ratio and plasticity index. But it is seen that the relationship exists for a wide range .Similarly Mayne and Swanson 1981 also proposed a cam clay prediction of undrained initial tangent modulus with respect to undrained shear strength in terms of over consolidation ratio, Cc and friction angle. With this method, the range is smaller than proposed by Duncan and Buchignani 1976 but still the relationship exists for certain range.

Specific to study site, in IPET report, V-19(IPET 2007), the stiffness of the soil was estimated by using the relation as E/S_u to be 92. Hurricane Protection Office (HPO) 2008 did the soil structural analysis for the same site in February 2007 before the full scale load test in which two analysis were conducted, one using the soil modulus obtained from tri-axial shear strength testing (TXT) and the other from in situ Pressure Meter Test (PMT) performed at the outfall canals where E/S_u for TXT was 48-68 and G/S_u for PMT was 100-220. So comparing all this approaches, it is seen that there is not a unique value of stiffness parameters and uncertainties certainly exist in estimating stiffness parameters by any kind of empirical equations. So it is necessary to treat these uncertainties by calibrating them accurately. In this study, back numerical analysis was done by using inclinometer measurement of the full scale test as the datum to calibrate the stiffness parameters of the individual soil layers. Though the complete report of parametric study has not been presented in this paper, the paper has recommended the final calibrated moduli required for the further numerical analysis.

As an outline of this paper, the brief description about the full scale test is given in the second chapter of this paper. In the third chapter, a detailed account on the modeling has been presented. The main focus and the uniqueness of this study is the development of the algorithm for the initialization of the model which suits for the total stress analysis; proper modeling of the gap occurrence ; and the alteration of the shear strength of marsh layer for different loading condition. In the fourth chapter of the paper, the results from the numerical simulations are evaluated and finally the conclusion from the study is presented in the last chapter.

Full scale loading test

Since the data and the information from the full scale loading test were used as the datum of this study, it is relevant to discuss briefly about the full scale loading test.

Objectives

The full scale load test was conducted in the critical section of the I-wall in the London Avenue canal of New Orleans by US Army Corps of Engineer (USACE), New Orleans District from August 17, 2007 to August 31, 2007 with the main objective to find out whether the safe water elevation (SWE) can be increased within the city's outfall canals during the closure of the temporary gated structure during the hurricane season or not. The closure of the temporary gated structure is needed to prevent the storm surge to enter the outfall canals from Lake Pontchartrain. Since the pumping capacity of the gate is not equal to city's inland drainage system, it is necessary to increase the safe water elevation to reduce the interior flooding during the closure of the gate. Increasing the safe water elevation would allow the city to pump the water into the canal at higher rate which reduces the risk of flooding inland and at the same time it would also allow the flood gates to remain open for the longer time.

The other objective of the test was to calibrate the models in order to study the behavior of the flood walls.

Test procedure and the measurements

A cofferdam was built along 150 feet of the critical section of I-wall on the east side of the London Avenue canal to isolate the I-wall section from the rest of the water in the canal and to aid the slow, incremental and controlled raising of water level against the wall. Then the sophisticated instruments were installed to monitor the response of the wall and the foundation soil underneath due to the impact of different water level. The Instruments that were used during the test were the inclinometers to measure the horizontal soil deformation; tilt meters to record the rotation of the monolithic base; piezometers to measure the pore pressure; and the distance measuring device to measure the deflection of wall. Once the site set up was ready, the water was raised in controlled incremental manner and the response of the wall and the foundation was monitored by scientists and engineers. The data like the load deflection curve and the inclinometer readings presented in the documents (HPO 2008, Conroy and Berry 2008) was used for this study. Fig.1 shows the fluctuation of water level and the deflection of the monolithic base for different water level (HPO 2008, Conroy and Berry 2008)

In Fig. 1, it can be seen that the loading of the wall was done in two phases.

Fig1. Comes here

Numerical Model

Modeling tool and the condition

Fast Lagrangian Analysis of Continua (FLAC^{3D}) developed by ITASCA was used for the numerical analysis of the flood wall. It is a finite difference code to solve quasi- static problem using dynamic approach. To model the static response of a system, damping is used to absorb kinetic energy. It has the capability of large strain calculation mode.

To reflect the short period of loading condition and the undrained behavior of the embankment and the foundation soil (marsh stratum) total stress analysis was conducted. The other reason of selecting the total stress analysis is that the total stress strength parameters are readily available in the literature (IPET 2007) whereas effective stress strength parameters are not and estimation of effective stress parameters using empirical relations may add uncertainty in the simulation.

Model geometry

Since I-wall is a long system with the homogenous property, the system can be analyzed using the plane strain model. To mimic the plane strain behavior using FLAC^{3D} ,only one zone was constructed along the out of plane direction and the velocity of the grid points lying on the front and the back plane along this direction was constrained to zero.

Dunbar and Britsch III 2008 and Rogers et al. 2008 have given the detail outline of the geologic set up of the New Orleans area and the history of its formation. The load test site consists of four layers of foundation soil and the embankment backfill soil layer. The FLAC^{3D} is constructed to represent those layers with sheet pile and the concrete capping. The lower most soil layer is the stiff clay known as Bay sound clay. This layer extends below -44.4 ft measured in NAVD88 scale. The stiff clay is overlain by beach sand that extends from -44.4 ft to -21.4 ft. Above beach sand layer, up to -11.4 ft, is the silty-sand layer which is overlain by the marsh layer. The marsh layer is divided in to two parts namely Marsh-crest and the Marsh-toe as they exhibits different shear strength. The embankment with backfill material extends up to 2.2 ft above the marsh layer. The sheet pile is embedded inside the reinforced concrete for 2.75 ft from the base of I-wall. The reinforced concrete capping extends from 0.2 ft up to 13.2 ft and the tip of the sheet pile is at -21.4 ft.

The main characteristics of the FLAC^{3D} numerical model were that soil was represented by the brick type zones each having eight grid points possessing elasto- plastic behavior. Similarly, sheet pile was represented by linearly elastic two dimensional shell type finite elements known as the 'embedded liner' slaved with an interface. The reinforced concrete I-wall was represented by the brick type zones having eight grid points possessing isotropic linearly elastic behavior and the soil/wall interface was represented by three nodes triangular elements.

The typical I-wall section is shown in Fig. 2 which has the base of 1.75 feet and tapered to 1 foot above.

Fig.2 comes here

The model geometry is illustrated in Fig. 3

Fig.3 comes here

Initial soil model parameters

The Mohr Coulomb constitutive model was used to represent the stress strain behavior of the soil**.** This model was used since it is simple and has less parameter than other advanced constitutive model so that it has the lower degree of uncertainty. This model requires basically five parameters: cohesion, c; internal friction angle, Φ; mass density ,ρ; shear modulus, G; and bulk modulus, K**.** For the numerical analysis, the strength parameters and the young's modulus of soil in IPET report, Volume V, Appendix 19, (IPET 2007) were taken as the starting parameters. The Poisson's ratio was taken as 0.48 to account the undrained behavior of the soil for the cohesive soil and for silty sand and beach sand; poison's ratio was estimated using the relation derived from the theory of elasticity (e.g. Terzaghi 1943):

$$\upsilon = \frac{K_0^t}{1 + K_0^t} \qquad (1)$$

Where, v is the Poisson's ratio and K_0^t is the equivalent coefficient of lateral earth pressure at rest condition for total stress analysis. K_0^t is estimated using equation (12)

Once the undrained elastic young's modulus and the Poisson's ratio were known, the bulk modulus and the shear modulus were calculated using the following relations:

$$G = \frac{E}{2(1+\nu)} \qquad (2)$$

$$K = \frac{E}{3(1-2\nu)} \qquad (3)$$

where G and K denote the shear modulus and bulk modulus respectively and E denotes the elastic young modulus of the soil. The elastic modulus in IPET report was estimated to be 92 times the undrained strength. The elastic moduli of sand and silty sand layers were calculated with the same relation for cohesive soil assuming the undrained strength of sand to be 1500 (this value was used to calculate the young's modulus only).The starting model parameters for the soil are given in Table 1:

Here comes the table 1

The parameters of silty sand were assumed to be the one for the topmost sand layer of London Avenue Canal which is given in IPET report, Volume V- Appendix 19, (IPET 2007).

Model parameters for the I-wall and sheet pile

To model the sheet pile, structural element of the FLAC^{3D}, known as “embedded liner” was used. The structural element is basically the shell element having resistance to both membrane loading and the bending loading. The shell element in FLAC^{3D} can possess either of the isotropic, orthotropic or anisotropic properties. In this analysis, the sheet pile was assumed to possess an isotropic elastic property. It requires three parameters to define the mechanical properties of these elements: density, ρ; elastic modulus, E; Poisson’s ratio, υ; and the thickness, t. Elastic modulus, density and the poisons ratio can be found in any steel tables. The sheet pile used in London Avenue canal is CZ-101(HPO 2008). The geometrical parameters of this type of sheet pile are found in the manufacturer table (sheetpile.net). To account the simple geometry of the shell element used in the simulation against the complex one of the real Z-type sheet pile, equivalent thickness of the shell element was estimated using the relation:

$$t_{eq} = \sqrt[3]{I} \qquad (4)$$

Where, t_{eq} denotes the equivalent thickness of the embedded liner element used in the simulation and I denotes moment of inertia in in^4 per unit foot width of the sheet pile

To model the concrete I-wall section, FLAC^{3D} zones were used. Elastic model was used to model the stress strain behavior of the concrete. The main parameters required for this model are: the density, ρ; bulk modulus, K and shear modulus, G. The density of the concrete is readily available in any literature. Bulk and shear modulus are calculated using the equation (2) and equation (3) where E is young’s modulus of concrete estimated from equation (8). The Poisson’s ratio of concrete can vary from 0.15 to 0.2 (Bowles, 1982). In this simulation the Poisson’s ratio

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was assumed to be 0.2. The young's modulus of normal weight concrete was estimated using the relation (Wang and Salmon1998):

$$E_c = 57{,}000.\sqrt{f_c^{'}} \qquad (5)$$

Where, E_c is the elastic modulus of concrete in psi and $f_c^{'}$ is the compressive strength of the concrete that varies from 3500 to 5000 psi for usual non pre-stressed reinforced concrete (Wang and Salmon 1998). In this numerical simulation, it was taken to be 3925 psi. This value is the average value of the compressive strength measured by taking samples from the existing I-wall. The details can be found in Appendix 15, Volume V of IPET report (IPET 2007). For plane strain condition, the elastic modulus estimated using the equation (5) was modified using the relation given in FLAC structural elements manual (Itasca 2006):

$$E_{c,plane-strain} = \frac{E_c}{(1-\nu^2)} \qquad (6)$$

Where, ν denotes the poison's ratio.

Actually, the reinforcement was disregarded while calculating the elastic modulus, because the moment of inertia for the transformed reinforced cement concrete section is only 5% and 7% more than that of the plain cement for the upper portion and the lower portion of the existing I-wall. And as far as the cracking is concerned, the tensile stress developed in the concrete for maximum 7 feet of loading condition is below the tensile strength of the concrete given by the equation (7) (ACI 318-05, 2005)

$$f_{ct} = 7.5\sqrt{f_c^{'}} \qquad (7)$$

So the cracking was not considered in the numerical analysis.

Model parameters for the soil–wall interface:

As described in the previous section, structural element of the $FLAC^{3D}$, known as "embedded liner" was used to model the sheet pile. Apart from having structural behavior of the shell type element, the embedded liner has also the tangential and normal interaction between it and the $FLAC^{3D}$ grid. This tangential and the normal behavior is controlled by the "embedded liner" interfacial properties. This means that the embedded liner has the interface slaved with it so that building of separate interface is not necessary.

The normal behavior of the soil- sheet pile interface is governed by the normal coupling springs properties like the stiffness per unit area, K_n and the tensile strength f_t. The tangential or the shear behavior of the soil sheet pile interface is governed by the shear coupling springs properties like stiffness per unit area, K_s; cohesive strength c; residual cohesive strength c_r ; the friction angle ϕ and the interface normal stress σ_n. The tangential behavior of the interface is so modeled that if the interface fails in tension, then the effective cohesion drops from c to residual cohesion c_r and the tensile strength is reduced to zero. But if the relative displacement is such that there is no more separation between the soil and the sheet pile, then the normal stress will again develop. This behavior of the liner interface in $FLAC^{3D}$ provides the possibility of modeling gap occurrence and closing of it very accurately. Fig.4 and Fig. 5 show the normal and shear directional behavior of the interface.

Here comes Fig. 4 and Fig. 5

In the case of interface between the reinforced concrete capping and the soil, a separate interface represented by triangular elements with 3 nodes was built up. These elements are characterized by the normal and shear stiffness with sliding properties. The constitutive model of these interfaces is defined by linear Coulomb shear strength model which limits the shear force acting in the interfacial node.

For both cases of sheet pile-soil interface and reinforced concrete wall-soil interface, a schematic of the FLAC^{3D} interface elements showing constitutive behavior are shown in Fig.5.

Here comes Fig 6.

As mentioned earlier, the main interfacial properties required for the simulation are the stiffness and the strength parameters. The FLAC^{3D} manual (Itasca 2006) points out that the strength properties of the interface are important, but the stiffness properties are not. But if there is a large contrast of the stiffness in the system, it takes a lot of time for numerical simulation to converge. So to economize the simulations, the FLAC^{3D} Theory and Background Manual (Itasca 2006) recommend that the K_n and K_s can be estimated by thumb of rule which is to be 10 times the stiffest neighboring zone.

$$K_n \approx K_s \approx 10.\max\left[\frac{K+\frac{4}{3}G}{\Delta z_{\min}}\right] \tag{8}$$

Where K & G are the bulk and shear moduli respectively and Δz_{min} is the smallest width of an adjoining zone in the normal direction. The max [ ] notation indicates the maximum value over all the zones adjacent to the interface to be used.

As far as the strength parameters of the interface are concerned, for cohesive soil, they are given in terms of adhesion which is equal to 0.5 to 0.7 times the undrained strength of the adjacent soil (Bowles 1996). In this Simulation, the cohesion was estimated using the relation:

$$c \approx 0.67.S_u \qquad (9)$$

lower case s

Where, s_u denotes the undrained shear strength of the adjacent soil and c is the cohesion of interface.

For the case of cohesion less soil, the strength parameters are given in terms of angle of friction. For steel pile embedded in silty sand (dirty sand), the interface angle of friction, δ was taken to be 14^o (Bowles 1996; USACE 1994).

Modeling Hypothesis

As discussed earlier, the main objective of this study was to develop proper analytical procedures that reflect the realistic behavior of the flood wall system. What happens in the system is not always tangible. So it was necessary to develop some hypothetical tool which had to be verified using numerical experiment. Two major hypotheses were made to study the behavior of the I-wall due to water load. The validity of these hypotheses was tested by comparing the results from numerical analysis with the measurements from the full scale load test.

Hypothesis 1: Formation of gap

The gap formation was not considered in the previous design of flood walls system in New Orleans. Most of the design guidelines then published by USACE (e.g. examples), considered the formation of tension cracks to some depth formed due to the cohesion of the soil but didn't take into account the formation of gap due the relative deformation of the soil/wall system. After

Katrina, investigations made by IPET (IPET 2007) showed that the major aspect relating to the performance of flood wall system is related to the formation of gaps between I- wall and the levee embankment soil or foundation soil. After this, many papers have been published pointing out the significant effect of the gap formation on the overall stability of the flood wall system (give the name of the papers). The observations made during the field visits after the Hurricane Katrina showed that gap occurred at several places due to canal water loads. But at the same time, during the field visits, occurrence of gap was not observed at all places. To elaborate more, even for the two sections adjacent to each other having the seemingly identical soil conditions and the same loading conditions is liable to gap occurrence for one section and not liable to gap occurrence for the other one. So, the factors governing the gap occurrence in the flood wall system might not be only very few. Therefore it is not always possible to predict whether the gap will occur or not. So incorporation of the gap formation can only be regarded as the hypothesis which has to be tested using the numerical experiments. In this study, the numerical simulation results have been compared for the system when gap occurs and when it doesn't with the measured one. In the case when gap formation was considered, it was assumed that the formation of the gap allows water to penetrate through it and exert a direct hydrostatic force on the exposed sheet pile and the soil adjacent to it.

However, unlike in numerical analysis, in the case **of design of floodwalls**, as the hydrostatic force acting on the exposed sheet pile is more than the active earth pressure that the wall would experienced if there were no gap, gap formation is taken to be more critical and recommended to be incorporated for the safe and conservative design, no matter whether gap occurs or not.

Hypothesis2: Under seepage induced strength reduction of marsh layer:

The investigation performed by IPET has pointed out that the subsurface erosion/piping is the main cause of the failure of I-walls in London Avenue Canal. Most of the papers published aftermath of Katrina has also indicated the same thing. Some paper (e.g. IPET 2007) indicated the evidences of sink holes and sand boils at the crest and toe of the levees in London Avenue Canal after Katrina .Though the full scale load test was taken on London Avenue canal (the section near to the one where piping and subsoil erosion was reported to be more prevalent during Katrina), it might be misleading to adopt that piping occurred during the load test, since there were no evidence reported during the load test regarding piping/erosion. It is obvious that the piping will occur for the sand layer. But the subsurface setup of the full scale test location shows that the sand layer is overlain by the marsh stratum which is fairly impermeable than the sand layer. So the occurrence of piping failure for such geological setup cannot be justified since piping is the process of removing soil from the exit point advancing up gradient. Piping might occur in the case where there are discontinuities in the marsh stratum exposing the sand layer to the ground surface or due to the development of the tensile stresses in the marsh layer promoting the propensity of vertical piping in this layer. And since piping is a progressive event, the duration of the full scale test was not long enough to lead such event.

From the above discussion, we (authors) are not trying to advocate that the load test section will not suffer from the piping failure at all. The section is liable to piping problem for the large and sustained loading condition. This is because the sustained and large hydraulic loads generate higher seepage force which might be potential enough to rupture the marsh layer and break it throughout its depth that promote vertical piping through this layer. But before this action could take place, the marsh layer on the land side might face some strength reduction which is justified by the explanation in the following paragraph. In other words, the strength reduction of the marsh might be the case which when exacerbated by the high seepage force on the sand layer underneath causes it to rupture making vertical path to promote piping.

So in this study, another aspect for impending failure is considered. Formation of gap causes the shortening of the water path to weak marsh layer which causes an under seepage induced strength reduction of the layer. Since the strength of the marsh is dependent on the effective stress, the seepage force in the marsh would decrease the effective stress thus decreasing the strength. This aspect has also been discussed by Seed et al. (2008) and Ubilla et al. (2008). IPET 2007 on page V-42 has also given the similar explanation regarding the reduced shear strength for the London Avenue North breach near the load test site. The strength is however recovered when the water level drops increasing the effective stress in the marsh.

Most of the literature indicates that there is a correlation between the shear strength of the marsh and the effective vertical stress but have not proposed the unique correlation factor (give references if any). So the quantitative reduction of the of shear strength using any theoretical expression is not possible. So in this study the reduction of the strength was taken to be only hypothetical which had to be tested and calibrated using the numerical experiment. Reduction of shear strength can only be realized when there is a gap and since the marsh is about 100 times impermeable than the levee fill material (IPET 2007), the reduction of strength in the levee fill material would not be that dominant as compared to the marsh layer. So reduction of strength was considered only in the marsh layer.

The motivation toward the reduction of the strength during the high flood level was evolved by observing the load deflection curve of the full scale load test. From the measured load deflection curve (Fig.1), it is seen that some portion of the deformation was not recovered during the unloading indicating the yielding of some soil portion. The curve also indicates that some soil already started yielding when the water level reached 5.5 feet. And since for the initial original soil strength, the system behaves elastically, the load deflection curve indicates implicitly that there might be a reduction of the strength of certain soil layer during the high flood level which leads to the unrecoverable deformation. Some might argue of using the low shear strength starting from the low water level, but experience showed that this would affect in increasing the deflection at lower water level as well which contradicts the measured load deflection curve.

Simulating Procedure

In this section, the detailed numerical procedure has been discussed. First, the general procedure which is common to any type of geotechnical numerical analysis is discussed. Apart from these general procedures, the special and unique numerical approaches that have been formulated and implemented in this study such as the initialization procedures, the gap formation, and the strength reduction are discussed separately.

General procedure

The natural soil layer up to marsh layer was built up and run for the initial state. For this, gravity was turned on gradually in an incremental manner to reduce the dynamic impact. Alternately, the initialization of the existing natural soil layer could also have been done layer by layer of small thickness.

Then the embankment of backfilled material was built on above the natural soil layer. The sheet pile was erected and reinforced cement concrete capping was constructed with suitable interface between the reinforced cement concrete and the embankment soil. Again the model was run for the elastic equilibrium. Elastic equilibrium was chosen so that the plastic flow would not occur during the initial equilibrium state.

The suitable sub routine was used to initialize the accurate horizontal stress. For this, the equivalent coefficient of lateral earth pressure at rest (K_0) for total stress analysis was calculated using the K_0 values of the corresponding soils at effective stress condition.

The water was then added up to Mean Water Level (MWL) and run for elastic equilibrium. The water was added in an incremental manner to avoid dynamic effect. Then the soil was changed to Mohr Coulomb elastic perfectively plastic model and again run for the equilibrium. After the equilibrium was reached, the displacement and the velocity of the whole system were initialized to zero.

The water level in the canal was raised according to the loading sequence in the full scale load test. To reduce the dynamic impact, water level was raised in an incremental manner. The response of the system was observed for each water level.

The above procedure was repeated for each modified set of parameters.

Initialization

The initial stress condition in the system is very important in the numerical simulation of many geotechnical problems since the stress strain behavior of the soil depends on the stress level. It is difficult to achieve the initial stress in numerical simulation when there are discontinuities in the geometry since plastic flow occurs at these points (reference if any why don’t you refer FLAC manual?). To avoid this problem, the initialization was done in two steps, as discussed earlier, initially by bringing the model into elastic equilibrium to inhibit the yielding of soil before assigning the elasto-plastic properties of the soil.

During the initialization process, FLAC^{3D} calculates the vertical stress accurately but the calculation of the horizontal stress might not be the one as per the appropriate value coefficient of lateral earth pressure at rest (K_0). So an algorithm was devised to assign the appropriate value of K_0 for different soil layers.

Actually, evaluation of the K_0 is very difficult since it depends on the geological and the stress history of the soil layers. There are many theoretical and empirical correlations to estimate this value for normally consolidated soils and over consolidated soils. But these correlation are valid for over consolidated soil only if the over consolidation is due to simple unloading (Lancellota 1995). The over consolidation due to the other effects like desiccation cannot be addressed with those formula. For the study site, luckily the soil deposits are from normally consolidated state to only slightly over-consolidated state (IPET 2007). So, the existing theoretical and empirical correlations can be applicable to evaluate K_0 with confidence.

In the literatures, K_0 values are referred for the effective stress case. Since the numerical simulation was done for the total stress case using the saturated unit weight, equation (9) was developed to calculate the "equivalent K_0" for the total stress analysis. The "equivalent K_0" incorporates the effect of horizontal pressure due to water as well. The relation was derived assuming that the total horizontal stress developed for the effective stress case and the total stress case will be the same.

$$K_0^t = \frac{\Sigma K_0 \sigma_{vi}^{'} + \sigma_w}{\Sigma \sigma_{vi}} \quad (10)$$

Evaluating the stress term in equation (9),

$$K_0^t = \frac{\Sigma K_0 \gamma_i^{'} h_i + \gamma_w \Sigma h_i}{\Sigma \gamma_{sati} h_i} \qquad (11)$$

Simplifying,

$$K_0^t = \frac{K_0 \gamma_i^{'} + \gamma_w}{\gamma_{sat}} \qquad (12)$$

Where, K_0^t denotes "equivalent coefficient of lateral earth pressure at rest condition for total stress analysis", K_o is the coefficient of lateral earth pressure at rest condition for effective stress condition, $\sigma_{vi}^{'}$ is the effective overburden pressure for the ith layer, σ_{vi} is the total overburden pressure for the ith layer, σ_w is the water pressure, h_i of depth of ith layer, $\gamma_i^{'}$ is the effective unit weight of soil of ith layer and $\gamma_{sat\,i}$ is the saturated unit weight of soil of ith layer

With equation (11), the equivalent coefficient of lateral earth pressure for total stress analysis can be used for the FLAC^{3D} simulation. The equation (10) is simplified to equation (11) to ease its use in devising a subroutine in the simulation and the K_0^t calculated using equation (12) gives an average error of only 2.5 % when compared to that using equation (11).

To use equation (12), K_0 value for the case of effective stress is needed for different soil layers. Since embankment clay, marsh and the stiff clay are from normally consolidated to slightly consolidated soils (IPET 2007), K_0 are within the range between 0.60 and 0.63 (Seed et al 2008).The value of 0.6 was chosen in this study.

Similarly, for beach sand and silty-sand, which are also normally consolidated, K_0 was estimated using the relation (Jaky, 1944):

$$K_0 = 1 - \sin(\phi') \qquad (13)$$

Gap formation

Different papers have devised different logics regarding the likelihood of formation of gaps and their depths. In the IPET report (IPET 2007), the depth of the gap is estimated numerically by checking the effective stress of the soil in the neighborhood of the sheet pile and has assumed that the gap occurs if the soil undergoes the tensile stress.

The same principle is echoed in the paper by Brandon et al. (2008), in which the depth of gap has been calculated analytically by comparing the total lateral earth pressure and the water pressure for different depths. In their paper it has been concluded that the depth of the gap solely depends upon the cohesion and the submerged unit weight of the soil independent of the load condition.

In the above discussions, the theory behind the calculation of the depth of the gap assumes that the sheet pile is rigid which rotates about its tip developing active earth pressure on the flood side and the passive earth pressure on the protected side. The method proposed by Brandon et al. (2008) can be an effective way for the calculation of an approximate depth for the hand calculation. But this might not be the case if the sheet pile is flexible. So the logics in predicting the depth of gap should be compatible with respect to the soil/ structure deformation and is only possible through numerical simulation.

In this study, the relative displacement between the sheet pile and the soil adjacent to it was monitored. If the relative displacement was such that the soil/wall interface was in tension, the soil pile interface was removed and the water load was applied to the sheet pile and the soil adjacent to it. This was achieved by deleting the link element that connects the soil zones with the structural nodes. The link in FLAC^{3D} is actually a special element which provides the means by which the structural element (sheet pile) interacts with the grid (soil). The removal of the link allows the structure and the soil adjacent to behave independently. The sub routine was so developed that the link was reinstalled when the soil/wall interface undergoes compression. This reflects the case when the wall is unloaded.

Strength reduction

As the gap forms reducing the seepage path to marsh layer, the strength of the marsh layer was reduced when the water level was high and was resumed to the original strength when the water level dropped to the MWL.

As discussed earlier, the correlation factor recommended by different papers that correlates the shear strength of the marsh to the effective stress is not unique (example with references). So trial values of the reduced strength were used for the marsh beneath the crest (since the under seepage is dominant at this section) when the water level reached 5.5 feet, 6.5 feet and 7 feet and the response of the flood wall system is observed. As the shear strength of the marsh soil of similar site at 17th Street canal near to the load test site varies from 50 to 920 psf (Sasankul et al 2008) the trial values that have been used were within this range.

Since the stiffness and the undrained shear strength of the soils have a direct correlation, the reduction of the shear strength should induce a decrease in the stiffness of the marsh soil. This will also influence the stiffness and strength of soil/wall interface as well. But incorporation of all these effects cannot be handled manually in the numerical simulation as they add complexity. This study has only been limited to the fact that the influence of gap will reduce the seepage path of the water increasing the pore pressure in the marsh layer thus decreasing the effective stress resulting in reduction of the strength for the high flood level and vice versa. This is not that implausible. But at the same time, this concept has opened the door for further study to introduce new constitutive model for the marsh layer and the soil/ wall interface which can handle all those effects.

Comparison of model predictions and test data

Summary of comparison

Three cases each of which with considering the gap formation and without gap formation were compared. The result from each of these cases were compared with the deflection curve of the monolithic base measured during the full scale load test and the Root Mean Square (RMS) of the percentage discrepancy between the measured and computed deflection at different WL were compared for each of the cases: This is illustrated in Table 2.

Table 2 comes here

Load Deflection curves

The load deflection curves in terms of water level versus deflection of the wall at monolithic base was plotted and compared with the measured load deflection curve for the different cases discussed in 4.1.

When comparing the load deflection curves for different cases, it is seen that the IPET Elastic modulus is highly underestimated (Fig.7). It can be seen that though the underestimation is not that significant for the higher water level, it is much significant for the low water level.

When the elastic modulus is adjusted to match the inclinometer results (discussed in next section), the load deflection curve is somewhat different (Fig.8). The adjustment has not made that much improvement in predicting the behavior. In this case, it is seen that the deflection from the simulation for the low water level is in close agreement with that of the measured one but it fails to predict the behavior for higher water level.

When the elastic modulus of the soil layer is adjusted and the strength of the marsh layer beneath the crest is reduced during the high water level, there has been a significant improvement in the approximation of the behavior of the system as reflected by load deflection curves for the numerical simulation and measured one (Fig.9). Due to the reduction of the strength, part of the deformation of the system has not been recovered. On top of that, when comparing the load deflection curves for this case, the approximation is again improved when the gap formation is incorporated than when it is not. So this implies that the hypothesis regarding the gap occurrence and strength reduction best represent the phenomena occurring in the flood wall system during the load test as reflected through the load deflection curve. Though the phenomenon of the strength alteration of the marsh layer at higher water level cannot be observed visually, there were some evidences that proved the occurrence of the gap during the load test.

Fig.7, 8, 9 comes here (Include the figure regarding the gap occurrence)

Soil deformation along the depth

The verification of the calibrated soil stiffness parameters can only be justified by comparing the soil deformation along the depth. During the full scale load test, the differential inclinometer was used in measuring the soil deformation (HPO 2008). The inclinometer measurement was done below the crest and the toe represented by IP-2 andIP-3 respectively. Since the measurement were done for all the water heights of the full scale load test and it was not convenient to include all the measurements, the measurements corresponding to typical loading conditions were taken into consideration for the comparison with the simulated results.

Fig. 10 shows the measured deformation of the soil along the depth for the hydraulic pressure corresponding to 6.0 ft height of water during the first phase test and 7.0 ft height of water during the second phase and the simulated results corresponding to the same loading conditions for the calibrated parameters. Though there are some discrepancies between the measured deformations with that from the simulation, the calibration of the parameters has given a close approximation of the behavior of the system.

Fig.10 comes here

Horizontal deformation of embankment

The verification of the calibrated parameters was also done for the soil and wall deformation at different points of the levee and flood walls. The points that were chosen were the top of the wall, the bottom of the wall and the edge of the crest. The load deformation curves were measured for hydraulic pressures corresponding to different water level during the first and second phase of the full scale load test. Comparing the results from the numerical analysis with the measured one, it shows that the calibration of the parameters in close approximation of the behavior of the system. This is shown in Fig. 11.

Fig. 11 comes here

Bending Moments, wall configuration and the state of soil

Though report on the bending moments, soil/wall configuration and the state of the soil has not been provided in the documents presented by HPO 2008 or Conroy and Berry (2008) so that the comparison can be made, authors of this paper noticed that it is necessary to present them to give an insight of the behavior of the flood wall system so that it can pave a path for the improved design procedure. Fig. 12 shows the comparison of the bending moments for the hydraulic pressure corresponding to 7 feet of water level for the system with calibrated stiffness and reduced strength of the marsh for the case with - and without gap. It is seen that the bending moment corresponding to gap formation is more critical than that without gap formation. Fig. 13 shows the exaggerated view of soil and wall configuration for the same loading condition. From this figure, it is seen that the gap was only partially developed for the water load corresponding to 7 feet height. This result contradicts with other literatures in which they point out that the gap normally occurs up to silty sand layer. These literatures assume that the sheet pile is a rigid structure but from Fig 12, it can be seen that the sheet pile in this study behaved as if it a flexible structure. Fig. 14 shows that the marsh layer lying between the toe of the levee and the center of the levee has been yielded. This is reflected in the load deflection curve and the history of the deflection of the monolithic base which is given in Fig. 15. Unfortunately, we don't have any evidences supporting the yielding of the marsh layer in the full scale load test.

Fig.12, 13, 14, 15 comes here

Conclusion

The objective of this study was to develop a proper numerical procedure to study the behavior of the flood wall system and to calibrate the soil parameters that can best represent the site characterization by comparing the model predictions with the full-scale test program. From the study it can be concluded that the adjustment of the modulus of the soil in IPET report and the reduction of the strength in the marsh layer during high flood with incorporation of gap gives the better result which matches closely with that of the field data. This implies that the numerical procedures adopted in this study and the calibrated modulus gives a better approximation of the behavior of the flood wall system.

From the study, it was also found out that the marsh layer is the most susceptible layer to yielding and this yielding was primarily due to the strength reduction of the layer induced by the high underwater seepage during larger load promoted by the opening of the gap reducing the seepage path. Though the seepage analysis was not conducted in this study, many papers (e,g, IPET 2007) has already proved that the opening of gap induces the high pore pressure on the land side of flood wall system. So, we recommend that this situation should also be incorporated while analyzing and designing the I-wall in New Orleans area because, apart from the high uplift pressure above the sandy layer, the strength reduction of the marsh layer might contribute significantly for the instability of the I-wall.

Using the numerical technique that has been adopted in this study and the calibrated stiffness parameter, the further analysis of the flood wall system to evaluate the proposed retrofitting techniques will be significantly more reliable.

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[2]Hurricane Protection Office(HPO)of the US Army Corps of Engineers and the St. Louis District Corps of Engineers (2008) " The London Avenue Site Specific Load test" *Appendix-D: Analysis of the London Avenue Canal Load test-Section1 Soil Structure Interaction Analysis,*1-17

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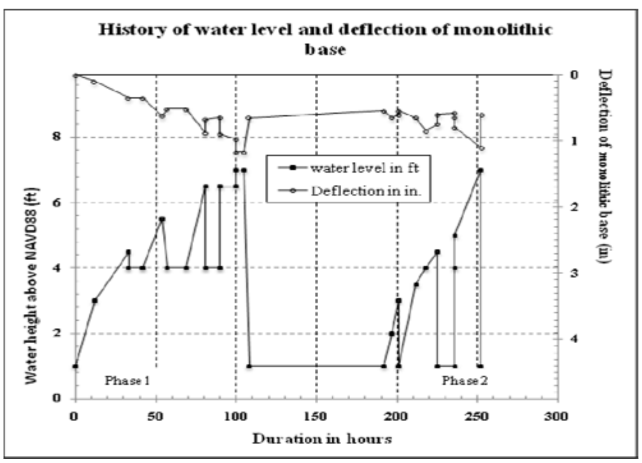


Fig1. Loading sequence and deflection pattern during the full scale load test (Source: Conroy et al, 2008)

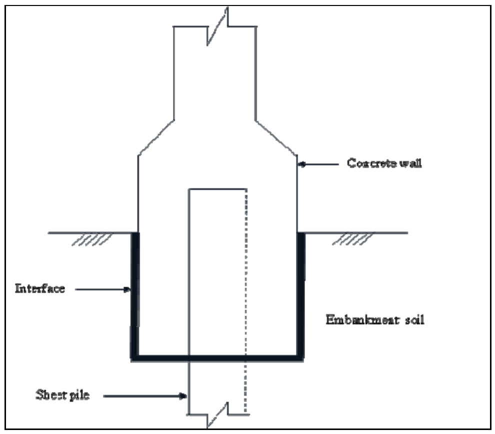


Fig.2: Typical I -wall section

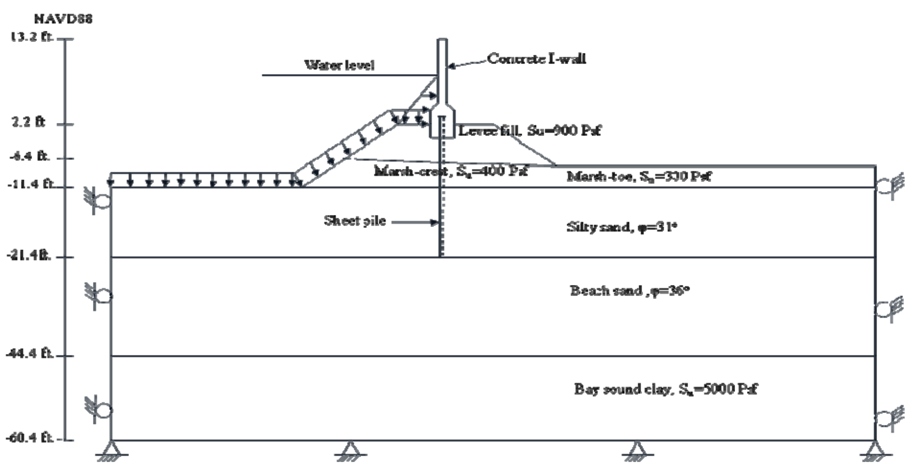


Fig. 3: Model geometry with soil profile

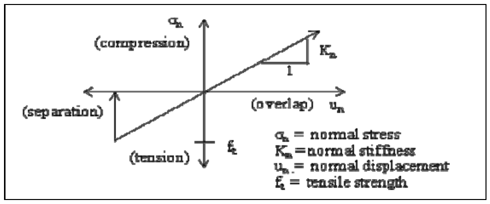
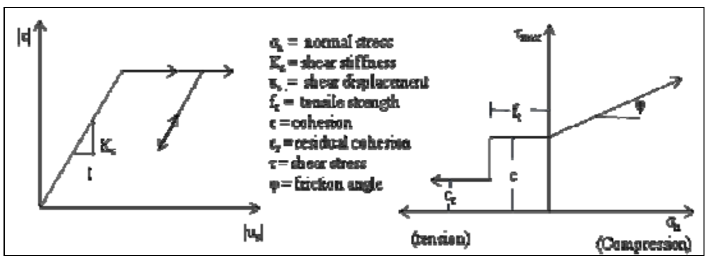
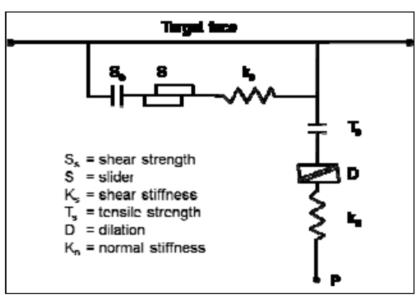


Fig.4: Normal directional interface behavior for sheet pile (Adapted from Itasca, 2006)



(a) Shear stress versus total relative displacement (b) shear-strength criterion

Fig. 5: Shear-directional interface behavior for sheet pile (Adapted from Itasca, 2006)



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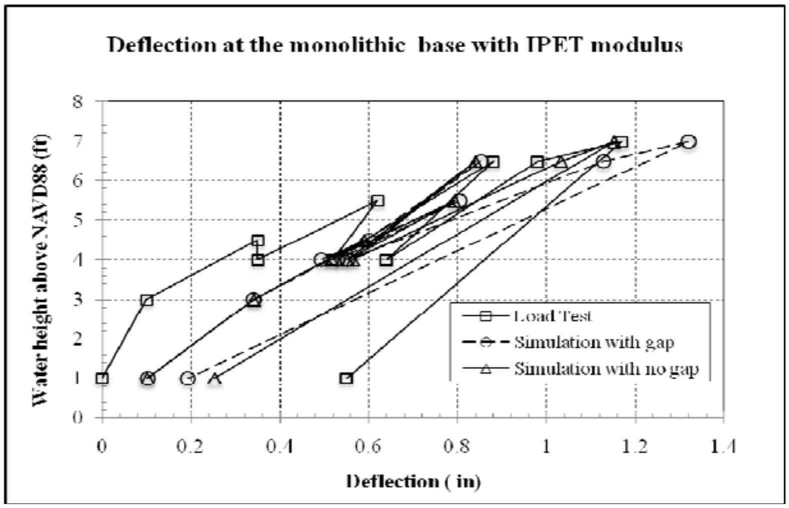


Fig 7: Load deflection curve for the numerical simulation with IPET elastic modulus

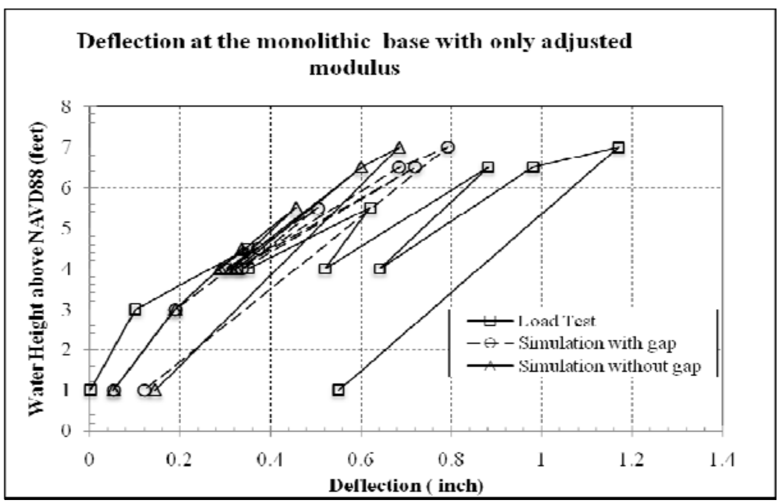


Fig. 8: Load deflection curve of the numerical simulation with adjusted elastic modulus

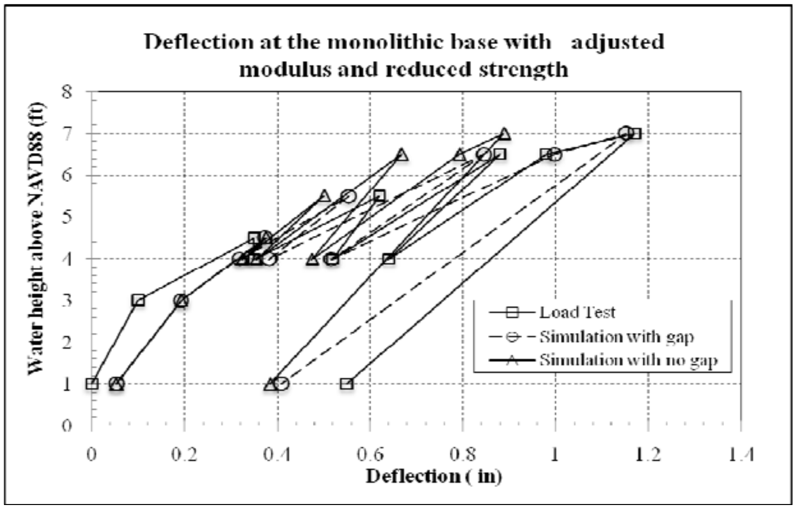


Fig 9: Load deflection curve of the numerical simulation with adjusted elastic modulus and strength reduction

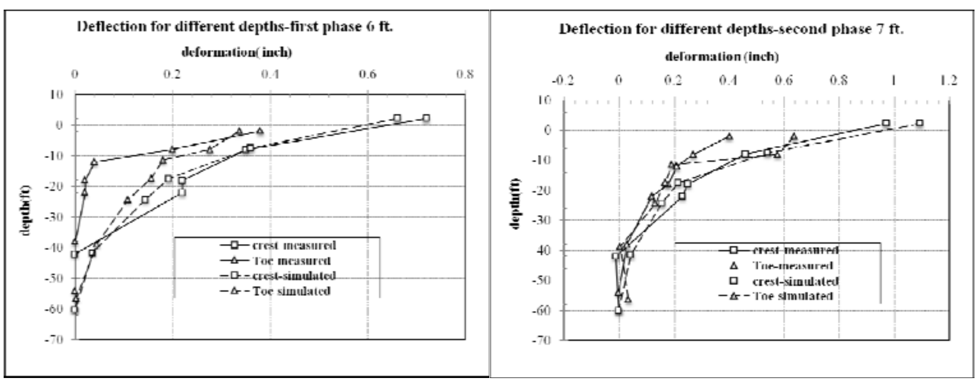
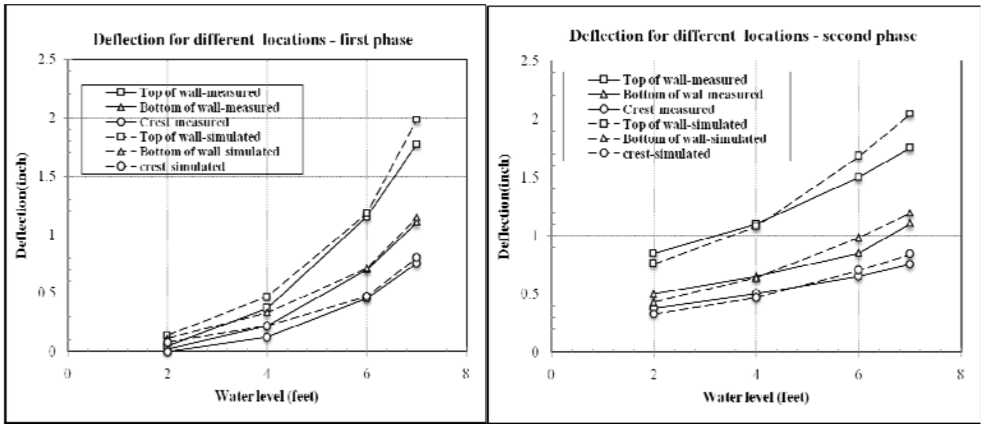


Fig. 10: Deformation of soil at different depths



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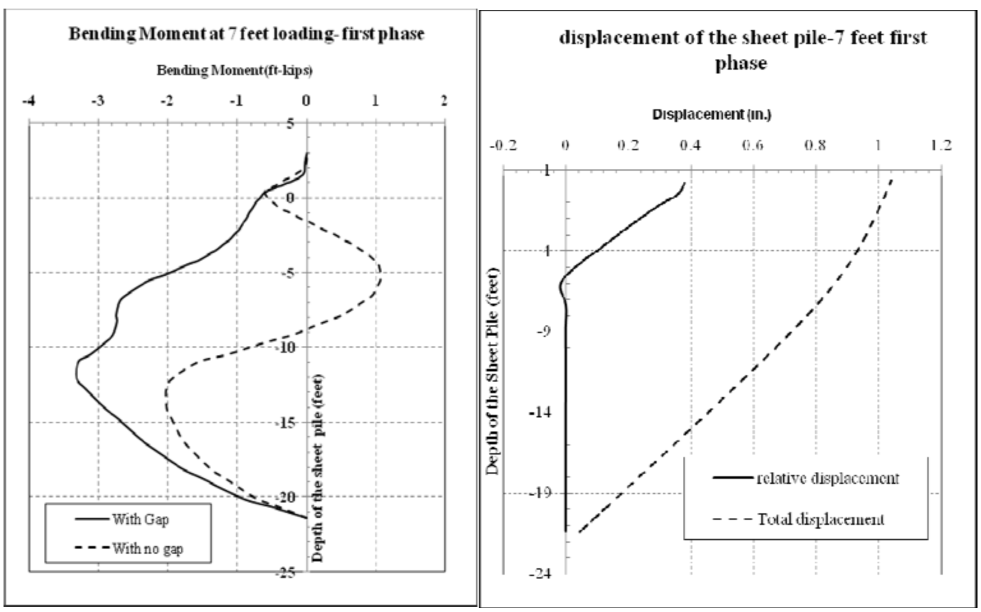

Fig.12: Bending Moment distribution and displacement along the sheet pile

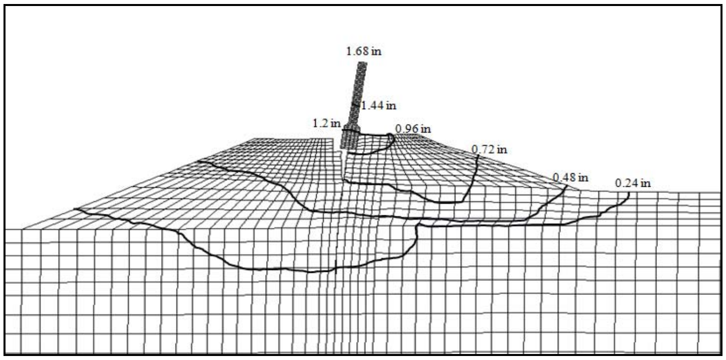

Fig.13: Configuration of wall at 7 feet loading showing the counters of horizontal displacement

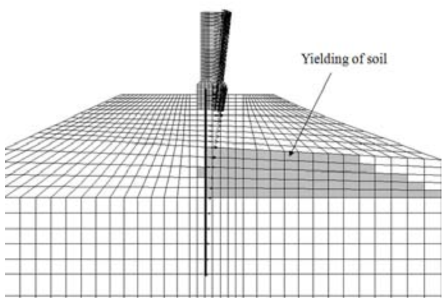

Fig.14: State of the soil at 7 feet loading with displacement vectors for flood wall

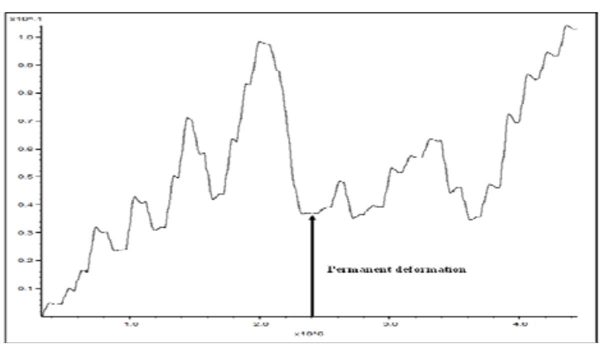

Fig15: History of the monolithic base deflection

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Soil layer	Shear Strength	Saturated density (Slug/cf)	Elastic modulus (Psf)	Poisson's ratio
Levee-fill	Su=900 Psf,φu=0°	3.39	8.28E+04	0.48
Marsh(Toe)	Su=300 Psf,φu=0°	2.49	2.76E+04	0.48
Marsh(Center)	Su=400 Psf, φu=0°	2.49	3.68E+04	0.48
Silty Sand	c'=0 ,φ'=31°	3.66	1.38E+05	0.43
Relic Beach Sand	c'=0 ,φ'=36°	3.79	1.38E+05	0.41
Bay sound Clay	Su=5000 Psf,φu=0°	3.89	4.60E+05	0.48

Table 1: Initial soil model parameters

Case	With Gap	Without gap
1. IPET elastic modulus	37	35
2. Adjusted elastic modulus without strength reduction	38	46
3. Adjusted elastic modulus with strength reduction	15	23

Table 2: RMS values (in percentage) for the different cases

Soil layer	Adjustment in elastic modulus
Levee-fill	1 times the IPET elastic Modulus
Marsh	3 times the IPET elastic Modulus
Silty Sand	2 times the IPET elastic Modulus
Relic Beach Sand	3.5 times the IPET elastic Modulus
Bay-sound Clay	12 times the IPET elastic Modulus

Table 3: Adjusted stiffness parameters for the soil

Note: The IPET elastic modulus corresponds to modulus in Table 1.