February 21, 2007

Analysis of the London Avenue Canal Load Test - Section 1
Soil Structure Interaction Analysis

General

A soil structure interaction (SSI) analysis was completed at the London Canal NE wall, denoted as Section 1. Analyses were conducted to predict the behavior of the existing I-walls for differing water levels during the London Canal load test. Goals of this work include:

- Predict wall response
- Evaluate critical moments/loads
- Set criteria for terminating the test based on study results

To evaluate displacements, sheet pile conditions and the potential for global instability, FLAC (Fast Lagrangian Analysis of Continua) models were developed using current survey information and geologic stratigraphy from the original project GDM. Two analyses were performed to estimate the structure response using soil modulus values obtained from laboratory triaxial shear strength testing (TXT) and from in-situ pressuremeter tests (PMT) performed at the three outfall canals.

Geometry

Recent survey data was used to determine the ground surface and top of wall information. Models were developed using simplified stratigraphy based on the FEM seepage study. The section stratigraphy is shown on the FEM seepage figures and consists of approximately horizontal soil layers that vary in elevation as much as 3-5 ft. This section was simplified by setting horizontal soil layer boundaries. This was done to efficiently use the I-wall template created by Itasca Consulting for Task Force Guardian work.

The design section includes: clay embankment (El. +2.5 ft NAVD88), underlain by marsh material (El -6.5 ft to -13.5 ft), underlain by a sand strata (El. -13.5 ft to -51.5 ft) that mantles a lower clay deposit. The thickness of the entire marsh layer was based on its thickness at the levee toe. Setting stratigraphy in this way reduces the levee depth at centerline from 11.5 ft to 9 ft and is conservative for estimating wall response. Correspondingly, the top of sand layer is lowered by 1 ft in the model, and will also be conservative. This same approach was used when comparing the template model to the more rigorous IPET model results. Comparison of results were very reasonable and within the margin of error of selecting soil modulus values.

The existing floodwall has a top elevation of about +12.9 ft consisting of a concrete cap and CZ-101 sheetpile to a tip El of -21.5 ft. The sheet pile from an old wall remains in the levee about 3 ft landward of the existing wall. The old wall was also assumed to be CZ-101 sheet pile.

Foundation parameters

The FLAC analyses are based on simple Mohr-Coulomb constitutive models (elastic-perfectly plastic soil behavior). Unit weight and shear strength parameters were based on values selected for the limit equilibrium analyses. Soil modulus values were determined assuming the soils are linearly elastic and isotropic using either E/Su with Poisson's ratio from the IPET report or selected pressuremeter test values of G/Su with Poisson's ratio. The parameters used for analysis are presented in table 1. G/Su values from the PMT are about 7 to 14 times higher than comparative TXT values, which were the basis for IPET studies. These values are expected to bound the actual field conditions. The at-rest earth pressure coefficients are computed only to initialize stresses during model development so equilibrium can reached in fewer computational steps.

Table 1. Summary of Soil Parameters

Material	unit wt. (pcf)	unit wt/g	Su (psf)	f' (deg)
Levee	109	3.385	900	0
Marsh (centerline	80	2.484	400	0
Marsh (toe)	80	2.484	300	0
Beach Sand	115	3.571	0	30
Bay Sound Clay	102	3.168	779	0

	Model Values using TXT Data					
Material	E/Su	E (psf)	Poisson	G (psf)	G/Su	K (psf)
Levee	48	43,200	0.47	1.47E+04	16	2.40E+05
Marsh (centerline	48	19,200	0.47	6.53E+03	16	1.07E+05
Marsh (toe)	48	14,400	0.47	4.90E+03	16	8.00E+04
Beach Sand	-	418,000	0.3	1.61E+05	-	3.48E+05
Bay Sound Clay	68	52,972	0.47	1.80E+04	23	2.94E+05

	Model Values using PMT Data			
Material	G/Su	G (psf)	K (psf)	ko
Levee	220	1.98E+05	1.32E+05	0.89
Marsh (centerline	100	4.00E+04	2.67E+04	0.89
Marsh (toe)	100	3.00E+04	2.00E+04	0.89
Beach Sand	-	2.30E+05	1.54E+05	0.43
Bay Sound Clay	160	1.25E+05	8.31E+04	0.89

Structural parameters

The two current wall structures were modeled as three beams: (1) the sheet pile from the abandoned wall; (2) the upper concrete portion of the existing I-wall; and (3) the sheet pile supporting the existing wall. Interface elements were applied to the existing wall below the ground surface. These elements allow slip and separation of the soil and wall. The shear strength of the interfaces was set at 90% of the shear strength of the weakest soil layer. Interface elements were not used along the old wall since gap formation was not expected to be a problem.

In FLAC the structural beam properties are formulated in plane stress (like a plate) and are adjusted for plane strain conditions by dividing Young's modulus by $1-\nu^2$ (ν=Poisson's ratio). The structure parameters presented in table 2 do not show the adjusted Young's modulus.

Table 2. Summary of Structural Parameters

Member	E (psi)	I (in^4/ft)	EI ($lbft^2/ft$)	A (in^2/ft)	EA (lb/ft)	ν
Concrete (1 ft wide)	3,000,000	1,728	3.60E+07	144	4.32E+08	0.20
CZ-101	29,000,000	65.01	1.31E+07	6.08/1.804=3.37	9.77E+07	0.30

Piezometric Conditions

Pore-water pressures in the beach sand materials significantly affect the results. Total head conditions for the seepage model without impeded drainage were included in the initial FLAC models, but the model failed to reach equilibrium as the pore-water pressures lifted the marsh material for the WL=1.0 ft condition. The maximum total head beneath the blanket was selected at the head condition that yielded a vertical effective stress of nearly zero at the sand/marsh interface (see figure 1). This total head line, of El. -4.0 ft, was used to represent the pore-water pressures in the beach sand material from the protected side levee toe and landward.

The FEM seepage analyses indicate that the total head of El. -4.0 ft occurs at distances of about 50-80 ft beyond the levee toe based on the canal water level. Applying a total head of -4.0 ft, landward of the levee toe, under estimates the head from the levee toe to distances up to 80 ft from the levee toe and over estimates the head farther landward.

The head in the beach sand beneath the levee and towards the canal was uniformly varied from El. -4.0 ft at the protected side levee toe to the canal elevation at the point of open seepage entry of the top of sand strata and canal slope.

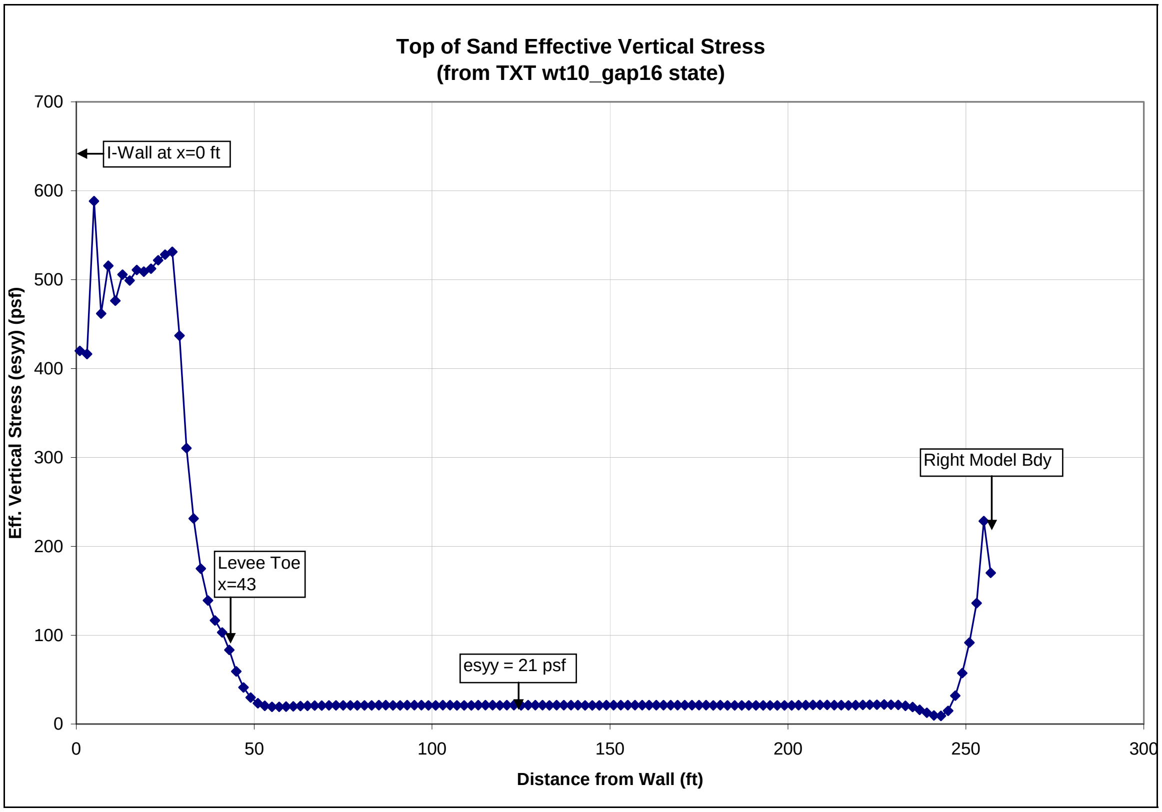

Figure 1 Effective vertical stress at top of beach landward of I-wall.

Loading Conditions and Gap Formation

Canal water loadings are modeled as mechanical pressures acting normal to the ground surface and horizontal on the wall. When a gap is included between the soil and I-wall a horizontal mechanical pressure is added to both the soil and the wall to the depth of the gap.

Gap development is modeled following the procedure used in the IPET report. The total horizontal stress in the element adjacent to the wall is compared to the hydrostatic pressure that would exist if a gap were present. If the hydrostatic water pressure exceeds the total horizontal stress it is assumed that a gap would form. Each zone is checked as canal water levels are incrementally raised. The gap was deepened in 1 to 2 ft steps as needed.

In both models there were elements where a gap would develop to some depth and then the horizontal stress was high enough to exceed the hydrostatic pressure. Then below this non-gap area there were elements where a gap could again develop if the hydrostatic pressure could reach to those depths. Judgment was used to progress the gap in these areas. If the mesh appeared as though it was being pulled by the displacing wall, a gap was added and the model was re-run to equilibrium and gap conditions were checked again. Also, if the zone of higher horizontal stress was small, a gap was added. This likely resulted in non-uniform gap progression as presented in the results.

Figure 2 shows screen captures of the model progression at different water elevations and gap/crack depths. In this figure the ini_real.sav step is the model run to equilibrium using real soil properties with a starting canal water El. 0 ft (NAVD88) and starting landside total head in the beach sand of El. -5.0 ft. At step wt1_gap0, the canal water level is raised to El. 1.0, the landside head is raised to El. -4.0, and the gap depth below the top of levee is 0 ft. After this state the canal water levels are raised in 1 ft increments and the gap is progressively deepened.

As assumed by IPET, the crack was limited to where the sheet pile intersected the beach sand deposit. It is assumed that the clean beach sand has no free standing height and will collapse against the wall. Also, there is assumed to be no head loss from the bottom of the crack to the sheetpile tip due to the relatively high permeability of the sand. In this way there is no additional mechanical pressure added to the sheet pile below the crack depth.

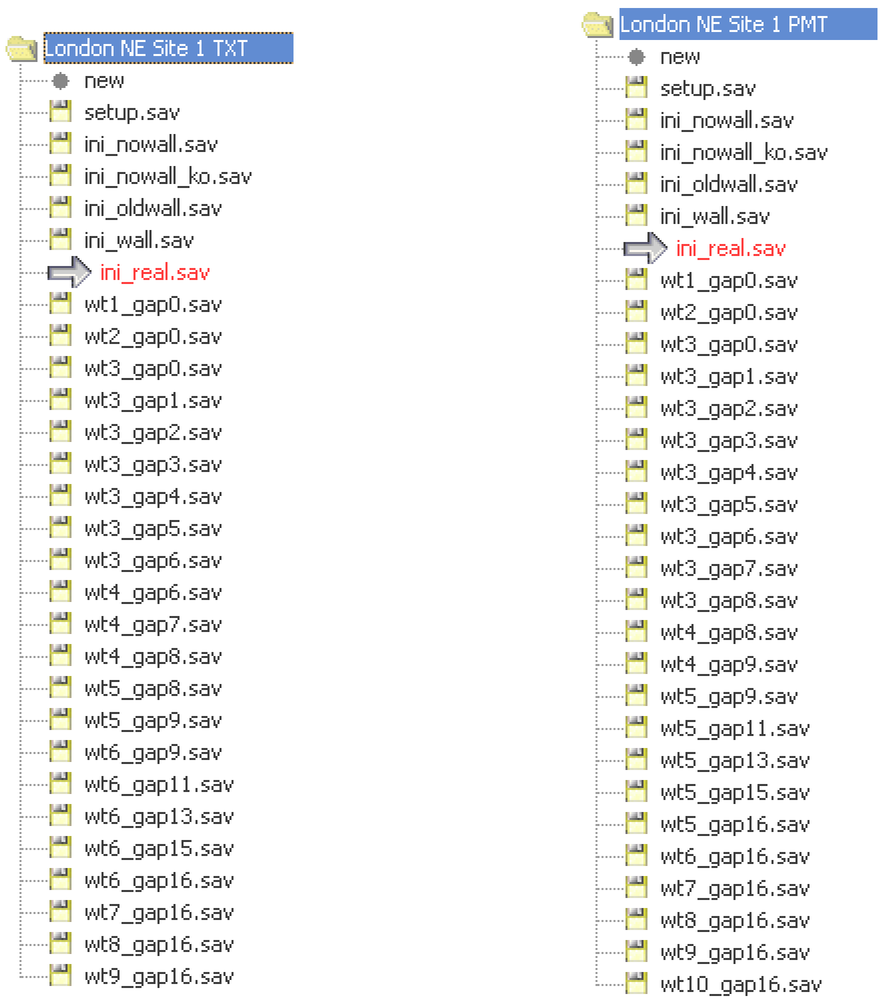

Figure 2 SSI model progression of water loading and gap/crack development.

FLAC Model

The two FLAC models are similar except for the foundation material modulus values presented in Table 1. The mesh is generally constructed of quadrilateral elements about 2 ft square as shown in figure 3. The left and right boundaries are fixed in the x-direction (rollers) and fixed in both x and y directions (pinned) along the base. The boundaries were set at a distance that has minimal effect on results. The right boundary is a distance of 264 ft from the existing wall. The conservative approach of assigning zero effective stress at the top of sand to the right boundary limit allows greater displacement near this limit than will likely be experienced. The walls are attached to the mesh at nodal locations. Because of the selected mesh size, the old wall is located only 2 ft landward of the existing wall.

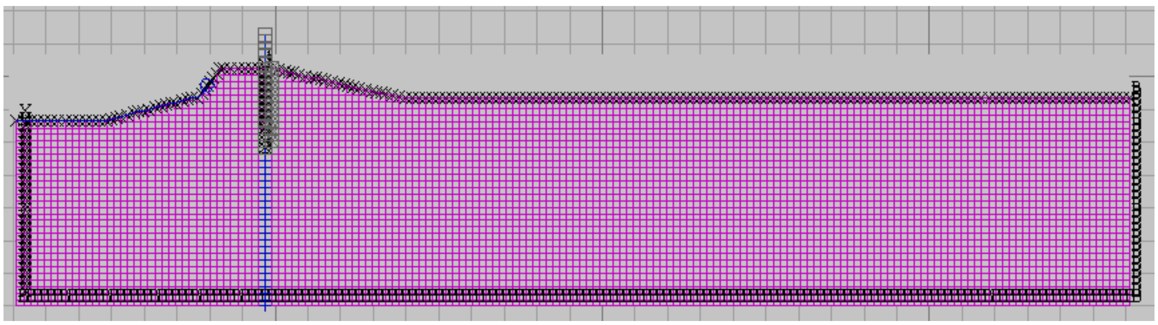

Figure 3 FLAC mesh configuration with existing wall and old wall on protected side.

The FLAC model is built slowly using small load increments as discussed above and in the IPET report. Factor of safety computations were completed using the c-phi reduction technique on the ultimate soil strengths and interfaces. The factor of safety steps are not shown in figure 2 but were completed on the project state for those water levels 3-10 ft with the greatest gap depth.

Figure 4 shows the selected stratigraphy by plotting contours of cohesion in 25 psf increments. The vertical line shown in the middle of the figure is the location of the existing I-wall, where 2-halves of the model are joined, and is not the I-wall itself. The I-wall and old wall are shown in figure 5 with interface elements extended to 1 zone below the sheet pile tip of the existing wall.

Results

Results of the SSI analyses are very sensitive with respect to soil modulus values and uplift conditions beneath the marsh material. The selection of the pore-water pressure in the beach sand materials, so that the effective vertical stress at the top of the sand layer is near zero, results in very low shear strength at the top of the sand strata. The outcome of low shear strength at the top of sand is high shear strain and deformation.

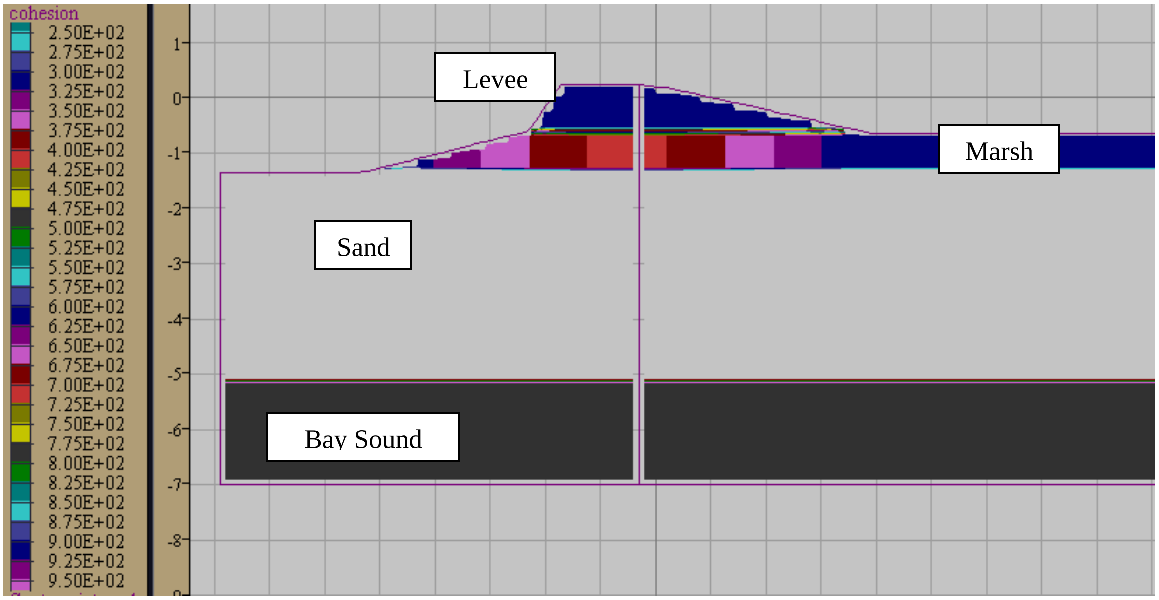

Figure 4 Stratigraphy shown by soil cohesion (sand ϕ’=30deg; c=0).

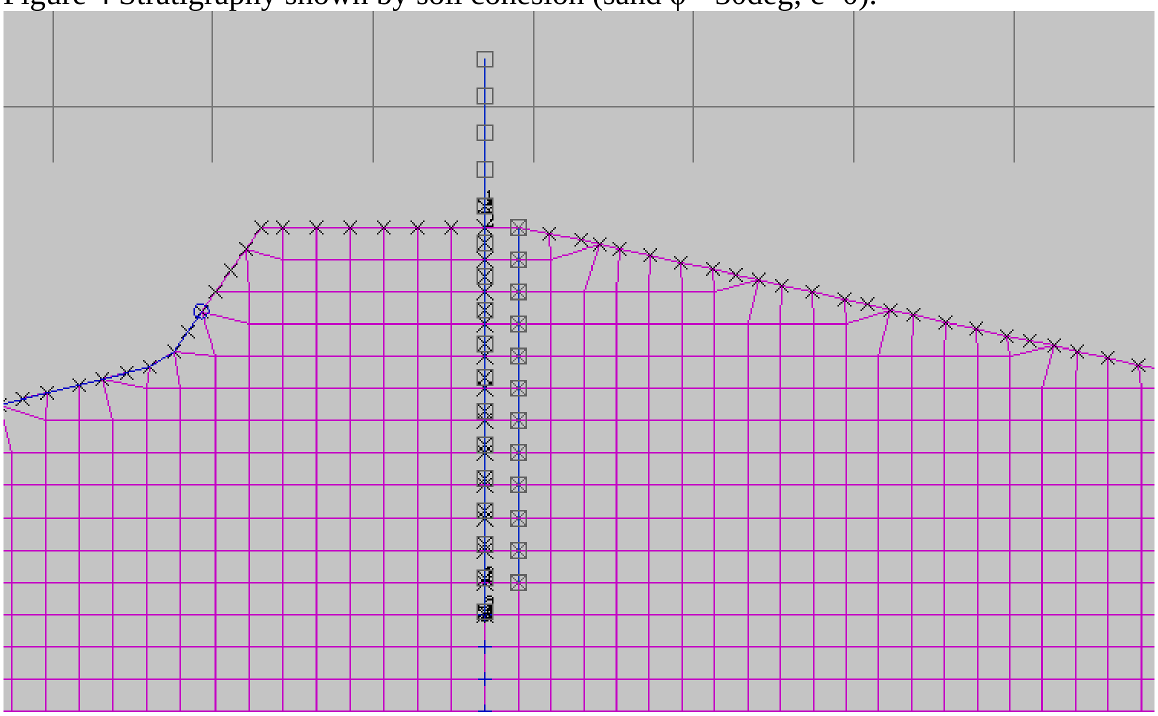

Figure 5 Existing and old wall locations. Existing wall interface extends below pile tip. The old wall is connected to the mesh at nodal points with no interface.

Deformation

Figure 6 presents the computed deflection of the existing I-wall using TXT modulus parameters. Using parameters based on the TXT data results in large deformations. Note that there is little bending in the deflected members so bending moments are small. Figure 7 presents the deflected shape of the model magnified 20X for the canal water level at El. 6.0 ft. Note the localized shearing in the zones below the marsh layer as seen by the angular grid distortion. Shearing is also evident in the plot of shear strain increment shown in figure 8. Note in figure 8 that the boundary conditions do not restrict the development of the shear strain increment for this water level.

Similarly, figures 9 through 11 depict results based on PMT modulus parameters. Deformations are significantly lower than those predicted using TXT parameters but the landward extent of shear strain is about the same for the PMT and TXT models.

Figure 12 compares the differential displacement of the wall from the top of wall to the top of berm. This displacement will be monitored during the load test using tiltmeters and ground surveys. Although the displacements are significantly affected by the soil modulus values, they will also be affected by the piezometric conditions. Figure 1 summarized the effective vertical stress at the top of the sand under high uplift conditions. Under this condition, there is unrecoverable displacement in the sand as shown by the plasticity presented in figure 13. Under lower uplift conditions, the available shear strength will increase in the sand and displacements should decrease.

In the current model, the high uplift in the sand strata is resulting in minimal shear strength beneath the marsh material. Figures 14 and 15 show the displacement vectors and the plasticity indicators for the wt10_gap16 state in the PMT model. Displacement is occurring in the marsh material as this is loaded laterally. Figure 16 presents the stress tensors and shows that the major principle stress is σ_{xx}. The application of high uplift under the entire marsh layer may not be realistic at great distances from the wall and the model will need to be changed when real piezometer data is obtained.

Gap Development

IPET demonstrated the significance of the formation of the flood side gap. Under the high head conditions modeled for the open seepage entrance the gap forms quickly to the top of sand layer for both modulus conditions (TXT and PMT values). Figure 17 shows that the gap tip reaches the top of sand at canal water levels of 5 or 6 ft. This differs from the IPET report where the gap reached the top of sand between canal water levels of 6 to 8 ft. This difference is likely affected by differing uplift assumptions and the wall location within the levee section. Figure 18 shows displacement vectors for a low canal water level and the location of the existing I-wall is seen near the landside levee crest. This condition results in low horizontal stress on the flood side of the wall and these low stresses are easily overcome by the hydrostatic water pressure when water exceeds the top of levee. It is interesting that the gap progresses slightly faster under the higher modulus condition using the PMT values.

Factor of Safety

The factors of safety computed using limit equilibrium methods and FLAC will differ since the pore-water pressures in the beach sand are different. Figure 19 summarizes the factors of safety for both the PMT and TXT based models. Since ultimate strengths are the same in these models, there is little difference in computed factor of safety. Since the gap forms at nearly equal canal water levels, only minor differences are observed at water levels below El. 6 ft.

As with the computed deformations, factors of safety will be affected by uplift conditions. The uplift conditions used in these models are lower than computed by the FEM seepage at the levee toe and resulting factors of safety are expected to be higher than computed from the limit equilibrium analyses.

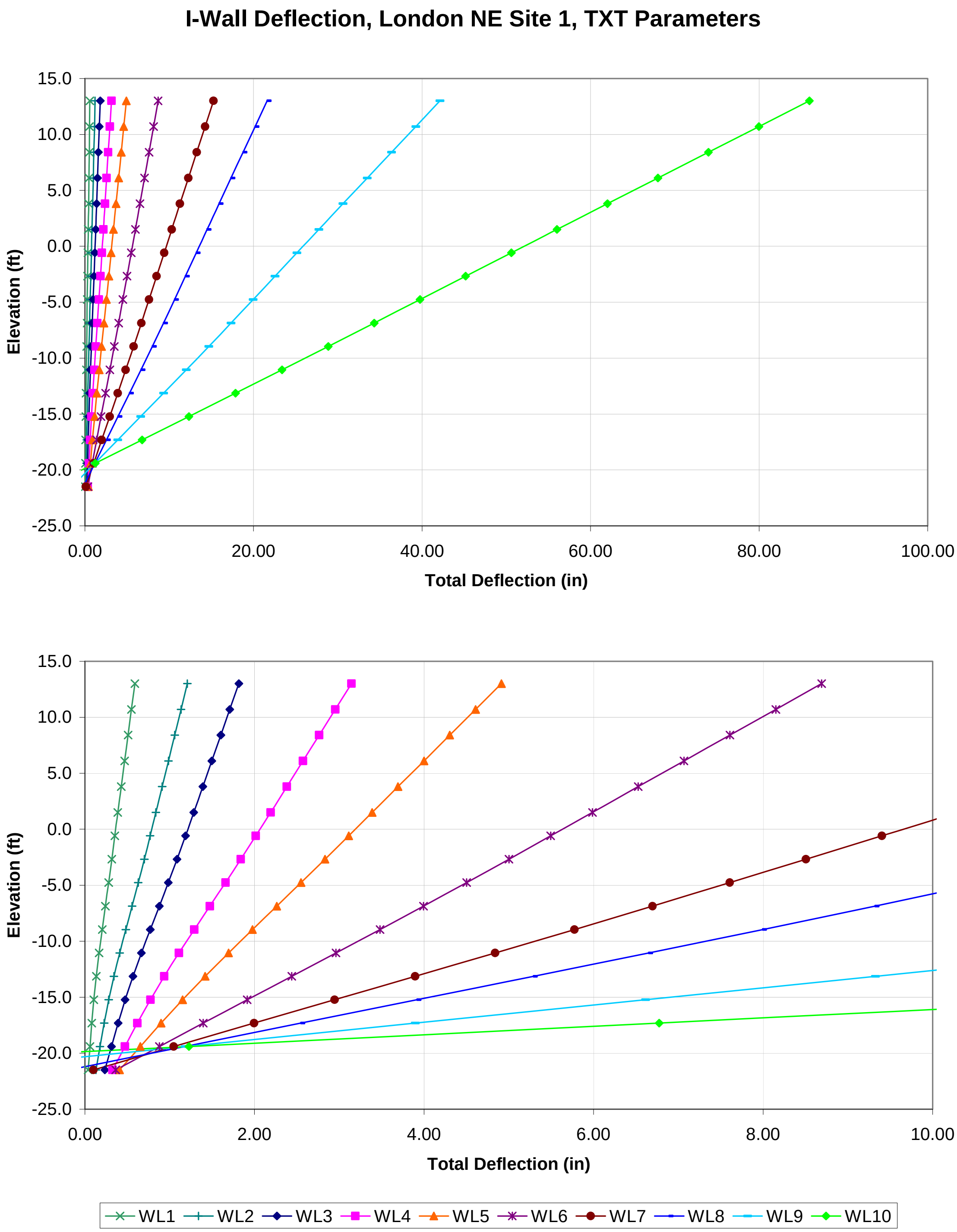

Figure 6 Full and enlarged plots of I-wall deflection using TXT parameters.

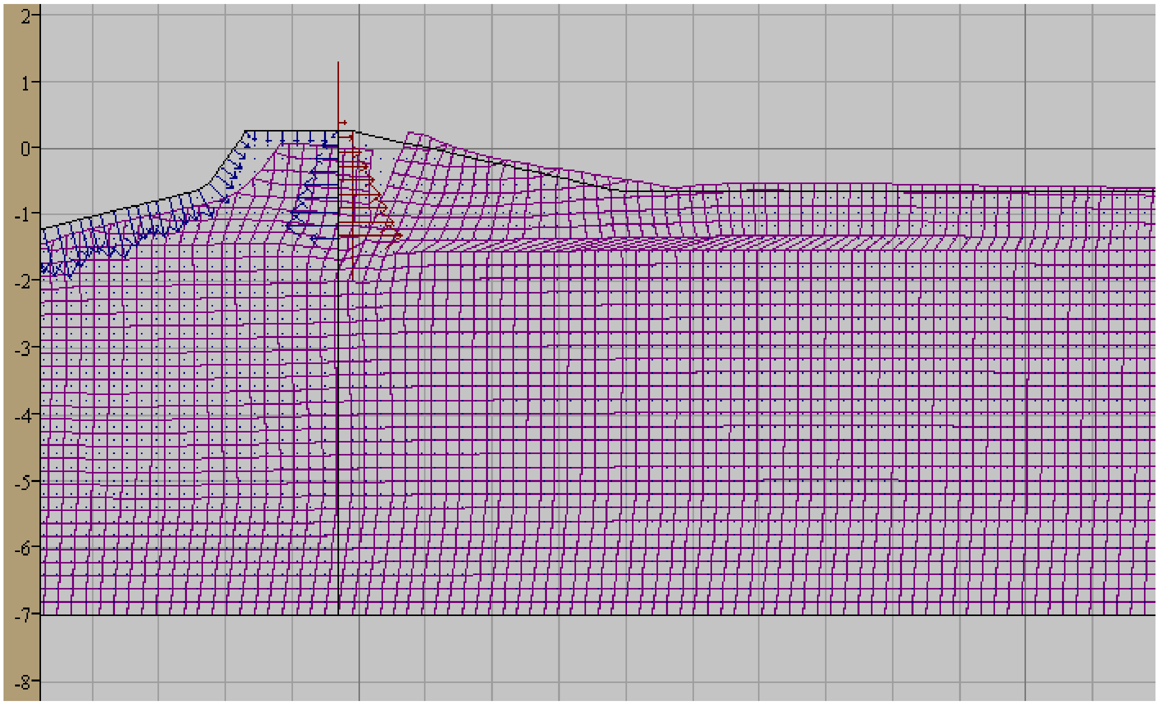

Figure 7 Deformed mesh (x20) for TXT wt6_gap16 state. Note shearing at top of sand.

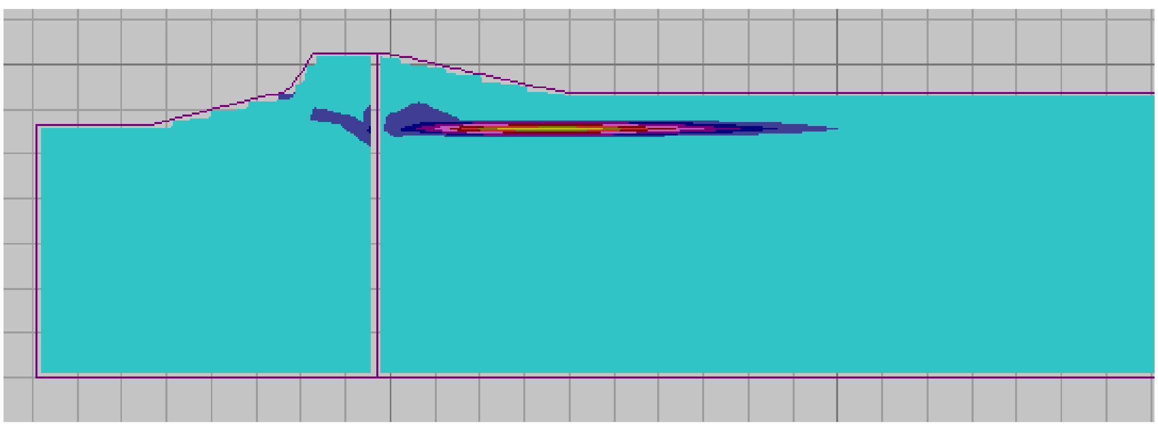
Figure 8 Shear strain increment in beach sand identifying zones of high shear.

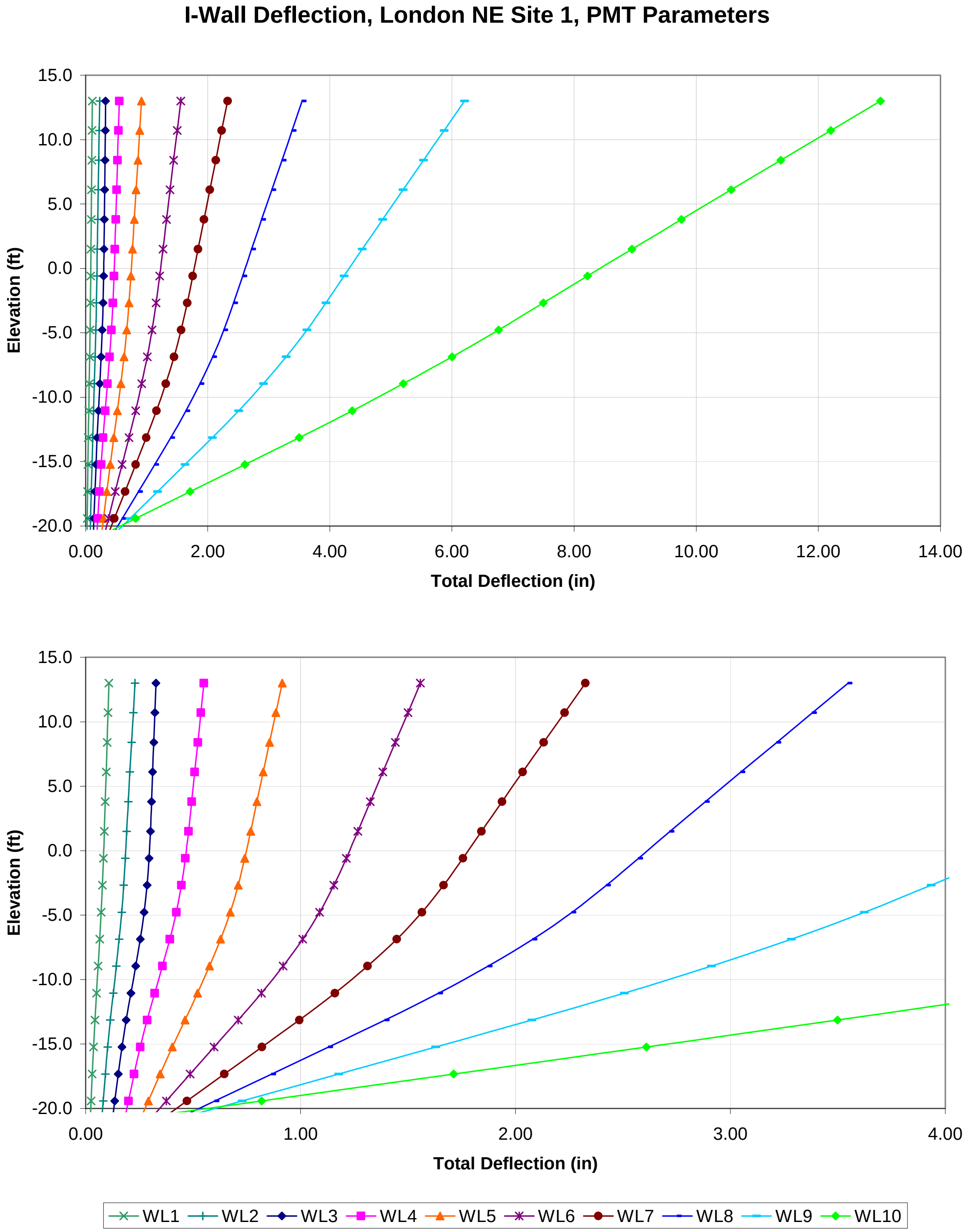

Figure 9 Full and enlarged plots of I-wall deflection using PMT parameters.

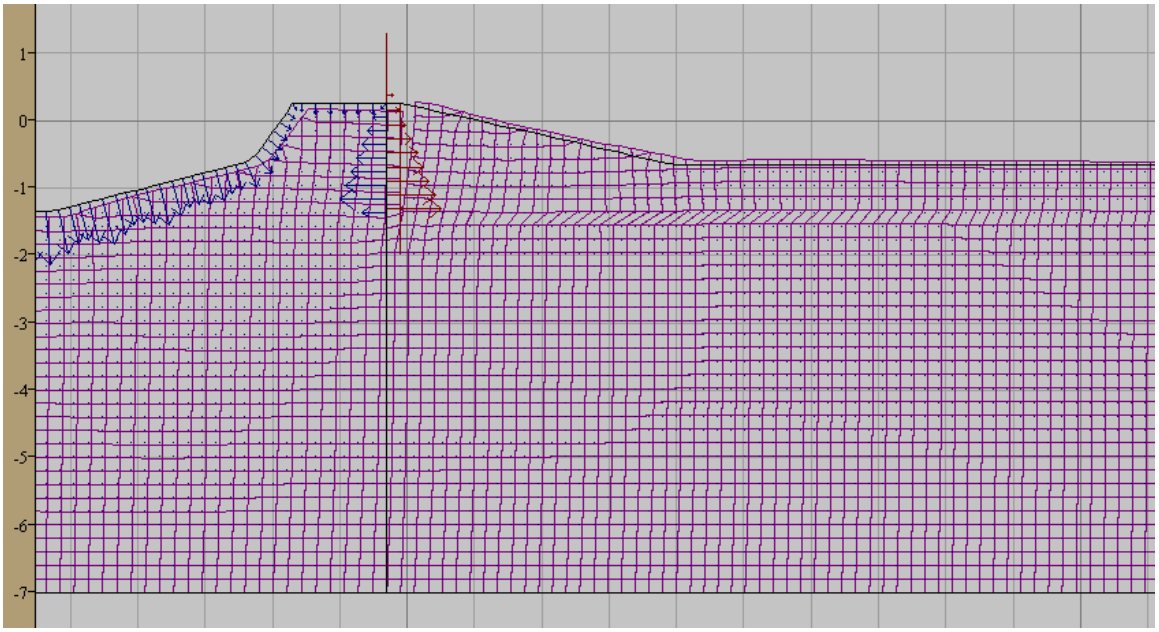

Figure 10 Deformed mesh (x40) for wt6_gap16 state. Note less shearing at top of sand than TXT model.

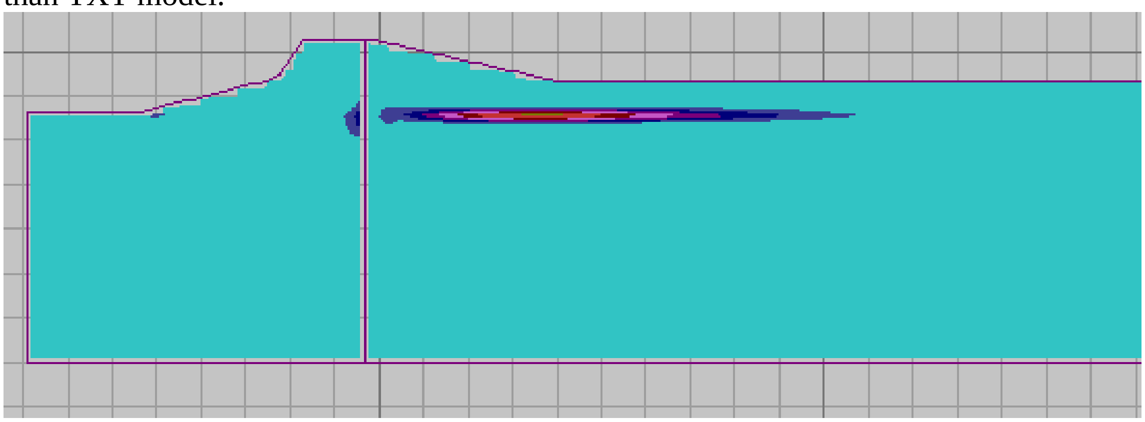

Figure 11 Shear strain increment in beach sand for wt6_gap16 state. The magnitude of shear strain increment is about 4 times lower than for the TXT model but the extent is similar (about 100 ft landward of the existing wall).

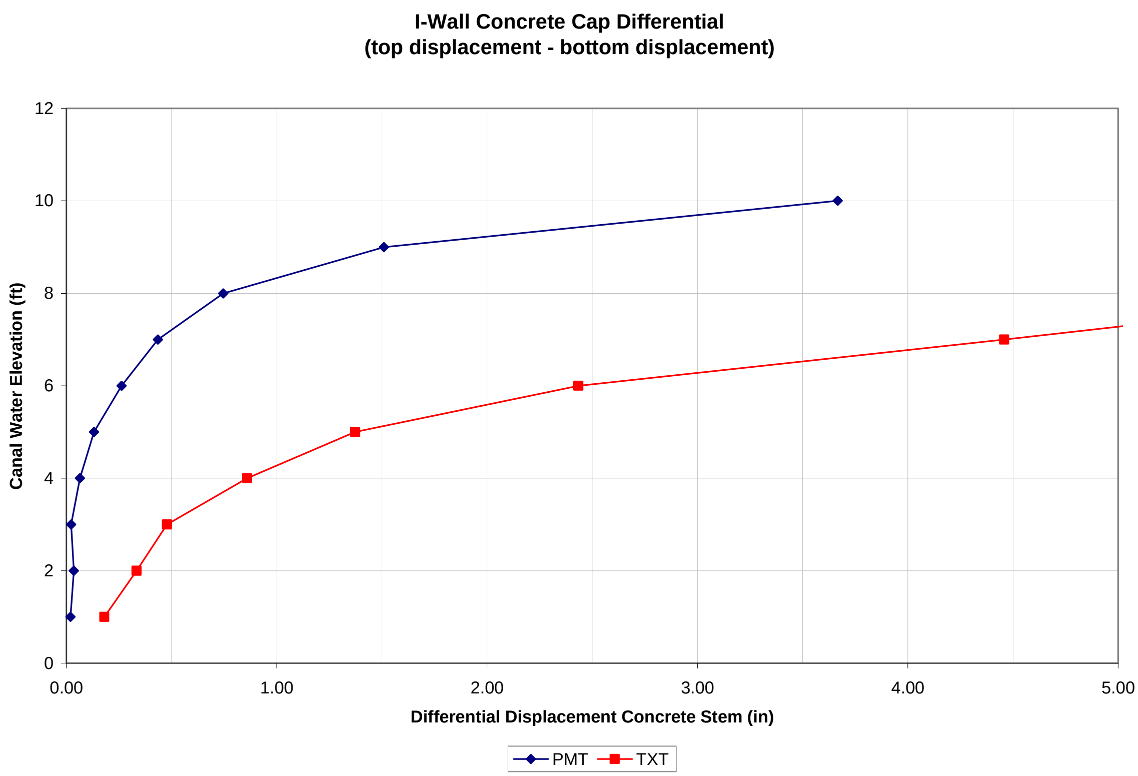

Figure 12 Differential displacement of the concrete cap.

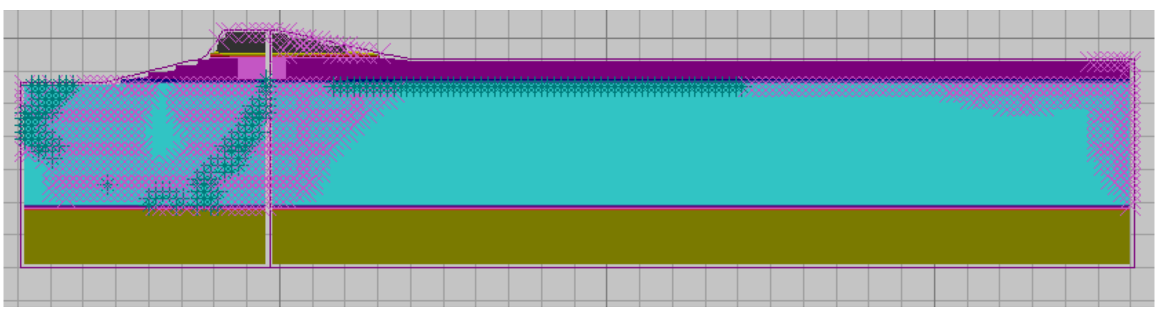

Figure 13 Zones of plasticity (blue X's) show zones yielding in shear. Note the extent of shear beneath the marsh material for the PMT wt6_gap16 state.

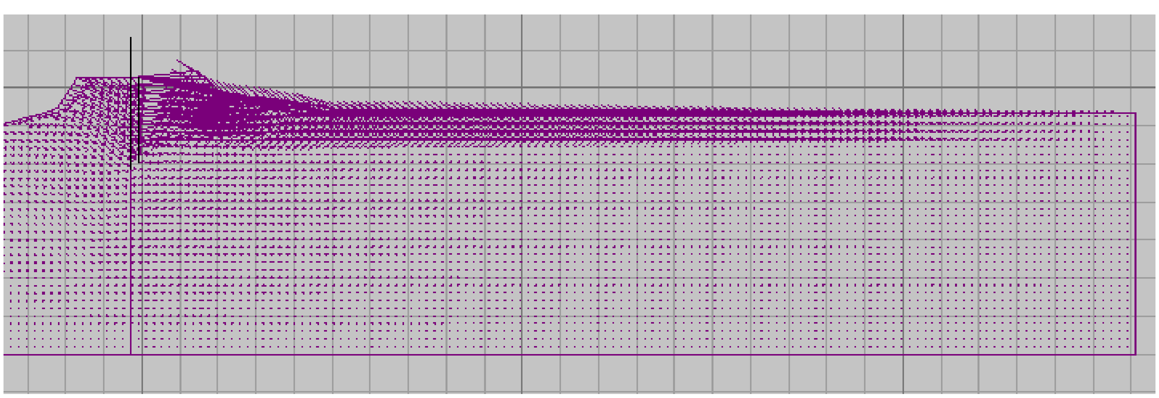

Figure 14 Displacement vectors for PMT wt10_gap16. Note that displacement horizontal in the marsh material.

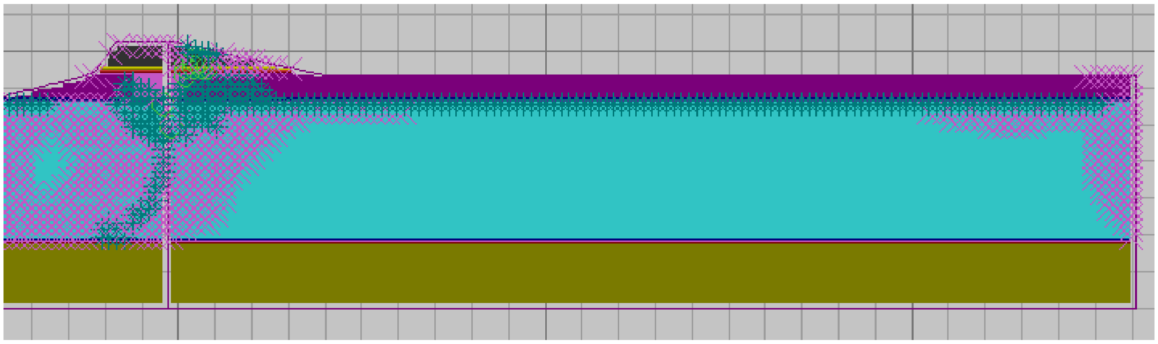

Figure 15 Zones of plasticity show yielding in shear to the right boundary. This condition is due to high uplift in the entire sand strata.

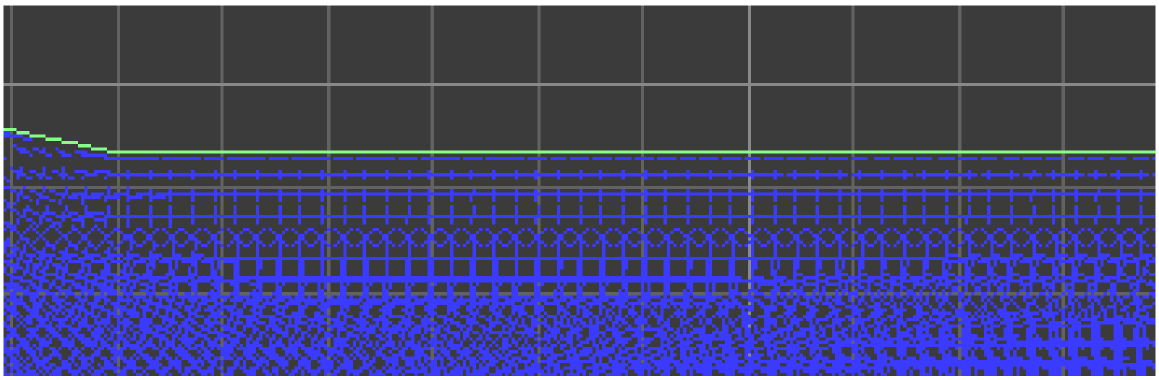

Figure 16 PMT wt10_gap16 state showing stress tensors in marsh material. Note the maximum principle stress is σ_{xx}, not σ_{yy} (typical for marsh layer to the right boundary)

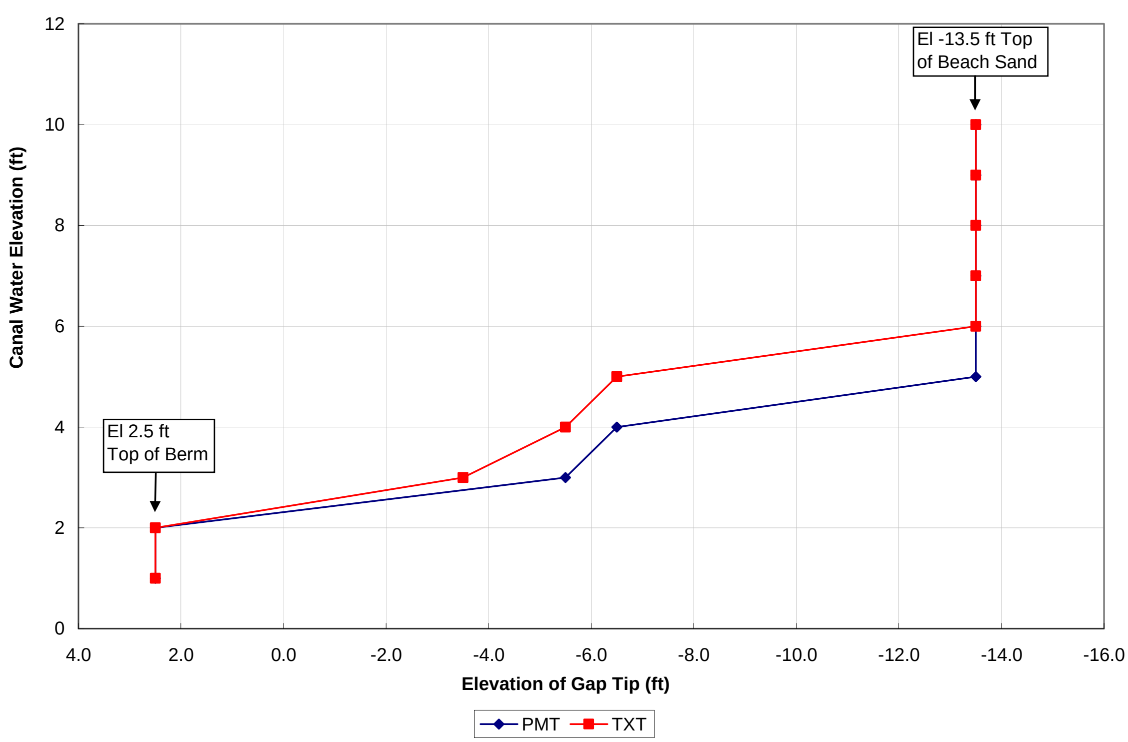

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Figure 17 Flood side gap progression with canal water level.

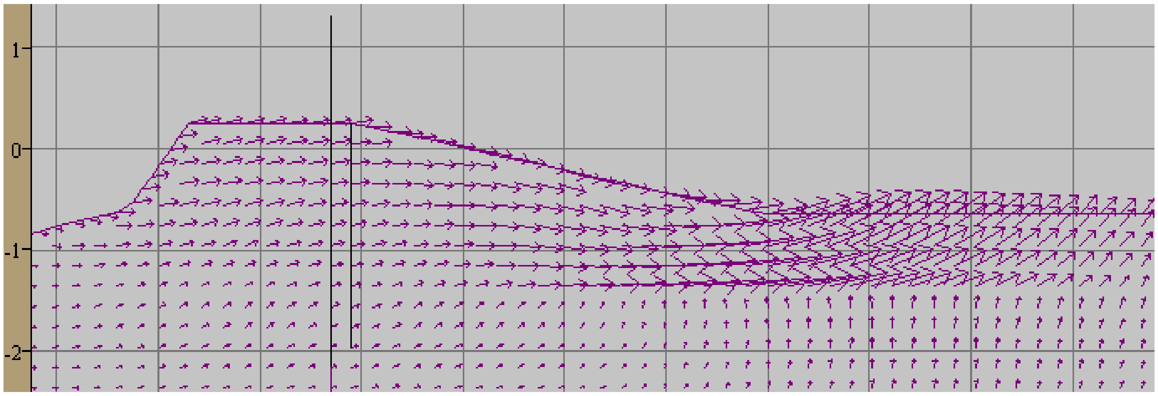

Figure 18 Displacement vectors for PMT wt1_gap0 state.

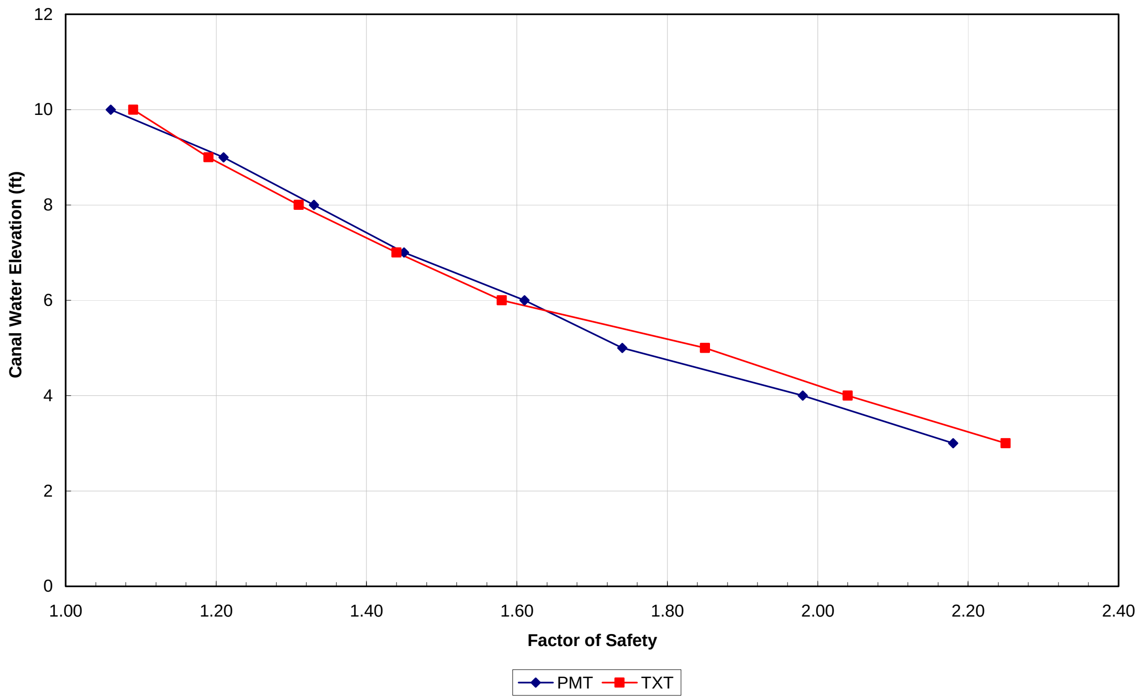

Figure 19 Factors of Safety for Varying Canal Water Elevations

Load Test Conclusions

There are two critical uncertainties associated with predicting performance of the I-wall when estimating deformation and factor of safety. These uncertainties are:

(1) Pore-water pressures in the beach sand
(2) Site specific soil modulus values

The pore-water pressures estimated from the FEM analyses exceeds the overburden stress from the marsh layer so simplifying assumptions were made for these calculations. These assumptions reduced the uplift near the levee toe and increased the uplift at the right boundary. Simplifying the uplift conditions leads to conservative results for deformation but may over estimate the stability since the computed heads near the levee toe were reduced in the FLAC analyses. Also, relationships from triaxial testing and from pressuremeter testing were used to bound the expected modulus values. Since these relationships were developed as a function of material strength, the estimates of modulus will vary if the in-situ strength differs from that assumed. This is compounded by the greater uncertainty in establishing the strength to modulus relationships.

From these analyses it is suggested that criteria used to halt the test be based on field observations and measured conditions coupled with judgment based on experience and seepage, stability and deformation model behavior. Wall displacement should not damage the existing water stops or overstress the structure. Structure overstress does not appear to be a concern using 2D conditions, but deflection between monoliths, especially at the cofferdam tie-in monoliths, should be closely monitored. Figure 20 summarizes the displacement of the wall at the top of levee (El. 2.5 ft) for differing canal water levels. Using the PMT parameters a deflection limit of 1.5 inches on the wall at the top of levee would equate to a canal water level of about 6 ft.

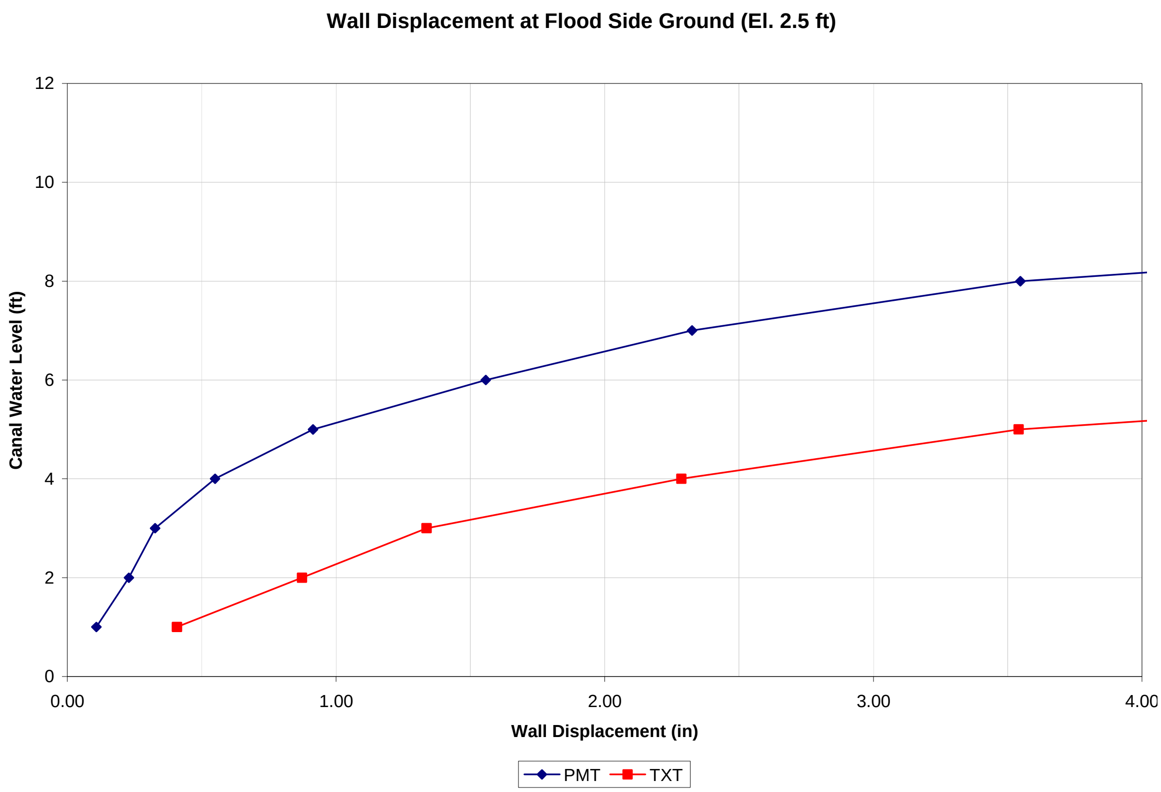

Figure 20 Wall Displacements at the Top of Levee.

From a slope stability approach, the factor of safety from limit equilibrium analyses should be used for decision purposes. Since results are sensitive to pore-water pressure conditions models should be established using the most recent boring and testing results. These can be updated in the field during the load test process.