February 10, 2007

Analysis of the London Avenue Canal Load Test - Section 1

Seepage and Stability Analysis

Thomas L. Brandon

Geometry

The basic geometry used for the analyses was based on pre-Katrina LIDAR surveys and post-Katrina measurements taken for the east bank I-wall, just south of R. E. Lee Avenue. The elevation of the tip of the sheet pile was assumed to be located at elevation -21.5 ft NAVD88. A schematic of the basic geometry is shown in Figure 1. This geometry is assumed to be representative of that found between GDM stations 107+00 to 114+00 on the east bank of London Avenue Canal, and this area has been termed "Section 1."

Erosion, Piping, and Heave

Finite element analyses of seepage beneath the I-wall were performed using the computer program SLIDE[1]. The characteristics of the cross section analyzed are shown in Figure 1. The relevant materials are the sand at the base of the section, the overlying marsh layer, and the clayey levee fill. The permeability of the sand, based on field pumping tests, was $1.5x10^{-2}$ cm/sec. The permeability of the marsh layer was estimated as $1x10^{-5}$ cm/sec, and the permeability of the levee fill and the Bay Sound clay was estimated as $1x10^{-6}$ cm/sec. The seepage analyses conducted assumed that a steady state seepage condition had been achieved.

The hydraulic boundary conditions used in these analyses are also shown in Figure 1. The canal water level was varied in the analysis, from El. 1 ft to El. 9.5 ft NAVD88. A constant-head boundary condition was imposed at the far protected side boundary, with a head value of -7 ft. This is approximately equal to the elevation of the ground surface. The location of the protected side constant head boundary is approximately the same distance from the I-wall as used in the other London Avenue Canal I-wall analyses reported in the IPET report.

Seepage analyses were conducted for conditions where a *gap* exists between the I-wall and the canal-side embankment, extending down to the marsh/sand interface, and for conditions where *no gap* exists. The presence of the gap allows a direct conduit of flow from the canal to the underlying sand layer via a flow path next to the I-wall. It should be noted that in all analyses, there is direct communication at the bottom of the canal between the canal water and the underlying sand layer. In actuality, the canal may be "silted in," and this would provide an impeded flow boundary.

Six-node triangular elements were used for the seepage analyses. The general finite element mesh used in the analyses is shown in Figure 2. There may have been slight variations in the mesh shown and those used in the different analyses since SLIDE requires that a new mesh be automatically regenerated when changes in the boundary conditions are imposed.

---

[1] Available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5

The results of the seepage analyses are analyzed in two different ways. First, the exit gradient at the toe of the protected side embankment has been calculated, and contours of vertical hydraulic gradient have been plotted to evaluate the potential for erosion or piping. Second, the effective stress at the marsh/sand interface has been calculated to evaluate the potential for heave.

The value of hydraulic gradient that would theoretically cause erosion of the marsh layer is

$$i_{critical} = \frac{\gamma_b}{\gamma_w} = \frac{\gamma_s - \gamma_w}{\gamma_w}$$

where $i_{critical}$ = hydraulic gradient that would cause erosion of the marsh material, γ_b = buoyant unit weight of the marsh material, γ_w = unit weight of water, and γ_s = saturated unit weight of marsh material. With γ_s for the marsh assumed to be 80 pcf, the critical hydraulic gradient is 0.38. It is important to note that the above analysis assumes that the only force resisting erosion is the weight of the marsh material. The contribution of the shear resistance of the marsh soil to combat erosion is not included in the analysis.

Shown in Figure 3 are locations where the vertical hydraulic gradient exceeds the critical hydraulic gradient of 0.38 for a canal water level of 1 ft NAVD88, and no gap exists between the I-wall and the protected side soil. As shown by the figure, there are substantial areas where the gradient exceeds the critical gradient. Figure 4 shows the results of an identical analysis, except that a gap is now present between the I-wall and the canal side embankment down to the marsh/sand interface. The presence of the gap only slightly increases the zone where the hydraulic gradient exceeds the critical hydraulic gradient. The maximum hydraulic gradient for canal water elevation of 1 ft is 0.71 for the “no gap” case and 0.74 for the “gap” case.

Figures 5 and 6 show similar results for a canal water level of 4 ft NAVD88, for the “no gap” and the “gap” case. Similar to the previous results, the presence of the gap only slightly increases the zone where the hydraulic gradient exceeds 0.38. The maximum hydraulic gradient for canal water elevation of 4 ft is 1.00 for the “no gap” case and 1.07 for the “gap” case.

Figures 7 and 8 show the results for a canal water level of 6 ft NAVD88, for the “no gap” and the “gap” case. The maximum hydraulic gradient for canal water elevation of 6 ft is 1.19 for the “no gap” case and 1.26 for the “gap” case.

In summary, even for modest canal water levels, a hydraulic gradient greater than 0.38 exists both for conditions where no gap exists, and for conditions where the gap extends down to the marsh/sand interface. The presence of the gap does not greatly influence the exit gradient, as the majority of the flow occurs through the canal bottom, and little head loss takes place in the sand layer.

The computer program PHASE2[2] was coupled with SLIDE to calculate the effective stresses existing at the marsh/sand interface. Total stresses at the interface were calculated using the finite element method with PHASE2 assuming linear elastic behavior of the soils, and pore pressures were calculated using the results of the finite element seepage analysis. Shown in Figure 9 is the zone where the pore pressures exceed the total stress for a canal water elevation of 1 ft for the “no gap” condition. Figure 10 shows the results of an identical analysis, but includes

[2] Available from Rocscience Inc., 31 Balsam Avenue, Toronto, Ontario, Canada M4E 3B5

the condition where a gap exists between the I-wall and the canal side embankment down to the marsh/sand interface. As indicated from the figures, the pore pressure acting at the base of the marsh layer exceeds the total stress, indicating a potential heave condition. The presence of the gap between the I-wall and canal side soil tends to increase the pore pressure at this interface, exacerbating the heave problem. This heave condition could result in a rupture of the marsh layer at one or more weak points, creating an upward flow of water through the rupture. This flow would relieve the high water pressure locally, and create a new hydraulic boundary condition at the point of rupture. Volume V of the IPET report provides details on an analysis technique to estimate the severity of this condition, but that analysis has not been conducted as part of this study.

Figures 11 and 12 show similar results for a canal water level of 4 ft for the “gap” and “no gap” conditions. As expected, the zone of negative effective stress increases with increasing canal water level, and potential heave problems are more pronounced.

Similar to the results of the erosion analysis, even low canal water levels tend to produce heave conditions which can be deemed unsafe when there is a direct conduit of flow from the bottom of the canal to the sand layer. The low unit weight of the marsh layer does not provide much resistance to either erosion or heave. The presence of the gap slightly increases both the potential for erosion and heave.

Shear Stability Analyses

The results of the finite element seepage analyses were also incorporated into the stability analyses. The stability analyses were performed for the same cross section used for the seepage analyses, which is shown in Figure 1. The same hydraulic boundary conditions were used in the stability analysis as the seepage analysis.

Standard Penetration Tests performed in the area of Test Section 1 showed that the upper areas of the sand layer had Standard Penetration Test blow counts (N_{SPT}) in the range of 2 to 14 blows, which would correspond to values of ϕ' in the range of 30 degrees to 32 degrees. In order not to overestimate the strength of the sand, a value of ϕ' = 30 degrees was used in the stability analyses.

The marsh was treated as an undrained material, with s_u = 300 psf, and $\phi_u = 0$. The unit weight of the marsh was assumed to be 80 pcf. The levee fill was also treated as an undrained material, with s_u = 900 psf, and $\phi_u = 0$. In only a few cases, the slip circles slightly intersect the levee fill, therefore, the levee strength has a very small influence on the calculated values of factor of safety. The unit weight of the levee fill was assumed to be 109 pcf. These are the same undrained shear strengths and unit weight values used in the IPET report for the west bank failure south of R. E. Lee Avenue.

Analyses were conducted assuming intimate contact between the I-wall and the canal side soil, and for conditions where a gap extended to the marsh/sand interface. For the “gap” analyses, it was assumed that the crack would not extend into the sand, because the sand is cohesionless, and would be expected to slump and fill any gap. Reduced hydraulic pressures, owing to head losses in the sand, were applied to the sheet pile section embedded in the sand. In addition, an active earth pressure force, calculated based on the effective stresses incorporating the reduced pore pressures, was also applied to the sheet pile in this area. This “half-cracked” or “partial gap” condition, described in detail in the IPET report, was used in all stability analyses for the “gap” analyses.

Figures 13, 14, and 15 show the critical failure surfaces for the “no gap” condition for canal water elevations of 5.5, 7.5, and 9.5 ft, respectively. Figures 16, 17, and 18 show the critical failure surfaces for the “gap” condition for canal water elevations of 3.5, 5.5, and 7.5 ft, respectively.. The factors of safety calculated for these analyses are also shown plotted in Figure 19. As shown by Figure 19, a canal water level of about 9.2 ft produces a factor of safety of unity for the “no gap” condition. For the “gap” condition, the canal water level that results in a factory of safety of unity is about 6.2 ft. This demonstrates the considerable effect that the presence of the gap has on shear stability, and this effect is much greater than the influence of the gap on erosion and heave.

The results of the seepage and stability analyses, as applied to the proposed test section, allow the following observations to be made:

- Critical hydraulic gradients will exist for canal water elevations greater than 1 ft.
- Conditions for heave exist for canal water elevations greater than 1 ft.
- If a gap *does not* form behind the I-wall, shear failure should not occur until the canal water elevation reaches about 9.2 ft.
- If a gap *does* form behind the I-wall, shear failure should not occur until the canal water elevation reaches about 6.2 ft.

The analyses contained several conservative assumptions, which would have a great impact on the results.

- It was assumed that there is a direct hydraulic connection between the canal bottom and the sand layer. If the canal bottom is “silted in,” the pore pressures at the toe will be much lower, resulting in lower hydraulic gradients, reduced uplift pressures, and higher factors of safety.
- It was assumed that the constant head boundary is located about 150 ft from the I-wall. If the constant head boundary is closer to the I-wall, the pore pressures at the toe will be lower, resulting in lower hydraulic gradients, reduced uplift pressures, and higher factors of safety.
- It was assumed that the drained friction angle of the sand was 30 degrees. This would represent a lower bound value, and it probable that the true friction angle of the sand would be greater than 30 degrees.

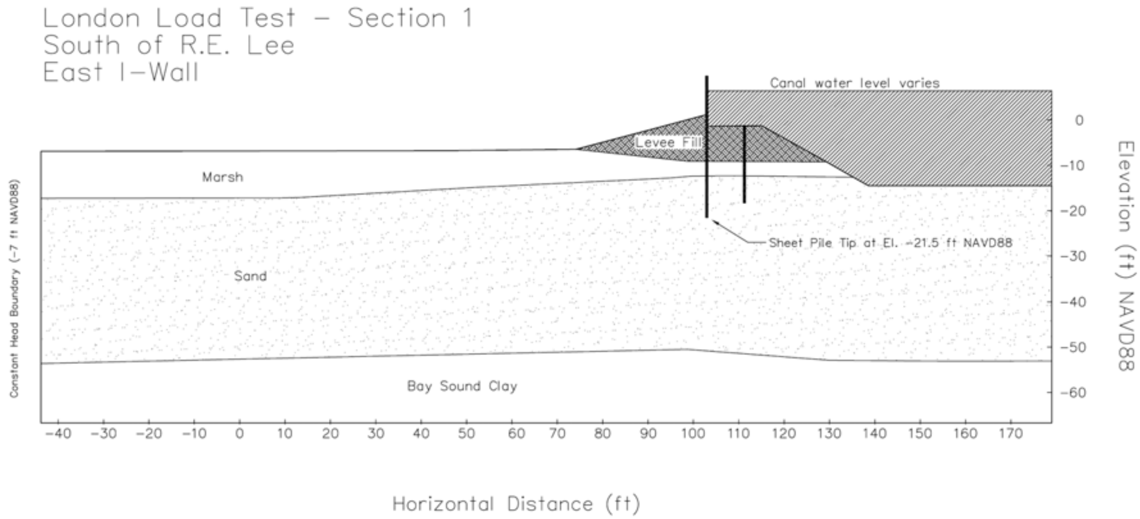

Figure 1 Schematic of London Avenue Canal load test section 1. This figure shows the domain and the hydraulic boundary conditions used in the seepage analysis.

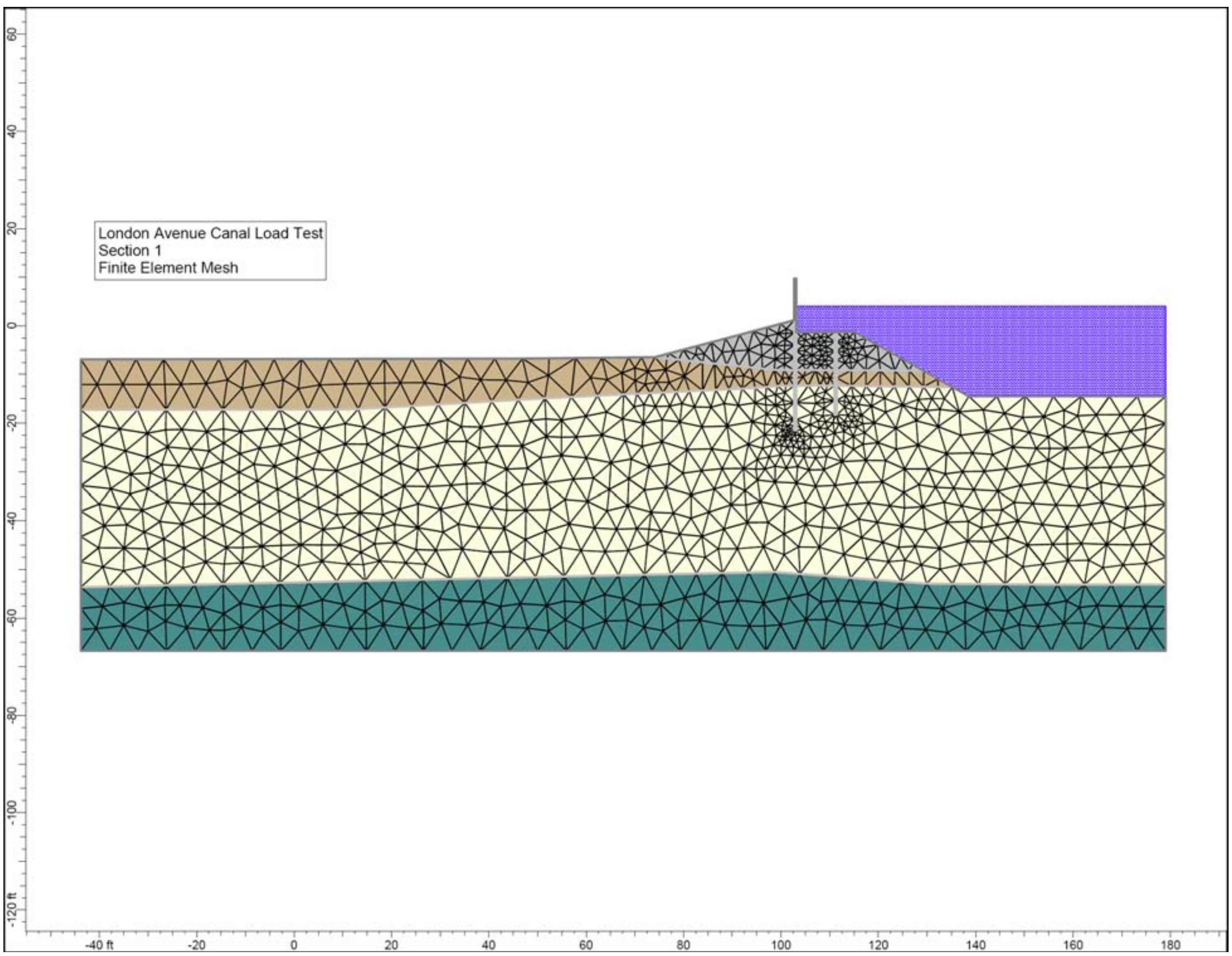

Figure 2 Finite element mesh used in the seepage analysis.

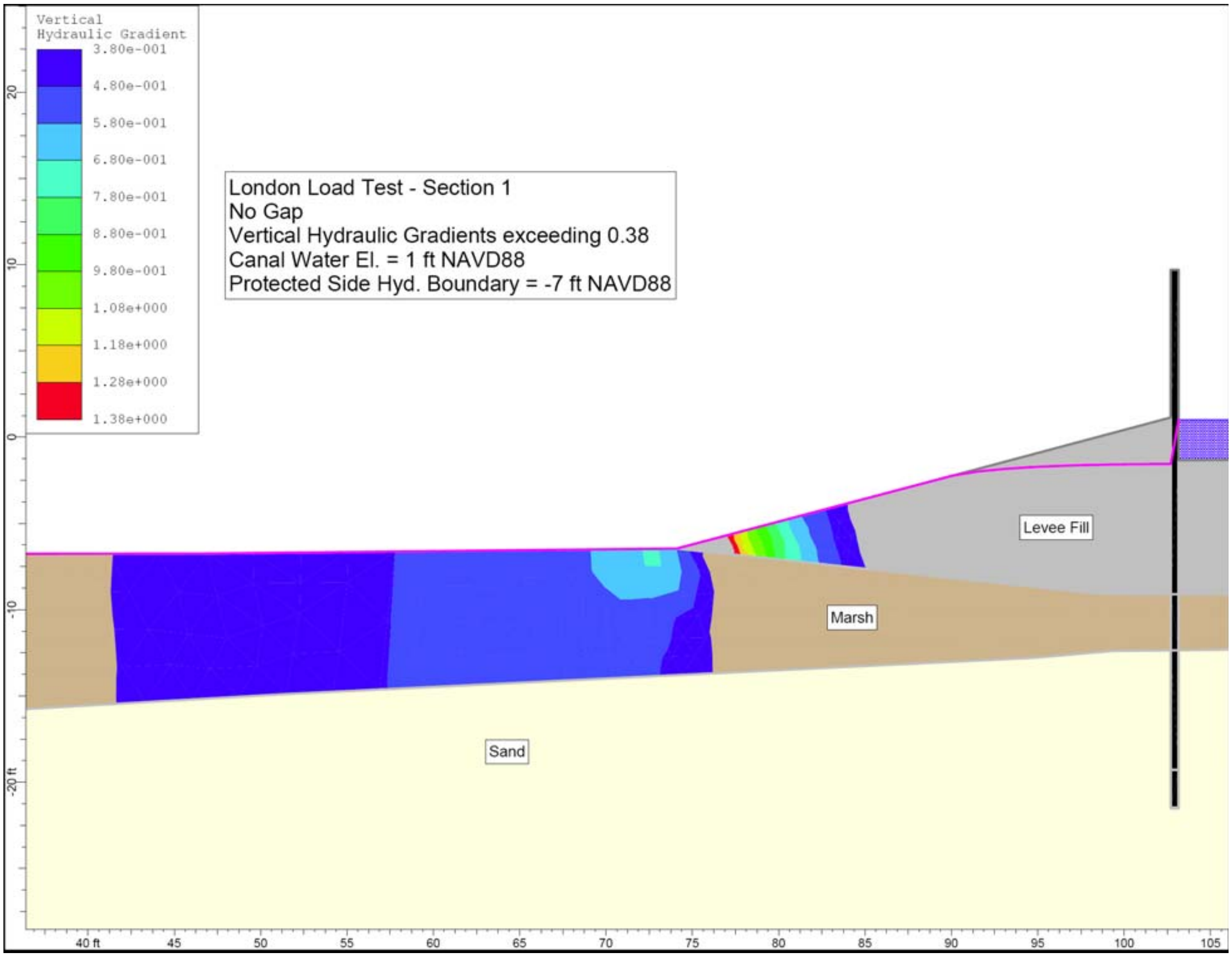

Figure 3 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 1.0 ft NAVD88, and no gap is present.

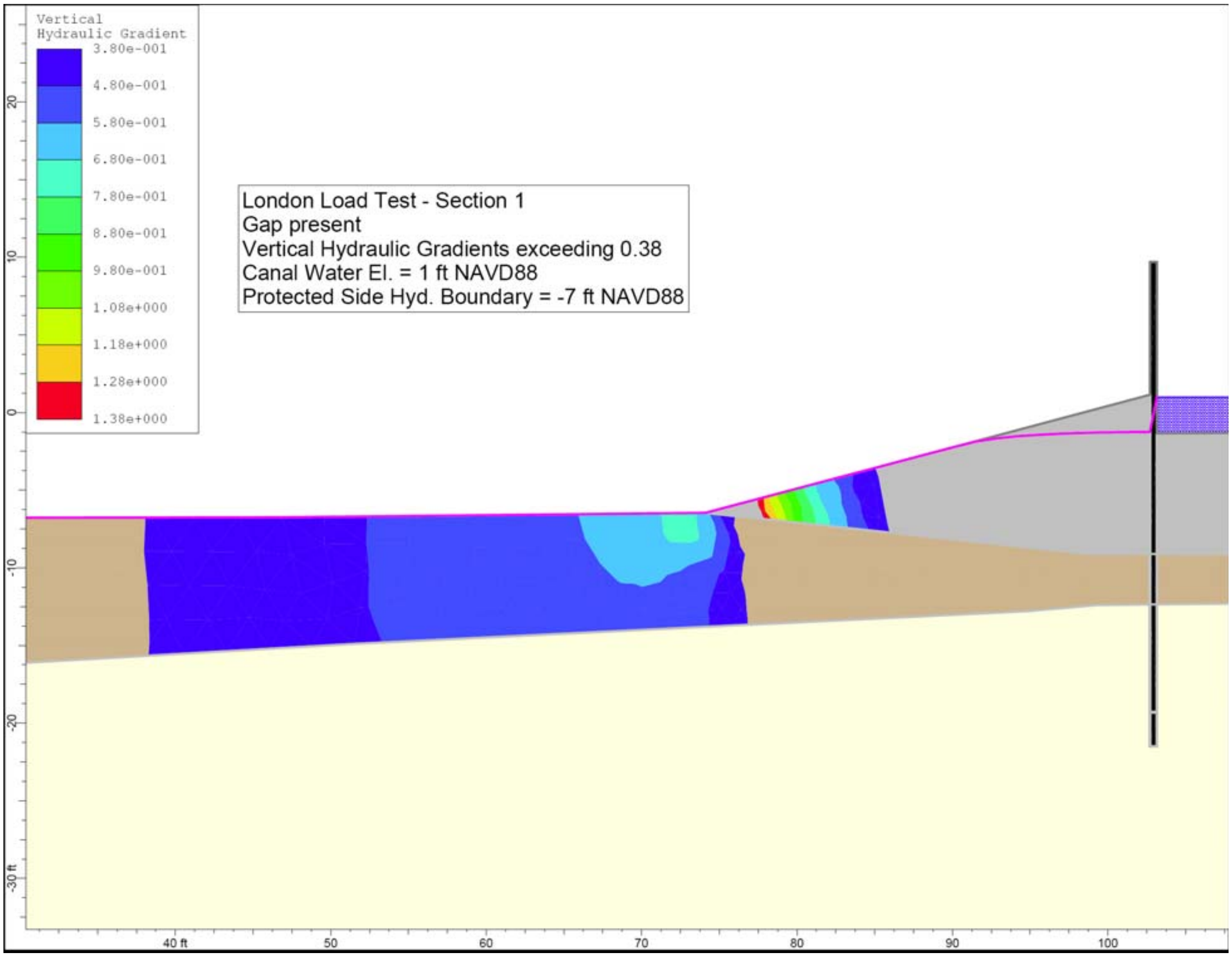

Figure 4 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 1.0 ft NAVD88, and a gap is present.

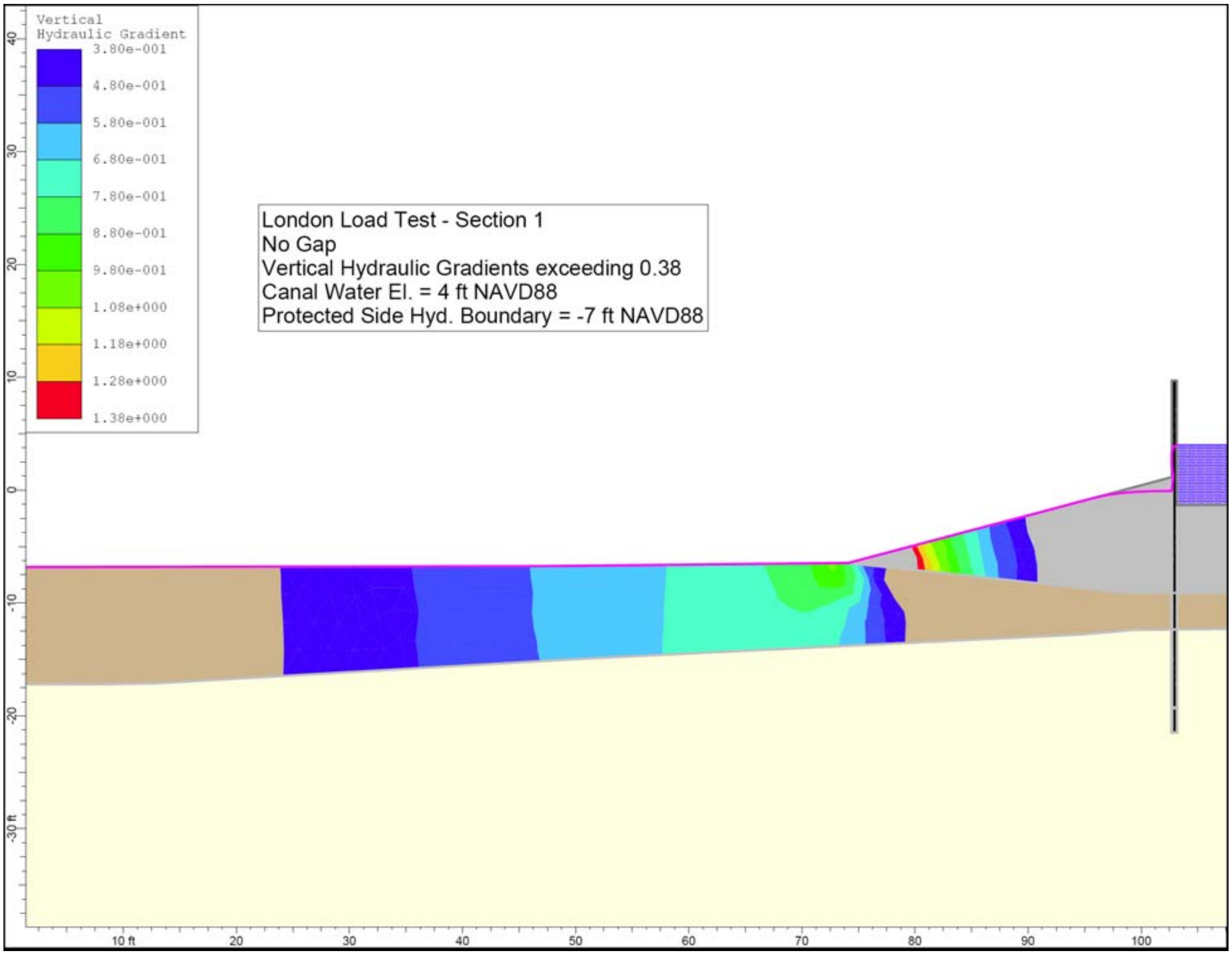

Figure 5 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 4.0 ft NAVD88, and a gap is not present.

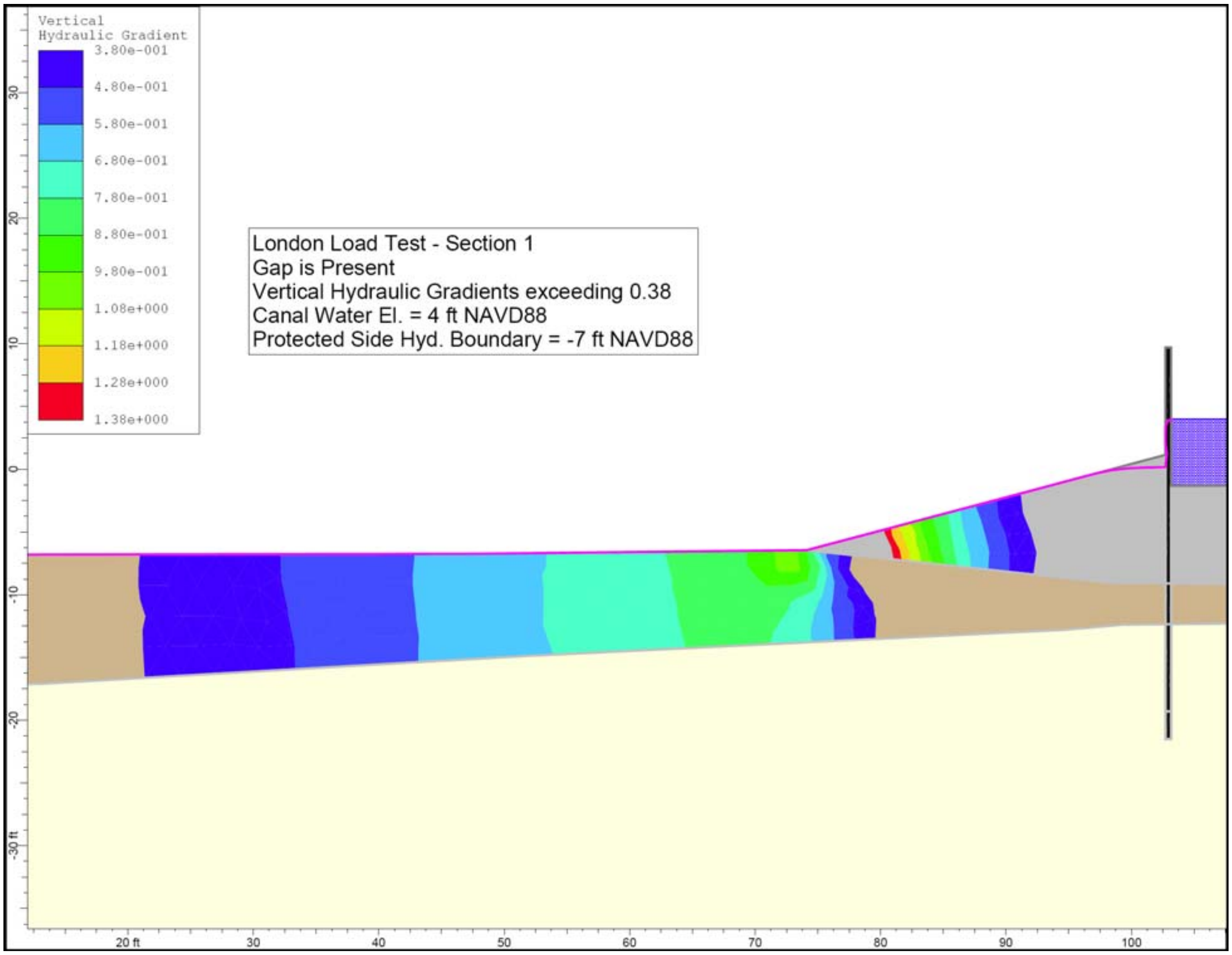

Figure 6 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 4.0 ft NAVD88, and a gap is present.

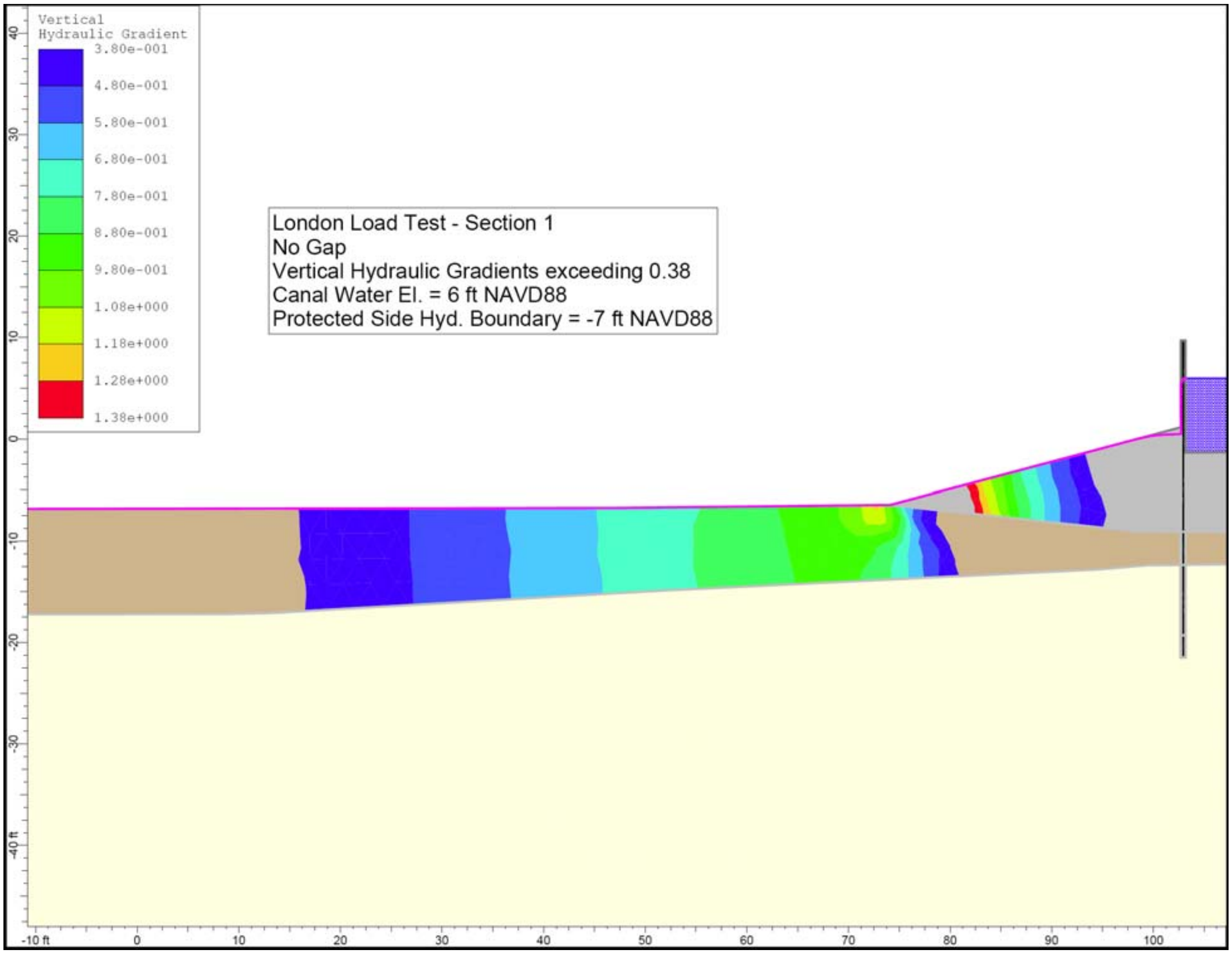

Figure 7 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 6.0 ft NAVD88, and a gap is not present.

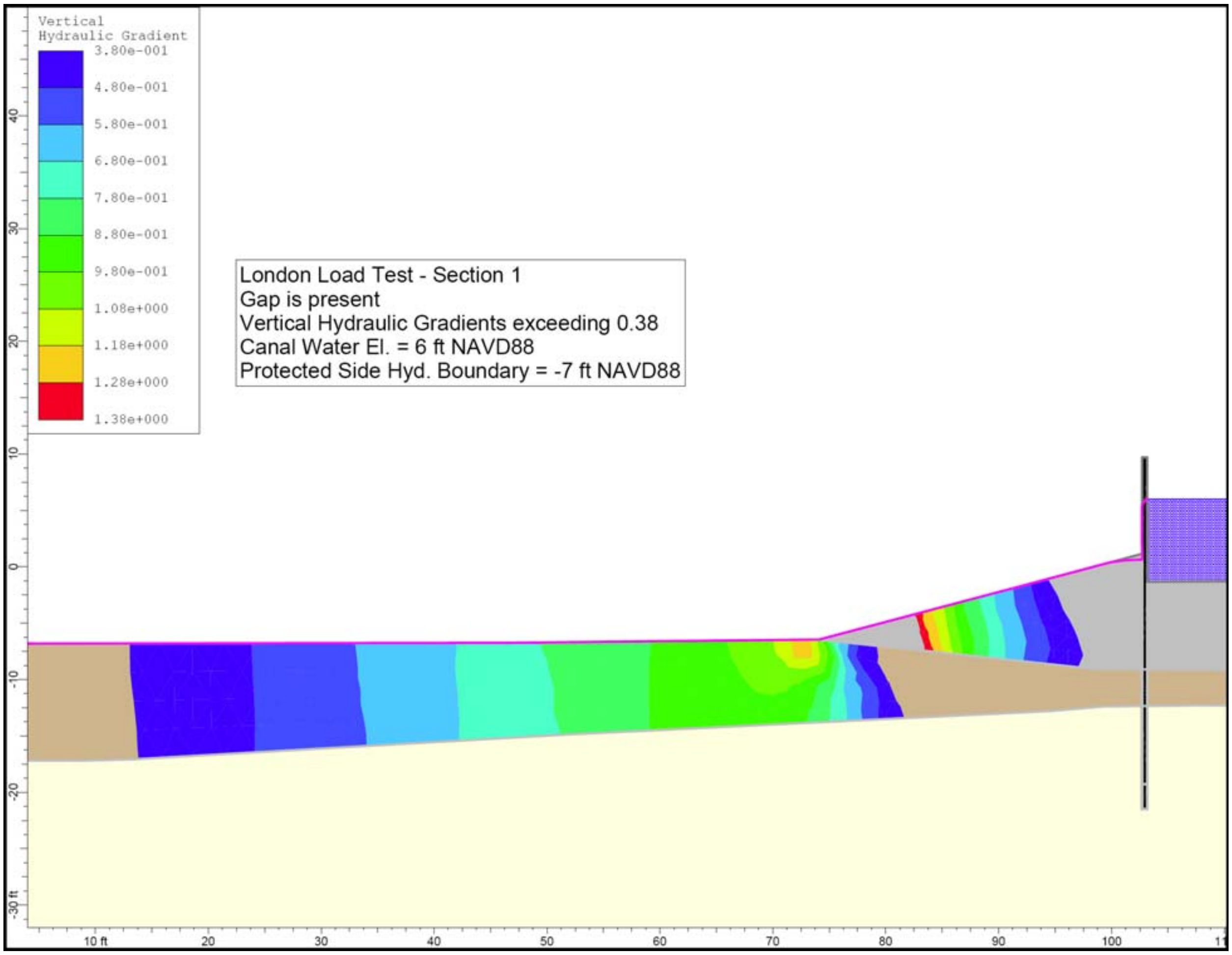

Figure 8 Contours of vertical hydraulic gradient showing gradients exceeding 0.38. The canal water level is 6.0 ft NAVD88, and a gap is present.

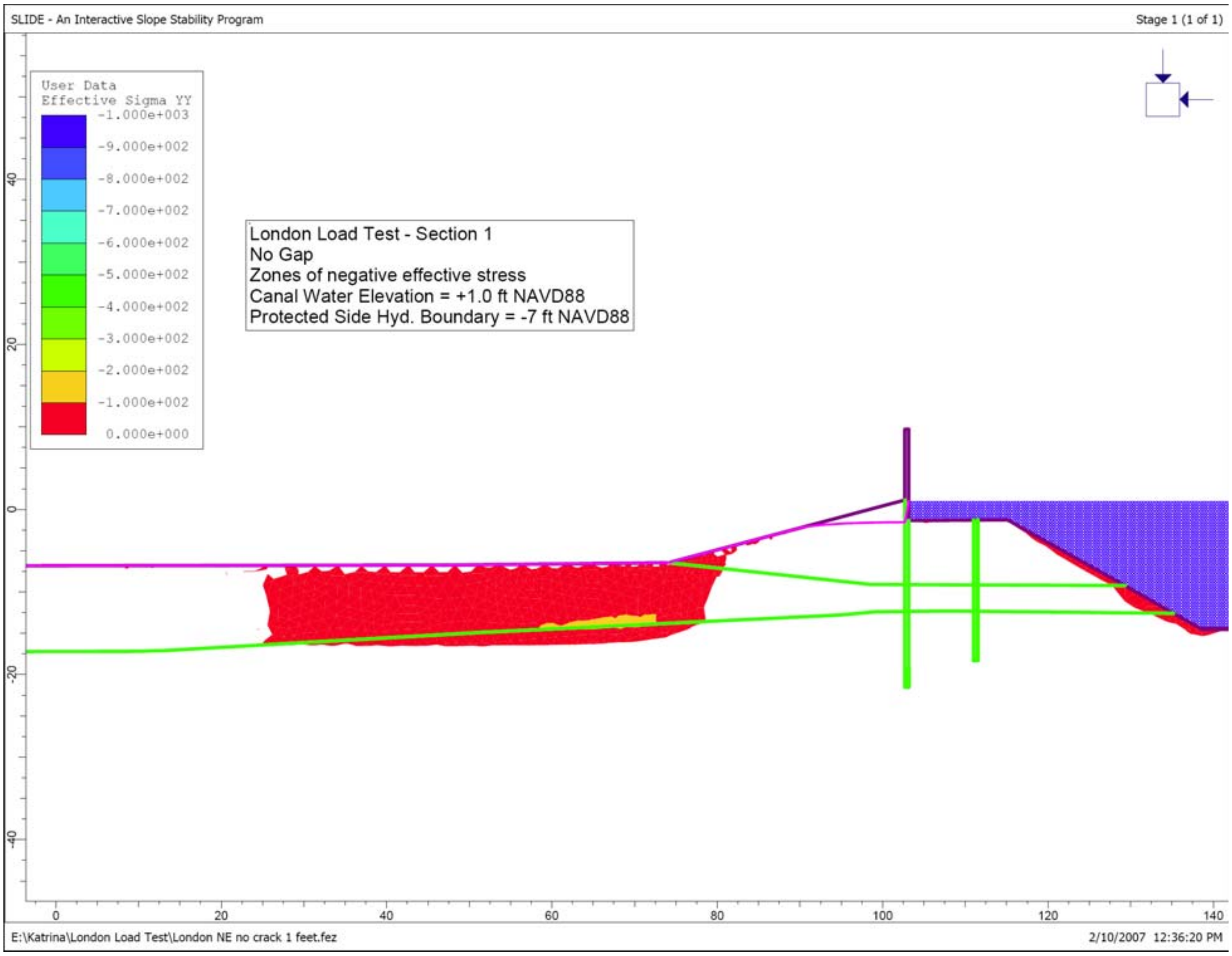

Figure 9 Areas of negative effect stress, indicating potential heave. The canal water level is 1.0 ft NAVD88, and a gap is not present.

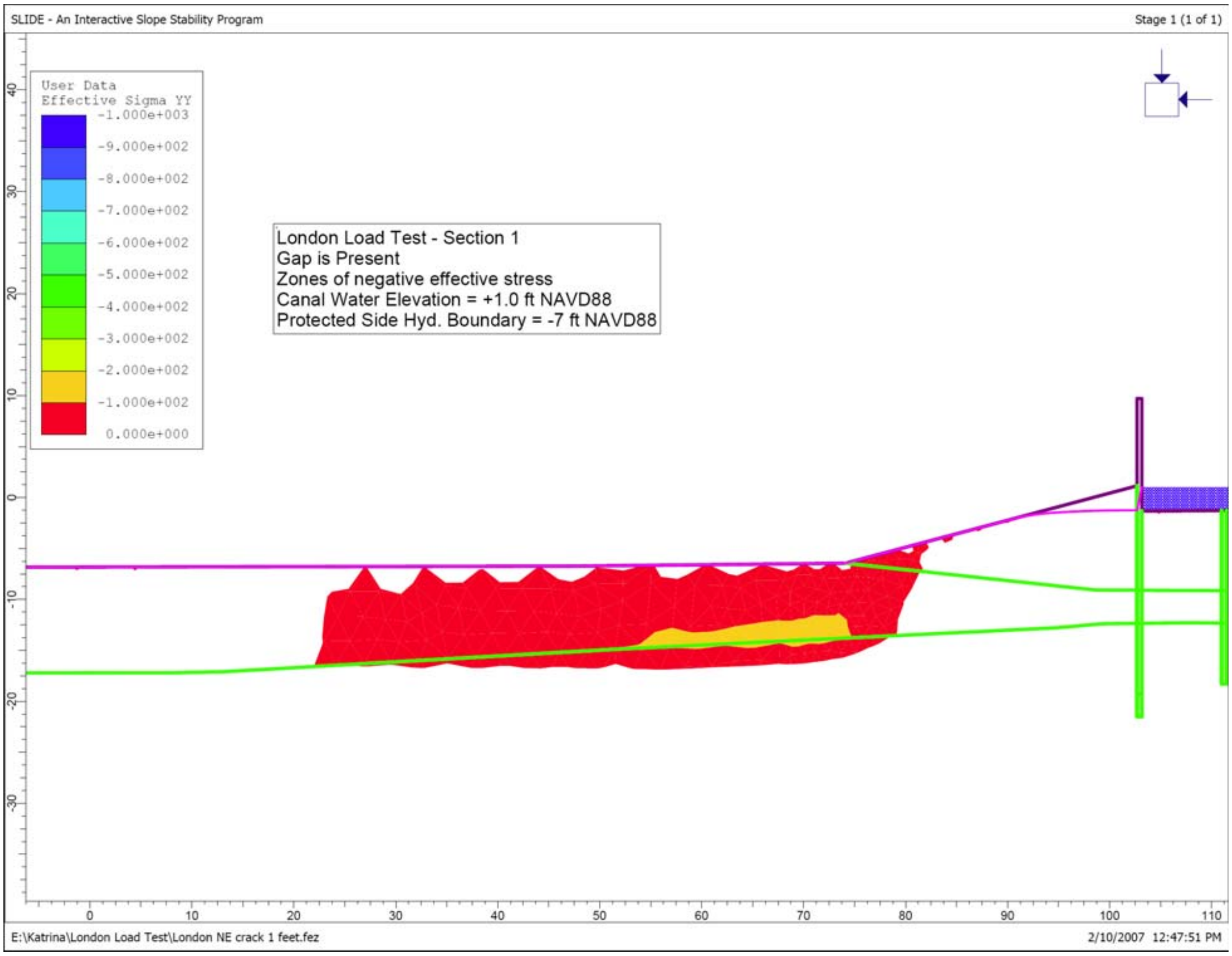

Figure 10 Areas of negative effect stress, indicating potential heave. The canal water level is 1.0 ft NAVD88, and a gap is present.

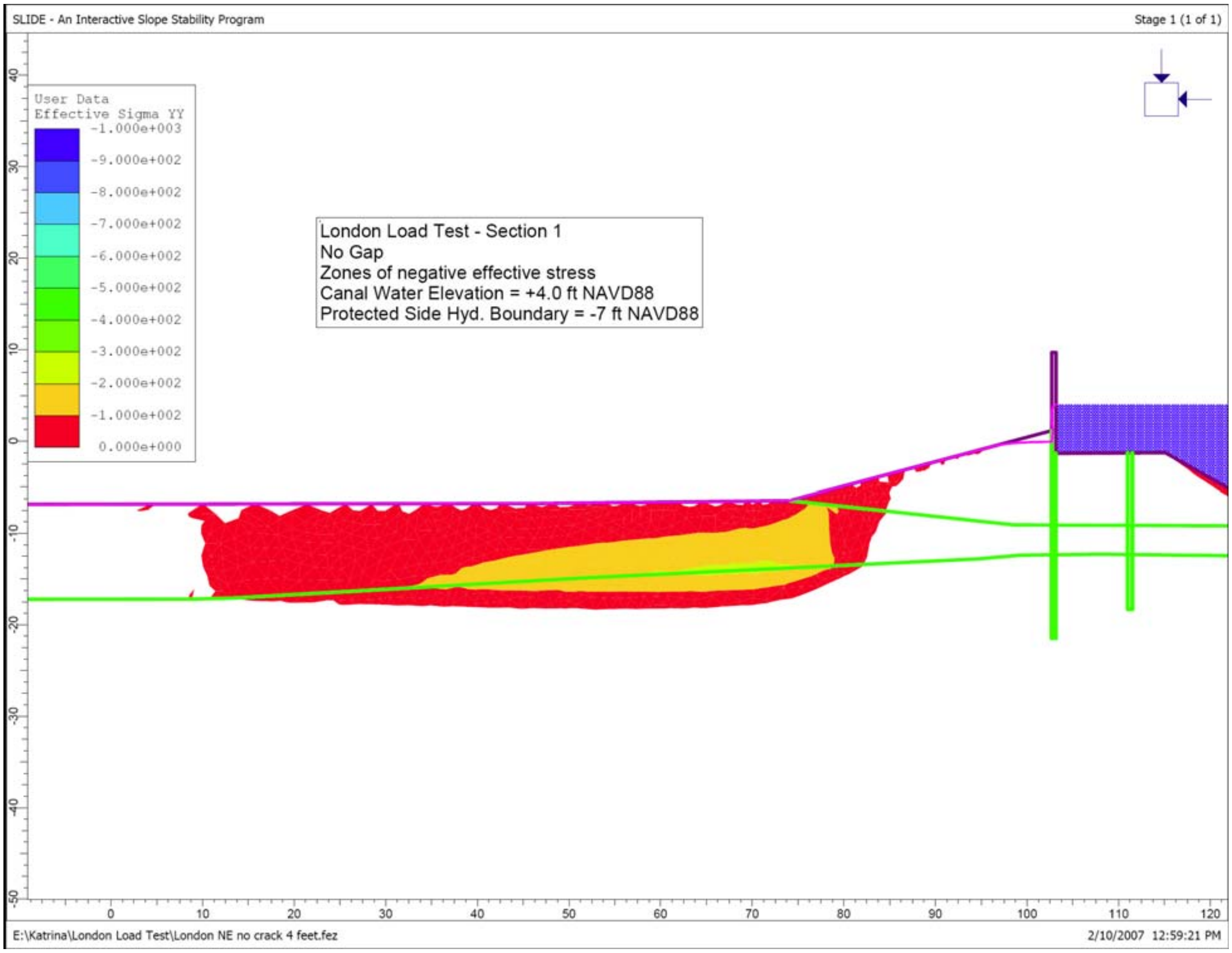

Figure 11 Areas of negative effect stress, indicating potential heave. The canal water level is 4.0 ft NAVD88, and a gap is not present.

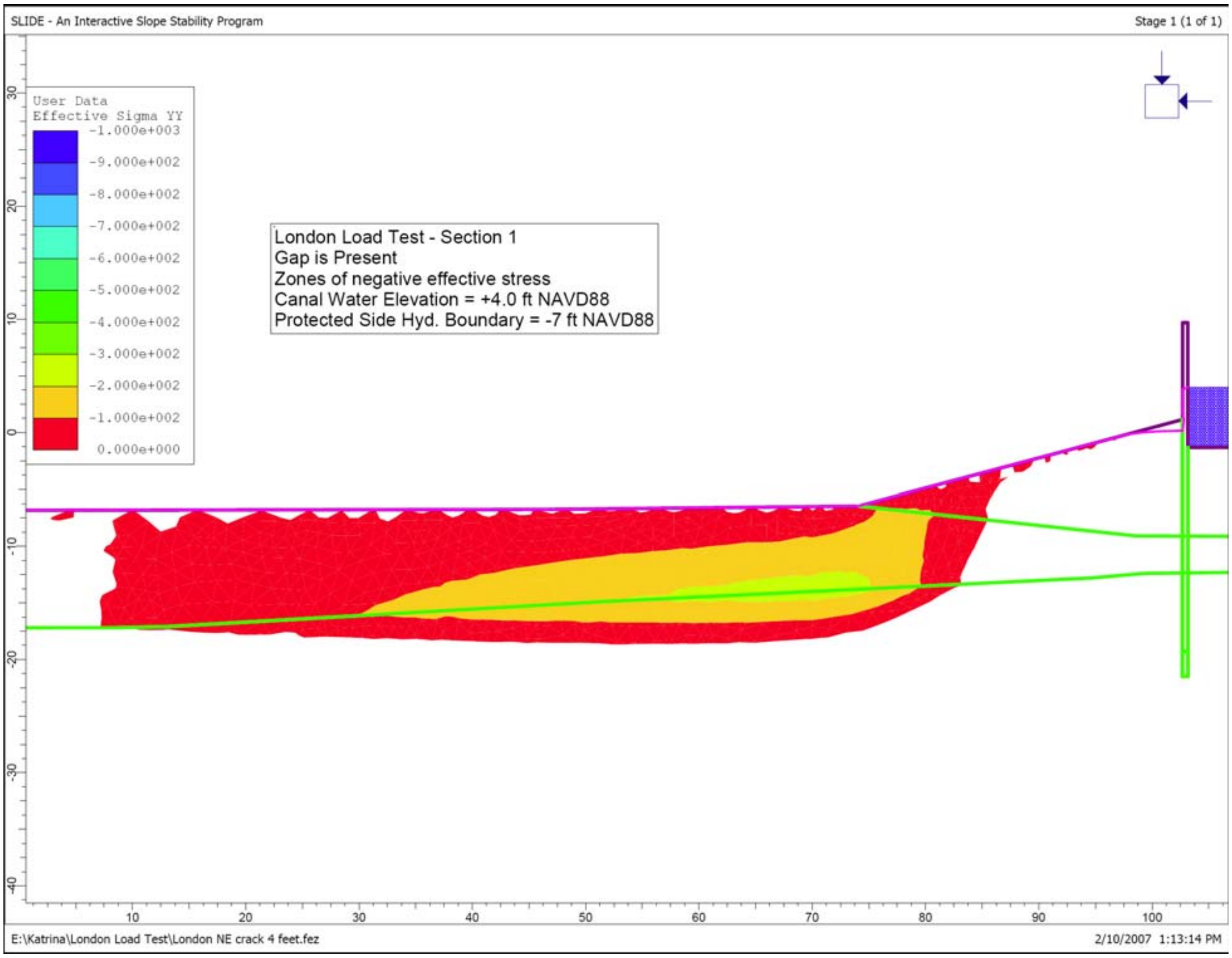

Figure 12 Areas of negative effect stress, indicating potential heave. The canal water level is 4.0 ft NAVD88, and a gap is present.

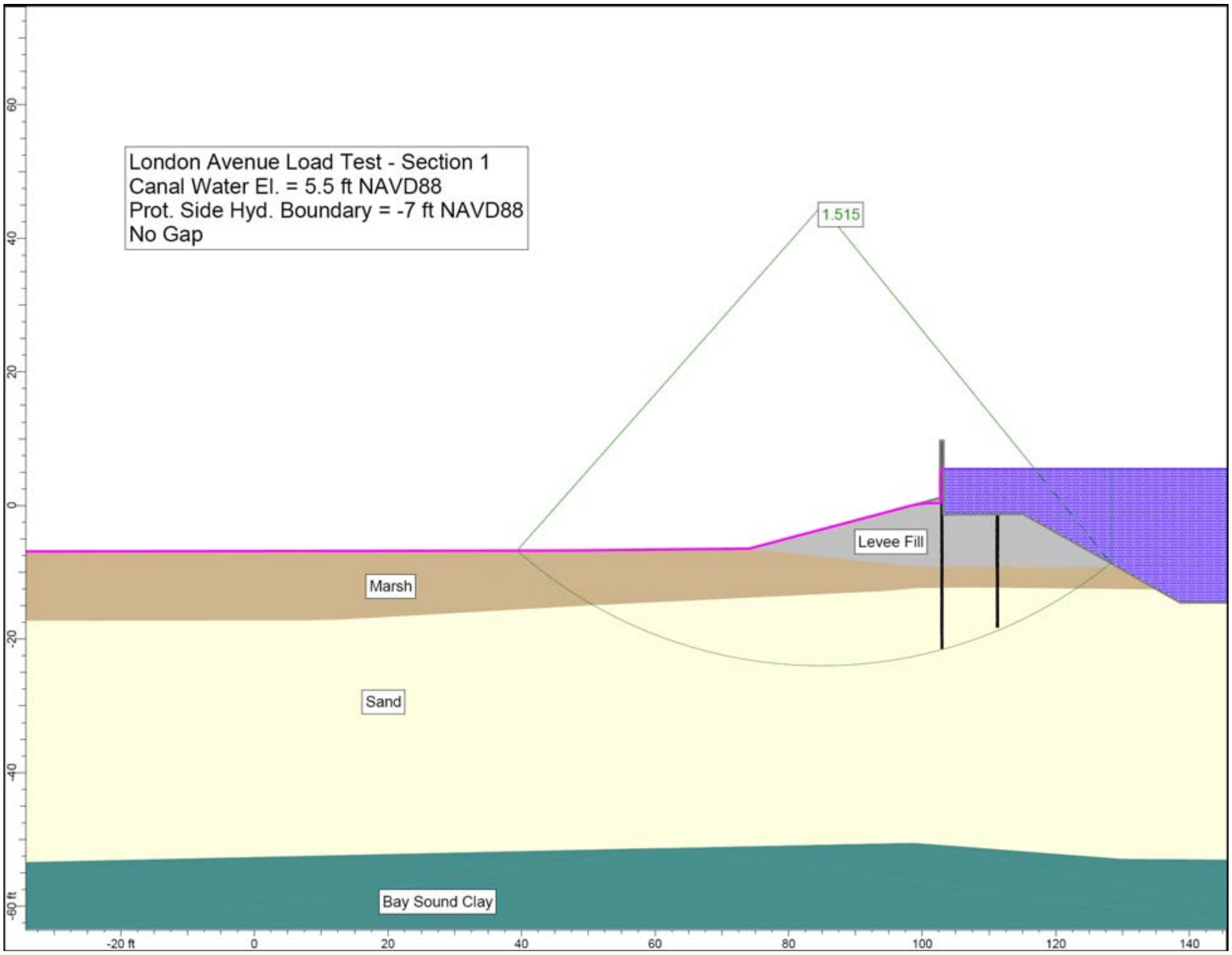

Figure 13 Critical failure surface for canal water level of 5.5 ft NAVD88, for the condition of "no gap."

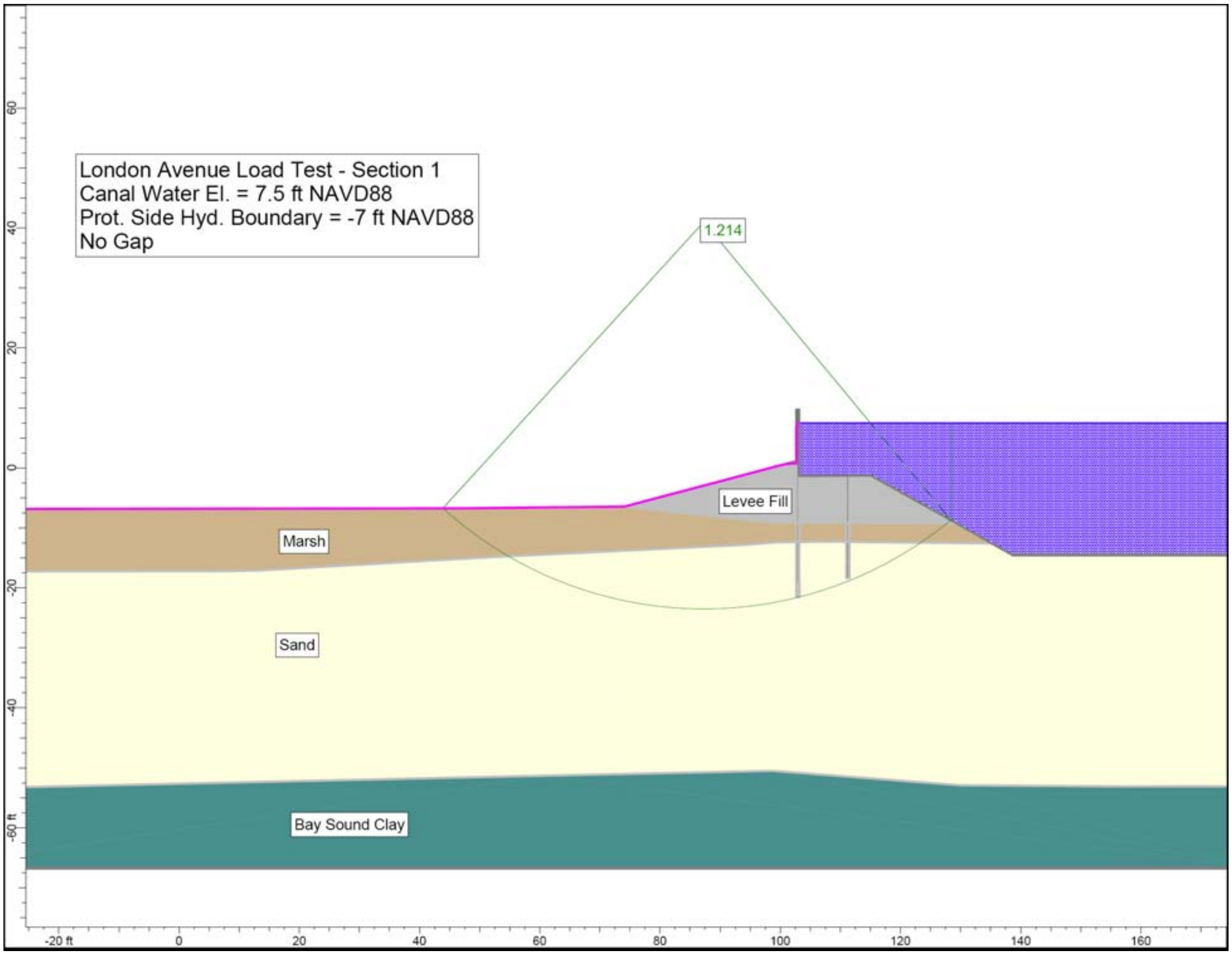

Figure 14 Critical failure surface for canal water level of 7.5 ft NAVD88, for the condition of "no gap."

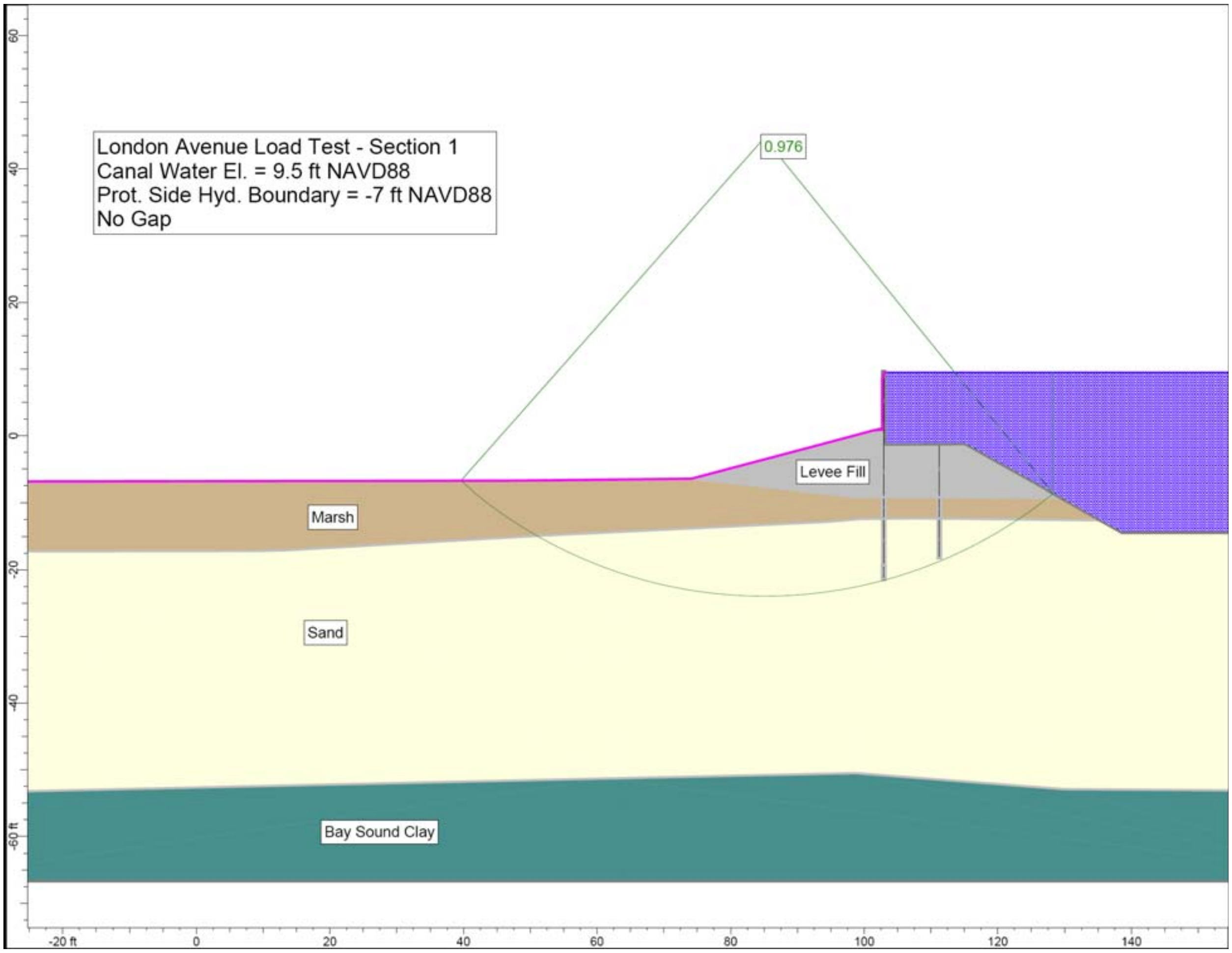

Figure 15 Critical failure surface for canal water level of 9.5 ft NAVD88, for the condition of "no gap."

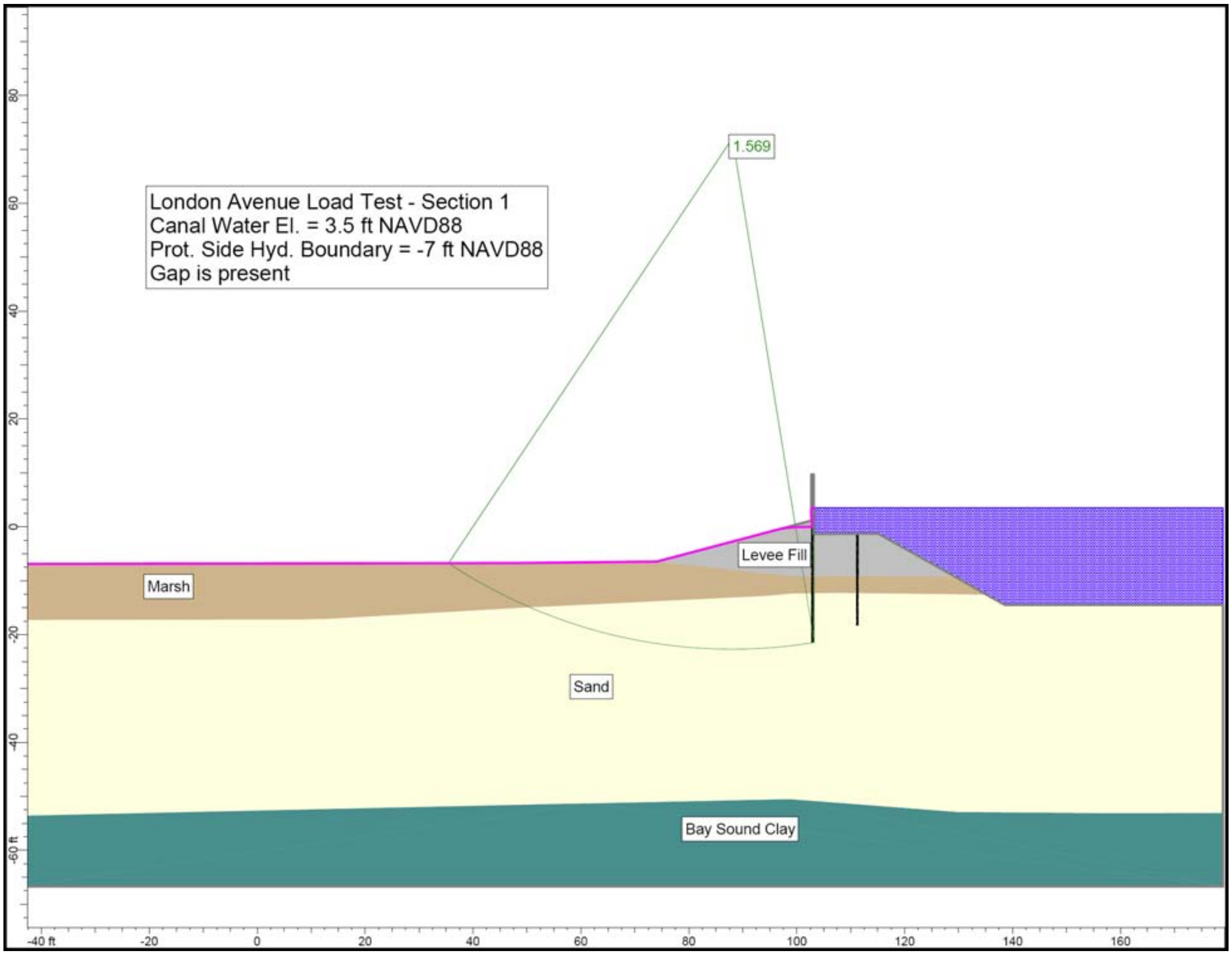

Figure 16 Critical failure surface for canal water level of 3.5 ft NAVD88, for the gap condition.

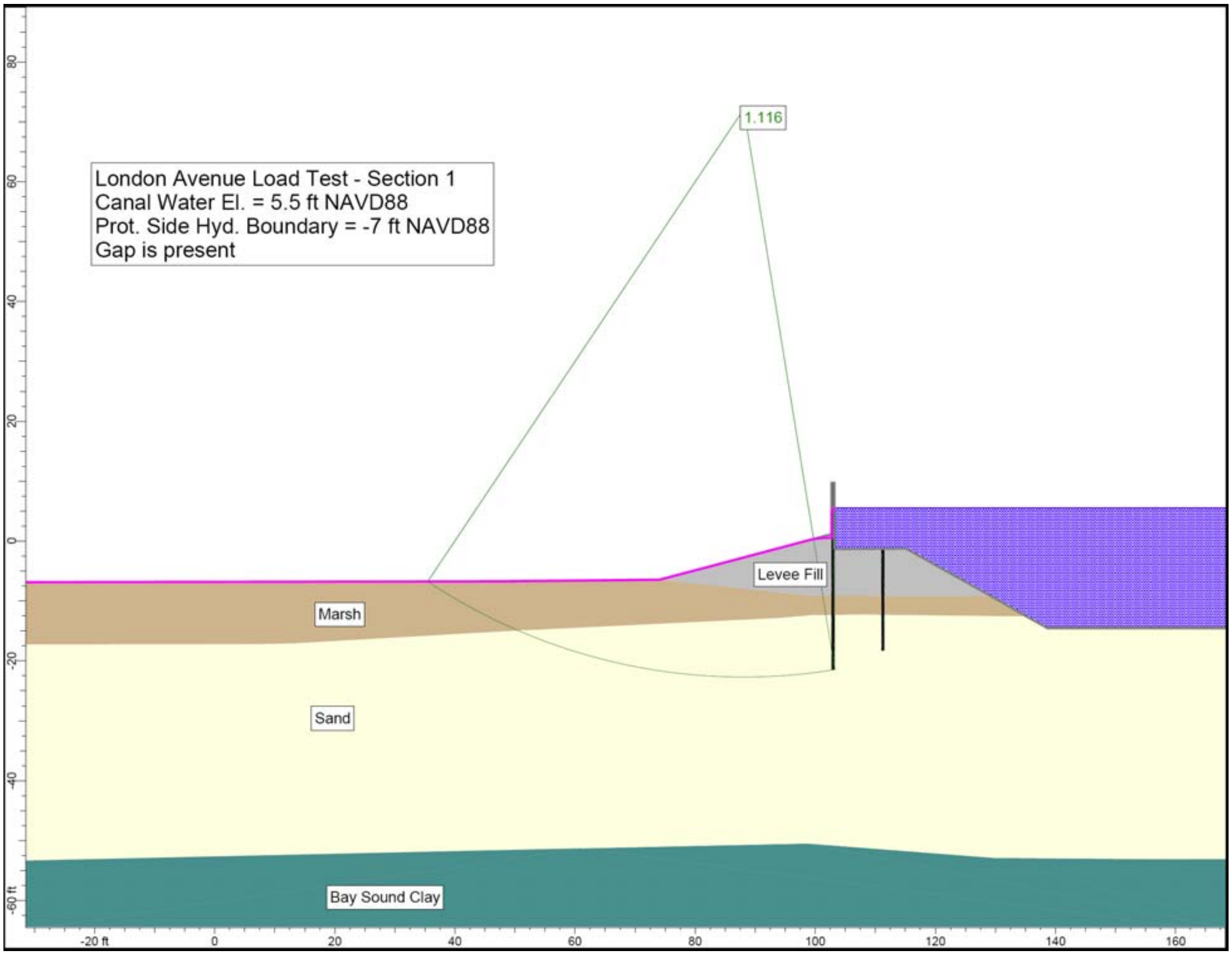

Figure 17 Critical failure surface for canal water level of 5.5 ft NAVD88, for the gap condition.

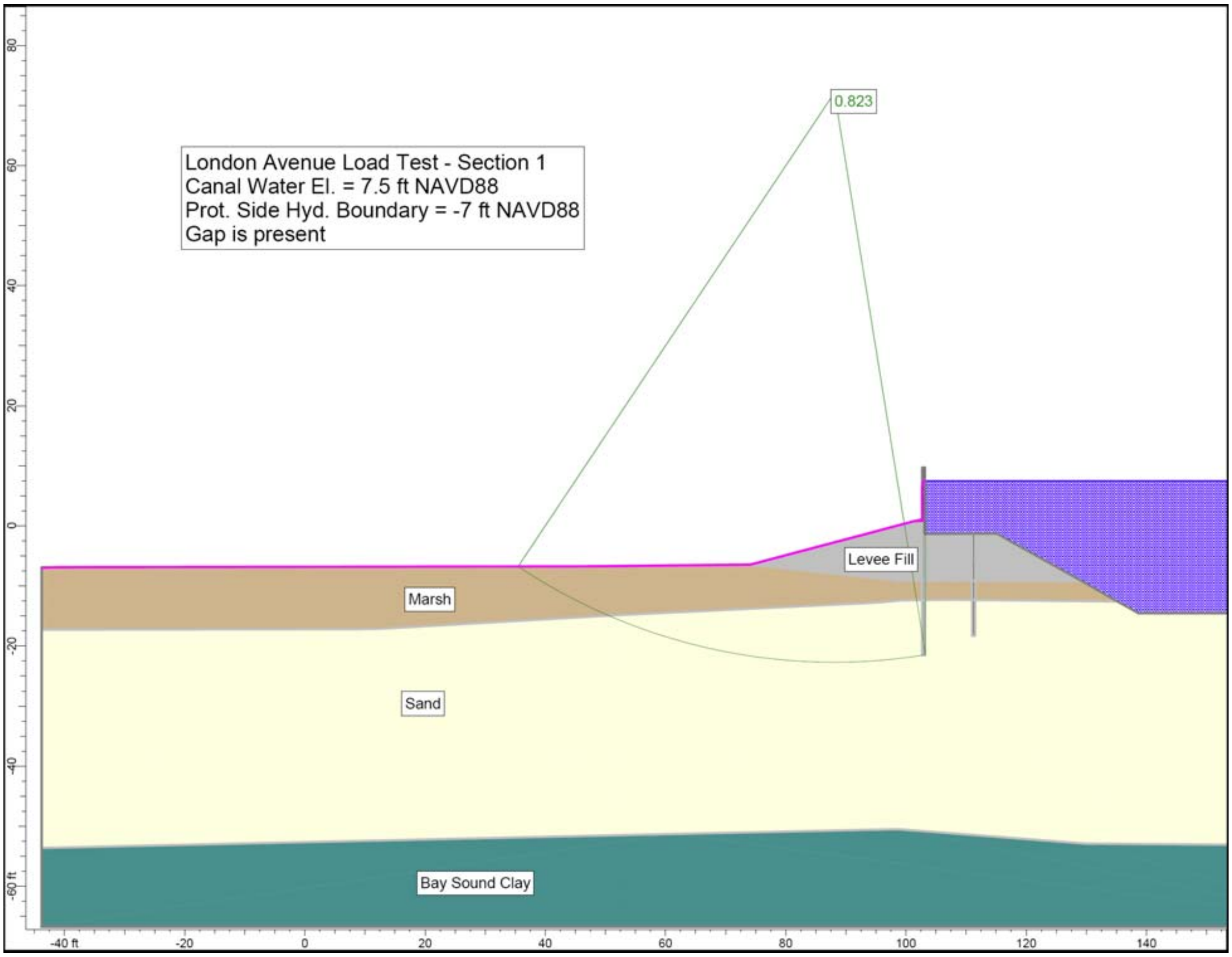

Figure 18 Critical failure surface for canal water level of 7.5 ft NAVD88, for the gap condition.

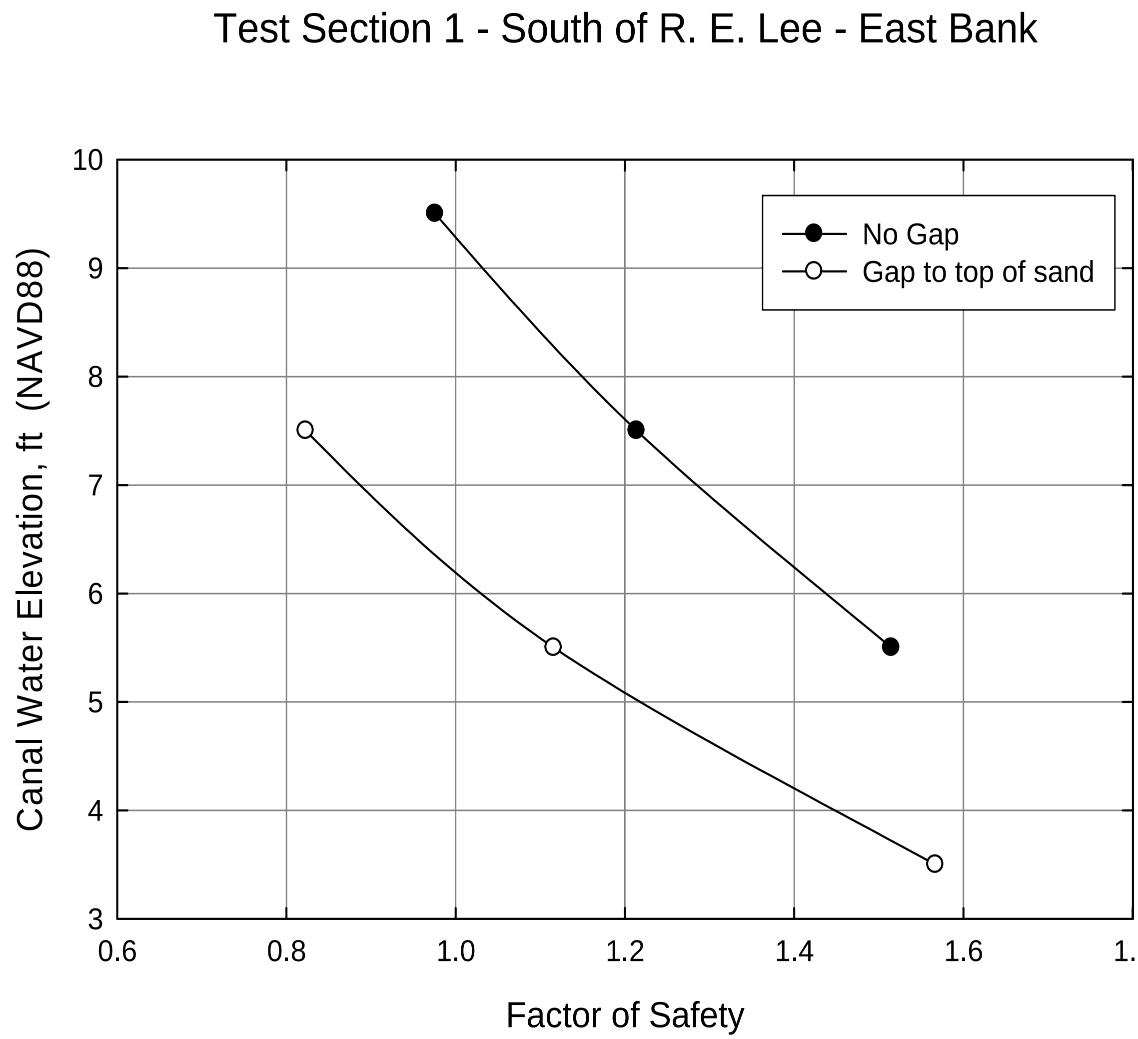

Figure 19 Factors of safety for the “gap” and “no gap” conditions as a function of canal water level.