FINITE ELEMENT ANALYSIS OF DISPLACEMENT PATTERNS IN A SAND FILL ON SOFT GROUND

By:

W. H. Roth[1]
T. D. Swantko[1]
L. D. Handfelt[1]

[1]Dames & Moore, Los Angeles

ABSTRACT

The displacement patterns within a sand fill resulting from large differential settlements within the underlying natural soils were investigated using the finite element method. The sand fill is located on the edge of an organic soil area. The largest portion of the fill directly overlies natural sands, but the fill partially extends onto highly compressible organic soils. The numerical study was undertaken to assist in defining the safe distance from the edge of the soft ground area where a structure could be supported within the sand fill. Consequently, the objective of the study was to define general displacement patterns within the fill rather than to provide precise predictions of deformation magnitudes.

The sand fill was modeled using an incremental nonlinear elastic FE program with hyperbolic stress-strain relationships. The boundary conditions were chosen such that the lower boundary (the contact between sand fill and the organic soils) was incrementally dropped simulating settlements within the soft soils estimated from one-dimensional consolidation theory.

The sand fill was instrumented with settlement plates and slope indicators to check the analytical results. Although the actual construction sequence and configuration of the sand fill was different than originally planned and what was assumed for analysis, many aspects of the study were still applicable for comparison of field behavior with the analytical results. In particular, the relationships of vertical settlements to horizontal spreading and the limits of stable zones within the sand fill agreed reasonably well with the predicted behavior.

INTRODUCTION

In spite of their frequent applications in the area of soil mechanics and foundation engineering, numerical tools, such as the finite element method, are still associated by many practicing soils engineers with prohibitive time and/or cost effort. In the past, the increasing complexity of computer codes involved with numerical methods also tended to discourage many to venture beyond conventional analyses involving closed form analytical solutions or simplified limiting equilibrium methods. However, in the last decade, economic finite element computer codes have been developed and tested so thoroughly, that utilizing them even on a limited budget has become feasible. Also, it is neither necessary nor always possible to intimately understand the numerical procedure of a computer code in order to successfully use it. Provided the user has a good practical understanding of the technical problem and a fundamental knowledge of engineering mechanics, he is able to detect gross errors by spot checking the results with simple hand calculations.

Most numerical tools are applicable only for a certain range of conditions, and engineering judgement is necessary to recognize their limitations. While today there are practically no limits on the capability of computational techniques, serious handicaps still exist with expressing material behavior by the proper constitutive laws. In some cases, sophisticated numerical tools would be no more reliable than simplified conventional analysis techniques. The case described in this paper is felt to be an example of successfully and economically combining both approaches. A conventional one-dimensional consolidation analysis was used to establish displacement boundary conditions for a numerical deformation analysis of a sand fill partially overlying soft organic soils. The main purpose for this study was to define the edge of the deformed zone in the sand fill resulting from compression of the organic soils. The results were used to identify stable ground for siting a structure.

SITE CONDITIONS

Figure 1 shows a plan view of the project site. The contours shown indicate the depths to which organics were present. The deep organic area to the southwest was to be used for landscaping and the proposed building was to be located in the area where all the organics would be replaced with compacted fill. For architectural reasons, it was desirable to locate the building as close to the edge of the organic area as practical.

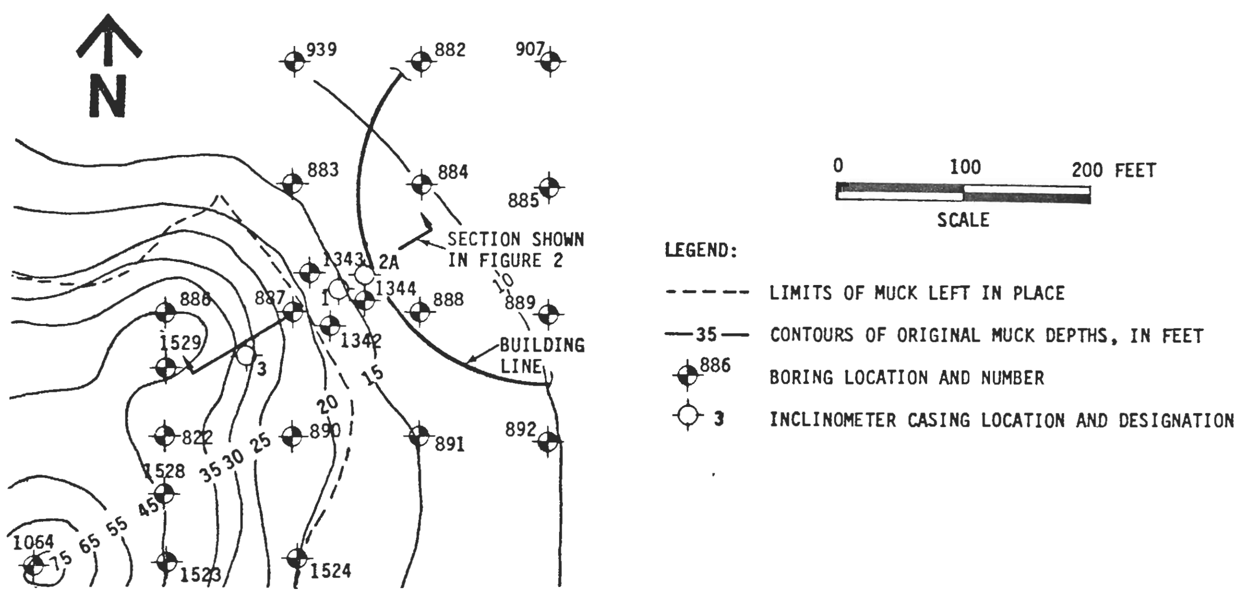

Figure 1. Plan View of Project Site

It was decided to surcharge the deep organic area to complete most of the primary consolidation settlement prior to construction of the building. Even so, it was estimated that up to 2 feet of long-term settlement would still occur in the area due to secondary compression of the organic soils. In addition, due to a tight construction schedule, there was the possibility of construction proceeding well before primary consolidation was completed. Consequently, it was necessary to define a distance from the edge of the organic area where a structure could be safely located, considering that a total vertical settlement of almost 7 feet could occur in the deep organic soils area to the southwest.

During grading operations the organics were excavated to the limits indicated by the dashed line shown on Figure 1. Figure 2 shows the subsurface profile at the site following grading. Prior to developing the area, the natural ground

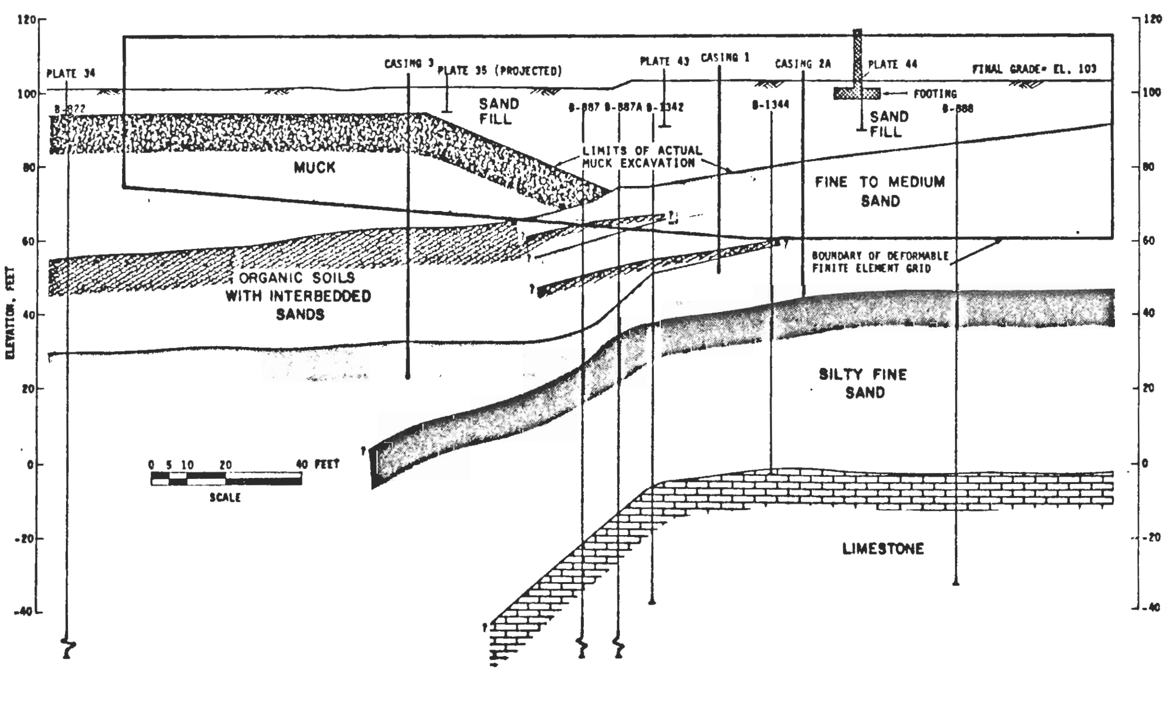

Figure 2. Subsurface Section Showing Locations of Borings, Inclinometer Casings, Settlement Plates, and Building Foundation

surface was at about Elevation +94 feet. Before building construction started, the general area was to be surcharged to Elevation +115 feet. The final grade after surcharging would be about +103 feet. In the deep organic soil area to the southwest, the fill was placed directly on the natural surface. The plan was that the surcharge would be removed from the building area after about six weeks, since the surcharge here was generally precautionary. Within the deep organic area, the surcharge could be inplace for one year or more.

Beneath the general building site, the compacted fill replaced the excavated very soft soils and peat which had extended down to about Elevation +80 feet. Below the fill, and extending to about Elevation +45 feet are generally medium dense to dense, fine to medium sands. Below the sands to the top of the limestone formation (about Elevation -1 foot) are loose to medium dense silty fine sands.

A much poorer and variable subsurface profile exists to the southwest. Here, the entire profile is depressed due to past sinkhole activity. Organic soils and peat underlie the fill in this area to about Elevation +30 feet. Above about Elevation +60 feet, are soft, highly compressible, organic soils and peat, locally referred to as muck. Below about Elevation +60 feet, the organics are stiffer and interbedded with sands.

FINITE ELEMENT ANALYSIS OF SAND FILL

The purpose of the finite element analysis was to evaluate the displacement pattern within the sand fill to identify stable ground near the edge of the settling soft ground area. Taking into account the lack of understanding of the constitutive behavior of the muck, it was not justified to attempt to model the muck by finite element techniques. Therefore, it was decided to restrict modeling to the sand fill, for which proven constitutive relationships were readily available. The compression of the underlying muck and organic soils was analyzed utilizing conventional one-dimensional consolidation theory. The calculated settlements due to the load of the sand fill were used as boundary conditions for the modeled fill.

The nonlinear elastic finite element computer program, ISBILD, developed by Ozawa and Duncan (1973), was used to perform the analysis. The program uses the formulation for the variable tangent modulus developed by Duncan and Chang (1970) based on hyperbolic stress-strain relationship and stress-dependent initial elastic moduli. The program also employs analogous expressions for the Poisson's ratio developed by Kulhawy, et al. (1969).

The two-dimensional, plane strain program approximates the nonlinear behavior of soil by successive increment procedures. The loading is subdivided into a number of small increments, and soil behavior is assumed to be linear within each increment. The elastic modulus and Poisson's ratio values for each element are iteratively reevaluated for each loading increment based on the new confining stress and the cumulative shear strain. The input soil parameters are usually determined by a series of routine triaxial compression tests. Many sands have been tested by researchers in the past and ample data is available in the literature.

Nonlinear elastic models can be used successfully for moderately strained soils. However, if the stress levels in the soil approach failure conditions, the soil should be modeled by other means to allow for plastic behavior. Important indicators of unrealistic results due to limitations of the constitutive model include the buildup of tension in elements, and the development of shear stresses higher than the soil's strength. For the profile analyzed, it was found that reasonable results were obtained for the range of settlement profiles expected up to a maximum settlement of almost 7 feet.

MATERIAL PROPERTIES

In lieu of actual laboratory test data for the sand fill, data for sand with similar grain size characteristics as compiled by Kulhawy et al (1969) was used. The selected material was Sacramento River Sand tested for various relative densities. Figure 3 shows a comparison of the grain size distribution curve for Sacramento River sand with the range of grain sizes for the actual sand fill. The minimum relative density of the fill was estimated to be 60 percent. For these conditions, the following dimensionless parameters were selected from the paper referenced above. Defining the elastic modulus:

$K = 545$, $n = 0.54$, and $R_f = 0.86$;

and defining Poisson's ratio:

$G = 0.55$, $F = 0.26$, and $d = 2.1$

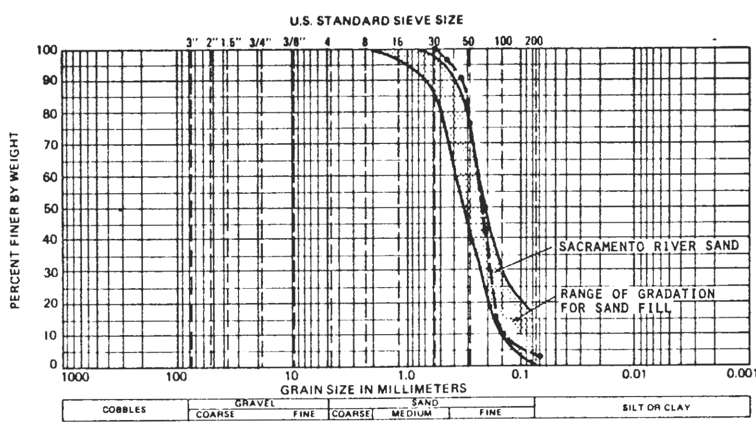

Figure 3. Gradation of Fill Sands and Sacramento River Sand

Reference is made to the above cited literature for the definitions of these parameters. The internal friction angle of the sand was estimated to be 32 degrees; cohesion was zero.

The ground water table within the fill was assumed to be at Elevation +96 feet. Wet and buoyant unit weights of 120 and 60 pounds per cubic foot, respectively, were used. The initial stress conditions were calculated by estimating K_o, the ratio of horizontal stress over the overburden stress. This ratio was estimated to vary across the fill as shown in Figure 4(a). In assigning the K_o values, substantially

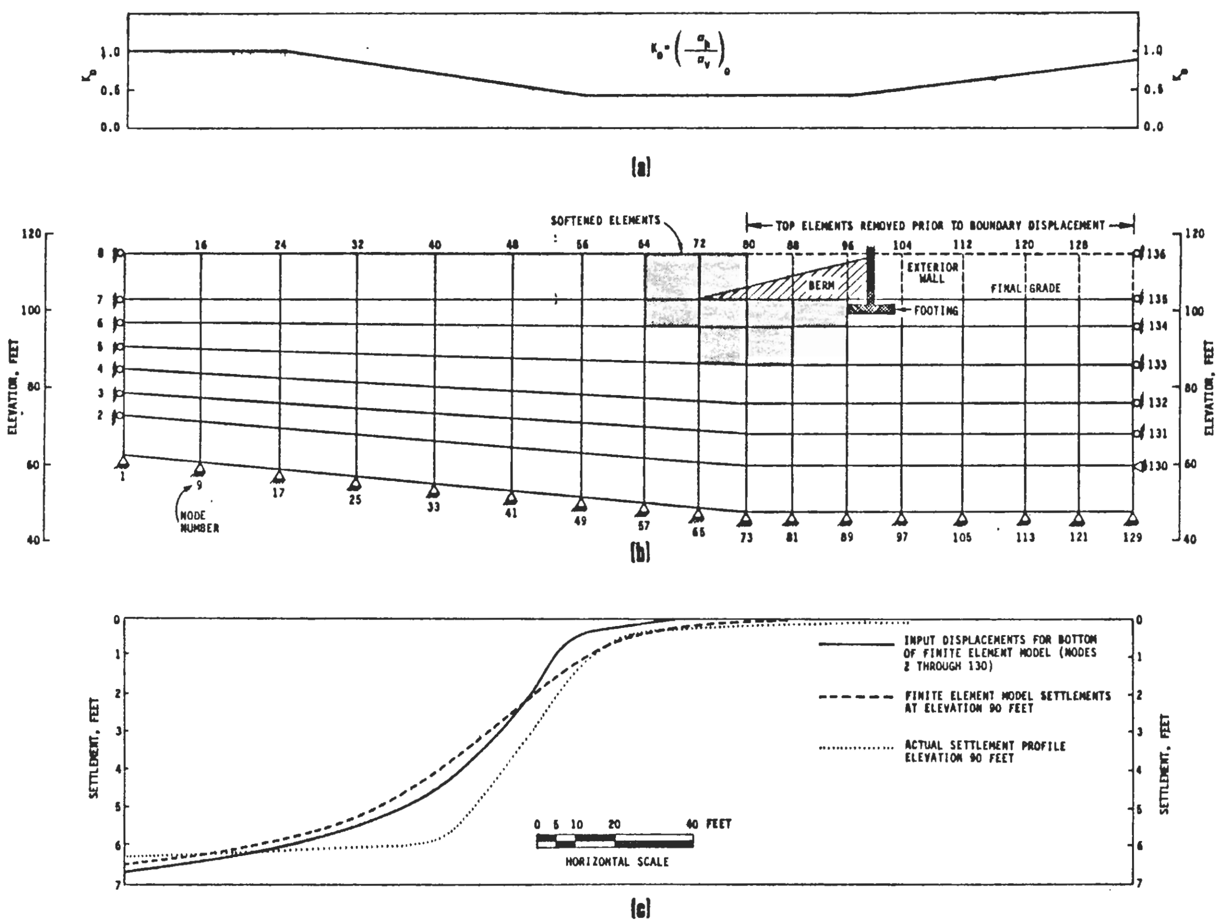

Figure 4. Sand Fill Model and Input (a) Assumed Initial Stress Conditions, (b) Finite Element Grid Used to Model Sand Fill, and (c) Calculated and Actual Settlement Profiles

lower K_o values were assigned in areas of the fill where extension was expected to take place as a result of initial settlements during placement of the fill.

MODEL SETUP AND COMPUTATIONAL PROCEDURE

Figure 4(b) shows the finite element grid used to model the sand fill. The top of the grid includes a 12-foot-high surcharge over the entire area which extends to Elevation +115 feet. For the program used, it was not possible to directly specify nonzero boundary displacements. Therefore, a boundary displacement was simulated for the overlying fill by compressing a bottom row of "dummy" elements with applied forces. The dummy elements were assigned a modulus orders of magnitude stiffer than the modeled sand, and were setup as linear elastic, so a simple relationship between displacements and applied forces could be used. For convenience, the boundaries of the grid representing the modeled soil body (sand fill and natural sands) are superimposed on Figure 2.

Shown in Figure 4(c) are profiles of vertical displacements. The solid line indicates the calculated settlement profile for the organics which was used as the input displacements for the bottom of the sand fill model. The dashed line represents the vertical displacements predicted by the finite element model at Elevation +90 feet and, for comparison, the dotted line is the actual field settlement profile measured at Elevation +90 feet.

All load changes and the settlement increments were applied as nodal point forces. The initial stress conditions considered the surcharge inplace over the entire area. For the first computational step, the surcharge load was removed from the building area while an assumed building load (including the weight of a planned berm surrounding the structure) were applied simultaneously. Subsequent to analyzing these load changes, five displacement increments were applied to the bottom boundary. These displacement increments were percentage increments of the total settlement profile shown in Figure 4(c). The percentage increments applied were 4, 8, 16, 32, and 40, which, when summed, equal 100 percent. The nonlinearity of the soil was closely followed by using smaller displacement increments in the beginning. During preliminary runs, the application of the same total displacement in five equal increments had resulted in unrealistic results because of early development of shear stresses exceeding the soil's actual strength.

The program's restart option made it possible to interact between displacement increments to correct local stress inconsistencies due to the limitations of the nonlinear elastic model. It was necessary to interrupt the analysis for excessive stress buildup only once, after the third displacement increment, when 28 percent of the total displacement was applied. The excessive stress buildup was corrected by assigning artifically low moduli to the affected elements; by thus softening these elements their shear failure was simulated. The elements for which such modifications were necessary are shaded in Figure 4(b). As can be seen, the elements requiring softening are limited to a shallow zone. After application of the last displacement increment, additional elements were approaching critical stress conditions indicating that further displacement would result in widespread overstressing and thus, would render unreasonable results. No tension buildup occurred in any of the runs.

ANALYSIS RESULTS

The calculated nodal point displacement fields were plotted for each displacement increment applied to the bottom boundary. After 28 percent of the total settlement was applied, a fairly stable zone at the proposed foundation level was established which did not change appreciably during application of additional displacement increments. Figure 5 shows the displacement fields within the model after the third and fifth increments, equivalent to 28 and 100 percent of the total vertical settlement considered. The dashed lines in Figures 5(a) and 5(b) mark the edge boundaries beyond which calculated horizontal displacements were smaller than 1/4 inch. Within the overall accuracy of this analysis, any displacement less than about 1/4 inch was considered to indicate stable ground.

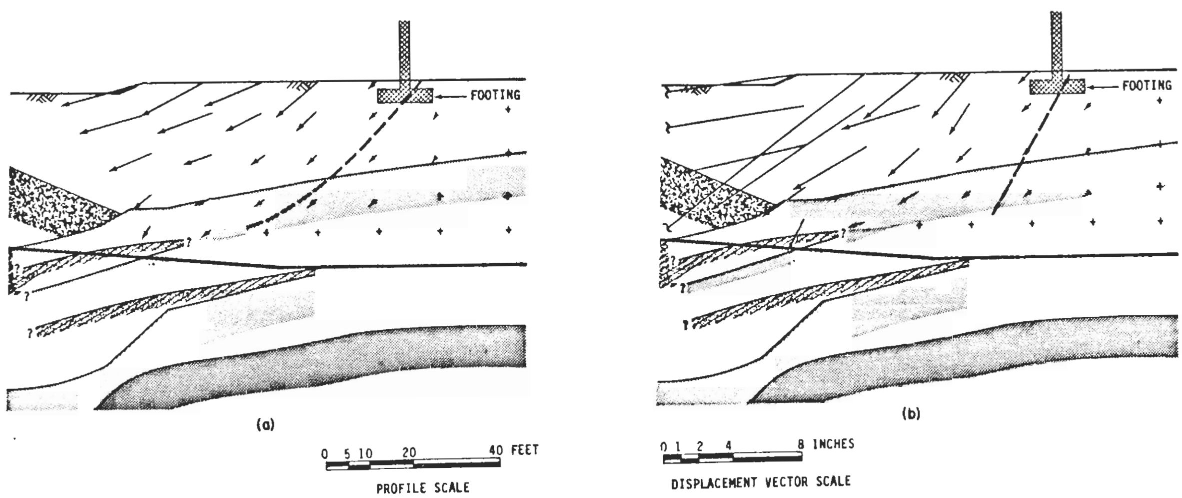

Figure 5. Displacement Fields with Boundaries of Stable Zones. (a) After Third Displacement Increment (28 percent of Total). (b) After Fifth Displacement Increment (100 percent of Total)

Although the total applied bottom boundary displacements more than tripled from the third to the fifth increment, the edge boundaries of both displacement fields intersect the final grade essentially at the same point. However, at depth, the edge boundary after the fifth increment is rotated further to the right, and the average inclination is noticeably steeper than after the third increment.

Based on the results of this analysis, it was concluded that the assumed building location was stable. This conclusion was based, in part, on the fact that the building would be circular in plan and, that, in the analysis, the building was assumed to be located at its closest approach to the deep organic profile. Also, the analysis had considered the problem to be two-dimensional (plane strain), and considered what was actually the worst organic profile in the area. As can be seen by the organic depth contours in Figure 1, the area represented by the analyzed profile is actually fairly localized. Finally, also ignored were the stabilizing and reinforcing effects the structure itself would have on the soil through friction along the base of the foundation and the strength of the structural framing.

Supplementary runs made on a slightly modified profile showed that the location and orientation of the edge boundary of the displacement field depends on the depth to the bottom boundary and the shape of the applied settlement profile. In general, it appears that as the thickness of the sand fill decreases, the slope of the edge boundary becomes flatter. Analysis of a sand fill only 25 feet thick showed the edge boundary to be approximately a straight line inclined at 30 to 35 degrees to the horizontal. In comparison, the almost 45-foot-thick sand fill analyzed in Figures 5(a) and 5(b), shows an edge boundary inclined at an average of 40 to 60 degrees.

The supplementary studies were used to evaluate the effects of a change in the actual grading scheme for development of the site area. The finite element model used in the primary analysis considered a homogenous sand fill within the boundary shown in Figure 2. However, changes in the initial exavation plan resulted in significantly less organic material actually being excavated than was assumed in the analysis. The modified grading plan left an additional wedge of muck inplace as shown in Figure 2. This resulted in a steeper settlement profile (dotted line in Figure 4) than considered in the analysis (dashed line). In addition, the settlement contribution of these organics is acting on the sand fill at a level significantly above the assumed bottom boundary. Superimposing the results of the supplementary study of the thinner sand fill model, it was concluded that the change in the excavation plan would not alter the location of the stable zone boundary and, therefore, the building could be located as analyzed.

COMPARISON OF MEASURED AND CALCULATED DISPLACEMENTS

Actual displacements within and adjacent to the organic soil area were monitored using surface settlement plates and inclinometer casings. This data was used to check the deformations predicted from the numerical analysis. The settlement plates were installed in a 100-foot grid pattern. The locations of Plates 34, 35, 43, and 44 are shown in Figure 2. Three inclinometer casings, designated 1, 2A, and 3, were installed to monitor the lateral deflections occurring in the fill and upper sand profile. The locations and depths of these casings are shown in Figures 1 and 2.

The additional muck left in place by the change in the grading plan resulted in a steeper settlement profile near the top of the fill (where it was measured) than the analysis predicted based on the settlement profile applied to the bottom boundary. These differences have been previously discussed and are compared in Figure 4(c).

The measured and calculated profiles of horizontal displacements at inclinometer locations are compared in Figure 6. The reliability of the data from Casing 3 is reduced by its tendency to buckle due to vertical settlement of the fill. Also, Casing 1 is known to have been disturbed by construction equipment prior to the time the last profile was recorded. However, it is not likely that this disturbance reached deeper than approximately 10 to 15 feet. Taking into account the shortcomings of the instrumentation data and the differences between the actual and analyzed profiles, it is concluded that the finite element results agreed reasonably well with the actual field displacements.

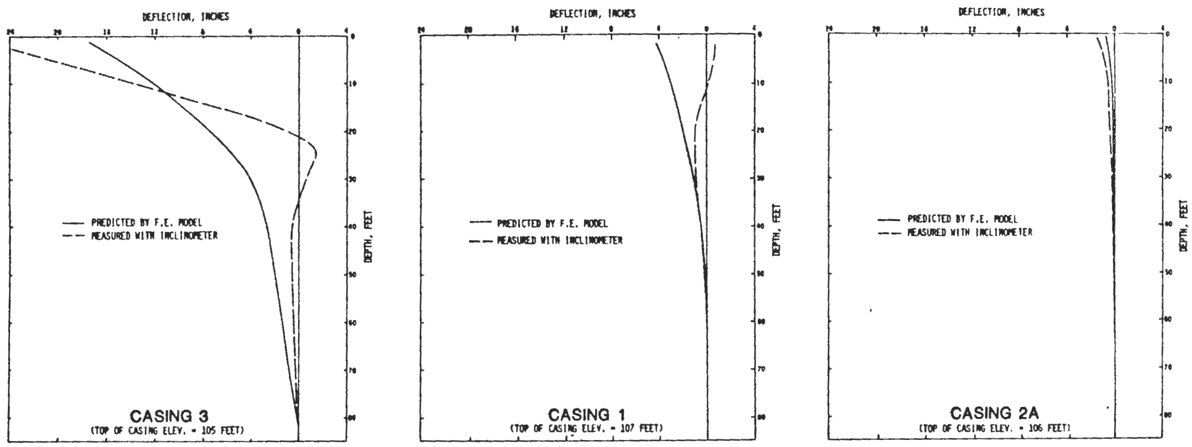

Figure 6. Comparison of Predicted and Measured Horizontal Deflections at Inclinometer Casing Locations

SUMMARY AND CONCLUSIONS

Displacement patterns within a sand fill which partially extends onto highly compressible organic soils were analyzed using finite element techniques. The study was undertaken to assist in defining a safe distance from the edge of the soft ground area where a structure could be supported within the sand fill. The sand fill was instrumented with settlement plates and inclinometers to check the analytical results. On the basis of analyses of several different profiles, it was concluded that the location and shape of the edge boundary of the stable zone within the sand fill model depends on:

1. The shape of the settlement profile due to the compressive layer;

2. The magnitude of the settlements; and

3. The depth to the compressive layer underlying the fill.

In general, these studies showed that, as the depth of the compression layer increases, the average inclination of the boundary of the stable zone becomes steeper. However, sufficient parametric work was not performed during this study to support generalization of the results discussed.

The subject study is an example of how standard numerical tools can be used without jeopardizing a project budget with costly and time consuming program development efforts. Using a well documented computer program within its proven realm of applications made it possible to keep labor and computer costs low. For the modeling of the one subsurface profile discussed in this paper (excluding the supplementary studies) the total CPU time was 2-1/2 minutes on a CDC 6600 computer with the costs for storage devices approximately equaling the computation costs. Approximately 40 man-hours were required for model setup, program execution, and data reduction.

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