Field Performance of a Lateral Earth Support System

Comportement In Situ d'un Système de Soutènement Ancré

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SYNOPSIS The report describes the construction of a relatively new lateral earth support system for deep excavation. Unlike the conventional systems that serve to retain the soil behind a vertical cut, this new procedure is based on the concept of soil reinforcement. The current paper presents the results of a recent study wherein the above mentioned system was applied to a 10 meter deep excavation. The primary objectives of this project were (1) to monitor the field performance of the new system both during and following the period of construction, and (2) to verify the methodology recently developed by the authors for the design of analysis of such systems. The former objective was achieved by installing an extensive field instrumentation network, while the latter goal was realized by comparing the field measurements to the predictions obtained with the finite element program REA developed by Herrmann (1978).

INTRODUCTION

In recent years, a relatively new lateral earth support system for deep excavation has been introduced. Unlike the conventional systems that serve to retain the soil behind a vertical cut, this new procedure is based on the concept of soil reinforcement. That is, the native soil adjacent to the excavation is strengthened so that it can stand unsupported at depths which would normally require the installation of sheet piling or soldier piles and bracings.

Briefly, the new system is composed of an array of reinforcing anchors that are grouted into the soil mass, a wire-mesh reinforced shotcrete panel facing, and rows of re-bars which form horizontal wales at each anchor level. The various components of this new system are shown in Figure 1.

Excavation begins from the ground level. After each layer of excavation, reinforcement is immediately applied to both the native soil and the exposed cut. The system provides a rather unique means of temporary earth support, with the advantages that it (1) requires no pile driving, (2) prevents any loosening or sloughing of the soil, and (3) provides an obstruction free site for subsequent foundation work. The system has so far been successfully used for thousands of square meters of excavation, to depths of up to about 25 meters.

An analytical procedure for evaluating the stability of the system was first proposed by Bang (1979), and was later verified by means of centrifuge model studies (Shen et al, 1979). A method for predicting the behavior of the system at the working-stress level, as well as a detailed description of the construction procedure, have been provided by both Bang (1979) and Shen et al (1979).

The current paper presents the results of a recent study wherein the above mentioned system was applied to a 10 meter deep excavation. The primary objectives of this project were (1) to monitor the field performance of the new system both during and following the period of construction, and (2) to verify the methodology recently developed by the authors for the design and analysis of such systems. This latter goal was realized by comparing the field measurements with the predictions obtained by means of the finite element program REA (Herrmann, 1978).

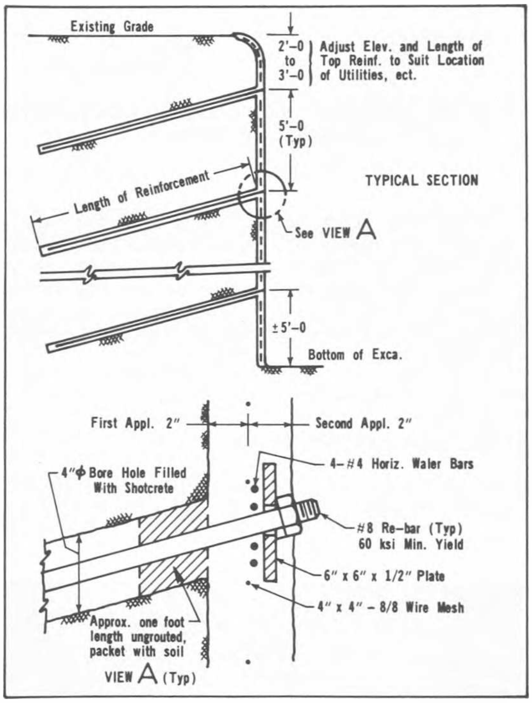

FIG. 1 Construction Details - A New Shoring System

SITE AND SOIL CONDITIONS

The construction site was located on the Davis Campus of the University of California, due north of the sanitary landfill facility. The terrain was flat, and the soil deposit was composed primarily of interbedded layers of sandy silts and silty clays. Also present in this typical Putah Creek alluvial deposit were lenses of fine river sand, coarse sand, and fine gravel. Undisturbed samples were taken to a depth of 25 meters using a pitcher sampler tube. A gravel layer and the groundwater table were encountered at this depth.

During the excavation, it was observed that the soil deposit was extremely variable in both the vertical and the horizontal directions. Undrained triaxial confined compression tests were performed in order to determine the shear strengths of the various soil types that were encountered. The Mohr-Coulomb failure envelope associated with these test results is shown in Figure 2. As a result of the very inhomogeneous nature of the excavation site, it would be impossible to delineate the various soil layers when performing the finite element analysis. Hence, it was concluded that either an average failure envelope or a set of upper and lower bounds would be the most appropriate indicator of the strength properties of the entire soil mass.

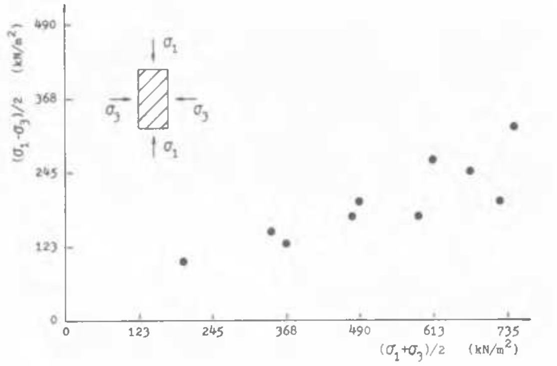

FIG. 2 Results of the Undrained Triaxial Confined Compression Tests

THE LATERAL EARTH SUPPORT SYSTEM

A view of the completed excavation, with the lateral earth support system in place, is shown in Figure 3. The 10 meter deep excavation was constructed in 5 lifts, with each lift being approximately 2 meters deep. The geometry of the vertical facing and the layout of the reinforcing anchors are shown in Figure 4. The remaining three sides of the excavation were left untreated, and had slopes of about 45 degrees.

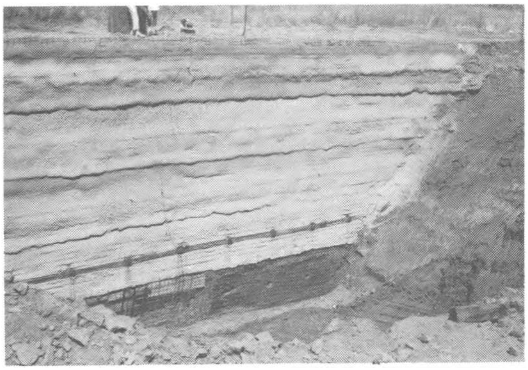

FIG. 3 A View of the Excavation Site

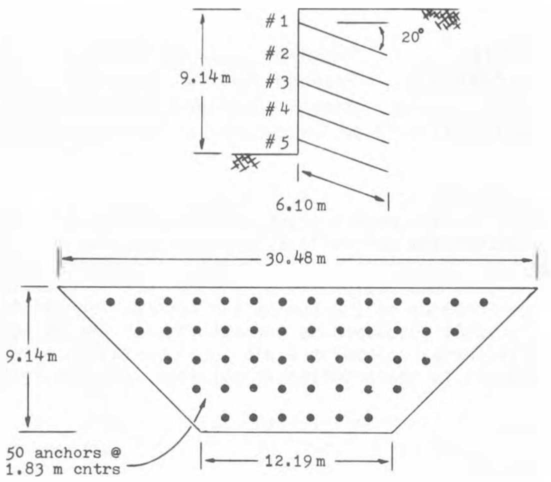

FIG. 4 Orientation of the Reinforcing Anchors

INSTRUMENTATION AND MEASUREMENT

Extensive instrumentation was installed at the site so that the field performance of the system could be monitored. Four borehole inclinometers were installed along the centerline of the excavation in order to obtain a set of horizontal displacement profiles. The inclinometers were placed at distances of 1.52, 4.57, 9.14, and 15.24 meters behind the edge of the vertical cut. The two boreholes closest to the excavation (#1 and #2) were each 18.29 meters deep, while the remaining two boreholes (#3 and #4) were each 12.19 meters deep. A network of surface movement markers was installed prior to the start of excavation so that the horizontal and vertical deformation of the ground surface could be monitored by transit survey. Finally, the centermost column of reinforcing anchors was equipped with resistance type strain gages. Four sets of gages were spot welded to each anchor rod so that the magnitude and the distribution of the axial force in each bar could be measured. A complete set of borehole inclinometer and survey readings was taken before and after each lift of excavation. Strain gage readings were made after each bar had been grouted into place, and after the completion of each subsequent lift.

The horizontal deflection profiles yielded by inclinometers #1 and #2 are shown in Figures 5 and 6, respectively. It may be observed from these records that:

a) the horizontal deflection decreases with increasing depth from the ground surface;
b) the horizontal deflection decreases with increasing distance behind the vertical cut;
c) the horizontal deflection increases as the excavation depth increases; and,
d) the horizontal deflections tend to be relatively small at depths greater than the current excavation depth.

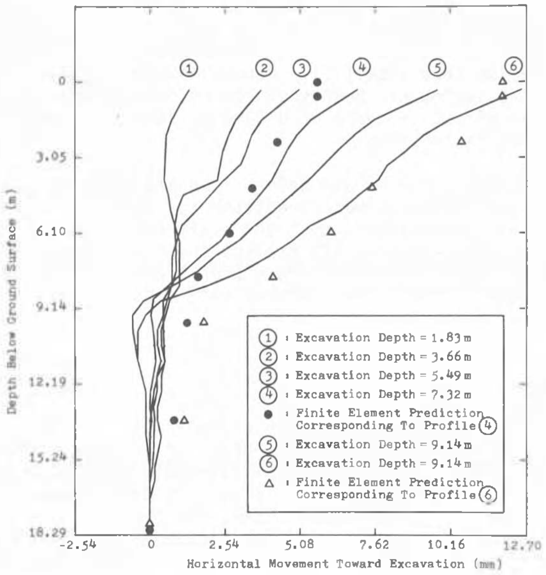

FIG. 5 Horizontal Deflection Profile At Inclinometer #1 (1.52 meters behind the excavation)

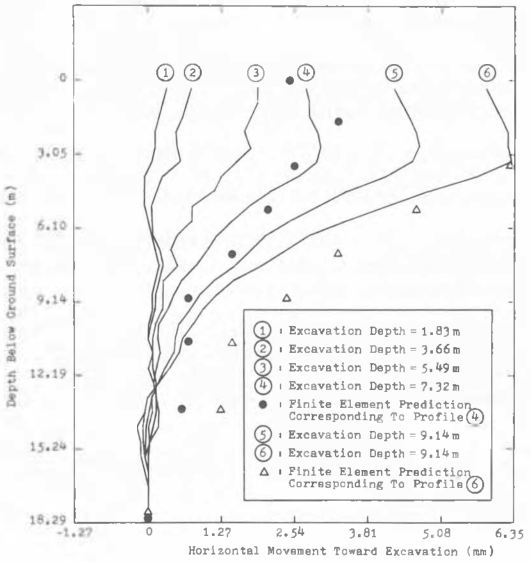

FIG. 6 Horizontal Deflection Profile At Inclinometer #2 (4.57 meters behind the excavation)

It is also of interest to note that profiles #5 and #6 (Figures 5 and 6) were both taken after the final lift of excavation had been completed. Profiles #5 were obtained immediately after completion, whereas profiles #6 were recorded two weeks later. Hence, there was a slight amount of short term movement. Additional readings were continued throughout the wet winter and spring months. During this period, several large earthquakes centered approximately 150 kilometers southeast of the site shook the area. However, no movements greater than those reflected by profiles #6 were recorded. Thus, it may be concluded that the medium term stability of the system was quite high.

The record of ground surface movements obtained by transit survey is shown in Figure 7. As may be seen, the survey readings are in close agreement with the inclinometer measurements. In both cases the maximum displacements are quite small - approximately 1 cm at the center of the uppermost edge of the excavation.

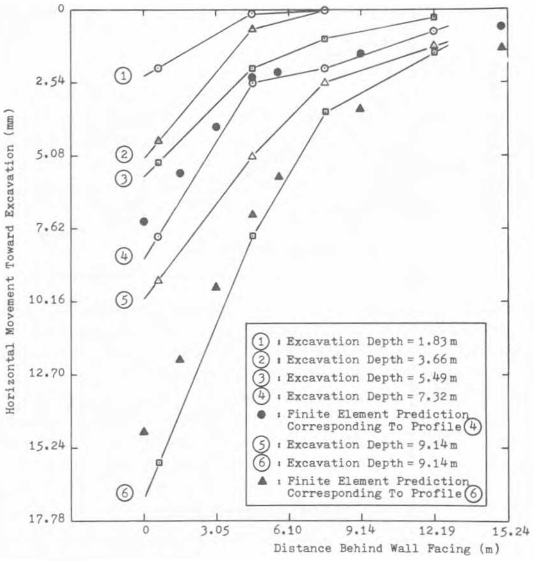

FIG. 7 Horizontal Movement of Ground Surface at the Excavation Centerline (by field transit survey)

The axial force distributions in the 2nd, 3rd, and 4th level anchor rods are shown in Figure 8. It may be noted that, for a given excavation depth, the maximum axial force at the various levels is approximately the same. The force distributions correspond most closely to the "rigid facing" distributions established by Mitchell (1977) while researching the development of friction in Reinforced Earth. That is, the force distribution is roughly triangular, with a maximum at or near the wall facing, and decreasing to zero at the free end.

COMPARISON WITH THE FINITE ELEMENT PREDICTIONS

In order to verify the analytical procedure recently reported by Bang et al (1980), the behavior of the wall was predicted through analyses using the finite element program REA developed by Herrmann (1978). Due to the extremely heterogeneous nature of the actual excavation site, during the finite element analyses the soil was modelled as a homogeneous deposit, and was assigned a set of average strength properties.

The predicted horizontal deflection profiles of inclinometers #1 and #2 (1.52 and 4.57 meters behind the vertical cut, respectively) are shown in Figures 5 and 6, respectively. For each of these locations, the predicted profiles corresponding to excavation depths of both 7.32 and 9.14 meters are presented. In all instances the agreement between the measured and the predicted movements is quite good. As may be seen, for depths less

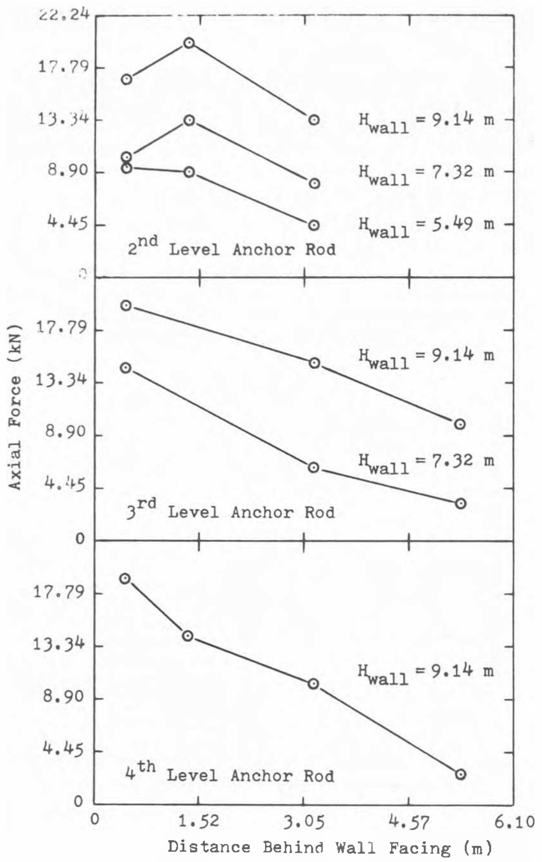

FIG. 8 Measured Axial Force Distribution in 2nd, 3rd and 4th Level Anchor Rod

than about 6 meters the agreement is remarkably close. However, for greater depths the finite element predictions tend to exceed the actual measured ground movements. It is likely that these discrepancies are primarily the result of the somewhat inaccurate modeling of the soil deposit, rather than any fundamental error in the analytical technique.

As may be observed in Figure 7, the predicted ground surface movements are likewise in excellent agreement with the results of the field transit surveys. The predicted axial force distributions in the centermost column of reinforcing anchors are not shown. However, these too were relatively close to the measured distributions presented in Figure 8.

SUMMARY AND CONCLUSIONS

A 10 meter deep field prototype of the new lateral earth support system was constructed during the summer of 1979 at a cost of about $110 per square meter of wall. As the various field measurements reveal, the measured horizontal and vertical movements of the ground adjacent to the excavation were very small, and well within the limits associated with the conventional lateral support systems. Although the system is intended to serve as a temporary support only, the fact that no additional movements were recorded during a 9 month period following the completion of construction - a period which included a very wet winter and spring, and several small earthquakes - indicates that the system also has a great deal of medium term stability. Hence, the new lateral earth support system has been shown to be both an economical and an effective means of providing temporary support in deep excavations.

The measured forces and deformation are in good agreement with the finite element predictions. Thus, the study has also demonstrated that the newly developed analytical procedures can correctly predict the field behavior. Hence, these analytical procedures provide a rational approach to the economical and safe design of the system.

ACKNOWLEDGMENTS

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