Predicted and Observed Behavior of a Deep-Soil-Mixing Braced Wall

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Soil-mixing technique has many applications and is gaining popularity in the construction industry. An experience wherein a braced deep-soil-mixing (DSM) wall 18.3 m (60 ft) deep was used for an 11.3-m excavation 11.3 m (37 ft) deep under difficult subsurface conditions is presented. Reinforcing beams were installed inside the overlapping DSM columns, which made up a continuous retaining wall. Two levels of bracing struts at depths of 3 and 7.6 m (10 and 25 ft) were used for the complete excavation. Preconstruction prediction of the wall deflection was obtained using a beam-column computer solution, BMCOL76. Soil-structure interaction was modeled by specific nonlinear soil resistance curves. The stiffness of the composite wall was appropriately modeled on the basis of the properties of the soil-grout mixture. The measured behavior of the wall was compared with the preconstruction prediction.

Though soil-mixing technique has been widely used in many countries (*1–3*), it is a relatively new concept in the United States. The method has been applied in the industry so recently in this country that even the terminology of its many applications is a subject of debate (*4–6*). The construction application of soil mixing typically results in a series of interconnecting soil-grout mix columns referred to as "soil mixing walls," "soil-cement mixing," or "soil cement in situ walls." A special branch of this technique applied to deeper ground is called "deep soil layer mixing" or "deep soil mixing." For the particular application described herein and also because of its specific service-marked term, deep-soil-mixing wall, or DSM wall, is used throughout this paper. Applications of deep soil mixing (DSM) or shallow soil mixing (SSM) include soil stabilization, underwater soil improvement, soil remediation, foundation elements, and retaining walls. There is little documentation concerning the use of DSM or SSM techniques in soil improvement and stabilization projects (*7–9*).

This paper describes the use of a DSM wall 18.3 m (60 ft) deep as a temporary structural retaining wall for an excavation 11.3 m (37 ft) deep. DSM provided an attractive construction alternative in this case where a restricted construction area, contaminated subsoils, and shallow groundwater level limited the use of other conventional retaining structures. The predicted and observed behavior of the DSM wall are described.

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37-ft EXCAVATION

The plan area of excavation was about 99.7 × 16.2 m (327 × 53 ft). The bottom of the excavation was about 11.3 m (37 ft) below existing ground surface in most of the area. The excavation area was located in a built-up area and part of the excavation boundary was only a few feet away from an adjacent facility. Intolerance of the adjacent facility to construction vibration restricted many retaining wall construction methods, such as pile driving. The situation was further complicated by another restriction minimizing the pumping of contaminated groundwater and excavation of contaminated subsoils during construction of the retention system. These environmental concerns led to a search for a retaining system that would also serve as a groundwater cutoff wall. The final decision was made that a reinforced DSM wall system should be used. The reinforced DSM system was perceived to provide a structural wall, and its construction would neither require excavation and dewatering nor produce any vibration during construction. The DSM wall system consisted of 344 DSM columns with appropriate reinforcement. Inclinometers were placed at selected locations to monitor the movement of the wall throughout the excavation process. Figure 1 presents the general project layout with inclinometer locations.

SUBSURFACE CONDITIONS

The project area is located on the upper Texas Gulf Coast and situated on Holocene alluvium (fluvial deposit) overlying the Pleistocene sediments. The generalized soil profile at the project site is shown in Figure 2. The six zones of the project area are summarized as follows:

- Zone 1 [depths of 0 to 3 m (0 to 10 ft)] consists of loose to medium-dense fine sand and silty fine sand.
- Zone 2 [depths of 3 m to 7.9 m (10 to 26 ft)] consists of very soft to firm clay with sand layers.
- Zone 3 [depths of 7.9 m to 15.2 m (26 to 50 ft)] consists of medium-dense to very dense fine sand.
- Zone 4 [depths of 15.2 m to 25.3 m (50 to 83 ft)] consists of firm to very stiff clay.
- Zone 5 [depths of 25.3 m to 34.7 m (83 to 114 ft)] consists of medium- to very dense silty fine sand and fine sand.
- Zone 6 [depths below 34.7 m (114 ft)] consists of firm to hard clay and silty clay.

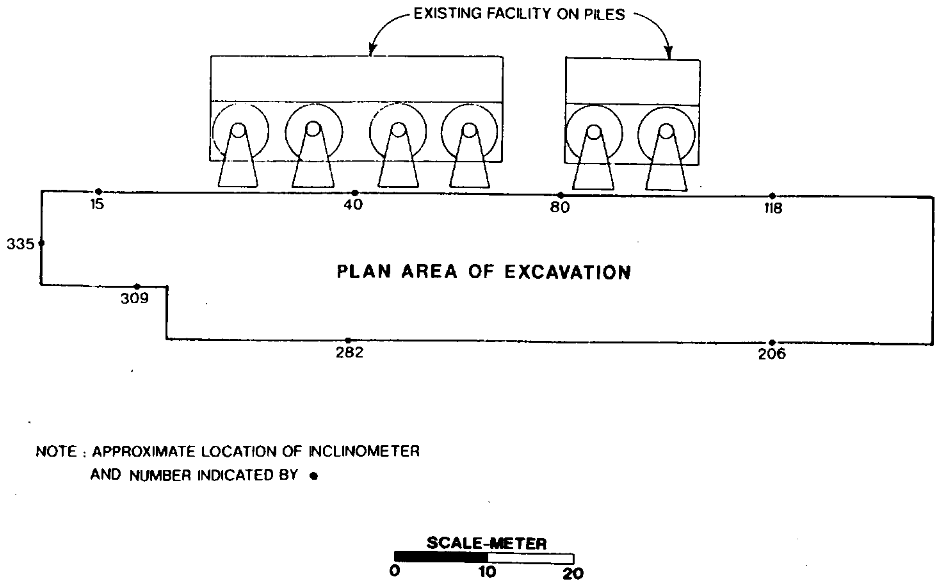

FIGURE 1 **Plan area of excavation.**

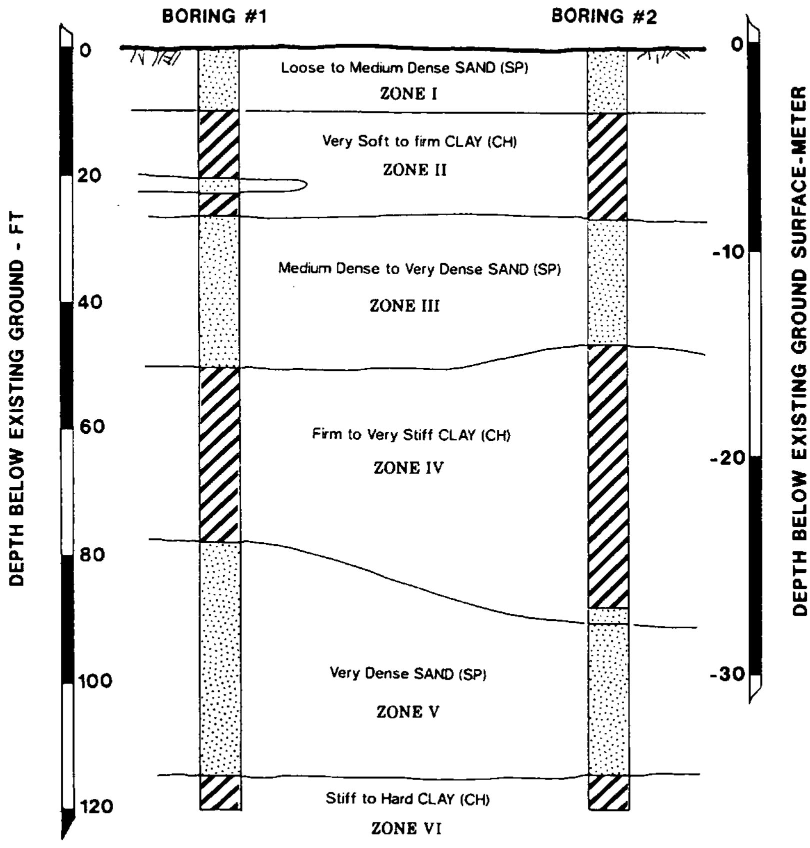

FIGURE 2 **Generalized soil profile.**

Figures 3 and 4 present the uncorrected standard penetration tests (SPTs) "N" profile and the undrained shear strength profile, respectively. The groundwater level was observed to be 2.4 m (8 ft) deep. The bottom of the excavation was about 11.3 m (37 ft) below ground and based within the Zone 3 cohesionless soils. Because of the high groundwater level, conventional excavation would require dewatering or a cutoff wall penetrating into the Zone 4 cohesive soils. The deep soil mixing technique provided an attractive means to construct a system serving as both a groundwater cutoff curtain and a retaining wall.

DSM WALL

The equipment used to construct the DSM wall included a specially designed soil mixing rig and a grout mixing plant. The rig used a 1335-KN (150-ton) crane with a supporting set of leads which guided four hollow-stemmed augers. A series of overlapping auger flights 914 mm (36 in.) in diameter were welded to those four augers. As the discontinuous auger flights were advanced into the ground, a cement-based grout was pumped through each hollow-stemmed auger shaft and discharged at the bottom of the auger. The soilcrete columns were produced by the rotation of the beaters along the auger stem while drilling. Once the required depth was achieved, the auger stem rotation was reversed. Mixing continued as the augers were withdrawn.

Continuity of the soilcrete columns was achieved in two ways: (a) the four augers laterally overlapped each other on each stroke drilled, up to 229 mm (9 in.), as shown in Figure 5 (*top*), and (b) each stroke drilled was vertically overlapped by one auger from the previous stroke, which created a continuous soilcrete wall, as shown in Figure 5 (*bottom*). Each DSM column in this study was designed to penetrate at least 0.6 (2 ft) into the relatively impervious cohesive soils (Zone 4), which is about 15.2 m (50 ft) below ground surface.

A wide flange (WF) 24 × 104 steel beam was inserted into each DSM column before the soilcrete was set to serve as structural reinforcement. A vibratory hammer (Model ICE 815) was used to vibrodrive the reinforcing beam below the bottom of the soilcrete column into the Zone 4 cohesive soils to tip at 18.3 m (60 ft) below ground surface. Minimum vibration was induced during driving within the deeper cohesive layer.

SOILCRETE

The soilcrete was formed by blending the grout and the soil in each DSM column. To achieve the desired strength of the soilcrete mixture, the design mix included 200 kg (450 lb) of Type I portland cement and 2 kg (4.5 lb) of M-1 Wyoming Gel injected into 0.76 m^3 (each cubic yard) of soil. The grout used in this project was prepared in a 3790 L (1,000 gal) storage tank with a specific gravity of 1.35 and a water/cement

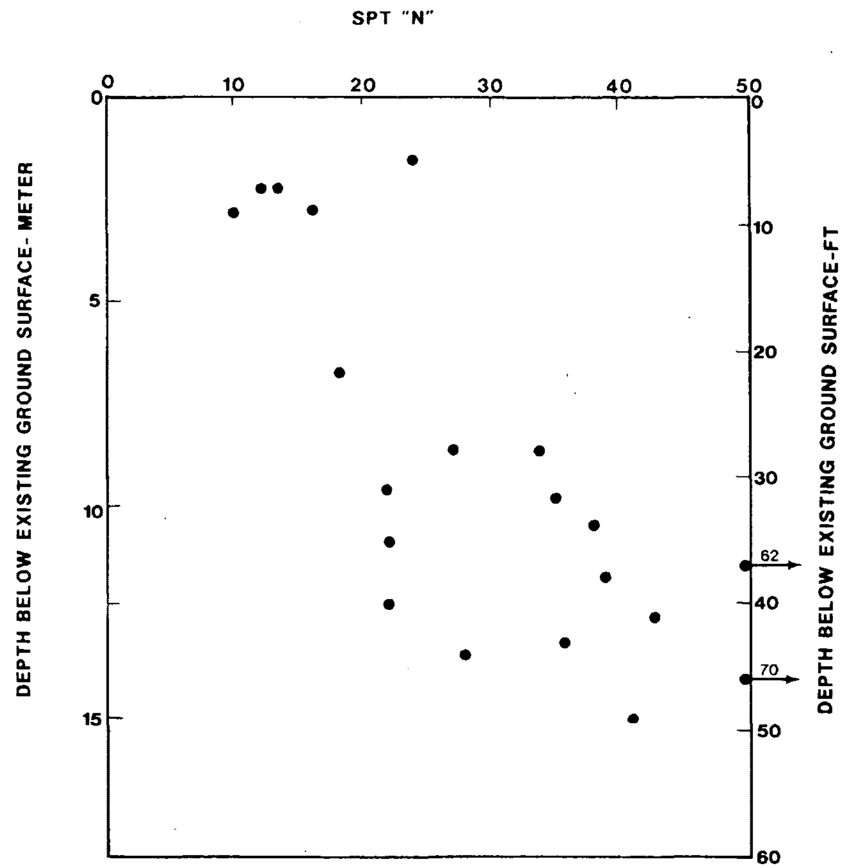

FIGURE 3 SPT "N" profile.

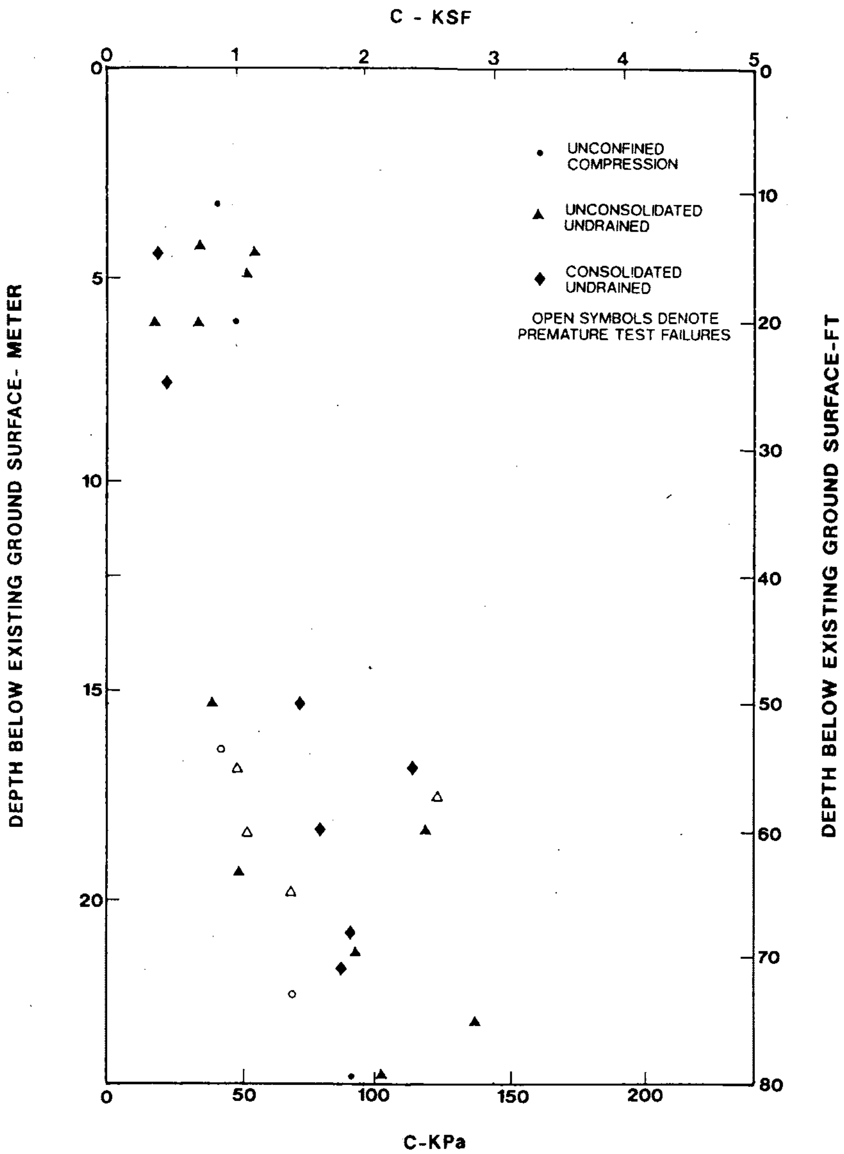

FIGURE 4 Undrained shear strength profile.

ratio of 1:2. A grout flow rate of 83 to 106 L/min (22 to 28 gal/min) was measured at the four pumps that delivered the grout to the shafts.

Many soilcrete samples were obtained throughout the construction process. Each sample was about 76 mm (3 in.) in diameter and 152 mm (6 in.) high. The samples were capped and allowed to cure at ambient temperatures of about 70°F. The samples were tested for unconfined compressive strengths and permeability in the laboratory. Table 1 summarizes the results of laboratory testing on the soilcrete samples obtained at various depths. Specific depth information was not recorded. The strength data exhibit large scatter, as evidenced by their mathematical means and their coefficients of variation. The data scatter reflects the heterogeneity of the subsoils and suggests a relatively nonuniform or nonhomogeneous mixture in the soilcrete column. The trend, however, indicates about a 100 percent increase in compressive strength between the 3- to 7-day and the 7- to 28-day curing periods. Moduli of elasticity of the soilcrete samples, based on the stress-strain relationships from the unconfined compression testing, were estimated to range from 0.1×10^6 KPa to 0.26×10^6 KPa (1.48×10^4 to 3.75×10^4 psi). The coefficients of permeability of the four soilcrete samples ranged from 1.32×10^7 to 7.10×10^7 cm/sec.

PRECONSTRUCTION PREDICTION

Because of the innovative use of the DSM columns as a retaining wall system, extensive numerical analyses were performed before the final design scheme was selected. Analyses were performed to optimize the depth of wall, the selection of reinforcing beam, and the bracing of the DSM wall. The performance of the wall was evaluated based on the predicted load-deflection characteristics using a beam-column computer solution BMCOL76 (*10*). The computer procedure used the

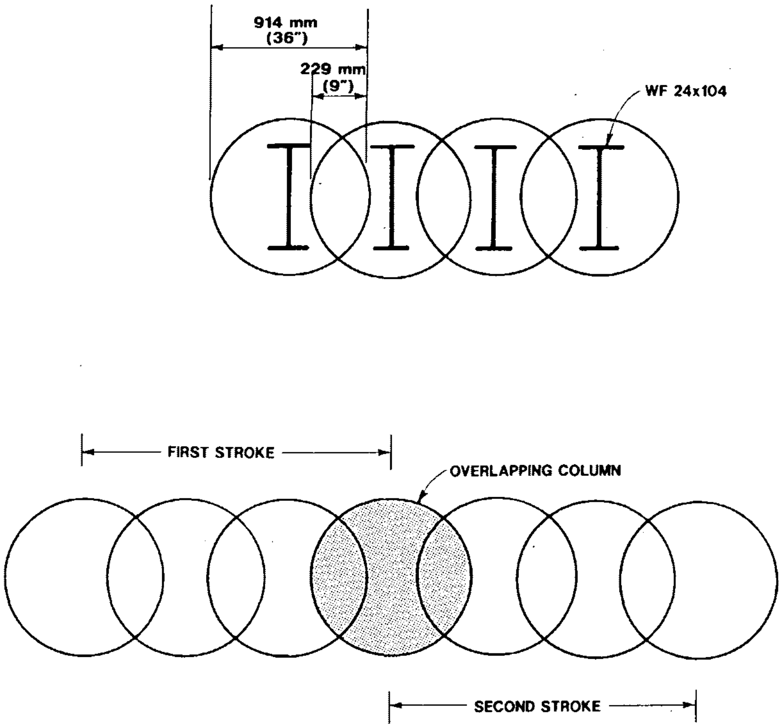

FIGURE 5 Overlapping of DSM columns: lateral overlap, *top*; vertical overlap, *bottom*.

TABLE 1 Properties of Soilcrete on Basis of Laboratory Testing

3-day strength (KPa)	7-day strength (KPa)	28-day strength (KPa)	Permeability (cm/sec)
113.0	184.0	517.4	1.32×10^{7}
126.1	306.6	765.5	3.40×10^{7}
136.4	311.4	834.4	3.60×10^{7}
141.9	421.7	961.2	7.10×10^{7}
144.0	438.2	1001.1	
149.5	498.8	1121.0	
170.9	509.2	1245.0	
171.6	531.2	1263.6	
206.7	633.9	1485.5	
209.5	672.5		
254.2	695.2		
284.6	959.1		
416.2			
427.9			
491.9			
500.2			
Mean - 246.7	Mean - 513.3	Mean - 1021.8	
Median - 189.5	Median - 504.3	Median - 1001.1	
Standard Deviation - 135.7	Standard Deviation - 208.8	Standard Deviation - 294.2	
Coefficient of Variation 55%	Coefficient of Variation 41%	Coefficient of Variation 29%	

Note: data are arranged in ascending order.
1 KPa = 0.145 psi

TABLE 2 Soil Parameters Used for Preconstruction Prediction

Range of Depth (ft)	Soil Type	Undrained Shear Strength (KPa)	Angle of Friction
0 - 8	Sand	--	29
8 - 26	Clay	23.9	--
26 - 50	Sand	--	34
50 - 60	Clay	86.1	--

Note: 1 KPa = 20.9 psf

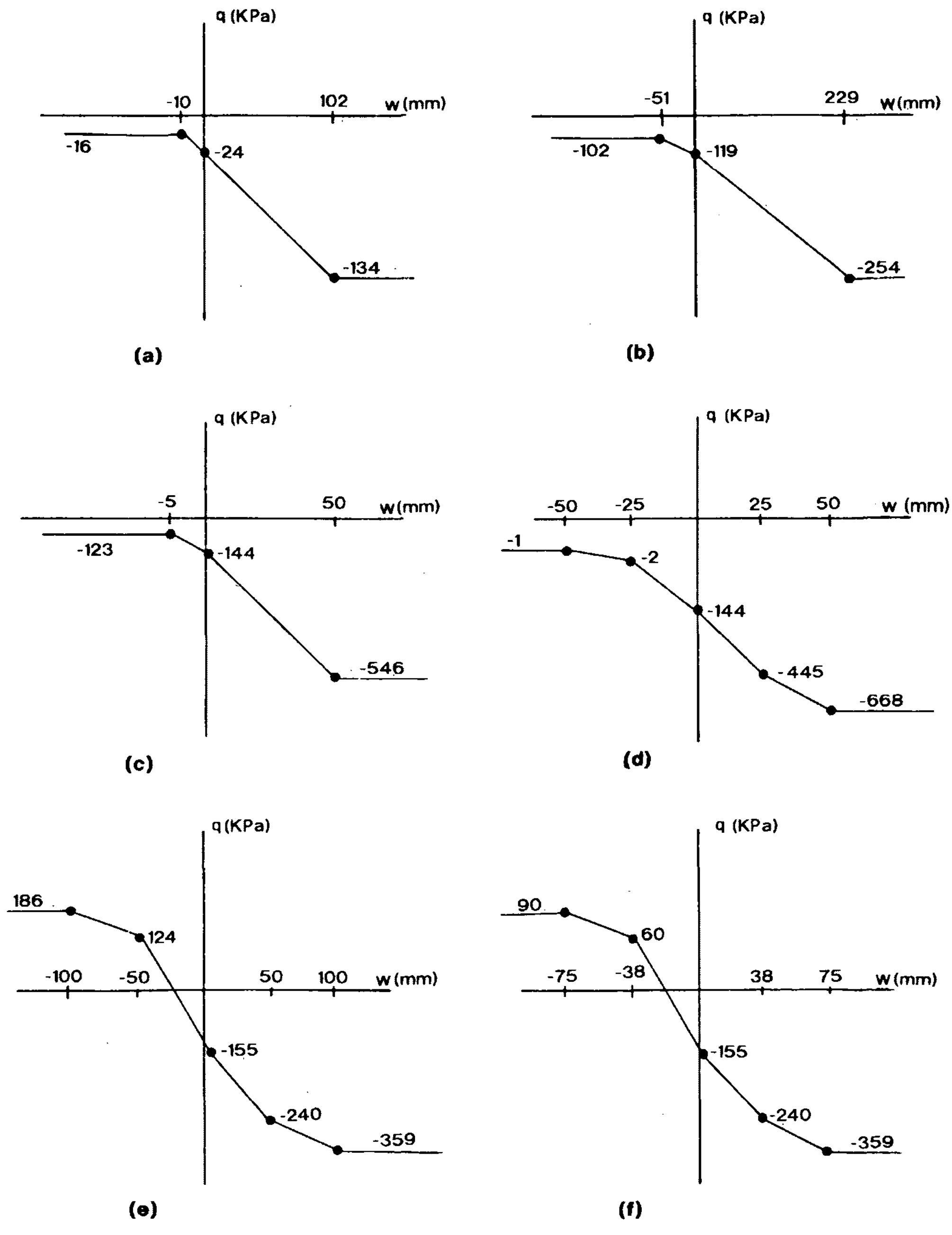

FIGURE 6 *q-w* curves for 37-ft excavation: *a*, 2.4-m depth; *b*, 7.9-m depth; *c*, 11.3-m depth; *d*, 15.2-m depth; *e*, 15.5-m depth; *f*, 18.3-m depth (negative sign denotes direction toward excavation).

concept of Hetenyi's beam on elastic foundation theory (*11*) but permitted the soil to be modeled as a nonlinear medium. Two important factors affecting the flexible wall behavior were the effects of soil-structure interaction and the wall stiffness.

The mechanism of soil-structure interaction can be modeled by entering nonlinear soil resistance relationships, termed *q-w* curves, into the computer procedure. The required *q-w* curves are typically obtained from classical wall-deflection requirements for active and passive stress states (*12,13*). Complete details of the generation of *q-w* curves at any depth along the wall are detailed by Haliburton (*14*). It is known that lateral earth pressures (both active and passive) change with time for retaining structures as strength of clay soil changes with time (*15*). Williams and Baka (*16*) have shown that mobilized active pressures, in the case of a cantilever wall system, can increase 50 percent over a 30- to 60-day period as drained shear strength of clay soil is gradually developed. Daniel and Olson (*17*) concluded that the failure of an anchored bulkhead was due to the lack of understanding of the soil behavior, particularly that the fully drained strength of cohesive soil would be less than that of the undrained strength.

The preconstruction analysis assumed the after-construction (short-term) condition because the final structural slab and concrete perimeter wall would be cast immediately after completion of excavation. The earth pressures developed for the analysis were based on short-term soil parameters (i.e., undrained shear strengths for cohesive soils). Soil parameters used to develop active and passive pressures are presented in Table 2. However, the actual lateral pressures realized during construction might be in the transition from undrained shear strength to drained shear strength as excess pore water pressures would be partially dissipated during the construction time period. Certain conservatism and judgment based on past experience were imposed in selecting the appropriate coefficients of lateral earth pressures in the preconstruction prediction process. A set of *q-w* curves used to analyze the complete excavation under the final design scheme is shown in Figure 6. The *q-w* curves were generated on the basis of the short-term soil parameters to closely model the temporary nature of the wall.

Another important parameter in the analysis was the bending stiffness of the composite wall (*EI*), where *E* is the modulus of elasticity of the wall material and *I* is the moment of inertia. On the basis of the laboratory test results of the soilcrete as described previously, the *EI* for the DSM wall without reinforcing steel was found to range from 1.88×10^6 to 4.77×10^6 N-m^2 (6.55×10^8 to 1.66×10^9 lb-in.^{2}) per 0.3 m (1 ft) width of the wall. The *EI* for the reinforcing beam was 1.19×10^8 N-m^2 (4.13×10^{10} lb-in.^{2}) per 0.3 m (1 ft) width of the wall. The *EI* of the soilcrete column was about 2 to 4 percent of the *EI* of the reinforcing beam. Thus the *EI* of the reinforcing beam was conservatively taken as the *EI* of the composite wall in this study.

The cross bracings used for the braced excavation were 16.1 m (53 ft) long, hollow steel tube sections (TS16×16). Com-

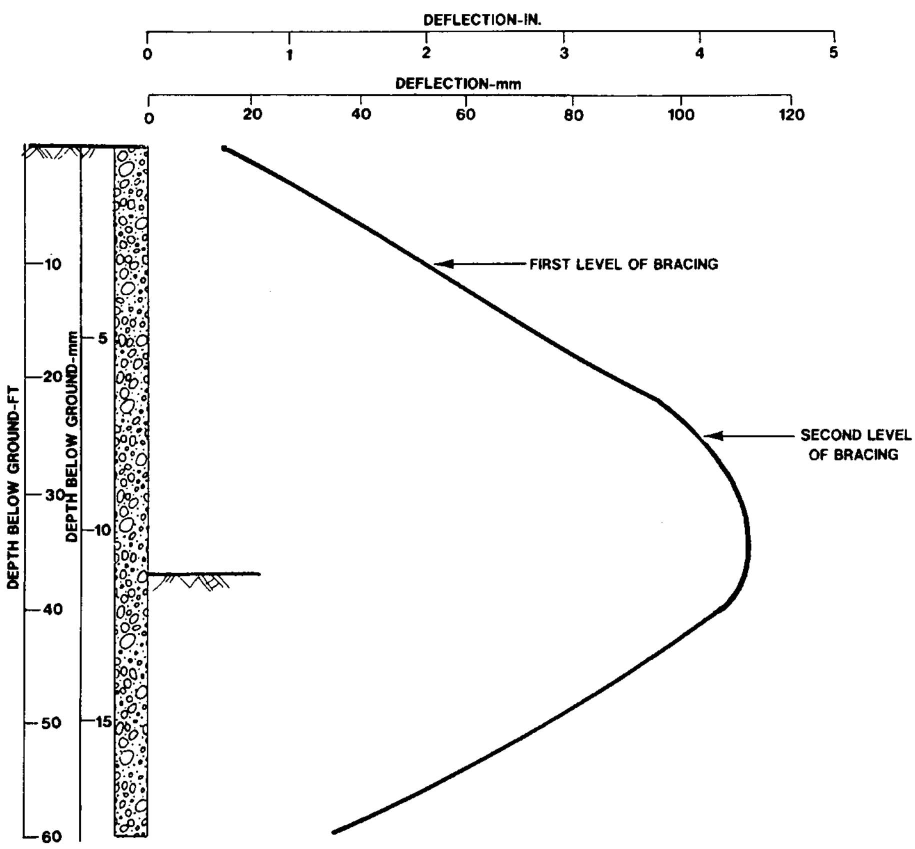

FIGURE 7 Predicted deflection profile.

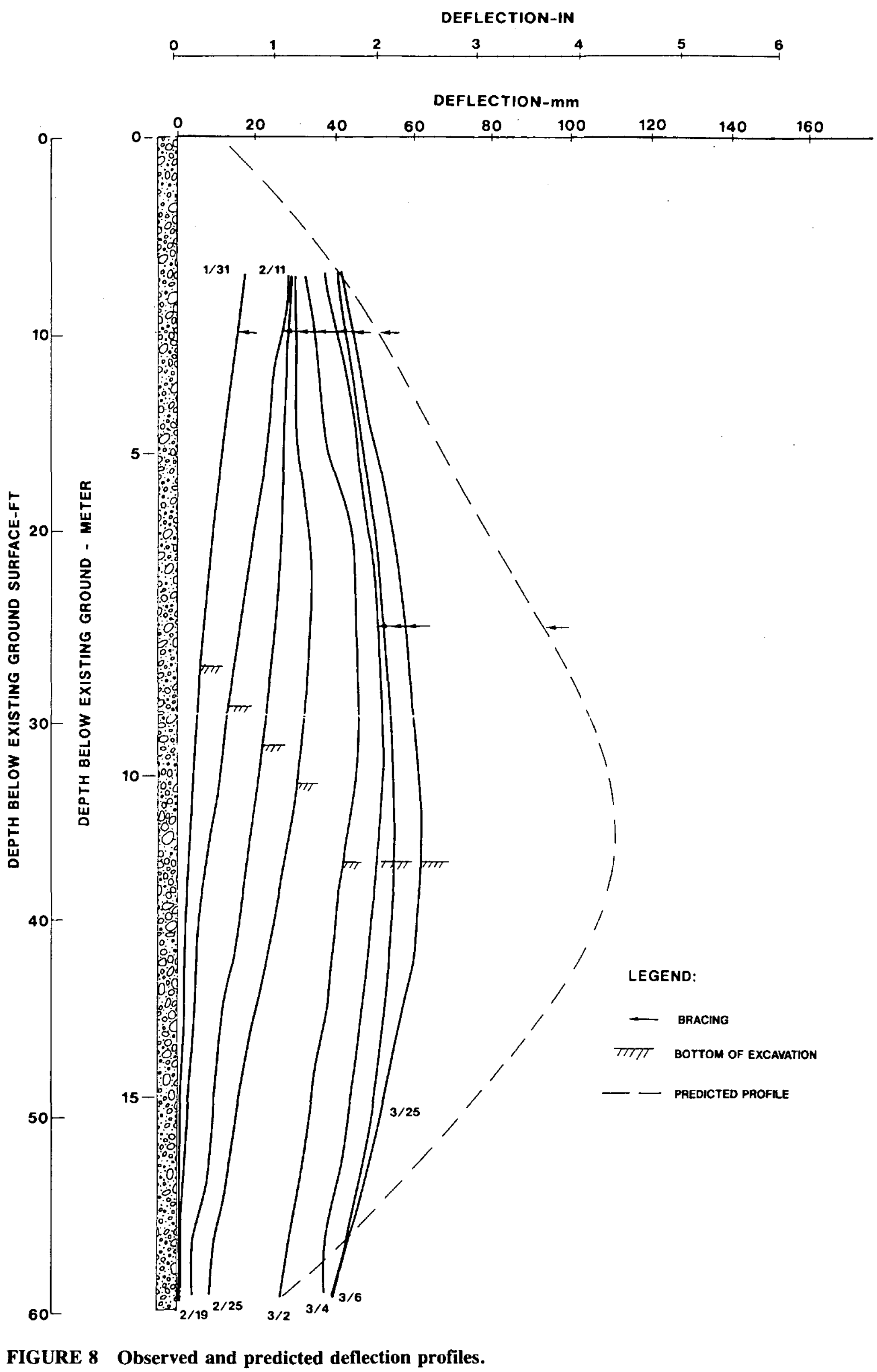

FIGURE 8 Observed and predicted deflection profiles.

plete excavation required two levels of bracing struts at respective depths of 3 m (10 ft) and 7.6 m (25 ft). The cross bracings were modeled as elastic springs. For such a long slender section of each cross bracing, the buckling load controls the ultimate axial behavior of the tube. On the basis of the critical buckling load and the allowable stress for the steel tube section, the spring constant for each cross bracing was calculated to be 1.43×10^3KN/m (8.18×10^3 lb/in.) per 0.3 m (1 ft) width of the wall.

Figure 7 presents the preconstruction prediction of the wall movement under the complete excavation condition. The wall was predicted to move laterally toward the excavation from the top to the toe, with the largest deflection occurring at the base of the excavation. The predicted deflection profile was used as a baseline for the movement criteria for the construction phase.

OBSERVED BEHAVIOR

During excavation, the DSM wall system behaved generally as expected. The DSM wall provided a complete cutoff of groundwater flow into the excavation except at one location where one DSM column had not been constructed in complete continuity and was subsequently repaired by in-place grouting. The cross-bracing loads were not measured in this study. Figure 8 shows the progressive movements of the DSM wall at various stages of construction. The deflection profiles were obtained from the Inclinometer No. 118 shown in Figure 1. Inclinometer No. 118 was selected because more data had been collected at this location. Because of construction and early termination of the monitor program, other inclinometers were not used for comparison purposes because of insufficient data. However, similar performance was observed for other inclinometers where readings could be obtained in the early stage of construction.

The preconstruction deflection prediction plotted in Figure 8 shows that the observed behavior of the DSM wall approaches toward a final profile similar to the predicted profile with the same order of magnitude in actual deflections. The predicted maximum deflections were larger than those measured; this may be due to the conservative selection of soil parameters in the active zone, which is usually done in practice. However, the measured deflection magnitudes are gradually increasing with time. This phenomenon appears to be consistent with the comments by other researchers (*16,17*) that the decrease of soil strength due to the long-term drained condition of clay soil would be gradually realized with time.

CONCLUSIONS

By analyzing and measuring the behavior of a braced DSM wall as a structural retaining wall, the following conclusions may be drawn:

1. A DSM wall successfully served as a groundwater cutoff and retaining wall at a restricted site where dewatering and driving vibration needed to be minimized.

2. Wall performance can be practically predicted before construction using beam-column analysis with appropriate nonlinear q-w curves so that baseline information can be established for subsequent monitoring activities.

3. In this particular case, the bending stiffness of the wall system was controlled by the reinforcing steel, and the behavior of the cross bracing was governed by the critical buckling load.

4. The short-term behavior of the DSM wall agreed reasonably well with the preconstruction prediction using short-term soil parameters. Progressive movement of the wall indicated that the conversion of drained shear strength from undrained shear strength of cohesive soil was gradually taking place.

REFERENCES

1. B. B. Broms and P. Boman. Lime Columns—A New Foundation Method. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 5, No. 4, 1977, pp. 539–56.
2. B. H. Jasperse and C. R. Ryan. Geotech Import: Deep Soil Mixing. *Civil Engineering*, Dec. 1987, pp. 66–68.
3. P. Hann. Taipei Untangles Its Rail Lines. *Engineering News-Record*, Feb. 1986, pp. 41–42.
4. R. Lundgren. Discussion: Deep Oil Mixing at Jackson Lake Dam. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 117, No. 12, 1991, pp. 1975–76.
5. O. Taki and W. Fillmore. Discussion: Deep Soil Mixing at Jackson Lake Dam. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 117, No. 12, 1991, pp. 1976–78.
6. C. R. Ryan and B. H. Jasperse. Discussion: Deep Soil Mixing at Jackson Lake Dam. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 117, No. 12, 1991, pp. 1978–79.
7. Weak Seabed Strata Yields Strong Support. *Engineering News-Record*, March 1983, pp. 30–31.
8. S. Sawyer. *Site Demonstration Report—In Situ Stabilization/Solidification*, Hialeah, Florida. EERU Contract 68-03-3255. Environmental Protection Agency, Cincinnati, Ohio, Nov. 1988.
9. C. R. Ryan and B. H. Jasperse. Deep Soil Mixing at Jackson Lake Dam. *Proc., Foundation Engineering Congress*, Vol. 1, June 1989, pp. 354–67.
10. H. Matlock, D. Bogard, and I. P. Lam. *BMCOL76: A Computer Program for the Analysis of Beam-Column Under Static Axial and Lateral Loading*. University of Texas, Austin; Ertec, Inc., June 1981.
11. M. Hetenyi. *Beam on Elastic Foundation*. University of Michigan Press, Ann Arbor, 1946.
12. G. B. Sowers and G. F. Sowers. *Introductory Soil Mechanics and Foundations*. MacMillan Publishing Co., Inc., New York, 1970.
13. *Engineering and Design, Retaining Walls*. EM 1110-2-2502. U.S. Army Corps of Engineers. 1975.
14. T. A. Haliburton. *Soil Structure Interaction*. Technical Publication 14. School of Civil Engineering, Oklahoma State University, 1971.
15. C. E. Williams and J. A. Focht III. Long-Term Performance of Cantilevered Retaining Walls. *Proc., Spring Texas Section Meeting*, Fort Worth, Tex., 1987.
16. C. E. Williams and J. E. Baka. Predicted and Measured Performance of Cantilevered Retention System, *Proc., Fall Texas Section Meeting*, El Paso, Tex., 1985.
17. D. E. Daniel and R. E. Olson. Failure of an Anchored Bulkhead. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 108, No. 10, 1982, pp. 1318–1327.

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