Design and Performance of a Reinforced Embankment for Mohicanville Dike No. 2 in Ohio

J. M. DUNCAN, V. R. SCHAEFER, L. W. FRANKS, AND S. A. COLLINS

Mohicanville Dike No. 2 is a rim dike on the Mohicanville Reservoir in Wayne County, Ohio. Constructed on a weak peat and clay foundation, the dike failed during construction and was 22 ft below its design height of 28 ft. Of a number of alternatives considered for raising the dike to its design height, construction of a reinforced embankment afforded the best combination of cost and reliability. Finite element and conventional limit equilibrium analyses were conducted to determine the reinforcing required for stability of the embankment. To achieve the factor of safety required for design, a reinforcing force of 30,000 lb/ft was required. A heavy steel mat was selected for the reinforcing material. (Although the steel reinforcement will probably corrode in time, it is only needed for the first few years of the embankment's life; after the foundation gains strength through consolidation, the reinforcement will no longer be required for stability.) The embankment was instrumented to measure reinforcement forces, settlements, horizontal movements, and pore pressures. Instrumentation studies have shown that the performance of the embankment is fully acceptable and that finite element and slope stability analyses provide an effective means for designing reinforced embankments and for anticipating their performance.

Mohicanville Dike No. 2 is a rim dike on the Mohicanville Reservoir in Wayne County, Ohio. Originally constructed by the U. S. Army Corps of Engineers in 1936, the dike is 28 ft high and about 1,800 ft long. The dike suffered a number of failures during construction and could not be raised above 12 ft owing to the weakness of the peat and clay foundation at the site. Subsequent settlement reduced the height of the dike to about 6 ft, and it was maintained at this height—about 22 ft below design grade—until its reconstruction in 1984 and 1985.

Alternatives for raising the dike to its design height were evaluated in the late 1970s by Law Engineering Testing Company working for the Huntington District of the Corps of Engineers to determine the most feasible method *(1)*. Because of the weakness of the foundation soils, construction of a conventional embankment was infeasible, no matter how flat the slopes. The depth of the peat and clay was so great (about 60 ft) that excavation of the weak materials was not economically feasible. Displacement of the soft foundation soils was considered but was rejected because of the large quantities of fill required and the uncertain quality of the resulting structure. Use of a concrete flood wall was considered but rejected because of the poor foundation support. Eventually it was decided that the best alternative for raising the dike would be construction of a reinforced embankment.

Use of a reinforced embankment as a permanent water-retaining structure is not common and may be unprecedented in the United States. However, the infeasibility of other solutions made this design necessary at Mohicanville. Furthermore, although the structure is permanent, the reinforcement will only be needed to improve stability during the first 10 years of its life. After that, the foundation will have gained sufficient strength by consolidation that the embankment will be stable without reinforcement.

The use of reinforcement to improve embankment stability is fairly new, and design procedures are still being developed. For Mohicanville, both finite element analyses and conventional equilibrium slope stability analyses were performed. A complete set of limit equilibrium analyses were performed by Law Engineering Testing Co. *(1)*, and finite element analyses were performed by the U.S. Army Engineers Waterways Experiment Station (WES) *(2)* in conjunction with the senior author. Finite element analyses were used to estimate the force in the reinforcing and the horizontal and vertical movements of the embankment. The limit equilibrium analyses were used to evaluate the factor of safety with respect to shear failure through the embankment and its foundation and to determine the amount of reinforcement required for stability.

Because of the unusual design concept and the importance of the structure, the Huntington District installed a large number of instruments in the embankment and the foundation to confirm that forces in the reinforcement, movements of the embankment, and pore pressures in the foundation were within acceptable limits during and following construction. The information derived from these instrumentation studies has been used to monitor construction progress and assess the accuracy of the finite element analyses and stability analyses, as explained subsequently.

After construction, a second finite element analysis was performed by Schaefer and Duncan *(3)* for the Huntington District. The purpose of this new analysis was to more closely represent the actual field conditions at the instrumented sections, including two layers of reinforcement as actually installed at the Sta. 9+00 cross section and slightly different foundation and embankment strengths than had been used in

J. M. Duncan, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Va. 24061. V. R. Schaefer, Department of Civil Engineering, University of New Mexico, Albuquerque, N. Mex. 87131. L. W. Franks, Huntington District, U.S. Army Corps of Engineers, 502 8th St., Huntington, W. Va. 25701. S. A. Collins, Federal Energy Management Administration, 1040 Austin Ave., Atlanta, Ga. 30307.

the original analyses performed by WES. These changes were found to have only small effects on the calculated results. Both the original WES analyses and the more recent analyses performed by Schaefer and Duncan *(3)* are in good agreement with the field measurements, indicating that finite element analyses combined with conventional limit equilibrium analyses provide a suitable basis for design of reinforced embankments on weak foundations.

PROPERTIES OF THE DIKE AND FOUNDATION

Mohicanville Dike is located on a glaciated plateau of glacial till in a moraine belt. The site contains a peat bog that developed in postglacial kettle holes *(4)*. The foundation soils consist of peat overlying soft clay, as shown in Figure 1. Soil properties were assessed through field vane shear tests and laboratory triaxial, consolidation, permeability, compaction, and classification tests *(1)*. A summary of the test results is shown in Table 1.

The foundation clay ranges in thickness from 10 to 60 ft and varies across the site from a silty clay to an organic clay. The shear strength of the clay where it has not been loaded by the old dike is small, due to the small unit weight of the overlying peat deposits. Typical undrained shear strength values range from 300 to 1,000 lb/ft^2, as shown in Figure 1. Stress-strain characteristics of the clay are shown in Figure 2a, consolidation test results in Figure 3.

The peat varies from fibrous near the ground surface to amorphous in the lower portions of the deposit, and is 16 to 20 ft thick in the virgin state. Where it was compressed under the weight of the old embankment, the thickness of the peat was reduced to 11 to 15 ft. Typical undrained shear strengths for the peat ranged from about 150 lb/ft^2 in the virgin state to about 500 lb/ft^2 where it was compressed under the old embankment, as shown in Figure 1. Stress-strain characteristics of the peat are shown in Figure 2b, consolidation characteristics in Figure 3.

The permeability of the peat was assessed by performing field and laboratory permeability tests. In Figure 4 the results are compared with values for California peats obtained by Weber *(5)*. For the California and the Mohicanville peats, the variations of permeability with consolidation pressure are similar. The line shown in Figure 4 was used in the finite element analyses to represent the variation of permeability with effective stress.

TABLE 1 SUMMARY OF SOIL PROPERTIES

Soil Property	Foundation Clay	Peat	Embankment Fill
Unified Classification	CL, CH, and OH	Pt	CL
Dry Unit Weight, pcf	OH: 40 to 84 CL-CH: 60 to 91	10 to 36	113 to 120
Water Content, %	OH: 37 to 67 CL-CH: 28 to 65	280 to 540	15 to 18
Liquid Limit	28 to 80	--	27 to 57
Plastic Limit	16 to 37	--	17 to 21
Plasticity Index	14 to 43	--	10 to 37
Specific Gravity	2.61 to 2.80	1.50	2.70 to 2.80
% Finer than			
#4	100	--	73 to 100
#10	100	--	60 to 95
#40	96	--	40 to 90
#200	90 to 95	--	25 to 80
2 micron	10 to 40	--	10 to 20
Undrained Shear Strength, psf	400 to 1000	200 to 500	3000 to 6000
Ø', degrees	25 to 29	17 to 32	32
c', psf	0 to 500	200 to 400	200
Permeability, ft/yr	0.1 to 10	see Figure 4	0.1 to 1

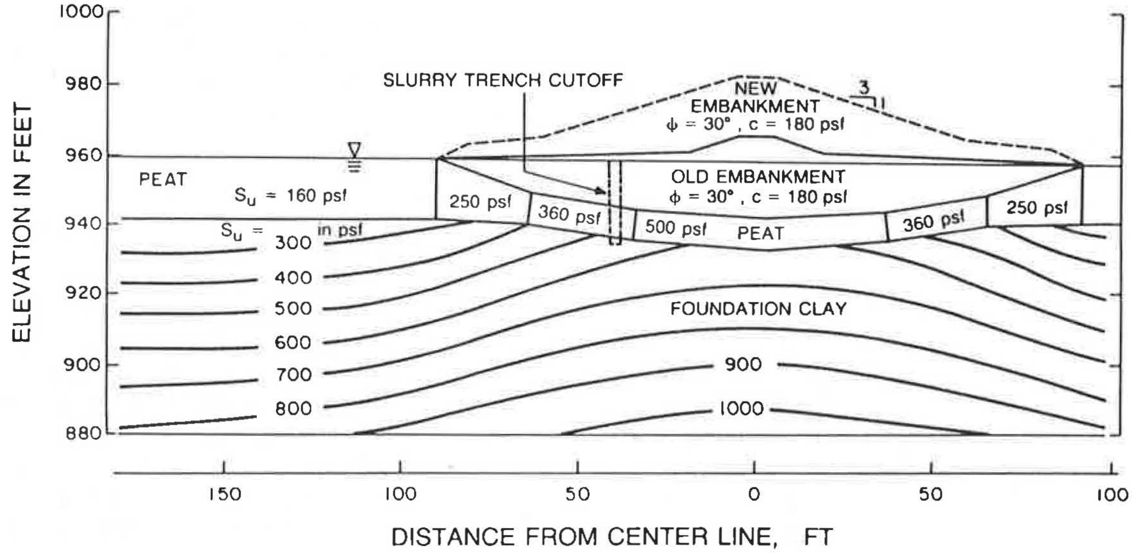

FIGURE 1 Typical cross section and undrained shear strengths [after Collins et al. *(1)*].

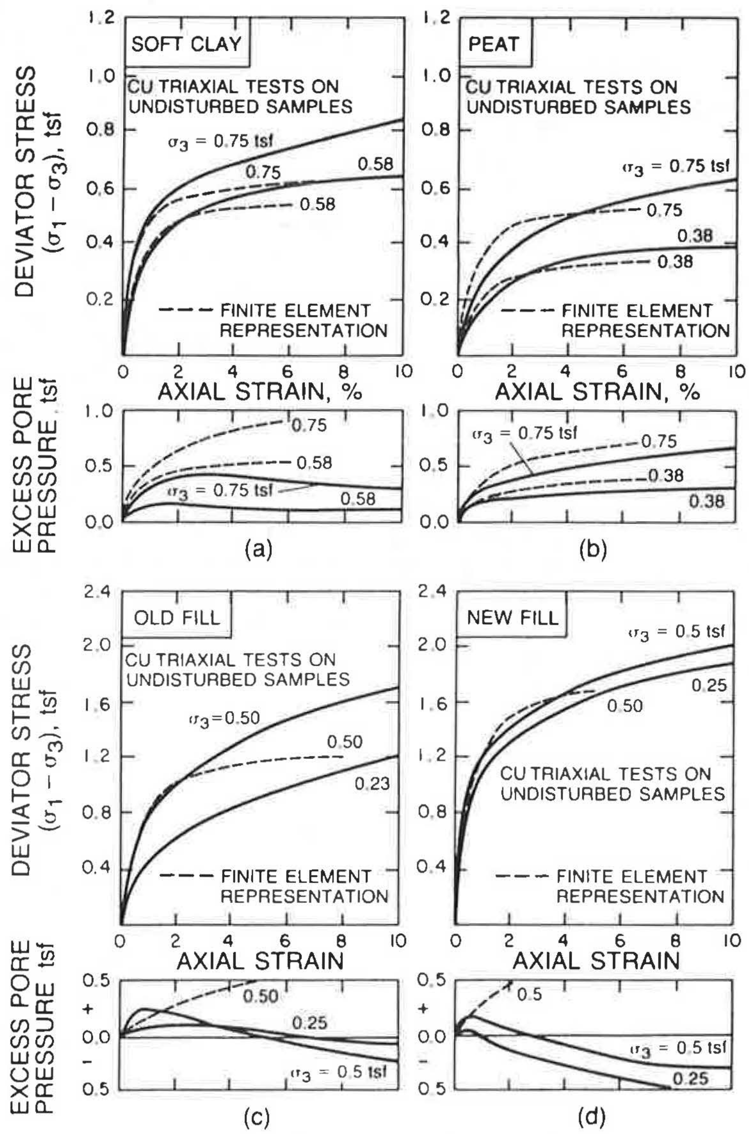

FIGURE 2 Stress-strain curves from consolidated undrained triaxial tests with pore pressure measurement.

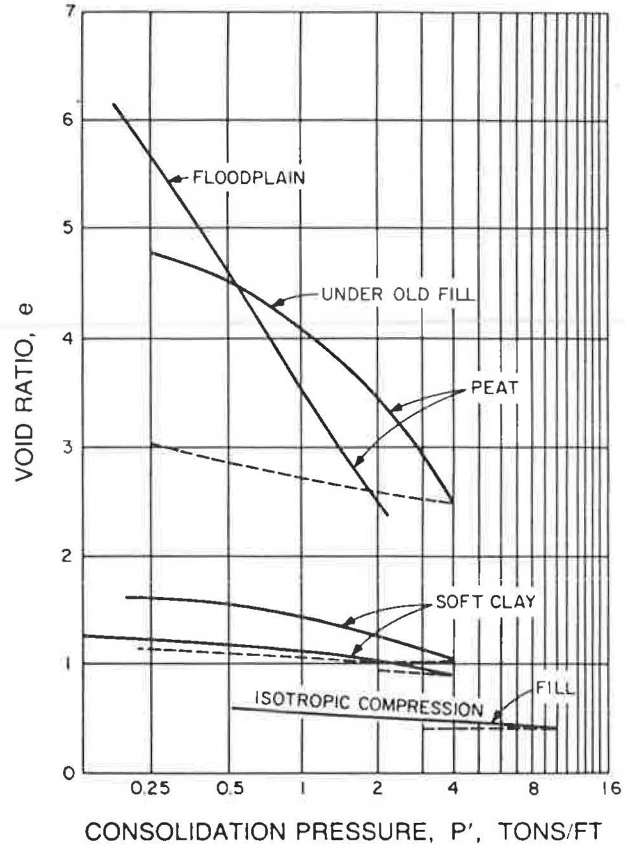

FIGURE 3 Consolidation curves for embankment and foundation soils.

Both the old and new embankment fill materials were derived from glacial tills in the surrounding uplands. The fill material grades as gravelly sandy clay with zones of gravelly clay. Pockets of poorly graded sand, silt, silty sand, and clayey gravel are also present. As it was compacted in the new embankment, the fill exhibits good shear strength characteristics and is quite ductile at the in-place water content of −2 to +2 percent with respect to optimum. The strength of the old fill is lower, especially in areas where previous failures occurred.

The stress-strain and strength characteristics of the old and new fill are shown in Figures 2c and 2d. The tests on the new fill were performed on block samples taken from the embankment after construction. The new fill exhibits higher strength and modulus than the old fill. Consolidation characteristics of the old fill are shown in Figure 3.

PROPERTIES OF THE REINFORCEMENT

The stability analyses performed for design by Law Engineering Testing Co. *(2)* indicated that a reinforcing force of about 30,000 lb per foot of embankment would be required to raise the factor of safety to a value of 1.3 at the end of construction, as required by Corps of Engineers design standards. The calculated distribution of reinforcing forces across the embankment required for a factor of safety equal to 1.3 is shown in Figure 5. The finite element analyses performed before construction *(2)* showed that a stiff reinforcing material would be required to achieve this amount of reinforcing force under working conditions.

To meet these requirements of stiffness and strength, a specially fabricated steel mesh was used. The mesh consists of No. 3 bars spaced 2 in. apart along the length of the dike, welded into a mesh with No. 2 bars parallel to the embankment axis, which are spaced on 6-in. centers. This mesh provides an ultimate reinforcement strength of 48,000 lb per foot of embankment.

The mesh was transported to the site in 8-ft-wide rolls and was unrolled at the site by the same machine used to roll it in the fabricating plant. When unrolled, the strips of mesh were cut into two pieces, each 8 ft wide and 160 ft long. These strips were dragged into position on the embankment with a bulldozer and end loader. The reinforcement extended 80 ft upstream and downstream from the centerline of the embankment between Station 3+00 and Station 14+00.

The reinforcing mat was placed at elevation 960 ft, approximately 4 ft above original ground elevation. In most areas, about 6 to 8 ft of old fill were excavated to reach this elevation before placement of the steel mesh. In one area where exceptionally large settlements occurred, additional fill had to be placed to increase the surface elevation to 960 ft before the steel mesh was placed. A second layer of reinforcing was placed at elevation 961 ft between Station 8+40 and Station 9+40 as an added precaution, because of the uncertain

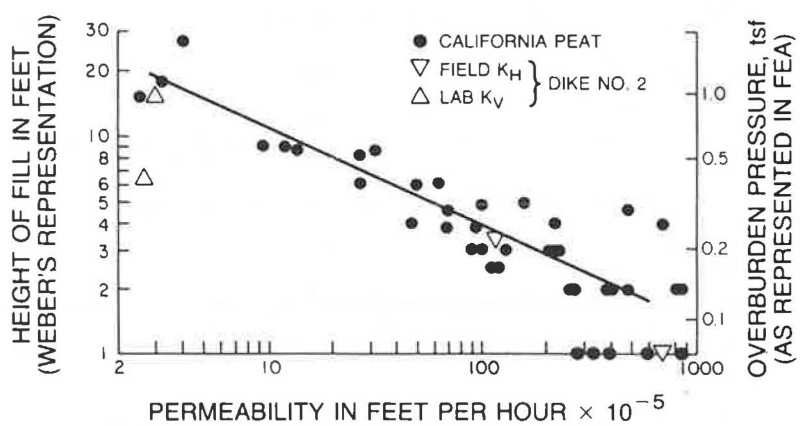

FIGURE 4 Variation of permeability with loading [after Collins et al. *(1)* and Weber *(5)*].

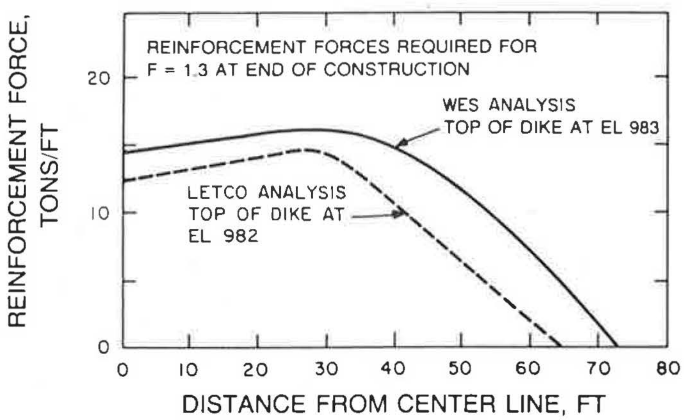

FIGURE 5 Distribution of required force from reinforcement to provide minim factor of safety of 1.3 [after Fowler et al. *(2)*].

foundation conditions in this area. Extensive failure occurred in this area in 1936 during the original construction, and a localized failure occurred in 1983 during construction of a slurry trench.

FIELD MEASUREMENTS

The dike, its foundation, and the reinforcement were instrumented extensively, as shown in Table 2. A total of 39 piezometers of three different types were used to measure foundation pore pressures. Some 13 inclinometers were installed to measure movements of the embankment and the foundation; 9 of these are vertical, and 4 are horizontal, placed just above the reinforcing mesh to determine settlements at this elevation. Also, 12 settlement plates were installed to measure vertical movements near the reinforcement level, and 25 surface monuments were installed to supplement the other measurements.

Strain gauges on the reinforcement provided a direct means for determining the force in the steel reinforcement throughout construction. Of 76 strain gauges installed on the reinforcing mesh, only 2 have failed, and the data appear to be consistent and reliable.

Reinforcement Forces

Values of reinforcement force measured at the embankment centerline at Stations 6+55 and 8+00 are shown in Figure 6, which covers the period from August to November, 1984. The force in the reinforcement increases as the embankment height increases. Between Points A and B in Figure 6, the reinforcing force increases approximately linearly with embankment height. In November, construction was halted for the winter and was not resumed until June 5, 1985. Placement of the first 3 or 4 ft of fill on the embankment after the winter shutdown induced little additional force in the steel. Aging of the recently compacted embankment fill over the winter may have caused the stiffness of the fill to increase sufficiently to affect its interaction with the reinforcement and the foundation. This possibility is currently under investigation through laboratory tests and finite element analysis. At more than 14 ft, as further fill was placed on the embankment the rate of increase of reinforcing force returned to that before the winter shutdown.

An interesting phenomenon is indicated by the data at Point C in Figure 6. At a constant embankment height of 19 ft, the reinforcement force increased by about 2 tons/ft over a period of 9 days. This increase in force is thought to be due to undrained creep in the foundation soils.

TABLE 2 INSTRUMENTATION AT MOHICANVILLE DIKE NO. 2

	Station						
Instrument	4+75	6+55	8+00	9+00	11+00	12+20	Total
Piezometers							
Open tube	1	3		4		1	9
Electric	1	2		3		1	7
Pneumatic	4	8		7		4	23
							39
Inclinometers							
Vertical	1	3		3	1	1	9
Horizontal	1	1		1		1	4
Strain gauges on steel	2	29	2	29 lower 10 upper	2	2	76
Settlement plates	3	3		3		3	12
Surface displacement monuments	5	5	5	5	5		25

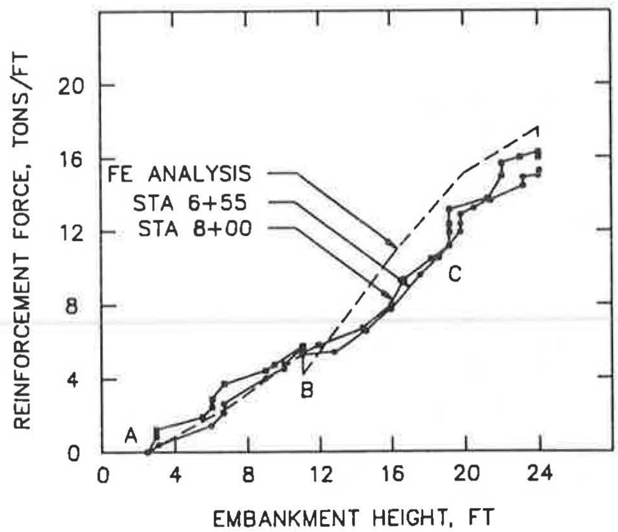

FIGURE 6 Centerline reinforcement forces versus embankment height for Stations 6+55 and 8+00.

Between Stations 8+40 and 9+40, two layers of reinforcement were used, spaced 1 ft apart vertically. This area is where the worst failures occurred during construction of the original embankment, where failure occurred in a wall of the slurry trench, and where the foundation conditions are most uncertain. The measurements of reinforcing force made in this area are shown in Figure 7. The lower layer of steel carries considerably greater force than the upper layer, although both have the same properties. The fraction of the total reinforcement force carried by the lower layer increases with increasing embankment height. After construction of the embankment was completed (Point *D* in Figure 7), the force in the lower layer of steel mesh increased slightly, whereas the force in the upper layer decreased by about 50 percent. Although many factors may be involved in this complex behavior, the most important factor appears to be that the effectiveness of embankment reinforcement is improved by placing it lower within the embankment, and the lower layer of steel mesh is thus in a position to be more effective than the upper layer.

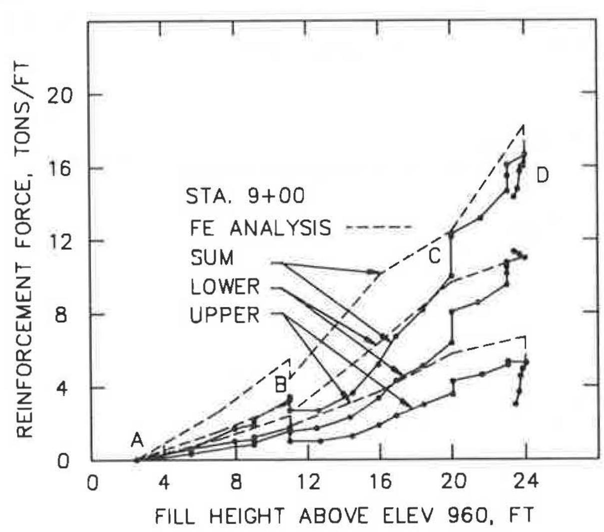

FIGURE 7 Centerline reinforcement forces versus embankment height for Station 9+00.

Distributions of the reinforcement forces across the embankment at Station 6+55 are shown in Figure 8 for various times during construction. Throughout construction, the maximum force occurs at the center of the embankment, as would be expected. In the downstream portion of the embankment, the variations of force with distance from the centerline are smooth and regular, indicating that the measurements probably contain little scatter. In the upstream portion of the embankment, the reinforcement forces are more erratic and are believed to be influenced by the slurry trench cutoff wall, which was located 45 ft upstream from the centerline.

Pore Pressures

Values of pore pressure measured during construction are shown in Figure 9 for piezometers in the peat and in Figure 10 for piezometers in the foundation clay. In both the peat and the clay, the increase in pore pressure is greater beneath the center of the embankment than for the piezometers located upstream and downstream. Both in the peat and in the clay, the response for piezometers downstream from the center is slightly greater than for piezometers the same distance upstream. This difference may be due to the effect of the slurry trench cutoff, which is restricting drainage of the foundation soils. The cutoff is located 45 ft upstream from the embankment centerline and probably restricts lateral migration of high pore pressures from the center of the embankment in the upstream direction.

The rate of increase of pore pressures beneath the center of the embankment during construction was approximately 1 ft of increase in pressure head for each 1 ft of increase in embankment height. Because the moist unit weight of the embankment fill is approximately twice the unit weight of water, this response corresponds to a value of the pore pressure ratio r_u, approximately equal to 0.5 (r_u = change in pore pressure divided by change in overburden pressure). The pore pressures both in the peat and in the clay decreased appreciably during the winter shutdown (Point *B* in Figures 9 and 10.)

Settlements and Horizontal Movements

Settlements measured by a horizontal inclinometer located at Station 6+55 are shown in Figure 11. The settlement at the beginning of the second construction season was largest at the downstream toe, where approximately 0.5 ft of settlement occurred. Subsequently, during the second construction season, and after the end of construction, the settlements near the center of the embankment were greater than those that occurred upstream and downstream.

The pattern of settlements shown in Figure 11 may be due to two separate influences, both of which tended to cause the downstream settlements to be larger than the upstream settlements. One is the influence of the old fill. The upstream portion of the old dike at this station was considerably thicker than the downstream portion. This greater fill thickness would have the

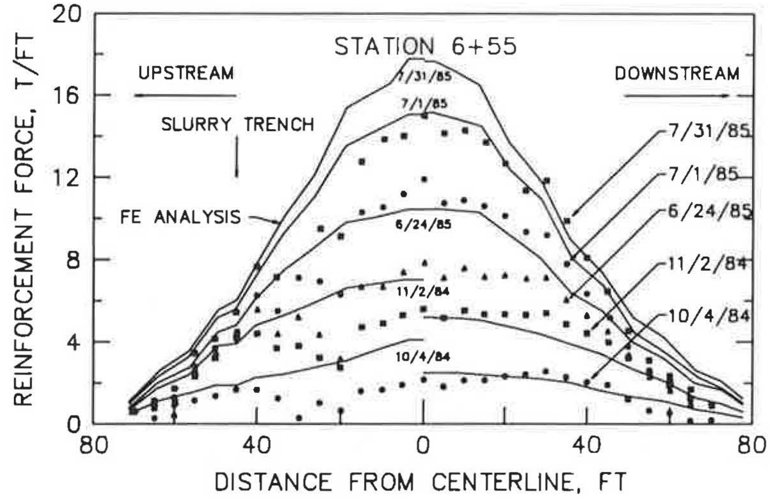

FIGURE 8 Reinforcement force distribution about the centerline for Station 6+55.

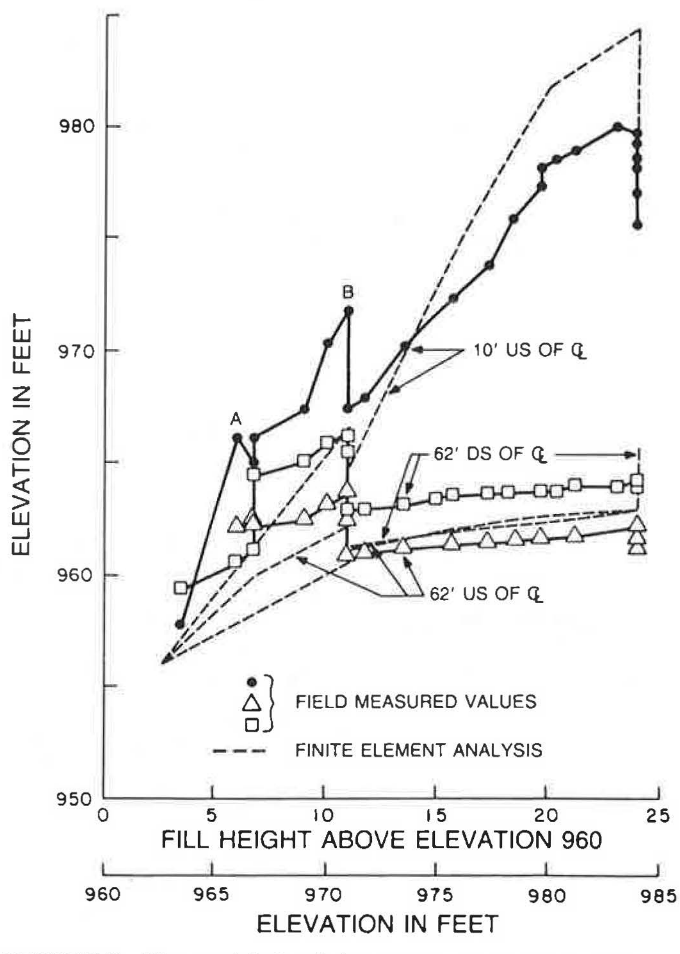

FIGURE 9 Piezometric levels in peat versus embankment height for Station 6+55.

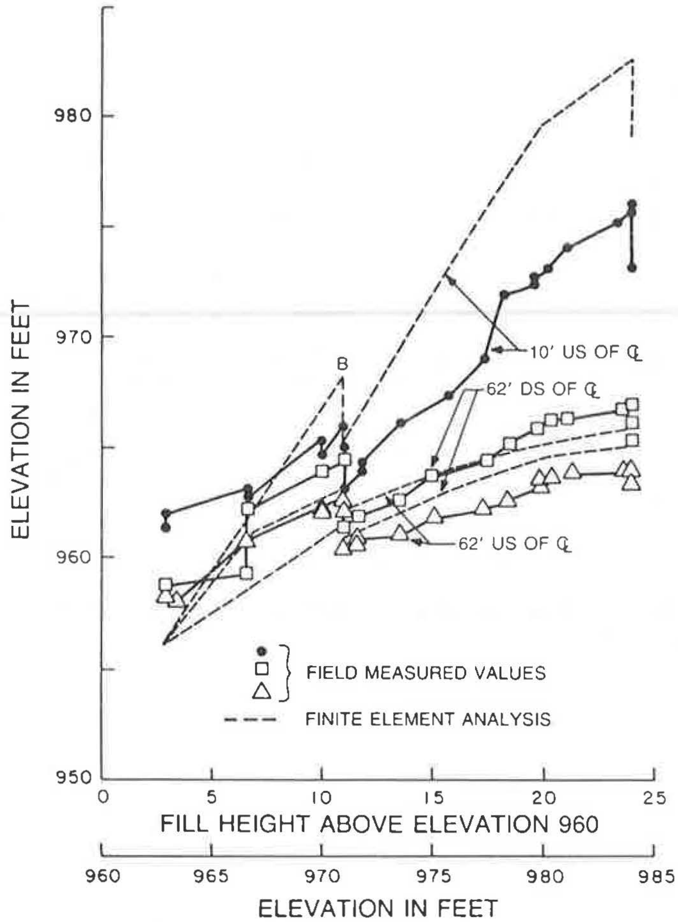

FIGURE 10 Piezometric levels in foundation clay versus embankment height for Station 6+55.

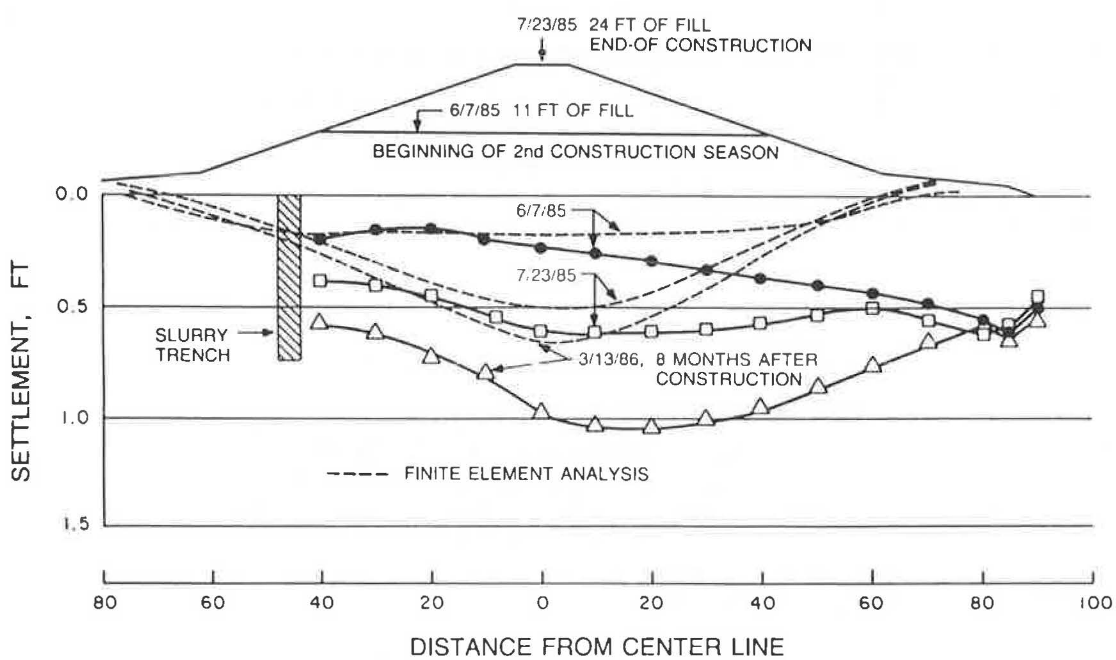

FIGURE 11 Settlement profile along Station 6+55.

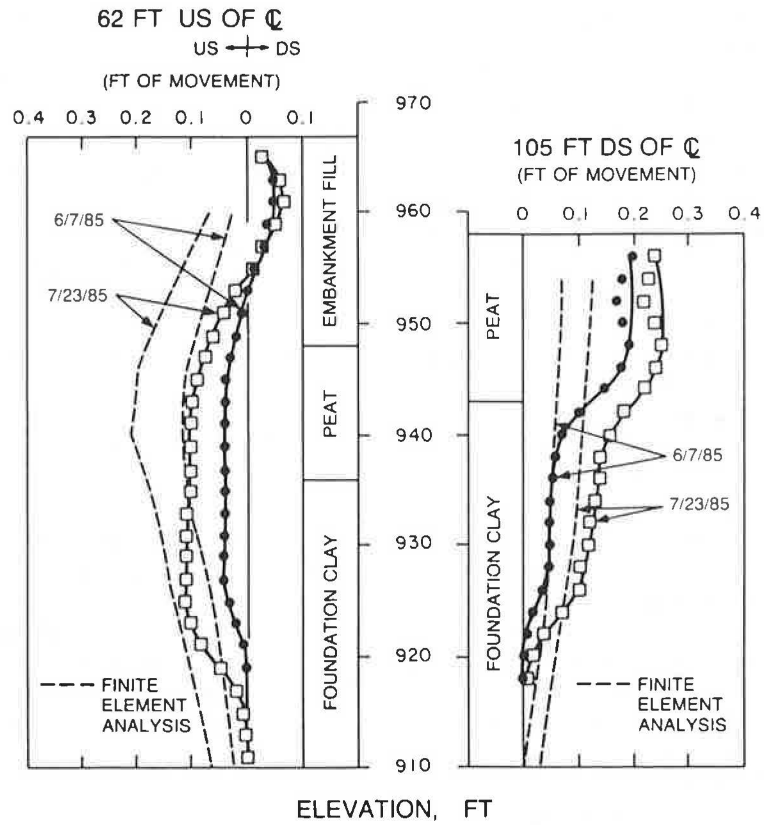

FIGURE 12 Movements of vertical inclinometers near embankment toes at Station 6+55.

effect of preconsolidating the upstream area more than the downstream area and would lead to smaller settlements upstream *(6)*.

A second influence is the slurry trench cutoff. This cutoff, by inhibiting drainage of the foundation soils in the upstream direction, may have effectively trapped pore pressures in the peat and clay beneath the upstream central portion of the embankment. As a result the settlements beneath the upstream portion of the embankment were smaller than those beneath the downstream portion. According to this explanation, the settlements upstream and downstream would become more nearly equal with time.

Horizontal movements measured near the upstream and downstream toes of the embankment at Station 6+55 are shown in Figure 12. The movements are small, the largest measured movement being less than 0.25 ft. In both cases, most of the shear deformations that give rise to the horizontal movements occur in the clay layer, within the depth interval between elevations of 915 and 930 ft. Also the movements at the level of the reinforcement are small, as expected. The measured movements at the elevation of the reinforcement are in fact less than 0.02 ft.

FINITE ELEMENT ANALYSES

As mentioned previously, finite element analyses were performed for design at WES *(2)*, and additional analyses were performed by Schaefer and Duncan *(3)* after construction, in connection with evaluation of the results of the instrumentation program. The same computer program was used in both analyses. The program, CON2D, was developed originally by Chang and Duncan *(7)* and was subsequently modified by Duncan et al. *(8)* and by Schaefer and Duncan *(2)*.

CON2D uses a modified form of the Cam Clay constitutive model and is capable of analyzing consolidation of soils as well as deformations under undrained and fully drained conditions. The stress-strain parameters used in the model can be evaluated using the results of conventional laboratory strength and consolidation tests. Reinforcing in embankments is modeled by bar elements with zero flexural stiffness, and slip between the reinforcement and the adjacent soils is modeled by special interface elements.

The degree of agreement between the actual stress-strain characteristics of the soil and those modeled by CON2D can be evaluated by using the parameters to calculate triaxial stress-strain curves, and comparing the computed stress-strain behavior with experimental results. Comparisons for the clay and peat from the foundation of Mohicanville Dike and for the old and new embankment fill are shown in Figure 2. The stress-strain curves are modeled accurately by the modified Cam Clay parameters used in the finite element analyses. The degree of agreement is not as good for the pore pressures, but the results shown in Figure 2 represent the best agreement that could be achieved within the constraints of the constitutive model. In selecting the parameters used in the analyses, emphasis was placed on matching the actual behavior of the soils in the small-strain range, because it was known that the actual strains would be small, provided that the design analyses were correct and the embankment remained stable. In the range of strains below 2 percent, the calculated pore pressure variations were in reasonable agreement with the measured values.

Results of the analyses performed by Schaefer and Duncan *(2)* are shown in Figures 6 through 12, together with measured values from the field instrumentation studies.

Reinforcement forces at the embankment centerline are shown in Figure 6. The calculated values are in good agreement with those measured during the first construction season. The calculated reduction in force during the winter shutdown due to consolidation of the foundation soils is considerably greater than the amount that actually occurred. After winter shutdown the calculated rate of increase in force was slightly greater than that measured. As explained previously, it is believed that the smaller rate of increase in reinforcement force after the winter shutdown may have been due to an increase in stiffness of the fill that resulted from aging during the shutdown period. This effect was not represented in the analyses presented here but will be considered in further studies.

Calculated and measured reinforcement forces at Station 9+00, where two layers of reinforcement were used, are shown in Figure 7. The calculated values are higher than the measured values through most of the embankment construction, and the difference is greatest during the first part of the second construction season. The calculated and measured values agree well in two respects. One is the decrease in reinforcing force during the shutdown period and during the period following construction. The other is the tendency for the upper layer of reinforcing to be less effective than the lower layer and for the force in the upper layer to decrease at a faster rate than that in the lower layer.

One aspect of the interaction between the reinforcing and the embankment was not modeled by the analyses. During a pause in the second construction season, the reinforcing force increased. As mentioned previously, this increase in force may have been due to undrained creep in the foundation. A similar increase near the end of construction may also have been due to creep effects. Because the modified Cam Clay model used in the analyses does not simulate creep effects, the finite element analyses do not simulate this aspect of the actual behavior. The field data indicate that after a period of 3 or 4 weeks the effects of creep die out, and the reinforcing force begins to decrease as the foundation soils consolidate.

Calculated distributions of reinforcing force across the embankment are compared to the measured values in Figure 8. The calculated distributions are discontinuous at the centerline because they were calculated by using two half-meshes rather than a whole mesh. It can be seen that the agreement is quite good overall, especially for the downstream half of the embankment. The behavior of the upstream half of the embankment was apparently affected to some degree by the slurry trench cutoff. Because the cutoff was represented in the analyses by only one element's width, its interaction with the foundation and the embankment and its effect on the behavior may not have been accurately reflected in the calculated results.

In Figures 9 and 10, calculated pore pressures are compared to the measured values. The calculated values are smaller than the measured values at the early stages and larger than the measured values at the later stages. These differences appear to be consistent with the differences between the calculated and measured laboratory test results, as shown in Figure 2 and discussed previously. The calculated amounts of decrease in pore pressure during shutdown periods and after construction are in good agreement with those measured, indicating that the consolidation characteristics of the foundation soils are reasonably accurately represented in the analyses.

The calculated settlements are compared with those measured in Figure 11. The agreement is not good at the early stages. As noted previously, the settlements at this time were strongly affected by the variations in preconsolidation pressure from upstream to downstream, and this detail of the initial conditions was not represented in the finite element analyses. The settlements that occurred during the second construction season and those that occurred after construction are in better agreement with the calculated values.

Calculated horizontal movements are compared with those measured near the toes of the embankment as shown in Figure 12. The calculated variations of horizontal movement with depth are much more uniform than the measured values. As mentioned previously, much of the measured lateral movement was caused by deformations of the soils between elevations of 915 and 930 ft, indicating existence of a soft or weak zone in this area. The fact that such a weak zone was not represented in the finite element analyses is probably responsible for the differences between the measured and the calculated horizontal movements.

CONCLUSIONS

The experience gained from design, construction, instrumentation, and analysis of the Mohicanville Dike has provided information of considerable value with regard to the behavior of reinforced embankments on weak peat and clay foundations.

The experience has shown that it is feasible to effectively stabilize an embankment on a weak foundation using a single layer of reinforcement near the base of the embankment. Because the Mohicanville Dike is 28 ft high, and the necessary improvement in safety factor was considerable (about 40 percent), the amount of reinforcing force required was large (about 30,000 lb per foot of length of embankment). A steel mat, specially fabricated for the job, proved to be an economical reinforcing material from the points of view of both initial cost and construction feasibility. Although the steel will probably corrode in time and its reinforcing capacity will decrease, the foundation soils will have consolidated and gained strength by that time, and the reinforcing will no longer be needed to maintain the stability of the embankment.

The instrumentation studies performed on the embankment during and following construction have provided valuable information regarding the accuracy of the finite element analyses and the limit equilibrium slope stability analyses used to design the embankment. Comparisons of the calculated and measured reinforcement forces indicate that the finite element analyses provide an effective means of estimating the amount of reinforcing force that would develop during construction and the rate at which the force would decrease after construction as the foundation soils consolidate. The measured movements of the embankments have been small, consistent with the expected behavior of an embankment that has a factor of safety equal to 1.3 at the end of construction. Thus the combination of finite element analyses to estimate reinforcing forces and conventional limit equilibrium analyses to calculate a factor of safety appears to provide an effective approach for design of reinforced embankments on weak foundations.

ACKNOWLEDGMENTS

Many people have contributed to the design, construction, and evaluation of the performance of Mohicanville Dike No. 2. The writers wish to specifically acknowledge the contributions of James Coffman and Thomas Plummer of the Huntington District; Jack Fowler, Roy Leach, John Peters, and Ray Horz of WES; and George Sowers and David Wheeless of Law Engineering Testing Company; without whose efforts the design, construction, and instrumentation studies would not have been possible.

REFERENCES

1. S. A. Collins, W. Rogers, and G. F. Sowers. *Report of Embankment Reanalysis—Mohicanville Dikes.* Huntington District, U.S. Army Engineers, Huntington, W.Va., July 1982.
2. J. Fowler, R. E. Leach, J. F. Peters, and R. C. Horz. *Mohicanville Reinforced Dike No. 2 Design Memorandum.* Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Sept. 1983.
3. V. R. Schaefer and J. M. Duncan. *Evaluation of the Behavior of Mohicanville Dike No. 2.* Huntington District, U.S. Army Corps of Engineers, Huntington, W.Va., Nov. 1986.
4. G. W. White. *Glacial Geology of Wayne County, Ohio.* Ohio Geological Survey Report 62, 1967.
5. W. G. Weber, Jr. Performance of Embankments Constructed over Peat. *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 95, No. SM9, Jan. 1969, pp. 53–77.
6. S. A. Collins. *Analysis of Instrumentation and Evaluation of Embankment Performance: Mohicanville Dike No. 2.* Huntington District, U.S. Army Corps of Engineers, Huntington, W.Va., June 1986.
7. C-S. Chang and J. M. Duncan. Analysis of Consolidation of Earth and Rockfill Dams. Report TE 77-3. U.S. Army Engineers Waterways Experiment Station, Vicksburg, Miss., Sept. 1977.
8. J. M. Duncan, T. B. D'Orazio, C-S. Chang, K. S. Wong, and L. Namiq. *CON2D: A Finite Element Computer Program for Analysis of Consolidation.* Report UCB/GT/81-10. College of Engineering, University of California, Berkeley, 1981.

Publication of this paper sponsored by Committee on Mechanics of Earth Masses and Layered Systems.