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INFLUENCE OF REINFORCEMENT MODULUS ON DESIGN AND CONSTRUCTION OF MOHICANVILLE DIKE No. 2

L'INFLUENCE DU MODULE DE RENFORCEMENT SUR DESSIN ET CONSTRUCTION DE LA DIGUE MOHICANVILLE No. 2

EINFLUSS DES GEOTEXTIL-VERFORMUNGSMODULS AUF ENTWURF UND BAU DES MOHICANVILLE-DAMMS Nr. 2

To raise the existing Mohicanville Dike No. 2 to required grade it was found necessary to use tensile reinforcement to limit lateral spreading. Conventional limit equilibrium and finite element analyses were used to determine reinforcement working load, placement of reinforcing layers, and reinforcement stiffness. It was found that a number of commercially available geotextiles would provide adequate strength to supply the working load. However, geotextiles would not be sufficiently stiff to take up the working load without excessive mobilization of foundation strength. Steel wire mesh was found to provide both adequate stiffness and strength required for the design. Field placement of the wire was found to be practical and construction has been successfully completed.

Um den bestehenden Mohicanville-Damm Nr. 2 zu efordernen Grad zu erheben, man fand notwendig, Spannungsstarkung zu benutzen, um seitliche Ausbreitung begrenzen zu konnen. Begrenztes ubliches Gleichgewicht und begrenzte Grundbestandteil (Element)(Faktor) wurden benutzt, um Verstarkungsmaximalbelastung, Ausstellung von verstarkten Schichten und verstarkende Festigekeit zu bestimmen. Man fand, dass einige kommerziel vorhandende Geotextilien genugende Starke verschaffen wurden, um die Wirkungsbelastung zu geben. Doch Geotextilien hatten nicht genugende Steifheit, die Maximalbelastung ohne ubermassige Mobilisierung des Grundkraftes zu absorbieren. Man fand, dass der Drahtnetz sowohl genugende anordnungserforderliche Steifheit als auch Starke verschaffen wurde. Man fand das Feldlagern des Drahtnetzes praktisch, und die Konstruction ist erfolgreich abgeschliessen worden.

INFLUENCE OF REINFORCEMENT MODULUS ON DESIGN AND CONSTRUCTION OF MOHICANVILLE DIKE NO. 2

Background

Mohicanville Dike No. 2 is a flood control dike constructed as part of the Mohicanville Dam and Reservoir project located in the Muskingon Watershed of Ohio (as shown in Figure 1) near Mohicanville, Ohio. Originally designed to the same elevation, el 998, (304 m) as the Mohicanville Dam, the dike was constructed in 1937 but continued foundation failures during construction in the peat and soft clay foundation finally led to a decision to stop construction. A record pool height and subsequent seepage through and/or under the dike alerted the Huntington District to the need to upgrade the flood control system from an existing crest, el 965, (294 m) to a proposed elevation of 983 (300 m). After extensive subsurface investigations and detailed analysis of slope stability and seepage, it was determined that no acceptable factor of safety could be obtained for raising the dike without some type of reinforcement. It was also decided that a finite element and conventional analysis be conducted to determine the type and amount of reinforcement needed and the construction requirements. The dike was completed to design height in August 1985.

Purpose

This paper discusses the design parameters, stability analyses, construction techniques, and expected behavior of the dike during and after construction. The purposes of the study was to: (1) design a 24 ft (7.3 m) high flood control reinforced earth dike on a soft foundation; (2) describe construction techniques for raising a dike on a soft foundation; and (3) to design an instrumentation monitoring system which provided information necessary for monitoring the dike safety during and after construction.

The Problem

The problem was that the foundation material of peat and clay were too soft to support the full height of the embankment. The reinforcement was designed to cause the embankment to behave like a semi-rigid body and settle vertically and uniformly with a minimum amount of horizontal displacement. During initial construction (1936), the embankment had been raised approximately 6 ft (1.8 m) above peat surface when longitudinal cracks appeared in the dike and foundation and a major shear failure occurred. Construction was stopped for a brief period of time but fill placement was eventually resumed. After about eight days the embankment and foundation began cracking, spreading, and displacing vertically and horizontally. During the next season (1937), an attempt was made to dewater the foundation, but when construction was resumed in May, cracks began to form and in August another major failure occurred in the same area. Construction was terminated at about 15 ft (4.6 m) above original grade. The crest height since 1937 subsided to approximately 7 ft (2.1 m) above the original ground surface.

Alternative construction methods such as displacement section construction and staged construction techniques were considered but were eliminated in favor of a reinforced embankment. The embankment was designed to an elevation that allows for anticipated consolidation of about 3 ft (0.9 m). During construction in 1936-37, borrow measurements indicated about 142,000 cubic yards (108,600 cubic meters) of fill material was placed while current cross sections exhibited only about 91,000 cubic yards (69,600 cubic meters) above the original ground. In the failed area (Station 9+00)

(2.7 m) near the center of the dike between each abutment, there was about 4 to 6 volumes of fill material below grade for each volume above grade. It was decided that conventional methods of construction would not be acceptable for the new dike. Staged construction without displacement and/or reinforcement would be a lengthy construction process because only about 2 to 3 ft (0.6 to 0.9 m) of embankment could be constructed before construction would have to be halted to allow dissipation of pore water pressure and a corresponding gain in foundation shear strength.

Design and construction techniques for building reinforced embankments on soft foundations is a relatively new area that was pioneered by the late Dr. Allan Haliburton and geotechnical engineers at the Waterways Experiment Station, Vicksburg, MS. Experience with using reinforcement is too limited to rely on conventional analysis for a relatively large and steep embankment whose failure could result in loss of life and property if the reservoir was lost. To supplement the conventional analyses, a finite element analysis was conducted to model behavior of the foundation, embankment, and reinforcement. The conventional limit equilibrium and finite element analyses were used as complementary design tools wherein the limit equilibrium method gave an estimate of stability and the finite element method was used to determine the proper location of the instrumentation used to monitor the safety and behavior of the embankment during and after construction.

Site Condition

Extensive subsurface investigations were conducted at various time segments: (1) prior to 1935 for the original design; (2) in 1970 for embankment reanalysis; and (3) in 1980 for embankment reanalysis and consideration for reinforcement. During the foundation investigation in 1980, a number of vane shear strength, triaxial shear strength, permeability, consolidation characteristics, water content-density relationships, Atterberg limits, grain size distribution, and visual classification tests were conducted. Also a number of falling head permeability tests were conducted and piezometers installed in the foundation. A typical cross-section of the dike and foundation materials used for design analysis is shown in Figure 1.

Embankment. The embankment was constructed from local material that consisted of glacial till composed of gravelly sandy clay (CL) with zones of gravelly clay sand (SC) and minor amounts of sand (SP), silt (ML), silty sand (SM), and clayey gravel (GC). Subsurface investigation indicated large deformations of the embankment and discontinuities in the embankment and foundation materials during original construction were responsible for the low shear strengths measured.

Peat. The layer of peat shown in Figure 1 generally varied from the upper portion being fibrous peat to the lower portion being amorphous peat. The original depth before construction in 1937 was about 16 to 20 ft (4.9 to 6.1 m). Undrained shear strengths used in the analysis for the peat layer are shown in Figure 1.

Foundation clay. The foundation clay layer extended to about 60 ft (18.3 m) below the peat layer and was classified as a medium plasticity silty clay (CL) with thin zones of high plasticity clay (CH) and organic clay (OL and OH). The shear strengths used in the analysis are shown in Figure 1.

Fabric-Reinforced Embankment Design

A fabric-reinforced embankment is subject to the same failure potential mechanisms as a conventional embankment on soft soils: (1) rotational slope/foundation failure; (2) foundation displacement; and (3) horizontal splitting and spreading. To prevent these potential failure mechanisms required large reinforcement forces and low strains. Forces calculated for the potential failure modes are resisted by reinforcement tensile strength while spreading displacements are controlled by reinforcement tensile modulus. Friction between the soil and fabric or reinforcement must be large enough to prevent sliding of the embankment on the reinforcement or the foundation. Proper construction techniques can be used to develop reinforcement tensile forces at relatively small strains. Design criteria for embankment design are still being refined but at present, the work by the authors and the late Dr. Haliburton is compiled as the state-of-the-art in a manual for the FHWA entitled "Use of Engineering Fabrics in Transportation-Related Applications," December 1984.

The reinforcement design is based primarily on the criteria that the reinforcement should provide enough resistance to provide a factor of safety ( ) of 1.3 against mobilization of the full resistance of the foundation soil. A conventional limit equilibrium analysis was used to determine the required reinforcement force for FS = 1.3. To achieve a factor of safety of 1.3, the reinforcement should be stiff enough to take up the additional force prior to excessive elongation of the reinforcement and subsequent excessive deformation of the embankment and foundation. The required stiffness was determined from a finite element analysis of the reinforced embankment. The finite element analyses also provided predictive values of embankment deformation, construction-induced pore pressure, and rates of consolidation.

Slope stability analysis. A conventional circular arc limit equilibrium slope stability analysis was conducted to determine the unbalanced moment without reinforcement. It was assumed that for the embankment to fail in the rotational mode, the reinforcement must tear or separate. To prevent failure, the reinforcement strength can be added to the resisting forces. This approach assumes the following: (1) reinforcement tensile strength and soil shear strength are mobilized simultaneously and (2) the critical failure location will be the same for the non-reinforced and the reinforced embankments. The Fellenius circular arc method of slope stability analysis was used to determine the minimum factor of safety with no reinforcement as shown in Figure 1. A factor of safety of 0.89 was calculated which was well below 1.3 which is recommended by the Corps of Engineers. The required resisting moment needed for a factor of safety of 1.3 was determined and the required tensile force needed for the reinforcement was determined from the following equation:

$$T_{1.0} = \frac{AM - \frac{AM}{FS}}{R} \qquad (1)$$

where

$T_{1.0}$ = required reinforcement tensile strength (working force) without factor of safety applied to reinforcement
AM = active moment
RM = resistive moment
FS = factor of safety against full soil strength mobilization
R = radius of critical arc, moment, arm

The maximum required tensile strength calculated by the conventional analysis was about 32.4 kips/lin ft (1050 kN/m). Assuming a factor of safety of 1.5 for the reinforcement, the maximum required tensile strength was calculated to be about 48.6 kips/lin ft (710 kN/m).

Finite Element Analysis

A finite element analysis was conducted to predict embankment performance using idealized subsurface conditions. The finite element analyses was primarily intended to supplement the conventional circular arc limit equilibrium computation and to compare field instrumentation measurements with predicted rates of consolidation, embankment deformation, pore pressure variations and tensile reinforcement force and pullout. The primary advantage of using the finite element analysis in design was the ability to rationally consider the influence of reinforcement modulus and/or stiffness and multi-layer configurations on shearing reinforcement forces (Figure 2).

One of the objectives of using the finite element analysis was to achieve compatibility with the conventional limit equilibrium analyses. As noted above, an assumption of the limit equilibrium analysis is that the tensile strength of the reinforcement and the shear strength of the soil is mobilized simultaneously. However, to achieve the required factor of safety of 1.3, the working force in the reinforcement must be obtained before 70 percent of the soil strength is mobilized. Therefore, the influence of reinforcement stiffness relative to that of the soil must be accounted for to ensure the desired factor of safety against full soil strength mobilization is achieved. The relationship between reinforcement stiffness and force is illustrated in Figure 2. Where it is seen that the required reinforcement working force is achieved only for stiffness comparable to the steel wire. Also shown on the figure are the stiffness versus strength values for various reinforcement materials. The analysis indicates that even though many fabrics have sufficient strength to meet design requirements their stiffness is too low to mobilize the required reinforcement force without excessive mobilization of foundation soil strength.

The finite element analysis was "calibrated" to the limit equilibrium analysis by adjusting the soil properties so that the calculated deformations and stress in the finite element analysis of an unreinforced section were as large as implied by the factor of safety of 0.89 determined from the limit equilibrium method. Once compatibility between the two methods is achieved then the reinforcement design was based on the following:

a. The reinforcement tensile force or working force determined by the limit equilibrium method (equation 1) at a factor of safety of 1.3 for the soil was about 32.4 kips/ft (1050 kN/m).

b. The finite element analysis was used to determine the reinforcement stiffness or modulus and layer configuration and location to achieve the desired working force.

In addition, the reinforcement type must exhibit high moduli, low elongation, low creep, high contractor survivability, high abrasion and puncture resistance, high ultra-violet resistance, high acid and basic resistance, high electrical resistivity, resistance to salts and phosphates, and must carry an ultimate tensile force of 1.5 times the working force or about 48.6 kips/ft (710 kN/m).

The procedure outlined above has helped to define and emphasize the advantages and disadvantages of both the limit equilibrium and finite element analysis. The limit equilibrium nalysis has considerable precedence over other design techniques and it can be used to calibrate the material properties selected for the finite element analysis. The primary advantage of the finite element analysis is that it can be used to determine the relationship between reinforcement force and modulus and/or stiffness but its accuracy is dependent on the material properties selected.

Reinforcement. The maximum required tensile reinforcement occurs at the point of maximum settlement of the dike which was at or near the embankment centerline. The maximum force predicted by the finite element analysis was 36 kips/lin ft (526 kN/m) for the end of construction. The required force should reduce to 20 kips/lin ft (292 kN/m) in 10 years after consolidation has occurred.

The gradient of tensile force per unit length of reinforcement had a maximum value of 0.3 ton/ft^2 (28.7 kN/m) at about 50 ft (15.2 m) from the centerline, Figure 3. This gradient is important because it is equal to the shear stress between the reinforcement and soil. It was determined from laboratory pullout tests that slippage between the reinforcement and fill material may occur if the reinforcement gradient exceeds this maximum value or if the maximum gradient occurs further from the centerline where the overburden stress is less. Ultimate slip resistance between the soil and steel wire was found to be adequate to resist complete pullout (Figure 3).

Settlement and lateral displacement. Settlement normally consists of two components: (1) immediate settlement related to shear strains during construction and (2) settlements related to volumetric changes caused by consolidation after construction. Vertical settlement predicted for the end of construction were predicted to be about 1.0 ft (0.3 m) and subsequent consolidation of about 2 ft (0.6 m) of which 0.4 ft (0.12 m) should occur in the first year after construction. The finite element analysis indicated that reinforcement has only moderate influence on immediate settlement and long-term consolidation is virtually unaffected by reinforcement stiffness.

The primary difference in vertical subsidence and lateral movement is that lateral spreading is greatly influenced by reinforcement stiffness. Reinforcement reduces the embankment spreading by providing stiffness at the base directly proportional to the amount of stretch in the reinforcement. A significant amount of lateral movement in the foundation below the reinforcement was predicted during construction with little or no lateral displacement after construction. The maximum amount of spreading at the embankment toe predicted using steel wire was about 1 inch.

Pore pressure. Excess pore pressure induced in the peat and clay layers during construction was predicted to increase at a maximum rate of 1.7 ft (0.5 m) of piezometer head for each foot or 0.3 m of fill. Maximum rate of consolidation was predicted along the embankment centerline with a pore pressure reduction of about two percent per month after construction. Excess pore pressures were not expected to dissipate beneath the outer portions of the embankment until a significant reduction in pore pressure was achieved near the embankment centerline.

Field Measurements

Field measurements were made of movements, pore pressures, and reinforcement force and comparisons were

made between measured and predicted values throughout construction. The measured values agreed well with the predictions based on the finite element analysis. Pore pressures were found to differ from predicted values by only a few percent under the embankment centerline. Field pore pressures were considerably less than predicted near the embankment toe; a trend that was also reflected in the settlement profile. It is believed that differences in observed and predicted values of pore pressure was primarily due to difficulties in defining the permeability of the soft peat near the embankment edge. The reinforcement force was accurately predicted by the analysis although several days were required after fill placement before the load was reflected in reinforcement force measurements. This time delay is probably due to time-dependent (creep) response of the soil not accounted for in the analysis.

Another important application of the finite element analysis was the assessment of the performance during construction based on revisions to the analysis. At one section of the dike pore pressure heads were found to be significantly higher than predicted by pre-construction analysis. A comparison between field conditions and those assumed in the analysis indicated that the difference in pore pressure was a result of the lower general ground surface at that section. The additional pore pressure head was approximately equal to the additional weight of fill needed at the section to achieve the required initial grade. Differences also were noted between actual subsurface conditions and conditions assumed in the analysis. A revised analysis was performed to better depict the actual field conditions. Close agreement was found between the revised analysis and measured performance. Importantly, the revised analysis revealed that construction could be completed without exceeding design values of reinforcement force.

Construction

Photographs of the reinforcement placement procedures are shown in Figures 4 to 6. The wire was transported to the site as rolls then straightened into flat sheets prior to placement. After placement, the wire was covered with approximately 2 ft (0.6 m) of lime-treated soil. Subsequent fill placement proceeded as for conventional embankment construction. Problems were encountered during initial phases of wire placement because of excessive curvature of the reinforcement mats. The curvature required overlapping the mats over 51 inches (1.3 m) at the ends to achieve complete coverage at embankment centerline. Improved quality control during fabrication eliminated the problem.

Conclusions

Embankment construction was terminated in November 1984 because of poor weather conditions that made it difficult to achieve proper compaction of the fill material. Embankment construction began in June 1985 and was completed July 1985. A construction report and analysis of instrumentation measurement will be prepared in FY86.

It is concluded that preliminary measurements of vertical and lateral movement, pore pressure, and tensile load in the reinforcement have not exceeded the predicted values. The field measurements agree well with results of the finite element analysis indicating that useful predictions of field performance can be obtained despite the complexity of the problem.

Acknowledgment

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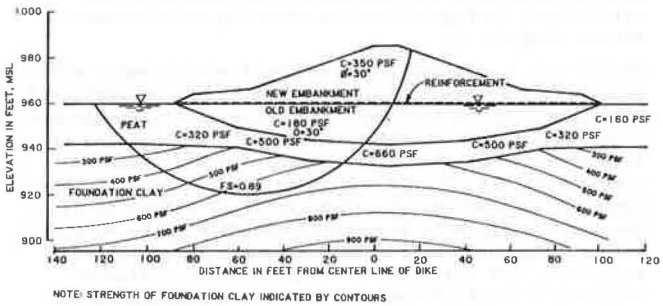

Figure 1. Mohicanville Dike #2. Limit Equilibrium Stability Analysis

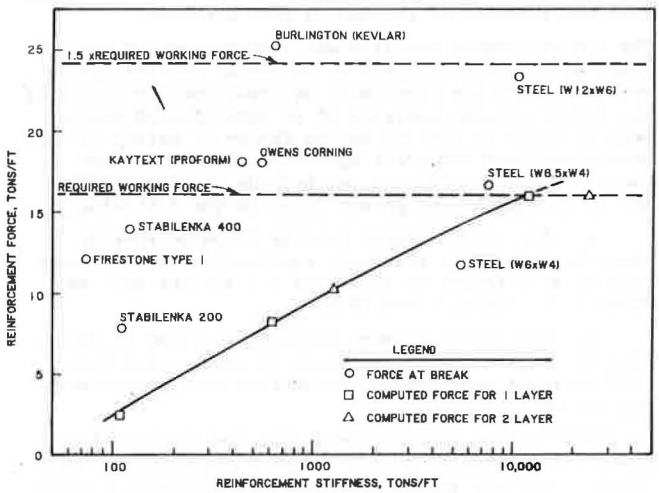

Figure 2. Reinforcement force and strength versus stiffness, K_s

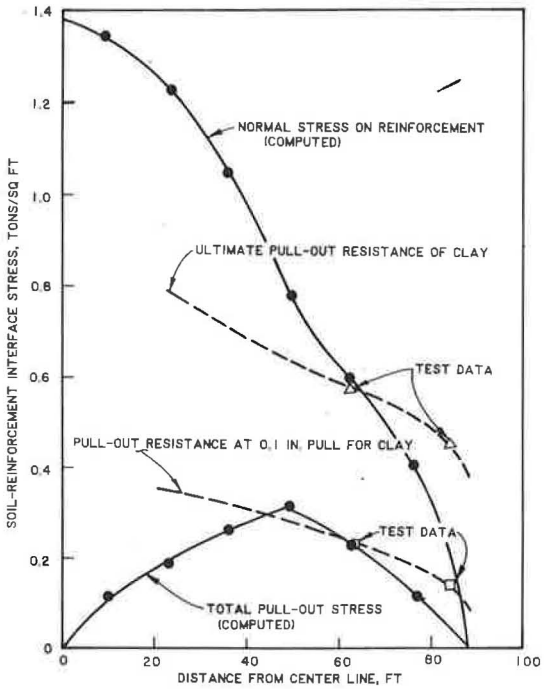

Figure 3. Total pullout force versus distance from centerline for steel reinforcement at end of construction

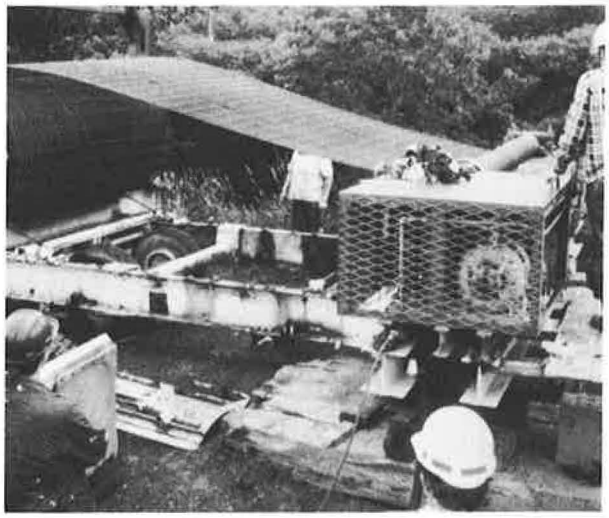

Figure 4. Welded steel wire being bent in straightening machine

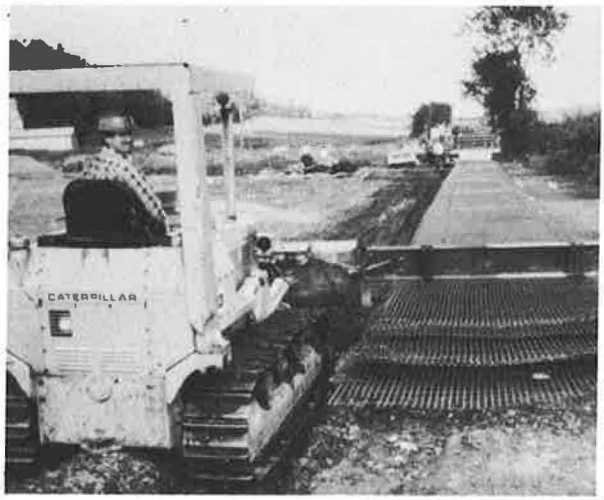

Figure 5. Welded steel wire being stacked on construction site

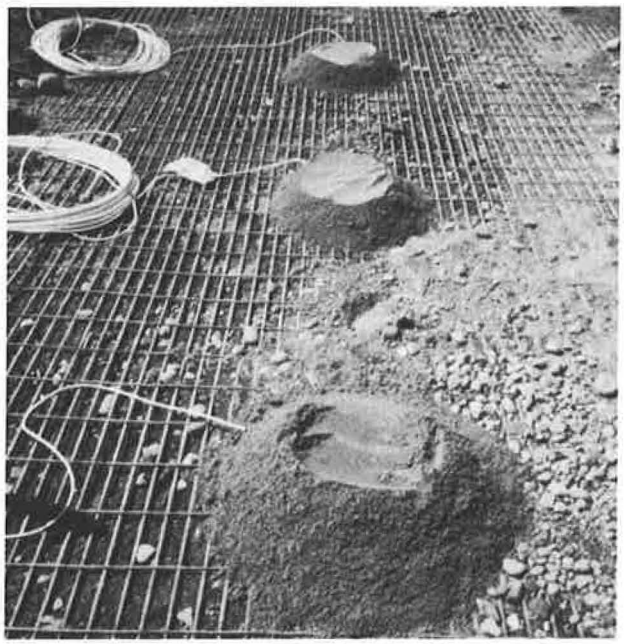

Figure 6. Strain gage locations being protected by sand piles prior to placement of sand blanket