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A Geotextile Reinforced Embankment for a Four Lane Divided Highway—
U.S. Hwy. 45, West Bend, Wisconsin

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SYNOPSIS: Geotextile reinforcement was used to construct an embankment for a four lane divided highway over up to 22 feet of low strength peat. The embankment had heights up to 7 feet. Special field testing and conventional laboratory tests were performed to measure the shear strength and compressibility. Stability analysis indicated that geotextile reinforcement could be used to construct a stable embankment on the peat deposit, provided the geotextile had sufficient strength to prevent rotational shear failure and to limit lateral deformation of the embankment. Construction of the embankment was begun in late summer of 1984. The highway opened for traffic in late 1985. Performance of the embankment was monitored during and after construction. The design, construction procedures, and results of the settlement monitoring program are presented.

BACKGROUND

The existing U.S. Highway 45, located south of West Bend, Wisconsin, had experienced a poor safety record. The Wisconsin Department of Transportation (WDOT) planned to construct a four lane divided by-pass around the city. The new alignment crossed the edge of Mud Lake, a filled glacial lake. The lake contained significant peat deposits.

Traditionally, the peat soils would have been displaced or excavated and replaced with granular fill. However, the following constraints precluded the conventional approach: 1) No disposal sites were available for the organic soils; 2) Since the area was considered a wetlands, there were environmental concerns related to excavating the organic soils; 3) The large volumes of soils involved would have resulted in large costs; 4) To allow direct observation of the foundation, expensive dewatering would have been required.

As a result of the constraints, the design engineers, J.C. Zimmerman Engineering Corporation, contacted their geotechnical engineering consultants, STS Consultants, Ltd., concerning the feasibility of constructing the embankment over the organic soils.

SOIL CONDITIONS

Borings performed by WDOT showed that the organic soils occurred over a span of 2300 feet. The peat occurred in two "basins" that were nearly separated by a ridge of inorganic soil. In the south basin the peats were up to 22 feet thick and in the north basin were up to 18 feet thick. Since the embankment crossed the west edge of the lake, the ground surface declined to the east and the organic soils became thicker to the east.

Three distinctive organic layers were encountered. The upper layer was a root mat having a thickness of zero to 2 feet. The second layer was a relatively decomposed fibrous peat ranging in thickness from 4 to 18 feet. The fibrous peat, which occurred at all locations that were explored, had losses on ignition ranging from 60 to 90% and water contents typically ranging from 100 to 1000%. The third layer, which was primarily encountered in the south basin, was an amorphous type sedimentary peat found below the fibrous peat. The sedimentary peat had ignition losses of less than 10% and water content ranging from 40 to approximately 250%.

The organic soils were underlain by inorganic silty and sandy soils that were generally in a loose to medium dense condition.

The strength of the organic soils was initially measured in the WDOT laboratory by means of numerous unconfined compression tests and by a single direct shear test.

In order to provide better definition of the shear strength of the organic soil, the geotechnical engineer undertook a program of field testing. This included both conventional vane shear tests and a new small-diameter plate bearing device. This device consisted of a 3 inch diameter disc that was connected to a smaller diameter push rod by means of a load cell. Due to the small diameter plate and the weak soils, it was possible to manually push the rod into the soil. By virtue of its location, the load cell ignored the effect of friction on the rod. The shear strength of the organic soil was calculated using the bearing capacity equation for "deep foundations".

Typical strength results from laboratory and field tests are shown on Figure No. 1. The data shown is from the south basin and represents all three organic layers. As shown on the figure, the strengths ranged from 30 to.
520 psf, with the lower strengths measured in the amorphous peat layer.

In the authors' opinion, the in-situ test results appeared to provide the most reliable shear strength for the organic material.

To evaluate embankment settlement numerous standard consolidation tests were performed in the WDOT laboratory. These tests indicated compression indices ranging from 0.1 to 5.0. The tests performed on fibrous peat had compression indices greater than 1.7. Tests on sedimentary peat indicated compression indices of 1.0 or less.

Additionally, several long term consolidation tests were performed to measure the secondary compression coefficient. The tests indicated secondary compression coefficients ranging from 0.009 to 0.029. The lowest coefficient was measured for sedimentary peat, while the higher values were measured on fibrous peat. The 1/3 order of magnitude variation in secondary compression coefficient between the two types of peat, indicated relatively uniform secondary compression behavior.

SOIL MODEL FOR STABILITY ANALYSES

A soil model was developed for each of the zones so that slope stability analyses could be performed.

For the purposes of this paper, only the analysis of the most critical zone will be reviewed. The soil model for this zone, illustrated on Figure No. 2, included fibrous peat occurring in a layer extending from 4 to 15 feet below the surface. This layer was thickest at the western end of the cross-section and the thickness decreased toward the east. On the basis of field testing, this layer was assigned a shear strength of 250 psf.

At the east end of the cross-section, the fibrous peat was underlain by sedimentary peat having a thickness of 15 feet near the extreme east end of the section. The thickness of this layer decreased to the west and the layer tapered out completely about 20 feet east of the embankment centerline. This sedimentary peat layer was assigned a shear strength of 100 psf based on the field test results.

The fibrous and sedimentary peats were underlain by a localized sandy sedimentary peat deposit that extended about 20 to 30 feet on either side of the embankment centerline. This layer was assigned a shear strength of 400 psf.

The organic soils were underlain by inorganic silts and sands which were assigned a friction angle of 26 degrees.

DESIGN REQUIREMENTS

An embankment having a maximum height of 7 feet was necessary to meet final, after settlement, grade requirements.

For the normal long term situation a factor of safety of 1.5 was required. Although somewhat conservative, it was felt that this factor of safety would result in a better long-term performance of the embankment. However, for a short term condition, such as initial fill placement, or the placement of surcharge fill, a factor of safety of 1.3 was considered adequate.

STABILITY ANALYSIS

The stability of the unreinforced embankment was first analyzed for bearing capacity and rotational shear stability using conventional geotechnical techniques. The slope stability analyses were made using the computer program STABL developed by Purdue University. This program calculates the factor of safety by the
The analyses employed the modified Bishop method which is applicable to circular failure surfaces and the simplified Janbu method applicable to failure surfaces of general shape.

The STABL program features unique random techniques for the generation of potential failure surfaces for subsequent determination of the more critical factors of safety. Typically, 50 to 100 potential failure surfaces were analyzed for each case. Both circular arc and sliding block surfaces were considered in the analyses.

For normal weight fill, factors of safety ranged from 0.72 for 4:1 embankment side slopes, to 0.87 for 8:1 side slopes. Both values are well below the desired factor of safety. The critical result of the slope stability analysis was checked manually using the method of slices. The manually calculated factors agreed within 0.1, which was considered good agreement.

For the analysis, it was apparent that the embankment could not be constructed without some form of subgrade or embankment modification. Since excavation and replacement was not a viable alternative, other methods including the use of wick drains, stone columns, light weight fill, piles, as well as soil reinforcement were considered. After a comparison of the methods with respect to feasibility, performance and cost, it was apparent that the use of geosynthetics was the most effective alternative. The use of metallic reinforcement was excluded due to the high corrosion potential in the acidic organic soils and relatively high cost.

**EMBANKMENT REINFORCEMENT**

By placing high tensile strength reinforcement at the base of the embankment, the stability of the embankment could be improved through increased shear resistance offered by the reinforcement. In addition, the reinforcement theoretically provided additional stiffness to the base of the embankment, allowing for a more uniform distribution of embankment loads. As the reinforcement should also reduce shear stresses at the embankment subgrade interface, it aided in reducing the potential for lateral spreading of the embankment over the weak subgrade.

Other reasons for using geosynthetic reinforcement included:

1. Allowing for initial support of vehicles out over the soft soil deposits, so that fill could be placed.
2. Providing for more controlled construction, less disturbance, and less displacement of the organic soil during construction.
3. Preventing the embankment from penetrating downward into the soft subgrade.
4. Maintaining the integrity and uniformity of the embankment construction. The reinforcement was not anticipated to reduce settlement of the embankment but was assumed to provide for a more uniform settlement. As such, the geosynthetic was anticipated to reduce differential settlement at points of transition in organic soil thickness.

**REINFORCEMENT REQUIREMENTS**

Analyses were carried out in order to determine the strength of the reinforcement necessary to geotextile the embankment to be constructed to its full height. The calculation method utilized the critical failure surface from the slope stability analysis of the unreinforced section to determine the resisting moments required to raise the factor of safety above 1.3 at the end of construction. This additional resisting moment was then assumed to be developed by the reinforcing element. The strength required of the reinforcing and its location was then determined analytically. The long-term factor of safety was met through post-construction (consolidation) shear strength gains in the subgrade.

Several methods of analyzing the required strength of the geosynthetic were used as summarized in the FHWA Geotextile Engineering Manual, (Christopher and Holtz, 1985). The methods included those proposed by Fowler, 1980 and Wager, 1981. The Fowler method assumes that the reinforcement is placed in tension by alignment tangent to the failure surface such that the resisting moment provided by the geotextile is equal to the radius of the circle times the allowable strength in the reinforcement. Please refer to Figure No. 3. The increase in resisting force is defined by the following equation:

\[
M_r = TR
\]  

The Wager method uses a vector approach which accounts for soil-fabric interaction plus the strength of the textile. The Wager method allows for the soil-fabric friction by adding the geotextile tensile strength \(T\) times the height \(Y\) of the radius point of the slip circle above the fabric, to the textile strength times the horizontal component of the
radius (X) times the tangent of the embankment fill friction angle (Ø). See Figure No. 3. Therefore, the increase in resisting moment (Mₚ) provided by the geosynthetic is defined by the following equation:

\[ Mₚ = T X + T X \tan \theta \]  

(2)

For highly deformable soils such as peat, it was our opinion that the Wager method provided a realistic model and was used for the final selection. For other less deformable soil conditions, the Fowler and Wager methods may be non-conservative (Bonaparte and Christopher, 1987).

The geosynthetic reinforcement analysis indicated that at the most critical location, a total reinforcement tensile strength on the order of 1500 lb/ft would be required. Even though this is a relatively high geosynthetic strength requirement, it could be easily be met by commercially available products if several layers of geosynthetic were used. By considering several layers, other efficiencies could be gained as strength requirements were neither uniform across the site nor along the alignment.

Several items were required to assure compatibility of the multiple layers. Firstly, the reinforcement layers were separated by a minimum of one foot of granular soil such that maximum soil-fabric friction could be achieved by each layer. Second, characteristics were required of each layer such that strength requirements were achieved at compatible strains. Finally, a re-analysis was made to verify the strength requirements for each successive layer.

The reinforcing requirements were also evaluated with respect to the ability of the reinforcement to limit lateral movement of the embankment. An analysis was made to determine the factor of safety against the embankment fill sliding laterally on top of the reinforcing material. A soil-fabric friction angle of 25° was required. An analysis was then made to determine the strength of the reinforcing required to resist substantial lateral movement. The force to be resisted was assumed to be the force resulting from the active lateral pressure at the base of the embankment with an applied factor of safety of 1.5.

As substantial movement was anticipated along the alignment of the embankment during construction, the amount of lateral spreading analysis was also used to determine the required geotextile strength in that direction.

A limiting design strain was then established to control the lateral and longitudinal movement at the design strength requirements. A limiting strain of less than 10% was selected to reduce the potential for tension cracking in the embankment following construction (5% induced strain was assumed during construction). Geotextile requirements will be detailed in a later section.

**TOE BERM REQUIREMENTS**

The reinforced embankment was then checked for overall bearing capacity failure. Since the reinforcement was designed to prevent local shear failure, the stresses at the base of the embankment could then be assumed to be distributed more over the full width of the embankment. A classical (Prandtl) analysis averaging the strength of all soils within the classical failure zone indicated a factor of safety in excess of 1.5. However, it is unlikely that an embankment that is wide relative to the thickness of the underlying soft layer would fail in this mode. A more probable mode of failure would involve the lateral squeezing of soils from beneath the embankment. An elastic shear stress versus shear strength analysis (Jurgenson, Boston Society of Civil Engineers, 1934) at the edge of the embankment indicated an unsafe condition in the lower strength subgrade area (factor of safety approximately 1). Passive pressure and shear resistance analysis indicated higher factors of safety. Due to the possibility of low factors of safety in these areas, special construction procedures were recommended to increase stability, including the use of a berm at the toe of the embankment in those sections to provide additional lateral resistance and the construction of a berm prior to to construction of the embankment to contain soil and prevent it from squeezing laterally. These construction techniques are typically referred to as mud wave construction techniques and will be reviewed further in the construction details section.

**EMBANKMENT SETTLEMENT**

The settlement of the embankment was calculated two ways using the results of the laboratory consolidation tests. The first way involved a conventional consolidation theory using the compression index measured by the conventional laboratory consolidation tests. Using this method, extremely large settlements were predicted. In more cases, the predicted settlement exceeded the height of the embankment. While it is certainly possible for the settlement to exceed the height of fill placed, this was not judged to be likely based upon past experience.

The second method of predicting settlement was more simplistic. This method used the results of the long-term consolidation tests that were performed in the laboratory. In this method, the predicted settlement was equated to the compression measured in the laboratory under a similar pressure times the ratio of the thickness of the compressing soil in the field to the laboratory sample thickness. The results of three long-term consolidation tests performed at a constant load increment of 1000 psf, the maximum pressure expected to result from embankment construction, were used for this analysis. The pertinent compression was taken to be that occurring at the completion of full primary consolidation.
Based upon this analysis the calculated primary settlement, under an embankment load of 1000 psf and assuming a 20 foot peat thickness, ranged from 8 inches to 28 inches.

In addition to the primary consolidation discussed above, secondary compression of the peat was anticipated. The secondary compression was computed, based upon the coefficient of secondary compression determined from the long-term consolidation tests.

The predicted total settlement including the primary consolidation and the secondary compression is summarized on Figure No. 4. The wide band of settlement resulted from variations in laboratory consolidation test results.

The noted settlement was for the conditions of 20 feet of compressing soil and an embankment load corresponding to 1000 psf. Where the embankment was lower or where the thickness of the organic soil was less, proportionately less total settlement was anticipated.

GEOTEXTILE SPECIFICATIONS

The specifications that were prepared for the project are summarized on Table No. 1 which follows:

Each fabric roll was required to be marked showing the type of fabric upon delivery to the field. Two (2) copies of the mill certificates for the geotextile were required to be provided with each shipment of the fabric.

Testing by an independent agency was specified to confirm the design parameters. A complete design parameter test series was required for the first shipment to the site. Additional sets of strength and modulus parameters were required for each additional 10,000 square yards used on the project.

Since the geotextile contributed significantly to the strength of the embankment, seams were allowed only in the transverse direction, the direction in which the shearing stresses were lower. The seams were required to develop the specified strength of the geotextile in the cross direction. The seams were required to be sewn with thread having equal or greater strength and durability as the material of the geotextile. The seams were specified to be double sewn with parallel stitching approximately 1/2 inch apart. Chain-lock seams were required to reduce the potential for unraveling. The sewn-fold was required to be placed on the upper surface of the geotextile to facilitate observation of the seams.

CONSTRUCTION DETAILS

The following items briefly highlight major construction procedures:

- A well-graded granular fill was specified to facilitate placement and compaction. It was also felt that granular fill would be more tolerant of the anticipated settlement.
- The bottom 1.5 feet of fill was specified to contain less than 5% fines in order to function as a drainage layer.
- Side slopes were specified to be 4 (horizontal) : 1 (vertical) or flatter. A 10 foot wide toe berm was required for embankment heights greater than 10 feet.
- The lower geotextile reinforcement layer extended across the full embankment width. The upper geotextile extended 10 feet beyond the embankment crests.
- The east half of the embankment was filled only to half height during the first construction season. Thus, it served as a temporary berm for the higher western half of the embankment.
- The temporary berm allowed a surcharge to be placed on the western half of the embankment to accelerate settlement.
A minimum of 3 feet of separation between the peat and the pavement subgrade was required.

Felled trees were left in place to create a "sacrificial" geotextile directly above the felled trees. Drainage fill was placed on the "sacrificial" fabric to provide a working platform.

The contractor opted to place a low strength "sacrificial" geotextile directly above the felled trees. The large movement observed at the area was detected. The settlement which had occurred at the time of pavement placement of excessive fill in this area, which overstressed the seam.

The placement of subsequent fill was initiated at the toe of the embankment and proceeded toward the center. The settlement occurring at the toes further tensioned the fabric.

No turning of the fill placement vehicles was allowed on the first lifts of fill.

The height of fill piles was restricted to 3 feet before blading. No fill piles were allowed to remain overnight. Side slopes were not allowed to become steeper than 4:1 at any time.

EMBANKMENT PERFORMANCE

Monitoring of the construction operations and the performance of the embankment was provided by WDOT.

The instrumentation that was installed included settlement plates, pore pressure piezometers and inclinometers. The majority of the instrumentation was concentrated where the organic soils were weakest and thickest. The primary data related to settlement of the embankment is discussed in the following paragraphs.

The during construction and immediate post-construction performance of the embankment, as determined by monitoring the settlement plates, was as predicted. In most areas, the settlement which had occurred at the time of pavement construction was in the range of 2 to 4 feet. The one exception was in the localized area where overfilling caused shear displacement that resulted in 6 feet of apparent settlement.

It is believed that early during construction, a localized seam failure occurred in the sacrificial fabric at this location. The failure likely resulted from the unintentional placement of excessive fill in this area, which overstressed the seam. Several feet of excess fill was placed in this area before the failure was detected. The large movement observed at the east side at Station 395+00 appeared to result from shear displacement—not consolidation settlement.

The area was repaired by filling the depression with light weight fill (branches and twigs) up to the surrounding fill surface. The area was then covered with an additional sacrificial geotextile layer, which overlapped a minimum of 5 feet over the stable surrounding fill. Conventional procedures then resumed, including placement of the two high strength geotextile reinforcing layers.

The post-paving performance of the embankment has been as good or slightly better than predicted. Survey markers installed on the pavement, shortly after placement, experienced from zero to 5 inches of settlement during the subsequent year and a quarter. Since post-paving differential settlement has occurred over a long span, the distortion is minimal and rideability of the section is considered good.

CONCLUSIONS

1. Geosynthetic reinforcement can be engineered, using procedures discussed herein, to allow the support of embankments over weak foundations.

2. Geosynthetic reinforcement is a cost-effective method. It is conservatively estimated that $400,000 was saved on the Highway 45 project, when compared to more conventional alternatives.

3. The settlement of embankments can be predicted using methods discussed herein.

4. The geotextile reinforced embankment can be tolerant of significant settlement.

5. The post-pavement construction settlement has been slight and the rideability of the highway is excellent.

6. Close construction monitoring is important and should be considered an extension of the design process.

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