Design and Construction of Highway Embankments over Amorphous Peat or Muck

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Construction of highway embankments over deposits of amorphous peat and muck is made difficult by the low shear strengths, high compressibilities, and excessive amounts of creep typically associated with soils of this type. The objective of this research was to develop a procedure for the design and construction of low embankments over such materials for use by the Indiana Department of Highways. This paper begins with a review of the compression behavior of these soils, including a method for predicting embankment settlements from the results of laboratory tests. A soil testing program is then developed for determining parameters required for embankment design and construction. Field vane shear tests are recommended for the measurement of the undrained shear strength, and creep tests are recommended for calculating parameters to predict settlement. The paper concludes by presenting a procedure for the design and construction of embankments over amorphous peats and mucks. The procedure relies on stage loading, preloading, and in some instances geotextiles, to overcome the problems ordinarily encountered during construction over such soft soils.

Numerous deposits of amorphous peat and muck are located within the State of Indiana, and elsewhere. Many difficulties are encountered when highway embankments are constructed over these soft soils. In the past, highway engineers have relocated roadways to avoid construction over peat or muck. In other instances, the organic material was excavated and replaced with a more suitable material. However, as neither of these methods is economical by modern standards, highway departments have been forced to develop more sophisticated methods which allow construction directly across deposits of such materials.

Two characteristics of amorphous peats and mucks make them undesirable as materials for embankment foundations. Materials of this kind compress excessively when subjected to an applied load. Most of the compression results from the relatively high amounts of secondary compression associated with organic soils. These deformations occur over a long time, which compounds the problem. Deposits of these materials possess low preconsolidation pressures, so a large compression response is likely even at low stress levels. Amorphous peats and mucks are also characterized by very low shear strengths. Shear failures, which are both expensive and time-consuming to renovate, can occur very easily either during or after construction. Deposits of amorphous peat and muck vary greatly, so that representative values of compressibility and shear strength are difficult to define. Details of peat and muck behavior are covered in previous reports, Gruen and Lovell (*1*) and Joseph (*2*).

As a result of these typical characteristics, efforts to construct highway embankments over these materials have often resulted in poor performance in the form of excessive total settlements, large differential settlements, and shear failures. Moreover, attempts to predict embankment settlements from the results of laboratory tests are often unsuccessful.

It is the aim of this paper to present a procedure for designing and building embankments over amorphous peat and muck, as developed by Crowl and Lovell (*3*). This procedure will include use of a soil testing program to determine parameters needed during embankment construction. In addition, by using the method for settlement prediction presented here, it is hoped that more reliable settlement predictions may be achieved.

PROCEDURE FOR DESIGN AND CONSTRUCTION

Site Exploration

An important step in determining the behavior of embankments over amorphous peat or muck is obtaining reasonable soil parameters for analysis. However, before these parameters can be established, representative soil characteristics of the deposits must be determined. This is not an easy task when dealing with materials of this type.

Finding representative characteristics of the deposits is difficult because amorphous peats and mucks typically vary so much. To find the range of existing soil conditions at the site, a preliminary soil survey should be conducted. Disturbed samples may be taken at various depths using a hand auger or a power auger, and used to determine water

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content, organic content, and specific gravity. As the compressibility and preconsolidation pressure of the soil throughout the deposit are also needed, creep tests should be performed on undisturbed samples. Field vane shear tests should be conducted throughout the site at various depths to determine the undrained shear strength.

The most critical value to be measured during the site investigation is the undrained shear strength. The designer must select a conservative value because of the low factors of safety used in design. For projects such as earth dams Terzaghi and Peck (*4*) recommend spacing borings at a maximum of 100 feet. The variability of deposits of amorphous peat and muck requires more extensive testing. For the purposes described in this paper, a spacing of not greater than 25 feet along the embankment centerline is recommended. Tests should be performed near the surface, at mid-depth, and near the bottom of the deposit to obtain sufficient information regarding the strength profile.

The values of undrained shear strength obtained from field vane shear testing depend on the rate of loading, soil anisotropy, and progressive failure according to Bjerrum (*5*). These limitations should be considered when interpreting field vane shear tests for use in design analyses. It should also be noted that the behavior of amorphous peats and mucks is assumed to be similar to that of soft clays, and that the use of field vane shear testing in the manner described above applies to these materials as well.

During site investigation, the depth of the amorphous peat or muck in the region of the proposed embankment must be determined. A procedure for estimating the thickness of peat deposits, provided in ASTM Standard Specification D4544-86, uses graduated steel rods of 9.5 ± 1.0 mm diameter and 1.0 or 1.2 m length. The rods can be threaded together for use in deposits of any reasonable thickness. Testing involves pushing or driving the rod into the deposit until resistance to penetration is observed to increase sharply. The depth at which resistance increases should be recorded as the deposit thickness. If sampling is desired, a piston-type sampling device as described in MacFarlane (*6*) can be attached to the rod assembly. This method has a number of limitations, and the Standard Specification should be consulted.

The soft clays and marls which often underlie amorphous peats and mucks can influence the behavior of the constructed embankment, and should be sampled as well to determine their effects on embankment behavior. It is advisable to continue sampling with depth until a layer of adequate strength is reached.

Embankment Design

Embankments constructed over soils of this nature can be designed with or without geotextiles, depending on the initial shear strength of the deposit. In some instances, geotextiles are necessary to allow construction to begin. Geotextiles have been found not only to reduce the horizontal deformations of embankments, but to increase stability, bridge weak areas of the subsoil, and provide a barrier between embankment and foundation soils. This paper will cover the design of embankments with or without geotextiles.

Embankments without Geotextile Reinforcement

After the site investigation, the results of field vane shear tests provide a range of values of the undrained shear strength in the deposit. Rather than using an average value of the shear strength, a conservative value (in some cases the lowest measured value) should be used during design analyses. The variability typical of these soils can cause considerable variation in shear strength, and the average value could be significantly greater than the measured lower values.

The factor of safety used in this paper for overall bearing capacity, rotational failure, and lateral squeeze is 1.3. Attewell and Taylor (*7*) state that for embankments constructed on a compressible foundation, a factor of safety on the order of 1.5 is ordinarily used during stability analysis. Values as low as 1.2 have been used when soil data and site conditions were well established. When analyzing the stability of a preloaded embankment, Stamatopoulos and Kotzias (*8*) state that a factor of safety ranging from 1.1 to 1.3 can be used, assuming that the correct input values have been used during analyses. Thus, although a value of 1.3 is used herein, when selecting a factor of safety, considerable judgment based on previous experience should be exercised.

Overall Bearing Capacity: The overall bearing capacity calculation is a simple one. This step is used to find an approximate value of the allowable height. For a strip loading on soils of this type, the bearing capacity equation reduces to

$$q = cN_c \tag{1}$$

where

q = ultimate pressure (psf),
c = undrained shear strength (psf), and
N_c = bearing capacity factor determined from Vesic (*9*).

The maximum allowable load providing a safety factor of 1.3 should be calculated. Once the allowable load is known, the height of this load is found as

$$H = \frac{q_{\text{all}}}{\gamma} \tag{2}$$

where q_{all} = allowable pressure (psf) and γ = unit weight of embankment soil (pcf).

Lateral Squeeze: The weight of an embankment will tend to squeeze the foundation soil laterally. Jurgenson (*10*) states that the force required to cause lateral squeeze of a soil between two rigid plates is equal to

$$P = \frac{1}{a} cBL^2 \tag{3}$$

where

P = total applied load (lb),
a = one half of deposit thickness (ft),
c = undrained shear strength (psf),
B = length of embankment (= 1 ft for unit length), and
L = one half of embankment base width (ft).

A diagram illustrating these variables is provided in Figure 1. The total load, P, for the height of the embankment found in the previous step is then calculated for a unit length of embankment. From this, the required value of the undrained shear strength is

$$c_{req} = \frac{Pa}{BL^2} \tag{4}$$

The resulting factor of safety (C_{avail}/c_{req}) must be greater than 1.3. If this is not the case, the height of the first load may be decreased, or the geometry of the embankment can be adjusted to provide a longer base length.

In addition, lateral squeeze has been evident in instances where the applied stress produced by the embankment is greater than three times the undrained shear strength of the foundation soil. Therefore, the value of c_{req} calculated in Equation 4 must also be at least one-third of the applied stress. If this is not the case, the required shear strength (c_{req}) should be increased to satisfy this criterion. The resulting factor of safety (c_{avail}) must be greater than 1.3.

Embankment Spreading: The lateral earth pressure developed within the embankment, as shown in Figure 2, must be resisted by shearing stresses at the base. If sufficient resistance is not provided by the foundation, the embankment may become unstable. The lateral earth pressure force, P_a, developed within a noncohesive embankment is

$$P_a = 0.5\gamma H^2 \tan^2\left(45 - \frac{\phi}{2}\right) \tag{5}$$

where

γ = unit weight of embankment soil (pcf),
H = height of embankment (ft), and
ϕ = angle of friction of embankment soil.

The resistance, P_r, provided by the foundation soil is

$$P_r = cl \tag{6}$$

where c = undrained shear strength (psf) and l = lateral distance from crest to toe of embankment (ft).

A safety factor of 2 against embankment spreading is suggested for geotextile reinforced embankments (*11*) and has been adopted for unreinforced embankments as well. A calculated factor of safety less than this value will require the use of a lesser height of load.

Rotational Failure: To investigate the stability of the embankment with respect to rotational failure, STABL4 (*12*) or STABL5 (*13*) should be used. The stability analysis should be performed for the allowable embankment height

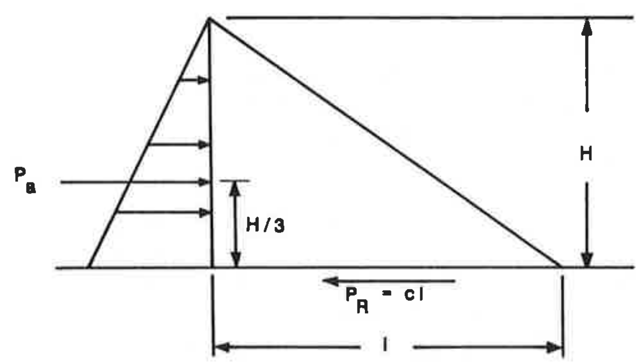


FIGURE 2 Embankment spreading.

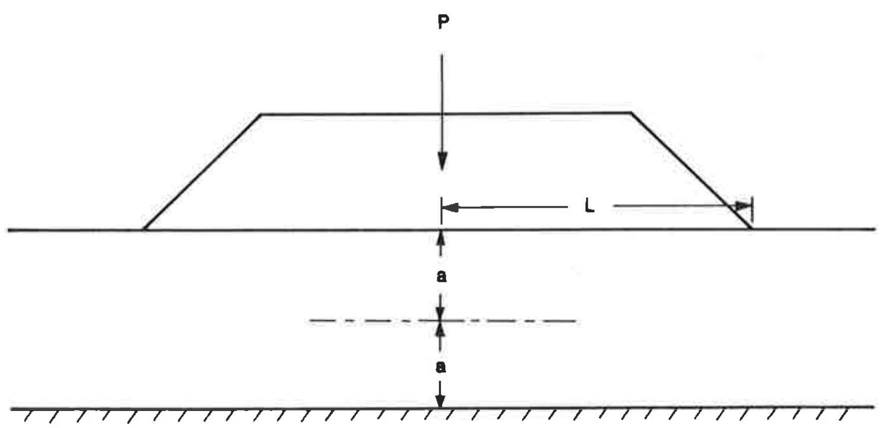


FIGURE 1 Description of variables in Equation 3.

found after the preceding analyses. If the stability analysis yields a factor of safety less than 1.3, another iteration should be performed using a lesser height of load.

Resistance provided by the embankment material in unreinforced embankments may be included in the stability analysis only if an overconsolidated or desiccated layer exists at the surface of the deposit. Otherwise, any lateral movements in the embankment can create tension cracks, sharply reducing the resistance within the embankment.

Geotextile Reinforced Embankments

If geotextiles are used during embankment construction, the allowable safe height of construction is increased as a result of the stabilizing action of the reinforcement. This section will discuss the design of geotextile reinforced embankments, and is based on a design manual by Christopher and Holtz (*14*). The manual provides a more in-depth coverage of the topic, and is recommended reading when designing with geotextiles.

Overall Bearing Capacity: The overall bearing capacity is calculated in the same manner as for the unreinforced embankments. Once again, the recommended factor of safety is 1.3. Once the allowable pressure is calculated, the safe height can be calculated. For geotextile reinforced embankments, the average applied pressure can be estimated as $P/2L$, where P and L are as illustrated in Figure 1.

Lateral Squeeze: Geotextiles have no influence on the extent of lateral squeeze. The required value of the undrained shear strength is therefore found in the same manner as unreinforced embankments. Embankments constructed with geotextiles require a factor of safety of 1.3 against lateral squeeze.

Rotational Failure: Using STABL6 (*15*), a stability analysis should be performed for the calculated height of load. The value of the fabric strength required should be adjusted until the minimum factor of safety is 1.3.

Embankment Spreading: When constructing embankments with geotextiles, the lateral earth pressures exerted by the fill are resisted by the reinforcement. If sufficient friction is not developed between the embankment and the reinforcement, or the foundation and the reinforcement, the embankment may become unstable. Instability may also occur if the foundation soils beneath the embankment cannot resist the applied shear stress.

These two failure modes dictate that the reinforcement must provide enough frictional resistance to prevent sliding along the interface. In addition, the tensile strength of the geotextile must be adequate to prevent rupture or tearing. The lateral earth pressure developed within a noncohesive embankment is given in Equation 5. The resisting force, P_r, provided by the geotextile is found as

$$P_r = 0.5\gamma lH \tan \phi_{sf} \tag{7}$$

where ϕ_{sf} = soil-fabric friction angle and l = lateral distance from crest to toe of embankment (ft).

The value of ϕ_{sf} is equal to

$$\phi_{sf} = \tan^{-1}(4P_a/\gamma lH) \tag{8}$$

The specified value of ϕ_{sf} should be at least 2/3 (ϕ_{soil}).

A factor of safety against embankment spreading is found by dividing the resisting force by the actuating forces. A minimum factor of safety of 2 is recommended by Fowler (*11*).

The lateral earth pressures must be resisted by tension forces in the reinforcement. To prevent splitting or tearing, Fowler recommends a minimum factor of safety of 1.5. The resulting required fabric strength is given by

$$T_f = 1.5P_a \tag{9}$$

where T_f equals fabric tension force.

Limit Fabric Deformation: The stresses required to resist lateral spreading are developed through strain in the geotextile. The modulus of the geotextile controls the amount of strain. The resulting distribution from lateral spreading is assumed to vary linearly from zero at the toe to its maximum value beneath the crest of the embankment.

This assumption is not conservative considering that most geotextiles possess stress-strain curves that develop concave-upward, not linearly. A factor of safety equal to 1.5 should be used to determine the geotextile tensile modulus, E_f. If the required modulus is calculated from the tensile strength force, T_f, the factor of safety is included. The minimum geotextile tensile modulus, E_f, required is found as

$$E_f = \frac{T_f}{\varepsilon_{max}} \tag{10}$$

where ε_{max} is the maximum strain in percent expected in the geotextile along the embankment centerline.

Using the assumed linear strain distribution, the maximum strain is two times the average strain beneath the embankment. A value of 5 percent average strain is recommended for design. The maximum strain would then be 10 percent, and the required fabric tensile modulus may be found as

$$E_f = 10T_f \tag{11}$$

The embankment will also deform until the required fabric strain develops to prevent a rotational stability failure. The actual behavior of the embankment in this condition is unknown, and assumptions outlined in Christopher and Holtz (*14*) have been used. The resulting minimum required modulus to control a rotational failure is found as

$$E_{fr} = \frac{T_{fr}}{0.10} \tag{12}$$

where T_{fr} = required tensile strength of fabric and E_{fr} = minimum fabric modulus.

For additional information on geotextile selection and evaluation, the reader should refer to a modern comprehensive reference, i.e., Christopher and Holtz (*14*); Mitchell and Villet (*16*).

Stage Loading

The softness of amorphous peats and mucks often makes construction to the full height in one stage impossible, particularly if a surcharge is to be placed. Construction will therefore have to be performed in stages. Once the maximum first load, as calculated in the preceding analyses, has been applied, the foundation will begin to consolidate. The consolidation will result in a strength gain, allowing further loads to be placed without inducing failure in the embankment foundation.

To determine the duration of each stage load required for consolidation to occur, pore pressure transducers should be placed in the foundation. After excess pore pressures induced by the previous loading have dissipated, no further strength gain will develop. Field vane shear tests should then be performed in the foundation beneath the embankment, and in areas next to the embankment to determine the extent of the strength gain. Using the increased values of undrained shear strength, the aforementioned analyses should be performed to calculate the allowable height of the second stage load. The procedure of applying the load, allowing pore pressures to dissipate, measuring the increased shear strength, and placing subsequent loads should continue until the final embankment or surcharge height is reached.

Preloading

One of the problems of building highway embankments over amorphous peat and muck is the large amount of secondary compression taking place over an extended period. To reduce the amount of settlement that occurs during the service life of an embankment, a surcharge in excess of the final design embankment height should be placed. The necessary height of surcharge is found by first using the Gibson-Lo model (*17*) to predict settlements produced by the design height of the embankment.

The Gibson-Lo model provides a prediction of the one-dimensional compression of soils. This model is stated in the following equation from Edil and Simon-Gilles (*18*):

$$\varepsilon(t) = \Delta\sigma[a + b(1 - e^{-(\lambda/b)t})] \qquad (13)$$

where

$\varepsilon(t)$ = strain at time t,
$\Delta\sigma$ = applied stress increment,
a = primary compressibility,
b = secondary compressibility, and
λ/b = rate factor for secondary compressibility.

The input parameters required for this model are obtained from the results of creep tests. To obtain the most accurate results, the creep tests should be performed at stress levels simulating actual field loading, rather than using a conventional load increment ratio.

Creep testing begins by reconsolidating the samples at their preconsolidation pressure in the loading frame. Edil and Simon-Gilles (*18*) recommend sustaining the load until deformation is reduced to 0.001 to 0.003 mm/day. At this point, the next load is applied, corresponding to the stress level induced by the design embankment height. The load should be sustained until enough data are collected to accurately calculate the values required for the Gibson-Lo model. For the materials tested during this project, a load duration of 10,000 minutes was found to be sufficient.

After creep tests are completed, the parameters required for the Gibson-Lo model are found using a method presented by Lo, Bozozuk, and Law (*19*). In this method, the logarithm of strain rate is plotted versus time, as shown in Figure 3. The straight line portion of this curve corresponds to the time range of secondary compression. If the straight line is extended back to the *y*-axis, the parameters can be found by solving simultaneously the following equations:

$$\text{line slope} = 0.434(\lambda/b) \qquad (14)$$

$$y\text{-intercept} = \log(\Delta\ \sigma\ \lambda) \qquad (15)$$

$$a = \frac{\varepsilon_f}{\Delta\sigma} - b + be^{-(\lambda/b)t_f} \qquad (16)$$

where ε_f = last strain reading and t_f = time of last strain reading.

The three parameters a, b, and λ/b obtained from laboratory tests depend somewhat on the value of stress increment, final stress level, and the average strain rate. Stress increments less than approximately two times the stress level tested in the laboratory will cause little variation in the value of the parameters. However, this is true only for laboratory conditions. The parameters obtained from the analysis of field and laboratory performances are different as a result of the discrepancies between these conditions.

During research conducted by Edil and Mochtar (*20*), both the laboratory and field behaviors of organic soils under loading were observed. The results were compared to determine any relationships between the two conditions. From this comparison correlations could be developed between the model parameters for laboratory and field performance. Figure 4 provides a curve of consolidation stress versus primary compressibility, and distinguishes between data points from laboratory tests and from field observations. The figure indicates that the primary compressibilities in the field and the laboratory are comparable for the same stress level. Therefore, the laboratory value of the parameter a will compare with the field value when

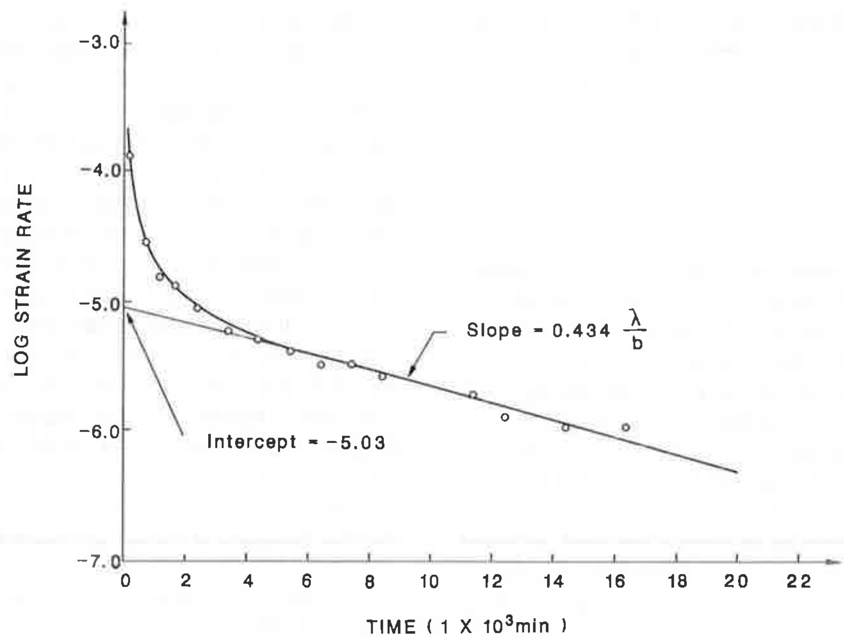


FIGURE 3 Plot of log strain rate with time from laboratory tests (*19*).

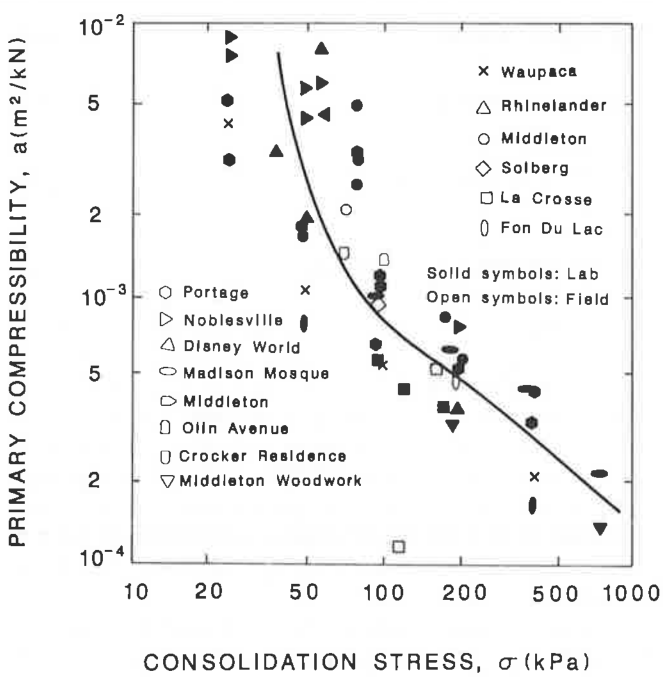


FIGURE 4 Primary compressibility versus stress level (*20*).

the variation of soil properties is considered. In addition, the curve fitted through the plotted points can be used to correct the value of the parameter *a* when a prediction is desired at a different stress level.

Figure 5 provides a curve of the secondary compressibility factor, *b*, versus stress level. The points plotted are for peat data only. As this figure illustrates, the field value of *b* is higher than the laboratory value at equivalent stress levels. Using Figure 5, a plot of b_{field}/b_{lab} versus consolidation stress was constructed as illustrated in Figure 6. Once again, it should be noted that Figure 6 represents data from observations made on peat only.

A plot of strain rate versus λ/b is provided in Figure 7. This figure indicates that no correlation exists between $(\lambda/b)_{lab}$ and $(\lambda/b)_{field}$.

As previously mentioned, the value of a_{lab} is approximately equal to a_{field}, and therefore no correction will be required. Figure 6 can be used to calculate b_{field} from the results of laboratory tests. Using Figure 7, the value of $(\lambda/b)_{field}$ can be determined. If the field strain rate is not known from previous experience, Edil and Mochtar (*20*) recommend using a value two to three orders of magnitude smaller than that observed in the laboratory.

It should be recognized that the recommended correlations in Figures 4 through 7 are best-fit lines through data with a considerable amount of scatter, and that these correlations therefore provide only an approximate relationship between laboratory and field performances. Using them can help improve predictions; however, they still may not provide sufficient reliability, and should therefore be used with caution. As a result, the use of laboratory test results for settlement prediction is still a matter for question. The most reliable settlement predictions can be obtained by observing field performance for calculating the Gibson-Lo model parameters.

Using the corrected parameters, settlement prediction can be conducted using the Gibson-Lo model. The amount of settlement expected within the service life of the embankment can be deduced from the results of settlement

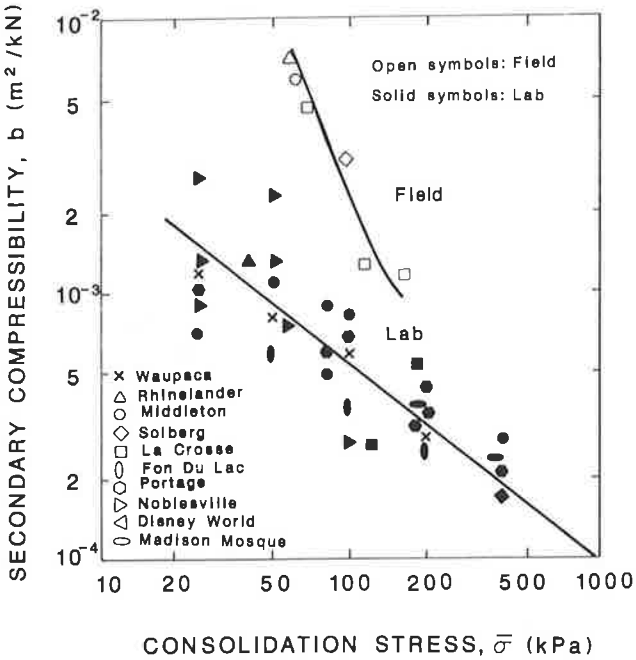


FIGURE 5 Secondary compressibility versus stress level (*20*).

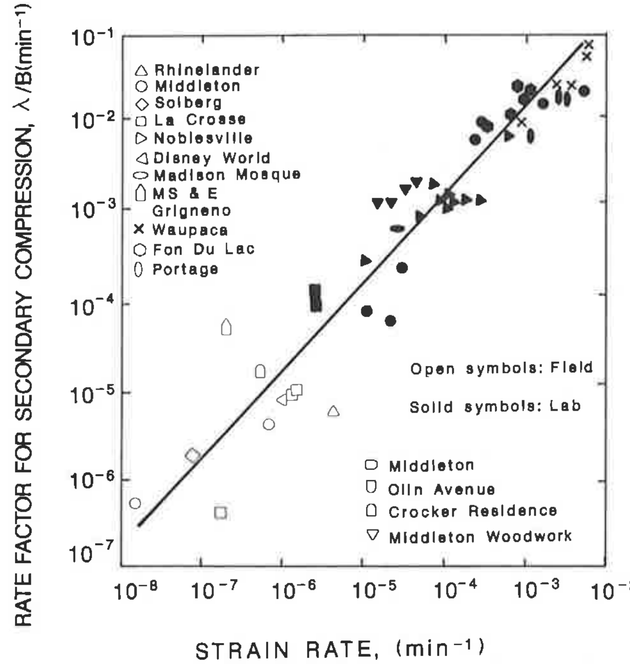


FIGURE 7 Dependency of rate factor for secondary compression on average strain rate (*20*).

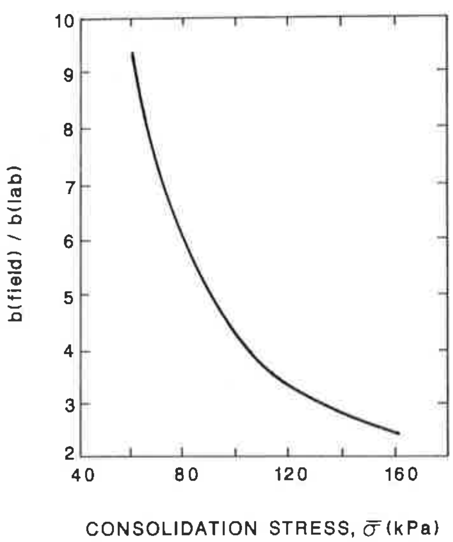


FIGURE 6 Correction curve for laboratory values of secondary compressibility (*20*).

prediction. The objective of the surcharge is to induce that amount of settlement during the time required for primary consolidation. To calculate the height of surcharge required to accomplish this, the Gibson-Lo model should be used to predict the settlement produced by various heights of surcharge, until the appropriate value is obtained. The results of creep tests simulating loading by the design embankment height may be used, so long as the stress increase of the surcharge plus the embankment is less than twice that used during these tests, as concluded by Gruen (*17*). After the height of surcharge is determined, the analyses presented for embankment design must be performed, to ensure that the surcharge does not create any instabilities.

Field Observations

To help monitor the behavior of the deposit of amorphous peat or muck when loaded, several field observations should be made. The most obvious of these is a record of settlements along the embankment centerline. These measurements can be compared with the predicted settlements to check their accuracy; they can also be used to calculate the field strain rate of the deposit so that the rate factor for settlement prediction can be corrected if necessary. Settlement measurements will also be used to determine when the required amount of settlement has occurred, allowing the surcharge to be removed.

Inclinometers should also be placed in the embankment site to measure any lateral movements of the embankment. Data obtained from inclinometers should be interpreted carefully, as these soft materials can flow around the inclinometer. As mentioned previously, pore pressure transducers should be installed to observe the dissipation of excess pore pressures. All types of field instrumentation should be installed to provide redundancy. This will allow for any equipment that becomes inoperable or is disturbed during construction.

Embankment Materials

Deposits of amorphous peat or muck occur in low-lying areas and are very wet. Therefore, portions of the embankment will become saturated, particularly as settlement occurs. Because of this, a well-graded material possessing a limited amount of fines should be chosen for construction above the water table. This will allow embankment drainage and will reduce the effects of wetting/drying or freezing/thawing.

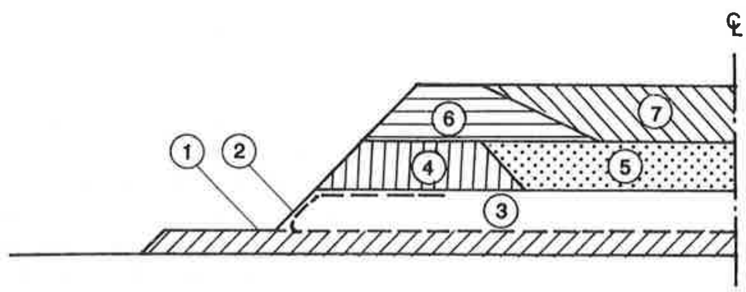
Construction Sequence

Barsvary et al. (*21*) present a sequence of construction for embankments over soft subsoils. A diagram of their procedure is illustrated in Figure 8. Before actual construction begins, they recommend placing a working platform on the foundation soil for construction mobility and easier placement of the geotextile. If geotextiles are to be used, they should be placed on the working platform, transverse to the alignment of the embankment. After placing the embankment to a height of one foot, the geotextile should be folded back on top of this material as shown in the figure. The geotextile should then be anchored by compacting earth above the folded region as in Step 4. The core of the embankment is then placed and compacted. Subsequent lifts should then be constructed by placing and compacting the edges as shown in Step 6, followed by placement and compaction of the embankment core. Compaction lifts should be kept at about the same level, to aid compaction by lateral constraint.

CONCLUSIONS

This paper has investigated the problems associated with the construction of highway embankments over amorphous peats and mucks, and a number of conclusions have been reached.

1. The use of relationships developed by Edil and Mochtar (*20*) correlating the results of laboratory tests with field behavior will improve the results of settlement predictions made with the Gibson-Lo model. However, these correlations are approximations and should be used with caution.
2. Field vane shear testing may be a useful method for measuring the undrained shear strength of amorphous peats and mucks. However, the limitations of vane shear testing, as discussed in Bjerrum (*5*), should be considered when interpreting test results. It should also be restated that a conservative value of the undrained shear strength should be used during design analyses because of the variability normally encountered with these soils.
3. In order to construct embankments over deposits of amorphous peat or muck, stage loading will be required in most instances, especially when a surcharge is to be applied. The strength gain from consolidation will allow placement of subsequent loads without inducing failure in the foundation.
4. To reduce the amount of settlement during the service life of the embankment, a surcharge should be placed to speed up compression of the foundation.
5. For deposits of extremely soft materials, geotextiles may be required for successful construction. To supplement the information provided in this report, the reference by Christopher and Holtz (*14*) should be consulted.



STAGE I

1. Place working platform
2. Place geotextile transverse to alignment
3. Place 0.3 m granular and fold back geotextile
4. Place and compact earth to anchor geotextile
5. Place and compact embankment core

STAGE II

6. Place and compact earth to profile grade at edges
7. Place and compact earth to profile grade at core

FIGURE 8 Construction sequence (*21*).

6. The procedure presented in this paper has recently been submitted to the Indiana Department of Highways, which provided financial support for this research project. At this time, the design and construction procedure is still in the experimental stage and has not been implemented. Therefore, several test embankments should be constructed using this method to evaluate its validity and usefulness.

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