TREATMENT OF SOFT FOUNDATIONS FOR HIGHWAY EMBANKMENTS

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● Since time immemorial, engineers have constructed or have attempted to construct roads across areas of soft foundation materials of low bearing strengths with varying degrees of success. Such material is usually encountered in swamps, marshes, bogs, river and lake bottoms, and lacustrine clay areas. Almost every state has its examples where exceedingly excessive amounts of fill, far beyond the estimated quantities, were needed to complete an embankment across such material. Over-runs exceeding 2,000 percent have been experienced in the past in attempting to construct such fills. Even after the embankments have been finally constructed, their poor performance has warranted the needless expenditure of funds to maintain safe riding qualities of the pavements built on them.

Many methods have been used for constructing embankments on soft foundations. Some are as old as the art of engineering, others are of recent development. The methods described herein are limited to those that have been utilized in the design of specific projects by the New York State Department of Public Works. Any methods omitted from the following are not included solely because they were not applicable to the various problems. The omitted methods may have application for other problems in other areas.

It is necessary to emphasize here that the selection of any method of foundation treatment for highway embankments is wholly governed by the economics of the problem and the end results desired, based on the class of road under construction. Obviously, it is not necessary or economical to provide an elaborate and expensive method for the treatment of soft foundations for the embankments of a low-class road. Conversely, a method that is satisfactory both from economic and performance points of view for a low-class road may be entirely unsatisfactory for a high-class road. Therefore, the foundation treatment method chosen must be governed primarily by both economical and performance considerations in conformance with the class of project.

DISTRIBUTION AND CHARACTERISTICS OF SOFT-FOUNDATION AREAS

Generally speaking, all deposits of soft materials have three conditions in common; flat topography, poor drainage, and fine-grained or organic soils. Flat topography causes poor surface drainage which permits long periods for sedimentation and the formation of fine-grained or organic deposits. Geologically, such soft deposits can usually be classified as lacustrine, organic, or alluvial.

In New York, fresh and salt marshes and swamps are encountered bordering the many bays and estuaries in the vicinity of the lower Hudson River and Long Island. These usually occur in the form of soft peats and mucks, sometimes underlain by soft silts and clays, and may extend to considerable depths. Fresh water swamps with similar conditions may also be encountered along the shores and inlets of the various large lakes of the state, such as Ontario, Erie, and Champlain. Embankment construction along the shores of these bodies of water may encounter soft organic foundations wherever shallow stagnant waters occur. In the central part of the state, especially in the vicinity of Syracuse, many of the organic deposits are underlain by soft plastic marl. Numerous other organic deposits are scattered throughout the state at random within areas of flat topography and poor drainage. The peats and mucks encountered usually have high moisture content, great compressibility, low permeability, low density, and low shear strength. The marls generally possess a fair shear strength, low to medium plasticity, high compressibility and medium permeability.

A considerable portion of New York was covered by post-glacial lakes resulting in lacustrine soils at these locations. In these areas varved plastic clays may occur to depths greater than 200 ft. Some of these lacrustrine soils are covered by a thin layer of granular outwash material, but in most instances the clays are found at the surface, with the upper 5 to 20 ft. consisting of a brown oxidized layer. These varved lacustrine materials vary widely in shear strengths according to location, may be heavily precompressed or normally loaded, and usually possess low coefficients of consolidation and from low to medium plasticity. The more extensive areas of these so-called clays have been tenta tively defined and named according to their general locations, such as Hudson-Champlain, St. Lawrence, Oneida, Ontario, Geneva, and Buffalo.

Silts of postglacial alluvial origin are often encountered along the sides and on the bottoms of the various rivers and lakes of the state, and may be organic or inorganic. These alluvial silts vary widely in shear strengths, usually are normally loaded and very compressible, and possess medium coefficients of consolidation and low plasticity.

All of the above-discussed materials are also often encountered in small localized areas throughout the state.

LOCATING AND DEFINING AREAS OF SOFT FOUNDATIONS

The correct, accurate solution of any foundation problem demands a thoroughly adequate subsurface exploration program to furnish sufficient field and laboratory information. It must be emphasized that many of the following methods of foundation treatment require complete shear and consolidation characteristics profiles of the questionable layers in order to produce a safe yet economical design. This is particularly true of the more elaborate expensive methods of foundation treatment, such as stabilizing berms and sand drains. Insufficient field and laboratory information may result in unsafe embankments or uneconomical over-safe designs. The additional subsurface exploration required to produce a sufficient amount of information costs only a very small fraction of the expense involved in correcting a major shear failure.

U. S. Department of Agriculture Soil Survey Bulletins and Maps, U. S. Geological Survey Topographic Maps, aerial photographs, site inspections, and preliminary soil surveys all aid in detecting the presence of such deposits, and are useful in planning an adequate subsurface exploration program. Occasionally, soft materials are covered by a thin veneer of stable material of adequate shear strength, and their presence may not be detected except by subsurface explorations.

The subsurface-exploration program should consist of two phases. The first phase includes taking a sufficient number of holes by hand- or power-operated equipment to define the horizontal and vertical limits of the deposit and to obtain proper samples for evaluating the general soil characteristics. This information should be adequate to permit the preparation of soil profiles. In the second phase, undisturbed sample holes are located and undisturbed samples obtained representative of the entire profile to permit the necessary laboratory structural testing. For some projects the first-phase program may be sufficient to furnish the necessary information for evaluating the proper method of treatment for the foundation.

An appreciation of soil and geologic origins, combined with an understanding of methods of reconnaissance, can be of great value in planning the preliminary location of a route and the corresponding preliminary subsurface exploration. Highway engineers occasionally overlook the fact that changing the line and grade of a proposed design to avoid or minimize the difficulties associated with soft foundations may often result in a more economical design and a more satisfactory highway. It is often more economical to avoid a serious foundation problem by changing line or grade than to attempt an elaborate solution. Soils-engineering knowledge should be fully utilized not only in solving subsurface problems but in avoiding them if possible.

MAJOR ENGINEERING REQUIREMENTS

Although the selection of any method of soft-foundation treatment is governed in a general sense by the economic considerations involved and the result desired, such selection is also specifically governed by certain engineering requirements which must be satisfied. Each embankment must satisfy two engineering requirements: stability and settlement.

Stability—First, an embankment must be stable against any lateral displacement. The safety factor against slope and foundation failure must preclude any possibility of such failures if the embankment is to serve satisfactorily its intended purpose.

Settlement—A highway embankment may be safe against failures but may be entirely unsatisfactory because of settlement. The two settlement considerations involve the overall magnitude and the time and rate of settlement. Differential settlements are usually more detrimental than uniform settlements of much higher magnitude. Detrimental differential and total settlements result in broken, dangerous pavements and excessive maintenance costs. Differential settlements adjacent to firm structures, such as bridge abutments, are specifically objectionable, and require continued maintenance to permit safe driving. Uniform settlements over long stretches reflecting gradual transitions may not be objectionable and may not affect the road riding qualities. For such conditions uniform settlements up to 2 ft. have been experienced without affecting the quality of road.

The time and rate of settlement are also important considerations. In general, all detrimental settlement must take place before any permanent pavement is placed. To accomplish this may require the use of methods to accelerate settlement, such as sand drains or surcharges. If these are not possible or economical, it may be necessary to install a temporary pavement on the embankment while the detrimental settlements are taking place, and to construct the permanent pavement after settlement has stopped.

Settlement and stability are interrelated in that the strength of the underlying soft material depends on its degree of consolidation under the load imposed by the embankment. An increase in the degree of consolidation under the embankment load will proportionately increase the strength.

ENGINEERING FACTORS AFFECTING METHODS OF TREATMENT

Every problem involving the design and construction of an embankment on a soft foundation usually has several methods of solution. The most satisfactory solution depends upon several considerations.

Embankment Dimensions—The desired height and other dimensions of the embankment greatly influence the choice of the most satisfactory method of foundation treatment. The higher the embankment the greater the stresses it imposes on the foundation and the greater the settlement. A narrow, high embankment is sunk by displacement more satisfactorily than a low, wide one, for which the possibilities are greater for local embankment failures and entrapment of compressible material. Wider embankments cause deeper bulbs of pressure, which produce greater settlements in deep compressible layers.

Foundation Characteristics—The strength, profile, and dimensions of the soft foundation influence the choice of methods which can be used economically for its treatment. These considerations are general and usually obvious, and they will not be discussed here.

Construction Materials—Much depends on the availability and cost of the various construction materials near the site. For example, if suitable granular material is not available, methods involving underwater placement may not be economical, or, if borrow material is scarce, methods requiring large quantities of borrow would not be feasible. The use of light-weight materials, such as slag or cinders, generally is not considered at great distances from the source of these materials.

Construction Schedule—The permissible construction schedule greatly influences the selection of the correct method of soft foundation treatment. If the pavement is to be placed immediately after the completion of the embankment which itself is rapidly constructed, then all detrimental settlement must be completed rapidly. Methods involving long-time detrimental settlements cannot be considered in this instance. Furthermore, the rapid construction of an embankment on a deep deposit of soft material implies an almost instantaneous application of load. The underlying soft material may not gain any strength by consolidation, and the safe heights of embankments so placed may be severely limited. Conversely, slow and delayed construction may permit a sufficient degree of consolidation to reflect definite strength increase in the foundation. If considerable time is planned between the grading and the paving contracts, the interval may be profitably used to allow the major portion of the expected foundation settlement to take place, resulting in a more satisfactory pavement.

Location—The topographic and man-made conditions surrounding the site also govern the method of foundation treatment selected. Sinking a fill by explosives cannot usually be done in populated areas. Narrow rights-of-way may limit the use of wide berms, or the formation of mud waves by planned displacement. Constructing an embankment adjacent to an existing structure supported on spread footings on a deep clay deposit may result in detrimental settlements of the existing structure. Care must be exercised not to disturb existing railroads and utilities on soft foundations. Their presence usually limits the method of construction and treatment.

POSSIBLE METHODS OF TREATMENT

The possible methods of treatment of soft foundations for high embankments which have been used by the New York State Department of Public Works are shown in Table 1. The methods listed under "Removal"

TABLE 1

POSSIBLE METHODS OF TREATMENT OF SOFT FOUNDATIONS FOR HIGHWAY EMBANKMENTS

I. Removal—by:
   A. Excavation
      1. Complete
      2. Partial

   B. Displacement
      1. Embankment weight
         a. Normal height
         b. Surcharge
      2. Using explosives

II. In-Place treatment
   A. Primarily for Stability Requirements
      1. Slow-rate construction
      2. Use of light-weight material
      3. Use of stabilizing berms
      4. Drainage interception

   B. Primarily for settlement requirements
      1. Normal construction
      2. Two-stage construction
      3. Surcharge
      4. Heavy compaction

   C. For settlement and stability requirements
      1. Vertical sand drains
      2. Combinations of above methods

satisfy both requirements for stability and settlement. The in-place treatment methods have been listed under the major requirements with which they are generally associated: stability, settlement, and stability and settlement combined. Descriptions of specific projects making use of these methods have been included in the discussions of the individual methods.

EXCAVATION

Consideration is given to excavation whenever the soft material is reasonably shallow, the necessary borrow is readily available, and the project is of such nature requiring embankment stability in a relatively short time. Excavation is generally economical where areas are available adjacent to the excavation for disposal of the unsuitable material, so that excavation and side casting can be accomplished in one operation. After completion of the embankment, the side-cast spoils can be bulldozed to flatten the side slopes, or leveled to fit adjacent topography. Generally, the water table is close to the surface, and the excavated areas become filled with water. In such instances the material used for backfill to several feet above water should be granular to compact readily as dumped under water.

The width of excavation is controlled by the dimensions required to eliminate settlement under the roadway and shoulder areas, and to insure stability against failure of the side slopes. Generally, to estimate quantities the width considered includes the rectangular portion made by a 45-deg. line starting 1 ft. beyond the edge of pavement, continuing downward and outward to the bottom of the proposed excavation, and then vertically upward to the original surface. If such a section does not offer stability to the shoulders and side slopes, the section is widened accordingly. In cases where the soft material is quite shallow, the maximum full width between toes may be excavated. The dimensions used to obtain the width of excavation are indicated in Figure 1.

Complete Excavation

Excavation may be either complete or partial. Complete excavation should remove all of the unsuitable material to firm bottom and insure against subsequent shear failure and settlement. It is generally considered when the soft foundation is of reasonable depth, the quantities involved are relatively small, the embankment is of such width to indicate possible entrapment of objectionable quantities of the soft material if not removed, and the project is of such nature to require stability in a short time.

On existing projects in New York State, complete excavation has been obtained economically to depths up to 35 ft. Where quantities of excavation of unsuitable material have been large, more favorable contract prices have been realized by setting up this excavation as an item separate from the regular common excavation item for construction.

Partial Excavation

Frequently an extremely soft, compressible material of low bearing strength is underlain at fairly shallow depths by a somewhat stronger material compressible to a lesser degree. The surface deposit will not safely support an embankment of the desired height, but the second layer will. Furthermore, the surface deposit is the seat of a greater portion of the expected settlement. In this case it is often advantageous, economically, to excavate the surface layer and build on the underlying stronger but still compressible material, planning on letting the embankment undergo the smaller settlement caused by the second layer. Having reduced the thickness of the compressible layer by partial excavation, the time required to eliminate settlement within the thickness remaining will become considerably shorter. Figure 1 shows a cross-section of a portion of a state highway project crossing a swamp deposit in southern New York, for which partial excavation was recommended for construction. The soil profile consisted of a layer of very soft peat and organic silt overlying a layer of soft silt with a trace of fine sand, over firm bottom of sand and gravel. Both compressible layers were of variable thicknesses, with the peat and organic silt averaging 15 ft and the lower silt layer averaging 20 ft. Ground water was at the surface.

Although the upper peat and organic silt layer was sufficiently soft to displace partially under the weight of the embankment, the embankment was of sufficient width to result in possible entrapment of a good portion of the layer. This would produce bad differential settlements. Figure 1 compares the stability and settlement values for both building on and removing the upper soft peat and organic silt layer. Excavating this upper layer and placing the embankment directly on the lower layer, insured stability, reduced the magnitude and time of settlement, and permitted completion within the same construction period.

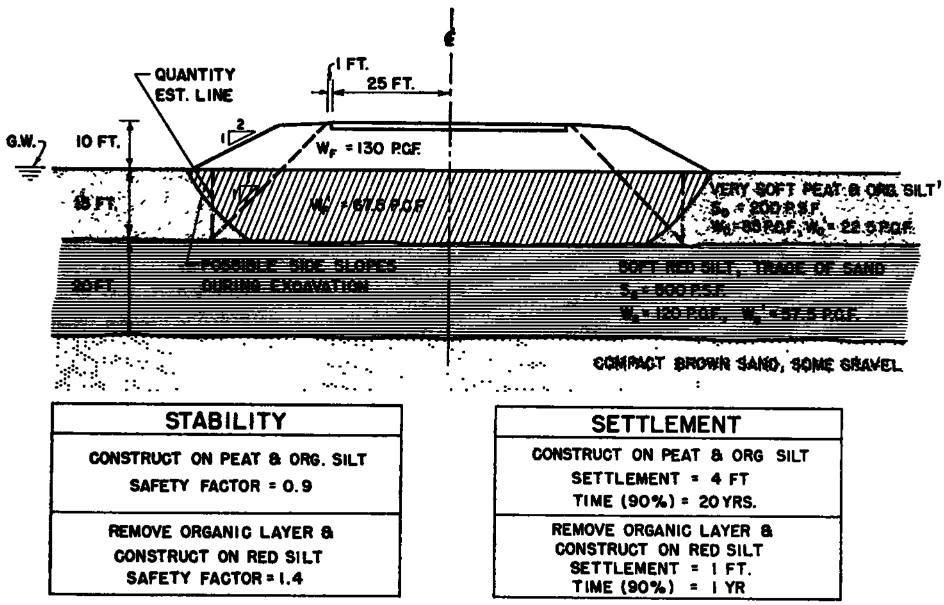

Figure 1. Partial-excavation method of construction.

Partial excavation is also used in uniformly soft deep deposits where the remaining material beyond the depth of excavation can be displaced by the embankment weight or surcharge. Such a method is recommended only when the soft deposit is rather deep, the embankment is generally wide enough to hinder full displacement by standard methods, the topography is such to permit displacement of the unexcavated soft material, and the embankment is sufficiently high to furnish the necessary weight for displacement. Generally, in such cases a surcharge is added beyond normal embankment height to permit rapid adjustment for settlement, if a portion of the underlying soft material remains entrapped within the embankment area.

DISPLACEMENT

Whenever the average stress in the foundation, caused by the embankment weight, exceeds the strength of the material sufficiently to overcome any restraining force present, displacement occurs in the direction of least resistance. The amount or extent of displacement is dependent on the interrelation of the superimposed weight, the strength of the underlying soil, uniformity of material and profile, the geometric relation of width and depth, and the presence or absence of any restraining forces.

As the soft material displaces from under the embankment, it builds up as mud waves on the sides or in front of the embankment. These waves act as passive restraining forces, reducing the tendency for further displacement. In some instances the embankment weight is sufficiently great with respect to the strength of foundation to displace all of the soft material. In other instances weight additional to that of the embankment is required to overcome the restraining forces present, or developed, and permit full displacement. Generally, in uniformly soft materials, once an embankment has started to sink, it will so continue if maintained at the same top elevation, and if any mud waves formed adjacent to it are removed. This will occur because the fill material replacing the soft foundation is of greater density, and proportionately increases the total superimposed weight. However, if not removed as it forms, it is possible for the mud wave to build up sufficient restraint to check any further sinking.

A wide embankment placed on a soft foundation sinks unevenly across its width, with the maximum settlement occurring about half-way between the toes and the center line. This type of settlement has been described by others as "heart shaped," due to its resemblance to the lower part of an inverted heart. This tendency leads to the possibility of entrapping a central core under the embankment, producing undesirable differential settlements after construction.

Displacement has been considered as taking place either normally under the weight of the regular embankment, or with additional aid, such as removal of mud wave, surcharge, use of explosives, or combinations of these. These methods are covered below.

Normal Displacement by Embankment Weight

Some foundation soils, such as soft mucks and peats, organic silts, and marls, may be displaced from under the embankments by merely superimposing the normal embankment heights. Displacement by normal height embankment is generally considered when the weight of embankment planned is sufficient to displace the material, and when any material that may be entrapped under the embankment will have characteristics and dimensions resulting in settlement adjustment under the normal weight, within the time permitted by the construction period before paving. For some projects, provisions are made to remove the mud waves as they develop adjacent to the embankment, and to cast the material beyond the limits required to permit free displacement of all underlying material.

One highway project so designed crosses a section of a bay leading to Lake Ontario, north of Rochester. The soil profile consisted of a thin layer of soft organic silt 5 ft. thick, over a layer of soft marl 20 ft. thick, over sand. The surface of the organic silt was under three feet of water. The embankment ranged in height from 13 to 37 ft. above the water. A typical section of this project is shown in Figure 2. The strength of the marl was such to permit displacement under any embankment higher than 10 ft. The first ten feet of embankment consisted of granular material, placed to float on the marl and provide a working platform above the water surface. The remaining embankment was common borrow. Provisions were made to remove the mud waves raised above the water level, to permit easier displacement of the underlying marl. It was anticipated that some marl would be entrapped near the center of embankment, but its thickness, characteristics, and drainage were such to allow easy adjustment within the construction period.

Surcharge

A surcharge may be added to the embankment during construction if its normal height is not sufficient to displace the underlying soft material, or if there is a possibility of entrapping enough material to result in undesirable differential settlement. This surcharge helps in two ways: (1) by inducing additional displacement, and (2) by causing the normal settlement to take place more rapidly.

A project in which a surcharge was used to sink a highway embankment to firm bottom was recently built in swamp areas adjacent to a bay in New York City. The profile consisted of a surface fibrous mat 3 ft. thick, covering a layer of organic silt 15 to 20 ft. thick. Figure 3 shows the construction procedure schematically. Several steps were taken to aid displacement. The fibrous mat was removed before construction, the mud

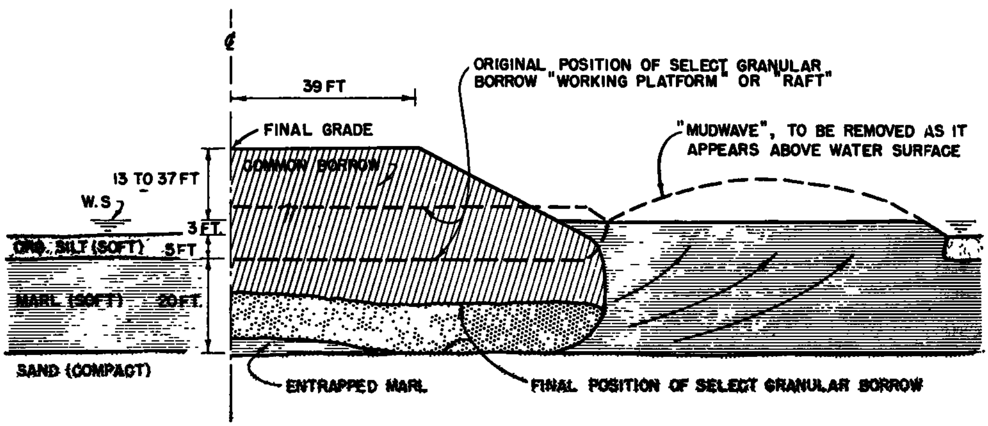

Figure 2. Sinking an embankment by displacement due to its weight.

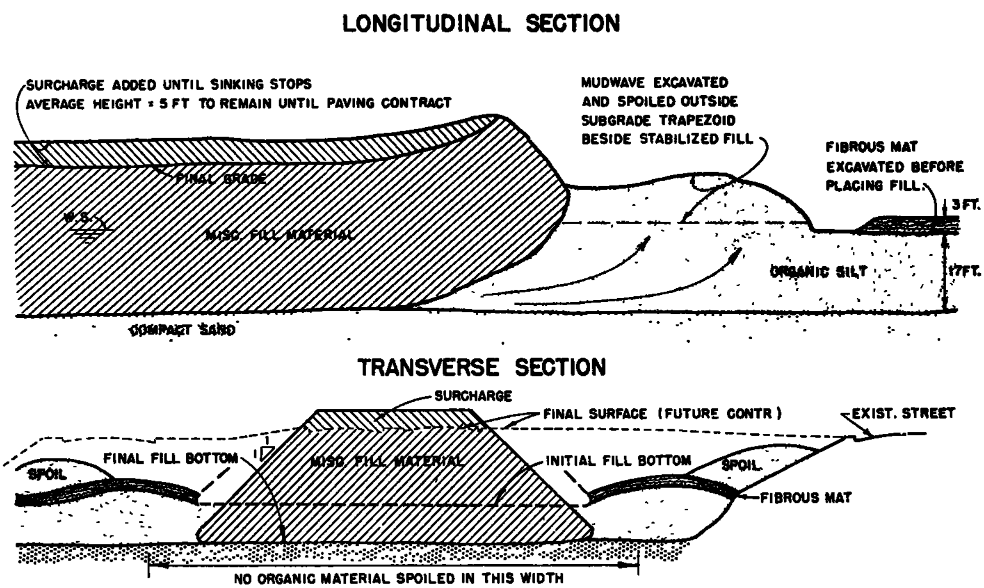

Figure 3. Sinking a fill by displacement using a surcharge.

waves formed ahead of the fill were removed, and a surcharge was added.

The material used for the embankment was quite stony, large quantities being spoil from rock excavations. Because of this, it was impossible to probe through the fill after construction to determine the degree of displacement obtained. However, settlement readings made on the surface for a period of time after construction indicated that the embankment had reached stability soon after completion.

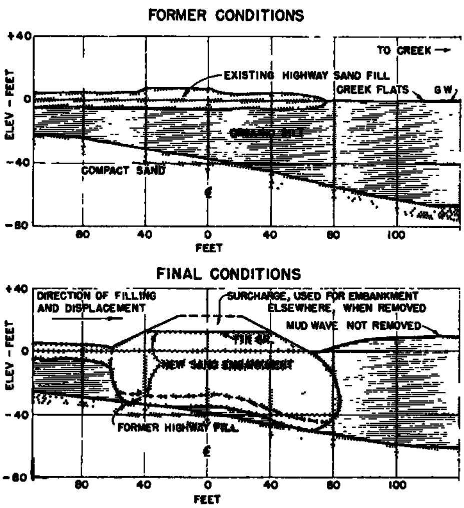

Figure 4. Using a surcharge to sink a fill.

Another project in which a surcharge was used to displace material was along a bay leading to the Hudson River near Peekskill. Here, the road alinement was along the bay at the foot of adjacent hills, and followed an old existing road which had been floated on the soft foundation. The profile then consisted of sand and gravel fill up to 10 ft thick over a soft organic silt layer of variable thickness, over sand. In the past, additional sand fill had been dumped adjacent to the old roadway to widen the surface sand layer. Typical sections for this project, showing the original profile and construction methods used, are given in Figure 4. The organic compressible layer was wedge shaped, widening toward the bay. Although the same general alinement was followed for the new road, it was wider and had a higher grade line. A surcharge of ten feet was used during construction to aid in the displacement of the underlying organic silt. To insure displacement toward the bay, the embankment and surcharge were constructed in sections starting from the hillside and working toward the bay. The mud waves formed indicated that most of the organic silt was displaced toward the bay. The surcharge was maintained for the winter months, during which time no settlement was noticed, and stability had been reached.

Explosives

Explosives for settling embankments have been used in New York on a number of projects to aid in the displacement of soft foundations, whenever the weight alone, or weight and surcharge, were not sufficient to accomplish the displacement. Depths of unsuitable material displaced by this method have varied from 5 to 24 ft. The amount of dynamite used has ranged from ¾ to 1¼ lbs. per cu. yd. of material displaced. In general, these projects were done under contract on a unit price basis per pound of dynamite used. Subsequent performance of these embankments has been highly satisfactory, and the method is still considered where applicable and economical.

Embankment settlement with explosives is restricted to areas where explosives can be used with safety, and where there is no danger from vibration to existing structures or utilities. Most of the general methods of soft foundation displacement and fill settling with explosives have been used in New York, depending on the existing local conditions. Insofar as these methods have been covered in other publications, they will not be discussed. Figure 5 shows typical sections of a project built some years ago in the vicinity of Saranac Lake in Upstate New York, in which explosives were used to displace the soft peat and muck. On this project, the swamp depth ranged to 12 ft. It was first necessary to break the mat crust on the swamp surface, and liquify the underlying peat and muck, by propagation ditch shooting. The embankment was then progressed into the liquified swamp, and was built up to sufficient height to push ahead the loose material. When displacement stopped and the peat and muck stiffened and piled up along the front, charges of dynamite were placed in the peat around the blunt-nosed point of the embankment, and fired by propagation. The enbankment was progressed by this toe shooting method. Soundings were made repeatedly along the side slopes, and where soft pockets were entrapped, additional dynamite was placed and fired to settle the embankment.

In recent years other methods of stabilizing embankment foundations have proven more economical than using explosives, and for this reason explosives have not been used as often as in the past. Most roadway sections being built are wider, and require larger quantities of dynamite per cubic yard of material to be displaced the greater distances.

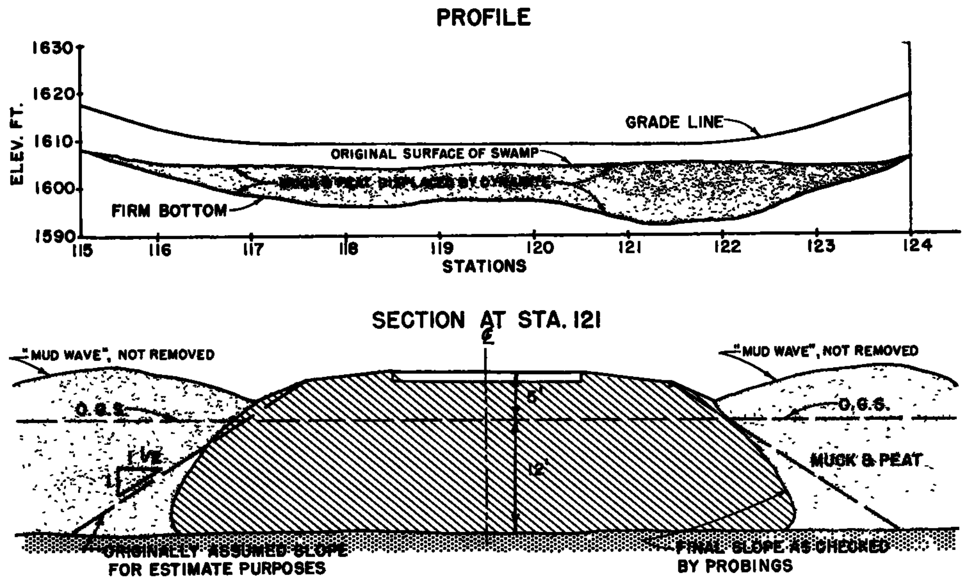

Figure 5. Sinking a fill by displacement using explosives.

IN-PLACE TREATMENT

Removal of soft foundation materials by excavation or displacement involves the use of direct methods which have in common the replacement of the unsuitable or undesirable materials with materials of acceptable quality. The nature of many highway projects is such that a solution by removal will result in excessive quantities, difficult and impractical construction, and unreasonably high costs. In addition, many materials generally classified as soft and weak, although not completely adequate for embankment foundations in their natural state, can be made suitable with special treatment or with modifications in design.

The methods of treatment of soft foundation materials which involve the retention and the actual use of these materials as part of the foundations have been referred to as in-place treatments. As previously mentioned, in-place treatment methods should satisfy the two requirements of stability and settlement, as required by the specific projects. For some projects, only one of the two requirements may be critical, whereas the other may present no problem. Other projects involve the need to satisfy equally both requirements. Conversely, some of the methods listed for in-place treatment are intended to satisfy only one requirement, whereas others could be used to satisfy both. In many instances several methods are combined for the solution of specific projects.

In-Place Treatment for Stability

Where it is planned to retain the soft material as part of the embankment foundations, the requirement for stability against lateral movement should be satisfied. Stability can be increased by decreasing the foundation stress, increasing its strength, or by providing confining or restraining forces. These are discussed in the methods listed below.

Slow-Rate Construction

In some instances the strength of a soft foundation layer is not sufficient to support, without possible failure, the weight of an embankment constructed under normal procedures. However, due to its relatively

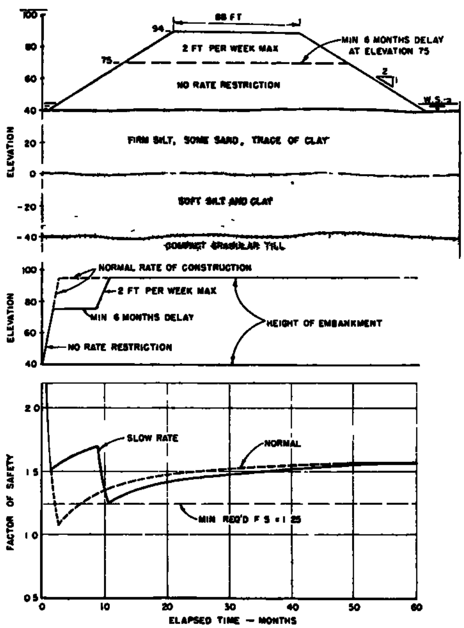

Figure 6. Slow rate of construction.

rapid settlement characteristics, such a foundation may gain sufficient strength during a slow rate or partially delayed construction period to permit the completion of the embankment to its required height. For the method to be effective, the layer should have such consolidation properties and thickness to permit rapid settlement under the weight of the part of the embankment initially placed. For a proper evaluation, complete settlement and stability analyses should be made, along with the effect of the rate of settlement on strength increase.

Figure 6 summarizes the conditions for a project built across the northern end of Seneca Lake in central New York, which included a delayed and slow construction rate to insure stability. The foundation profile consisted of a layer of firm silt and sand from 25 to 45. ft. thick, over a compressible layer of silt and clay from 30 to 60 ft. thick. The material below this was compact granular till. The normal lake level at the site was five feet above lake bottom. The proposed embankment across the lake varied in height from 20 to 60 ft. above the lake bottom. Stability analyses showed that the foundation would safely support a fill up to 40 ft. high. For a higher fill there would have been danger of shear failure if the embankment were built too rapidly.

At the location shown in section in Figure 6, the firm silt and sand layer and the underlying soft silt and clay layer were each approximately 40 ft. thick, with an embankment height of 54 ft. At this location a normal construction rate to full height would have a factor of safety of only slightly greater than unity, which was not considered high enough to assure stability. However, with a minimum of six months delay in construction after having reached a height of 35 ft., and a reduced rate not to exceed 2 ft. per week thereafter to full height, sufficient strength increase in the foundation was realized during construction to obtain a minimum value of 1.25 for the factor of safety, which was considered adequate for this project.

Light-Weight Material

The use of light-weight material for an embankment reduces the embankment weight and the foundation stress by the ratio of the density of that material to the density of common borrow. Light-weight materials generally used are cinders and slag. These, of course, can be considered for use only when they are available locally. Generally, cinders and slag have a total in-place density of 75 to 95 lbs per cu ft. and, compared to common borrow, show an equivalent weight reduction of 30 to 50 percent.

A project for which light-weight material was used to reduce the stress on a soft foundation was a grade separation approach embankment for a city street in South Buffalo. A section of the embankment and foundation is shown in Figure 7. The foundation consisted of a shallow layer of firm sand and silt overlying a layer of soft silt with fine sand and clay, some 60 ft. thick. In this case the slag used for the embankment was a material readily available, and was actually more economical to use than common borrow. Actual in-place density tests made on the slag ten years after construction showed it to have an overall average density of 93 lbs. per cu. ft. The embankment was built to a height of 40 ft. Using slag for the embankment, the factor of safety obtained against foundation failure was 1.25 or greater, whereas with local common borrow the expected factor of safety would have been less than unity, and failure during construction would have probably occurred.

of circle, while the resisting forces include the shearing strength along the arc of failure and the total weight on the low side of the center of circle.

If the factor of safety resulting from such a computation indicates that failure might occur during or soon after construction, the stability can be improved by adding stabilizing berms as counterweights over the original ground. To be effective, these stabilizing berms should be built of such width to have their center of gravity well beyond the center of the most critical sliding circle, obtained for the original section, and to cover the toe of this circle. The width and height of berm required for any project are also obtained by trial and error in the same manner.

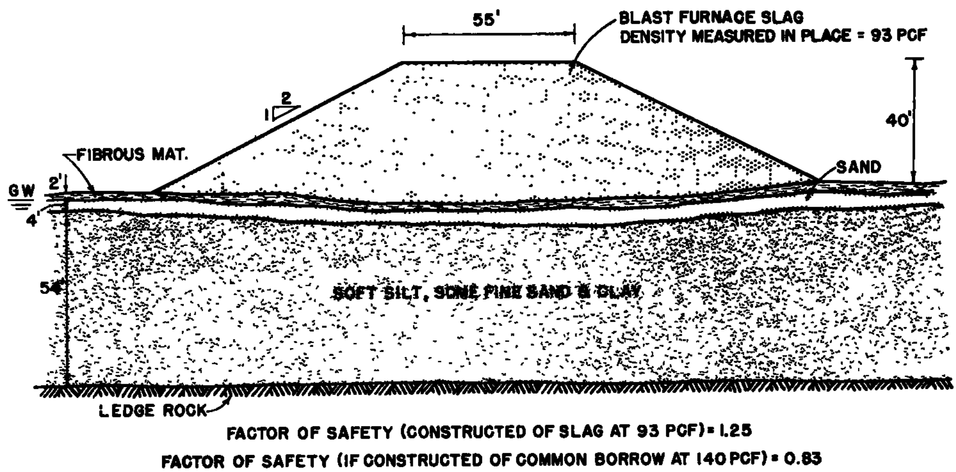

Figure 7. Use of light-weight material in embankment construction.

Stabilizing Berms

If a soft, uniform layer of sufficient thickness fails under the weight of an embankment, the failure usually occurs in the form of a rotating slide, with the failure surface approximating a circular cylinder. In cross-section, this failure surface usually approximates a circular arc. For any section having specific strength values, there is one most critical circle showing the lowest factor of safety against failure, which can be determined by trial and error. Stability computations for any circular arc are made by equating the moments, about the center of the circle, of the actuating forces against the resisting forces. The actuating forces include the total weight on the embankment side of the center

It should be pointed out that in determining heights of berms, the stability of the berm itself must also be satisfied. A typical example of such a stability analysis is given in the appendix. The use of stabilizing berms only satisfies the requirement for stability. Gradual settlement of the embankment due to the consolidation of the underlying foundation soil may follow, and should be considered as a separate requirement.

Berms are also used to correct failures which occur during or after construction. In such cases the position of the failure arc can usually be ascertained from the locations of the shear cracks, mud wave, and depth of firm layer. This information can be used to compute the average shear strength of the foundation material during failure. A stabilizing berm can then be designed to counterbalance the actuating forces.

A project for which stabilizing berms were used to obtain stability was a section of New York State Thruway in the vicinity of Albany. The foundation conditions in the Albany area consist mainly of thick deposits of lake-laid clays extending to depths of 200 ft. or greater. The area is randomly dissected by cut sections involved excavation of large quantities of soft wet plastic silt and clay which was not suitable for normal embankment construction. Consequently, the berms were made so that they could be constructed of this wet material. Therefore, in addition to being used for stability, the berms also provided a place to deposit unsuitable material.

Several feet of settlement was expected under these high embankments. Due to the

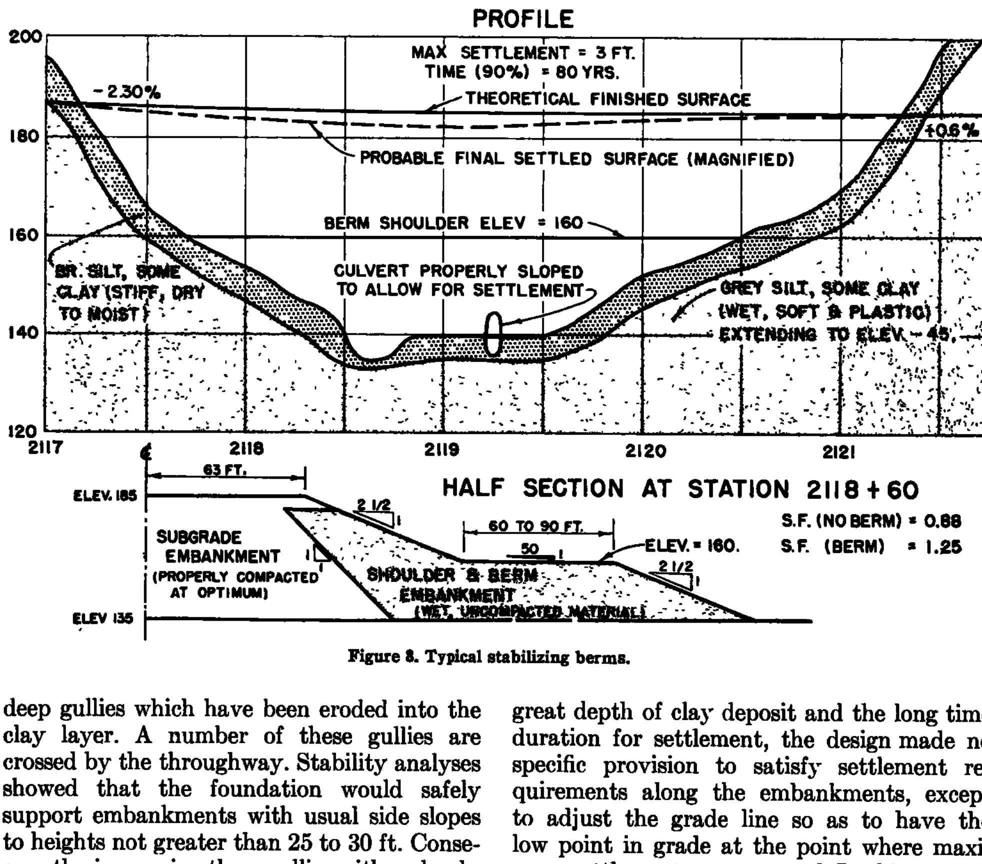

Figure 8. Typical stabilizing berms.

deep gullies which have been eroded into the clay layer. A number of these gullies are crossed by the throughway. Stability analyses showed that the foundation would safely support embankments with usual side slopes to heights not greater than 25 to 30 ft. Consequently, in crossing these gullies with embankments up to 50 ft high, stabilizing side berms were necessary to obtain stability. These berms were from 50 to 100 ft wide and from 15 to 25 ft high. Figure 8 shows a typical section across one of these gullies.

A design factor was the lowering of the grade line to decrease the heights of the fills in the vicinity of the gullies. Cuts were thereby located adjacent to the gullies. The deeper great depth of clay deposit and the long time duration for settlement, the design made no specific provision to satisfy settlement requirements along the embankments, except to adjust the grade line so as to have the low point in grade at the point where maximum settlement was expected. In this manner future settlement will increase the grade slightly toward the low point, and provide gradual transitions without any undesirable differential settlement.

Drainage Interception

Side-hill construction of embankments sometimes leads to foundation failures caused by ground water accumulation and movement. The ground-water movement may be deeply seated and not readily intercepted by the common highway practices of ditches pressures which increase the buoyancy of the foundation soil and thereby decrease its effective weight and shearing resistance.

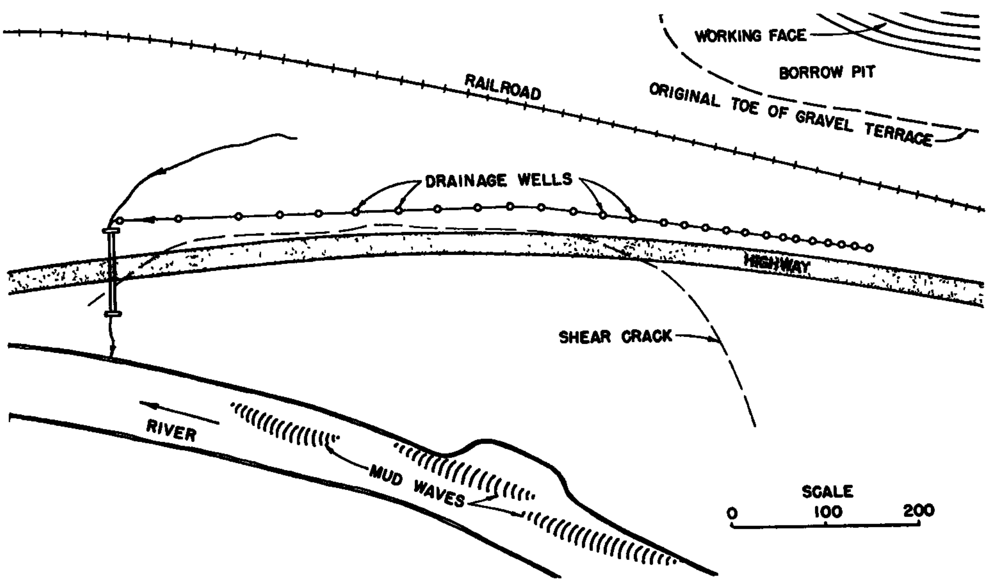

Figure 9. Plan of slide stabilization by drainage.

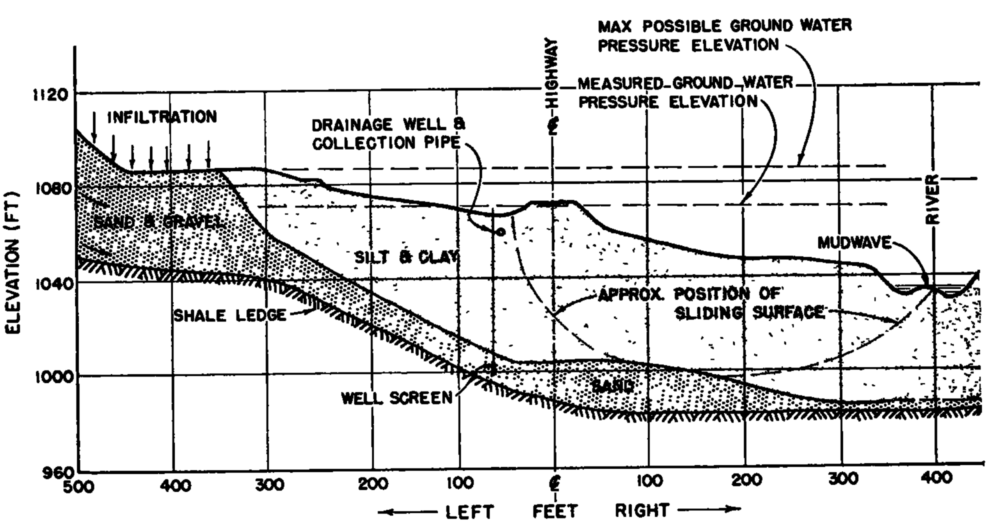

Figure 10. Profile of slide stabilization by drainage.

and underdrains. Such ground water conditions can influence stability in several ways: by developing large seepage forces in the down hill direction which act to increase the sliding forces, and by developing artesian

Where such problems exist, adequate subsurface exploration is essential, with careful observations made of water conditions. In many instances the severity of the problem is not recognized in the investigation and design stages, and the conditions remain unnoticed until during or after construction. Such was the case in the relocation of a state highway in a river valley west of Oneonta, Chenango County. Soon after construction, a slide developed involving a section of the roadway, as shown in Plan and Profile in Figures 9 and 10.

The soil conditions under the embankment consisted of approximately 60 ft. of impervious silt and clay over ten to twenty feet of sand. This sand stratum was continuous under the slide area, became thinner toward the center of the valley, and thicker in the direction toward the hillside. It terminated in a gravel terrace along the valley wall. The gravel terrace provided an infiltration area to supply the sand aquifer. Artesian pressures were made possible by the shape of the impervious silt and clay stratum and the position of the sand aquifer.

During the spring rains, when the artesian pressures were highest, the slide moved. Mud waves, which formed in the river channel, were eventually washed down river. This action continued until the area between the highway and river was badly disturbed by secondary shear planes.

Berms were considered as a method of stabilizing the slide area, but were believed to be impractical due to the proximity of the river to the highway, and to the weakness of the soil near the river bank. Permanent drainage wells placed in the sand aquifer were believed to offer a safer and more positive solution to the problem. Actual measurements of artesian pressure several months after a slide movement occurred, indicated ground water pressure elevations which were 15 feet below the level of the gravel terrace, but which were approximately 25 ft higher than the level of the ground adjacent to the river. Since these values had been taken after movement had stopped and the roadway filled back to grade, the pressures measured were probably lower than those existing at the time of failure. A stability analysis of the section considering the artesian pressure conditions showed that by permanently lowering the artesian pressure elevation approximately 10 ft. below the measured values, a reasonable margin of safety could be attained. The design was completed on this basis.

The drainage wells consisted of 2½-in.-diameter well points with 40-mesh screen, embedded in the sand aquifer. The well points were connected by riser pipes to a collection header which was located at a specified depth. The spacing of the wells was varied with the header elevation to provide the necessary drawdown. Figures 9 and 10 show the locations of the drainage wells.

In-Place Treatment for Settlement

Embankments for which the stability requirements have been satisfied may still show considerable settlement long after construction.

In many instances a foundation is sufficiently strong to support the weight of a normal embankment without anticipating shear failures, but due to the compressibility of the foundation material, objectionable settlements of long duration may occur after construction. The eventual solution to eliminate or reduce the effects of settlement will depend on the amount and rate of settlement expected, the uniformity and continuity of embankment and foundation, the presence of points of discontinuity such as bridge abutments, and the quality of roadway constructed and results expected.

Normal Construction

Normal construction procedures are generally followed in cases where the settlement expected is minor, or is uniform for long sections with proper transitions at the ends of the section, and the quality of road is not affected. In these instances the settlement is expected but no precautions are taken to reduce or eliminate it.

Two-Stage Construction

Two-stage construction infers delaying the placement of the final pavement after embankment construction until the detrimental settlement has been eliminated. This method is used mainly where the total settlement expected after construction is not excessive but is reasonably rapid, where normal construction practices would result in objectionable differential settlements, where additional borrow for surcharge is not readily available, and where the schedules of construction permit such delays. The postponement of placing the final pavement may vary from one construction season to several years. Sometimes, if it is planned to use the roadway during this period, a temporary pavement may be placed, which can be readily built up as settlement occurs. Construction men generally refer to this method as permitting the section to "season."

Surcharge

An additional height is often added to the normal height of embankment as a surcharge to expedite the settlement. It is feasible only where the soft foundation is still sufficiently strong to permit the addition of the surcharge without failure. The actual amount of surcharge needed depends on the thickness and the time-settlement relations of the compressible layer, the normal embankment height, and the overall time available to complete the project. The additional weight of surcharge increases the amount of settlement obtained per unit time. The surcharge can be removed after the amount of settlement expected under the normal height embankment has been reached. If the compressible layer is deep or if the embankment is high, the amount of surcharge required, to noticeably affect the normal time-settlement relations, will be so great as to make the method uneconomical. Material used for surcharge may require double payment for handling unless use can be found for this quantity on other sections of the project after the surcharge is no longer needed.

A project in which a surcharge was used to expedite settlement was a portion of New York State Thruway west of Syracuse. The compressible layer was marl approximately 16 ft. thick, underlain by relatively firm silt and fine sand. The embankment height to grade was approximately 4 ft. For the normal embankment, the settlement expected was 0.8 ft., realized in approximately three years. A surcharge height of four feet added to the embankment reduced the time for the normal settlement to seven months. This relationship is shown plotted in Figure 11.

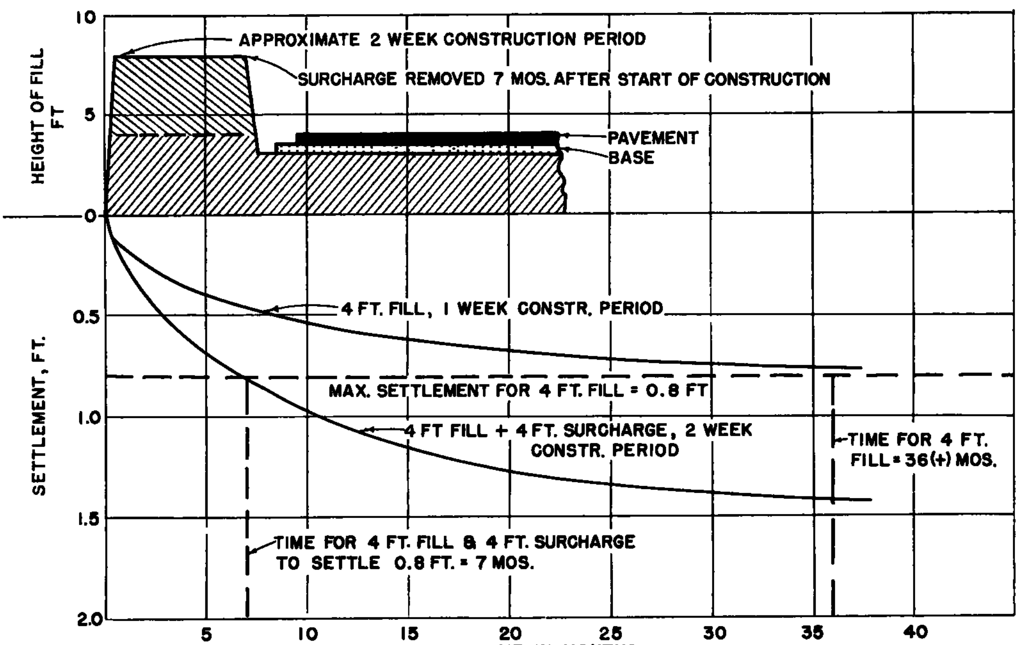

Figure 11. Using a surcharge to accelerate settlement.

Heavy Compaction

In some instances reasonably shallow foundation materials in a relatively loose state can be compressed by heavy pneumatic-tired rolling to eliminate any possible settlement due to the subsequent weight of an embankment. For effective results, such materials should contain granular sizes and be above the water table. A project in which such heavy rolling was used was a section of the throughway within the City of Buffalo. The section involved was approximately ½ mi. long and followed the alinement of the old Erie Canal. The canal had long been abandoned and through the years had been filled with rubble, sand, cinders, and other inert waste material, to a height of 10 to 15 ft. above the canal bottom. Gradeline for the project was established from 3 to 5 ft. above the existing rubble fill.

The foundation for this project was rolled with a heavy pneumatic-tired roller weighing from 30 to 45 tons, distributed over four tires. Ten to 15 passes of this roller caused settlement averaging approximately 1 ft. before stability was reached. Following this foundation rolling the embankment and pavement were constructed. The project has proven very satisfactory.

Another project so designed was reported at the January 1951 meeting of the Highway Research Board, and has been described in the Highway Research Board *Bulletin 42*.

In-Place Treatment for Stability and Settlement

Many soft foundations are critical both with respect to stability and settlement. In such cases the methods of treatment to satisfy one requirement may directly affect the other requirement. In many instances several individual methods are combined to satisfy the requirements for stability and settlement. The cases discussed below are typical of such projects.

Sand Drains

Sand drains provide a means of accelerating settlement and permit a rapid gain in strength for soft foundations, by providing drainage channels through which excess water may be removed. The increase in strength is often needed to permit the foundation to carry heavy embankments. Sand drains are especially effective in deep, soft and compressible soil formations. By proper design of drain sizes and spacings and by the use of controlled rates of construction, embankment heights heretofore impossible can now be built over deep soft foundations.

A careful analysis of stability and settlement is required for each project which makes use of sand drains. Published information on the design of sand drains is readily available, and this discussion will be limited to the description of a typical project.

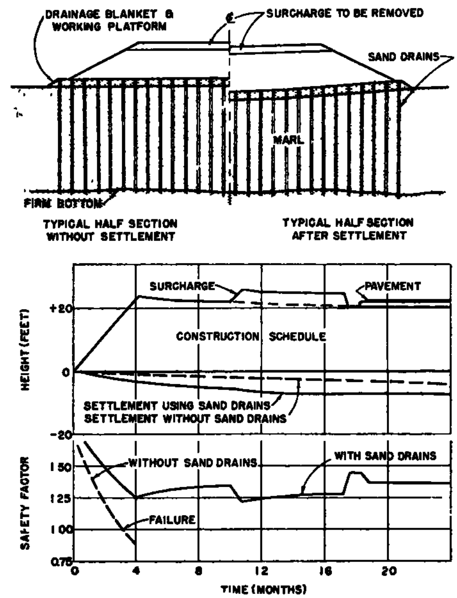

Figure 12. Vertical sand-drain installation.

Near Syracuse the throughway crosses a preglacial valley which has been filled with lacustrine sediments. The section requiring sand drains was approximately 2,000 ft. long and included a bridge requiring a high gradeline for clearance. The foundation consisted of a deposit of soft marl which varied in thickness from 10 to 65 ft., supporting an embankment 23 ft. high. Sand drains spaced between 8 and 12 ft. on centers with diameters of 18 to 22 in. were required to promote consolidation of the foundation at a desirable rate, and to develop the necessary increase in strength to support the embankment. A horizontal blanket of free draining sand and gravel was provided to be placed on the ground surface to carry away the water which was squeezed out by consolidation. Figure 12 shows the controlling conditions for a section of this project.

Without special treatment, the marl would not have supported the required height of embankment. By using both sand drains and a controlled rate of construction, the embankment and surcharge could be placed safely. The time required to eliminate settlement was reduced from ten years or greater to within the overall construction period. Provisions were also made for the installation of pore-water pressure measuring devices and settlement platforms to aid in controlling the rate of construction.

level by dikes and gates to provide a wild life refuge (see Fig 13).

The general considerations which affected the overall design were: (1) the water level in the area to be maintained at the minimum elevation required by the U. S. Department of Agriculture, from one to 2 ft. above the average ground surface; (2) a definite scarcity of granular material suitable for underwater placement, or for possible use as a drainage blanket in connection with sand drain installation; (3) an adequate supply of glacial till soil suitable for common borrow and satisfactory for above-water placement; and

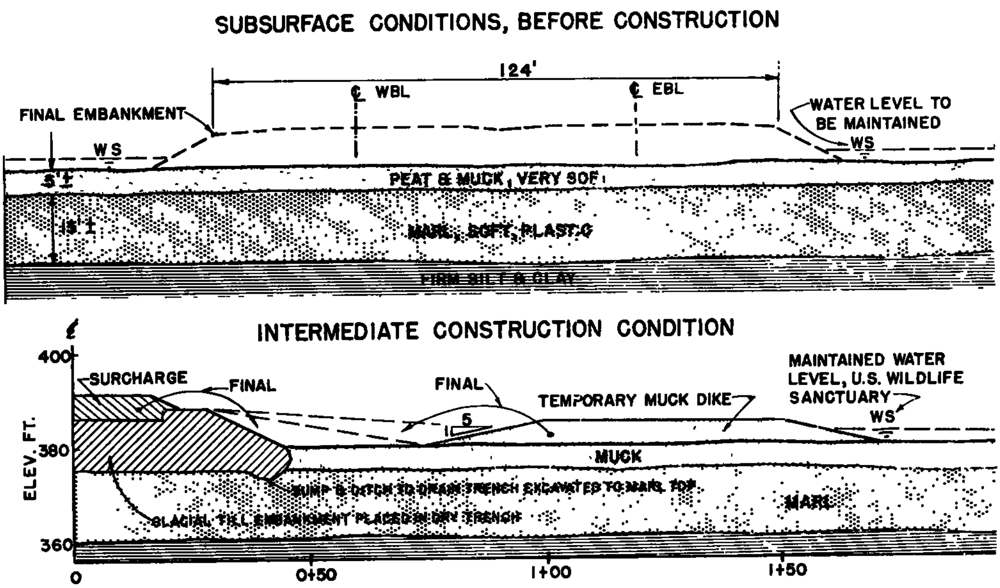

Figure 13. Combination of partial excavation and surcharge.

Partial Excavation and Surcharge

A project requiring the use of partial excavation combined with a surcharge height above the normal embankment included a section of New York State Thruway across Montezuma Swamp in west-central New York. The soil profile in this area consisted of a layer of peat and muck varying in thickness from 3 to 10 ft. but averaging 5 ft., underlain by a deposit of marl from 7 to 15 ft. thick, averaging 10 ft. The marl in turn was underlain by a deposit of silt and clay, which was reasonably firm. The normal water surface in this area was from 1 to 2 ft. above the ground surface, maintained to this minimum

(4) a time lapse of one year, and possibly two years, between the grading and paving operations.

Stability and settlement analyses made from the results of laboratory tests on representative undisturbed samples of the various layers indicated the following:

(1) The peat and muck layer would be unstable under the desired height of embankment, and would permit shear failures. Due to the approximately 150-ft. width of the embankment section used, such failures would take place along the sides, but would entrap the major portion of the peat and muck under the central section of embankment.

(2) The underlying marl layer would be stable under the weight of embankment and a moderate surcharge height, especially if the excavated peat and muck were spread beyond the embankment to add some counterweight.

(3) The peat and muck, being highly compressible, in combination with the marl would cause large detrimental settlements of long duration if left in place or entrapped by normal construction, and a satisfactory pavement could not be placed during the desired time interval.

(4) With the removal of muck, the settlement expected in the marl layer could be eliminated within a short time after construction, aided by a moderate surcharge placed on the normal embankment. A surcharge of such height to stabilize the muck and marl layers combined would cause failure in the marl layer.

Based on these considerations, the final design approved for construction involved: (1) removal of the muck, to be used as temporary dikes around the area of excavation and as eventual light berms adjacent to the future embankments; (2) dewatering of the embankment area by pumping, or where possible by gravity drainage to the nearby river, and building the embankment in the dry directly on the marl at the bottom of the excavated trench; (3) using glacial till as common borrow placed in layer construction, with the first layer of sufficient thickness to provide support for the equipment and to cause no disturbance of the marl layer; (4) continuing the embankment beyond normal height with a surcharge varying from 3 to 5 ft. high, depending on local requirements; and (5) surcharges to be maintained a minimum of one year.

CONCLUSIONS

The delineation of areas containing soft materials which may prove troublesome for highway embankments, and the tentative evaluation of applicable methods of solution should be undertaken during the early stages of the design of a project. In many instances, the treatment required for a soft foundation may be so costly as to justify a change in line and grade to avoid or minimize the effects of such foundations. A properly designed highway will reflect the least overall expenditure consistent with the quality desired, based on the proper evaluation of all the factors which affect design.

In the solution of a project involving soft foundations, the various methods discussed in this paper are considered as competing with each other. By the very nature of each problem, some methods do not apply. Others may offer equally satisfactory results. The choice of the best method is dependent on

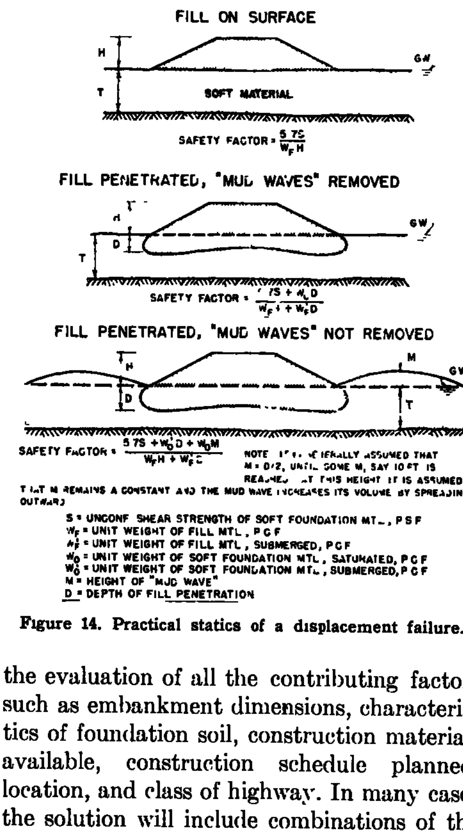

Figure 14. Practical statics of a displacement failure.

the evaluation of all the contributing factors such as embankment dimensions, characteristics of foundation soil, construction materials available, construction schedule planned, location, and class of highway. In many cases the solution will include combinations of the methods listed.

The minimum acceptable factors of safety noted in some of the project examples, contained herein, are not rigidly established values. The minimum factor of safety for any problem should be established by an evaluation of the reliability of the field and laboratory information and the probable accuracy of the design computations.

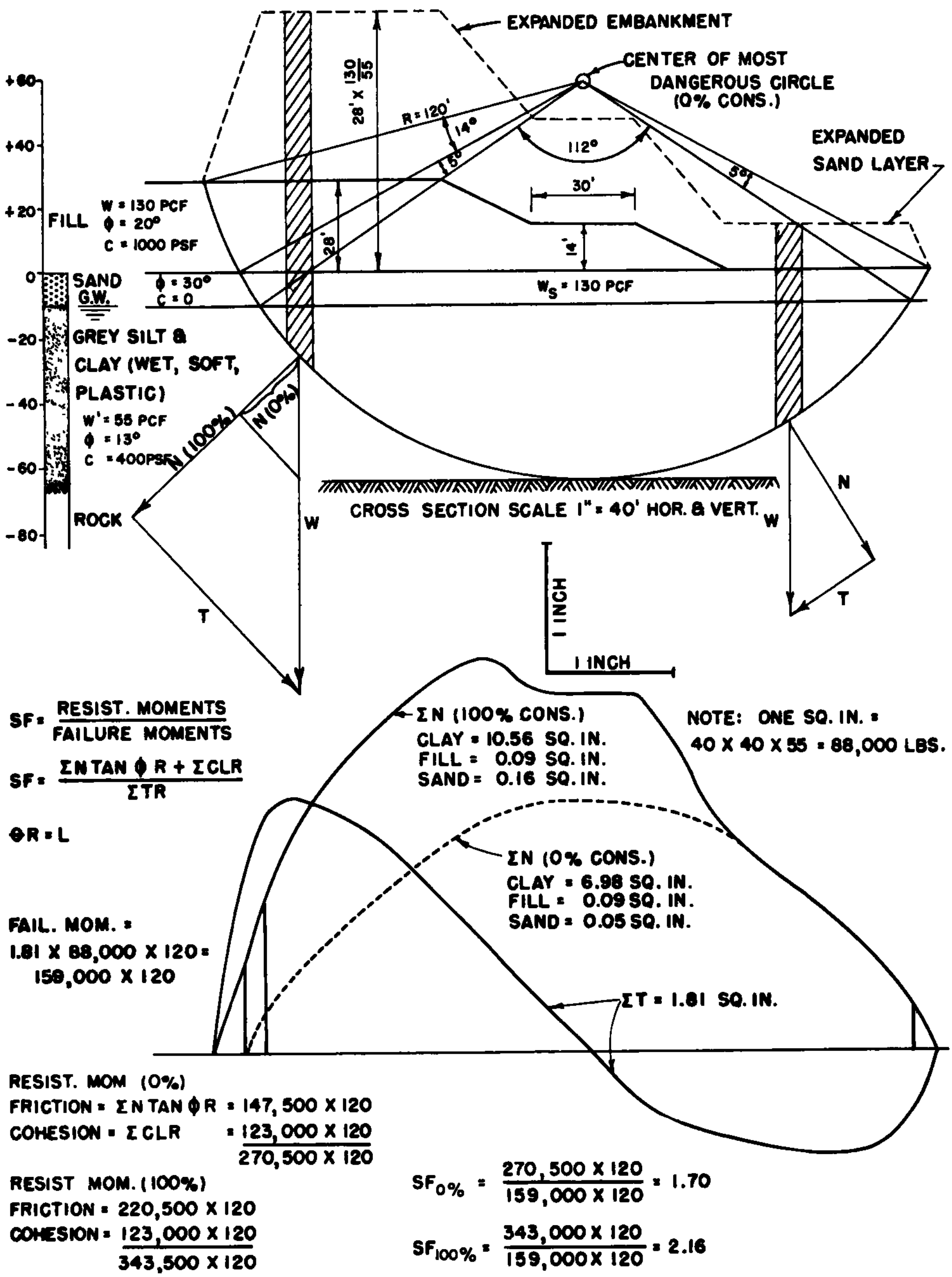

Figure 15. Typical rotational-failure analysis.

APPENDIX

Stability

Embankment failures generally follow one of two common types: the displacement or sinking type of failure and the rotational type of failure. The former consists of the embankment sinking as a unit in the soft foundation producing "mud waves" on both sides (Fig. 14). The latter consists of the embankment breaking and one section rotating downward and outward producing a mud wave on that side (Fig. 15). In design, all critical embankments should be checked for both types of failures.

Figure 14 shows a practical and convenient method for computing the stability of the displacement type of failure. Although the equations are simple and semiempirical, they have proven satisfactory when used for design purposes and for the investigation of existing failures. These equations may either be used to design embankments supported on soft foundations, or to solve problems involving fill and surcharge heights necessary to produce displacement.

Figure 15 shows a graphical method for the analysis of a typical rotational type of failure involving a fill with a berm. This method is more convenient than the usual methods of analysis and possesses several features that should be mentioned here. Attention is called to the method of reducing the densities of the fill and underlying layers to a common density, usually the submerged weight of the plastic layer. This is done by proportionately expanding the heights or thicknesses of the heavier materials.

It is important to note several considerations. The location of the center of the most critical circle without the berm differs from that with the berm included. The most critical circle is not always tangent to the firm bottom; the depth of the circle depends upon the general slope of the fill and the strength characteristics of the soft foundation. The location of the center of the most critical circle depends also upon the degree of consolidation of the soft foundation under the embankment load. Thus this method may be used to account for stability involving all degrees of consolidation. For simplification, however, the figure shows the same critical circle for 0 and 100 percent consolidation. Actually the circle shown is correct for 0 percent.

The location of the most critical circle for any condition must be determined by trial and error. There are, however, several rules too involved to mention here, to aid in locating that circle. Following these rules the most critical circle can usually be located in a very few trials.

The method illustrated can be used to accurately analyze rotational failures by accounting for such complexities as sloping water tables, variable densities and strengths, artesian pressures, degrees of consolidation and irregular ground surfaces.

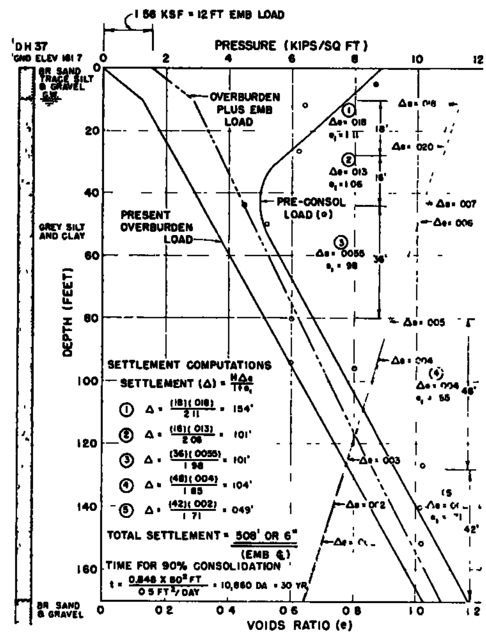

Figure 16. Typical embankment-settlement computation.

Settlement

A typical settlement computation is shown in Figure 16. This illustrates the consolidation characteristics of a Hudson-Champlain clay in the vicinity of Albany. The clay is precompressed at this immediate location, by dessiccation near the surface and probably by a layer that has been removed in the past.

The preconsolidation points from the laboratory tests have been plotted and averaged by a curve. The present overburden pressures computed from measured densities are also plotted. The embankment pressures at the various depths are added to the overburden pressures. The voids ratios and changes in voids ratios are plotted from the rebound and recompression characteristics of the consolidation data. The settlement amount and time is then computed using the common methods of increments.

DIFFERENTIAL THERMAL ANALYSIS

T. William Lambe, *Assistant Professor of Soil Mechanics, Massachusetts Institute of Technology*

● The student usually expects to spend the first part of his course in soil mechanics in learning of what natural soil is composed and the effect of the various constituents on the engineering properties of the soil. Like the uninitiated who reads any of the common books on soil mechanics, the student is surprised to find that soil mechanics is really *soil MECHANICS.*

The explanation of this unequal emphasis is twofold. First, the identification of the minerals which make up the soils with which engineers commonly work is seldom a simple job. This difficulty is faced particularly with the fine-grain soils, which furnish the most serious engineering problems. Second, even if the mineralogical composition of a particular soil were known in detail, it might be of only limited value, because of the scant knowledge of the relationship between the composition and the engineering behavior of soil. The narrow boundaries of knowledge on this complex relationship are being widened at an encouraging rate. The purpose of this paper is not, however, to discuss this progress, but to deal with the first problem—that of identification. This paper is limited to the identification of finely grained minerals in soils by the differential thermal analyzer.

Of all the procedures used for identification of the clay mineral species in the clays (X ray, electron microscope, staining tests, etc.) the differential thermal analyzer[1] appears to be the most practical for the soils engineer. It is not unreasonable to think that the differential thermal analyzer might become standard apparatus in well-equipped soil-mechanics laboratories. The analyzer will be of more value to the engineer as more is learned about the operation of the apparatus and the interpretation of the data. Also more information about the relationships between soil composition and soil behavior will increase the value of the analyzer.

The measurement of thermal changes in soils dates back to Le Chatelier, who, in 1887, published thermal data on halloysite, allophane, kaolin, pyrophyllite, and montmorillonite. Le Chatelier used a single platinum–platinum+10-percent-rhodium thermocouple for his work. Roberts-Austen is credited (Orcel, 1935) with the first use of the differential thermocouple for thermal analysis. The differential thermal analyzer has since been used by scientists, especially in England, France, Japan, and the United States, in the fields of metallurgy, ceramics, agronomy, geology, and engineering.

THEORETICAL CONSIDERATIONS[2]

The differential thermal procedure[3] consists of simultaneously heating a test sample and a thermally inert substance from room temperature to a little over 1000 C., and measuring any differences in temperature between the sample and the inert substance by means of a differential thermocouple. Since the inert substance undergoes no thermal reactions, the difference in temperature between it and the sample is a measure of the thermal reactions which occur in the sample. The analytical results are portrayed as a curve, sometimes called a thermogram, in which the abscissa is the sample temperature and the ordinate is the difference in temperature between the sample under test and the thermally inert reference. Analysis of the unknown consists of comparing the new curve with curves of known minerals.

[1] The reader should not infer that all natural clays can be completely analyzed by the thermal method. Occasionally supplementary tests (surface area measurements, chemical analysis, etc.) may be needed and are usually desirable for aid in the identification of a soil.

[2] See Tripp (1948) for a thorough treatment of the theoretical considerations.

[3] In the appendix is given an example of a differential thermal test and analysis.