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Mapping and Prediction of Limestone Bedrock Problems

FRANCIS T. ADAMS and C. WILLIAM LOVELL

ABSTRACT

Site evaluation for transportation routes and engineering structures located in limestone bedrock regions is often difficult. The highly irregular soil-bedrock interface and the presence of subsurface cavities make the results of such an investigation highly uncertain. A methodology has been prepared to assist the engineer with the planning of a thorough site evaluation that will minimize the degree of uncertainty. Preliminary studies consisting of the compilation of data from existing sources such as physiographic, engineering soils, pedologic, surficial and bedrock geology, topographic, drainage, and overburden thickness maps and reports are essential. A review of the current methods of remote sensing such as aerial photography, reflective and thermal infrared imagery, radar, radiometric data, and multispectral imagery is included. Combining this information with a knowledge of certain geologic indicators will identify areas that are more suitable to the siting of the route or structure under study. Preparation of a sinkhole density map is advocated for projects involving large areas. Areas of uncertainty can be further investigated employing geophysical techniques such as gravity, ground-probing radar, magnetics, seismic refraction, or electrical resistivity. The problems for which each of these techniques is appropriate are discussed. The compiled information can then be used as a guide to plan the location of subsequent borings.

When planning a transportation route or selecting a site for a structure in a limestone region, the engineer is faced with many unique problems that can often be solved only with experience gained from similar previous situations. However, it is unfortunate that even with such experience, many competent engineers are often fooled by the highly erratic nature of the soil and bedrock associated with karst regions. Selection of the appropriate techniques for site investigation and knowledge of their limitations are necessary to the correct interpretation of the collected data.

Two major engineering problems associated with the development of a residual soil weathered from carbonate rock are (a) the highly irregular soil-bedrock contact and (b) the frequent development of subsurface cavities that may or may not manifest themselves at the surface as sinkholes. The first problem may cause cost overruns when attempting to predict the amount of rock to be excavated in cut-and-fill operations for highways and makes the design and construction of deep foundations difficult. The second problem may lead to catastrophic collapse during or after construction, because of either alteration of the surface drainage or increased load from an embankment.

A methodology has been prepared that will enable the engineer to make a step-by-step evaluation of a proposed site. A review of the current techniques of remote sensing and geophysical exploration was conducted to determine the most useful methods for performing a site investigation in a limestone region.

METHODOLOGY

The proposed methodology of site investigation for locating highway routes or other engineering structures in limestone regions is shown in Figure 1. It considers a wide variety of techniques that have been used successfully for investigations in karst regions. Of course, none of the methods can be used in all situations, so the physical situations and types of problems for which each is best suited are emphasized in the following sections.

PRELIMINARY STUDIES

The first step shown in Figure 1 is to determine the physiographic characteristics of the area under consideration. Various references can be consulted for this purpose (1,2,3,p.318). From this information the engineer can immediately anticipate the types of problems that are likely to be encountered at the site under investigation. Many states have mapped the surficial soils on the basis of parent material, landforms, or some other general engineering classification. Indiana, Arizona, Kentucky, Illinois, and Rhode Island have prepared such maps on a county basis. A more comprehensive program of soil mapping is being carried out by the Soil Conservation Service of the U.S. Department of Agriculture. Although mapping has been for agricultural purposes, the pedologic soils can be correlated with specific engineering properties, and engineering data are often included in many of the more recent survey reports. Surficial and bedrock geology maps are good sources of information. State geological surveys, in cooperation with the U.S. Geological Survey, have prepared these maps at a scale of 1:250,000. The extent of a limestone area can be seen immediately from such a map. Drainage maps may also be available and are useful in locating sinkholes and other surface features of karst topography. Overburden thickness maps provide general information on the depth to bedrock at a given site. Stereoscopic aerial photographs are an invaluable tool for delineating features of a site and getting an initial feel for the area without actually visiting it. They should be available from state or local planning agencies. Small-scale satellite imagery (ERTS) can be obtained from the National Aeronautics and Space Administration for use in large site selection studies. Although solution features are difficult to distinguish because of the small scale, regional trends may be observed.

RECONNAISSANCE DATA

If the information discussed previously is insufficient or not available, a remote sensing survey may

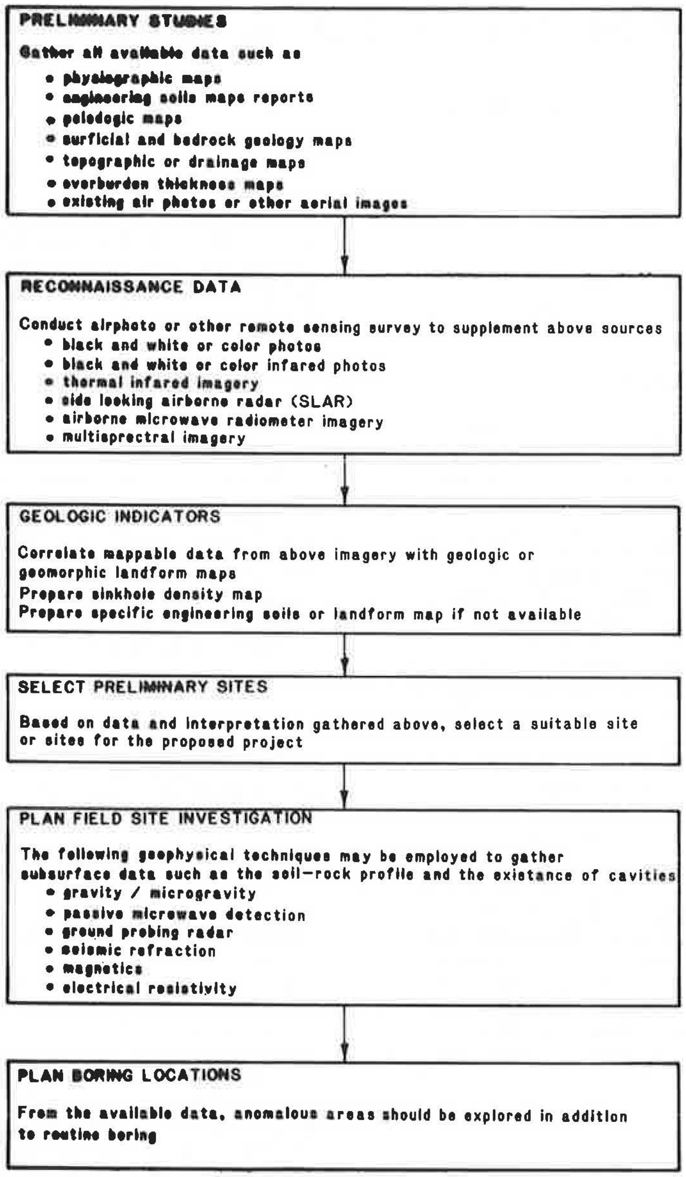

FIGURE 1 Methodology of site evaluation.

be conducted. The most common type of imagery is black-and-white or color photography, usually at scales between 1:24,000 and 1:62,500 (4,p.18). An experienced aerial photograph interpreter can delineate problem areas such as solution zones based on subtle topographic or reflectance features. Variations in topographic, drainage, and erosion characteristics, as well as vegetation, land use, and soil tones are important patterns that the interpreter looks for. Black-and-white or color reflective infrared photographs (about 0.7 to 1.1 μm in wavelength) have been used to map slight differences in soil or moisture content or both. Color infrared photographs are especially useful if a combination of films and filters is employed. An important clue for detecting solution activity in limestone is vegetative stress in the vicinity of a developing cavity. Such stress induces a change in reflectance (5). Near-surface features such as faults, fractures, and joints can also be delineated, which is important because they act as channels along which active dissolving takes place (4).

Thermal infrared imagery records emitted radiation in the 8 to 14 μm wavelength band. Sinkholes are linked to subsurface drainage paths, which have air and water flowing through them, often at temperatures different from the ambient temperature. Some studies have been performed using thermography to distinguish differences in soil texture with good results (6). Both Stohr (7) and Rinker (8) concluded that thermal infrared imagery cannot distinguish sinkholes by itself, and that stereoscopic aerial photographic coverage or ground control or both are required supplements.

Side-looking airborne radar (SLAR) has been used to map fracture and fault traces (5). Airborne microwave radiometer surveys can detect subsurface voids by recording anomalies in the radiometric temperature of the ground surface. From data taken over a previously mapped cave, Dedman and Culver (9) have concluded that there is a correlation between temperature lows and the location of subsurface voids. Although not perfect, the technique shows sufficient promise to warrant more research. A relatively new technique of remote sensing is multispectral imagery, which requires an airborne line scanner capable of recording data in several synchronous spectral bands. Good discussions are given by Wagner (10), West (11), and Mathews et al. (12). Multispectral imagery may be a valuable, if expensive, tool for delineating limestone residual soil boundaries.

GEOLOGIC INDICATORS

After enough information has been gained from the sources discussed, geologic and geotechnical interpretations are made to identify areas of high collapse potential. Williams and Vineyard (13) have compiled a list of geologic indicators of potential collapse in areas of limestone and dolomite in Missouri. They concluded that potential for catastrophic collapse is enhanced if the residual soil cover (a) is between 12 and 30 m (40 to 100 ft) thick, (b) contains the relict structure of parent rock, (c) has a low plasticity clay fraction (ML A-7-5), and (d) is draining poorly. Surface water that is diverted to the subsurface and a high density of natural sinkholes often indicate areas with a high potential for collapse.

For studies that encompass large areas, such as selection of transportation routes, preparation of a sinkhole density map may be helpful. Such a map was developed for Lawrence County, Indiana (Figure 2), and was prepared by simply counting the number of sinkholes per square mile from an existing drainage map. Correlation of sinkhole density with geologic formation is possible, and routes can be aligned so that areas of high sinkhole concentration are avoided.

Most of this information has addressed only the problem of detection of subsurface solution features in karst areas. Delineation of the soil-bedrock contact must be made with more sophisticated techniques and will be discussed in the section on field investigation.

PRELIMINARY SITE SELECTION

Selection of preliminary sites can now be made based on the following points. The area should (a) be located within reasonable distance of existing transportation routes; (b) be accessible to exploration and construction equipment; (c) have a minimum of solution activity; (d) have favorable drainage characteristics; (e) contain a minimum of potential solution zones such as caves, joints, fractures, or faults; and (f) be located in reasonable proximity to engineering construction materials.

FIELD INVESTIGATION

When the site or sites have been selected, a field investigation can be performed. Problem areas can be

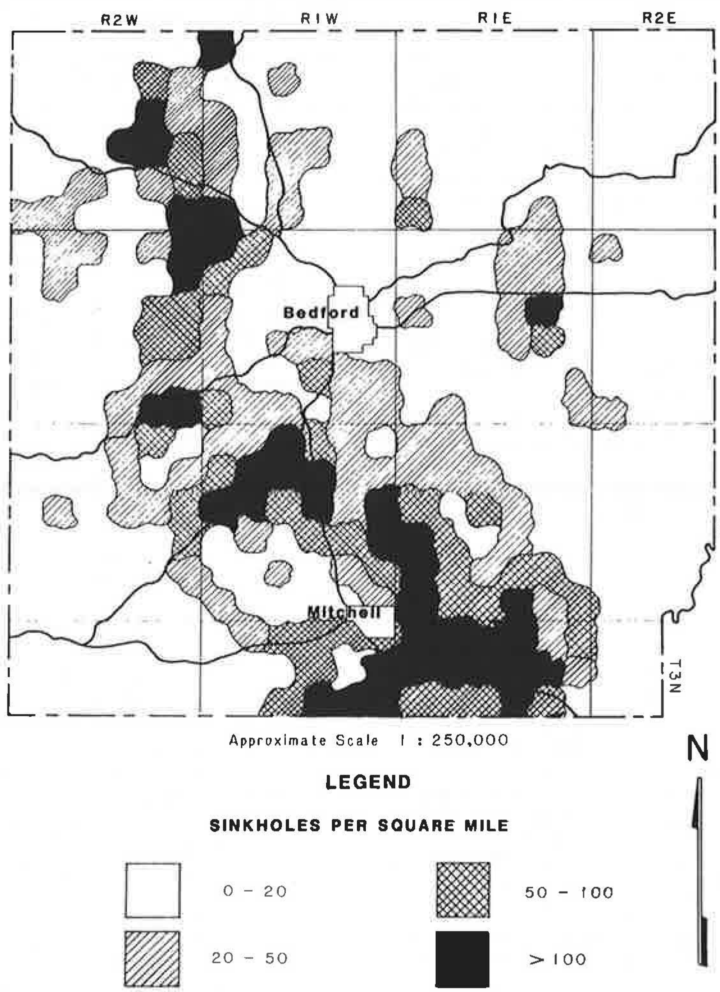

FIGURE 2 Sinkhole density map of Lawrence County.

identified from the preliminary reconnaissance work, and it is on these that much of the subsurface investigation should be concentrated. Geophysical techniques are efficient in limestone regions, and several techniques are discussed.

Gravity

Gravimetric techniques have long been used to detect anomalies in the earth's gravity field caused by differences in density among earth materials (14,15). Several studies have been conducted recently to evaluate the potential use of gravity methods for detecting subsurface voids (16-18), but the technique must often be coupled with another method to give satisfactory results.

Radar

Impulse or ground-probing radar has been the subject of recent studies (18,19). The reliability of this technique is questionable because of the wide variety in the electromagnetic properties of earth materials. However, if the equipment is chosen so that the output signal wavelength is comparable in size to the features that are to be detected, and if these features are not too deep, the technique can provide useful information about the subsurface profile.

Magnetics

Magnetic surveying is a technique that has shown limited potential in locating subsurface cavities (20). McDowell (21) describes a magnetic survey that was conducted over clay-filled sinkholes in chalk. McDowell concluded that although the method was successful in locating many of the sinkholes, the site must be free of any metal refuse that will interfere with the signal.

Seismic Refraction

Seismic refraction is a useful technique for delineating soil-bedrock interfaces in a limestone region, and it provides a quantitative measure of the depth to bedrock. Brooke and Brown (17) used refraction to locate a small cave and fault trace north of Santa Cruz, California. Although this is an instance where the technique was used for cavity detection, soil-bedrock profiling is the most efficient use of seismic refraction in limestone areas.

Electrical Resistivity

Much of the recent literature on detection of subsurface cavities in limestone has concluded that electrical resistivity methods are very efficient (17-19). Bates (22), in a comprehensive study of geophysical techniques used for detecting subsurface cavities, concluded that electrical resistivity methods give the best results. Good discussions of the details of the technique are given by Fountain (18), Clayton et al. (20), and Bates (22). Interpretation procedures can be found in Bates (22) and in the U.S. Army Corps of Engineers geophysical exploration manual (23). Problems associated with this method include outside interference with current, poor resistivity contrast between materials, dipping interfaces, and limitations due to equivalence and suppression in depth sounding (20).

Enough information should now be available for the engineer to plan an intelligent boring scheme to verify features identified by the remote sensing and geophysical surveys. Problem areas should be explored in detail, but no area should be completely ignored.

SUMMARY AND CONCLUSIONS

Problems that are unique to limestone bedrock and the residual soil derived from it require well-planned, thorough site evaluations if engineering structures and transportation routes are to be successfully constructed in such regions. Preliminary studies consisting of the compilation of data from existing sources of information are essential. Reconnaissance information may be required for large projects if existing sources are scarce. The preliminary information may reveal certain geologic indicators of potential problems in limestone, and the possible sites can be reduced to one or two areas. Locations where problems are anticipated should then be surveyed using a geophysical method or combination of methods. When the surveys have been completed and sufficient data have been collected, suspected problem areas can be verified with subsequent borings.

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Characteristics of Sinkhole Development and Implications for Potential Cavity Collapse

BYRON E. RUTH and JANET D. DEGNER

ABSTRACT

The results of investigations conducted in the karstic coastal plain of Florida indicate that the potential for cavity collapse is significantly greater in topographic lows where existing sinkholes, depressions, and other surficial features are lineated. Fracture traces and these lineaments can be established by interpretation of aerial photographs. In one case study it was found that almost all recent sinkholes occurred in the vicinity of the intersection of lineaments. A high vertical permeability to promote solution of carbonates and to induce piping of overburdened soils seems typical of many cavity and sinkhole systems. Collapse of overburden into cavities generally appears to be triggered by depression of the water table followed by wet surface conditions (e.g., well pumping and irrigation). Details of these investigations are presented with some emphasis on the lack of definitive geophysical and remote sensing methods for cavity detection. Several examples are presented to illustrate the difficulty in detecting subsurface cavities in conventional foundation investigations and in the analysis of the in situ conditions triggering the collapse. The subsidence of a portion of Interstate highway is discussed in relation to the observed in situ conditions.

Sinkholes and karst terrain are generally associated with the solution of limestone or, to a lesser degree, dolomite in regions with at least moderate rainfall. Arid or semiarid regions normally do not exhibit significant karst development, although any solution features observed in these dry regions are probably remnants from periods of more humid climatic conditions (1).

Nearly 15 percent of the contiguous United States has soluble rock at or near the surface (2). Puerto Rico is noted for its extensive solution features, particularly its pepinos. Gypsum beds in Canada provide a spectacular example of sinkholes. Even in the high velt near Johannesburg, South Africa, solution of dolomite rock occurs although the average annual rainfall is only about 35 in.

Karst terrain exists in many locations throughout the world, but it does not seem to have an extensive effect on highway alignment, site selection, foundation investigations, or performance of structures. Concern about the hazards associated with cavity collapse is mainly evident when a dramatic failure or a substantial economic loss occurs--for example, the Winter Park, Florida, sinkhole in 1981.

Sinkholes are continually developing in the more active regions of Florida. Many of these go undetected because they occur in sparsely developed regions. However, the probability of structural damage caused by cavity collapse will most likely increase as development and urbanization become more intensive.

There are numerous factors that influence the solution process and the development of subsurface cavities. It is essential that the soluble rock have a reasonable degree of water flow. This means that the rock must have adequate permeability and flow of water to allow replacement of water saturated with dissolved ions with less saturated water. Permeability has been characterized as either primary porosity (intergranular) or secondary porosity (fractures and joints) (3).

Primary porosity is dependent on interparticulate openings in the rock, shell beds, or coquina deposits. Numerous small solution pits may form because of downward flow of water in deposits located above the water table. Horizontal flow below the water table can produce small caves or irregular conduits (3).

Secondary porosity refers to flow along cracks, joints, and discontinuities that are generally caused by tectonic movements and stress release in the rock. Highly fractured zones in carbonate rocks yield greater quantities of water than nonfractured zones (4-6). Flow is often much greater than through interparticulate openings. Consequently, the rate of solution may be significantly greater, especially as the cracks, joints, and fracture zones become enlarged and the water flow rate increases. Water flow along bedding planes is also categorized as secondary porosity.

Local or regional variations in topography and stratigraphy also have a definite effect on water availability and flow through soluble formations. Slightly higher terrain elevations with slopes adequate to promote surface runoff and to minimize water retention have a tendency to reduce solution activity. Similarly, low permeability layers, such as clays, effectively reduce the downward flow of water but may concentrate water at discontinuities in the stratigraphy that can produce accelerated solution and sinkhole development. Terrains with perched water tables are usually not candidates for sinkholes except in zones of discontinuities where horizontal flow and secondary porosity are high.

Climatic variations and pumping from wells in shallow and deep aquifers can conceivably increase water flow and solution activity. This may be a long-term effect, but it is also important from the standpoint of increasing the potential for inducing roof collapse or raveling of existing solution cavities.

Variation in water table may also be attributed to the presence of cavities and conduits that provide a high downward permeability. There is evidence to suggest that the water table is often depressed in the vicinity of cavities. Observations of existing sinkholes provide a rationale for this: Sediment and debris washed into the throat of the sinkhole often provide a seal that allows water to accumulate until sufficient hydraulic pressure results in a piping failure that drains the pond. Some sinkholes undergo this process repeatedly. After the pond empties, the shallow water table is depressed in the zone surrounding the sinkhole. Other factors influencing cavity development and collapse are discussed in subsequent sections of this paper.

The detection of cavities in rock and overburden is usually a difficult task. Geophysical and aerial remote sensing methods are not always reliable because of in situ conditions and operational limitations of remote sensing equipment. Considerable emphasis has been placed on using resistivity and seismic (e.g., wave-front analysis) techniques. The results obtained using these geophysical methods have been quite variable; the investigation reported by Love (7) is a good example. Anomalies on thermal infrared imagery have been used to identify subsurface cavities, but air temperatures, wind, density of vegetation, and other adverse factors can affect the reliability of this technique.

The investigations presented in this paper were directed toward the use of aerial photography for identification of fracture traces and lineaments and their relationship to the formation of sinkholes. The results of these investigations suggest that the intersections of lineaments constitute potentially high-risk areas for cavity collapse. This is a subjective technique that depends on stratigraphy and local conditions. It is not intended as a method to pinpoint the exact location, size, or depth of a cavity.

CAVITY AND SINKHOLE CHARACTERISTICS

In general, the relationship between the size of sinkholes and the cavity is unknown before collapse. The size of observed sinkholes ranges from about 1 ft to more than 300 ft (0.3 to 91 m) in diameter and 150 ft (69 m) or more in depth. A study conducted in Alabama indicated that the average sinkhole is about 10 ft (3 m) wide, 12 ft (3.7 m) long, and 8 ft (2.4 m) deep (8). Sinkholes that develop in Florida are comparable although it is not unusual to have larger ones that fall in the 20- to 30-ft (6- to 10-m) diameter class.

Figure 1 shows a sinkhole that occurred in the parking lot of the Maracaibo Apartments in Gainesville, Florida, in 1982. Unfortunately, a new 1982 Oldsmobile parked directly over the cavity was buried in the throat of the sinkhole when the collapse occurred. The sinkhole and a portion of the parking lot were located in a lineated depression that was only several feet lower than the surrounding relatively flat terrain.

A recent investigation on Interstate 75 near Alachua just north of Gainesville, Florida, revealed a small cavity in the median at a depth of 45 ft (14 m) that extended to 67 ft (20 m). Borings were 5 ft (1.5 m) on center and only two borings encountered the cavity. Only 5.5 yd^3 (4.2 m^3) of grout were required to fill the cavity. Obviously, on the basis of the amount of grout and the spacing of bore holes, the cavity was narrow and lineated.

Adjacent to this small cavity was a zone of major

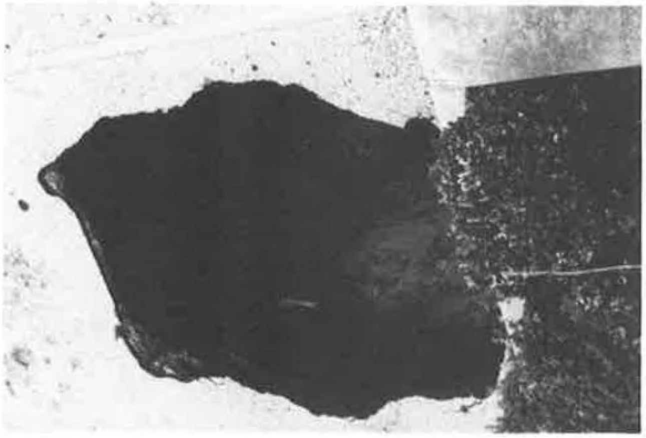

FIGURE 1 Sinkhole in parking lot at the Maracaibo Apartments, Gainesville, Florida.

FIGURE 2 Subsidence on I-75 near Alachua, Florida.

subsidence that was located in a topographic low. The southbound lanes of I-75 were closed because of concern about the possibility of a collapse after a depression and cracking occurred in the pavement (Figure 2).

The distressed area extended from the western portion of the median, west across the pavement, and about 30 ft (9 m) beyond the right-of-way to surface cracks, which were displaced vertically about 5 in. The observed cracks did not form a continuous circular shape but, when mapped (as shown in Figure 3), they revealed an elliptical pattern approximately 140 ft (43 m) wide that was estimated to be more than 180 ft (55 m) long. Borings down to 75 ft (23 m) indicated sandy soils with isolated clayey soil lenses. The water table was not encountered at this depth, even though there had been considerable rainfall for several months preceding the subsidence.

Soft gray limerock was encountered as shallow as 33 ft (10 m) and as deep as 46 ft (14 m) in the median. There were several exceptions where borings terminated at 76 ft (23 m) and 101 ft (31 m) showed sands exclusively. To the west the depth to rock generally increased to 66 ft (20 m) or more. One boring taken in the lowest area southwest of and outside the failure boundary indicated rock at 149 ft (45 m). The surface features, subsurface soils, and depth to bedrock indicated that portions of the subsidence area are located over a narrow solution valley or fracture zone that has been filled with sands and stratified lenses of clay.

Investigations are currently being conducted using resistivity, ground-penetrating radar, and a cone penetrometer in an attempt to identify the source of the problem. However, it is hypothesized that it is a deep-seated failure, induced by localized piping into a cavity, which resulted in the overstressing of overlying soils.

The foundation investigation performed for construction of the Chemical Engineering Building at the University of Florida missed a cavernous cavity system. During excavation of the elevator shaft, a void, which led to the discovery of the cavity, was encountered. A spelunker mapped the cavity (Figure 4). The potential for missing small or narrow lineated cavities with borings for foundation investigations is greater than one might expect.

Sinkholes are usually formed by the collapse of

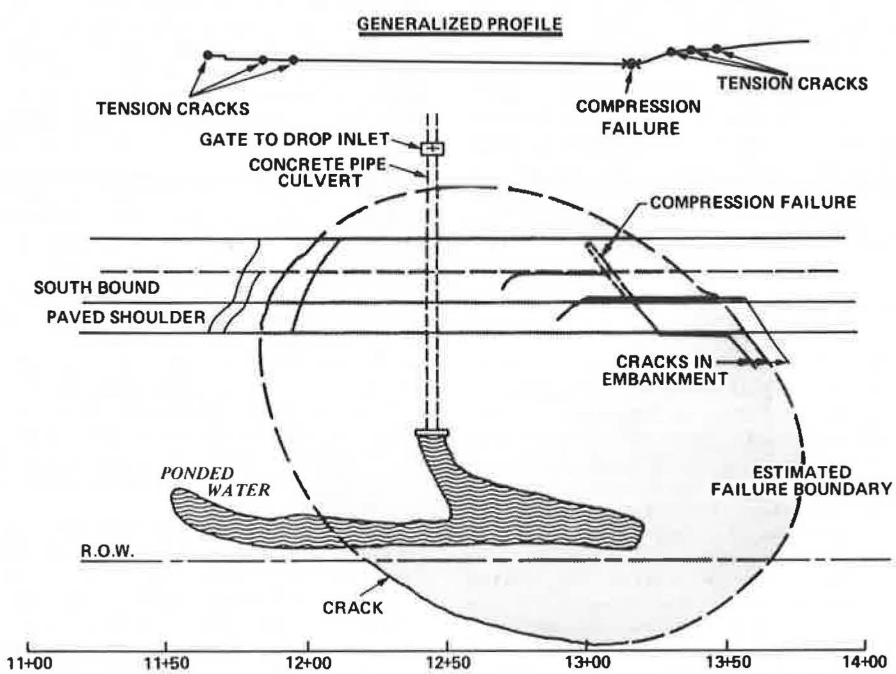

FIGURE 3 Subsidence on I-75 (MP 396.2).

CHEMICAL ENGINEERING BUILDING

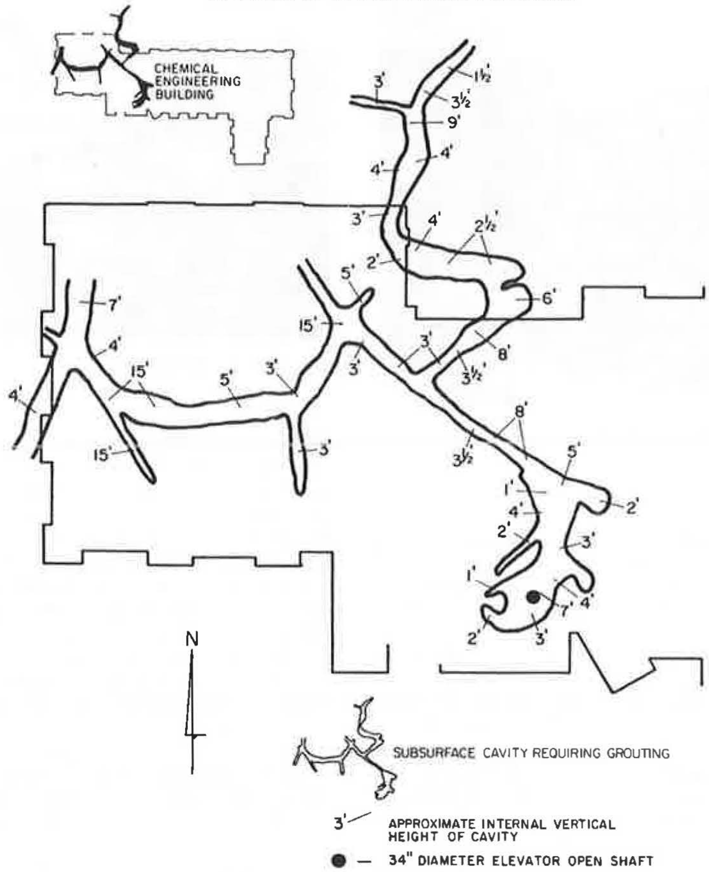

FIGURE 4 Subsurface cavity encountered during excavation of elevator shaft.

rock roofs or by raveling of unconsolidated overburden into an existing cavity (9). Sinkholes often enlarge shortly after initial collapse as a result of localized slope failures or raveling or both. Small vertical sinkholes [e.g., 15 ft (4.6 m) or less in diameter] may retain their initial shape, because of arching action, unless raveling of the vertical faces is pronounced.

The detection of cavities, particularly small ones, is generally not possible unless extensive effort is expended in a subsurface exploration program. However, there are indicators that assist in the pinpointing of hazardous zones and thereby confine subsurface exploration investigations to smaller, more well-defined areas.

REMOTE SENSING INVESTIGATIONS

Land satellite (LANDSAT) imagery and aerial photography have been used to investigate the reliability of using lineaments (photolinears) to identify the most probable locations for sinkhole occurrence. Aligned depressions or dolines and their intersections indicate more active areas that have a greater potential for cavity collapse. Satellite imagery is valuable for interpretation of major depositional and structural features. However, because of its limited resolution, it is not as useful for doline mapping as is high-quality aerial photography.

The identification and mapping of depressions, old sinkholes, and other features from aerial photography is relatively easy except where features are masked by dense vegetation. An investigation conducted near Pierson, Florida, provides a comparison of LANDSAT, high-altitude color infrared, and low-altitude aerial photography (10). The study area is located in the Crescent City-Deland Ridge Section of the Central Lake District. The area is typified by sand hills with summits generally 80 to 100 ft (24 to 30 m) in elevation. Pliopleistocene sand and shell rest directly on the Florida Aquifer (11). Although limited in scope, the study indicated that the intersection of lineaments corresponded to de- pressed zones that contained one or more sinkholes.

Another site near Dover, west of Plant City, Florida, was selected for investigation. During a period of subfreezing temperatures in January 1977, strawberry farmers began pumping from wells to protect their crops from frost damage. The drawdown in an observation well reached almost 10 ft (3 m) in about 4 days, but drawdown in other wells was reported to be 60 ft (18 m). The potentiometric elevations at the observation well of the Riverview Weather Station compared with air temperatures are shown in Figure 5. Numerous sinkholes were reported to have developed in strawberry fields and citrus groves. Twenty-two sinkholes were documented by Hall and Metcalfe (12).

This area lies physiographically in the Hillsborough Valley of the Ocala Uplift District. It is described by Brooks (11) as having sluggish surface drainage and many karst features in a valley from which much of the surficial clastic sediments have been removed. The stratigraphy consists of 10 to 20 ft (3 to 6 m) of surficial sands and 20 to 50 ft (6 to 15 m) of clayey sands, phosphoritic sand and

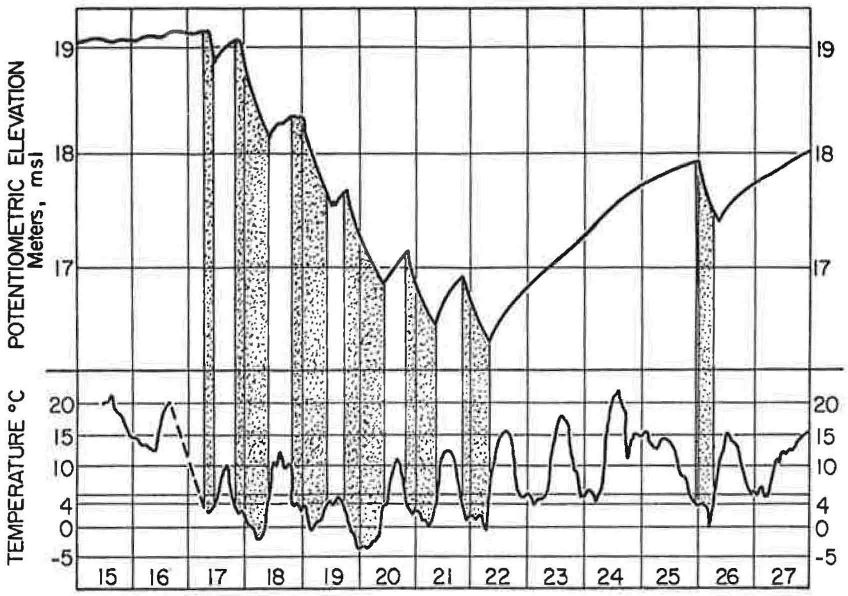

FIGURE 5 Comparison of hydrograph from proposed Thonotosassa well field and thermography from Riverview weather station, Jan. 1977 [after Hall and Metcalfe *(12)*].

gravels, and clayey limestones and dolomities of the Miocene Hawthorne formation underlain by limestone.

Various types of imagery were studied, but Agricultural Soil Conservation Service (ASCS) photographs were used to develop lineament in the study area. The ASCS photographs selected for interpretation were INDEX ASCS-2-68DC Item 5, 1-21-68, BQF-4JJ 32-37, 84-89, and 153-159.

The lineaments were mapped primarily from soil and vegetation tonal alignments, and the linears formed by a few old sinkholes. The lineaments were transferred to a base map that identified the location of the 22 sinkholes. Figure 6 shows this map that illustrates the location of wells, strawberry fields, and new sinkholes in relation to lineaments.

These lineaments are similar to the northeast and northwest trends mapped by Vernon (13) in 1951. The majority of the sinkholes occurs at or near the intersection of lineaments and in slightly lower or depressed areas of the terrain (14). The area south of FL-574 had the greatest activity (17 sinkholes), which is probably attributable to greater drawdown and intensity of overhead irrigation for freeze protection of crops.

Cavity collapses that occurred during construction of the runway for the new Southwest Regional Airport in Lee County provided an opportunity to evaluate photolinears (lineaments) in a different geologic setting. The runway is bisected by a scarp at 25 ft (7.6 m) elevation, which delineates the

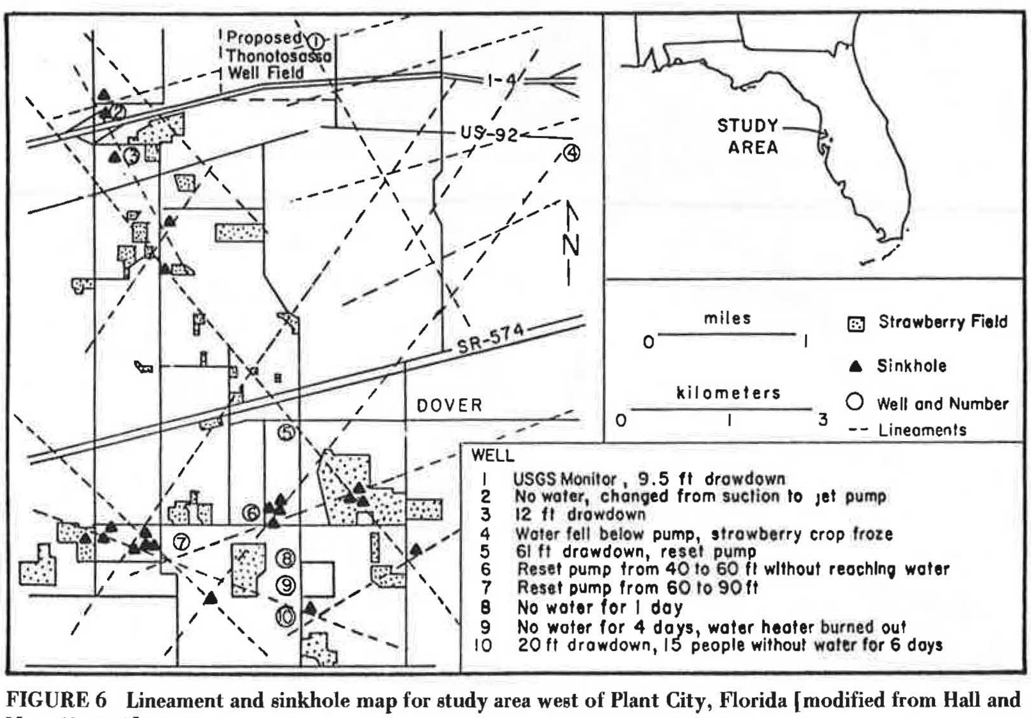

FIGURE 6 Lineament and sinkhole map for study area west of Plant City, Florida [modified from Hall and Metcalfe *(12)*].

boundary between two physiographic provinces, the Immokalee Rise to the east and the Southwestern Slope to the west (15,16).

The Immokalee Rise is characterized as a broad area of land slightly higher than that surrounding it, and typically 30 to 40 ft (9 to 12 m) in elevation above mean sea level (11). This area consists of flatwoods with some wet prairie and cypress swamp. Soils consist of fine sands, and sandy loam and clay, with numerous pockets of shell sands and muck. Permeability of the sandy clay is about one-tenth of that occurring in the fine sands (6 to 20 in./m). The depth of unconsolidated soils (about 20 ft or 6 m) is relatively thin. Water table levels between wet and dry seasons fluctuate about 2 to 15 ft (0.6 to 4.6 m) below the land surface.

LANDSAT, false color composite, high-altitude color infrared, and Florida Department of Transportation black-and-white panchromatic aerial photographs (March 1979) at a nominal scale of 1:24,000 were used for interpretation of regional and local terrain features (17). Interpreted photolinears were

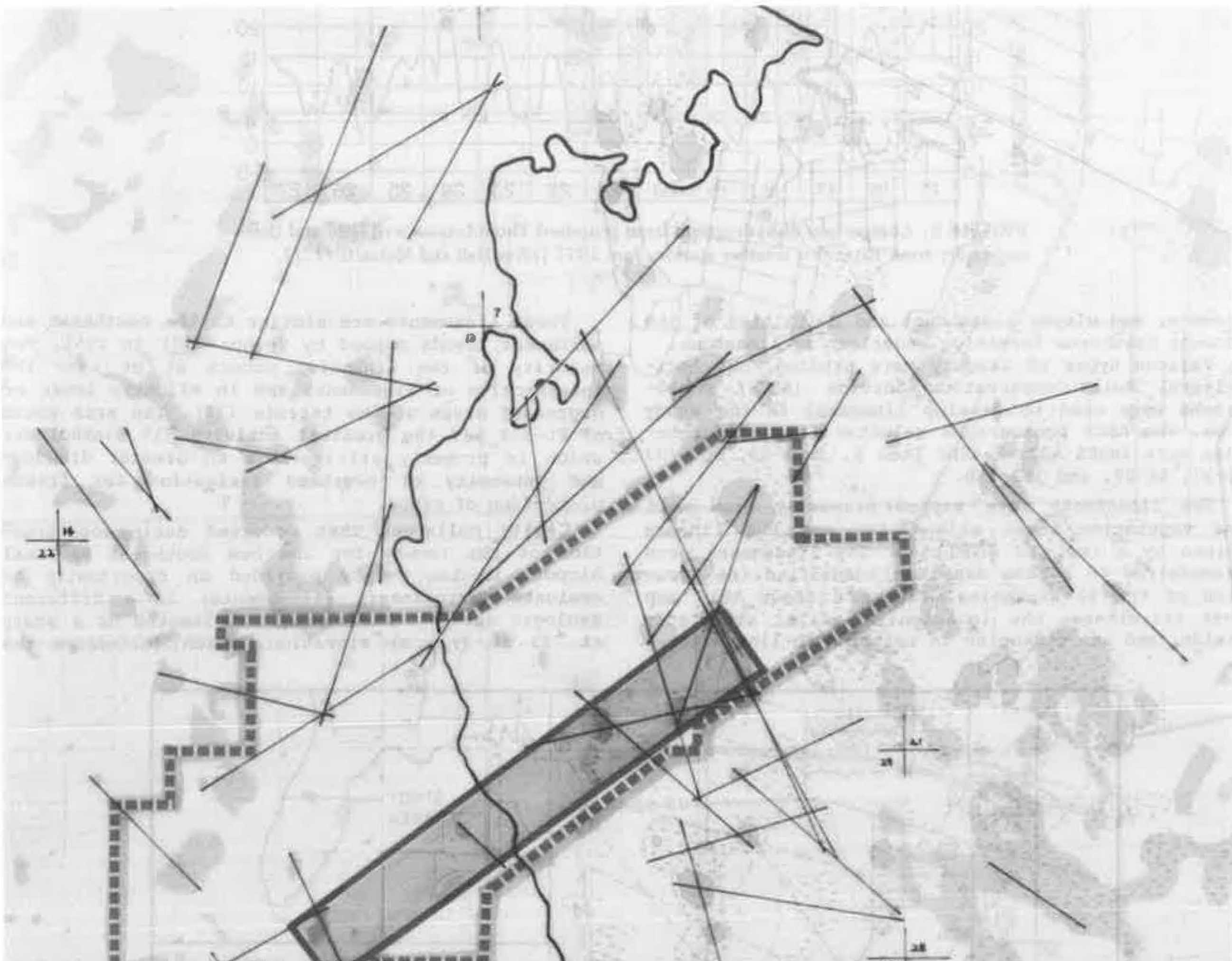

FIGURE 7 Photolinear map of the Southwest Regional Airport site and vicinity, Lee County, Florida.

transferred to a U.S. Geological Survey 7.5-min topographic quadrangle map. Figure 7 shows that the northeastern portion of the airport runway is located in a zone of the Immokalee Rise that has a predominance of photolinears and intersections of these linears. It was also obvious that numerous depressions and wet, swampy areas are located within this zone.

The potential for subsurface cavities and sinkhole development, based on the mapped photolinears, was interpreted as high in the northeastern segment of the runway and relatively low in the southwestern portion. These results correlated well with the sinkholes that had developed during construction. Information about the exact location of these sinkholes was not available, so a direct correlation to photolinear intersections was not attempted.

SUMMARY

Recent subsidence and collapse features in the karstic coastal plains of Florida are generally found in association with old sinkholes and depressions in topographic positions that are low relative to the surrounding areas. Overburden often consists of either permeable soils or more impermeable soils with localized zones of high vertical permeability.

Subsurface cavities and smaller solution channels may provide an excellent conduit for localized removal of surface water and groundwater, depending on the stratigraphy and availability of water from the surrounding area. It has been observed that the water table is often depressed in the vicinity of cavities. This is, to a degree, substantiated by the response of old sinkholes that repeatedly fill until washed-in sediment plugging the hydraulic connection is removed (e.g., by piping). This empties the pond and depresses the shallow water table.

The frequency of sinkhole occurrence has generally been associated with the depression of potentiometric levels resulting from drought or removal of groundwater by pumping of wells. Previously submerged soils overlying subsurface cavities are subjected to an increase in stress because the soil is no longer in a buoyant condition. Noncohesive soils may have a tendency to ravel or spall, eventually failing by collapse or through the continued erosion of soil. The latter case lends itself to hourglass erosion and the formation of a more conically shaped sinkhole.

Cavity collapse seems equally dependent on the availability of surface water to increase the weight of the overburden and, consequently, stresses in the dome of the cavity. The potential for piping to weaken the system is equally high, particularly where sufficient permeability exists or small conduits have been formed. Rainfall and irrigation water that flow on the surface or migrate in the permeable shallow subsurface soils toward topographic lows containing cavities appear to be a major factor in triggering collapses.

The low probability of locating cavities during conventional foundation investigations suggests that more emphasis should be placed on the use of suitable geophysical and remote sensing methods. In situ conditions such as depth to water table, very dry soils, major irregularities in the soil-rock contact surface, and depth and size of cavity influence the selection and use of these methods. Furthermore, it is imperative to have at least some knowledge of the stratigraphy, soils, and geologic conditions before conducting any subsurface exploration program.

Foundation investigations can best be planned using information derived from remote sensing methods that is supplemented by available hydrologic and geologic maps and literature, particularly when alternate sites are being evaluated. Interpretation of low- to medium-altitude aerial photographs for analysis of terrain is essential for identifying surficial and certain subsurface anomalies that affect the location and type of exploration equipment used to evaluate foundation conditions.

The results of investigations in Florida's karst terrain suggest that fracture traces and other photolinears (dolines) are good indicators of the most active solution conditions. The risk of cavity collapse or encountering a cavity along these linears, particularly at the intersection of a photolinear, is quite high. Other investigators have demonstrated that these intersections also provide greater permeability and well productivity.

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Corrective Procedures for Sinkhole Collapse on the Western Highland Rim, Tennessee

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ABSTRACT

Sinkhole collapse poses an increasing problem for the engineer and geologist. This is partly because no systematic procedure exists for repairing such collapses. Any successful repair procedure for use in the karst limestones of middle Tennessee should include efforts to identify and control water movement into the collapse site. A geologically based approach to collapse repair that takes advantage of the causal and correlative factors responsible for nearly 100 collapses in the study area and an extensive fluorescein dye-tracing program are described. Corrective procedures deal not only with the collapse itself but with the rerouting, to the extent possible, of runoff and groundwater at the collapse site. A high-permeability, graded rock fill is placed in the collapse, and water movement into the site is minimized.

Karst topography in general poses several highway engineering problems, the most serious of which is collapse. Within the last decade, contract costs for correcting sinkhole collapses involving bridges and highways in Tennessee and Alabama exceeded $10 million unadjusted for inflation (1). Significant progress has been made in the last decade in identifying the geologic and hydrologic factors and construction practices that contribute to sinkhole collapse (2,3). Field research to date deals primarily with recognition, detection, and prediction of sinkhole collapse (4,5). Little published work deals with correcting sinkhole collapse.

Sinkhole collapse is defined here as the nearly vertical downward movement of some portion of either the bottom or flank slope of the karst depression, along a defined failure surface, into an underlying void (cavity, solution-enlarged joints, or joint intersection).

A geologically based engineering approach to correcting collapse in the karstic limestones of the Western Highland Rim of Tennessee is described. Primary emphasis is on recognizing and effectively countering the geologic and hydrologic factors responsible for collapse. Secondary emphasis is on a collapse repair technique. Careful attention to surface and subsurface geologic and hydrologic conditions at the collapse site should ensure successful repair efforts.

GENERAL GEOLOGIC SETTING

The karst topography of the northern part of the Western Highland Rim is developed on the gently dipping west-northwest flank of the Nashville Dome (Figure 1). Dominant karst landforms include dolines (sinkholes), disappearing streams, and an extensive cave network. The karst landscape is underlain, from oldest to youngest, by Warsaw, St. Louis, and Ste. Genevieve limestones (formations). Karstification is most evident in the St. Louis and Ste. Genevieve limestones, particularly where these crop out north and east of the Cumberland River. Table 1 gives a generalized geologic description of the karst units for engineering use.

The carbonate bedrock underlying the study area has three major joints sets oriented N70°E to N80°E, N20°E to N40°E, and N20°W to N30°W. The bedrock dips about one-half of a degree to the northwest. Joints in the bedrock play a primary role in controlling groundwater movement. The low-angle dip of the bedrock to the north and northwest is a secondary factor influencing groundwater movement.

The limestones of the study area differentially weather into clayey, cherty residuum. This residuum typically consists of 0-70 ft of highly weathered, buff to red, angular to blocky, porous chert pebbles and cobbles incorporated in a yellow to red highly mottled clay matrix (CH-CL). The extent and particle-size distribution of the chert vary within and be-

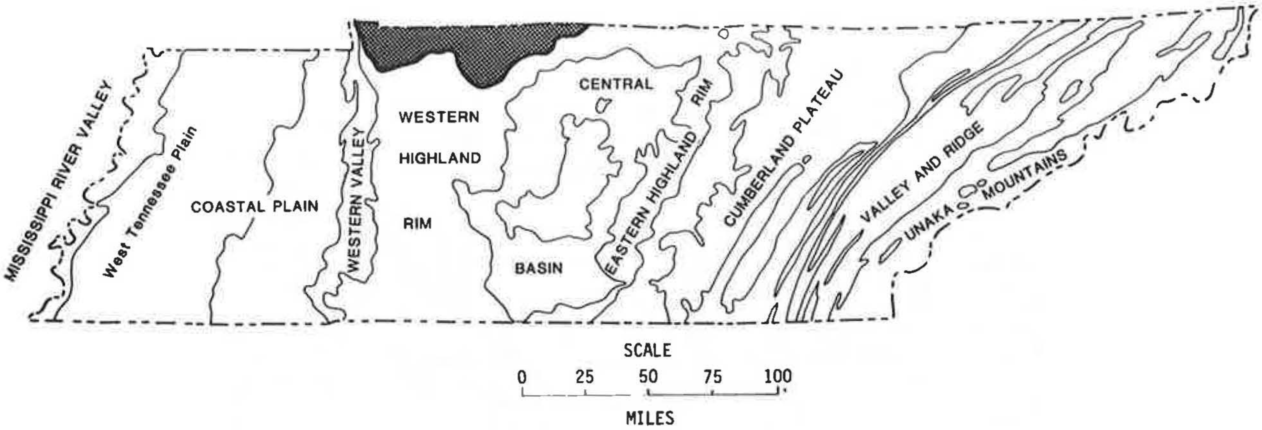

FIGURE 1 Physiographic map of the karst area of the Western Highland Rim.

TABLE 1 Generalized Geologic Description of Karst Units for Engineering Use

Formation	Characteristics	Karst Expression (sinkholes/ square mile)	Collapse Risk
Ste. Genevieve limestone	Gray to yellow-brown, fine- to coarse-grained, thin- to thick-bedded, thickness: 0-90 ft	0-25	Moderate to severe, particularly on uplands near Red River
St. Louis limestone	Brownish-gray to yellowish-brown, fine- to coarse-grained, thin- to thick-bedded, thickness: 0-200 ft	0-40	Moderate to severe
Warsaw limestone	Brownish-gray to yellowish-brown, fine- to coarse-grained, thin- to very thick-bedded, thickness: 0-200 ft	0-5	Low

tween the residuum units derived from each formation.

Loess, composed of brown to tan silt (ML-MH), caps many hilltops and some gently sloping surfaces surrounding many karst depressions north of the Cumberland River. The loess is up to 4 ft thick. Under the influence of gravity, loess is transported downslope by sheet wash and channelized flow, resulting in the mixing of loess with the underlying residuum. This mixing produces a third unconsolidated unit--silty colluvium. The silty colluvium consists of 0-15 ft of brown to tannish yellow clayey silt (ML-CL) within which occur minor quantities of sand- and pebble-sized chert derived from the underlying residuum.

All of the nearly 100 collapses examined in the study area occurred in the soil overlying solution-enlarged joints in the bedrock. The boundary or contact between the soil and bedrock can best be termed pinnacle in nature. Soil-bedrock contacts in the area have a depth-to-bedrock variation so great that 10 to 40 ft of difference in the depth to soluble rock is common at sites less than 50 ft apart horizontally.

GENERAL HYDROLOGIC SETTING

The general hydrologic setting is one of the most important factors affecting the stability of karst depressions. Drainage is principally in the subsurface and consists of an interconnected system of caves, enlarged vertical fractures, and smaller cavities developed along bedding planes. The open joints (vertical fractures) in the carbonate rock extend downward from the surface and are open in varying degree to at least the elevation of the bedrock channels beneath the alluvium of the Cumberland and Red rivers. The Cumberland River serves as the major discharge point for groundwater moving in the subsurface.

The water table surface, to the extent it exists in the study area, neither closely "mimics" the landscape, nor is it always laterally continuous (Figure 2). The well-developed fracture system in the limestones; the great variation in fracture permeability within the joints, caves, interconnecting passages, and bedding planes; and fluorescein dye tests suggest that an open groundwater system exists in the karst terrain of the Highland Rim.

Fluctuations in groundwater levels are of special interest because of their observed effect on the shear strength of the soil occupying the voids in jointed rock and bridging the pinnacles along the soil-bedrock contact. Fluctuations in the local water table are of two general kinds--seasonal and pulse. The water table drops during late summer and early fall and rises in late winter and early spring. Pulse loadings, from a single storm event or series of storm events, locally can trigger a rise in static water level of as much as 10 to 50 ft above its elevation before the storm events. Rapid fluctuations occur in areas where the system is less open and groundwater movement is restricted to smaller joints and fewer solution cavities. The greater the fracture permeability in a localized area, the smaller will be any local groundwater level fluctuations.

Sinkholes in the area play a dual role. First, the sinkhole acts as a collecting basin for runoff. Second, surface water, collected in the depression, drains vertically through the residuum into bedrock fractures and downward to recharge the water table. The water table is directly connected to the Cumberland and Red rivers by seeps and springs associated with solution openings above and below river level.

Field data plus limited data from other sources

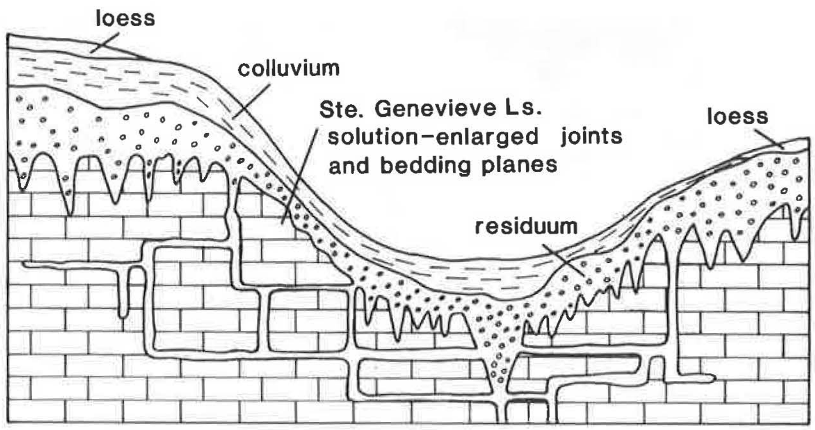

FIGURE 2 General geologic conditions controlling groundwater occurrence in the study area.

(6-8) indicate that water as well as air may be found in some joints and small cavities along bedding planes. Significant quantities of groundwater do occur in the clayey chert-rubble zones that mark the very irregular and pinnacled soil-bedrock contact. A water table occurs near the top of the carbonate bedrock and will also appear in the cherty clay residuum where these deposits are particularly thick along the very irregular soil-bedrock contact. The local water table has a low hydraulic gradient during most of the year. Rapid increases in the hydraulic gradient occur locally in the study area during periods of heavy or prolonged precipitation when recharge through sinkholes exceeds the ability of the groundwater system to transmit it, via springs, to the Cumberland and Red rivers. Water, moving along the most open fractures and bedding planes in the general direction of the dip of carbonate bedrock, is locally forced under excessive hydraulic head to back-flood the nearest available solution-enlarged joints or joint intersections. Substantial volumes of groundwater collect where joints and bedding planes intersect major solution-enlarged joint sets. The water, temporarily forced to rise upward and laterally through solution-enlarged joints and bedding planes, frequently encounters solution openings containing sediment. Core samples from such settings have shown the sediment to be saturated, soft, and weak. The extent of this back-flooding is primarily a function of joint permeability and hydraulic head. Back-flooding is a short-lived phenomenon. Within hours, or at most a few days, back-flooding of the rock fractures, bedding planes, and caverns is eliminated as a result of base flow into the Red and Cumberland rivers. The hydraulic gradient is then lowered to its pre-storm value.

Groundwater recharge occurs primarily through sinkhole swallets and solution-enlarged joint sets. The amount of recharge taking place directly through the residuum is not known although some soil units have such a high clay content as to preclude significant amounts of recharge. Static water levels in the water table range from 50 to 125 ft below ground level in the study area (7). This wide range reflects the erratic and discontinuous nature of the groundwater system.

COLLAPSE CRITERIA AND MECHANISM

A brief synopsis of the geologic and hydrologic criteria associated with nearly 100 collapses in the study area is given here so the reader can evaluate the remedial measures taken to correct sinkhole collapse. More detailed discussions of these factors can be found elsewhere (4).

All sinkhole collapses occurred in the soil, which exists as a mantle above a pinnacled bedrock surface. Soil thickness varied from 0 to 70 ft, reflecting differences in weathering intensity and the resulting irregularities in the pinnacled soil-bedrock contact. The relation between soil characteristics and sinkhole collapse is largely a function of texture, frequency of wetting, vertical and horizontal permeability, and the cumulative effects that these factors have on the shear strength of the soil.

Collapse sites are underlain primarily by the Ste. Genevieve and St. Louis limestones of Mississippian age. Although more collapses occurred in the soil overlying the Ste. Genevieve than in that over the St. Louis limestone, lithologic differences within bedrock formations appeared as great as the differences between formations, especially in the case of the St. Louis limestone.

Collapse occurrence correlates with three systematic joint sets in the study area (N70°E to N80°E, N20°E to N40°E, N20°W to N30°W). Fifty-eight collapses occurred in sinkholes with a long-axis orientation (bearing of the maximum depression diameter) parallel to the N70°E to N80°E joint set and 30 collapses occurred in karst depressions with a long-axis orientation parallel to the N20°E to N40°E joint set. Nine collapses occurred in sinkholes with a long-axis orientation parallel to the N20°W to N30°W joint set. The N70°E to N80°E joint set showed larger solution openings than either of the other two joint sets. The solution enlargement of the N70°E to N80°E set averaged 1 ft and ranged between 0.25 and 3 ft. The width of the solution-enlarged N70°E to N80°E joint set varied little between the St. Louis and Ste. Genevieve limestones. The tendency for collapse to occur in sinkholes with the long-axis orientation parallel to the N70°E to N80°E joint set reflects the ease with which groundwater migrated along this joint set saturating the soil bridging the solution-enlarged joints.

Depression geometry correlates with collapse. Collapse occurred most frequently in sinkholes with an L/W ratio (maximum depression diameter/minimum depression diameter, measured at right angles to maximum diameter) approaching circularity ($L/W \leq 1.80$). The L/W ratio, before collapse, in part reflected the engineering characteristics (cohesion, angle of internal friction, Atterburg limits) of the soil occupying the void in the carbonate bedrock and the size of the void.

A strong positive correlation exists between monthly collapse rates and the extent to which precipitation deviates (deficits or surpluses) from long-term monthly means. The correlation reflected fluctuating groundwater levels, following periods of precipitation surplus or deficits, in an area noted for large variations in vertical and horizontal fracture permeability.

Differential settlement of the soil along the pinnacled bedrock surface resulted in sediment bridging the rock pinnacles (soil arch). Wetting of the soil-arch sediments by groundwater, locally back-flooding the nearest available joint intersections, reduces the shear strength of the soil arches. Groundwater temporarily back-flooding the joints beneath the soil arch provides a significant vertical stress to support the soil bridging the void in the bedrock, even though wetting lowers the shear strength of the soil. With falling groundwater levels, soil in the arches was no longer supported by this vertical stress. The reduction in the shear strength of the soil arches eventually reached a threshold value (shear stress induced by the weight of soil-arch sediments $\geq$ shear strength of the arch sediments) such that overburden weight could no longer be transmitted to the bedrock pinnacles providing lateral support for the soil arch. With the threshold exceeded, a final spalling of sediments (along the underside of the soil arch) produced sinkhole collapse (Figure 3).

Since the field inventory began in 1973, approximately 60 percent of the nearly 100 collapses have been man induced. None of the collapses was triggered by groundwater withdrawal, in marked contrast with many of the collapses described in Alabama and Florida. Approximately two-thirds of the induced collapses were due to the alteration of surface drainage during residential and commercial development. Approximately one-third of the collapses resulted from artificial fills being placed in karst depressions.

The disruption of surface drainage in a "depression-pocked" terrain to accommodate construction frequently increases collapse risk for reasons related to the hydrologic functions of sinkholes. Karst depressions serve as point sources of groundwater recharge via solution-enlarged joints and bedding planes and as temporary storage basins for runoff. Urbanization frequently results in additional runoff, generated by increased impermeable surface area being rerouted to the nearest sinkhole. Collapse risk increases where the construction site is located along the N70°E to N80°E joint set between the sinkhole receiving the added runoff and the springs discharging the groundwater. The hydraulic gradient steepens between the sinkhole, which serves as the master collecting basin, and the springs after heavy or prolonged precipitation. The increased water volume, recharging the groundwater, moves down the hydraulic gradient as a water pulse. Frequently the solution-enlarged joints and bedding planes cannot effectively transmit the pulse load and back-flooding occurs in the solution-enlarged joints and joint intersections. Repeated back-flooding of the residuum increases the stress levels and redistributes the stresses acting on the soil arches, so that the vertical effective stress component exceeds zero along the underside of the soil arch. A stable effective stress distribution is reestablished by the spalling of the sediment from the underside of the arch (see Figure 3). Collapse occurs if a stable effective stress distribution is not reestablished as a result of spalling.

Even where surface drainage is not changed by urban development, the additional runoff caused by increased areas of impermeable surface can contribute to sinkhole collapse. The ponded water that collects in the depressions adds weight to the soil arch. The ponded water that does not evaporate infiltrates through the silty colluvial and residuum units at rates that range widely because of vertical and lateral differences in permeability. The contact between the residuum and colluvium is not horizontal but generally concave. The concave residuum-colluvium contact is interrupted by relict joints in the residuum that intersect the colluvium; by abandoned swallets; by previous collapse features; and by chert pebbles on the remnant, erosional surfaces developed locally on the cherty, clayey residuum. The infiltrated water, not stored in the chert-rubble zone along the soil-bedrock contact, migrates downward wetting the soil bridging the bedrock

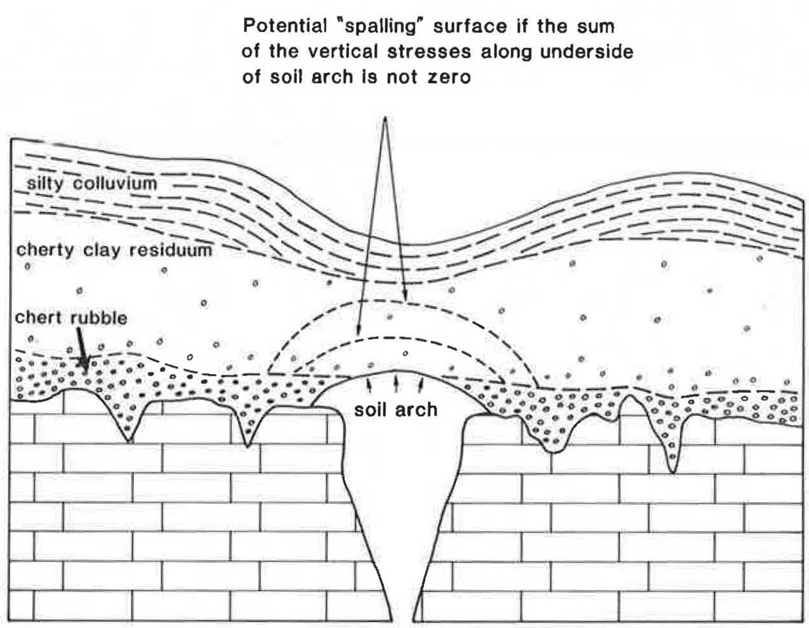

FIGURE 3 Generalized cross section of the soil arch and potential spalling surfaces (dashed lines).

voids. This process is most effective where relict joints are preserved in the residuum. Vertical percolation contributes to sinkhole instability by reducing the cohesive strength of the residuum partly filling the bedrock void. However, this is not as significant a collapse factor as is wetting due to back-flooding of groundwater through fractures beneath the soil arch.

The placement of artificial fill in a sinkhole frequently triggers collapse for reasons directly related to karst processes. The differential solution of the carbonate bedrock produces a thick cherty clayey residuum draped over an exceptionally irregular and pinnacled soil-bedrock contact. The contact surface is made even more irregular by the solution-enlarged joints and the presence of cavities along the soil-bedrock contact. The settlement of the residuum along this irregular contact is uneven resulting in the sagging of residuum into some voids and the bridging of other voids between pinnacles along the contact surface. The soil arches formed where the residuum has sufficient shear strength to bridge the pinnacles are at best unstable, particularly with fluctuations in moisture. Filling the sinkholes results in added stress being applied to the soil arch beneath the surface. The added weight of the fill can result in the accelerated compression of the residuum bridging the void in the bedrock. The added compression increases the bulk density of the soil units disturbing the effective stress distribution within the arch. The added effective stress frequently triggers a spalling of sediment from the underside of the soil arch in an attempt to reestablish equilibrium. Collapse occurs when equilibrium is not reestablished by sediment spalling along the underside of the arch.

Excavation for fill soil near a sinkhole can expose openings in the soil or along the soil-bedrock contact. Any new openings at the surface provide additional routes for direct recharge of groundwater. Rapid downward movement of water has a tendency to erode and transport unconsolidated deposits into the subsurface along solution-enlarged joints. This piping phenomenon can trigger collapse at the surface or initiate cavities in the soil that themselves frequently collapse. Even discounting the piping phenomenon, the added groundwater recharge due to the additional openings produced by grading can compromise the stability of the soil arch by increasing the back-flooding phenomenon that is so closely related to sinkhole collapse in the study area.

CORRECTIVE PROCEDURES

Water is the critical element in any effective collapse repair procedure. A remedial technique that does not include efforts to reroute surface and subsurface water away from the site risks failure.

The first step in the corrective technique requires excavation of the soil within the collapse down to bedrock. The bedrock surface is carefully cleaned to remove as much sediment as possible from the solution-enlarged joints. The Brunton bearing and amount of solution enlargement are measured for all joints exposed in the collapse. The amount of sediment occupying the joints at the site provides important information. Field experience suggests that the greater the solution enlargement and the more sediment-free a joint set is, the greater is the groundwater volume carried by the fractures.

The second step involves a detailed examination of all roadcuts in the area for joints and solution cavities developed along bedding planes. Joint bearings and solution features in the roadcuts are measured and compared with those at the collapse site. Available aerial photographs and topographic maps are examined for evidence of joint control of sinkhole major-axes orientations. The topographic and geologic setting of all springs is identified and correlated with the joint sets for dye-tracing purposes.

Efforts to control groundwater movement through the collapse site require that flow paths be determined if at all possible. Step three in the corrective technique involves an extensive dye-tracing program to locate groundwater flow patterns. Fluorescein ($C_{20}H_{12}O_5$) is best suited for tracing groundwater flow in karst terrains for a number of reasons (9). Fluorescein in groundwater can be detected in the field without a fluorometer. An experienced investigator can frequently detect fluorescein visually. Fluorescein has a higher absorption capacity than rhodamine WT on activated charcoal packets placed in fractures at the collapse site. Most important for field tests, the success of the fluorescein dye tracing is more dependent on the total quantity of dye passing through the activated charcoal packet placed in the joint than on the peak concentration of the dye reaching the collapse site. This fact is particularly important in settings where the fluorescein-charged water must infiltrate through fine-grained soils before entering rock fractures below the surface.

Individual sinkholes in the immediate area of the collapse are charged with a volume of fluorescein dye. Sinkholes with a major axis parallel to the N70°E to N80°E joint set are tested first. Experience shows these joints carry significant groundwater and strongly correlate with collapse. Fluorescein is mixed in an aerated tank in a concentration varying from 1 lb/100 gal (diffuse-flow conditions) to 1 lb/200 gal of water (conduit-flow conditions) depending on whether the dye-charged water is placed directly in an opening (sinkhole swallet) or must infiltrate through the soil. When it reaches the subsurface, the fluorescein-charged water will migrate down the hydraulic gradient with a velocity directly proportional to the gradient. Typical groundwater velocities in the study area are 100-300 ft/hr.

The presence of fluorescein-charged groundwater at the collapse site is detected using activated coconut charcoal packets. The packets consist of 5 in. x 6 in. fine nylon mesh, stapled pads containing two teaspoons of activated charcoal. Each pad is placed upright and perpendicular to the rock fractures exposed in the collapse. The activated charcoal will absorb any fluorescein passing through the packet. The packets should be left in the fractures several hours before they are removed. This gives the fluorescein-charged water the opportunity to reach the collapse if the charged water is being carried to the site from the point of injection. Next, each charcoal packet is emptied into a small container. The charcoal granules are covered with a 5 percent solution of potassium hydroxide (KOH) in 70 percent isopropyl alcohol. Any fluorescein present has been absorbed by the activated charcoal. The KOH in the alcohol replaces the fluorescein which then is released into the alcohol turning the solution a very distinctive yellow-green (a positive test). The process is repeated in each sinkhole until all sinkholes within the immediate area of the collapse are dye tested. The average time for the fluorescein dye to reach the collapse site from a dye-charged sinkhole reflects the direction and slope of the hydraulic gradient as well as the relative secondary permeability of the joint set.

A few days later, a second set of fluorescein dye tests is conducted to determine groundwater movement

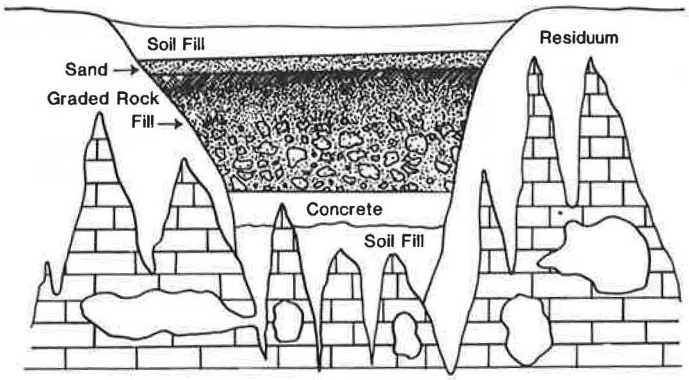

FIGURE 4 Cross section of corrected collapse.

from the collapse to any resurgent springs located earlier. The paths of minimum travel distance to the collapse are plotted on the topographic map for all dye-test sinkholes. Minimum travel plots are also made from the collapse to the resurgent springs serving as discharge points.

Next, a detailed surface-water movement map is prepared to determine which sinkholes are major points of groundwater recharge to the collapse site. Typically, in the study area, the major points of groundwater recharge are the larger, deeper, more structurally aligned (major axis parallel to joint set) sinkholes.

Careful surface grading around the collapse minimizes the volume of surface water recharging groundwater up the hydraulic gradient from the collapse site. Efforts are made to divert surface-water movement into drainage segments or into sinkholes between the collapse site and the resurgent springs or stream (i.e., farther down the local hydraulic gradient). Any groundwater movement would then be down the hydraulic gradient and away from the collapse site. Efforts should be made to reduce the joint permeability at the collapse site because groundwater will, to some extent, continue to move under the collapse site from recharge points outside the immediate area.

High-density concrete is placed in the solution-enlarged joints over a compacted soilfill (CH) placed in the joints to a depth of 1 ft below the top of the rock surface (Figure 4). The concrete is placed inside the solution-enlarged joint overlapping the top to form a plug incapable of entering the joint. The next step involves either using a graded rock filter or a high-density concrete to fill the collapse. In either case, the filter or concrete is placed in the collapse site to within 1 to 3 ft of the surface. Backfill, consisting of an earth fill compacted to a minimum density of 90 percent of the maximum standard AASHTO density, is placed on a sand blanket over the concrete or filter. The compacted controlled fill should allow construction to take place directly over the sinkhole without significant settlement. The ground surface over the collapse site is then graded to slope away from the backfill. Grading the slope away from the backfill prevents water collecting over the repaired collapse.

SUMMARY

A geologically based approach to collapse repair has been developed for use in the karstic limestones of the northern portion of the Western Highland Rim of Tennessee. The technique is based on taking advantage of the significant lithologic, structural, geomorphic, and hydraulic factors found to correlate strongly with collapses in the study area. The technique may not be effective in areas where such geologic and hydrologic factors are not as significant in initiating collapses.

ACKNOWLEDGMENTS

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Use of Sinkholes for Drainage

DAVID L. ROYSTER

ABSTRACT

Sinkholes have long been used for drainage in the design and construction of highways. Decisions to use them, however, seem to be based more on expediency than on engineering and scientific analysis. This practice has apparently resulted from the traditional notion that the processes and mechanisms of sinkhole development and proliferation are not totally analyzable and, therefore, not predictable. Although it is true that investigations of sinkholes and sinkhole-prone topography rarely produce absolute and finite data that can be applied in a strict quantitative sense, there are methods of analysis that, when coupled with experience and judgment, can be used with a high degree of success. In general, these analyses require information regarding such factors as geologic structure (e.g., joint orientation, direction and angle of dip); depth, direction of flow, and slope of the groundwater table; thickness and makeup of the regolith; degree of solution development and present level of activity; relief and topographic expression; size of area being drained by the sinkhole; location of known or assumed points of discharge; precipitation and flood levels; well locations and level fluctuations; location and size of swallets; and potential for further residential or commercial development. As with sinkhole problems in general, problems resulting from their use for drainage fall into two categories: those related to subsidence, or collapse, and those related to flooding. Regardless of which of these problems is being considered, the designer must always be alert to the fact that the treatment being applied to one may result in the development of the other. The success of future preventive and corrective measures will depend on more intensive geological and geotechnical investigations, as well as more stringent land use restrictions and building codes.

Much has been written about the cause and development of sinkholes and the recognition of flood- and collapse-prone sinkhole terrain in relation to highway planning and design, yet very little seems to have been published about their treatment. There are probably several reasons for this, the principal one being that most treatments are viewed as tentative or experimental, and, because scientists and engineers are sometimes reluctant to write about things that are inconclusive or projects that are incomplete, case histories do not get written. A second reason, allied to the first, is that investigations of sinkholes rarely produce absolute and finite data that can be applied in strict quantitative analysis. Treatments, therefore, must be based on experience and judgment and a more or less qualitative analysis and evaluation of whatever information is available. Such a practice, in effect, involves "design without numbers," which runs contrary to the precise analytical methods of most scientists and engineers.

In general, sinkhole problems can be placed in two categories: those related to subsidence, or collapse, and those related to flooding. Regardless of which of these problems is being considered, it must always be kept in mind that the treatment being applied to one problem may result in the development of the other. In other words, the treatment of a collapse problem may produce flooding, and the treatment of a flooding problem may result in subsidence or collapse. It is important, therefore, that the designer be alert to any side effects that might result from a particular design.

Several experiences with both collapse and flooding problems associated with highway design and construction are described.

FACTORS TO BE CONSIDERED

Sinkholes have long been used for drainage in the design of highways. Unfortunately, however, little attention has been paid to the side or long-term effects of such practices, especially in areas of continuing commercial or residential development. Furthermore, decisions to use sinkholes have been based more on expediency than on engineering and scientific analysis, apparently because, as indicated previously, sinkholes are not considered totally analyzable. There are several factors, however, that should be evaluated when traversing sinkhole topography, especially when considering the use of sinkholes for drainage. Among these are

- Depth, direction of flow, degree of fluctuation, and slope of the groundwater table;
- Thickness and makeup of the regolith (overburden) and depth to in-place rock;
- Geologic structure (e.g., joint orientation, direction and angle of dip);
- Relief and topographic expression;
- Degree of solution development and present level of activity;
- Size of the area being drained by the sinkhole;
- Presence and size of swallets;
- Potential for further residential or commercial development;
- Location of known or assumed points of discharge (e.g., springs, seeps); and
- Historical data (i.e., observed floodings, subsidences or collapses versus amount and intensity of precipitation).

Probably the two most significant of these factors in determining the use of sinkholes for drainage are depth, direction of flow, degree of fluctuation, and slope of the groundwater table and thickness and makeup of the regolith and depth to in-place rock. With this information it is often possible to make a reasonable judgment about subsurface storage capacity and flow-through potential of a particular area.

APPLICATIONS

An example of the importance and application of the aforementioned factors involves the correction of a subsidence problem along TN-76 in Montgomery County, Tennessee.

In August 1975 the paving of a 2.98-mile, four-lane section of TN-76, for which the grading and drainage had just been completed, was let to contract. The paving was completed in May 1976. By the following spring (1977), a 20-ft section of the southbound lane at Vaughn's Road began to subside (Figure 1). An investigation revealed that an approximate 200-ft section of both lanes had been constructed over a sinkhole that contained at least two swallets. The elevation of the lowest portion of the sink before filling was approximately 498 ft. The elevation of the finished pavement at this point was 518 ft, which meant that the maximum filling of the sink was 20± ft. The easternmost swallet had been left uncovered to accommodate drainage in this area. All drainage from an area of approximately 400,000 ft² was directed to this one swallet.

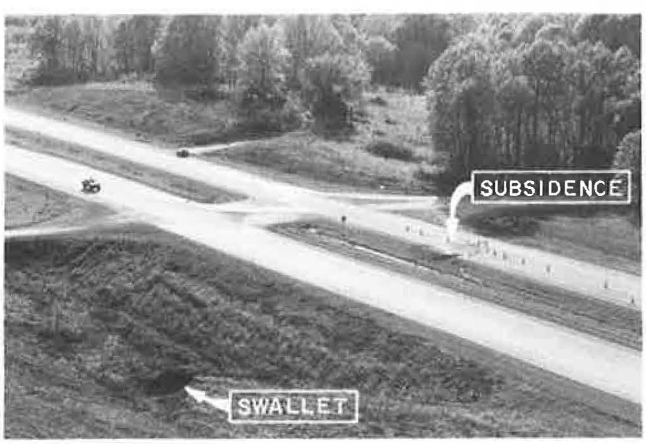

FIGURE 1 Subsidence caused by piping from water introduced into a sinkhole along TN-76.

The subsidence was repaired with a bituminous mix and the lane reopened to traffic. In the spring of 1978, however, the subsidence recurred, which resulted in the closing of the left southbound lane and shoulder area. Soon thereafter a subsurface investigation was initiated. Fifteen borings were made in and around the area of the subsidence and around the perimeter of the sink to determine the type and condition of the fill material, overburden, and in-place rock. Standpipe piezometers were placed in a number of these borings to determine the depth, slope, and fluctuation of the water table.

Because of the necessity of getting the project under contract by August, only 3 weeks of readings of water depths were possible. The water table--except for 1 day in which the elevation stood at 428 ft in response to 2.5 in. of rain over a 3-day period--remained steady at elevation 426 ft. The easternmost piezometer generally read 0.2± in. higher than the one installed some 200 ft to the west. This was taken as an indication, supported by the surface contour configuration, that the water table sloped in a northwesterly direction. From this a determination was made that the subsidence was caused by piping from a localized high-water condition that occurred during periods of high and intense precipitation. Most of the water was introduced through the one remaining swallet in the sink area (Figure 2).

The corrective measure involved sealing the swallet and rerouting the surface drainage toward the Red River drainage area northwest of the project (Figure 3). It was theorized that, because any of the rerouted water entering the subsurface would do so on the downslope of the normal water table relative to the roadway and because there were at least 80 ft of "freeboard" between the surface and the normal water table, the localized high-water condition would be eliminated. This, in turn, was expected to eliminate the piping problem. In the 5 years since the completion of the project (constructed at a cost of $91,316) no further problems have developed (Figure 4).

Another example of subsidence resulting from directing surface water into a swallet on the upslope side of the water table occurred approximately 1 mile from the TN-76 and Vaughn Road project. This problem, which has not been corrected, affects the eastbound on-ramp of Interstate 24 at TN-76 (Figure 5). All of the surface water in the southeast quadrant of the interchange is directed into a sinkhole approximately 100 ft south of the on-ramp. Although a subsurface investigation has not been conducted, indications are that the surface water entering the swallet is flowing back under the ramp in a northeastern direction and producing a piping action that is causing the subsidence. The problem will be corrected as soon as funds become available.

Several other subsidence and collapse problems have occurred in and around the I-24 and TN-76 interchange since construction began in 1975. A large portion of the entire interchange lies within the sink area. One of the more serious problems involved one of the piers on the I-24 bridge that carries the westbound lane across TN-76 (Figures 6 and 7). During construction, water collecting around the pile cap following a heavy rainstorm resulted in collapse of the soil around the cap. The collapse was caused by a piping action along the H-piles that penetrated to bedrock. The situation posed no great problem for the pier; problems did, however, involve the falsework that supported the beams and other structural components that were being placed. Had the collapse enlarged, some of the falsework would definitely have been affected. The problem was solved by filling and sealing the collapse and by diverting the

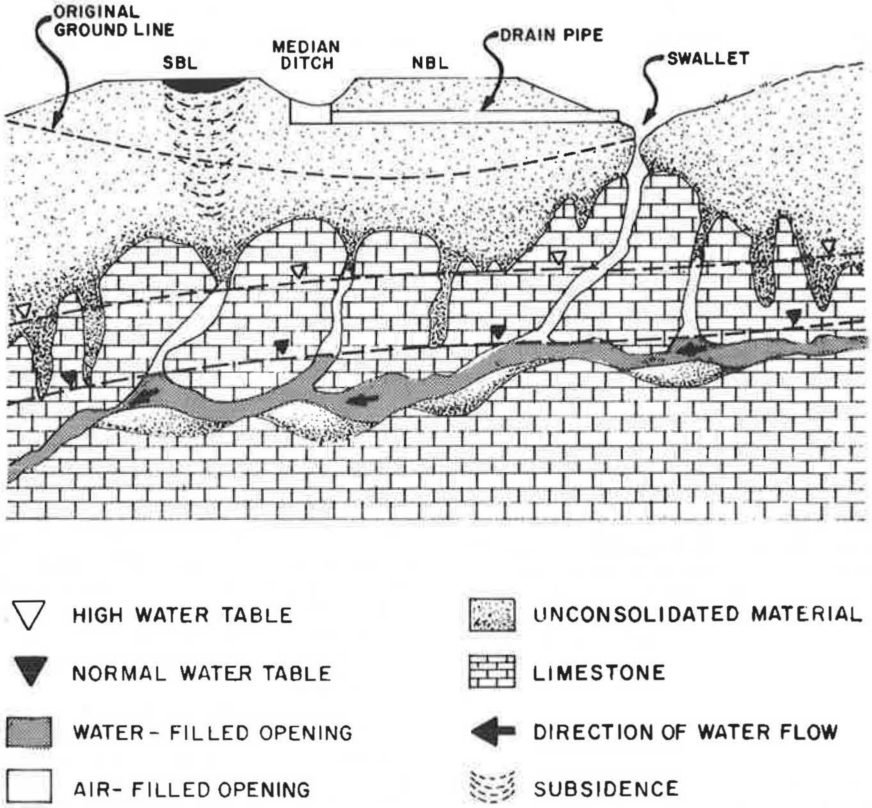

FIGURE 2 Schematic cross-sectional diagram of the TN-76 subsidence problem.

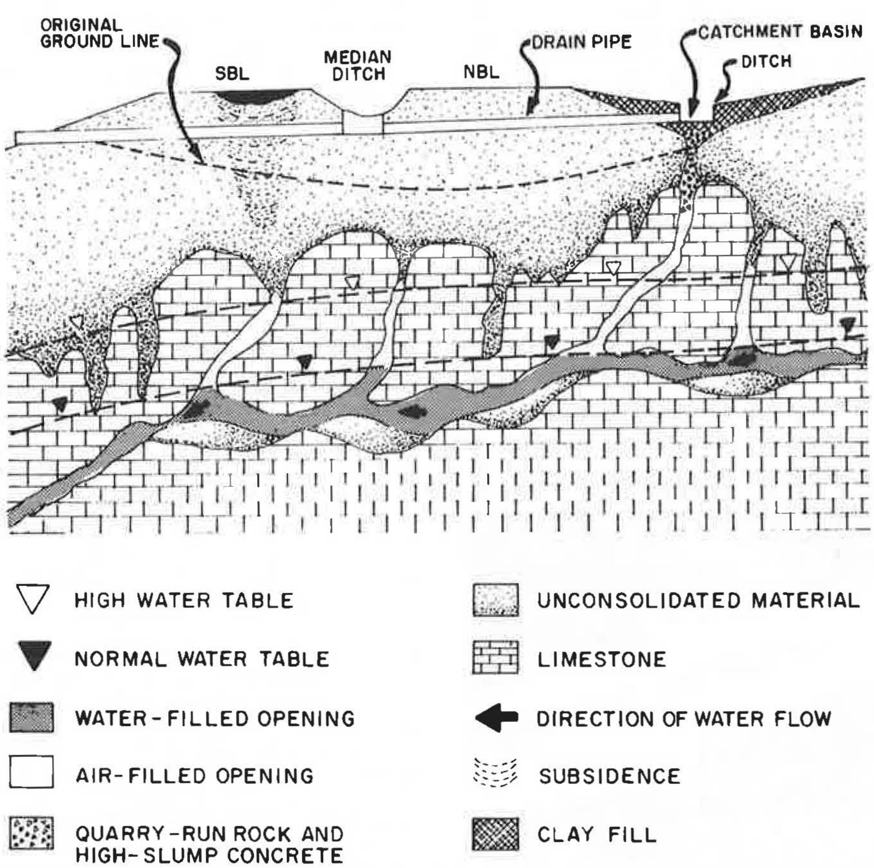

FIGURE 3 Schematic cross-sectional diagram of the TN-76 subsidence correction.

FIGURE 4 The corrective measure involved sealing and filling the sinkhole and re-directing the surface water toward the Red River drainage area.

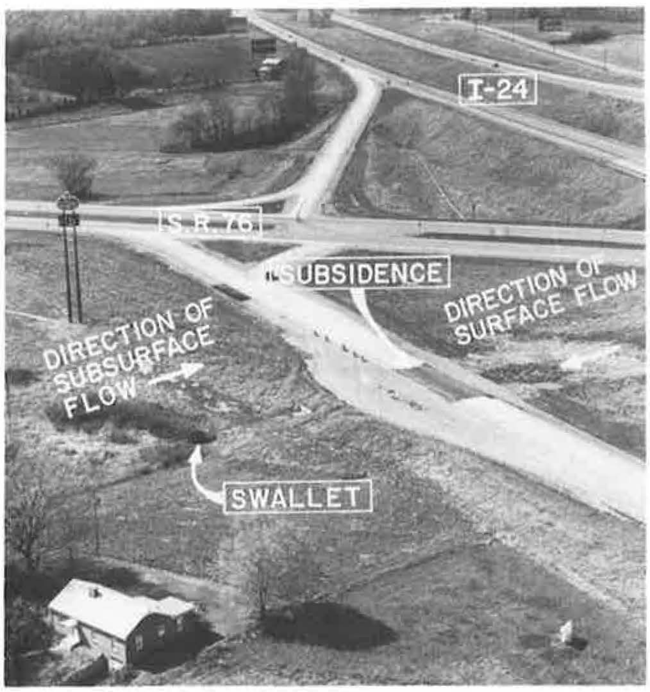

FIGURE 5 Subsidence of the I-24 eastbound on-ramp as viewed in April 1980.

surface water away from the area. This treatment seems to work well where there is a thick soil overburden (50± ft. in this case), where the normal water table is well below the soil-rock interface (25± ft in this example), where the water table does not fluctuate excessively (piezometers installed in the problem area indicated variations of no more than 5 ft over a 3-month period), where there are significant voids in the in-place rock caused by the solution of joints and bedding planes (several corings at the bridge site proved this to be the case), and where there are no major swallets in the immediate area on the up-slope side of the water table relative to the problem area.

Kemmerly (1), who has done considerable work in the Montgomery County, Tennessee, area writes: "Natural and artificially induced water-table fluctuations and the alteration of natural drainage patterns are the major cause of sinkhole collapse" (1,p.20). This observation is supported in the writings of Newton (2-4), Williams and Vineyard (5), Sowers (6), Foose and Humphreville (7), and Poland (8) as well as in those of various others writing on the subject in recent years.

FIGURE 6 Collapse of soil around a pile cap along westbound I-24 over TN-76 bridge in Montgomery County.

FIGURE 7 Close-up view of the collapsed condition shown in Figure 6.

Although collapse and subsidence usually produce the most dramatic and devastating problems caused by water table fluctuations and drainage pattern alterations in karst areas, flooding can also be a problem. Flooding occurs when water flowing into the subsurface exceeds the storage and flow-through capacity of the internal drainage system, or when swallets and caves and other openings to the subsurface become blocked through natural or man-made causes.

Tennessee has experienced a number of flooding problems in recent years due primarily to development along highways that traverse karst areas. Sinkholes and caves that have been used for drainage along highway alignments, and that have functioned effectively for many years, frequently reach a point where they can no longer accommodate the additional run-off that results from increasing commercial and residential development. A number of these problems in Tennessee were described by Moore (9) in 1980. A more recent case, however, and one that probably can be considered a classic among these kinds of problems, involves a rather large depression (270± acres) in north central Montgomery County, Tennessee. The entire depression, which will be referred to as the "Ringgold Sink," is drained by two swallets at the low point near its center. TN-12 (US-41-A) traverses northwesterly through the depression some 500± ft west of the swallets. The average per day traffic volume for this four-lane highway is approximately 30,000 vehicles.

A 1957 U.S. Army Corps of Engineers 7.5-min quadrangle of this area shows only five buildings within the 515-ft contour, which essentially delimits the depression or drainage basin (Figure 8). Since that time the approximately 270 acres in the basin have been almost totally developed both commercially and residentially. With each passing year flooding has become more and more prevalent, resulting in closures of the highway for periods of up to 24 hr.

In response to a request by the city of Clarksville in late fall 1982, the Soils and Geological Section of the Tennessee Department of Transportation initiated an investigation to develop corrective measures for the flooding problem. The request was prompted by roadway closures on August 31 following 4.03 in. of precipitation on August 30 and 31, and on September 13 following rainfall totaling 3.26 in. on September 11 and 13. During these periods the roadway at the low point in the depression was covered by 4 to 5 ft of water. The elevation of the roadway at this point is approximately 478 ft. The elevation of the low points in the two swallets is 464 and 467 ft, which means that the maximum depth of water in the depression during these floodings was 18± ft. Four of the nine commercial buildings, whose foundations are near the low-point grade of the roadway, within the basin were also affected.

During the investigation, 22 borings were made in and around the two swallets. Standpipe piezometers were placed in five of the borings. Water level readings were made approximately every 3 days for 3 months and compared with rainfall amounts during this period. Though no roadway closures occurred during these 3 months, rainfall amounts were relatively high, especially during December when 10.06 in. were recorded at the Clarksville Sewer Plant. Historically, closures have occurred with a 4-in. rainfall over a 24-hr period, or when there was a 3.5± in. rainfall followed by another 3.5± in. rainfall in 2 or 3 days. There was a 1.57 in. rain-

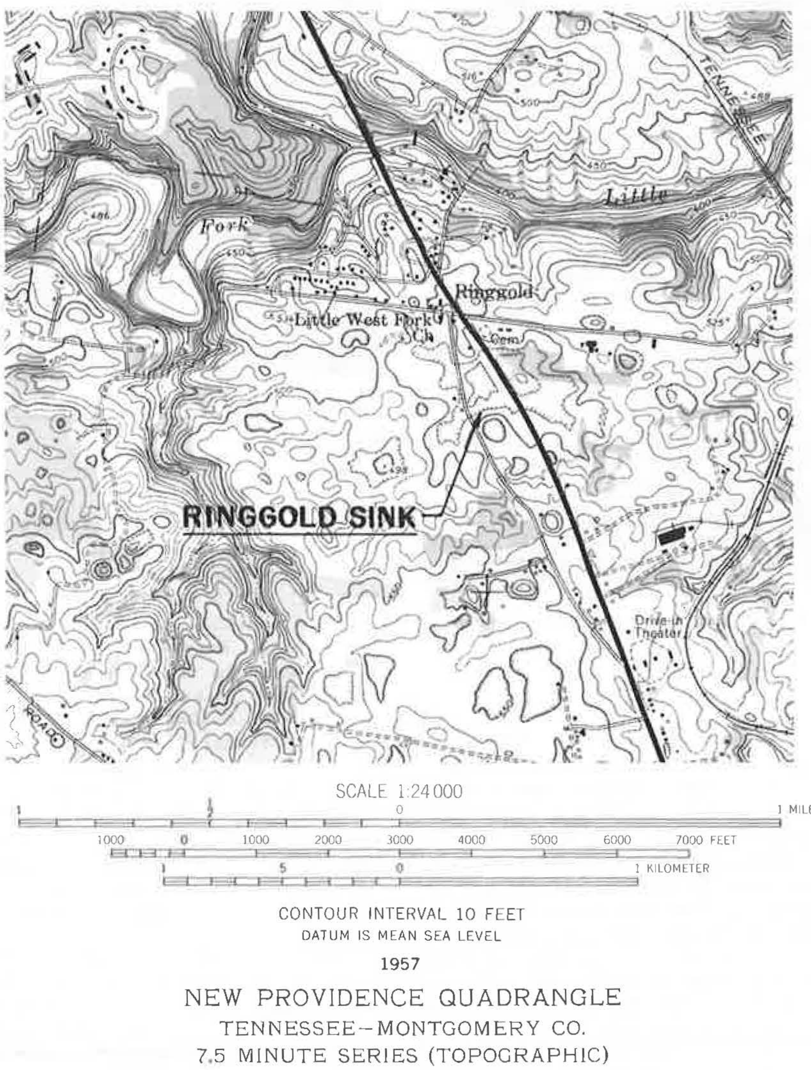

FIGURE 8 Ringgold Sink location map.

fall on December 3-4, 1.80 in. on December 10, 1.45 in. on December 14, and 0.83 in. on December 15. In addition, 3.96 in. fell between December 24 and 27. (As a comparison, the next closure occurred on May 19, 1983, when 4.6 in. fell in a 24-hr period. This had been preceded by 2.63 in. on May 15-16.) Piezometer readings on December 26, during the rainy period, showed only a 1.5 to 2.0 ft rise in the water table. By December 29, all levels were back to normal. It is possible that during the investigation the overall drainage of the swallets was improved somewhat by the borings and by the clean-out work that was necessary to gain access for the drilling equipment. A number of dead trees, as well as considerable refuse (discarded rubber tires and so forth), were removed in the clean-out operation.

As is the case with most sinkholes in the area, the swallets, or tubes, that lead to the subsurface in the Ringgold Sink have developed along pinnacles of limestone near the surface. The surface of the in-place rock drops off rather dramatically in all directions from the pinnacles. For example, at swallet 1, the pinnacles occur within 7 to 10 ft of the surface in an approximate 30 ft radius, with the rockline dropping off 40 to 50 ft outside this zone in a distance of only 30 to 40 ft. Most of the surface water appears to flow along the soil-rock interface before percolating deeper into the subsurface through solution-enlarged joints and bedding planes, and ultimately into possibly larger channels and conduits that feed the two or three springs that empty into Little West Fork River some 2,000 ft to the north. The elevation of the flow line of the springs where they exit is 385± ft. The elevation of the water table beneath the depression, as measured in the five piezometers, varied between 419.5 and 421.5 ft. As might be expected, the greatest number of cavities (and these appeared mostly to be soil-filled) occurred in the pinnacle areas. Core recovery outside these areas where the regolith was 40 to 50 ft thick usually ran 90 to 100 percent. The overburden consisted largely of a cherty clay, with clay contents generally ranging between 45 and 60 percent and plasticity indices between 25 and 30. Approximately 3 or 4 ft of silt and organic material overlay the residuum in swallet 1 (Figure 9).

To gain some feel for the drainage characteristics of the swallets, and to determine if the water table could be affected in close proximity to the piezometers, several 2,000-gal tanks of water were pumped into 5-in. polyvinyl chloride casings set in the overburden to the top-of-rock. The water was pumped at the maximum pump rate (2,000 gal in approximately 12 min). No rise in the water levels in any of the five piezometers was detected. Furthermore, there were no observable blockages or lags in the flow rate into and through the 5-in. casings. Although no particular significance was placed on this testing outcome, it did serve to strengthen the already existing notion that the drainage within the swallets might be improved somewhat, at least in the short run.

Several corrective alternatives were considered. The first was to raise the grade of the roadway above the present flood level. This would have solved the problem for the roadway but not for the

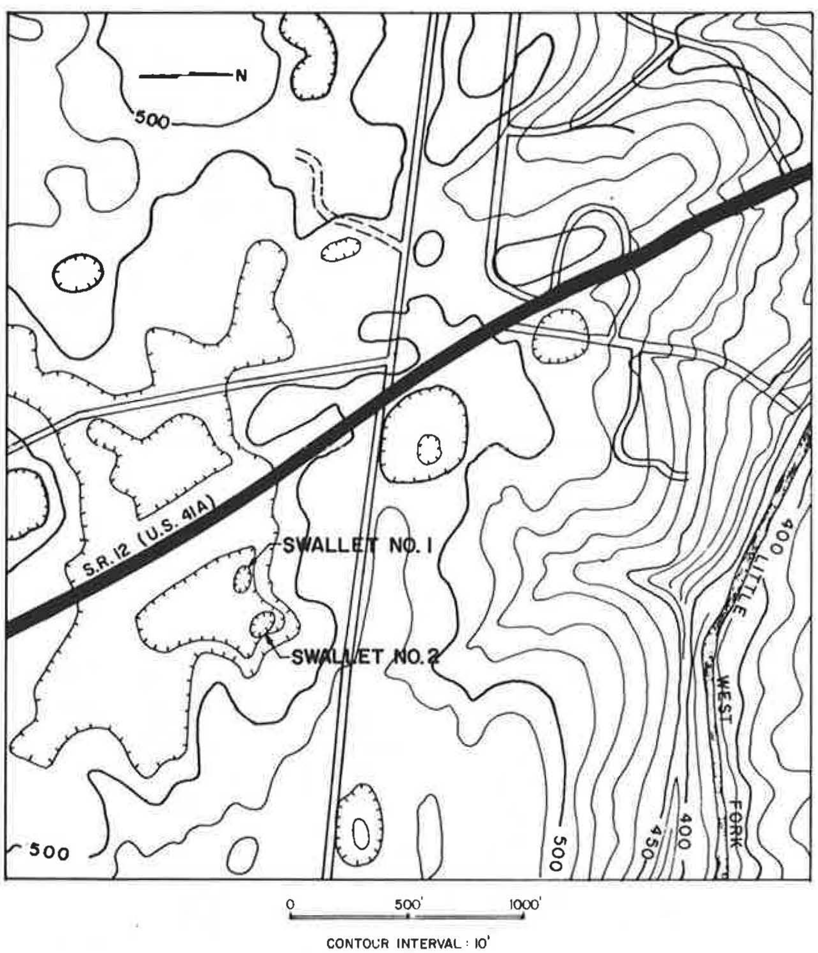

FIGURE 9 Location of swallets within Ringgold Sink.

adjoining commercial establishments. Too, there was a problem of access that could not be reasonably handled. The Hydraulics Office of the Division of Structures considered a pumping station alternative. The pumps would have been activated when flood levels came within 2 or 3 ft of the roadway. The cost of the pumps and installation was estimated to be around $225,000. This did not include drainage easements that would have been necessary, nor did it include the cost of maintaining the system. A third alternative involved the excavation of holding ponds. This idea was eliminated, however, because of the limited area for construction, because of pollution problems that would probably develop, and because of the danger of collapse due to piping.

The most obvious alternative considered involved the construction of a storm sewer installed in a cut-and-cover trench, or a combination of cut-and-cover and tunneling. The excavation would have extended to a maximum depth of 35 to 40 ft and would have been 2,000± ft in length. It would have required a 72-in. pipe. The cost of this alternative, excluding drainage easements, which would have been rather substantial, was estimated at around $300,000.

The final repair alternative considered involved improving the drainage of the existing swallets. This improvement would consist of excavating the swallet areas to in-place rock, installing perforated standpipes (3 to 4 ft in diameter) and backfilling with drainage stone (Figure 10). Also included were grouted riprap ditches, which would direct the water into the sumps, and several siltation fences.

Excavation, as well as drainage stone placement, will be accomplished with a clamshell to avoid compaction and consolidation in and around the swallets. The construction of sump 1 will be completed before beginning work on sump 2 to minimize siltation and the possibility of blockage in case of flooding while work is under way. All excavated material will be disposed of outside the drainage basin. The backfill material (drainage stone) will be required to meet the following gradation: 60 to 40 percent between 2 and 1 ft along its maximum dimension, 40 to 20 percent between 1 ft and 6 in., 20 to 10 percent between 6 and 2 in., and 5 to 0 percent less than 2 in. In addition, filter fabric will be placed on the excavated slopes to minimize erosion and siltation at the drainage stone-soil interface (Figure 10).

The cost was estimated at around $60,000. One of the keys to the success of the sump design will be the flood levels of Little West Fork. The normal pool elevation is 383 ft, the 2-yr level is 397 ft, the 10-yr level is 402 ft, and the 100-yr level is 408 ft. Obviously, these levels will affect the groundwater level beneath the sink and in the area between the sink and the river. The 44-ft difference in elevation between the normal water table beneath the sink and the lowest swallet entrance does not leave much additional storage capacity, especially because much of this volume is composed of a relatively highly plastic clay with little void space.

The most obvious problem with the sump alternative, then, is the uncertainty that it will work. There are simply too many unknowns and too many factors that must be left to chance for this to be offered as the ultimate corrective measure in this type of situation. It is one thing to use a sinkhole for drainage in a small, relatively isolated, undeveloped area but quite another in a large, highly developed area, especially where there is already a flooding problem. As indicated previously, however, it was generally felt throughout the investigation that the drainage in the swallets could be improved, possibly even to the point of handling the 2- to 5-year flood levels. Consequently, the department recommended that the sump alternative be constructed, but only on the condition that the city of Clarksville prohibit further development within the basin for 5 years. At the same time the city was encouraged to set aside a portion of its annual budget for each of those 5 years to fund the storm sewer construction. As of this writing (July 1983) the city is waiting for a response from the metropolitan planning commission before committing itself to the recommendation.

LAWMAKING AND TECHNICAL RESPONSIBILITIES

The best way to avoid collapse and flooding problems in sinkhole areas is to simply prohibit all types of construction for all time. This, of course, is not a realistic or acceptable approach, especially if you live in an area like the one that has been described. The sensible approach is to establish restrictions in the obvious problem areas by prohibiting certain types of development and by tailoring the designs of those developments that are permitted to fit the conditions at the site.

The Ringgold Sink problem is a classic example of the failure of government to enact and enforce land use statutes, zoning ordinances, and building codes

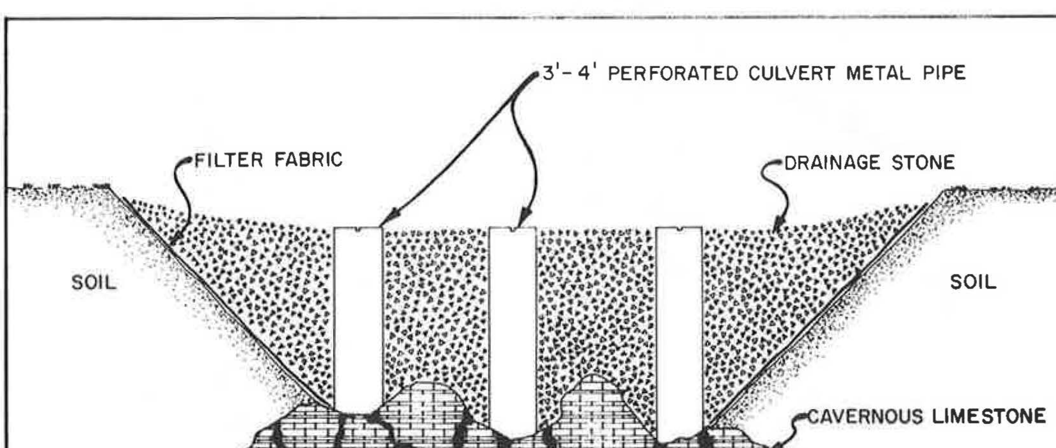

FIGURE 10 Schematic cross section of sump design for the Ringgold Sink.

relative to geologic hazards. Although the problems are not generally as catastrophic or serious as are those discussed here, developments in major sinkhole areas must be controlled in much the same way as those in flood plains, those near active geologic faults, those near active volcanoes, and those in avalanche and landslide areas. If the public good is to be served, all levels of government must put politics and special interests aside and make the hard decisions that so often are associated with land use questions, especially where geologic hazards are involved.

Those people with technical responsibilities--geologists, engineers, planners, and architects--must do a better job of communicating their knowledge and understanding of sinkholes to both the lawmakers and the public. They must also improve their own skills and expertise through research and investigation. As stated previously, a great deal of information has been published about the cause and development of sinkholes and the recognition of flood- and collapse-prone terrain in relation to planning and design, yet very little has been published with regard to treatment. Simply identifying a problem or problem area is not enough. Professionals and technologists in both the public and private sectors must always take that second step: the development of cost-effective alternatives for solving the problem.

CONCLUSION

Sinkholes have long been used for drainage in the design and construction of highways. Decisions about their use, however, seem to be based more on expediency than on engineering and scientific analysis and planning. Furthermore, little attention is paid to the long-term effects of such practices, especially in areas of continuing residential or commercial development.

Although sinkholes can, and sometimes must, be used for drainage in certain areas and in certain situations, decisions about their use should be based on site-specific geologic and geomorphologic information. To fail to consider such factors (e.g., depth, direction of flow, degree of fluctuation, and slope of the groundwater table; geologic structure; thickness of overburden and depth to in-place rock; relief and topographic expression; degree of solution development and present level of activity) may very often result in future, if not immediate, problems.

In addition to the need for improved analytic methods in collapse and flood preventive and corrective considerations, governmental lawmaking bodies must assume a greater responsibility for enacting and enforcing zoning ordinances and building codes in sinkhole areas. There are places where total restriction on development is the only answer to collapse or flooding problems. In others, only minimal limitations are all that may be required. In either case, and for all situations in between, such restrictions can only be instituted by governments--preferably the local government. How responsive they are will depend on the concern of the public and the persuasiveness of the local scientific and engineering community.

ACKNOWLEDGMENTS

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Road Construction in Palsa Fields

J. HODE KEYSER and M. A. LAFORTE

ABSTRACT

Palsa is an important feature of the discontinuous permafrost regions of northwestern Quebec. Because of the development of hydroelectric complexes along La Grande and Great Whale rivers, the road network will be expanded by the addition of 2000 km of road with many sections crossing palsa fields. Problems related to the design, construction, and maintenance of roads in palsa fields are identified and described. The observations are mainly based on the performance of a test embankment built 3 years ago on a large palsa and the performance of 620 km of road, paved in 1976, that cuts through several palsa fields. The topics discussed are topology, occurrence and distribution of palsa fields in northern Quebec, dating of palsa ice, description of a typical palsa field, description of the physical characteristics of a typical palsa, temperature regime in the palsa, performance of an instrumented test embankment 3 years after construction, performance and maintenance history of a 6-year-old paved road that crosses several palsa fields, and predicted versus observed rate of settlement of existing embankments. Based on the results of these investigations, recommendations are made for the design and maintenance of roads that cross palsa fields.

The development of a road network in subarctic Quebec is relatively recent. All the principal and some local roads were built after 1972. With the development of the La Grande hydroelectric complex, approximately 2000 km of roads are being added. The principal access road runs from Matagami to LG-2; it is 620 km long, and was paved between 1974 and 1976 (Figure 1).

The area under consideration is 40 000 km², extends from the 50th to the 56th north parallel, and is divided into two main topographical regions: (a) the lowlands, at an altitude under 200 m, that are composed of a silty clay plain south of the 52nd degree parallel, and a glaciomarine plateau to the north and (b) the highlands that are 150 to 250 km from the shores of James Bay and present a low rocky plateau strewn with lakes.

The area bedrock is essentially Precambrian with many outcrops in the highlands. Unconsolidated deposits are composed mainly of marine silty clay and beach deposits in the lowlands and glacial till and fluvio-glacial sands and gravels in the highlands. Almost everywhere peat deposits can be found in surface depressions.

The climate is of a subarctic continental type with maritime influence from the James Bay. The average freezing index is around 2500°C per day, and the thawing index is less than 1500°C per day; mean air temperature varies from 0 to -4°C. The in-place snow cover varies from 45 to 65 cm.

Although the area is in a discontinuous permafrost zone (1), permafrost features are scarce in the highlands except at the tops of bare hills. In the lowlands, the main permafrost feature is palsa [for definition, see Stanek (2)]. As the development of the area proceeds from south to north and from west to east, most of the roads are or will be located in the lowland area where muskeg and palsa fields are frequent.

The problem of design and maintenance of roads crossing palsa fields is dealt with. Conclusions are based on observations of the performance of an experimental embankment constructed over a palsa, on data gathered through subsurface investigations of a projected road 100 km long between LG-2 and GB-1, and on the evaluation of five settlement sites along the 620-km James Bay access road.

PALSA TOPOLOGY AND OCCURRENCE

Palsa can be defined as a discontinuous permafrost feature; it is a mound created by the formation and growth of an ice core under favorable microenvironmental conditions (2). Two main types of palsas are identified in subarctic Quebec: nonwooded palsas and wooded palsas (3). Both types occur in the lowland areas at elevations not exceeding 200 m and can be found in clusters of 3 to 10 in both dry and wet areas.

Nonwooded palsas are mainly located north of the -3°C annual isotherm in the coastal zone of Hudson Bay. They are principally of organic origin (90 percent) and have either round or irregular shapes. They can be isolated or form important palsa fields up to 5 km² in area (palsa plateau north of Great Whale). The origin of nonwooded palsas is thought to be the degradation of ancient permafrost.

Wooded palsas are found principally in the southern part of the territory where the mean annual temperature varies from -1°C to -4°C. They are normally covered with black spruce and tamarack, lichens and peat moss, forming a drunken forest at the edge of the palsa. Many palsas present signs of degradation, with cracks and water ponding at the surface. More than 400 sites of wooded palsas have been identified by Dionne (3); the first access road cuts through at least five zones (4). Each zone could have 5 to 10 palsa fields.

Palsas are generally considered relic permafrost, under tree cover, protected by microclimatic phenomena. However, a dating test by the O-16/O-18 method indicates that the palsas along the LG-2-GB-1 access road were formed during the last 40 years (5).

DESCRIPTION OF A TYPICAL PALSA FIELD

A palsa field can be defined as an assembly of individual palsas in an environment that favors palsa formation. A typical palsa field in northern Quebec is shown in Figure 2.

Palsa fields have been characterized by in situ geotechnical and geophysical surveys; by borings, soundings, and sampling; and by testing. More than 10 palsa fields were studied in 1980-1982. Borings were made either with or without B or N casing, and samples were taken with thin-wall tubes or split spoons in the disturbed and undisturbed state (4).

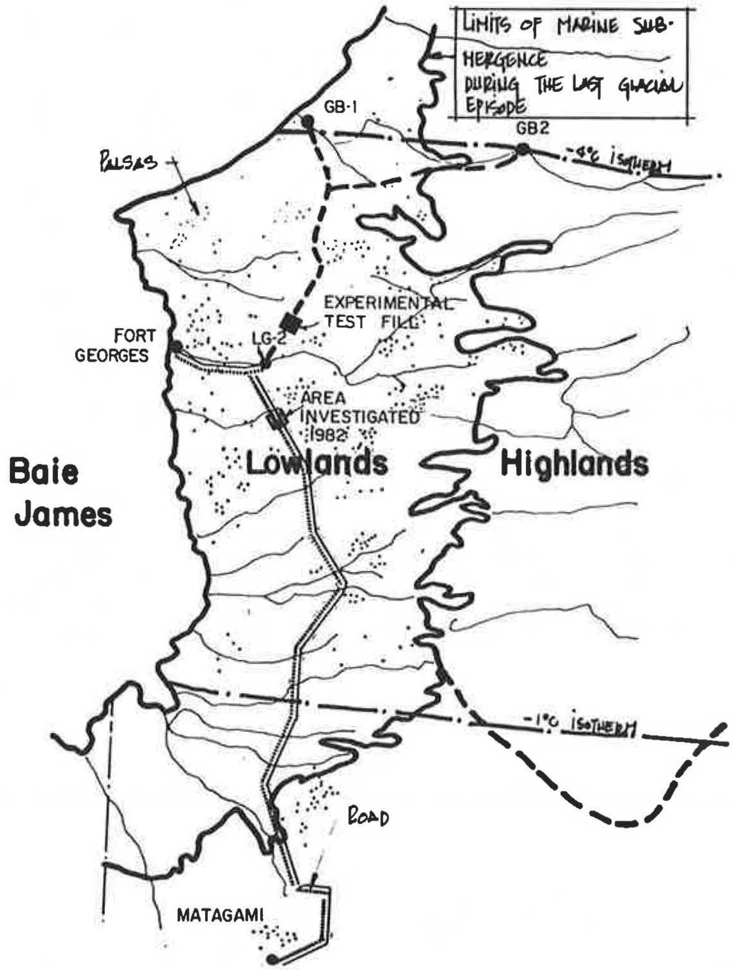


FIGURE 1 La Grande hydroelectric complex.

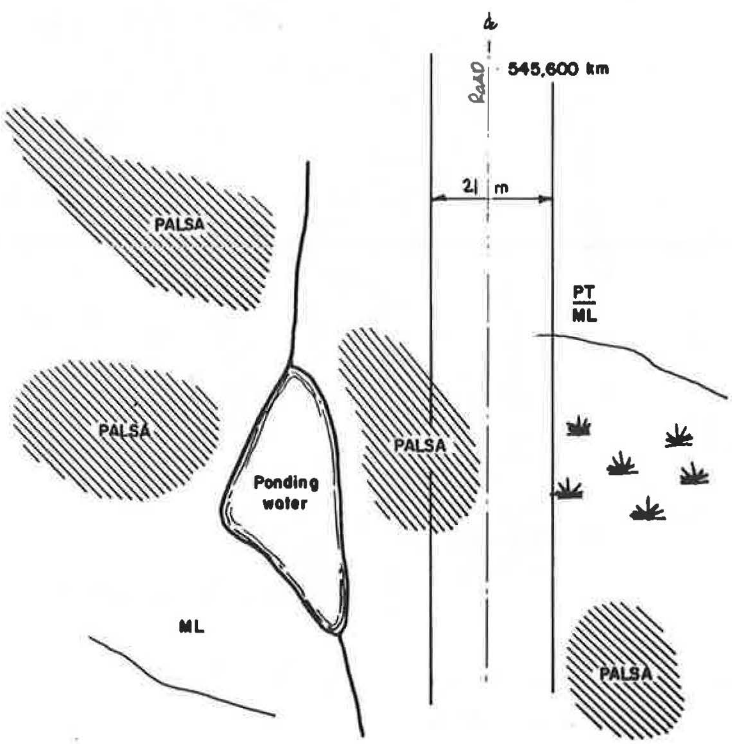


FIGURE 2 Typical palsa field.

The samples were examined in a cold room and in the field to determine the density and water and ice content. Classification tests were also made: peat was classified according to the Von Post index (6) and the frozen structure was classified according to ASTM standard procedure (7).

There could be a palsa field in every low-lying area in a particular region; but more often palsa fields are distant from each other. A palsa field generally contains 3 to 10 palsas; the distance between individual palsas is generally less than the width of the palsas.

The drainage pattern in a palsa field is not well defined and is sometimes influenced by the underlying rock. Usually the water table is close to the surface, and, most of the time, a pond can be found in the low areas of a palsa field.

DESCRIPTION OF A TYPICAL PALSA

Palsas are usually small and circular or ellipsoidal in shape; the length of a palsa generally varies from 10 to 100 m, and palsas have a thickness of 1 to 8 m with a maximum of 3 m above the surrounding terrain. The surface of a palsa is often hummocked and contains small depression zones that are at times unfrozen (see Figure 3). The peat layer at the surface of the palsa is often cracked around the boundaries and is accompanied by a slipping surface and, sometimes, exposed ice. Degradation is worst in the open areas or in old burnt surfaces; in the deep wood-covered areas, the surface is hummocky but the peat layer is not degraded. Around the palsa, underneath the pond, or in the natural peat cover, the soil is unfrozen and boundaries between frozen and unfrozen material are well defined.

All the wooded palsas investigated are composed of four typical layers: a layer of peat at the top, a silty clay layer with no excess ice, ice interstratified with silt, and the unfrozen soil. Several typical borehole sections are shown in Figure 4 and the general characteristics of each layer are given in Table 1.

The thickness of the peat layer varies from 0.60 to 2.70 m. The peat material is generally black or brown with a low fiber content and high mineral content. The Von Post index is always higher than 6. Surface cover is normally lichen or moss with an active layer varying from 20 to 50 cm. The frozen peat is generally classified as Nbn because it is well bonded with a slight excess of ice, its density

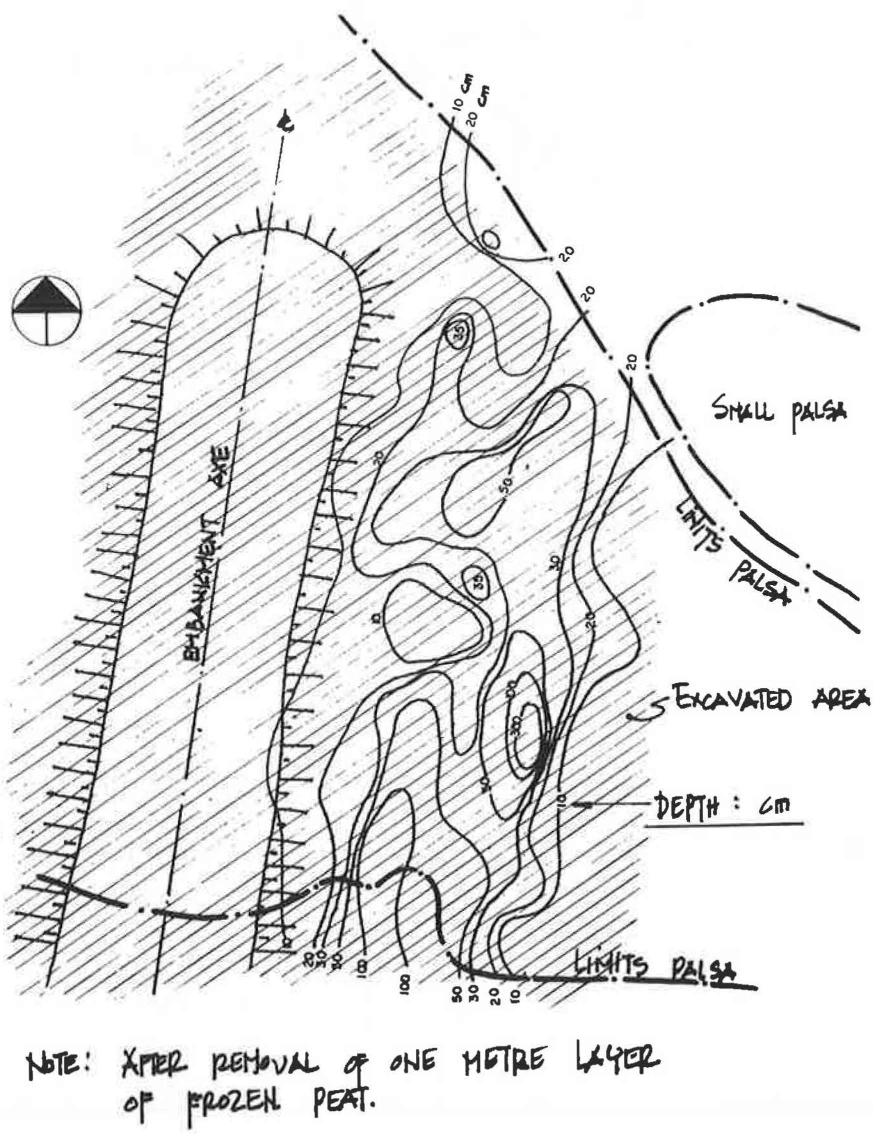
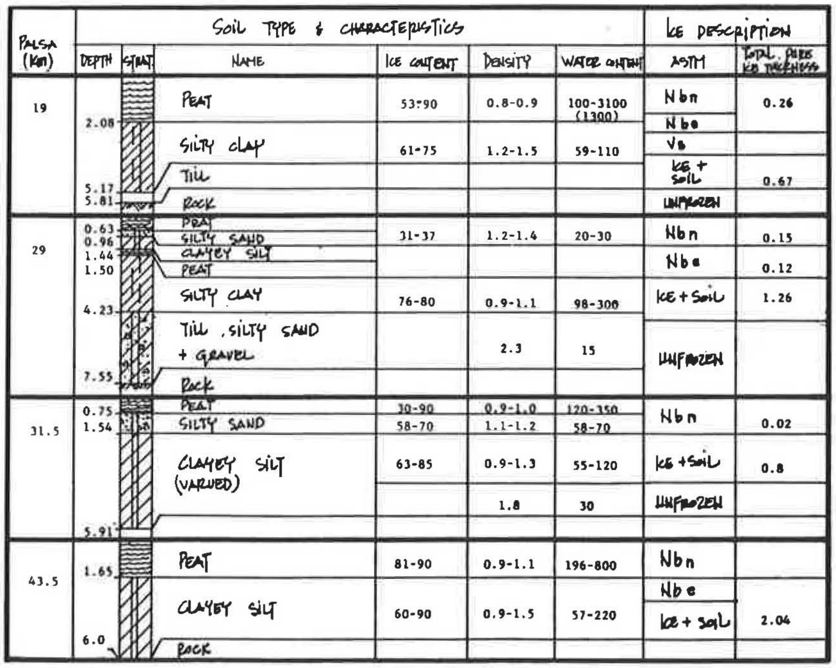


FIGURE 3 Typical unfrozen depression zones at the surface of a palsa.



PALSA (Km)	DEPTH	NAME	ICE CONTENT	DENSITY	WATER CONTENT	ASTM	TOTAL PURE ICE THICKNESS
19	2.08	PEAT	53-90	0.8-0.9	100-3100 (1300)	Nbn	0.26
	5.17	SILTY CLAY	61-75	1.2-1.5	59-110	Nbe	
						Vs	
		TILL				ICE + SOIL	0.67
	5.81	ROCK				UNFROZEN	
29	0.63	PEAT	31-37	1.2-1.4	20-30	Nbn	0.15
	0.96	SILTY SAND					
	1.44	CLAYEY SILT				Nbe	0.12
	1.50	PEAT					
	4.23	SILTY CLAY	76-80	0.9-1.1	98-300	ICE + SOIL	1.26
	7.55	TILL, SILTY SAND + GRAVEL		2.3	15	UNFROZEN	
		ROCK					
31.5	0.75	PEAT	30-90	0.9-1.0	120-350	Nbn	0.02
	1.54	SILTY SAND	58-70	1.1-1.2	58-70		
		CLAYEY SILT (VARVED)	63-85	0.9-1.3	55-120	ICE + SOIL	0.8
				1.8	30	UNFROZEN	
	5.91						
43.5	1.65	PEAT	81-90	0.9-1.1	196-800	Nbn	
		CLAYEY SILT	60-90	0.9-1.5	57-220	Nbe	
						ICE + SOIL	2.04
	6.0	ROCK					

FIGURE 4 Description of typical palsa materials.

generally varies between 0.8 and 0.9 g/cm³, and in situ water content is between 250 and 2500 percent.

Between the peat layer and the icy core there is normally 0.5 to 1.0 m of well-frozen silty clay. This material, which is typically well bonded with some excess ice, is classified as Nbe. It has an in situ water content that varies between 50 and 110 percent and a frozen density that varies between 1.3 and 1.6 g/cm³.

The layer forming the icy core of the palsa is usually 1 to 3 m thick. It is composed of a highly segregated soil, formed by ice lensing the silty clay. The individual ice lenses can attain 10 to 15 cm in thickness. The frozen classification of the material varies from V_s (excess ice with ice stratification) to V_i (ice with soil inclusions) to "ice + soil." The ice is either transparent or opaque and represents up to 70 percent of the material by vol-

TABLE 1 General Characteristics of Palsa Materials

Material and layer thickness	Physical properties in frozen state			Properties in unfrozen state			Water content %		Thermal conductivity (Watt/°km²)		
	Classification	Density g/cc	Ice content in % of volume	Description	Anticipated settlement in % of original height	Saturated shearing strength[a]	Frozen state	Unfrozen state[b]	Frozen state	Unfrozen state Unsaturated	Unfrozen state Saturated
Peat (0.6-2.7 m)	Nbn	0.8-0.9	30-50	Compressible fibered mineral peat	30-40	.1 p_o	250-2500	.6	1.7	0.5	0.6
Silty clay (0.5-1.0 m)	Nbe	1.3-1.6	30-60	Slightly consolidated fine soils	50-60	.15 p_o	50-110	.5	2.7	1.7	1.7
Icy core interstratified with silt (1.0-3.0 m)	$V_s V_i$ to (ice and soil)	1.0-1.4	80-150	Normally consolidated fine soil in suspension	60-70	.05-0.1 p_o	80-150	.5	3.0	2.0	-
Glacial till or silty clay	-		None	Over consolidated soil	< 5	.2 p_o			-	-	-

[a]Undrained shearing strength as related to effective pressures of earth in place.
[b]Unfrozen water content as a fraction of total water content.

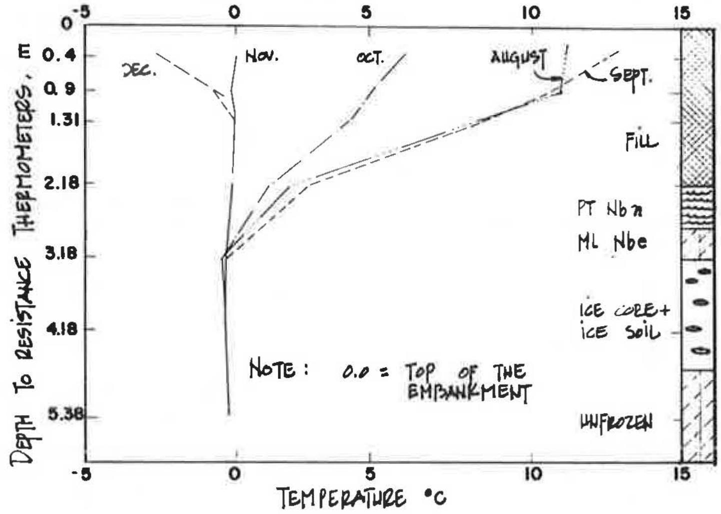


FIGURE 5 Thermal regime of a palsa: test embankment, access road to GB-1, 1981.

ume. In this zone, the soil is often dry and near freezing temperature. The water content of the material from the icy core varies between 80 percent (V_i, V_s) and 150 percent (ice + soil) and its frozen density varies from 1.0 to 1.4 g/cm³. The material underlying the ice core is generally either glacial till or silty clay, is normally completely thawed, but in some cases may be in a partly frozen state with visible ice inclusions (V_i).

The temperature profile of a typical palsa is shown in Figure 5. The profile, valid for 1981-1982, indicates that the temperature of the icy core is between -1°C and 0°C and that it has varied within a range of 0.3°C within the year.

PERFORMANCE OF AN EXPERIMENTAL EMBANKMENT OVER PALSA

An experimental embankment was built by Hydro-Quebec in 1981 over a palsa along the proposed access road to the Great Whale hydroelectric complex. The purpose of the test was to evaluate the rate of settlement and to identify problems associated with settlement prediction and embankment performance. As shown in Figure 6, the embankment was built on a typical small wooded palsa, 20 x 45 m, protruding 1.2 m above the surrounding muskeg. The stratigraphy of the palsa is typical: a layer of peat at the top followed by a layer of silty clay and an icy core. Before the construction of the embankment the protruding surface of the palsa was leveled leaving frozen peat at the surface.

Figure 7 shows the layout of test sections and the location of the instruments. The embankment was divided into three sections according to the treatment of the leveled ground (frozen peat): In one section a geotextile was placed between the peat and the fill; in the second section the peat was covered with 15 cm of sand and 10 cm of polystyrene insulation; and in the third section the frozen peat was left bare.

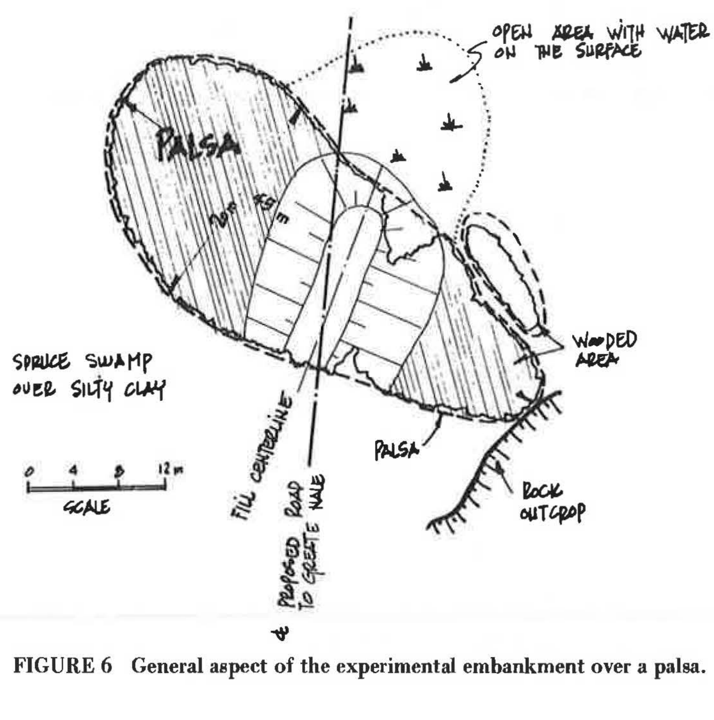


FIGURE 6 General aspect of the experimental embankment over a palsa.

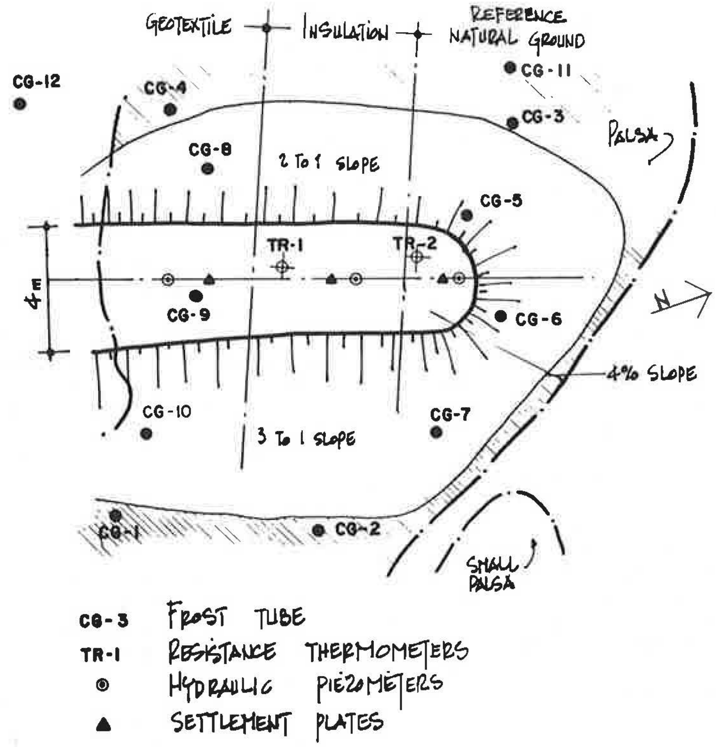


FIGURE 7 Characteristics of the experimental embankment and location of instruments.

The 2.3-m embankment was built with uniform sand containing little gravel. The slope of the fill was 2 to 1 in the western direction and 3 to 1 in the eastern direction. As shown in Figure 7, the embankment was instrumented with 12 frost tubes, 2 x 5 resistance thermometers, 3 x 2 hydraulic piezometers, and 3 settlement plates.

Because on-site testing facilities were limited, prediction of long-term settlement was based on simple tests. Thawing was predicted by thermal calculation using the modified Berggren formula (8), and settlement was evaluated using frozen soils classification, ice content, and frozen density (9,10). The prediction of the most probable long-term settlement was based on average in situ frozen soil properties, available air temperature data on a monthly basis, and instrument readings made since August 1981. The results of observations to date (December 1982) indicate the following:

1. As illustrated in Figure 8, predicted settlement is rapid at first as the peat material thaws, then becomes slower as thawing penetrates the ice core of the palsa. A rough estimate indicates that the coefficient of variation of predicted settlement can be as high as 20 percent due to the imprecision of initial input data and the natural variation in the properties of the palsa material.
2. Settlement of the fill without maintenance (no snow removal in winter and no leveling in summer) is in the lower range of prediction: about 25 cm in the insulated section and close to 50 cm elsewhere, of which 15 to 20 cm occurred during the first 4 months.
3. Settlement differential is highly correlated with the depth of the thaw front; this depth varied from 40 to 90 cm with a maximum thaw occurring under the slope of the embankment. Measured relative settlement is from 20 to 50 percent of the thawed

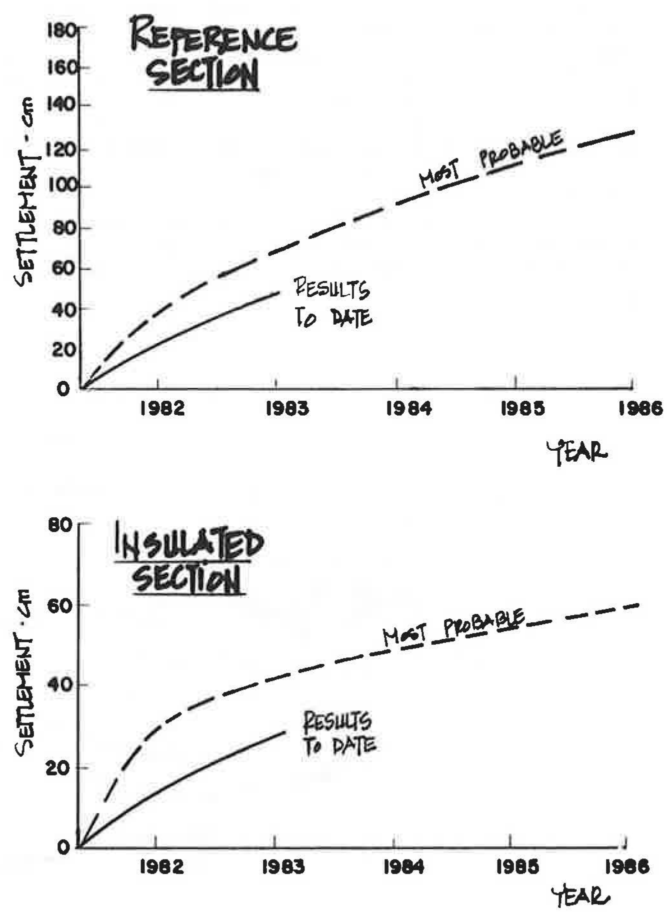


FIGURE 8 Predicted and actual settlement of the test embankment.

depth, which varies with exposure and lateral variation in the natural ice content of the material.

4. Settlement begins in the summer as soon as the fill is thawed, and continues until early winter (December) as the material underneath continues to thaw while the embankment above is again freezing.

5. Maximum thickness of the frozen ground after 2 years decreased from 2.5 m originally to less than 1.5 m, and some areas under and around the embankment thaw completely.

6. Thickness of frozen ground under the fill varies seasonally as the thaw proceeds from both the surface and the bottom. Figure 9 shows typical progress of the thaw line underneath the embankment.

7. Cracks appeared in the slopes and near the edges of the surface soon after construction and continued to develop during the thaw season. Cracking occurs because the thaw is deeper alongside the embankment.

8. Protection of the natural state of peat cover around the embankment is of the utmost importance. As shown in Figure 10, the disturbance of peat by tracked vehicles on the west side of the embankment changed the thermal regime sufficiently to cause complete melting of the palsa and the formation of cracks in the embankment. In contrast, on the east side, where the surface of the palsa has not been disturbed, the active layer is less than 20 cm thick with no degradation of the palsa.

PERFORMANCE OF ROADS THROUGH PALSA FIELD

Description of the Road

The Matagami-LG-2 road is the first paved road ever built in subarctic Quebec. This road, 620 km long, extends north to the 53rd parallel. The pavement is 8 m wide and the shoulders are unpaved. The pavement is composed of 8 cm of high-quality bituminous surfacing, 20 cm of crushed stone base, and 45 to 95 cm of granular subbase. On muskeg, the pavement profile is normally kept 1 to 2 m above the surrounding terrain.

At the time of construction, it was found that frost penetration varied within a range of 3 to 5 m under the pavement and that frost did not disappear until late September in the northern part of the road. Although massive ice was encountered locally during ditch excavations, the existence of palsa was yet unknown at that time.

Evaluation of the Road

The pavement has been subjected to periodic evaluations since the road was opened. Dynaflect deflections were measured once in the summer of 1978 at a rate of four measurements per kilometer. Road roughness has been evaluated eight times since opening of the road to traffic--twice in the winter to measure the effect of winter. The degradation of the surface was identified and quantified in terms of extent and severity in 1978, 1980, and 1982.

In general, the pavement has performed satisfactorily to date, except in settlement zones where periodic leveling and reloading have been required.

Road Settlement

Severe settlement zones where the pavement is badly deteriorated represent around 0.8 percent (or 5 km) of the road length; in known degrading palsa fields settlement zones represent up to 15 percent of the total length (15 km). Figure 11 shows the variation in the lengths of the settlement zones: The average length is about 90 m and the range extends from less than 10 m to more than 300 m.

It is believed that most if not all the major settlements are due to the presence of palsas. Indeed, an extensive investigation conducted in five settlement zones (11) revealed that

1. Major settlements occur mainly in palsa fields;

2. Settlement far exceeds that which is predictable by geotechnical calculation; the level of the soil-embankment contact varied from 1.5 to 2.5 m below the natural ground level whereas 20 to 30 cm could have been anticipated from soil consolidation;

3. Palsas are still in existence and cause degrading even 10 years after construction of the embankment (1972-1973 ; in one location, where no palsa was visible along the road, an icy core about

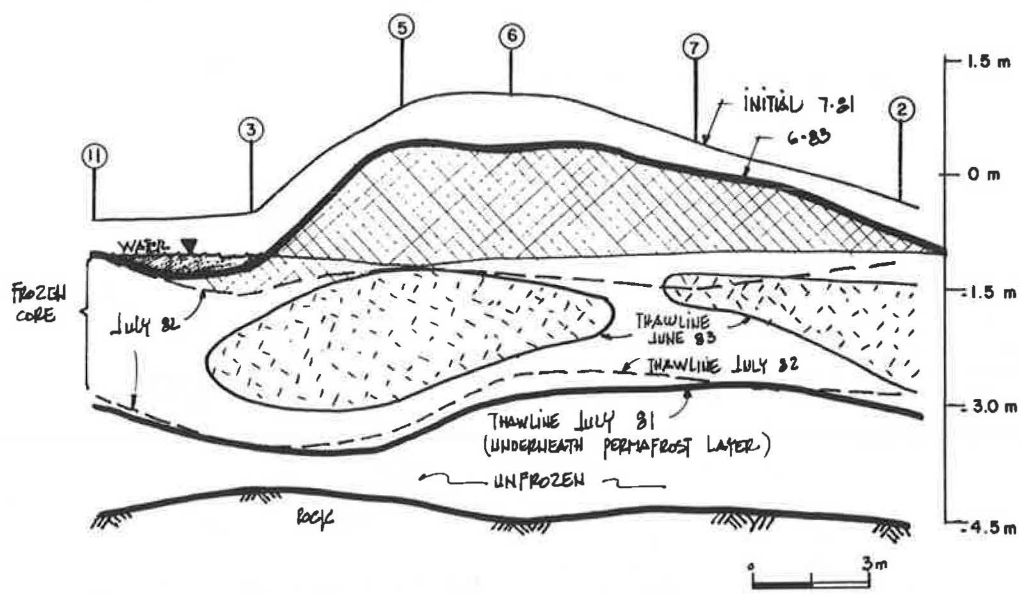


FIGURE 9 Progress of thaw line underneath test embankment (1981-1983).

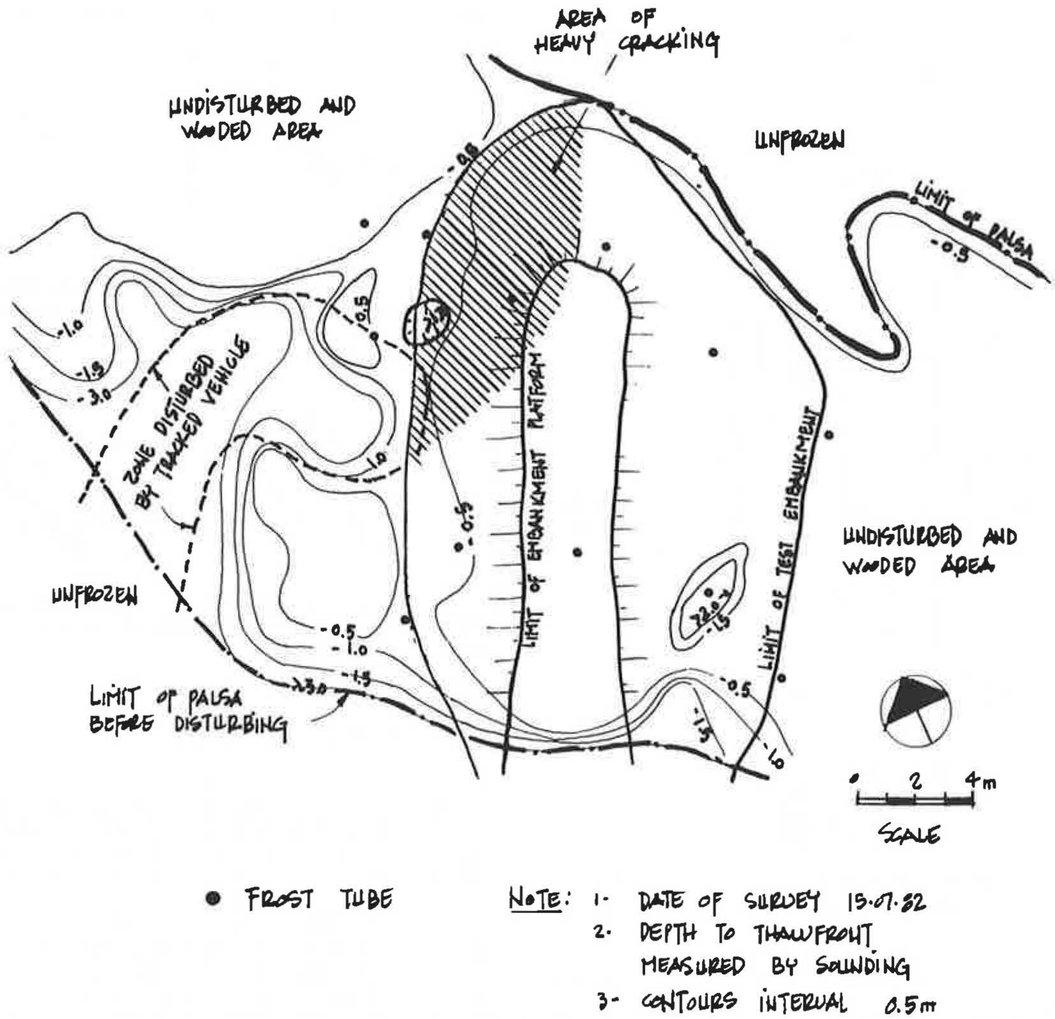


FIGURE 10 Melting of palsa and cracking caused by the disturbance of peat by tracked vehicles around test embankment.

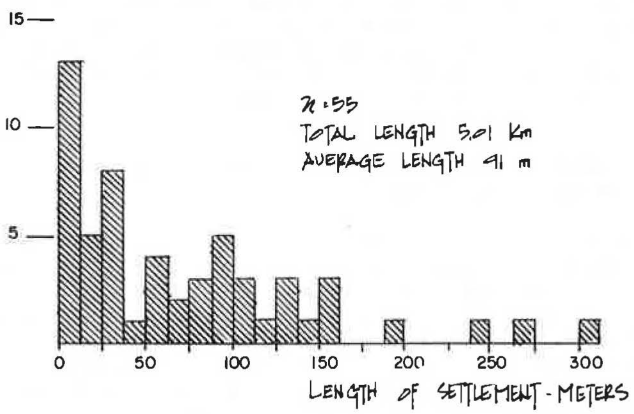


FIGURE 11 Histogram showing variation of length of settlements.

10 m in diameter and 1 m thick (Figure 12) was found 4.2 m from the surface of the road; the temperature of the ice was about 0°C; and

4. Settlements generally appear either in late fall or in early summer and are highly differential; they are rapid at first (10 to 20 cm in a very short period) and slower during each following thaw season (5 to 10 cm or more).

To evaluate past and future settlements, several test pits and borings have been made. The results, described hereafter, are shown in Figure 13. Note that

- Palsas under settlement sections have completely disappeared except under the settlement area detected in 1981 and described earlier;
- Total settlement varies between 1.0 and 1.3 m;
- Maximum annual rate of settlement varies between 20 and 30 cm; and
- Settlements observed on the road compare well with those of the experimental embankment.

The small palsa found in 1981 settled 8 cm in early summer, became unstable in fall, and suddenly settled 20 cm in the spring of 1982.

Road Stability

The stability of the palsa embankment system depends to a great extent on how the road covers the palsa. (Figure 14).

1. When the road cuts through the palsa and the palsa is large compared with the width of the embankment (as is the case of the experimental embankment), the performance of the road will be as shown

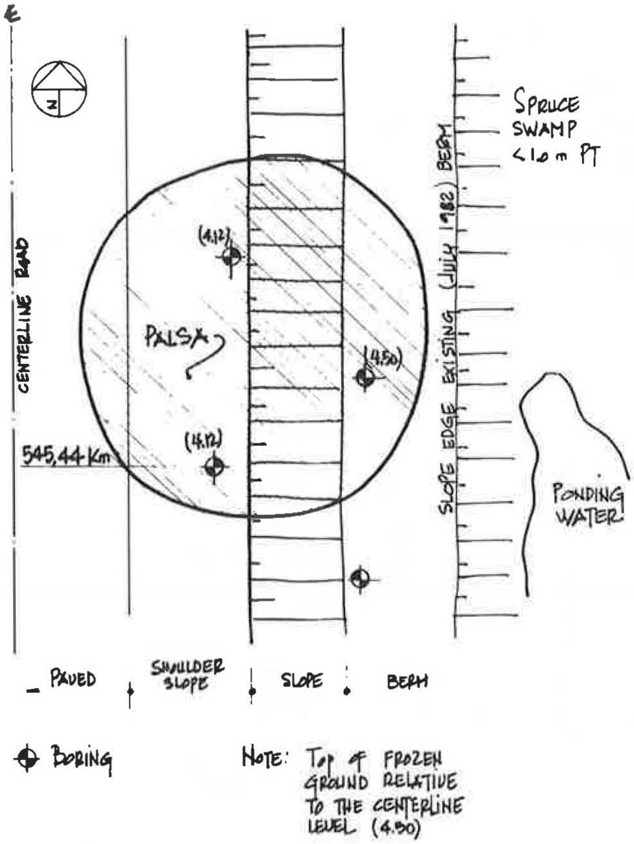


FIGURE 12 Degradation caused by a small palsa found 8 years after construction underneath 4.2 m of fill.

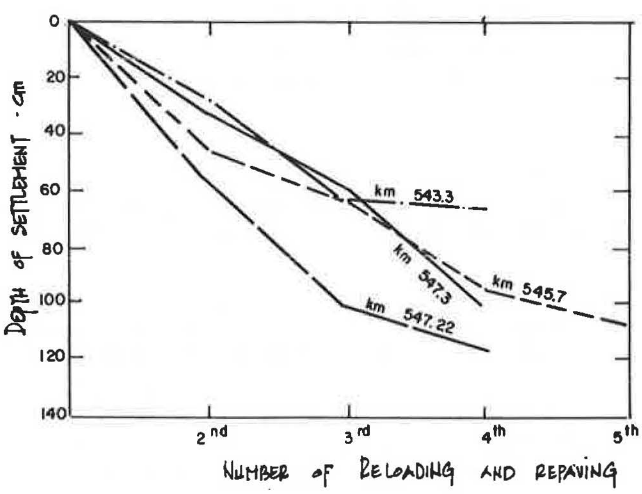


FIGURE 13 Progressive subsidence of five settlement areas over palsa field.

in Figure 15; the differential evolution of the thaw front underneath and along the sides of the embankment will create local instability under and at the edges of the embankment.

2. When the palsa is smaller than the width of the embankment, thawing of palsa will create a locally unstable area where depressions will eventually occur.

3. When the road embankment cuts through the edge of a palsa and covers it only partly (most cases), differential settlements will result between

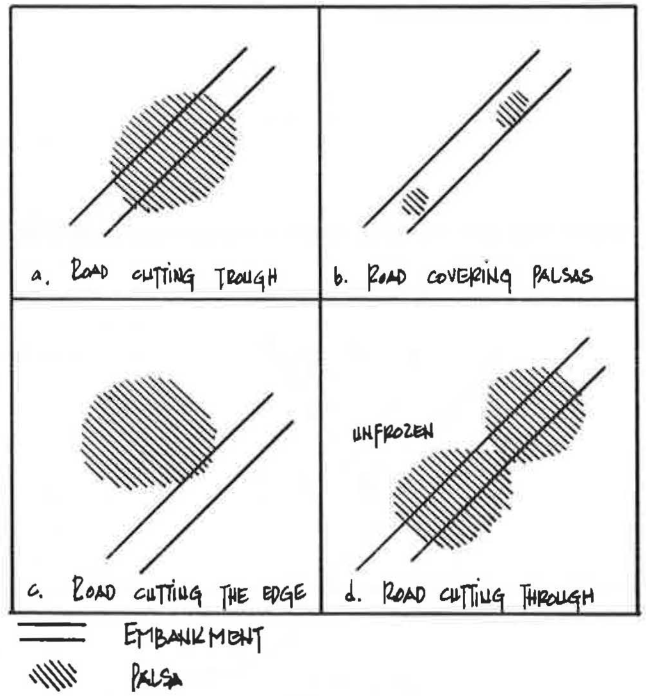


FIGURE 14 Typical palsa crossing.

the unfrozen zone, which will settle rapidly, and the frozen zone where the settlement rate will depend on the progress of the thaw front. Furthermore, instability will result laterally toward the frozen zone as the palsa thaws.

4. When the embankment cuts through several palsas separated by nonfrozen zones, local instability and differential settlement are concentrated at the transition zones, but, because the drainage distance is small, the pore water pressure is low and consequently the stability problem is attenuated.

Note that stability can be greatly affected by the quality of the embankment material; for instance, it was found that where the material is too fine and uniform it will not respond well to dynamic or pluvial compaction and will stay loose beneath the water table and above the thawed soil.

RECOMMENDATIONS FOR DESIGN AND MAINTENANCE OF EMBANKMENT OVER PALSA

Because palsa is a relatively high-temperature permafrost feature, any disturbance of the thermal regime by road or other types of construction may or will initiate thawing. Because palsa has a high ice content, its melting may lead to serious stability problems. Based on all field investigations and laboratory test results, a design approach is suggested. The main steps are shown in Figure 16 and briefly described.

1. Locate palsa field; this can best be done by interpretation of aerial photographs;
2. Select alignments to avoid palsa fields; if this is impossible follow step 3;
3. Determine the exact location of palsas and their geometric and geotechnical properties: the problem of detecting palsas has been discussed elsewhere (12,13); however, based on work in subarctic Quebec where the overburden is relatively thin, more

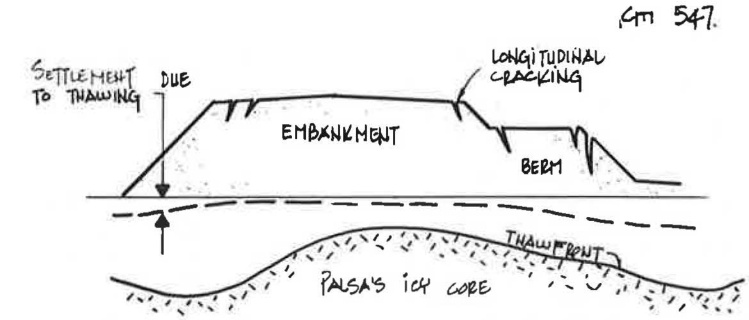


FIGURE 15 Typical thaw-front profile underneath embankment over palsa and related distress.

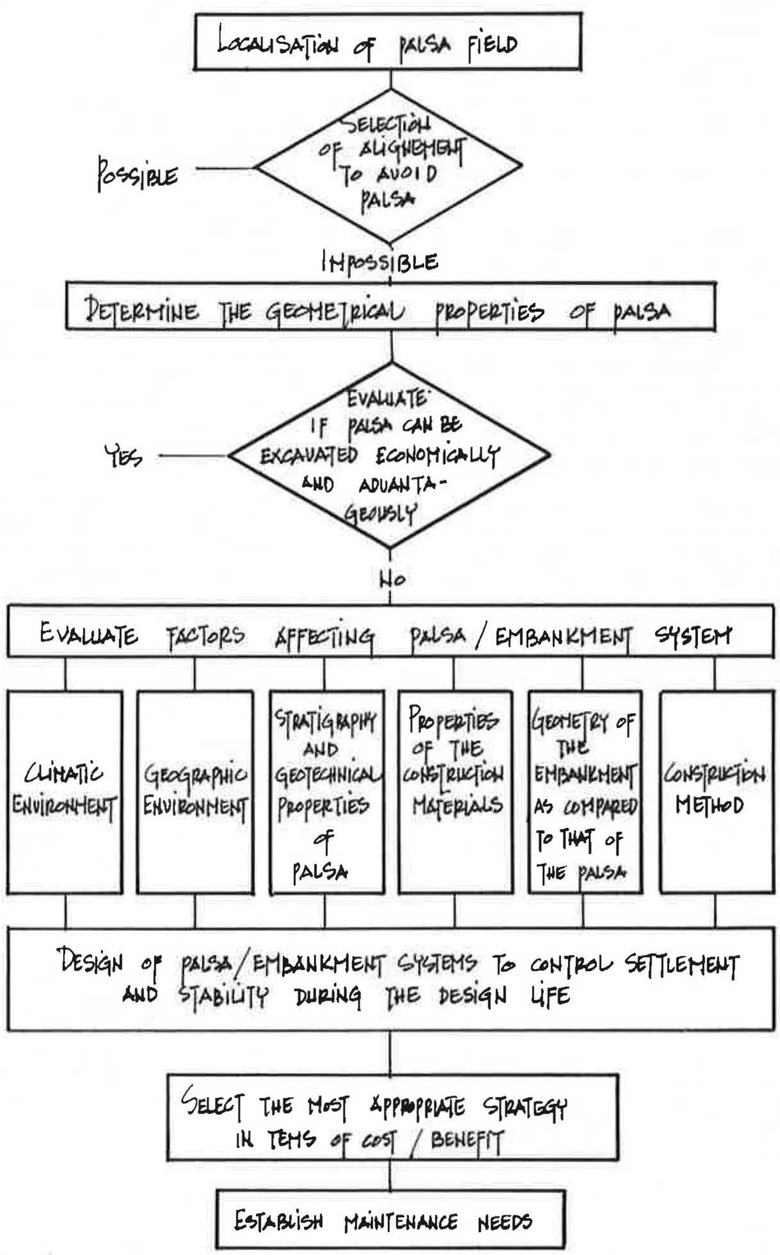


FIGURE 16 Steps involved in the design of embankment over palsa.

research is needed to develop reliable detection methods (14,15);

4. Evaluate if the palsa can be excavated advantageously; this is usually the case when the palsa is small (Figure 14b) or when the proposed embankment cuts through the edge of the palsa (Figure 14c);

5. Define parameters influencing the design of palsa embankment system; the main parameters are (a) the climatic environment of the palsa: temperature, snow and rain precipitation, wind speed, duration of sunlight, freeze-thaw frequency; (b) the geographical environment: profile of palsa above surrounding ground, drainage condition, condition of peat and vegetation, existence of degraded zones and water ponding; (c) the stratigraphy and geotechnical properties of palsa materials such as those given in Table 1; (d) the geometry of the embankment compared with that of the palsa: height, width, and lateral slope of the embankment, relative position of the embankment over the palsa, exposure of the embankment to wind, sun, and snow; (e) the properties of embankment materials: type, permeability, capillarity, filtration capability, saturated and unsaturated thermal conductivity, and mechanical stability when saturated; and (f) the construction method: water level and thaw profile during construction and degradation of the natural environment during construction.

6. Design the palsa embankment system to control settlement and stability.

The designer should bear in mind that, considering the nature and the thermal regime of palsas described in this paper, complete frost protection is uneconomical and impossible even with artificial insulation. Therefore, any design will eventually result in a progressive degradation and disappearance of the palsa. The object of the design is to assure stability and to either accelerate the settlement rate or lengthen the settlement period in relation to the proposed uses and anticipated life of the road.

Note that different approaches can be used in the design: The rate of settlement can be controlled by the thickness of the embankment that, when dry, acts as an insulating layer; the total amount of settlement can be diminished by partial excavation of the palsa's icy core; and the stability of the system can be assured by a proper selection of embankment material and appropriate construction control to assure the preservation of the environment surrounding the palsa.

In any case, when the thawed material under the fill is saturated peat or silty clay of very low bearing capacity or both, the system must be stabilized; it might be advantageous to use geotextiles either as an anticontaminant or as a reinforcing material. If high instability is suspected, berms can be designed for the embankment in the same way as is done for embankments over unstable muskeg (16).

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Settlement Rates in the Varved Clays of the Hackensack Meadowlands

A. A. SEYMOUR-JONES

ABSTRACT

Field settlement and piezometer data for four highway construction projects have been used to determine the effective rates of consolidation in the varved clays of the Hackensack meadowlands. The data were used to evaluate the relative performance of sand drains installed by displacement and nondisplacement methods and areas where sand drains were not used. The effects of the use of sand drains and sand drain spacing are evaluated.

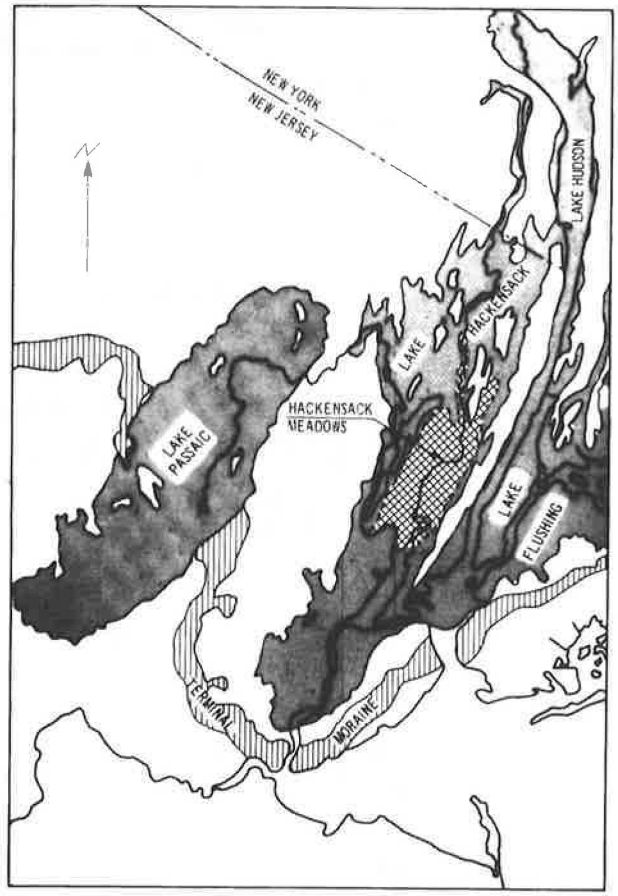

FIGURE 1 Glacial geologic setting of the Hackensack meadowlands.

Field settlement and piezometer data have been analyzed for four highway construction areas in the Hackensack meadowlands to determine the effective field rate of consolidation. These data are compared with laboratory and field permeability tests that were made as part of the original design. The field consolidation data were also used to evaluate the effectiveness of sand drains, the relative efficiency of displacement and nondisplacement sand drains, and the effect of sand drain spacing on the rate of consolidation.

LOCATION

The Hackensack meadowlands are located in northeastern New Jersey, approximately 3 miles west of New York City as shown in Figure 1. This site is a former glacial lake that extended over a considerably larger area known as Glacial Lake Hackensack (1). The four highway construction areas analyzed are portions of the New Jersey Turnpike and are labeled A, B, C, and D in Figure 2.

GEOTECHNICAL CHARACTERISTICS

Typical boring logs for these four areas are shown in Figures 3-6. The general soil profile consists of a surface layer of peat, which has been covered or replaced by fill in built-up areas, underlain by a sand layer in most locations. The varved clays that are the topic of this paper are located beneath this sand layer. The varved clays range from 65 to 130 ft in thickness at these locations. Beneath the varved clays is a sand and gravel glacial till layer that overlies a shale and sandstone bedrock.

The varved clays consist of individual varves 1/16 to 1/2 in. thick. Each varve consists of a spring-summer deposition that varies from a fine sand or silt to a clayey silt and a fall-winter deposition that varies from a silty clay to a clay. Close visual analysis of many boring samples shows that the initial spring deposition is a very thin parting of fine sand or silt overlain by the clayey silt deposited during the remainder of the spring-summer period. In some samples this sand or silt parting was missing. Figure 7 shows plasticity data for whole varves and for the separate varve components.

The boring profiles in Figures 3-6 are for the four areas and show that the upper 20 to 30 ft of the varved clay have been desiccated resulting in overconsolidation of the deposit beneath this desiccated crust by 0.5 to 2.0 tsf. These four figures show that conditions within the varved clay are very similar at areas A, C, and D where the overconsolidation due to desiccation is approximately 0.5 tsf. The overconsolidation at area B is significantly greater, approximately 2.0 tsf, as shown in Figure 4. The effect of this greater overconsolidation on the shear strength can also be seen in this figure. The present overburden pressure or overburden pressure noted in these four figures is the overburden pressure before construction of the highway projects discussed in this paper. Imposed highway embankment loads for these areas ranged from 0.6 to 1.8 tsf.

Numerous horizontal permeability tests, both laboratory and field, were made on these varved clays as part of the original highway design. Figure 8 shows permeability test results for area C, which

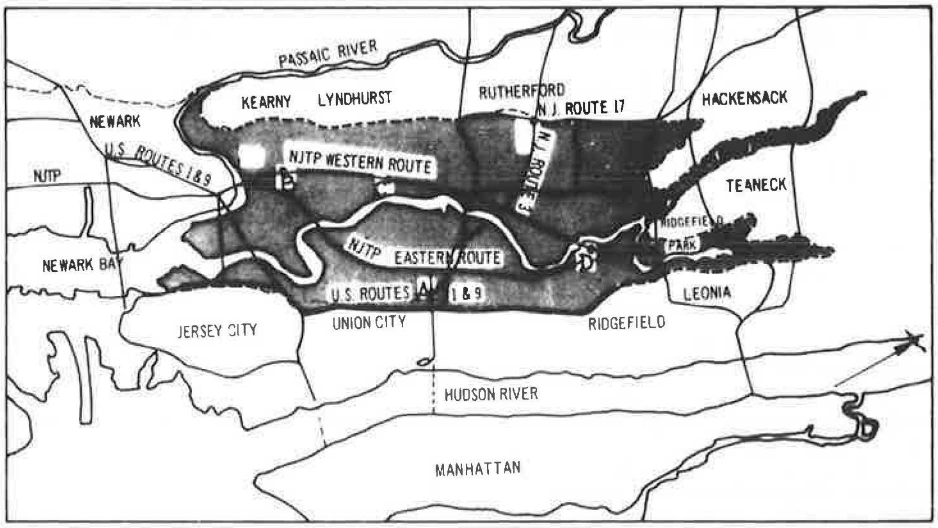

FIGURE 2 Location of study areas.

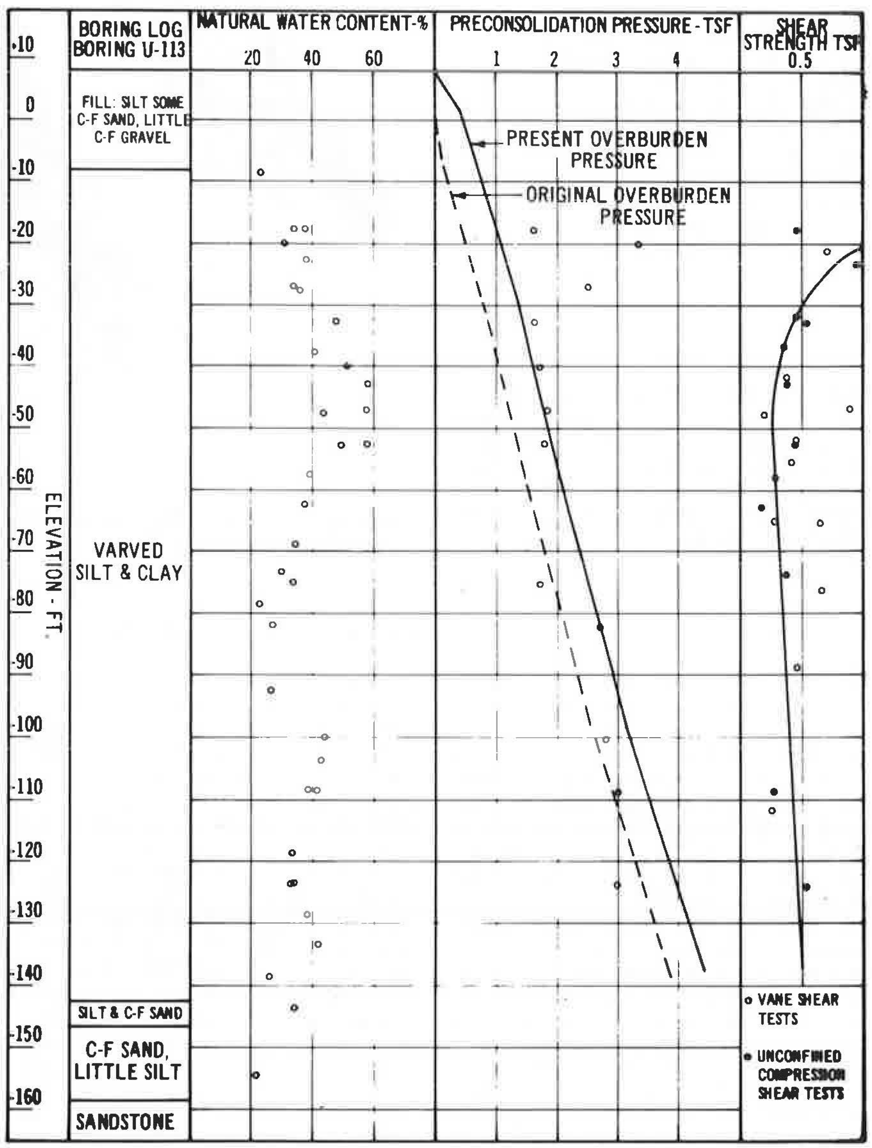

FIGURE 3 Typical boring log for area A.

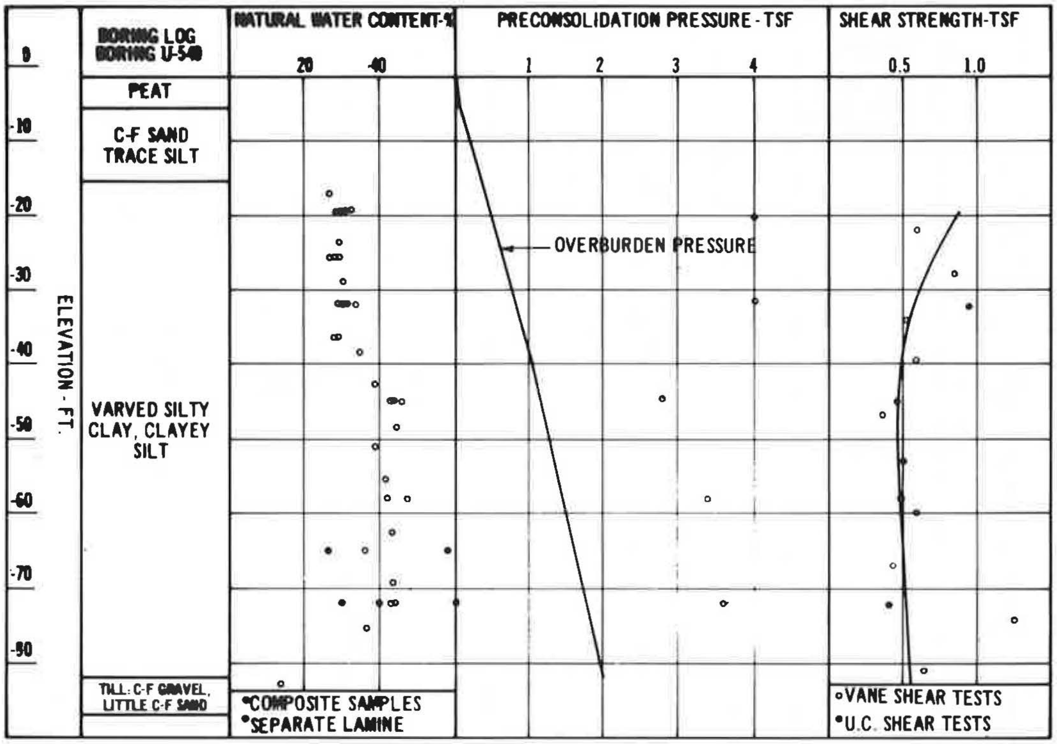

FIGURE 4 Typical boring log for area B.

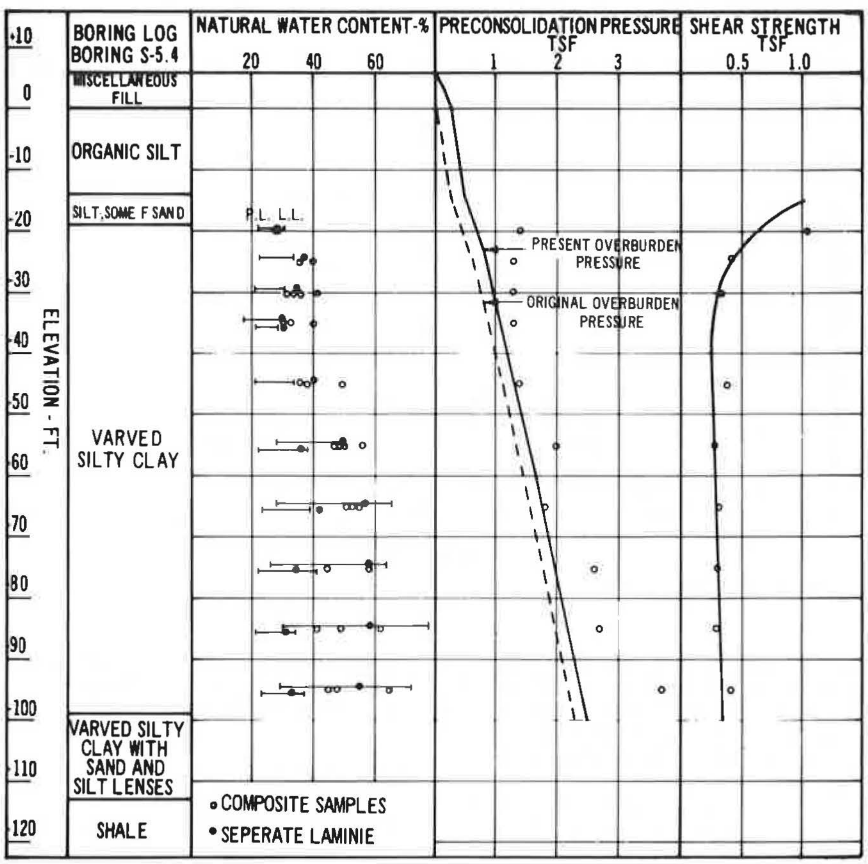

FIGURE 5 Typical boring log for area C.

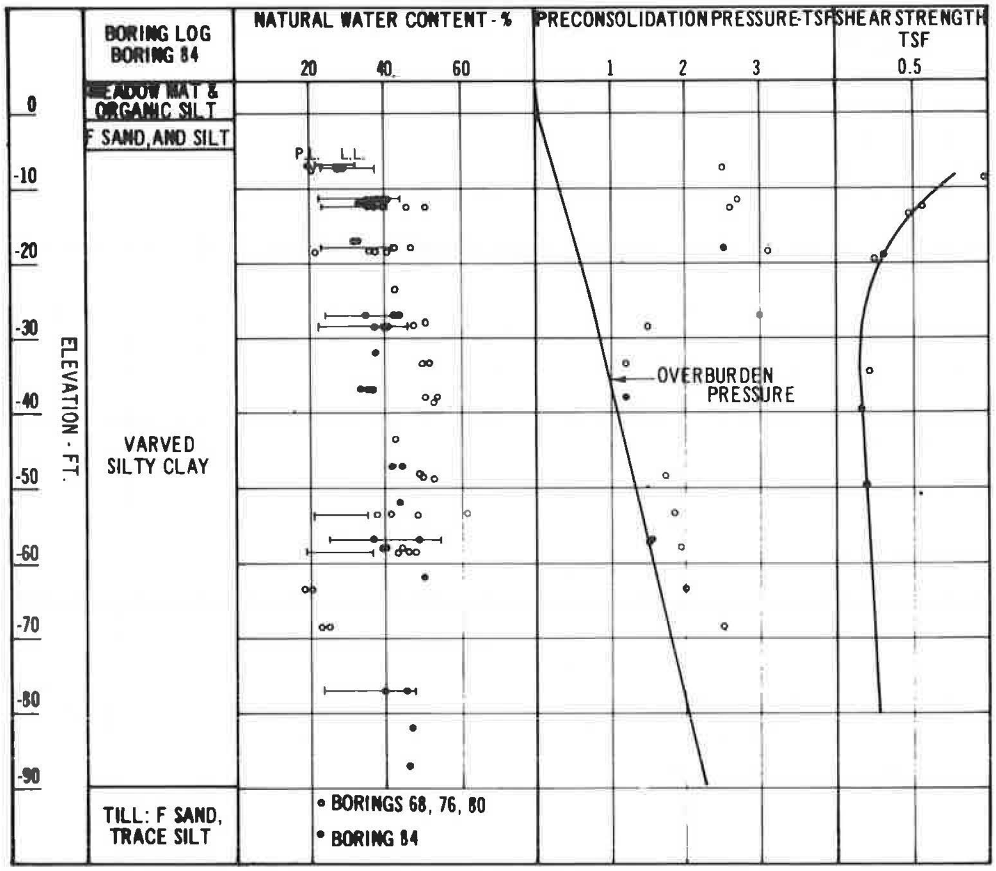

FIGURE 6 Typical boring log for area D.

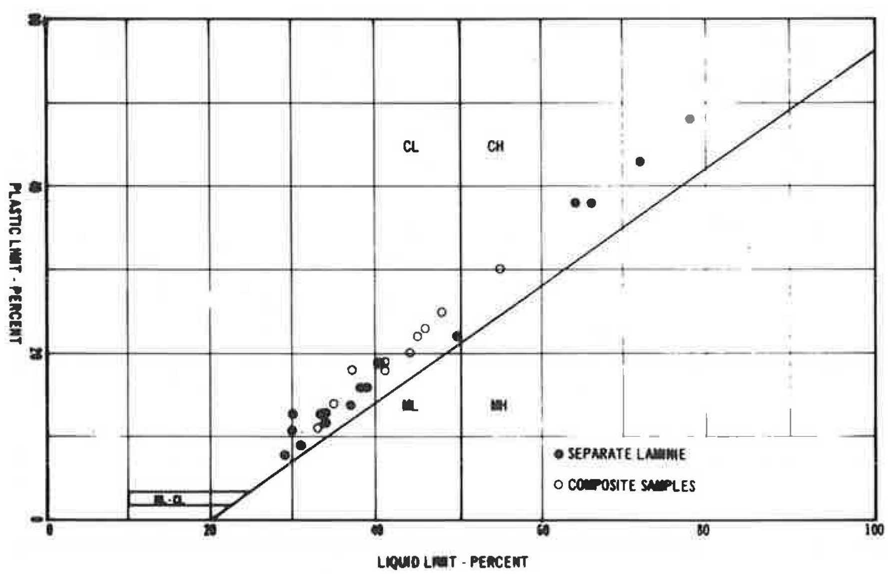

FIGURE 7 Atterberg limit data for varved clays.

are typical for these varved clays. Included in this figure are two large-scale permeability tests made on individual sand drains by Casagrande and Poulos (2). These tests consisted of a single wash sand drain and a single driven sand drain that were instrumented with adjacent piezometers to determine the effective rate of horizontal permeability for each. Casagrande and Poulos (2) have shown the horizontal permeability to be 8 to 20 times greater than the vertical permeability for these varved clays.

Permeability data obtained from areas B and D are similar to those for area C. The noticeable decrease in permeability with depth is significant.

EMBANKMENT CONSTRUCTION METHODS

Typical sections of the embankment construction for non-sand drain and sand drain areas are shown in Figure 9. In the areas where sand drains were not

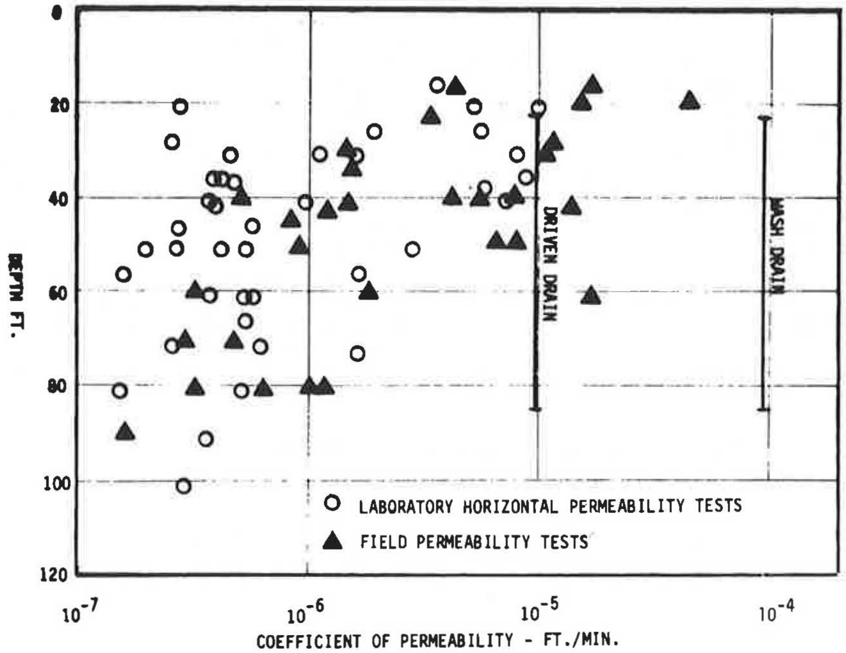

FIGURE 8 Laboratory and field permeability data for area C.

used the sequence of construction employed was to (a) excavate the surface layer of peat and then place clean backfill; (b) install settlement platforms, piezometers, and any other instrumentation; and (c) place the embankment fill at a controlled rate to maintain embankment stability. In the areas where sand drains were employed the construction sequence was similar except that the sand drains were placed after the backfilling of the peat excavation and before the installation of the instrumentation.

Two types of sand drains were used: displacement

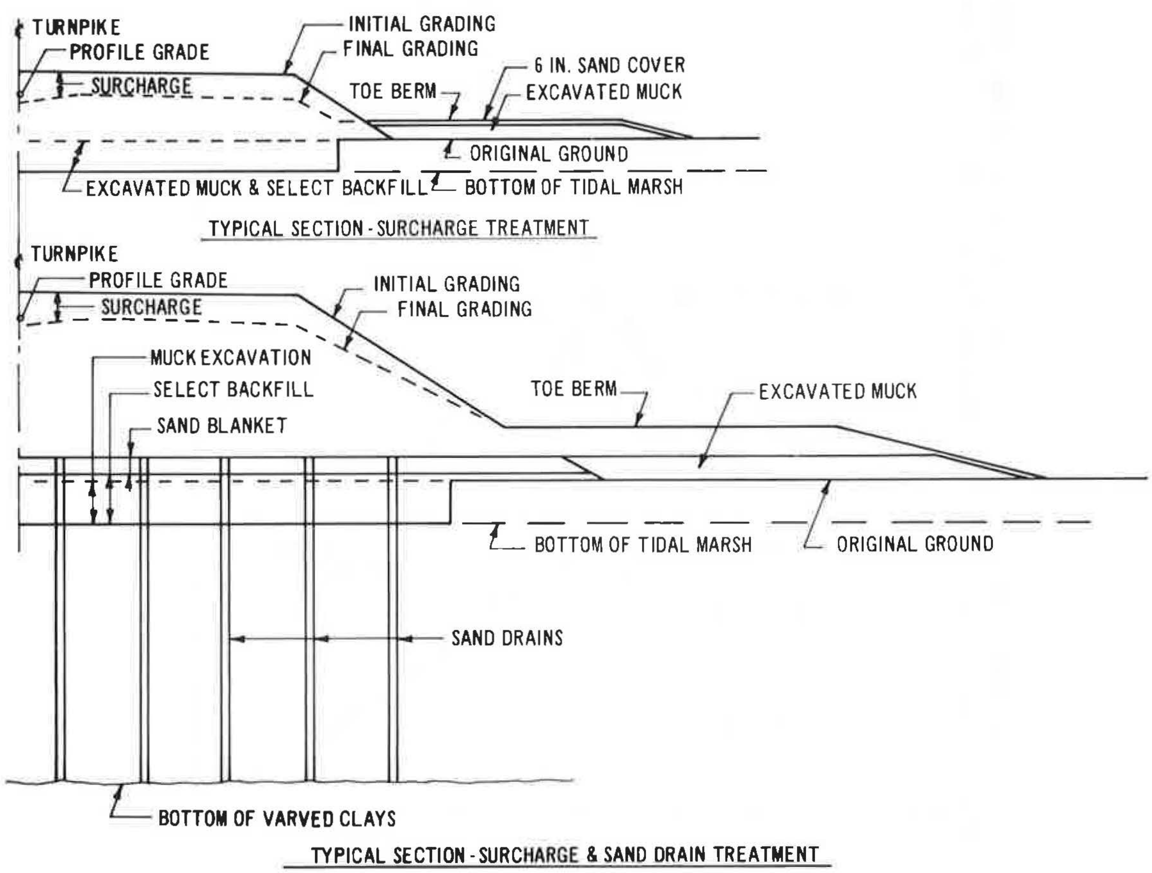

FIGURE 9 Typical embankment sections for area C.

and nondisplacement. The displacement sand drains were used in areas A, B, and D and were constructed by driving a closed-end, 18-in.-diameter mandrel and placing sand as the mandrel was withdrawn. The non-displacement sand drains used in area C were placed by the Raymond method, which consists of jetting a 20-in.-diameter hole with a fish-tail bit and jet pipe, inserting an 18-in.-diameter closed-end mandrel, and placing sand as the mandrel is withdrawn. Summaries of the treatment methods for the four areas are given in Table 1.

FIELD DATA AND ANALYSIS

The field data analyzed consisted of settlement readings obtained from settlement platforms and pore pressure readings obtained from piezometers located beneath the center of the embankments and within the varved clays below the upper desiccated portion at depths ranging from 30 to 100 ft below the original ground surface. Typical piezometer and settlement platform data for area C are shown in Figure 10. The platform settlement readings were plotted against the square root of time to determine the point of 90 percent theoretical consolidation as developed by Taylor (3).

The piezometer readings were analyzed using Skempton's relationship between applied pressure and pore pressure (4). Normally the values for Skempton's coefficient, A, are obtained from triaxial tests with pore pressure measurements. Because such

TABLE 1 Summary of Treatment and Observation Methods

Area	Treatment Methods and Sand Drain Spacing[a]: Surcharge without Sand Drains	Displacement Sand Drains (ft)	Nondisplacement Sand Drains (ft)	Observations: Piezometers	Settlement Platform
A		8, 10, 14			X
B		14		X	X
	X			X	X
C			20, 40	X	X
	X				X
D		14, 16, 20		X	X

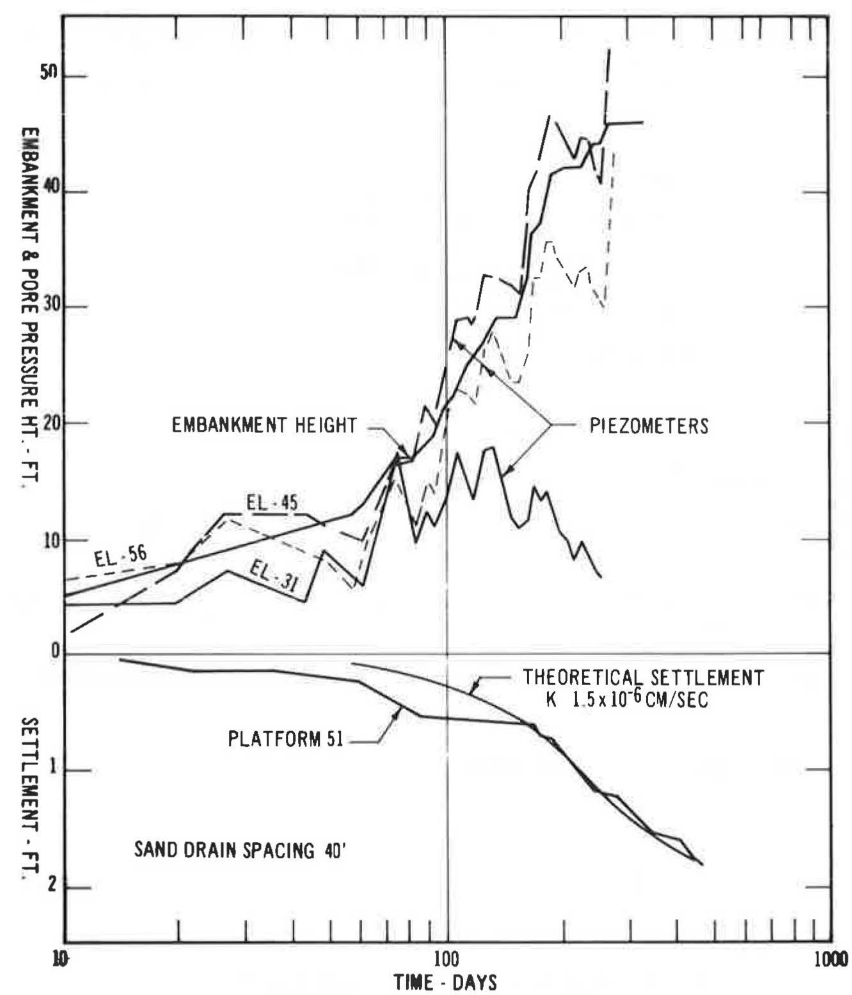

FIGURE 10 Typical piezometer and settlement platform data for area C.

data were unavailable, values for A recommended by Skempton (4) were used: A = 1.0 for conditions where the applied load exceeded the soil preconsolidation pressure and A = 0.5 for conditions where the applied load was judged to be less than the soil preconsolidation pressure.

The previously noted field data provided the basis for calculating the coefficient of consolidation. The theory developed by Barron (5) was used for the areas where sand drains were employed. The methods of analysis developed by Fungaroli (6) and that developed by Davis and Poulos (7), which are based on horizontal drainage only, were used for the non-sand drain areas. A significant difference in the calculated coefficient of consolidation was obtained by the latter two methods and is discussed in the next section of this paper.

It was found that the range in values of the calculated coefficients of consolidation was four orders of magnitude. This range was much wider than expected. In an attempt to explain this wide range of results it was decided to use the calculated coefficient of consolidation data to calculate horizontal permeabilities that could be compared with results of laboratory and field horizontal permeability tests made during the design phases. The relationship between horizontal permeability and coefficient of consolidation is

$$K_h = C_r\,(\Delta\varepsilon/\Delta P)\,\gamma$$

where

K_h = horizontal permeability (cm/sec),
$\Delta\varepsilon$ = change in strain due to embankment load,
ΔP = change in stress due to embankment load (kg/cm²),
γ = unit weight of water = 0.001 kg/cm³, and
C_r = coefficient of consolidation (cm²/sec).

For the determination of $\Delta\varepsilon$ and ΔP, calculated settlements were determined by use of one-dimensional consolidation tests. The value of $\Delta\varepsilon$ determined was the total calculated settlement divided by the varved clay deposit thickness. The ΔP used was the average imposed stress over the depth of this deposit. The calculated horizontal permeabilities are shown in Figure 11 and are summarized in Table 2. The equation is the classic consolidation equation developed by Terzaghi for consolidation by vertical drainage modified for horizontal drainage by Barron (5).

An independent assessment of the potential horizontal permeability of the sand and silt partings was developed using data provided by Burmister (8) and is discussed in the next section.

DISCUSSION

A comparison of the various permeability values calculated from the field settlement and piezometer data is worthwhile to evaluate the range of results and the probable causes of these variations. A comparison of these data with the laboratory and field permeability test results provides a basis for judging how useful the latter are for design.

Figure 11 shows the permeability values calculated using the field settlement platform and piezometer readings. The range of values is quite large, about four orders of magnitude. However, there are certain trends that can be observed from this plot. These trends are (a) the non-sand drain areas have much higher permeability values than the sand drain areas, (b) within each of the four separate construction areas the data for the sand drains cover a relatively small spread, and (c) the sand drain data for areas A and B are significantly smaller than those for areas C and D. The remainder of this discussion evaluates potential causes of these observed conditions.

Settlement Platform Versus Piezometer Data

The ranges of calculated horizontal permeability values based on piezometer data shown in Figure 11 are about two to three times the range of those calculated from settlement platforms in each of the

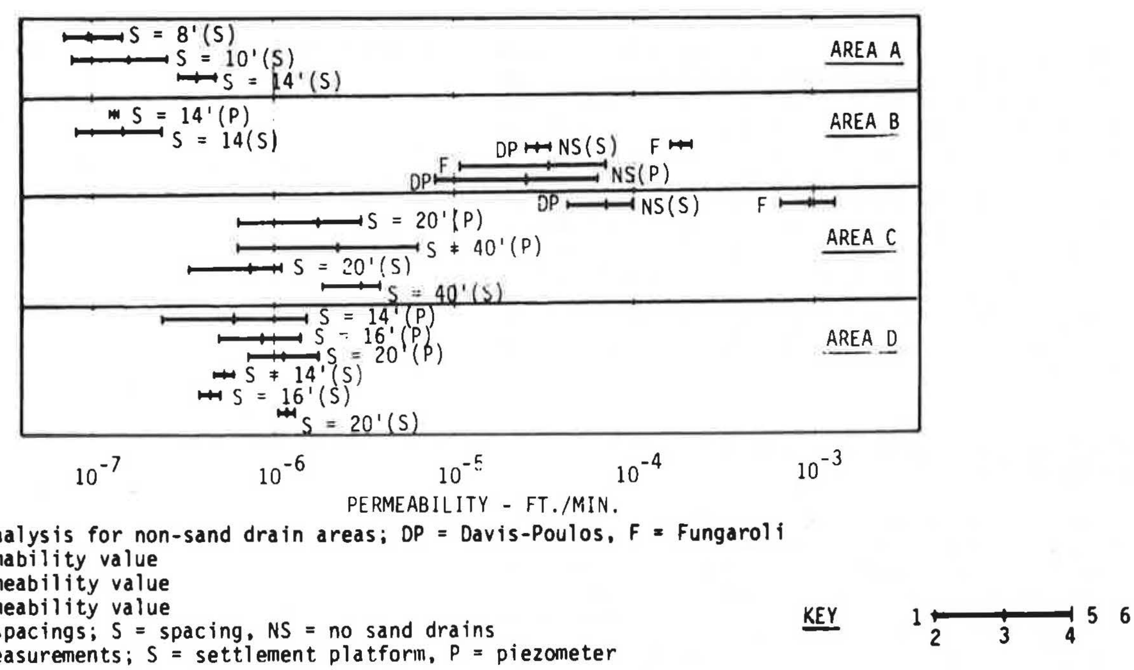

FIGURE 11 Summary of calculated horizontal permeabilities from field settlement and piezometer data.

TABLE 2 Summary of Calculated Horizontal Permeabilities

Area	Sand Drain Spacing [ft (m)]	Data Source[a]	No. of Data	Horizontal Permeability [ft/min x 10^{-6} (cm/sec x 10^{-6})] Maximum	Minimum	Average
A	8	S	6	.14	.06	.10
	(2.44)			(.07)	(.03)	(.05)
	10	S	7	.26	.08	.16
	(3.05)			(.13)	(.04)	(.08)
	14	S	6	.48	.30	.38
	(4.27)			(.24)	(.15)	(.19)
B	14	S	2	–	–	.12
	(4.27)					(.06)
		P	4	.24	.08	.14
				(.12)	(.04)	(.07)
	n/s[b]	S	4	210	160	180
				(105)[c]	(80)[c]	(90)[c]
				36	26	30
				(18)[d]	(13)[d]	(15)[d]
		P	2	72	10	34
				(36)[c]	(5)[c]	(17)[c]
				66	8	26
				(33)[d]	(4)[d]	(13)[d]
C	n/s	S	9	1,260	640	1,120
				(630)[c]	(320)[c]	(560)[c]
				100	44	70
				(50)[d]	(22)[d]	(35)[d]
	20	S	5	1.10	.34	.74
	(6.10)			(.55)	(.17)	(.37)
		P	15	3.0	.60	1.72
				(1.5)	(.30)	(.86)
	40	S	9	3.8	1.8	3.0
	(12.19)			(1.9)	(.90)	(1.5)
		P	18	6.2	.60	2.2
				(3.1)	(.30)	(1.1)
D	14	S	2	.56	.46	.52
	(4.27)			(.28)	(.23)	(.26)
		P	7	1.52	.24	.60
				(.76)	(.12)	(.30)
	16	S	2	.50	.38	.44
	(4.88)			(.25)	(.19)	(.22)
		P	8	1.40	.50	.86
				(.70)	(.25)	(.43)
	20	S	2	1.26	1.04	1.16
	(6.10)			(.63)	(.52)	(.58)
		P	7	1.76	.72	1.12
				(.88)	(.36)	(.56)

[a]S indicates data from settlement platform readings, P indicates data from piezometer readings.
[b]n/s indicates no sand drains used.
[c]Data based on Fungaroli method of analysis.
[d]Data based on Davis and Poulos method of analysis.

individual areas. This result is reasonable because the piezometers represent only conditions at a local point within the varved clay deposit, whereas the settlement platforms represent the average condition for the full depth of the deposit. Figure 8 shows, based on laboratory and field tests, a wide range of permeabilities due to the natural variability of the soil. This is confirmed by field piezometer data. It appears that averages of the permeability results from a number of piezometers in any area provide a reasonably good representation of the average horizontal permeability for the total varied clay deposit thickness as determined from the settlement platform data.

Comparison of Horizontal Permeability in Different Areas

Horizontal permeability data for areas A and B are about one order of magnitude smaller than those for areas C' and D. If only the data for sand drains with 14-ft spacing are reviewed (Table 3), the permeabilities for areas A and D are generally close together and significantly higher than those for area B. The one known significant difference between the soils of area B and those of the other two areas

TABLE 3 Selected Horizontal Permeability Data

Area	Sand Drain Spacing (ft)	No. of Data	Average Horizontal Permeability (ft/min x 10^{-6})
Comparison of Data for 14-ft Sand Drain Spacing			
A	14	6	0.38[a]
B	14	2	0.12[a]
C	14	2	0.50[a]
		7	0.60[b]
Comparison of Displacement and Nondisplacement Sand Drains			
C	20[c]	5	0.74[a]
		15	1.72[b]
D	20[d]	2	1.16[a]
		7	1.12[b]
Comparison of Fungaroli with Davis and Poulos Methods of Analysis			
B	–[e]	2	34[b,f]
			26[b,f]
		4	180[a,f]
			30[a,g]
C	–[e]	9	1,120[a,f]
			70[a,g]

[a]From settlement platform data.
[b]From piezometer data.
[c]Nondisplacement sand drains.
[d]Displacement sand drains.
[e]Non-sand drain areas.
[f]Fungaroli method of analysis.
[g]Davis and Poulos method of analysis.

is that the area B soils have a higher preconsolidation, about 2 tsf compared with about 0.5 tsf for the other areas. This preconsolidation could have an effect on horizontal permeability, but it is doubtful if it is of any significance compared with the natural variation in the horizontal permeability discussed previously. The reason for this statement is that the imposed loadings in some sections of areas A and D were of a magnitude of 1.8 tsf and consequently resulted in total pressures of up to 2.0 tsf, the preconsolidation pressure for area B. If maximum pressure had a major effect on horizontal permeability, the permeabilities of these areas should all have been relatively close. Consequently the difference in effective horizontal permeability for area B compared with the other three areas must be due to depositional or other conditions rather than to the difference in preconsolidation pressure.

Displacement Versus Nondisplacement Sand Drains

The calculated horizontal permeability data provide a means of evaluating the relative efficiency of displacement and nondisplacement sand drains. The horizontal permeability test drains plotted in Figure 8 show that the nondisplacement sand drain horizontal permeability is one order of magnitude higher than that for the displacement sand drains. The data in Figure 11 and Table 3 indicate that the 20-ft drain spacing for area C where nondisplacement sand drains were used had essentially the same permeability as did displacement sand drains with the same spacing in area D. Permeabilities for both driven- and wash-type sand drains are one order of magnitude lower than that of the lowest test drain shown in Figure 8.

The wash drain used for the tests (2) shown in Figure 8 was constructed by jetting a pipe down and then backfilling with sand. The production nondisplacement sand drain was constructed by a different method, as noted previously. The different construction methods employed could possibly explain part of the difference between the permeabilities calculated for the two types of nondisplacement sand drains. However, the difference between the test and production displacement sand drains cannot be explained on this basis because the construction of both types was essentially the same. It is believed that there is another factor causing the difference. This is discussed later.

Non-Sand Drain Areas

Analysis of the non-sand drain calculated horizontal permeability data provides some interesting results. The calculated horizontal permeabilities for the non-sand drain areas are at least one to two orders of magnitude greater than that for any of the sand drain areas. The most logical explanation of this difference is the disturbance effect on the soil permeability resulting from the sand drain construction.

Based on the permeability data for the two test sand drains reported by Casagrande and Poulos (2), the calculated permeabilities for these non-sand drain areas provide reasonable results and indicate that the theoretical methods used to calculate the permeabilities appear to be valid. Both methods (6,7) used to calculate the horizontal permeability based on the field settlement and piezometer data are based on horizontal drainage only. This condition is reasonable for this varved clay deposit because of the very high ratio of horizontal to vertical permeability noted previously (2).

A comparison of the results of the two methods of analysis was made (Table 3) and is of interest. Results for the two methods are fairly close based on the piezometer data. Horizontal permeability results for the settlement platform data using the Fungaroli method are about one order of magnitude greater than results obtained using the Davis and Poulos method. Laboratory permeability test data on prepared samples of silts and fine sands (8) were used to help evaluate this difference in results. Based on past visual examinations of varved clay samples it was judged that a high percentage of the varves could contain 1 to 5 percent silt or fine sand and silts. The permeability results for this 1 to 5 percent content of (a) a coarse silt, (b) a fine sand and coarse silt, and (c) a fine sand were calculated from the Burmister data and plotted in Figure 12 along with the calculated permeabilities for the non-sand drain areas. This comparison and a comparison of the other permeability data shown in Figures 8 and 11 indicate that the field permeability data calculated by use of the Davis and Poulos method appear to be the more reasonable.

A detailed review of the theoretical background of these two methods of analysis seems to be needed because there is no obvious reason for this difference. One possible reason for the difference in results obtained using the two methods of analysis

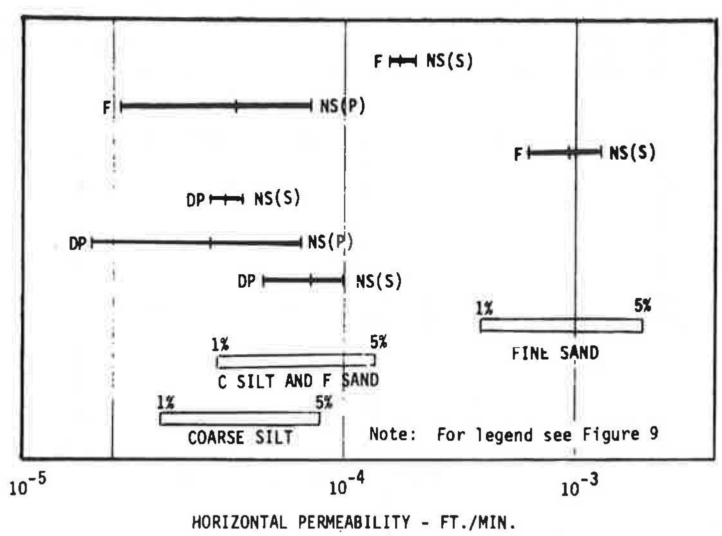

FIGURE 12 Field permeability data for non-sand drain areas.

is the boundary drainage conditions. The Fungaroli method assumes the soil area beyond the limits of soil consolidation swells in volume equal to the volume of soil consolidation. The Davis and Poulos method assumes the soil area beyond the limits of soil consolidation is free draining.

The calculated permeabilities for these two non-sand drained areas equal or exceed the highest field and laboratory permeability tests as do the results for the two test sand drains shown in Figure 8. These two observations indicate that for varved clay deposits small-scale laboratory and field permeability tests generally will result in calculated permeabilities that are significantly less than the true effective horizontal permeabilities of the deposit.

Effect of Sand Drain Spacing

The range of calculated field horizontal permeabilities for all the sand drain areas is of two orders of magnitude and this range can be attributed at least in part to the normal variability in the in situ permeability shown in Figure 8 and discussed previously. There is another possible contributing factor, the spacing of sand drains. The relationship between the average calculated horizontal permeability and the sand drain spacing is shown in Figure 13. These data indicate that the closer the sand drain spacing, the lower the calculated horizontal permeability. From these data it appears that smear or disturbance from the driving of the sand drains has a significant effect on the horizontal permeability and the dependent rate of consolidation. A similar trend for sand drains in tidal marsh deposits has been noted (9).

The calculated horizontal permeabilities for the non-sand drain areas are of one to two orders of magnitude greater than the calculated horizontal permeabilities for the sand drain areas. This condition may be the result of the lack of any disturbance or smear of the more permeable portion of the varve layers in the non-sand drain areas.

The data presented in the two preceding paragraphs give strong evidence that the use of sand drains in varved clay deposits greatly reduces the effective horizontal permeability of the deposit. The reduction in permeability due to the use of sand drains in tidal marsh deposits (9) was much less.

This summary of field permeability data provides a guide for the calculation of settlement rates for future construction projects in the varved clays of these areas of the Hackensack meadowlands. Laboratory or small-scale field permeability tests can provide a basis for estimating consolidation rates, but a significant number of tests, at least 6 to 12, are necessary to develop the range of permeability conditions.

Figure 11 also provides a guide for the choice of design permeability values. The mid-to-lower portion of the permeability range is applicable for sand drains; the lower portion should be used for closely spaced drains, and the midportion used for more widely spaced drains. The upper portion of the permeability range should be applicable for non-sand drain areas.

Depending on comparative soil types and index properties, these data may provide useful guidance for other areas containing varved clays within both the Hackensack meadowlands and other glacial lake deposits.

CONCLUSIONS

Following are the conclusions derived from the comparison of horizontal permeability of the varved clay deposit calculated from field settlement and piezometer data.

The wide range in horizontal permeabilities derived from small-scale laboratory and field tests was confirmed by the embankment piezometer data.

The actual embankment settlement rates indicate that the effective horizontal permeability for the sand drain areas falls in the lower half of the range of results obtained from the small-scale laboratory and field permeability tests.

There is a significant difference in the horizontal permeability of area B compared with areas A, C, and D that is apparently due to causes other than the difference in preconsolidation pressure.

The type of nondisplacement sand drain used showed no significant improvement in efficiency over the standard displacement sand drain in this varved clay deposit.

These data give strong evidence that the use of sand drains in varved clays causes a significant reduction in the horizontal permeability of the soil. It also has been observed that the closer the sand drain spacing, the greater the reduction in horizontal permeability. These conclusions show the significant disturbance effect that sand drains can have on varved clay permeability.

The Poulos-Davis method of determining settlement rate appears to provide more realistic results than the Fungaroli method for this varved clay deposit, when both methods are based on consolidation resulting from horizontal drainage only.

The effective horizontal permeability, based on the data from the non-sand drain areas and on the single jetted test drain described by Casagrande and

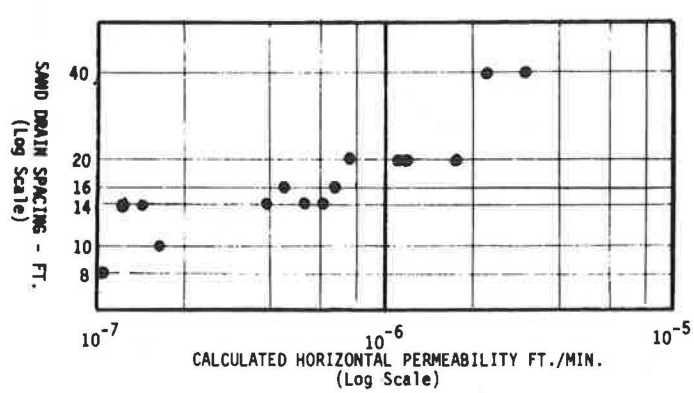

FIGURE 13 Calculated horizontal permeability versus sand drain spacing.

Poulos, seems to be significantly greater than indicated by most horizontal permeability data determined by small-scale laboratory and field permeability tests.

ACKNOWLEDGMENTS

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Pedotechnical Aspects of Organic Soil Classification and Interpretation

GILBERT WILSON

ABSTRACT

Exploration and classification of organic soils in transportation research is done primarily to predict performance and impacts of construction activities. In the preliminary stages, published maps are included in the data base. There is a continuing need for improved methods of interpreting surveys performed by mapping agencies as the state of their art develops. For Canadian soil survey applications, the pedotechnical setting sheet has been proposed. The setting sheet is a modular framework in which soils and landscape data pertinent to engineering are presented graphically. The site-specific appearance of the mapping unit data has resulted in slow acceptance. This question is addressed using the case history of a geotechnical site appraisal for embankment construction over highly organic soils. A feel for soil behavior is developed as the site investigation proceeds. In retrospect, it is seen that the graphic data, which are superimposed on the setting sheet background, pertain to the central concept of the mapping unit, and they are presented in this form in order to pass on the feel for soil behavior to others, with minimum effort and cost.

In transportation research the interest in classification of highly organic soils stems from the need to better predict performance and impacts of construction (1). For site appraisals, published maps and surveys may represent the only data base and interpretations of mapping units are provided in many areas (2). A continuing need exists for improved methods of classification and interpretation. For geotechnical applications, improvement should be such that a better feel for the soils mapped can be developed (3). The practical uses and limitations of existing classification schemes for organic soils are discussed by tracing the stages of a typical but difficult site investigation. Stemming from this is a proposal to make more effective use of this type of site experience and to assure that the information gained is made available for subsequent application.

METHOD AND MATERIALS

The method adopted is to review an actual case history of a geotechnical appraisal of a site for a low embankment structure, which took place more than 12 years ago, and then to compare the information available at the time with what is available today to see whether the same problems are recurrent.

Preliminary information for this site appraisal was obtained from the only readily available source, the Yarmouth County Soil Survey Report 9 (1960) (4) (Figures 1 and 2). Information from that source is then supplemented by field tests taken during the site investigation described (5). As the program of field testing proceeds, those responsible gradually develop a better feel for assessing the engineering parameters of the soils at the site. Although initially it might appear that these parameters are only applicable on a site-specific basis, by the time the investigation is complete it may be noted that a classification of regional significance develops.

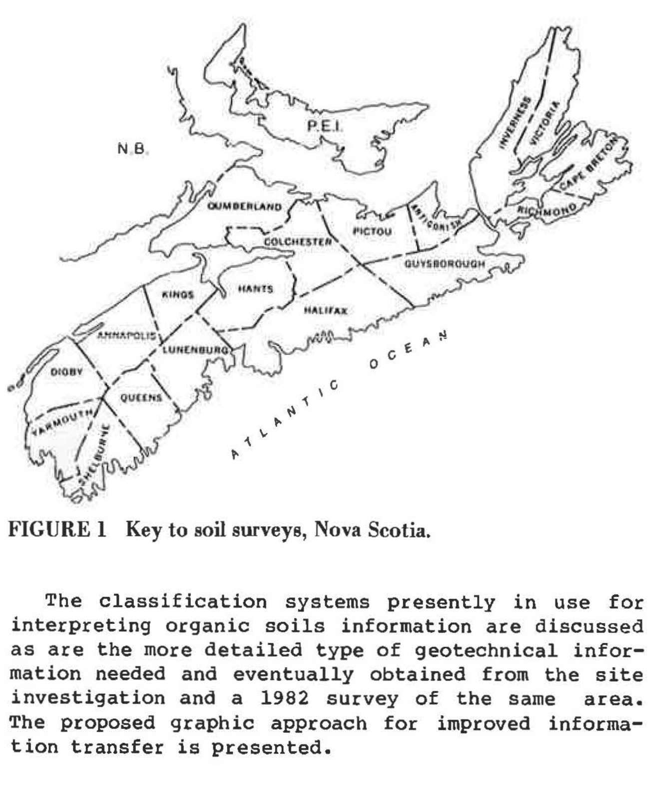


FIGURE 1 Key to soil surveys, Nova Scotia.

The classification systems presently in use for interpreting organic soils information are discussed as are the more detailed type of geotechnical information needed and eventually obtained from the site investigation and a 1982 survey of the same area. The proposed graphic approach for improved information transfer is presented.

CASE HISTORY

A site appraisal was required to assess feasibility of embankment construction. Only a minimum amount of field work was to be done. (Note that appraisals may sometimes be required on condition that no evidence of exploratory work at sites be made public because of land speculation and other considerations.)

Published Surveys

In 1972 the only readily available source of soils information was the Yarmouth County Soil Survey Report. The site was located in mapping unit SM (Figure 2), which was defined on the map legend under the heading Miscellaneous Soils as salt marsh (SM), grey silt loam over dark grey silt loam, tidal deposit.

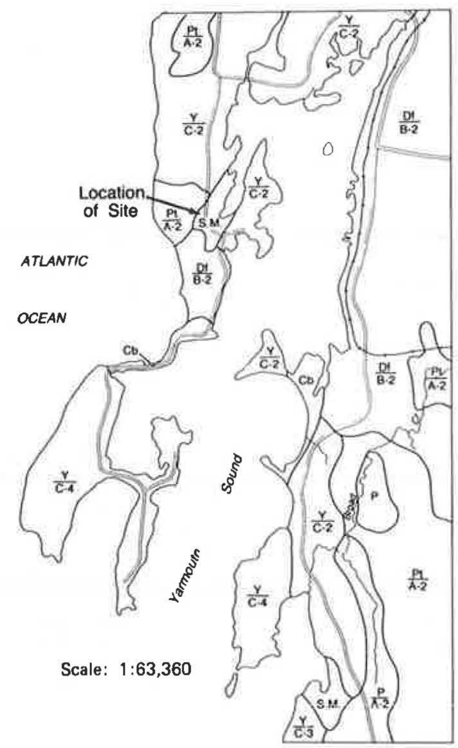


FIGURE 2 1958 soil map.

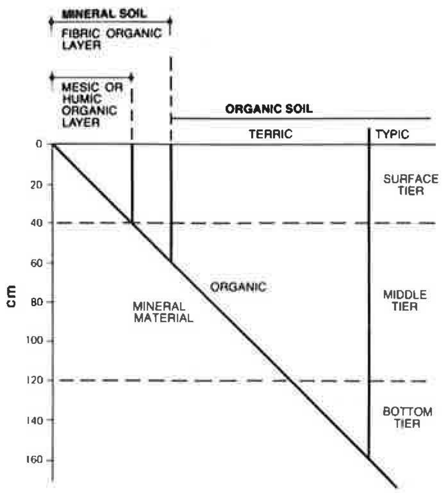
In the text of the soils report the mapping unit was further described as follows:

> The areas of salt marsh have developed as a result of repeated salt-water flooding of low lying coastal areas. Deposition of sediments at high tide has built up deep, medium textured deposits along tidal stream channels and in protected bays and inlets. The sediments deposited by tidal action in Yarmouth County are gray to olive in colour and are a uniform silt loam in texture. The surface is covered with salt-tolerant vegetation, chiefly marsh grass, sea blite and spurrey. Utilization: At present the salt marshes are of no value for agriculture. A number of areas in the county are under consideration for reclamation. If properly dyked and drained, the soils should be very fertile and productive (4,pp.35-36).

Interpretation for Site Appraisal

Consideration of different approaches to soil classification is necessary for interdisciplinary exchanges of information. The SM soil series was classified as miscellaneous and not organic soil because it did not meet the (pedological) requirements of 30 percent OM (6) (Figure 3). This requirement is contrasted with the geotechnical classification (7) (Figure 4). However, in terms of mode of deposition, salt marshes are generally equated with the "filling-in" process of peat soil deposition (8) (Figure 5). In geotechnical terms, this translates as normally consolidated soft or loose material.

The land use capability classification suggested no conflict between agriculture and proposed road construction (except for possible areas to be reclaimed as mentioned previously).



VON POST SCALE OF DECOMPOSITION

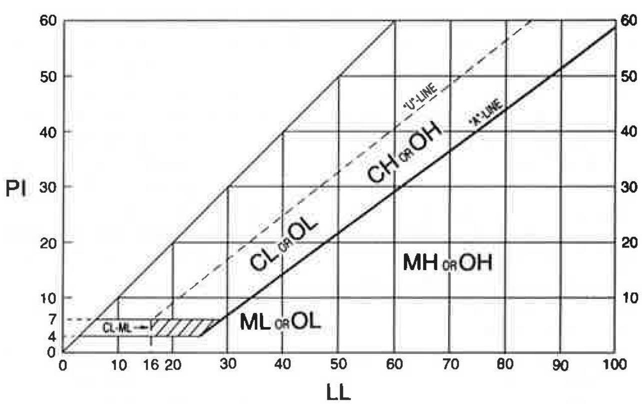
FIBRIC (Of)
1 **Undecomposed:**
2 **Almost undecomposed:**
3 **Very weakly decomposed:**
4 **Weakly decomposed:**
no peat substance escapes between the fingers

MESIC (Om)
5 **Moderately decomposed:**
6 **Strongly decomposed:**
a third of the peat escapes between the fingers

HUMIC (Oh)
7 **Strongly decomposed:**
half the peat escapes between the fingers
8 **Very strongly decomposed:**
9 **Almost completely decomposed:**
10 **Completely decomposed:**

	OM = 30%	
Organic soil	>	Mineral soil

FIGURE 3 Organic soil classification–pedology.



$$\frac{LL\,(air)}{LL\,(oven)} = .75$$

Organic soil	<	Inorganic soil

FIGURE 4 Organic soil classification–geotechnics.

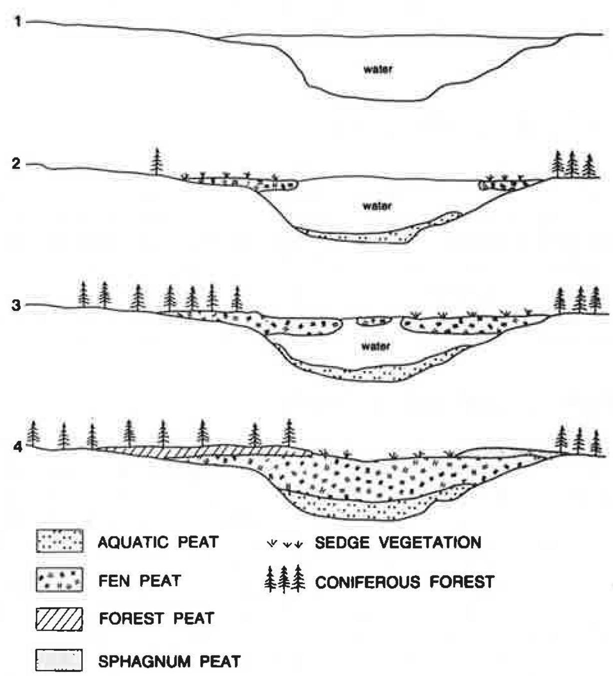


FIGURE 5 Stages of "filling-in" process.

Low embankment structures required for road construction might be feasible, but a rationale for deriving engineering strength parameters from organic soil classification symbols could not be found. In situ strength parameters normally will result from additional investigations performed at the site. The interpretation sheet (Figure 6) illustrates briefly the nature of the parameters required. Embankment height was related (by slip-circle and bearing-capacity analyses) to soil strength where the latter increases with depth as in normally consolidated soils. A brief investigation was undertaken to provide the required samples for laboratory testing and characterization of the SM soils at the site.

Site Investigation

Tidal conditions in the salt marshes indicated drilling from a fishing boat in the tidal channels would be most practical. The first significant finding was that the deposit in places exceeded 100 ft (approximately 30 m) in depth and appeared to be, as expected according to the "filling-in" process, loose and normally consolidated.

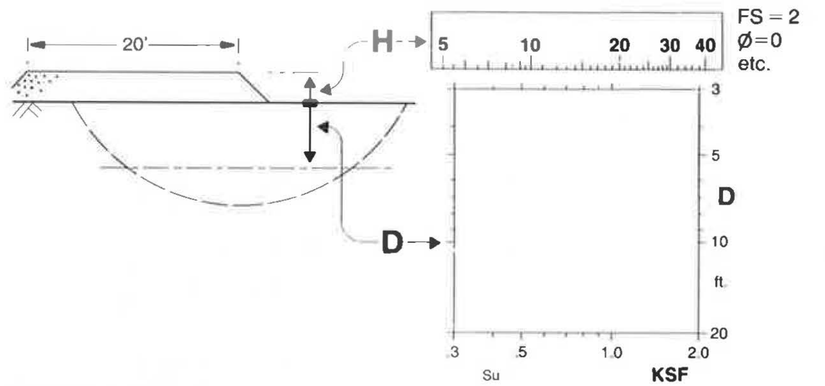


FIGURE 6 Embankment height: interpretation required.

The next significant finding, however, was the erratic nature of the test results obtained from the samples recovered. Undrained shear strength (Su) values varied from 0 to 1.0 KSF (0 to 50 Kpa), while direct shear tests indicated 0' values up to 50 degrees. Sample disturbance was suspected as the major cause, partly because of the influence on the sampling process of the vegetation and organic fibers coupled with adverse drilling and sampling conditions in the tidal zone. The first phase of the site investigation consequently did not greatly improve on the published soil survey information, and a reasonable interpretation of the shear strength characteristics of the SM soils could not be given.

Additional Laboratory Work

A laboratory testing program using the SHANSEP system of reconstituting soil samples was attempted (9). The reconstituted samples seemed to account for the disturbance of the softer soils, but the values obtained were still much less than those of the other (apparently less "disturbed") soils. The second phase of the site investigation consequently did not greatly improve on the initial information.

Additional Field Vane Testing

A program of additional in situ testing using the field vane was attempted to determine whether the SHANSEP reconstituted strength values were reasonable. The results of the in situ testing indicated only that strength values appeared to be as much as 3 to 4 times greater than those obtained by the other methods. It has also been reported elsewhere (10) that maximum torque in organic soils due to strength of fibers may occur at vane rotations exceeding 270 degrees. The additional in situ testing resulted in still less confidence in the site investigation work. This type of confusion, which often occurs when attempting to obtain practical interpretations from organic soil tests, has also been reported by others (11).

Additional in situ Testing

There is a method of soil sampling by which the undrained shear strength of cohesive soils in situ can be obtained while a good-quality undisturbed sample of the soil is recovered for laboratory verification (12). The in situ testing program was extended once more to include the square tube tests. The result was confirmation (for the SM soils in this area) that normal relationships generally existed between strength values determined by undrained compression tests on undisturbed samples and torque tests (e.g., field vane, square tube). The initial field vane test results were then suspect and eventually the high values that had previously been obtained were traced to errors due to inaccurate torque wrench calibration.

Interpretation of Extended Investigation

By checking and rechecking in situ strengths with carefully selected undisturbed samples, it was eventually substantiated that, despite the fact that the deposit was normally consolidated according to its depositional history and the "filling-in" process, the in situ strength varied significantly both horizontally and vertically throughout the deposit.

When this had been confirmed, confidence in the site investigation results was regained and sample disturbance was considered as only a minor factor. It was then possible to consider the genesis of such erratic strength characteristics of a normally consolidated loose deposit.

The soil moisture diagram (4) indicates that, for soils exposed at the surface during the months of June, July, and August, there is a potential soil moisture deficiency that could result in overconsolidated (stronger) surface soils due to moisture tension. Stronger soils could exist alongside normally consolidated deposits underlying the tidal channels. These two different surface conditions could account for the horizontal strength variations. If, however, at the same time, the land surface was also slowly subsiding, similar combinations of overconsolidated and normally consolidated soils could be expected to be repeated in depth. The dynamic action of the highly specialized vegetation would be evident maintaining the ground surface near the mid-tide level, keeping pace with subsidence, and acting as a medium for soil particle attraction and soil accumulation in the tidal zone.

With this interpretation it was possible to get a better feel for the probable engineering performance of these salt marsh soils. For low embankment construction, the marsh soils between the tidal channels might be significantly stronger than the soils underlying the channels. Consequently further investigation was warranted.

Additional Testing Between Channels

Additional testing of the tidal land between the channels confirmed higher strength values. It was

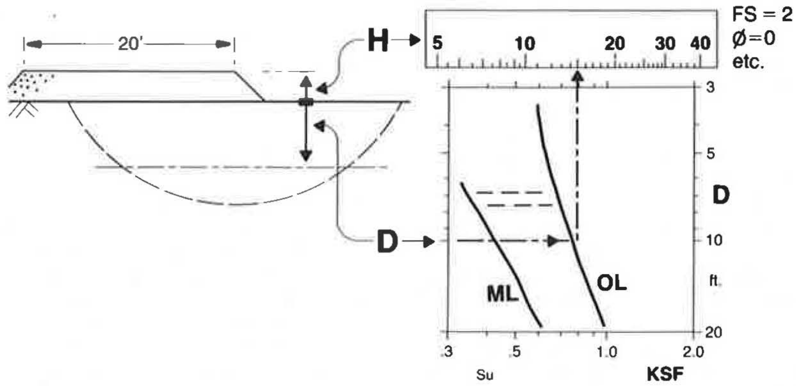


FIGURE 7 Embankment height: interpretation completed.

finally possible to complete the soil strength interpretation sheet and equate the SM map unit soils in terms of an appraisal for embankment construction (Figure 7). The low plastic organic silts (OL soils) between the channels had undrained strength values in a range that could be defined by a strength profile increasing from 1/2 KSF at 3 ft to 1 KSF at 20 ft. The dashed lines indicate that minor layers of normally consolidated low plastic silts (ML soils) could be expected at odd intervals. In terms of embankment height, this strength profile could be interpreted as indicating construction of embankments up to 15 ft (approximately 3 m) to be generally feasible.

The ML soils underlying the channels tend, however, to be mainly normally consolidated, with only minor layers of the stronger OL soils, indicated by the same dashed lines. Piled foundations for crossing structures would probably be required in these channel areas.

APPLICATION OF CASE HISTORY

Site investigations for embankments and other structures have been shown to be educational experiences for those taking part. Engineers generally have a more confident feel for the total soil environment at the site afterwards. A considerable waste of time and effort results if all of this site information is then lost or not made easily available to others. It should be noted that the hypothesis of a subsiding coastline was upheld when, toward the end of the investigation, some peat was recovered at a depth of 100 ft (approximately 30 m). Carbon dating indicated that coastline submergence has been occurring at an average rate of approximately 1 ft (0.3 m) per 100 years for the past 10,000 years.

It would be of value to know if other investigators would be likely to repeat the same lengthy process to answer a similar request today. Since the site investigation in 1972, the sources of published information for the area have increased. A section of the up-to-date Surficial Geology map (1982) is shown in Figure 8 (13). Neither the SM unit nor its surficial geology equivalent is described on the new map. This new map was compiled mostly for geochemical and mineralogical purposes as illustrated by the symbols and by the coastal section (personal communication with authors). Given time for scientific search and research, papers written on the question of submerging coastlines in Nova Scotia could be found, but the engineer involved in preliminary appraisals would have to be aware of the condition beforehand in order to find the information (14,15). Different personnel conducting a modern investigation would probably have to repeat the "educational" experience before being able to give a realistic appraisal of the same soils.

Soil surveys are based on the recurrence of similar landscape patterns within the same climatic region. Certain landscape parameters are likely to be common to all similar landscapes classified as one mapping unit. The mapping unit (e.g., SM) can be defined in terms of these general parameters. A specific map unit like the one SM unit discussed, will have these parameters plus others that are specific to it. Strength might appear to be in this category. Characteristics typical of a subsiding coastline, however, are likely to be of regional

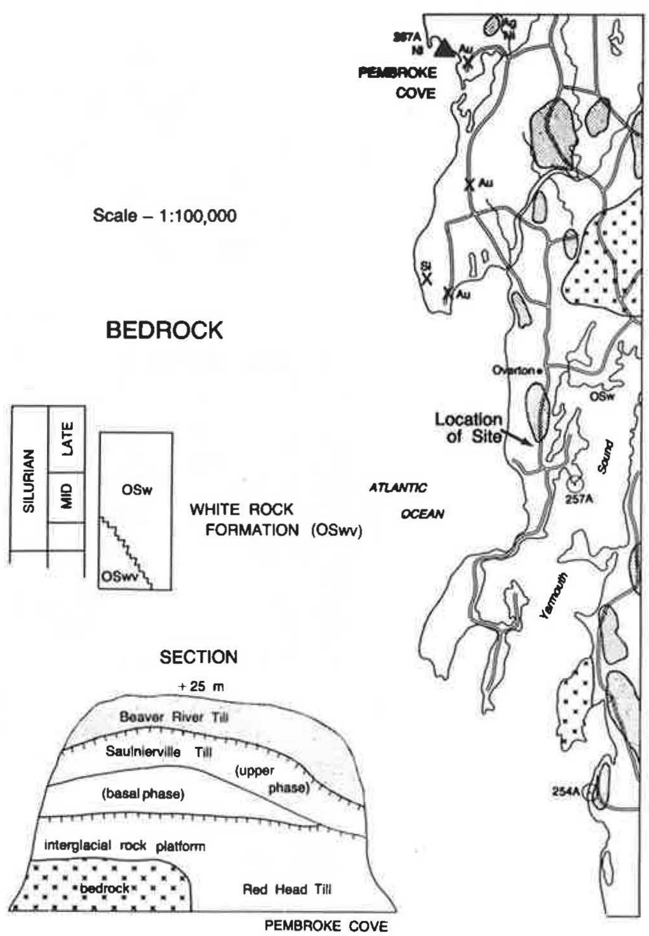


FIGURE 8 Pleistocene geology (13).

significance (14), and the nature of the strength profile given should also be characteristic, in a general way, of all landscapes denoted by the mapping unit. The interpretation sheet (Figure 7) could be considered an effective addition to the classification symbols ML and OL in describing the nature of the SM unit soils and their interpretation in terms of low embankment construction.

In the process of updating soil survey maps the addition of a graphic classification scheme defined by the setting sheet (16) has the advantage of making a considerable amount of in situ information readily available (Figures 9 and 10). In part 1 of Figure 9, for example, the story of the tidal channels and the land in between, the subsiding coastline, and so forth is told using only two or three lines and a few symbols.

Part 2 of Figure 9 is the soil-moisture diagram given in the soils report (4). Part 4 in Figure 10 shows textural characteristics and the guidelines by which determinations can be quickly and easily made (17). With information presented in this form, educational experiences do not have to be repeated indefinitely.

Also of interest are the land use interpretations. Reclamation of salt marsh soils by drainage, in the light of known regional coastal subsidence, no longer seems to be sound agricultural practice. It is evident that updating of soil surveys using simple graphics as illustrated would also improve the validity of land use interpretations.

CONCLUSIONS

Some very useful information for site appraisal purposes can be gained from existing organic soil classification systems using published surveys (in this case, a 1958 soil survey map). It often requires a considerable amount of detailed field work to improve on the information given.

As is true for most classification systems, improvements can be justified. Case history analysis can illustrate the nature of the improvements required. The graphic system proposed has had slow acceptance in Canadian soil surveys, partly because graphics give the impression that too much information of a site-specific nature is being given (18). The foregoing discussion demonstrates that the geotechnical use of published soil survey information has nothing to do with engineering design for specific sites and that site investigations cannot be circumvented by this type of generalized information. Site selection on the other hand can be made effectively on the basis of appraisals. In addition,

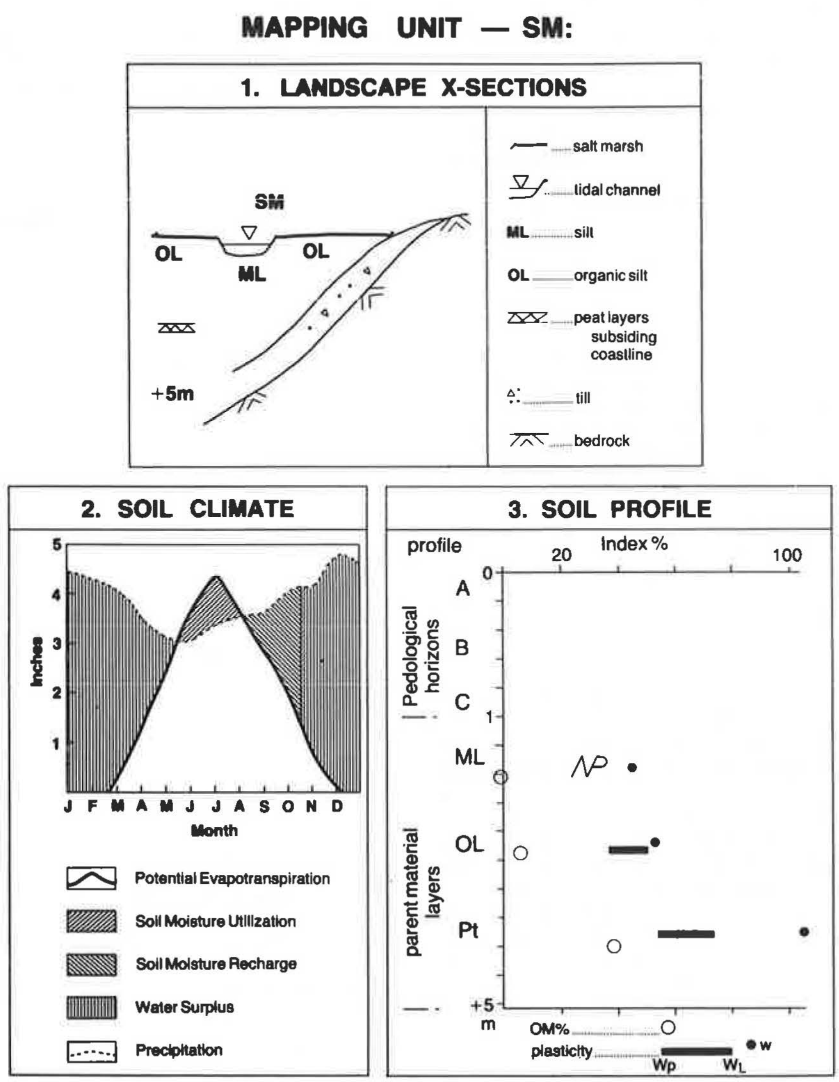


FIGURE 9 Setting sheet: parts 1, 2, and 3.

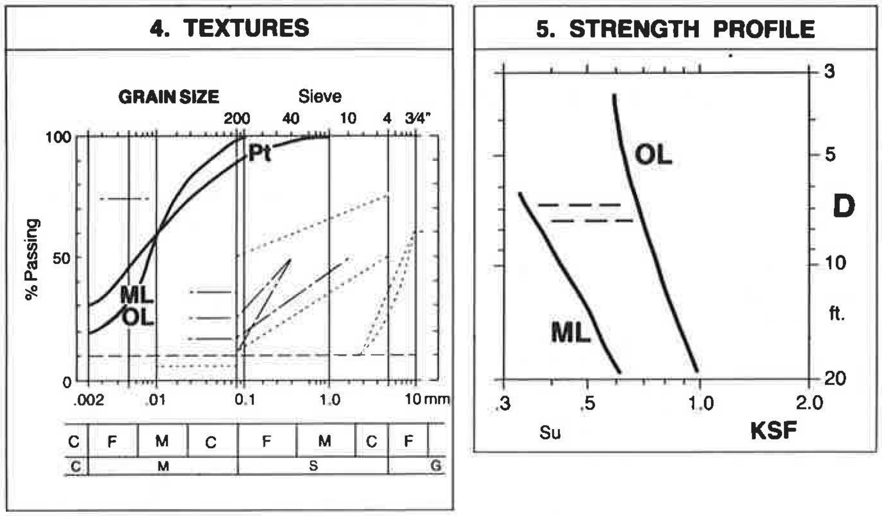


FIGURE 10 Setting sheet: parts 4 and 5.

the interpretation of site information can be greatly improved if a feel for soil behavior has already been developed from existing information.

These conclusions have been reinforced by the remarks of a reviewer who draws attention to yet another recent site investigation in this region where the same problems were found in attempting to interpret field tests in this type of organic soil.

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Pedologic Classification of Peat

RICHARD W. FENWICK and WILLIAM U. REYBOLD

ABSTRACT

Peat is classified in the order Histosols in the U.S. Department of Agriculture pedologic classification system. Histosols constitute one of the 10 orders of the hierarchical system that was developed from 1951 to 1974. (It was published as Soil Taxonomy, A Basic System of Soil Classification for Making and Interpreting Soil Survey, USDA Agriculture Handbook 436, 1975.) Histosols, unlike other soils, developed primarily from organic parent material and are saturated with water unless they are artificially drained. They are characterized by low bulk densities and are subject to large degrees of subsidence when drained; therefore, they present unique problems for use and management. The formation of Histosols, the differentiae used in classification, and the properties identified that affect engineering use are discussed.

The terms "peat" and "organic soils" have been used and are still used in a general sense to refer to soils that are characterized by a high organic materials content. The origin of the organic materials is the vegetation that has grown and is presently growing on the area. These plant remains are in various stages of decay and decomposition. Numerous criteria and subsequent systems have been used in classifying organic soils. Most were developed for a specific region or purpose. The scheme that appears in the U.S. Department of Agriculture pedologic classification system (1) is the result of the efforts of many soil scientists who were interested in the classification and management of peat soils. Peat soils are classified as Histosols. Some of the criteria used in Soil Taxonomy (1) for the various hierarchical classifications of Histosols are outlined and criteria that affect engineering use are identified.

DISTRIBUTION AND EXTENT

Histosols occur on all continents and within all of the major climatic zones of the earth--even in arid regions, as long as water is present. Histosols occur predominantly, however, in areas where precipitation usually exceeds evapotranspiration. Canada and the United States have approximately 14 percent of the known organic soils in the world (2). In the United States, organic soils are concentrated mostly in the northern states and along the Atlantic and Gulf coasts. Table 1 gives, by state, in the "Histosols Currently Mapped in State" column, the extent of Histosols identified to date in the National Cooperative Soil Survey (NCSS) program. The column "Estimated Total Histosols in State" shows the total hectares of Histosols in each state using the figures in the "Histosols Currently Mapped in State" column as a basis for the estimate.

TABLE 1 Extent of Histosols in the Coterminous United States (Tidal Marsh Not Included)

State	Histosols Currently Mapped in State (ha)	Histosols as a Percentage of All Mapped Soils in State	Estimated Total Histosols in State (ha)
Alabama	244 000	0.3	38 800
California	19 300	0.08	30 400
Connecticut	2700	0.2	26 800
Florida	545 600	6.7	898 400
Georgia	12 100	0.1	14 900
Idaho	6100	0.06	13 400
Illinois	54 800	0.6	86 000
Indiana	104 000	1.3	120 000
Iowa	9900	0.08	11 300
Louisiana	239 200	3.2	366 400
Maine	33 000	0.9	71 600
Maryland	1000	0.04	1000
Massachusetts	21 000	1.3	25 800
Michigan	193 400	2.5	373 600
Minnesota	318 400	3.0	612 800
Mississippi	29 900	0.3	36 200
Missouri	300	0.0	500
Montana	1000	0.01	3700
New Hampshire	10 400	0.6	13 800
New Jersey	17 200	0.9	17 200
New York	77 200	1.0	121 600
North Carolina	121 200	1.6	200 400
Ohio	52 000	0.6	62 800
Oregon	11 500	0.1	24 600
Pennsylvania	5300	0.05	5800
Rhode Island	10 500	3.9	10 500
South Carolina	43 200	0.6	46 400
Texas	1400	0.0	1600
Vermont	4800	0.3	7100
Virginia	20 300	0.4	40 200
Washington	27 800	0.2	34 300
Wisconsin	369 500	3.6	502 000
Total	2 393 400	0.5	3 819 900

CLASSIFICATION OF HISTOSOLS

Order

Histosols are commonly called bogs, marshes, moors, muskegs, peats, or mucks (3). Most are deep organic materials, but a few are shallow over rock or fractured rock and rubble. Histosols, unlike other soils, are derived primarily from organic parent material. To be included within the order Histosols, a soil must be composed of organic materials in more than 50 percent of the upper 80 cm of the profile unless the soil rests on solid rock or fills the interstices of fragmented rock, in which case the thickness requirements are waived. Soils composed of 75 percent or more (by volume) sphagnum moss must extend to a depth of 60 cm or more to qualify as Histosols. The organic materials that make up Histosols that are saturated with water contain at least 12 to 18 percent organic carbon by weight, depending on the clay content of the mineral fraction. With few exceptions, Histosols are constantly saturated with water unless they are artificially drained. For naturally unsaturated organic soils, the minimum organic carbon content requirement is 20 percent by weight. Because of their organic character, Histosols have low bulk densities, generally less than 0.25 g/cm^3.

Suborder

Three broadly defined states of decomposition are recognized for organic (histic) materials: little decomposed (fibric), moderately decomposed (hemic), and highly decomposed (sapric). The predominance of a particular decomposition state within a given profile gives rise to the names of the three most common suborders of Histosols: Fibrists, Hemists, and Saprists. A fourth suborder, Folists, is recognized. The suborders are defined in further detail as

1. Three suborders of Histosols that are saturated with water 6 months or more of the year or have artificial drainage:

Fibrists are composed of fibrous plant remains so little decomposed that they are not destroyed by rubbing and their botanic origin can be readily determined. Soils in this suborder tend to have the highest moisture content, commonly between 850 and 3000 percent of dry weight, and the lowest bulk density, less than 0.1 g/cm^3. Fibric horizons or layers are designated in NCSS profile descriptions as Oi.

Hemists contain organic materials that are decomposed enough that the biologic origin of two-thirds of the volume cannot be easily determined, or they contain fibrous materials that can be largely destroyed by rubbing. They are wet, with moisture content commonly between 450 and 850 percent of dry weight, and usually have a bulk density between 0.1 and 0.2 g/cm^3. Hemic horizons or layers are designated in NCSS profile descriptions as Oe.

Saprists consist primarily of highly decomposed organic materials. Commonly, few plant remains can be identified botanically. The moisture content is normally less than 450 percent of dry weight and the bulk density of the organic materials is usually greater than 0.2 g/cm^3. Sapric horizons or layers are designated in profile descriptions as Oa.

2. A suborder of Histosols that is never saturated with water for more than a few days following heavy rains:

Folists are composed of litter, leaves, twigs, and branches in various states of decomposition, ranging from nearly undecomposed to, more commonly, highly humified materials. The organic materials must contain at least 20 percent organic carbon and rest either on bedrock or on fragmental materials that have interstices filled or partly filled with organic materials.

Great Group

At the great group level of classification, Histosols are separated based primarily on the soil temperature regime. The prefixes Cryo-, Boro-, Medi-, and Tropo- designate the most common great groups of Histosols. For example, Borohemists are Hemists with a frigid soil temperature regime. In addition, the term Sphagno is added as a prefix when a Fibrist is composed of three-fourths or more (by volume) sphagnum moss (i.e., Sphagnofibrists). Hemists with significant quantities of sulfidic or sulfuric materials are designated as Sulfihemists or Sulfohemists at the great group level. Soil Taxonomy recognizes 20 great groups of Histosols. Soils have currently been classified into 16 of these 20 great groups.

Subgroup

At the subgroup level, intergrades (transitional forms to other orders, suborders, or great groups) and extragrades (forms that are not typical of the great group but do not indicate transitions to other soils) are recognized.

A typical profile of a Borofibrist would be classified in the Typic subgroup, and the Borofibrist intergrading to a Borohemist would be classed in the Hemic subgroup (Hemic Borofibrist). If the soil were shallow over bedrock (an extragrade feature), the subgroup would be Lithic (Lithic Borofibrist). Soil Taxonomy presently recognizes 124 subgroups of Histosols. Soils have been classified into 55 of these subgroups.

Family

Subgroups are subdivided into families. Each soil family name consists of the subgroup name and several additional adjectives for class names based on particle size, mineralogy, reaction, temperature, and soil depth.

Series

Soil series are subdivisions of families that provide additional homogeneity of recognizable properties and features. Because series are commonly named for geographic locations, the name seldom indicates soil properties; series descriptions, however, convey the greatest amount of soil property information and are of special value for local investigations. Information given for soil series in published soil surveys of the NCSS include depth, percent organic matter, particle size distribution of the mineral fraction, percentage rock fragments, bulk density, permeability, available water capacity, soil reaction, and estimated subsidence. There are currently 201 soil series in the Histosol order.

PHYSICAL CHARACTERISTICS WITH ENGINEERING SIGNIFICANCE

Soil properties identified in the classification scheme that have significance for engineering use include state of decomposition, bulk density, soil temperature, reaction, ferrihumic material, sulfidic material, depth to bedrock, depth and thickness, particle size and mineralogy, and presence of marl or diatonaceous earth in the mineral layers.

Organic deposits (Histosols) subside after drainage (4). The potential rate of subsidence of Histosols after drainage is critical in making decisions regarding their use. The state of decomposition, bulk density, soil temperature, thickness of material, and percentage clay in the mineral fraction are soil properties used to estimate potential subsidence. Reaction and presence of sulfidic materials affect corrosion of steel or concrete conduits and are used to estimate such corrositivity, and the presence of ferrihumic material affects excavation.

Trafficability estimates are made considering thickness of organic material, bulk density, depth to bedrock or ferrihumic material, kind and depth of mineral layers present, and state of decomposition of the organic material. All of the soil properties used in the classification scheme are considered in the design, construction, and maintenance of drainage systems.

SUMMARY

Organic soils (Histosols) are classified based on quantitative criteria that can be determined in the field by visual observations and by simple field tests. The order is identified by content of organic material; the suborder by the degree of decomposition of the organic materials; the great group by soil temperature; subgroups by intergrades to other great groups of organic soils; and the family by particle size, mineralogy, reaction, temperature, and soil depth.

The criteria used to classify peat soils identify soil properties that have significance for engineering purposes. Nomenclature used in the classification scheme is connotative and enables recognition of the properties.

The NCSS classifies and maps soils using Soil Taxonomy. Soil survey maps at scales of 1:15,840, 1:20,000, or 1:24,000 are available for about 1,660 counties in the United States. The maps and descriptions of peat soils can help engineers plan and conduct soil investigations for engineering purposes.

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Compression of Peat Under Embankment Loading

H. ALLEN GRUEN and C. W. LOVELL

ABSTRACT

Peat and organic soil are commonly avoided as sites for highway construction. There are situations when this is not possible or economical, and the peat must be dealt with. If the organic accumulation is relatively shallow, excavation and replacement are feasible. However, for deeper deposits other alternatives, including the preloading technique discussed here, need to be considered. Preloading both strengthens the peat, so that it can safely carry the intended load, and achieves long-term compression in an accelerated period. Prediction of the settlement of peat under both the service load and the preload is important. Rheological parameters can be derived from field testing to allow use of a method that predicts settlements and controls duration of preload. A case study involving a highway compares results predicted by the method with actual measurements.

Building highways over peat and other highly organic deposits has been avoided by engineers whenever possible. It has been customary to go around peat lands when planning a highway, and this is still the preferred solution. However, there are times when passing the highway alignment over the deposit may be an effective alternative.

When these deposits are relatively shallow (less than 5 m), excavation and replacement by granular materials are commonly used. However, when the deposits are deeper or of a large lateral extent, special foundation treatment is usually required.

One such treatment is preloading. As a result of expansion into areas with poor foundation soils, preloading techniques through surcharging have been developed with some success as a means of in situ improvement of soil properties. Preloading accelerates settlement and strengthens the deposit so that an embankment can be supported without failure or excessive settlement.

A major drawback to preloading peat has been the inability to predict the deformation characteristics of the organic deposit under loading. This lack of knowledge becomes apparent when attempting to determine the surcharge magnitude and duration required to accelerate settlement. The time rate and magnitude of settlement to be expected with peat are at best uncertain. Methods currently used to predict settlement give poor results when applied to large strain materials with significant secondary compression effects (i.e., peats). Thus, after a preload has been applied to peat, the rate and magnitude of settlement are often uncertain, and consequently the required duration of the surcharge period is unknown.

A technique to accurately control the duration of the surcharging period so that construction may be completed in the minimum amount of time is presented.

GIBSON AND LO MODEL

Gibson and Lo (1) proposed a rheological model that applies to large strain soils that exhibit secondary compression. This theory assumes that the structural viscosity of the soil is linear. For large values of time, the deformation behavior, $\varepsilon(t)$, may be written as

$$\varepsilon(t) = \Delta\sigma \left[a + b\left(1 - e^{-(\lambda/b)t}\right)\right] \qquad t > t_a \tag{1}$$

where a, b, and λ are empirical parameters that can be determined from deformation response data; $\Delta\sigma$ is the increase in vertical stress; and t_a is the time after which the stress has become fully effective. This model has been shown to closely model both laboratory and field behavior of peat (2,3).

Dhowian (4) derived the following method for determining the rheological parameters to be used in the Gibson and Lo model. If Equation 1 is differentiated with respect to time, the rate of strain obtained is

$$\partial\varepsilon(t)/\partial t = \Delta\sigma\lambda e^{-(\lambda/b)t} \tag{2}$$

Taking the logarithm of both sides in Equation 2, the following linear relation is obtained:

$$\log_{10}[\partial\varepsilon(t)/\partial t] = \log_{10}\Delta\sigma\lambda - 0.434(\lambda/b)t \tag{3}$$

which in a simplified form is the following straight line:

$$Y = C + D(t) \tag{4}$$

where

$Y = \log_{10}[\partial\varepsilon(t)/\partial t]$ = log of strain rate,
$C = \log_{10} \Delta\sigma\lambda$ = line intercept, and
$D = -0.434(\lambda/b)$ = slope of the line.

The parameters are determined by plotting the logarithm of strain rate against time from compression results for a particular soil. A straight line is then drawn through these points. The slope (D) and the intercept (C) of this line yield the values of b and λ. The primary compressibility parameter (a) is found by substituting the known quantities into Equation 5:

$$a = [\varepsilon(t)/\Delta\sigma] - b + be^{-(\lambda/b)t} \tag{5}$$

APPLICATION

The Gibson and Lo model may be used to extrapolate field settlement curves and predict field settlement under other than the applied stress level. This will be illustrated later by an example. The actual surcharged embankment is constructed in the field and settlement data are recorded. After a short period (normally less than 3 months), the load has become fully effective and sufficient data are available to determine the rheological parameters. This method has been computerized (3) so that data can be entered as they are collected, refining the rheological parameters to a greater accuracy as settlement progresses. When these parameters have been determined for a given deposit, the settlement behavior can be extrapolated to any time.

In a similar manner, using Equation 1, the stress change term ($\Delta\sigma$) can be chosen to predict the settlement behavior under other loads. Varying the stress change term in Equation 1, while using one set of rheological parameters (a, b, and λ), assumes that these parameters are constant with strain rate and that strain is a linear function of stress at any given time. This is not completely correct for peat. However, Gruen and Lovell (3) have shown that, for the stress change levels normally involved in the preloading of peat, the violation of these assumptions causes small and acceptable errors.

ILLUSTRATION

This method will be illustrated by a case history. A highway was to be built over an extensive deposit of peat and highly organic materials at Walt Disney World, Florida. Preliminary investigations and rough settlement calculations resulted in the selection of a surcharged embankment section to be placed on the deposit. Settlement plates were placed and the embankment was constructed. Settlement was monitored at regular intervals. In a short time excess pore pressures had dissipated (end of primary consolidation), and the rheological parameters for the model could be determined. Table 1 shows the observed settlement data and the calculated logarithm of strain rate. The movement of settlement plate 89 during the first 3 months and the embankment load are shown in Figure 1. Note that primary consolidation appears to end at approximately 40 days.

TABLE 1 Observed Settlement Data

Time (Days)	Settlement (cm)	Strain	Change In Time (Days)	Log Change In Strain / Change In Time	Midtime (Days)
0	0	0			
			5	-2.62	2.5
5	3.66	0.012			
			5	-2.13	7.5
10	14.9	0.049			
			5	-2.36	12.5
15	21.6	0.071			
			5	-2.32	17.5
20	29.0	0.095			
			10	-2.49	25.0
30	38.7	0.127			
			10	-2.74	35.0
40	44.2	0.145			
			10	-3.00	45.0
50	47.2	0.155			
			10	-3.30	55.0
60	48.8	0.160			
			10	-3.10	65.0
70	48.8	0.168			
			10	-3.30	75.0
80	52.7	0.173			

To determine the rheological parameters, the logarithm of strain rate was plotted against time as shown in Figure 2. Only the data for times after primary consolidation had occurred were used in determining the best fit line shown in Figure 2. In this example, the plotted points before 40 days are disregarded because the deformation behavior during this period is controlled by hydrodynamic effects. After the applied load has become fully effective (excess pore water pressure equals zero), the logarithm of strain rate plots approximately as a linear function of time. The rheological parameters b and λ are calculated from the slope and intercept of the line as shown in Figure 2. The rheological parameter a is determined from Equation 5.

Using these parameters in Equation 1, the settle-

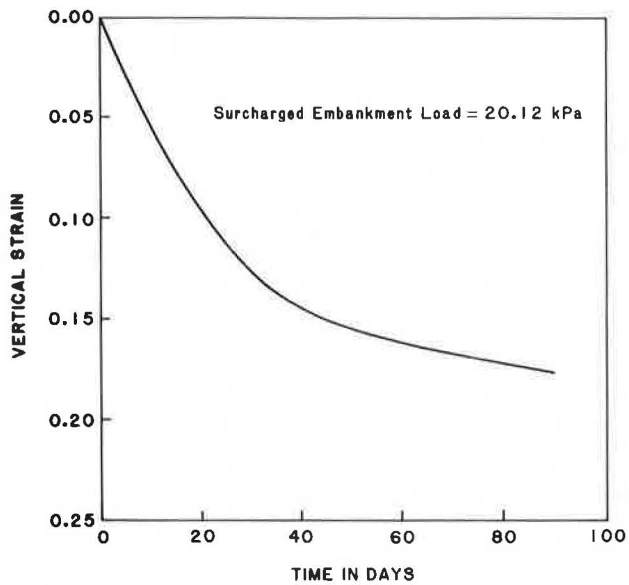

FIGURE 1 Settlement data from plate 89.

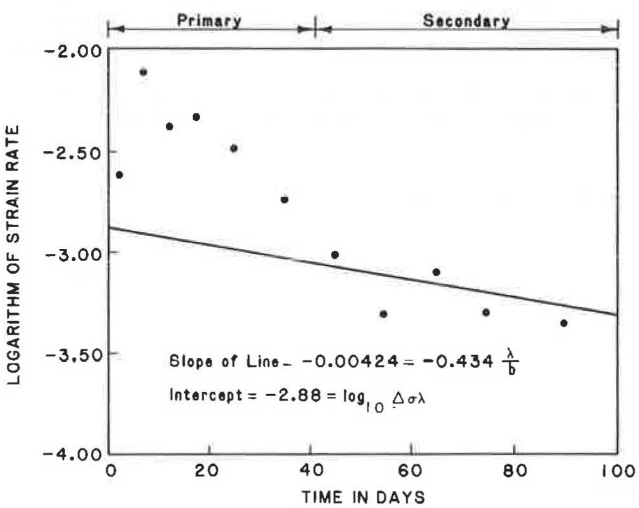

FIGURE 2 Logarithm of strain rate versus time.

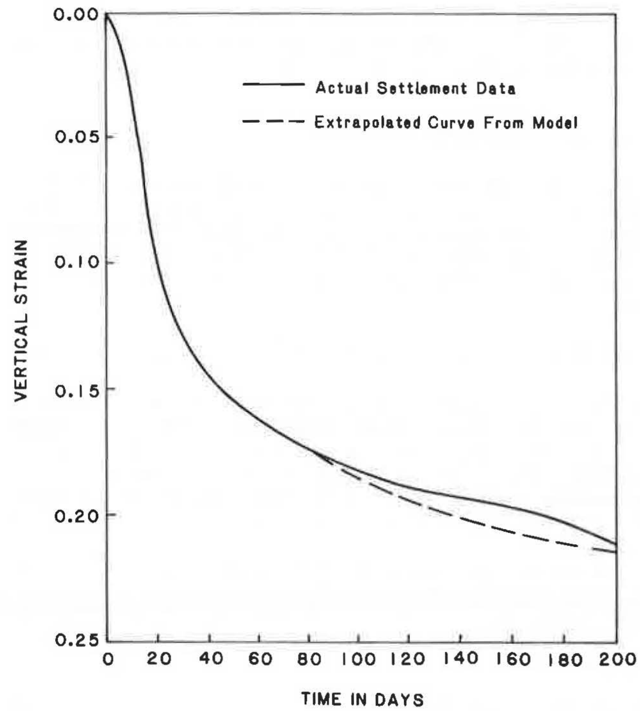

FIGURE 3 Actual and estimated settlement behavior.

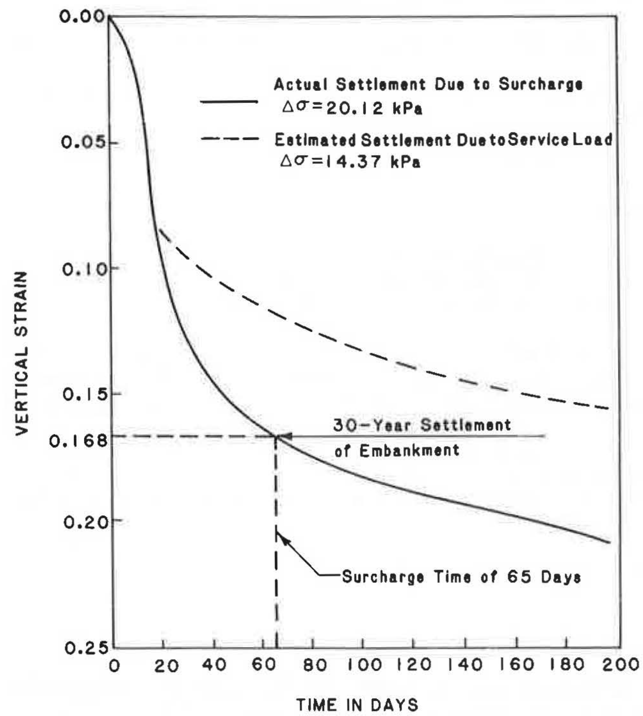

FIGURE 4 Actual field settlements and anticipated settlement due to service load only.

ment record can be extrapolated as shown in Figure 3. This extrapolation agrees very well with actual settlements that subsequently occurred. At this point it is desired to estimate the settlement behavior of the deposit under only the service load (embankment with no surcharge). This is accomplished by using the calculated rheological parameters in Equation 1 along with a stress change ($\Delta\sigma$) corresponding to the anticipated service load. The estimated settlement behavior of the deposit under the service load is shown with the actual settlement curve due to the surcharge load in Figure 4. In this case it is assumed that the surcharge is intended to eliminate the settlements expected under the service load over a period of 30 years. As shown in Figure 4, the estimated strain in 30 years is 0.168.

The surcharge should remain in place until the desired settlements have occurred (roughly 70 days). During this time, settlement data should continue to be collected and used to refine the parameters used in the model. This approach can be considered somewhat of an observational method, in that the model becomes more and more accurate as settlement continues, providing more data for determination of the parameters. Determination of the rheological parameters and prediction of settlement have been simplified by use of the computer program given by Gruen and Lovell (3).

After sufficient settlements have occurred, the surcharge is removed and construction of the highway is completed. It should be noted that the settlement data used in this illustrative example were obtained from an embankment loading of peat at Walt Disney

World, Florida (see the paper by Swantko et al. in this Record). The actual design incorporated preloading; however, the rheological model approach was not used in the project. The use of the rheological model for other cases is reported by Gruen and Lovell (3).

CONCLUSION

If peat is to be used directly as a foundation material, its properties must be improved by preloading. Using preliminary settlement estimates, the magnitude and duration of preloading can be predicted and a surcharge applied. After the primary strain portion under the surcharge load has occurred, the Gibson and Lo theory can be applied to determine the rheological parameters used for the model. According to Landva (5) the field settlements under embankment loading have normally entered the secondary strain portion within 3 to 4 months. Using these rheological parameters, the surcharge settlement curve can be extrapolated and the settlement curve for the final design load can be estimated. These two curves can be compared so that the duration of preloading is sufficient to accelerate the anticipated settlements caused by the service load. Using the Gibson and Lo theory in this manner will give more accurate control over preloading than do other methods currently used. If the deposit is fairly uniform, a test section may be built to determine the rheological parameters for the deposit. These parameters can be used with the model for designing subsequent embankment sections and preloading programs.

ACKNOWLEDGMENTS

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Experience with Development of Peat Deposits at Walt Disney World, Florida

THOMAS D. SWANTKO, STEPHEN W. BERRY, and WILLIAM P. RINGO

ABSTRACT

Considerable experience was gained with the development of sites underlain with peat and highly organic soils at Walt Disney World in central Florida. The most straightforward approach was to totally excavate the organics and replace them with sand fill. The excavation techniques and equipment used depended on the total depth of organics, the size of the area to be excavated, and the seasonal groundwater conditions. Various surcharging techniques were used to stabilize the peat before construction. Several specialized approaches were used successfully: (a) A portion of a major lagoon was developed by compressing a thick organic profile by surcharging. Vertical compressions of up to 15 ft (3.0 to 4.6 m) were achieved, thus avoiding the need for significant excavation and disposal. (b) A controlled surcharge program was used to develop over 3,000 linear ft (0.9 km) of a major four-lane access road over an organic profile extending to depths of 10 to 14 ft (3.0 to 12.2 m). Surcharging was found to stabilize the peat by removing primary consolidation and reducing the rate of secondary compression. (c) A section of an elevated monorail system in a deep organic area was developed in a phased sequence of surcharging, partial removal of surcharge, driving piles, and additional surcharge removal. (d) A finite-element program was used to assess the general vertical and horizontal displacement pattern within a sand fill extending partly over highly compressible organic soils. The purpose of this study was to evaluate the distance from the edge of the soft ground area where a structure could be safely supported in the sand fill.

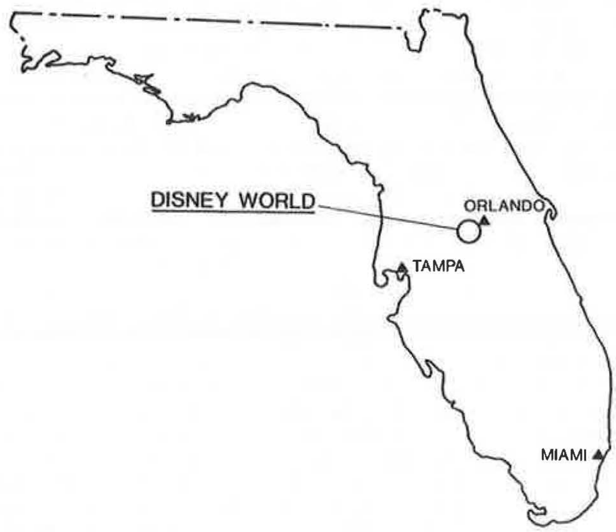

FIGURE 1 Location map.

Walt Disney World is located in central Florida about 15 miles (24 km) south of Orlando (Figure 1). The Disney World property contains approximately 42 square miles (109 km²) of land. Before development, much of the property was covered with dense vegetation and there were large areas that were swampy and underlain by peat and highly organic soils.

The initial development of the property began with the construction of the Magic Kingdom in the late 1960s. A site was selected where the near-surface soils consisted primarily of sands, which minimized grading and allowed shallow foundations to be used for support. However, it was still necessary to develop some of the peat areas.

The peat areas encountered during initial construction generally contained less than 10 ft (3.0 m) of organics. For the most part, the organics were excavated and replaced with sand fill. However, this straightforward approach to the development of peat sites was not always possible during the recent development of the 500-acre (202-ha) Experimental Prototype Community of Tomorrow (EPCOT) site, for which field investigations began in 1978.

The majority of the EPCOT site is underlain from the surface by about 40 ft (12.2 m) of medium dense to dense sand. However, the site is bordered by heavily vegetated, low-lying ground and there was a large low-lying area covering approximately 50 acres (20 ha) located near the center of the site. In these low areas, peat and organic soils were typically found to extend from the surface to depths of 5 to 20 ft (1.5 to 6.1 m); in some areas the depth of organics exceeded 60 ft (18.3 m). The nature of the development planned and the location and the extent of the peat deposits required considering alternatives to the total excavation and replacement techniques used previously. The selection of an alternative depended on time and economic constraints as well as engineering judgment, including an assessment of the uncertainties and their potential impact on scheduling, initial costs, and projected maintenance costs.

INVESTIGATION OF PEAT AREAS

The identification of peat areas was initially made on the basis of a visual inspection of the site. Peat was generally found in lower ground areas, containing mature bay, black gum, cypress, and some pine trees, with a thick undergrowth of low brush around the perimeter. There was also often evidence of standing water. Because Florida terrain is typically flat, the term "lower ground" meant that the ground surface was only 1 to 4 ft (0.3 to 1.2 m) lower in elevation than the surrounding "higher ground."

A preliminary estimate of the lateral extent of the peat was made on the basis of soil maps from the Soil Conservation Service, U.S. Department of Agriculture, and a review of aerial photographs and topographic maps. The photographs, including infrared photographs, were examined for vegetation patterns or other surficial features that might define changes in subsurface conditions. Topographic maps identified the limits of the lower ground areas. Sometimes, the lower ground areas or the vegetation patterns were nearly circular in plan, suggesting a relationship with past sinkhole development (collapses of cavities within the underlying limestone formation).

Additional preliminary information on the peat areas was developed using a hand-probing program. The probe used consisted of 1/2-in. (1.3-cm) diameter steel rods that were carried in 5- to 10-ft (1.5- to 3.0-m) sections. The probes were typically performed throughout the area of interest on a grid pattern using 50- to 100-ft (15.2- to 30.5-m) spacing. The rods could be pushed by hand through organic profiles approximately 20 ft (6.1 m) deep without encountering much resistance. By using wrenches or a 40-lb (18-kg) drop hammer, the rods could be advanced to greater depths [in one instance in excess of 70 ft (21.3 m)]. It was not uncommon to find peats interbedded with sand layers; therefore, the probing did not stop at the first sign of an increase in resistance. It is recognized that conclusions based on such probing are fairly subjective. However, if probing is done carefully, useful information can be developed to help in planning a more detailed drilling and sampling program. For example, the results of the probing were used to develop a rough contour map of organic thickness.

Conventional truck-mounted drilling equipment could not be used in the peat areas. Therefore, the boring and sampling program required the use of a track- or skid-mounted rig. The skid rig was moved through the area by clearing a path through the vegetation and using the remaining trees to winch the rig from location to location. Drilling was done either by rotary-wash or chop-and-wash techniques. Water for the drilling operation was obtained from the near-surface water table.

The rigs used for this study were capable of drilling to depths of 230 ft (61 to 70 m). Most of the borings were advanced into the underlying limestone formation to evaluate its integrity and check for the presence of large cavities. The borings were advanced at least 15 to 20 ft (4.6 to 6.1 m) below the last organic layer and into competent material.

Samples of the peat and organic soils were obtained using a variety of sampling techniques, including a split-barrel sampler with Shelby tube extension, and a piston sampler. It was extremely difficult to obtain good samples within the upper 5 ft (1.5 m) of the profile due to the presence of abundant coarse roots and the extensive root systems in this zone.

NATURE OF PEAT

The peat sites primarily contained mature tree growth that would be classified as coverage type A, after Radforth (1). The ground surface was underlain by a "rootmat" of living and partially decomposed root systems. The rootmat layer was typically 1 to 5 ft (0.3 to 1.5 m) thick and contained very little, if any, soil material. These interwoven root systems provided a natural reinforcement that could support light vehicles.

Below the rootmat, the peat ranged from "stringy" and "spongy," to highly decomposed vegetation matter, to material that contained equal amounts of organics and mineral soils. The organics were usually in the form of very fine root fibers with a diameter of less than 1/32 to 1/64 in. (0.8 to 0.4 mm). Below a depth of about 5 ft (1.5 m), the fiber structure was usually highly decomposed. Some samples showed no evidence of fibers. These lower peats would generally fall into categories 1 to 5, amorphous granular, of the Radforth classification of peat structure (1), with most samples in categories 4 and 5.

The typical ranges in the physical properties of the peat are

Property	Typical Range of Test Results
Wet density	60 to 80 pcf (1.0 to 1.3 g/cm³)
Moisture content	140 to 1200 percent (based on dry density)
Dry density	4 to 32 pcf (0.06 to 0.5 g/cm³)
Organic content	50 to 95 percent (based on dry weight)
Ash content	5 to 50 percent (based on dry weight)
Shear strength	$<$ 100 to 400 psf ($<$ 4.8 to 18.6 kPa)

It was not unusual for the organics to be interbedded with loose sands. In addition, soft to medium-stiff organic layers were sometimes encountered below stiffer layers. It is believed that the stiffer layers had consolidated through desiccation during fluctuations in the groundwater level.

Laboratory consolidation tests showed that the peat was highly compressible, with samples consolidating from one-fourth to one-half their original thickness under moderate loads. The test samples were first consolidated under a nominal load of 100 to 200 psf (4.8 to 9.6 kPa), which simulated placement of a thin working platform of fill over the peat and also tended to remove some of the disturbance caused during sampling. The samples were then loaded to anticipated field levels.

Typical summary plots of primary consolidation (expressed as sample strain) versus the log of the applied load, and the coefficient of secondary compression (C_S) versus the log of the applied load are present in Figures 2 and 3, respectively. Note that the results in Figure 2 show an increase in sample stiffness when samples from progressively deeper strata are compared.

EXCAVATION TECHNIQUES

General

The most positive approach to the development of a peat deposit is complete removal of the organics by excavation and replacement with sand fill. The techniques used to excavate the peat depend primarily on the total depth of the organics, the size of the area to be excavated, and the seasonal groundwater conditions.

Front-End Loaders

During the dry season, the peat and organic soils could be excavated to depths of about 6 ft (1.8 m) using front-end loaders working from the underlying natural sands. The use of front-end loaders allowed good control for cleaning of the bottom of the excavation to remove all of the organics. However, if

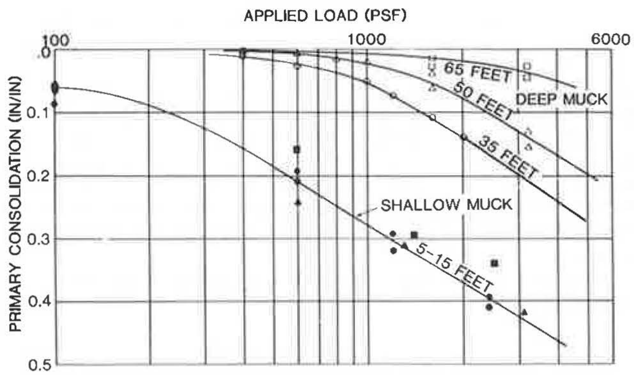

FIGURE 2 Primary consolidation versus log of applied load.

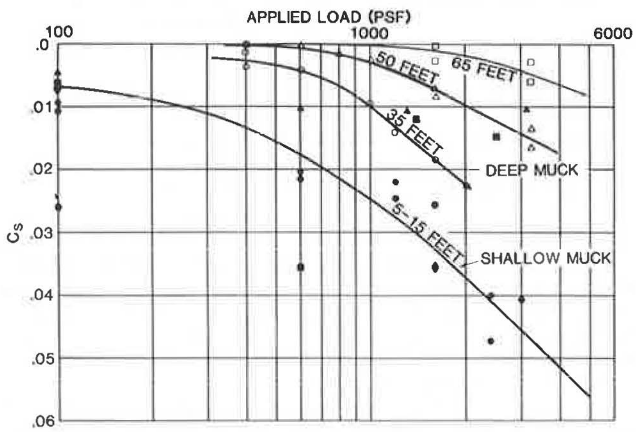

FIGURE 3 Coefficient of secondary compression versus log of applied load.

the excavation became wet, the use of front-end loaders generally had to be discontinued because of poor trafficability.

Backhoes

Backhoes were used for excavation of organic materials that were known to be wet. The water flowing into the excavation was diverted from the face of the cut to one or more sumps. These sumps were simply holes dug below the level of the excavation. Pumps were placed in the sumps to discharge the water away from the work area. The intakes on the pumps were protected by strainers. The strainers were often clogged by fiberous and woodier portions of the peat, and it was necessary to continuously switch sumps or pumps to maintain the dewatering operation.

In the usual backhoe operation, the machine cut a strip as far out as it could reach, working parallel to the edge of the peat deposit. All the organic material was removed down to clean sand. The excavated material was loaded directly into scrapers or trucks for disposal. As the backhoe moved, the excavated zone was filled with sand and compacted.

If during the excavation operations the inflow of groundwater became large enough that the bottom of the excavation could not be visually inspected, the operations had to be stopped until the excavation could be pumped dry and the bottom observed. During this stage of the operation, the contractor often used a backhoe to bail portions of the excavation to speed the dewatering operation. Ideally, if the sand under the organics could be dewatered by deep wells or properly placed well points, a more efficient demucking operation could be conducted.

Draglines

For larger areas and deeper excavations, a dragline was used. However, the use of draglines caused a sloppy operation, because of the overflow of material from the bucket and the highly disturbed state of the excavated organics. The water at the face of the excavation could be controlled with difficulty by means of sumps and temporary ditches. Removing the water allowed the bottom of the excavation to be cleaned by bulldozers.

As did the backhoes, the dragline excavated a strip as far out as it could reach and each strip was then filled with compacted sand. Because of the method of dragline operation, the leading edge of the backfilled sand adjacent to where the equipment sat was partly removed and wasted when excavating the next organic strip.

The excavated muck was wet and sloppy and was often placed behind the dragline and allowed to dry before being hauled to the disposal area. The working of draglines, the cleaning of the bottom of the excavation, the fill placement, the dewatering, the shifting of pumps, and the positioning of the scrapers or trucks being filled required constant direction. However, when working well, the choreography of a dragline demucking operation was truly impressive.

Displacement

Another means of removing peat was by displacement. In this method of removal, a bulldozer was used to push the mud and muck out, and the area behind the bulldozer was filled with compacted sand. Soft organic materials on the order of 5 ft (1.5 m) deep could be removed by this method; if done carefully, deeper displacement was also possible.

It was necessary to keep the working pad low, no higher than 3 or 4 ft (0.9 to 1.2 m) above the bottom of the excavation, and to push down and out with the bulldozer blade. A sand berm carried in front of the blade formed a temporary dike for the water and displaced organic material. If the sand berm was allowed to get too high, it flowed over and covered the muck being removed. It was also necessary in this operation to be careful that there were no reentrant angles to trap the mud wave being pushed in front. Occasionally, it was necessary to excavate the contaminated sand berm or mud wave ahead of the bulldozer to prevent a buildup of this material from flowing back into the excavation or onto the newly placed fill.

Dredge

If the excavation area is large enough, the use of a small hydraulic dredge may be justified. In this operation, the area was cleared and grubbed using light draglines with 1-yd^3 (0.8-m^3) buckets operating on mats. The draglines were equipped with an L-shaped clearing bucket, which was used to windrow the cut trees and grub the surface material and then work the spoil pile to the edge of the peat deposit, where it was loaded into trucks or scrapers by front-end loaders and taken to the disposal area.

The dredge was equipped with a revolving cutter head that could slice through any roots and smaller stumps. When the cutter head became clogged, it was cleaned by reversing. The bottom of the hydraulically dredged area was extremely irregular. The dredge operator could tell when he was no longer pumping organics and was excavating sand by the increase in voltage required and the smoother pumping action. Because the dredging occurred under the water surface, it had to be controlled by soundings and sampled by hand probes. Redredging of missed areas was often required. When dredging an enclosed area, it was necessary to have a makeup water supply to keep the dredge floating and to produce a slurry of organic material that could then be pumped to the selected disposal area. Disposal can be a problem, particularly because of state and federal environmental regulations covering handling of such materials and disposal sites.

SURCHARGING TECHNIQUES

Creation of a Major Lagoon

A central feature of the EPCOT project is a large man-made lagoon that covers approximately 35 acres (14 ha). A large portion of the lagoon falls within the limits of a major peat area. The location of the organic area with respect to the lagoon is shown in Figure 4. To aid in interpreting the depth of the organic materials present throughout this area, a computer program entitled Surface Approximation and Contour Mapping (SACM) was used. This program can approximate any surface through irregularly distributed points using a weighted least-square fit. Information from borings and hand probes was used to develop the isopachs of organic thickness shown in Figure 4. As shown, a large portion of the area planned for the lagoon was underlain by more than 30 ft (9.1 m) of organics.

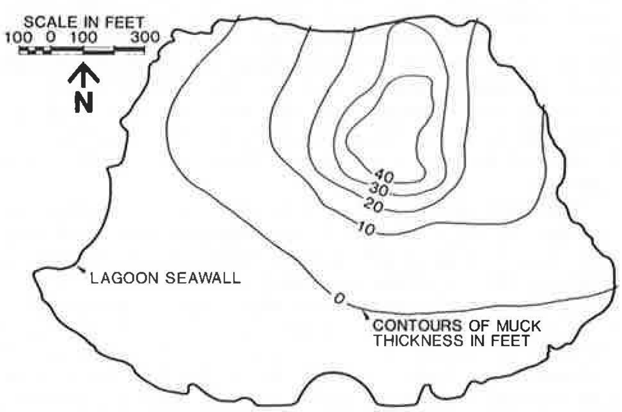

FIGURE 4 Lagoon site plan.

Because of the depth of the organics, it was not considered economical or practical to excavate completely. However, due to the high compressibility of the material, an alternative involving complete excavation of some areas and surcharging of the remainder was used to develop the planned grades within the lagoon. The concept was to contain the perimeter of the organics and then uniformly load the center to compress the surface of the peat below the planned lagoon bottom.

The original surface of the organics was at elevation +94 ft. The planned bottom of the lagoon was at elevation +84 ft. A 3-ft (0.9-m) layer of sand was planned over the bottom of the lagoon to act as a filter over the organics left in place. On this basis, the surface of the organics was required to be compressed from elevation +94 ft down to about elevation +81 ft, or about 13 ft (4.0 m) vertically (Figure 5).

In planning the construction sequence, it was necessary to consider the stability of the soft organics before they were consolidated by the surcharge. The surcharging had to be done carefully without creating unbalanced loadings on the surface because of the risk of generating mud waves. The construction approach used is shown in Figure 6. The first stage consisted of excavating a "collar" around the perimeter of the area and filling it with sand to help maintain stability by confining the peat. The organics were removed in the collar area either by dragline or by dredging to a point where the bottom of the muck was about 15 ft (4.6 m) deep. The face of the muck was cut at a slope of approximately 3 horizontal to 1 vertical. The collar excavation was then backfilled with sand. This backfilling had to be done carefully because the weight of the sand fill against the 3-to-1 muck slope could cause a heaving-type failure. However, localized heaving did not represent a serious problem because of the high-surcharge fill subsequently placed in the area.

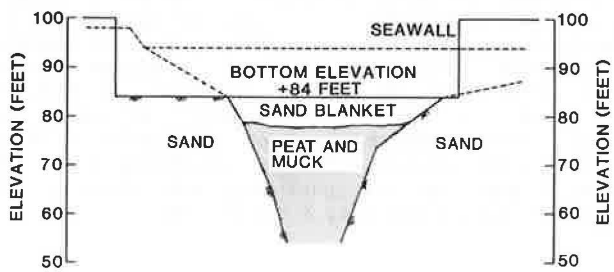

FIGURE 5 Proposed final grading.

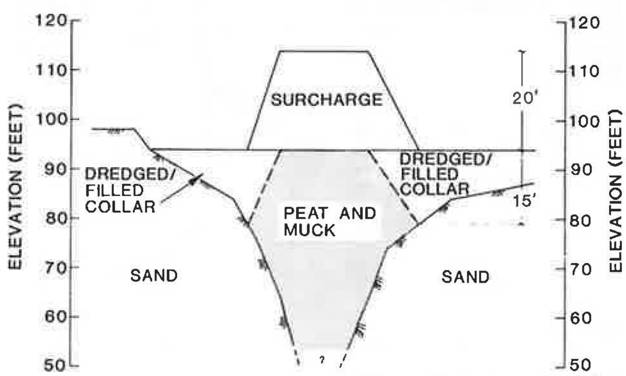

FIGURE 6 Construction phase.

The majority of the muck removal in the collar area was accomplished using a 16-in. (40.6-cm) cutter-head dredge. The dredged muck was pumped through pipes to disposal in excavations that had been developed as borrow pits about 1 mile (1.6 km) south of the project. The borrow pits were developed as a series of cells that also served as sedimentation basins for the dredge effluent. Makeup water for the dredging operations was repumped from the borrow pits, from a point at the opposite end from the discharge line, thus forming a closed-loop system.

Before surcharging the remaining peat area, large trees and brush were removed, but the surface rootmat was left undisturbed to serve as a reinforcing layer to support the surcharge soils and minimize the tendency for minor instability problems. The initial thickness of sand fill was placed hydraulically by dredge, using sands coming from the other portions of the lagoon excavation. This initial fill was placed uniformly over the peat surface in layers no more than 2 to 3 ft (0.6 to 0.9 m) thick. Small dozers were then used to place a dike over the collar area to contain the next layer of dredged sand.

After the initial layer of fill was placed over the entire area, settlement plates were installed to monitor the vertical compressions. The settlement plates consisted of 2-ft (0.6-m) square steel base plates with 1-in. (2.5-cm) diameter riser pipes. Additional fill was then placed over the area using very flat side slopes with no layer more than 2 ft (0.6 m) thick. After completing the third lift, conventional earthmoving equipment was used to place the remainder of the surcharge. The settlement pipes were raised by adding additional pipe sections as the surface of the fill increased. The settlement pipes were surveyed daily during placement of the surcharge to monitor potential instability problems. As the surcharge reached its design height, the monitoring was performed less frequently. The total surcharge placed over the peat was approximately 20 to 24 ft (6.1 to 7.3 m) thick.

Typical settlement curves observed during the surcharging are shown in Figure 7. The surcharge was placed over a 2-month period, and the full surcharge remained in place for a period of 4 to 8 months. During the surcharge program, the surface of the peat was observed to compress vertically as much as 15 ft (4.6 m). After the required compression occurred, the surcharge was removed, leaving about a 3-ft (0.9-m) blanket of sand overlying the compressed peat. As the surcharge was removed, the observed rebound was only a few inches.

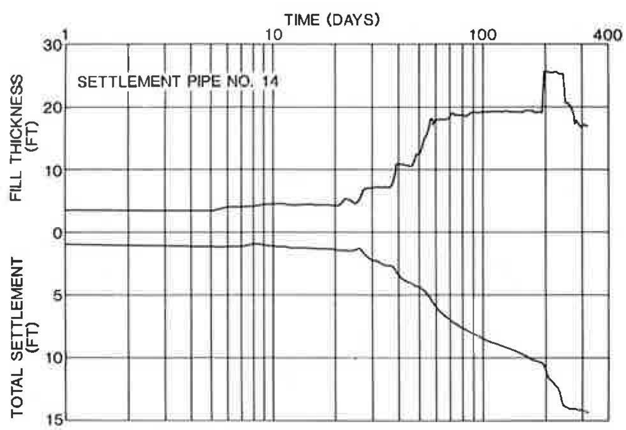

FIGURE 7 Typical settlement plots, lagoon area.

During the course of the program, portions of the surcharge were approved for removal when sufficient compression was achieved. To speed consolidation of other areas, some portions of the released surcharge material were added to other areas to create additional load. Ultimately, the surcharge material was used as fill in other portions of the project.

Overall, the surcharging program was a success. With the exception of two localized areas, the entire organic surface was compressed to the desired elevation. These localized areas required excavation of a few feet of organics and replacement with sand fill. An important advantage of the surcharging was that it avoided significant problems with excavation and disposal.

A similar concurrent program was used to create a smaller pond located to the north. The pond bottom was developed using a surcharge of 10 to 12 ft (3.0 to 3.7 m) to accomplish vertical compressions of 7 to 8 ft (2.1 to 2.4 m).

Roadway Embankment Surcharging

More than 3,000 linear feet (0.9 km) of a major four-lane access road and a large parking area were planned in areas underlain to depths of 10 to 40 ft (3.0 to 12.2 m) with peat and highly organic soils. The construction alternatives were (a) total removal of the organic material and replacement with compacted fill, (b) partial excavation and replacement, or (c) building directly on the organics following a controlled surcharging program.

The disadvantages of both total and partial removal were the cost and technical difficulties of excavation, the quantity of compacted backfill required, and the problems associated with disposal of large quantities of organic spoil. Particular problems were associated with partial muck removal, because, when the upper rootmat layer was removed, the remaining peats were particularly sensitive and the controlled placement of fill soils directly on this disturbed surface would have been extremely difficult.

The use of surcharging, or preloading, techniques to stabilize peat deposits is not unique, and surcharging has been done with varying measures of success (2-5). If done carefully, surcharging can remove the expected large primary consolidations and reduce the magnitude of long-term, secondary compressions.

The depth beyond which surcharging becomes economically attractive compared with full muck removal and replacement depends on the size of the area involved, disposal problems, and the construction time available. From a practical construction standpoint, at muck excavation depths in excess of 5 to 10 ft (1.5 to 3.0 m), the types of required excavation equipment change as do the difficulties associated with dewatering. On the basis of all factors involved for this project, it was believed that 5 ft (1.5 m) was a practical limit for complete muck removal.

The placement of an embankment, or strip surcharge, is more difficult to control than the areal surcharges discussed previously, because of the absence of confinement along the edges of the embankment and the potential for lateral squeezing of the organics at depth. Also, the rate of loading of subsequent lifts must be controlled so that the increase in shear strength that comes with consolidation takes place before the next load increment is placed.

The embankment surcharge program started with the complete excavation of organic soils to the 5-ft (1.5-m) depth contour and replacement with sand fill. In the area to be surcharged, the entire right-of-way was cleared; however, no grubbing was performed and the rootmat was left intact. Trees and brush were cut to a height about 1 ft (0.3 m) above the rootmat and removed. The initial layer of fill placed over the rootmat was pushed out using light dozers, such as a D-3 or D-5 Caterpillar tractor with wide tracks, to avoid creating a mudwave. This initial fill layer, designed to be not more than 3 ft (0.9 m) thick, was placed across the entire right-of-way. The front of the layer was kept at least 40 ft (12.2 m) ahead of the following layer. No grade changes of more than 18 in. (45.7 cm) in height were allowed to occur on this first lift. The second layer of fill was about 2 ft (0.6 m) thick and was spread over the first layer with equipment not heavier than a D-6 Caterpillar tractor. Scrapers were allowed to operate on the second layer provided excessive rutting was not observed.

The rate of loading of the peat was carefully controlled. Rough estimates of the initial critical embankment height that could be supported by the peat were based on experience (2,4,5). In general, after placement of the second lift of fill, a limiting loading rate of 1 to 1.5 ft (0.3 to 0.5 m) of fill per week was used. This rate was derived during the surcharge program using settlement plate data and observations of general stability. In addition, two test sections were developed where inclinometer casings and pore pressure transducers were installed to help monitor stability. As the surcharge approached its final height, no fill was added unless the daily settlement rate was less than 1/4 in. (0.6 cm).

The final grades planned for the roadway were typically 5 to 9 ft (1.5 to 2.7 m) above the original grades in the area. In general, the height of the surcharge was extended to an elevation 5 to 7 ft (1.5 to 2.1 m) above the planned final grade for the roadway surface. When placing the surcharge fill, side slopes of 3 horizontal to 1 vertical or flatter were maintained. Final roadway slopes were trimmed somewhat flatter, between 5:1 and 6:1, to allow for maintenance mowing of the grass.

Preliminary estimates of settlements and settlement time rates anticipated under the surcharge loads were based on laboratory consolidation tests using the following equations after MacFarlane (2):

$$S_o\text{field} = (H_o \text{ field} \times S_o \text{ lab})/(H_o \text{ lab}) \qquad (1)$$

$$t_o\text{field} = [(H_o\text{field})^i \times t_o\text{lab}]/(H_o \text{ lab})^i \qquad (2)$$

where

S_o = settlement due to primary consolidation,
H_o = initial thickness,
t_o = time required to complete S_o, and
i = exponential parameter, generally 1.5 to 2.0.

Secondary compression was estimated by the use of the following equation (2):

$$S_s = C_s H \log_{10} t/t_o \qquad (3)$$

where

S_s = magnitude of secondary compression;
C_s = coefficient of secondary compression; slope of settlement-log time plot divided by the thickness of the sample at the completion of primary consolidation;
t = field time considered; and
H = thickness of peat at t_o.

Prediction of the magnitude of field settlements and settlement time rates for peat deposits is difficult at best. Some minimum level of field investigation, together with laboratory testing, is required to make the initial judgment as to whether surcharging is practical from a scheduling standpoint and is likely to result in an acceptable condition from a long-term performance standpoint. However, due to the uniqueness and horizontal and vertical variability of each peat deposit, no amount of laboratory testing can substitute for empirically developed data. Therefore, a settlement monitoring program should be included as part of any surcharge program in order to evaluate the positive effects of the surcharging, to provide data for judging when the surcharge can be removed, and to predict long-term performance. These field data will also allow empirical adjustment of the predictions made from laboratory tests and thus improve available analytic approaches.

For this project, settlement pipes were installed at 100-ft (30.5-m) spacing along each major section of the surcharge. The pipes were installed after placing the initial layer of fill. The contractor was responsible for protecting the pipes and there was a specific penalty for any settlement pipe destroyed and not replaced promptly.

A subsurface profile along a surcharged road section is shown schematically in Figure 8, and a typical settlement curve observed during the surcharge period is shown in Figure 9. In general, the areas surcharged were underlain by 10 to 14 ft (3.0 to 3.7 m) of organic material. Overall, the organic profile was compressed about 5 to 6 ft (1.5 to 1.8 m) under a total fill and surcharge load of about 1,500 psf (72 kPa). This settlement represents about 35 to 45 percent of the initial thickness of the peat layer. The observed settlement is generally in good agreement with the prediction obtained using Figure 2. Approximately 2 ft (0.60 m) of the settlement took place during placement of the initial 3-ft (0.9-m) layer of fill and is believed to have resulted primarily from compression of the open-gapped rootmat layer. Primary settlements occurred during a 40- to 90-day period.

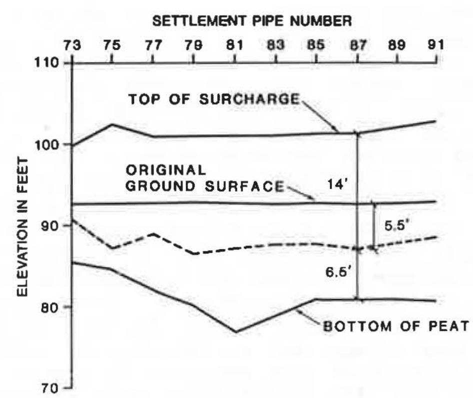

FIGURE 8 Schematic profile along surcharged road section.

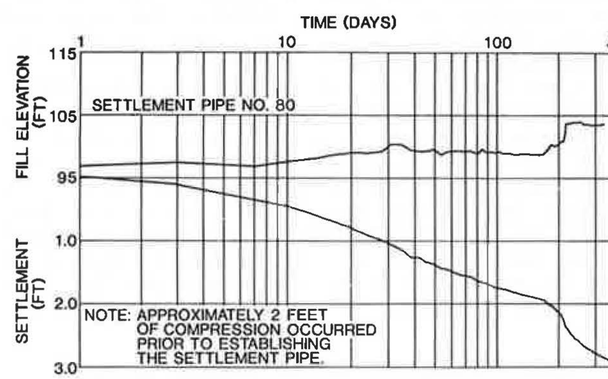

FIGURE 9 Typical settlement plot along surcharged roadway.

Approximately 4 ft (1.2 m) of surcharge were removed from the area to establish final planned grades after primary consolidation had been completed; this represented approximately 30 percent of the total fill plus surcharge load that had been applied. In the 1.5 to 2 years following removal of the various surcharges, the secondary compression has typically been measured as approximately 0.5 to 1.0 in. (1.3 to 2.5 cm). To date, the measured values of secondary compression are about one-half the values obtained using Equation 3 and Figure 3, which indicates another positive effect of the surcharging. This reduction in the rate of secondary compression after surcharging was also observed by Samson and LaRochelle (4) for a similar level of loading.

The laboratory work of Dhowian and Edil (6) suggests that loaded organic deposits may undergo a tertiary settlement phase that occurs after the secondary phase and at an accelerated rate compared with the settlement rate during the secondary phase. This tertiary phase may begin several years after completion of primary consolidation. However, the tertiary settlement phase apparently has not been observed in the field and may be a laboratory phenomenon. Periodic settlement readings are still being taken to record the performance of the surcharged areas on this project. These data will provide information on the long-term settlement rates, including the possibility of tertiary settlements.

OTHER CONSTRUCTION ON PRECOMPRESSED MUCK

A section of an elevated monorail system crosses an area underlain with a thick organic profile. Within this area, the monorail is supported on long piles driven to refusal into the underlying limestone formation. Before installing the foundations, a 10- to 12-ft (3.0- to 3.7-m) high surcharge was placed over the organics to compress the surface of the peat below the elevation of the bottom of a pond planned in this area. After the required vertical compression was achieved, approximately one-half of the surcharge soils were removed along the monorail alignment. The remaining fill was left over the compressed peat to serve as a working platform to support the construction and pile-driving equipment.

Following driving of the piles and installation of the pier columns and the beamway, all but a 2-ft (0.6-m) blanket of sand was removed from the area. The pond was then filled with water. Because the organics had been surcharged to a fairly high level, the 2-ft (0.6-m) blanket of sand left in place did not represent a load sufficient to cause any further movements within the peat profile. Therefore, the final condition was similar to having the pile-supported monorail piers installed across an undisturbed peat deposit.

During installation of the piers, inclinometer casings were installed to monitor any horizontal displacements occurring near the piers. In addition, settlement plates installed as part of the surcharging program were used to continue to monitor any vertical settlements in the area. After the pier columns were installed, survey monitoring continued on both the vertical and the horizontal position of each of the piers. This monitoring verified that there were no movements of any of the piers.

FINITE-ELEMENT ANALYSIS OF DISPLACEMENT PATTERNS IN A SAND FILL ON SOFT GROUND

As mentioned earlier, it is difficult to predict with much confidence the field performance of peat on the basis of laboratory test results. The best predictions are based on empirical correlations derived from actual field observations. However, sophisticated numerical techniques can still provide useful information and can be used to advantage.

A finite-element analysis was undertaken to evaluate the displacement pattern developed within a sand fill resting partly on an organic profile. The purpose of the analysis was to evaluate the distance from the edge of the soft ground where a structure could be safely supported on the sand fill. The generalized subsurface profile considered in the study area is shown in Figure 10. The sand fill was modeled using an incremental nonlinear elastic finite-element program. The details of the analysis have been described by Roth et al. (7). The lower boundary of the model was taken as the contact between the sand fill and the organics. In the study, the lower boundary was incrementally dropped, simulating settlement within the soft soils.

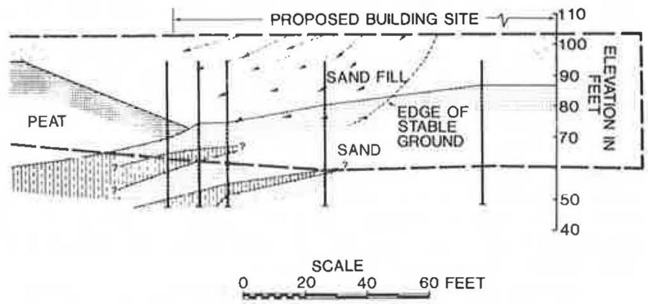

FIGURE 10 Vector displacement pattern in sand fill.

The displacement pattern predicted within the sand fill is also shown in Figure 10. During actual construction, the fill was instrumented with settlement plates and inclinometer casings to check the analytical results. Although the actual construction sequence and configuration of the sand fill were somewhat different than originally planned and assumed for analysis, many aspects of the study were still applicable for comparison of field behavior with the analytical results. In general, the predicted relationships of vertical settlements to horizontal spreading and the limits of stable zones within the sand fill were found to be in good agreement with actual behavior.

On the basis of analyses of several different profiles, it was generally concluded that the location and shape of the edge boundary of the stable zone within the sand fill model depended on

- The shape of the settlement profile resulting from compression of the organic layer;
- The magnitude of the settlements;
- The depth to the compressive layer underlying the fill; and
- The fact that as the depth of the compression layer increases, the average inclination of the boundary of the stable zones appears to become steeper.

SUMMARY AND CONCLUSIONS

The development of peat deposits poses significant problems for the geotechnical engineer as well as the developer. The physical and engineering characteristics of the organic materials are quite dissimilar from those of typical mineral-type soils for which theoretical relationships and modeling techniques have been developed to predict performance. As a result, much of the ability to predict the performance of peat under short- and long-term loading comes from direct observation and empirically derived relationships.

In developing peat deposits, the first choice is usually to completely remove and replace the peat materials with compacted sand fill. Such excavations must be well planned and controlled. However, excavation and replacement are not always economically practical, and there are additional problems with disposal of the excavated materials.

A variety of lessons was learned during development of peat deposits at Walt Disney World in Florida. The types of construction equipment and procedures used to excavate or partially excavate such materials depended on the total depth of the organics, the size of the area to be excavated, and the seasonal groundwater conditions. Surcharging techniques were used to complete primary consolidation before construction and to reduce the rate of secondary compression. This approach is not unique and has been used successfully by others (2-5). For this project, it was found that gross settlement and time rate predictions, made using Equations 1 and 2 after MacFarlane (2), provided good estimates for planning purposes. However, the primary judgment about when to remove the surcharge was based on monitoring actual field settlements during the surcharge program.

To a large extent, working with peat is a "learn by doing" proposition. When concerned with developing peat sites, both the geotechnical engineer and the developer should recognize the need to maintain flexibility in design schedules to cover the uncertainties in the predictions. As experience on a site is gained, the ability to predict peat behavior improves. Peat materials do lend themselves to engineering logic, but the logic uses direct practical experience to empirically adjust the predictions made from laboratory tests and analytical theories.

ACKNOWLEDGMENT

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Construction and Performance of Pavement over Muskegs

J. HODE KEYSER and M. A. LAFORTE

ABSTRACT

The long-term performance of embankment and pavement over muskegs along the James Bay access road is described. The road was built in 1973 in a discontinuous permafrost region. The embankments were designed by a preloading method based on the Von Post classification. Twenty-four embankment sections over deep muskegs were selected from a total of 54 for detailed performance studies. They have been characterized by their distribution along the road; their geometry; the type and quality of their underlying soil; the Von Post classification; the shear strength and depth of their peat materials; and the height, type, and typical cross section of their embankment. The performance of pavement built over deep muskegs has been defined in terms of long-term settlement, change in pavement roughness, structural behavior, and deterioration of the surface. On the whole, 8 years after construction, the performance of pavement built over deep muskeg is satisfactory: Long-term settlement generally varies between 25 and 50 percent of the thickness of the peat deposit except where ice is present under the peat; the riding quality, which still has a good rating, is 50 percent rougher over peat deposits than elsewhere; there is a loose relationship between the height of embankment and the maximum deflection; the dynamic modulus of peat under the embankment is on the order of 50 mPa under Dynaflect loading conditions; and longitudinal cracking is two to four times greater over peat deposits than in other areas.

The techniques of design and construction of highways over muskeg are well documented in the literature (1,2) and generally fall within four categories:

- Complete or partial removal of peat material underneath the roadway,
- Stabilization of material by draining and preloading,
- Building of pile-supported roadway through peat deposits, and
- Building the embankment using bridging techniques and delaying pavement construction to allow postconstruction settlements.

In northern regions, where muskeg occurrence is great and where it is not always possible to relocate a road to avoid muskegs, complete or partial excavation of peat material is recommended (3) only if the depth of peat is less than a meter, drainage can be improved, and the underlying material has a good bearing capacity. In all other cases, it is preferable to build the embankment over the muskeg after limited removal of vegetation, using bridging techniques where necessary, and to accept some postconstruction settlements.

Although the technique of building embankments over muskeg is important and will eventually gain more importance with the rapid development of the north, very little is known about its reliability; indeed, a literature survey revealed that very few papers dealing with the performance of roads over muskegs (3,4) place great emphasis on the relation between the properties of the peat and the long-term performance of the embankment.

An attempt will be made to describe the long-term performance of embankment and pavement over muskegs along the James Bay penetration road. The design of the road was based on simple relations established by Lefebvre et al. (5) between the Von Post classification of peat (6) and its short-term physical and mechanical behavior.

THE ROAD

The principal access road to the James Bay hydroelectric complex in northern Quebec was built during the summers of 1972 and 1973 in an almost virgin territory. The access road is 620 km long and runs from Matagami (North 49.8 degrees) to LG-2 (North 53.8 degrees). It was paved between 1974 and 1976, and since then its general performance had been periodically monitored according to a pavement maintenance management system (7).

The territory where the road is situated is a discontinuous permafrost area (8) that can be divided in two geomorphologically different zones as shown in Figure 1:

1. From kilometers 0 to 275, approximately, the road crosscuts the James Bay lowlands (9); principal soil types are classified A-5 to A-7, mostly silty clay or clayey silt, sometimes varved, with low shear strength and consolidation. The southern part of the region is heavily wooded with spruce and birch.
2. From kilometers 275 to 600, the undulated terrain exhibits rock outcrops aligned in the east-west direction and is crosscut by low valleys and moraine plateaus (9); principal soil types are classified A-1 to A-4; in the low areas and depressions the soils are classified A-6 or A-7. Between kilometers 400 and 530 and kilometers 550 and 600, the sandy soils found almost everywhere along the road have jack pine vegetation and almost no organic soil. In the low area, spruce swamps or string bogs are common along slow-draining creeks.

DISTRIBUTION OF MUSKEG

Mean distribution of principal soil types is given in Table 1 for each geomorphologic region. Although the whole area is considered a territory with a moderate (north) to a high (south) probability of

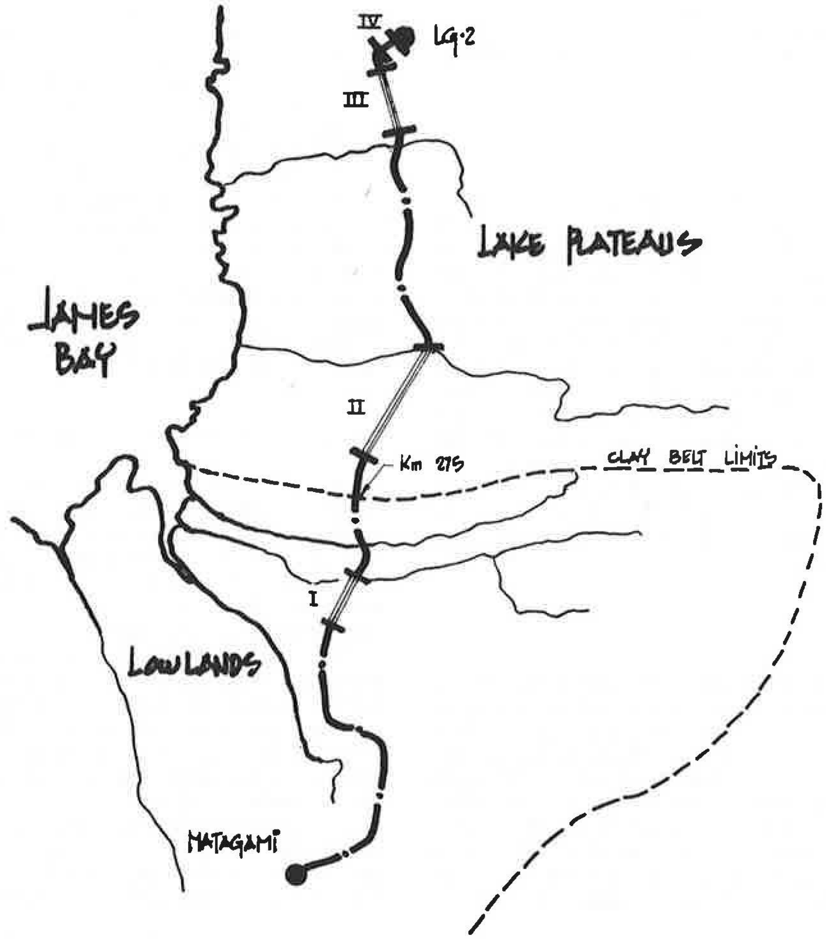

FIGURE 1 High muskeg occurrence zones (I, II, III, & IV), James Bay access road.

TABLE 1 Mean Soil Distribution Along the James Bay Access Road[a]

Soil types	Mean %	Range %
Southern section (0-275 km)		
Till	8	4-15
Sand or gravel	5	0- 7
Silt	22	5-40
Clay	55	50-60
Peat:		
- all deposits	-	6-37
- deposits of more than one meter deep	9	2- 8
Rock	5	0-10
Northern section (275-620 km)		
Till	20	8-32
Sand or gravel	37	20-57
Silt	13	6-10
Clay	7	0-20
Peat:		
- all deposits	-	10-30
- deposits of more than one meter deep	9	2-12
Rock	16	3-33

[a]According to the original geotechnical investigation before construction; the actual road has less than 5% of peat section having more than one meter on depth: 3% have more than 150 m long.

occurrence of muskeg areas (2), most of the muskeg areas crossed by the road are less than a meter in depth and are concentrated in four well-defined zones.

Zone 1

Zone 1 is between kilometers 150 and 230; it is situated south of the Broadback River in a clay belt that is typical of the James Bay lowlands. Peat sections are typically spruce swamps on clay deposit and are generally heavily wooded and concentrated in low areas. Thin peat deposits are very frequent; however, only 10 to 15 deposits are more than 1.5 m in depth. The Von Post classification index (6) of peat material varies between H-2 and H-10, often with an index from H-2 to H-4 at the surface (0-0.5 m) and from H-6 to H-10 toward the bottom of the deposit. Some deposits show only one category of peat material. The H-10 material is therefore rare; most of the peat material is H-7 or H-8 at most. The underlying material is silty clay and often very hard at the contact zone.

Zone 2

Zone 2 is between kilometers 315 and 400; it is located in the southern part of the plateau region where thin sheets of muskeg are found in the depressions between low-lying outcrops. Peat sections are typically spruce swamps or areas of low vegetation over clayey silt deposits. Peat deposits are often more than 300 m long with 1.2 to 2.5 m of peat material. The Von Post classification indices vary from H-4 to H-8 and the underlying clayey silt is often very soft with low consolidation or shear strength.

Zone 3

Zone 3 is between kilometers 535 and 570; it is located in an area similar to zone 2 but at the edge of a vast sandy deposit with frequent sandy or silty sand underlying material. Palsas [discontinuous permafrost characteristic features (10)] are common in this zone that is in the dryest part of a muskeg area, under low peat cover (1-1.5 m maximum), at the edge of the muskeg area, and in close contact with inorganic highly freezing silty material. In this zone, at least five palsa fields were crosscut by the road (11).

Peat deposits in the zone vary between 100 and 400 m in length and are 0.5 to 2.5 m in depth. The Von Post classification index is typically H-7 to H-10, and in at least one area (km 549.7) there is a deposit of organic brown clay of high shearing resistance (Su > 90 kPa). Underlying soil is typically a low resistance clayey silt, dense sandy silt, or silty sand.

Zone 4

Zone 4 is located between kilometers 604 and 618 where the road runs along the La Grande River valley. Muskegs are common in a low-lying area between outcrops where spruce or bush vegetation is scarce. More than 25 percent of the total road length in this section is muskeg.

Other Sections

Other peat sections along the road are typically bog areas adjacent to lakes or along creeks meandering between cliffs. Most of those areas are unvegetated short stretches with deep (2-5.5 m) low bearing capacity materials overlying rock or sandy-silty deposits. The thickest rockfill was used over many of these sections.

Careful selection of the road alignment before construction and relocation of alignment during construction avoided many important muskegs that can be seen from the roadway. In fact, the road crosses 54 deep (> 1.0 m deep and > 150 m long) muskegs that together represent 16 km of road, 60 percent of which are located north of kilometer 275. Figure 2 shows typical longitudinal profiles of the muskegs. A summary of the general characteristics of each muskeg is given in Table 2.

DESIGN OF EMBANKMENT OVER MUSKEG

To minimize long-term settlement the design of embankment over muskeg was based on the concept of consolidation before paving. The embankment and the subbase of the pavement were built during the summer of 1973, and the road was opened to heavy traffic and maintained during the 3 years before paving in the summers of 1975 and 1976.

On deep muskegs, the duration and magnitude of loadings were based on the Von Post classification index (Table 3) using a relationship established between the Von Post scale (VPS) and the following properties: specific gravity, natural void ratio, virgin compression index, rebound index, and deformation modulus (5).

As indicated by the authors (5), this approach, based on simple tests and classifications, has proven to be very practical when a good estimate of the order of magnitude is desired and when

- There are a good number of muskegs coupled with a difficult field access;
- The properties of peat vary considerably from one location to another within and between muskegs;
- It is almost impossible and very costly to perform a comprehensive analysis of each muskeg; and
- Based on field settlement measurements, the calculated settlement tends to overestimate (VPS > 5) or underestimate (VPS < 5) the true settlement.

PERFORMANCE OF PAVEMENT BUILT OVER MUSKEGS

The entire 620 km of road have been subjected to periodic evaluation since the road was paved:

- Dynaflect deflection measurements were made once in the summer of 1978 at a rate of four measurements per kilometer;
- Road roughness was evaluated eight times during the summer, and twice in the winter to measure the effects of winter; and
- Degradation of the surface was identified and quantified in terms of extent and severity in 1978, 1980, and 1982.

Generally, until now, the pavement has performed satisfactorily over peat deposits except for the several settlement zones where palsas were encountered and where leveling and reloading were periodically required.

A detailed survey of road sections built over major muskeg was done in June 1983. The survey consisted of

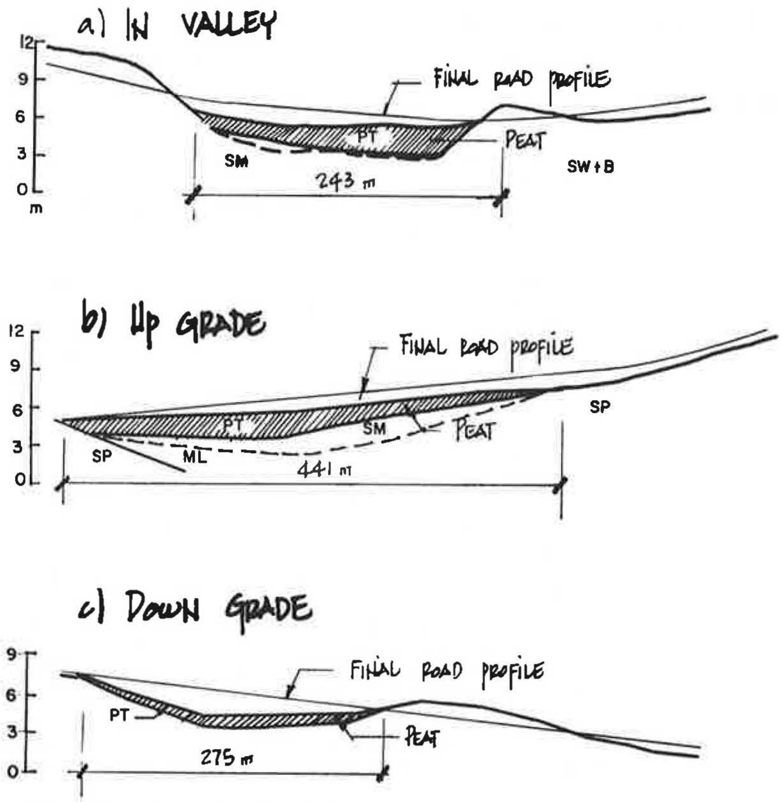

FIGURE 2 Typical profiles of peat deposits.

TABLE 2 Characteristics of 54 Deep Muskegs[a] Crossed by the James Bay Access Road (16.81 km or 2.7 percent of road)

Regional Distribution		km	Number	Length/km
James Bay Lowlands		0-150	4	1.66
		150-275	13	4.6
Lake Plateaus		279-450	18	6.2
		490-620	16	3.35
		600-620	3	1.00
Initial[b] Peat Thickness (m)		km	%	
<1.5		3.75	22.3	
1.5-2.5		11.00	69.4	
2.5-3.5		1.60	9.5	
>3.5		0.7	4.1	
Soils Type underneath Peat		km	%	
CL	A-7	6.55	39	
ML, CL, ML thin	A-5, A-6	6.00	35	
SM-SP	A-2, A-3, A-4	3.60	21	
SP	A-1	0.41	2.4	
Rock		0.46	2.5	
Topography		km	%	
Up grade		5.2	31.5	
Down grade		3.2	17	
Crossing of valley		6.0	35.5	
Plateau		3.0	17	
Maximum thickness of embankment (m)		km	%	
<1.5		2.7	16.1	
1.5-2.5		11.5	68.4	
2.5-3.5		2.0	11.9	
>3.5		1.0	6	

[a] Peat deposits having more than 150 m in length and more than 1.0 m in depth.
[b] Before construction of embankment.

TABLE 3 Degree of Decomposition After Von Post (5)

DEGREE OF DECOMPOSITION VON POST'S SCALE	INFORMATION FOR IDENTIFICATION
H_1	Completely unconverted and mud-free peat which, when pressed in the hand, only gives off clear water.
H_2	Practically completely unconverted and mud-free peat which, when pressed in the hand, gives off almost clear colourless water.
H_3	Little converted or very slightly muddy peat which, when pressed in the hand, gives off marked muddy water. The pressed residue is somewhat thick.
H_4	Badly converted or somewhat muddy peat which, when pressed in the hand, gives off marked muddy water. The pressed residue is somewhat thick.
H_5	Fairly converted or rather muddy peat. Growth structure quite evident but somewhat obliterated. Some peat substance passes through the fingers when pressed but mostly muddy water. The pressed residue is very thick.
H_6	Fairly converted or rather muddy peat with indistinct growth structure. When pressed at most 1/3 of the peat substance passes through the fingers. The remainder extremely thick but with more obvious growth structure than in the case of unpressed peat.
H_7	Fairly well converted or marked muddy peat but the growth structure can still be seen. When pressed, about half the peat substance passes through the fingers. If water is also given off, this has the nature of porridge.
H_8	Well converted or very muddy peat with very indistinct growth structure. When pressed, about 2/3 of the peat substance passes through the fingers and at times a somewhat porridgy liquid. The remainder consists mainly of more resistant fibres and roots
H_9	Practically completely converted or almost mudlike peat in which almost no growth structure is evident. Almost all the peat substance passes through the fingers as a homogeneous porridge when pressed.
H_{10}	Completely converted or absolutely muddy peat where no growth structure can be seen. The entire peat substance passes through the fingers when pressed.

- Boring, sampling, and measuring the static and dynamic shear strength using the Corps of Engineers type of penetrometers on the virgin peat outside the road, in the ditches, or through the embankment;
- Topographic surveys and determination of the profile of embankment resting on deep peat deposits; and
- Mays ride meter pavement roughness measurements in the most heavily used lane, along the muskeg and 2 km on either side of the muskeg.

CHARACTERISTICS OF 24 DEEP MUSKEGS

Twenty-four road sections crossing muskegs deeper than 1.5 m and more than 150 m long were selected for detailed performance studies using pavement evaluation data obtained since 1978 and results from detailed surveys made in 1982 and 1983. Table 4 gives the characteristics of the 24 most important peat deposits:

- Location and length of muskeg;
- Depth, Von Post classification, and shear strength of peat material;
- Type, quality, and estimate of shear strength of underlying material;
- Height, type, and typical profile of the embankment actually observed in the field; and
- Evaluation of embankment settlement 10 years after construction (June 1983); the settlement was estimated by boring through the fill material and by topographic surveys of embankments compared with the natural surrounding peat and embankment level.

TYPICAL CROSS SECTION OF SETTLED EMBANKMENT OVER PEAT DEPOSIT

Two typical cross sections of embankment were noted over peat deposits along the road (Figure 3):

- Where embankment height is less than 1.5 m, as it is in a spruce swamp muskeg, ditches 0.5 to 1.0 m deep are along both sides of the embankment and the actual level of the pavement surface has more or less reached the level of the muskeg due to settlement of the embankment. Fill material is typically uniform sand or sandy gravel, and the actual slope of embankment is small (i.e., 3 to 5 horizontal for 1 vertical).
- Where embankment height is more than 1.5 m and peat deposits are deep, with or without trees, and have open water, ditches are nonexistent and the slope of the embankment is less than 1.5 to 2.0 horizontal for 1 vertical. Most of the time, the rockfill has been placed over a bed of trees bridging the fill. Penetration of the fill into the peat deposit can be as much as 1.5 m, and the actual road level is 1.5 to 4 m above the surrounding muskeg.

TABLE 4 Description of Deep Peat Deposits (>1.5 m) Along the James Bay Road[a]

		Nature of peat deposit			Underlying material		Embankment			Settlement After 10 years	
Station km	Unit of peat deposit	Depth m	Von Post	Su[a]	Type	Su[a]	Height	Type[b]	Class (fig.3)	Meter	% peat depth
152.8	0.29	1.4	H-8	12-30	ML	100-200	1.4	SG	1	0.8	57
183.7	0.415	4.6	H-5		CL		1.3	SG	1	0.6	13
193.7	0.365	2.28	H-5	0-25	CL	40-90	1.7	SP-SG	1	1.3	57
202.7	0.14	2.2	H-6	45	IP=10 CL	40-60	1.51	Rock fill	1	0.7	32
206.5	0.21	1.5	H-4	0-15	ML sabl.		2.15	SG Rock fill	2	0.8	53
226.4	0.152	2.74	H-3	0-15	IP=10 ML	60-100	1.95	SG	1	0.9	32
230.5	0.46	3.66	H-2	0-25	IP=5 ML	30-150	1.23	SG	1	1.7	46
316.3	0.20	2.5	H-5	0-20	ML	45-70	2.33	SG	.2		
318.5	0.27	2.35	H-8	0-25	ML-C1	70-80	1.65	SG	2	0.90	38
319.0	0.31	2.30	H-8	0-17	ML-CL	70-80	1.65	SG	2	0.75	33
336.4	0.50	2.5	H-7	0-15	ML-SM		1.0	SP	1	0.75	28
337-6	0.19	2.1	H-7	0-27	ML-SM	75-80	2.1	SG	1	0.75	36
388.9	0.75	3.0	H-8	6-48	ML	80-105	2.7	SG Rock fill	1	1.0	33
518	0.23	2.5	H-10	0-12	SM-SP	N=25	4.2	SG Rock fill	2	0.90	36
548.1	0.21	2.5	H-6	0-20	ML		2.28	SG	1 Berm	0.70	28
549.7	0.285	2.0	H-10	93+	SM-ML	N=20	2.2	SG	1	0.75	37.5
550-4	0.14	2.0	H-7	12-30	ML	66	2.7	SG	1	0.6	30
550.6	0.40	2.1	H-4	3-18	ML	6.6-1.2	2.7	SG	2		
552.5	0.28	2.0	H-6	18-39	IP=3 ML		1.3	SG	1		
558.4	0.21	1.6	H-7	6-27		67+	2.6	SG	2	0.5-0.9	31-22
559.3	0.213	1.8	H-7	0-12	ML		2.57	SG	2		
604.1	.411	1.95	H-6	30-40	ML	120	2.35	SG-SP	2	0.6	31
609.9	.305	2.6	H-4 H-5	12-50	ML	110	1.65	SG	2	0.51	25
613.2	0.274	2.1	H-4	50-60	ML	140	2.30	SG	2	0.8	38

[a]Su, undrained shear strength resistance, KPa.
[b]According to unified soil system classification.

GEOTECHNICAL BEHAVIOR OF PEAT

Shear Strength of Peat

Shear strength of peat is always low. It is normally less than 40 kPa and less than 20 kPa for peat in the virgin state for most deposits outside the road. Under the embankment, the peat was consolidated and a notable increase in shear strength was observed; as shown in Figure 4, this is true even under low embankment heights.

Von Post Index and Long-Term Settlement of Embankment over Peat

Although a certain relationship exists between the Von Post index and settlement of peat after a week (5), this analysis did not confirm any clear relationship between settlement after 10 years and the Von Post index. This is shown in Figure 5. Long-term settlement of peat has been measured and varies between 13 and 57 percent of the thickness of peat deposits. The great variation can be attributed to different initial water contents and compressibility of the peat material, and to the type of material used as fill and occasional bridging with logs.

RIDING QUALITY OF PAVEMENT OVER PEAT DEPOSITS

A Mays ride meter was used to determine the riding quality of a pavement built over fill resting on peat deposits in each of the four muskeg zones. The riding quality was measured in the most heavily traveled lane. In most cases the riding quality was measured 2 km before the muskeg, over the muskeg,

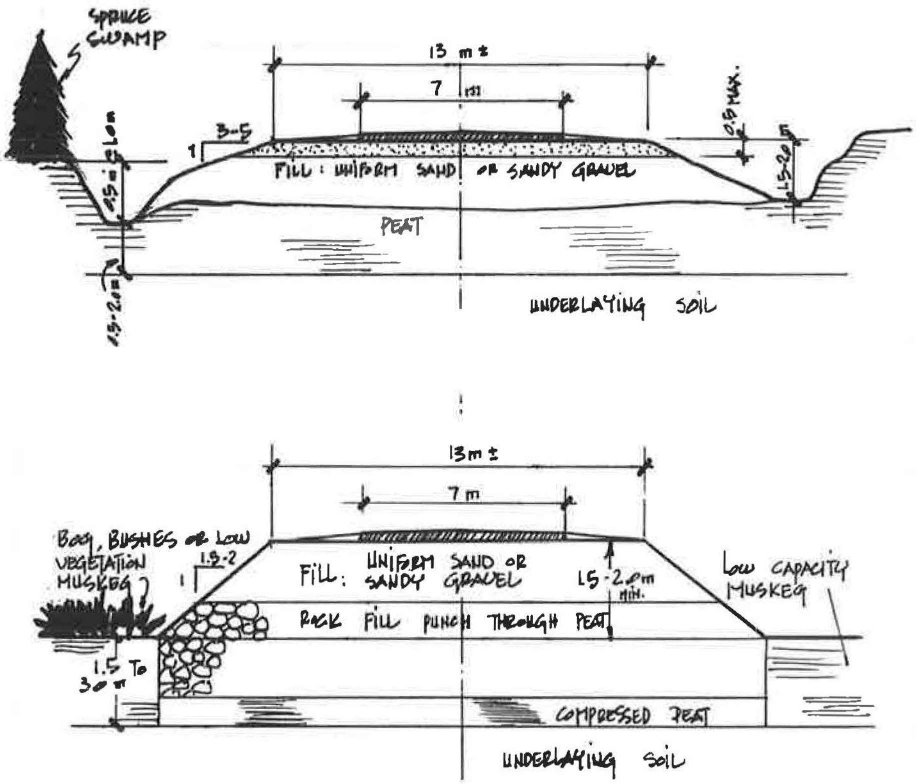

FIGURE 3 Typical transverse profile of embankment over peat sections.

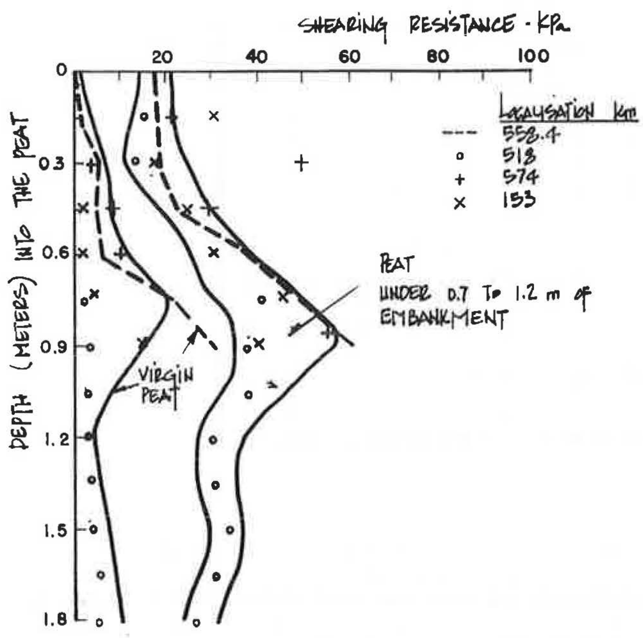

FIGURE 4 Increased shear resistance of peat material under low fill.

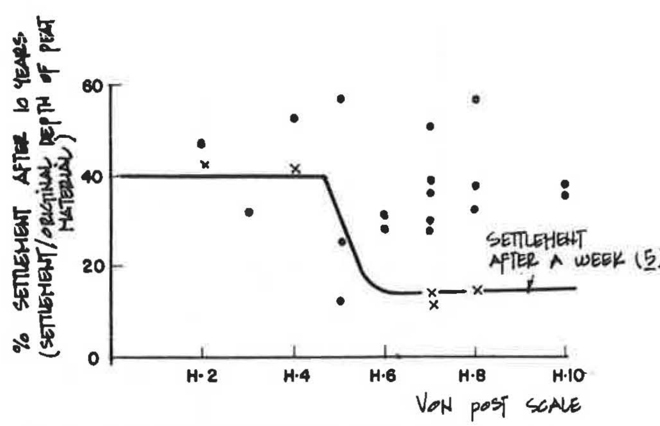

FIGURE 5 Relationship between long-term settlement of peat under 1.0 to 2.2 m of fill and Von Post classification *(5,6)*.

between peat deposits, and 2 km after the last peat deposit. The riding quality of minor peat deposits of less than 1.0 m in depth or less than 0.15 km in length was also evaluated.

Table 5 gives a summary of test results of the evaluation conducted in June 1983. As indicated, the pavement is about 50 percent rougher over peat deposits than over sections with no peat (Mays RCI 131 versus 87 in./mile). A particularly important point in the data observed is the transition zone between peat and no-peat sections. As the data given in Table 6 indicate, the riding quality of the pavement in the transition zone is about 40 percent rougher than the riding quality of the pavement outside transition zones (Mays RCI 109 versus 81 in./mile).

CRACKING OF ASPHALT PAVEMENT OVER PEAT DEPOSITS

The extent and severity of cracking have been evaluated every 2 years since the paving of the access road in 1976. The amount of longitudinal or transverse cracking was measured, and the extent of longitudinal cracking, which includes centerline, lane, or edge cracks (Figure 6a), is expressed in meters per kilometer of length of the road.

Transverse cracking is classified in four types and is expressed in the number of cracks per kilometer of road as shown in Figure 6b:

TABLE 5 Riding Quality of Pavement Over Deep Peat Deposits (>1.5 m) Along the James Bay Road Compared with Adjacent Pavement

Pavement over peat deposit					Pavement outside peat deposits (in relation to peat deposit)					
			Riding confort index		Before		Between		After	
Localization	Nb of 1/20 m sections	Total length	Nb	Mean in/mile	Nb	Mean in/mile	Nb	Mean in/mile	Nb	Mean in/mile
150-156	3	0.88	12	100	37	116	22	93	13	96
182-186	2	0.88	12	125	32	108	17	134	5	174
196-200	2	0.65	8	195	22	122	7	113	13	104
202-208	2	0.49	6	193	9	145	54	120	21	106
225-232	4	0.81	10	168	21	100	14	131	41	110
315-322	5	1.85	23	102	20	57	16	58	28	69
332-340	2	0.81	10	58	33	70	7	62	31	51
386-394	2	1.45	18	82	36	50	21	46	20	52
534-536	2	0.56	7	106	6	46	13	45	18	41
546-553	10	2.58	32	177			54	114		
553-562	9	2.41	30	126	18	119	39	100	24	84
561-565	1	0.25	3	84	17	63			23	71
572-580	5	0.88	12	111	21	59	25	74	48	62
602-606	4	1.61	20	141	17	53			14	66
608-616	9	2.49	31	200	8	65	48	96	8	73
TOTAL	62	18.6	231	1968	391	1173	283	1186	307	1159
AVERAGE				131		84		91		83

TABLE 6 Riding Quality at Transition Zones and Outside Transition Zones

Localisation km	At transition zones		Outside transition zones		
	Number	Riding quality	Number	Riding quality	Ratio
150-156	7	99	65	106	0.93
182-186	7	160	47	117	1.37
196-200	4	123	38	114	1.08
200-208	4	116	80	93	1.25
225-232	9	128	67'	109	1.17
315-322	16	119	48	61	1.95
332-340	4	77	67	60	1.28
386-394	7	83	70	44	1.89
534-536	2	51	37	43	1.19
546-565	37	96	138	97	0.99
602-606	5	119	26	47	2.53
608-618	12	133	52	79	1.68
TOTAL	114	109	817	81	1.44

Note: The length of transition zone is 160 m: 80 m before the peat deposit and 80 m on the peat deposit.

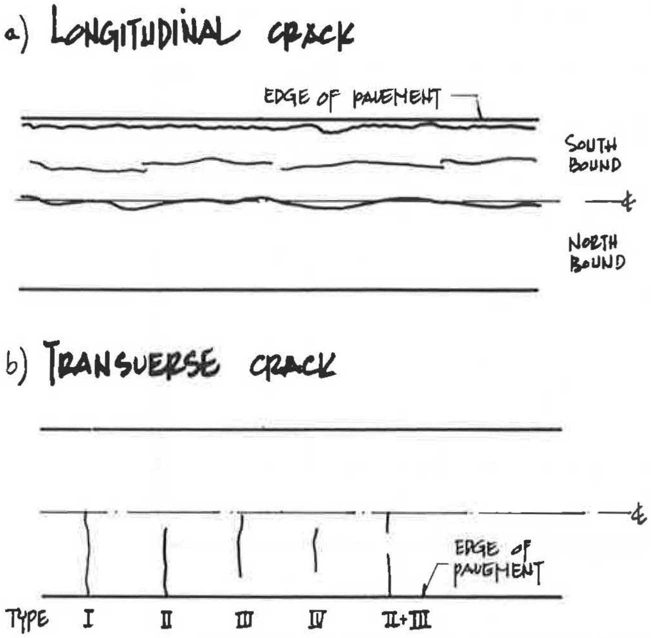

FIGURE 6 Typical crack patterns.

- Type I cracks extend from the centerline to the edge of the pavement;
- Type II cracks extend from the edge of the pavement toward the centerline;
- Type III cracks extend from the centerline toward the edge but do not cross the complete lane; and
- Type IV cracks are typically lane cracking without crossing the edge and the centerline.

A summary of the extent of cracking (measured in June 1983) of the 24 sections of pavement built over peat deposits is given in Table 7.

Longitudinal cracking is common in both directions in pavements built over peat deposits. As the data given in Table 8 indicate, the length of longitudinal cracks in pavement built over peat deposits is generally 2 to 4 times greater than in other areas. An attempt to correlate length of cracking with either height of fill or depth of peat deposits was not successful even where longitudinal cracking is a visual sign of embankment instability.

Types II, III, and IV transverse cracks are common over peat deposits. Type I cracks usually occur at the beginning and the end of the peat deposits or directly above a culvert. Sometimes frost cracks, which extend beyond the pavement into the shoulders, are also found.

TABLE 7 Cracking of Pavement Built Over Deep Peat Deposits (>1.5 m) Along the James Bay Road

Station km	Length of peat deposits	Longitudinal cracking[a]					Transverse cracking[a]				
		Center line	Lane cracking		Edge cracking						
			N	S	N	S	I[b]	II	II et III	III	IV
152.8	0.29	3	3.6				3	9		16	10
183.7	0.415	28	9.1	104	40.3	20.4	19	4		24	4
197.7	0.365	82.3	11.6	65.5	36.6		4			9	4
203.6	0.14	31.4	10.6	30.5			$4^{(2)}$			2	4
226.4	0.152	15.2			38.1			4		3	6
230.5	0.46	198.1	74.7	74.7			12	3	6	34	22
206.5	0.21		45.7	22.9	152.4			1		2	1
316.3	0.20	9.1	30.5	45.7	45.8		$5^{(4)}$			2	
318.5	0.27		6.1	25.9		265	$8^{(6)}$	13	5	39	15
319.0	0.31				79.3		$4^{(2)}$			1	1
336.4	0.50	21.3	4.6	10.7	30.7	131	23	2	3	24	
337.6	0.19	6.1	9.1	27.4	31.1		2			40	
388.9	0.75	39.6	38.1	33.6	369		$11^{(2)}$	1		3	1
518	0.23	97.5	28.9	6.1	3.1		14	2	1	16	16
548.1	0.21	9.1	82.3	82.3			64	14	3	19	17
549.7	.285	48.8	16.8		88.4		6		3	3	2
550.4	0.14		12.2	9.2	12.2		7			4	10
550.8	0.40	26.5	6.1	67	45.7	60.9	49	10	4	16	6
552.9	0.28	76.2		44.1		15.2	28		4	16	0
558.4	0.21	26.5					10			8	3
559.3	0.213	25.9	30.4	4.6			4			4	
604.1	.411	241.7	356.6	201.1	33.5		115	41	20	60	25
609.8	.305	35.0	42.7	17.3		7.6	14	26	32	.27	8
613.2	.274	100.5	13.7	128.3	9.1		28	8	6	25	3

[a]See Figure 6 for description of cracks.

[b]$4^{(2)}$ = 2 out of 4 transverse cracks are crossing the shoulders on both sides.

TABLE 8 Cracks in Pavement on Peat Deposits Compared with Cracks in Pavement on Ordinary Soil

Localization	Longitudinal cracks m/km			Type 1 transverse cracks nb/km		
	On peat deposits	On ordinary soils	Ratio	On peat deposit	On ordinary soils	Ratio
km 180-240	664	18	37.0	19	12	1.6
km 330-370	787	173	4.5	22	32	0.7
km 370-390	635	79	8.0	15	19	0.8
km 540-560	407	70	5.8	95	69	1.4
km 600-620	1160	40	29.0	143	69	2.1
TOTAL	3653	380	9.6	294	201	1.5

Types II and III cracks often occur together and could represent developing type I cracks. As the data in Table 8 indicate, there are generally fewer type I cracks over peat sections than over other sections, except between kilometers 600 and 620 where the traffic is heavy.

Table 9 and Figure 7 present data on cracks over peat deposits from 1980, 1982, and 1983 surveys. Cracking increases rapidly with time: Longitudinal cracks and particularly edge cracks almost doubled between 1982 and 1983. However, type I transverse cracks did not increase substantially during the same period.

STRUCTURAL BEHAVIOR OF PAVEMENT OVER PEAT DEPOSITS

Analysis of structural behavior of pavement over selected peat sections has been done using Dynaflect deflection data, measured every 0.25 km along the road, collected in 1978 and check data collected in 1979. Maximum deflection of pavement over peat deposits is generally higher than deflections measured over other soil types, whatever the height of embankment. Studies (3,4,12) suggest that a minimum acceptable height of embankment over peat deposit is necessary to assure acceptable performance; 1.0 to 1.3 m of embankment is generally suggested.

TABLE 9 Cracking of Pavement on Peat Deposits—Comparison of 1980, 1982, and 1983 Surveys

Type of crack	Unit	Cracking - Average		
		1980	1982	1983
Transversal, Type 1	Nb/km	38	47	50
Longitudinal, All types	M/km	180	298	615
Lane cracking		-	210	287

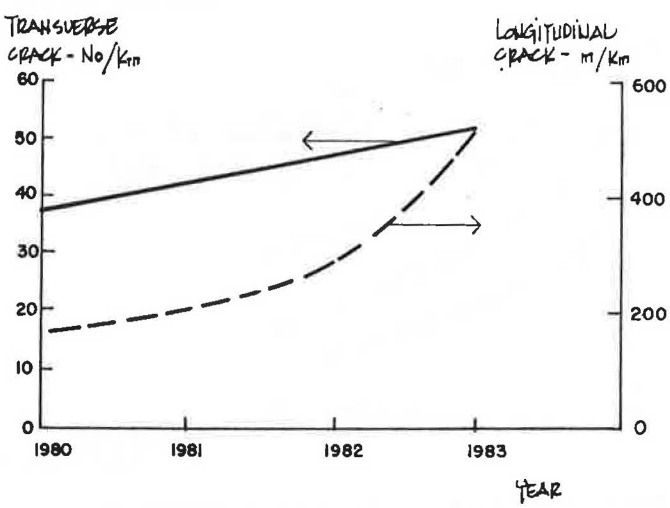

FIGURE 7 Progression of cracking of pavements built over peat deposits, James Bay access road.

The relationship between measured maximum Dynaflect deflection (S_1), deflection at 1.2 m (4 ft) from the loading point (S_5), height of embankment over peat deposit above ditch level (type I embankment), and height above level of surrounding muskeg (type II embankment) is shown in Figures 8a nd 8b. Using statistically derived data it is possible to determine the required height of embankment to limit the deflection to a certain design value. As an example, an embankment height of 3 m or more will always result in a maximum deflection of less than 1.0 E-3 in., and (at a height of 1.2 m) a deflection of less than 0.3 E-3 in.

Summary results of an evaluation of the dynamic modulus of compressed peat 10 years after construction are given in Table 10. The modulus was calculated with the FHA program OAF for Dynaflect loading condition and for 40 kN wheel load. The following constants were used in the calculation:

	Density (kN/m³)	Thickness (m)	Poisson's Ratio
Bituminous surface	22.76	6.4	0.40
Base course	21.90	45.7	0.37
Embankment	19.15	See Table 10	0.45
Compressed peat	10.98		0.50

Examination of all test results indicates that the dynamic modulus of compressed or consolidated peat is quite constant with a mean value of 7 ksi under Dynaflect loading and 12 ksi under 40 kN wheel loads. Eighty-five percent of the values are equal to or greater than 6 and 2, respectively, for the two loading conditions.

SEVERE SETTLEMENT OF PAVEMENT ON PEAT DEPOSITS

A detailed survey of severe settlement zones where pavement was badly deteriorated revealed that settlement is due to the presence of palsa (or its ice core) buried under muskegs covered with black spruce and tamarack. Indeed, these settlement zones are characterized by the fact that their settlement far exceeds that which is predictable by geotechnical calculation; the level of soil-embankment contact generally varied from 1.5 to 2.5 m below natural ground level, whereas 20 to 30 cm could have been anticipated from soil consolidation.

Settlement generally appears either in late fall or in early summer and is highly differential. Settlement is rapid at first and slower during each following thaw season. As shown in Figure 9, total settlement of pavement over palsa varies between 1.0 and 1.3 m and the maximum annual rate of settlement varies between 20 and 30 cm. The problem of design and maintenance of pavement over palsa is discussed by Keyer and Laforte in another paper in this Record.

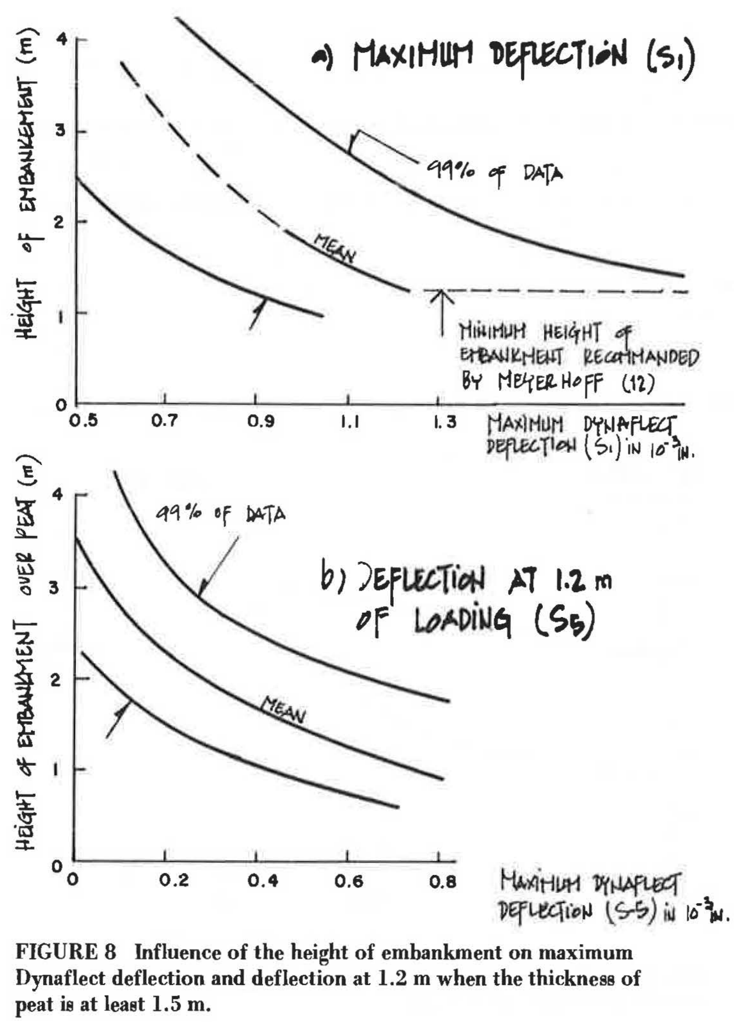

FIGURE 8 Influence of the height of embankment on maximum Dynaflect deflection and deflection at 1.2 m when the thickness of peat is at least 1.5 m.

TABLE 10 Dynamic Modulus of Peat Subjected to 10 Years of Consolidation

Peat Von Post Scale	Heigth of embankment cm	Under Dynaflect loading		Under 40 kN (9 000 lb) wheel load	
		ksi	mPa	ksi	mPa
H2	124	9	59	17	121
H4	39	8	57	12	81
H5	69	7	51	12	81
	78	6	43	11	77
	160	2	15	5	33
H7	78	10	72	18	127
H8	35	7	48	10	70
Mean	83	7	49	12	84
Range	35-66	2-10	15-72	5-17	33-127
85%	Value above	6	43	10	70

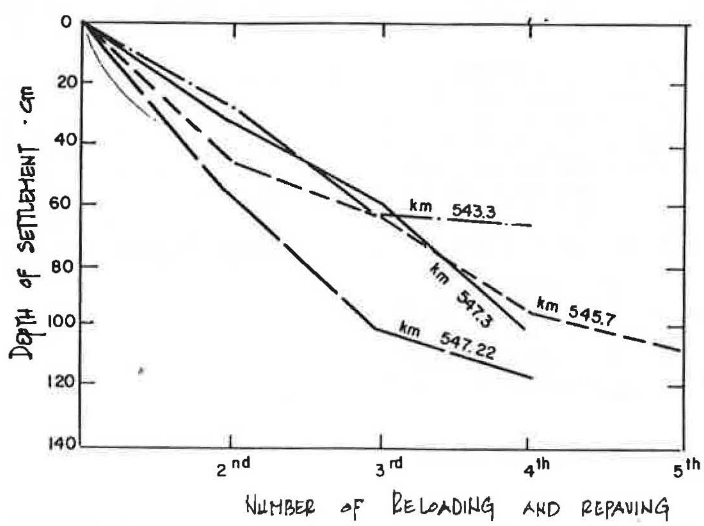

FIGURE 9 Progressive subsidence of five settlement areas over palsa field.

CONCLUSIONS

The following conclusions refer to the performance of pavement and embankment built over muskegs in northern Quebec. The design of embankment is based on relations established by Lefebvre et al. between the Von Post classification of peat and its physical and mechanical behavior under loading.

Along the 620 km of the James Bay penetration road, which run straight north from parallel 49.8° N to 53.8° N, the length of peat deposits along different sections covers a range that varies between a minimum of 6 percent and a maximum of 37 percent of the length of each section. One-third of all peat deposits are more than a meter deep. Most of the muskeg areas along the road are concentrated in four well-defined zones 80 km, 85 km, 35 km, and 14 km in length. The 620 km of road cross 54 deep muskegs that together represent about 16 km of road. The characteristics of each muskeg have been defined in terms of regional distribution, initial peat thickness, type of soil underneath the peat, topography, and maximum thickness of embankment.

The design of embankment over muskeg was based on the concept of preconsolidation; the order of magnitude of the duration of load was evaluated using the Von Post classification of peat. Design approach based on simple classification was found to be practical for northern roads in view of the great number of muskegs, the difficulty of field access, the variability of peat deposits, and the difficulty of predicting settlement with precision.

The performance of pavement and embankment built over muskegs has been evaluated periodically by Dynaflect deflection measurements, Mays road roughness measurements, and condition surveys. On the whole the performance of pavement 8 years after construction is satisfactory.

Two typical cross sections of settled embankment over peat deposits were noted: embankment made of sand or sandy gravel, which has settled to the level of the surrounding muskeg, and embankment made of rockfill constructed over a bed of trees, which is 1.5 to 5 m above the surrounding muskeg. For peat deposits thicker than 1.4 m the long-term settlement generally varies between 0.5 and 1.0 m or between 25 and 50 percent of the depth of the peat deposit. No clear relationships can be found between long-term settlement and the following properties: depth of peat deposit, Von Post classification of peat, shear strength of peat, material underlying peat deposit, embankment height, and type of embankment.

Shear strength of peat is always low; it is normally less than 20 kPa in the virgin state and 40 kPa under embankments.

Although the riding quality of pavement on peat deposit is still rated good, the pavement surface is 50 percent rougher over peat deposits than over no-peat sections. The riding quality of pavements in transition zones between peat and no-peat sections is about 40 percent rougher than that of the no-peat sections.

Transverse cracking is not influenced by the presence of peat deposits; however, longitudinal cracks in pavement built over peat deposits are generally 2 to 4 times larger than cracks in other areas and are increasing rapidly. It has not been possible to correlate the length of longitudinal cracking with either the height of fill or the depth of peat deposits.

Maximum Dynaflect deflection of pavement over peat deposits is generally higher than deflections measured over other soil types whatever the height of embankment. There is a loose relationship between the height of embankment and the maximum deflection.

The dynamic modulus of peat under embankment is quite constant; it is around 50 mPa (7 ksi) under Dynaflect loading and 80 mPa (12 ksi) under 80 kN (18 kips) axle loads.

Severe settlement of fill on peat deposit encountered along the road is due to the presence of ice under the peat. Total settlement of pavement over palsa varies between 1.0 and 1.3 m and the maximum annual rate of settlement varies between 20 and 30 cm.

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