Highway Design and Construction Over Peat Deposits in Lower British Columbia

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Peat or muskeg is common in the lower mainland area of British Columbia near Vancouver. Initially, roads and highways were constructed around these soft deposits. Recent, more intensive industrial and residential development, however, has forced the construction of transportation facilities through peat areas.

Excavation and displacement, the normal methods of dealing with peat, are very expensive. Their potential high cost forced the Department of Highways of British Columbia to investigate the use of preconsolidation. Several test sections were constructed, instrumented, and evaluated. Results indicated that this technique could provide stable high-standard highways with acceptable riding characteristics.

Portions of several major highways have been constructed successfully in British Columbia using preconsolidation. The most recent and most important is several miles of the Trans-Canada Highway in the Vancouver area which has been constructed to full freeway standard. Data obtained from the field and laboratory are outlined. In particular, some of the general properties of the peats are summarized and methods for predicting settlement, evaluating stability, and determining pavement thickness are discussed.

In general, in the lower mainland area of British Columbia peat has not been as critical a construction material as is often suggested. Frequently, it has been easier to handle than the very soft inorganic clays which often underlie the peat.

• ON THE west coast of North America, the region including Seattle, Wash., and Vancouver, B. C., contains extensive organic swamp land or organic terrain. These peat deposits have for many years been extensively mined for horticultural purposes near Vancouver, B. C., but only within the last ten years have they been traversed by major highways.

DEFINITION OF TERMS

Some confusion has arisen over the terms: peat, muskeg, organic terrain, and swamp deposits. Peat is the common term used quite widely through the English-

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speaking world for soil composed predominantly of noncolloidal organic material. Muskeg seems to have been derived from the North American Indian "maskek" (Cree), "mashkig" (Ojibway), and "maskeg" (Chippewa) meaning a swamp. The word is more prevelant in Canada than elsewhere. It was apparently taken over from the Indian by construction men to express the drama of building over the peat bogs which are common in northern Canada. The term "muskeg" carries the connotation of a swampy environment as well as the meaning of a peat soil.

A swamp or bog is any very wet ground. Swamp deposits may be either organic or inorganic. They vary greatly in character.

"Organic terrain" was introduced by Radforth (1) and is primarily applicable to his work in connection with vegetal cover and surface topography, particularly with reference to their generic connection with the underlying soil. For soil engineering purposes, the term "peat" appears most useful and is used extensively by the authors. Certainly as soon as a sand blanket is placed over peat, the terms "muskeg" or "organic terrain" should no longer be applied to the soil. Neither is it appropriate to use the terms "organic terrain" or "muskeg" with reference to a soil sample in the laboratory.

DISTRIBUTION OF PEAT LANDS

Although the Province of British Columbia contains 366,250 sq mi (nearly 100,000 more than Texas), most of the 1,750,000 population is huddled in the valley of the Lower Fraser River. This region is called the Lower Mainland. At the mouth of the Fraser River is Metropolitan Vancouver (Fig. 1). As Vancouver has developed, the better land has been used first, so that the extensive peat deposits of the area have not been used for major construction until the last ten years.

In the Municipality of Richmond, just south of Vancouver City, there are two bogs, which together constitute 30 percent of the land area or about 15 sq mi. Peat is now being mined from these bogs. Some secondary roads traverse them and, in 1959, the Deas Tunnel Throughway was constructed across the westerly extremity of the peat area. The peat in Richmond is of variable thickness. The maximum depth known to have been measured is about 20 ft. It is commonly underlain by a silty clay with low but significant strength. Major marine and industrial construction is to be expected in Richmond on peat lands near the Fraser River.

The Municipality of Delta, south of the Fraser River, contains the Delta Bog which is 20 sq mi in extent comprising 30 percent of the municipality. This bog has recently been traversed by the Deas Tunnel Throughway and by the Canadian National Railways. Peat is being mined from the Delta Bog. The peat varies in thickness up to 30 ft. It is underlain either by sand or by silty clay of medium strength.

Moving eastward up the Fraser River, there are peat bogs along the river in North Surrey, Port Moody, and Maillardville. In these areas the peats are usually shallow—up to 15 ft and underlain by a silty clay of medium strength. Occasional depths of peat to 25 ft have been observed. Major highway construction crossed these deposits in the Maillardville area in 1954 with the Lougheed Highway and in 1961 with the Burnaby Freeway Section of the Trans-Canada Highway.

There are other scattered peat deposits in the lower mainland; for example, the small one near Chilliwack crossed by the Trans-Canada Highway. But the peat bogs with which most experience has been gained are in Burnaby which contains 5 ½ sq mi of peat comprising about 20 percent of its land area. The South Burnaby Peat Bog has not yet been the scene of major construction but the Central Burnaby Peat Bog is now being crossed by a major freeway (Fig. 2). In this deposit there is up to 40 ft of peat underlain by up to 40 ft of exceedingly soft and sensitive silty clay.

HIGHWAY CONSTRUCTION EXPERIENCE

When it was no longer possible to avoid the peat bogs completely, some secondary roads were constructed through them. The first roads on Lulu Island, south of Vancouver, used side borrow construction (Fig. 3, 4, and 5). Peat was excavated from the ditch area to facilitate drainage and piled on the grade. This was followed by local

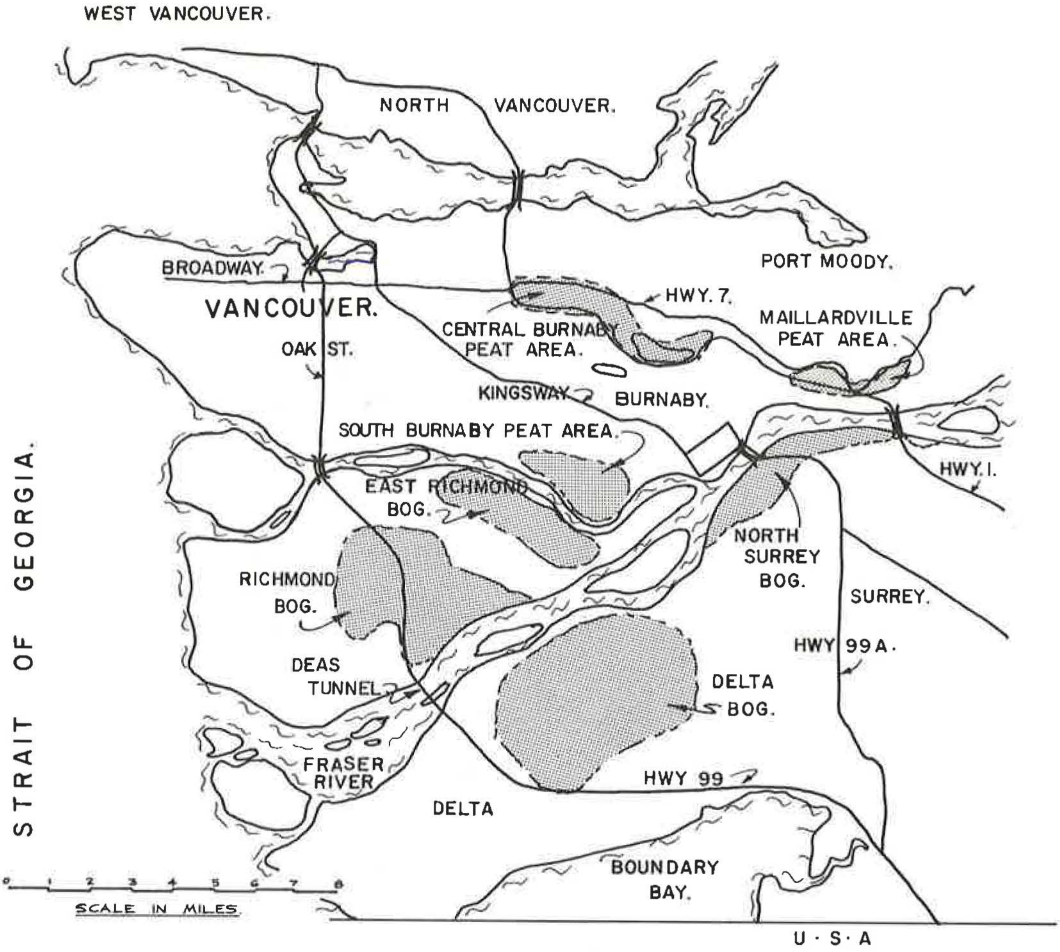


Figure 1. Greater Vancouver peat areas.

soil and a layer of gravel of sufficient thickness to carry the traffic. Needless to say, these roads suffered severe distortion and settlement during their first years of use. However, gravel was continuously added and, finally, an asphalt surface. The riding qualities of these roads are substandard but most of them are still in use and carrying heavy traffic.

In the Vancouver area, it was common practice to place several feet of gravel directly on top of the peat. No attempt was made to provide drainage ditches. As settlement occurred, more gravel was placed, ultimately followed by asphaltic pavement. These roads exhibit inferior service characteristics but at the present carry considerable traffic.

In North Surrey, considerable use was made of corduroy. Gravel was placed on top of the timbers and continually added as settlement took place and increased standards were required. This was followed by a concrete or asphaltic surface. On major thoroughfares several surface courses were placed. For example, a test hole drilled on the existing Trans-Canada Highway near the south end of the Pattullo Bridge at New Westminster revealed the profile given in Table 1. The history of this section is self-evident.

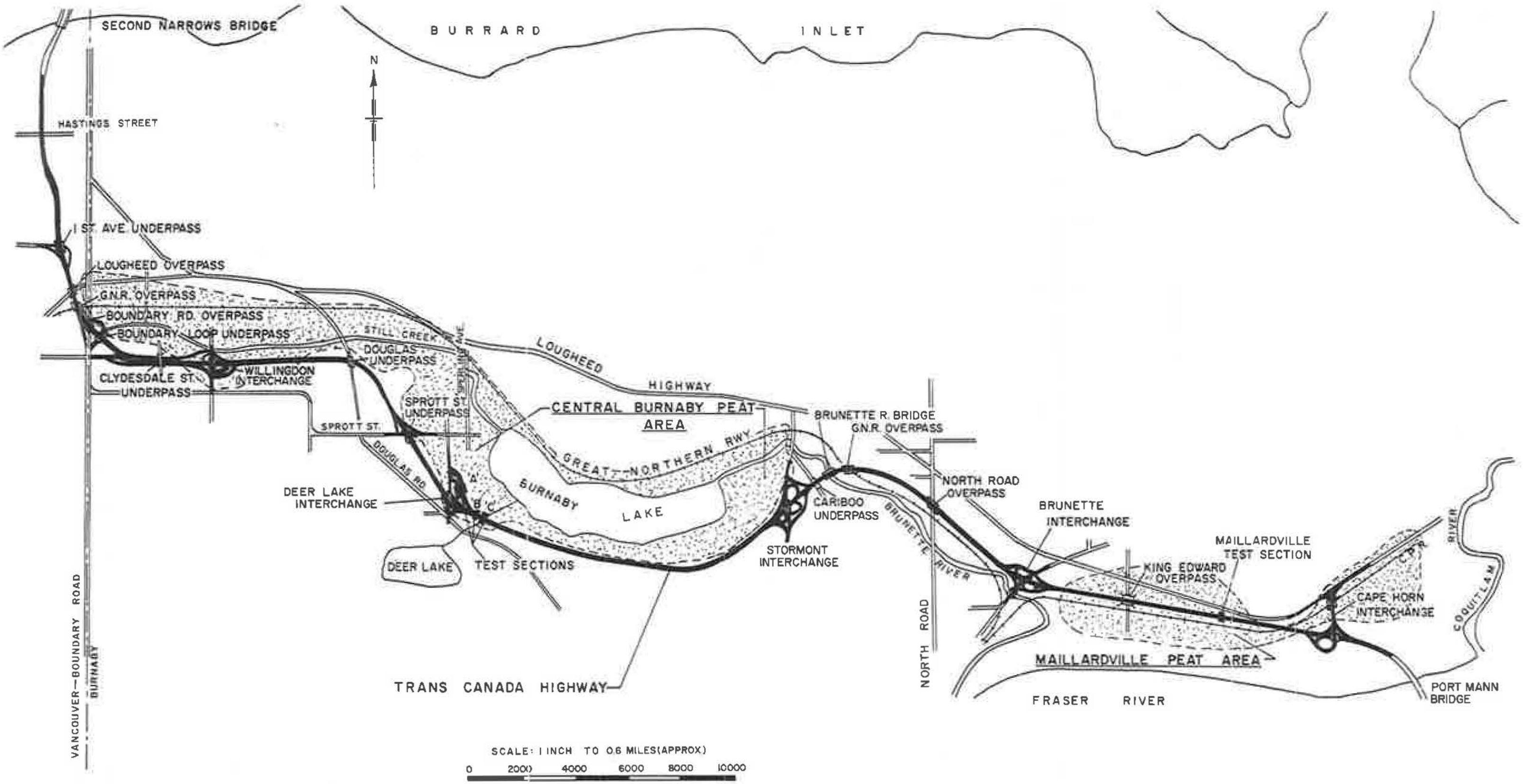


Figure 2. Vancouver, Fraser River section, Trans-Canada Highway.

Figure 3. Road built by side borrow.

Figure 4. Road built by adding gravel.

TABLE 1

PROFILE OF TEST HOLE NEAR PATTULLO BRIDGE, NEW WESTMINSTER

Depth (ft)	Material
0 - 1	Asphaltic concrete[a]
1 - 3	Gravel
3 - 3.7	Concrete
3.7 - 7	Gravel
7 - 8	Wood (corduroy)
8 - 17	Peat
>17	Blue clay

[a]Original thickness, 3 in.

Figure 5. Lulu Island test section.

Maillardville Section of Lougheed Highway—Sand Drains

The first major highway construction project over peat was the Lougheed Highway near Maillardville, built in 1954. The profile consisted of up to 25 ft of peat and highly organic clay overlying up to 15 ft of soft to stiff clay, in turn underlain by a dense till.

Excavation or displacement was considered desirable but was ruled out because of excessive cost. Instead, it was decided to "float" the road over the muskeg and accept moderately extensive future maintenance. As an experiment, vertical sand drains (Fig. 6) were installed over a 1,000-ft section to assess their effectiveness in peat. The drains were installed at a spacing of 15 ft in a triangular pattern, to a depth of 25 to 35 ft using the closed mandrel method. It was calculated theoretically that the drains should have increased the rate of settlement by at least 8 times. Detailed analysis of the data indicated the sand drains increased the rate of primary settlement by only about 15 percent (Fig. 7).

No piezometers were installed to measure pore pressures. However, the section with sand drains exhibited no vibration when heavy vehicles passed by, whereas the

Figure 6. Filling sand drain with sand.

section without sand drains underwent a very noticeable jelly-like vibration. From this it was inferred that the sand drains allowed rapid pore pressure dissipation and hence the drains were considered to be functioning.

One major error in construction occurred on this project. The majority of culverts were constructed on pile foundations. These, of course, did not settle with the grade and within a few months severe bumps developed at all culvert locations. These locations still require occasional repair.

Deas Tunnel Throughway—Excavation

In 1957, a four-lane freeway from Vancouver to the United States border was planned, with 2 miles of the location crossing peat ranging in depth from 6 to 11 ft. During the early stages of design it was considered that the peat could possibly be preconsolidated to provide a stable grade without costly excavation of the peat. A test section was proposed and constructed. Unfortunately, results were not available in time for the final design. Consequently, it was decided to excavate the peat completely and replace it with sand dredged from the Fraser river. About one year after the freeway was opened, differential settlement became noticeable in some of the areas where the peat was removed. By September 1962 this differential reached as much as 3 in. over a length of 50 ft. Drilling revealed pockets of peat and organic silt under the sand fill in the areas of settlement indicating that all the organic material was not removed during construction. This is one of the main difficulties encountered when excavation or displacement of the peat is employed.

Lougheed Highway at Boundary Road

The first project to employ the surcharge or preconsolidation method was the reconstruction of the Lougheed Highway at Boundary Road in Vancouver in 1958. About 1,200 ft of the original highway had been constructed on top of a peat deposit up to 14 ft deep. Reconstruction was to upgrade the highway from two to four lanes.

The entire fill was brought to grade followed by a 5-ft surcharge topped by a temporary surface which was used by traffic for three months. Following removal of the surcharge, base gravel and asphaltic concrete were laid. Field instrumentation, comprised of settlement plates and piezometers, was used to control the rate of construction. No difficulties were encountered. In the first four years of service from 1958 to 1962, no differential settlement or pavement failure was apparent.

Burnaby Freeway Section of the Trans-Canada Highway

The Burnaby Freeway Section of the Trans-Canada Highway connects the Second Narrows Bridge with the Port Mann Bridge. It is a 12-mi stretch of ultimate 8-lane freeway designed to the highest freeway standards, fully grade separated with some 22 grade separation structures in the 12 mi. Traveling from Port Mann towards Vancouver, the highway crosses a number of peat deposits. First, almost 2 mi of the Maillardville peat area is traversed. A test section was built in this area in 1958 (Fig. 8). The Caribou Interchange and the Freeway westward from the Interchange skirt the Central Burnaby peat bog with some of the secondary roads crossing the peat. The Deer Lake Interchange, however, with the sections of the Freeway and secondary roads close to it, overlies the deepest section of the Central Burnaby Peat Bog. The original highway line was laid out to avoid this deep and very difficult section of bog,

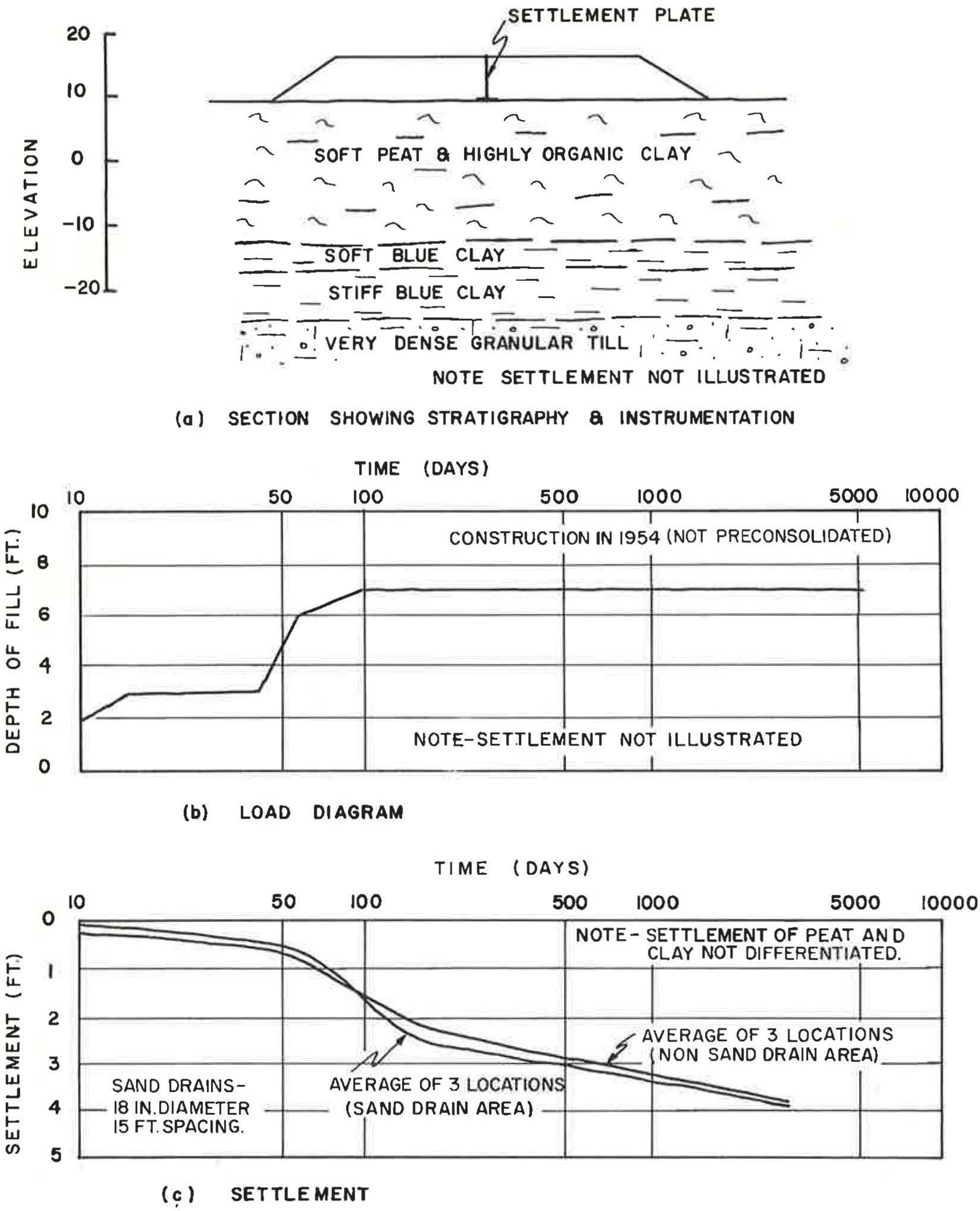


Figure 7. Lougheed Highway at Maillardville.

but the Municipality requested that the highway use the poorest soil in the area to leave the better ground for commercial and other development. The Provincial Government agreed to do this, recognizing that it involved considerable extra cost. Three test sections were built in the Deer Lake Interchange area (Figs. 9, 10, and 11). In the Willingdon Interchange area, there are about ¾ mi of freeway and ¾ mi of secondary roads which cross peat deposits similar but not quite so severe as the Deer Lake Interchange. In total, about 4 mi of this 12-mi stretch of freeway is across peat, and

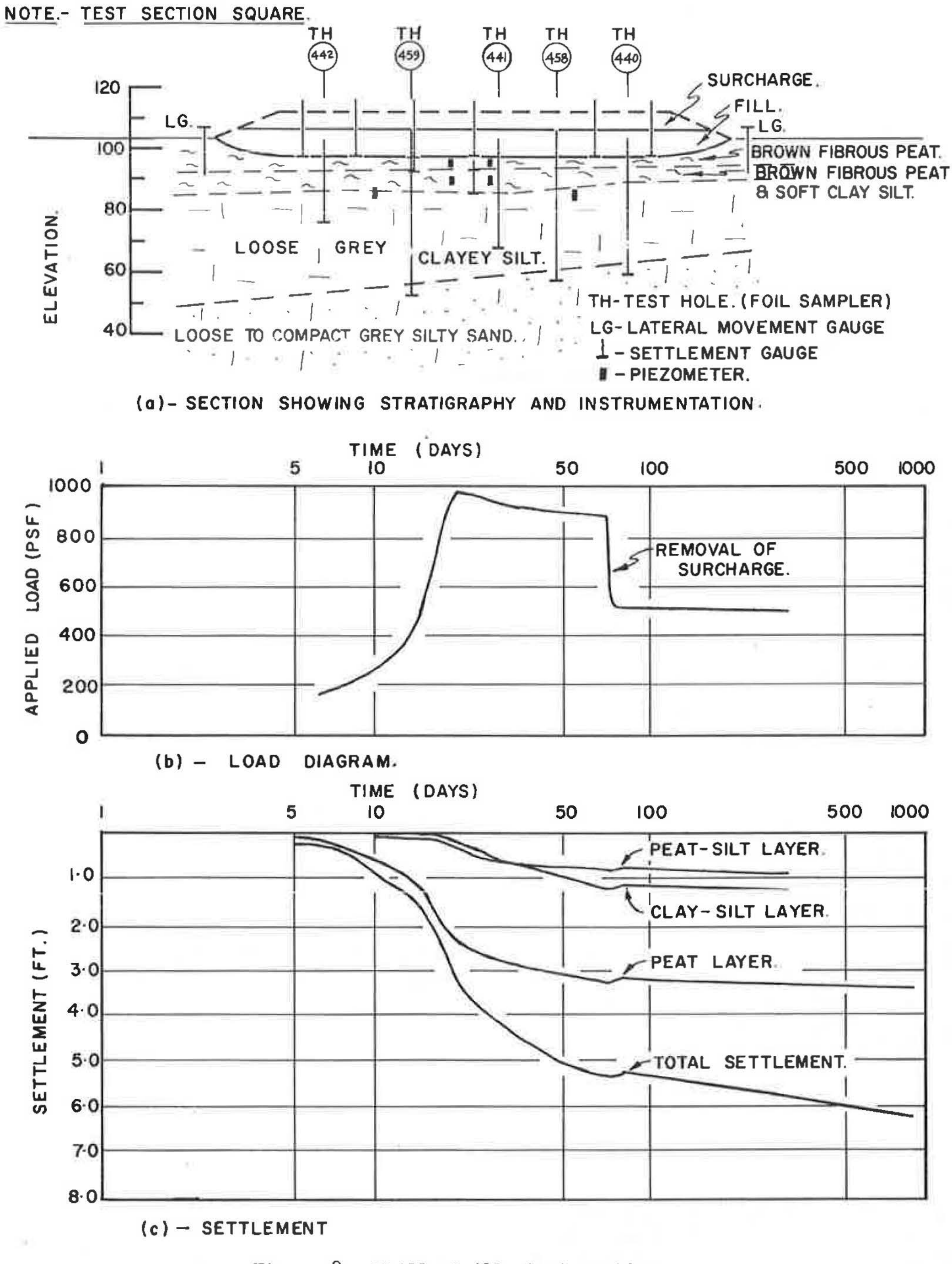


Figure 8. Maillardville test section.

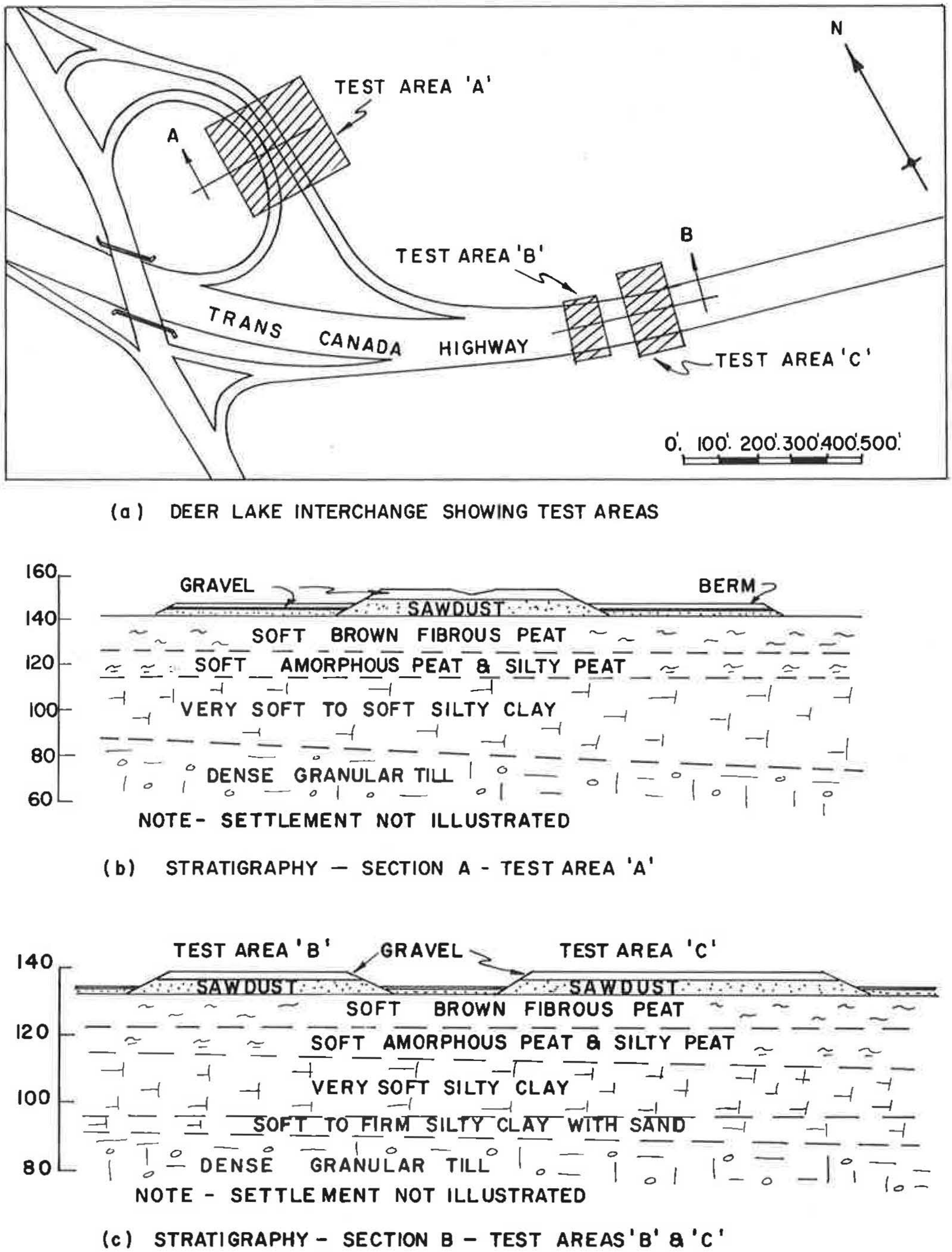


Figure 9. Deer Lake interchange.

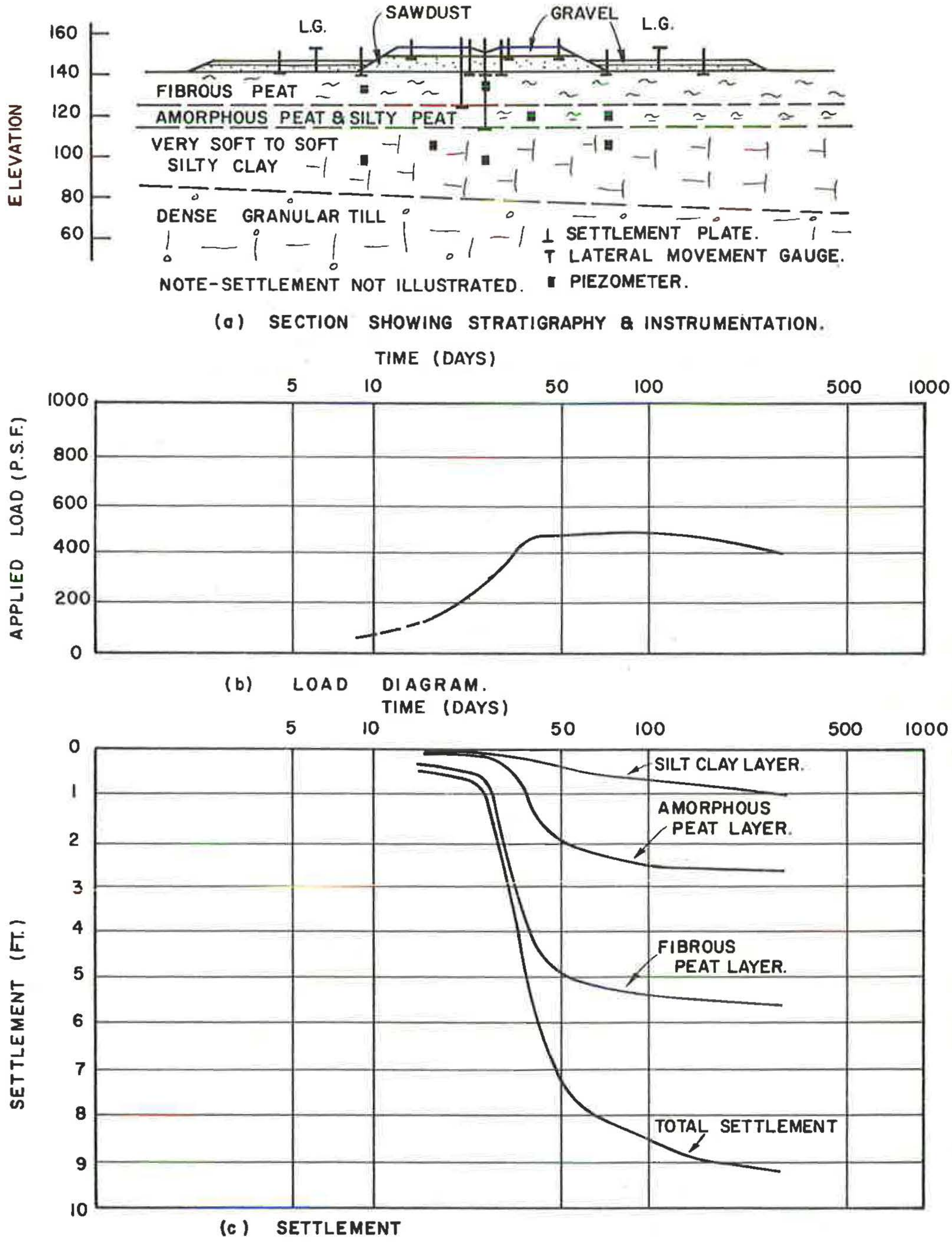


Figure 10. Test area A.

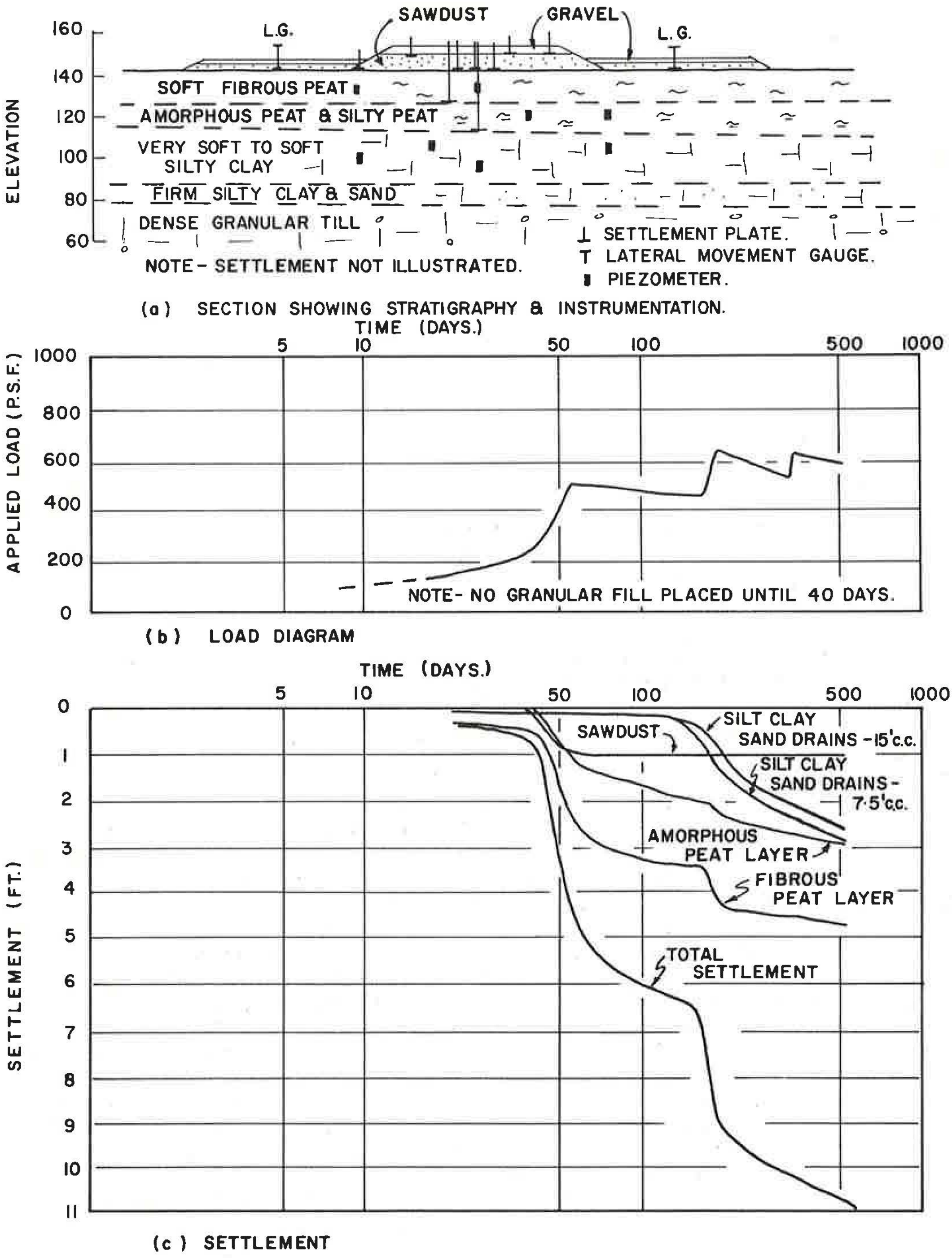


Figure 11. Test area B.

in addition, several miles of secondary roads cross peat areas. This highway has been under construction since 1959. It will be open to traffic in 1963. Two basic methods of peat treatment have been employed. For shallow deposits, preload only has been used, whereas, in deep deposits underlain by soft clay, preload and lightweight fill have been employed. These two types of treatment are considered separately.

Shallow Deposits: Preload Only. —Three principal sections of the freeway, that in the Maillardville area, that near Boundary Road, and that near Stormont Avenue, traverse comparatively shallow peat deposits, up to about 10 or 15 ft in thickness. The Maillardville test section (Fig. 8) is typical of this kind of problem and treatment. A cost analysis showed that, at these locations, it was less expensive to treat the highway by preloading than to remove the peat. Preload was therefore added in the amounts necessary to give a load during surcharge equal to about 150 percent of the ultimate load. The peat consolidated quite rapidly and there was generally no stability problem because the underlying materials were reasonably firm. In several sections, high fills were built, two of these to heights over 20 ft, to accommodate grade separation structures. One of these was at King Edward Avenue in the Maillardville area and the other at First Avenue. At these locations, continuing consolidation of the underlying materials caused substantial long-term settlements and much of the rather long 2- to 3-year construction period was used in loading and surcharging these fills.

Deep Deposits: Preload with Lightweight Fill. —At two locations along the freeway, near Willingdon Avenue and near Deer Lake Interchange, very deep peat deposits were crossed which were underlain by very soft clay. The Deer Lake Interchange section, which was the most difficult and the most extensively instrumented, is shown in Figure 9. The stratigraphy in this figure shows that underlying the soft fibrous peat and the soft amorphous peat, which together extend to a depth of as much as 35 ft, there is a layer of extremely soft, sensitive, silty clay extending to a depth of as much as 75 ft. The moisture content of the fibrous peat is generally between 400 and 1, 200 percent and that of the amorphous peat between 200 and 600 percent. This corresponds to void ratios in the fibrous material of 8 to 17 and in the amorphous material of 3 to 8. The shear strength shows a slight trend to decrease with depth in the peat to a minimum in the order of 0. 05 tons per sq ft. The soft silty clay has a moisture content decreasing with depth from about 200 to about 40 percent. The shear strength increases with depth from a minimum of about 0. 04 tons per sq ft. The rate of increase of shear strength with effective pressure (C/P) is about 0. 4. The sensitivity is about 5 to 7.

Thus, there were two major problems facing the treatment of this difficult bog: First, calculations from laboratory data and full-scale test sections showed that 6 ft of granular material would produce a settlement of about 11 ft, 8 of it arising from the peat and 3 from the clay. Second, a load of even 6 ft of granular fill was enough to cause a base failure in the underlying clay. This situation, combined with a high water table and the proximity of Burnaby Lake, created a very difficult design problem. After a careful study of all possible alternatives, the solution adopted was the addition of a layer of lightweight fill before the application of the granular fill and surcharge. The lightweight fill was sawdust (Fig. 12) and its thickness varied from 3 to 12 ft. The function of the sawdust was to provide volume without extra weight (Fig. 13). To achieve a satisfactory detailed design, it was necessary to predict the amount of settlement with considerable accuracy so that the sawdust could be placed to accurate thickness and grade before being covered by granular material. The top sawdust surface must be so placed that after settlement it will be just below water level and thereby not be subject to decay. The surface of the sawdust must be low enough so that after settlement there is room for the design thickness of granular material, whereas it must be high enough to reduce the total load adequately. If too much granular material were used, a stability problem would result. To achieve the necessary fairly close tolerances, three test sections were constructed and carefully instrumented.

The soft clay underlying the peat created a much greater problem than did the peat itself. In fact, the authors have been led to the conclusion that, in many of the instances where peat has been given a reputation of being a very difficult material, it is

Figure 12. Sawdust placement, Trans-Canada Highway, Vancouver.

Figure 13. Checking quality of sawdust.

not the peat itself which is the troublemaker but rather it is the soft clay which very commonly underlies peat. Because the peat has such a low unit weight and is a very recent normally loaded deposit, any clay which underlies it, even though it may be at a depth of 30 or 40 ft, will probably be exceedingly weak. On this project, a great deal of study has been given to the soft clay and to its performance with and without sand drains. Although some very interesting findings were uncovered during the investigation with regard to the very soft clay, these results are considered to be beyond the scope of this paper, which concentrates on the treatment of the peat soils.

The general arrangement of the lightweight fill construction technique employed is shown in Figure 14. Inasmuch as it is rather unusual to use sawdust as a permanent material in a major highway, a few comments on its properties may be appropriate. From a study of sawdust durability, the authors came to the conclusion that the sawdust would be in danger of rotting if it were not continuously submerged. It was also found that sawdust above the water table is subject to a hazard of spontaneous combustion. A number of records were found of instances where spontaneous combustion in sawdust piles had occurred. The top surface of sawdust must therefore be kept at an elevation not higher than the ground-water table. In the Vancouver area, sawdust is available in large quantities. The contract price on this project for several hundred thousand cubic yards was about $0.70 per cu yd for supply and placement. The sawdust has performed very well as a construction material. It was found that the sawdust could be worked in any Vancouver weather. Neither the compactive effort nor the moisture content were critical. It was compacted solely by the passage of the trucks in a predetermined pattern. Under the preload, which was in the order of ½ ton per sq ft, the sawdust compressed to some 80 or 90 percent of its original thickness, but this consolidation was uniform and rapid. The trafficability of the sawdust during construction was excellent. There was no difficulty in driving over it even with standard passenger cars. It has acted as a much more satisfactory base for the compaction of granular material than did the peat. The sawdust has acted as a frictional material with an angle of internal friction $\varphi = 50^0$ as determined by laboratory direct shear tests. The required pavement thickness is considered to be at least 1 ft less over sawdust than directly over the peat.

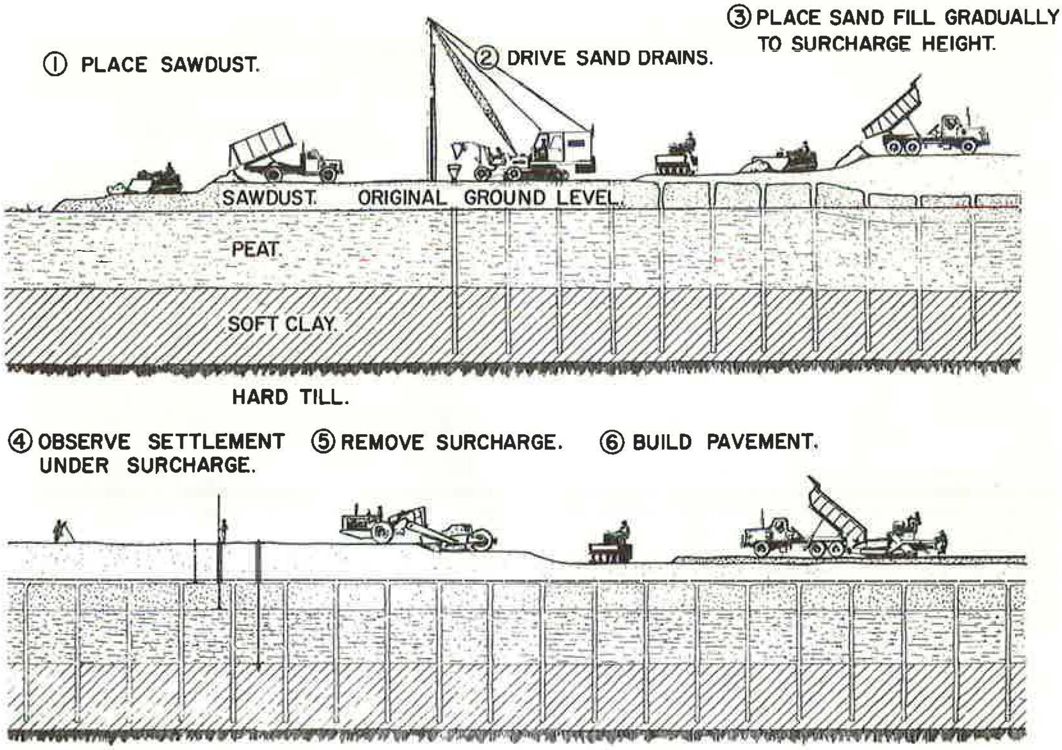


Figure 14. Construction technique using sawdust.

Unit Weight

The unit weight of peat is best determined on large chunk samples or samples obtained in large-diameter thin-walled samplers. The wet unit weight of pure peat has generally ranged from 55 to 75 pcf. Higher unit weight is associated with a higher inorganic content. Dry unit weights as low as 4 pcf are common for uncontaminated samples.

At a few locations, lightweight fill was required above the water table and in these locations lightweight concrete aggregate was used.

PEAT PROPERTIES

Many of the tests used to determine the physical characteristics of inorganic soils may also be used for peat. The most significant of the properties are unit weight, moisture content, air content, specific gravity, permeability, shear strength, and compressibility. Because of the high moisture content, and difficulty of obtaining and trimming samples, considerable care must be taken to obtain statistically significant results. Test procedures employed and the general range of test values obtained in British Columbia are summarized.

Moisture Content

To determine the moisture content, the samples are air dried at a constant temperature of 110 C until a constant weight is obtained. The test temperature and length of drying time are closely controlled.

Moisture contents of pure peats have generally ranged from about 500 to 1,500 percent with occasional values exceeding 2,000 percent. Values less than 500 percent generally indicate the presence of inorganic constituents. A manifold variation in moisture content, (i.e., from 100 to 400 percent or from 400 to 1,200 percent) may exist erratically within 1 ft. To reduce this scatter, moisture content samples should not be smaller than 10 cu in.

Moisture content is the least expensive and most used test. Many investigators have related it to such things as void ratio, specific gravity, coefficient of compressibility.

Void ratio is often computed from moisture content. The specific gravity must be determined or estimated (usually about 1.5 to 1.6 for pure peat). The sample is sometimes assumed 100 percent saturated but preferably a degree of saturation is known. It is desirable to determine gas content on representative samples so that this may be used in calculating void ratio. Otherwise, the fully saturated assumption may give a 10 percent error in void ratio. Void ratios are very high. For example, at a moisture content of 1,000 percent, the void ratio approximates 18.

Air Content

The air (or gas) content of peat is difficult to measure and no widely recognized method is yet available. In British Columbia, air contents have been estimated from data obtained during the consolidation test. Results give values of 7 to 10 percent. Promising performance has been obtained from a few tests using an air meter of the type commonly used for concrete testing.

The air content is of considerable theoretical and practical importance. All physical tests are affected by it and in the field, permeability, rate of consolidation, and measurement of pore pressures, are all believed to be substantially affected by the presence of air.

Specific Gravity

The specific gravity is affected principally by the presence of inorganic material. For pure peat the specific gravity ranges from about 1.5 to 1.6. The lower limit represents the average specific gravity of lignin and cellulose.

Accurate measurement of specific gravity is difficult. The most common method is to take a representative sample and fire it at 1,400 F for 3 hr and weigh the residue. The weights of soil solids and woody material are thereby determined and the specific gravity of the peat calculated assuming the specific gravity of the soil as 2.70 and that of the woody material as 1.50.

Permeability

The permeability of peat reduces considerably with increased load. The permeability of virgin peat tested in British Columbia generally ranges from 10^{-2} to 10^{-4} cm per sec. After settlement takes place under a load equivalent to only a few feet of fill, the permeability reduces to about 10^{-6} cm per sec, and under loads equivalent to 6 to 8 ft of fill "k" reduced to 10^{-8} to 10^{-9} cm per sec. Figure 15 shows peat permeabilities determined on the Burnaby Freeway project.

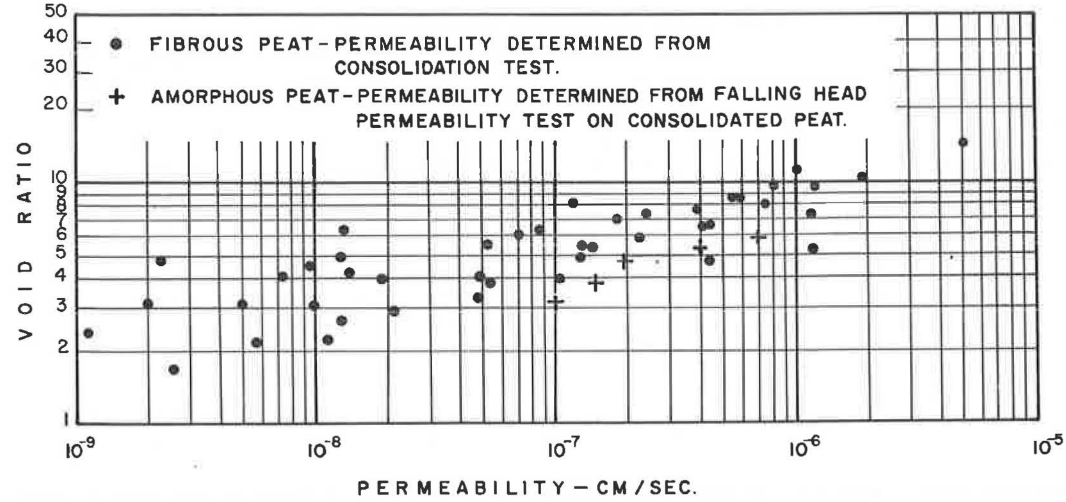


Figure 15. Void ratio vs permeability.

Considering the high void ratio of peat, greater permeability would be expected. The low permeability appears partly due to the manner in which peat retains moisture and partly due to the presence of gas. The low permeability explains the great difficulty encountered in attempting to drain peat.

Shear Strength

The common procedure for determining the shear strength of soil is to obtain undisturbed samples in the field and perform unconfined or triaxial compression tests in the laboratory. This procedure is not very rewarding with organic material. Not only is it difficult to obtain good samples in the field, but it is also difficult to trim samples in the laboratory and during testing the samples usually show great strains and distortions so that the results are difficult to interpret.

A second method of assessing the shear strength is to perform a stability analysis on a section where failure has occurred or can be induced by excavating a trench with a vertical face to such a depth that failure occurs. This method gives an average value of shear strength that is usually more accurate than laboratory tests. This method has been used with some success but the interpretation may still be difficult.

A third method of assessing shear strength involves measurement in situ using the vane shear apparatus. The validity of the vane test in peat might reasonably be questioned in the light of the high permeability which suggests that drainage may occur around the vane and influence the test, and in the light of the strong fibers and roots often encountered. Vane tests have been used extensively, however, and, in British Columbia, have been found to give quite satisfactory results.

In British Columbia, very few laboratory tests have been performed due to the difficulty of sampling and testing. The few results that have been obtained revealed an angle of internal friction φcq of about 25^0 and φ' of about 35^0. This relatively high value combined with the initial high permeability indicates that the strength of peat in practice does not usually create a stability problem providing reasonable rates of loading are employed.

The peat strength frequently shows an indefinite relationship with depth (Fig. 16). This is not surprising because the peat is normally loaded and has a very small submerged unit weight. The strength in the Sperling area (Fig. 17) shows a pronounced decrease with depth. This is due principally to the character of the peat which, at this location, becomes decidedly less fibrous and more amorphous with depth.

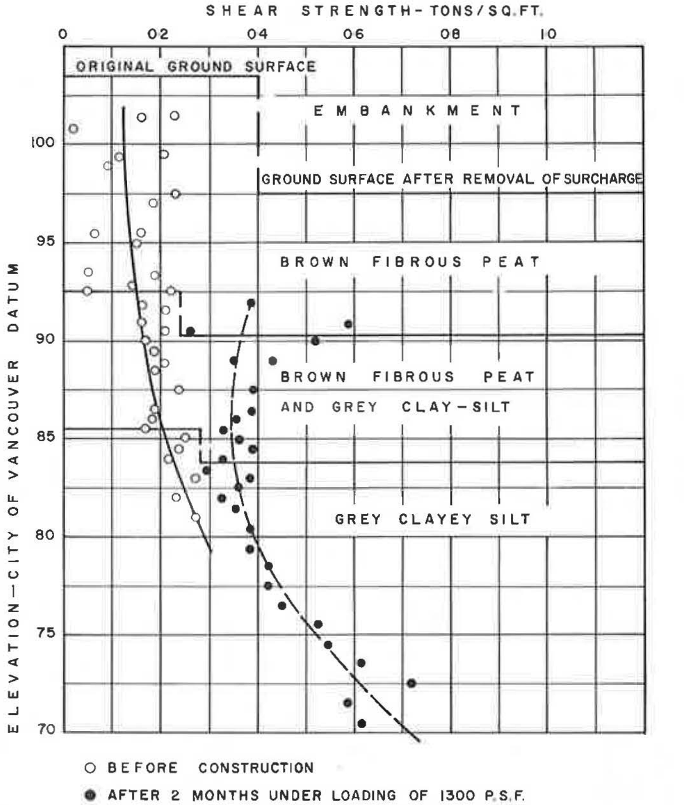


Figure 16. Vane shear strength, Maillardville test section.

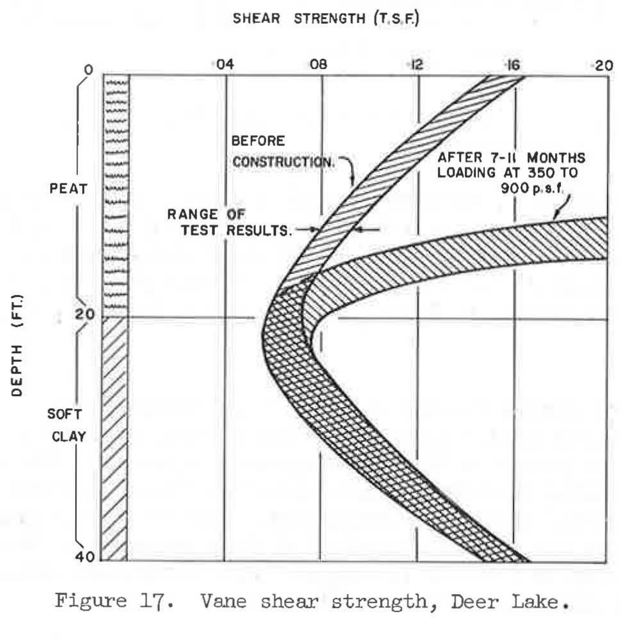
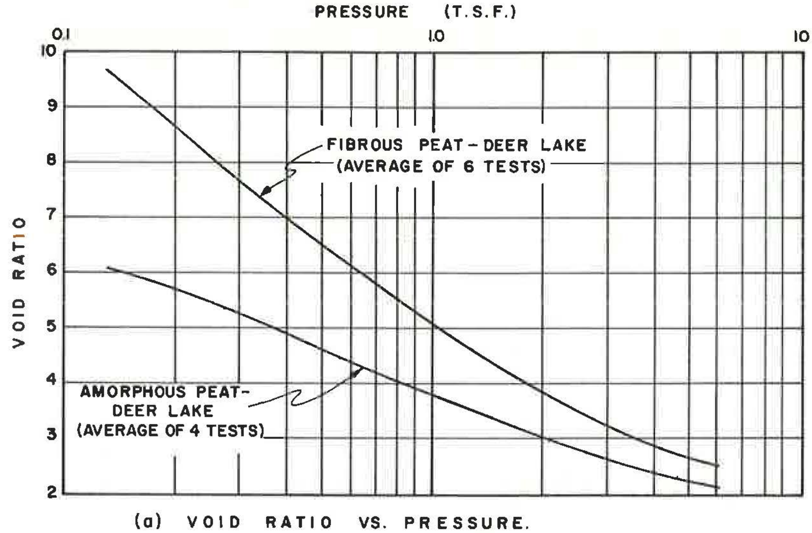


Figure 17. Vane shear strength, Deer Lake.



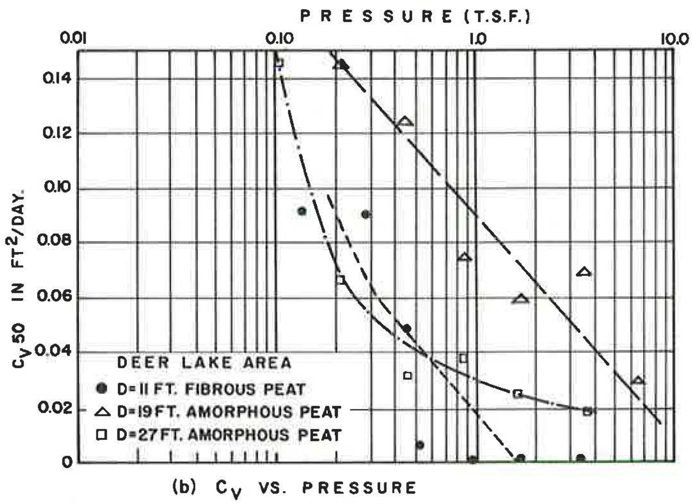


Figure 18. Consolidation test data, Deer Lake area.

Considerable increase in strength is noted in peats that have been loaded, as shown in Figures 16 and 17. At a test section on Lulu Island, original vane strengths averaged 200 psf and nine months after an 8-ft thickness of sand was placed (0. 4 tons per sq ft) the vane strength had increased to an average of 1, 300 psf. Numerous remolded tests were performed with the vane and indicated a sensitivity of 1. 2 to 2. 4.

Consolidation

The consolidation characteristics of the peat are considered in greater detail later. In general, two types of testing were used. First, there were the usual consolidation tests as for inorganic soil using 2 ½-in. diameter specimens. The results show a wide range (Fig. 18). During the test, C_V decreases by a factor between 5 and 100. This is caused largely by the change in permeability. The second test type frequently used is a single increment test or a test in which the load sequence on the sample is made identical with that expected for the prototype. This test has the advantage of requiring fewer assumptions for extrapolation to full size.

Correlation of Mechanical Properties

Numerous attempts have been made to correlate various properties of peat, particularly with moisture content. Some general correlations appear to exist (Fig. 19).

The relationship between specific gravity and moisture content arises from the fact that peat is a mixture of woody material with a specific gravity of about 1.5 and inorganic material with a specific gravity of about 2.7. At high moisture contents, the peat is all organic and the specific gravity is constant. At lower moisture contents, the specific gravity is higher. The moisture content of specific gravity relationship shown in Figure 19 only holds for virgin, normally loaded peat. If the peat is dried or compressed, the relationship changes drastically.

The relationship between moisture content and void ratio is a mathematical one involving specific gravity and gas content. For virgin, normally loaded peat an almost linear relationship has been observed (Fig. 19).

The relationship between void ratio and consolidation properties which have been observed is shown in Figure 19. When working in the same area, they can be used for preliminary estimates, but for any important or final work, some laboratory testing of consolidation properties is desirable.

SETTLEMENT

One of the major practical problems in building over peat is to predict the magnitude and rate of settlement. The authors' experience to date in British Columbia permits the assemblage of data correlating laboratory and field consolidation. These data are summarized in Tables 2 and 3.

Predicting Magnitude of Settlement

Column 15 shows that the laboratory tests give a reliable estimate of the magnitude of field settlement due to consolidation. From this, it may also be deduced that field settlement is largely due to consolidation. This deduction is confirmed by field observation providing the shear strength is not approached or exceeded. At a number of locations, toe stakes and tilt meters have been used to measure horizontal displacement. During construction, horizontal movements are usually considerable (i.e., in the order of 1 to 3 ft), but these movements account for less than 10 percent of the settlement. Except in cases of serious instability when corrective measures are required, the horizontal movements decrease rapidly as consolidation takes place and seem to be of no concern after surcharge is removed. Soft clay under the peat does, of course, introduce complications.

Both laboratory and field time curves usually show the characteristic S shape on semilog paper as for inorganic soils. With laboratory tests, it is sometimes necessary to take readings at very early times (even to the extent of using a motion picture camera) in order to get the early part of the curve. In the field, the early part is often observed through a complicated loading schedule. It is usually not difficult, however, to establish 100 percent primary consolidation. The terms "primary" and "secondary" consolidation are used not to describe any physical phenomenon in the soil but purely to refer to two empirical parts of the consolidation time curve. The primary and secondary parts are separated by a characteristic concave curve when the consolidation time curve is plotted on semilog paper.

Predicting Rate of Settlement

In the light of the great variations in C_V (Fig. 18), it is not appropriate to apply the Terzaghi theory of consolidation. Nevertheless, one might expect the time to 100 percent primary to be proportional to the square of the thickness because this relationship is derived directly from Darcy's Law. From Table 2, Column 9, however, it appears that the exponential "i" is, on the average, closer to the power $1\frac{1}{2}$ than to the power 2. The following reasons are suggested for this deviation from theory:

1. Horizontal drainage in the prototype. Test section A which was 100 ft square overlying 30 ft of peat settled much more rapidly than the actual road fill which was a strip 200 ft wide.

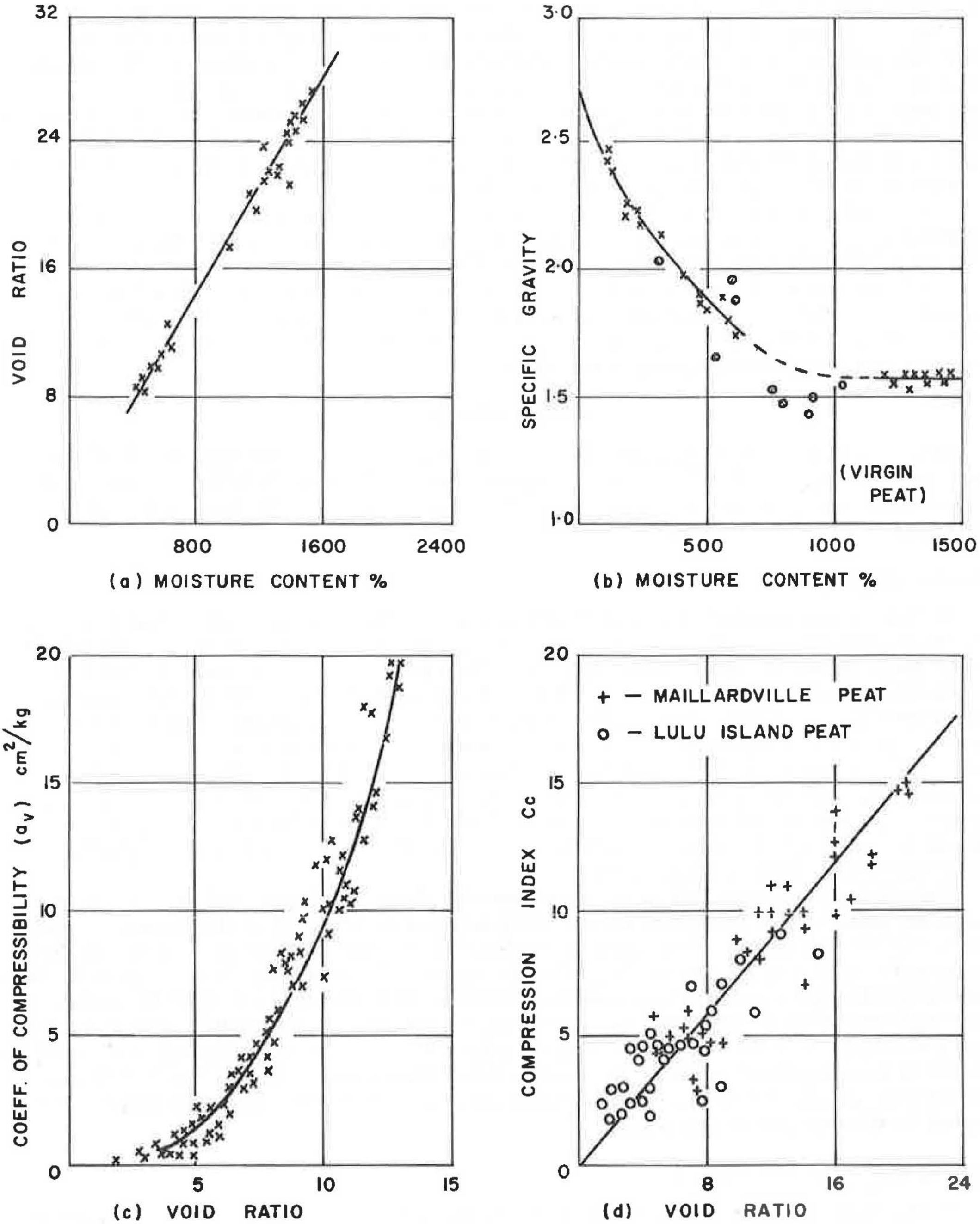


Figure 19. Correlation of various physical characteristics of peat tested in British Columbia.

2. Higher horizontal permeability. No concrete evidence of this is available to the authors from experience with peat, but it is frequently a very significant factor for inorganic soils.

3. If there is any physical significance to the 100 percent primary point on the consolidation curve this significance is obscure and thus the theoretical basis for the method of analysis is weak.

The wide difference in the literature on the value "i" is notable. Hanrahan (2, 3) gives data to support a value of 2. Lake (4) purports to show that the factor is 1. The authors find that 1. 5 can be used with sufficient accuracy for practical purposes in British Columbia.

Residual and Rebound Characteristics

Because in most instances primary consolidation in the field in peat is completed within a few weeks or months, it is more important than with clay to estimate the rate and magnitude of the secondary compression. The secondary curve is usually a straight line on a logarithmic plot.

If the coefficient of secondary compression C_{sec} is defined as the amount of compression per unit thickness of soil occurring during one cycle, the secondary settlement can then be calculated from

$$S_{sec} = C_{sec} H \log \frac{t_2}{t_1} \qquad (1)$$

in which

S_{sec}	=	magnitude of secondary settlement from time t_1 to time t_2;
C_{sec}	=	coefficient of secondary compression;
H	=	thickness of layer at time t_1;
t_1	=	time for 100 percent primary consolidation; and
t_1 to t_2	=	time over which it is desired to calculate the secondary settlement.

Values observed for C_{sec} are given in Columns 10 and 11 of Tables 2 and 3.

Table 2 shows that the rate of secondary consolidation C_{sec} is usually much greater, by a factor up to 5, in the field than in the laboratory. The field factor is also observed to be quite variable. This may be the explanation, at least in part, for the great unevenness that develops on roads over peat that has not been preloaded. Field values of C_{sec} have been observed to range between 2 and 16 percent. The value of this coefficient has been observed to depend, to a substantial extent, on the load history of the deposit. Preloading is effective in reducing C_{sec}. This factor appears also to be influenced by the magnitude of the load. Table 3 shows C_{sec} to decrease by a factor of 7 with a load increase of a factor of 4. This trend has been observed in the experience of the authors for single increment tests but not so markedly for standard consolidation tests. The value that is tolerable depends on the thickness of the peat and the highway design.

The secondary settlement as just analyzed assumes no shear deformation due to overloading. The occurrence of shear deformation in the field can be determined by installing and observing lateral movement and elevation hubs near the toe of the fill. It is difficult to obtain significant measures of the magnitude of movement in this way. Slope indicator installations have proved the best for giving a reliable measure of horizontal movement but the amount of movement that can be tolerated by the equipment is a severe limitation.

TABLE 2

COMPARISON OF FIELD AND LABORATORY PEAT CONSOLIDATION TESTS

Test Section	Type of Test: Laboratory: No.[a]	Type of Test: Laboratory: Description	Type of Test: Field	Original Thickness: Lab,[b] H_l (in.)	Original Thickness: Field,[c] H_f (in.)	Original Thickness: Ratio, H_f/H_l	Time to 100% Primary Consolidation: Lab, T_l (min)	Time to 100% Primary Consolidation: Field, T_f (min × 10^4)	Time to 100% Primary Consolidation: Ratio T_f/T_l	i as Parameter[d]	$C_{sec} = \frac{H_2 - H_1}{H_1 \log \frac{T_2}{T_1}} \times 100$: Lab	C_{sec}: Field	C_{sec}: Ratio, Field/Lab	Magnitude 100% Primary: Orig. Height (%): Lab	Magnitude 100% Primary: Orig. Height (%): Field	Magnitude 100% Primary: Ratio, Field/Lab	Consolidation Load
(1)		(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
A	2	H122A, S_1, W = 825 S_5, W = 1,245	90' × 110' at Deer Lake interch.; no sand drains; fibrous peat	1.0	192	384	62	1.4	226	0.9	3.0	4.6	1.5	22	32	1.45	0.24
	1	H122A, S_8, W = 564 Avg. W = 430	Amorphous peat	1.0	168	336	16	3.6	2,250	1.3	1.9	4.7	2.5	22	16	0.73	0.24
B	2	H461, S_2, W = 550 S_9, W = 700	100' × 150', Sta. 370 + 00; sand drains at 7½' and 15'; fibrous peat	0.75	168	450	17	13.0	7,600	1.5	2.5	2.1	0.83	31	32	1.04	0.4
	3	H461, S_{18}, W = 465 S_{23}, W = 460 S_{30}, W = 257	Amorphous peat	0.75	152	406	24	25.0	10,400	1.5	2.2	5.0	2.3	24	22	0.92	0.4
C	5	H461, S_2, S_9, S_{18}, S_{23} and S_{30}	120' × 200', Sta. 372 + 00; sand drains at 15' c.c.; fibrous and amorphous peat	0.75	372	990	20	47.0	23,500	1.5	2.3	9.4	4.1	26	22	0.85	0.35
Willingdon highway	2	H443, S_{2B}, W = 1,130 H444, S_{2C}, W = 687	170' stripload, Sta. 237 + 00; sand drains; fibrous peat	0.75	138	368	25	17.3	6,900	1.5	3.5	16.2	4.6	33	25	0.76	0.4

[a] Of standard consolidation tests.
[b] D. D.
[c] S. D.
[d] $\ln (H_f/H_l)^i = T_f/T_l$.

Rebound can be quite a significant matter in the removal of surcharge (Fig. 20). At some locations in the Burnaby Freeway rebounds of over 1 ft have been observed. In both the field and the laboratory rebound is found to be much greater if over 80 percent of the applied load is removed. This became apparent on the Burnaby Freeway at locations where all granular fill had to be removed to adjust the sawdust thickness. Field rebound has generally been found to be in the order of double laboratory rebound. There are a number of factors at work here:

1. The most obvious is elastic rebound which would not be observed in the laboratory consolidation test because of the lateral confinement but which may be a factor in the field. Rebound movements, however, have generally been too slow for elastic phenomena which must be rapid. There appears in the field to be a substantial part of the rebound occurring at a rate that would indicate a fast consolidation type of movement and another part that would be as expected for a normal consolidation movement.

2. It is observed in the field that, usually where the load removed is less than 80 percent of that added, the rebound is about 5 percent of the total settlement. This compares with about 2 percent in the laboratory. On very shallow deposits (i.e., less than 10 ft), however, the magnitude of the field rebound for less than 80 percent load removal tends to remain in the order of 0.2 to 0.3 ft. The explanation would appear to be in the few feet of woody surface mat which may well be expected to rebound a rather uniform amount at what might be a rapid consolidation-type time rate.

3. Gas expanding and coming out of solution probably is a factor although it is a difficult one to evaluate. One would expect it to be rapid like an elastic movement and to be observed in both field and laboratory. This gas expansion is provisionally considered to be a minor factor.

It seems that none of the preceding gives a satisfactory explanation for the large rebound on full removal of load. This is observed in the field to

TABLE 3

COMPARISON OF TWO THICKNESSES OF LABORATORY PEAT COMPACTION TESTS

Type of Test		Original Thickness			Time to 100% Primary Consolidation			i^b	C_{sec}			Magnitude 100% Primary			Consolidation Load
												Orig. Height (%)		Ratio L/S	
Smaller Sample	Larger Sample	Small[a] (in.)	Large[a] (in.)	Ratio, L/S	Small (min)	Large (min)	Ratio, L/S		Small	Large	Ratio, L/S	Small	Large		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Single increment load, H248, S_{1A}, W = 750	Single increment load, H249, S_{1D}, W = 523	1.0	1.75	1.75	60	125	2.1	1.3	2.4	1.1	0.5	33	21	0.64	0.5
Single increment load, H248, S_{1B}, W = 774	H248, S_{1D}, W = 464	1.0	1.75	1.75	40	170	4.2	2.5	0.3	0.2	0.7	38	40	1.05	2.0

[a] D.D.
[b] As in Table 2.

be 15 to 35 percent of the settlement. From a practical point of view, it is concluded that full unloading should be avoided wherever possible, and that, if it is necessary, some load should be reapplied as quickly as possible.

Surcharging

Preconsolidation is now a standard procedure for constructing highways over peat in British Columbia. For each application careful preliminary studies are usually required to ensure success. Such a study discloses the amount and timing of surcharge necessary to increase the rate and magnitude of settlement to such a point that the settlement expected under the final load in 25 years can be obtained during the construction phase.

A design procedure has been proposed by the authors (5) which is shown in Figure 21. Curve ABC is the calculated 25-year field load-settlement curve. Curve DEF is a similar calculated field load-settlement curve for the allowable construction period; in this case, three months. Curves ABC and DEF are constructed from laboratory data using the consolidation principles described under consolidation and the three preceding sections.

Point D indicates that if a 0.3-ton load is placed it will settle 2.5 ft in three months. Curve GDB represents the minimum load that must be added to maintain the top of the fill at the required finished roadway elevation. The slope and shape of this curve are affected by the location of the water table and the unit weight of the fill materials. Point B represents the ultimate condition that must be achieved and the horizontal projection of B to line DEF, point E, gives the load required to obtain the 25-year settlement in three months. Where conditions vary, new curves must be constructed.

Instrumentation is necessary so that during construction the actual performance may be compared with that predicted. It is important that contractual arrangements be such that changes in the amount and duration of surcharge may be readily made.

The field data should be compared with the laboratory data and decisions made while construction is in progress as to magnitude of surcharge and time of surcharge removal. Figure 22 is an example of a comparison between field and lab observations. Figure 22a shows a time settlement curve for a fill and surcharge placed in three stages with each stage left on long enough to extrapolate the 25-year settlement under each loading. The amount of settlement under each load is plotted as points X, Y, and Z in Figure 22b, thereby giving a field 25-year load-settlement curve which may be compared with the 25-year load-settlement curve calculated from laboratory tests as shown in Figure 21. In Figure 22b, line GKD defines the load which must be employed to maintain the required finished roadway grade. In this example, to maintain a grade line 5 ft above original ground a load of 0.5 tons per sq ft is required (point K). The load-settlement curve from laboratory data is also shown. In this particular example, it is seen that the surcharge added at 60 days was somewhat excessive, unless only about one month were available for surcharging. In this particular instance, however, a one-month surcharge period is not practical, because stability considerations required a greater time.

It is very important that fills be constructed in definite lifts which are placed very rapidly. Three lifts of 3 ft each, placed at one-month intervals, and each placed rapidly, are much preferable to twelve 9-in. lifts at one-week intervals. Control and analysis can only be satisfactorily carried out for substantial lifts placed rapidly. Stability requirements must, of course, also be satisfied.

EMBANKMENT STABILITY

The properties of peat, in combination with the very soft clay that frequently underlies it, create embankment and slope stability problems that require special attention.

General Stability

In the authors' experience, soft clay under the peat is always present when embankment stability is a problem. It seems that the geological history of the develop-

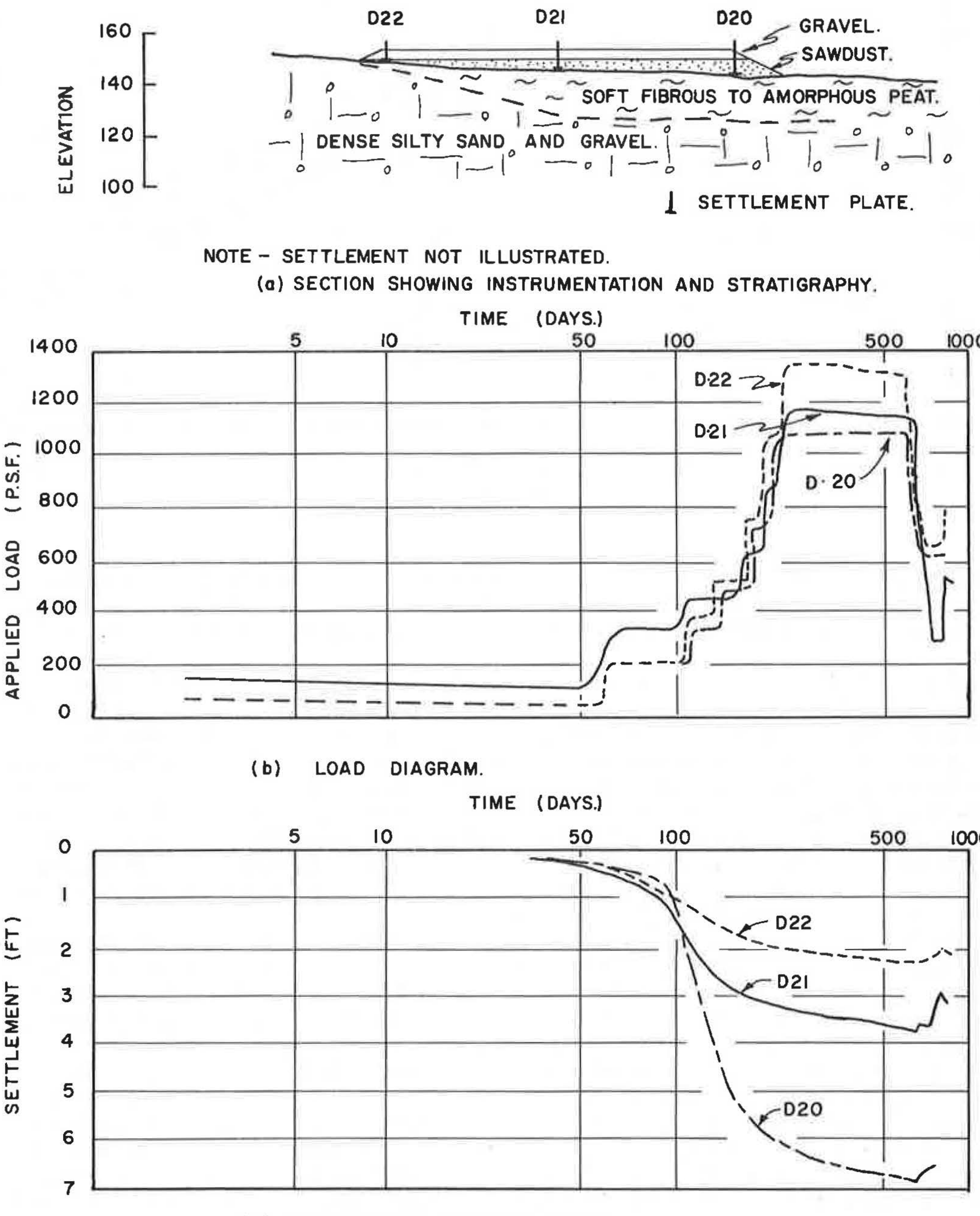


Figure 20. Sprott Street area.

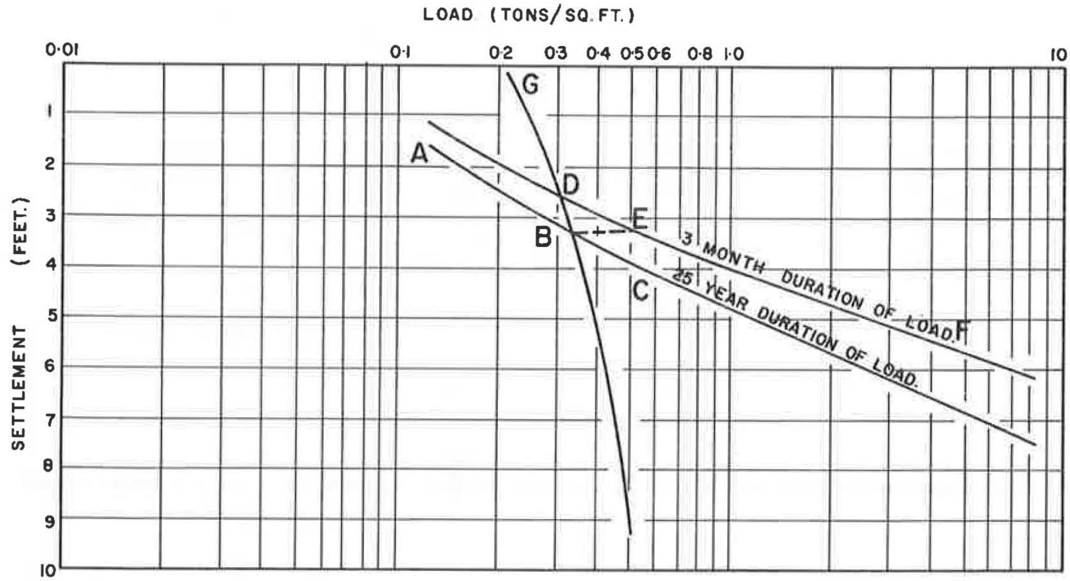


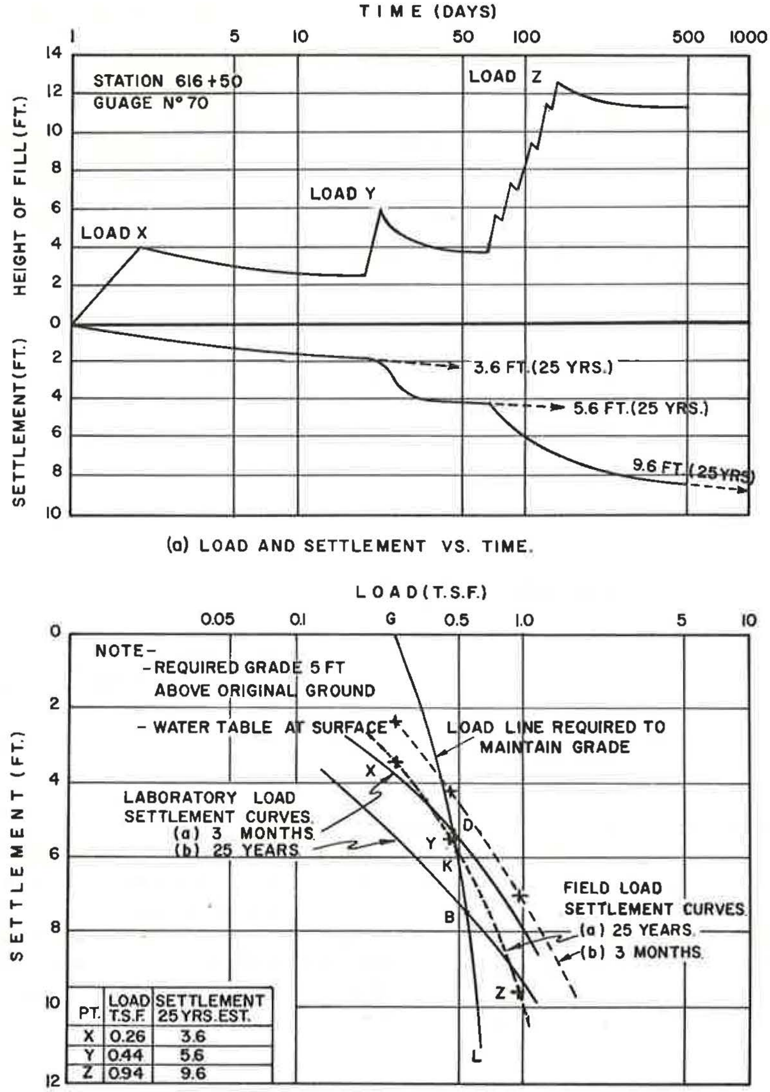
Figure 21. Graphical determination of surcharge load required for peat treatment by preconsolidation.

ment of peat frequently involves normally loaded soft clays and silts just below the peat. Even at great depth, these may be extremely soft because the unit weight of peat is so low. Beneath 40 ft of peat, the effective pressure on the clay may be only 200 psf.

Figure 17 shows a typical strength depth profile. The data for this particular figure come from field vane tests in the Deer Lake area. The field vane test results are in reasonable agreement with laboratory tests, and many vane tests have been taken so that for any critical section the shear strength is known. Based on this strength profile and pertinent data concerning unit weights, the stability of embankment can quite readily be analyzed using standard computation procedures. A block slide moving on the boundary between the peat and clay is found to be the most critical condition. Because laboratory tests show the expected gain-in-strength characteristics of the peat and clay, it is possible to develop a theoretical step program of loading, resting, and reloading until the desired height is achieved.

The first approach in a stability analysis is to take these measured strengths and the measured unit weights, and perform the usual circular arc and block-slide type of analysis. The calculation heights are frequently found to show the desired embankment heights to be unstable, but because both the peat and the clay increase in strength under load, it is possible to calculate a step loading program to satisfy the theoretical stability requirements. This is only part of the story, however, as demonstrated by the Willingdon Avenue slip which occurred in December 1960 (Fig. 22). The actual reasons for this slip include several factors beyond the scope of this paper, such as remolding of the clay by sand drains. Nevertheless, the slip also illustrated a fundamental element in such problems. This section is indicated to be stable by a stability analysis which does not allow for the relative stress-strain properties of the peat and the clay.

The relative stress-strain properties of the peat and the clay are most significant. Figure 23 shows a comparison of the stress-strain properties of the peat and the clay at Willingdon. At a 2 percent strain, the clay may already have passed its maximum stress, whereas the peat is stressed to only 25 percent of its maximum. At a 5 percent strain, the clay has dropped to its remolded strength whereas the peat is only at about 40 percent of its maximum. Therefore, the full strength of the peat and the clay



(a) LOAD AND SETTLEMENT VS. TIME.

(b) CHECK OF SURCHARGE EFFICIENCY.

Figure 22. Comparison of laboratory and field settlement estimates, Fraser Mills area.

cannot be mobilized at the same time. Thus, considering the block-sliding analysis, the resisting pressure from the peat will be below full passive pressure and the driving pressure will be above full active until after the shear resistance of the clay has been reduced to its remolded strength. The big problem in stability analysis is thus the selection of the correct strength to use in the analysis. By a careful selection, first, of the correct peat strength curve as shown in Figure 24 and, then, of the crucial strain with the corresponding strength in peat and clay, it was not difficult to obtain an analysis that fully explained the Willingdon slip and other slips in the Burnaby Freeway.

Through careful application of these design analyses and close field control, embankments have been successfully built over the soft peat on this project to granular fill heights of as much as 25 ft. Where the soft clay is present, the maximum height is much less.

Embankment stability on peat soils is not considered at present to be a problem that can be fully resolved in the laboratory or the office. At present, full-scale field

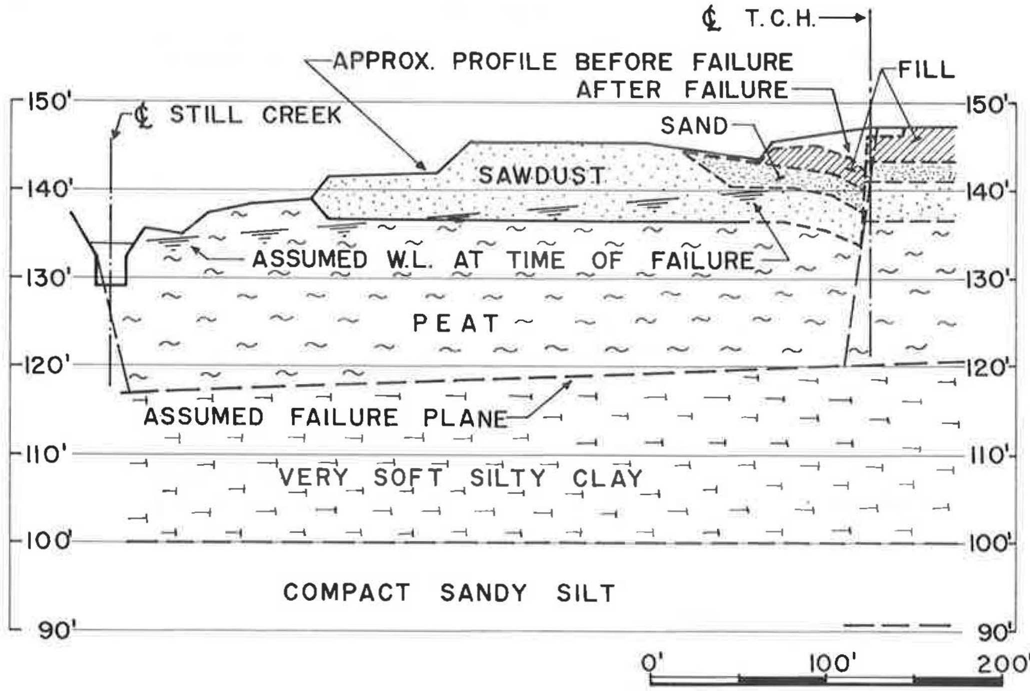


Figure 23. Willingdon failure.

tests and careful field instrumentation and control of construction are considered essential. These are the procedures that have been followed in achieving the stable 25-ft embankments. It is not, of course, possible to build high embankments without experiencing very substantial movements. It is difficult to extrapolate laboratory strain measurements to the field but, in general, horizontal movements in the order of 5 percent of the peat thickness have been experienced in British Columbia. To achieve optimum economy it is usually desirable to design to such a factor of safety that some slips are experienced on a major highway project over peat.

Bridge Abutments

At bridge abutments (on piles or on fill), it is important to analyze the several components of the fill movement carefully. Provided such an analysis is carefully done and provision for movements is built into the structure, movements of several feet after construction have been accommodated without difficulty. In such continuing movement, the underlying clay is, of course, a more significant factor than the peat. The most troublesome movement in some structures has been the spreading apart of the abutments due to differential settlement in the approved fill. The usual practice in modern highway bridges is to build small bridge seats which are quite unsatisfactory for such conditions. It is interesting to compare this practice with early railway bridge building practice where every bridge seat was designed for the convenient accommodation of movements. In modern highways through swamp ground, great cost savings can be achieved by building on top of the soft materials, but it is essential that both embankments and structures be designed to accommodate the movement and to work together in so doing.

PAVEMENT DESIGN

The majority of asphaltic concrete surfaces placed on roads "floated" over muskeg or constructed on corduory have been subject to distortion, differential settlement, cracking, and breakup. When the use of pre-loading was contemplated, detailed consideration was given to the problem of providing a pavement structure with long-term structural and service adequacy. The methods of pavement design were reviewed and four of these appeared to offer promise: plate bearing, Benkelman beam deflection, shear strength, and elastic theory.

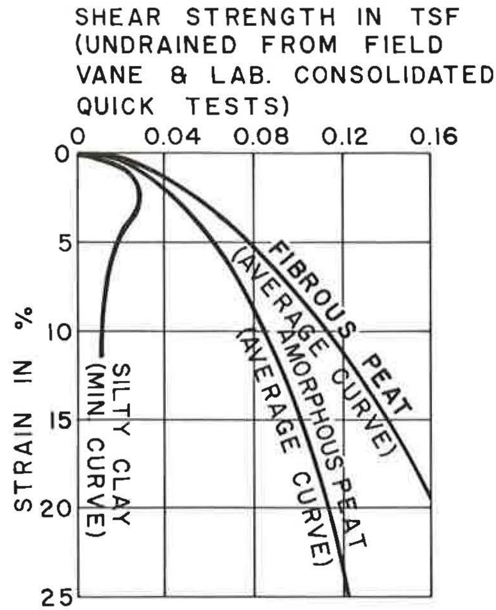


Figure 24. Stress-strain curves of peat and clay.

Plate Bearing

Several plate bearing tests were attempted on the virgin peat but proved difficult to interpret due to the extremely high strain. On peats that had been pre-loaded, however, useful results were obtained. Tests at four separate locations yielded pavement thickness requirements ranging from 42 to 47 in. for load conditions representing repeated applications of 18,000-lb single-axle loads.

Benkelman Beam Deflection

A field study comprising the determination of the Benkelman beam deflection, thickness of existing pavement, and visual inspection of the surface condition was performed on numerous highways constructed on peat. All roads studied carried at least moderate traffic and had been in service at least 8 years. Figure 25 shows the results of the survey.

The road surface was in good condition at over 95 percent of the locations where the pavement thickness exceeded about 45 to 48 in. and the deflection was less than 0.035 in. This suggests that a pavement thickness of 48 in. is adequate on preconsolidated peat.

Shear Strength

The shear strength method developed in England (6) is based on the theory that sufficient pavement thickness is required to reduce the stress in the peat below the shear strength. Based on field vane shear tests, a depth of pavement of 22 in. was indicated. This is too little judging by performance in the field (Fig. 26). The discrepancy may be due to the vane shear test not being reliable in peat, peat having a moderate angle of shearing resistance, or radius of curvature and not shear strength being the limiting design factor.

Elastic Theory

A radius of curvature criterion developed from elastic theory (7, 8) can be used to evaluate pavement thickness provided the modulus of elasticity of the subsoil layers can be estimated. Representative values as determined in British Columbia by the plate bearing test are (a) asphaltic concrete, 2,000 to 200,000 tons per sq ft; base gravel, 1,000 to 1,500 tons per sq ft; silty sand, 200 to 300 tons per sq ft, and peat, 10 to 30 tons per sq ft. The modulus for asphaltic concrete varies considerably with temperature. Using a lower value representative of warm weather conditions and applying the radius of curvature design method, a 48-in. pavement thickness is indicated.

Assessment of the preceding design methods suggests that 42 to 48 in. of pavement is adequate for preconsolidated peat. In British Columbia the present design require-

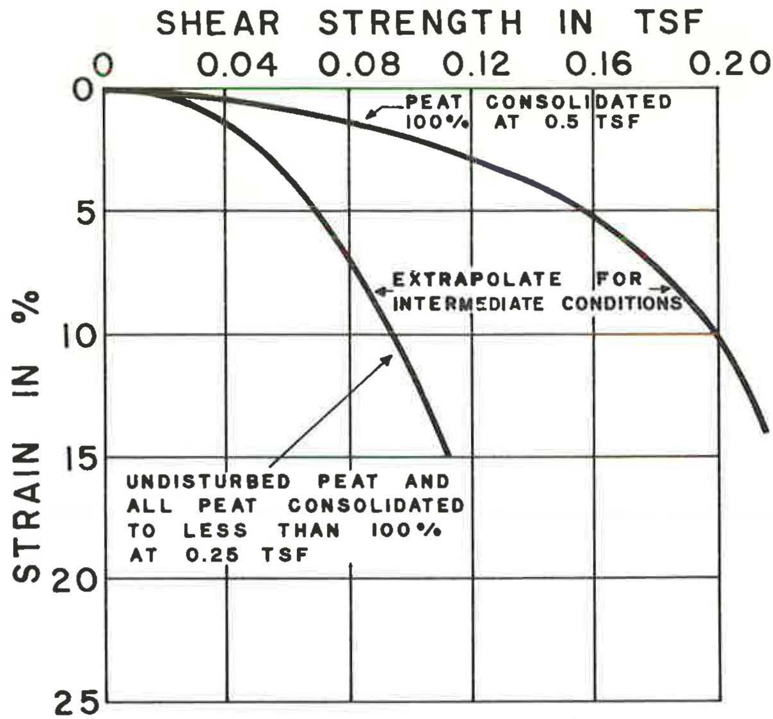


Figure 25. Peat strength properties for stability computations.

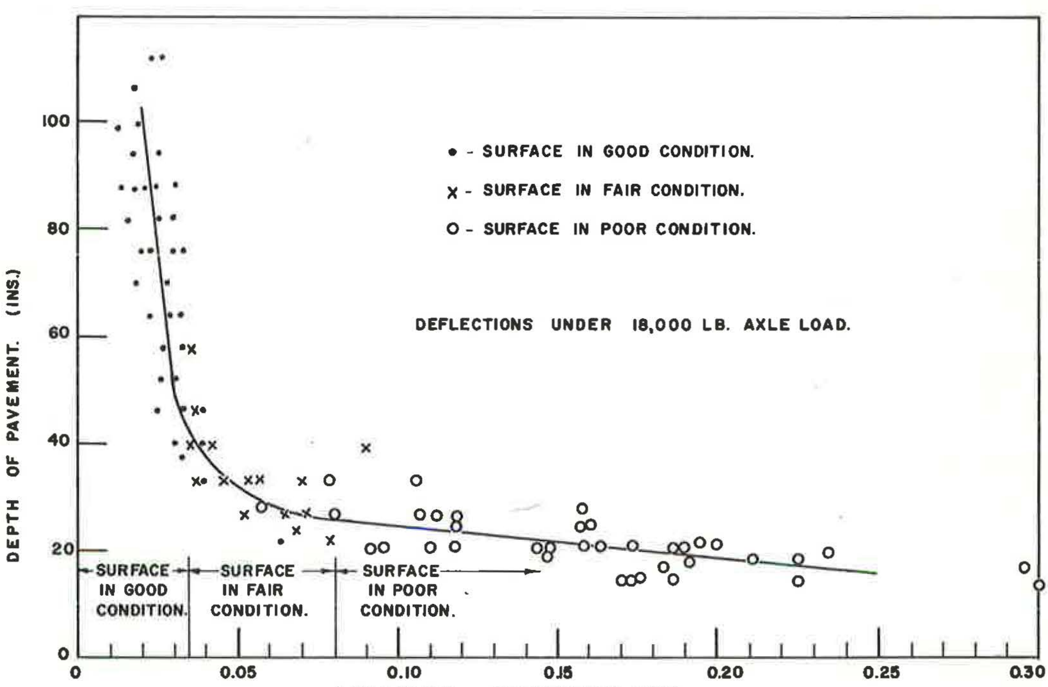


Figure 26. Depth of pavement on peat vs pavement deflection.

ments are 48 in. for secondary and primary highways and 54 to 60 in. on freeways. These values are based on data representing peats encountered in the lower British Columbia mainland. Though they may be valid for preconsolidated peats generally, it is suggested that they not be used in other areas without verification. Where sawdust is used, the top foot of it is considered to be part of the pavement.

SUMMARY AND CONCLUSIONS

1. Peat is sometimes referred to as muskeg, organic terrain, swamp or bog land. It is composed principally of dead vegetal matter in various stages of decay.
2. Peat, of depths up to 40 ft, covers over 50 sq mi of the lower mainland area of British Columbia, which is now developing rapidly.
3. Early highways were constructed around the peat where possible. If a peat bog had to be crossed, the road was either built on corduory or "floated" on the surface. Neither method has proven successful under heavy traffic conditions. Complete removal of the peat by excavation or displacement has become common for major highway across peat, but this becomes very expensive for depths over 8 to 10 ft.
4. Construction using preconsolidation has been used more recently and very successfully on numerous sections of major highways in British Columbia. The technique has proven less expensive than excavation or displacement for depths of peat exceeding about 5 ft.
5. The peats in the lower mainland region of British Columbia fall into two principal categories: fibrous and amorphous. Some correlation exists between moisture content and the other engineering properties.
6. Peat time-settlement curves commonly show a shape on which 100 percent primary consolidation can be identified.
7. Comparison of laboratory and field data suggests that the magnitude of the apparent primary settlement varies directly as the thickness of the peat and that the time rate of apparent primary settlement varies as the thickness to the power 1.5 (in comparison with the power 2 for the classical theory).
8. The apparent secondary settlement follows a straight line on a semilog plot of time-settlement. The slope of this line, the coefficient of secondary consolidation, is usually much greater in the field than in the laboratory and may decrease substantially with increased load.
9. Rebound may be a significant factor when peat is unloaded, particularly if the load is fully removed.
10. Settlement in peat areas may be greater than the depth of granular fill applied. Sawdust is being used as a weightless spacer to overcome this problem on several sections of freeway over peat near Vancouver.
11. Fill stability is generally not a problem in peat areas because the rapid settlement is associated with a rapid increase in shear strength. Where major stability problems do exist, they are usually associated with soft clay underlying the peat.
12. Although much progress has been made in measuring strength and consolidation properties of peat in the laboratory, it is still considered necessary to employ full-scale test sections on major projects in unfamiliar areas.
13. Benkelman beam deflection tests, plate bearing tests, and elastic theory indicate a pavement thickness of 42 to 48 in. for primary highways on preconsolidated peat in the British Columbia lower mainland. A thickness of 54 to 60 in. is recommended for freeways with a 1-ft reduction being allowed if a sawdust blanket is used.

ACKNOWLEDGMENTS

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Res. Record No. 7

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Discussion

I. C. MACFARLANE, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada—The authors are to be congratulated on their excellent comprehensive study of an important project, which has made a real contribution to peat technology. The British Columbia approach to highway construction over organic terrain has displayed both orginality and imagination and is now being followed, with some success, in other parts of Canada.

The authors quite rightly point out that confusion in terminology has arisen (indeed, it has been there from the beginning) and any effort to clarify the situation is to be commended. The National Research Council was the first organization in Canada to undertake scientific research in muskeg and from the earliest stages of this program, it has been pointed out that muskeg (or organic terrain) is a terrain condition and that peat is a material (12, 13). The two terms are not—nor has it ever been implied—any more interchangeable than are, say, clay plain and clay. Therefore, the writer would most certainly agree that it is inappropriate to refer to a peat sample in the laboratory as a sample of muskeg or organic terrain.

The writer is particularly interested in some of the physical characteristics of Vancouver peats inasmuch as they agree in general with his own results for peats of a more fibrous nature obtained elsewhere in Canada. In particular, the relationship between specific gravity and water content of peats from northern Ontario, as shown by Figure 27, exhibit the same shape as is observed in Figure 19b. Specific gravity of soil solids was determined by pulverizing the oven-dried peat and following with only minor modifications the usual specific gravity determination procedure for inorganic soils, taking special care that all the air was excluded from the samples. Specific gravity of pure peats was found to be between 1.5 and 1.7 regardless of the degree of humification. The lower limit of pure peats would appear to be those exhibiting a specific gravity of about 1.65, a moisture content of the order of 500 percent and an organic content of about 80 percent.

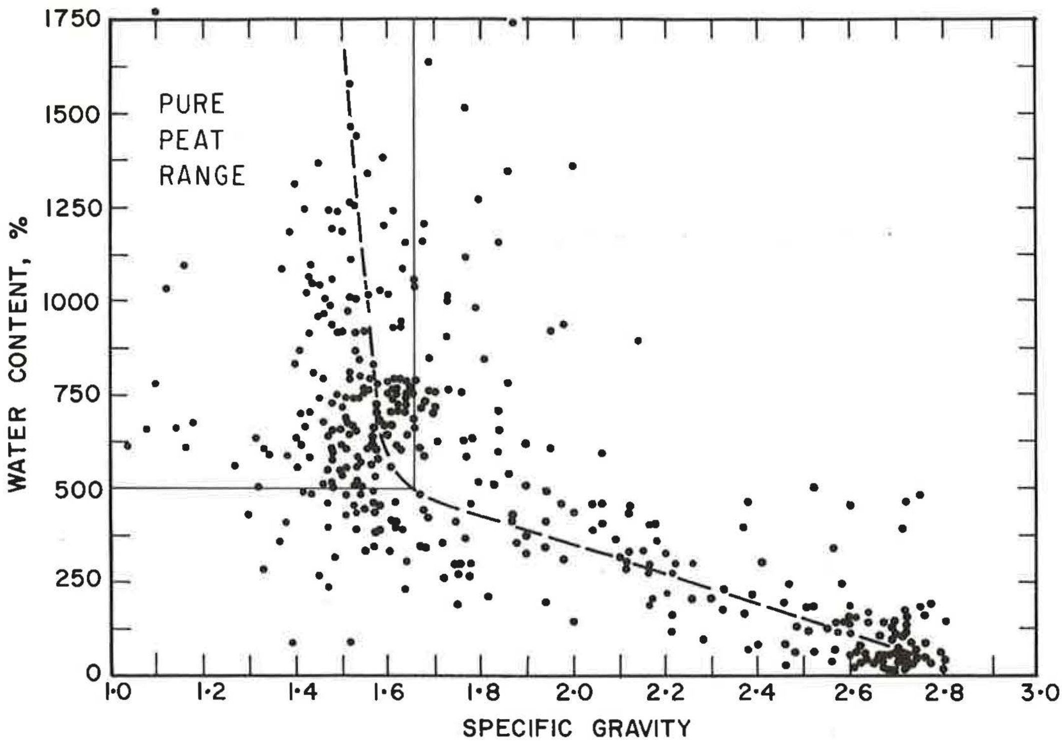

Figure 27. Water content vs specific gravity.

With regard to the water content of peats, concern has often been expressed about the effect of the drying temperatures normally used for water content determinations of inorganic soils. An extensive series of water content tests has been carried out at the Division of Building Research on both non-woody fine-fibrous and amorphous-granular peats. Results have indicated that drying temperatures of 105 to 110 C for water content determinations are too high for peat, and charring occurs with consequent errors in the results. The report of this investigation has not yet been issued but preliminary indications are that a drying temperature of about 85 C is more appropriate for peats than is the normal 110 C temperature used for inorganic soils.

Efforts have been and are being made to correlate the easily determined physical characteristics of peat (such as water content, specific gravity, and organic content) with shear and consolidation characteristics. Some small success is apparent for peats below the pure peat range; i. e., peat with some degree of mineral soil contamination. In the pure peat range, however, these correlations have not been as readily evident and much more work needs to be done.

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