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Aggregates, Marsh Deposits, and Asphaltic Membranes

4 Reports

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Contents

Highway Design and Construction Over Peat Deposits in Lower British Columbia

NORMAN D. LEA, N. D. Lea and Associates, Vancouver, B. C.; and
C. O. BRAWNER, Materials Engineer, British Columbia Department of Highways, Victoria, B. C.

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

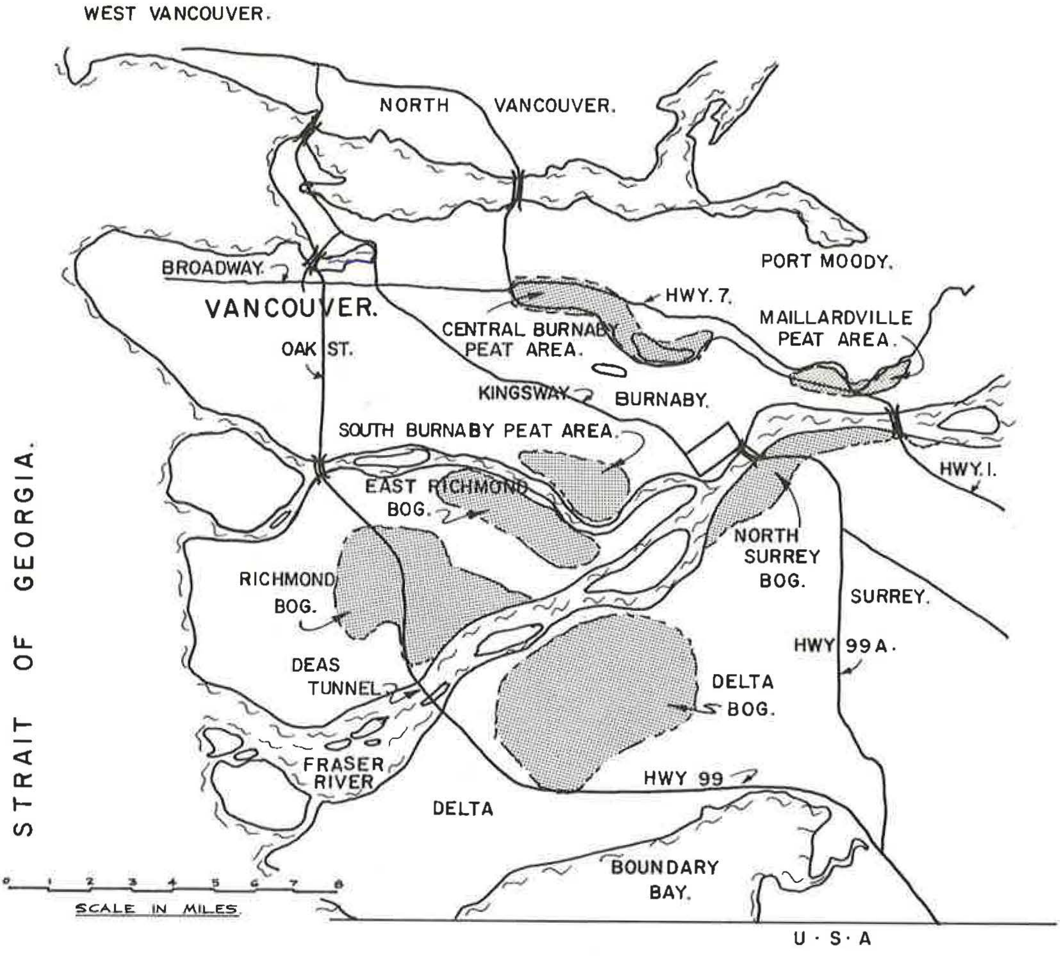


Figure 1. Greater Vancouver peat areas.

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.

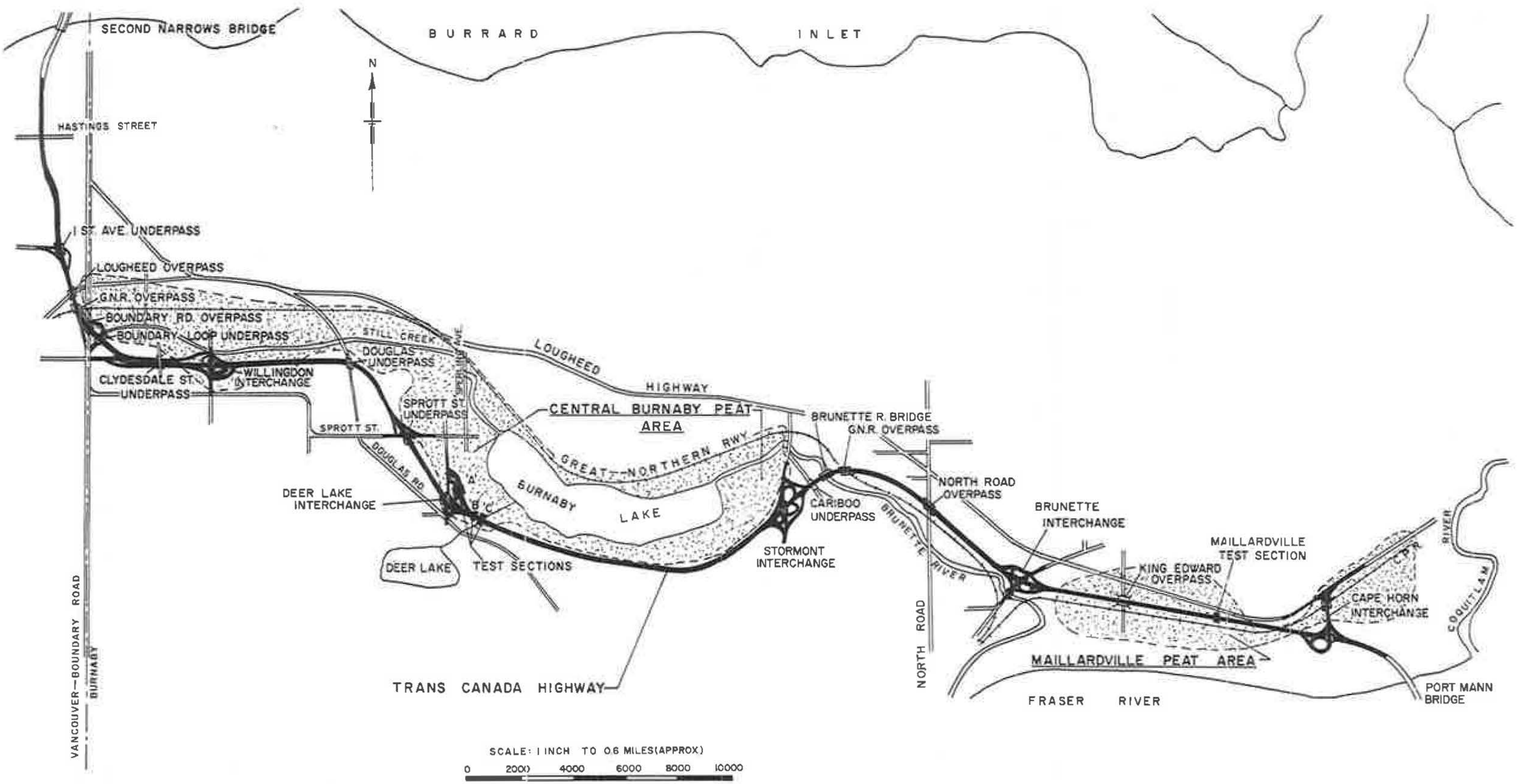


Figure 2. Vancouver, Fraser River section, Trans-Canada Highway.

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,

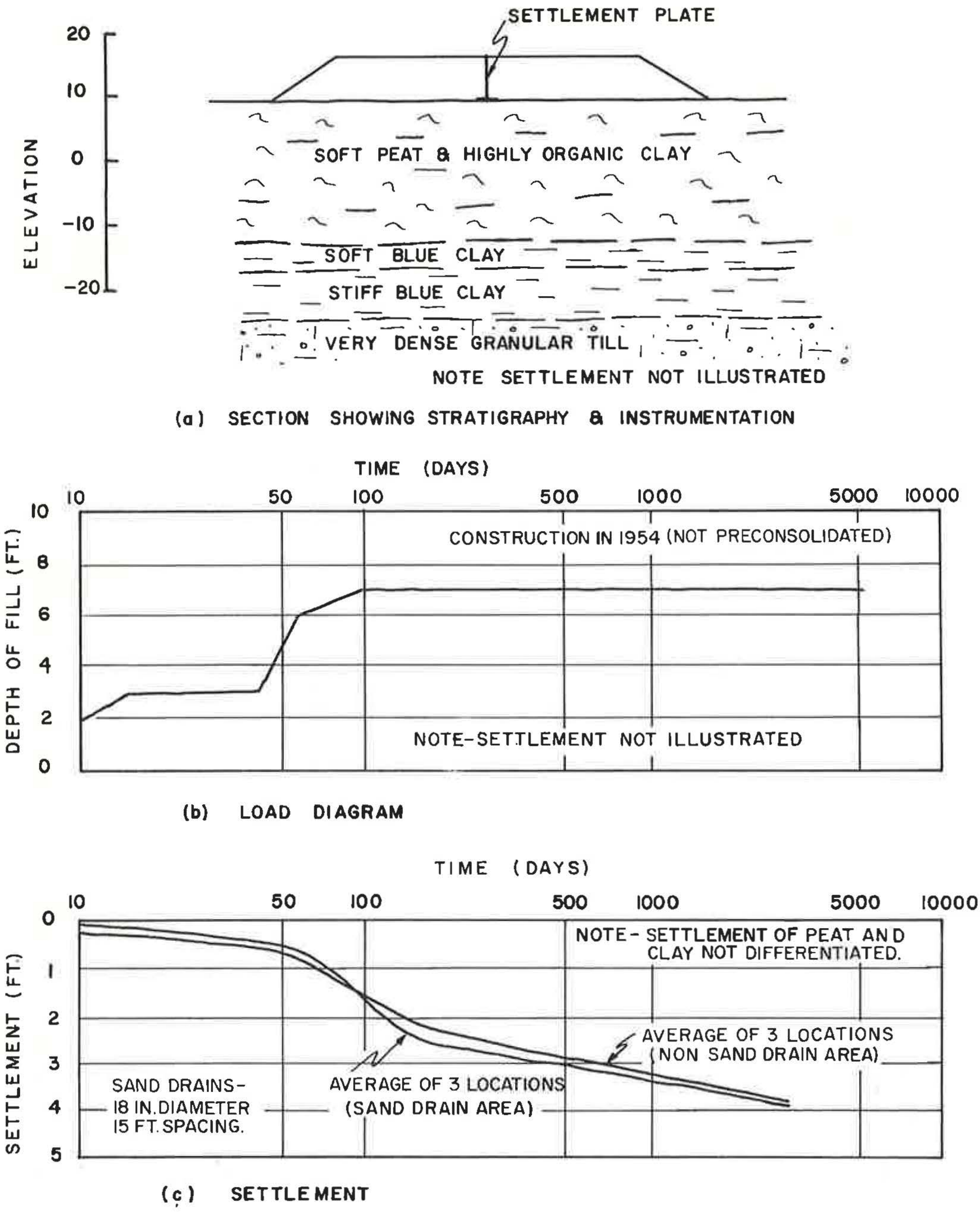


Figure 7. Lougheed Highway at Maillardville.

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

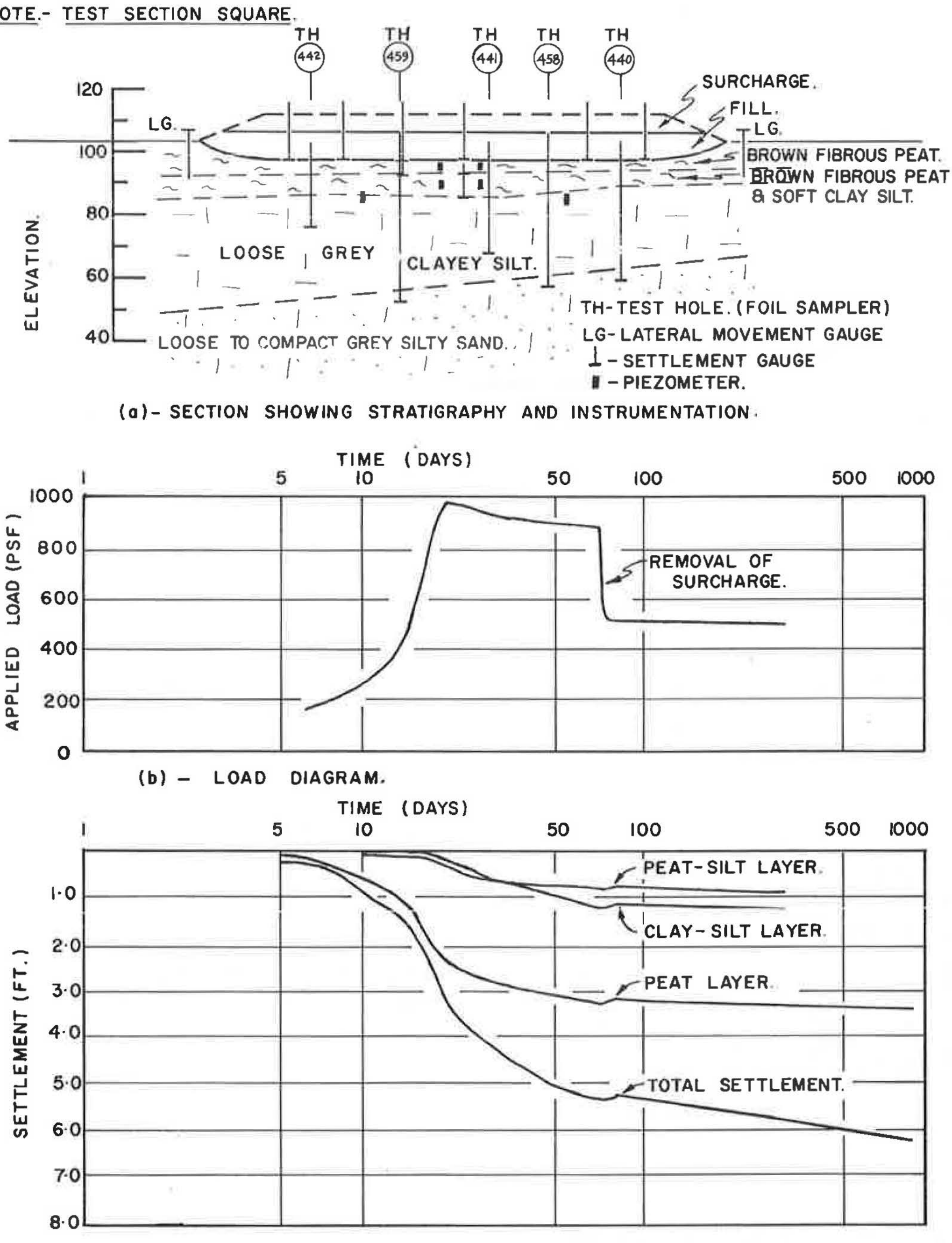
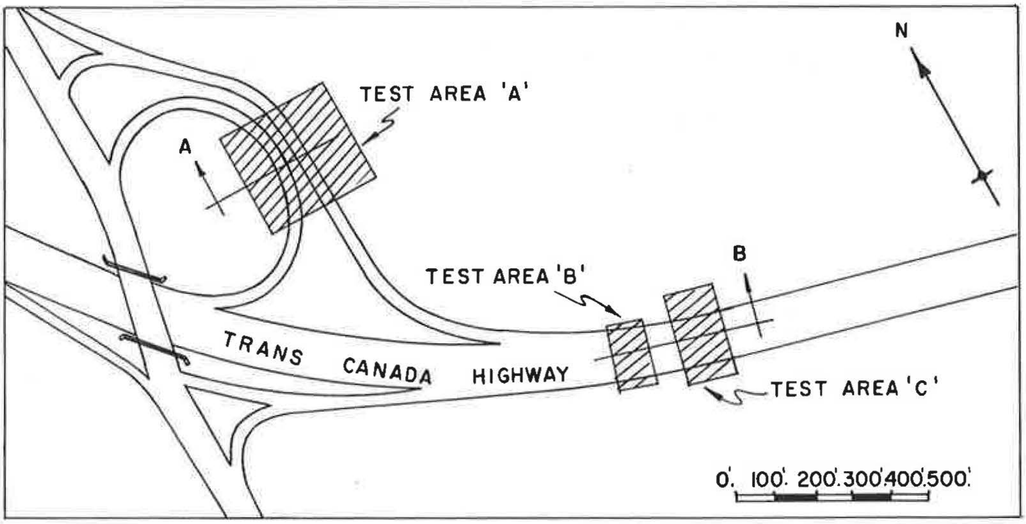
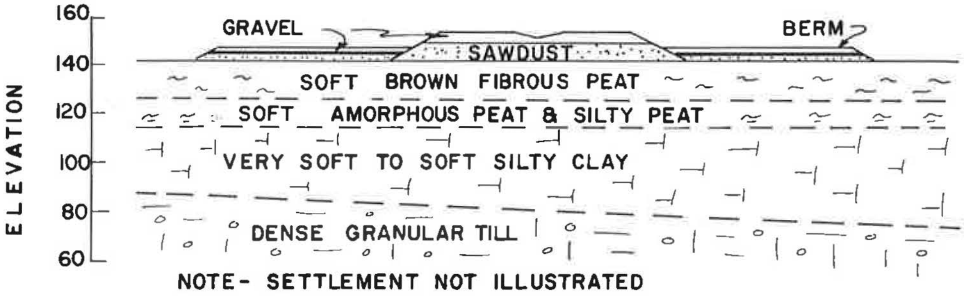


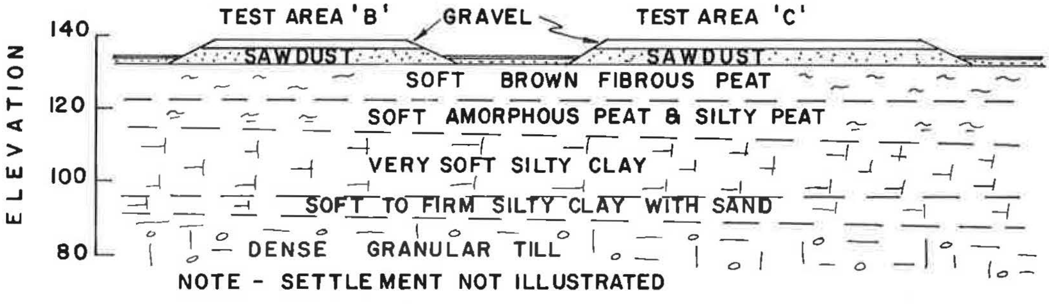
Figure 8. Maillardville test section.



(a) DEER LAKE INTERCHANGE SHOWING TEST AREAS



(b) STRATIGRAPHY — SECTION A - TEST AREA 'A'



(c) STRATIGRAPHY - SECTION B - TEST AREAS 'B' & 'C'

Figure 9. Deer Lake interchange.

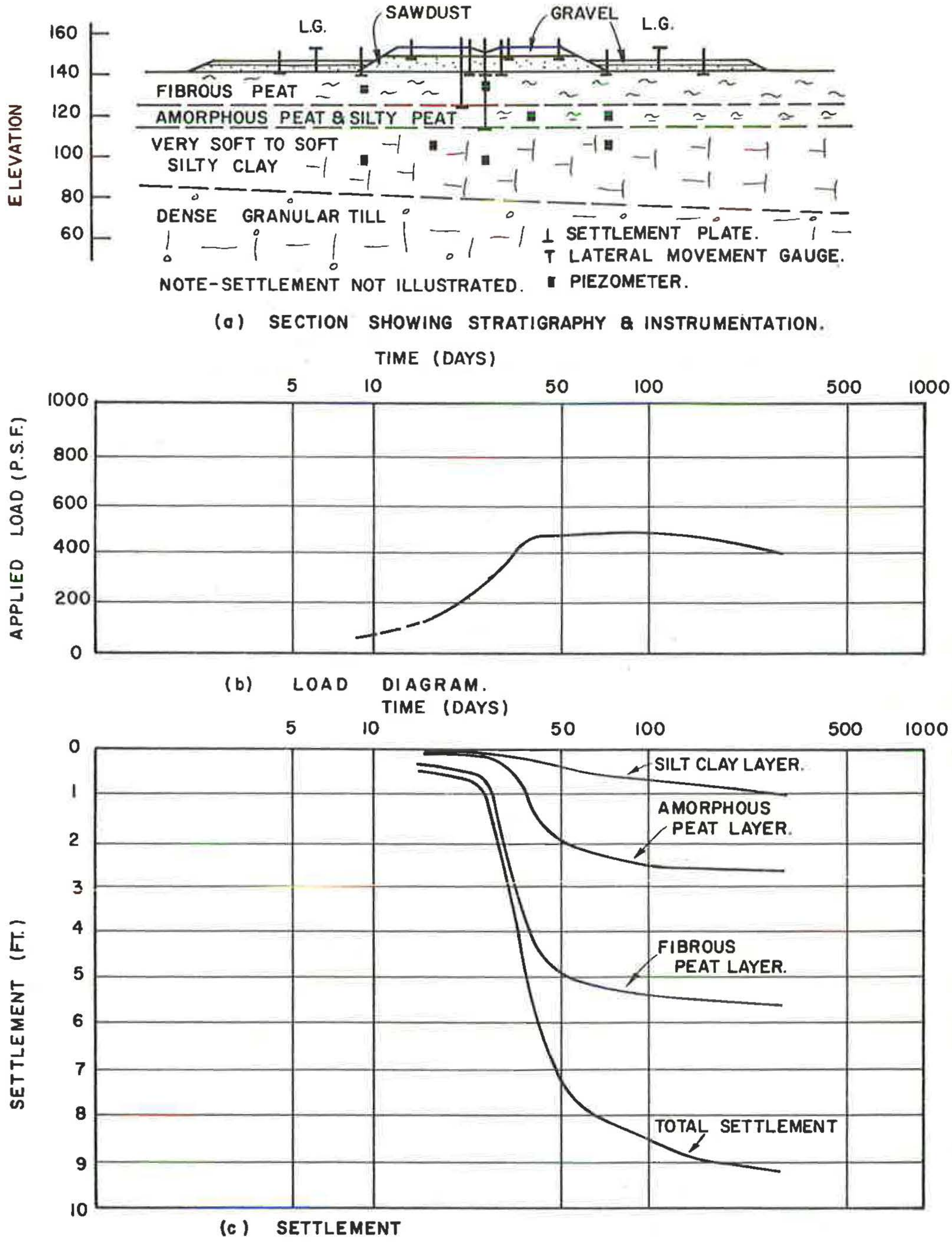


Figure 10. Test area A.

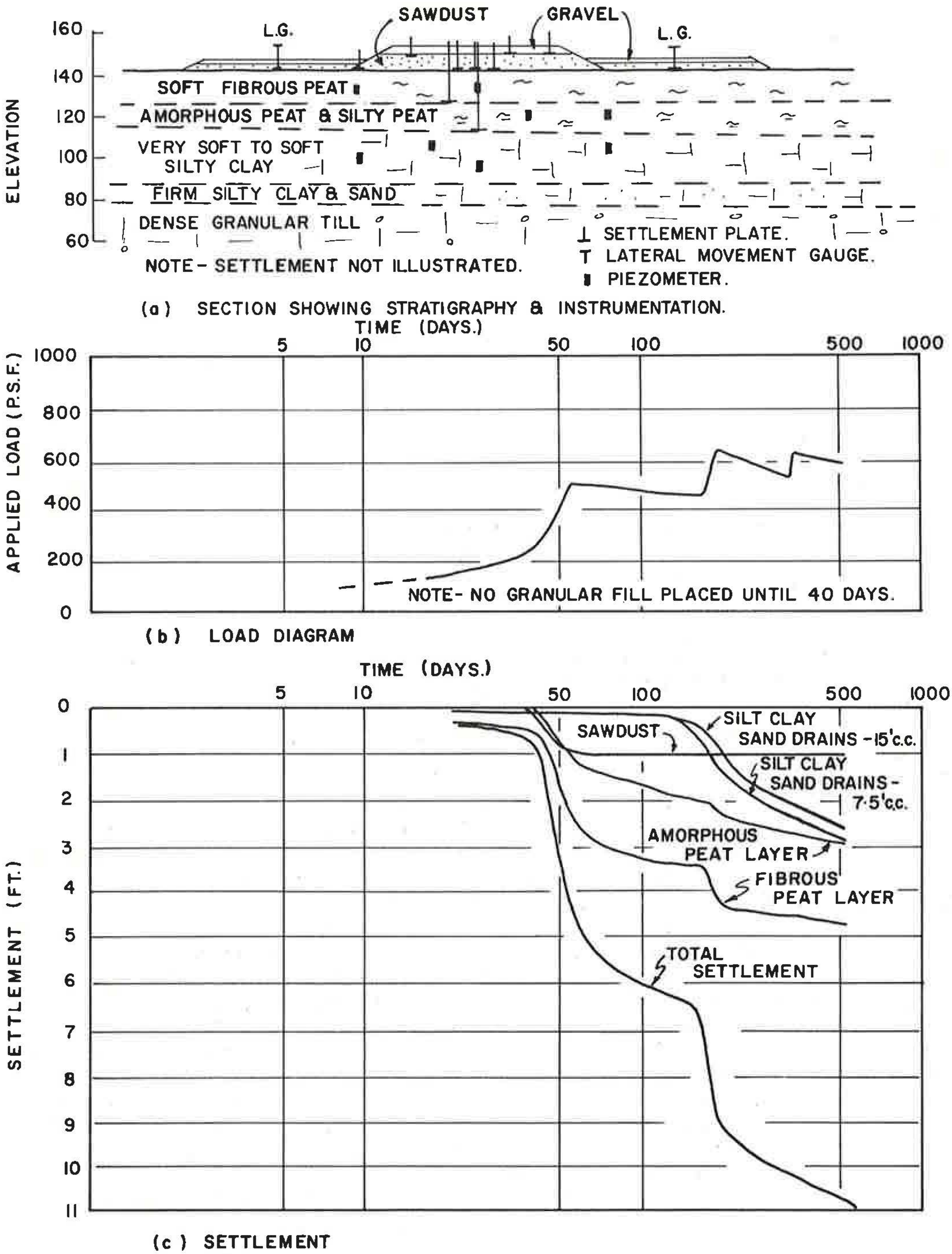


Figure 11. Test area B.

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 $\frac{1}{2}$ 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.

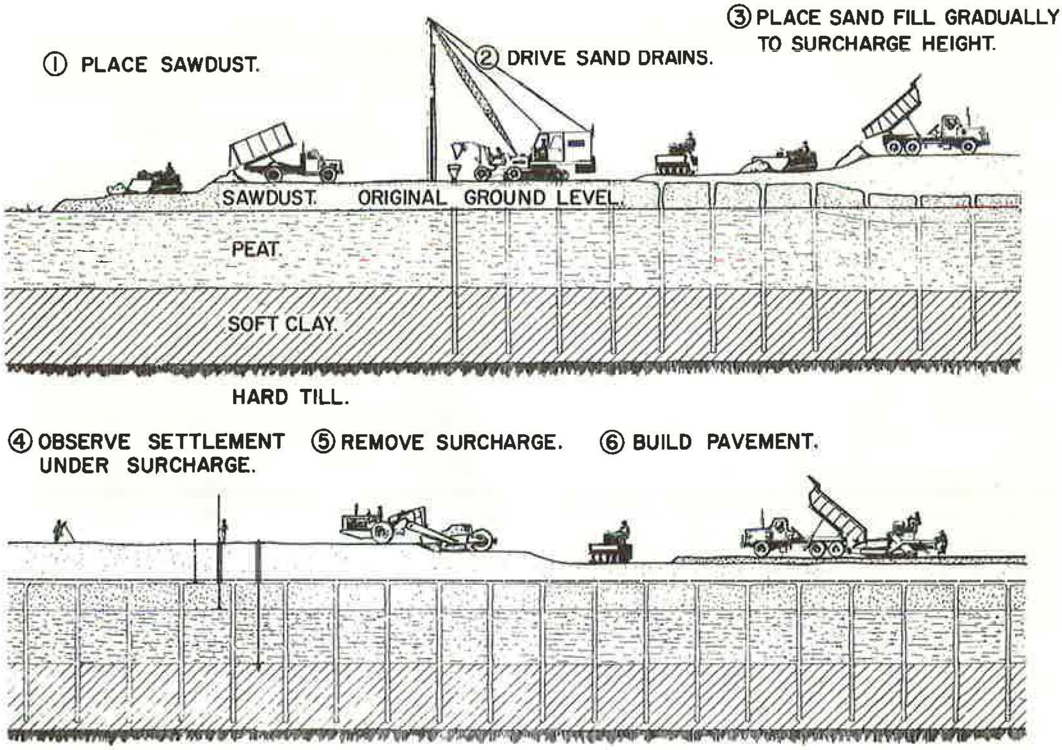


Figure 14. Construction technique using sawdust.

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.

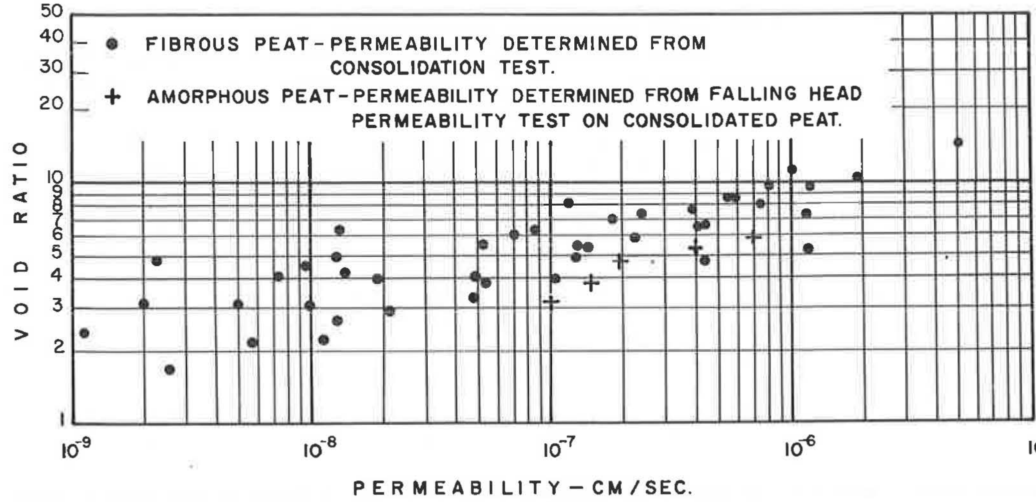


Figure 15. Void ratio vs permeability.

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.

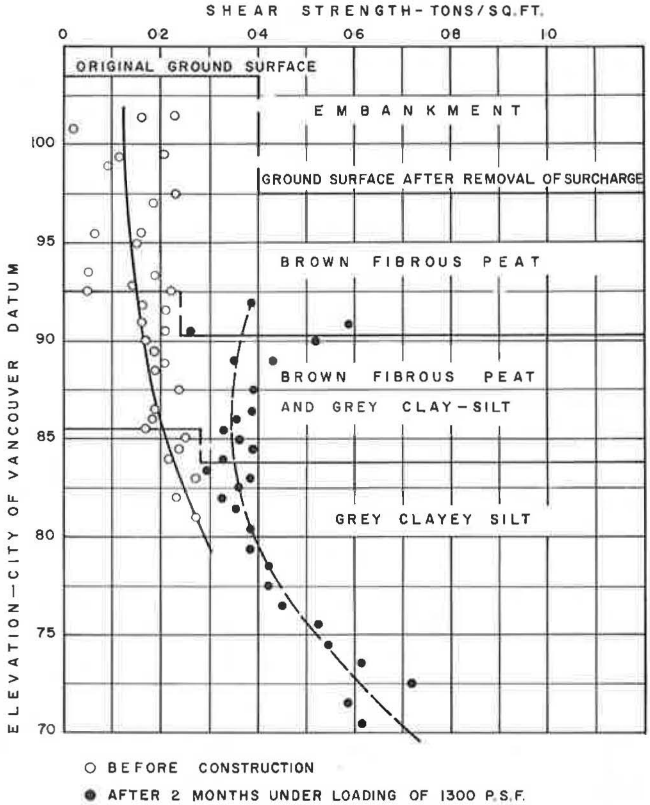


Figure 16. Vane shear strength, Maillardville test section.

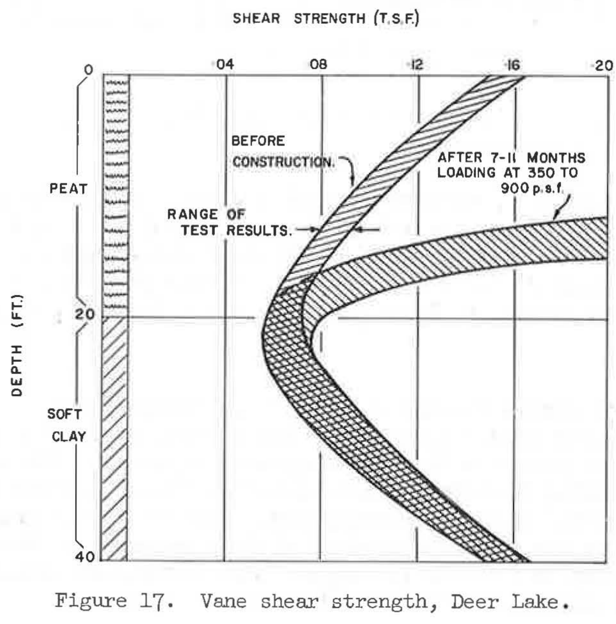


Figure 17. Vane shear strength, Deer Lake.

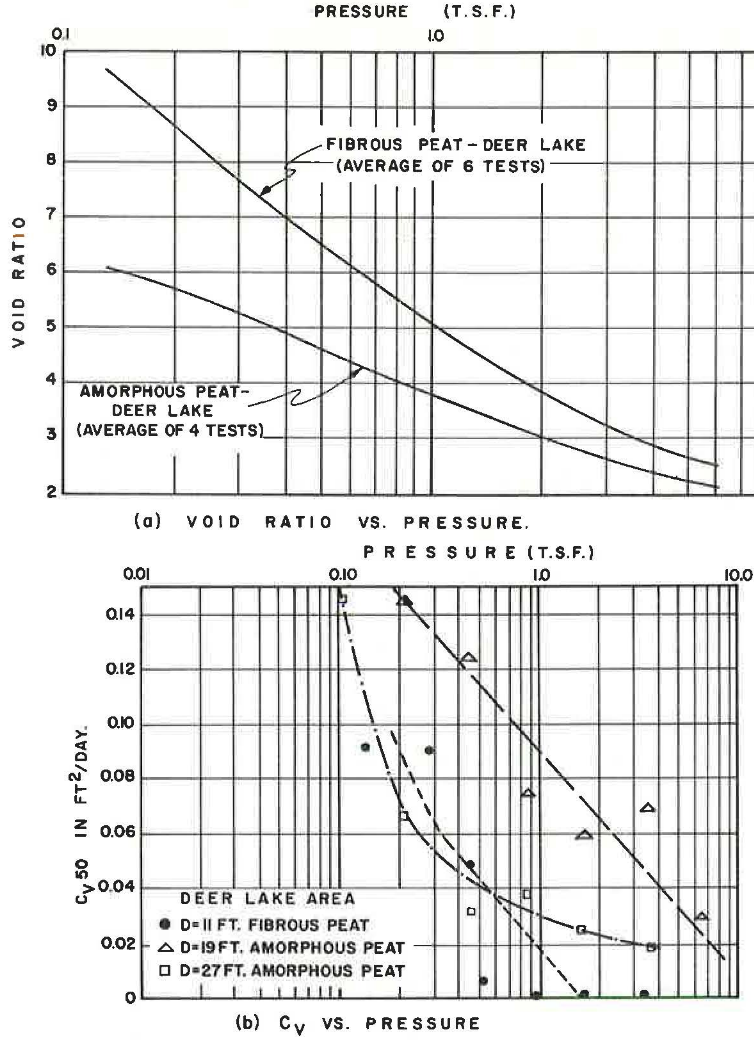


Figure 18. Consolidation test data, Deer Lake area.

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.

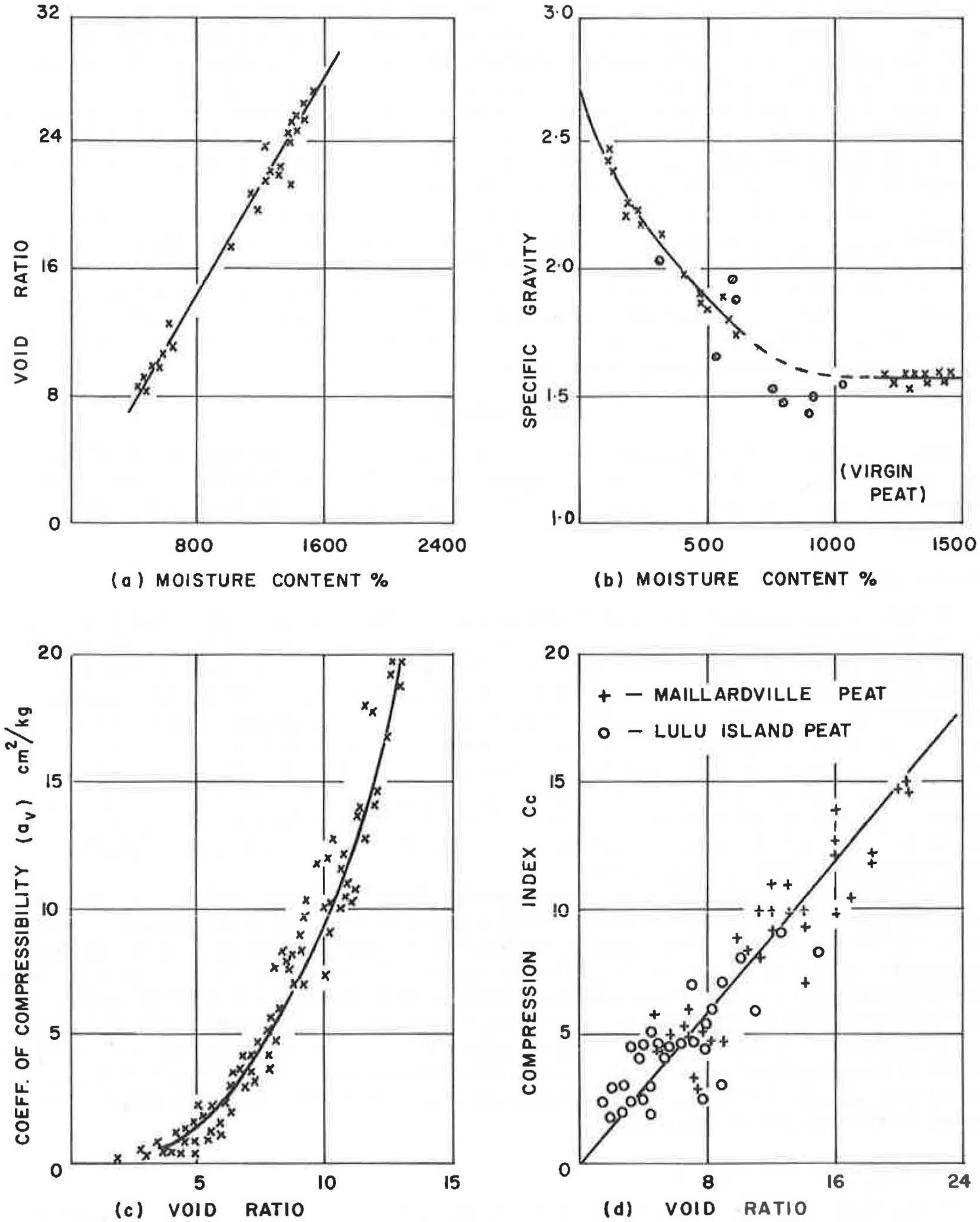


Figure 19. Correlation of various physical characteristics of peat tested in British Columbia.

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 (%)			
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	Ratio L/S	
(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-

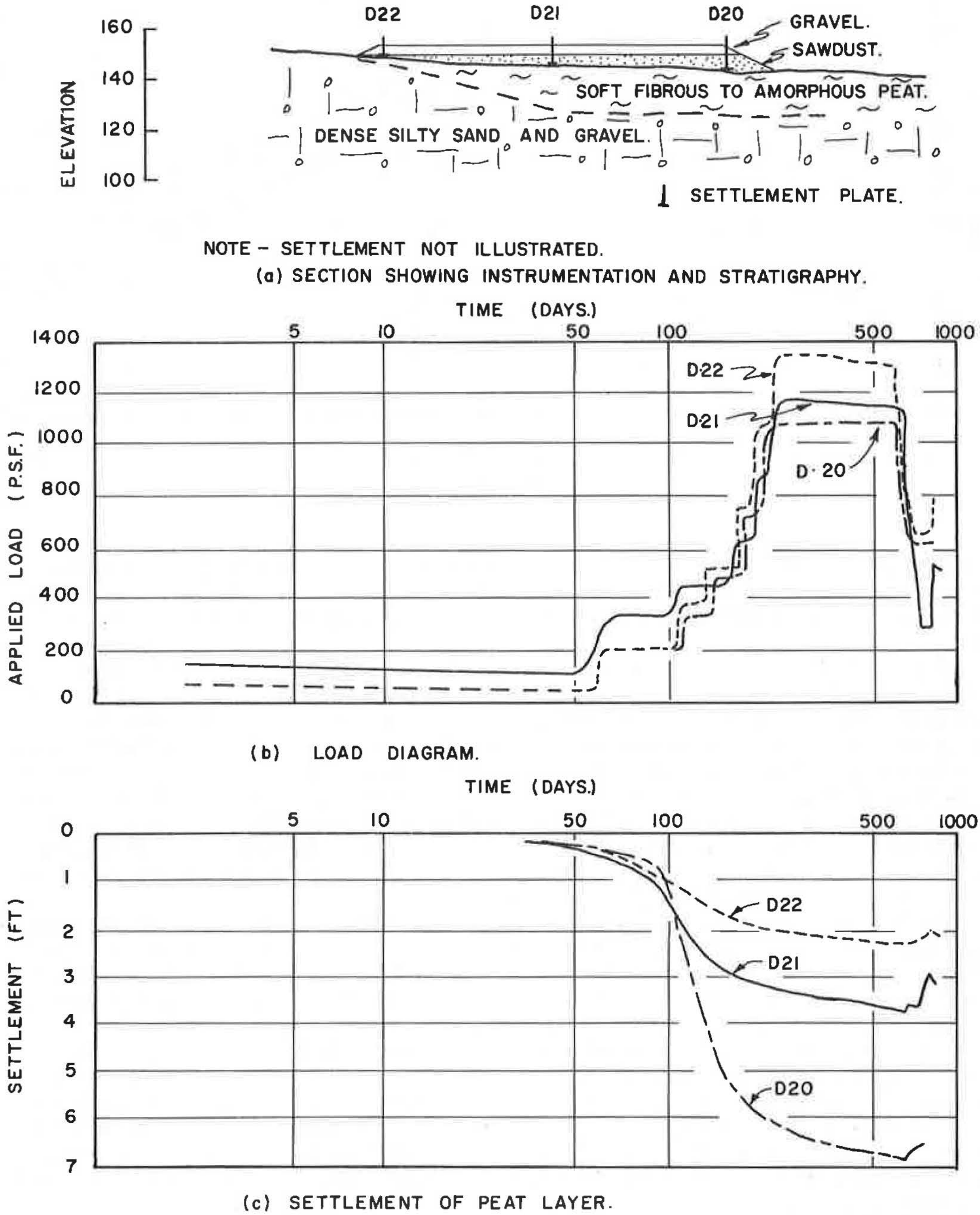


Figure 20. Sprott Street area.

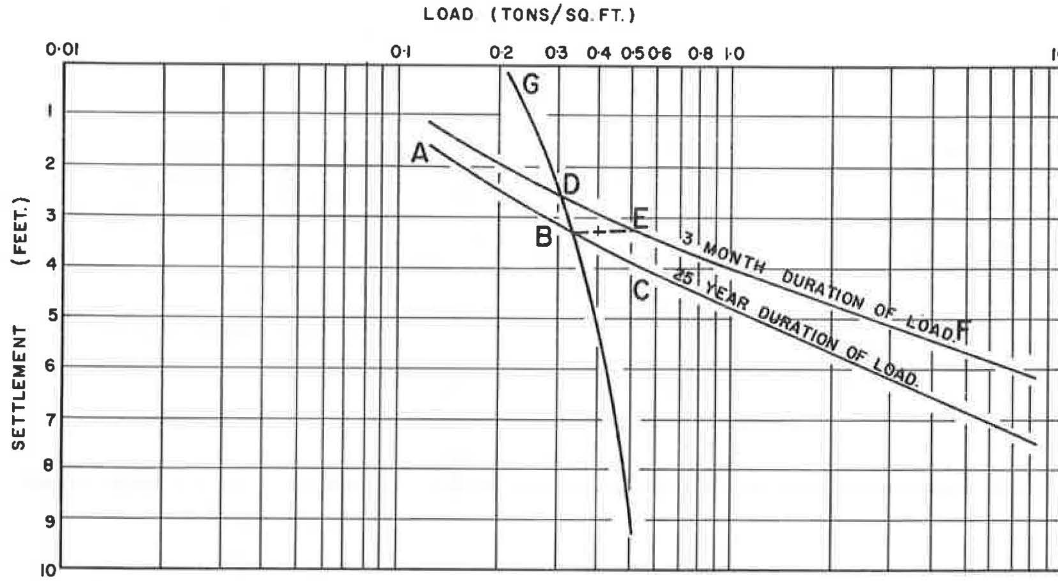


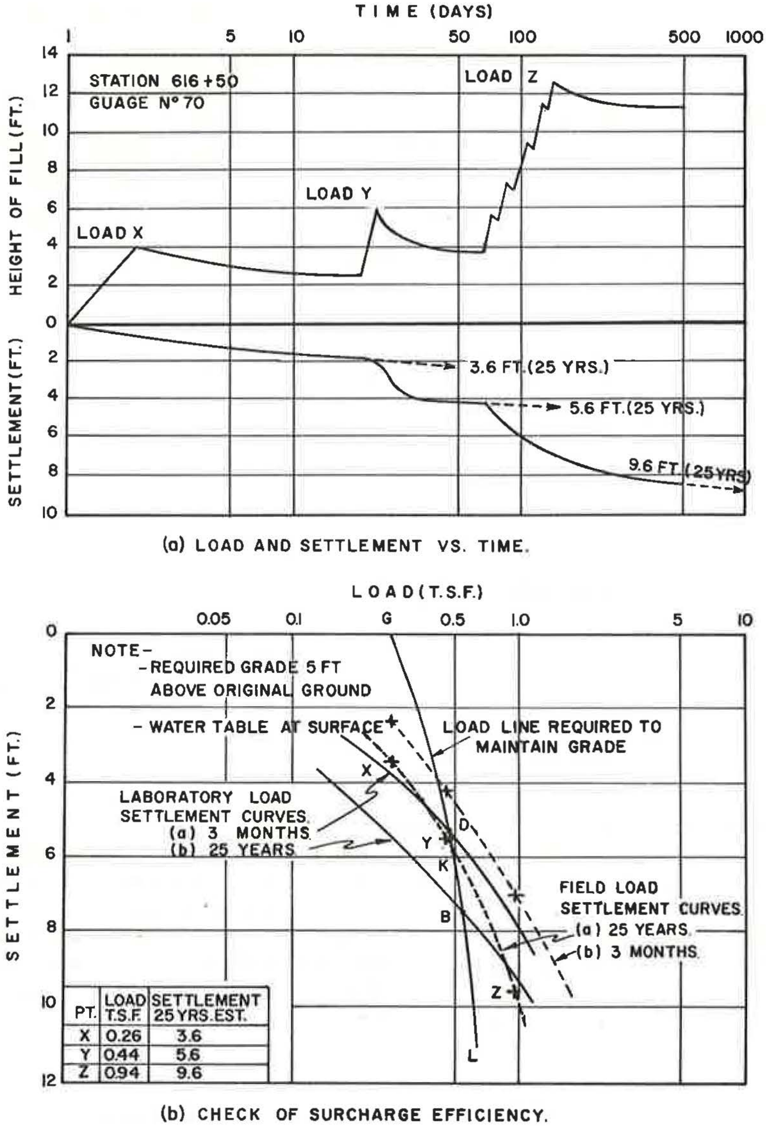
Figure 21. Graphical determination of surcharge load required for peat treatment by preconsolidation.

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



PT.	LOAD T.S.F.	SETTLEMENT 25 YRS. EST.
X	0.26	3.6
Y	0.44	5.6
Z	0.94	9.6

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

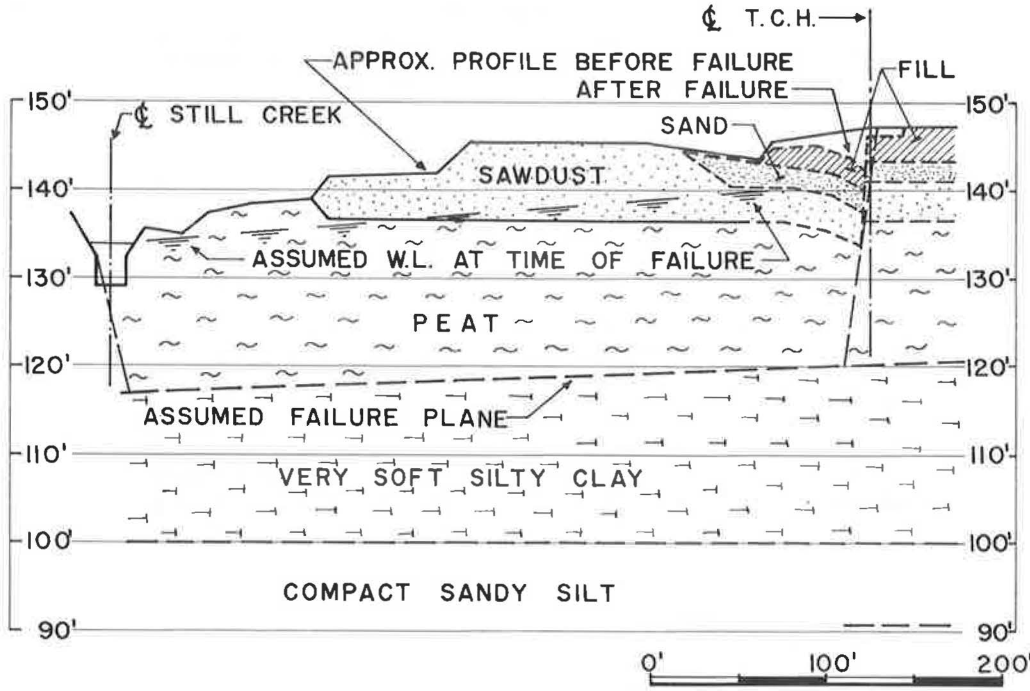


Figure 23. Willingdon failure.

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.

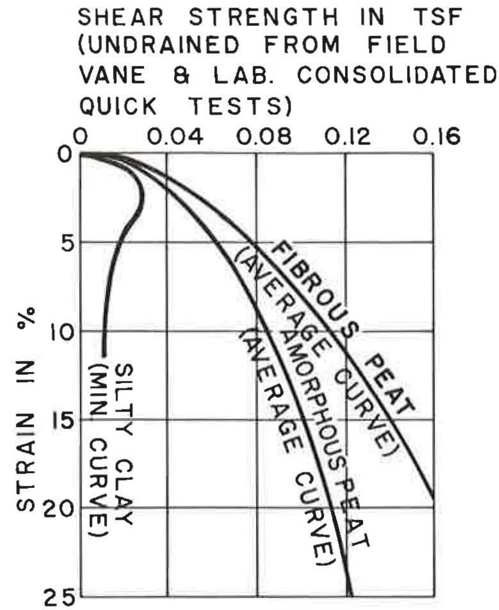


Figure 24. Stress-strain curves of peat and clay.

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 preloaded, 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-

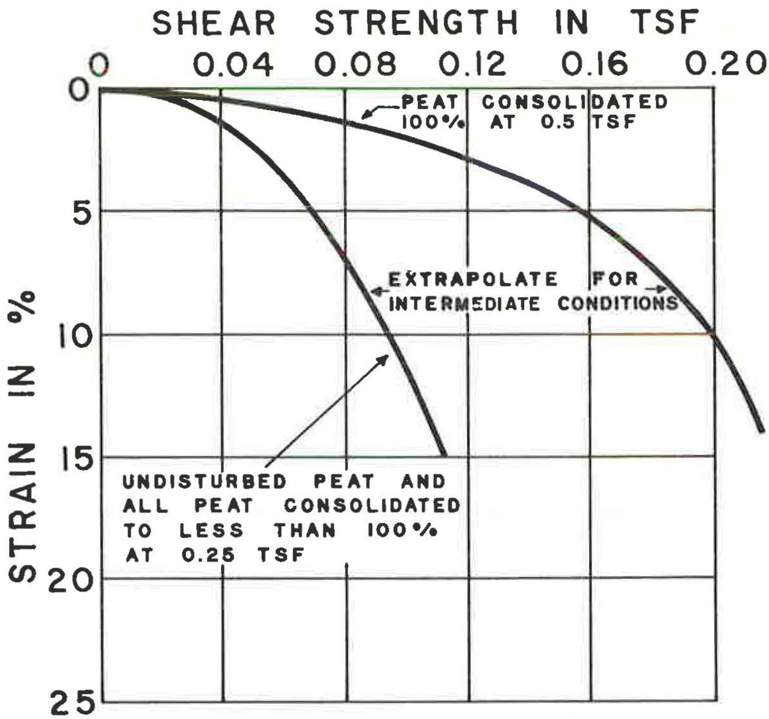


Figure 25. Peat strength properties for stability computations.

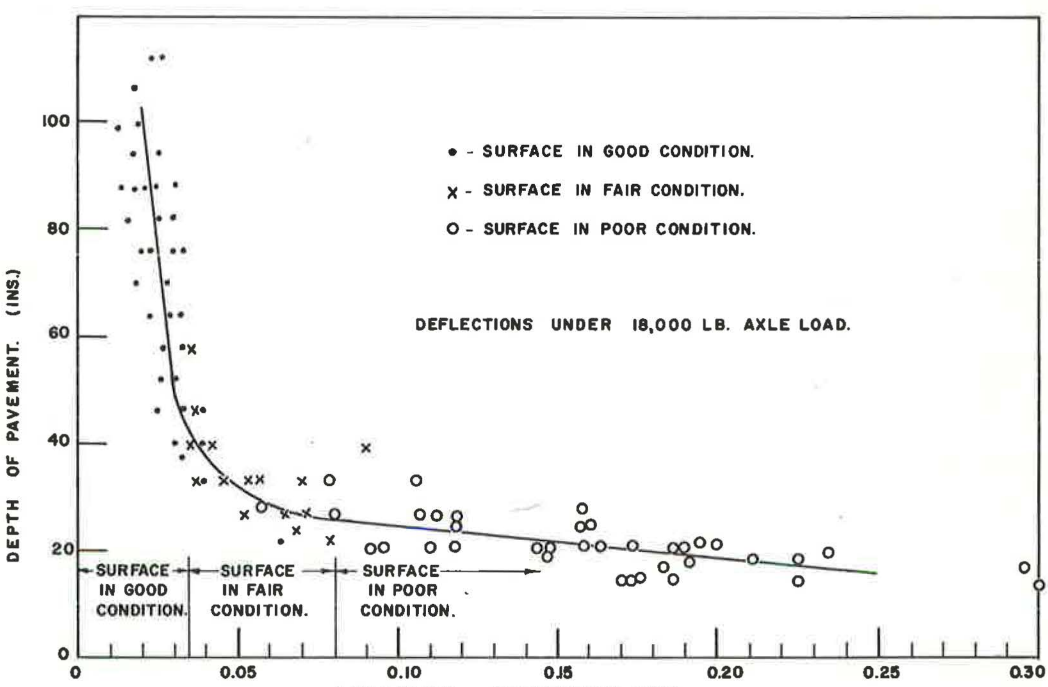


Figure 26. Depth of pavement on peat vs pavement deflection.

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

Appreciation is expressed to P.A. Gaglardi, Minister of Highways for British Columbia, for his permission to publish the data in this paper. The assistance of E. M. Hoy, Senior Soils Engineer of the Foundation of Canada Engineering Corporation Limited, in assembling and reviewing data is gratefully acknowledged.

 Rec. Record No. 7

REFERENCES

1. Radforth, "Suggested Classification of Muskeg for the Engineer." Eng. Jour. (Nov. 1952).
2. Hanrahan, "The Mechanical Properties of Peat with Special Reference to Road Construction." Inst. of Civil Engineers of Ireland (1952).
3. Hanrahan, "An Investigation of Some Physical Properties of Peat." Geotechnique (Sept. 1954).
4. Lake, Proc., 7th Muskeg Research Conf. (1961).
5. Lea, N. D., and Brawner, C. O., "Foundation and Pavement Design for Highways on Peat." 40th Convention of Canadian Good Roads Assoc., Vancouver (1959).
6. Glossop, R., and Golder, H. Q., "Construction of Pavements on Clay Foundation Soil." Inst. of Civil Engineers, Road Research 14, London (1944).
7. Odemark, N., "Undersokningav Elasticitetsegenskaperna hos Olika Jordarter Samt Teori for Berakning av Bellaggningar Engligt Elasticitetsteorin." Statens Vaginstitut, Meddelande, Vol. 77 (1949).
8. Odemark, N., "Om Vagens Konstruktion Vid Hoga Hjultryck." Statens Vaginstitut, Specialrapport 6 (1956).
9. Brawner, C. O., "The Principle of Pre-Consolidation in Highway Construction over Muskeg." Proc., 5th Muskeg Research Conf., Winnipeg (March 1959).
10. Lea, N. D., "The Mechanical Properties of Peat." Proc., 4th Muskeg Research Conf., Ottawa (March 1958).
11. Miyakawa, I., "Some Aspects of Road Construction over Peaty or Marshy Areas in Hokkaido." Civil Engineering Research Inst., Hokkaido Development Bureau, Sapporo, Japan (June 1960).

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.

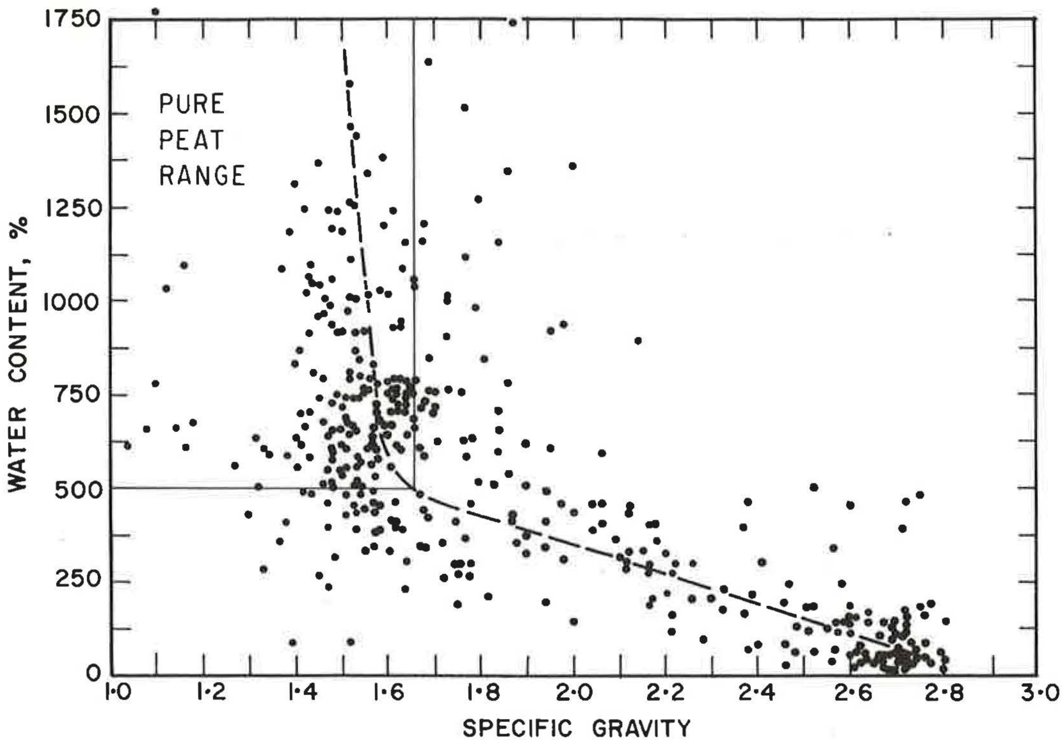


Figure 27. Water content vs specific gravity.

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.

REFERENCES

12. Radforth, N. W., "A Suggested Classification of Muskeg for the Engineer." Eng. Jour. 35: 1199-1210 (Nov. 1952).

13. MacFarlane, I. C., "Guide to a Field Description of Muskeg." National Research Council, Associate Committee on Soil and Snow Mechanics, Tech. Memo. 44, Ottawa (June 1958).

Asphalt Membranes in Expressway Construction

FREDERICK A. HARRIS*, Soils Engineer, Southern Inspection Service, Houston, Texas

Envelope-type asphalt membranes have provided excellent stabilization of plastic earth fills for bridge abutments in urban expressway construction in Houston. Test fills have been observed over a 14-year period with excellent findings. The observations include detailed construction records of moisture-density conditions, followed by annual continuous core-drilling of fills and testing of cores for moisture content, density, and triaxial compressive strength. Test holes penetrated the entire depth of fill as well as the compacted subgrade below fills. For comparison, nearby soils of the same nature as the fills but not protected with the asphalt membranes have been sampled and similarly tested at intervals as found convenient.

Other observations have included the physical appearance of such fills, lateral movements in fills (or more properly, the lack of such movements), stability of the membranes under extreme drying conditions, and appearance of the membranes when exposed during subsequent stage construction. One of the most startling conclusions from these performance observations is that the membraned fills are in many cases more stable than concrete pavements placed on the fills.

The use of asphalt membranes in earth fills in expressway construction in Houston represents a major use of this type of design, having been used on 54 structures requiring 104 abutment fills of volume of over 400,000 cubic yards and treated with approximately 1,600,000 gal of grade OA-55 oil asphalt. It would be difficult to estimate the future demand for asphalt membranes in Houston's expressways as its use has been adopted as standard design for all fills except those having unusually low plastic properties.

Surface and buried types of asphalt membranes have also been used with good success, and these are discussed briefly with comments on general performance and design criteria.

Coverage of 1 gal per sq yd of grade OA-55 oil asphalt has been found to be sufficient to maintain a continuous membrane even under most adverse conditions and to maintain essentially constant moisture content, density, and compressive strength in the fills they envelop.

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*Formerly Sr. Lab. Engr., Houston Urban Expressways, Texas Highway Department.
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• ASPHALT MEMBRANES have been extensively and successfully used in urban expressway construction in Houston. Figure 1 shows the three general types used—surface, buried, and envelope membranes. The last type is the most comprehensive and was used to stabilize medium to highly plastic clay soils known by past experience to be unsuitable for bridge abutment fills due to lateral and vertical seasonal movements. Expansion joints of only 1-in. width were provided between these fills and the bridge abutments. It was therefore imperative that soil movements be restricted to less than ½ in. because some of the total joint opening would be reduced by seasonal expansion and contraction of concrete in the bridges themselves.

For these reasons it was decided to initiate a field research program of annual observations of soil conditions in several of such fills and supplementary observations at less frequent intervals of widths of joint openings at all bridge ends on the Gulf Freeway—a 50-mi length of urban and rural limited-access highway. It includes an approximately equal number of overpasses constructed on urban section with membrane protection and on rural section without protection.

CONSTRUCTION PROCEDURE

Figure 2 shows details of the envelope-type membranes used to completely "wrap-up" fills on urban type structures. The area to receive fill was first stripped of all organic vegetation, then compacted to 100 percent standard Proctor density (AASHO Method T99-38) at optimum moisture content to a depth of approximately 6 in. and then fine-graded to produce a smooth surface. OA-55 grade asphalt was then applied at rate of 1.0 gal per sq yd to the entire subgrade surface and for a distance of a few feet beyond the proposed toe of fill slopes. The fill core was then constructed in layers 6 to 8 in. thick to the same density requirements and the sides fine-graded to 1 on 1.5 slope. OA-55 asphalt was then applied to the fine-graded upper surface and side slopes at rate of 1.0 gal per sq yd in such a manner as to intersect and tie into the bottom membrane. Top soil previously stripped from the fill subgrade was then used to cover the side membranes and to flàtten the side slopes to 1 on 4 or flatter; these slopes were then block-sodded to prevent erosion and to provide an architecturally pleasing outward appearance. The upper membranes were covered with an appropriate base course for the expressway pavement.

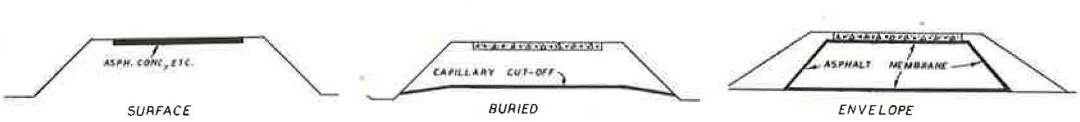

Figure 1. Functional types of membranes.

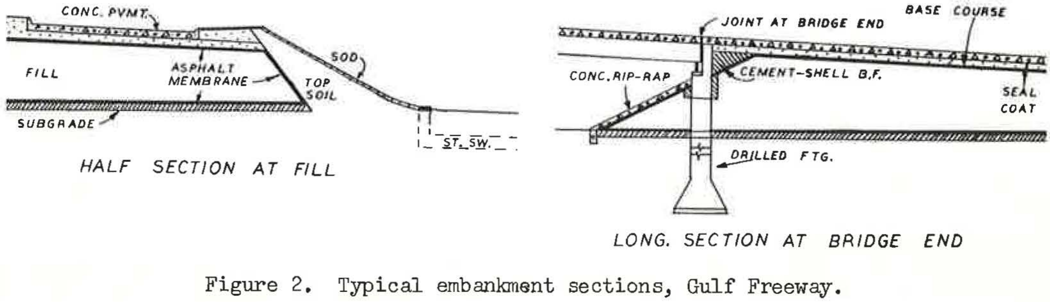

Figure 2. Typical embankment sections, Gulf Freeway.

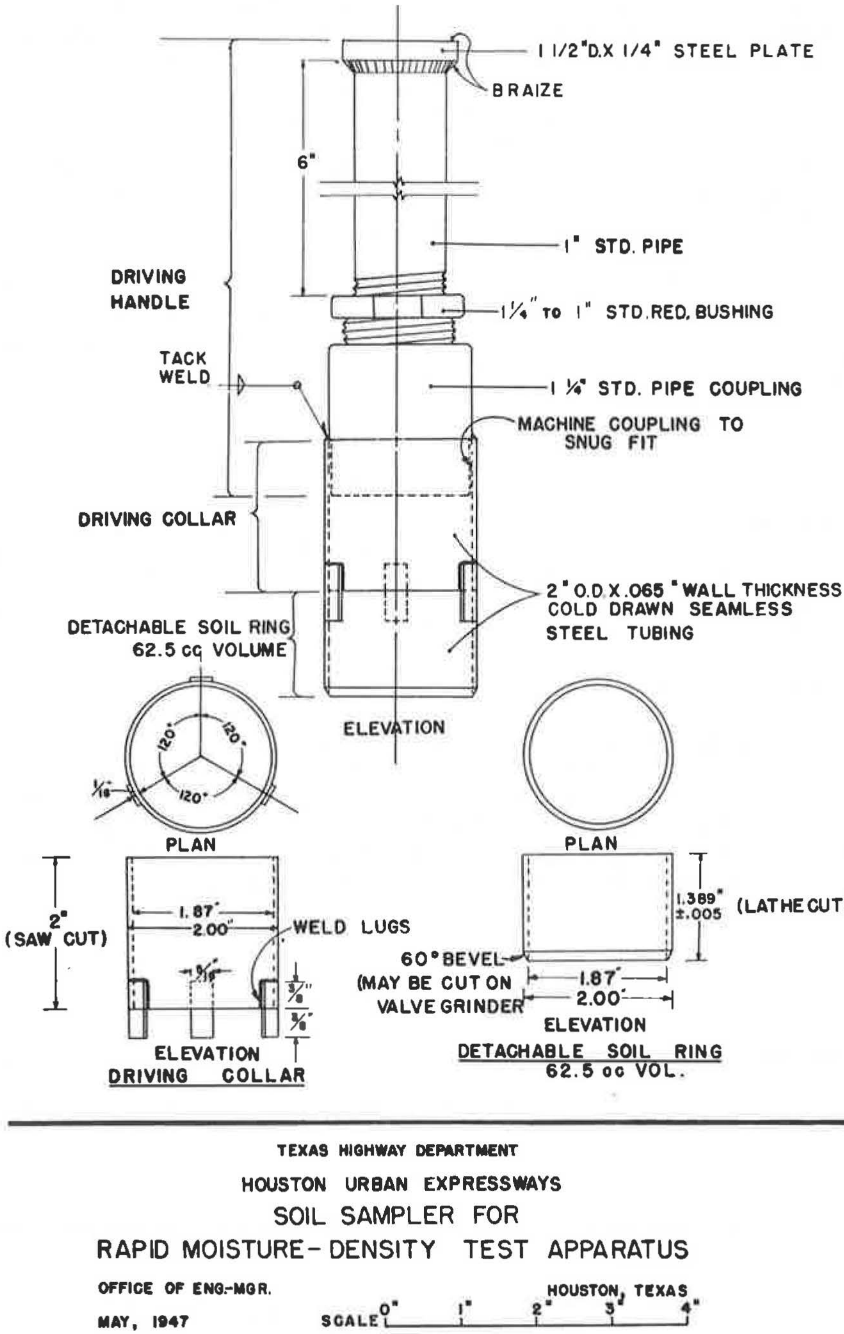

Figure 3. Ring density apparatus.

The first layer of each fill was compacted with bulldozers and other track-type equipment (in order not to puncture the asphalt membrane); subsequent layers were compacted with sheep's-foot rollers. Fine-grading of side slopes was done with motor graders to provide a smooth surface and included cutting of a small trench at the fill toes to expose the lower membrane temporarily until application of the side membranes; this was done to insure that the side membrane would tie in to the lower membrane at all points, thus resulting in a complete and continuous asphalt membrane envelope.

The soils in fills and fill subgrade varied from CL to CH unified soil classification with maximum and minimum limits approximately as follows: optimum moisture content, 23 to 28 percent; optimum dry density, 96 to 108 pcf; and plasticity index, 30 to 50 percent.

RESEARCH PROGRAM

Six representative abutments were selected as test fills. Moisture content and density of compacted subgrade and each layer of each fill were determined using Harris ring density apparatus (1). Figure 3 shows the field apparatus. Accurate records of these tests were maintained to serve as initial conditions for the proposed future observations. All available data on soil conditions in the vicinity of the tests fills were also assembled and reviewed to serve as initial conditions for untreated soil. After completion of construction, each test fill was cored to obtain undisturbed cores; these were tested to determine moisture content, density, and compressive strength. The latter tests were made with triaxial compressive test apparatus and the results were calculated to obtain compressive stress-strain curves.

At yearly intervals each fill was again core-drilled without use of drilling water; undisturbed cores were taken at approximately 2-ft intervals in each fill and in earth subgrade. These were tested to determine moisture content, density, and compressive strength as previously described. Average values of the test results for fill, subgrade, and the underlying natural soil were then compared with initial conditions and plotted against time in years. Figures 4 through 9 show such records over a period of 14 years to date. After 8 years, the time interval was lengthened to 5 years due to the consistency found in earth fill by yearly observations.

TEST RESULTS

Figures 4 through 9 show that moisture content, density, and shearing resistance of all fills and subgrades were essentially constant through 8 years of service. Conditions found at the last 5-yr observation were also essentially constant for all fills except East Approach Wayside (Fig. 7) which showed some unexplainable variations. This may be the result of a localized variation between test hole locations. Wide variations have been noted in soil conditions in nearby untreated soil, thereby definitely proving the efficiency of the envelope-type membranes.

It has been suggested that similar membranes be used with plastic soils to construct stable base courses for pavements (2) results of this investigation indicate such proposal to be entirely feasible and such construction of a permanently stable nature. These results also indicate that membrane thicknesses in excess 1.0 gal per sq yd are not required for soil conditions prevailing at the sites and the construction procedures used. This amount of coverage gives continuous membranes from $\frac{1}{16}$ to $\frac{1}{4}$ in. in thickness and has been found to be sufficient even under very adverse conditions, as discussed later.

STABILITY OF JOINT OPENINGS AT BRIDGE ABUTMENTS

The purpose of the envelope-type membranes was to provide stable fills for bridge abutments. The field and laboratory data obtained from the test fills seemed to indicate that excellent stability was being attained. However, it still remained to prove or disprove the effectiveness of the membranes in maintaining essentially open joints at bridge ends. It was therefore decided to examine after several years service the condition of 1 in. expansion joints at each bridge abutment, including the test fills and all other fills on the 50-mi length of the Gulf Freeway. An approximately equal number of fills with and without membranes are existing and were all constructed at approximately the same time (within a few years of each other). Observations were made in the spring of the year and on clear days when the air temperature was between 60 and

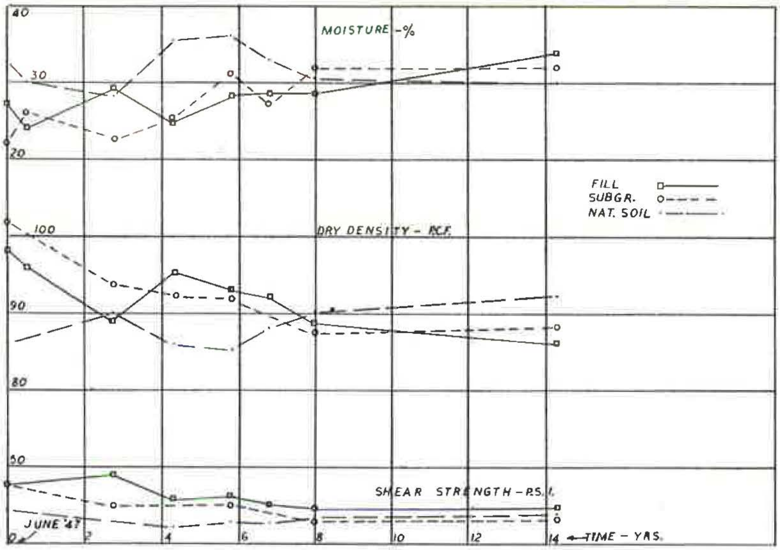

Figure 4. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, West Lombardy.

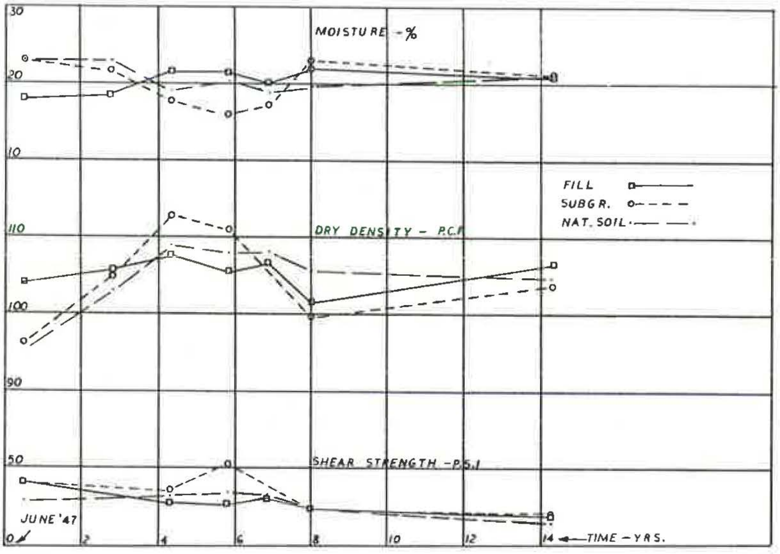

Figure 5. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, West Calhoun.

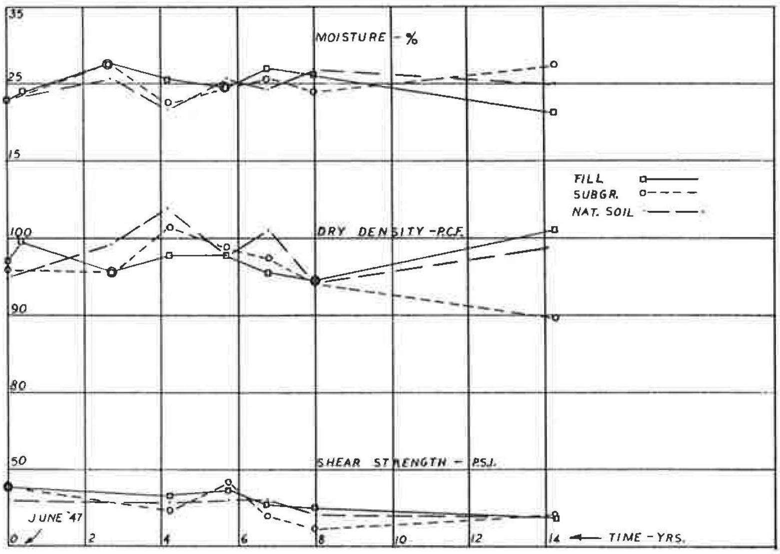

Figure 6. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, East Calhoun.

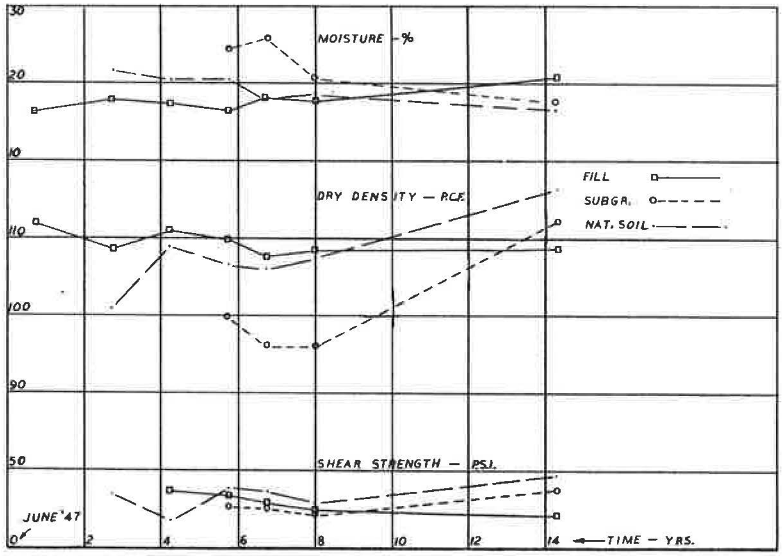

Figure 7. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, East Wayside.

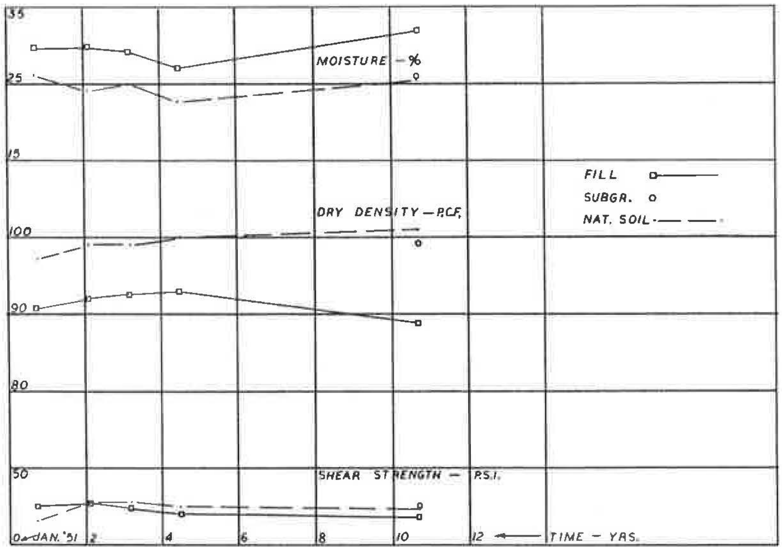

Figure 8. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, East Sims.

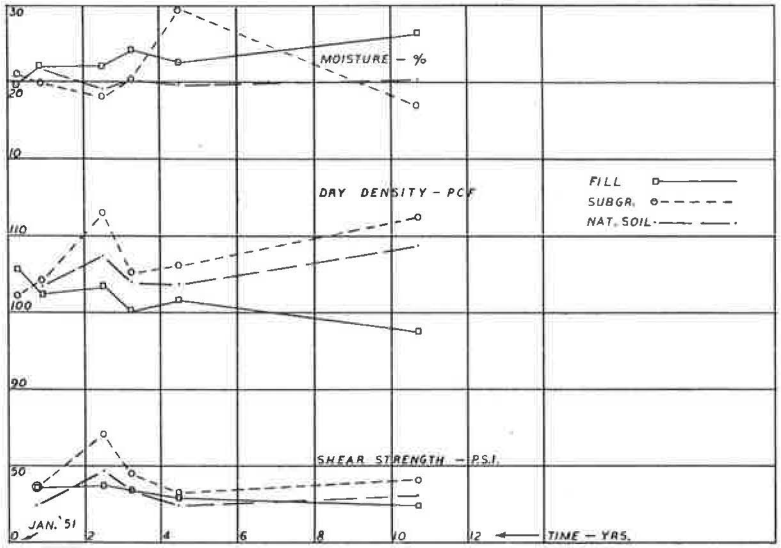

Figure 9. Moisture, dry density, and shear strength of fill, subgrade, and natural soil vs time, West Sims.

80 F, so as to eliminate as nearly as possible the effect of concrete expansion in the bridges themselves and in the adjoining concrete pavement slabs. Table 1 summarizes the conditions found in 1956 (maximum of 8 years after construction). Tables 2 and 3 summarize the conditions found in 1962 (maximum of 14 years after construction). These figures show excellent performance of the membraned fills, and poor to mediocre performance in the majority of the fills constructed without membranes.

OTHER OBSERVATIONS

A striking demonstration of the efficiency of such membranes was provided late in the summer of 1952 by what at first appeared to be a failure in one of the 4-year-old

TABLE 1

BRIDGE ABUTMENT SURVEY, APRIL 19, 1955, US 75, GULF FREEWAY[a]

Fill Type	Structure	Height (ft) Fill	Height (ft) Cut	Structure Length (ft)	Condition of Expansion Joints in Bridge
Membraned	Velasco-IGNRR	10		958	Fully open
	Scott St.	10		676	Very slight closure
	Cullen Blvd.	9		704	Fully open
	Calhoun Rd. -HB&TRR	10		1,192	Fully open
	Lombardy St. -HB&TRR	10		1,208	Approx. 1-in. closure at each end
	Telephone Road	11		366½	Closed due to 1-in. movement of pavement slabs into bridge
	Wayside Drive	8	4	292	Fully open
	Bray's Bayou	2-17		370	Fully open
	Grigg's Rd.	10-11		1,115½	Fully open except possibly some closure on south end
	Woodridge St.	9	3	295	Fully open
	Str. 12 (Reveille)	12	3	200	Fully open; anchor bolts on rocker arms too tight
	Str. 13 (Reveille)	13	4	165	Fully open; anchor bolts on rocker arms too tight
	Park Place Rotary	10		710	Fully open
	Sims Bayou	0-14		223	Fully open
Plain	Howard Street	12		184	Fully closed
	Garden Villas	20		181.6	Fully closed
	Ellington Field	19		231½	Fully open
	Clear Creek	16		451½	Some closing, not serious
	FM 518	17		181½	Some closing, not serious
	Dickinson Bayou	13½		290½	Fully open
	Camp Wallace	16		182	Fully open
	FM 1765 (St. 348 Ext)	17		181.9	Fully closed
	FM 519	15		181.9	Fully closed
	Rt. Lane at Galv. "Y"	20		244½	Some closure on north end

[a]Weather: clear and mild; temperature, 70 to 80 F.

TABLE 2

BRIDGE ABUTMENT SURVEY, MAY 3, 1962, US 75, GULF FREEWAY

Fill Type	Structure	Height (ft) Fill	Height (ft) Cut	Structure Length (ft)	Condition of Expansion Joints at Bridge Ends
Membraned	Velasco-IGNRR	10		958	Fully open, except at pavement ends joint closed completely
	Scott St.	10		676	Open $\frac{7}{8}$ in., except at pavement ends joint closed $\frac{1}{2}$ to $\frac{3}{4}$ in.
	Cullen Blvd.	9		704	Open $\frac{1}{2}$ in.
	Calhoun Rd-HB&TRR	10		1,192	Fully open, except at pavement ends joint closed $\frac{3}{4}$ in.
	Lombardy St.-HB&TRR	10		1,208	Open $\frac{1}{2}$ in. on right, closed on left
	Telephone Rd.	11		336.5	Fully open, except at pavement ends joint closed completely
	Wayside Drive	8	$\frac{1}{3}$	292	Open $\frac{5}{8}$ to $\frac{7}{8}$ in., except at pavement ends joint closed completely
	Bray's Bayou	2-17		370	Open $\frac{1}{4}$ in., except at pavement ends joint closed completely
	Griggs Road	10-11		1,115.5	Open $\frac{3}{4}$ to 1 in., except at pavement ends joint completely closed
	Woodridge St.	9	$\frac{1}{4}$	295	Fully closed
	Str. 12 (Reveille)	12	3	200	Completely open
	Str. 13 (Reveille)	13	4	165	Almost completely closed at pavement ends, joint varies from open to closed
	Park Place Rotary	10		710	Completely open except at pavement ends joint completely closed and $\frac{1}{2}$-in. differential uplift evident
	Sims Bayou	0-14		223	Open $\frac{7}{8}$ in. same closure at pavement ends
Plain	Howard St.	12		184	Completely closed, distress
	Garden Villas	20		181.6	Completely open, except partially closed at pavement ends
	Ellington Field	19		231.5	Completely open
	Clear Creek	16		451.5	Completely closed, distress
	FM 518	17		181.5	Completely open, except $\frac{1}{2}$ closed at pavement ends
	Dickinson Bayou	13.5		290.5	Completely closed
	Camp Wallace	16		182	Completely open
	FM 1765	17		181.9	Completely closed
	FM 519	15		181.9	Completely closed
	Rt. Lane at Galv. "Y"	20		244.5	Completely closed

TABLE 3

BRIDGE ABUTMENT SURVEY, MAY 3, 1962, US 75, GULF FREEWAY, SUMMARY OF ABUTMENT JOINTS IN FILLS

Fill Type	Fill No.	Condition of Joint	% of Total[a]
Membraned	2	Completely or almost completely closed	14
	5	Partially closed	36
	7	Completely or almost completely open	50
Total	14		100
Plain	6	Completely or almost completely closed	60
	0	Partially closed	0
	4	Completely or almost completely open	40
Total	10		100

[a]Of each type.

membraned fills, but which on further examination proved to be limited to the soil above the upper asphalt membrane. During a long dry season, a crack of up to 12-in. width and 80 ft in length opened near the crown of this fill. Figure 10 shows a man standing in this crack; his feet are resting on the upper membrane where the crack stopped. The crack was cleaned out in several places and the membrane found to be intact in all cases and showing numerous shiny surfaces. Figure 11 shows one such test pit with the membrane cut and peeled back to allow drilling of a test hole. The exposed surface of the fill was intact and showed even the marks of the grader blade used in fine grading of slopes.

Tests of samples from this test hole and from others in the clay above the membrane showed conditions as summarized in Figure 12. Shrinkage cracks developing in the clay above the upper membrane had become so numerous that the weight of the laterally unsupported prisms of soil produced sliding forces greater than the shearing resistance of the warm asphalt membrane, and these soil prisms accordingly slid downhill and closed together in much the same manner as stacking a deck of playing cards. The clay fill under the membrane was in essentially the same condition as constructed (25 to 28 percent moisture) and did not show even small cracks; the clay above the membrane varied from hard and dry to soft and wet in consistency with moisture contents varying from 7 to 35 percent.

It was concluded from these observations that the 1.0-gal per sq yd coverage

Figure 10. Construction engineer in crack, Str. 13, summer 1952.

Figure 11. Crack and test pit, membrane peeled back from embankment, Str. 13, summer 1952.

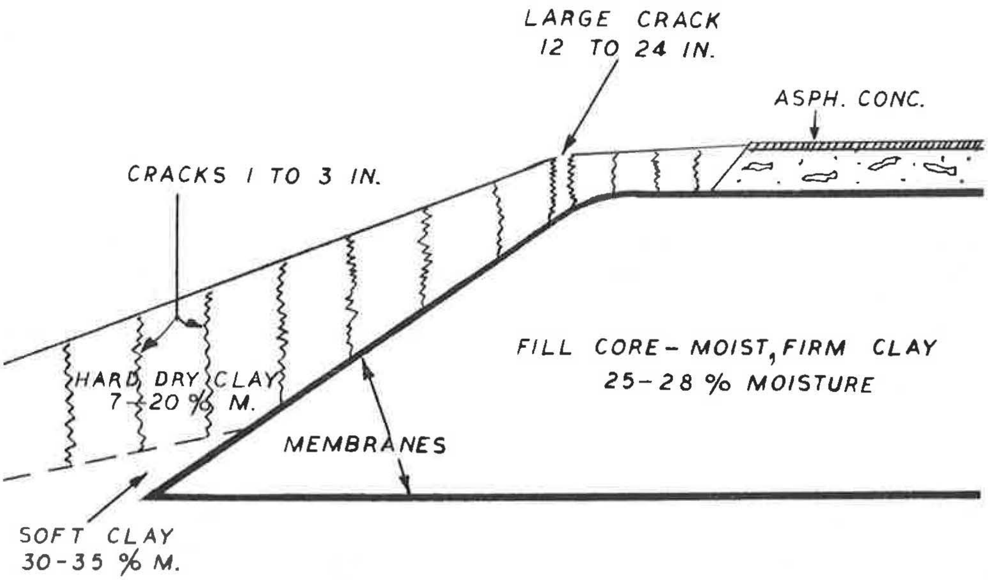

Figure 12. Cross-section near bridge end, Str. 13, summer 1952.

of OA-55 asphalt is sufficient to maintain a continuously stable membrane under adverse conditions, and that such membranes will effectively and permanently stabilize the most plastic soils encountered in the Houston area. Also, there is no reason to believe that such membranes will perform any less efficiently in highly plastic than in moderately plastic soils.

Much of the expressway work in Houston is necessarily accomplished by stage construction. On many such projects built initially several years ago, the original construction has been recently tied into for expansion of these facilities and it has been necessary to make excavations into and through membraned fills. This has afforded opportunities to examine visually the membranes and the protected portions of such fills. In all such cases, the membranes have been found to be intact and the fills in excellent condition; in most cases each individual layer of fill is easily discernible.

SURFACE- AND BURIED-TYPE MEMBRANES

Figures 1 and 2 show that only the envelope-type membranes can afford complete and permanent stabilization of the contained soil. However, the high degree of insurance afforded by envelope membranes is not necessary in all phases of expressway construction, and further examination of Figure 2 shows some examples of buried- and surface-type membranes incorporated in the Gulf Freeway construction. Other arrangements of buried and surface types have been used in other road and street projects to inhibit or prevent edge failures (2, 3). Experience with such single-layer membranes justifies the following conclusions:

1. Single-layer membranes prevent capillary migration of moisture upward through the membrane.
2. Similarly, such membranes prevent passage of surface runoff downward through the membrane.
3. Shrinkage forces during dry seasons stop at the membrane, except at and beyond membrane edges. Hence, the effect of any single-layer membrane placed in the vicinity of pavement edges is to move the edge of the zone of seasonal moisture fluctuation from the pavement edge to the outside edge of the membrane.
4. Shrinkage forces will migrate downward and inward from the membranes outer edge on an angle of up to 45°.

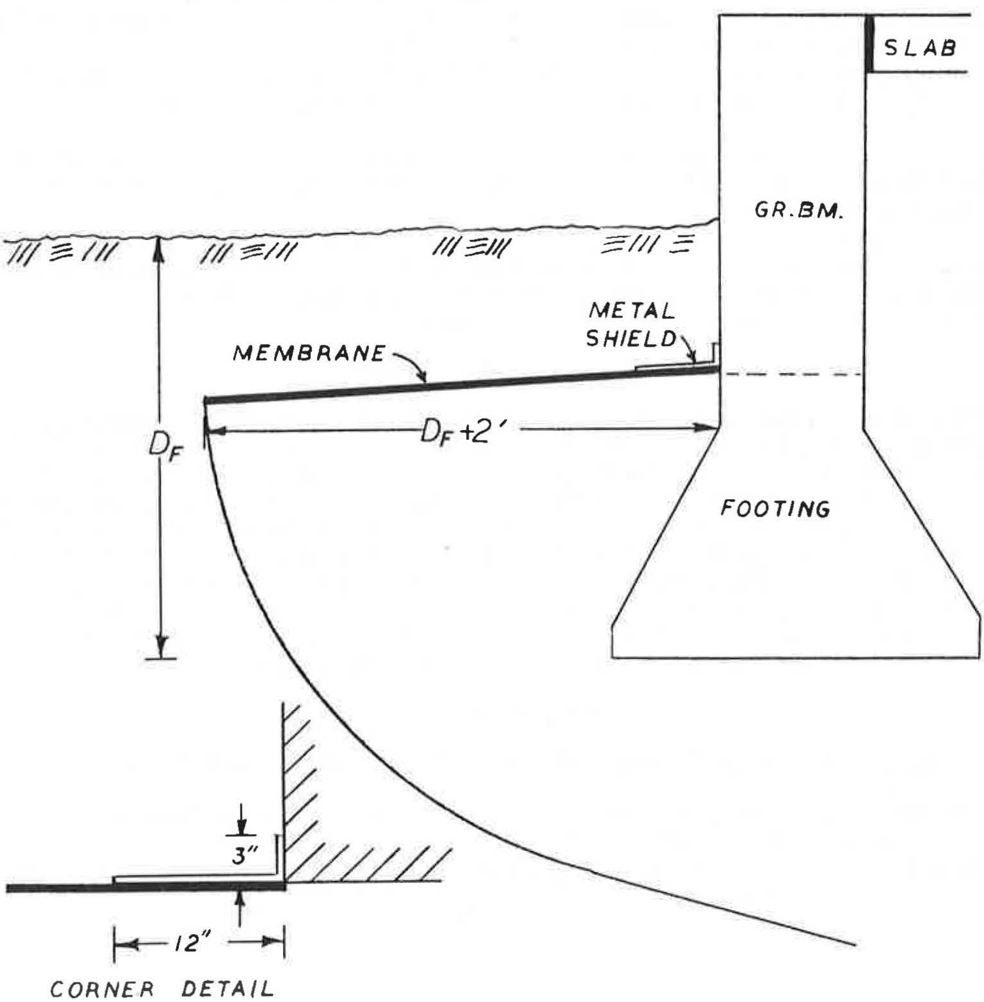

Figure 13. Example of single buried type of asphalt membrane.

5. The zone of effective moisture stabilization afforded by any single-layer membrane is therefore limited to an approximately cylindrical volume contained in vertical section within a quarter circle having center at inner membrane edge and radius equal to the membrane width, as shown in Figure 13.

6. Single-layer membranes are therefore influenced by soil and climatic conditions and the degree of stabilization required or desired. Their design is therefore necessarily predicated on individual requirements and soil conditions in each case. In some cases, for example, the membrane may consist of a small percentage of emulsion or cut-back asphalt mixed with a 4- to 6-layer of soil or base material; in other cases, it may consist of a seal coat over a relatively pervious base course. Other variations are evident.

SUMMARY OF FINDINGS

1. Envelope membranes such as employed in Gulf Freeway abutment fills afford complete and permanent stabilization of plastic soils.

2. Coverage of 1.0 gal per sq yd of OA-55 asphalt is sufficient to provide continuously permanent membranes even under adverse conditions of use.

3. The preceding conclusions are based on observations of moisture content, density, and compressive strength in test fills over a period of 14 years.

4. These findings and conclusions are further confirmed by outward stability as evidenced by observations of expansion joint openings at bridges having membraned and unmembraned abutment fills after 10 to 14 years of service.

5. There is some evidence to the effect that the test fills are consolidating and gaining strength very slowly under the effect of their own weight at constant moisture content.

6. Single-layer membranes (surface and buried types) afford a lesser degree of stabilization, and have a definite range of applicability in expressway and other highway construction.

7. Design criteria have been established and are given in the preceding section (conclusion 5 and Fig. 13) for single-layer membranes, whereby the designer may establish membrane types, widths, and thicknesses consistent with economic considerations and soil conditions in each individual case.

ACKNOWLEDGMENTS

The author is deeply indebted to A. C. Kyser, Engineer-Manager, Houston Urban Expressways, Houston, Texas, for allowing and encouraging the research program described in this paper to be continued after his separation from State service, and for allowing him to examine at intervals the data being accumulated. Also greatly appreciated are the many courtesies and cooperation afforded by Joe M. Battle, Assistant Engineer-Manager, and E. B. Ward, Sr., Laboratory Engineer, and for reviewing drafts of the paper. The writer is indebted also to his present employer, Philip D. Barbard, for encouraging him to prepare this paper and allowing him to take time from more urgent duties to make the additional studies necessary to bring the findings up to date, and to the Asphalt Institute for sponsoring the paper.

REFERENCES

1. Harris, F.A., "Rapid Moisture-Density Test." Roads and Streets, 90:85 (Oct. 1947).
2. Benson, J.R., "The Use of Asphalt Membranes in Highway and Airfield Pavements." Proc. AAPT, 22:56 (1953).
3. Kyser, A.C., Jr., "Design of Cross Section as Influenced by Edge Failure Problems." Texas Highways, 2:No. 1, p. 53 (Nov. 1954).

Studies on Soil-Aggregate-Sodium Chloride-Stabilized Roads in Franklin County, Iowa

J. J. MARLEY, Assistant Soils Engineer, Iowa State Highway Commission; and
J. B. SHEELER, Associate Professor of Civil Engineering, Iowa State University of Science and Technology

Sodium chloride-stabilized roads in Franklin County, Iowa, were studied to gain a better insight into the time rate of material loss from the surface. The roads were located in the same general area and constructed of materials from the same sources so that the main variable was duration of service. The thickness of the soil-aggregate-sodium chloride surface courses and the amount of float material on the surface of the road were determined. The float material was tested in the laboratory to study any gradation changes that might have occurred following displacement from the intact surface course. The component materials of the soil-aggregate mix were tested in the laboratory to determine what changes in physical properties could be attributed to the addition of sodium chloride.

The results of these investigations show that the thickness of the surface course decreases and the amount of float material increases as the period of use increases. These tests also show that the silt-clay content of the float material is less than in the soil-aggregate surface course. The addition of sodium chloride to the soil-aggregate mix caused no noticeable changes in the Atterberg limits of the mix, whereas significant changes in the Atterberg limits of the soil component were noted after the addition of sodium chloride.

• THIS STUDY was conducted to examine the performance characteristics of low-cost roadway surfaces of soil-aggregate-sodium chloride mixtures. Many roads have been successfully stabilized with sodium chloride. However, little information is available on either the properties of the road materials or the effects of sodium chloride on the materials.

The performance of some of the sodium chloride-stabilized roads in Franklin County, Iowa, and the performance of some nearby non-chemically treated roads have been studied. The study of sodium chloride-stabilized roads was restricted to those in which the binder soil used in construction came from the same source. The effects of sodium chloride on some of the engineering properties of the soil and soil-aggregate mixtures used were studied in the laboratory.

Paper sponsored by Committee on Soil-Sodium Chloride Stabilization.

LITERATURE REVIEW

Sodium chloride has no cementing properties, and its use as a soil stabilizer is restricted to soils that are mechanically stable themselves. Such soils are soil-aggregate mixtures used as subbase courses, base courses, and low-cost roadway surface courses.

Strahan (18) made the first systematic study of soil-aggregate surface courses during the 1920's, although the art of soil-aggregate stabilization had been practiced by the Romans.

The theory of soil-aggregate stabilization has been stated, and the design and construction of soil-aggregate mixtures has been surveyed (7, 24). Standard specifications for the design of soil-aggregate mixtures have been published (1).

Sodium chloride has never been widely used as a dust control because it is only slightly hygroscopic (23) and also because considerable corrosion potential was attributed to it (8). Its use as a soil-stabilizing agent has been reported in widely varying geographical locations in the United States (13).

Possible factors in the mechanism of sodium chloride stabilization have been set forth by several authors (12, 16, 23). Some of these are increased density of soil-aggregate, low permeability due to clay expansion, moisture retention, lowered freezing point of the solution of sodium chloride in water, recrystallization of sodium chloride, increase in cohesion of clay due to sodium ions, increase in surface tension of water, and increase in solubility of carbonate minerals.

The relative effects of these mechanisms have never been fully evaluated. However, investigation of the effects in general of sodium chloride on the physical properties of soil-aggregate mixtures were made at Iowa State University (5). The addition of sodium chloride was found to increase the maximum Proctor density of the mix at a lower optimum moisture content. No other significant changes in the physical properties of the mixtures were found.

LOCATION OF ROADS AND MATERIALS STUDIED

The roads studied are located in Franklin County, Iowa, in the north central part of the State (Fig. 1). According to the Iowa Geological Survey (9), the county is divided into two areas of glacial drift. The western five-eighths of the county is covered by Cary glacial drift, and the eastern three-eighths by Iowan glacial drift.

Location of Roads

Sodium chloride-stabilized roads were constructed in Franklin County in 1956, 1957, 1958, and 1959. One road was chosen for investigation for each year of construction (Fig. 2):

1. Road S-4 1956.—Starting at the northeast corner of section 10 in Oakland township going west to the northwest corner of the northeast quarter of section 7 (NE cor NE ¼ 90-22-10 to the NW cor NE ¼ 90-22-7). This section of road is 3.5 mi long.
2. Road S-3 1957.—Starting at the northeast corner of section 12 in Lee township going west to the northwest corner of section 11 in Oakland township (NE cor NE ¼ 90-21-12 to the NW cor NW ¼ 90-22-11). This section of road is 8 mi long.
3. Road S-2 1958.—Starting at the northeast corner of section 10 in Oakland township going south of the southeast corner of the northeast quarter of section 22 (NE cor NW ¼ 90-22-10 to the SE cor NE ¼ 90-22-22). This section of road is 2.5 mi long.
4. Road S-1 1959.—Starting at the southeast corner of section 3 in Oakland township going north to the northwest corner of section 35 in Morgan township and then east to the northeast corner of section 35 and then north to the northeast corner of section 25 in Morgan township (SE cor SE ¼ 90-22-3 to the NW cor NW ¼ 91-22-35 to the NE cor NE ¼ 91-22-35 to the NE cor NE ¼ 91-22-25). This section of road is 4 mi long.

Roads that had no sodium chloride or binder soil mixed with the aggregate of the surface course were also investigated. These nonstabilized roads surfaced with gravel only were also constructed in 1956, 1957, 1958, and 1959 (Fig. 3):

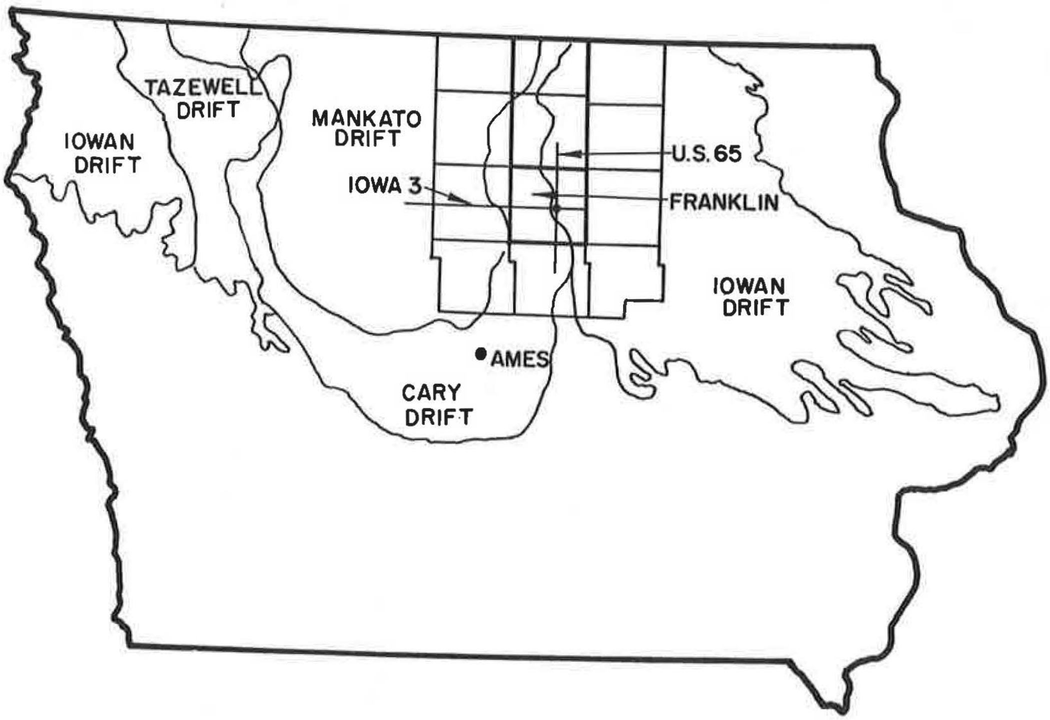

Figure 1. Map of Iowa showing Franklin County and Wisconsin glacial drift borders.

1. Road G-4 1956.—Starting at the northwest corner of section 8 in Oakland township going south to the southwest corner of section 17 and then east to the southeast corner of section 17 (NW cor NW $^{1}/_{4}$ 90-22-8 to the SW cor SW $^{1}/_{4}$ 90-22-17 to the SE cor SE $^{1}/_{4}$ 90-22-17). This section of road is 3 mi long.

2. Road G-3 1957.—Starting at the northeast corner of section 12 in Oakland township going south to the southeast corner of section 25 (NE cor NE $^{1}/_{4}$ 90-22-12 to the SE cor SE $^{1}/_{4}$ 90-22-25). This section of road is 4 mi long.

3. Road G-2 1958.—Starting at the northeast corner of section 4 in Oakland township going south to the southeast corner of section 4. Also starting at the northwest corner of section 4 and going to the southwest corner of section 4. (NE cor NE $^{1}/_{4}$ 90-22-4 to the SE cor SE $^{1}/_{4}$ 90-22-4 also NW cor NW $^{1}/_{4}$ 90-22-4 to the SW cor SW $^{1}/_{4}$ 90-22-4). The total length of these two sections of road is 2 mi.

4. Road G-1 1959.—Starting at the northeast corner of section 2 in Oakland

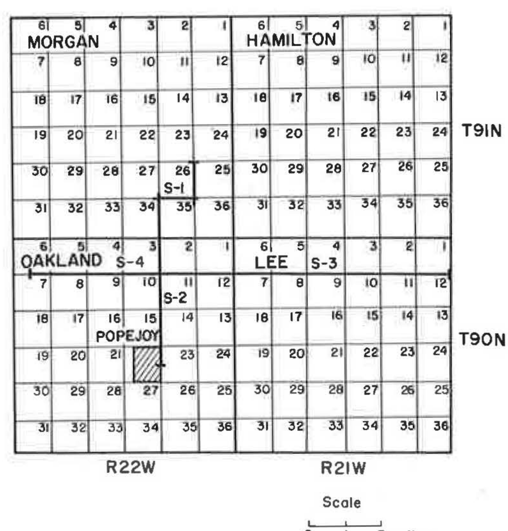

Figure 2. Map of four townships in southwestern Franklin County locating soil-aggregate-sodium chloride-stabilized roads studied.

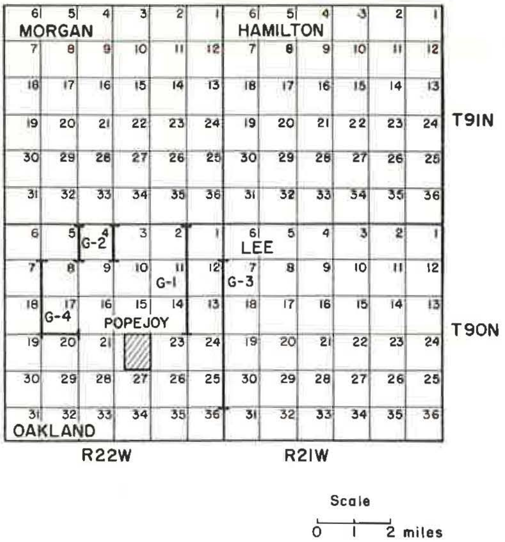

Figure 3. Map of four townships in southwestern Franklin County showing non-chemically treated roads studied.

township going south to the southwest corner of section 14 (NE cor NE 1/4 90-22-2 to the SE cor SE 1/4 90-22-14). The length of this section of road is 3 mi.

Sources of Materials

The materials used to surface these roads were locally available gravels and glacial clay. The sodium chloride mixed with the soil materials was a crushed rock salt imported from St. Louis, Mo. The gravel came from two sources located within Franklin County. All the binder soil used came from one source (Fig. 4).

The first aggregate source (A-1) is located in Reeve Township in the northwest quarter of section 21 (NW 1/4 91-20-21). The second source of aggregate (A-2) is located in Morgan Township in the southwest quarter of section 30 (SW 1/4 91-22-30). The binder soil (C-1) is located in Osceola township in the northwest corner of section 20 (NW cor NW 1/4 90-19-20).

Aggregate A-2 was used on all the non-stabilized roads investigated and on stabilized roads S-1, S-2, and S-4. It was also used on the western 3 mi of stabilized road S-3 (from the NW cor NW 1/4 90-22-11 to the NW cor NW 1/4 90-21-7). Aggregate A-1 was used for the eastern 5 mi of stabilized road S-3 (from the NE cor NE 1/4 90-21-12 to the NW cor NW 1/4 90-21-8).

Geological and Climatological Considerations

The area in which the roads are located is covered by Cary glacial drift. The principal soil association is the Clarion-Webster association (19). The aggregate sources also lie within the area covered with Cary drift, and the source of the binder soil is in a part of the county covered with Iowan drift.

The mean annual rainfall in Franklin County is 31 in., and the mean annual

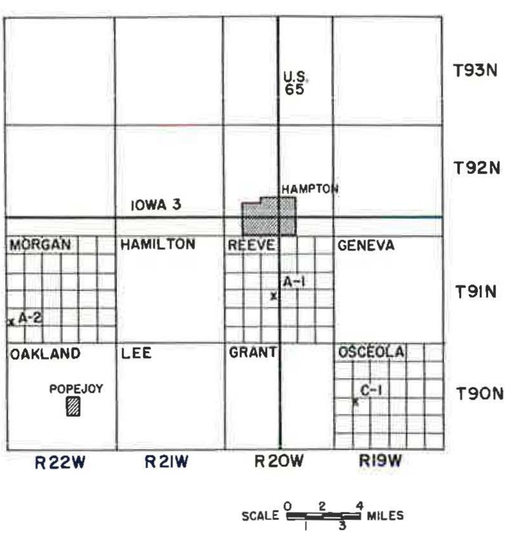

Figure 4. Map of Franklin County showing sources of materials used in construction of roads studied.

TABLE 1

RAINFALL AND EXTREME TEMPERATURES RECORDED AT HAMPTON, IOWA, IN THE PERIOD 1956-1959[a]

Year	Temperature (°F)			Rainfall (in.)
	Highest Recorded	Lowest Recorded	Max. Annual Diff.	
1956	96	-23	119	24.6
1957	95	-18	113	31.8
1958	92	-20	112	22.2
1959	93	-21	114	30.7

[a]From records of U.S. Weather Bureau (20, 21, 22).

temperature is 45 F. Rainfall and temperature data recorded in the period 1956-1959 at Hampton, Iowa, the county seat of Franklin County, are given in Table 1.

DESIGN, CONSTRUCTION AND MAINTENANCE OF ROADS

Design of Stabilized Roads

The soil-aggregate-sodium chloride-stabilized surfaces were designed to meet the gradation and plasticity requirements of the Iowa State Highway Commission standard specifications (11). The thickness of the stabilized surface was designed to be 4 in. and the width 24 ft. The road had a maximum crown of 6 in. in 12 ft and a minimum crown of 4 in. in 12 ft. Aggregate A-1 was mixed with 20 percent soil C-1 by dry weight of the aggregate. Aggregate A-2 was mixed with 12.5 percent soil C-1 by dry weight of the aggregate. These proportions of soil and aggregate produced mixtures that meet the gradation and plasticity specifications. The specifications require a plasticity index for the soil aggregate mixture of between 5 and 12 percent.

A surface course 4 in. thick, 24 ft wide, and 1 mi long contained 1,564 cu yd of soil-aggregate. At the specified rate of 12 lb per cu yd of compacted mixture, 9.39 tons of sodium chloride would be required per mile. The application rate was 10 tons of sodium chloride, which was slightly in excess of the specified minimum for a surface course. Compacted to a density of 130 pcf, the total weight of the material in such a surface course would be 2,746 tons. Based on the dry weight of the soil-aggregate 0.36 percent of sodium was added. An addition of 0.5 percent has been found to be the optimum addition for reducing the optimum moisture content and increasing the maximum density (5).

Construction of Stabilized Roads

A road mix type of construction was used. The aggregate and binder soils were first spread on the road, then dry sodium chloride was distributed with a mechanical spreader. The soil, aggregate, and sodium chloride were mixed in place. Sufficient water was added to bring the mixture to the optimum moisture content, and the materials were given a final mixing with a single rotor-type rotary mixer.

The materials were then spread, shaped, and compacted. The crown was shaped just before the final compaction of a section. The specifications required that the mix be compacted to 95 percent of the standard Proctor density for the mix.

Maintenance of Stabilized Roads

In the regularly scheduled maintenance program of Franklin County, further chemical surface treatment and blading is required. Three tons of calcium chloride pellets (75 to 80 percent pure) per mile were added. This serves as a dust control. The calcium chloride is spread with a mechanical spreader.

Blading is carried out only when indicated by the surface condition of the road and then only after a rainfall. Blading after a rainfall prevents the surface of the road from being unduly disturbed and also insures that the loose material which is bladed to the heavily traveled area of the road will have sufficient moisture for recompaction (13).

The chemical surface treatment maintenance program was actually carried out on roads S-3 and S-4 in 1958 and on roads S-1, S-2, and S-4 in 1959. In other years, the chemical surface treatment was omitted because of insufficient county maintenance funds.

Construction and Maintenance of Nonstabilized Roads

Construction and maintenance procedures of non-chemically treated surface courses were not as closely controlled as those on stabilized surface courses. The surfacing materials were hauled from the source, placed on the grade, and spread with a blade grader. Compaction was left to the action of traffic, with no moisture, density, or thickness controls exercised.

No chemical was applied for dust control, and the roads were bladed whenever necessary, without regard to moisture.

INVESTIGATIONS

Field

Two types of data were obtained directly from measurements in the field: the thickness of the soil-aggregate surface, and the amount of loose or float material on the surface.

A trench sample was dug one-half the width of the surface course (3). The depth of the surface course was measured at 1-ft intervals from the shoulder to the centerline of the road. The thickness of the surface course was measured at 1-mi intervals, and the side of the road on which the trench was cut was alternated at each sample location. The average thickness of the surface course at a trench sample location is given by

$$t_a = \frac{\frac{1}{2} t_1 + t_2 + t_3 + \ldots + \frac{1}{2} t_n}{n-1} \qquad (1)$$

in which t is the thickness of the surface, the subscripts refer to the position of the thickness measurement (transverse to the centerline of the road), and n is the number of points at which the surface course thickness was measured.

The amount of float material on the surface of the road was measured by collecting the loose material within the area of a wooden template with a soft broom. The template facilitated the gathering of data by providing a constant sampling area. A broom with soft bristles was used to minimize any loosening of firmly held material. The rectangular 3- by 6-ft template was constructed of 1- by 2-in. lumber with corner braces to prevent distortion. The float material was swept into containers and weighed in the field. A representative sample was taken for laboratory testing, and the rest of the material returned to the surface of the road.

Other field work included the gathering of samples of aggregate and binder soil for laboratory determinations of Atterberg limits, mechanical analyses, and other tests.

Laboratory

The float material, the aggregates, the binder soil, and the soil-aggregate mix were investigated in the laboratory. The laboratory tests on the float material from the soil-aggregate-sodium chloride surface courses included a mechanical analysis of the material from each sample location.

Various soil classification and identification tests were conducted on the binder soil. These tests were performed in accordance with the standard ASTM procedures and included mechanical analysis, Atterberg limit tests, pH determination, organic matter determination, and X-ray diffraction analysis to determine the predominant minerals (1, 2).

The tests run on the gravels included mechanical analyses, abrasion tests, and soundness tests. These tests were performed in accordance with ASTM procedures. The abrasive resistance of the aggregates was determined by the use of the Los Angeles machine and the soundness of the aggregates was determined by the use of a saturated solution of sodium sulfate. Tests on the soil-aggregate mixtures were performed on the mixtures combined in the proportions for which they had been designed. Tests on these mixtures included mechanical analyses, Atterberg limit tests, and moisture-density relationships with and without the addition of 0.5 percent sodium chloride.

To determine the effect of sodium chloride on the engineering properties of the binder soil C-1, varying percentages of sodium chloride and water were added and the mixture was tested. Atterberg limit tests and X-ray diffraction analyses were performed on these mixtures. The X-ray diffraction analyses were run on dry soil-sodium chloride mixtures, on mixtures containing water, and on mixtures containing ethylene glycol.

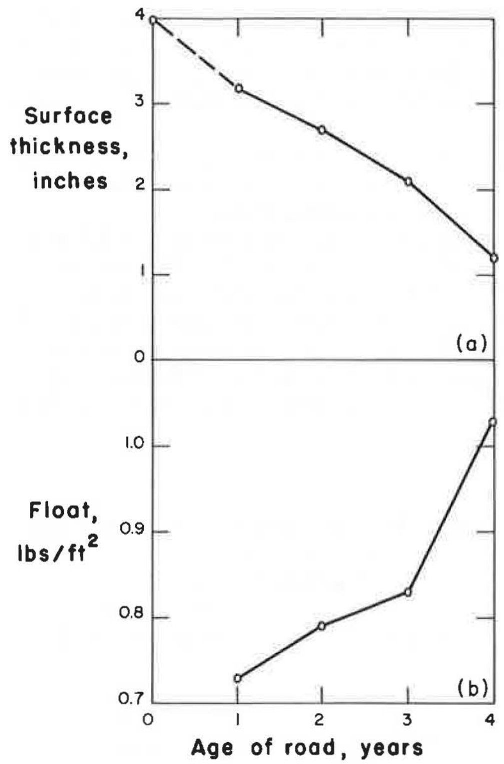

Figure 5. Field data: (a) thickness of surface course vs age of road; (b) amount of float material vs age of road.

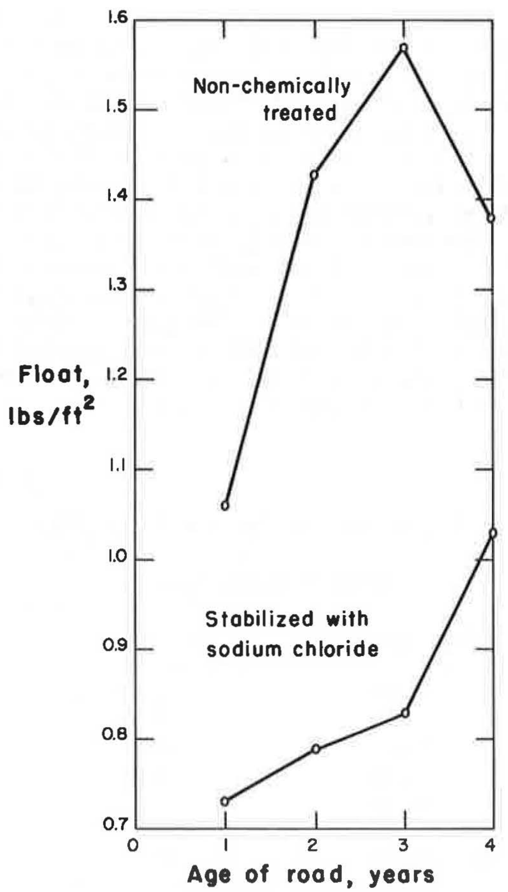

Figure 6. Amount of float material vs age of road.

RESULTS

Field Investigations

The data collected in the field are plotted in Figure 5. Figure 5a shows the thickness of the surface course vs the age of the road; Figure 5b shows the amount of float material vs the age of the road. These curves show that the thickness of the stabilized surface course decreases with time, and the amount of float material per square foot of roadway surface increases with time.

The data show that the float material on non-chemically treated roads increased for three years and then decreased. Figure 6 shows the amount of float material vs the age of the road. The upper curve is for the non-chemically treated surfaces and the lower curve is for the soil-aggregate-sodium chloride stabilized surfaces. These curves indicate that the soil-aggregate-sodium chloride surfaces have less float material than the non-chemically treated surfaces.

The curves in Figures 5 and 6 do not include data from that portion of road S-3 in which aggregate A-1 was used. Thus these curves include only the data from roads on which the same aggregate was used.

Table 2 compares surface thickness and float material on road S-3 for the different soil-aggregate.

Laboratory Investigations

The various size fractions of the float material are plotted vs road age in Figure 7, which does not include the data from the portion of road S-3 where aggregate A-1 was used.

These curves show that after the road has been used for one year the fine fraction (the silt and clay) in the float material is about 60 percent of what it was in the original mix. After the first year, the fine fraction of the float material remains nearly constant. This should be expected, because the fine fraction in the float material is readily transported by the wind and is quickly blown away.

The sand and gravel contents of the float material seem not to have a direct relationship to the age of the road. However, the percentage of gravel tends to increase and the percentage of sand to decrease with the increasing age of the road.

The results of the laboratory tests performed on the soil C-1 are shown in Table 4. X-ray diffraction analysis of the fraction of the soil C-1 passing the No. 200 sieve showed that the predominant minerals in this material are montmorillonite, illite, quartz, and feldspars. The portion of the road constructed with aggregate A-1 and 20 percent binder soil showed both a greater surface thickness and a greater amount of float material than did the section constructed using aggregate A-2 and 12.5 percent soil C-1. The properties of aggregates A-1 and A-2 are summarized in Table 3. The

TABLE 2

COMPARISON OF FLOAT MATERIAL AND SURFACE THICKNESS ON ROAD S-3

	Float Material (psf)		Thickness (in.)	
	Aggregate A-1	Aggregate A-2	Aggregate A-1	Aggregate A-2
	1.16	0.34	2.3	1.8
	1.04	0.62	2.1	1.6
	0.78	0.64	2.2	1.9
	0.88	0.48	1.8	1.4
	0.65	--	2.1	--
	0.47	--	2.4	--
Avg.	0.83	0.52	2.1	1.8

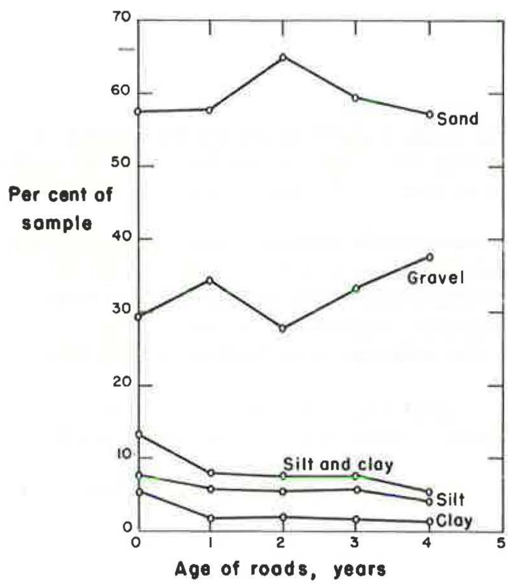

Figure 7. Various size fractions of float material vs age of road.

TABLE 3

ENGINEERING PROPERTIES OF AGGREGATES A-1 AND A-2

Property	Value	
	A-2	A-1
Gravel (%)	33	37
Sand (%)	59	53
Silt (%)	5	5
Clay (%)	3	5
Soil class.	A-1-b(0)	A-1-b(0)
Los Angeles abrasion loss[a] (%)	48.2	35.7
Soundness test[b] (%)	9.58	7.97
Plasticity index[c]	Nonplastic	Nonplastic

[a]ASTM designation C 131-55T.
[b]ASTM designation C 88-59T. (loss after 5 cycles).
[c]Test run on that fraction passing No. 40 sieve.

engineering classification of both of these aggregates is A-1-b(0). This type of material is well graded, and except for the plasticity requirements it would meet the specifications of the Iowa State Highway Commission for soil-aggregate surface courses. Both aggregates are nonplastic. Aggregate A-1 showed a greater resistance to abrasion and more resistance to the sodium sulfate test than did aggregate A-2. Aggregate A-1 showed an abrasion loss of 35.7 percent compared with a loss of 48.2 percent for aggregate A-2. The loss after 5 cycles of immersion in a saturated solution of sodium sulfate and oven-drying was 7.97 percent for aggregate A-1 and 9.58 percent for aggregate A-2. These tests indicate that aggregate A-1 is less susceptible to being broken up by the action of traffic and is probably less susceptible to the action of freezing and thawing than is aggregate A-2.

The physical and chemical properties of soil C-1 are given in Table 4. The results of the tests performed in the soil-aggregate mix are given in Tables 5 and 6. The mechanical analysis of the soil-aggregate mix with aggregate A-1 shows that it contains 11 percent 5-μ clay (material finer than 0.005 mm) and the soil-aggregate mix with aggregate A-2 contains 6 percent clay. Both soil-aggregate mixtures contain 30 percent gravel. The plasticity index of the A-1 mix is 5 percent and of the A-2 mix 7 percent. The Atterberg limits of these soil-aggregate mixtures are unchanged by the addition of 0.5 percent sodium chloride. The addition of 0.5 percent sodium chloride caused an increase in the density of 2.5 percent for the A-1 mix and 2.8 percent for the A-2 mix.

The results of the Atterberg limit test on soil C-1 and soil-sodium chloride mixtures are given in Table 7. Figure 8 shows these results vs the percent of sodium chloride added. The addition of 0.5 percent sodium chloride by dry weight of the soil caused a slight increase in the liquid limit of the soil. The liquid limit of the soil with the addition of 1 percent sodium chloride is nearly the same as it is with the addition of 0.5 percent sodium chloride. Additions of sodium chloride above 1 percent cause the liquid limit of the soil to decrease. The addition of sodium chloride caused the plastic limit of the soil to increase at a nearly constant rate.

TABLE 4

PHYSICAL AND CHEMICAL PROPERTIES OF SOIL C-1

Property	Value
Gravel (greater than 2.0 mm) (%)	3
Sand (2.0 mm to 0.074 mm) (%)	55
Silt (0.074 mm to 0.005 mm) (%)	21
Clay (less than 0.005 mm) (%)	21
Liquid limit (%)	28
Plastic limit (%)	14
Plasticity index (%)	14
Carbonate content (%)	2.35
Organic matter (%)	0.11
Cation exchange capacity[a]	11.5
pH	8.1
Predom. clay minerals[b]	Montmorillonite, illite
Other predom. minerals[b]	Quartz, feldspars
Eng. soil class.	A-6(3)
Textural soil class.	Sandy clay loam

[a]Milliequivalents per 100 g.
[b]From X-ray diffraction analysis.

TABLE 5

SOME ENGINEERING PROPERTIES OF AGGREGATE A-2 PLUS 12.5 PERCENT SOIL C-1[a]

Property	Addition of Sodium Chloride	
	0.0	0.5
Optimum moisture content (%)	9.5	8.7
Standard Proctor density (pcf)	128.7	132.3
Liquid limit (%)	22	22
Plastic limit (%)	15	15
Plasticity index (%)	7	7

[a]Size fractions in mixture: gravel, 30 percent; sand, 57 percent; silt, 7 percent; and clay, 6 percent.

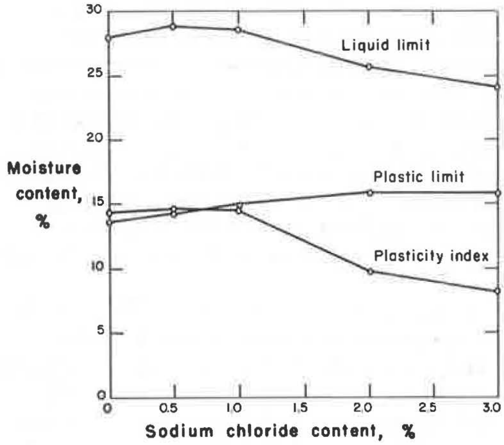

Figure 8. Atterberg limits vs percent of sodium chloride added.

TABLE 6

SOME ENGINEERING PROPERTIES OF AGGREGATE A-1 PLUS 20 PERCENT SOIL C-1[a]

Property	Addition of Sodium Chloride	
	0.0%	0.5%
Optimum moisture content (%)	11.2	9.6
Standard Proctor density (pcf)	131.1	134.3
Liquid limit (%)	21	21
Plastic limit (%)	16	16
Plasticity index (%)	5	5

[a]Size fractions in mixture: gravel, 30 percent; sand, 53 percent; silt, 6 percent; and clay, 11 percent.

The net effect on the plasticity index of the soil is that the plasticity index remains nearly constant for additions of sodium chloride up to 1 percent, due to the increase in both the liquid limit and the plastic limit. When amounts of sodium chloride above 1 percent are added, however, the plasticity index of the soil is decreased from 14 at 1 percent to 8 with the addition of 3 percent sodium chloride. This decrease in plastic limit is caused both by the decreasing liquid limit and the increasing plastic limit.

X-ray diffraction analyses of soil-sodium chloride mixtures passing the No. 200 sieve did not show any effect on the minerals due to the addition of sodium chloride, nor did these analyses show any crystalline sodium chloride in the mixtures.

ANALYSIS OF RESULTS

Field Investigations

The increase in the amount of float material on the surface of the soil-aggregate-sodium chloride-stabilized surface courses would probably continue until the surface course was worn away and only loose aggregate particles remained on the roadbed. If the surface course were completely worn away, there would be no further source for float material. Thereafter, as the float material is removed from the surface of the road by wind, water, or action of traffic, the amount of float material on the road surface would decrease. It was not possible to verify this hypothesis in the field because no soil-aggregate-sodium chloride-stabilized surface course was constructed in Franklin County before 1956.

The amount of float material on non-chemically treated roads increased for three years, and then it decreased. The decrease indicates that all the aggregate on the road surface is present as float material. As this float material is removed from the surface of the road, it is not replaced from a surface course. It was not possible to evaluate the amount of float material on non-chemically treated roads in Franklin County further, because on no roads suitable for investigation had aggregate been added more than four years

TABLE 7

RESULTS OF ATTERBERG LIMIT TESTS FOR THE FRACTION OF SOIL C-1 PASSING NO. 40 SIEVE WITH VARYING AMOUNTS OF SODIUM CHLORIDE

Percent NaCl Added	Atterberg Limit (%)		
	Liquid Limit	Plastic Limit	Plasticity Index
0.0	28	14	14
0.5	29	14	15
1.0	29	15	14
2.0	26	16	10
3.0	24	16	8

earlier. The only aggregate on road G-4 was the float material. The thickness of the surface course on this road was virtually zero, and the surface of the road was markedly darker in color than other roads because of the exposure of the dark subgrade soil.

The reason for the increase in the amount of float material on these roads is believed to be as follows. The aggregate particles on the surface of the road are loosened from the soil matrix by the action of traffic. Eventually, these particles become completely free of the soil matrix and lie on the surface of the road. This leaves on the surface of the road a void space that was previously occupied by an aggregate particle. The fine soil surrounding this hole is then laterally unsupported, and the sides are caved in by the action of traffic. The loose fine material is picked up by the action of traffic, thrown into the air, and blown away from the road. This action leaves other aggregate particles exposed on the surface of the road and subject to the same cycle of events. Thus the thickness of the stable portion of the surface course is decreased.

The coarse aggregate particles loosened from the surface accumulate on the surface of the road; this accounts for the increase in the amount of float material. Some of these particles, however, are either thrown to the shoulders of the road or are ground into smaller particles. These smaller particles could then be blown away in the same manner as the fine soil. This would account for the reduction in the amount of float material after the source is depleted.

The fine particles in the float material are reduced in the first year, and thereafter the fine fraction of the float material is nearly constant (Fig. 7). The fine fraction of the float material is quickly removed from the surface of the road by wind action. In the non-chemically treated roads the process is probably accelerated, because there is no binder soil added to the aggregate. The surface is readily disturbed by the action of traffic, and any fine binder material is removed rather quickly, leaving the aggregate particles free to be easily thrown off the road or ground into smaller particles.

After four years of use, the stabilized road S-4 had a surface course 1 to 2 in. thick, and the non-chemically treated road G-4 had only float material remaining. These findings are in close agreement with results reported on data from Butler County, Iowa (16). Non-chemically treated roads were worn out in approximately four years, and the soil-aggregate-sodium chloride surface courses were 1 to 2 in. thick at the end of four or five years.

The portion of road S-3 constructed with aggregate A-1 had a greater surface thickness and a greater amount of float material than that constructed with aggregate A-2 after three years of use. Aggregate A-1 was mixed with a greater proportion of binder soil in construction of the surface course. The use of a greater percentage of binder soil C-1 probably increased the resistance to loosening of the aggregate particles from the soil-aggregate A-1 mix. The soil-aggregate A-1 mix contained 11 percent clay compared with 6 percent clay in the soil-aggregate A-2 mix. If this greater percentage of clay made it more difficult for the aggregate particles to be removed from the road surface, then the rate of wearing would be reduced, and this road (S-3 using aggregate A-1) would have a greater remaining thickness of surface course.

Also, after the aggregate particles had been loosened from the soil matrix, the greater resistance to abrasion exhibited by aggregate A-1 would prevent it from being ground into smaller particles as readily as aggregate A-2.

Laboratory Investigations

A reduction of the plasticity index of the binder soil C-1 could be brought about by the addition of a sufficient amount of sodium chloride (Fig. 8). The data indicate that 1.5 to 2.0 percent sodium chloride is needed to cause an appreciable effect. The reduction of plasticity index is due both to a reduction of the liquid limit and an increase in the plastic limit. Most of the reduction in plasticity index is due to lowering the liquid limit. It is desirable in soil stabilization to reduce the plasticity index by raising the plastic limit.

The amount of sodium chloride necessary to affect noticeably the plastic properties of the soil is greater than that usually added to soil-aggregate mixtures. The addition of 0.5 percent sodium chloride had no effect on the liquid and plastic limits of the soil-aggregate mixtures (Tables 5, 6). The percentage of sodium chloride added to the soil-aggregate mix is computed on the basis of the dry weight of the mix (5). The Atterberg limits are then run on the fraction passing the No. 40 sieve. This test procedure, which introduces sand into the soil fraction on which the test is being run, may account for the fact that the plastic properties do not change when sodium chloride is added. If the Atterberg limits were determined only for the silt and clay fraction of the soil-aggregate mix, the addition of 0.5 percent sodium chloride to the soil aggregate would most likely affect the plastic properties of this portion. A change in the basis of computation shows that 0.5 percent of the total soil-aggregate is 2.9 percent of the silt-clay fraction of the A-1 plus C-1 soil-aggregate mix and 3.8 percent of the silt-clay fraction of the A-2 plus C-1 soil-aggregate mix (Fig. 8).

The changes in the plastic properties noted in the soil-sodium chloride mixtures are probably due in part to the ion exchange which takes place when sodium chloride is added to the soil (6), but these effects have not been evaluated as a part of this investigation. Part of the reduction of the liquid limit is possibly due to the partial dispersion of the clay fraction of the soil by sodium chloride (23). The particles in such a dispersed system are able to slide over one another at a lower moisture content than is necessary in a system that is not dispersed.

The mechanism causing the rise in the plastic limit of the soil which accompanies an increase in sodium chloride content may be hypothesized through physico-chemical considerations. The plastic limit of a soil is defined as the minimum moisture content at which the soil-water system behaves as a plastic solid. In order for the material to behave in this manner, the soil particles must be bound together by interparticle forces which when totaled can be expressed as some given amount of energy per unit weight of dry soil. This energy is supplied for the most part by the surface tension of the liquid in the capillary spaces of the soil. Any change in the surface tension of the liquid causes a change in the interparticle forces, as does any change in the size of the capillary spaces. Because an increase in the sodium chloride concentration causes an increase in the plastic limit, the size of the capillary spaces must necessarily increase. An increase in the volume of water per unit weight of soil means that the space occupied by water must become larger. If an increase in volume of water per unit weight of soil is accompanied by an increase in the surface tension of the liquid, it is possible to maintain a constant total interparticle energy per unit weight. This can be done by maintaining the proper ratio of the amount and concentration of sodium chloride solution to capillary size. A soil containing a sodium chloride solution as the liquid should therefore exhibit a higher value for the plastic limit because more solution is needed to maintain a constant ratio of total surface tension per unit weight to total volume of liquid per unit weight.

X-ray diffraction analyses of the float material and of samples from the soil-aggregate-sodium chloride surface course did not show any recrystallized sodium chloride. This is probably because any crystalline sodium chloride present in these samples was in small amount and as crystals was too small to be detected by the procedures used. The same results were obtained with soil-sodium chloride mixtures prepared in the laboratory. No changes in the spacing of the clay mineral lattice were noted after the addition of sodium chloride to the binder soil in the laboratory.

The most significant contribution made by sodium chloride to the stability of soil-aggregate mixtures is probably the stabilization of the moisture content of the mixture. A soil-aggregate mix without a chemical additive is relatively impermeable; however, the addition of sodium chloride should decrease the permeability of the mix by causing greater clay expansion when wet. Sodium chloride should also prevent the moisture content of the soil-aggregate mix from falling too low during periods of dry weather. The mechanisms of this effect are the lower vapor pressure of the sodium chloride solution causing a lower rate of evaporation, the crystallization of sodium chloride in void spaces preventing evaporation of moisture from some of the void spaces, and the hygroscopicity of sodium chloride.

If the moisture content of a soil-aggregate surface course can be kept within limits so that the moisture content in the mix never becomes low enough to allow raveling or dusting on the surface, or great enough to permit the normally applied loads to rut the surface, a conservation of material and more desirable riding qualities of the road will result.

CONCLUSION

The durability of soil-aggregate-sodium chloride surfaces is affected by the quality of aggregate and the amount of clay-size material used in the construction of the surface course. The addition of sodium chloride causes a reduction in the liquid limit and an increase in the plastic limit of the glacial soil studied in this investigation and contributes to the stability of soil-aggregate mixtures.

ACKNOWLEDGMENTS

The subject matter of this paper was obtained as part of the work being done by Soil Research Laboratory, the Iowa Engineering Experiment Station, Iowa State University under sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission.

Special appreciation is extended to W. H. Jorgenrud, County Engineer, Franklin County, Iowa, for the information on the construction of the road surfaces and for the opportunity to conduct field tests and gather samples, and to the members of the Iowa Engineering Experiment Station Staff who assisted in gathering samples and field performance information.

REFERENCES

1. "1958 Book of ASTM Standards Including Tentatives. Part 4: Cement, Concrete Mortars, Road Materials, Waterproofing, Soils." ASTM (1958).
2. "1959 Supplement to Book of ASTM Standards Including Tentatives. Part 4: Cement, Concrete, Mortars, Road Materials, Waterproofing, Soils." ASTM (1959).
3. Baylard, E. M., "Performance Study of Calcium-Chloride Treated Roads." HRB Proc., 31:336-348 (1952).
4. Belcher, D. J., "A Field Investigation of Low Cost Stabilized Roads." Purdue University, Engineering Experiment Station, Research Series Bull. 81 (1941).
5. Gow, A. J., Davidson, D. T., and Sheeler, J. B., "Relative Effects of Chlorides, Lignosulfonates and Molasses on Properties of a Soil-Aggregate Mix." HRB Bull. 282, 66-83 (1961).
6. Grim, R. E., "Clay Mineralogy." McGraw-Hill (1953).
7. Huang, E. Y., "Manual of Current Practice for the Design, Construction and Maintenance of Soil-Aggregate Roads." Department of Civil Engineering, University of Illinois (1959).
8. Hubbard, P., "Dust Preventives and Road Binders." Wiley (1910).
9. "Annual Report, 1905." Iowa Geological Survey (1906).
10. "An Engineering Report on the Soils, Geology, Terrain and Climate of Iowa." Iowa State Highway Commission (1960).
11. "Standard Specifications for Construction on Primary, Farm to Market, and Secondary Roads and Maintenance Work on the Primary Road System." Iowa State Highway Commission (1960).
12. Looker, C. D., and Spencer, W. T. "Use of Sodium Chloride in Road Stabilization." HRB Proc., 18: pt. 2, pp. 257-274 (1938).
13. "Salt for Road Stabilization." Salt Institute, Chicago (1960).
14. Sanborn, A. F., "An Annotated Bibliography on Soil Stabilization with Sodium Chloride Including a Review of the Literature." Internat. Salt Company, Scranton (1956).
15. Sheeler, J. B., "Sodium Chloride and Calcium Chloride Treatment of Secondary Roadway Surfaces." Iowa State University, Engineering Experiment Station (1959).
16. Sheeler, J. B., "Sodium Chloride Stabilized Roads in Iowa." HRB Bull. 282, 59-65 (1961).
17. Spangler, M. G., "Soil Engineering." 2nd ed., Internat. Text Book Co. (1960).

18. Strahan, C. M., "A Study of Gravel, Topsoil and Sand Clay Roads in Georgia." Public Roads, 10:117-136 (1929).
19. "Soil Survey of Franklin County, Iowa." U. S. Department of Agriculture (1938).
20. "Climatological Data, Iowa Section." Vol. 67, 1956, U.S. Department of Commerce (1957).
21. "Climatological Data, Iowa Section." Vol. 67, 1957, U. S. Department of Commerce (1958).
22. "Climatological Data, Iowa Section." Vol. 67, 1958, U. S. Department of Commerce (1959).
23. Woods, K. B. (Ed.), "Highway Engineering Handbook." McGraw-Hill (1960).
24. Wooltorton, F. L. D., "The Scientific Bases of Road Design." Edward Arnold, London (1954).

Appendix

TABLE 8

ABRASION OF COARSE AGGREGATE BY USE OF LOS ANGELES MACHINE[a]

Aggregate	Sample Weight (g)			Wear (%)
	Original	Final	Loss	
A-1	2,500	1,607	893	35.72
A-2	2,500	1,296	1,204	48.16

[a]ASTM designation C 131-55T.

TABLE 9

SOUNDNESS TEST OF AGGREGATE A-1 USING SATURATED SOLUTION OF SODIUM SULFATE[a]

Sieve Size		Grading of Original Sample (%)	Wt. of Test Fraction Before Test (g)	% Passing Finer Sieve After Test[b]	Weighted Avg.[c] (%)
Passing	Retained on				
No. 100		15.6	--	--	--
No. 50	No. 100	7.8	--	--	--
No. 30	No. 50	24.4	100	8.71	2.12
No. 16	No. 30	14.4	100	12.53	1.80
No. 8	No. 16	12.2	100	9.09	1.11
No. 4	No. 8	15.6	100	11.58	1.81
3/8-in.	No. 4	10.0	100	11.33	1.13
Total		100.0	500	--	7.97

[a]ASTM designation C 88-59T.
[b]Actual percentage loss.
[c]Corrected percentage loss after five cycles.

TABLE 10

SOUNDNESS TEST OF AGGREGATE A-2 USING SATURATED SOLUTION OF SODIUM SULFATE

Sieve Size		Grading of Original Sample (%)	Wt. of Test Fraction Before Test (g)	% Passing Finer Sieve After Test[a]	Weighted Avg.[b] (%)
Passing	Retained on				
No. 100		13.5	--	--	--
No. 50	No. 100	11.3	--	--	--
No. 30	No. 50	22.6	100	11.75	2.66
No. 16	No. 30	18.1	100	11.44	2.07
No. 8	No. 16	13.5	100	13.64	1.84
No. 4	No. 8	12.4	100	13.66	1.69
3/8-in.	No. 4	8.6	100	17.32	1.32
Total		100.0	500	--	9.58

[a]Actual percentage loss.
[b]Corrected percentage loss after five cycles.

TABLE 11

FLOAT MATERIAL AND THICKNESS OF SOIL-AGGREGATE-SODIUM CHLORIDE-STABILIZED SURFACES

Road	Sample No.	Float Material (psf)	Thickness (in.)
S-4	1	0.56	1.4
	2	0.89	1.3
	3	1.48	0.8
	4	1.27	1.2
	5	0.95	1.4
	Avg.	1.03	1.2
S-3[a]	1	1.16	2.3
	2	1.04	2.1
	3	0.78	2.2
	4	0.88	1.8
	5	0.65	2.1
	6	0.47	2.4
	Avg.	0.83	2.1
S-2	1	0.59	2.7
	2	0.93	3.0
	3	0.60	2.1
	4	1.03	2.4
	Avg.	0.79	2.7
S-1	1	0.46	3.3
	2	0.80	2.9
	3	0.74	3.4
	4	1.05	3.1
	5	0.59	3.4
	Avg.	0.73	3.2

[a]Includes only that section of road S-3 on which aggregate A-2 was used for construction.

TABLE 12

FLOAT MATERIAL ON NON-CHEMICALLY TREATED SURFACES

Road	Sample No.	Float Material (psf)
G-4	1	1.64
	2	1.11
	3	1.35
	4	1.43
	Avg.	1.38
G-3	1	1.67
	2	1.47
	3	1.43
	4	1.58
	5	1.69
	Avg.	1.57
G-2	1	1.43
	2	1.48
	3	1.35
	4	1.45
	Avg.	1.43
G-1	1	1.08
	2	1.01
	3	0.97
	4	1.13
	5	1.09
	Avg.	1.06

TABLE 13

RESULTS OF MECHANICAL ANALYSES OF FLOAT MATERIAL ON SOIL-AGGREGATE-SODIUM CHLORIDE-STABILIZED SURFACES

Road	Sample No.	Float Material (%)			
		Clay	Silt	Sand	Gravel
S-4	1	0	4	59	37
	2	3	9	60	28
	3	1	2	54	43
	4	1	1	46	52
	5	1	5	66	28
	Average	1.2	4.2	57.0	37.6
	Range	0-3	1-9	46-66	28-52
	Max. var.	3	8	20	16
S-3[a]	1	0	3	56	41
	2	3	6	70	21
	3	2	4	42	52
	4	2	10	70	18
	Average	1.75	5.75	59.50	33.00
	Range	0-3	3-10	42-70	18-52
	Max. var.	3	7	28	34
S-3[b]	1	3	8	42	47
	2	2	5	60	33
	3	0	4	45	51
	Average	1.67	5.67	49.00	43.67
	Range	0-3	4-8	42-60	33-47
	Max. var.	3	4	18	14
S-2	1	1	4	63	32
	2	2	9	65	24
	3	1	5	62	32
	4	4	4	69	23
	Average	2.0	5.5	64.75	27.75
	Range	1-4	4-9	62-69	24-32
	Max. var.	3	5	7	8
S-1	1	0	3	51	46
	2	4	9	58	29
	3	1	5	62	32
	4	2	8	58	32
	5	2	5	60	33
	Average	1.8	6.0	57.8	34.4
	Range	0-4	3-9	51-62	29-46
	Max. var.	4	6	11	17

[a]Portion that used aggregate A-2.
[b]Portion that used aggregate A-1.

Relative Wear Resistance of Soil-Aggregate Mixtures

A. G. QAISSAUNEE, Assistant Professor, University of Kabul, Afghanistan; and
EUGENE Y. HUANG, Associate Professor of Civil Engineering, University of Illinois, Urbana

This investigation attempts to establish a relationship between the rate of wear due to the loosening and loss of particles and some of the characteristics of soil-aggregate mixtures. A total of 18 treatments consisting of two types of aggregate material, three gradations within each material, and three plasticity indexes within each gradation were studied. A specially designed apparatus, employing a wire brush as an abrading tool, was used to abrade about 50 g from the bottom of each sample. The samples were compacted in the standard $\frac{1}{30}$-cu ft compaction mold at optimum moisture and were cured at an oven temperature of 115 F for 9 hr. The rate of wear, defined as the total amount of material abraded divided by the time it took to abrade it, was determined for each specimen. The data were analyzed by the IBM 650 computer and the equations relating the rate of wear as measured in this experiment and the kind of material, the gradation index, and the plasticity index were deduced.

The results of this investigation indicate that, within the range of the variables studied, the rate of wear due to the loosening of particles varies with the gradation index and the plasticity index of the soil-aggregate mixtures and is also greatly influenced by the geometric characteristics of the aggregates. The resistance to wear of all mixtures increases as the plasticity index increases, and for every plasticity index 2 to 9, there is an optimum gradation index which results in the lowest amount of wear as compared with other mixtures having the same plasticity index. Of the two aggregate materials used in this investigation, the one consisting of the more angular and irregularly shaped particles exhibits better wear resistance qualities than the other material.

• SOIL-AGGREGATE mixtures composed of a more or less controlled combination of coarse aggregates, sand, and binder have been used as surfacing materials for low-cost, low-traffic highways for many years. An enormous number of miles of surfaced

Paper sponsored by Committee on Soils-Calcium Chloride Roads.

highways in this country, estimated at over 50 percent of the total mileage, is included in this category. One of the desired qualities of a roadway surface is the ability to resist traffic wear, but these surfaces, and for that matter, many other stabilized surfaces, are somewhat limited with respect to this ability. It is estimated that under normal traffic conditions for which these soil-aggregate surfaces may be economically justified, on the average 0.25 to 1 in. of surface material is lost annually (1). This loss of material must be compensated for in the course of maintenance operations, or it may be controlled to a certain degree by the addition of certain chemical stabilizing agents. Replacement of the surface material lost through wear and erosion often requires as much as 50 percent of the local maintenance dollar (2). This expenditure places a heavy burden on the limited budgets of local highway organizations.

A literature survey has indicated that no basic research has ever been undertaken to evaluate the relative wear resistance of different soil-aggregate mixtures. A few field studies have been carried out to measure the amount of loss from untreated soil-aggregate surfaces and surfaces treated with calcium chloride. Some of the findings of these studies are summarized as follows:

1. Loss from surfaces treated with calcium chloride was about one-third as much as that from the untreated surfaces (1).
2. Loss increased with an increase in traffic volume (1, 3).
3. Loss of depth is progressive but not at a uniform annual rate (4).
4. The loss is not consistently related to the volume of traffic (4).
5. Surfaces with 15 percent or more material retained on the No. 10 sieve resist depth losses much better than surfaces with little or no coarse material (4).

These more or less qualitative, and in some cases contradictory, conclusions by early investigators are indicative of the complex nature of the problem and demonstrate the need for some basic research and quantitative data in this area.

GENERAL CONSIDERATION OF PROBLEM

Traffic wear from soil-aggregate surfaces is dependent on four general factors: (a) the traffic conditions, (b) the roadway geometrics, (c) the climatic conditions, and (d) the characteristics of the soil-aggregate mixture.

Traffic action is instrumental in loosening the particles and removing them from the surface. The traveling vehicle exerts forces which in turn produce shearing stresses on the surface of the road. These shearing stresses must be resisted by the shearing strength of the materials on the surface. The magnitude of these forces and the resulting stresses depend on a number of traffic factors, including traffic volume, type of vehicles using the surface, speed of travel, wheel load, tire type and inflation pressure, acceleration and deceleration, and impact effect.

Of these traffic factors, the most important one generally recognized by engineers is that of traffic volume. Whether the loss is due to fatigue or instantaneous bond failure, every passage of the load means more loss. The total amount of wear varying with the volume of traffic has been realized by many engineers and the volume of traffic has often been used as the criterion for the maintenance operations and betterment program of soil-aggregate roads. Generally speaking, the problem of traffic wear is not very serious on a well-designed and well-built soil-aggregate surface, as long as the daily traffic is below 50 vehicles. When the daily traffic approaches 50 to 100 vehicles per day, dust becomes a problem (5). Dust palliatives can be applied economically when the highway carries approximately 150 to 200 vehicles per day. Finally, a point is reached where a soil-aggregate surface cannot be economically held by any known methods, and a higher-type surface must be provided. There is no universal criterion for these operations, however, because of the variations of the other general factors influencing the amount of wear.

The roadway characteristics, such as horizontal and vertical alignments, the relative elevation of the roadway with respect to the surrounding land, and the condition of the surface, act as modifiers in that they modify the stresses or the process of removal. The amount of wear at the approach to the horizontal curve, in the horizontal curve, and at the exit from the curve is often more than that on the tangent, because of the deceleration and acceleration of vehicles necessary to negotiate the curve and the centrifugal force involved while on the curve. Similarly, the deceleration and acceleration of vehicles on a grade result in an increase in the magnitude of shearing stress and in turn cause more wear. Removal of the material from the surface by wind is facilitated by a high-grade line and is aided by the force of gravity and erosion on steep grades. The smoothness or roughness of the surface affects the impact and the tractive forces acting on the surface and in turn the degree of wear.

Climatic factors such as precipitation, temperature, wind, and humidity together with the characteristics of the mixture determine the availability of moisture in the surfacing materials. It is generally realized that the actual wear takes place primarily during dry weather and that as long as there is a certain amount of moisture available, the fine particles are not as readily whipped up by traffic. When this moisture is lost through evaporation by the actions of wind and temperature, the dust problem initiates and the particles on the surface are gradually loosened by the actions of traffic. Since the average soil moisture parallels in general the average annual rainfall (6), the in-place moisture content for a given soil-aggregate mixture is generally high in areas of high rainfall, resulting in a relatively short period of active wear. On the other hand, a soil-aggregate mixture in a wet climate may be under a plastic condition for a long period of time and hence be subject to deformation, leading to rugged surface irregularities and to high rates of wear when climatic conditions are favorable.

The characteristics of the soil-aggregate mixture influencing its resistance to wear are those that govern the strength of the mixture in general and the ability of the mixture to retain moisture in particular. The strength of the mixture is derived from the mechanical interlocking of the granular fraction and the binding action of the fine soil fraction. The ability of the mixture to retain moisture, as well as its capacity to develop cohesion or binding action, depends on the quantity and quality of the binder. In practice, these principal characteristics of soil-aggregate mixtures may be controlled in specifications by placing limits on gradation and plasticity index. In this connection, it is generally realized that soil-aggregate surface materials must be reasonably well graded to provide an adequate internal skeleton of grains of aggregate touching each other or interlocking with each other to furnish internal stability. Most important of all, the quantity and quality of the binder must be such that the binder is capable of cementing the material together and yet when it expands in the presence of excessive moisture it just fills the rest of the voids in the mixture without destroying its internal stability. The quantity of the binder is indicated by the material passing the No. 200 sieve; the quality of binder is generally controlled by specifying a plasticity index for the mixture.

Other characteristics of a soil-aggregate mixture which are related to its wear resistance include the geometric characteristics and size of its particles. Because part of the shearing resistance of a mixture is derived from particle interlocking, it is reasonable to expect that angular, flat, or elongated, and roughly textured particles are more resistant to wear than rounded and smooth-faced aggregate because of higher particle interlocking and greater mutual protection. Likewise, wear from surfaces containing larger aggregate particles is often less than those containing smaller particles because they provide better protection to the mortar surrounding them, thus making their removal more difficult.

MECHANISM OF TRAFFIC WEAR

Traffic wear from the soil-aggregate surfaces takes place in two ways: (a) the loosening and subsequent loss of materials, and (b) the polishing and breakdown of individual particles. The first kind of traffic wear arises when the fine material in the surface is loosened by traffic and whipped away. This component of wear is largely determined by the binding properties of the sand-clay mortar and the ability of the binder material to hold the larger particles in place. The second kind of wear is essentially a function of the hardness and toughness of the aggregate particles and may

Figure 3. Wear apparatus.

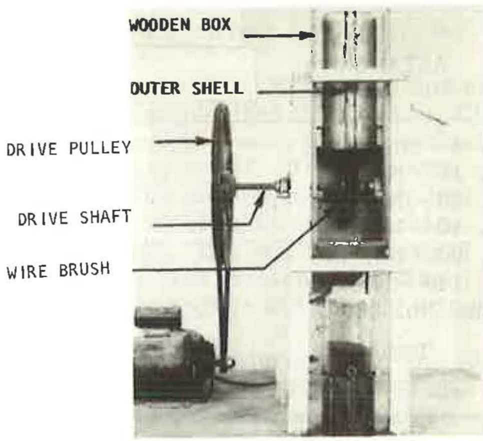

Figure 4. Wear apparatus with front removed, and loading assembly.

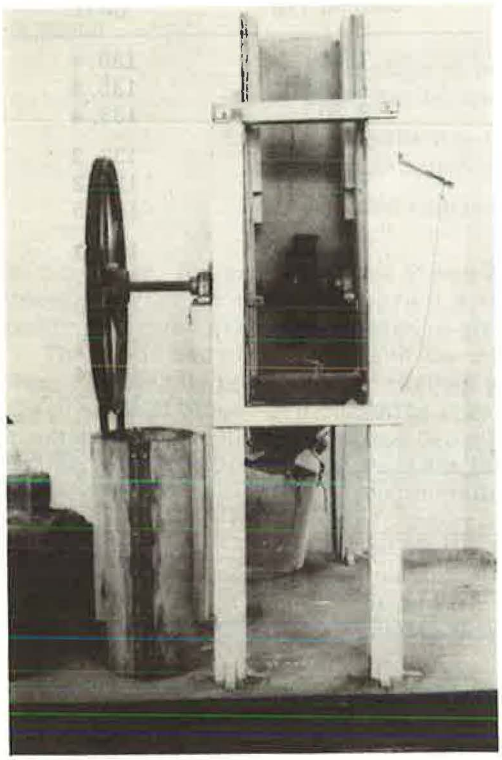

Figure 5. Wear apparatus with front and outer shell removed.

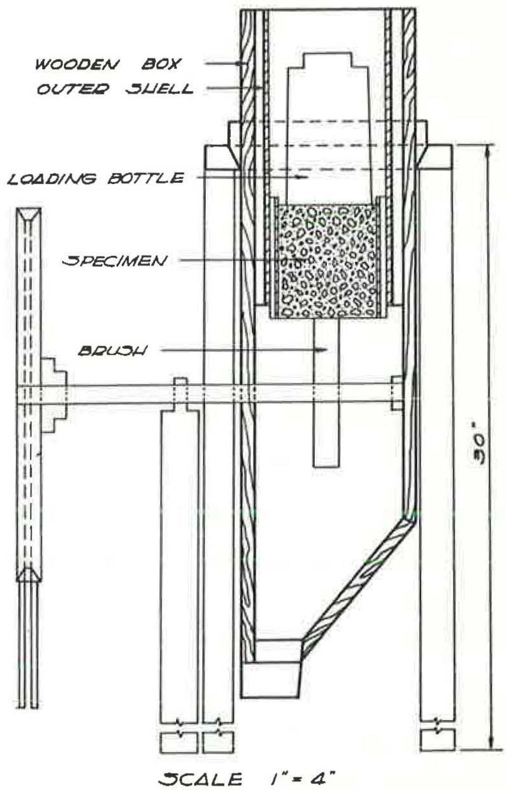

Figure 6. Cross-section of wear apparatus.

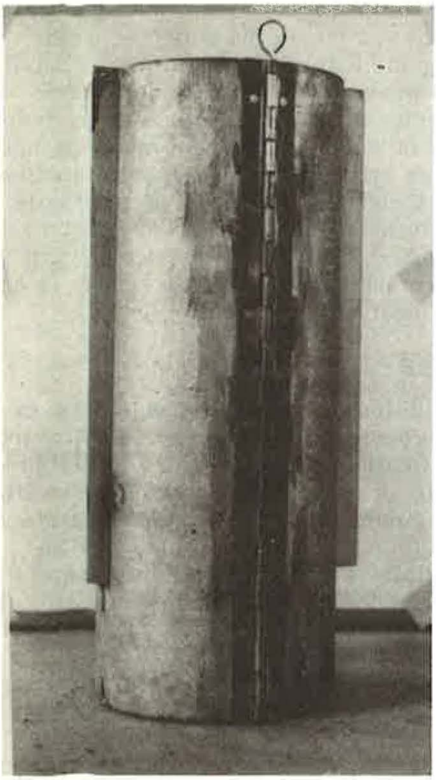

Figure 7. Outer shell.

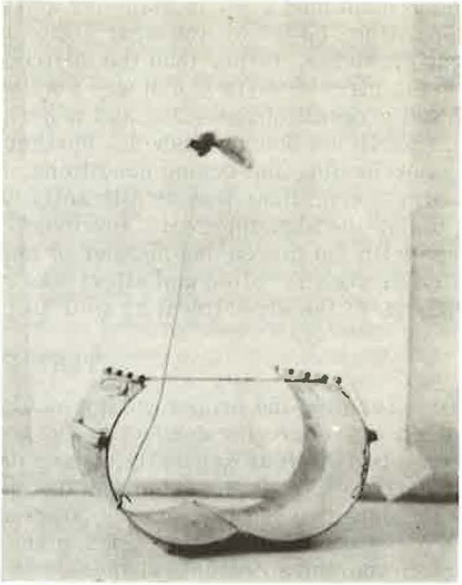

Figure 8. Specimen holder.

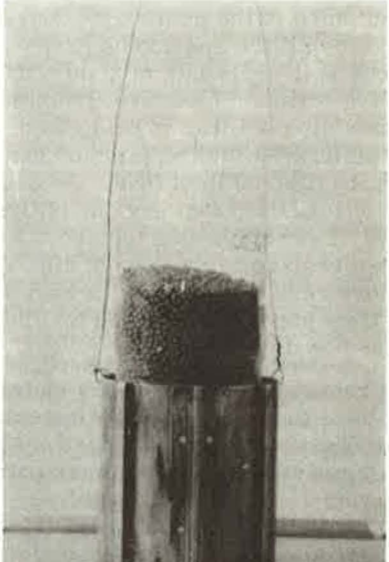

Figure 9. Loading assembly.

is used to turn the wire brush and to reduce the speed. It receives its power from a 1/3-hp motor through a 3/4-in. motor pulley. The turning speed of the pulley-operated wire brush is 1,725 rpm.

The specimen holder shown in Figure 8 is made from 1/16-in. thick sheet metal. Its inside diameter is 4 3/16 in. and it is 4 3/4 in. high. It is hinged in two places and is provided with one removable pin. There are four lugs welded to it to provide a close fit inside the outer shell. The loading assembly (Fig. 9) consists of the sample, the specimen holder, and a bottle partially filled with lead shots. The total weight of the load may be adjusted by an increase or a decrease of the number of lead shots in the bottle.

In regard to the abrading tool for the wear device, before the wire brush was finally adopted, several other abrading materials had been considered and experimented. In this connection, a 6-in. "industrial heavy duty solid" rubber wheel was the first used. It was hoped that the results obtained might make later correlations with field observations easier, but only the rubber wheel showed any wear and the method had to be discarded. Subsequently, a pair of medium grade grindstones 6 in. in diameter and 1 in. wide was experimented. With this type of apparatus, however, the wear action taking place was due to the polishing of the larger particles, rather than the desired loosening and loss of fine particles. Finally, the 6-in. circular wire brush was employed. The wear or loss of material by this tool was a result of loosening and removal of particles with no indications of the individual particles being polished. Further, the rate of wear for different mixtures under the same loading and curing conditions, or the same mixtures under different loading and curing conditions was significantly different. Consequently, the wire brush was adopted as the abrading tool. However, the effectiveness of the brush was found to decrease with the increasing number of specimens tested. This fact was viewed with particular consideration and effort was subsequently made to remove this effect in the design of the experiment as well as in the data analysis.

TEST PROCEDURE

To determine the proper curing and loading conditions of the specimens and to establish a procedure for conducting the wear test, a series of trial tests was performed. Because traffic wear generally occurs during dry weather, it was decided to measure the relative wear after dry-curing the specimens in an oven. Previous studies in Illinois indicate that the in-place moisture content of a number of soil-aggregate surfaces in DeWitt County during the summer season range from 27.8 to 59.8 percent of the optimum moisture contents of these surface materials, with only 3 out of a total of 37 values exceeding 50 percent (10). In an effort to produce a moisture content within the preceding range for the soil-aggregate mixtures in this study, an arbitrary temperature of 115 F was selected and a series of tests was performed on a typical material to determine the curing time required. The results of this study indicated that a curing time of 9 hr would produce a moisture content of approximately 40 percent of the optimum moisture of the material. Consequently, this curing condition was chosen as the standard for all the specimens prepared for this study. The average moisture contents of various soil-aggregate mixtures after curing, expressed as a percentage of their respective optimum moisture contents, are summarized in Table 3.

To establish the load to be applied, another series of tests was performed on the typical material after the samples had been cured for 9 hr. Observations were made with 10, 15, and 20 lb of load. As expected, it was found that the wear varied with the amount of load, and that any one of these loads would be satisfactory for the particular mixture. To avoid having too short a contact time for the mixtures that were believed to be less resistant to wear, or slippage of the belt with mixtures that were believed to be more resistant to wear, a load of 12 lb was selected for use in this study.

The final procedure adopted for the preparation of specimens and for the wear test is outlined as follows:

1. Proportion 2,200 g of dry material for any given gradation and plasticity index.
2. Place the 2,200 g of dry material in the mixer, mix for 2 min, add sufficient water to obtain the predetermined optimum moisture content, and mix for 3 min more.
3. Compact the sample to maximum dry density in three equal layers in the 4-in. mold having a capacity of $^1/_{30}$ cu ft, using a 5.5-lb rammer and a 12-in. drop.
4. Level off the top and weigh the sample and the mold.
5. Extrude the compacted sample, weigh and place it in the oven with the temperature of the oven controlled at 115 F.
6. Remove the sample from the oven after 9 hr and weigh it.
7. Place the sample in the specimen holder, with the lower end in contact with the wire brush, close and put in the removable pin. Two to four narrow sheets of paper must be placed between the specimen and specimen holder to insure that the specimen and specimen holder act as one unit and that the specimen cannot be pressed out.
8. Put enough lead shot in the loading bottle such that the total weight of the specimen, the specimen holder, the loading bottle, and the lead shot is 12 lb.
9. Place the specimen holder with the accompanying specimen in the apparatus and center the loading bottle on the top of the specimen.
10. Raise the whole assembly and turn on the switch.

11. At approximately 30 sec from the time that the switch was turned on, bring the specimen into contact with the brush, abrade about 50 g of material, raise the assembly, turn off the switch, and record the total time in seconds.

12. Collect all the material abraded and record the weight and total time that the specimen was in contact with the brush.

13. Calculate the rate of wear by dividing the weight of the material abraded by the total time expressed in grams per second.

TABLE 3

AVERAGE RELATIVE MOISTURE CONTENTS OF SOIL-AGGREGATE MIXTURES TESTED

Material	Gradation Index	Plasticity Index	Percent of Optimum
113	0.3	2.0	42
		6.0	44
		10.0	48
	0.4	2.0	35
		6.0	43
		10.0	48
	0.5	2.0	36
		6.0	41
		10.0	46
178	0.3	2.0	42
		6.0	42
		10.0	49
	0.4	2.0	38
		6.0	41
		10.0	48
	0.5	2.0	35
		6.0	42
		10.0	47

EXPERIMENTAL DESIGN

The experiment was performed according to a statistical procedure to eliminate personal bias in the interpretation of the results and to determine the effects of the selected controlled factors on the wear resistance of soil-aggregate mixtures on the basis of mathematical probabilities. As described previously, the study involved two types of material, three gradations within each material, three plasticity indexes within each gradation or a total of 18 treatments. It was decided to replicate each treatment six times, and to use a total of six brushes for the entire study. The specimens were to be prepared, cured, and tested under uniform conditions in groups of nine. These conditions suggested a factorial design with six blocks. Each block, employing a different brush, contained all 18 treatments in a randomized order, with the restriction that gradation indexes and plasticity indexes be equally represented in the first and second halves of each block.

To determine the nine treatments assigned to the first half of the blocks and those to the second half of the blocks, the signs of the products of the orthogonal polynomials were determined first. The nine plus signs were assigned to the first half of the blocks and the nine minus signs to the second half. The nine specimens that were to occupy each half of a block were then numbered one to nine, and the random arrangement of these specimens was determined by the use of a random numbers table (11). The arrangement of the tests, designated in terms of "material-gradation index-plasticity index," is given in Table 4.

ANALYSIS AND RESULTS

The rate of wear of various soil-aggregate mixtures obtained according to the preceding procedure is summarized in Table 5. The results in the table represent the combined effect of the many factors involved in the experiment. Not only the variations of the material factors (such as the type of aggregate, gradation index, and plasticity index) but also the differences in the effectiveness of the brushes used and the decrease of the effectiveness of a given brush with the increasing number of specimens tested are reflected in the results of the tests. The mathematical relationship between the rate of wear as measured by the aforementioned procedure and the several factors involved was determined using the general regression model:

$$\hat{y} = \overline{y} + \sum C_i X_i \qquad (2)$$

in which

$\hat{y}$ = rate of wear;
$\overline{y}$ = average rate of wear for all observations;
X_i = independent variables, p, p^2, g, g^2, m, pg, p^2g^2, pg^2, p^2g, mpg, mpg^2, mp^2g^2, mgp^2, mp, mp^2, mg, mg^2, B_1S, B_1S^2, B_2S, B_2S^2, B_3S, B_3S^2, B_4S, B_4S^2, B_5S, B_5S^2, B_1, B_2, B_3, B_4, B_5, S, and S^2;
C_i = regression coefficients;
g = gradation index;
p = plasticity index;
m = material;
B_i = brush i; and
S = sequence.

The data were analyzed by the IBM 650 computer using the library routine program "STAMP" of the University of Illinois Digital Computer Laboratory by the method of least squares. Based on the results with a significance level of at least 95 percent from the regression analysis, two equations for the materials tested in terms of all the independent variables previously listed were obtained. Considering an average brush which would not decrease its effectiveness following usage, the two equations were subsequently reduced to the following forms in terms of only the material factors:

$$y_{113} = 48.25 - 230.6g + 328.4g^2 + 0.87gp^2 + 13.8gp - 0.75p - 0.33p^2 - 33.0g^2p \qquad (3)$$

$$y_{178} = 54.22 - 248.0g + 328.4g^2 + 22.5gp - 4.24p - 0.09p^2 - 33.0pg^2 + 0.27gp^2 \qquad (4)$$

The standard error of the estimate is 0.74, and the multiple correlation coefficient is 0.97.

The preceding equations indicate that the rate of wear as measured in this study varies with quadratic functions of the gradation index and the plasticity index. The relationship between the rate of wear and the gradation index for the two materials with various plasticity indexes are shown in Figures 10 and 11. These graphs indicate that the rate of wear decreases as the plasticity index increases, and that for each

TABLE 4
RANDOM ARRANGEMENTS OF SOIL-AGGREGATE MIXTURE SPECIMENS

Order of Testing	Material-Gradation Index-Plasticity Index					
	Block 1	Block 2	Block 3	Block 4	Block 5	Block 6
1	178-0.4-10.0	113-0.5-10.0	113-0.5- 2.0	178-0.4-10.0	178-0.4- 2.0	113-0.4- 6.0
2	178-0.5- 6.0	178-0.4- 2.0	113-0.5-10.0	114-0.3- 2.0	113-0.5-10.0	178-0.3- 6.0
3	113-0.5- 2.0	178-0.3- 6.0	113-0.3-10.0	113-0.5-10.0	178-0.4-10.0	113-0.3-10.0
4	113-0.3- 2.0	113-0.3-10.0	113-0.4- 6.0	113-0.3-10.0	178-0.5- 6.0	178-0.5- 6.0
5	178-0.3- 6.0	178-0.5- 6.0	113-0.3- 2.0	178-0.5- 6.0	113-0.3- 2.0	178-0.4- 2.0
6	113-0.5-10.0	113-0.3- 2.0	178-0.4- 2.0	178-0.3- 6.0	113-0.4- 6.0	113-0.3- 2.0
7	113-0.3-10.0	113-0.4- 6.0	178-0.5- 6.0	178-0.4- 2.0	178-0.4- 6.0	113-0.5-10.0
8	178-0.4- 2.0	178-0.4-10.0	178-0.4-10.0	113-0.5- 2.0	113-0.5- 2.0	113-0.5- 2.0
9	113-0.4- 6.0	113-0.5- 2.0	178-0.3- 6.0	113-0.4- 6.0	113-0.3-10.0	178-0.4-10.0
10	113-0.4-10.0	113-0.3- 6.0	178-0.5-10.0	113-0.4- 2.0	178-0.3-10.0	113-0.5- 6.0
11	113-0.5- 6.0	178-0.5-10.0	113-0.4- 2.0	113-0.4-10.0	113-0.4- 2.0	178-0.3-10.0
12	178-0.3- 2.0	178-0.5- 2.0	178-0.5- 2.0	178-0.5-10.0	178-0.4- 6.0	113-0.4- 2.0
13	178-0.4- 6.0	113-0.4-10.0	178-0.3- 2.0	178-0.3- 2.0	178-0.3- 2.0	113-0.4-10.0
14	178-0.5- 2.0	178-0.3- 2.0	178-0.3-10.0	178-0.5- 6.0	113-0.4-10.0	178-0.4- 6.0
15	178-0.5-10.0	178-0.4- 6.0	113-0.5- 6.0	178-0.5- 2.0	178-0.5- 2.0	178-0.3- 2.0
16	113-0.4- 2.0	113-0.4- 2.0	113-0.3- 6.0	113-0.4- 6.0	178-0.5-10.0	178-0.5- 2.0
17	178-0.3-10.0	178-0.3-10.0	178-0.4- 6.0	178-0.3-10.0	113-0.3- 6.0	113-0.3- 6.0
18	113-0.3- 6.0	113-0.5- 6.0	113-0.4-10.0	113-0.3- 6.0	113-0.5- 6.0	178-0.5-10.0

TABLE 5

RESULTS OF WEAR TESTS OF SOIL-AGGREGATE MIXTURES[a]

Material	Gradation Index	Plasticity Index	Rate of Wear (g/sec)					
			Brush 1	Brush 2	Brush 3	Brush 4	Brush 5	Brush 6
113	0.3	2.0	11.10 (4)	4.82 (6)	11.34 (5)	10.44 (2)	9.72 (5)	9.64 (6)
		6.0	7.14 (18)	4.29 (10)	10.02 (16)	9.20 (18)	8.57 (17)	7.74 (17)
		10.0	7.70 (7)	3.54 (4)	8.14 (3)	8.31 (4)	5.72 (9)	4.83 (3)
	0.4	2.0	7.36 (16)	3.80 (16)	8.60 (11)	9.87 (10)	7.82 (11)	6.70 (12)
		6.0	6.48 (9)	2.71 (7)	7.69 (4)	9.91 (9)	7.08 (6)	6.76 (1)
		10.0	5.75 (10)	1.80 (13)	6.78 (18)	6.34 (11)	6.14 (14)	4.78 (13)
	0.5	2.0	13.98 (3)	5.89 (9)	13.04 (1)	12.18 (8)	12.10 (8)	11.02 (8)
		6.0	6.10 (11)	2.47 (18)	7.44 (15)	6.50 (16)	6.82 (18)	6.28 (10)
		10.0	5.89 (6)	2.38 (1)	7.46 (2)	6.36 (3)	6.68 (2)	4.65 (7)
178	0.3	2.0	8.80 (12)	4.61 (14)	9.82 (13)	8.82 (13)	8.75 (13)	7.88 (15)
		6.0	8.33 (5)	4.92 (3)	8.50 (9)	6.92 (6)	5.92 (7)	6.94 (2)
		10.0	3.45 (17)	0.88 (17)	4.79 (14)	4.13 (17)	3.81 (10)	3.67 (11)
	0.4	2.0	6.26 (8)	5.00 (2)	8.50 (6)	7.05 (7)	8.60 (1)	6.40 (5)
		6.0	4.70 (13)	2.00 (15)	6.79 (17)	5.73 (14)	6.25 (12)	4.60 (14)
		10.0	4.02 (1)	1.48 (8)	6.20 (8)	5.80 (1)	5.21 (3)	3.66 (9)
	0.5	2.0	9.96 (14)	4.17 (12)	11.56 (12)	10.10 (15)	13.14 (15)	8.92 (16)
		6.0	7.78 (2)	3.54 (5)	7.16 (7)	8.24 (5)	8.42 (4)	7.78 (4)
		10.0	3.21 (15)	1.44 (11)	5.23 (10)	5.51 (12)	5.20 (16)	4.18 (18)

[a]Sequence of test shown in parenthesis after value of rate of wear.

plasticity index 2 to 9, there is a gradation index at which the rate of wear is a minimum. This gradation index is referred to hereafter as the optimum gradation index. These optimum values are given for the two materials in Table 5.

The trends of these optimums are not the same for the two materials. The optimum gradation index for material 113 increases as the plasticity index increases, whereas that for material 178 changes little or none at all. It seems that the difference is due to the influence of the geometric characteristics of the aggregates in these mixtures. For a mixture with sharply angular particles such as material 178, the protection shared by adjacent particles became very important. Because this mutual protection also depended on the relative position of the particles or simply the gradation index, it is conceivable that some optimum combination of the particle index and the gradation index would provide maximum protection irrespective of the plasticity index.

Figures 10 and 11 also show that the general shape of the curves changes gradually and approaches a straight line as the values of the plasticity index are increased. It has been described previously that wear due to loosening of particles is controlled primarily by cohesion. Because cohesion is reflected by the plasticity index, the rate of wear decreases as the plasticity index increases. As the plasticity index increases, the relative importance of cohesion also increases. This fact is illustrated by the flatness or the decrease in curvature in the proximity of the optimum points as the plasticity index increases.

To make a comparison of the wear resistance of all the soil-aggregate mixtures tested, two terms, "relative wear" and "relative wear resistance," are introduced here. The relative wear of a given mixture is defined as the ratio of wear at optimum gradation index for the given plasticity index to the wear at optimum gradation index for a mixture composed of material 178 and having a plasticity index of 9.0. Relative wear resistance is defined as the reciprocal of the relative wear. The relative wear and relative wear resistance values are given in Table 6. The relationship between relative wear resistance and plasticity index for the two materials is shown in Figure 12. As expected, the material with a higher particle index has a superior quality of wear resistance.

From the practical point of view, the results of this study provide a quantitative set of data relating wear resistance to gradation and plasticity index of soil-aggregate mixtures, which may be of some value in the understanding and future control of the wear of these materials. These results indicate that within the range of the variable studied,

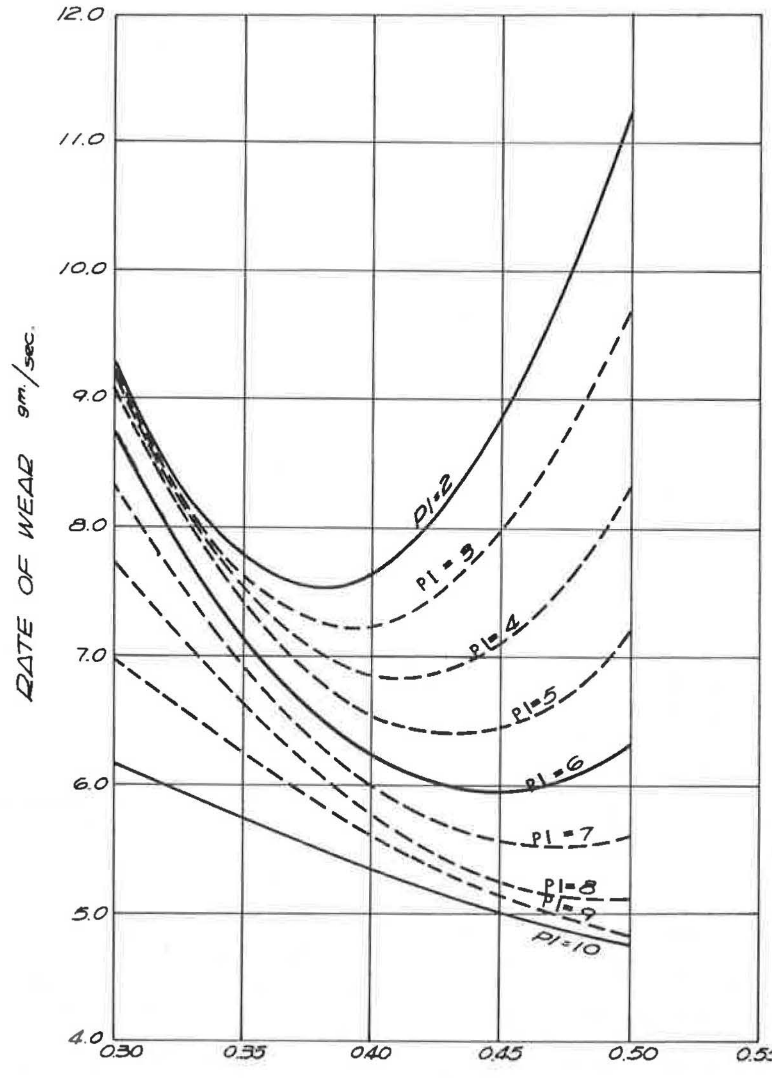

Figure 10. Relationship between rate of wear and gradation index for material 113.

an increase in the plasticity index increases the wear resistance, and that for any given plasticity index there is an optimum gradation index. There are also clear indications that the particle index plays an important role in the mechanism of wear resistance. The fact that, as the plasticity index increases, the rate of wear in the proximity of the optimum gradation index becomes less sensitive to changes in the gradation index appears rather significant. It means that at higher plasticity indexes a wider range of gradations can be used without increasing the rate of wear by any significant amount. The importance of this phenomenon becomes rather obvious when one considers the fact that even with the best practical degree of field control, it is difficult to produce a mixture with a given optimum gradation index.

It must be noted, however, that the findings are limited to the types of the materials and the curing and test conditions under which they were investigated. For field applications, additional studies are needed to substantiate the laboratory findings and to evaluate the other major variables not covered in this investigation.

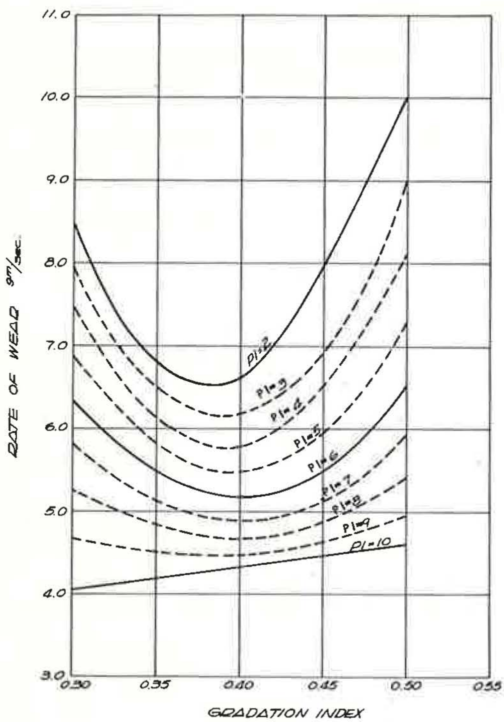

Figure 11. Relationship between rate of wear and gradation index for material 178.

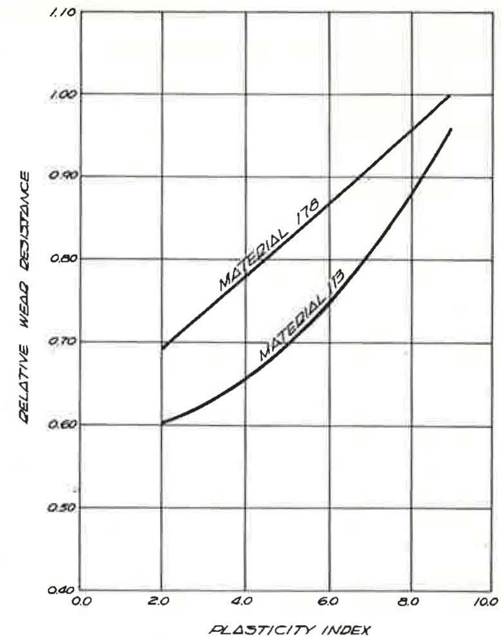

Figure 12. Relationship between relative wear resistance and plasticity index.

TABLE 6

OPTIMUM GRADATION INDEX AND RELATIVE WEAR RESISTANCE OF SOIL-AGGREGATE MIXTURES

Material	Plasticity Index	Opt. Gradation Index	Relative Wear	Relative Wear Resistance
113	2.0	0.38	1.68	0.60
	3.0	0.40	1.61	0.62
	4.0	0.41	1.52	0.66
	5.0	0.43	1.42	0.70
	6.0	0.45	1.32	0.76
	7.0	0.47	1.23	0.81
	8.0	0.50	1.14	0.88
	9.0	0.58	1.04	0.96
178	2.0	0.38	1.46	0.68
	3.0	0.39	1.37	0.73
	4.0	0.39	1.29	0.78
	5.0	0.39	1.22	0.82
	6.0	0.40	1.15	0.87
	7.0	0.40	1.09	0.92
	8.0	0.39	1.04	0.96
	9.0	0.38	1.00	1.00

SUMMARY AND CONCLUSIONS

In this study the characteristics of soil-aggregate mixtures controlling the wear or loss of material from soil-aggregate surfaces due to the loosening of the fine particles were investigated. A total of 18 treatments consisting of two types of material, three gradations within each material, and three plasticity indexes within each gradation were studied. A specially designed apparatus, employing a wire brush as an abrading tool, was used to abrade about 50 g of material from the bottom of each specimen. The specimens were compacted in the standard $^1/_{30}$-cu ft compaction mold at optimum moisture content, and then oven-cured for 9 hr at a constant temperature of 115 F. The rate of wear, defined as the total amount of material abraded divided by the time it took to abrade it, was determined for each specimen.

The experimental design used to collect the data was a factorial design with six randomized and restricted blocks. The restriction was equal representation of gradation indexes and plasticity indexes in the first and second halves of each block. A different brush was employed to test each block. The data were analyzed by the IBM 650 computer using the library routine program "STAMP" by the method of least squares. The output was used to write the regression equations relating wear to the type of material, gradation index, and plasticity index.

Based on the results of this research, the following conclusions regarding the relative wear resistance of soil-aggregate mixtures have been drawn. It is to be understood that these conclusions are based on the relationships established in this test series, applying the wear apparatus and the procedures developed in this investigation.

1. Using the wear apparatus and the procedures developed in this investigation, a wear index reflecting the traffic wear due to the loosening of particles may be obtained.
2. Within the range of variables studied, wear due to the loosening of particles varies with quadratic functions of gradation index and plasticity index.
3. Resistance to wear increases with an increase in plasticity index.
4. For every plasticity index there is an optimum gradation index which will result in the lowest amount of wear as compared to other mixes having the same plasticity index.
5. Any deviation from the optimum gradation index has a decreasing effect on the change in the rate of wear as the plasticity index increases. Thus, at higher plasticity indexes, wider ranges of materials can be used without decreasing the wear resistance of the mixtures by a significant amount.
6. The trend followed by this optimum gradation index is greatly influenced by the particle index, or the geometric characteristics of the aggregates.
7. Materials with higher particle index values have better wear-resistance qualities.

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REFERENCES

1. Baylard, E. M., "Performance Study of Calcium-Chloride-Treated Road." HRB Proc., 30:337-348 (1952).

2. "Construction and Maintenance of Local Rural Roads." American Road Builders' Association. Tech. Bull. 180, 3-15 (1951).
3. Swinton, R. S., "Measurement of Wear on Gravel Roads." HRB Proc., 17; pt. 1, pp. 323-328 (1938).
4. Strahan, C. M., "A Study of Gravel, Top Soil and Sand-Clay Roads in Georgia." Public Roads, 10:117-136 (1929).
5. Huang, E. Y., "Manual of Current Practices for Design, Construction, and Maintenance of Soil-Aggregate Roads." University of Illinois Experiment Station (June 1959).
6. Eno, E. H., "Some Effects of Soil, Water, and Climate upon the Construction, Life, and Maintenance of Highways." Ohio State University Experiment Station, Bull. 85 (Nov. 1934).
7. Terzaghi, K., and Peck, R. B., "Soil Mechanics in Engineering Practice." Wiley (1948).
8. Woolf, D. O., "The Relation Between Los Angeles Abrasion Test Results and the Service Records of Coarse Aggregates." HRB Proc., 16:350-9 (1937).
9. Huang, E. Y., "A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles." American Society for Testing Materials (in press).
10. Huang, E. Y., "In-Situ Stability of Soil-Aggregate Road Materials." ASCE Jour. Highway Div. (May 1962).
11. Brunk, H. D., "An Introduction to Mathematical Statistics." Ginn (1960).