EFFECT OF VARYING THE QUANTITY AND QUALITY OF THE SOIL PORTION OF HIGHWAY AGGREGATES ON THEIR STABILITY

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SYNOPSIS

The combination of the plasticity index and liquid limit as $\frac{PI}{LL} \times 100$, for convenience designated as "D", was found useful to indicate the quality of soil Three soils of different qualities were added to each of three similarly graded specimens of coarse material to produce a maximum density grading Thirteen per cent passing the No 40 sieve, and two other gradings which differed only in that the per cents passing the No 40 sieve were 23 and 33 respectively were used The stability of the nine combinations was determined by the California bearing value test

These tests indicated that there is a definite relationship between the quantity D × soil per cent and the stability which should be of considerable value in the design of subgrades, base and surface courses It also appears that the practice of rejecting aggregates on the basis of liquid limit and plasticity index tests without providing for consideration of the relative importance of the quantity of soil in the aggregate tested may often be uneconomical.

In many instances the material proposed for use as cushion, ballast, base, top course or bituminous aggregate is found to be satisfactory and desirable in all respects except that the soil portion is not acceptable. As specifications are generally written now such a material could not be used.

In the discussion of situations of this type two questions usually arise. First, is the quantity of fine material large enough to cause failures if it is used? Some may hold that even relatively small quantities have a very serious effect while others might argue that the effect would be negligible It is apparent that the effect would be unimportant for very small quantities and would become more and more serious as the quantity increased. The difficulty is due to uncertainty regarding the manner in which the effect increases.

The second question which arises is: can an acceptable material be produced by adding satisfactory fines, eliminating a part of the unsatisfactory fines, or both? Here, too, the difficulty is due to uncertainty as to the effect of variation in the quantity and quality of fines. Economic considerations will rarely justify the use of a mixing plant to accomplish the addition of selected fines, but it is believed that they could be satisfactorily incorporated in an aggregate already on the road by using a motor patrol and sprinkling as is frequently done when adding binder.

These situations suggested that an investigation of the effect of varying the quantities of soils of different quality when combined with material retained on No 40 sieve be made. For the purposes of this study the fines or soil portion of a mineral aggregate was considered as the portion passing a No. 40 sieve An aggregate was considered to be a graded mixture of coarse material and fines or soil.

The liquid limit and plasticity index were selected as criteria of the quality of soils and it was decided to use the bearing value test originated by the California Department of Public Works, Division of Highways, to measure the effect of varying the quantity of soils of different quality combined with coarse material. The California Public Works Standard Specifications (1935) state the following regarding this test: "It is used to determine the bearing value of foundation soil and material proposed for base, subbase, binder and surfacing. The test is performed only on that portion of the material which will pass a screen with circular openings one inch in diameter. The bearing value of binder material is determined after the material has been blended with clean sand and gravel, sufficient binder being used to provide the maximum amount of dust permitted in the type of work proposed."

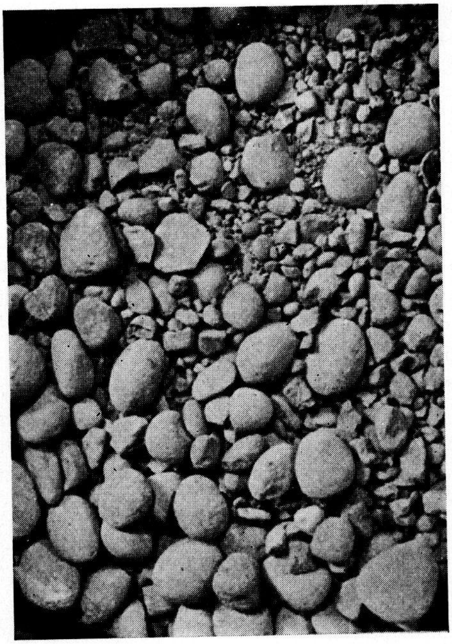


Figure 1. Character of Aggregate Used

The soils used consisted of a clay which was obtained on the east bank of Oak Creek west of Corvallis, Oregon, and a sandy loam which was secured one mile north of Corvallis, Oregon, on State Highway No. 26. A sieve analysis of each soil was made. The liquid limits and plasticity indices of the clay and loam, and of mixtures of the loam with 20, 40, 60 and 80 per cent clay were determined.

From a large stockpile of Willamette River gravel sufficient 1-in. minus material for the tests proposed was selected. Particular care was taken to eliminate all flat and angular pieces. The rounded character of the coarse material is indicated in Figure 1. Rounded aggregate was selected in order that mechanical interlocking of the particles would not obscure the effects which it was desired to study.

The gravel was washed and separated into sizes retained on the $\frac{3}{4}$-in., $\frac{1}{2}$-in, $\frac{3}{8}$-in., $\frac{1}{4}$-in., No. 4, No. 10 and No. 40 sieves. It was then recombined to make three different gradings for nine specimens each weighing approximately ten pounds. The gradings were based on a maximum density grading computed from the rule that the percentage passing each sieve equals the square root of the ratio of the diameter of the sieve to the maximum diameter of the aggregate. The proportions of all gradings retained on the No. 40 sieve were the same, but later soil was added to make the percentages passing the No. 40 sieve 13, 23, and 33. The resulting gradings were designated as No. 1, No. 2, and No. 3 respectively. In addition to these, one specimen, designated No. 0, containing no material passing a No. 40 sieve was prepared.

TABLE 1

Soil	A	B	C
LL	25.4	33.0	44.0
PI	4.1	13.5	21.2
% Clay	20	60	100

On the basis of the soil mixture tests, three soils were selected, the best being one which just met the usual specification requirements. These soils were designated A, B, and C. Their liquid limits, plasticity indices and clay content are shown in Table 1.

Then 13, 23 and 33 per cent of soil A was added to three of the specimens of coarse material which were designated

No. 1-A, 2-A, and 3-A respectively. In like manner soils B and C were added to the prepared coarse material and the resulting, specimens designated No. 1-B, 2-B, 3-B; 1-C, 2-C, and 3-C. That is, samples having the same number have the same grading and same percentage of soil while those having the same letter contain the same soil. The gradings are indicated in Table 2 and shown graphically in Figure 2.

Using the standard Proctor cylinder and tamper, the optimum moisture content of each specimen was determined. The results are shown in Table 4.

For the bearing value test sufficient material to provide a specimen about five inches high and six inches in diameter when compacted was first brought to the optimum moisture content previously determined and then consolidated in a cylindrical mould 6 in. in diameter under a load of 2000 lb. per sq. in.

A piston with a bearing area of 3 sq. in. was then caused to penetrate each specimen at the rate of 0.05 in. per minute.

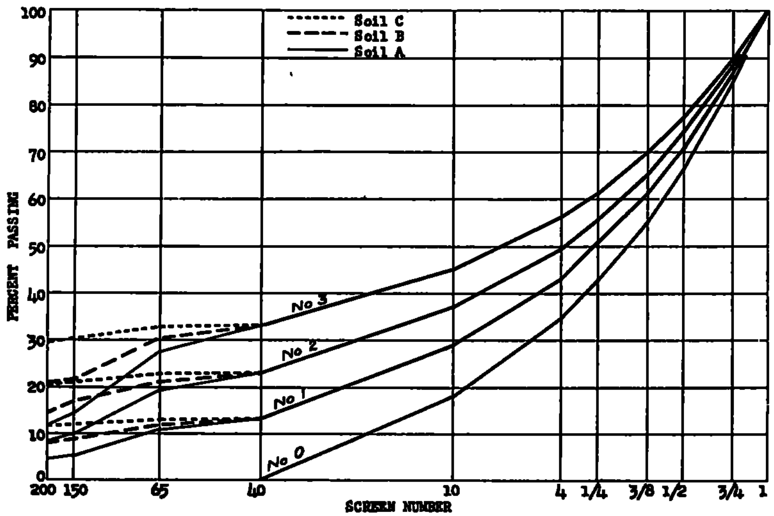


Figure 2. Grading of Specimens

TABLE 2

Sample No.	0	1-A	1-B	1-C	2-A	2-B	2-C	3-A	3-	3-C
% passing										
1 in.	100.0	100.0	100 0	100 0	100.0	100.0	100.0	100 0	100.0	100 0
¾ in.	85.0	87.0	87 0	87 0	88.3	88 3	88.3	90 0	90.0	90 0
½ in.	66 6	71 0	71 0	71 0	74 3	74 3	74 3	77.7	77.7	77.7
⅜ in	55.1	61 0	61 0	61 0	65 4	65.4	65 4	70.0	70 0	70 0
¼ in.	42 4	50 0	50.0	50 0	55 8	55 8	55 8	61.4	61 4	61.4
No. 4	34 5	43 0	43 0	43 0	49 5	49 5	49 5	56 2	56.2	56 2
No. 10	18 4	29 0	29 0	29 0	37.2	37 2	37 2	45 3	45 3	45 3
No. 40	0	13 0	13 0	13 0	23.0	23 0	23 0	33 1	33.1	33 1
No 65	0	10 9	11.9	12 9	19 3	21 0	22 8	27 7	30.3	32 8
No 150	0	5 6	8 8	12 0	9 9	15 5	21.2	14 2	22 4	30 4
No 200	0	4.6	8 1	11.6	8 2	14 4	20 6	11.8	20 6	29 5

TABLE 3

BEARING VALUE TEST DATA

Pen in	Aver. Load lb per sq in	Ratio to Std Value	Ratio to Ult Std Value
Specimen No 0			
		%	%
0 1	937	46 8	21 8
0 2	1590	56 8	37 0
0 3	1810	53 2	42 1
0 4	2000	52 7	46 5
0 5	2015	46 8	46 8
Specimen No 1-A			
0 1	1058	52 9	24 6
0 2	2244	80 1	52 2
0 3	2748	80 9	63 9
0 4	3094	81 5	72 0
0 5	3335	77 6	77 6
Specimen No 1-B			
0 1	997	49 8	23 2
0 2	1958	69 9	45 6
0 3	2420	71 2	56 3
0 4	2846	74 8	66 1
0 5	3200	74 4	74 4
Specimen No 1-C			
0 1	1066	53 3	24 8
0 2	1811	64 7	42 1
0 3	2232	65 6	51 9
0 4	2551	67 2	59 3
0 5	2788	64 9	64 9
Specimen No 2-A			
0 1	1063	53 1	24 8
0 2	2232	79 6	51 9
0 3	2660	78 3	61 8
0 4	2975	78 3	69 2
0 5	3249	75 6	75 6
Specimen No 2-B			
0 1	752	37 6	17 5
0 2	1398	50 3	32 5
0 3	1734	51 0	40 3
0 4	2012	52 7	46 8
0 5	2272	52 8	52 8

TABLE 3—*Concluded*

Pen. in	Aver Load lb. per sq in.	Ratio to Std Value	Ratio to Ult. Std. Value
Specimen No 2-C			
0 1	722	36 1	16 8
0 2	1234	44 1	28 7
0 3	1521	44 7	35 4
0 4	1650	43 4	38 4
0 5	1794	41 7	41.7
Specimen No 3-A			
		%	%
0 1	1144	57 2	26 5
0 2	1992	71 2	46 3
0 3	2366	69 6	55 1
0 4	2642	69 5	61 5
0 5	2825	65 7	65 7
Specimen No 3-B			
0 1	638	31 9	14 8
0 2	910	32 5	21 2
0 3	979	28 8	22 8
0 4	1050	27 6	24 4
0 5	1110	25 8	25 8
Specimen No 3-C			
0 1	486	24 3	11 3
0 2	553	19 8	12 9
0 3	560	16 5	13 0
0 4	581	15 3	13 5
0 5	605	14 1	14 1

The loads required to obtain penetration in increments of 0 1 in. were recorded to a total penetration of 0.5 in The load at a penetration of 0 5 in is referred to as the ultimate load After completion of the test on the upper face, the specimen was reversed, reconsolidated, and the test duplicated on the opposite end. The results of the two tests were averaged Figure 3, illustrates the method of making the test.

The loads for each increment of penetration may be expressed as percentages of the ultimate load obtained on a standard specimen or the load for each increment may be expressed in percentage of the load required to obtain the same penetration on a standard specimen. Accordingly, it appears that an aggregate may be rated on the basis of the average of the ratios of its bearing value for each increment of penetration to the corresponding standard value. The use of this average ratio is believed to be desirable as a basis for comparison as some variation in individual ratios occurs when the piston encounters large particles of the aggregate.

The loads for the standard specimen used by the California Department of Public Works and in this investigation are as follows:

Penetration	Load lb. per sq. in.
in.	
0.1	2000
0.2	2800
0.3	3400
0.4	3800
0.5	4300

The ratings of subgrades and bases used by the California Department of Public Works are as follows:

% of Standard	Classification
0–5	Very poor subgrade
5–10	Poor to questionable subgrade
10–20	Fair to good subgrade
20–30	Very good subgrade
30–50	Good sub-base
50–80	Good gravel base
80–100	Good crushed rock or gravel base

Specifications usually require bases to show bearing values 80 per cent of the standard.

Bearing value test data are summarized in Tables 3 and 4.

The average percentage of ultimate standard bearing value was plotted against penetration for two arrangements of the test data each consisting of three sets of curves.

In the first group, Figures 4, 5, and 6, the effect of change in the quantity is indicated, the same soil being represented by the three curves in each particular set.

Figure 4 indicates that changing the percentage of the "A" soil from 13 to 33 has a relatively small effect on the bearing value. The bearing values shown in Figure 5, indicate that variation of the quantity of the less desirable "B" soil has a definite effect, while reference to Figure 6 shows that variation in the percentage of the clay-like "C" soil results in a serious decrease in the bearing value.

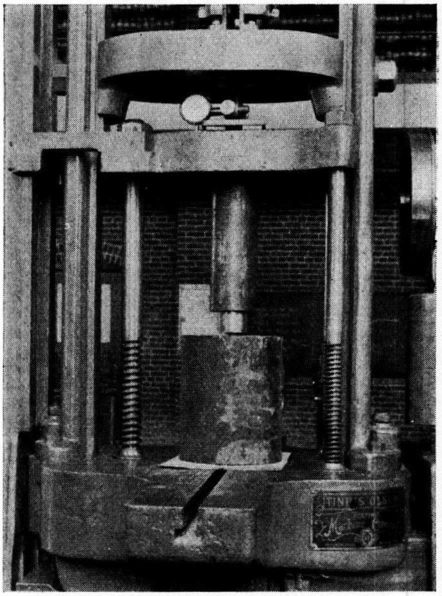


Figure 3. Method of Making Bearing Value Test

In the second group, Figures 7, 8 and 9, the effect of change in the quality of the soil is indicated, the grading of the aggregate or the percentage of soil represented by the three curves in each particular set being the same.

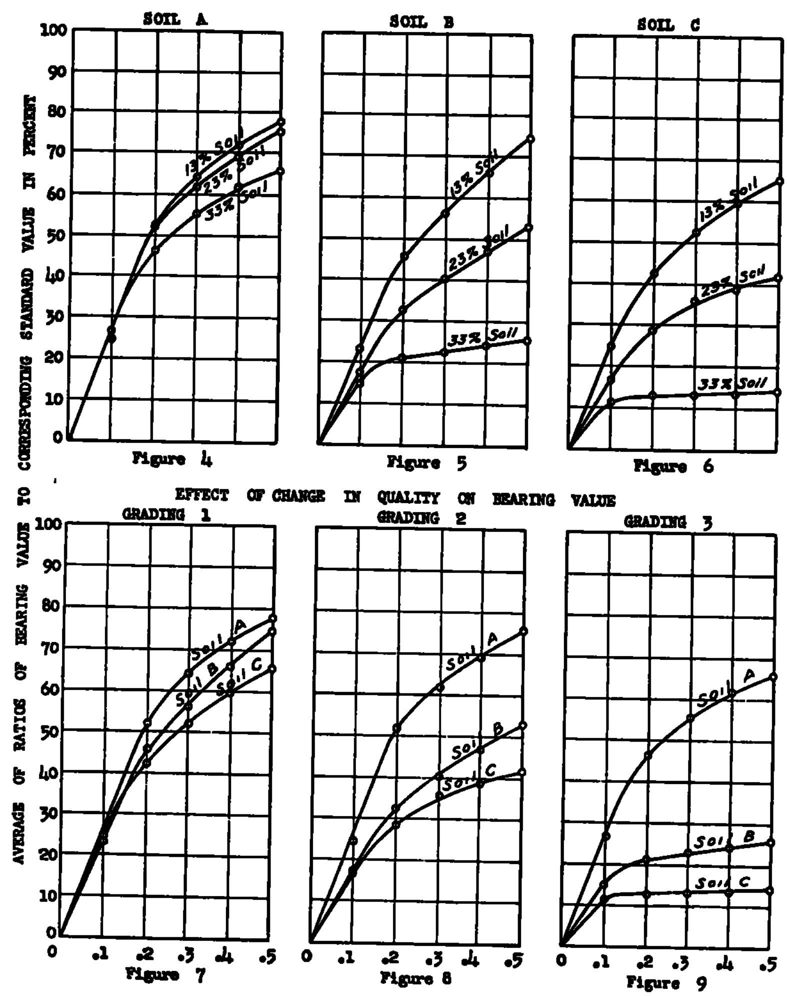
Figure 7, representing the maximum density grading set, indicates that change

TABLE 4

Summary of Bearing Value Test Data

Spec. No.	Optimum Moisture	Ult Bearing Value lb per sq in	Percentage of Ult Std. Value	Aver. of Ratios to Correspond Std Values	Proportion of Soil	"D" $100\frac{PI}{LL}$	Soil % × D
	%				%		
0	5.0	2015	46.8	51.2	0	0	0
1-A	6.0	3335	77 6	74 6	13	16 1	2 1
1-B	7 0	3200	74 4	68 0	13	40 9	5 3
1-C	7 5	2788	64 9	63 1	13	48 2	6.3
2-A	7.0	3249	75 6	73 0	23	16 1	3 7
2-B	7.5	2272	52 8	48 9	23	40 9	9 4
2-C	8 5	1794	41 7	42 0	23	48 2	11.1
3-A	7 5	2825	65.7	66 6	33	16.1	5.3
3-B	8 0	1110	25 8	29.3	33	40 9	13.5
3-C	9 5	605	14 1	18 0	33	48 2	15 9

EFFECT OF CHANGE IN QUANTITY ON BEARING VALUE



PENETRATION IN INCHES

in the quality of soil from PI = 4.1 and LL = 25.4 to PI = 21.2 and LL = 44.0 has relatively little effect on the bearing value. This is probably due to the fact that in this grading most of the load is transferred directly from grain to grain of the larger material while the soil merely fills the voids.

Figure 8, representing gradings containing 23 per cent of soil, indicates some decrease in bearing value as the quality of the soil changes, and Figure 9, repre-

3. If it is proposed to improve the quality of the soil in an aggregate by admixture of better material, the resulting bearing value can be predicted.

These curves also indicate the desirability of the generally used limitations of the liquid limit and plasticity index, namely 25 and 6. The bearing value of aggregates containing soil A which conforms closely to these requirements is noted to decrease much less rapidly as the

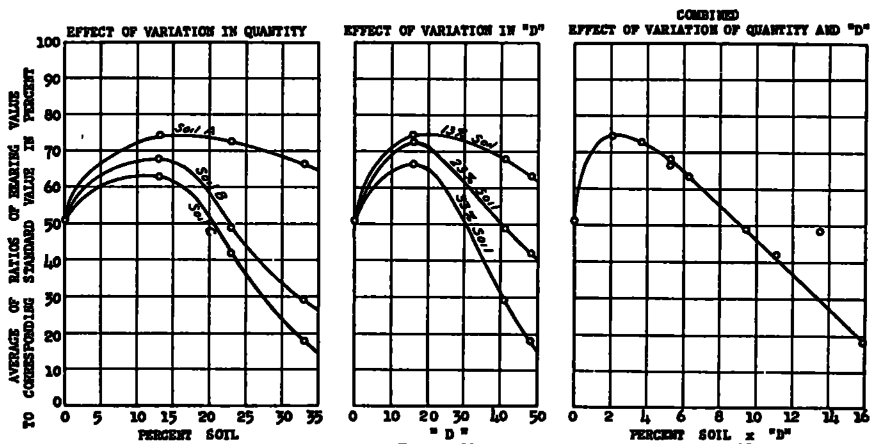


Figure 10 Figure 11 Figure 12

senting gradings containing 33 per cent of soil, shows a considerable decrease as the quality changes.

Plotting percentage of soil against the average of the ratios of the bearing value for each increment of penetration to the corresponding standard values for each specimen, three curves were obtained, Figure 10, which meet at the average ratio determined for specimen with grading No. 0, in Table 1. The usefulness of such a set of curves is illustrated by the following:

1. The stability of an aggregate containing 10 per cent of soil C is equal to one containing 17 per cent of soil B, or 35 per cent of soil A.
2. If 10 per cent of the soil in an aggregate containing 33 per cent of soil C is eliminated, the bearing value will be increased 133 per cent.

percentage increases than those containing other soils.

The relationship between stability and quality could be partially indicated by using either the liquid limit or plasticity index as a measure of quality. However, as both liquid limit and plasticity index are indicative of the quality of soils, it seemed desirable to combine them as a quantity $100\frac{PI}{LL}$. For convenience this quantity is referred to as "D". As far as is known this number has no physical significance, but, as will appear later, it seems to be a particularly convenient reference value for the comparison of various soils.

Plotting "D" against the average of the ratios of the bearing value for each increment of penetration to the corresponding standard values for each specimen, Figure 11, resulted in three curves which are somewhat similar to those in Figure 10. The value plotted for D = 0 was obtained from the test of Specimen No 0 which contained no material passing a No. 40 sieve. It seems reasonable that if a specimen containing soils having PI = 0 were tested that a greater bearing value would be obtained. The value plotted may be considered to represent the minimum bearing value when "D" equals zero. The maximum value could not be greater than that obtained for specimen 1-A. This suggests the possibility that "D" should be expressed as $100\frac{PI+L}{LL}$ where "L" is some value whose effect on "D" in the case of the other specimens is negligible.

Carrying one step further the idea that the quantity or number "D" to a considerable extent reflected the relative qualities of soils, it appeared that the influence of the soil in a given specimen on its bearing value would be proportional to "D" as well as to the quantity of soil, or if the product of the percentage of soil and the "D" value for each specimen were plotted against their respective bearing values that all the points would fall on one curve.

Figure 12 shows the curve which resulted The points, it will be observed, fall so as to locate a curve very definitely. From this curve it is apparent that the stability for all practical purposes varies directly as the percentage of soil and "D" for values of the quantity Soil% × D from 2 to 16. Also that for values of Soil% × D greater than 2, the aggregate having the larger Soil% × D will have the lesser stability.

As a result of this series of tests it was found that there appears to be a definite relationship between the plasticity index and liquid limit and stability which should be of considerable value in the design of subgrades, base and surface courses. These tests also suggest that the practice of rejecting aggregates on the basis of liquid limit and plasticity index tests without providing for consideration of the relative importance of the quantity of soil in the aggregate tested may often be uneconomical. That is, local materials which upon further investigation would prove to be satisfactory or which could be readily modified may be passed over in favor of more expensive materials.

The investigation reported herein was performed by the writer at Oregon State College, School of Engineering, while on leave from the Public Roads Administration.

DISCUSSION ON STABILITY OF HIGHWAY AGGREGATES

Mr O. L. Stokstad, *Michigan Highway Department:* In an attempt to check on the causes of failure in low cost road construction the soil engineers in the eight districts of Michigan were asked to list in the order of importance the factors which in their experience had contributed to or caused failures in gravel roads constructed to serve as bases for higher type surfaces.

Causes of failures in the order of importance·

1. Clay subgrade
   a. Lack of granular sub-base.
   b. Unconsolidated clay fills
   c. Wet clay masses in a sand fill or sub-base
2. Black top-soils and mucky soils in the subgrade
3. Subgrade drainage
   a. High water table
4. Too heavy an application of clay in stabilizing a sand subgrade
5. Frost heaves

6. Surface drainage—no ditch section
7. Miscellaneous
   a. Settlement over unexcavated muck
   b. Thin gravel
   c. Sugar sand
   d. Compaction difficulties due to clay and moisture in the gravel during cold or rainy weather
   e. Heavy trucking during construction.

This outline illustrates the importance of plastic soils as a factor which should be considered in any program of highway construction.

In regions of deep frost penetration the most common method for building roads over plastic soils consists of providing for granular sub-base (Fig. 1). The thickness of this ballast should vary with different soils and theoretically it should be practicable to measure the supporting power of the subgrade at its weakest moment and use a thickness of sand necessary sufficiently to distribute the anticipated load concentrations. In practice the limitations imposed by various construction problems may control the minimum thickness which is practical.

For instance there are limits beyond which it is impractical to go with respect to refinements in grading operations. The grade for the sub-base for instance may not be as true to line and cross section as the finished roadway and if the sub-base is too thin a variation of 0.2 ft may become serious. If the contractor's equipment travels on the sub-base during wet weather some of the sub-base may displace underlying clay and force clay wedges up through the sand Conventional paving operations require a certain amount of equipment travel on the grade in advance of the paver. When sub-base material consists of very incoherent sand it may be necessary to stabilize the top 2 or 3 in. of the sub-base with loam or sandy clay. It is necessary that the sub-base be sufficiently thick so that this operation does not involve too great a fraction of the sub-base thickness and thus destroy its effectiveness. Borrow pits from which sub-base materials are obtained are usually not absolutely uniform and occasionally they will yield a load of inferior quality. The thinner the sub-base the more serious becomes the effect of such slightly off color loads of

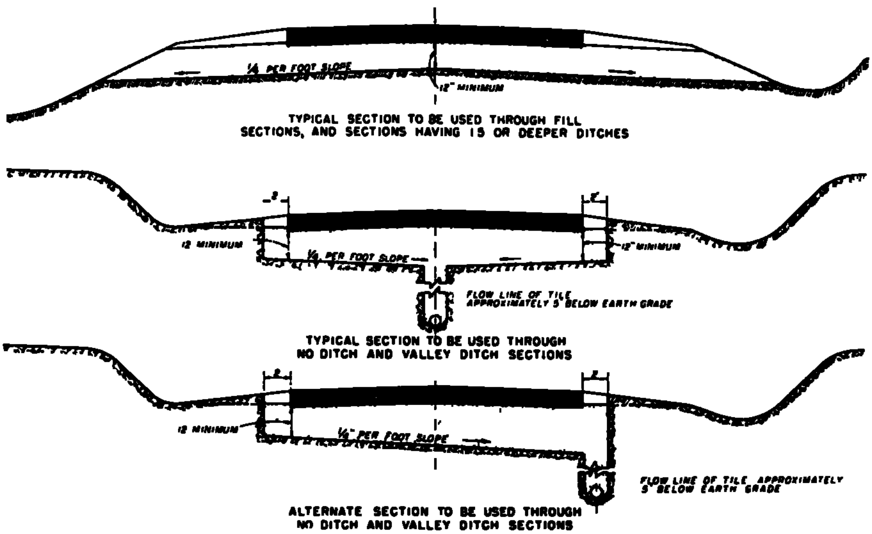


Figure 1

material. Many small local failures in gravel and bituminous surfaces may be traced to so-called clay gobs in the sub-base material. These are some of the factors which effect sub-base construction and their influence is such that sub-base thicknesses in this State of less than 12 in. are impractical. Judging from service records this thickness seems to be sufficient. Such failures on sub-base sections as have been studied have resulted largely from conditions for which control during construction was responsible. The Munising and Ontonagon soils of the Upper Peninsula of Michigan are two which require greater sub-base thickness than the 12-in. minimum made necessary by limitation imposed by construction techniques

In a climate such as Michigan's all soils with a clay and silt content of approximately 20 per cent or more suffer from loss of stability during spring thaws The more clayey soils at the time of maximum spring break-up are often so soft that the usual methods for measuring bearing values cannot be used. The manipulating action of traffic during this period may cause sub-grade soil to become fluid. The presence of organic matter in the soil aggravates this condition which accounts for the serious failures so often experienced on level clay plains where highway grades have been constructed of excavation from shallow roadside ditches.

In practicing the art of highway foundation engineering the soils engineer soon learns the soil individuals which are susceptible to spring break-up and roughly to what extent this susceptibility may go. At this point the science of soil mechanics may be used to give numerical value to bearing capacities as they change with the various seasons.

When gravel roads on plastic soils have been maintained for many years they often seem to develop a certain freedom from spring break-up.

This phenomenon is probably due to the fact that bad spots have been excavated and repaired each spring for so many years that a state of equilibrium has been approached between service requirements and minimum roadway condition insisted upon by the public to satisfy these requirements. Such a state of equilibrium is upset when the old gravel is covered with a new surface of a different type. An impervious surface retards the drying time and thus prolongs the break-up period. A higher type surface also tends to increase the volume of traffic. In spite of these factors there are many miles of old gravel on plastic soil foundations giving satisfactory service as bases for higher type surfacing It is important to remember, however, that small changes in verticial alignment such as is necessary to take the wallops out of an old gravel road may destroy sections of old gravel enough to create weak spots and resulting failures in the new road.

Fortunately much of the region of deep frost penetration is also a region of glaciation and glaciated regions usually have supplies of sand and gravel available for sub-base construction without excessive transportation costs. There are large areas on the other hand, where the supply of granular material is definitely limited such as in the basin of the former glacial Lake Maumee in southeastern Michigan and northern Ohio. The foundation problem in this area has not been solved as is evidenced by concrete pavements in which individual pieces are pumping mud and water under the impact of heavy traffic. Plastic soils have a double effect on highway surfacings.

Frost heaving, which always occurs in connection with freezing and which is always more or less differential, gradually breaks a pavement into smaller and smaller pieces. Then after thus breaking the slab, these soils, under the influence of a prolonged rainy period or spring break-up, lose their capacity to properly support the weakened structure.

The pedologist has succeeded in developing methods for gathering and classifying a wealth of soil engineering knowledge. The specialist in soil mechanics has succeeded in giving quantitative expression to many soil properties. The chemist is striving to make plastic soils permanently hydrophobic on a basis where his techniques can be applied to large volumes of soil. It is the soil engineer's function and obligation to assemble and study the contributions of these specialists, mix them with discipline acquired on survey and construction field studies and from this procedure develop the art of highway foundation engineering.