Laboratory Investigation on the Use of High Moisture Content Soils in High Fills

Y. H. HUANG, Assistant Professor of Civil Engineering, University of Kentucky, and
F. D. SHEPARD, Highway Research Engineer, Virginia Highway Research Council

A laboratory investigation on the use of high moisture content soils in high fills is described. The purposes were to investigate (a) the effect of thixotropy, initial moisture content and density, and consolidation on the shear strength of clay-like soils with moisture contents up to 8 percentage points above optimum; (b) the effect of initial moisture content and density on the consolidation characteristics of these soils; and (c) the possibility of using these soils in high fills and the manner in which they should be used.

Two soils—a Dunmore silt loam and a Clarkesville clay loam—were investigated. Soil specimens were fabricated by static compaction and later investigated for shear strength by the direct shear test and for consolidation characteristics by the conventional consolidation test. The results show that, when a soil is consolidated under high pressures, the initial moisture content and density have little effect on the final shear strength. This leads to the conclusion that high moisture content soils can be used if they are placed in the lower part of a high fill and if sufficient time is allowed for them to consolidate. A procedure for using these soils in high fills is also suggested.

•THE development of the Interstate Highway System has introduced a new problem into the highway field, namely, the shortage of suitable soils required for high fills. This shortage is caused in part by the limitations in moisture contents on soils that can be used. Since some of the soils from cuts and surrounding areas have moisture contents exceeding these limitations, large quantities of soils, which are mostly clayey, often have to be wasted and a suitable borrow found, thus increasing tremendously the cost of construction.

This problem is particularly critical in the Ridge-and-Valley Province of Virginia where high moisture content clay-like soils are frequently encountered. Virginia used to limit the moisture content of embankment soils to not exceed 3 percentage points above the optimum determined by the AASHO standard compaction test. However, a recent study of 159 soil survey samples from three Interstate projects in this area showed that only 11 percent of the samples met this limitation, while 54 percent had moisture contents more than 10 percentage points above optimum. Though the 1966 edition of Virginia Specifications has changed the limitation to 5 percentage points above optimum, this revision still cannot prevent the waste of large amounts of wet soils. Since many of these wet soils appear quite stiff and have the ability to support the rollers, a study on the possibility of using them is of practical significance.

The chief reason for not using these wet clayey soils in highway fills is that they cannot be compacted effectively to a desirable density. Even if this density can be achieved by increasing the compactive effort, the use of higher moisture contents during

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compaction will result in different soil structures which, in turn, may have adverse effects on soil properties.

The effect of structures on the behavior of compacted clays was discussed by Lambe (1, 2), employing the principles of colloid and crystal chemistry. He indicated that a clayey soil compacted on the dry side of optimum generally resulted in a flocculated structure, whereas that compacted on the wet side of optimum generally resulted in a dispersed structure.

Seed and Chan (3, 4) made an extensive investigation on the structure and strength characteristics of compacted clays. They substantiated Lambe's findings and demonstrated that for the same compacted density a flocculated structure generally exhibited less shrinkage, more swelling, higher swelling pressure, a steeper stress-strain curve, greater undrained strength, and lower pore pressure when compared with a dispersed structure. However, if the undrained strength and pore pressure were measured at large strains, the initial structure might be disturbed and its effect might become not so apparent. They also found that static compaction generally resulted in a flocculated structure, while dynamic or kneading compaction generally resulted in a dispersed structure.

Though clays compacted wet of optimum generally result in a dispersed structure and less strength, their strength may sometimes increase substantially with time. This thixotropic effect has been found in many fine-grained soils (5, 6). Mitchell (6) attributed the thixotropy in soils to the initial non-equilibrium of interparticle forces after compaction. He found that the thixotropic effect would occur if the initial structure was dispersed artificially to an extent greater than dictated by the interparticle forces. The role of water content in determining thixotropic behavior is to alter interparticle forces. If it is of a magnitude such that a dispersed structure can be induced by means of externally applied forces, even though the soil particles would tend to flocculate when at rest, the soil will be thixotropic. If the water content is such that the soil assumes a stable dispersed or flocculated structure independent of applied forces, the thixotropic effect will be negligible.

Since density and moisture content appreciably affect the engineering properties of soils, they should be properly controlled in the construction of fills. However, it is also felt that these two factors may not be so critical if the soil is placed under high fills. It is possible that under high overburden pressures soils with widely different initial densities and moisture contents after compaction may be finally consolidated to nearly the same density and moisture content, if given adequate time. Furthermore, the undesirable effect of high initial moisture contents, which induce a weaker structure, may be offset by thixotropic regains. In other words, consolidation along with thixotropy are the mechanisms which may change an initially weak soil into a much stronger one. If this is found true, there should be no reason to believe that those clays with moisture contents exceeding specification limits cannot be used under high fills.

One may argue that the suggestion of permitting a higher moisture content, or a lower density, for soils placed in high fills is contrary to normal practice since many specifications require greater density for deeper fills. However, these specifications are based on the assumption that the fills are massive with a long drainage path that requires a very long time to consolidate, so most of the settlements occur after the pavement is placed. If adequate drainage is provided in the fill such that most of the consolidation is completed before paving, a lower density for deeper fills might be permitted.

The purposes of this study were to investigate (a) the effect of thixotropy, initial moisture content and density, and consolidation on the shear strength of clay-like soils with moisture contents up to 8 percentage points above optimum; (b) the effect of initial moisture content and density on the consolidation characteristics of these soils; and (c) the possibility of using these soils in high fills and the manner in which they should be used.

In this study two soils, one a Dunmore silt loam and the other a Clarkesville clay loam, were investigated (7). Since both soils exhibit the same type of behavior, only data on the Dunmore silt loam will be presented. However, it must be borne in mind that the conclusions based on the Dunmore silt loam are also supported by extensive data on the Clarkesville clay loam.

Though the moisture contents of the soil used in this investigation were limited to 8 percentage points above optimum, it is believed that the conclusions drawn from this study are also applicable to higher moisture contents as long as the wet soil can support the rollers and other construction equipment.

PRELIMINARY CONSIDERATIONS

Before conducting any shear tests, three factors must be carefully considered: types of shear test, degrees of saturation, and increments of consolidation pressure.

Types of Shear Test

Because of the large number of tests required for this investigation, the direct shear test was employed instead of the conventional triaxial shear test.

When a soil is placed on the top of a fill, it is not subject to any significant consolidation pressures. Since the loads imposed by traffic are transient in nature, a quick shear test employing a very small normal pressure will simulate the actual field conditions.

When a soil is placed under high fills, it is subject to considerable overburden pressures. Since these pressures are applied slowly and a sufficient period of time can be made available for the soil to consolidate, a consolidated quick test should be employed.

The most direct way of finding the combined effect of consolidation and thixotropy is to consolidate the soil in a direct shear machine and then test it after a desirable period of time. However, this was impracticable because some specimens had to be left in the machine for 90 days to complete a test. Consequently, a modified procedure utilizing a confining device, as will be described later, was employed.

Degree of Saturation

For a soil compacted to a given density, the degree of saturation has a great effect on its shear strength. An unsoaked soil specimen is stronger than a soaked one because the increase in moisture content after soaking decreases the cohesion. Since a fill may be subject to severe weather conditions during construction, it is desirable that the specimen be completely soaked before testing.

The soaking of compacted soils generally results in swelling, the magnitude of which depends on the moisture content used during compaction and the confining pressure applied during soaking (4). Soils compacted dry of optimum may show a greater strength than those wet of optimum when unsoaked or soaked under high confining pressures; however, the reverse may be true if they are soaked under low confining pressures because of the larger swelling exhibited by the dry soils. Consequently, conclusions based on unsoaked specimens may not be applicable to soaked specimens. This clearly indicates that an arbitrary method of soaking must be established before any conclusions can be drawn.

In this investigation, both unsoaked and soaked specimens were employed. When the specimens, after being fabricated, were not subjected to consolidation before shear, the direct shear test was conducted at the molding moisture content and the specimens were not soaked. If the specimen was subjected to consolidation, it was first soaked under an arbitrary consolidation pressure of 0.23 ton per sq ft (equivalent to 200 grams of load on the 1:40 level arm) for 24 hours and the soaking continued as the consolidation pressure increased.

Increments of Consolidation Pressure

In order to simulate construction procedures, consolidation pressures were applied in increments. The first increment was 0.23 ton per sq ft and this was doubled each time until the desired consolidation pressure was obtained. When the specimens had been fully consolidated to the given pressure, they were kept in a confined state for a given period of time until being sheared.

This method of applying the consolidation pressure in the direct shear test is quite similar to that in the consolidation test so that the results obtained from the shear test can be related to those from the consolidation test.

SOIL CHARACTERISTICS

The soil reported in this paper was taken from Interstate Highway 81, about 2 miles north of Marion, Virginia. The physical properties of the soil were as follows:

1. Pedological classification	Dunmore Silt Loam
2. Specific gravity of solids	2.70
3. Atterberg limits	
a. Liquid limit	70
b. Plastic limit	42
c. Plasticity index	28
4. AASHO standard compaction	
a. Optimum moisture content, percent	32.6
b. Maximum dry density, lb per cu ft	86.6
5. Grain size analysis, percent	
a. Clay (smaller than 0.002 mm)	63
b. Silt (0.002 to 0.05 mm)	28
c. Sand (greater than 0.05 mm)	9
6. HRB classification	A-7-5 (19)

PROCEDURE

The procedure involved the preparation and testing of laboratory-compacted specimens. Two types of specimens were fabricated—strength and consolidation specimens.

The moisture-density relationship was obtained using the AASHO standard compactive effort, and the dry densities at moisture contents of 0, 3, 5, and 8 percentage points above optimum were determined. Soil specimens having these moisture contents

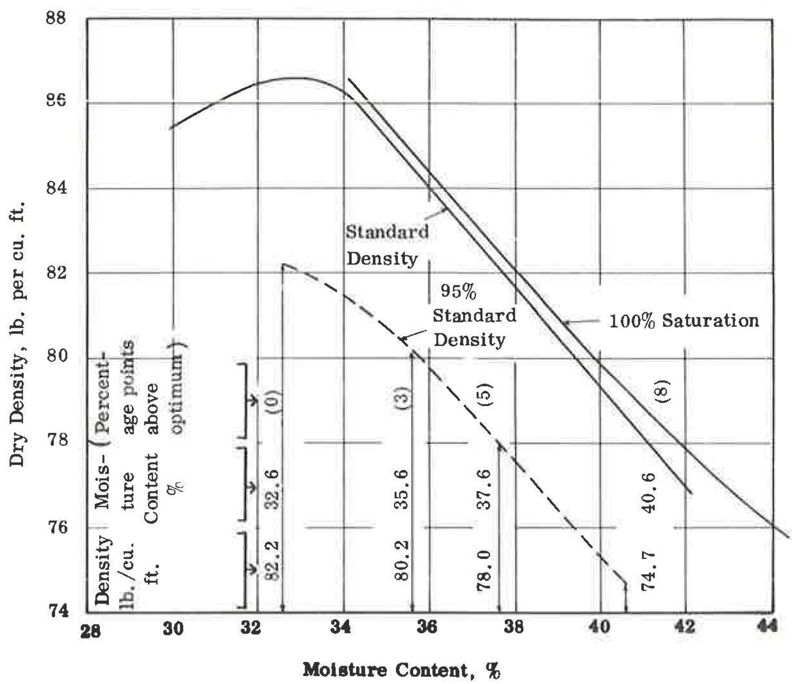

Figure 1. Moisture-density relationship.

were then compacted by a static method to densities corresponding to 95 percent of those obtained by the standard effort. It was believed that these densities could be easily obtained in the field.

Figure 1 shows the moisture-density relationship. The particular moisture contents and densities used are also indicated.

Preparation of Strength Specimens

The strength specimens were prepared by statically compacting the soil in a split ring 2.5 in. in diameter and 0.75 in. in height. Split rings were used so that the specimens could be tested in a direct shear machine. To prevent drainage between the rings, vacuum grease was applied and adhesive tape was secured around the outside to keep the two rings together. To facilitate compaction, the soil having approximately the desirable moisture content was placed in a mold 4.5 in. in diameter with the split ring embedded at the center. The whole mold assembly was then statically compacted by a Soiltest Versa Tester. When the soil was compressed to the required density, the load was released and the specimen removed from the mold. The top and bottom faces of the specimen were trimmed flush with the split ring and the trimmings used for water content determinations. The specimen was then weighed so that its density could be found.

An attempt was made to control the moisture content within limits of ± 0.5 percent and the density within ± 0.5 lb per cu ft. This requirement was generally fulfilled except for a very limited number of specimens. However, in no case was a discrepancy of more than ± 1 percent in moisture content or ± 1 lb per cu ft in density permitted.

Since the shear strength was investigated under three consolidation pressures (0, 1.85, and 7.40 tons per sq ft), four moisture contents (0, 3, 5, and 8 percentage points above optimum), and three curing times (1, 14, and 90 days), a total of 36 tests were required. Each test consisted of three specimens, so that a total of 108 specimens were fabricated.

In the case of zero consolidation pressure, the compacted specimens were placed in plastic bags and cured for 1, 14, and 90 days before testing. The direct shear test was conducted at the molding moisture content, and no soaking of the specimens was required.

In the case of 1.85 and 7.40 tons per sq ft consolidation pressures, the compacted specimen, with both top and bottom porous stones in place, was positioned in a confining device consisting of two plates and four connecting bolts as shown in Figure 2. The device was then placed in an empty pan and centered in a conventional consolidation apparatus. After an initial consolidation pressure of 0.23 ton per sq ft had been applied, the specimen was soaked by filling the pan completely with water. The load was then doubled every 24 hours until a consolidation pressure of 1.85 or 7.40 tons per sq ft was obtained. After the specimen had been consolidated under the prescribed pressure, it was kept in the consolidated state by slightly tightening the four bolts. The specimen was then cured in a moisture room for 1, 14, and 90 days before testing.

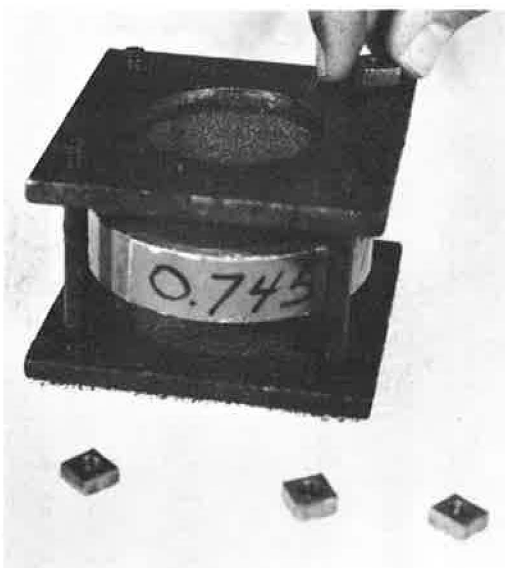

Figure 2. Confining device.

Preparation of Consolidation Specimens

The consolidation specimens were prepared in the same manner as the strength specimens except that a fixed consolidation ring was used instead of a split ring. The specimen, after being compacted and checked for density, was used immediately for the consolidation test.

Testing

The strength specimen, after being cured for a given time, was set in a conventional direct shear machine and tested immediately. In the case of unconsolidated specimens, a very small normal load, say 0.23 ton per sq ft, was applied to seat the specimen. In the case of consolidated specimens, a normal load equal to the original consolidation pressure, i.e., 1.85 or 7.40 tons per sq ft as the case might be, was applied. After the normal load had been put on, the nuts on the confining device were loosened and the undrained shear test was performed immediately. The test was conducted at a fairly quick rate, say 0.15 in. per minute, until the peak strength was obtained.

The consolidation specimens were tested on a standard consolidation apparatus. The specimens were soaked under a consolidation pressure of 0.23 ton per sq ft for 24 hours. Then consolidation pressures of 0.405, 0.81, 1.62, 3.24, 6.48, and 12.96 tons per sq ft were successively applied, and each maintained for a 24-hour period. The dial was read at 5, 10, 20, 30, and 60 seconds, then 2, 4, 8, 15, 30, and 60 minutes, and finally 2, 4, 8, 24 hours.

TEST RESULTS AND DISCUSSIONS

The test data on shear strength are summarized in Table 1. These data contain the information from which some of the figures and tables to follow were obtained and the final conclusions were drawn.

Effect of Curing Time on Shear Strength

The test on unconsolidated and unsaturated specimens is one of the most direct and simple methods for finding the effect of curing time on strength. Here specimens are tested in the same conditions as they are molded, the only variable being the curing time.

TABLE 1

DATA ON SHEAR STRENGTH, DENSITY AND MOISTURE CONTENT

Consolidation Pressure (ton/sq ft)	Time (days)	Moisture Content Above Optimum											
		0 Percent			3 Percent			5 Percent			8 Percent		
		S (lb/sq ft)	γ (lb/cu ft)	ω (%)	S (lb/sq ft)	γ (lb/cu ft)	ω (%)	S (lb/sq ft)	γ (lb/cu ft)	ω (%)	S (lb/sq ft)	γ (lb/cu ft)	ω (%)
0 Unsaturated	1	1894	82.5	32.5	1493	80.7	35.8	1011	77.8	37.9	752	75.0	41.1
		1836	81.6	32.6	1531	80.1	35.7	1113	77.9	37.9	788	74.2	40.7
		2059	81.9	32.5	1623	80.0	35.6	1067	77.7	38.0	798	74.5	40.3
	14	1873	81.7	32.9	1531	80.3	35.8	1030	78.0	37.1	854	75.2	40.3
		1966	81.9	32.9	1475	80.3	35.8	1120	77.6	37.2	806	74.3	41.0
		1846	82.2	32.7	1456	80.4	35.8	1038	77.7	37.6	788	74.0	41.0
	90	2075	82.6	32.2	1688	80.8	35.0	1118	78.3	37.9	668	74.7	40.2
		2121	82.4	31.9	1539	80.1	35.9	1099	77.8	38.1	909	74.9	40.3
		2102	83.1	31.8	1596	80.3	35.0	1070	78.6	37.2	890	74.7	40.4
1.85 Saturated	1	2615	82.6	32.8	2588	80.7	35.7	2216	77.8	37.7	2012	74.9	41.0
		2709	82.3	32.7	2375	80.3	35.7	2068	77.9	37.3	1963	74.9	40.5
		2709	81.9	32.6	2347	79.7	35.7	2189	77.7	38.1	1988	74.9	40.8
	14	2504	82.3	31.8	2402	79.6	36.0	2161	78.2	37.7	1994	75.2	40.4
		2496	83.1	31.6	2496	80.5	35.5	2058	77.5	38.0	1938	75.2	40.7
		2664	82.7	32.7	2272	81.0	35.7	2133	78.2	37.1	1938	74.2	40.7
	90	2412	82.2	32.1	2375	79.8	35.5	2076	77.3	37.0	1892	75.0	39.7
		2458	82.6	31.8	2364	79.6	35.3	1984	78.6	37.3	1752	75.2	40.1
		2550	82.6	32.0	2216	79.9	35.3	2068	78.4	37.6	1781	74.5	41.0
7.40 Saturated	1	5491	81.8	32.6	5574	79.9	36.1	4935	78.5	37.0	4910	75.0	40.7
		5436	81.3	33.0	4730	80.8	35.6	5129	78.4	36.9	4968	74.4	40.7
		5278	81.5	33.0	4647	80.7	35.7	5296	78.3	36.8	4805	75.1	40.8
	14	5240	81.9	32.7	5129	80.5	36.0	4916	78.1	37.8	5008	74.2	40.9
		5054	81.8	32.6	5102	80.7	35.7	5018	78.3	37.5	4916	75.1	40.8
		5018	81.7	32.8	5148	80.4	35.9	5064	77.8	38.0	5046	74.9	40.7
	90	5054	82.1	32.5	5027	80.6	35.7	4694	77.9	36.7	4860	74.4	40.5
		5240	82.9	32.4	5008	80.6	35.8	4939	77.4	37.3	4833	74.2	40.3
		5129	82.7	33.0	5370	79.8	35.5	4968	77.[illegible]	37.0	5129	74.3	40.3

Note: S = shear strength determined by the direct shear test; γ = dry unit weight; ω = moisture content.

TABLE 2

EFFECT OF CONSOLIDATION PRESSURE, CURING TIME, AND MOISTURE CONTENT ON SHEAR STRENGTH[a]

Consolidation Pressure (ton/sq ft)	Time (days)	Moisture Content Above Optimum (%)			
		0	3	5	8
0 Unsaturated	1	1929.7	1549.0	1063.7	779.3
	14	1895.0	1487.3	1062.7	816.0
	90	2079.3	1607.7	1095.7	822.3
1.85 Saturated	1	2677.7	2436.7	2157.7	1987.7
	14	2554.7	2390.0	2117.3	1956.7
	90	2473.3	2318.3	2042.7	1806.3
7.40 Saturated	1	5401.7	4983.7	5120.0	4894.3
	14	5104.0	5126.3	4999.3	4990.0
	90	5141.0	5135.0	4867.0	4940.7

[a]Shear strength is in lb per sq ft.

The test on consolidated and saturated specimens is another way of checking the thixotropic effect. It has been known for some time that for a given set of initial conditions there exists a unique relationship between the void ratio at failure and the shear strength (8, 9). As long as there is no structural disturbance this relationship appears to be independent of all other variables. Specimens with the same initial conditions and consolidated under the same pressure should arrive at the same void ratio. This void ratio is maintained by the confining device throughout the curing period. If there is no thixotropic effect, or structural change, during the curing period, the reapplication of the normal pressure during shear should cause very little change in void ratio. Since these specimens have the same void ratio after consolidation and since in the quick test this void ratio is equal to the void ratio at failure, the same strength should be obtained no matter how long the specimens are cured.

Table 2 shows the effect of consolidation pressure, curing time, and moisture content on shear strength. The shear strength is the average peak strength obtained from three test specimens.

A glance at Table 2 shows that curing time has very little effect on shear strength. To find whether curing time had a significant effect on shear strength, an analysis of variance (10) based on the shear strength shown in Table 1 was made; the results are given in Table 3.

The F ratio is a ratio of the between-group mean square and the within-group mean square. If this ratio exceeds a given value corresponding to the degrees of freedom for the variance estimate and the confidence level, the hypothesis that the two mean squares estimate the same variance is rejected, and curing time is considered to have a significant effect on shear strength.

Table 3 shows that, of the 12 sets of tests, 9 indicate the insignificance of curing time at the 95 percent confidence level, while the other three have F ratios only slightly over the 5.14 criterion. This may indicate that under the given methods of sample preparation and testing the thixotropic effect is practically nonexistent. This lack of thixotropy may be attributed to the following two causes:

1. In the range of moisture contents employed, static compaction always results in a flocculated structure, which lacks the thixotropic effect. If dynamic or kneading compaction were employed, a dispersed structure might be induced and the soil might become thixotropic (5, 6).
2. The shear strength is the peak strength which occurs at relatively large displacements. Since these large displacements cause a complete disturbance of the structure, the effect of initial structure may not be reflected in the shear strength (3, 4).

Though the shear strength at smaller displacements was also investigated and found to be non-thixotropic, it is possible that the direct shear test, even at small displacements, still causes a considerable disturbance to the soil structure. If the shear strength were determined at small strains by a triaxial compression test, the soil might exhibit thixotropic effect.

This explanation on the causes of non-thixotropy is purely conjectural. It is quite

TABLE 3

EFFECT OF CURING TIME ON SHEAR STRENGTH

Consolidation Pressure (ton/sq ft)	Moisture Content Above Optimum (%)	F Ratio	Effect of Curing Time
0	0	0.13	Insignificant
	3	2.79	Insignificant
	5	2.42	Insignificant
	8	0.23	Insignificant
1.85	0	5.61	Significant
	3	0.84	Insignificant
	5	2.57	Insignificant
	8	11.47	Significant
7.40	0	6.72	Significant
	3	0.21	Insignificant
	5	2.35	Insignificant
	8	0.54	Insignificant

Note: At 95 percent confidence level, the effect of time is significant when F value is greater than 5.14.

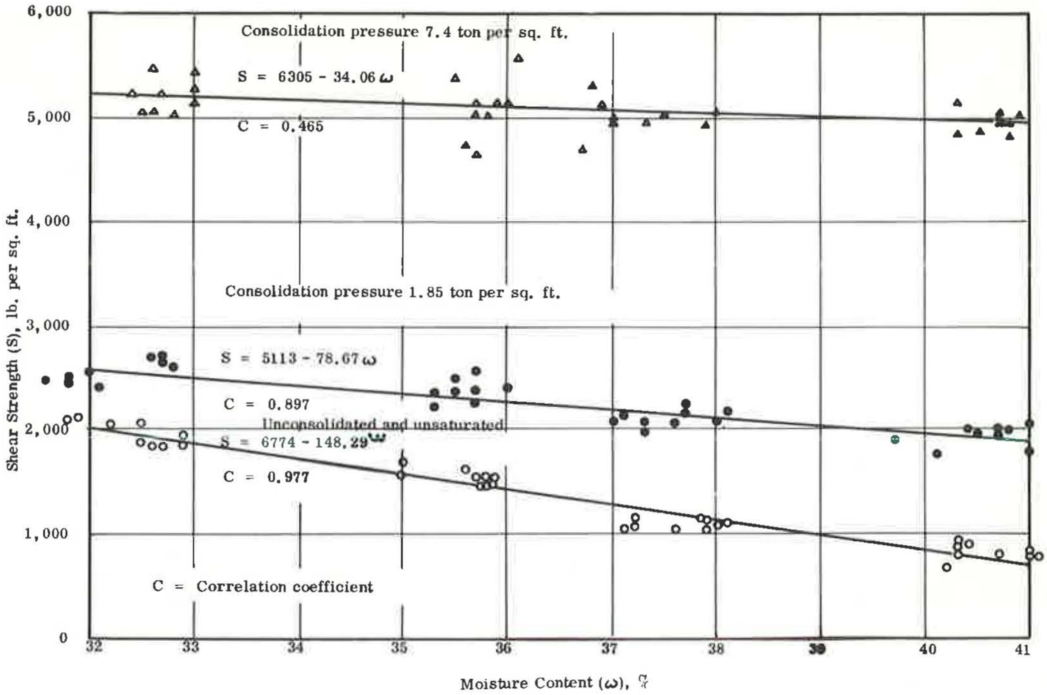

Figure 3. Shear strength vs moisture content under different consolidation pressures.

possible that the soil is non-thixotropic no matter what methods of compaction or tests are employed. Leonards (9) determined the unconfined compressive strength of a compacted Fort Union clay and found that the strength remained unchanged for a period of one year. The lack of thixotropy may indicate that density is one of the most important factors affecting strength.

Effect of Moisture Content on Shear Strength

Since curing time has practically no effect on shear strength, the shear strength at different curing times and moisture contents as listed in Table 1 can be plotted together as shown in Figure 3. The least-square lines together with the formulas relating strength to moisture content were obtained by standard linear regression techniques (10). The figure shows that there exists a straight-line relationship between shear strength and moisture content; the lower the consolidation pressure, the more rapid the change in strength with moisture content.

The correlation coefficient C as shown in the figure is a measure of the validity of the correlation. A coefficient of 1 indicates a perfect correlation, whereas a coefficient of 0 indicates no correlation at all. For unconsolidated and unsaturated specimens, there is an excellent correlation between shear strength and moisture content, but the correlation becomes poorer as the consolidation pressure increases.

The effect of consolidation pressure on the slopes of these straight lines and on the correlation coefficients indicates that as far as shear strength is concerned the initial moisture content is more important for unconsolidated specimens but becomes less important as consolidation pressure increases.

Effect of Density on Shear Strength

Since under a given compactive effort there exists a unique relationship between moisture content and density, the shear strength is related to density as well as to moisture content. In fact, density is a better indicator of strength than moisture

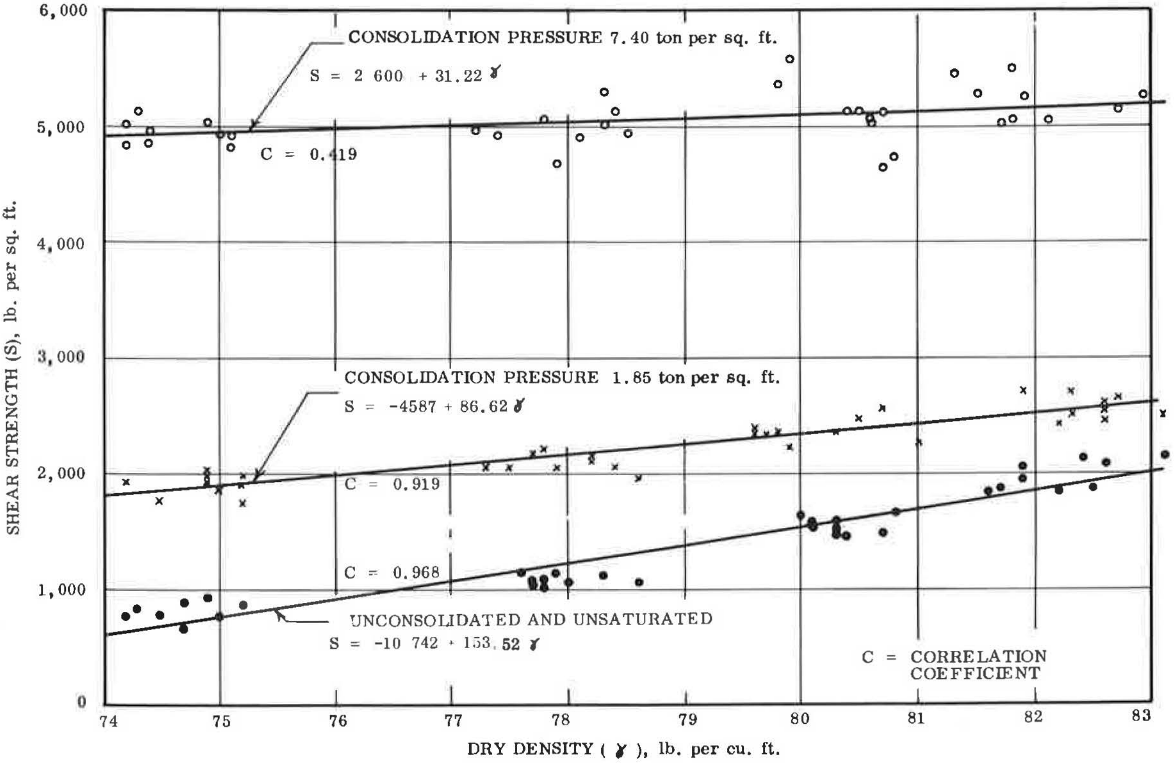

Figure 4. Shear strength vs density under different consolidation pressures.

content because the latter does not convey any meaning unless the compactive effort is specified.

Figure 4 shows that there is also a straight-line relationship between shear strength and dry density. The straight-line relationship between shear strength and moisture content and between shear strength and dry density indicates that the moisture-density relationship must be linear. This is generally true except at the vicinity of the optimum moisture content, as was shown in Figure 1.

In Figure 4 the vertical intercept between any two straight lines indicates the increase in shear strength due to the given increase in consolidation pressure. Since a larger intercept indicates a more rapid increase in strength with the increase in consolidation pressure, the figure shows that consolidation has more effect in increasing the shear strength at lower densities than at higher densities.

It should be noted that these straight lines may not give the best fit. For the unconsolidated and unsaturated specimens, a curve concaved slightly upward would give a better fit; it was found that a plot of density vs logarithm of shear strength would result in a straight line that gives a better correlation coefficient.

Figure 5 is a replot of the least-square lines shown in Figure 4, when the shear strength is expressed as a percentage of the strength at a density corresponding to 95 percent of the maximum density. It can be clearly seen that density has a tremendous effect on shear strength for unconsolidated specimens; however, its effect on strength diminishes appreciably when the consolidation pressure increases. For example, the shear strength at a density of 75 lb per cu ft is only 33 percent of that at 82.2 lb per cu ft. Nevertheless, if the soil in these two widely different compacted states is placed in high fills under a consolidation pressure of 7.4 tons per sq ft, the low-density soil will arrive at a shear strength 95 percent of the high-density soil. This may indicate that low-density or high moisture content soils can be used in high fills if they are given sufficient time to consolidate.

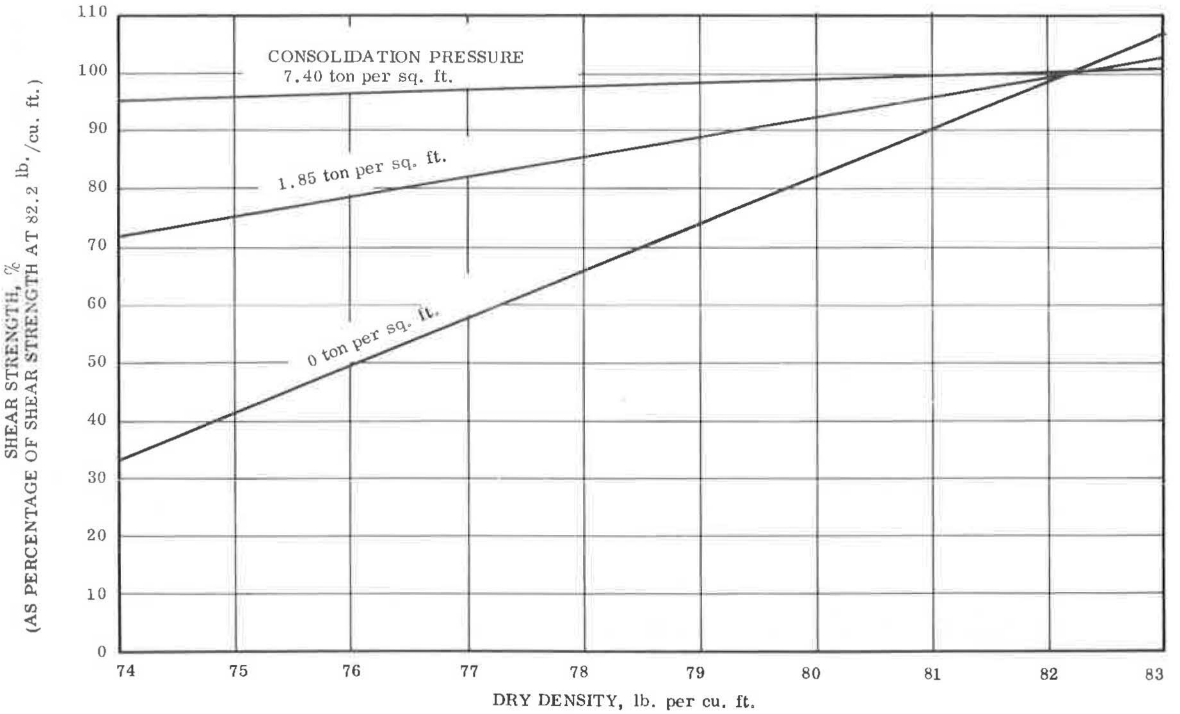

Figure 5. Variation of shear strength with density under different consolidation pressures.

Effect of Consolidation Pressure on Void Ratio

Figure 6 shows the relationship between void ratio (e) and the logarithm of pressure (log p), when the soil is fully consolidated under the given pressures. The figure shows that the e-log p curves converge as the consolidation pressure increases. This substantiates the previous finding that soils with widely different initial densities but consolidated under high pressures will exhibit only slight differences in strength because the differences in density or void ratio become smaller as the consolidation pressure increases.

The void ratios corresponding to the desirable and actual densities are marked on the left of the figure. As Figure 6 shows, under small consolidation pressures the soil compacted at the optimum moisture content results in a decrease in density. This is due to the swelling of the soil when soaked.

A SUGGESTED PROCEDURE FOR USING HIGH MOISTURE CONTENT SOILS

The investigation indicates that, though there is no evidence of thixotropic regains in the soil studied, consolidation alone can increase the shear strength appreciably. The effect of consolidation on shear strength is more noticeable at lower initial densities than at higher densities. Under high consolidation pressures a soil with widely different initial densities and strengths can finally arrive at some densities and strengths which are only slightly different. If soils having a density lower than or a moisture content greater than the specification limit are placed under high fills and time is allowed for them to consolidate, there should be no reason to believe they cannot be used.

In order to use wet soils properly, two questions must be answered:

1. Where should they be placed, or what is the minimum consolidation pressure required?

2. What is the time rate of consolidation, or will the consolidation or settlement be essentially complete before the pavement or other structures are placed on the fill?

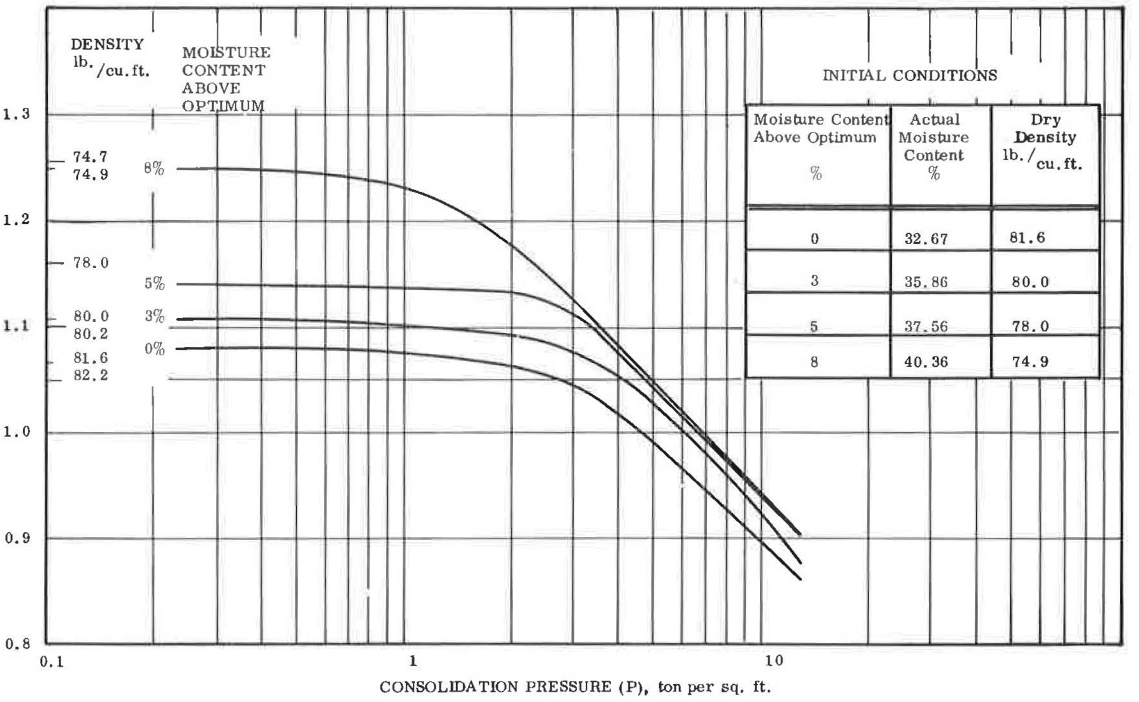

Figure 6. Void ratio vs logarithm of consolidation pressure under different initial conditions.

Minimum Consolidation Pressure Required

The lack of thixotropy in the soil indicates that density is the most important factor affecting shear strength, the effect of structure being negligible. A given density can be obtained by either compaction or consolidation. In fact, consolidation is a better process than compaction because it causes a dissipation of the pore pressure and thus increases the effective stress. If the soil with a moisture content of 40.36 percent and a density of 74.9 lb per cu ft is used and the minimum density requirement is 82.2 lb per cu ft, it can be found from Figure 6 that the soil must be placed under a consolidation pressure of not less than 4.8 tons per sq ft, which is equivalent to a fill height of about 90 ft.

Under a given consolidation pressure, the void ratio at 100 percent consolidation as shown in Figure 6 will generally take a very long time to attain. In engineering practice it is not necessary to have the fill 100 percent consolidated before the pavement or other structures are placed. A certain percentage of consolidation, say 95 percent, may be considered satisfactory so long as the remaining consolidation or settlement is small and within the specified tolerance.

Let e_0 be the initial void ratio, or the void ratio obtained when the soil is soaked under a pressure of 0.23 ton per sq ft, and e_{100} be the final void ratio for a given consolidation pressure. The void ratio at 95 percent consolidation is

$$e_{95} = e_0 - 0.95\,(e_0 - e_{100})$$

$$= 0.95\, e_{100} + 0.05\, e_0$$

Based on the final void ratios and the initial void ratios, the relationship between the void ratio at 95 percent consolidation and the consolidation pressure is plotted in Figure 7. The minimum pressure required to consolidate the soil with a moisture content 8 percentage points above optimum to a density of 82.2 lb per cu ft is 5.3 tons per sq ft, which is equivalent to a fill height of 100 ft.

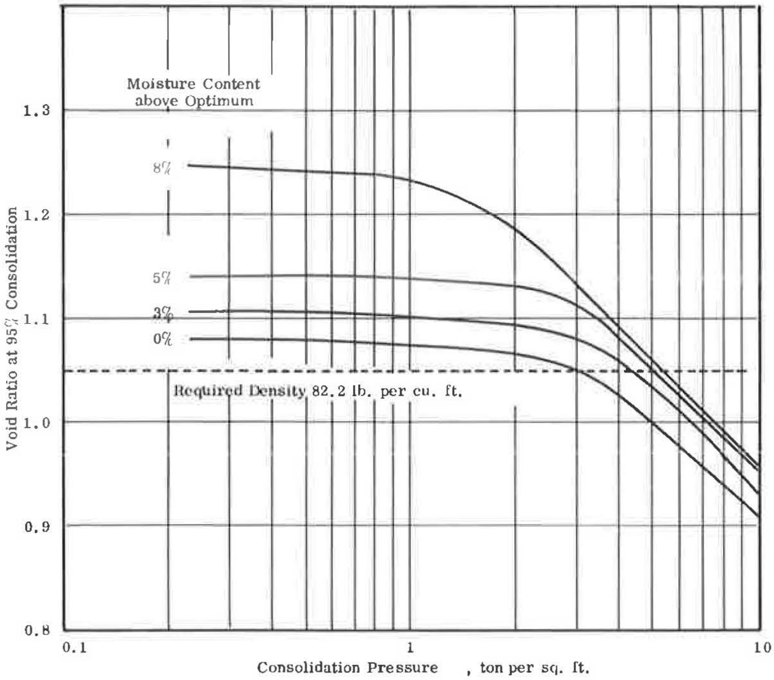

Figure 7. Void ratio at 95 percent consolidation vs consolidation pressure.

The selection of 95 percent consolidation is for illustration purposes only. Properly, the percentage should be chosen so that the remaining settlement will not be detrimental to the overlying structures. The remaining settlement can be determined by the following formula:

$$\Delta H = \frac{e_{95} - e_{100}}{1 + e_{95}} (2H)$$

where

ΔH = remaining settlement after 95 percent consolidation, in ft;
$2H$ = thickness of consolidated layer, in ft;
e_{95} = void ratio at 95 percent consolidation; and
e_{100} = void ratio at 100 percent consolidation.

For example, if the soil with a moisture content 8 percentage points above optimum is 20 ft thick and subject to an average consolidation pressure of 5.3 tons per sq ft, then from Figure 6 e_{100} = 1.032, and from Figure 7 e_{95} = 1.049, and

$$\Delta H = \frac{1.049 - 1.032}{1 + 1.049} \times 20 = 0.166 \text{ ft} = 2 \text{ in.}$$

Time Rate of Consolidation

If it is found that the consolidation must be 95 percent complete before the construction of any overlying pavements, then a question immediately arises as to the actual time required to effect 95 percent consolidation. This can be determined from Terzaghi's theory of consolidation by the following formula:

$$t = T_V \frac{D^2}{C_V}$$

where

t = time, in days;
T_V = time factor depending on the initial pore pressure distribution and the percent consolidation;
D = length of drainage path, in ft, equal to half thickness of consolidation layer if drainage is provided at both top and bottom;
C_v = coefficient of consolidation, in sq ft per day.

If the initial pore pressure is constant or varies linearly with depth, then $T_V = 1.129$ at 95 percent consolidation.

In the above example, if drainage is provided at both top and bottom of the 20-ft wet layer, then D = 10 ft. Since the consolidation coefficient of the soil was found to be 0.22 ft²/day based on the consolidation test, the time required to effect 95 percent consolidation is

$$t = 1.129 \times \frac{(10)^2}{0.22} = 513 \text{ days}$$

If the above time is considered too long, additional drainage must be provided to reduce D. If additional drainage is provided at the mid-height, then D = 5 ft or t = 128 days.

It was not the intention of this study to investigate the design and construction of drainage facilities. However, it must be pointed out that a very simple method of providing drainage is to place sand layers at frequent intervals in the fill. This "sandwich construction," wherein the wet clay layers are alternated with a granular, permeable material, has been suggested by Baker and Gray (11). Though they arbitrarily recommend that the wet clay be deposited in 1-ft layers, the maximum thickness of clay layers could be determined from the theory of consolidation as described here.

CONCLUSIONS

In this study the possibility of using high moisture content soils in high fills was judged on the basis of two considerations: (a) they must have sufficient strength to prevent shear failures, and (b) they must not cause the overlying structures to settle excessively.

A laboratory investigation of the soil used in this study revealed the following important facts:

1. Under the given methods of sample preparation and testing, the soil did not exhibit thixotropic behavior.
2. The shear strength increased appreciably with the increase in consolidation pressure, the rate of increase being more remarkable at low densities than at high densities.
3. The initial density had a tremendous effect on shear strength when the soils were not subjected to consolidation; however, its effect on strength diminished appreciably when the consolidation pressure was increased.
4. Under high consolidation pressures, a given soil with widely different initial densities and strengths finally arrived at some densities and strengths which were only slightly different.
5. Based first on the assumption that density is the most important factor affecting strength and then on Terzaghi's theory of consolidation, a procedure was suggested to determine the depth below which wet soils could be placed and the length of drainage path required to effect a given degree of consolidation.

Due to the large varieties of soil properties and fill dimensions, no analyses of stability and settlement for any fills were attempted. However, the facts clearly indicate that soils having moisture contents well above optimum can be used in high fills if proper drainage is provided. In fact, the consolidation process is superior to that of compaction from the standpoint of ultimate strength, as demonstrated by Leonards (9). Unfortunately, the beneficial effect of consolidation has not been properly utilized in the construction of fills.

It is believed that wet soils, unless too wet to be handled with construction equipment, can be used as fill materials if proper drainage is provided. However, if suitable materials can be found nearby, it may be more economical to use the borrow materials rather than the excavated materials because the cost of installing drainage facilities may be prohibitive. On the other hand, if large quantities of the soils excavated are too wet and suitable borrow cannot be obtained at a reasonable cost, the installation of drainage facilities in lieu of wasting the excavated materials may be the more economical practice. This demonstrates that a cost analysis must be conducted before any decisions on the use of high moisture content soils are made.

ACKNOWLEDGMENTS

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Discussion

J. A. TICE, Highway Research Engineer, Virginia Highway Research Council—The authors are to be congratulated on a well-written paper. It is hoped that this discussion will show another way of approaching the problem of how to best use wet materials in highway fills.

The results presented offer further proof that the shear strength increases with consolidation pressure, and corroborate the generally held idea that samples having initially different moistures and densities consolidate to a common void ratio. This latter point can be seen by extrapolating the e-log pressure curves in Figures 6 and 7. The data on thixotropy are interesting and indicate that samples wetter than those tested may possibly show some strength gains due to thixotropy. In any event the gains appear to be small.

The remainder of this discussion will be devoted to the section titled "A Suggested Procedure for Using High Moisture Content Soils," for it is here that the writer's opinion differs from that of the authors. The underlying philosophy of this section seems to be that if, after compaction and consolidation, the wet material has a void ratio corresponding to the void ratio of 95 percent of the standard AASHO maximum density, the wet soil can be used. This philosophy is based on the assumptions that (a) density is the most important factor affecting strength and (b) achievement of 95 percent of the standard AASHO maximum density is a desirable objective. Neither of these assumptions is particularly relevant to the main purpose of Huang and Shepard's study, namely, to investigate the possibility of using these soils (wet soils) in high fills and the manner in which they should be used.

The key questions concerning an embankment are (a) can it be built without undergoing a shear failure and (b) will future settlement affect the highway? Although the authors treat the second question, they ignore the first, evidently assuming that if 95 percent standard AASHO maximum density is achieved, there will be no problems. This is not so. Achievement of a specified density does not mean that the consolidated shear strength will be sufficient to support the height of fill needed to produce the required consolidation pressure. Regardless of the placement conditions of the soil, consolidation of the embankment will result in an equilibrium being established between the weight of the embankment and the density (void ratio) of the soil. If, at this equilibrium, the shear strength of the soil is sufficient to support the embankment, the actual density is irrelevant (considering here embankments at least 10 ft in height where the traffic load is negligible compared to the weight of the embankment). Huang and Shepard recognize that trying to compact the wet soil to 95 percent standard AASHO maximum density may be harmful, but still they believe that achievement of this density by consolidation is a desirable goal. As they point out for one of the soils used in the study, requirement of this density results in the requirement that about 90 ft of fill be above the wet material. Fills this high are rare, at least in Virginia. Were density after consolidation disregarded, it is quite possible that much lower fills could be built with wet materials in the lower portion.

Since, in the writer's opinion, shear strength is a major consideration in embankment design, and since shear strength, an easily measured property, increases with consolidation pressure, why concern ourselves with the ultimate density? The writer believes a more reasonable procedure would be one based on the increase of shear strength with consolidation pressure. This would also allow more flexibility in the use of wet materials in fills. An alternative approach has been developed, but time and space do not permit a detailed explanation of this alternative.

As can be seen, the writer's main objection to Huang and Shepard's procedure is their emphasis on attaining a consolidated density equal to a compacted density required by existing specifications. Although specification limits are important, the results of this research project show that existing specifications are not adequate with regard to the use of wet materials and that, if wet materials are to be used, specifications on placement density as well as on placement water content need to be revised.