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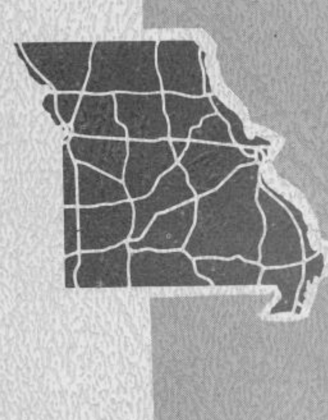
71-9

MOISTURE, DENSITY AND SLOPE REQUIREMENTS IN HIGH FILLS



MISSOURI STATE HIGHWAY DEPARTMENT
FEDERAL HIGHWAY ADMINISTRATION

MOISTURE, DENSITY AND SLOPE REQUIREMENTS IN HIGH FILLS

STUDY NO. 71-9

Prepared by

MISSOURI STATE HIGHWAY DEPARTMENT

Division of Materials and Research

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ABSTRACT

Eight soils commonly used in highway construction in Missouri were selected for study based on diversity in geologic origin, areal distribution and range in physical characteristics. Physical indices, consolidation and shear strength characteristics were determined. Theoretical pore pressures possible in field conditions were calculated based upon procedures outlined by Hilf. Effective stress stability analyses were performed to relate molding moisture at constant compactive effort to height of fill, angle of slope and factor of safety. In all cases, saturated strengths were assumed as a limiting condition. Potential settlements were related to molding moistures at constant compactive effort for various heights of fill. Limited investigation was made of the effects of variable compactive efforts. Results of the study are believed to be in reasonable agreement with limited data available from case studies.

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LIST OF ABBREVIATIONS AND SYMBOLS

C_c	–	compression index
CH	–	inorganic clay of high plasticity (ASTM D-2487-69 Classification of Soils for Engineering Purposes)
CL	–	inorganic clay of low to medium plasticity (ASTM D-2487-69 Classification of Soils for Engineering Purposes)
C_v	–	coefficient of consolidation
c	–	cohesion intercept on y-axis for Mohr's envelope of shear strength based on total stresses
c'	–	cohesion intercept on y-axis for Mohr's envelope of shear strength based on effective stresses
e_c	–	void ratio at preconsolidation stress
e_o	–	void ratio of trimmed specimen
F.S.	–	factor of safety
h	–	height
KSF	–	kips (1000 lbs.) per square foot
LL	–	liquid limit
ML	–	inorganic silt (ASTM D-2487-69 Classification of Soils for Engineering Purposes)
N	–	normal component of overburden load
O.M.	–	optimum moisture
P_a	–	atmospheric pressure
P_c	–	preconsolidation stress
PI	–	plasticity index
P_2	–	intensity of overburden load
R	–	shrinkage ratio
R_u	–	pore pressure ratio
S	–	shear test, drained
S.G.	–	specific gravity
SL	–	shrinkage limit
S_o	–	degree of saturation, original
s	–	shear strength of soil
u	–	intensity of the internal excess pressure or pore pressure
u_a	–	intensity of the internal excess gas (air) pressure
u_w	–	intensity of the internal excess fluid pressure
V_a	–	volume of gas (air) in a unit total volume of soil mass
V_w	–	volume of fluid (water) in a unit total volume of soil mass
w	–	moisture content
γ	–	density, weight per unit volume
Δ	–	increment of change

σ	–	total normal stress
$\bar{\sigma}$	–	effective normal stress
ϕ	–	angle of inclination of Mohr's envelope for shear strength based on total stresses
ϕ'	–	angle of inclination of Mohr's envelope for shear strength based on effective stresses
X	–	a factor for determining internal excess pressures relating to the degree of saturation of the soil and to the intensity of the overburden load

INTRODUCTION

The Missouri State Highway Department has experienced massive fill failures in recent years where design densities were achieved. Investigation indicated the fill soils were placed very wet of optimum. Other fills, placed dry of optimum, have exhibited severe deformations with adverse effects on pavements and structures. Dips and sags in fill sections throughout the state clearly indicate that design and compaction practices have not always achieved desirable results.

Missouri standards require a minimum of 90% of AASHO T-99 maximum density except near bridge ends and in the top 18 inches of the subgrade beneath flexible pavements. Moisture controls, as used, primarily serve to control the effects of swell on the pavement or to facilitate attainment of design density where tests are made with difficulty as in rocky clay soils.

Although foundations of embankments are investigated and analyzed routinely with respect to stability and settlement, these characteristics of fills are rarely considered except on the basis of past experience. It seemed desirable, therefore, that consideration should be given to developing a background of engineering data useful for predicting behavior of specific soil types, with respect to settlement and stability, when used in high fills. Secondly, it was hoped that development of this data would lead to more general application in the form of design criteria and specifications to govern construction.

CONCLUSIONS

A background of basic soils engineering data has been accumulated for the eight soils studied. Significant differences were found in physical properties and predicted behavior. These differences are discussed in detail under a section of this report entitled "Implications of the Study with Respect to Soil Type".

The test data derived from the study soils has been used to develop predictions of stability and settlement characteristics of embankments of 30 to 80 feet in height. These characteristics suggest limitations, with respect to embankment height, slope, moisture content and density, which are believed valid for the particular soils as tested, subject to the assumptions and limitations inherent in the study. Care should be exercised in extrapolating data and predicted behavior to other than the study soils.

Most of the study soils were shown to be capable of developing internal pore pressures in embankments of 80 feet or less in height, when compacted at moisture contents above optimum, such that embankment stability would be effected adversely. The two soils of loessial origin were found to be most sensitive to pore pressure development and a residual clay least sensitive.

The most desirable range of moisture contents to minimize both settlement and stability problems was found generally to be somewhat dry of optimum. For the CL and CH soils studied, however, there appears to be a danger that compaction very dry of optimum, without greater compactive effort to further modify soil structure, can lead to objectionable settlements. These settlements could occur with increases in moisture content at some time after completion of a fill and appear as a collapse phenomena.

The two soils of loessial origin did not appear prone to either settlement or stability problems when compacted dry of optimum provided normal minimum densities were achieved.

Assuming the normal 90% of AASHO T-99 maximum density requirement is always met, settlements within embankments constructed of any of the study soils should rarely be of great concern for fills less than about 50 feet in height. For fills above this height, special density requirements may be justified.

Areas for additional research are indicated. There is a need for relating field tests and field construction conditions to those of the laboratory. Particular problem areas include curing periods and molding techniques to achieve desired densities while simulating field compaction. The influence of variable gravel content on residual soils deserves investigation with respect to pore pressure development, strength and consolidation characteristics. Such soils are difficult to test for field control purposes and moisture controls properly designed for optimum performance would be of great practical value.

Also needed are more detailed analyses of embankments as constructed, with respect to pore-pressure development and strength, particularly where failures are involved. Investigations of such failures have implications beyond the immediate repair of the failed section and should be encouraged.

IMPLEMENTATION

There should be no "cookbook" approach to the implementation of the results of this study. Rather, it is suggested that soils specialists may best use this report as a guide, tempered by experience and awareness of the limitations and assumptions of the study and the variables between laboratory and field, in developing specific recommendations for specific problems.

The limitations suggested with respect to embankment heights, slopes, factors of safety and soil moisture contents are not the only considerations which should determine slope design. Swelling pressures, wet-dry and freeze-thaw cycles may also adversely effect the performance of slopes. Empirical suggestions are made under "Implications of the Study with Respect to Soil Types" for minimal slope designs to cope with these problems with the alluvial and glacial soils. Foundations, unusual phreatic conditions or susceptibility to erosion, for example, all may dictate even flatter slopes than suggested by this study.

There would appear to be little point in requiring special moisture or density controls for fills with heights within a range such that settlement or stability problems are shown to be unlikely and where normal minimum density requirements are likely to be achieved. Since the amount of settlement tolerable normally depends on how it will appear and feel in service, careful consideration should be given before applying special controls of any kind for control of settlement only.

The need for moisture controls for possible stability problems should be determined by evaluating both the moisture content of available borrow and the indicated slopes for adequate stability. Decisions on use of moisture controls should then be made by comparing the costs of flattened slopes without moisture control vs. steeper slopes with controls.

The moisture control expected to have most frequent application is an upper limit for the loessial soils which have been shown to be subject to serious stability problems when compacted wet of optimum. This confirms field experience. No evidence was found to indicate need for a lower moisture limit on such soils provided normal minimum density requirements are met. Contract special provisions should be developed to govern moisture control in accordance with these findings.

Consideration should also be given to requiring field moisture determinations as a part of the soil survey for use in decisions involving moisture control vs. slope flattening.

SCOPE

Eight soils, commonly used in highway construction in Missouri were selected for study. Factors considered in the selection were diversity of plasticity, areal distribution and geologic origin as well as past histories, whenever possible, as problem soils. Non-plastic or granular soils were excluded from consideration.

The selected soils were sampled and prepared for standardized tests with added procedural requirements to facilitate achieving specific aims of the study. Tests included indices tests, moisture-density relations, consolidation and drained direct shear. Moisture equalization and reduction of the effects of thixotrophy were provided for by prolonged curing periods during sample preparation. A basic premise utilized in the study involves use of the consolidation test on dynamically molded specimens as an indicator of retained energy and use of consolidation strain rates to calculate and predict theoretical development of internal pressures in field applications. Drained direct shear tests are used as a basis for determining effective stresses with simplified procedures originally utilized by Hilf[1]. The more rigorous theoretical concepts of Bishop[2] and Skempton[3] are recognized but are considered too difficult for routine application in the laboratory for determination of shear strengths. Strength values considered were limited to saturated strengths determined from a range of molding moistures and densities. Strength parameters as used in the stability analyses were adjusted by a factor based upon the correlation of effective angle of internal friction vs. the plastic index as reported by Bjerrum and Simons[4]. The effect of negative pore pressure on unsaturated soil strength was ignored as a justifiable simplifying assumption.

Settlements are predicted from consolidation data through a range of moistures and densities for fills of 30, 50 and 70 feet in height so as to permit estimation of intermediate or excess heights.

Stability was analyzed by effective stress methods using the concepts previously described. A computer program, based on the Swedish circle method of analysis and using automatic search features, was used to analyze failures limited to embankments. The effect of foundations was ignored. Various heights of fills were analyzed so that curves could be developed relating slope angle and fill height, through a range of 30 to 80 feet, to molding moisture and factor of safety.

SOIL TYPES

Factors considered in selecting soils for this study included geologic origin, areal distribution, frequency of usage in highway construction and, wherever possible, a past history of construction or maintenance problems. Only fine grained soils were considered but, within this limitation, the widest possible range of texture and plasticity was sought.

To encompass a variety in geologic origins, two soils were selected from each of four categories, glacial, loessial, residual and alluvial. The general locations within the state from which the soils were selected are shown in Figure 1. For convenience, the soils are identified as R-1 through R-8 in the figure and throughout this report. A more detailed description follows of the selected soils.

R-1. This is a CL-ML soil of loessial origin from the bluffs adjacent to the Missouri River bottom in Clay County. Past soil survey practice has been to classify this soil, along with other loess deposits, only by a pedologic name, Knox. Loess and associated glacial and paleosoil stratigraphy in this area is complex and differentiation of the various units difficult. The unit sampled for study is tentatively identified as the Peoria loess from the Wisconsian stage of the Pleistocene.

The Peoria and other loesses commonly classified as Knox are used extensively in highway construction along the Missouri River and especially in the urban areas of Kansas City and St. Joseph. Fills of 50 to 60 feet in height are not uncommon. It does not have a reputation for being a problem soil except for erosional characteristics.

R-2. This is a CL soil of loessial origin found in the eastern part of the state, principally on the bluffs adjacent to the Mississippi River. Soil surveys commonly classify the soil by a pedologic name, Memphis. The unit sampled from Route 79 in Ralls County is tentatively identified as Roxana loess from the Wisconsian stage of the Pleistocene.

This soil is commonly found in fairly shallow deposits, usually overlying glacial tills north of the Missouri River. Extremely high fills built entirely of this soil are not common but some homogeneous fills of 50 feet in height have been built. Natural moisture contents are frequently high and its reputation as a construction material is poor. Adverse behavior encountered ranges from excessive elasticity when worked wet, as it frequently is, to sloughs and massive slides during construction.

R-3. This is a CH soil of residual origin classified in soil surveys as Crawford. Its occurrence is primarily in the western plains area of the state. It is derived from weathering principally of Mississippian age limestones and frequently is found with a high content of admixed chert gravel.

Crawford does not have a reputation as a problem soil. The sample selected for study from near Springfield in Green County is chert free and is somewhat more plastic than is typically encountered.

R-4. This is a CH soil of residual origin derived from decomposition of dolomitic limestones and found throughout the Ozark region of southern Missouri. It is classified pedologically in soil surveys as Clarksville. Usually, it is associated with varying amounts of admixed chert gravel but pockets of chert free clay are common. The sample selected for study from near Ellington in Reynolds County is free of significant granular content and is average in plasticity for such occurrences.

Some of the highest fills in the state have been and are to be built of Clarksville soils. Past construction and maintenance history has been mixed. While slides are not common, post-construction settlements have been serious in some cases and only unsightly in other cases.

R-5. This is a highly plastic CH soil of alluvial origin sampled from the Missouri River bottom in Holt County. It is classified in soil surveys as Wabash. The principal occurrence is throughout the glacial plains area of the state along tributaries of the Missouri and Mississippi Rivers. The sample selected is the most highly plastic of all the soils studied and, while atypical of most Wabash alluvium, is representative of a type of soil frequently encountered in fill construction from side borrow. Its reputation is poor. Fortunately, it is most frequently used in fills of low to moderate heights.

R-6. This is a CL glacial till sampled from Putnam County. The sample appears representative of the average glacial till found in this state with an insignificant granular content. Soils such as the one sampled have not been considered especially troublesome in past construction of fills which have been generally of moderate height. (However, some associated deposits of gley and gumbotil have been extremely troublesome. In physical characterics these appear most like R-5).

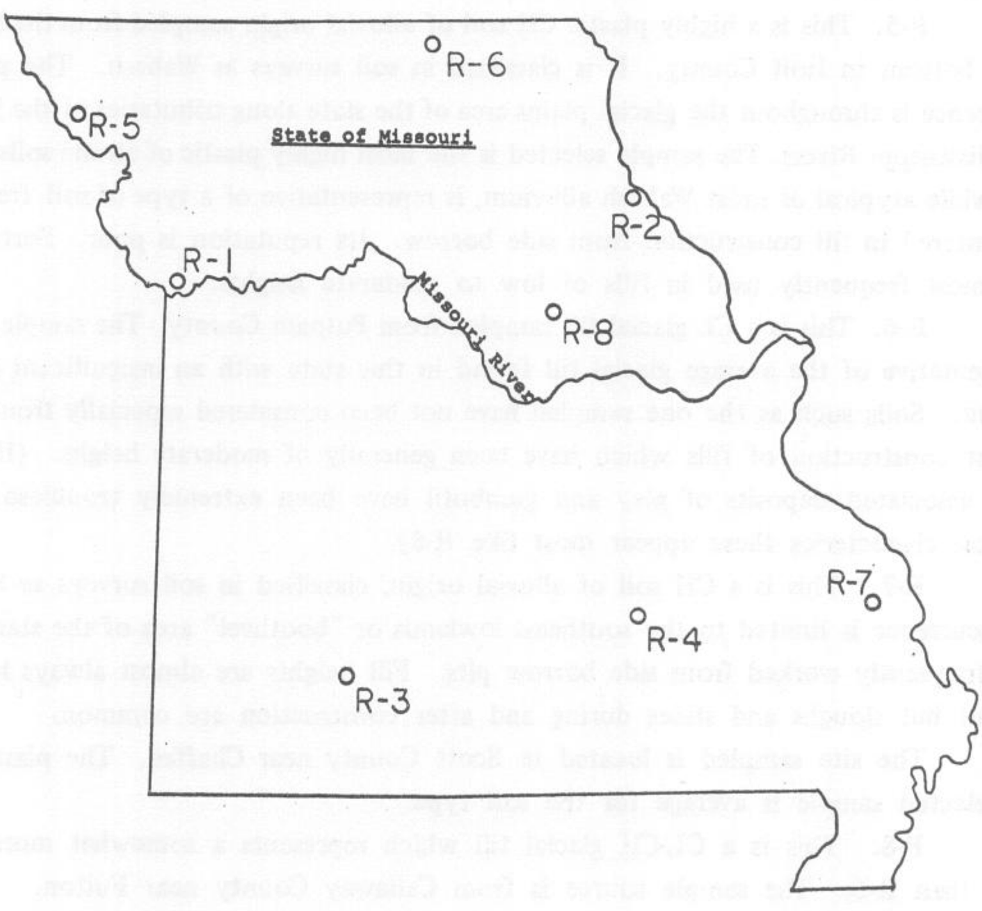
R-7. This is a CH soil of alluvial origin, classified in soil surveys as Sharkey. Its occurrence is limited to the southeast lowlands or "bootheel" area of the state where it is frequently worked from side borrow pits. Fill heights are almost always less than 40 feet but sloughs and slides during and after construction are common.

The site sampled is located in Scott County near Chaffee. The plasticity of the selected sample is average for the soil type.

R-8. This is a CL-CH glacial till which represents a somewhat more plastic range than R-6. The sample source is from Callaway County near Fulton.

FIG. 1

General location of sampled sites of study soils



SAMPLE PREPARATION AND ROUTINE AND INDICES TESTS

The samples of these soils were prepared for testing by oven drying for a minimum of 72 hours followed by crushing and screening through the #4 sieve. The soil was then mixed, split and stored in sealed polyethylene bags until testing.

Routine and indices tests performed included:

1. Mechanical analysis
2. Atterberg limits
3. Shrinkage limit
4. Shrinkage ratio
5. Specific gravity
6. Moisture density relations (AASHO T-99, Method C) with the following additional procedural requirements:
    a. The soil-water mixtures were aged (a minimum of 24 hours for clay) prior to molding to provide time for moisture equalization through the soil particles.
    b. Interfaces of the layers of the compaction samples were defined by narrow foil strips to facilitate identification during trimming of the consolidation sample.
    c. The compaction test specimens were ejected from the mold, wrapped in Saran and foil, waxed, and stored in controlled humid conditions.
    d. The specimens were aged (a minimum of 72 hours for silts and one week for clays) before testing to reduce possible effects of thixotrophy.

A summary of indices and routine test results for all of the study soils, R-1 through R-8, is included as Table 1. The A-line plot showing the range of the plasticity of the study soils is shown in Figure 2. The relationship of the moisture density curves is illustrated in Figure 3.

TABLE 1

Summary of Routine and Indices Test Data

Sample No.	Classification			LL	PI	% Smaller Than				SL	R	AASHO T-99 M.D., (pcf)	O.M.,%	S.G.
	Pedologic	ASTM	AASHO			#10	#40	#200	2μ					
R-1	Knox	CL-ML	A-4-(8)	31	8	--	100	94.2	17.5	20.8	1.71	110.3	15.8	2.60
R-2	Memphis	CL	A-6-(10)	37	16	--	100	98.2	23.5	17.4	1.8	108.6	17.3	2.62
R-3	Crawford	CH	A-7-6-(20)	59	31	97.0	95.6	93.9	53.5	20.0	1.86	95.7	27.4	2.67
R-4	Clarksville	CH	A-7-6-(20)	69	41	97.2	95.3	77.8	59	18.4	1.76	90.7	28.5	2.68
R-5	Wabash	CH	A-7-6-(20)	78	50	--	--	100.0	63	14.4	2.01	89.5	27.9	2.62
R-6	Glacial Till	CL	A-6-(12)	39	17	98.6	91.9	71.6	33	13.9	1.96	113.2	15.5	2.66
R-7	Sharkey	CH	A-7-6-(19)	56	36	--	99.8	97.8	43.5	12.7	1.95	97.8	23.6	2.61
R-8	Glacial Till	CL-CH	A-7-6-(8)	51	33	96.2	90.8	83.1	39	11.3	2.03	108.6	18.0	2.67

Key to Symbols

LL .... Liquid limit
PI .... Plasticity index
SL .... Shrinkage limit
R .... Shrinkage ratio

M.D. .... Maximum density
OM .... Optimum moisture content
S.G. .... Specific gravity

FIG. 2

A-Line plot of study soils .

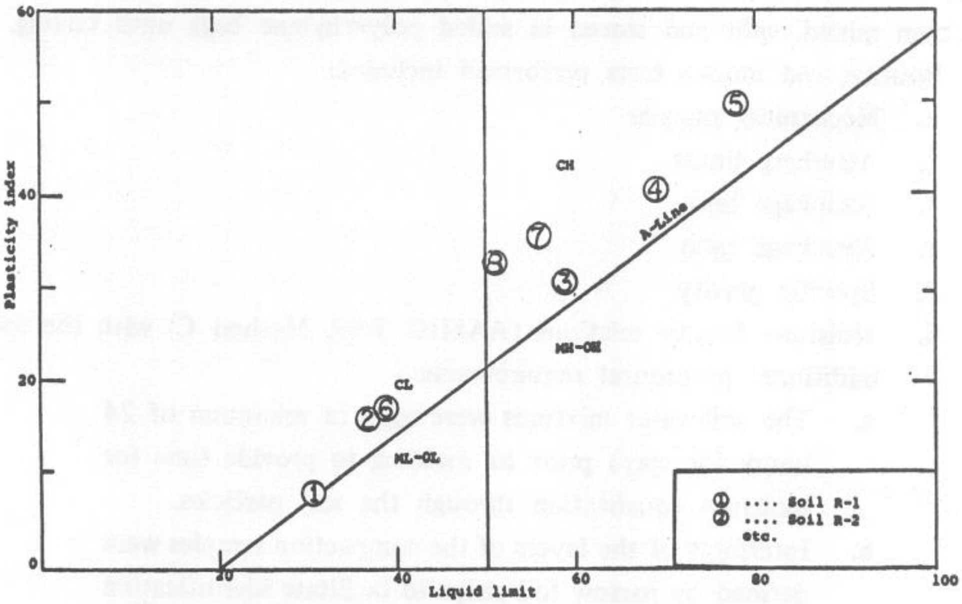
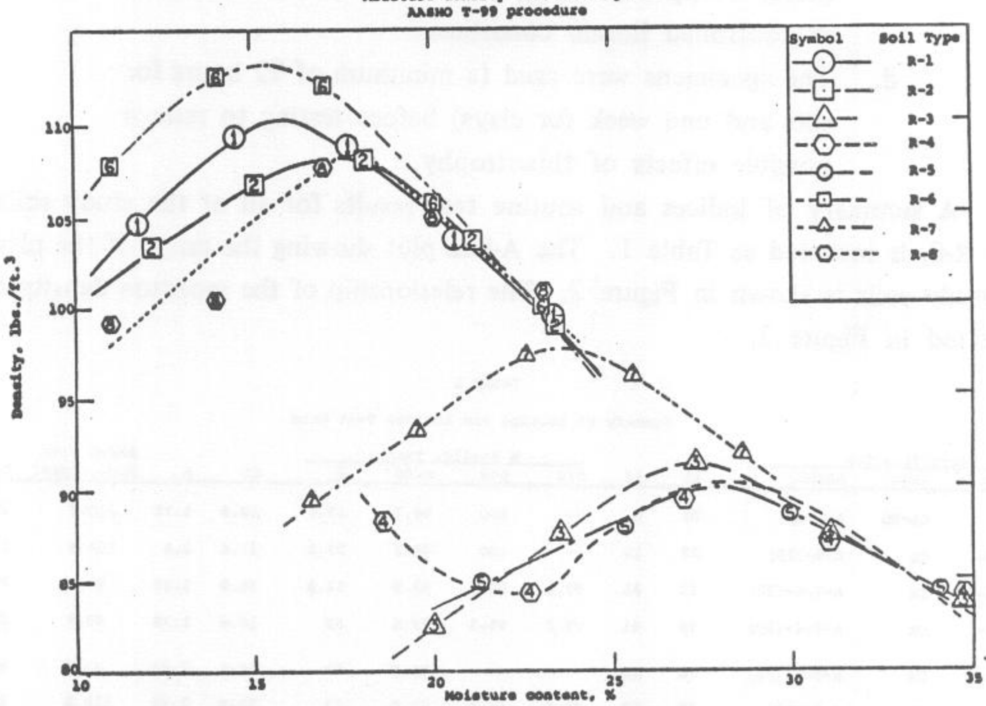


FIG. 3

Moisture-density relationships
AASHO T-99 procedure



PREPARATION OF CONSOLIDATION SAMPLES

The basic series of tests for each soil were performed on five specimens molded for determination of an AASHO T-99 moisture-density relationship curve. These five specimens were defined by points on the curve at 3 to 4% moisture intervals.

Soils R-2 and R-4 were chosen for additional testing due to their past history of stability or settlement problems and because they differed widely in plasticity. Three specimens of each soil were molded in accordance with AASHO T-180 (Method C). Three additional specimens of each soil were molded using T-99 procedures except that the energy input was reduced from 25 to 10 blows. This will be referred to elsewhere in the text and tables as "T-99, reduced effort".

The moisture-density relationship curves from the varied compactive efforts used with R-2 and R-4 are shown in Figures 4 and 5.

FIG. 4

R-2 Moisture density relationship for varied compactive effort

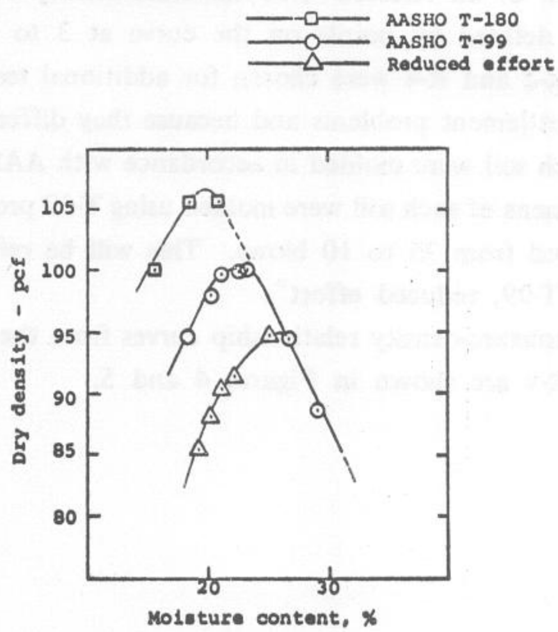
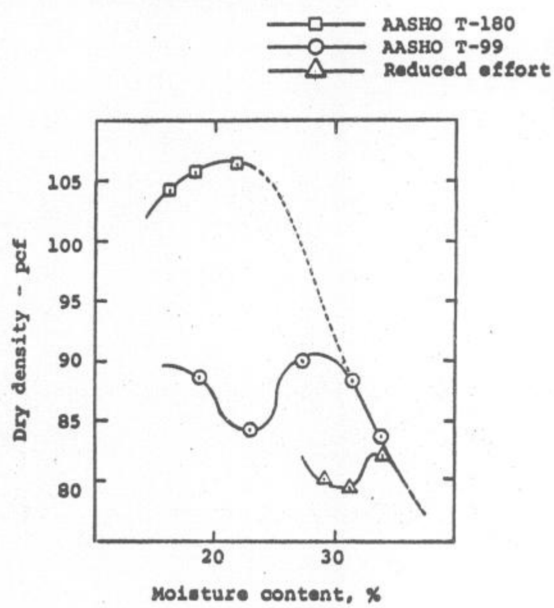


FIG. 5

R-4 Moisture density relationship for varied compactive effort



PREPARATION AND TESTING OF SHEAR SAMPLES

Static molding to predetermined densities was used to form the drained direct shear (S) test specimens. Loaded specimens of silty soils were soaked for a minimum of 3 days and the more plastic clays for 7 days to insure saturation before shearing. The coefficient of consolidation (C_V) and the preconsolidation pressure (P_C) were used to determine the initial load, the rate of loading and rate of applied shear stress.

The shear strengths were determined on samples prepared at varying degrees of compaction and moisture. These were:

1. 90% T-99 maximum density at optimum moisture.
2. 95% T-99 maximum density at optimum moisture - 25% of optimum moisture.
3. 95% T-99 maximum density at optimum moisture.
4. 95% T-99 maximum density at optimum moisture + 25% of optimum moisture.
5. 100% T-99 maximum density at optimum moisture.

Shear strengths are reported in Table 2 as effective stress shear strength parameters, Ø' and c', from drained consolidated tests which have been corrected for reduction of cross-sectional area. Ø' for the case of c'=0 was also determined for normal loads in excess of the preconsolidation value.

TABLE 2

Drained Shear Test Data

	90% Maximum Density			95% Maximum Density									100% Maximum Density		
	Optimum Moisture			Optimum Moisture - 25%			Optimum Moisture			Optimum Moisture + 25%			Optimum Moisture		
Soil	Ø'	c'	Ø'(c' = 0)	Ø'	c'	Ø'(c' = 0)	Ø'	c'	Ø'(c' = 0)	Ø'	c'	Ø'(c' = 0)	Ø'	c'	Ø'(c' = 0)
R-1	37	70	37	37	0	37	38	0	38	37	195	38	37.5	100	38
R-2	36	0	36	34	210	35	36	0	36	35	100	35	34	310	36
R-3	26	390	28	26	360	28	27	370	29	25	505	28	24	530	27
R-4	25	350	27	25	350	28	25	280	27	24	290	25	24.5	360	27
R-5	21	280	23	20	390	22.5	20	340	22.5	19	460	22	20	455	23
R-6	28.5	105	29	27.5	170	28	27	220	28	28	190	29	26	325	27
R-7	25.5	200	27	24	225	25	24	220	25.5	24	250	25.5	23	470	26
R-8	23	125	25	23	275	24.5	24	160	25	21.5	300	23.5	22	360	34.5

NOTES: Maximum density and optimum moisture as determined by AASHO T-99, Method C.

CONSOLIDATION TESTING

The consolidation tests were performed in accordance with ASTM D2535-70 in two series to be referred to as inundated and non-inundated.

Specimens for the non-inundated series were prepared by trimming from the middle of the top lift of the compaction test specimens. The specimens for the inundated test series were trimmed from the middle of the middle lift of the compaction test sample. After application of the 1/4 KSF load, the specimens were inundated and subsequent loads were applied as required to restrain swell. All consolidation tests were continued until a minimum of 3 points approximated a straight line on a void ratio, log of pressure plot, or to a maximum load of 48 KSF.

The results of all consolidation tests are summarized on Tables 3 through Table 12.

TABLE 3
Consolidation Test Data

Soil R-1 (Knox)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-99	Non-inundated[1]	11.8	.525	95.8	58.5	8.8	.503	.048	---
"	"	14.4	.450	100.8	83.5	18.6	.427	.082	---
"	"	17.7	.493	97.8	93.0	1.0	.474	.071	10
"	"	20.4	.565	93.3	94.0	0.7	.525	.097	11
"	"	23.3	.622	90.0	97.1	0.6	.591	.103	10
AASHO T-99	Inundated[2]	11.7	.506	96.9	60.0	15.8	.493	.114	---
"	"	14.4	.455	100.4	81.9	14.5	.438	.080	---
"	"	17.5	.487	98.2	93.3	11.2	.476	.078	10
"	"	20.4	.557	93.8	95.1	1.0	.518	.092	11
"	"	23.2	.629	89.6	95.7	0.6	.605	.107	13

TABLE 4
Consolidation Test Data

Soil R-2 (Memphis)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-99	Non-inundated[1]	13.1	.561	96.2	61.3	6.6	.517	.174	30
"	"	15.0	.517	99.1	76.2	12.6	.493	.134	10
"	"	15.9	.493	100.8	84.7	10.5	.465	.119	7
"	"	17.3	.489	101.0	92.2	7.4	.452	.092	5
"	"	17.9	.486	101.2	96.4	5.3	.455	.111	10
"	"	21.4	.567	96.0	98.5	2.3	.538	.134	4
"	"	23.7	.663	90.5	93.7	1.1	.633	.161	4
AASHO T-99	Inundated[2]	12.3	.567	96.0	56.7	6.0	.561	.182	40
"	"	14.7	.512	99.5	75.3	14.8	.498	.130	30
"	"	15.9	.500	100.2	81.9	8.7	.473	.116	12
"	"	17.0	.489	101.0	91.1	6.5	.471	.104	15
"	"	17.9	.485	101.4	96.5	5.9	.437	.111	10
"	"	21.4	.560	96.5	96.2	1.6	.547	.131	3
"	"	23.7	.638	91.9	96.9	0.9	.607	.152	3

1. Consolidated at molded moisture content in saturated atmosphere.
2. Consolidated with specimen inundated at ¼ ksf load.

Key to Symbols

w, ... Initial moisture content
e_o .... Void ratio of trimmed specimen
M.D. .. Maximum density
S_o .... Initial degree of saturation
P_c .... Preconsolidation pressure
e_c .... Void ratio at preconsolidation pressure
C_c .... Compression index
C_v .... Coefficient of consolidation

TABLE 5

Consolidation Test Data

Soil R-3 (Crawford)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.²/day x 10^{-2}
AASHO T-99	Non-inundated[1]	19.4	1.022	84.9	50.8	20.4	.965	.432	---
"	"	24.0	.822	94.1	77.8	17.8	.790	.174	---
"	"	26.7	.736	98.9	97.0	11.2	.647	.186	---
"	"	28.7	.809	94.6	94.6	5.7	.742	.204	3
"	"	33.4	.922	89.3	96.7	7.4	.866	.242	1
"	"	34.9	.972	87.0	95.8	3.3	.952	.258	1
AASHO T-99	Inundated[2]	20.4	.902	90.2	60.3	2.6	.896	.268	10
"	"	23.7	.791	95.8	80.0	5.9	.778	.254	2.5
"	"	26.4	.737	98.8	95.7	5.9	.719	.148	1.7
"	"	28.9	.819	94.3	94.1	5.7	.742	.228	2
"	"	33.4	.922	89.3	96.8	5.4	.898	.248	2
"	"	34.6	.962	87.4	96.1	3.8	.933	.280	1

TABLE 6

Consolidation Test Data

Soil R-4 (Clarksville)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.²/day x 10^{-2}
AASHO T-99	Non-inundated[1]	18.8	.850	99.4	59.3	1.7	.838	.282	7
"	"	23.1	.913	96.1	67.7	7.6	.886	.436	---
"	"	26.9	.764	104.3	94.2	8.1	.729	.216	---
"	"	30.6	.864	98.7	94.8	5.5	.807	.246	3
"	"	34.1	.974	93.2	93.8	1.9	.867	.266	2
AASHO T-99	Inundated[2]	18.7	.781	103.3	64.2	2.1	.761	.220	5
"	"	23.0	.856	99.1	72.0	5.8	.835	.245	7
"	"	26.3	.760	104.6	90.3	5.8	.760	.232	2
"	"	31.0	.862	98.8	96.1	4.5	.834	.254	2
"	"	33.9	.961	93.8	94.5	1.8	.925	.260	2

TABLE 7

Consolidation Test Data

Soil R-5 (Wabash)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.²/day x 10^{-2}
AASHO T-99	Non-inundated[1]	22.3	.884	96.3	66.2	10.0	.858	.502	12
"	"	26.3	.705	106.4	97.9	10.5	.680	.218	---
"	"	27.4	.789	101.5	90.8	7.1	.780	.238	0.4
"	"	30.1	.794	101.0	99.1	8.3	.732	.238	0.5
"	"	31.4	.841	98.6	97.7	8.1	.811	.291	0.6
"	"	34.5	.841	98.6	96.8	4.2	.867	.313	0.7
"	"	38.1	1.041	88.9	95.8	3.6	.931	.398	1.2
AASHO T-99	Inundated[2]	21.4	.791	101.3	70.8	3.8	.798	.252	16
"	"	25.4	.724	101.9	91.9	6.3	.728	.092	90
"	"	27.3	.813	100.1	87.8	6.6	.787	.284	0.4
"	"	29.0	.781	101.9	97.1	5.1	.778	.315	0.6
"	"	31.3	.841	98.6	97.5	5.1	.831	.303	0.5
"	"	34.3	.926	94.2	97.0	3.4	.911	.316	0.2
"	"	38.2	1.039	89.0	96.4	2.3	1.000	.366	0.7

1. Consolidated at molded moisture content in saturated atmosphere.
2. Consolidated with specimen inundated at ¼ ksf load.

Key to Symbols

w, ... Initial moisture content
e_o .... Void ratio of trimmed specimen
M.D. .. Maximum density
S_o .... Initial degree of saturation

P_c .... Preconsolidation pressure
e_c .... Void ratio at preconsolidation pressure
C_c .... Compression index
C_v .... Coefficient of consolidation

TABLE 8
Consolidation Test Data

Soil R-6 (Glacial Till)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-99	Non-inundated[1]	11.4	.428	102.3	62.8	21.4	.429	.332	---
"	"	14.1	.408	103.8	93.5	8.0	.409	.166	2.1
"	"	16.7	.432	102.0	94.0	5.0	.439	.144	1.8
"	"	20.2	.533	95.3	92.2	1.7	.532	.190	1.3
"	"	23.7	.647	88.7	94.4	0.85	.648	.212	1.5
AASHO T-99	Inundated[2]	11.5	.455	100.4	64.8	2.8	.455	.156	1.8
"	"	14.1	.477	98.9	88.8	4.5	.476	.170	2.1
"	"	16.6	.475	99.0	93.8	3.5	.476	.160	1.7
"	"	20.0	.528	95.6	92.4	1.6	.529	.180	1.3
"	"	23.5	.632	89.5	95.1	1.0	.632	.194	1.5

TABLE 9
Consolidation Test Data

Soil R-7 (Sharkey)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-99	Non-inundated[1]	17.3	.824	91.1	54.8	8.7	.762	.544	7
"	"	20.2	.670	99.5	78.4	8.3	.586	.314	9
"	"	22.7	.600	103.8	98.9	12.6	.573	.264	---
"	"	25.7	.682	98.7	98.1	5.4	.642	.214	0.6
"	"	28.8	.766	94.0	98.0	3.8	.756	.331	0.5
AASHO T-99	Inundated[2]	16.3	.729	96.1	58.0	3.7	.690	.252	0.7
"	"	19.1	.601	103.7	83.0	5.2	.579	.167	1.0
"	"	22.6	.619	102.6	95.1	6.3	.610	.231	0.7
"	"	25.3	.669	99.6	98.8	4.8	.660	.224	0.5
"	"	28.5	.768	93.9	96.9	2.6	.748	.293	0.5

TABLE 10
Consolidation Test Data

Soil R-8 (Glacial Till)

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-99	Non-inundated[1]	11.7	.734	88.4	42.4	11.7	.620	.458	---
"	"	14.4	.645	93.2	59.4	13.4	.583	.390	---
"	"	17.4	.485	103.2	95.4	12.9	.432	.188	---
"	"	20.4	.581	97.0	93.5	5.1	.525	.180	0.8
"	"	23.2	.671	91.7	91.9	2.1	.658	.234	0.6
AASHO T-99	Inundated[2]	11.5	.656	92.6	46.7	2.0	.628	.212	2.0
"	"	13.8	.536	99.8	68.7	4.5	.527	.188	1.5
"	"	16.8	.498	102.3	90.2	6.8	.498	.178	1.2
"	"	20.1	.579	97.1	92.7	4.2	.555	.203	1.0
"	"	23.4	.658	92.5	94.7	2.3	.639	.225	0.7

1. Consolidated at molded moisture content in saturated atmosphere.
2. Consolidated with specimen inundated at ¼ ksf load.

Key to Symbols

w, ... Initial moisture content
e_o .... Void ratio of trimmed specimen
M.D. .... Maximum density
S_o .... Initial degree of saturation
P_c .... Preconsolidation pressure
e_c .... Void ratio at preconsolidation pressure
C_c .... Compression index
C_v .... Coefficient of consolidation

TABLE 11
Consolidation Test Data

Soil R-2 (Memphis)

AASHO T-180 and Reduced Effort Compaction Procedures for Specimen Preparation

Compactive Effort	Test Condition	Soil w, %	R-2 e_o	(Memphis) % of M.D. (T-99)	S_o(%)	P_c(ksf)	e_c	C_c	C_v ft.^{2}/day x 10^{-2}
AASHO T-180	Non-inundated[1]	10.5	.487	101.2	56.3	41.7	.456	---	---
"	"	13.3	.417	106.4	83.8	20.4	.388	.040	---
"	"	15.6	.418	106.0	97.8	11.5	.383	.080	117
"	Inundated[2]	9.7	.458	103.2	55.5	24.0	.460	.092	32
"	"	12.9	.377	109.1	90.0	5.0	.376	.033	---
"	"	15.5	.411	106.6	98.7	3.7	.392	.064	4
Reduced Effort[3]	Non-inundated[1]	14.0	.716	87.5	51.1	1.7	.677	.214	70
"	"	15.0	.672	90.0	58.7	3.5	.636	.117	120
"	"	15.9	.632	92.1	65.9	8.3	.578	.241	35
"	"	16.9	.616	93.1	72.1	3.0	.532	.155	30
"	"	19.8	.569	96.0	91.0	3.2	.556	.130	25
"	Inundated[2]	13.8	.712	87.9	50.9	1.6	.689	.147	30
"	"	15.0	.672	90.0	58.5	1.9	.663	.183	35
"	"	15.8	.626	92.5	66.2	2.5	.623	.183	35
"	"	17.1	.587	94.7	76.2	6.6	.689	.153	30
"	"	19.8	.565	96.0	91.6	3.8	.575	.136	20

TABLE 12
Consolidation Test Data

Soil R-4 (Clarksville)

AASHO T-180 and Reduced Effort Compaction Procedures for Specimen Preparation

Compactive Effort	Test Condition	w, %	e_o	% of M.D. (T-99)	S_o%	P_c(ksf)	e_c	C_c	C_v ft./^{2}day x 10^{-2}
AASHO T-180	Non-inundated[1]	16.9	.651	109.8	66.8	22	.642	.260	---
"	"	15.8	.673	107.0	83.1	23.4	.658	.204	---
"	"	21.6	.596	115.3	97.1	20	.587	.116	---
AASHO T-180	Inundated[2]	16.8	.670	111.5	69.3	24.5	.650	.094	---
"	"	15.7	.587	119.1	77.3	23.4	.582	.128	---
"	"	21.3	.575	116.9	99.4	24.6	.561	.100	---
Reduced Effort[3]	Non-inundated[1]	28.5	.905	96.8	84.8	2.3	.880	.240	2
"	"	31.2	.970	93.4	86.2	2.1	.946	.260	2
"	"	33.7	.979	93.3	92.9	2.1	.921	.272	2
"	Inundated[2]	28.4	.920	96.7	84.1	2.2	.905	.250	3
"	"	31.0	.935	95.7	90.0	3.5	.905	.274	2
"	"	34.4	.975	93.2	94.5	2.9	.920	.248	2

1. Consolidated at molded moisture content in saturated atmosphere.
2. Consolidated with specimen inundated at ¼ ksf load.
3. Compaction effort reduced to 60% AASHO T-99 effort.

Key to Symbols

w, ... Initial moisture content
e_o .... Void ratio of trimmed specimen
M.D. .. Maximum density
S_o .... Initial degree of saturation
P_c .... Preconsolidation pressure
e_c .... Void ratio at preconsolidation pressure
C_c .... Compression index
C_v .... Coefficient of consolidation

THEORY AND APPROACH

Terzaghi's[5] effective stress equation is expressed as:

$$\bar{\sigma} = \sigma - u$$

where σ is total stress, $\bar{\sigma}$ is effective stress, and u is the pore pressure or the internal pressure of the fluids in the soil.

In a compacted cohesive soil the two fluids, air and water, are not at equilibrium. According to Bishop, the equation may be modified to:

$$\bar{\sigma} = (\sigma - u_a) + X\,(u_a - u_w)$$

where u_a is internal air or gas pressure, u_w is internal water pressure, and X is a factor relating to the degree of saturation varying from 0 for a totally dry soil to 1.0 for a fully saturated condition.

Blight[6] subsequently indicated that although the factor X cannot be satisfactorily determined, the sums of the two independent stress components, $(\sigma - u_a)$ and $(u_a - u_w)$, control the acting effective stress rather than the actual values of σ, u_a, or u_w. The component $(\sigma - u_a)$ indicates that internal gas pressures directly effect the intergranular effective stress of a compacted cohesive soil. The component $(u_a - u_w)$ is related to the relative amount of soil suction and directly effects the acting normal forces.

As the component $(u_a - u_w)$ only effects the normal forces acting in the soil, the basic tests conducted for this study were consolidation tests under saturated conditions to determine the compressibility of the soil structure and drained direct shear tests to determine the shear stress parameters. For these tests, conducted at atmospheric pressure, the negative capillary forces were considered at a minimum due to prolonged inundation, and the degree of saturation at the maximum that would be expected under field conditions. Additional, non-inundated consolidation tests were conducted for comparison purposes.

Yoshimi and Osterberg[7] have indicated that, for compacted cohesive soils, no outflow of water occurs with compression in the range of degree of saturation of 70 to 97%. This is due to $(u_a - u_w)$ being a negative stress under the condition of atmospheric pressure with no flow occurring against this negative gradient. This condition offers validity to assumptions of $\bar{\sigma} = \sigma - u_a$, with u_a determinable by volumetric strain measured from consolidation testing, and of no drainage occurring from a compacted soil mass. The assumption of $u_a = u_w$ for compacted cohesive soils should thus be satisfactory for establishing design criteria based upon the worst field condition, i.e., saturation.

This procedure of calculating u_a from volumetric strain was initially presented by Hilf and has been further verified by Yoshimi and Osterberg.

For the purposes of this study, the effective shear strength is determined by the modified Coulomb equation:

$$s = (\sigma - u_a) \tan \phi' + c'$$

where s is shearing resistance, tan ϕ' is the tangent of the angle of internal friction, and c' is the cohesion intercept from the direct shear test.

The pressure u_a was determined by Hilf's simplified formula which is based upon Boyle's Law and Henry's Law for the compressibility and solubility of gasses in solutions, in this case basically air and water. Hilf's simplified formula is:

$$u = \frac{P_a \Delta}{V_a + h (V_w - \Delta)}$$

where Δ is volumetric strain, P_a is atmospheric pressure, V_a is the initial volume of air, V_w is the initial volume of water and h is Henry's constant.

For comparison purposes the calculated internal excess pressures for the eight soils are tabulated in terms of the pore pressure ratio, R_u

$$R_u = \frac{u}{\gamma_d h}$$

where γ_d is the maximum dry density determined by the AASHO T-99 procedure and h is the height of fill.

Stability analyses were prepared by further modifying the Coulomb equation as follows:

$$s = N (1 - R_u) \tan \phi' + c'$$

(or)

$$s = N (1 - R_u) \tan \phi' \text{ (for the case of } c' = 0),$$

(whichever value is greater)

where N is the normal component of the weight of a slice of the circular failure arc. Shearing resistance across the base of each slice of the Swedish circle type stability analyses was computed on the basis of the formula which provided the maximum resistance to failure for that slice.

ANALYSIS OF DATA

Settlement Prediction

Settlements for the eight soil types have been calculated and are presented in Figure 6 through Figure 13 in terms of fill heights of 30, 50 and 70 feet. These calculated settlements are based on the following assumptions:

1. Vertical loads at any point in an embankment can be approximated by the height of fill at that point times a wet density equal to maximum dry density at optimum moisture.
2. The settlement of a finite layer is a function of the decrease in void ratio between the point of preconsolidation (P_c) and the vertical load (P_2) of the layer.
3. No settlement occurs under vertical loads less than the preconsolidation pressure.
4. No allowance is made for recompression or swell.

The predicted settlements are shown as total settlement in feet vs. placement moisture at a constant compactive effort. Settlements are also tabulated in Table 13 in terms of predetermined degrees of placement moisture, at optimum, optimum - 20% of O.M., and optimum + 20% of O.M. In both the figures and tables the settlements are expressed for both inundated and non-inundated cases. The effect of inundation on soils of low density compacted dry of optimum is apparent in Table 14. The degree of collapse which occurred during these series of consolidation tests is reflected by settlements that are predicted to occur at fill heights of 30, 50 and 70 feet through a range of placement moistures varying from optimum - 30% of O.M. to optimum - 10% of O.M. The settlement differentials between the inundated and non-inundated cases for the same heights of fill and placement moistures are indicative of what could occur should inundation or saturation occur after fill completion.

Consolidation samples, as trimmed from the mid-points of lifts from those compaction specimens molded dry of optimum, generally were calculated to have higher densities than the entire compaction specimens from which they were trimmed. This difference, significant only dry of optimum, varied from as little as 1% for an ML-CL soil to as much as 7% for a CH soil. Differences in densities were greater for the inundated series trimmed from the middle lifts of compaction specimens than for the non-inundated series trimmed from the top lifts. This indicates that settlements occurring with actual fill placement may be somewhat greater than the calculated settlements presented.

Consolidation test data from specimens of soil types R-2 and R-4, prepared by dynamic molding by "T-99 reduced effort" (See Preparation of Consolidation Samples) and by AASHO T-180, were used to predict the range of settlements that could occur under fill heights of 30, 50 and 70 feet for these varied compactive efforts. These predicted settlements are presented in Figures 14 and 15 and included in Table 15. Since the previously cited figures and tables are based upon constant compactive effort, with initial density variable as a function of moisture content, data was retabulated in Table 15 to reflect predicted settlements at a constant degree of density. 90% of T-99 maximum density, the minimum normally acceptable under Missouri specifications, was selected for this table.

TABLE 13

Predicted Fill Settlement (In Feet) vs. Placement Moisture

Soil	Test Condition	Optimum Moisture - (20% x O.M.) Fill Height 30 ft.	50 ft.	70 ft.	Optimum Moisture Fill Height 30 ft.	50 ft.	70 ft.	Optimum Moisture + (20% x O.M.) Fill Height 30 ft.	50 ft.	70 ft.
R-1	Non-Inun[1]	0	0	0	0	0.4	0.65	0.3	0.7	1.15
"	Inundated[2]	0	0	0	0	0.4	0.65	0.35	0.8	1.25
R-2	Non-Inun[1]	0	0	0	0	0	0.1	0	0.3	0.7
"	Inundated[2]	0	0	0	0	0	0.1	0.2	0.6	1.0
R-3	Non-Inun[1]	0	0	0	0	0	0.15	0	0.2	0.5
"	Inundated[2]	0.05	0.3	0.7	0	0	0.2	0	0.1	0.5
R-4	Non-Inun[1]	0	0	0.2	0	0	0.15	0.25	1.1	2.1
"	Inundated[2]	0	0	0.2	0	0	0.3	0.25	1.2	2.3
R-5	Non-Inun[1]	0	0	0	0	0	0	0	0.1	0.5
"	Inundated[2]	0	0.25	0.8	0	0	0.2	0	0.3	1.1
R-6	Non-Inun[1]	0	0	0	0	0.1	0.5	0.2	0.5	1.3
"	Inundated[2]	0	0.3	0.9	0	0.3	0.9	0.2	0.7	1.4
R-7	Non-Inun[1]	0	0	0	0	0	0.1	0	0.2	1.0
"	Inundated[2]	0	0.1	0.4	0	0.1	0.4	0.1	0.8	1.9
R-8	Non-Inun[1]	0	0	0	0	0	0.2	0.1	0.2	1.5
"	Inundated[2]	0	0.1	0.5	0	0	0.3	0.1	0.4	1.3

1. Consolidated at molded moisture content in saturated atmosphere.

2. Consolidated with specimen inundated at ¼ ksf loading.

TABLE 14

Predicted Fill Settlement vs. Placement Moisture
(Settlement in feet calculations based on both non-inundated and inundated consolidation for the fill heights and placement moistures indicated)

Soil	Optimum Moisture - (30% x O.M.) Fill Height						Optimum Moisture - (20% x O.M.) Fill Height						Optimum Moisture - (10% x O.M.) Fill Height					
	30		50		70		30		50		70		30		50		70	
	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun	Non Inun	Inun
R-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
R-2	0	0	0	0	0.1	0.1	0	0	0	0	0	0	0	0	0	0	0	0
R-3	0	0.2	0	1.3	0	2.5	0	0.05	0	0.3	0	0.7	0	0	0	0	0	0
R-4	0	0	0.7	0.4	1.5	0.8	0	0	0	0	0.2	0.2	0	0	0	0	0	0.2
R-5	0	0	0	0.4	0	1.3	0	0	0	0.25	0	0.8	0	0	0	0	0	0.1
R-6	0	0.2	0	0.75	0	1.5	0	0	0	0.3	0	0.9	0	0	0	0.2	0	0.65
R-7	0	0	0	0.3	0	1.0	0	0	0	0.1	0	0.4	0	0	0	0	0	0
R-8	0	0.1	0	0.4	0	0.9	0	0	0	0.1	0	0.5	0	0	0	0	0	0.2
Additional tests on specimens compacted at 40% of AASHO T-99 compactive effort[1]																		
R-2	0.6	0.6	1.9	1.9	3.5	3.5	0.2	0.4	0.7	1.5	1.3	2.5	0	0.2	0.1	0.8	0.35	1.6
R-4	No Data						Insufficient Data						0	0	0	0	0	0
Additional tests run on specimens compacted																		
R-2	0	0	0	0.05	0	0.1	0	0	0	0.1	0	0.2	0	0	0	0.1	0	0.3
R-4	0	0	0	0	0	0	0	0	0	0	0	0	No Data					

1 All references to optimum moistures are to those of AASHO T-99

TABLE 15

Fill Settlement When Compacted at 90% Maximum Density Vs. Varying Placement Moisture Contents

Soil Type	Compacted At AASHO T-99 Effort: 90% Maximum Density at Dry Side of Optimum[2] Fill Height			Compacted At Reduced Effort[3]: 90% Maximum Density at At Optimum Fill Height			Compacted At AASHO T-99 Effort: 90% Maximum Density at Wet Side of Optimum[2] Fill Height		
	30 ft.	50 ft.	70 ft.	30 ft.	50 ft.	70 ft.	30 ft.	50 ft.	70 ft.
R-1	0	0	0	No Information			.6	1.0	1.7
R-2	0	0	.4	.1	.3	.7	.6	1.7	2.8
R-3	---	.4	0.9	No Information			---	0.1	0.5
R-4	0.2	0.8	1.7 (see note 1)	.2	.8	1.7	0.6	2+	3+
R-5	0	.25	0.8	No Information			0.2	1.0	2.0
R-6	0.3	1.2	2.4	No Information			0.8	1.8	2.9
R-7	---	0.4	0.9	No Information			0.3	1.2	3.0
R-8	0.2	0.5	1.1	No Information			0.3	1.1	2.1

Note 1 Specimen molded at 18.7% H_2O and 93.9% maximum density
Note 2 Moisture content is that which gives indicated percent of maximum density with compactive effort used for AASHO T-99
Note 3 Compacted with 40% of compactive effort used in AASHO T-99
Note 4 All references to % of maximum density and optimum moisture are those of AASHO T-99

FIG. 6

Predicted settlement vs. placement moisture, Soil R-1

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

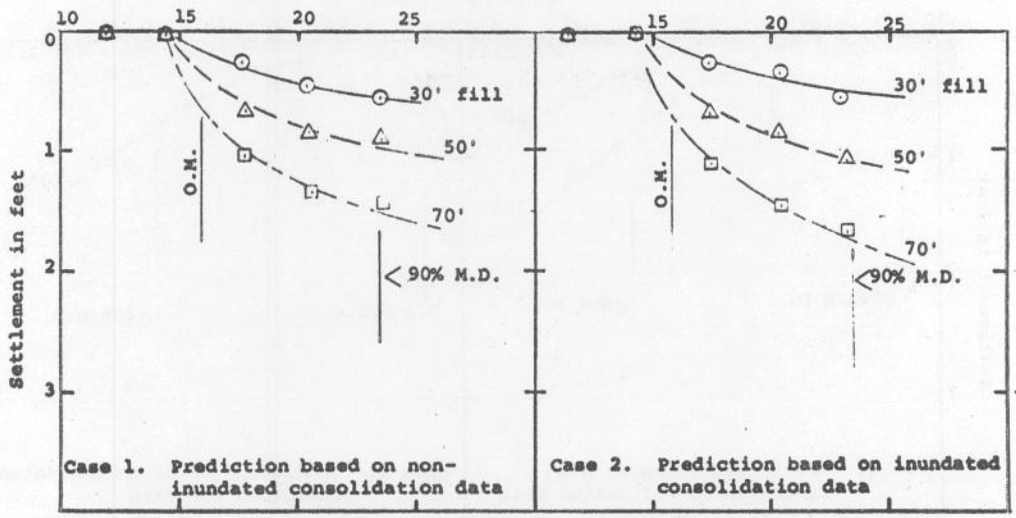


FIG. 7

Predicted settlement vs. placement moisture, Soil R-2

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

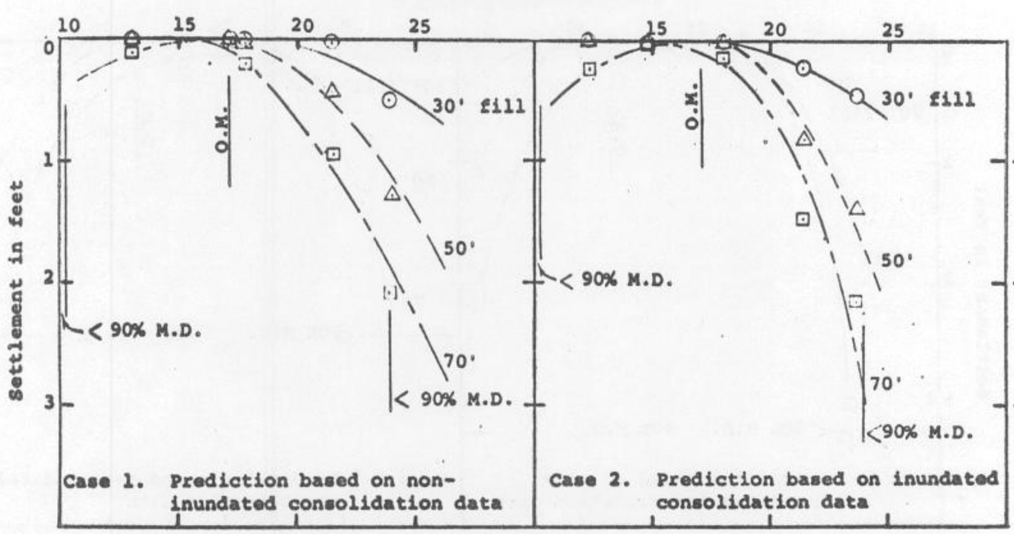


FIG. 8
Predicted settlement vs. placement moisture, Soil R-3

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

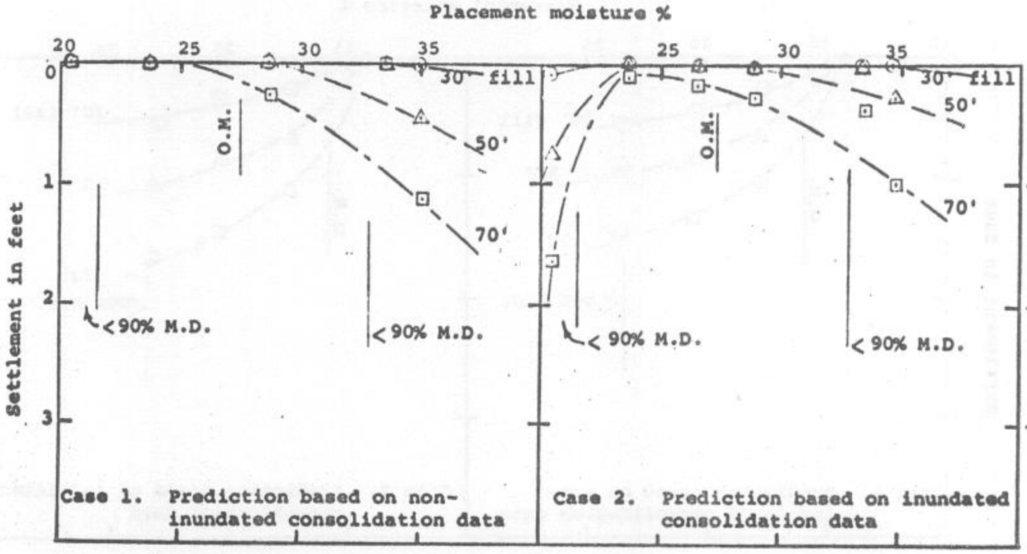


FIG. 9
Predicted settlement vs. placement moisture, Soil R-4

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

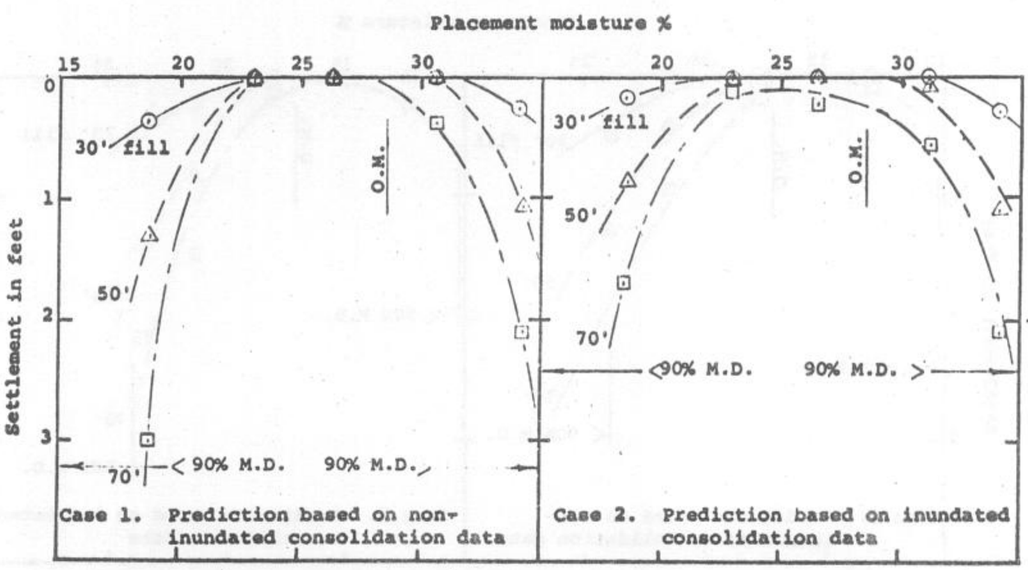


FIG. 10
Predicted settlement vs. placement moisture, Soil R-5

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

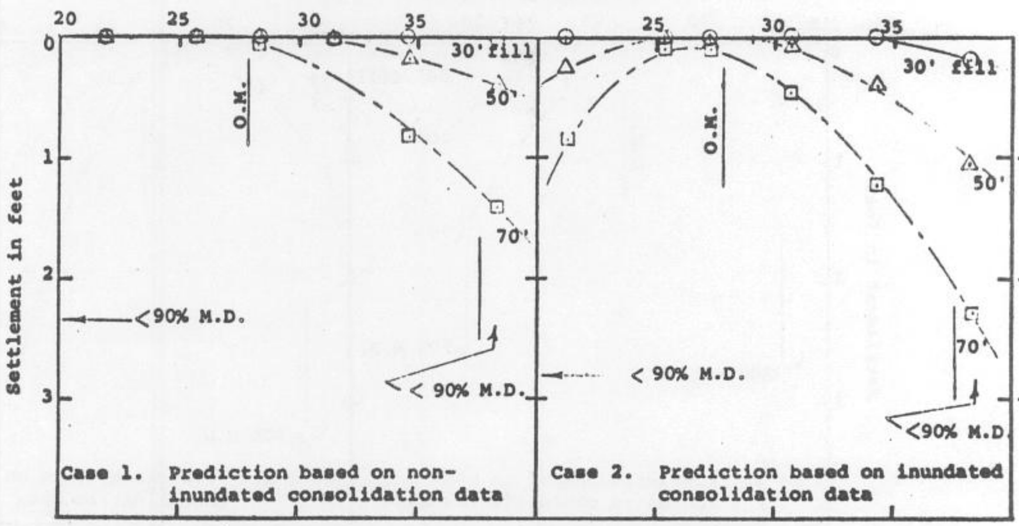


FIG. 11
Predicted settlement vs. placement moisture, Soil R-6

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

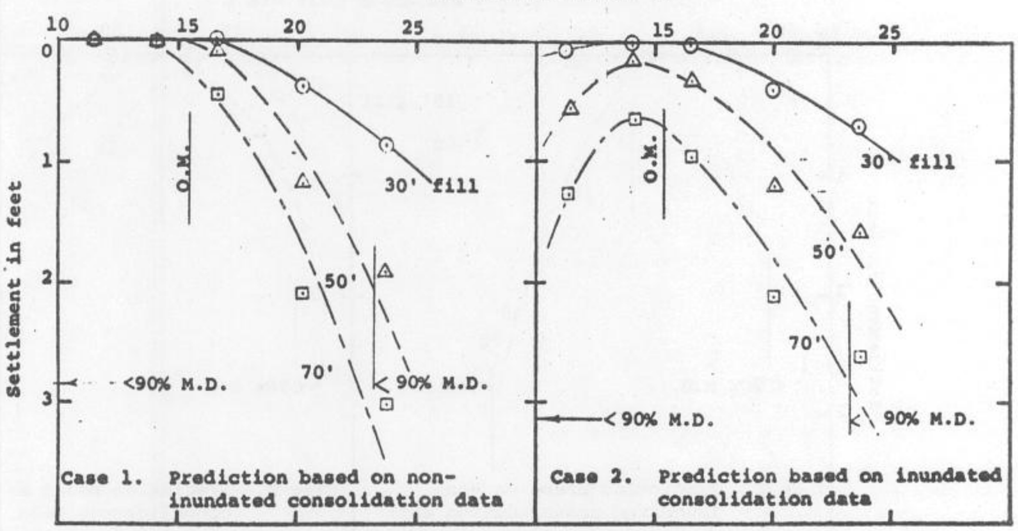


FIG. 12

Predicted settlement vs. placement moisture, Soil R-7

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

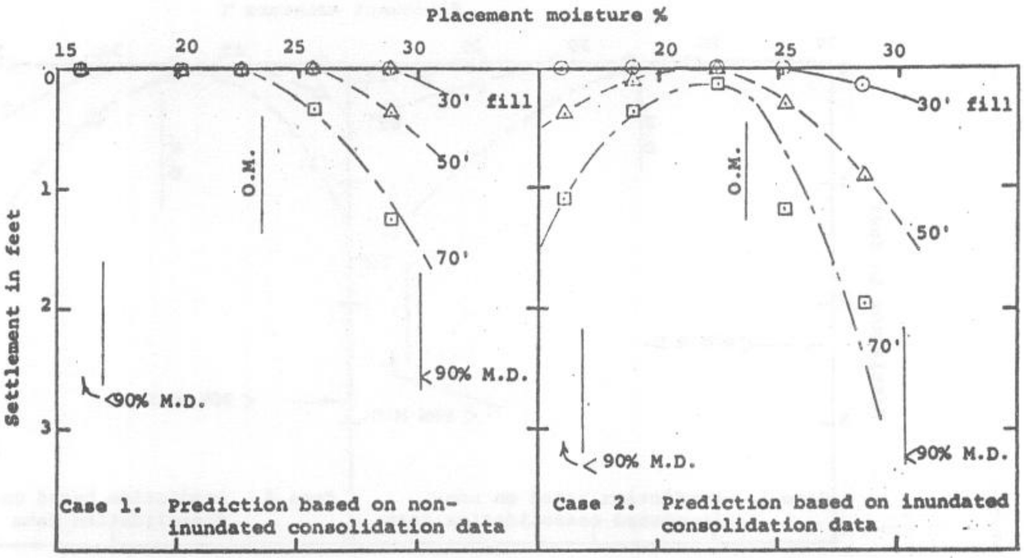


FIG. 13

Predicted settlement vs. placement moisture, Soil R-8

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

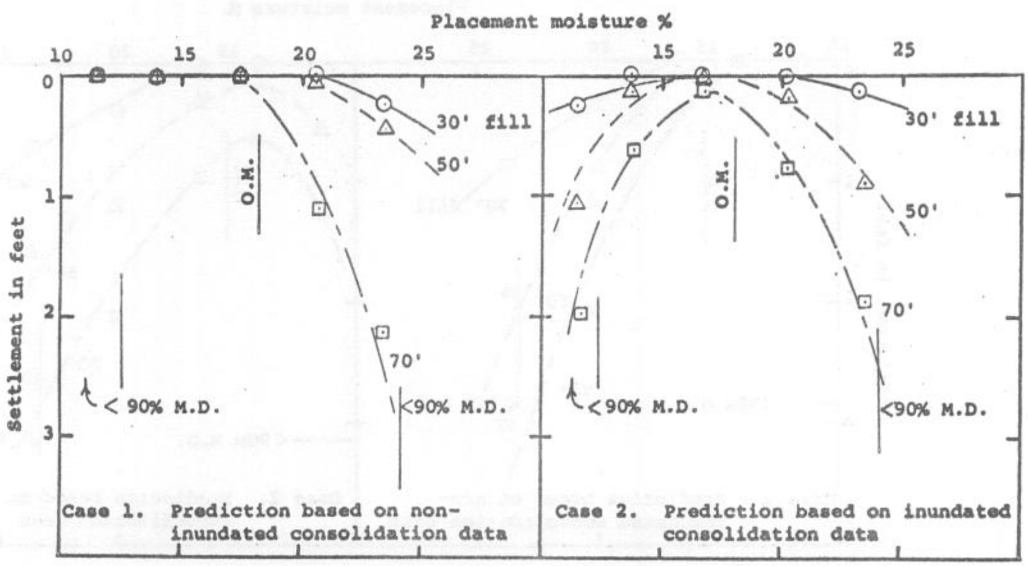


FIG. 14
Predicted settlement vs. placement moisture, Soil R-2
Reduced compaction effort*

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill

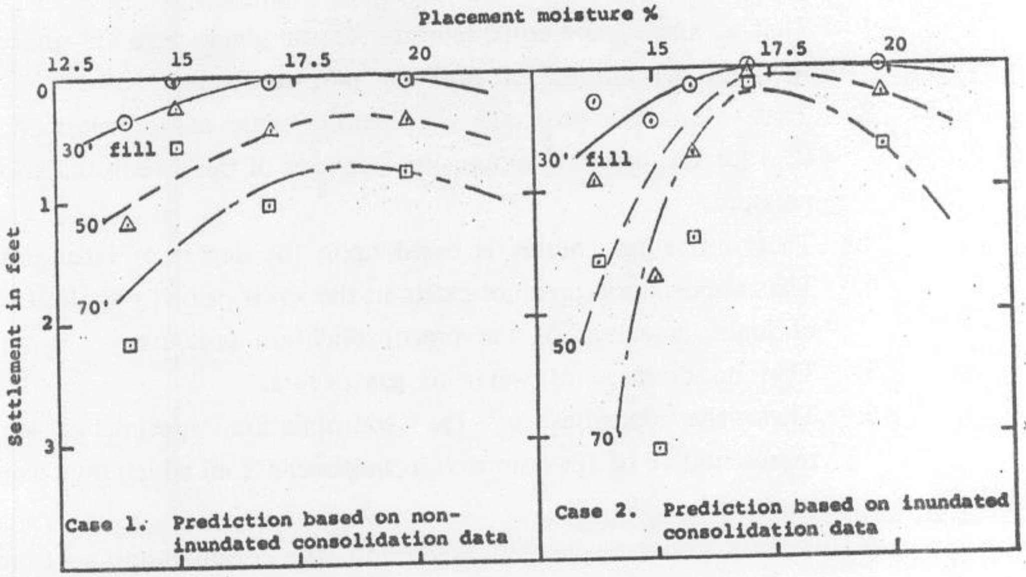
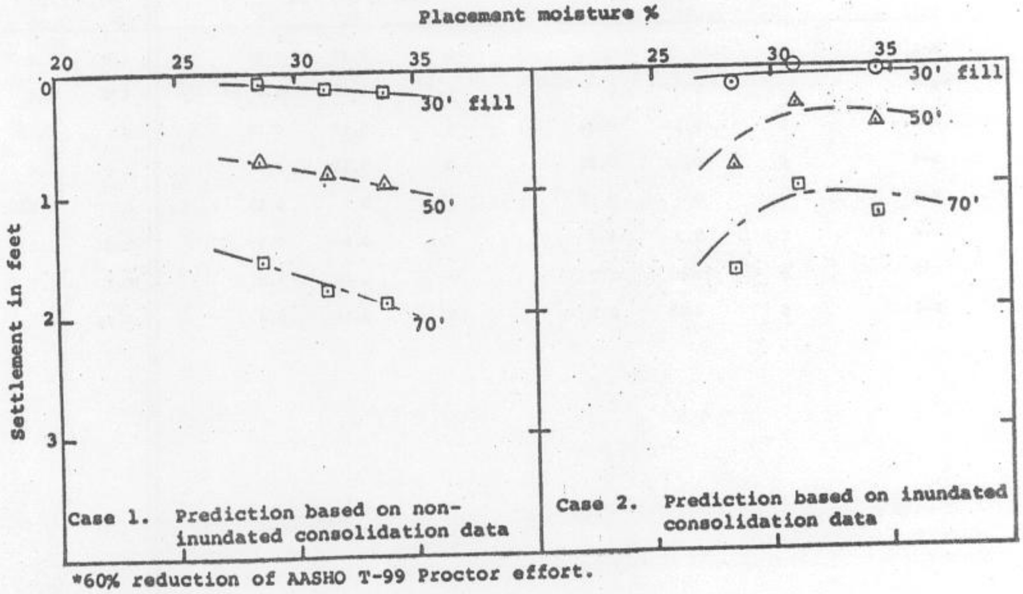


FIG. 15
Predicted settlement vs. placement moisture, Soil R-4
Reduced compaction effort*

Symbol	Equivalent P_2 loading
○	30' fill
△	50' fill
□	70' fill



*60% reduction of AASHO T-99 Proctor effort.

Excess Internal Pressure

Theoretical internal excess pressures are tabulated for heights of 30, 50 and 70 feet in Table 16, for each of the soil types, in terms of the pore pressure ratio R_u. These are presented in terms of degree of placement moisture for the range of optimum - 20% of O.M. to optimum +20% of O.M. These pore pressures, calculated according to Hilf's procedure, are based upon the following assumptions:

1. That u_a and u_w are equal (surface tension phenomena are ignored with no corrections for capillary pressures.)
2. That volume change occurs as a function of the compression index (C_c) for the pressure increments in excess of the preconsolidation pressure.
3. That initial gas volume is based upon the degree of saturation.
4. That atmospheric pressure exists in the voids prior to application of loads in excess of the preconsolidation pressure.
5. That no drainage of water or gas occurs.
6. That the densities of the consolidation specimens were representative of the compaction specimens from which they were trimmed.

As discussed under Settlement Prediction, the consolidation specimens actually tended to be somewhat denser than the total compacted specimen at moisture contents drier than optimum. Except wet of optimum where they are most significant, the calculated pore pressure ratios could therefore be somewhat high.

TABLE 16

Calculated R_u Values vs. Placement Moisture

	Optimum Moisture - (20% x O.M.)			Optimum Moisture			Optimum Moisture + (20% x O.M.)		
	Fill Height, Ft.			Fill Height, Ft.			Fill Height, Ft.		
Soil	30	50	70	30	50	70	30	50	70
R-1	0	0	0	0	0.25	0.85	1.0	1.0	1.0
R-2	0	0	0	0.2	1.0	1.0	1.0	1.0	1.0
R-3	0	0.14	0.14	0	0.15	0.18	0	0.18	0.9
R-4	0	0.03	0.05	0	0.12	0.35	1.0	1.0	1.0
R-5	0	0	0.11	0	0	0.15	0	0.78	1.0
R-6	0.1	0.3	0.4	0.1	0.8	0.9	1.0	1.0	1.0
R-7	0	0.03	0.2	0	0.08	1.0	0.9	1.0	1.0
R-8	0	0.05	0.1	0	0.25	0.4	0.45	1.0	1.0

Stability

The initial determination of the stability of various slope configurations was accomplished with the use of Singh's[8] stability charts. Based on these initial findings, computer analyses based on the Swedish circle type solution were used to determine stability at varying degrees of consolidation according to the principle of effective stress. The analyses were based on the following assumptions and conditions.

1. Foundation soils were not considered and all failures were limited to the fill.
2. Slopes considered varied between extremes of 1.5 and 4 to 1 depending upon the degree of slope required for factors of safety in the range of 1.0 to 1.5.
3. Loading was assumed instantaneous with no drainage occurring.
4. Both tan ϕ' and c' as averaged from test data were modified for use in stability analyses. Bjerrum and Simons reported a correlation of effective angle of internal friction (ϕ') to plastic index indicating that ϕ', for the case of $c' = 0$, decreases with increasing plasticity. This correlation, as shown in Navdocks DM-7, Soil Mechanics, Foundations and Earth Structures[9], is reproduced in Figure 16 of this report with a plot of the eight tested soils superimposed. The average tested strengths of these soils were adjusted for use in the analyses as follows: The tested shear strengths, both ϕ' and c', are adjusted by dividing them by a ratio of the tested angle of internal friction, for the case $c' = 0$, to the angle represented by the average correlation minus one standard deviation. Both averaged and modified shear strength parameters with the adjustment factors used are tabulated in Table 17.
5. It was assumed that the circle furnishing the lowest factor of safety was determined by the program used, a major modification of Bureau of Public Roads Program S-3. Choice of alternate parameters, tan ϕ' and c' or tan ϕ' for the case of $c' = 0$, was made by the program for each slice of each failure circle.
6. The fill was zoned into strata of constant vertical depth with the depth of the first stratum determined by the height of fill required to equal the preconsolidation pressure as determined by consolidation testing. The pore pressure ratio (R_u) was considered zero for this upper strata. Additional strata were

zoned down to a maximum depth of 80 feet. An average theoretical pore pressure ratio was used for each stratum. The average pore pressure ratios used in the analyses are listed in Table 16.

The results of these stability analyses are graphically illustrated in Figures 17 through 26 as fill height vs. degree of slope for various placement moistures and for two factors of safety, 1.0 (or failure) and 1.5, the minimum normally sought for long term stability.

TABLE 17

Shear Strength Parameters

Soil	Average values Tan Ø'	c'	Tan Ø' (c' = 0)	Adjustment factor	Adjusted parameters, as used for stability analyses Tan Ø'	c'	Tan Ø' (c' = 0)
R-1	.754	65	.767	-23%	.581	50	.591
R-2	.700	103	.715	-23%	.539	79	.551
R-3	.488	411	.539	-9.0%	.444	374	.491
R-4	.460	308	.503	-9.1%	.418	280	.457
R-5	.358	396	.410	+5.2%	.377	417	.431
R-6	.522	193	.547	-1.0%	.517	191	.542
R-7	.446	232	.474	+1.0%	.450	234	.478
R-8	.421	245	.452	---	.421	345	.452

FIG. 16

Relationship of plastic index to effective angle of internal friction

(8 Missouri soils plotted on Bjerrums and Simons correlation, as shown in Navdocks DM-7)

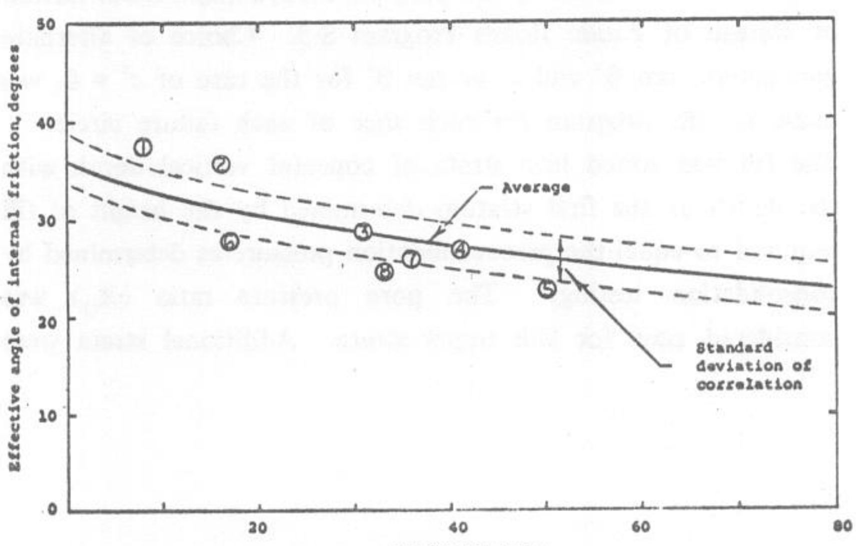


FIG. 17

Soil R-1

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content

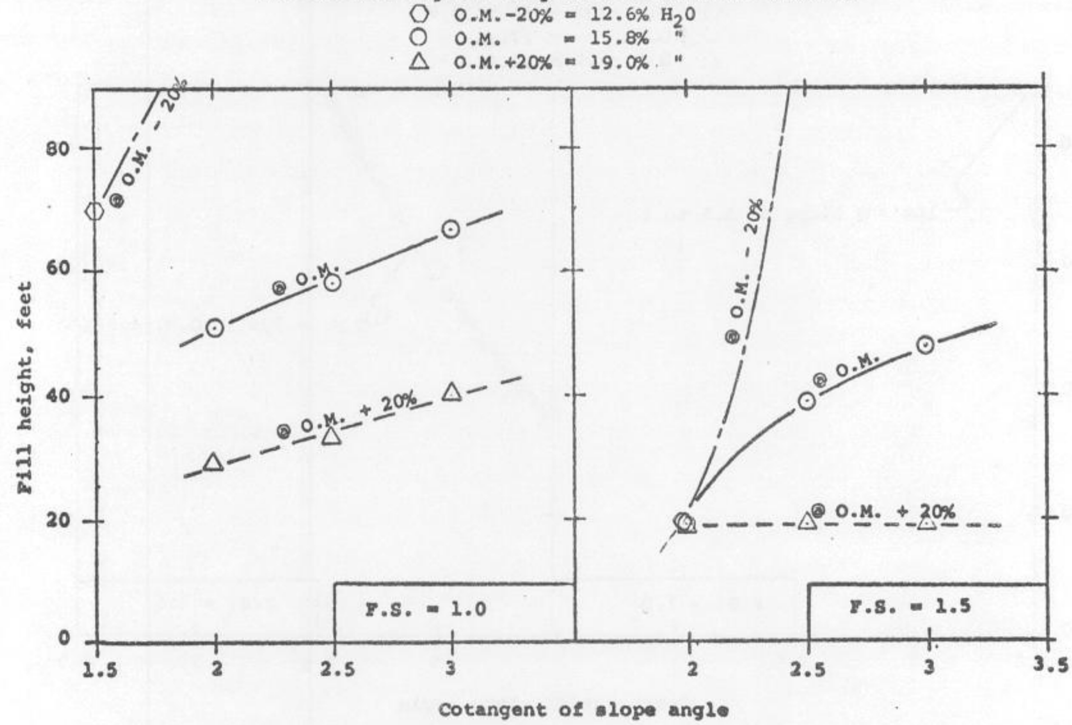


FIG. 18

Soil R-2

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content

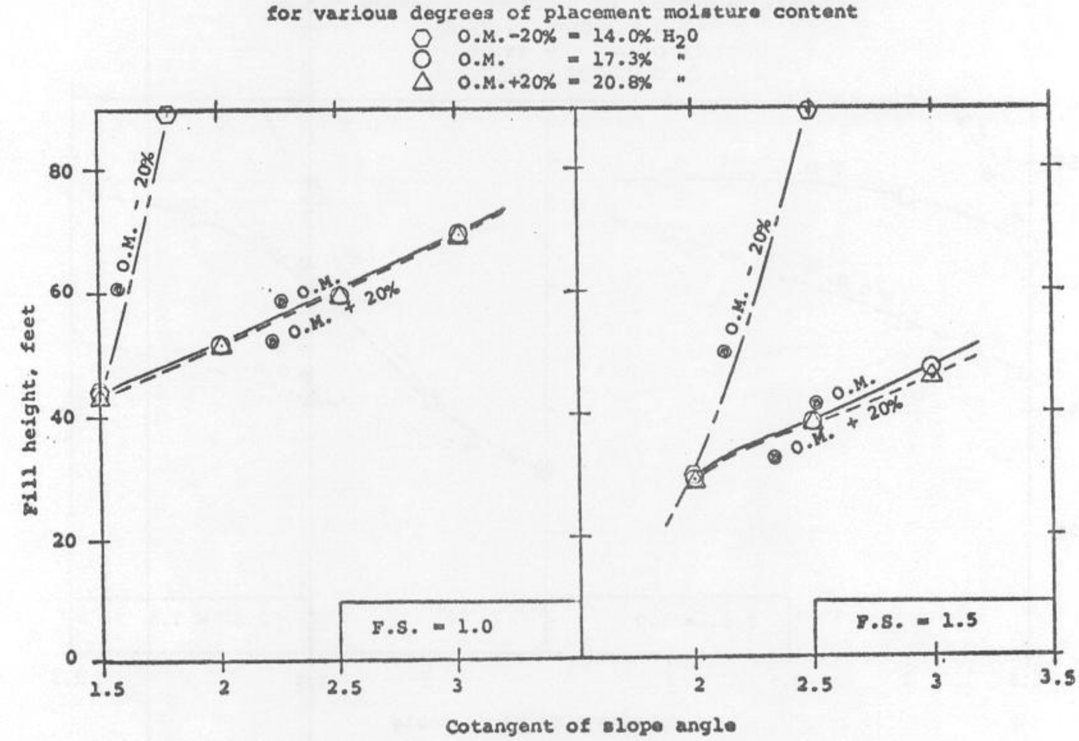


FIG. 19

Soil R-3

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content
⬡ O.M.-20% = 21.9% H_2O
○ O.M. = 27.4% "
△ O.M.+20% = 32.8% "

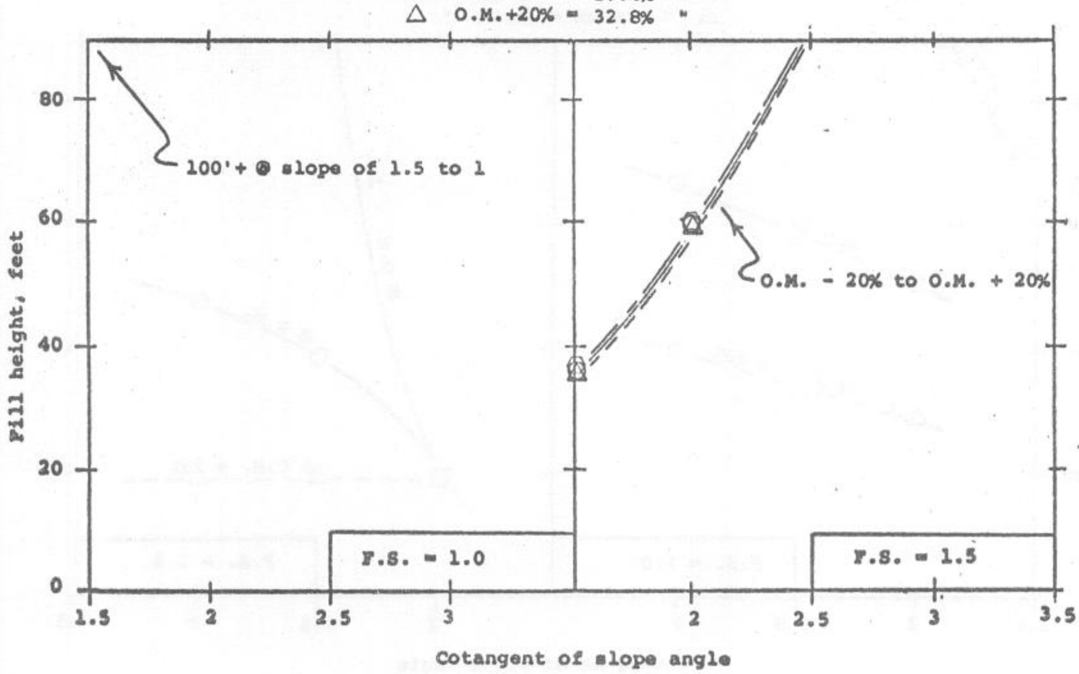


FIG. 20

Soil R-4

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content
⬡ O.M.-20% = 22.8% H_2O
○ O.M. = 28.5% "
△ O.M.+20% = 34.2% "

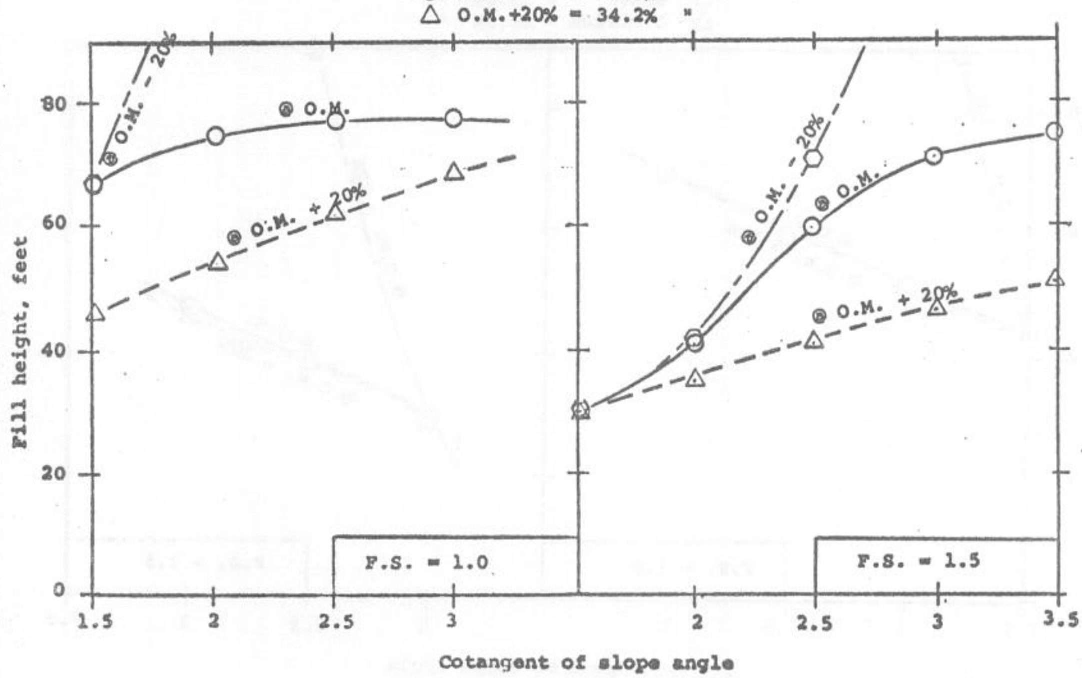


FIG. 21

Soil R-5

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content
⬡ O.M.-20% = 22.3% H_2O
○ O.M. = 27.9% "
△ O.M.+20% = 33.5% "

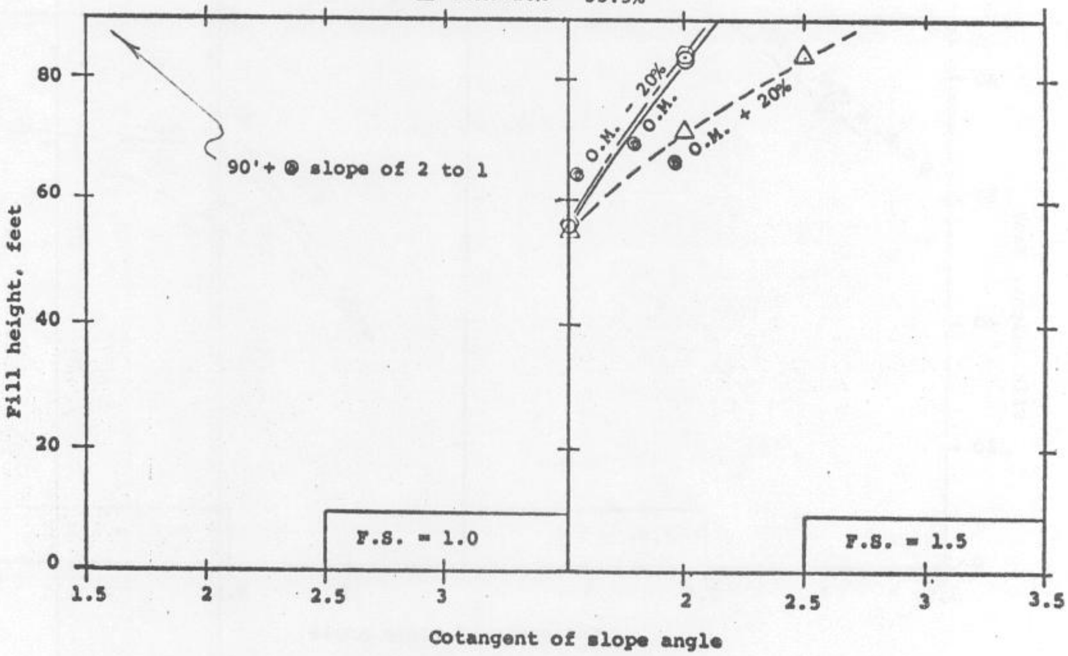


FIG. 22

Soil R-6

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content
⬡ O.M.-20% = 12.4% H_2O
○ O.M. = 15.5% "
△ O.M.+20% = 18.6% "

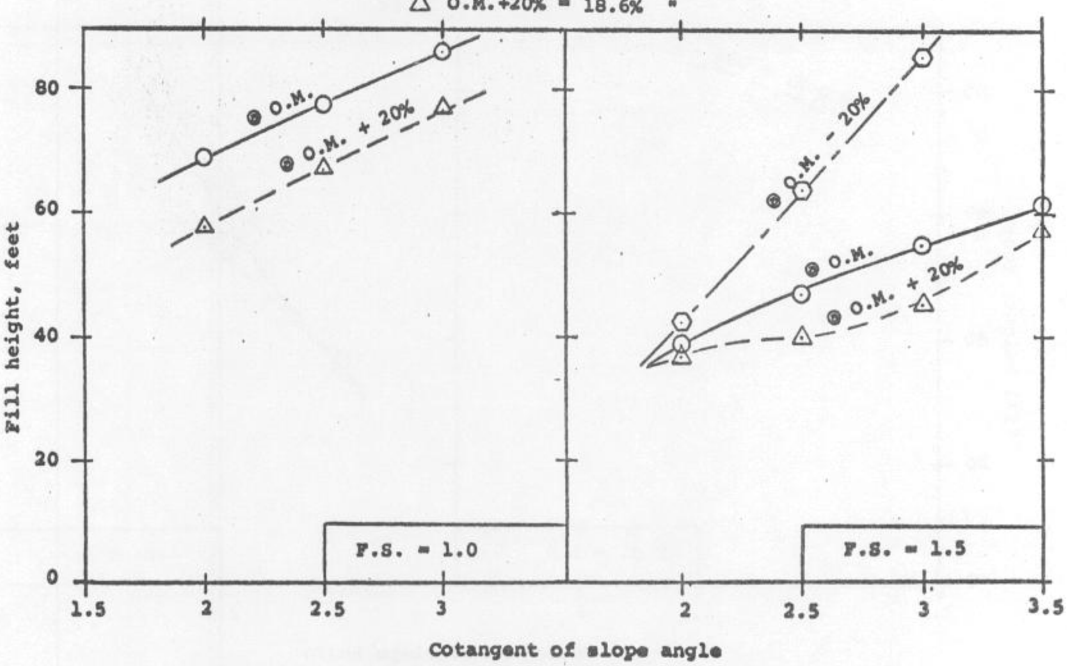


FIG. 23

Soil R-7

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content

⬡ O.M.-20% = 18.9% H_2O
○ O.M. = 23.6% "
△ O.M.+20% = 28.3% "

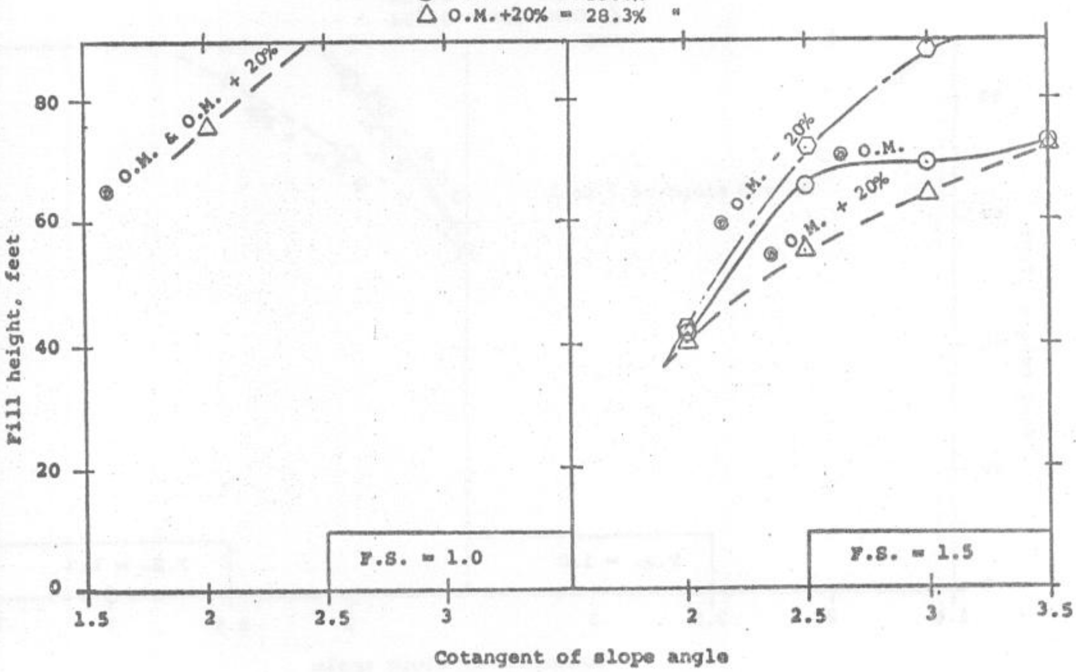


FIG. 24

Soil R-8

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content

⬡ O.M.-20% = 14.4% H_2O
○ O.M. = 18.0% "
△ O.M.+20% = 21.6% "

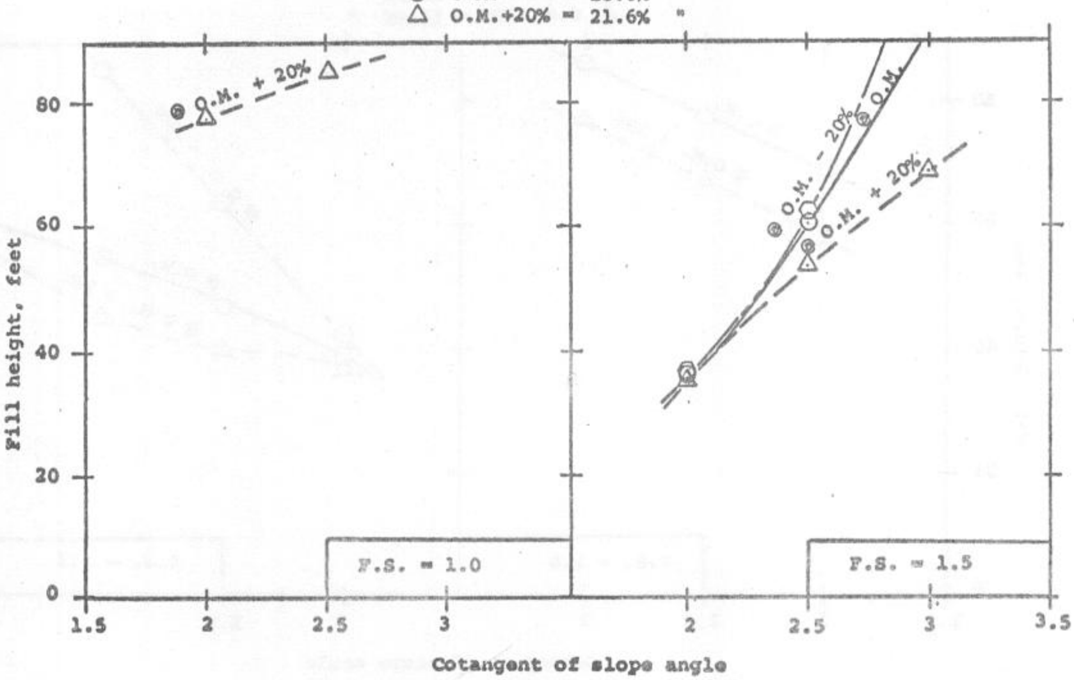


FIG. 25

Soil R-2

Compacted @ reduced effort[1]

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content[2]

⬡ O.M.-20% = 13.8% H_2O
○ O.M. = 17.3% "
△ O.M.+20% = 20.8% "

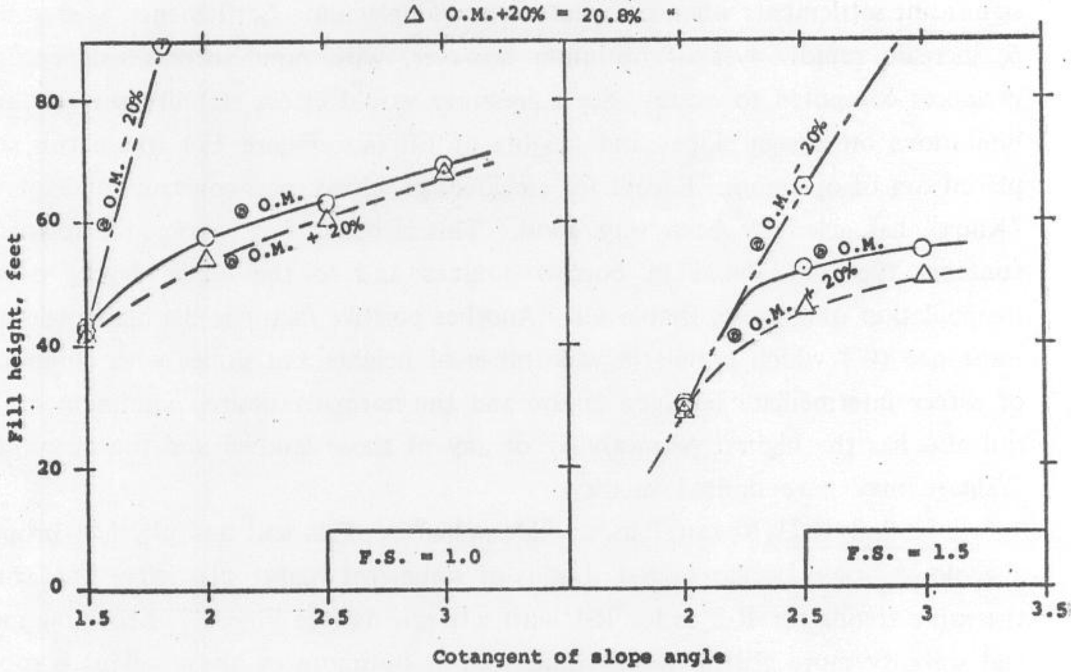


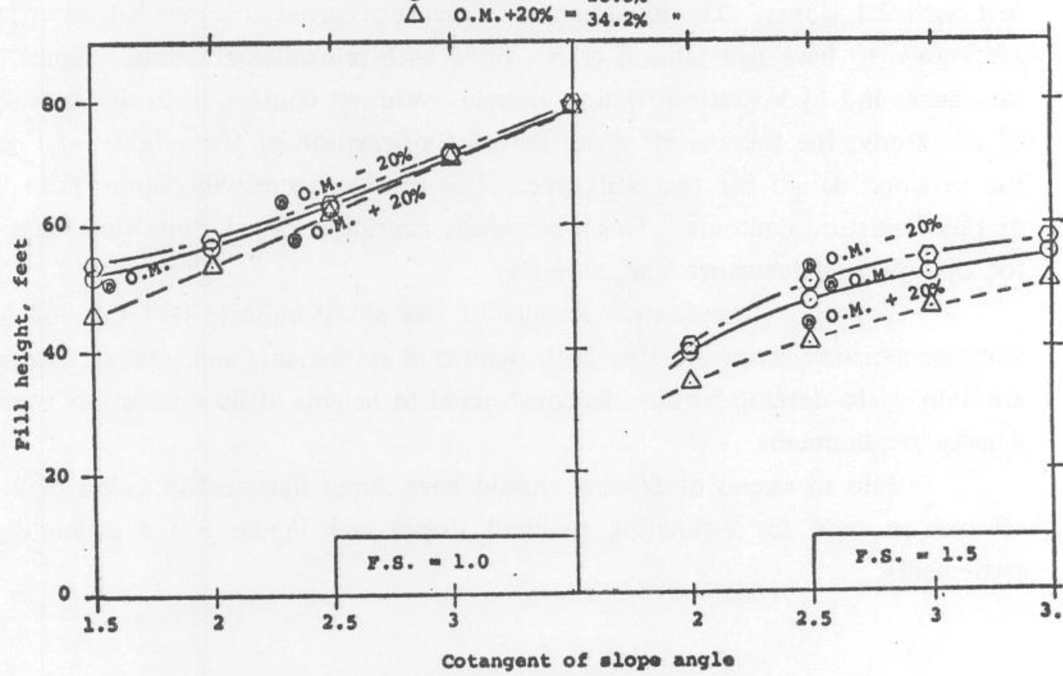
FIG. 26

Soil R-4

Compacted @ reduced effort[1]

Allowable fill height vs. cotangent of slope
for various degrees of placement moisture content[2]

⬡ O.M.-20% = 22.8% H_2O
○ O.M. = 28.5% "
△ O.M.+20% = 34.2% "



1. Compacted at 60% of AASHO T-99 (method C) effort
2. Degree at moisture determined from AASHO T-99 (method C) optimum moisture

IMPLICATIONS OF THE STUDY WITH RESPECT TO SOIL TYPES

Soil R-1 (Peoria? loess, "Knox"). This study shows that this soil does not exhibit significant settlements when compacted dry of optimum. Settlements, as shown in Figure 6, increase rapidly wet of optimum however, with rapid increases of excess internal pressures computed to occur. Such pressures would effect stability severely and suggest limitations on design slopes and heights of fill (see Figure 17) where this soil will be placed wet of optimum. Except for erosional problems, past construction experience with "Knox" has generally been very good. This is believed primarily due to low moisture contents typically found in borrow sources and to the quick drying possible with manipulation of a loose, friable soil. Another positive factor is the high angle of shearing resistance ($\emptyset'$) which permits a wide range of heights and slopes with computed factors of safety intermediate between failure and the normally desired minimum of 1.5. This soil also has the highest permeability of any of those studied and the assumption of no drainage may have limited validity.

Soil R-2 (Roxana? loess, "Memphis"). This soil has physical properties and a geologic origin similar to R-1, but is of somewhat higher plasticity. This study shows the same trends for R-2 as for R-1 with settlements (see Figure 7) becoming more severe and stability more critical when compacted at optimum or above. This is substantiated by field experience. R-2 borrow sources frequently are very wet and the soil is sufficiently plastic that considerable manipulation is required for drying. At least two massive fill slides are known to have occurred in fills of 46 and 48 foot heights, with 2 to 1 slopes, where placement moistures were well above optimum and densities averaged about 95% of maximum. Figure 18 indicates that the factor of safety should reach 1.0 when R-2 is placed at optimum moisture or above (density in excess of 90%) to a height of 52 feet with 2:1 slopes. The documented failures occurred at somewhat lower heights but are known to have had tension cracks filled with rainwater at failure. Since the effects of cracks, and hydrostatic pressures therein, were not considered in the stability analyses of this study, the failures offer substantial confirmation of the validity of Figure 18 for use in slope design for this soil type. Use of this figure will require prior knowledge of field moisture contents. This will permit consideration of slope flattening vs. drying for optimum performance and economy.

Soil R-3 (Crawford). Results of this study indicate that this soil has a wide working moisture range and that both significant settlements and internal excess pressures are unlikely to develop within fills constructed to heights of 80 feet or less within normal density requirements.

Fills in excess of 60 feet should have slopes flattened in excess of 2:1. Figure 19 can be used for estimating required slopes and Figure 8 for estimating possible settlements.

Soil R-4 (Clarksville). Study results indicate a range of moisture contents below optimum moisture at standard compactive effort for most satisfactory performance in terms of both settlement and stability. This is indicated by comparison of Figure 9 and Figure 20. Lower moisture contents are shown to lead to high settlements, possibly a collapse phenomenon, under high loads. This has been confirmed by field experience where fills of Clarksville, compacted dry of optimum and near the lower limit of standard density requirements (90%), have subsequently exhibited deformations in pavements and guardrails after increases in soil moisture.

Wet of optimum, internal excess pressures are computed to develop with resulting decreases in slope stability. These, as indicated in Table 16 and Figure 20, suggest imposing limitations on either placement moisture or on slope design for normally desired factors of safety. Field experience with Clarksville however does not generally confirm study results with respect to stability. This is believed due to the fact that the study soil is atypical of Clarksville as generally used en masse. While pockets of chert-free clay similar to that used in the study are frequently found, these are normally spread in thin lifts. En masse, the typical Clarksville fill has a substantial admixed content of chert gravel which is believed to substantially change behavior. This gravel content possibly makes fills so permeable that significant pore pressures are impossible. There are also obviously beneficial effects on shearing strength from the gravel content.

It is therefore concluded that:

(1) Study results appear fully applicable only to a soil similar to that used in the study, i.e., substantially rock free.

(2) Field experience would suggest that the need for a lower limit on moisture content, indicated by the study, is valid regardless of rock content.

(3) Further investigation of rocky Clarksville soil is indicated to determine strength parameters, permeabilities and pore pressure development. Large diameter triaxial testing is believed the best vehicle for determining these properties.

Soil R-5 (Wabash). This alluvial soil appears to be fairly non-critical with respect to moisture content as it effects mass fill stability within the limits of normal density requirements. Figure 10 indicates settlements possible with various molding moistures. For minimal settlements, moisture controls, particularly an upper control, are indicated. Considering that borrow sources are usually quite wet, there appears to be little practical need for a lower moisture limit.

Field experience with very high fills of this material is limited. However, shallow sloughs and slides are encountered in fills of low height. This suggests a more conservative approach to slope design than indicated by Figure 21. Such distress is believed to be basically a surface phenomenon, not considered in stability analyses of this study, resulting from high volume change characteristics. Wet-dry cycles are believed to ultimately reduce strengths near the surface to near residual values and lead to development of cracks which fill with rain water and produce shallow failures. Experience indicates the need for slopes of at least 3:1 to control such surface failures in this soil, regardless of height of fill.

Soil R-6 (Glacial till). Settlements are shown to be minimal at or below optimum moisture with significant increases possible with increasing moisture content. Internal excess pressures are computed to develop rapidly at optimum and above. These factors would suggest an upper limit on moisture control if fill heights are in such a range that settlement or stability, as indicated by Figure 11 and Figure 22, are likely to be a problem. Natural moisture contents of most borrow sources however, tend to be near optimum moisture. This is believed to be the principal reason why massive failures are unknown except with seepage pressures from external sources.

Shallow failures, essentially surface sloughs, are not uncommon with 2:1 slopes for reasons previously discussed for soil R-5. Experience indicates that slopes of at least 2.5:1 are needed to control such failures in this soil.

Soil R-7 (Sharkey). Study results indicate optimum performance, in terms of both settlement and stability, is obtained with moisture contents dry of optimum. Borrow sources, however, are almost invariably very wet of optimum and drying of this tenacious clay is difficult. This suggests that slopes should generally be designed conservatively, assuming placement wet of optimum and using Figures 12 and 23. Past experience indicates surface failures of the type previously discussed for R-5 and R-6 often occur and that slopes of at least 3:1 are required regardless of fill height or placement moisture.

Soil R-8 (Glacial till). Settlement data indicates that optimum performance is achieved with a moisture range from slightly above optimum to several percentage points below optimum with the permissible spread a function of fill height and tolerable settlement. From a stability standpoint, best performance is achieved with moisture contents below optimum (refer to Figure 24). Like R-6, natural moisture contents of borrow sources are usually near or below optimum. Most distress observed in the field has been the type of shallow surface failure previously discussed for R-5, R-6 and R-7. Slopes of at least 2.5:1 are believe necessary, based on experience, to control this type of failure regardless of fill height or placement moistures.

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