

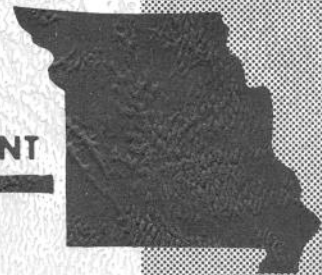
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MISSOURI COOPERATIVE HIGHWAY RESEARCH PROGRAM
FINAL REPORT

79-1

AN INVESTIGATION
OF THE ENGINEERING PROPERTIES
OF THREE PROBLEM EARTH MATERIALS

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT



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THREE PROBLEM EARTH MATERIALS

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Prepared By

MISSOURI HIGHWAY AND TRANSPORTATION DEPARTMENT

Division of Materials and Research

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The opinions, findings and
conclusions expressed in this
publication are not necessarily
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ABSTRACT

The three materials studied are the claystones of the Cheltenham formation, the clay shales of the Maquoketa formation and gley. A performance survey was conducted on roadways where these materials had been encountered. Samples were obtained and tested for determination of engineering properties. Gley was found to be associated with some degree of slope distress or failure in all cases. Problems with the Maquoketa clay shales were found to be less common. The only serious problem linked to the claystones was sometimes serious erosion of slopes due to lack of vegetative cover. Laboratory testing revealed the average drained shear strengths of each of the three materials to be lower by at least seven degrees than ordinary soils of like plasticity. All were found to be relatively impermeable. The claystones were indicated to have fairly low volume change potential with that of the other two materials moderately high. A failure mechanism is postulated for the study materials and a simplified procedure proposed for slope design. Conclusions are offered with respect to average slope requirements and the need for restrictions on use in subgrades.

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INTRODUCTION

Three earth materials have established especially poor reputations for use in roadway construction in Missouri. These are the claystones of the Cheltenham formation in the east-central part of the state, the clay shales of the Maquoketa formation in the northeast part of the state; and gleys, lacustrine deposits found in scattered locations throughout the glaciated, northern half of Missouri. Problems which have come to be associated with the use of these materials include slides in both cuts and fills, difficulty in establishing vegetative covers and differential swells in subgrades with resulting pavement distress. The various remedial treatments proposed have included extensive flattening of cut and fill slopes and undergrading and wasting, whenever possible, of the excavated materials. However, despite such generally poor reputations, the degrees of distress experienced vary and instances can be found of satisfactory performance. This study was proposed as a general background study to gain insights into the engineering characteristics of these problem materials and further to determine if discrimination is possible, what usages are permissible and what restrictions are necessary.

CONCLUSIONS

The performance survey found all known occurrences of gley, where used in highway construction, to be associated with some degree of slope failure or distress. Slope failures in Maquoketa clay shale were less common and more apt to be associated with seepage problems and unfavorable topography. Cheltenham claystone slopes were found to be free of slides and significant sloughing but, except where capped or mixed with soil, to be largely devoid of vegetation and subject to sometimes severe erosion.

The most significant conclusion from the laboratory testing was that all of the study materials have drained shear strengths which are lower on average by at least seven degrees than ordinary soils of like plasticity. Reflecting differences in average plasticity, gley has the lowest average drained shear strength of the three materials and claystone the highest. Other tests show claystone to have fairly low volume change potential while Maquoketa clay shale and gley have moderately high potential. In practical terms, all of the study materials are indicated to be virtually impermeable.

It is postulated that an effective loss of cohesion at the slope surface, as a consequence of volume change phenomena, contributes to the erosion of claystone and to sloughs and shallow slides in gley and Maquoketa clay shale. A simplified method of slope stability analysis, based on the assumption of zero cohesion, is proposed for use with the study materials. Assuming average shear strengths, this approach indicates slopes should be on the order of 4:1 for gley, 3:1 for fill slopes of Maquoketa clay shales and 3:1 for claystones where soil caps are used to support vegetation. Individual investigation and analysis are recommended, however, for any of the study materials when used in large quantities, in high fills or in deep cuts. The study results are considered to have questionable validity with respect to design of cut slopes in Maquoketa clay shale.

Current practice with respect to treatment of subgrade soils is considered adequate for handling of the claystones and Maquoketa clay shales. Gley, however, should be undergraded in cut sections and prohibited from placement in the top of the subgrade.

IMPLEMENTATION

This report will be circulated throughout the Department for use by those who conduct and use soil surveys. It is hoped that the information contained herein will contribute to an understanding of the study materials and the problems associated with their use. Conclusions based on considerations of average properties are believed applicable for use in preliminary planning and for devising treatments for use of minor quantities of the study materials based on correlations to index properties. However, should large quantities of any of these materials be encountered or should their use be anticipated in deep cuts or high fills, individual investigations including shear tests and stability analyses should be performed.

SCOPE

Constructed projects where the three study materials were known to have been used were surveyed for evidence of slides, sloughs, erosional problems and objectionable subgrade heave. Samples were collected for determination of index properties including Atterberg limits, moisture-density relations, grain size analysis and ASTM classification. Selected representative samples were used for performing shear tests and consolidation tests from which permeability values were determined. At least 16 slow, drained direct shear tests were performed on each of the study materials. Shear strengths so determined were correlated to plasticity and these correlations compared to correlations published in the literature. Limited testing was also performed, for comparison purposes, on materials with which two of the study materials are commonly associated. A hypothesis was developed for the mechanism of slope failure in the study soils and a simplified method of estimating slope stability proposed for slope design. Conclusions are reached with respect to probable slope requirements for materials of average properties and the need for restrictions on use in subgrades.

THE STUDY MATERIALS

Gley.

"Gley", as the term is used in this study, refers to Pleistocene clays which are believed to be principally of lacustrine origin from glacial or inter-glacial periods with parentage from associated clay tills. Other origins have been assigned to this or similar deposits as well as other names, including "gumbotil" and "accretion-gley." Those deposits developed on Kansan till and overlain by Illinoian Loveland loess have been given the name Ferrelview formation by Howe and Heim⁽¹⁾ who acknowledge the occurrence of similar but older, unnamed deposits which are both underlain and overlain by till. Some presumably older deposits excavated on highway projects have been revealed to be both overlain and underlain by till, to be lenticular in shape and to have occasional stringers of sand and silt along the perimeters. The principal areas where gley has been found in roadway soil surveys are outlined in Figure 1. However, gley deposits are considered possible anywhere within the limits of glaciation across the north half of the state.

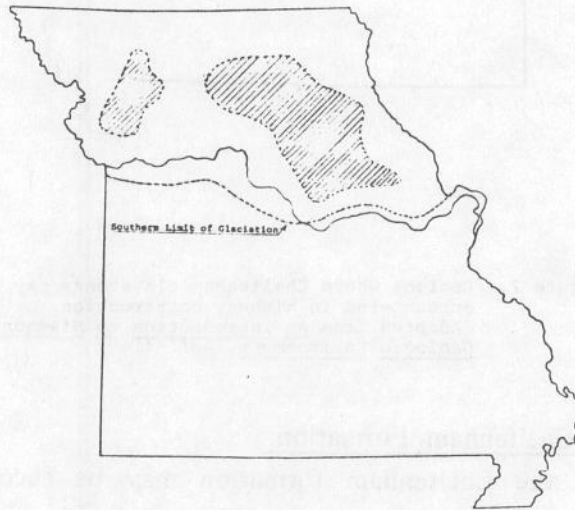


Figure 1. Areas within which gley has been identified in roadway soil surveys.

Gley is a distinctive light gray in color and can be easily distinguished from the dark gray (unoxidized) or brown (oxidized) clay tills. Texturally too it is distinctive. Whereas most of the associated clay tills are at least slightly sandy and have occasional pebbles and boulders, gley normally contains only traces of fine sand. It is very tenacious and exhibits little or no structure. Freshly excavated exposures are stiff to hard but

swell rapidly and become very soft when exposed to moisture. Wet and swollen exposures dry slowly and develop thin crusts over a soft, sticky interior. X-ray diffraction tests, performed as part of an unpublished study jointly conducted with the Division of Geology and Land Survey, indicated the clay fraction of the material herein called gley, as well as most of the associated glacial soils, to be predominantly of the montmorillonite family with lesser amounts of illite.



Figure 2. Regions where Cheltenham claystones may be encountered in highway construction. (Adapted from An Introduction to Missouri's Geologic Environment, Ref. 4)

The Claystones of the Cheltenham Formation.

The claystones of the Cheltenham formation may be encountered in highway construction principally in the east-central part of the state in the areas outlined in Figure 2 although patches may be found as far east as St. Louis County and as far west as Morgan County. The Cheltenham is the basal Pennsylvanian formation in this region and unconformably overlies carbonate rocks principally of the Mississippian and Ordovician periods. The formation also contains shales, which may predominate in northern and western exposures, as well as coarser detrital materials in the form of sandstones, chert conglomerates and chert rubble or residuum which may intergrade with the claystones. Locally, thin coal seams and underclays may be found. The Stratigraphic Succession in Missouri(2) reports that north of the Missouri River, the Cheltenham is ".... more or less like bedded deposits laid down on a solution surface, but toward the south the clays

tend to be confined to filled-sink structures, each of which is of restricted area." Mineral and Water Resources of Missouri(3) reports the blanket-like deposits north of the river average 15 feet in thickness while the filled-sink deposits to the south may extend to depths of as much as 100 feet. Sandstone is sometimes found as "rimrock" outlining claystone filled sinks. Such an occurrence is revealed in an exposure on Route 50 in Gasconade County as shown in Figure 3.



Figure 3. Cheltenham claystones in filled sink deposit on Rte. 50 in Gasconade County. (Note sandstone "rimrock" in foreground).

All of the Cheltenham claystones are locally called "fireclay" but not all deposits have commercial value. Refractory grades, which include "plastic," "flint," and "burley," are relatively free of quartz, illite and other impurities and meet specific firing requirements. Such clays have a high kaolinite content with some of the best grades containing more than 85 percent of this mineral(3). All x-ray diffraction tests performed for the Department for the purpose of establishing mineral values for right-of-way acquisition have indicated the clay minerals to be predominantly of the kaolinite family with lesser amounts of illite.

The Cheltenham claystones are usually hard when freshly excavated and break with

a conchoidal fracture as opposed to the platy fracture of the laminated and fissile shales. Most claystones are white to gray in color but may show variegations ranging through purple, red, maroon and rarely, black.

Clay Shale of the Maquoketa Formation.

The clay shale of the Ordovician age Maquoketa formation, as encountered mainly in the northeastern Missouri counties of Marion, Ralls, Pike and Lincoln, is the third problem material studied. The formation is described by The Stratigraphic Succession in Missouri(2) as exhibiting two distinct lithologies in this area. The lower to basal portion of the section is characteristically dolomitic or "flaggy" and poses few problems in highway construction. The upper or clay shale portion, which is the subject of study, is typically fissile and somewhat silty textured with some thin calcareous and dolomitic shale layers. It weathers rapidly on exposure to a soft clay which is pale blue-green to yellow-green in color. Fresh surfaces may display a range of colors from dull green to brown and dark gray.



Figure 4. Principal outcrop areas of the Maquoketa formation in northeast Missouri. (Adapted from the Geologic Map of Missouri, Ref. 5).

The average thickness of the Maquoketa in the study area in northeastern Missouri is 100 feet where it outcrops, in association with other Ordovician formations, in the region outlined in Figure 4. The section in this region is atypical of the formation as found south of the Missouri River where it thins rapidly and a dolomitic to calcareous facies predominates. In Jefferson County, for example, Department geologists have measured thicknesses ranging from as little as one foot to a maximum of 26 feet. In this area, roadway cuts are relatively stable on slopes which approach the vertical.

THE PERFORMANCE SURVEY AND SAMPLING PROGRAM

Specific locations for surveying and sampling were determined using geologic maps, departmental files of soil survey and slide investigation reports and the memories of departmental geologists and engineers.

Most sampling was done by digging disturbed samples from natural exposures or from highway cut or fill slopes. Only gley and associated clay till were of such consistency that they could be sampled in an undisturbed state using thin-walled tubes. Samples of the Maquoketa and of the Cheltenham claystones were available from deep cores retained from surveys for proposed roadways. In regions where claystones were being mined for refractory use, exposures in pits, spoil piles and stockpiles were also available for sampling in a disturbed condition.

Gley.

Gley was surveyed and sampled at 39 roadway sites in 12 counties through the area shown on Figure 1. At 18 of these sites, slides and sloughing were evident. At 11 other sites, slides had been repaired and at 10 sites the exposed gley showed evidence of erosion and minor sloughing. Undisturbed samples were obtained using a power drill and 3-inch diameter, thin-walled tubes to a maximum depth of 16 feet at one site on Rte. 63 in Randolph County. Soil samples were also taken adjacent to failed areas on Route 63 in Randolph County for comparison purposes. These non-failed areas were found to be, as expected, composed of a slightly sandy clay till with no evidence of gley. In the areas surveyed, almost all slope failures in evidence were judged to be associated with the occurrence of gley. In all instances, gley could be readily distinguished from the associated clay tills on the basis of color and texture. The predominant slope in the failed areas was 2:1 (two horizontal to one vertical), reflecting standard design practice at the time.

Fill slope failures typically resulted in undermining of the guard rail with slumping extending into the shoulder. Repairs were initiated before such failures could affect the pavement. However, some gley-related failures are so extensive as to be beyond the capacity of maintenance forces to repair and either have been or will be repaired by contract. A typical fill slide involving gley is shown in Figure 5. This is from Route 63 in Randolph County,

Cut failures, while unsightly, are less serious and require less urgent attention. However, they must ultimately be treated since the failures eventually block drainage by filling ditches and/or encroach on adjacent private property.

Pavement distress due to the presence of gley in subgrades was most evident in cut sections. The resulting pavement unevenness is not so abrupt as to endanger vehicular

control but, viewed from a distance, clearly reveals differential swells of considerable magnitude. Such distress is less evident in fill sections, probably due to spreading and mixing with other soils.



Figure 5. Progressive slope failure of fill built partially of gley on Rte. 63, Randolph County.

Claystones of the Cheltenham Formation:

Twelve highway cut slopes with claystone exposures were sampled. At all locations the excavated claystone was apparently incorporated into adjacent fills. At five other locations commercial stockpiles of refractory grade claystone were sampled and at three locations old mining pits were sampled. Core samples were also available from a proposed relocation of Route 50 in Gasconade County where a deep claystone deposit in a sink type structure had been investigated. Overall this sampling represented claystone from eight counties covering the area shown on Figure 2.

The performance survey indicated that the fill slopes, almost all 2:1, were generally in good condition where significant quantities of claystone were used. In most of these fills, however, soil and rock were randomly mixed with the claystone. Slopes of dumped strippings from clay pits and of claystone wasted from roadway excavation as unsuitable material were found as steep as 1:5 to 1. These slopes, while badly eroded and lacking vegetation, did not exhibit slides or significant sloughing even after periods of up to thirty years.

Roadway cut slopes in claystone deposits ranged from 1.5:1 to 3:1 and, except where capped with a soil cover, showed a consistent lack of vegetative cover with accompanying weathering and erosional effects. Again, this did not appear to have seriously adverse effects on the overall performance of the slopes. No slides were found and sloughs, where present, were quite small. The beneficial effect of capping such slopes with soil was apparent on Route 63 in Maries County where the 3:1 slopes of a deep cut have a dense vegetative cover and are presently free of any form of distress. Cut slopes in Cheltenham materials, principally shales, were capped in the vicinity of the Routes 740 and WW interchanges with Route 63 in Boone County with generally good but variable success which appears related to the quality and thickness of capping materials used.

During recent construction of roadways relocated as a consequence of the construction of Clarence Cannon Dam and Reservoir, the Corps of Engineers as the funding agency prohibited all Pennsylvanian materials encountered in excavation from placement in any of the massive embankments which could be subject to inundation. As a consequence, a large volume of material was wasted. (Some doubt as to the need for this action was one reason for including the claystones in this study.) Cut slopes in the claystones and shales in the Clarence Cannon Dam Reservoir area are all 3:1. No slides or sloughs are evident but some erosional problems are becoming evident in those areas where growths of vegetation have not been successfully established. Soil caps were not planned but, as a consequence of the contractor's grading operations, some soil was smeared over portions of the slopes. Where this was done, vegetation has been most successful.

Pavement distress was not found which could definitely be attributed to the usage of claystones in the subgrades. One location on Route 100 in St. Louis County where the pavement was badly heaved and broken was initially thought to be underlain with claystone due to an adjacent cut slope exposure. However, subsequent investigation indicated a waxy textured residual clay, not a Cheltenham claystone, to be responsible for the problem.

The performance survey, while confirming that there are problems in establishing vegetation and controlling erosion with claystone slopes, indicated that these problems are relatively minor and that the material may be undeserving of much of its poor reputation.

Clay Shales of the Maquoketa Formation.

The performance survey and associated sampling of the Maquoketa formation was confined to the area where the material was considered to be a problem, the region outlined in Figure 4. As previously noted, the Maquoketa is found elsewhere where its characteristics and behavior are distinctly different.

The upper or clay shale portion of the Maquoketa formation was sampled at 12 locations. Six of these locations were roadway cut slopes, two were creek channel exposures, three were from areas proposed for future construction and one was in a repaired slide area. Samples of the lower portion of the formation were also taken at several locations for comparison purposes.

Three of these sites involved proposed relocations of Route 79 through Pike and Lincoln Counties. Prior investigations in these areas had included cores taken completely through the formation. Samples of these cores taken at close intervals were tested to determine index properties.

Much of the Maquoketa's poor reputation has been earned in the vicinity of the town of Clarksville in Pike County where a sidehill fill for Route 79, built of Maquoketa



Figure 6. Abandoned Rte. 79 at Clarksville. (Roadway was relocated after green-type failures involving Maquoketa shale).

over Maquoketa talus, has been a severe maintenance problem for years. Extensive creep type movements and sliding resulted locally in a buildup of several feet of asphaltic

surfacing material in attempts to maintain the riding surface. The roadway was eventually relocated farther into the bluff where a new problem developed in the form of massive rockfalls as weathering of the newly exposed Maquoketa undermined the overlying formations. The new embankment has since been relatively stable although persistent creeping and sliding of the slopes below the roadway suggest that the instability problem has not been eliminated but only delayed. A portion of the old, distressed roadway is shown in Figure 6.

Extensive and detailed investigations have shown that the problems at Clarksville are due not only to the presence of Maquoketa clay shales but are compounded by the sidehill location and by seepage percolating downward through the overlying carbonates and accumulating on the more impermeable shale. Such seepage leads to swelling and softening of the shale exposures and contributes to saturation of the embankment and the shale talus which forms the embankment foundation.

North of Clarksville most of the slopes involving this clay shale are failing to some degree. A 3:1 cut slope has a large shallow slide. Two long, sidehill cuts where 2:1 cut slopes intercept the shale on its natural slope of approximately 3.5:1 are sloughing throughout their length. A short fill with slopes as steep as 1.5:1, lying between these cuts and which is constructed mainly of the clay shale, has a small slide requiring maintenance of the shoulder. The other slope in this area is a larger cut slope on 2:1 which is also failing due to slides and sloughs.

Those fill slides involving other embankment materials on Maquoketa foundations are considered outside the scope of this study and were not included in the survey. While the fissures and scarps of failed fills permit ready inspection to determine the constituent material, it is more difficult to determine the makeup of fills which have not failed. Consequently, the extent of successful use of the clay shales cannot be gauged. It should be noted in passing however that shallow slides are fairly common in natural slopes which involve various combinations of Maquoketa clay shale, some of which is glacially scoured, tills and loess. Such occurrences, when observed in soil surveys for roadway relocations, have contributed to the caution with which this material is viewed.

It is significant that, in the bluff areas along the Mississippi River south of Clarksville which is a region of high topographic relief, the extensive natural exposures in the upper part of the Maquoketa rarely have slopes steeper than 3.5:1 and frequently are 4.5:1 or flatter.

While there is evidence of swell in Maquoketa clay shale subgrades, this swell does not appear to be so differential in nature as to be a particular problem. This probably can be attributed to the great thickness of the material where it is encountered, resulting in a fairly high degree of uniformity in subgrades in both cuts and fills.

THE TESTING PROGRAM

The testing program for the materials sampled during the performance survey encompassed the following:

1. Plasticity and classification tests on all of the samples.
2. Grain size analysis, shrinkage ratio, and shrinkage limit determinations on representative samples.
3. Moisture and density relationships (AASHTO T-99, Method C) on selected representative samples.
4. Direct shear and consolidation testing of representative samples.

Average results of indices test data, together with the number of tests performed, are shown in Table 1.

TABLE 1
Averages of Routine and Indices Test Data

Material	No.	LL	S.D.	No.	PI	S.D.	No.	No. 40	No. 200	2u	No.	S	R	No.	MD (PCF)	OM %	No.	G
Gley	110	59.4	8.3	106	38.9	6.9	24	97.8	87.2	47.4	10	9.7	2.1	22	98.3	22.0	6	2.66
Clay till	18	40.7	6.5	18	24.4	4.8	18	93.3	70.7	30.4	2	12.6	2.0	15	108.3	16.5	2	2.67
Claystone (Cheltenham)	58	32.7	8.6	33	14.0	5.7	31	94.0	79.9	38.6	18	14.7	1.9	16	113.8	14.1	18	2.64
Maquoketa Clay Shale	31	45.4	4.4	31	21.5	4.2	13	99.1	98.2	49.2	14	17.6	1.8	9	107.0	18.1	5	2.73
Lower Maquoketa	12	30.1	4.9	12	10.1	2.4	5	92.4	86.8	21.1	7	14.6	1.9	5	119.0	13.6	4	2.74

Key to Symbols

LL.....Liquid Limit	MD.....Maximum Density
PI.....Plasticity Index	OM.....Optimum Moisture
S.....Shrinkage Limit	G.....Specific Gravity
R.....Shrinkage Ratio	S.D.....Standard Deviation
	No.....Number of Tests

A limited number of moisture samples were also obtained before such sampling was abandoned as invalid due to the wide variations anticipated as a consequence of the shallow depths being sampled and the extreme seasonal changes during the time of the surveys.

A representative sample of each of the study materials was submitted for fertility testing. Fertility results are shown in Table 2.

The shale and claystone were generally too hard for undisturbed sampling and could only be tested as remolded specimens. Material for preparing remolded specimens for consolidation and shear testing was slaked by wet-dry cycling where necessary, oven dried, crushed and screened through the No. 10 sieve. The soil was then mixed, split and stored

in sealed plastic bags. Soil-water mixtures were prepared at optimum moisture and cured 24 hours prior to molding of test specimens to provide time for moisture equalization through the soil particles. The specimens were then statically molded, generally at 95% of maximum density and at optimum moisture, wrapped in Saran and foil, waxed and stored in a constant humidity room a minimum of 24 hours before testing.

Similar tests were performed on undisturbed specimens of gley. The undisturbed samples were wrapped in both Saran and foil, sealed in wax and stored in the constant humidity room.

TABLE 2

Fertility Analyses of Selected Samples

<u>Material</u>	<u>O.M.</u> <u>%</u>	<u>P₂O₅</u> <u>lb./acre</u>	<u>K</u> <u>lb./acre</u>	<u>Mg</u> <u>lb./acre</u>	<u>Ca</u> <u>lb./acre</u>	<u>NA</u>	<u>pH</u>	<u>M.E.</u>
Gley	0.3	138	60	492	1218	0	7.1	5
Claystone	0.2	19	236	400	1288	0	7.1	5
Clay Shale	0.7	493	80	400	4760	0	7.5	14

Key To Symbols

O.M.	Organic Matter	Ca	Calcium
P ₂ O ₅	Phosphate	NA	Neutralizable Acidity
K	Potassium	pH	Acidity
Mg	Magnesium	M.E.	Total Exchange Capacity

Consolidation Testing.

Consolidation tests were performed for assessment of swelling potential and to provide data for calculation of permeability. The tests were performed in accordance with AASHTO T-216 and were rebounded from each load exceeding 2,000 psf. A series of consolidation tests were also performed without rebounding on 5 specimens of gley obtained from a moisture-density relations test where each specimen was compacted with the same compactive energy but at a different moisture content with resultant variable densities. These specimens were trimmed from the compaction samples in which lifts were defined by foil markers. The series of tests noted as non-inundated were trimmed from the middle of the top lift. The second series, noted as inundated, was similarly trimmed from the middle lift. Where noted as non-inundated, water was not added to the specimen until the applied load exceeded the apparent preconsolidation load of the compacted specimen. Where noted as inundated, water was added at the start of the test.

The results of these series of tests are summarized on Tables 3, 4 and 5. Typical void ratio versus loading curves of the three materials are shown in Figures 7 through 9.

TABLE 3

Average Consolidation Test Data

Material	No. Tests	Load ksf	Cv, ft. ² /day x 10 ⁻²	C _c	Calculated k, ft./day
Gley (undisturbed)	6	2	0.2	0.220	3.8 x 10 ⁻⁶
		4	0.3		1.3 x 10 ⁻⁶
		8	0.2		6.7 x 10 ⁻⁷
		16	0.1		2.2 x 10 ⁻⁷
Gley	4	2	1.1	0.250	2.2 x 10 ⁻⁵
		4	0.2		1.9 x 10 ⁻⁶
		8	0.1		4.5 x 10 ⁻⁷
		16	0.1		2.0 x 10 ⁻⁷
Clay till (Undisturbed)	2	2	0.2	0.200	3.8 x 10 ⁻⁶
		4	0.3		4.0 x 10 ⁻⁶
		8	0.2		1.1 x 10 ⁻⁶
		16	0.2		4.0 x 10 ⁻⁷
Claystone (Cheltenham)	2	2	4.5	0.137	5.5 x 10 ⁻⁵
		4	5.1		3.2 x 10 ⁻⁵
		8	4.3		1.2 x 10 ⁻⁵
		16	5.6		1.3 x 10 ⁻⁵
Clay Shale (Maquoketa)	2	2	1.2	0.265	2.7 x 10 ⁻⁵
		4	1.4		1.7 x 10 ⁻⁵
		8	1.4		7.5 x 10 ⁻⁶
		16	1.8		6.0 x 10 ⁻⁶

Key to Symbols

- Cv.....Coefficient of Consolidation
- Cc.....Compression Index
- kPermeability

TABLE 4

Swell (Volume Change) of Consolidation Test Specimens

Material	Remolded Test Conditions	Initial Swell* Percent	Swelling Index, C _s
Gley	95% MD @ OM	5.2	.046
Gley (Undisturbed)	----	1.0	.090
Clay Till (Undisturbed)	----	1.1	.080
Claystone (Cheltenham)	95% MD @ OM	3.7	.035
Clay Shale (Maquoketa)	95% MD @ OM	7.7	.073

*Restrained by 62.5 psf load.

TABLE 5

Consolidation Test Data for Compacted Gley

W	Percent of MD	S _o (%)	e _o	Pc _c (ksf)	e _c	C _c	C _v ² ft./day (# 8 ksf)	k ft./day (calculated)
Non-Inundated Tests								
16.5	92	64.2	.715	12.0	.689	.426	2.4	5.9 x 10 ⁻⁴
19.4	99	86.3	.624	6.0	.596	.120	1.5	3.9 x 10 ⁻⁵
22.2	97	88.9	.694	5.2	.671	.208	0.01	3.1 x 10 ⁻⁶
25.3	93	88.1	.798	3.2	.770	.256	0.005	8.7 x 10 ⁻⁶
28.0	89	92.4	.810	2.0	.763	.273	0.003	4.5 x 10 ⁻⁶
Inundated at start of test with 1/8 ksf loading								
16.6	92	64.4	.716	3.5	.698	.214	0.05	5.5 x 10 ⁻⁵
19.4	99	80.4	.619	6.8	.592	.198	1.00	4.9 x 10 ⁻⁶
22.1	97	87.7	.701	6.5	.678	.286	0.01	4.7 x 10 ⁻⁶
24.8	93	87.6	.788	4.2	.753	.313	0.005	6.7 x 10 ⁻⁶
28.2	89	88.8	.883	2.1	.861	.301	0.003	6.8 x 10 ⁻⁶

Note: Tests performed on AASHTO T-99, Method C, moisture-density relations test specimens (maximum density 104 pcf, optimum moisture 20.5%).

Key to Symbols

- W.....Moisture Content
- e_o.....Original Void Ratio
- MD....Maximum Density
- C_c....Compression Index
- k.....Permeability
- P_c.....Apparent Preconsolidation Load
- e_c.....Void Ratio at Apparent Preconsolidation Load
- S_o.....Original Percent Saturation
- C_v.....Coefficient of Consolidation

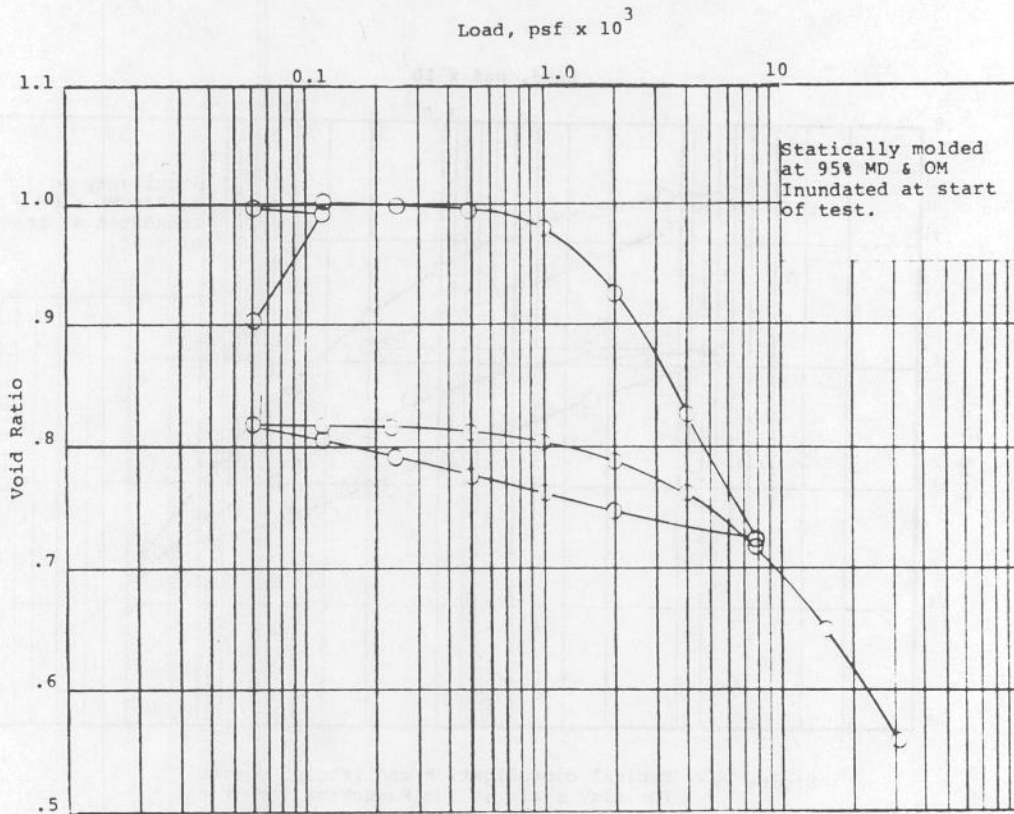


Figure 7. Typical consolidation and rebound curves for gley.

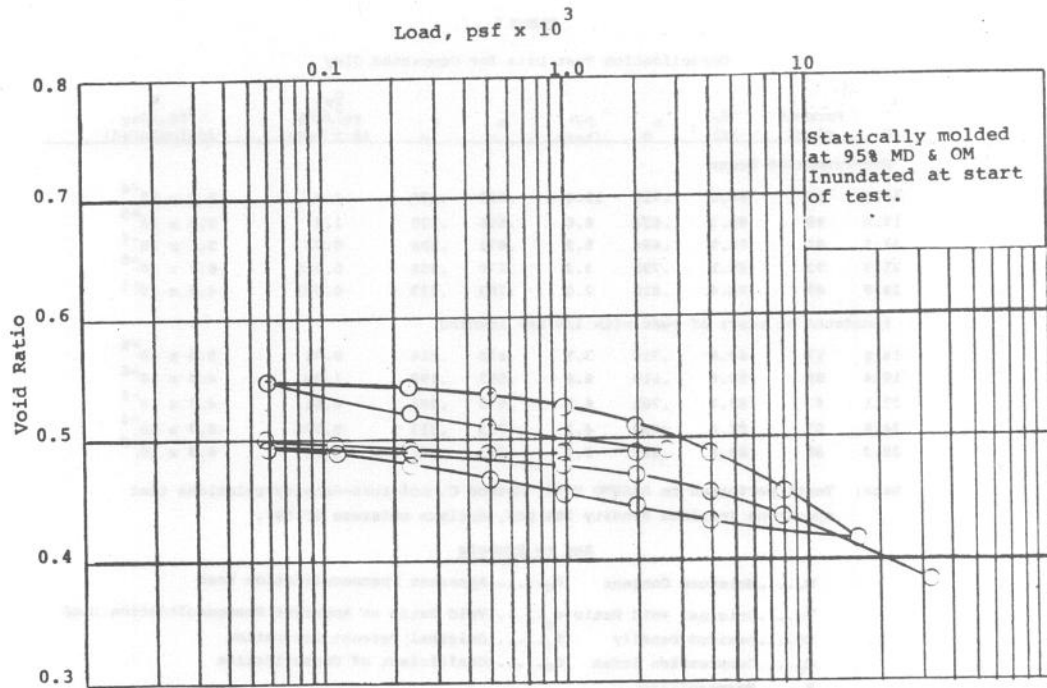


Figure 8. Typical consolidation and rebound curves for Cheltenham claystone.

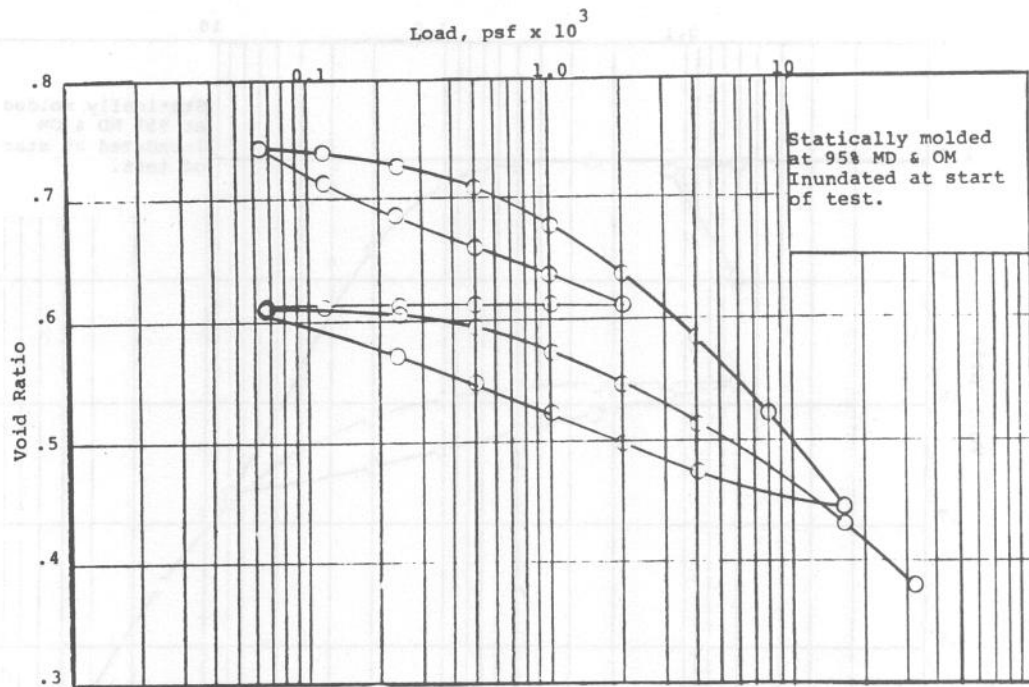


Figure 9. Typical consolidation and rebound curves for clay shale of the Maquoketa formation.

Shear Testing.

Most drained, direct shear test specimens were formed by static molding to 95 percent of maximum density at optimum moisture content. Some tests were also performed on consolidation test specimens after completion of the consolidation tests. In any case, specimens were soaked a minimum of 7 days before shearing to insure saturation. The samples were then sheared at the slowest possible rate, approximately 2×10^{-4} inches/min., in accordance with AASHTO T-236. Shear test results are summarized in Table 6 as effective angles of internal friction, ϕ' , and cohesion, c' , as well as ϕ' where c' is zero, i.e., the apparent cohesion through the range of preconsolidation being disregarded.

Permeability.

The permeability values were calculated from consolidation test data according to procedures outlined by Lambe(6). The initial step requires determining the coefficient of compressibility, (A_v), as follows:

$$A_v = \frac{0.435 C_c}{P}$$

where C_c is the compression index and P is the average load for the pressure increment or $\frac{(P_1+P_2)}{2}$. The permeability, k , then can be calculated as:

2

$$k = \frac{C_v A_v \gamma_w}{1 + e}$$

where C_v is the coefficient of consolidation, γ_w is the unit weight of water and e is the void ratio corresponding to the loading increment used.

As permeability normally varies for each loading increment of the consolidation test, permeabilities are shown in Table 3 for loads ranging from 2 to 16 ksf. For tests on compacted gley, as shown in Table 5, permeability values are calculated only for the apparent preconsolidation load value.

File Search.

Departmental files were also searched for test data previously developed on the problem materials. Most available data was in the form of classification tests, Atterberg limits, moisture-density relations, etc. Only a few shear tests were available. In all cases, raw work files were carefully reviewed to verify unequivocal identification of the material as well as compliance with the conventions used in this study.

ANALYSIS OF DATA

Classification and Index Tests.

To aid in comparison and analysis, classification and indices test data were averaged for the various materials, standard deviations of data for significant properties determined, the data summarized (Table 1) and displayed graphically in various figures. Figure 10 is an A-line plot showing the range of the classification data while Figures 11 through 13 show the grain size distributions of the study materials. Figure 14 shows graphically the data dispersion for liquid limits. Figures 15 and 16 display correlations of clay sized particles to liquid limit and plasticity index. Also included in the latter figures, for comparison purposes, are some data from the lower or "flaggy" portion of the Maquoketa

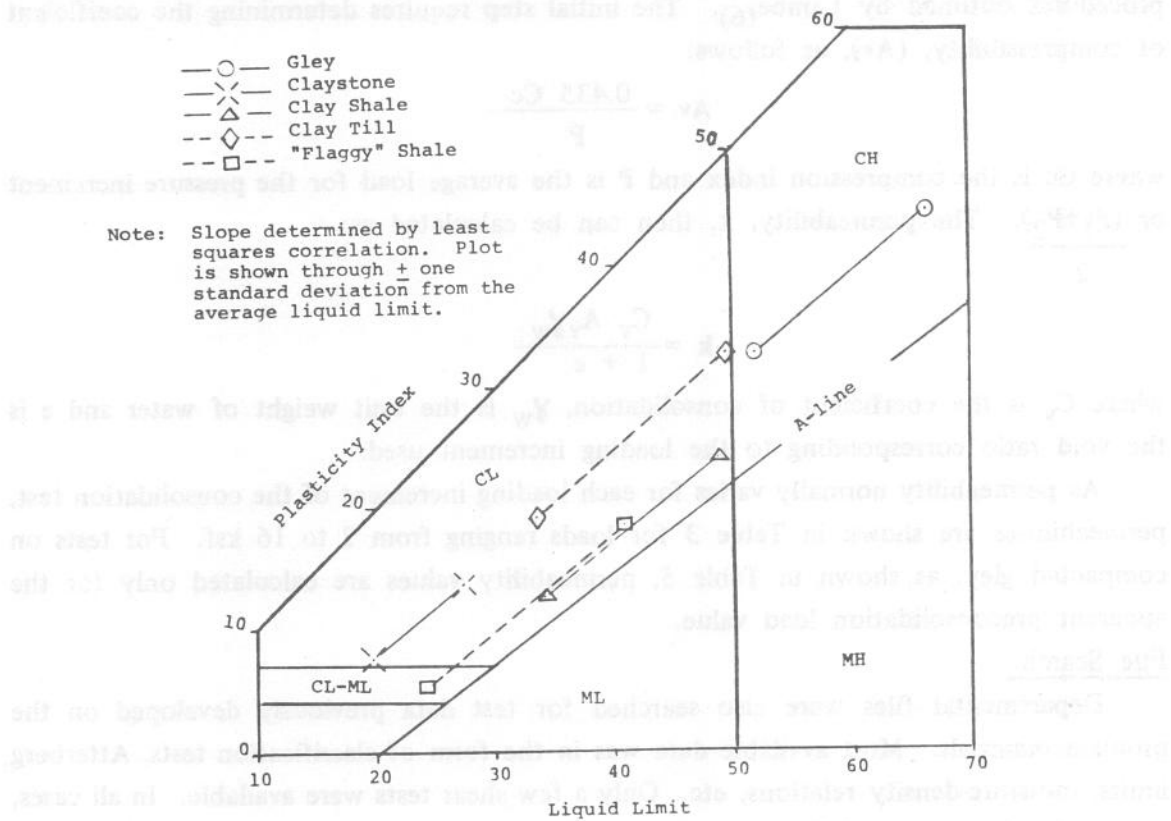


Figure 10. A-line plot of problem materials.

formation and from the clay tills commonly found in association with the gleys.

The claystones and the Maquoketa clay shales are shown to be predominantly CL by ASTM classification and the gleys predominantly CH. The average liquid limits are 59, 46 and 33 for gley, Maquoketa clay shale and claystone respectively while plasticity indices average 39, 22 and 14. By such plasticity criteria, gley is clearly the worst of

the three materials, the claystones the best and the Maquoketa clay shale intermediate.

Test data from the lower or "flaggy" portion of the Maquoketa reveal an average ASTM classification of CL, an average liquid limit of 32 and an average plasticity index of 12. When it is considered that this represents only that portion of the lower Maquoketa which can be broken down for testing, i.e., other than the stony flags, whereas almost all of the upper part can be degraded, it is readily apparent why there is a marked difference in the behavior of the upper and lower portions of the formation.

The clay tills sampled in conjunction with the gleys show an average liquid limit of 41 and an average plasticity index of 24. Both values are significantly lower than those determined for gley.

The average grain size curves for the study materials, as shown in Figures 11, 12 and 13, do not exhibit large difference in percentages of clay-sized particles, i.e., smaller than 2 microns. However, the curve shape and slopes are quite different. This suggests, in view of the significant differences in plasticity characteristics, that the degree of uniformity of size grading and the activity and mineralogy of the clay fraction are more likely to account for behavioral differences rather than the relative percentages of clay.

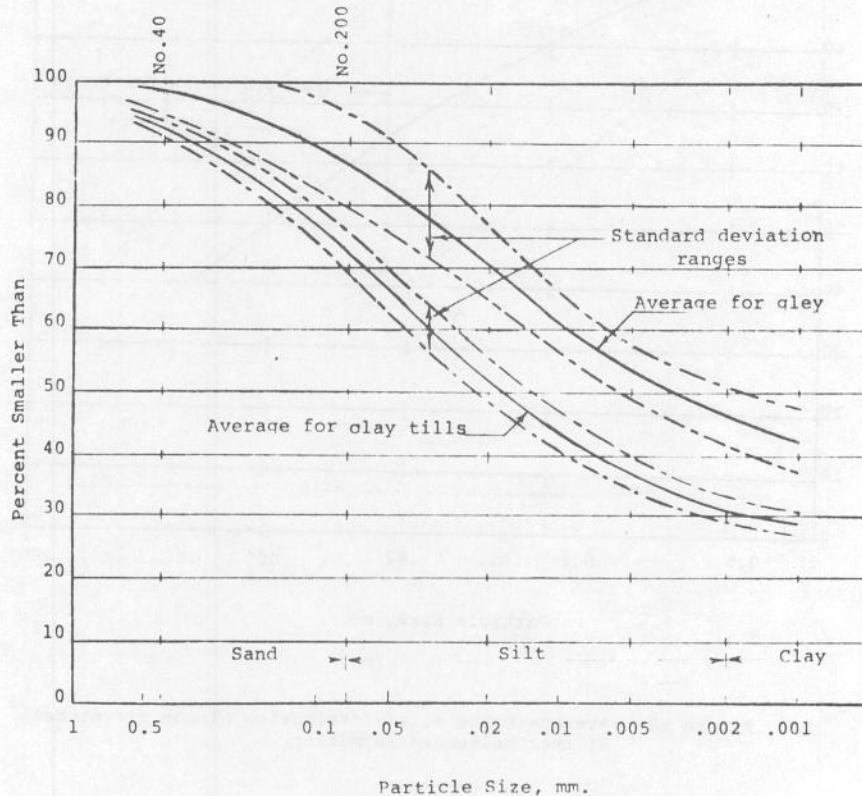


Figure 11. Average grain size distribution of gleys and associated clay tills.

When analyzed in terms of activity index, defined as the quotient of PI divided by the percent of clay, gley is shown to be the most active, with an average index of 0.83, followed by Maquoketa clay shale at 0.45 and then by claystones at 0.36. This is believed to reflect mineralogical differences in the clay fractions of the three materials. Available x-ray diffraction test data show the clay fraction in gley to be composed predominantly of the montmorillonite family, the most active of the clay minerals, along with lesser amounts of illite. The claystones are shown to be composed predominantly of clays of the kaolinite family, among the least active, along with some lesser amounts of illite. No such data are available for the Maquoketa but the intermediate degree of activity calculated for this material would suggest that illite may be the predominant clay mineral.

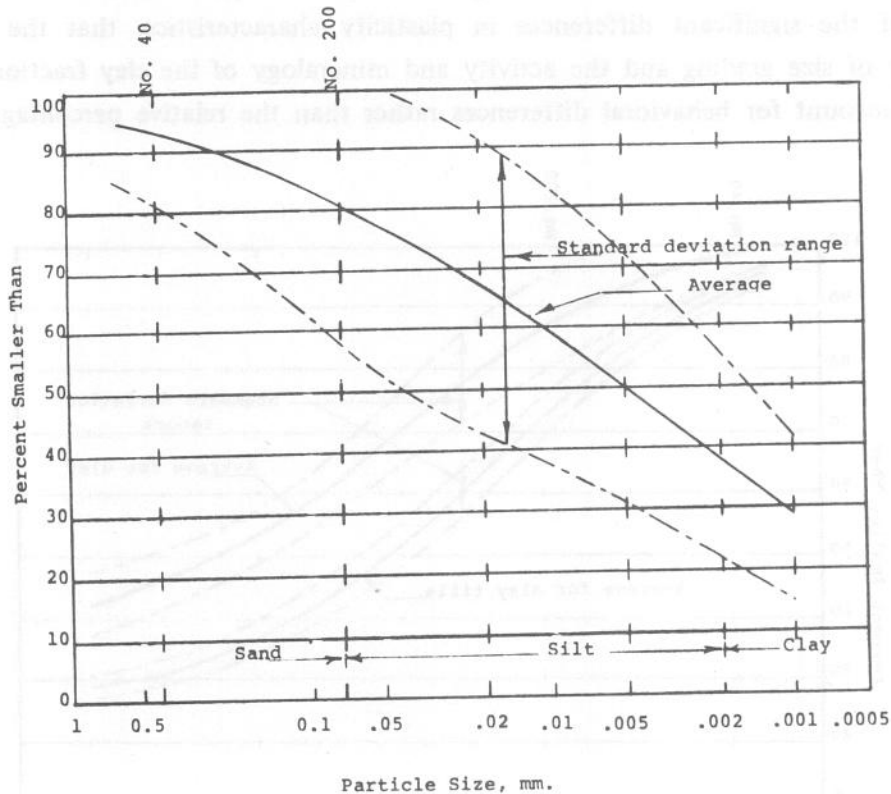


Figure 12. Average grain size distribution of the claystones of the Cheltenham formation.

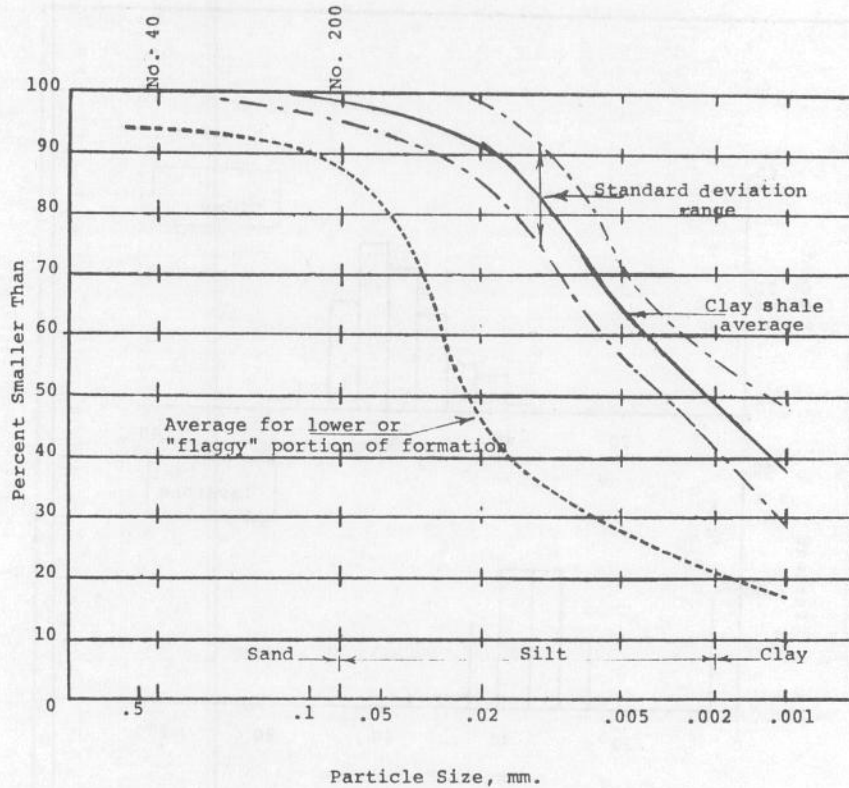


Figure 13. Average grain size distribution of the clay shales of the Maquoketa formation.

Significant insights into behavioral differences of two of the study materials are offered by comparison of grain size distribution curves of materials with which they are commonly found in association. Gleys are shown to have less sand and silt and a correspondingly higher percentage of clay sized material than do the associated clay tills, 47 vs. 30 percents respectively, as shown on Figure 11. These two percentages have approximately the same ratio as do the ratios of the average liquid limits and plasticity indices of the two materials, indicating gley and till to be closely related. It is consistent with the derivation postulated for gley that it should be enriched in clay with correspondingly less sand and silt than the parent till from which it is believed to be derived. This clay enrichment is believed to account for the differences in shear strength and expansion characteristics between gley and clay till.

The curve included in Figure 13 for the lower or "flaggy" portion of the Maquoketa shows less than half as much clay content as for the clay shale as well as a curve indicative of a higher degree of gap grading with greater potential for particle to particle contact

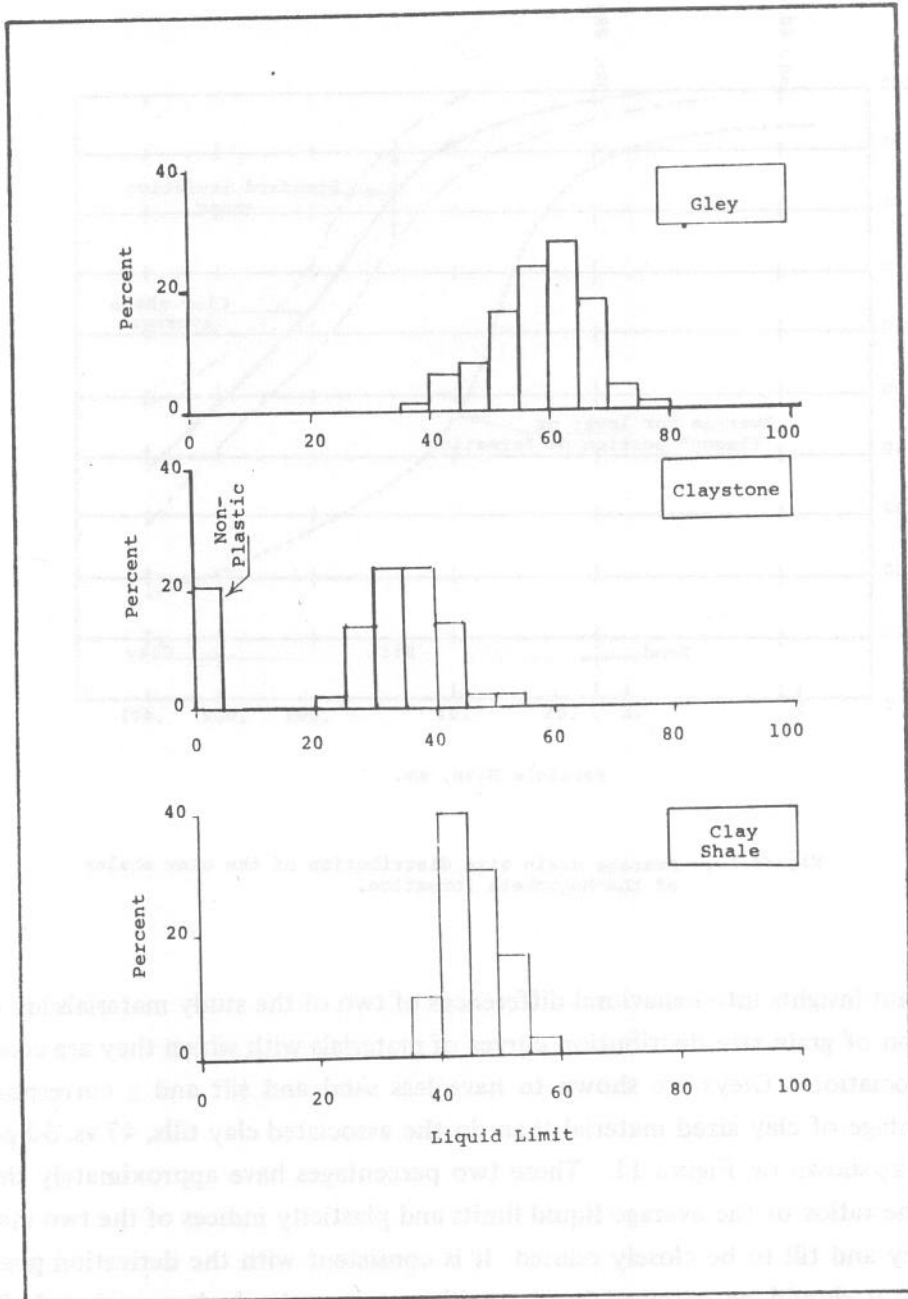


Figure 14. Liquid limit distributions of samples of the problem materials.

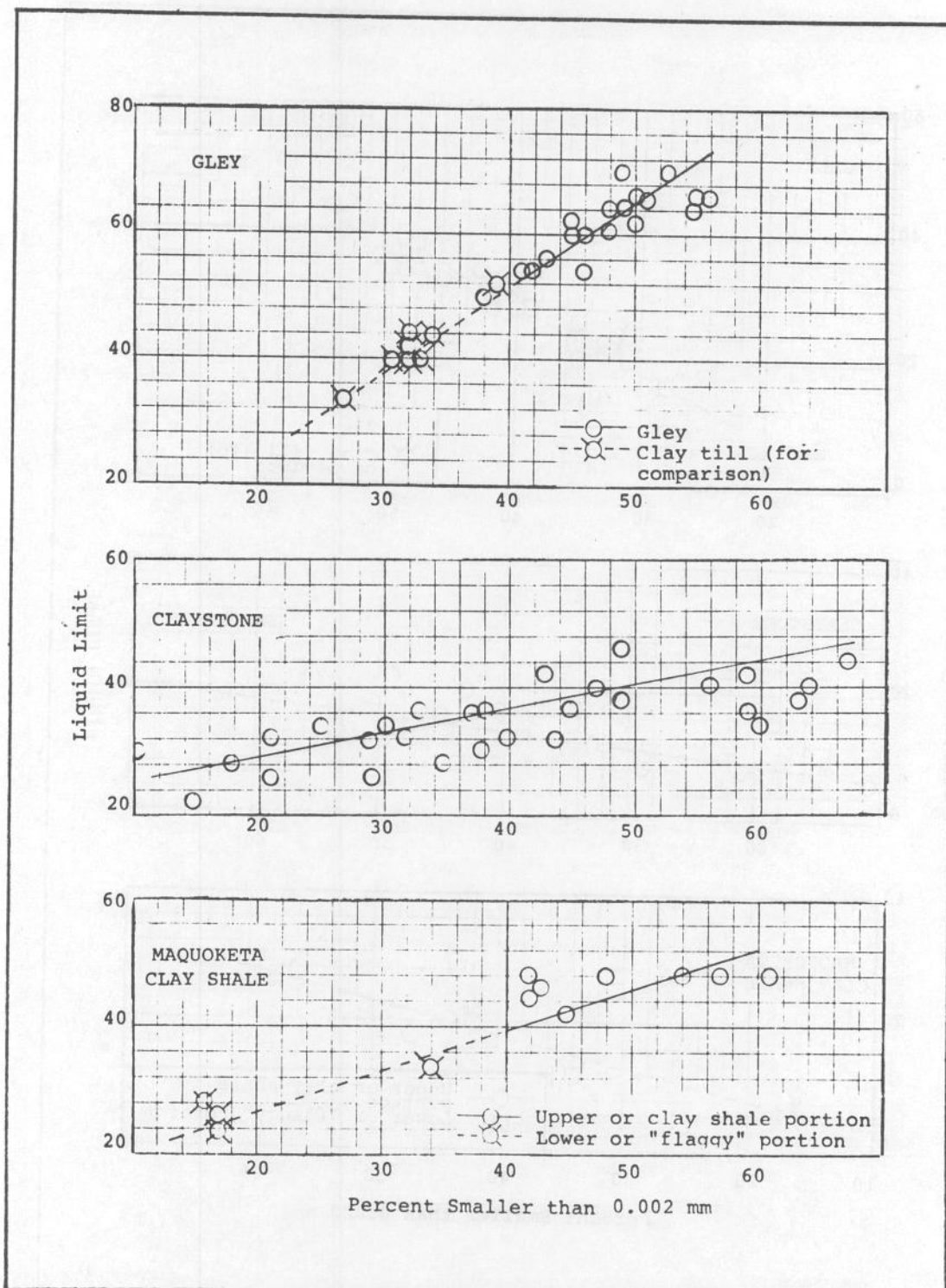


Figure 15. Percent smaller than 0.002 mm vs. liquid limit for the problem materials.

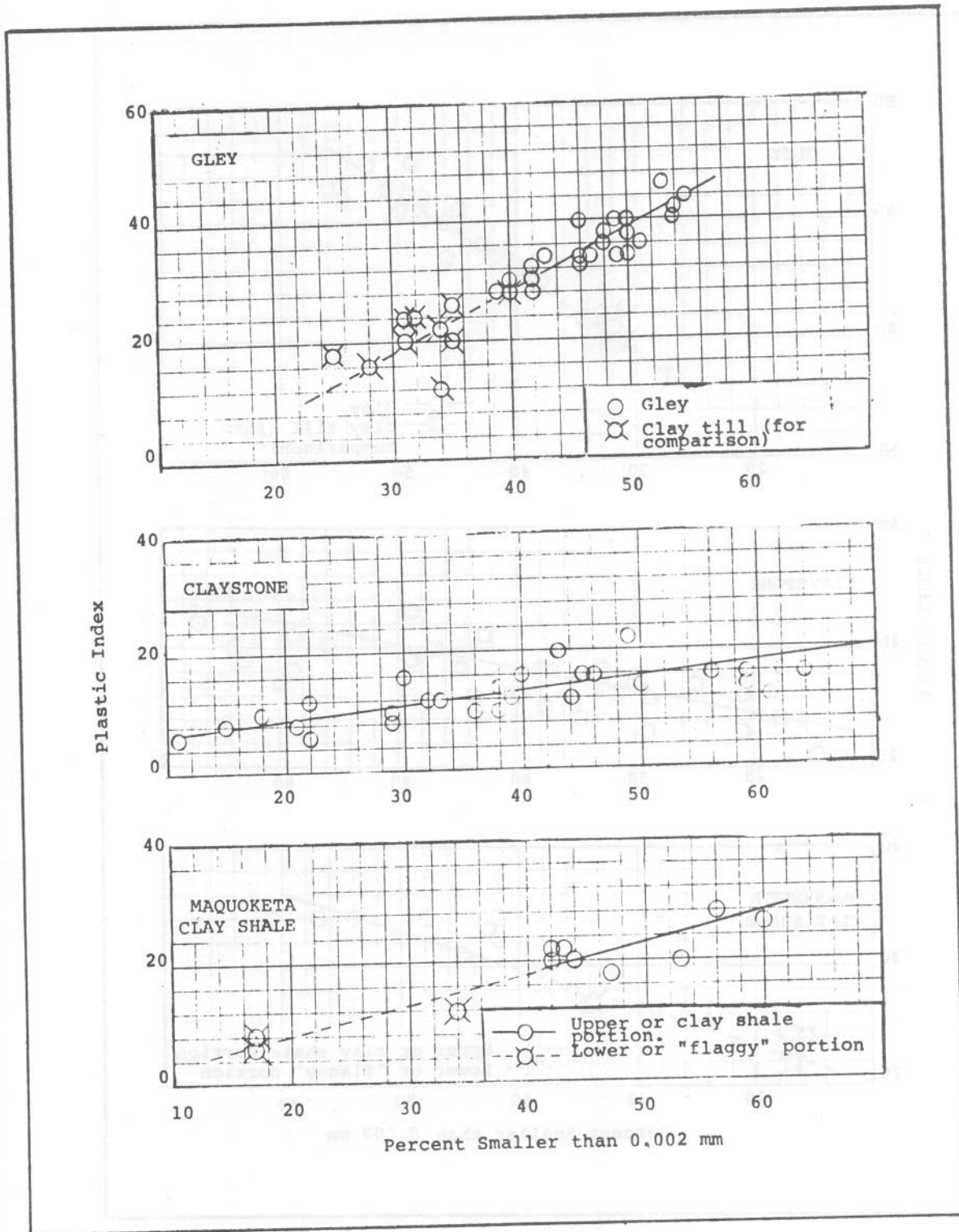


Figure 16. Percent smaller than 0.002 mm vs. plasticity index for the problem materials.

of the coarser fraction and for higher permeability. (Again, almost all of the clay shales of the upper portion of the formation are degradable for testing while only about half of the lower part of the formation can be degraded).

Atterberg limits provide a basis for computing potential for volumetric change, VC, using the formula:

$$VC = (w_1 - S)R$$

where w_1 is a given water content, S is the shrinkage limit and R is the shrinkage ratio. Using the average test values shown in Table 1 and substituting the liquid limit for w as maximum possible water content, gley of average properties is computed to have a maximum potential volume change of 104 percent, clay shale 52 percent and claystone 34 percent. A measure of potential shrinkage from the compacted state may be similarly computed by substituting the moisture of compaction for w_1 . Using optimum moistures, gley's potential shrinkage averages 26 percent, Maquoketa clay shale only about a half percent and claystone none.

TABLE 6

Average Slow Direct Shear Test Data

	No. Tests	ϕ' Degrees		c' psf		ϕ'^0 ($c'=0$)	
		Avg.	S.D.	Avg.	S.D.	Avg.	S.D.
Gley	20	13.8	3.2	522	162	19.7	2.8
Gley (remolded)	5	16.6	2.0	390	137	20.6	0.7
Clay Till	5	22.9	2.5	232	148	23.9	1.8
Claystone	22	24.1	4.2	225	102	25.6	4.5
Maquoketa Clay Shale	16	19.1	2.9	249	179	22.4	1.6
Lower Maquoketa	7	26.9	6.4	258	203	29.6	4.6

S.D. = Standard Deviation

ϕ' = Angle of inclination of Mohr's envelope for shear strength based on effective stresses.

c' = Cohesion intercept on Y axis for Mohr's envelope for shear strength based on effective stresses.

Shear Tests.

Analysis of the average slow, direct shear data of Table 6 confirms the trends shown by the plasticity data. The average effective angle of internal friction, ϕ' , where cohesion or c' is zero, is lowest for gley, averaging 19.7 in the undisturbed state and 20.6 degrees remolded. Claystone is highest at 25.6 degrees and Maquoketa clay shale is intermediate at 22.4 degrees. (The comparison samples of clay till average 23.9 degrees, significantly higher than the gley, and those of the lower Maquoketa, averaging 29.6 degrees, are distinctly higher than the clay shale portion). The same relative trends in ϕ' are also in evidence through the range of preconsolidation where lower values of ϕ' are reported in combination with c' . However, an inverse relationship holds true for c' , gley being the highest and claystone slightly lower than Maquoketa.

Figure 17 is perhaps the most significant of this report in terms of the insights offered into the engineering behavior of the three problem materials. This figure shows the correlations established for the three materials between plasticity index and ϕ' (where c' is zero). Also superposed is the well-known correlation by Bjerrum and Simons⁽⁷⁾. Unpublished correlations prepared from all available test data for Missouri soils have compared well to the Bjerrum and Simons correlation. However, the three study materials, excepting those claystones of lowest plasticity, are at significant variance from this correlation. All three materials are shown to have angles of internal friction which average at least 7 degrees less than the Bjerrum and Simons correlation. The claystones however have a significantly lower angle only when the plasticity index exceeds about 10.

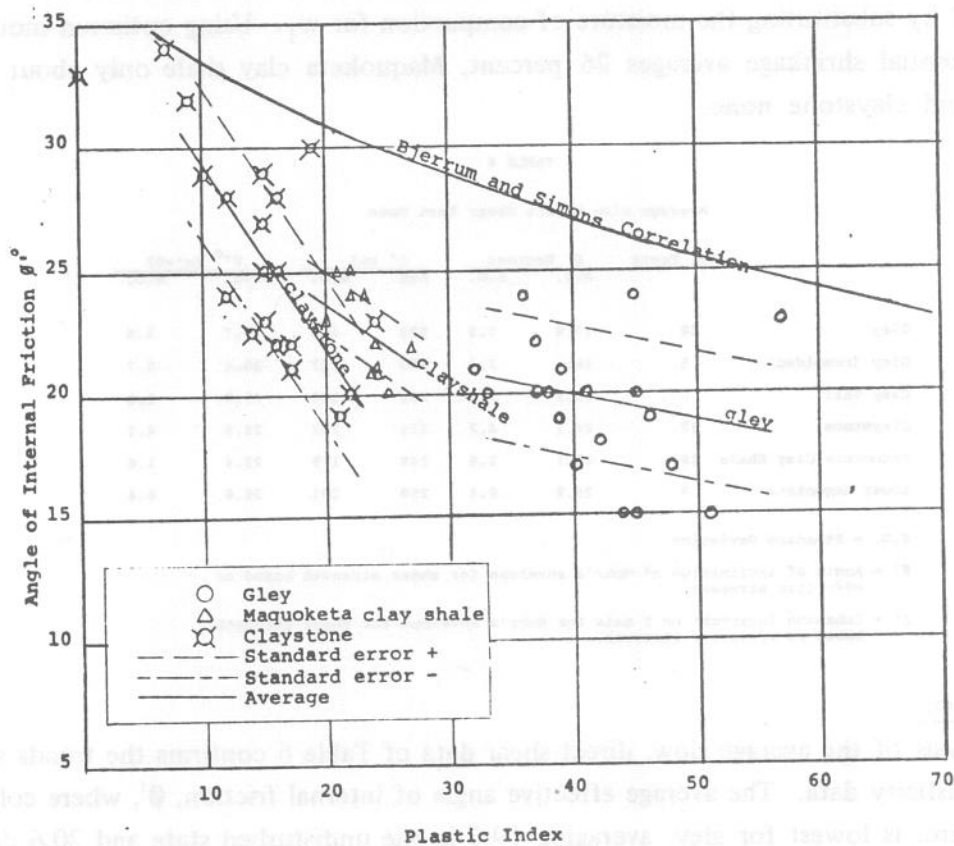


Figure 17. Correlation of plastic index to the angle of internal friction.

Consolidation and Permeability Tests.

Consolidation tests were performed primarily to provide a basis for calculating permeability values and secondarily to assess swell potential through rebounding. Since

internal settlement of fills has not been shown to be a problem, there was little reason to assess compressibility although appropriate test properties are included in the tables if this should be of interest. Consolidation, permeability and swell data are reported on Tables 3, 4 and 5 while Figures 7, 8 and 9 are typical consolidation and rebound curves developed from statically remolded specimens of each of the study materials.

Both the compression index, C_c , and the rebound index, C_s , are measures of the change in void ratio per log cycle of change in loading. Compression indices are quite low for claystone and moderate for both Maquoketa and gley. Considerable variation in the values for gley were determined however for the various types of samples tested which included undisturbed, statically remolded and dynamically compacted. Swell indices of statically remolded specimens were lowest for claystone, intermediate for gley and highest for Maquoketa. An undisturbed gley specimen however had an even higher swell index than the Maquoketa. Considering the variations revealed in compression indices for various sample types, it is probable that similar variations would occur in swell indices if different types of samples were tested. The swell and rebound data developed from the consolidation tests are therefore considered somewhat inconclusive. Additional testing of undisturbed samples and of dynamically compacted specimens molded to duplicate a range of density and moisture conditions would be desirable to further quantify the swell potential of these materials.

Permeability values determined for all three materials varied with the degree of load restraint. In the case of gley, where both undisturbed and statically compacted samples were tested, the determined values of permeability not surprisingly also varied somewhat by type of sample and compacted density. For statically remolded specimens where direct comparisons can be made, gley permeabilities were on the order of 10^{-5} to 10^{-7} feet per day depending on load restraint, the clay shale 10^{-5} to 10^{-6} feet per day and the claystone all in the 10^{-5} feet per day range. In practical terms, all are virtually impervious.

DESIGN IMPLICATIONS

The study materials have disparate properties as measured by plasticity, activity index, swelling potential and shearing resistance. They are alike however in that their angles of internal friction are consistently low as compared to other soils of like plasticities. The performance survey indicates that the problems experienced differ greatly in degree, ranging from erosion of bare slopes to sloughs to progressively deeper slides. All these differing forms of distress appear related however in that they are associated with at least some degree of long-term surface weathering involving desiccation cracking during dry periods and/or swelling and softening when moisture is available with resultant loss of cohesion at the slope surface. Cracking and/or softening appear to extend in significant degree to depths of as much as several feet in the case of gley or Maquoketa clay shale. While the claystones seem similarly afflicted to only a minor degree, the paucity of vegetation on claystone slopes contributes to ease of erosion of this surface layer of cohesionless material.

The assumption of loss of cohesion at the surface of a slope affords a convenient and simple means of analyzing long-term slope stability. In the absence of cohesion, the factor of safety can be expressed simply as a function of the ratio of the tangents of the angles of internal friction, ϕ' , and the slope angle. Seepage effects can be approximated by multiplying the tangent of ϕ' by a factor r_u , the ratio of the effective weight of the material, after deducting the uplift component of any selected degree of seepage pressure, to the total weight of the material. Factor of safety curves can be developed, as in Figure 18, for tangents of ϕ' vs. the cotangents of the slope angle. Such curves are applicable for analyzing any height of slope using ϕ' where cohesion is zero.

Judgement is required however in selecting an appropriate value of r_u between the extreme conditions of no seepage ($r_u = 1.0$) and maximum seepage stress ($r_u \approx 0.5$). It is postulated that seepage effects have least effect on those soils which exhibit the lowest degree of volume change, hence least swelling, least loss of cohesion, shallowest depth of desiccation cracking, etc; hence, least capacity to absorb water during rain, etc. Contrarily, those soils with the highest degree of volume change potential should have greatest potential for development of seepage stresses. On this basis it could be argued that for a given factor of safety the claystones require the lowest, even no, reduction in $\tan \phi'$ for seepage, gley requires the largest reduction and Maquoketa clay shale some intermediate degree of reduction. Similar judgement is also required in selection of an appropriate minimum factor of safety. Because the loss of all cohesion is admittedly an extreme and simplifying assumption, factors of safety approaching 1.0 may be appropriate under circumstances where the consequences of failure are least serious, as

in cuts or fills of minimal height, and/or where there are few uncertainties about the occurrence, the properties or the probable use of the material under consideration. Higher minimum factors of safety, perhaps approaching 1.5, would thus appear justifiable under circumstances of greatest risk and uncertainty.

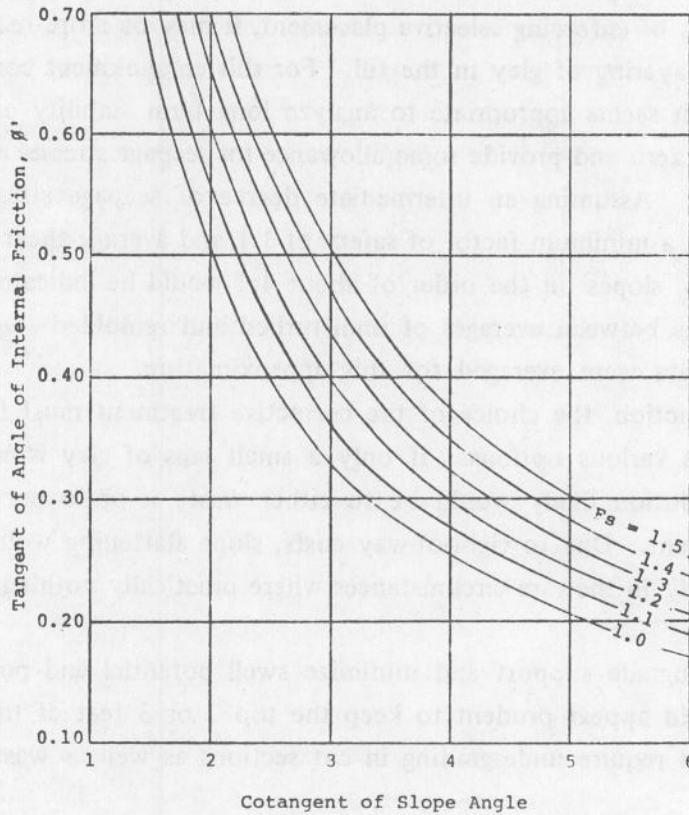


Figure 18. Factor of safety, F_s , curves based on cotangents of slope angles vs. tangents of the angles of internal friction, ϕ' , for the general case where cohesion is zero.

Gley.

Gley, by field experience and most laboratory criteria, is the worst of the study materials. Observations of the numerous failures in the field suggest that surficial exposures of gley, in the absence of load restraint and with seasonal availability of moisture, swell to the point where there is almost total loss of cohesion at the surface with this loss extending in lessening degree with increasing depth. Further, the material is so impermeable that it tends to support perched water tables in both cuts and fills. This introduces seepage stresses and provides an added source of water to satisfy the high swell potential.

These considerations suggest that in new construction gley should be wasted or, if used, either buried deep in the core of an embankment or the slopes should be flattened severely. For cases where gley is afforded adequate load restraint, it would appear logical to analyze slope stability using both effective stress parameters ϕ' and c' with appropriate assumptions as to the degree of seepage stresses possible with a given slope template. Given the difficulties of enforcing selective placement, it may be more realistic to assume full width, random layering of gley in the fill. For this embankment condition, and for cut slopes in gley, it seems appropriate to analyze long term stability only in terms of ϕ' where cohesion is zero and provide some allowance for seepage stresses using the criteria previously discussed. Assuming an intermediate degree of seepage stress equivalent to an r_u value of 0.75, a minimum factor of safety of 1.1 and average shear test parameters as shown in Table 6, slopes on the order of about 4:1 would be indicated for long term stability. Differences between averages of undisturbed and remolded shear strengths are not great and all data were averaged for this approximation.

On new construction, the choice of the corrective treatment must be based on the relative costs of the various options. If only a small lens of gley is encountered, the most economical solution likely would be to either waste it or to bury it in the core of a large embankment. Due to right-of-way costs, slope flattening would likely be the option of choice only in the rare circumstances where practically nothing else is available for use but gley.

To improve subgrade support and minimize swell potential and possible pavement irregularities, it would appear prudent to keep the top 2 or 3 feet of the subgrade free of gley. This would require undergrading in cut sections as well as wasting or selective placement in fills.

For repair of slides on existing roadways, the limitations of the existing right-of-way will rarely permit flattening of gley slopes to the required degree. Excavation of the disturbed gley even over-excavation to a temporarily steep slope, followed by replacement with selected material to provide load restraint, is ordinarily the most economical and rapid repair. Depending on the presence of perched water tables and seepage, chimney and blanket drains may also be required. Needless to say, each slide correction requires individual analysis with solutions tailored to individual circumstances.

Claystone.

Field observations show that the Cheltenham claystones are not particularly prone to sloughs or slides on even relatively steep slopes. The main problem appears to be that the material is so lacking in available moisture, as well as being relatively infertile, that it is extremely difficult to establish any form of vegetation to control erosion of a thin, cohesionless, weathered layer which develops at the surface. The performance survey indicated that this erosional problem is most severe in cut slopes and is frequently

absent in fills where some degree of intermixing is provided with other soils or with rock. It is probably more unsightly than serious, with cleaning of ditches the most persistent maintenance problem.

While undisturbed samples of claystone could not be tested, data from remolded tests are believed applicable to both cuts and fills. Supporting this viewpoint are the generally similar performances observed for steep cut and fill slopes in the field, the absence of bedding or laminations such as shale exhibits and the random particle orientations likely in any material comprising sink fill.

It appears that slopes of claystone may be designed for relatively long term stability against slides or large sloughs on the assumption of relative permanence of both ϕ' and c' , permitting slopes as steep as 2:1 in many cases. However, where it is deemed desirable to control erosion, as would normally be the case, caps of more pervious and fertile soils appear necessary to support vegetation. Because seasonal development of seepage stress is possible in a more pervious layer superposed on a less pervious layer, some degree of slope flattening is indicated to prevent sloughing of the cap during wet periods. Using Figure 18 and the same criteria for cap stability as for gley, i.e., average ϕ' with zero cohesion at the cap-claystone contact, an r_u factor of 0.75 and a minimum factor of safety of 1.1, slopes on the order of about 3:1 would be indicated where caps are superposed on claystone slopes with average properties.

The foregoing considerations suggest the likelihood that for most occurrences, the optimum cut slope design would be 3:1. Only where appearance is considered unimportant, as on some supplementary roads, and where ditches are designed to accommodate debris storage should steeper slopes be considered. However fill slopes possibly could be as steep as 2:1 depending upon the degree of intermixing with other materials and the nature of those materials.

The swell potential of claystone in subgrades does not appear sufficiently serious to warrant extraordinary measures. Present practice, cut compaction and adjustment of moisture content above the optimum for those subgrade soils with liquid limits of 40 or greater, seems adequate.

Maquoketa Clay Shales.

The Maquoketa clay shales, by most of the criteria discussed, are intermediate in degree of potential for swell and desiccation. Using Figure 18, and assuming an average value of ϕ' from Table 6, an r_u value of 0.87 (equating to only a minimal degree of seepage stress and approximately half of that assumed for gley) and a minimum factor of safety of 1.1, slopes of about 3:1 would be indicated for long-term stability of Maquoketa clay shales when used in fills.

It is not proposed that similar criteria should be extended to cut slopes in this material.

Since all of the shear test data were developed from remolded material, it is likely that the effective angles of friction of in-situ material could be significantly lower due to the likelihood of a prevailing particle orientation associated with the laminar shale bedding. This is supported by observations of natural slopes which are flatter than 3:1.

Since Maquoketa clay shales in subgrades rarely cause serious problems, probably because of relative uniformity of swell experienced, extraordinary corrective measures do not seem indicated. Current Missouri specifications, which would require cut compaction and adjustment of subgrade moistures to optimum or above, should suffice.

General.

Table 6 and Figure 17 show that the shear strengths of the study materials, while quite low on average, are also variable. Foregoing conclusions with respect to slope design were based upon considerations of average properties and, as such, are believed applicable for purposes of preliminary planning and for devising treatments for use of minor quantities of the study materials. However the variability of the test data suggests that, when the study materials must be handled in large quantities, in deep cuts or in high fills, individual investigations should be performed, including shear testing and stability analysis. If this is done, required slopes for individual sites are likely to vary, plus or minus, from the predicted average slopes.

REFERENCES

1. Howe, W. B., and Heim, G. E. Jr., The Ferrelview Formation (Pleistocene) of Missouri, Department of Business and Administration, Division of Geological Survey and Water Resources, 1968.
2. The Stratigraphic Succession in Missouri, Department of Business and Administration, Division of Geological Survey and Water Resources, 1961.
3. Mineral and Water Resources of Missouri, Department of Business and Administration, Division of Geological Survey and Water Resources, 1967.
4. Stout, L. N. and Hoffman, D., An Introduction to Missouri's Geologic Environment, Missouri Geological Survey and Water Resources, 1973.
5. Anderson, K. H., et al, Geologic Map of Missouri, Department of Natural Resources, Division of Geology and Land Survey, 1979.
6. Lambe, William T., Soil Testing for Engineers, John Wiley and Sons, New York, 1951.
7. Bjerrum, L., and Simons, N. E., "Comparison of Shear Strength Characteristics of Normally Consolidated Clays", Proc., ASCE Research Conference on Shear Strength of Cohesive Soils, 1960.

