EVALUATION AND TREATMENT OF EXPANSIVE VOLCANIC SOILS US95, OWYHEE COUNTY, IDAHO

FINAL REPORT

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ABSTRACT

Soils at several locations along the first 18.5 miles of US95 in Owyhee County, Idaho, have been found to meet the diagnostic criteria for highly expansive soils. Many of the soils have liquid limits greater than 150 percent, shrinkage limits less than 18 percent, cation exchange capacities greater than 50 cmol/kg, Activity ratios greater than 1.5 and free swells greater than 100 percent. X-ray diffraction shows that the soils contain as much as 60 percent montmorillonite by weight. One-dimensional swell tests of specimens compacted to AASHTO T-99 maximum dry unit weight at water contents near optimum exhibited as much as 30 percent swell when inundated under small surcharge stresses. Swell pressures up to 6 tons/ft^2 were measured for the compacted soils.

The cause of the soil expansion is intake of water into the montmorillonite, an expanding lattice clay mineral. In order for potentially expansive soils to actually swell in an engineered structure, they must initially be in a water deficient condition as a result of stress or climate or both, and then water must become available as a result of a change in the soil's environment. The water deficient condition of near surface soils in Owyhee County is a result of the semi-arid climate of the region. The annual precipitation in the lower parts of the region averages eleven inches per year and mainly occurs as snowfall. Small amounts of water are therefore made available after pavement construction as result of infiltration of precipitation and concentrated runoff into the exposed but fairly impervious soils. More importantly, water becomes available as a result of changes in the evapotranspiration regime brought about by paving. In semiarid climates with deep water tables, decreased evapotranspiration invariably produces increases in water content of soils beneath covered areas. Pavement heaving resulting from either of the water sources mentioned may take several years to become noticeable.

In contrast to the swelling behavior shown by the compacted soils, more or less undisturbed, intact test specimens of the highly plastic Owyhee soils failed to exhibit significant swell or swell pressures upon inundation. It is suggested that the substantially reduced swelling of the impervious soils is due to their untypical origin. Many of the well-known and widely distributed expansive soils of the United States originated from shales formed from weathered basic igneous rocks and tephra deposited in seawater. The expansive soils of Owyhee County are believed to have developed mainly from subaerial hydrothermal alteration and devitrification of welded and slightly welded ignimbrites. The retention of some residual welding and or cementation in the intact material is offered as a possible explanation for the apparent lack of significant swelling. Other factors contributing to the reduced swelling of the undisturbed material are their generally lower insitu unit weights and higher water contents as compared to the remolded and compacted soils. If the results of the laboratory tests on the intact soils are generally applicable to the field, swelling of the Owyhee soils will likely be minimal as long as they can be preserved in an intact, undisturbed condition.

When the soils are remolded during construction and when pavement structural sections are to be placed on incoherent colluvial soils of high swelling potential, stabilization techniques will have to be employed to minimize and delay swelling. Standard stabilization methods include chemical treatments to reduce the expanding lattice minerals' affinity for water and the construction of physical barriers to prevent the ingress of water. The laboratory test program showed that lime is an effective stabilizer of the Owyhee soils, and the treatment did not produce detrimental expansive sulfate reactions even though the soils contain small amounts of gypsum.

The use of lime treatment for the Owyhee soils is limited to new pavement construction. Hydraulic barriers in the form of horizontal and vertical membranes coupled with shoulder and ditch paving are recommended for both new construction and existing pavements in areas currently exhibiting distress. While these measures cannot prevent all future swelling of the potentially expansive pavement subgrades, they will reduce the amount of swelling and delay its occurrence.

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CHAPTER ONE

INTRODUCTION

This report contains the results of the research conducted for Research Project 154, FC# 00-185, "Evaluation and Treatment of Expansive Volcanic Soils-US95, Owyhee County, Idaho." The project began February 1, 2000, and terminated on June 30, 2002. Expansive soils are defined in this report as any soil, which under certain conditions in certain environments is capable of exhibiting increases in volume (swelling) while experiencing increases in water content.[1]

Project Objectives

The objectives of the project were as follows:

1. Identify the causes of the swelling of the subgrade soils underlying the first 18.5 miles of US95 in Owyhee County, Idaho.
2. Identify methods to evaluate the swelling of expansive soils.
3. Identify treatment methods that will stabilize the subgrade soils for both existing and new construction pavements.

Project Scope

The scope of the project included the following activities:

1. Review the literature dealing with expansive soils in pavement applications.
2. Sample the soils in the project study area.
3. Perform appropriate laboratory tests to evaluate the soils and treatment methods.
4. Prepare interim and final reports.

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[1] A number of terms defined in a variety of ways have been used in the literature to describe concepts and phenomena associated with soil swelling. In an effort to minimize possible confusion, terms used in this report are defined where they first appear and again in a glossary contained in Appendix Two.

Organization of the Report

Chapter Two of the report is a description of the problem of swelling soils in the first 18.5 miles of US95 in southwestern Idaho, including the history of past construction and treatments along the route. Chapter Three contains a brief discussion of the geology of the study area and the hypothesized origin of the expansive soils and rocks. Insitu physical properties are given. The literature review on methods and criteria for identifying expansive soils and assessing the degree of expansion potential is in Chapter Four. Chapter Four also contains the results of classification tests and some of the swelling tests performed on the project soils. When evaluated using the criteria from the literature it can be seen some of the Owyhee soils classify as having an extremely high swell potential. Alternative treatment methods for expansive soils are discussed in Chapter Five. The effectiveness of lime treatment for the Owyhee soils is evaluated in detail. Conclusions and recommendations from the study are given in Chapter Six.

CHAPTER TWO

BACKGROUND: SWELLING SOIL ON US95, OWYHEE COUNTY, IDAHO

Introduction

This chapter reviews the history of construction and swelling soil problems in the first 18.5 miles of US95 in Owyhee County. The review represents an interpretation of records provided to the Principal Investigator by District 3 of the Idaho Transportation Department (ITD). The chapter contains a resume of the activities completed for this study including a description of the field and laboratory work performed by the Principal Investigator and students at the University of Idaho and Boise State University under Research Contract No. 154.

Construction of US95 from MP 0.0 to 18.5

It was not possible to determine the exact dates of all the construction and rehabilitations of the current alignment of the first 18.5 miles of US95 from the records available. The records included three sets of construction plans (F-3111(32) for grading, dated 1981; FLH 11-1(6) for grading, dated 1982; and FLH 11-1(7) for grading and paving, dated 1983), a Vicinity Sketch from a Phase II report dated April 4, 1991, and various other Phase II and III reports. Therefore, what follows is only a best estimate of the actual construction dates.

1972: Construction from the Oregon line (MP 0.0) to MP 6.36 under ST 3111(521). Expansive soils were treated by compacting slightly above optimum water content and separating them from the ballast with an asphalt membrane.

Early 1980's: Construction on new alignment from MP 6.36 to MP 18.5, with 1.25 ft of lime-stabilized subgrade between MP 6.36 and MP 13.2.

1985: Cover coat applied to new alignment of US95 from MP 13.4 to MP 20.67.

1989: Reconstruction between MP 16.7 and MP 17.9 to correct pavement heaves with 1.25 ft of lime stabilized subgrade.

1992: Reconstruction between MP 0.22 and MP 5.33 to correct pavement heaves with 1.25 ft lime stabilized subgrade.

Concerns with Expansive Clays

It appears that the first geotechnical study for the new "Oregon Line to Elephant Butte" construction was the Phase II Report completed for MP 0.0 to MP 6.5. In this report the presence of plastic soils with liquid limits of as much as 125 and commonly above 70 was mentioned. Special treatment to deal with these soils involved compaction at water contents slightly above the optimum, the use of impermeable asphalt membranes and higher than usual ballast thicknesses. A Phase I Geologic Reconnaissance Report completed by Humphrey (1971) for anticipated new construction between Mileposts 16.0 and 18.0 likewise did not specifically mention expansive clays.

The first comprehensive geotechnical study performed for the new Oregon Line to Elephant Butte alignment was completed in 1979 by personnel of the Federal Highway Administration (Ulrich, 1979). Potential problems expected from formations encountered in the project area were described and concerns about the stability of fills constructed with the clay soils were expressed. It was suggested that the expansive CH clays would require special treatments such as lime or the use of encapsulating membranes to make them suitable for subgrades.

In an intra-department correspondence dated 1986, the ITD Soils and Foundations Engineer reported that severe pavement heaves were being produced by expanding subgrade soils between Mileposts 0.0 and 6.36 and also between 16.7 and 17.9, with most of the distress occurring at gradepoints (Smith, 1986). The communication also referred to a sampling and testing program completed in 1985 which revealed that the clay subgrade soils were moderately to highly expansive, and that most insitu water contents were between 6 and 25 percent above the (T-99) optimum water contents. Methods for stabilizing expansive subgrades were mentioned, and additional studies were suggested.

In 1988, an intra-departmental correspondence titled "Swelling Soils - Oregon Line to Elephant Butte" presented a thorough examination of the swelling soils and the associated problems within the study area (Nottingham, 1988). The geology of the area and the origin of the problem soils was described. Drainage maintenance deficiencies were identified as the major source of the water causing the swelling. The apparent effectiveness of the lime stabilized subgrade in preventing heaves between Mileposts 6.36 and 13.2 was noted. The use of lime in future construction and/or rehabilitation of pavements in areas underlain by the expansive soils was suggested.

In a followup to the Nottingham report (also in 1988), the ITD Geotechnical Engineer supported the concept of lime subgrade stabilization with "5 percent lime" and further suggested that during the pavement reconstruction the stabilized subgrades be maintained in a condition as close to saturation as possible, and that an asphalt membrane be placed on top of the treated subgrade. After considering and rejecting the use of pressure injected lime fly ash slurry and a patented soil stabilizer known as Condor SS, in subsequent phase reports for projects F-3111(49) ITD engineers recommended using1.25 ft thick subgrades treated with 5 lbs/ft^2 quicklime (80 percent CaO) compacted to 95 percent of T-99 maximum dry unit weight. An asphalt membrane (CRS-2R) was to be placed on top of the lime stabilized subgrade.

Beginning in October of 1999, discussions concerning the swelling soils problem on US95 were initiated between personnel of District 3 and the Department of Civil Engineering at the University of Idaho. An agreement between ITD and the University for research titled "Evaluation and Treatment of Expansive Volcanic Soils-US95, Owyhee County" was finalized in February, 2000. The research activities completed by the University under this contract are briefly described in the following paragraphs.

University of Idaho Research Activities

Table 2.1 contains a list of research activities planned for the project at the University of Idaho. As shown in the table, some of the effort in the project was completed by the Idaho Transportation Department (ITD) and some was performed under a subcontract with Boise State University (BSU). Each of the activities and the results are described in the following paragraphs.

Develop Work Plan - A meeting of the Principal Investigator and members of the ITD Advisory Board for the project were held in Boise in November 1999. Sampling sites and the test program were discussed at the meeting, and a field trip to the study area was made by the Principal Investigator and the District 3 Geological Engineer.

Table 2.1 Description of research activity

Activity	Activity Number	Performing Organizations
Develop work plan	1	UI, ITD, BSU
Literature review	2	UI
Field drilling and sampling	3	UI, ITD
Lab tests to characterize the soils	4	UI, BSU
Treatment evaluation tests	5	UI, BSU
Laboratory scale treatment evaluations	6	UI
Final report	7	UI, BSU

Literature Review - The review of the technical literature was performed by the Principal Investigator at the University of Idaho. The review encompassed methods to identify and characterize expansive soils, test methods to quantify swelling and treatment methods to reduce or eliminate soil swelling. The results of the review appear in Chapters Four and Five.

Field Drilling and Sampling - Ten four-pound grab samples were collected from the surface in November 1999. The small disturbed samples were used for preliminary plasticity tests, mineral identification and for selecting drilling locations. The road log is contained in Appendix Four.

Drilling and sampling of the subsurface soils in the project study area was performed by ITD in March 2000. Drilling was performed dry using hollow stem augers. The sampling equipment available for the project was ITD's CME continuous sample tube system. With this system the soil sample is retained inside 3.25 inch inside diameter by 2.5 ft long plastic liners. The liners fit inside a 5 ft long split-tube barrel sampler, which has an outside diameter of 4.00 inches. The non-rotating liner and split-tube barrel are pushed into the soil just ahead of the bit of the rotating hollow stem auger. Samples recovered with these tools cannot be considered to be undisturbed due to the combined wall thickness of the liner and split-tube barrel of 0.375 inches, which results in a kerf (or area ratio) of 51 percent. Recommended values for open-drive undisturbed sampling in fine-grained soils are less than 10 to 15 percent (Hvorslev, 1949). Table 2.2 is a list of the borings with their locations and depths. Logs of the borings completed by the UI Graduate Research Assistant are included in Appendix Five.

Fifty pound grab samples of surface soils were collected at Mileposts 2.3East, 17.0West, 17.5West and 17.7West in September 2000. These samples were used in the compaction and treatment evaluation tests.

Lab Tests to Characterize the Soils - Common index property tests to characterize the expansiveness of the soils, x-ray diffraction analysis to identify clay minerals and other compositional analyses were performed on representative project soils both in their natural and treated conditions. Many of these test results are presented in Chapters Four and Five where their significance and applications to the swelling problem are discussed. The results of additional X-ray diffraction performed on lime-treated compacted samples are contained along with cation exchange capacity, chemical composition, grain size distribution, and specific gravity test results in Appendix Three.

Table 2.2 Locations and depths of borings completed in March 2000

Mile Post	ITD AH No.	Centerline Offset		Depth feet
		Direction	Distance, ft	
1.2	11	East	35	27.6
2.3	5	East	20	28.1
6.8	12	East	30	27.6
7.7	13	East	30	5.6
7.7	14	West	30	27.5
9.1	4	East	35	26.5
17.0	3	East	35	30
17.6	7	West	50	28.1
17.7	2	West	50	25
17.7	6	West	60	25.1
17.9	1	West	~35	~50
17.9	8	East	45	28.1
18.4	9	East	35	16.9
18.4	10	East	22	27

Treatment Evaluation Tests - Tests were preformed on specimens from Mileposts 2.3 and 17.0 to determine the appropriate lime treatment levels for swell prevention. Chemical properties were subsequently measured for the treated soils. The principal results of these tests are in Chapter Five with the remaining supporting results summarized in Appendix Three.

CHAPTER THREE

GEOLOGY, SOILS AND CLIMATE

Introduction

This chapter contains a brief description of the geology, soils and climate of the study area. The discussion of geology is taken mainly from the Intra-Departmental Correspondence by Nottingham (1988), although the entire document is not reproduced herein. Direct quotes from Nottingham are indicated by italics. The section on the physical properties of the soils includes the results of tests on samples collected by ITD for previous design studies as well as results for samples collected for this study during March 2000.

Geology

Geologic Setting - *The Owyhee Uplands section of the Columbia Intermontane province is a high plateau south of the Malheur-Boise-King Hill section (Ross and Savage, 1967). Much of the surface is at an elevation of 4,000 to 5,000 feet, but several mountain masses rise to 8,000 feet. Lower areas are deserts because of low precipitation. Most of the lavas of the Owyhee Uplands are older than those of the Snake River Plain to the north, and although some basalt flows are present, many of the rocks are rhyolites and welded tuffs. Miocene basalt covers several lower areas north of Silver City and the mountainous uplands are flanked by stream-dissected silicic volcanic flows, ash deposits and wind-blown loess deposits. The core of the Owyhee Mountains, including War Eagle and South Mountain, is mainly granitic rock.*

Structurally, the Owyhee Uplands section is an uplifted area with doming and block faulting common. Because of the high elevation and long erosion activity the Owyhee Uplands is one of the more deeply dissected sections of the Columbia Intermontane province.

Rhyolitic rocks were erupted from vents in and adjacent to the Owyhee Mountains and Owyhee Plateau of southwestern Idaho from 16 m.y. ago to about 10 m.y. ago as reported by Ekren, McIntyre and Bennett in 1984. They were deposited on a highly irregular surface developed on a variety of basement rocks that include granitic rocks of Cretaceous Age, quartz latite and rhyodacite tuffs and lava flows of Eocene Age, andesite and basaltic lava flows of Oligocene Age, and latitic and basaltic lava flows of early Miocene Age.

The rhyolitic rocks are principally welded tuffs that, regardless of their source, have one feature in common -- namely internal characteristics indicating en masse, viscous lava-like flowage. The flowage features commonly include considerable thicknesses of flow breccia at the bases of the various cooling units. On the basis of the tabular nature of the rhyolitic deposits, their broad areal extents, and the local preservation of pyroclastic textures at the bases, tops, and distal ends of some of the deposits, they [Ekren, et al., 1984] have concluded that the rocks were emplaced as ash flows at extremely high temperatures and that they coalesced as liquids before final emplacement and cooling. All ash-flow tuffs emplaced in the Owyhee region between 16 m.y. ago and 10 m.y. ago show evidence of high emplacement temperatures whether derived from sources within the highlands or in the adjacent western Snake River Plain.

The recognition of ash-flow tuffs has long presented a difficult problem because of the lack of knowledge concerning their mode of deposition and the reliability and limitations of field and laboratory criteria (Ross and Smith, 1961). Non-welded ash-flow tuffs are often confused with tuffs of other origins and welded tuffs are often confused with lava flows, normal vitric tuffs are sometimes fused in contact with lava flows or shallow intrusions and these may be mistaken for welded ash-flow tuffs.

The most important single criterion for recognition of the pyroclastic nature of ash-flow tuffs in the field seems to be the presence of pumice fragments. A principal characteristic of ash-flow tuffs is their common occurrence in thick units (tens of feet) of typically nonsorted or nonbedded materials. This characteristic is in direct contrast to ash-fall tuff deposits of comparable thickness in which pronounced bedding is nearly always present. Different zones of single flow units have undergone various degrees of consolidation giving a layered appearance commonly mistaken for the bedding of several flows.

Rhyolite lavas rarely occur without glassy flow breccia at the base. The lavas move so slowly that their outer margins chill, are broken and are overridden by the liquid interiors. Ash flows were emplaced at speeds of 100 Km/hour and typically are marked by several meters of nonwelded tuff without flow brecciation. In SW Idaho brecciation of tabular sheets of welded tuff occurred at the same time the ash was welding and the basal part was being chilled (Ekren and others, 1984).

Volcanic-derived sediments of the Sucker Creek Formation and Poison Creek Formation as shown on the Generalized Geologic Map [Figure 3.10] *and described in the stratigraphy section comprise the problematic soils in the project area.*

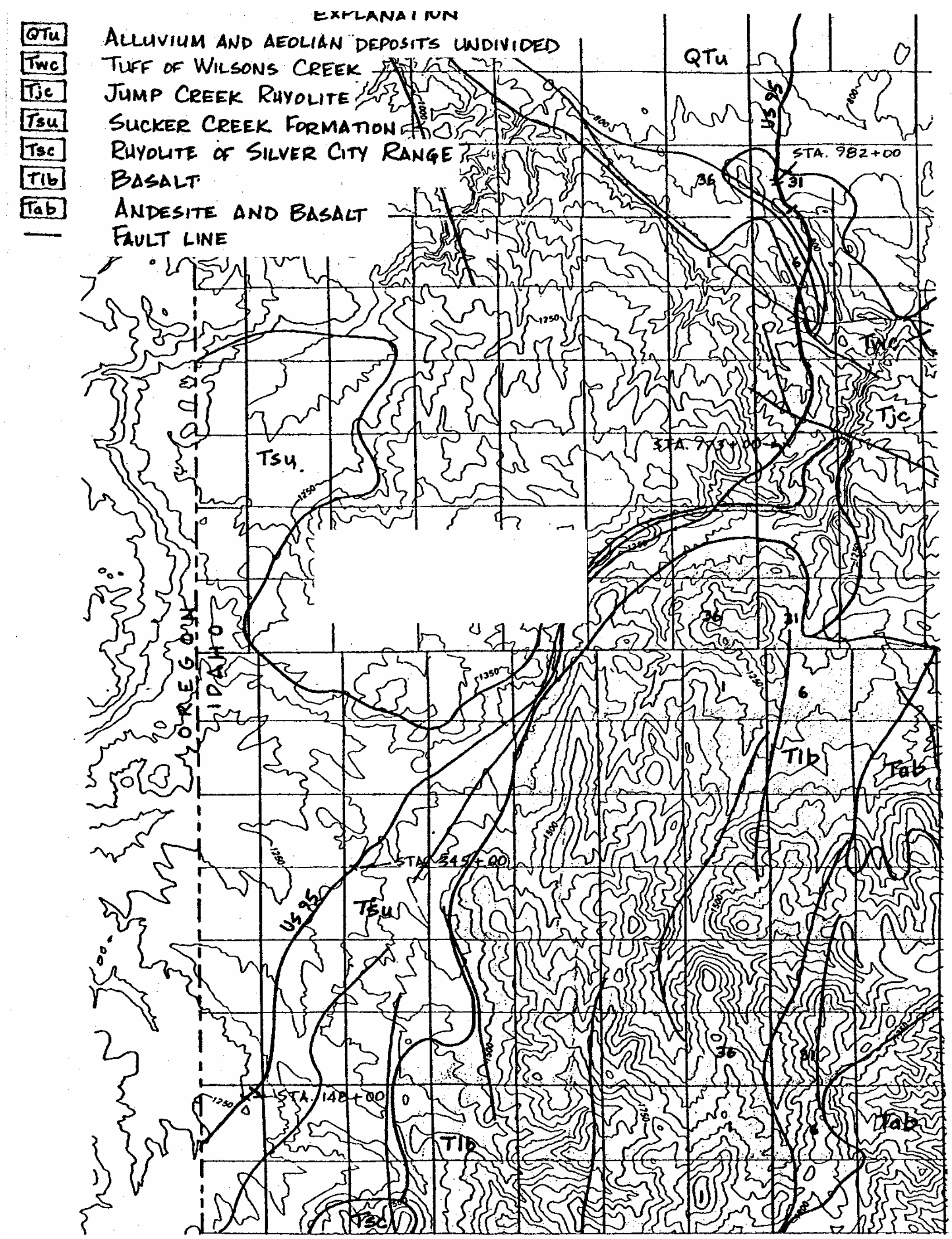

Figure 3.1 Generalized geologic map of the study area (adapted from Nottingham, 1988)

Stratigraphy

COLLUVIUM AND TERRACE GRAVELS (Quaternary, 0-5 meters)

This unit consists of slope wash and talus deposits of angular rhyolite and vitrophyre cobbles and boulders mixed with terrace gravels which are poorly graded in a brown sandy silt.

JUMP CREEK RHYOLITE (Miocene Age, 0-250 meters)

Conspicuously porphyritic rhyolite with alternating crystal-poor and crystal-rich layers containing plagioclase as large as 15 mm; massive in places with eutaxitic foliation suggestive of welded tuff; flow layered in other places. Thick flow breccias at base in many exposures indicate that the rock is either a lava or a tuff that was remobilized; rock is grayish red, medium gray or brownish gray; weathering dark gray and dark brownish gray; vitrophyres are black. The rhyolite rests on the montmorillonite rich Sucker Creek Formation and, as a consequence, landslides are common.

SUCKER CREEK FORMATION (Miocene Age, 0-500+ meters)

Altered and vitric nonwelded bedded tuff, volcanic sandstone, arkose, granite-cobble conglomerate, and minor carbonaceous mudstone; intruded locally by thin basalt dikes. Most of the beds are yellowish gray or yellowish brown; conspicuous white beds of tuffaceous sandstone and siltstone are found locally; most granitic cobbles in the conglomerate are well rounded and are set in a well-cemented conglomeratic sandstone matrix. A few conglomerate lenses contain abundant cobbles of rhyolite, latite, and basalt as well as granite. This unit is called ash-tuff in the report for project FLH 11-1(3).

POISON CREEK FORMATIONS (Miocene Age, 0-120+ meters)

Gray, buff, and white lacustrine and stream silt, sand and clay, mostly tuffaceous and in places much altered to montmorillonite. This unit corresponds to the ash-tuff unit in report FLH 11-1(3).

Clays in the Owyhee Soils

According to Ulrich (1979), both the Sucker Creek and the Poison Creek formations contain substantial quantities of montmorillonite. The expansive clay strata are presumably weathering products of the ash-flow tuffs found in these formations as well as transported and sedimented materials eroded from the weathered ignimbrites. Nearly all the host rock in the area, including the rhyolite beds, have experienced some degree of alteration to clays which include smectites, illites and the kaolin group as well as mixed-

Table 3.1 Physical and index properties of US95 soils in June 1970 (Nottingham, 1988)

Milepost (approx.)	Station	Offset feet	Depth feet	Soil	Liquid Limit percent	Plastic Index percent	Water Content Percent	
							T-99 opt	Natural
2.6	135+00	0	15.5-40.5	clay	202	167	50.0	84.4
2.7	141+00	0	11.7-27.5	clay	125	89	42.6	75.8
3.5	184+90	15L	12.3-18.5	brn clay	135	94		
3.5	186+70		0	sandy silt	54	14	26.9	41.1
3.9	206+90	0	0	tan clay	99	58	43.8	78.8

layer clays. In the eastern parts of the region the smectites tend to be beidellite and mixed-layer clays (Post et al., 1997), whereas in the western part of the region which includes the US95 soils, the predominant smectite is montmorillonite (see Table 4.1). Both these 2:1 layer expanding lattice clay minerals are common argillic weathering products of basalts and other mafic rocks weathering under arid and semi-arid climates (Arnold, 1984). Devitrification of the large amounts of glass in the siliceous ash-flow tuffs also produces crystobalite and alkali feldspars, the latter being highly susceptible to argillic alteration (Ross and Smith, 1961; Williams and McBirney, 1979).

Condition of the Expansive Soils in Owyhee County

The physical condition of the soils along the US95 alignment has been investigated several times since the problems associated with expansive soils began to appear. Tables 3.1 through 3.3 show plasticity data and both natural and T-99 optimum water contents as reported by Nottingham (1988). Figure 3.2 shows the distribution of natural water contents with depth at five stations on the alignment as of March 2000. Table 3.4 lists plasticity data and water contents measured for materials beneath the pavement between Mileposts 16.9 and 18.0 in 1992 as reported by Moles (1992).

Climate of the Study Area

Figure 3.3 lists values of the average annual values of temperature in degrees Fahrenheit, precipitation in inches, Thornthwaite Moisture Index (TMI) and freezing index in degree-days for Owyhee County reported in a previous study (Hardcastle, 1992). According to criteria suggested by Thornthwaite (1948), the climate in Owyhee County would be described as semiarid.

Table 3.2 Physical and index properties of US95 soils in swell areas in January 1985 (Nottingham, 1988)

Milepost (approx.)	Depth feet	Soil Type	Liquid Limit percent	Plastic Index percent	Water Content Percent	
					T-99 opt	Natural
0.4	2.8-5.0	gray clay	87	60	34.4	40.0
1.22	2.0-5.0	brown clay	83	50	34.2	34.4
2.02	2.0-5.5	gray clay	85	63	30.1	36.5
2.4	1.9-5.0	red clay	81	53	34.5	43.6
2.5	1.8-5.0	yellow clay	115	87	36.5	61.9
2.6	2.0-4.2	brown clay	51	25	30.0	34.9
3.1	2.4-4.4	brown clay	85	59	36.0	43.7
3.25	2.0-5.0	yellow clay	99	72	29.0	42.4
3.75	2.2-5.0	brown clay	83	46	35.5	53.9
5.4	2.1-5.2	brown clay*	43	22	16.6	20.7
5.55	2.1-5.2	brown clay	108	86	39.6	51.0
6.18	2.0-5.1	gray clay	113	86	41.4	55.5
17.56	4.5-7.5	gray clay	104	75	43.0	48.6
17.60	3.0-7.5	gray clay	104	75	43.0	60.6
17.65	3.0-7.5	gray clay	104	75	43.0	47.1
17.7	3.0-7.5	gray clay	104	75	43.0	54.7
17.8	3.0-7.5	gray clay	104	75	43.0	56.0

*with gravel

Table 3.3 Physical and index properties of US95 subgrade soils in October 1986 (Nottingham, 1988)

Milepost (approx.)	Depth feet	Soil Type	Liquid Limit percent	Plastic Index Percent	Water Content Percent	
					T-99 opt	Natural
0.4	2.7-5.7	clay	78	43		39, 56
2.015	2.9-5.0	clay	93	64		44, 48
2.35	2.8-6.0	silty clay	87	55		38
2.5	2.7-6.0	clay	140	94	45	70, 69
2.5	2.9-6.0	clay	178	135	44.7	62
2.72	3.3-3.8	silt, gravel				55
3.10	3.8-5.0	clay	113	69		52, 46
3.25	2.9-6.5	clay				43
3.75	3.5-6.5	silt				53, 57
5.60	4.5-7.5	clay	134	82		66, 63
6.18	3.5-7.4	OH	57	27	29	25, 59
6.2	6.4-7.4	OH	57	27	28.7	31
17.30	4.0-7.5	clay	124	75	33	56, 39
17.3	5.6-6.6	clay	124	75	32.9	59
17.65	3.5-7.0	clay			37	53
17.65	3.8-7.0	clay	95	55	37	53
17.77	3.8-7.5	clay				74, 69
17.83	3.7-7.0	clay				67, 67

Table 3.4 Physical and index properties of subgrade materials from MP 16.9 to 18.0 (Moles, 1992)

ITD Drill Hole No.	Sample Interval feet	Material	InSitu Water Content percent	Liquid Limit percent	Plastic Index percent	T-99 Compaction	
						Opt. Water Content percent	Max. Dry Unit Weight, lbs/ft^3
HAH-1	1.0-2.2	LTSG*	69.2				
HAH-1	2.2-3.1	CH	74.7				
AH-2	0.4-1.1	AggBase	4.6				
AH-2	3.3-5.2	LTSG	64	NP	NP	35.8	79.4
AH-2	5.2-7.3	CH	66.5	135	91		
AH-3	0.45-1.3	AggBase	4.2				
AH-3	3.4-5.1	LTSG	58.4	99	41		
AH-3	5.1-7.4	CH	63.5	141	104		
HAH-4	1.1-2.2	LTSG	73.2				
HAH-4	2.2-2.8	CH	68.4				
AH-5	0.45-1.2	AggBase	4.1				
AH-5	3.4-5.1	LTSG	63.1			35.8	79.4
AH-5	5.1-7.5	CH	65.9	136	86		
AH-6	0.4-1.1	AggBase	4.5				
AH-6	3.6-5.0	LTSG	65.7	103	33		
AH-6	5.0-7.0	CH	63.9	131	76	44	64.7
HAH-7	1.1-2.4	LTSG	30.4				
HAH-7	2.4-3.0	CH	26.6				
AH-8	0.5-1.1	AggBase	4.4				
AH-8	3.0-4.4	LTSG	23.2		NP	35.8	79.4
AH-8	4.4-7.4	CH	66.5	89	48		
AH-9	0.5-1.1	AggBase	4.5				
AH-9	3.5-5.7	LTSG	33.5			35.8	79.4
AH-9	5.7-7.4	CH	50.1	93	44		

*Lime-treated subgrade

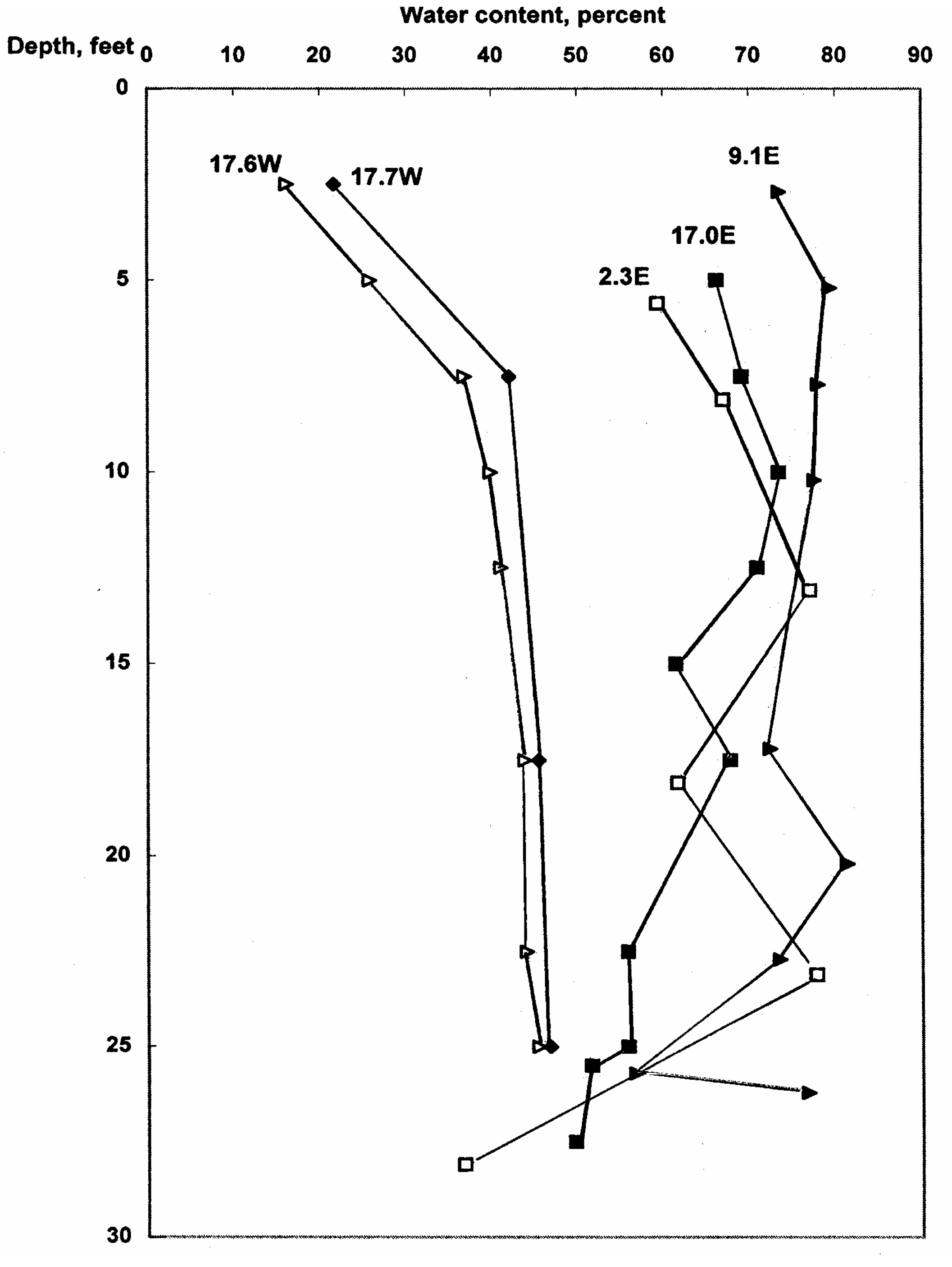

Figure 3.2 Distribution of natural water contents with depth, March 2000, at 5 locations on US95

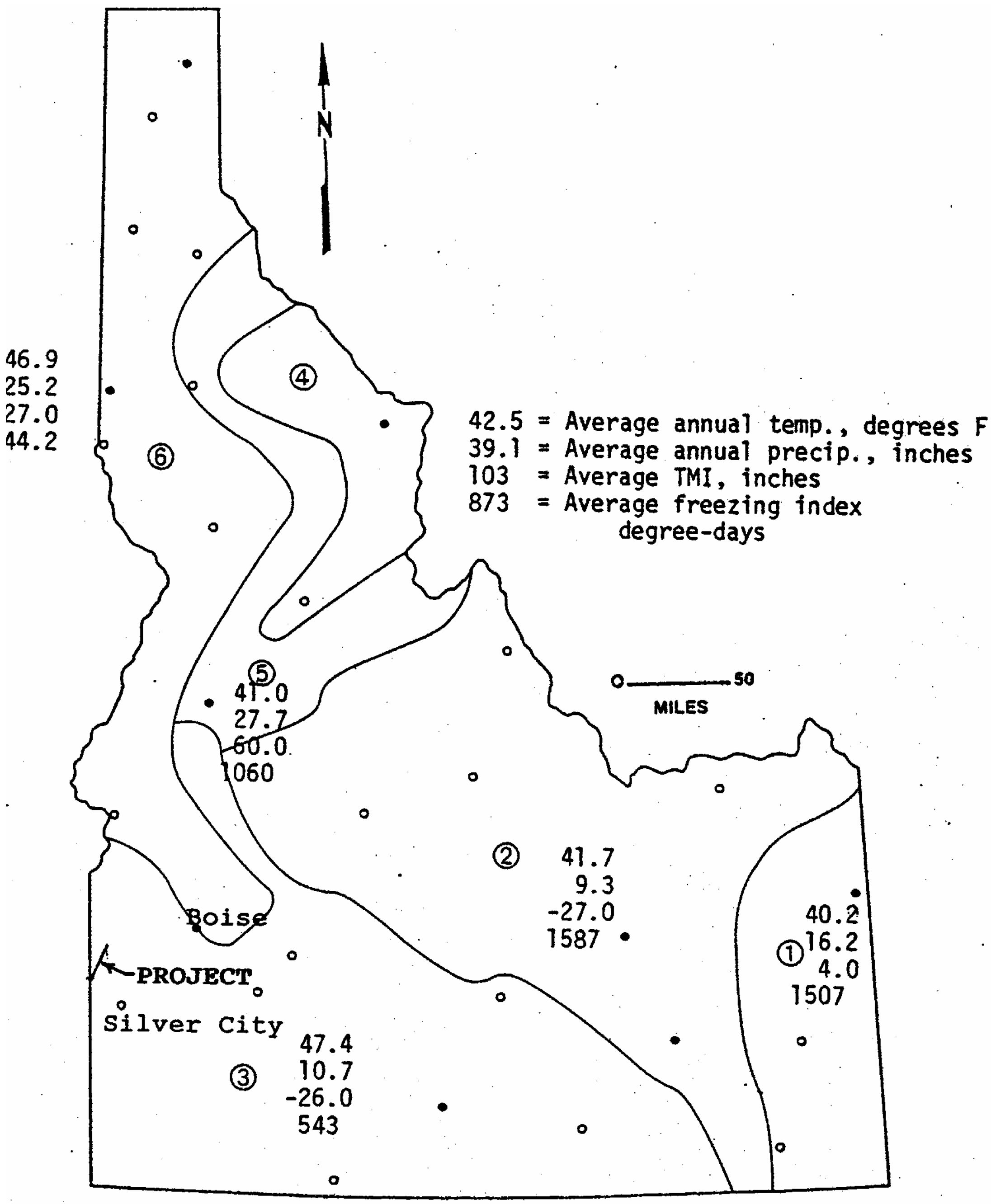

Figure 3.3 Generalized climate parameters for Idaho (Hardcastle, 1992)

Conclusions

Laboratory tests performed by ITD have consistently shown that the US95 alignment is underlain by highly plastic and therefore potentially expansive soils. The tests results presented in the tables and the figure indicate that insitu water contents are consistently higher than the optimum water contents for AASHTO T-99 compaction. In some cases, the natural water contents are as much as 35 percent greater than T-99 optimum. This result suggests that if the subgrades are compacted to dry unit weights approaching the T-99 maximums at water contents close to the T-99 optimums, the compacted soils would be both denser and dryer than the natural subgrades. In this condition the highly plastic subgrade soils would certainly be in a potentially expansive state and could be expected to swell as they experience the inevitable post-paving water increases induced by the paving-caused changes in the evapotranspiration regime (as discussed in Chapter Four).

The water content data in Table 3.4 includes values measured in 1992 for the base and lime-treated subgrade as well as for the untreated subgrade. Surprisingly, the data show that the water contents of the lime-treated subgrades are generally very close to those of the untreated clay subgrades. One would expect that equilibrium water contents (and suctions) of the lime-treated subgrade would be lower than the untreated CH clay subgrades.

The plasticity results in the table indicate that the lime-treatments (presumably 5 percent) were not always effective in reducing the clay plasticity to levels associated with non-swelling clays. Finally, the low water contents consistently measured for the aggregate base indicate that this material is close to being free-draining and likely to have a high permeability relative to the permeability of the soils.

CHAPTER FOUR

IDENTIFICATION AND CHARACTERIZATION OF EXPANSIVE SOILS

Introduction

Laboratory methods and criteria for assessing the swelling potential of soils are reviewed in this chapter. Also presented are results of laboratory tests performed to characterize the swelling behavior of the US95 soils. The purpose of the literature review is to provide a basis for comparing the physical and chemical properties of the Owyhee soils with those of other swelling soils described in the literature. Such comparisons will aid in the evaluation of the applicability and probable success of treatment methods that could be applied to the Owyhee County soils.

Wetting-induced expansion of subgrade soils detrimental to pavements requires three conditions be met. First, a soil that contains materials capable of increasing their volume when subjected to increases in water content must be present. This requires that the soil contain expanding lattice type clay minerals. Expanding lattice clay minerals can change the amount of water held within the clay crystal lattice through direct adsorption and by adsorption of hydrated exchangeable cations. The more water taken into the crystal lattice and adsorbed onto external surfaces of the clay particles, the more volume the soil will have. Second, the soil must either exist in or be put into a condition and stress state where it has a tendency to take additional water into the clay mineral lattices. In other words, the soil must have an affinity for water or a water deficiency consistent with its stress state and water content. When the first two requirements are met, a potentially expansive soil exists. In order for a potentially expansive soil to actually expand (swell), a third condition must exist, that is, water must become available to be taken into the soil in response to its current water deficiency. This chapter reviews criteria and laboratory tests used to identify and characterize the expansiveness of soils.

Test methods are grouped into five categories. The first includes methods, which involve direct determination of the presence of expanding lattice clay minerals. The second category includes methods, which involve indirect assessments of soil mineralogy using common soil index properties tests. The third category utilizes soil physical properties, which are more difficult to measure than index properties like the Atterberg limits. Laboratory swelling tests that yield indices of swelling but which don’t produce fundamental parameters that can be used in to calculate heaves or settlements in field applications comprise the fourth category. The fifth category includes the methods involving direct quantitative measurements of the expansion of soils as they are wetted from an initially drier condition to a final wetter condition. The first four approaches provide an indication of the presence of expansive clay minerals in the soil and the possible significance of the swelling soil problem. Only the last method directly assesses the expansion potential of the soil in the specific conditions and environments in which it may exist in the field.

Identification of Swelling Clay Minerals

Expansive soils must contain minerals, which increase their volume when wetted. Identification of the type and amount of expanding lattice clay minerals in a soil is fundamental to its swell potential evaluation. It is also believed that the total smectite mineral content of soils is a reliable indicator of swell potential (Olson et al., 2000). Of course, identification of the presence of expanding lattice clay minerals in a soil indicates only that the soil could expand if the two other requirements stated above are met.

Methods used to identify clay minerals in soils include X-ray diffraction, electron microscopy, differential thermal analysis and wet chemical analysis (Mitchell, 1993). Because of the requirements for special, often expensive apparatus and skills, the direct identification methods are not routinely used in civil engineering practice. The results of X-ray diffraction analyses performed on the Owyhee soils are given in Table 4.1. The smectite contents of the soils indicate that the soils are very expansive.

Table 4.1. X-ray mineralogical analyses of five US95 soils from Owyhee County

Mile Post	**Minerals, percent**					
	Smectite	**Glass**	**Quartz**	**Feldspars**	**Zeolites**	**Gypsum**
2.3	60	35		5		
2.3	60	25		15[1]		
9.1	50	35		15[2]		
12.8	30	30	10	5		5
17.0	50		20[3]		15[4]	10

[1]Andesine, [2]Orthoclase and Oligoclase, [3]Cristobalite-Tridymite,

[4]Phillipsite and Clinoptilolite-Heulandite

Index Property Tests for Evaluating Swell Potential

The indirect methods to assess soil expansiveness in terms of swell potential utilize empirical relationships among easily measured index properties and the one-dimensional swelling response of soils after they have been brought to some specific initial state and then inundated. Initial states for swell tests have been specified in terms of dry unit weight, water content and method of compaction. The swell potential of a soil is usually described qualitatively using such terms as low, medium, high and very high corresponding to the amount of one-dimensional volume change occurring after the test specimen is inundated. The swell tests used in the correlation are described below.

The first and still most commonly used index properties correlated with swell potential are the soil consistency limits. Plasticity limits are only indirect measures of the presence of expansive clay minerals in soils, and by themselves are insufficient to predict the actual amount of swelling that can be expected unless the initial soil condition of interest is exactly the same condition as was used in the original development of the swell prediction relationship. This is rarely the case.

Soil plasticity is an indirect measure of the colloidalness of the soil and its ability to interact with water. Soil plasticity limits used to identify expansive soils include the shrinkage, plastic and liquid limits and the plasticity index. Examples of early and still widely used plasticity criteria are given in Table 4.2 and Figure 4.1. The plasticity properties of the Owyhee soils determined in this study are given in Table 4.3. Although the relationships of the consistency limits of the Owyhee soils are not always consistent with those suggested by Holtz and Gibbs, it’s clear that the Owyhee soils are well beyond the range of “very high” expansiveness.

Table 4.2 Expansive soil classification based on soil plasticity (Holtz and Gibbs, 1956)

Shrinkage Limit	**Liquid Limit**	**Plastic Index**	**Potential for volume change**
>15	20-35	<18	Low
10-15	35-50	15-28	Medium
7-12	50-70	25-41	High
<11	>70	>35	Very High

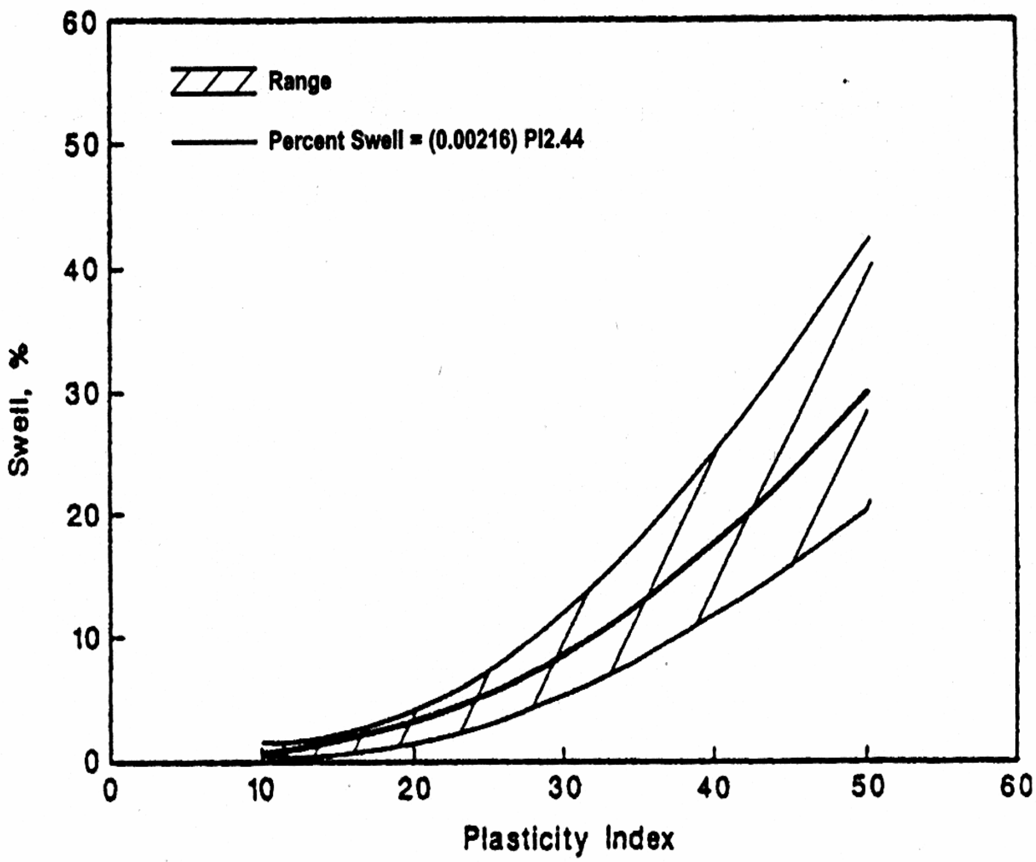

Figure 4.1 Swell potential as a function of soil plasticity index (Seed, et al., 1962)

Table 4.3 Plasticity limits of US95 soils from Owyhee County

Mile Post	Depth, feet	Shrinkage limit, percent	Liquid limit, percent	Plastic Index, percent	Volume change potential
0.6	0	16	133	80	very high
1.2	2.6		103	50	very high
2.3	0 4.8 5.6	11	192 130 168	140 69 110	very high very high very high
6.8	2.6		129	95	very high
7.7	2.5 10		NP 105	NP 78	low very high
9.1e	5.0		95	73	very high
12.8	0	28	50	17	medium
17.0e	5.0		127	79	very high
.6w	8.1		185	136	very high
17.7w	8.0		129	81	very high
17.7w	2.0		58	41	high
17.8w	0	17	73	37	high
17.9e	5.3		NP	NP	low
18.4e	2.6		44	25	medium

Advanced Soil Physical Properties

More recent empirical relationships between expansion potential and physical properties of soils include combinations of soil plasticity and other soil properties such as colloids content, cation exchange capacity, soil suction, properties of the soil moisture characteristic curve, and the suction compression index. The tests for these physical properties can be substantially more difficult than the Atterberg limits tests. Examples of these relationships are discussed in this section.

Colloids (or clay) content is defined as the percent by weight of soil particles smaller than 0.002 mm. Colloids content is usually determined in hydrometer tests. The influence of colloids content on soil expansion potential is through its indirect relationship to the density of positive charge deficiency of clay minerals. The finer the particles in a soil, the greater the likelihood that the soil contains charged particles. Example of the use of colloids content to assess swell potential is given in Figures 4.2 and 4.3. The Activity term in the figures is defined as the dimensionless ratio of plasticity index to colloids content, with both in percent. Table 4.4 shows plasticity and colloids content of some of the US95 Owyhee County soils.

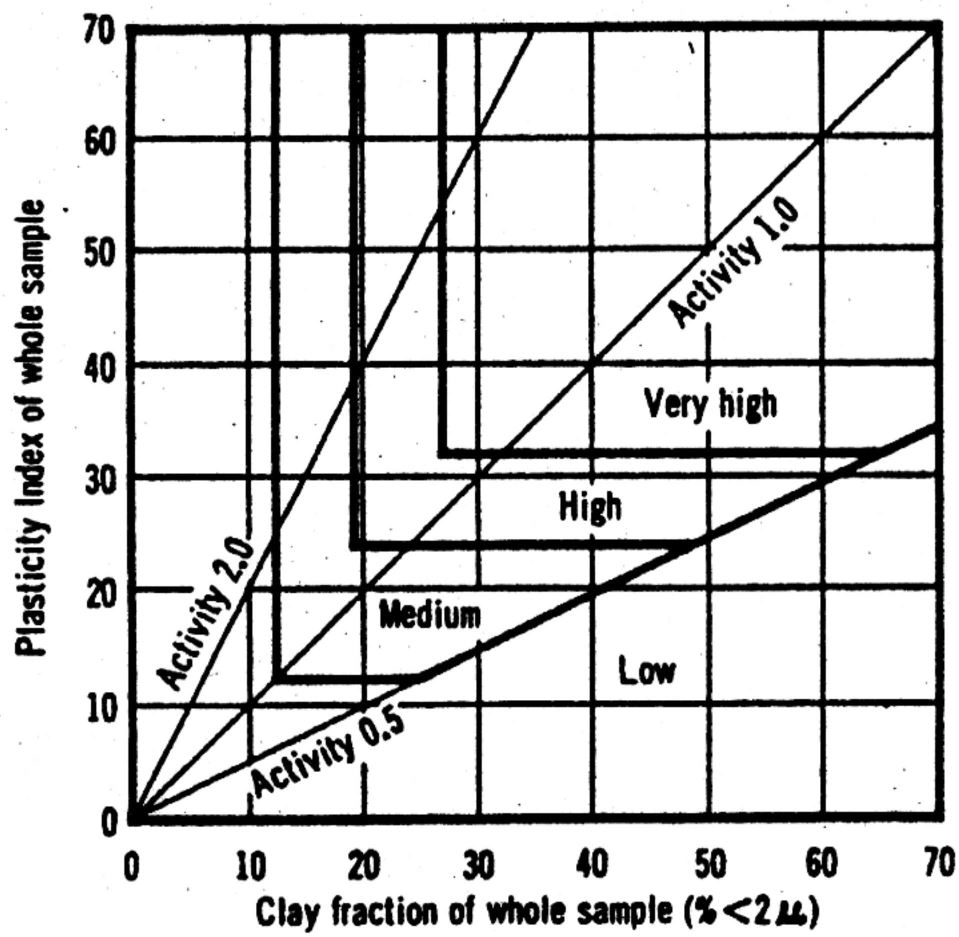

Figure 4.2 Swell potential as function of colloids content and Activity (Seed, et al., 1962)

Table 4.4 Cation exchange capacity (CEC), Activity and swell parameters of US95 soils

Mile Post	**Depth feet**	**Plasticity Index Percent**	**Cation Exchange Capacity meq/100g**	**Colloids content percent**	**Activity**	**Estimated COLE Percent**	**Expansiveness Rating Figure 4.5**
1.2E	2.6	50	49	52	0.96	10	very high
2.3E	0	69	82	70	0.99	15	very high
2.3E	5	110	86	73	1.5	15	very high
9.1E	5	73	51	23	3.2	3	very high
12.5E	0	17	10	21	0.81	1	low
17.0E	5	85	64	66	1.29	14	very high
17.0 W	0	79	66	68	1.16	14	very high
17.7 W	8	81	48	55	1.47	12	very high

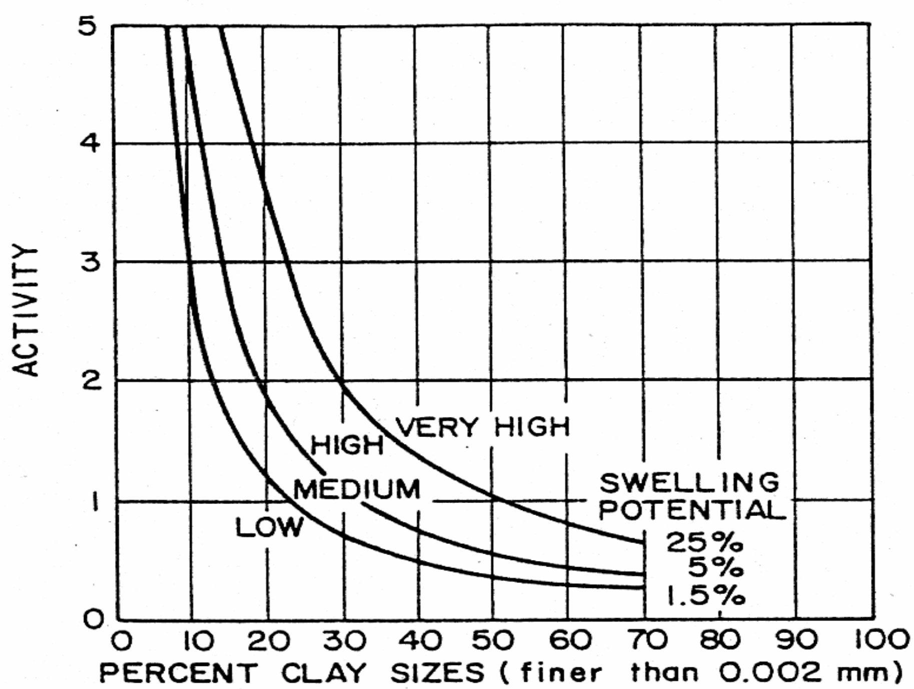

Figure 4.3 Expansiveness as a function of colloids content and Activity (Van der Merwe, 1964)

When colloids content is combined with a direct measure of positive charge deficiency the empirical relationships of Figures 4.2 and 4.3 may be improved. Cation exchange capacity (CEC) is a direct measure of the quantity of cations adsorbed on the water accessible surfaces of clay minerals required to neutralize the positive charge deficiency of the clay particles. CEC is expressed in milli-equivalents per 100 grams of dry soil. Figure 4.4 is a relationship developed by Holt (1963) in which a normalized cation exchange capacity, CEC_c , and the Activity are used to indicate the presence of expansive clay minerals without the need to perform the direct identification procedures mentioned above. Normalized cation exchange capacity, CEC_c , is the conventional cation exchange capacity in milli-equivalents per 100 grams divided by the colloids content in percent. Table 4.4 contains the cation exchange capacity and colloids contents measured for selected US95 Owyhee county soils of this study. The data show that the soils contain montmorillonite, which is consistent with the X-ray diffraction results.

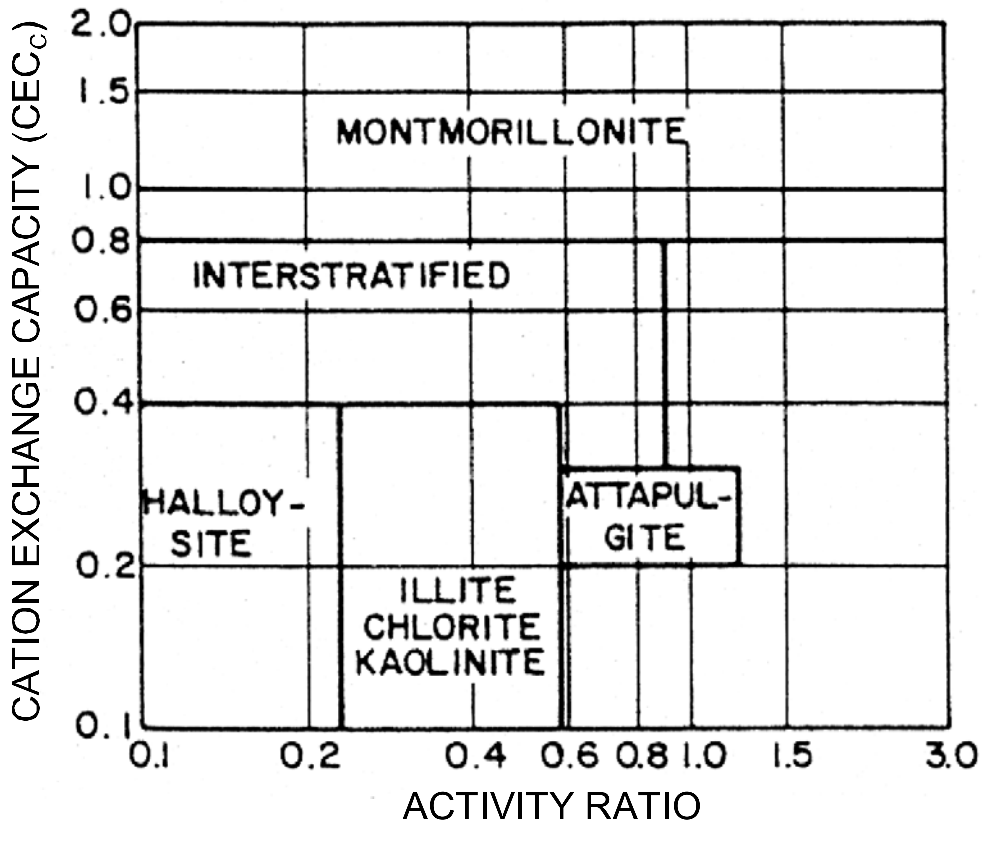

Figure 4.4 Clay mineralogy as a function of Activity and Cation Exchange Capacity (Holt, 1969)

McKeen and Hamberg (1981) extended the CEA_c-Activity relationships of Figure 4.4 to develop both an approximate method for estimating directly the qualitative swell potential of a soil based on the soil property known as coefficient of linear extensibility or COLE and a method for estimating the COLE value itself. COLE represents the change in dimensions due to shrinkage of a soil mass as it dries from a moist condition defined as the water content corresponding to a suction of 5 psi to the oven-dry condition. COLE is a kind of reverse swelling and is determined in a test involving finding the dry unit weight of the soil for the two specified conditions (Nelson and Miller, 1992). Once a COLE has been determined, the qualitative swell potential of the soil can be estimated from COLE and the colloids content using Figure 4.5. If the COLE values are not available, but activity and cation exchange capacity data are, Figure 4.6 can be used to identify the appropriate region of Figure 4.5 and thus provide estimates of both soil expansiveness and COLE. Table 4.4 contains estimated COLE values based on data from Table 4.3 along with the evaluation of the soil expansiveness according to these criteria.

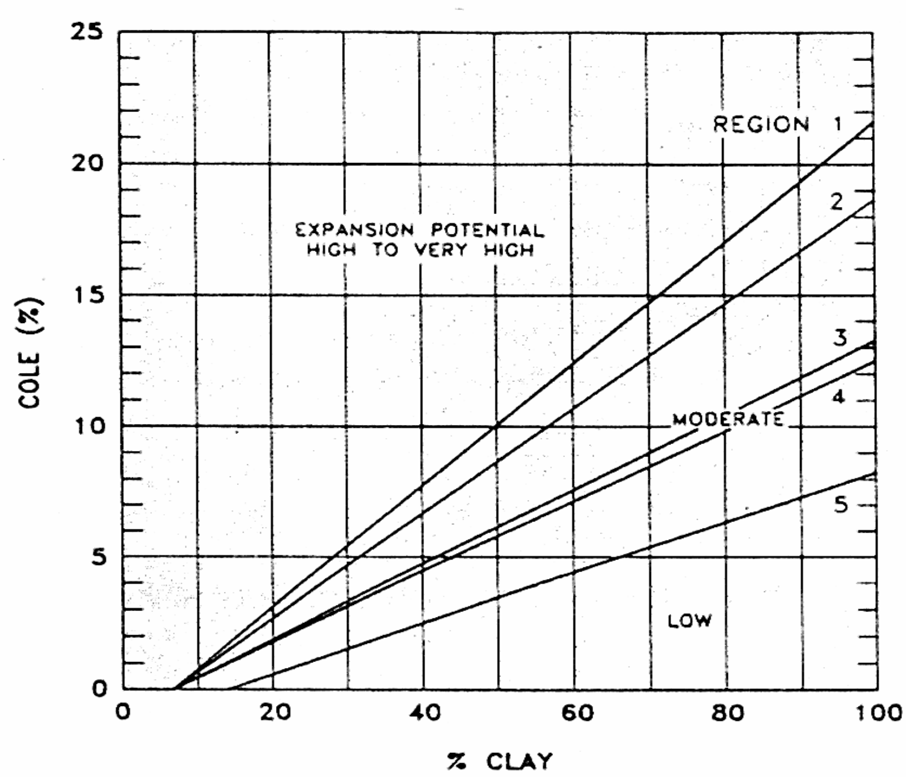

Figure 4.5 Swell potential as a function of colloids content and COLE (McKeen and Hamberg, 1981)

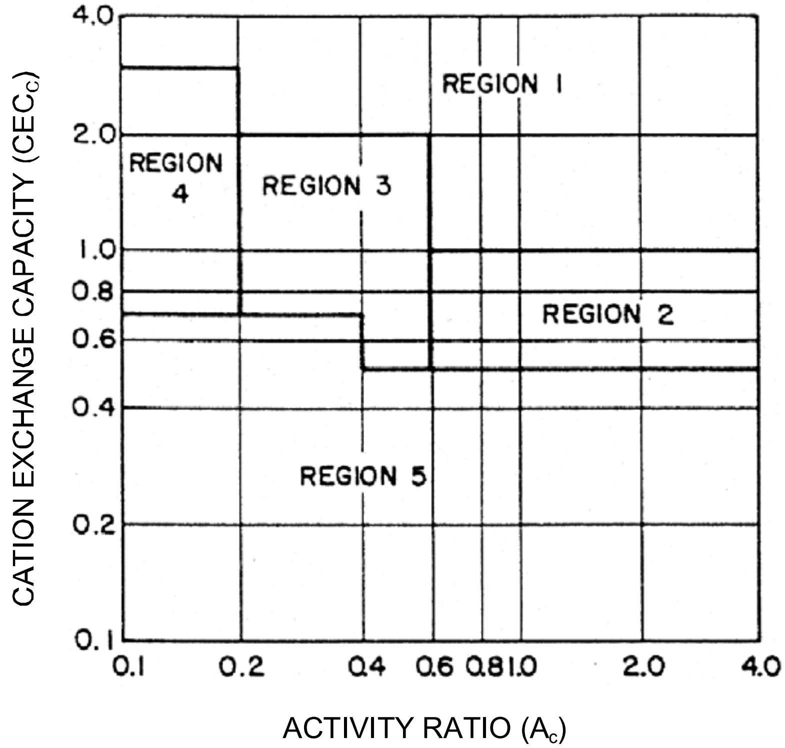

Figure 4.6 Soil expansiveness and COLE regions as a function of Activity and Cation Exchange Capacity (McKeen and Hamberg, 1981)

The final indirect method to be presented in this section is one developed by McKeen (1992, 2001) that incorporates the dimensionless slope of the soil suction-water content relationship of a soil, $\Delta h/\Delta w$, and the suction compression index, C_h. An example of the suction-water content relationship for a soil (also called a soil water characteristic curve or SWCC) is shown in Figure 4.7. The dimensionless slope of the SWCC is obtained when suction is expressed in units of pF, and water content is expressed in percent. The suction unit pF is the logarithm to the base 10 of the suction head, h, expressed in cm of water. The dimensionless suction compression index, C_h, is the ratio of the change in dimensionless vertical strain, ε_v, produced by drying or wetting a soil divided by the change in the suction, in units of pF, accompanying the change in water content. Figure 4.8 illustrates the definition of suction compression index for one-dimensional volume changes. Classification of soil expansiveness using these criteria is given in Table 4.5.

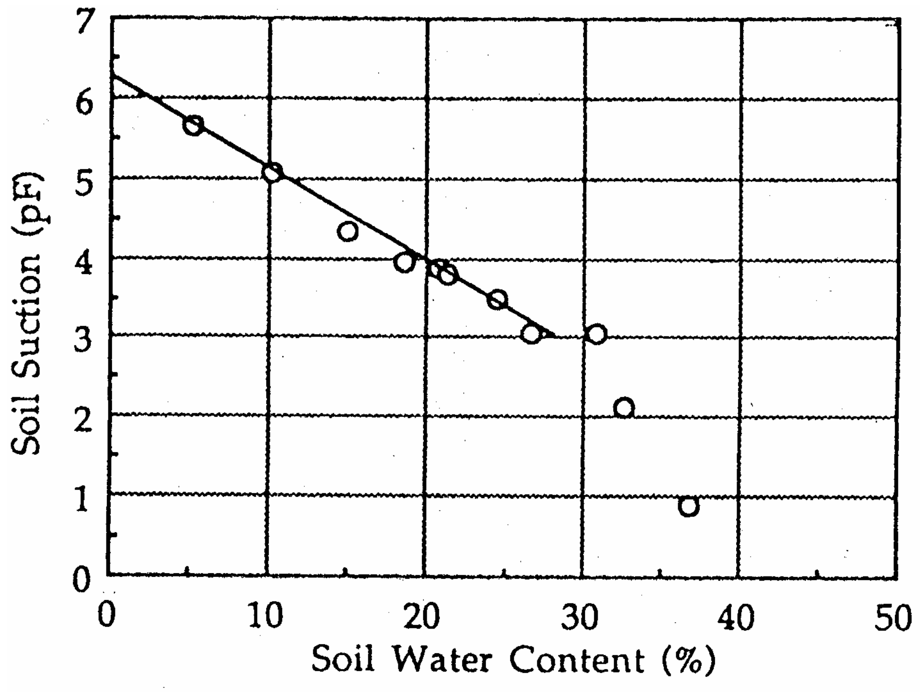

Figure 4.7 Example of the relationship between soil suction and water content (McKeen, 1992)

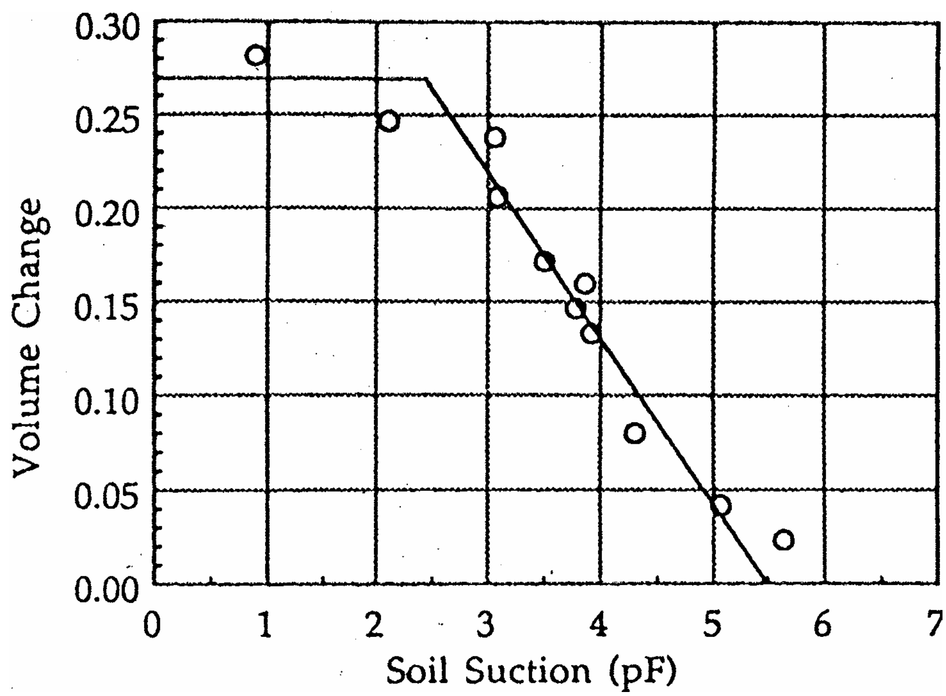

Figure 4.8 Example of the relationship between volume strain and soil suction (McKeen, 1992)

Table 4.5 Soil expansiveness classification based on soil properties related to soil suction (McKeen, 1992)

Category	$\Delta h/\Delta w$ (pF/%)	C_h ε_{vol}/pF	Expansiveness Classification
I	> -6	-0.227	Very High
II	-6 to -10	-0.227 to -0.120	High
III	-10 to -13	-0.120 to -0.040	Moderate
IV	-13 to -20	-0.040 to NE[1]	Low
V	< -20	[1]Non-Expansive	[1]Non-Expansive

The testing effort required to evaluate the expansiveness of a soil using these criteria goes well beyond measuring soil index properties, but as was the case for the approach described in the previous paragraphs, it is possible to use the criteria even if only index properties are available. For example, empirical relationships to estimate the soil water characteristic curves from plasticity and grain size data have been published in both the soil science and geotechnical literature. Figure 4.9, from Zapata et al. (2000), gives soil water characteristic curves of fine-grained soils as functions of plasticity index and percent fines. The curves in the figure could be used to estimate the slope of the SWCC, and Figure 4.10 could be used to estimate the suction compression index, Ch, as a function of the slope of the SWCC.

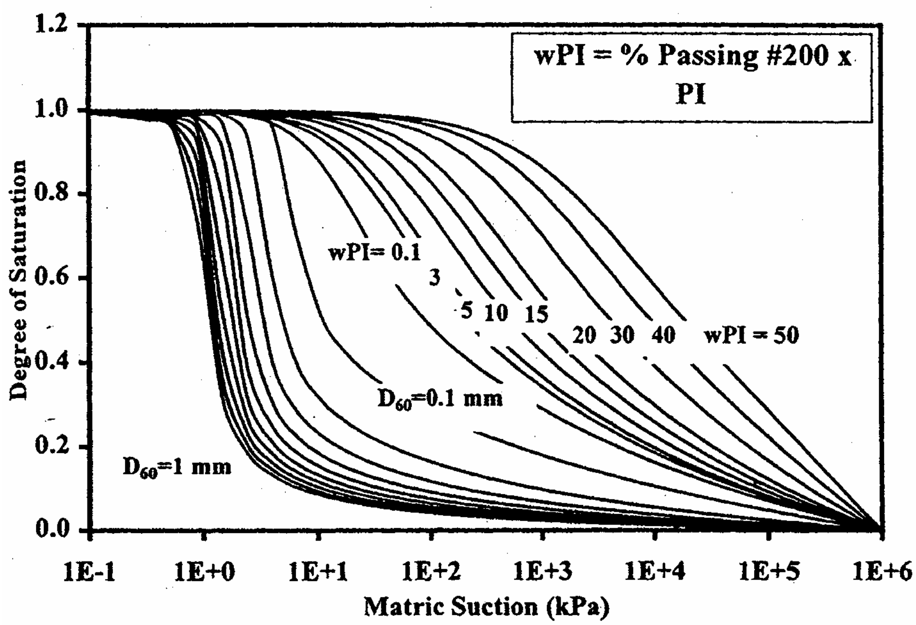

Figure 4.9 Relation of soil water characteristic curves, soil plasticity and percent fines (Zapata, et al., 2000)

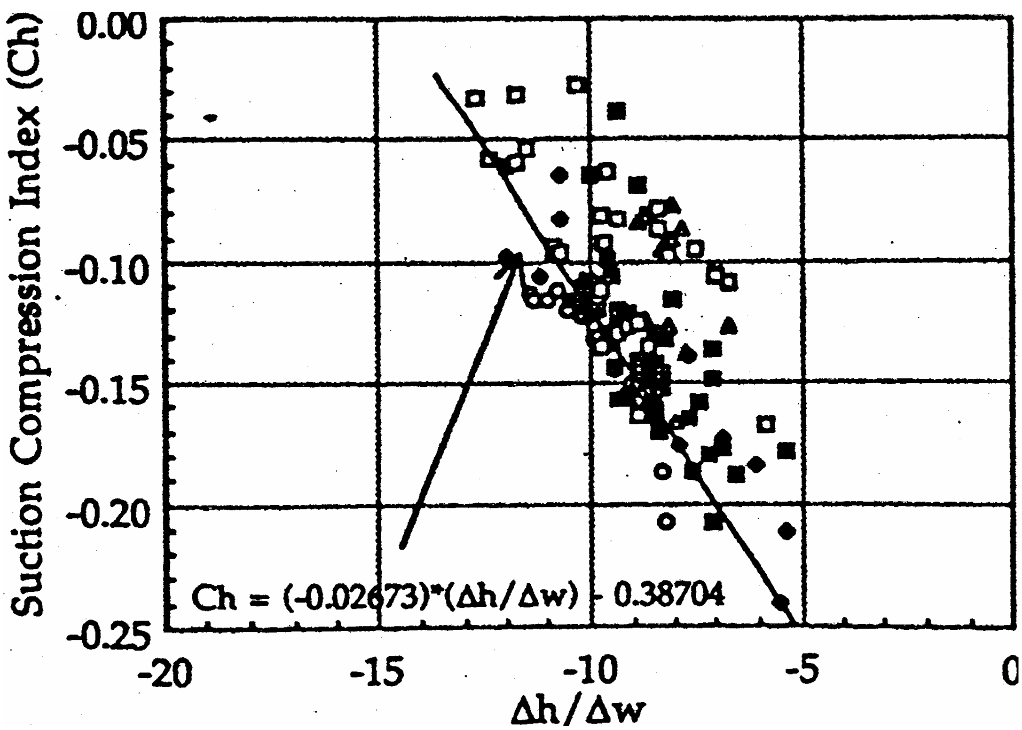

Figure 4.10 Relation of suction compression index, C_h, to the slope of the soil water characteristic curve (McKeen, 1992)

Qualitative Swelling Tests

Two test procedures involving actual soil swelling have been developed. The free swell test originated by Holtz and Gibbs (1956) consists of a simple comparison of the bulk volume of 10 mm^3 of air- dry minus 40 mesh and the bulk volume of the same quantity of air-dry soil after it is sedimented in 100 mm^3 of distilled water. Both volume measurements are made in a standard 100 mm^3 graduated cylinder. Free swell, in percent, is defined as the ratio of the wet bulk volume to the dry bulk volume. Holtz and Gibbs suggested that soils with free swells greater than 100 percent can cause considerable damage to lightly loaded structures. Soils with free swells in excess of 50 percent could present swell problems in the field. Free swells measured for the US95 soils are given in Table 4.6.

The second swelling test used to provide a qualitative indication of potential swelling problems is the American Society for Testing and Materials Standard Test Methods for Expansion Index of Soils D4829-88 (ASTM, 1988). The expansion index, EI, of a soil is 1000 times the difference between the initial and final heights of a compacted test specimen after inundation divided by the initial height. The initial height of the test specimen is about one inch. The soil is compacted into a rigid ring one inch high by four inches in diameter using 15 blows of 5.5 pound standard Proctor compaction hammer at an initial degree of saturation between 40 and 60 percent. The specimen is inundated under a 1.0 lb/in^2 vertical surcharge stress. These conditions are not designed to duplicate any actual field condition; they produce only the expansion index, EI, used to classify the potential expansion of the soil in the qualitative terms of Table 4.7.

Table 4.6 Free swell of US95 soils from Owyhee County

Mile Post	**Depth Feet**	**Free Swell, %**
1.2E	2.6	100
2.3E	0 5.6	200 270
6.8E	2.6	70
7.7E	2.5 10.0	30 160
9.1E	5.0	40
17.0E	5.0	100
17.6W	8.1	140
17.7W	8.0	100
17.7W	2.0	100
17.9E	5.3	0
18.4E	2.6	70
18.4E	7.0	90

Table 4.7 Expansion potential from Expansion Index (ASTM, 1988)

Expansion Index, EI	**Expansion Potential**
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
>130	Very high

One-Dimensional Swell Tests

The only general methods for evaluating the swelling behavior of expansive soils under conditions realistically representative of field conditions and the changes in soil conditions and stress states that are likely to occur in the field are the various one-dimensional loading-wetting tests performed in the oedometer. The American Society for Testing and Materials has adopted two one-dimensional test types for determining the response of soils to changes in water content. Standard Test Method D4546-90 One-Dimensional Swell or Settlement of Cohesive Soils is performed on either undisturbed or remolded soils using conventional oedometers in one of three methods. In Method A the test specimen is allowed to expand after being inundated under a 1.0 kPa vertical stress. The change in the specimen height expressed as a percentage of the original height is defined as the "free swell." After free swell is complete, the specimen is loaded incrementally as in the conventional one-dimensional compression test until the specimen height is equal to or smaller than its original height (usually 1.00 inch). The swell pressure is defined as the vertical stress required to bring the specimen back to its original thickness.

Method B is similar to Method A except that the initial vertical stress under which the specimen undergoes its free swell is selected by the operator. In Method C the test specimen is inundated, and increments of vertical stress are applied as required to prevent the specimen from changing its thickness after inundation. This procedure requires an adjustable oedometer. The swell pressure is the smallest vertical stress that has to be applied to prevent any swelling.

In addition to the standard procedures of ASTM, other generally similar one-dimensional swell test procedures and apparatus have been described in the literature. Among the more recent developments of the one-dimensional swell test techniques are those that incorporate high air entry value porous ceramic disks at the bottom of the test specimen in the standard consolidation cell (Mou and Chu, 1981; Schreiner and Burland, 1987). This apparatus modification permits soil swelling to be measured for any desired levels of initial and final water contents and suctions up to complete saturation. Water contents are changed by controlling the soil suction during the test. Such tests take substantially more time and the special apparatus requires special care to operate successfully. Methods A and B of ASTM D4546 were used in this study. Figure 4.11 shows results of a test performed using Method A for one of the US95 soils.

Table 4.8 summarizes results of one-dimensional swelling tests performed on the soils sampled in drill holes adjacent to US95 in Owyhee County. Test specimens were trimmed from the more or less "undisturbed" field samples, subjected to either 100 or 500 lbs/ft^2 vertical surcharge stress and then inundated, that is, the sample basin in the oedometer was filled with water. One-dimensional swell test results for compacted surface soils from two locations along the route of US95 are given in Table 4.9.

The test specimens were compacted to dry unit weights produced by the standard Proctor (AAHTO T-99) effort at water contents close to optimum. Swell data like that plotted in Figure 4.11 was recorded for both the "undisturbed" and compacted test specimens.

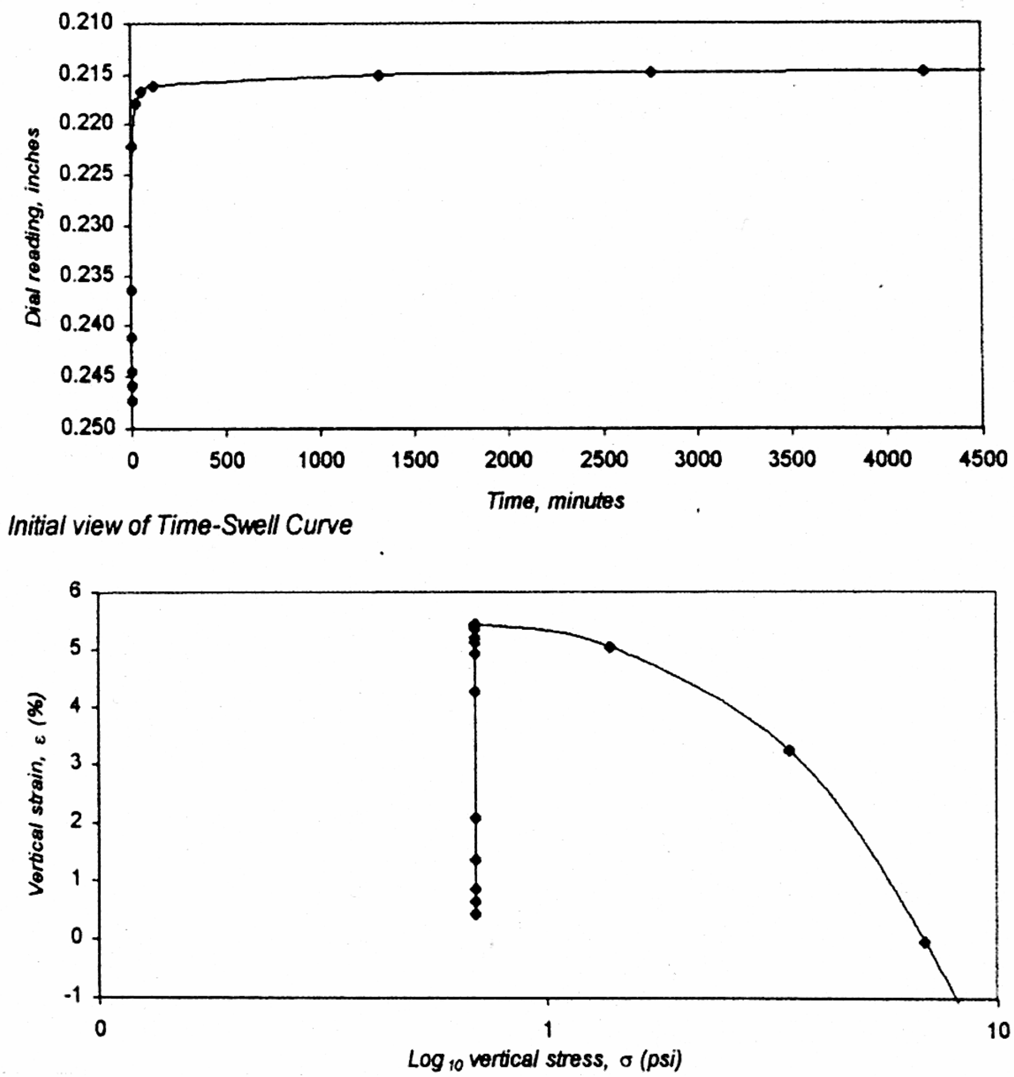

Figure 4.11 One-dimensional swell response of "undisturbed" sample, MP 17.0, 5ft depth, initial w = 46%, final w =81%, swell pressure = 0.5 tsf

Table 4.8 One-dimensional swell test results for “undisturbed” samples of US95 soils

Mile Post	Depth feet	Vertical pressure tons/ft^2	Initial water content percent	Initial dry unit weight lbs/ft^3	Swell pressure tons/ft^2	Swell ($\Delta H/H_o$) percent
1.2E	2.6	0.25	35	47	0.3	0.2
2.3E	4.8	0.25	60	52	0.3	0.0
2.3E	5.6	0.25	54	57	0.4	0.3
2.3E	6.0	0.05	48	67	0.2	1.0
2.3E	8.1	0.25	59	51	0.4	0.6
6.8E	2.6	0.25	91	33	0.3	0.0
7.7E	2.5	0.25	29	54	0.2	0.0
7.7E	10	0.25	69	41	0.2	0.0
17.0E	5.0	0.05	46	44	0.5	5.5
17.6W	8.1	0.05	28	67	0.4	5.1
17.6W	8.1	0.25	39	49	0.4	0.4
17.7W	2.0	0.25	16	81	0.25	0.0
17.7W	2.6	0.05	15	77	0.2	0.31
17.7W	8.0	0.25	32	73	0.8	1.8
18.35	7.0	0.25	41	82	0.25	0.0

Table 4.9 One-dimensional swell test results for compacted US95 soils

Mile Post	Initial water content percent	Final water content percent	Initial dry unit weight lbs/ft^3	Relative compaction percent	Swell pressure tons/ft^2	Swell ($\Delta H/H_o$) percent
2.3	32	70	70	99	6.0	31
2.3	39	47	52	73	4.0	18
17.0	30	48	54	72	6.0	23
17.0	46	87	33	46	3.0	9.1
17.5	23	35	77	100	1.8	10.6
17.5	42	71	41	55	0.4	1.1

Conclusions

All the empirical criteria based on soil index properties indicate that many of the soils along the current alignment of US95 in Owyhee County are very highly expansive. The most expansive soils are located at MP 2.3 and between MP 17.0 and MP 18.4. The one-dimensional swell test results summarized in the Table 4.8 for “undisturbed” test samples from the first 18.4 miles of US 95 lead to the conclusion that the swelling of the soils following inundation at their current natural water contents was either very small or nonexistent, a result which is contrary to what might be expected from the application of all the criteria based on soil index properties measured on remolded soils. On the other hand, the swelling of compacted specimens of the soils was more like what would be expected from the various swell classification schemes applied in the preceding paragraphs. The significance of these results is discussed in a later section of the report.

CHAPTER FIVE

TREATMENTS FOR EXPANSIVE SOILS

Introduction

Methods used to prevent or reduce swelling of pavement subgrade soils are presented in this chapter. The results of tests performed to evaluate the effectiveness of lime treatment of the expansive US95 soils are also provided. Although the term "lime treatment" has been used to describe the application to soils of both CaO (quicklime) and $Ca(OH)_2$ (hydrated or slaked-lime), only hydrated lime was used in this study.

Methods for treating pavement subgrade soils to prevent or control swelling have been grouped in several ways. The usual subdivisions used for treatments designed to prevent swelling are (1) the introduction of chemicals into the soil to reduce or eliminate its swell potential, (2) provision of hydraulic barriers to prevent the entry of water into expansive soils, (3) mechanical methods which provide physical restraint for the soil to keep it from swelling, and (4) compacting soil in a manner which reduces its suction, that is, its affinity for water. Another approach to categorizing treatment methods would be to group the methods into categories related to how they address the three requirements necessary to have soil expansion. These requirements were discussed in the introduction to Chapter Four and included the following: (1) the soil has to contain expanding lattice clays or other hydrateable minerals, (2) the soil has to exist in or be brought to a water deficient condition in terms of its water content and stress state, and (3) water has to become available to enter the soil. A combination of these approaches is used in the following sections.

Chemical Soil Treatments

Treatment methods directly addressing the presence of expansive clays in soils include the so-called chemical methods of soil stabilization. In this approach substances are added to the soil which effectively reduce the affinity of clay minerals for water or change the clay minerals into non-expanding lattice materials. The latter effect is achieved by either destroying the clay minerals through what has been termed artificial weathering or by reactions between additive and clay that produce cementitious materials at the expense of the clay. Changes in the chemical environment brought about by chemical additives also may make the clay require less water to satisfy its charge deficiency through the phenomenon of ion crowding, that is, by replacing higher volume hydrated monovalent cations with divalent cations and increasing the concentration of cations in the soil water (Petry and Armstrong, 2001). Added chemicals are conveniently divided into calcium-based and non-calcium based stabilizers.

Calcium-Based Stabilizers - Portland cement and the various forms of lime are examples of the calcium-based materials used to modify and stabilize clayey soils. *Soil modification* with lime refers to the immediate reduction in plasticity and water content that results from the flocculation and agglomeration of the clay particles produced by the depression of the adsorbed water films of the clay. Adsorbed water film thicknesses are reduced both as a result of Ca^{+2} cations replacing monovalent cations (Na+ and K^{+}) and the increase in the total electrolyte concentration of the soil water. Both these phenomena result in a decrease in the soil suction or water deficiency and the swelling potential. *Soil stabilization* with lime is due to pozzolanic reactions in which new crystalline cementitious materials are produced as the added lime raises the soil pH and thus the solubility of silica and alumina (Transportation Research Board, 1987).

Calcium-based stabilizers have been around for many years and have a well-documented record of improving workability and decreasing expansiveness in soils with expanding lattice clay minerals. Lime treatment offers the advantage of fairly simple and reliable test methods to determine the required treatment level achieve the required amount of lime modification and swelling reduction (Eades and Grim, 1966). The major disadvantage of all calcium-based stabilizers that increase the pH of the treated soil is the possible reaction of lime with soluble sulfates. This so-called "sulfate reaction" reduces the amount of lime available to participate in the stabilizing pozzolanic reactions, and even worse, the lime can react with the sulfates to produce the highly expansive hydrated minerals ettringite and thaumasite (Mitchell, 1986)

The beneficial effects of hydrated lime treatment for the US95 soils in Owyhee County are illustrated in Table 5.1. The test results summarized in the table show that the $Ca(OH)_2$ is effective in reducing the plasticity as well as the swell pressure and volume changes in the soils. The results also suggest that while the pH results indicate that a treatment level of nine percent is adequate, the swell data indicate that a treatment in excess of this may be required if all the soil swelling is to be eliminated.

Residual swelling exhibited by the inundated laboratory test specimens containing nine percent lime and the presence of sulfates in the soils suggested that the sulfate reaction that produces the expansive minerals ettringite and thaumasite might be occurring in the soils at the higher $Ca(OH)_2$ treatment levels.

Table 5.2 shows soluble sulfate levels measured for untreated soils in the project area. Table 5.3 shows similar results for the lime treated compacted surface soil samples from Mileposts 2.3 and 17.0.

Table 5.1 Effects of $Ca(OH)_2$ treatment on US95 soils

Mile Post	$Ca(OH)_2$ Percent	PH	Liquid Limit Percent	Plastic Index Percent	Compaction Water Content percent	Dry unit Weight lb/ft^3	Swell Pressure tons/ft^2	Swell percent
2.3	0	7.7	192	140	32	71	6.0	31.4
	3	9.3	145	112	34	70	2.0	15.8
	6	11	103	55	34	67	0.9	2.2
	9	11	99	44	34	69	0.6	0.8
17.0	0	8	91	50	28	75	6.0	22.5
	3	9.2	102	45	34	72	6.0	18.6
	6	11	80	32	32	72	1.2	4.3
	9	11	80	27	33	67	0.4	1.3

Table 5.2 Sulfate levels in natural, untreated US95 soils

Mile post	Depth feet	SO_4-S microg/g (ppm)
1.2	2.6	130
2.3	0	430
2.3	0.5	1,400
2.3	0	1,500
2.3	5	510
9.1	5	25
12.5	0	6
17.0	0	140
17.0	5	150
17.0	8	710
17.7	8	440

Table 5.3 Compositional analysis of lime treated compacted US95 soils from Mileposts 2.3 and 17.0

Mile Post	Applied $Ca(OH)_2$ percent	pH	Cation Exchange Capacity meq/100g	Exchangeable Cations, meq/100g				SO_4-S ppm	$CaCO_3$ per-cent
				Ca^{+2}	Mg^{+2}	Na^{+}	K^{+}		
2.3	0	7.7	56	26	8	23	3	1400	3
	3	9.3	54	34	2	22	3	2200	6
	6	11	46	35	2	15	3	210	7
	9	11	55	36	3	20	3	1200	12
17.0	0	8	52	27	13	5	2	140	4
	3	9.2	54	33	4	5	2	69	7
	6	11	50	38	7	9	3	77	9
	9	11	64	34	5	5	2	66	12

In order for the sulfate reaction to occur in soils, at least ten percent clay and a pH greater than 10.5 are required (Hunter, 1988). The minimum amount of soluble sulfate required has been estimated as ranging from 2,000 ppm (Petry, 1992; Perrin, 1992) to as low as 700 ppm (Hunter, 1988). Puppala, et al., (1999) detected ettringite in soils having as little as 320 ppm soluble sulfates. Unlike clay swelling which can take place as the colloids condense water from its vapor in the soil atmosphere, the sulfate reaction requires that there be sufficient liquid water in the soil to maintain the sulfates in solution. The reaction is believed to occur in as little as a "few months " after lime treatment (Hunter, 1988), and can continue to occur for years as long as the soil pH remains above 10.4 and sulfates remain in solution (Puppala, et al., 1999).

To evaluate the possibility of the sulfate reaction in US95 soils, five samples of the $Ca(OH)_2$ treated material were examined by X-ray diffraction after they were subjected to innundation for a period of approximately one year. The results of the X-ray diffraction analysis of the treated soils from Mileposts 2.3 and 17.0 are given in Table 5.4. The presence of ettringite or thaumasite was not observed in the treated soils (Post, 2002). The soluble sulfate levels or the lime levels used in the US95 soils are apparently not sufficient to produce the sulfate reaction. In any case, due to the extremely dry climate in which the soils exist, it's not likely that there is sufficient water to maintain the sulfates in solution long enough for the reaction to occur in the field.

Table 5.4 X-ray diffraction analysis of lime treated soils from US95 after soaking and aging

Mile Post	$Ca(OH)_2$ treatment percent	Constituents				
2.3	0	Smectite	Orthoclase	Andesine		Trace[1]
	9	Orthoclase	Andesine	Smectite	Mg-Calcite	Trace[2]
	9	Smectite	Andesine	Orthoclase	Calcite	
17.0	6	Orthoclase	Andesine	Calcite	Smectite	Clinoptilolite
	6	Orthoclase	Andesine	Clinoptilolite	Smectite	Mg-Calcite

[1]Trace constituents are Calcite and Jarosite

[2]Trace constituents are Talc and Gypsum

The use of lime to stabilize expansive clay soils is considered to be most effective when dry CaO or $Ca(OH)_2$ or slurried $Ca(OH)_2$ can be mixed directly with the soil along with an appropriate amount of water, allowed to cure, followed by mixing with additional water and possibly lime for compaction, and finally, compacted to dry unit weights close to the T-99 maximum. There are several variations on this approach (Transportation Research Board, 1987). The technique of injecting lime slurries under pressure directly into soils beneath the surface has had mixed success. Problems are usually attributed to incomplete penetration of homogeneous, intact clays (Pengelly, et al., 1997).

Non-Calcium Based Chemical Stabilizers - A variety of chemical additives which have demonstrated ability to improve the engineering properties and workability of soils and soil-aggregate mixtures have been developed. More recently, partly in response to the increasing recognition of the sulfate problem described above, some of the old and some new chemical admixtures have been put forth as being effective alternatives for the calcium-based treatments of expansive clay soils. Many of the non-calcium based stabilizers are proprietary, and the mechanisms of their stabilizing action are not clearly understood. Some are believed to act in like calcium-based stabilizers in that they break down the clay minerals (weathering) thereby reducing the amount of clay available to expand. Other suggested mechanisms include replacement of adsorbed monovalent cations (ion exchange), which lowers the water deficiency (suction) in the soil in a manner similar to the role of the Ca^{+2} in lime treatment. Table 5.5 is a partial list of non-calcium based chemical stabilizers purported to be effective in reducing soil swelling.

Table 5.5 Non-calcium based chemical soil stabilizers

Stabilizing Chemical or Trade Name	Manufacturer or Supplier	Reference or Source[1]	Remarks
Ammonium and Potassium Lignosulfates	Hayward-Baker Fort Worth, TX	Pengelly et al., 1997	Injectable
Barium Chloride Barium Hydroxide	Various	Ferris et al. 1991	Expensive, for high SO_4 clays
BIO-CAT (bioenzyme)	Soil Stabilization Products, Inc. Merced, CA	Scholer, 1992	
CBR-PLUS	CBR-PLUS North America, Inc. Vancouver, B.C.	Company brochure	
Condor SS (sulfonated naphthalene)	Earth Science Products Corp. Wilsonville, OR	Scholer, 1992	Not effective with Smectite
Consolid-444 (ammonium chloride)	American Consolid, Inc. Davenport, IA	Scholer, 1992	No contact
EcSS 3000 (sulfonated oil)	Environmental Soil Stabilization, L.L.C. Arlington, TX	Company brochure	Injectable, high SO_4 clays
EMC Squared (bioenzyme)	Soil Stabilization Products, Inc. Merced, CA	Company Brochure	Proprietary, for high SO_4 clays
HIExC	Environmental Soil Stabilization, L.L.C. Arlington, TX	Sarkar et al., 2000	Injection in high SO_4 clays
Perma-Zyme (enzyme)	International Enzymes @perma-zyme.com	Scholer, 1992	
PSCS-320 (bioenzyme)	Alpha Omega Enterprises	Scholer, 1992	
Road Bond EN1 Sulfonated D-Limonene	C.S.S. Technology, Inc. Weatherford, TX 76086	Katz et al., 2001	

[1]Citations of references are contained in Appendix One.

The review of the available literature for non-calcium based stabilizers revealed that many of the non-calcium based chemicals seem to be most effective in treating soils, aggregates and soil-aggregate mixtures that contain only small amounts of clay of low plasticity. The applications have been mainly to surface courses and bases for low volume roads. Beneficial effects (stabilization) in these applications refer to increases in strength and cohesiveness, improvements in workability, and decreases in compressibility of the treated materials. With the exception of EcSS-3000, EMC-SQUARED, HIExC, and Roadbond EN-1, there is no experience, nor claim for the use of most of the proprietary chemicals specifically to reduce swelling of highly plastic clay soils. As indicated in Table 5.5, the advantages claimed for the first three products relates to their ability to stabilize expansive clay soils containing high levels of sulfates.

Some additional perceived limitations of the use of the non-calcium based stabilizers include the as yet relatively small number of well-documented field trials and demonstrations and difficulty in determining appropriate treatment levels. Currently there are no simple, rapid, widely recognized test methods to determine appropriate treatment levels. The materials may not be available everywhere, and costs may be high at the treatment levels required for very highly plastic, expansive clays.

Hydraulic Barriers

Hydraulic barriers are used to prevent the entry of both the liquid and vapor forms of water into expansive soils. Barriers may be designed to completely encapsulate the expansive soil or they may be intended to act as cutoffs for either horizontal or vertical flows.

Encapsulation - This treatment provides for the complete isolation of expansive soils from all water sources. Encapsulating barriers have been constructed using both asphaltic materials, and more recently, with geomembranes. Encapsulation can only be used with new pavement construction.

Horizontal Membranes - Single waterproof layers of asphalts and geomembranes have been installed both above and below compacted and natural subgrades for the purpose of intercepting liquid water and preventing its entry into expansive soils. Horizontal barriers installed above the expansive soil layer are designed to prevent water present in base courses from being drawn into the expansive soil, which has a much higher affinity for water (suction) than typical aggregate base materials. Membranes placed below the potentially expansive subgrade during new construction or rehabilitation are designed to prevent water from being transported by capillary rise into the subgrade.

The horizontal barrier approach is obviously not available for remediation of existing expansive soil subgrades. Furthermore, when membranes are placed on top of compacted or natural subgrades in semiarid climates like that of Owyhee County, it's virtually impossible to prevent an increase in the water content of the potentially expansive subgrades. This situation is due to the fact that water in its vapor phase will continue to move into the potentially expansive soil as a result of changes in temperature and evaotranspiration regime brought about by the membrane and the overlying pavement. Research at Fort Collins, Colorado, has shown that while horizontal membranes can be effective in reducing the entry of surface water, their major benefits in arid and semiarid climates are to increase the time required for water content increases and swelling to occur. They also tend to make the water content increase and thus the swelling more uniform when it does occur (Nelson and Miller, 1992).

Deep Vertical Moisture Barriers (DVMB) - A typical DVMB is shown in Figure 5.1 (Snethen, 1979). Installing the hydraulic barriers in a vertical orientation at the margins of the pavement permits their use both in new construction and as a remedial technique for existing pavements. DVMB's have been constructed mainly with Portland cement concrete and geomembranes. More recently, geocomposites and highly plastic clays have been used. For the reasons mentioned above for horizontal membranes, DVMB's have been shown to be effective in reducing but not entirely eliminating soil expansion beneath pavements. DVMB's also delay the swelling and tend to make it more uniform (Nelson and Miller, 1992; Steinberg, 2000).

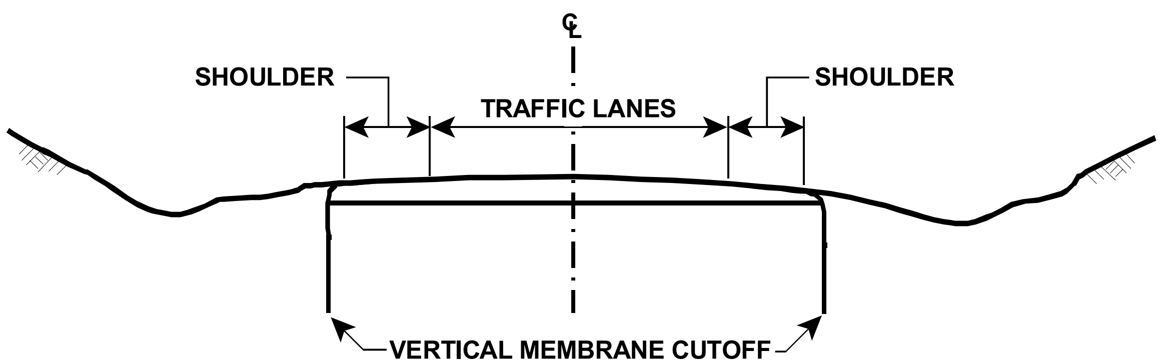

Figure 5.1 Deep vertical moisture barrier, DVMB (Snethen, 1979)

Mechanical Treatments

The three types of treatments described as mechanical are surcharge stresses, fiber reinforcement and Geogrids.

Surcharge Stress - This reliable technique involves placing a sufficient quantity of non-expanding material (ballast) over an expansive soil so that the vertical effective stress on the expansive soil equals or exceeds its swell pressure. If the surcharge material has a dry unit weight of 125 lbs/ft^3, each foot of thickness provides a surcharge stress of 125 lbs/ft^2 or 0.063 tons/ft^2. It's clear that unless high fills are required for highway grade considerations, the surcharge method is likely to be cost-effective only for expansive soils with low swelling pressures.

The surcharge material itself can be a stabilized or encapsulated expansive soil or more commonly, a non-expansive borrow material. In some applications part of the expansive soil is removed prior to placement of the surcharge material. The successful application of this approach requires that laboratory swell tests be performed to determine the swell pressure as the soil as goes from its initial moisture condition to the final stable moisture condition. This treatment method can only be used in conjunction with new construction of the pavement section.

Fiber Reinforcement - The use of randomly oriented polypropylene fibers 25 to 50 mm in length for stabilizing expansive soils has been investigated by Puppala and Musenda (2000). In tests involving 0.3, 0.6 and 0.9 percent fibers by dry weight of compacted soil, the inclusion of the fibers increased the unconfined compressive strength over that of the untreated high plasticity CH soils. The strains required to mobilize the increased strengths were larger than for the untreated soils. While the fibers slightly reduced the shrinkage of soil pastes, the conventional swelling measured in oedometer tests increased when the soils were treated with fibers. The increased free swell was attributed to a more uniform distribution of moisture in the compacted samples caused by the moisture paths created by the fibers. The use of 0.9 percent fiber reduced the swell pressures in the soil with a liquid limit of 82 percent from 0.26 tons/ft^2 to 0.22 tons/ft^2. In the slightly less plastic soil (liquid limit of 73 percent), the swell pressure was reduced from 0.39 tons/ft^2 to 0.22 tons/ft^2. The investigators concluded that "the mechanisms causing the swell pressure reductions still need to be evaluated."

The use of fibers for reducing swelling doesn't address any of the three required conditions for swelling. Fibers don't change the swelling clay minerals in any way; fibers don't reduce the water deficiency condition of the soil from what it is in its compacted condition and fibers don't make water unavailable to the potentially expansive soil.

Geogrids - Like fibers, geogrids can provide mechanical reinforcement of soft, compressible clay subgrades. Geogrids increase the strength and bearing capacity of the soil and can therefore be considered to be a soil stabilizer. Geogrids are promoted by their manufacturer as being an alternative to lime stabilization for heavy clays containing sulfates (Tensar Earth Technologies, Inc., 2001), but it is difficult to see how they can prevent swelling of soils containing highly expansive clay minerals. Like fibers, geogrids don't alter the structure of expanding lattice clay minerals in soil; geogrids can't reduce the water deficiency if the soil's mineraology, condition and environment create a water deficiency; and geogrids cannot prevent the entry if water becomes available to a soil possessing the first two requirements. It does seem likely that like fibers, geogrids may reduce swell pressures by providing some restraint in the form of tensile reinforcement. By providing paths for the entry of moisture, geogrids may also promote more uniform soil swelling when used with uniform soils.

Miscellaneous Treatments

Methods categorized as miscellaneous in this report include control of compaction variables, replacement fill, prewetting, and sub-drainage. None of these methods is believed to have application to the swelling soils problem of US95 in Owyhee County.

Control of Compaction Variables - Compacting subgrade soils to low dry unit weights on the wet side of optimum water content using kneading methods has been found to be an effective means to reduce swelling for moderately plastic soils (Seed, Mitchell and Chan, 1960). Compaction of clays under conditions that produce high shear strains and locally oriented clay particle arrangements has been demonstrated to result in less water deficiency and therefore, reduced swelling. The limitation on this approach is that the lowered dry unit weights and higher water contents are also accompanied by lower subgrade strength and stiffness and an increased tendency for soil shrinkage to occur if the soils are subjected to drying.

Replacement Fill - This technique refers to the process whereby expansive subgrade soil is removed and replaced with a non-expansive material. Replacement materials have included non-expansive borrow as well as expansive soils treated with lime or other swell-prevention chemicals. Depending on the thickness of the expansive soil in the subgrade, it may be replaced entirely, or it may be removed only to the depth at which the weight of the non-expansive material is equal to the swell pressure of the expansive subgrade soil. According to Chen (1988), the Federal Highway Administration has recommended that the required thickness of the replacement material be estimated from the expansive soil's plasticity using the relationships of Table 5.6. Based on these plasticity criteria, the most highly plastic Owyhee County soils would require about six feet of replacement fill. A fill thickness of six feet provides a surcharge stress of less than 0.5 tons/ft^2. This thickness may be adequate for the undisturbed US95 soils but clearly it's inadequate for the remolded soils.

Pre-Wetting - Experience with the techniques of ponding, sprinkling or injecting large amounts of water into natural subgrade soils prior to pavement construction for the purpose of reducing post-construction swelling has been reviewed by Chen (1988) and Nelson and Miller, (1992). Pre-wetting programs are designed to provide water to satisfy the clay's water deficiency. However, the reductions in soil swelling are usually accompanied by weakening, softening and decreases in the workability of the soil. Depending on the permeability of the treated soil, the pre-wetting process may require months or even years to be completed.

Table 5.6 Relation of required surcharge fill thickness to soil plasticity (Chen, 1988)

Plasticity Index, percent	**Thickness of Undercut and Replacement Fill, feet**	
	Interstate Highways	**Secondary and State Highways**
10-20	2	2
20-30	3	2
30-40	4	3
40-50	5	3
>50	6	4

Sub-Drainage - The use of sub-surface drain tile or perforated pipe wrapped in fabrics or graded granular filters below the edges of pavements has also been reviewed by Chen (1988) and Nelson and Miller (1992). Properly designed and constructed side drains parallel to the pavement alignment can be effective in intercepting and rapidly removing subsurface water flowing at atmospheric pressures through materials of low suction potential. Sub-drains will not have any effect on water moving into expansive soils in the vapor phase or water moving in unsaturated soil in response to gradients in soil suction. In fact, sub-surface drains may enhance the opportunity for atmospheric water to enter the subgrade by exposing more surface area of expansive high suction subgrade soils to water at atmospheric pressure. Good surface drainage and the prevention of ponding is more likely to be effective than sub-surface drains placed in highly impermeable, highly plastic soils. This is particularly true in a semiarid climate like that of Owyhee County.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

Introduction

As stated previously, three conditions must exist in order for a clayey soil to expand and heave when subjected to increases in water content under light surcharge stresses such as those created by pavements. First, the soil must contain clay minerals that increase volume as a result of the entry of water into expandable crystal lattices. Second, the soil must exist in or be brought to a condition in which it has a water deficiency in its existing stress state. In other words, the soil must exist at a water content at which its suction is sufficiently large so that it takes in water and expands against the stresses acting on it if the water becomes available. The third condition that causes the potential expansion to actually occur is that due to some change in the soil environment, the water to enter the soil (and reduce its suction) becomes available. This chapter summarizes the principal findings of the research and presents recommendations for dealing with the expansive soils within the context of the three required conditions. The recommendations include specific measures to minimize and delay the continued heaving of the existing pavement on the first 18.5 miles of US95. Recommendations for subgrade treatments in new pavement construction in the area also provided.

Conclusions

Soils sampled at several locations along the first 18.5 miles of US95 in Owyhee County have been found to satisfy all the diagnostic criteria of soils having extremely high swelling potential. Tests performed on remolded samples show that many of the soils have liquid limits in excess of 150 percent, shrinkage limits less than 18 percent, cation exchange capacities greater than 50 cmol/kg, Activity ratios greater than 1.5 and free swells greater than 100 percent. X-ray diffraction analyses show that some of the soils contain as much as 60 percent of the expanding lattice clay mineral montmorillonite. One-dimensional swell tests performed on specimens compacted to 100 percent of the AASHTO T-99 maximum dry unit weights of about 75 lbs/ft^3 near their optimum water content of about 35 percent swelled as much as 30 percent under small overburden stresses. Swell pressures measured for the compacted soils were as large as 6 tons/ft^2. Therefore, the swelling potential exhibited by the Owyhee soils can be described as extreme in comparison to most of the materials described in the literature. If the soils are used as compacted subgrade beneath pavements without specific treatments to reduce the expansion potential or measures to keep water from entering them, pavement distress due to heave can be expected to occur eventually.

The annual precipitation in Owyhee County averages only 11 inches and much of it occurs as snow. In addition to infiltration of rainfall and runoff into exposed soils, changes in the evapotranspiration regime produced by covering the soils invariably results in increases in compacted subgrade water contents in semiarid climates like that of Owyhee County. Swelling due to decreases in evapotranspiration is slow to occur and may require several years to produce an equilibrium condition. In view of the fact that much of the precipitation in Owyhee County occurs as snow and may be sublimated back into the atmosphere, it's possible that a substantial portion of the US95 soil swelling may be due to the decrease in evapotranspiration brought about by paving.

In contrast to the swelling behavior shown by the compacted soils, more or less intact samples taken from below the loose incoherent surface colluvial materials did not swell or develop large swell pressures in laboratory one-dimensional tests. This behavior is believed to be due the unusual origin of the Owyhee County soils. Many of the well-recognized and widely distributed expansive soils in the United states originated from shales formed from weathered basic igneous rocks and weathered air-fall tuffs deposited in seawater. The expansive soils of Owyhee County are believed to have developed from subaerial hydrothermal alterations and devitrification of welded to slightly welded ash flow tuffs (ignimbrites). The retention of bonds from the initial welding and post-weathering cementation in the undisturbed, intact material could explain the absence of significant swelling in the more or less intact subsurface samples. Other possible reasons for the reduced swelling of the highly colloidal more or less intact materials include their typically low unit weights and high initial natural water contents as compared to the water contents and dry unit weights produced by compacting the soils.

Two points concerning the locations of the existing pavement distress attributed to heaving are noted. First, it should be recognized that not all the soils encountered along the existing US95 alignment are expansive and not all the pavement on the study alignment appears to be undergoing damage from swelling soils. Second, swelling related distressed areas appear to be mainly at transitions between cut and fill sections (grade points) as at Milepost 2.3, and at locations where the pavement is close to the natural ground surface in relatively flat areas underlain by incoherent colluvial materials.

The first point may seem obvious in view of the fact that at grade points the fill side is always lower in elevation than the cut and, therefore, receives concentrated runoff from the cut. More significant, however, is the high probability that in spite of a specification that the fill be constructed of imported non-expanding borrow, some remolded potentially expansive soil from the adjacent cut inevitably gets incorporated into the compacted fill at the transition. The higher initial suction of the compacted expansive soil and its increased exposure to surface water at the grade point could be the cause of the pavement distress typically observed at these locations.

In the relatively flat area between approximately Mileposts 17.0 and 18.4, the pavement section appears to have been constructed mainly on low, compacted fills placed on incoherent colluvial materials. Natural water contents of the near surface colluvium from borings made outside the pavement section at Mileposts 17.6 and 17.7 shown in Figure 3.2 are similar to Proctor compaction optimums. Compaction of these highly plastic materials at these water contents puts them in a potentially highly expansive condition. It's now clear from the swell pressures measured for the compacted material that the 1.25 ft thickness of lime-treated subgrade could not provide sufficient vertical surcharge to prevent swelling of the underlying untreated compacted colluvium.

The only soil modifiers that have been conclusively demonstrated to reduce the plasticity and swelling of very highly plastic soils containing substantial amounts of smectite are the calcium-based stabilizers Portland cement and lime. The soil modifier used tested in this study was hydrated lime, $Ca(OH)_2$ which proved to be generally effective at treatment levels of about six to nine percent. ASTM test method D6276-1999a, Standard Test Method for Using pH to Estimate the Soil-Lime Requirement for Soil Stabilization, should be used to determine treatment levels.

Although compositional analysis showed that the soils contain up to 2,200 ppm soluble sulfate, the development of the expansive hydrated mineral ettringite was not detected more than one year after lime treatment. Given the climate and topography of the current US95 alignment, it's highly unlikely that the sulfates present in the soil would be found in solution.

Recommendations

Recommendations dealing with two areas are given in this section. Recommendations for minimizing and delaying future heaves of the existing pavements are given first followed by recommendations for new or rehabilitated pavements. Finally, suggestions for further research are given.

Existing Pavements - Two alternatives from the treatments discussed in Chapter Five should be considered for reducing future heaving of the existing pavements. The recommended approaches represent attempts to prevent the entry of atmospheric (liquid) water into the soils and to create conditions where the water content increases due to changes in evapotranspiration will tend to be more uniform and slower in developing. The first alternative consists of the creation of continuous horizontal membrane from the pavement surface to the bottom of ditch utilizing shoulder and ditch paving in cut sections. Shoulder and ditch paving will reduce exposure of the subgrade to infiltration of surface water during and shortly after precipitation events and will also make it easier to inspect and maintain the ditches to prevent ponding of surface water.

The use of subsurface drains is not recommended. An alternative to paving the ditches in cut sections is to install deep vertical moisture barriers (DVMB) at the outside edges of paved shoulders. A continuous membrane should be formed by the paved shoulder and the DVMB.

In fill sections, shoulder paving should be extended to the top of fill slopes. Aggressive toe of slope drainage is also recommended.

The use of injected chemicals to address the first condition required for soil expansion noted in the introduction is not recommended for either existing pavement or new construction for two reasons. First, with the exception of the calcium-based stabilizers, the effectiveness of available injectable chemicals for modifying very high plasticity materials like the Owyhee soils has not been demonstrated. Second, in consideration of the very low permeability of the highly plastic materials, the likelihood of achieving a uniform treatment level of the compacted subgrade and any underlying colluvial material even with calcium-based stabilizers is believed to be small. Uneven treatment of the expansive soils could very well exacerbate the differential subgrade swelling problem by increasing the hydraulic conductivity of localized zones in the material, thereby providing improved access for infiltrating surface water to the remaining unmodified material. Finally, because the actual thickness of the existing layers of compacted subgrade or colluvial material overlying the intact undisturbed subgrade soil or rock is not likely to be uniform or known, a very conservative, costly injection depth would be required.

New Construction - Once the decision is made to remove the existing structural section and expose the subgrade down to natural colluvium or intact material, additional alternatives to those mentioned above become available. The first alternative recommended for cut sections is to remove all the existing compacted subgrade along with any obviously incoherent colluvial subgrade material beneath the traveled way down to the surface in the intact, coherent subgrade soil (or rock). Based on the results of the boring made outside the existing pavements, the maximum depth to intact soil or rock is believed not to exceed five ft. Intact, coherent subgrade material or rock should also be exposed at least from a point beneath the edge of the traveled way out to the centerline of the drainage ditch.

All backfill used to establish the pavement grade preferably should be non-expansive fine-grained (preferably clayey) borrow of low hydraulic conductivity. Alternatively, the backfill may be lime-stabilized expansive soil treated to the level indicated by ASTM Standard Test Method D6276 referred to above. The required treatment level with hydrated lime will be as high as nine percent. The use of non-expansive clay borrow is preferred because lime treatment of the expansive clay borrow will substantially increase its hydraulic conductivity. The objective of a low hydraulic conductivity material is to create an impermeable but non-expansive subgrade fill prism which will prevent the movement of any liquid water onto the surface of the underlying natural soil or rock..

Base and ballast materials used in new construction over expansive soils should be well-graded materials with non-plastic fines having hydraulic conductivities less than 10^{-6} cm/s. If such materials are not available and conventional free-draining base and ballast materials are used, an impervious asphalt or geosynthetic membrane should be placed on the surface of the subgrade prior to placing any base or ballast. Shoulders and ditches should be paved as described above in the recommendations for existing pavements. An alternative to paving the ditches in cut sections is to install 8 ft deep vertical moisture barriers located at the outside edge of paved shoulders. A continuous membrane should be formed by the paved shoulder and the DVMB.

Non-expansive or lime treated soils should be used to construct fills. Fill areas should be stripped of all obviously incoherent colluvial material or down to a depth such that the total thickness of the fill prism is at least twenty ft. Shoulder paving or a horizontal membrane should be extended to the top of fill slopes. Aggressive toe of slope drainage is also recommended.

Future Research - Efforts should be made to involve manufacturers and suppliers of proprietary non-calcium based stabilizers in a test programs to evaluate the effectiveness of their products for reducing the expansiveness of highly plastic soils like those of Owyhee County. These cooperative research efforts could begin with laboratory programs. Field trials will no doubt be expensive, and due to the time required for possible swelling to develop, they will require extended periods of time to fully evaluate their outcomes.

Research to confirm and fully examine the reasons for the apparent absence of swelling in the highly plastic intact soils (ash-flow tuff) and the sulfate reaction is recommended.

APPENDIX ONE

REFERENCES

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APPENDIX TWO

GLOSSARY OF TERMS

Actual swell -percent, the ratio of the increase in total volume divided to the original total volume exhibited by a soil as it increases its original water content to some higher water content (where it still may have some unsatisfied water deficiency).

Coefficient of linear extensibility (COLE) -a measure of the change in dimensions due to shrinkage of a soil clod as it dries from the moist condition at the water content corresponding to a suction of 5 psi (33 kPa) to the oven dry condition. COLE = $[(\gamma_{d\text{ Oven dry}})/(\gamma_{d\text{ Moist}})]^{1/3} - 1$

Expansion index- dimensionless (ASTM D4829-88) 1000 times the difference between the initial and final height of a 1 inch thick compacted specimen after inundation divided by the initial height an "index" property like for example plasticity index. Has no relation to any filed condition, used only for comaprisons. (also see swell potential)

Expansive clay mineral -a clay mineral of the expanding lattice type that can change its volume by taking available water into the crystal lattice through direct adsorption or hydration of adsorbed cations.

Expansive soil -a soil that under some condition is capable of increasing its volume when wetted, normally, a soil containing expansive clay minerals

Free swell -percent, ratio of the final apparent total volume of to the initial apparent total volume of 10 cm^3 of air-dry minus 40 mesh soil after it has been poured into 100 cm^3 of distilled water contained in a 100 cm^3 graduated cylinder. Percent free swell = $(V_{wet} - V_{air\ dry})/V_{air\ dry})$

Hydrated lime -calcium hydroxide, $Ca(OH)_2$, also known as slaked lime.

Lime modification -the immediate reduction in plasticity and water content that occurs from the flocculation and agglomeration of the clay particles produced by the of depression of the adsorbed water films of the clay. Adsorbed water film thicknesses are reduced both as a result of Ca^{+2} cations replacing monovalent cations (Na+ and K^+) and the increase in the total electrolyte concentration of the soil water. These phenomena result in a decrease in the soil suction or water deficiency and the expansiveness of the soil.

Lime stabilization -increase in strength and stiffness due to the pozzolanic reactions in which new crystalline cementitious materials are produced as added lime raises the soil pH and thus the solubility of silica and alumina (Transportation Research Board, 1987).

Lime content -dry weight hydrated lime per total dry weight of solids expressed as percent.

One-dimensional swell -percent, the ratio of the change in the vertical dimension of a laterally confined test specimen accompanying inundation.

Quicklime -Calcium oxide, CaO, quicklime.

Swell-percent, the ratio of the change in the vertical dimension to the original vertical dimension of a laterally confined soil as its water content increases, the vertical strain during swelling, $\Delta H/H_o$

Swell pressure -the vertical stress that must be applied to an expansive soil to prevent it from swelling.

Swell potential -percent, the ratio of the change in the vertical dimension to the original dimension of a laterally confined soil as it takes in water starting at some specified initial condition and stress state and ending at a condition where its water deficiency is completely satisfied (soil water potential equal to zero). In order for a soil to have a swell potential, it must have a water deficiency (suction) in its initial specified condition. The initial condition is defined in terms of dry unit weight, particle structure, water content, and stress state (vertical stress).

Swelling index -slope of the void ratio versus the log of the effective vertical stress on the rebound curve of a one-dimensional compression test

Swell-susceptible soil -a soil containing sufficient expansive clay or other minerals which exists in a condition and stress state such that if water becomes available to it, it will increase its volume

Surcharge pressure -the vertical stress applied to the test specimen during the measurement of one-dimensional swell.

APPENDIX THREE

ADDITIONAL TEST RESULTS

Table A3.1 Compositional analysis of untreated US95 soils

Mile Post	Depth feet	Cation Exchange Capacity meq/100g	Exchangeable Cations, meq/100g				SO_4-S ppm	$CaCO_3$ per-cent
			Ca^{+2}	Mg^{+2}	Na^{+}	K^{+}		
1.2E	2.6	49	37	7	6	3	130	3.0
2.3E	0.0	82	49	9	24	4	1500	4.1
2.3E	5.0	86	49	9	23	4	510	5.7
9.1E	5.0	51	29	9	6	3	25	2.7
12.5E	0.0	10	6	2	0.2	1	6	2.0
17.0E	8.0	64	89	4	2	2	710	3.1
17.0W	0.0	66	49	15	5	2	150	5.1
17.7W	8.0	48	65	2	8	1	440	6.5

Table A3.2 Specific gravities and colloids content of US95 soils

Mile Post	Depth feet	LAB	Specific gravity	Colloids content percent
0.6	0.0	BSU	2.64	
2.3E	0.0	BSU	2.72	75
2.3E	0.0	UI	2.79	72
2.3E	2.5	BSU	2.85	
7.7E	0.0	UI		
9.1E	5.0	UI	2.65	75
12.8	0.0	BSU	2.66	
17.0W	0.0	UI	2.80	73
17.5W	0.0	UI	2.76	
17.7W	0.0	BSU	2.59	
18.4E	2.6	UI	2.76	

APPENDIX FOUR

ROAD LOG

Mile Post	Observation
0.0	Turn around in flat; pavement good
0.5	Small cut, grade point; pavement rough
0.9	MP 1 sign
1.1	Big cut in brown and gray bedded material; bumps
1.9	MP 2 sign
2.2	Grade point to cut both sides, brown, purple fat clays, gypsum crystals; bump at grade Point
2.7	Large fill; good
2.9	MP 3 sign
3.1	Grade point to cut at 3.1; bumps
3.5	Heavily eroded (rilled) high cut on east side in bedded brown and gray materials; bumps
3.7	Bumps
3.9	Flat on grade to MP 4.4
4.5	Cut, rough
5.0	Grade starts uphill, culvert
5.1	Cut in white material
5.3	Cut in red, rusty colored material
5.6	Bumps
5.9	Small cut to grade point, then fill to MP 6.3; bumps
6.9	Cut in brown material
7.0	High fill; good
7.5	Fill; good
7.8	Cut in yellow-white material; bump at grade point
8.3	Fills and flats; good
8.8	Cut, pavement in poor condition
9.0	Patched

Mile Post	Observation
10.0	Fill; excellent
10.5	Half-cut in red rock
10.8	Cut in brown sediments; rough
11.3	Good
11.8	Half cut in yellow sandy indurated material, cobbles, red rhyolite; good
11.9	Fill; good
12.1	Cut in sediments, breccia flows; good
13.0	Cut in bedded materials; good
13.5	Cut in colorful mixed strata; good
13.6	High fill; good
13.8	Cut in colorful rocks; good
14.2	Viewpoint
14.4	Cut in red, brown rhyolites; good
14.8	End of full cut, half-cut continuing on west side to 15.0
15.1	Cuts in reddish breccia, red rhyolites
15.6	High half cut on east in yellow rhyolite; good
15.9	Full cut; good
16.3	High, long fill; good
16.7	Sign "Bumps next one mile," half cut on west side, flow rocks; bumps
17.0	Half cut on west side, purple rhyolite above brown soil, fault; rough
17.3	Half cut on west in brown soil; bumps
17.5	At grade; rough
18.0	End of bumps
18.4	POE building

APPENDIX FIVE

BORING LOGS

Boring I.D. AH-2 **M.P.** 17.7 **Location** ~50 ft west of CL **Date** 3/20/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group	Names Class	Remarks
0 ft			
	Tube 1. (0.5-2.5)		
1	1-ft layer dark olive gray		
	crumbly appearance		
2	light olive gray		w_n = 21.7% @ 2'
	Tube 2. (2.5-5.0) crumbly smooth		2.5' -- w = 17.4 %
3			
4	white crystal deposits		
5	Tube 3. (5.0-7.5)		
	dispersed deposition of white particles (from this depth to 8-ft)		
6	olive		
	fractures		
7			
	Tube 4. (7.5-10.0)		
8			Swell test @ 8' w = 32.2%
	fractures		
9		red-brn-orange deposition	
		" "	
10	Tube 5. (10.0-12.5)		w_n = 42.1% @ 10'
		" "	
11			
	disking - ½"	" "	
12			
	Tube 6. (12.5-15.0)	" "	
13			
		" "	
14			
		" "	
15	Tube 7. (15.0-17.5)	platy smooth	15' -- w = 40.8 %
		" "	
16			
		" "	
17			
	Tube 8. (17.5-20.0)		w_n = 45.5% @ 17.5'
18			
	disking - ½" and crumbly soil (evidence of soil compression)		
19			
20	Tube 9. (20.0-22.5)		

Boring I.D. AH-2 **M.P.** 17.7 **Location** ~50 ft west of CL **Date** 3/20/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	olive	
21	disking - ½"	
	~2" layer of white-yellow material	
22		
	Tube 10. (22.5-25.0)	
23		
24	rusty layer	
25	borehole depth	w_n = 46.8% @ 25'
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40 ft		

Boring I.D. AH-3 **M.P.** 17.0 **Location** ~35 ft east of CL **Date** 3/21/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0-2.5)	
1		
	~2.5' aggregate base	
2		
	Tube 2. (2.5-5.0)	
3	dark brown moist	
	light olive brown w/ white lenses (small veins)	
4	molting	
	loose in tube (breaks up) highly fractured hard clay	
5	Tube 3. (5.0-7.5)	w_n = 66.3% @ 5'
	recovery ~ 70 % in this sample tube	Swell test @ 5' w = 68.7%
6		
7		
	Tube 4. (7.5-10.0)	w_n = 69.2% @ 7.5'
8		
9		
10	Tube 5. (10.0-12.5)	w_n = 73.5% @ 10'
	tan 1" lense	
11	tan 1" lense	
	fractures	
12		
	Tube 6. (12.5-15.0)	w_n = 70.9% @ 12.5'
13		
14		
15	Tube 7. (15.0-17.5)	w_n = 61.5% @ 15'
	4" tan layer	
16	1" white crystals	
	white streaks	
17		
	Tube 8. (17.5-20.0)	w_n = 67.8% @ 17.5'
18	white crystals	
19		
20		

Boring I.D. AH-3 **M.P.** 17.0 **Location** ~35 ft east of CL **Date** 3/21/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	disked - 3/4"	
21		
22		
	sample smaller than tube, i.e. loose in the tube	w_n = 55.9% @ 22.5'
23		
24		
25		
26		
27		
	disked - 3/4"	
28		
	occasional rust streaks	
29		
30	Sampled depth	
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		

Boring I.D. AH-4 **M.P.** 9.1 **Location** ~35 ft east of CL **Date** 3/22/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0.6-2.6)	
1	loose in tube (breaks up)	
	highly fractured, light olive brown	
2		
	Tube 2. (2.6-5.1)	w_n = 73.6% @ 2.7'
3		
	fractures	
4		
		w_n = 79.5% @ 5.2'
5	Tube 3. (5.1-7.6)	all -40 S at 5.2' easy grind,
		black coating on some dry partings
6		Swell test @ 5' w = 71.3%
	fractures	
7		
	Tube 4. (7.6-10.1)	w_n = 78.1% @ 7.7'
8	~1" layer light yellowish	
	brown, sandy appearance	
9		
10	Tube 5. (10.1-12.6)	
		w_n = 77.6% @ 10.2'
11		
	light yellowish brown lense	
12		
	Tube 6. (12.6-15.1)	
13		
14		
	light yellowish brown lense	
15	Tube 7. (15.1-17.6)	
	light yellowish brown lense	
16		
	~4" layer olive brown w/ olive yellow mixed in	
17		
	Tube 8. (17.6-20.1)	w_n = 72.5% @ 17.2'
18		
	~6" layer olive brown w/ some olive yellow lenses	
19		
20		

Boring I.D. AH-4 **M.P.** 9.1 **Location** ~35 ft east of CL **Date** 3/22/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	Tube 9. (20.1-22.6)	w_n = 81.4% @ 20.2'
21		
	bad core sample "broken up"	
22		
	Tube 10. (22.6-25.1)	w_n = 73.6% @ 22.7'
23		
24		
25		
		w_n = 56.8% @ 25.7'
26		
	Sample depth	w_n = 76.8% @ 26.2'
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40 ft		

Boring I.D. AH-5 **M.P.** 2.3 **Location** ~20 ft east of CL **Date** 3/23/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ________ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** Yes

Depth	Moisture Color Group Names Class	Remarks
0 ft		
1	No core	
2		
3	Tube 1. (3.1-5.6)	looks “wet” (very moist)
	Brown-orange, moist stiff clay	3.1' -- w = 61.3 %
4	olive brown	Swell test @ 4.8' w = 59.3%
		" " w = 7.7% “AD”
5	brown	" " @ 5.6' w = 54.3%
	Tube 2. (5.6-8.1)	" " @5' w = 55.2%
6	olive brown, moist	" " @5.6' w = 66.7 %
	cracking	w_n = 59.4% @ 5.6'
7		
	red streaks, wet at 8'	
8	Tube 3. (8.1-10.6)	Swell test @ 8.1' w = 58.5%
	olive brown, moist	" " w = 8.3% “AD”
9	cracking	w_n = 67.0% @ 8.1'
10		
	Tube 4. (10.6-13.1)	
11	orange streaks	dispersed white crystal clumping
12		
13	Tube 5. (13.1-15.6)	
		w_n = 77.0% @ 13.1'
14	3" dark brown layer	
		Atterberg Limit test
15		[1]brown, shiny, smooth, slight
	Tube 6. (15.6-18.1)	grit sound
16		[2]olive, shiny, smooth
17	cracking	
18	Tube 7. (18.1-20.6)	18.1' -- w = 57.7%
	cracking	w_n = 61.7% @ 18.1'
19		
	dark brown	
20		

Boring I.D. AH-5 **M.P.** 2.3 **Location** ~20 ft east of CL **Date** 3/23/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** Yes

Depth	Moisture Color Group	Names Class	Remarks
20 ft			
	Tube 8. (20.6-23.1)		
21	dark brown		
	tan, greenish, sandy		
22			
23	Tube 9. (23.1-25.6)		w_n = 77.8% @ 23.1'
24			
25			
	Tube 10. (25.6-28.1)		
26			hole is filling up with water
	dark yellowish brown layer	sandy material	possibly due to migration from
27	light olive brown layer	" "	hillside-sandy layer (lense)
	dark yellowish brown layer	" "	
28	Sample depth		w_n = 36.8% @ 28.1'
29			
30			
31			
32			
33			
34			
35			
36			
37			
38			
39			
40			

Boring I.D. AH-6 **M.P.** 17.7 **Location** ~60 ft west of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0.6-2.6)	
1	dark brown-organic material	
	grayish brown to pale brown	Swell test @ 2.0' -- w = 15.1%
2		Swell test @ 2.6' w = 15.5%[1]
	Tube 2. (2.6-5.1)	w_n = 16.2% @ 2.6'
3	dispersed white particles	3.0' -- w = 14.0%[2]
	throughout tubes 2 and 3	
4		
5	Tube 3. (5.1-7.6)	w_n = 25.9% @ 5.1'
	gray brown	
6		
	vertical fractures	
7		
	Tube 4. (7.6-10.1)	w_n = 36.9% @ 7.6'
8		
	dispersed rust spots	
9	fractures	
	soil compression is evident	
10	Tube 5. (10.1-12.6)	w_n = 39.9% @ 10.1'
	light brownish gray	
11	w/ layering of light gray	
	and brown	
12	fractures	
	Tube 6. (12.6-15.1)	w_n = 41.1% @ 12.6'
13		
	grayish brown	Atterberg Limit test
14		[1]sounds and feels gritty, does
	fractures	take water well
15	Tube 7. (15.1-17.6)	
		Moisture content test
16		[2]crumbly (solid)
	fractures	
17		
	Tube 8. (17.6-20.1)	w_n = 43.9% @ 17.6'
18		
	compression damage to 22.6'	
19		
	fractures	
20		

Boring I.D. AH-6 **M.P.** 17.7 **Location** ~60 ft west of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** No

Depth	Moisture Color Group Names Class	Remarks
20 ft		
21	Tube 9. (20.1-22.6) grayish brown fractures	
22	Tube 10. (22.6-25.1)	wn = 44.1% @ 22.6'
23	fractures	
24		
25	Sample depth	w_n = 45.5% @ 25.1'
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		

Boring I.D. AH-7 **M.P.** 17.6 **Location** ~50 ft west of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0.6-3.1)	
1	light gray	
2		
3	Tube 2. (3.1-8.1)	3.1' -- w = 28.4%
	soil compression	
4	pale olive	
	white particle deposits	
5	throughout this tube	
6		
7		
8	Tube 3. (8.1-10.6)	Swell test @ 8.1' w = 39.3%[1]
	fractures	
9	some vertical fractures	
10		
	Tube 4. (10.6-13.1)	
11	~1/2" layer-white to yellow	
	~1/2" layer -- “rust” color	Atterberg Limit test
12		[1]smooth, shiny and wet
	“rust” lense -- fractures w/ vertical fractures	
13	Tube 5. (13.1-15.6)	
	dispersed white particles	
14	fractures	
15	“rust” lense	
	Tube 6. (15.6-18.1)	
16	fractures	
	“rust” lense	
17	“zebra striping” – compaction	
18	Tube 7. (18.1-20.6)	
	~1" layer-dark olive	
19		
	fractures	
20		

Boring I.D. AH-7 **M.P.** 17.6 **Location** ~50 ft west of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth Moisture Color Group Names Class Remarks

20 ft

Tube 8. (20.6-23.1)

21

fractures

22

23 Tube 9. (23.1-25.6)

fractures

24

dark olive

25

Tube 10. (25.6-28.1)

26

fractures

27

28 Sample depth

29

30

31

32

33

34

35

36

37

38

39

40

Boring I.D. AH-8 **M.P.** 17.93 **Location** ~45 ft west of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0-2.6)	
1	dark grayish brown	
	light gray	
2	grayish brown	
	Tube 2. (2.6-5.1)	
3	light olive brown	2.8' -- w = 14.1%[1]
	fractures	
4		
5	Tube 3. (5.1-7.6)	
	~1' layer-broken up	Swell test @ 5.3' -- w = 19.8%[3]
6	fractures w/ some vertical	
	fracturing	
7		
	Tube 4. (7.6-10.1)	
8	appears that larger particles	
	leaving air voids	
9	white crystals	
10	Tube 5. (10.1-12.6)	
	white crystals	
11	pale olive	
	some fractures	
12		
	Tube 6. (12.6-15.1)	12.8' -- w = 62.8%[2]
13	white crystals	
14		
	white crystals	
15	Tube 7. (15.1-17.6)	
	~5" - sample missing	Moisture content test
16		[1]crumbly, smooth
		[2] crumbly, smooth
17	white crystals	
	Tube 8. (17.6-20.1)	
18		
	"rust" lense w/ white crystals	Atterberg Limit test
19	some fracturing	[3]sounds gritty
20	Tube 9. (20.1-22.6)	

Boring I.D. AH-8 **M.P.** 17.93 **Location** ~45 ft west of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth	Moisture Color Group	Names Class	Remarks
20 ft			
	~5" region of disking		
21			
22			
	Tube 10. (22.6-25.1)		
23			
	white crystals		
24			
25	Tube 11. (25.1-27.6)		
	~4" - sample missing		
26			
27	fractures		
	Sample depth		
28			
29			
30			
31			
32			
33			
34			
35			
36			
37			
38			
39			
40			

Boring I.D. AH-9 **M.P.** 18.35 **Location** ~35 ft east of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth	**Moisture Color Group Names Class**	**Remarks**
0 ft		
_	Tube 1. (0-2.6)	
1	grayish brown	
_	vertical fractures	
2		
_	Tube 2. (2.6-5.1)	Swell test @ 2.6' -- w = 15.7%[1]
3	white crystals	3.0' -- w = 18.1%[2]
_	sample “broken up”	
4		
_	light olive brown particles	
5	Tube 3. (5.1-7.6)	
_	light olive brown	
6		
_	~3" layer-dark gray	Atterberg Limit test
7	white crystals	[1]feels and sounds gritty
_	Tube 4. (7.6-10.1)	
8	~1.5' of damaged sample	
_	retrieved as spiral shape with	Moisture content test
9	air void-mat’l appears clayey	[2]crumbly, smooth
_	~1' layer-larger particles w/ white crystals	
10	Tube 5. (10.1-12.6)	
_	the sample contained in this	
11	tube appears multi-colored	
_	from yellows to reds -- clay	
12		
_	Tube 6. (12.6-15.1)	
13	light gray	
_	~1' layer-white to black	
14	deposition	
_	light olive brown w/ dispersed deposition of the above layer	
15	Tube 7. (15.1-16.8) "	
_	"	
16	"	
_	Refusal -- 16.8' "	
17		
_		
18		
_		
19		
_		
20		

Boring I.D. AH-9 **M.P.** 18.35 **Location** ~35 ft east of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture	Color	Group	Names	Class	Remarks
20 ft						
21						
22						
23						
24						
25						
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						
38						
39						
40						

Boring I.D. AH-10 **M.P.** 18.35 **Location** ~22 ft east of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0-2)	
1	~1' no sample retrieved	
	pale yellow-white crystals	
2	crumbly material	
	from 2' to 4.5' no sample	
3	retrieved	
4		
	Tube 2. (4.5-7)	4.5' -- w = 12.3%[1]
5	pale olive	
6	white-black-brown crystals	
	in an ~5" layer-olive yellow	
7	Tube 3. (7-9.5)	Swell test @ 7' -- w = 41.3%[2]
	white crystals	
8	pale olive	
9		
	Tube 4. (9.5-12)	
10		
	olive gray	Moisture content test
11		[1]crumbly, smooth
	small fractures	
12	Tube 5. (12-14.5)	
	small fractures	Atterberg Limit test
13	"	[2]slight grit sound but smooth
	"	
14	"	
	Tube 6. (14.5-17)	
15	small fractures	
	"	
16	"	
	"	
17	Tube 7. (17-19.5)	
	olive	
18		
19		
	Tube 8. (19.5-22)	
20		

Boring I.D. AH-10 **M.P.** 18.35 **Location** ~22 ft east of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	pale olive	
21	some disking	
	some vertical fractures	
22	Tube 9. (22-24.5)	
	light olive gray	
23	thin layering of black mat’l	
	layers wander to vertical	
24		
	Tube 10. (24.5-27)	
25	pale yellow	
	fractures -- “broken up”	
26		
27	Sample depth	
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		

Boring I.D. AH-11 **M.P.** 1.2 **Location** ~35 ft east of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group	Names Class	Remarks
0 ft			
	Tube 1. (0-2.6)		
1	~6" layer grainy light		
	yellowish brown		
2	very dark gray-crumbly		
	Tube 2. (2.6-5.1)		Swell test @ 2.6' - w = 34.9%[1]
3			3.0' -- w = 37.2%
4	~1" layer-white & org/brn		
	fractures		
5	Tube 3. (5.1-7.6)		
	fractures		
6			
			Atterberg Limit test
7			[1]no grit-smooth-shiny-mat'l
	Tube 4. (7.6-10.1)		holds on to water, hard to wet
8			all soil
	fractures		
9			
10	Tube 5. (10.1-12.6)		
	fractures w/ vertical fractures		
11	dark gray-crumbly		
	~1" layer-gray		
12			
	Tube 6. (12.6-15.1)		
13	~1" layer white to pale yellow		
	~12" layer-very dark gray		
14	fractures		
15	Tube 7. (15.1-17.6)		
	olive gray		
16			
	~5" layer-yellowish red		
17			
	Tube 8. (17.6-20.1)		17.6' -- w = 43.3%
18	olive		
19			
	pale olive		
20	Tube 9. (20.1-22.6)		

Boring I.D. AH-11 **M.P.** 1.2 **Location** ~35 ft east of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ________ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ________

Depth Moisture Color Group Names Class Remarks

20 ft

21 ~4" layer-pale yellow

22 white particles
Tube 10. (22.6-25.1)

23

24
olive

25 Tube 11. (25.1-27.6)

26
gray

27
Sample depth

28

29

30

31

32

33

34

35

36

37

38

39

40

Boring I.D. AH-12 **M.P.** 6.8 **Location** ~30 ft east of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0-2.6)	
1	~1' air void w/ organic mat'l	
	light yellowish brown	
2		
	Tube 2. (2.6-5.1)	Swell test @ 2.6' -- w = 90.5%[1]
3		3.0' -- w = 88.4%[2]
4		
	some vert/horiz fractures	
5	Tube 3. (5.1-7.6)	
	light olive brown	
6		
	some vert/horiz fractures	
7		
	Tube 4. (7.6-10.1)	
8		
		Atterberg Limit test
9		[1]smooth and slippery
10	Tube 5. (10.1-12.6)	
		Moisture content test
11		[2]smooth and platy
12		
	Tube 6. (12.6-15.1)	
13	olive gray	
	light olive gray	
14	~6" layer-gray	
	white/black/red-brn particles	
15	Tube 7. (15.1-17.6)	
	white crystals	
16		
	~10" layer-light yellowish brown	
17	gray-white crystals	
	Tube 8. (17.6-20.1)	
18	grayish brown	
19	gray	
	thin multi layering of dark gray, red-brn, light brownish gray	
20	Tube 9. (20.1-22.6)	

Boring I.D. AH-12 **M.P.** 6.8 **Location** ~30 ft east of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	disking-grayish brown	
21		
	vertical fractures	
22	sample "broken up"	
	Tube 10. (22.6-25.1)	
23	olive gray	
24		
	black particles	
25	Tube 11. (25.1-27.6)	
	light gray olive	
26		
27		
	Sample depth	
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		

Boring I.D. AH-13 **M.P.** 7.7 **Location** ~30 ft east of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water**

Depth	Moisture Color Group Names Class	Remarks
0 ft		
1	Tube 1. (0-2.6) olive gray-crumbly-moist	
2	olive yellow Tube 2. (2.6-4.9)	2.4' -- w = 62.5%[1]
3		
4	olive yellow white crystals	
5	Tube 3. (4.9-5.6) appears to be compressed	
6	Sample depth-rejected	
7		Moisture content test
8		[1]gritty
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		

Boring I.D. AH-13 **M.P.** 7.7 **Location** ~30 ft east of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	**Moisture**	**Color**	**Group**	**Names**	**Class**	**Remarks**
20 ft						
21						
22						
23						
24						
25						
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						
38						
39						
40						

Boring I.D. AH-14 **M.P.** 7.7 **Location** ~30 ft west of CL **Date** 3/24/2000 **Sheet** 1 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ______ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ______

Depth	Moisture Color Group Names Class	Remarks
0 ft		
	Tube 1. (0-2.6)	
1	light olive brown-broken up	
	black	
2		
	Tube 2. (2.6-5.1)	Swell test @ 2.5' -- w = 29.3%[1]
3		3' -- w = 34.1%[3]
4	fracture	
5	Tube 3. (5.1-7.6)	
6		
7		
	Tube 4. (7.6-10.1)	
8		
9		
	olive yellow	
10	Tube 5. (10.1-12.6)	Swell test @ 10' -- w = 69.0%[2]
	black	
11		
12		
	Tube 6. (12.6-15.1)	12.5' -- w = 58.9%[4]
13		
14		
		Atterberg Limits
15	Tube 7. (15.1-17.6)	[1]sounds gritty
		[2]mat'l slides off test device
16		
		Moisture content tests
17	fractures	[3]gritty
	Tube 8. (17.6-20.1)	[4]gritty
18	light olive brown	[5]gritty
	black	
19		
	olive brown	
20	Tube 9. (20.1-22.6)	

Boring I.D. AH-14 **M.P.** 7.7 **Location** ~30 ft west of CL **Date** 3/24/2000 **Sheet** 2 of 2

Project Name U.S. Highway 95 Expansive Volcanic Soils

Driller J. Deberry **Technician** ________ **Inspector** JHH

Drilling method 6" HSA **Sampler** 4" split spoon **Ground Water** ________

Depth	Moisture Color Group Names Class	Remarks
20 ft		
	dark grayish brown	
21		
22		
	Tube 10. (22.6-25.1)	22.5' -- w = 29.7%[5]
23		
24	fractures	
	light yellow brown	
25	Tube 11. (25.1-27.6)	
	olive yellow	
26		
27	olive	
	Sample depth	
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		
39		
40		