AN ABSTRACT OF THE THESIS OF

Gregory H. Clemmons for the degree of Master of Science

in Civil Engineering presented on June 22, 1979

Title: Evaluation of Coastal Oregon's Marginal Aggregates

Redacted for privacy

Abstract Approved: ______________________

R.G. Hicks

Coastal Oregon is deficient in quality construction aggregates. There is, however, an abundance of lower quality, or marginal, aggregate available for construction purposes. An evaluation of these marginal aggregates is the purpose of this paper.

There are four classes of marginal rocks found on the Oregon Coast. Basalts, high in mechanical strength but susceptible to chemical weathering, offer the most likely source of material to the Central and Northern Oregon Coast. Sandstones, which exhibit poor mechanical strength characteristics, are also potential rock sources. Sands and dredged materials, because of poor gradation, require stabilization or blending to provide sufficient stability, but offer a feasible alternative to importation of quality aggregate.

Descriptions of tests used to evaluate the mechanical and chemical degradation of the lower quality aggregate found on the coast are included, as well as a summary of the results of such tests. An intensive testing program was undertaken in two phases. Phase I, which consisted

of conventional durability testing was used to select appropriate aggregates to be tested in Phase II, or the repeated load test program.

Evaluation of the permanent deformation, resilient modulus and degradation due to loading was done on open graded samples and dense graded samples, each in the wet and dry condition. From this testing it was found that the marginal quality aggregates performed as well as the high quality aggregate when dry, but performed rather poorly in the wet state, suggesting waterproofing as a means of upgrading or beneficiating the aggregate.

The degradation analysis proved to be of little benefit when comparing the marginal aggregates with the high quality aggregates. The mechanical action of the compaction process and the repetitive loading degraded the aggregates but when they were tested in the wet condition they tended to re-cement upon drying. The results were not indicative of what actually occurred.

Blending a high quality basalt with a low quality basalt yielded favorable results with respect to permanent deformation and resilient modulus.

An Evaluation of Coastal Oregon's
Marginal Aggregates

by

Gregory H. Clemmons

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Completed June 22, 1979

Commencement June 1980

APPROVED:

Redacted for privacy

Professor of Civil Engineering in Charge of Major

Redacted for privacy

Head of Civil Engineering Department

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Date Thesis is Presented June 22, 1979

Typed by Laurie Campbell for Gregory H. Clemmons

ACKNOWLEDGEMENTS

The author expresses his sincere thanks to his major professor, Dr. R.G. Hicks, for his guidance and counseling during the conduct of this project. Sea Grant, Oregon State University, is also acknowledged for the financial assistance it provided.

The author would also like to thank the following people for the assistance they provided: Vanit Chintakovid, Gary Evans, Rob Vanderzanden and Chen Chang . The coordinated effort provided by these people greatly assisted the author in the literature review and testing phase of the project. The efforts of Laurie Campbell and Debbie Whitcomb, who typed this thesis, are appreciated.

The cooperation of the Federal Highway Administration, the United States Forest Service, the Oregon State Highway Division, numerous counties, and aggregate suppliers is also acknowledged.

Finally, the author wishes to express sincere thanks and appreciation to his wife Janet for her encouragement and understanding which greatly simplified the task of completing the work.

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AN EVALUATION OF COASTAL OREGON'S MARGINAL AGGREGATE

CHAPTER 1. INTRODUCTION

Many areas along Oregon's coast are deficient in quality construction aggregate (1). This deficiency is the result of numerous factors, all contributing to the problem. Briefly, these factors include environmental restrictions on newly proposed and existing mining operations, energy constraints which result in higher extraction and transportation costs, and detrimental physical characteristics of the available aggregates. Increasing population and decreasing reserves of relatively good quality aggregate mean that this deficiency can only become more severe over time unless corrective action is taken now.

A current solution to this problem is to import quality aggregate from areas that have abundant reserves. Figure 1 illustrates the origins and destinations of most of the imported aggregates. A good amount of this imported aggregate comes from the Willamette Valley, where about seventy percent of the state's population resides. The increasing population of this area is certain to impose more demand on the aggregate reserves. Environmental legislation, in the form of local land-use ordinances, state policies and federal guidelines also limit the amount of extractable aggregate in the Willamette Valley. The costly practice of carrying aggregate by truck through the coastal mountains and then returning empty will become more expensive as fuel and labor costs continue to rise. As the Willamette Valley supplies decrease and transportation costs escalate, other alternatives of supplying the coastal areas with aggregate will need to be considered. One alternative is to

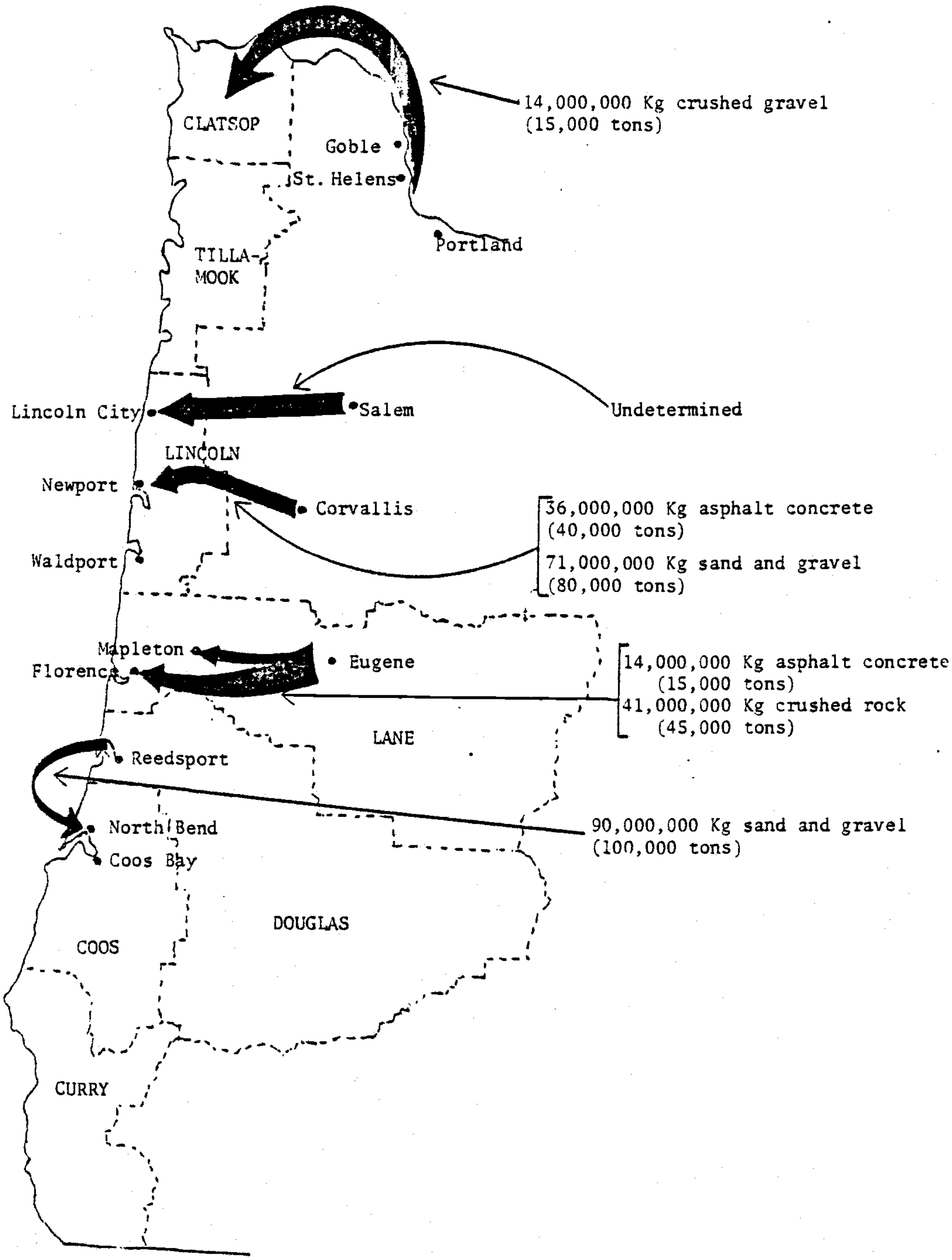

FIGURE 1. Annual Aggregate Importation to Coastal Oregon
(Source: Reference 33)

make use of the abundantly available, lower quality aggregates found near the coast of Oregon. However, numerous problems can result from their use in road construction. The purpose of this report is to characterize and evaluate the available aggregates found in these areas. The results of the report will hopefully lead to a better understanding of the advantages and disadvantages of using them as an alternative to costly importation from distant areas such as the Willamette Valley or other, aggregate-rich, coastal areas.

CHAPTER 2. AGGREGATE CHARACTERISTICS

Local aggregates which have the possibility of being used more extensively for construction purposes include: basalts, sandstone (or siltstone), dune sands, beach sands, and dredged spoils. The use of these materials, however, presents specific problems to the engineer. Table 1 is a listing of each aggregate type with its associated problem or problems. A description of the problems occurring with the use of these aggregates, tests used to evaluate their potential performance, and a summary of available testing data will be presented.

Evaluation of a particular aggregate requires an understanding of the problems likely to occur with its use. Knowing the problem potential will lead to the proper selection of the tests to be performed, and therefore, useless or redundant tests will be avoided. This section of the report will describe some of the problems encountered with using marginal or low quality rocks and will discuss the factors contributing to these problems.

Possibly the two most abundant aggregates found along the Oregon Coast are basalt and sandstone. Basalt is sometimes of marginal quality for highway construction because of the potential for degradation. Sandstone is almost always marginal, because of degradation potential. The two aggregates, however, undergo two different forms of degradation.

West, Johnson and Smith (2) defined degradation as "... the breakdown of aggregate pieces into smaller particles through chemical and/or physical processes." This definition encompasses both chemical, or weathering, degradation and physical, or mechanical, degradation. Some

TABLE 1

MARGINAL COASTAL AGGREGATES AND ASSOCIATED PROBLEMS

TYPE OF AGGREGATE	PROBLEM
Marine Basalt	Low Resistance to Chemical Degradation
Sandstone and Siltstone	Low Resistance to Mechanical Degradation
Sand, Beach and Dune	Low Stability Because of Poor Gradation Environmental Restrictions
Dredged Spoils	Poor Gradation Possibility of High Organic Content

basalts are susceptible to the former, while most sandstones fail by the latter.

BASALTS

Basalts are igneous rocks. Where the parent magma, or lava, was deposited and solidified is directly related to their performance in a road section. There are three broad classes of basalts to be considered: submarine, aerial and intrusive. Submarine basalts were deposited in water, with subsequent rapid cooling; aerial basalts were deposited in air or over land, with slower cooling; and intrusives were deposited within the earth and experienced variable cooling rates. The rate of cooling determines the grain size that was developed and also the quality of the rock to some extent.

Submarine basalts, with very rapid cooling, often fail to develop a distinguishable grain pattern. The substance that is formed under these conditions is termed glass. This glass, which is metastable or susceptible to breakdown, will sometimes alter to clay minerals through weathering action. If the clay minerals are expanding clays, such as smectites, degradation will occur with subsequent road failure (2). It is generally believed that the submarine basalts are potentially poor materials for road construction purposes (3). However, Van Atta and Ludowise (4,5) suggest that such a generalization should not be made. The reasoning is that degradation potential is not only a function of rate of cooling, but also of the mineral components of the parent magma. Certain quarry sites labeled as submarine exhibit a wide range of durability values, from poor to good. Also, some aerial basalts fail on some tests.

Recognition of the different types of basalts can sometimes be made at the quarry (6). A spherical shaped mass, termed a pillow, indicates a submarine basalt. An example of this is shown in Figure 2. These pillow basalts or pillow lavas result from rapid cooling of the flow when exposed to water. Pillows are composed largely of unstable glasses on the perimeter.

Columnar formations, as seen in Figure 3, usually indicate an aerial flow basalt, dike or sill. Aerial flows are variable in thickness, but usually exhibit a porous broken top grading down into a more dense, coherent body. Columnar jointing, the result of shrinkage on cooling, may be seen, with the joints perpendicular to the cooling surface. Glass is present chiefly in the broken porous top of the flow (34). If glass is not present in any significant amount, a more reliable source of quality aggregate is indicated.

Intrusive basalts result from solidification of the lava beneath an insulating rock cover. Such intrusive bodies may be lens-shaped masses of approximately uniform thickness, when compared to its lateral extent, emplaced parallel to the bedding of the intruded rocks. These are termed sills. They may also be tubular bodies that cut across the bedding of the intruded rocks, and are then called dikes. Both may exhibit columnar jointing with columns perpendicular to the cooling surface of the rock body. Vertical, pipelike conduits, or intrusive breccias, as shown in Figure 4, are formed when intruding lava encounters water-saturated rock, causing rapid chilling and steam explosions which tear it apart. These breccias are much less common than flows,

FIGURE 2. Pillow Basalt from Yaquina Head Quarry
Newport, Oregon

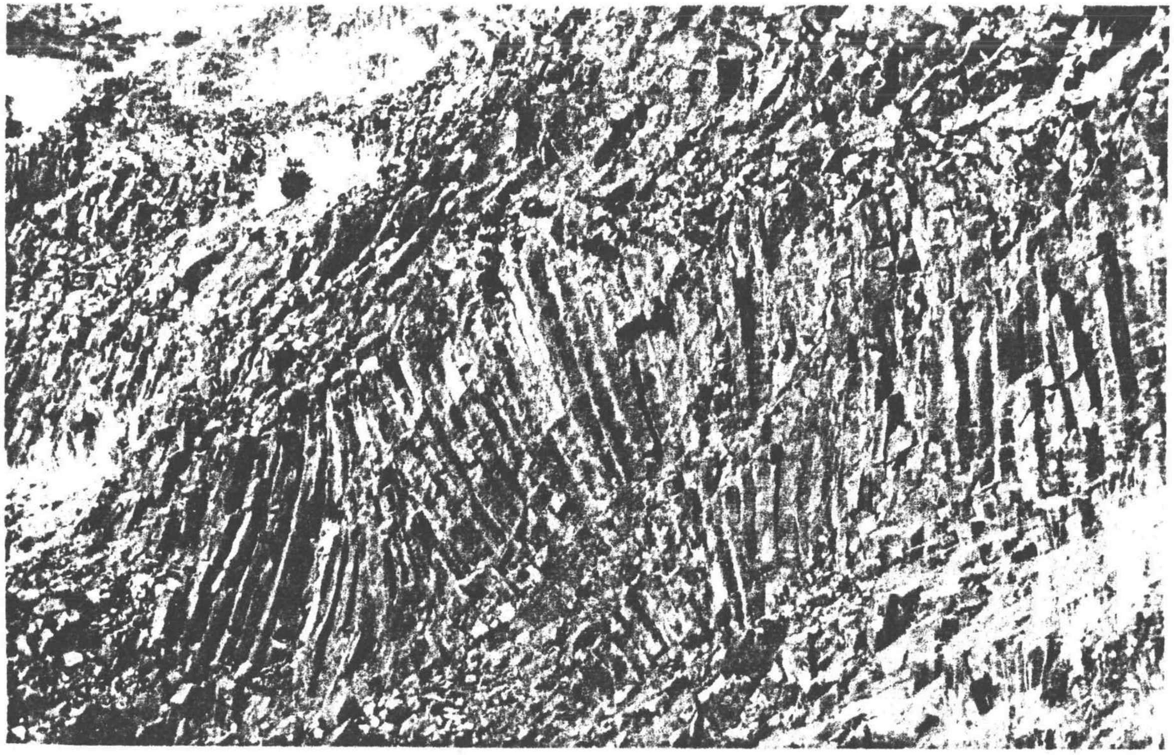

FIGURE 3. Columnar Basalt from Yaquina Head Quarry, Newport, Oregon

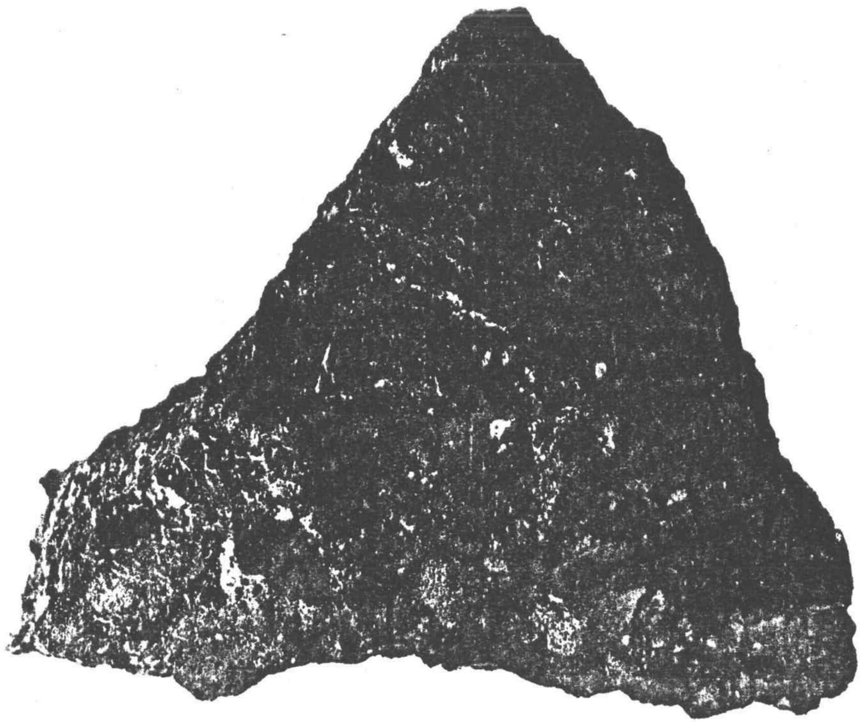

FIGURE 4. Breccia from Yaquina Head Quarry, Newport, Oregon

dikes or sills (34).

Problems incurred with the use of basalts, specifically submarine basalts, are well-documented and it is almost certain that many other road failures of this type are not recognized as such. For example, poor drainage conditions will cause a much more rapid failure of a road section made of marine basalt than a road section constructed of high quality aggregate. However, the failure may be attributed solely to the drainage problem and not the interaction of the water with the low quality rock.

In 1948, Melville (7) described the development of a plastic layer below a macadam surface on a Virginia highway. Rapid breakdown of an unsound rock was believed to be the cause. It was particularly troublesome because the rock had met the Virginia road specifications. This spurred the development of a new accelerated weathering test which attempted to describe the weathering or chemical degradation potential of a particular rock. The test consisted of a wet grinding process in which a rock sample was placed in a ball mill and rotated in water for two days. Atterberg limits were evaluated to determine the plasticity of the degraded material. Other weathering tests have since evolved, including the California Durability, Oregon Aggregate Degradation, and Washington Durability Tests.

In 1953, Minor (8) recognized the degradation susceptibility of highly altered basalts and suggested an abrasion test similar to the Los Angeles Abrasion (LAA). This modified LAA test differed from the conventional test by eliminating the standard charge of steel balls used to provide a grinding action. He also discussed the use of four percent portland cement to hold the degrading aggregate in place and thus reduce the manufacture of plastic fines caused by the kneading and pumping of traffic.

Erickson (9) discussed the degradation of Columbia River basalts found in Idaho in 1955. His discussion focused on the comparison of different degradation tests. Scott (10), also working with these Idaho basalts, determined that the percentage of secondary or alteration minerals could be directly correlated with performance. Petrographic analyses were used in determining the amount of secondary minerals present. It was found that less than 20 percent secondary minerals resulted in good performance, 20 to 35 percent was borderline or marginal, while greater than 35 percent resulted in almost certain road failures.

Sibley (11), working in Washington, determined that the plasticity of fines generated in a modified Los Angeles Abrasion test could be reduced by approximately 60 percent with the addition of portland cement. Also in that study, a kneading compaction test (15,000 tamps at 500 psi) was conducted. In this case, cement treatment of the fines produced during compaction failed to reduce the plasticity index as it did in the modified Los Angeles Abrasion test.

Numerous other accounts of road failures because of degrading basalts are available in the literature (12-16). The general conclusion is that the production of plastic fines in altered basalts is the principle cause for this failure. A discussion by Krebs and Walker (27) explains the significance of the presence of clays in a structural section such as a road base. Aggregate degradation can lead to the production of fine-grained soil. Fine-grained soil consists of material smaller than a No. 200 (0.074 mm) sieve opening and can be further classified as silt or clay. The change from silt to clay begins at about 0.005 mm and is essentially complete at 0.001 mm. There are five broad classes of clay minerals: Kaolinite, Halloysite, Illite, Montmorillonite and Chlorite. Because of a lack of interlayer bonding, Montmorillonite clays are particularly troublesome when present in structural section. This lack of bonding in Montmorillonite allows almost unlimited expansion to accommodate the presence of water, hence we have the term "expanding clays." The development of Montmorillonite clays occur in the presence of ferromagnesien minerals such as those common to basalts and volcanic ash, rocks which are in abundance in the Pacific Northwest. Efforts to beneficiate or improve their performance have been met with little success (17). Beneficiation measures should recognize the potential for chemical degradation, a relatively long-termed process when compared to mechanical or load associated degradation. A logical solution to the chemical degradation problem would be to isolate the aggregate from water. Because of their widespread availability in the Pacific Northwest, and their relatively strong mechanical characteristics, efforts to improve these basalts would appear to be warranted.

SANDSTONE

More than 75 percent of all aggregates exposed at the surface of the earth are sedimentary (18). The Coast Range and coastal areas of Oregon exhibit an abundant amount of sandstone and siltstone, which, if strengthened, may provide substantial amounts of locally available aggregate for construction purposes.

Sandstone is composed of cemented sand grains. These grains are no different than the sand found on beaches or in dunes. The cementation is provided by the precipitation of mineral matter in the pores of the sandstone. The cementing material may be added from outside the rock by migrating solutions or may possibly result from the reorganization of mineral matter already present within the rock by solution from grains and precipitation within pores. The strength of sandstone is determined by the strength of the cementing agent, which unfortunately, is not adequate for most road construction purposes. In fact, conventional testing of these sandstones is difficult to control because of this low strength. Significant reduction in sizes is experienced when the sandstone is shaken over sieves and when it is compacted (Figure 5). Field compaction and subsequent traffic loading will result in dense gradations and subsequent loss of permeability. The presence of water will result in instability and failure. Huddleston (19) tested some of the locally available sandstone found on the Oregon Coast and determined that the addition of portland cement greatly enhanced the strength of these materials. Metcalf and Goetz (20) suggested the use of asphalt to improve the stability of sandstone, although the high absorption (10-12%) typical of sandstone makes this relatively expensive.

Sandstone has been used in only limited amounts on the Oregon Coast. The United States Forest Service has built roads using this aggregate for a base on lightly-traveled, dead-end roads of short expected life (one or two years) (21). The BLM has built test sections on the Central Oregon Coast. No evaluation of their performance is as yet

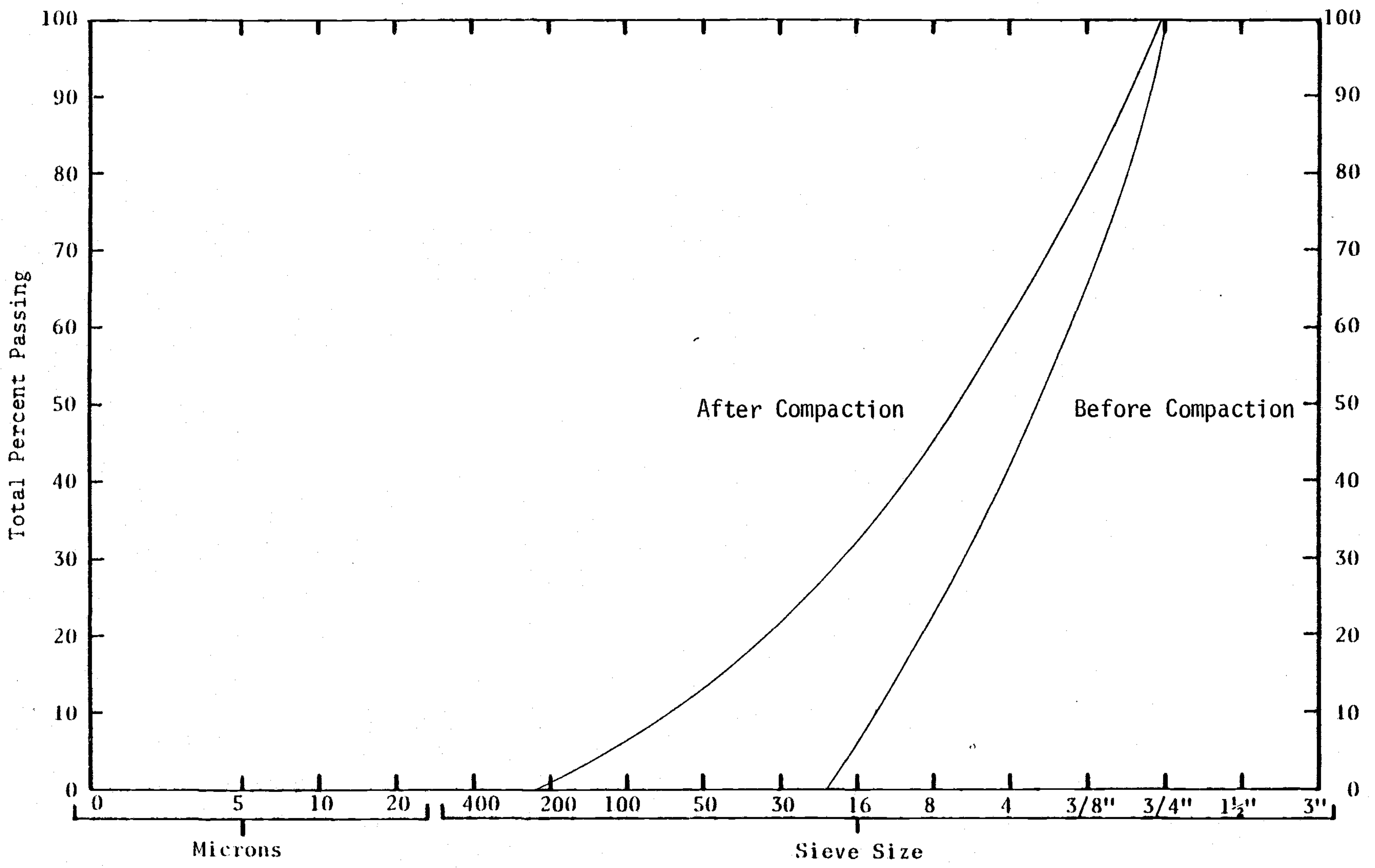

FIGURE 5. Grading Analysis Showing Breakdown of "Big A" Sandstone as a Result of Modified Proctor Compaction

available because of the relatively recent construction.

SANDS

Dune sands and beach sands are relatively abundant on the Oregon Coast. However, two problems exist with the use of these sands for construction purposes. The first problem concerns the instability of sand when used untreated. The second problem is the lack of available extraction sites because of environmental restrictions.

Oregon's first experience with portland cement stabilized dune sands was with the construction of a section of the coastal highway (U.S. 101) near North Bend during the period from 1939 to 1941. This section of highway performed very well, but was uneconomical because of high cement requirements (14 percent). High cement contents are required to provide the strength from grain interlocking that is absent in these poorly-graded sands. Borgen (22) conducted research in 1961 on cement stabilization of dune sands found along the coast of Oregon. He cited two reasons for the high cement contents, one of which was the high void ratio. The other reason for high cement content was the presence of organic acids found in sands lying directly beneath surface vegetation. The organic acid is the result of leaching of decaying matter throughout the upper sand layer. Satisfactory strength of these acidic, uniformly graded sands could be attained only with the addition of approximately 14 percent portland cement by weight. Dune sands from greater depths, free of organic acids, realized the same strength gain with about 11 percent portland cement.

The United States Forest Service (21) has utilized sand for lightly-traveled roads. Sutton Lake Campground Road (No. 1749) was constructed utilizing emulsified asphalt (CSS-1 and CSS-1h) mixed in place with the native sand and sealed with a chip seal. Sand Beach Campground Road (No. S3001) was constructed by mixing either CSS-1 or CSS-1h emulsion with the native sand in a pugmill. This road was also surfaced with a chip seal. Both roads, although subjected to only light campground traffic, have performed well since construction over six years ago.

Blending of sands with other aggregates was done by the Oregon State Highway Department on the Bullards Bridge Project in Coos County. Pea gravel and terrace gravels were mixed with the local sands and used as a base with adequate results (30).

The other major problem with using Oregon's dune and beach sands is potentially more restrictive. Because of the scenic beauty of the Oregon coast, large-scale mining operations are limited. Some small-scale coastal operations are supplying sand for small construction projects, such as subbases for parking lots or for fill purposes (39,40). In general, however, the use of these sands for construction purposes is limited. One major concrete supplier (24) has found it necessary to import approximately 5,000 cubic yards annually from the Willamette Valley. This is required because of the lack of available sand supplies in the area.

DREDGED MATERIALS

Aside from the Umpqua River, dredged spoils are relatively unused on the Oregon Coast. Table 2 is a listing of the material dredged from the major coastal rivers. Like the dune sands and beach sands, these

TABLE 2

LOCATION, TYPE AND AVERAGE ANNUAL AMOUNTS OF DREDGED MATERIALS FROM COASTAL OREGON FROM THE YEARS 1973 to 1977 (Source: Reference 33)

County	Location	Amount Cubic Yards	Total Cubic Yards	Total Cubic Meters	Type of Materials
Clatsop	At the Mouth of the Columbia River Oregon and Washington	5,878,624	6,665,000	5,095,600	Sand and Silt
	Skipanon Channel, Oregon	50,050			
	Tongue Point, Piers 7 & 8, Oregon	40,900			
	Columbia Slough (Operation Fore-sight)	26,310			
	Astoria Turning Basin	669,102			
Tillamook	Tillamook Bay and Bar, Oregon	24,701	133,000	101,700	Sand
	Wilson-Trask River, Oregon	108,163			
Lincoln	Depoe Bay, Oregon	12,437	652,000	498,500	Sand
	Yaquina Bay and Harbor, Oregon	639,165			
Lane	Siuslaw River, Oregon	237,654	238,000	182,000	Sand
Douglas	Umpqua River, Oregon	323,812	499,000	381,500	Sand
	Smith River, Oregon	174,941			
Coos	Coos Bay, Oregon	2,666,273	2,754,500	2,105,900	Sand and silt
	Coos and Millicoma Rivers, Oregon	35,851			
	Coquille River, Oregon	52,314			
Curry	Chetco River, Oregon	43,370	187,200	143,200	Sand
	Rogue River Harbor at Gold Beach, Oregon	106,282			
	Port Orford, Oregon	37,514			

water-deposited materials exhibit relatively uniform grading. The material deposited by relatively slow moving waters tends to be smaller in size, while faster moving waters will tend to deposit larger-sized aggregates. Figure 6 shows relatively large aggregate deposited by the Chetco River, while Figure 7 displays the smaller sand sizes deposited by the slower moving Siuslaw River.

Testing of dredged aggregates has not been done at OSU at this time, but their durability is expected to be fairly high. The weaker rocks erode more as they are transported downriver and leave only the stronger ones. The problem of uniform grain size distribution could be partially solved by blending the spoils from areas exhibiting different flow velocities. As the economics of hauling aggregates from distant sources becomes prohibitive, these materials will play an important role in relieving local aggregate shortages. At the same time, the problem of disposing of the dredged spoils in an environmentally acceptable manner would be eased.

In concluding this chapter, it is known that aggregates of marginal quality are abundant along the coast of Oregon (33). However, economical solutions to the problems of degradation and poor size distribution are, as of this date, elusive. With rising fuel costs it may be that a previously uneconomical treatment method, such as portland cement, lime, or asphalt emulsion will become cost effective in the not too distant future.

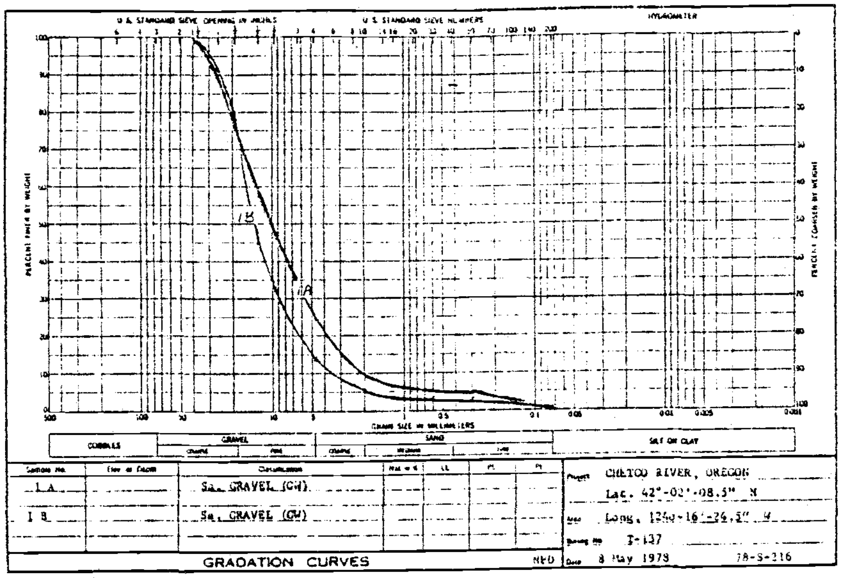

FIGURE 6. Aggregate Deposited by the Chetco River (Reference 35)

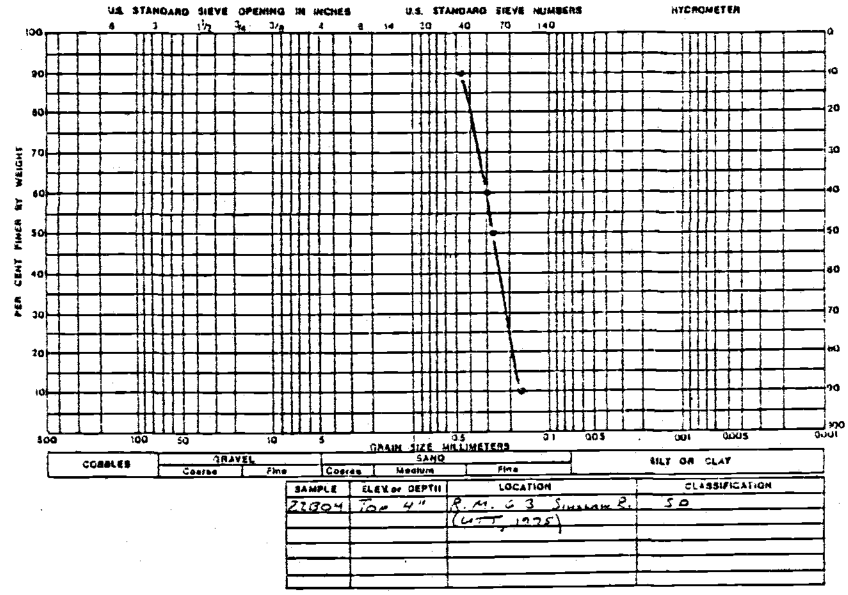

FIGURE 7. Aggregate Deposited by the Siuslaw River (Reference 36)

CHAPTER 3. METHODS OF EVALUATION

Evaluation of an aggregate can be accomplished in many ways. Of interest to the highway engineer is the sample gradation and the mechanical and chemical durability of the aggregate. It is the purpose of this chapter to describe the different durability tests that are currently used in the Northwest.

Several durability tests for construction aggregate are in use today. These tests include the following:

Property	Test Method
Mechanical Degradation	(1) Los Angeles Abrasion (LAA)
Chemical Degradation	(1) California Durability (2) Oregon Aggregate Degradation (OAD) (3) Washington Durability (4) Idaho Durability (5) Accelerated Weathering (Dimethyl sulfoxide or ethylene glycol)
Mineral Content	(1) Petrographic Analysis
Past Performance	(1) Field Evaluations of Aggregate

In the past 25 years, much has been written about the problem aggregate marine basalt, and it is unfortunate that many chemical weathering tests have been developed to predict essentially the same thing. Each Northwest state has developed its own testing procedure to measure the production of plastic fines under simulated weathering conditions. A discussion of these tests as well as the other principal tests conducted on coastal aggregates follows. The Idaho Durability test will not be considered because of its infrequent use in coastal Oregon. Following a description of the different methods of evaluation, a discussion of the detailed test program to be conducted is given.

LOS ANGELES ABRASION TEST (LAA) (OSHD TEST METHOD 211-74 METHOD B)

The Los Angeles Abrasion Test, which measures mechanical degradation was first developed in 1916 (27). It consists of rotating 5,000 grams of aggregate in a hollow steel cylinder for 500 revolutions with a charge of 11 steel balls of a specified size. The rock is then sieved over a 1.68 mm (No. 12) opening and the percent that passes is recorded as the LAA value. Thirty-five percent loss is a typically maximum value allowable; however, this changes according to the construction requirements. The Oregon State Highway Division (OSHD) uses 30%, 35% and 45% for the surface, base and subbase, respectively, as the maximum percent loss. The Federal Highway Administration (FHWA) and the United States Forest Service (USFS) use 35% in all locations.

CALIFORNIA DURABILITY (TEST METHOD NO. CALIF. 229-E)

The California Durability Test is a chemical degradation, or weathering test. The coarse size aggregate (2,500 g) is placed in a steel canister with 1,000 ml of water. The canister is placed in the durability shaker and the sample is agitated for ten minutes. After shaking, the water and material finer than a 0.074 mm (No. 200) sieve is drained into a sand equivalent tube containing 7 ml of sand equivalent stock solution. The liquid and suspended solids are mixed and then allowed to settle for 20 minutes. The sediment height is observed as an indirect measure of the size of the clay particles that are produced. Thus, the durability factor for the coarse sized material, D_c, is obtained. The fine-sized material, smaller than the 1.75 mm (No. 4) sieve, is tested in a manner similar to the sand equivalent test, but with a 10-minute agitation period. The value obtained by dividing the sand height by the clay height is the durability of the fines, or D_f.

This test is fairly involved and requires close adherence to the specifications for consistent results. It yields a measure of the amount of clay sized material that can be expected under weathering and loading conditions, thus it is a reliable measure of the quality of basalts and other clay bearing aggregates. The FHWA and USFS consider failure to be 35 percent or greater for either D_c or D_f.

OREGON AGGREGATE DEGRADATION (OAD) (OSHD TEST METHOD 208-75)

The Oregon Aggregate Degradation Test (OAD) uses a different means of attaining the same results as the California Durability Test: production of clay sized particles under the action of weathering. The test is frequently referred to as the "Oregon Air" because the sample is agitated in water by means of compressed air instead of a mechanical shaker. It briefly consists of placing a carefully prepared specimen (100 g) of material passing the 1.70 mm (No. 10) and retained on the 1.61 mm (No. 20) sieves, adding 100 ml of water and agitating with compressed air for 20 minutes. The measure of degradation is done in two ways. A sediment height test, similar to the California Durability test is performed. An

8.9 cm (3 1/2 inch) sediment height is considered the maximum allowable for most purposes. Next, the sample is rescreened over a 1.61 mm (No. 20) sieve. This measures an aggregate's susceptibility to mechanical degradation in the presence of water. Thirty-five percent loss is considered to be the maximum allowable loss.

This test is somewhat simpler than the California Durability Test, but still requires close attention to the specifications when preparing the sample for consistent results. The FHWA and USFS do not normally perform this test.

WASHINGTON DEGRADATION (WSHD TEST METHOD NO. 113A)

The Washington Degradation test is perhaps the simplest durability test to perform. It also has the advantage of having only one result to measure durability. The test consists of placing 500 g of 1.27 cm (1/2 inch) to 0.64 cm (1/4 inch) and 500 g of 0.64 cm (1/4 inch) to 1.70 mm (No. 10) sized aggregate in a plastic Tupperware canister, adding 200 ml of water and agitating in the California Durability shaker for 20 minutes. After agitation, the water is drained into a sand equivalent tube with 7 ml of the stock solution. A sediment height is read after 20 minutes and from this, a durability factor is read from a table. The test yields results similar to the California Durability and OAD tests and is usually run on basalts and other clay bearing aggregates. Durability values less than 50 percent indicate marginal or lower quality rock. The State of Washington is the main user of this test.

ACCELERATED WEATHERING TEST (FHWA REGION 10)

The susceptibility of an aggregate to weathering or breaking when exposed to water is estimated by this method of testing. Dimethyl Sulfoxide (DMSO) or Ethylene Glycol (E.G.) can be used. These liquids tend to penetrate through the micropores and larger vesicles which are abundant in lower quality basalts and attack the expanding clays within the rock. The expansion forces result in fracturing of the rock.

This method, which was developed by the U.S. Army Corps of Engineers, is used by the Federal Highway Administration (Region 10) as a backup test for borderline results of the California Durability. More than four rocks broken out of the ten original rocks is a failing test. The test method was recently changed from using the ten-rock sample to a procedure similar to the Sodium Sulfate Soundness Test to determine a weighted loss from various sizes. No significant amount of test data is available for correlation of the new test with other tests and field performance.

PETROGRAPHIC ANALYSIS

West, Johnson and Smith (2) and many others have recommended that petrographic analysis and X-ray examination be conducted on all basalts. Degradation potential of a basalt is a function of the minerals present in the particular rock. Chief contributions to weathering in basalts are glass and smectite clay, which is a member of the Montmorillonite family (4). This type of clay is highly reactive with water. The purpose of the petrographic test is to detect the presence of glass and clay. The type of clay present requires X-ray examination. These tests, however, are difficult to perform because of the need for specialized personnel and equipment. Also, a particular quarry site cannot be evaluated on the basis of one test. Many quarry sites in western Oregon exhibit marine, intrusive and aerial deposits. These different regions of a pit will exhibit different mineral characteristics, therefore complete petrographic analysis of a pit would be cost prohibitive. If precise control on the location of rock excavation can be attained, the analysis is worthwhile, however, such control is difficult. Petrographic analysis is generally looked upon as a test used for research purposes.

FIELD EVALUATION

Perhaps the best way of evaluating a rock is a subjective rating of the rock in service. If a rock is known to perform well, testing is not required. However, field evaluation is probably the most questionable "test" available for several reasons.

Aggregate quality will vary throughout a particular quarry. Good performance in one region does not necessarily imply good performance of any other region. Construction practices, loading conditions and climate variations will also influence, to a great extent, the performance of an aggregate. Lastly, one evaluator may recognize different problems as being aggregate associated or nonaggregate associated. A subjective evaluation is difficult to substantiate when approval or rejection of a contractor's bid is on the line.

COMPARISON OF TESTS

Correlation of test results with field performance has been attempted. Lund (28) correlated field performance as measured subjectively by a rating team, with Oregon Aggregate Degradation, California Durability, Washington Durability, Los Angeles Abrasion (LAA), and Sodium Sulfate Soundness (SS). Both LAA and SS failed to correlate very well, but the Oregon, California and Washington tests all performed satisfactorily with the Washington test giving the best results. This is interesting, because the test procedure for the Washington Durability test is by far the most straightforward and simplest to perform.

Breese (29), in a study to evaluate different test methods for the Nevada State Highway Department, discussed the relative merits of the Oregon, Washington and California tests. No specific recommendations were made in this report. Also, this study did not directly correlate test results with field performance.

Field performance, because of different construction, loading, climate, evaluation, etc., is inherently difficult to measure accurately, so one questions the results of correlation studies. Also, to be truly representative, a series of such tests should be conducted on a particular rock type, i.e., basalt, gabbro, sandstone, etc. No such evaluation has been performed.

SUMMARY OF AVAILABLE TEST DATA

Tables 3 through 7 are listings of test data that have been compiled on selected quarry sites found in or near Coastal Oregon. Table 8 is provided as a key to the terminology used in these tables. These

TABLE 3

SUMMARY OF AGGREGATE TESTS - BASALT

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. GR.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
Tobe Creek 35-02-0001	Basalt	55		58	37	23		9		
		44		74	34	26		7		
		36		68	47	24		5		
		44			45					
		22(A.R.)		68	41	23				PI = 4
		27(moist)	3.09/3.03	60	42					LL = 26
		36		65	37	21				
		30	2.95	55	25	22	8	1		PI = 6 LL = 27 Wt. Ave: DMSO 36.54 E.G. 6.61
South Fork Alsea Quarry 35-02-0002	Basalt	87	2.84	93	95	16		0		
		85		93	94	17		0		
		79		90	92	16		0		
Mary's Peak Quarry 35-02-0026	Marine Basalt		2.83		67	20				
			2.86	44	27	45				
		59	2.96F 2.83C	31	30	67	10	0		
		51	2.86F 2.69C	37	27	25	10	3		
		45 28(A.R.) 57(Man) 25(A.R.) 55(Man)			21					
Siletz Quarry 35-21-0016	Basalt	37(Man)	2.60F 2.69C	36	30	26		1		
Kaufmann 35-21-0019	Weathered Basalt	68(Man)	2.84F 2.87C	80	78	18		0		S.S. 8 Strip < 95%
Ocean Lake S & G 35-21-0027 See pg. 14 for more	Basalt	66	2.85F 2.89App	85	91	13		0		S.S. 5% Strip < 95%
Hill Top & Roads End 35-21-0028	Basalt	70	2.73F 2.70App	74	64	19				S.S. 9%
Morris 35-21-0029	Basalt	75	2.74F 2.73App	58	43	16				S.S. 14%
Kinchloe Quarry 35-06-0003	Marine Basalt			52	33	15		0		
				49	32	13	0	0		
Kasper Quarry 35-06-0011	Marine Basalt	68mfg	2.97F	62	60	26		2		
		77mfg	2.98F	70	48	25		0		S.S. = 12
		71mfg	2.99F	63	56	29		2		
		32(A.R.)		61	45			0		S.S. = 14
				52	26	29				PI = 7 LL = 24
Gray Quarry 35-06-0016	Basalt Flow (Marine)	31(A.R.)		45	40					S.S. = 32%
				52	48	19				LL = 21
		38(A.R.)		48	46	17				S.S. = 8%
				58	47	15				
Indian Creek Quarry 35-06-0017	Marine Basalt		2.85F	34	30	23	10	5		

TABLE 3 (continued)

SUMMARY OF AGGREGATE TESTS - BASALT

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. GR.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
Berry Creek (Ray Wells)	Basalt	38(A.R.)		36	33	22				
		43(A.R.)		36	22	22				
		50(A.R.)		50	30	25				
		35		44	31					
		40		50	31					
		39(A.R.)		53	25	27				
		47		49	42					PI=7 LL=16
Ocean Lake	Basalt		Bulk=2.85 SSD=2.87			12.14			16.78%/ 0.6 in	
			Bulk=2.85 SSD=2.87			11.70			15.17/ 0.5	
			Bulk=2.85 SSD=2.88			12.52			14.69/ 0.6	
			Bulk=2.85 SSD=2.87			13.30			14.01/ 0.4	
			Bulk=2.85 SSD=2.87			13.74			15.04/ 0.5	
			Bulk=2.86 SSD=2.88			11.96			16.90/ 0.5	
			Bulk=2.87 SSD=2.87							
Ansley Ranch Quarry Boekelman Quarry 35-06-0021	Columnar over Marine Basalt	50mfg	2.98F	66	40	26	4	0		
		55mfg	2.86F	56	40	21	6	1		
		64mfg	2.93F	58	47	22	5	0		
		65mfg	2.96F	70	58	26	2	0		
		69mfg	2.98F	78	80	19	4	0		
		55mfg	2.99F	61	42	25	4	0		
		49mfg	2.86F	35	30	21	4	0		PI=4 LL=28
				27	20		8	7		PI=13 LL=38
				63			5	0		
				54		18	2	0		
Kenstone Quarry 35-06-0041	Basalt	41mfg	2.88F	4	22	37	7	2		Strip < 95%
		49(A.R.)		34	34		7	5		PI=7 LL=29
"County" Pit 35-06-0052	Basalt			51	26	24				W.D. = 13
Woodward #2 35-06-0060	Altered Marine Basalt	38mfg	2.75F	46	32	24				Strip < 95%
		44mfg		27	25	29				
		43mfg	2.83F	25	26	32				
		44mfg		30	26	30				
Waterman Quarry 35-06-0064	Metamorphic (Basalt?)		2.77F	58	33	21		0		Strip < 95%
				43	28	29				
Norway Rock Products 35-06-0079	Sub-Basalt (Submarine)			24	26	19	10	10		
Highway 42 Quarry 35-06-0095	Submarine Basalt	53mfg	2.97F	62	48	22	10	0		
				54		20	9	5		

TABLE 4

SUMMARY OF AGGREGATE TESTS - SANDSTONE

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. GR.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
Little Wolf Creek Quarry 35-10-0044	Sandstone	29	2.64F	65	40	32				S.S. = 30
		35	2.63F	70	40	40				S.S. = 21
		21	2.66F	54	30	36				S.S. = 83
Old Wolley Quarry 35-10-0127	Sandstone	27(Man)	2.64F	38	26	62	0	0		
		34(Man)	2.77F	52	28	56	0	0		
Manasha (Owner) 35-10-0151	Sandstone	58(Man)	2.59F	70	42	43				
		49mfg	2.68F	74	37	44				
		50mfg	2.67F	74	42	39				
		37(A.R.)		67	39	41				
N. Fork Coquille 35-06-0049	Sandstone	21	2.67F			96				
			2.66F			77				
		17				95				
				28	29	85		0		
BLM Pit 35-06-0054	Sandstone	38mfg	2.63F	65	31	49				S.S. = 73%
		46mfg	2.63F	51	34	48				S.S. = 92%
										Strip < 95
		49mfg	2.63F	54	31	49				S.S. = 63
		50mfg	2.63F	45	33	59				S.S. = 92%
		51mfg	2.61F	59	33	58				S.S. = 90
Buck Peak	Sandstone			48	35	15		0		PI = 4
										LL = 22
		31mfg	2.61F	46	28	47	0	0		
		30mfg	2.62F	50	29	49	0	0		
		28mfg	2.64F	36	27	52	0	0		
		33mfg	2.60F	54	28	47	0	0		
Moon Creek 35-06-0079	Sandstone	27mfg	2.67F	27	29	71	1	1		
		27mfg	2.70F	38	29	58				
		32mfg	2.62F	47	30	71				

TABLE 5

SUMMARY OF AGGREGATE TESTS - GABBRO

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. GR.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
Will. Ind. 35-21-0030	Gabbro		2.71F	82	60	22		0		Strip < 95%
Greenleaf Creek 35-20-0063	Gabbro	65	2.76F	67	46	30		9		Strip < 95%
		73	2.74F	76	58	24		1		Strip < 95%
		71	2.67F	78	58	23		0		Strip < 95%
		74	2.75F	74	54	25		3 of 8 (4)		Strip < 95%
		70	2.67F	80	67	24		3		
		58	2.72	80	50	29	6	2		
Deadwood Quarry 35-20-0048	Gabbro Ledge Rock	74	2.86F	85	64	17		0		Thin Section S.S. = 2% Strip < 95
		70	2.74F	82	65	16	0	0		
		73	2.77F	85	68	15	0	0		
		41	2.67F	67	39	33	2	0		
East Roman Nose 35-10-0055	Gabbro	80(Man) 49(A.R.)	2.78F	74	65	18		0		
		88(Man) 41(A.R.)	2.74F	78	74	18		0		
		40(A.R.) 80(Man)	2.79F	76	73	18		0		S.S. = 1%
Bridge Creek Quarry 35-10-0187	Sandstone & Conglomerate	50(A.R.)		26	30	66		1		
		63mfg		19	27	82		0		
		72mfg		19	29	85		0		
Wooley (Owner) 35-10-0208	Gabbro			80	60	19		0		
		63mfg	2.79F	87	68	17	0	0		

TABLE 6

SUMMARY OF AGGREGATE TESTS - SAND AND GRAVEL

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. Gr.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
Timmons Quarry 35-36-0044	Gravel	37		74	48	13				
		52		76	68	13				
		59		76	57	12				
				78	91	14	2	0		Wt. Ave: DMSO 3.3 E.G. 0.1
Gooseneck R.Q. 35-27-0004	Gravel		2.94	78	73	15				Sodium Sulfate 6
		82(Man)	2.94	76	77	16				Sodium Sulfate 8 S.E. = 12 as Received
		42(A.R.)			54	18		0		
Morse Bros. 35-02-0028	Gravel	68(Man)	2.75	78	82	17	1	0		Wt. Ave: DMSO 9.03 E.G. 0.5
Umpqua River 35-10-0024	River Gravel	78(A.R.) 80(Man)	2.70F	71	78	14				
Govt. Owned 35-29-0027	Quarry Rock	65	2.82	70	46	28	10	3		
Sand Dune 35-29-0051	Dune Sand	R @ 300 psi = 78--2.67--Density = 100 PCF--% w.c. 17--AASHO								A-3(0)
Slide Creek Quarry 35-02-0025	Diabase Dike or Sill	69	2.90	76	68	20	0	0		Strip < 95
		81	2.97	76	72	13	0	0		
		61	2.81	63	55	35	3	0		
		69	2.85	85	67	24	0	0		
Flat Mtn. Quarry 35-02-0029	Igneous Intrusion	58(Man)	2.78	66	43	26	0	0		
		56(Man)	2.75	78	53	18	0	0		Report Given
			2.77F							
			2.71C							
Dovre Peak West 35-29-0047	Intrusive; Diorite?	58	2.72	68	41	21	2	2		

TABLE 7

SUMMARY OF AGGREGATE TESTS - MISCELLANEOUS

Pit Name and Number	Type of Rock	Sand Equiv.	Sp. GR.	California D_c	California D_f	LAA	DMSO	E.G.	OAD	OTHER
BLM 35-21-0025	Andesite			70	40	22		3		PI = 4 LL = 36
BLM 35-21-0026	Quartz Diorite			63	47	28		1		
Beckley Thomas Quarry (Ten-mile Quarry) 35-10-0036	Metamorphased Vol-canic	58		71	52	22		0		
		64		70	51	20		0		
		70		78	74	19		0		
		53		73	47	20		1		
		90		71	50			0		PI = 4 LL = 21
		86		70	53			0		
		77			66	20		1		
Esmond Creek Quarry 35-10-0164	Gabbro Sill	71mfg	2.72F	82	69	18				S.S.= 7%
Nelson Ridge Quarry 35-20-0019	Gabbro (Diabase)	59	2.78F	76	49	25		4		Strip < 95%
		71	2.80F	85	66	22		0		Strip < 95
		63	2.79F	78	59	25		0		Strip < 95
		71	2.81F	76	63	23	0	3		Strip < 95
Langlois Quarry Sullivan Ranch Quarry 35-08-0058	Metamorphic	73mfg	2.91F	73	61	10	0	0		
		57(A.R.)	2.99F	74	66	14	0	0		
			2.93App.							
					98	15	0	0		
McDowell Quarry 35-36-0047		27		56	40	14				PI = 2 LL = 28
		20		36	26	25				PI = 8 LL = 38
Gleneden Beach	Beach Sand	98	Bulk=2.63 SSD=2.65							FF = 2.75

TABLE 8

KEY TO TERMINOLOGY USED IN TABLES 3 THROUGH 7

1. The pit numbering system as developed by the FHWA is described as follows:

35	-	29	-	0008
State (Oregon)		County (Tillamook)		Pit Number

County	Number
Tillamook	29
Yamhill	36
Polk	27
Benton	02
Lincoln	21
Lane	20
Douglas	10
Coos	06
Curry	08

2. Abbreviations

Sp. Gr.	-	Specific Gravity
F	-	Fines
C	-	Coarse
D_c	-	Durability of coarse-sized aggregate
D_f	-	Durability of fine-sized aggregate
LAA	-	Los Angeles Abrasion
DMSO	-	Dimethyl Sulfoxide
E.G.	-	Ethylene Glycol
OAD	-	Oregon Aggregate Degradation
S.S.	-	Sodium Sulfate Soundess
PI	-	Plastic Index
LL	-	Liquid Limit
N.P.	-	Non-plastic
Man,mfg	-	Manufactured by laboratory crushing
A.R.	-	As Received

3. Typical specification values for durability tests

Sand Equivalent	-	35% minimum
California Durability	-	35% minimum
LAA	-	35% maximum
OAD	-	3.5 in.maximum sediment height
	-	35% maximum passing the No. 20 sieve

tables do not represent all of the private and commercial quarries, but it is believed that they give a representative view of what is available. The data were provided mostly through the efforts of the Federal Highway Administration, Region 10. Test data were also provided by the Oregon State Highway Division, the United State Forest Service, various counties, and private operators. Table 9 presents the range in test values that was found from these tables.

From Tables 3-7, it is evident that many of the quarries exhibit variable results for the durability tests. This reflects the varying nature of a rock deposit from one zone to another, even though they may be only a few feet apart. This variability presents a problem for the materials engineer when prospecting for new sites or accepting or rejecting potential sources presented as a bid item by contractors. At this point, the materials engineer must rely on past performance of the rock and on his own engineering judgment. Experience is a valuable tool when evaluating aggregate.

As a matter of interest, regression analysis was performed on the values obtained for DMSO testing, E.G. testing and California Durability testing. The results are listed as follows:

$$D_f = 47.5 - 1.18\ (\mathrm{DMSO}) \qquad r = -0.26$$

$$D_c = 65.81 - 2.04\ (\mathrm{DMSO}) \qquad r = -0.50$$

$$D_f = 50.16 - 2.27\ \mathrm{EG} \qquad r = -0.36$$

$$D_c = 56.00 + 0.18\ \mathrm{EG} \qquad r = 0.12$$

D_c and D_f represent the California Durability factors for the coarse and fine aggregate and DMSO and EG represent the number of rocks broken in the ten-rock test. The relatively low correlation coefficients (r)

TABLE 9. Ranges in Durability for Coastal Aggregates

MATERIAL	RANGE
Basalt	$D_c = 23$ to $D_c = 93$ $D_f = 25$ to $D_f = 45$ LAA = 15 to LAA = 50
Sandstone	$D_c = 27$ to $D_c = 74$ $D_f = 27$ to $D_f = 42$ LAA = 32 to LAA = 96
Desirable Values	$D_c > 35$ $D_f > 35$ LAA < 35

indicate that the tests are probably measuring different qualities of the rock. Tables 3-7 do seem to substantiate the fact that the Accelerated Weathering tests are measuring the amount of expanding clay minerals within a rock. The sandstones, which have little clay, generally showed no reaction with either the DMSO or the E.G. Also, the DMSO reaction appears to be much more violent in the basalts than does the E.G. reaction. The average number of rocks broken in the ten-rock test was 5.3 for DMSO and 2.4 for E.G.

There are many methods of evaluating the durability of an aggregate. Different areas of the country have different methods of estimating the performance of local aggregates. In the Northwest, where basalts are common, each state has developed its own test for detecting potential production of clay particles. An evaluation of the different tests was performed by Lund for the U.S. Forest Service and it was found that the Washington Durability test was the most reliable and also the simplest. Adoption of this test would help to alleviate confusion among different suppliers and also shorten laboratory testing time.

CHAPTER 4. TEST PROGRAM

Selection of typical high-quality and low-quality aggregates and the test program conducted on them will now be discussed.

The purpose of this report is to evaluate the marginal quality aggregates which are in abundance along the Oregon Coast. Figures 8 and 9 show the relative amounts of these aggregates. To describe these marginal aggregates, extensive testing is required on these as well as quality aggregates for comparison purposes. Based on discussions with local aggregate users, various sources of both high and marginal quality aggregate were selected. These sources are listed in Table 10. Figure 10 shows the location of these sources.

The test program was conducted in two phases. Phase I consisted of the conventional tests previously described. From the results of this phase, aggregates were selected for testing in the repeated load test program of Phase II.

The selection of the types of tests to be conducted on the various aggregates should simulate in-service conditions. To date, laboratory evaluation of coastal aggregates has been limited primarily to the standard durability tests such as the California Durability, Oregon Aggregate Degradation, Washington Durability and the Los Angeles Abrasion tests. All but the last recognize the fact that some aggregates, basalts in particular, degrade much more rapidly in an environment with water present. These tests rely on agitation in water for an indication of the amount of chemical degradation or weathering that can be expected in service. The Los Angeles Abrasion Test is typically used to estimate

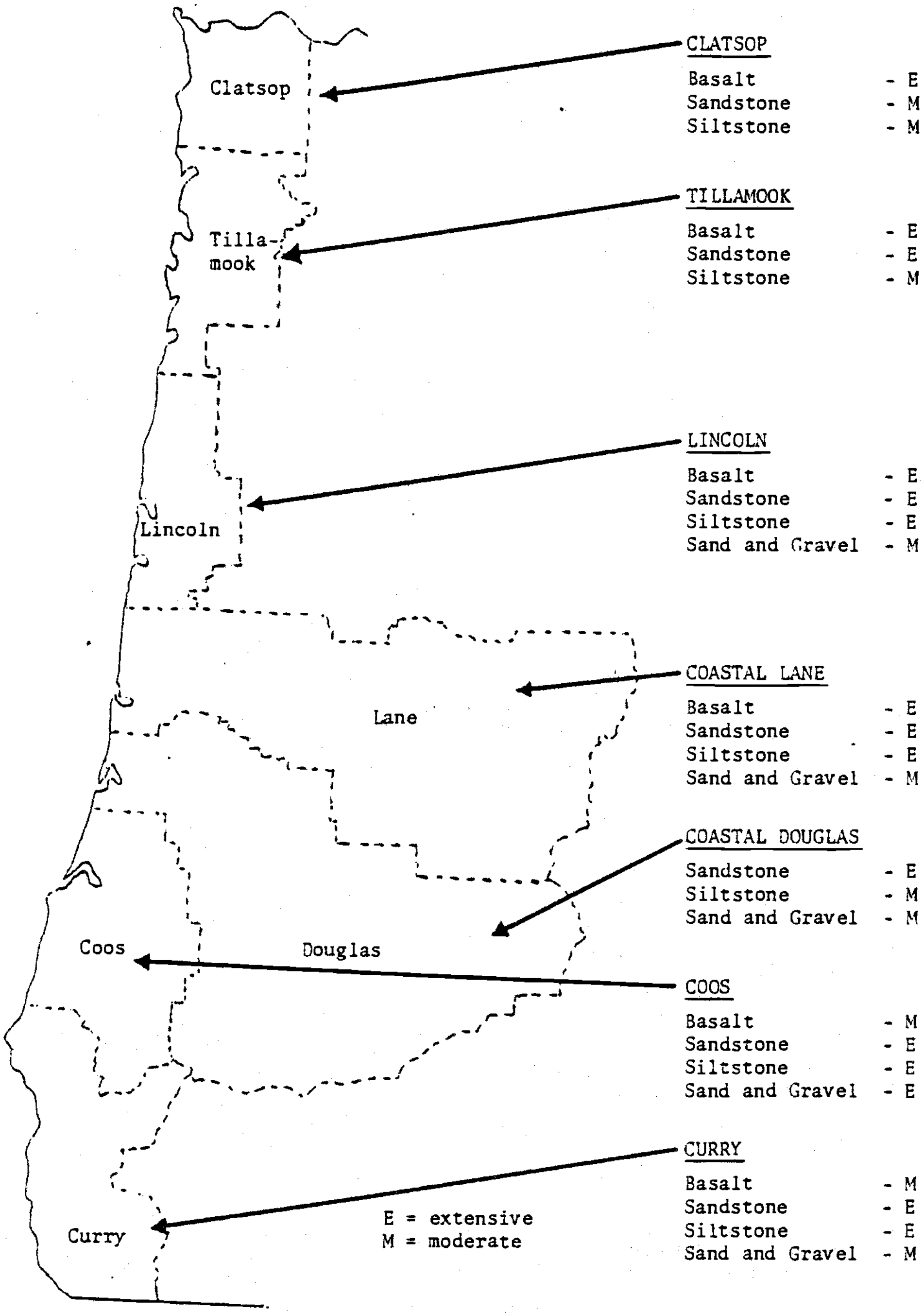

FIGURE 8. Availability of Land Based Marginal Aggregate in Oregon's Coastal Counties (Reference 33)

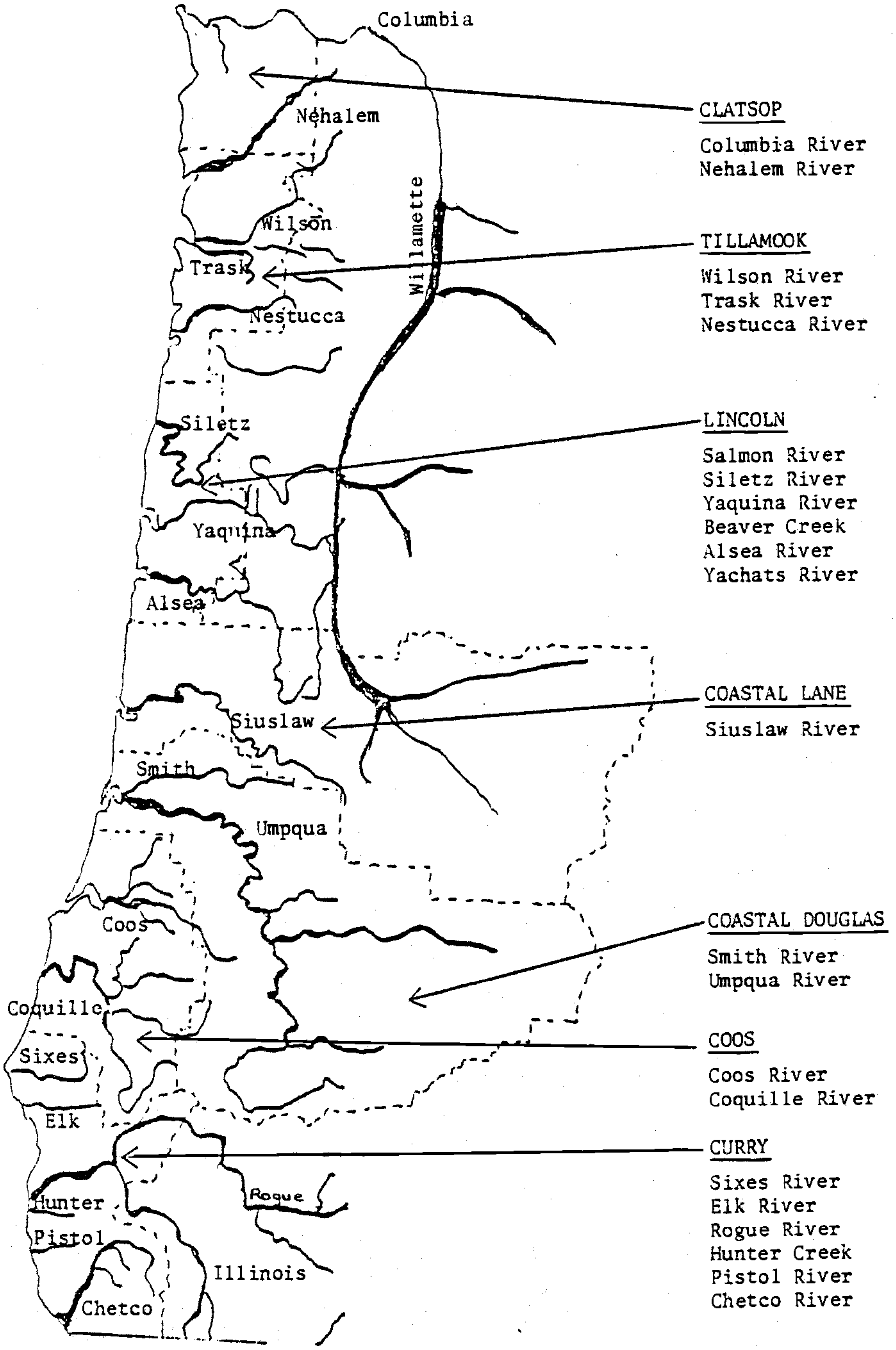

FIGURE 9. Availability of River Aggregate in Oregon's Coastal Counties (Source: Reference 33)

TABLE 10

SOURCES OF COASTAL AGGREGATE TO BE EVALUATED

Source	Type of Rock
Coffin Butte	Basalt
Berry Creek	Basalt
Ocean Lake	Basalt
Eckman Creek	Basalt
Yaquina Head	Basalt
Big A Cut	Sandstone
Morse Bros.	Siltstone
Toledo Sand & Gravel	Gravel (Basalt and Granite)

the amount of mechanical degradation that will occur. It is most applicable to the evaluation of rocks with low mechanical stability such as sandstone and siltstone.

Other, less common, tests applicable to Oregon's basalts include petrographic analysis and accelerated weathering with the aid of dimethyl sulfoxide (DMSO) or Ethylene Glycol (E.G.).

PHASE I

The preliminary test program, or Phase I, was conducted to provide a basis of selection of aggregates to be evaluated during Phase II. The tests conducted during Phase I included the chemical degradation indicators, i.e., California Durability, Oregon Aggregate Degradation, Washington Durability and DMSO and the mechanical degradation indicator test (Los Angeles Abrasion).

A field performance evaluation of these aggregates was considered, but because of rating subjectivity and lack of construction control, maintenance records and traffic data, this approach was rejected. This led to the development of Phase II of the test program.

PHASE II

This testing phase was designed to simulate the loading conditions present at the base level of a road section.

The development of repeated load testing apparatus in conjunction with the advancement of MultiLayer Analysis with the aid of computers has opened up a new realm of testing available to the highway materials engineer. With this type of test, any material can be subjected to a number of loading conditions. The number, frequency and duration of loads can be varied, as well as load intensity and confining pressure, so that a lifetime of service can be simulated in a relatively short time under precisely controlled laboratory conditions.

To investigate the performance of the selected aggregates, a combination of loading and environmental conditions was used to simulate the expected conditions in a typical road section. The aggregate samples were prepared to meet the United States Forest Service (Region 6) gradation requirements for an open-graded sample and a dense-graded sample (Table 11). The optimum density and moisture content were found by AASHTO Method T-180 (Modified Proctor). The sample was then prepared by vibratory compaction to this optimum condition and placed in the MTS (Materials Testing System) for repeated load testing. The sample preparation procedure and MTS operation is described in detail in Appendix A. For each rock type and each gradation, a sample was tested in a dry condition and a wet condition. The stresses at the interface between the subgrade and base caused by an 18 kip (80 kN) axle load were calculated for a 3-inch (7.62 cm) asphalt concrete wearing surface and a 12-inch (30.5 cm) base of unbound aggregate. Figure 11 shows the road section assumed. The method used, was a three-layer system analysis presented in "Principles of Pavement Design," by Yoder and Witczak (31). The modulus values assumed were for ideal conditions. The modulus for the asphalt concrete wearing course was assumed to be 300,000 psi (20,000 MN/m^2). The aggregate base modulus value was assumed to be 30,000 psi (2,000 MN/m^2) and the subgrade modulus was assumed to be 15,000 psi (1,000 MN/m^2). The calculated stress values resulted in a confining stress of 7 psi (480 kN/m^2) and an axial stress of 13 psi (900 kN/m^2)

TABLE 11

GRADATIONS USED FOR SAMPLE PREPARATION

Sieve Size	Percent Passing	
	Open-Graded	Dense-Graded
3/4	100%	100%
1/2	51%	-
No. 4	10%	43%
No. 10	3%	33%
No. 200	1%	6%

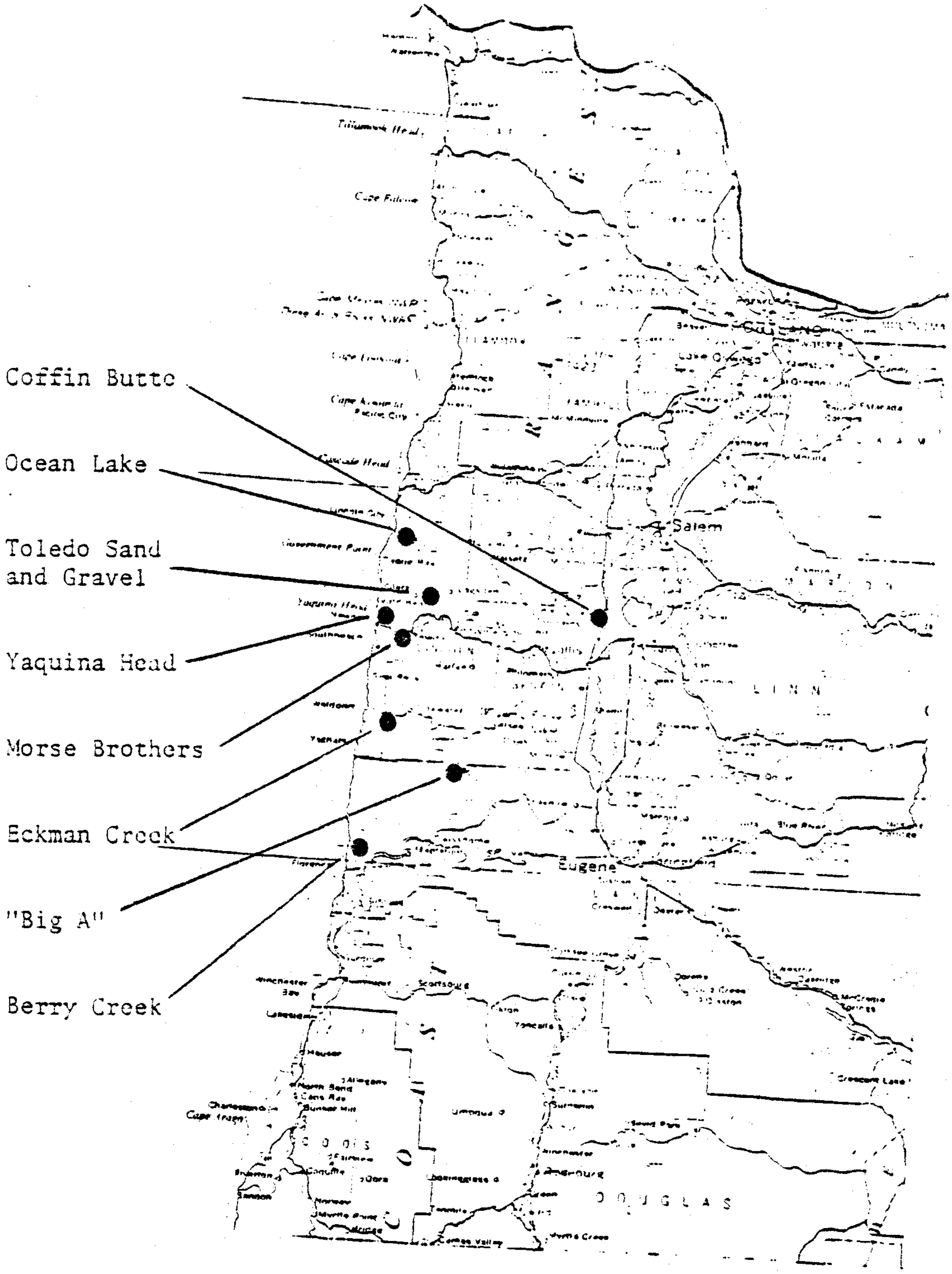

FIGURE 10. Location of Aggregates Tested

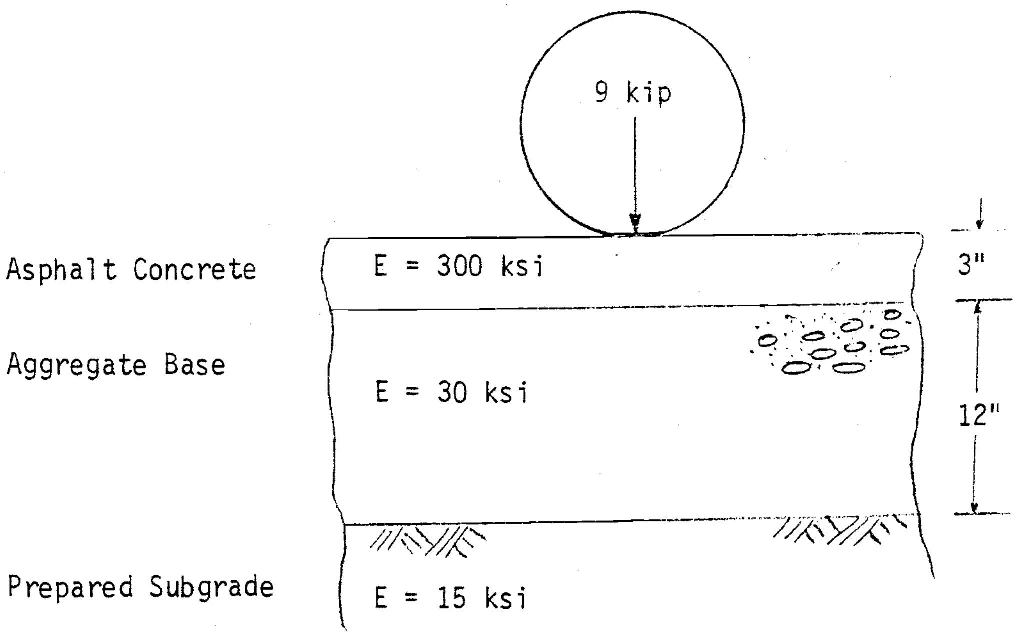

Figure 11. Road Section Used in Calculating Stress Conditions to use for Permanent Deformation

at the bottom of the base. These values were felt to be somewhat lower than what would be expected under adverse conditions. Also, the greatest stress will occur at the top of the base. For this reason, the values used during testing were somewhat higher than these results.

The main thrust of this phase of testing was to measure: 1) the amount of permanent deformation or plastic strain that would occur under repeated loading; 2) the resilient modulus, or stiffness, of the specimen; and 3) the change in surface area of the sample after modified proctor compaction, vibratory compaction and repeated load testing. Comparisons of the performances for the different gradations and moisture contents and for the different types of rocks were then made.

The calculation of the before treatment surface area is shown in Table 12. Calculation of after treatment surface areas were conducted by breaking down the tested specimen, drying if necessary, resieving and calculating the resulting surface areas as done in Table 12.

The permanent deformations of the aggregate samples were measured throughout the testing period by reading a dial gauge attached to the MTS load cell. Readings were possible to the nearest 0.001 inch (.0254 mm) with interpolation to the nearest 0.0001 inch (.00254 mm). The MTS was programmed to apply its repeated load once every two seconds for a duration of 0.12 seconds, corresponding to a wheel passing over at approximately 15 mph (24 km/hr) (37). This provided a recovery time of 1.88 seconds which was felt to be sufficient to allow full recovery of elastic deformation so that only plastic, or permanent, deformation was present at the beginning of each new load application. Initial buildup of permanent deformation was high for all aggregate samples with a

TABLE 12

SURFACE AREA CALCULATION

STANDARD SURFACE AREA, DENSE-GRADED (BEFORE COMPACTION)

Size	3/4"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
% Passing	100	.58	.43	.35	.27	.21	.21	.10	.06
SA Factor	2	2	2	4	8	14	30	60	160
Surface Area	2	1.16	.86	1.40	2.16	2.94	4.50	6	9.60

Total Surface Area = 30.62 ft^2/lb

STANDARD SURFACE AREA, OPEN-GRADED (BEFORE COMPACTION)

Size	3/4"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
% Passing	100	.32	.10	.04	.02	.015	.014	.012	.010
SA Factor	2	2	2	4	8	14	30	60	160
Surface Area	2	.64	.20	.16	.16	.21	.42	.72	1.60

Total Surface Area = 6.11 ft^2/lb

subsequent slowing of the rate of increase of deformation. From the change in sample height thus obtained, the plastic strain was calculated. A plot of plastic strain versus total number of cycles showed that the response is approximately linear on a semilog graph. The sample was allowed to undergo approximately 35,000 load repetitions at a confining stress of 10 psi (690 kN/m^2), and an axial stress of 35 psi (2400 kN/m^2). These values were used to approximate the average stress condition, at mid-base level, when subjected to truck loading.

After this sequence of loading, the sample was readied for the resilient modulus testing by attaching the linear variable differential transformers. These LVDT's allowed accurate measurement of the elastic deformation that occurred with each load application. Measurement of this deformation was done on a two-channel Hewlett Packard Oscillographic Recorder (Model 7402) with the other channel used for measuring the applied load. An example of the readout is shown in Figure 12. The resilient modulus was calculated by dividing the deviator stress (axial stress (σ_1) minus confining stress (σ_3) by the resulting axial elastic strain. This was done for four different confining stress levels and four different stress ratios (σ_1/σ_3) at each confining stress level to account for a wide array of possible loading conditions. The confining stresses used for resilient modulus calculations were 15 psi (1035 kN/m^2), 10 psi (690 kN/m^2), 5 psi (345 kN/m^2), and 3 psi (115 kN/m^2). The stress ratios used were 3.5, 3.0, 2.5 and 2.0. The sample was preconditioned for approximately 1000 repetitions at the highest confining stress and stress ratio. Even though the sample had been subjected to approximately 35,000 loading cycles at σ_3 = 10 psi (690 kN/m^2)

and σ_1 = 35 psi (2415 kN/m^2), the introduction of the heavier loading condition of σ_3 = 15 psi (1035 kN/m^2) and σ_1 = 52.5 psi (3622 kN/m^2) resulted in additional plastic deformation. Also, the initial values of resilient modulus at this new stress level were significantly lower than the values obtained after several hundred repetitions. Therefore, the sample was allowed to stabilize for approximately 1,000 cycles before measurements were begun. After resilient modulus testing at all combinations of confining stress and stress ratio, the highest level was repeated for verification of accuracy and correct seating of the LVDT's.

To summarize the test program, Figure 13 was prepared. The purpose of the testing was to demonstrate the difference in performance between high quality and marginal quality construction aggregates found on the Oregon Coast. The influence of water and gradation on the performance of the aggregates was also determined. Once the conventional testing of Phase I was completed, the aggregates to be tested in Phase II were selected. These were to be a high-quality basalt and a marginal quality basalt and sandstone. The effect of blending equal proportions of the high-quality and marginal quality basalts was also to be investigated.

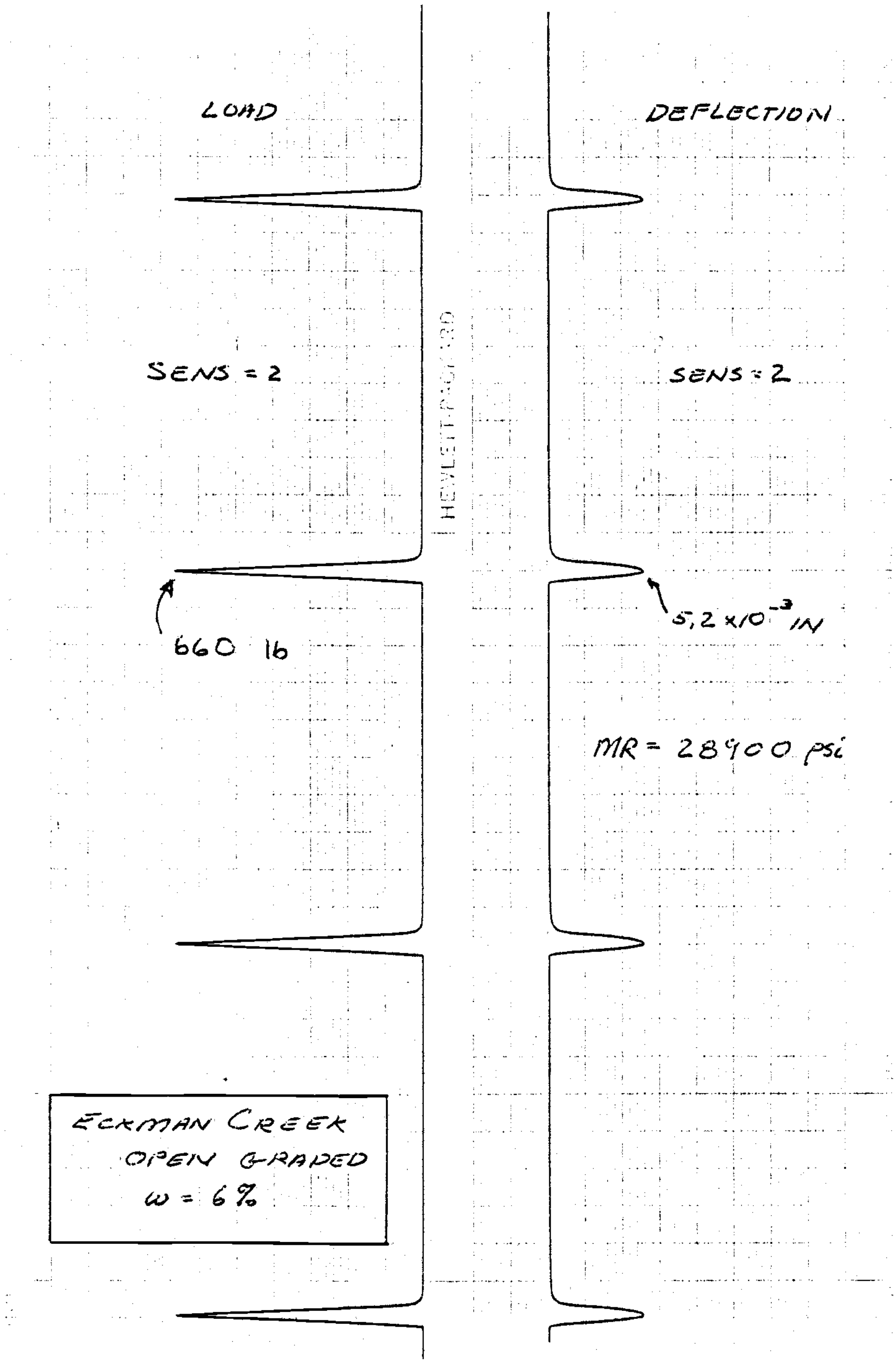

FIGURE 12. Deflection of an Aggregate Sample Resulting from an Applied Axial Stress of 52.5 psi (3625 kN/m^2) and a Confining Stress of 15 psi (1035 kN/m^2).

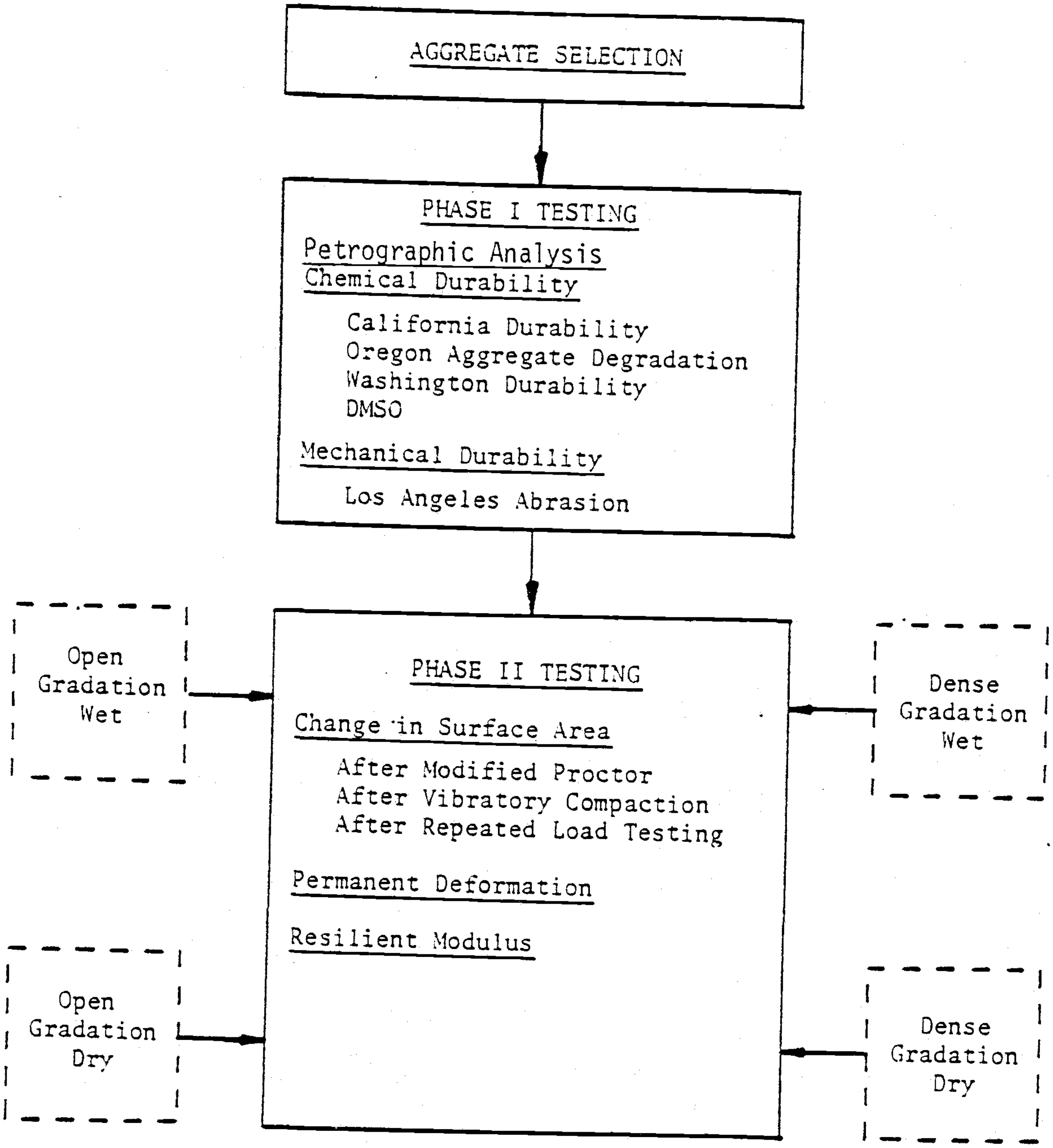

FIGURE 13. Test Program for Evaluation of Coastal Oregon's Marginal Aggregates

CHAPTER 5. RESULTS

The purpose of this chapter is to summarize the results of the two phases of testing.

PHASE I

The standard durability test results obtained in Phase I are shown in Table 13. The results of the petrographic analysis are shown in Appendix C.

From the results shown in Table 13, it is evident that the aggregate sample from Eckman Creek quarry is low quality, failing every test except the Los Angeles Abrasion test, which was borderline. In particular, it performed very poorly in the DMSO test and Washington Durability test. This rock is from a marine basalt deposit and exhibits numerous vesicles or void spaces which would indicate extremely rapid cooling. These vesicles occurred when the volcanic gases, attempting to rise to the surface of the liquefied deposit, were "frozen" in place. The rapid cooling resulted in a large amount of volcanic glass being formed. Volcanic glass has been shown to be detrimental to the performance of aggregate for road building purposes. It can alter to secondary clay minerals such as the expanding smectite type clays. The presence of a large amount of vesicles indicates an abundant amount of surface area available for reaction with water. This combination of large surface area and abundant glass explains the relatively poor performance in the conventional durability tests.

The Ocean Lake basalt performed well on all of the Phase I tests.

TABLE 13

SUMMARY OF DURABILITY TESTS

Rock Source	Type	Cal. Dur D_c	Cal. Dur D_f	OAD H	OAD % Pass	Wash. Dur.	DMSO	LAR
Coffin Butte	Basalt	78	57	0.9	18	56	7	16.8
Berry Creek	Basalt	68	35	2.9	23	28	4	27
Ocean Lake	Basalt	81	43	1.4	16	66	0	13
Eckman Creek	Basalt	26	44	10.5	37	16	10	34
Yaquina Head	Basalt	74	49	2.6	17	29	3	
"Big A"	Sandstone	33	19	4.6	92	36	Not applicable	95
Morse Bros.	Siltstone	<30	11	15	85	<30	Not applicable	63
Toledo Sand and Gravel	Gravel (Basalt and Granite)	68	77	1.4	15.3	28	4	16
Typical Acceptance Values		25 (min)	35 (min)	3.5 Max	35 Max	50 Min	4 Max	35 Max

For this reason it was selected as the high-quality aggregate to be tested as a control in Phase II.

The two sedimentary rocks selected ("Big A" sandstone and Morse Bros. Siltstone) both performed poorly in the Phase I test program. In fact, the tendency to degrade under relatively light physical agitation (i.e., mechanical sieving) presented problems throughout both phases of testing. The size comparisons used for the Oregon Aggregate Degradation (% passing the No. 140 sieve after agitation) and the Los Angeles Abrasion test were uncontrollably biased because of this weak mechanical strength. The "Big A" sandstone was selected for further testing because it appeared to be slightly stronger than the siltstone.

In general, the results of the Phase I test sequence were in the same approximate range as shown in Tables 3 through 7.

PHASE II

The purpose of this section of the report is to describe the permanent deformation, stiffness, and degradation characteristics of the aggregates when subjected to the different loading conditions described in Chapter 3. The aggregates tested in this phase were: Eckman Creek basalt, Ocean Lake basalt, "Big A" sandstone, and an equally proportioned blend of the two basalts. Each aggregate was tested in an open gradation and a dense gradation. For each gradation, several specimens were prepared in a wet and dry condition so that the degradation could be measured after the different loadings (modified proctor compaction, vibratory compaction and repeated load testing) had been applied.

To determine the optimum density and moisture content, modified proctor compaction (AASHTO T-180) was performed. These results are summarized in Table 14 and the curves are furnished in Appendix B. Some general conclusions can be reached from Table 14. The high-quality Ocean Lake basalt has slightly higher maximum dry densities than the other rocks for both the open and dense gradations. Also, the optimum moisture contents were lower for the Ocean Lake samples. This most probably results from the higher specific gravity of this rock as compared to the more porous Eckman Creek basalt and "Big A" sandstone. Huddleston (19) found the saturated surface dry (SSD) specific gravity to be 2.55 and 2.31 for the Eckman Creek basalt and "Big A" sandstone, respectively. From Table 3 it was found that the specific gravity (SSD) for the Ocean Lake basalt was 2.87.

Table 15 shows the density and moisture conditions of the aggregate samples that were subjected to repeated load testing.

Degradation Results

The results of the degradation analysis are presented in Figures 14 through 21. These results are quantified by means of surface area calculations in Table 16. From these results, the following general observations can be made:

1) Aggregate from Ocean Lake, a high-quality rock as determined in Phase I, proved to resist degradation much more readily than either the "Big A" sandstone or the Eckman Creek basalt.
2) More degradation occurred in an open-graded aggregate than a dense-graded aggregate.
3) The wet sandstone, to a significant degree, and the wet Eckman Creek basalt, to a lesser extent, tended to breakdown substan-

TABLE 14

RESULTS OF MODIFIED PROCTOR COMPACTION

Source	Gradation	Optimum Moisture	Maximum Dry Density
Ocean Lake	Open	1.5%	122 lb/ft^3 (1960 kg/m^3)
	Dense	5.0%	144 lb/ft^3 (2300 kg/m^3)
Eckman Creek	Open	6.5%	116 lb/ft^3 (1800 kg/m^3)
	Dense	6.3%	129 lb/ft^3 (2068 kg/m^3)
"Big A"	Open	7.0%	117 lb/ft^3 (1875 kg/m^3)
	Dense	8.0%	121 lb/ft^3 (1940 kg/m^3)
Blend	Open	4.0%	119 lb/ft^3 (1915 kg/m^3)
	Dense	6.0%	134 lb/ft^3 (2150 kg/m^3)

TABLE 15

MOISTURE CONTENTS, w, AND PERCENT OF MAXIMUM DENSITY, γ_d OF AGGREGATE SAMPLES USED IN REPEATED LOAD TESTING

Source	Dry	Wet
Ocean Lake		
Open	γ_d = 101% w = 1.5%	γ_d = 100% w = 6.5%
Dense	γ_d = 104% w = 1.5%	γ_d = 100% w = 5.5%
Big A		
Open	γ_d = 100% w = 2.0%	γ_d = 100% w = 11.2%
Dense	γ_d = 103% w = 2.0%	γ_d = 99% w = 8.9%
Eckman Creek		
Open	γ_d = 99% w = 2.5%	γ_d = 99% w = 8.6%
Dense	γ_d = 100% w = 2.5%	γ_d = 100% w = 9.0%
Blend		
Open	γ_d = 99% w = 2.0%	γ_d = 96.3% w = 5.0%
Dense	γ_d = 103% w = 2.0%	γ_d = 100% w = 10.4%

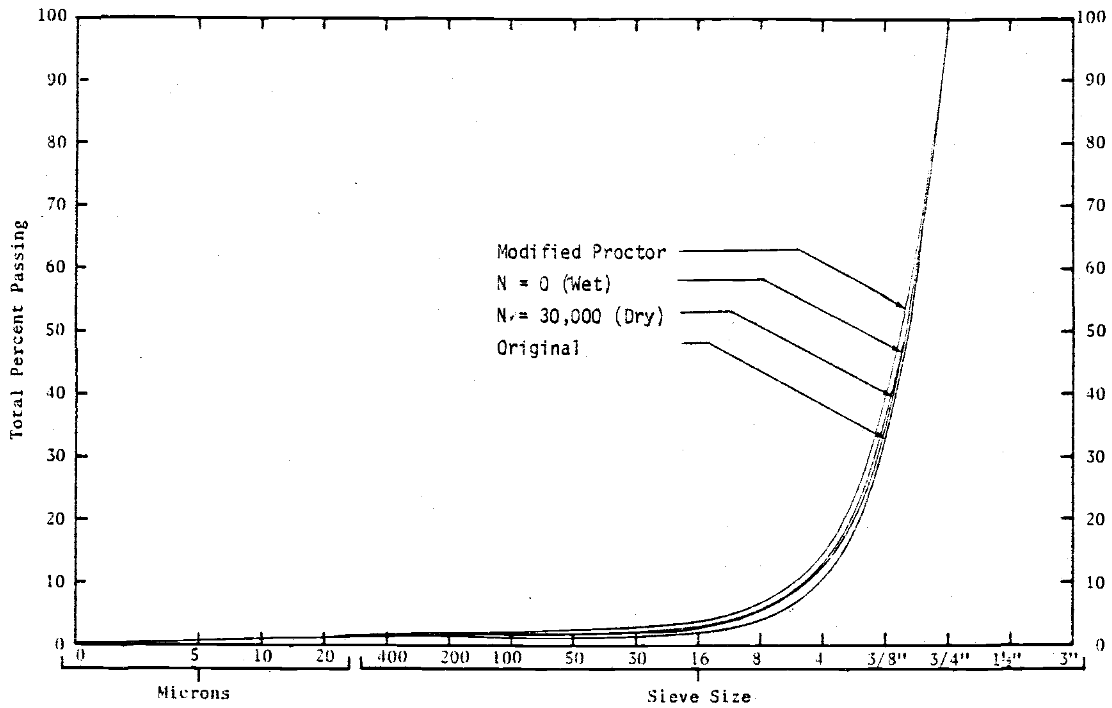

FIGURE 14. Grading Analysis: Open Graded Ocean Lake

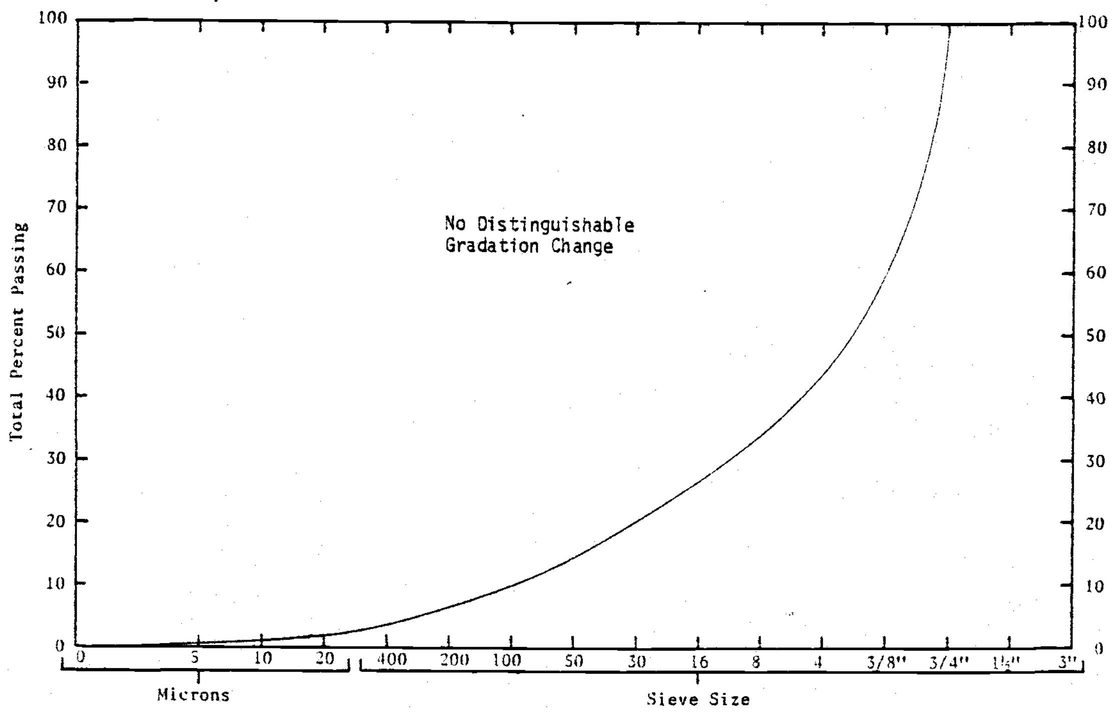

FIGURE 15. Grading Analysis: Dense Graded Ocean Lake

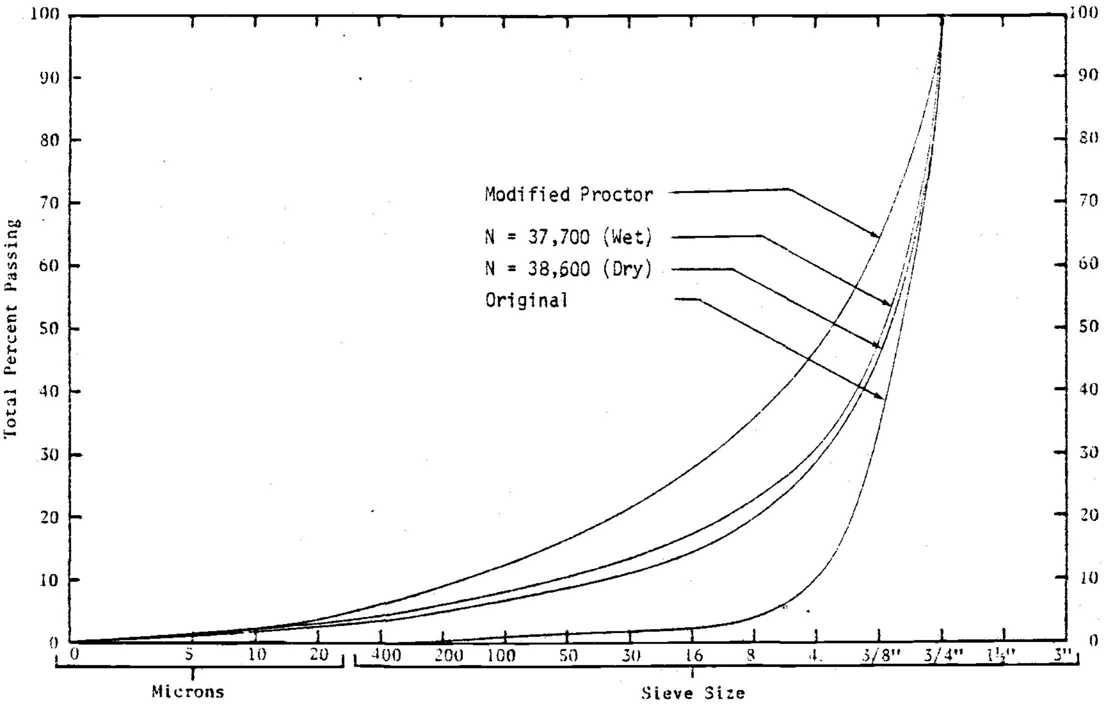

FIGURE 16. Grading Analysis: Open Graded "Big A"

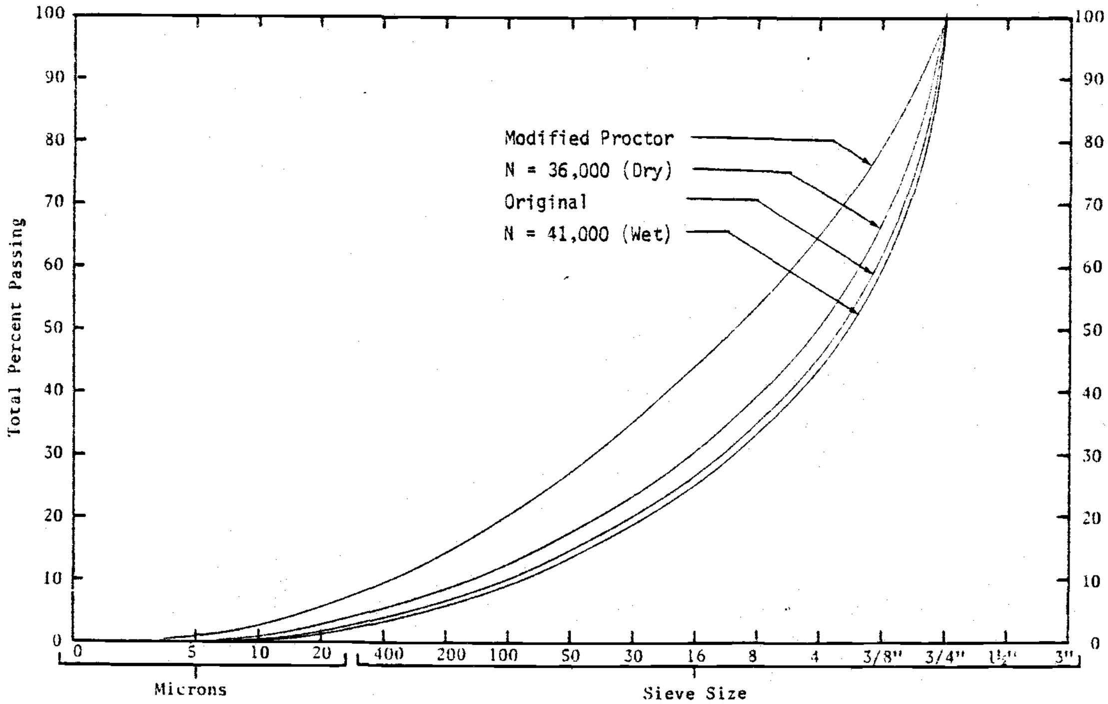

FIGURE 17. Grading Analysis: Dense Graded "Big A"

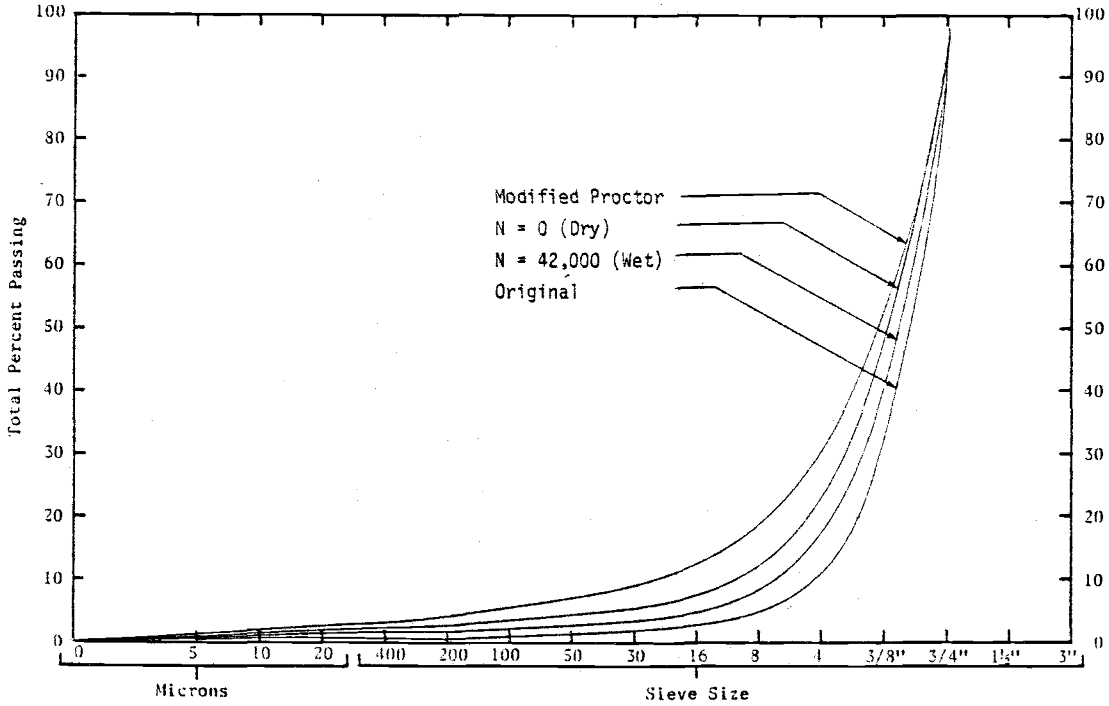

FIGURE 18. Grading Analysis: Open Graded Eckman Creek

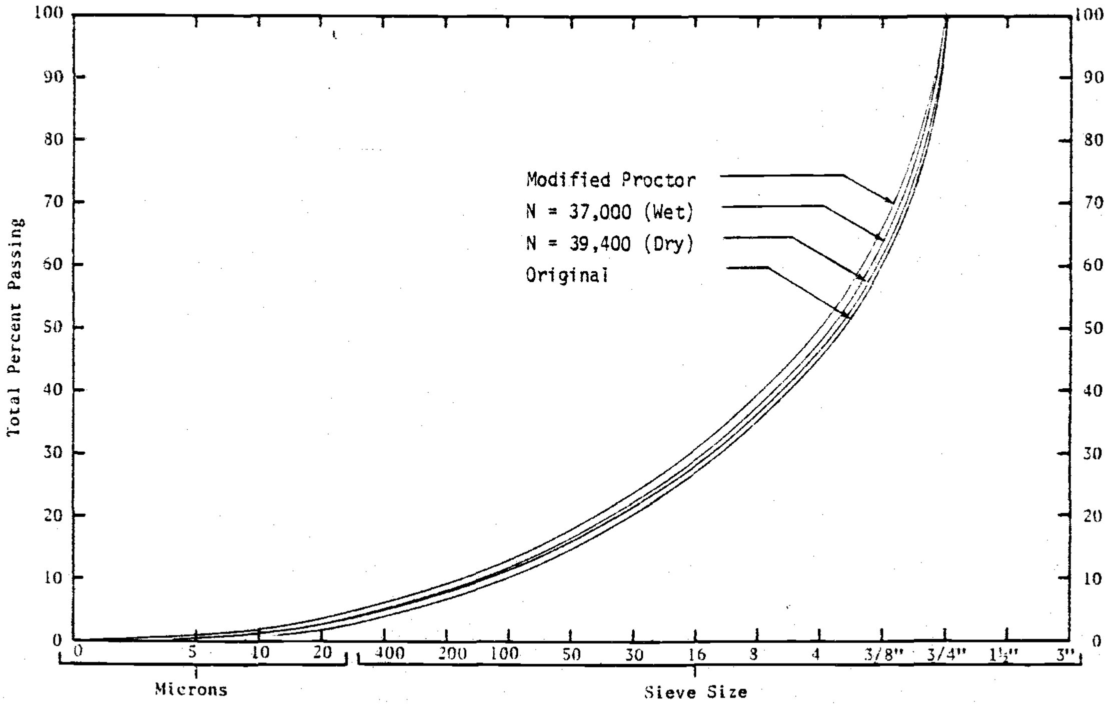

FIGURE 19. Grading Analysis: Dense Graded Eckman Creek

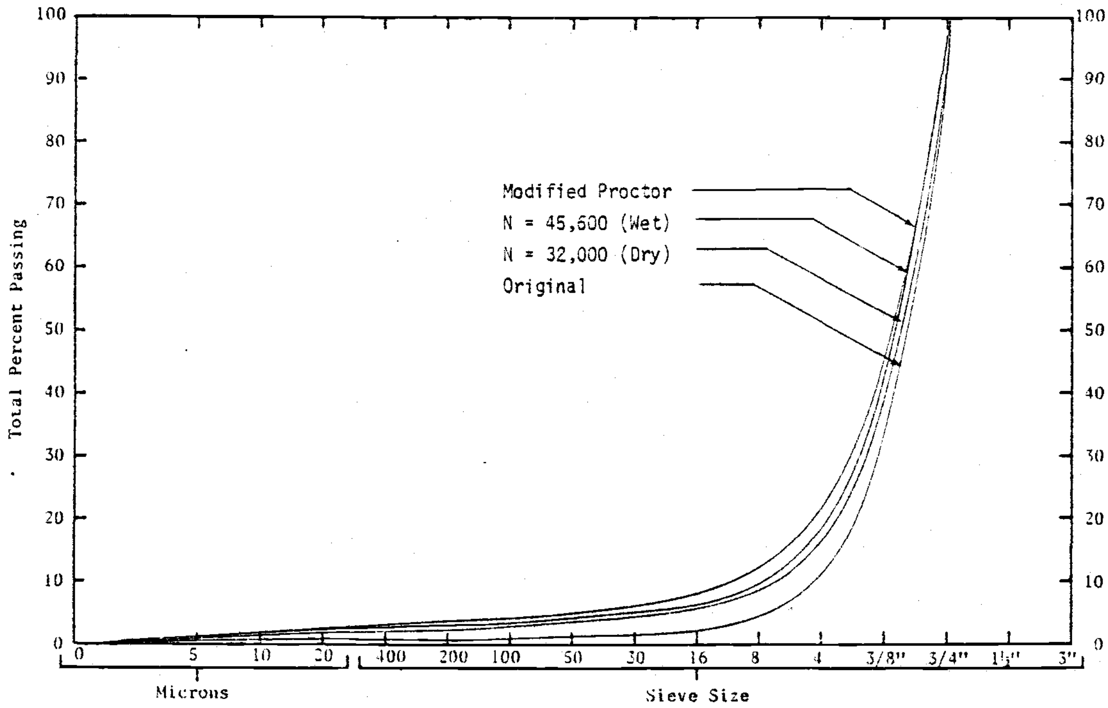

FIGURE 20. Grading Analysis: Open Graded Blend

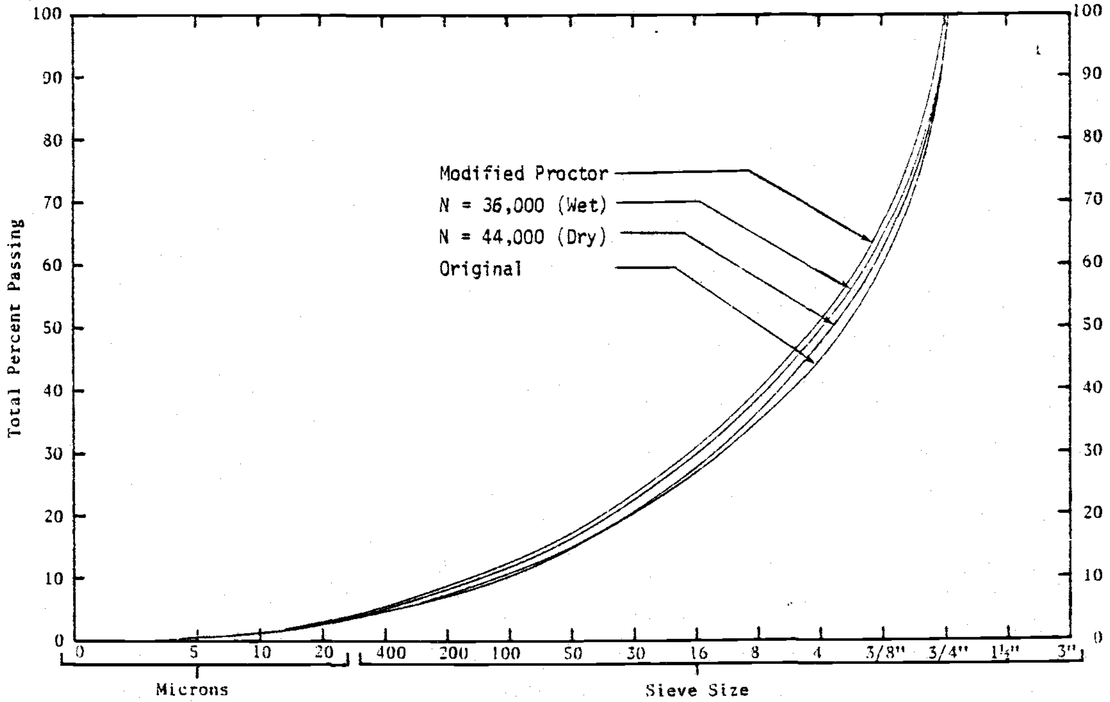

FIGURE 21. Grading Analysis: Dense Graded Blend

TABLE 16

RESULTS OF DEGRADATION ANALYSIS

Source	Gradation	Original Surface Area lbs/ft^2 (kg/m^2)	% Change in Surface Area After: Modified Proctor	Vibratory Compaction Dry	Vibratory Compaction Wet	Repeated Loading (N≈35000) Dry	Repeated Loading (N≈35000) Wet
Ocean Lake	Open	6.11 (1.25)	62%*	90%	29%**	12%	-34%**
	Dense	30.62 (6.27)	4%	0.59%	0.98%	5.9%	-15.4%**
"Big A"	Open	6.11 (1.25)	337%	222%	285%	388%	192%**
	Dense	30.62 (6.27)	86%	14%	-52%**	0.9%	-18%**
Eckman Creek	Open	6.11 (1.25)	306%	64%	78%	31%	46%
	Dense	30.62 (6.27)	4.9%	2.9%	1.2%	2.2%	3.2%
Blend	Open	6.11 (1.25)	102%	28%	62%	55%	61%
	Dense	30.62 (6.27)	11%	10%	-18%**	10%	12%

* Sample originally prepared to a much more open gradation without fines.

** Re-cementing of aggregate upon drying results in underestimation of change in surface area.

tially but recement when drying. This was visibly noticeable upon sample breakdown but was virtually unavoidable. The potential change in surface areas for these two samples would be significantly higher in actual practice.

4) Compaction by modified proctor produced, as expected, the largest amount of breakdown in aggregate grain size. Vibratory compaction resulted in less breakdown, in general. Additional breakdown due to repeated loading in the MTS produced only a small change in surface area.

5) Under heavy loading conditions, as in the modified proctor compaction, the benefit of blending Eckman Creek basalt with Ocean Lake basalt was appreciable for an open-graded mix. Further testing is required to substantiate the premise that blending is a cost-effective way of easing the problems of using low-quality aggregates.

Plastic Strain Results

The results of the plastic strain measurements are plotted in Figures 22 through 29. In an effort to standardize the results, the plastic strain (ε_p) was calculated by using the dial reading at the tenth load repetition as the reference reading. This provided each sample with sufficient time to remove any surface irregularities that may have given a false plastic strain value. This reading was then subtracted from each subsequent dial reading to obtain the permanent deformation. Dividing this value by the specimen length yielded the plastic strain. This value is of importance in designing road bases and

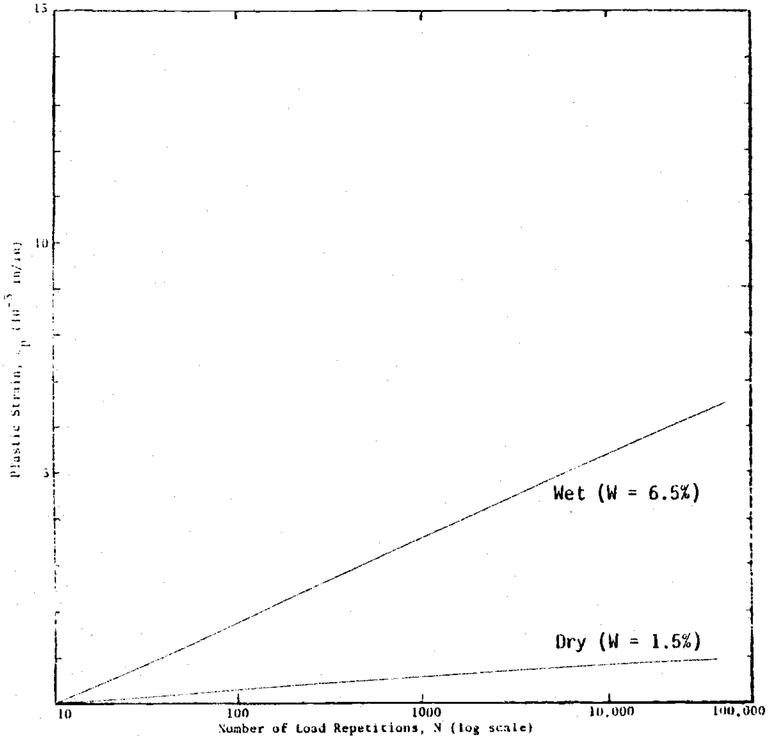

FIGURE 22. Cumulative Plastic Strain as a Function of Number of Loa
Repetitions: Open Graded Ocean Lake

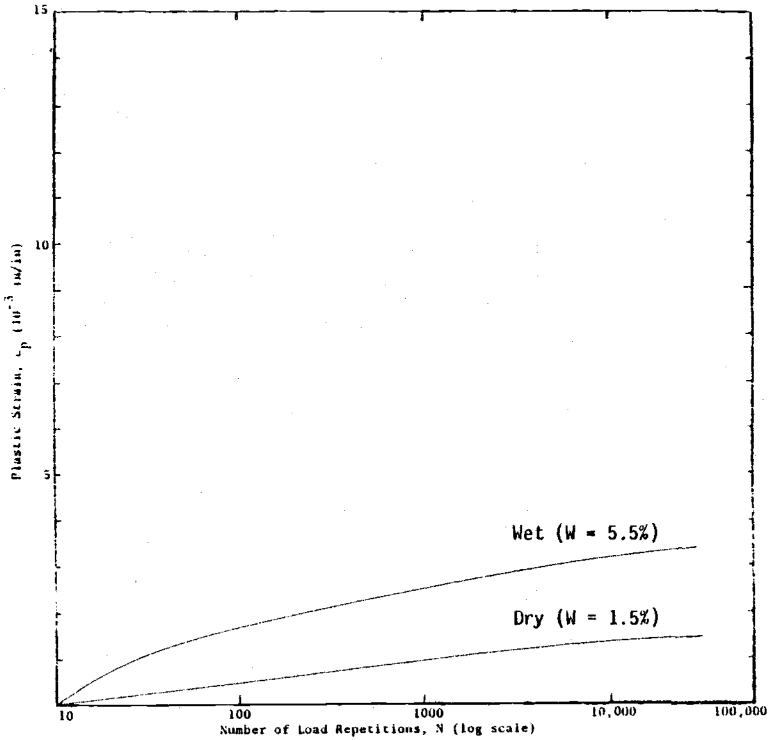

FIGURE 23. Cumulative Plastic Strain as a Function of Number of
Load Repetitions: Dense Graded Ocean Lake

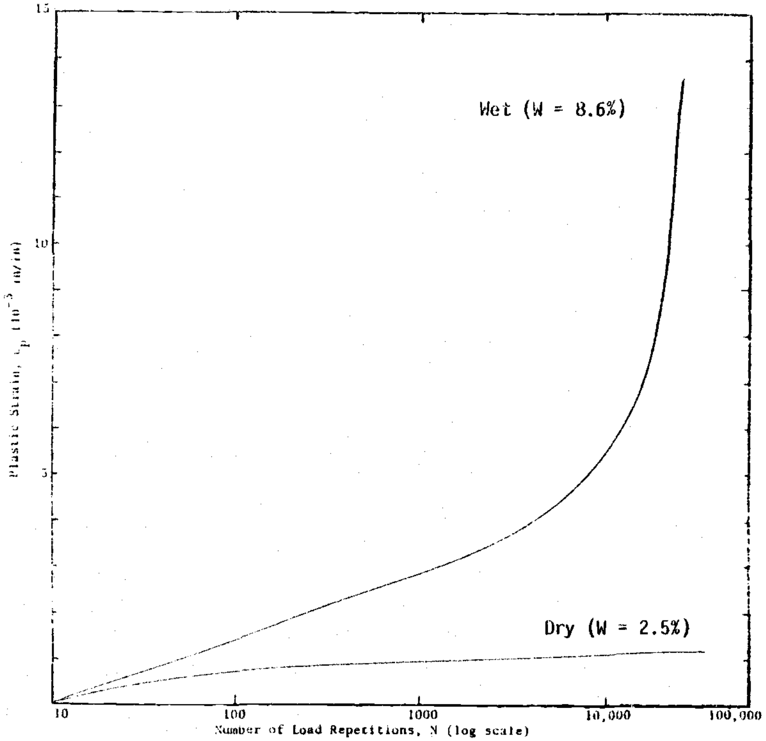

FIGURE 24. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Open Graded Eckman Creek

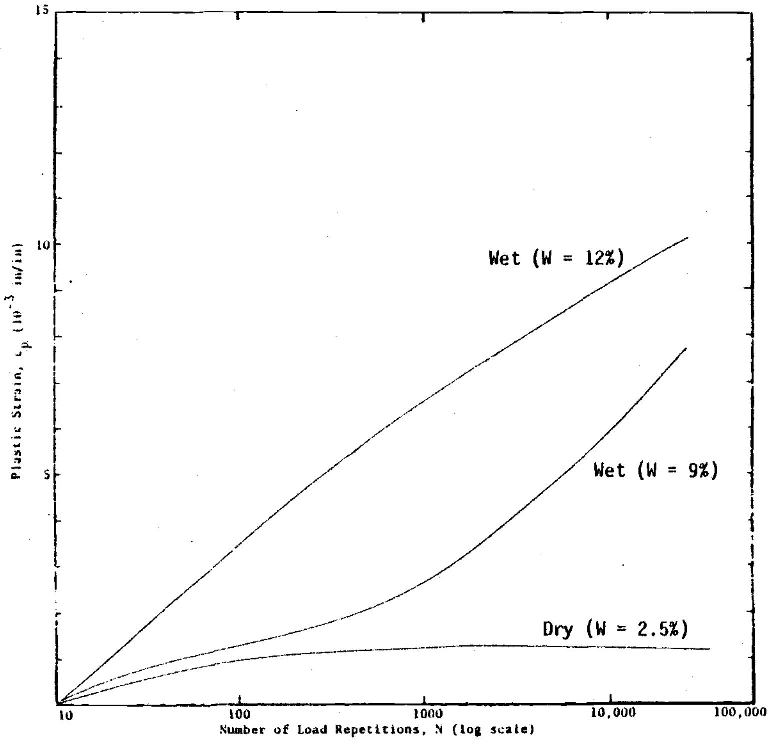

FIGURE 25. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Dense Graded Eckman Creek

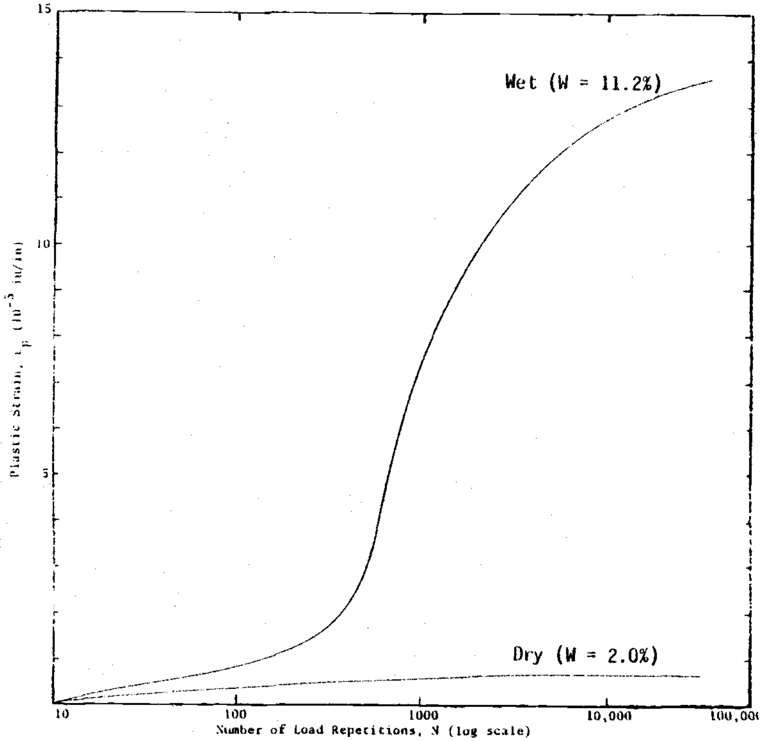

FIGURE 26. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Open Graded "Big A"

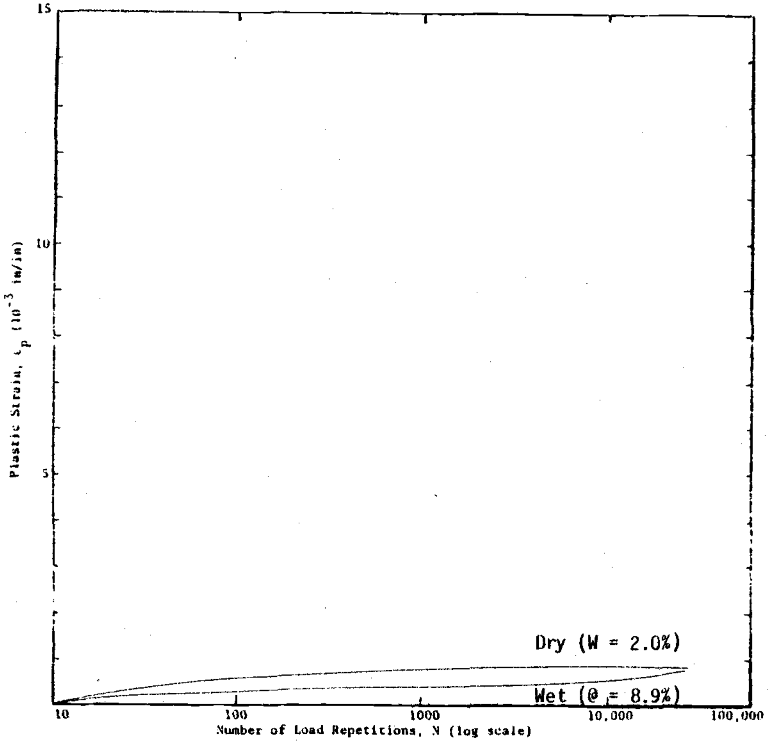

FIGURE 27. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Dense Graded "BIG A"

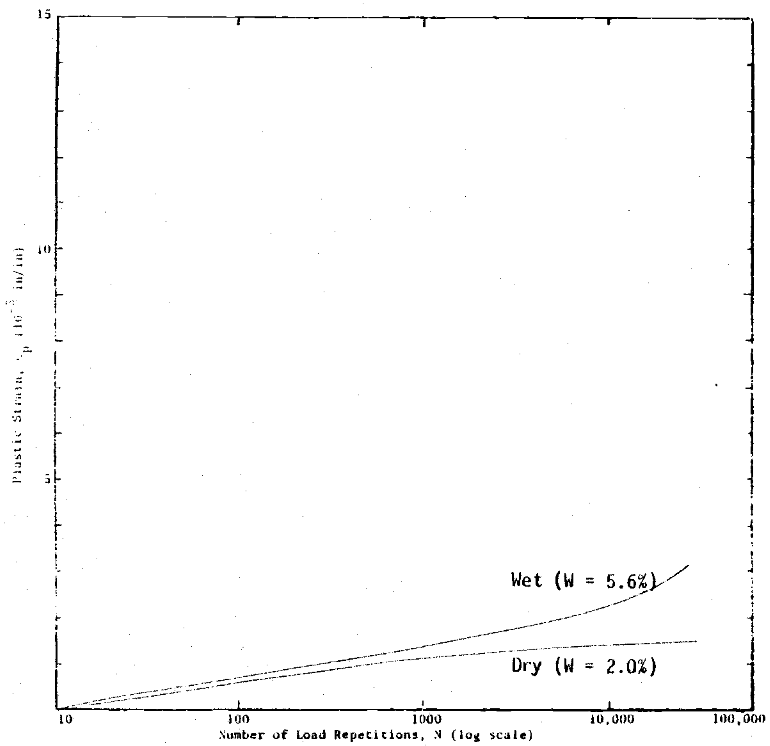

FIGURE 28. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Open Graded Blend

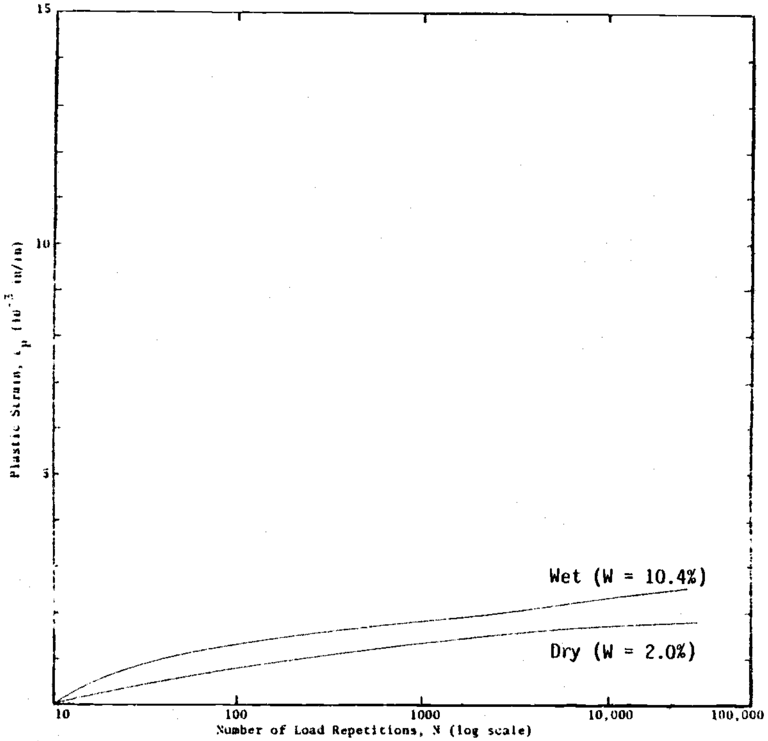

FIGURE 29. Cumulative Plastic Strain as a Function of Number of Load Repetitions: Dense Graded Blend

subgrades in order to predict rutting potential.

Table 17 summarizes this plastic strain value for each aggregate after 35,000 cycles of loading. These values demonstrate the adverse effects of water in any road base. In all but one case ("Big A" dense) the plastic strain experienced in a wet environment was significantly greater than that experienced in a dry state. The performance of the dense-graded sandstone may be attributed to the fact that the wet sample, when compacted, broke down to such a degree that an extremely dense specimen resulted. The amount of voids available to accommodate permanent deformation were practically nonexistent. This is not, however, a desirable condition to have in a roadway. Differential stresses experienced when a wheel passes over this section would almost certainly result in breaking of the bond that was developed.

As seen in Table 17, plastic strain values for all dry samples is approximately the same with the average being 1.20×10^{-3} in/in. The average plastic strain for the wet samples was about 5.25 times as great as the dry samples. Unlike the dry samples, however, twice the plastic strain was experienced in the wet open-graded samples as the wet dense-graded samples. Deleting the unusual results of the densely-graded "Big A" did not significantly alter the overall results for the wet specimens. Of particular interest was the performance of the low-quality aggregates in a wet condition. They tended to experience large amounts of plastic strain. This would appear to substantiate the findings of the conventional durability tests performed in Phase I.

TABLE 17

PLASTIC STRAIN (IN/IN x 10^{-3}) AFTER 35,000 LOADING REPETITIONS AT σ_1 = 35 psi and σ_3 = 10 psi

Source	Wet		Dry	
	Dense	Open	Dense	Open
Ocean Lake	3.45	6.25	1.40	0.96
Eckman Creek	10.2	11.0	1.35	1.24
"Big A"	0.60	13.20	0.70	0.60
Blend	2.55	3.20	1.80	1.55
Averages	4.20	8.41	1.31	1.09
	6.31		1.20	

The blended specimen produced good results with respect to plastic strain. In fact, the blend experienced less strain than the quality aggregate from Ocean Lake. Repeat testing would be required to determine, with confidence, if blending is a feasible alternative to easing local aggregate shortages.

In general, the measurement of plastic strain proved to be extremely sensitive to moisture conditions. Figure 25, the plastic strain curve for dense-graded Eckman Creek basalt, demonstrates the effect of additional moisture. Figure 30 compares the results obtained by Barksdale (41) with the results for the dense-graded Ocean Lake basalt. They are approximately equal.

Resilient Modulus Results

The resilient modulus is a measure of the amount of elastic strain for a given stress condition. It is expressed, mathematically, as follows:

$$M_R = \sigma_d / e_e \tag{1}$$

where $M_R \equiv$ Resilient Modulus (psi)

$\sigma_d \equiv$ deviation stress (psi)

$= \sigma_1 - \sigma_3$ (psi) (axial minus confining stresses)

$e_e \equiv$ elastic strain

There are two conventional methods of expressing resilient modulus

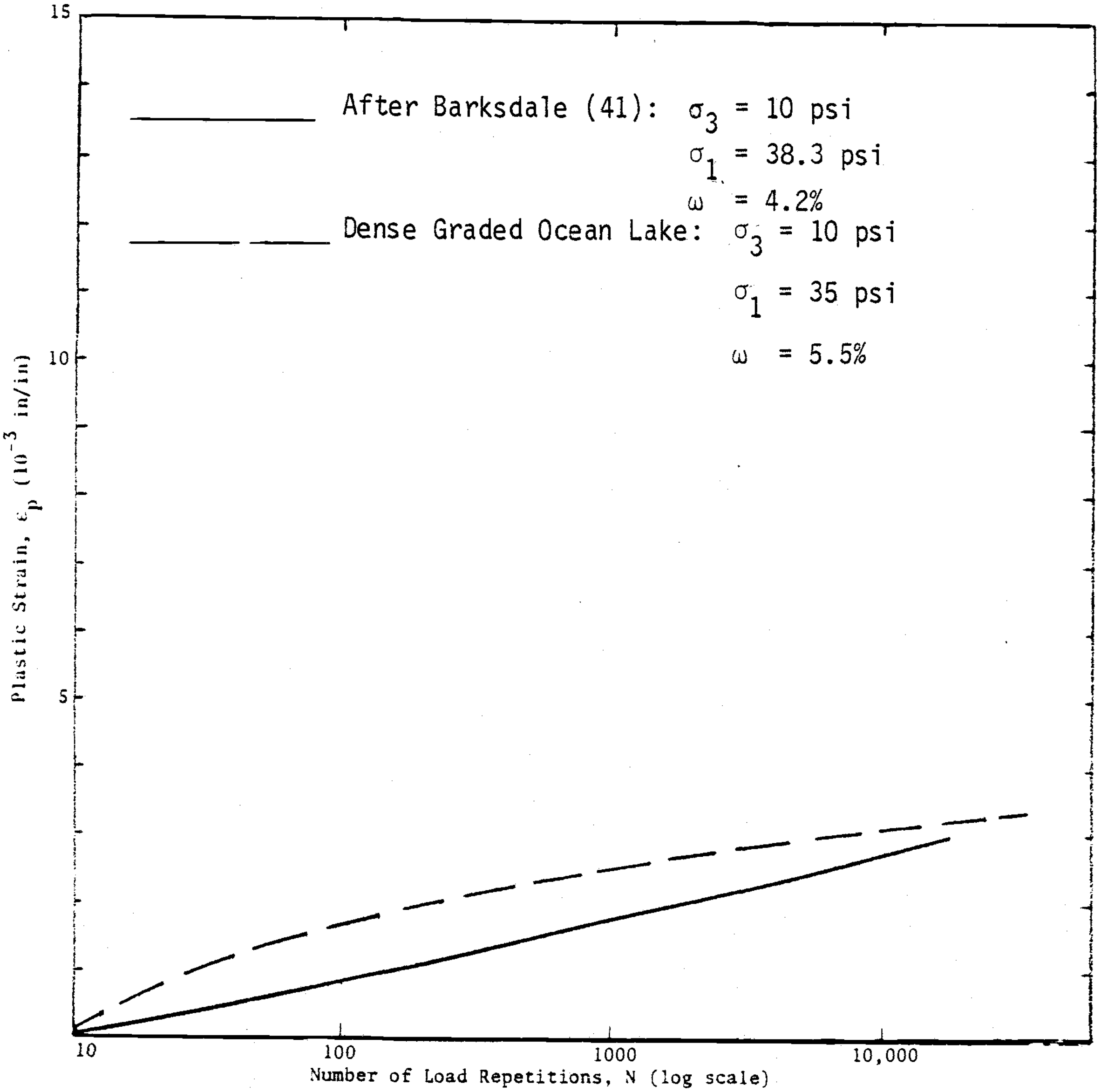

FIGURE 30. Comparison of Development of Permanent Deformation for Dense Graded Ocean Lake and Results Obtained by Barksdale (41)

response to varying stress conditions (32). These are as follows:

$$M_R = k_1 \sigma_3^{k_2} \tag{2}$$

where k_1 and k_2 are regression constants

$$M_R = k_1' \theta^{k_2'} \tag{3}$$

where $\theta \equiv$ sum of principal stresses

$= \sigma_1 + 2\sigma_3$

k_1' and k_2' are regression constants.

The results of regressing the individual resilient modulus values against σ_3 and then θ are shown in Table 18. Upon plotting the results on Log-Log graph paper it was noticed that there was a definite pattern to the variation about the regression line, suggesting the action of a variable that was not accounted for. This is shown for the wet, open-graded sample for Ocean Lake in Figures 31 and 32. All samples tested followed this pattern. In an attempt to account for this variation, linear multiple regression was performed by regressing resilient modulus against confining stress and axial stress. The results of this regression are included in Table 18.

The r-squared value for each regression equation is given in Table 18. This value is a measure of the amount of variation that has been removed by using the particular equation developed. Values of 1.00 indicate that the data exactly fits the model, while values of zero indicate no correlation. Regressing resilient modulus against confining

TABLE 18

SUMMARY OF RESILIENT MODULUS TESTING

Ocean Lake

Open-Graded

Dry

$M_R = 6363\ \sigma_3^{.6562}\ (r^2 = .819)$

$M_R = 2053\ \theta^{.6888}\ (r^2 = .936)$

$M_R = 7573 + 391\sigma_1 = 350\sigma_3\ (r^2 = .99)$

Wet

$M_R = 5725\ \sigma_3^{.6360}\ (r^2 = .812)$

$M_R = 1769\ \theta^{.7178}\ (r^2 = .941)$

$M_R = 6794 + 887\sigma_1 - 339\sigma_3\ (r^2 = .99)$

Dense-Graded

Dry

$M_R = 6628\ \sigma_3^{.6327}\ (r^2 = .805)$

$M_R = 2203\ \theta^{.6670}\ (r^2 = .928)$

$M_R = 7345 + 955\sigma_1 - 582\sigma_3\ (r^2 = .997$

Wet

$M_R = 6130\ \sigma_3^{.6552}\ (r^2 = .82)$

$M_R = 1980\ \theta^{.6870}\ (r^2 = .93)$

$M_R = 7115 + 311\sigma_1 - 190\sigma_3\ (r^2 = .987$

Eckman Creek

Open-Graded

Dry

$M_R = 11434\ \sigma_3^{.4873}\ (r^2 = .78)$

$M_R = 4474\ \theta^{.5530}\ (r^2 = .891)$

$M_R = 14880 + 945\sigma_1 - 686\sigma_3\ (r^2 = .96)$

Wet

$M_R = 4453\ \sigma_3^{.5849}\ (r^2 = .78)$

$M_R = 1593\ \theta^{.620}\ (r^2 = .907)$

$M_R = 5195 + 572\sigma_1 - 385\sigma_3\ (r^2 = .993)$

Dense-Graded

Dry

$M_R = 4812\ \sigma_3^{.6620}\ (r^2 = .82)$

$M_R = 1531\ \theta^{.6960}\ (r^2 = .936)$

$M_R = 5897 + 710\sigma_1 - 357\sigma_3\ (r^2 = .99)$

Wet

$M_R = 4001\ \sigma_3^{.6066}\ (r^2 = .77)$

$M_R = 1371\ \theta^{.6440}\ (r^2 = .897)$

$M_R = 4505 + 583\sigma_1 - 431\sigma_3\ (r^2 = .996)$

"Big A" Sandstone

Open-Graded

Dry

$M_R = 5338\ \sigma_3^{.4738}\ (r^2 = .73)$

$M_R = 2265\ \theta^{.5084}\ (r^2 = .878)$

$M_R = 6841 + 472\sigma_1 - 417\sigma_3\ (r^2 = .49)$

Wet

$M_R = 2490\ \sigma_3^{.5642}\ (r^2 = .76)$

$M_R = 921\ \theta^{.598}\ (r^2 = .89)$

$M_R = 2892 + 312\sigma_1 - 236\sigma_3\ (r^2 = .996)$

Dense-Graded

Dry

$M_R = 8577\ \sigma_3^{.3808}\ (r^2 = .69)$

$M_R = 4259\ \theta^{.598}\ (r^2 = .835)$

$M_R = 10533 + 560\sigma_1 - 560\sigma_3\ (r^2 = .996)$

Wet

$M_R = 8725\ \sigma_3^{.3090}\ (r^2 = .58)$

$M_R = 4806\ \theta^{.3430}\ (r^2 = .739)$

$M_R = 10316 + 501\sigma_1 - 644\sigma_3\ (r^2 = .98)$

Blend

Open-Graded

Dry

$M_R = 3639\ \sigma_3^{.5217}\ (r^2 = .78)$

$M_R = 3440\ \theta^{.5540}\ (r^2 = .911)$

$M_R = 11057 + 326\sigma_1 - 550\sigma_3\ (r^2 = .99)$

Wet

$M_R = 7901\ \sigma_3^{.6049}\ (r^2 = .82)$

$M_R = 2806\ \theta^{.633}\ (r^2 = .931)$

$M_R = 9330 + 100\sigma_1 - 530\sigma_3\ (r^2 = .996)$

Dense-Graded

Dry

$M_R = 5620\ \sigma_3^{.6089}\ (r^2 = .79)$

$M_R = 1933\ \theta^{.6440}\ (r^2 = .921)$

$M_R = 6090 + 696\sigma_1 - 275\sigma_3\ (r^2 = .98)$

Wet

$M_R = 13357\ \sigma_3^{.4485}\ (r^2 = .68)$

$M_R = 5905\ \theta^{.483}\ (r^2 = .825)$

$M_R = 15909 + 1219\sigma_1 - 1202\sigma_3\ (r^2 = .988)$

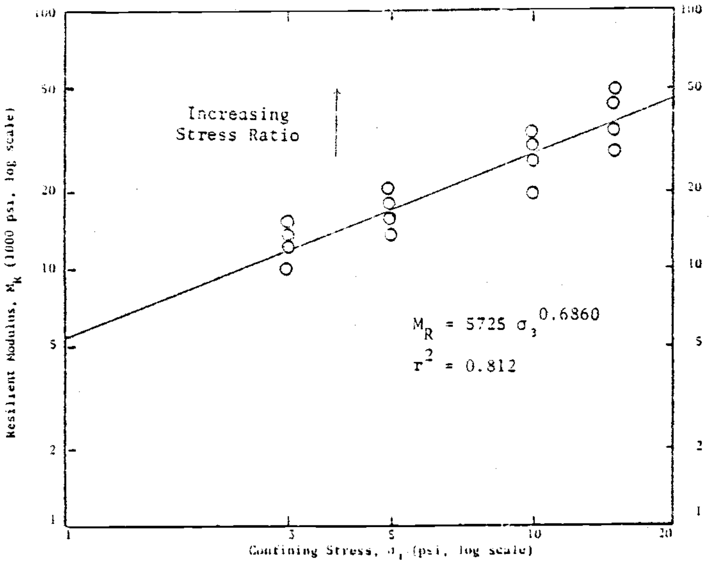

FIGURE 31. Variation of Resilient Modulus with Confining Stress: Open Graded Ocean Lake (Wet)

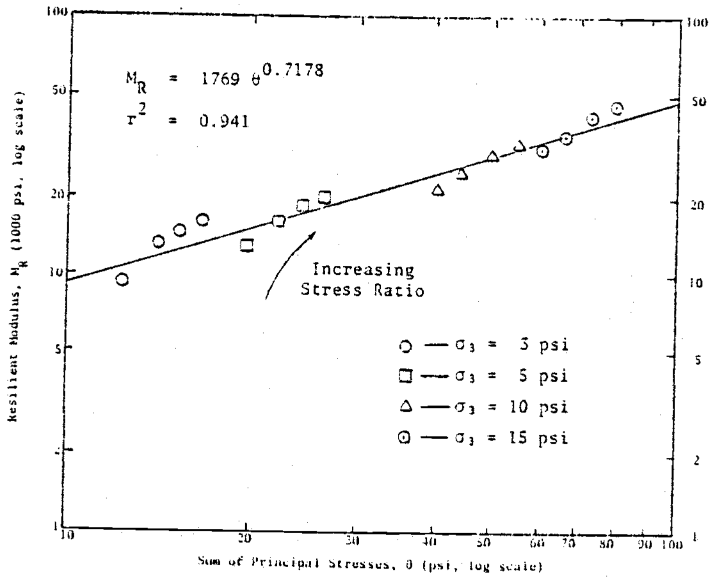

FIGURE 32. Variation of Resilient Modulus with the Sum of the Principle Stresses: Open Graded Ocean Lake (Wet)

stress resulted in an average r-squared value of 0.76. By considering the variable θ, this value was improved to 0.89. By considering the independent action of the variables comprising θ, confining stress and axial stress, the average r-squared value was improved to 0.988, or almost complete removal of error. So, for one series of tests on each specimen, the multiple regression approach appeared to give the best approximation of the expected value of resilient modulus. Plots of resilient modulus as a function of σ_3, θ and $\sigma_1 + \sigma_3$ are presented in Figures 33 through 48. These plots can be used to describe the effects of different gradation and moisture contents on the stiffness of the sample. To quantify the differences, the relations developed for M_R vs. θ will be used. From Figure 37 for Ocean Lake it appears that the gradation change and moisture change result in little significant stiffness change throughout the range of θ. Comparing Ocean Lake with the other aggregates (Figures 38 through 40) shows that the effect of water on the stiffness of the sample is more pronounced for the lower quality aggregate. General statements about the relative stiffnesses of the lower quality aggregate and the blend when compared to the high-quality rock are not statistically warranted. To do this, a more intensive test program, with several replications at all treatment levels, would be required which was beyond the scope of this project. Also, the various plots of the resilient modulus response lead to the conclusion that a testing error was made on the wet specimen for the dense-graded blend. This error can most probably be attributed to an LVDT alignment problem. This problem was prevalent throughout the early stages of resilient modulus testing. Several results of this sort were discarded and the tests were performed again.

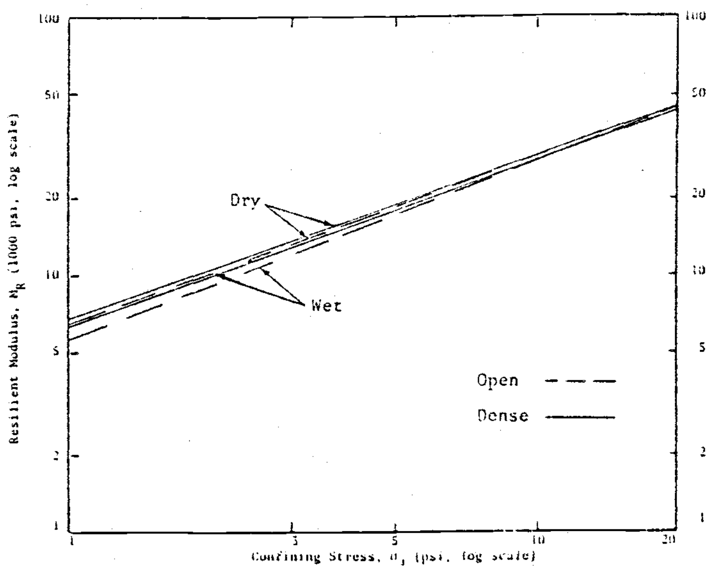

FIGURE 33. Variation of Resilient Modulus with Confining Stress: Ocean Lake

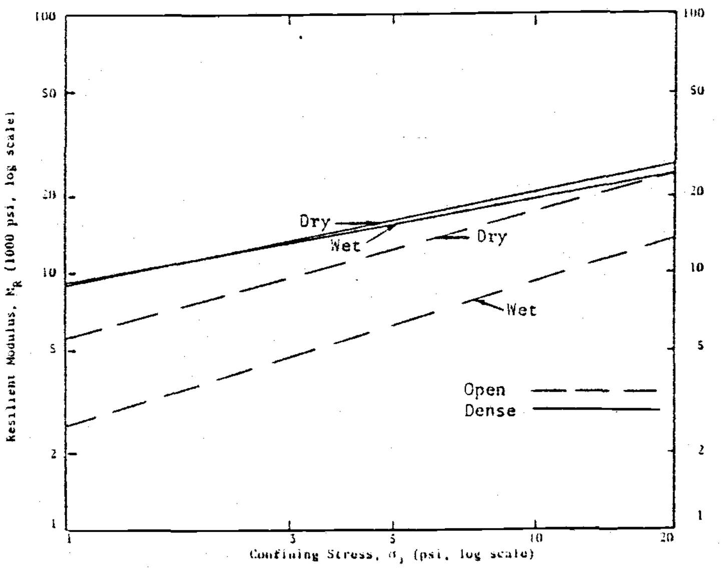

FIGURE 34. Variation of Resilient Modulus with Confining Stress: "Big A"

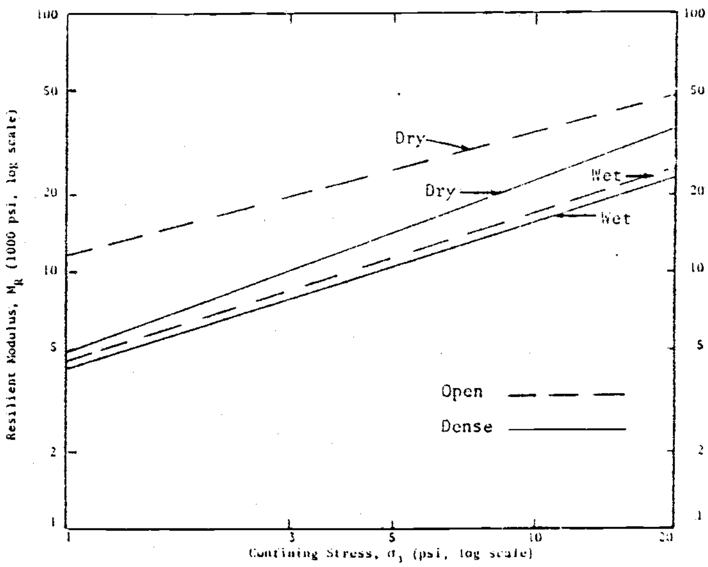

FIGURE 35. Variation of Resilient Modulus with Confining Stress: Eckman Creek

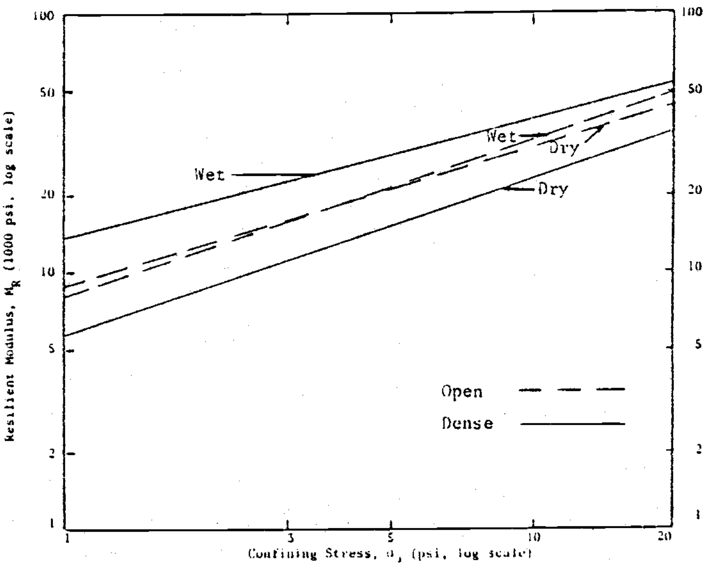

FIGURE 36. Variation of Resilient Modulus with Confining Stress: Blend

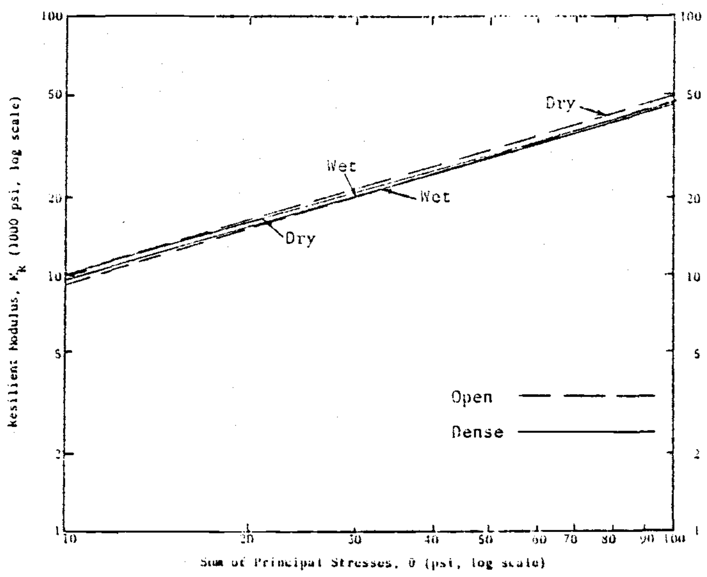

FIGURE 37. Variation of Resilient Modulus with the Sum of the Principle Stresses: Ocean Lake

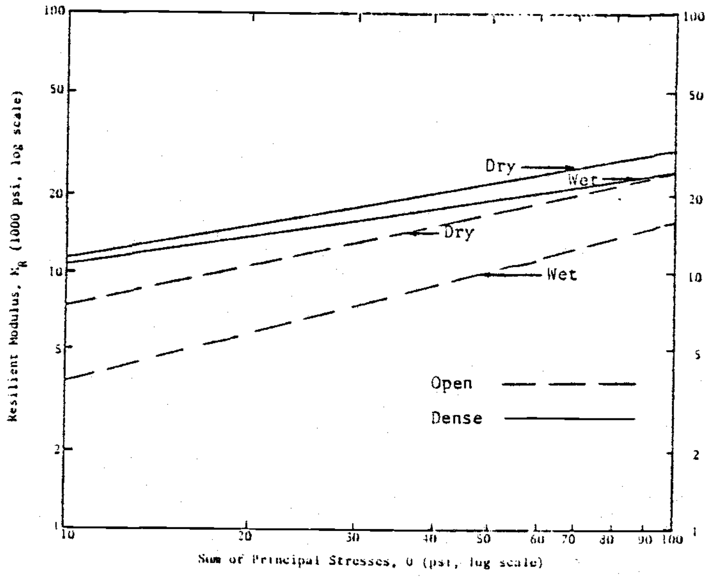

FIGURE 38. Variation of Resilient Modulus with the Sum of the Principle Stresses: "Big A"

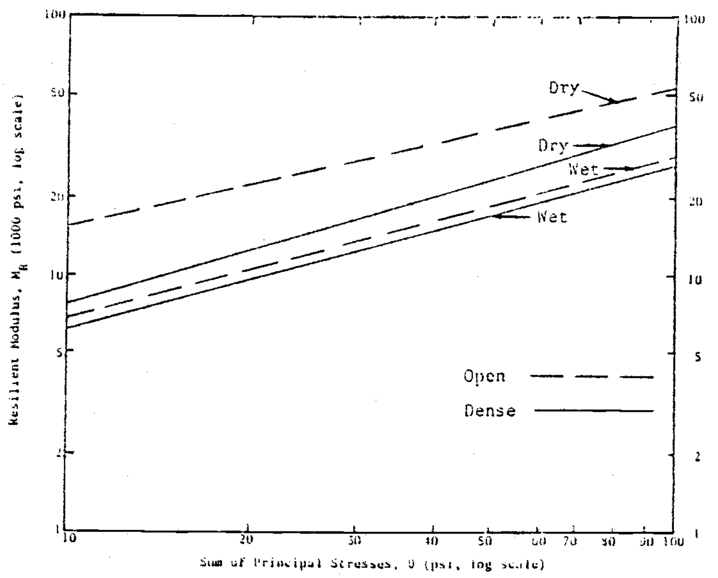

FIGURE 39. Variation of Resilient Modulus with the Sum of the Principle Stresses: Eckman Creek

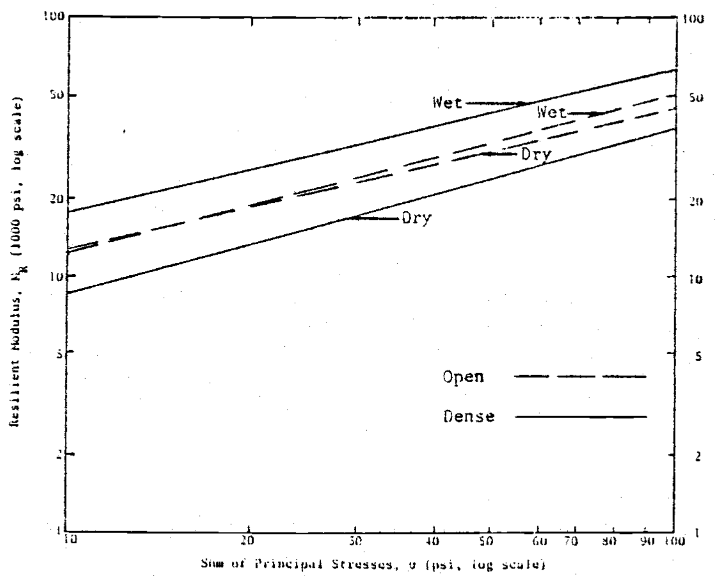

FIGURE 40. Variation of Resilient Modulus with the Sum of the Principle Stresses: Blend

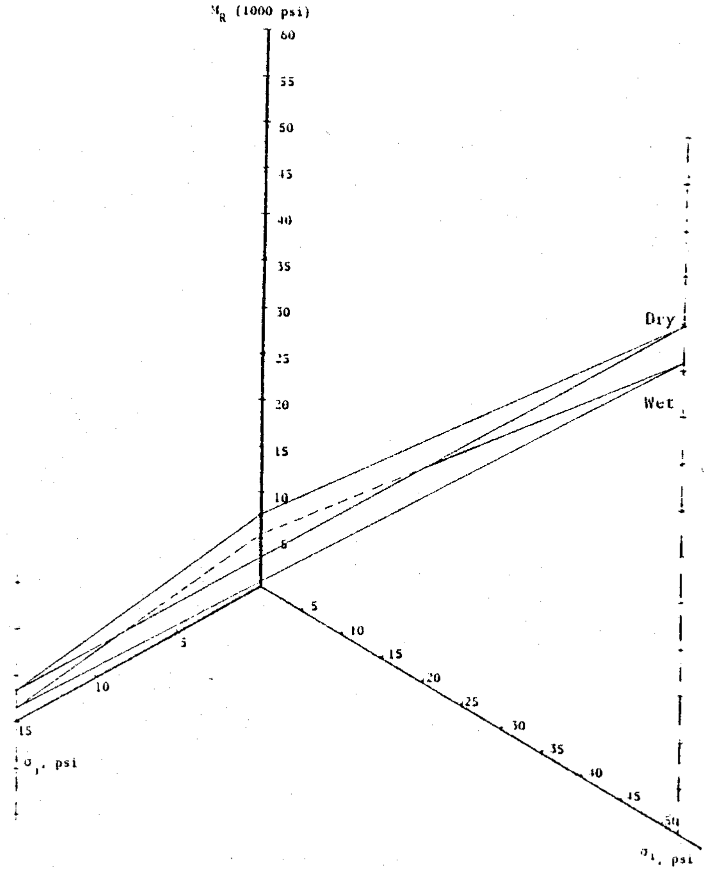

FIGURE 41. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Open Graded Ocean Lake

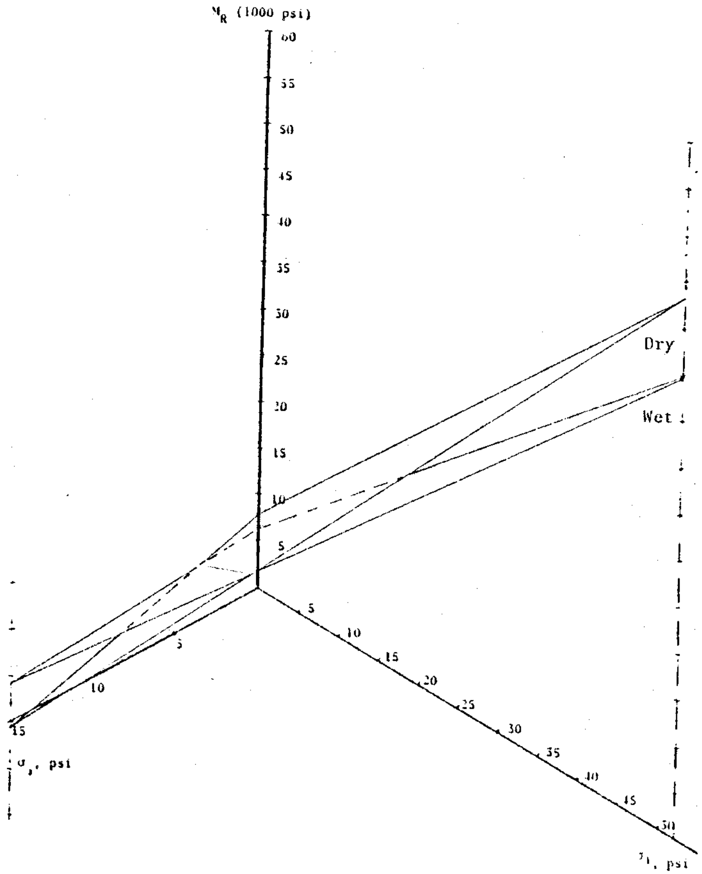

FIGURE 42. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Dense Graded Ocean Lake

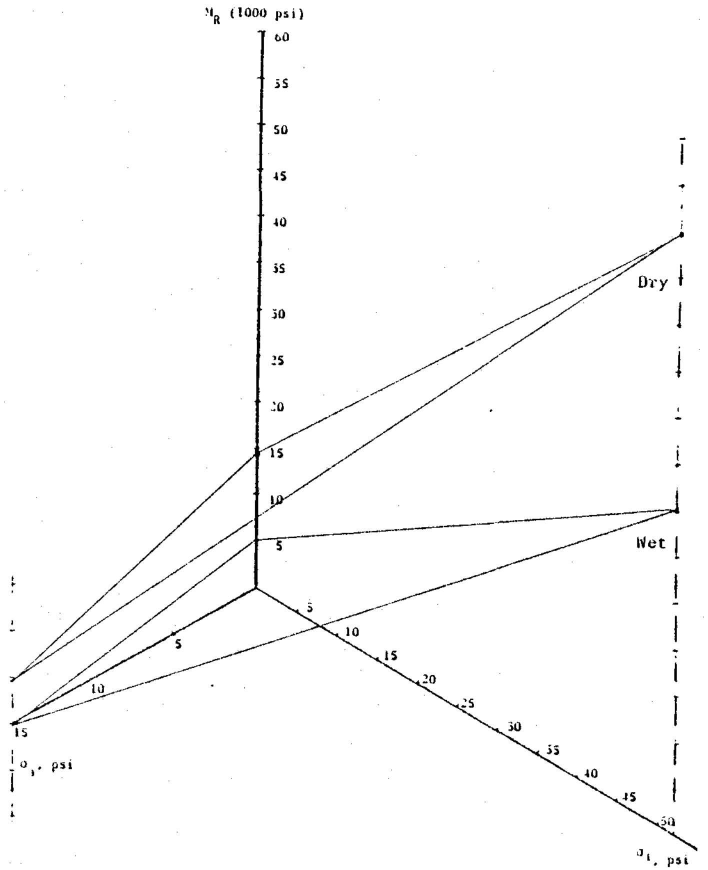

FIGURE 43. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Open Graded Eckman Creek

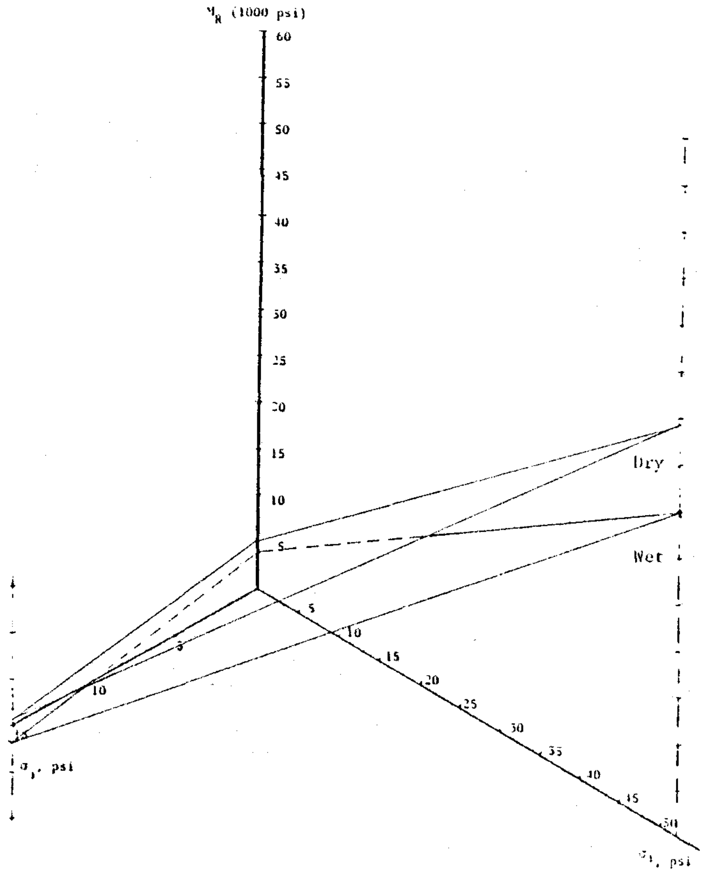

FIGURE 44. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Dense Graded Eckman Creek

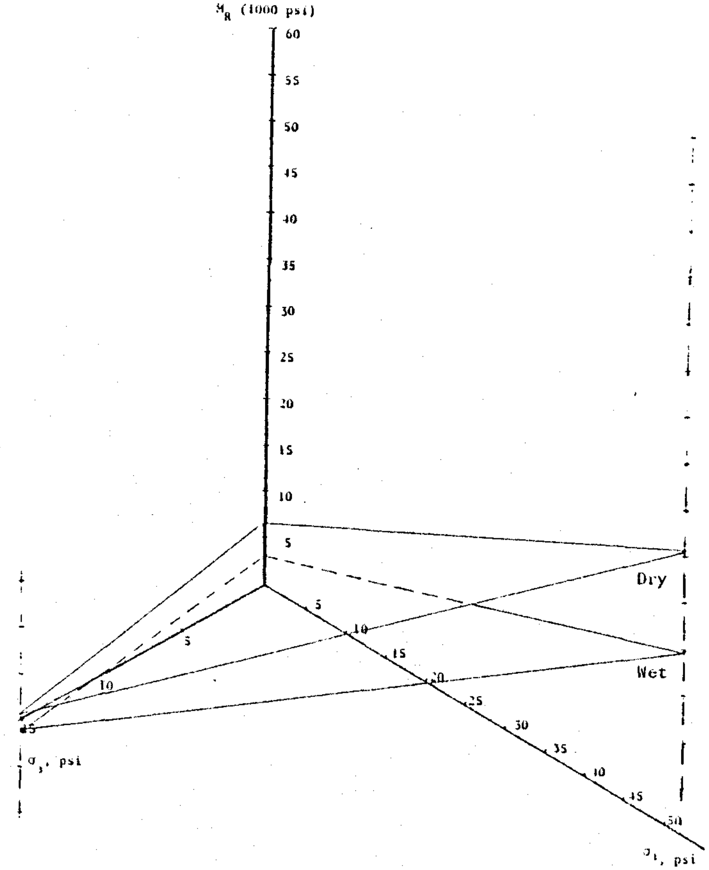

FIGURE 45. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Open Graded "Big A"

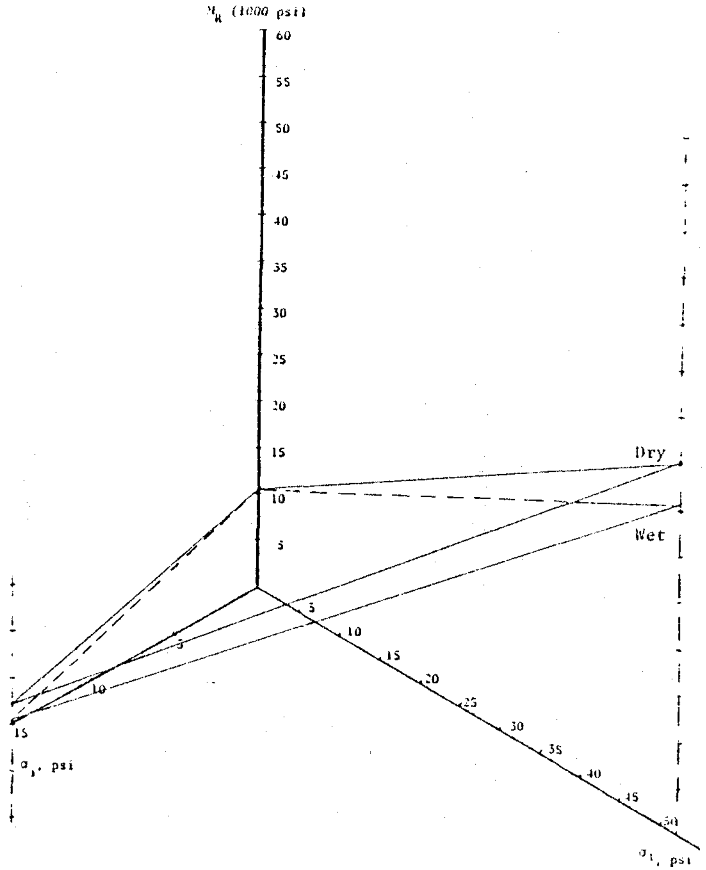

FIGURE 46. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Dense Graded "Big A"

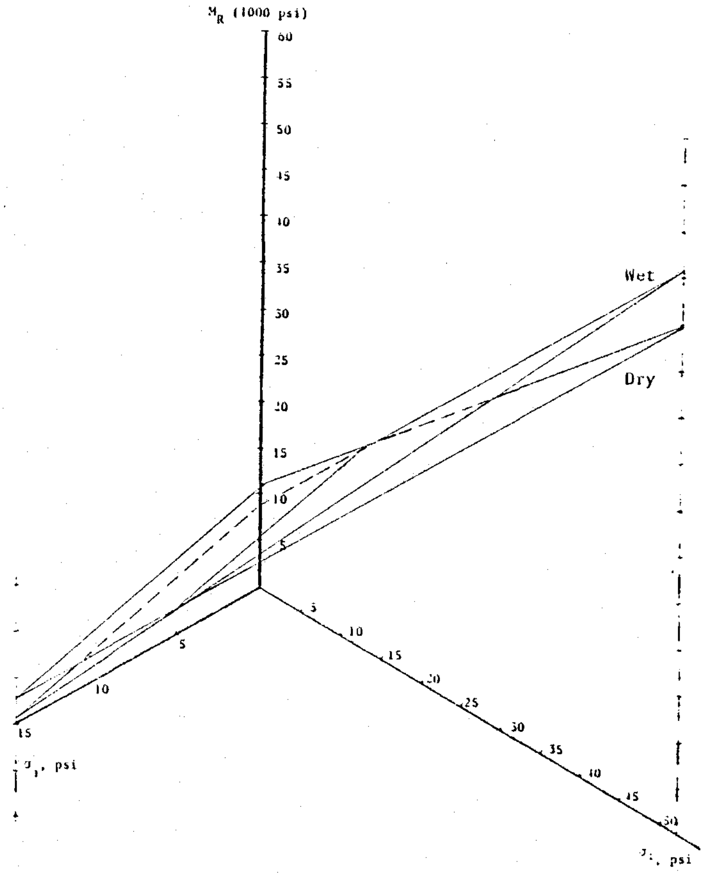

FIGURE 47. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Open Graded Blend

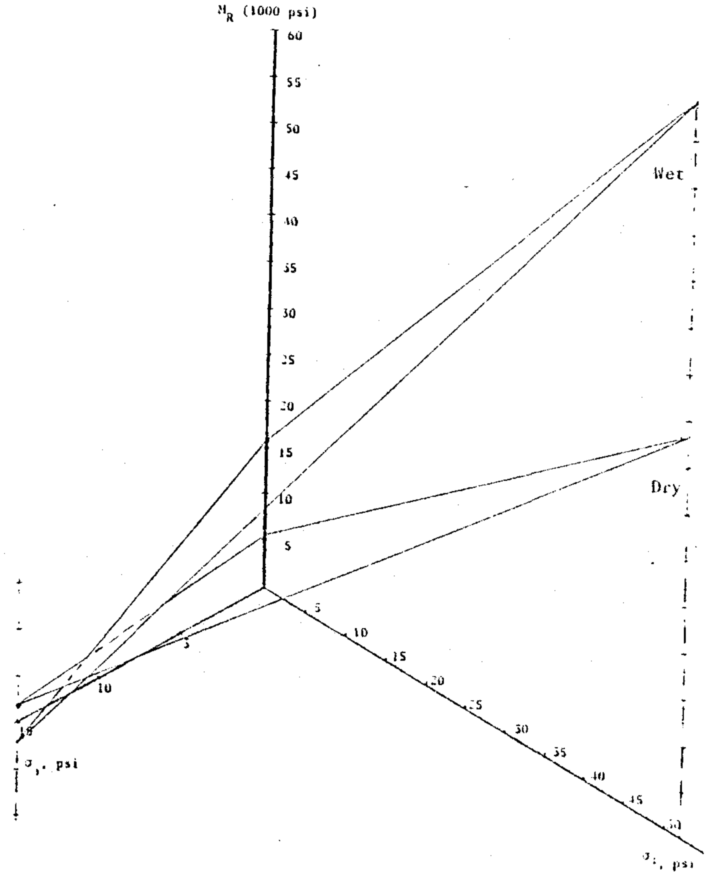

FIGURE 48. Resilient Modulus Response as a Function of Axial Stress (σ_1) and Confining Stress (σ_3): Dense Graded Blend

Unfortunately, a mechanical failure of the MTS and limited time would not permit a repeat testing of this sample. It is believed that the resilient modulus value for the wet, densely-graded blend is actually somewhat lower than its dry counterpart.

CHAPTER 6. DISCUSSION

Up to this point, a definition of "marginal" aggregate has been purposely avoided until information concerning the quality of the coastal aggregates could be presented. Frequently, aggregate specifications are written to allow acceptance of lower quality aggregates when high-quality rock is not locally available. It would seem reasonable then to state that the term "marginal" would imply a range in acceptable values. Erickson (9) recommented that 20 to 35 percent secondary minerals would give marginal performance. The adoption of a similar "marginal" range in values for the standard durability tests is recommended.

Classification of an aggregate source as marginal would indicate that special design considerations would be required for use in road construction. Special design considerations could result in a number of modifications. These modifications would include blending with quality rock and admixture beneficiation such as asphalt emulsions, portland cement, lignins, and industrial wastes.

Availability of construction aggregate is a function of several interacting factors. The low quality of the different aggregates found on the Oregon Coast is only one of these. The other factors include conflicting land-use, long haul distances, excessive extraction and preparation costs and escalating transportation costs. These factors are largely controlled by societal values.

Because of the low bulk value of aggregate, it is most economical when located close to the construction site, or population. Locating a quarry too close to residential areas results in dissatisfaction of the people residing in the neighborhood. Blasting to break the rock masses is restricted and objections to heavy haul traffic are raised because of noise and dust. Also, many potential sites of quality aggregate have been eliminated because of residential and business activities on the site.

Yaquina Head Quarry in Newport, Oregon, is experiencing these land-use problems. This quarry provides much of Newport's quality construction aggregate but is unfortunately located quite prominently on one of the coast's most scenic and publicly accessible sites. There has been quite a bit of pressure exerted on the owner of this quarry to close down. However, eliminating the site will almost certainly impose added importation from the Willamette Valley. One other alternative would be to improve the lower quality basalts and sandstones which lie relatively hidden only a few miles inland.

Failures of the aggregate in a road is difficult to substantiate. They will usually show up as alligatoring or eventual potholing, both water related. Removal of the water or waterproofing the aggregate would result in adequate performance of the road section. Measurement of actual degradation has been done by Minor (13). This involved statistical sampling of the aggregate gradation on a newly constructed road and repeating this after a year's service. Comparison of the gradations before and after showed that the aggregate (marine basalt) was experiencing extreme breakdown. This is one method of detecting a problem aggregate's performance in a road.

CHAPTER 7. CONCLUSIONS & RECOMMENDATIONS

The purpose of this report was to evaluate marginal quality aggregates found along the Oregon Coast. This was done in several different stages. A literature review was conducted to: 1) aid in problem definition; 2) characterize local aggregates; 3) determine specific problems with aggregate usage on Oregon's Coast (limited reserves of quality aggregates); and 4) determine appropriate testing measures to be conducted on these aggregates.

After the literature review was completed, aggregate samples were collected and Phase I and Phase II testing was performed. The results of the testing program led to the following conclusions:

1. Low-quality aggregates, as determined by the conventional testing performed in Phase I, experienced greater amounts of plastic strain, indicating a tendency for rutting behavior, especially in the wet condition.
2. The measurement of degradation after two types of compaction (modified proctor and vibratory compaction) and repeated load testing was virtually impossible because of the tendency of the wet material to re-cement together upon drying.
3. The adverse effect of water on the strength of the sample was particularly pronounced for the low-quality Eckman Creek basalt and "Big A" sandstone.
4. Some benefit can be derived from blending as was done with the Ocean Lake and Eckman Creek basalts, particularly in the control of plastic strain.

In conducting this research, not all variables that may influence the performance of aggregate in a road section were accounted for. Perhaps the most significant variable not considered was time. It is believed that the response to moisture of a rock, marine basalt in particular, occurs over a period of time. The results of this curing process can only be measured under actual service conditions. Another important variable not considered was the kneading action imparted by wheel loads. The repeated load testing performed in the MTS did not provide this more extreme condition of loading. Also to be considered would be the effect of different moisture contents and stress ratios on permanent deformation.

In conclusion, it is believed that these lower quality aggregates can be used, if properly treated, for road construction purposes. Use of these lower quality aggregates as base materials and the higher quality rock as surfacing would provide the coastal areas with significant savings of the dwindling quality aggregate supplies. Most agencies recognize this and relax their specifications when justified. However, the two low quality aggregates tested fell well below these relaxed specifications. It is not felt that gradation changes offer any measurable relief from this problem so it appears that admixture treatment should be considered. Blending of a low-quality aggregate with a high-quality aggregate appears to warrant further investigation for determining the optimum combination. The physical strength of the untreated basalts are comparable. Therefore, sealing the marginal quality marine basalt from the adverse effects of water would be a viable means of treatment. It is recognized that the more logical choice of sealants would be either asphalt cement, emulsified asphalt or portland cement. Using these materials as sealants would provide the added benefit of increased strength and, therefore, reduced quantities required for construction purposes. Also to be considered is the use of wood by-products called lignins.

Based on the results of this study, the following recommendations are made:

1. Perform more testing on the untreated aggregate to provide statistical significance to the effect of moisture on plastic strain and resilient modulus.
2. As determined from this testing it was found that plastic strain could be significantly lessened when marginal aggregate and quality aggregate are blended. Therefore, further investigation of the optimum blend should be considered.
3. Investigate the feasibility of using additives such as asphalt cement, emulsified asphalt, portland cement, and lime to negate the detrimental effects of marginal aggregates found along the Oregon Coast. Cady, et al., (17) determined that soft, easily abraded aggregates (e.g. sandstone and siltstone) can be improved by epoxy impregnation. Such treatment and subsequent abrasion testing should be considered. Aggregates prone to weathering degradation proved to be especially difficult to improve, however, impregnation with polymer additives might offer some hope. Neither of these methods are presently economically feasible however.
4. Develop typical specifications or guidelines for using both

treated and untreated marginal aggregates for construction purposes. Developing these specifications would require a consideration of the differences in permanent strain and modulus for quality aggregates and low quality aggregates when wet. Layer coefficients would be required and construction of a test road or sections would be desirable to justify these specifications.

5. Finally, a standardized test procedure for detecting degradation potential should be adopted by all road building agencies in the Northwest. The Washington Durability and DMSO tests are recommended for basalts. They are the least time consuming tests, are easily read and interpreted, and require only a nominal cost for purchasing the equipment and materials. It is believed that even the less affluent cities and counties could afford these items. The Los Angeles Abrasion test should be performed on aggregates suspected of having poor mechanical strength such as sandstone.

    Washington Durability values of 60%, 50% and 40% are suggested for the surfacing, base and subbase, respectively. Los Angeles Abrasion values of 35%, 45% and 50% are suggested for the same components of a road section.

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APPENDIX A

RESILIENT MODULUS TEST PROCEDURE

This appendix is a manual for performing resilient modulus tests on soils with the MTS (Materials Testing System) electrohydraulic closed loop test system at OSU. It was taken from a report written by George Filz (39) in August of 1978. It contains sections on the MTS system, calibration of both the load cell and the LVDT's to the HP (Model 7402) recorder, a description of the resilient modulus test procedures, and a sample data sheet.

The procedures discussed herein were developed during the summer of 1978 to measure the resilient modulus* of volcanic materials used in roads in Eastern Oregon and modified to conduct permanent deformation and resilient modulus testing on coastal aggregates. For each material, tests were performed at a variety of combinations of water content and dry density. For the permanent deformation, the sample was subjected to repeated loading at a confining stress of 10 psi (690 kN/m^2) and a deviation stress of 25 psi (1730 kN/m^2). For the aggregate base materials, the resilient modulus was evaluated at the following confining pressures and stress ratios for each confining stress.

σ_c, psi	(kN/m^2)	σ_1/σ_2
3	(210)	2.0
5	(350)	2.5
10	(690)	3.0
15	(1040)	3.5

*The resilient modulus, M_R is given by: $M_R = \Delta\sigma_d / \Delta\varepsilon$

where: $\Delta\sigma_d$ = the change in deviator stress during the load pulse
$\Delta\varepsilon$ = the change in strain due to the load pulse.

The resilient modulus increases with the number of load repetitions but tends to approach a limiting value. To develop this ultimate modulus, the sample was preconditioned for 1000 cycles. The sample would be preconditioned at the combination of confining pressure and deviator stress which produces the greatest deflection of the sample to insure removal of any permanent deforamtion. For the stress combinations used in base material testing, the choice was simple: precondition at σ_c = 15 psi (1040 kN/m^2) and σ_d = 37.5 psi (2600 kN/m^2). Once the sample has been preconditioned sufficiently, it is only necessary to condition the sample for about 100 repetitions at each combination of confining pressure and deviator stress before measuring the resilient modulus.

THE MTS SYSTEM

Since the load applied to the sample is read using the load cell in the triaxial cell and the HP Strip Chart Recorder (Model 7402), a system completely independent from the MTS system, it is not necessary to perform any calibration or zeroing on the MTS system. Simply adjust the load delivered to the sample by the MTS system until the desired load amplitude is indicated on the HP recorder.

A variety of wave forms are available using the Digital Function Generator of the MTS system. The settings described below generate a triangular wave pulse of 0.12 seoncds duration, delivered every 2.0 seconds.

(1) Be sure that cooling water is available to the MTS System before turning hydraulic power on.

(2) On the MASTER CONTROL PANEL, turn the CONSOLE POWER switch ON.

(3) On the COUNTER PANEL, set the COUNTER INPUT control to OSCILATOR and the COUNT MULTIPLIER to X1. Press both the PRESET COUNT and TOTAL COUNT reset buttons. While pressing the PRESENT COUNT reset button,

set the preset count to 43,200 (one day).

(4) This step establishes the load duration, frequency and pattern. On the DIGITAL FUNCTION GENERATOR, set the CONTROL MODE switch to LOCAL. Set RATE 1 to 6.0×10^{-2} seconds and RATE 2 to 9.4×10^{-1} seconds. Make sure that the STOP AT ZERO, RETURN TO ZERO and HOLD buttons are all out, but do not press the START button. The OUTPUT ZERO indicator should be lit. Set the BREAKPOINT PERCENT control to 0.0 and the BREAKPOINT switch to LOCAL REVERSE. Press the HOLD AT BREAKPOINT button and make sure that the RAMP THRU ZERO button is out. Press the RAMP and INVERT buttons.

(5) This step is an initial condition of zero axial load. On the CONTROLLER panel, press the LOAD ± PERCENT button. Put the SET POINT control at 500 and the SPAN 1 control at 0. Set the METER switch to DC ERROR and the GAIN control to 8.2.

(6) On the LIMIT DETECTOR module, located behind the swinging CONTROLLER panel, set the IND-INTLK-PROG switches for XDCR1 and XDCR3 to IND. For XDCR1, set the UPPER AND LOWER controls to 500 and set the UPPER and LOWER controls to 900 and set the UPPER and LOWER switches to + and -, respectively.

(7) On the LOAD CONTROL DC CONDITIONER module, the farthest module to the right behind the CONTROLLER panel, set the RANGE switch to 4. This will light the 10% LOAD indicator on the CONTROLLER panel.

(8) Press the INTERLOCK RESET button on the CONTROLLER panel.

The MTS system is now ready for use in resilient modulus testing. The load delivered by the system is the sum of a steady load and a repetitive load. The magnitude of the steady load is determined by the SET POINT control with numbers above 500 giving tension and numbers below 500 giving compression. The amplitude of the repetitive load is determined by the SPAN 1 control with the load amplitude increasing as the SPAN 1 setting is increased. During resilient modulus testing, the steady load should be kept as small as possible. Therefore, the SET POINT control is only used to raise and lower and triaxial cell and to apply enough force to keep the rod firmly against both the MTS load cell and the load cell inside the triaxial cell. The value of the SET POINT control will probably always be between 490 and 510 for these tests.

To apply a load to the sample, check that the DC ERROR is near zero than press the HYDRAULIC PRESSURE button once to light the LOW indicator. Press it again to light the HIGH indicator. If at any time things look wrong (for example, a sudden jump of the base or a loss of confining pressure), press the EMERGENCY STOP button. With the SET POINT control, the sample may be raised until the rod is in contact with both load cells. Pressing the START button on DIGITAL FUNCTION GENERATOR sends the load waveform to the SPAN 1 control. Increasing the SPAN 1 setting from zero will begin applying the load pulse to the sample. The SPAN 1 control should be adjusted until the desired load pulse amplitude is indicated on the HP recorder. While increasing the SPAN 1, it may be necessary to increase the SET POINT value (i.e., apply more tension) in order to decrease the steady load on the sample.

To stop loading the sample, press the STOP AT ZERO button on the DIGITAL FUNCTION GENERATOR. The triaxial cell may then be lowered using the SET POINT control. Turn the hydraulic power off with either the HYDRAULIC OFF or EMERGENCY STOP buttons, since these buttons perform the same function.

CALIBRATING THE LOAD CELL

To calibrate the load cell the following steps are suggested:

(1) Connect the load cell to one channel of the HP recorder through the triaxial cell base.

(2) On the HP recorder, set the following control on the load channel:

SENSITIVITY	= OFF
FILTER	= 50 (to filter elctronic noise at 50 Hz)
OPR-BAL	= OPR
ZERO SUPPRESSION POLARITY	= OFF

CAL	=	0.0
OFFSET	=	0.0
BRIDGE	=	FULL
ATTENUATOR	=	1

(3) With the chart speed at 1 mm/second, use the PEN POSITION control to set the chart pen to the center of the paper.

(4) Set the OPR-BAL switch to BAL.

(5) Adjust the C BAL and R BAL controls:

(a) Increase the SENSITIVITY until the pen just deflects off the charge paper. Turn the control back one step so that the pen is back on the paper.

(b) Adjust the C BAL control for minimum pen deflection from zero.

(c) Adjust the R BAL control for minimum pen deflection from zero.

(d) Repeat a, b and c until the SENSITIVITY control is on 0.1 mV/V/Fs and the pen is as close to zero as possible.

(e) Set the OPR-BAL switch to OPR.

(f) With the SENSITIVITY still at 0.1, adjust R BAL until the pen is exactly on zero.

(6) Calibrating the load cell:

(a) With the SENSITIVITY control on OFF, use the PEN POSITION control to set the pen 5 mm from the right hand edge of the chart paper.

(b) Set the load cell, which is still connected to the recorder, on the floor and stack approximately 400 pounds of weights on the cell.

(c) With the SENSITIVITY control on 1 mV/V/FS, adjust the vernier control until the pen deflects 1 mm to the left for every 10 pounds on weight on the load cell.

(d) To verify the accuracy of the load cell, load weights from 100 to 700 pounds by 100-pound increments on the cell and read the pen deflection on the chart, using an appropriate SENSITIVITY setting. A linear regression may be run between the known weights on the load cell and the chart pen deflection.

CALIBRATING THE LVDT's

To calibrate the LVDT's the following steps are suggested:

(1) On the HP recorder, set the following controls on the LVDT channel:

SENSITIVITY	= OFF
FILTER	= 50
OPR-BAL	= OPR
ZERO SUPPRESSION POLARITY	= OFF
CAL	= 0.0
OFFSET	= 0.0
BRIDGE	= FULL
ATTENUATOR	= 1

(2) With the chart speed at 1 mm/second, use the PEN POSITION control to set the chart pen to the center of the paper.

(3) Set the OPR-BAL switch to BAL.

(4) Preliminary adjustment of the C BAL and R BAL controls:

(a) Disconnect the LVDT's from the HP recorder.

(b) Increase the SENSITIVITY until the pen just deflects off the chart paper. Turn the control back one step so that the pen is again on the chart.

(c) Adjust the C BAL control for minimum pen deflection from zero.

(d) Adjust the R BAL control for minimum pen deflection from zero.

(e) Repeat steps b, c and d until the SENSITIVITY control is on 0.1 mV/V/FS and the pen is close to zero as possible.

(5) Connect the LVDT's to the recorder through the test base.

(6) Mount the LVDT's and their cores on the Schaevitz calibration mounts (Figure A1).

(7) With the micrometer, insert the core of one LVDT into its LVDT towards the neutral point. The neutral point has been reached when, with the OPR-BAL switch on BAL, the pen has minimum deflec-

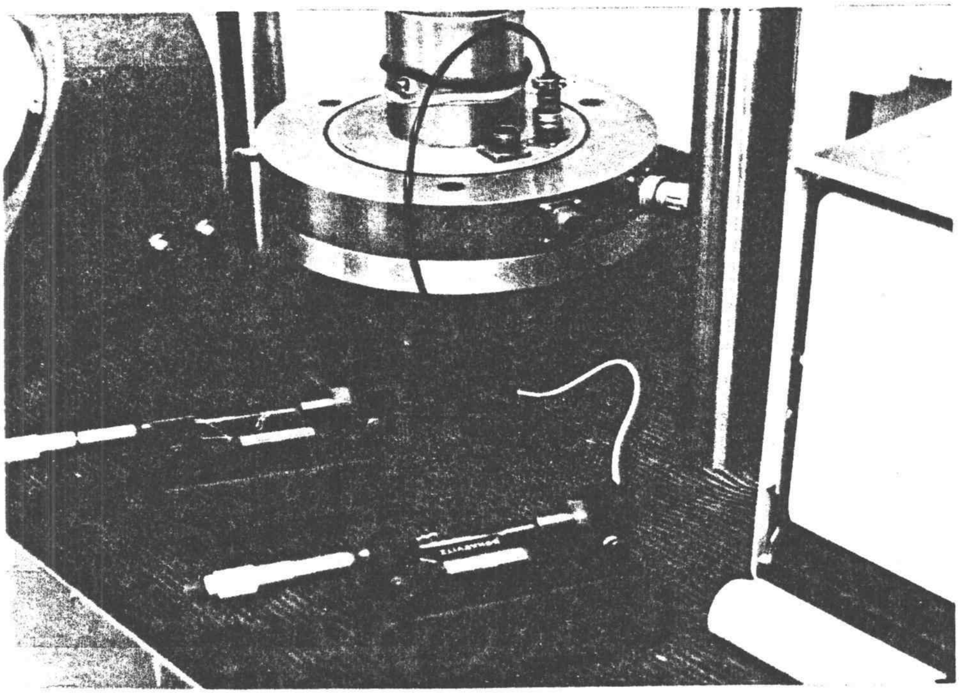

FIGURE 1. LVDT Cores Mounted in Schaevitz Micrometers for Calibration

tion from the chart zero line. These micrometers have a little push-pull slop, so it is necessary to approach all measurements from a consistent direction. Find the neutral point for this LVDT by adjusting the micrometer and increasing the SENSITIVITY until it is 0.1 mV/V/FS. Note the micrometer reading at the neutral point.

(8) Repeat Step 7 for the other LVDT and its core. Remember to decrease the SENSITIVITY before beginning to insert the second LVDT core.

(9) Final adjustment of the C BAL and R BAL controls:

(a) Without moving the LVDT cores from their neutral points, perform steps 4b through 4e.

(b) Set the OPR-BAL switch to OPR.

(c) With the SENSITIVITY control still at 0.1 mV, adjust R BAL until the pen is on the chart paper zero.

(10) Calibrating the LVDT's:

(a) Set the SENSITIVITY control on OFF.

(b) Use the PEN POSITION control to set the pen on the left hand edge of the chart paper.

(c) Set the SENSITIVITY control to 5 mV/V/FS.

(d) Move the LVDT cores in 0.05 inches from their neutral points.

(e) Use the VERNIER control to adjust the pen position until it is exactly on the right hand edge of the chart paper. Thus, on the 5 mV scale, 5 centimeters equals 0.05 inches of displacement of the LVDT's.

(f) The LVDT's have now been calibrated to the HP recorder. Once calibrated, it is important to avoid switching the two LVDT cores.

(11) To verify the LVDT calibration, check the pen displacement, on appropriate SENSITIVITY scales, corresponding to different core displacements from the neutral points. Do not exceed ± 0.1" from the neutral points, as this is the limit of the linear range for the LVDT's.

RESILIENT MODULUS AND PERMANENT DEFORMATION TEST

Procedure

The following steps are suggested for sample preparation:

(1) Prepare the soil at the desired water content and store in the humidity room.

(2) Prepare the mold to receive the soil:

(a) Place the membrane on the test base with approximately 3/4" of the membrane extending down onto the stand.

(b) Roll the rubber O-ring up to its notch.

(c) Place about six wraps of black plastic tape around the membrane and O-ring at the top of the test base. By increasing the diameter of the membrane with the black tape, the two-piece mold will clamp more securely onto the test base.

(d) Clamp the two-piece mold around the membrane. The mold should set on top of the taped O-ring. While putting the mold on the base, try to keep wrinkles from developing in the cloth near the bottom of the mold.

(e) After the mold has been firmly clamped in place with the two C-clamps, stretch the membrane over the top of the mold and tape it in place.

(f) Place a fillet of vacuum grease in the crack between the mold and the taped O-ring around the top of the test base. A small amount of vacuum grease may be required where the two mold halves contact.

(g) Place the three wooden blocks under the mold (Figure A2). This insures the proper height and level for the mold.

(h) Apply a vacuum to the mold and check to make sure the membrane is pulled out against the mold.

(i) Drop two or three filter papers down into the mold to cover the vacuum hole in the center of the test base.

(3) Compact soil into mold:

(a) Remove enough soil from the humidity room to perform the test.

(b) Weigh a sample of the soil (200-500 g) to use for a moisture determination.

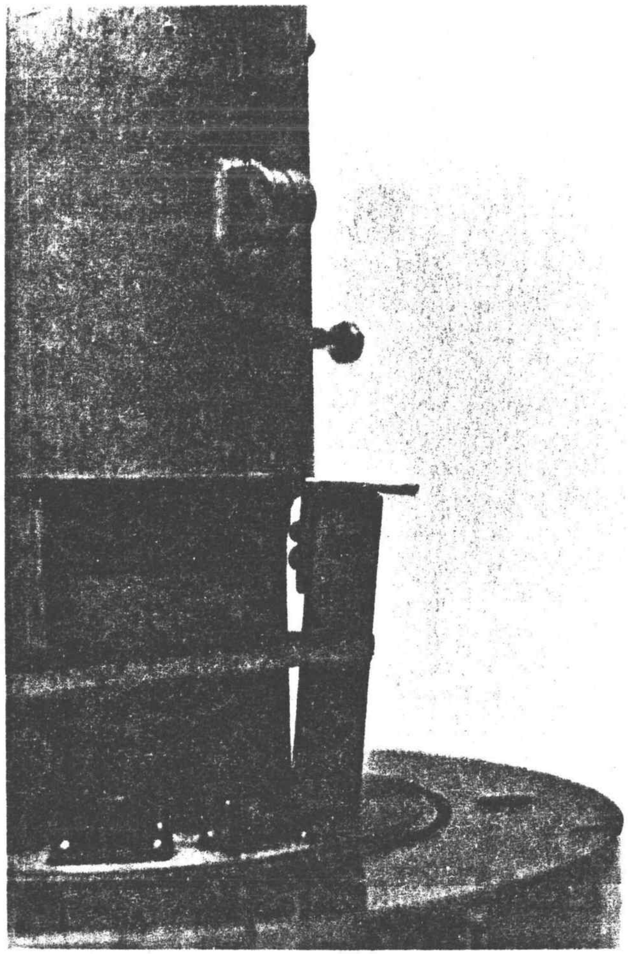

FIGURE A-2. Wood Blocks in Place, Insuring Proper Height and Level of Sample

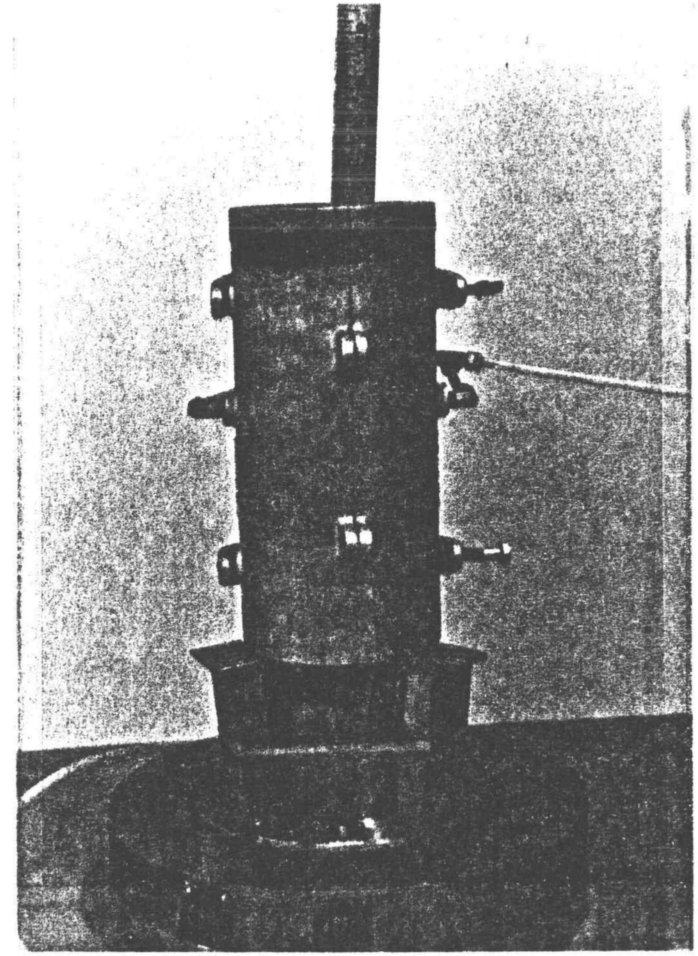

FIGURE A-3. Assembled Mold with Two Lifts of Compacted Soil

(c) Weight five equal batches of the soil such that each batch has the necessary weight to provide the required dry density when compacted into a two-inch lift, i.e.:

$$\text{weight/batch} = (\gamma_d)(1+w)(\pi r^2(2''))$$

where: γ_d = dry density of soil
w = water content
r = radius of test base ($\sim$ 2")

(d) With a scale, measure the distance from the top of the test base to the top of the mold ($\sim$ 10 1/8").

(e) Carefully place the first batch of soil into the mold.

(f) Use the vibration compactor to compact the soil into the mold and measure the thickness of the layer this compactive effort produces (Figure A3). Continue compacting the soil until a two-inch layer is achieved.

(g) Compact the next three lifts into the mold. Do not, however, rely on the compactive effort used to achieve the two-inch thickness in the first layer. Use less time than the first layer required and measure the accumulated soil column height. Then use more blows until a thickness of 4, 6, or 8 inches is achieved.

(h) Tape the extension collar on top of the mold (Figure A4).

(i) Compact the final layer of soil into the mold until it is just below the top of the two-piece mold.

(j) Remove the extension collar.

(k) Use the finishing plate and one or two more blows of the impact hammer to finish the surface of the soil.

(l) Place the load cell on top of the soil, remove the tape holding down the rubber membrane, pull the membrane up around the load cell, and roll the rubber O-ring down into its notch on the load cell.

(m) Switch the vacuum from the mold to the base.

(n) Remove the three wooden blocks, remove the two C-clamps, and use a screwdriver in the filed notch on the mold to separate the mold halves (Figure A5). Remove the mold.

(o) Wipe the vacuum grease off the tape over the O-ring.

(4) Measure the height of the soil column with a scale (approximately 10").

(5) Precondition the soil:

(a) Screw the large aluminum disk onto the MTS ram.

(b) Set the triaxial cell test base onto the aluminum disk.

(c) Connect the cable from the load cell to the test base and connect the cable from the HP recorder (the load channel) to the test base.

(d) Assemble the triaxial cell (Figure A6).

(e) On the MTS system, set the following controls:

SPAN	= 0
SET POINT	= 500
LOAD PERCENT	= 10
STROKE PERCENT	= 100
POWER	= ON
COUNTER INPUT	= OSCILLATOR
COUNT MULTIPLIER	= X1
CONTROL MODE	= LOCAL

Reset all interlocks and press RESET to turn the blue light off. Turn METER switch to DC error.

(f) After checking to see that the DC error is zero, press the HYDRAULIC PRESSURE switch once to light the LOW indicator and then again to light the HIGH indicator. With the SET POINT control, ease the triaxial cell up until the rod which transfers force from the MTS load cell to the load cell on top of the sample is just touching the MTS load cell. Monitor the force transmitter to the sample on the HP recorder, and do not let it exceed ten pounds.

(g) Close the valve on top of the triaxial cell and, with the regulator, apply confining pressure to the sample. The confining pressure may tend to push the load cell and ram down while holding the rod up against the MTS load cell. When this begins to happen, adjust the SET POINT control to bring the load cell back into contact with the rod. Then continue applying confining pressure until the desired quantity is reached.

(h) Remove the vacuum from the soil and leave the sample in the drained condition. Membrane leaks may be detected by inserting the end of the hose leading from the test base into a small beaker of water. This is important when performing tests on wet samples, as the sample may tend to dry if the leak is large enough.

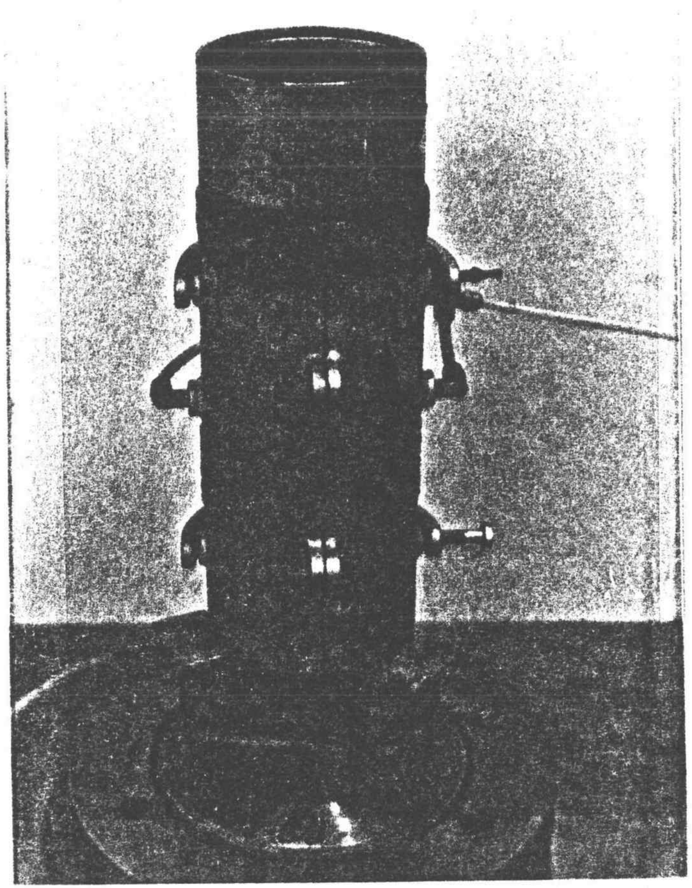

FIGURE A-4. Extension Collar Attached for a Final Lift of Soil

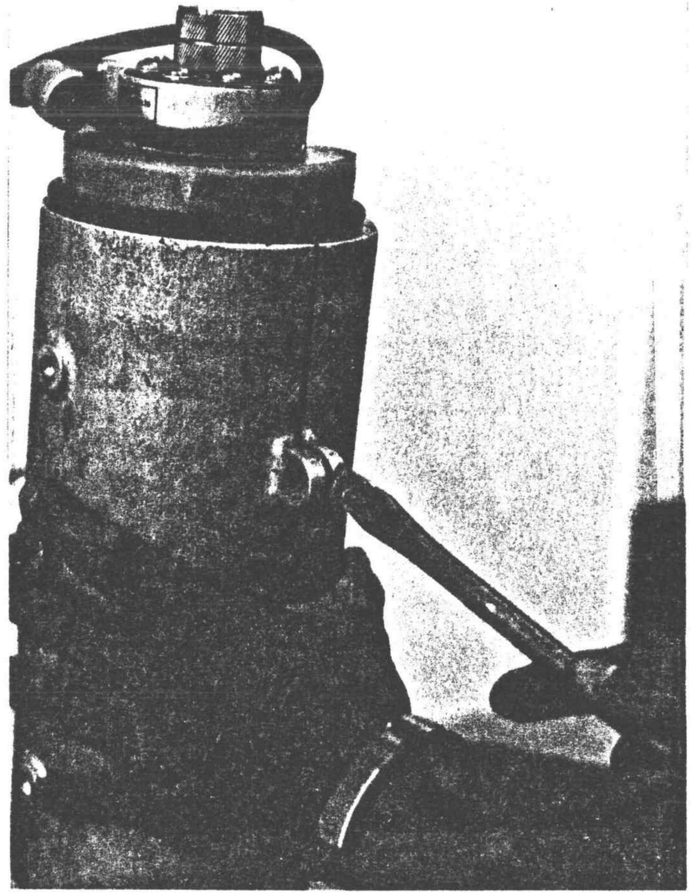

FIGURE A-5. Separation of Mold Halves with Screwdriver

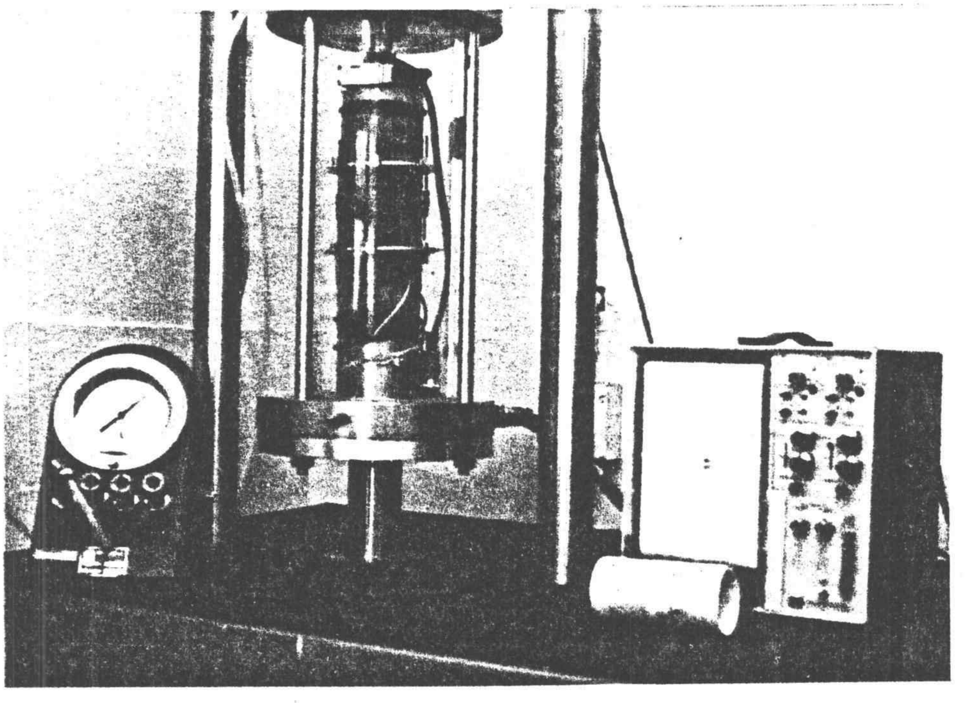

FIGURE A-6. Assembled Triaxial Cell

(i) Attach the dial gage to the MTS and obtain an initial height reading for the triaxial cell base.

(j) With a wave form selected on the DIGITAL FUNCTION GENERATOR panel, press the START button and increase the SPAN setting until the desired pulse amplitude is reached, as indicated on the HP recorder. As the SPAN setting is increased, the SET POINT control will need to be adjusted to bring the constantly applied load back toward zero. This constantly applied load is necessary in order to keep firm contact between the rod and both load cells. This constant load is more visible at lower confining pressures, which require lower deviator stress, and therefore, a more sensitive scale on the HP recorder. In no case should this steady load exceed ten pounds.

(k) To test for permanent deformation, apply a repeated load (σ_1 = 35 psi and σ_3 = 10 psi) for approximately 35,000 cycles. Check both the load pulse amplitude and the confining pressure several times during loading. Take dial gage readings so that the interval between readings is approximately double the previous interval, beginning at N = 10. (10, 20, 50, 100 ...).

(6) Mounting the LVDT's.

(a) Apply a vacuum to the soil.

(b) Press the STOP AT ZERO button on the MTS DIGITAL FUNCTION GENERATOR.

(c) Turn the regulator valve off.

(d) Use the SET POINT control to lower the triaxial cell. If the sample was preconditioned under high confining pressure, lowering the triaxial cell will allow some of the confining gases to escape, since the rod is held up against the MTS load cell. After the cell is completely lowered, release the remaining confining pressure with the valve on top of the triaxial cell.

(e) Remove the top plate of the triaxial cell and then remove the plastic confining cell.

(f) Place the bottom LVDT clamp about three inches up from the bottom of the sample. Use a rubber band to hold the clamp in place. Gently set the three 3.9" wood blocks on the mounted LVDT clamp with the top of the blocks leaning in against the sample and secure in place with string. Place the remaining clamp which holds the LVDT cores on top of the three wood spacers. Again, use a rubber band to hold the clamp in place. Squeeze the two clamps together using the 3.9" blocks to create a four-inch gage length from center thickness to center thickness on the two clamps. Take special care when placing the clamps to make sure that they are level and that the rods holding the LVDT cores are lined up with the center of the holes in the LVDT clamp. Placing a doubled thickness of paper between the knurled nut and the top LVDT clamp will aid in centering the LVDT in the bottom hole.

(g) Connect the cable from the LVDT's to the test base. Connect the cable from the HP recorder to the test base. Set the ZERO SUPPRESSION POLARITY switch to OFF, the OFFSET control to 10.0, and the CAL control to 0.00.

(h) Put one of the LVDT's into its hole in the LVDT clamp so that the core is near the middle of the LVDT. With the recorder SENSITIVITY control on OFF, zero the pen on the center of the chart using the PEN POSITION control. Using the 1 mm/sec chart speed and gradually increasing the SENSITIVITY control, move the LVDT so that it is 0.01" below its neutral point. This will correspond to a chart pen deflection of 12.5 mm to the left of center on the 2 mV/V/FS SENSITIVITY control setting, since only one LVDT is being used. Firmly secure the LVDT in its proper position, making sure that the LVDT is not twisted and thus hindering the free movement of the core.

(i) Repeat the last step for the remaining LVDT. Now the total chart pen deflection should be 25 mm to the left of zero on the 2 mV/V/FS SENSITIVITY setting. This corresponds to a distance of 0.01" from the neutral point, since both LVDT's are being used.

(7) Test for Resilient Modulus

(a) Reassemble the triaxial cell. Raise the head with the SET POINT control.

(b) Apply the required confining, steps 5e to 5h above. Remove the vacuum.

(c) Apply the laod pulse to the soil by setting the SPAN control to zero and then using step 5i above. When the confining pressure and the load pulse amplitude are both correct, press the COUNTER RESET button and condition the soil for 100 repetitions.

(d) During conditioning, use the ZERO SUPPRESSION POLARITY switch and the CAL control to get the deflection pulse centered on the chart paper. Then increase the recorder SENSITIVITY until the maximum pen movement is achieved without going off the chart paper or lighting the OVERLOAD indicator. Then the initial LVDT offset of 0.01" was incorrect and needs to be redone in order to decrease the amount of zero suppression required. If the OVERLOAD indicator light is due to too much positive zero suppression, then an initial LVDT offset of more than 0.01" is required. Conversely, too much negative zero suppression implies that the initial offset should be less than 0.01".

(e) After conditioning the sample for 100 repetitions at a particular confining pressure and deviator stress, take a reading of load and deflection using the 25 mm/second chart speed. The zero suppression for the LVDT's may have to be modified for each reading.

(8) Measure the final water content.

(a) Use steps 6b to 6e to begin removing the sample. Notice that no vacuum is necessary here.

(b) Remove the LVDT's. Remove the LVDT clamps and scrape all the dried epoxy off them with a screwdriver. Unhook all cables from the test base. Remove the two remaining long bolts from the test base. Remove the load cell from the top of the sample.

(c) With the membrane still taped to the test base, empty the soil into a pan, keeping track of the soil which came from the top, middle, and bottom of the soil column. Place some soil from each of these three regions into a pan to be weighed for a water content determination.

Figure A-7. Resilient Modulus Data Sheet

CONFINING PRESSURE σ_3 - psi	PARAMETER	STRESS RATIO σ_1/σ_3: 2.0	2.5	3.0	3.5	COMMENTS
3		1	2	3	4	
	DEVIATOR STRESS σ_d	3	4.5	6	7.5	
	σ_1^2/σ_3 (psi)	12	18.75	27	36.75	
	σ_1 (psi)	6	7.5	9	10.5	
	$\theta = \sigma_1 + 2\sigma_3$ (psi)	12	13.5	15	16.5	
	LOAD REQUIRED (lb)	75.4	94.2	113	132	
	SPAN 1 SETTING (VOLTS) (APPROXIMATE)	0.372	0.465	0.558	0.651	
	ϵ_e (IN/IN)					
	$M_R = \sigma_d/\epsilon_e$ (psi)					
5		5	6	7	8	
	σ_d	5	7.5	10	12.5	
	σ_1^2/σ_3	20	31.25	45	61.25	
	σ_1	10	12.5	15	17.5	
	θ	20	22.5	25	27.5	
	LOAD REQUIRED	126	157	188	220	
	SETTING	0.62	0.775	0.93	1.09	
	ϵ_e					
	M_R					
10		9	10	11	12	
	σ_d	10	15	20	25	
	σ_1^2/σ_3	40	62.5	90	122.5	
	σ_1	20	25	30	35	
	θ	40	45	50	55	
	LOAD	251	314	377	440	
	SETTING	1.24	1.55	1.86	2.17	
	ϵ_e					
	M_R					
15		13	14	15	16	
	σ_d	15	22.5	30	37.5	
	σ_1^2/σ_3	60	93.75	135	183.75	
	σ_1	30	37.5	45	52.5	
	θ	60	67.5	75	82.5	
	LOAD REQUIRED	377	471	565	660	
	SETTING	1.86	2.32	2.79	3.25	
	ϵ_e					
	M_R					

APPENDIX B

The figures contained in this appendix are the results of the modified proctor test (AASHTO T-180).

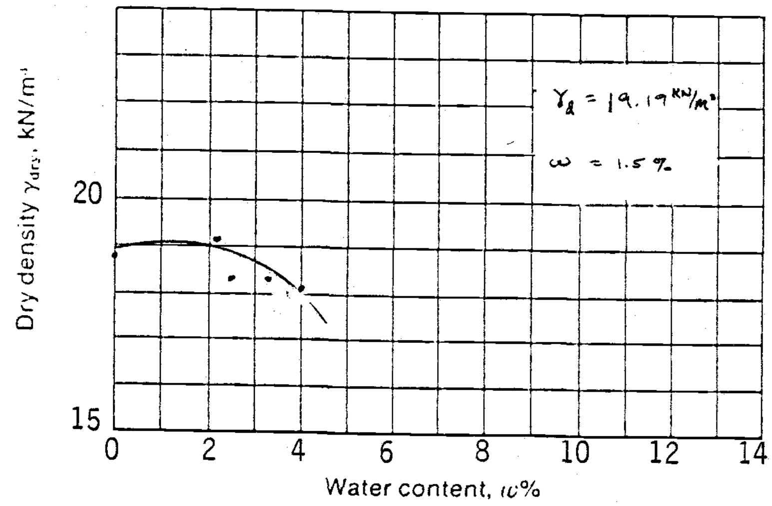

Fig. B-1 Ocean Lake, open graded moisture-density relationship

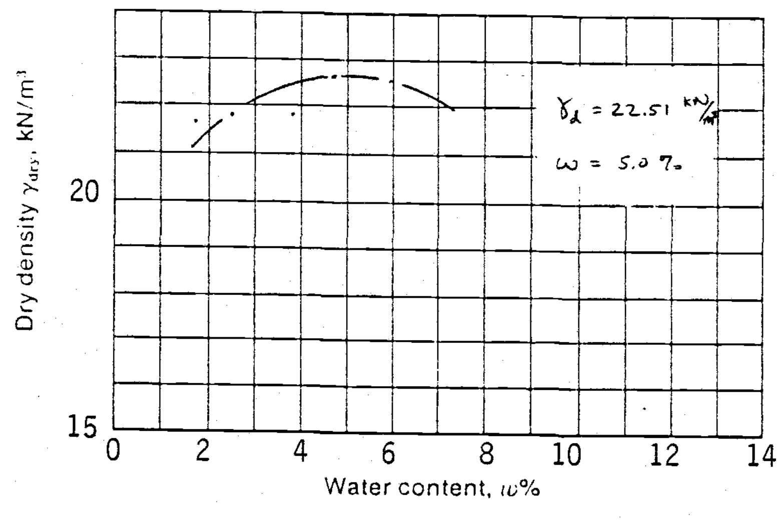

Fig. B-2 Ocean Lake, dense graded moisture-density relationship

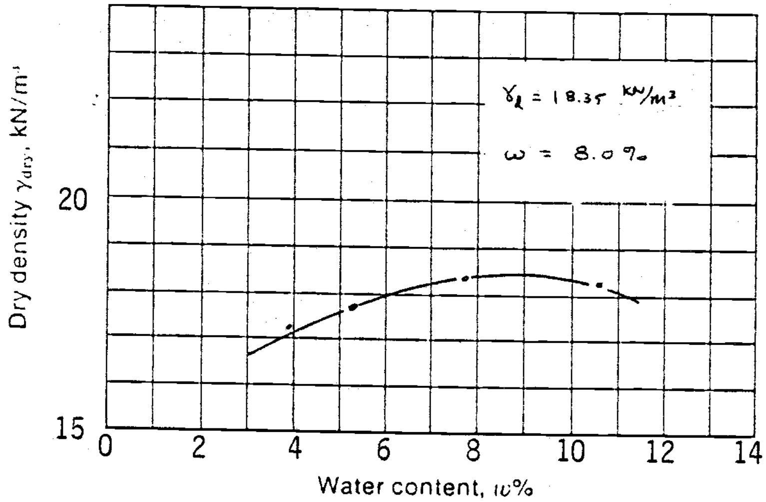

Fig. B-3 Big A sandstone, open graded moisture-density relationship

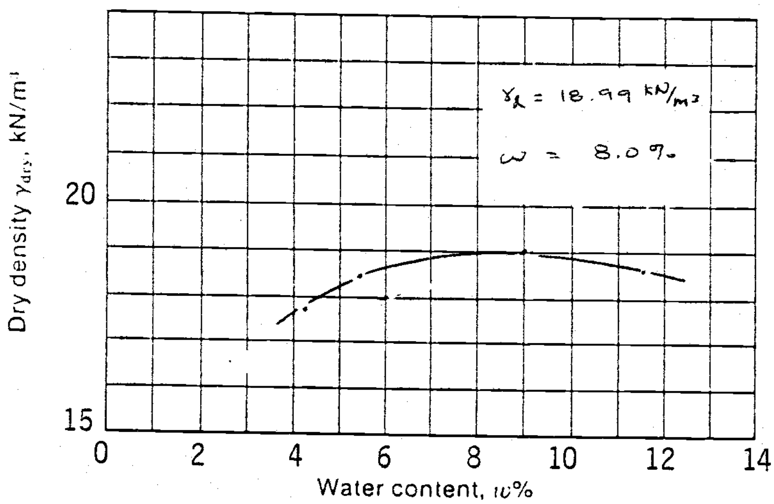

Fig. B-4 Big A sandstone, dense graded moisture-density relationship

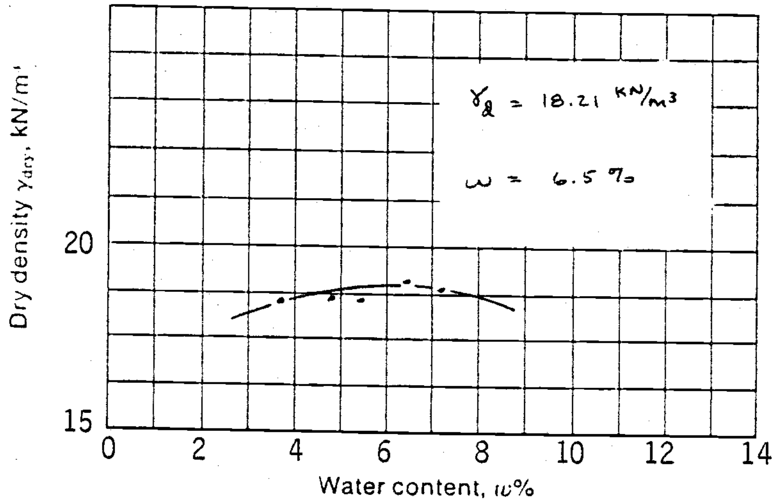

Fig. B-5 Eckman Creek, open graded moisture-density relationship

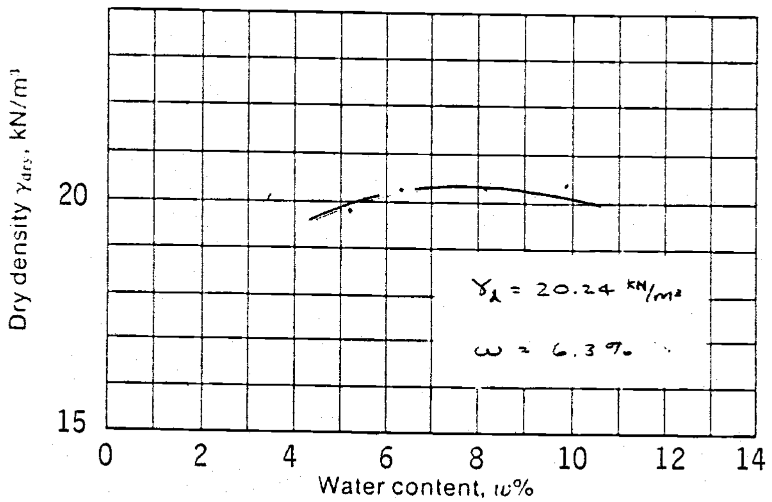

Fig. B-6 Eckman Creek, dense graded moisture-density relationship

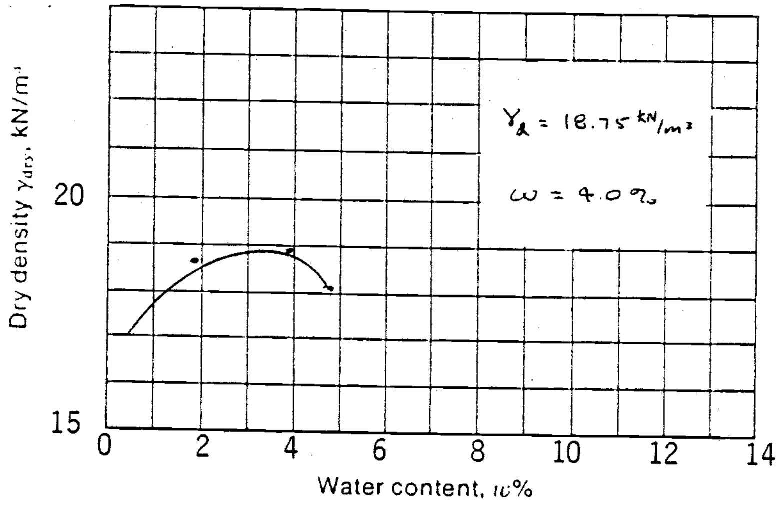

Fig. B-7 Blend, open graded moisture-density relationship

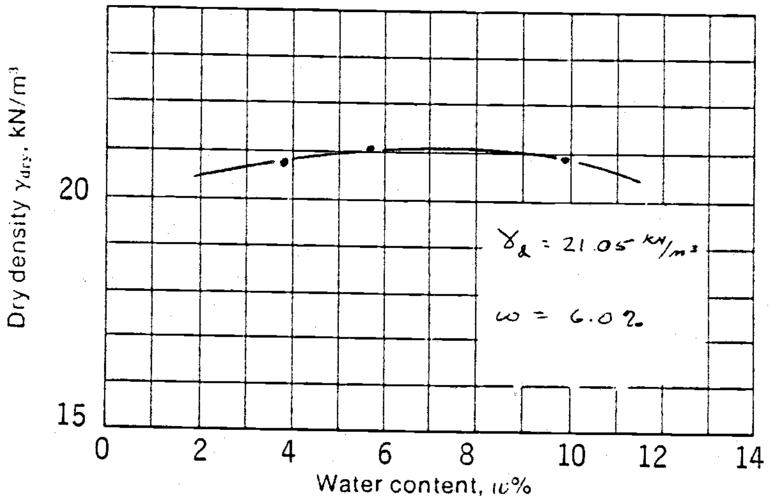

Fig. B-8 Blend, dense graded moisture-density relationship

APPENDIX C

PETROGRAPHIC RESULTS

This appendix presents the results of petrographic analysis performed by Dr. Harold Enlows, Department of Geology, OSU.

The durability of basalt is a function of the mineralogy of the deposit. Glass, which results from rapid cooling, is a component that is believed to cause rapid chemical degradation or weathering. Glass, being relatively unstable, will alter into deleterious by-products such as smectite clays. The smectite clays will expand significantly, resulting in fractures of the rock and subsequent plastic slurries in the base or subbase levels of the roadbed.

A ranking of the different basalt samples based on percent of glass and smeltite clay indicates that the breccia samples from Berry Creek and Yaquina Head are very poor quality. Berry Creek Breccia has approximately 47 percent glass and smectite while Yaquina Head Breccia is very high with 75.8 percent glass and smectite. These samples were suspected of having high glass and smectite contents and were tested to show that the mineral composition of an aggregate source varies. The Olivine sample from Yaquina Head has only 0.8 percent smectite and no glass, while Berry Creek Vesicular Basalt has 25.4 percent glass and smectite. The breccia samples are not truly representative of the quarry quality, since they represent only a small fraction of what is available, so they will not be considered in the ranking. Petrographic analysis was not conducted on the Ocean Lake aggregate because the sample selected did not contain any sizes large enough for thin slicing.

Based on these results, the following ranking, from high quality to low quality is applicable: Yaquina Head, Coffin Butte, Berry Creek and Eckman Creek. It is believed that Ocean Lake would approximate Yaquina Head in quality. These results substantiate the findings of the conventional durability tests.

TERMINOLOGY

The terminology petrographers use in describing textures of igneous rocks can be a bit confusing. Following are some definitions.

Aphanitic. Constituent minerals cannot be identified with the naked eye or an X10 lens.

Porphyritic. The rock shows two different sizes of minerals. The larger we term "phenocrysts" and they are enclosed in a "groundmass" of smaller grains.

Glomeroporphyritic. A porphyritic texture in which the phenocrysts tend to clump together.

Anhedral. Depending upon the number of crystal faces surrounding a grain we may term the grain:

Euhedral - if it is totally surrounded by crystal faces.

Subhedral - Partially surrounded by its crystal faces.

Anhedral - no crystal faces, grain quite irregular.

Alteration minerals. Following crystallization of the minerals from the magma, or the lava, hot waters or solutions escaping from the hot flow or cold waters from meteoritic sources (weathering) may alter the original minerals forming new ones. Often these are clay minerals.

Glass being metastable, is particularly apt to alter. If the glass is iron rich, as it is in basalts, it often forms green or greenish brown alteration products, clay minerals or chlorite.

Lithic. (rock xenolith).

Tachylyte. Black, basaltic glass, usually rich in magnetite.

Breccia. A rock formed of angular fragments cemented together.

Clast. Single rock fragment in a breccia.

REPORT SHEET G413

Name of Rock BASALT Number of rock and slide SG-1

Location Coffin Butte

Macroscopic Description

Dark gray, aphanitic rock with an even scattering of small olive gray to greenish amydules.

Microscopic Description

Texture Porphyritic, in places glomero-porphyritic. Phenocrysts of anhedral plagioclase and clinopyroxene up to 1 mm long in a diabasic groundmass. A number of round to irregular vesicles filled with green alteration product and a little interstitial glass altering to greed products.

Mineralogy

(1) Essential and major minerals

Phenocrysts Bytownite (An_{76})
Augite (appears to be a subcalcic augite)

Groundmass Plagioclase
Clinopyroxene (augite)
Magnetite
Brownish, magnetite rich glass

(2) Varietal mineral, if any

(3) Accessory minerals

(4) Alterations

Both glass and pyroxene altering to a green to greenish brown, cryptocrystalline to fibrous material of low birefringence. Suggest smectite (nontronite). The nontronite also fills vesicles.

(5) Remarks

Analyst H. E. Enlows

Results of a grain count

Plagioclase	233	- 31.9%
Pyroxene	254	34.7
Smectite?	120	16.4
Magnetite	75	10.2
*Glass	49	6.7
	731	99.9

*The glass is altering to smectite, so it is possible the smectite content should be increased and the glass decreased.

REPORT SHEET G413

Name of Rock Basalt Breccia Number of rock and slide SG-2

Location Yaquina Head

Macroscopic Description

A dark gray (N3), aphanitic, vesicular breccia with angular clasts varying in size from over 1 cm to less than 1 mm in average diameter. Occasional feldspar laths up to 2 mm long can be seen in the clasts. Some clasts appear to be "welded" together but many are cemented by calcite. The cementation is very firm. Many vesicles are filled with calcite.

Microscopic Description

A poorly sorted breccia. Clasts of porphyritic, vesicular basalt of at least two different textures are bound in a matrix of smaller fragments, apparently tiny fragments of tachylyte or palagonite.

Mineralogy

(1) Essential and major minerals

Clasts. The mafic clasts are porphyritic with phenocrysts of Plagioclase (An_{50} plus) and an occasional quartz grain. The majority reach lengths of 0.2 to 0.1 mm, or diameters of the same.

The matrix of the clasts are largely tachylyte containing many tiny microlites of feldspar and magnetite.

(2) Varietal mineral, if any

(3) Accessory Minerals

(4) Alterations

Vesicles may be filled with calcite, more generally with a green birefringent material (smectite?), more rarely with zeolite or nests of anhedral quartz grains.

The tachylyte is in places altered to a birefringent green material (smectite?) or to a feebly birefringent brownish palagonite.

(5) Remarks

Analyst H. E. Enlows

Grain Count

Tachylyte	-	378	-	50.0%
Plagioclase		48		6.3
Smectite		95		12.6
Palagonite		100		13.2
Calcite		131		17.3
Quartz		3		Tr
Zeolite		1		Tr
		756		99.4

REPORT SHEET G413

Name of Rock Olivine Basalt Number of rock and slide SG3

Location Yaquina Head

Macroscopic Description

Medium dark gray (N4) aphanitic rock coated with an olive gray material on fractured surfaces.

Microscopic Description

Texture Porphyritic. Phenocrysts Plagioclase (An_{54}plus) laths up to 0.6 mm long, some with reaction rims. Quartz, also with reaction rims. Olivine altering to iddingsite.

Groundmass A mixture of glass, magnetite and tiny devitrification crystals in the glass, small placioclase laths and small pyroxene crystals.

Mineralogy

(1) Essential and major minerals

Plagioclase	Some lithic fragments (xenoliths) are present.
Olivine	These are similar to the rock itself but
Rare quartz	slightly different in color, coarser texture
Clinopyroxene	and show a little flow texture.
Magnetite	Several odd masses consisting of an intergrowth of brown glass and clinopyroxene surrounded by a reaction rim of magnetite.

(2) Varietal mineral, if any

(3) Accessory minerals

(4) Alterations

Olivine to iddingsite.

Several veinlets of smectite not over 0.05 mm wide but extending across the thin section.

(5) Remarks

The quartz phenocrysts are surrounded by reaction rims consisting of brown glass, clinopyroxene and magnetite.

Many plagioclase phenocrysts are skeleton crystals with reentrants and inclusions of brown glass.

Analyst H. E. Enlows

Grain Count

The fine groundness of glass, magnetite, small plagioclase laths and pyroxene was simply counted as "groundmass".

The Olivine was extensively altered to iddingsite, so both were recorded. Skeleton crystals of plagioclase were interesting and recorded seperately.

Groundmass	469	76.5%
Plagioclase phenocrysts	83	13.5
Plagioclase skeleton crystals	19	3.1
Glass and clino-pyroxene masses	5	0.8
Quartz phenocrysts	6	0.8
Smectite veinlet	5	0.8
Olivine	13	4.2
Iddingsite	13	4.2
	613	99.7

PETROGRAPHIC REPORT

Name of Rock Basaltic (or andesitic) Breccia

Location Berry Creek　　　　Number of rock and slide SG 4b

Macroscopic Description

Volcanic breccia, olive gray, clasts up to 3 cm in diameter but average nearer 5 mm, many as small as 1 mm or less. Most clasts dense, some scoriaceous, some fresh, some weathered or altered, angular to subrounded.

Microscopic Description

Texture

Breccia, clasts angular to subrounded cemented by calcite. Clasts seem to be textural variations of the same rock type. (Berry Creek Basalt or Andesite?) Glass and plagioclase predominate, clinophyroxene less important and seen usually as small groundmass grains. Magnetite accessory. Occasional large plagioclase and pyroxene clasts seen. The following seem to be major rock types.

(1) Largely dark glass with plagioclase microlites in flow lines. Plagioclase near An_{50}.

(2) Largely plagioclase phenocrysts set in plagioclase microlites with no flow texture. Phenocrysts of plagioclase up to 6.5 mm long, An_{58}. Lesser amounts of glass and clinopyroxene not largely altered to a brownish green material, smectite? Minor magnetite.

(3) A vesicular variety of (1), the vesicles filled with smectite? and smectite replacing some of the dark glass.

Cement

Both calcite and smectite? are found in interstitial spaces as cement.

Grain Count

The grain count was made as if the rock were homogeneous, no account was made of porosity or different rock types.

Glass	170 points	28% of total rock
Plagioclase	164	27
Magnetite	34	6
Pyroxene	23	4
Smectite	114	19
Calcite	95	16
	600	100

Remarks

Rounding of the clasts indicate some transport or perhaps working by moving water. Many of the clasts were strongly altered to smectite, perhaps by contect with heated waters. Some glass appeared fresh, other masses strongly altered to smectite.

Analyst H.E. Enlows

REPORT SHEET G413

Name of Rock Vesicular basalt Number of rock and slide SG 4

Location Berry Creek

Macroscopic Description

Medium dark gray (N4), aphanitic and finely vesicular.

Microscopic Description

Texture

Porphyritic and finely vesicular.

<u>Phenocrysts</u> Plagioclase (An_{60} plus), laths up to 1 mm long Clinopyroxene, rather rare

<u>Groundmass</u> Smaller plagioclase laths (An_{50}plus), clinopyroxene, magnetite and brown glass

Mineralogy

(1) Essential and major minerals

Plagioclase

Clinopyroxene

Glass

Magnetite

(2) Varietal mineral, if any

(3) Accessory minerals

(4) Alterations

(5) Remarks

Extremely fresh

Analyst H. E. Enlows

Point Count

		Mineral percent	Total content including porosity
Plagioclase -	246 -	40.5% -	34.3%
Clinopyroxene	128	21.1	17.8
Glass	154	25.4	21.4
Magnetite	79	13.0	11.0
		100.0	
Pores	111		15.5
			100.0

PETROGRAPHIC REPORT

DETRITAL SEDIMENTARY ROCKS

Sample Number SG 5 Formation ______ Age ______

Location Curry Co. Rock Name Volcanic Arenite

Classification used Gilbert

Description of Hand Specimen

A medium gray (N5), well-cemented sandstone, grains angular and less than 1 mm in average diameter. Clasts appear to be predominantly mafic volcanic fragments with minor feldspar and pyroxene.

Observations both microscopic and macroscopic

Grains angular to subrounded, some elongate and flattened.

Framework: 91 % of rock

Porosity 0 Median grain size 0.3 - 0.5 mm Sorting Fair

Mineralogy

Mafic volcanic rock clasts.

Various kinds in various stages of alteration, many highly vesicular but flattended and often shaped by harder grains to occupy intersticial positions.

(1) Dark, devitrified mafic glass in various stages of preservations. Alters to smectite?

(2) Vesicular glass with flow texture, altering to green alteration products.

(3) Porphyry, plagioclase phenocrysts in a glassy matrix.

(4) Epidote and quartz or epidote and feldspar. It is suggested that somewhere in the provenance hydrothermal solutions had been at work forming epidotized and silicified rock masses.

clasts (cont.)

(5) Scoria, vesicles filled with smectitie and calcite.

(6) Silicified feldspar grains.

Mineral Costs

Plagioclase, some fresh some extensively altered to kaolinite, sericite or calcite.

Clinophyroxene

Hornblende

Quartz

Silicified rock fragments, or masses of fine-grained quartz.

Magnetite and Pyrite

Matrix: % practically none

Weak, vesicular grains apparently were deformed fitting into interstitial positions between the firmer mineral and rock grains.

% 9%

Cement:

Kaolinite
Smectite
Calcite
Fine masses of anhedral quartz grains

Inference and Conclusions

Maturity Quartz, quartzite, and chert % 3.5

Feldspars and granitic rock fragments % 15.3

Unstable fine-grained rock fragments % 71.0

Analysis of Maturity Misc. mineral fragments: 10.0
Hornblende, pyroxene, magnetite, epidote
immature

Provenance

Rock Types largely mafic volcanics. At least one area containing the results of silicification and epidotization.

Relief	High
Climate	Moist

Dispersal or Transportation Running water, short distance only

Depositional Environment

Analyst H. E. Enlows

Point Count

Clasts

Mafic volcanics -	410	71.1%		Smectite? -	32	56.1%
Plagioclase	88	15.3		Kaolinite	22	38.6
Clinopyroxene	27	4.7		Calcite	2	3.5
Hornblende	27	4.7		Quartz	1	1.6
Silicified grains	16	2.8				99.8
Epidote grains	1	Tr				
Quartz	4	0.7				
Magnetite	4	0.7				
		100.0				

PETROGRAPHIC REPORT

Name of Rock   Porphyritic basalt   Sample Number   SG 7

Location   Eckman Creek

Macroscopic Description

Greenish black (5GY 2/1), vesicular and porphyritic rock. Some vesicles filled with a green alteration product, probably smectite. The phenocrysts are large (up to 1 cm. long) plagioclase laths and smaller pyroxenes, the matrix dark and aphanitic, perhaps partially glassy.

Microscopic Description

Phenocrysts - Labradorite (An_{58}) and clinopyroxene

Matrix - Smaller plagioclase laths, smaller clinopyroxenes, magnetite and glass, now largely altered to a greenish-brown alteration product, probably smectite.

Grain Count

Plagioclase	334	55.7%
Smectite and glass	180	30.0
Pyroxene	60	10.0
Magnetite	26	4.3
	600	100.0