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Title: Properties of Marginal Aggregates Treated with Asphalt Emulsion

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The purpose of this report is to address the problem of supplying construction aggregates to Oregon's coastal areas that are currently, or will in the near future, be deficient of good quality materials. This is done primarily by studying methods of utilizing marginal aggregates that are abundantly available in the deficient areas. The methods considered include various beneficiation procedures, such as admixture stabilization, pretreatment, blending, and reinforcement. Beneficiation by admixture stabilization is further analyzed by an experimental program designed to evaluate properties of marginal aggregates treated with asphalt emulsion. The aggregates tested include a high quality basalt, two marine basalts, a sandstone, and a fine grained dune sand. Mix properties evaluated include diametral resilient modulus, split tensile strength, and diametral fatigue life for both unconditioned specimens and specimens conditioned by moisture exposure. Layered elastic system pavement design principles are then implemented with the dynamic test results obtained from the study aggregates and with typical properties of a hot mix asphalt concrete to determine required pavement thicknesses. From this information, layer equivalencies are determined for comparing required pavement thicknesses of emulsion treated marginal aggregates with a hot mix asphalt concrete. Structural layer coefficients are

then derived for use in standard pavement design methods. Finally, conclusions are given for utilization of the marginal aggregates treated with asphalt emulsion and recommendations given for additional research.

Properties of Marginal Aggregates Treated with Asphalt Emulsion

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1.0 INTRODUCTION

This report is one of a series of reports evaluating the problem of supplying construction aggregates to Oregon's coastal areas that are currently, or will in the near future, be deficient of good quality materials. This problem is exemplified by current practices of importing quality aggregates to deficient areas. The origins and destinations of some of this imported construction material are shown in Figure 1.1. As seen from the figure, much of this aggregate comes from the Willamette Valley, resulting in poor energy usage and high costs from trucking aggregate through the coastal mountains and returning empty. The legislative ban on usage of proposed and existing rock quarries in this area provides additional problems (4). As fuel costs will continually increase, land use and environmental legislation and resource availability limit the amount of quality aggregate that could potentially be exported, alternative methods must be developed to supply the coastal area with construction material.

1.1 Problem Definition

The major problems evolving from this lack of quality materials are 1) high construction and energy costs resulting from using imported quality aggregates, and 2) potential of early distress and poor performance of roadways constructed of marginal aggregates. Some of the tradeoffs between using quality and marginal aggregates are given in Table 1.1. To ease or eliminate these problems, various alternatives are available

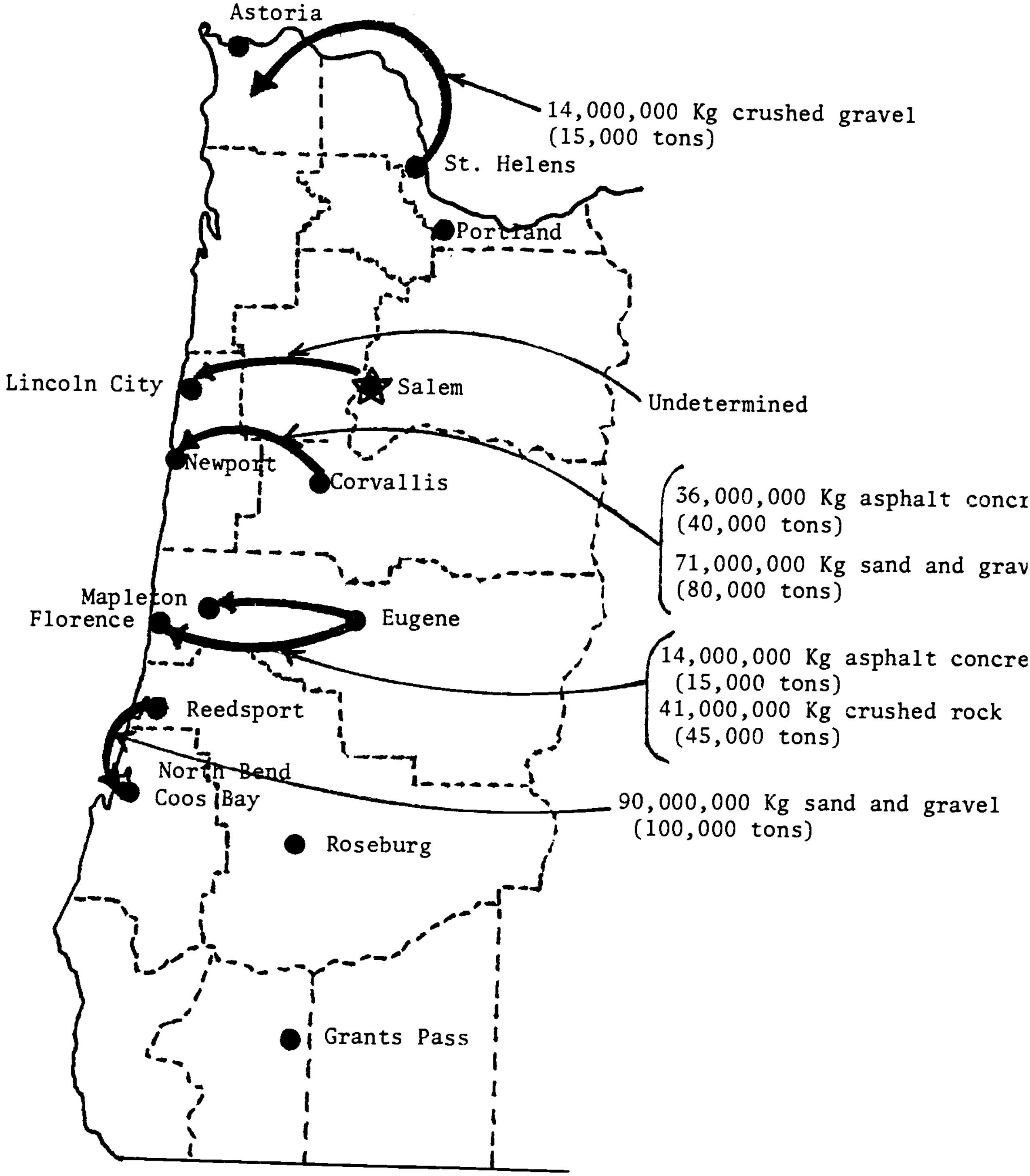


Figure 1.1. Annual Aggregate Importation to Coastal Oregon
(Source: Reference 1)

Table 1.1. Tradeoffs Between Good Quality and Poor Quality Aggregate (2).

Good Quality Aggregate	Poor Quality Aggregate
Advantages	
1. High quality.	1. Use of local material, resulting in energy conservation.
2. Good performance.	2. Low cost.
3. Development of new technology not needed.	3. Preservation of good resources.
4. Can directly be used.	4. Not against zoning regulations.
	5. Not against public opinion.
	6. Can be used through beneficiation.
Disadvantages	
1. Need to import, resulting in excessive energy consumption.	1. Low quality.
2. High cost.	2. Performance questionable.
3. Depleting good resource.	3. Development of new technology possibly required.

for the coastal area. As shown in Table 1.2, there are numerous advantages and disadvantages to the predominant methods currently available. In this report, emphasis is given to beneficiation as a means of providing the required materials.

1.2 Purpose of Study

The fact that many areas along Oregon's coast are deficient in quality construction aggregate has been well defined and documented in previous publications (1,2,3,4). This deficiency has been studied in light of the extent of the shortage (4), the aggregate needs and problems in the coastal area (3), alternative ways of supplying aggregates (2), and properties of untreated marginal aggregates found abundantly in the deficient areas (1). The results of these studies have indicated a need to determine the feasibility of treating locally available aggregates with low cost stabilization admixtures to provide acceptable paving mixtures. Studies have been performed and are currently underway studying the properties of coastal marginal aggregates treated with portland cement (5,6). The purpose of this report is to evaluate the option of treating locally available and abundant marginal aggregates with asphalt emulsions. A review of various treatment methods and previous experience is given, along with an extensive analysis of the properties characterizing the study aggregates when treated with asphalt emulsion. This study is concluded with design recommendations for use of the marginal aggregates treated in this manner.

1.3 Research Approach

The procedure followed in carrying out this study is shown in Figure

Table 1.2. Summary of Advantages and Disadvantages of Various Methods (2).

Alternatives	Advantages	Disadvantages
Importation	- High quality material can be used - Good performance	- High cost - Energy consumption - Depleting good resources - Environmental impact
Dredging	- Availability of low quality material - Use of previously wasted material - Improve navigation and mitigate flooding - Land improvement - Preservation of good resources	- Environmental impact - Cost impact by environmental requirements
Beneficiation	- Availability of lower quality materials - Can use local materials - Lower costs possible - Preservation of good resources	- Performance questionable - New technology possibly required
Mining Wastes	- Availability of local materials - Land improvements - Potential cost saving - Potential energy saving - Use of waste - Preservation of good resources	- Leaching possibility - Performance questionable
Recycling	- Materials conservation - Energy conservation - Preservation of investment - Cost savings	- Limited performance data - Environmental impact
Re-evaluation of Specifications	- Use of quality materials in most critical locations - Use of local materials - Preserves resources	- Education - Economics - Political factor
Re-evaluation of Land Use Practice	- More quality aggregate available - Potential low construction cost	

1.2. Initially, and throughout the study period, the most current relevant literature was gathered and reviewed. Various individual meetings were held with federal, state, and county officials, along with the aggregate suppliers who are responsible for the construction materials used in the coastal roadway system. Some of the agencies contacted are listed in Table 1.3. After general trends, problems, and possible solutions were determined for the coastal area, a "brainstorming" meeting was called to determine, more precisely, specific problems and discuss possible solutions for supplying acceptable aggregates to the shortage areas. In this meeting, the materials to evaluate, tests to perform, types of information needed by users, along with many other factors concerned with the project were discussed and evaluated by the professionals in attendance. The conclusions determined from this meeting, along with the literature review and various individual meetings, provided guidance in development of the project experiment design and design recommendations.

The experiment design involves the selection of test aggregates and the determination of their properties, the asphalt emulsion mix design, and the selection of tests and test procedures for the dynamic test program. The aggregates selected were those found to be most abundant and easily accessible. The basic physical and durability aggregate properties were determined and are reported in the experiment design. The mix design procedure used and the results obtained for the aggregates selected are also given in this section. The dynamic test program describes the procedures and results from modulus, strength, and fatigue testing laboratory fabricated specimens of the selected aggregates mixed with asphalt emulsion. The results from this are given in Chapter 4.

Upon completion of the experimental phase, the test results were

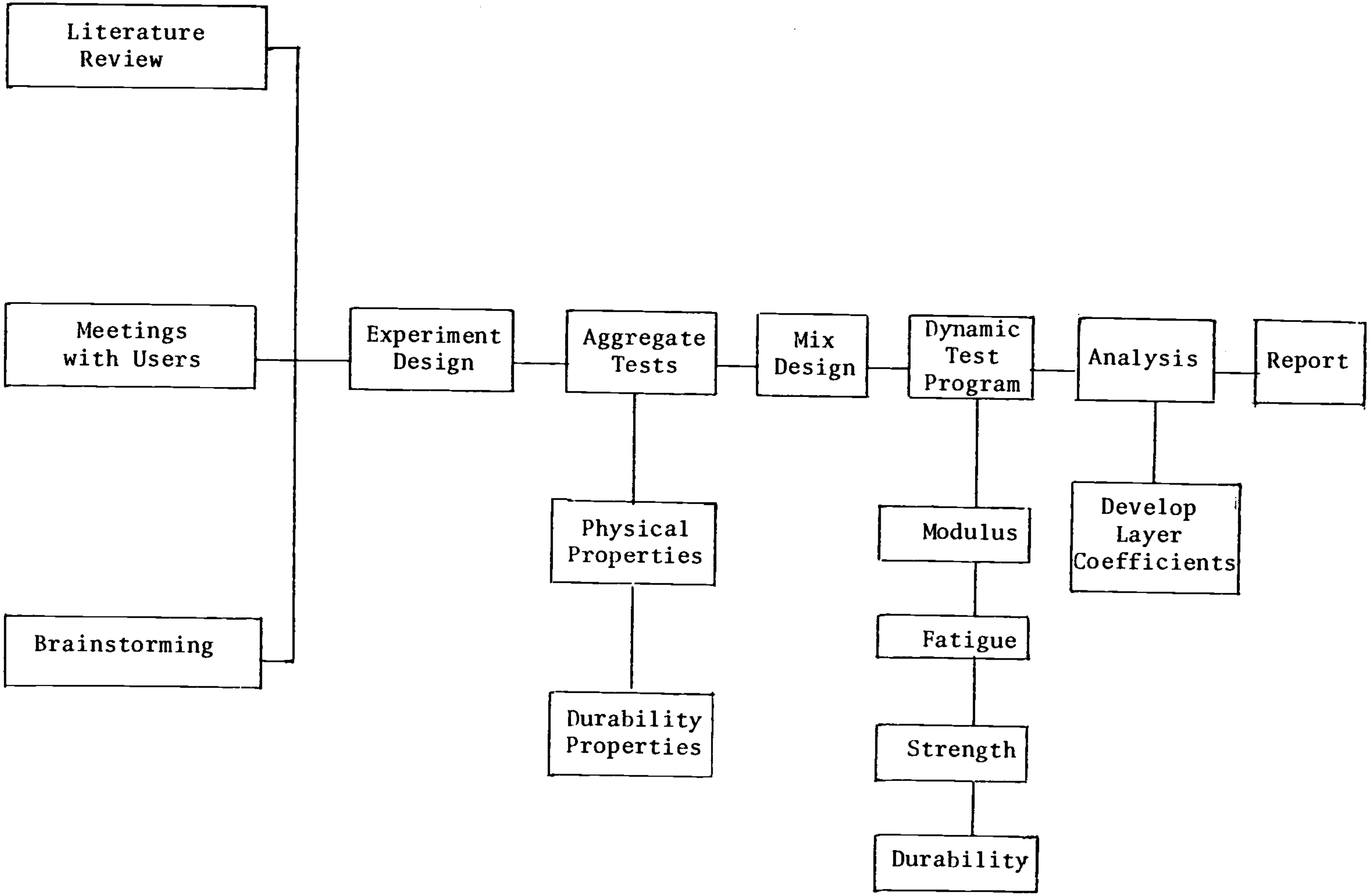


Figure 1.2. Research Approach

Table 1.3. Agencies Contacted for Project Study.

Federal	Federal Highway Administration Bureau of Land Management U.S. Bureau of Mines U.S. Forest Service U.S. Army Corps of Engineers
State	Oregon State Highway Department Division of State Lands Department of Land Conservation Oregon Coastal Zone Management Association, Inc. Department of Geology and Mineral Industries
County	County Engineers, Roadmasters, and Public Works Directors
Private	Morse Brothers, Inc. Wildish Sand and Gravel Oceanlake Sand and Gravel Eckman Creek Quarry Ray Wells Quarries Vern Stocker Montagne-Bierly Associates, Inc. Chevron, USA, Inc. Bohemia, Inc., Umpqua Division Al Pierce Lumber Co. Asphalt Pavement Association of Oregon Oregon Concrete Aggregate Producers Association Eugene Sand and Gravel Ross Island Sand and Gravel Company Johnson Rock Products Yaquina Head Quarries G and P Excavation

analyzed. This was done by simulating the different pavement mixes performance in typical pavement structural sections, and developing layer equivalencies for comparison with asphalt cement concrete mixes composed of quality aggregates. Finally, from this analysis, design recommendations are made.

2.0 METHODS OF UPGRADING MARGINAL AGGREGATES

The purpose of this chapter is to discuss potential beneficiation methods for marginal aggregates in the Oregon coastal area. The aggregates of primary interest (judged by availability and abundance), are marine basalts, which exhibit poor durablity characteristics, sandstones and siltstones, which provide low strengths because of poor grain interlock, and sands and dredged spoils, which have low stabilities and excessive amounts of fines. This chapter includes a discussion of the various methods available for upgrading marginal aggregates, background on previous experience of beneficiating the study aggregates, and recommendations for the treatment of these marginal aggregates.

2.1 Methods of Beneficiation Considered

Problems such as high plasticity, excessive degradation under repeated loading, or low strengths found in marginal aggregates can be mitigated by the addition of relatively small amounts of a second material. This addition of a small amount of the second material is referred to as admixture stabilization, and generally improves a road construction material's performance by increasing the strength, increasing the volume stability, and/or altering its permeability (7).

Pretreatment of marginal aggregates for improvement of properties before utilization in portland cement concrete and bituminous concrete pavements is a method which has recently received much attention and investigation (8). This pretreatment involves internal and external aggregate coatings and aggregate impregnation.

Removing deleterious particles from an aggregate supply by such mechanical means as heavy media separation, "jiggling," elastic rebound and other similar techniques could prove to be another practical method for improving aggregates susceptible to such separation measures.

The blending of marginal aggregates with higher quality materials will provide satisfactory road building material and is practiced regularly in many parts of Oregon (9,10).

A brief summary of costs for various alternatives is given in Table 2.1. The following sections give additional information on the beneficiation methods discussed above.

2.2 Admixture Stabilization

A great number of materials, both natural and synthetic, have been used or proposed for use as road construction material stabilizers. By far the most commonly used admixtures for soil stabilization are the cementing agents portland cement, asphalt, and lime. Mixes composed of these substances are often modified with additional admixtures (e.g., air entrainers, anti-stripping agents, etc.) to provide special effects and benefits. Some of these materials have been suggested for use based upon their chemical properties, while others have been suggested in hopes of finding a market or disposal location for industrial by-products.

Benefits gained from the chemical characteristics of the admixtures include: 1) the effect due mainly to chemical interaction with the soil, such as with lime, 2) the effect of the chemical due to interaction with the soil and to its own properties, as with portland cement, and 3) the effect due primarily to its own chemical properties, as with sodium silicate (7).

Most of the admixtures available are only effective for a limited

Table 2.1. Summary of Costs for Various Alternatives of Supplying Aggregate, 1979 (2).

Descriptions	Alternative of Supplying									
	Importation			Dredging	Cost of Beneficiation of Local Materials					
					Admixture Stabilization				Pretreatment	Reinforcement
	Truck	Railroad	Barge		Asphalt Treated	Asphalt Emulsion Treated	Portland Cement Treated	Lime Treated		
Transportation Cost ($/ton-mile)	0.05-0.13	0.075-0.122	0.006-0.015							
Cost ($/ton)				0.28-1.31	8-24	8-21	4-15	3-19	0.36-58.36	
Cost of fabrics ($/sq yd)										0.8-0.9
Installation Cost ($/sq yd)										0.05-0.10

1 ton = 1.016×10^3 kg.
1 mile = 1.609 km.
1 sq. yd. = 8.361×10^{-1} sq. meters.

number of soil types. In order to determine the admixtures most appropriate for Oregon's coastal aggregates, the advantages the admixtures provide must be defined.

Two major classifications of admixtures based upon their mechanism of stabilization are: 1) cementing admixtures and 2) waterproofing admixtures. These are listed in Tables 2.2 and 2.3. Other miscellaneous admixtures that have potential for improving the properties of the marginal aggregates are listed in Table 2.4.

The following paragraphs present a brief discussion of these divisions.

Cementing Admixtures - This group of materials includes admixtures which stabilize soils as a result of the cementation of material particles into a structural mass. The subdivision of non-reactive admixtures refers to instances where no chemical reactions between the admixture and the soil are required to provide the desired effects. Reactions between the admixture components and water may take place, such as the hydration of portland cement and the pozzolanic reaction of lime and flyash (7).

Reactive admixtures are those for which the stabilizing cement is a product of a chemical reaction between the admixture and some component of the material to be stabilized. An example of this is the cementitious materials produced by the reaction between lime and clay particles (7).

Waterproofing Admixtures - Waterproofing admixtures are added to reduce the effects of water on the performance of road construction materials. These admixtures may or may not directly impart stability to the roadway. Their principal advantage is in allowing compacted materials to retain their stability when exposed to a wet environment.

Table 2.2. Cementing Admixtures (After 7).

Non-Reactive Cements	Reactive Cements
Portland Cement	Portland Cement
Asphalts	Lime
Lime-Fly Ash	Acid and Alkaline Inorganic Chemicals
Cement-Fly Ash	Phosphoric Acid and Phosphates
Rosin (a natural resin)	Hydrofluoric Acid
Synthetic Resins	Sodium Hydroxide
Aniline Furfural	
Polyvinyl alcohol	
Polyvinyl acetate	
Resorcinal-Formaldehyde	
Synthetic Polymers	
Calcium Acrylate	
Polyurethanes	
Sodium Silicate	
Chrome-lignin	
Sulfur	
Cottrel flour	

Table 2.3. Waterproofing Admixtures (After 7).

Asphalts
Quaternary Ammonium Chloride Salts
Natural Resins
Vinsol
Shellac
Synthetic Resins
Aniline Furfural
Urea-Furfural
Siliconates
Alkyl Chlorosilanes
Amines

Table 2.4. Miscellaneous Admixtures (After 7).

Vegetable Oils
Cottonseed
Linseed
Tung
Castor
Mineral Oil
Molasses
Detergents
Paraffin
Plasticized Sulfur
Latex
Wood lignins
Polyester fiber

Miscellaneous Admixtures - Miscellaneous admixtures proposed for stabilization of construction materials are listed in Table 2.4. Although these materials have not historically received extensive usage, they might be considered as economical or technically viable alternatives in some circumstances.

The suitability and effectiveness of the principal admixture types for use with subgrade soils is summarized in Table 2.5. These recommendations are made based upon the gradation and plasticity characteristics of the soil and the previously defined characteristics of the admixtures. For instance, the rate and the extent of chemical reactions usually increases with the surface area available to participate in the reaction. Because of this, reactive cementing admixtures would be expected to be most effective for large surface area (fine-grained) soils (7).

Admixtures can be considered as cementing agents or as modifiers, generally being mixed with soil and then compacted in lifts. Cementing agents, such as lime, lime-pozzolan, portland cement, asphalt cement, sulfur and other chemicals, interact with the soil to produce structural materials. Modifiers tend to waterproof, dustproof, or otherwise improve the stability of the construction material. Modifiers include the above mentioned cementing agents, as well as dust palliatives, calcium chloride, salts, and other materials (11).

2.2.1 Asphalt

The principal classes of asphalt used in soil stabilization are asphalt cement, liquid asphalts, and asphalt emulsions. Asphalt cement refers to asphalt that is refined to meet specifications for paving, industrial use, and special purposes. Its penetration is usually between

Table 2.5. Admixtures Suggested For Use With Different Subgrade Soil Types (As from 7).

Soil Type	Most Effective Admixture Type
Coarse granular soils	Soil-asphalt, soil-cement, lime-fly ash
Fine granular soils	Portland cement, lime-fly ash, soil-asphalt, chloride salts
Clays of low plasticity	Portland cement, chemical waterproofers, lime
Clays of high plasticity	Lime

40 to 300, and is of a penetration grade or viscosity grade designation (12).

Liquid asphalt (also called cutback) is asphalt cement which has been liquefied by blending with a petroleum solvent. Upon exposure to atmospheric conditions, the solvents evaporate, leaving the asphalt cement to perform its function (12). Typically naphtha, gasoline, kerosene, diesel, or other oils are used as solvents to produce desired properties of Rapid-Curing (RC), Medium-Curing (MC), or Slow-Curing (SC) materials. Due to recent legislation, the use of cutback asphalts must be greatly reduced or eliminated. This restriction was brought about because of the excessive amount of hydrocarbons released into the atmosphere upon curing of the liquid asphalts (13).

Criteria for evaluating asphalt mixes consist primarily of 1) stability, 2) durability, 3) tensile strength, 4) flexiblity, and 5) fatigue resistance. The stability refers to the resistance of a mix to deform under loading. Stability, durability, and tensile strength require an intermediate amount of asphalt in the final mix to produce optimum properties. Flexibility refers to a mix's ability to conform to long term variations in base and subgrade elevations. For this property, as the asphalt percentage increases, its flexibility increases. The fatigue resistance refers to the ability to bend repeatedly without fracturing. Here, also, higher asphalt contents produce better results (11). The best mix possible for a given aggregate source would require a balance of the five properties discussed above. Most aggregates require about 4 to 7 percent (by weight) asphalt cement.

Field Experience With Local Aggregates - The Bullards Bridge-Bandon and Davis Slough-Bullards Bridge sections of the Oregon Coast Highway consist of plant mixed, asphalt-treated native materials (consisting of sand, pea-gravel, and terrace gravel) that have provided excellent performance. These materials were used with a low penetration grade asphalt to provide a much lower cost project than could have been obtained by importing quality materials (14).

A study was conducted in 1969 by the Oregon State Highway Division (15) to determine if asphalt and portland cement could be used to beneficiate materials such as 1) sand from the Winchester Bar of the Umpqua River, 2) vesicular basalt, 3) Kincheloe quarry marine basalt, and 4) argillite. For each of the materials, variations were made in gradation and binder content for the mix. Performance criteria were compressive strength and retained Hveem stability (retained meaning test results after simulated degradation) for portland cement and compressive strength, retained compressive strength, stability, and retained stability for asphalt concrete mixes. It was determined that for portland cement treatment of the sands, increased strength occurs with coarser gradations and increased cement content. For the marine basalts, increasing the cement content increased the strength in all cases, but as would be expected, was less pronounced for the coarser samples. The finer graded samples gave significantly higher strengths, but the percent retained stability was superior for the coarser samples. Asphalt treatment was not recommended for the sands because of poor resistance to moisture. Test values obtained for the asphalt treated marine basalt were thought to be below desirable limits, and therefore, cement treatment was recommended (15).

Sands have historically been beneficiated for construction materials in a number of projects using several different methods. Extensive testing on sand-asphalt and sand-cement mixes has recently been performed at the Georgia Institute of Technology (16). The criteria used to evaluate the mixes were properties of fatigue and rutting for sand-asphalts and fatigue and shrinkage for sand-cements. The most important advantage of using sand-asphalt bases compared with sand-cement was determined to be that shrinkage cracking is not a problem. In both cases the mixes were found to have lower strengths than conventional mixes. As a result, if not properly designed and constructed, pavements using sand-mixes have the potential for exhibiting excessive rutting and premature fatigue distress. It was also found that sand-asphalt mixes with high asphalt contents and low air voids have a significantly longer fatigue life. Greater fatigue resistance and workability were found to exist for gap graded mixes than for conventional mixes, while providing adequate levels of stability and durability. The addition of crushed stone to a pure sand mix was found to significantly reduce rutting (16).

Another study recently completed by the U.S. Army Engineer Waterways Experiment Station (17) concluded that asphaltic concrete can be made from almost any material with 100 percent passing the 38 mm (1-1/2 in.) sieve to about 15 percent passing the .075 mm (No. 200) sieve. The primary aggregates evaluated in this study were sand, clayey sands, clayey-gravelly sand, and blends to be used in limited traffic and short service life pavements. In the use of sand-asphalts, it was suggested that rutting might be limited by lateral containment of the mix, by paving the shoulders.

The Pennsylvania State University (18) has recently completed a study investigating methods of improving the water resistance of asphalt concrete composed of marginal aggregates. The methods considered here include the use of anti-stripping additives, surfactants added to the aggregate, hydrated lime, aggregate pretreatment, aggregate coatings, and sulfur-extended binders for asphalt concrete composed of marginal river gravels, granites, and a marginal basalt obtained from Roseburg, Oregon. One important finding resulting from this study is that the properties of the basalt mixture are greatly improved by treatment with hydrated lime. Test results indicate that the resilient modulus can be increased by 64% and tensile strength by 5% over the values of an untreated mix. Conditioned samples (vacuum saturated, frozen, and placed in hot water bath) demonstrated a 200% increase in modulus and 68% increase in tensile strength with the lime treatment, as well as a significant reduction in stripping.

In a study of the effect of moisture on the modulus of asphalt mixes, Schmidt (19) concluded that moisture resistance could be greatly improved by 1) use of harder grades of asphalt, 2) the use of the highest asphalt content feasible, and 3) pretreatment of the aggregate with a lime slurry. He also found that the detrimental effect of moisture is generally reversible upon drying of the mix.

2.2.2 Asphalt Emulsions

Emulsified asphalt is asphalt cement (~60%) prepared with a small amount of emulsifying agent (~1%) and water (~40%), resulting in the formation of minute globules of asphalt suspended in a medium of water. Depending upon the emulsifying agent, the asphalt particles in the emulsion may have either cationic, anionic, or nonionic surface charges. Emulsions may be produced with rapid (RS), medium (MS), or slow setting

(SS) characteristics, large or small residual asphalt contents and with various base asphalt types for various requirements of different aggregates and environments.

The use of asphalt emulsions provide several advantages, primarily by resulting in mixes that are versatile, economical, and low or non-polluting (20). Some of the advantages of open graded asphalt emulsion pavements are listed in Table 2.6. High production rates are possible with relatively low investments in construction equipment (20). This beneficiation method has also proven to be quite simple and economical. Construction costs, compared to those for hot asphalt cement, are greatly reduced because of the elimination of the operations of heating and drying the aggregate, aggregate screening, and maintaining the asphalt and aggregate at mixing temperatures. Since dryers are not needed to heat the aggregate, no smoke is produced, dust emissions are generally quite low, a fire hazard is eliminated, and further conservation of energy is obtained by reducing fuel costs (20,21,22).

Marine basalts have historically been used in open graded asphalt emulsion mixes and have provided very good results (24,25). One reason for using an open graded mix is to allow for minor degradation of the basalt with the resultant formation of a more dense pavement layer. This has been done on both Oregon State Highway Division (24) and Federal Highway Administration-Bureau of Land Management (25) projects.

The FHWA-BLM Surfacing Study Team (25) is currently investigating the use of sandstones and basalts treated with portland cement, asphalt, asphalt emulsion, and lime in a number of test roads in the Oregon Coastal area. They have found that several test sections comprised of open graded emulsion treated basalts and dense graded sandstones on top of

Table 2.6. Advantages of Open Graded Asphalt Emulsion Pavements (13,22,23,41).

1. Lower in cost than conventional dense-graded hot mix for high performance type pavements because of less equipment needed, mixes are prepared cold, This also gives a savings in fuel cost.

2. Good serviceability with low maintenance requirements.

3. Coarse textured pavement allows for better drainage from surface, increased skid resistance, and improved stripe visibility.

4. Elimination of aggregate drying and screening operation reduces plant costs and dust pollution at the mixing plant site.

5. A savings in the cost of energy needed to produce the paving mixture is realized by elimination of the aggregate screening operations and the need to maintain asphalt at operating temperatures.

6. Reduced fire hazards.

7. Reduction of asphalt oxidation as thicker asphalt films are used and heating is not required.

8. Pavements are highly resistant to cracking, even under heavy loads. These pavements have successfully carried heavily loaded logging trucks (up to 890 kN, 200,000 pounds, gross) without distress (23).

9. Improved pavement flexibility and increased fatigue resistance.

sandstone bases perform very well. A summary of some of the test roads currently under investigation by this team is given in Table 2.7. Of particular interest here is the Nestucca River Road. A test section of this road that has performed exceptionally well was comprised of an open-graded, emulstion treated marine basalt. The basalt typically met all of the aggregate specifications except for the Dimethyl Sulfoxide (DMSO) test, in which all 10 rocks would fracture. As discussed later, this is very similar to properties found for the Eckman Creek study aggregate.

The Siuslaw National Forest has used pure blow sand in two projects, the Sand Beach Park south of Tillamook and the Sutton Creek Beach Park north of Florence. The sand was mixed with CSS-1 emulsion and used as a surfacing mix, covered by a chip seal. Both of the roads, constructed by slightly different methods, have proven to be quite successful (26).

Douglas County has reported very good results from using asphalt emulsion and 1 to 2 percent portland cement with a sand aggregate in the construction of the Winchester Bay Road, County Road 252 (27).

Many other successful projects using asphalt emulsion and marginal aggregates, dune sands, beach sands, and alluvial sands, are well documented by Chevron, USA, Inc. (28).

2.2.3 Portland Cement

Of the more conventional stabilization techniques, cement stabilization is by far the most common (29). It may be added to all types of soils, except those having more than 1-2 percent organic material, and will have an effect broadly proportional to the amount of cement added (29). Cement requirements range from 4 to 15 percent by weight, typical-

Table 2.7. Summary of Major SST Test Roads (25).

Project	Construction Completed	No. of Sections	Surfacing Type	Surfacing Depth	Basis of Studies	Results to Date
Nestucca River Rd. BLM 10	Summer 1976	5	Std. bit. plant-mix; Rd. mix w/ seal coat; & Open grd. marine basalt plant-mix w/seal coat.	Pave 3" on 4" base; pave 4" on 2" base; pave 3" on 4" base (existing subbase under all sections).	Deep, steep canyon with degrading aggregates & high maintenance cost. Determine a pavement structure that will be suitable. Possibly upgrade local aggregate.	During 1st fall road-mix showed distress. This has not spread, considered a construction defect. No other distress to date.
Moon Creek Rd. BLM 1672	Under Contract	8 wedges	Local aggregate, marine basalts, and sandstone, emulsion mixes, w/seal coat.	3.5 to 9 inches, directly on subgrade (4 on level & 4 on grade).	Implement studies on degrading rock.* Establish tentative equivalency factors. Compare wear, flat and steep grades. (*Microscopically identify bad-actors.)	Under construction.
		3 Std.	Emul. tr. plant-mix base w/seal coat & base w/soil binder.	2" treated on 2" of water bound on 12" of sandstone sub-base. 4" on existing base.	Compare soil bound w/asphalt bound base material.	Binder provided misc. work, but contractor elected source contained binder quality & none was added.
Elk Creek Rd. BLM 629	Fall 1972	3	Bit. open graded water bound base lime treated base all w/seal coats.	10" (All on subgrade) 16" 10"	Upgrade local aggregate with an admixture to cut haul costs on good rock.	Microscopic examination of fragments recovered in the summer of 1977 showed: water bound slight degrading; asphalt tr. trace of degrading; lime treated still clear.
Berry Creek Rd. BLM 409	Summer 1975	9 steps	Base w/BST. Gravel w/P.I. Quarry base w/ S.E.	2",4",6", & 8" on 6" of topping. 8" on topping. 8",10",& 13".	Confirm "R" value design method. Compare gravel (50% fractured river rock) w/binder to quarry rock w/ fracture for serviceability. Check for pattern of rock loss under traffic. Demonstrate maintenance type light emulsion mat.	Under designed sections are now failing under the mat; however, the unoiled sections are not? Shifting of material under traffic shows: crowns become supers, & supers go flat, etc. Data still being gathered.

Table 2.7. Summary of Major SST Test Roads (25) (continued).

Project	Construction Completed	No. of Sections	Surfacing Type	Surfacing Depth	Basis of Studies	Results to Date
Blue Ridge Rd. BLM	Summer 1975	2	Open graded bit. treated base w/ seal coat.	2" on variable base and subbase according to "R" value design.	Can an open graded mat of minimum depth serve with adequate base underneath? Does a light dust treatment retain fines and reduce maintenance?	The bituminous mat is in good condition and being monitored. Maintenance costs on the spur roads cannot be separated out, so this phase is lost.
		4	Base treated and untreated w/dist. oil.	4" to 6" admin. des. spur roads.		
Whitcomb Creek	Summer 1974	none	Aggregate surfacing w/P.I.	Variable 4" to 8" on existing subbase.	An old spec was used because it fit the project needs: gravel surface, overburden on the listed source.	Maintenance was not as low as hoped, and rock loss appears excessive. More detailed study of other projects will be necessary.

1" = 25.4 mm.

ly increasing with increasing soil plasticity (11). Compacted soil-cement must contain sufficient cement to withstand standard laboratory freeze-thaw and wet-dry tests and meet weight loss criteria, and contain enough moisture for maximum compaction (30).

A study was recently completed by Oregon State University (6) to evaluate cement stabilization of poor quality aggregate sources of Waldport marine basalts, three gradations of Tyee sandstone, a moderately weathered granite and two types of decomposed granite. Of particular interest to this study are the results found for the marine basalt and sandstones. To sufficiently meet strength and durability requirements, the marine basalts were found to require a minimum of 6% (by weight) cement to be mixed with 13% water. The sandstones typically require 5% cement and 12 to 14% water, both materials producing 7-day cure strengths of greater than 3450 kN/m^2 (500 psi). Typical requirement ranges of unconfined compressive strengths for soil-cement mixtures with sandy and gravelly soils are 2070 to 4140 kN/m^2 (300 to 600 psi) (6). This study also determined six major factors that are important to control in cement stabilization projects. These factors are: gradation, moisture content, cement content, degree of mixing, and degree of compaction and curing. The greatest attention should be given to control of cement and moisture content (6).

2.3 Pretreatment

As previously stated, the pretreatment of marginal aggregates with external and internal coatings and impregnation of aggregate pores could prove to be a viable solution to the aggregate's usage. In a recent study by Pennsylvania State University (8), mitigation of aggregate problems of freeze-thaw resistance, alkali-silica reaction, alkali-carbonate reaction, and D-cracking (primarily in limestones) for portland cement concretes and stripping and degradation in bituminous pavements were evaluated. These problem areas with the upgrading materials selected for study are shown in Table 2.8.

One of the most important considerations for this method would be the cost effectiveness of the process and the materials used, as these factors will have as much bearing on their usage as the actual effectiveness of the treatment. Typical costs for various treatment materials studied by Pennsylvania State University are given in Table 2.9. As seen from this table, some balances can be found for high cost materials and low application requirements, however, this is not always the case. The availability of these materials to a localized area, such as the Oregon Coast, would also have a great influence on their cost and usage.

The application process would need to be considered for different treatment of the materials to be utilized. The primary methods for applying these materials are soaking, vacuum, pressure, solvent exchange, vacuum-soak, vacuum pressure, and combinations of the above methods (8).

A summary of recommendations for upgrading marginal aggregates for portland cement concretes and bituminous pavements are given in Figures 2.1 and 2.2, respectively. A summary of some of the conclusions determined by Pennsylvania State are as follows (8):

1. Freeze-thaw sensitive aggregates can be rendered innocuous by impregnation or coating, using a variety of treatment materials.
2. The use of impregnants, in general, does not appear to be

Table 2.8. Summary of Aggregate Problems and Treatment Procedures and Materials Selected for Penn State Study (As from 8).

Aggregate Problem Area	Selected Upgrading Treatment Procedures	Selected Upgrading Material*
I. Portland Cement Concrete Mixtures		
A. Freeze Thaw	Impregnation Coating	E, MMA, BLO, PEG, KL, and S E and LOE
B. Alkali-Silica Reaction	Coatings	E and LOE
C. Alkali-Carbonate Reaction	Coatings Admixtures	E and LOE DS, LC, and FC
D. D-Cracking	Impregnation	E, MMA, BLO, PEG, KL, and S
II. Bituminous Concrete Mixtures		
A. Stripping	Coatings Admixtures	E and KL HL, A, SD, and SA-1
B. Degradation	Impregnation	E and KL

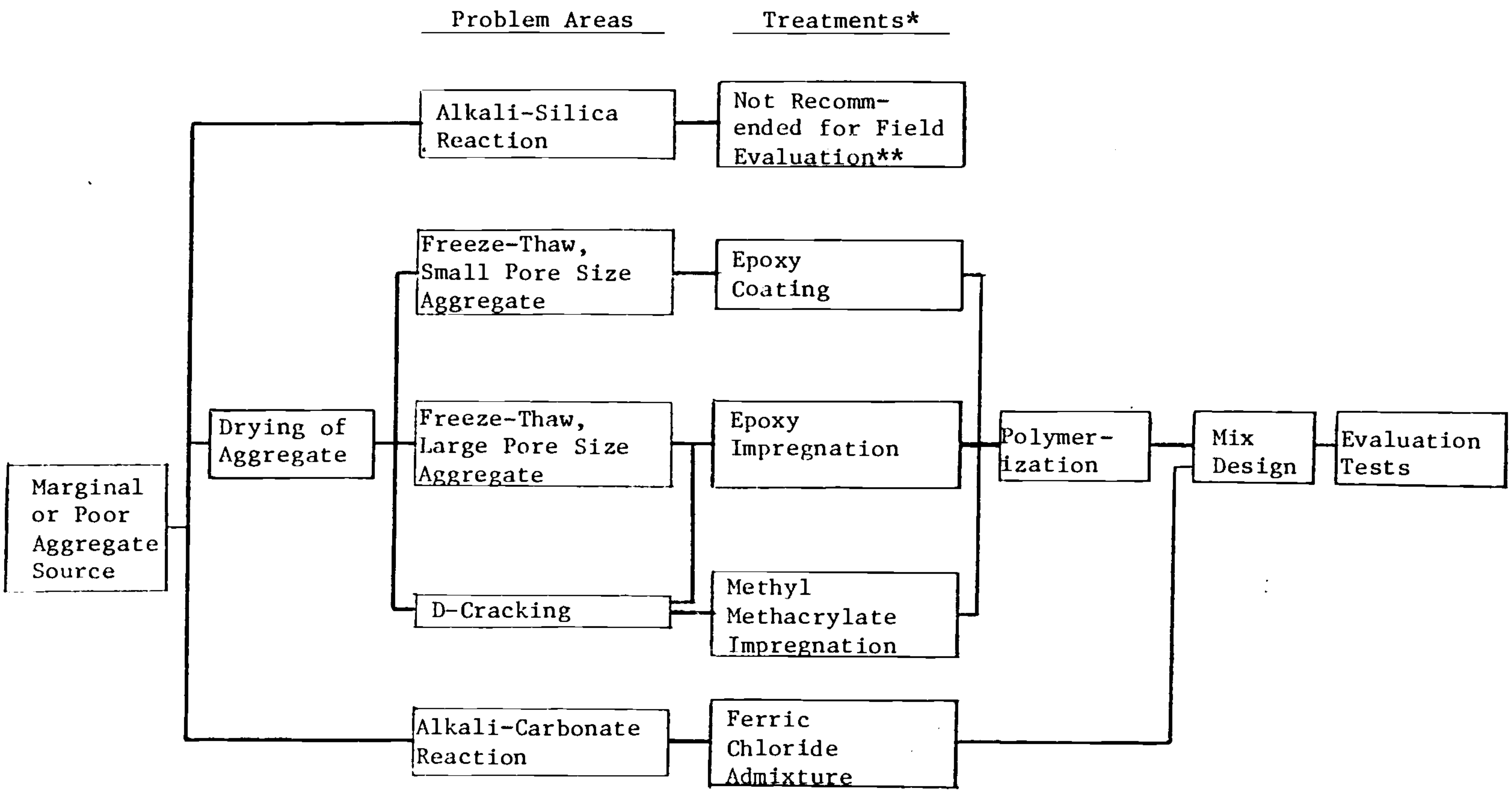
*Epoxy (E), Methyl Methacrylate (MMA), boiled linseed oil (BLO), ethylene glycol (PEG), Kraft lignin (KL), sulfur (S), linseed oil emulsion (LOE), dimethyl sulfoxide (DS), lithium carbonate (LC), ferric chloride (FC), hydrated lime (HL), amine (A), sodium dichromate (SD), and SA-1 pretreatment (SA-1).

Table 2.9. Estimated 1978 Costs of Treatment Materials (8).

Treatment Material	Treatment Rate (Percent)	Treat. Mat'l. Cost, \$/1,000 kg of Agg. Treated
Epoxy Coating (expoxy:triethylene tetramine 100:14.1 phr)	3.7 (a)	64.31
Linseed Oil Emulsion Coating	1.8 (a)	41.98
Epoxy Impregnation (epoxy:triethyl-ene-tetramine:xylene 100:14.1:25 phr)	2.4 (a)	35.38
Methyl Methacrylate Impregnation	2.6 (a)	36.04
Boiled Linseed Oil Impregnation	2.7 (a)	16.63
Polyethylene Glycol Impregnation	4.9 (a)	47.97
Sulfur Impregnation (sulfur: asphalt 100:2.5 phr)	4.1 (a)	4.51
Kraft Lignin Impregnation	3.0 (b)	3.96
Dimethyl Sulfoxide Admixture	1.0 (c)	1.63
Lithium Carbonate Admixture	5.0 (c)	14.07
Ferric Chloride Admixture	7.0 (c)	4.02
Acid Wash Pretreatment	0.6	23.00
Hydrated Lime Admixture	1.0	0.40
Sodium Dichromate Admixture	0.1	0.82
Amine Admixture	0.5 (d)	2.00

(a) By weight of aggregate; average of test aggregates.

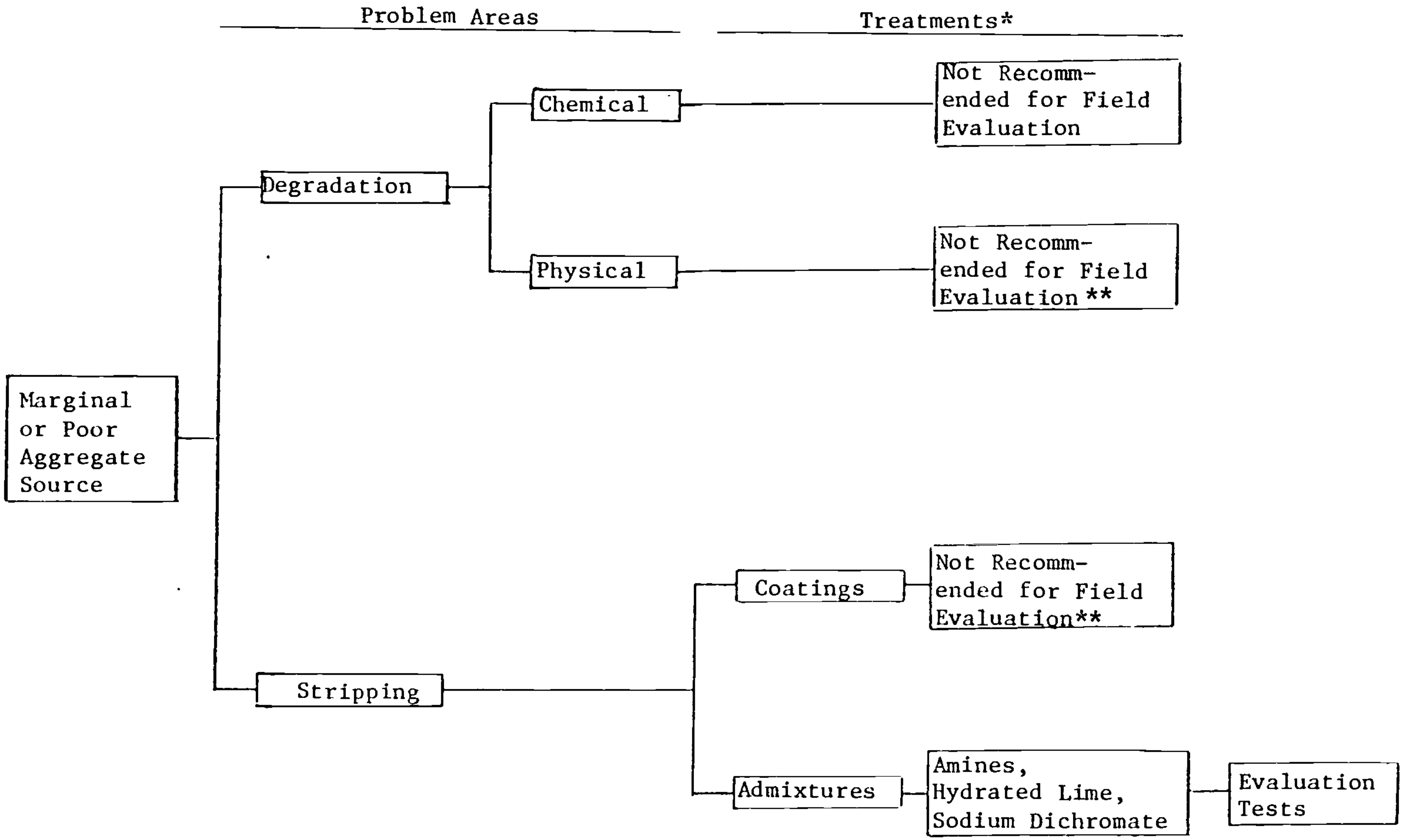
(b) Assumed value.
(c) By weight of mixing water; at maximum effectiveness.
(d) By weight of asphalt cement. Assumed 6% A.C. content in mixture.
(e) 1 lb. = 4.536×10^{-1} kg.



* The treatment methods and materials should not be interpreted as the optimal means of upgrading a marginal or poor aggregate. Modifications of other treatment methods and materials not listed above, but used in this study, may add to the list of successful upgrading treatments.

** Due to inconclusive results in the laboratory experiments caused by low reactivity of the test aggregates used.

Figure 2.1 Summary of PCC Aggregate Upgrading Recommendations. (as from 8)



* See footnote Figure 2.1
** Further laboratory study needed.

Figure 2.2 Summary of BC Aggregate Upgrading Recommendations.
(as from 8)

feasible for upgrading degradation susceptible aggregates for bituminous paving mixtures. Soft, easily abraded aggregates can, however, be improved by epoxy impregnation.

3. Several treatment materials appear to have detrimental effects on mechanical properties (particularly compressive strengths) of PCC mixtures.
4. Epoxy coatings adversely affect the mechanical properties (modulus) of some bituminous paving mixtures, especially those with rounded aggregates and softer binders.
5. Anti-stripping admixtures apparently have no ill effects on the mechanical properties of bituminous paving mixtures.
6. Unmodified raw kraft lignin is not a suitable impregnant or coating material for either PCC or bituminous paving mixtures. It reacts with the binder in both cases producing unacceptable mixes.
7. Treatment material costs are generally favorable for admixtures and impregnants (sulfur, kraft lignin, boiled linseed oil, etc.), but restrictively high for epoxy coatings.

Schmidt (19) concluded that pretreatment of aggregates with a lime slurry before combining with asphalt will greatly improve the mixes properties when exposed to moisture in terms of both stripping and retained modulus. As previously discussed, this benefit was also found by Chehovits (18) in studying modulus and tensile strength properties of a marginal basalt taken from Douglas County, Oregon.

2.4 Blending

The method of blending of marginal aggregates with quality aggregates to extend a quality aggregate supply is used extensively, but is not well documented. Both Benton County (10) and Coos County (9) among others have used this method to extend quality aggregate supplies. Many sands throughout the United States have been successfully used in asphalt bases and surface mixes by blending with other materials (31). The proportions of aggregates to be blended would be dependent upon properties of both aggregate supplies.

Barksdale (16) found that in blending crushed stone with pure sands, the rutting potential of the resulting asphalt mix is significantly reduced. Test results obtained by Grau (17) indicate that performance of flexible pavements with sand mixes is improved by the inclusion of coarser aggregate. Grau also suggests that the sand-aggregate ratio of a marginal material to be used in a zero-slump portland cement concrete mixture should be 25 or greater. In modulus testing, a 50-50 blend of the study aggregates, Oceanlake basalt and Eckman Creek marine basalt, Clemmons (1) found that the blend experienced less plastic strain than either the good or poor quality material tested separately. He also found in durability testing that for an open-graded mix, under heavy loading conditions, the benefit of blending these two materials is quite appreciable.

2.5 Reinforcement

Reinforcement, as used in this text, means the inclusion of foreign materials in a soil mass to increase stability and bearing capacity and reduce deformations. Of primary interest in this field is the use of geotechnical fabrics. The major fabrics currently available are composed primarily of polyethylene, polyester, or polypropylene fibers and can be in a woven or nonwoven form in a continuous sheet. They have historically been used for providing functions of reinforcement, drainage, filtering, separation, and erosion control in road building applications (32). Fabrics have been found to reduce aggregate depth requirements (26) and in some cases, they have effectively replaced lime stabilization (33). These benefits indicate that fabrics could be used to mitigate strength and stability problems found with marginal aggregates.

2.6 Methods Recommended For This Study

The percentage of materials passing the .075 mm (No. 200) sieve size and the plasticity of a soil have been found to be valuable guides to determine the suitability of lime, cement or asphalt as a stabilizing agent (34). Figure 2.3 presents a guide based upon these criteria. This guide presents a good basis for admixture selection, however, other factors must also be considered. For instance, it is generally recognized that asphalt is not a suitable stabilizer for materials that continue to break down under repeated loads (6). This would tend to make some marginal aggregates, such as different types of sandstones and siltstones, unsuitable for treatment with asphalt. Also, the use of lime-pozzolan cementing agents require a relatively good quality material to be effective, thus limiting their usage with some marginal aggregates (35).

In the case of a uniformly graded sand, high cement contents, ranging from 7 to 11 percent are required because of the amount of voids between soil particles that must be filled if the soil particles are to

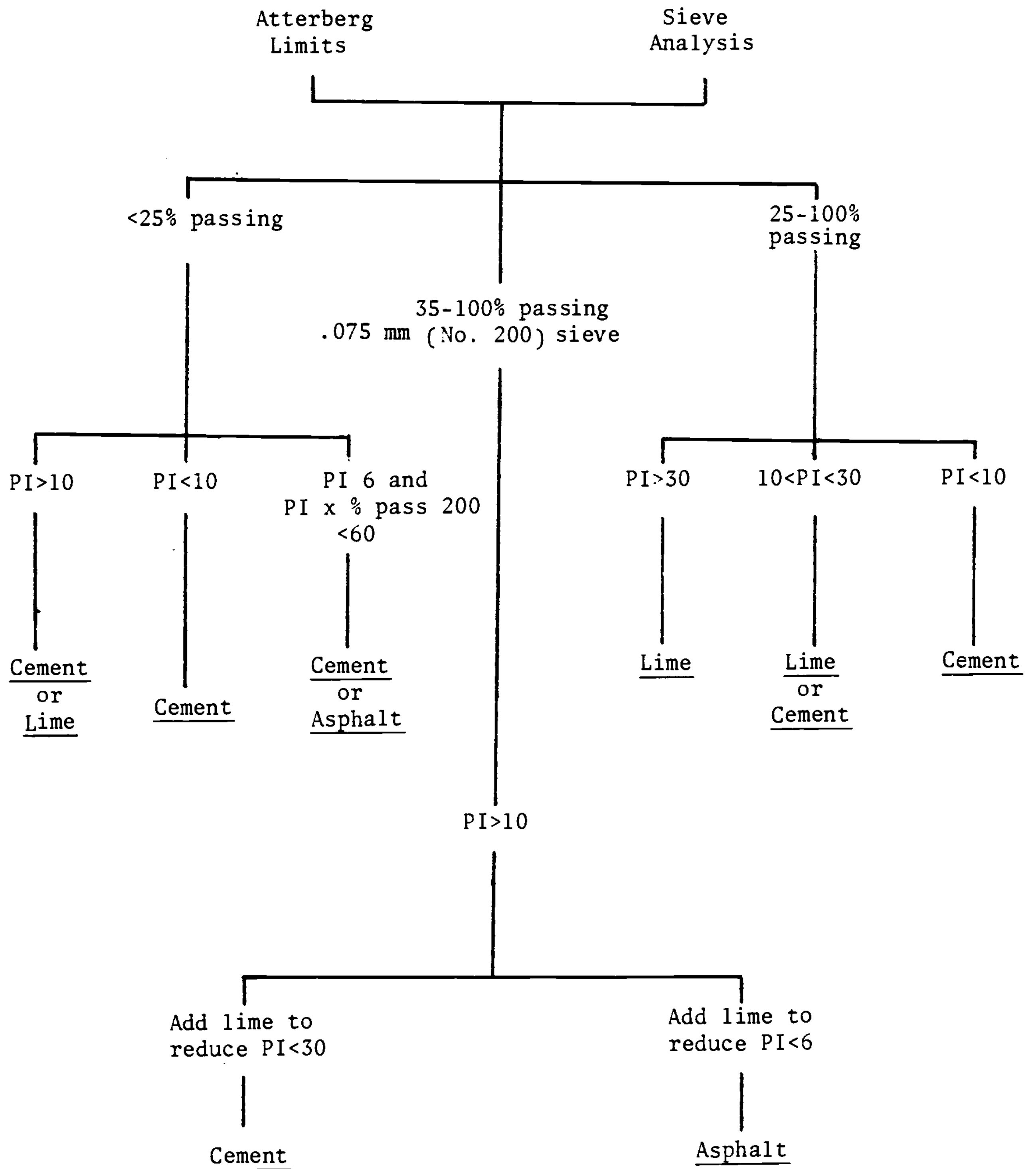


Figure 2.3 Guide For Choice of Stabilizers.
(as from 34)

be cemented together (36). For a fine-grained soil, such as clay, cement contents ranging from 13 to 16 percent can be required because of the comparatively greater number of soil particles that would require cement (36).

Table 2.10 gives a comprehensive guide to the selection of admixtures for specific soil types. Based upon Figure 2.3 and Table 2.10 and the previous discussion in this report, the recommended stabilization methods of Table 2.11 are given for marginal aggregates found in the Oregon coastal area. The aggregates listed here are described in greater detail in Chapter 3.

These upgrading methods require further analysis to determine mix properties of fatigue resistance, resilient modulus, and split tensile strengths. The remainder of this report discusses the use of emulsions to stabilize quality basalts, marine basalts, and sands. A companion report discusses the use of cement to stabilize marine basalt, sandstone and dredged spoils (5).

Table 2.10. Comparison of Materials and Stabilization Methods (After 29,37).

Material	Untreated	Cement Treatment	Lime Treatment	Bitumuls & Bitumuls-Cement Treatment
Open-graded aggregate	Not suitable. lacks cohesion.	Not suitable.	Not suitable.	Provides tensile strength. Acts as a drainage layer. Significant thickness reduction possible. Used as a surface.
Processed dense-graded aggregate	Suitable. Thicker pavement structure needed than when treated.	Suitable. Some thickness reduction possible*.		Can be constructed in any thickness needed to support loads. Significant thickness reduction possible.
Natural gravel, slags, shells, reclaimed demolition waste, synthetic aggregate	Fines may be needed to prevent raveling.	Probably suitable; depends on grading*.	Not suitable, except if non-plastic. 2-4% binder needed.	Significant thickness reductions possible. Bitumuls-cement preferred where curing is a problem.
Clean sand	Coarse material may be needed for strength and fines to prevent raveling, improve workability.	Unsuitable; results in brittle mix.	Unsuitable; no reaction.	Significant thickness reductions possible.
Clayey sand, loam, silty sands	Coarse material needed for strength.	4-8 percent used.	May be suitable, depending on clay content**.	May be suitable. Some thickness reductions possible. Bitumuls-cement preferred. Careful field control needed.
Sandy clay	Not usually suitable.	4-12 percent needed.	4-8 percent, depending on clay content.	May be suitable for light traffic. Laboratory tests required.
Heavy clay	Unsuitable.	Unsuitable. Mixing may be assisted by pretreatment with 2% lime rather than 8-15 percent cement.	Most suitable. 4-8 percent, depending on clay content.	Not suitable.

* Requires minimum thickness of 4-6 inches of well-graded material.

** Lime may render the material non-plastic if clay content is low. Usually requires less than 4 percent lime.

Table 2.11. Summary of Recommended Stabilization Methods For Oregon's Coastal Aggregates.

Material	Stabilization Method
Marine Basalt	Asphalt emulsion Portland cement
Sandstone and Siltstone	Portland cement
Sands	Asphalt emulsions Portland cement Lime-pozzolan
Dredged Spoils	Portland cement Lime, lime-pozzolan

3.0 EXPERIMENT DESIGN

This chapter describes the experiment design for the asphalt emulsion testing program. A flow diagram is given on Figure 3.1. As seen from this diagram, the program was carried out in four main stages, these being

1) the selection and determination of aggregates and basic aggregate properties,
2) the asphalt emulsion mix design,
3) the dynamic test phase, and
4) the analysis of the dynamic test results.

Greater detail of this testing program is given in the following sections.

3.1 Materials

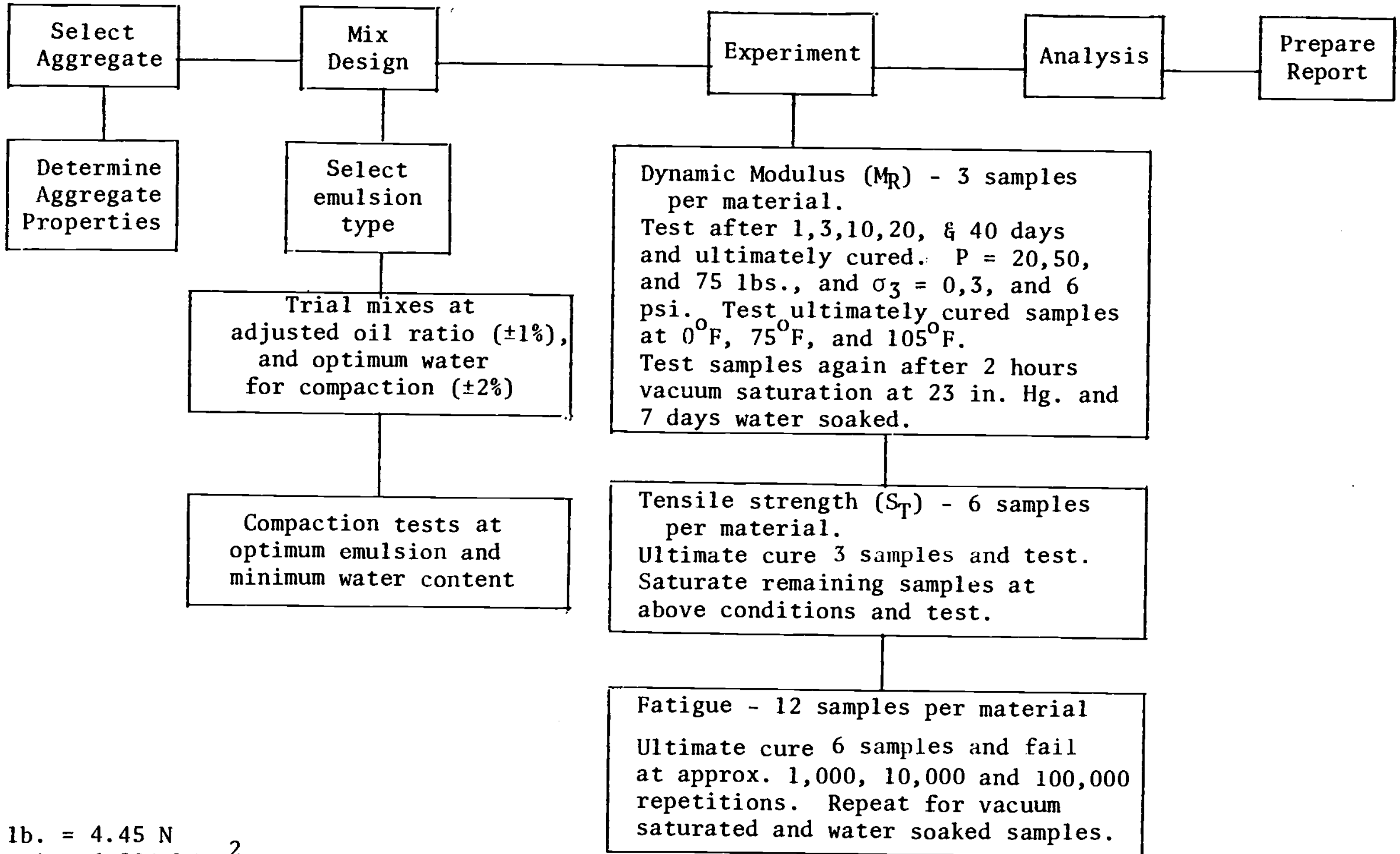
This section of the report discusses

1) the aggregates selected,
2) the tests performed,
3) summary of test results, and
4) a discussion of the test results.

3.1.1 Aggregates Selected

The major classifications of aggregates chosen for evaluation are basalts, sandstones, sands, and dredged spoils. These have been chosen because of their historically marginal nature and their relative abundance in Oregon's coastal area.

The basalts chosen for study were sampled from the Oceanlake Sand



1 lb. = 4.45 N
1 psi = 6.894 kN/m^2
1 in. = 25.4 mm

Figure 3.1. Asphalt Emulsion Test Program

and Gravel Quarry near Lincoln City, the Berry Creek (Ray Welles Construction) Quarry, near Florence, and the Eckman Creek Quarry, near Waldport. The Oceanlake material is a high quality aerial basalt which was chosen primarily for performance comparison with other treated materials. The Berry Creek and Eckman Creek aggregates are a marginal quality marine basalt, which have historically been used in construction with varying degrees of success.

The sandstone selected was sampled from the Big A Cut, adjacent to a Siuslaw National Forest Service road near Alsea. This material is actually an interbedded sandstone and siltstone of the Tyee formation. A limited amount of testing on this material has previously been performed by Huddleston (6), and is currently being evaluated by Chang (5).

The sands selected for study are a dune sand and a coarser graded beach sand. The dune sand was obtained from Vern Stocker, in South Beach (south of Newport). The beach sand was obtained from an old stockpile of material, previously taken from Gleneden Beach, which is south of Lincoln City. State regulations currently prohibit the excavation of this material for construction purposes.

The dredged spoils were sampled from a stockpile owned by the Al Pierce Lumber Co. of Coos Bay, near the Coos Bay-North Bend bridge on U.S. Highway 101. The material was dredged from the Coos River as part of a Corps of Engineers project, and consists primarily of a clean, finely graded sand with intermixed shell fragments. The locations of these materials are shown on Figure 3.2.

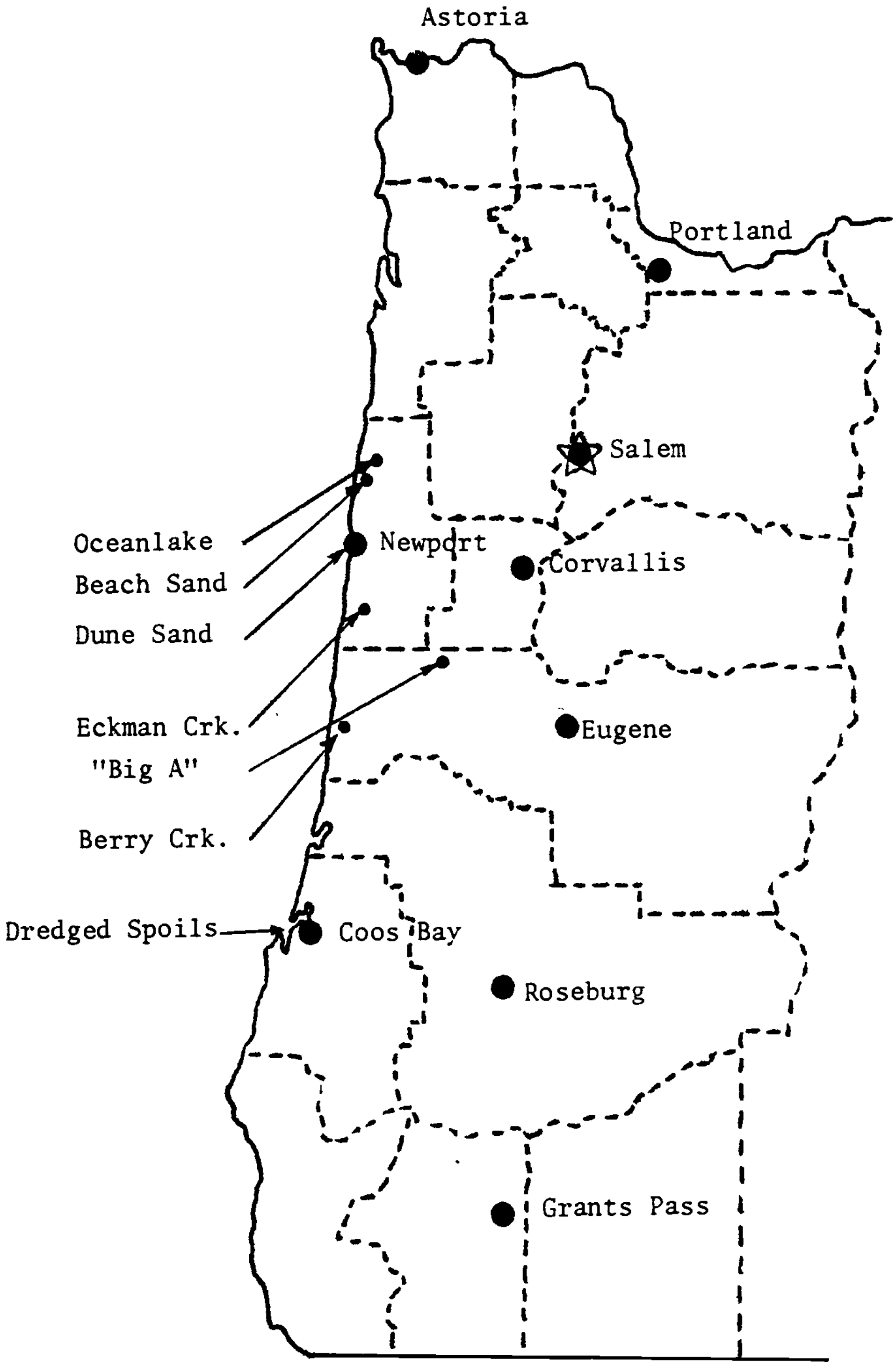


Figure 3.2. Location of Aggregates Tested

3.1.2 Aggregate Properties

The tests performed on the aggregates are listed in Table 3.1 along with the appropriate test designation. The tests are primarily concerned with the identification of basic physical and durability properties of the aggregates, and with determining appropriate admixture contents.

The Ten Rock test was also performed on Berry Creek and Eckman Creek aggregates treated with different admixtures. CMS-2 treatment consisted of thickly coating the specimens with emulsion, air curing for 24 hours, and oven curing for 24 hours at 60°C (140°F). For the sulfur treatment, the sulfur was heated to the liquid phase before coating. A thick slurry was also made from portland cement and hydrated lime materials in order to coat the aggregates.

3.1.3 Results

The open gradation, given in Figure 3.3, was used to determine the moisture-density properties of the basalt aggregates. This was done to compare with results of the same aggregates used in open graded asphalt emulsion mixes. The sandstone was prepared in the dense gradation given in Figure 3.3. This gradation for this material has previously been proven by Huddleston to provide optimum stabilized characteristics when treated with portland cement. A dense graded sandstone emulsion mix has also been successfully used on a FHWA test road (25). Typical U.S. Forest Service, Region 6, aggregate gradation specifications for emulsion treatment are given in Tables 3.2 and 3.3.

The gradations for the dune sand, beach sand, and dredged spoils were determined "as received" and are given in Figures 3.4, 3.5, and 3.6.

Table 3.1. Aggregate Tests.

Test	Designation
Gradation	OSHD 204-71 AASHTO T 27-74
Specific Gravity and Absorption	OSHD 202-71 (fines) AASHTO T 84-74 (fines) OSHD 203-71 (coarse) AASHTO T 85-74 (coarse)
Moisture-Density	AASHTO T 134-74
Sand Equivalent	OSHD 101-71 AASHTO T 176-70
Oregon Air Degradation	OSHD 208-77
Washington Durability	WSHD 113A
California Durability	Calif. 229-E
Los Angeles Abrasion	OSHD 211-72 AASHTO T 96-74
Centrifuge Kerosene Equivalent	Calif. 303-F
Ten Rock Test (DMSO)	FHWA Region 10 Test Method

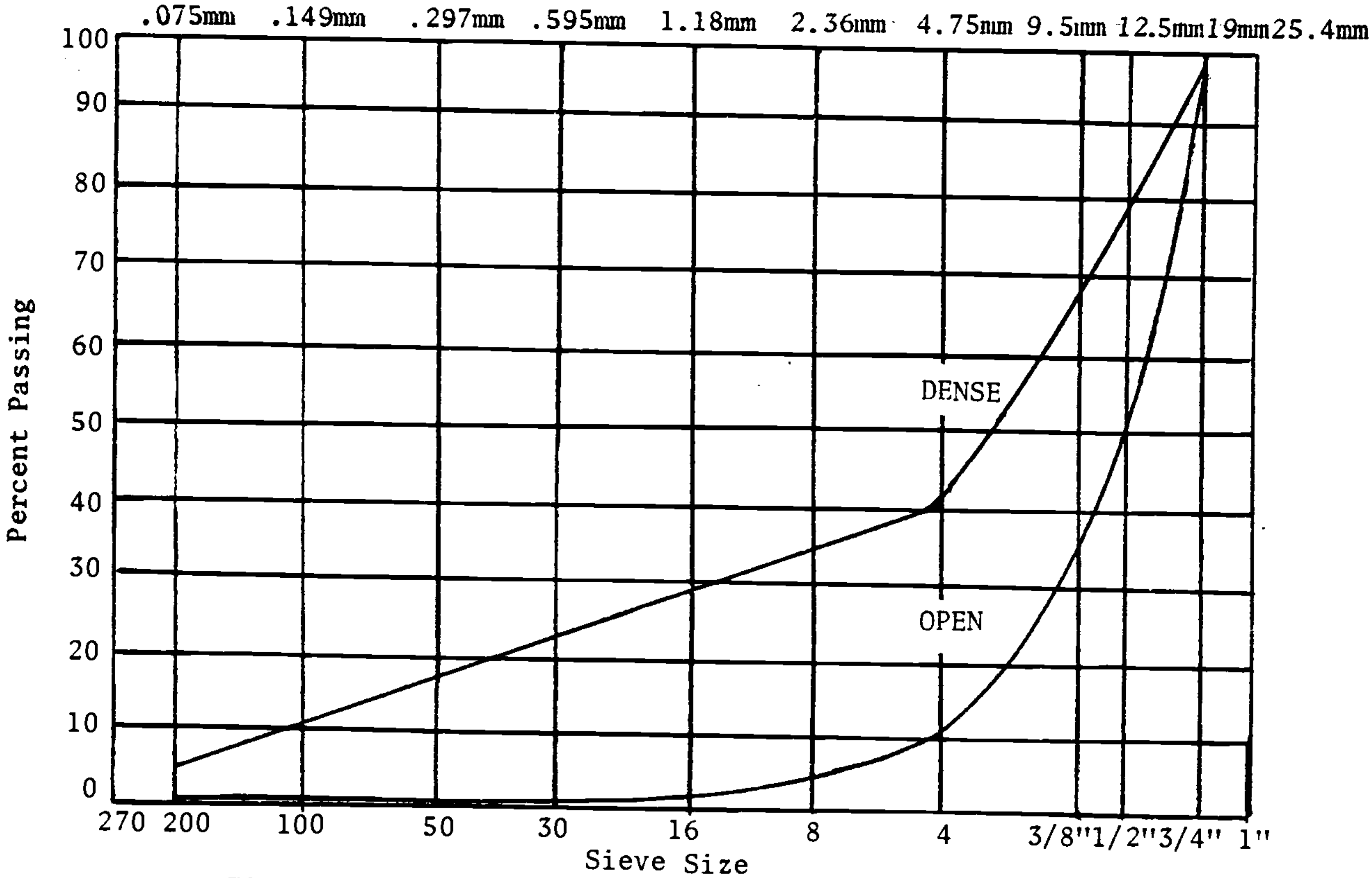


Figure 3.3. Aggregate Gradations for Basalt and Sandstone

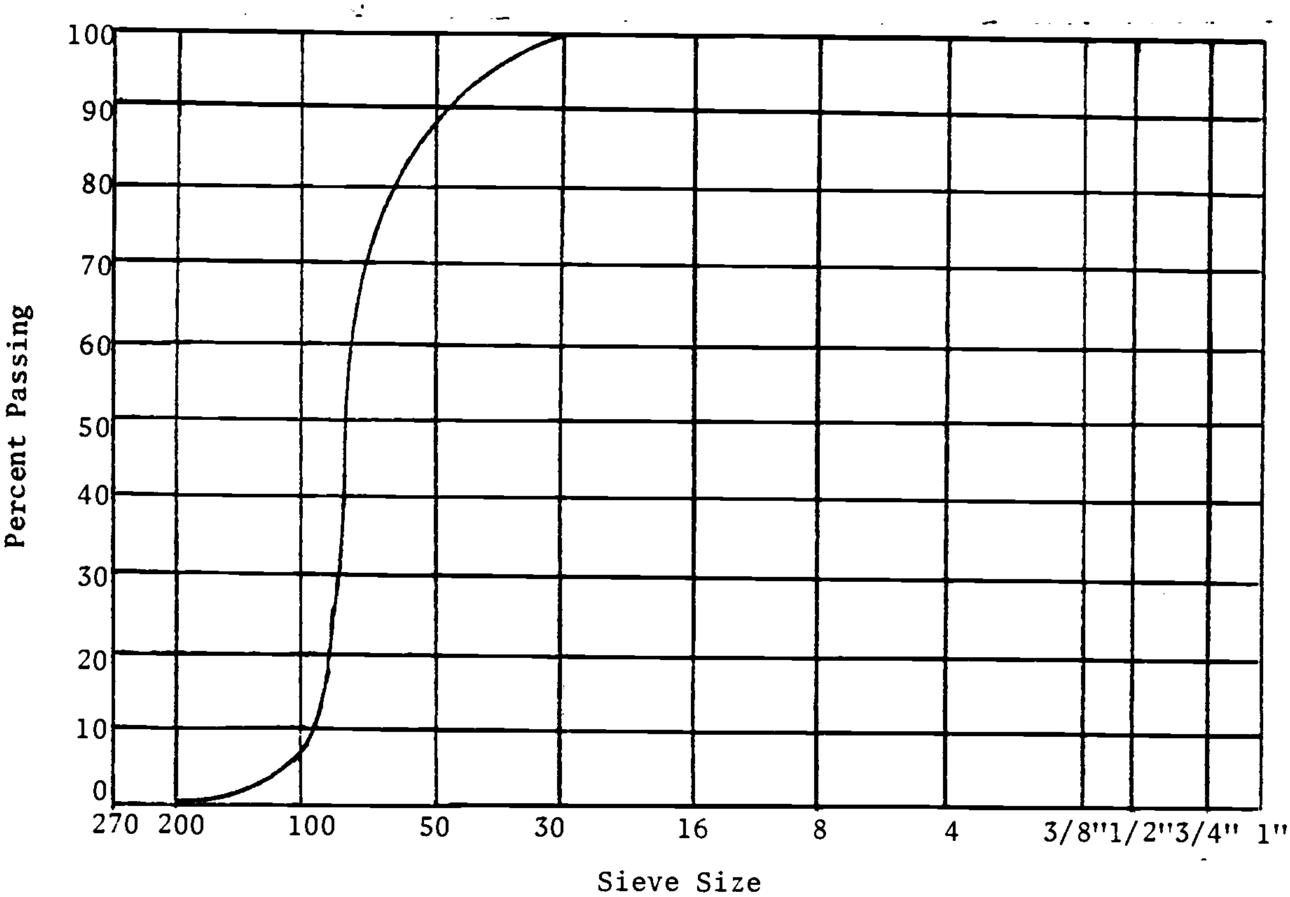


Figure 3.4. Dune Sand Gradation

Table 3.2. Typical Aggregate Specifications For Open Graded Asphalt Emulsion Mixes, USFS

Sieve Size	% Passing
19.0 mm (3/4")	100
12.5 mm (1/2")	45-70
4.75 mm (No. 4)	0-20
2.0 mm (No. 10)	0-6
.075 mm (No. 200)	0-2

Table 3.3. Typical Aggregate Specifications For Dense Graded Asphalt Emulsion Mixes, USFS

Sieve Size	% Passing
19.0 mm (3/4")	100
4.75 mm (No. 4)	51-63
2.0 mm (No. 10)	30-44
.075 mm (No. 200)	2-10

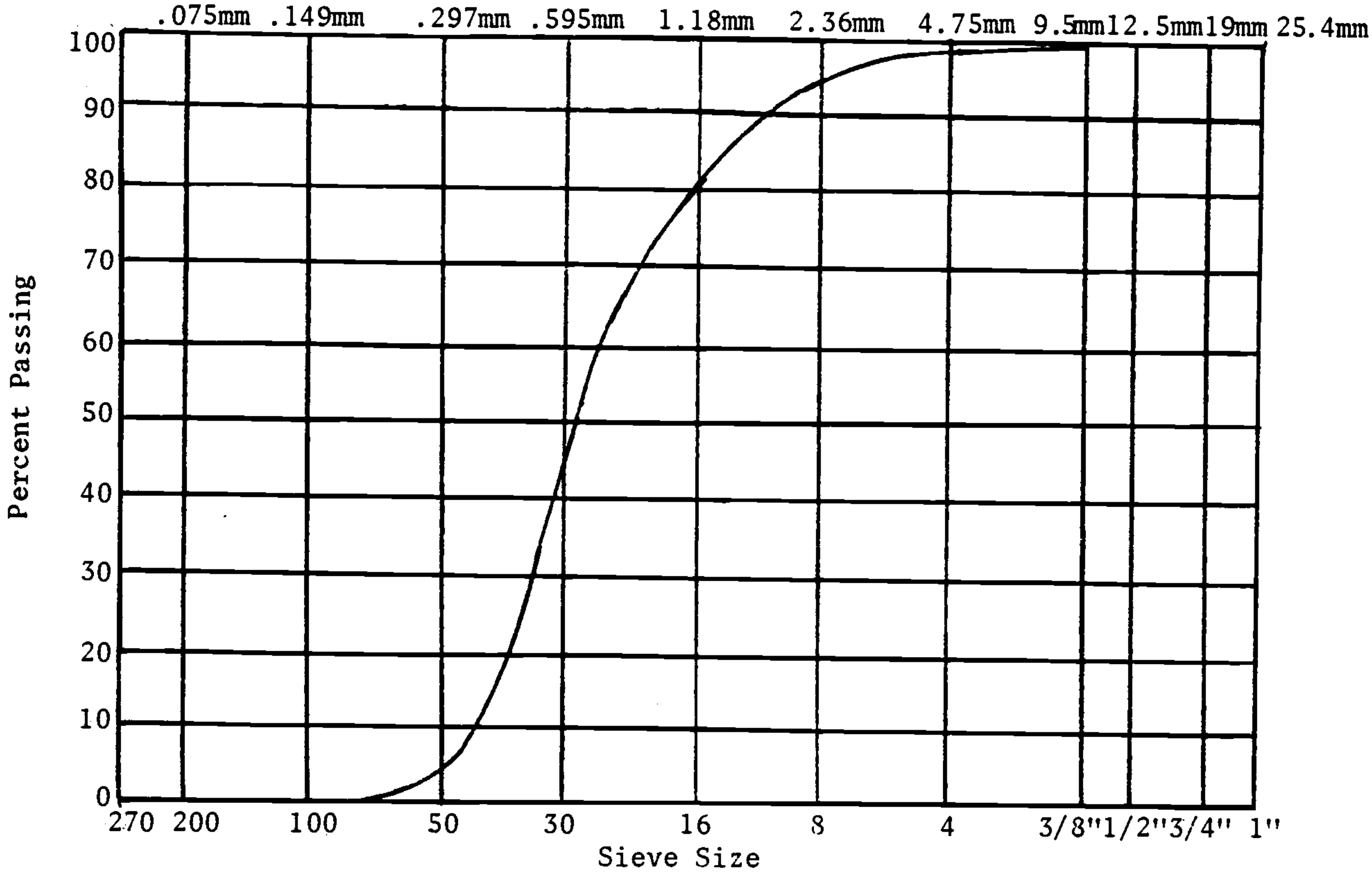


Figure 3.5. Beach Sand Gradation

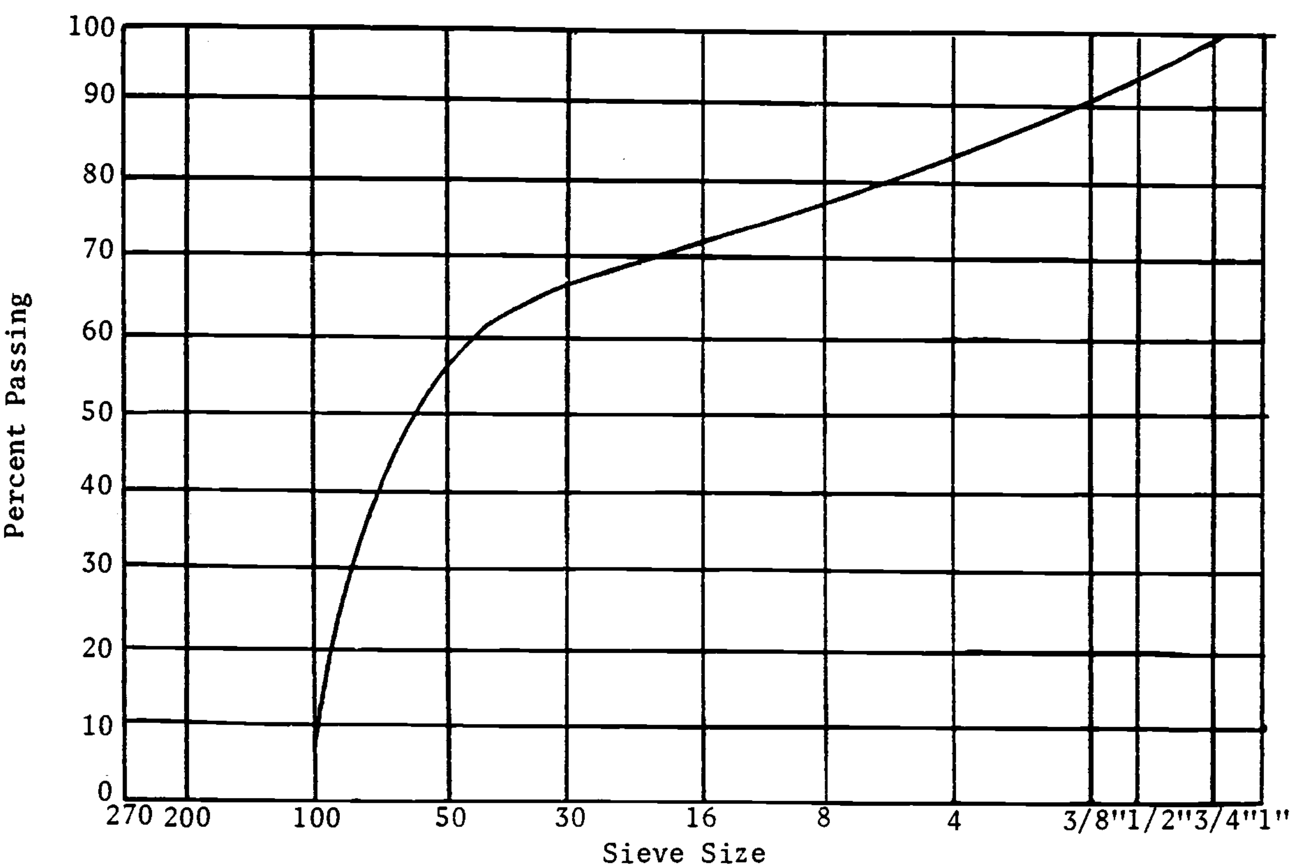


Figure 3.6. Dredged Spoils Gradation

Table 3.4 lists the aggregate properties for the study materials, as determined according to the test methods listed in Table 3.1. Because of the aggregate particle size of the dune sand, beach sand, and dredged spoils, certain tests were not able to be performed. These are indicated by a dash where the property value would normally be. Typical specifications are given at the bottom of the table.

3.1.4 Discussion of Results

As seen from Table 3.4, the Oceanlake basalt is the only aggregate which meets all of the specifications given. The sands and dredged spoils meet all of the specifications for which they could be tested, except for gradation. The Berry Creek basalt failed to pass the Sand Equivalent and Washington Durability, and is borderline for the California Durability (fines) and the Ten Rock test. The Eckman Creek basalt failed the Oregon Air Degradation, Washington Durability, California Durability, and Ten Rock tests. The Big A sandstone failed all of the tests except for the Sand Equivalent and California Durability of coarse particles.

Although only one set of samples were examined in the Ten Rock test of the treated materials, the results indicate that the performance of the Eckman Creek and Berry Creek aggregates can be slightly improved through treatment with admixtures.

3.2 Emulsion Mix Design

The purpose of this analysis was to observe the effect of fluid content (water and emulsion) on workability of a mix, the time to initial break, the thickness and percent of coating of asphalt on the aggre-

Table 3.4. Aggregate Properties.

Aggregate	S.G. (SSD)	Absorption (%)	Moisture-γ: γ_d (pcf)	Moisture-γ: w/c (%)	SE	OAD: D_{20}	OAD: H (in)	Wash. Dur.	Cal. Dur. Dc/Df	LA Abrasion (%)	% Oil Retained	CKE (adj)	Ten Rock: Std.	Ten Rock: CMS-2	Ten Rock: Sulfur	Ten Rock: portland cement	Ten Rock: Hydrated Lime
BASALT																	
Oceanlake	2.88	1.0	122*	1.5	66	16	1.4	66	81/43	13	2.6	-	0	0	-	-	-
Berry Creek	2.64	3.0	108*	4.7	27	23	2.9	28	68/35	27	3.0	-	4	0	2	2	3
Eckman Creek	2.68	5.0	116*	6.5	37	37	10.5	16	26/44	34	3.4	-	10	10	10	8	10
SANDSTONE																	
Big A	2.39	10.0	121**	9.0	36	92	4.6	36	33/19	95	6.0	7.8	-	-	-	-	-
SAND																	
Stocker Dune Sand	2.76	.6	108	12.5	87	-	-	-	-/96	-	-	2.84	-	-	-	-	-
Gleneden Beach	2.66	.6	108	4.2	100	12.2	.1	-	-/99	-	-	1.67	-	-	-	-	-
DREDGED SPOILS																	
Coos Bay	2.25	1.4	111	9.5	86	19.8	2.0	-	-/86	-	-	2.75	-	-	-	-	-
Typical Specifications					35 min.	35 max.	3.5 max.	50 min.	25/35 min.	35 max.			4 max.				

* Open Gradation
** Dense Gradation
\- Was not tested
1 pcf = 16.02 kgs/m^3
1 in. = 25.4 cm

gates, and compacted mix densities in order to optimize the mix proportions used in dynamic testing and strength evaluation.

The physical properties of the study aggregates have previously been discussed. The aggregates, aggregate gradations, and emulsion types selected for further testing are listed below:

Aggregate Type	Aggregate	Gradation	Emulsion Type
Basalt	Oceanlake	Open[1]	CMS-2
Marine Basalt	Berry Creek Eckman Creek	Open Open	CMS-2 CMS-2
Sandstone	Big A	Dense[1]	CMS-2s
Sand	Stocker Dune Sand	As Received[2]	CSS-1 CSS-1 w/ 1.5% portland cement

[1]Figure 3.3
[2]Figure 3.4

These aggregates have been selected on the basis of meeting suitability requirements for treatment with asphalt emulsion, accessibility of aggregate resources, and previous success in other regions by treating these aggregate types. The properties of the emulsions used fall within the specifications given in Table 3.5.

3.2.1 Mix Design Procedure

The procedure used for all of the study aggregates to determine the optimum emulsion and water contents was developed by the U.S. Forest Service, Region 6, for Open-Graded Emulsified Asphalt Pavements (R6-355(79)). This method is also quite similar to that used by the Federal Highway Administration, Region 10.

Table 3.5. Asphalt Emulsion Specifications (37).

Type	Medium-Setting		Slow-Setting	
Grade	CMS-2		CSS-1	
	Min.	Max.	Min.	Max.
Tests on Emulsions:				
Viscosity SSF @ 77°F sec.			20	100
Viscosity SSF @ 122°F sec.	50	450		
Settlement, 5 days, %[a]		5		5
Storage stability test, 1 day[b]		1		1
Coating ability & water resistance:				
Coating, dry aggregate	Good			
Coating, after spraying	Fair			
Coating, wet aggregate	Fair			
Coating, after spraying	Fair			
Particle charge test	Positive		Positive[c]	
Sieve test, %		0.10		0.10
Cement mixing test, %				2.0
Distillation:				
Oil distillate by vol of emulsion, %		12		
Residue, %	65		57	
Tests on Residue from Distillation Test:				
Penetration, 77°F	100	250	100	250
Ductility, 77°F, 5 cm/min., cm	40		40	
Solubility in trichloroethylene, %	97.5		97.5	

(a) The test requirement for settlement may be waived when the emulsified asphalt is used in less than 5 days' time; or the purchaser may require that the settlement test be run from the time the sample is received until it is used, if the elapsed time is less than 5 days.
(b) The 24-hour (1 day) storage stability test may be used instead of the 5-day settlement test.
(c) Must meet a pH requirement of 6.7 maximum (ASTM E-70) if the particle charge test is inconclusive.
(d) 1°C = (°F-32)5/9.

3.2.2 Mix Design Results

To determine the approximate beginning emulsion content, the oil ratios for the study aggregates were determined. The results from the specific gravity and CKE test are given in Table 3.6. Only the percent oil retained was required for the open graded mixes. The dense graded mix requires both the percent oil retained and a centrifuge kerosene equivalent value. The sands require only a centrifuge kerosene equivalent value to be obtained. The oil ratios found to be required for the study aggregates are given in Table 3.6. The method used to obtain the oil ratio is California Test Method 303-F.

Trial 500 gram mixtures of the aggregates with various amounts of emulsion and water were then prepared for a visual examination of mix characteristics. The characteristics evaluated included 1) workability of the mix, 2) thickness of residual asphalt on the aggregate particles, 3) percentage of aggregate in the mix coated with asphalt, and 4) presence of excess fluids after mixing. The workability of the mix was recorded as good, fair, or poor after mixing in the emulsion for 30-45 seconds. Presence of excess fluids was also noted at this time. The film thickness was evaluated after the mix had completely broken (turned black), and was estimated as thin, medium, or thick. Percentage of aggregate coated was also evaluated after the mix had broken. The results of this testing are given in Appendix A. The optimum proportions were selected according to the following specifications for a range of one percent moisture:

Workability	fair to good
Film thickness	moderate to thick
Coating	90-100%
Excess fluids	little to none

Table 3.6. Oil Ratio Required for Study Aggregates.

Gradation	Aggregate	K_c	K_f	K_m	Oil Ratio
Open	Oceanlake	1.15	-	-	5.73
	Berry Creek	1.32	-	-	5.98
	Eckman Creek	1.50	-	-	6.25
Dense	Big A	2.50	1.63	1.93	8.75
As	Dune Sand	-	.90	-	5.20
Received	Beach Sand	-	1.20	-	5.0
	Dredged Spoils	-	1.01	-	5.50

Open Graded Oil Ratio = $1.5\ K_c + 4.0$

Dense Graded Oil Ratio, K_c, K_f, and K_m determined from aggregate properties and California Test Method 303-F.

K_c, K_f, K_m = Coefficients determined from CKE and % oil retained tests.

Oil Ratio = Approximate required emulsion content.

As this mix design was carried out for the purpose of comparing different aggregate mixtures under controlled environmental conditions, it was not necessary to determine the minimum temperature for 90% coating, as is required in the Forest Service procedure for specific asphalt emulsion mixes in certain applications.

The dry density of the open graded mixes was then determined by the following procedure: 1) a 1200 gram aggregate sample is mixed with the optimum emulsion content and minimum water content; 2) the amount of mix required to form a compacted specimen approximately 102 mm (4 inches) diameter by 63 mm (2.5 inches) high is added to a secured mold, which is placed in the kneading compactor shown in Figure 3.7; 3) approximately 20 tamping blows are applied at 1725 kN/m^2 (250 psi) to achieve preliminary compaction; 4) the mold is then loosened and 150 tamping blows at 3450 kN/m^2 (500 psi) are applied; 5) the mold is then carefully removed and a 56 kN (12,600 lb) static leveling load is applied to the specimen, using the double plunger method; 6) the height and weight of the specimen are measured; and 7) the specimens are then oven-dried at 110° ± 5°C (230° ± 9°F) for 24 hours and the dry density is determined.

Due to the instability of the sandstone and dune sand mixes, the kneading compactor could not be used for compaction. For these materials, a 178 kN (40,000 lb) static load was applied using the double plunger method to achieve compaction. This was followed by the same cure period and dry density determination as required for the open graded mixtures.

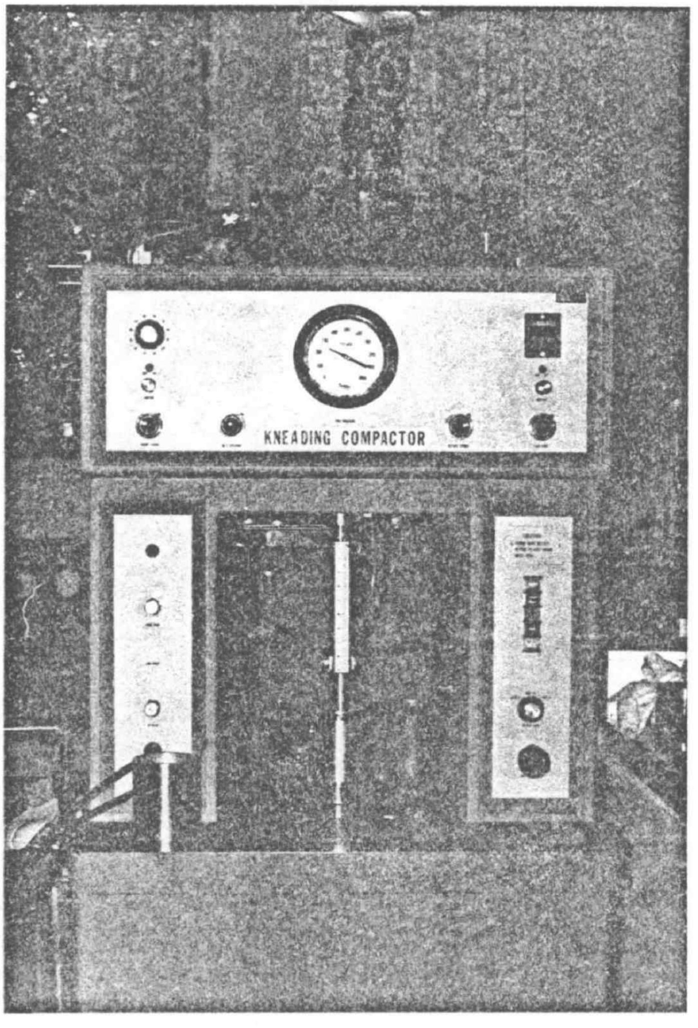


Figure 3.7. Kneading Compactor

3.2.3 Discussion of Results

From the results of this test program, the optimum emulsion contents listed in Table 3.7 are recommended for use in future testing of the study aggregates. Because of the high fluid contents required for the Big A sandstone, and the excessive degradation characteristics of this material when wet, it was not tested in the dynamic test program. Also, the dune sand was tested only with the portland cement, as less fluids are required and faster specimen curing is obtained.

3.3 Dynamic Test Program

To simulate the performance of the emulsion treated aggregates in an actual pavement section, a series of dynamic tests are required to characterize the materials behavior. To evaluate the structural performance of the aggregate mixtures according to layered elastic theory, and to study durability characteristics of the mixes, the following dynamic test program was employed.

3.3.1 Selection of Tests

The properties determined for the asphalt emulsion mixtures are the dynamic modulus, fatigue characteristics, tensile strength, temperature susceptibility, and durability characteristics. The Oregon State University diametral testing apparatus shown in Figure 3.8 was used to determine modulus and fatigue characteristics of all test samples. Tensile strength properties were determined using the Mechanical Test System (MTS) shown in Figure 3.9. Durability characteristics were determined by means of vacuum saturation tests to find the effect of moisture on modulus, tensile strength, and fatigue strength. The tests used to

Table 3.7. Optimum Emulsion Contents.

Aggregate	Emulsion Type	Emulsion Content, %	Water Content, %	Dry Density, pcf (typical)
Oceanlake	CMS-2	5.0	0-1	123
Berry Creek	CMS-2	6.0	2-4	123
Eckman Creek	CMS-2	6.0	3-4	130
Big A Sandstone	CMS-2s	12.0	12-14	114
Dune Sand	CSS-1	8.0	9-12	104
Dune Sand	CSS-1 + 1.5% portland cement	7.0	9	116

1 pcf = 16.02 kgs./m^3

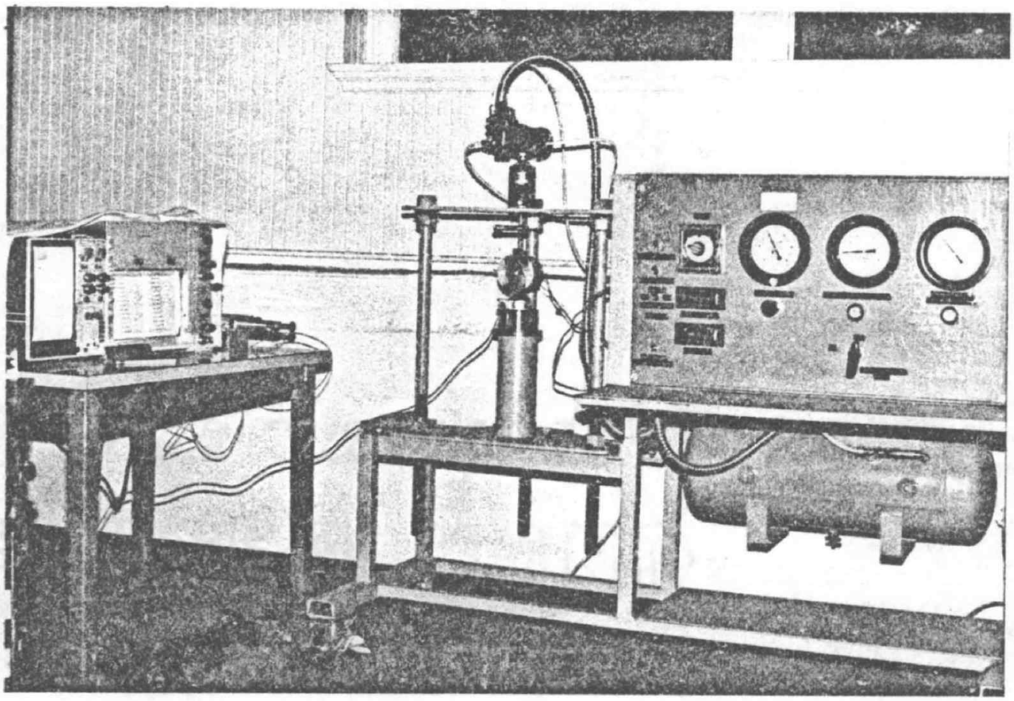


Figure 3.8. OSU Diametral Test Apparatus

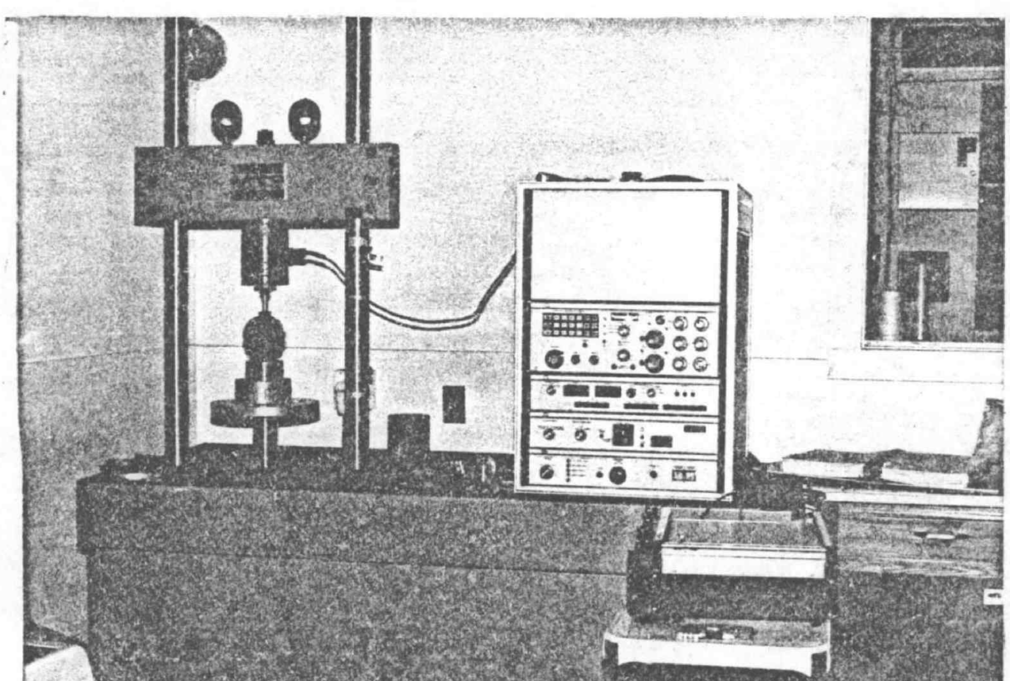


Figure 3.9. Split Tension Test on Mechanical Test System (MTS)

evaluate the marginal aggregate mixes are summarized in Table 3.8.

3.3.2 Dynamic Test Procedures

Modulus. For the diametral modulus, the samples were prepared using a kneading compactor according to the method prescribed in the mix design for density testing. Samples are 102 mm (4 inch) diameter by 63 ± 8 mm (2.5 ± .3 inches) thick. For each aggregate type, three samples were tested after time intervals of about 1, 3, 20, and 40 days of curing at 24 ± 3°C (75° ± 5°F). The general equation for the resilient modulus from the diametral test is as follows (39):

$$M_R = \frac{P(\nu + .2732)}{t(\Delta h)}$$

where M_R = resilient modulus, psi

P = dynamic portion of the load, lbs.

t = thickness of specimen, inches

Δh = the horizontal elastic deformation inches (on the OSU testing system, this is the sum of the two output channel pen deflections)

ν = Poisson's ratio (assumed to be 0.35 for asphalt emulsion mixes).

The resilient modulus of each material type was determined over a range of confining stresses of from 0 to 41 kN/m^2 (0 to 6 psi) and a dynamic load range from 89 to 334 N (20 to 75 lbs), using the test procedure given in Appendix C. Displacement transducers attached to a yoke and the confining stress apparatus used for this testing are shown in

Table 3.8. Tests for Asphalt Emulsion Stabilized Mixes.

Test	Property Evaluated
(1) Diametral	(1) Modulus
	(2) Fatigue
	(3) Temperature Susceptibility
(2) Static Loading	(1) Split Tensile Strength
(3) Durability (After Vacuum Saturation)	(1) Modulus
	(2) Tensile Strength
	(3) Fatigue

Figure 3.10. As shown in Figure 3.11, a dynamic load is applied to the samples, from which measurements are recorded from the displacement transducers and from a load cell located underneath the sample being tested. The duration of the deviator stress pulse was 0.10 second, which corresponds with about 48 kms/hr (30 mph) actual conditions for a tire load, and was applied at a frequency of 30 load cycles per minute.

Fatigue. For fatigue testing, after measurement of the initial elastic strain obtained from applying a constant dynamic load, six samples of each aggregate type were failed at approximately 1000, 10,000, or 100,000 load repetitions. All samples were air cured for 4 days at 24 ± 3°C (75 ± 5°F) and placed in the desiccator shown in Figure 3.12 at 79.4 kg/m^2 (23 in Hg.) vacuum until all of the moisture was removed to obtain the ultimate cured condition. Loads were applied at 60 cycles per minute and 0.10 second duration until failure, using the diametral setup shown in Figure 3.13. Failure of the sample was determined by an approximately 12 mm (1/2 inch) wide vertical crack across the diameter of the sample as shown in Figure 3.14. This vertical crack breaks an electrical circuit going through the metallic tape shown in Figure 3.14, stopping the test at the exact number of failure load applications.

Split Tensile Strength. For this test, the samples were cured according to the requirements for the previously mentioned fatigue testing, at 24 ± 3°C (75° ± 5°F). A static load was applied to each sample at a rate of 51 mm (2 inches) per minute in the manner shown in Figure 3.15. Continuous measurements of horizontal and vertical deformation were recorded on an X-Y recorder connected to the Mechanical Test System (MTS) shown in Figure 3.9, from which the tensile strength was

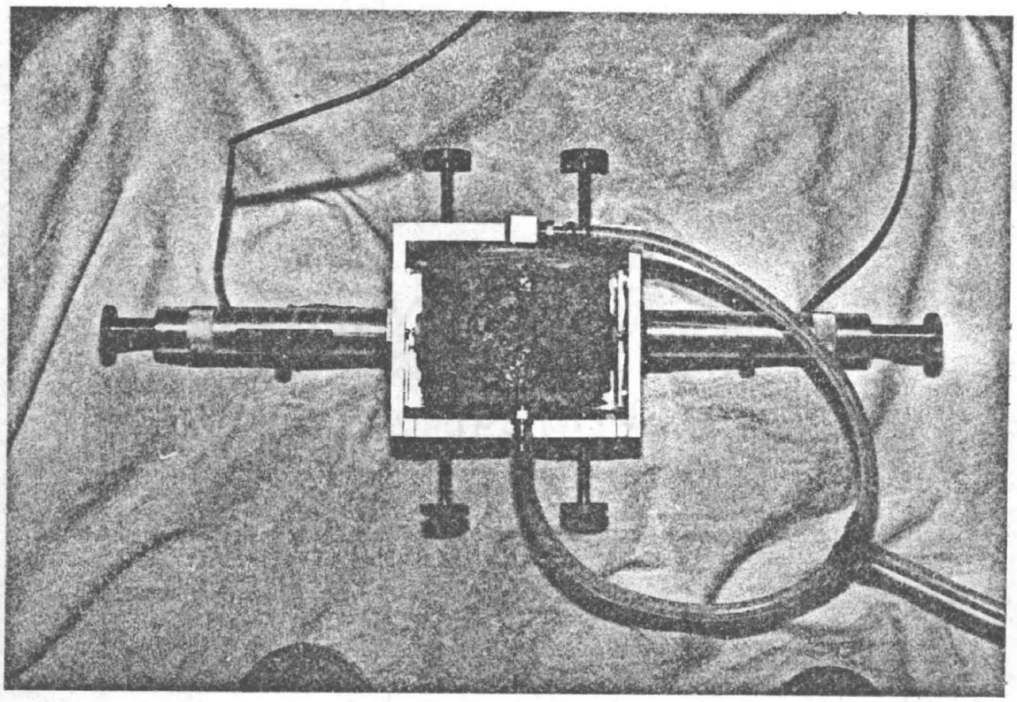


Figure 3.10. Displacement Transducers and Confining Stress Apparatus on Test Specimen

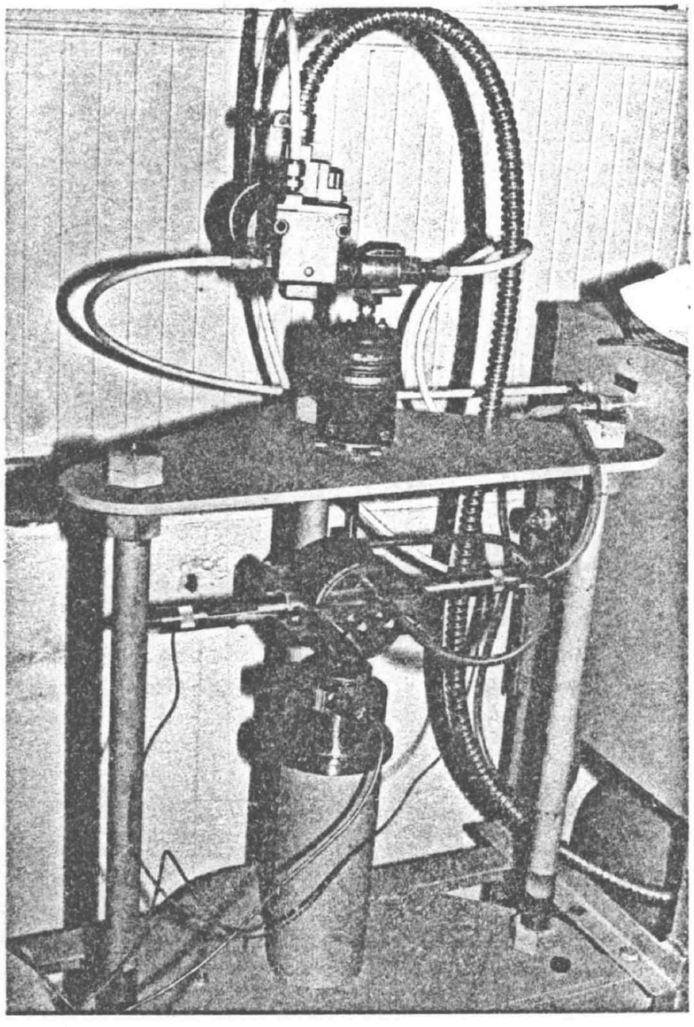


Figure 3.11. Resilient Modulus Test

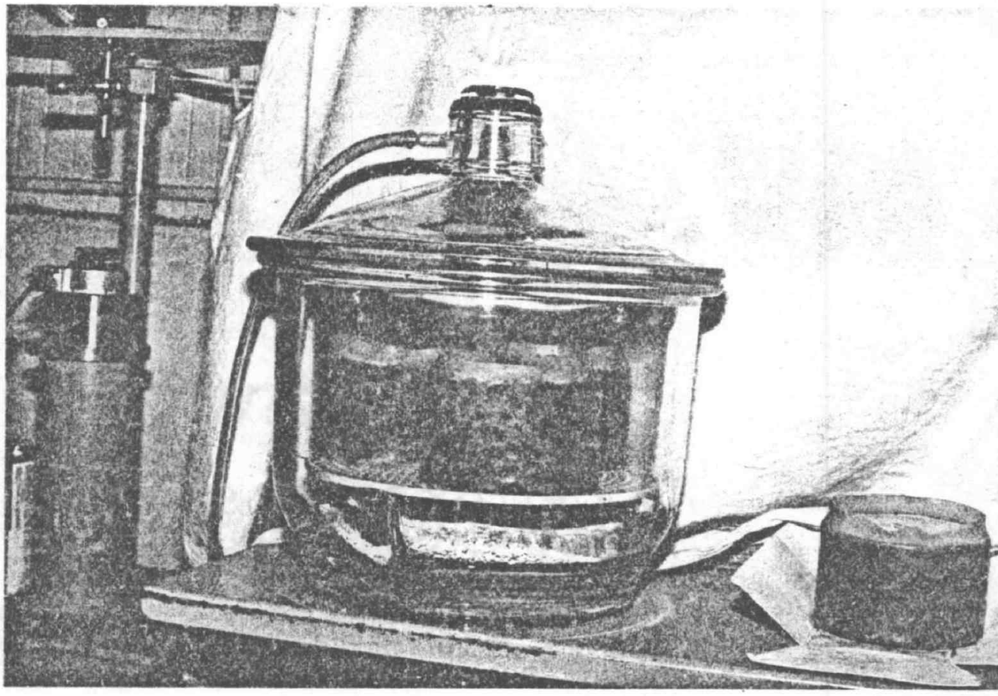


Figure 3.12. Vacuum Desiccator With Samples Being Cured

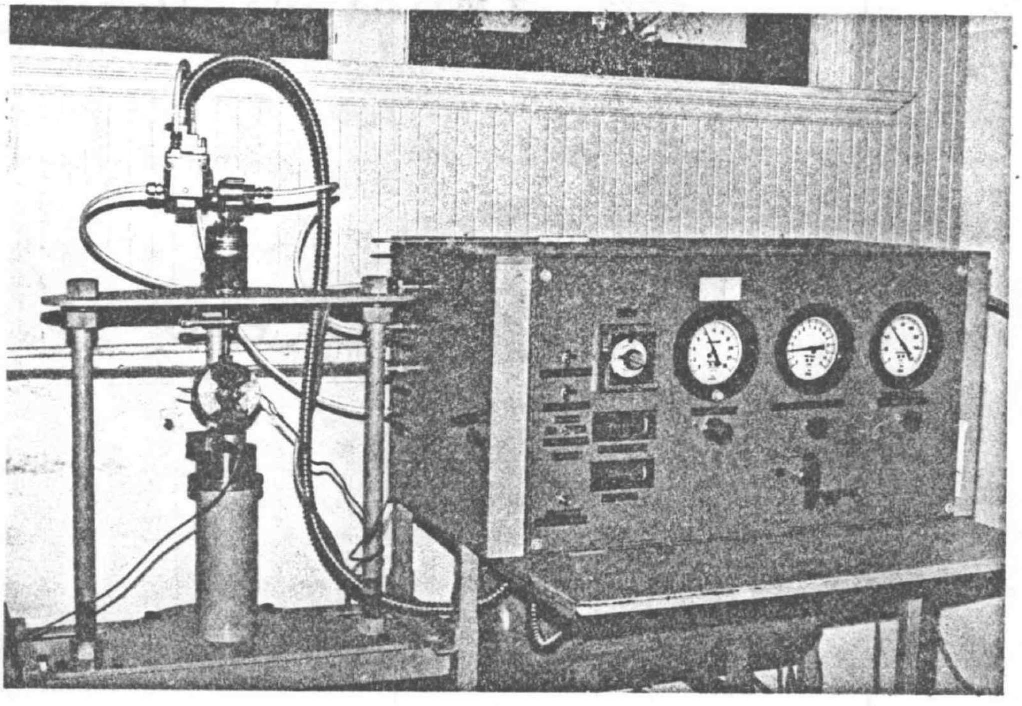


Figure 3.13. Diametral Setup for Fatigue Testing

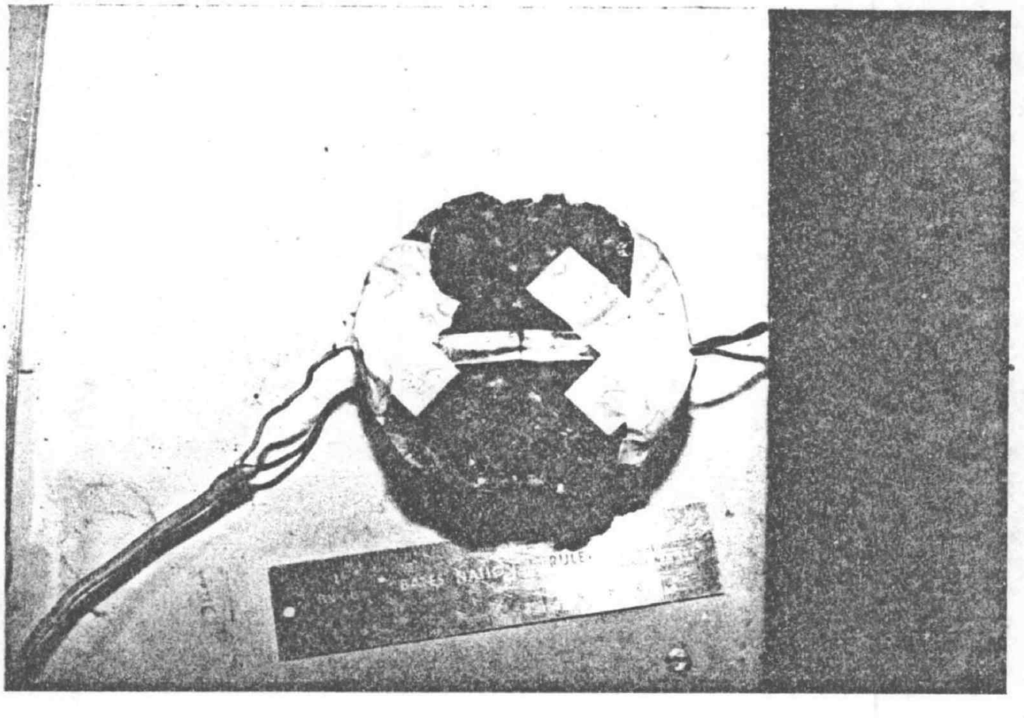


Figure 3.14. Sample Failed from Fatigue Testing

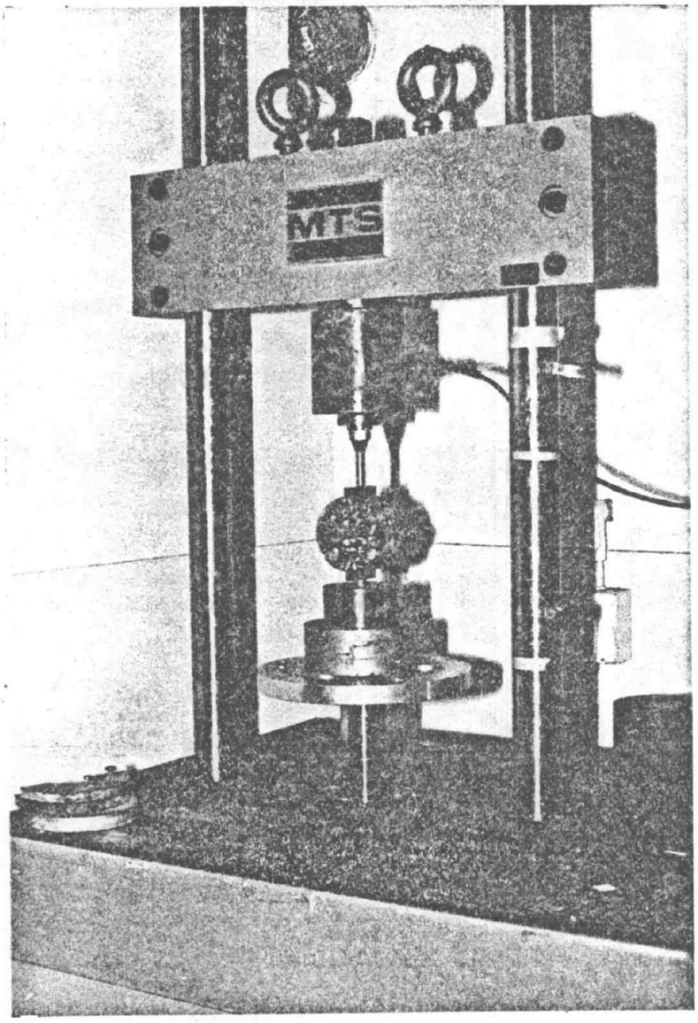


Figure 3.15. Sample Being Failed in Split
Tension Test

determined for ultimately cured samples and for samples conditioned by a 2-hour 79.4 kg/m^2 (23 inch Hg.) vacuum saturation with a 7-day soak period. This conditioning is similar to that used by Lottman (39) in considering short-term durability characteristics of asphalt concrete. The tensile strength (S_T) is determined from the equation (38):

$$S_T = \frac{P_{max}(k)}{ht.}$$

where S_T = Tensile Strength, psi,

P_{max} = maximum load, lbs.,

k = geometric constant, $2/\pi$ (diameter),

ht. = sample height, inches.

3.3.3 Dynamic Test Program Analysis

From the laboratory characterization of the study materials and use of the layered elastic theory, layer thicknesses can be determined to preclude fatigue and rutting for the treated marginal aggregates in pavement sections. The use of fatigue curves similar to Figure 3.16 and design thickness curves such as given in Figure 3.17 allows easy determination of layer equivalencies from defined layer thicknesses. The following steps are required:

(a) Determine design life of pavement,

(b) Determine critical strain level based on design life for both materials (ε_1, ε_2; Figure 3.16),

(c) Determine layer thicknesses for previously determined strain levels (h_1, h_2; Figure 3.17),

(d) Calculate layer equivalency as Eq. = $\frac{h_2}{h_1}$.

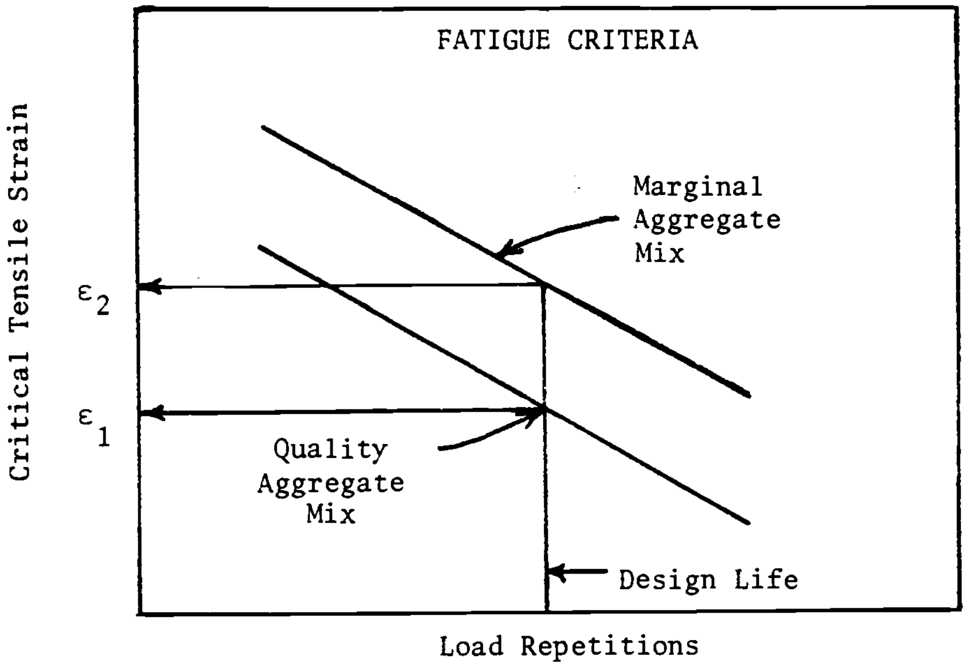


Figure 3.16. Typical Fatigue Test Results

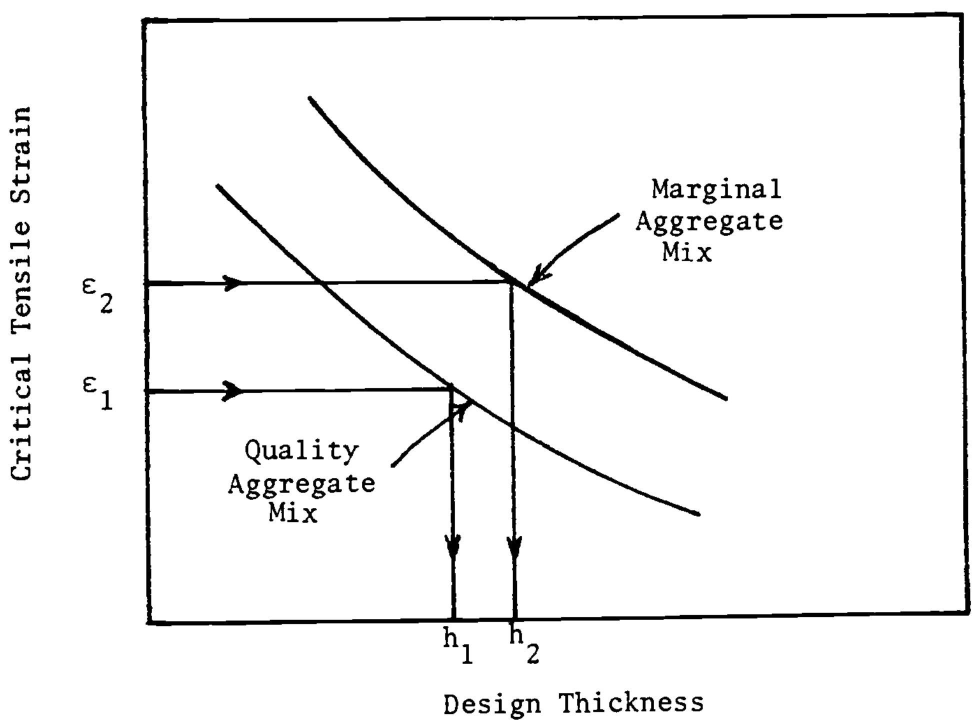


Figure 3.17. Typical Design Thickness Curve Based on Critical Strain Level

4.0 DYNAMIC TEST RESULTS

The purpose of this chapter is to report results obtained from tests on laboratory-prepared samples of emulsion-treated Berry Creek, Eckman Creek, Oceanlake, and Dune Sand materials. The test program is previously defined in Chapter 3. Included in this chapter is a summary of mix properties, resilient modulus values for various curing times, conditioning, temperatures, tensile strength, and fatigue properties for unconditioned and conditioned samples. Individual test results for each test specimen are given in Appendix B.

4.1 Mix Properties

Table 4.1 summarizes the properties of the specimens according to the aggregate type. The highest densities were obtained with the Eckman Creek aggregate, most likely attributable to excessive degradation characteristics. The degradation experienced by each of the open graded mixes after compaction is indicated by the increase in surface area shown in Table 4.2. This is also shown by comparing original and final gradations in Figures 4.1, 4.2, and 4.3. The dune sand specimens had the lowest density, along with the lowest specific gravity of the aggregates. As the required added water content varied slightly from the beginning to the end of specimen fabrication, an average value is used for comparison. This variation probably resulted from differences in absorbed moisture at the time of mixing.

4.2 Resilient Modulus

The resilient modulus of each of the study aggregate mixes was

Table 4.1 Mean Values and Coefficients of Variation for Density of Laboratory Specimens.

Aggregate	Emulsion Content, %	Mean Added Water %	Total Number of Specimens	Density Mean, pcf	Density Std. Dev.	C.V., %
Berry Creek	6.0	1.7	31	124.8	3.3	2.7
Eckman Creek	6.0	2.7	24	130.4	2.9	2.2
Oceanlake	5.0	1.1	36	125.9	2.0	1.6
Dune Sand	7.0	7.2	28	114.1	2.5	2.2

1 pcf = 16.02 $kgs./m^3$

Table 4.2. Surface Area Calculation Before and After Compaction.

Sieve Size	19.0 mm (3/4")	9.5 mm (3/8")	4.75 mm (No. 4)	2.36 mm (No. 8)	1.18 mm (No. 16)	.600 mm (No. 30)	.300 mm (No. 50)	.150 mm (No. 100)	.075 mm (No. 200)	Total Surface Area, ft^2/lb	Percent Change %
	STANDARD SURFACE AREA, OPEN GRADED (BEFORE COMPACTION)										
SA Factor	2	2	2	4	8	14	30	60	160		
Open Graded % Passing	1.00	.41	.10	.04	.03	.025	.02	.015	.01		
Open Graded Surface Area	2.0	.82	.20	.16	.24	.35	.60	.90	1.60	6.87	-
	STANDARD SURFACE AREA, OPEN GRADED (AFTER COMPACTION)										
Berry Creek % Passing	1.0	.59	.27	.15	.09	.07	.05	.03	.02		
Berry Creek Surface Area	2.0	1.18	.54	.60	.72	.98	1.5	1.8	3.20	12.52	82
Eckman Creek % Passing	1.0	.63	.40	.23	.16	.12	.08	.06	.04		
Eckman Creek Surface Area	2.0	1.26	.80	.92	1.28	1.68	2.40	3.60	6.40	20.34	196
Oceanlake % Passing	1.0	.49	.26	.11	.06	.05	.04	.03	.02		
Oceanlake Surface Area	2.0	.98	.52	.44	.48	.70	1.20	1.80	3.20	11.32	65

1 ft^2/lb = 4.88 m^2/kg

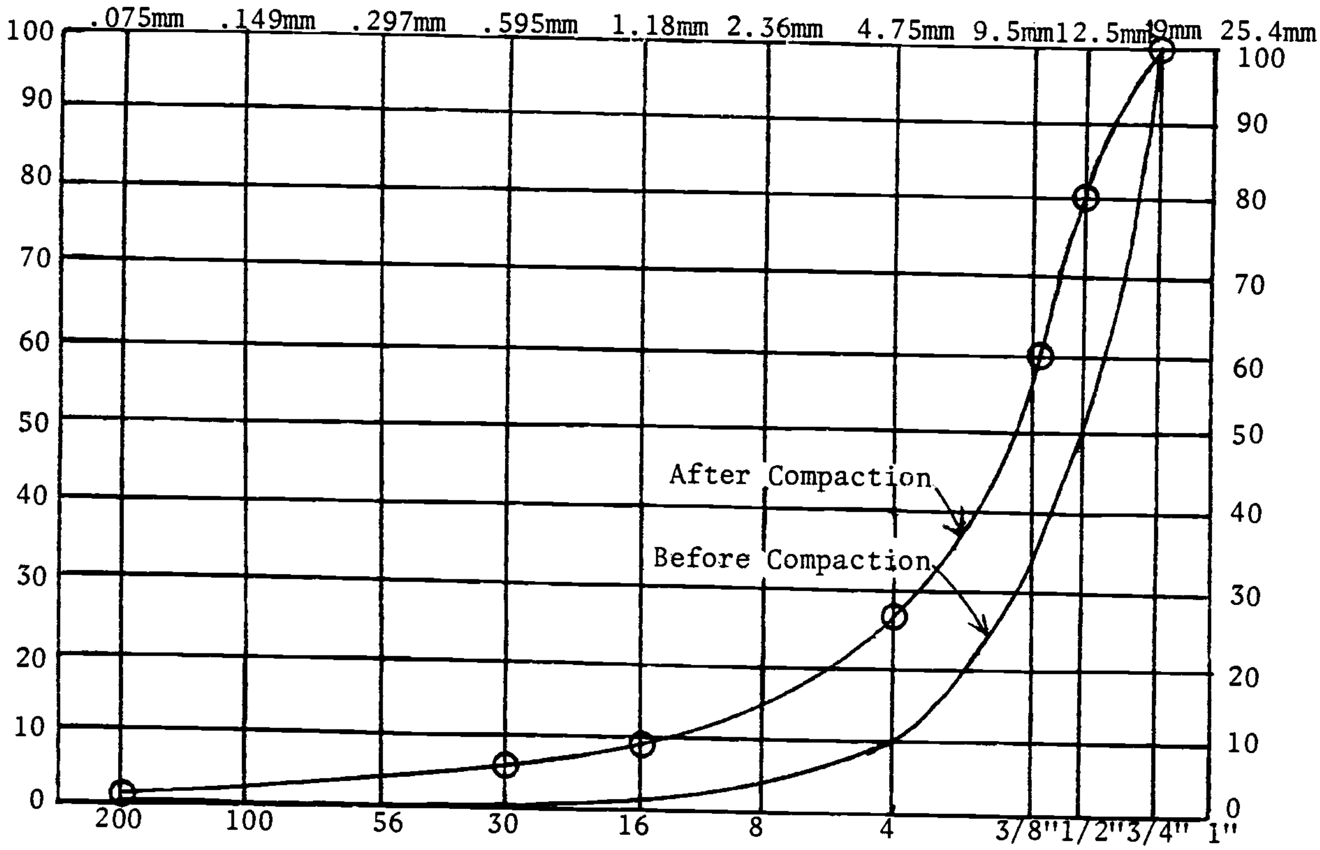


Figure 4.1. Effect of Compaction on Gradation, Berry Creek

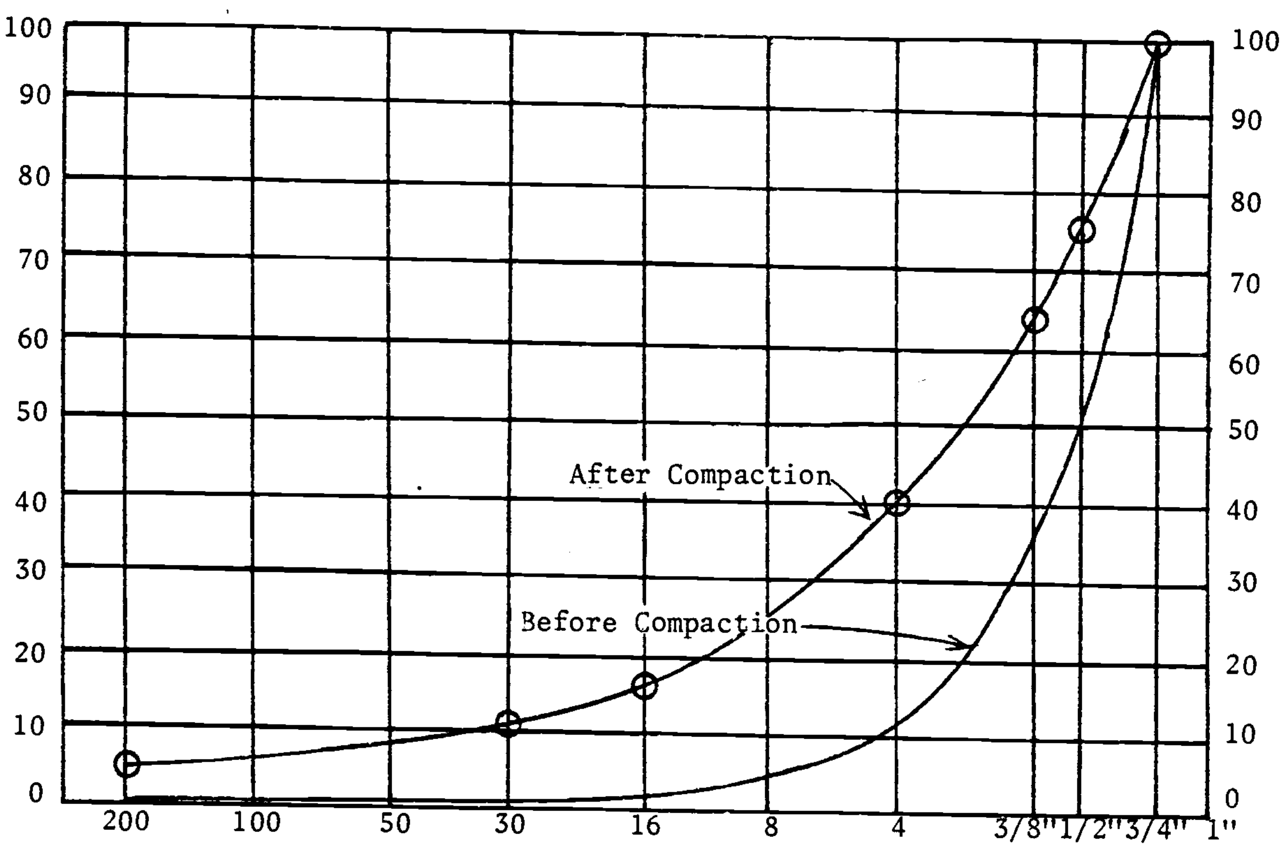


Figure 4.2. Effect of Compaction on Gradation, Eckman Creek

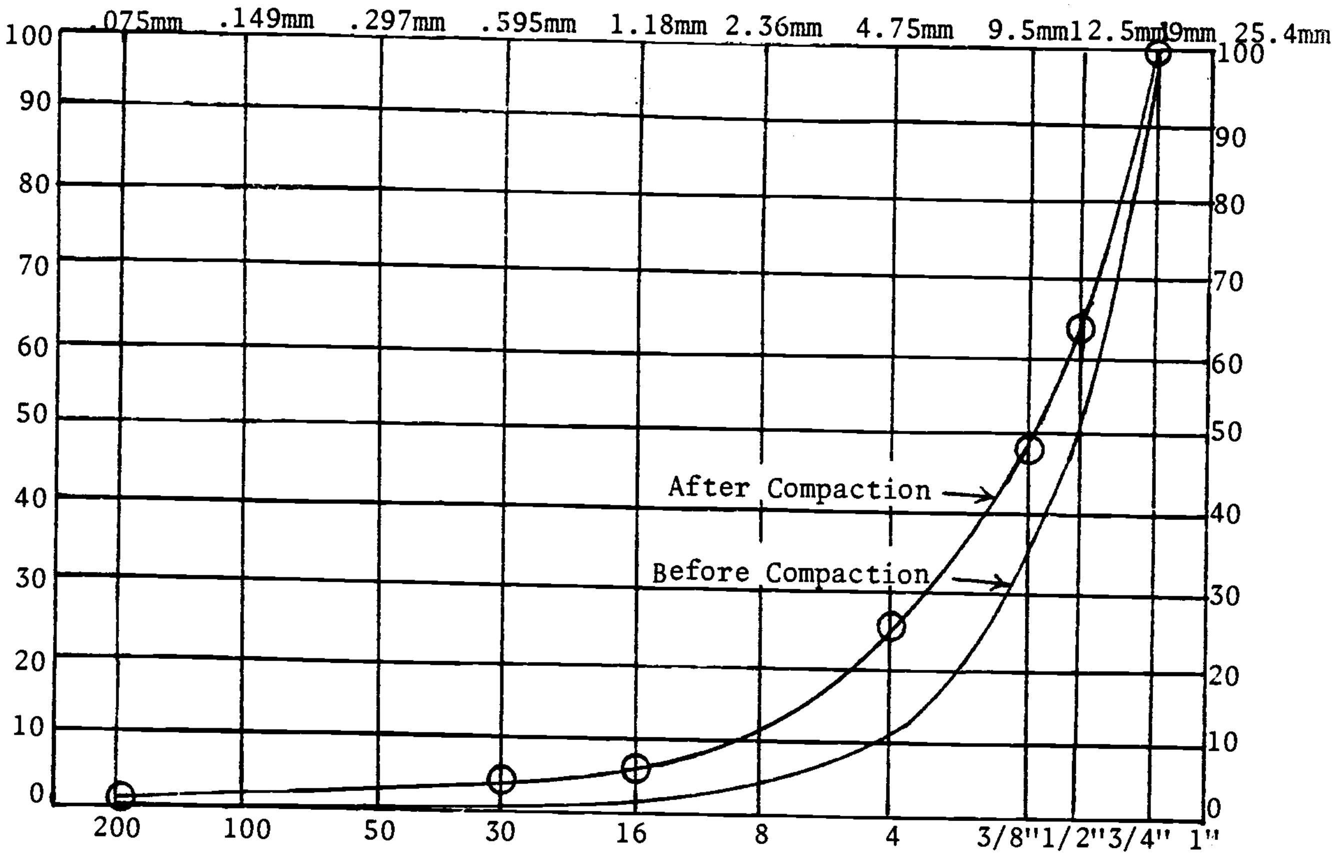


Figure 4.3. Effect of Compaction on Gradation, Oceanlake

determined as a function of curing time, at various testing temperatures, and before and after the water-conditioning procedure, as previously described in the experiment design. The "ultimate cure" condition listed herein is assumed to occur after the vacuum desiccation process previously described.

4.2.1 Modulus vs. Time

The relationship between the modulus and the number of days cured for each of the mixes is shown in Figures 4.4, 4.5, 4.6, and 4.7. The data points represent the average of three samples tested. As seen from these curves, a minimum of thirty days air cure is required for all of the specimens to approach an ultimate cure under laboratory conditions. For this testing, the ultimate cure was assumed to be obtained when all of the moisture was desiccated from the samples and the slope of the M_R vs. time curve became zero. The slopes of the curves are all quite similar, except for the dune sand. This curve rises much faster in the first ten days of cure, probably caused by the hydration of the portland cement additive. This substantiates other findings (19) stating that the inclusion of this material will significantly decrease the time required for curing emulsion mixes.

Also shown on these curves is the influence of confining pressure (σ_3) on the modulus. Generally, the confining stress has a greater influence on the samples in the early stages of cure. As the samples become stiffer, the confining stress has less effect. This effect is best exemplified by the dune sand curves.

As seen from these figures and Table 4.3, the Eckman Creek mix resulted in the highest ultimate modulus values, followed by the Berry

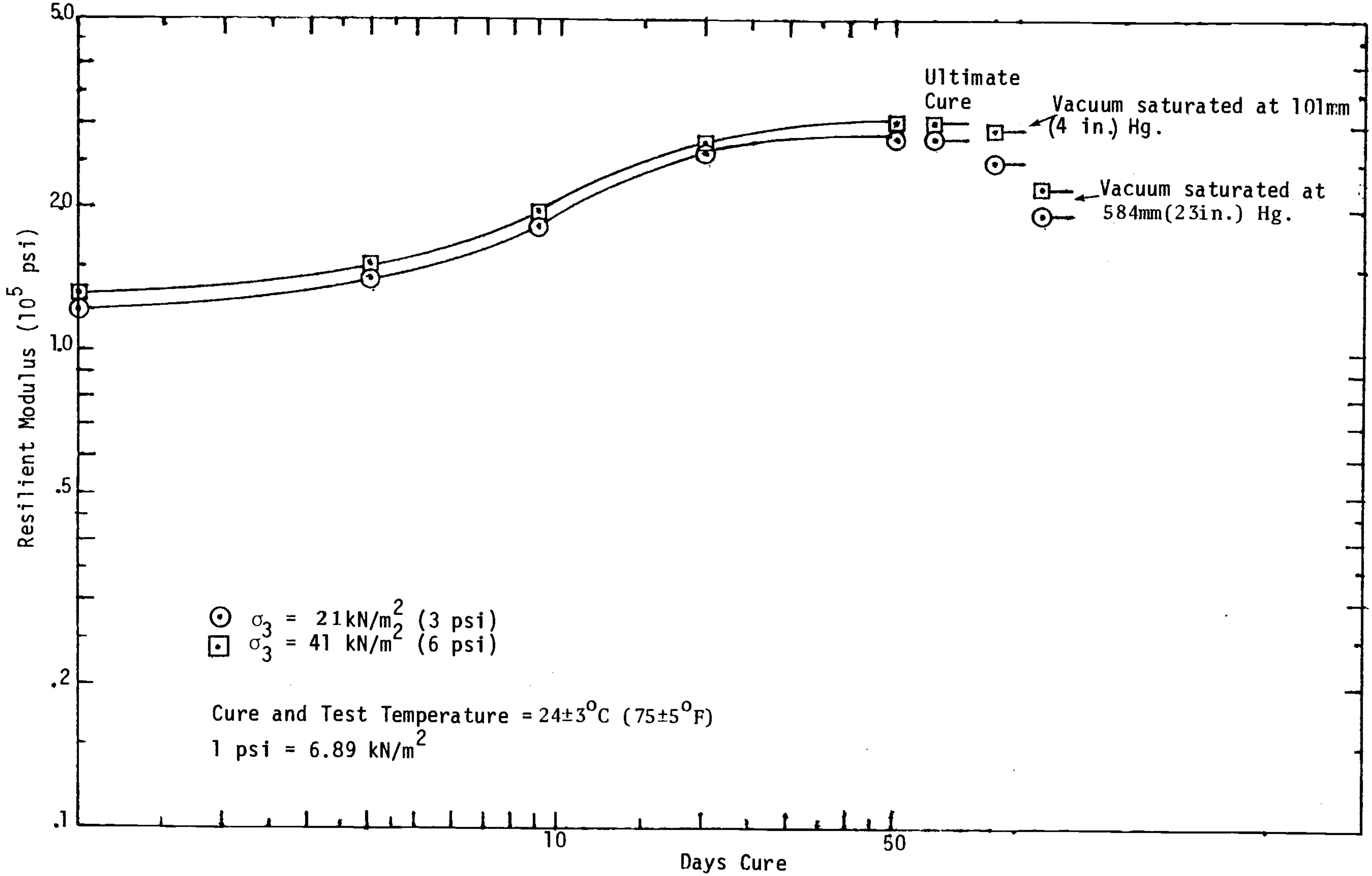


Figure 4.4. Variation in Modulus with Time of Curing, Berry Creek

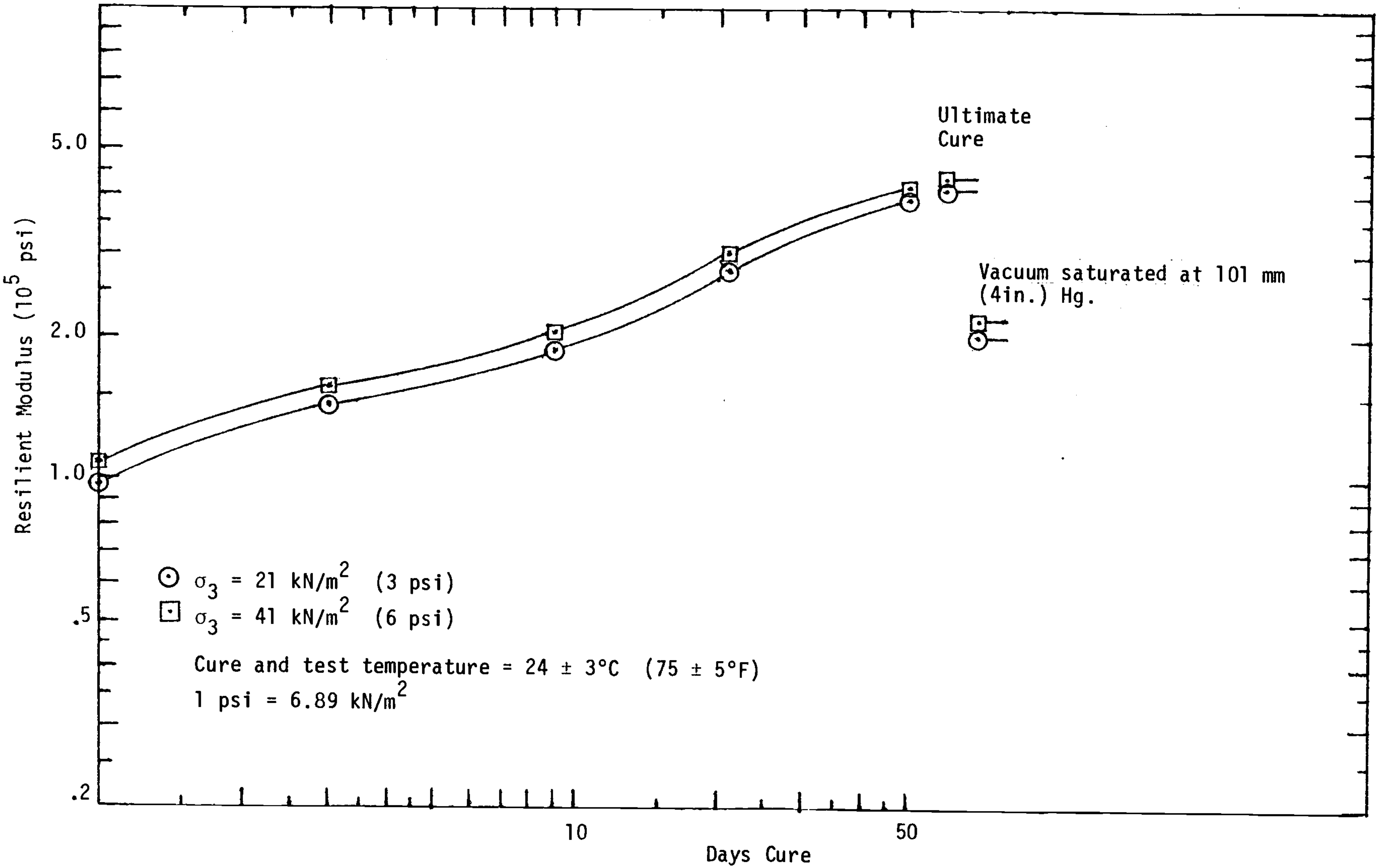


Figure 4.5 Variation in Modulus with Time of Curing, Eckman Creek

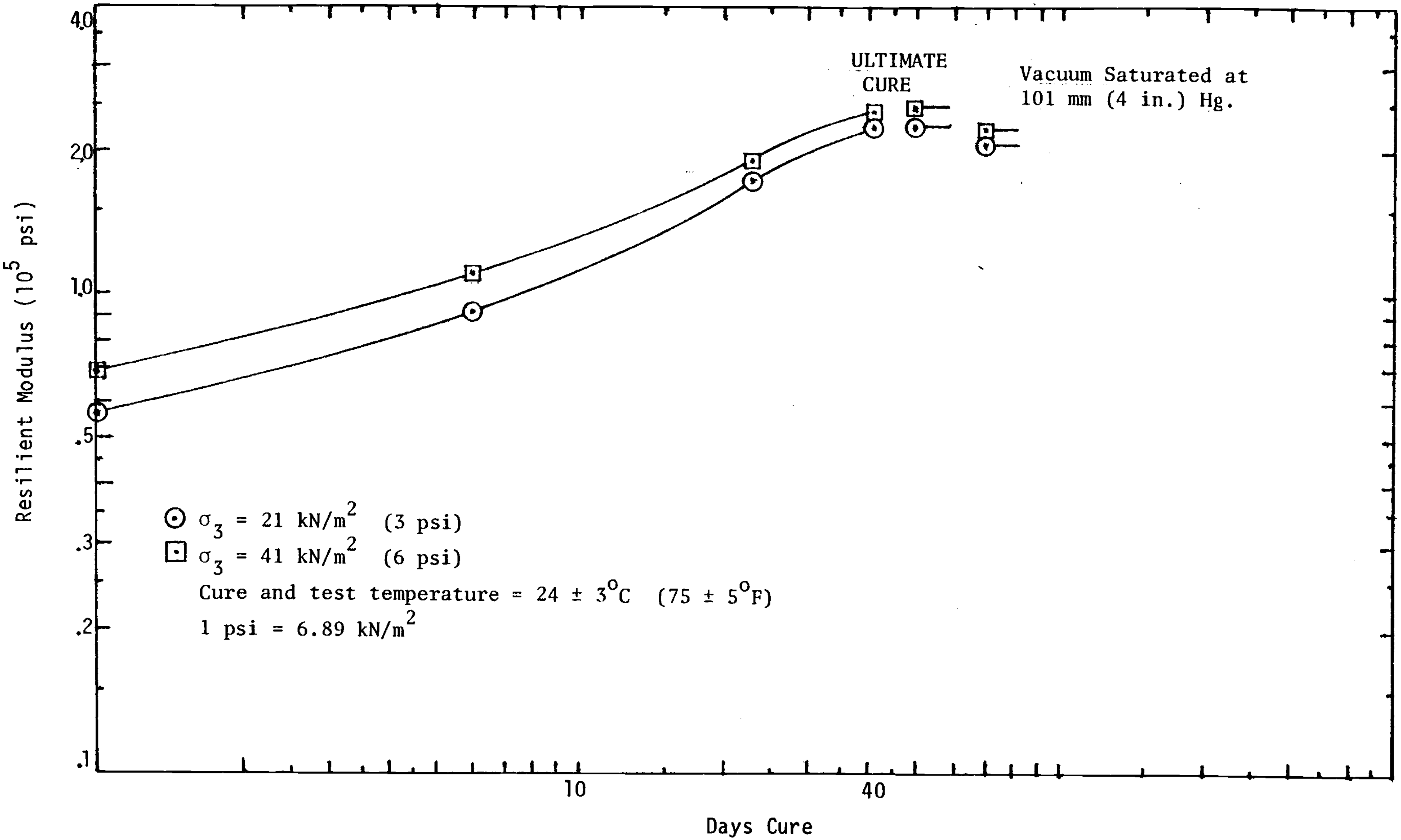


Figure 4.6 Variation in Modulus with Time of Curing, Oceanlake

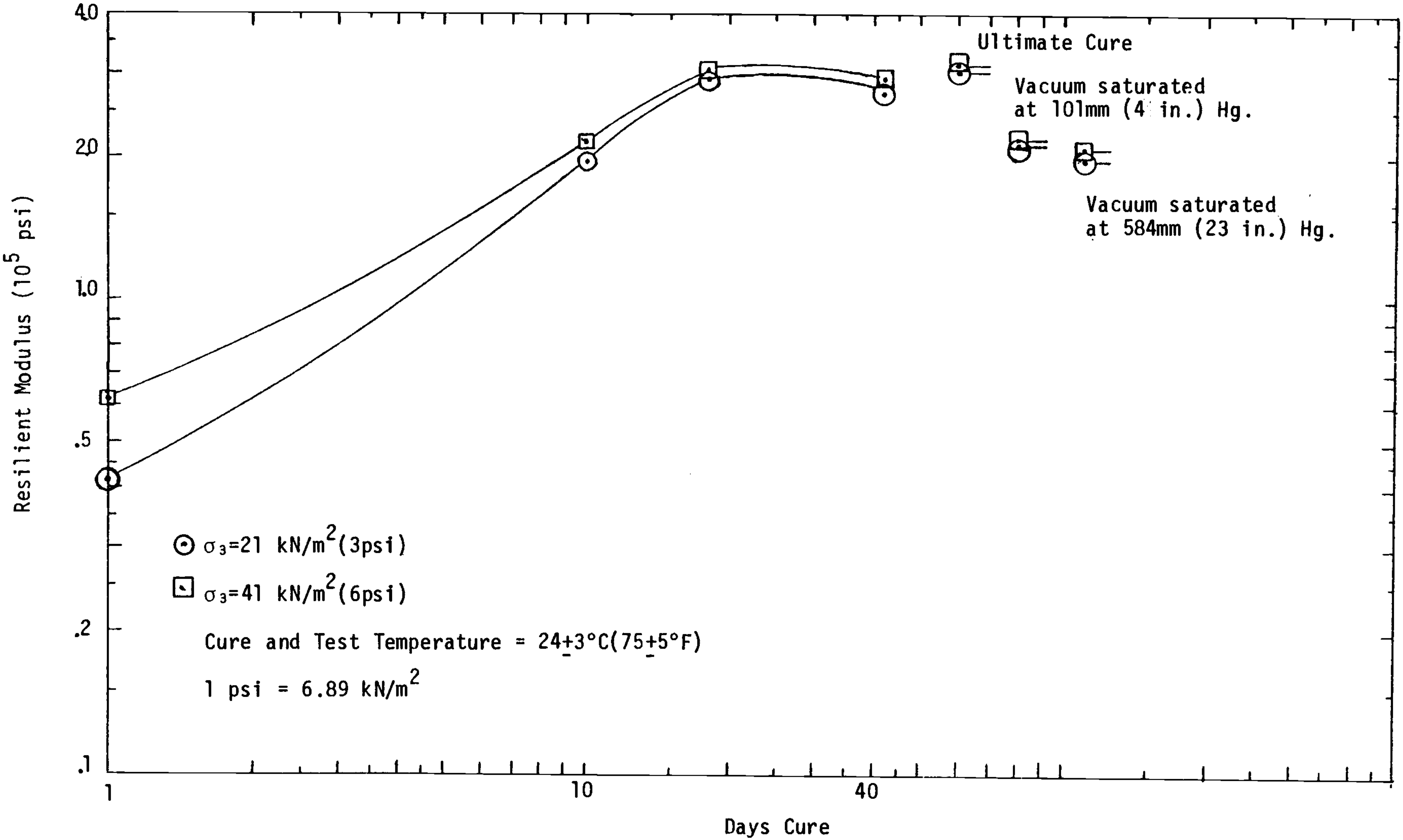


Figure 4.7 Variation in Modulus with Time of Curing, Dune Sand

Table 4.3. Effect of Vacuum Saturation on Modulus.

Aggregate	Confining Stress, psi	Modulus, psi			Percent Reduction	
		Ultimate	After 4 in. Vac. Sat.	After 23 in. Vac. Sat.	After 4 in.	After 23 in.
Berry Creek	Ø	175,000 (4)	-	110,000 (3)	-	37.1
	3	266,000 (10)	179,000 (8)	167,000 (6)	32.7	37.2
	6	292,000 (10)	210,000 (8)	193,000 (6)	28.1	33.9
Eckman Creek	Ø	219,000 (5)	-	98,500 (2)	-	55.0
	3	366,000 (11)	202,000 (3)	121,000 (2)	44.8	66.9
	6	394,000 (11)	219,000 (3)	152,000 (2)	44.4	61.4
Oceanlake	Ø	140,000 (6)	-	87,600 (6)	-	37.4
	3	233,000 (12)	210,000 (3)	118,000 (3)	9.9	49.4
	6	260,000 (12)	222,000 (3)	144,000 (3)	14.6	44.6
Dune Sand	Ø	171,000 (8)	-	128,000 (3)	-	25.1
	3	232,000 (12)	185,000 (6)	194,000 (6)	20.3	16.4
	6	246,000 (10)	201,000 (6)	214,000 (6)	18.3	13.0

note: 1) number in parentheses indicate number of specimens tested
2) 1 inch = 25.4 mm
3) 1 psi = 6.89 kN/m^2

Creek, Oceanlake, and dune sand, respectively. As previously discussed, the Eckman Creek aggregate degraded considerably upon compaction, resulting in a significantly higher density and denser gradation than the other mixes. As the same compactive effort was used for all of the mixes except the dune sand, this would explain the magnitude of the modulus values obtained for this mix. The dune sand, although lacking in strength gain from coarse particle grain interlock, contained a higher asphalt content than the other mixes, and 1.5% portland cement. The Oceanlake mix contained less residual asphalt than the other mixes, along with less mixing water. The open graded mixes, having consistently higher densities than the dune sand mix, resulted in higher modular values, which is consistent with results presented by Hicks, et al. (38) as shown in Figure 4.8.

4.2.2 Modulus vs. Conditioning

The effect on modulus of conditioning is given in Table 4.3. The greatest loss in modulus resulted from vacuum saturating and water soaking the Eckman Creek test specimens. This material lost approximately 45% of its ultimate stiffness by the 102 mm (4 inch) vacuum saturation and about 60% by the 584 mm (23 inch) vacuum saturation-water soak process. The dune sand consistently experienced little reduction in stiffness at both levels of vacuum saturation.

It is evident that both the confining stress and the level of vacuum saturation have a significant effect on the stiffness loss resulting from conditioning the test specimens. Generally, testing at a higher confining stress and after a lower level of vacuum saturation results

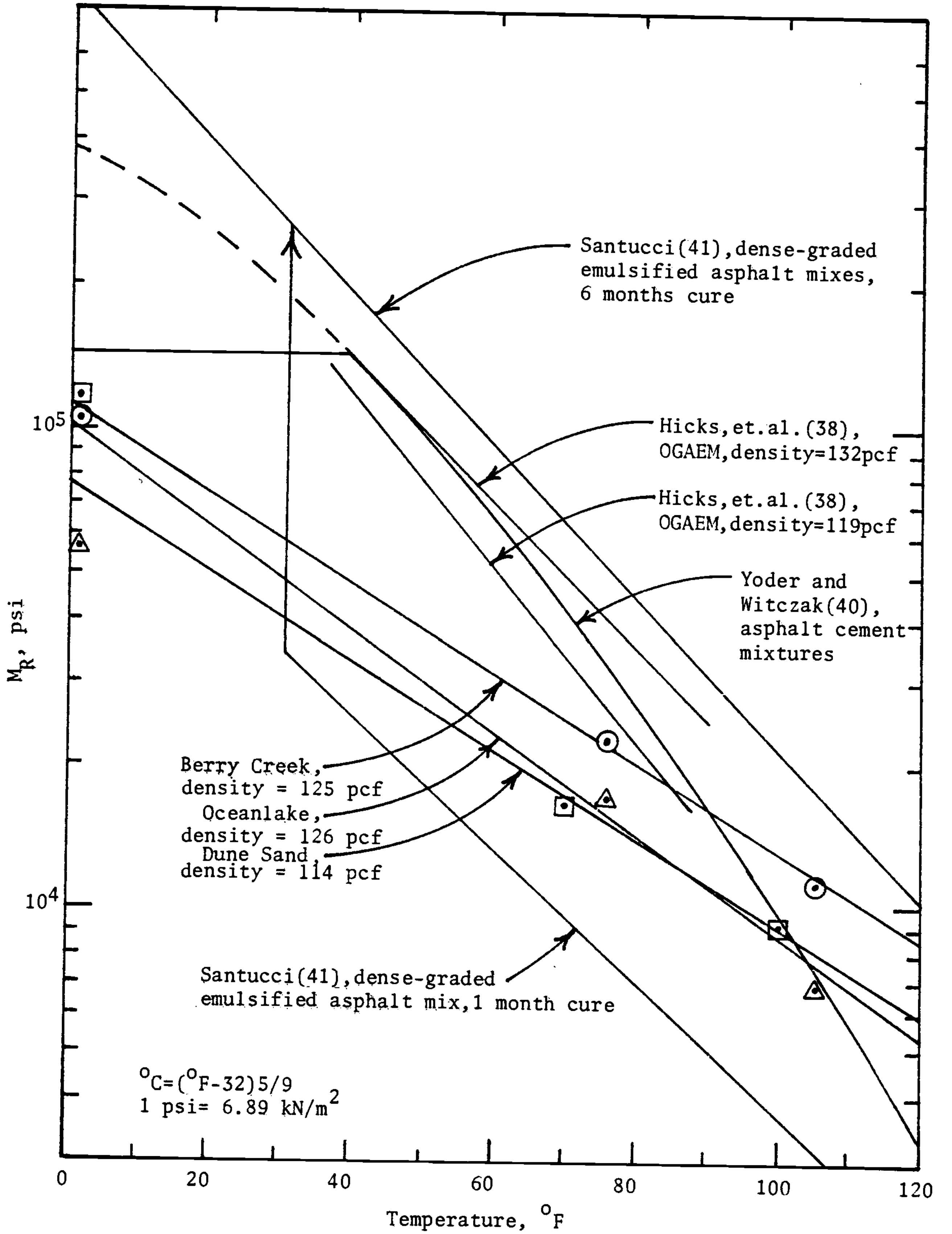


Figure 4.8. Resilient Modulus vs. Temperature

in the lowest percent reduction in stiffness; however, more testing would be required to substantiate this. Also, testing of cores taken from test roads exposed to various moisture conditions would aid in correlating the exposure time and level of vacuum used in laboratory conditioning to actual conditions. Lottman (39) is currently correlating properties of laboratory mixes exposed to a standard saturation and conditioning process with test results from asphalt concrete pavement cores exposed for various amounts of time. Results from this study are expected to give a much better insight to the relation between laboratory conditioning procedures and actual field performance.

4.2.3 Modulus vs. Temperature

The dependency of the modulus on the test temperature for each of the aggregate mixes is given in Figure 4.8. In all cases, the study aggregate mixes compare very well to the values predicted for hot mix concrete by Yoder and Witczak (40). The dependence of the modulus on temperature for hot mix concrete can be predicted by the following equation (40):

$$E_1 = \frac{k_o}{k_1{}^{q^{d_1}}}$$

where E_1 = asphalt concrete modulus, psi

k_o = regression constant, 3.8×10^6

k_1 = regression constant, 1.0046

d_1 = regression constant, 1.45

q = temperature, °F

Considerably more testing is required to determine an equation of this type for the study aggregate mixes.

The curves developed by Santucci (41) represent the variation of modulus with the mean monthly pavement temperature, and with the time period after construction for a specific mix. The modular values of samples not ultimately cured that were tested below 0°C (32°F) were found to equal values of the six month cured condition. Santucci suggests that emulsified asphalt mixes placed in parts of the Southwest, Texas, and Florida are expected to reach their ultimate design modulus in six months, while a two-year cure period is assumed for emulsion mixes placed in northern regions. The study specimens were vacuum desiccated to obtain a near ultimate cured condition.

The curves presented by Hicks, et al. (38) represent the variation of modulus of ultimately cured open graded emulsion mixes as a function of temperature and density. These curves indicate that higher modular values can be obtained with increasing mix densities.

In general, all of the curves reported by others present steeper slopes than those found for the study aggregates. As all of the values obtained fall within acceptable ranges, this is considered to be a factor of the test procedure used, and not of significant importance.

4.3 Indirect Tensile Strength

The indirect tensile strength for each of the aggregate mixes was determined at the ultimate cured condition and after the samples were vacuum saturated at 584 mm (23 in) of Hg pressure for two hours and soaked in water for seven days. The results of this testing are summarized in Table 4.4.

Table 4.4. Effect of Vacuum Saturation on Tensile Strength.

Aggregate	Average Density pcf	Average Tensile Strength, psi		Percent Reduction
		Ultimate Cure	After 23" Hg. Vac. Sat.	
Berry Creek	124.7	27.1 (3) s = 3.1	24.8 (9) s = 5.45	8.5
Eckman Creek	130.4	26.3 (4) s = 5.04	21.9 (3) s = 3.27	16.7
Oceanlake	126.4	16.8 (6) s = 3.68	18.8 (4) s = 5.28	-11.9
Dune Sand	114.5	19.1 (5) s = 4.61	18.7 (3) s = 0.87	2.1

Note: numbers in parentheses indicate number of samples tested

s = standard deviation
1 inch = 25.4 mm
1 psi = 6.89 kN/m^2
1 pcf = 16.02 kg/m^3

Using statistical analysis techniques, specifically the F-test, it can be shown that at a 1% significance level (α = .01), the population variances of the sample information are equal. Assuming that both sample populations of each aggregate type have relative frequency distributions that are approximately normal, Students "t statistic" has been used to analyze the information obtained. This analysis indicates that the unconditioned values, and the strength reductions listed, can be accepted with about 75% confidence for the Berry Creek, 85% for the Eckman Creek, and less than 70% for the dune sand. The apparent increase in strength exhibited by Oceanlake material is accepted with 75% confidence. Based on similar testing, Lottman (39) has stated that such an increase is not unusual.

In comparing the ultimate strengths of the different aggregates, the Berry Creek and Eckman Creek mixes have consistently shown higher values than the Oceanlake or dune sand. For the open graded mixes, this can be attributed to the denser gradation obtained upon compaction and the higher amount of emulsion used in preparing the Berry Creek and Eckman Creek specimens. With these admixture contents, the test results of the conditioned marginal aggregates are significantly greater than the unconditioned strengths of the quality aggregates. All of the values obtained were lower than those reported ny Adedimila (42) in tests on mix asphalt samples.

The results of this testing indicate that tensile strength loss from moisture exposure is primarily a function of the quality of the aggregate. This is based on the fact that the Oceanlake and dune sand aggregates exhibited zero and slight strength reductions compared to that obtained for the marine basalts. However, because of possible differences in the homogeneity of the samples, the relatively low confidence intervals obtained, and the small number of samples tested, it is felt that there is insufficient evidence to make definite conclusions from these results.

4.4 Fatigue

The diametral fatigue curves obtained for the study aggregates are given in Figures 4.9, 4.10, 4.11, 4.12, and 4.13 for unconditioned and moisture-conditioned specimens. Table 4.5 summarizes these fatigue equations along with other applicable equations obtained for similar pavement mixes. Individual test results are given in Table B-4 of Appendix B.

4.4.1 Before Conditioning

Figure 4.9 gives the fatigue curves of the unconditioned study aggregate mixes along with results reported by other investigators (41,42). The fatigue equations are summarized in Table 4.5. The strain vs. fatigue life equations are given in the form of:

$$N_f = k_1 \left(\frac{1}{\varepsilon}\right)^{n_1}$$

where N_f = number of repetitions to failure,

ε = initial strain in the mixture, and

k_1 and n_1 = constants.

In analyzing the fatigue equations, a low k_1 value generally indicates a high fatigue life for the pavement mix. A low n_1 value indicates a steep slope in the strain vs. number of repetitions relationship.

In discussing or comparing the fatigue life of laboratory tested open graded emulsion mixes, the intrinsic nature (high void content) of

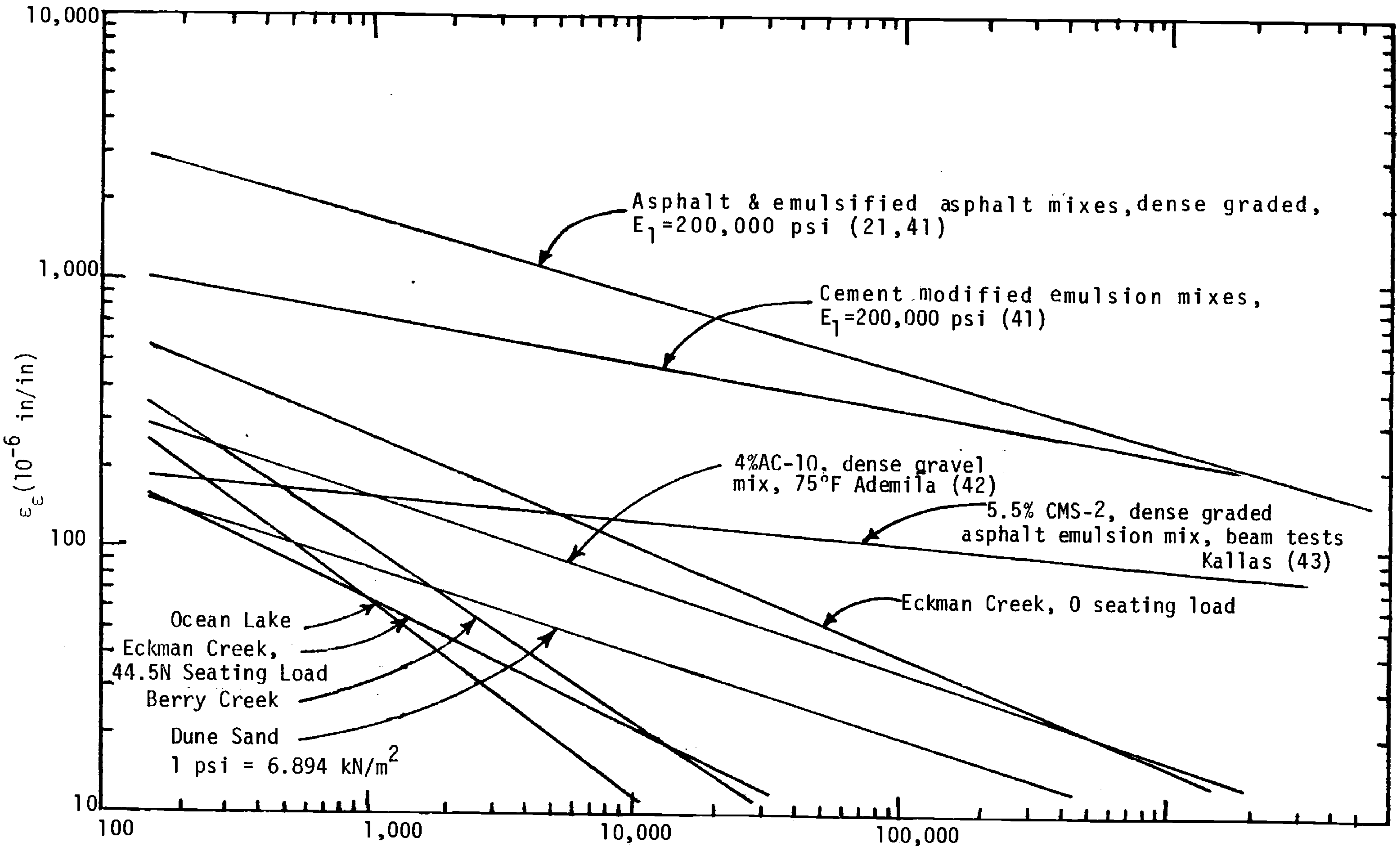


Figure 4.9 Diametral Fatigue Curves of Unconditioned Aggregate Mixes

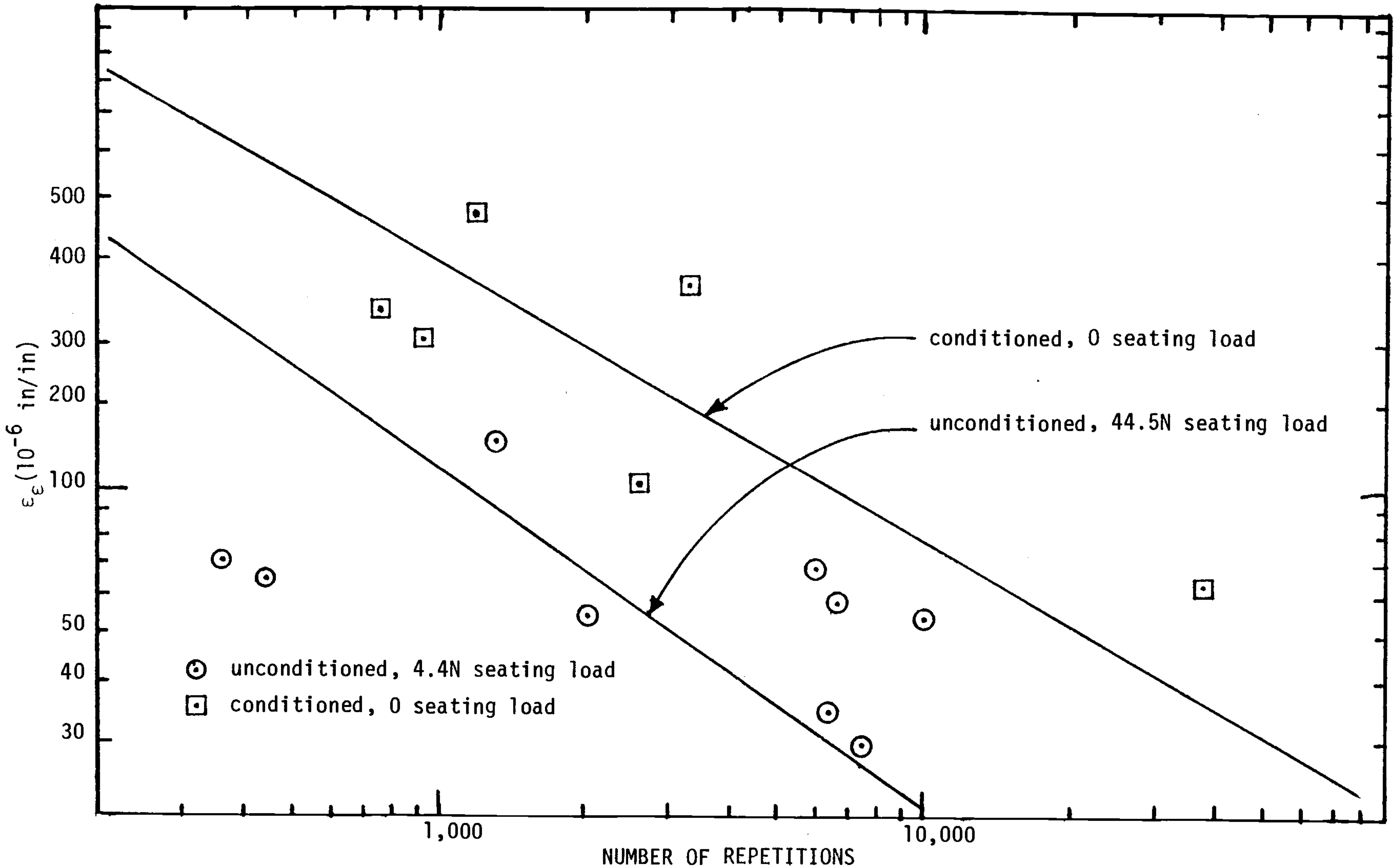


Figure 4.10 Effect of Conditioning on Diametral Fatigue, Berry Creek

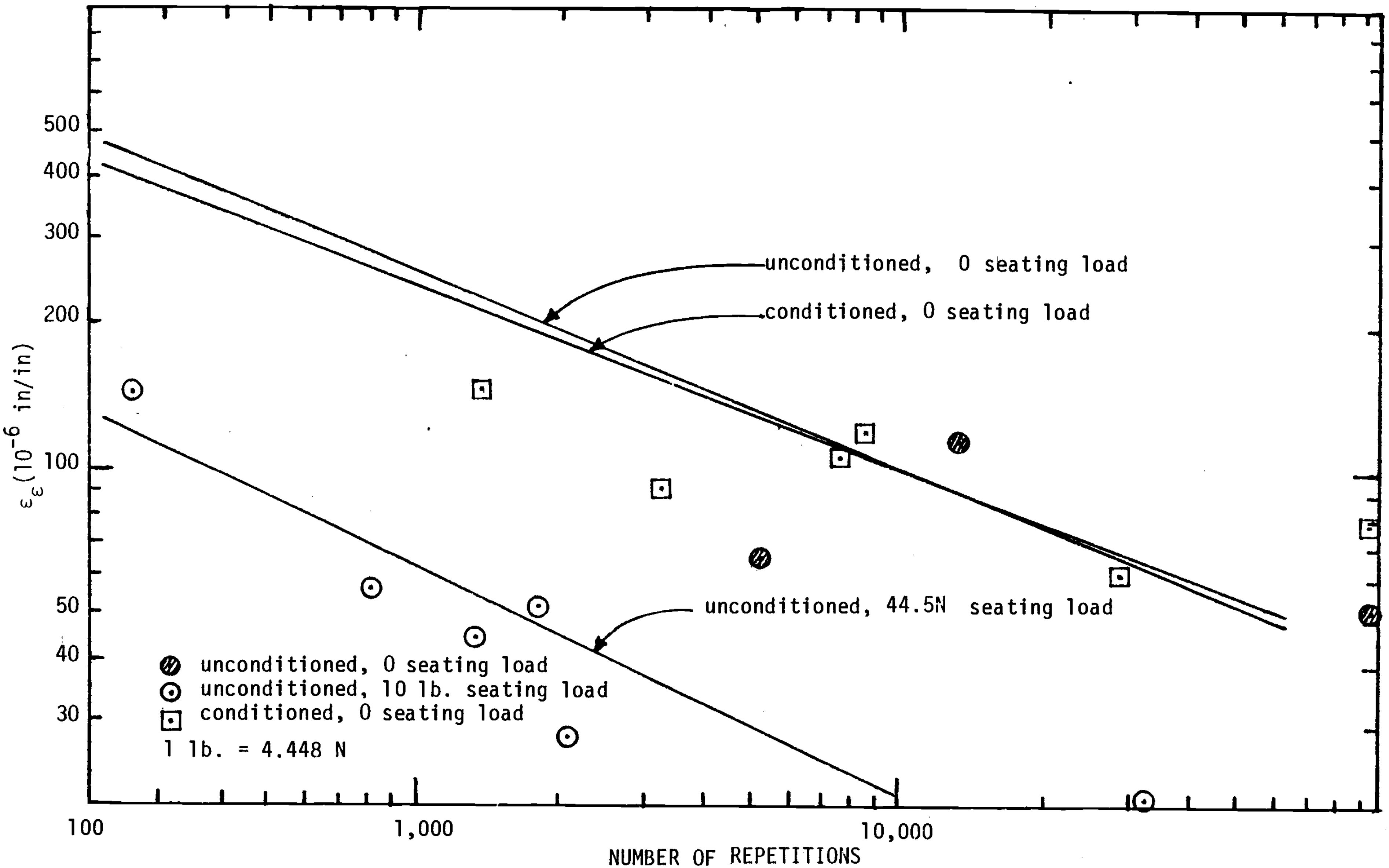


Figure 4.11 Effect of Conditioning on Diametral Fatigue Results, Eckman Creek

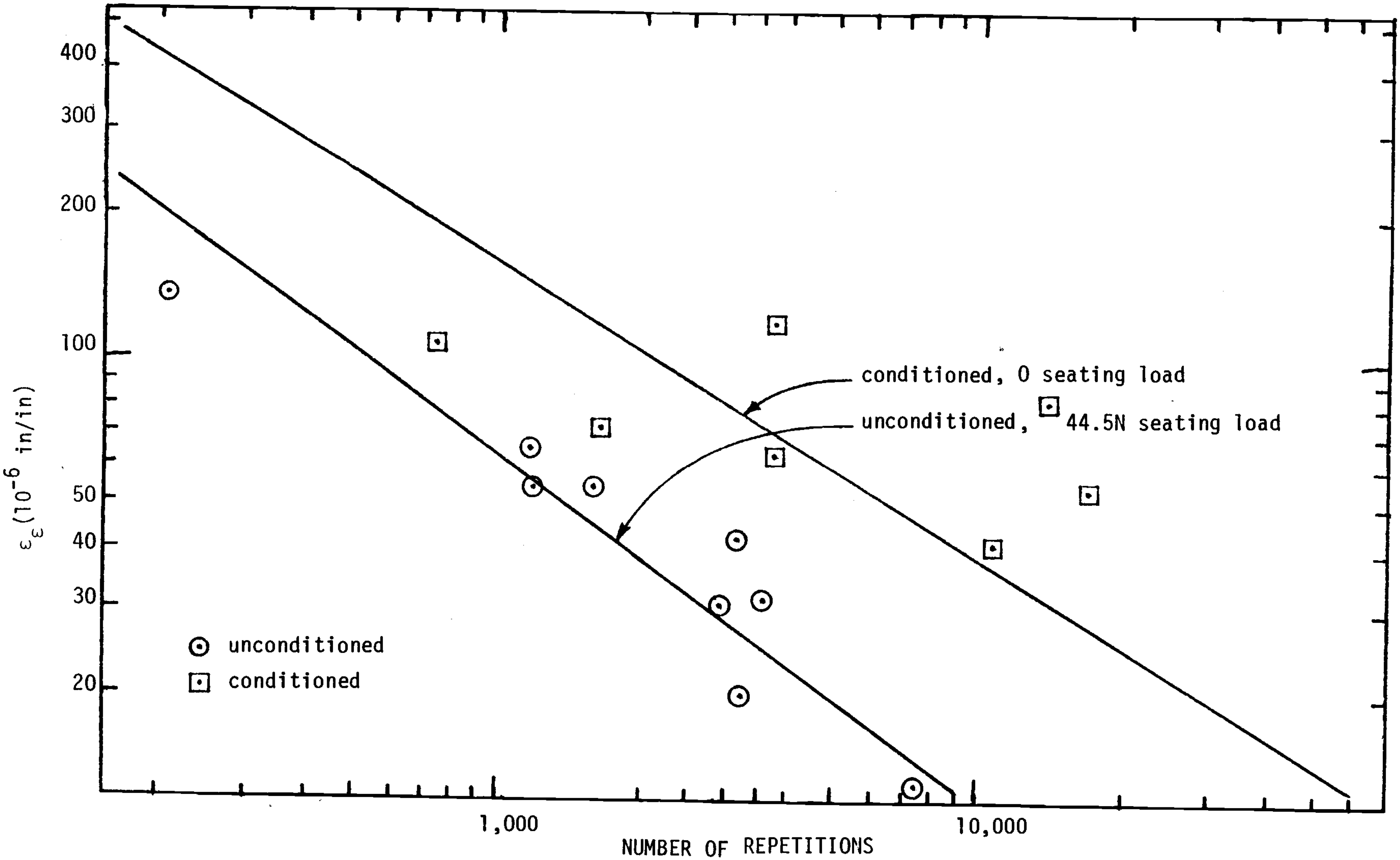


Figure 4.12 Effect of Conditioning on Diametral Fatigue, Oceanlake

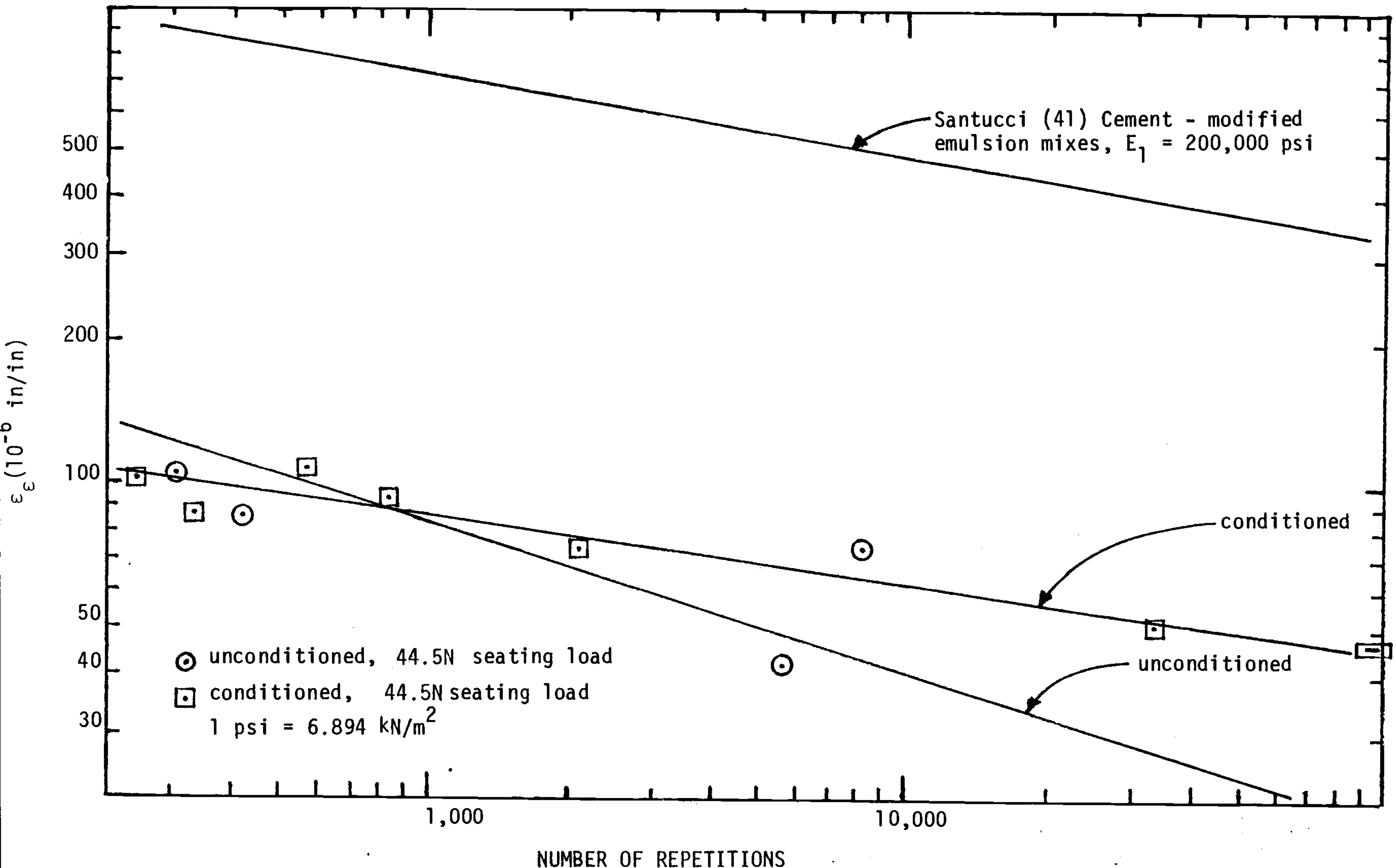


Figure 4.13 Effect of Conditioning on Diametral Fatigue, Dune Sand

Table 4.5. Summary of Fatigue Equations ($N_f = k_1 (\frac{1}{\varepsilon})^{n_1}$)

Aggregate	k_1	n_1	r
Berry Creek, unconditioned, combined seating loads	7.67×10^{-4}	1.53	.55
Berry Creek, conditioned, 0.0 seating load	8.50×10^{-4}	1.72	.78
Eckman Creek, unconditioned, 0.0 seating load	1.78×10^{-6}	2.44	.60
Eckman Creek, unconditioned, 10.0 lb. seating load	1.61×10^{-6}	2.09	.88
Eckman Creek, conditioned, 0.0 seating load	3.65×10^{-7}	2.61	.69
Oceanlake, unconditioned, combined seating load	1.63×10^{-3}	1.38	.93
Oceanlake, conditioned, 0.0 seating load	6.59×10^{-4}	1.63	.47
Dune Sand, unconditioned, 10.0 lb. seating load	9.13×10^{-11}	3.20	.73
Dune Sand, conditioned, 10.0 lb. seating load	1.56×19^{-26}	7.09	.96
Dense-Graded Crushed Gravel, 5.5% CMS-2, 15% voids, beam tests, Kallas (43)	5.012×10^{-43}	11.9	.42
Dense Gravel Mix, 4% AC-10, 75°F, diametral tests, Adedimila (42)	2.10×10^{-9}	3.06	.97
Emulsified Asphalt Mixes, Air Voids = 5%, E_1 = 200,000 psi, beam tests, Santucci (41)	1.81×10^{-7}	3.53	
Cement-Modified emulsion mixes, Air Voids = 5%, E_1 = 200,000 psi, beam tests, Santucci (41)	4.98×10^{-16}	5.80	

1b = 4.45N

1 psi = 6.894 kN/m^2

these materials must be considered. In fatigue testing asphalt cement concretes, Adedimila (42) found that increasing air voids above 3% results in a decrease in fatigue life. This is consistent with fatigue life adjustments for varying void contents recommended by Santucci (41). As previously discussed, the extremely high void content in open graded mixes allows for much greater flexibility than conventional mixes, and when used in pavements, they have seldom failed from fatigue (21,41). In the laboratory testing of the study open graded materials, a majority of the samples failed primarily from deformation, rather than a distinct vertical crack through the center of the specimen.

A majority of the specimens were tested with a 44.5 N (10 lb.) seating load in order to secure the specimen in the testing frame and to induce a pulsing load rather than an impact load. As seen from the two Eckman Creek curves comparing a zero seating load and a 44.5 N seating load, this procedure adversely affects the fatigue life.

In studying variations of hot mix fatigue lives with test temperature, percent asphalt cement, and test method, Adedimila (42) found typical ranges in k_1 of 5×10^{-20} to 5×10^{-5}, and in n_1 of 2.5 to 6.3. Considering the values of $k_1 = 5.012 \times 10^{-43}$ and $n_1 = 11.9$ obtained by Kallas (43) for beam tests on a dense graded emulsion mix, the values obtained from this study can be considered appropriate for open graded emulsion mixes. The cement-modified dune sand mix results in a flatter slope than typically found for asphalt and emulsified asphalt curves, which is consistent with results found by Santucci (41).

The correlation coefficients (r) found for the study mixes are lower than those typically found for asphalt cement mixes (42,43). This indicates that comparatively more scatter in fatigue data exists for open graded emulsion mixes than for hot mixes. This is consistent with results found by Kallas (43) with beam flexural tests on emulsion mixes (Table 4.5) and is attributable primarily to a lack of homogeneity in samples, with asphalt coatings on aggregates and distribution of asphalt less uniform than that obtained with asphalt cement mixes.

The curve on Figure 4.9 representing a dense gravel mix with 4% AC-10 asphalt was provided to compare results of similar residual asphalt contents (3.3% asphalt in Oceanlake, 4% residual asphalt in Berry Creek and Eckman Creek) tested under similar procedures (diametral fatigue) and conditions. As seen from Figure 4.9, the study aggregate mixes compare much more favorably with this curve than for the curve reported by Kallas (43) from beam tests, with both of the mixes being dense graded. Kallas reports that from beam flexural tests, the fatigue curves for dense graded emulsion mixes are consistently flatter than for asphalt cement mixes. As the fatigue curve given in Figure 4.9 by Santucci (41) for asphalt and emulsified asphalt mixes was also derived from beam flexural tests, and from the comparison of diametral test results of these materials, the tendency for flatter slopes does not appear to be verified by this study.

The fatigue curves given by Santucci (41) have been developed from flexural beam tests and have been shifted as recommended by Van Dijk (44) to account for differences in laboratory and field crack propagation. The accuracy of these curves in predicting actual fatigue failure has been verified by a number of studies (21,41,45). On the basis of the modular values obtained for the study aggregate mixes, these curves are felt to provide a good representation of the behavior of the study aggregate mixes. This assumption is discussed further in

Section 4.5.

4.4.2 After Conditioning

The fatigue curves showing the effect of water conditioning on each of the study aggregate mixes are given in Figures 4.10, 4.11, 4.12, and 4.13. As seen from these figures, the conditioned open graded curves are consistently higher than those for unconditioned specimens. As seen from Figure 4.11, the primary reason for this shift is attributable to the effect of the seating load. All of the conditioned specimens were tested without a seating load, while the unconditioned curves were developed from samples tested both with and without a seating load.

All of the cement-modified dune sand mixtures were tested with a 44.5 N (10 lb.) seating load. For this mix, an intersection between the two curves exists at about 1000 load repetitions, after which the conditioned samples appear to have a greater fatigue life. This tendency also occurs for the Eckman Creek mix at about 10,000 load repetitions, however, little test data exist to support it.

In general, the water conditioning procedure has very little, if any, effect on the initial strain vs. load repetition relationship of the study aggregate mixes if the tests are made with the same seating load.

4.5 Discussion of Results

The conditioning process used in this study consisted of vacuum saturating the samples at either 101 mm (4 in.) or 584 mm (23 in.) of mercury pressure for a period of two hours and soaking the samples in water for seven days. For short term moisture damage prediction in asphalt concrete mixes, Lottman (39) prescribes vacuum saturation at 660 mm (26 in.) of mercury for a period of 30 minutes, followed by two hours of water soaking. Chevron (37) has a moisture conditioning procedure that calls for vacuum saturating samples for one hour at 101 mm (4 in.) of Hg. and soaking in water for an additional hour. Schmidt (46) has tested asphalt concrete samples after saturation at 30 mm (1.18 in.) of mercury with various temperature cycles over a number of days exposure. Jimenez (47) reports that soaking specimens at 381 mm (15 in.) Hg. of vacuum pressure for 15 minutes plus 15 minutes at ambient pressure is equivalent to soaking at one hour at 101 mm (4 in.) Hg. vacuum followed by one hour of soaking. As previously shown, the level of vacuum pressure has a significant effect on the amount of damage experienced by the samples, although it is still uncertain as to how this relates to actual conditions.

Recommended Modular Values - As the retained stiffness at 41 kN/m^2 (6 psi) confining stress for each of the aggregate mixes conditioned with the 101 mm (4 in.) vacuum saturation process was greater than 1379 x 10^3 kN/m^2 (200,000 psi), this value is used in the development of structural layer coefficients. A confining stress of 41 kN/m^2 (6 psi) is chosen to represent conditions existing in an actual pavement. This is the standard modulus value also recommended for design of open mixes by Hatch (21), indicating that the modular values for all of the open mixes (including those with marginal aggregates) are at an acceptable level.

The low levels of stiffness reduction found for the dune sand mix treated with portland cement are consistent with results found by Schmidt (19). Schmidt also found that the modulus of exposed and unexposed cement treated sand mixes continue to increase for a considerable period of time. For the study sand mix, as the higher level vacuum saturated samples resulted in higher modular values, curing time is seen to be more influential than the level of vacuum used. The sand tested by Schmidt is quite comparable to the dune sand studied here, however, a harder based asphalt in the emulsion (85/100 penetration compared to 100/250) was used. This resulted in conditioned modular values of greater than the 2068 x 10^3 kN/m^2 (300,000 psi) found in this study. From this fact and similar test data (19) indicating that much higher modular values are obtained from lower penetration base asphalts, the stiffness of the dune sand is likely to be improved significantly by using a harder based asphalt emulsion such as CSS-1h. For the development of structural layer coefficients for the dune sand, a value of 2068 x 10^3 kN/m^2 (300,000 psi) is assumed.

Recommended Fatigue Relationships - Due to the fact that little or no information is available relating diametral fatigue curves of open graded emulsion mixes to actual performance obtained in field conditions, the previously discussed curves developed by Santucci are used for further design analysis. The discrepancies between the fatigue results obtained in this study and actual field conditions might be remedied by further testing varying such test factors as the seating load, load duration and frequency. Although not considered in this study, these latter two factors have been proven (48) to significantly affect diametral fatigue performance of hot mix asphalts. Up to a certain level, increasing rest periods have been found to significantly increase the fatigue life of laboratory tested specimens. With a constant load duration, frequency variations of 3 to 30 cycles per minute have no effect on fatigue life.

However, increasing frequencies from 30 to 100 cycles per minute decrease the fatigue life (48). A constant value of 60 cycles per minute was used in this testing, indicating that an additional shift would be appropriate based on loading frequency. In future diametral fatigue testing of open graded asphalt emulsion mixes, no seating load should be used and the load frequency should be limited to 30 cycles per minute.

5.0 DEVELOPMENT OF STRUCTURAL LAYER COEFFICIENTS

In this chapter, the results of the previous chapters are used to develop design recommendations for use of the study aggregates in high performance pavements. The pavement performance analysis procedures are given, along with design requirements for use of marginal aggregate mixes and quality mixes. With this information, layer equivalencies are determined from which layer coefficients are developed for use in standard design techniques. Finally, the results of this analysis are compared with standards used by other agencies.

5.1 Approach

For this task, layered system elastic theory principles (49) are implemented using properties of the study aggregates treated with emulsified asphalt and of hot mix asphalt concrete in order to determine relative layer equivalencies. These equivalencies compare required pavement thicknesses of the study aggregate mixes with thicknesses required for a high quality hot mix pavement. Pavement failure criteria are defined by establishing the critical strain levels in a structural section for a given number of load repetitions. With the aid of a computer, the maximum tensile strain levels in the pavement mixes are calculated over a range of pavement thicknesses. Using this information, a sufficient thickness is determined to limit strain in the bottom of the surfacing layer (fatigue) and in the top of the subgrade (rutting) for various amounts of traffic.

The procedure followed in calculating layer equivalencies has been previously outlined in Section 3.3.3 of the Experiment Design. From the layer equivalencies determined, layer coefficients are developed for use in AASHTO pavement thickness design procedures (50).

5.2 Failure Criteria

As previously stated, prevention of fatigue cracking and pavement rutting are of primary concern in this design method. Fatigue cracking is defined by excessive horizontal strain (ε_t) in the bottom of the surfacing layer. Rutting is characterized by excessive vertical strain (ε_v) on top of the subgrade. As discussed in the literature review, it is believed that the fatigue behavior open graded emulsified asphalt pavements is close to that developed by Santucci (41) and is given in Figure 5.1 for the emulsified asphalt mixes and Figure 5.2 for the cement modified asphalt emulsion mixes. These curves were developed from extensive laboratory testing and shifted to the right to more closely represent actual conditions (41). They have also been proven to closely simulate actual fatigue behavior (21,44,45). Significantly more testing and comparison with test roads would be required before shift factors for the diametral fatigue curves for the study aggregate mixes could be developed.

The subgrade strain criteria, also given by Santucci, is shown in Figure 5.3. This curve has also been shown to closely simulate actual conditions for open graded asphalt emulsion mixes (21,41,45) and was empirically developed for a limiting rut depth of 9.5 mm (3/8 inch). The critical strain levels given by this curve are used for all of the material types under study.

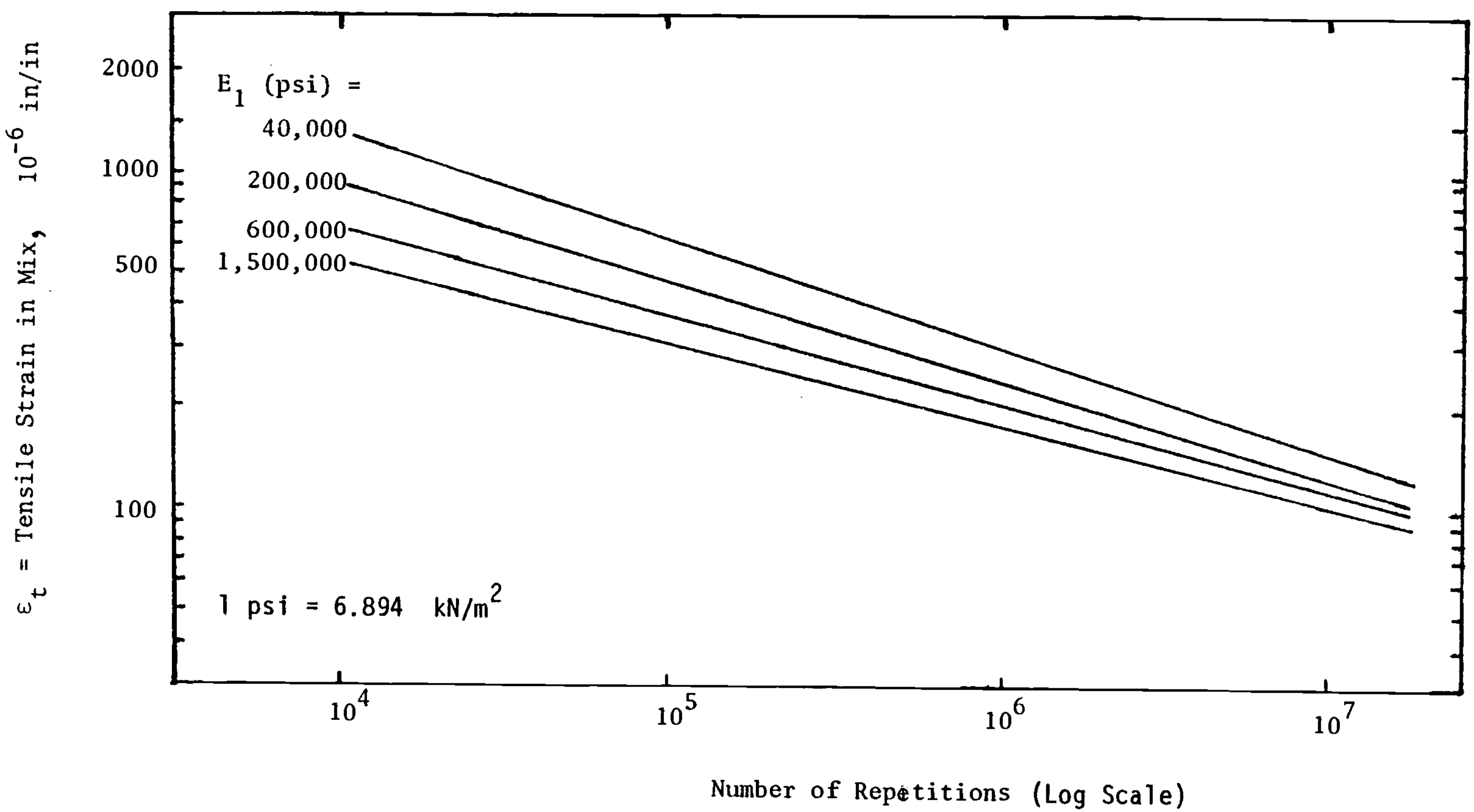


Figure 5.1. Fatigue Criteria for Asphalt and Emulsion Mixes (after reference 41)

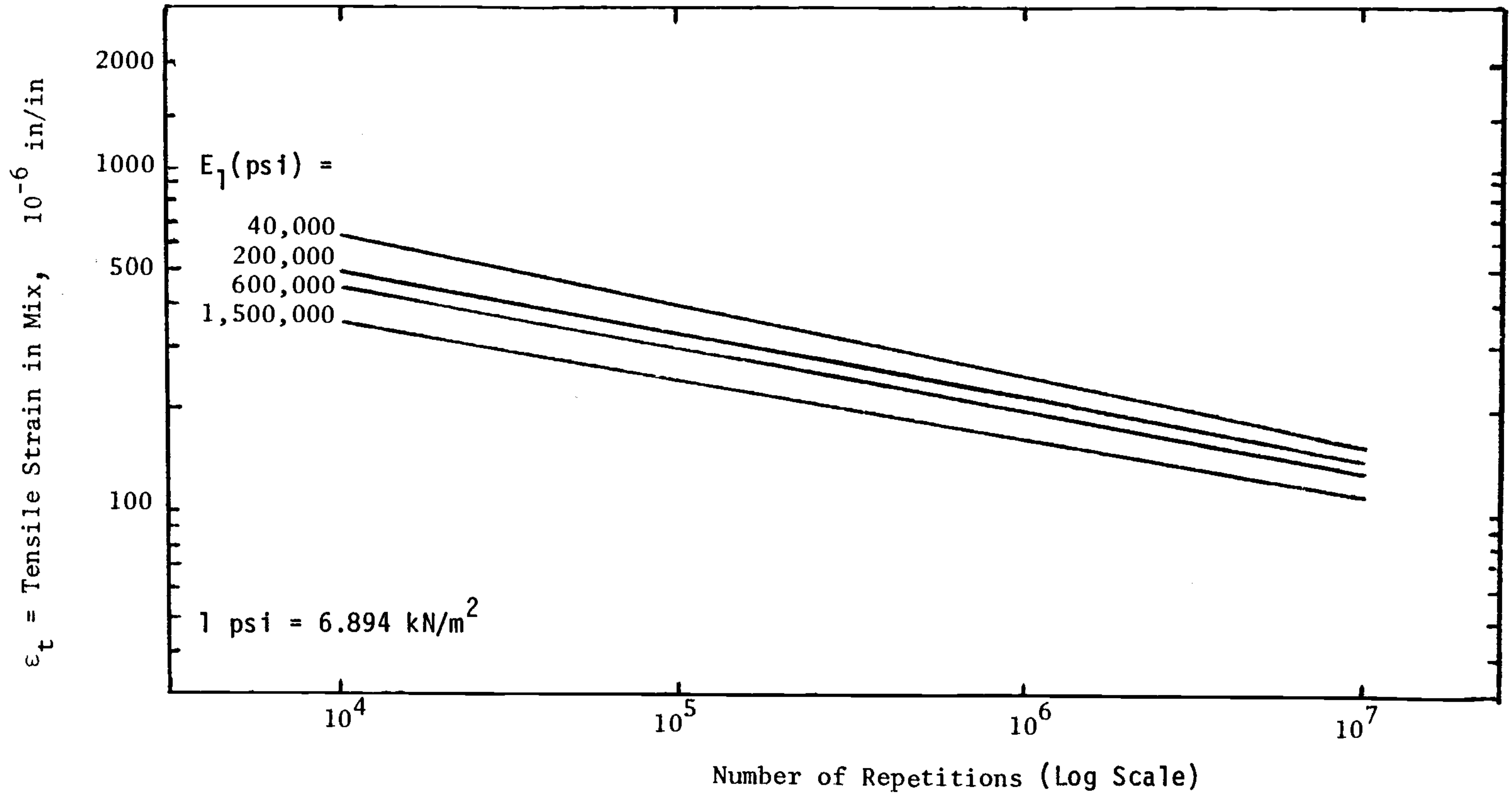


Figure 5.2. Fatigue Criteria for Cement-modified Emulsfied Asphalt Mixes

(after reference 41)

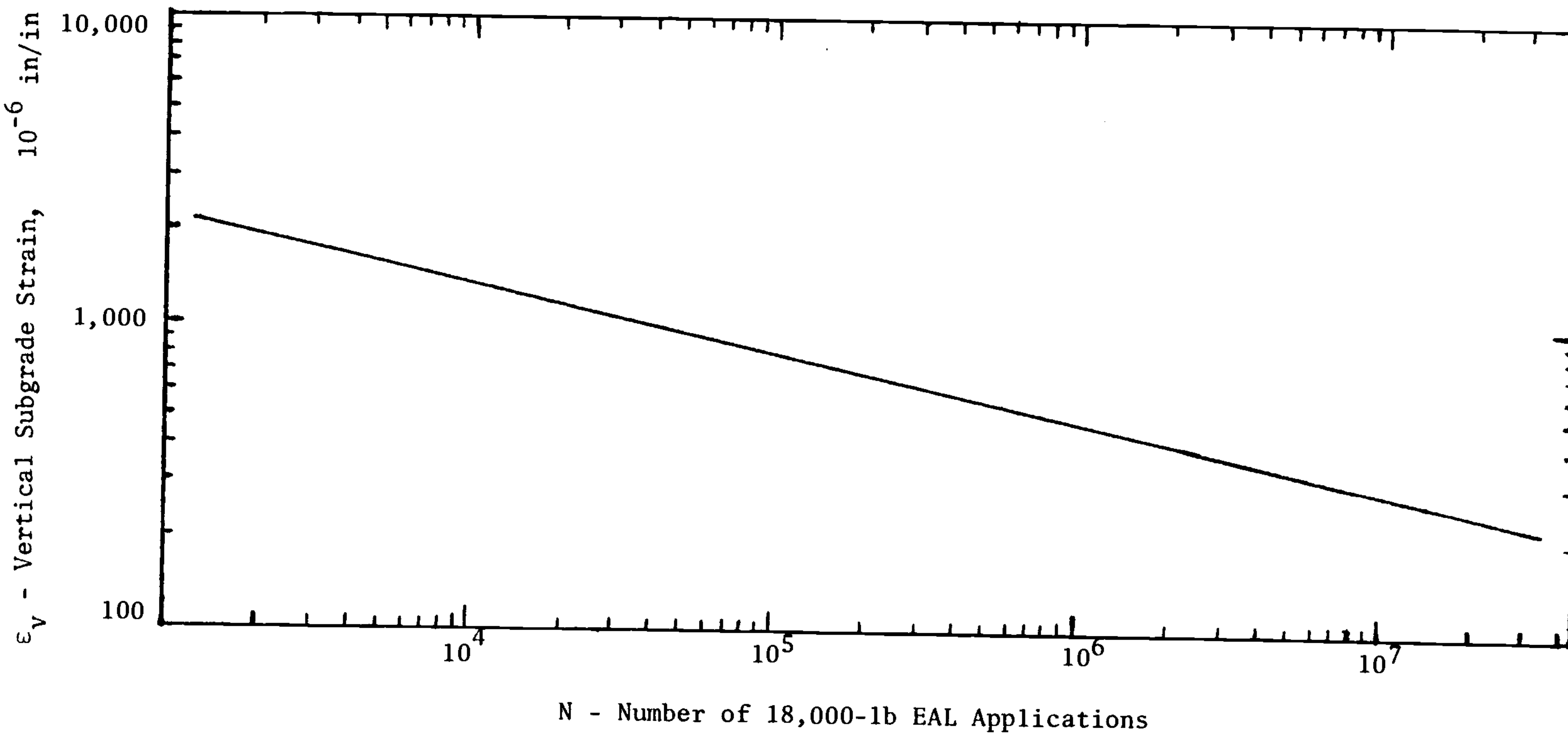


Figure 5.3. Subgrade Strain Criteria
(after reference 41)

5.3 Development of Layer Equivalencies

With the failure criteria known, the maximum strain levels in a given pavement system must be analyzed in order to develop equivalent thicknesses of the marginal aggregate mixes and hot mix asphalt concrete. These strain levels are analyzed for the typical pavement cross-sections shown in Figure 5.4. ELSYM5, a layered system analysis computer program (49), was used for this strain analysis, the results of which are given in Tables 5.1 and 5.2. Loading was assumed to be an 80 kN (18 kip) single axle applied by two 20 kN (4.5 kip) circular loads with application pressures of 586 kN/m^2 (85 psi) and spaced 312.9 mm (12.32 inches) apart. Poisson's ratio is assumed to be 0.35 for all layers. Cement-modified mixes are reported to have slightly lower values (40), however, this is not expected to significantly affect results. The maximum strain was determined at the bottom of the surfacing layer and at the top of the subgrade for varying magnitudes of surfacing mix and subgrade modulus and for different surfacing layer and base layer thicknesses. This information is plotted in Figures 5.5 to 5.12 to illustrate the surfacing layer thickness vs. strain relationships. With this information, layer equivalencies can be determined by obtaining the critical strain levels from Figures 5.1, 5.2, and 5.3 for a given number of 80 kN (18 kip) equivalent axle load (EAL) applications and then entering the surfacing layer thickness vs. strain charts to obtain the required layer thickness. As the fatigue curves of the marginal aggregates appear to be the same whether conditioned or unconditioned by moisture, the fatigue curves discussed previously are assumed to accurately reflect the behavior of these aggregates. The hot mix modulus is assumed to equal 2758 x 10^3 kN/m^2 (400,000 psi) (21), the cement-modified emulsion

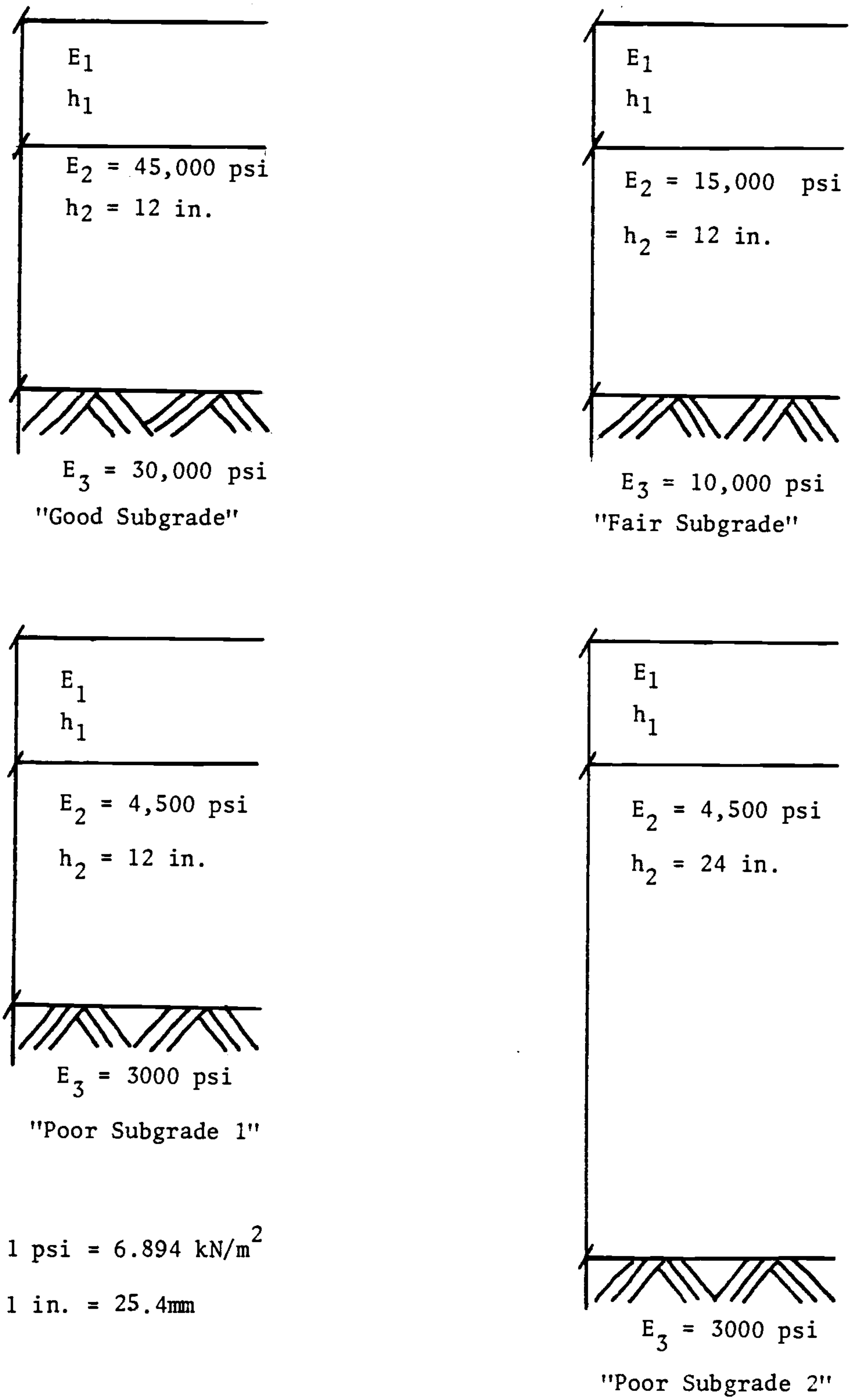


Figure 5.4. Pavement Cross-sections Analyzed

Table 5.1. Maximum Tensile Strain at the Bottom of the Surfacing Layer ($x10^{-6}$ in/in) (after reference 21).

Modulus of Surfacing Mix	"Good" Subgrade (*1) (Subgrade Modulus = 30,000 psi)				"Fair" Subgrade (*1) (Subgrade Modulus = 10,000 psi)			
	Surfacing Layer Thickness				Surfacing Layer Thickness			
	2"	4"	6"	8"	2"	4"	6"	8"
400,000	271	216	153	115	592	375	249	178
300,000	281	242	176	132	657	439	296	214
200,000	286	277	207	158	743	540	372	275
100,000	264	322	256	196	844	726	527	397
50,000	210	339	285	221	844	895	687	524

Modulus of Surfacing Mix	"Poor" Subgrade (*1) (Subgrade Modulus = 3,000 psi)				"Poor" Subgrade (*2) (Subgrade Modulus = 3,000 psi)			
	Surfacing Layer Thickness				Surfacing Layer Thickness			
	2"	4"	6"	8"	2"	4"	6"	8"
400,000	1117	589	366	247	1095	574	357	242
300,000	1300	712	450	308	1277	695	438	301
200,000	1585	923	595	415	1561	901	581	405
100,000	2111	1384	927	667	2092	1357	906	652
50,000	2589	1960	1367	1016	2583	1933	1341	995

(*1) Base Modulus = 1.5 (Subgrade Modulus), Base Thickness = 12".
(*2) Base Modulus = 1.5 (Subgrade Modulus), Base Thickness = 24".
Based on a constant Poisson's Ratio of 0.35 and two 4.5 kip Circular Loads with a Contact Pressure of 85 psi, 12.32 inches apart.
1 psi = 6.894 kN/m^2.
1 inch = 25.4 mm

Table 5.2. Maximum Compressive Strain at the Top of the Subgrade ($x10^{-6}$ in/in) (After reference 21).

Modulus of Surfacing Mix	"Good" Subgrade (*1) (Subgrade Modulus = 30,000 psi) Surfacing Layer Thickness				"Fair" Subgrade (*1) (Subgrade Modulus = 10,000 psi) Surfacing Layer Thickness			
	2"	4"	6"	8"	2"	4"	6"	8"
400,000	356	271	204	157	996	658	448	322
300,000	365	283	217	171	1022	702	491	360
200,000	373	297	235	189	1052	761	552	415
100,000	386	318	263	218	1096	848	652	512
50,000	397	337	287	245	1134	919	741	606

Modulus of Surfacing Mix	"Poor" Subgrade (*1) (Subgrade Modulus = 3,000 psi) Surfacing Layer Thickness				"Poor" Subgrade (*2) (Subgrade Modulus = 3,000 psi) Surfacing Layer Thickness			
	2"	4"	6"	8"	2"	4"	6"	8"
400,000	2817	1534	930	609	1292	868	591	415
300,000	2959	1691	1055	707	1326	925	650	468
200,000	3136	1915	1242	860	1367	1001	731	544
100,000	3375	2287	1584	1152	1423	1115	862	675
50,000	3549	2618	1931	1470	1470	1207	979	799

(*1) Base Modulus = 1.5 (Subgrade Modulus), Base Thickness = 12".
(*2) Base Modulus = 1.5 (Subgrade Modulus), Base Thickness = 24".
Based on a constant Poisson's Ratio of 0.35 and two 4.5 kip Circular Loads with a Contact Pressure of 85 psi, 12.32 inches apart.
1 psi = 6.894 kN/m^2.
1 inch = 25.4 mm.

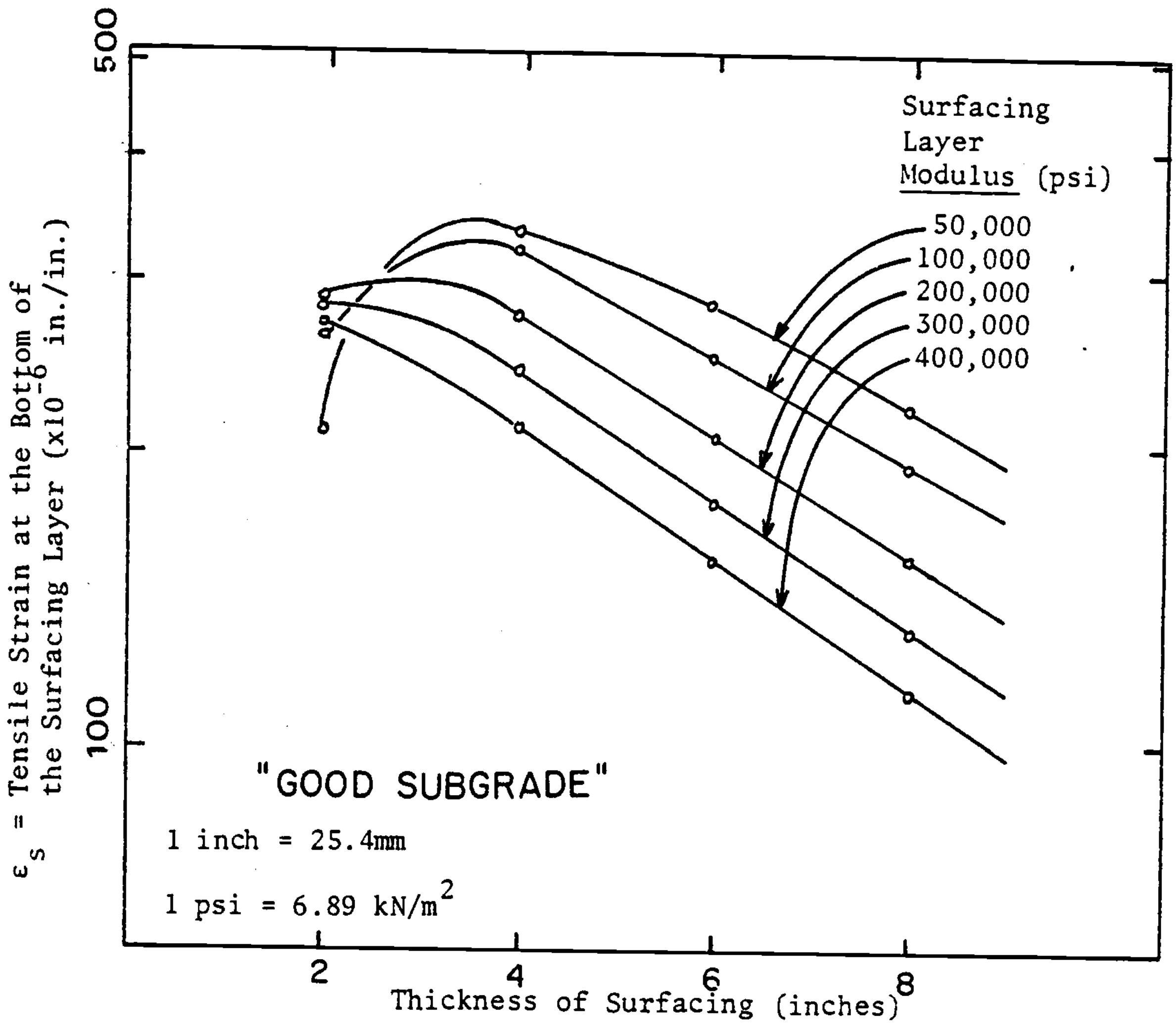


Figure 5.5 Variation of Tensile Strain at the Bottom of the Surfacing Layer With Thickness of the Surfacing layer, assuming: (after 21)

a) Subgrade Modulus = 30,000 psi
   Base Modulus = 45,000 psi
   Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
   Contact Pressure = 85 psi
   Load Spacing = 12.32 inches

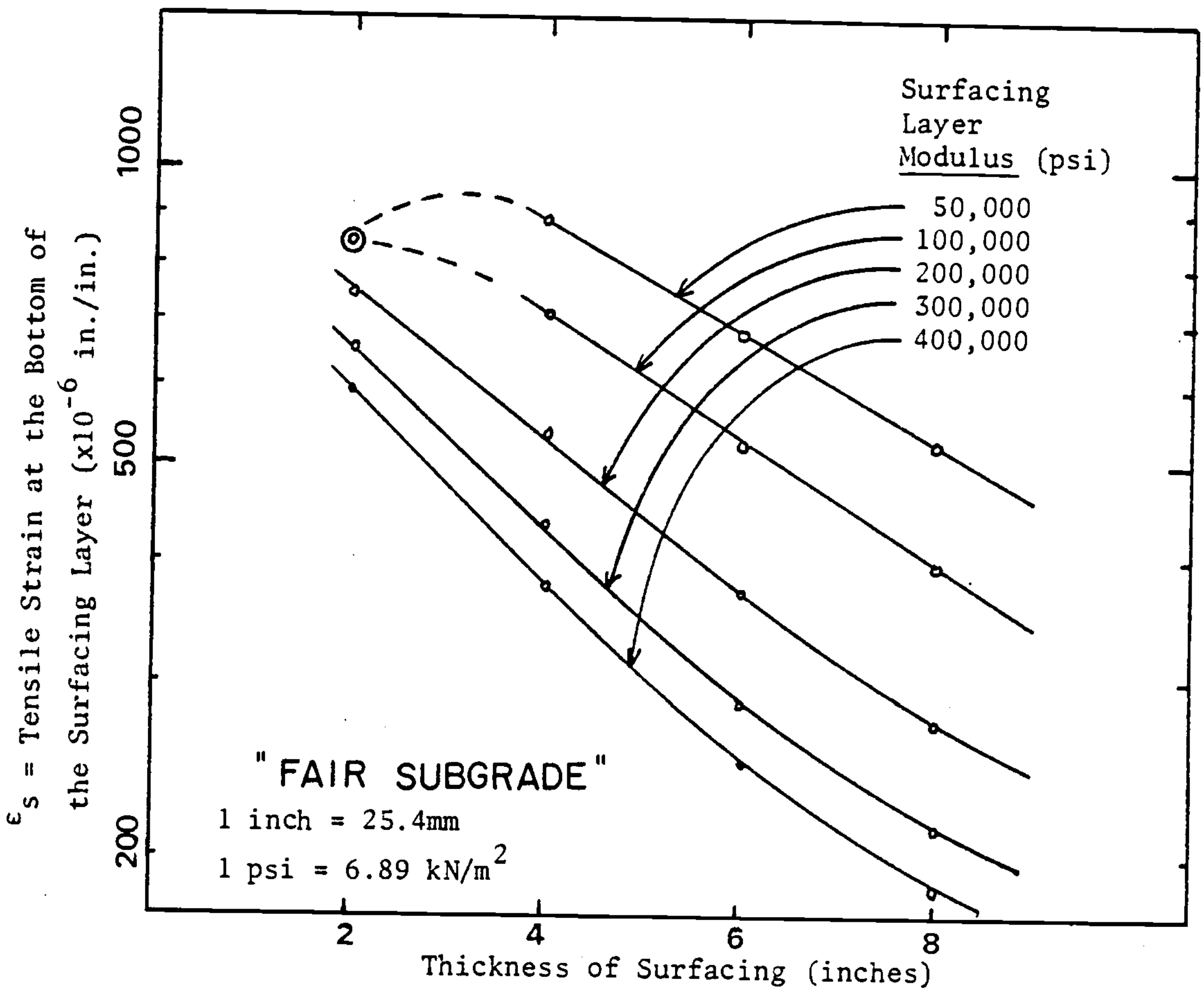


Figure 5.6 Variation of Tensile Strain at the Bottom of the Surfacing Layer With Thickness of the Surfacing Layer, Assuming: (after 21)

a) Subgrade Modulus = 10,000 psi
Base Modulus = 15,000 psi
Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
Contact Pressure = 85 psi
Load Spacing = 12.32 inches

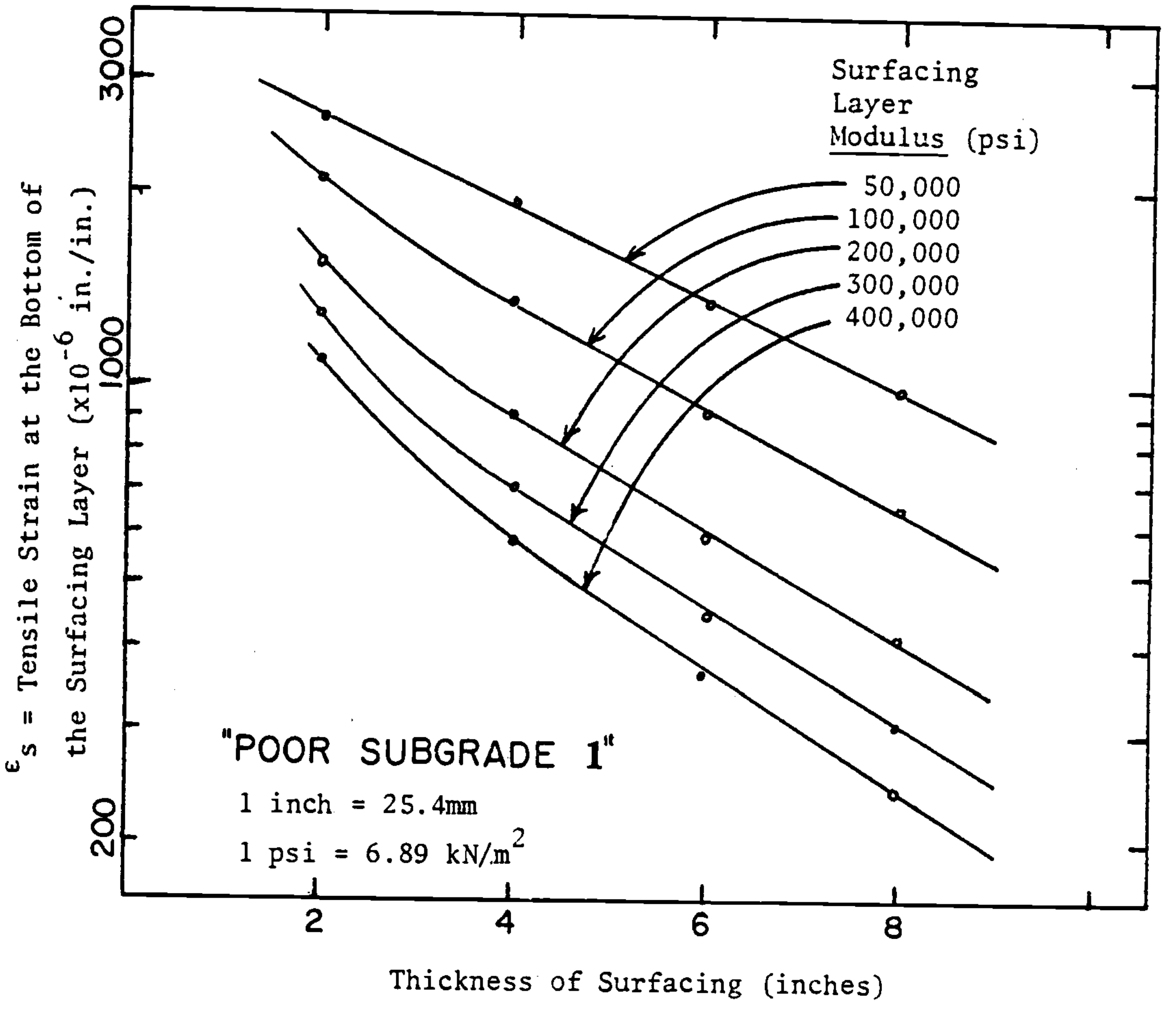


Figure 5.7 Variation of Tensile Strain at the Bottom of the Sufacing Layer With Thickness of the Surfacing Layer, Assuming: (after 21)

a) Subgrade Modulus = 3,000 psi
Base Modulus = 4,500 psi
Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
Contact Pressure = 85 psi
Load Spacing = 12.32 inches

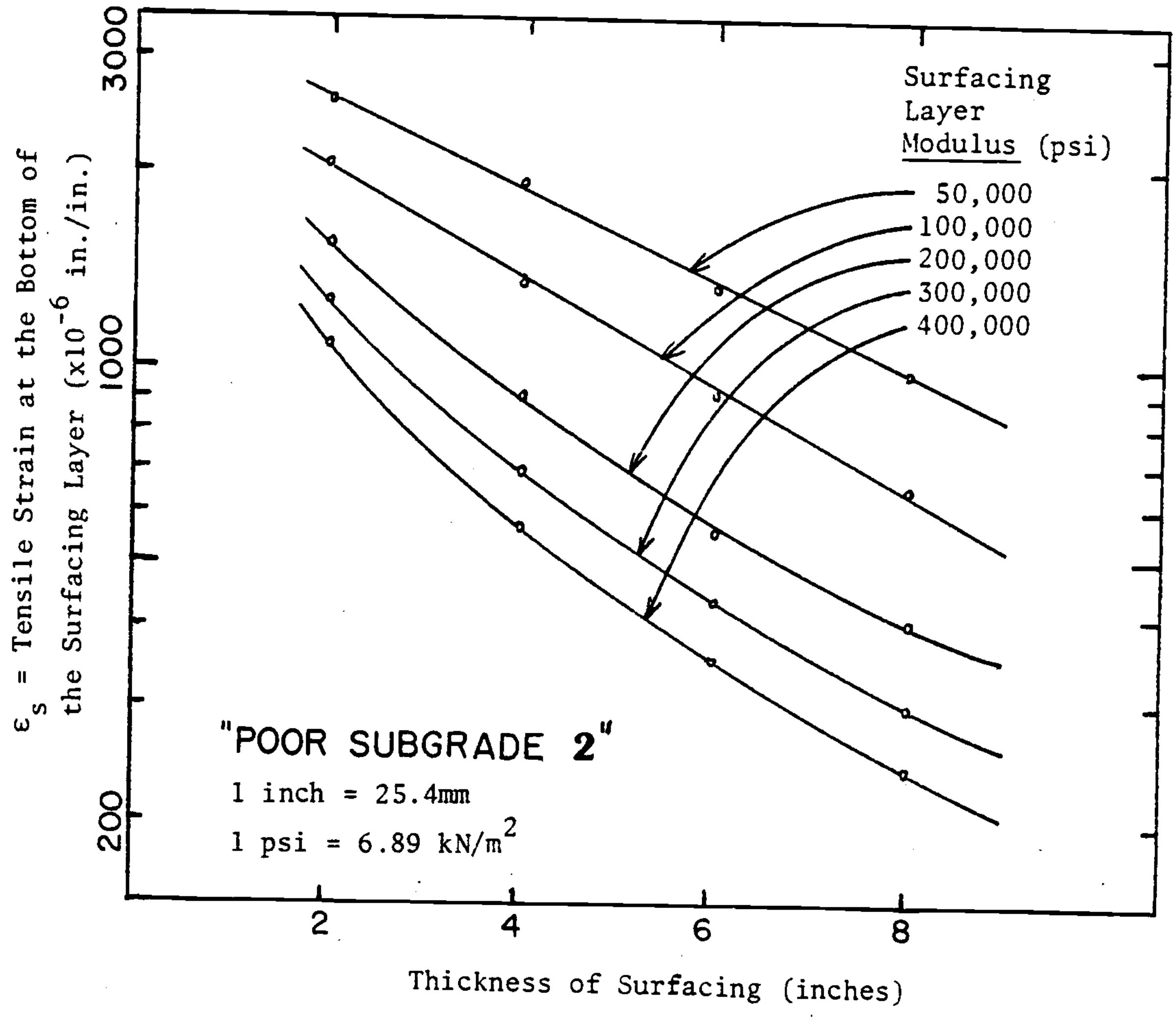


Figure 5.8 Variation of Tensile Strain at the Bottom of the Surfacing Layer With Thickness of the Surfacing Layer, Assuming: (after 21)

a) Subgrade Modulus = 3,000 psi
Base Modulus = 4,500 psi
Base Thickness = 24 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
Contact Pressure = 85 psi
Load Spacing = 12.32 inches

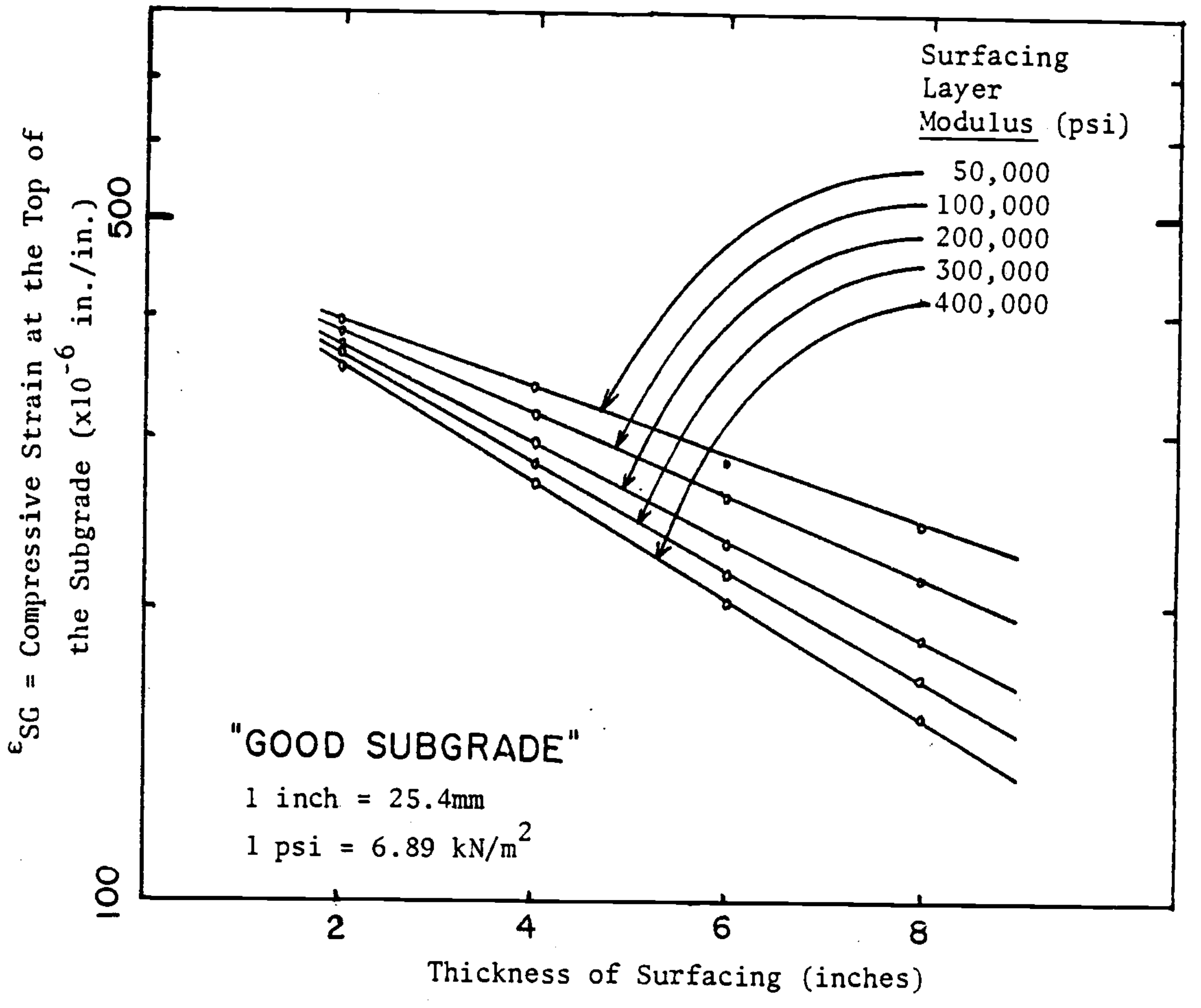


Figure 5.9 Variation of Compressive Strain at the Top of the Subgrade with Thickness of the Surfacing Layer, Assuming: (after 21).

a) Subgrade Modulus = 30,000 psi
   Base Modulus = 45,000 psi
   Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
   Contact Pressure = 85 psi
   Load Spacing = 12.32 inches

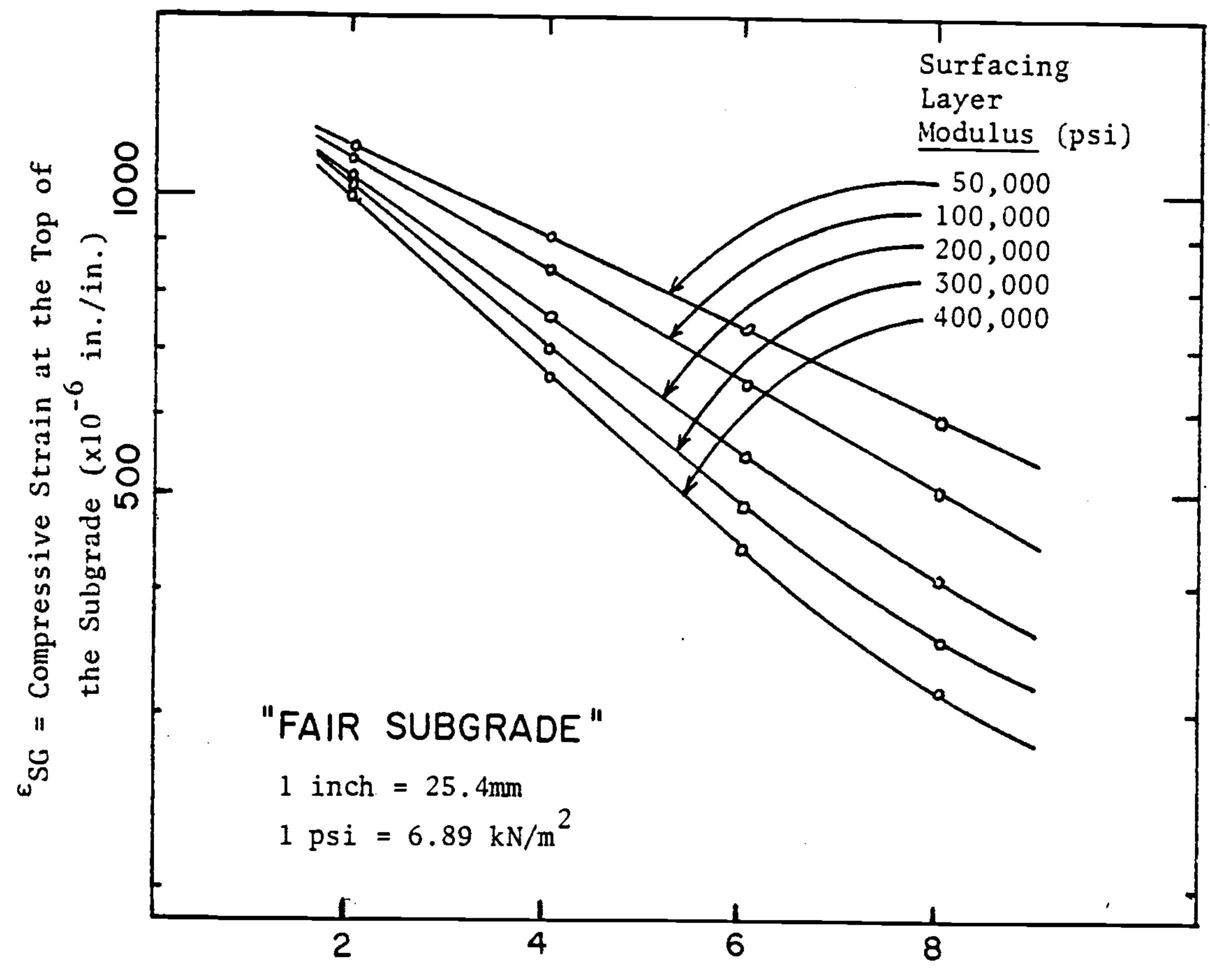


Figure 5.10 Variation of Compressive Strain at the Top of the Subgrade with Thickness of the Surfacing Layer, Assuming: (after 21)

a) Subgrade Modulus = 10,000 psi
Base Modulus = 15,000 psi
Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
Contact Pressure = 85 psi
Load Spacing = 12.32 inches

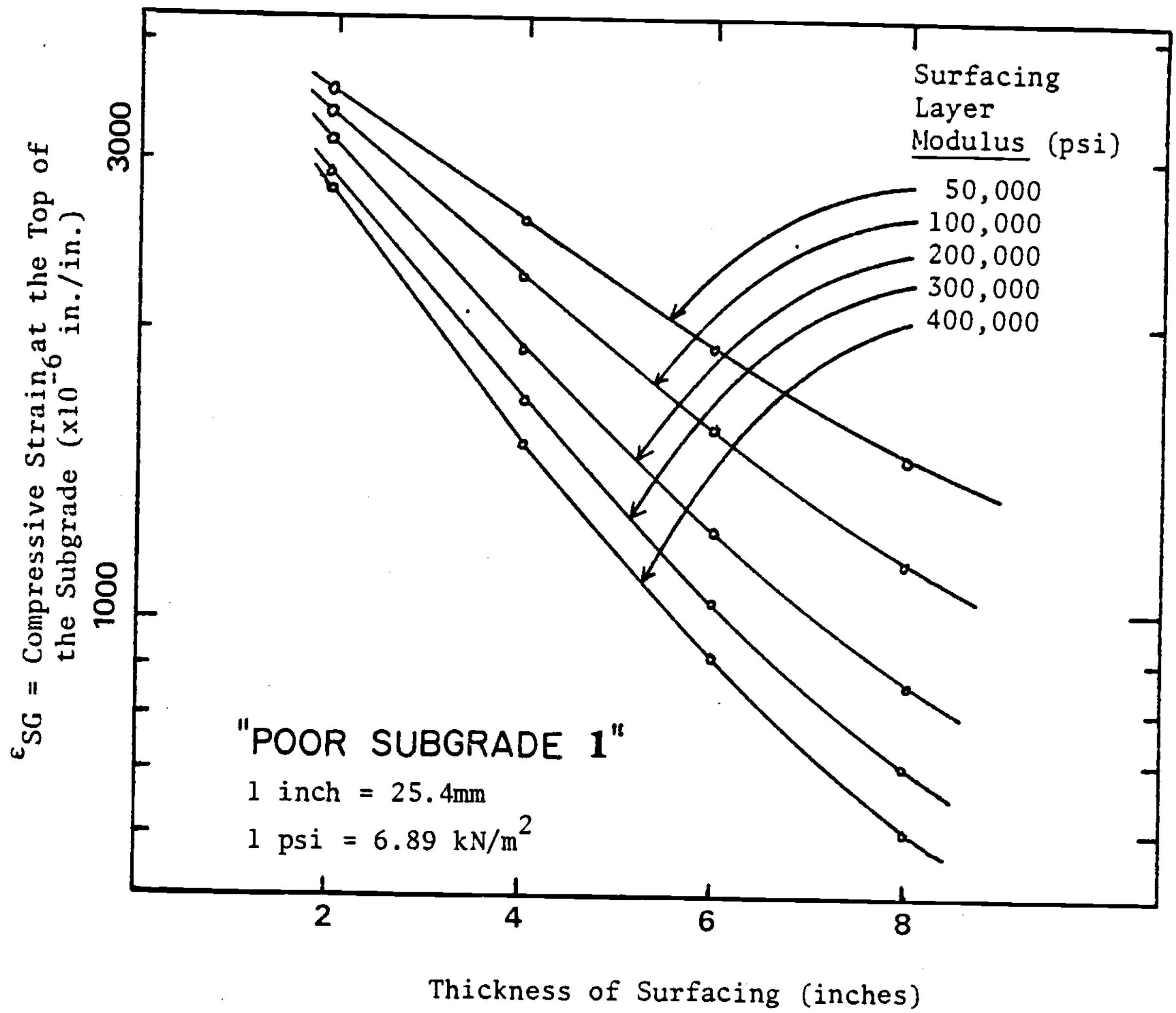


Figure 5.11 Variation of Compressive Strain at the Top of the Subgrade with Thickness of the Surfacing Layer, Assuming:

a) Subgrade Modulus = 3,000 psi
Base Modulus = 4,500 psi
Base Thickness = 12 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
Contact Pressure = 85 psi
Load Spacing = 12.32 inches

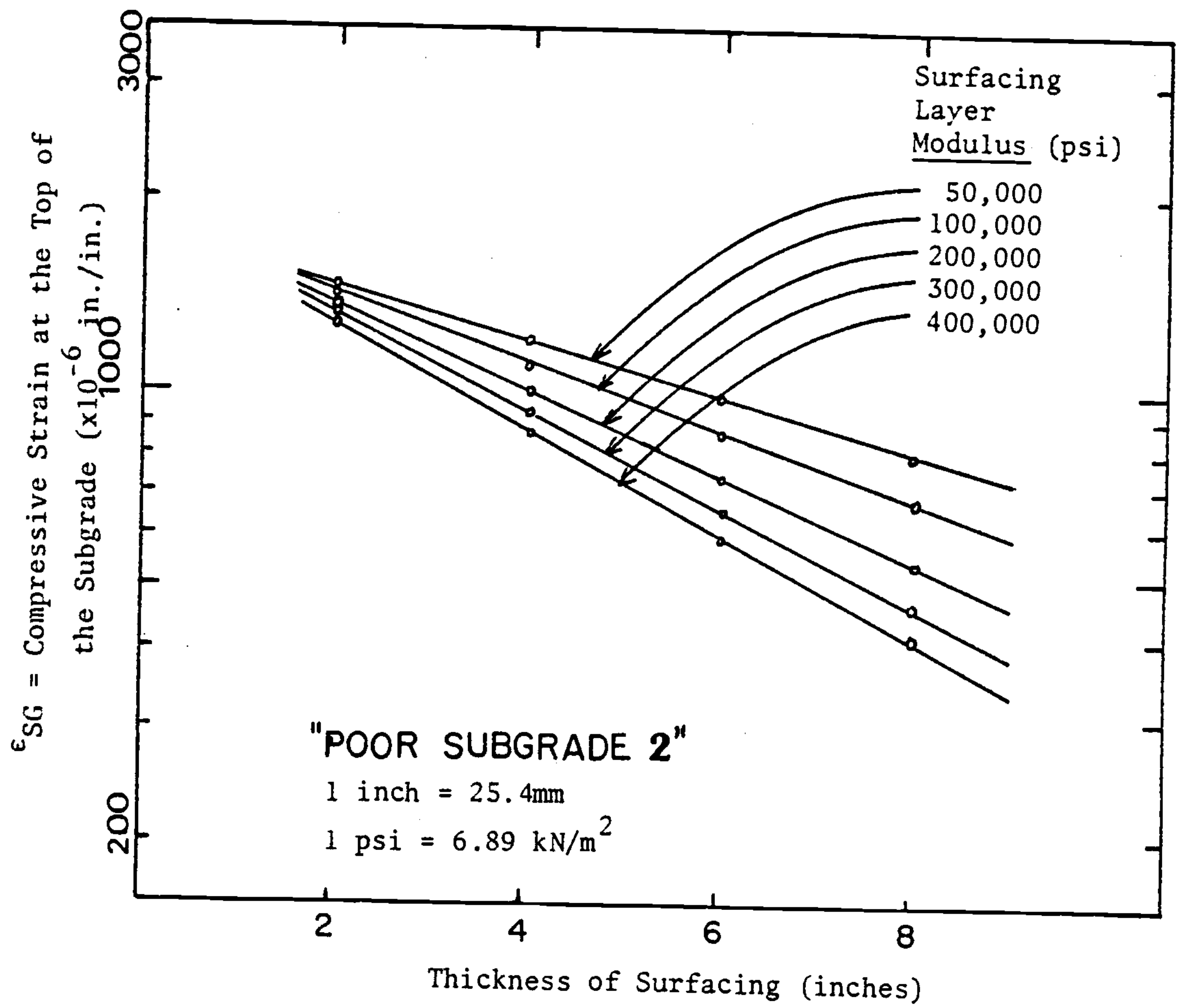


Figure 5.12 Variation of Compressive Strain at the Top of the Subgrade with Thickness of the Surfacing Layer, Assuming:

a) Subgrade Modulus = 3,000 psi
   Base Modulus = 4,500 psi
   Base Thickness = 24 inches

b) Poisson's Ratio = 0.35

c) Two 4.5 KIP Circular Loads
   Contact Pressure = 85 psi
   Load Spacing = 12.32 inches

mix 2068 x 10^3 kN/m^2 (300,000 psi), and the marginal emulsified asphalt mix 1379 x 10^3 kN/m^2 (200,000 psi) for the development of layer equivalencies. This development is followed by a discussion investigating the effect on layer equivalencies of varying both of the emulsion mix modulus values.

The layer equivalencies determined from Santucci's fatigue criteria are shown in Tables 5.3 and 5.4. The thickness design and layer equivalencies determined from rutting criteria are given in Table 5.5. From examination of these tables, it can be seen that the design thickness is controlled by fatigue criteria in all cases except for the "Poor Subgrade 1" condition, where a greater thickness is required to preclude rutting in the pavement. In Tables 5.3 and 5.5 no general trends are evident in the layer equivalencies obtained. As seen from these tables, low variances are obtained from the average layer equivalency values of 1.27 and 1.24 for the open graded emulsion mixes, and 1.09 for the rutting criteria values obtained for the cement-modified mix. However, in Table 5.4, it is evident that the cement-modified layer equivalencies based on fatigue criteria are highly dependent upon the level of traffic. Significantly greater relative thicknesses are required at lower traffic repetitions, this difference diminishing with increasing traffic level. This difference arises because of the significantly lower tensile strains allowed for the cement-modified mixes in comparison with the hot mix. To provide the most conservative design thicknesses based upon the results of this analysis, the layer equivalencies given in Table 5.6 are recommended. From these values, layer coefficients for other design procedures are developed in the following section.

An important consideration in the use of the layer equivalencies

Table 5.3. Thickness Design Based on Fatigue Criteria, Open Graded Emulsion Mixes (E. Mix) (after reference 21).

Design Load Applications, 18 kip EAL		10^4	5×10^4	10^5	5×10^5	10^6
E. Mix Allowable Tensile Strain, 10^{-6} (Figure 5.1)		900	570	470	295	245
Hot Mix Allowable Tensile Strain, 10^{-6} (Figure 5.1)*1		720	480	400	260	215
Design for "Good Subgrade" (Figure 5.5)	E. Mix Thickness	---	---	---	---	4.9"
	Hot Mix Thickness	---	---	---	---	4.0"
	Layer Equivalency*2	---	---	---	---	1.23
Design for "Fair Subgrade" (Figure 5.6)	E. Mix Thickness	---	3.6"	4.7"	7.5"	8.6"
	Hot Mix Thickness	---	2.8"	3.7"	5.8"	6.8"
	Layer Equivalency	---	1.29	1.27	1.29	1.26
Design for "Poor Subgrade 1" (Figure 5.7)	E. Mix Thickness	4.1"	6.2"	7.3"	9.8"	---
	Hot Mix Thickness	3.3"	4.9"	5.7"	7.7"	---
	Layer Equivalency	1.24	1.27	1.28	1.27	---
Design for "Poor Subgrade 2" (Figure 5.8)	E. Mix Thickness	4.0"	6.1"	7.1"	9.7"	---
	Hot Mix Thickness	3.2"	4.8"	5.5"	7.6"	---
	Layer Equivalency	1.25	1.27	1.29	1.28	---

Average E. Mix Layer Equivalency = 1.27, σ = .02, variance = 1.5%.
*1 = Assumes Hot Mix Resilient Modulus = 400,000 psi, E. Mix Resilient Modulus = 200,000 psi.
*2 = Layer Equivalency = Emulsified Asphalt Mix Thickness/Hot Mix Thickness.
1 psi = 6.894 kN/m^2; 1 inch = 25.4 mm.

Table 5.4. Thickness Design Based on Fatigue Criteria, Cement-modified Mixes (CMM)

Design Load Applications, 18 kip EAL		10^4	5×10^4	10^5	5×10^5	10^6
Cement-Modified Mix Allowable Tensile Strain, 10^{-6} (Figure 5.2)		460	365	320	240	210
Hot Mix Allowable Tensile Strain, 10^{-6} (Figure 5.1)*1		720	480	400	260	215
Design for "Good Subgrade" (Figure 5.5)	CMM Thickness	---	---	---	---	4.7"
	Hot Mix Thickness	---	---	---	---	4.0"
	Layer Equivalency*2	---	---	---	---	1.18
Design for "Fair Subgrade" (Figure 5.6)	CMM Thickness	---	4.6"	5.4"	7.0"	8.0"
	Hot Mix Thickness	---	2.8"	3.7"	5.8"	6.8"
	Layer Equivalency	---	1.64	1.46	1.21	1.18
Design for "Poor Subgrade 1" (Figure 5.7)	CMM Thickness	6.1"	7.1"	7.9"	9.2"	---
	Hot Mix Thickness	3.3"	4.9"	5.7"	7.7"	---
	Layer Equivalency	1.85	1.45	1.39	1.19	---
Design for "Poor Subgrade 2" (Figure 5.8)	CMM Thickness	5.7"	6.9"	7.6"	---	---
	Hot Mix Thickness	3.2"	4.8"	5.5"	---	---
	Layer Equivalency	1.78	1.44	1.38	---	---
Average CMM Layer Equivalency		1.82	1.51	1.41	1.20	1.18

Total Average CMM Layer Equivalency = 1.43, σ = .23, variance = 16%.
*1 - Assumes Hot Mix Resilient Modulus = 400,000 psi, CMM Resilient Modulus = 300,000 psi.
*2 - Layer Equivalency = CMM Thickness/Hot Mix Thickness.
1 psi = 6.894 kN/m^2
1 inch = 25.4 mm

Table 5.5. Thickness Design Based on Rutting Criteria, Open Graded Emulsion Mixes and Cement-modified Mixes.

Design Load Applications, 18 kip EAL		10^4	5×10^4	10^5	5×10^5	10^6
Allowable Subgrade Strain, 10^{-6} (Figure 5.3)		1020	920	800	570	490
Design for "Good Subgrade"						
(Figure 5.9)	E. Mix Thickness	---	---	---	---	---
	CMM Thickness	---	---	---	---	---
	Hot Mix Thickness	---	---	---	---	---
	Layer Eq. E. Mix	---	---	---	---	---
	Layer Eq. CMM	---	---	---	---	---
Design for "Fair Subgrade"						
(Figure 5.10)	E. Mix Thickness	---	2.8"	3.7"	5.8"	6.7"
	CMM Thickness	---	2.6"	3.4"	5.2"	6.0"
	Hot Mix Thickness	---	2.4"	3.1"	4.8"	5.5"
	Layer Eq. E. Mix	---	1.17	1.19	1.21	1.22
	Layer Eq. CMM	---	1.08	1.10	1.08	1.09
Design for "Poor Subgrade 1"						
(Figure 5.11)	E. Mix Thickness	6.7"	7.7"	8.6"	---	---
	CMM Thickness	5.8"	6.6"	7.3"	---	---
	Hot Mix Thickness	5.3"	6.0"	6.6"	---	---
	Layer Eq. E. Mix	1.26	1.28	1.30	---	---
	Layer Eq. CMM	1.09	1.10	1.10	---	---
Design for "Poor Subgrade 2"						
(Figure 5.12)	E. Mix Thickness	2.9"	4.7"	5.4"	7.6"	8.6"
	CMM Thickness	2.5"	4.0"	4.8"	6.7"	7.6"
	Hot Mix Thickness	2.3"	3.7"	4.4"	6.3"	7.0"
	Layer Eq. E. Mix	1.26	1.27	1.23	1.21	1.23
	Layer Eq. CMM	1.09	1.08	1.09	1.06	1.09

Average E. Mix Layer Equivalency = 1.24, σ = 0.4, variance = 3.2%.
Average CMM Layer Equivalency = 1.09, σ = .01, variance = 0.9%.

1 inch = 25.4 mm

Table 5.6. Recommended Layer Equivalencies for Hot Mix Thicknesses.

Design Load Applications, 18 kip EAL	Open Graded Emulsion Mix	Cement-Modified Mix
10,000	1.27	1.82
50,000	1.27	1.51
100,000	1.27	1.41
500,000	1.27	1.20
1,000,000	1.27	1.18

are their variation with the resilient modulus of the mix. Standard values of 1379×10^3 kN/m^2, 2068×10^3 kN/m^2, and 2758×10^3 kN/m^2 (200,000 psi, 300,000 psi, and 400,000 psi) have been assumed in this study, however actual values may conceivably range from less than 345×10^3 kN/m^2 (50,000 psi) to greater than 6894×10^3 kN/m^2 (1,000,000 psi) for the pavement mixes under study. Hatch (21) has calculated layer equivalencies for the variation in surfacing modulus of open graded emulsion mixes in the standard pavement sections given on Figure 5.4. These values are plotted on Figure 5.13 according to the fatigue criteria model and the rutting criteria model. As the same allowable subgrade strain is applicable to both open graded emulsion and cement-modified emulsion mixes, the rutting criteria layer equivalencies apply to both mixes. Table 5.7 lists cement-modified emulsion mix layer equivalencies as they vary with modulus, subgrade condition, and design life. As seen from these curves, the open graded emulsion layer equivalency depends significantly upon the subgrade condition and the modulus of the mix, especially at low modulus and poor subgrade conditions. From Table 5.7, a similar variation with modulus exists for the cement-modified material, however the difference resulting from changing subgrades is less pronounced, and the layer equivalencies are not consistently lower with better quality subgrade conditions. Because of this fact, the level of traffic and the modulus of the mix appear to be the most significant factors affecting the magnitude of these layer equivalencies.

5.4 Recommended Structural Layer Coefficients

In the AASHTO design procedure (50), a weighted structural number is used in the following design equation:

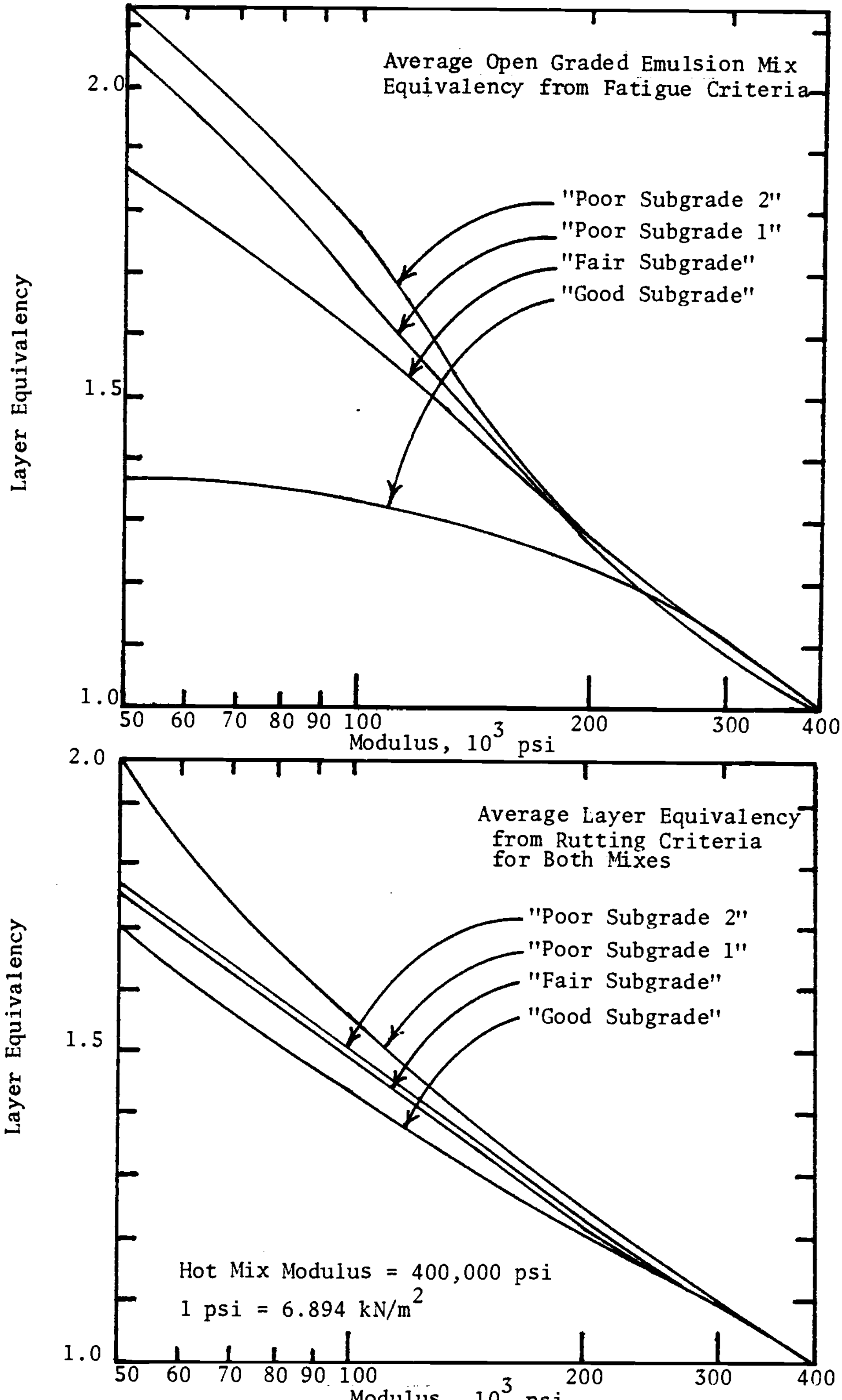


Figure 5.13 Variation of Layer Equivalency with Pavement Modulus

Table 5.7. Variation of Cement-modified Mix Layer Equivalencies with Modulus, Subgrade, and Design Life from Fatigue Criteria.

Design Life, EAL	Pavement Subgrade	Modulus, psi		
		200,000	300,000	400,000
10^4	"Good"	-	-	-
	"Fair"	-	-	-
	"Poor 1"	2.09	1.85	1.55
	"Poor 2"	2.03	1.78	1.45
	Average	2.06	1.82	1.50
5×10^4	"Good"	-	-	-
	"Fair"	2.11	1.64	1.52
	"Poor 1"	1.71	1.45	1.27
	"Poor 2"	1.75	1.44	1.23
	Average	1.86	1.51	1.34
10^5	"Good"	-	-	-
	"Fair"	1.76	1.46	1.32
	"Poor 1"	-	1.39	1.19
	"Poor 2"	1.69	1.38	1.18
	Average	1.73	1.41	1.23
5×10^5	"Good"	-	-	-
	"Fair"	1.52	1.21	1.12
	"Poor 1"	-	1.19	1.05
	"Poor 2"	-	-	1.07
	Average	1.52	1.20	1.08
10^6	"Good"	1.38	1.18	1.03
	"Fair"	1.43	1.18	1.06
	"Poor 1"	-	-	-
	"Poor 2"	-	-	-
	Average	1.41	1.18	1.05

$$SN = \sum_{i=1}^{n} a_i D_i$$

where SN = weighted structural number,

a_i = layer coefficient of the i-th layer, and

D_i = depth of the i-th layer.

The pavements structure is deemed adequate when the summation of coefficients times depths is greater than or equal to the appropriate weighted structural number. Use of this equation is illustrated in Figure 5.14.

Layer coefficients vary depending upon the material type and the design traffic count. Coefficients established for hot mix bituminous pavements are given in Table 5.8. In order to develop layer coefficients for utilizaton of the marginal aggregates, the hot mix coefficients are adjusted by comparing equivalent structural sections of the two materials as follows:

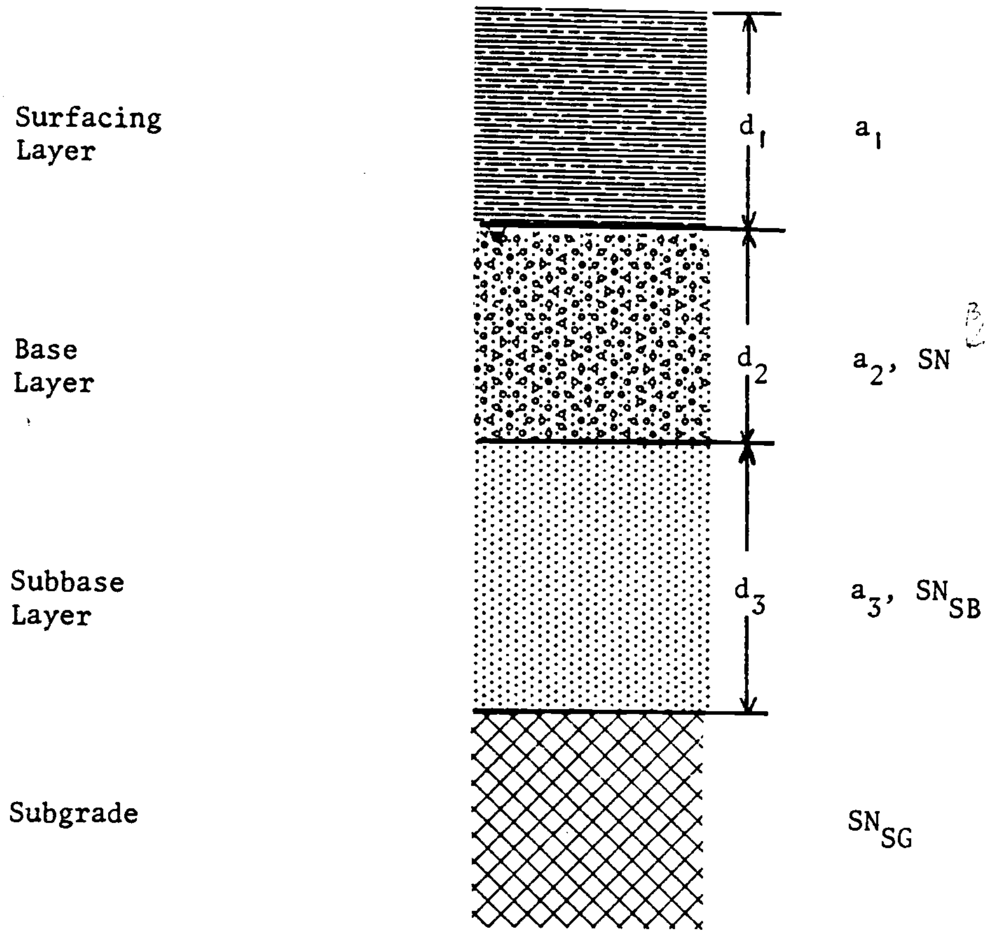
$$D_{\text{Emulsion Mix}} (a_{\text{Emulsion Mix}}) = D_{\text{Hot Mix}} (a_{\text{Hot Mix}})$$

where a = layer coefficient, and

D = design thickness.

From this equality, and the fact that the layer equivalencies previously developed are simply the ratio of emulsion mix design thickness over hot mix design thickness, the following equation is used to adjust hot mix layer coefficients:

$$a_{\text{Emulsion Mix}} = \frac{a_{\text{Hot Mix}}}{\text{Layer Equivalency}}$$



Design Equations -

$$SN_B \leqslant a_1 d_1$$

$$SN_{SB} \leqslant a_1 d_1 + a_2 d_2$$

$$SN_{SG} \leqslant a_1 d_1 + a_2 d_2 + a_3 d_3$$

Figure 5.14 Illustration of the AASHO Design Equations (after 21)

Table 5.8. Layer Coefficients for Hot Mix Bituminous Pavements (a_1) (after Reference 52).

Layer Coefficient (a_1)	Total 18-kip Equivalent Axles
0.42	<10,000
0.40	10,000 - 60,000
0.38	60,000 - 120,000
0.36	120,000 - 350,000
0.34	350,000 - 1,000,000
0.32	1,000,000 - 3,000,000
0.30	>3,000,000

This adjustment is given in Table 5.9 for the hot mix coefficients given in Table 5.8, using the marginal aggregate mix layer equivalencies determined in Table 5.6.

Table 5.10 lists typical layer equivalencies that have been specified by different agencies. These are listed as they would apply to the study aggregates. In comparing the open graded emulsion mix layer equivalencies of this study with those specified by the Forest Service, the Forest Service values are considerably higher. As the study values are modified from the same hot mix coefficients as specified by the Forest Service, the Forest Service coefficients appear a bit conservative unless low modulus values are being obtained from the mixes (note Figure 5.13). In comparing the cement-modified mix layer equivalencies, several interesting differences can be noted. For one, the relationships between increasing relative thicknesses and design traffic life is opposite for the two sets of values. The Forest Service values indicate that higher relative thicknesses of the cement-modified mixtures are required for higher traffic loads, while this analysis indicates that at higher traffic loads, the cement-modified mix becomes more equivalent to a hot mix. In this study, this relationship is attributable to the fatigue criteria used. In this fatigue model (Figure 5.2), a much lower critical strain is allowed at the lower traffic loads than for hot mix, with this difference diminishing at higher traffic loads. Another difference is noted in considering the layer equivalency magnitudes. The Forest Service values indicate more conservative relative thicknesses are required at traffic loads of greater than 10,000 EAL in comparison with the study values obtained. For traffic loads less than 10,000 EAL, a smaller relative thickness is required. More information relating to

Table 5.9. Recommended Layer Coefficients for Marginal Emulsified Asphalt Mixes.

Total 18-kip Equivalent Axles	Open Graded Emulsion Mix Layer Coefficient	Cement-Modified Emulsion Mix Layer Coefficient
<10,000	.33	.23
10,000 - 60,000	.31	.26
60,000 - 120,000	.30	.27
120,000- 350,000	.28	.28
350,000 - 1,000,000	.27	.28
1,000,000 - 3,000,000	.25	.27
>3,000,000	.24	.27

Table 5.10. Typical Layer Equivalencies.

	EAL, 18 kip	Hot Mix Coefficient	Open Graded Emulsion Coeff. (Quality)	Open Graded Emulsion Coeff. (Marginal)	Cement-Mod. Emulsion Coeff. (Sand Agg.)	OGE Mix Layer Eq. (Quality)	OGE Mix Layer Eq. (Marginal)	CMM Layer Eq. (Sand)
USFS (52)	18 kip Axles							
	<10,000	.42	.27	.25	.28	1.56	1.68	1.50
	10 - 60,000	.40	.25	.23	.26	1.60	1.74	1.54
	60 - 120,000	.38	.23	.21	.24	1.65	1.81	1.58
	120 - 350,000	.36	.21	.19	.22	1.71	1.89	1.64
FHWA (51)		.28	.28		.21	1.00		1.33
Oregon DOT (24)		-	-		-	1.11		1.11
Washington Highway Department (21)		-	-		-	1.10		-

field performance would be needed to substantiate these values.

The Federal Highway Administration (51) specifies equivalent thicknesses of hot mix and open graded emulsion, however, the hot mix coefficient specified is much lower than that used by the Forest Service. This results in larger pavement thicknesses using the FHWA method given the same design conditions. The cement-modified emulsion mix equivalencies from this study range from 1.18 to 1.82, with an average value of 1.42. These values are dependent upon the level of traffic, however they are predominantly higher than the single value of 1.33 specified by the FHWA.

6.0 CONCLUSIONS AND RECOMMENDATIONS

This report summarizes the findings of a study to develop methods of supplying construction aggregates to coastal Oregon areas. This has been done with a primary interest in beneficiating abundant and locally available marginal aggregates, these being marine basalts, sandstones, dune sands, and dredged spoils which have historically not been used, or used with limited success.

The results of Chapter 2 indicate that a number of methods are available for upgrading the properties of the study aggregates for use in pavement construction. For admixture stabilization, the use of asphalt, asphalt emulsions, portland cement, and lime in mixes with the study materials have been shown to perform successfully in a number of studies and test projects. The pretreatment of aggregates, particularly marine basalts with hydrated lime, has been proven in laboratory tests to significantly improve durability properties. The blending of high quality aggregates with poor quality materials, although seldom practiced, offers an excellent method of extending the high quality materials available in these areas along with those presently being imported. Finally, the use of fabrics in roadway construction with the study aggregates offers great potential in providing increased subgrade and base stabilization and in reducing aggregate depth requirements.

The experimental program discussed in Chapter 3 provides a valid system to evaluate stiffness, strength, fatigue and durability properties of quality aggregate, marine basalt, and cement-modified dune sand emulsion mixes, the results of which are given in Chapter 4. Similar experimental programs have been used in other studies of this type.

The test results and discussion given in Chapter 4 substantiate the idea that the marginal aggregates can be treated with emulsions to provide high quality pavements. The marine basalts are recommended for construction in open graded mixes treated with CMS-2 emulsion, while the dune sand should be treated with small amounts of portland cement and CSS-1h emulsion. The stiffness of these mixtures after moisture exposure does not fall below conventional modular values, and the fatigue life does not appear to be affected by such conditioning. Although little information is available for comparing the tensile strengths obtained from these materials with results found by others, the results appear to be at an acceptable level and do not indicate a significant loss from moisture conditioning. Again, little or no information is available for comparing diametral fatigue results of open graded emulsion ixes. In comparing the results found here with results from other test methods and materials, the diametral test does not appear to be acceptable for open graded mixes.

Finally, the layer equivalencies and structural layer coefficients developed in Chapter 5 allow for the use of standard design procedures in utilizing these marginal aggregates with practically any required conditions. These also allow for cost comparisons to be made easily once the prices of the emulsion mix and hot mix are determined for a given situation.

In conclusion, the findings of this study indicate that marginal aggregates can be used to provide quality roadways, resulting in significant energy and construction cost savings. Recommendations for further research include:

1) Testing of open graded emulsion mixes of marine basalt pre-treated with hydrated lime,
2) Testing of emulsion treated sandstone of a higher grade than the intermixed sandstone and siltstone obtained for this study,
3) Development of improved mix design procedures, possibly including diametral modulus testing, to determine optimum emulsion contents, and
4) Testing of the marginal aggregates in wheel track tests or trial pavement test sections to further refine failure criteria for these materials.

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APPENDIX A

Table A-1.1. Summary of Mix Design Characteristics.

Aggregate	Sample	Emulsion Content, %	Added Water, %	Workability	Film Thickness	Coating, %	Excess Fluids
Berry	1	5	3.5	good	thick	80	none
Creek	2	5	4.5	good	thin	75	none
(CMS-2	3	6	1.0	good	med-thick	85	none
emulsion)	4	6	2.0	good	medium	95	none
	5	6	3.5	good	medium	90	none
	6	6	4.5	good	medium	90	none
	7	7	3.5	good	medium	95	none
	8	7	1.0	good	med-thick	90	none
	9	7	2.0	good	medium	98	quite a bit
Eckman	1	5	0	good	medium	75	none
Creek	2	5	1	good	med-thick	95	none
(CMS-2	3	5	2	good	medium	90	none
emulsion)	4	6	0	good	med-thick	80	none
	5	6	1	good	thick	70	none
	6	6	2	good	med-thick	95	none
	7	7	0	good	med-thick	80	none
	8	7	1	good	thick	90	none
	9	7	2	good	med-thick	95	yes
Oceanlake	1	4.5	0	good	thick	75	none
(CMS-2	2	4.5	1	good	medium	99	slight
emulsion)	3	4.5	2	good	thin-med.	100	considerable
	4	5.0	0	good	med-thick	90	none
	5	5.0	1	good	medium	100	slight
	6	5.0	2	good	med-thin	100	considerable
	7	5.5	0	good	thick	98	none
	8	5.5	1	good	med-thick	99	slight
	9	5.5	2	good	medium	100	considerable

Table A-1.1. Summary of Mix Design Characteristics (continued).

Aggregate	Sample	Emulsion Content, %	Added Water, %	Workability	Film Thickness	Coating, %	Excess Fluids
Big A Sandstone (CMS-2 emulsion)	1	10	12	good	thick, patchy	80	none
	2	10	14	good	medium	80	none
	3	10	16	fair	varies, patchy	85	yes
	4	11	12	good	varies, patchy	85	none
	5	11	14	good	medium	90	slight
	6	12	12	good-fair	medium	90	none
	7	9	12	fair	varies, patchy	75	none
	8	12	14	good	med-thick	98	slight
	9	14	10	good	medium	98	none
Dune Sand (CSS-1 emulsion)	1	5	10	good	medium	70	none
	2	5	12	fair	medium	80	none
	3	5	14	fair	medium	85	none
	4	6	10	fair	medium	80	none
	5	6	12	fair	medium	90	none
	6	6	14	fair	medium	90	slight
	7	8	12	good	med-thick	100	none
	8	7	12	fair	medium	95	none
	9	7	14	fair	medium	100	slight
Dune Sand (CSS-1 emulsion and 1.5% portland cement)	1	8	9	good	med-thick	98	none
	2	8	11	good	medium	95	none
	3	7	12	fair	medium	95	none
	4	7	14	fair	medium	98	slight
	5	7	9	fair-good	med-thick	98	none
Criteria for All Mixes	-	-	-	fair-good	med-thick	90-100	slight-none

APPENDIX B

Table B-1.1. Mix Properties.

Aggregate	Sample	Height, in.	Density, pcf	Emulsion Content, %	Water Content, %
Berry Creek	1	2.40	124.3	6.0	4.0
	2	2.45	122.7	6.0	4.0
	3	2.50	123.2	6.0	4.0
	4	2.56	121.9	6.0	5.5
	5	2.56	120.5	6.0	5.0
	6	2.58	121.4	6.0	4.5
	7	2.50	123.6	6.0	2.0
	8	2.54	122.7	6.0	1.5
	9	2.32	126.8	6.0	1.5
	10	2.46	125.4	6.0	1.0
	11	2.50	124.3	6.0	1.0
	12	2.54	123.1	6.0	1.0
	13	2.56	123.1	6.0	1.0
	14	2.42	126.8	6.0	.5
	15	2.26	122.3	6.0	.5
	16	2.52	122.3	6.0	1.0
	17	2.36	130.4	6.0	1.0
	18	2.48	128.2	6.0	1.0
	19	2.44	128.6	6.0	1.0
	20	2.44	130.2	6.0	1.0
	21	2.40	131.0	6.0	1.0
	22	2.36	120.2	6.0	1.0
	23	2.60	122.8	6.0	1.0
	24	2.05	126.6	6.0	1.0
	25	2.44	117.8	6.0	.5
	26	2.52	129.6	6.0	.5
	27	2.56	126.4	6.0	.5
	28	2.56	125.2	6.0	.5
	29	2.56	123.6	6.0	.5
	30	2.50	129.3	6.0	.5
	31	2.64	125.4	6.0	.5
Average			124.8	6.0	1.71
Standard Deviation			3.31		1.54

Table B-1.2. Mix Properties (cont.)

Aggregate	Sample	Height, in.	Density, pcf	Emulsion Content, %	Water Content, %
Eckman Creek	1	2.25	127.9	6.0	4.0
	2	2.45	130.2	6.0	5.0
	3	2.50	128.5	6.0	5.0
	4	2.30	130.5	6.0	4.0
	5	2.38	131.6	6.0	3.0
	6	2.48	128.5	6.0	3.0
	7	2.36	133.7	6.0	3.0
	8	2.42	133.7	6.0	2.5
	9	2.52	131.8	6.0	2.5
	10	2.52	129.2	6.0	2.5
	11	2.48	131.8	6.0	2.5
	12	2.36	137.7	6.0	2.5
	13	2.44	129.9	6.0	3.0
	14	2.38	129.6	6.0	3.0
	15	2.40	128.1	6.0	3.0
	16	2.32	123.9	6.0	1.5
	17	2.20	131.2	6.0	1.5
	18	2.44	129.1	6.0	1.5
	19	2.44	128.5	6.0	2.0
	20	2.48	127.9	6.0	2.0
	21	2.48	128.8	6.0	2.0
	22	2.52	134.3	6.0	1.5
	23	2.40	133.9	6.0	1.5
	24	2.48	128.2	6.0	1.5
Average			130.4	6	2.7
Standard Deviation			2.9		1.0

Table B-1.3. Mix Properties (cont.)

Aggregate	Sample	Height, in.	Density, pcf	Emulsion Content, %	Water Content, %
Oceanlake	A	2.46	126.9	5.0	.5
	B	2.56	123.5	5.0	.5
	C	2.52	126.2	5.0	.5
	1	2.48	128.5	5.0	1.0
	2	2.50	128.6	5.0	1.0
	3	2.52	126.9	5.0	.5
	4	2.52	127.1	5.0	1.0
	5	2.50	129.1	5.0	1.0
	6	2.62	126.6	5.0	.5
	7	2.52	126.1	5.0	1.0
	8	2.52	125.8	5.0	1.0
	9	2.52	126.1	5.0	1.0
	10	2.48	127.5	5.0	1.0
	11	2.52	126.0	5.0	1.0
	12	2.56	124.8	5.0	1.0
	13	2.56	124.6	5.0	1.0
	14	2.42	124.0	5.0	1.0
	15	2.40	129.8	5.0	1.0
	16	2.36	127.3	5.0	1.0
	17	2.52	125.2	5.0	1.0
	18	2.52	125.3	5.0	1.0
	19	2.46	128.6	5.0	1.0
	20	2.36	123.6	5.0	1.0
	21	2.44	126.3	5.0	1.0
	22	2.36	124.4	5.0	1.5
	23	2.40	128.4	5.0	1.5
	24	2.52	126.6	6.0	2.0
	25	2.56	124.2	5.0	1.5
	26	2.52	126.5	5.0	1.5
	27	2.52	126.9	6.0	2.0
	28	2.46	126.2	5.0	.5
	29	2.56	123.9	5.0	1.0
	30	2.48	125.1	5.0	1.0

Table B-1.3. Mix Properties (cont.)

Aggregate	Sample	Height, in.	Density, pcf	Emulsion Content, %	Water Content, %
Oceanlake (cont.)	31	2.50	123.3	5.0	1.0
	32	2.56	122.9	5.0	1.0
	33	2.56	120.9	5.0	1.0
Average			125.9	5.0	1.1
Standard Deviation			1.95		

Table B-1.4. Mix Properties (cont.)

Aggregate	Sample	Height, in.	Density, pcf	Emulsion Content, %	Water Content, %
Dune Sand	A	2.52	116.4	8.0	9.0
	B	2.50	116.2	8.0	9.0
	C	2.52	115.5	8.0	9.0
	1	2.56	114.5	8.0	9.0
	2	2.52	115.3	8.0	9.0
	3	2.56	114.4	8.0	9.0
	7	2.44	114.3	7.0	8.0
	8	2.32	116.4	7.0	8.0
	9	2.52	111.9	7.0	8.0
	10	2.56	112.3	7.0	6.5
	11	2.48	117.3	7.0	7.0
	12	2.56	113.9	7.0	7.0
	13	2.48	118.2	7.0	7.0
	14	2.48	117.8	7.0	7.0
	15	2.52	115.4	7.0	7.0
	16	2.52	114.4	7.0	6.0
	17	2.32	116.2	7.0	6.5
	18	2.28	114.1	7.0	9.0
	19	2.56	110.7	7.0	6.0
	20	2.48	114.4	7.0	6.0
	21	2.52	111.0	7.0	6.0
	22	2.44	114.3	7.0	6.0
	23	2.52	113.5	7.0	6.0
	24	2.60	108.0	7.0	6.0
	25	2.44	113.7	7.0	6.0
	26	2.32	111.6	7.0	6.0
	27	2.36	109.8	7.0	6.0
	28	2.52	112.0	7.0	6.0
Average			114.1	7.21	7.18
Standard Deviation			2.5	.42	1.24

Table B-2.1. Dynamic Modulus vs. Conditioning and Confining Pressure.

Aggregate Berry Creek Sample	Average Dynamic Modulus, psi: Ultimate Cure			After 4" Vacuum Saturation			After 23" Vacuum Saturation		
	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi
1		195,000	213,000		152,000	179,000			
2		202,000	215,000		136,000	158,000			
4		277,000	300,000		192,000	256,000		208,000	224,000
5		278,000	302,000		274,000	303,000		189,000	209,000
6		291,000	318,000		288,000	325,000		192,000	235,000
8		230,000	255,000		131,000	155,000	91,000	126,000	157,000
9		318,000	335,000		145,000	172,000	133,000	175,000	205,000
10		235,000	265,000		110,000	133,000	107,000	112,000	129,000
11	266,000	290,000	345,000						
12	190,000	345,000	373,000						
17	112,000								
18	133,000								
Average (# of Samples)	175,000 (4)	266,000 (10)	292,000 (10)		179,000 (8)	210,000 (8)	110,000 (3)	167,000 (6)	193,000 (6)
Standard Deviation	68,900	49,200	54,100		67,500	73,700	21,200	38,900	41,300

Note:
1) 10 lb. seating load
2) 0.10 second load duration
3) 30 cycles per minute load frequency
4) 1 psi = 6.894 kN/m^2

Table B-2.2. Dynamic Modulus vs. Conditioning and Confining Pressure. (continued)

Aggregate Eckman Creek	Sample	Average Dynamic Modulus, psi								
		Ultimate Cure			After 4" Vacuum Saturation			After 23" Vacuum Saturation		
		σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi
	1		338,000	353,000		219,000	235,000			
	2		432,000	462,000		163,000	174,000			
	3		480,000	495,000		223,000	249,000			
	4	234,000	323,000	350,000						
	5	251,000	368,000	409,000						
	6	178,000	341,000	365,000						
	10		401,000	433,000						
	11		369,000	387,000						
	12		329,000	372,000						
	13	215,000	340,000	369,000				95,000	137,000	171,000
	14	217,000	308,000	337,000				102,000	104,000	132,000
Average (# of samples)		219,000 (5)	366,000 (11)	394,000 (11)		202,000 (3)	219,000 (3)	98,500 (2)	121,000 (2)	152,000 (2)
Standard Deviation		27,200	52,200	50,400		33,600	39,900	4,950	23,300	27,600

note:
1) 10 lb. seating load
2) 0.10 second load duration
3) 30 cycles per minute load frequency
4) 1 psi = 6.894 kN/m^2

Table B-2.3. Dynamic Modulus vs. Conditioning and Confining Pressure. (continued)

Aggregate Sample	Average Dynamic Modulus, psi								
Oceanlake	Ultimate Cure			After 4" Vacuum Saturation			After 23" Vacuum Saturation		
	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi
1		219,000	233,000		232,000	241,000			
2		259,000	290,000		214,000	227,000			
3		199,000	224,000		184,000	198,000			
4		306,000	331,000						
5		272,000	292,000						
6		217,000	237,000						
7		339,000	369,000						
8		285,000	314,000						
9		203,000	231,000						
16	191,000			82,000			44,000		
17	174,000			78,000			112,000		
18	116,000			63,000			79,000		
19	101,000	158,000	189,000				103,000	119,000	147,000
20	132,000	197,000	236,000				116,000	144,000	173,000
21	125,000	145,000	169,000				72,000	90,000	111,000
Average (# of samples)	140,000 (6)	233,000 (12)	260,000 (12)	74,000 (3)	210,000 (3)	222,000 (3)	87,600 (6)	118,000 (3)	144,000 (3)
Standard Deviation	35,000	59,300	59,600	10,000	24,200	21,900	27,800	27,000	31,000

note:
1) 10 lb. seating load
2) 0.10 second load duration
3) 30 cycles per minute load frequency
4) 1 psi = 6.894 kN/m^2

Table B-2.4. Dynamic Modulus vs. Conditioning and Confining Pressure. (continued)

Aggregate	Sample	Average Dynamic Modulus, psi								
		Ultimate Cure			After 4" Vacuum Saturation			After 23" Vacuum Saturation		
Dune Sand		σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi	σ_3 = 0 psi	σ_3 = 3 psi	σ_3 = 6 psi
	A		261,000	281,000		186,000	193,000		252,000	262,000
	B		350,000	354,000		259,000	262,000		163,000	179,000
	C		296,000	308,000		189,000	200,000		177,000	186,000
	7		228,000	244,000		143,000	162,000	111,000	174,000	198,000
	8		310,000	324,000		180,000	211,000	140,000	230,000	272,000
	9		230,000	243,000		155,000	177,000	132,000	169,000	186,000
	10	163,000	166,000	177,000						
	11	110,000	98,000	110,000						
	12	153,000	174,000	186,000						
	18	216,000	208,000	220,000						
	19	132,000	101,000	121,000						
	20	116,000								
	21	220,000								
	22	205,000								
	23	160,000								
Average (# of samples)		171,000 (8)	232,000 (10)	246,000 (10)		185,000 (6)	201,000 (6)	128,000 (3)	194,000 (6)	214,000 (6)
Standard Deviation		40,000	74,900	72,600		40,500	34,600	15,000	37,200	41,800

note:
1) 10 lb. seating load
2) 0.10 second load duration
3) 30 cycles per minute load frequency
4) 1 psi = 6.894 kN/m^2

Table B-3.1. Tensile Strength of Unconditioned Samples.

Aggregate	Sample	Unconditioned Tensile Strength, psi
Berry Creek	17	24.0
	18	30.1
	19	27.1
	Average	27.1
	Std. Dev.	3.1
Eckman Creek	10	22.5
	11	24.1
	12	24.8
	22	33.7
	Average	26.3
	Std. Dev.	5.04
Oceanlake	3	19.8
	4	22.3
	5	14.4
	28	17.1
	29	13.4
	30	13.5
	Average	16.8
	Std. Dev.	3.68
Dune Sand	1	15.2
	2	17.1
	3	16.9
	27	26.9
	28	19.4
	Average	19.1
	Std. Dev.	4.61

1 psi = 6.894 kN/m^2

Table B-3.2. Tensile Strength of Conditioned Samples

Aggregate	Sample	Unconditioned Tensile Strength, psi
Berry Creek	1	28.6
	2	26.7
	3	22.0
	4	21.8
	5	15.4
	6	19.6
	8	26.7
	9	31.8
	10	30.7
	Average	24.8
	Std. Dev.	5.45
Eckman Creek	13	25.7
	14	20.2
	15	19.9
	Average	21.9
	Std. Dev.	3.27
Oceanlake	B	13.2
	C	15.4
	16	22.2
	18	24.2
	Average	18.8
	Std. Dev.	5.28
Dune Sand	7	17.7
	8	19.2
	9	19.2
	Average	18.7
	Std. Dev.	0.87

1 psi = 6.894 kN/m^2

Table B-4.1. Fatigue Test Results, Unconditioned Samples.

Aggregate	Sample	Density pcf	Dynamic Load lbs.	Dynamic Modulus psi	Initial Tensile Strain 10^{-6} in/in	Nf
Berry Creek	7	123.6	100	182,000	71.3	353
	14	126.8	20	112,000	29.8	7,480
	15	122.3	50	110,000	65.2	435
	16	122.3	25	103,000	34.8	6,431
	*27	125.2	27	56,400	59.6	6,482
	*28	126.4	45	43,800	129.0	1,281
	*30	129.3	49	93,700	67.5	5,896
	*31	125.4	30	66,500	54.8	2,000
Eckman Creek	4	130.5	75	234,000	45.2	1,353
	5	131.6	100	251,000	52.2	1,816
	6	128.5	200	178,000	147.0	265
	7	133.7	50	335,000	21.0	33,663
	8	133.7	125	296,000	56.6	810
	9	131.8	65	314,000	27.7	2,111
	*23	133.9	58	168,000	47.0	did not fail + 195,300
	*23	133.9	81	92,800	117.0	13,511
	*24	128.2	66	101,000	65.9	5,292
Oceanlake	10	127.5	50	205,000	31.9	3,500
	11	126.0	75	149,000	65.0	1,150
	12	124.8	75	180,000	52.7	1,520
	13	124.6	50	212,000	29.8	2,916
	14	124.0	20	213,000	12.6	7,383
	15	129.8	23	163,000	19.4	3,244
	*31		32	76,300	54.1	1,148
	*32		85	78,700	137.0	200
	*33		27	75,700	44.4	3,136
Dune Sand	14	117.8	81.6	290,000	74.4	8,184
	15	115.4	127.0	156,000	105.2	300
	24	108.0	94.3	281,000	41.9	5,612
	25	113.7	104.0	161,000	85.6	411

note:
*) 0.0 lb. seating load
1) 10 lb. seating load
2) 0.10 second load duration
3) 1 cycle per second load frequency
4) 1 lb. = 4.45N
5) 1 psi = 6.894 kN/m^2

Table B-4.2. Fatigue Test Results, Conditioned Samples.

Aggregate	Sample	Density pcf	Dynamic Load, lbs.	Dynamic Modulus, psi	Initial Tensile Strain, 10^{-6} in/in	N_f
Berry Creek	20	130.2	84.8	30,100	375	1,155
	21	131.0	66.8	34,000	265	3,198
	22	120.2	25.0	16,900	204	897
	23	122.8	53.0	28,000	237	725
	24	126.6	26.5	41,500	101	2,555
	26	129.6	25.4	50,800	64.4	37,520
Eckman Creek	16	123.9	65.7	62,700	146.6	1,411
	17	131.2	61.5	94,000	91.4	3,300
	18	129.1	40.3	50,000	107.0	7,663
	19	128.5	37.1	81,000	60.7	29,500
	20	127.9	61.5	66,900	120.0	8,649
	21	128.8	44.5	74,000	78.7	did not fail @ 100,000
	21	128.8	89.1	63,900	182.0	4,778
Oceanlake	19	128.6	56.0	69,000	107.4	712
	20	123.6	51.0	102,000	68.9	873
	22	124.4	39.2	75,400	71.5	1,600
	23	128.4	40.3	86,000	63.2	3,653
	24	126.6	29.7	91,800	41.6	10,076
	25	124.2	41.3	42,900	122.0	3,701
	26	126.5	29.7	70,900	53.9	16,566
	27	126.9	31.8	49,900	82.1	13,758
Dune Sand	18	114.1	75.0	105,000	101.8	250
	19	110.7	36.0	62.700	72.7	2,091
	20	114.4	50.9	76,000	87.2	325
	21	111.0	42.4	116,000	47.2	did not fail @ 139,000
	21	111.0	74.2	102,000	93.7	814
	22	114.3	95.4	118,000	108.0	560
	23	113.5	44.5	111,511	51.4	33,283

note:
1) seating load = 0.0 lbs.
2) samples conditioned with 2 hour, 23 inch Hg. vacuum saturation and 7 day water soak
3) average modulus of 20, 50, and 75 lb. dynamic loads
4) modulus determined from given dynamic load
5) 1 lb = 4.45N
6) 1 psi = 6.894 kN/m^2

APPENDIX C

INDIRECT TENSILE TEST METHOD
FOR
RESILIENT MODULUS OF BITUMINOUS MIXTURES

1. Scope

1.1. This method covers procedures for preparing and testing laboratory fabricated or field recovered cores of bituminous mixtures to determine resilient modulus values using the repeated-load indirect tensile test. The procedure described covers a range of temperatures, loads, loading frequencies, and load durations. The minimum recommended test series consists of testing at 41, 77*, and 104°F (5, 25*, and 40°C) at a loading frequency of 0.33 to 1.0 Hz for each temperature. This recommended series results in 9 test values for one specimen which can be used to evaluate the overall resilient behavior of the mixture.

2. Applicable Documents

2.1. ASTM Standards:

D 1559 Resistance to Plastic Flow of Bituminous Mixture Using Marshall Apparatus

D 1561 Preparation of Test Specimens of Bituminous Mixture by Means of Kneading Compactor

D 3515 Hot-Mixed, Hot Laid Asphalt Paving Mixture

D 3496 Method for Preparation of Bituminous Mixture Cylindrical Specimens

*or ambient laboratory temperature as appropriate.

D 3387 Test for Compaction and Shear Properties of Bituminous Mixtures by Means of the U.S. Corps of Engineers Gyratory Testing Machine (GTM).

3. Summary of Method

3.1. The repeated-load indirect tensile test for resilient modulus is conducted by applying compressive loads with a haversine, square wave, or trapezoidal wave form. The loads act parallel to and along the vertical diametral plane of a cylindrical specimen of asphalt concrete (Figure 1) at a given temperature and loading frequency. The resulting recoverable horizontal deformation of the specimen is measured and used to calculate the resilient modulus of elasticity with an assumed value of Poisson's ratio or with a calculated value using the measured recoverable vertical deformation.

4. Significance and Use

4.1. The values of the resilient modulus and resilient Poisson's ratio can be used for bituminous paving mixture design, as a supplement to standard values already used. The resilient properties can also be used in layered elastic analysis and thickness design of pavements. Since the procedure is non-destructive, the test method may further be used in research investigations such as evaluation of materials performance with time (e.g., exposure tests). The method is not intended for use in specifications.

5. Apparatus

5.1. Testing machine - The testing machine should have the capa-

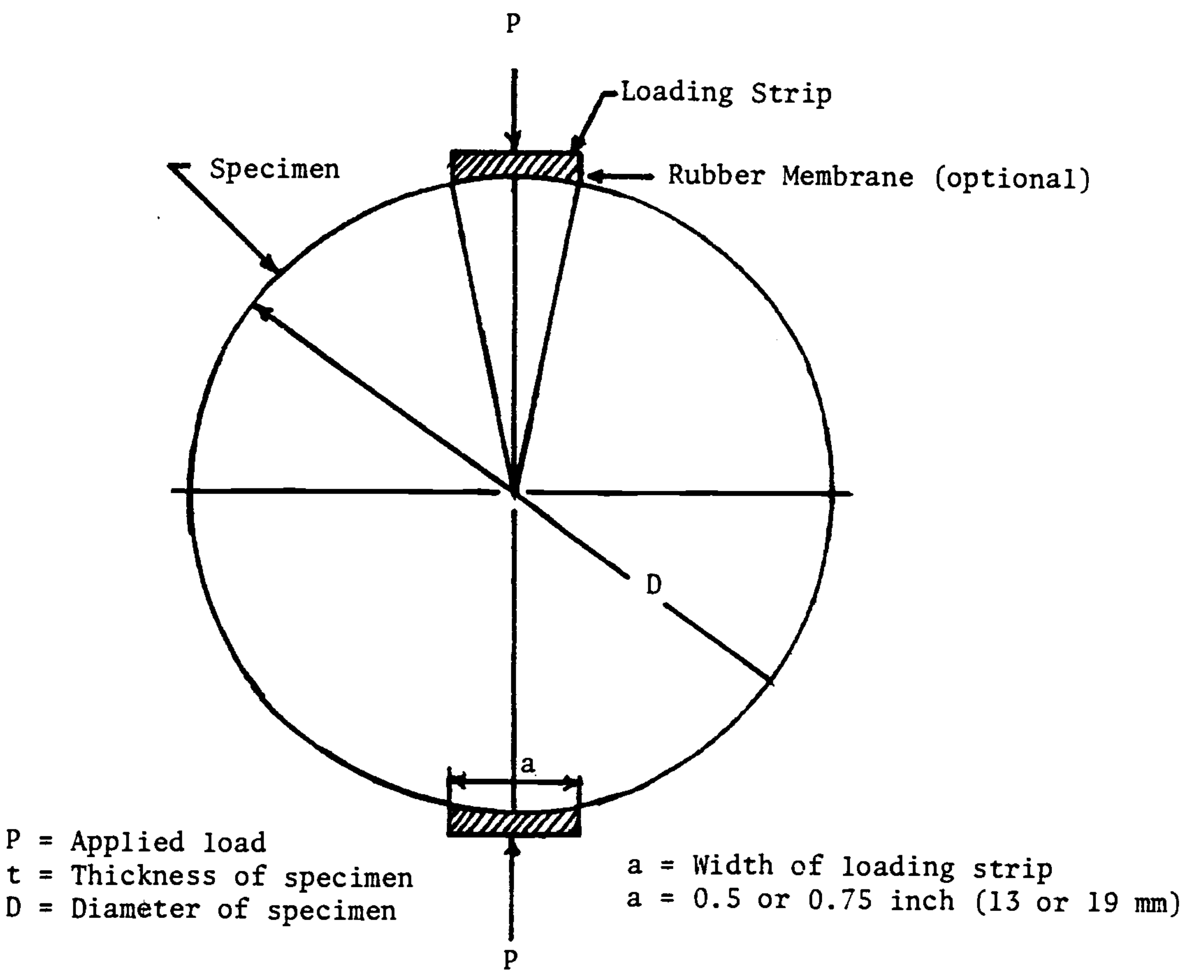

Fig. 1. Indirect Tensile Test

bility of applying a load pulse over a range of frequencies, load durations, and load levels.

Note 1 - An electro-hydraulic testing machine with a function generator capable of producing the prescribed wave form has been shown to be suitable for use in repeated-load indirect tensile testing. Other commercially available or laboratory constructed testing machines such as those using pneumatic repeated loading can also be used. However, these latter machines may not have the load capability to handle larger specimens at the colder testing temperatures.

5.2. Temperature control system - The temperature control system should be capable of control over a temperature range. The temperature chamber should be large enough to hold an adequate number of specimens for a period of 24 hours prior to testing.

5.3. Measurement System - The measurement system should include a recorder or other measuring device for the horizontal and vertical deformations. If Poisson's ratio is to be assumed, then only horizontal deformations must be recorded. Loads should be measured and recorded or accurately calibrated prior to testing. The system should be capable of measuring deformations in the range of 0.00001 inches (0.00025 mm) of deformation. An alternate system could give deformation readout directly by suitable calibration of the loading and measurement components.

5.3.1 Recorder - The recorders should be independent of frequency for tests conducted up to 1.0 Hz.

5.3.2 Deformation Measurement - The values of vertical and horizontal deformation are measured by LVDT's or other suitable devices. The horizontal LVDT's should be at mid-height opposite each other on the specimens horizontal diameter. The sensitivity and type of measurement device should be selected to provide the deformation readout required in Section 4.3.

Note 2 - The Trans-TEX Model 350-000 LVTD and Statham UC-3 tranducers have been found satisfactory for this purpose.

Note 3 - The gages should be wired to preclude the effects of eccentric loading so as to give the algebraic sum of the movement of each side of the specimen. Alternatively, each gage can be read independently and the results summed separately.

5.3.3. Load Measurement - Loads are measured with an electronic load cell capable of satisfying the specified requirements for load measurements in Section 5.3.

5.4. Loading Strip - A steel, brass or aluminum curved-loading strip with radius equal to that of the test specimen is required to transfer the load from the testing machine to the specimen. The load strip shall be 0.5 or 0.75 inches (13 or 19 mm) wide for 4.0 or 6.0 inch (102 or 150 mm) diameter specimens, respectively; edges should be rounded in order to not cut the sample during testing. For specimens with rough textures, a thin hard rubber membrane attached to the loading strip has been found effective in reducing impact loading effects if vertical defomrations are not monitored.

6. Specimens

6.1. Laboratory Molded Specimens - Prepare the laboratory molded specimens according to acceptable procedures such as ASTM MEthods D 1561, D 1559, D 3496 and D 3387. The specimens should have a height of at least

2 inches (50 mm) and a minimum diameter of 4 inches (102 mm), but not less than four times the maximum nominal size of the aggregate particles.

6.2. Pavement Cores - Core samples from an inservice pavement should have a minimum height of 1.5 inches (38 mm) and diameters of at least 4 inches (102 mm) but not less than four times the maximum nominal size of the aggregate particles. Cores should have relatively smooth parallel surfaces.

Note 4 - Laboratory molded specimens and pavement cores with diameters of 6 inches (150 mm) and heights of 3 inches (75 mm) or more have been used.

7. Procedures

7.1. Place test specimens in a controlled temperature cabinet and bring them to the specified test temperature. Unless temperature is monitored, and the actual temperature known, the specimens should remain in the cabinet at the specified test temperature for at least 24 hours prior to testing.

Note 5 - A dummy specimen with a thermocouple in the center can be used to determine when the desired test temperature is reached.

7.2. Place specimen into loading apparatus and position the steel or aluminum loading strips. Adjust and balance electronic measuring system as necessary.

7.3. Apply a preconditioning loading consisting of a repeated haversine, or other suitable waveform, to the specimen without impact for a minimum period sufficient to obtain uniform deformation readout. Depending upon the loading frequency, a minimum of 50 to 200 load repetitions is generally sufficient; however, the minimum for a given situation must be determined so that the resilient deformations are stable. A complete test will usually include measurements at three temperatures, e.g., 41 ± 2, 77 ± 2, and 104 ± 2°F (5, 25, and 40°C), at one or more loading frequencies, e.g., 0.33, 0.5 and 1.0 Hz, for each temperature. Recommended load range is from 10 to 50 percent of the tensile strength. Tensile strength can be determined from a destructive test on a specimen and the equation of Section 8.3.

Note 6 - Load duration is the more important variable and it is recommended that the duration be held to some minimum which can be recorded. The recommended range for load duration is 0.04 to 0.4 sec., with 0.1 sec. being representative of transient pavement loading. Recommended frequencies are 0.33 to 1.0 Hz. In lieu of tensile strength data, load ranges from 25 to 200 lbs per inch of core or specimen thickness can be used.

7.4. Monitor the vertical and horizontal deformations during the test.

Note 7 - A typical load pulse-deformation trace is shown in Figure 2, along with notations indicating the load-time terminology.

7.5. Each test should be completed within two minutes from the time specimens are removed from temperature control cabinet.

Note 8 - The two minute testing time limit is waived if loading is conducted within a temperature control cabinet meeting requirements in Section 5.2.

7.6. Each specimen should be tested more than once by rotating the specimen and loading through another diametral plane. Three laboratory fabricated specimens or three cores are recommended for a given test series with variables of temperature, load duration, and load. In order to reduce permanent damage to the specimen, testing should begin at the lowest temperature, shortest load duration, and smallest load. Subsequent testing on the same specimen should be for conditions producing progressively lower moduli. Bring specimens to specified temperature before each test.

Note 9 - If excessive total deformation, i.e., greater than 0.001 inch (0.0254 mm), occurs during a test, reduce the applied load, the test temperature, or both.

8. Calculations

8.1. Measure the average recoverable horizontal and vertical deformations over at least three loading cycles (see Figure 2) after the repeated resilient deformation has become stable.

8.2. Calculate the resilient modulus of elasticity E_R and Poisson's ratio ν using the following equations:

$$E_R = \frac{P(\nu + 0.27)}{t\Delta_x}, \text{ psi}$$

$$\nu = 3.59 \frac{\Delta_x}{\Delta_y} - 0.27$$

where P = repeated load, lb.

ν = Poisson's ratio

t = thickness of specimen, in.

Δ_x = recoverable horizontal deformation, in.

Δ_y = recoverable vertical deformation, in.

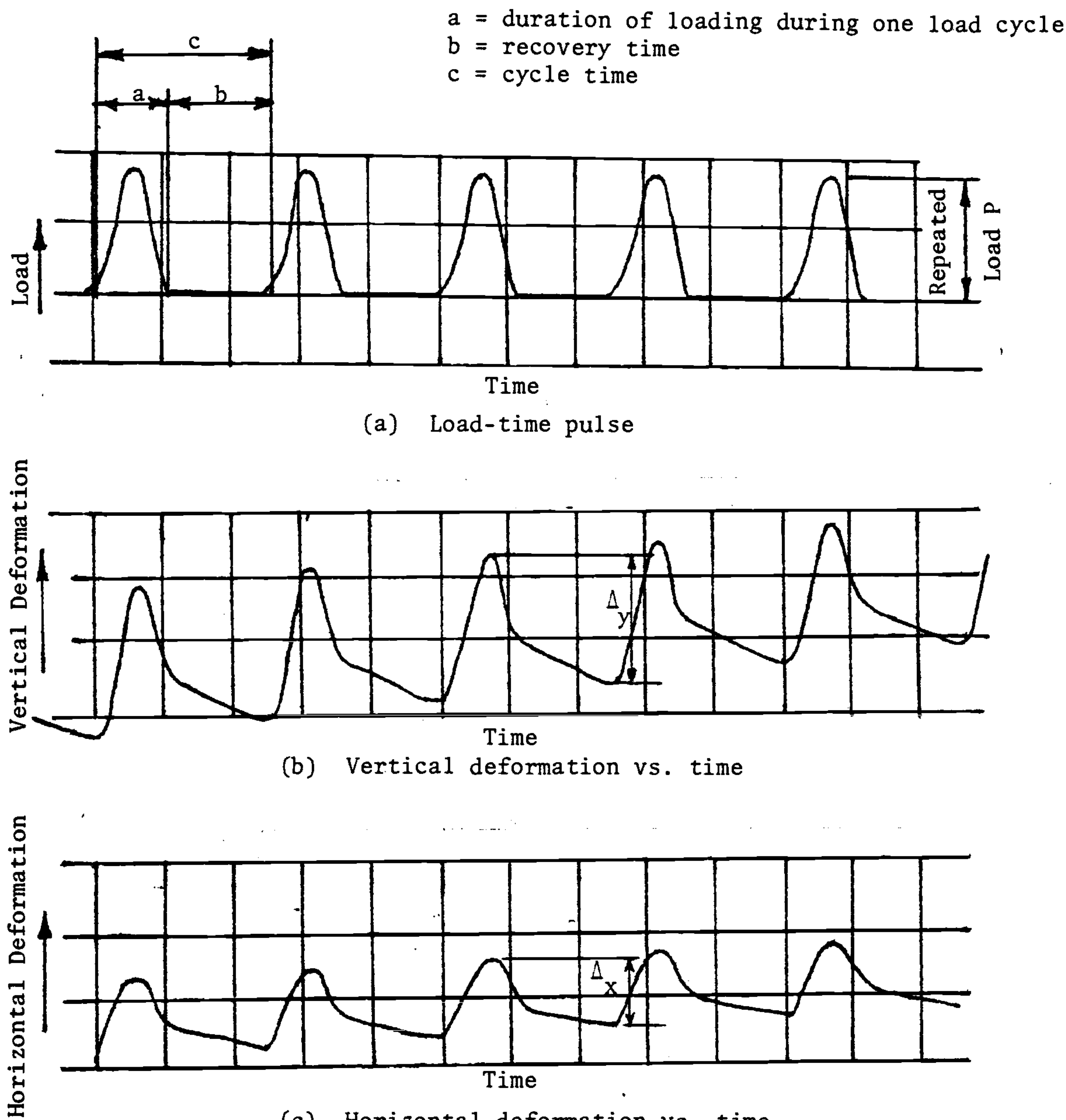

Figure 2. Typical load and deformation versus time relationships for repeated-load indirect tensile test

Note 10 - Poisson's ratio can be calculated using the above equation for 4-inch and 6-inch diameter specimens with 0.5 inch or 0.75 inch wide loading strips, respectively, or the value can be assumed in which case vertical deformations are not required. A value of 0.35 for Poisson's ratio has been found to be reasonable for asphalt mixtures at 77°F (25°C).

8.3. The tensile strength S_T can be calculated using the following equation:

$$S_T = \frac{2P_{ult}}{\pi t D}$$

where P_{ult} = the ultimate applied load required to fail specimen, lb.

t = thickness of speciment, in.

D = diameter of specimen, in.

9. Report

9.1. Report the average resilient modulus at temperatures of 41, 77, and 104°F (5, 25, and 40°C) for each load and load frequency used in the test.

10. Precision

10.1. The precision of the method is being established.