FINAL REPORT

SUBGRADE ELASTIC MODULUS FOR ARIZONA PAVEMENTS

by
Robert W. Crossley, P. E.
&
George H. Beckwith, P. E.

Submitted to
Arizona Department of Transportation
Highways Division

for
Research Project - HPR 1-15 (156)
Sponsored by
Arizona Department of Transportation
in Cooperation with
U.S. Department of Transportation
Federal Highway Administration

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/AZ-75/142		

4. Title and Subtitle

SUBGRADE ELASTIC MODULUS FOR ARIZONA PAVEMENTS - FINAL REPORT

5. Report Date: January 1978

6. Performing Organization Code:

7. Author(s): Robert W. Crossley, P.E. & George H. Beckwith, P.E.

8. Performing Organization Report No.:

9. Performing Organization Name and Address

Sergent, Hauskins & Beckwith, Inc.
3940 West Clarendon Avenue
Phoenix, Arizona 85019

10. Work Unit No. (TRAIS):

11. Contract or Grant No.: HPR 1-15 (156)

12. Sponsoring Agency Name and Address

Arizona Department of Transportation
206 South Seventeenth Avenue
Phoenix, Arizona 85007

13. Type of Report and Period Covered: FINAL REPORT 1975 to 1978

14. Sponsoring Agency Code:

15. Supplementary Notes

In cooperation with U.S. Department of Transportation, Federal Highway Administration

16. Abstract

Elastic modulus of subgrade is an important input parameter in the newly developed rational methods for the design of pavements. In order to provide guidance for the selection of realistic values of E_s for pavement design in Arizona, testing was performed at five field sites typical of Arizona subgrades. Testing involved plate bearing tests, refraction seismic, CBR, Dynaflect, and Road Rater. Correlations are made and guidelines are presented for incorporation of the findings into a pavement management system.

17. Key Words	18. Distribution Statement
Subgrade Elastic Modulus, Pavement Management System, Plate Bearing Test, Refraction Seismic, CBR, Dynaflect, Road Rater, Subgrade Moisture	No Restriction. This report is available to the public through: NTIS Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	372	

Form DOT F 1700.7 (8-72)

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ACKNOWLEDGEMENTS

The writers wish to extend thanks to Mr. Gene R. Morris and Mr. Ben W. Ong of Research Division, Arizona Department of Transportation for their encouragement and cooperation during this project. Thanks are also due to Mr. John B. Hauskins, Jr., and Mr. John F. Eisenberg who provided access and assistance with the ADOT computer, to Mr. George B. Way for providing environmental data, and to Mr. John Burns and Mr. David Allocco who arranged for use of the Dynaflect.

Special thanks are due to Mr. Dale S. Parker of SHB, for allowing the use of his ranch property in Rainbow Valley for extensive in-situ testing, and to Mr. Malcolm J. Bartlett of Kaiser Aetena and Mr. Palmer L. Clarkson, Jr. of Fortune Properties, Inc., who likewise provided access to sites at McCormick Ranch.

Finally, the participation of over 30 employees of Sergent, Hauskins, and Beckwith, including drillers, equipment operators, field and laboratory technicians, draftsmen, engineers, and typists is appreciated. Worthy of special citation are senior technician Mark LaCross who performed the many plate bearing tests, to draftsman David M. Farr for the work done in preparing the many figures in this report, and to technical secretary Mrs. Nita J. Feutz, who typed the manuscript for this report.

ABSTRACT

Elastic modulus of subgrade is an important input parameter in the newly developed rational methods for the design of pavements. In order to provide guidance for the selection of realistic values of E_s for pavement design in Arizona, testing was performed at five field sites typical of Arizona subgrades. Testing involved plate bearing tests, refraction seismic, CBR, Dynaflect, and Road Rater. Correlations are made and guidelines are presented for incorporation of the findings into a pavement management system.

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LIST OF ABBREVIATIONS AND SYMBOLS

A	Coefficient in resilient modulus equation
a	Radius of a plate
AASHTO	American Association of State Highway and Transportation Officials
ADOT	Arizona Department of Transportation
ADT	Average Daily Traffic
ASTM	American Society for Testing Materials
B	Bulk Modulus
B	Footing Width
B	Exponent in resilient modulus equation
CBR	California Bearing Ratio
cm	Centimeter
CRT	Cathode Ray Tube
D	Confined Modulus
DSM	Dynamic Stiffness Modulus
d	Dynaflect deflection reading
d_1, d_2, etc.	Dynaflect reading based on numbered sensor
E	Young's Modulus, Modulus of Elasticity
E_s	Modulus of Elasticity of soil or subgrade*
E_{seis}	Modulus of Elasticity of soil under seismic loading
Ec	Modulus of Elasticity of soil determined by consolidation test
Ep	Modulus of Elasticity of soil determined by Menard Pressuremeter
E_{dynf}	Modulus of Elasticity of soil determined by Dynaflect
E_{RR}	Modulus of Elasticity of soil determined by Road Rater

*In Section III and IV of this text E_s is the modulus of subgrade calculated from a large (30") diameter plate bearing test under 30-50 psi.

E_{d_1}, E_{d_2}, etc.	Modulus of Elasticity of soil determined by Dynaflect using only the numbered sensor
E_1, E_2, etc.	Modulus of Elasticity of material in a layered elastic system
E^*_1, etc.	Elastic Modulus in layered system determined by computer iteration
$\|E^*\|$	Complex Modulus
EAL	Equivalent Axel Load
ECI	Elastic Compliance Index
ERF	Environmental Reduction Factor
^oF	Degrees Farenheit
FAA	Federal Aviation Administration
G	Shear Modulus
H	Elastic Constant in Ahlvin and Ulery expression
h_1, h_2, etc.	Thickness of layer
I_p	Influence factor
K_1	Coefficient in resilient modulus equation
K_2	Exponent in resilient modulus equation
k	Modulus of Subgrade Reaction, on spring constant of soil
k_h	Horizontal modulus of subgrade reaction
LL	Liquid Limit
M	Modulus of Deformation
MESL	Membrane Encapsulated Soil Layer
M_R	Modulus of Resilience
N	Penetration Number for blows per 6 inches
NA	Not Available
NDT	Non-destructive Testing
n_h	Rate of change of k_h with depth
P	Load on plate
PCA	Portland Cement Association

PI	Plasticity Index
p	Pressure on loaded area
psi	Pounds per square inch
R	Stabilometer Value
S	Spreadability
SDI	Subgrade Damping Index
SHB	Sergent, Hauskins, and Beckwith
SSV	Soil Support Value
SPT	Standard Penetration Number
V	Volume
V_o	Initial Volume
V_s	Velocity of Shear Wave
V_c	Velocity of Compression Wave
w/c	Moisture Content
γ_d	Dry density
Δ	Incremental change
ε	Strain
ε_t	Tangential Strain
ε_v	Vertical Strain
ρ	Mass Density
ρ	Deflection
θ	Sum of principal stresses ($\sigma_1+2\sigma_3$)
σ_1	Major Principal Stress
σ_3	Minor Principal Stress
σ_d	Deviator Stress
ν	Poisson's Ratio

PROJECT SUMMARY

The Arizona Department of Transportation is in the process of developing a pavement management system which will include rational pavement design and overlay procedures based on fundamental material properties. Elastic models will likely be used; hence a body of knowledge concerning the elastic behavior of typical subgrade soils in Arizona is needed.

In January, 1976, Sergent, Hauskins, and Beckwith, Inc., Consulting Geotechnical Engineers, was commissioned to undertake a study of subgrade elastic modulus as it pertains to pavement design in Arizona. The research effort was to focus particularly upon the response of partially saturated and cemented desert soils which are widespread in the state, and which have received little attention in the general engineering literature.

The study consists of five elements. Firstly, a literature review of the subjects of elastic properties of soils, pavement design, and environmental changes in subgrades was performed. Secondly, a sensitivity analysis and review was made to determine the importance, in terms of economy of design, of subgrade modulus as an input parameter. Thirdly, an extensive program of comparative field testing at five test stations, typical of Arizona subgrades, was formulated and performed. Testing involved repetitive and non-repetitive plate bearing tests, refraction seismic, in-situ CBR, Dynaflect, and Road Rater. The fourth element involved the correlation of data from the various tests - including comparison of findings from the five sites with findings presented by researchers in other parts of the country

and throughout the world. The fifth element consists of a series of conclusions and recommendations for incorporating the findings of this study into future pavement design and management systems.

The two year project is presented herein with the general conclusion that elastic modulus of subgrade for most typical Arizona soils is far from an intrinsic material property, but is a highly dependent variable. Due to the cemented, dessicated, and fissured state of most soils in this semi-arid environment, correlations of E_s with index properties are impractical. The most responsible approach to the selection of meaningful values of E_s for pavement design lies in the application of a statistically valid body of in-situ test data, combined with a thoughtful amount of engineering judgement and intuition.

SECTION I. INTRODUCTION, OVERVIEW, AND DISCUSSION OF THE USE OF ELASTIC MODULUS IN PAVEMENT DESIGN

1.1 Introduction

Continually rising costs coupled with an increasing scarcity of quality roadbuilding materials in many parts of the world in recent years have created a desire on the part of engineers for optimization in the design of concrete and asphalt pavements. As is the case today in many areas of engineering technology, empirical methods for pavement design, founded on what had been considered "successful" past designs, are giving way to mechanistic procedures utilizing fundamental material properties.

Today in virtually every political subdivision, responsible agencies have adopted or are in the process of developing more sophisticated procedures which seek to model pavement performance in a rational manner. In the United States, the Federal Highway Administration has urged each State Department of Transportation to develop its own pavement design and management systems, incorporating regional considerations of climate, subsurface environment and economics of construction. For several years, the Arizona Department of Transportation, in cooperation with the universities and private consultants, has been working toward developing such a system.

With very few exceptions, each of the several dozen new pavement design methods which are described in the

technical literature utilize a layered systems approach to model pavement structures. Inasmuch as the behavior of a layered system depends on the stress-strain performance of the various layers, a model characterizing the load response of each material is a vital element in every pavement design method. The simplest and most widely used model for materials response is the linear elastic model, although more sophisticated models such as visco-elastic, elasto-plastic, and finite element models are also found.

In a typical mechanistic flexible pavement design procedure based on elastic theory, a three layer system consisting of an asphaltic concrete or bituminous bound layer, an unbound granular base layer, and an infinite subgrade is described. Response of each of the materials is a function of the two fundamental elastic parameters, modulus of elasticity (E) and Poisson's ratio (ν), which must be assigned to each layer. Other important input parameters include the traffic data (the number of wheel loads, size, and tire pressure), and thicknesses of the various layers. With these, the performance of selected pavement sections can be modeled.

The purpose of this research effort is to focus on a single input parameter, the elastic modulus of the subgrade. The research is intended to provide insight into the nature of subgrade response as it affects typical pavement sections in Arizona and the Southwest.

Dr. R. A. Jimenez of the University of Arizona is actively engaged in developing a rational method for the design

of asphalt pavements in Arizona (1, 2)* based on a three layer elastic model and employing modulus ratios to calculate response. Several other projects are on-going at the Arizona Department of Transportation to determine materials' behavior and environmental conditions under Arizona pavements.

While a final design procedure and pavement management system will be several years in development, it is certain that knowledge of subgrade modulus will be required in three areas.

These are:

- New Pavement Design - The load response of the soil or compacted subgrade is essential in predicting the behavior of any pavement system.
- Evaluation of Existing Pavements - Measured values of elastic modulus for both surface and subgrade layers of existing pavement structures provide excellent indices of deterioration which can be used in planning pavement rehabilitation.
- Pavement Overlay Design Procedures - As is the case with design of new pavements, many agencies are developing overlay procedures which are based on elastic theory. For these, subgrade elastic modulus is an important input parameter.

*Numbers in parentheses correspond to references listed at the end of the report.

The purpose of this project was to develop methods for determining subgrade modulus for Arizona conditions. In the course of this study, the current trends in pavement design were integrated with our experience and a considerable amount of field testing in developing the recommendations contained herein.

1.2 Summary of Program

1.2.1 General

While a broad understanding of the subject of elastic properties of soils is an overall objective of this effort, a more specific goal is to critically examine the load response of typical Arizona soils for which little information is available in the published literature. For this reason, major emphasis was focused on cemented desert clayey sands, sandy clays, and clay-sand-gravel mixtures, and on other partially saturated or desiccated cohesive soils. The body of knowledge available in technical literature concerning the behavior of more classical soil types, such as clean sands and saturated normally consolidated clays, is considered adequate. These will, undoubtedly, receive even more scrutiny from researchers in the next few years and therefore received less attention in this study.

The research effort was divided into five general headings. These are the literature review, sensitivity analysis, field testing, data correlation, and recommended evaluation procedures.

1.2.2 Literature Review

An effort was made to compile a complete bibliography of research related to the subjects of elastic modulus of soils and pavement design. From the various abstract services and other listings, over 400 papers were referenced. More than half of these were obtained and reviewed by the investigators during the course of the project. References will be made to significant findings of many of these research efforts throughout this report.

Specific topics which were researched include:

- Elastic properties of soil and rock, especially as applied to geotechnical engineering or pavement design. Particular attention was devoted to articles concerning materials testing, both in the laboratory and in-place, in order to obtain an understanding of the various ways of quantifying the load response of the materials.

- Relationship between statically and dynamically derived moduli, differences between moduli determined by various tests, effects of size of loaded areas, vertical confining pressure, strain level, and variation in modulus as a function of temperature and moisture content.

- Pavement design methods - every available rational method of pavement design and overlay design which came to our attention, regardless

of the degree of refinement, was reviewed. The suggested methods of characterizing subgrade response were of special interest.

- Environmental influences as they affect pavement performance. Studies and case histories of moisture and temperature changes beneath pavements were of interest, particularly those involving pavements in arid regions. An exception within this topic is that of frost related pavement distress. Because of the low rainfall-high temperature environment throughout most of Arizona, the "spring thaw" problem, which is a major concern in many states, was not closely scrutinized.

1.2.3 Sensitivity Analysis

The individual importance of a single input variable within a design system containing many inputs must be measured from the standpoint of economy. In the case of a pavement design procedure, which demands several traffic and materials behavior parameters which are usually not readily available at the beginning of a project, the question of the expense a user may reasonably justify to refine each input value is of major importance.

This report will show that most means of determining moduli are costly in terms of time and money. Therefore, the effect of variations of subgrade modulus on the overall effectiveness of a design procedure must be placed clearly in perspective.

Many sensitivity studies have been performed on specific systems (e.g. 3,4,5) to determine the relative importance of each of the numerous input variables. Rather than duplicate the findings of others, the authors sought a means of presenting the role of subgrade modulus in terms of pavement thicknesses required for various volumes of traffic.

Results of the sensitivity study are given in section 1.9.

1.2.4 Field Testing

A decision was made early in the project to emphasize in-situ testing rather than attempt to test "undisturbed" samples in the laboratory. This decision was made for two reasons.

First, there is a wealth of information in the literature resulting from the dynamic testing of soils in the laboratory, utilizing both undisturbed and reconstituted materials. It was felt that more laboratory data would not add significantly to the existing body of knowledge. Secondly, the Arizona soils of interest are often cemented to varying degrees and are usually jointed or fissured in-place. In many respects, their behavior resembles that of soft rock rather than soil. The taking of "undisturbed" samples in the field is difficult at best and often results in stress-relief which creates further fissuring. Reconstituted samples or samples trimmed for testing are unlikely to duplicate the type of load response which would occur in-situ.

For these reasons, all testing, other than grain-size analysis and Atterberg Limits, were performed in-place at five selected field test stations. The program involved plate bearing tests, refraction seismic, CBR, Dynaflect, and Road Rater testing. Detailed discussions of the test stations and the testing procedures are given in Section II.

1.2.5 Data Correlation

Values of elastic modulus have been calculated for each test. The results of all testing are presented in Section III. In Section IV, an attempt is made to correlate the values derived from the various methods.

1.2.6 Recommended Evaluation Procedures

Recommendations regarding the determination and selection of subgrade elastic modulus values for use in pavement design projects in Arizona, are given in the latter part of Section IV. These procedures make use not only of field and laboratory testing, but also include an evaluation of the likelihood of changes in the subsurface environment during the design life of the project.

1.3 History of the use of Elastic Theory in Soil Mechanics and Pavement Design

In recent years it has become common to find published procedures for analyzing problems of foundation behavior

and soil-structure interaction employing models based on elastic theory. To be used effectively, these methods require the integration of two contrasting disciplines; the precise and methodical theory of elasticity and the empirical and interpretative discipline of soil mechanics.

The classical theory of elasticity had it beginnings as early as the 17th Century with the published work of John Hooke. Since that time, the development of elastic theory has been carried on primarily by mathematicians and physicists such as Rankine, Coulomb, Lord Rayleigh, Otto Mohr and others. Until recently, elastic theories were used primarily to describe the behavior of a material whose properties were homogeneous and isotropic, that is, a material which today would be called a "continuum". With the advent of high-speed computers containing large storage capacities, it has been possible to extend elastic analyses to provide solutions to the response of media which are layered or contain other complex geometries.

In contrast, the theories of soil behavior and the art of "soils engineering" have been developed, for the most part, by practitioners during this century and are based largely on empirical evidence. Until recently, soil has been treated as a "particulate" material reflecting the observation that it is composed of discrete grains or particles.

In the early work of pioneers in the field, such as Terzaghi, Casagrande, Bishop and others, major emphasis

was placed on the size and gradation of particles, shape of grains, void ratio, friction and cohesion developed by particle contact, and the role of pore water in the determination of soil behavior; particularly for fine grained soils.

During the 1950's and 60's, the state of the art allowed engineers to predict settlement of footings or embankments placed on a given soil based on the results of laboratory tests, such as density, void ratio and consolidation. In the same manner, the load-carrying capacity of a pavement subgrade could be estimated using indices such as CBR or R values.

The worth of any design technique is best evaluated by the performance of the completed product. There have been numerous indications that soil deformations predicted from classical soil mechanics are much greater than actually occur. This may be due largely to the fact that tests, such as the laboratory consolidation test, measure deformation in one direction only while, in reality, deformation takes place in three dimensions.

By 1970, practitioners of soil mechanics, rock mechanics and engineering geology (who were now calling themselves "geotechnical engineers") began to see widespread use of analyses which treated soil and rock as though they were elastic materials. On encountering formulas that demanded a value of E (modulus of elasticity) and ν (Poisson's ratio) for subsurface materials, many engineers simply returned to the more time-honored, but conservative, analyses.

Today, based on extensions of technology developed in the 30's and 40's, the stresses and settlements beneath footings of various shapes can be predicted from elastic analyses using tables or formulas developed by Burmister (6), Giroud (7), Ahlvin and Ulery (8), Poulos (9), Ueshita and Meyerhof (10) and others. Likewise, pile settlements, both individually and in groups, have been treated elastically by Poulos and Davis (11) and Poulos and Mattes (12).

The analyses of lateral capacities of piles and of pile deflections can be made using the elastic approaches developed by Broms (13), Davidson and Gill (14) and Reese and Matlock (15).

During the past two decades, a new methodology has been developed to analyze dynamic response of machine foundations and structures subjected to seismic vibrations. At the low strain levels involved, most material remains in the elastic range; thus, it is no surprise that dynamic analyses of foundations have generated an even greater need for data on the elastic properties of soil and rock. The pioneers in this field have included Hardin, Richart, Seed, Whitman, Lysmer, Hall and Drnevich (see references 16, 17 and 18).

Finally, in the area of pavement design, there always has been a need for some form of stiffness factor for bearing capacity evaluation of the subgrade. Newer pavement design methods which use the computer (such as the CHEVRON and BISTRO Programs) are nearly always based on multilayer elastic theories, as are some of the more simplified

longhand methods. Obviously, the results are only as meaningful as the elastic parameters which are used to characterize the subgrade materials.

The physical and mechanical behavior of soils have been found to vary widely with geographical location in such a complex way that subgrade stiffness cannot usually be estimated with sufficient accuracy on the basis of classification and the use of "typical" values. It follows then that an engineer seeking to use elastic analyses for projects in his area, should not be content to pick values out of a handbook, but should investigate the meaning of the term as applied to his design situation.

1.4 Available Rational Pavement Design Methods

1.4.1 General

In most geographical provinces, the final responsibility for design of highway and airfield pavements resides in some division of a Department or Ministry of Transportation. In the case of highway pavements, construction usually proceeds on a continuing basis from year to year, and design techniques tend to become established routines.

Those engineers whose job it is to actually design pavement sections are likely to be confined by departmental directives to a structured, step by step procedure. The method being used at a given agency may have been developed originally by or for that

agency, or it may have been borrowed (possibly from afar) and modified for local use.

Pavement technology is usually divided into two general categories, that concerned with flexible (asphaltic concrete or bituminous bound) and rigid (Portland Cement concrete) pavements. Hybrid structures, such as those involving stabilizing bases, lime-cement-fly ash sections, or bituminous overlays of concrete pavements often do not fit neatly into either category.

Regardless of the materials comprising a pavement, the characteristics of the subgrade are important. For conventional flexible pavements, the compacted subgrade, along with the aggregate base, is an essential structured element. In the case of rigid pavements, deflections, moments and shears are functions of interaction with the subgrade which acts as a foundation.

Many more sophisticated design procedures have been published in recent years, particularly for flexible highway pavements. These methods are the result of research by universities, industry, government agencies and private engineering consultants, or in most cases, cooperative efforts involving two or more of these entities. The published methods available in 1977 vary greatly in their degrees of sophistication, stages of development and particularly in their relative amounts of field verification. It is almost certain that as more agencies take careful note of

their needs in the area of pavements, the number and complexity of design systems will grow.

While it is not the purpose of this effort to judge the merits of any design procedure, a brief mention of many methods is presented. A further look at these methods, with respect to the ways the various researchers recommend characterizing subgrade response is presented in section 1.8.

1.4.2 Rigid Airfield Pavements

Because of the nature of airfield construction, designs are more likely to be pursued on an individual basis, rather than on a production basis, as is often the case with highways. Factors such as the multiplicity of aircraft types and loading, relative uniformity of subsurface conditions over the length of construction, and intense interest in the project on the part of a handful of intended users, dictate the more individual approval. There are only a few published methodologies, four of which are noted here.

A. Portland Cement Association Method (Reference 19) This thickness design method, presented in chart form, is easy to follow. For several specified aircraft, thickness requirements are presented in terms of repetitions of load to failure for various stiffnesses of subgrade. Subprocedures are available to analyze mixed load of traffic.

B. Corps of Engineers Method (Reference 20)
This method is similar to the PCA method, making use of charts for light, medium, and heavy aircraft. A subsystem for the reduction in subgrade strength during frost action is a major feature.

C. FAA Method (Reference 21)
The charts prepared by the PCA were incorporated into computer programs employing more sophisticated traffic analysis subsystems. Thickness requirements for single gear and dual-tandum gear aircraft are given in terms of a 20-year, 20,000 pass life.

D. Yang Method (Reference 22)
In his textbook, Yang describes an intricate deterministic analysis approach to the design of airport pavements. The subsystems are based on experience gained during the rebuilding of Newark Airport by the Port of New York Authority.

Some of the many subsystems included in the method are, vehicle-pavement interaction models, progressive deterioration models and cost-benefit (fiscal management) models. Heavy emphasis is placed on dynamic testing of materials. While the textbook example is valid only for Newark, Yang states that the models can be tailored and input parameters obtained for other airports. Obviously, the use of this method, or any of its subsystems, can be justified only on major projects requiring in-depth analyses.

The Yang method is one example, of which there are several in the highway pavement area, where the models are highly advanced and seemingly all possible factors related to pavement performance are included. Questions arise, however, as to field verification of the procedure. Also, there arises in the minds of many practical engineers, the suspicion that our ability to conceive and formulate performance models may have outdistanced our ability to provide meaningful input parameters.

1.4.3 Flexible Airfield Pavements

The design methods for flexible airfield pavements are largely empirical and have been in use for many years. As with rigid pavements, a key element is the design aircraft, its wheel load configuration and the traffic mix. Three methods are referenced:

A. Corps of Engineers Method (Reference 23)
This empirical method was developed during World War II and is based on CBR. Modifications have since been made to accommodate the heavier wheel loadings. The method is presented in the form of charts which allow rapid thickness evaluation for different loadings and traffic patterns.

B. FAA Method (Reference 24)
This method also uses CBR and is given in the form of charts which present base thickness and

bitumen thicknesses required for single and dual wheel gear loads.

C. Asphalt Institute Method (Reference 25)
This is a "full depth" asphalt design procedure. It is similar to the Asphalt Institute's method for the design of highway pavements, but special attention is given to the problem of mixed traffic loadings at airfields. Subsystems dealing with the fatigue and deformation properties of asphalt are a major feature. Thus, the Asphalt Institute method is more deterministic than the other two methods.

1.4.4 Rigid Highway Pavements

The economics of construction normally dictate that rigid highway pavements be placed where roads must carry a high volume of heavy traffic. The performance of rigid slabs is dependent to a large degree upon the strength of the concrete and to a somewhat lesser degree on the strength of subgrades.

Probably because of the low pavement mileage built of PCC (compared to flexible pavement mileage), few agencies have developed design methods of their own. The two methods given here (PCA & AASHTO) appear to be sound theoretically and apparently few problems have arisen with their use.

A. Portland Cement Association Method (Reference 26)
The Westergaard stress analysis technique is the

basis for this method. Traffic is divided into load axle groups with provisions incorporated to allow for traffic growth in the future. The concrete strength and thicknesses required for an anticipated traffic load over a 40-year life, can be determined using a tabular procedure.

B. AASHTO Method (Reference 27)
This easy-to-use method was developed in the 1960's, based on results of the AASHO road test in Illinois. Utilizing a set of nomograms, it is a simple procedure to determine a necessary slab thickness based on conditions of traffic, concrete strength and subgrade support for a 20-year life.

1.4.5 Flexible Highway Pavements

Highway pavements utilizing bituminous bound layers, either as structural units or for the purpose of sealing and smoothing the riding surface, are, by far, the most abundant in terms of mileage throughout the world. Accordingly, these have received the most attention of researchers. For many years, empirical techniques and local engineering judgment formed the basis for most designs. Since the 1950's, there has been an increasing effort to replace empiricism with rational methods in the design of flexible pavements.

At the time of this writing, the Fourth International Conference - Structural Design of Asphalt

Pavements, has just been completed. This international gathering of pavement specialists has been held at five year intervals since 1962. At this recent conference, emphasis was placed on complete (or nearly complete) pavement design systems. Methods were presented from every corner of the developed world.

Compared to the pavement types mentioned previously, the number and degree of sophistication of design systems and subsystems dealing with flexible highway pavements seems overwhelming. This abundance of technology is partly due to the universal acceptance of asphalt pavements, but there are other factors. Among them are:

- The constituents and behavioral properties of bituminous mixes cover a very broad range.

- There are several manifestations of distress and failure in asphaltic concrete pavements (fatigue cracking, rutting, low temperature cracking, etc.). Most means of quantifying states of pavement deterioration are, to varying degrees, subjective.

- Some forms of distress are not a function of traffic, but result from aging or environmental attack on the bituminous materials. In some regions, these are important considerations.

- Climate and subsurface environmental conditions vary widely from one location to another.

• The economics of construction (and rehabilitation) are a function of geography and are subject to change with time.

• Traffic loads on asphalt pavements range from a few vehicles a day to many thousands.

• Legal load configuration and tire pressures vary greatly from country-to-country.

At the recent international conference, there was a debate among the practitioners over whether design methods were becoming so complex as to be cumbersome for the average engineer to understand and use. A plea was made that methods must be simplified by the pavement specialists, in order to be accepted by the general engineering community.

Those who advocate sophistication urged complete understanding of all mechanicisms as a basis for future simplification and development of workable techniques. Those who argued for simplification first, stated that few organizations were rushing to take advantage of these new tools. It was noted that there is little benefit in developing even more complicated and expensive methods of analysis, because if they are not widely used, their accuracy will never be verified.

Nevertheless, several of the methods deserve brief mention here and in section 1.8, as they will undoubtedly be in the forefront of pavement design

technology. The first three (VESYS IIM, PDMAP, and Shell) are the most inclusive as of this date. Two others (Chevron Research and Kentucky) are in a form which can be easily used by practicing engineers and were employed in our sensitivity study (section 1.9).

A. VESYS IIM (Reference 28)

The VESYS IIM system is probably the most advanced and all-inclusive flexible pavement design method available today. It has been formulated within the U. S. Department of Transportation-Federal Highway Administration in cooperation with engineers from the Massachusetts Institute of Technology and Austin Research Engineers. The most striking features of VESYS are the models used for materials characterization, particularly the creep-compliance curves which are necessary to describe visco-elastic behavior. Employing the multitude of inputs supplied by the design engineer, computer programs calculate the anticipated performance in terms of rutting, roughness and crack damage.

The method is being implemented on an experimental basis by the Departments of Transportation in Utah and Florida for the purpose of verifying and refining the many subsystems and improving the method as a whole. VESYS IIM is understood in depth by only a few engineers, therefore, its adoption by existing agencies is likely to be slow. One of the biggest problems is said to be

the collection and characterization of field and laboratory data in exactly the way the developers of the various models intended.

B. PDMAP (Reference 29)

A sophisticated computer program has been developed by Woodward-Clyde consultants of San Francisco for the analysis of fatigue and permanent deformations of flexible pavements. Since this firm is a consultant to Arizona Department of Transportation in the development of a pavement management system, the PDMAP procedure is of special importance to Arizona. As is true of most of the newer systems, models to characterize materials behavior are an important feature. The developers admit there are problems with field verification, especially in the area of damage criteria for the fatigue model.

PDMAP has been used on field test information from California and Minnesota where verification continues. Closely allied with PDMAP is another computer model, called COLD, for analyzing low temperature cracking.

C. Shell Method (Reference 30)

Many years of quality research in pavement and asphalt technology at the Koninklijke/Shell Laboratorium in Amsterdam, and elsewhere on the European continent, have resulted in a well documented Shell Method for the design of flexible

pavements. At this time, the method has been refined to the point where it is inclusive, but in a form that can be used without difficulty by most engineers. The method has been reduced to charts, making computers unnecessary. Like all methods, there is a need for further verification of the systems; which is continuously being performed in the field.

D. Kentucky Method (Reference 5)
Probably the most advanced and inclusive procedure developed completely in-house by a State Department of Transportation, is the Kentucky Flexible Pavement Design and Management System. The procedure and its subsystems are available in chart and graph forms. They include models for analyzing fatigue and rutting deterioration of pavements, traffic mixes and economic analyses.

The Kentucky Highway Department has performed several sensitivity studies of their own models and have provided considerable field verification for the method. While it is tailored specifically for use in Kentucky and, therefore, some subsystems may not be applicable in other regions, the procedures are well worth considering.

E. Chevron Research Method (Reference 31)
Probably the simplest method to use while still incorporating sophistication, is the Chevron Research Method. Charts have been prepared to

determine thickness requirements for full depth asphalt pavements based on rutting and fatigue. The major emphasis in the subsystems concerns the properties of various asphalt mixes.

F. Other Design Procedures

Many other pavement design procedures have been advanced here and abroad. The most widely used is the AASHTO method (27). This procedure, which makes use of layer equivalency ratios, has been adopted in some form by many of the state Departments of Transportation. The Asphalt Institute (32) has a widely used method that is still being modified. The National Crushed Stone Association (33) has developed, in cooperation with the Corps of Engineers, a straightforward method which is largely empirical.

Several state Departments of Transportation have published flexible design methods. Notable among these are; Ohio, Kansas and California. The Texas Transportation Institute, in cooperation with independent consultants and university researchers, has published numerous excellent papers dealing with pavement design subsystems.

Research in the area of flexible pavements is increasing abroad also. In Canada, researchers at the Department of Transportation (CDOT) and the University of Waterloo have contributed substantial knowledge. In the United Kingdom, the work being done at the Road Research Laboratory,

Crowthorne, and the University of Nottingham, deserves special note. Other pavement design methods, some very sophisticated, have come from France, Germany, Belgium, Ireland, Denmark, Italy, Hungary, the USSR, South Africa, and Australia.

It is not within the scope of this project to perform more than a general evaluation of the various pavement design procedures or subsystems. Our main interest has been in the various conventions which pavement experts from throughout the world have chosen or recommended to characterize subgrade response.

In most cases the subgrade (soil), as well as other materials in the pavement prism, has been modeled elastically. On the surface this appears to combine sophistication with simplicity. But soil does not behave elastically in most situations and therefore an elastic response model is an approximation of what really occurs. For this reason, there have evolved many ways of defining and measuring soil response for incorporation into design procedures. A look at the current state-of-the-art concerning elastic properties of soils is presented in the following section.

1.5 The Concept of Modulus as Applied to Soils

1.5.1 General

In classical theory of elasticity, the modulus of

elasticity (or Young's modulus) of a material is that numerical value which expresses the proportionality of stress to strain. Graphically, it is the slope of a stress versus strain relationship.

An examination of a typical stress to strain plot, such as given in Figure 1, will show that a single unique value of Young's modulus can be obtained only if the stress-strain relationship is linear. Otherwise, the elastic modulus must be expressed as a function of stress level.

In the real world, few earth materials exhibit a linear stress-strain relationship through a significant range of stresses. It is probably a very rare soil or soft rock which displays a linear response to loading except at very low strain levels.

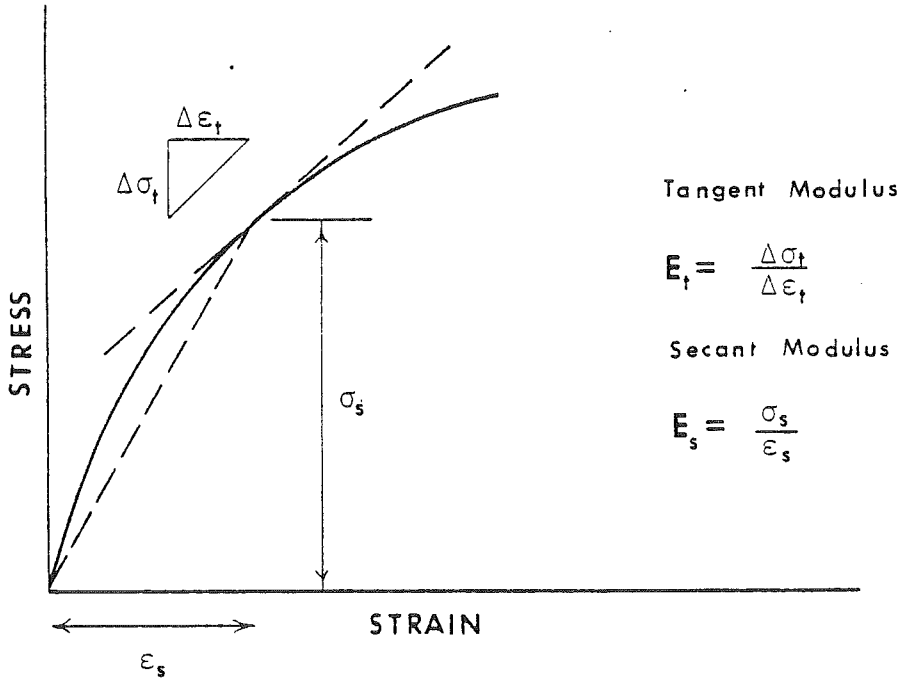


Figure 1. Graphical Representation of Elastic Modulus

Engineering formulas which make use of an elastic model demand a single number to describe what is, in essence, the "response" of the material to being loaded. In most cases, this toys with reality. The response of the material is a function of stress level, f(p). In fact, as will be shown in later sections, the load response is a function of many more than one variable. In addition to the magnitude of stress, the response may be dependent upon the size and shape of the loaded area, the thickness(es) of the loaded layer(s), and the relative stiffness of the various materials in the layered system. In designing a facility, such as a paved roadway, the possibility that the response will change with time or with seasonal variations in environmental factors (i.e., moisture content, temperature) must also be considered.

In the early studies which gave rise to the discipline known as soil mechanics, the three phase (solids, water, and air) component system of soil, along with the particulate nature of the solid component, was recognized and examined. These unique characteristics of soil made necessary an entirely new methodology to describe its mechanical behavior. As opposed to metal, wood, brick, or stone, which could be considered a "continuum", soil was labeled a "discontinuum". Experiments have shown that most materials which are continuums are adaptable to theories of elasticity or plasticity at least through some stress range. However, one may properly question whether a particulate system, such as soil, is

receptive to elastic analyses, especially when subjected to tensile stresses.

The consensus among active researchers in the profession today seems to be that on a macro scale, soil and earth materials do indeed exhibit behavior similar to that of a continuum under many conditions of loading. In contrast, rock, which was assumed to be a continuum and highly adaptable to elastic theory, has been found to behave on a macro scale more as a discontinuum. The engineering behavior of rock in road cuts or large excavations is governed by the natural planes of weakness, such as bedding planes, joints and fissures, rather than by the strength properties of the rock substance.

Returning to the response of earth materials to load, one can see there is a definite anomaly. On the one hand, the engineer seeks to use design techniques which require that the response of the soil to load be expressed as a single numerical parameter much as one should insert a value of bulk density or specific gravity of solids into a formula.

Yet, in reality, the response (hence modulus) is not an intrinsic property, but a highly dependent variable. The engineer knows that he is using expressions which were developed for a "continuum", yet, intuitively, he senses that his three phase particulate system is somewhat different.

On the other hand, engineering is an art which, throughout history, has merged scientific doctrines

with empirical evidence to develop sound and beneficial design procedures. An objective of this study is to seek such an accommodation as can be applied to pavement design in Arizona.

In technical literature, one often sees the term "modulus" (usually in conjunction with a modifier) used rather casually and denoted by any of a myriad of subscripted alphanumeric characters (e.g., E_z, E_o, N_x, D, M_R, G). The burdening variety of names and notations leads the novice to wonder whether or not the authors are talking about the same thing. In general, they are. All of the "modulus" terms are measures of response of a medium or layered system to stress. The confusing multiplicity of terms is due to the many ways in which response can be induced, charted and quantified. Specific engineering analyses require parameters which quantify the likely behavior of materials under a stress system similar to that to which the material will be subjected during its design life. For instance, an analysis of the foundation for a sensitive piece of vibrating machinery requires quantification of soil response under that peculiar type of loading. A long slender pile carrying a lateral load in the same soil would generate an entirely different response from the soil.

In this chapter, some of the more common moduli will be discussed, although the treatment is not intended to (nor could it) touch on every response term one may encounter in the literature.

1.5.2 Compression Modulus (E)

In continuum mechanics, the most commonly used response parameter is simply the ratio of the induced stress to resultant strain termed the "modulus of elasticity", or "Young's modulus", and is universally designated by the character "E". In the literature of soil mechanics, the expression "Young's modulus of the soil" is occasionally found, but because soil is not truly elastic, many writers prefer a substitute term. Modulus of elasticity and compression modulus are most commonly used. These carry the designator E, thus affirming that the parameter is rooted in elastic theory. Farther afield in connotation, though not in physical meaning, is the term "modulus of deformation", sometimes designated as M, which seems to be in little favor of late. The wider acceptance of elastic analyses in soil engineering in recent years seems to have eliminated some of the artificial distinctions of terminology, and there now appears to be no need to apologize for referring to E as Young's modulus of soil.

The compression modulus (E) is expressed in units of stress; strain being dimensionless. In the English system, values are usually given in psi.

Ideally, the E modulus defines the response of an earth material loaded over any area (or loaded at a point); that is, shape and size of the loaded area are not of consequence in the definition. In reality, any load must be applied and measured over a specific

area. The soil whose response is measured may rest under a footing, a plate or a beam in the case of field tests, or it may be a sample of finite dimensions in the laboratory. In the literature there are many correlation equations which seek to eliminate boundary effects in an attempt to provide a more nearly intrinsic property to be called "compression modulus".

An example is the work published by McLeod (34) of the Canadian Department of Transportation concerning the influence of plate size on the unit load causing a given deflection. Another is an empirical equation given by Vesić (35), which will be discussed further in section 1.5.3.

In all such studies, correlations are probably applicable only to the particular soil types which are the object of a given investigation. Universal applicability of such relationships is unlikely; therefore, the need arises to provide a standard compression modulus at a selected stress level and bearing pressure.

Because of the inherent ambiguity surrounding modulus of elasticity, with regard to soil, the character E is often found to be subscripted in the literature. This in itself leads to more confusion, but it is necessary to alert the user to some special constraint which may be important in the use of the parameter.

The most commonly seen subscript is E_s. Unfortunately, the writer knows of three contexts in which this subscript has been used. The most common application of E_s is to denote that the modulus was computed as a secant modulus (as opposed to a tangent modulus; see Figure 1. That is, the response is a measure of total strain, rather than a ratio of incremental values.

In the soil mechanics literature, secant values of modulus are by far the more common and, in this study, E modulus values will be assumed to be secant values except in the regime of very low strains where the initial tangent modulus and secant modulus are virtually synonymous.

E_s is also used (especially in the literature concerning pavements) to specify the material in question is native soil or subgrade, as opposed to the man-made layers in the pavement systems. Moduli of pavement and base course layers are usually designated E_1 and E_2, respectively, and subgrade modulus is E_3 in a three layer system. In this study, the preceding definition of E_s will apply.

A third type of modulus, which sometimes receives the designation E_s, is the seismic or dynamic modulus. This also has other designators as will be discussed later in this chapter.

Many other subscripts (and superscripts) which are found attached to the E modulus in the literature

have meaning only as defined in that specific treatise. In another author's work, the same subscript may be used for an entirely different purpose. A few general examples taken at random from the literature include E_o for the modulus of soil in a completely dry state, E_n for the modulus determined in the unconfined compression test, E_c for the modulus determined in the consolidation test (this probably is the same as D, the constrained modulus to be discussed later) and E_p for the modulus determined using the Menard Pressuremeter. In one respected text, E_f refers to the soil modulus applied to a loaded footing, while in another, E_f is the secant modulus at failure of a triaxially loaded specimen.

While the means of quantifying a stress-strain relationship is essentially the same for all the many subscripted variations of E, the numerical values which are obtained by testing a given material usually vary widely as will be seen in sections 1.6 and 1.7.

1.5.3 Modulus of Subgrade Reaction

Perhaps the oldest, and most widely used, response parameter in soil mechanics is the modulus of subgrade reaction which is nearly always designated k. This parameter has been used for many years in evaluating subgrades for rigid pavement design, and variations of k (namely k_h and its derivative n_h) are very important in the analysis of laterally loaded piles.

Physically, the modulus of subgrade reaction is the ratio of stress to deflection upon a loaded area. Its units of measurement are stress divided by deflection, usually expressed in psi per inch of deflection or pci.

The k modulus was first described by Westergaard, and was called the "dense liquid constant". This name invokes the idea of soil being modeled as a dense liquid, rather than an elastic solid. Presumably then, the soil deforms without volume change.

In more recent writings, k is referred to as the spring constant of the soil. Such terminology envisions a model in which a rigid body transmits load to a flexible soil whose response would mathematically be the same as the spring in the classical Hooke model. In the context of the subject of rheology, where the response of a material is modeled in terms of a spring, sliding mass, and dashpot, the use of k implies a purely elastic system. A visco-elastic system would require a spring constant and a dashpot constant, while an elasto-plastic system would require spring and sliding block constants.

In the early works after Terzaghi, the word "coefficient" was used rather than modulus in connection with k. This has lead to confusion because, in some papers concerned with piles, "coefficient of subgrade reaction" and "constant of subgrade reaction" are used for another purpose. These are discussed later in this chapter.

The k modulus is similar to the E modulus in many respects, and one might successfully argue that they are, in reality, the same thing. The difference lies more in their connotation and use than in what one physically measures in the field.

Historically, k is measured in the field using plate bearing tests. There has been no attempt until recently to tie k to elastic theory. There has always been a recognition that the size and shape of a loaded area strongly influences the k value measured and, as is the case with E, expressions of correlation have been proposed. An example is Terzaghi's (36) expression $k_m = k\ (\frac{m + 0.5}{1.5\ m})$, where k is the modulus of subgrade reaction for a square plate of dimension B and k_m is the modulus for a rectangular footing with dimensions B x mB. Another Terzaghi equation relating modulus to plate size is, $k_s = k\ (\frac{B + 1}{2\ B})^2$ where k is the modulus determined from a 1 foot square plate and k_s is the appropriate modulus for an area of dimension B.

The above equations apply only to sands. For clays, the relationship $k_s = \frac{k}{B}$ is given. Vesić (35) has given an expression for correlating k modulus with footing size based on E_s (Young's modulus of the soil). The equation is: $k'_s = 0.65 \sqrt[12]{\frac{E_s\ B^4}{E_b\ I}}\ \frac{E_s}{1-\nu^2}$

where B is the footing width E_b is the elastic modulus of the footing and I is the moment of inertia

of the footing. Also, $k'_s = k_s B$. This equation seems to indiate that Vesić believes the E modulus to be a much more fundamental parameter of soil than the k modulus.

The problem with all of the above correlation equations is that they probably are valid only for ideal materials. The degree of error (and the consequences) which may result from applying these relationships to a specific earth material will probably remain unknown.

Pursuant to a discussion of modulus of subgrade reaction (k), it is appropriate to discuss briefly the relatively similar terminology used for load-deflection analyses of laterally loaded piles (or soil-pole interaction theory as it is sometimes called). As in other analyses, the concept of (k_x or k_h) as a spring constant of the soil is used. However, since large depths are involved, usually an increase in stiffness of the soil with increased vertical confining pressure must be included in the analysis. For this reason, Terzaghi in 1955 introduced a term called "constant of horizontal subgrade reaction" designated n_h. This term could be applied to those soils (especially sands) whose stiffness increases linearly with depth, i.e., $k_x = k_o + n_h x$; x being depth below the ground surface.

Since Terzaghi's first writings on the subject, there has been no standardization of notation or terminology in the later works on soil-pole interaction theory, and confusion is the order of the

day. Researchers such as Broms (37) and Davidson and Gill (38) have generally followed Terzaghi's (39) conventions (with some additions of their own) but Matlock and Reese (40), who have published voluminous research on this and related subjects, use k for the derivative of the subgrade reaction modulus with respect to depth instead of n_h and use E_s as a spring constant rather than k. In a recent work in the field of load-deflection analysis of piles by Poulos (41, 42), another change in notation and a shift closer to pure elastic analysis can be noted. Poulos' soil stiffness is given in terms of Young's modulus (E) rather than k and n_h is the derivative used to denote the variation of E with depth. Spring constants (k) are not used except in general discussion.

Dimensionally, the terminology of soil-pole interaction theory provides confusion when compared with similar terms from other areas of geotechnical engineering. One would expect the first derivative of the spring constant to be given in units of pounds per inch^4, and a few writers have used this. Most, however, such as Reese and Matlock, prefer to express deflections and spring constants as per unit width of pile. This results in constants of subgrade reaction being given in terms of pounds per inch^3 and spring constants in pounds per inch^2.

It should be evident that the terminology and conventions of soil-pole interaction analyses, while similar to those of other areas of geotechnical engineering, are unique and should be used only

in their own context. An excellent summary of the subject of horizontal subgrade reaction has recently been published by Audibert and Nyman (43).

In conclusion, it is the writer's opinion that in future treatises dealing with foundations, piles or layered pavement systems, modulus of subgrade reaction (k) will increasingly give way to Young's modulus (E). As more sophisticated analyses, especially those dependent upon computers such as finite element, require E moduli to describe the soil's response to load, engineers are becoming more comfortable using a Young's modulus for soil. Apparently a major reason for the increasing popularity of E versus k lies in the belief that E is dependent upon fewer variables than k and, thus, is slightly closer to being an intrinsic property of a given earth material.

1.5.4 Shear Modulus (G)

The shear modulus of soil designated G (and sometimes called modulus of shear deformation) is a useful parameter because of its precise derivation from elastic and vibration theory. Within the range of low strains, G comes very close to being an intrinsic property of most earth materials. Two equations widely used in soil dynamics describe fundamental relationships.

The first defines shear modulus $G = \rho V_s^2$ where ρ is the mass density and V_s is the velocity of shear

wave travel through the material. Most engineers can agree that ρ and V_s are intrinsic properties of a given earth material that can be determined definitively through field or laboratory testing (though with great difficulty in the case of shear wave velocity).

A second equation relates shear modulus to compression modulus.

$$G = \frac{E}{2\ (1+\nu)}$$

It would appear at first glance that this equation provides the means to define E intrinsically as well. One must remember, however, that the shear modulus is measured as elastic vibrations travel through a medium, the shear wave being only one of many waves. Most of the field geophysical techniques in use today produce unit strains which are usually on the order of 10^{-4} to 10^{-6} inches per inch. Thus, the shear modulus and the Young's modulus derived in such a manner are applicable only to that regime of low strains, which for most earth materials is considerably different than the response under a plate or foundation where several tenths of inches of deflection may be involved.

As a possible remedy to the problem of strain level dependence, engineers from Shannon and Wilson (44) have sought to develop an in-situ impulse test. Waves are generated by the free fall of a heavy hammer on a specially designed anchor which is placed in a borehole. They have measured shear wave velocity

at strain levels as high as 3×10^{-3} inches per inch, which is close to the strain produced by strong motion earthquakes. Correlations of shear modulus with elastic modulus at even higher strain levels are needed, however.

For sands, Hardin and Drnevich (45) and Seed and Idriss (46) recommend that G modulus be determined in the field or laboratory using shear wave velocity at the low strain levels, and then reduced by some empirical method to obtain reasonable values of modulus at other strain levels. For saturated clays, G moduli values determined by laboratory and by field techniques were found to be farther apart, yet there seems to be no reason why modulus reduction techniques similar to those used for sands cannot be developed for soils of all types.

1.5.5 Dynamic Modulus

Inasmuch as a large part of the recent research in the field of load response of soils makes use of vibration techniques, it is not surprising to see response parameters with designations such as "dynamic modulus". Unfortunately, the exact meaning and characterization of the term varies widely.

For example, it was previously noted that E_s is used in some works to denote a Young's modulus determined seismically (presumably at low strain levels). In other works, dynamic modulus and shear modulus (G) are used interchangeably. Recent articles by Hall

(47) and Weiss (48) of the U. S. Army Corps of Engineers Waterways Experiment Station use a parameter called the dynamic stiffness modulus (DSM) in the methodology applied to their nondestructive testing of pavements. The DSM is a spring constant and must be determined by field testing as is the subgrade reaction modulus (k).

In essence, dynamic moduli are the same as the response parameters discussed earlier in this chapter. The distinction arises from the type of loading and the range of strains through which they apply.

1.5.6 Resilient Modulus (M_R)

Modulus of resilience (M_R), also called resilient modulus of deformation, is an index of the response of materials to repeated load and is primarily used in the field of pavement design. A specially designed triaxial testing cell is necessary to perform the test. Response properties of both bituminous bound and unbound materials may be determined in this manner.

Resilient modulus is defined as the ratio of deviatoric stress to recoverable strain. Figure 2 provides a graphical presentation of its meaning.

$$M_R = \frac{\sigma_d}{\varepsilon_{rec.}}$$

To be meaningful in design, the value of M_R must be related to a particular value of deviator stress and number of cycles of loading.

The laboratory test is difficult and expensive to run due to the care necessary in sample preparation. There has been increasing interest in the recent literature concerning M_R. Many pavement analysts treat M_R as an appropriate substitute for E to describe the load response of soil, especially a soil subjected to cyclic loading. More will be said concerning triaxial testing in section 1.6.4.

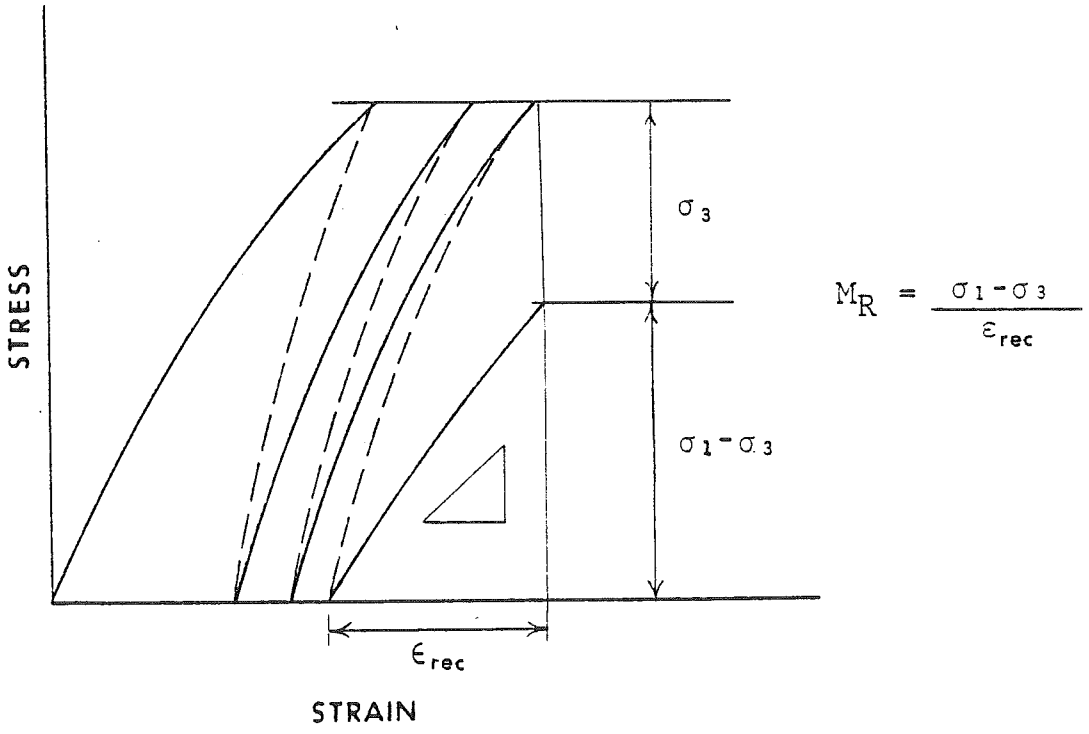


Figure 2. Graphical Presentation of Resilient Modulus

1.5.7 Confined Modulus

Confined or constrained modulus usually refers to the compression modulus derived from the one-dimensional consolidation test, or any test in which volume change is permitted in one direction only. This modulus receives various designations in the literature, most commonly E_c or D. It is very important for the engineer to recognize that since real world loading situations result in volume change in three directions, the constrained modulus must be corrected. From elastic theory comes the relationship

$$D = \frac{E\ (1-\nu)}{(1+\nu)\ (1-2\nu)}$$

By inspection, it can be seen that the conversion is heavily dependent upon the value of Poisson's ratio; thus, the engineer desiring to use this expression must have a feel for the ν of his material.

1.5.8 Bulk Modulus

Occasionally in the soil mechanics literature, bulk modulus (B) or (K), also called modulus of volume compressibility, is found. This term is an index of percent change in volume of a material in response to a given load. It is defined as $B = \sigma/\Delta v/v$. Where volumetric strain is equal in all three directions, elastic theory provides that $B = \frac{E}{3\ (1-2\nu)}$. Percent change in volume can be measured only on a sample

of finite dimensions and the equalization of strains in all directions is most difficult to achieve. For these reasons, bulk modulus is seldom used in engineering analysis.

1.5.9 Complex Modulus

Complex modulus $|E^*|$ is a special term used in asphalt technology to describe the stress-strain relationship of a visco-elastic material under simulated traffic in the laboratory. Its use is described by Papazian (49). Essentially, complex modulus compares maximum stress to maximum strain, which in a pulse or sinusoidal load situation may not occur at the same instant. Apparently, complex modulus is not used in connection with earth materials.

1.6 Determination of Modulus in the Laboratory

1.6.1 General

Virtually any test in which load and deformation are measured can provide a means of calculating a value of elastic modulus of the soil. Unconfined compression, uniaxial compression, plane strain and triaxial testing are well known in soil mechanics, and are often used to determine and compare modulus values.

What the engineer is attempting to accomplish is to solve a boundary value problem in soil mechanics (in this case the layered pavement) by inserting a laboratory determined parameter which hopefully simulates

in-place soil behavior. On large jobs involving great distances, such as highway pavements, the production conscious engineer would prefer to use the simplest procedure which yields acceptable results. Several comparison studies have been performed whose objectives were to determine modulus for a particular soil type by several methods and compare them with field experience. A few of these studies merit brief mention.

Forsyth, et al (50) from the California Department of Transportation, investigated the response of a stiff residual clay (actually a weathered shale). Moduli were back calculated from plate bearing tests and then also determined by means of unconfined compression, triaxial, consolidation, Menard Pressuremeter and seismic techniques. Unconfined compression and triaxial tests resulted in moduli too low, probably because of sample disturbance. The other test methods produced a wide range of stress dependent values.

Krizek and Corotis (51), investigated the response of well graded sand-gravel and sandy silt mixtures in laboratory tests and compared them with the response of compacted fills of the same material. Values obtained from uniaxial compression, plane strain, conventional triaxial and true triaxial tests showed that modulus was highly dependent on the stress path and type of test. For the compacted granular soils, dry density and stress level were the most important variables affecting modulus.

Shields and Bauer (52), investigated the response of a sensitive clay both in the laboratory and in-situ. They found that for this particular soil, there are two ranges of E_s values. An upper range of values was obtained from consolidation, triaxial and plate bearing tests; a lower range from unconfined compression and pressuremeter tests. These results contrast with those of Forsyth, although it is virtually impossible to compare results of testing involving different laboratories and different soils.

Thompson and Robnett (53), sampled fine grained soils from sites throughout the State of Illinois. Attempts were made to correlate resilient modulus from the triaxial test with soil classification and CBR tests. In their initial report, correlations were found to be not completely satisfactory.

It has been found by virtually every investigator who has undertaken the task of studying the load response of earth materials, that modulus values must be qualified as to the conditions of the test and the means of interpreting the data.

1.6.2 Classification Tests

There is a natural tendency for engineers to measure those properties of materials which are easy to measure in order to obtain a maximum amount of information for a given effort. Soil classification tests, dependent on grain-size distribution and plasticity index, are part of nearly all soil investigations.

This is partly due to the usefulness of these parameters, but also because they can be scheduled for high volume production.

It is not surprising, therefore, that many designers seek a correlation between Unified, AASHTO, or FAA soil classifications and elastic modulus. An example of such correlations is the graph given in Figure 3, which has been published by the Portland Cement Association. It can be immediately seen that the correlations allow wide latitude in interpretation. They reflect the general observation that coarse granular soils, such as SW, GW and GP, are stronger than CL, ML and CH soils.

Such correlation charts may be acceptable where very imprecise and conservative designs are to be used, but it should be obvious that they are fundamentally inadequate. We have previously stated that modulus of soil is a function of many variables, such as stress level, size of loaded area, and particularly, moisture content. In no way does a classification test account for these. This is particularly true in Arizona where naturally occurring lime cementation is often present, acting to strengthen dry soil. Classification tests have no inherent way of accounting for the stiffness of such soils.

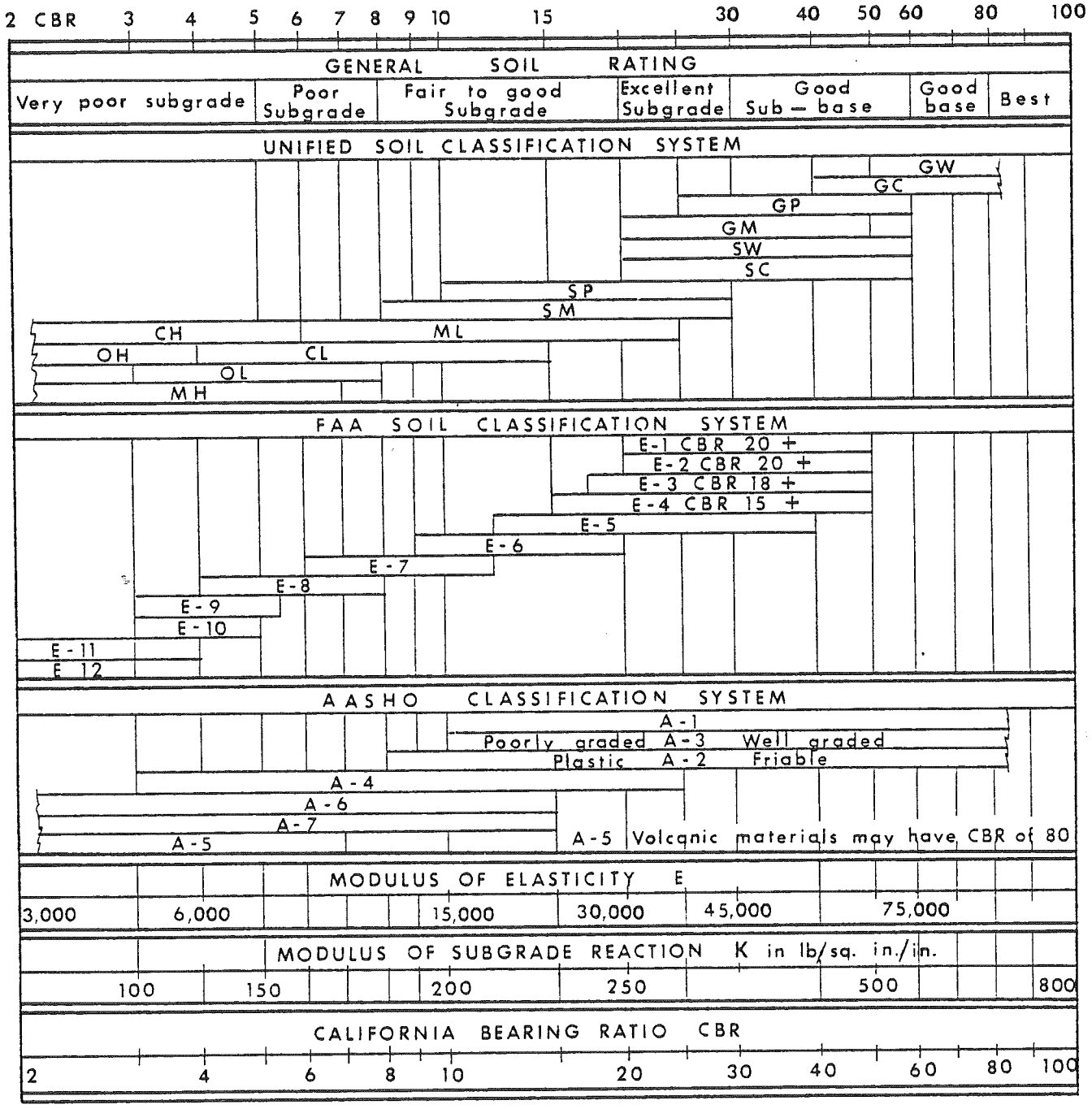
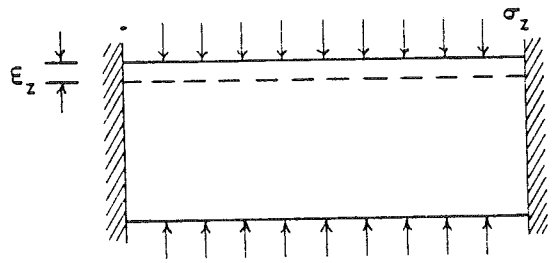


Figure 3. Correlation of Modulus Values with Soil Classification (after Reference 19)

Attempts have also been made to correlate modulus with relative density or void ratio in granular materials. An example is Figure 4, taken from a textbook by Richart, Hall and Woods (54). The problem with such a graph is that it applies only to very dry sand, probably only to uniformly graded sands, and is applicable only at very low levels of strain. This last problem can be partly overcome by applying strain-reduction factors such as given by Seed and Idriss (46) in Figure 5.

1.6.3 Uniaxial Compression Tests

As mentioned in section 1.5, the confined modulus D of a soil sample can be determined from the familiar one-dimensional confined compression test.



$$D = \frac{\sigma_z}{\varepsilon_z}$$

Figure 6. Uniaxial Compression Test

This is shown in Figure 6. This constrained modulus may be converted to the consolidation elastic modulus E_c by the formula

$$E_c = \frac{(1+\nu)\ (1-2\nu)}{(1-\nu)}\ D$$

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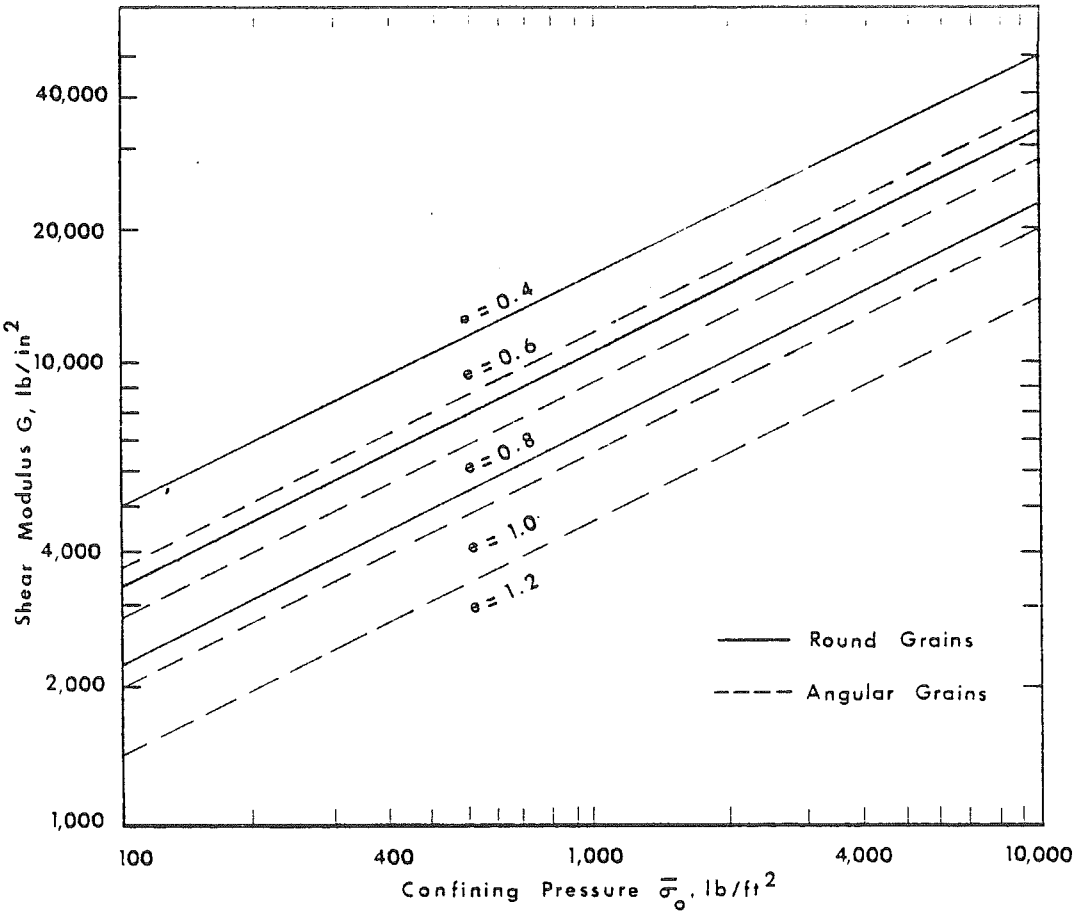


Figure 4. Variation of Shear Modulus of Dry Sands with Void Ratio and Confining Pressure (after Reference 54)

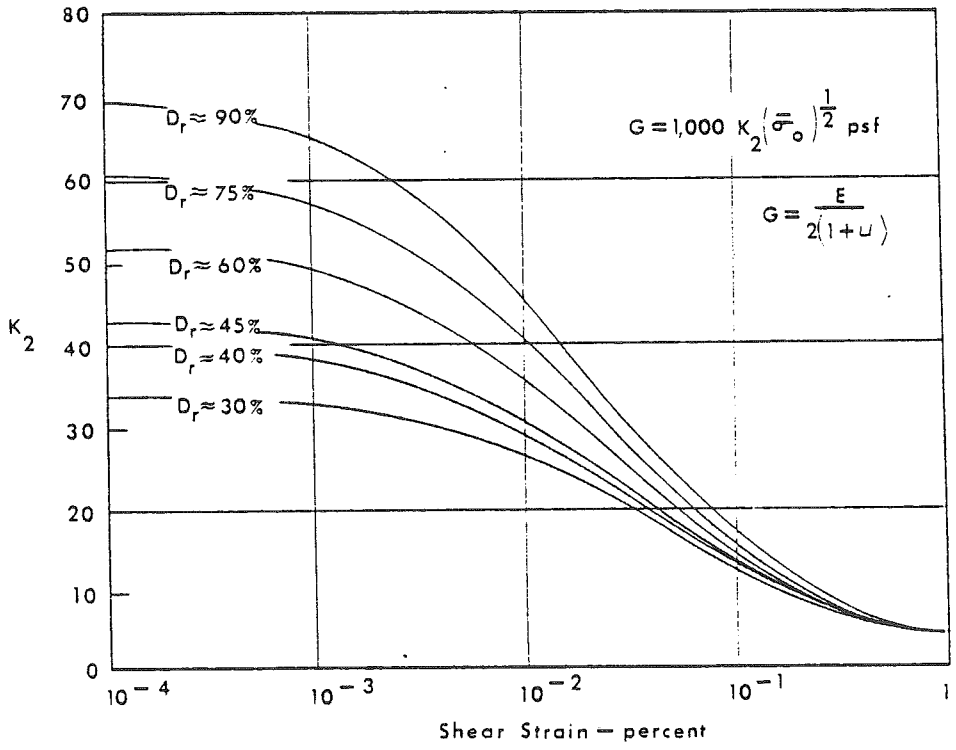


Figure 5. Shear Moduli of Sands at Different Relative Densities (after Reference 46)

An immediate disadvantage can be seen with this method. A proper conversion from the one-dimensional modulus to the three-dimensional modulus requires a very precise value of Poisson's ratio. Values of Poisson's ratio, accurate beyond the first decimal place, are very expensive to obtain, and then may be applicable only to a limited range of strains.

Balakrishna Rao (55), performed a laboratory investigation of the response of partially saturated soils in one-dimensional compression. Five typical soil types were examined, Ottawa sand, concrete sand, lean clay, fat (highly plastic) clay, and sandy clay. It was interesting to note that because of the boundary conditions imposed by the test, increasing degrees of saturation resulted in higher modulus values, probably because of limited drainage. In a field situation, the opposite would nearly always be true. Increased moisture results in increased deformation for a given load.

The uniaxial consolidation test is not recommended for determination of modulus values for pavement design for reasons stated above; namely the problems involving Poisson's ratio, the fact that the stress field and drainage conditions imposed in the test are unlike those in-place, and for many soils, because of the difficulty in obtaining a representative undisturbed sample.

1.6.4 Unconfined Compression Test

For firm, cohesive soils or cores of intact rock, the stress-strain relationship and E modulus

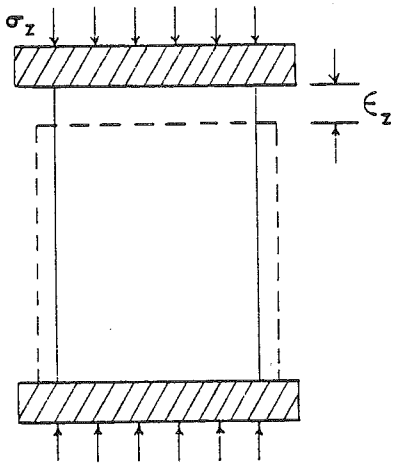


Figure 7. Unconfined Compression Test

can easily be determined from the unconfined compression test. With precise measurements of lateral strain, Poïsson's ratio can also be determined. Obviously, a disadvantage of this method is that it can only be performed on cohesive materials.

Unconfined compression tests were part of the previously cited studies by Shields (52) and Forsyth (50). Both reported low modulus values (compared to other methods) which may seem strange inasmuch as stiff samples are involved. The low values reported may be the result of the strain level or of the stress path. They may also be the result of the absence of confining pressure.

The absence of confining pressure in the unconfined compression test, like the total confinement imposed by the consolidometer, creates a situation which is not in accord with in-situ conditions. It is generally assumed that modulus increases with confining pressure, though more rapidly for sands than clays, see Figure 8.

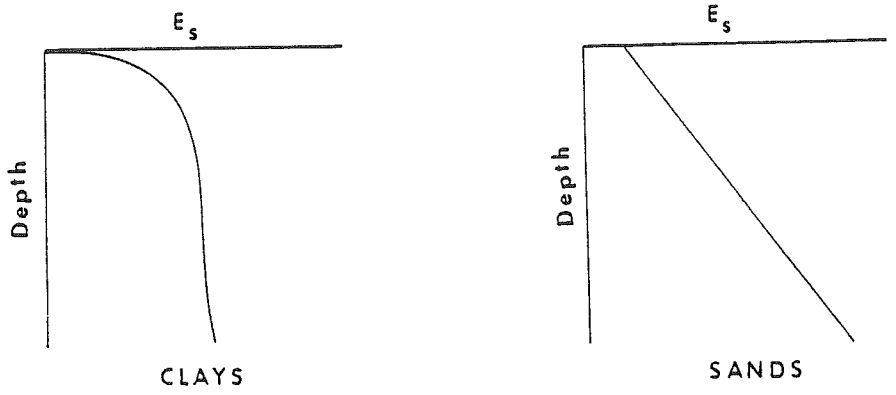


Figure 8. Effect of Confining Pressure on Subgrade Modulus

For this reason, triaxial testing is the preferred "sophisticated" means of determining modulus in the laboratory.

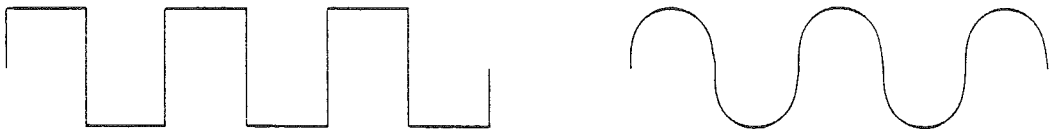
1.6.5 Triaxial Testing

A value of elastic modulus can be obtained from the conventional triaxial test in the form

$$E = \frac{\sigma_1 - \sigma_3}{\varepsilon}$$

This is not, however, the usual means of presenting triaxial data for use in pavement design technology.

The triaxial cell has been adapted for cyclic (dynamic) testing of pavement materials under conditions which simulate repetitive traffic loading. The loading program may be in the form of discrete pulses (square waveform) or a sinusoidal load of selected frequency and amplitude.



9a. Discrete Pulse 9b. Sinusoidal

Figure 9. Types of Loading

The procedure was apparently first established by Hveem (56) as a way of describing the nonlinear load response of pavement materials.

As discussed in section 1.5.6, modulus is given as M_R, the resilient modulus, which is the ratio of the repeated deviator stress to recoverable strain (see Figure 2).

It should be noted that the resilient modulus has been pursued with regard to both bituminous bound materials as well as unbound materials. In fact,

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the volume of test data for bound materials far outweighs that for soils. In many pavement design methodologies, M_R is considered an appropriate substitute for E_s, for all layers in the prism.

In the past ten years, a voluminous amount of laboratory data has been reported involving dynamic triaxial testing. The California DOT/University of California, Berkeley, and the U. S. Army Corps of Engineers Waterways Experiment Station have been most actively involved. Generally, several tests are performed on a given sample of material so that a regression analysis can be made.

For a given number of cycles of loading, resilient modulus is expressed in either of two ways

$$M_R = K_1 \sigma_3^{K_2}$$

or;

$$M_R = K_1 \theta^{K_2}$$

σ_3 is the confining pressure, and θ = sum of the three principal stresses = $\sigma_1 + 2\sigma_3$. The use of θ is given as a function of the total stress regime.

Many investigators have published values for K_1 and K_2 for various materials. A recent compilation is available in a report by Chou (57) for granular materials.

Chisolm and Townsend (58, 59) reported that $M_R = 2570\ \theta^{.865}$ for gravelly sand where θ is less

than 100 psi. For clays, M_R is always 5 to 12 times the static unconfined compression modulus. This is because resilient modulus measures only recoverable strain, while the modulus calculated from an unconfined compression test includes both elastic and plastic deformation.

Resilient modulus may be plotted as a function of deviator stress or number of cycles of loading

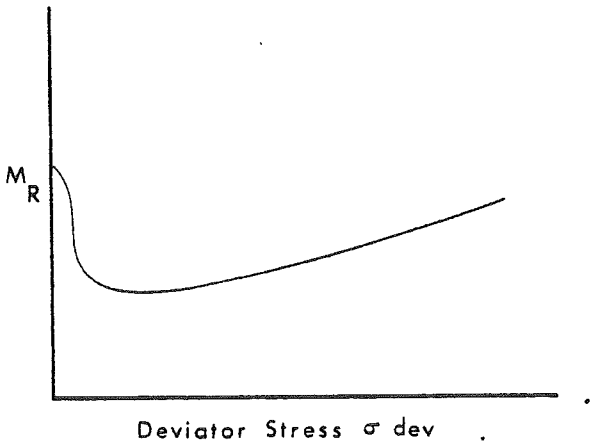


Figure 10a. Typical Relationship of Resilient Modulus as a Function of Deviator Stress

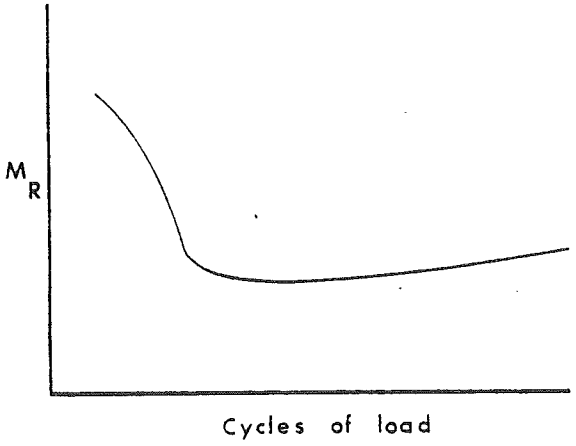


Figure 10b. Typical Relationship of Resilient Modulus as a Function of Cycles of Load

For sands, the plots appear similar. The question arises as to the deviator stress level and number of cycles of loading which are appropriate for typical pavement design problems. Kalcheff and Hicks (60), report that for sands, 50 to 100 cycles is sufficient to characterize the materials response. Probably, a deviator stress in the range of 20 to 30 psi is realistic for pavement design.

In summary, dynamic triaxial testing is the most sophisticated means of characterizing subgrade response in the laboratory today. It will undoubtedly be the subject of more research in the future. The advantages and disadvantages of using resilient modulus can be summarized as follows:

Advantages

- M_R is defined as a function of stress ($M_R = K_1\theta^{K_2}$) and therefore carries no pretense of being an independent variable, as does E_s. The simple function shown above is almost as easy to use in a computer program as a fixed value.

- The use of M_R and repetitive testing incorporates the dynamic nature of the problem. The stress path is more like that which occurs in the prototype structure.

Disadvantages

- Repetitive triaxial testing is expensive. In order to define K_1 and K_2, it is necessary to prepare and test several specimens. This is probably not practical on a production basis.

- In any procedure involving the testing of several samples, there is likely to be random scatter in the test results. Unless the body of data is large, a realistic median value cannot be determined.

- There are boundary limitations in the triaxial testing apparatus which may introduce discrepancies

which render the model a poor representation of the field situation. An example might be the effect of size of the loaded area. Another is the strain softening effect displayed by many soils under plate loading. In reality, modulus decreases as failure approaches, but the equation $M_R = K_1\theta^{K_2}$ indicates that modulus increases with increasing stress unless K_2 is negative.

Within the elastic stress range, the above equation is useful only for appropriate confining pressures.

1.6.6 R Value

R value, also called stabilometer value, is an index developed by the California Department of Transportation and used in many of the states which employ the AASHTO pavement design system.

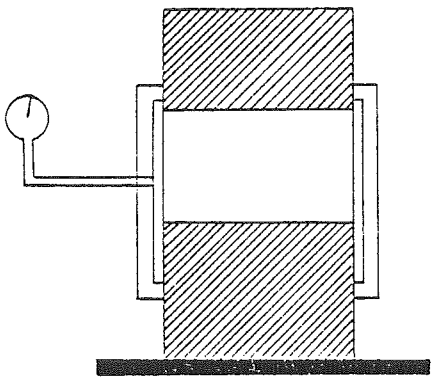


Figure 11. Stabilometer Device

R is basically a measure of resistance to deformation of a sample of material under an applied vertical pressure of 160 psi. As is the case with CBR, most laboratories serving pavement design agencies are equipped to run R values on a volume basis. Obviously, it would be desirable to have a correlation between R value and E_s.

Figure 12 shows the correlation used by the Arizona Department of Transportation. R is correlated with k, E, and Soil Support Value (SSV) which is a function of index parameters. As stated in section 1.6.2, field verification for such correlations is generally lacking. Because of problems involving boundary conditions, sample disturbance and stress path, correlations between laboratory indices and in-place response of soils are very subjective at best.

Apparently few investigators have scrutinized R value closely. Nielson, et al (61), reported correlations between k and R for clayey soils. Using a least squares curve fitting technique, they reported that $k\ (pci) = 0.401 + 2.546\ R - 0.042R^2 + 0.0008R^3$.

1.6.7 CBR

The California Bearing Ratio (CBR) was adopted by the California DOT before World War II and subsequently abandoned in favor of R value. CBR is used today by a number of agencies however, including the Corps of Engineers.

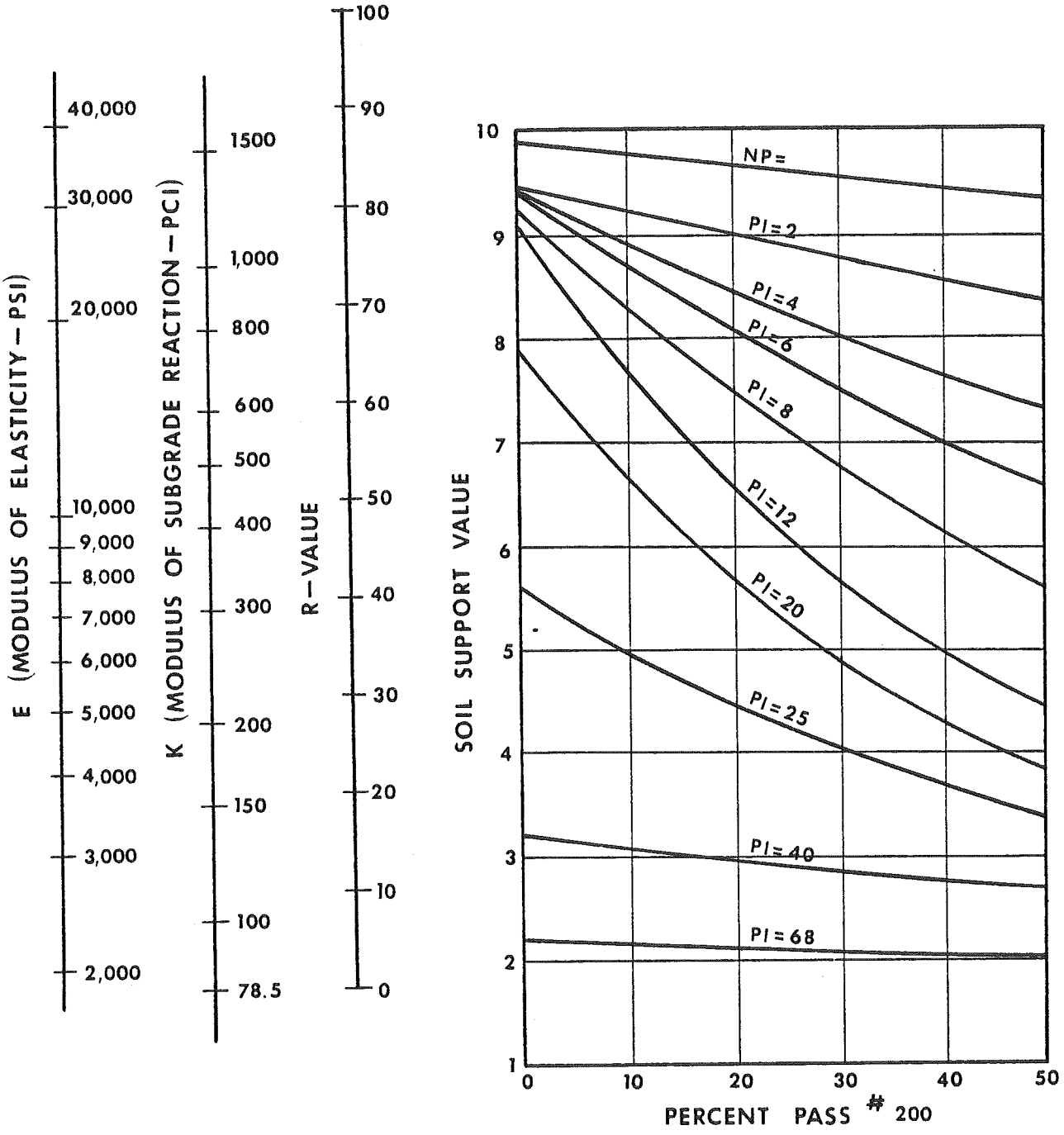


Figure 12. Soil Strength Correlation Chart used by Arizona Department of Transportation

The CBR is a penetration test. Basically, it is the ratio of the load required to cause penetration of a 3 square inch circular plunger into the soil at a specified rate compared to the load required to cause penetration at the same rate of a reference material. The test is performed both in the laboratory (in a compaction mold) and in-place in the field. Theoretically, the results should be the same, but, in reality, the confining effect of the mold may affect values for some very coarse grained materials. The samples are usually tested at a moisture condition simulating the worst possible design consideration, namely, saturation of the subgrade.

Throughout the literature concerned with pavement design, one sees the correlation attributed to Heukelom and Foster (62), that E_s = 1500 x CBR (in psi) or E_s = 10 x CBR (MN/m^2). This relationship has rarely been challenged. In actuality, Heukelom and Foster reported that E ranged from 700 to 2800 times the CBR value. For AASHTO road test subgrade materials (an A-6 soil), Marek and Dempsey (63) reported that E = 2640 x CBR in the unfrozen state. Nielson, et al (61) investigated in depth the problems of correlating E_s with CBR and R value. They found that for granular soils in the Las Cruces, New Mexico area, at 0.1 inch penetration

$$E' = 312 \text{ CBR (psi)}$$

E' is a modulus of soil reaction used in culvert analysis and is equal to 1.5 x D, the constrained

modulus. This would correspond to the following set of correlations.

For ν =	0.1	E_s = 203 x CBR
	0.2	E_s = 187 x CBR
	0.3	E_s = 154 x CBR
	0.4	E_s = 97 x CBR

Obviously there is some discrepancy among what the various investigators have found.

Kirwan and Snaith (64) state that, with regard to Irish glacial tills, considerable error resulted from trying to use a fixed ratio between CBR and resilient modulus. Likewise, Thompson and Robnett (53) reported that fine grained soils of similar CBR may exhibit widely varying values of resilient modulus.

Pichumani (65), in testing several airfield pavement subgrades, noted that the relationship E = 1500 x CBR generally resulted in much higher values of E than were derived from plate bearing tests. The major problem with CBR may be that the piston area in contact with the subgrade is so small (3 square inches) that perimeter/area ratio effects are dominant and, in many cases, mass behavior of the subgrade is not being analyzed.

1.6.8 Resonant Column

As will be discussed in detail in section 1.7, seismic velocities provide a means of calculating low strain elastic modulus values. Normally, field

techniques are employed to determine shear wave velocity. (From this, G and E can be calculated if Poisson's ratio is known or assumed.) The desire for sophisticated laboratory techniques has led to the development of the resonant column.

Anderson (66) performed extensive laboratory work involving the shear velocity of cohesive soils. He noted that strain amplitude and number of cycles of loading are fundamental independent variables upon which V_s depends. Anderson also found that laboratory values of V_s were always lower than the field (in-situ) values for the same material. This was confirmed by Stoke and Richart (67) who noted that resonant column values of V_s significantly underestimated the field values. It was also noted that V_s increased with vertical confining pressure. Similar results were reported by Cunny and Fry (68). Comparative values of in-situ and laboratory determined shear and compression wave velocities from 14 widely placed sites in the continental United States showed that laboratory moduli ranged within $\pm$ 50 percent of the in-situ moduli. Resendiz, et al (69) published work dealing with E modulus of saturated clay in Mexico City. Their conclusion was that only upper and lower bounds of moduli can be determined from laboratory tests.

In summary, it must be said that the use of any of the laboratory methods described in this chapter for the determination of modulus involves some compromise and will probably require some adjustments

if meaningful values of modulus are to be used for pavement design. The biggest drawbacks to laboratory testing are:

- size of specimen - invariably one uses a smaller specimen than desirable.
- difficulties in modeling correctly the boundary and environmental (i.e., soil-moisture interaction) conditions which are critical to the response of subgrades in the field.

1.7 Determination of Modulus In-Situ

1.7.1 General

It should be apparent by now that load response parameters (i.e., moduli) of earth materials are very sensitive to a number of factors. A given native deposit of soil or weathered rock could reasonably be assigned elastic modulus values spanning an entire order of magnitude, depending on the geometry, size and path of loading and on possible changes in the soil-moisture regime. The engineer seeking a k value or an E value for use in a specific analysis should give full consideration to the practical significance of these factors.

A modulus value should be calculated or chosen based on a testing procedure which closely simulates the geometry and level of deflection of the design product during its operating life. For instance, if a value of E is needed for the analysis of a vibratory

machine foundation, a modulus determined dynamically at low strain level should be used. On the other hand, for pavement design, a modulus compatible with the shape and magnitude of the design wheel load is appropriate.

Experience with local conditions is essential to any engineer dealing with pavements. In almost every project where load tests were performed, or postconstruction settlement records are available, it is possible to back calculate modulus values. Likewise, a pre-design field testing program to characterize subgrade response on major projects should be encouraged. The money involved in testing will likely be overshadowed by the savings in construction costs.

1.7.2 Settlement Records - Shallow Foundations

Values obtained from programs set up to monitor settlement of structures with shallow spread-type footings of any common shape (rectangular, circular or continuous) may be inserted into appropriate equations for the deflection of loaded areas on or below the surface of an infinite elastic media. Compilation of such equations may be found in numerous references, such as in chapters 3, 4 and 5 of "Elastic Solutions for Soil and Rock Mechanics" by Poulos and Davis (70), chapter 6 of "Foundation Engineering" by Leonards (71), chapter 14 of "Soil Mechanics" by Lambe and Whitman (72), chapter 5 of "Foundation Analysis and Design" by Bowles (73), and recent

papers by Mitchell and Gardner (74) and Poulos (75). In all cases, a value for Poisson's ratio of the soil must be known or assumed.

At first glance, it may seem that the preceding references contain a plethora of expressions and associated graphs and tables which have been taken from elastic theory and adapted for soil mechanics. In fact, most are of the general form $E = \frac{(1-\nu^2)p}{\rho} I$ where ρ is the settlement, p is the applied pressure and I is an "influence" factor which takes into account such variables as size and shape of the loaded area and the relative rigidity or flexibility of the loaded plate or footing.

1.7.3 Settlement Records - Pile Foundations

Modulus can be back calculated from the results of pile load tests. For contact piles built to develop both friction and end-bearing, elastic techniques developed by Poulos and Davis (75, 76) can be used.

Uncertainty arises in the interpretation of the resulting modulus. Since a thick stratum of soil is involved, the question becomes one of where the computed modulus applies. The analysis was developed for a medium whose elastic modulus is constant with depth. Thus, for a layered soil, or a soil whose modulus increases as the vertical confining pressure, such an analysis may not be reasonable. On the other hand, it may be within reason to draw a pressure bulb around the pile and use stress analysis to

locate a virtual point (a point where the increase in stress is about .5 times the maximum pressure increase) and designate that the computed value of modulus be assigned there.

Nevertheless, pile load tests, while valuable for the information they give, are probably the most indecisive means of determining the subgrade modulus.

1.7.4 Plate Bearing Tests

Probably the most practical means of obtaining modulus values in-situ as input parameters in foundation or pavement design systems is the plate bearing test.

Where a plate bearing test is performed with circular plates directly on an "elastic subgrade", the general Boussinesq equation is applicable

$$E = p \frac{(1-\nu^2)\ 2aI}{\rho}$$

where a is the plate radius. I, the influence factor, is generally given to be 1.0 for the center of a flexible plate (uniform load) and $\pi/4 = 0.79$ for a completely rigid circular plate. Bowles (73) publishes a solution where I (rigid) is given as high as 0.88. In this report, due to the loading geometry used in the performance of plate bearing tests, modulus will be calculated using $I = 0.80$ for plates smaller than 18 inches and $I = 0.85$ for plates 24 and 30 inches in diameter where some relaxation of

rigidity was likely. Compared to the changes in moduli with stress level, the differences due to influence factors are moot.

The only substantive difference between moduli determined from plate bearing tests and those from settlement data lies in the time element. Readings from plate bearing tests usually provide only primary settlement response, while settlement values from structures taken over a long time frame will incorporate secondary compression and creep potential of the soil.

Where a plate bearing test is imposed on a layered system, such as a pavement, modulus of the subgrade may also be back calculated, although a multilayered elastic analysis must be used. Several suitable methods for hand calculation are available in chapter 6 of Poulos and Davis (70). The drawback to multilayer analysis is that to calculate a modulus for the third layer, compression modulus for the upper two layers must be assumed. A sensitivity study using one of the typical three layer equations, however, showed that the final result is not sensitive to the value chosen for E_1, and only moderately sensitive to E_2 (see Figure 17).

The writer has analyzed, for the purpose of determining modulus, plate bearing tests performed by this firm at four existing airfields in the Southwest. These are given in Figures 13 through 16 and A-1-1 through A-1-8. In these, the general

dependence of modulus upon stress level is evident; in most cases, secant subgrade modulus decreases significantly as higher loads are applied.

In all cases, a three layer elastic system was assumed and Vesić's (77) method was the basis for calculation of subgrade modulus. A Jones' (78) analysis was used to calculate stress levels on the interfaces.

Other multilayer elastic methods were checked for comparison. Vesić's method was chosen, not because of greater sophistication or accuracy, but because the method is most easily adapted to the programmable calculator.

In only one case was data available to compare E_s, determined by measuring deflection of a plate placed on a pavement (multilayer analysis), with values of E_s determined from a plate directly on that subgrade. Such data was available in one spot at Grant County Airport. As can be seen in Figure 13, the correlation is fair, hence, plate bearing tests on pavements (which are more frequently employed because of the costs and inconvenience of digging and sealing a pit on an operational roadway or runway) do provide an acceptable means of quantifying load response of a subgrade.

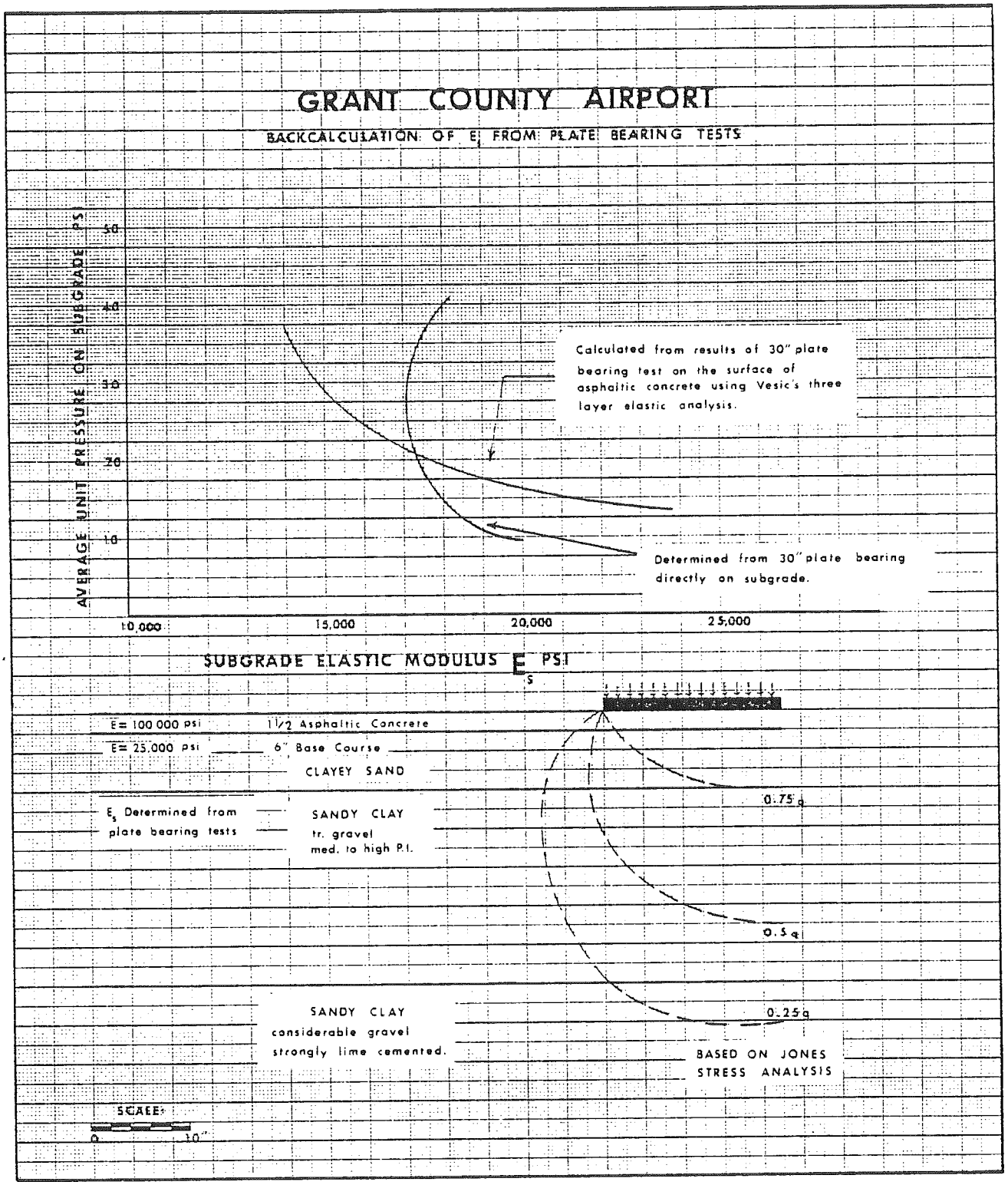


Figure 13. Results of Plate Bearing Tests
Grant County, New Mexico Airport

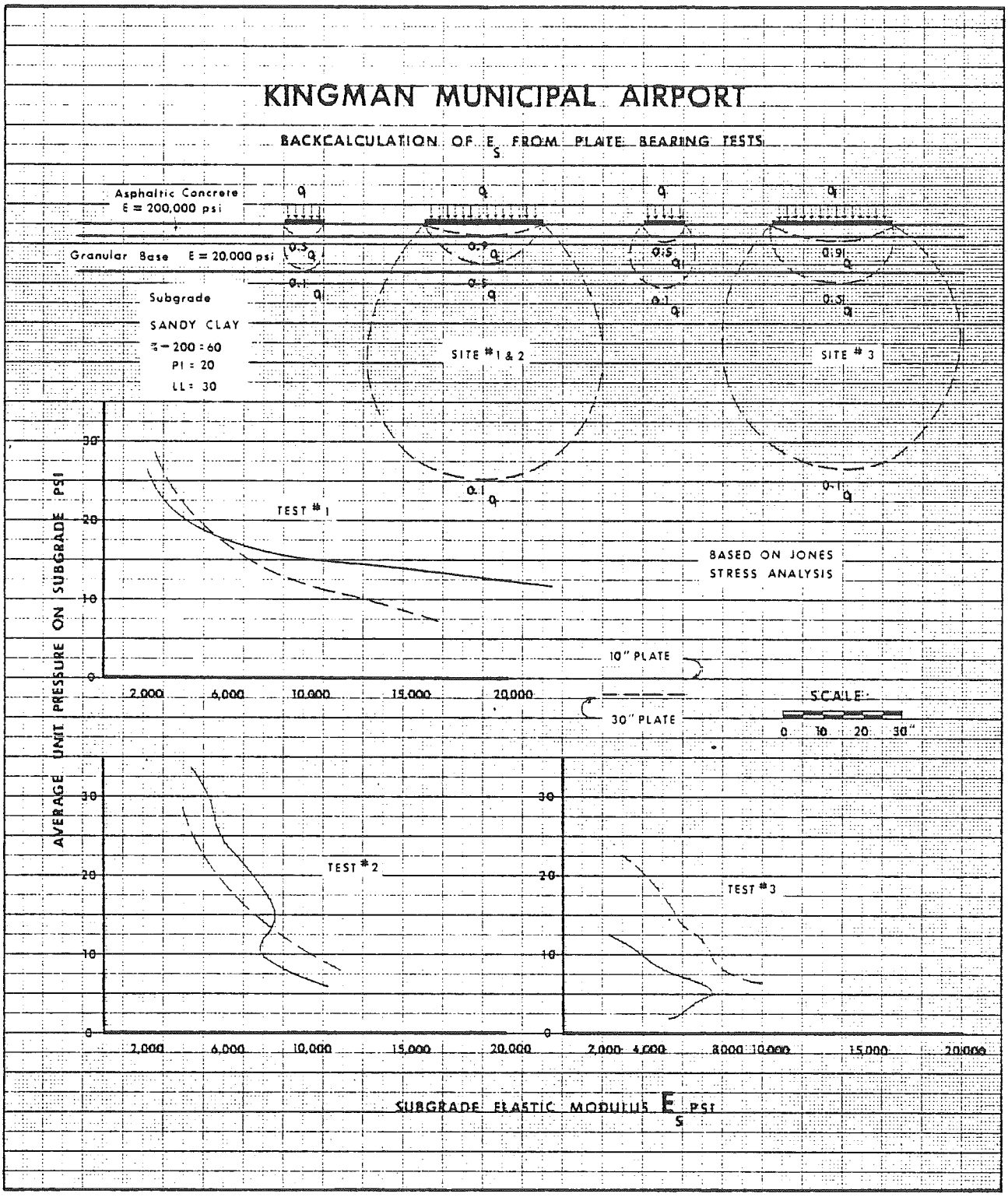


Figure 14. Results of Plate Bearing Tests
Kingman, Arizona Airport

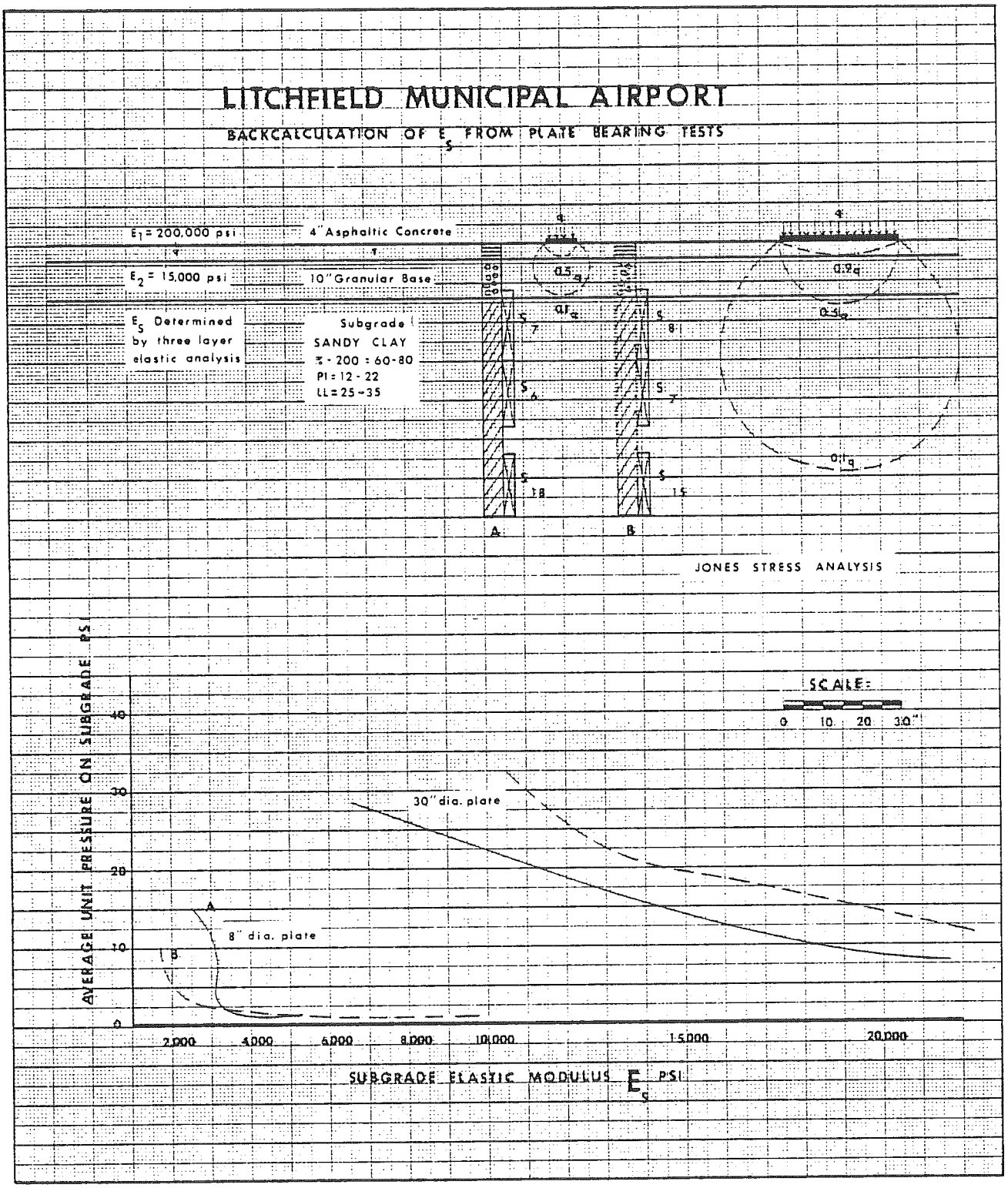


Figure 15. Results of Plate Bearing Tests
Litchfield Park, Arizona Airport

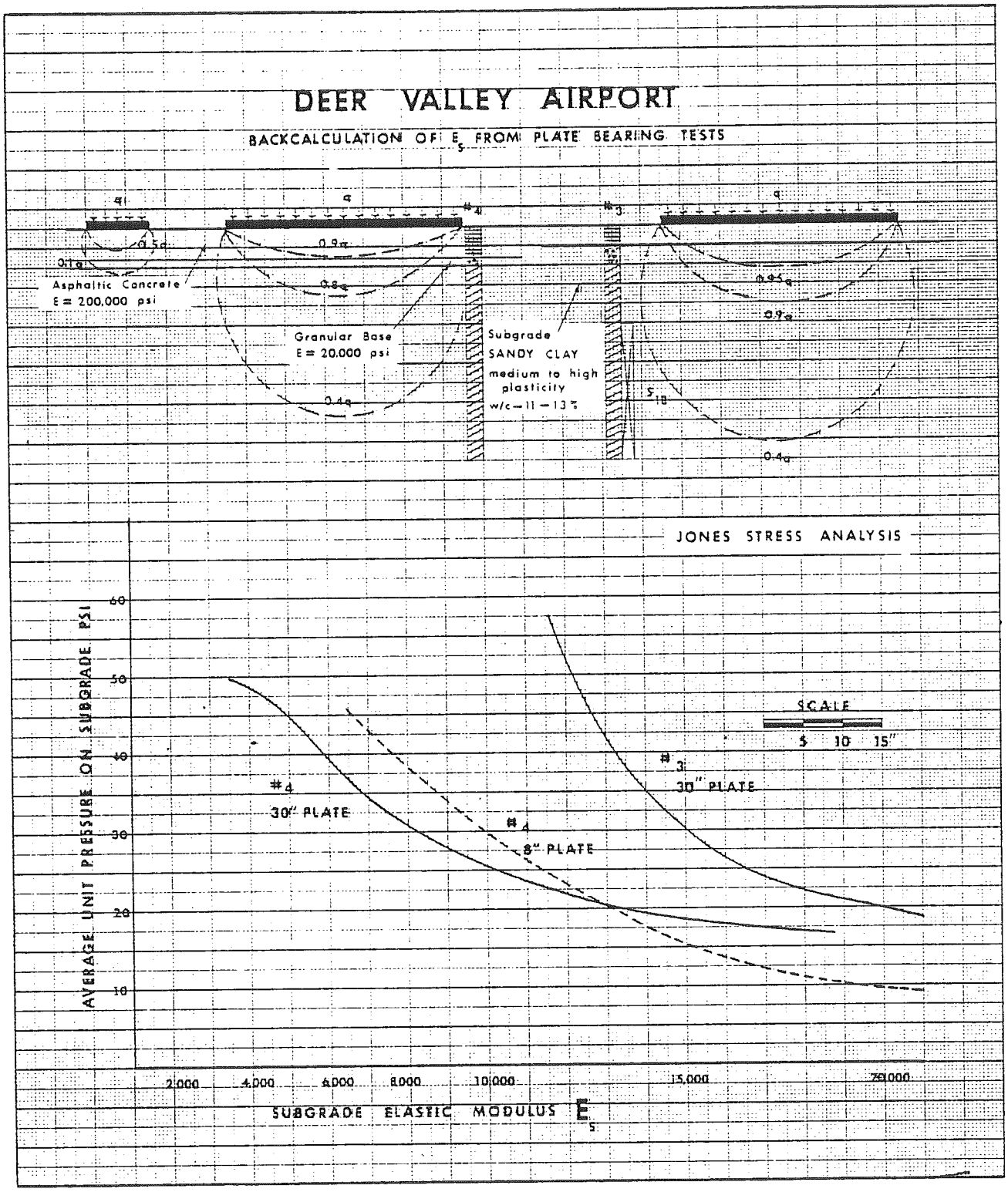


Figure 16. Results of Plate Bearing Tests
Deer Valley Airport, Phoenix, Arizona

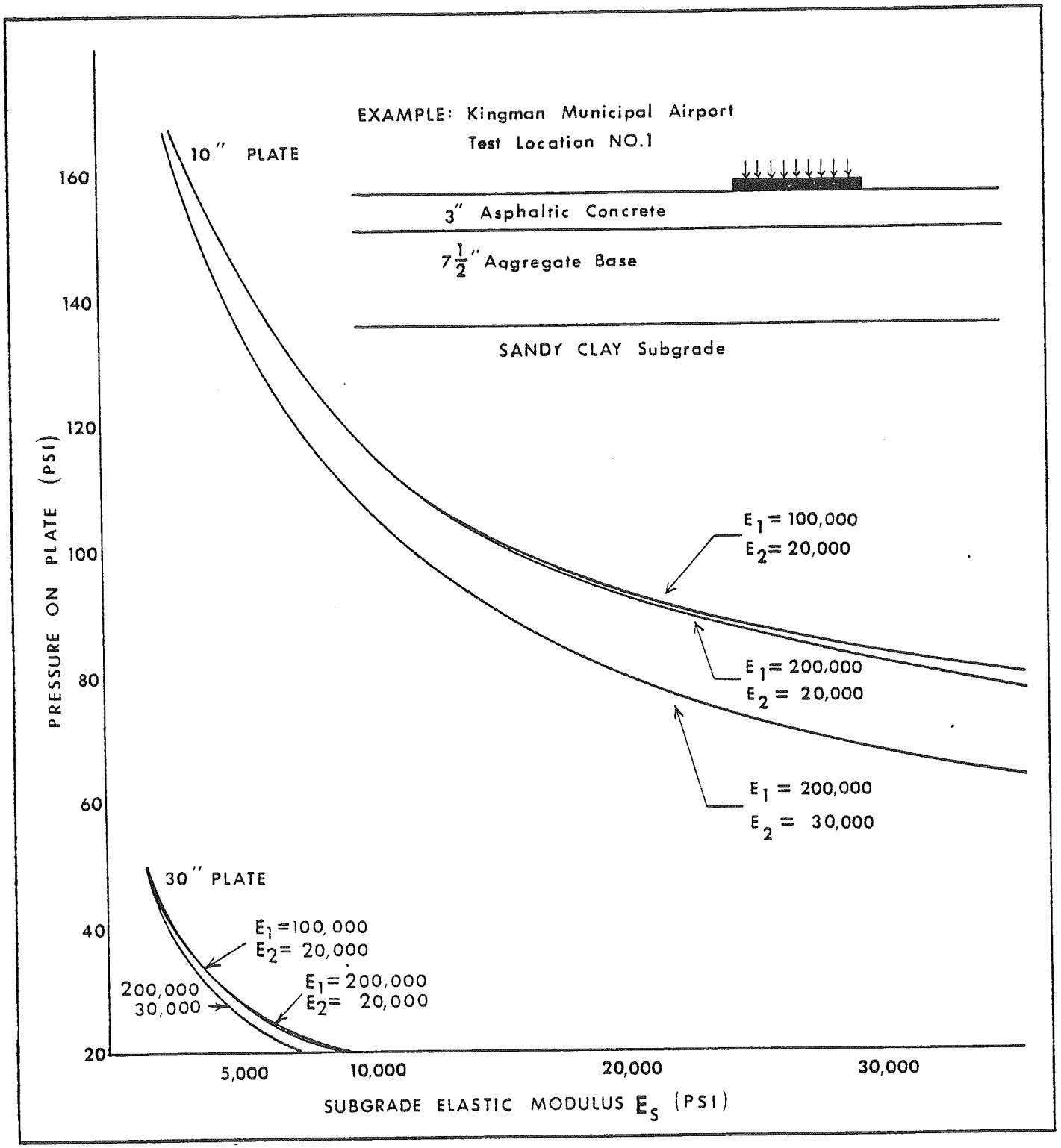


Figure 17. Sensitivity of Subgrade Modulus (E_3) to Moduli of Base and Surface Layer in a Three Layer Elastic Analysis

Further discussion of plate bearing tests and their usefulness will be presented in Sections III and IV, in light of the data gathered for this project. The advantages and disadvantages are briefly summarized.

- Plate bearing tests provide an accepted, widely used, and standardized means of evaluating the load response of a subgrade. Once the equipment has been mobilized, the test is relatively easy to run and understand. With a stress deflection plot, E or k modulus may be determined either by the tangent or secant method, using the virgin curve, rebound curve, or cyclic curve (in the case of a repetitive load test) depending upon the needs of the designer.

- Unfortunately, plate bearing tests are expensive in terms of time and money. A single stress-deflection curve is essentially only one particle of data as far as a statistical analyses is concerned. Because of the inherent lateral variations in earth materials' properties, it is desirable to gather several particles of data for comparison. A single plate bearing test may be misleading.

1.7.5 Standard Penetration Testing

An ever present ingredient in subsurface investigations, worldwide, is penetration testing. The amount of energy expended in driving a rod, hammer or sampler through a measured thickness of the

subsurface is usually a reliable index of the strength of the material in that stratum.

Many different types of penetrometers are used in various countries. In the United States, the standard split barrel spoon sampler fits this purpose. Using an ASTM certified procedure (see section 2.4.2A), the standard penetration resistance (N value) is obtained.

Remarkably, very few researchers have attempted to correlate subgrade modulus with standard penetration number. Several handbooks, in proposing moduli for loose, medium and dense "sands" or "soft and stiff" clays, use standard penetration number as a general index. These are not, however, derived from specific studies.

Martin (79) has recently published a correlation between standard penetration number and E modulus from the pressuremeter in residual soil from the Piedmont area of the eastern United States. Martin's field data showed considerable scatter and a least squares tend line was used to characterize the relationship.

Ohsaki and Iwasaki (80) have published the correlation $G = 1200\ N^{0.8}$, G is in (tons/square meter). This equation was a "best fit" among dozens of samples in Japan, where modulus was measured in-situ. Even so, the actual plot (G vs N) given in the paper shows considerable scatter of the data points.

Mitchell and Gardner have published a table, which summarizes correlations between N and E_s, based on writings from six research efforts. All deal with cohesionless materials. The differences in the resulting curves are so great as to make one doubt that there can be any correlation between penetration numbers and moduli.

1.7.6 CBR

Since the California Bearing Ratio is virtually the same test whether performed in-situ or in a laboratory compaction mold, little more can be added to the discussion in section 1.6.7.

It would be interesting to compare CBR values before and after construction. In Colorado, the Department of Transportation (81) undertook a most revealing study. In 1970, records were pulled on sections of highways around the state, built 15 years earlier. These sections were opened and in-place CBR's run. The results showed there was no correlation between this in-place CBR and the CBR on soaked samples run 15 years ago. The major finding of the project was a fact which is obvious to experienced highway engineers; that on sections with good service, moisture in the subgrade had remained virtually unchanged. On sections where poor service was indicated, subgrade moisture had increased drastically.

This study is not cited to belittle CBR, but to indicate that precision in design and preconstruction

testing does not guarantee a successful prediction of postconstruction behavior. In Colorado, it appears that mechanical and chemical degradation of the subgrade with time and traffic may have helped render the design CBR's meaningless.

1.7.7 Seismic Techniques - Impulse Methods

The most objective methods of determining elastic modulus in-situ are the seismic methods. It can be readily seen by inspection of the two equations presented in section 1.5.4

$$G = \rho V_s^2$$

$$E = 2G\ (1+\nu)$$

that if one can measure the in-situ shear wave velocity of an earth material, G is virtually automatic, and knowledge or assumption of ν will yield the compression modulus. This value of modulus is intrinsic for that soil at the lower range of strains and probably can be adjusted for other strain levels.

Unfortunately, the desired result is not easy to achieve in the field. When an impulse is sent through an elastic medium, several types of waves are generated. Compression waves travel fastest, and are the first to be seen on a wave trace. Compression wave velocity is easily computed, but the first arrival of the shear wave is obscured and its velocity is difficult to determine.

With accurate values of shear and compression wave velocity, Poisson's ratio can be calculated from the relationship

$$\frac{V_c}{V_s} = \frac{2\ (1-\nu)}{1\ -\ 2\nu}$$

and the modulus value can then be obtained precisely using $E = 2(1+\nu)G$.

Many techniques have been employed and other are in the development stage for determining V_s. Most are time consuming, expensive, and require sophisticated interpretation of the data.

Seismic techniques fall into several categories. The simplest is surface refraction seismic in which only compression wave velocity of the various layers is determined. E modulus is therefore calculated by assuming a value of Poisson's ratio. Seismic refraction surveys may also be designed to determine both shear and compression wave velocity, although the effort involved in placing the first arrival of the shear wave is considerable. Both types of seismic refraction surveys were used in this project and are described in Section II.

The crosshole seismic techniques make use of an impulse generated by a special hammer and clamp mechanism which is placed against the wall of the hole or trench. A receiver in the second hole some distance away records the wave train. Under favorable conditions, the pulse so generated will be rich

in shear waves and weak in compression waves. A trained geophysicist can thus determine the time of first arrival of shear waves (hence V_s). An obvious drawback to this method is the need to drill two holes at each test site and to manually transport and place the geophysical gear.

The Downhole-Uphole technique is similar to cross-hole in that a mechanical source rich in shear wave is created. Downhole is said to be especially useful where a low velocity layer underlies higher velocity material.

Rayleigh wave correlation techniques have always held promise for engineers. Rayleigh waves are surface waves whose velocity is within 5 to 10 percent of shear wave velocity over the entire spectrum of possible Poisson's ratios (see Figure 18). Rayleigh waves may be generated by either an impact or vibratory source. Basically, the impact method involves setting off small charges at increasing distances from both ends of a geophone spread. The arrival of R waves is characterized by high amplitude motions of the wave trace.

The trend in research and development today lies with impact testing. It is desirable to use a relatively light weight, air-transportable piece of equipment to nondestructively test the condition of existing pavements. Such an impact tester has been put into operation by the Dutch Shell Laboratorium (82) and is under development by the Civil

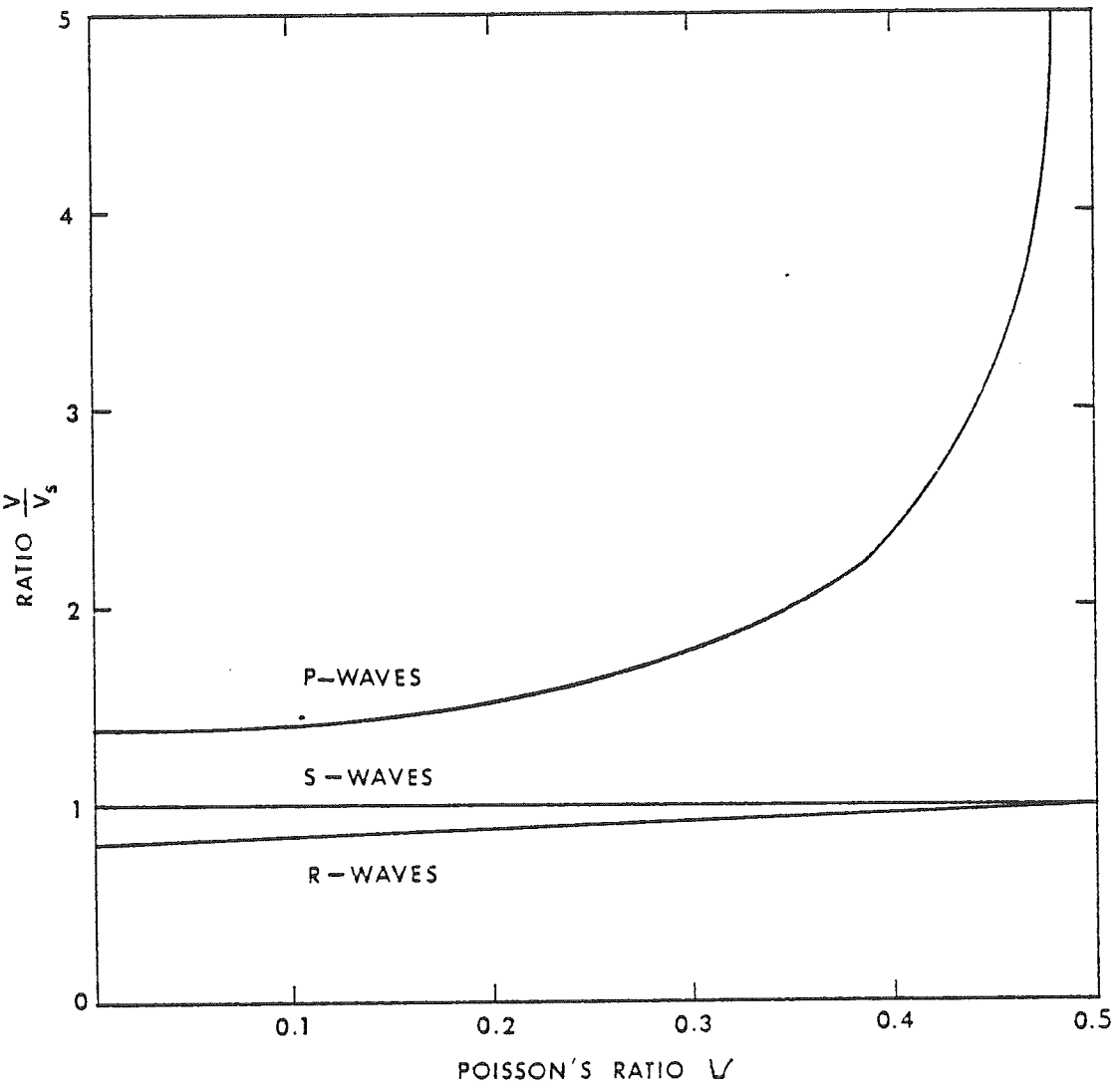


Figure 18. Relationship Between Shear Wave Velocity and Rayleigh and Compression Wave

Engineering Research Facility (CERF), Kirtland AFB, New Mexico (personal communication).

Problems involving collection of data are not difficult. By using a free fall drop hammer and a specially placed train of geophones, wave traces (motion-time plots) can be gathered on a high volume schedule. The problem lies in interpretation.

Apparently the mathematical science of "wave synthesis" can be employed to sort out values of V_s and V_p of various layers. "Digitization" of the entire wave trace is accomplished using the computer. By analyzing numerous possibilities, the wave trace is replaced by several others which detail what the response of the pavement would have been under a vibrating load of certain frequency. By analyzing wave forms of the several frequencies, it should be possible to obtain V_s and V_p values.

At the time the writer talked to CERF personnel in 1976 about this development, the shift in research from steady-state oscillators to impact testers was just beginning. An obvious problem to be solved was the computer programming of the wave digitization and analysis, with the goal of containment of the computer core and work station in a van-sized compartment. Another possible stumbling block is the problem faced by those who have tried to use Dynaflect data to back calculate stiffness of various layers in a pavement. It may be that there is more

than one reasonable profile which would generate the given response spectrum.

The Dutch Shell Falling Weight Deflectometer (FWD) is used to determine overlay design thickness needed for existing pavements. A suite of values of maximum deflection is analyzed statistically using the BISAR computer program. Subgrade modulus is one of the products, although overlay requirement is the major output.

1.7.8 Steady-State Vibration Techniques

Evaluation of existing pavements is a subject that is receiving increased attention as continually greater porportions of many transportation budgets are being committed to maintenance of existing facilities rather than new construction. Testing can be either destructive (boreholes or dug pits in pavements) or nondestructive. The former is definitive, but usually involves the closing of a road or runway resulting in inconvenience and economic loss. The need to gather extensive data quickly without inconvenience has been the motivation behind the development of nondestructive pavement evaluation equipment; most of which imposes steady-state vibratory loads on pavements.

The earliest device was the Benkelman beam, which measured deflection under a moving wheel load.

The Dynaflect, Road Rater and LaCroix Deflectograph followed. These highly mobile devices are used

primarily to obtain deflection basins on asphalt pavements. The Dynaflect is the most widely used worldwide.

Finally, the heavy vibrators such as the WES 16-kip semi-van transported vibrate have been built and used to test heavy airport runways.

These machines are meant to evaluate the stiffness (hence competence) of a pavement at a given spot, and only indirectly can be used to evaluate subgrade conditions. In this research effort, Dynaflect and Road Rater readings were taken directly on the surface of subgrades in an attempt to provide design moduli. Apparently, this has rarely been undertaken by pavement researchers.

Methods have been described where Dynaflect data on pavements is placed in a computer program and by regression, elastic moduli and Poisson's ratio are calculated for various layers. These will be commented on again in Section III.

Under development at the present time by Waterways Experiment Station, is a 350-pound van-mounted wave velocity vibrator which, if successful, will be ideal for state agencies in their testing programs. Rayleigh wave correlation techniques will be used to provide V_s and V_p of several layers. Computer space and costs are the problems to be overcome.

1.7.9 Pressuremeter

The Menard Pressuremeter is basically an expansive membrane which is operated in a small groove or a small borehole in soil or rock. As gauged pressure is applied to the sides and bottom of the cut, deformation is measured by monitoring the volume of fluid injected into the membrane.

Modulus is given as

$$E_p = (1+\nu)2V_o \frac{\Delta P}{\Delta V}$$

where V_o is the initial volume and ΔP and ΔV are the incremental changes in pressure and volume.

The pressuremeter is not unlike a number of in-situ strength tests used primarily in rock mechanics techniques in tunneling and underground opening problems. Tests such as flat-jack, borehole extensometer and borehole plate loading tests are mentioned in the literature, but will not be discussed here.

Few researchers have endorsed pressuremeter testing as a suitable means of determining modulus for geotechnical applications. In most tests, it is felt that only the horizontal subgrade reaction is being measured. Bronstein, et al (83) reported that the pressuremeter modulus was predictably less than the modulus determined in a plate bearing test. These authors proposed correction factors based on soil properties to make the various moduli conform.

In summary, it must be said that no field method for measuring load response of soil incorporates all the quantities one would like - i.e., definitive, easy and inexpensive to perform, and reproducible. The plate bearing test, for all its drawbacks, probably will remain the method against which all others are judged.

1.8 Discussion of Modulus as an Input Parameter in Rational Pavement Design Procedures

The many rational procedures for the design of airfield and highway pavements, both rigid and flexible, were discussed in section 1.4. It has been shown in sections 1.5, 1.6 and 1.7 that numerous conventions may be used to characterize load response of soils. Inasmuch as structural design of a pavement is a problem involving load response of the various components, it is instructive to look at the models selected by the developers of pavement design methods to characterize the response of subgrades.

Table 1 provides a list of available methods according to category. Each entry contains a summary of the published comments or recommendations regarding subgrade response.

A look at the table is revealing. In the traditional methods for the design of rigid highway and airfield pavements, a Westergaard "dense liquid" model employing the k modulus is preferred. The 30-inch plate bearing test is the most common way of determining k, this being the largest plate which is practical to use in the field.

Apparently k is preferred over E for rigid slabs because of the fact that wheel loads are distributed through the concrete and applied to the subgrade over a semi-infinite area. Hence, the spring constant (k) of the soil is less dependent upon size of the loaded area and the perimeter-area ratio effect. Witczak (84) states that the thickness of a rigid highway pavement is relatively insensitive to the value of k modulus and therefore the use of handbook values is probably justified.

An exception to the traditional procedures is Yang's method (22) which is applicable to the design of airfield pavements for the new generation of jumbo jets. Yang uses an E_s value to characterize the response of airfield subgrade (the design E_s is an average value of many plate bearing test values).

This E_s is then used to calculate an appropriate dense liquid constant k, according to either a formula given by Vesic (85)

$$k = \frac{E_s}{1-\nu^2} \quad \frac{1}{Z}\sqrt[3]{\frac{E_s}{E_c}}$$

where Z = slab thickness and E_c = modulus of concrete, or an empirical relationship found at Newark

$$k = \frac{E_s}{1-\nu^2} \cdot \frac{1}{2(a+z)}$$

where z = slab thickness
a = ratios of design wheel load

This k value is then used in another formula to calculate the bending moment in the slab.

The published methods for the design of flexible airfield pavements utilize various techniques for characterizing

Table 1

Summary of Pavement Design Methods

RIGID AIRFIELD PAVEMENT DESIGN METHODS		
Name of Method	Means of Characterizing Subgrade Response	Remarks
PCA Reference (19)	k modulus-determined from 30" plate bearing test @ 0.05" deflection for typical subgrades. For stabilized bases recommend modulus be taken @ 10 psi.	The suggested characterization applies only to two layer systems and for single, dual wheel and dual-tandem wheel aircraft only. For three layer systems, E modulus and modulus ratio procedures are recommended. For wheel configurations of large aircraft, special charts are given. Gives a chart for correlation of k modulus related to soil properties, where plate bearing tests are impractical.
Corps of Engineers Reference (20)	k modulus-determined from plate bearing tests. Size and deflection level are at the option of the designer, but should be representative of the design aircraft. 10 psi is recommended.	The field value of k should be adjusted according to the judgment of the designer for ultimate local conditions, such as moisture increase.
FAA Reference (21)	k modulus - from plate bearing tests - virtually the same as Corps of Engineers procedures with latitude given the designer to decide appropriate test conditions.	
Navy Method Reference (93)	k modulus from 30" diameter plate bearing tests. Follow ASTM D1196 (non-repetitive)	A procedure is given to modify the subgrade reaction modulus based on the thickness of a base course.
Yang Method Reference (22)	E_s - subgrade elastic modulus is taken as the tangent modulus in the 40 to 60 psi range on 30" plate bearing tests. For design values @ Newark, a statistical analysis of 33 tests was used.	It was noted that a Westergaard model of k modulus works well for concrete pavements 6"-10" thick with wheel radius 4"-6". Westergaard model is not good for heavy aircraft/thick pavement case. Correlations are suggested.

Table 1 (cont.)

FLEXIBLE AIRFIELD PAVEMENT DESIGN METHODS		
Name of Method	Means of Characterizing Subgrade Response	Remarks
Corps of Engineers Reference (23)	Design charts use CBR value. Should use the value measured at either 0.1" or 0.2" penetration, whichever is greater.	The CBR is determined in the laboratory under conditions expected to develop in the field.
FAA Reference (24)	Subgrade rating (designated F1 through F10). This F value is obtained from correlations with CBR or with the FAA soil classification.	The design subgrade rating is modified based on considerations of site drainage and frost susceptibility.
Asphalt Institute Reference (32)	Elastic modulus (E_s) can be determined by any of three recommended methods (1) M_R from repeated load triaxial compression test on compacted specimens. (2) E_s = 1500 CBR (3) Calculate E_s from plate bearing tests. Recommends 30" plate @ 0.5" deflection on the 10th cycle of a repetitive load test.	Also includes a correlation table relating FAA subgrade rating to E_s. It is recommended that the design subgrade value be 85th percentile of all values.
Navy Method Reference (93)	Requires several non-repetitive (ASTM D1196) and one special repetitive (four cycle to 0.1") plate bearing tests. Non-repetitive tests on 15" and 30" plates to 0.15" deflection.	Plot the values of load at specified deflections on a design chart to obtain design thickness of pavement.
Air Force Method Reference (94)	Charts use CBR, similar to Corps of Engineers. Recommends in-place testing if possible.	Design subgrade CBR should be 25 percentile value.
Canadian DOT Reference (95)	S (subgrade support value) determined from plate bearing test-specifically a 30" plate @ 0.2" deflection.	McLeod's plate size influence charts are referenced so that any size plate could be used.

Table 1 (cont.)

Name of Method	Means of Characterizing Subgrade Response	Remarks
RIGID HIGHWAY PAVEMENT DESIGN METHODS		
PCA Reference (26)	k - modulus. Recommends the use of 30" plate bearing test, design loading or deflection not specified. Also includes a graph for correlating k modulus with CBR, R value and Unified, AASHTO and FAA soil classification systems.	Recommends use of a reasonable k modulus determined for summer or fall subgrade conditions. Increased k values for winter (frozen) conditions or decreased values for spring thaw tend to compensate on a yearly basis.
AASHTO Interim Guide Method Reference (27)	k - modulus. Recommends the use of 30" plate bearing test - pressure level not specified.	
FLEXIBLE HIGHWAY PAVEMENT DESIGN METHODS		
VESYS IIM Reference (28)	D (t) defined as the creep compliance function. a_i and b_i are permanent deformation parameters. Must be determined in the laboratory using prepared specimens in an incremental static-dynamic test. The viscoelastic response is determined in a 1000 second creep test. Permanent deformation response comes from a series of dynamic (haversine) pulses.	The formula $D(t) = \frac{\varepsilon(t)_{zz}}{\sigma_{zz} - 2\nu\sigma_{rr}}$ where σ_{zz} = axial load without confinement σ_{rr} = confining pressure ε_{zz} = time dependent axial strain ν = Poisson's ratio a_i = IS/e b_i = 1-S e = resilient dynamic strain at 200 load repetitions I = intercept on a log-log plot of accumulated permanent strain vs. load number S = slope of above line
PDMAP Reference (29)	$M_R = A\sigma_d^B$ = determined from repeat load triaxial testing. Coefficient A and exponent B must be determined by user for each design section.	Variables A and B given for season of the year at Minnesota test station and were also calculated for undisturbed roadbed samples taken at the AASHO Road Test.

Table 1 (cont.)

FLEXIBLE HIGHWAY PAVEMENT DESIGN METHODS (Cont'd.)		
Name of Method	Means of Characterizing Subgrade Response	Remarks
Shell Method Reference (30)	E_3 - Elastic modulus of subgrade-Suggested that E_3 be determined from dynamic triaxial testing or from the empirical relationship $E_3 = 10^7 \times CBR$ in N/m^2.	1 N/m^2 = 1.45 x 10^{-4} psi. Relationship translates to E = 1450 x CBR, in psi.
AASHTO Method Reference (27)	Uses the Soil Support Value (S). Numbers on a scale of 1-10. Value of 3 was assigned to subgrades at the AASHO Road Test.	Each user agency must establish their own correlations between S and some standard parameter such as CBR, R value or triaxial strength.
Asphalt Institute Method Reference (32)	E_s - from correlations with CBR, R value, or from plate bearing tests. Recommends a 12" plate, repetitive testing, the modulus to be taken at the 0.2" deflection on 10th repetition.	
National Crushed Stone Association Method Reference (33)	CBR (laboratory-compacted sample) or in-situ if possible. Degree of compaction and soaking at the discretion of user.	Also contains a table correlating CBR with Unified or AASHTO soil classifications.
California DOT Method Reference (84)	R value (Hveem Stabilometer)-remolded or undisturbed samples.	
Canadian Good Road Association Method Reference (96)	E_s - based on repetitive plate bearing test. Use 30" plate, 10 repetitions, measure modulus at 0.5".	
Kansas Method Reference (97)	E_d = subgrade modulus of deformation. Determined from triaxial test under saturated conditions.	

Table 1 (cont.)

FLEXIBLE HIGHWAY PAVEMENT DESIGN METHODS (Cont'd.)		
Name of Method	Means of Characterizing Subgrade Response	Remarks
Kentucky DOT Method Reference (5)	E_3 = 1500 CBR*	*Kentucky CBR similar to conventional CBR. Soak time is much longer to allow swelling soils to reach a steady-state.
Chevron Research Method Reference (31)	E_s - Subgrade Elastic Modulus-at the discretion of the user. Recommends the repeated load triaxial test or the CBR relationship E = 1500 x CBR.	
Arizona Method Reference (2)	E_3 from correlation charts (CBR, R-value and classification).	
WES Procedure Reference (98)	M_R - Resilient Modulus use cylindrical, recompacted specimens in triaxial chamber using constant σ_3	Recommends the format $M_R = A\theta^B$ where $\theta = \sigma_1 + 2\sigma_3$.
Ohio DOT Method Reference (99) (100)	Not spelled out - subgrade support characteristics from tests on undisturbed samples or from Dynaflect measurements in-situ.	Two layer analysis in Reference (99) correlates subgrade modulus with Dynaflect deflection.
ESSO-France Method Reference (101)	E_s = 10 CBR (E_s in MPa)	Corresponds to E_s = 1450 x CBR, in psi.
South African Method Reference (102)	$M_R = A\theta^B$ for granular soils from dynamic triaxial test, free draining and few repetitions. E = K x CBR for cohesive soils K is a factor between 5 and 20, averaging 10 (metric-MPa).	ν = 0.35 (assumed)

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Table 1 (cont.)

FLEXIBLE HIGHWAY PAVEMENT DESIGN METHODS (Cont'd.)		
Name of Method	Means of Characterizing Subgrade Response	Remarks
Brown and Pell Method Reference (103)	E = 10 CBR (metric)	
Ireland Method Reference (104)	E - compression modulus selection method unspecified - according to the judgment of the user.	
Belgian Road Research Method Reference (105)	For granular soils E_s = 10 x CBR (metric) For cohesive soils $E_s = \frac{43}{PI + 0.715}$	
German Method Reference (106)	E_{dyn}, ranges between 1.2 and 2.5 x E_{v2}. E_{dyn} is the dynamic modulus for use in design. E_{v2} is the modulus calculated from a plate bearing test using 30cm plate.	
Italian Method Reference (107)	E from a plate bearing test program involving series of repetitive (two cycle) loads on 60cm diameter plates.	The empirical procedure for calculating a design E_s and ν is given. Several tests must be run to obtain a statistically meaningful design value.
Hungary Method Reference (108)	E_3 = 10 x CBR (metric)	
USSR Method Reference (109)	E_s - apparently determined from plate bearing tests (details unspecified).	

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the subgrade, but most rely on CBR or plate bearing tests. In the realm of flexible highway pavement design, there are many different standards to be followed in the treatment of subgrade response. There seems to be a parallel between the overall degree of sophistication incorporated in a given method and its procedure to model the subgrade. For instance, in VESYS IIM, which is the most intricate of the pavement design systems, a nonlinear elasto-plastic subgrade model is called for. For most methods, purely elastic models of subgrade response are preferred. The rheology of soils is too imprecise to justify the effort of determining creep and permanent deformation indices. Of the approximately 20 methods listed, five recommend dynamic triaxial testing to determine M_R for the subgrade. These are in the form $M_R = A\sigma_d^B$ or $M_R = A\theta^B$. It should be noted that this is also a common way to test and characterize the response of bituminous bound materials and asphaltic concrete.

Another 10 methods suggest that users adopt simple correlations, most often, the Heukelom and Foster relationship $E = 1500 \times CBR$. Only four of the methods rely strongly on in-situ testing, while most endorse laboratory tests as being suitable and probably more economical.

In most agencies, worldwide, pavement designers prefer that subgrade samples be tested in ways that simulate the poorest possible environmental condition, namely, saturation of the subgrade. Accordingly, pavements are often designed and built as if subgrade support was minimal. The reasonableness of such a philosophy will be discussed in Section IV.

As stated previously, there is a variance among the methods on the importance placed on obtaining precise values of E_s. Those content to use simple correlations such as E = 1500 x CBR apparently feel that such relationships are sufficiently accurate within the context of the many independent variables in a pavement design problem to provide an economical pavement.

In any design procedure, one of the engineer's responsibilities is to decide how to proportion finite fiscal resources to obtain values for his input variables. Obviously, it makes little sense to spend thousands of dollars per mile seeking an exact value of a parameter which has negligible effect upon the thickness of the final pavement. With respect to subgrade modulus, a sensitivity analysis may show this in proper perspective.

1.9 Sensitivity Analysis

Most of the methods for the design of flexible pavements described in the previous section are based upon limiting values of stress, strain and deflection within the highway prism. Multilayer elastic analyses requiring digital computer programs are an essential tool in the determination of accurate stress and strain values. Most pavement design techniques focus primarily on two criteria as keys to successful design. These are the maximum value of tensile strain in the asphaltic concrete layer, which is known to be an index of the potential for fatigue-type failure in pavements, and the compressive strain at the top of the subgrade, which is an indicator of rutting-type failure potential.

Successful implementation of any such pavement design methodology hinges on two areas of technology concerning materials behavior. First, the stress, strain and deflection values calculated in any layered elastic program depend upon realistic input values of modulus of elasticity (E) and Poisson's ratio (ν) of the various materials in the highway prism. Secondly, there must be a set of limiting strain criteria regarding fatigue and rutting, the acceptance of which yields a viable and economically sound pavement design.

Multilayer elastic analysis programs such as CHEVRON (86) and BISAR (87) are widely accepted today by pavement analysts. Tensile and compressive strain limit criteria are not as well established although several authors and agencies have published suggested criteria which may be useful in a comprehensive pavement design methodology. Notable among these are works by Dorman and Metcalf (88), Finn, et al (89), Edwards and Valkening (90), Claessen, et al (91) and Richards (92).

The purpose of this chapter is to present some of the apparent relationships between subgrade elastic modulus, E_s, and thickness of asphaltic pavements using the latest state-of-the-art techniques. Additionally, an analysis of the sensitivity of concrete pavements to k modulus using AASHTO is given. For purposes of this discussion, strain criteria from references 5 and 28 will be used exclusively as the methods involved incorporate sophistication which is typical of all the latest methods, yet are in chart form and can be readily used.

The thickness design procedures developed by the Chevron Research Company and presented by Santucci (31) pertain to full thickness asphaltic concrete pavements. Using values calculated from the CHEVRON multilayer elastic analysis program, Santucci presents design charts which can be used to determine both maximum tensile strain, ε_T, in the asphalt and vertical compressive strain, ε_v, in the subgrade for various combinations of asphalt modulus, E_1 (which is primarily a function of age and temperature) and subgrade modulus, E_s. Other charts relate the two strain criteria to the life expectancy of the pavement in terms of the number of 18-kip EAL's to failure.

The subgrade compressive strain criteria, in terms of number of allowable load repetitions, were borrowed from the experience of Monismith and McLean (personal communication) with typical subgrades in California. The tensile strain criteria in asphalt was developed from a large body of fatigue test data performed in the Chevron Research Laboratory. A unique feature of the Chevron method is the inclusion of the effect of volume of air voids and volume of binder on the fatigue life of asphalt as parameters independent of elastic modulus.

Examples of how data derived from the Santucci charts can be used to show the importance of subgrade modulus in the design life of full thickness asphalt pavements can be seen in Figures 19, 20 and 21. Figure 19 relates required asphalt thickness to subgrade modulus for various values of 20-year Average Daily Traffic (ADT) assuming failure due to rutting.

Figures 20 and 21 relate the same requirements, assuming failure due to fatigue in the asphalt. Such curves must be developed for each desired quality of asphalt as defined by air voids, binder content, and elastic modulus.

A design is performed by checking the thickness of the asphalt required to achieve the desired pavement life under each of the two possible failure modes (it is assumed other failure mechanisms such as thermal cracking will be otherwise prevented). The required thickness is the larger of the two numbers.

Figure 22 is a similar design curve showing the role of subgrade modulus in the design of a conventional three layer pavement system. It is derived from nomographs and strain criteria published by the Kentucky Department of Transportation (5). This particular set of curves applies to prisms where the base is twice as thick as the asphalt and the subgrade is assumed to be infinite. The design thickness is that needed to insure the desired pavement life based on both asphalt fatigue and rutting. Similar curves could be developed for other ratios of asphalt to base thickness. As was the case with Santucci's work, values of tensile and compressive strain in the prism were derived using the CHEVRON program. Limiting strain criteria in asphalt are based on research by Dorman and Metcalf (88) of the Shell Oil Company. Subgrade compressive strain criteria are the result of the Kentucky Department of Transportation experience. Southgate notes that in Kentucky, subgrade strain criteria generally govern.

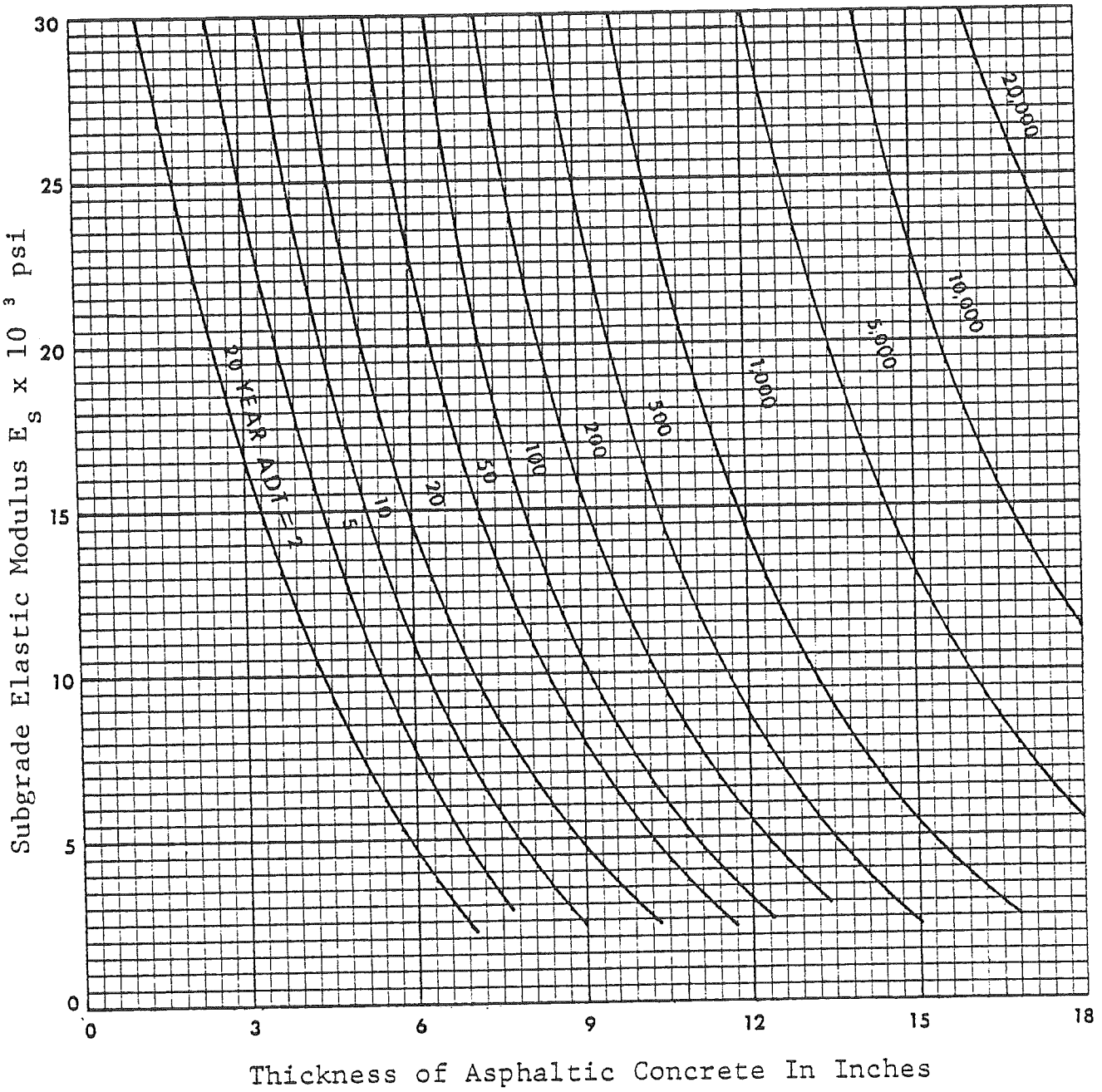


Figure 19. Design chart showing role of subgrade elastic modulus (E_s) in design of full thickness asphaltic concrete pavements using Chevron Research Method.

Failure the result of excessive compressive strain at the top of the subgrade (rutting).

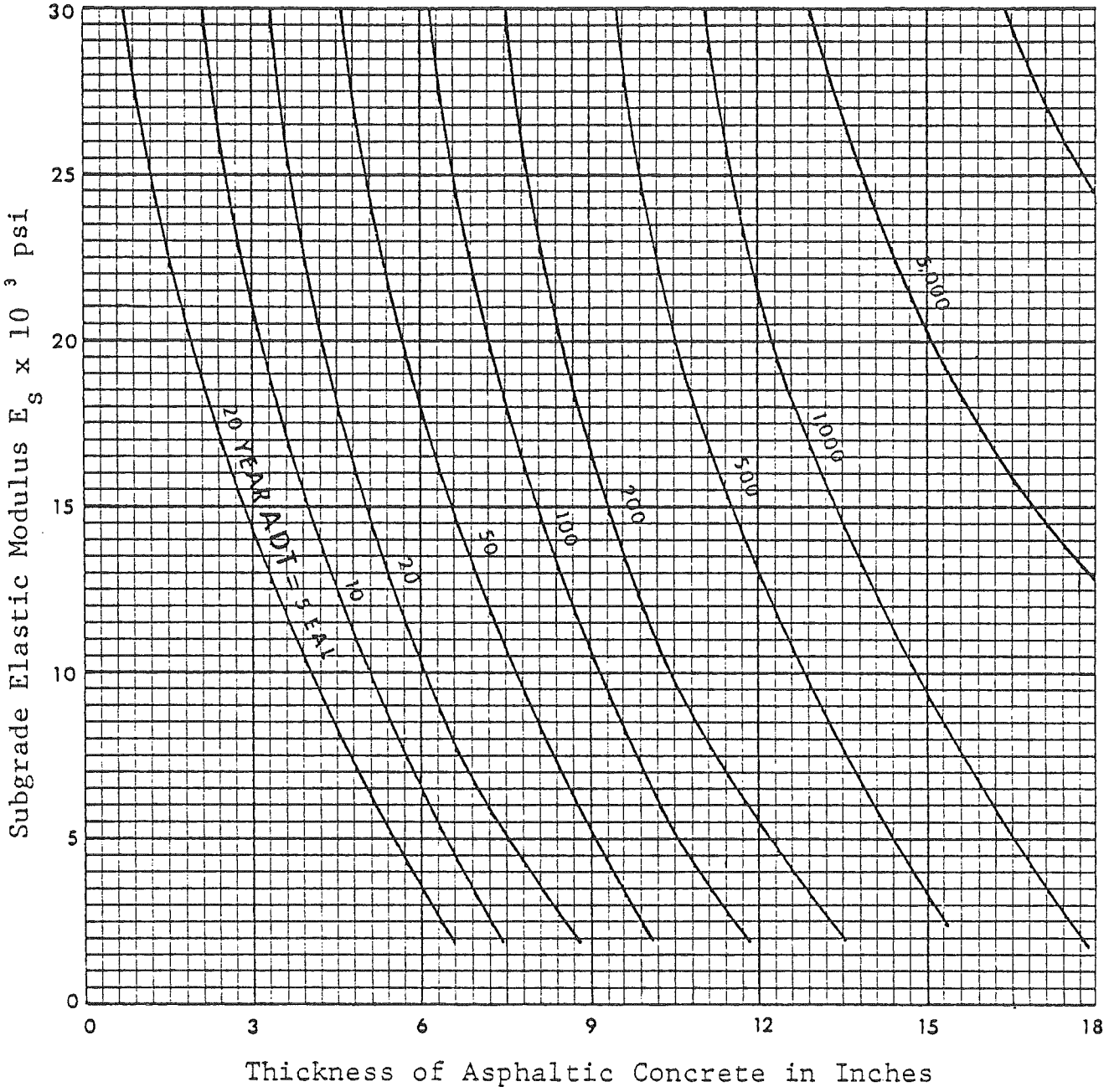


Figure 20. Design chart showing role of subgrade elastic modulus (E_s) in design of full thickness asphaltic concrete pavements using Chevron Research Method.

Failure the result of excessive tensile strain at bottom of asphaltic concrete layer (fatigue).

V_v = 5% V_b = 11% E_1 = 200,000 psi

V_v = Volume of air voids
V_b = Volume of asphalt residue

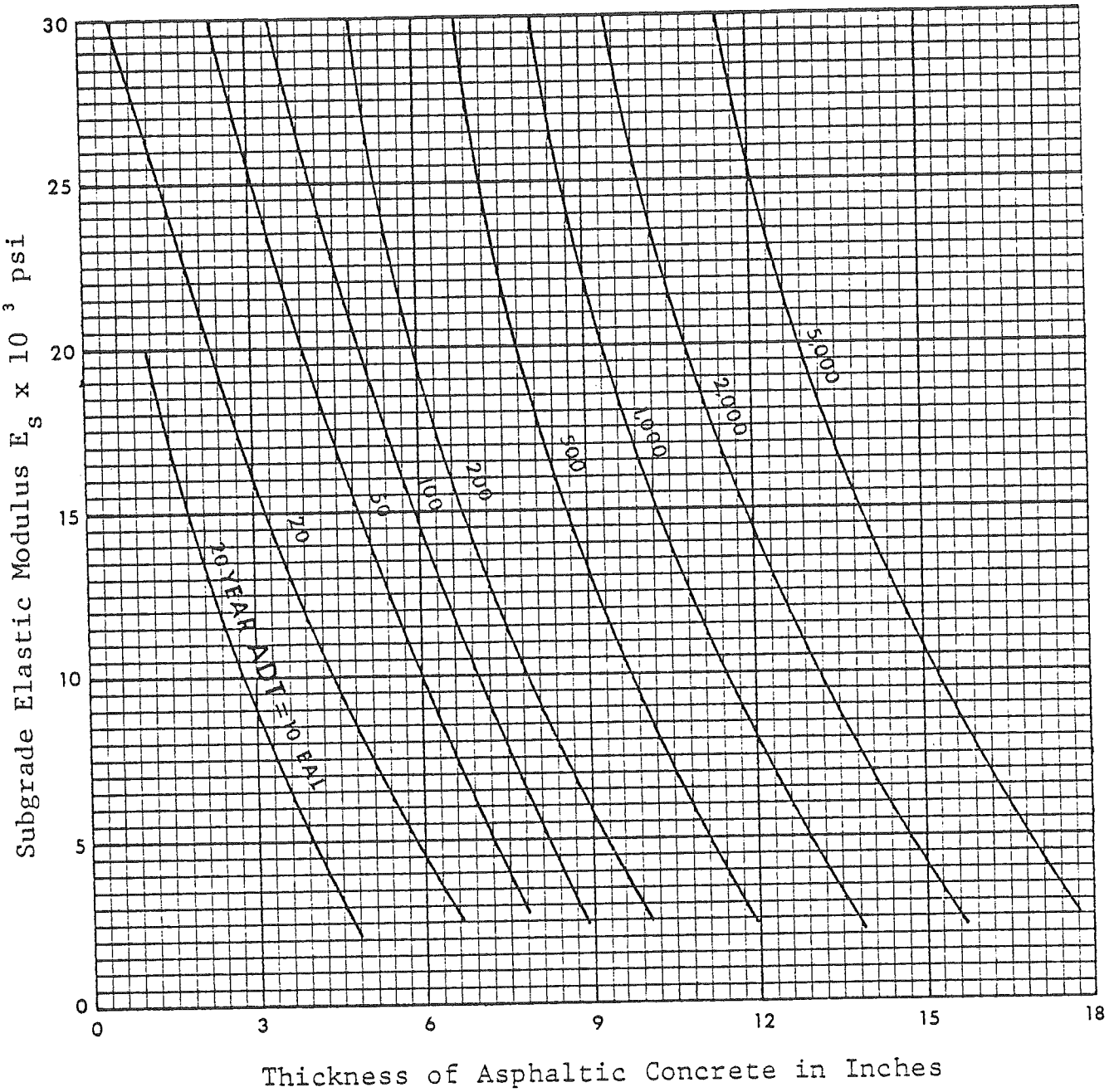


Figure 21. Design chart showing role of subgrade elastic modulus (E_s) in design of full thickness asphaltic concrete pavements using Chevron Research Method.

Failure the result of excessive tensile strain at the bottom of asphaltic concrete layer (fatigue).

V_v = 3% V_b = 13% E_1 = 200,000 psi

V_v = Volume of air voids
V_b = Volume of asphalt residue

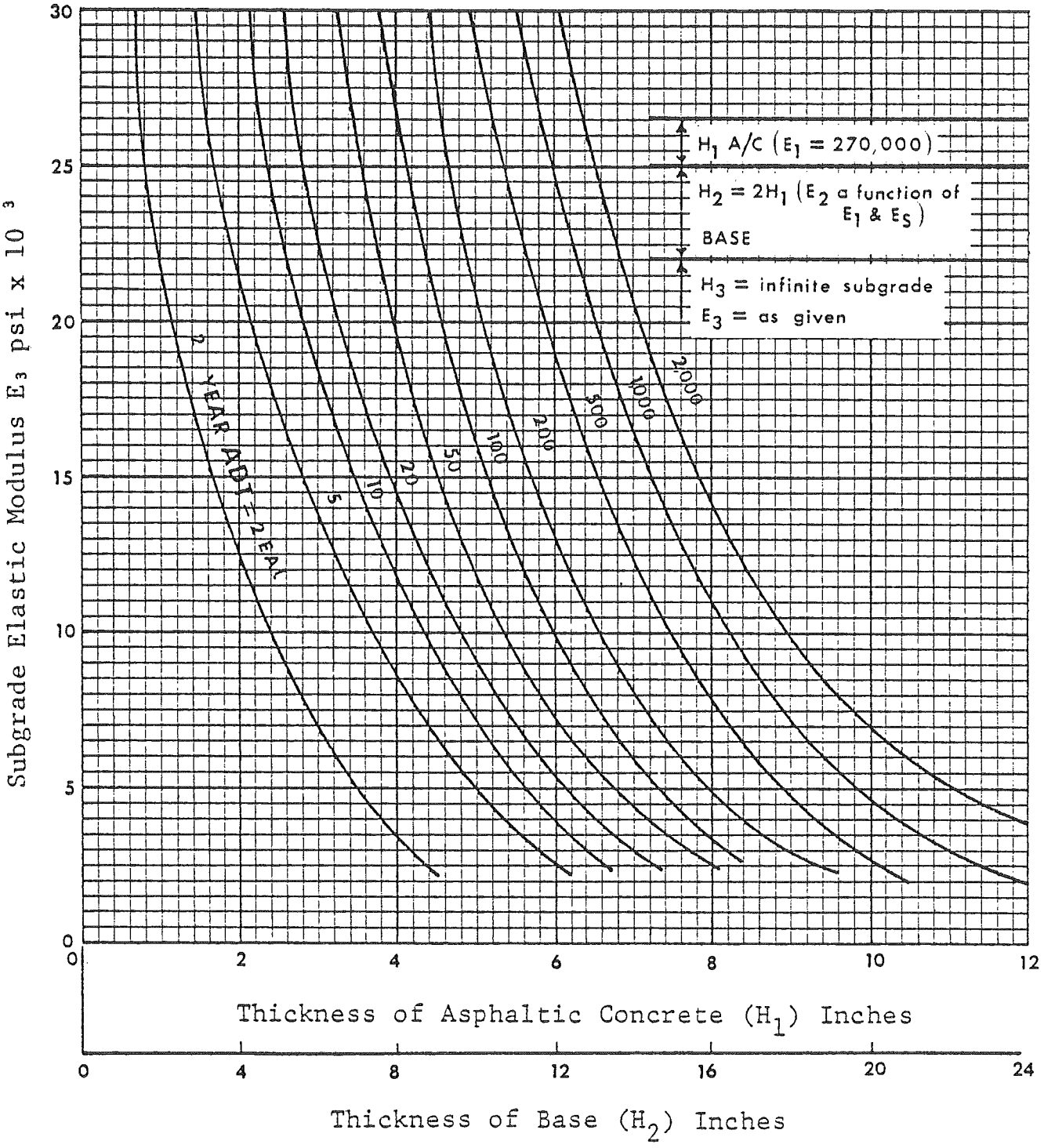


Figure 22. Design chart showing role of subgrade elastic modulus (E_s) in flexible pavement design using Kentucky Method.

A few spot checks of a two layer system using the Kentucky charts show them to be apparently more conservative than the Chevron charts. While elastic modulus of asphalt is a design input in the Kentucky method, more sophisticated asphalt quality parameters are not. In Figure 22 of this report, the subgrade stiffness is given as an elastic modulus, E_s; however, in Southgate's analysis, stiffness of the subgrade is presented in terms of "Kentucky CBR". This parameter is similar to the conventional CBR and the widely accepted (though often questioned) relationship given by Heukelom and Foster of $E_s = 1500$ CBR is assumed. In the Chevron method, E_s is considered an intrinsic response parameter and its correlation with tests or other indices is left to the judgment of the user.

It has been stated earlier that there is no universal standard for defining, determining or bracketing the response parameter we call E_s, the elastic modulus of a subgrade. For this reason, it is difficult to compare pavement design methods. Nevertheless, personal communication by the writer with both authors seems to indicate that both understand E_s to be an index of a covered subgrade's response under an 18-kip load with 70 to 90 psi tire inflation pressure.

From the four figures in this review, certain principles can be derived.

1. As subgrade modulus increases, the life of a pavement increases for a given highway section.

2. For a given subgrade modulus, the life of a pavement increases logarithmically with section thickness.

3. The importance of subgrade modulus in any design method is most keenly felt in the range of E = 5000 to 25,000 psi (which is the range of most Arizona subgrades). As an example, using Figure 22, a pavement 4 inches thick over an 8 inch base and a subgrade whose subgrade modulus is 10,000 psi has a 20-year ADT life of about 7. This same section with a subgrade modulus of 20,000 psi has an ADT of 50 and with E = 30,000 psi, ADT = 150. Similarly, for a full thickness pavement of 6 inches over a 10,000 psi subgrade, 20-year ADT is 8 while for E = 20,000 psi, ADT = 48.

Therefore, the evidence seems to indicate that in using recently developed flexible pavement design methods, the economic benefits to be derived from applying a thoughtful and physically reliable value of E_s justify a reasonable effort in obtaining such values.

Figure 23 is similar to earlier figures, whereby the role of subgrade reaction k in the thickness design of rigid pavement slabs is shown. In can be seen that according to the nomograms in the AASHTO procedure, the thickness of slab for a given daily traffic volume is only slightly sensitive to the subgrade modulus, especially for high volume roads. It should be remembered that this statement applies only to standard 18-kip axle loads, not to large or catastrophic loads, which must be analyzed by a method such as Yang's. Accordingly, this research effort is focused primarily on the response of flexible highway pavements.

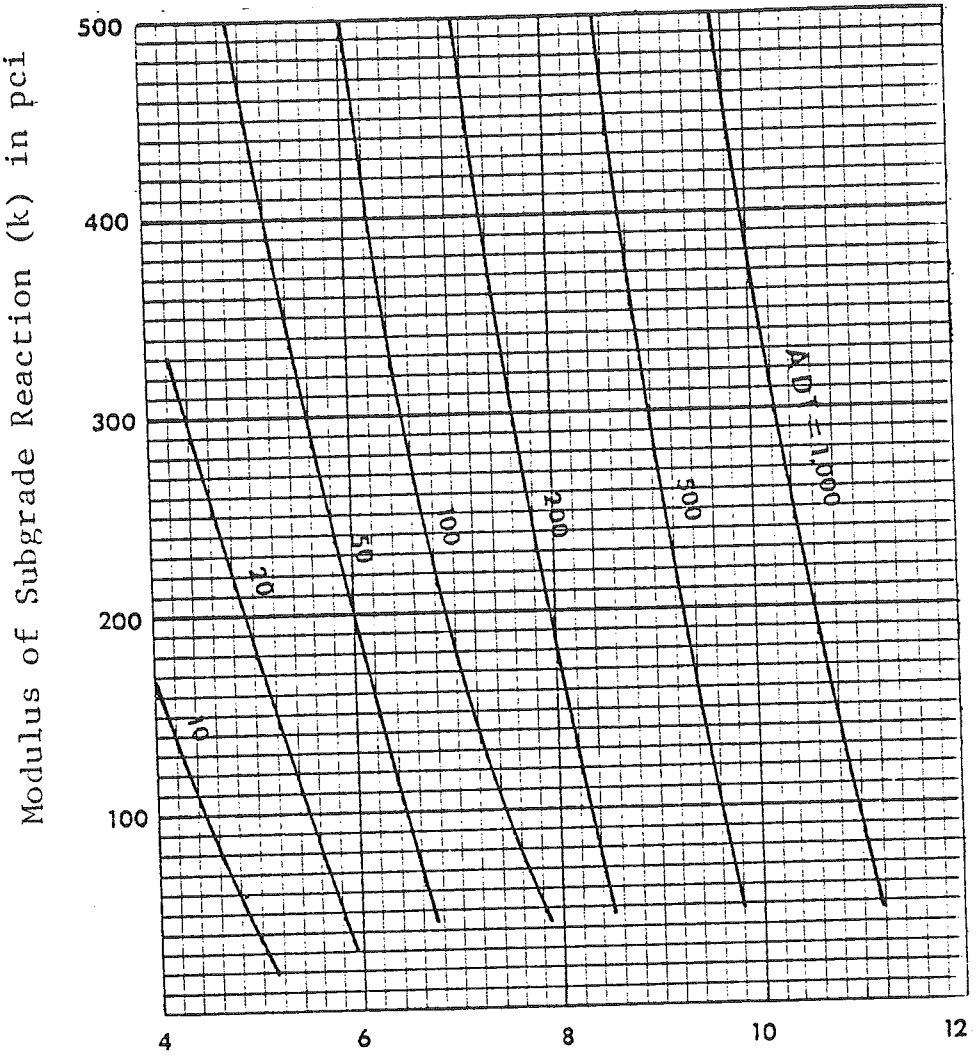


Figure 23. Design showing role of modulus of subgrade reaction in the thickness requirement for rigid pavements. Derived from design charts in Reference 27.

SECTION II. FIELD TESTING PROGRAM SITES AND TEST METHODS

2.1 Goals of Field Testing Program

2.1.1 General

While the worldwide technical literature contains numerous discussions of elastic modulus of soils, there can be found many seeming inconsistencies and contradictions. Reported load response behavior of various soils in different localities are often dissimilar, leading one to question whether an agency such as Arizona Department of Transportation can or should rely on information developed elsewhere.

The purpose of this field testing program was to gather and analyze a substantial amount of information from sites which are typical of Arizona subgrades, but which at the same time are unlike the classical soil types which have been the object of many prior investigations.

Accordingly, field testing was undertaken in two phases. In Phase I (fall, 1976) three sites were examined, with emphasis placed on precision in quantifying subgrade response. At the two Phase II test sites, (spring and summer, 1977), the emphasis was shifted to collecting large amounts of data within limited areas to investigate various

factors including the degree of lateral variation in response for typical Arizona soils.

In both phases, the plan involved the measurement of modulus by several methods and through a wide range of loadings and stress paths. Comparison of values obtained by different methods were to be used to develop criteria for selection of design values of E_s.

2.1.2 Physical Aspects of Testing Related to Modulus

A major objective of the testing program was to investigate to what degree widely reported behavior concerning response of soils to loading would be manifested at the selected test stations. Such widely reported observations include:

A. Modulus is dependent on the size of a loaded area. Following Burmister's pioneer work with plate bearing tests in the early 1940's, McLeod (34) and Housel, using data from several Canadian airports and from the Hybla Valley Road test, reported that there is a straight-line relationship between unit load and perimeter-area ratio (P/A). An example is shown in Figure 24. In Figure 25, we have back calculated values of E from the suite of data in Figure 24. It can be seen that as the size of the plate increases there is generally a higher modulus for all stress ranges. The reason for this is usually assumed to be the fact that loads on larger plates are transmitted deeper into the subgrade where stiffness is greater.

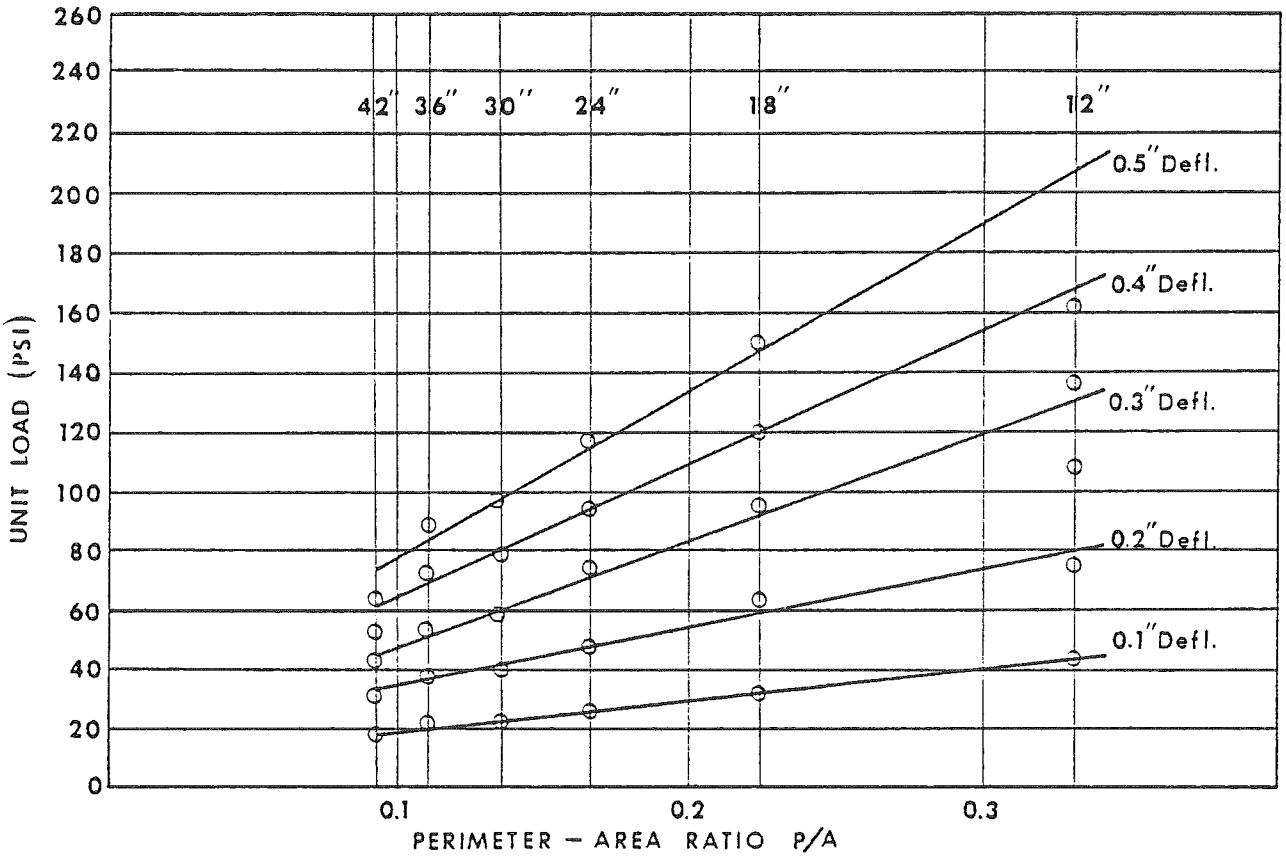


Figure 24. Influence of Plate Size on Unit Load (after McLeod - Ref. 34)

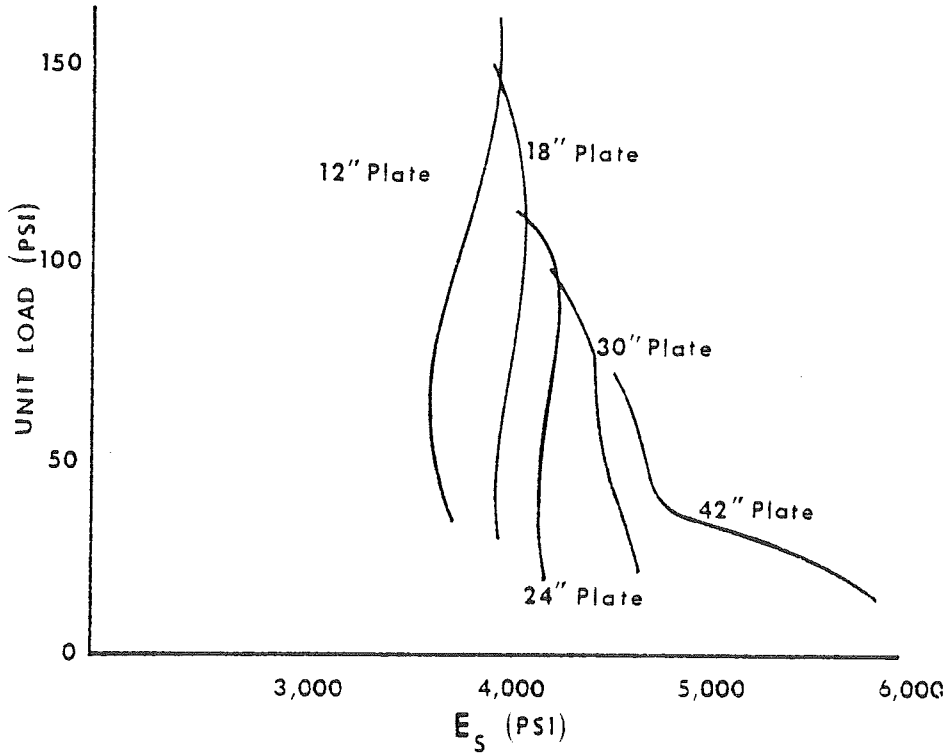


Figure 25. Backcalculation of E_s from Data Presented in Figure 24

B. Modulus is dependent on stress path. Cyclic (repetitive) load tests are thought by many investigators to be superior indicators of subgrade performance compared to the single cycle test. A disadvantage is that they are expensive tests to perform. Consulting airport engineer Reinard Brandley (110), has run numerous plate bearing tests on airport pavements and subgrades. He reported erratic and uncertain results when back calculating E moduli from the first load cycles of plate bearing tests. Brandley reported great success, however, using the third, fourth and fifth cycles of a repetitive load test.

C. Modulus is dependent on strain level. At very low levels of strain, soils are elastic with deformation being fully recovered upon release of the load. Beyond the elastic range, plastic deformation occurs, and to describe the load response by using only an E modulus is, in fact, an over-simplification.

Virtually all experience has shown that modulus usually decreases with increasing stress level. This is fundamental when one remembers that, graphically, E modulus is the slope of a line joining the origin and a point on stress-deflection plot. In soils, as bearing capacity failure is approached, the stress-deformation plot becomes flatter and obviously the secant slope must become flatter also.

The most pertinent question is, for a specific soil, how much adjustment for differing strain levels is appropriate. As mentioned earlier, Seed and Idriss (46) published correlation charts for clean sands and for saturated clay, based on laboratory experience (see Figure 5). It is not known to what degree these findings are applicable for very dry and cemented soils.

D. Modulus is dependent on duration of loading. Odometer tests have shown that time dependent settlement is an important element in the load response of clayey soils. Long-term settlements may be classic "consolidation settlements" where water is exuded from the soil pores or may consist of long-term plastic deformation (creep). For this reason, most specifications regarding the running of plate bearing tests require that some minimum rate of deformation be reached before the next load increment is applied. In the case of traffic loading, however, durations are short and a modulus incorporating time dependent deformation may not be desirable.

E. Modulus is dependent on frequency of loading. Traffic loads are generally random-pulse loads. There is a rest period between pulses during which the soil may completely or partially rebound.

A test which uses a steady-state vibratory load may induce a response which is frequency as well

as amplitude dependent. Like other materials, soils exhibit resonance at natural frequencies. Even discounting resonance, the large amount of energy being supplied by a high frequency oscillator is certain to promote a response which is different from that resulting from a transient load.

In summary, the testing program was tailored to examine these accepted principles concerning the physical aspects of modulus and to put each in a proper perspective with regard to Arizona subgrades.

2.1.3 Environmental Aspects Related to Modulus

For earth materials in-place, the environmental factors influencing load response may include:

- changes in temperature
- changes in moisture content
- mechanical or chemical deterioration

Actually, these three are interrelated. Changes in temperature, of course, have a substantial effect on subgrade stiffness where freezing occurs. Several investigators (111) have found that the temperature changes within the range of ground temperature variations above freezing which occur in Arizona have only a slight effect on compressibility. Effects are largely confined to the long-term "consolidation" settlements and are not considered to be of practical significance in this study. Chemical and mechanical

deterioration of subgrades proceed more rapidly in a high moisture environment. Water is the medium of transfer which allows introduction or removal of ions from a soil mineral structure. The fabric as well as the structure of most soils is primarily a function of its moisture history. It can be said then that the "environmental condition" of a subgrade, for all intents and purposes, refers to its "moisture condition".

Water in subgrades is invariably destructive to modulus. The mechanisms by which soils lose strength as a result of moisture increase are complex and are the subject of many (possibly conflicting) theories. Proctor (112), explained the loss in strength in terms of a breakdown in capillary (tensile) forces between particles. Hogentogler (113) discussed the absorptive forces created on particle surfaces under low degrees of saturation. Greater amounts of moisture thicken the water film, providing lubrication and decreased particle friction which allows greater movement of individual grains. Hogentogler described swelling as the addition of water to such a degree that individual soil particles are displaced. Hilf (114) emphasized the theory of pore water pressures in unsaturated soils to explain the role of menisci shapes in developing the friction forces which give strength to soils. The physico-chemical aspects are treated by Lambe (115). At low degrees of saturation in clayey soils, a flocculated (attractive forces dominating) structure is likely. With greater amounts of moisture, and

depending on the ion content in the pore fluid, a dispersed structure (repulsive forces dominating) may result, with consequent lower strength.

The theory of effective stress, as it applies to the engineering properties of partially saturated soils and the soil structure created by the processes of formation described in Section 2.2.2, has been treated by a number of investigators, including Blight (116, 117, 118), Burland (119), Newland (120) and Barden and Sides (121). Negative pore water pressures or soil suction, pore air pressures and indirectly pseudochemical phenomena are considered in this body of theory.

From the practical viewpoint concerning pavements, the action of repeat loading is important. Intermittent pulse loads resulting from traffic create within a wet subgrade, local hydrostatic forces which could be described as being the result of miniature diaphragm pumps. The resultant pressures may momentarily be high enough to rout individual soil particles, resulting in larger void spaces in the subgrade. For desert soils, the combined effects of traffic and prolonged exposure to excess moisture will dissolve and remove naturally occurring cementation, leaving a weaker structure.

Roadbuilders throughout history have recognized the problems with pavement performance created by excess water in the base and subgrade layers. The best defense is usually assumed to be good drainage

design. In spite of good intentions, however, planned drainage is not always successful in keeping moisture out of subgrades. For this reason, many pavement design procedures are predicated on the assumption that the subgrade is in its poorest possible state (saturated). This is why the soaked CBR is widely employed.

In most climatic regions there is a tendency for moisture to increase with time beneath a pavement. This is particularly the case where a layer of open graded base course rests above a relatively low permeability subgrade. In arid regions, however, this has been found not always to be the case as will be discussed at length in Section IV.

Moisture change beneath pavements is a subject of great interest to pavement analysts today. Excellent field study programs have been undertaken by various agencies. In Pennsylvania, Cumberledge, et al (122), selected five representative test sites under existing pavements and instrumented them with moisture and temperature sensing devices. At three sites, the subgrade was sandy, at the other two, the subgrade was fine grained. Readings of the temperature and moisture were taken once a month. Deflection basins at the sites were also determined on a monthly basis using the Road Rater.

The results were as expected; a definite relationship could be seen between moisture and deflection. At all test sites, an increase in surface deflection

generally accompanied each increase in subgrade moisture except when freezing occurred, in which case, the increased deflections occurred during the spring thaw. The most noticeable effect of soil type was that for the sandy subgrade, moisture contents dropped back to their base level at a faster rate than did clays following the thaw.

An Oklahoma study by Haliburton (123) showed similar trends regarding seasonal moisture changes. In the mostly fine grained Oklahoma subgrades, there was usually an increase in water content, due to seepage, over the construction water content until a value of 1.1 to 1.3 times the plastic limit of the soil was achieved. It was also noted that if a pavement becomes cracked, rainfall infiltration into the subgrade will produce localized wet/dry cycles, which often lead to distress.

An interesting study in a state where few subgrades are subject to long-term freezing was performed by the Texas Transportation Institute (124). At most test stations, greater deflections were recorded during the spring and summer months, but only in the northern panhandle were they pronounced. Deserving special note is the statement that seasonal changes in deflection at a given pavement section are rarely as large as the differences attributed to spatial variations in stiffness of the subgrade-pavement system itself.

It should be noted that the Arizona Department of Transportation has implemented a subgrade environmental monitoring program which is similar to that of Pennsylvania. The results will be published in late 1978. Inasmuch as seasonal rainfall and temperature patterns in Arizona are less predictable from year to year than in the more northerly states, one might expect less clearly defined trends regarding seasonal variations in moisture and pavement stiffness.

Returning to the goals of our testing program in relation to the effects of "environmental condition" on subgrade response, there was never a doubt that increasing the moisture content in a subgrade would lessen its stiffness (lower the modulus). A purpose of the testing was to quantify in some way this moisture-dependence of modulus for our typical soils.

2.2 Distribution of Soils in Arizona

2.2.1 General Geography and Climate

In general, Arizona is located in one of the driest regions of the United States. Most commonly, the climate is referred to as "semi-arid", reflecting the fact that, while rainfall is sparse, the region falls short of being a desert in the truest sense of the term.

The state is most easily divided into three geological and physiographic provinces (see Figure 26).

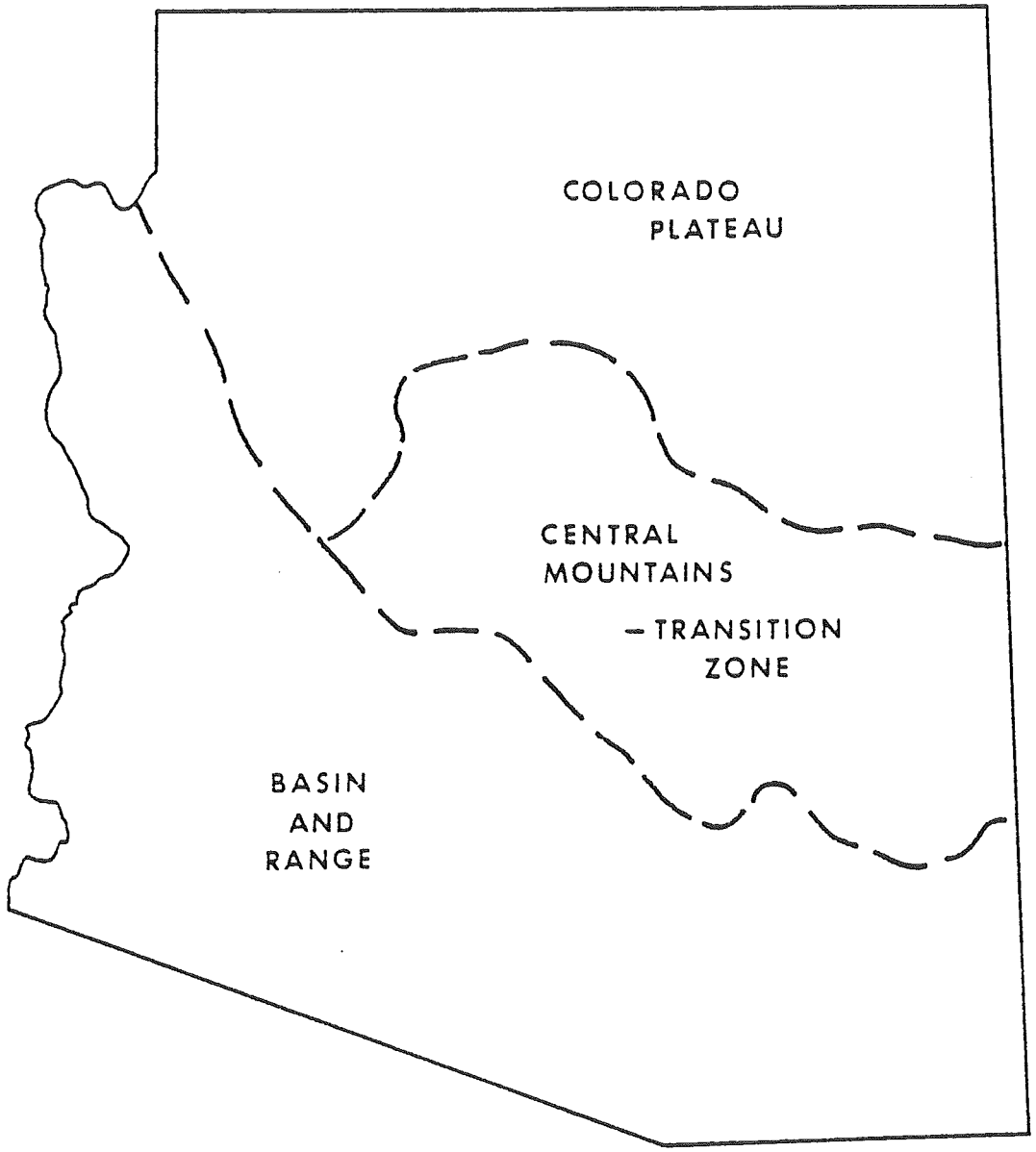


Figure 26. Physiographic Provinces in Arizona

The southern and western parts of the state fall into the Basin and Range province, the northern part contains the Colorado Plateau and between the two is the Central Mountains Transition Zone.

Throughout the state, rainfall and mean annual temperature are generally almost a linear function of elevation, although localized anomalies occur.

The Basin and Range province, which contains the bulk of the state's people and economic activity, is characterized by sharp blocky mountains rising above wide and gently sloping valley floors (basins). These valley floors are referred to as "the desert", although only a few hundred square miles in Yuma County meet the accepted criteria for a desert. These intermountain valleys contain grass and low brush. Elevation of valley floors ranges from 200 feet to about 3000 feet.

The mean annual daytime temperature is in the mid-80's (F), with summer days in the 100-115^{o} range. Rainfall is very sparse, ranging from as low as 3 inches per year to 9 to 10 inches near the mountain fronts.

The Colorado Plateau is a regional uplift consisting of thick strata of sedimentary rock. The present day landscape is mainly the result of wind and water erosion. In most locations, the plateau is gently dipping and is dissected by numerous parallel streams. Elevations range from 5000 to about

7500 feet. Rainfall is in the 10 to 20 inch annual range, and temperatures are moderate. Visually, the terrain must still be considered semi-arid.

The Central Mountains and Transition Zone contain peaks to 12,600 feet. At these elevations, rainfall ranges to as high as 30 inches per year. In higher regions, protection of subgrades from winter frost damage is an important consideration. Nevertheless, data published by the Corps of Engineers (125) concerning the Mean Freezing Index indicates that only a very small proportion of Arizona can expect degree-day readings below 32°F to exceed 100, which is quite temperate compared to the more northerly states.

2.2.2 Desert Soil Conditions

Throughout the Basin and Range province, most of the highways traverse the intermountain basins. The soils covering these basins are primarily alluvial, that is, they have been deposited by flowing water, and are derived from granitic, volcanic or metamorphic parent rock.

Two general categories of alluvial deposits predominate. Recent alluvial fan deposits and alluvial deposits in the floodplains of intermittent streams cover much of the basins of southern, central and western Arizona. Over the remaining parts of these basins, erosion surfaces expose older continental

alluvial deposits and, to a lesser extent, playa (ancient lake bed) deposits.

A detailed study of the Rio Grande Valley in southern New Mexico by Ruhe (126), provides an insight into the history and processes of formation of these deposits as well as their interrelationship with engineering properties. The area of this study is a direct geologic counterpart of many of the basins in Arizona. Extensive carbon 14 dating in Ruhe's study indicates the recent alluvial deposits were formed in the past 10,000 years during the arid period since the last ice age. The engineering properties of these soils, which are very sensitive to weakening by moisture increase, appear to be primarily the result of processes of sedimentation peculiar to a dry environment. Most of the older deposits are more than 50,000 years old, and many are several hundred thousand to several million years in age. Owing to the complex processes of induration and cementation which are not fully understood, these soils are generally very firm and less sensitive to moisture changes.

Both the younger and older alluvial deposits are composed predominantly of silty sands, sandy silts, clayey sands, and lean sandy and silty clays. However, highly plastic (and highly expansive) clays are occasionally present in both the alluvial deposits and in the lake bed deposits.

Much of the recent alluvium consists of low gradient alluvial fan deposits sedimented in a manner as

described by Blessenback (127) or of intermittent stream and slope wash deposits which involve a similar depositional sequence. In this depositional sequence, each layer is laid down by a flash flood or sheet flow. The layer dries out before the overlying layer is deposited. Sedimentation of the next layer wets only the surface of the underlying material. In this way, the deposit, as a whole, is built up without being consolidated under its own weight at high moisture contents. During the process of formation, the silt and sand grains become "tack welded" and together with minor clay fractions and trace amounts of lime give the deposits moderate dry strength. These deposits are greatly weakened by increased moisture and sometimes will settle under their own weight upon saturation. The more severely moisture sensitive of these deposits are primarily silty or slightly clayey granular materials.

Although "collapsing" soils have been recognized to some degree, at least since the 1930's, the types of "collapsing" soils which occur in the Southwest have only been widely treated in engineering literature for about the past 20 years. General reviews of the subject have been presented by a number of authors, including Dudley (128) and Bara (129).

For the older alluvium, the presence of substantial calcareous cementation creates a major element of strength and stiffness. Although the means by which the cementation is formed is uncertain for many

deposits, infiltration of intermittent rainfall to a shallow depth below the surface has undoubtedly been a major means of deposition. The water picks up mineral salts as it percolates through the soil and later evaporates leaving the salts as cementation in a well defined horizon. The fluctuation of water tables and evaporation of brines in small lake beds also probably formed some of the older cemented basin deposits. Most of the older alluvium possesses at least moderate to strong calcareous cementation and some deposits are cemented to rocklike hardness. Such calcareous deposits are often termed "caliche" and have been discussed, in detail, by Reeves (130) who presents a map showing their very wide distribution in Arizona.

A portion of the older basin alluvium deserving special mention is the conglomerate. At the time of or shortly following the Basin and Range Orogeny (early Tertiary), these silty or clayey sand-gravel-cobble mixtures were laid down close to mountain fronts. They have since been cemented, occasionally to the point of being lithified. The local name for many of these deposits is Gila Conglomerate, after the considerable extent of such material along the Gila River. The fanglomerate cut test section is an example of this type of formation.

2.2.3 Plateau and Mountain Soil Conditions

The soils of the Colorado Plateau are mostly alluvial or residual deposits composed of highly plastic

clays, sandy clays and silty sands derived from the sedimentary rock strata. Often, however, residual soils at depth are covered by several feet of wind-blown sand or floodplain alluvium. As is the case in the deserts, surface alluvial soils of the Colorado Plateau and Central Mountains have generally become lime cemented. Because the sedimentary deposits of the Colorado Plateaus are predominantly interbedded sandstones and shales (siltstones, mudstones and claystones), both the residual and alluvial soils derived from these formations are highly variable and all combinations of sand-silt-clay mixtures are found. Many of the soils are gypsiferous and ion exchange phenomena associated with highly soluble salts have a major influence on the soil's behavior.

Severely "collapsing" soils are found in recent alluvial fan and intermittent stream deposits derived from the sedimentary rock. The formational sequence is the same as described in section 2.2.2. Likewise, cemented alluvium on the Colorado Plateau is similar to deposits described in section 2.2.2.

Highly plastic and highly expansive soils associated with clay-shales are widespread. The most widely distributed are the Chinle Formation soils which have been related to severe highway subgrade problems over much of northern Arizona. The extent and character of this formation is outlined in detail by Stewart, et al (131).

Expansive soils associated with sedimentary units of the Mesa Verde Group, including the Mancos Shale, are also widely distributed in northeastern Arizona; mostly on the Navajo Indian Reservation.

Much of northern Arizona and the Central Mountains consists of basalt and other products of a volcanic environment. These include deposits of cinders, welded tuffs and highly plastic (and highly expansive) clays which are derived from the weathering of volcanic ash. Often these products are lime cemented and form a very hard near surface material.

Most of the clayey subgrades of northern and central Arizona are very dry and desiccated much of the year. On the Colorado Plateau, the near surface soils exhibit stress relief (overburden has been eroded away) and, therefore, do not fit the classical model of a saturated normally consolidated clay.

2.2.4 Other Soils

Clean sands and sand-gravel mixtures are found in and near many of the drainage channels throughout the state. Such deposits are free of cohesive fines and cementation. Likewise, many nonplastic windblown deposits of dune sand many feet thick are to be found in northern Arizona. There appears to be no reason why their behavior should not be the same as that described in numerous reports on cohesionless granular soils.

The classical condition of saturated normally consolidated clays is practically nonexistent in Arizona. Because of long periods of exposure to the sun, clays crack and spall and usually become highly overconsolidated due to the internal stresses of desiccation. This results in stress regimes which are very complex and are often similar to stress relief phenomena.

2.3 Site Selection

2.3.1 General

In the first phase of field testing, it was desired to choose three sites where detailed comparative testing could be performed. Available roadways presently in graded and drained condition were reviewed in a meeting with Arizona Department of Transportation personnel. The three sites, I-10 near Buckeye, Lake Pleasant-Morristown Road, and Cherry Road near Cienega Creek were chosen, based on considerations of subgrade properties typical of those of interest, proximity to Phoenix for travel considerations, and minimal interference from traffic.

Following analysis of data from the first three sites, the authors sought other test stations with the same emphasis on subgrade properties, but with an additional element - destructability. Accordingly, Phase II field testing was performed on private property where permission was obtained to dig deep trenches for subsurface testing and where artifical

saturation of the subgrade could be performed without disturbance of a highway right-of-way.

The Rainbow Valley and McCormick Ranch test sites met these needs.

At this point, the physical environment surrounding each of the sites will be discussed.

2.3.2 Sites for In-Situ Testing

A. The I-10 Buckeye test site consists of a continuous strip approximately 230 feet long located within the outside westbound lane of Interstate 10, just east of the Miller Road Interchange. The location is noted on the vicinity map shown as Figure 27. The testing reported herein took place subsequent to final grading of the roadway, but prior to paving.

The test strip begins at station 6073+00 and extends to 6075+30. At this location, the Interstate Highway traverses a series of alluvial fans of comparatively recent age emanating from the White Tank Mountains. This particular spot was chosen for examination because the roadway is cut into the native materials from stations 6073+00 to 6075.00. The last 30 feet of the test strip is a fill section covering a small arroyo.

Three exploratory borings, which consisted of continuous split spoon sampling (ASTM D1586 in

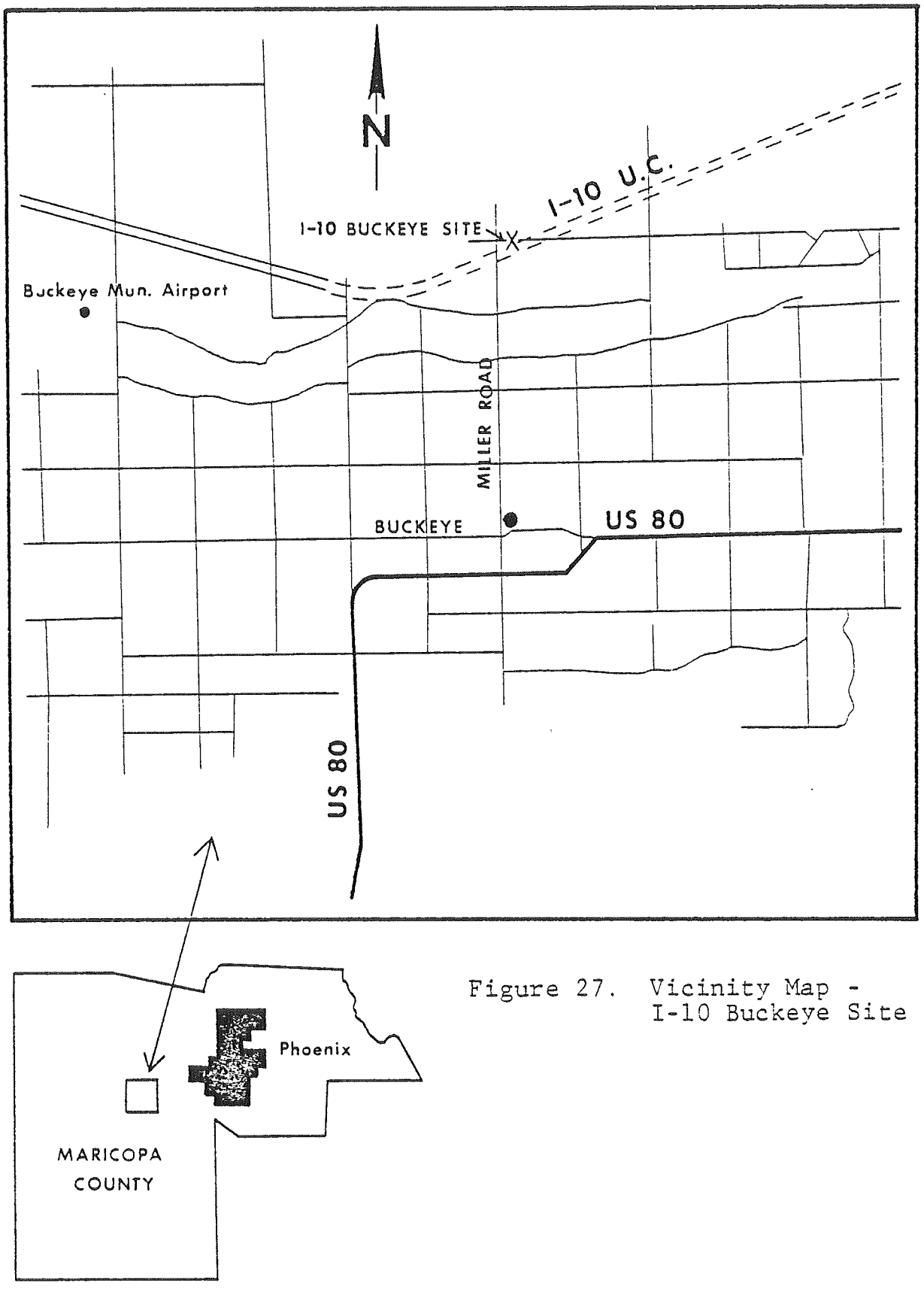


Figure 27. Vicinity Map - I-10 Buckeye Site

12-inch increments, were performed at stations 6073+10, 6074+00 and 6075+10. The logs of these borings are given in section 3.1.1. The native materials in the cut section were found to consist predominantly of sandy clay and clayey sand of low to medium plasticity. The soil profile is moderately cemented, as indicated by number of blow counts recorded for the 6-inch increments of sampler penetration. Such a degree of cementation is considered very typical of alluvial deposits in the deserts of southern and western Arizona.

The fill material above the arroyo is apparently locally derived, and subgrade has been scarified and recompacted to at least 6 inches throughout the test section. The physical and engineering properties of the fill were found to be little different from adjacent native materials. This seems to indicate that upon drying, a recompacted desert soil containing considerable calcium carbonate may, within a few months, "set up" to a degree of hardness comparable to its original condition.

Beyond station 6075+00, the fill reaches to a depth of about 4½ feet overlying loose silty sand and gravel from the arroyo.

The value of the I-10 Buckeye site lies in the fact that it is most typical of Basin and Range terrain, with lime cementation playing a major role.

The degree of cementation is in the middle range of conditions insofar as induration and strength properties are concerned.

B. Lake Pleasant-Morristown Road Site

The proposed Lake Pleasant to Morristown Road, when completed, will connect Morristown on U. S. 89 south of Wickenburg with Interstate 17 near New River. The segment of graded and drained roadway which was selected for testing is located between the Agua Fria and New Rivers, approximately 3 miles southeast of Lake Pleasant. A general location of the site is shown on Figure 28.

The alignment traverses a pediment surface of a type which is common along mountain fronts in the Southwest. This is a gently inclined erosional surface veneered with a foot or so of fluvial gravel and residual soil. Exploratory drilling was attempted at several locations along the roadway, but at most of the locations, the auger met refusal on rock within the upper 2 feet. A cut section where the road drops from the pediment to floodplain terraces of the Agua Fria River revealed the near surface rock (and likely the cause of refusal) to be a fanglomerate; that is, a heterogeneous mixture of the transported rock fragments which have been so strongly cemented by calcium carbonate that the material is in fact lithified.

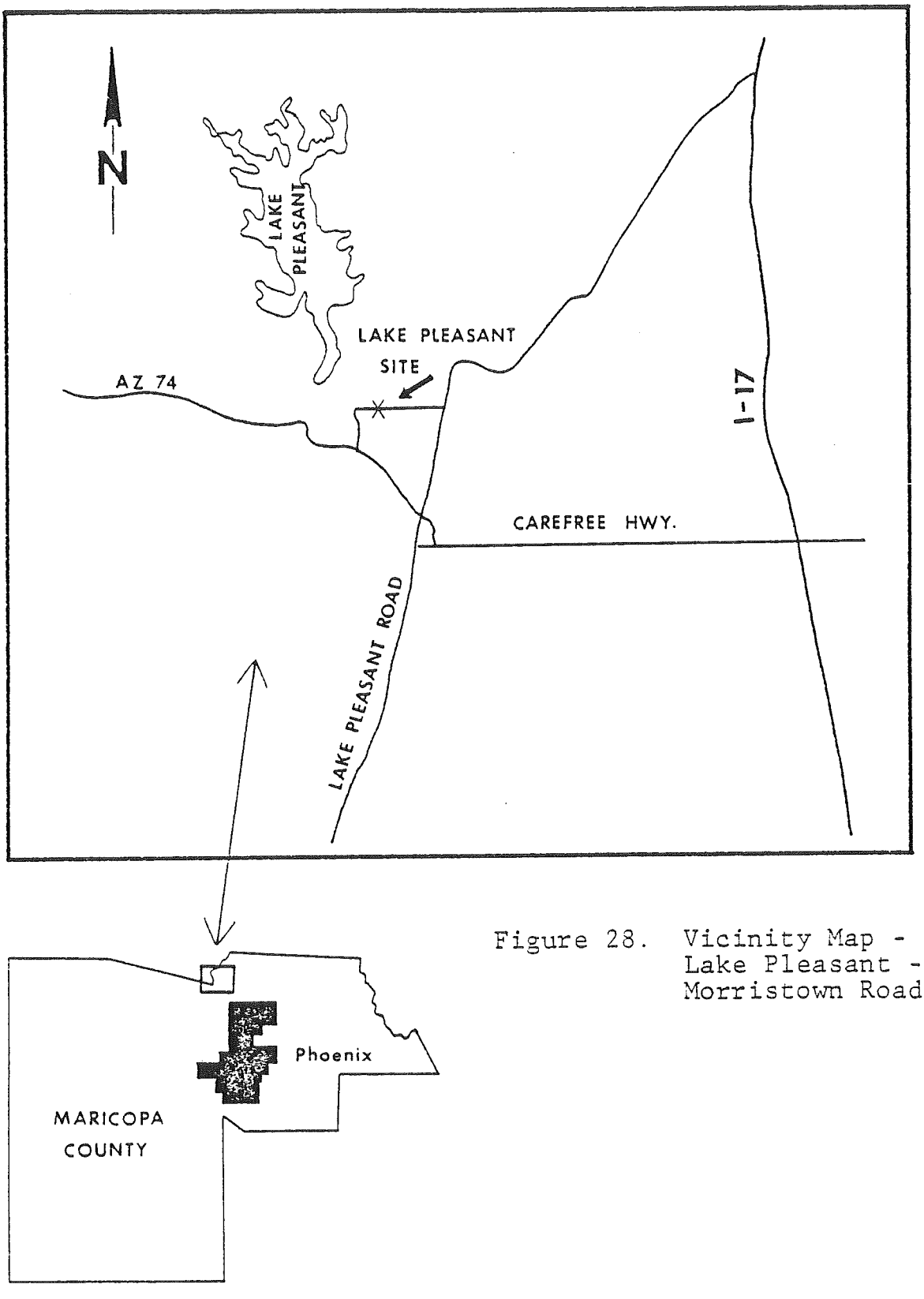


Figure 28. Vicinity Map - Lake Pleasant - Morristown Road Site

It was decided that testing would be performed at three stations along the roadway. At the first spot, designated the "On-Grade" site and represented by boring LP-1 in section 3.1.2, there was found to be approximately 1½ feet of unconsolidated material above the rock. The upper several inches have been recompacted to form the roadway. The second spot was approximately 1000 feet to the west and very near the Arizona Department of Transportation's subsurface environmental monitoring station. It was designated the "Orange" site and rock was found to be approximately 1 foot below the surface (see log LP-2 in section 3.1.3). At the crest of the grade where the roadway began to drop from the pediment surface, the test spot designated "Fanglomerate Cut" was placed. Here, the lithified alluvium was at the surface.

The Lake Pleasant-Morristown Road site was intended to be representative of the upper bound of hardness which could be found among cemented subgrade profiles. The Fanglomerate Cut location certainly met this criteria. The other two spots were chosen for testing in order to examine the response of 1 to 2 feet of typical compacted desert alluvium placed over a stiffer material.

C. Cienega Creek Site

The Cienega Creek site is located on Arizona 169 (Cherry Road) immediately west of the interchange

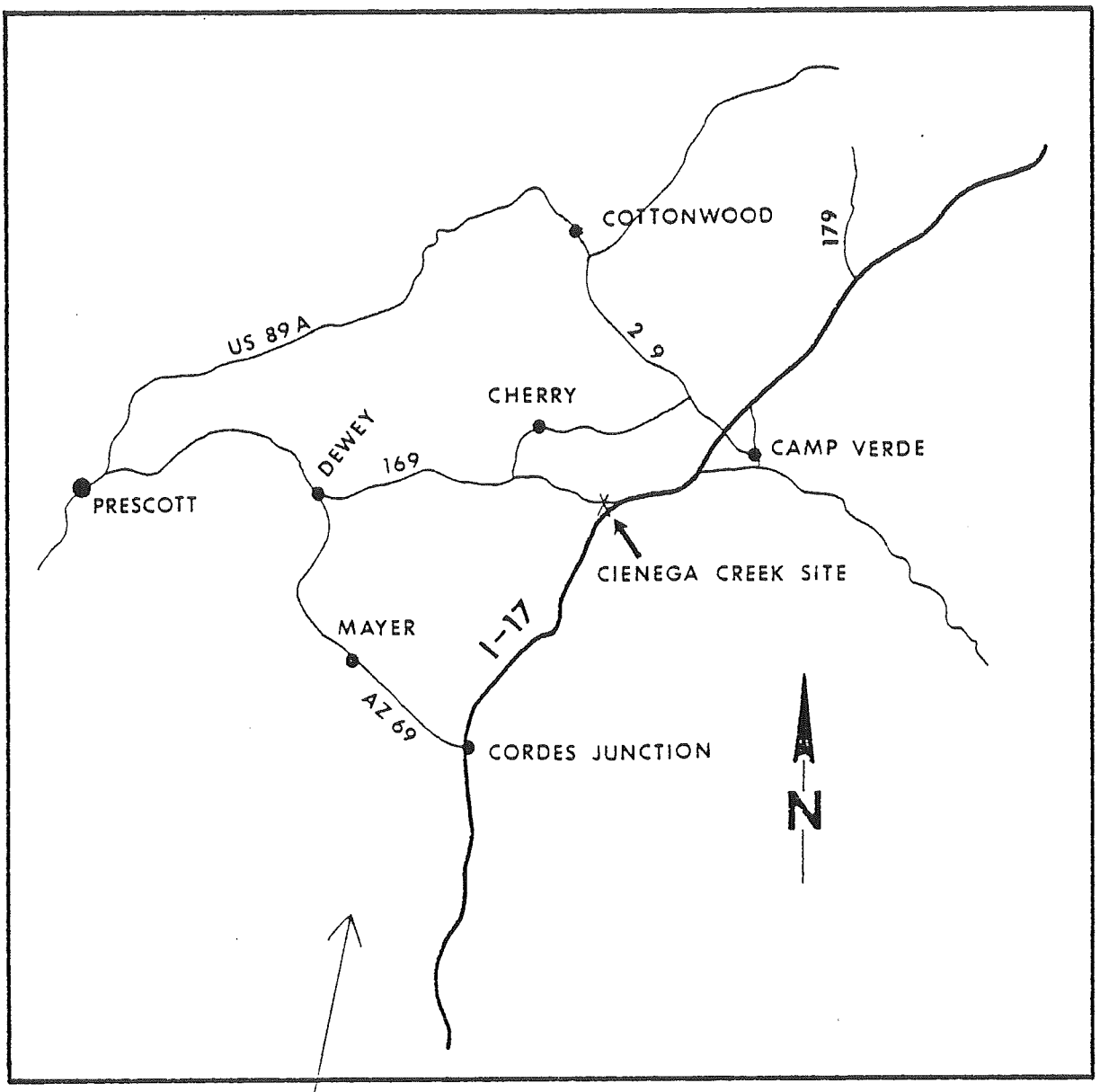
with Interstate 17 in eastern Yavapai County. A vicinity map is provided by Figure 29. The graded and drained roadway consists of a foot of clayey sand subbase course over a subgrade consisting of 3 feet of highly plastic clay. Beneath the clay is a moderately to slightly weathered granite. A log of the split spoon boring is given in section 3.1.3.

The Cienega Creek site is typical of many in the central mountains of Arizona. The elevation is approximately 5000 feet and annual rainfall is more than twice that which falls on the desert. Soils at this site are predominantly residual, but the upper few feet may also contain clays derived from weathered volcanic ash deposits. Classification test data indicates that they possess a moderately expansive potential.

A subgrade such as is found at Cienega Creek may be expected to be more sensitive to moisture changes than desert sites. The plate bearing and geophysical tests were performed in the late fall while the surface of the roadway was in an extremely dry condition. The resulting desiccation (which creates the same effect as overconsolidation in the clays) probably provided a stiffer condition than would be expected at other times of the year.

D. Rainbow Valley Site

In the second phase of field testing, objectives



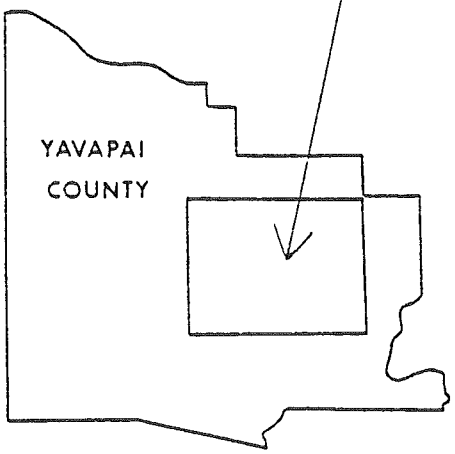


Figure 29. Vicinity Map - Cienega Creek Site

were to examine the effect upon subgrade response of saturation and of changes in modulus with depth or confining pressure. For these, a special test station was needed - one with complete freedom from traffic or other outside interference and one where permission to dig into the subgrade could be obtained. In the Rainbow Valley test site, these requirements were met.

The Rainbow Valley test site is located on an isolated piece of ranch property. This parcel is located on an alluvial fan emanating from the Buckeye Hills. It is approximately 30 miles west of Phoenix and 4 miles southeast of Buckeye. An area of approximately 150 feet square was made available. For purposes of testing, a 100 by 75 foot grid was charted and staked, and all test locations are tied to this grid.

The soil at the Rainbow Valley site, as given in the composite log on Figure 39 (section 3.1.4), is generally a silty sand with trace amounts of clay and some gravel. In the topmost foot of native material, the lime cementation is predominantly weak, but below this, the cementation is moderate to strong in most places.

The Rainbow Valley site proved to be extremely valuable from the standpoint of data acquisition from an uncompacted subgrade. Like the I-10 Buckeye site, Rainbow Valley is typical of desert subgrades with regard to the degree

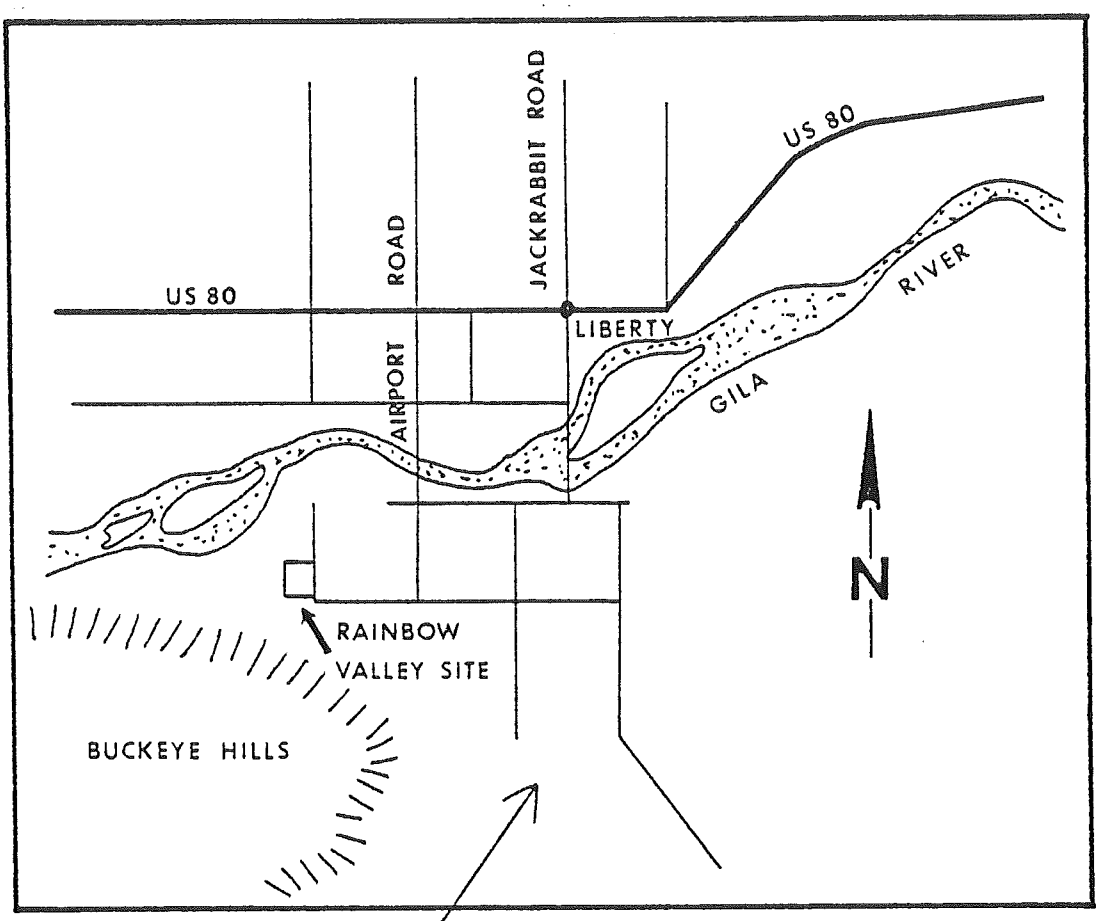
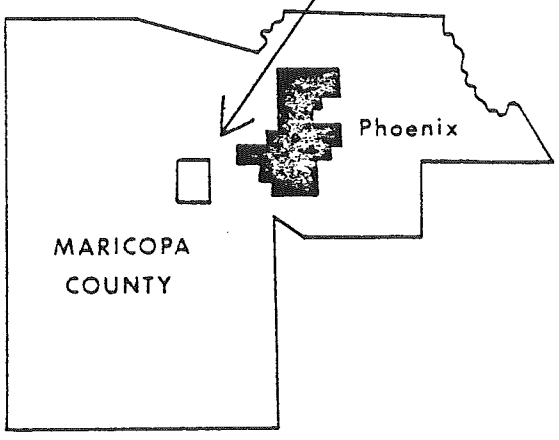


Figure 30. Vicinity Map - Rainbow Valley Site



and spatial variation of cementation. Because of the favorable working arrangement, the volume of response data collected at Rainbow Valley is greater than at the other test sites.

E. McCormick Ranch Site

The McCormick Ranch site provided a unique opportunity to perform testing on a controlled engineered fill of material that is similar to many subgrades in the basins of southern and western Arizona. At the same time, a nearby site consisting of the same material in native condition was available for comparison testing.

McCormick Ranch, in north Scottsdale (see Figure 31), is geologically much younger than the other test sites, and is typical of reworked basin alluvium. As seen in the description on Figure 41 (section 3.1.5), the soil type is a borderline silty clay - clayey silt. Near the surface of the native (desert) site, very little cementation has developed.

The fill, which was the site of most of our tests, was placed in 1972 to a depth of about 3 feet. Compaction was to 95 percent of Standard Proctor (AASHTO T99 or ASTM D698). There has been no activity on this site during the 5 years following the placement of the fill. Test grids, 50 feet by 75 feet, were staked at both the fill and undisturbed desert locations for control of test placements.

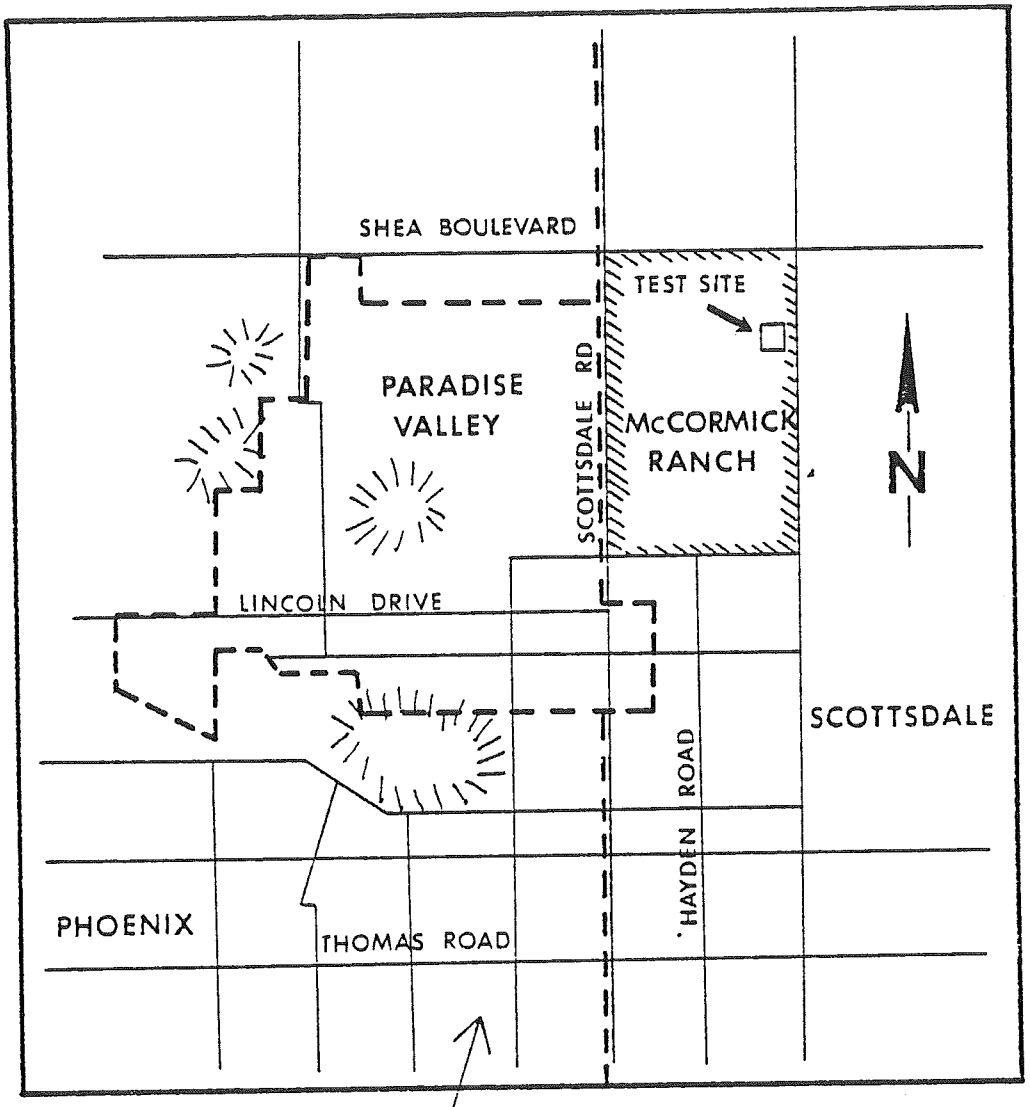


Figure 31. Vicinity Map - McCormick Ranch Site

Like Rainbow Valley, the selection of this test site provided an excellent opportunity to examine spatial variation in modulus. The difference, however, lay in the fact that for a relatively homogeneous fill, little variation in modulus was expected. The McCormick Ranch sites also provided the opportunity to observe the effects of increased moisture on firm, though uncemented, subgrade materials.

2.4 Outline of Field Testing Program

2.4.1 General

As a first step at each of the five test sites, exploratory drilling and sampling, soil classification tests and dry density and moisture content determinations were initiated in order to characterize the subgrade environment in terms of commonly used parameters. In Phase I of the field testing, elastic modulus was determined using four field techniques; namely, repetitive plate bearing tests, surface refraction seismic, Dynaflect and Road Rater. In Phase II, four techniques were also employed, repetitive and nonrepetitive plate bearing tests, refraction seismic, in-situ CBR and Dynaflect.

Visual summaries of field testing at each of the various sites are provided in Section III. Each field procedure and its attendant means of determining modulus is discussed below.

2.4.2 Test Methods

A. Standard Penetration Testing and Sampling

At each of the test locations in Phase I, continuous sampling of the subgrade to a depth of 5 feet or refusal was accomplished using a 2 inch O.D., 1 3/8 inch I.D. split barrel spoon sampler. In some cases below 2 feet, reaming of the hole with the 4½ inch continuous flight auger was necessary to alleviate friction on the sampler. An index of driving energy (and hence the stiffness of the subgrade) was recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the sampler in 6 inch increments. This is the ASTM D1586 procedure by which standard penetration number (N) is obtained.

At the Phase II sites, the number of locations where sampling was performed was increased, in order to examine spatial variations, but the depth of examination was reduced to 24 inches.

B. Classification/Atterberg Limits Tests

Samples were examined and visually classified in the field. Laboratory index tests including grain-size analysis (ASTM D422) and Atterberg Limits (ASTM D423 and D424) were performed on many of the samples to aid in classification. Results of these tests are available in section 3.1. In addition, moisture contents of

various samples were obtained by ASTM D2216. These values of moisture content are shown on the logs and figures which are presented in Section III.

C. In-Situ Density

In-situ values of dry density and moisture content were determined at various locations at each of the test stations using the sand cone density method (ASTM D1556). The results are available in section 3.1. These values were necessary for use in E modulus calculations using the shear and compression wave velocity values.

D. Plate Bearing Tests

In the first phase of testing, 6 cycle repetitive plate bearing tests were performed in accordance with ASTM D1195. The SHB truck-mounted load frame which was specifically designed and built for pile and plate load tests was used to provide the reaction weight. Load was applied to circular, 3/4 inch thick steel plates. Plates 12, 18, 24 and 30 inches in diameter were used at various spots in the testing program.

The plates were seated on a level bed of Ottawa sand 1/8 to 1/4 inch thick and were pyramided as recommended in the ASTM specifications. A calibrated ram action hydraulic Porta-pump jack was used to apply load to the plates. At first, a

hand pump was used to raise the ram; however, it was found that efficiency could be increased by using an air compressor in conjunction with an air to hydraulic pump to provide the necessary fluid pressure to the Porta-pump.

Measurements of deflection were taken on three Ames dials bearing near the edges of the plates. Loads were applied and deflection readings taken after the rate of secondary compression was less than 0.001 inch per minute for 3 minutes as specified by ASTM standard. Loads were then released, the rebound noted, and the load reapplied. This sequence was repeated six times for each load increment and was generally done for four or five values of load so as to be able to plot an accurate load deflection curve.

In the second phase of testing, both repetitive and nonrepetitive (ASTM D1196) plate bearing tests were included at the two test stations. The larger number of tests were nonrepetitive as it was felt that a greater volume of useful information could be gathered by this procedure. The nonrepetitive tests were performed using the same procedure as described above, the only deviation being that the load is not released until the end of the test.

Elastic modulus can be calculated from the plots of stress versus deflection for the various plate bearing tests.

The general formula which applies is:

$$E = \frac{qb\ (1-\nu^2)}{\rho} I_p$$

where

E = elastic modulus of the subgrade (psi)

q = pressure in (psi)

b = diameter of plate in inches

ν = Poisson's ratio

ρ = deflection

I_p = influence factor, dependent upon shape and rigidity of the plate.

There are several schools of thought on the way in which E modulus should be calculated from plate bearing tests. Since soil rarely behaves as a linearly elastic material over more than a very narrow stress range, the computed value of E is a function of the stress level. In any design problem, a stress level comparable to that in the prototype structure is the one which should be used.

It will be shown in Section III, that the values of modulus for a given soil calculated using the above formula often differ greatly, depending on the size of the plate. Again, the question arises as to which size is most appropriate for determining elastic modulus for highway pavement design. It will be shown in Sections III and IV that the larger plates usually yield more realistic values for pavement design problems.

The question also arises, when dealing with a repetitive plate bearing test, as to whether the E modulus should be measured using the virgin curve, or using a rebound point as a new zero (see Figure 32). In many instances, there may be large differences in the calculated values of modulus. The answer should depend on the type of analysis for which the modulus is intended. In dealing with pavement systems and the response of subgrades under repeated loads, it might, at first, appear that a modulus measured on the rebound curve would be appropriate. Indeed, the resilient modulus M_R is physically compatible with such a convention.

On the other hand, when dealing with highway pavement structures, the comparatively small and short-lived stresses which are transmitted to the subgrade may provoke a response which is nearly in the elastic range, therefore, modulus calculated from the virgin stress-deformation curve is realistic.

It should become even more reasonable, in light of the large spatial variations in subgrade response, to use the simpler conventions to characterize subgrade modulus.

E. Surface Refraction Seismic

It was stated earlier in this report that precise means of assigning a value of elastic

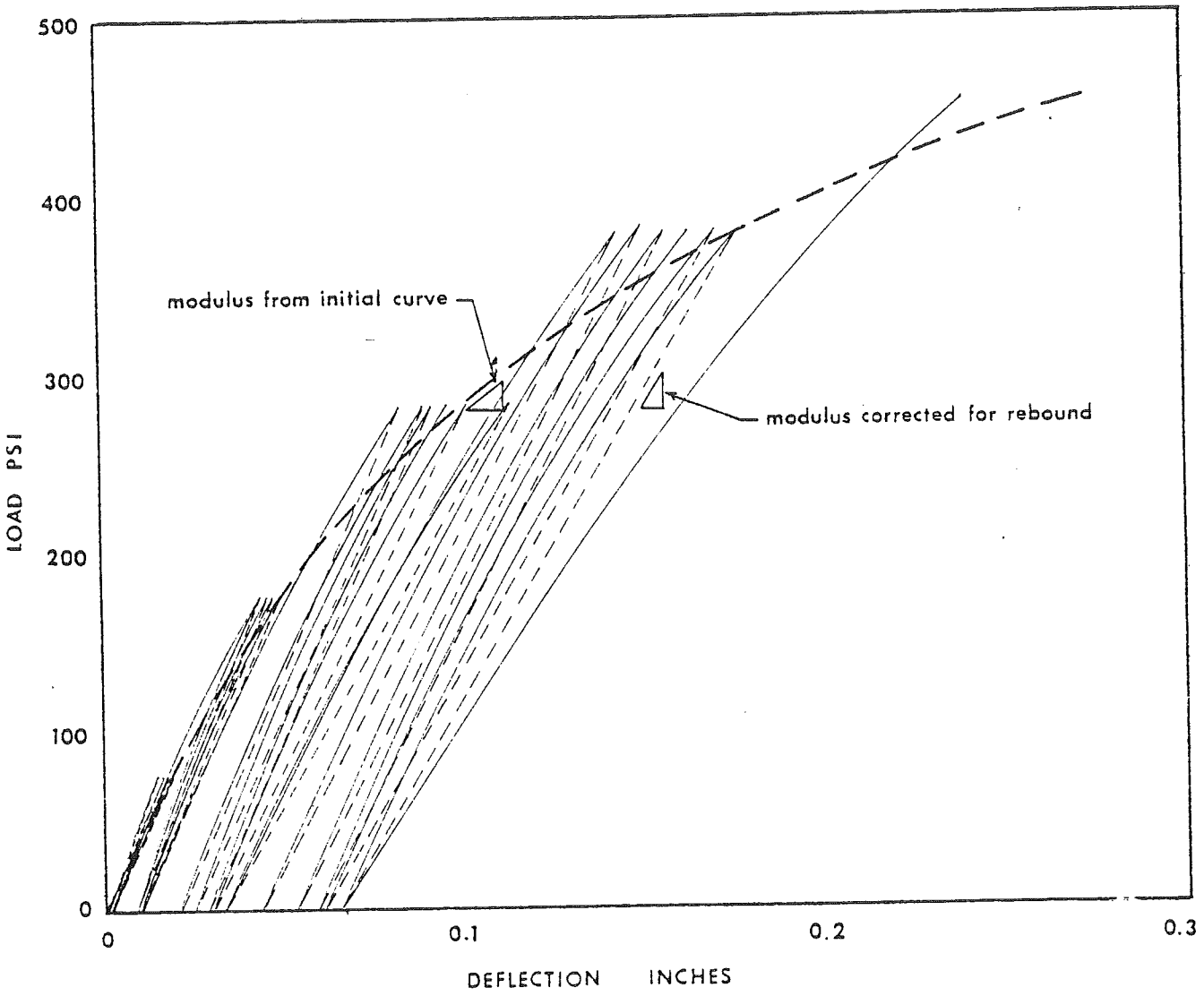


Figure 32. Example of Stress Path for a 6-cycle Plate Bearing Test

modulus to an earth material can be accomplished if measured values of shear and compression wave velocities are known. Elastic modulus (E) is defined by the relationship

$$E = V_p^2 \rho \frac{(1+\nu)(1-2\nu)}{(1-\nu)}$$

where

V_p = compression wave velocity

V_s = shear wave velocity

ρ = mass density

ν = Poisson's ratio

Poisson's ratio is precisely defined by the relationship

$$\nu = \frac{(V_p/V_s)^2-2)}{2(V_p/V_s)^2-1)}$$

In routine geophysical field work, the generation and recording of arrival times of compression waves presents little difficulty, inasmuch as they travel fastest through any medium and are first to be seen on a wave trace. The later arrival of the shear wave is obscured and, therefore, special techniques and interpretations are required to determine shear wave velocity. In section 1.7, crosshole seismic and Rayleigh wave correlation techniques were briefly discussed as the most widely used techniques. Such techniques are preferred for examining deep profiles.

During the first phase of field testing, it was desired to precisely determine shear and compression wave velocities of the topmost 5 feet of subgrade at each test site. Consulting geophysicist Richard T. Zbur of Albuquerque, New Mexico, was employed as a subconsultant. Mr. Zbur utilized another method of generating shear waves called the unidirectional method; specifically, the "horizontal traction method".

This method involved striking a wooden plank (84 inches by 11 inches by 7 inches) placed on the ground surface with the long axis of the timber oriented perpendicular to the geophone line. A vehicle was driven upon the plank to establish a secure coupling with the ground surface. The timber was struck horizontally on one end with the sledgehammer and the initiated shear waves were recorded with the aid of the horizontal geophones. The plank was then struck on the opposite end and the identifiable reverse phase of the shear waves was recorded. Vertical impacts striking the timber produced good compressional waves and these arrivals were detected by the vertical geophones.

Geophysical equipment utilized included a portable analog/digital, high speed, dry recording seismograph system consisting of a Nimbus Model ES-1200 12-channel signal enhancement engineering seismograph. The high-speed oscillograph recorder provided recording capability of 0.1

microsecond resolution. Low frequency (4.5 hertz) Mark L-1 vertical and horizontal geophones were used.

The compression and shear waves are generated using a hammer-impact energy source. A 20 pound sledgehammer, with a piezoelectric impact switch attached, was used as the energy source.

Spread lengths and geophone spacing were held to small dimensions owing to the depths to be surveyed and the resolution required in this study. Spread lengths of 30 feet employing six pair vertical and horizontal geophones spaced at 5 foot intervals were used to monitor the seismic wave arrivals.

It is well to remember that the amplitude of subgrade displacements generated by these impacts were probably on the order of 10^{-4} to 10^{-6} inches and that the subgrade elastic moduli thus calculated apply to that strain level. We have stated previously that elastic moduli of soils are particularly strain sensitive. The values of modulus calculated should be adjusted downward in some way when dealing with larger amplitudes of displacement.

It has been previously mentioned that Hardin and Drnevich (45) have made such adjustments for sand.

In a like manner, it is not unreasonable to assume that in cohesive soils or cemented soils, shear and compression wave velocities (and hence elastic modulus), are functions of amplitude of displacement or vibration. Therefore, the values derived should be considered the "upper bounds" of modulus and the effects of strain softening carefully considered.

In Phase II, surface refraction seismic testing was performed using the Bison 1570B Signal Enhancement Seismograph. With this instrument, only compression wave velocity is recorded and it is therefore necessary to assume values of Poisson's ratio to calculate E modulus. Because of the savings in cost and the desire to gather a large volume of data in Phase II, this compromise was considered acceptable.

Compression wave velocity is determined by measuring the time of travel of a wave created by the impact of a hammer blow on a metal plate placed a known distance from the recording geophone. The waveform is viewed by the operator on a CRT screen. A unique feature of this model is its ability to add several pulses to form a combined trace, in order to more clearly identify the arrival time.

F. California Bearing Ratio

The field in-place CBR tests were performed according to specifications published by the U. S.

Army Corps of Engineers. The assembled apparatus for performance of the test was fitted for placement against the back bumper of a drill rig.

In running the test, a standard 3 square inch piston is jacked into the ground at a constant rate. The load causing specified penetration (0.1", 0.2", etc.) is then compared with the standard in order to calculate CBR. The CBR value must be related to modulus by some empirical means such as E = 1500 x CBR.

The results of CBR tests performed at Phase II sites are given in section 3.4.

G. Dynaflect

The Dynaflect is an electromechanical device used to measure induced dynamic deflections on pavement structures. The dynamic force generator is mounted on a small trailer which is towed behand a van or truck. It employs counter rotating masses which consist of unbalanced flywheels. Transfer of load is through two steel wheels placed 20 inches center to center. The peak to peak dynamic force is 1000 pounds and is applied to a frequency of 8 hertz.

The deflection basin is charted from readings taken by five motion sensing accelerometers which are set along a line between and parallel to the wheel paths at 1 foot spacings.

The Arizona Department of Transportation Dynaflect was utilized at each of the five sites. Readings were taken at several designated spots in the various test strips. At the Phase I sites, an attempt was later made to operate the Road Rater at precisely the same spots. Readings were taken and corrected for temperature by ADOT personnel using procedures described by Southgate (132).

The calculation of moduli from Dynaflect data has been the subject of several investigative efforts. Three of the most important, which will be discussed here, involve the work of Scrivner and others (133) at the Texas Transportation Institute, Vaswani (134) at the Virginia Department of Transportation, and Majidzadeh (135) at Ohio State University. Other excellent work has been performed by the Utah and California Departments of Transportation. All of the above analyses made use of two or three layered elastic systems.

Work done by Scrivner has results in a new computer program called "ELASTIC MODULUS II" which is capable of converting Dynaflect deflections to elastic moduli in two layered systems. In this analysis, use is made of d_1 and d_3 which are deflections at the center and third stations. The 500 pound loads transmitted by each wheel are considered point loads. The back calculation of modulus makes use of Burmister's early work on

two layered elastic systems. The computer program converges on the pairs of E values by means of an iterative process. The difficulty with ELASTIC MODULUS II is that the set of values determined for E_1 and E_2 may not be unique, and judgment may be required for selection of the proper set.

Vaswani did extensive work using all five sensors and developed the concept of spreadability as an index parameter. Spreadability is defined as

$$S = \frac{d_{max.} + d_2 + d_3 + d_4 + d_5}{5d_{max.}}$$

where

$d_{max.}$ = deflection at the sensor between the two force wheels

d_2 = deflection of second closest sensor, etc.

Vaswani showed that for a semi-infinite homogeneous elastic subgrade, the spreadability will theoretically be 31.35 percent. For systems with a strong upper layer over a weak subgrade, the spreadability will be greater than the above value, while for a weak top layer over a strong base the spreadability will be less.

Charts prepared by Vaswani summarize the computed modulus of elasticity of subgrades for various

thickness-strength combinations of flexible pavements based on maximum Dynaflect deflections. It appears, however, that because subgrades are rarely perfectly elastic, spreadability of the subgrade is usually not 31.35 percent, thereby introducing a degree of uncertainty into the analysis. Nevertheless, Vaswani's findings are important because they put into perspective the importance of maximum deflection as an index of upper layer stiffness and deflection of the outer sensors as an index of subgrade stiffness. Majidzadeh summarizes these and other Dynaflect procedures, examining all the parameters which have been suggested. Majidzadeh concludes with a series of charts of his own for determining subgrade modulus in a two layer system using d_1, d_2, d_5 and an assumed or known value of E_1 (modulus of the pavement).

The writer has examined all three of these methods and found them wanting for purposes of interpreting Dynaflect readings taken directly on a subgrade. In examining deflection basins generated by both the Dynaflect and Road Rater, it was found that the subgrade response rarely resembles the homogeneous isotropic model. Furthermore, it seems apparent that the response varies spatially even over a 5 foot radius to such a degree that a method which relies heavily on d_1, such as Vaswani's, or d_5, such as Majidzadeh's, may give an exaggerated rather than representative value of modulus.

In section 3.5, the writer will suggest an empirical method for determining E_s modulus from Dynaflect data taken directly on a subgrade, using the Ahlvin and Ulery expression for deflections caused by uniform circular loadings.

H. Road Rater

The Road Rater is an electrohydraulic oscillator whose mission is similar to that of the Dynaflect; that is, to apply a dynamic load to a pavement structure and measure its response in the form of a deflection basin. The Road Rater employs a hydraulic actuator which is mounted on the front of an over-the-road van and is operated from the cab. Hydraulic flow is directed to either side of the piston causing the mass to be displaced in an up/down motion, thus, applying a sinusoidal dynamic force to the pavement. The contact with the pavement takes place through two 4 inch by 7 inch rectangular steel pads spaced 6 inches apart. Measurements of deflection are taken at four locations, 1 foot apart, with the closest reading between the two pads. As with the Dynaflect, motion sensing accelerometers are used.

The advantage of using the Road Rater, as compared to the Dynaflect, is that the dynamic load and frequency may be changed as needed in order to apply the necessary amount of energy to a

pavement system to invoke a meaningful response. This can be very important when dealing with thick rigid pavement slabs.

The Road Rater is most commonly used for street systems or airfield evaluation. For such projects, usually only one load and frequency is used throughout. In this research effort, it was hoped that by using various frequencies and dynamic loads at the same spot, discernible trends regarding apparent values of E modulus could be determined.

A Model 400 Road Rater from La Belle Consultants of Santa Ana, California, was used at all three sites. This particular machine operates at discrete frequencies of 10, 20, 25, 30 and 40 hertz. An attempt was made to place the pads on exactly the same lines as were the Dynaflect wheels. At each test station, deflection basins were measured using several frequencies and loads. Each change in frequency results in a change in the peak to peak dynamic load according to the relationship

$$P = \pm .0511\ W\ f^2\ D$$

where

P = peak dynamic load in pounds
W = weight of the mass in pounds (160 lbs.)
f = frequency - cycles per second
D = amplitude of displacement of the mass in inches

It can be seen from the formula that the dynamic load can also be varied by changing the amplitude of displacement of the hydraulic cylinder. The static load about which the sinusoidal load is applied can also be varied by changing the hydraulic gauge pressure in the cylinder. After deflection basins had been measured at each of the test spots using a gauge pressure of 600 psi, the Road Rater returned to the various stations to measure deflections using a higher gauge pressure (and hence greater static load) of 900 psi. An unedited copy of the La Belle report is given in the Appendix.

At the present time, the writer knows of no published method of analyzing Road Rater data to determine subgrade modulus. An empirical procedure is given by the writer in section 3.6.

SECTION III. RESULTS OF FIELD TESTING

3.1 Results of Soil Characterization Testing

3.1.1 I-10 Buckeye Site

Figure 33 provides a visual summary of the testing performed at the I-10 Buckeye test station. Figures 34a through 34c are logs of the three penetration borings which were performed at the test site to characterize the subsurface materials. Standard penetration numbers (6-inch increments) are shown on the logs as are the laboratory determined moisture contents. The results of sieve analysis and Atterberg Limits testing of selected samples are given in Table 2.

It should be remembered that this particular location was chosen for the test section because it provided one of the few cut sections along the graded and drained alignment. The borings show there is little physical difference between the scarified and recompacted subgrade material near the surface and the undisturbed soil beneath. The soils can be generally described as sandy clay-clayey sand of low plasticity, and weakly to moderately lime cemented. One would expect that cementation near the surface would have been destroyed during the road building. It appears, however, that the residual lime is sufficient; such that, given a few months of drying, recementation occurs to a degree which renders the

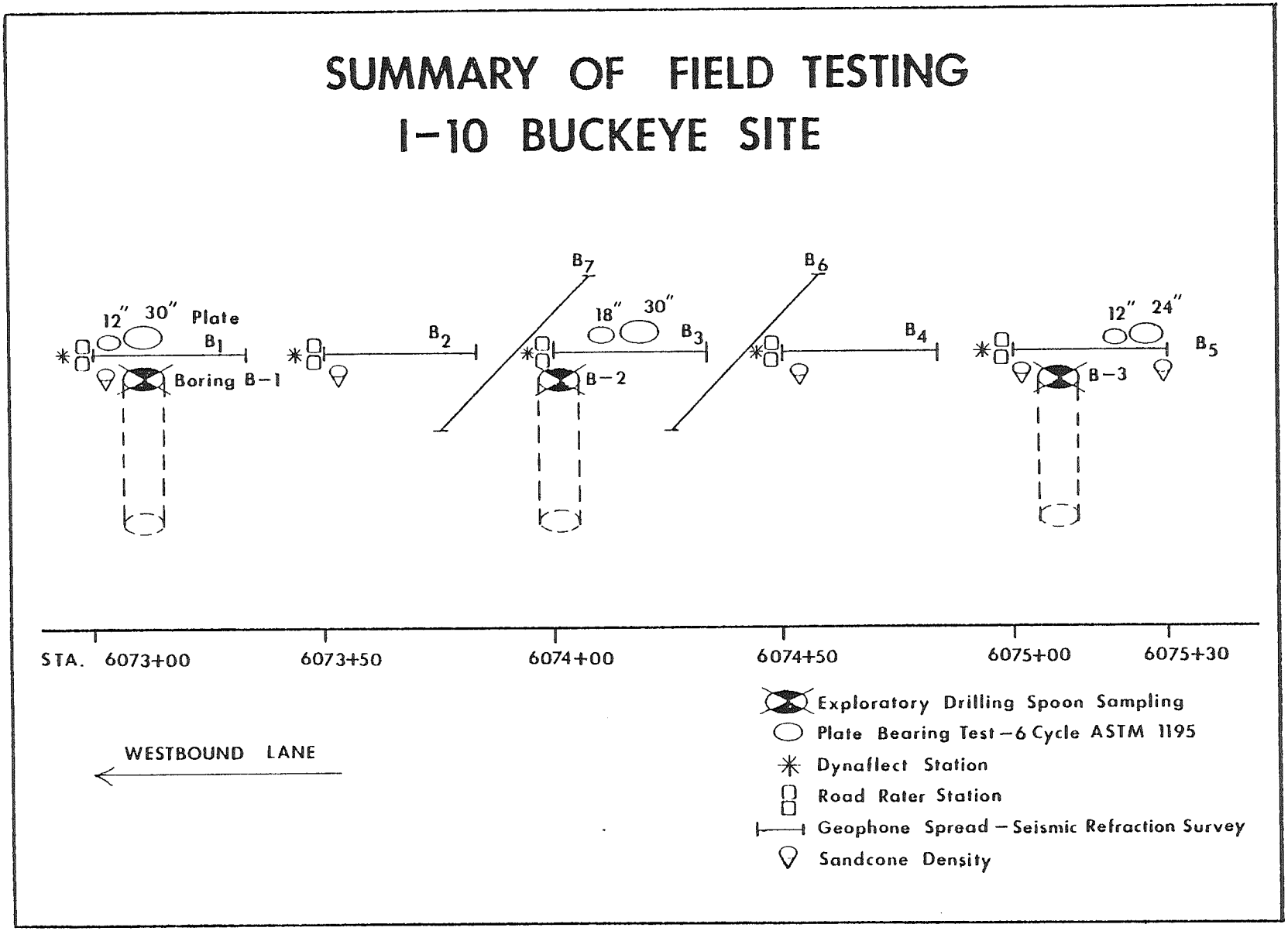


Figure 33. Visual Summary of Field Testing - I-10 Buckeye Site

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JOB NO. E76-3 DATE 7-23-76

LOG OF TEST BORING NO. B-1

Location I-10 Buckeye Site-Sta. 6073+10

RIG TYPE CME-55

BORING TYPE Split Spoon Sampler

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				S	8-8			CL		Cut Section Compacted Subgrade SANDY CLAY, weakly lime cemented, low plasticity, light brown
				S	12-16			CL		
				S	12-24			SC		Native SANDY CLAY, some gravel, moderately cemented, low plasticity, light brown
				S	10-17					
5				S	16-15					CLAYEY SAND, some gravel, moderately lime cemented, low plasticity, light brown Stopped sampler at 5'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 34a. Log of Subsurface Boring - I-10 Buckeye Site

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JOB NO. E76-3 DATE 7-23-76

LOG OF TEST BORING NO. B-2

Location I-10 Buckeye Site-Sta. 6074+00

RIG TYPE CME-55

BORING TYPE Spilt Spoon Sampler

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								CL		Cut Section Compacted Subgrade SANDY CLAY, weakly lime cemented, low plasticity, light brown
				S	8-14		0.5			
				S	10-13		2.7	SC		
				S	8-12		2.5			
				S	15-16		5.2			Native CLAYEY SAND, some gravel, moderately lime cemented, low plasticity, light brown
				S	30-26		5.8	SC		CLAYEY SAND, some gravel, strongly lime cemented, low plasticity, light brown
5										Stopped sampler at 5'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 34b. Log of Subsurfa Boring - I-10 Buckeye Site

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for Arizona Pavements

PROJECT

JOB NO. E76-3 DATE 7-23-76

LOG OF TEST BORING NO. B-3

Location I-10 Buckeye Site-Sta. 6075+00

RIG TYPE CME-55

BORING TYPE Split Spoon Sampler

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				S	13-17					Fill Section over wash
				S	22-16					Compacted FILL
				S	10-16			CL		SANDY CLAY, weakly lime cemented, low plasticity, light brown
				S	13-10					
				S	10-4			SM		Native Alluvium
5										SILTY SAND & GRAVEL, well graded, subangular, nonplastic, gray
										Stopped sampler at 5'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 34c. Log of Subsurfac Boring - I-10 Buckeye Site

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soil as stiff or stiffer than in its original condition.

Results of the surface in-place densities are given in Table 3. It should be noted these were taken on a day following a rainfall; hence the discrepancy in degree of saturation compared to the time of spoon sampling and performance of plate bearing tests.

Table 2
Summary of Soil Classification Testing
I-10 Buckeye Site

Boring No.	Depth	Unified Classification	LL	PI	Percent Passing 200	40	16	10	4
B-1	0-1'	CL	31	10	55	77	91	95	99
B-1	1'-2'	CL	31	10	52	70	81	84	88
B-1	2'-3'	SC	30	10	38	54	69	78	87
B-1	3'-4'	SC	32	13	25	39	58	72	85
B-1	4'-5'	SC	35	15	28	42	62	74	87
B-3	4'-5'				19	38	61	71	79

Table 3
Summary of In-Place Densities*
I-10 Buckeye Site

Station	Dry Density (pcf)	w/c Percent	Degree of Saturation Percent
6073+00	103.2	10.0	44
6073+50	129.8	6.5	63
6074+50	125.4	8.1	67
6075+00	125.6	6.7	56
6075+30	125.2	7.7	63

*Performed 10/25/76 - following recent rainfall

3.1.2 Lake Pleasant Site

In Figure 35, a visual summary of testing at the three spots along the Lake Pleasant-Morristown Road is given. Logs from the On-Grade site and Orange site are presented as Figures 36a and 36b. Penetration of the hard fanglomerate by the split spoon and conventional flight auger met refusal; hence no subsurface profile is given for the Fanglomerate Cut site.

Generally, the compacted subgrade for this proposed highway is a clay, sand and gravel mixture. Laboratory testing revealed the following classification information for the compacted top foot of material.

Unified Classification: GC
LL = 30
PI = 11
Percent -200 = 48
Percent -40 = 55
Percent -16 = 59
Percent -10 = 62
Percent -4 = 67

In Table 4, the densities and moisture contents of the upper 6 inches are given.

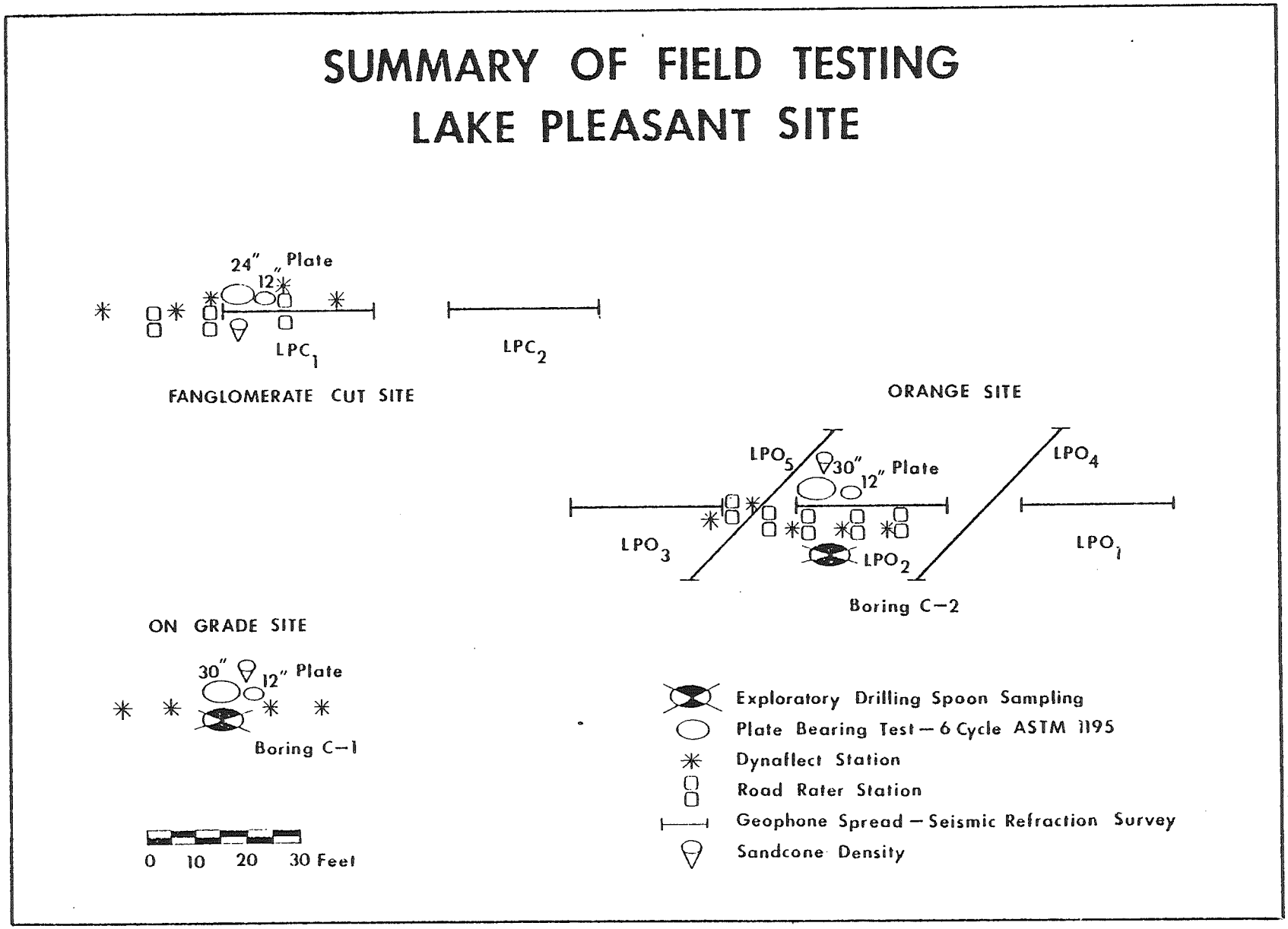


Figure 35. Visual Summary of Field Testing - Lake Pleasant Site

Subgrade Elastic Modulus Study
for Arizona Pavements

PROJECT

JOB NO. E76-3 DATE 7-23-76

LOG OF TEST BORING NO. LP-1

Location Lake Pleasant-Morristown Road-On-Grade Site

RIG TYPE CME-55

BORING TYPE Split Spoon Sampler with 4½" Flight Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			S	S 8-42				GC		Compacted Native Material CLAYEY SAND & GRAVEL, well graded, angular to subangular, low plasticity, brown
				S 50-60						Native FANGLOMERATE
5										Auger refused at 2'2"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 36a. Log of Subsurfac Boring - Lake Pleasant (Or Site

PROJECT Subgrade Elastic Modulus Study for Arizona Pavements

JOB NO. E76-3 DATE 7-23-76

LOG OF TEST BORING NO. LP-2

Location Lake Pleasant-Morristown Road-Orange Site

RIG TYPE CME-55

BORING TYPE Split Spoon Sampler with 4½" Flight Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			S	S	11-26 100/1"		1.7	GC		Compacted Native Material CLAYEY SAND & GRAVEL, well graded, angular to subangular, low plasticity, brown
										FANFLOMERATE
5										Auger refused at 1'1"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 36b. Log of Subsurface Boring - Lake Pleasant (On-grade) Site

Table 4

Summary of In-Place Densities*

Lake Pleasant Sites

Location	Dry Density (pcf)	w/c Percent	Degree of Saturation Percent
On-Grade Site	116.1	9.8	61
Orange Site	111.8	11.1	61
Fanglomerate Cut	119.1	8.3	56

*Performed 10/25/76 - following recent rainfall

3.1.3 Cienega Creek Site

Figure 37 is a visual summary of all testing performed at the Cienega Creek site. Figure 38 is a log of the exploratory boring taken at the site.

The subgrade profile consists of a foot of clayey sand with gravel over a highly plastic clay. Table 5 presents a summary of classification test data from the spoon samples taken. Table 6 is a summary of the in-place moisture and density data.

The split spoon boring was performed in May, 1976 on a day when the compacted subgrade was very damp, as noted by the 14.3 percent water content. The Phase I testing was performed in September and October of that year, following several months of dry weather. In addition, the subgrade had been further compacted as a result of heavy use by loaded logging trucks. The degree of compaction is revealed in the high dry density values (136 pcf) noted at the time of testing.

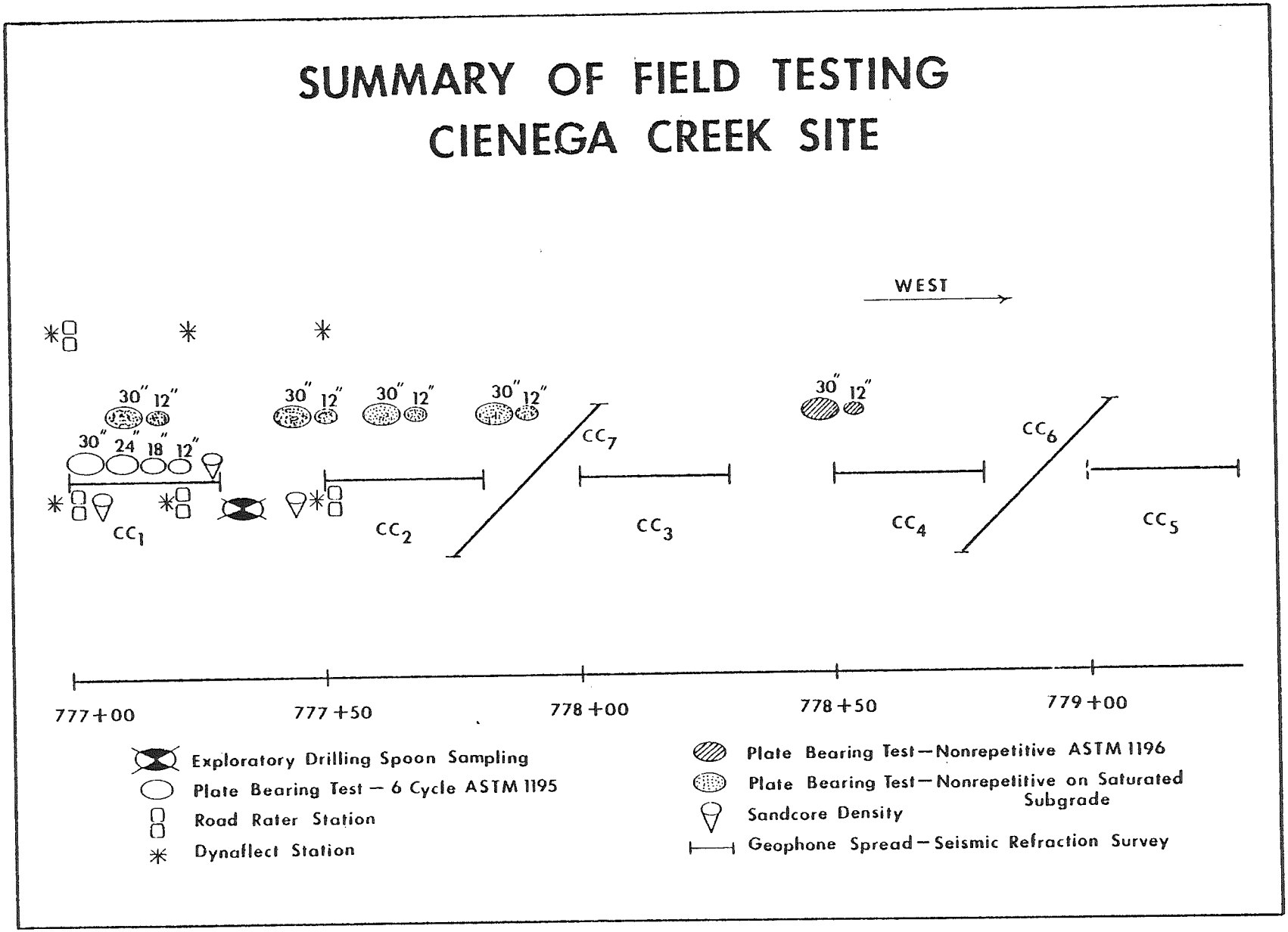


Figure 37. Visual Summary of Field Testing - Cienega Creek Site

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JOB NO. E76-3 DATE 5-5-76

LOG OF TEST BORING NO. CC-1

Location Cienega Creek Site-Sta. 777+00

RIG TYPE CME-75

BORING TYPE Split Spoon Sampler with 4½" Flight Auger

SURFACE ELEV.

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Percent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				S	10-11		14.3	SC		Base Coarse CLAYEY SAND, trace of gravel, well graded, medium plasticity, reddish-gray
				S	9-11		24.1			
				S	14-12		21.6	CH		
				S	19-18		21.1			Subgrade CLAY, small amount of sand & gravel, high plasticity, gray
				S	50-165		14.9			GRANITE, moderately to slightly weathered, gray
5										Stopped auger at 4'6" Stopped sampler at 5'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Auger cuttings. B - Block sample
S - 2" O.D. 1.38" I.D. tube sample.
U - 3" O.D. 2.42" I.D. tube sample.
T - 3" O.D. thin-walled Shelby tube.

Figure 38. Log of Subsurface Boring - Cienega Creek Site

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Table 5

Summary of Soil Classification Testing

Cienega Creek Site

Boring No.	Depth	Unified Classification	LL	PI	Percent Passing 200	40	16	10	4
CC-1	0-1'	SM-SC	26	6	33	56	76	85	93
CC-1	1'-2'	CH	63	41	70	80	86	89	94
CC-1	2'-3'	CH	70	50	92	95	97	98	100
CC-1	3'-4'	CH	64	46	84	90	93	96	100
CC-1	4'-5'	CL	46	22	58	72	83	87	91

Table 6

Summary of In-Place Densities*

Cienega Creek Site

Station	Dry Density (pcf)	w/c Percent	Degree of Saturation Percent
777+00	135.1	7.0	83
777+25	136.8	5.1	65
777+50	136.2	5.8	72

*Performed 10/26/76

3.1.4 Rainbow Valley Site

In an effort to delineate the lateral variations in subgrade properties at the Rainbow Valley site, 19 penetration borings 2 feet deep were performed within the 75 by 100 foot grid station. The results are shown in graphical perspective on Figure 39. Generally, the soil can be described as a silty sand with a small amount of clay and scattered pockets of gravel. In the top foot to 18 inches, the sand is uncemented

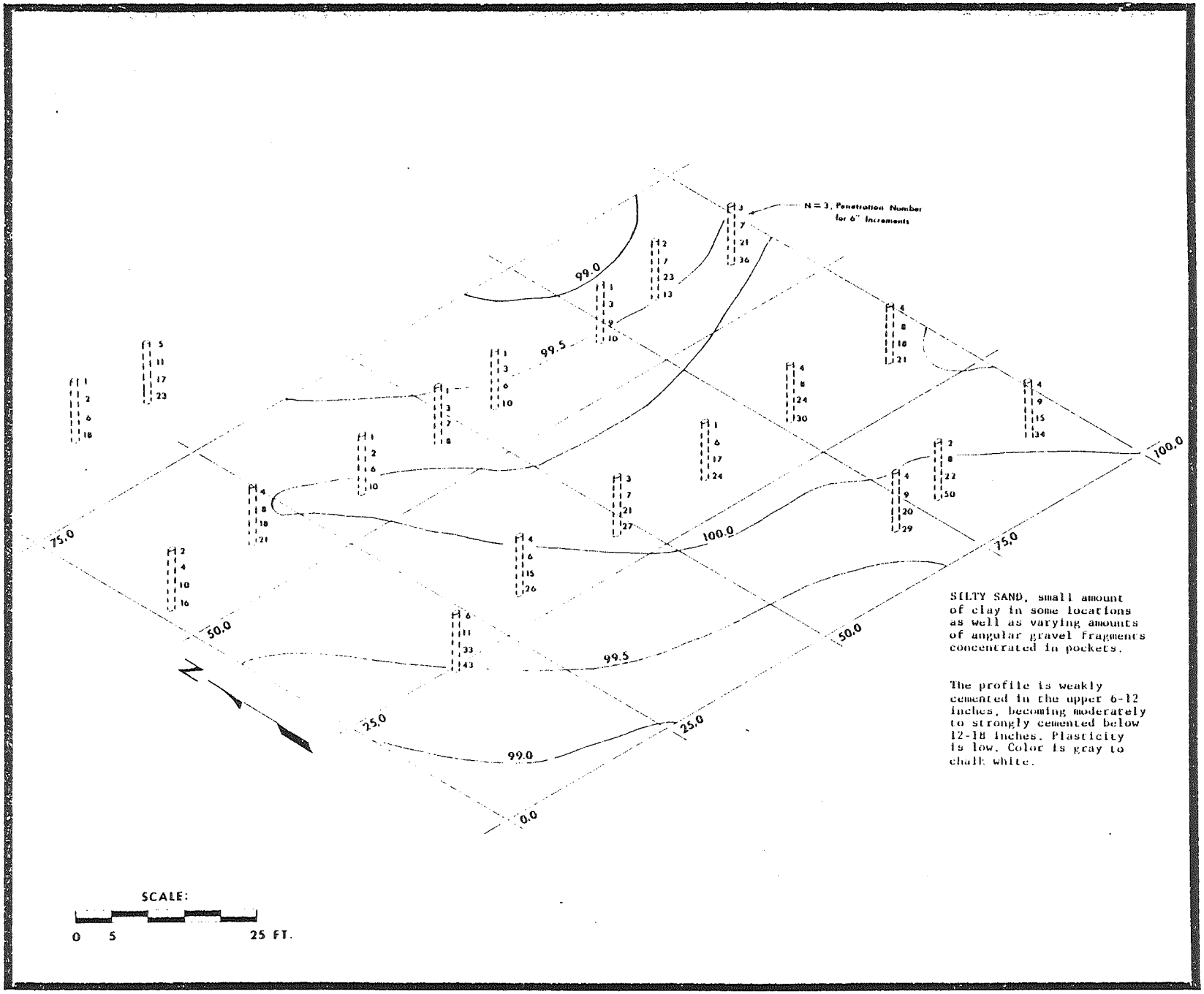


Figure 39. Visual Summary of Penetration Testing - Rainbow Valley Site

or very weakly cemented, but below this, there is moderate to strong lime cementation over most of the site.

Table 7 is a listing of the classification data from selected samples. In Figure 40, in-place densities are shown graphically.

It is instructive to view Figure 39 from the standpoint of variation in subgrade hardness (i.e., the cementation) as reflected by corresponding penetration numbers at the various locations. Such lateral variations in degree of cementation must be considered typical of soil profile development in this part of the country.

The 6 inch penetration numbers seem to suggest that cementation is greater in the southeast quadrant of the test grid, although a close look in the field revealed that there are exceptions. After the surface testing, a trench was dug so that tests could be set on the more strongly cemented strata. Watching the blade during trenching revealed considerable differences in stiffness over a few feet. Also, natural jointing and fissuring within the caliche could be seen on the fresh surfaces.

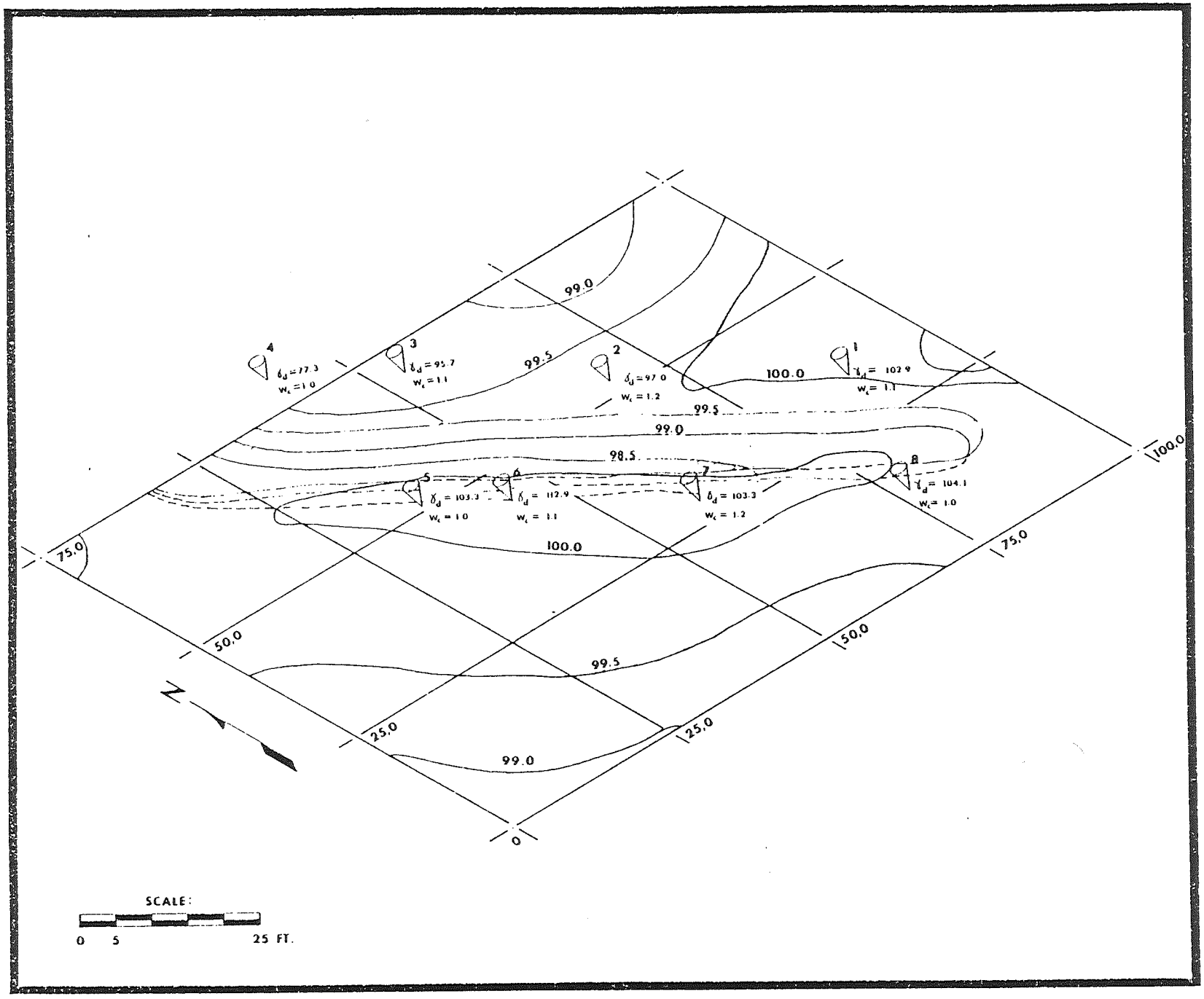


Figure 40. Visual Summary of In-Place Densities - Rainbow Valley Site

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Table 7
Summary of Soil Classification Testing
Rainbow Valley Site

Location	Depth	Unified Classification	LL	PI	Percent Passing 200	40	16	10	4
101.3E-65.9N	0-1'	SM	34	9	45	65	78	84	91
90.1E-66.8N	1'-2'	SM-SC	27	5	31	48	65	74	85
25.0E-65.9N	1'-2'	SM	34	10	49	70	85	91	97
55.5E-38.2N	1'-2'	ML	42	14	54	73	88	92	98

3.1.5 McCormick Ranch Site

Graphical presentations of the results of penetration testing at the McCormick Ranch desert and compacted fill sites are given in Figures 41 and 42, respectively. In Table 8, which contains the classification test data from soil samples, it can be seen that the physical properties of soil at the fill and undisturbed site are virtually identical. However, the penetration numbers reveal the difference in firmness.

The soil at McCormick Ranch is a lean silty clay and appears to be only weakly cemented in the top 2 feet of undisturbed soil. The penetration tests show that the fill is very uniform in stiffness throughout.

Figure 43 shows the results of a consolidation test of an undisturbed sample from the desert site. The large loss in volume upon saturation indicates that this is a somewhat moisture sensitive soil.

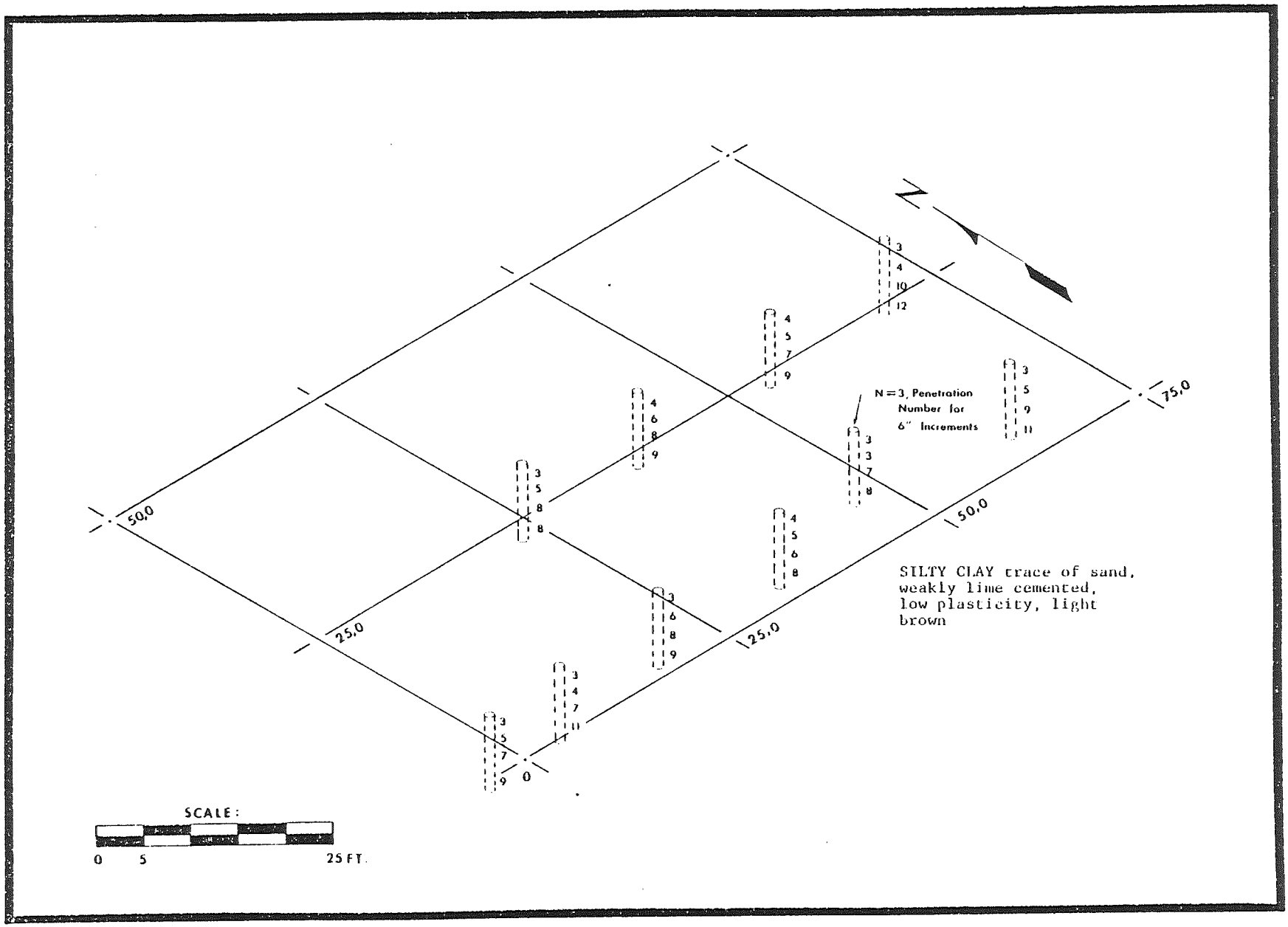


Figure 41, Visual Summary of Penetration Testing - McCormick Ranch Desert Site

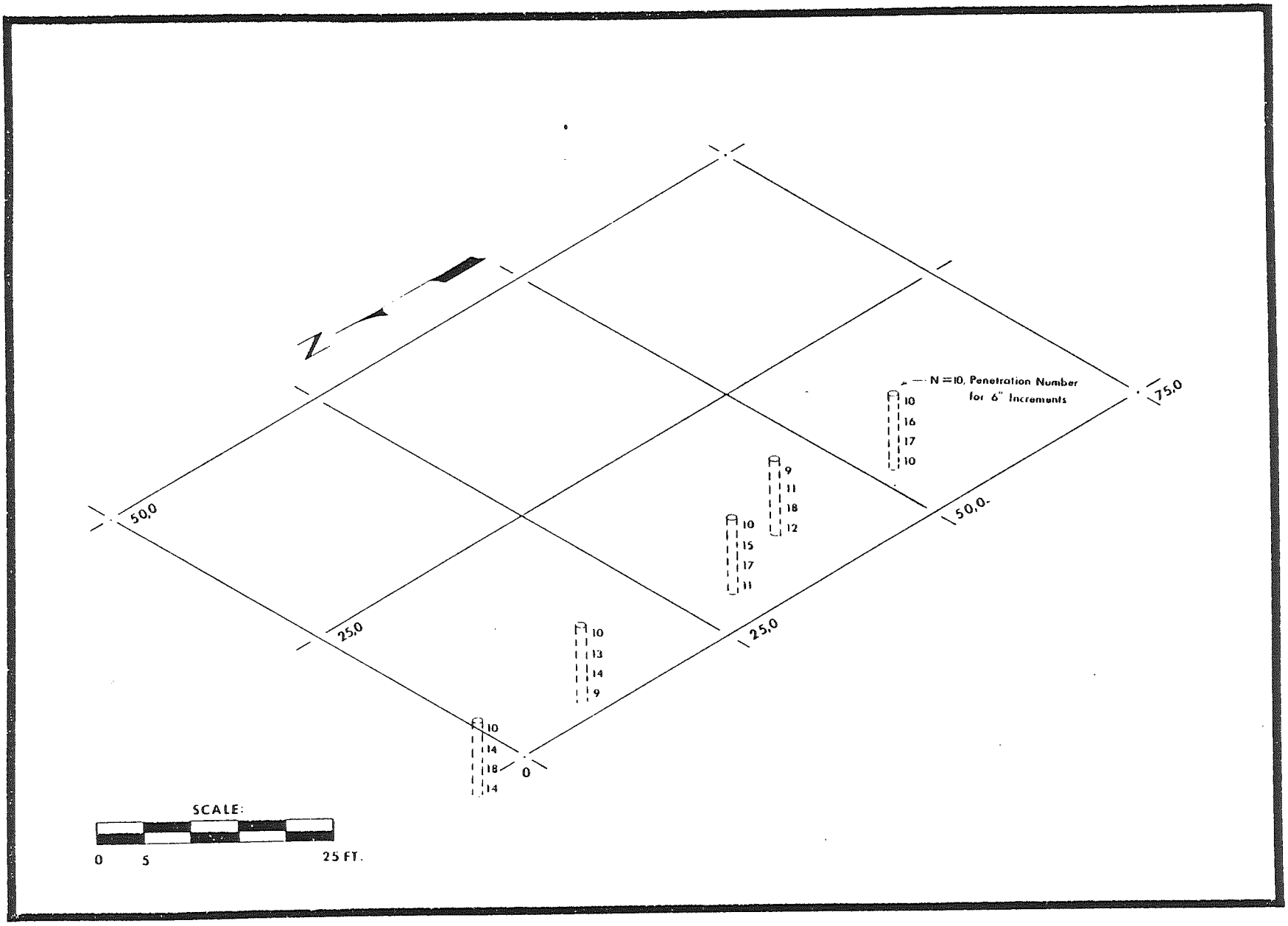


Figure 42. Visual Summary of Penetration Testing - McCormick Ranch Fill Site

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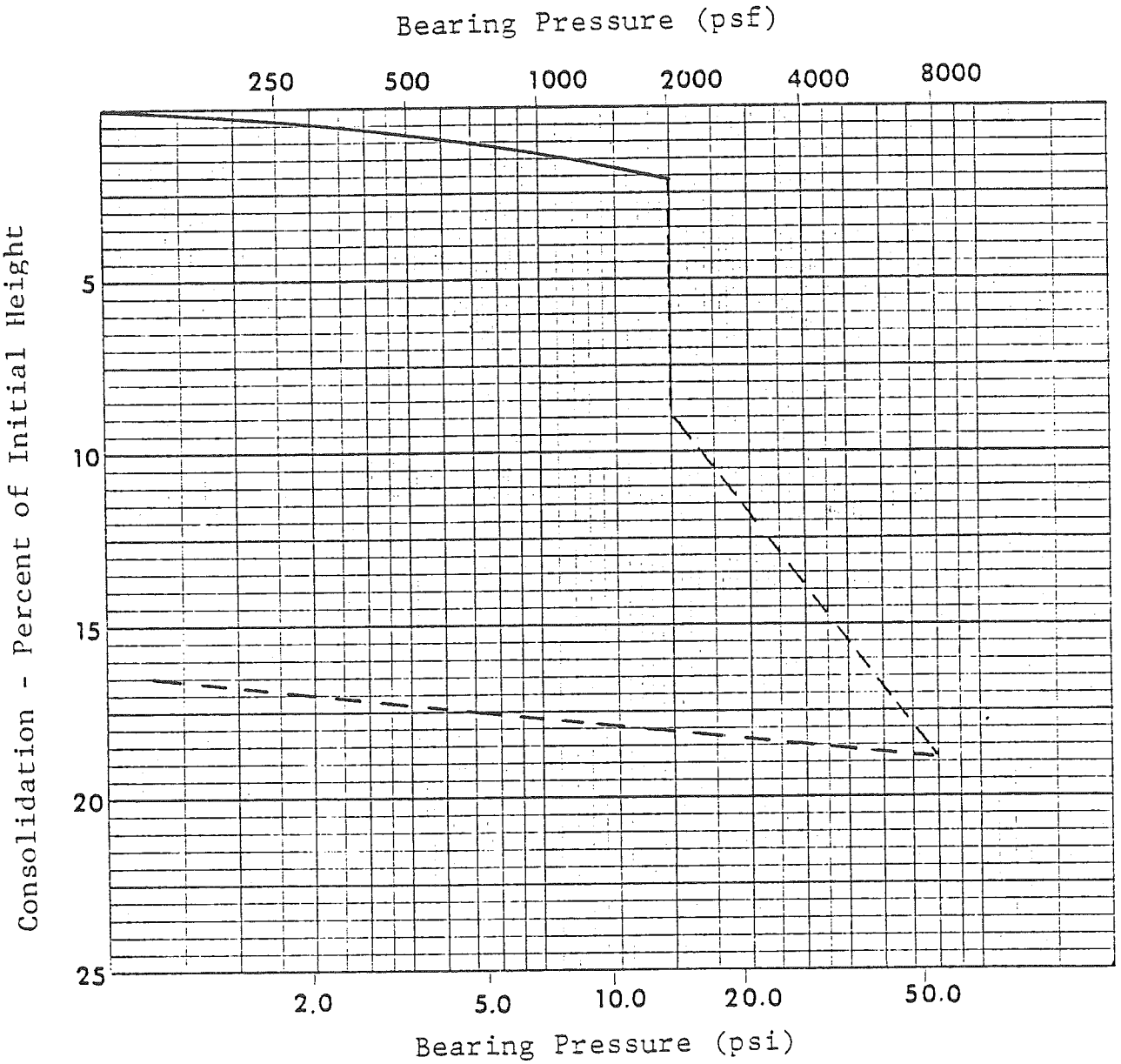


Figure 43. Results of Consolidation Test on Undisturbed Surface Soil - McCormick Ranch Desert Site

In Figure 44, in-place density and moisture determinations in the fill are displayed. As previously stated, the fill was compacted to 95 percent of Standard Proctor (ASTM D696).

Table 8
Summary of Soil Classification Testing
McCormick Ranch Sites

Location	Depth	Unified Classification	LL	PI	Percent Passing 200	40	16	10	4
Desert -1.9E-7.0N	1'-2'	ML-CL	23	7	63	88	94	96	99
Desert 31E-32.2N	0-1'	ML-CL	22	7	58	87	95	98	100
Desert 25.2E-9.0N	1'-2'	CL	24	8	56	87	95	96	99
Fill 60.0N-15.0E	1'-2'	ML-CL	24	6	80	98	99	100	
Fill 17.4N-10.2E	1'-2'	CL	27	9	83	98	99	100	

3.2 Results of Plate Bearing Tests

3.2.1 General

At the three Phase I test sites, repetitive plate bearing tests (ASTM D1195) were performed using various sizes of plates (12", 18", 24", 30"). All were performed on the surface of compacted subgrades under in-situ moisture conditions. At the Phase II sites, both repetitive and nonrepetitive (ASTM D1196) tests were performed on dry and artificially saturated subgrades. Complete results of all plate bearing tests

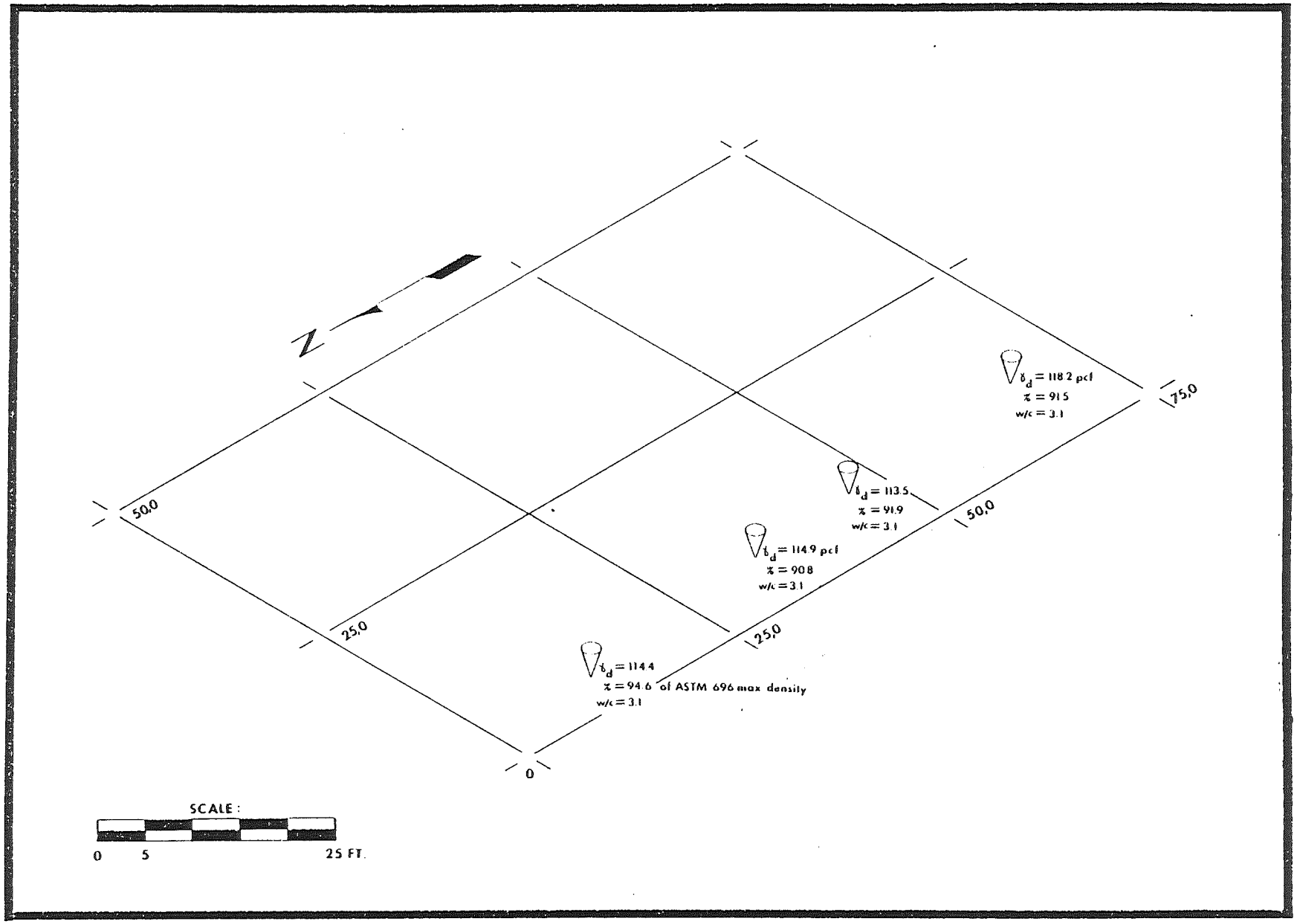


Figure 44. Visual Summary of In-Place Densities - McCormick Ranch Fill Site

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(in the form of stress-deflection plots) are available in the Appendix, Figure A-3.2-1 through Figure A-3.2-134. A general discussion of results at each site follows.

3.2.2 I-10 Buckeye Site

A graphical presentation of the six repetitive plate bearing tests performed at this site is given in Figure 45. Plots of E modulus versus stress level based on the sixth cycle of loading are included. A comparison of values from the six tests at the 30 to 50 psi range is given below. This stress range is considered the most appropriate for highway subgrades.

Table 9
Results of Plate Bearing Tests
I-10 Buckeye Site

Location	Plate Size	E @ 30-50 psi Sixth Cycle	E @ 30-50 psi First Cycle
Station			
6073+02	12" Plate	29,000 psi	34,800 psi
6073+12	30" Plate	25,000-34,000	29,000-38,000
6074+10	18" Plate	39,000	40,000
6074+12	30" Plate	17,500	20,600
6075+10	12" Plate	50,000	60,000
6075+15	24" Plate	29,000	36,000

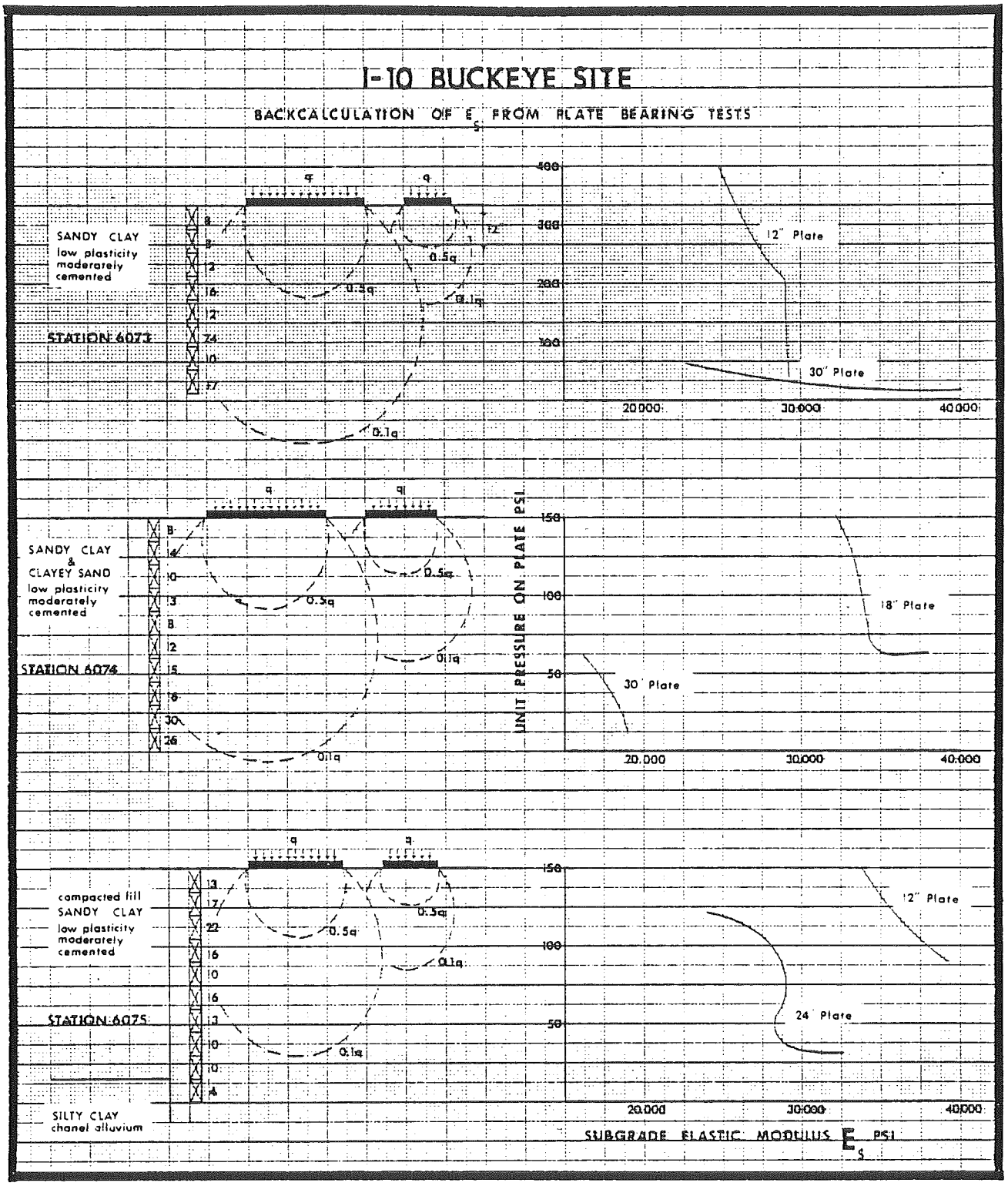


Figure 45. Graphical Summary of Plate Bearing Tests - I-10 Buckeye Site

A few observations regarding these tests can be made.

- The compacted subgrade is very stiff in its dry condition. With the exception of the 12 inch plate tests, it was impossible to achieve even 0.1" deflection. There is no way to compare the load-deflection relation to plate size as McLeod (34) did, inasmuch as the subgrade at the I-10 Buckeye site is an order of magnitude stiffer than the Canadian sites.

- While the six plate bearing tests show this subgrade to be very stiff, there is considerable variation in the response from test to test. This is revealed in the fact that the modulus-stress level curves are not all of the same general shape. In all but the first 12 inch test, modulus decreased with stress level (strain softening), though the rate of decrease varied widely.

- In general, the larger plates yielded smaller values of modulus at a given stress level. That is the opposite of what McLeod found (see Figure 25).

- There was a reduction of 10 to 20 percent in modulus values calculated from the sixth cycle compared to the first cycle. One exception was noted where there was virtually no difference. The repetitive testing did not eliminate spatial variations in response at this site.

- There appears to be no noticeable difference in the shape of the curves or values of modulus taken from tests over the 4 feet of fill (station 6075+00), as compared to those taken on the cut. This would seem to indicate that fills built of desert soils, which are high in lime, shortly after recompaction achieve a degree of recementation and resultant stiffness which compares favorably with that of undisturbed soil.

It must be concluded that subgrade modulus (E) is not clearly defined by a single plate bearing test. In order to enable the application of engineering judgment in the selection of a design value, several tests are needed. Discussion of this and other limitations of the plate bearing tests are given in section 4.1.

3.2.3 Lake Pleasant Site

A graphical presentation of the six repetitive plate bearing tests, performed in pairs, at three separate locations along the Lake Pleasant-Morristown Road is given in Figure 46.

Values of E modulus for the compacted subgrade material at the On-Grade and Orange locations were back calculated using the Ueshita and Meyerhof (136) analysis for a circular load on an elastic material underlain by a rough rigid base. At the Fanglomerate

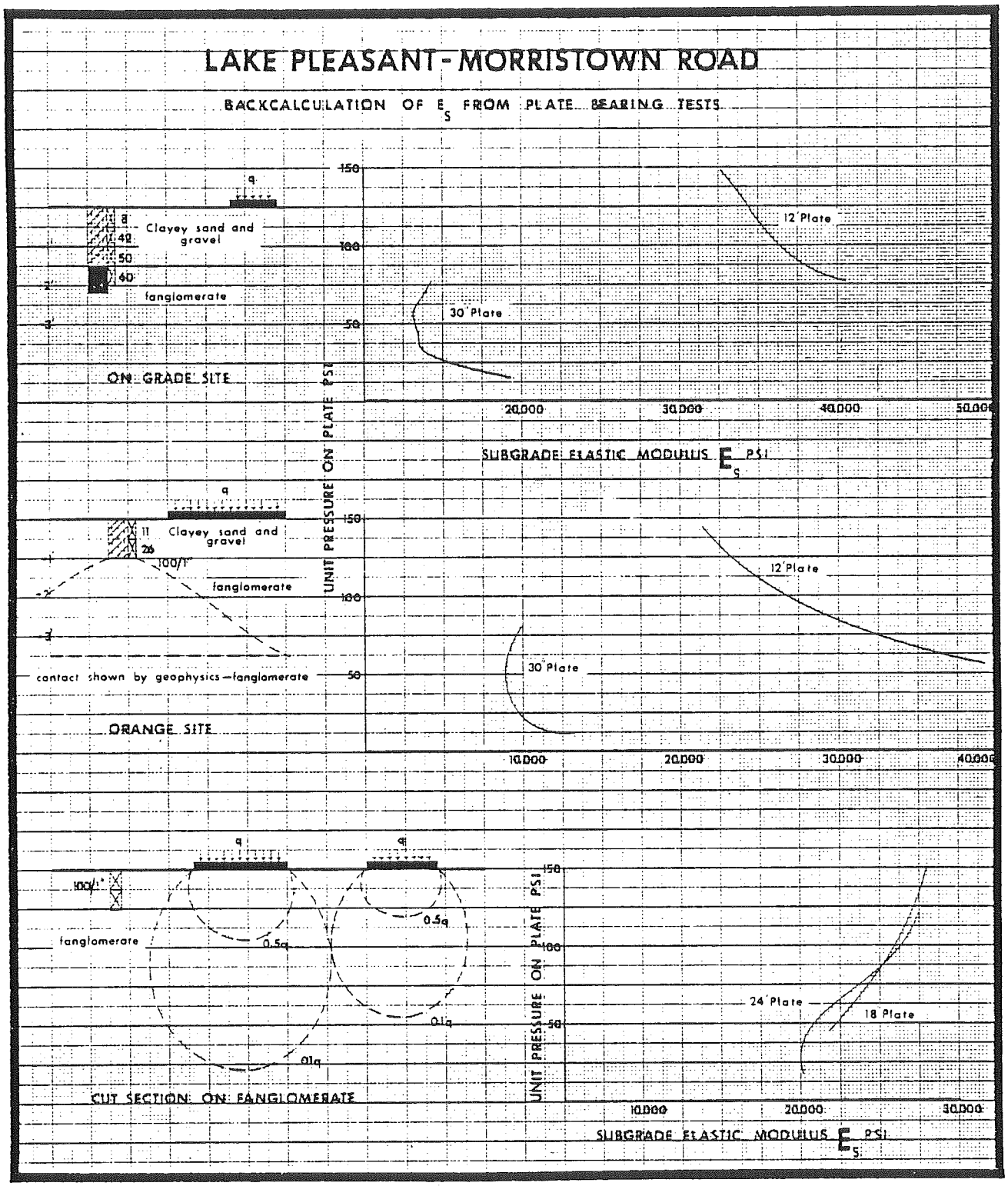


Figure 46. Graphical Summary of Plate Bearing Tests - Lake Pleasant Sites

Cut location, a homogeneous elastic subgrade was assumed. A comparison of values from these tests at the 30 to 50 psi stress level is given below.

Table 10
Results of Plate Bearing Tests
Lake Pleasant Sites

Location	Plate Size	E @ 30-50 psi Sixth Cycle	E @ 30-50 psi First Cycle
On-Grade Site	12" Plate	57,000 psi	58,000 psi
On-Grade Site	30" Plate	13,000-15,000	16,000-18,000
Orange Site	12" Plate	54,000	60,000
Orange Site	30" Plate	9,000-10,000	11,000-12,000
Fanglomerate Cut	18" Plate	22,000	26,000
Fanglomerate Cut	24" Plate	21,000	23,000

The most startling observation to come from these plate bearing tests is the wide variance in response of the 12 and 30 inch plates; very high moduli were recorded for the 12 inch plates at both the Orange and On-Grade locations. Since the 30 inch plates are influencing a stratum known to be lithified (1 to 3 feet below the surface) to a much greater degree than the 12 inch plates, one would expect the opposite to be true. This same relationship regarding plate size was observed, however, at the I-10 Buckeye site, as will be discussed in detail in Section IV, the fact that the subgrades consist of embedded gravel and small cobbles in a cemented matrix seems to be a major factor in causing this anomaly.

The two plate bearing tests located on the Fanglomerate Cut are unique among all the plate bearing

tests in the entire project. At this spot where the subgrade is, in fact, a cut surface on soft rock, the load deflection relationship exhibits strain hardening (an increase in stiffness as the stress increases).

A look at the seismic refraction data found in section 3.3.2, may offer an explanation. At the Fanglomerate Cut section, a Poisson's ratio of 0.18 was revealed in the near surface material, indicating greater vertical compressibility than exists for most subgrades of compacted soil. The seismic E modulus is about 68,000 psi which is almost three times the value determined from the plate bearing tests. It appears likely that the compressibility of the near surface portion of this fanglomerate is influenced by stress relief phenomena.

The excavation of this cut allowed the rock to expand vertically as stress field readjustment took place. Upon reloading to the preexcavation confining pressure during the load test, Microscopic expansion cracks were again closed, creating a tighter (and stiffer) structure. Compressibility continued to increase (hence modulus increased) up to a stress level of about 200 psi. Here strain softening began, probably as a result of a new set of expansion cracks and fissures directly resulting from the plate load.

3.2.4 Cienega Creek Site

In Figure 47, a graphical presentation is given of

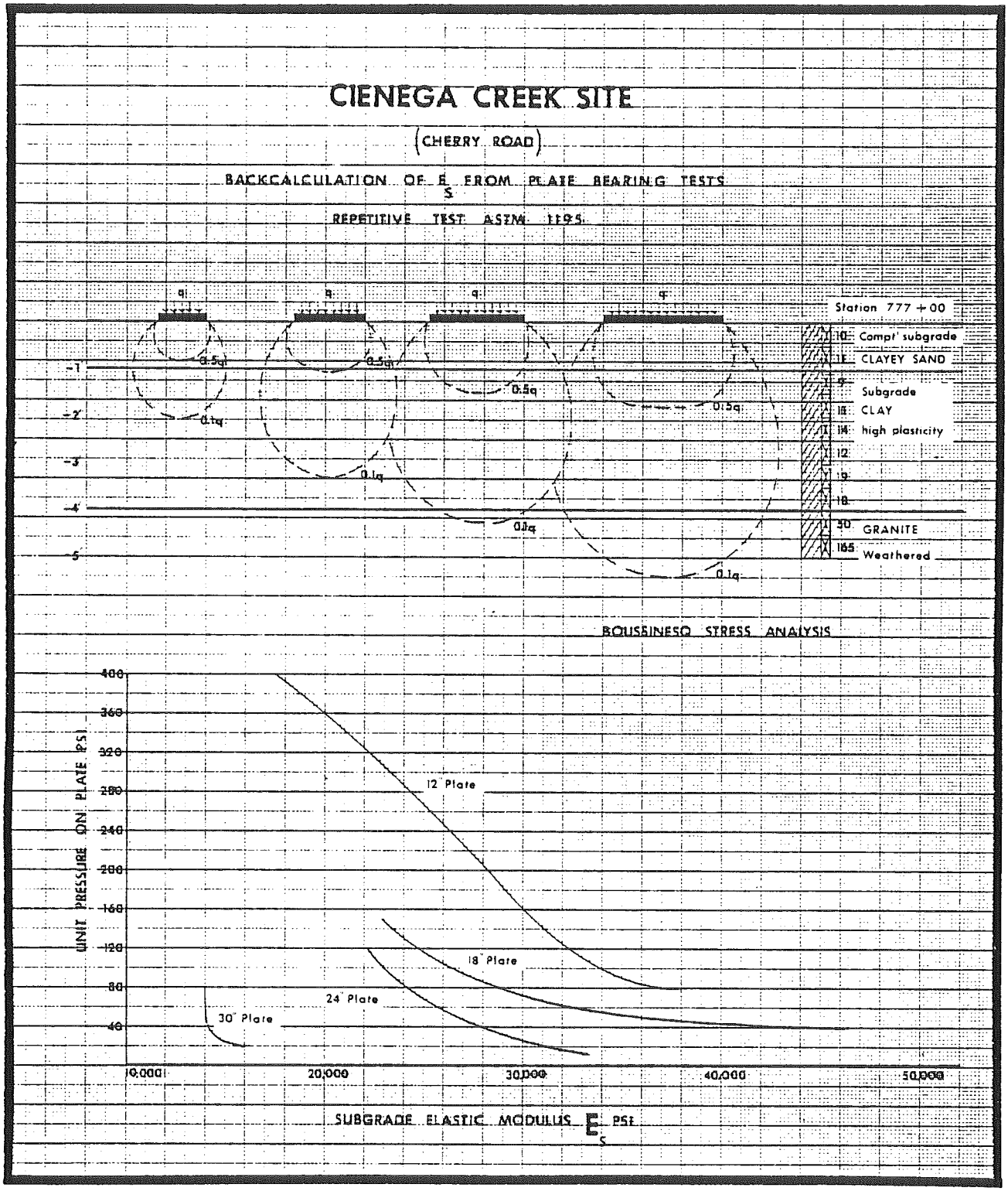


Figure 47. Graphical Summary of Repetitive Bearing Tests - Cienega Creek Site

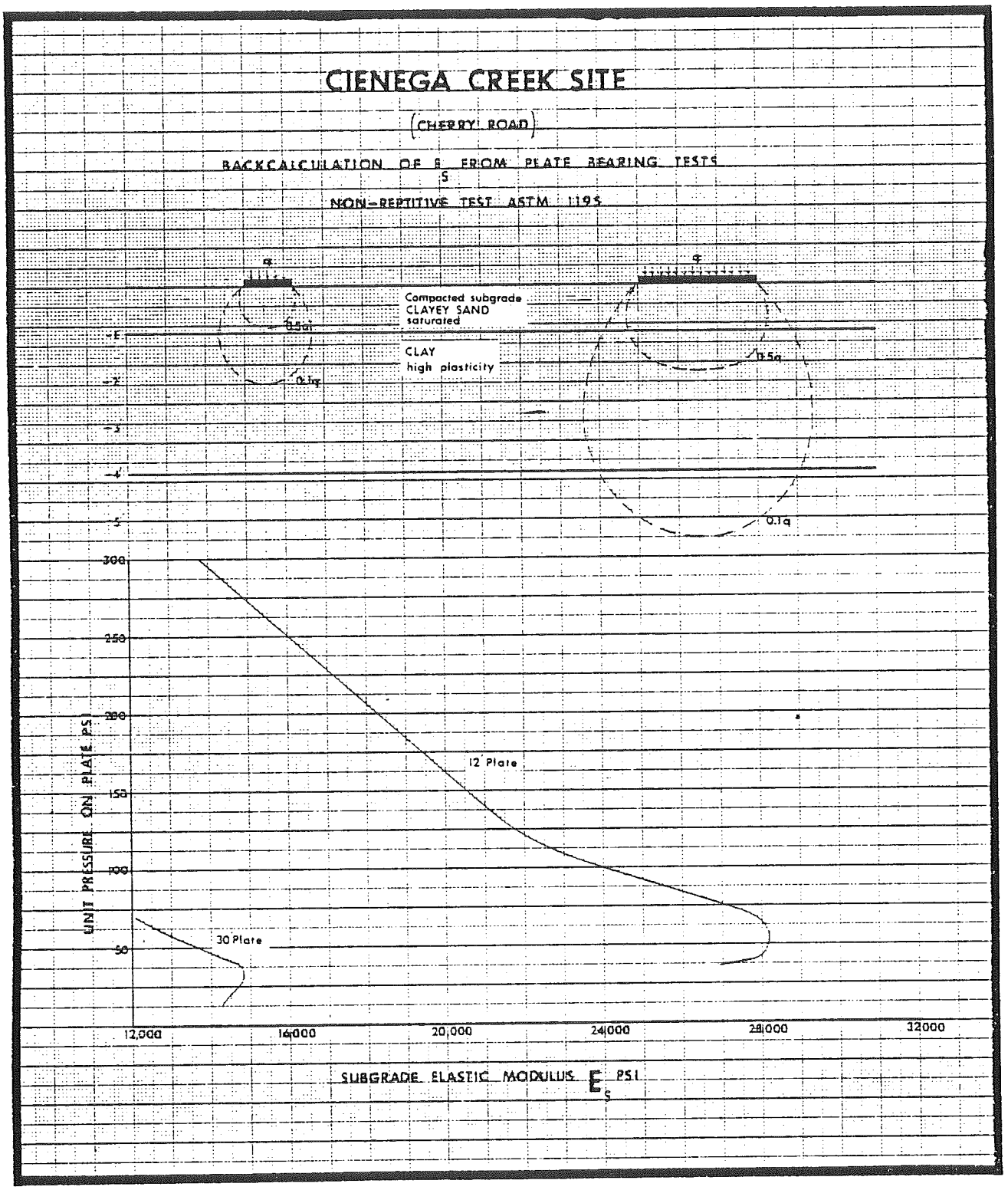


Figure 48. Graphical Summary of Non-repetitive Plate Bearing Tests - Cienega Creek Site

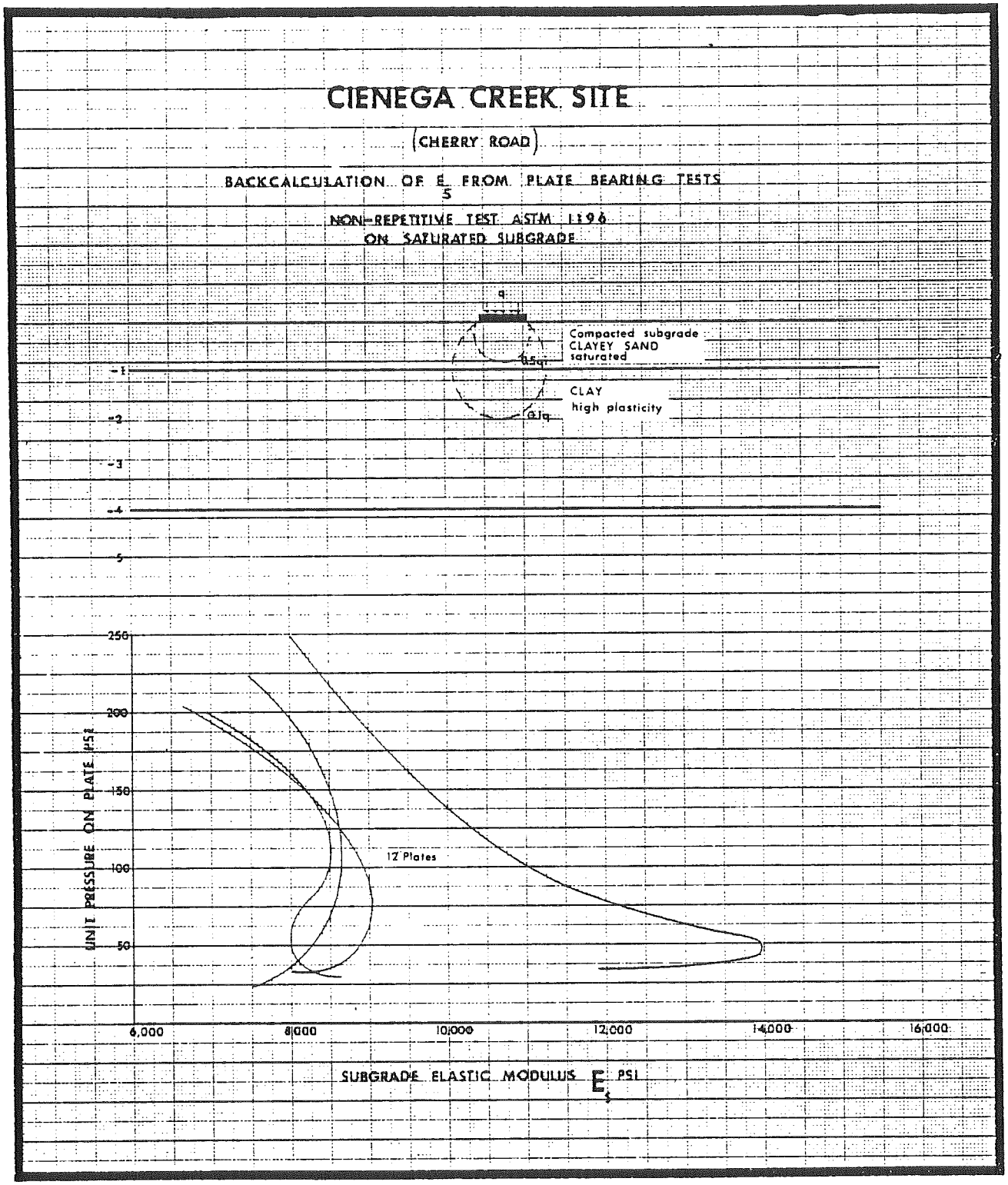


Figure 49. Graphical Summary of 12" Diameter Plate Bearing Tests on Saturated Subgrade - Cienega Creek Site

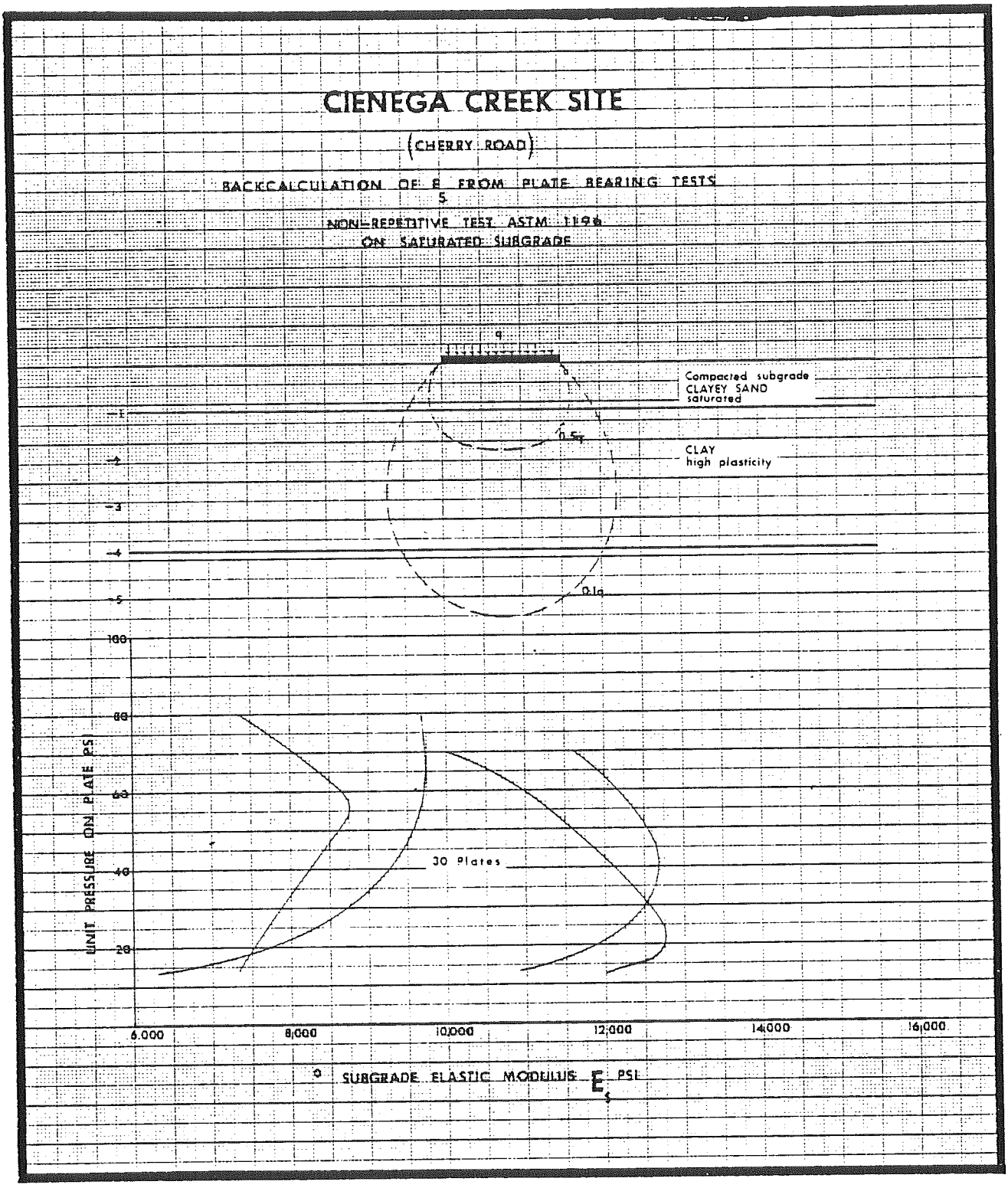


Figure 50. Graphical Summary of 30" Diameter Plate Bearing Tests on Saturated Subgrade - Cienega Creek Site

the four repetitive plate bearing tests performed during Phase I at the Cienega Creek test site. Figure 48 presents the results of two nonrepetitive tests performed during Phase II at the same location, and Figures 49 and 50 present the results of four 12 inch and four 30 inch plate bearing tests performed on this subgrade under conditions of artificial moisture increase.

Figure 47 reveals that with the subgrade in a dry condition, lower moduli are measured with the larger plate, just as was the case for the I-10 Buckeye and the Lake Pleasant tests. As previously stated, this is the opposite of the situation found by most researchers. Nevertheless, a plot of unit load versus perimeter/area ratio at 0.1 inch deflection comes close to the straight line relationship found by McLeod (34), albeit the slope is much steeper (see Figure 51).

The eight plate bearing tests performed on the saturated subgrade at this test station seem to show the opposite trend, regarding modulus and plate size. With one exception, the 12 inch plate produced greater deformation at a given stress level than did the 30 inch plate. In comparing the results of tests on dry and saturated subgrades, it is noted that the 12 inch plates registered about a 60 to 70 percent reduction in modulus due to saturation while the 30 inch plates registered only a 10 to 20 percent loss.

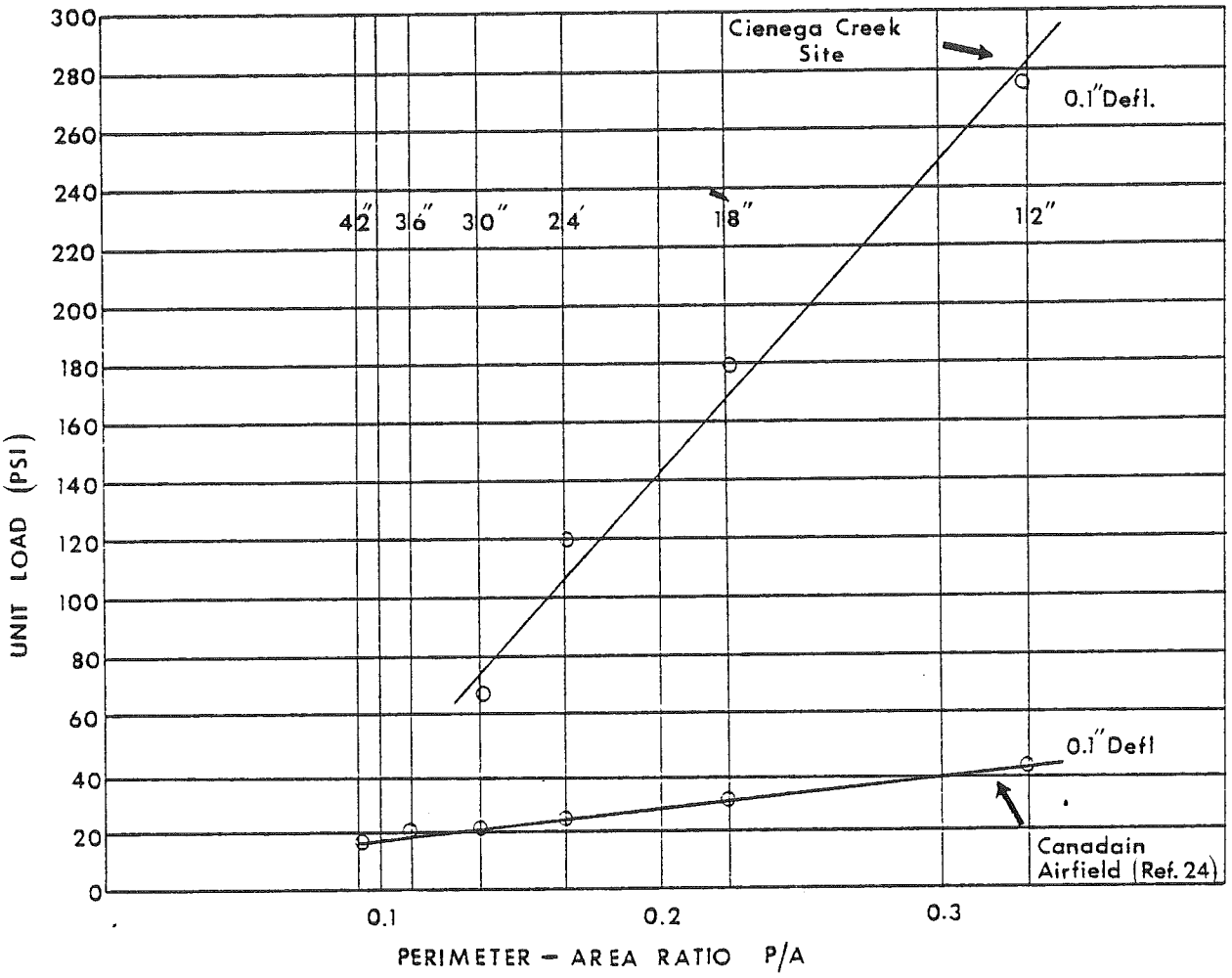


Figure 51. Influence of Plate Size on Unit Load at Cienega Creek Site

Several possible conclusions could be drawn concerning the behavior on evidence from all the tests; the following is concluded:

- The topmost 6 to 12 inches of exposed subgrade at the Cienega Creek site is in a very dense state due to desiccation and compaction resulting from the heavy logging trucks. Below about 1 foot, the soil contains more moisture and is presumably somewhat softer. In its dry state, the upper foot of the subgrade acts as a beam which possesses high flexural stiffness, hence the high modulus values resulting from the small plates.

- Because of the high density and low permeability of the clayey sand subgrade at the test site, our method of overnight soaking caused only slight impairment to the stiffness of the material in-place, as noted by the static test. Probably the top 2 to 4 inches of soil were softened, as reflected in the increased deflections on the 12 inch plates.

- It appears highly probable that a long period of near saturation of the subgrade, in combination with the random pulse loads produced by traffic, would result in more severe deterioration of the subgrade stiffnes than created by saturation alone. Thus, moduli calculated from the tests on saturated subgrade provide but an upper bound relative to a ratio of wet/dry moduli.

3.2.5 Rainbow Valley Site

A total of 80 plate bearing tests were performed at the Rainbow Valley test grid as part of a planned effort to derive a considerable body of test data from a small typical site. The testing program can be broken down as follows:

- Near Surface Tests - 1"-4" below existing grade (SPT = 2-6)
  - 4 repetitive tests - (1 each 12", 18", 24" and 30" plates)
  - 44 nonrepetitive tests - (1 each 18" and 24"; 20 30"; and 22 12" plates)
- Below Surface Tests - 12"-18" below grade - dry surface (SPT = 15-30)
  - 3 repetitive tests - (1 each 12", 18" and 30" plates)
  - 11 nonrepetitive tests - (5 30"; and 6 12" plates)
- Below Surface Tests - 12"-18" below grade - saturated subgrade
  - 18 nonrepetitive tests - (1 each 18" and 24"; 8 30"; and 8 12" plates)

Figures 52, 53 and 54 are graphical presentations of the results of the 48 near surface, the 14 subsurface and the 18 saturated tests, respectively. Complete load-deflection and modulus-stress curves are found in Figures A3.2-27 to A3.2-106 in the Appendix. In addition, a tabular listing of the test results of the 30 to 50 psi modulus values is provided in Table 11.

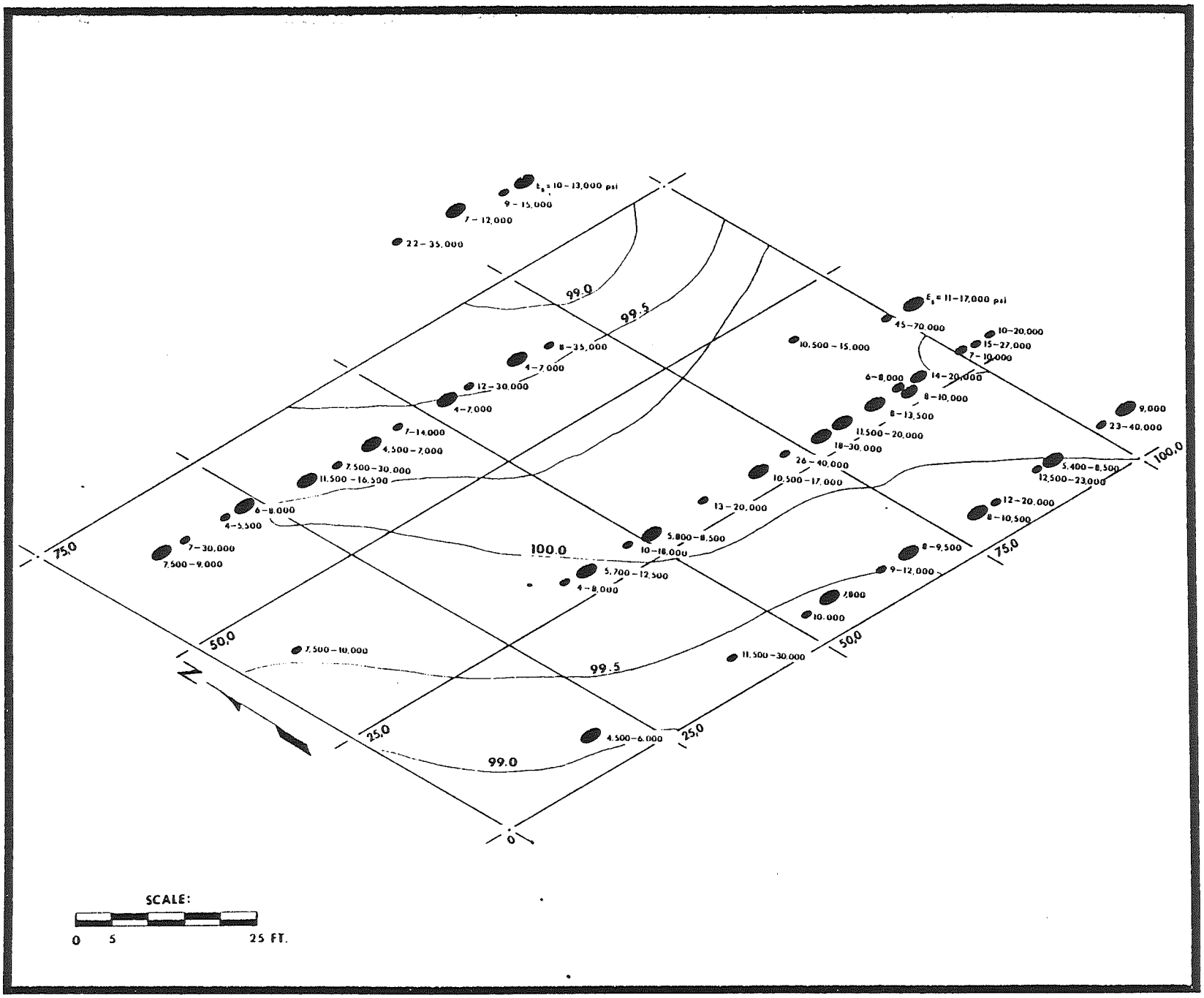


Figure 52. Visual Summary of Near Surface Plate Bearing Tests - Rainbow Valley Site

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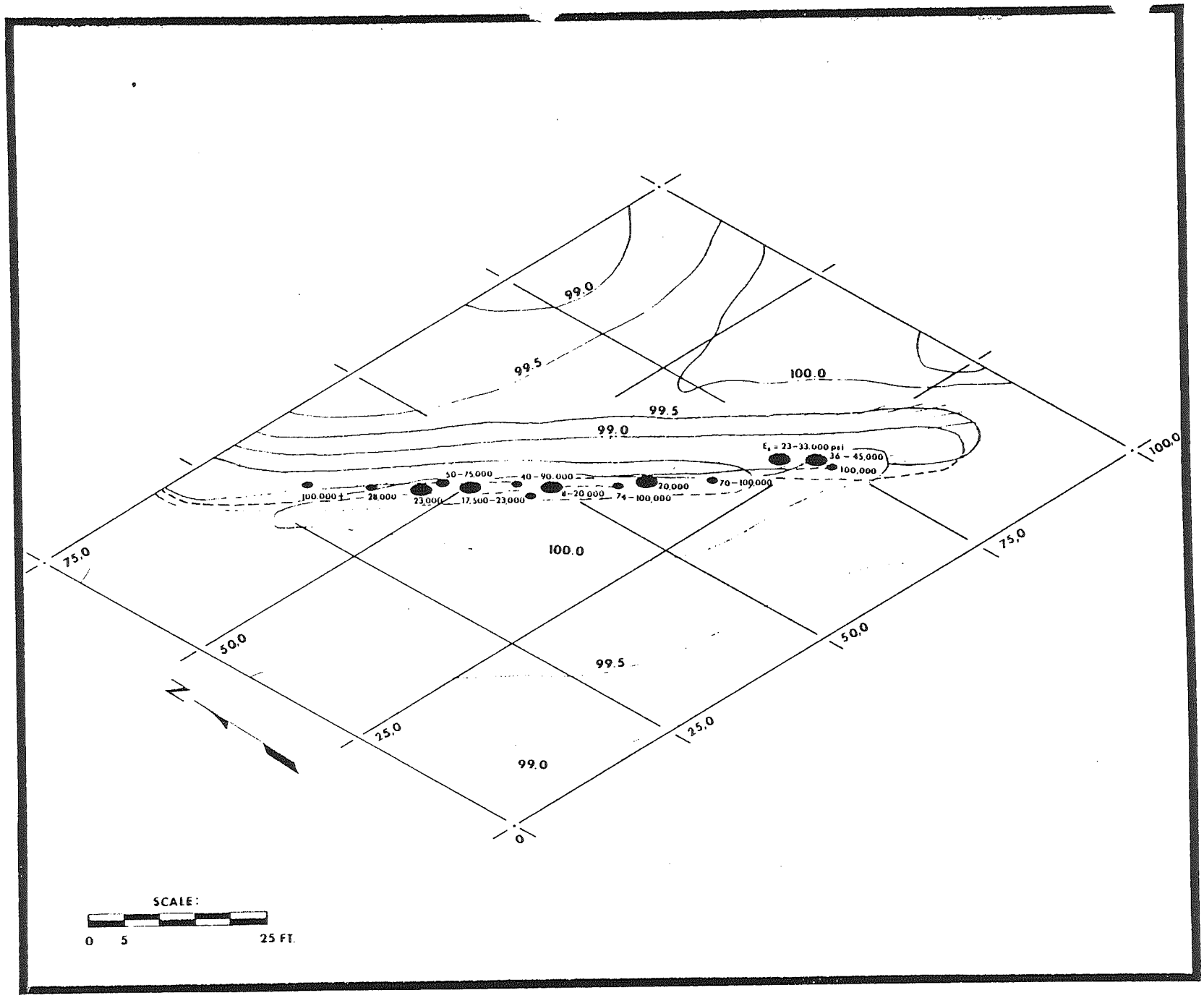


Figure 53. Visual Summary of Subsurface Plate Bearing Tests - Rainbow Valley Site

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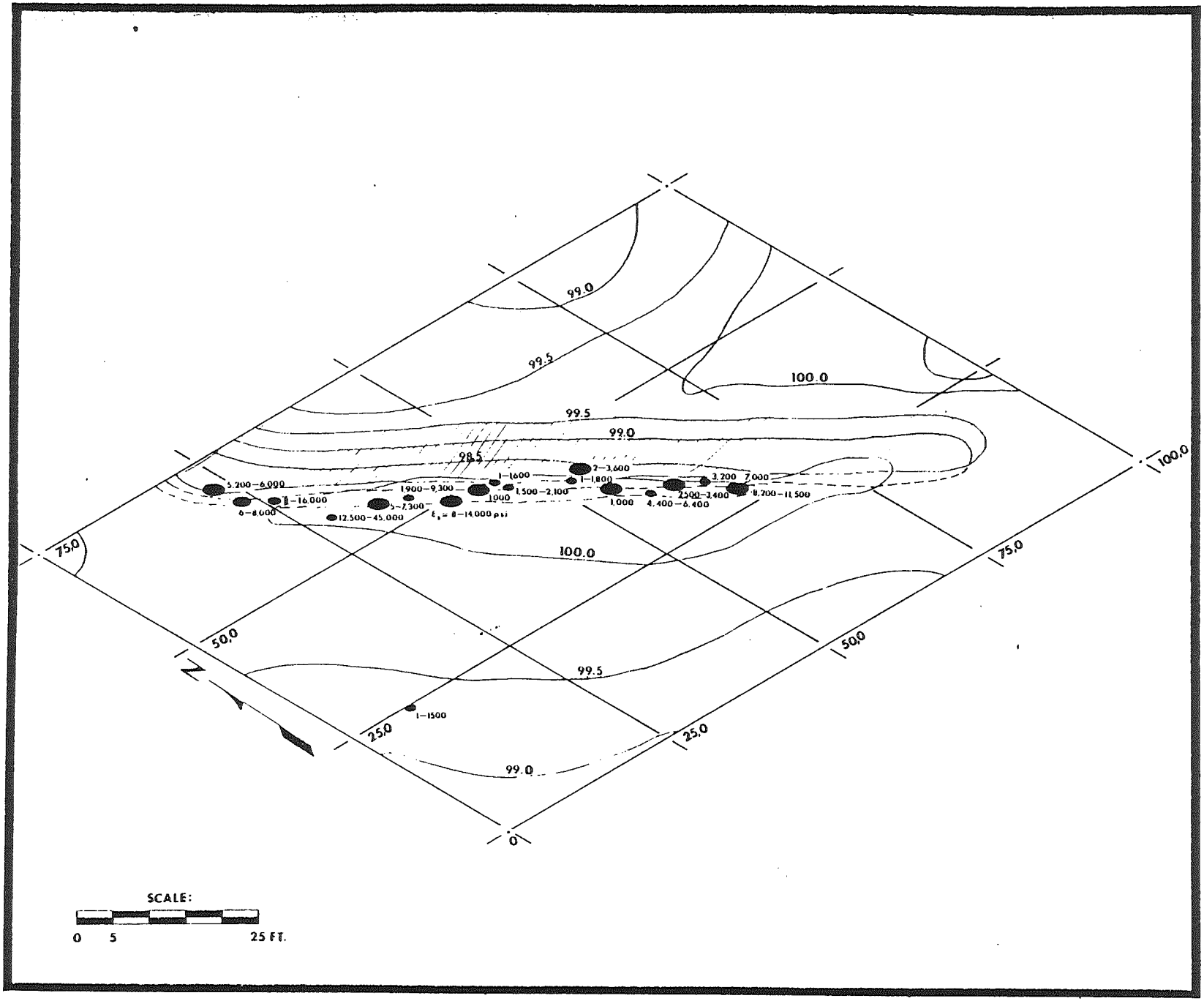


Figure 54. Visual Summary of Plate Bearing Test on Saturated Subgrade - Rainbow Valley Site

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Table 11
Results of Plate Bearing Tests
Rainbow Valley Site

No	Location (E,N)	Plate Size	E@ 30-50 psi 1st Cycle	E@ 30-50 psi 6th Cycle
RV-1	105.9, 27.5	12"	10,000-20,000psi	
RV-2	103.9, 27.5	12"	13,000-27,000	9000-15,000
RV-3	101.0, 28.1	18"	7000-10,000	
RV-4	90.0, 28.4	18"	6000-8000	4500-5500
RV-5	94.5, 28.5	24"	14,000-20,000	
RV-6	90.5, 28.2	24"	8000-10,000	5000-7000
RV-7	86.2, 28.9	30"	8000-13,500	6000-10,000
RV-8	81.6, 28.9	30"	11,500-20,000	
RV-9	77.0, 29.0	30"	18,000-30,000	
RV-10	73.3, 29.2	12"	26,000-40,000	
RV-11	68.7, 29.3	30"	10,500-17,000	
RV-12	60.7, 29.5	12"	13,000-20,000	
RV-13	51.9, 29.2	30"	5800-8500	
RV-14	48.0, 29.2	12"	10,000-18,000	
RV-15	41.3, 29.3	30"	5700-12,500	
RV-16	37.4, 29.0	12"	4000-8000	
RV-17	103.3, 7.3	30"	9000	
RV-18	101.4, 7.1	12"	23,000-40,000	
RV-19	93.3, 6.7	30"	5400-8500	
RV-20	91.1, 6.7	12"	12,500-23,000	
RV-21	32.0, 6.0	12"	12,500-20,000	
RV-22	78.3, 6.0	30"	8000-10,5000	
RV-23	68.0, 5.5	30"	8500-9500	
RV-24	64.0, 5.5	12"	9000-12,500	
RV-25	55.7, 5.1	30"	7800	
RV-26	52.0, 5.2	12"	10,000	
RV-27	24.2, 5.1	30"	4500-6000	
RV-28	40.7, 5.2	12"	11,500-30,000	
RV-29	10.0, 65.5	30"	7500-9000	
RV-30	13.0, 65.0	12"	7000-30,000	
RV-31	20.0, 65.2	12"	4000-5500	
RV-32	23.5, 65.0	30"	6000-8000	
RV-33	32.9, 64.6	30"	11,500-16,500	
RV-34	36.5, 64.5	12"	7500-30,000	
RV-35	42.9, 64.3	30"	4500-7000	
RV-36	46.0, 64.2	12"	7000-14,000	
RV-37	54.2, 64.2	30"	4000-7000	
RV-38	57.3, 64.0	12"	12,000-30,000	
RV-39	65.0, 64.0	30"	4000-7000	
RV-40	69.0, 63.9	12"	8000-35,000	

Table 11 (cont.)

No	Location (E,N)	Plate Size	E@ 30-50 psi 1st Cycle	E@ 30-50 psi 6th Cycle
RV-41	90.0, 89.7	30"	10,000-13,000	
RV-42	86.3, 89.7	12"	9000-15,000	
RV-43	75.7, 91.5	30"	7000-12,000	
RV-44	72.2, 91.7	12"	22,000-35,000	
RV-45	104.1, 39.9	30"	11,000-17,000	
RV-46	100.9, 40.1	12"	45,000-70,000	
RV-47	89.3, 44.7	12"	10,500-15,000	
RV-48	82.5, 32.0	Subsurface 30"	36,000-45,000	36,000-45,000
RV-49	80.0, 28.3	Subsurface 12"	100,000	100,000+
RV-50	72.0, 34.0	Subsurface 30"	23,000-33,000	
RV-51	69.0, 37.5	Subsurface 12"	70,000-100,000	
RV-52	63.8, 42.8	Subsurface 30"	20,000	
RV-53	62.0, 44.9	Subsurface 12"	74,000-100,000	
RV-54	57.0, 49.5	Subsurface 30"	8000-20,000	
RV-55	54.0, 52.2	Subsurface 12"	40,000-90,000	
RV-56	50.0, 56.0	Subsurface 30"	17,500-23,000	
RV-57	48.5, 57.0	Subsurface 12"	50,000-75,000	
RV-58	46.0, 59.7	Subsurface 30"	23,000	
RV-59	41.3, 64.0	Subsurface 12"	28,000	
RV-60	37.0, 69.0	Subsurface 12"	100,000+	
RV-61	54.9, 52.0	Subsurface 18"	40,000-80,000	
RV-62	26.7, 75.7	Saturated 30"	5200-6000	
RV-63	29.9, 73.0	Saturated 24"	6000-8000	
RV-64	32.0, 70.5	Saturated 18"	8000-16,000	
RV-65	38.0, 63.3	Saturated 12"	12,500-45,000	
RV-66	40.0, 61.3	Saturated 30"	5100-7300	
RV-67	42.7, 59.3	Saturated 12"	1900-9300	
RV-68	46.0, 55.8	Saturated 30"	8000-14,000	
RV-69	58.0, 48.0	Saturated 12"	1000-1800	
RV-70	51.5, 55.0	Saturated 30"	less than 1000	
RV-71	53.0, 54.0	Saturated 12"	1000-1600	
RV-72	51.0, 55.5	Saturated 12"	1500-2100	
RV-73	58.7, 48.5	Saturated 30"	2000-3600	
RV-74	61.0, 45.0	Saturated 30"	1000	
RV-75	63.3, 41.3	Saturated 12"	4400-6400	
RV-76	65.0, 40.0	Saturated 30"	2500-3400	
RV-77	67.0, 37.3	Saturated 12"	3200-7000	
RV-78	69.7, 35.0	Saturated 30"	8200-11,500	
RV-79	7.0, 24.5	Saturated 12"	1000-1500	
RV-80	9.2, 41.3	12"	7500-10,000	

Some preliminary observations regarding the results of plate bearing tests at the Rainbow Valley site are as follows:

- Lateral variation in subgrade stiffness over the 100 x 75 foot test area is evident. In the softer near surface soils (N = 1-4 blows per 6 inches), moduli below about 15,000 psi were noted with a few exceptions. As was the case with the compacted subgrade, there was a tendency to register higher modulus values with the small plates compared to the larger plates. The difference was nowhere near as great as at the Lake Pleasant sites, however.

- The plate bearing tests performed on the cut surface (12-18" below original ground, where N = 15-30) showed the same trends as did the near surface tests - the only difference being the magnitude of the modulus values. There was also variation in stiffness laterally. Moduli associated with the small plates were greater than those assigned to the larger plates, with much larger differences in value than was found in the near surface.

- For the saturated tests which were performed along the same surface (18-21") below grade, a different situation was found regarding the plate size. The larger plate tests yielded moduli which were generally the same or less than those of the smaller plates. Also, the loss in stiffness upon saturation of these native soils was much more extreme

than was the case for the highly compacted soil at Cienega Creek. For the 30 inch plates, modulus values are 10 to 25 percent of the dry values, and for the 12 inch plates, the range was 5 to 10 percent.

- A partial explanation for the parity in modulus values among both large and small plates on the saturated soil lies in the method of preparation of the test strip. Post-testing examinations show that cementation was loosening by the soaking for about 6 to 10 inches. This includes the major area of stress influence for 12 inch plates, but the influence of the 30 inch plate is felt much deeper. A more complete discussion of the role of plate size on the response of dry and wet subgrades is given in section 4.1.

- Without exception throughout the 80 tests, strain softening was found. In the 48 tests on dry near surface soil, the amount of softening was usually significant, even in the range between 30 and 50 psi. This can be seen in the modulus values in Table 11.

- The two repetitive tests (ASTM D1195) performed on the freshly cut surface exhibited an interesting induced strain phenomena. At the lower values of stress, upon release of the load as required by the test procedure, the plate rebounded to higher than its original plane, (i.e., there was complete elastic recovery and more). This can be explained

by stating that in the recently exposed cemented soil at the bottom of the trench, vertical stress relief which should accompany the unloading had not been complete. The plate load, however, induced further release, which was noted in the dial readings upon unloading.

While this and the phenomena found at the Fanglomerate Cut site may appear to be of only passing interest to the pavement designer, they illustrate a point concerning the difficulty involved in obtaining design modulus values. These stress relief phenomena which are reflected in the plate bearing test results involve at most 10 to 20 mils. The effect of even these small deflections on calculated values of modulus is tremendous for the range of stresses below 30 psi. When one considers that within both natural and reconstituted masses of earth material, vertical and horizontal stress fields which defy evaluation are usually present (particularly in soils where tensile stresses due to cementation or desiccation are involved), it is not surprising that the load response induced by plate bearing tests varies so greatly over limited areas.

3.2.6 McCormick Ranch Sites

The plate bearing tests at the two McCormick Ranch sites consisted of the following:

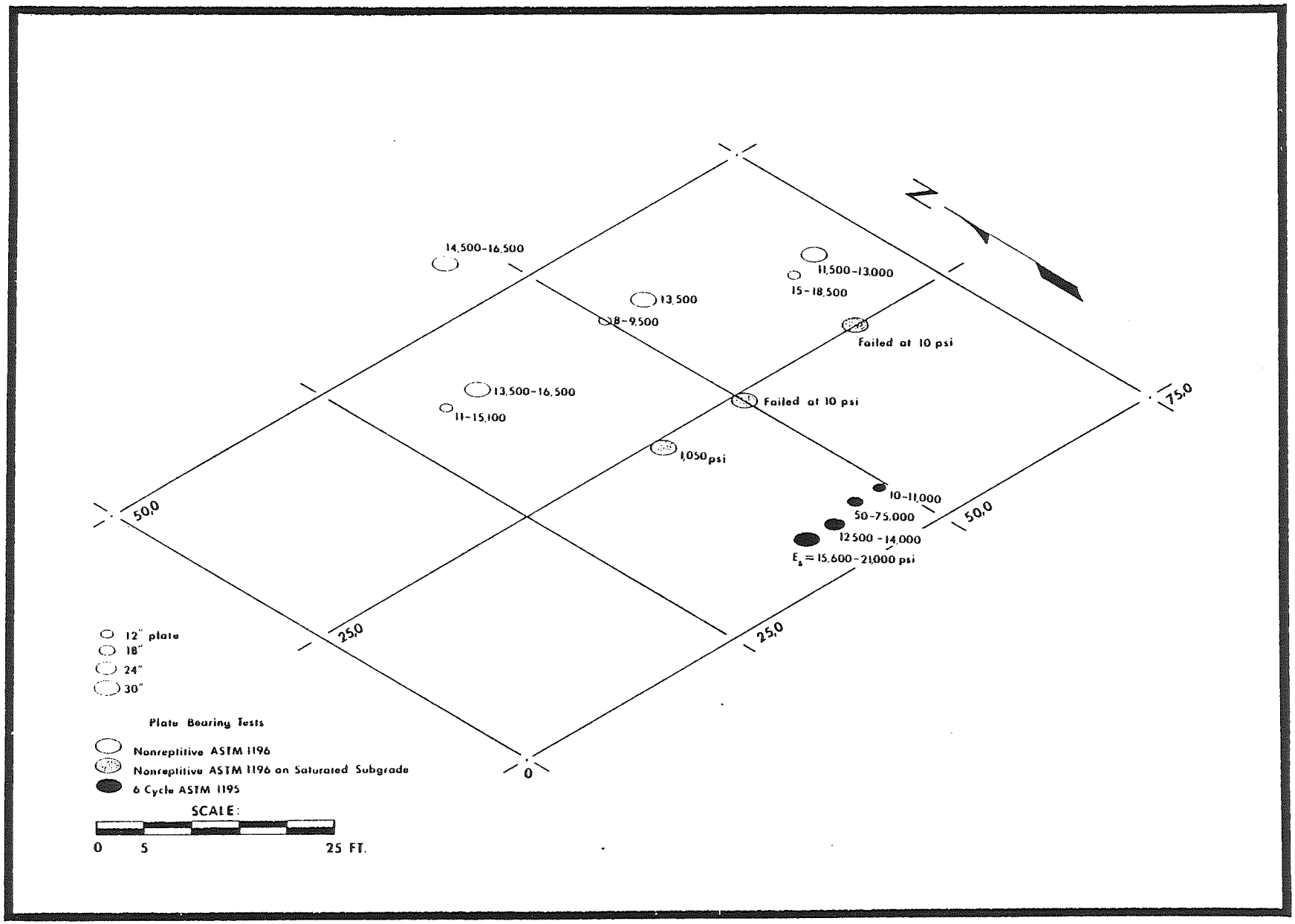


Figure 55. Visual Summary of Plate Bearing Tests - McCormick Ranch Desert Site

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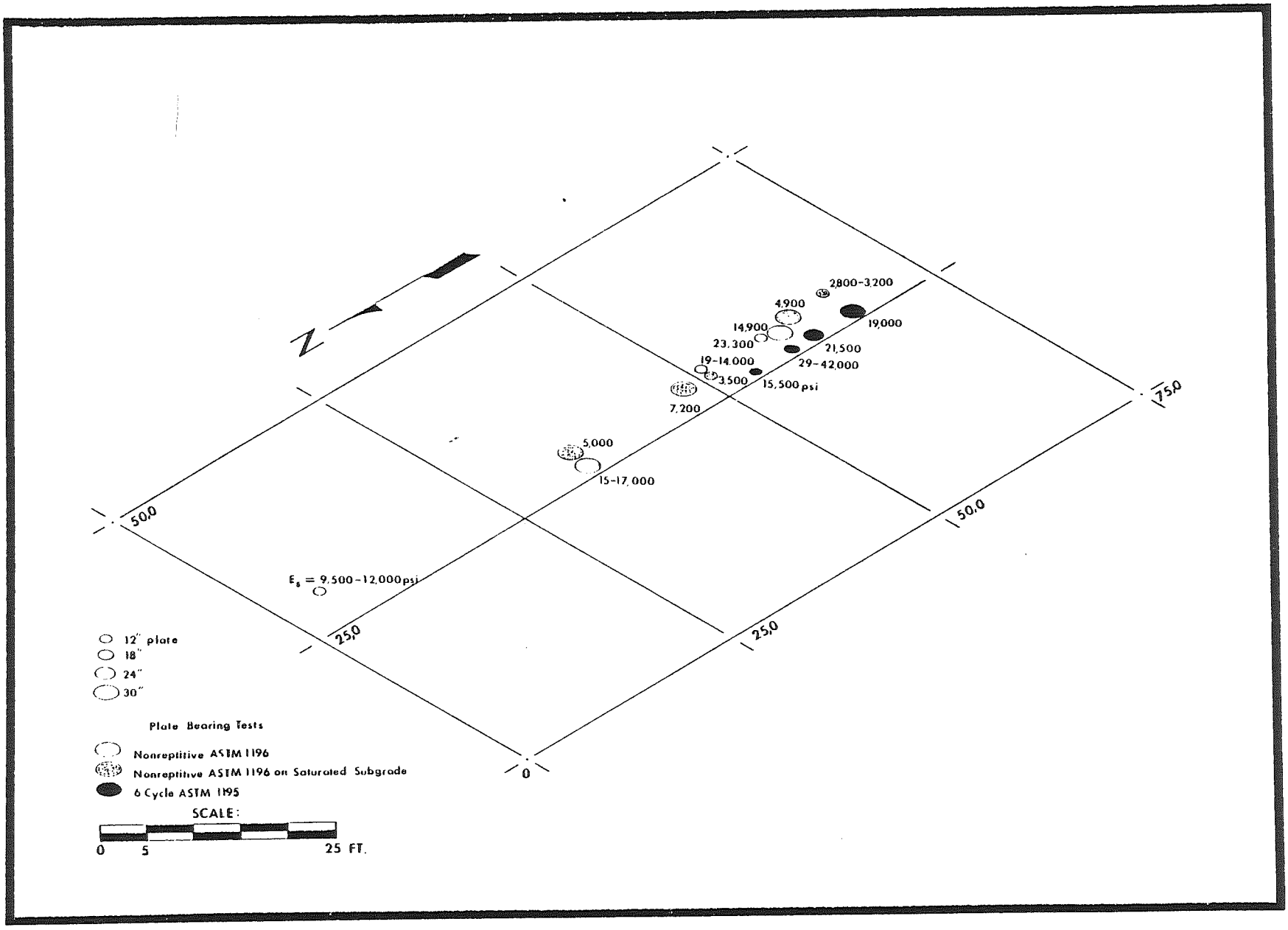


Figure 56. Visual Summary of Plate Bearing Tests - McCormick Ranch Fill Site

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- At the desert (undisturbed) site - 14 tests
  (4 repetitive - 1 each 12", 18", 24" and 30")
  (7 nonrepetitive - 4 30"; and 2 12" tests)
  (3 nonrepetitive-saturated subgrade - 3 30" tests)

- At the compacted fill site - 14 tests
  (4 repetitive - 1 each 12", 18", 24" and 30")
  (5 nonrepetitive - 2 30"; and 3 12" tests)
  (5 nonrepetitive-saturated subgrade - 3 30"; and 2 12" tests)

Figures 55 and 56 are graphical presentations of the results of these plate bearing tests at the desert and fill sites. Complete load-deflection and modulus-stress curves are found in Figures A3.2-107 through A3.2-134 in the Appendix. Table 12 is a tabular listing of the tests.

- The response shown by the plate bearing tests at the desert site are somewhat different from what had been found at the earlier sites. On this fine grained surface, the 12 inch plates seemed to generate lower modulus values than the 30 inch plates. This is understandable since the soil profile does become stiffer below 6 inches and there is no gravel or blocky cemented soil to cause resistance to movement of the small plate.

- The near surface soil at McCormick Ranch has some of the characteristics of a "collapsing" soil. When saturated, the modulus becomes so low that resistance to load is minimal.

Table 12
Results of Plate Bearing Tests
McCormick Ranch Site

No	Location (E,N)	Plate Size	E@ 30-50 psi 1st Cycle	E@ 30-50 psi 6th Cycle
Mc-1	39.5, 5.9-Desert	30"	20,000 psi	15,600-21,000
Mc-2	42.7, 6.2-Desert	24"	14,000-15,500	12,500-14,000
Mc-3*	46.7, 7.3-Desert	18"	55,000-66,000	50,000-75,000
Mc-4	49.7, 7.0-Desert	12"	15,000-18,000	10,000-11,000
Mc-9	54.0, 41.9-Desert	30"	13,500	
Mc-10	50.0, 41.9-Desert	12"	8000-9500	
Mc-11	35.0, 42.2-Desert	30"	13,500-16,500	
Mc-12	31.9, 42.2-Desert	12"	11,000-15,100	
Mc-13	69.0, 35.0-Desert	30"	11,500-13,000	
Mc-14	65.7, 34.7-Desert	12"	15,000-18,500	
Mc-15	46.5, 58.7-Desert	30"	14,500-16,500	
Mc-26	40.0, 24.0-Desert	Saturated 30"	1050	
Mc-27	50.5, 24.0-Desert	Saturated 30"	failed at 10 psi	
Mc-28	64.0, 25.0-Desert	Saturated 30"	failed at 10 psi	
Mc-5	66.2, 26.2-Fill	30"	21,000-18,000	19,800
Mc-6	62.2, 26.0-Fill	24"	24,700	21,500
Mc-7*	58.0, 26.0-Fill	18"	32,400-42,000	29,000-42,000
Mc-8	54.5, 26.0-Fill	12"	17,500	15,500
Mc-16	58.5, 27.5-Fill	30"	14,900	
Mc-17	58.4, 27.5-Fill	12"	23,300	
Mc-18	34.7, 27.9-Fill	30"	15,000-17,000	
Mc-19	52.0, 30.0-Fill	12"	19,000-14,000	
Mc-20	5.3, 30.7-Fill	12"	9500-12,000	
Mc-21	34.0, 28.3-Fill	Saturated 30"	5000	
Mc-22	48.7, 29.7-Fill	Saturated 30"	7200	
Mc-23	52.0, 30.0-Fill	Saturated 12"	3500	
Mc-24	62.5, 30.0-Fill	Saturated 30"	4900	
Mc-25	66.3, 30.0-Fill	Saturated 12"	2800-3200	

*Results of these tests are highly suspect.

- The results of plate bearing tests on the controlled fill provide an insight into the inherent variations in earth materials. Both the penetration tests and the refraction seismic results (see section 3.3.5) affirm that this fine grained fill is as nearly a uniform earth mass as can be built. Therefore, it was expected that very little variation would exist in the plate bearing moduli. In fact, E moduli from 9500 to 42,000 psi were found although the 14,000 to 24,000 range contains most of the values. Perhaps an explanation lies in the fact that this fill (which does contain some natural lime cementing) has been in place for 5 years and has undergone several cycles of wetting (from the infrequent rainfalls) followed by long periods of drying. Thus, subtle changes in the horizontal and vertical stress fields are present to a degree which is reflected in the results of these tests.

Other items of note concerning plate bearing tests on the controlled fill are:

- It is not possible to say what the effect, if any, plate size has been on the load response of this fill.

- The tests on the artifically saturated fill revealed about a 75 percent loss in stiffness as compared to the dry condition.

3.3 Results of Refraction Seismic

3.3.1 I-10 Buckeye Site

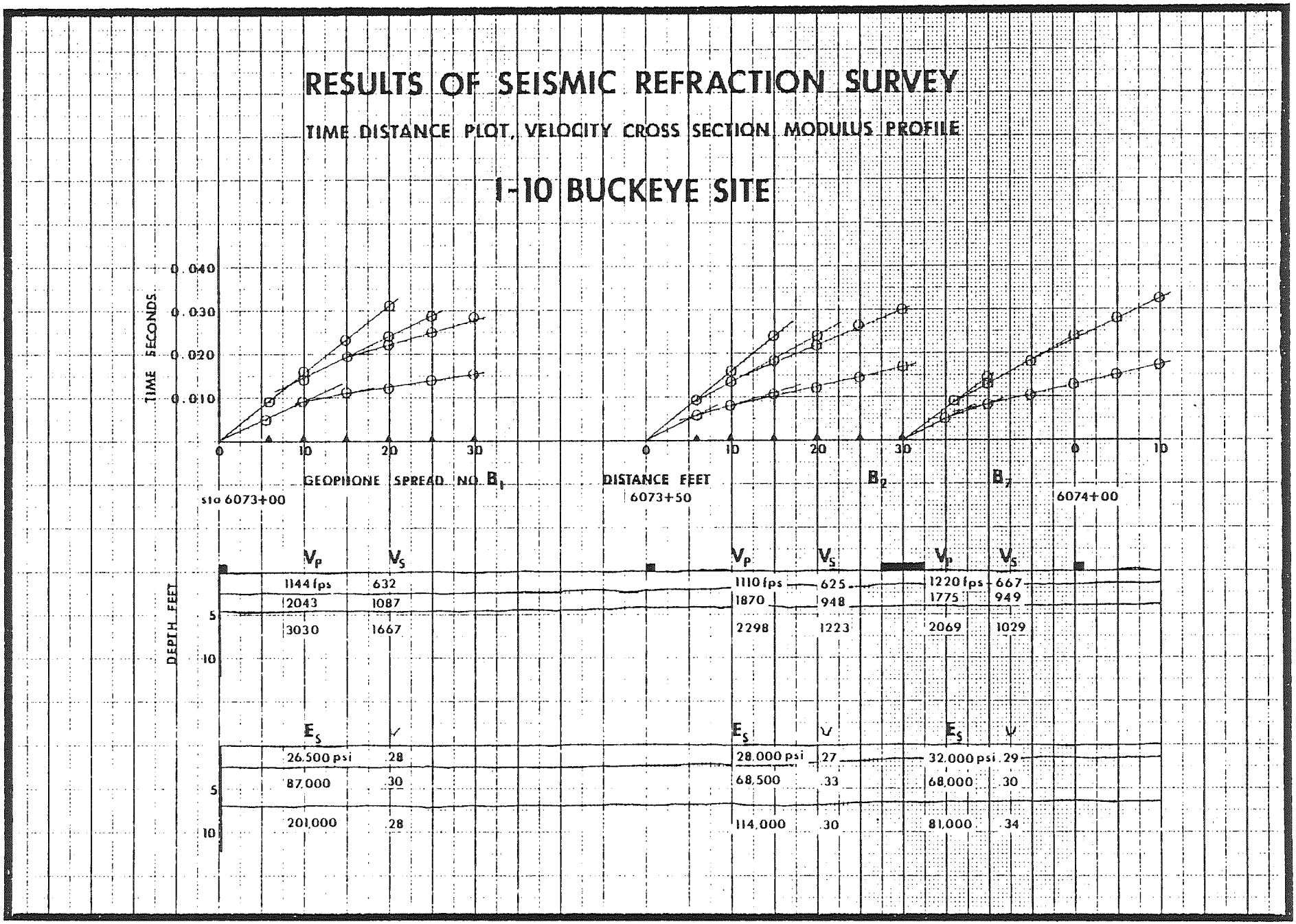
Results of the Seismic Refraction Survey at this site are given in Figure 57. It appears that the modulus determined by geophysics in the upper 2 feet compares very favorably with the moduli derived from the smaller plates in the 30 to 50 psi range. Moduli are only 50 to 75 percent greater than calculated using the large plates.

This was somewhat, though not totally, unexpected. As previously stated, it was expected the values of modulus calculated from plate bearing tests would be smaller than values determined seismically due to the difference in strain level. Based on the Hardin and Drnevich findings, it was thought that the appropriate reduction would fall between about 25 and 75 percent of the plate bearing values. Such was not the case at this site, however.

An interesting finding in the geophysical study at this site is that Poisson's ratio in the cut section ranges from about 0.27 to 0.33 but, in the deep compacted material, is 0.17 to 0.19, indicating a somewhat more compressible subgrade structure.

3.3.2 Lake Pleasant Site

Seismic refraction lines were run at the Orange site and at the Fanglomerate Cut. The results are presented in Figure 58.


RESULTS OF SEISMIC REFRACTION SURVEY
TIME DISTANCE PLOT, VELOCITY CROSS SECTION, MODULUS PROFILE
I-10 BUCKEYE SITE
TIME SECONDS
0.040
0.030
0.020
0.010
0
10
20
30
GEOPHONE SPREAD NO. B1
sta 6073+00
DISTANCE FEET
6073+50
B2
B7
6074+00
DEPTH FEET
5
10
Vp
Vs
1144 fps
632
2043
1087
3030
1667
1110 fps
625
1870
948
2298
1223
1220 fps
667
1775
949
2069
1029
Es
ν
26,500 psi
.28
87,000
.30
201,000
.28
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68,000
.30
81,000
.34

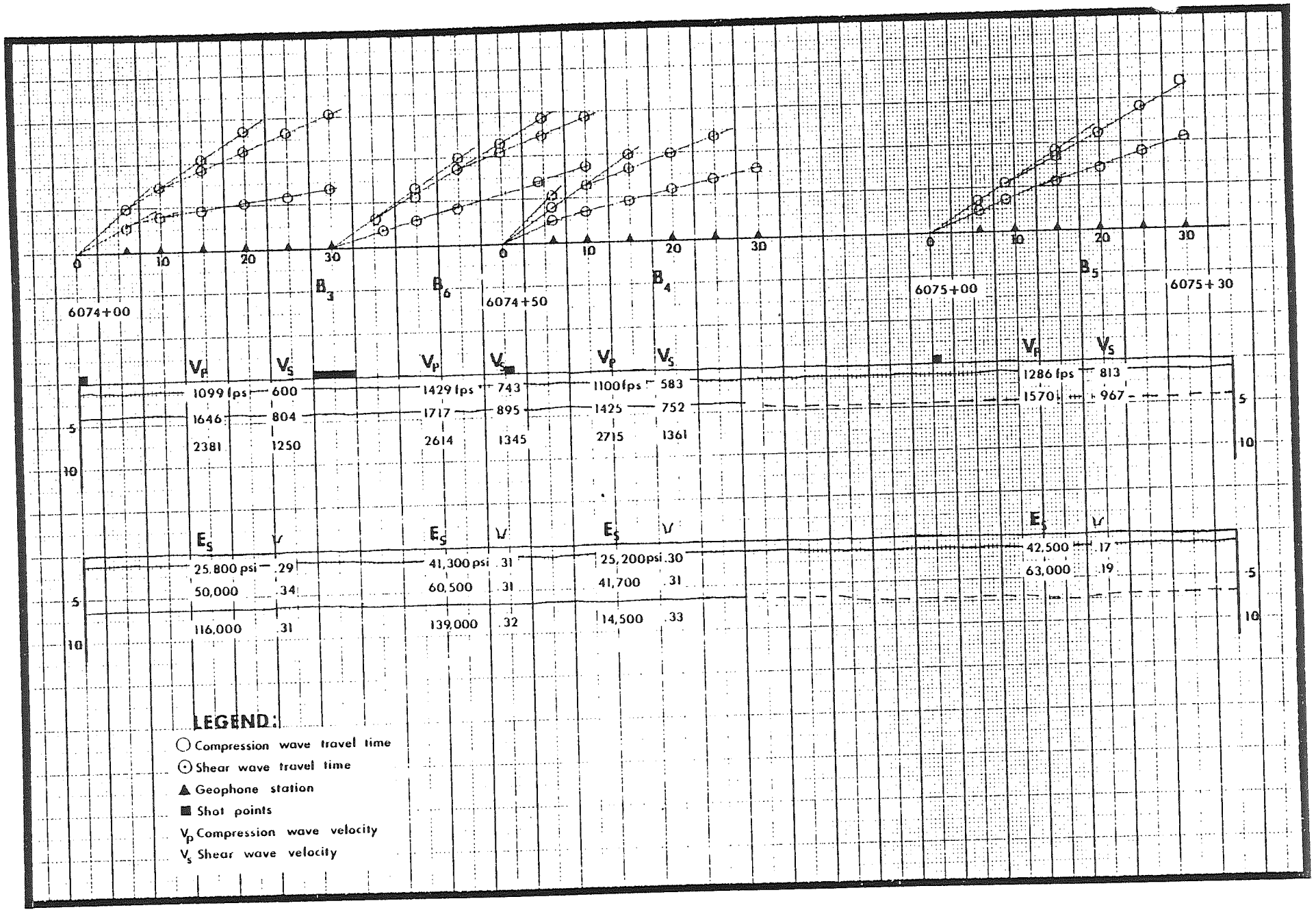
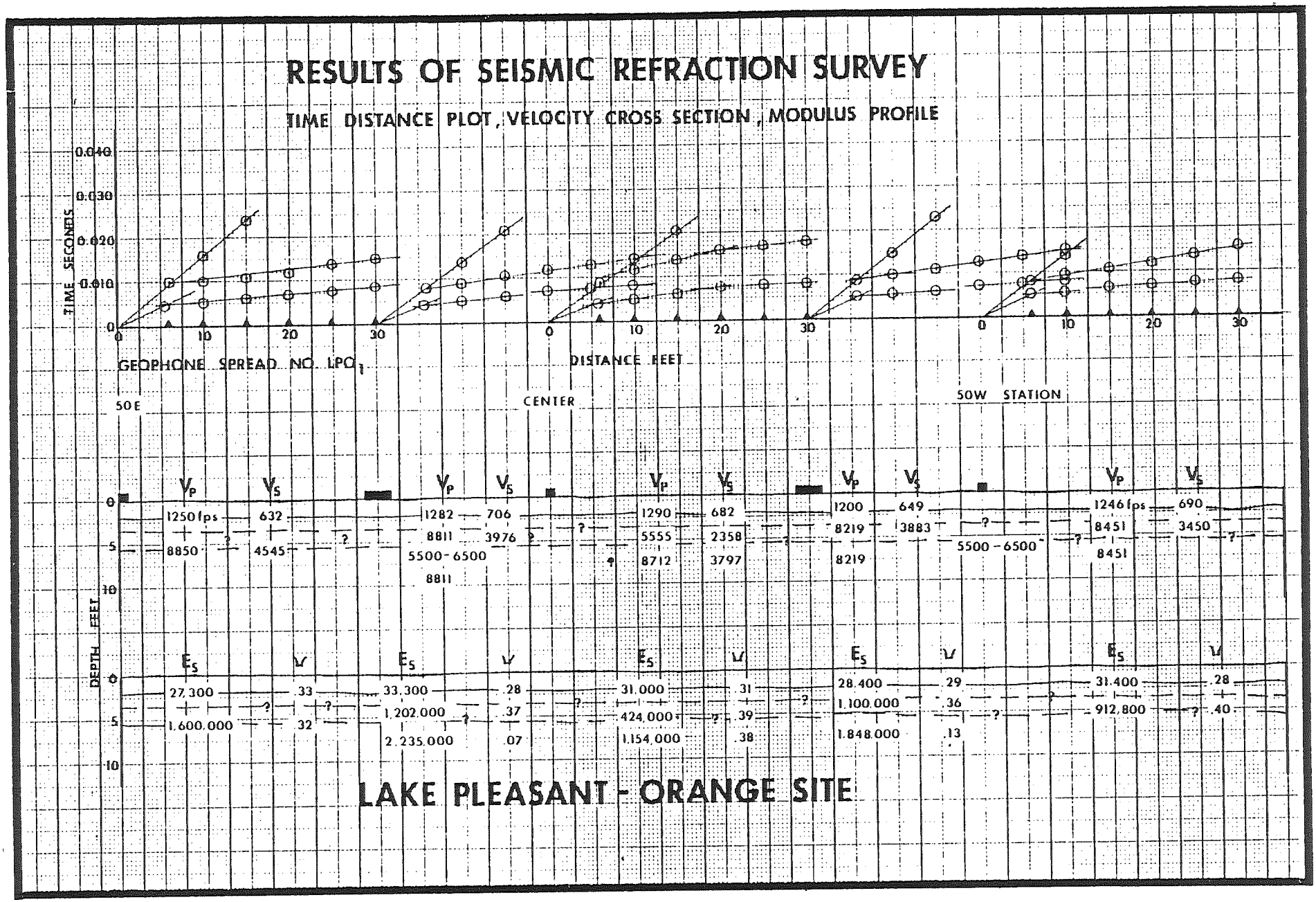


Figure 57. Graphical Summary of Refraction Seismic - I-10 Buckeye Site

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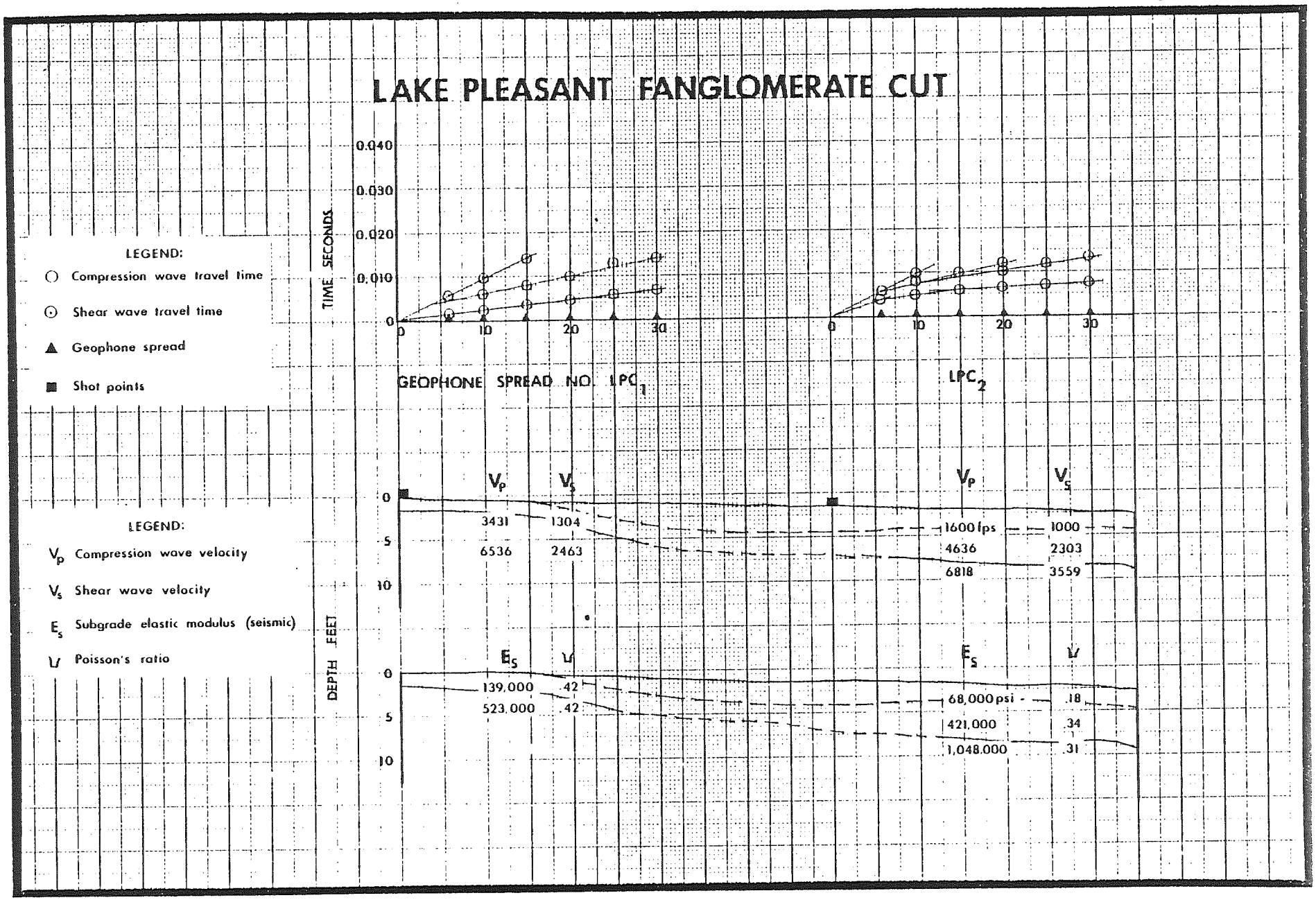


Figure 58. Graphical Summary of Refraction Seismic - Lake Pleasant Sites

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The seismic refraction study showed the Fanglomerate Cut to have three layers from the standpoint of seismic velocity. Near the surface, the elastic modulus is 68,000 psi. Below 2 feet, E increases to the 100,000 to 400,000 range and below about 5 feet is 500,000 to 1,000,000 psi. These values are very reasonable for softer rock, thus, reinforcing the suspicion that there is a stress relief or possibly a surface weathering phenomena which is affecting the load response at this site. This idea is reinforced by the fact that Poisson's ratio is 0.18 for the near surface compared to 0.34 to 0.42 for the deeper rock, indicating there is somewhat greater vertical compressibility at the surface.

At the Orange site, the seismic moduli for the upper 3 feet of compacted clayey sand and gravel were about three times the E value resulting from the large plate bearing test in the 30 to 50 psi range. On the other hand, E_{seis} is less than the value from the 12 inch plate. This would tend to add support to the suggestion given in section 3.2.3 that the 12 inch plate bearing tests on subgrades containing gravel and cobbles or blockey cemented materials do not measure the "mass response" of the subgrade.

3.3.3 Cienega Creek Site

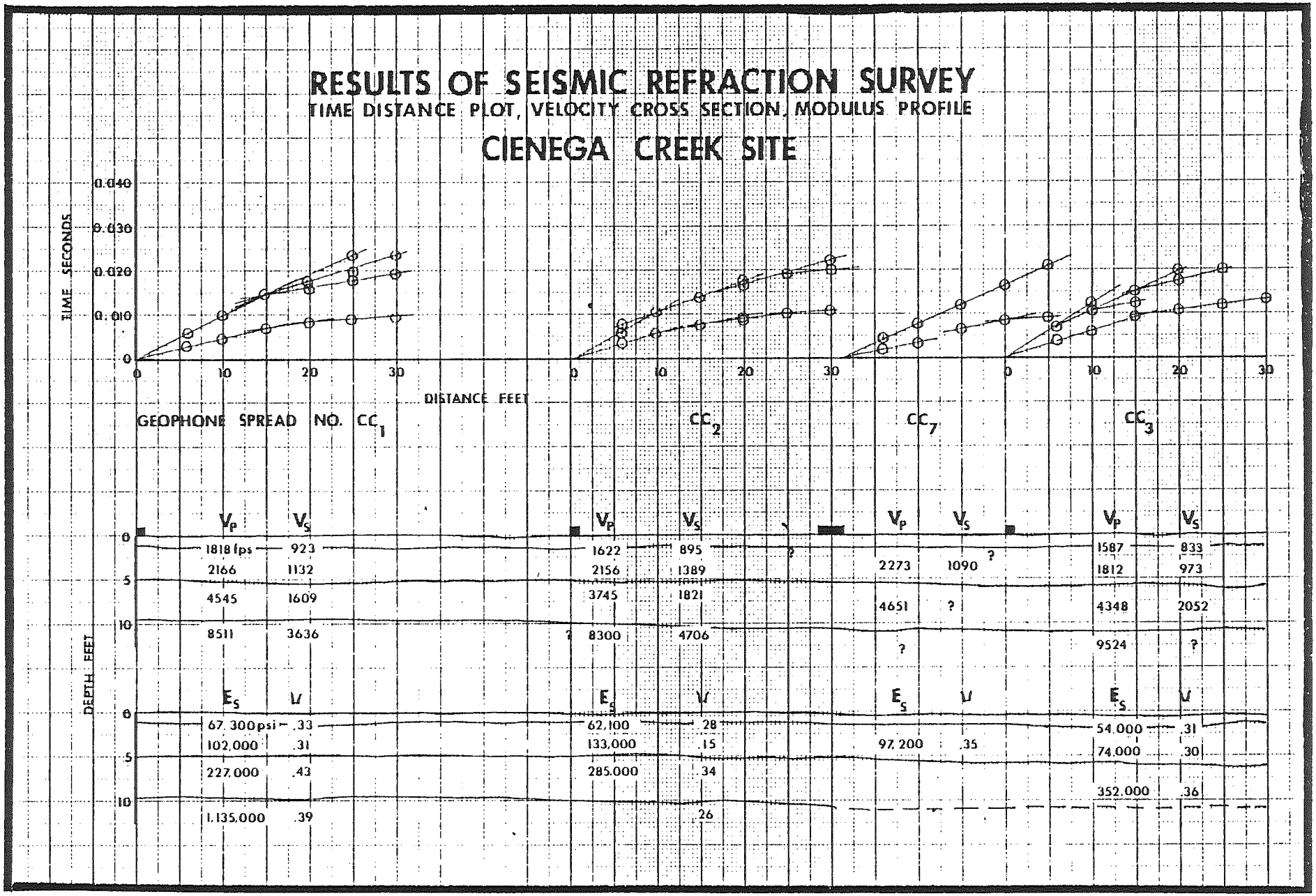
The refraction seismic survey at the Cienega Creek site is presented in Figure 59. It revealed E_{seis} modulus values of 50,000 to 70,000 psi in the compacted subgrade, 75,000 to 130,000 psi in the native

clay subgrade, and 180,000 to 350,000 psi in the weathered granite. These results are consistent with published data on strain softening, as the near surface values are in the range that was expected (about four to five times the values determined from plate bearing tests).

Moduli calculated for the second layer are higher than surface values by 50 to 100 percent. This may seem analmous inasmuch as it was stated earlier that beneath the surface on this dry, densely compacted road, the soil is higher in moisture content and other tests indicate it is somewhat softer. This may still be true. A drawback to the use of seismic velocities is that in two phase systems (soil-water) they do not properly reflect the loss of strength which accompanies increasing amounts of moisture in a soil at higher strains. In fact, the opposite result may occur. Near saturation in a dense clayey material would probably create a higher shear wave velocity and, therefore, a greater E modulus would be indicated. Lower moduli at higher strains may be affected by pore pressure buildup in saturated soils.

3.3.4 Rainbow Valley Site

At both the Phase II test sites, compression wave velocity only was determined, therefore, the E_{seis} values are dependent on assumed values of Poisson's ratio.



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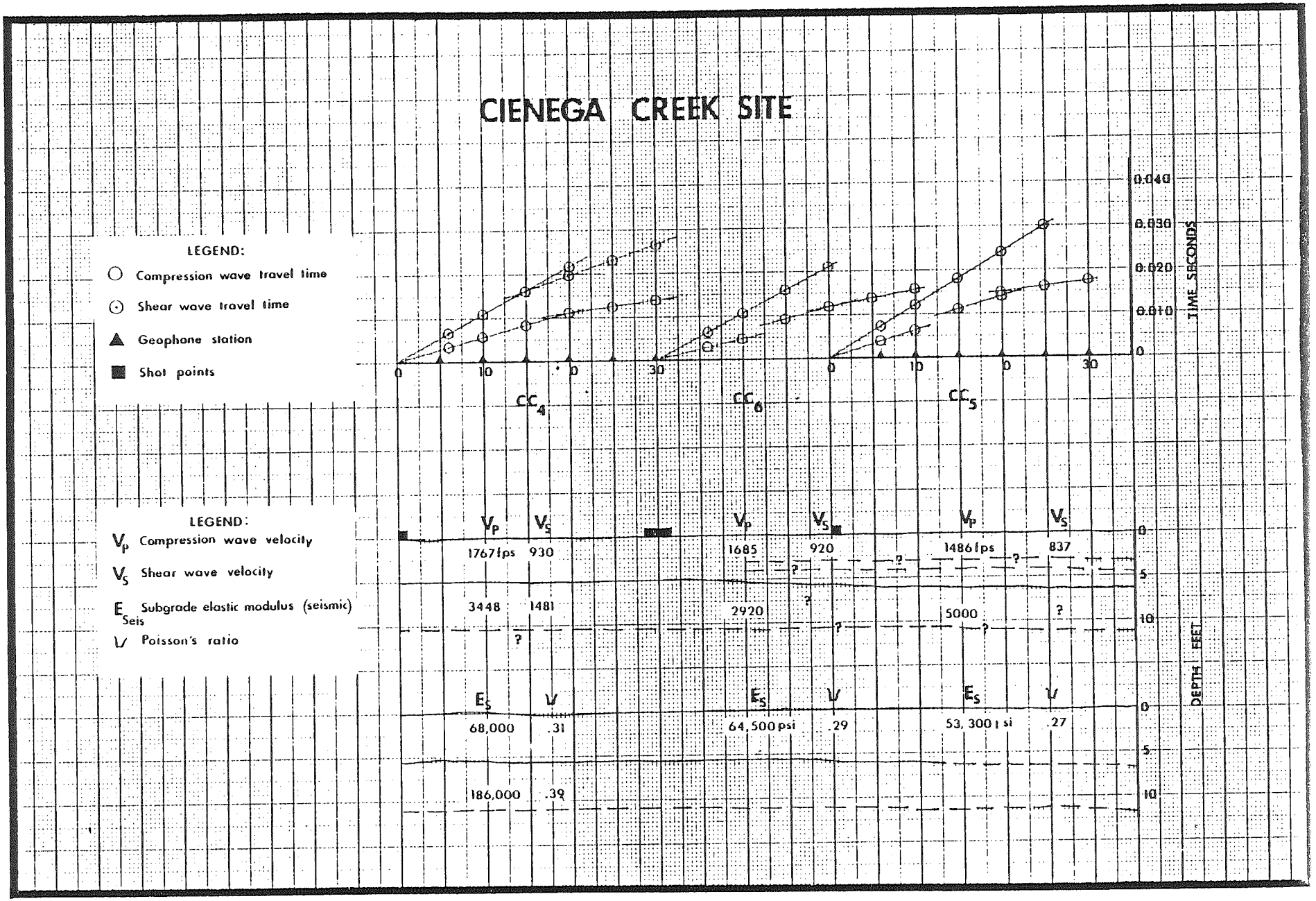


Figure 59. Graphical Summary of Refraction Seismic - Cienega Creek Site

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At the Rainbow Valley site, 12 traces were run in the near surface and six were run in the trench at the 18 to 24 inch below grade level. Based upon our experience at the three Phase I sites, 0.3 was chosen as the probable Poisson's ratio for the weakly cemented near surface soil, and 0.2 for the soils in the excavated trench.

Results of the refraction seismic survey at Rainbow Valley are given in Table 13 and are displayed visually in Figure 60. Time-distance plots and a more complete tabulation of the data are given in the Appendix.

Because E_{seis} depends on the velocities of wave travel over significant distances in the subgrade, it was thought that this value of modulus might be different from others, in that local stress field variations and small locally soft or stiff zones would have little effect on value. Thus, it was thought that E_{seis} might closely represent the mass property of the subgrade.

The results, as seen in Table 13, however, show that E_{seis} indicates considerable lateral variation in stiffness at this site. Most important, however, is the fact that, taken collectively, there was no difference between E_{seis} values from the lines on the surface as compared to those in the trench. Ten foot stationing was used to derive the time-distance plots. Apparently this was not adequate to identify a surface stratum less than a foot thick. This

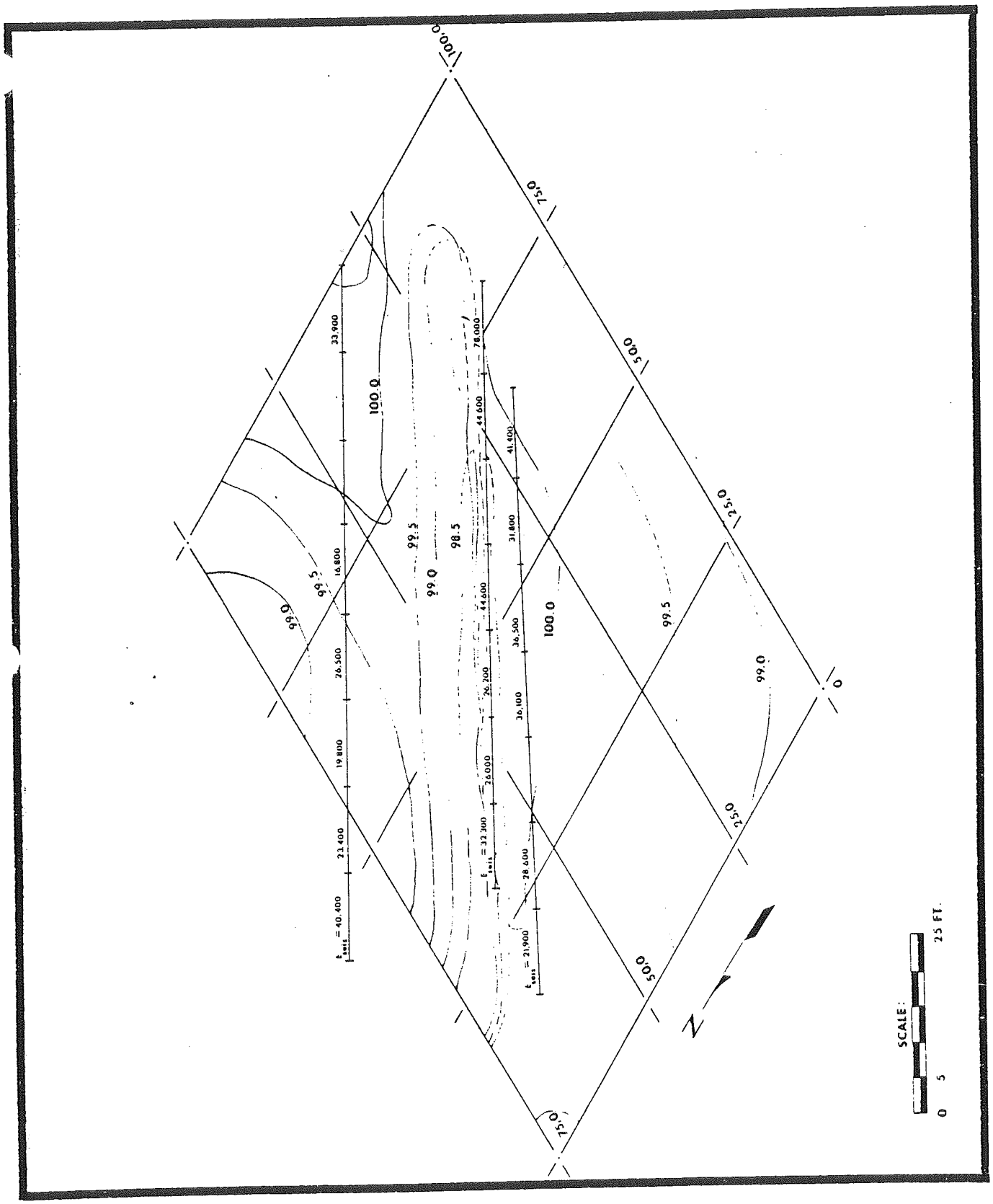


Figure 60. Visual Summary of Refraction Seismic - Rainbow Valley Site

demonstrates another possible disadvantage of using seismic refraction to determine E_s. To identify the modulus of a thin softer stratum over a much harder material would require that about a 2 foot shot point spacing be used. This may result in problems of interpretation (due to scatter), and would probably be more expensive than running a conventional line. Fortunately, this situation will seldom arise where compacted subgrades are involved.

Values of E_{seis} from the excavated trench show the same relationship to the values calculated from the plate bearing tests, as was the case at the I-10 Buckeye site. That is, modulus values were 50 to 75 percent greater than determined from the 30 inch plates and compared well with the values assigned from the 12 inch plates. It must, therefore, be concluded that in stiff cemented soils, the reduction in modulus for increased strain (from 10^{-6} in/in to about 10^{-3} in/in) is not as drastic as is the case for dry cohesionless soils.

Table 13
Summary of Refraction Seismic Testing
at Rainbow Valley Site

Near Surface - ν = 0.3 (assumed) γ_d = 95 pcf

Line No.*	V_c	E_{seis} (psi)
1	1630	40,400
2	1240	23,400
3	1140	19,800
4	1320	26,500
5	1050	16,800
6	1490	33,900
7	1650	41,400
8	1445	31,800
9	1550	36,600
10	1540	36,100
11	1200	21,900
12	1370	28,600

Excavation - ν = 0.2 (assumed) γ_d = 105 pcf

13	1260	32,300
14	1130	26,000
15	1135	26,200
16	1480	44,600
17	1390	39,400
18	1960	78,000

*See Figure 60 for location of lines

3.3.5 McCormick Ranch Site

E_{seis} was determined along 12 lines at the McCormick

Ranch controlled fill site. Locations are given on Figure 61. A compilation of time-distance data is included in the Appendix.

The refraction seismic data proved that as far as mass response of the fill is concerned, there is very little lateral variation. Seismic methods apparently are effective in averaging out local variations in stiffness such as noted by plate bearing tests.

The 12 seismic traces are virtually identical, therefore, no tabulation of results is presented.

The material in the upper several feet has a velocity of 1150 feet per second. Using $\nu = 0.35$ and $\gamma = 120$ pcf, this translates to an E_{seis} of 21,300 psi. This value is about 10 to 50 percent greater than derived from the plate bearing tests (both large and small) adding further credibility to the thesis that this is the appropriate range of difference between modulus values determined by the two methods.

3.3.6 Findings Concerning Poisson's Ratio

It is often assumed that since the range of Poisson's ratio of elastic materials is from 0.0 to 0.5 and because most elastic solutions incorporate the term in the form $(1 - \nu^2)$, that accuracy is not critical where selection of the value is concerned. For this reason, little research has been published regarding in-situ values of Poisson's ratio.

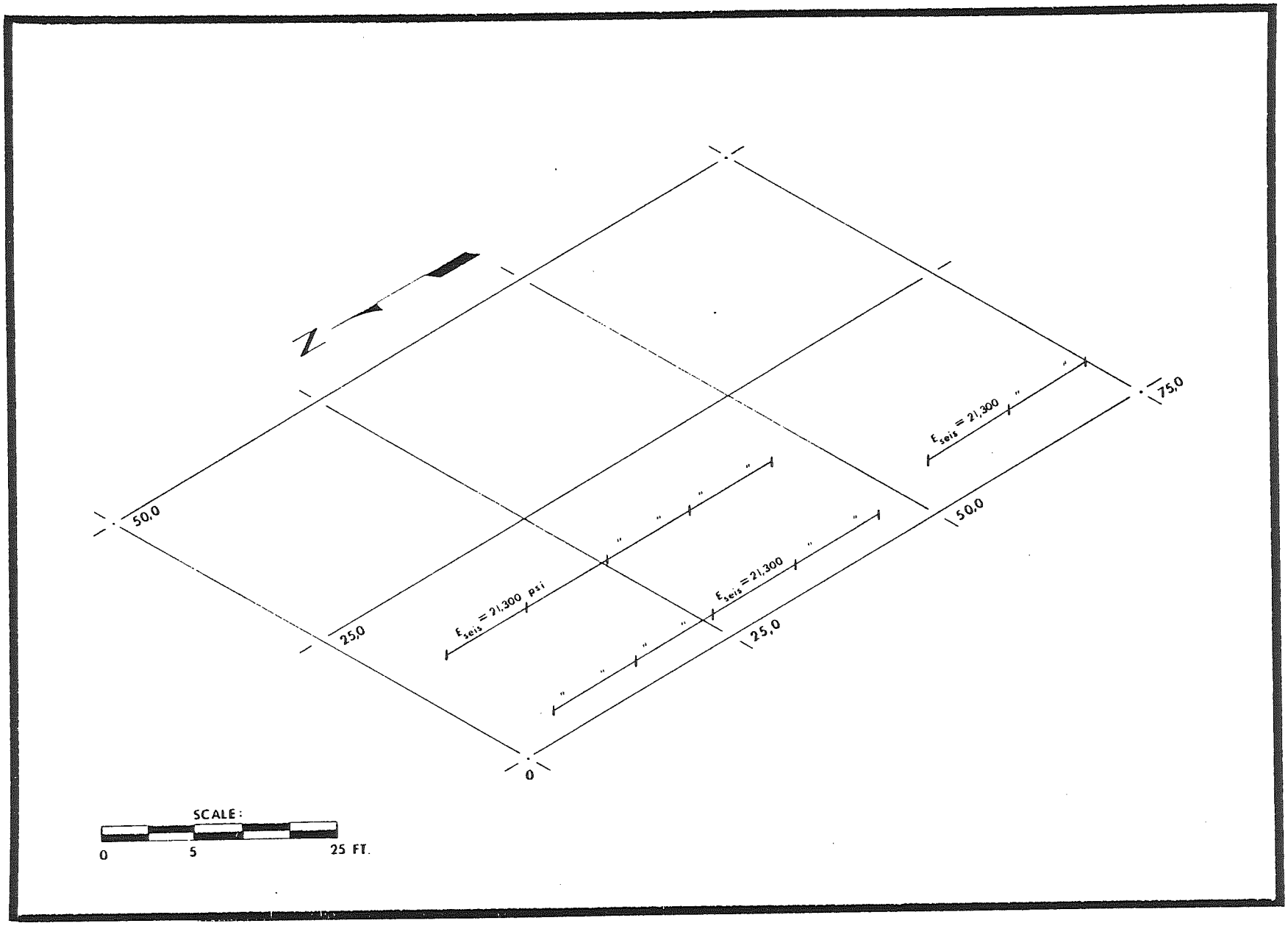


Figure 61. Visual Summary of Refraction Seismic - McCormick Ranch Site

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Included as part of the results of the surface refraction seismic work performed by our consultant Richard T. Zbur, are derived values of Poisson's ratio at each of the Phase I test sites. Based on Zbur's results, a listing has been compiled to suggest values of Poisson's ratio applicable to typical Arizona soils.

The interested reader is referred to Zbur's study (137) on cemented desert alluvium at Estancia Valley, New Mexico, where similar values of Poisson's ratio were recorded.

Correlation of Poisson's Ratio

Subgrade Type	Poisson's Ratio
Fine grained soils, low plasticity, weakly to moderately lime cemented (native)	0.28 to 0.34
Fine grained soils, moderate lime content (recompacted & recemented)	0.16 to 0.20
Clayey sand & gravel mixtures (recompacted, not cemented)	0.26 to 0.33
Sand & gravel, strongly cemented	0.36 to 0.42
Medium to highly plastic clays & clayey sands (desiccated)	0.28 to 0.32
Medium to highly plastic clays & clayey sand, near plastic limit	0.38 to 0.45
Cemented soils subjected to vertical stress relief	0.10 to 0.20

3.4 Results of Field CBR Testing

3.4.1 Rainbow Valley Site

In Figure 62, the results of 12 in-situ CBR tests are given graphically. For comparison, results of the 30 inch plate bearing tests are also imposed.

A definite parallel between CBR value and stiffness determined by the other methods is indicated by the results.

An objection which is often raised in relation to the CBR test concerns the danger that since the piston area is so small (3 square inches) it may meet stiffer than typical resistance (such as an embedded cobble or calcareous concretion), the result being an erroneous reading. During testing, however, a careful visual control was employed in an attempt to avoid such occurrences. It is believed that success was achieved both at Rainbow Valley and McCormick Ranch sites in obtaining CBR results representative of the mass of the soils.

It appears that a reasonable correlation between CBR design subgrade modulus E_s would be

$$E_s = 500 \times CBR \text{ (psi)}$$

This is near the lower range of comparison ratios reported in the literature.

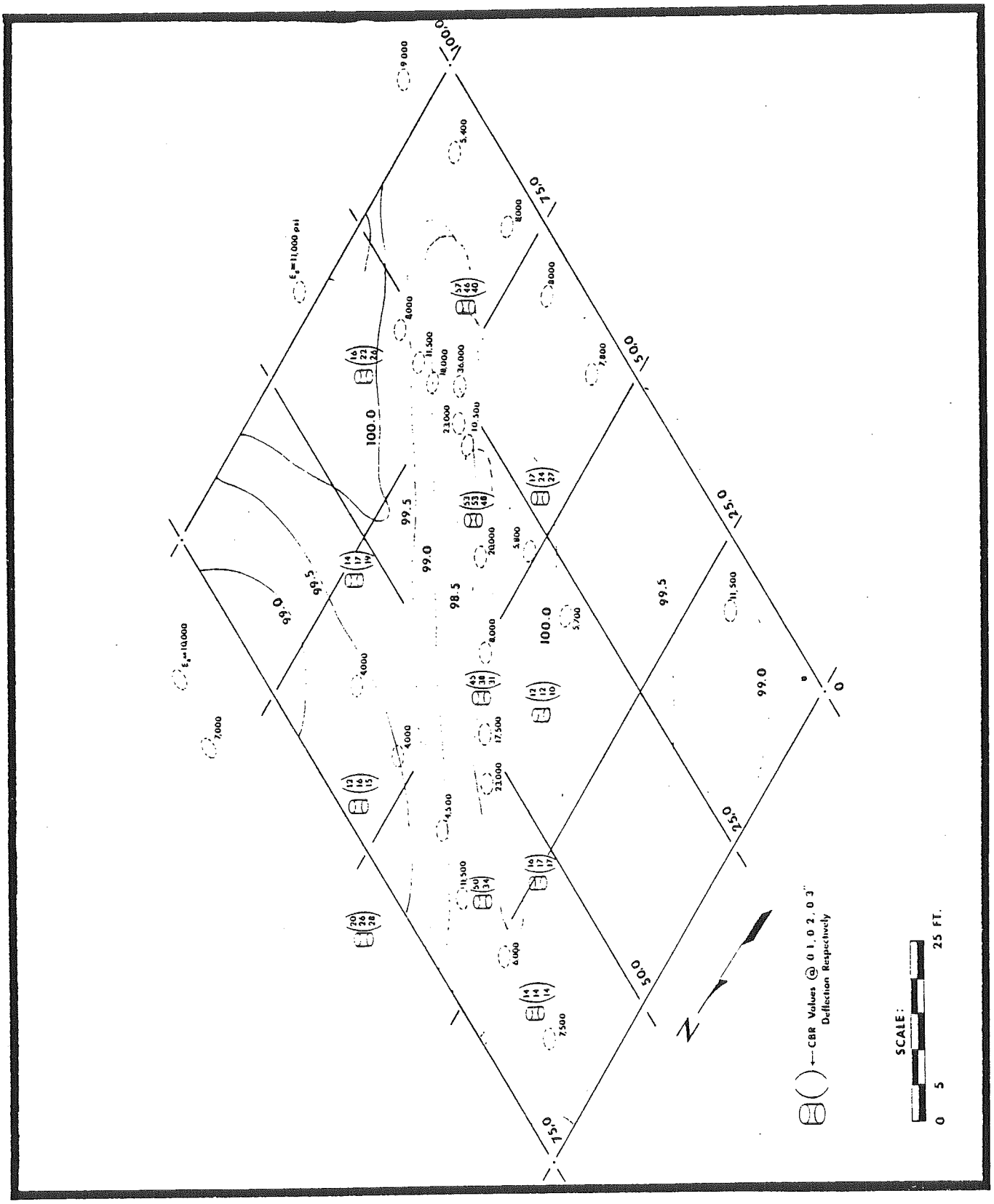


Figure 62. Visual Summary of CBR Testing - Rainbow Valley Site

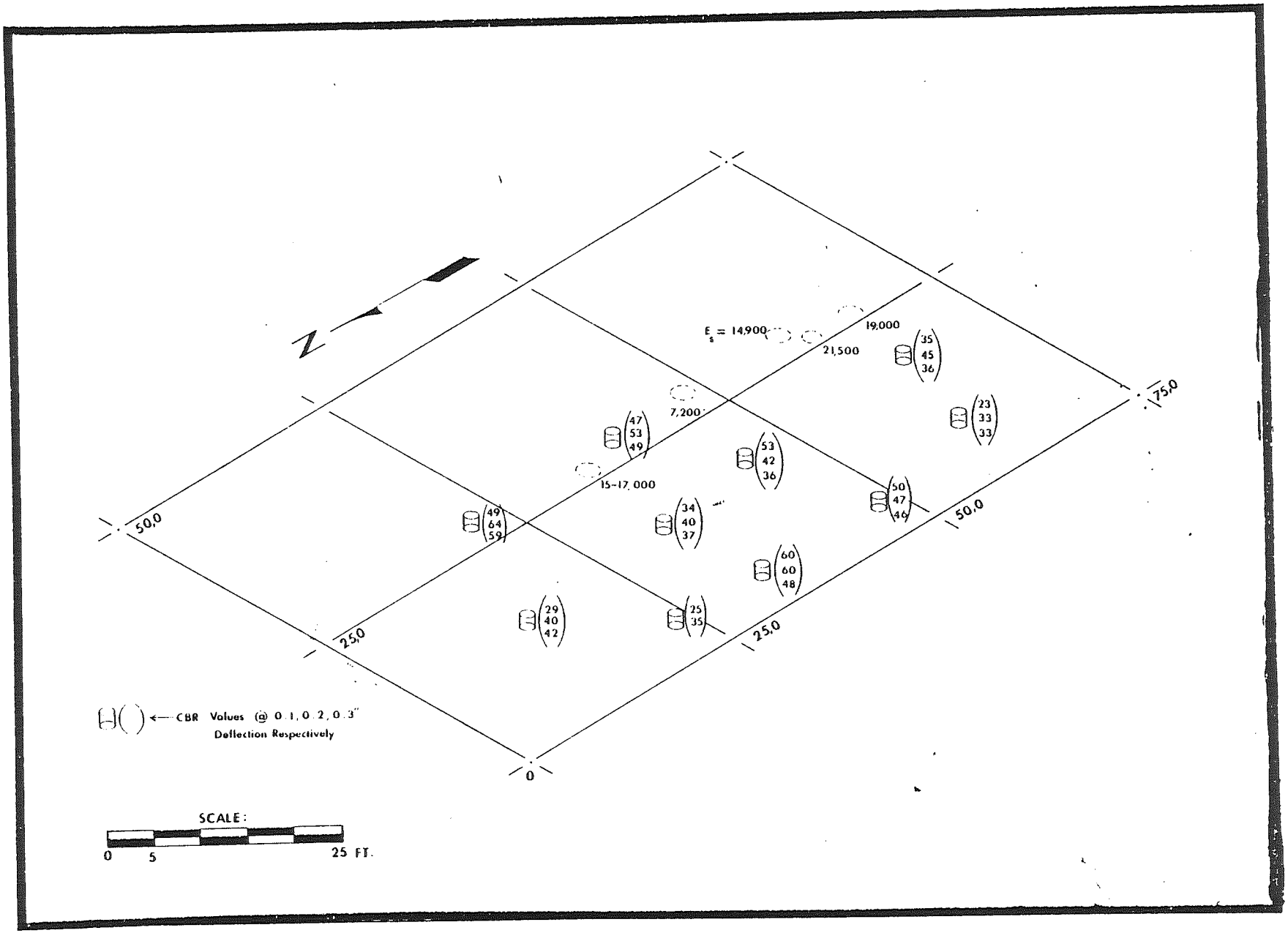


Figure 63. Visual Summary of CBR Testing - McCormick Ranch Fill Site

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3.4.2 McCormick Ranch Site

The 10 in-place CBR's exhibited a greater amount of scatter than expected for the uniform fill surface. Nevertheless, the average of the 10 values is 40, which, when compared to the E_s values derived from the 30 inch and 12 inch plate bearing tests, confirms that the relationship found at Rainbow Valley is reasonable, e.g.,

$$E_s = 500 \times \text{CBR in psi}$$

Results of the CBR's at the McCormick Ranch fill are displayed in Figure 63.

3.5 Results of Dynaflect Testing

3.5.1 General

The calculation of E modulus of soil using remote surface deflections induced by a vibratory load is obviously much more difficult than analyzing a situation where a static load and straightforward geometry are involved. The use of the term "remote deflections" refers to the fact that with the Dynaflect (and Road Rater), the motion sensing devices monitor deflections not at the spot where the loaded area contacts the surface, but at discrete distances within its sphere of influence. From the five readings of the Dynaflect sensors, one can plot a deflection basin, which serves as an indicator of the response of the material(s).

As previously mentioned, three well-known investigators (Scrivner, Vaswani, and Majidzadeh) have presented methodologies which include the means to derive values of subgrade modulus, E_s, from sets of Dynaflect measurements on pavements. Of these, only Majidzadeh has dealt with measurements taken directly on subgrades. No correlations using the Arizona subgrade deflection basins could be found.

The major shortcoming of the existing body of research developed by the three investigators mentioned above, is that most of the effort has been spent sorting out the response of multiple layers from single arrays of Dynaflect readings. The requirements of this project are probably simpler, but, apparently have received little attention. That is, the use of Dynaflect readings to calculate E_s of a homogeneous elastic continuum (which, however simplified, is the model most pavement analysts use to describe the behavior of subgrades) has not been previously considered in detail.

It was hoped by the authors at the outset that a means could be found to incorporate into the Dynaflect problem the Ahlvin and Ulery (8) solution for the elastic deflections at some distance from a known circular load. Although the Ahlvin and Ulery analysis applies to the static load situation, rather than vibratory, it was thought that empirical adjustments could be applied which would result in credible design values of E_s.

The Ahlvin and Ulery analysis is as given below in Figure 64.

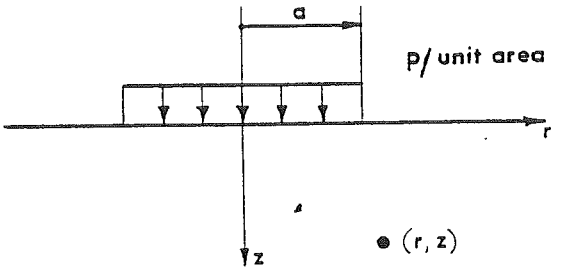


Figure 64. Ahlvin & Ulery Expression for Deflection Due to a Uniformly Loaded Circular Area.

H is an influence factor which is a function of the ratio r/a where r is the distance from the point of interest to the center of the loaded area.

Figure 65 is a graphical presentation of the Dynaflect sensor configuration and its adaptation to the Ahlvin and Ulery expression. The actual shape of the wheel print is probably an oval, but for analysis, a circle with a radius of 1.12 inches is employed. Computations are made using the influence factors, H, tabulated for each of the values of deflection, d_1, d_2, d_3, d_4, d_5. For discussion, these calculated moduli are labeled E_{d_1}, E_{d_2}, E_{d_4}, and E_{d_5}.

Table 14 is a tabular listing of all Dynaflect readings taken at each of the five sites. The process of

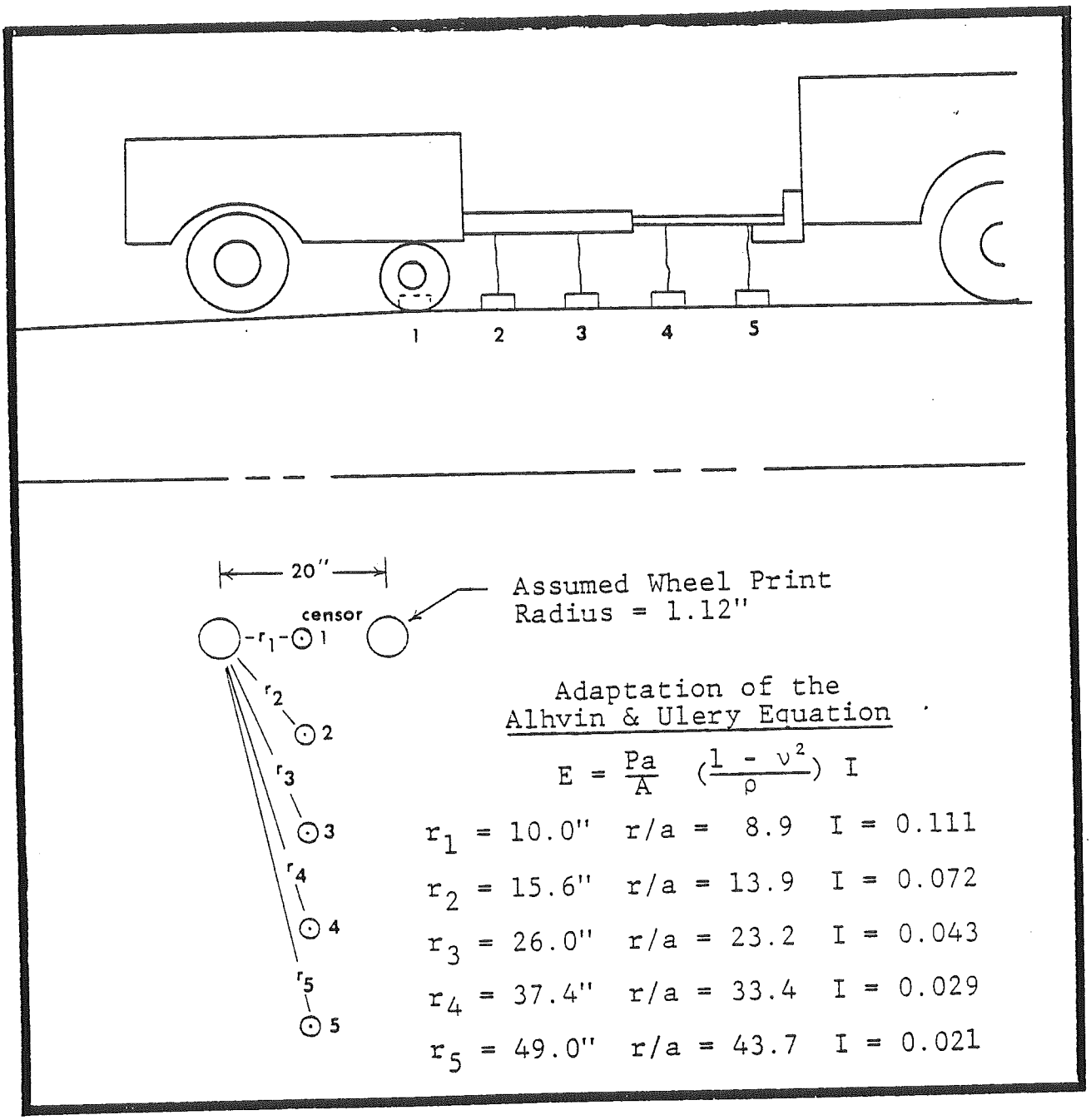


Figure 65. Dynaflect Configuration

developing an empirical procedure for calculating an elastic modulus of a subgrade from Dynaflect data (E_{dyn}) which correlates well with E_s from plate bearing tests was one involving numerous trial-and-error manipulations of data. It was immediately discovered that the maximum deflection, d_1, (hence E_{d_1}) was, of itself, an insufficient indicator of subgrade response, due to scatter. As the three major publishers of Dynaflect research found, it is essential to make use of all five sensor readings and resulting deflection bowls to analyze the materials' response.

Examination of the Dynaflect data from the Arizona subgrade test sites revealed three significant findings.

- With only a few isolated exceptions, the modulus values (E_{d_1}, E_{d_2}, E_{d_3}, etc.) calculated from the deflections became increasingly greater as each more distant sensor was considered.

- For some of the sensor stations, the moduli derived from the first two sensor readings (E_{d_1} and E_{d_2}) correlated fairly well with E_s. At others, they were much higher than E_s. At no location were E_{d_1} and E_{d_2} consistently lower than E_s.

- At all five test locations, scatter in the readings hinder the analyses, but not to any greater degree than was the case with plate bearing test data. This is surely due to the inhomogeneity of the subgrades and their stress fields.

It was somewhat surprising to find that first sensor deflections at the Rainbow Valley near surface where E_s = 6000-10,000 psi were smaller (hence, greater E) than in the subsurface excavation where E_s = 12,000-25,000 psi. A careful look at the deflection bowls, however, revealed significant differences in shape of the deflection basins. Scrivner and the other Dynaflect researchers have used two index parameters, the Base Curvature Index (BCI) and Surface Curvature Index (SCI) to aid in their analyses of deflection bowls (see Figure 66).

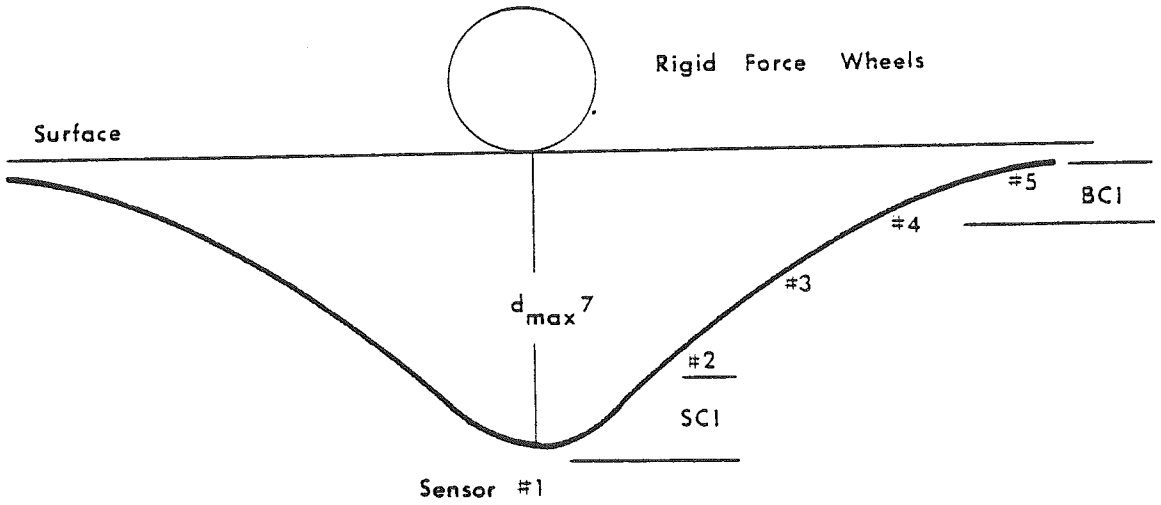


Figure 66. Deflection Basin Configuration for Dynaflect (After Majidzadeh. Ref. 100)

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The three Dynaflect researchers assign as the major cause for differences in shapes of deflection bowls, the differences in stiffness of materials in layered systems. This is undoubtedly true for pavements, but the authors of this paper have concluded that where the Dynaflect sensors are placed on soil, the most important variable influencing the shape of deflection bowls is internal damping of the vibrations within the soil.

Damping in soils is a very complex phenomena and is an important new area of research. The best summarization of work on the subject is the Seed and Idriss (46) paper. For sands, Hardin and Drnevich reported shear strain, mean principal stresses, void ratio, and frequency as the most important variables influencing degree of damping. Seed stated that factors affecting damping in wet clays are very difficult to determine. Obviously, the degree of saturation is important, but the role of in-situ stress fields is unknown.

One can only speculate concerning the factors that influence damping in blocky cemented or desiccated soils such as are common in Arizona. Analysis of the variations of the five sensor readings from the Dynaflect provides an insight into the problem.

For a subgrade which is truely elastic, homogeneous and isotropic, it would be expected that $E_{d_1}=E_{d_2}=E_{d_3}=E_{d_4}=E_{d_5}$. It was noted, however, that in most cases $E_{d_1}<E_{d_2}<E_{d_3}<E_{d_4}<E_{d_5}$, and to a considerable

degree, this is because of damping. (If the non-agreement of E_{d_1} values were primarily the result of random variations in subgrade stiffness, then there would be a random scatter of E_{d_1} values.) It was noted, however, that for most Dynaflect arrays where E_{d_1} and E_{d_2} were very close, these moduli were also within the range of E_s (from the 30 inch plate at 30 to 50 psi). Where E_{d_2} is much greater than E_{d_1}, some reduction was desired. Additionally, for arrays where E_{d_3}, E_{d_4}, and E_{d_5} were close, such as was the case for the Rainbow Valley near surface tests, considerable reduction was necessary to bring compliance with E_s.

Accordingly, the authors have defined two index parameters similar to those widely used by Scrivner, Vaswani and Majidezah. These are the Elastic Compliance Index (ECI) which is an evaluation of the relationship of the deflections at the nearest two sensors, and the Subgrade Damping Index (SDI) which is determined using the deflections from the three farthest sensors. These two indices are used to determine a reduction factor which is applied to E_{uncorr} - which is an average of E_{d_1} and E_{d_2}.

The Elastic Compliance Index is defined as:

$$ECI = \frac{E_{d_2}}{E_{d_1}} - 1$$

In only a handful of cases was this value negative and then the amount no more than .06. In such instances, the absolute value should be used.

The Subgrade Damping Index is equal to:

$$SDI = \frac{E_{d_3}/E_{d_1} + E_{d_4}/E_{d_1} + E_{d_5}/E_{d_1}}{3} - \frac{E_{d_3}}{E_{d_1}}$$

or;

$$\frac{-2\ E_{d_3} + E_{d_4} + E_{d_5}}{3\ E_{d_1}}$$

SDI is, therefore, an indicator of whether the vibratory loading is being felt 2 to 4 feet from the contact wheels or whether natural attenuation has reduced the expected deflections to some degree.

A low value of ECI indicates that within the stress range caused by the vibratory load, the soil is behaving according to the elastic model, and E_{uncorr} is close to E_s. On the other hand, a high value of ECI indicates that the energy of the Dynaflect loading is rapidly dissipating and damping must be taking place within the soil. Therefore, the E_{uncorr} is too small due to energy loss) and must be reduced if compliance with E_s is desired. It was found, however, that where higher values of SDI are calculated (continued damping beyond 2 feet is indicated), the need to reduce E_{uncorr} was not as great. On the other hand, where SDI is low or negative, a near elastic response is indicated, and a reduction factor based on the ECI was confirmed.

An analysis of all available Dynaflect data from the five stations resulted in the production of Figure 67. The use of this chart is straighforward. Entering with values of ECI and SDI from an array of

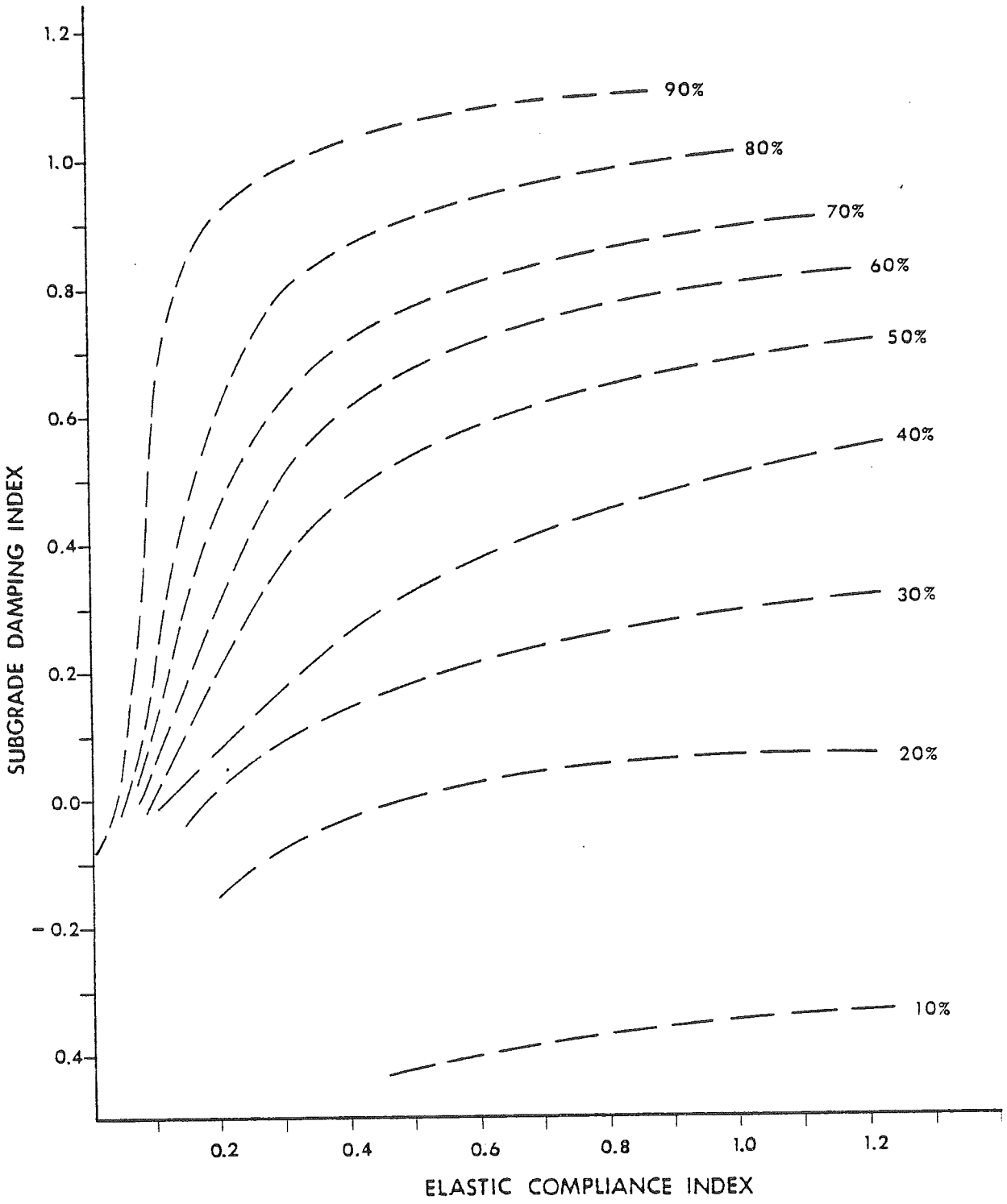


Figure 67. Chart Showing Suggested Reduction Factor to Relate E_{dynf} with E_s.

deflections, a reduction factor is determined which, when applied to E_{uncorr}, should bring it close to E_s.

An example problem is provided below.

Given: At Lake Pleasant On-Grade site - one set of deflections included: d_1-d_2-d_3-d_4-d_5 = 1079-568-265-139-85 (mils x 10^3)

From the Ahlvin and Ulery equation:

$P = 500$ lbs.

$$p = \frac{P}{A} = 500 \text{ lbs}/\pi(1.12^2) = 127 \text{ psi}$$

$\nu = 0.3$

$$E = pa\,\frac{(1-\nu^2)}{\rho/2} \quad H = \frac{258.8\ H}{\rho}$$

$$E_{d_1} = \frac{(258.8)\ (.111)}{(.001079} = 26{,}631$$

$$E_{d_2} = \frac{(258.8)\ (.072)}{(.000568} = 32{,}815$$

$$E_{d_3} = \frac{(258.8)\ (.043)}{(.000265)} = 42{,}006$$

$$E_{d_4} = \frac{(258.8)\ (.029)}{(.000139)} = 54{,}010$$

$$E_{d_5} = \frac{(258.8)\ (.021)}{(.000085)} = 63.957$$

From these, $E_{uncorr} = \frac{26{,}631 + 32{,}815}{2} = 29{,}255$

$$ECI = \frac{32{,}815}{26{,}631} - 1 = 0.23$$

$$SDI = \frac{-2(42.0) + 54.0 + 63.9}{3(26.6)} = 0.42$$

From Figure 67, reduction factor = 62 percent

E_{dynf} = (0.62) (29,255) = 18,138 psi

This method is strictly empirical; it has no theoretical or mathematicl foundation. Nevertheless, when allowances are made for the natural variability in stiffness of subgrade, it is a reasonable means of converting Dynaflect readings taken on soil to E modulus values which are within the range of accuracy required for pavement designs.

The success of this adjustment technique will be discussed with respect to the individual sites in section 3.5.2.

Regarding evaluation of the Dynaflect, the practice of using a light vibrator which generates small deflections to simulate the response of a larger load is often questioned. Advocates of the use of the Dynaflect and Road Rater have often assumed that the vibratory loads they generate elicit the same response as would the 9000 pound standard wheel loading. The modulus calculations presented herein seem to bear this out. The very fact that although the maximum Dynaflect deflections are less than .002", but yet the back calculated E modulus values are in the same range as those produced by plate bearing tests in the 0.05" to 0.1" deflection range, strongly supports the contention that nondestructive vibratory testing of this type can be of significant value.

3.5.2 Discussion of Dynaflect Moduli

Table 14 is a tabular listing of all Dynaflect data taken in connection with this project, including the

Table 14
Summary of Dynaflect Testing

No.	E(uncorr.) (psi)	Possion's Ratio	Dynaflect Readings (Mils x 10^3) w_1, w_2, w_3, w_4, w_5	ECI	SDI	Reduction Factor	E_{dynf} (psi)	Probable E_s based on (30" p.b.t. @ 30-50 psi)
I-10 Buckeye Site								
6074+50	18,800	.28	1931-802-307-154-92	.56	.79	70%	13,200	20,000
6074+00	19,700	.28	1842-772-357-190-128	.54	.41	42%	8,300	20,000
6075+30	21,800	.27	2080-673-248-104-65	1.00	1.59	100%	21,800	25,000
6073+00	26,800	.27	1278-584-337-226-154	.41	.03	23%	6,200	25,000
6073+50	36,600	.17	980-426-267-184-131	.49	.01	21%	7,700	25,000
6075+00	40,100	.17	1486-327-160-89-68	1.94	.43	35%	14,000	25,000
Fanglomerate Cut (Lake Pleasant)								
+30	10,500	.20	2649-1987-937-511-284	.13	.30	75%	7,900	20,000
+10	13,700	.20	2157-1419-766-426-216	.01	.34	100%	13,700	20,000
+50	22,200	.20	1788-700-255-111-66	.65	1.30	95%	21,100	20,000
+40	24,300	.20	1192-823-445-246-133	.06	.29	92%	22,300	20,000
+20	28,700	.20	1277-568-255-153-91	.31	.45	56%	16,100	20,000
On-Grade Site - (Lake Pleasant)								
+30	23,100	.30	1561-653-265-111-57	.55	1.43	100%	23,100	18,000
+20	26,100	.30	1135-672-322-176-102	.09	.35	85%	22,200	18,000
+40	28,100	.30	1135-587-265-131-82	.25	.52	68%	19,100	18,000
+10	29,300	.30	1079-568-265-139-85	.23	.42	62%	18,100	18,000
+50	41,400	.30	795-388-210-114-82	.32	.24	44%	18,200	18,000
Orange Site (Lake Pleasant)								
+20	14,900	.30	2044-1135-539-246-111	.16	.90	92%	13,700	18,000
+50	16,800	.30	1987-937-322-111-49	.37	2.52	100%	16,800	18,000
+30	17,700	.30	2015-851-312-131-91	.53	1.06	90%	15,900	18,000
+10	18,400	.30	1816-851-284-125-65	.38	1.37	100%	18,400	18,000
+40	21,400	.30	1618-719-284-116-75	.45	1.10	92%	19,700	18,000
Cienega Creek Site								
	19,900	.33	1513-837-333-161-71	.17	.98	93%	18,500	14,000*
	23,600	.33	1287-697-297-97-40	.19	1.99	100%	23,600	14,000
	24,200	.33	1223-697-322-167-76	.13	.67	88%	21,300	14,000
	25,300	.33	1094-708-322-174-73	.00	.61	100%	25,300	14,000
	25,600	.33	1030-740-386-232-103	.09	.32	86%	22,000	14,000
	27,000	.33	1030-665-311-174-84	.00	.43	100%	27,000	14,000
Rainbow Valley Site - Near Surface								
15	26,300	.30	1229-621-311-136-105	.28	.50	62%	16,300	10,000
9	30,300	.30	1164-505-272-142-70	.49	.65	58%	17,600	7,000
13	30,300	.30	1339-466-239-149-85	.86	.32	33%	10,000	6,000
10	32,900	.30	1242-427-259-129-101	.88	.37	36%	11,800	7,000
8	33,600	.30	1164-427-272-136-140	.76	.16	25%	8,400	7,000
12	32,400	.30	1048-485-226-134-78	.40	.33	42%	13,600	6,000
14	31,200	.30	1456-427-213-128-78	1.21	.40	35%	10,900	10,000

* E_s = 14,000 Using 30" Plates E = 24,000-30,000 Using 24" Plates

Table 14 (cont.)

No.	E(uncorr.) (psi)	Possion's Ratio	Dynaflect Readings (Mils x 10^3) w_1, w_2, w_3, w_4, w_5	ECI	SDI	Reduction Factor	E_{dynf} (psi)	Probable E_s based on (30" p.b.t. @ 30-50 psi)
			Rainbow Valley Site - Near Surface (cont.)					
7	37,000	.30	1087-382-226-136-113	.84	.06	21%	7,800	7,500
11	33,600	.30	1164-427-181-134-82	.76	.00	18%	6,000	6,000
32	35,500	.30	996-430-208-144-96	.50	.01	20%	7,100	8,000
36	33,600	.30	1022-465-190-141-102	.42	-.12	18%	6,000	6,000
37	42,000	.30	712-415-257-165-125	.11	.01	45%	18,900	8,000
42	40,500	.30	776-411-219-133-105	.22	.05	30%	12,100	8,000
39	39,000	.30	791-435-198-146-79	.17	.07	40%	15,600	8,000
33	40,900	.30	807-393-196-155-124	.33	-.19	17%	7,000	7,000
38	44,600	.30	751-356-217-145-115	.36	-.03	20%	8,900	8,000
6	44,200	.30	893-323-149-120-70	.79	-.09	17%	7,500	7,000
31	45,900	.30	801-325-156-125-70	.59	-.04	18%	8,200	8,000
5	46,200	.30	796-323-123-95-60	.59	-.10	17%	7,800	7,500
4	52,300	.30	679-291-142-107-66	.51	-.03	19%	9,900	7,500
35	55,300	.30	766-249-144-124-84	.99	-.26	12%	6,600	8,000
34	54,500	.30	727-262-138-120-77	.79	-.23	13%	7,100	8,000
41	54,400	.30	716-265-133-121-82	.75	-.32	11%	6,000	7,000
40	58,600	.30	593-264-138-113-77	.45	-.16	16%	9,400	7,000
			Rainbow Valley - Excavation					
28	16,100	.20	1757-1133-430-215-152	.00	.38	100%	16,100	18,000
26	18,100	.20	1362-1015-508-260-117	.00	.57	100%	18,100	18,000
25	20,400	.20	1497-840-358-156-141	.15	.42	78%	15,900	15,000
27	21,500	.20	1543-742-391-221-139	.34	.28	44%	9,500	23,000
30	23,400	.20	1367-703-404-182-86	.26	.78	83%	19,400	25,000
20	26,100	.20	1087-699-511-265-123	.00	.36	100%	26,100	24,000
21	24,200	.20	1203-737-388-207-147	.05	.22	92%	22,300	23,000
29	24,600	.20	1308-664-397-208-156	.27	.22	45%	11,100	25,000
19	31,000	.20	1009-543-408-207-116	.20	.33	62%	19,200	25,000
18	33,800	.20	951-485-369-201-116	.27	.26	47%	15,900	28,000
23	28,800	.20	1436-485-239-116-105	.92	.38	36%	10,400	12,000
24	37,100	.20	854-446-336-181-132	.24	.16	43%	16,000	12,000
17	40,700	.20	776-408-323-188-155	.23	.05	30%	12,200	26,000
16	36,100	.20	912-446-226-129-111	.32	.09	29%	10,500	20,000
22	47,400	.20	640-362-194-144-87	.14	.00	34%	16,100	20,000
			McCormick Ranch Fill Site					
3	27,500	.30	1233-571-282-130-64	.40	.91	82%	22,500	18,000
8	29,100	.30	1094-566-253-134-74	.25	.52	68%	19,800	18,000
14	34,800	.30	946-462-249-128-70	.32	.51	57%	19,800	18,000
11	32,300	.30	865-576-214-122-60	.02	.48	98%	31,600	18,000
5	37,100	.30	657-589-272-143-73	.27	.34	51%	18,900	18,000
4	31,000	.30	997-544-189-127-66	.18	.27	63%	19,500	18,000
6	32,700	.30	1147-450-195-146-67	.65	.24	32%	10,500	18,000
13	32,400	.30	968-514-183-110-57	.22	.47	67%	21,700	18,000
1	36,800	.30	731-525-213-137-71	.09	.22	80%	29,400	18,000

Table 14 (cont.)

No.	E(uncorr.) (psi)	Possion's Ratio	Dynaflect Readings (Mils x 10^3) w_1, w_2, w_3, w_4, w_5	ECI	SDI	Reduction Factor	E_{dynf} (psi)	Probable E_s based on (30" p.b.t. @ 30-50 psi)
McCormick Ranch Fill Site (cont.)								
7	41,600	.30	585-525-195-117-97	.27	.04	28%	11,600	18,000
2	42,300	.30	617-472-190-128-69	.15	.14	55%	23,300	18,000
12	44,100	.30	621-429-185-106-53	.06	.38	92%	40,500	18,000
10	43,400	.30	643-429-177-115-58	.02	.24	98%	42,500	18,000
9	43,300	.30	759-372-149-74-56	.32	.43	52%	22,500	18,000

calculated values of ECI, SDI, E_{dynf} and comparisons with the probable E_s values as determined by the 30 inch plate bearing test at 30 to 50 psi.

It can be seen the empirical reduction technique worked very well for the Lake Pleasant On-Grade and Orange sites, and for the Rainbow Valley near surface tests. It worked fairly well at the McCormick Ranch fill, the Rainbow Valley excavation, and on the Fanglomerate Cut. Questions arise as a result of the I-10 Buckeye and Cienega Creek values, but there are explanations which appear satisfactory.

At the I-10 Buckeye site, five of the six values of E_{dynf} were much lower than E_s. However, as stated earlier, the Dynaflect readings and densities were performed on a morning following an infrequent rainfall, which produced higher than normal moisture contents. This appears to explain the lower values.

Another problem with using the Dynaflect was found in analysis; six arrays are too small a sample to provide a needed statistical comparison of data.

At the Fanglomerate Cut site, two of the five E_{dynf} values appear to be low, although the possibilities of local resonance and other unexplained responses to the vibratory loads are possible in this cut. As stated previously, the five arrays of data at the On-Grade and Orange sites are in good agreement with the E_s values from the plate bearing tests.

The Cienega Creek site is the only one where the corrected values of E_{dynf} were larger than the E_s values from the 30 inch plate bearing tests. It will be recalled, however, that at this site, the top foot of soil was very dense and dry compared to the clay from 1 to 5 feet. The 24 inch diameter plate bearing test, which was more indicative of the stiffness of the dry upper layer than the 30 inch test, produced a much higher modulus, and one that compared very favorably with E_{dynf}. If it is assumed the proposed correlation technique produces modulus values which apply mainly to the upper few inches of a subgrade, then the results at Cienega Creek can be considered successful.

At the Rainbow Valley site, 32 arrays were taken on the near surface where E_s was found to be in the range of 6000 to 10,000 psi. There was considerable scatter, but 75 percent of the corrected E_{dynf} values fell in this range, with a median value (50th percentile) of 8400 psi. Likewise in the subsurface excavation, where E_s ranged from 12,000 to 26,000 psi, the median of 15 E_{dynf} values was 16,000 psi. About one-half the arrays produced values of E_{dynf} which seemed low, but many of the other half were very close to E_s. A visual summary is presented in Figure 68.

The Dynaflect results from the McCormick Ranch fill (see Figure 69) were subject to the greatest amount of scatter. Since the refraction seismic testing showed no scatter, it must be concluded that the

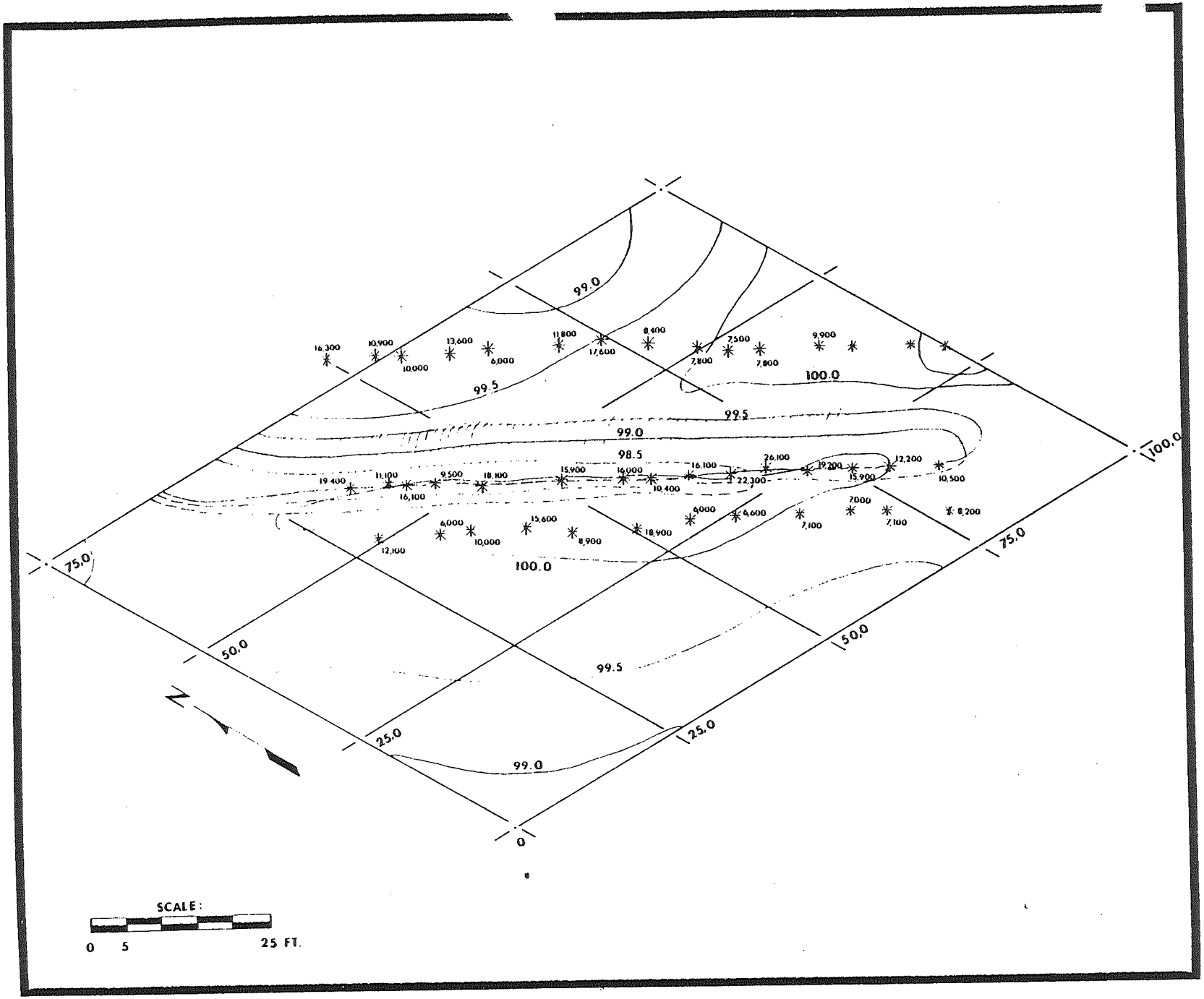


Figure 68. Visual Summary of Dynaflect Testing at Rainbow Valley Site

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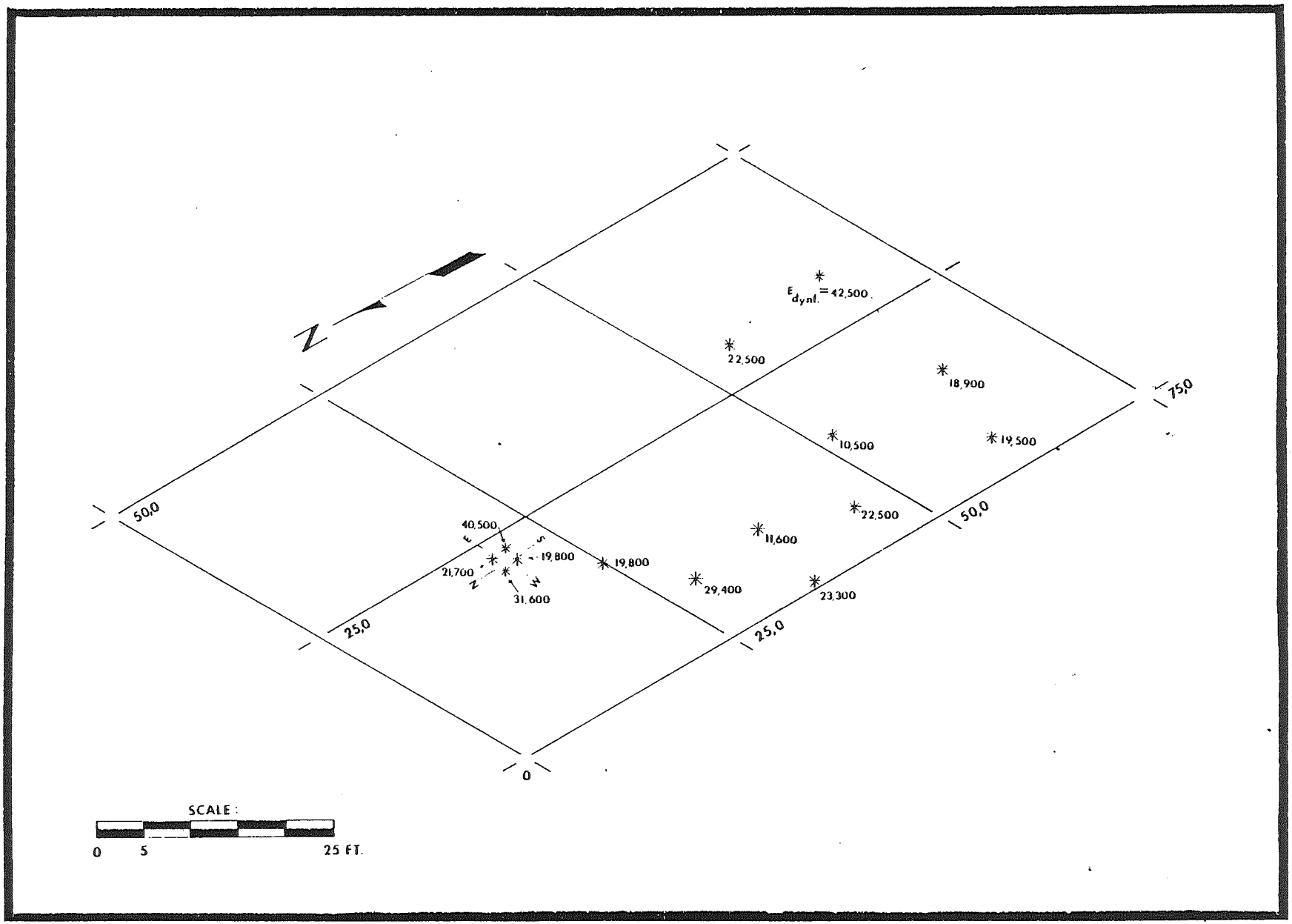


Figure 69. Visual Summary of Dynaflect Testing at McCormick Ranch Fill Site

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Dynaflect does not measure "mass response" of subgrades, but like the plate bearing tests, the deflections are highly influenced by differential cementation and local stress concentrations. Of the 14 arrays at this site, the median is 22,000 psi, which is somewhat higher than the average of four 30 inch plate bearing tests (18,000 psi), but compares very favorably with the E_{seis} (21,600 psi).

In conclusion, the elastic analysis and modulus reduction technique given in this section for Dynaflect data is recommended as a means of obtaining design values of E_s for use in pavement problems, provided an adequate number of arrays are available from a single design section to make a statistical judgment. It appears that 12 arrays should be considered the minimum for this purpose.

3.6 Results of Road Rater Testing

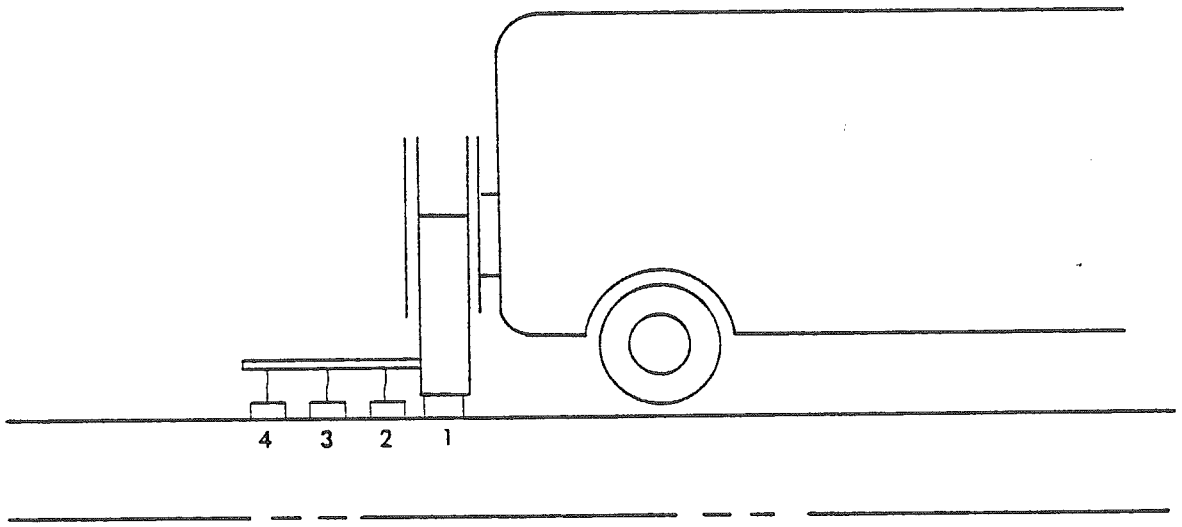
3.6.1 General

Nowhere in the literature could there be found references dealing with the calculation of E modulus of soils using Road Rater deflection bowls. (It is understood the Kentucky Department of Transportation will publish research dealing with this subject in the near future.) The authors of this report, therefore, set out to develop an empirical procedure similar to that used with the Dynaflect based on the Ahlvin and Ulery expression.

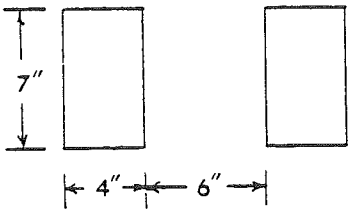
Basically, the Dynaflect and Road Rater are similar in function and in the types of subgrade response that they might be expected to induce. The major differences are in the geometry of the loading area and in the nature of the induced load. The Dynaflect operates only at 8 Hz and imparts a dynamic load of 1000 pounds. The model of Road Rater used for this project operates at discrete frequencies of 10, 20, 25, 30, and 40 Hz, but the dynamic loads (a function of frequency and piston displacement) are mostly smaller.

The application of the Ahlvin and Ulery analysis is shown in Figure 70. It will be found the adoption of a circular wheel print (instead of the rectangular) in the model causes problems with respect to analysis of the first sensor deflection. The model performs well for the other sensors, however. As was the case with the Dynaflect, each Road Rater deflection value rr_1, rr_2, rr_3, and rr_4 along with its appropriate influence factor H is inserted into the Ahlvin and Ulery expression for vertical deflection and values of E_{rr_1}, E_{rr_2}, E_{rr_3}, or E_{rr_4} back calculated.

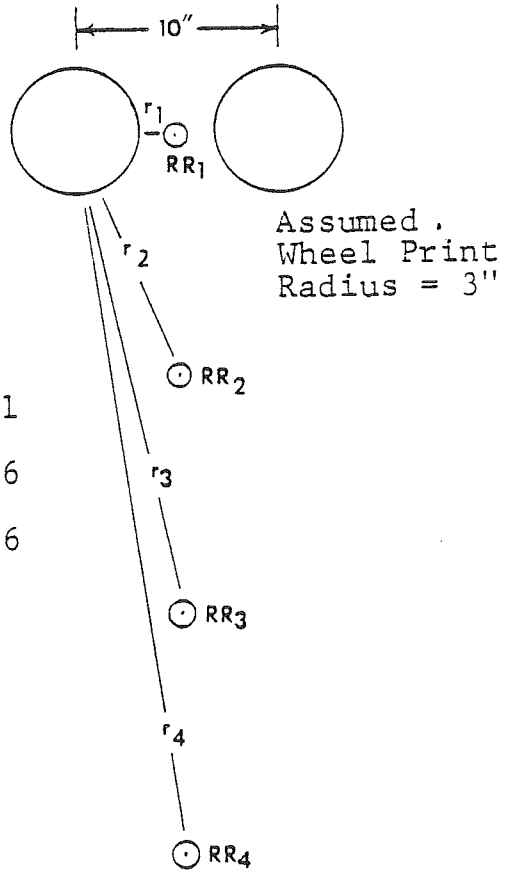
A complete copy of the LaBelle report of the Road Rater testing is produced in the Appendix. It can be seen that at nearly every test location at the Phase I test stations, an attempt was made to place the Road Rater at precisely the same spots as the Dynaflect. Road Rater deflections were induced at 10, 20, 25, 30, and 40 Hz, and, in many cases, more than one value of dynamic load was tried at a given



Actual Configuration



Assumed Configuration



Assumed Wheel Print Radius = 3"

Adaptation of the Alhvin & Ulery Equation

$$E = \frac{Pa}{A}\left(\frac{1 - \nu^2}{\rho}\right) I$$

r_1 = 5.0"	r/a = 1.67	I = 0.6468
r_2 = 13.0"	r/a = 4.33	I = 0.23481
r_3 = 24.5"	r/a = 8.1	I = 0.12576
r_4 = 36.3"	r/a = 12.1	I = 0.08346

Figure 70. Road Rater Configuration

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frequency. Arrays of deflections were then converted to modulus values. As with the Dynaflect, the authors sought an empirical technique to arrive at E_{rr} (Road Rater subgrade modulus) values which compared favorably with E_s derived from the 30 inch plate bearing tests.

It was found that the arrays of moduli from the 20 Hz deflection bowls were the most adaptable for developing a realistic correlation procedure. The peak to peak dynamic load at this frequency is only 327 pounds (at 0.05" displacement), but apparently the greater frequency provides an amount of energy to the subgrade which is comparable to the Dynaflect (and appropriate for a 9000 pound wheel load).

Table 15 is a tabular listing of all Road Rater 30 Hz - 327 pound deflection bowls produced at the three Phase I test sites. Included in the table are calculated values of modulus.

During the attempts to develop E_{rr}/E_s correlations, two factors were noted. The first sensor deflection, rr_1, was often unreasonable (and usually high, resulting in a low E modulus). Possibly this is due to the inadequacy of the model (circular loading area rather than the actual rectangular) in that the superimposition of vibrations so close to the source resulted in exaggerated deflections. With E_{rr_2}, E_{rr_3}, and E_{rr_4} values, there was usually a

Table 15
Summary of Road Rater Testing

Station	Poisson's Ratio	Road Rater Deflections (Mils x 10^3)	E_{RR_1}-E_{RR_2}-E_{RR_3}-E_{RR_4}	E_{RR}	Probable E_s based on 30" p.b.t. @ 30-50 psi)
I-10 Buckeye Site					
6073+00	.28	216-66-24-6	9600-11400-16800-44600	14,100	20,000
6073+00	.28	140-36-10-6	14800-20900-40300-44600	30,600	20,000
6073+50	.28	132-40-16-5	15800-18900-25400-53900	22,100	25,000
6073+50	.28	154-51-24-8	13500-14800-16900-33600	15,800	25,000
6074+00	.30	156-40-20-8	13100-18600-19900-33000	19,200	25,000
6074+00	.30	152-44-20-8	13500-16900-19900-33000	18,400	25,000
6074+50	.30	116-48-24-10	17700-15500-16600-26400	16,000	25,000
6074+50	.30	90-33-20-10	22700-22500-19900-26400	21,200	25,000
6075+00	.17	114-34-18-6	19200-23300-23600-47000	23,400	25,000
6075+00	.17	78-24-15-4	28000-33000-28300-70500	30,600	25,000
6075+30	.17	78-30-10-10	28000-26400-42500-28200	34,400	25,000
6075+30	.17	70-20-20-10	31200-39700-21200-28200	30,400	25,000
Fanglomerate Cut Site (Lake Pleasant)					
	.20	136-60-36-22	15900-13100-11700-12700	12,300	20,000
	.20	180-60-36-24	12000-13100-11700-11600	12,300	20,000
	.20	112-54-20-20	19300-14500-21000-13900	17,700	20,000
	.20	80-40-24-12	27000-19600-17500-23200	18,500	20,000
Orange Site (Lake Pleasant)					
	.30	54-28-14-4	37900-26500-28400-66100	27,400	18,000
	.30	88-34-20-12	23300-21900-19900-22000	20,900	18,000
	.30	90-54-30-12	22700-13800-13300-22000	13,500	18,000
	.30	124-44-20-12	16500-16900-19900-22000	18,400	18,000
	.30	90-30-10-7	22800-24800-39800-37800	32,200	18,000
	.30	90-38-16-6	22800-19600-24900-44100	22,300	18,000
	.30	84-30-20-8	24400-24800-19900-33000	22,300	18,000
	.30	360-190-90-35	5700- 3900- 4400- 7600	N/A	18,000
Cienega Creek Site					
777+00	.33	80-47-18-12	25100-15500-21700-21600	18,600	14,000*
777+00	.33	116-44-20-12	17300-16500-19500-21600	18,000	14,000
777+25	.33	90-47-30-14	22300-15500-13000-18500	13,700	14,000
777+25	.33	70-36-16-8	28600-20200-24400-32400	22,200	14,000
777+50	.33	66-36-18-6	30400-20200-21700-43100	20,900	14,000
777+50	.33	88-34-20-10	22800-21400-19500-25900	20,400	14,000
777+00	.33	58-32-22-8	34600-22700-17700-32300	20,200	14,000
777+00	.33	53-26-18-6	37800-28000-21700-43100	24,800	14,000

* E_s = 14,000 Using 30" Plates E = 24,000-30,000 Using 24" Plates

tendency for modulus to increase with each successive sensor (indicating damping), but there were more exceptions to this than was the case with the Dynaflect.

It was at first hoped that a Subgrade Damping Index tied to the E_{rr_4} value would be useful, however, no consistent results could be achieved of this parameter, and the idea was dropped. It was finally decided that the best procedure for choosing E_{rr} was simply to average E_{rr_2} and E_{rr_3}. This is what has been tabulated in Table 15.

Admittedly, this analysis is not as elaborate or refined as was that given for Dynaflect data. In retrospect, it is clear that not enough data was available from the Phase I sites to make authoritative conclusions.

As for the application of Road Rater deflections at other combinations of loading and frequency, it is believed that correlation techniques may be possible, but that there will be many unavoidable difficulties.

At the 10 Hz loading, it was found that the dynamic load is only 82 pounds and deflections (particularly at sensors 3 and 4) were so small as to be probably unrepresentative. At 20 Hz, good correlations were obtained using data applied to a 327 pound load. Fair correlations were possible using a 458 pound dynamic load at 20 Hz and also 511 pounds at 25 Hz. With the 30 and 40 Hz loadings, it was found that

modulus values were usually unrealistically low and there was almost random scatter of values as one proceeded from the closest sensor to the farthest. This would seem to suggest that at these high frequencies, damping of the vibrations in soils is not a factor but that local resonance is. This is not surprising when one considers the great amount of energy being applied to the subgrade.

3.6.2 Discussion of Road Rater Moduli

At the I-10 Buckeye site, a median of 12 values of E_{rr} was 22,100 psi (the range was 14,100 to 34,400 psi), while the 30 inch plate bearing tests suggested E_s is in the range of 20,000 to 25,000 psi. The median value shows good correlation, but the broad range of E_{rr} values indicates that at any given site, a large sample of arrays is essential in order to obtain a statistically sound design value of modulus.

At the Fanglomerate Cut, but four values are available; the median being 17,700 psi. This is slightly lower than expected (as were the Dynaflect moduli), but because of the stress relief which appears present at this site, the E_{rr} is credible.

At the Orange site (Lake Pleasant), the median of nine E_{rr} values is 22,300. This is slightly higher than the expected value based on plate bearing ($E_s \simeq 18,000$). Again, a broad spread in the values is noted.

At Cienega Creek, the median of seven values is 20,200 psi which compares very well with the Dynaflect moduli. Comments in section 3.4.2 regarding the low frequency vibratory loads primarily affecting the response of the upper 6 to 12 inches of stiff compacted clay at this site are, therefore, reinforced.

3.7 Summary of Subgrade Load Response

3.7.1 General

Table 16 is a concise listing of the results of in-situ testing at the five sites, given in terms of subgrade modulus. It is clear that none of the methods employed in this project emerged as completely authoritative in the determination of E_s. It was stated in a previous section that the plate bearing test has been so widely used for many years to evaluate pavements and subgrades, and that it is the standard against which any other method must be judged. This feeling is also strongly expressed by Kasianchuk and Argue (138) of the Canadian Department of Transportation, and is the results of their many years of in-situ testing of airfields and highways. In an attempt to establish the usefulness of wave propagation techniques as a means of nondestructive testing, these authors state that any serious attempt to replace the plate bearing test as the primary tool of field measurement must display good correlation with existing plate loading methods.

Table 16

Summary of Subgrade Elastic Modulus Values Determined by In-Situ Testing

Test Location	E_s (from 30"plates @ 30-50)	E_s (from 12"&18" plates	E_{seis}	E=500x CBR	E_{dynf}	E_{RR}	Remarks
I-10 Buckeye γ_d=125pcf w/c=1½% N = 8-13	17,500-34,000psi	29,000-50,000psi	25,000-42,000psi	-	Range[1] 6000-21,800psi	Median 22,100psi Range 14,100-34,400psi	[1]At time of Dynaflect w/c=6-10%
Lake Pleasant Orange γ_d=112pcf w/c=2% N = 11	10,000psi[2]	54,000psi	27,000-33,000psi	-	Median(5)* 16,800psi Range 13,000-20,000psi	Median 22,300psi -	[2]E_s from single test believed to be low.
Lake Pleasant On-Grade γ_d=116pcf w/c=2% N = 8	15,000psi[2]	57,000psi	-	-	Median 19,100psi Range 18,100-23,100psi	-	*Refers to number of values used to determine median
Lake Pleasant Fanglomerate Cut γ_d=119pcf w/c=1% N = 50/1"	20,000psi[3]	25,000psi	68,000psi	-	Median(5) 16,100psi Range 7900-22,300psi	Median 17,700psi	[3]Estimate based on 18"&24" tests.
Cienega Creek γ_d=135pcf w/c=4% N = 14	14,000psi	39,000-50,000psi	53,000-67,000psi	-	Median 22,000psi Range 18,500-27,000psi	Median 20,200psi	
Rainbow Valley (near surface) γ_d=80-100pcf w/c=1½% N = 1-4	6000-10,000psi	4000-35,000psi	16,000-41,000psi	6000-10,000psi	Median(32) 8400psi Range 6000-18,900psi	-	"Probably applies to cemented soil below 1.0 foot.
Rainbow Valley (excavation) γ_d=103-112 w/c=1½% N = 7-22	12,000-26,000psi	40,000-100,000psi	26,000-78,000psi	22,500 28,500psi	Median(15) 16,000psi Range 9500-26,100psi	-	
McCormick Ranch (fill) γ_d=90-100pcf w/c=2% N = 3	13,500-20,000psi	8000-18,500psi	21,700psi	-	-	-	
McCormick Ranch (fill) γ_d=115pcf w/c=3% N = 10	15,000-21,000psi	9000-23,000psi	21,300psi	11,500-30,000 Median(10) 20,500	Median(14) 22,000psi Range 11,600-42,500psi	-	

Comments concerning the comparison of subgrade modulus, E_s, with the results of other tests are given in the following section. The 30 inch plate with a loading of 30 to 50 psi has been designated as the reference modulus for the design of highway pavements. Smaller plates, such as the 12 inch, have been found to be too greatly affected by the local zones of stiffness and are, therefore, not indicative of mass response for many Arizona subgrades. The use of 30 to 50 psi as the applicable stress zone for design is believed to be conservative. Depending on the thicknesses and moduli of the bituminous bound and granular base layers, the pressure resulting from the 70 to 80 psi tire print which is imposed at the top of the subgrade should be in this range (or lower). For any other design problem (such as an airfield pavement, concrete pavement, footings, or slab foundations), a specially tailored approach should be employed to select a design value of modulus.

3.7.2 Correlation of E_s With In-Situ Density

Correlation was poor to nonexistent at all sites, as was expected. In dry desert soils, stiffness is largely independent of density or void ratio and is mostly a function of cementation or other interparticle bonds such as those caused by desiccation.(In uncemented sands and gravels, however, E_s is known to be a function of void ratio and relative density.)

Nevertheless, density is important when evaluating the potential for loss of modulus by a soil upon saturation. Most of the problem "collapsing" soils of the desert Southwest are of low density, because they have never had a chance to consolidate under their own weight. This was shown by the saturated plate bearing tests at the three sites where they were performed. For example,

Rainbow Valley	100-105 pcf	E_s(sat) = 10-30% E_s(dry)
McCormick Ranch Desert	90-100 pcf	E_s(sat) = 10-30% E_s(dry)
Cienega Creek	135 pcf	E_s(sat) = 50-80% E_s(dry)
McCormick Ranch Fill	115 pcf	E_s(sat) = 20-40% E_s(dry)

Density also seems to play a role in the damping of vibrations from the Dynaflect and Road Rater and in soil resonance associated with the Road Rater.

3.7.3 Correlation of E_s With Standard Penetration Number

Such correlations are generally unreliable. Obviously, greater moduli would be expected in soils with high standard penetration resistance, but, in reality, N appears to be a function of dynamic shear strength rather than stiffness.

Figure 71 shows graphically the relationship found at the five test sites between N and E_s. A general correlation is given, but many findings fall outside the limits of this relationship. For instance, at the Fanglomerate Cut, no penetration was possible

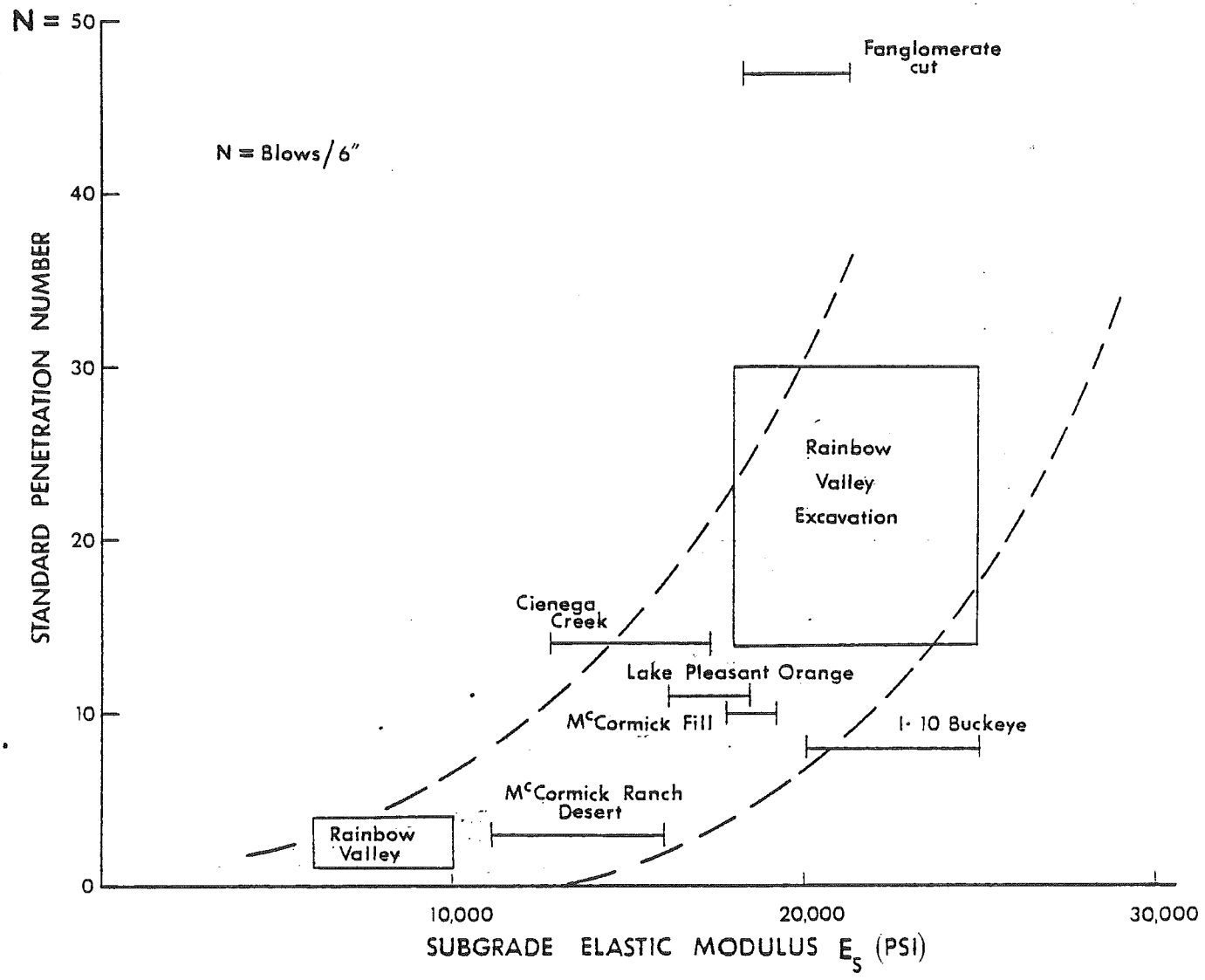


Figure 71. Relationship Between Standard Penetration Numbers and E_s at Test Sites

due to very strong cementation; yet because of the stress relief situation, a moderate elastic response was noted.

3.7.4 Correlation of E_s with E_{seis}

It was known in advance that a reduction factor would need to be applied to E_{seis} to bring the value into the range of E_s. Table 17 is a comparison of E_s and E_{seis} for the five sites. It would appear that for dry soils in Arizona, the reduction factor required to convert E_{seis} to E_s may range from 0.30 to 0.85.

Table 17

Correlation of E_s and E_{seis} at Test Locations

Site	E_s	E_{seis}	Required Reduction
I-10 Buckeye	20,000-25,000 psi	25,000-42,000 psi	0.60-0.80
Lake Pleasant Orange	18,000	27,000-33,000	0.60-0.80
Fanglomerate Cut	20,000	68,000	0.30
Cienega Creek	14,000-20,000	53,000-67,000	0.30
Rainbow Valley (surface)	6000-10,000	N/A	
Rainbow Valley (excavation)	18,000-25,000	26,000-78,000	0.30-0.70
McCormick Fill	16,000-20,000	21,300	0.75-0.95

It is believed that density is a key to the selection of the reduction factor. For very dense soils, such as the Fanglomerate Cut, Cienega Creek, and portions

of the Rainbow Valley excavation, high compression wave velocities were noted, resulting in higher moduli, with greater reduction required. In the less dense (but still firm) soils such as the McCormick Ranch fill or the Lake Pleasant Orange site, smaller reductions were indicated ($E_s = 0.80\ E_{seis}$). It is emphasized that these reductions apply only to soils in a condition of low moisture (S below about 30%). Near saturation would probably cause E_s to fall more dramatically than would E_{seis} and thus greater reductions would be mandated.

3.7.5 Correlation of E_s with CBR

The relationship given in this report that E_s = 500 x CBR in psi is based on empirical evidence. This ratio is much lower than usually reported (and only one-third the widely accepted E = 1500 x CBR). This study deals with in-situ tests on very dry subgrades; however, while the Heukelom and Foster relationship primarily applies to the laboratory test (with the soil in a compaction mold) using soaked material. For reasons which will be explained in Section IV, the authors believe that in dealing with subgrades in a desert environment, it is best to determine subgrade modulus under dry conditions and then apply corrections for moisture increase.

3.7.6 Correlation of E_s With Dynaflect and Road Rater Deflection Bowls

As was previously stated, the empirical techniques

given in this report for analyzing Dynaflect and Road Rater deflection basins are considered adequate for the predominant types of subgrades found in the state of Arizona. It is unlikely, however, that in their present form they would have universal application if applied to dissimilar soils throughout the world. Problems with the empirical analyses presented herein are partly due to the complex geometric arrangement of the loading areas of both machines, but are believed to be mainly due to the fact that damping and resonance in soils are very poorly understood. As is the case with in-situ stress fields, properties affecting damping or resonance in soils are undoubtedly highly anisotropic and vary spatially to a considerable degree in many soils. Scatter in the data will be an ever-present problem in the use of induced vibratory deflections to characterize soil response. Resonance will create an indication of lower than appropriate moduli while damping will produce subgrade moduli which appear to be greater.

3.7.7 Summary of Testing

The results of field testing at the five sites were very revealing but not entirely conclusive. At the root of the problem is the realization that no test, no matter how sophisticated or carefully performed, precisely duplicates the loading situation to which a pavement subgrade will be subjected over its design life. A second obstacle which is almost as great is the wide spatial variation in the response of subgrades over very short distances. The results

Table 18

Summary of Advantages and Disadvantages of In-Situ Testing Methods

Advantage	Disadvantage
Plate Bearing Tests	
•Test is straightforward, easy to run and understand •Geometry is uncomplicated; calculation of E_s can be made in accordance with a widely accepted methodology •Each test reveals E_s as a function of stress for the range of stresses applied.	•Time required to run each test (2-3 hours for ASTM D-1195, 4-6 hours for D-1196, longer if secondary compression is significant factor) •Once started a test cannot be interupted •Deflection readings are affected by passing traffic •Each test provides only one element of data for statistical analysis •The response of smaller plates is strongly influenced by heterogeneity of the subgrade, especially where large gravel, cobble, or blocky cemented materials are involved
Refraction Seismic	
•Definitive value of E modulus can be calculated from V_s and V_c; also Poisson's ratio •Many tests may be performed in a short time frame •Measures the mass response of soil, and is less affected by spatial variations in properties	•Modulus applies only to low strain levels (10^{-5} to 10^{-4}); must be reduced by some factor to the stress level of interest •Expensive equipment required to record V_s. Also requires expert interpretation of wave traces •Does not properly account for the loss in stiffness of a subgrade upon increase in degree of saturation
CBR	
•Test is widely accepted; can be performed by technicians, and equipment is available in nearly every agency •Inexpensive •Many tests can be performed in a day •Stiffness of the upper few inches is measured	•Contact area is small. Results may be influenced by an imbedded cobble, or blocky cemented matrix

Table 18 (cont.)

Dynaflect and Road Rater	
•Considerable data can be collected in a short time; therefore each particle of data has a low unit cost •Technique for conversion of deflection basin to modulus is easily adaptable to computer •Quickest way to collect a suite of values for statistical analysis	•The analysis of deflection bowls and calculation of modulus is empirical and imprecise •Deflection values are influenced by internal damping and resonance in soil, subjects which are poorly understood

of this field testing certainly reinforce Poehl and Scrivner's (124) findings in Texas mentioned earlier: that seasonal variations in the response of pavements in non-frost susceptible locations are not nearly as great as inherent spatial variations.

It is concluded that the plate bearing test should remain the primary reference for the measurement of subgrade response, and that correlative substitutes are available (in order of usefulness) in the field CBR, refraction seismic and Dynaflect. It is further concluded that in Arizona, the reference modulus of a subgrade (E_s) should apply to the subgrade in a condition of low moisture, to which reductions made necessary by moisture increase may be applied.

SECTION IV. APPLICATIONS, LIMITATIONS, AND IMPLEMENTATION OF RESULTS

4.1 Discussion of the Variability of Subgrade Modulus

4.1.1 General

It is clear from the findings presented in Section III that measured values of subgrade modulus are dependent on many variables, most of which are related to the testing procedure used to derive the parameter. In all cases a load is applied to the subgrade through some mechanism such as a load on a plate, a seismic wave, a vibratory pulse, etc. The expectation is that the charted response of the soil will provide insight as to the response of the prototype structure (subgrade in a layered pavement) during its operational loading situation. Thus a meaningful selection of E_s for design requires a keen awareness of the factors which produce variations in measured values of load response. In this section, a discussion of five independent variables which are of greatest concern in the pavement design problem is presented. These are: size of loaded area, stress (strain) level, lateral variations in stiffness, temperature and moisture effects, and load history. In each section our field results will be examined in the light of findings published by other researchers.

4.1.2 Variation in Subgrade Modulus Due to Size of Loaded Area

While engineering analyses treat E_s as if its value were independent of geometry, it has long been known that this is not the case. Since the 1930's, pioneers in the field of in-situ load testing (139) such as Housel, Palmer, Burmister, and McLeod have stated that the size of the loaded area greatly affects the deflection for a given stress level on the plate. As noted earlier, McLeod suggests that there is a straight-line relationship between unit support load at a given deflection level and perimeter/area ratio for plates ranging in diameter from 12 to 42 inches and possibly larger. (See Figure 24, section 2.1.2.) Burmister (140) notes that where small diameter plate tests are performed on the ground surface, edge punching shear effects are likely to cause unrepresentative displacements which dominate the settlement performance. As the size of the plate increases, however, edge shear and perimeter/area ratio become less important factors governing performance (see Figure 72). McLeod notes that as the size of a loaded area becomes much greater than 42 inches (as with a slab), the load response tends to become less dependent on size.

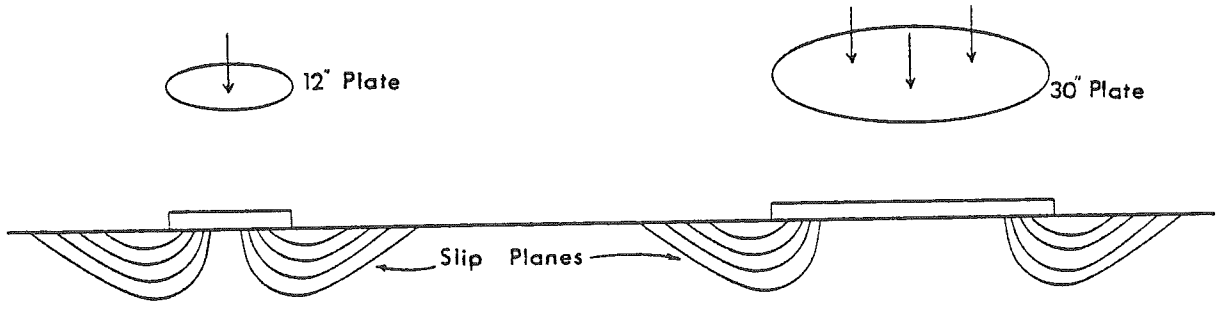


Figure 72. Effect of Plate Size on Edge Shear

In the example given by McLeod, which is a series of plate load tests from a Canadian airfield, the load deflection data resulted in moduli which increased slightly as plate size increased. This is not always the case, however. The experience at most of the test sites in Arizona, was that although the subgrades registered greater deflection per unit stress for large plates as did McLeod's, the degree of difference was so much greater as to result in moduli which decreased as plate size increased.

On the other hand, the findings of Housel and McLeod were confirmed by Soviet engineers Abelev and Rejtman (141) who concluded that when using the model of a semi-infinite solid for analysis of footings of large dimensions and lying on weak clayey soils, the modulus applied should be increased over that determined from a plate bearing test, due to bridging action.

Aside from the early published works of the aforementioned pioneers in load testing, there has been little said about plate size in the past several year. Barata (142) provides interesting reflections on the work of Housel and McLeod, in an attempt to modify their relationships to account for the increase in modulus with vertical confining pressure (i.e. depth). This is a topic which is very important when analyzing embedded footings, or end-bearing piers and is important to a lesser extent in pavement analysis. This is

because a plate load is transmitted to the supporting medium in the form of a pressure bulb whose size is approximately proportional to the size of the plate. Where layered subgrades are involved or where stiffness increases (or decreases) uniformly with depth in some fashion, the zone of influence must be considered in the analysis of plate bearing test data. Bauer et al (143) noted that to properly extrapolate the results of plate load tests, the actual thickness of soil undergoing deformation must be recognized and measured. The assumption of an infinite depth of homogeneous soil often leads to an over-estimation of the value of E modulus.

The Arizona subgrades examined in this research effort are different from those studied by most of the prior researchers in that they are much stiffer and drier. At all our test stations except the McCormick Ranch fill and desert sites, the small plates (12") indicated a greater stiffness (higher moduli) than did the larger plates.

At the McCormick Ranch fill and desert sites (the only locations where gravel and coarse sand were virtually absent) no differences in moduli could be seen based on plate size. Likewise, at the three locations where saturated tests were performed the size of loaded area was much less significant.

It appears that in dry and cemented subgrades, response of 12 inch and 18 inch diameter plates

is greatly influenced by the high stiffness of subgrade components, be they in the form of large gravel and cobble particles or blocky cemented matrices consisting of clay-sand-gravel mixtures. Support for this thesis is given by Dvorak (144) of Czechoslovakia, who reported that for plate bearing tests on coarse granular soils, meaningful data will be obtained only if the diameter of coarse particles is less than 0.3 times the diameter of the plate. It is likely that blocky cemented soils respond in a manner similar to that of large cobbles. Therefore below about 200 psi the load response is a function of interlocking and embedment strength of subgrade constituents, rather than an elastic response of a homogeneous mass. As larger plates are employed, a mass response of the subgrade is possible, as the size of the blocky constituents becomes a smaller fraction of the plate diameter.

This phenomena was most pronounced at the Orange and On-Grade sites (Lake Pleasant) and was very much in evidence at the I-10 Buckeye site and at Rainbow Valley, particularly in the trench where the spatial variation in cementation was very noticeable.

At the Rainbow Valley excavation, saturation of the cemented subgrade apparently broke enough bonds so that the "blocky matrix" phenomena was no longer operable and a mass response of the subgrade under 12 inch plates became possible. At the McCormick Ranch fill, and probably at Cienega Creek, saturation of the previously very desiccated fine

grained soil helped break the regime of high horizontal tensile stresses and allowed Burmister's edge punching shear mechanism to become a reality. The response was then more like those noted in the test section upon which the early researchers based their finding (which apparently were mostly soft resilient soils).

A pertinent question might be asked regarding plate sizes which can be successfully used in field testing programs. Since productivity is very important where large amounts of data must be collected, one might propose to use a 12 inch plate to chart a stress-deflection curve, and calculate modulus at some stress level beyond 30-50 psi where the effects of stiffer obstacles may be

Table 19
Correlation of Subgrade Modulus Values at Test Stations, Using 30" and 12" Plates

Site	E_s (30" Plate @ 30-50 psi)	Stress Level at Which E From 12" Plate is in the Range of E_s
I-10 Buckeye	20,000-25,000 psi	400 psi
Lake Pleasant Orange	12,000-18,000	200-300
Lake Pleasant	12,000-18,000	400-500
Cienega Creek	14,000	400-500
Rainbow Valley (surface)	6000-10,000	50-100
Rainbow Valley (excavation)	18,000-26,000	75-150
McCormick Ranch Fill	16,000-22,000	30-50

lessened. A look at Table 19 reveals that this is probably impractical. The level of stress resulting in a comparable range of E_s values for the 12 inch as for the 30 inch plates at 30-50 psi may be from 30 to 50 psi (hence no change) to over 500 psi. Keen engineering judgment would be required to select the appropriate range in a given situation.

In conclusion, it is recommended that for pavement design in Arizona, the 30 inch plate bearing test be adopted as the standard because:

- This is the closest to the size print a semi-trailer tandem-axle wheel will impose on the subgrade.
- Smaller plates are likely to be influenced by blocky or embedded constituents of the subgrade whose local stiffness is much greater than that of the subgrade mass.

4.1.3 Variations in Subgrade Modulus Due to Stress Level

At very low levels of stress, earth materials behave linear-elastically; that is, strain is directly proportional to stress and is completely recoverable. In this range of strains E modulus is a precise and descriptive index of response. As the stress level continues to increase, at some point plastic (nonrecoverable) as well as elastic deformation occurs in soil. The level at which plastic deformations begin and the proportion of strain which is recovered are internal, mass properties of the in-place material. The subjects of

repetitive loading and recoverable and nonrecoverable deformations will be examined in section 4.1.6. In this section we must deal with the fact that, all other variables affecting E_s being held constant, elastic modulus of soils is a function of stress (or strain) level.

Both our experience with desiccated, partially saturated, and cemented soils, as well as the experience of other researchers with classical soil types, indicate that E_s usually decreases with increasing levels of strain. This phenomena is known as "strain softening." The opposite effect, "strain hardening", has been noted in certain situations, especially where test conditions include lateral confinement.

There are actually two regimes of strain with which the analyst involved in field testing must be concerned. These are the elastic (seismic) range, which is about 10^{-6} in/in to 10^{-4} in/in and the elasto-plastic range, where strains are greater than about 10^{-4} in/in. In the former zone E_{seis} is an intrinsic material property (for all practical purposes), while in the latter E_s (referring to a secant modulus on a virgin stress-deflection curve) is a function of stress. For most geotechnical problems (other than dynamic analysis of foundations) the prototype loadings demand that an E_s beyond the elastic range be used.

Few investigators have compared E_{seis} with E_s beyond the elastic range on the basis of field evidence. It was previously mentioned that Seed

and Idriss (46) adapted research of Hardin and Drnevich in a form where the reduction in shear modulus (G) of sands of different relative densities based on strain level could be seen. They also presented a compilation of the findings of many researchers to provide a similar reduction scheme for saturated clays. These are given again as Figures 73a and 73b. This research is very important, as G can be directly related to E by knowing Poisson's ratio. A complication may exist if (as suspected) Poisson's ratio is itself a function of strain level. The writers believe that while Poisson's ratio does change in most soils as they undergo shear deformation, the reduction relationships given by Seed and Idriss (and given in this section) are so approximate and empirical as to be applicable to both E and G modulus.

Concerning E_s in the range of strains produced by plate bearing tests, Lo et al (145) found that for stiff fissured clays, tests using 12 inch plates on soil in deep test pits in the laboratory showed that as stress level increased, modulus decreased. Kersten (146) in Minnesota performed numerous plate bearing tests in the field using 12 inch plates. He found that for non-frozen plastic soils E modulus tended to decrease with stress level, while in granular soils, modulus increased.

When one compares the findings of these investigators as well as the McLeod and Housel data with the findings at the Arizona sites, there seems to be a question whether strain softening or strain

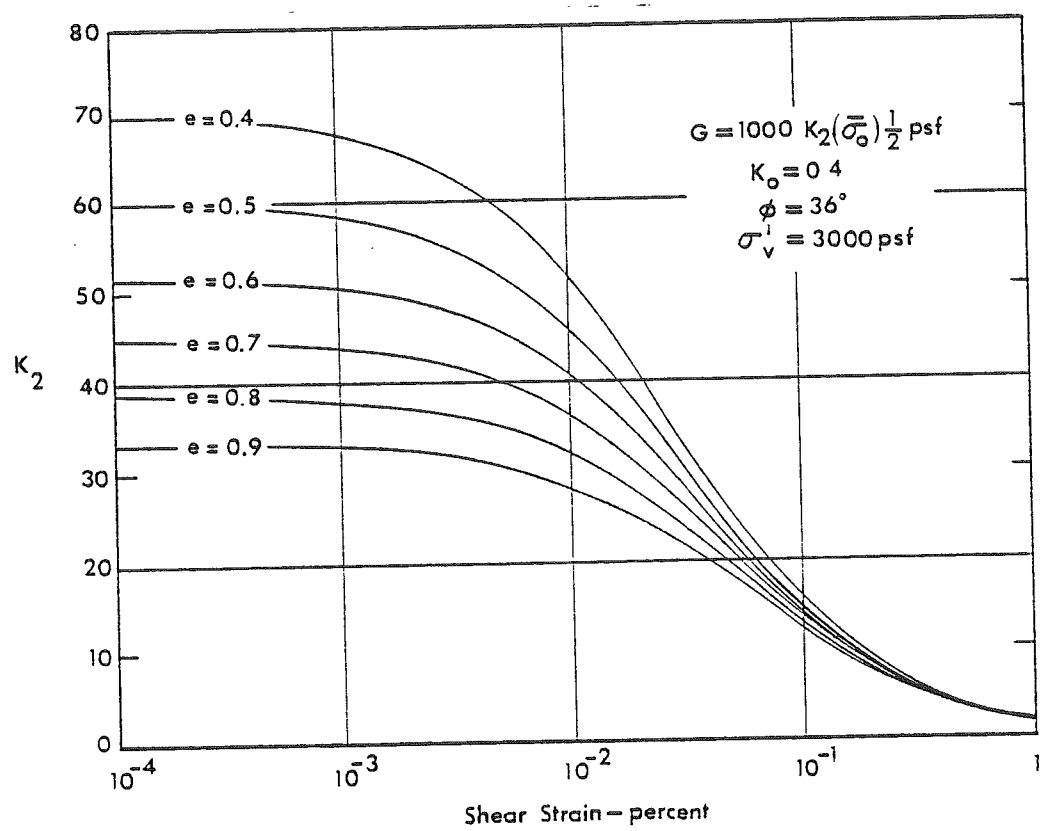


Figure 73a. Shear Moduli of Sands At Different Void Ratios (after Figure 46)

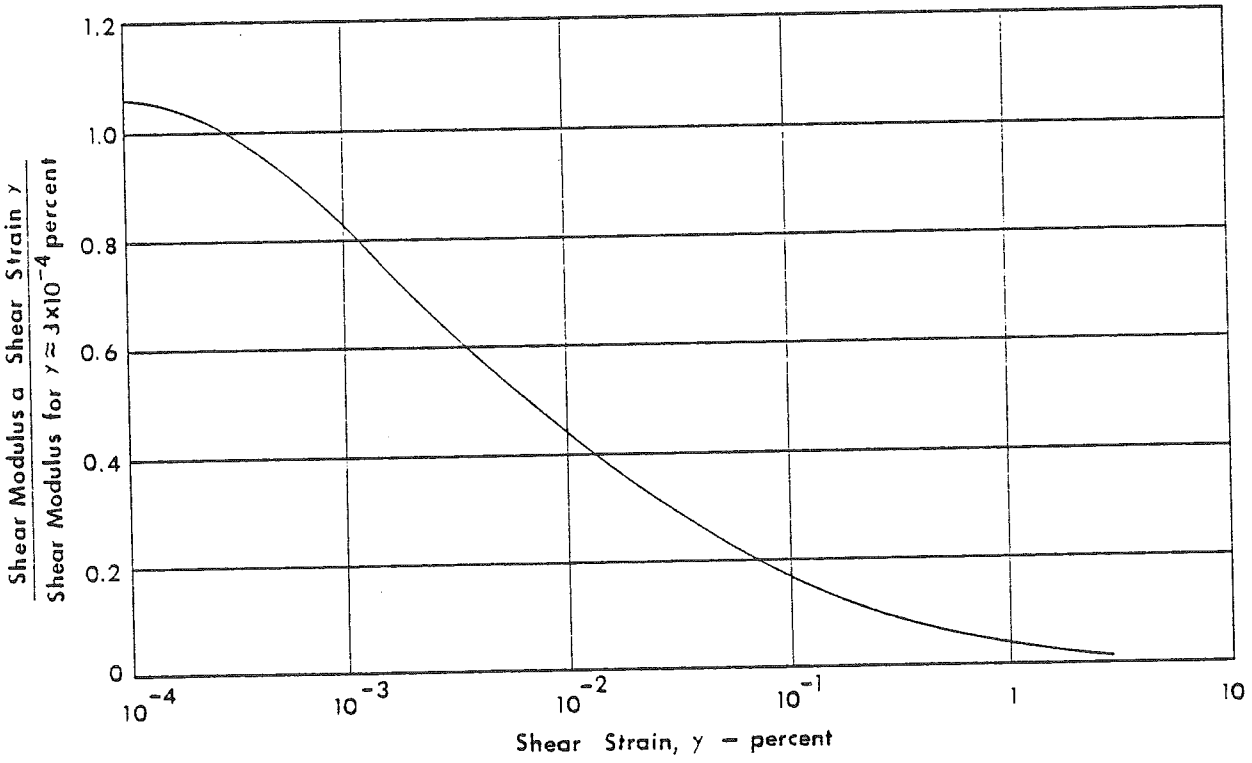


Figure 73b. Reduction of Shear Modulus with Shear Strain for Saturated Clays (after Figure 46)

hardening is the more common experience in dealing with loadings of in-place soils beyond the elastic range. In the data developed for this project, the only exceptions found to the strain softening syndrome involved the sections where stress relief or saturation of the subgrade was involved. At the Fanglomerate Cut and Rainbow Valley excavations, strain hardening played but a limited role. Once the micro-fissures and joints had been closed, the onset of strain softening could be noticed at higher stress levels. With the tests on saturated subgrades at the McCormick Ranch fill and Cienega Creek sites, the stress/modulus response was nearly linear. It appears that where the cementation or high tensile stresses in fine grained soils are broken down, they behave much like the silts and clays of high moisture and resilence upon which the studies of McLeod, Burmister, and Housel, are based.

To the pavement analyst, the question of dependence of modulus on stress level has importance in two areas. First, in using test data to select E_s, he must choose the stress range, as well as size of loaded area, which is applicable to his design prototype. This is why the range of 30-50 psi is recommended for highway pavements. Secondly, if the test data was derived using a strain level significantly different than will be associated with the prototype loading, an adjustment or extrapolation will be necessary. This is the purpose of the Seed and Idriss charts (Figures 73a and 73b).

Figure 74 is a graph showing an approximate relationship between the E_{seis} and E_s values from plate bearing testing at the Arizona test sites based on strain level. To plot this relationship, two assumptions are made.

- Deflection is related to strain in such a way that ρ = .001 inch (the threshold of Ames dial capability) is equivalent to $\varepsilon = 10^{-2}$ in/in. (In fact ε is related to ρ as a function of modulus and Poisson's ratio.)
- The seismically determined modulus applies to the strain range 10^{-6} to 10^{-4}.

According to Figure 74 there is a considerable range in appropriate reductions required at the Arizona sites. Generally it can be said that very dense and rock-like subgrade structures require the greatest reduction. Low density, fine grained, and weakly cemented subgrades show closer parity between E_s and E_{seis}.

4.1.4 Variation in Subgrade Modulus with Lateral Extent

The changes in response which are possible over very short distances were graphically shown by the results of the plate bearing, Dynaflect, and CBR testing at the Rainbow Valley and McCormick Ranch sites. Differences were expected at Rainbow Valley, since the undisturbed subgrade was known to contain variable degrees of lime cementation; but the spread of values at the controlled McCormick Ranch fill was greater than anticipated. The number of plate bearing tests taken at Phase I

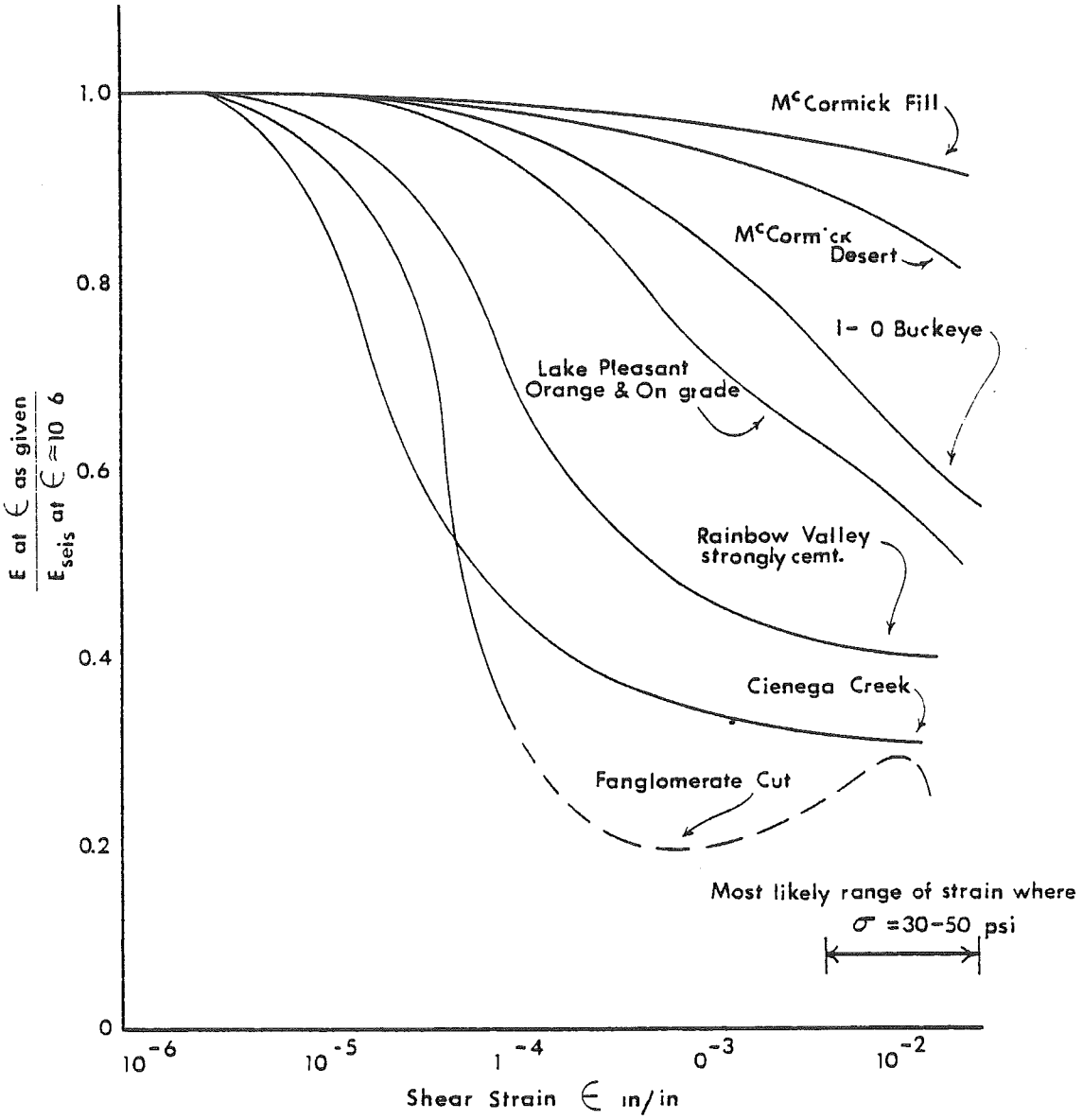


Figure 74. Reduction Required of E_{seis} Values Determined at Test Sites for Higher Strain Levels

sites was too small to evaluate lateral variations in stiffness, but Dynaflect data indicated the same potential for scatter as at the Phase II sites.

Very few studies have incorporated in a program of field testing, an attempt to bracket the range of measured stiffnesses possible in a material which is seemingly uniform. The Poehl and Scrivner (124) study using Dynaflect readings in Texas is most significant from our viewpoint. (See sections 2.1.3 and 3.7.7.) Meigh (147) in the Sixteenth Rankin Lecture presents a treasury of field and laboratory data in terms of the elastic properties of soft, differentially weathered, marine sedimentary rocks in Great Britain, such as the Keuper Marl. Meigh has encountered difficulties in data correlation similar to those described in this report, due to the complexity resulting from the bedding, jointing, stress relief, and differential weathering which are important characteristics of these materials.

It is difficult to say whether the range of variations in stiffness of Arizona subgrades is greater than would be expected in other parts of the world. The belief of the authors is that spatial variations are to be expected due to the complexity of the stress fields which have developed in Arizona subsurfaces during ages of desiccation, cementation, and removal of large thicknesses of overburden. In other climatic regions, deposits such as marine silts and clays or nearly saturated coastal plain deposits may tend to exhibit

uniform responses, while glacial tills, hardpans, and residual soils will behave in a very non-homogeneous manner.

Accepting the fact that Arizona subgrades do exhibit significant lateral variations in stiffness, the pavement design engineer is faced with two decisions.

- How large must a sample of field modulus values be in order to statistically bracket the range of E_s for a particular design section?
- From which point on the statistical distribution curve should one select the design value of E_s?

On the basis of the experience at these five test stations, it appears that the answer to the first question resides with the test being employed. For the CBR or Dynaflect a minimum of about 12 independently derived modulus values should be available for each designated design section. A large number of tests is necessary because of the potential for abnormally high or low values inherent in these methods. For 30 inch plate bearing tests or refraction seismic, as few as four values should provide a reasonable indication of the mass response of the subgrade.

These guidelines are founded primarily on the experience at McCormick Ranch fill and to a lesser extent at Cienega Creek. At these two sites, the physical makeup, density and compaction certainly qualifies them as homogeneous. At the McCormick fill E_s from four 30 inch plate tests ranged from

15,000-20,000 psi while the 10 seismic refraction lines resulted in no variation in modulus (21,200 psi). On the other hand the range of E_s from 10 CBR tests was 11,500 to 30,000 while the results of 14 Dynaflect bowls indicated the range of E_s to be 10,500 to 42,500 psi.

The question of whether to use a specified point such as the 75% or 25% percentile value in a suite of E_s values, or to calculate the arithmetic mean and standard deviation is a matter of engineering judgment and experience. In this research effort, the authors have preferred to use the median (50 percentile value) to define E_s where a statistical selection is involved. This decision was made because at these test sites there was no reason to believe that variations would tend toward either the high or low extreme. In some environments there might be inherent reasons why adjusted E_s values might range toward the high or low side of a median value determined from plate load tests. Most likely, where use of the Dynaflect or CBR is involved, subgrades in Arizona would tend to register responses which suggest higher modulus values. Possibly the 25 percentile value of a suite of 12 values is more appropriate and conservative. On the other hand for a micaceous soil, or one composed of weathered and dissolved rock (such as in Meigh's study) values of modulus resulting from CBR or Dynaflect would probably fall toward the low side of true mass response.

4.1.5 Changes in Subgrade Modulus With Time and Environmental Factors

An inherent shortcoming which is associated with the use of any of the newer rational pavement design procedures, is that the input parameters used to describe material properties, while representative of their behavior during or shortly following construction, may not be valid over the many years of the pavement's expected service life. The rational methods are designed to predict the cumulative distress (in the form of rutting or fatigue) which a given pavement will undergo. These are functions of strain levels in the pavement and subgrade layers, which themselves are functions (among other variables) of the subgrade stiffness.

If the eventual subgrade modulus of a pavement is lower than used in the program, the life of the pavement will be shorter than predicted. On the other hand, if the eventual modulus is much greater than used in the design procedure, the result will be the construction of a section which is thicker (and more expensive) than necessary to achieve the desired service life.

The construction of roadways usually involves the compaction of the subgrade at or near optimum moisture content, followed by the placement of subbase, base and bituminous layers. Factors which may cause changes in the subgrade modulus with time are given following:

Increase in E_s From Time of Construction	Decrease in E_s From Time of Construction
Freezing of Subgrade	-Loss of shear strength due to decreased grain friction resulting from higher moisture
Cementing of Subgrade	-Loss of cementation due to solution
Age-Hardening of Subgrade	-Routing of soil particles under combined action of traffic and excess water
	-Loss of density due to swelling of clays
	-Change in mineralogical composition or exchangeable ions in pore water

There have been very few field studies documenting changes in E with time, other than subgrade freeze-thaw investigations, which we have stated are not of major interest in this study. The excellent Colorado study (81) cited earlier is an example of the types of case histories which are pertinent to Arizona conditions. The above list shows that there are more mechanisms which promote loss of stiffness following construction than promote gain. The two opportunities for time dependent increase in stiffness are cementing and age-hardening. It was found at the I-10 Buckeye station that in soil containing naturally large amounts of a cementing

agent (in the Southwest, $CaCO_3$) recementation within the compacted subgrade takes place very rapidly. Age-hardening (thixotropy) in clay soil is possible, depending on its mineralogy, but is only possible under conditions of moisture stability.

It is significant that all of the mechanisms given as causes for deterioration of modulus involve the water content in the soil. Excess free water is destructive to the performance of virtually every type of subgrade. The most immediate effect of excess water is the lowering of shear strength as intergranular friction and tensile forces are destroyed. This permits greater permanent deformation. Secondly, under short-lived pulsating traffic loads, slow-draining, saturated soils can develop high local pore pressures which can dislodge and expel particles (if a path is available) resulting in lower densities and a deformable structure. Pore pressure buildup tends to reduce intergranular forces resulting in a more rapid rate of deformation with progressive application of loads. Thirdly, moving water will eventually dissolve cementaceous materials between soil grains, the result being a weaker matrix.

A fourth mechanism, the change in mineralogical composition, is difficult to document. Experience of this form has shown, however, that in some mine tailing piles, feldspar grains have weathered to form clay minerals (in wet, acidic environments) in less than five years. Such a process would be

detrimental in a pavement subgrade, because clay minerals cause an increase in plasticity (deformability) and contribute to the holding of moisture in soil. Even greater distress would occur if expansive varieties of clay are produced, such as results from the decomposition of volcanic ash or cinders.

Obviously, the testing program for this research effort could not incorporate a time dependent environmental analysis of subgrades. It is necessary to rely on comparisons of performance of the dry and saturated plate bearing tests. The loss in stiffness as a result of saturation can be summarized as follows:

Cienega Creek - 25-40% loss
Rainbow Valley Trench - 70-80% loss
McCormick Ranch Fill - 60-70% loss
McCormick Ranch Desert - 95% loss

The tremendous losses in stiffness at the Rainbow Valley trench and undisturbed McCormick Ranch sites is instructive regarding uncompacted subgrades. The Cienega Creek subgrade appeared to suffer the least, however, the 25-40 percent figure is based on the assumption that E_s = 14,000 psi (30" plate), while we have previously said that in the upper 6 to 12 inches, E_s is actually closer to 20,000 psi, indicating about a 60 percent reduction following saturation. This is the same range as found for the McCormick Ranch fill.

It appears that an appropriate rule of thumb is that a statically determined E modulus in dense cohesive or cemented soils will be reduced by about one-third as the moisture content is raised from a nearly dry to nearly saturated condition. A very low density fine grained soil would be expected to lose more, while a coarse sand and gravel mixture would probably retain much of its stiffness in spite of saturation.

A recent study by Richards (148) includes laboratory evidence concerning modulus of clay soil as a function of water content and soil suction.

The natural tendency of most soils to lose stiffness as excess water becomes available has been documented and quantified by static tests. The pavement behavior analysis problem, however, is a dynamic one. The combined action of dynamic loading and saturation is probably the most destructive of subgrades, so the reduction factor of one-third is probably only an upper bound.

4.1.6 Changes in Subgrade Modulus Under Repeated Applications of Load

The authors have chosen to retain the 30 inch plate bearing test as the standard for predicting subgrade response in the pavement design problem. While such a decision would have many supporters in the pavement design community, major criticism would come from those who point out that E_s is a static modulus of elasticity, while a pavement's

response under operating conditions is a dynamic response. This discrepancy has been the impetus behind dynamic triaxial testing and the use of resilient modulus for all layers in the pavement section.

The refraction seismic and Dynaflect testing methods make use of dynamic loads, but the E values calculated by this method are not dynamic moduli in the truest sense, as is M_R from the repeat-load triaxial test. In reality, traffic loads on subgrades are neither vibratory nor impact, but are random pulse loads which under normal circumstances involve rest periods of sufficient duration for elastic rebound to occur. Therefore dynamic triaxial testing is also an approximation of the actual loading situation, although the use of a square waveform comes close to reality. As shown in section 1.6, M_R must be stated in terms of the number of applications of the deviatoric load at which the value was calculated. An actual highway pavement will receive several million applications of the design load during its operating life. It is obviously impractical to apply this many repetitions in a test. As seen in Figure 10, M_R typically drops with the first several applications and then levels off (or increases slightly due to strain hardening). Several investigators have noted that in bound materials, 1000 cycles is sufficient to define resilient modulus. Kalcheff and Hicks (149) found that in sands, the response at 50 to 100 repetitions is sufficient to characterize the material.

The body of research concerning resilient modulus is valuable, even if one rejects repeat load testing (as the authors have) as too expensive and inconclusive. Those who would use a static modulus to describe the response of a subgrade over millions of cyclic pulse loads spanning 20 or more years, must attempt to resolve the differences between the test and operational situation, presumably, if environmental conditions such as moisture are held constant, E_s will decrease infinitesimally following each pulse due to internal stress readjustments and plastic deformation in the soil. The experience with resilient modulus seems to show, however, that eventually a steady state value of E_s will be reached, and strains will be of the visco-elastic (creep) nature.

While there could be found no published techniques in the literature for relating a static modulus to the steady state resilient modulus, the repetitive plate bearing test is a first step in providing guidance concerning the appropriate reductions. For an empirical procedure the authors selected the 30 inch repetitive tests at six test environments and calculated E_s at a loading increment near 50 psi from the first, second, fifth, and sixth cycles and noted the amount of decrease with each cycle. These are listed in Table 20 and plotted as Figure 75.

It is difficult to infer a steady state E_s value (in terms of its initial value) from but six repetitions, but for the compacted subgrades, it would

appear that 80% of the single cycle E_s is a minimum value. One can be certain that this is conservative because in reality the response after several cycles is one which should be measured from the point of recoverable deformation (rather than original zero) as is the resilient modulus. As noted in Section I such a slope is usually steeper than one connecting points on a virgin deflection curve, and, therefore, corresponds to a higher modulus.

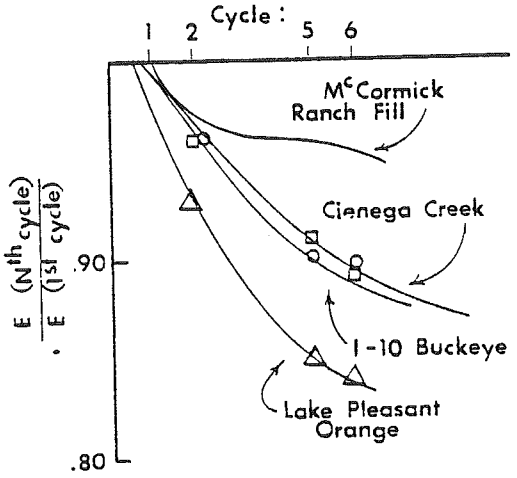


Figure 75. Reduction in Modulus Due to Cyclic Loading

Table 20. Reduction in Modulus Values Calculated at Various Cycles of Loading

Test Location	Stress Level	E_s (from 1st, 2nd, 5th & 6th cycles)	$\frac{E_n}{E_1}$ (n=1,2,5 & 6)
I-10 Buckeye	57.7 psi	27,300-26,300-24,800-24,300	1-.96-.91-.89
Lake Pleasant Orange	54.9 psi	21,900-20,500-18,700-18,400	1-.93-.85-.84
Cienega Creek	54.9 psi	16,500-15,900-15,000-14,800	1-.96-.90-.89
Rainbow Valley Surface	42.4 psi	10,000-8400-7300-7200	1-.84-.73-.71
Rainbow Valley Excavation	56.0 psi	40,600-40,600-39,400	1-.97-.97
McCormick Ranch Fill	56.6 psi	22,600-21,900-21,900-20,200	1-.96-.96-.89

4.2 Criteria for Selection of Subgrade Modulus for Use in Pavement Design Systems

4.2.1 Guidelines for Selection of E_3

In previous sections, methods of measuring subgrade modulus E_s of comparatively dry soils in Arizona have been described, as have techniques which may be of benefit in interpreting in-place test results. Four important points will be briefly summarized.

- Soil and subgrades do not behave elastically except at low level strains; nevertheless, a properly derived secant E modulus provides an adequate representation of the subgrade compressibility under highway type loadings.

- The most definitive field measurement of subgrade response is obtained with the 30 inch diameter plate bearing test at a stress level of about 30 to 50 pounds per square inch.

- Field (in-place) testing is considered to be much more representative of subgrade compressibility than laboratory testing due to the influence of cementation and desiccation in Arizona soils. Tests other than plate bearing may be employed, but an adjustment of some kind must usually be made to the modulus value, in order to obtain a value of E_s which is equivalent to that which would be determined by the plate bearing test.

- Two or possibly three adjustments must be made to the value of E_s pertaining to the 30 inch plate test in order to achieve a meaningful value of E_3, the modulus of the third layer, for use in a layered pavement design method.

The three adjustments are: (1) A factor to bring a measured modulus value into the range of E_s. The determination of this factor is dependent upon the test method. This first reduction eliminates the problems of geometry of loaded area and stress level. If large diameter plate bearing tests are performed, no adjustment is needed (other than a statistical one, to arrive at a mean or median value for a design section). (2) An adjustment to E_s to account for the dynamic and repetitive nature of pavement loadings. As discussed in section 4.1.6, this is a reduction factor which should not fall lower than 0.80 for compacted subgrades. (3) An Environmental Reduction Factor, herein designated ERF. This factor is to be based on an educated judgment of the expected environmental deterioration of modulus during the service life of the pavement. It is similar to Jiminez's Regional Factor, but should be more inclusive. Factors which must be considered when choosing the ERF for a design section are:

- annual rainfall*
- mean annual temperature*
- slope
- anticipated traffic
- drainage conditions
- thickness and makeup of pavement layers
- use of membrane seals or other moisture protection measures

*both functions of elevation in Arizona

In reality, the ERF is an index of the susceptibility of a subgrade to significant and permanent increase in postconstruction moisture content. This factor is as important as any in the pavement design system, as the results of the saturated plate bearing tests show that the ERF could be as low as 0.1. Guidance in the selection of this factor for Arizona subgrades is given in section 4.3.

4.2.2 Techniques Available to Aid in the Selection of E_3

The plate bearing test is most useful in determining E_s but, on a unit basis, is probably the most expensive and time consuming. Alternative field methods recommended in order are; CBR, refraction seismic and Dynaflect. The CBR is considered the best alternative to the plate bearing test because of its speed and simplicity. Its success depends upon careful selection of test locations by the field crew. Within a design section, random placement of tests is desirable and they should not be placed where visually harder or softer than normal spots in the subgrade can be seen.

Refraction seismic (using only compression wave velocity) can be very efficient, but must be used only where subgrades have low moisture content. A 5 foot (or smaller) shot point spacing is essential.

The commitment of the Dynaflect to a program of pavement design would require a careful monitoring

and constant refinement of procedures under the direction of an engineer/specialist.

It is highly probably that other, more efficient, in-situ subgrade response measurement techniques will become available in the next decade. The ideal system would consist of a small, mobile piece of equipment which, using an impact or vibratory load response, can quickly and accurately derive V_c and V_s values for strata in a layered pavement section. A prototype along these lines is being developed by CERF (see section 1.7.7). The Koninklijke/Shell Laboratorium (Dutch Shell) are using a Falling Weight Deflectometer device in pavement evaluation (82). The device is towed behind a truck or van, and is characterized by a 150kg (330#) mass which is dropped 400mm (15.7") onto a set of springs mounted on a rigid circular plate of 300mm (11.8") diameter. Deflections are measured at the center and at fixed distances from the plate. This data is recorded and analyzed in the control van.

The condition of the pavement is measured in terms of the modulus of the asphalt E_1. E_3 and E_2 (which is a function of E_3) and the layer thicknesses are inputs which must be known or assumed.

In its present configuration, the pavement evaluation procedure developed by Shell for the Falling Weight Deflectometer is similar to that developed by Texas Transportation Institute for the Dynaflect.

Should a major breakthrough take place in the digitization and analysis of waveforms, the Shell FWD hardware would be an excellent prototype for data development. It is not inconceivable, then, that in the future each state Department of Transportation will be able to acquire a device which can quickly obtain E_{seis} of various layers in a pavement. Regardless of what sophisticated measurement techniques are introduced in the future, the need to transform subgrade modulus values to those of the 30 inch plate at appropriate stress levels will remain.

The usefulness of laboratory testing and its relation to in-situ E_s will continue to be of interest to many. The straightforward, high production tests such as soil support value (based on PI and -200 fraction of a soil), R value and laboratory CBR will likely continue to be a part of pavement design technology. A look at soil support value in terms of Arizona Department of Transportation's correlation chart (see Figure 12, section 1.6.6) is given in Table 21. It can be seen that values of E modulus correlated with SSV are almost random when matched against the E_s values determined in-situ. For the most part, they are much lower than E_s in the dry to partially saturated state. Accordingly, SSV should be thought of as providing only a lower limit of E_s.

The more difficult and sophisticated laboratory tests (specifically triaxial testing) are useful

for research, but probably can never become an economically viable method of producing design information for cemented or very dry subgrades. It is, therefore, recommended that the Arizona Department of Transportation direct the major portion of its research effort in the future toward implementing efficient procedures for in-situ evaluation of soils and subgrade response.

Table 21
Comparison of Modulus Values Derived From Soil Support Value With E_s

Location	-200	PI	SSV	E (based on SSV) (psi)	E_s (psi)
I-10 Buckeye	55	10	4.7	5500	25,000
Lake Pleasant Orange	48	11	4.6	5200	18,000
Cienega Creek	33	6	7.4	15,000	14,000
Rainbow Valley (Near Surface)	45	9	5.7	8200	6000-10,000
Rainbow Valley (Excavation)	54	14	4.3	4300	12,000-26,000
McCormick Desert	60	7	5.7	8200	14,000
McCormick Fill	80	8	4.4	5000	18,500

4.3 Selection of the Environmental Reduction Factor (ERF)

4.3.1 Moisture Changes Beneath Covered Areas

The supporting power of a subgrade is dependent on

its moisture condition to a greater extent than upon any other variable. Therefore, the Environmental Reduction Factor, which is to be applied to the dry state E_s in order to produce a more accurate index of subgrade response during the working life of the pavement, is, in reality, a prediction on the likelihood of the equilibrium moisture content in subgrade becoming different from that of the test conditions. Because of its importance, the topic of moisture change beneath covered areas has been a major concern of pavement specialists for many years.

Interest in the subject began in the United States and in Great Britain toward the end of World War II. In 1945, Kersten (150) published an extensive study of moisture conditions under rigid and flexible airfield pavements throughout the U. S. A follow-up study was published by Redus (151) several years later. Redus found that while the degree of saturation in covered subgrade did not correlate with annual rainfall, there was a relationship between the equilibrium moisture and plasticity of the soil. That is, there were usually high degrees of saturation in the highly plastic clays, but rarely in the low to nonplastic subgrades. Kersten recognized that subgrade moisture was a function of year-round climate as well as physical properties of the soil. He found that, in most regions of the country, there could be found an increase in the moisture content of silty and clayey soils above the optimum moisture content and usually above the plastic limit.

In arid and semi-arid environments, however, the plastic limit was reached in only 10 percent of the cases involving clayey subgrades. The sandy subgrades under pavements in semi-arid environments were found to be extremely dry.

Toward the close of World War II, a substantial program of research on the prediction of subgrade moisture equilibrium was begun at the Road Research Lab (RRL) in Great Britain. At first, the emphasis was directed toward moisture change phenomena in subgrades and climates similar to those found in the United Kingdom, i.e., high rainfall, moderate temperatures and water table near the surface. A few years later, the effort was expanded to include tropical and desert environments. Use was made of overseas airfields for this purpose, primarily in Africa, the Mid-East and East Asia. The RRL group headed by Russam was also instrumental in initiating field studies by other groups in Israel and Australia.

The early investigators in the field of subgrade moisture change borrowed theories from soil physicists, agronomists and hydrologists concerned with flow through porous media. The problem of quantifying moisture movement in a partially saturated material is extremely difficult and uncertain. Both the development of rational methods of prediction, as well as interpretation of field data, are limited by the multitude of variables which are involved, such as the physical and physicochemical nature of

the soil, the effects of heat transfer within the subgrade, and the two phase (liquid/vapor) nature of moisture movement. In addition, complicated and often unknown boundary conditions are an important consideration in actual situations.

Today there is a considerable body of well developed rational theory regarding moisture movement under covered areas, as well as a considerable amount of field evidence on the subject. As is the case with current pavement design technology, however, there is a question as to whether the sophistication incorporated in the mathematical models is commensurate with the available means of collecting and interpreting measurements in the field. The interested reader is directed to a "Symposium in Print on Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas" (152) sponsored in 1965 by the CSIRO organization in Australia. An excellent summary of the conclusion of an international panel of experts was published by Aitchison, Russam and Richards (153).

In most of the recent literature, the moisture in a subgrade is quantified by its "moisture suction", which is an index of the energy required to cause water to flow in a partially saturated soil. Suction is preferred over moisture content because it relates to the potential for moisture increase (which depends on soil structure, permeability, etc.). A very compact desiccated clay soil would exhibit a high value of suction while a nearly saturated sandy silt would have a low value.

The pavement engineer concerned with the current state-of-the-art in design should look principally at the results of the many field studies. In seeking an Environmental Reduction Factor, it would be nearly impossible to develop a meaningful rational procedure to calculate a value. The engineer must rely primarily on judgment and experience.

The studies of the Crowthorne Road Research Group concerning moisture equilibria beneath pavements deserve the greatest amount of attention. Russam and others (154, 155, and 156) have divided subgrade environments into three categories for their purposes:

- Those where the water table is closer than 20 feet to the top of the subgrade and is maintained by abundant rainfall, impeded drainage or a nearby river, lake or sea.

- Those where the water table is deeper than 20 feet and rainfall is greater than 10 inches per year.

- Those where the overall climate is arid or semi-arid, rainfall is less than 10 inches per year and the water table is very deep.

Most of the southern, western and northern Arizona subgrades are in the third category, while most of the subgrades in higher elevations are described

by the second. Only in isolated circumstances can subgrade conditions of the first type be found in Arizona.

Russam presented examples of the first type of subgrade in both temperate climates (Great Britain) and tropical (Kenya, East Africa, and Singapore). In either case where the water table is shallow and the potential for evapotranspiration does not exceed the seasonal rainfall, soil moisture beneath impervious pavements comes into a static equilibrium with the water table. The only fluctuations are noted at the verges where cycles of wetting and drying occur in response to heavy rainfalls.

For the second class of subgrade environments, there is a tendency for the subgrades to experience an increase in moisture beyond construction moisture content to a degree somewhat related to the plasticity of the soil. Russam states that within a given climatic environment, the ratio of ultimate moisture content to the plastic limit tends to be constant. Russam (157) presents evidence of this from airfields in subtropical Rhodesia. Laxton (158) found that in the Australian airfields, the ratio of equilibrium water content to plasticity index is 0.83 to 1.23. Livneh, et al (159) reported that at an Lod airport, in the wetter region of Israel, the postconstruction wc/PI ratio reached 1.0 to 1.2.

The RRL findings at arid and semi-arid locations are of special interest to those working in Arizona. At

Abu-Siuer airfield in Egypt where rainfall is about 1 inch per year, Russam and Coleman reported there was no difference in moisture under the runway and adjacent uncovered soil (a nonplastic sand). Likewise, at an airfield in Aden, where a water table maintained by the sea was within 3 feet of the surface, measurements showed that moisture contents in the covered subgrade and the exposed soil were little different. At Khartoum, Sudan, the climate is very similar to Arizona (6 to 8 inch annual precipitation with considerably greater potential evaporation). At the airfield, a water table from the Nile River fluctuates between 23 and 32 feet below the surface. The moisture content of exposed soil fluctuated seasonally between 3 and 7 percent. Beneath the center of the taxiway, moisture content increased in 5 years from 6 to 18 percent, while there was no increase near the verge. An analysis of the clayey subgrade soil revealed that it contained a substantial percentage of expansive montmorillonite. It is therefore believed that this highly plastic material promoted upward movement of moisture from the water table. Near the verges where evapotranspiration was aided, permanent moisture buildup could not be maintained.

At the 1965 Symposium on Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas, a panel of experts agreed that where climate is arid throughout most of the year, and where normal care is taken to remove the surface waters from roadways, via camber of the roads and necessary drainage systems, the accumulation of water beneath road and

airfield pavements is prevented. In such circumstances, the soil moisture suction under sealed pavements is controlled by the mean atmospheric humidity and the moisture condition of the subgrade will differ little from that of uncovered soil.

In addition to the findings of the RRL, conclusions of the panel are supported by field evidence gathered by Livneh (160) in Israel, by de Bruijn (161) in South Africa, and Aitchison and Richards (162) in Australia. In an area of Israel where precipitation averages 8 inches per year, variations in the moisture content of a clayey loess were barely discernible, except immediately adjacent to the shoulders, where variations were ties to each of the infrequent heavy rains. Livneh also stated, based on findings throughout Israel, that in order for significant increases in the moisture beneath the pavements to be noted, there must be at least 6 inches of rainfall in a season and 12 inches of annual rainfall. Where 12 inches annually is not attained, there will be no change in the subgrade surface moisture content.

de Bruijn reported subgrade moisture data at a test station near Britstown in Central Karroo. The subgrade is a silty sand which is lime cemented. The annual rainfall is about 10 inches; therefore, conditions are very similar to those in southern Arizona. Under the center of a tarred road, the soil is in a desiccated condition.

Slightly higher moisture conditions were noted at the verges. At other test stations where rainfall was above 20 inches per year, higher moisture contents were found under the center of the pavement. In these environments, the verges were much drier; possibly because adjacent vegetation aided in removing the moisture.

Aitchison and Richards reported on field results dealing with moisture equilibria beneath pavements over the entire Australian continent. At each test station, moisture was measured at several spots to a depth of 5 feet by means of gypsum blocks. Two of the Australian sites are of particular interest to those working in Arizona. At a site near Alice Springs, where annual rainfall is 10 inches and the subgrade is a cemented sandy clay, no significant changes in soil suction were noted with time directly beneath the pavement. Moisture increases have been noted for a few weeks following heavy rainfalls, particularly near the verges. At the Woomera Village Aerodrome, the subgrade is strongly cemented by silicrete (which is more common in Australia than lime cementation). The average annual rainfall is less than 6 inches. At this site, the sensors revealed no changes in suction under the pavement or verges.

The results of these overseas studies suggest that moisture stability is the rule beneath covered areas in arid and semi-arid regions. Croney and Kulman (163) of RRL state that in dry arid climates, such

rainfall as there may be has no influence on the moisture condition under roads, and the moisture content of the subgrade will be close to the average value observed in the surrounding uncovered soil.

Several studies of moisture change have been performed in the U. S., though none in a truly semi-arid region. The most applicable to Arizona is the Poehl and Scrivner (124) study of seasonal changes in deflection using the Dynaflect. Notable field studies of moisture change beneath highway pavements have been performed in Missouri, Oklahoma, Pennsylvania, Ohio, South Carolina and Virginia. As previously mentioned, an extensive study of moisture change beneath Arizona subgrades will be published within a year, which should prove beneficial in the selection of the ERF.

In Missouri (164), one of the earliest field testing programs involved a 5 year study of moisture change beneath PCC pavements. Surprisingly, only very small moisture variations were noted under the centers of slabs. It was suggested that the placement of slabs on subgrades compacted above optimum moisture gives the best results.

Haliburton (123) reported that in the silty clays in north central and northeast Oklahoma, a year or two following the sealing of a subgrade, the moisture content achieved equilibrium at about 1.1 to 1.3 times the plastic limit. This is an area of

20 to 40 inches annual rainfall; thus, the climate and the environment are similar to Rhodesia or the wetter portions of Israel where Russam and Livneh, respectively, reported similar increases in post-construction moisture.

The Pennsylvania study by Cumberledge (122) and Majidzadeh's work in Ohio (99, 100) deal with subgrades subjected to predictable annual freeze/thaw conditions. At five sites in Pennsylvania, the highest moisture contents (and largest Road Rater deflections) were recorded following the spring thaw. The sandy subgrades exhibited the largest increase in seasonal moisture while the clayey soils exhibited only a mild rise.

In South Carolina, Chu, Humphries and Chen (165), monitored subgrade moisture at 30 sites throughout the state. It was found that in the resilient (micaceous) fine sands and silts, which are common in the piedmont and coastal plain regions of the eastern United States, there were usually increases in subgrade moisture beneath the center of the pavements to levels higher than optimum, although no real correlation with plastic limit was found.

In Virginia, Vaswani (166) monitored subgrade moisture under pavements in an environment similar to that in South Carolina. Following construction, the subgrade moisture rapidly increased and then leveled off; a situation which is typical in that type of climate. It was further found that subsoil

moisture in uncovered ground was greatly affected by rain, snow and air temperature, but the moisture beneath covered areas was only slightly affected.

In conclusion, it appears that field studies of subgrade moisture change conducted to date in the United States involved climates and subsurface environments which are much different from much of Arizona. Those conducted in the Mid-East, South Africa and Australia, however, do provide valuable insight.

4.3.2 Moisture Changes in Subgrades in the Southwest United States

At this time, few documented case histories involving moisture change beneath paved areas in the Southwest are available to the public.

Two studies by the Colorado Department of Highways are important, as they deal with subsurface environments and climates in that state which are very similar to those found in northern Arizona and the central mountains. In the first study (81), an evaluation was made in 1968-69 of pavement sections which were constructed 15 and 16 years earlier. A comparison of those sections which were failing or had failed and those which remained in good service is revealing.

- For all projects, there was no correlation between in-situ CBR and the laboratory CBR on

soaked subgrade materials performed 15 years prior. This would seem to indicate that either saturation was never achieved, or physical or chemical changes in the subgrade soil had taken place.

- There were no noticeable seasonal changes in subgrade moisture where the projects had good service records. Wherever undue distress or failure took place, the subgrade moisture was far in excess of optimum.

- Almost all weak spots under flexible pavements were associated with local areas of poor drainage.

The Colorado study indicated that, in that state, it is possible to maintain reasonably dry (optimum) moisture conditions beneath the flexible pavements. Where excess moisture does occur, however, distress results even though the subgrade stiffness criteria was based on a soaked CBR (presumably its poorest condition).

Expansive clays are a special problem to highway engineers throughout the world, and are found over wide areas of the western United States.

Brakey (167) presented research performed by the Colorado Division of Highways which dealt with moisture protection of subgrades on the expansive Mancos Shale. It was determined that on many roadways in

that state, postconstruction moisture increases in the granular base material were taking place as a result of hydrogenesis. This is the condensation of water vapor flowing through the large voids in the permeable base above the subgrade. The drops of water form on the granular particles and eventually reach the expansive subgrade causing distress. On test sections where the clay was protected by a membrane made of catalytically blown asphalt placed directly on the subgrade, no change in the subgrade moisture content was noted.

Hydrogenesis is not limited to open graded bases overlying expansive soils. The phenomena is probably more common than realized in semi-arid environments. Poor drainage conditions; that is, designs which allow runoff from infrequent heavy rainfalls to be trapped in a permeable subbase above a relatively impermeable subgrade are, undoubtedly, the most common reason for moisture buildup beneath pavements in areas where the water table is not a factor.

It appears, however, that significant postconstruction moisture increases in desert subgrades do not routinely occur.

Preliminary findings by engineers involved in Arizona Department of Transportation's subgrade environmental monitoring program indicate (personal communication) that at some desert stations subgrade moisture is increasing with time. No water table is

involved in such cases, therefore, either poor drainage or hydrogenesis is indicated. On the other hand, data from ADOT moisture monitoring stations near the Cienega Creek and Lake Pleasant (Orange) sites apparently have not shown permanent increases in moisture in the uncovered subgrades, nor is there a discernible pattern of seasonal change.

During the past 15 years, the firm of Sergent, Hauskins & Beckwith has had the opportunity to investigate subsurface conditions at several operating airports in the Southwest. The Kingman Airport was constructed as an Army airfield in the early 1940's. The section consists of 3 inches of asphaltic concrete over 9 inches of base. An Army report dated 1944 indicated the moisture content in the upper foot of the subgrade (a sandy clay) was 8 to 10 percent. An investigation by SHB in 1969 indicated much higher moisture contents in localized areas beneath the pavement (on the order of 11-15 percent). These increases were attributed to seepage through cracks which had developed in the asphalt. Other areas of the subgrade were found to be much drier than optimum. Kingman receives approximately 10 inches of rainfall annually.

At the Marine Corps Air Station in Yuma, where annual rainfall is about 4 inches, SHB investigated subsurface conditions beneath areas of distress on runways which, like the Kingman airfield, had been in service about 25 years. Most of the runway was

underlain by fine sands which were of very low moisture. Several expansive clay lenses crossed the runway area. It is believed that at the time of construction, these were compacted at near the plastic limit; however, at the time of the investigation, the moisture contents were well above the plastic limit. The situation appears very similar to that at Khartoum, except that no water table is involved. Therefore, the moisture increase in the clay must have come from another source. Possibly hydrogenesis has been a factor, although seepage through cracks was probably the major cause.

At the Grant County Airport (near Silver City, New Mexico), an investigation 5 and 10 years after two stages of construction revealed low subgrade moisture (very near optimum) in a lime cemented sandy clay subgrade. Annual rainfall is approximately 15 inches. It should be noted that this airfield was built with asphalt placed directly over a low permeability compacted sandy clay. It appears that this type of construction has been successful in preventing significant moisture infiltration and movement in the subgrade.

The Litchfield Park airfield west of Phoenix was investigated in 1970. This airfield is within a few miles of the I-10 Buckeye and Rainbow Valley sites, and native subgrade conditions are similar. Portions of the runway being investigated were constructed during World War II and others in 1955. Borings in the fine grained subgrade revealed that

in most cases moistures were well below the plastic limit. However, in a few cases, moistures had risen slightly; particularly near the verges of the pavement.

At Deer Valley Airport on the north edge of Phoenix, an investigation in 1971 revealed that soil moisture contents beneath the pavement were generally slightly below the plastic limit. The subgrade is a sandy clay which is weakly lime cemented. As was the case at Litchfield Park, a few isolated low spots adjacent to the pavement had contributed to ponding and moisture had increased in the upper few feet of pavement near the verges.

The evidence seems to suggest that large moisture buildups beneath pavements in Arizona are not inevitable, provided careful drainage of surface water is provided and hydrogenesis prevented. Many pavements in the Southwest have been in service for 20 years or more without any distress which could be attributed to a use in subgrade moisture. In most cases where moisture increases and distress have occurred, one of several measures might have been successful in maintaining the near optimum construction moisture content.

4.3.3 Means of Retarding Moisture Increases in Subgrades

Ideally, the impermeable seal coat, along with the usual camber of most asphalt roadways, would be expected to permit rainfall to quickly flow from the

top of a pavement over the shoulder and into a lateral ditch which carries the runoff to a natural watercourse. Hopefully, then, the structural section and subgrade are relatively unaffected by the periodic rainfall. Unfortunately, surface seals do not always remain impermeable. Joints and cracks, which can develop for a variety of reasons, in bituminous surface layers provide many opportunities for seepage to make its way into the bases and the subgrades. Problems arise if this seepage becomes trapped, which is likely in cases involving open graded or highly permeable bases and subbases overlying a low permeability subgrade. Such seepage tends to collect at the top of the subgrade, softening it, and creating the "trench" or "bathtub" condition of the structural section as described by Cedergren (168).

In his recent textbook, Cedergren (168) is emphatic concerning the need to construct roadways which provide rapid removal of rainwater through the pavement section. Cedergren believes that a highly permeable macadam type bound base used in conjunction with collector pipes is most satisfactory. He properly questions the trends incorporated into rational pavement design methods of the past three decades which emphasize strength and density in pavement preparation while ignoring the problem of drainage. Indeed many current rational methods, in relying on an index such as the soaked CBR, do attempt to provide a section which will perform even under the poorest condition of drainage. Cedergren points

out that such an approach does not take into account the dynamic nature of the destruction caused by excess water in a pavement section.

The authors cannot agree, however, with Cedergren's design philosophy if it is to be applied to regions where annual rainfall is less than 20 inches, and where evaporation potential is considerably greater than precipitation. Cedergren discounts the effectiveness of methods and materials designed to seal a subgrade from moisture infiltration. Case histories noted in sections 4.3.1 and 4.3.2 have shown that except where pavement cracking or very poor drainage procedures are present, subgrades in regions characterized by low rainfall and prolonged dry spells do not develop great increases in moisture content beneath paved surfaces. Problems do occasionally arise where an open graded base of high permeability is employed without drains or permeable shoulders.

The most successful techniques for preventing moisture increases are; (1) the placement of a full depth asphaltic concrete layer directly on dense compacted subgrade of low permeability, (2) the use of catalytically blown asphalt in a thin layer between the subgrade and subbase, (3) the use of membrane encapsulated soil layers (MESL) using a plastic sheeting or one of the newer high strength fabrics and, (4) the use of a rubberized asphalt membrane or surface seal.

Depending upon their location in relation to available construction materials, full depth asphaltic pavements are likely to involve higher initial cost than pavements containing layers of aggregate base. Case histories have shown that they perform well in climatic environments such as found in much of Arizona. The postconstruction investigation of the Grant County, New Mexico airport provides an excellent example.

Catalytically blown asphalt has been used successfully to protect expansive clays from moisture increase. Brakey (167) used a layer of catalytically blown asphalt to provide a water protective membrane above the expansive Mancos Shale in western Colorado. Gerhardt (169) used a similar membrane to prevent swelling of an alluvial clay deposit.

The development of asphalt-rubber membranes as seal coats, interlayers and as subgrade protection elements has been pursued actively by the Arizona Department of Transportation (170, 171), and will likely become a major tool of construction in this state in future years. In addition to providing an element of waterproofing to the structural layer, rubberized asphalt interlayers also have the capability of absorbing tensile flexural and shearing stresses, thus retarding crack propagation. Several test projects have been established involving roads in all climatic zones and with traffic volumes which range from light to very heavy. Many technical problems concerned with asphalt-rubber interaction, construction and rolling procedures

and surface preparation are being investigated. There is little doubt, however, that placement of a rubber-asphalt seal or membrane within a flexible pavement greatly reduces the potential for moisture increase in the subgrade.

4.3.4 Engineering Judgment in the Selection of the ERF

Returning to the stated purpose of this section, which is the selection of an environmental reduction factor to apply to E_s, the evidence clearly suggests that factors in Arizona are favorable for maintaining subgrades at near optimum moisture conditions. The low rainfall/high mean annual temperature regime over much of the state tends to promote moisture stability. Postconstruction moisture beneath pavements does occur in some instances, but these can usually be prevented by employing the appropriate design and construction techniques if anticipated.

The Environmental Reduction Factor ERF, like the design value for traffic 20 years in the future, requires intuitive judgment and a great degree of sophistication should not be expected. Values of ERF should probably consist of only one significant figure (e.g., 0.8, 0.7, etc.).

Covered subgrades in Arizona should be thought of as being in one of three possible moisture conditions. The nearly dry condition is a state where

degree of saturation is less than about 20 percent and a high degree of tensile stress exists due to desiccation. This is the range in which E_s is to be measured if possible. The partly saturated condition probably includes the range of S from about 20 to 80 percent, depending on the physical characteristics of the subgrade. This is the range of most compaction moisture contents and probably includes the equilibrium moisture content to which a covered subgrade will gravitate. In this middle range, most tensile stresses in soil are destroyed but the positive effects of cementation and grain interlocking remain. The near saturation condition describes a subgrade where a significant amount of free water is available. Under the pounding action of traffic, saturation will eliminate most of the effects of cementation, either mechanically or by solution, and will greatly weaken the interlocking forces of the individual grains. The loss in stiffness, which such a moisture condition induces, depends largely on the density and particle size distribution of the soil.

In assigning an ERF, the engineer must consider both the liklihood of moisture change, which is generally a function of external circumstances, along with the probable consequences of that moisture change concerning the subgrade response. The latter is a function of the internal makeup of the subgrade. Factors which should be considered in all cases include:

- Mean Annual Rainfall - It is obvious that in regions which receive larger amounts of rainfall, opportunities are greater for moisture increases into the subgrade to reach detrimental levels. It should be considered in those regions in Arizona where rainfall is in excess of 20 inches per year (generally above elevation 6000).

- Mean Annual Temperature - Ambient air temperature determines the rate at which a covered subgrade and its environs can return to the equilibrium moisture following infrequent heavy precipitation.

- Structural Design - The engineer must carefully consider whether the as-built structure will contribute to the elimination or to the collection of moisture in the subgrade. An open graded aggregate base adds to the strength of a section, but may act as a reservoir for seepage unless well constructed shoulder drainage is provided. Open graded bases also present an opportunity for hydrogenesis to take place. On the other hand, assurance that a well drained structure will be built can sometimes be used to justify an ERF close to 1.0. In general, granular bases are highly susceptible to moisture increases and require a substantial reduction of E_s. Full thickness asphaltic concrete or designs involving stabilized bases of low permeability are highly resistant to subgrade

moisture increase for typical Arizona conditions and are considered highly preferable. An ERF of 1.0 can usually be employed with these sections.

- Grade - Positive drainage minimizes the possibility of moisture buildup. In contrast, those features that allow long-term ponding of water near the pavement verge are likely to create moisture increases to some degree with any type of pavement structure.

- Subgrade Properties - Granular subgrades allow the drainage of moisture much more quickly than fine grained soils and are not greatly weakened by moisture increases. In contrast, most fine grained soils lose a substantial portion of their support capacity as a result of saturation.

- Moisture Protection Measures - The incorporation in the final design of special measures, such as membrane protection (see section 4.3.3), to retard the entry of moisture into a subgrade certainly justifies the use of an ERF near 1.0.

Figure 76 provides empirical guidelines for the selection of an ERF based on the field testing performed for this project as well as the experience of Sergent, Hauskins & Beckwith with other projects throughout the Southwest. This figure represents only a suggestion and is not intended to supplant the knowledge and experience of the engineer dealing with a specific situation.

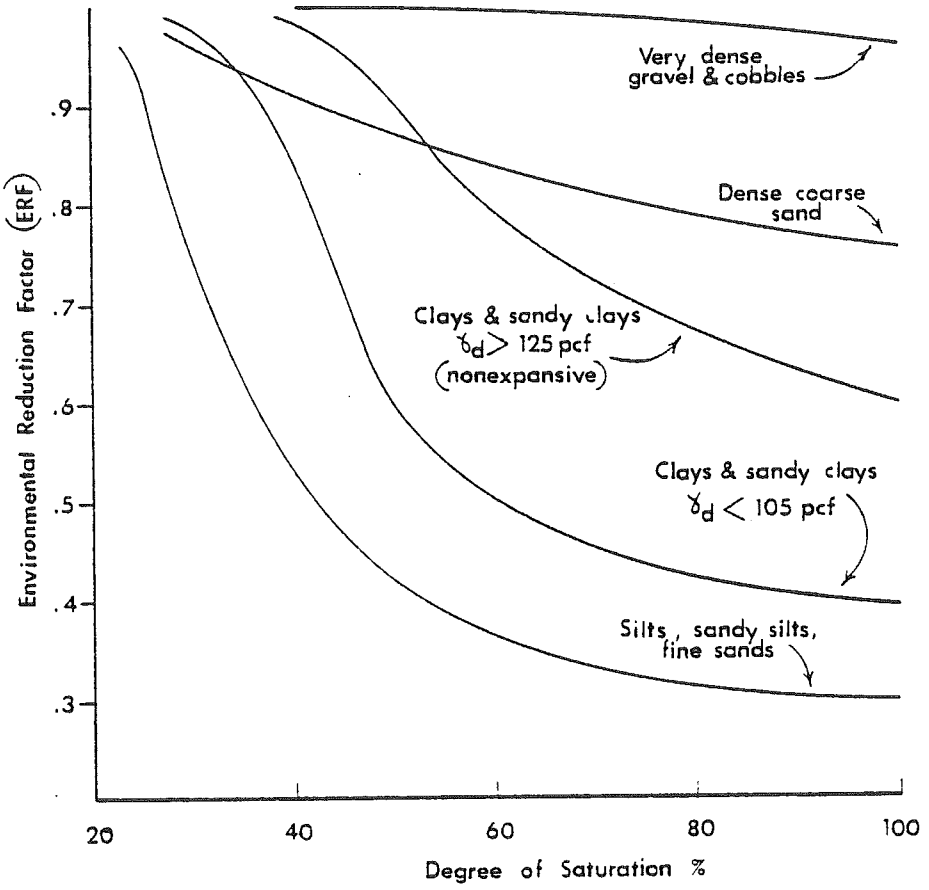


Figure 76. Suggested Relationship Between ERF and Degree of Saturation

4.4 Incorporation Into the Pavement Management System

4.4.1 New Pavement Construction

Figure 77 is a flow diagram which summarizes the suggested procedures of incorporating the most meaningful value of subgrade modulus into the eventual design and construction of new pavements.

A new segment of highway to be constructed should be divided into segments based upon considerations of geometric and geological similitude. As is currently the practice, a program of routine laboratory testing, along with field subsurface investigation, should be performed for each section. Based on the laboratory and field evidence, and on the experience of the design engineer, a preliminary value of E_3 can be assigned to the subgrade in order to perform a preliminary thickness design and to allow cost estimates and preparation of plans and specifications. This preliminary estimate of E_3 should be at least somewhat conservative in order to insure that costs are not underestimated.

At the time of construction of the graded and drained section, a detailed program of field testing should be initiated to provide a close determination of equilibrium subgrade modulus E_3. The field measurements of response can be determined by any of the four techniques emphasized in this report, or possibly by a method yet to be introduced; such as an impact device. In any case,

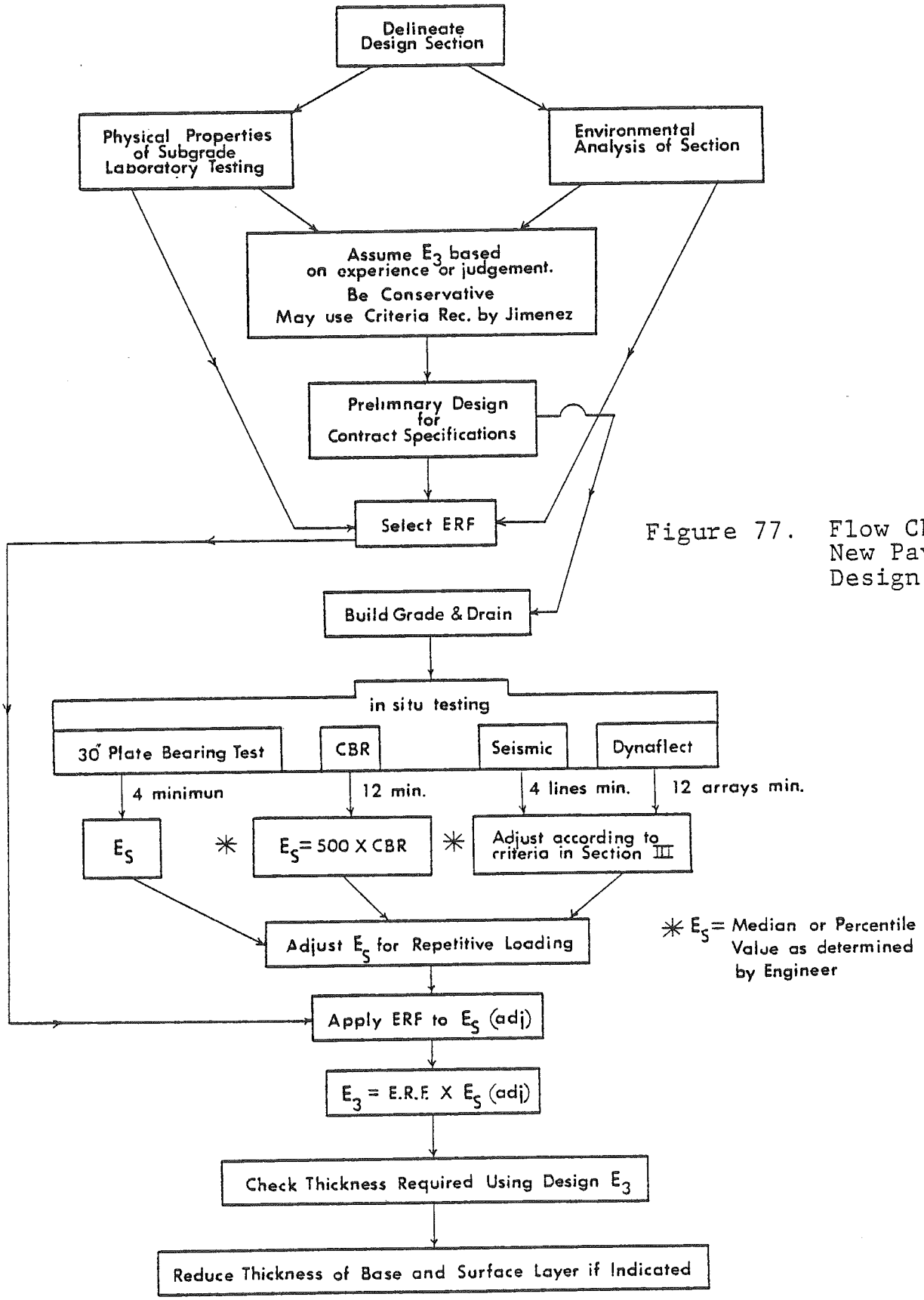


Figure 77. Flow Chart for New Pavement Design

a value of E_s must first be determined as being representative of the response of the subgrade:

- at the low moisture content that would be in equilibrium with the environment with good moisture protection of the subgrade (usually at optimum moisture or lower).
- loading over 30 inch circular area
- stress level of 30 to 50 psi

Two adjustments/reductions must be made. The first adjusts E_s to account for the repetitive nature of traffic loading. As discussed in section 4.1.6, the magnitude of the reduction depends upon the stiffness and resilience of the subgrade. In any case, the reduction factor should not be less than 0.8 for well compacted subgrades.

The more important reduction factor is the ERF. Figure 76 is presented as a guideline, but engineering judgment based on experience should be the primary element in the selection of this factor. Further, in the process of selection of the ERF, the pavement designer can incorporate the desired level of conservatism, based on the economics of a given project.

The corrected E_s, now designated E_3, can be used as input to a rational design method, such as that proposed by Jimenez (2), in which the thickness of the pavement structure elements is determined.

Two example problems are given to illustrate the use of the procedures described in this report.

Example No. 1:

A new section of highway is to be built in an alluvial valley in western Arizona. Laboratory data indicates:

Soil Classification: A-6
LL = 23
PI = 12

Percent -200 = 62
R value = 18
Lab CBR = 4.5

Because of a nearby source of quality aggregate, it is desired that a section be designed with at least two-thirds of the thickness composed of aggregate base. The environmental report indicates that the native soil is moderately lime cemented below 1 to 3 feet at most locations, except where the alignment crosses natural drainages.

Approximately two weeks after completion of the graded-and-drained construction, 12 in-situ CBR's were performed. At this time, it was found that for the compacted subgrade

σ_{dry} = 125 pcf, w_c = 5%, and S = 40%

The suite of CBR values was as follows:
32, 46, 49, 50, 50, 52, 54, 57, 64, 66, 69, 74

The median (high) of these values is 54; therefore, subgrade modulus is selected as

$$E_s = 500 \times 54 = 27{,}000 \text{ psi}$$

Because of the density and lime content of the subgrade, a reduction for cyclic loading of 0.90 appears to be adequate. An assessment of conditions affecting the ERF can be done on the basis of aggravating/mitigating circumstances. In this case:

Unfavorable	Favorable
• subgrade is a lean clay, low permeability	• rainfall is only 4 inches annually
• aggregate base may promote hydrogenesis	• soil contains a considerable amount of lime
	• design includes provisions for good lateral drainage

It is judged that during the 20 year design life of the pavement, the degree of saturation will rarely exceed 60 percent. An ERF of 0.80 is considered reasonable.

Therefore;

$$E_3 = (0.90)(0.80)(27{,}000) = 19{,}400 \text{ psi}$$

A 20 year design ADT of 50 (EAL's) per day is forecast. From Figure 22 (taken from Southgate's charts) it is seen that 4 inches of asphaltic concrete over 8 inches of base provides an adequate section.

Example No. 2:

An unimproved roadway in the central mountains, long used by local traffic, is to be regraded and paved. The plan is to use a full depth asphalt section over a compacted subgrade protected by a rubber-asphalt membrane. The subgrade soil is a highly plastic sandy clay with some gravel. The physical data is:

Soil Classification: A-7
LL = 50
PI = 25
Percent -200 = 78
R value = 10
Lab CBR = 3.0

A 20 year ADT of 10 is to be used for the design. The environmental analysis revealed a varying depth of clay from 1 to 3 feet over moderately weathered granitic rock. Immediately after compaction, four refraction seismic lines were performed, each showing a V_c of about 1750 fps in the upper layer of the subgrade. At the same time, testing revealed σ_d = 132 pcf, w_c = 6%, and S = 65%. A 12 inch diameter plate bearing test resulted in E ≃ 35,000 in the 30 to 50 psi range.

Assuming a value of Poisson's ratio of 0.33 for the partially saturated clay results in E_{seis} = 57,000 psi. Because of the high degree of saturation and density, it is believed that $E_s \simeq 0.3 E_{seis}$. Thus, an uncorrected E_s = 17,000 psi is selected.

Because of the high plasticity of the clay, 0.8 is considered an appropriate repeat load reduction factor. The ERF can be evaluated considering the following factors:

Unfavorable	Favorable
• rainfall = 20 inches per year	• full depth asphalt layer
• subgrade is a slow draining clay	• membrane seal
• clay may be slightly expansive	• subgrade compacted wet of optimum
	• good lateral drainage incorporated in design

Because the membrane seal and full depth asphalt are expected to be very effective, an ERF of 0.9 is selected. Then:

$$E_3 = (0.8)(0.9)(17{,}000)$$
$$E_3 = 12{,}200 \text{ psi}$$

From Figures 19 and 20 (Santucci's work), it is seen that on the basis of fatigue, 4½ inches of asphaltic concrete is needed, and on the basis of rutting, 5 inches is needed and therefore governs.

It should be noted that if one applied the physical properties of the soils in the two examples to the Arizona DOT Soil Strength Correlation Chart, E moduli of 4100 and 2800 psi, respectively, would be indicated.

The value of E_3 derived in the manner described herein is appropriate for use not only with the

Chevron Research or Kentucky Flexible Pavement Design Systems, but with any procedure employing a layered elastic model to describe the pavement response. The focus of the recommended procedure has been almost entirely concerned with highway pavements. An airfield could be evaluated in exactly the same manner, provided the appropriate size of loaded area and the level of stress are incorporated into test procedures.

4.4.2 Pavement Evaluation and Overlay Design

The incorporation of the findings concerning subgrade modulus derived from this study into pavement rehabilitation procedures is at least as important, if not more so, than those involving new construction. It appears that in the coming years, an increasingly larger portion of the Department of Transportation budget will be devoted to the maintenance and reconstruction of existing roadways.

Arizona Department of Transportation is currently conducting a perpetual inventory of pavement conditions throughout the state, using the Dynaflect. A computer program, employing Benkelman Beam equivalencies and Soil Support Value, is used to provide a qualitative evaluation of the pavement condition and recommended overlay thickness. Possibly in the future, one of the many emerging rational pavement overlay methods will be adopted and tailored for used in Arizona.

What are considered to be four of the most promising overlay design techniques are reviewed in Table 21.

The common denominator of all four methods is the use of NDT deflection bolws and layered elastic analyses to compute moduli of the three (or more) layers. With thicknesses and stiffness parameters assigned to each pavement layer, empirical and/or mechanistic procedures are then used to determine the number of load repetitions remaining before a condition deemed unacceptable is reached. Thickness of an asphaltic concrete overlay required to extend the life of a pavement is also determined in a rational manner.

The authors believe that the BISAR and ELSYM5 programs are excellent tools, but would prefer that they be modified to accept a value of E_3 determined by in-situ testing. The element missing from all of the methods described in Table 21 is a clear definition of the elastic modulus of the subgrade in terms of constraints upon its meaning. In every case, E_s is regarded as an intrinsic property. Generally, E_2 (modulus of the base course) is supposed to be a function of E_3 and the base layer thickness (see reference 172). Successive iterations then establish a set of moduli, E_1, E_2,... E_n, which mathematics confirm will produce the deflection basin found by the NDT device.

The Austin Research Engineers Method includes a subsystem which correlates the mathematically determined subgrade modulus with the "true" dynamic

Table 22

Summary of Pavement Overlay Design Methods

Name of Method	Technique for Measuring Condition of Pavement	Means of Characterizing Subgrade Stiffness
Austin Research Engineers-Treybig Reference (173)	Nondestructive Testing (Dynaflect, Road Rater, Benkelman Beam or LaCroix Deflectograph).	Resilient Modulus M_R from triaxial testing of undisturbed core samples, and calculated from NDT results. *Employs the computer program ELSYM5 (Reference 174) developed at the University of California to relate M_R to pavement deflection. *Irritative procedure to develop the true design subgrade modulus, using both laboratory and in-situ response data.
Shell Method Reference (82)	Maximum deflection determined by Falling Weight Deflectometer.	Uses BISAR computer program to produce a series of graphs which, using maximum deflection δ_o and a deflection bowl parameter Q_a as input, allow the user to select E_1, E_2 and E_3. ν is assumed to be 0.35 for all layers. E_2 is a function of E_3 and thickness of base h_2, according to criteria established by Heukelom and Klomp (Reference 172).
Danish Method Reference (175)	Dynamic plate load test using Falling Weight Deflectometer.	Uses CHEVRON n-layer program to produce a series of graphs, similar in function to the SHELL curves. They are based on a two layer system, and a layer equivalency procedure is required to extend to three layers.
New South Wales Method Reference (176)	Benkelman Beam Deflections.	Benkelman Beam deflection values are converted to E_s on the basis of two spreadability indices (similar to Vaswani's S). Authors strongly recommend that some values be verified by in-situ CBR testing.

modulus from cyclic triaxial testing. The authors would prefer to substitute some in-situ test, such as CBR for the triaxial, as was suggested by Rufford (177).

The authors would suggest that an overlay design method be developed for Arizona pavements which incorporates the best features of the presently available rational methods, and which is strongly rooted in in-situ testing. An outline of such a procedure is given in Figure 78.

Based on a statewide inventory of pavement conditions and expected traffic, a list of priorities would be set in a planned program of rehabilitation. The sections of highway to be overlaid or rebuilt should be divided into subsections according to stiffness and environment. Dynaflect tests should be run at frequent intervals to select representative test stations where intensive NDT coverage would be provided. At the test stations, an excavation of pavement and base should follow NDT so that some form of in-situ testing directly on the subgrade can be performed. From these, a statistically valid E_s can be developed.

A computer program can be used to fit the deflection bowl to a layered system with stiffnesses E_1^*, E_2^* and E_3^*. Poisson's ratios and layer thicknesses, as well as the shape of the deflection bowl, are the inputs. The computer value of E_3^* should then be compared with the in-situ value E_s. If they

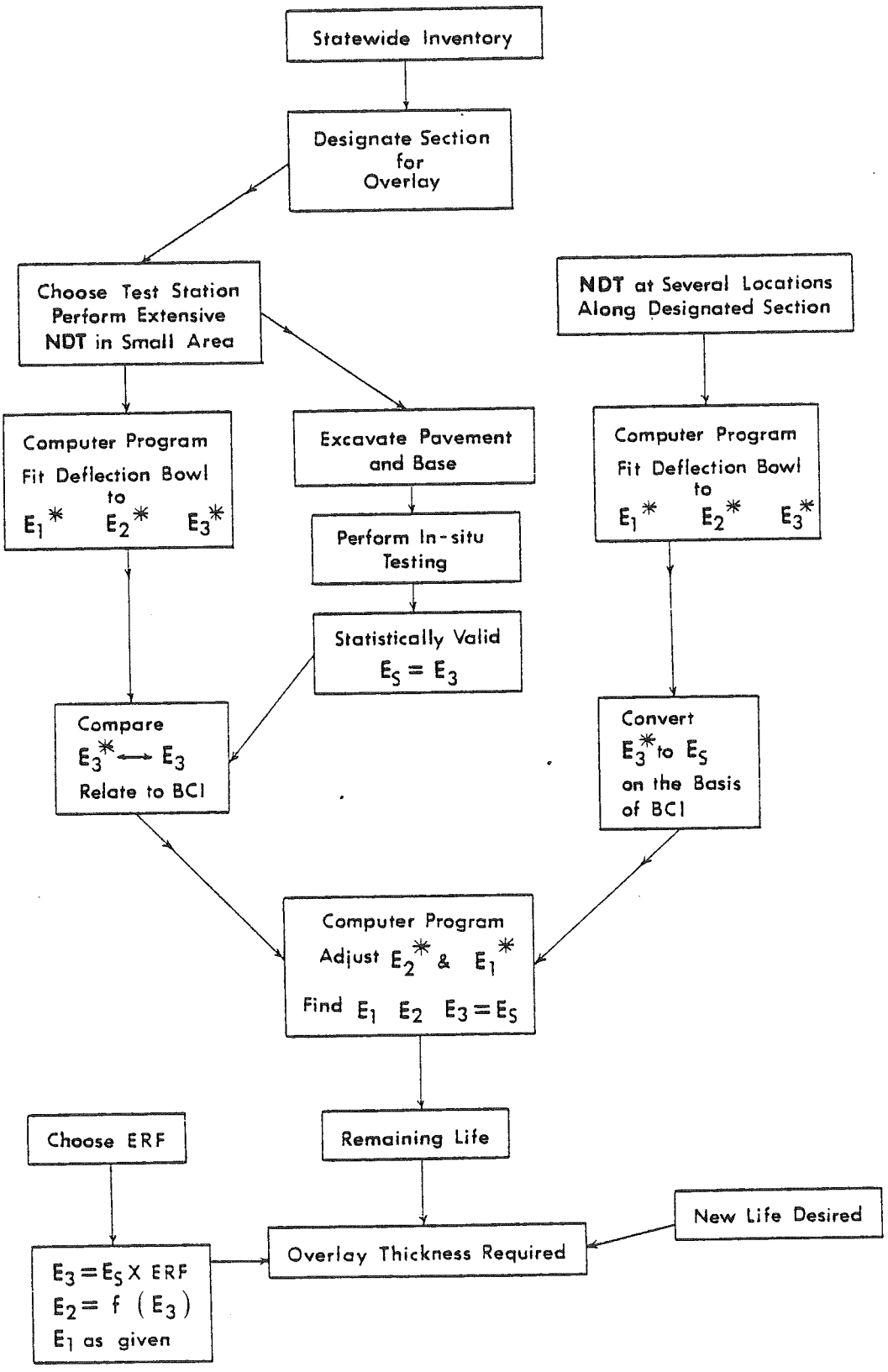


Figure 78. Flow Chart for Pavement Overlay Design

are greatly different (which is likely), an adjustment must be made. This is necessary because E_3 must indicate the approximate response of a standard wheel load under a layered pavement.

In most layered analysis programs, the stiffness of the base is deemed to be a function of the stiffness of the subgrade and the base thickness. Huekelom and Klomp (173) state that $E_2 = k.E_3$ where $k = 0.206 \times h_2^{0.45}$ (h_2 in mm and $2<k<4$). While it certainly is reasonable that E_s (and possibly E_1) is highly dependent on the stiffness of the infinite subgrade, it is felt that a rigid relationship between E_2 and E_3 is not workable. It should be possible to adjust E_2 and E_1 (within limits) so that along with the value of E_s, the actual deflection bowl is predicted.

The limits of E_1 and E_2 are important. E_2/E_3 should range between 1 and 4. (Many investigators state that the ratio of E_2/E_3 is between 2 and 4, however, the authors believe that in the case of very stiff subgrades, such as found in Arizona, ratios near 1 are possible.) The E modulus of asphaltic concrete is usually determined in the laboratory by such means as dynamic triaxial testing. The authors question, however, whether a finite core of bituminous bound material, when separated from its layered environment, will respond to loading in the same way as an infinite (in two directions) layer which must undoubtedly be affected by its support medium.

In summary, it is suggested that the deflection bowl and the in-situ value of E_s from a test section be used to calculate E_2 and E_1. More importantly, however, the computer value of E_3, the determined value of E_s and some index of the subgrade's response as reflected in the shape of the deflection bowl (such as Base Curvature Index) must be related. On the basis of this relationship, NDT deflection bowls from other locations throughout the designated section can be analyzed. E_3 from the multilayer analysis is converted to E_s which then becomes the sanctioned value of E_3. E_1 and E_2 would then be calculated to fit the deflection basin.

Following these steps, one of the available computer programs would be employed to determine the remaining life of the pavement at various locations and the thicknesses of overlay required.

The overlay design procedure discussed in this section is merely an outline. Specific details regarding in-situ and nondestructive testing as well as selection and manipulation of computer programs would necessarily be developed by the user agency. The outline given above is flexible enough to allow several alternative approaches. The key to any successful rehabilitation design program is accurate characterization of the material's properties in their functional environment. Regarding the subgrade, the point cannot be overemphasized that to describe a soil's response through a Young's

modulus (E) is an engineering simplification, and to be meaningful, the assigned value of E_s must apply to conditions of loading imparted by a standard wheel print through a layered section and upon the subgrade.

4.5 Summary of Conclusions and Recommendations

Major conclusions and recommendations developed in the course of this field study and literature review are as follows:

- The characterization of soil response by use of a Young's modulus, E, is an approximation. Few soils exhibit truly elastic behavior except at very low strain levels. Nevertheless, the use of methods of analysis employing E_s is a highly effective approach for pavement design and many other geotechnical engineering problems. Rational layered system methods of analysis are believed to be a major improvement over methods previously employed in Arizona. Among other advantages, these methods lend themselves to a much more analytical consideration of environmental effects. However, in applying elastic methods of analysis, it is extremely important to consider the geometry of the loaded area or stressed zone, the level of strain, variation of stiffness with depth, and effect of repeated loads.

- The load response of subgrades in Arizona is strongly influenced by factors not present in many other areas. The internal stresses caused by desiccation create a

high degree of overconsolidation, resulting in high horizontal stresses. Many soils possess a substantial degree of calcareous cementation. Recementation often occurs rapidly in calcareous fill materials. The process of soil formation in the desert environment often leads to moisture sensitive soil structures and large and erratic lateral variations in stiffness.

- Major savings in costs can often be effected by employing elastic methods of pavement design analysis in Arizona conditions and determining E_s by in-situ methods.

  For instance, if a design value of E_s = 5000 psi were used where a modulus of 15,000 psi actually represents the subgrade stiffness for a highway where the 20 year design ADT is 100, 7½ inches of asphaltic concrete over 15 inches of base would be indicated where 5 inches of asphaltic concrete over 10 inches of base would be determined with the use of 15,000 psi. Pavement structure thicknesses are highly sensitive to E_s over the range of 5000 psi to 25,000 psi. The majority of Arizona soils fall within these limits.

- In general, laboratory testing of Arizona soils is not the most effective means of evaluating response of pavement subgrades during their service lives. Index and classification tests can provide the basis for a relatively accurate evaluation of E_s for clean, uncemented cohesionless sands and gravels. However, laboratory tests have serious limitations for soils possessing a significant fraction of silt and/or

clay. Various laboratory tests designed to measure soil stiffness, including cyclic triaxial shear tests, are not believed to be an effective means of evaluating subgrade stiffness for most Arizona soils. After subgrade construction, the soils typically dry below compaction moisture contents. The complex internal stresses created by this desiccation are unlikely to be duplicated in the laboratory. The recementation of calcareous fills, which provides a major element of stiffness, is also unlikely to be duplicated in the laboratory. A large percentage of Arizona soils contain a substantial fraction of gravel and calcareous concretions which make necessary relatively large specimens in order to obtain the best quality laboratory test data.

- A procedure wherein a preliminary design is made on the basis of a limited testing program to provide a basis for preliminary cost estimates is recommended.

  In-situ testing of the completed compacted subgrade is then recommended to provide the basis for final design. Because of the wide lateral variations in subgrade stiffness, a relatively large number of tests will usually be necessary to establish design subgrade stiffness to a sufficient degree of accuracy. For highway pavements, E_s should be considered as corresponding to the response of a 30 inch diameter plate subjected to a 30 to 50 psi static load. Field tests recommended for evaluation of E_s are the in-situ CBR, refraction seismic and Dynaflect.

These three recommended methods enable efficient performance of a large number of tests.

In that the 30 inch plate bearing tests provide the reference criteria for evaluating the other tests, it is recommended that a program of periodic plate bearing tests be incorporated into the routine investigation procedure. With this program, in conjunction with routine classification testing, experience can be accumulated to develop the basis for more accurate preliminary estimates of E_s.

New in-situ testing techniques may be perfected in the near future which will afford a more efficient and accurate means of determining E_s. Development of new technology should be continuously monitored.

- As a part of the design procedure, prior to the use of test data in analysis, it is recommended that the values of Young's modulus be adjusted to consider the level of strain, environmental effects and the effects of repeated dynamic loads by the procedures presented herein.

- Seasonal variations in moisture and temperature affect subgrade response in Arizona to a substantially lesser degree than in many other areas. Environmental effects are usually much less significant than naturally occurring spatial variations in cementation and stress fields.

- Owing to calcareous cementation and the internal stresses due to desiccation, the elastic modulus of Arizona subgrades is usually substantially higher than the "handbook" values widely published in pavement design literature. The use of these "handbook" values usually leads to conservative designs in most areas of the state.

- For most areas of Arizona, the technology exists to maintain subgrades at or below the compaction water contents (or at low natural moisture contents), enabling the use of correspondingly higher subgrade moduli in design. The effectiveness of these measures has been demonstrated by a large and diverse body of international literature reporting moisture equilibria beneath pavements in arid regions. This data, accumulated over many years, is confirmed by case histories in Arizona and New Mexico.

- Careful verge drainage measures can sometimes protect pavements with permeable granular bases and justifies the application of a relatively high environmental reduction factor (ERF) to the subgrade modulus. However, a substantial calculated risk of environmental deterioration of subgrade stiffness is inherent with the use of clean granular bases. The use of various combinations of full thickness asphaltic concrete, stabilized bases and membrane protection of the subgrade (in conjunction with proper drainage) provides a very high assurance of subgrade moisture stability and justifies the use of an ERF at or near 1.0. The use of these

methods is strongly recommended where moisture sensitive subgrades are involved.

It is emphasized that the methods presented herein were developed specifically for Arizona conditions. Those evaluating the methods for use in other areas are cautioned to very carefully assess the similarity of the soil and climatic conditions they are dealing with to those covered by this study.

Although major advances in pavement design procedures have been made in recent years, it is emphasized, in conclusion, that engineering judgment remains the most important element in achieving good design. In order to fully exploit new pavement design technology, it is the authors' opinion that it is essential to assess every project on an individual basis and design a program for determining subgrade modulus using the methods most suited to the particular conditions involved.