Final Report

Use of the Dynamic Cone Penetrometer to Verify and Correlate with Soil Subgrade and Aggregate Base CBR Values for the Purposes of Design of Low Volume Traffic Asphalt Concrete Roadways

ODOT Project No. 2141

March 1999

Submitted to

Oklahoma Department of Transportation
Oklahoma City, Oklahoma

Submitted by

ERES

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Use of the Dynamic Cone Penetrometer to Verify and Correlate with Soil Subgrade and Aggregate Base CBR Values for the Purposes of Design of Low Volume Traffic Asphalt Concrete Roadways		5. Report Date March 1999 6. Performing Organization Code
7. Author(s) M.E. Ayers and D.A. Steele		8. Performing Organization Report No.
9. Performing Organization Name and Address ERES Consultants, Inc. 505 W. University Avenue Champaign, IL 61820-3915		10. Work Unit No. (TRAIS) 11. Contract or Grant No. ODOT Project No. 2141
12. Sponsoring Agency Name and Address Office of Materials and Research Oklahoma Department of Transportation 200 NE 21st Street Oklahoma City, Oklahoma 73105-3204		13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

The use of the Dynamic Cone Penetrometer (DCP) has increased dramatically over the past several years. The primary reason for the increased use stems from improvements to the device and the development of correlation equations for many strength/deformation parameters. However, no correlations were available to relate DCP penetration index (PI) to resilient modulus , and there were no specific PI/CBR correlations for Oklahoma soils.

This research was conducted to develop correlations between DCP penetration index values and soil strength parameters, including CBR and backcalculated resilient modulus. Correlations are presented for clay, silt, and sandy soil subgrades, and these correlations will facilitate the design, construction monitoring, and evaluation of low-volume roadways. The results of the study can also be used in the preliminary design and evaluation of asphalt concrete and portland cement concrete pavements.

17. Key Words Dynamic Cone Penetrometer (DCP), asphalt concrete pavement, low-volume roads, California Bearing Ratio (CBR), resilient modulus, pavement performance		18. Distribution Statement No restrictions	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No of Pages 65	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

ACKNOWLEDGMENT

This study is sponsored by the Oklahoma Department of Transportation through the Materials and Research Division. The authors wish to express their gratitude to Mr. Lawrence Senkowski, Mr. Curt Hayes, and the numerous ODOT Division personnel who assisted with the site selection, testing, and traffic control.

DISCLAIMER

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GUIDELINE PREPARATION

This project is being conducted under the supervision of Dr. Michael Ayers. Other project team members contributing to these guidelines include Douglas Steele, Shreenath Rao, Kelly Smith, and Robin Jones.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Problem Statement

The design and evaluation of low-volume roadways (both surfaced and unsurfaced) requires determination of the in situ support conditions in terms of the California Bearing Ratio (CBR) or resilient modulus (M_r). The dynamic cone penetrometer (DCP) can be used to estimate the support (strength/deformation) characteristics of a variety of materials ranging from fine-grained soils to unstabilized granular bases. However, there are no correlations relating DCP penetration index (PI) values to M_r and no specific PI/CBR correlations for Oklahoma soils.

Objective

The objective of the research is to develop correlations between DCP penetration index values and soil strength parameters, including CBR and backcalculated resilient modulus. Correlations between these parameters are developed for clay, silt, and sandy soil subgrades, and these correlations facilitate design, construction monitoring, and evaluation of low-volume roadways. The results of the study can also be used in the preliminary design and evaluation of asphalt concrete (AC) and portland cement concrete (PCC) pavements.

Background and Significance of Work

The use of the DCP has increased dramatically over the past several years. The primary reason for the increased use stems from improvements to the device and the development of correlation equations for many strength/deformation parameters.

Cone Penetrometer Testing

Cone penetrometer testing (CPT) encompasses many types of equipment and procedures. "Static" cone penetrometers, such as the Waterways Experiment Station (WES) cone, are advanced into the material being evaluated at a relatively slow rate. "Dynamic" cone penetrometers are advanced under impact loading.

CPT has developed in response to various problems or uncertainties associated with in situ evaluations. In response to the need for an expedient and "accurate" device to estimate the bearing capacity of subsurface soils, Sir Stanford Fleming [1] developed a simple form of static cone penetrometer in the 1870s. Numerous refinements of this basic concept led to the development of the WES cone and similar devices in current and widespread use.

The origin of DCP testing (specifically, standard penetration testing) is attributed to Colonel Charles Gow in 1902. In this case, an early form of the split spoon sampler was driven into the soil mass by a falling weight. Many offshoots of this basic concept are in use today, including the Standard Penetration Test, DCP, and similar devices.

All forms of in situ evaluation techniques have advantages and disadvantages that preclude their use in some circumstances. This study focuses on the DCP as the best available in situ evaluation technique to estimate the CBR and M_r of subgrade soils and granular materials. The advantages of the DCP include the following:

- The existence of a large database and correlations for determining the strength and deformation characteristics of subgrade soils and granular materials.
- Proven feasibility as an evaluation technique and design aid.

- Inexpensive to construct, operate, and maintain.
- Ability to differentiate and characterize subsurface layers.
- Utilization by personnel with minimum training.
- Assessment of in situ variability by conducting numerous short-duration tests (the average time to conduct a test is less than 5 minutes).

Historical Development of the DCP

Development of the handheld DCP is credited to A. J. Scala in the mid-1950s.[2] The DCP was developed in response to the need for a simple and rapid device for the characterization of subgrade soils. Flexible pavement design schemes used in Australia in the 1950s did not specifically require in situ strength tests of subgrade soils because available test methods were too complex and time-consuming.

The prototype DCP developed by Scala included a 20-lb drop hammer falling through a distance of 20 in. A 0.63-in diameter rod calibrated in 2-in increments was used to determine the penetration (generally after five weight drops). This configuration used a case hardened cone tip with a 30-degree included angle. Scala credits early developmental work by Haefeli, Amberg, and Von Moos [3] as the basis for this design.

Selection of the weight and drop height seemed somewhat arbitrary, possibly based on what the technician performing the evaluation could easily transport and use. The maximum penetration depth was approximately 30 in, limited by the length of the calibrated rod. Flexible pavement design and analysis procedures at that time did not require characterization of soils to depths greater than approximately 30 in.

For evaluations in which very stiff materials were encountered or where evaluation to greater depths was desired, removal of material to the depth of interest was suggested. The DCP was "driven" to the desired depth and a plot of the blows per inch versus depth was prepared. The strength of the soil was then based on the penetration/depth relationship.

Implementation of the DCP was facilitated by the development of correlations to CBR. In situ CBR and DCP tests were conducted at numerous sites with a broad range of subgrade soil types. A second series of tests were conducted to establish a relationship between DCP results and the bearing capacity of soils as estimated by a static cone. Shear strength for these fine-grained soils was also approximated by a correlation between CBR and shear strength.[4,5]

The developmental work by Scala proved the feasibility of the DCP as a design aid. Scala also proposed a pavement design scheme based on DCP/CBR correlations. An existing empirical relationship between pavement thickness and CBR was used to approximate the required thickness using DCP test results. The DCP was adopted by the Victoria Country Roads Board and gained relatively widespread acceptance.

The "next generation" DCP was developed by D. J. van Vuuren for the National Institute for Road Research, Pretoria, in the late 1960s.[6] The basic design and operating principles are identical to Scala's DCP. The primary differences lie in the dimensions of the apparatus. Van Vuuren's DCP consisted of a 22-lb weight falling through a distance of 18.1 in, a 0.63-in diameter penetration shaft calibrated in millimeters (or some multiple thereof), and a hardened cone tip measuring 0.79 inches in diameter with an included angle of 30 degrees. The DCP apparatus was designed to evaluate depths to 39 in. No basis for the revisions to the weight or the drop height are provided; it may be speculated, however, that several of the revisions were made with respect to changing the English units of measurement to metric.

Development of the DCP in Pretoria was to alleviate the problems associated with performing "field" CBR evaluations. Several alternatives, including the WES cone penetrometer, were evaluated. The DCP resulted in the best correlation to CBR and was therefore adopted for use.

Van Vuuren concluded that the DCP is well suited for the determination of CBR for a range of soil types. The DCP was thought to be applicable to soils with CBR values of 1 to 50. It was further concluded that the DCP was probably not suited for coarse-grained soils (coarse sand being the approximate upper bound). It was recommended that as the coarseness of the material increases, the number of DCP tests should increase to account for the higher variability.

The basic design of the DCP requires two persons for expedient operation, one person to perform the test, and another to read and record the data. A design modification in 1971, licensed to the South African Inventions Development Corporation, required only one operator.[7] The device was similar to its predecessors only in that a cone tip and calibrated shaft were advanced into the soil by a falling weight. In this case, the falling weight consisted of a sliding tube arrangement. Detailed specifications for this design are not available, but they are assumed to be similar to the van Vuuren DCP. No further mention of this device is evident in the literature.

An extensive evaluation of the existing roads in the Transvaal in 1973 necessitated the need for a rapid evaluation device suitable for use in fine-grained soils and base course materials.[8] The Transvaal Roads Department adopted the DCP for use at that time. Approximately 2,000 tests were conducted in this study. The effects of soil type, plasticity, moisture content, and density were evaluated. Test results indicated that the DCP reacted in a similar manner to CBR when the aforementioned parameters are varied. Increasing density and decreasing moisture content generally results in an increased CBR value. The effects of soil type and plasticity are not well defined due to differences in gradation, mineralogy, particle characteristics, and so on.

In 1975, a definitive work regarding DCP technology was prepared by E. G. Kleyn for the Transvaal Roads Department, Materials Branch.[9] The results of the testing program initiated in 1973 [8] were used as a basis for this study. Van Vuuren's basic design was utilized in Kleyn's work; however, the weight was reduced to 17.6 lb and the fall height increased to 22.6 in. Two cone tips of 30- and 60-degree included angles were evaluated. Numerous factors affecting DCP performance in laboratory and field testing were determined and are summarized below:

- Variability may be attributed to human factors, mechanical factors, and material factors.
- The reliability of DCP test results depends on adherence to established test procedures and attention to detail. No general error can be established.
- Mechanical factors primarily affect laboratory evaluations.
- The 60-degree cone tip was found to be considerably more durable than the 30-degree cone tip, particularly when evaluating granular materials. The 60-degree cone tip was found to react more quickly to material changes since the material is analyzed over a shorter distance (i.e., the 60-degree cone is "shorter" than the 30-degree cone).
- The DCP will give reliable readings in coarse-grained materials provided no large stones are encountered directly under the cone tip. Variability depends on the maximum aggregate size and the distribution of the larger particles. Acceptable readings were obtained in gravels with a maximum aggregate size of approximately 3 in. Gradation, density, moisture content, and plasticity were the most important material properties evaluated.

Kleyn pointed out a number of alternate uses for the DCP, including the control of earthworks and pavements, evaluation of pavements, and design of pavements.[9] Many experimental parameters are discussed in this report, including mold size effects, sample density stratification, and cone tip configuration.

Developmental work was continued through the remainder of the 1970s. Research conducted by Bester and Hallat [10] and De Villiers [11] produced correlations between DCP test results and unconfined compressive strength for materials similar to those evaluated by Kleyn.

Kleyn continued work on DCP-related material and presented three papers on the subject in 1982.[12,13,14] The configuration of the device was identical to that used in the previous study, but the 60-degree cone tip was used exclusively. The first of these papers introduced the concept of using DCP test results directly as a pavement design input. DCP test results are frequently plotted in terms of the number of blows or weight drops (x axis) versus depth (y axis). The slope of the resulting "DCP curve" at any depth is indicative of the strength of the material at that point. In this paper, Kleyn introduced the concept of a Layer-Strength Diagram (LSD) in which the penetration per blow (x axis) is plotted against depth (y axis). This approach makes it much easier to distinguish changes in material strength and layer interfaces.

The second paper [13] introduced the preliminary concept of using DCP test results to design a thin-surfaced unbound gravel pavement structure. A pavement design model was developed and subsequently correlated with the Heavy Vehicle Simulator (HVS) results for a number of pavement sections. This paper introduces the concept of a DCP Structure Number (DSN), which is given by the following equation:

$$\text{Layer DSN} = h/DN$$

where h is the layer thickness and DN is the DCP test result in terms of in/blow.

The DSN is equal to the number of blows required to penetrate a layer. The pavement DSN is the summation of all the layer DSNs comprising the pavement structure. The limiting depth for a pavement DSN was determined to be 800 mm (i.e., the stresses in the pavement at depths greater than 800 mm are insignificant). If the percentage of DSN (x axis) is plotted against depth (y axis), a Pavement Strength-Balance (PSB) curve results. The resultant PSB curve may then be compared to the curve obtained from field evaluations in which good to failed pavements were evaluated with the HVS. DCP test results can therefore be used as a direct design input using the PSB curves for typical pavements.

The third paper concerned utilization of the DCP to determine the structural capacity of an existing pavement in regards to rehabilitation.[14] The PSB approach introduced in reference 13 is used as a basis for this work. The concept of a "strength balanced" pavement allows for a number of alternative designs. The application of this principle to determine the optimum rehabilitation scheme is discussed.

The DCP was not a widely accepted technology in the United States until the early 1980s. A study conducted by Yoder, Shurig, and Colucci-Rios in Indiana [15] mentioned the DCP as a technique suitable for the determination of in situ CBR. This study focused primarily on the use of the Clegg Hammer and presented the DCP/CBR relationship determined by van Vuuren. Additional work to verify this relationship was not conducted.

It is interesting to note that the configuration of the DCP utilized in Yoder's study is different than that used in any previous work. Although not specified, the weight is estimated to be 22 lb falling through a distance of 29.35 in. The cone tip appears to be the standard 60-degree included angle cone, although a rounded cone tip is suggested for characterizing the bearing capacity of sand.

A further investigation of the Clegg Hammer, conducted by N. W. Garrick in 1983,[16] used the DCP to estimate subgrade CBR values. Data regarding the DCP tests were not provided.

Kleyn and van Heerden [17] further defined the role of the DCP as a means to optimize pavement rehabilitation. This work is essentially a continuation of the methodology suggested in reference 14 and uses the concept of a Strength-Balanced Pavement (SBP). The optimal SBP considers traffic loading, pavement support conditions, pavement depth, and associated costs. The rehabilitation selection uses DCP test results as a direct input.

Concurrent studies conducted by Smith [18] and Smith and Pratt [19] in Australia focused on the development of further DCP/CBR correlations. A wide range of material types were evaluated, and a general correlation was developed.

Development and refinement of DCP/CBR correlations was undertaken by Livneh and Ishai [20] in 1985. The configuration of the DCP used in this and subsequent studies in Israel is identical to the South African version, except that a 30-degree cone tip is used. Several additional reports published in 1985 (in Hebrew) compared DCP results and a modification to the WES cone (commonly referred to as the Airfield Cone) to CBR tests.[21,22] Acceptance of these studies was limited due to the lack of widespread distribution.

In 1987, Livneh presented a comparison of 21 DCP/CBR correlations taken from numerous sources. Correlations from Australia, England, South Africa, and Israel were included [23]. This paper was particularly important in that it synthesized the work of most of the principal investigators working on the development of the DCP. An important conclusion stated in this paper was that the variability associated with DCP testing is less than that associated with field CBR determinations. The maximum coefficient of variation associated with DCP tests performed on A-4, A-6, and A-7 soils is approximately 23 percent. The maximum coefficient of variation for CBR tests on comparable specimens is approximately 32 percent. This conclusion was based on the examination of a large field and laboratory database.

Livneh [24] provided DCP/CBR correlations derived from an extensive number of evaluations conducted in Israel. Livneh and Ishai [25] presented a paper at the Sixth International Conference on the Structural Design of Asphalt Pavements in Ann Arbor, Michigan, in July 1987. The principles governing use of the DCP for highway and airport pavement evaluation were discussed. These concepts are presented in a concise manner and are well detailed, providing a firm foundation from which to conduct detailed DCP evaluations and subsequent analyses.

Harison published a report in 1987 [26] in which a number of DCP/CBR correlations were developed for specific material types (clay, sand, and gravel), as well as for generalized materials. This study presented a simplistic mathematical model of DCP penetration. The following relationships between material parameters and DCP test results were noted:

- Density - An increase in density results in a decrease in DCP test values.
- CBR - An increase in CBR results in a decrease in DCP test values.
- Moisture - The effect of moisture depends on material type.

Younger, Tham, and Sundersan [27] presented the results of DCP evaluations conducted on roads in Bangladesh and Thailand. In Bangladesh, the strength-balance method of design, as originally proposed by Kleyn,[14] was used to predict pavement life. A comparison with observed pavement life showed the validity of this approach.

The comparative study in Thailand focused on predicting pavement life based on Benkelman Beam deflections and the DCP strength-balance method. A backcalculated value of the elastic modulus (E) using an E/CBR (CBR estimated by DCP) relationship did not result in measured deflections equaling estimated deflections. Although the indirect calculation of E was unsuccessful, it does provide a basis for further development.

The DCP strength-balance pavement design method shown in Kleyn's earlier works relies on a graphical approach to produce strength-depth curves. A paper by Kleyn, de Wet, and Savage [28] in 1987 concerned the mathematical development of these strength-depth curves. The equations were developed for use by design engineers without access to a computer. Sample calculations are provided and are compared with the earlier graphical formulation of the curves.

De Beer, Kleyn, and Savage [29] published a report in 1988 concerning the development of a universal classification system for thinly surfaced flexible pavements. Nine pavement strength-balance classification categories are described. Methodologies to categorize pavements evaluated with the DCP are discussed. Approximately 275 tests were conducted on pavements previously evaluated with the HVS to validate the proposed classification scheme.

Livneh [30] validated a number of previously published correlations relating DCP and several other test methods to CBR. In situ CBR tests conducted in test pits were compared with DCP test results at the same location. The results of these tests indicate that the estimation of CBR using the correlation equations is valid. Layer differentiation, as determined by DCP tests, was found to be sufficiently accurate when compared to test pit evaluation.

McGrath et al. [31] reported satisfactory results with the use of the DCP. A significant correlation between DCP values and typical soil strength parameters was found. However, limitations on the accuracy of DCP evaluations when testing cohesive soils in a highly plastic state were noted.

Ayers [32] and Ayers and Thompson [33] established a relationship between the DCP value and shear strength of granular materials. A general correlation between the shear strength of granular materials and DCP values was not determined because of the overwhelming effect of confining pressure on the shear strength. The correlations developed in this study related the deviator stress at various confining pressures and the DCP penetration rate through the use of multivariate linear regression analyses.

Ayers, Smith, and Thompson evaluated the relationship between in situ DCP values and laboratory shear strength and resilient modulus. The results of the study were inconclusive in regards to M_r; however, the shear strength correlations were confirmed for a variety of materials.

Hassan,[34] in a study conducted at Oklahoma State University under the direction of Dr. Ayers, investigated the parameters influencing DCP behavior as they relate to M_r. The effects of moisture, density, relative confinement, and other material characteristics were evaluated for a variety of fine-grained soils and granular materials. The results of this study serve to confirm many of the findings stated above. In addition, the likelihood of developing a M_r/DCP correlation was proved, particularly for granular materials.

The Minnesota Department of Transportation has conducted or sponsored extensive soil evaluation studies in recent years.[35, 36] Many of these studies focused on in situ evaluations of subgrade and base materials. These studies prove the adaptability of the DCP to widely varying conditions and material types. However, development of a realistic M_r/DCP correlation is still lacking.

This discussion of the historical development of the DCP has been somewhat abridged for clarity. The advancement of DCP technology may be attributed primarily to studies conducted in Australia/New Zealand, South Africa, and Israel. As the DCP technology base continues to expand, widespread acceptance is assured.

Current Utilization of the DCP

The DCP has gained increased acceptance in the last decade for a number of reasons, including the following:

- It is adaptable to many types of evaluations.
- There are no currently available rapid evaluation techniques that allow the characterization of such a broad range of material types (fine-grained soils to granular materials).
- DCP testing is economical.
- Numerous correlations exist that permit the estimation of CBR, unconfined compressive strength, and other parameters.
- DCP-based pavement design schemes and computer programs for data reduction and analyses have been continuously refined.

Thirteen countries have established standards for DCP use and are evaluating improvements to the apparatus and correlation equations. At the First World Symposium on Penetration Testing, it was suggested that the DCP apparatus and test procedures be standardized. The numerous variations in the DCP create problems when using the previously developed correlations.

The DCP is now widely accepted in the United States and is in use by many state departments of transportation, the U.S. Air Force, U.S. Army, and numerous others. The DCP is used in preliminary design, pavement evaluation, construction control, and as a stand-alone design method for low-volume roads using the strength-balance approach previously discussed.

TEST METHODOLOGY

The test methodology used in this study is divided into three phases: field data collection, data analysis, and development of the correlations.

The field data collection consisted of DCP, FWD, and CBR testing at seven test sites, chosen to be representative of different soils commonly encountered in Oklahoma. Included in these seven sites were a flyash stabilized subgrade and a highly compacted granular base material. At each test site, approximately 30 DCP and FWD tests were conducted at the same stations (locations). Following this, two to six samples were extracted at each site to determine in situ moisture and to provide sufficient materials to perform laboratory CBR tests. The in situ density was determined with a nuclear density gauge. The sample locations corresponded to selected DCP and FWD test stations.

Field data collection was divided into two phases. Initial testing (phase I) was conducted in March of 1997. The phase II testing was conducted in August of 1997 to determine the effect of moisture changes on DCP, M_r, and CBR test results.

DCP and FWD field data were analyzed by ERES. Laboratory CBR tests were conducted by Standard Testing at their facilities in Oklahoma City, Oklahoma. The DCP results were analyzed to determine the penetration curves for each test location and a representative PI per test point. The FWD data were collected and analyzed in terms of deflection measurements (deflection basins) that were used to backcalculate a subgrade modulus for each test location. Material samples taken at selected locations were used for one-point, soaked CBR tests conducted at field moisture content and density conditions in accordance with ASTM specification 1883-87.

Development of the DCP correlations was based on grouping of the test data by soil type and grouping all the data together in a common database. Correlations for phase I and phase II testing were developed and compared.

Site Information

Seven test sites were selected by the Oklahoma Department of Transportation Office of Materials and Research, as shown in figure 1. These sites included sand, silt, clay, a granular base, and a flyash stabilized clay. Five of the sites were chosen to represent the three broad categories of Oklahoma soil types typically encountered in road (highway) construction. The granular base and flyash stabilized sections were selected to determine the suitability of the DCP device in testing these material types. Table 1 provides information for each of the selected sites.

Sites 4 and 6 proved unsuitable for DCP characterization and are not included in the analysis. The FWD tests were conducted at these sites; however, the DCP either was not able to penetrate these materials or, when penetration was possible, the minimal penetration per blow made the test infeasible from a time standpoint. Site 7 also proved infeasible due to the presence of large rocks that either deflected the DCP or stopped penetration.

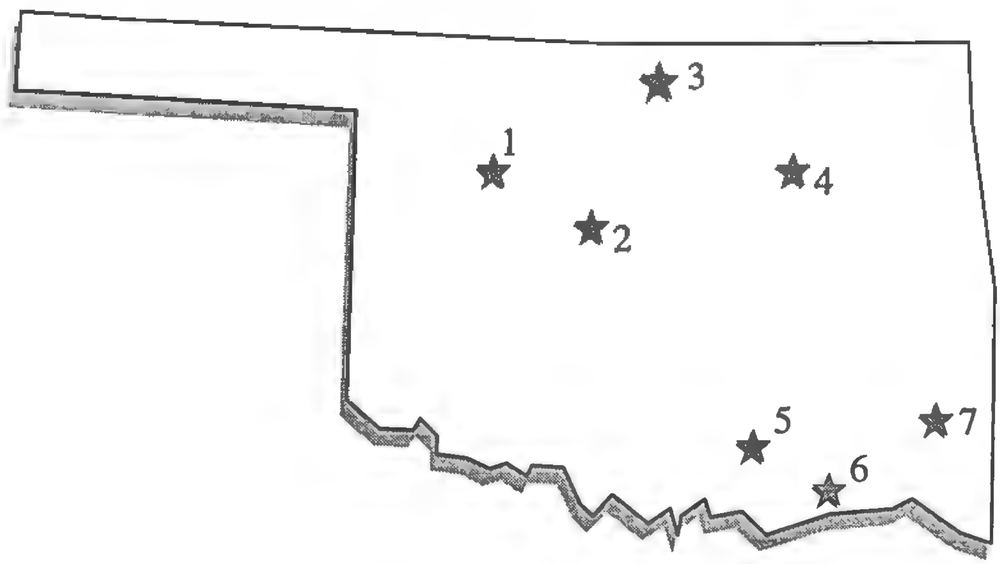


Figure 1. Site map.

Table 1. Site descriptions.

Site Number	Location (County)	Material Type	Soil Series	Surface
1	Major	Sand	Pratt	None
2	Kingfisher	Silt	Bethany	None
3	Kay	Clay	Lela	None
4	Creek	Aggregate Base	-	None
5	Choctaw	Clay	Burleson	Gravel
6	Choctaw	Clay - Fly Ash Stabilized	Roebuck	Oil and Chip
7	McCurtain	Clay	Goldston – Carnasaw-Sacul Association	Gravel/Fill

Dynamic Cone Penetrometer

DCP Test Methodology

Design details of the DCP used in this study are shown in figure 2.

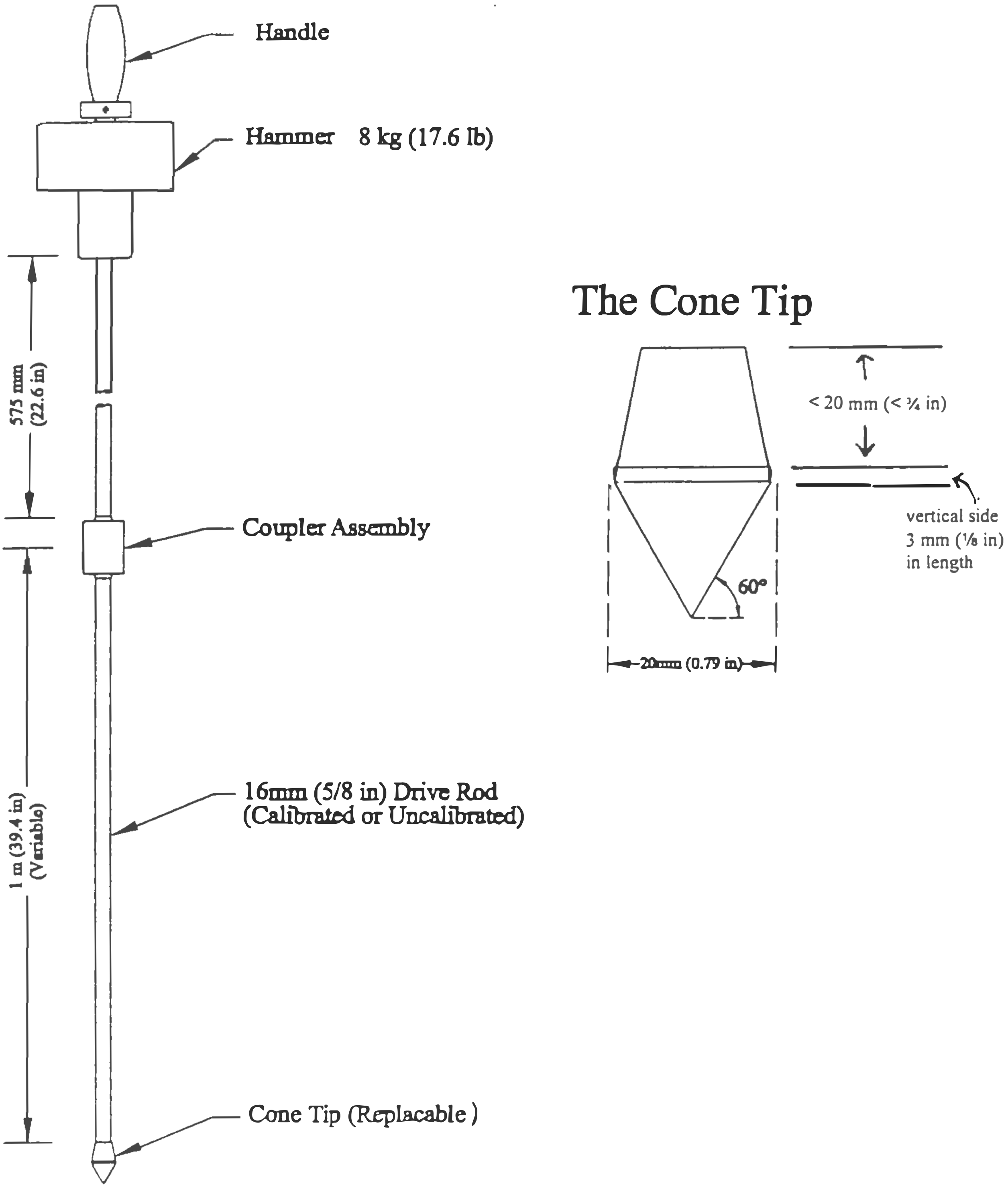


Figure 2. Design details of the Dynamic Cone Penetrometer used for testing.

At each location, testing began by placing the penetrometer cone tip on the subgrade and performing a seating blow to embed the cone in the subgrade material. The DCP apparatus was held in a vertical position throughout the test procedure. Subsequent blows were identical in that the hammer was raised until light contact with the handle was made. The hammer was then allowed to freefall until it impacted the coupler assembly, thereby advancing the cone tip. The resultant penetration was recorded in terms of the cumulative depth. Generally, blows were repeated until the total depth reached 30 in or until refusal.

The penetrometer data were analyzed by plotting the penetration curve (blow count vs. total depth) and determination of the penetration index, (penetration per blow) in units of inches per blow. Analytically, the PI is the slope of the penetration curve. This slope is dependent upon the stiffness of the material at each depth and can vary with depth at a given test point depending on the homogeneity of the subgrade material in the vertical direction. For example, very uniform roadbeds composed of a single soil type produce uniform, almost linear penetration curves. On the other hand, if the subgrade consists of several soil types or the same soil compacted in layers with differing densities, changes in the slope of the penetration curve are produced at the interface of the distinct layers.

The penetration curves for each test point and site were analyzed to determine the uniformity of the subgrade material. Based on the objectives of this project, it was desirable to characterize the subgrade by a single PI; therefore, the PI should only be averaged over the depth of material most representative of the roadbed. This analysis indicated 6 to 24 in was the appropriate range of depth. The PI at depths less than 6 in were influenced by increased compaction of the soil due to traffic or the presence of a granular layer. The PI determined at each test point served as the basis of the correlations with M_r and CBR.

DCP Test Results

Examples of representative penetration curves and PI values per depth curves for each test site are shown in figures 3 through 10. The curves for all test data are included in appendix A.

The PI results by station for each site are presented in figures 11 through 14. In each, the results for phase I and II testing are shown. The values shown are the average PI between the depths of 6 and 24 in.

The results for site 1 indicate PI values in the range of 0.1 to 0.5 in/blow for the phase I testing, with an overall mean of approximately 0.3 in/blow. In general, the results indicate a slightly stiffer subgrade for the portion between station 0+00 (County Road) and the end of the test section (station 10+00). PI values in this range indicate a soil of very good to good support. The phase II testing produced nearly identical results for the portion between station –5+00 and County Road; however, the remaining test points produced slightly higher results at the same location when compared to phase I testing.

Site 2 produced PI values in the range of 0.2 to 0.3 in/blow for phase I. The results were very uniform and correspond to a soil with good support. Phase II results were nearly identical for the same test points.

In the case of site 3, the PI produced the highest overall mean relative to the other sections, meaning the weakest subgrade of the four sites. The PI values range between 0.7 and 1.3 in/blow for phase I, with a mean of approximately 1.0 in/blow. As for phase II results, the portion between stations 0+50 and 5+50 produced nearly identical results, while the remaining section showed a significant decrease in penetration (a stronger response) for the same points tested in phase I. The magnitude of penetration for both phases is typical of a soil of very low bearing capacity.

Site 5 indicates a subgrade of high variability in the longitudinal direction. At the beginning of the section (stations 0+50 to 3+50), PI is in the range of 0.5 to 1.3 in/blow for both phases of testing, indicative of poor to very poor soils. From this point onward, the values are much lower, with a mean of approximately 0.4 in/blow. With the exception of a few points, the results of phase I and phase II are very similar.

Figure 3. Representative DCP penetration curve—site 1, phase I (sta 1+00).

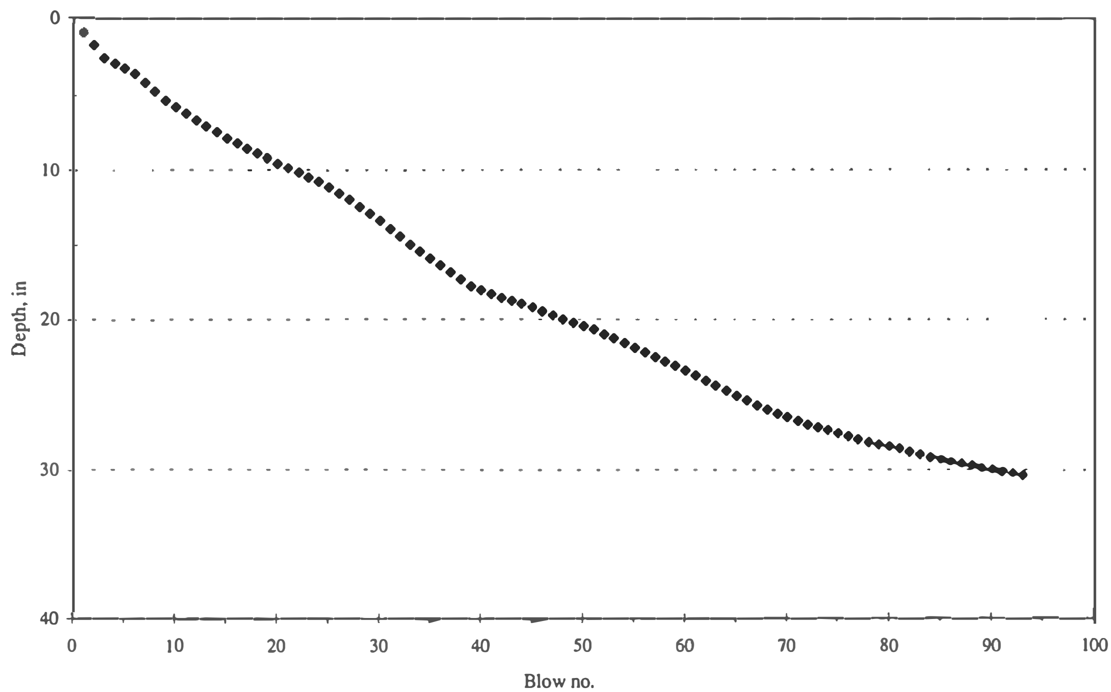


Figure 4. Representative Penetration Index vs. depth curve—site 1, phase I (sta 1+00).

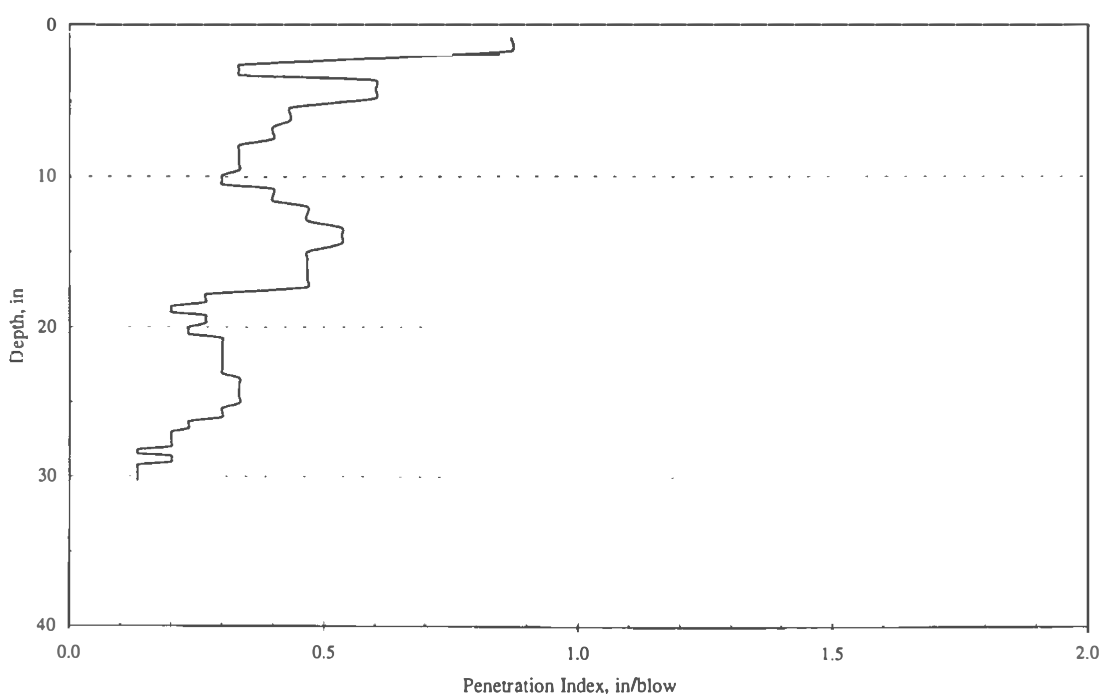


Figure 5. Representative DCP penetration curve—site 2, phase I (sta 5+50).

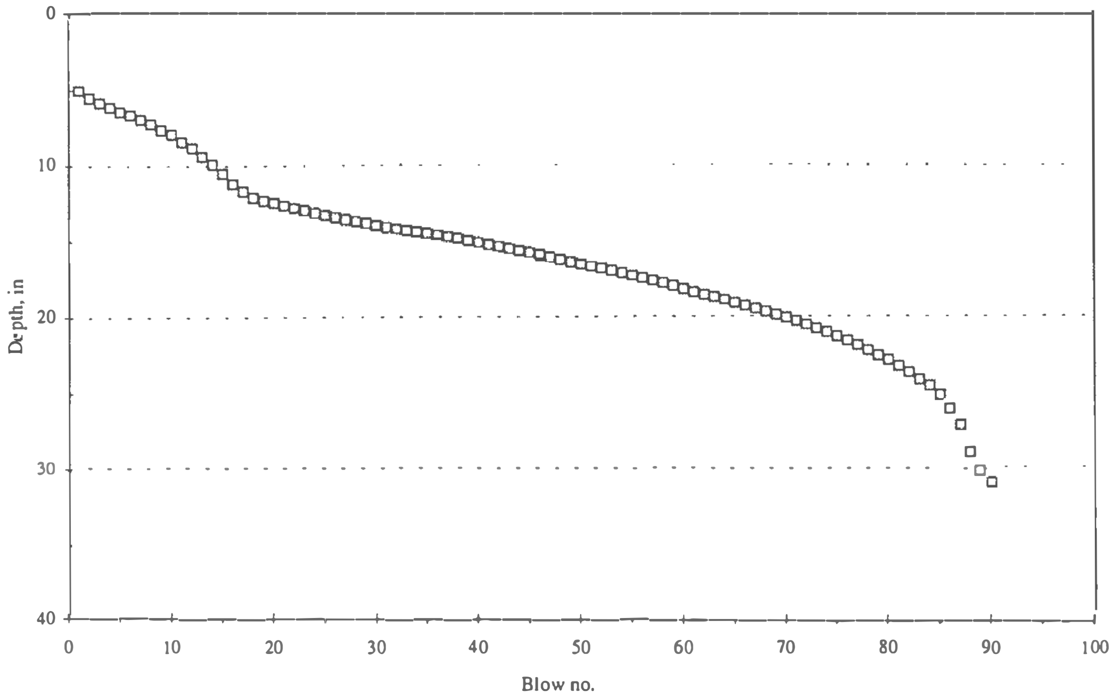


Figure 6. Representative Penetration Index vs. depth curve—site 2, phase I (sta 5+50).

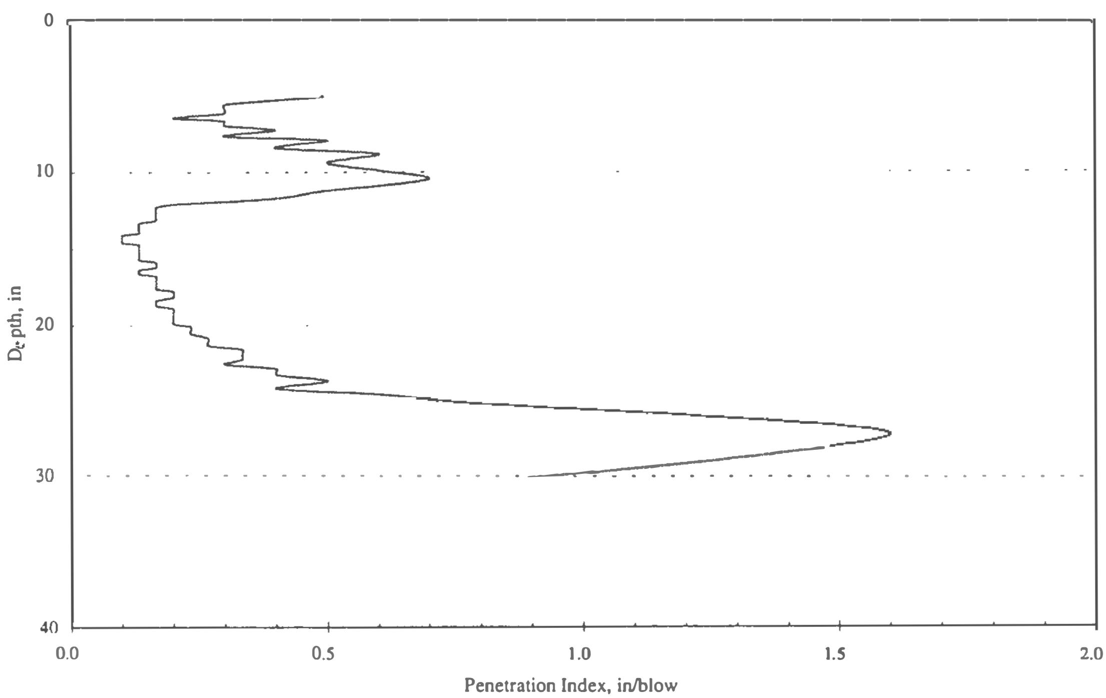


Figure 7. Representative DCP penetration curve—site 2, phase I (sta 1+00).

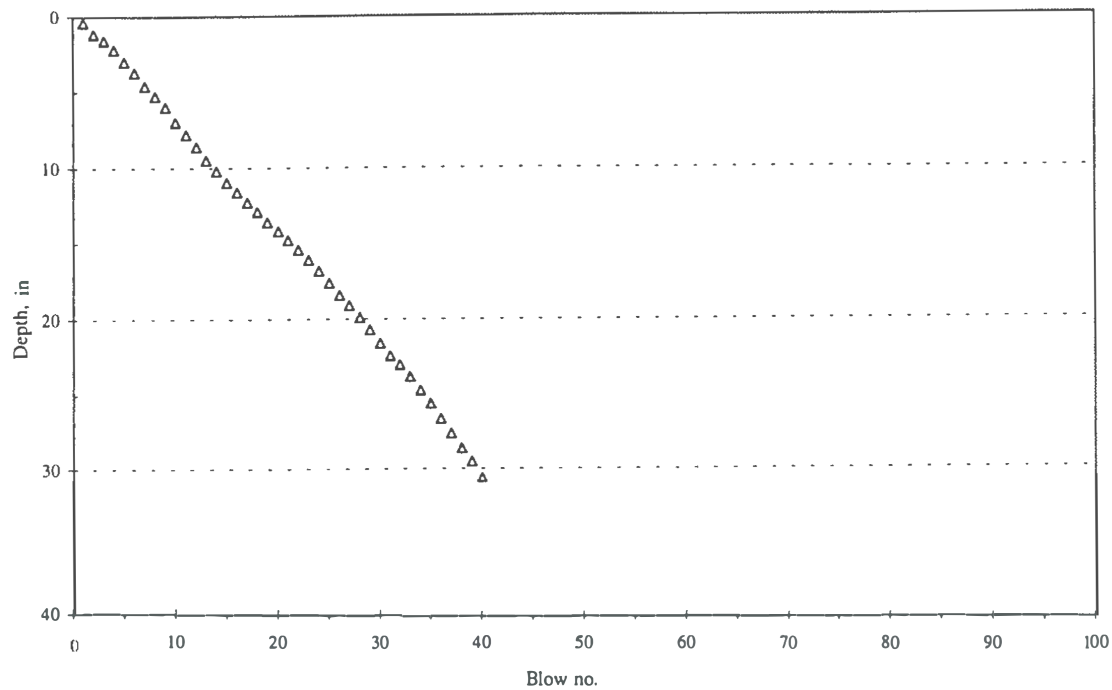


Figure 8. Representative Penetration Index vs. depth curve—site 3, phase I (sta 1+00).

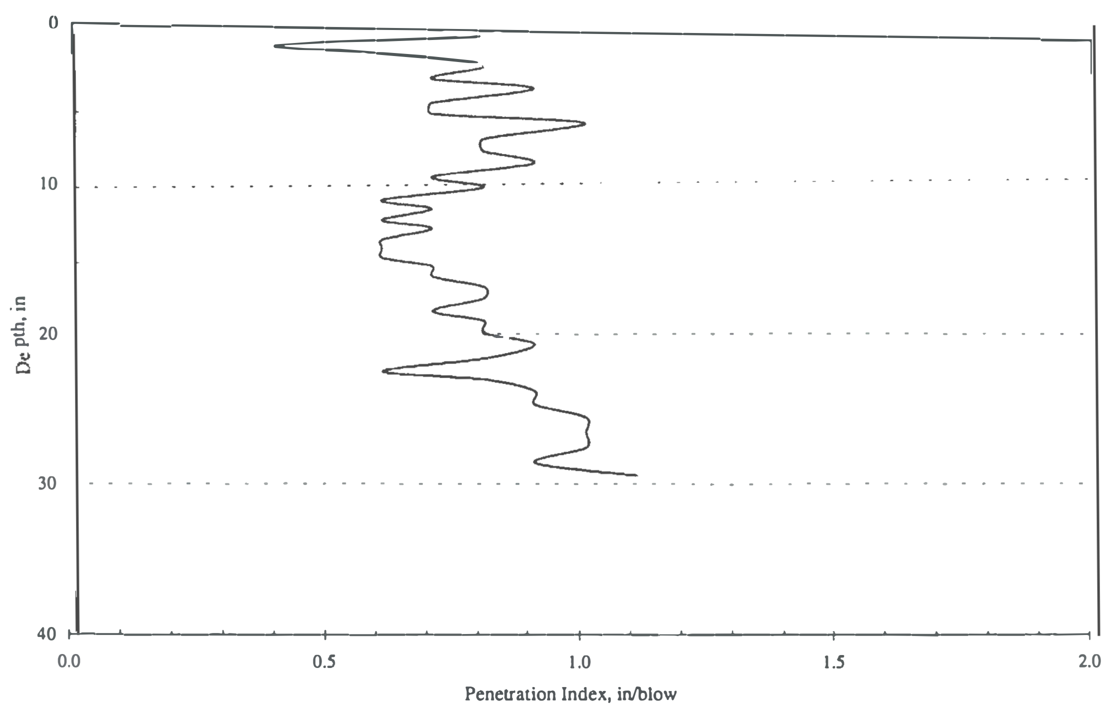


Figure 9. Representative DCP penetration curve—site 5, phase I (sta 11+00).

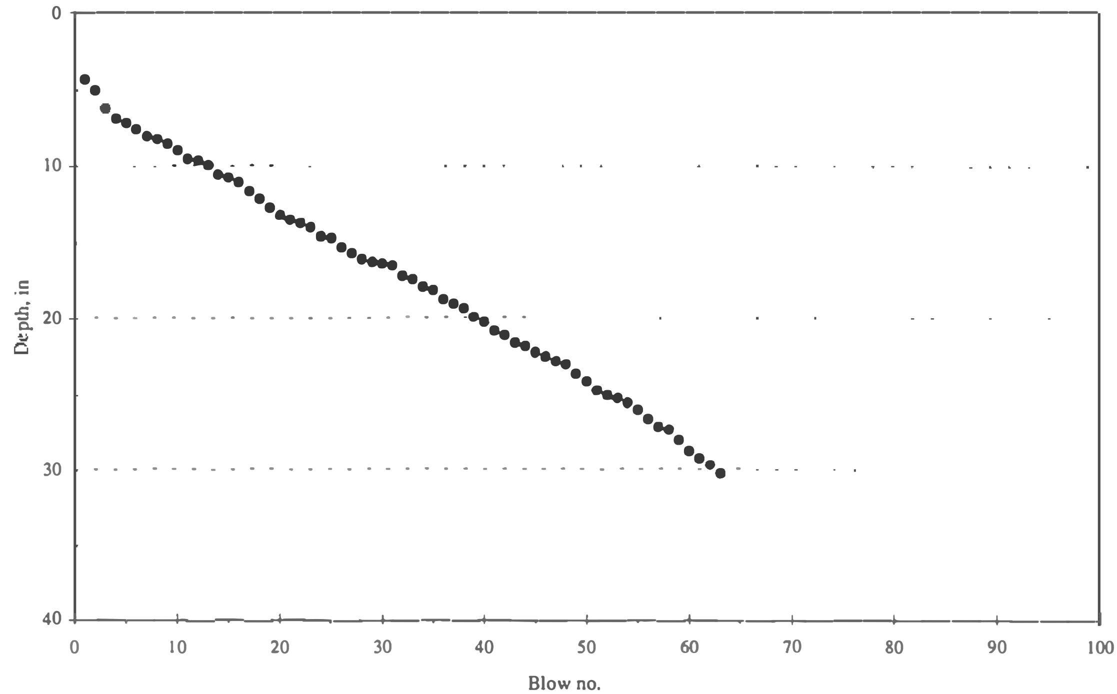


Figure 10. Representative Penetration Index vs. depth curves—site 5, phase I (sta 11+00).

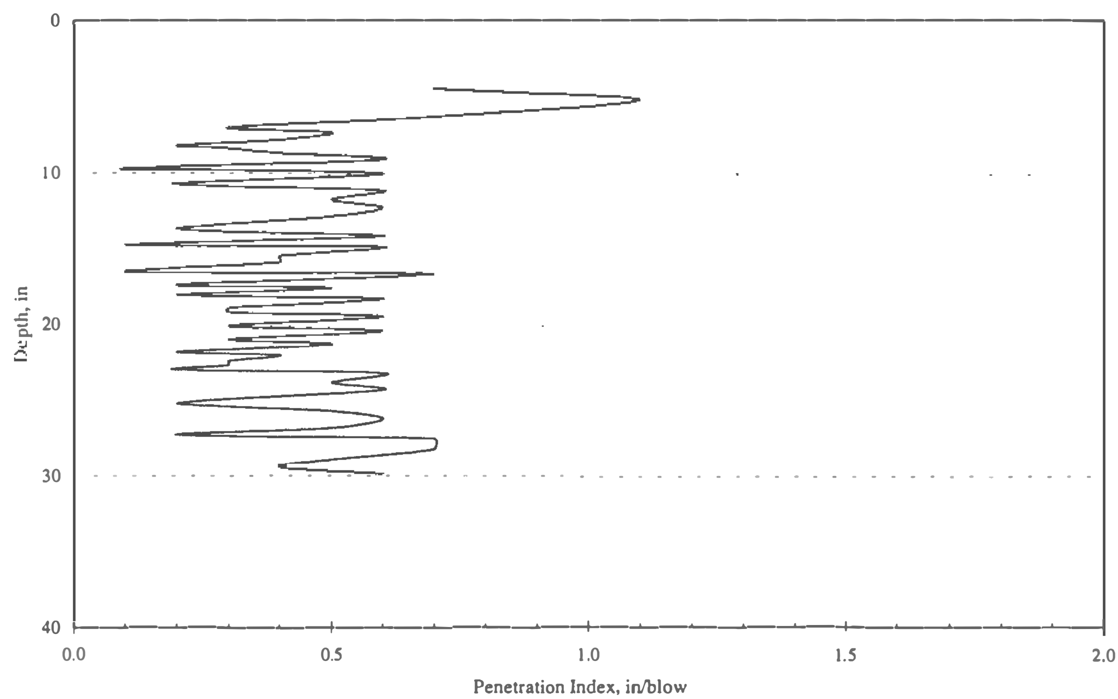


Figure 11. DCP Penetration Index vs. station, site 1.

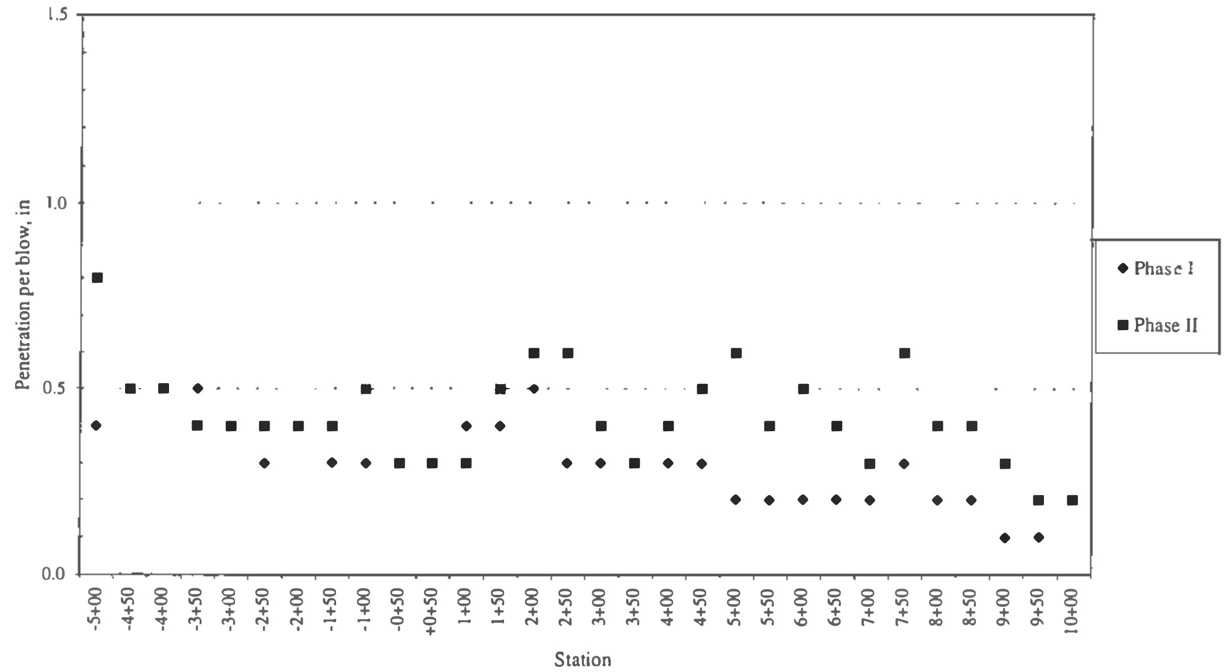


Figure 12. DCP Penetration Index vs. station, site 2.

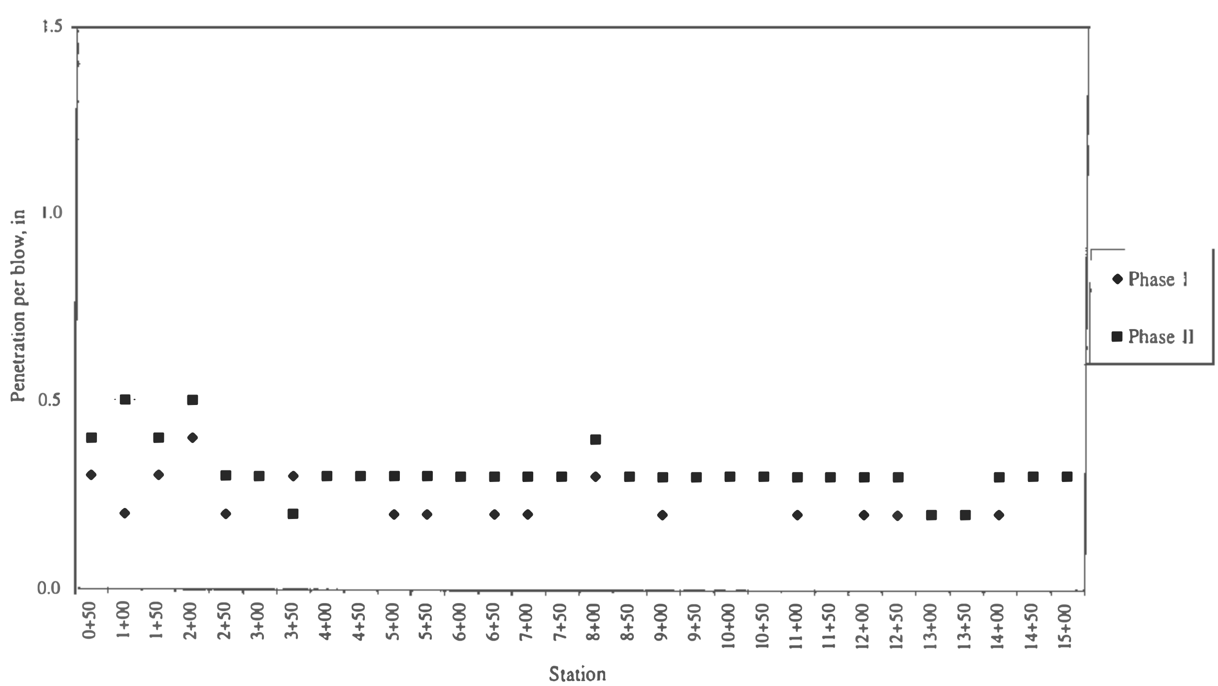


Figure 13. DCP Penetration Index vs. station, site 3.

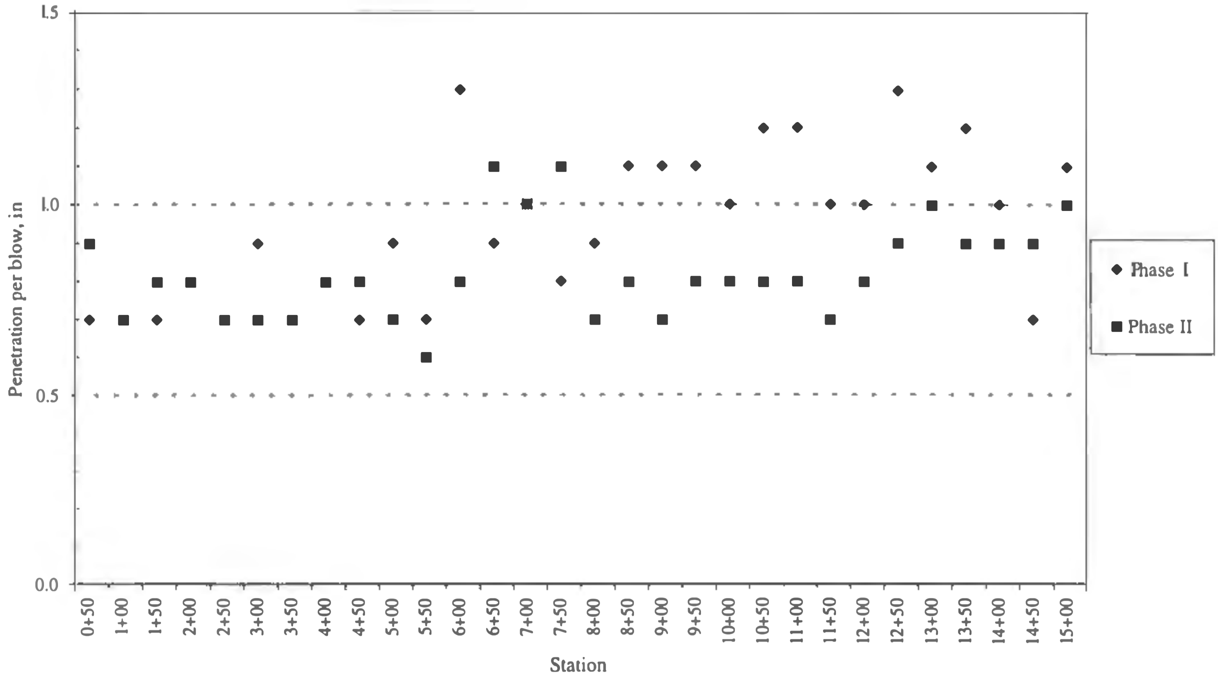
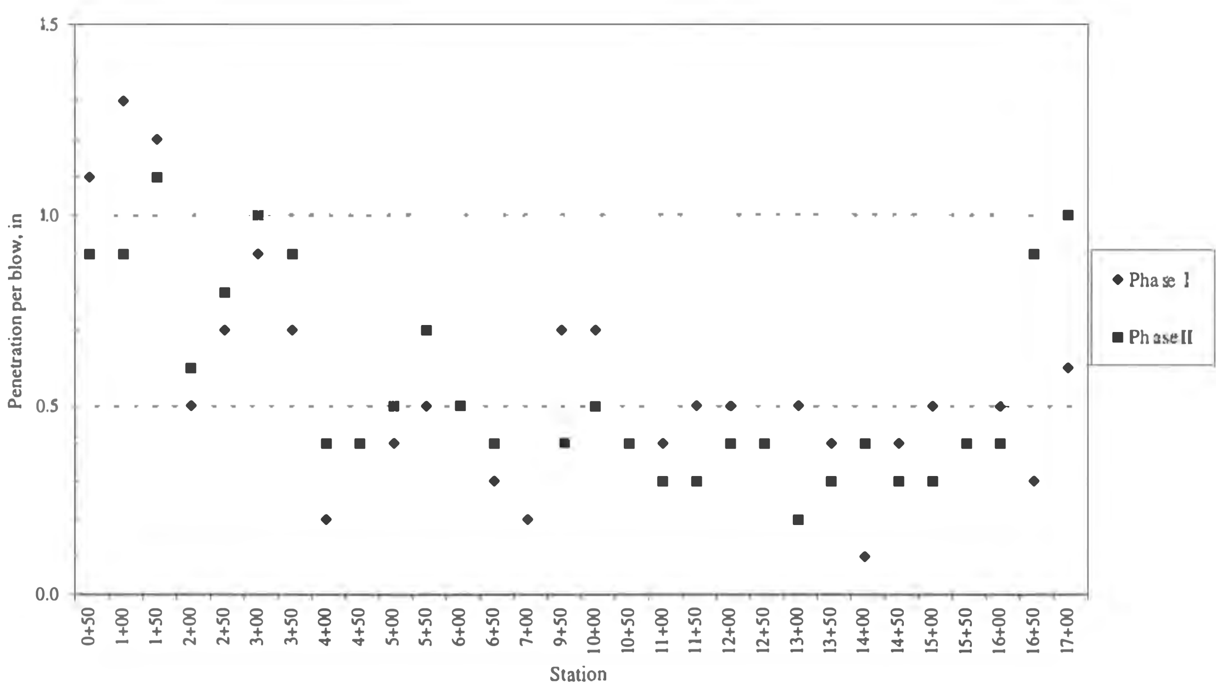


Figure 14. DCP Penetration Index vs. station, site 5.



Resilient Modulus

Resilient Modulus Test Methodology

The subgrade resilient modulus for the M_r vs. DCP correlation was determined using the FWD. The FWD is a dynamic plate load test that simulates the effects of a moving wheel load. During the FWD test, a circular load plate is lowered to the pavement surface and a mass is dropped from a predetermined height to produce an impulse load. The deflected pavement surface is measured by means of deflection transducers with one sensor placed in the center of the load plate (D0) and the remaining six sensors placed at intervals of 12 in away from the load plate (D12, D24, D36, D48, D60, D72). The FWD raw data are stored on a laptop computer and transferred to the office for in-depth analysis. Typically, analytical pavement models are used to determine the elastic moduli of the pavement layers and subgrade in a process known as backcalculation. In the case of flexible pavements, these models are based on the theory of elasticity, wherein each pavement layer and the subgrade are assumed to have linear elastic moduli (or stiffnesses).

The FWD data for the Oklahoma test sites were backcalculated using the method presented in the 1993 AASHTO Guide. This method determines moduli of a two-layer system (single pavement layer over subgrade) by use of an iterative procedure that utilizes equations based on elastic layer theory. First, an outer sensor is chosen to estimate the subgrade elastic modulus, E_s, based on the assumption that at distances sufficiently far from the load center the deflections are only a function of the subgrade stiffness and not of the upper pavement layers. Second, the maximum deflection, D0, is used in conjunction with the estimated E_s to determine the composite pavement modulus. With respect to determination of the correct outer sensor to use for E_s estimation, the AASHTO guide recommends using the sensor closest to the load plate which does not reflect the stiffness of the pavement layers, rather than merely using the farthest available sensor. This is because at increasing distances from the load the effects of other factors (such as stress-sensitive behavior and sensor accuracy) can distort the results and produce less accurate estimates of the true subgrade E_s.

In the case of FWD tests conducted directly over the roadbed surface (subgrade), the pavement can be modeled as a single layer system, and the portion of the backcalculation procedure that determines the composite pavement modulus is not utilized. Theoretically, according to the assumptions of elastic theory, any of the seven available deflection sensors could be used to estimate the subgrade elastic modulus, since moduli are assumed to be constant at all stress levels or distances from the load center. In reality, all pavement materials differ in varying degrees from their assumed idealistic behaviors. Therefore, the procedure used to determine subgrade moduli for the Oklahoma test sites involved two steps: the estimation of the subgrade elastic modulus at all seven sensor positions for each test point and a revision of the results to determine the sensor position that overall produced the most representative values for all test points. Having done this, the sensor positioned in the center of the load plate (D0) was selected as producing the most representative subgrade elastic moduli overall for the material types considered in this study.

The final step in the AASHTO backcalculation procedure is the correction of the FWD-determined subgrade elastic modulus to an equivalent value for the same material that would be determined by performing a laboratory resilient modulus test. To do this, the AASHTO guide recommends multiplying the FWD result by a factor of no greater than 0.33. This guideline was developed by several studies in which FWD testing and laboratory testing was conducted over the same soil types. The differences in results for the two methods is contributed to the inherent differences in the testing techniques, namely the mass of the soil tested, in-situ vs. laboratory behavior, and the method of sample loading. In this study a correction factor of 0.33 was applied to the FWD results to produce the presented values of subgrade M_r.

Falling Weight Deflectometer Test Results

The maximum deflection profiles for test sites 1, 2 , 3, and 5, normalized to 9,000 lb, are shown in figures 15 through 18. The maximum deflection (D0) is a good indicator of overall material or pavement stiffness (a lower deflection indicates a more rigid material). Therefore, in the case of deflections generated directly over the subgrade layer, the maximum deflection is inversely proportional to the overall quality of subgrade support. The D0 results for site 1 were the lowest and most uniform of all the test sections, showing a mean of approximately 40 mils. Site 2 resulted in maximum deflections slightly higher than site 1, but lower and more uniform than sites 3 and 4. The deflections in site 2 varied between 40 and 100 mils and were generally unchanged between phases I and II. Site 3 produced the highest mean deflection, with an average of approximately 125 mils during phase I. The results for phase II at this site were significantly lower, with a mean of approximately 80 mils, indicating a significant change in the subgrade response with respect to season. Site 5 showed great variability in the longitudinal sense, with lower deflections between stations 0+50 and 13+00 and high deflections from 13+50 to 17+00.

As discussed in the methodology section for resilient modulus, the backcalculation results at each deflection sensor were displayed graphically to determine the trend of M_r with respect to distance from the load center. The results of this analysis for phases I and II are shown in figures 19 and 20. Review of the curves shows the largest difference between test sites occurring at the maximum deflection position; the M_r values for sensors D12 through D72 are fairly uniform. The results at D0 produce reasonable backcalculated resilient moduli for the corresponding material types; for example, site 1 (sand) produced the highest value and site 3 (clay) produced the lowest. With the exception of slight increases in M_r for sites 1 and 3, the results for phase II produce nearly identical trends to phase I. The M_r values corresponding to both the D0 sensor (text) and D36 sensor (appendix B) were used in determining the correlations presented in this report.

The backcalculated M_r profiles for the four sites are presented in figures 21 through 24. Site 1 shows M_r values in the range of 4,000 to 9,000 psi, with higher values at the station 10+00 end. Values of this magnitude are reasonable but are slightly lower than what would be expected for a sand subgrade. Site 2 produced uniform, but low values, ranging between 3,000 and 6,000 psi, corresponding to a material of fair support. The mean M_r in site 3 is approximately 3,000 psi, and this is typical for a clay subgrade. The low variability of the results for site 3 indicates a very uniform material in both the horizontal and vertical directions. It is interesting to note that the M_r results for site 3 follow the same trend as seen in the maximum deflection profile, that is, a poorer result for the phase I testing relative to phase II. Site 5 indicates much variability over the length of the section, with values ranging from 2,000 to 8,000 psi. These values indicate a very poor to fair subgrade, consistent with the material in this section.

Figure 15. Maximum deflection normalized to 9,000 lbf, site 1.

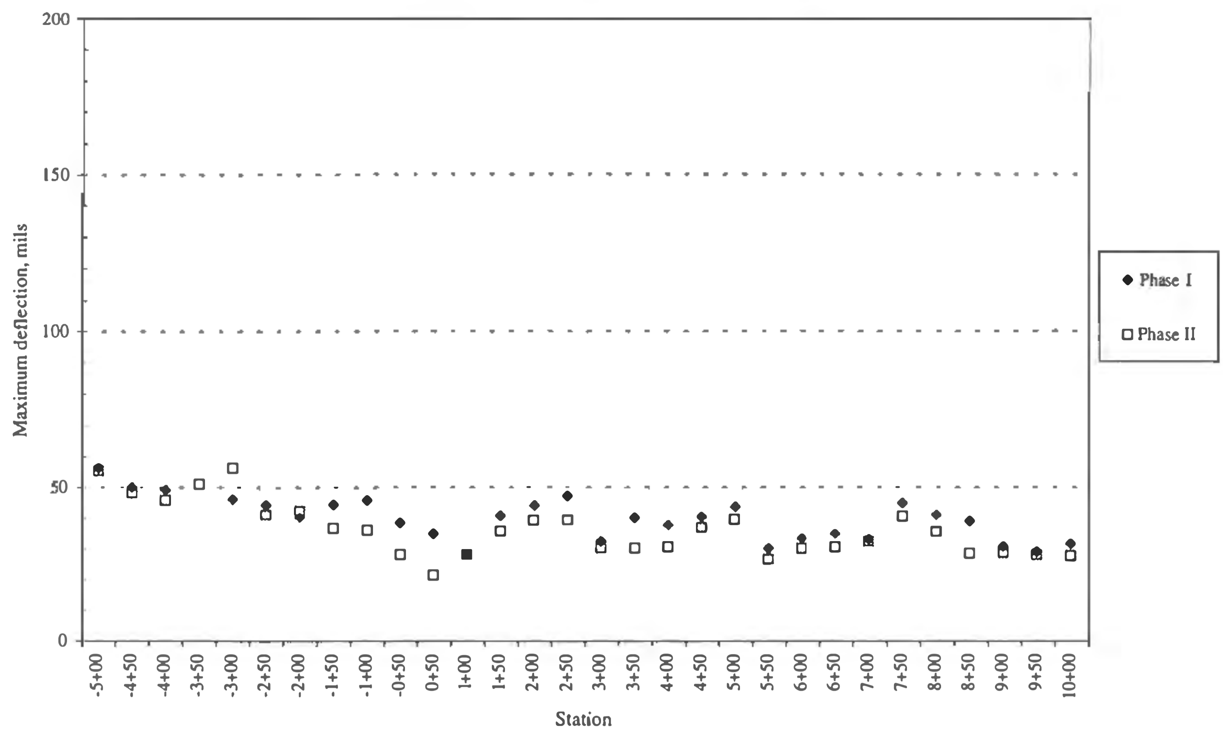


Figure 16. Maximum deflection normalized to 9,000 lbf, site 2.

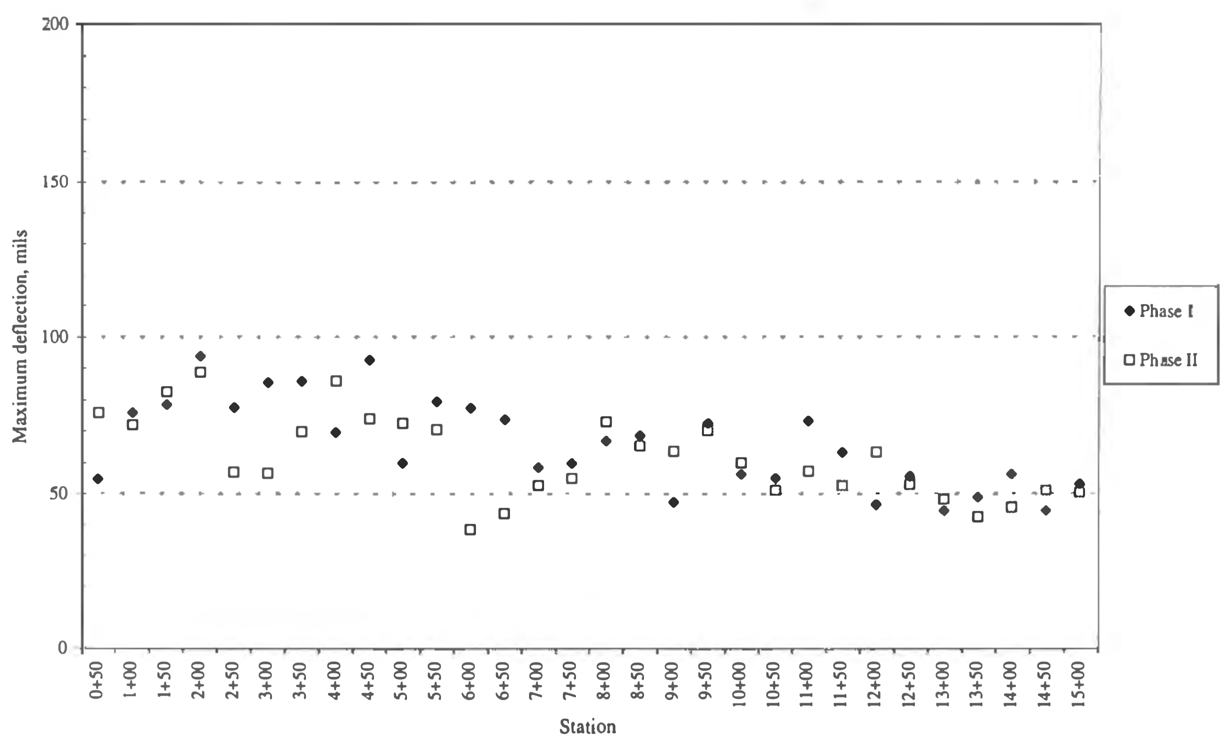


Figure 17. Maximum deflection normalized to 9,000 lbf, site 3.

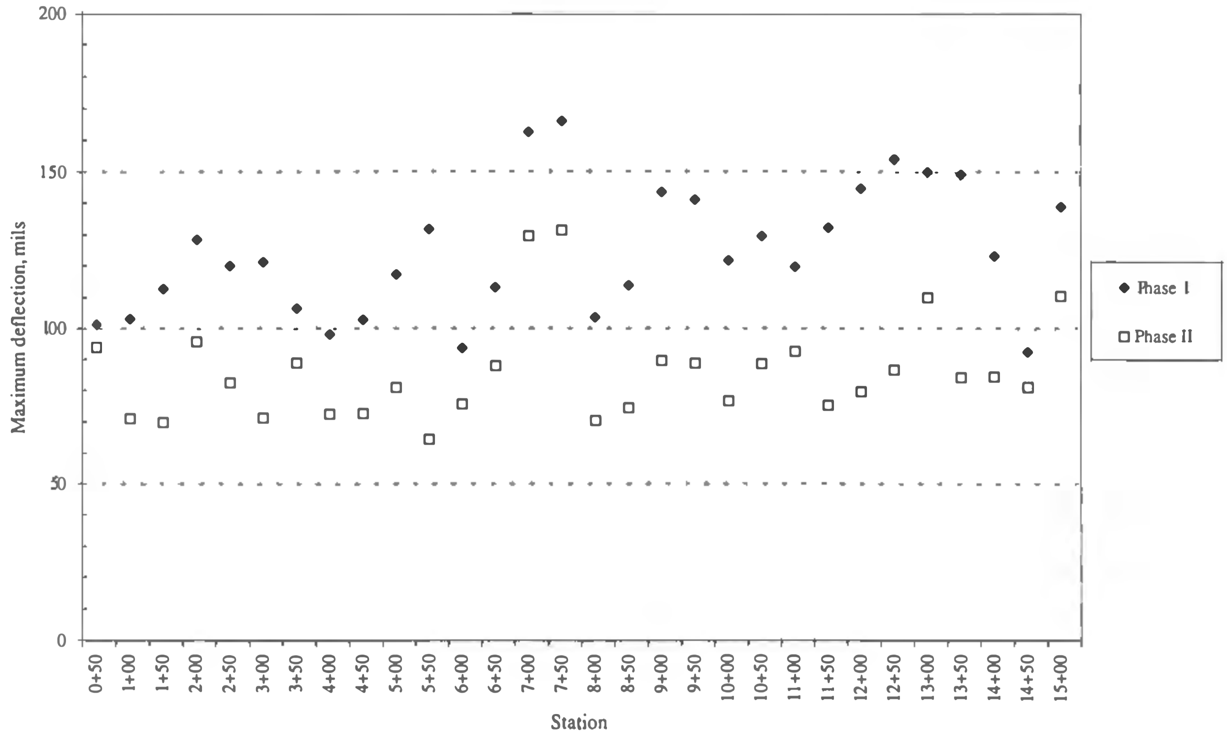


Figure 18. Maximum deflection normalized to 9,000 lbf, site 5.

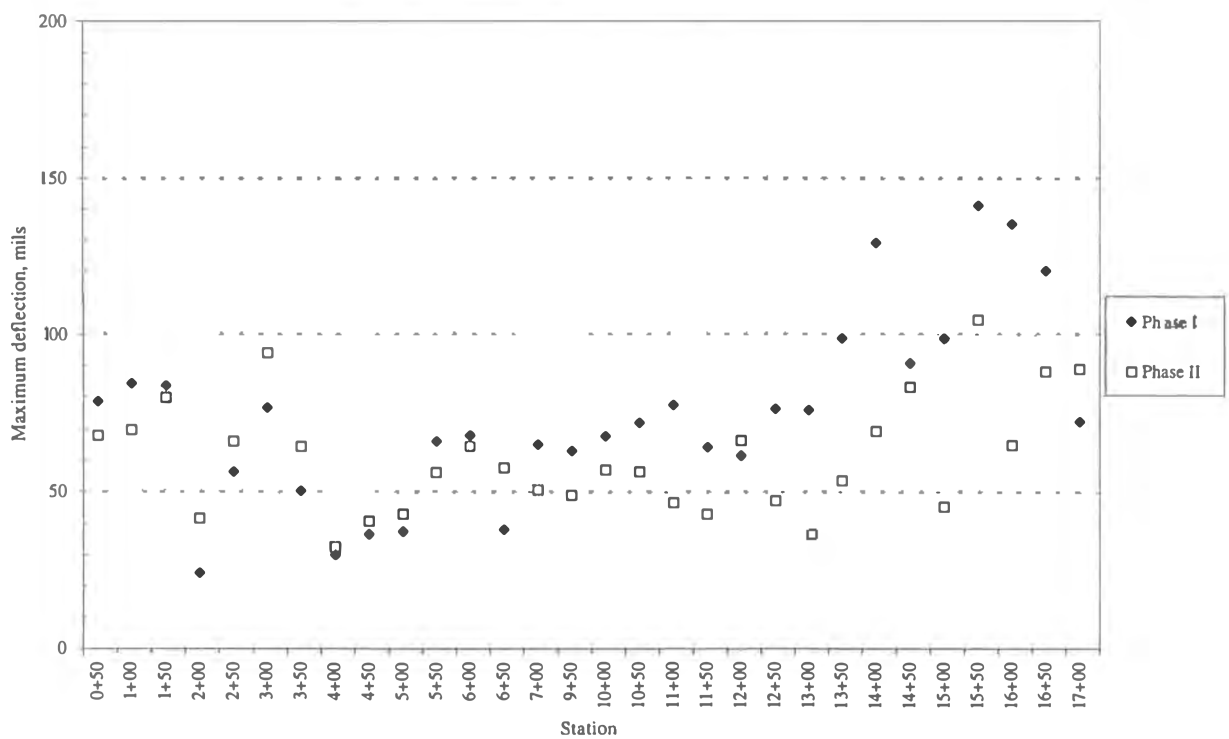


Figure 19. Subgrade resilient modulus calculated at each sensor—four sites, phase I.

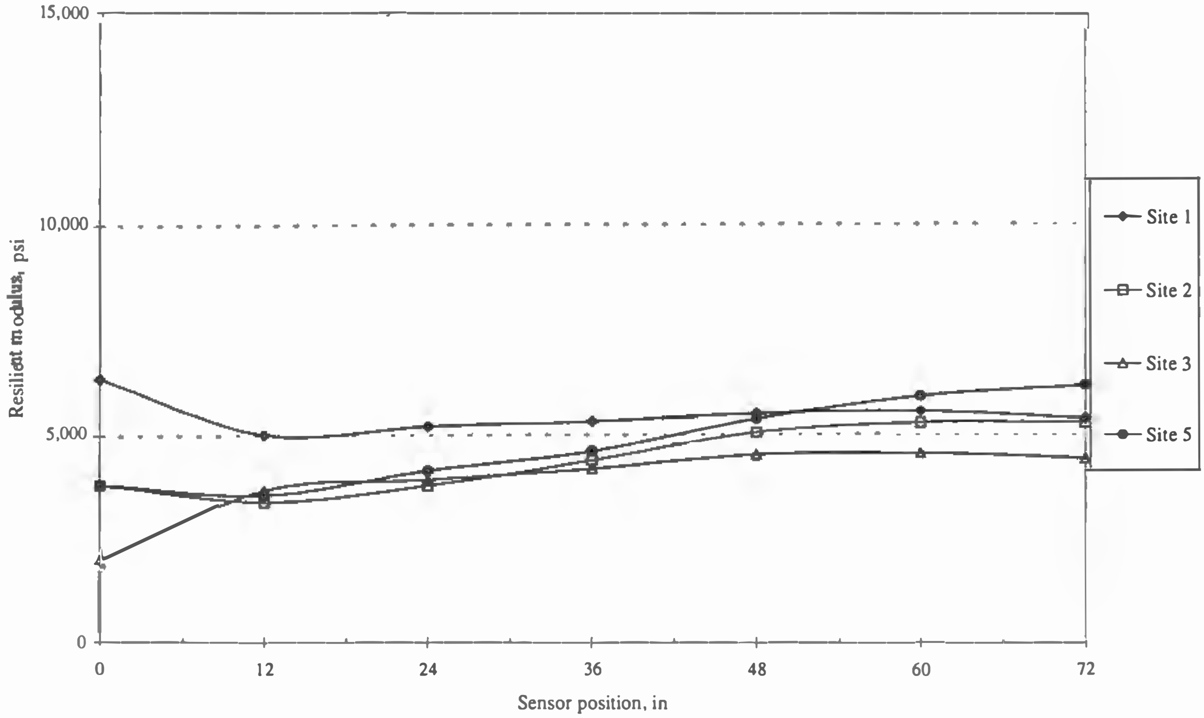


Figure 20. Subgrade resilient modulus calculated at each sensor—four sites, phase II.

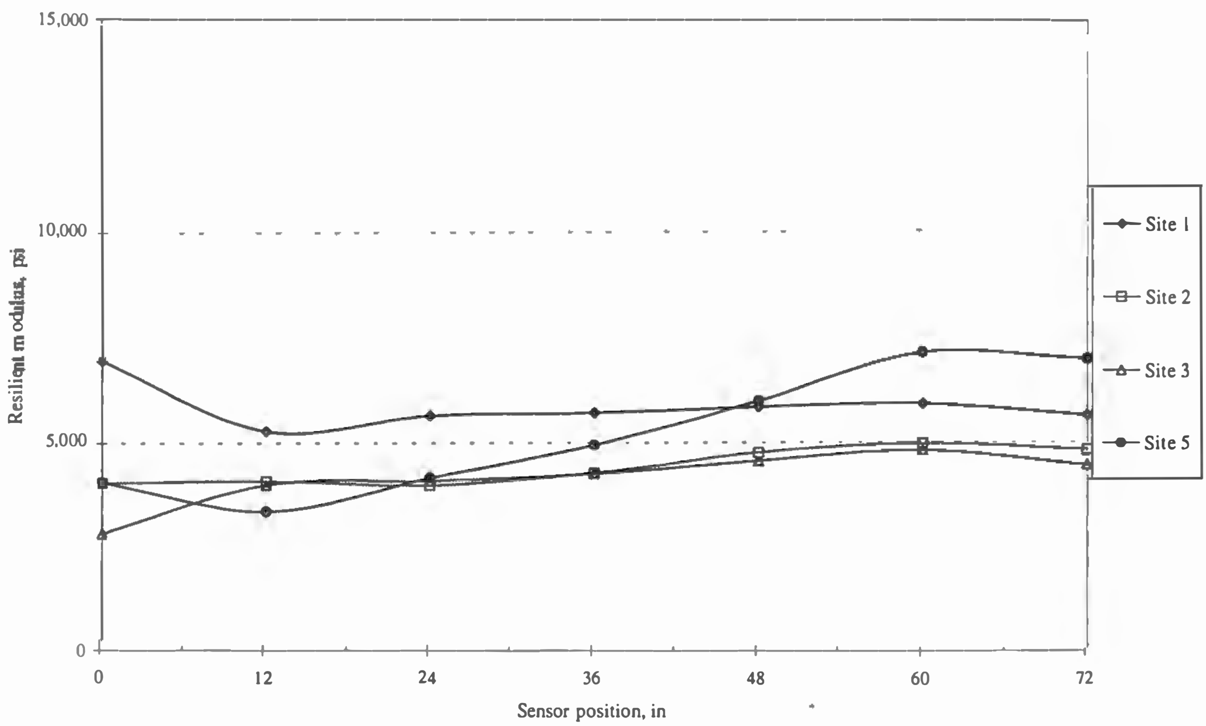


Figure 21. Backcalculated subgrade resilient modulus, site 1.

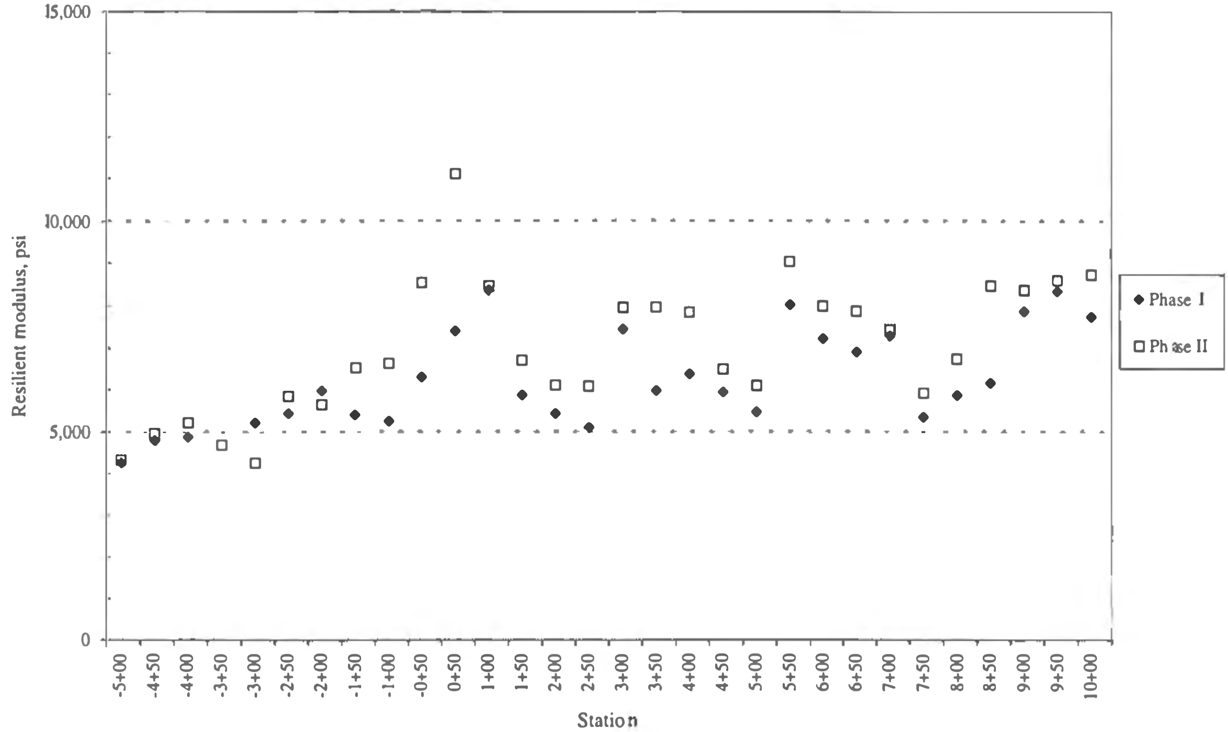


Figure 22. Backcalculated subgrade resilient modulus, site 2.

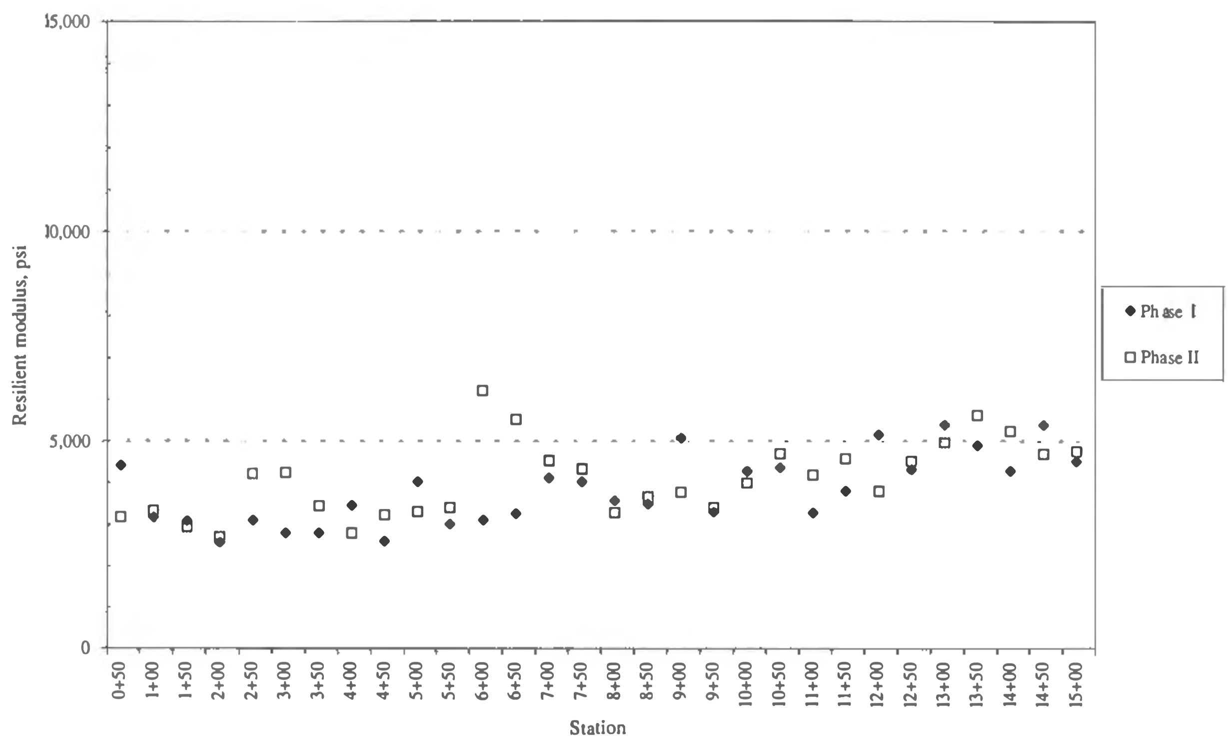


Figure 23. Backcalculated subgrade resilient modulus, site 3.

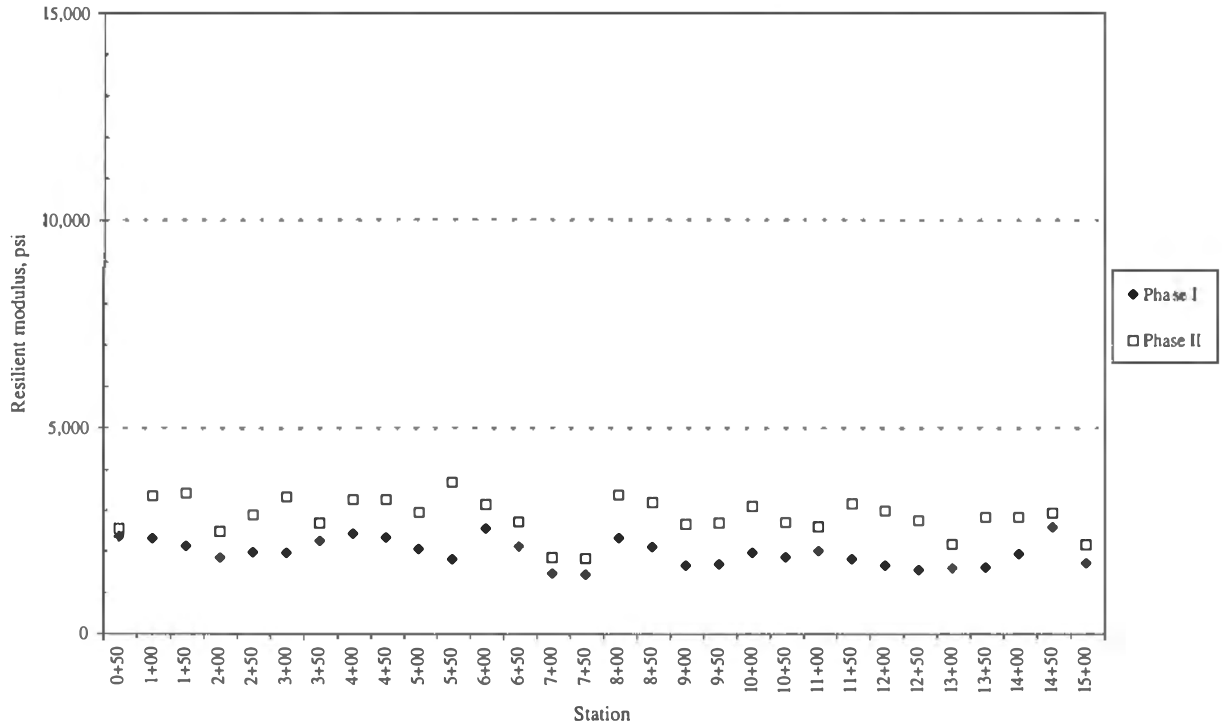
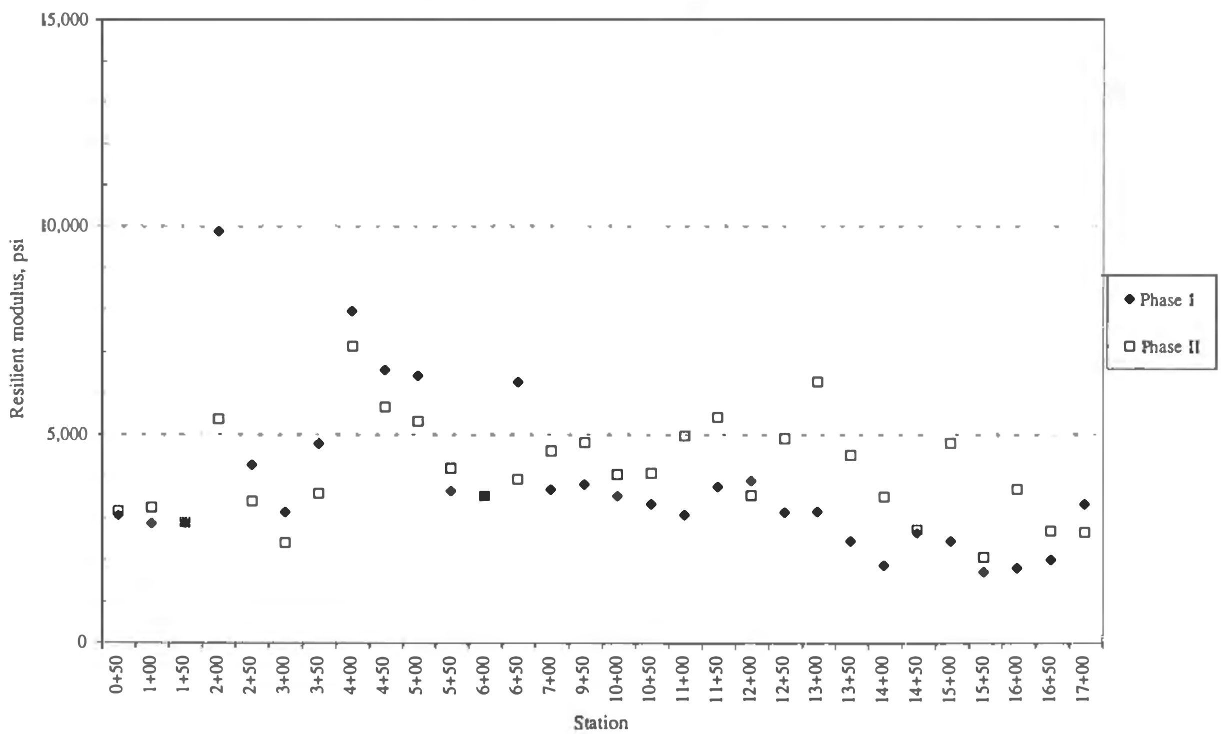


Figure 24. Backcalculated subgrade resilient modulus, site 5.



California Bearing Ratio

California Bearing Ratio Test Methodology

An estimate of soil bearing capacity at each site was made by performing CBR tests on a limited number of samples. CBR tests measure the shear resistance of molded samples to the penetration of a cylindrical piston at a slow rate of advancement. The CBR is defined as the ratio of the load required for the piston to penetrate a given depth over the load required to achieve the same penetration on a sample of high-quality crushed aggregate multiplied by 100. In this study, one-point, soaked CBR tests were conducted according to ASTM specification D 1883-87 on samples compacted to field moisture and density states. Soaked CBR testing of soils compacted to field densities represents the critical moisture state of the in-place material.

As per the contract requirements, limited moisture, density, and material samples were taken at each test location. In general, a minimum of two locations were chosen for CBR testing per test site. The maximum number of tests performed at a single site was six. Soil samples were taken between depths of 6 and 24 in, consistent with the range of DCP penetration index calculations.

California Bearing Ratio Test Results

The results for CBR testing of samples taken during both phases and compacted to moisture and density states representative of field conditions at the time of testing are presented in table 2. In addition to the testing done at field density for the samples obtained in phase, these specimens were also tested at a compaction level equivalent to 95 percent of maximum dry density, and the corresponding values are included in the table.

Table 2. Summary of the CBR test results for all sites, both phases.

General		Phase I			Phase II	
Site no.	**Site name**	**Station**	**CBR at field density**	**CBR at 95 % max. dry density**	**Station**	**CBR at field density**
1	Major	-5+00	16.3	28.1		
		-3+00	5.7	16.1		
		2+55	14.3	21.8	2+55	14.3
		5+00	16.0	20.9		
		7+50	na	11.2		
					8+00	10.1
2	Kingfisher	2+00	na	14.5	2+00	17.3
		7+00	20.0	28.4	7+00	20.1
					10+00	30.9
		11+00	na	35.9		
		13+00	16.6	15.3		
3	Kay	2+00	2.4	1.4		
		6+00	2.5	1.2		
					7+50	2.5
		9+50	3.2	1.5		
		14+00	na	1.4		
5	Choctaw	1+50	na	37.5		
		2+00	2.0	0.7		
					2+50	2.7
		6+00	2.2	1.1		
		11+00	0.8	1.5		
		11+50	na	30.1		
					13+50	13.5
		14+50	na	2.7		

As can be seen, the CBR results for sites 1 and 2 (sand and silt) reflect subgrade materials with CBRs greater than 10 percent and reaching a maximum of approximately 30 percent. A CBR value of 6 is generally considered a reasonable lower bound for roadway design. CBR values less than 6 may require stabilization or geotextile reinforcement to reduce the required thickness of granular material in low-volume road design. The results for sites 3 and 4 are poor but consistent with clays at saturated moisture conditions.

A summary of the moisture and density conditions to which the specimens were compacted is shown in table 3. These conditions were determined to be representative of field conditions at the time of testing by field nuclear density and laboratory moisture tests.

Table 3. Summary of the moisture and density states at the time of testing.

General		Phase I			Phase II		
Site no.	Site name	Station	Moisture content, %	Dry density, pcf	Station	Moisture content, %	Dry density, pcf
1	Major	-5+00	8.1	110.4			
		-3+00	11.1	103.9			
		2+55	14.5	103.4	2+55	14.5	103.4
		5+00	6.7	104.8			
		7+50	na	na			
					8+00	7.9	105.8
2	Kingfisher	2+00	na	na	2+00	15.5	115.2
		7+00	13.5	114.1	7+00	13.5	114.1
					10+00	10.6	113.0
		11+00	na	na			
		13+00	11.5	123.2			
3	Kay	2+00	14.8	105.3			
		6+00	20.0	101.6			
					7+50	23.3	102.5
		9+50	18.3	98.9			
		14+00	na	na			
5	Choctaw	1+50	na	na			
		2+00	17.6	102.8			
					2+50	16.8	102.4
		6+00	24.8	96.7			
		11+00	17.6	95.7			
		11+50	na	na			
					13+50	12.7	109.7
		14+50	na	na			

The phase 1 and phase 2 tests were intended to ascertain if seasonal variations in moisture would result in substantially different test values. In several cases, road maintenance, ponded water in the roadside ditches, and other factors necessitated sampling at different stations. Therefore, it is difficult to make direct comparisons of changes in the field moisture content on a station by station basis. However, based on the available data, the average moisture conditions were similar for both testing phases.

DATA ANALYSIS AND DEVELOPMENT OF CORRELATIONS

Resilient Modulus versus DCP Penetration Index

Based on the results of data analysis, resilient modulus versus DCP correlations were developed. First, results were compared point by point according to soil type (site), followed by a correlation including all data from all sites. The individual correlations for each soil type are shown in figures 25 through 28. In figure 29, the results for the four sections are presented on a single graph, where each site is plotted as a separate series. Finally, the correlation for all data from all sites is shown in figure 30. The same relationships are presented for phase II results in figures 31 through 36. A correlation based on the conglomerate results of both phases is shown in figure 37.

Based on the phase I results, the correlations for site 1 show that for PI in the range of 0.1 to 0.5 in/blow, M_r values from approximately 7,500 to 5,000 psi would be estimated. Site 2 shows low variability in both DCP and FWD results. The range of PI for site 2 of 0.2 to 0.4 in/blow results in M_r of 2,500 to 4,000 psi. Site 3 indicates that the estimated M_r is relatively insensitive to the wide range of DCP values, with all PI resulting in low M_r of approximately 2,000 psi. The DCP results for site 5 cover a very broad range, approximately 0.1 to 1.3 in/blow; however, the estimated M_r is fairly insensitive to this range and falls in the range of approximately 3,000 to 4,000 psi.

In terms of all results from all sites, sites 1 and 2 define the lower range of DCP values, while site 3 defines the upper range of PI (see figure 28). Site 5 is the most variable of all sites and has results extending over the entire range of DCP results. Figure 30 shows the correlation equation for all results taken as a single data set. Based on this relationship, the lowest PI value would estimate an M_r of approximately 5,500 psi, while the maximum DCP value of 1.3 in/blow results in an M_r of approximately 1,500 psi. Phase II correlations are very similar to those of phase I.

California Bearing Ratio versus DCP Penetration Index

The laboratory CBR results for phase I and II determined at field density were used to develop correlations to DCP values. The results of these relationships are shown in figures 38 through 41. A correlation based on the conglomerate results of both phases is presented in figure 42.

In the lower range of PI values, corresponding to sites 1 and 3, CBR values greater than 10 percent resulted. For DCP values in this range, the corresponding CBR values are between 10 and 40 percent, depending on the testing phase and density level. On the other hand, the PI values in the medium to high range correspond primarily to sites 3 and 4 and result in very low CBRs, typically less than 5 percent. It should be noted that when compared to typical CBR versus DCP correlations, the results shown here fall below the predicted curve. This is due in large part to the fact that the CBR tests conducted in this study were tested at saturated moisture conditions, rather than field moisture conditions. Testing at field moisture conditions would have produced results more similar to other published correlation curves; however, for design based on the most conservative CBR estimate, the soaked CBR test is the more appropriate method.

Figure 25. DCP Penetration Index vs. Mr—site 1, phase I.

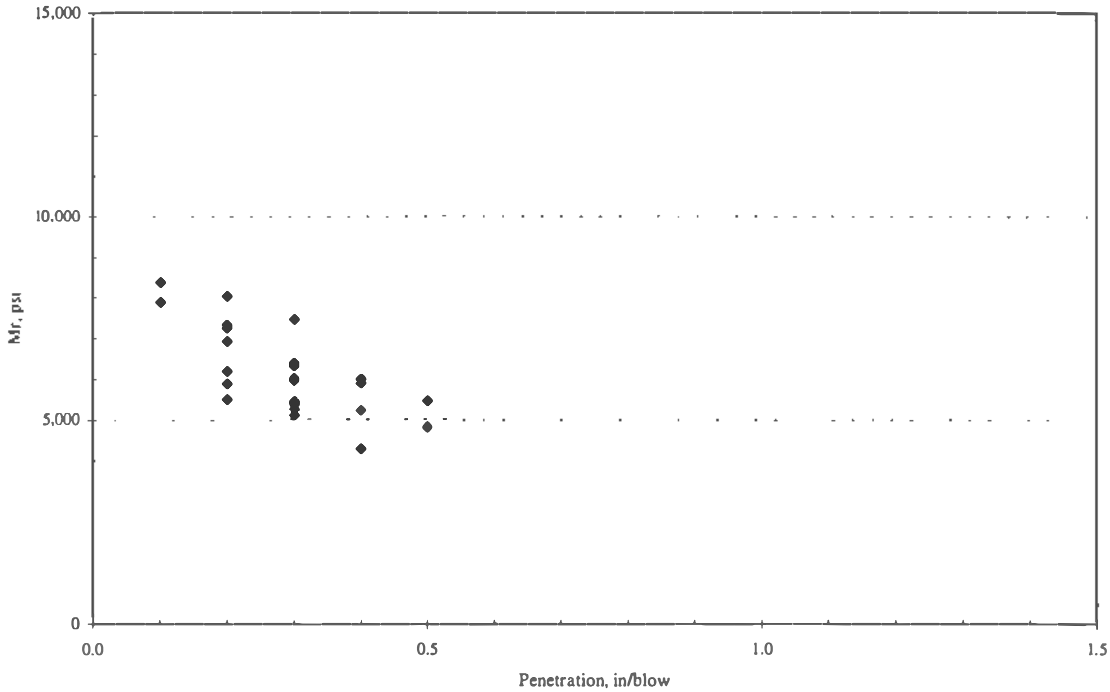


Figure 26. DCP Penetration Index vs. Mr—site 2, phase I.

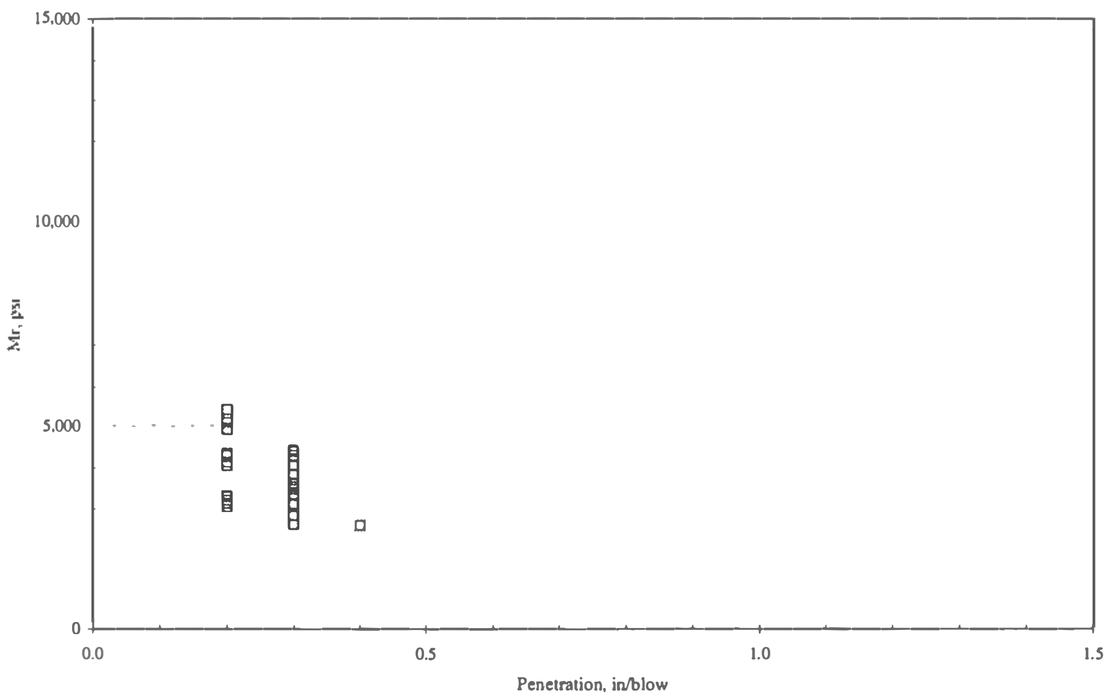


Figure 27. DCP Penetration Index vs. Mr—site 3, phase I.

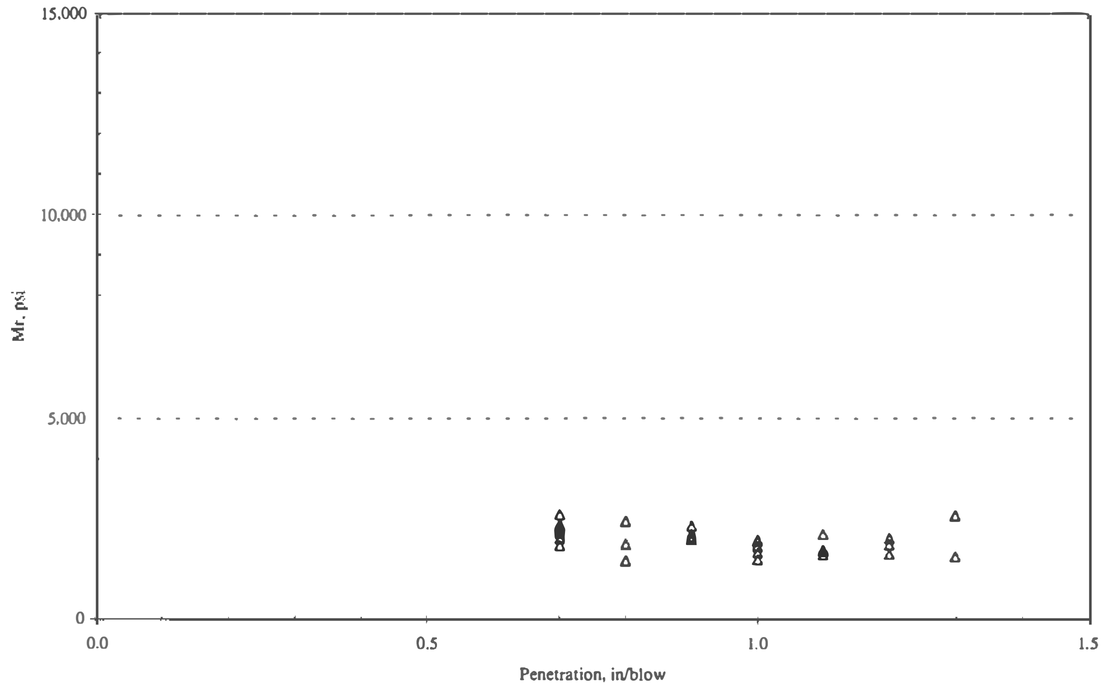


Figure 28. DCP Penetration Index vs. Mr—site 5, phase I.

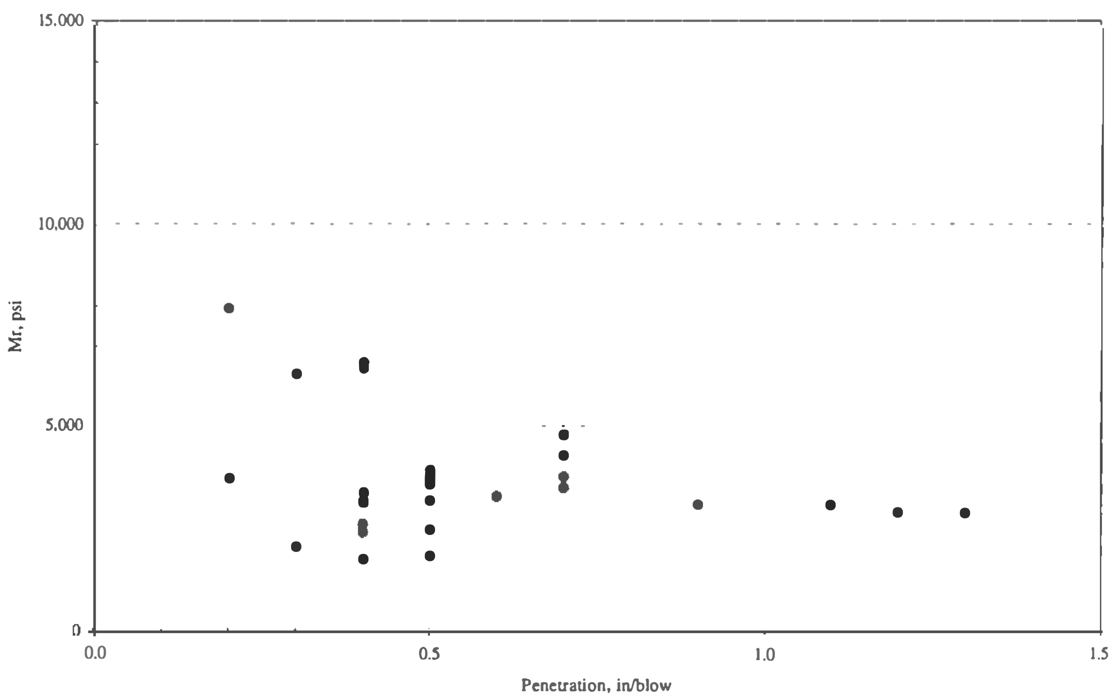


Figure 29. DCP Penetration Index vs. Mr—four sites, phase I.

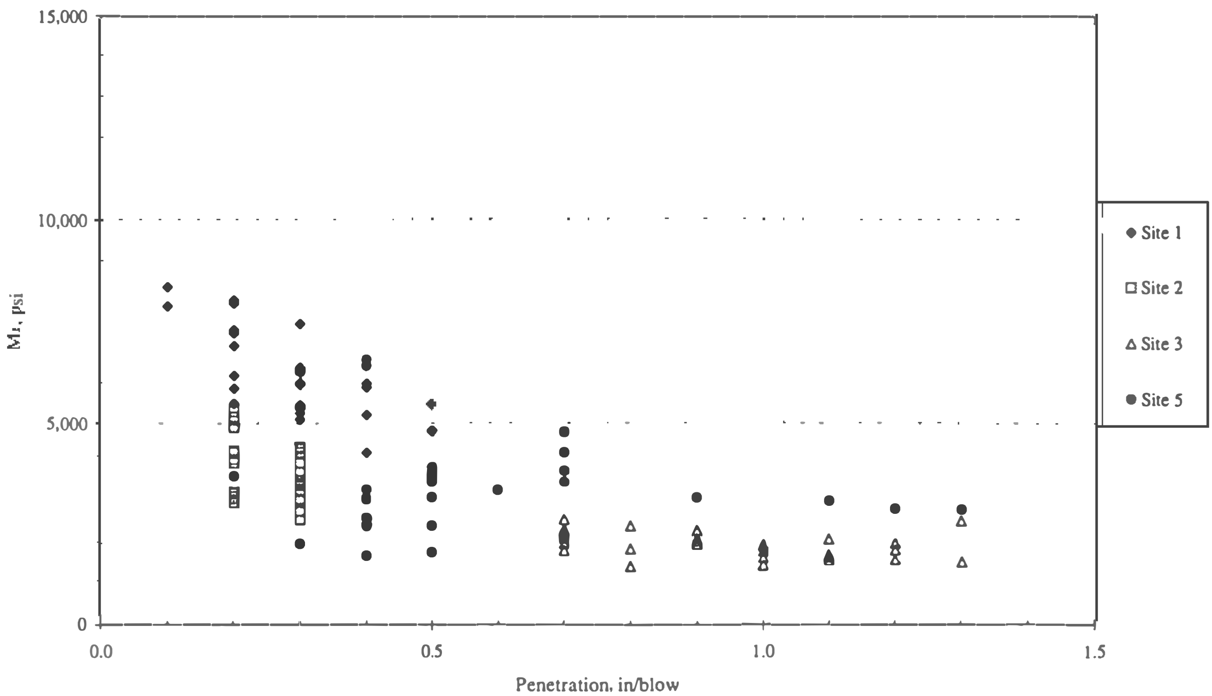


Figure 30. DCP Penetration Index vs. Mr—all sites, phase I.

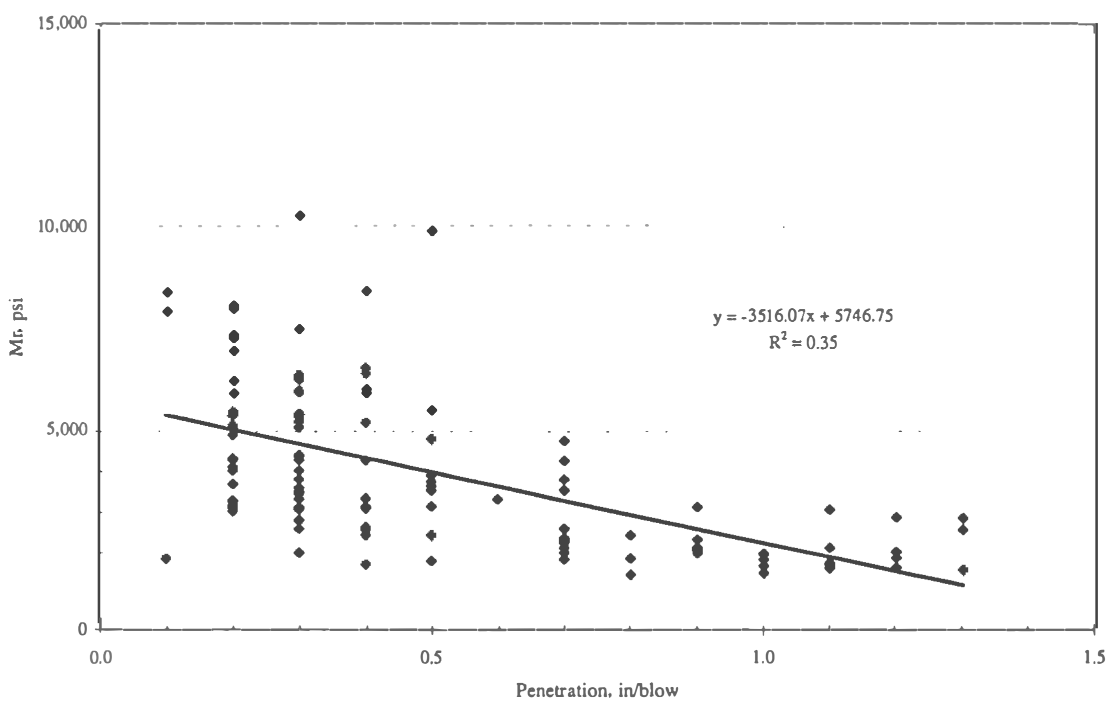


Figure 31. DCP Penetration Index vs. Mr—site 1, phase II.

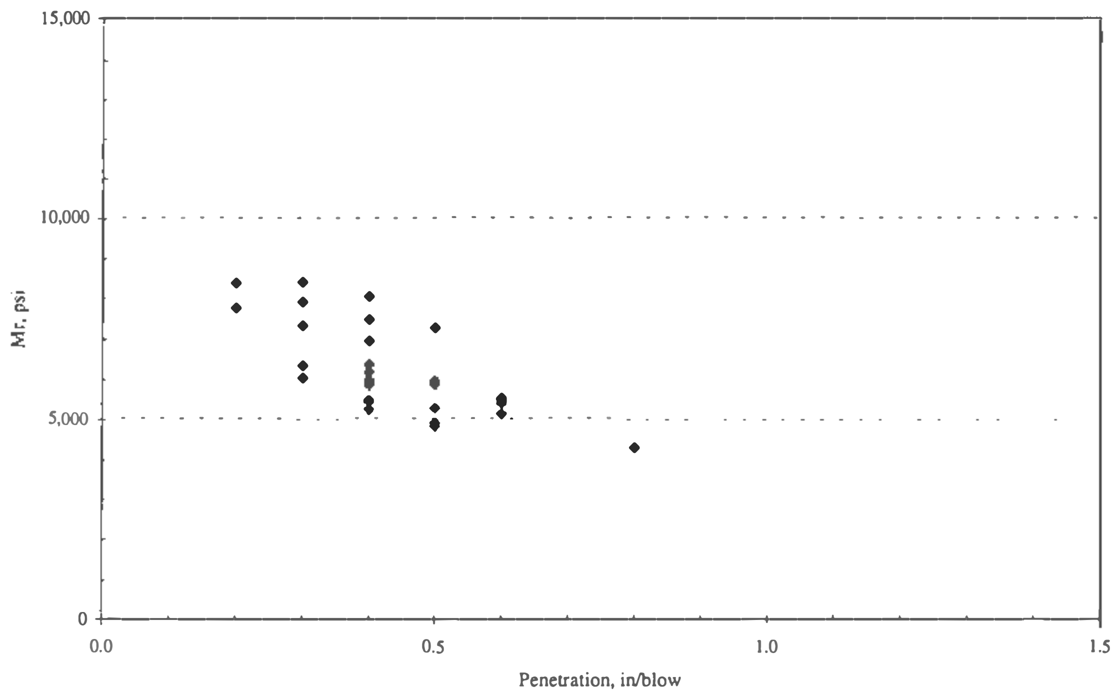


Figure 32. DCP Penetration Index vs. Mr—site 2, phase II.

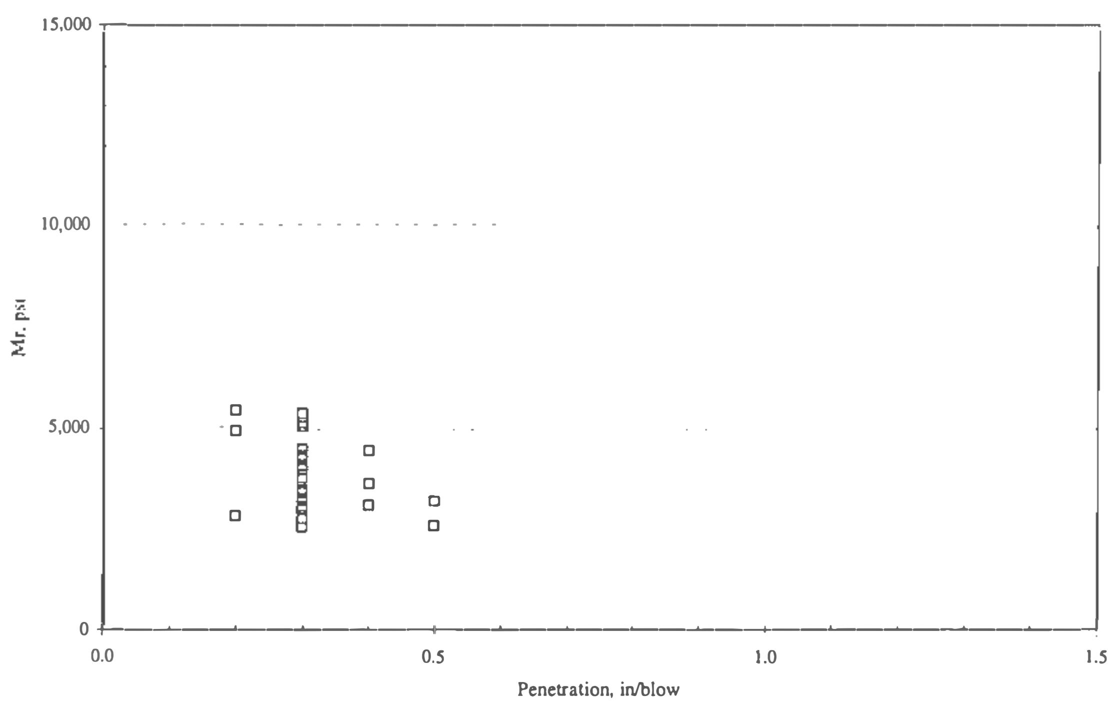


Figure 33. DCP Penetration Index vs. Mr—site 3, phase II.

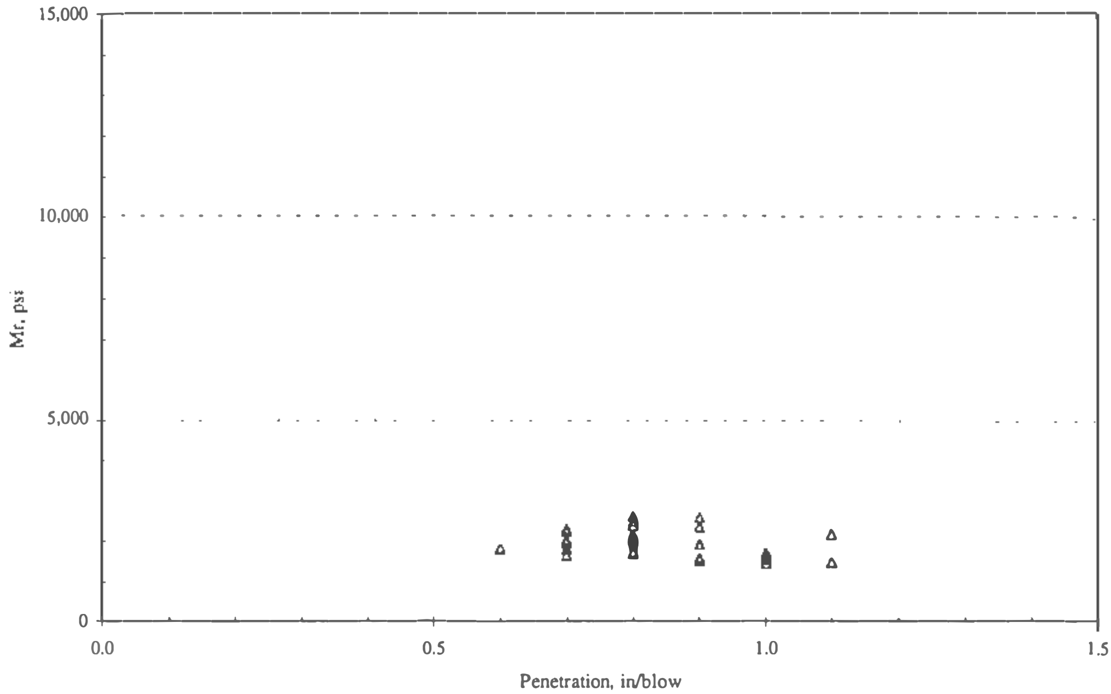


Figure 34. DCP Penetration Index vs. Mr—site 5, phase II.

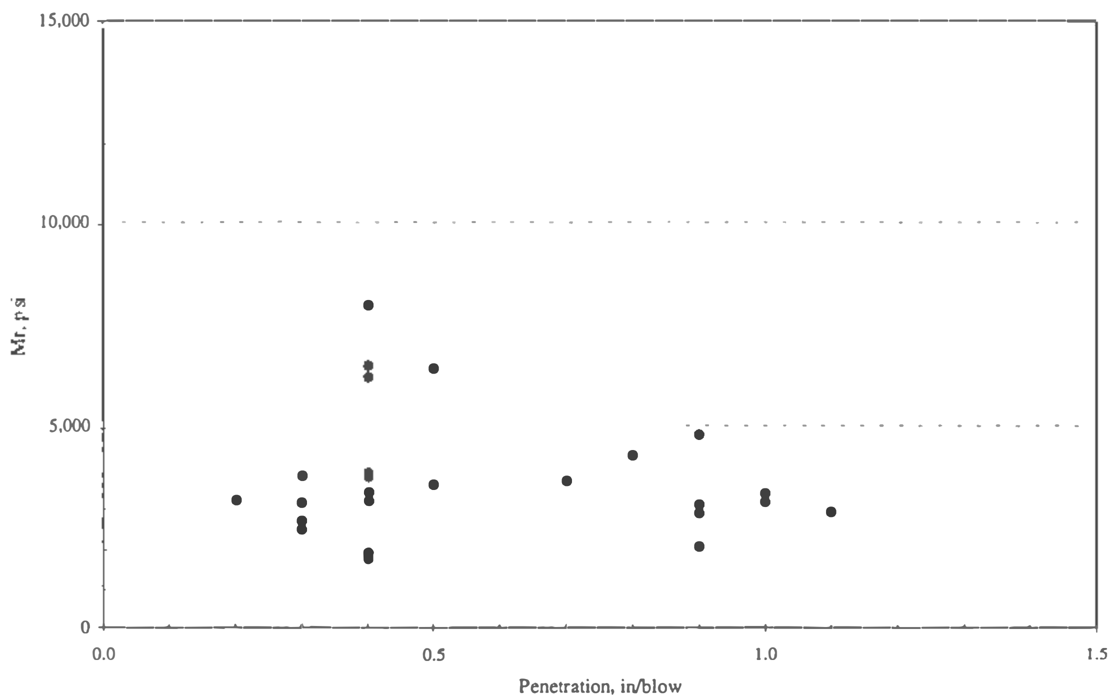


Figure 35. DCP Penetration Index vs. Mr—four sites, phase II.

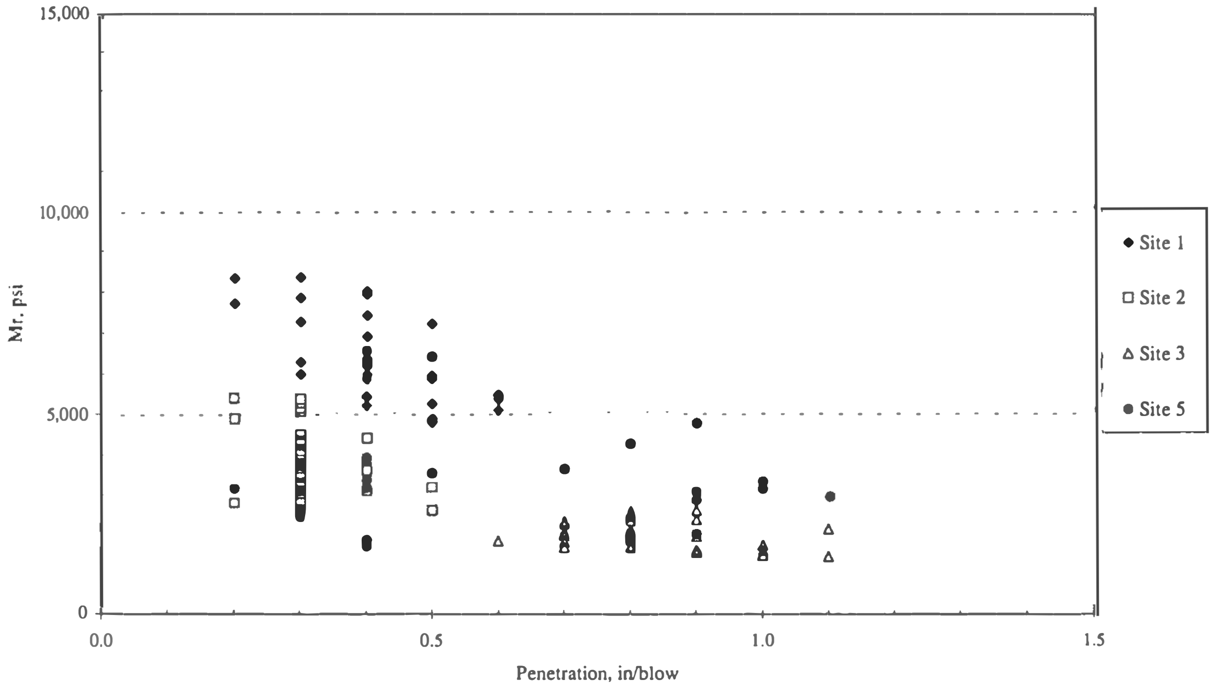


Figure 36. DCP Penetration Index vs. Mr—all sites, phase II.

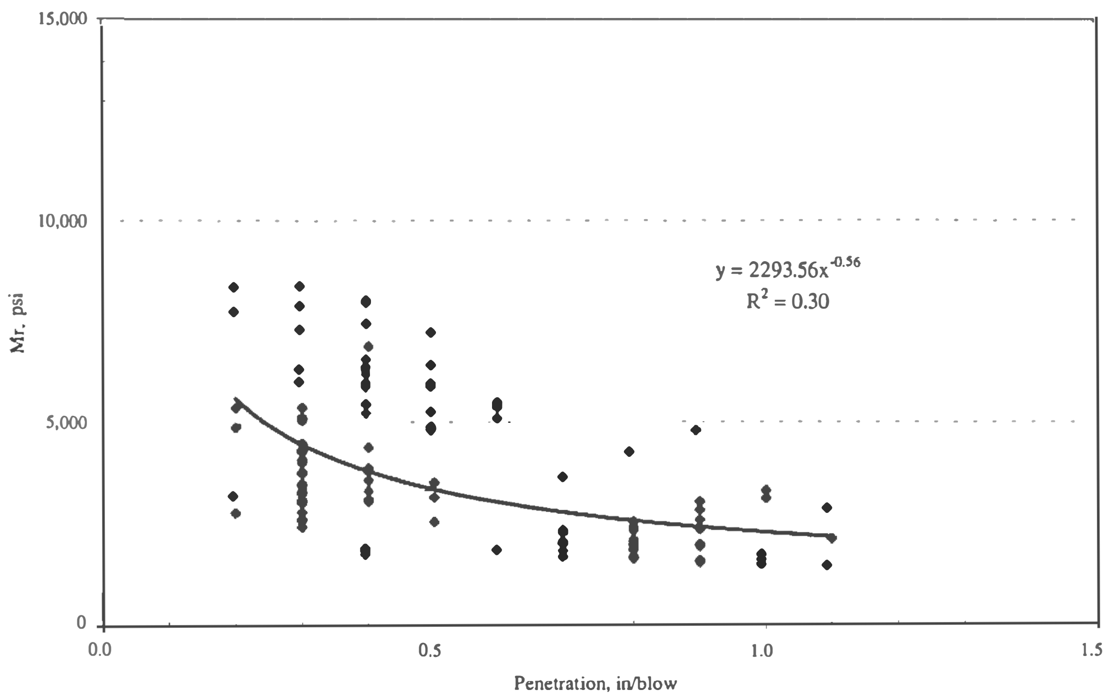


Figure 37. DCP Penetration Index vs. Mr—site 1, phases I and II.

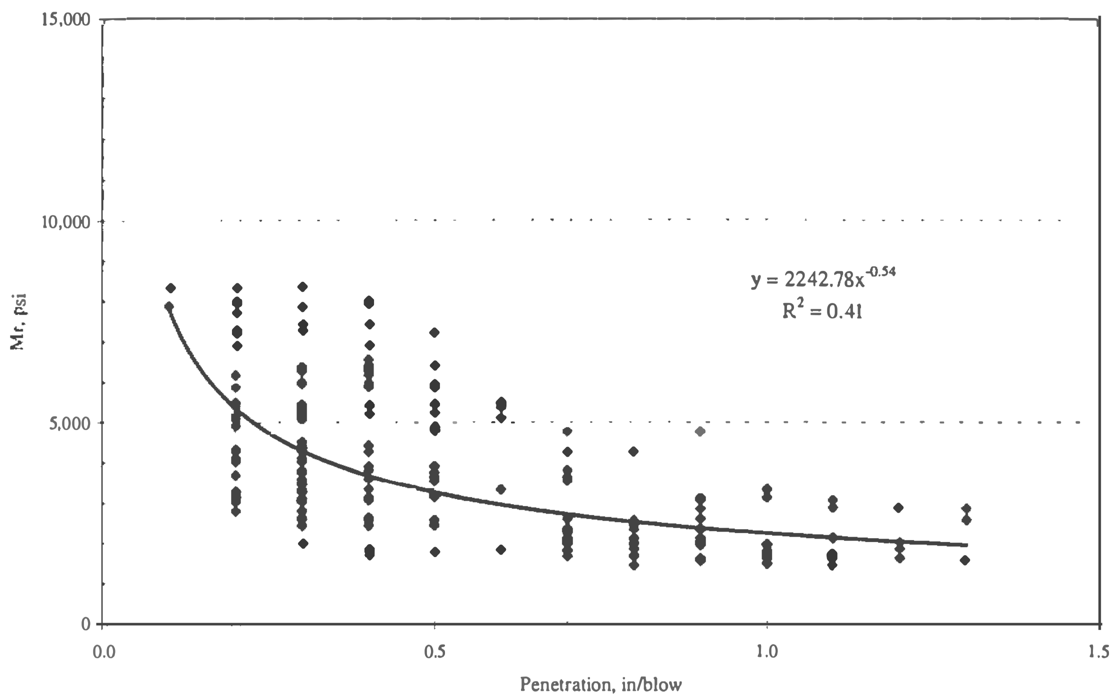


Figure 38. DCP Penetration Index vs. soaked CBR at field density—four sites, phase I.

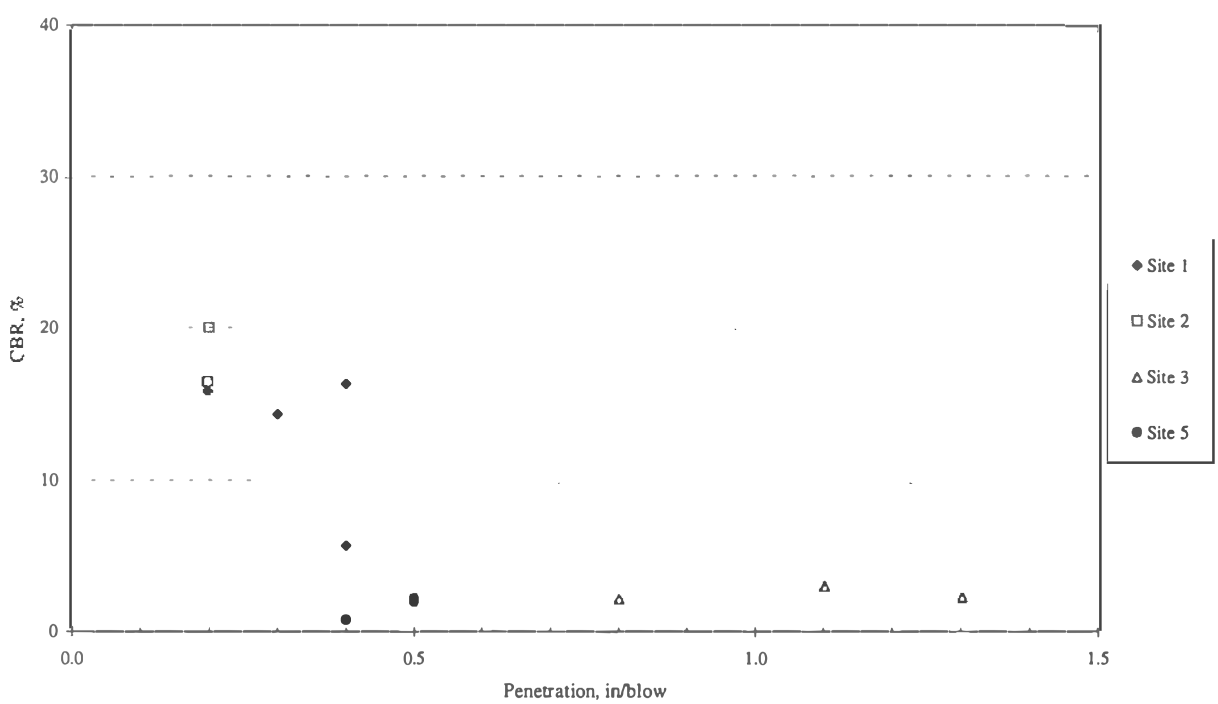


Figure 39. DCP Penetration Index vs. CBR—all sites, phase I.

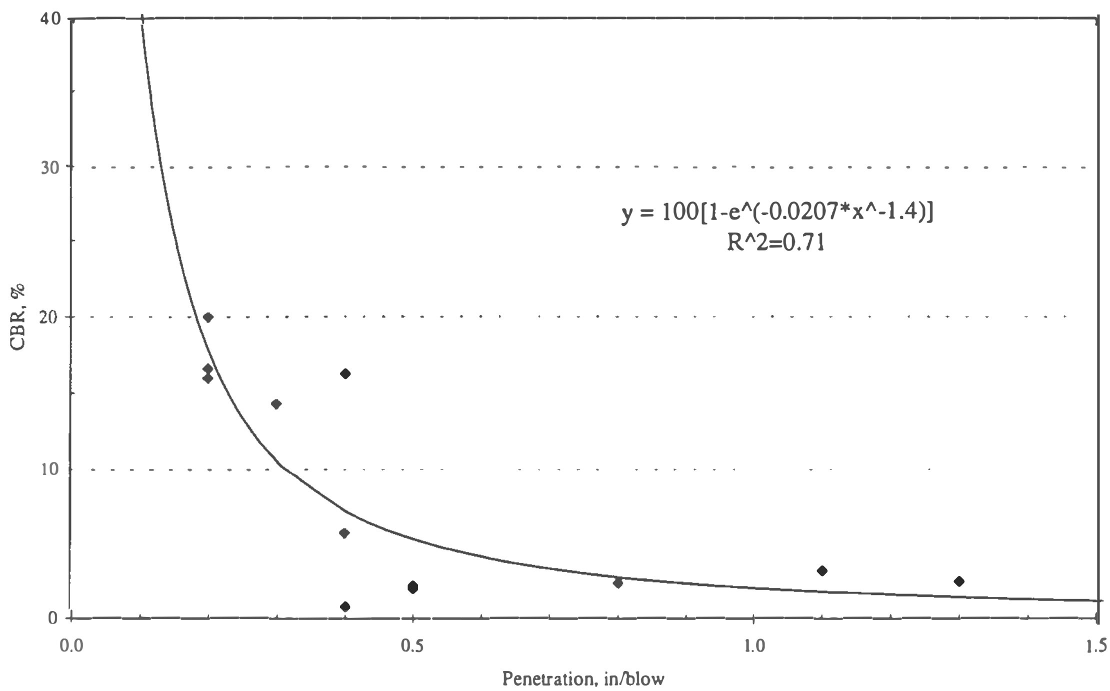


Figure 40. DCP Penetration Index vs. soaked CBR at field density—four sites, phase II.

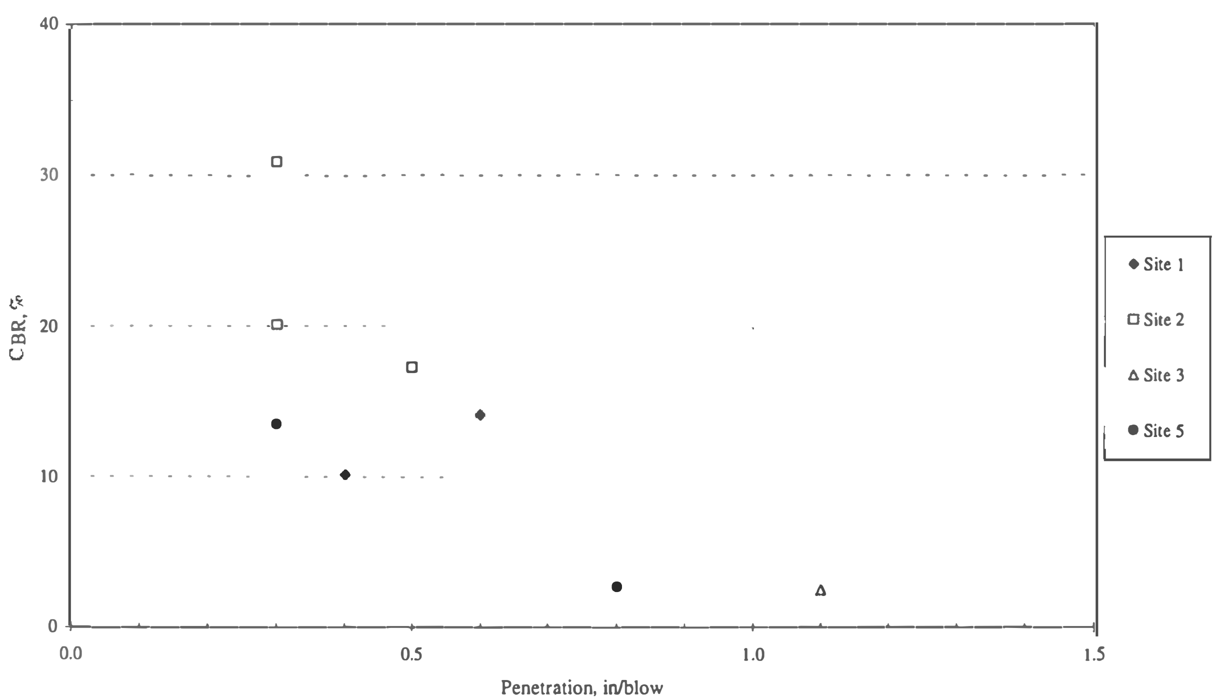


Figure 41. DCP Penetration Index vs. CBR—all sites, phase II.

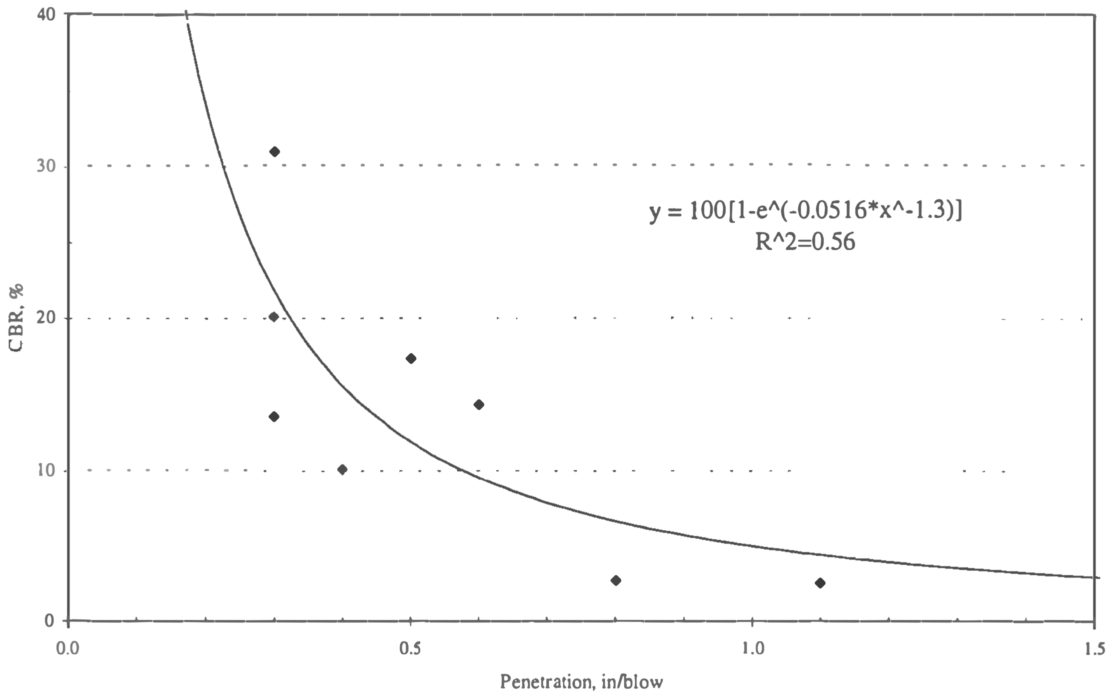
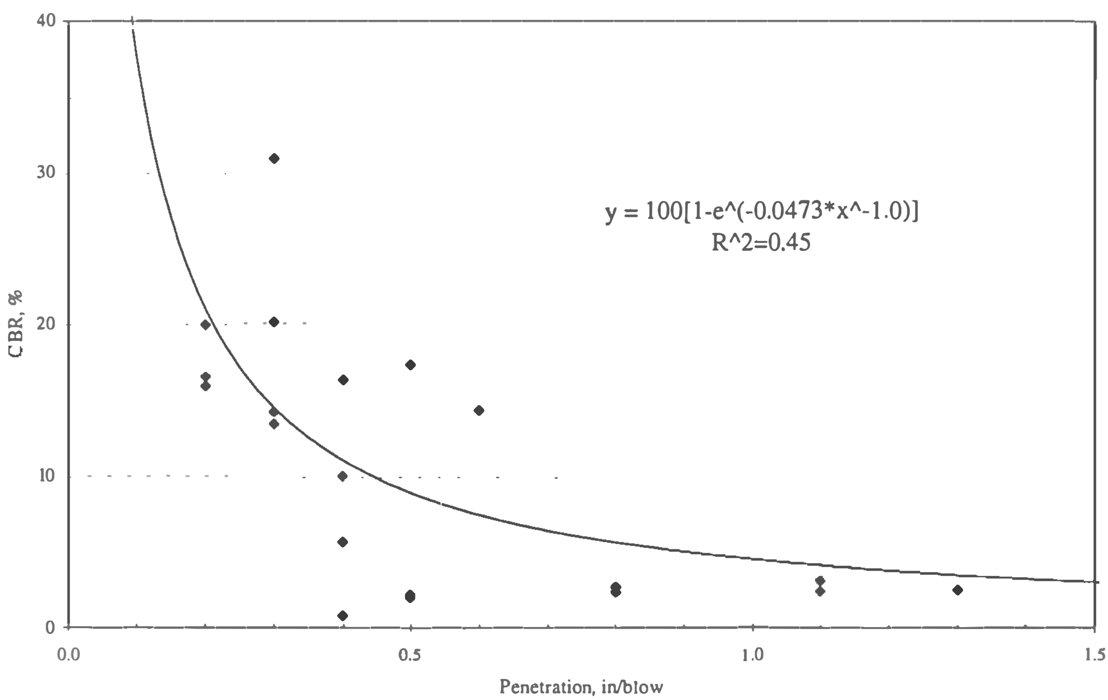


Figure 42. DCP Penetration Index vs. CBR—all sites, phases I and II.



CONCLUSIONS

Following is a summary of the important findings of this study:

- DCP, FWD, and CBR testing was conducted over seven test sites comprised of typical subgrade materials in Oklahoma. Due to the inability to perform DCP testing over three of the original sections, the results from only four sites were used in the development of the correlation equations presented in this report.

- DCP testing was conducted at the rate of approximately 30 tests per site using a standard DCP to a nominal depth of 30 in. Penetration curves were generated and analyzed for each test point, and the average PI between depths of 6 to 24 in was calculated for each test. PI profiles show sites 1 and 2 producing low penetration values, while site 3 resulted in very high penetrations. Site 4 was variable and produced medium to high PI.

- FWD data were used to backcalculate resilient moduli at the same locations at which DCP tests were conducted. A revision of the FWD results by sensor determined that the backcalculated moduli based on the maximum deflection (D0) produced the most reasonable values for the different soil types. In terms of maximum deflection, site 1 produced the lowest and most uniform deflection profile, indicating a good quality, uniform subgrade. On the other hand, site 4 resulted in the highest deflections and was the only site to show a significant difference in results with respect to season of testing.

- Backcalculated M_r values ranged from approximately 1,500 to 10,000 psi, with site 1 containing values in the higher range and site 3 consisting of low values. Results for the clay and silt subgrades are low and typical of these soil types. Backcalculated results for the sand subgrade are slightly low for this material, but reasonable.

- The results of laboratory, soaked CBR tests show results in basically two categories: sands and silts producing CBRs greater than 10 percent and reaching a maximum of 40 percent (depending upon the degree of compaction) and clay materials of CBRs less than 5 percent.

- Correlations between M_r and DCP data show different trends for each soil type in terms of magnitude and dispersion of data. When analyzed as a single data set, the results can be defined by a linear, inverse relationship between M_r and DCP. For the minimum and maximum DCP values encountered in this study, the aggregate correlation equation predicts M_r ranging from approximately 5,500 to 1,500 psi. These results are very typical for fine-grained soils, such as clays and silts. To obtain slightly more realistic results for granular materials, such as sands and gravels, it is recommended to use the individual correlation equation determined for site 1 of this study.

- A good correlation was found between DCP and soaked CBR data, with results generally characterized by two types: low DCP values (less than 0.4 in/blow) producing CBRs greater than 10 percent and reaching a maximum of 40 percent, and medium to high PI (0.5 to 1.5 in/blow) corresponding to very low CBRs, typically less than 5 percent. Although the estimated CBRs appear low when compared to other published correlation equations, this difference is due primarily to the difference in moisture content used for CBR testing. Most correlations are based on in situ CBR tests, while the soaked CBR tests are more representative of critical moisture states and give values consistent with current ODOT design procedures for determination of design subgrade value.

- Selection of the appropriate correlation equation for predicting either CBR or M_r is based on knowledge of the material being tested. If the material type is known, the correlation equation corresponding to either sand, silt, or clay should be selected. The general equations should be used if the material characteristics are unknown.

- The seasonal variation in DCP, CBR, and M_r was not found to have a substantial effect in this study due to minimal moisture changes during the 2 phases of testing. However, other studies have shown that moisture sensitive materials show a decrease in bearing capacity and stiffness with an increase in moisture content. Therefore, a more conservative support value for design purposes will be obtained if DCP testing is conducted during periods corresponding to high moisture content.

Recommendations for Continued Research

The results of this study verify the application of an existing DCP/CBR correlation for three broad categories of Oklahoma soils including clay, silt, and sand. The research also establishes DCP/M_r correlations for the same soil types. The DCP/M_r correlations are the only ones of their type found in the literature.

The following recommendations for further research will facilitate implementation of the developed correlations and the continued refinement of the correlations presented:

- The three broad categories of Oklahoma soils investigated should be expanded to include additional soil series.
- Laboratory M_r values should be determined for selected soil types to verify the backcalculated values.
- In situ CBR tests should be performed at selected sites, in addition to laboratory CBR tests.
- A computer program for DCP data entry and analysis should be developed to aid in using the developed correlations and promote more widespread use of the DCP.
- A computer program for the design of thinly surfaced and unsurfaced low-volume roads should be developed using DCP PI values to estimate soil strength/deformation properties.

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APPENDIX A

DCP penetration curves and PI vs. depth curves for all testpoints at all sites, both phases.

Figure A-1. DCP penetration curves—site 1, phase I (sta –5+00 to 0+50).

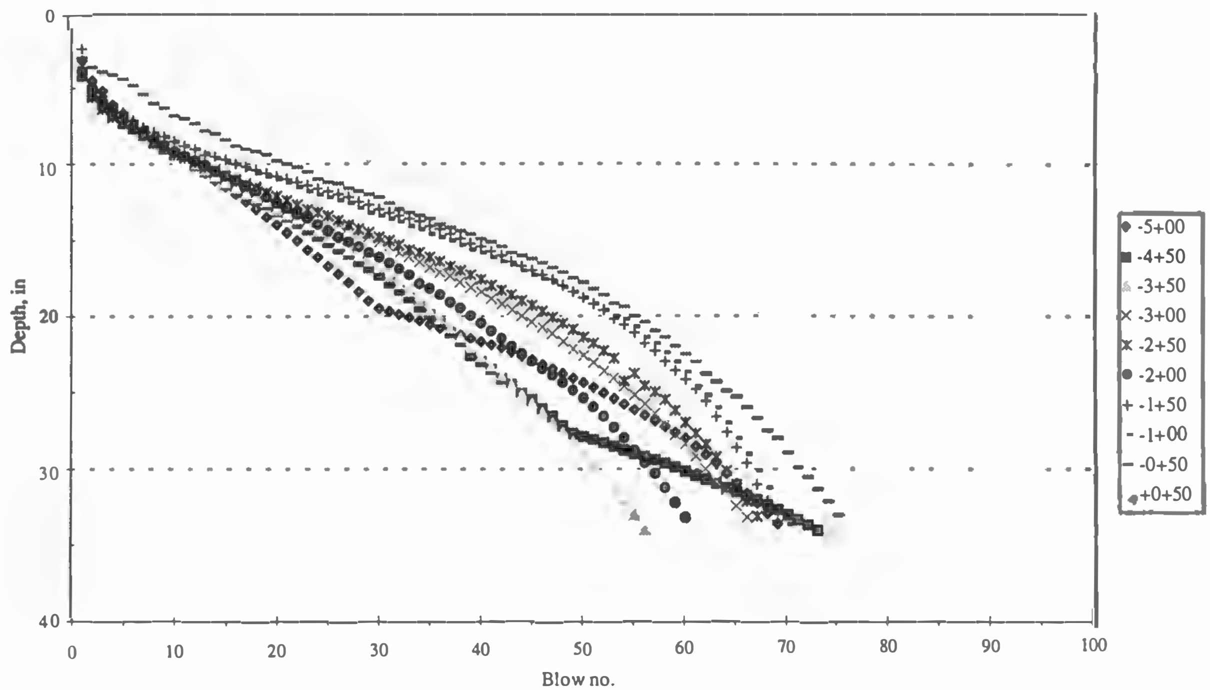


Figure A-2. DCP penetration curves—site 1, phase I (sta 1+00 to 5+50).

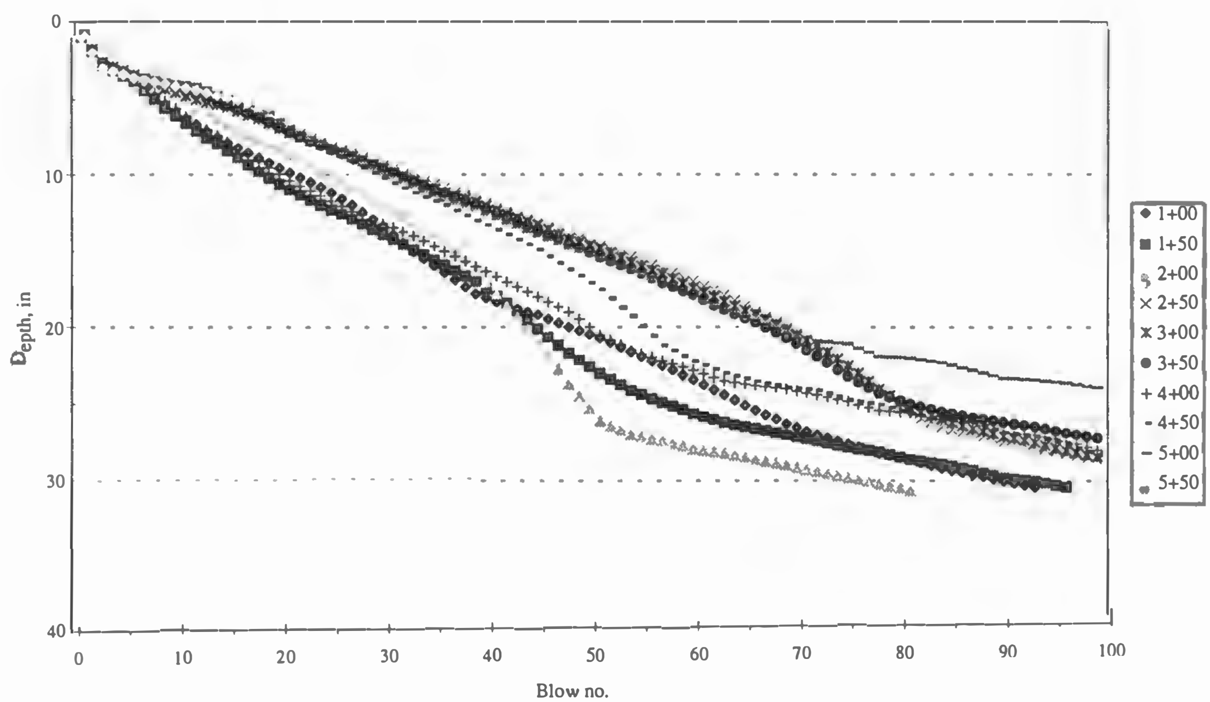


Figure A-3. DCP penetration curves—site 1, phase I (sta 6+00 to 9+50).

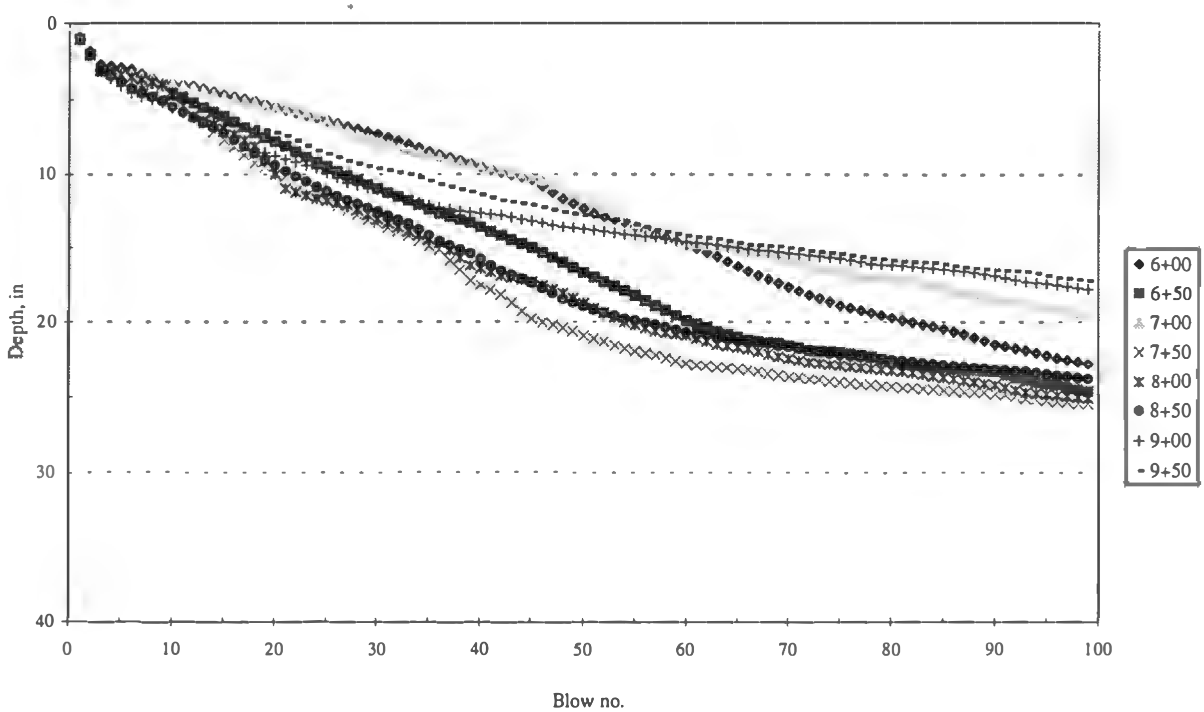


Figure A-4. DCP penetration curves—site 2, phase I (sta 0+50 to 5+00).

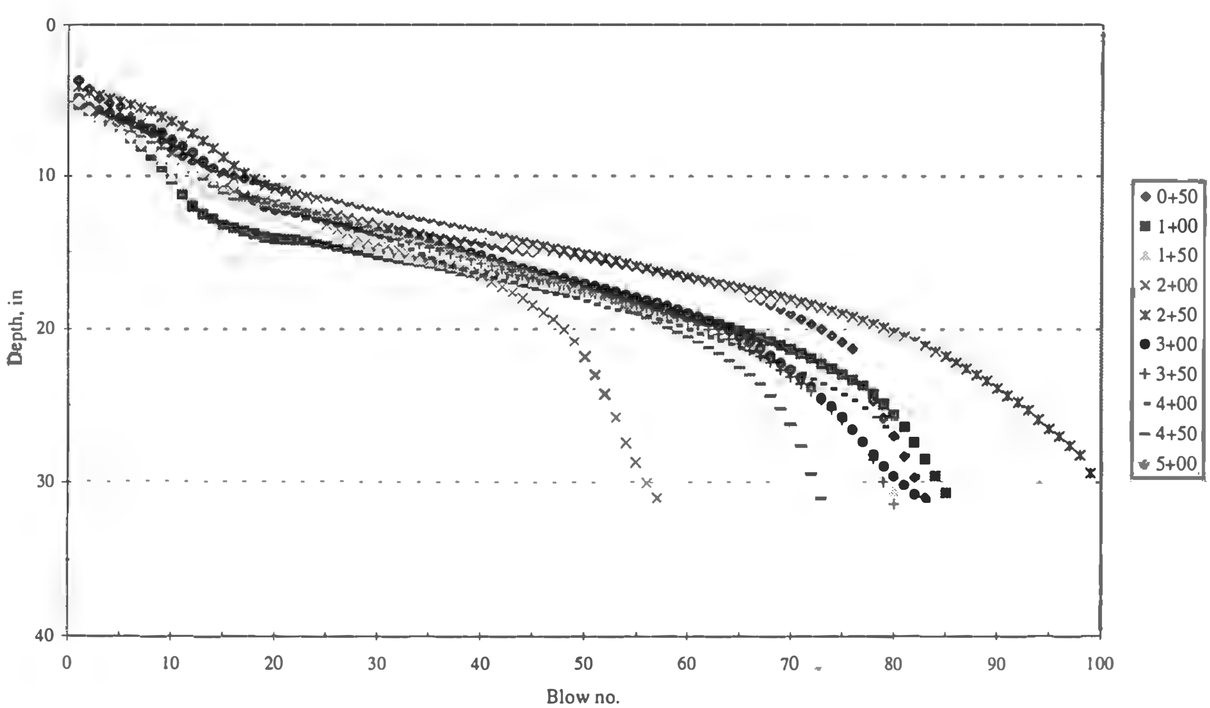


Figure A-5. DCP penetration curves—site 2, phase I (sta 5+50 to 10+00).

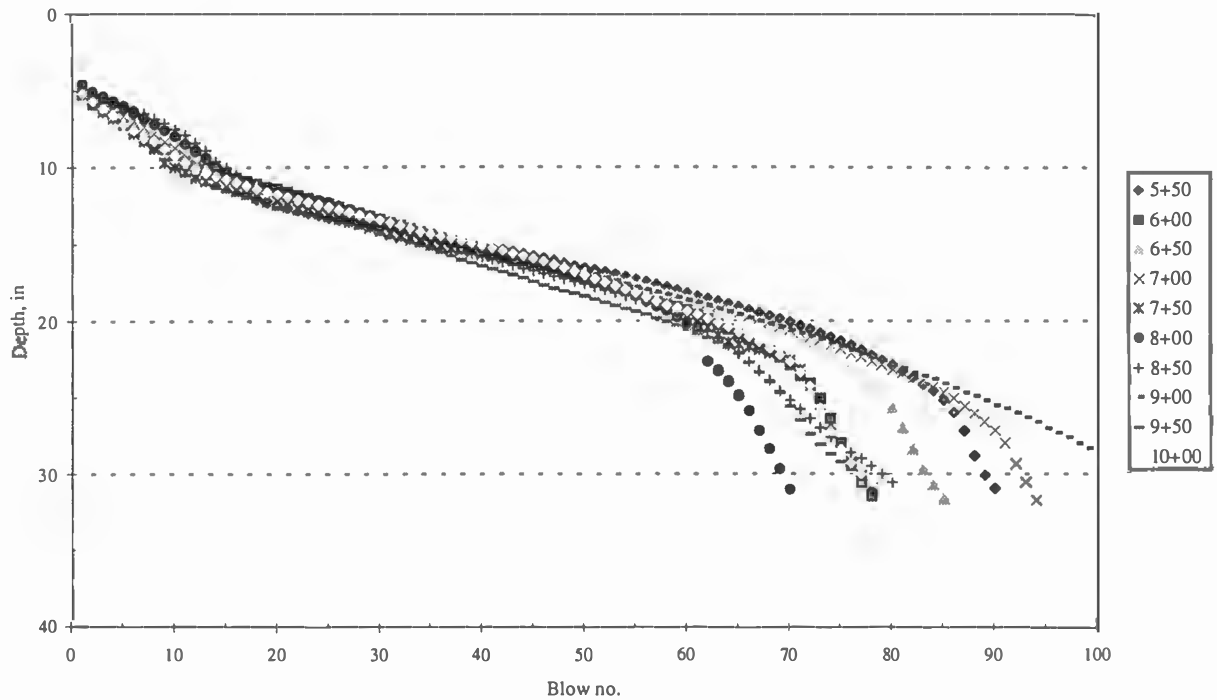


Figure A-6. DCP penetration curves—site 2, phase I (sta 10+50 to 14+50).

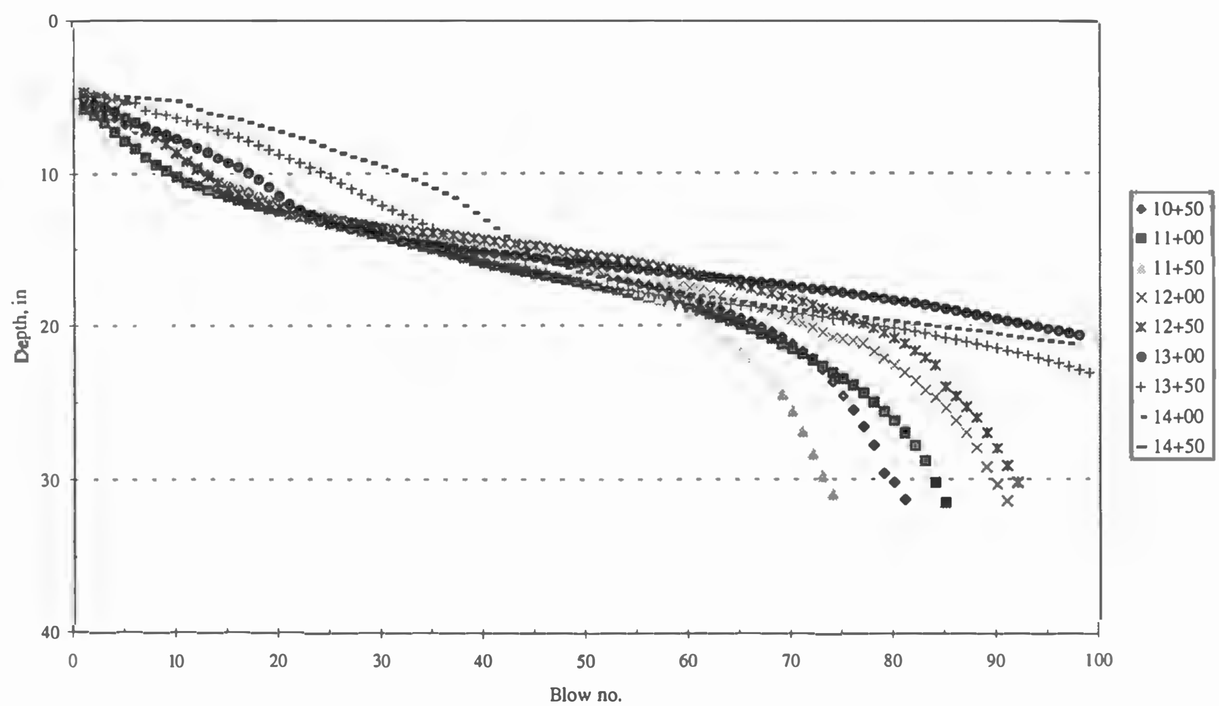


Figure A-7. DCP penetration curves—site 3, phase I (sta 0+50 to 5+00).

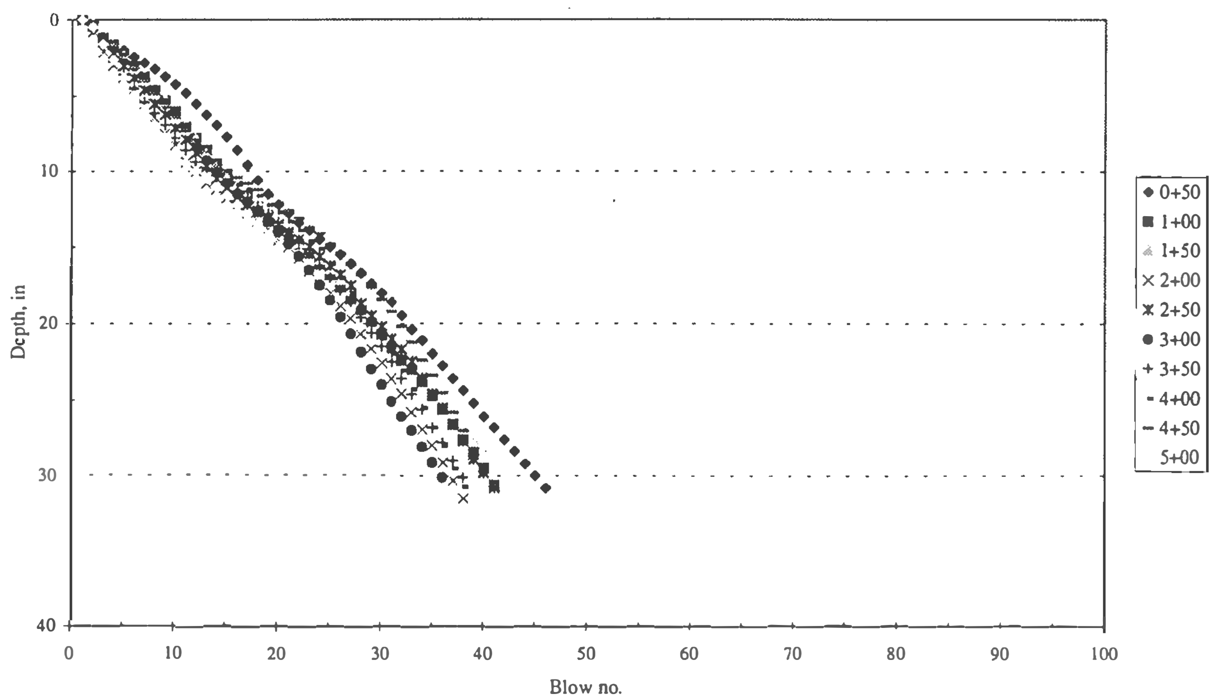


Figure A-8. DCP penetration curves—site 3, phase I (sta 5+50 to 10+00).

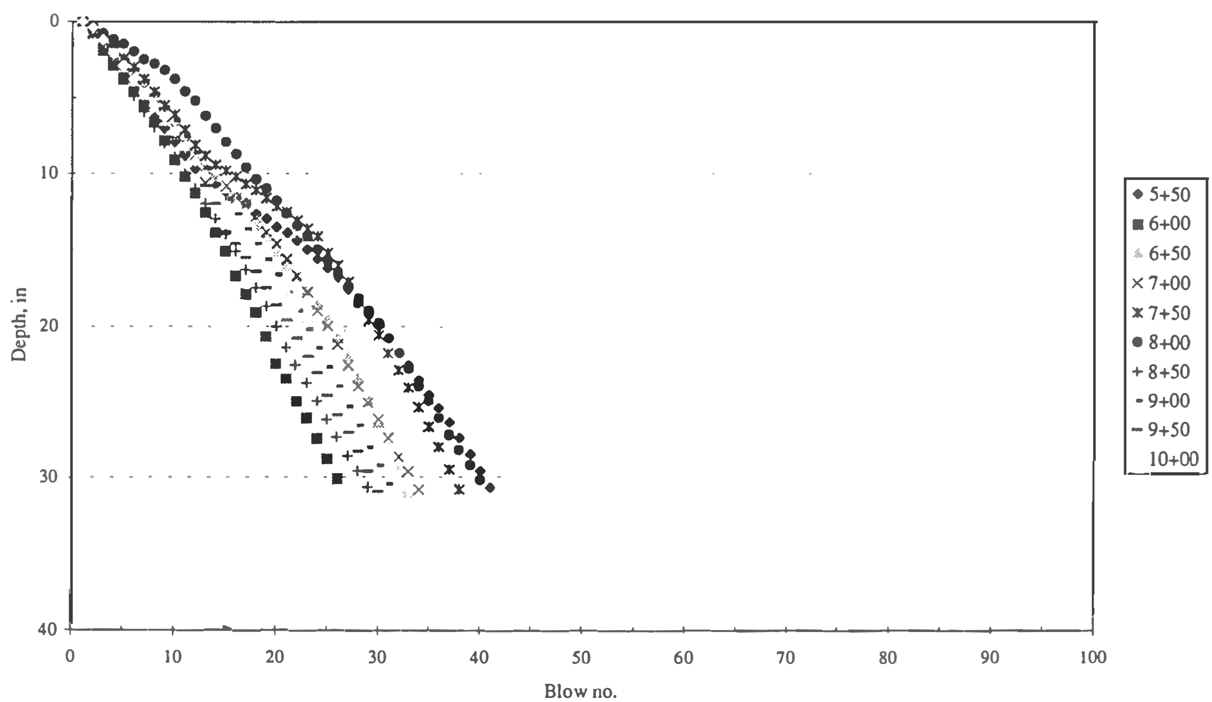


Figure A-9. DCP penetration curves—site 2, phase I (sta 10+50 to 15+00).

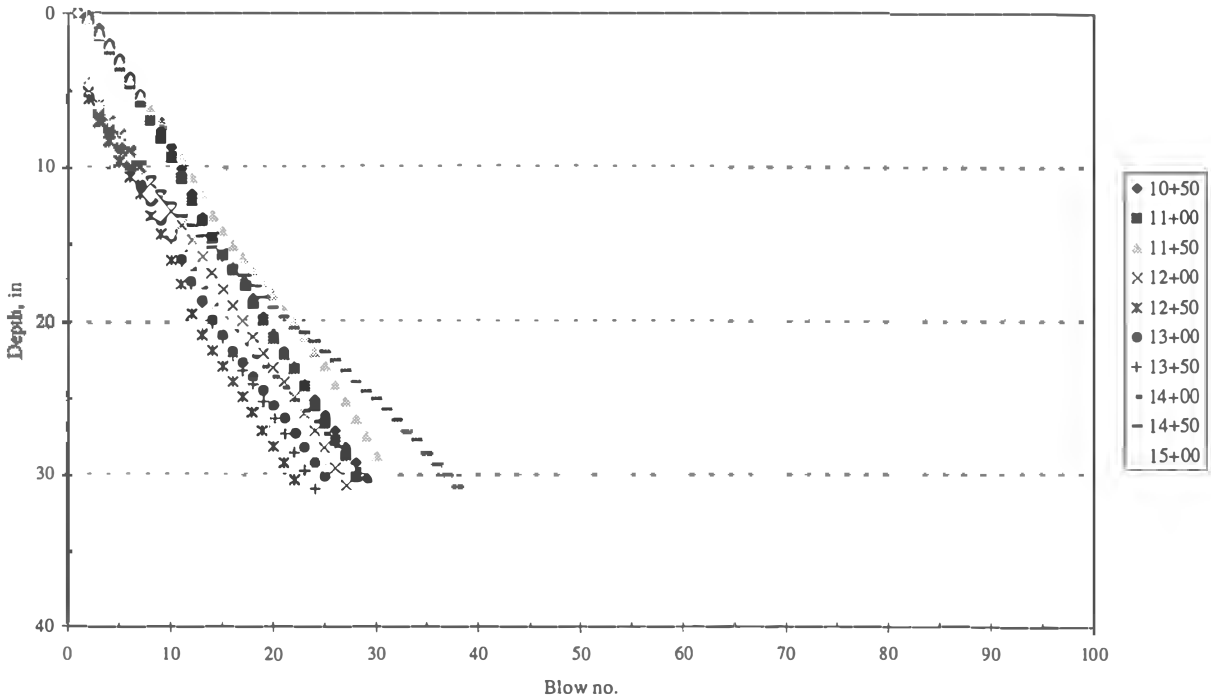


Figure A-10. DCP penetration curves—site 5, phase I (sta 0+50 to 5+00).

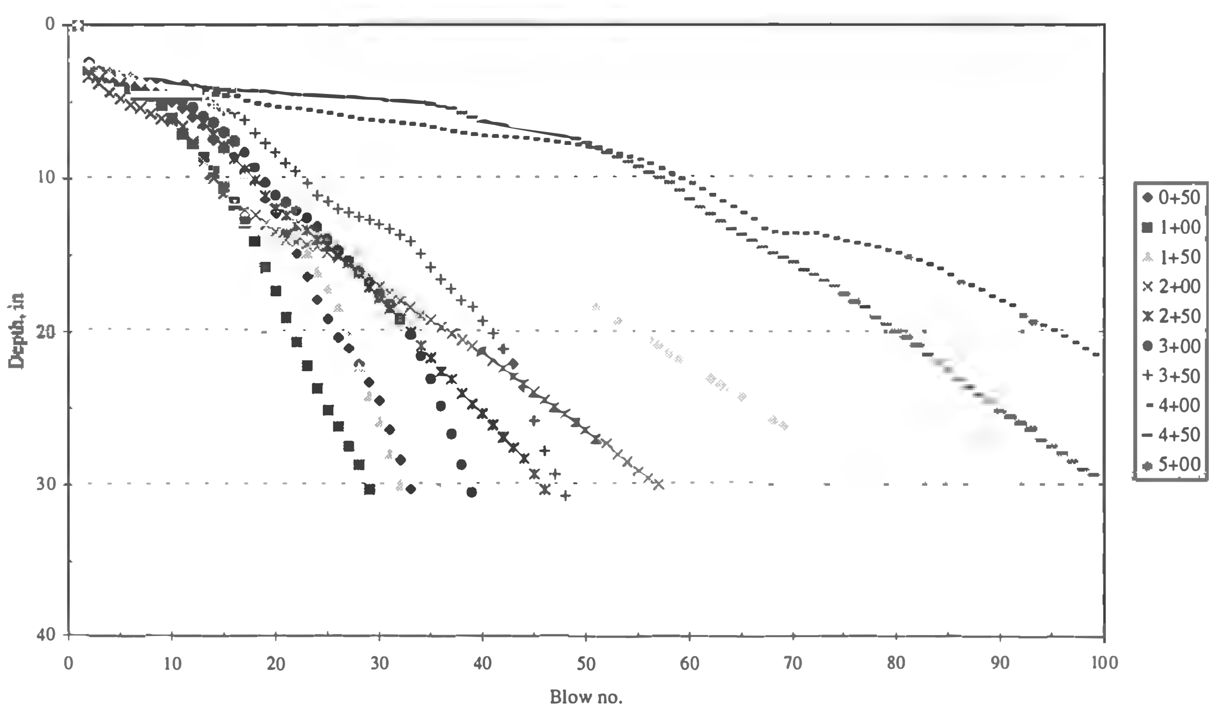


Figure A-11. DCP penetration curves—site 5, phase I (sta 5+50 to 12+00).

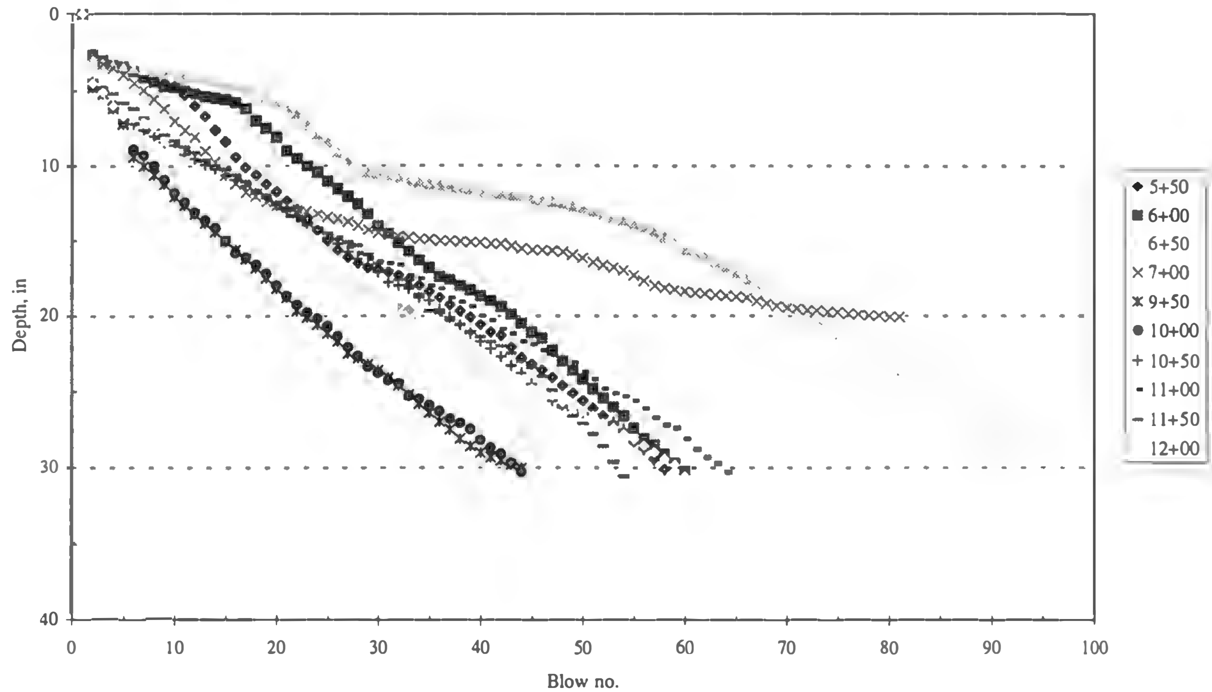


Figure A-12. DCP penetration curves—site 5, phase I (sta 12+50 to 17+00).

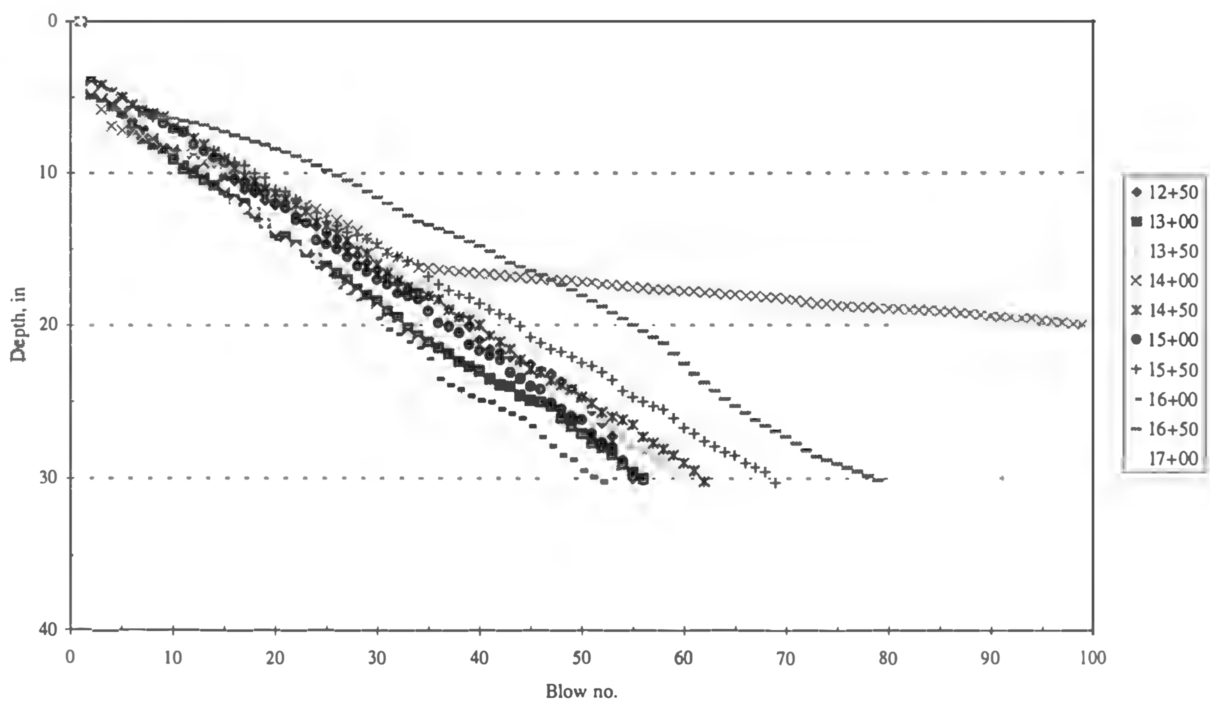


Figure A-13. DCP penetration curves—site 1, phase II (sta –5+00 to –0+50).

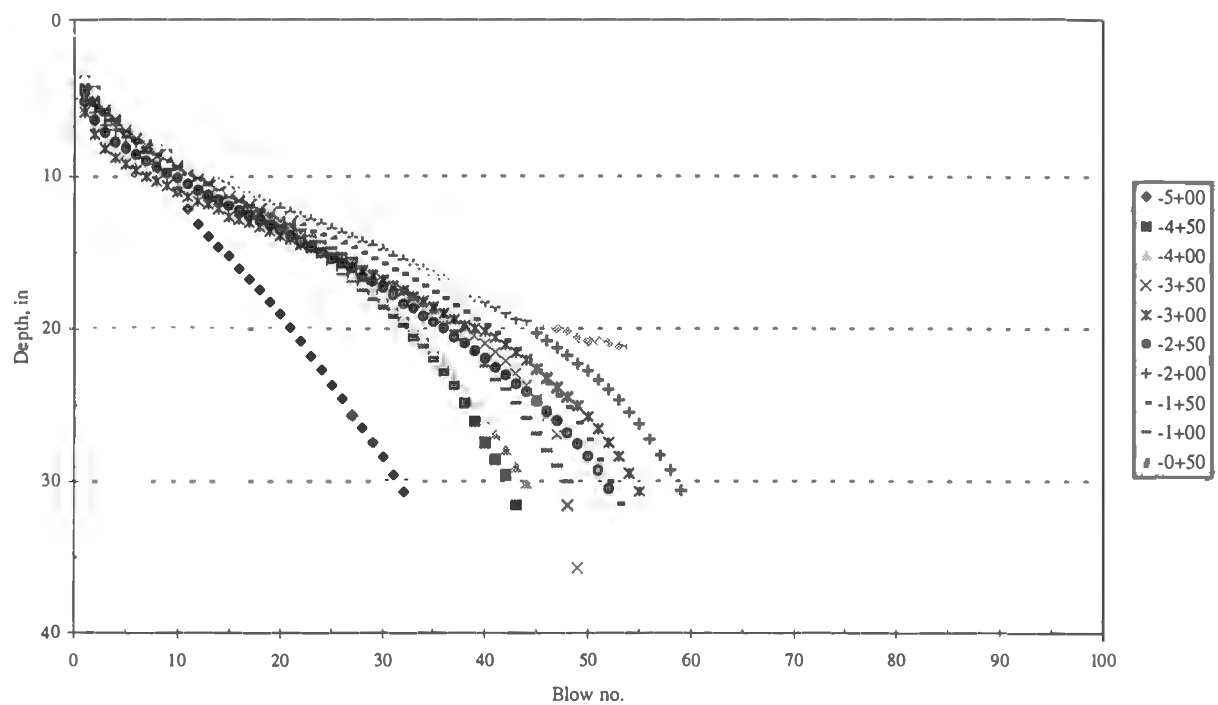


Figure A-14. DCP penetration curves—site 1, phase II (sta 0+50 to 5+00).

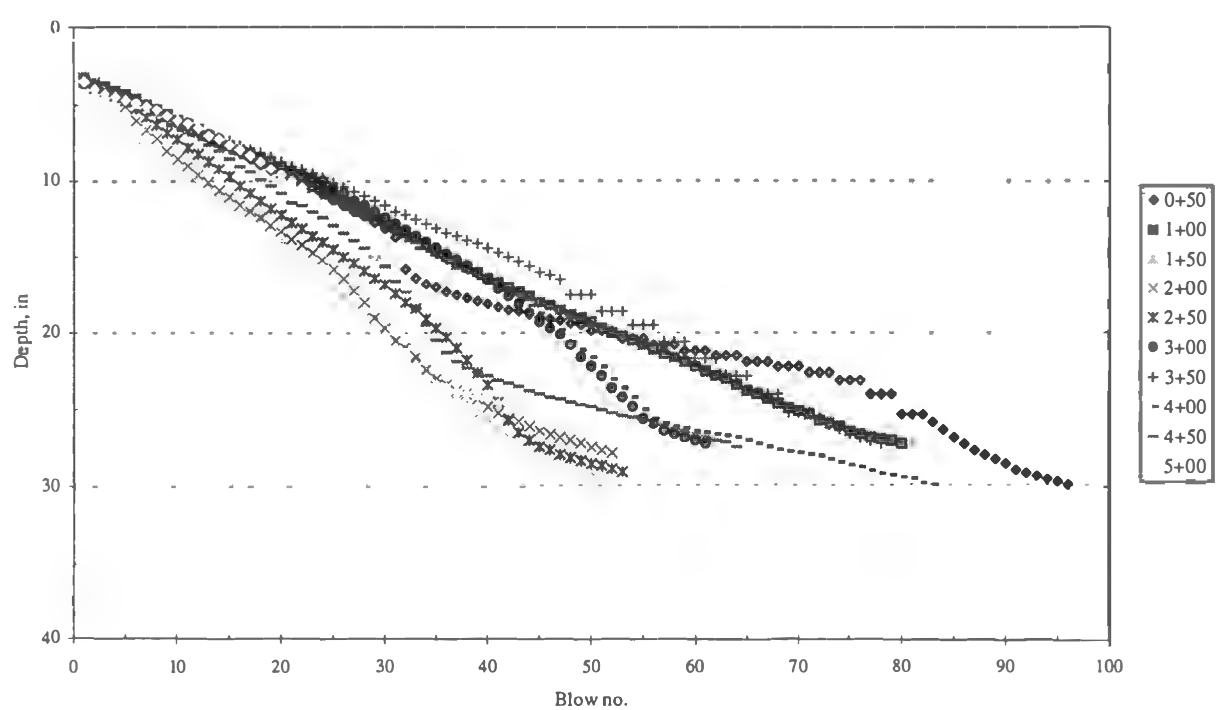


Figure A-15. DCP penetration curves—site 1, phase II (sta 5+50 to 10+00).

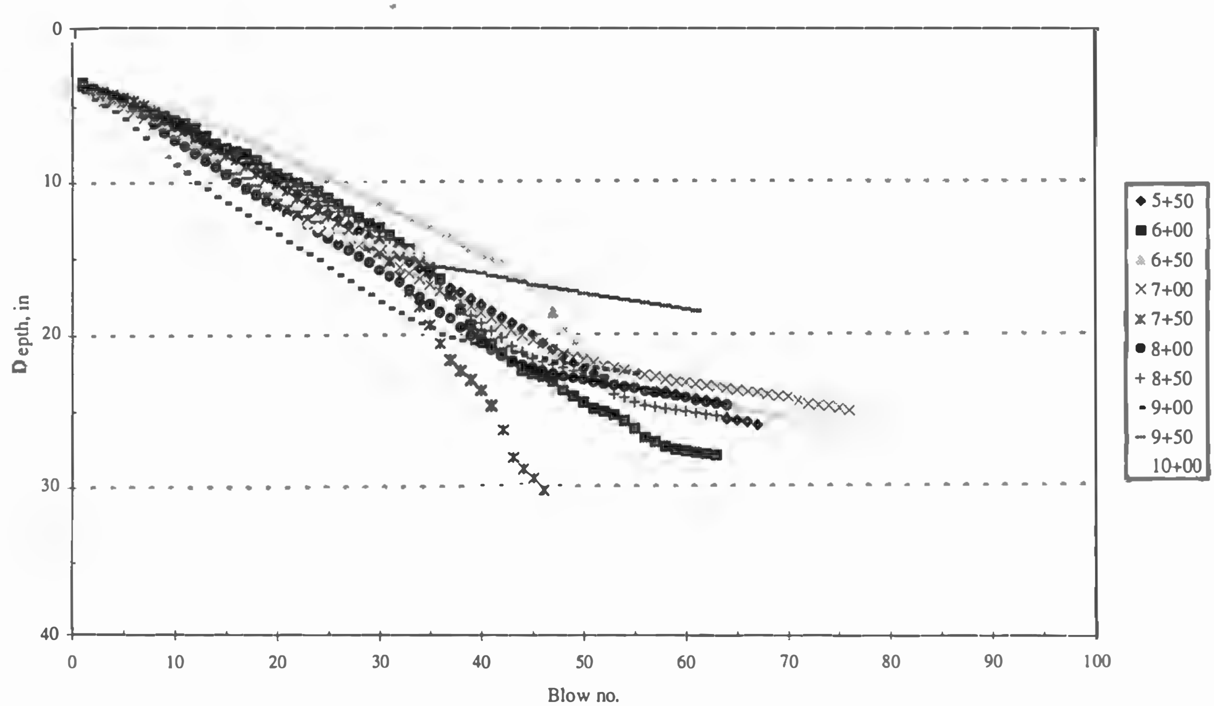


Figure A-16. DCP penetration curves—site 2, phase II (sta 0+50 to 5+00).

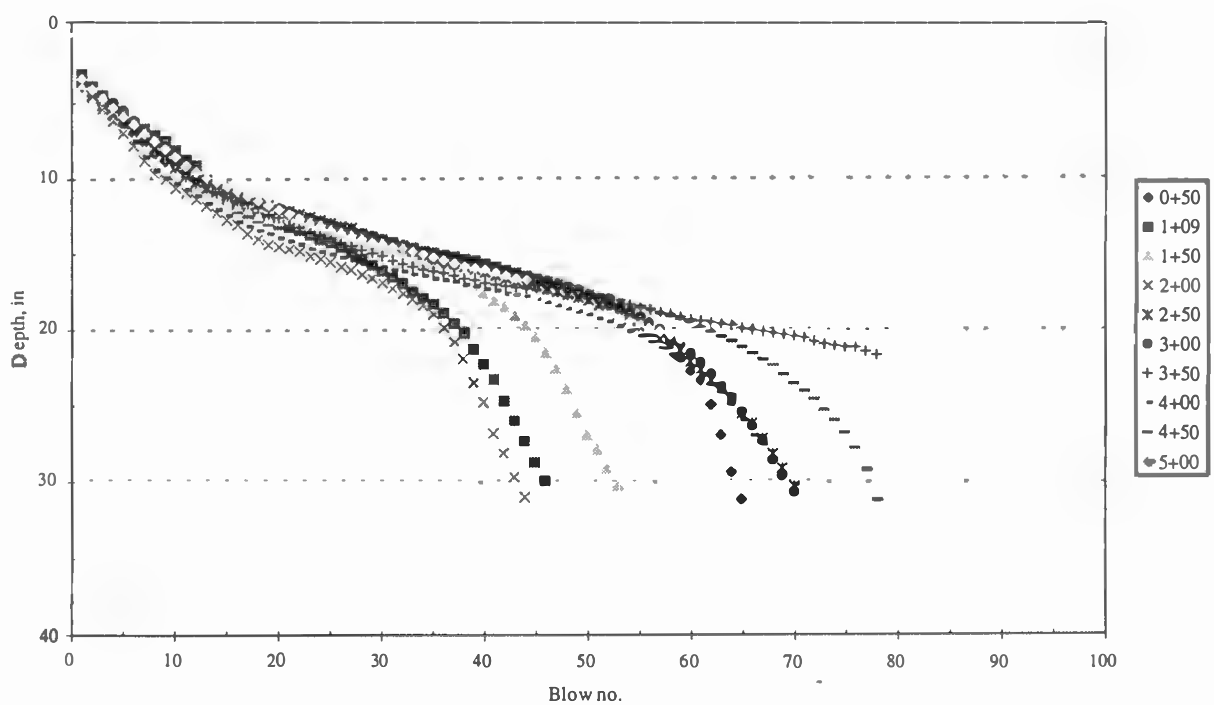


Figure A-17. DCP penetration curves—site 2, phase II (sta 5+50 to 10+00).

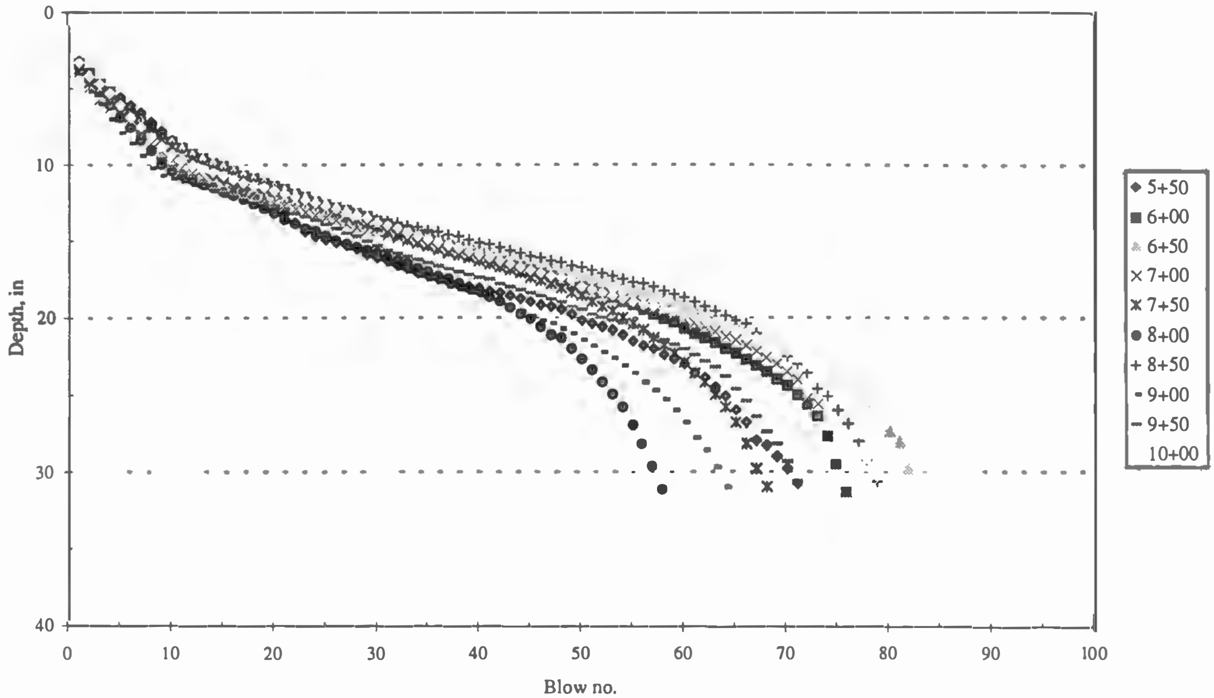


Figure A-18. DCP penetration curves—site 2, phase II (sta 10+50 to 15+00).

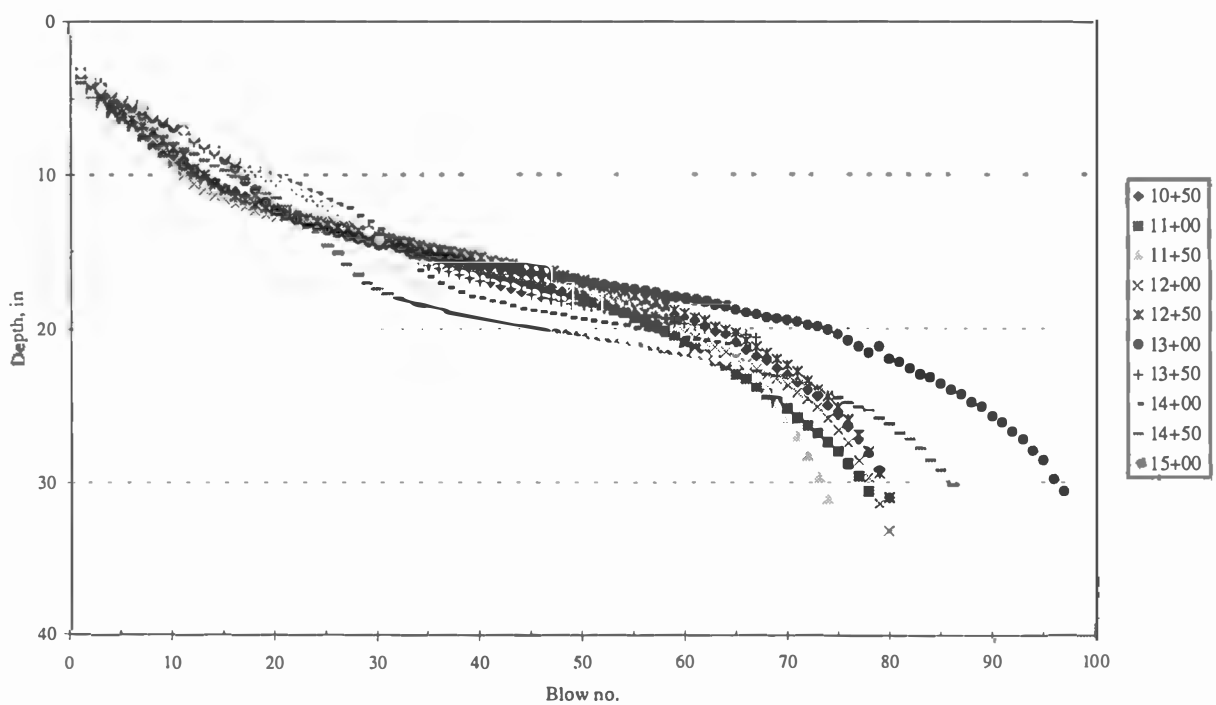


Figure A-19. DCP penetration curves—site 3, phase II (sta –5+00 to –0.50).

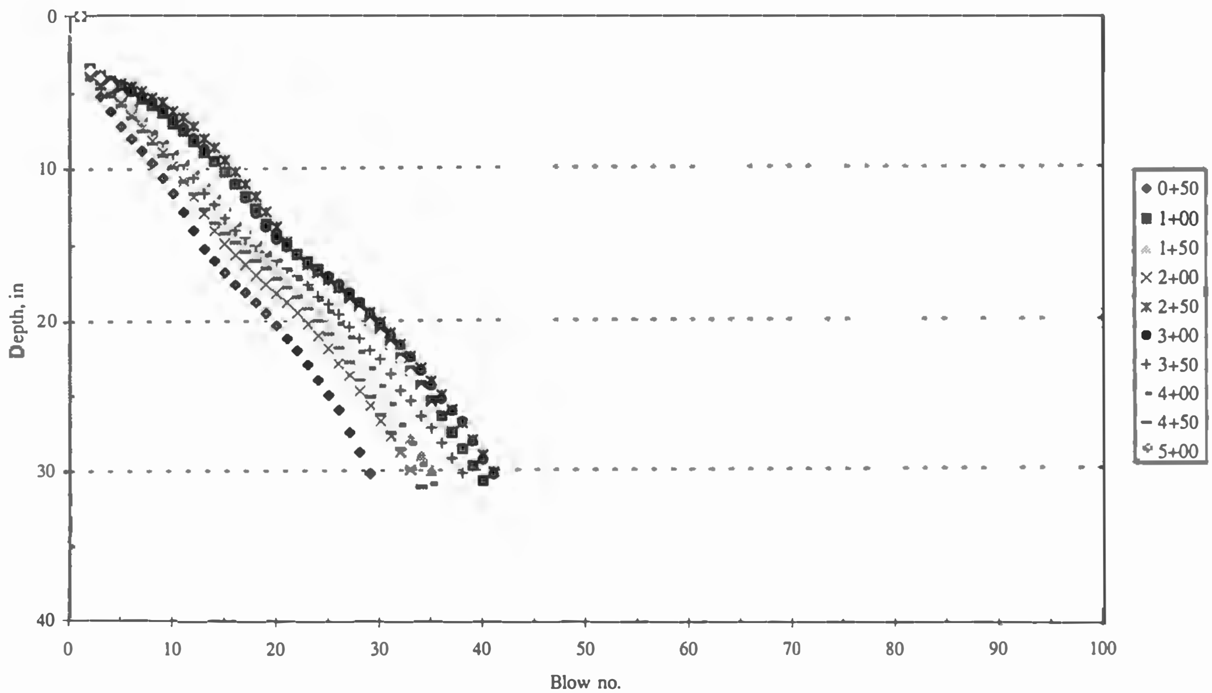


Figure A-20. DCP penetration curves—site 3, phase II (sta –5+00 to –0.50).

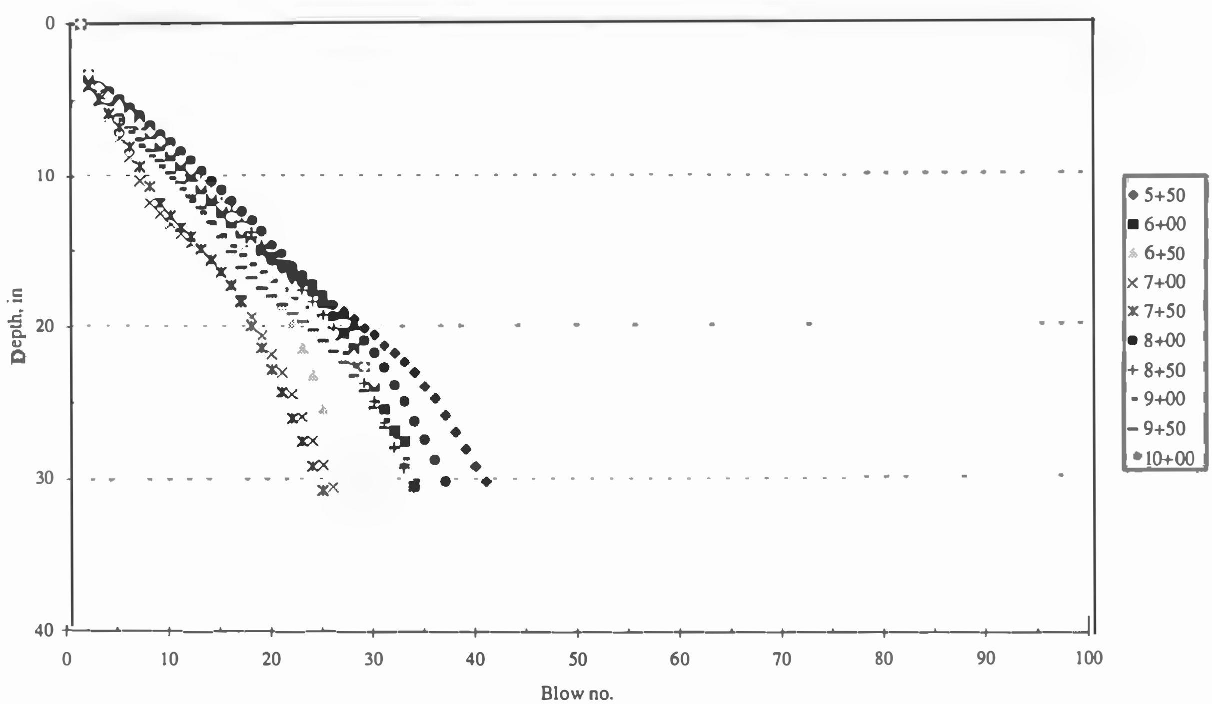


Figure A-21. DCP penetration curves—site 3, phase II (sta –5+00 to –0.50).

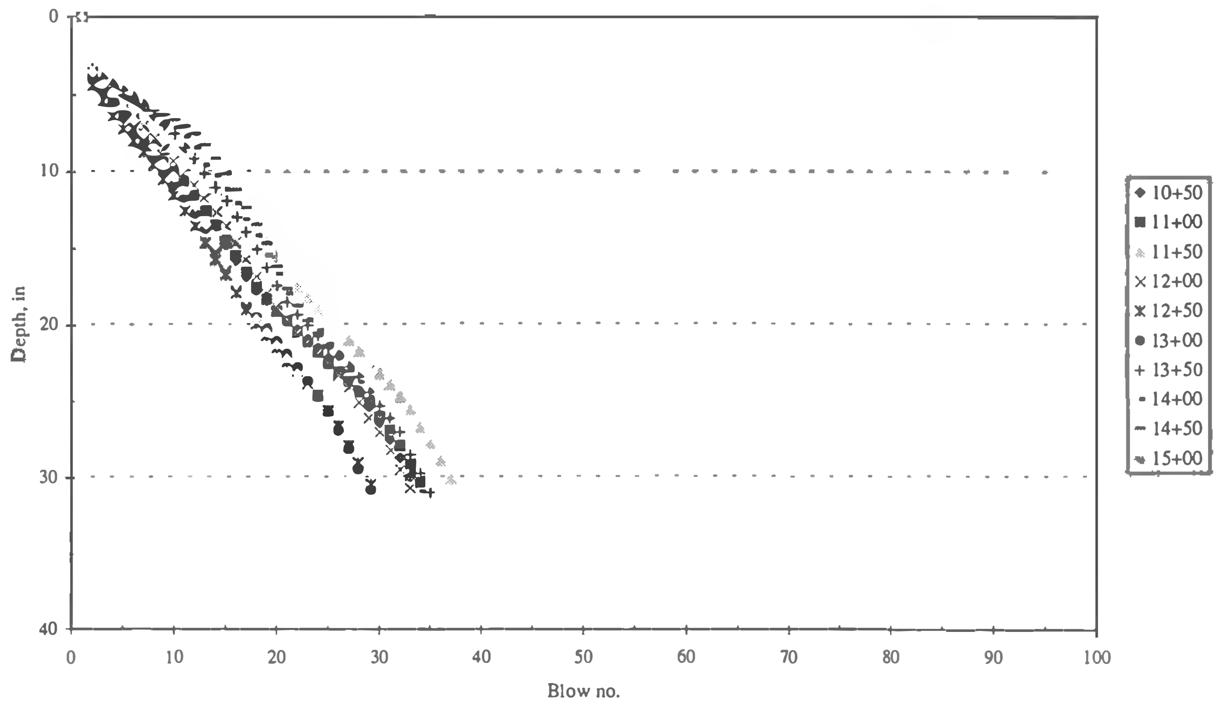


Figure A-22. DCP penetration curves—site 5, phase II (sta 0+50 to 5+00).

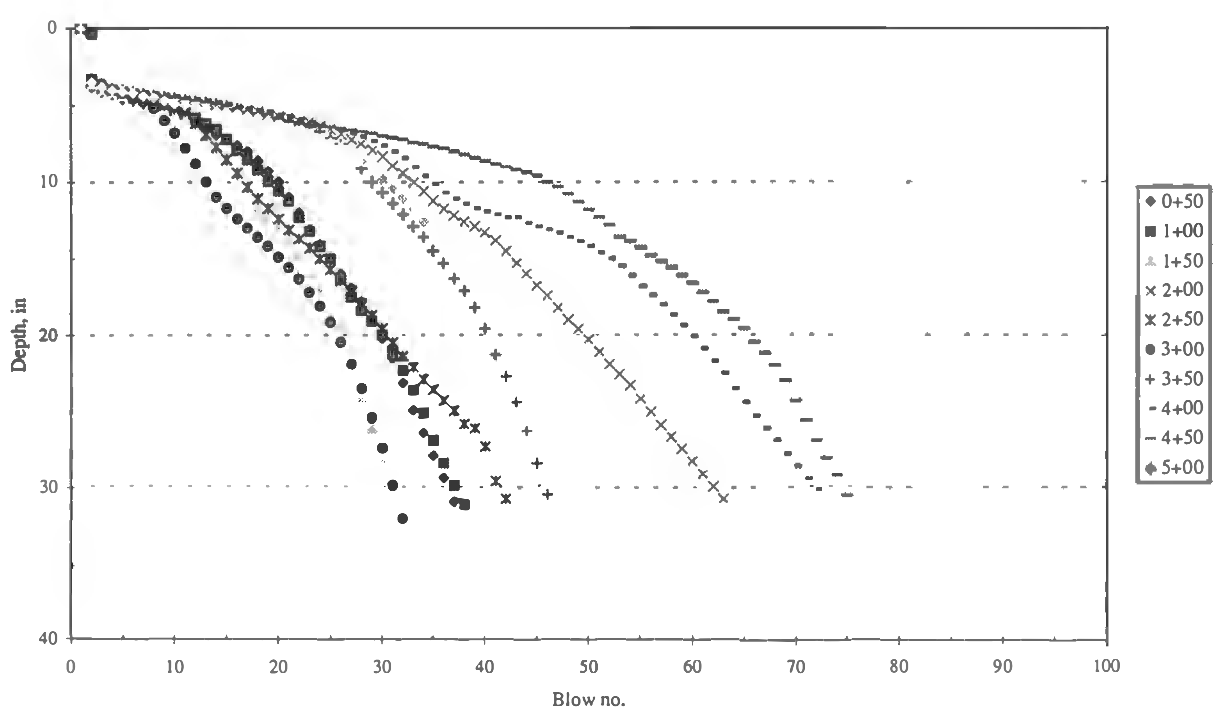


Figure A-23. DCP penetration curves—site 5, phase II (sta 5+50 to 12+50).

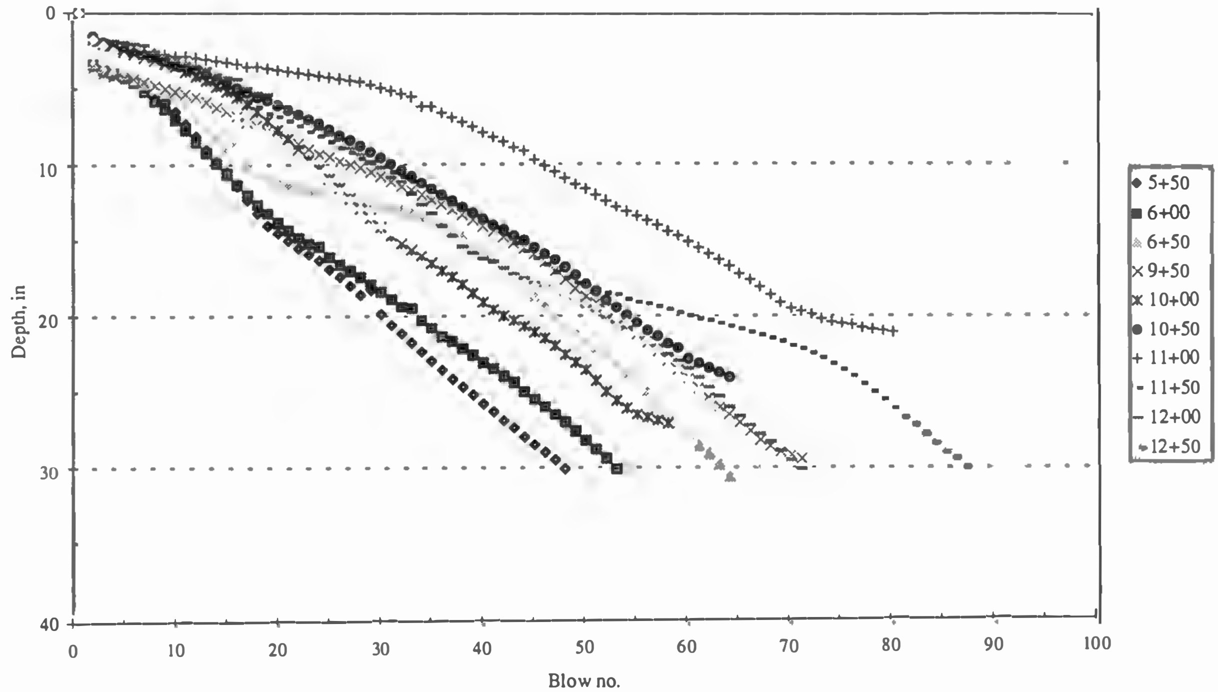
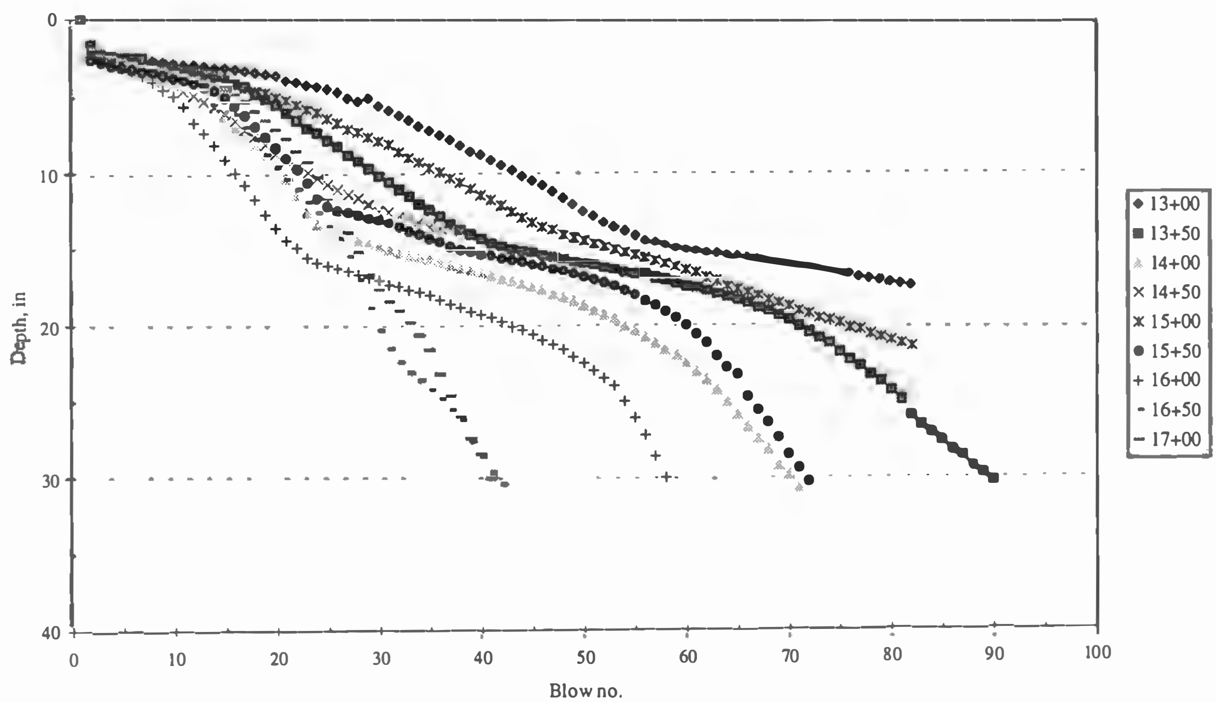


Figure A-24. DCP penetration curves—site 5, phase II (sta 13+00 to 17+00).



APPENDIX B

Alternative DCP vs. Mr curves using the 36-in sensor, both phases.

Figure B-1. DCP Penetration Index vs Mr at 36 in sensor—site 1, phase I.

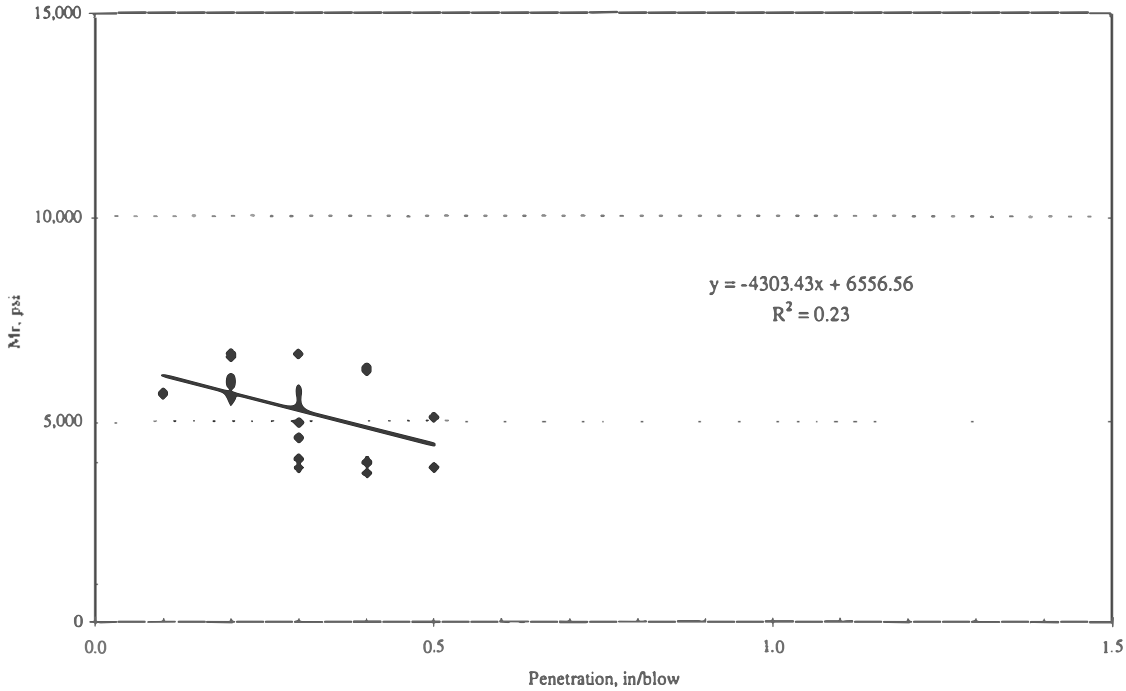


Figure B-2. DCP Penetration Index vs Mr at 36 in sensor—site 2, phase I.

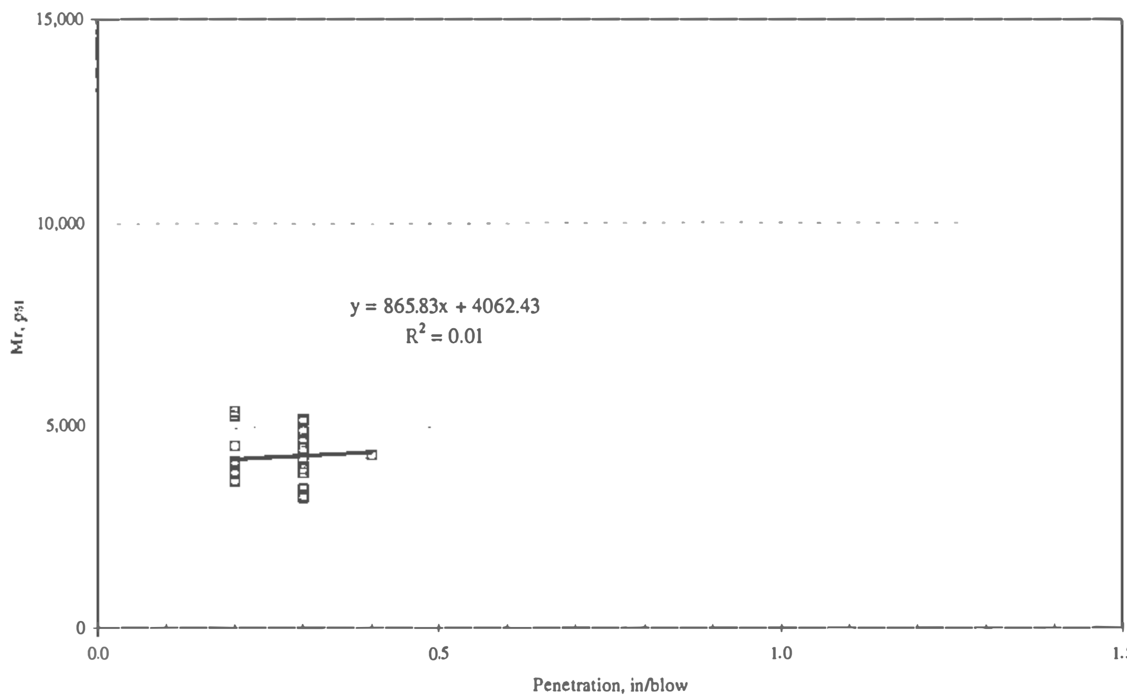


Figure B-3. DCP Penetration Index vs Mr at 36 in sensor—site 3, phase I.

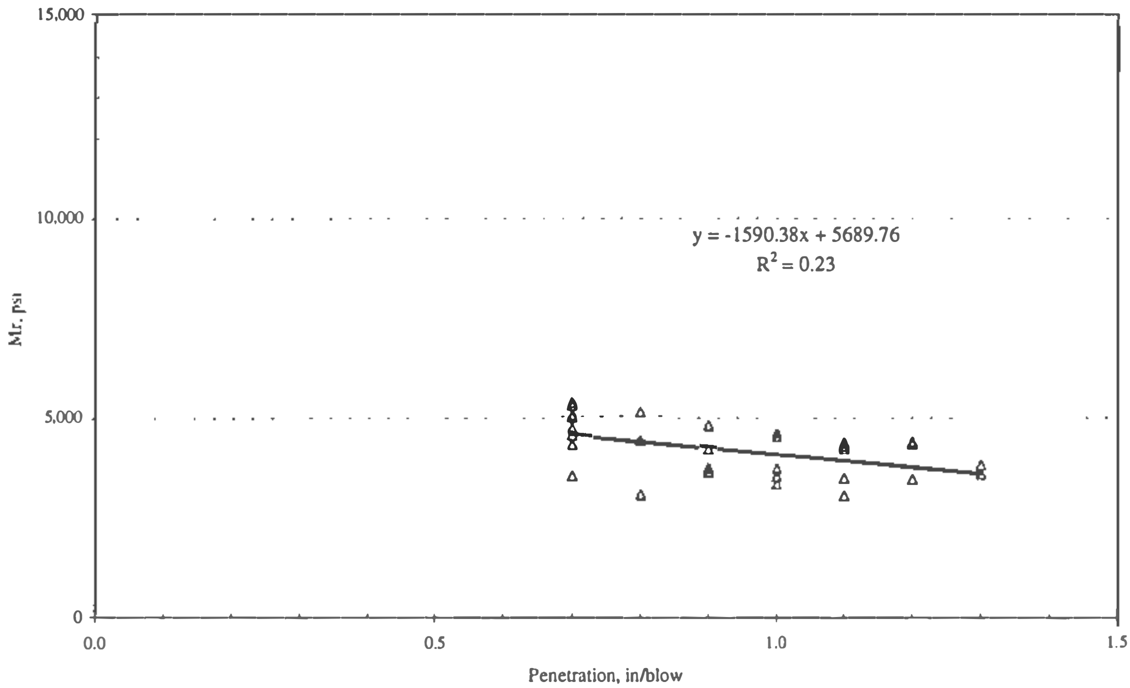


Figure B-4. DCP Penetration Index vs Mr at 36 in sensor—site 5, phase I.

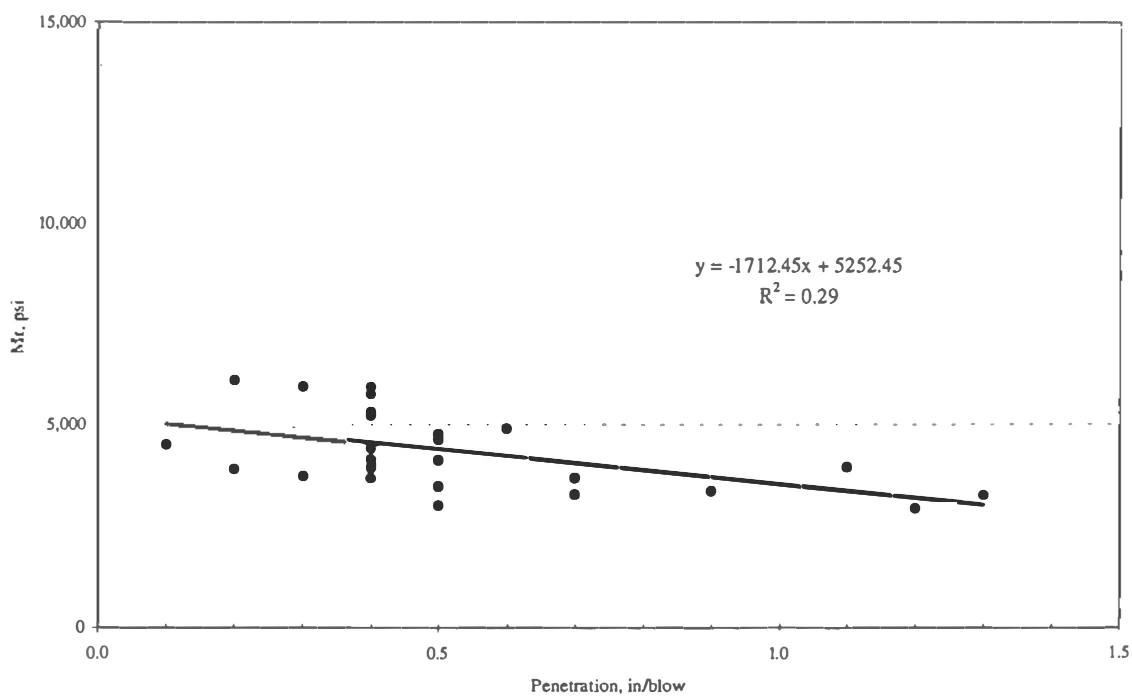


Figure B-5. DCP Penetration Index vs Mr at 36 in sensor—four sites, phase I.

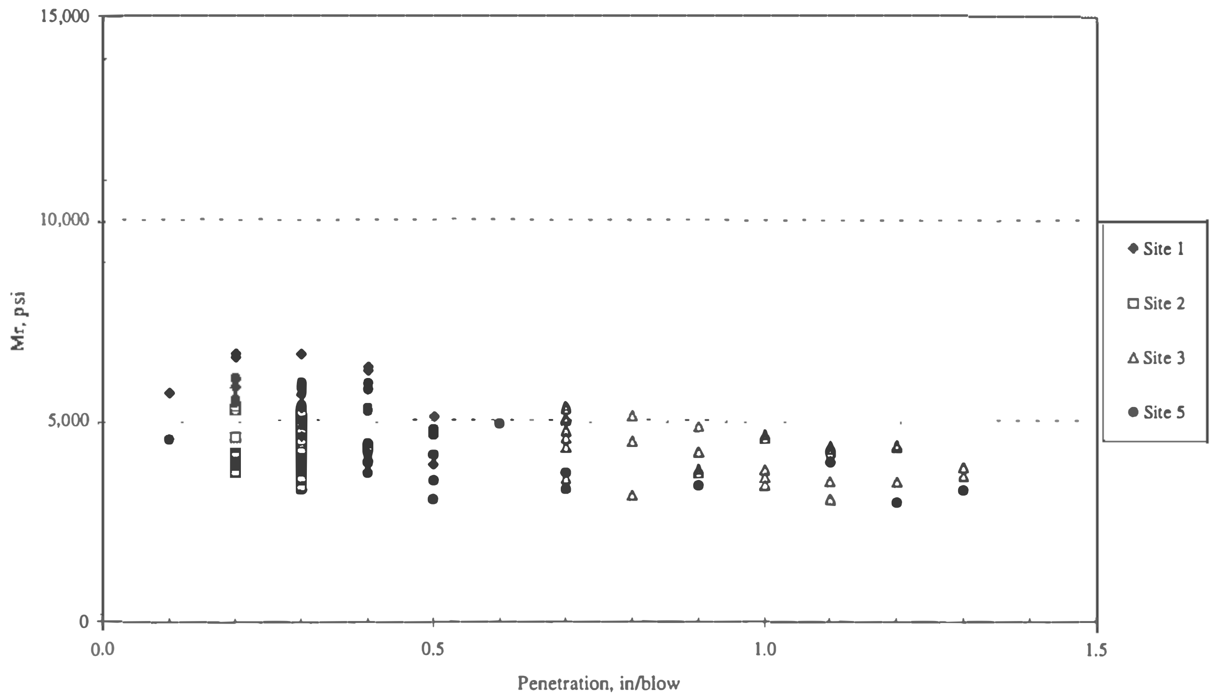


Figure B-6. DCP Penetration Index vs Mr at 36 in sensor—all sites, phase I.

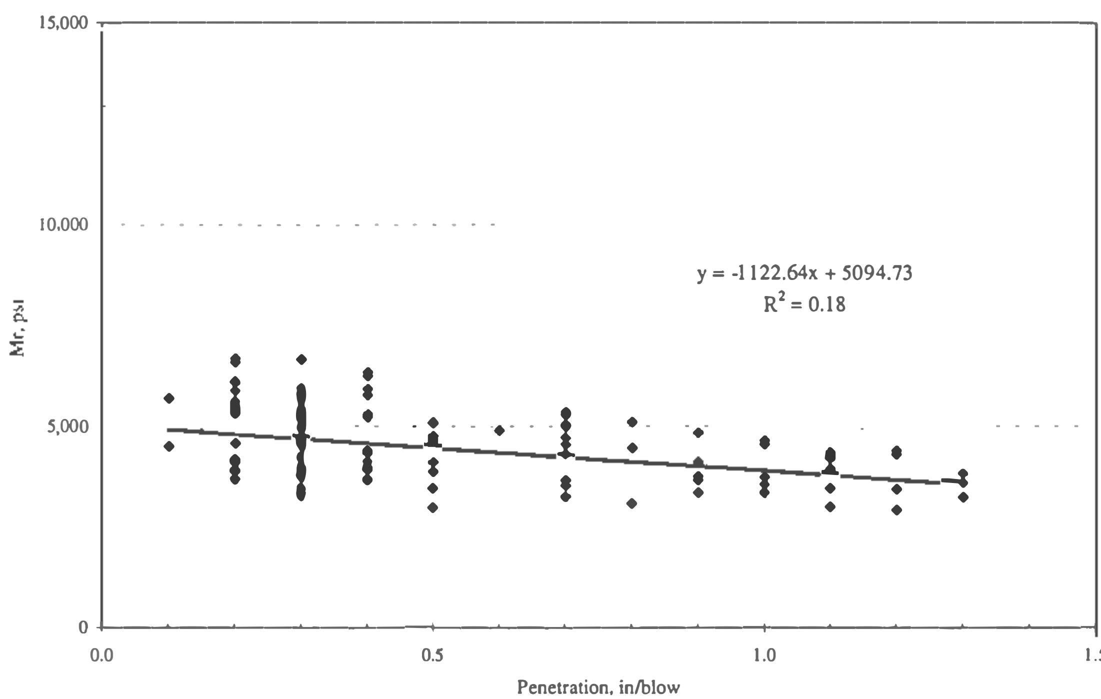


Figure B-7. DCP Penetration Index vs Mr at 36 in sensor—site 1, phase II.

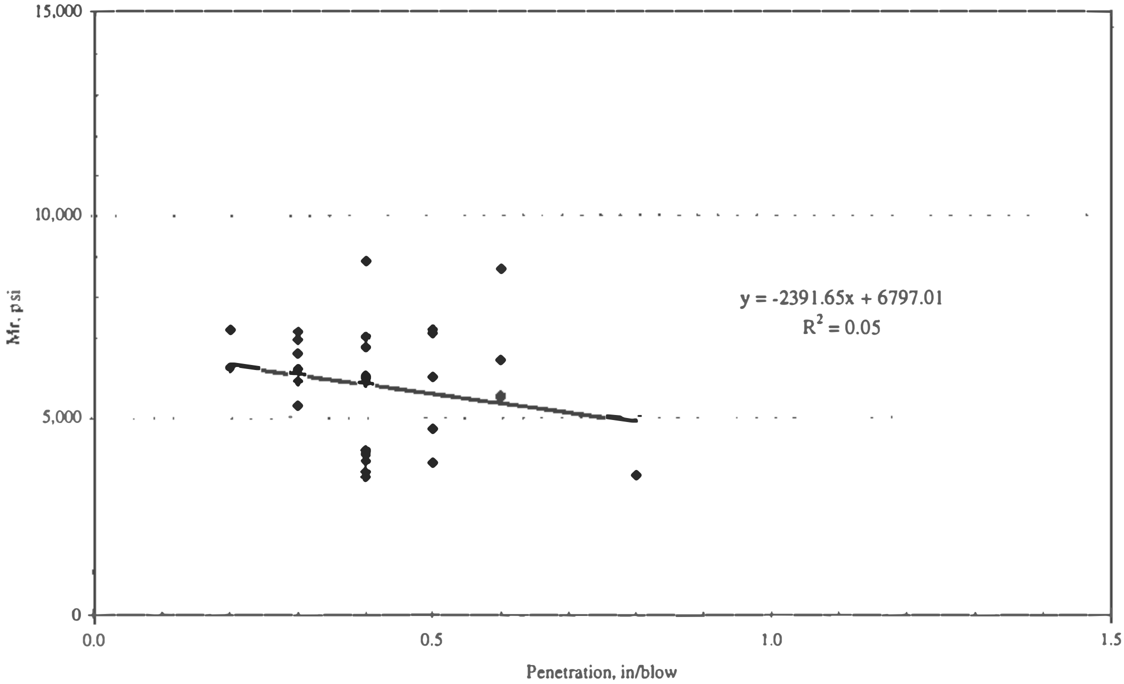


Figure B-8. DCP Penetration Index vs Mr at 36 in sensor—site 2, phase II.

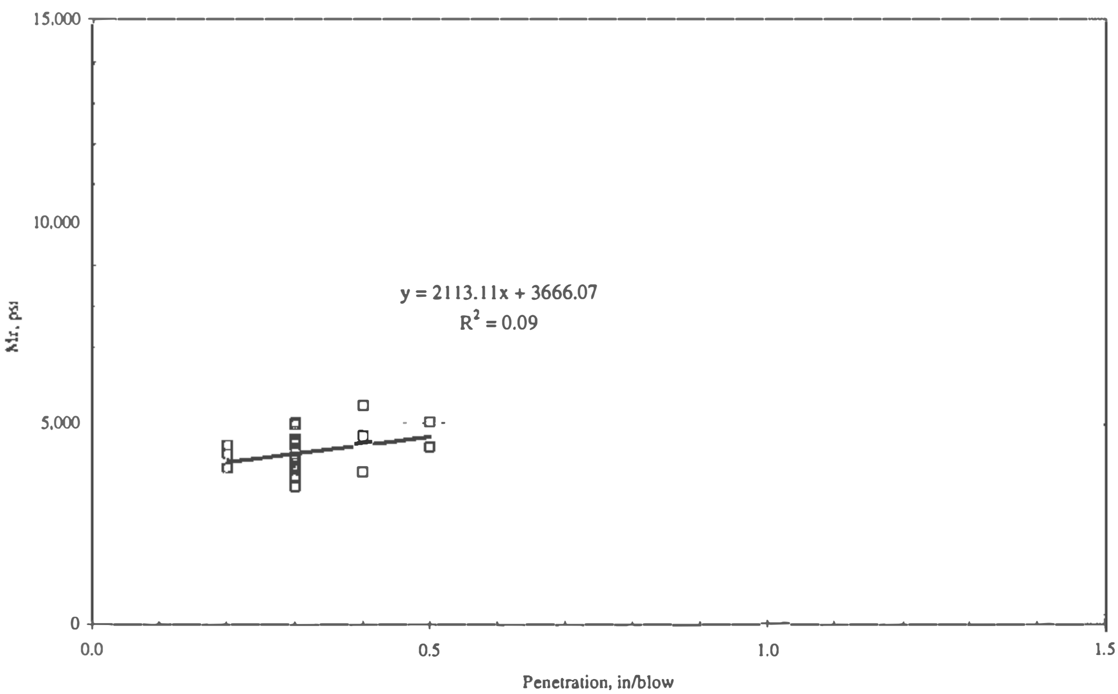


Figure B-9. DCP Penetration Index vs Mr at 36 in sensor—site 3, phase II.

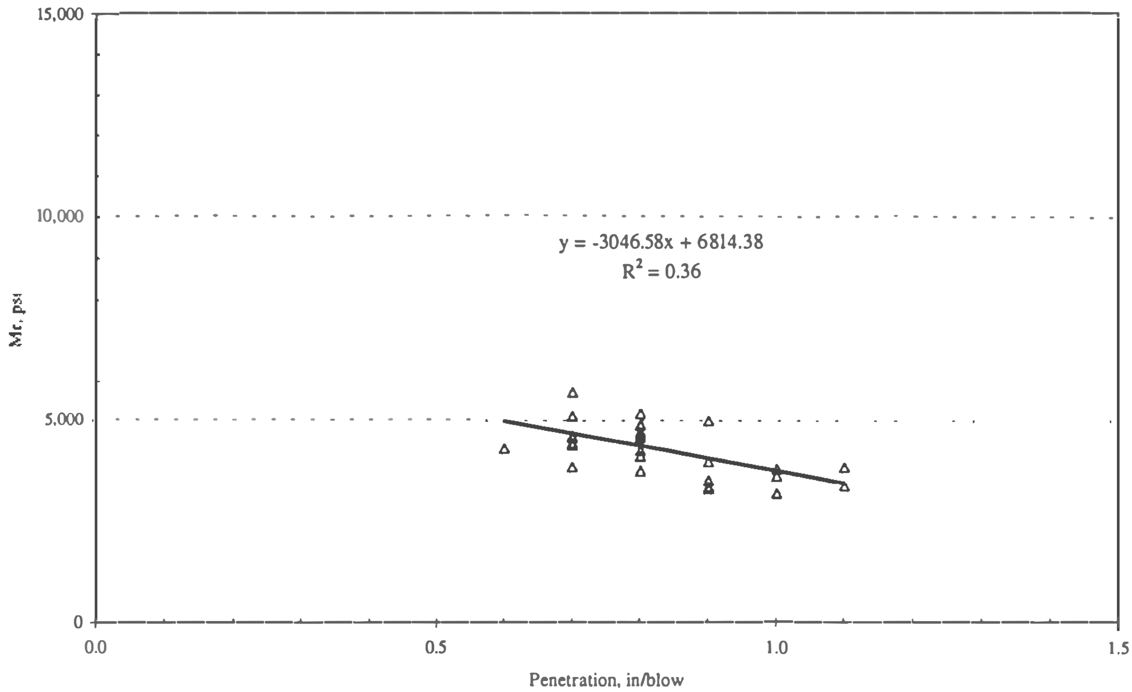


Figure B-10. DCP Penetration Index vs Mr at 36 in sensor—site 5, phase II.

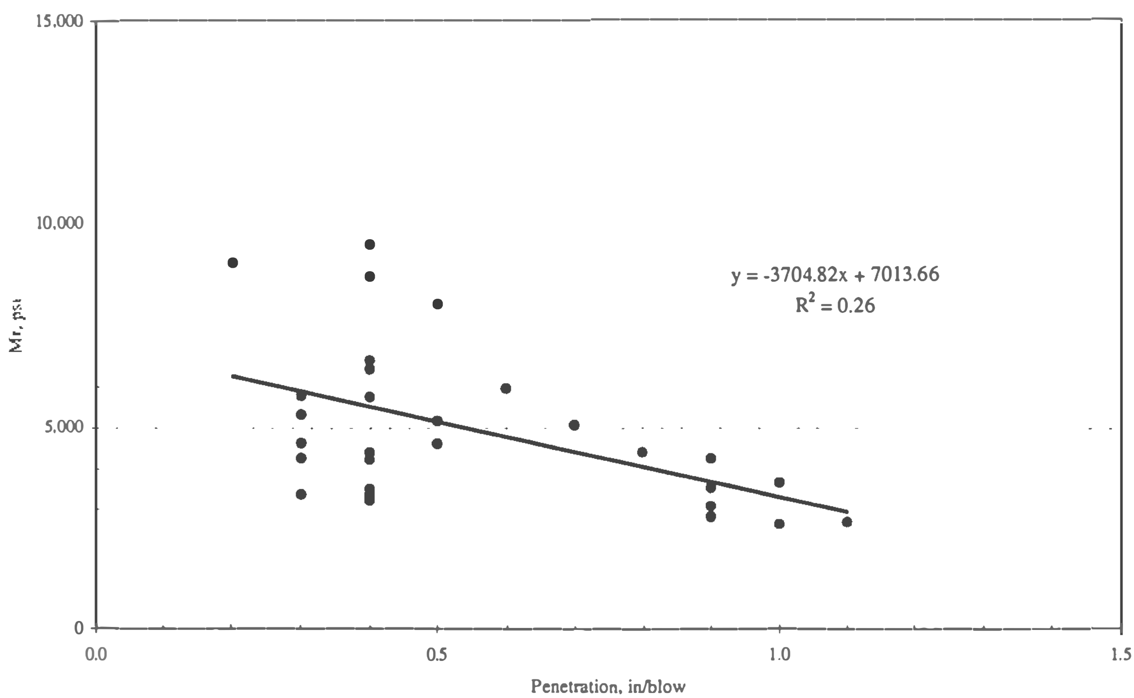


Figure B-11. DCP Penetration Index vs Mr at 36 in sensor—four sites, phase II.

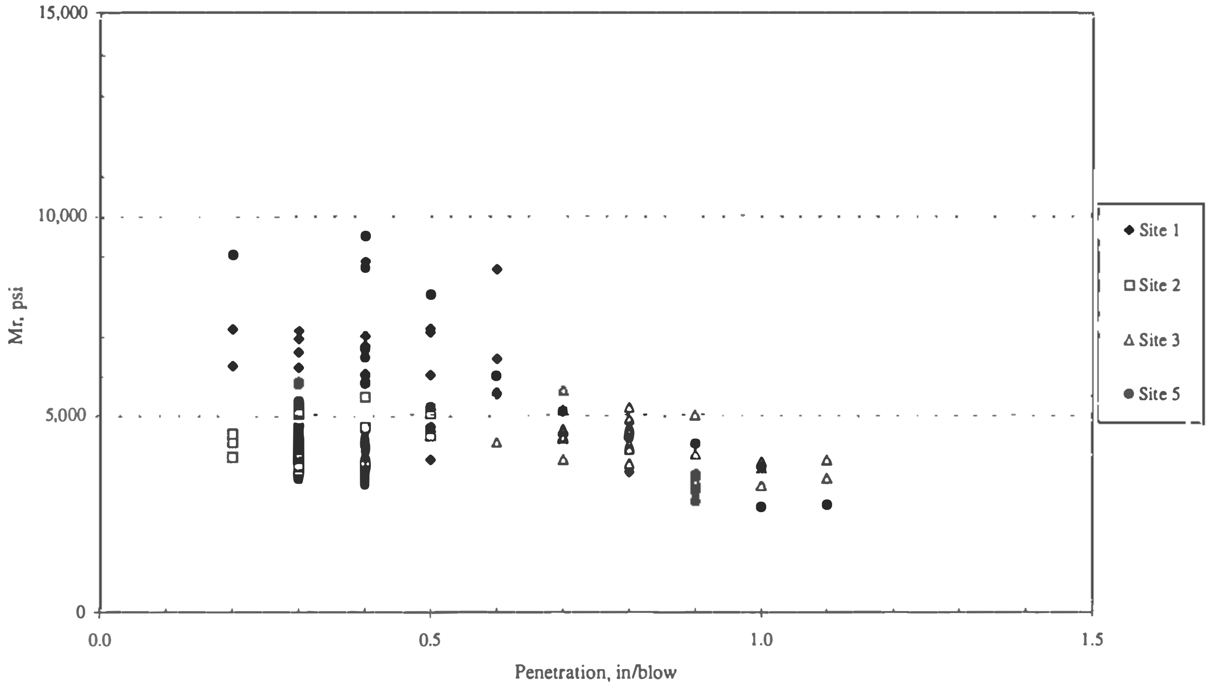
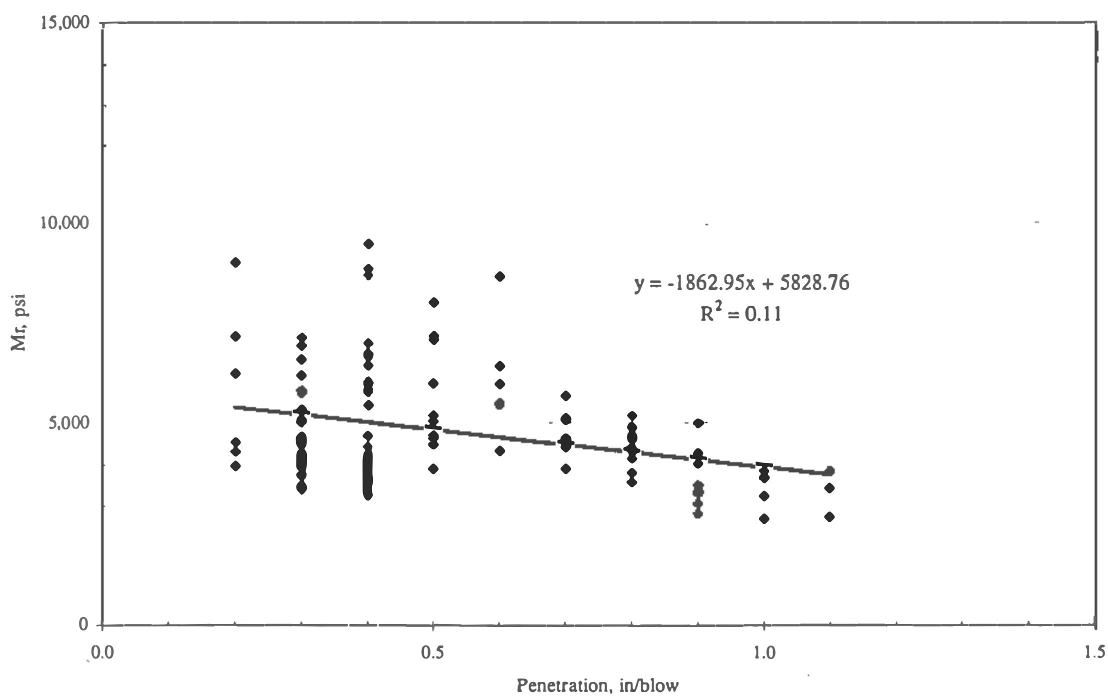


Figure B-12. DCP Penetration Index vs Mr at 36 in sensor—all sites, phase II.



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ERES Project Number 96-147-R1