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# LOAD-DEFLECTION BEHAVIOR OF LIME-STABILIZED LAYERS

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Army Construction Engineering Research Laboratory
Champaign, Illinois

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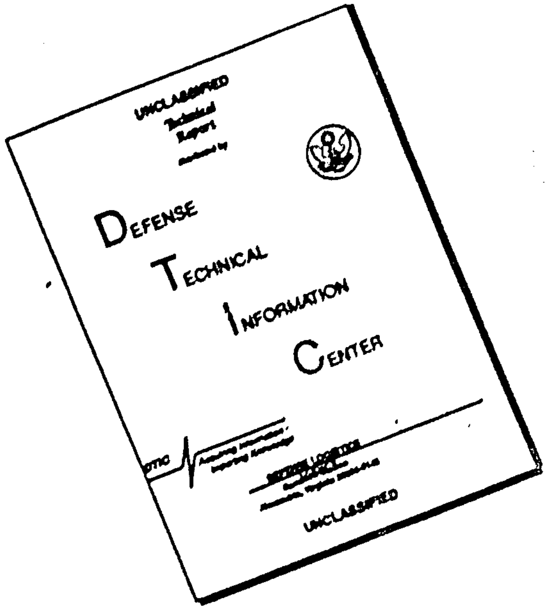
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The static and dynamic load-deflection response of soil-lime pavement layers was investigated. Soil-lime mixture layers 6, 9, and 12 in. thick were compacted over a stiff subgrade (k=450 psi/in.) and a soft subgrade (k=50 psi/in.). The soil-lime layers were statically and dynamically loaded after curing periods of 2, 14, 28, and 56 days. Supplemental subgrade soil and soil-lime mixture property data were also developed.

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The load-deflection data are summarized and analyzed. Various pavement behavior theories are evaluated to determine their capability to predict the load-deflection behavior of soil-lime pavement layers.

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## FOREWORD

This investigation was conducted for the Directorate of Military Construction, Office of the Chief of Engineers (OCE), as part of RDT&E Army Program, Project 4A664717D895 "Military Construction Systems Development;" Task 04, "Military Airfield Facilities;" Work Unit 002, "Load Deflection of Stabilized Layers." The work was performed by the Construction Materials Branch, Materials System and Science Division (MS), Construction Engineering Research Laboratory (CERL), Champaign, IL. The OCE Technical Monitor was S. Gillespie.

Personnel actively engaged in the planning and evaluation stages of this study were E.M. Condiff, H.R. Barrett, G. Schantz, R. Gunkel, J. Gambill, A. Jones, and L.P. Suddath. Dr. M.R. Thompson, Department of Civil Engineering, University of Illinois, Urbana-Champaign Campus, served as project consultant. Appreciation is expressed to Mr. E.A. Lotz, former Chief of MS, for his assistance and contributions to the successful performance of this investigation.

Mr. John J. Healy is Chief of MS. COL M.D. Remus is Commander and Director of CERL and Dr. L.R. Shaffer is Deputy Director.

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# LOAD-DEFLECTION BEHAVIOR OF LIME-STABILIZED LAYERS

## 1 INTRODUCTION

**Background.** Lime-stabilized soils have been used successfully in recent years as paving materials for both military and civilian construction. Extensive laboratory and field studies concerning the properties of soil-lime mixtures and the field performance of pavements containing soil-lime layers have demonstrated that soil-lime layers can be effective in pavement systems. Thompson has considered the general use of lime-treated soils in pavement construction and has advanced some tentative concepts for evaluating the structural behavior of soil-lime pavement layers.[1]

Even though soil-lime mixture properties have been studied in detail[2,3] and soil-lime layers have been extensively used in pavement construction, Rice's statement fairly well summarizes present capabilities:

> Data and analysis are lacking to determine the actual structural benefits imparted to a pavement structure by the incorporation of a stabilized layer in the pavement structure.[4]

It is significant to note that in Vietnam the U.S. Army Engineer troop units and the construction contractors built many miles of Line of Communication (LOC) roads and other construction which contained soil-lime layers. At that time no established procedures were available for adequately considering the structural benefits of soil-lime pavement layers. The U.S. Army's interest in using soil-lime mixtures in pavement construction is evidenced by studies at the Construction Engineering Research Laboratory (CERL) and the Waterways Experiment Station (WES).

Rice's CERL study indicated that lime stabilization appears "to increase the bearing capacity of the subgrade" for rigid pavements and that the soil-lime layers in the flexible pavements were approximately equivalent to high-quality, crushed stone.[5] It is important to note that the CERL investigation was a "model study" and the stabilized layers were quite thin: 3-in. maximum with a minimum of 1/2 in.

The WES Multiple Wheel Heavy Gear Load (MWHGL) study[6] of stabilized layers, done by Grau, included a full scale lime-stabilized section (3-in. asphalt concrete surface, 6-in. crushed stone, and a 15-in. soil-lime subbase). Based on the study, Grau concluded that:

1. Using stabilized structural layers in flexible pavement is highly recommended.
2. The performance of the lime-stabilized subbase material was as good as that of similar pavements constructed of unbound granular base and subbase materials when it was tested in the MWHGL test section at WES, and traffic-tested with a 360-kip, 12-wheel assembly.

The Air Force Weapons Laboratory (AFWL) also sponsored a recent WES study entitled "An Investigation of the Structural Properties of Stabilized Layers in Flexible Pavement Systems."[7] However, Grau's study at WES was the only one that included soil-lime.

Although substantial research has been directed to the problem of evaluating the structural properties of stabilized layers, only a limited effort has been specifically related to soil-lime layers An in-depth study of the structural behavior of soil-lime pavement layers is therefore justified.

[1] M. R. Thompson, "Lime-Treated Soils for Pavement Construction," *Journal of the Highway Division*, ASCE, Vol 94, No. HW2 (1968).

[2] M. R. Thompson, "Engineering Properties of Lime-Soil Mixtures," *Journal of Materials*, Vol 4, No. 4 (American Society for Testing and Materials, 1969).

[3] M. R. Thompson, *Shear Strength and Elastic Properties of Lime-Soil Mixtures*, Record No. 139 (Highway Research Board, 1966).

[4] J. L. Rice, *Stabilization for Pavements*, Technical Report S-11/AD763912 (Construction Engineering Research Laboratory [CERL], 1973).

[5] J. L. Rice, *Stabilization for Pavements*.

[6] R. W. Grau, *Evaluation of Structural Layers in Flexible Pavements*, Miscellaneous Paper S-73-26 (U.S. Army Waterways Experiment Station, 1973).

[7] W. R. Barker, W. N. Brabston, and F. C. Townsend, *An Investigation of the Structural Properties of Stabilized Layers in Flexible Pavement Construction*, Technical Report AFWL-TR-73-21 (Air Force Weapons Laboratory, 1973).



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**Study Objective.** Based on an awareness of the current, limited capabilities for evaluating the structural response of soil-lime pavement layers and the present status of soil-lime mixture technology, a research study was developed with the following general objectives:

1. To study the load-deflection behavior of typical soil-lime pavements
2. To evaluate the effect of pertinent factors such as: mixture strength, subgrade support, layer thickness, and type of loading on the load-deflection response
3. To establish the effectiveness of various pavement-behavior theories in predicting load-deflection behavior for soil-lime layers.

**Approach.** The response data available at present are insufficient for objectively evaluating the behavior of soil-lime pavement layers and the factors which influence that behavior (mixture properties, layer thickness, subgrade support, static vs dynamic loading, etc.). To achieve maximum benefits from this investigation, a simple pavement system (a soil-lime layer over a soil subgrade) was constructed in a laboratory test bin. The advantages of test bin construction were that it could be more accurately controlled than field operations, would minimize important environmental effects, and would permit the use of sophisticated and carefully controlled static and dynamic-loading apparatus and other instrumentation.

The investigation emphasized pavement response, rather than pavement performance under traffic loading. An important initial accomplishment was an understanding of the factors and parameters that influence pavement response; however, subsequent studies should be directed toward relating pavement response to pavement performance under traffic.

**Study Description.** The study was designed to include a range of parameters; the most significant parameters relative to soil-lime pavement load-deflection behavior were: layer thickness, mixture strength, type of loading, and subgrade support. Limitations imposed by loading capabilities, time, and test-bin size were considered in establishing the range of parameters.

*Mixture Strength.* When reactive soils are treated with lime, assuming adequate curing conditions of temperature and moisture prevail, the compacted soil-lime mixture develops increased strength as curing progresses. Thus, it is possible to study mixture-strength effects by varying curing time prior to testing. Curing times of 2, 14, 28, and 56 days were selected for this study.

*Subgrade Support.* To cover a range of typical conditions, both a weak and a stiff subgrade were used. Target values for the moduli of subgrade reaction were 50 psi/in. for the soft grade and 450 psi/in. for the stiff grade. Most soil-lime pavements are constructed in fine-grained subgrade areas; thus, k values in excess of 450 psi/in. are unlikely to be encountered in practice.

*Thickness of Stabilized Layers.* A wide range of constructed thicknesses can be achieved with various types of soil stabilization equipment. Although deep-layer stabilization procedures may be used to process up to 24 in. in one operation,* soil-lime layer thicknesses normally are 6, 9, and 12 in. The thicknesses used in this study were 6, 9, and 12 in.

*Type of Loading.* Many past studies have demonstrated that loading rates substantially affect pavement response. The availability of an MTS closed-loop testing system allowed application of both static and dynamic loads to the test section pavements. A 12-in. diameter, plate-loading device was used in the test program. (Note: For conditions of a 9,000 lb wheel load and an 80 psi contact pressure over a circular contact area, the diameter of the loaded area is 12 in.)

Table 1 summarizes the sections included in the laboratory testing program.

*M. R. Thompson, "Deep Plow Lime Stabilization for Pavement Construction," *Transportation Engineering Journal*, ASCE, Vol 98, No. TE2 (1972).

Table 1
Test Section Data

| Section Number | Soil-Lime Thickness, in. | Subgrade Support (k), psi/in. |
|---|---|---|
| 1 | 6 | 50 |
| 2 | 6 | 450 |
| 3 | 9 | 50 |
| 4 | 9 | 450 |
| 5 | 12 | 50 |
| 6 | 12 | 450 |

## 2 MATERIALS

**Soil.** Goose Lake Clay (GLC), a commercial clay distributed by the A.P. Green Refractories Co. of Morris, IL, was used in the test bin as the subgrade soil and also for soil-lime mixture preparation. Table 2 summarizes pertinent engineering properties of GLC. The moisture-density relation CE55[9] for GLC is shown in Figure 1. The principal clay mineral in GLC is kaolinite with some illite and quartz. GLC was selected because it is processed and uniform in properties, it will react with hydrated lime to achieve substantial strength increase, and it is similar to many soils that would be considered for lime stabilization.

**Lime.** A monohydrated, dolomitic lime, produced by the Marblehead Lime Co. of Thornton, IL, was used in the preparation of the soil-lime mixtures. The lime meets the specification requirements of ASTM C-207, Type N. Approximately 85 percent of the lime will pass the No. 325 sieve.

**Soil-Lime Mixtures.** Preliminary studies indicated that a 4 percent (based on dry weight of soil) lime treatment produced optimum compressive strength response for curing periods (at 73°F) up to 56 days—the maximum curing period planned for the test series. The compressive and flexural strength and stiffness properties of the soil-lime mixture (cured for 2, 7, 14, 28, and 56 days) were tested under both static and dynamic loadings.

[9] *Materials Testing*, Technical Manual 5-530 (Department of the Army, February 1966).

Table 2
Properties of Goose Lake Clay

Atterberg Limits
Liquid Limit, % - 30
Plastic Limit, % - 16
Plasticity Index, % - 14

Specific Gravity - 2.71

Moisture-Density Characteristics
$\gamma_D$ max., pcf - 124.8
Optimum moisture content, % - 11.0

Grain Size Distribution
% passing No. 200 sieve - 87
% $<2\mu$ - 26

Classification
Unified - CL
AASHO - A-6 (11)

The CE moisture-density relations for 4 percent lime-treated GLC are shown in Figure 1. The maximum dry density is 122 pcf and the optimum moisture content is 12.2 percent.

**Compressive Strength and Stiffness.** A series of Harvard miniature-sized specimens (1.3125 in. diameter x 2.816 in. length) were prepared with a drop-hammer compactor. The 4 percent soil-lime mixture was prepared at an optimum moisture content of approximately 12.2 percent and compacted to approximately 93-95 percent of maximum dry density.

For each series, seven specimens were cured in sealed containers at 73°F for 2, 7, 14, 28, and 56 days.

*Static Testing.* Following the designated curing period, four of the specimens were loaded to failure at a constant loading rate of 100 lb/min. Axial and radial deformations were measured continuously during loading. The static modulus of elasticity was calculated as the secant modulus at a stress level of 72 percent of the ultimate strength as recommended by Thompson.[10] Poisson's ratio values were calculated at various stress levels by dividing the diametral strain by the axial strain.

[10] M. R. Thompson, *Shear Strength and Elastic Properties of Lime-Soil Mixtures*, Record No. 139 (Highway Research Board, 1966).

Static compressive stress-strain curves and relations between stress level and Poisson's ratio are shown in Figures 2 and 3, respectively. Pertinent data are summarized in Table 3.

*Dynamic Testing.* Dynamic compressive testing was limited to determining the resilient modulus and resilient Poisson's ratio of three specimens subjected to various repeated stress levels. In general, the repeated stress levels were approximately 25, 50, and 75 percent of the static unconfined, compressive strength of the soil-lime mixture. One specimen was tested at each of the three stress levels. The repeated stress sequence was 500 applications of a 40 msec triangular stress pulse applied at a frequency of 20 cycles/min. Axial and radial deformations were monitored, throughout the repeated loading period.

Resilient moduli (repeated axial stress/recoverable axial strain) and resilient Poisson's ratio values (recoverable radial strain/recoverable axial strain) were calculated from the test data. The number of load applications (up to the 500 cycles applied) had no effect on the resilient moduli or resilient Poisson's ratio values; thus, the dynamic-response data summarized in Table 4 are for 250-load applications.

**Flexural Strength and Flexural Moduli.** Flexural-strength properties of the cured soil-lime mixtures were evaluated using 2 x 2 x 7 in. beams subjected to third point (2 in. - 2 in. - 2 in.) static and dynamic loading. The soil-lime beams were compacted in three equal layers with a full-face drop-hammer compactor. The mixtures were compacted at an approximately optimum moisture content of 12.2 percent to a dry density of approximately 112 pcf (92 percent of CE55).

The seven beams prepared for each series were cured at 73°F in sealed containers for periods of 2, 7, 14, 28, and 56 days. At the end of the curing period, SR-4 strain gages were cemented to the top and bottom of the specimens in the middle-third portion.

Four of the beams were tested under static loading conditions at a constant loading rato of 25 lb/min. The strain gages were monitored continuously during loading. Load and strain data were used to develop moment-curvature relations from which flexural moduli were calculated The static loading data are summarized in Table 5: modulus of rupture values, flexural moduli calculated for a stress level equal to 50 percent of the modulus of rupture, and tensile strains at failure.

Table 3

**Compressive Strength and Modulus of Elasticity Properties of Cured Soil-Lime Mixtures**

| Curing Period, days (a) | Compressive Strength, psi (b) | Secant Modulus, of Elasticity, psi (c) |
|---|---|---|
| 2 | 125 | 16,100 |
| 7 | 160 | 25,000 |
| 14 | 189 | 27,200 |
| 28 | 244 | 32,600 |
| 56 | 262 | 45,000 |

**Notes:**
(a) Curing temperature of approximately 73°F.
(b) Average of four specimens.
(c) Calculated at a stress level of approximately 72% of ultimate strength.

Table 4

Compressive Resilient Moduli and Poisson's Ratio
Properties of Cured Soil-Lime Mixtures

| Curing Period, days (a) | Stress Level psi | Stress Level % of Ultimate Strength | Resilient Modulus, ksi | Resilient Poisson's Ratio |
|---|---|---|---|---|
| 2 | 36 | 27 | 99 | 0.22 |
| | 67 | 51 | 125 | 0.71 |
| | 99 | 76 | 100 | 1.08 |
| 7 | 41 | 26 | 83 | 0.16 |
| | 81 | 51 | 71 | 0.20 |
| | 117 | 73 | 55 | 0.36 |
| 14 | 47 | 25 | 109 | 0.19 |
| | 93 | 49 | 109 | 0.27 |
| | 139 | 74 | 85 | 0.33 |
| 28 | 65 | 27 | 92 | 0.11 |
| | 130 | 54 | 92 | 0.21 |
| | 178 | 73 | 69 | 0.11 |
| 56 | 65 | 25 | 114 | 0.14 |
| | 148 | 55 | 117 | 0.24 |
| | 204 | 78 | 106 | 0.40 |

Notes:
(a) Curing temperature of approximately 73°F.

Table 5

Static Flexural Strength, Flexural Moduli, and
Tensile Failure Strain Data for Cured Soil-Lime Mixtures

| Curing Period, days (a) | Modulus of Rupture, psi (b) | Flexural Modulus, ksi (c) | Tensile $\epsilon$ at Failure, Microstrain |
|---|---|---|---|
| 2 | 32 | 71.1 | 644 |
| 7 | 34 | 124.1 | 493 |
| 14 | 42 | 133.3 | 415 |
| 28 | 54 | 198.6 | 552 |
| 56 | 55 | 216.0 | 420 |

Notes:
(a) Curing temperature of approximately 73°F.
(b) Average of four specimens.
(c) Modulus calculated at a stress level of approximately 50 percent of the modulus of rupture.

Dynamic testing was used to evaluate a dynamic flexural modulus. The repeated flexural-stress levels were approximately 25, 50, and 75 percent of the static modulus of rupture. One specimen was tested at each of the three stress levels under a repeated loading sequence of 500 applications of a 40 msec, triangular pulse applied at a frequency of 20 cycles/min. The SR-4 strain gages were monitored continuously during the test. Dynamic flexural moduli were calculated using the same procedures employed for the static test data. Since the number of cycles did not affect the dynamic flexural response (up to 500 stress applications considered), the dynamic flexural moduli reported in Table 6 are for 100 stress applications.

**Comments.** The reduction in maximum dry density and the increase in optimum moisture content for the soil-lime mixture effected by the lime treatment are typical (see Figure 1). Strength and modulus of elasticity values normally increase as curing times are lengthened (Tables 3, 4, 5, and 6 and Figure 2). The Poisson's ratio data in Figure 3 and the flexural failure-strain in Table 5 compare favorably with data developed at the University of Illinois for other soil-lime mixtures.[11]

Thompson has proposed that soil-lime mixtures tested in compression exhibit a "limiting failure strain" type behavior.[12] The flexural strain data in Table 5 also support a "limiting failure strain" approach. Limited flexural testing data developed in an early University of Illinois study[13] also appeared to indicate the development of maximum flexural resistance in a failure-strain range which is 400-500 microstrain, similar to the data in Table 5.

In summary, the material properties displayed by the cured lime-GLC mixtures agreed with previously developed data for other soil-lime mixtures. Therefore, soil-lime layers constructed using the lime-GLC mixture should exhibit typical load-deflection responses and also no difficulty should be encountered in extrapolating the behavior of the materials and pavements in this study to different subgrade soils and soil-lime mixtures.

[11] M. R. Thompson, "Engineering Properties of Lime-Soil Mixtures," *Journal of Materials*, Vol 4, No. 4 (American Society for Testing and Materials, 1969).

[12] M. R. Thompson, *Shear Strength and Elastic Properties of Lime-Soil Mixtures*, Record No. 139 (Highway Research Board, 1966).

[13] M. R. Thompson, "Engineering Properties of Lime-Soil Mixtures."

## 3 CONSTRUCTION AND TESTING PROCEDURES

**Test Section Construction.** All of the test items were constructed in an 8 x 8 ft reinforced concrete test bin, with wall thicknesses of 8 in., and a floor thickness of 12 in. The bin was deep enough to accommodate 42 in. of subgrade soil and soil-lime layers of thicknesses up to 12 in. Elastic layer and Westergaard-based calculations using typical properties for the subgrade and soil-lime mixture indicated that the bin dimensions were large enough to permit the development of slab action and deep enough to insure that the soil-lime layer response was controlled primarily by the subgrade soil and was not unduly affected by the concrete slab in the bin bottom.

*Material Preparation.* The subgrade soils and soil-lime mixtures were prepared in a turbine concrete mixer in approximately 800 lb batches. A predetermined amount of water was added slowly to the GLC and mixed until a homogeneous soil-water system was attained, generally about 5 min. The soil-lime mixture was prepared by dry-mixing the GLC and the appropriate amount of lime (4 percent lime content based on dry weight of soil), and then thoroughly incorporating enough moisture to achieve the desired moisture content. The soil-lime-water mixture was usually "wet mixed" for approximately 3 min.

Sufficient quantities of the GLC or the soil-lime mixture were prepared, covered to prevent moisture loss, and stockpiled immediately prior to placing the subgrade or soil-lime layer. Moisture was checked before placement to insure that the correct moisture content had been achieved.

*Material Placement and Compaction.* The GLC or soil-lime mixture was placed in the bin

Table 6

Dynamic Flexural Moduli of Cured Soil-Lime Mixtures

| Curing Period, days (a) | Flexural Stress Level psi | Flexural Stress Level % of Modulus of Rupture | Dynamic Flexural, Modulus, ksi |
|---|---|---|---|
| 2 | 7.5 | 23 | 178 |
| | 15.0 | 47 | 128 |
| | 24.0 | 75 | 133 |
| 7 | 7.8 | 23 | 174 |
| | 15.7 | 46 | 210 |
| | 25.6 | 75 | 197 |
| 14 | 12 | 29 | 218 |
| | 24 | 57 | 200(b) |
| | 36 | 86 | 165 |
| 28 | 15 | 28 | 273 |
| | 43 | 80 | 226(b) |
| 56 | 13.5 | 25 | 300 |
| | 30.0 | 55 | 250 |
| | 44.0 | 80 | 288 |

Notes:
(a) Curing temperature of approximately 73°F.
(b) Value used in finite-element analysis.

in approximately 6-in. lifts (loose thickness). The desired compaction was accomplished with an air-driven tamper (5-in. diameter tamping face). Compacted lift thicknesses varied from 2-1/4 to 3 in. Sufficient lifts were placed to achieve the desired thickness of compacted material.

The top surfaces of the subgrade and the soil-lime layer were leveled with a soil planer to proper elevations and thicknesses. An approximately 1/16-in. layer of paraffin was placed on the surface of the finished soil-lime layer to prevent subsequent moisture loss.

*Control Tests.* Moisture content, density, and *in-situ* California Bearing Ratio (CBR) tests were conducted during the placement of the subgrade material to insure that the compacted soil was properly and uniformly constructed. Soil-lime mixture control tests were for moisture content and density. Harvard miniature sized compression specimens were prepared from the soil-lime mixture and cured in sealed containers at 73°F for periods up to 56 days.

Table 7 summarizes the as-constructed subgrade properties of the test sections. Periodic checks following the completion of load-testing indicated that subgrade moisture and density properties had not significantly changed from as-constructed conditions.

The compressive strengths of the soil-lime mixtures cured for 28 and 56 days indicated that the mixtures placed in the test section were comparable to the mixtures used in the laboratory studies. The flexural strengths of beam specimens (2 x 2 x 7 in.) dry-sawed from some of the cured soil-lime slabs after the completion of load-testing also confirmed the fact that the soil-lime mixtures in the slabs were very similar to those previously evaluated in the laboratory studies. Based on these facts, it is assumed that the property data presented in Tables 3, 4, 5, and 6 are representative of the as-constructed and cured soil-lime layers.

**Testing Procedures.** The primary testing sequence for the soil-lime pavement sections included static and dynamic 12-in. diameter plate-loading following curing periods of 2, 14, 28, and 56 days. The entire testing sequence was accomplished on the same test section since the soil-lime pavement was not failed until the ultimate load test was conducted at the 56-day curing period.

*Loading Equipment.* An MTS closed-loop testing system with a 50k load-actuator was used for both static and dynamic testing. The 12-in. diameter loading device was a series of stacked aluminum plates (6, 9, and 12-in. diameters). Load was applied to the plate through a ball-type mechanism and was measured by using electronic load-cells of various appropriate ranges.

*Deflection Measurements.* Outputs from four linear potentiometers, spaced equally around the perimeter of the 12-in. loading plate, were electronically averaged and recorded as the plate deflection. Additional potentiometers measured pavement surface deflections at radial distances up to 42 in. away from the center of the loading plate.

During construction, a small plastic disc was placed on the surface of the subgrade immediately beneath the anticipated location of the center of the loading plate. After construction of the soil-lime layer, a small hole was drilled through the layer and a plastic push-rod was attached to the disc. The push-rod was extended through the load plate. Its movement, sensed by a potentiometer, was measured as the subgrade surface deflection.

*Typical Pavement Loading Operations.* After the appropriate curing time the soil-lime pavement sections were tested under static and dynamic loading conditions. In all cases, the loads were applied to the 12-in. diameter loading plate previously described. The plate was appropriately leveled and seated prior to the load test. The pavements were not tested to failure in order to preserve the sections for additional testing following an extension of the curing period.

The static loads were increased until the desired load was reached. The plate deflection was monitored after each load increment, and the next increment was not applied until the rate-of-accumulation of additional plate deformation was less than 0.002 in. in 10 min. Since the maximum static loads were substantially less than the ultimate load-carrying capacity of the pavement section, the plate deformation

**Table 7**

**Average Properties of Test Section Subgrades**

| | Soft Subgrade Sections (a) | Stiff Subgrade Sections (b) |
|---|---|---|
| A. Compacted dry density, pcf | 110 | 115 |
| B. Placement water content, % | 17 | 12.5 |
| C. Modulus of subgrade reaction psi/in.* | 50 | 450 |
| D. In-situ CBR* | 3 | 22 |

**Notes:**
(a) Sections 1, 3, and 5
(b) Sections 2, 4, and 6
* CE55 procedure

**Materials Testing*, Technical Manual 5-530 (Department of the Army, February 1966).

rate rapidly decreased soon after the load increment was applied. Plate load and deformation, pavement surface, and subgrade surface deflection data were recorded for the various load increments. Figures 4 to 9 present load-deflection (P-Δ) relations for all the sections.

After the static loading operations, dynamic loads of varying magnitude—but substantially less than the ultimate load-carrying capacity—were applied. The triangular load pulse was 40 msec in duration and the loading frequency was 20 cycles/min. Five hundred load repetitions were applied at each load magnitude and dynamic plate and subgrade surface deflections were recorded (Tables 8 to 13).

After the static-and dynamic-load testing had been completed for the 56-day curing period, the pavement sections were loaded to failure at the rate of 3000 lb/min. Plate and subgrade surface deflections were recorded during the test (Figures 10 to 15).

**Table 8**

**Dynamic Loading Data for Section 1**

| Cure Time, days | Plate Load, lb | Plate Pressure, psi | Plate Deflection at 100 cycles, 0.001 in. | Subgrade Surface Deflection at 100 cycles, 0.001 in. |
|---|---|---|---|---|
| 2 | 600 | 5.3 | 1 | 1 |
| | 1,500 | 13.3 | 3 | 2 |
| | 2,300 | 20.3 | 13 | 8 |
| 14 | 1,200 | 10.6 | 3 | 2 |
| | 2,600 | 23.0 | 9 | 6 |
| | 3,850 | 34.1 | 16 | 13 |
| 28 | 1,360 | 12.0 | 4 | 2 |
| | 2,850 | 25.2 | 10 | 7 |
| | 4,200 | 37.2 | 17 | 14 |
| 56 | 1,600 | 14.2 | 4 | 2 |
| | 3,400 | 30.0 | 10 | 8 |
| | 5,200 | 46.0 | 20 | 18 |

## Table 9

### Dynamic Loading Data for Section 2

| Cure Time, days | Plate Load, lb | Plate Pressure, psi | Plate Deflection at 100 cycles, 0.001 in. | Subgrade Surface Deflection at 100 cycles, 0.001 in. |
|---|---|---|---|---|
| 2 | 1,000 | 8.8 | 2 | 2 |
| | 2,100 | 18.6 | 3 | 2 |
| | 3,000 | 26.5 | 4 | 2 |
| 14 | 1,600 | 14.2 | 4 | -- |
| | 3,300 | 29.2 | 6 | 2 |
| | 4,600 | 40.7 | 8 | 2 |
| 28 | 2,000 | 17.7 | 6 | 2 |
| | 4,000 | 35.4 | 8 | 6 |
| | 5,800 | 51.3 | 10 | 7 |

## Table 10

### Dynamic Loading Data for Section 3

| Cure Time, days | Plate Load, lb | Plate Pressure, psi | Plate Deflection at 100 cycles, 0.001 in. | Subgrade Surface Deflection at 100 cycles, 0.001 in. |
|---|---|---|---|---|
| 2 | 1,800 | 15.9 | 10 | 5 |
| 14 | 1,000 | 8.8 | 8 | 5 |
| | 2,000 | 17.7 | 20 | 16 |
| | 2,800 | 24.8 | 32 | 27 |
| 28 | 5,250 | 46.4 | 6 | 5 |
| | 12,000 | 106.1 | 9 | 17 |
| | 17,500 | 154.7 | 37 | 30 |
| 56 | 3,250 | 28.7 | 9 | 3 |
| | 6,500 | 57.5 | 22 | 16 |
| | 10,000 | 88.4 | 37 | 29 |

## Table 11

### Dynamic Loading Data for Section 4

| Cure Time, days | Plate Load, lb | Plate Pressure, psi | Plate Deflection at 100 cycles, 0.001 in. | Subgrade Surface Deflection at 100 cycles, 0.001 in. |
|---|---|---|---|---|
| 2 | 1,900 | 16.8 | 3 | 1 |
| | [illegible],900 | 34.5 | 8 | 2 |
| | 5,900 | 52.2 | 12 | 5 |
| 14 | 3,750 | 33.2 | 6 | 2 |
| | 7,750 | 68.5 | 13 | 7 |
| | 11,500 | 101.7 | 23 | 13 |
| 28 | 4,750 | 42.0 | 7 | 2 |
| | 9,500 | 84.0 | 16 | 7 |
| | 13,500 | 119.4 | 25 | 14 |
| 56 | 5,500 | 48.6 | 7 | 3 |
| | 10,750 | 95.1 | 14 | 7 |
| | 14,250 | 126.0 | 19 | 15 |

## Table 12

### Dynamic Loading Data for Section 5

| Cure Time, days | Plate Load, lb | Plate Pressure, psi | Plate Deflection at 100 cycles, 0.001 in. | Subgrade Surface Deflection at 100 cycles, 0.001 in. |
|---|---|---|---|---|
| 2 | 2,200 | 19.5 | 4 | 2 |
| | 4,600 | 40.7 | 10 | 4 |
| | 7,000 | 61.9 | 17 | 8 |
| 14 | 3,500 | 31.0 | 5 | 2 |
| | 7,250 | 64.1 | 13 | 7 |
| | 10,000 | 88.4 | 21 | 12 |
| 28 | 4,375 | 38.7 | — | 4 |
| | 8,750 | 77.4 | — | 9 |
| | 12,500 | 110.5 | — | 16 |
| 56 | 5,000 | 44.2 | 9 | 3 |
| | 9,375 | 82.9 | 16 | 6 |
| | 13,750 | 121.6 | 25 | 14 |

## 4 STATIC LOADING—DATA ANALYSIS

**Static Loading.** In general, the various load-deflection (P-Δ) relations (Figures 4 to 9) illustrate the interactions among the various parameters of subgrade strength, thickness of soil-lime layers, and soil-lime mixtures strength (curing-period effect), and also prove that substantial load-carrying capacity can be developed in a soil-lime pavement system.

*Subgrade Strength Effects.* The influence of subgrade strength (as evaluated by the modulus of subgrade reaction) on load-carrying capacity is illustrated in Figures 16 and 17. The data are for 28-day curing periods and plate deflections of 0.02 in. and 0.06 in., respectively. Six-in. thickness data are not shown in Figure 17 since deflections of 0.06 in. were not obtained during the static plate-load testing.

The beneficial effect of increased subgrade support is obvious for all soil-lime layer thicknesses. Nussbaum and Larsen of the Portland Cement Association (PCA)[14] found similar "straight-line" relations between load-carrying capacity at a specified deflection and subgrade strength. It should be noted that in the PCA study several levels of subgrade strength were considered, in contrast to the two levels (50 psi/in. and 450 psi/in.) included in this study.

*Strength and Curing Effects.* Soil-lime mixture strength development in the test pavement was related to curing time as illustrated in Figure 2. Thus, increased test section curing time effected increased strengths in the soil-lime layer, while for practical purposes, the subgrade strength remained essentially unchanged.

It is important to note that both strength and moduli of elasticity properties of the soil-lime mixture increase with curing time (see Table 5). Therefore, the P-Δ response of the test sections should display a definite curing time effect. With the exception of section 2 (6-in. soil-lime, stiff subgrade), load-carrying capacity increased with extended curing. In section 2, the combined effects of stiff subgrade and thin layer thickness obscured the curing effects. The most dramatic increases in load-carrying capacity were obtained in the curing interval from 2 to 14 days, while somewhat reduced effects were noted for the longer curing periods. This is not surprising since modular ratio effects (E soil-lime/E subgrade) are more significant at lower values; i.e., the initial increases in strength and modular ratio (2 to 14 days in this study) were more beneficial than subsequent increases.

*Thickness Effects.* The influence of thickness on the 28-day curing P-Δ responses of the various sections is illustrated in Figures 18 and 19 for the soft and stiff subgrades, respectively. Similar trends were noted for the other curing periods. Thickness effects appear to be significant for both soft and stiff subgrade support conditions.

**Theoretical Analyses.** The procedures developed for predicting the load-deflection behavior of layered systems include elastic theory (Westergaard's dense liquid subgrade) and elastic-plate theory (elastic solid subgrade). For an extensive descriptive summary of the various procedures, consult Appendix G of the Allerton conference proceedings.[15] In all the above-listed procedures the materials are assumed to be linearly elastic, and the elastic properties are not stress-dependent. The various procedures have been used extensively with varying degrees of success in pavement analysis and design.

Selected P-Δ data from this study were analyzed using the various procedures. Table 13 summarizes soil-lime and subgrade soil-property data used in the analyses. Theoretical and experimental values were compared for the 28-day curing and a plate load of 9k, except for sections 1 and 2 (6 in. soil-lime layer thickness) where the comparisons were based on 5k and 6k loads, respectively. In some cases additional curing periods or plate loadings were also considered. The 28-day curing period was selected because the 28 and 56-day data were

[14] P. J. Nussbaum and T. J. Larsen, *Load-Deflection Characteristics of Soil-Cement Pavements*, Record No. 86 (Highway Research Board, 1965).

[15] R. W. Woodhead and R. H. Wortman, *Proceedings, Allerton Park Conference on Systems Approach to Airfield Pavements*, 23-26 March 1970, Technical Report P-5 AD 763212 (CERL, 1973), Appendix G.

quite similar and, at the end of the 28 days of curing, the soil-lime layer had developed sufficient flexural strength (54 psi) and stiffness (E = 199 ksi) to exhibit a slab- or "plate"-type structural behavior.

*Layered Elastic Analysis.* An n-layered elastic system computer program initially developed by Chevron and subsequently modified at the University of Illinois was used in the layered elastic analysis. A "rough interface" condition was assumed in the analysis and the material properties shown in Table 13 were used. Pertinent theoretical response data (surface deflection, flexural stress at the bottom of the soil-lime layer) and experimental data for the various sections are summarized in Table 14.

**Table 13**

**Material Property Data Utilized in Static Analyses**

**Subgrade Soil**

K, Modulus of Subgrade Reaction, psi/in.

50 (Soft Grade)

450 (Stiff Grade)

E, Soft Grade 880 psi (a)

1700 psi (b)

Stiff Grade 7960 (a)

(a) E calculated from 30 in. plate load test data on subgrade (Boussinesq theory).

(b) Determined from compressive stress-strain curves for compacted (kneading) soil samples (loading rate of 0.05 in./min). Samples were compacted to the average moisture-density conditions shown in Table 2.

**Soil-Lime Mixture**

Modulus of Elasticity, psi

199,000

(28-day curing, static testing, see Table 5)

A difficulty encountered in using the elastic-layer program to analyze the data was the adequate representation of the test section. The elastic-layer analysis assumes the bottom layer to be of semi-infinite extent, whereas in the test section the subgrade was 42 in. thick, the bottom of the test bin was 12 in. of concrete, and the bin was positioned on the concrete floor slab of the laboratory. Preliminary Bison strain-gage data obtained during the early stages of load-testing indicated that as little as 0.001 to 0.002 in. of deformation was being experienced in the bottom 6-in. layer of the subgrade, whereas the layered elastic theory data implied that detectable deformations should be occurring in the layer. It is apparent that the theory does not precisely represent the test section system.

*Elastic Plate Theory.* The dense liquid subgrade (Westergaard) and the elastic solid subgrade theoretical models were used to predict the behavior of the soil-lime test pavements. Moduli of subgrade reaction determined from 30-in. plate-loading tests were used for inputs in the Westergaard model. The moduli of elasticity (E) were calculated by using the plate-loading data for the subgrade and, in addition, compression specimens (2-in. diameter by 4 in.) were prepared by kneading compaction at the moisture and density levels indicated in Table 7. Static (deformation rate of 0.05 in./min) compressive stress-strain data were used to determine an E for use in the elastic subgrade model. Although the two determinations of E were quite different for the soft subgrade, they were approximately the same for the stiff subgrade condition.

Calculations were made using k values of 50 and 450 psi/in. for the Westergaard model and E values of 880 and 1700 psi (soft grade) and 8000 psi (stiff grade) for the elastic solid subgrade model (Table 14).

PCA's analysis[16] of their load-deflection data for cement-stabilized materials indicated that the relation

$$P = \frac{wk}{\alpha\left(\frac{a}{h}\right)^{\beta}}$$

in which

| | | |
|---|---|---|
| P | = | plate pressure, psi |
| w | = | plate deflection, in. |
| k | = | modulus of subgrade reaction, psi/in. |
| a | = | radius of plate |
| h | = | thickness of stabilized layer |
| $\alpha, \beta$ | = | experimentally determined factors |

[16] P. J. Nussbaum and T. J. Larsen, *Load Deflection Characteristics of Soil-Cement Pavements*, Record No. 86 (Highway Research Board, 1965).

## Table 14

**Theoretical Results and Measured Data**
**(Static Analysis—28-Day Curing Period)**

| Section No. | Measured Surface Δ, in. (a) | Layered Elastic Theory Δ, in. (b) | σ, psi (c) | $E_{sub.}$, psi |
|---|---|---|---|---|
| 1 | 0.050 | 0.087 | 142 | 880 |
| | | 0.057 | 123 | 1700 |
| 2 | 0.028 | 0.024 | 93 | 8000 |
| 3 | 0.075 | 0.107 | 136 | 880 |
| | | 0.070 | 122 | 1700 |
| 4 | 0.023 | 0.026 | 84 | 8000 |
| 5 | 0.035 | 0.086 | 82 | 880 |
| | | 0.054 | 75 | 1700 |
| 6 | 0.022 | 0.021 | 54 | 8000 |

| Section No. | Measured Surface Δ, in. (a) | Elastic Plate Theory: Dense Liquid Subgrade Δ, in. (b) | σ, psi (c) | k, psi/in. | Elastic Solid Subgrade Δ, in. (b) | σ, psi (c) | $E_{sub.}$, psi |
|---|---|---|---|---|---|---|---|
| 1 | 0.050 | 0.043 | 131 | 50 | 0.086 | 140 | 880 |
| | | | | | 0.055 | 123 | 1700 |
| 2 | 0.028 | 0.016 | 107 | 450 | 0.022 | 103 | 8000 |
| 3 | 0.075 | 0.043 | 124 | 50 | 0.104 | 137 | 880 |
| | | | | | 0.067 | 123 | 1700 |
| 4 | 0.023 | 0.014 | 90 | 450 | 0.023 | 92 | 8000 |
| 5 | 0.035 | 0.028 | 77 | 50 | 0.079 | 88 | 880 |
| | | | | | 0.050 | 80 | 1700 |
| 6 | 0.022 | 0.009 | 58 | 450 | 0.018 | 62 | 8000 |

Notes:
(a) 9k plate load except section 1 (5k) and section 2 (6k).
(b) Surface deflection at centerline of loaded area.
(c) Flexural stress at centerline of loaded area at bottom of soil-lime layer.

quite similar and, at the end of the 28 days of curing, the soil-lime layer had developed sufficient flexural strength (54 psi) and stiffness ($E = 199$ ksi) to exhibit a slab- or "plate"-type structural behavior.

*Layered Elastic Analysis.* An n-layered elastic system computer program initially developed by Chevron and subsequently modified at the University of Illinois was used in the layered elastic analysis. A "rough interface" condition was assumed in the analysis and the material properties shown in Table 13 were used. Pertinent theoretical response data (surface deflection, flexural stress at the bottom of the soil-lime layer) and experimental data for the various sections are summarized in Table 14.

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K, Modulus of Subgrade Reaction, psi/in.
50 (Soft Grade)
450 (Stiff Grade)

E, Soft Grade 880 psi (a)
1700 psi (b)
Stiff Grade 7960 (a)

(a) E calculated from 30 in. plate load test data on subgrade (Boussinesq theory).

(b) Determined from compressive stress-strain curves for compacted (kneading) soil samples (loading rate of 0.05 in./min). Samples were compacted to the average moisture-density conditions shown in Table 2.

**Soil-Lime Mixture**

Modulus of Elasticity, psi
199,000
(28-day curing, static testing, see Table 5)

A difficulty encountered in using the elastic-layer program to analyze the data was the adequate representation of the test section. The elastic-layer analysis assumes the bottom layer to be of semi-infinite extent, whereas in the test section the subgrade was 42 in. thick, the bottom of the test bin was 12 in. of concrete, and the bin was positioned on the concrete floor slab of the laboratory. Preliminary Bison strain-gage data obtained during the early stages of load-testing indicated that as little as 0.001 to 0.002 in. of deformation was being experienced in the bottom 6-in. layer of the subgrade, whereas the layered elastic theory data implied that detectable deformations should be occurring in the layer. It is apparent that the theory does not precisely represent the test section system.

*Elastic Plate Theory.* The dense liquid subgrade (Westergaard) and the elastic solid subgrade theoretical models were used to predict the behavior of the soil-lime test pavements. Moduli of subgrade reaction determined from 30-in. plate-loading tests were used for inputs in the Westergaard model. The moduli of elasticity (E) were calculated by using the plate-loading data for the subgrade and, in addition, compression specimens (2-in. diameter by 4 in.) were prepared by kneading compaction at the moisture and density levels indicated in Table 7. Static (deformation rate of 0.05 in./min) compressive stress-strain data were used to determine an E for use in the elastic subgrade model. Although the two determinations of E were quite different for the soft subgrade, they were approximately the same for the stiff subgrade condition.

Calculations were made using k values of 50 and 450 psi/in. for the Westergaard model and E values of 880 and 1700 psi (soft grade) and 8000 psi (stiff grade) for the elastic solid subgrade model (Table 14).

PCA's analysis[16] of their load-deflection data for cement-stabilized materials indicated that the relation

$$P = \frac{wk}{\alpha\left(\frac{a}{h}\right)^{\beta}}$$

in which

| | | |
|---|---|---|
| P | = | plate pressure, psi |
| w | = | plate deflection, in. |
| k | = | modulus of subgrade reaction, psi/in. |
| a | = | radius of plate |
| h | = | thickness of stabilized layer |
| $\alpha, \beta$ | = | experimentally determined factors |

[16] P. J. Nussbaum and T. J. Larsen, *Load Deflection Characteristics of Soil-Cement Pavements*, Record No. 86 (Highway Research Board, 1965).

adequately described the observed response. For the low (relative to soil-cement) flexural-strength cement-treated mixtures studied, $\beta$ ranged from 1.52 to 1.59, with an average of 1.57. $\alpha$ also varied with higher flexural-strength materials characterized by smaller $\alpha$ values. For the soil-cements investigated (adequate cement content to meet PCA durability requirements), $\alpha$ was equal to 0.58, and $\beta$ was 1.52.

Subsequent PCA field studies[17] indicated reasonable agreement between the PCA load-deflection relation and field response data. Subsequently, PCA developed a soil-cement thickness-design procedure[18] based on the load-deflection relation. Thompson[19] has suggested that the PCA-developed load-deflection relation might also be considered for use with cured soil-lime mixtures. To check the applicability of the PCA procedure, the P-Δ data for the soil-lime pavements tested in this study were analyzed using the suggested PCA relation. Based on the PCA approach used in analyzing the Minnesota Test Road data,[20] the soil-lime load-deflection responses for 0.02 in. and 0.04 in. were averaged to determine a representative value of w/P. Since the w/P values for the 28- and 56-day curing time P-Δ curves were approximately the same, data for the two curing times were averaged for subsequent use in the PCA relation. A summary of the wk/P data is shown in Table 15.

Log-log plots of wk/P vs a/h for the soft (k=50 psi/in.) and stiff (k=450 psi/in.) grades as well as $\alpha$ and $\beta$ values determined from an analysis of the plots, are shown in Figure 20.

[17]T. J. Larsen, *Tests on Soil-Cement and Cement-Modified Bases in Minnesota*, Bulletin D112 (Portland Cement Association, 1967).

[18]T. J. Larsen, P. J. Nussbaum and B. E. Colley, *Research on Thickness Design for Soil-Cement Pavements*, Bulletin D142 (Portland Cement Association, 1969).

[19]M. R. Thompson, "Lime Treated Soils for Pavement Construction," *Journal of the Highway Division*, ASCE, Vol 94, No. HW2 (1968).

[20]M. R. Thompson, *Shear Strength and Elastic Properties of Lime-Soil Mixtures*, Record No. 139 (Highway Research Board, 1966).

**Table 15**

**Summary of wk/P Data**

| Section No. | Curing Time, days | wk/P (*) |
|---|---|---|
| 1 | 28 | .046 |
| | 56 | .037 |
| | | av = .041 |
| 2 | 28 | .231 |
| | 56 | .221 |
| | | av = .226 |
| 3 | 28 | .038 |
| | 56 | .035 |
| | | av = .037 |
| 4 | 28 | .136 |
| | 56 | .138 |
| | | av = .137 |
| 5 | 28 | .022 |
| | 56 | .022 |
| | | av = .022 |
| 6 | 28 | .124 |
| | 56 | .132 |
| | | av = .128 |

***Notes:**
(a) Average value for deflections of 0.02 in. and 0.04 in.
(b) W = deflection, in.
k = modulus of subgrade reaction, psi/in.
P = plate pressure, psi

## Discussion

*Elastic Layer and Elastic Plate.* The layered elastic and elastic plate theories have been used for analyzing various types of pavement systems. A major requirement for an acceptable pavement analysis and design procedure is the adequate prediction of the pavement response to applied load. Figure 21 illustrates the efficiency of the various procedures for predicting surface deflection. The following observations are significant:

1. The Westergaard theory (elastic plate, dense liquid subgrade) consistently predicted low deflections, relative to measured values.

2. The elastic plate-elastic subgrade theory was quite accurate for the stiff subgrade conditions, but inaccurate (predicted high) for the soft, subgrade conditions when the E calculated from plate-load test data (880 psi) was used. A much better agreement between the predicted

and measured deflection was obtained for the soft subgrades when the E determined from the compressive stress-strain data (1,700 psi) was used in the analysis.

3. The elastic layer theory quite accurately predicted the response of the sections on the stiff subgrade, but predicted high values, relative to the measured values, for the soft subgrade conditions (subgrade E at 880 psi). The predictions were substantially improved when the subgrade E of 1,700 psi was used to represent the soft grade.

A point of major disparity is the comparison of the theoretical flexural stresses in the soil-lime layer (Table 14) and the 28-day flexural strength of the material, which was 54 psi. In all except the 12 in. thick sections (5 and 10), the theoretical flexural stresses were substantially greater than the flexural strengths of the soil-lime mixture. Note that the data in Table 15 are for plate loads substantially less than the ultimate load-carrying capacity of the soil-lime placement system, thus accentuating the discrepancy between predicted flexural stresses and flexural strength.

To further check the applicability of the elastic layer procedure, measured and theoretical surface-deflection profiles were compared, considering only those sections which showed good agreement between theoretical and experimental plate deflections. The comparative data are shown in Figures 22 to 25. It is quite apparent that the deflection profiles disagree substantially, even though the plate deflections were comparable. In general, the elastic layer theory predicts a greater degree of "slab action" (load distribution over a larger area) than the experimental data would suggest. The discrepancy is particularly accentuated for the soft-subgrade support conditions (Figures 22 and 24).

*PCA Load Deflection Theory.* Although the

$$\left(\frac{wk}{p} - a/h\right)$$

log-log plots (Figure 20) can be represented by a straight-line relation, it is quite obvious that one relation ($\alpha$ and $\beta$ constant) does not adequately describe the soil-lime layer load-deflection behavior for both soft and stiff subgrade support conditions.

It is interesting to note that although the $\beta$ values of 0.84 and 0.98 are not greatly different, the $\alpha$ values of 0.21 and 0.048 differ substantially. The $\alpha$ and $\beta$ values for the soil-lime pavements fall within the range of the field data reported by Larsen[21] for cement-stabilized materials, but do not compare favorably with the PCA laboratory test data.[22] In contrast to the PCA study in which $\alpha$ and $\beta$ were considered as constants for soil-cement quality materials, Nielsen's[23] analysis of his data for cement-treated sand bases indicated that $\alpha$ was a function of base thickness and k and $\beta$ were functions of layer depth. The results of this study also indicate that a "unique relation" ($\alpha$ and $\beta$ constant) of the form proposed by PCA for soil-cement cannot be developed for soil-lime pavements.

*General.* The experimental data and analysis results indicate that reasonable predictions of the load-deflection response (plate deflections) of soil-lime layers can be made using the elastic plate-elastic subgrade theory and the elastic-layer theory. Elastic-layer theory does not adequately estimate the deflection basin of the loaded soil-lime layer. The Westergaard approach did not accurately predict load-deflection response.

The PCA load-deflection theory cannot be used to accurately describe the behavior of soil-lime pavement systems. Apparently the $\alpha$ and perhaps the $\beta$ terms in the PCA theory are not constants for soil-lime layers but perhaps are influenced by other factors (subgrade support, layer thickness), thus severely limiting the usefulness and applicability of the procedure.

[21] T. J. Larsen, *Tests on Soil-Cement and Cement-Modified Bases in Minnesota*, Bulletin D112 (Portland Cement Association, 1967).

[22] P. J. Nussbaum and T. J. Larsen, *Load-Deflection Characteristics of Soil-Cement Pavements*, Record No. 86 (Highway Research Board, 1965).

[23] J. P. Nielsen, "Thickness Design Procedure for Cement-Treated Sand Bases," *Journal of the Highway Division*, ASCE, Vol 94, No. HW2 (1968).

It is important to note that agreement between experimental data and theoretical results for the sections with soft subgrades was achieved only when an alternate method, compressive stress-strain data, was used to establish an E value for the soft grade. The adequacy of the procedure in which E values are back-calculated from plate-load data should be carefully considered, particularly in the case of soft subgrades.

Although some of the theories offer potential for predicting the static load-deflection response of soil-lime layers, substantial discrepancies exist between the predicted flexural stresses and the flexural strength of the soil-lime mixture, and the deflection profiles which are theoretically measured. Most rationally based pavement design procedures include some provisions relative to allowable stresses and thus an accurate procedure is needed for predicting stresses in the paving materials. The PCA procedure for soil-cement thickness design is an exception since deflection and radius of curvatures, not stress, are the bases of the procedure.

Recent studies by the Federal Highway Administration (FHWA)[21] have demonstrated that linear viscoelastic theory is "much better" for predicting the response of a flexible pavement (asphalt concrete surface, crushed stone base, soil subgrade) to static loading rather than for dynamic loading conditions. In fact, if it is desirable to accurately predict the static response of pavements containing soil-lime layers, it would be appropriate to consider the possibility of using a viscoelastic theory similar to the one employed by Kenis in the FHWA investigation.

[21] W. J. Kenis, *Comparisons Between Measured and Predicted Flexural Pavement Responses*, Report No. FHWA-RD-72-10 (Federal Highway Administration, 1973).

## 5 DYNAMIC LOADING—DATA ANALYSIS

Recent pavement studies (see summary in Allen)[25] have shown that the accurate prediction of pavement response under dynamic loading conditions requires the use of materials-testing procedures and techniques capable of characterizing the stress-dependent properties of the materials in the pavement section. Finite element analysis methods have been developed for considering the dynamic loading of pavements. The procedure used to analyze the soil-lime pavement data has been considered in detail by Allen. The basic procedure, developed by E.L. Wilson and J.M. Duncan at the University of California, Berkeley, has been subsequently modified at the University of Illinois. Distinct advantages were attained by using the finite-element procedure for analyzing the dynamic-loading data. It was possible to accurately consider the stabilized layer-subgrade soil-test bin system. The bottom of the test-bin floor was assumed as a fixed lower boundary which was constrained vertically and horizontally. Thus, in effect, no deformation could accumulate below that level. In addition, the stress-dependent resilient moduli properties of the subgrade could be adequately considered.

**Material Properties.** Dynamic-compressive and flexural properties of the cured soil-lime mixtures were determined as described in Chapter 2. The test data are summarized in Tables 4 and 6.

The stress-dependent resilient behavior of the GLC subgrade soil was characterized by using a procedure developed by Robnett and Thompson.[26] Specimens (2-in. diameter by 4 in.) were compacted (kneading procedure) at moisture contents and densities similar to those achieved during the construction of the soft and stiff subgrades. The specimens were subjected

[25] J. J. Allen, *The Effects of Stress History on the Resilient Response of Soils*, Technical Report M-49/AD762194 (CERL, 1973).

[26] Q. L. Robnett and M. R. Thompson, "Interim Report-Resilient Properties of Subgrade Soils-Phase I-Development of Testing Procedure," *Civil Engineering Studies*, Transportation Engineering Series No. 5, Illinois Cooperative Highway Research Program, Series No. 139 (University of Illinois, 1973).

to repeated loading (various axial-stress levels, 60 msec pulse-duration, 20-load-applications per minute) and the resilient deformations were monitored. The resilient modulus was calculated as the repeated axial stress divided by recoverable resilient axial strain based on response data obtained following the application of 1000 conditioning stress applications (5 psi).

The flexural moduli of elasticity data in Table 6 were used to characterize the soil-lime layers in the finite element model. The resilient response relations in Figures 26 and 27 were used to represent the subgrade soils. Concrete in the test-bin floor was assumed to have an $E = 3 \times 10^6$ psi and $\mu = 0.17$.

**Finite Element Analysis.** Selected sections (Table 16) were analyzed using the finite element method to determine the adequacy of the technique for predicting the dynamic response of the test sections. Plots showing the relations between experimental and theoretical subgrade surface deflections are presented in Figures 28 through 36. Some concern was expressed about the validity of the plate-deflection data collected during dynamic testing. In many instances, the plate deflection data records showed substantial "bouncing," indicating that perhaps the plate did not maintain contact with the pavement surface at all times. Consequently, emphasis was placed on using subgrade deflection for checking the adequacy of the finite-element method.

**Discussion.** For most of the conditions considered, the finite element method rather accurately predicted the dynamic subgrade deflection. Table 16 qualitatively summarizes the extent of agreement between the experimental and theoretical data. In all cases, the "poor" and "fair" agreement ratings were for Sections 1, 3, and 5, constructed on soft subgrades. Difficulty was also experienced in getting good agreement between "experimental" and "theoretical" values for the elastic-layer and elastic-plate analyses of the static-loading data for the soft grade condition. Table 16 indicates that it is more difficult to accurately predict the dynamic behavior of sections with thinner soil-lime layers, soft subgrades, and short-time curing (lower-strength soil-lime mixtures).

Table 16

Summary of Finite-Element Analyses

| Section No. | Curing Period, Days | Soil-Lime E, (a) psi | Qualitative Agreement (b) Good | Fair | Poor |
|---|---|---|---|---|---|
| 1 | 28 | 226,000 | | | X |
| 2 | 28 | 226,000 | X | | |
| 3 | 28 | 226,000 | X | | |
| 4 | 2 | 128,000 | | X | |
| 4 | 28 | 226,000 | X | | |
| 5 | 2 | 128,000 | | | X |
| 5 | 14 | 200,000 | X | | |
| 5 | 28 | 226,000 | | | X |
| 6 | 28 | 226,000 | X | | |

Notes:
(a) Constant E value used in the analyses for the soil-lime layers.
E selected from Table 6 for approximately 50 percent stress level.
(b) Judgment based on qualitative examination of Figures 28 to 36.

Finite-element analyses were used to determine flexural stresses and strains at the bottom of the soil-lime layer for plate-loading which corresponded approximately to the maximum loads applied during the static-loading sequence. The data are summarized in Table 17. Comparison of the calculated stress and strain data with the soil-lime mixture property data in Table 5 indicates that, in general, the theoretical stresses and strains are less than the failure values determined from the static laboratory-testing program. It is encouraging to achieve a better agreement between the theoretical stresses and strains and measured material property data. Although one might question the desirability of comparing *dynamic* theoretical-analysis results with *static* materials-testing data, the flexural moduli of the soil-lime mixtures determined from static and dynamic testing do not differ substantially, although the dynamic moduli tend to be higher (Figure 37).

Dynamic surface deflection profiles were compared to further evaluate the finite element theory. Experimental and theoretical data for selected sections are shown in Figures 38 to 41. Because of concern about the validity of the plate deflection data for the dynamic loading condition, data are shown only for points not located on the load plate. Fairly good agreement is noted between the theoretical and experimental data. In contrast to the elastic layer theory, the finite element theory predicts less slab action or load transfer over a smaller area. It is important to recall that the subgrade response model used in the finite element analyses is assumed to be stress-dependent. Thus, it is possible to develop increased subgrade stiffness in those areas of low deviator stress which are further removed from the loading area.

**Dynamic vs Static Behavior.** It was apparent during the data analyses that the dynamic deflections were substantially less than the static values. In a comparison of static and dynamic subgrade surface deflection data (center of plate load), shown in Table 18 and Figure 42, the ratios of static to dynamic deflections varied from 1.5 to 3.0 with an average of 2.2.

Similar findings have been reported by Larsen[27] for cement-stabilized pavements in Minnesota. The effects of vehicle speed on total surface deflection and embankment deflection were studied for conventional flexible pavements at the AASHO Road Test. The AASHO study[28] indicated that increases in loading speed from 2 to 35 mph resulted in deflection reductions in the range of approximately 40 percent and the partial deflections (depth of the deflection basin measured under a 2 ft chord at the bottom of the deflection basin) were reduced on the order of 60 to 70 percent.

## 6 ULTIMATE LOAD TESTING—DATA ANALYSIS

**Meyerhof's Theory.** Studies[29,30] of lime-fly ash-aggregate and soil-cement-base course behavior have shown that ultimate load carrying capacity is much greater than predicted by elastic-layer theory. Meyerhof's[31] ultimate load theory correlated much better, for both static and fatigue loading, with observed experimental behavior. Field-performance data[32] also have indicated that the ultimate strength design approach is realistic.

Meyerhof's[33] equation for ultimate loading-carrying capacity under interior loading conditions was used to determine ultimate loads for the soil-lime layers tested in this study.

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[27] T. J. Larsen, *Tests on Soil-Cement and Cement-Modified Bases in Minnesota*, Bulletin D112 (Portland Cement Association, 1967).

[28] *The AASHO Road Test, Report 5—Pavement Research*, Special Report #61 (Highway Research Board, 1962).

[29] H. L. Ahlberg and E. J. Barenberg, *Pozzolanic Pavements*, Bulletin 473 (University of Illinois, 1965).

[30] E. J. Barenberg, *Evaluating Stabilized Materials* [unpublished], (University of Illinois, 1967).

[31] G. G. Meyerhof, "Load Carrying Capacity of Concrete Pavements," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol 88, No. SM3 (June 1962).

[32] E. J. Barenberg, "The Behavior and Performance of Asphalt Pavements with Lime-Flyash-Aggregate Bases," *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan (1967)*.

[33] G. G. Meyerhof, "Load Carrying Capacity of Concrete Pavements."

Table 17

**Summary of Flexural Stress and Strain Data From Finite Element Analyses**

| Section No. | Curing Period, Days | Plate-Loading (a) Total Load, lb | Plate Pressure, psi | Theoretical Values (b) Flexural Stress, psi | Flexural Strain, Microstrain |
|---|---|---|---|---|---|
| 1 | 28 | 6,220 | 55 | 41 | 252 |
| 2 | 28 | 6,882 | 60 | 41 | 251 |
| 3 | 28 | 9,000 | 80 | 42 | 230 |
| 4 | 2 | 8,010 | 71 | 26 | 235 |
| 4 | 28 | 20,000 | 177 | 92 | 197 |
| 5 | 2 | 9,000 | 80 | 23 | 193 |
| 5 | 14 | 12,860 | 102 | 38 | 194 |
| 5 | 28 | 16,075 | 128 | 48 | 246 |
| 6 | 28 | 24,870 | 220 | 79 | 105 |

Notes:
(a) 12-in. diameter plate.
(b) Based on finite element analyses, see Table 16. Values are for the centerline of the plate at the bottom of the soil-lime layer.

Table 18

**Static and Dynamic Loading Comparisons of Subgrade Surface Deflection**

| Section No. | Curing Time, Days | Plate Load, lb | Subgrade Surface Deflection, in. Dynamic | Static | Ratio, Static, Dynamic |
|---|---|---|---|---|---|
| 1 | 14 | 3,800 | .013 | .037 | 2.8 |
| | 28 | 4,200 | .014 | .035 | 2.5 |
| | 56 | 5,200 | .018 | .038 | 2.1 |
| 2 | 28 | 5,800 | .008 | .017 | 2.1 |
| 3 | 56 | 10,000 | .029 | .064 | 2.2 |
| 4 | 2 | 5,900 | .005 | .013 | 2.6 |
| | 14 | 11,500 | .013 | .023 | 1.8 |
| | 28 | 14,000 | .014 | .025 | 1.8 |
| | 56 | 14,250 | .015 | .023 | 1.5 |
| 5 | 2 | 7,000 | .008 | .024 | 3.0 |
| | 14 | 10,000 | .012 | .028 | 2.3 |
| | | | | | Av 2.2 |

$$P_0 = \frac{4 - f_b h^2}{\left(1 - \frac{a}{3L}\right) 6} \text{ for } a/L > 0.2$$

$P_0$ = ultimate load, lb

$f_b$ = modulus of rupture material, psi

h = slab thickness, in.

a = radius of circular loaded area

L = radius of relative stiffness

$L = 4\sqrt{Eh^3/12(1 - \mu^2)k}$

E = modulus of elasticity of material, psi

$\mu$ = Poisson's ratio

and k = modulus of subgrade reaction, psi/in.

Ultimate load-testing was conducted only after the 56-day curing period; thus, the results of only six tests are available. A flexural strength of 55 psi, a $\mu$ of 0.2, and an E of 216,000 psi were used based on the previous laboratory soil-lime mixture test data summarized in Table 5. Ultimate loads predicted by Meyerhof's theory are shown in Table 19. A comparison between the predicted ultimate load-carrying capacity and the plate-loading data is shown in Figure 43. Actual load-carrying capacities were substantially greater, by at least a factor of two, than the predicted values for all sections.

Based on the test data and the Meyerhof theoretical results, it is possible to evaluate the effects of subgrade support and slab thickness on ultimate load-carrying capacity, as shown in Table 20. In general, the theory underestimated the effects of subgrade support and overestimated the influence of thickness. Figure 44 illustrates the effect of thickness, as determined from plate load test data, for the soft and the stiff subgrade. The relative effect of thickness is more significant for the soft subgrade condition. Thickness effects for the stiff subgrade were nominal, as indicated by the load test data ultimate load ratios in Table 20.

**General Approach.** Examination of the ultimate load-deflection plots, Figures 10 to 15, indicates that the slopes of the P-Δ plots do not change substantially until a deflection of approximately 0.1 in. is reached. A majority of the ultimate load-carrying capacity is also developed prior to exceeding the 0.1 in. deflection value. Table 21 summarizes the load and deflection ratios for the sections. The load-ratio is the plate-load at a deflection of 0.1 in. divided by the ultimate load-carrying capacity, and the deflection ratio of 0.1 in. divided by the deflection required to develop the ultimate load.

The range of values for the load and deflection ratios is not large. Based on average values, 73 percent of the ultimate load-carrying capacity was developed at a deflection equal to about one-third the deflection required to develop the ultimate load-carrying capacity.

**Discussion.** Substantial ultimate load capacities are developed by cured soil-lime layers, as demonstrated by the data in Table 19. It is emphasized that the data in Table 19 are for only one curing period (56 days) and thus strength effects cannot be established.

Meyerhof's theory (interior loading conditions) did not satisfactorily predict the ultimate load response of the test sections, but was consistently conservative by at least a factor of two. The theory was verified qualitatively since both increased thickness and subgrade support contributed to the development of higher ultimate load-carrying capacities.

The general finding that a substantial portion (73 percent) of the ultimate load-carrying capacity can be developed at fairly low deflections of 0.1 in. was an encouraging development. Many expedient or low-traffic-volume pavements which display surface deflections in the range of 0.1 in. provide satisfactory performance. In order to maintain perspective, it should be noted that load-carrying capacities of 9k (highway truck traffic) were developed in the test section at deflections substantially less than 0.1 in., with the exception of Section 1, which is the thin soil-lime layer, weak subgrade.

Table 19

**Summary of Ultimate Load-Test and Predictions (56-Day Curing)**

| Section | Predicted Ultimate Load Carrying Capacity lb | Δ at Predicted Ultimate Load, in. (a) | Load Test Data (b) Max. Load, lb | Δ at Max. Load, in. | Load at Δ 0.1 in. (c) lb |
|---|---|---|---|---|---|
| 1 | 4,705 | 0.028 | 15,300 | 0.274 | 10,000 |
| 2 | 5,220 | 0.027 | 40,000 | 0.307 | 27,000 |
| 3 | 10,225 | 0.040 | 22,000 | 0.400 | 16,100 |
| 4 | 11,000 | 0.022 | 36,000 | 0.28 | 28,800 |
| 5 | 17,850 | 0.046 | 36,500 | 0.35 | 26,000 |
| 6 | 18,900 | 0.044 | 48,000 | 0.282 | 38,100 |

Notes:
(a) Deflection from ultimate plate-load data corresponding to the "predicted" ultimate load-carrying (Meyerhof's theory, interior loading).
(b) Values determined from ultimate plate-load data, Figures 10 to 15.
(c) Plate load corresponding to a plate deflection of 0.1 in.

Table 20

**Effect of Subgrade Support and Thickness on Ultimate Load-Carrying Capacity**

| Comparison | Ultimate Load Ratio (a) Meyerhof Theory | Load Test Data |
|---|---|---|
| **Subgrade Effects** | | |
| $P_{450}/P_{50}$(b) | | |
| H = 6 in. | 1.11 | 2.62 |
| H = 9 in. | 1.08 | 1.64 |
| H = 12 in. | 1.06 | 1.31 |
| **Thickness Effects** | | |
| $P_9/P_6$(c) | | |
| k = 50 psi/in. | 2.08 | 1.44 |
| k = 450 psi/in. | 2.1 | 0.9 |
| $P_{12}/P_6$ | | |
| k = 50 psi/in. | 3.8 | 2.38 |
| k = 450 psi/in. | 3.6 | 1.2 |
| $P_{12}/P_9$ | | |
| k = 50 psi/in. | 1.71 | 1.66 |
| k = 450 psi/in. | 1.72 | 1.33 |

Notes:
(a) Ratio of ultimate loads for the comparison indicated.
(b) Ratio of ultimate load for k = 450 psi/in. to ultimate load for k = 50 psi/in. for thicknesses (in.) indicated.
(c) Ratio of ultimate loads for the thicknesses (in.) indicated by the P subscripts.

Table 21

**Load and Deflection Ratios for 0.1 Inch Deflection**

| Section No. | Load Ratio (a) | Deflection Ratio (b) |
|---|---|---|
| 1 | 0.65 | 0.36 |
| 2 | 0.68 | 0.33 |
| 3 | 0.75 | 0.25 |
| 4 | 0.80 | 0.35 |
| 5 | 0.71 | 0.29 |
| 6 | 0.80 | 0.35 |
| | Av 0.73 | Av = 0.32 |

Notes:
(a) Load at a deflection of 0.1 in., divided by the ultimate load carrying capacity.
(b) Deflection of 0.1 in., divided by the deflection corresponding to the ultimate load-carrying capacity.

# 7 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are based on the results and discussion reported in the preceding chapters:

1. Substantial load-carrying capacity can be developed in soil-lime pavements. Increases in soil-lime layer thickness, soil-lime mixture strength, and subgrade support create additional load-carrying capacity.

2. For static-loading conditions, the n-layered elastic layer theory and the elastic plate on an elastic subgrade theory can estimate plate deflection with a fair degree of accuracy; however, neither theory accurately predicts the surface deflection basin at points remote from the loading plate. Careful consideration should be given to determining the appropriate moduli of elasticity values.

3. The Westergaard analysis did not accurately predict the behavior of the soil-lime pavements.

4. The n-layered elastic and the elastic plate on an elastic subgrade theories predict extremely high flexural stresses at the bottom of the soil-lime layer, as compared to static flexural strength data.

5. The PCA load-deflection response model has limited applicability for describing soil-lime pavement behavior. The results of this study indicate that the $\alpha$ and $\beta$ terms of the PCA load-deflection relation are not constant for soil-lime layers, but may vary depending on such parameters as subgrade strength, soil-lime mixture strength, and layer thickness.

6. For the same magnitude of loading, measured static pavement responses, as determined from subgrade surface deflection, were an average of 2.2 times greater than dynamic values.

7. Using a constant E value for the soil-lime mixture and a stress-dependent subgrade resilient modulus, the finite-element theory effectively predicted the dynamic behavior of the test pavements. Plate deflections were predicted more accurately than the surface deflection basins.

8. Soil-lime mixture flexural stresses and strains at the bottom of the pavement layers seemed reasonable and compared favorably with static flexural data when the finite element theory was used to predict the pavement response.

9. Soil-lime pavement systems are capable of developing substantial ultimate load-carrying capacities. In general, approximately 73 percent of the ultimate load-carrying capacity is developed at a deflection of 0.1 in., about one-third of the deflection noted at the ultimate load-carrying capacity.

10. Ultimate load-carrying capacity was affected by soil-lime-layer thickness and subgrade support. Larger ultimate load-carrying capacities were achieved with thicker pavement layers and increased subgrade support. Mixture strength was not a variable considered in this study, but Meyerhof's theory indicates that the load-carrying capacity should also increase with the development of higher flexural strengths in the pavement layer.

11. Meyerhof's ultimate-load theory did not accurately predict the behavior of the soil-lime pavements tested. The theory was conservative and the measured load-carrying capacities were at least twice those predicted using Meyerhof's equations for interior loading.

12. Qualitatively, the measured ultimate load-carrying capacities compared favorably with Meyerhof's theory. However, the theory underestimated the effect of subgrade support and overestimated the influence of pavement-layer thickness.

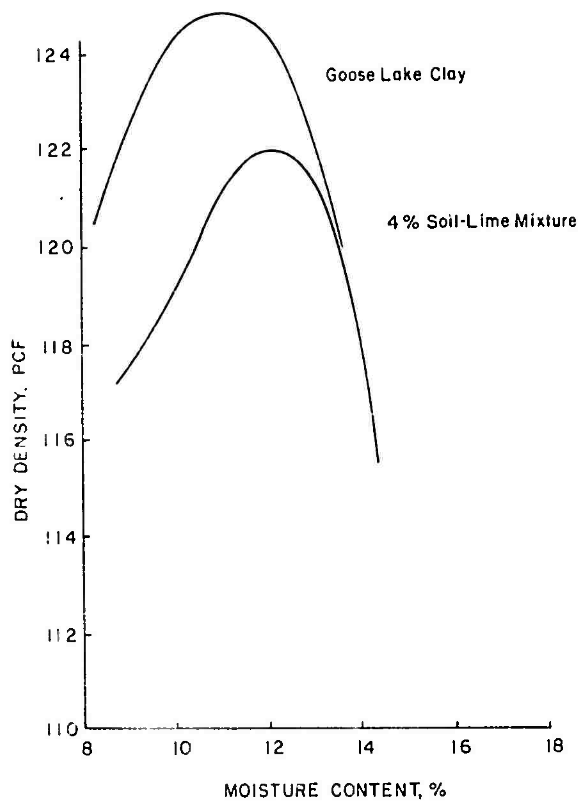


**Figure 1.** Moisture density relationship for GLC.

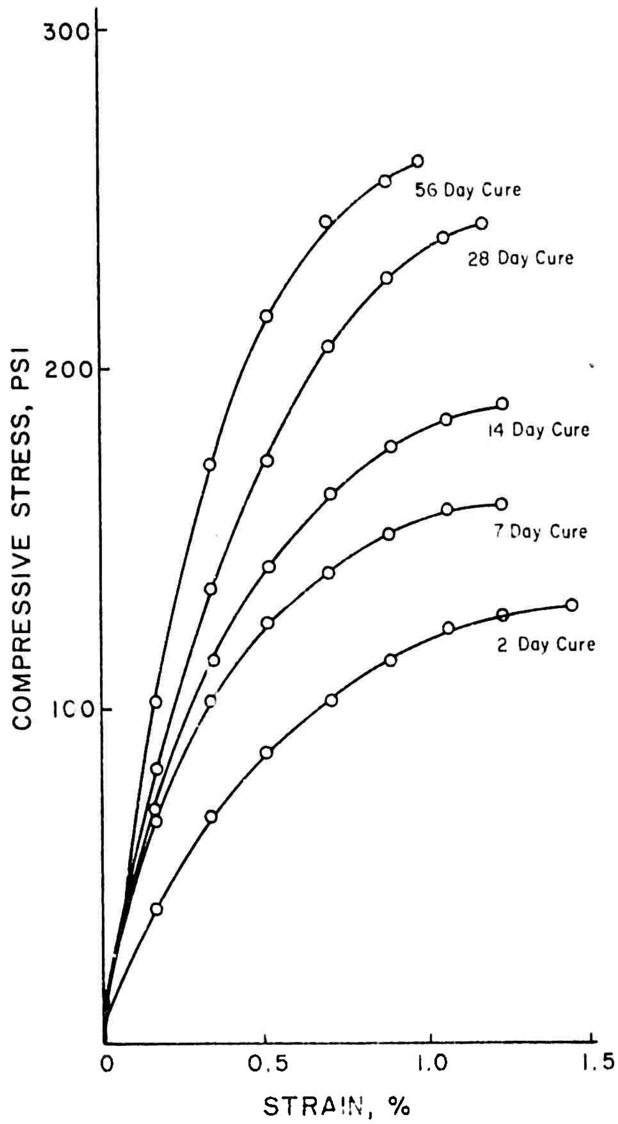


**Figure 2.** Static compressive stress-strain curves.

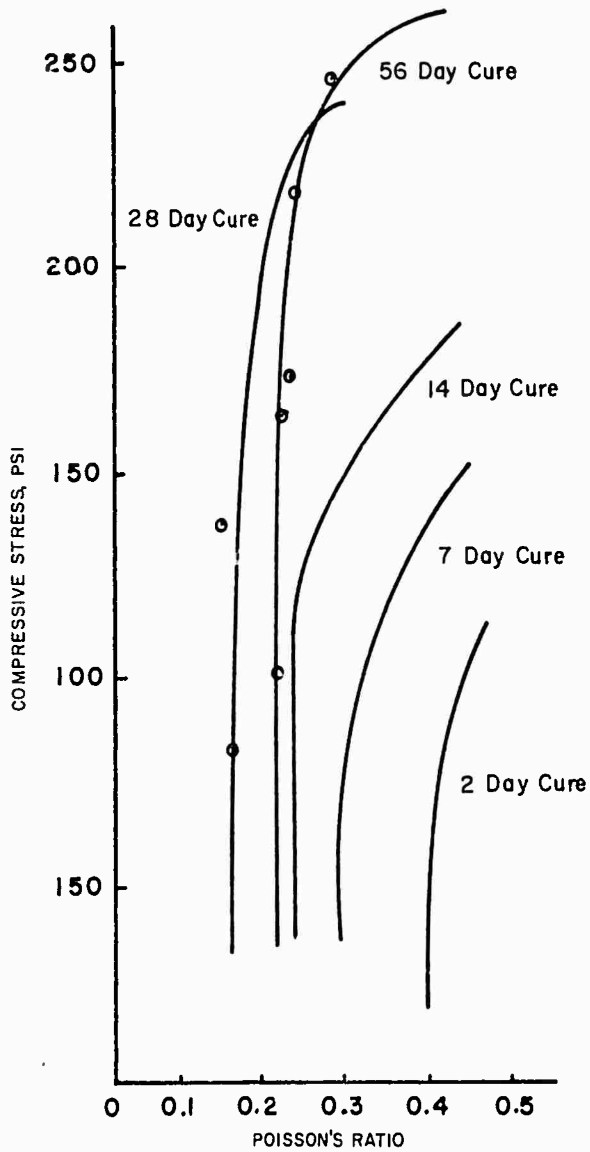


Figure 3. Compressive strength vs Poisson's ratio.

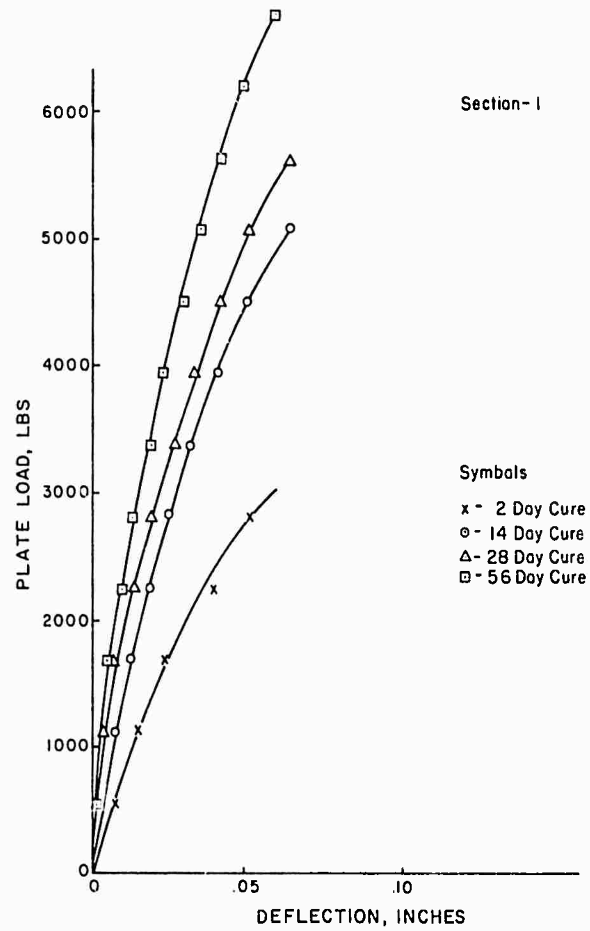


Figure 4. Load-deflection relationships for section 1.

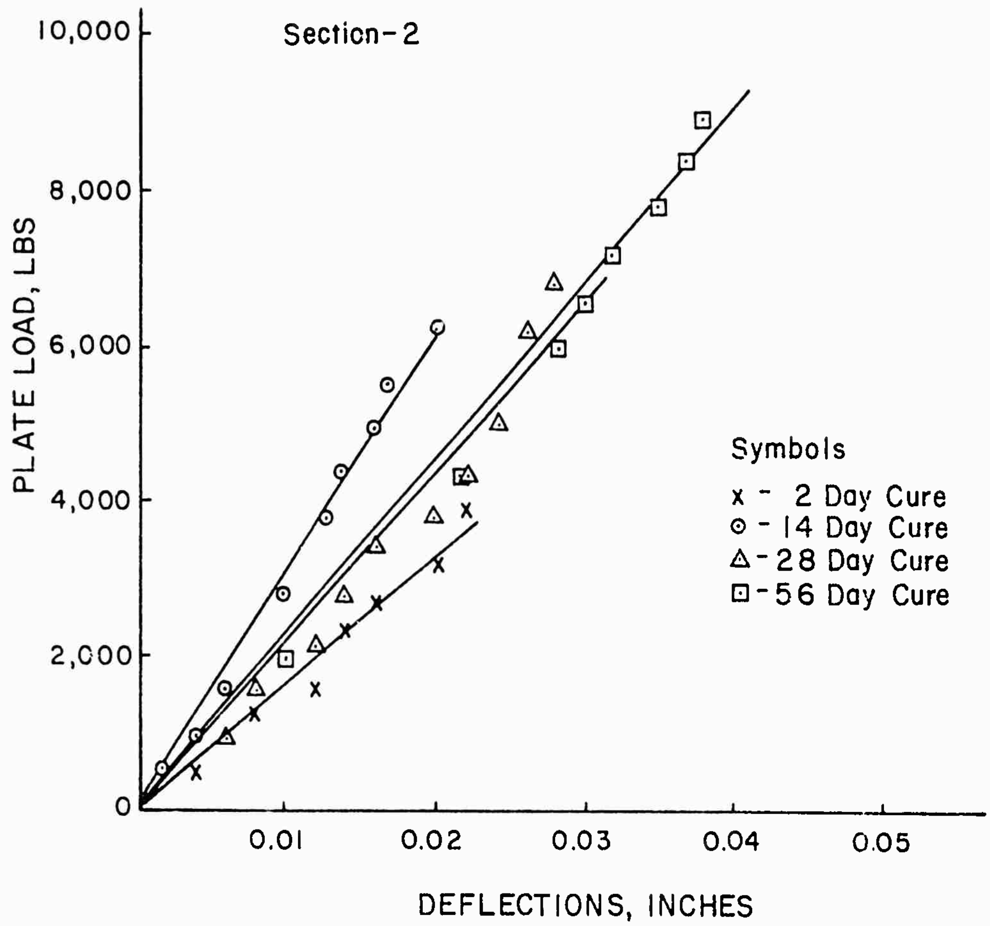


Figure 5. Load-deflection relationships for section 2.

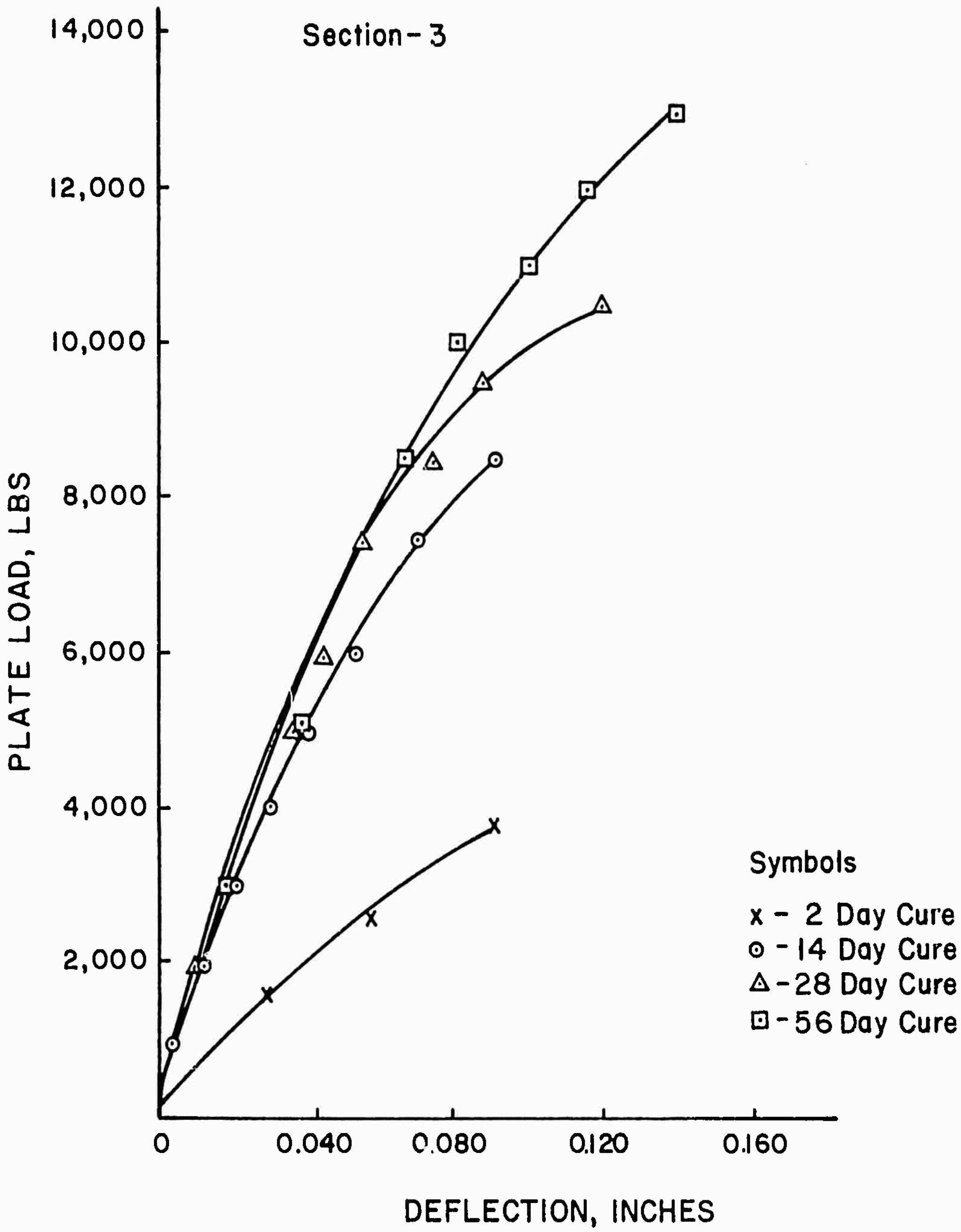


**Figure 6.** Load-deflection relationships for section 3.

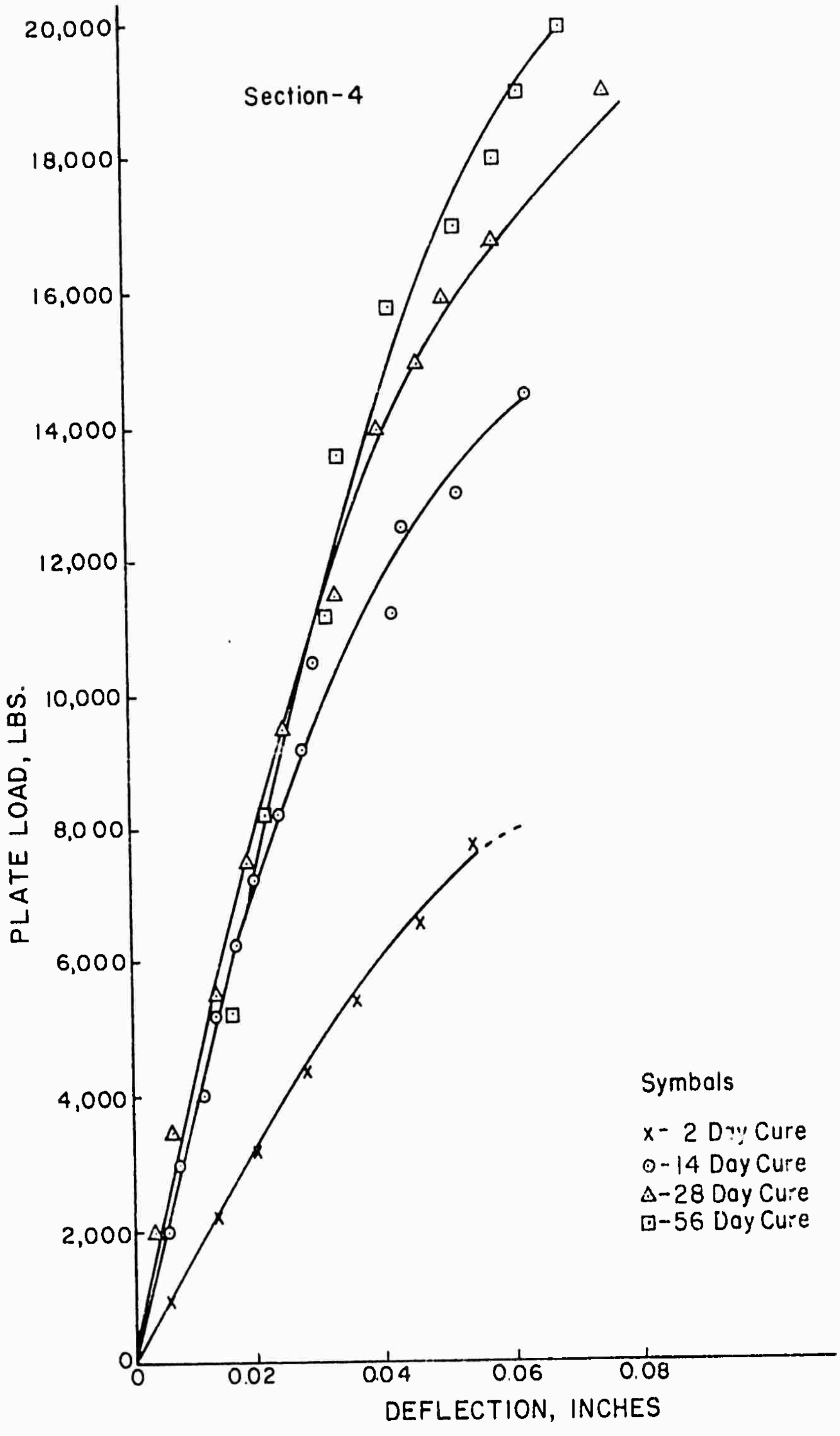


**Figure 7.** Load-deflection relationships for section 4.

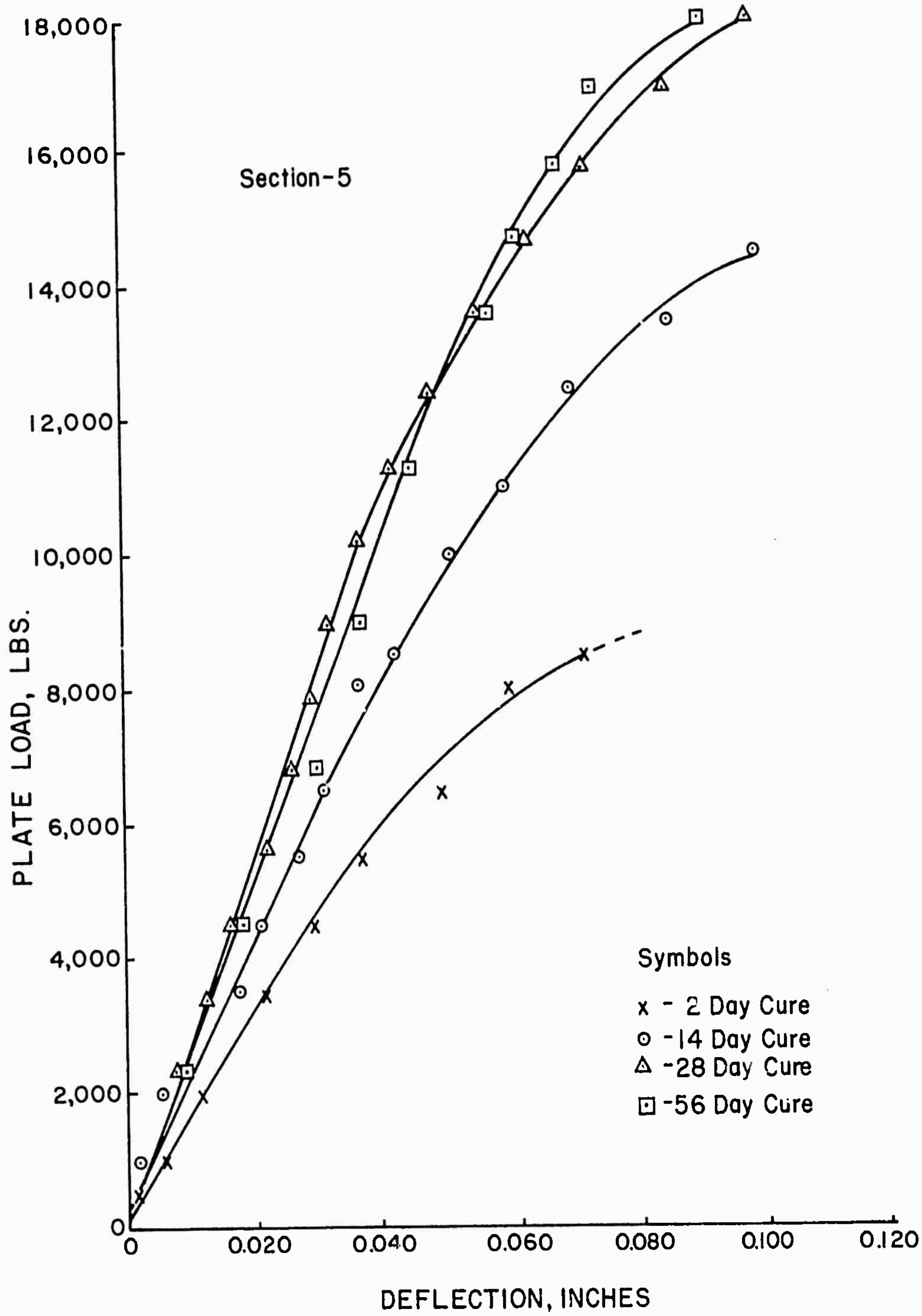


Figure 8. Load-deflection relationships for section 5.

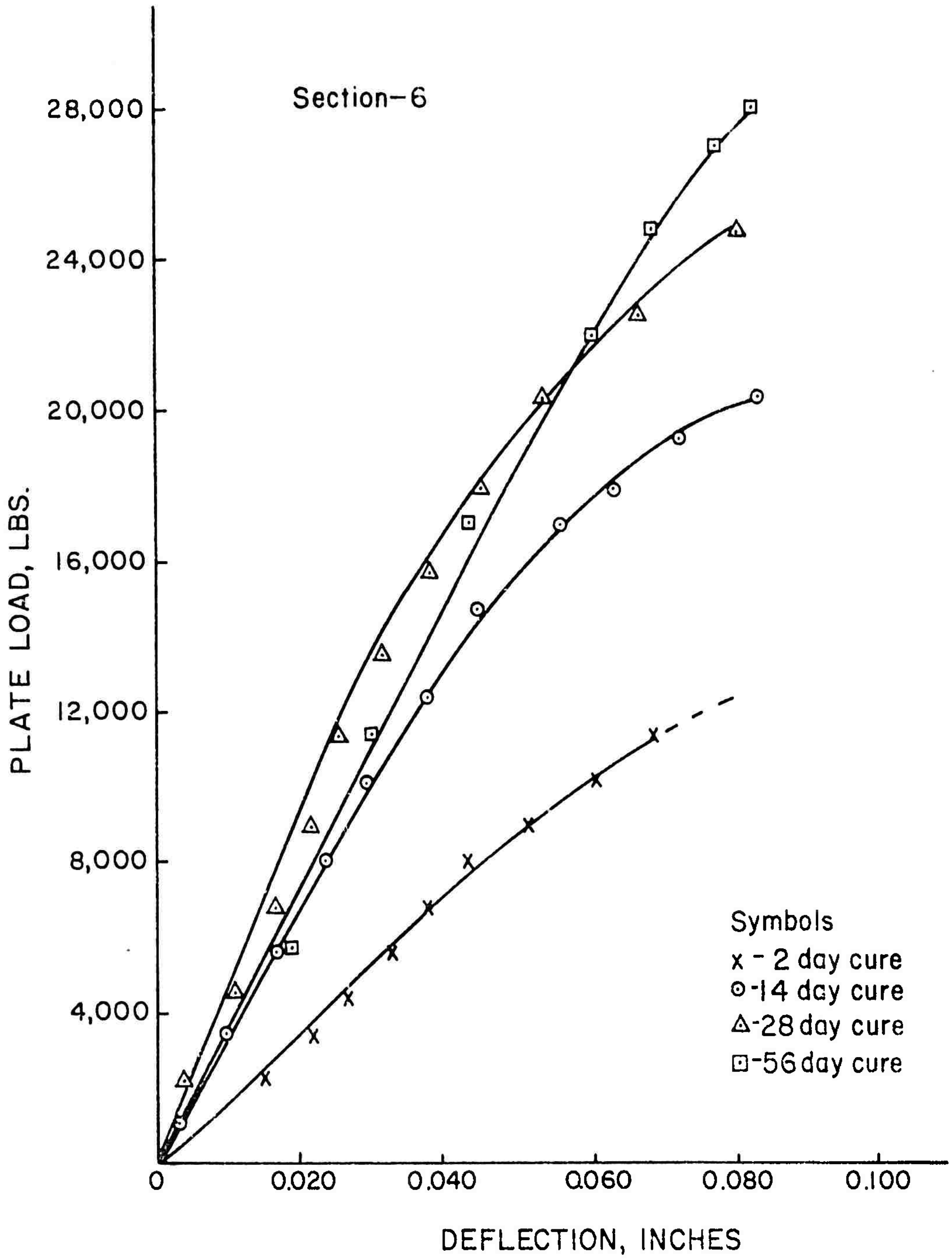


**Figure 9.** Load-deflection relationships for section 6.

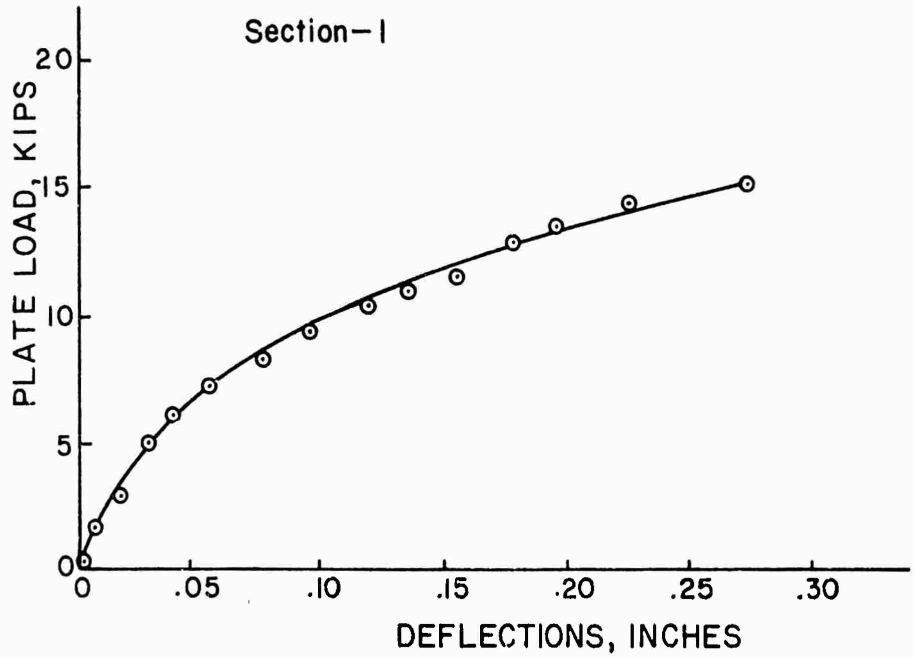


Figure 10. Load-deflection relationship for section 1 (ultimate load test).

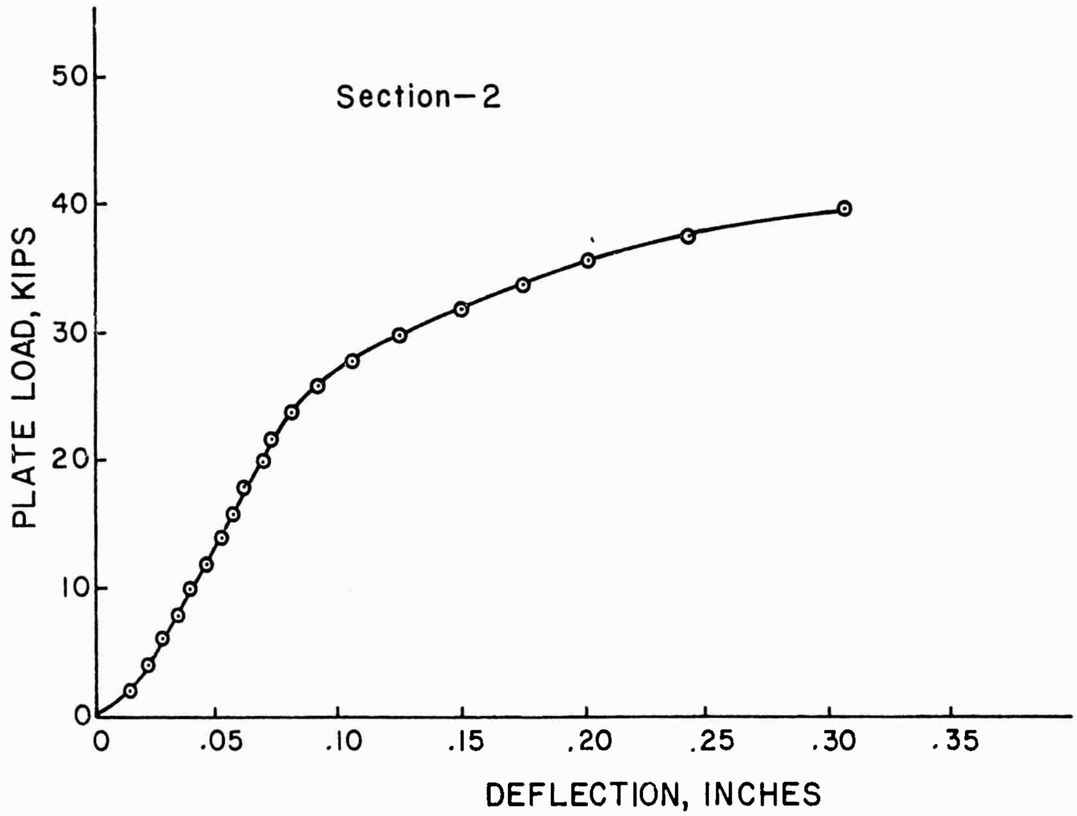


Figure 11. Load-deflection relationship for section 2 (ultimate load test).

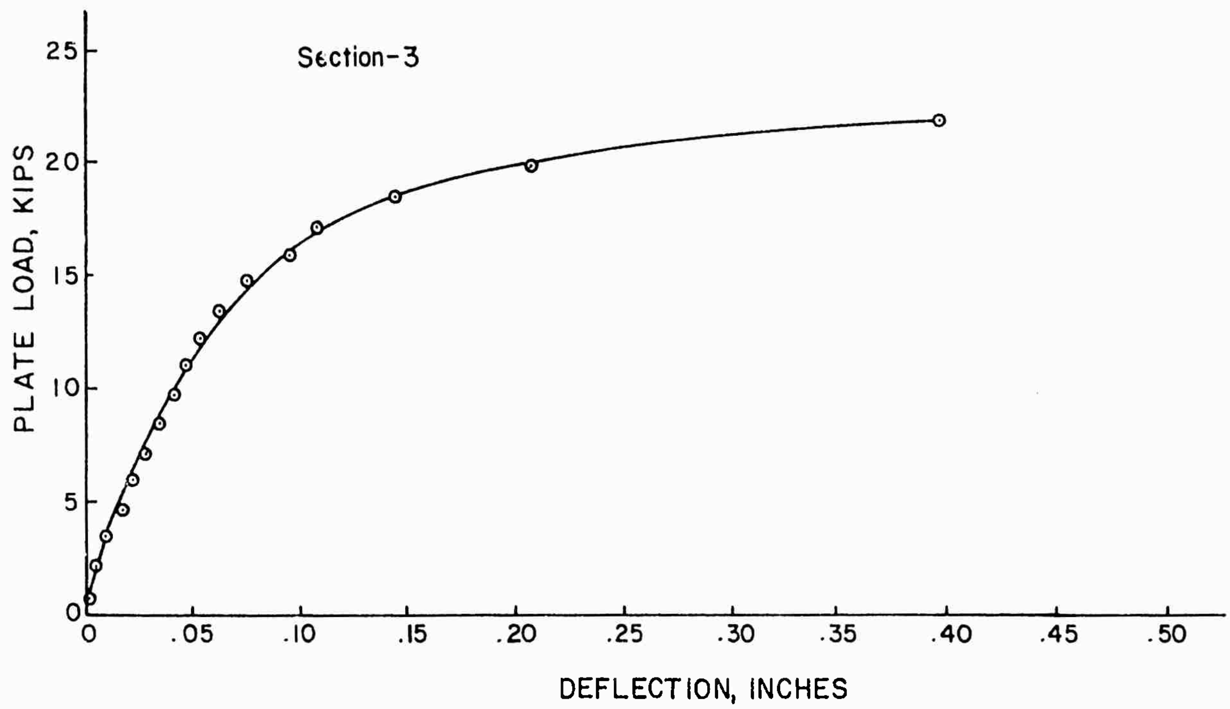


**Figure 12.** Load-deflection relationship for section 3 (ultimate load test).

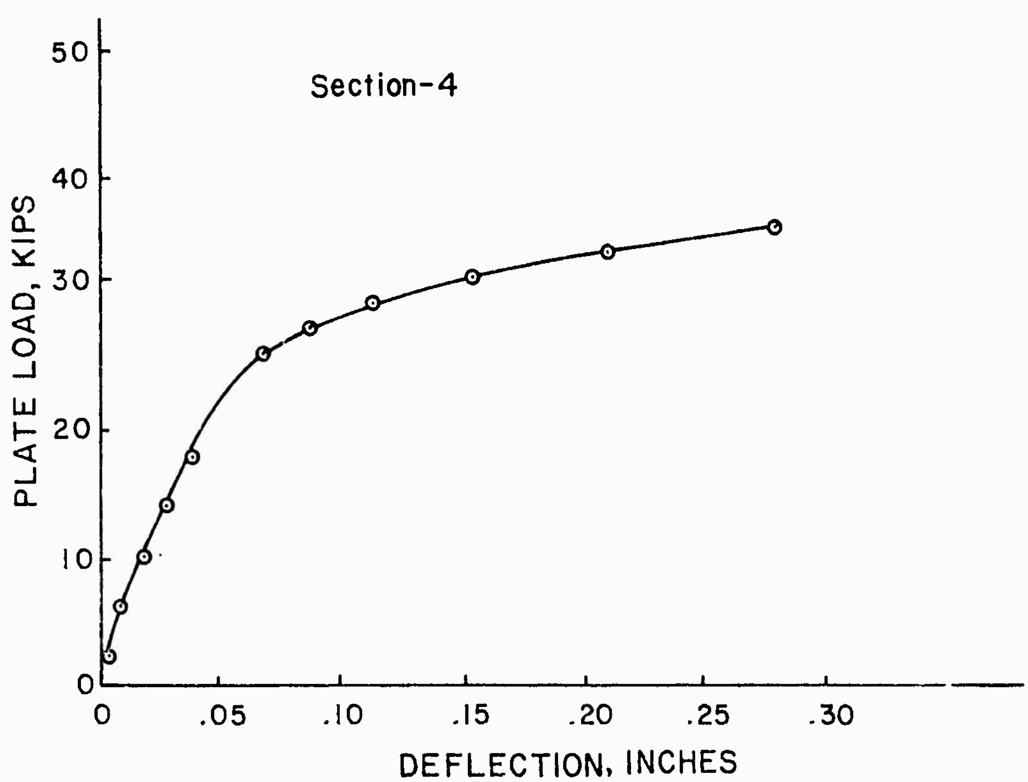


**Figure 13.** Load-deflection relationship for section 4 (ultimate load test).

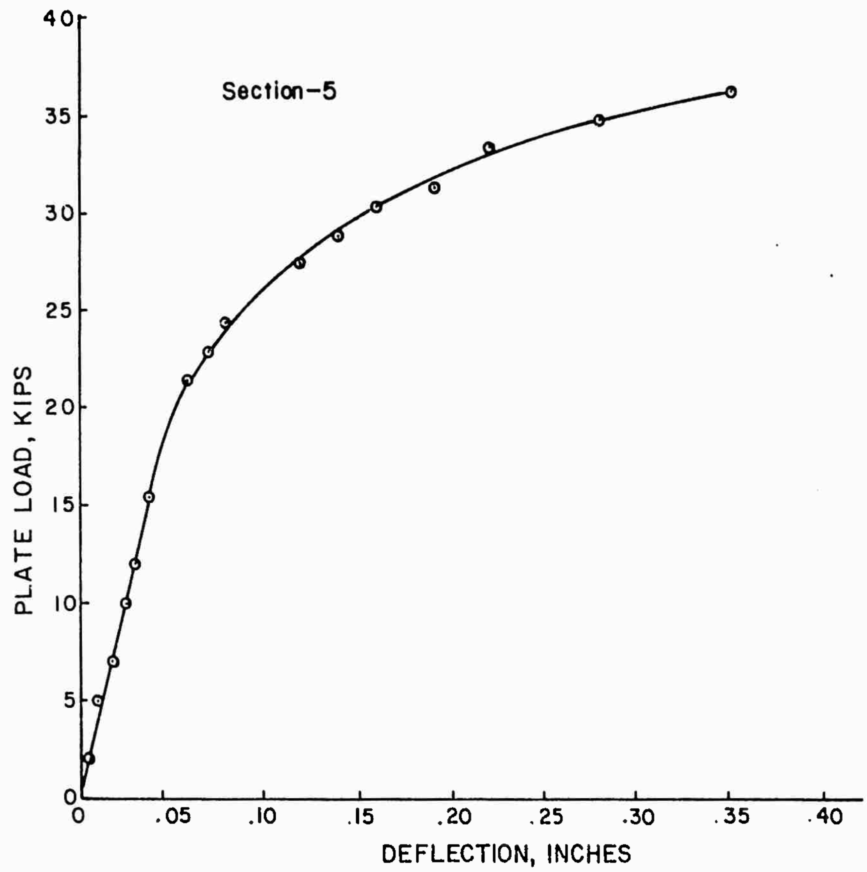


**Figure 14.** Load-deflection relationship for section 5 (ultimate load test).

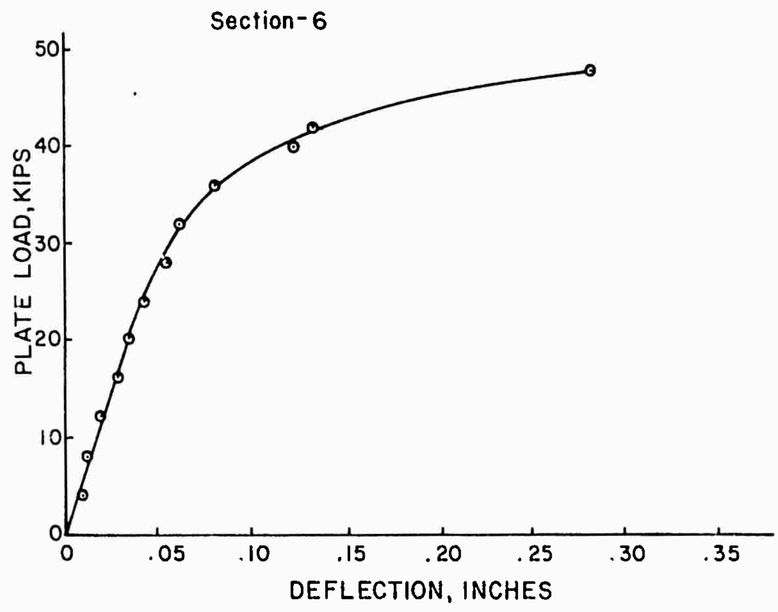


**Figure 15.** Load-deflection relationship for section 6 (ultimate load test).

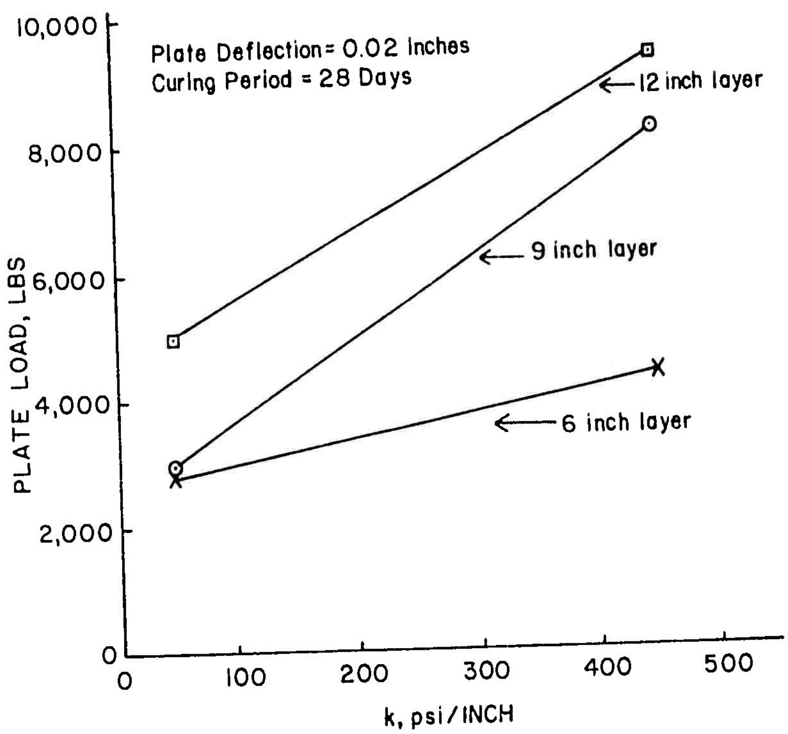


**Figure 16.** Influence of subgrade strength on load-carrying capacity.

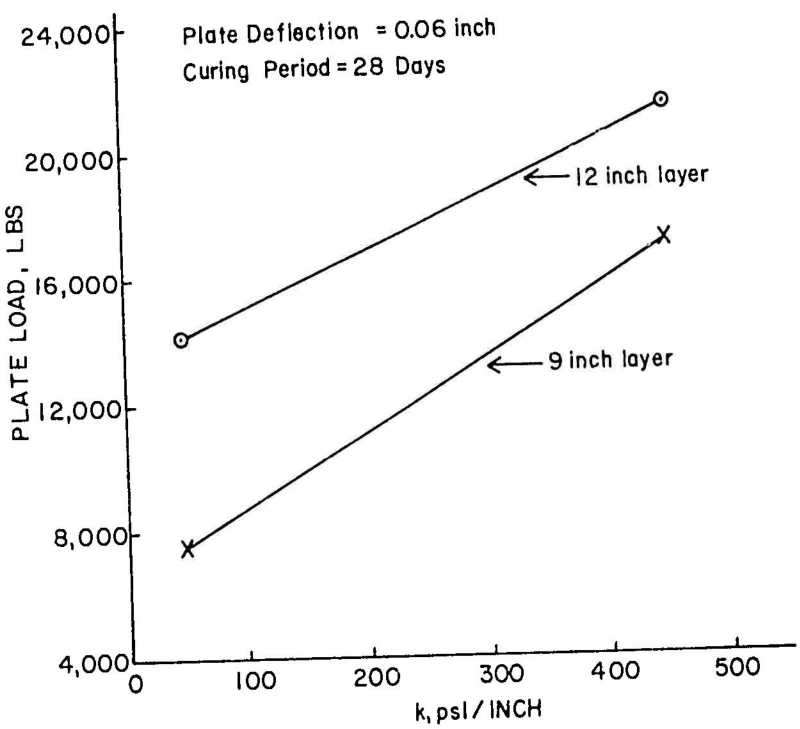


**Figure 17.** Influence of subgrade strength on load-carrying capacity.

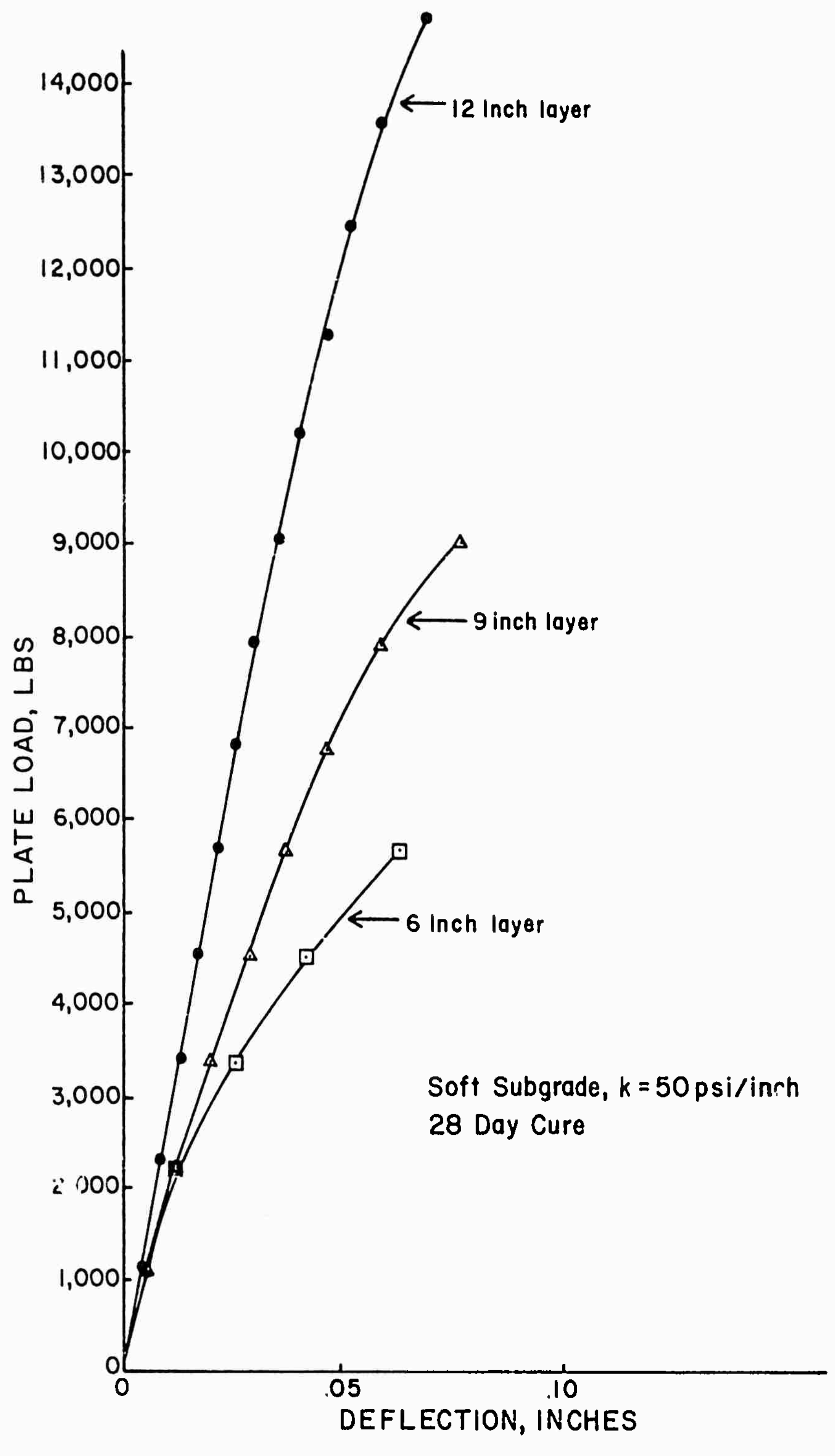


Figure 18. Influence of thickness responses for soft subgrade.

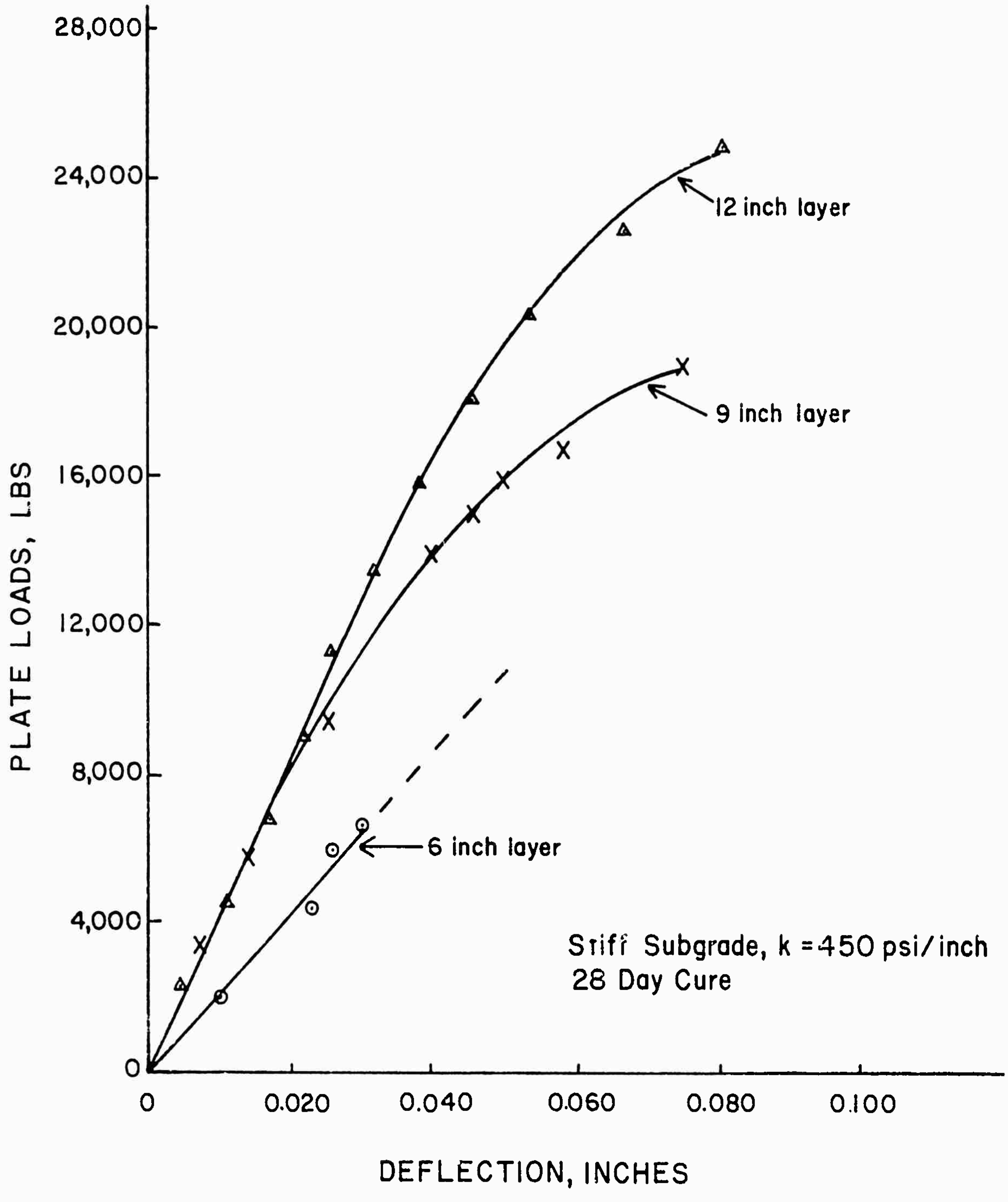


Figure 19. Influence of thickness responses for stiff subgrades.

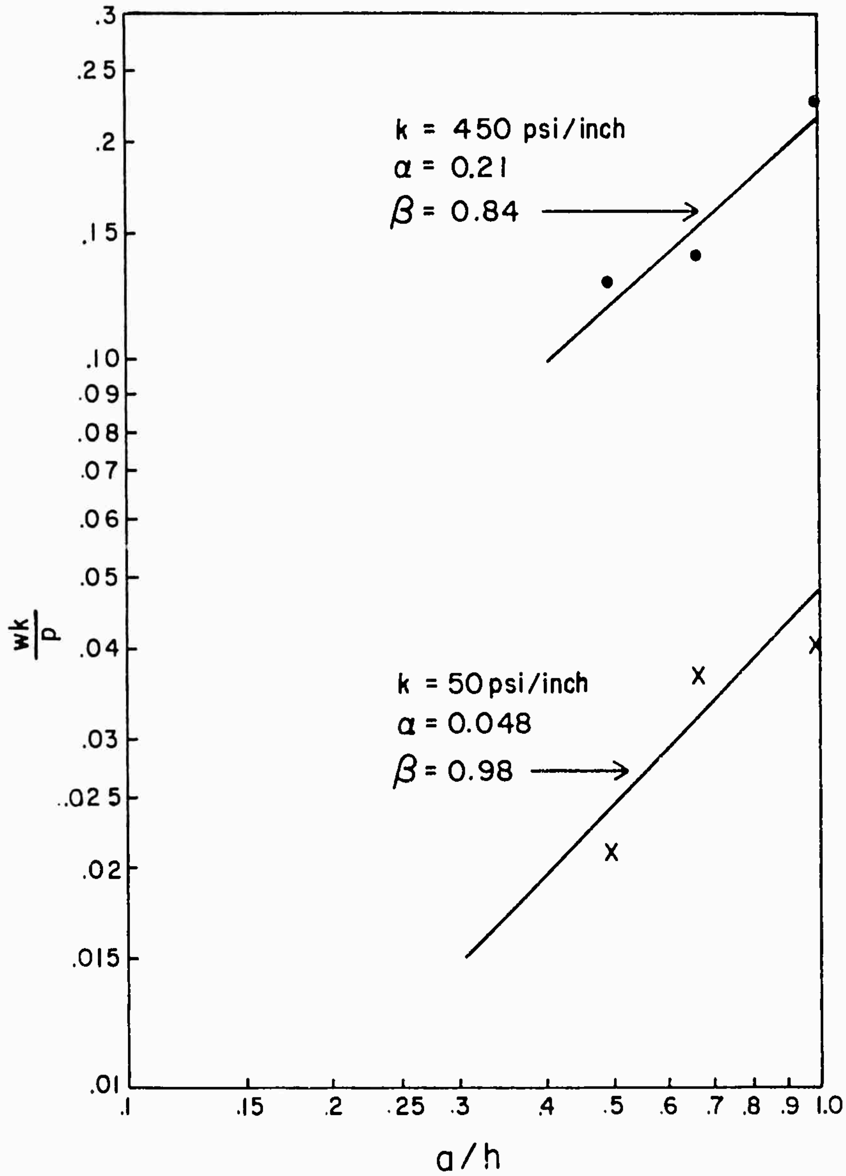


**Figure 20.** Log-log plots for the soft and stiff grades.

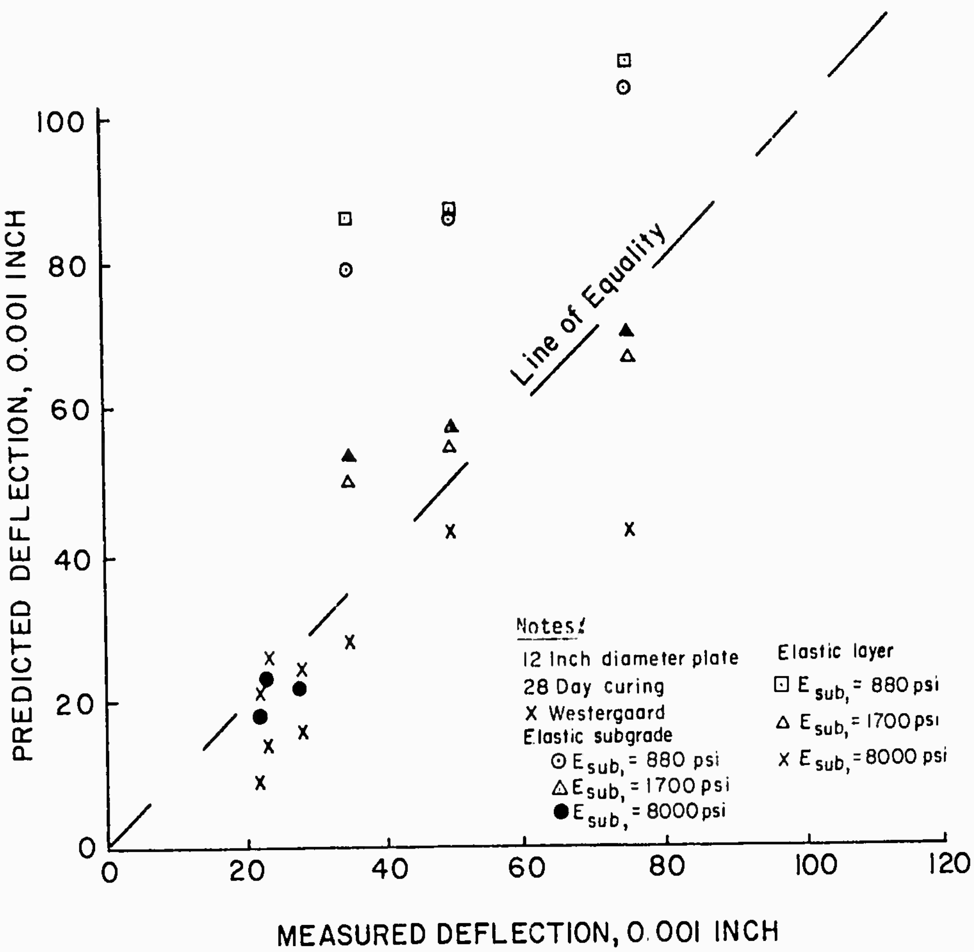


Figure 21. Prediction of surface deflection.

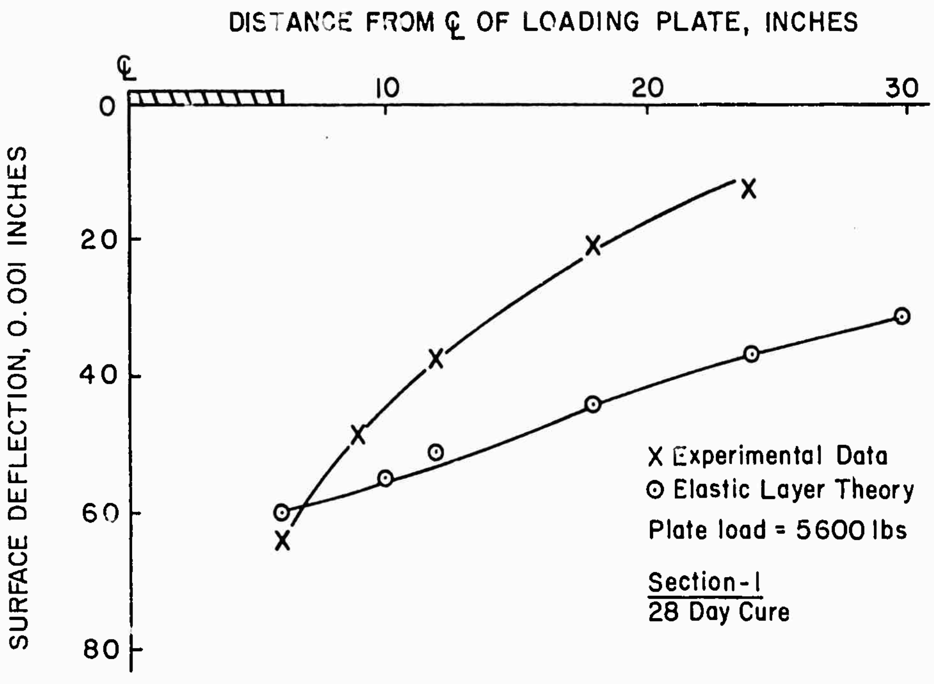


Figure 22. Surface deflection profile for section 1.

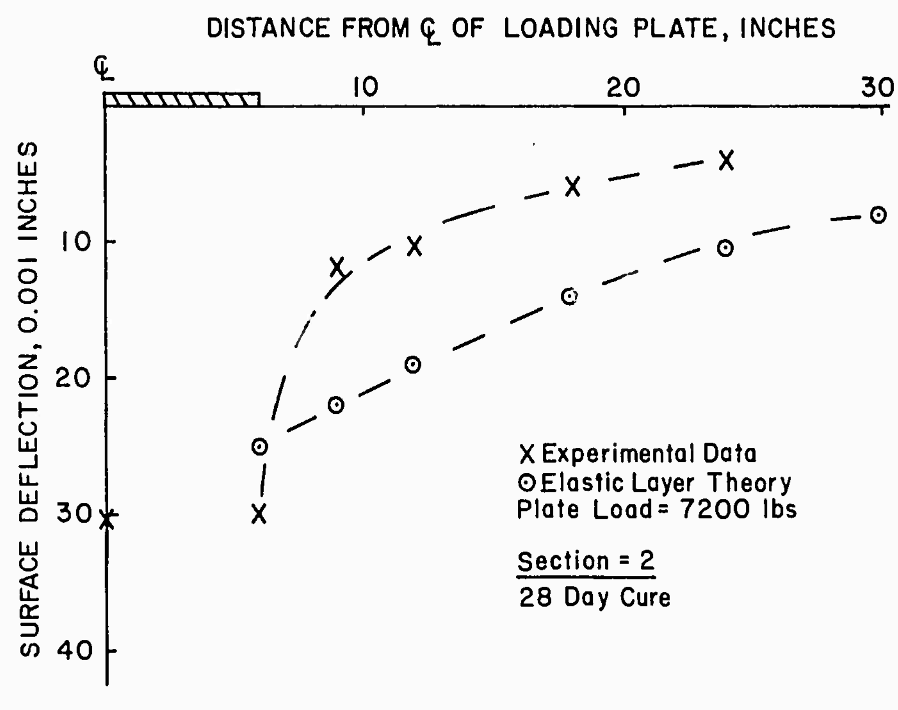


Figure 23. Surface deflection profile for section 2.

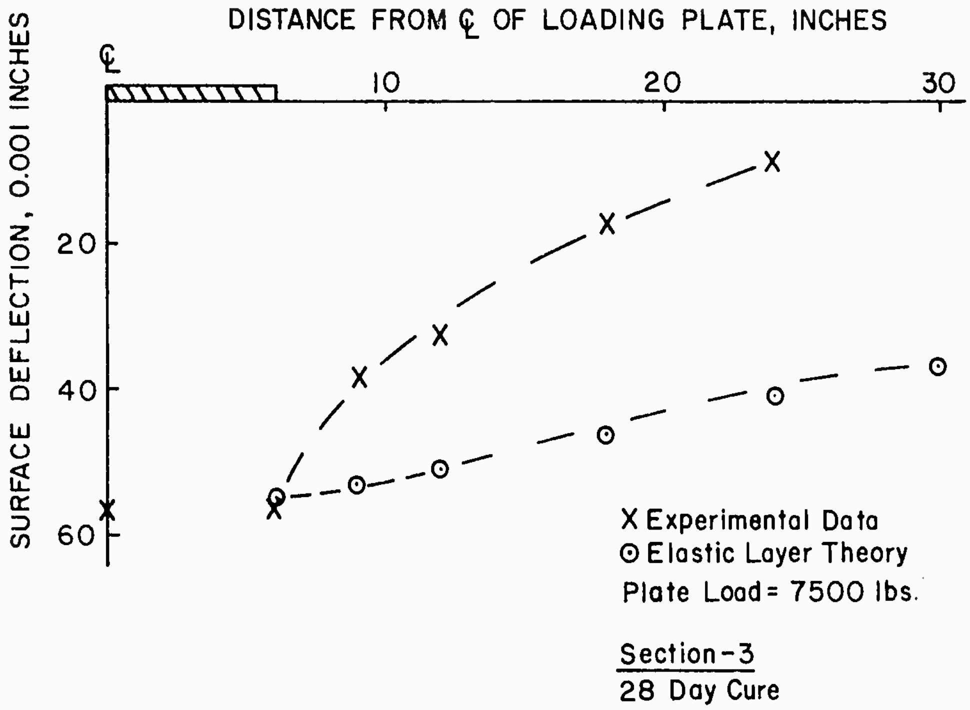


**Figure 24.** Surface deflection profile for section 3.

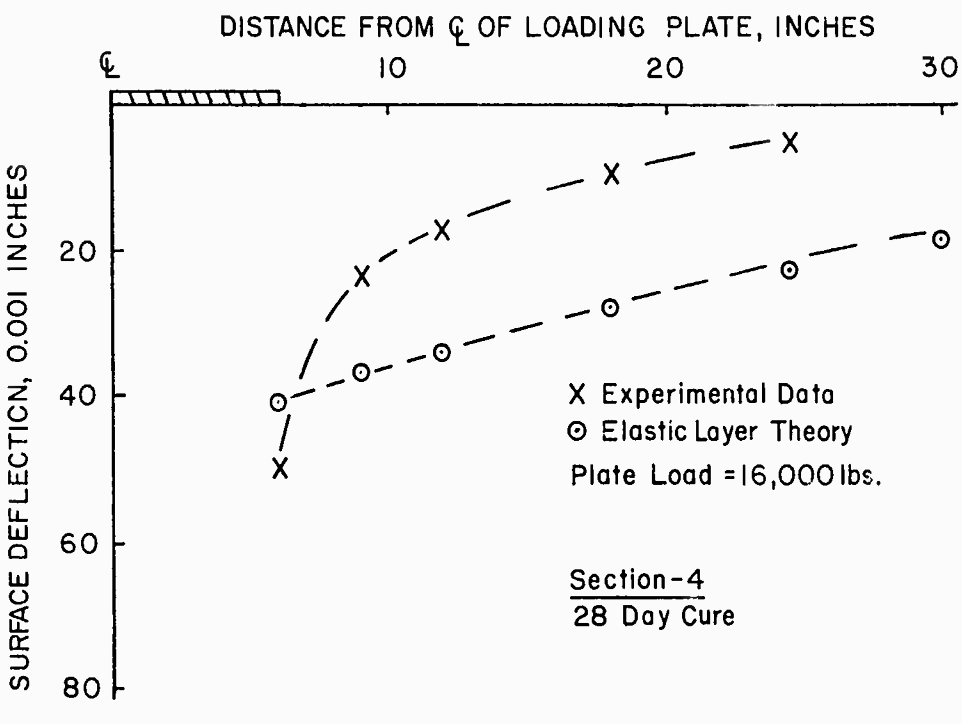


**Figure 25.** Surface deflection profile for section 4.

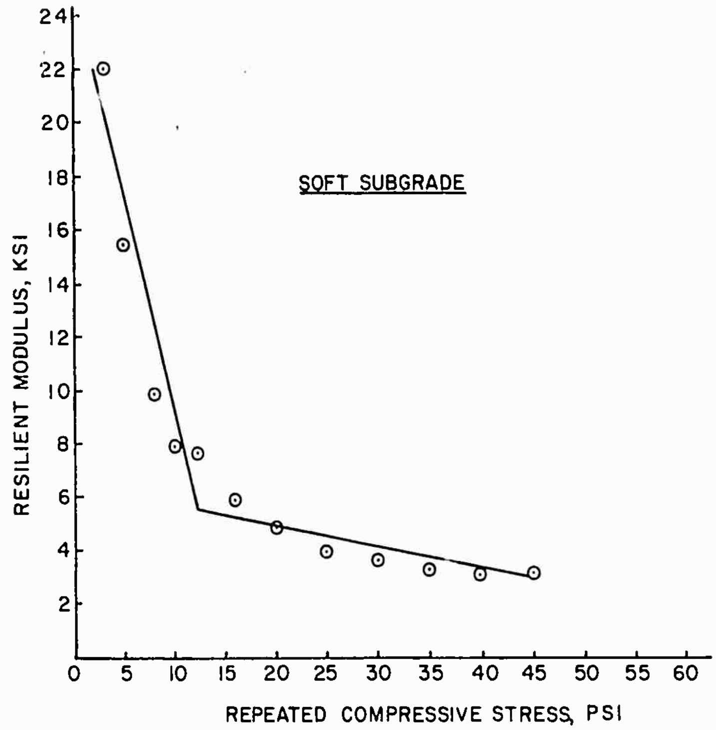


Figure 26. Resilient response—soft subgrade soil.

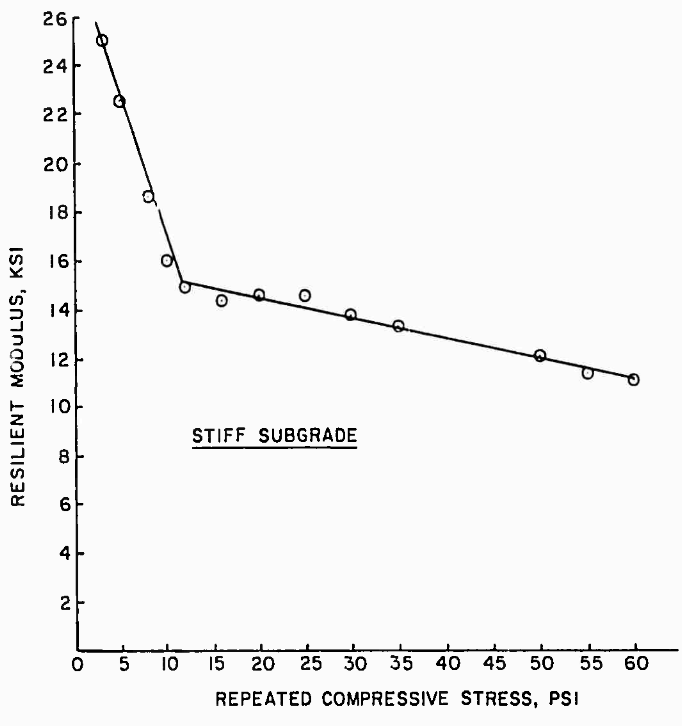


Figure 27. Resilient response—stiff subgrade soil.

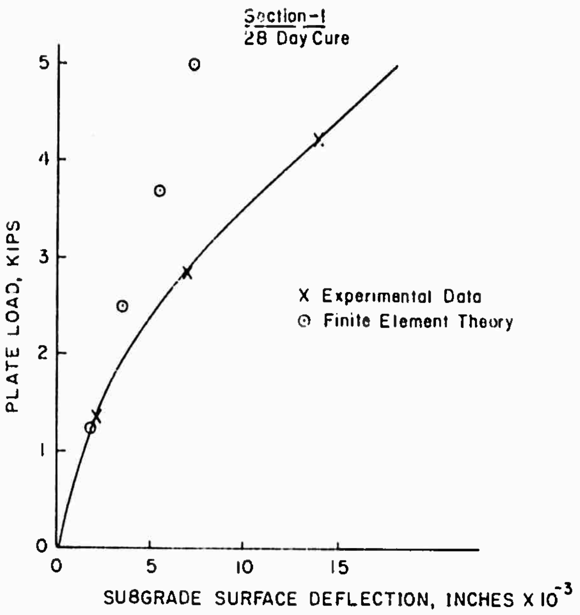


**Figure 28.** Experimental and theoretical subgrade surface deflections for section 1 (28-day cure).

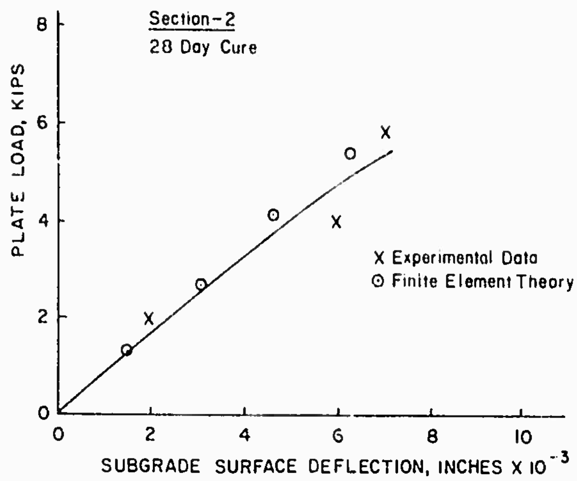


**Figure 29.** Experimental and theoretical subgrade surface deflections for section 2 (28-day cure).

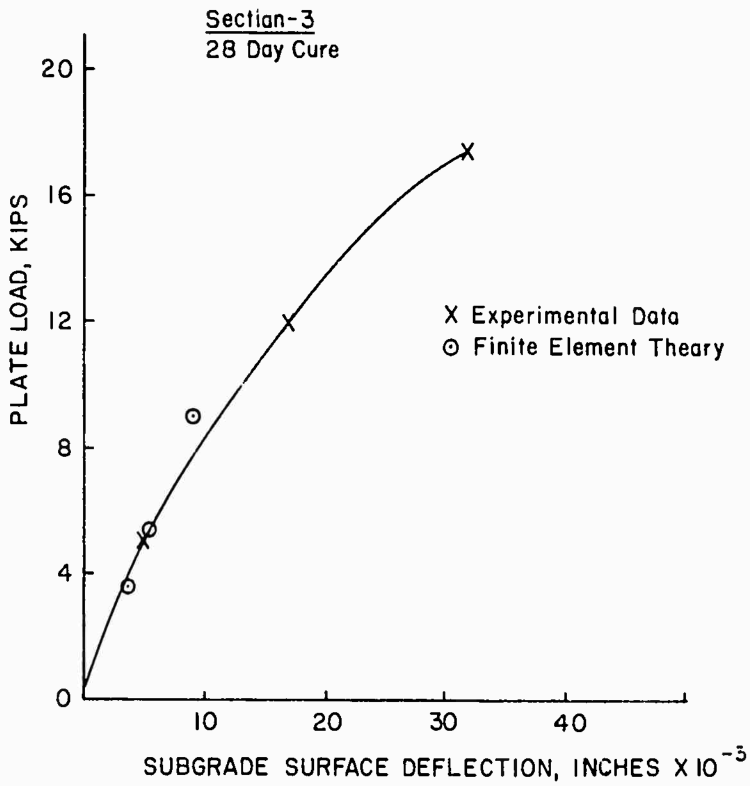


**Figure 30.** Experimental and theoretical subgrade surface deflections for section 3 (28-day cure).

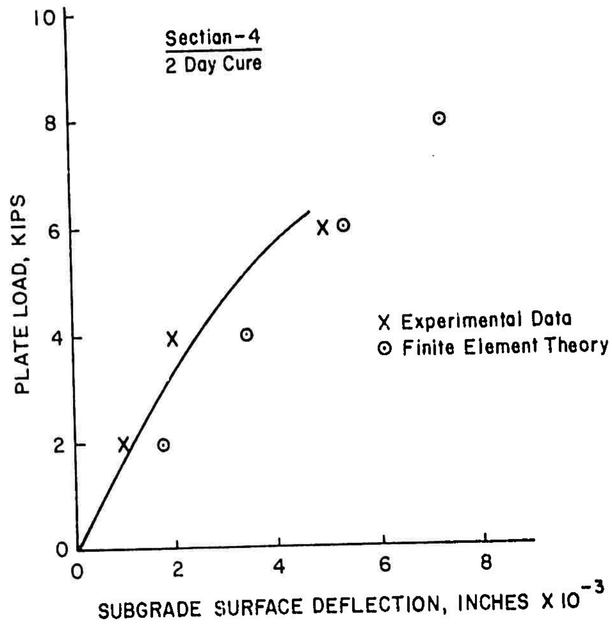


**Figure 31.** Experimental and theoretical subgrade surface deflections for section 4 (2-day cure).

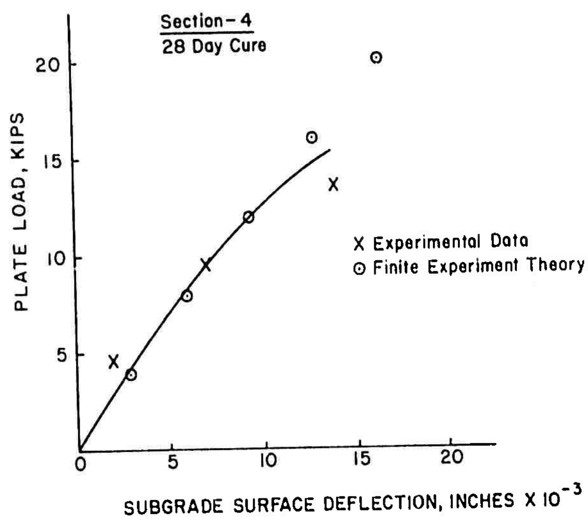


**Figure 32.** Experimental and theoretical subgrade surface deflections for section 4 (28-day cure).

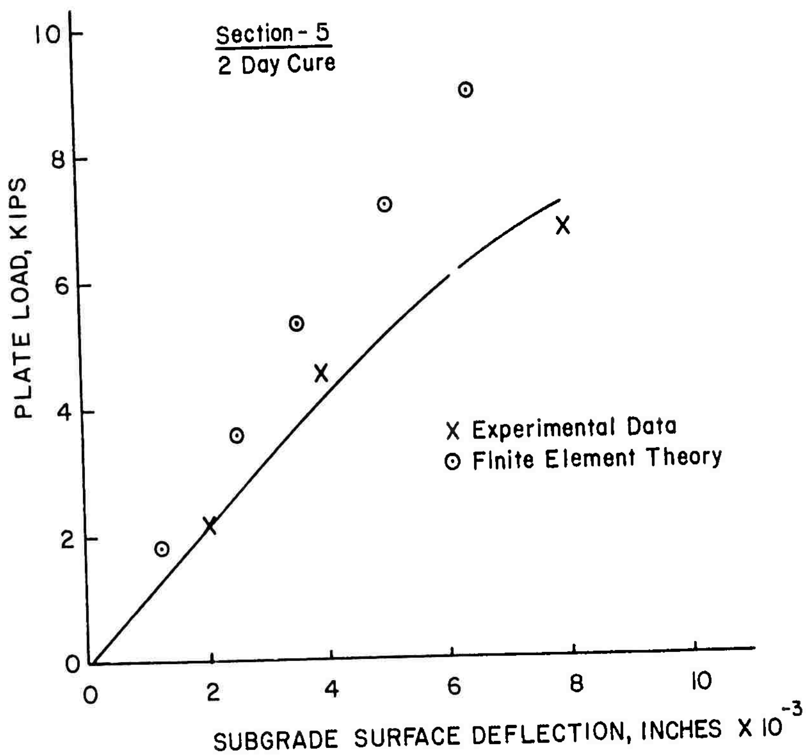


**Figure 33.** Experimental and theoretical subgrade surface deflections for section 5 (2-day cure).

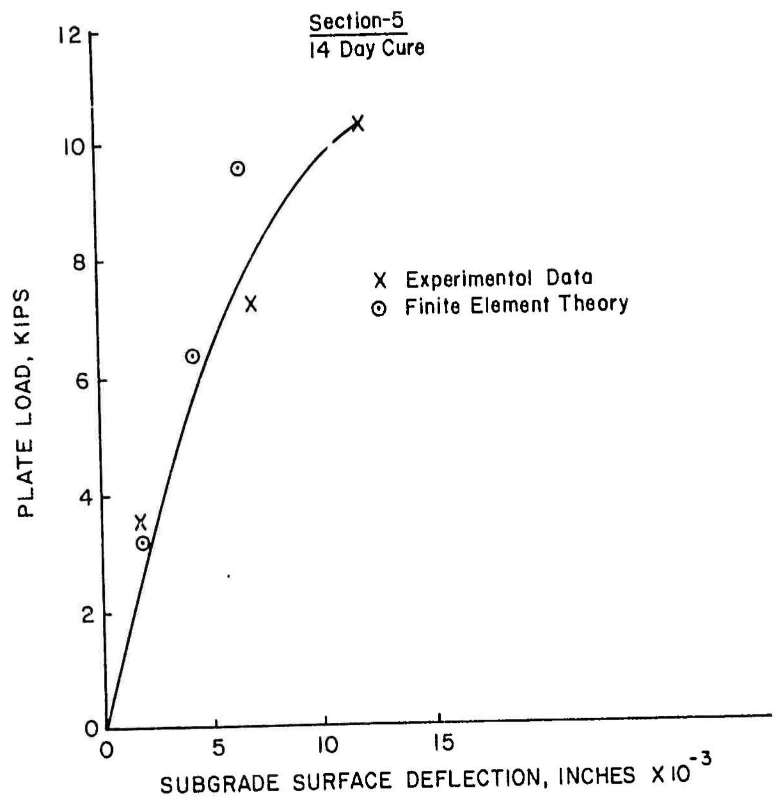


**Figure 34.** Experimental and theoretical subgrade surface deflections for section 5 (14-day cure).

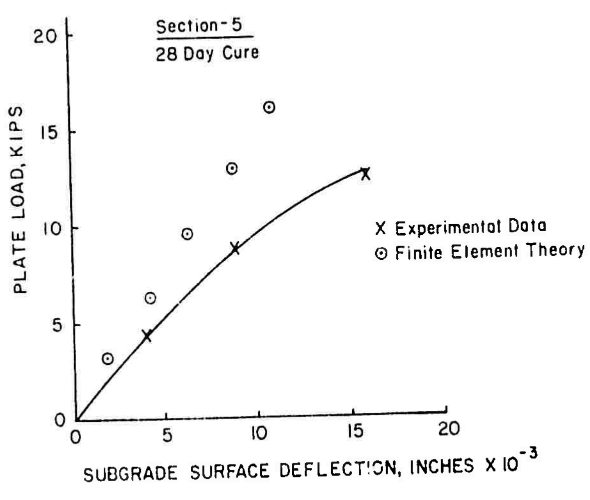


**Figure 35.** Experimental and theoretical subgrade surface deflections for section 5 (28-day cure).

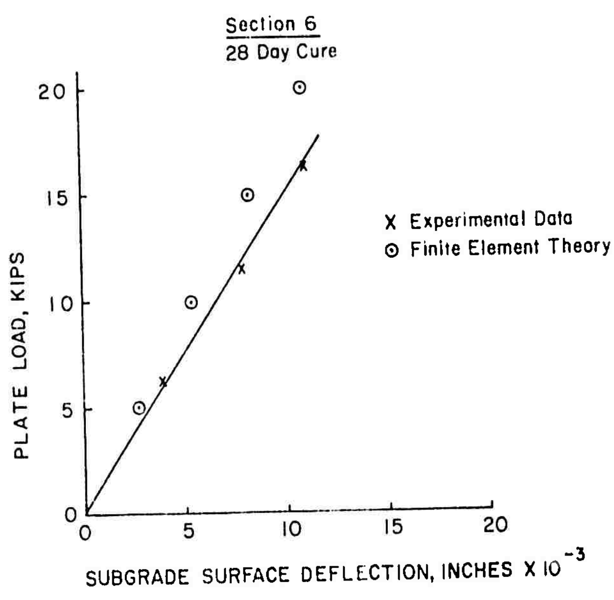


**Figure 36.** Experimental and theoretical subgrade surface deflections for section 6 (28-day cure).

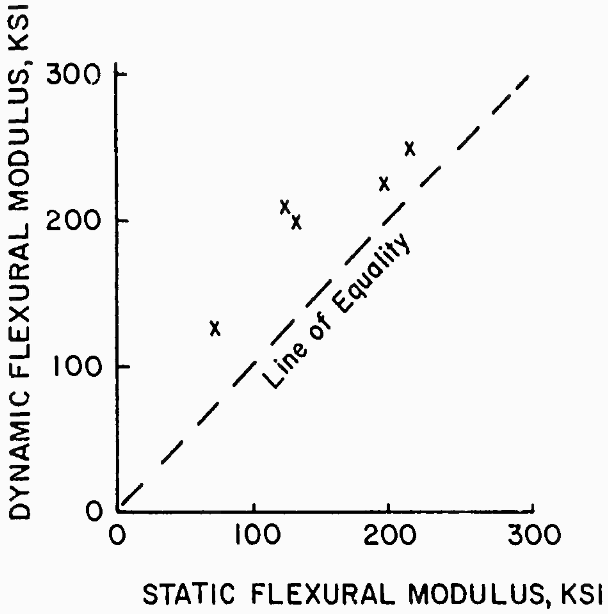


**Figure 37.** Flexural moduli from static and dynamic testing.

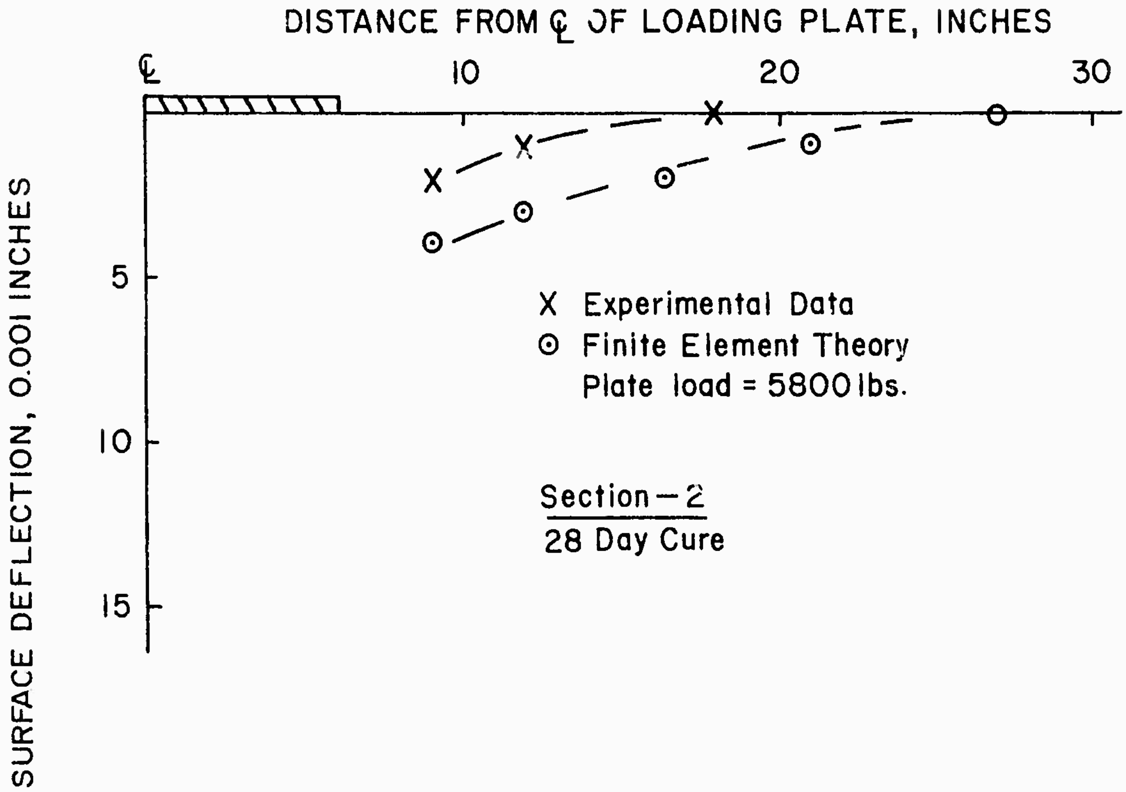


**Figure 38.** Dynamic surface deflection profile for section 2.

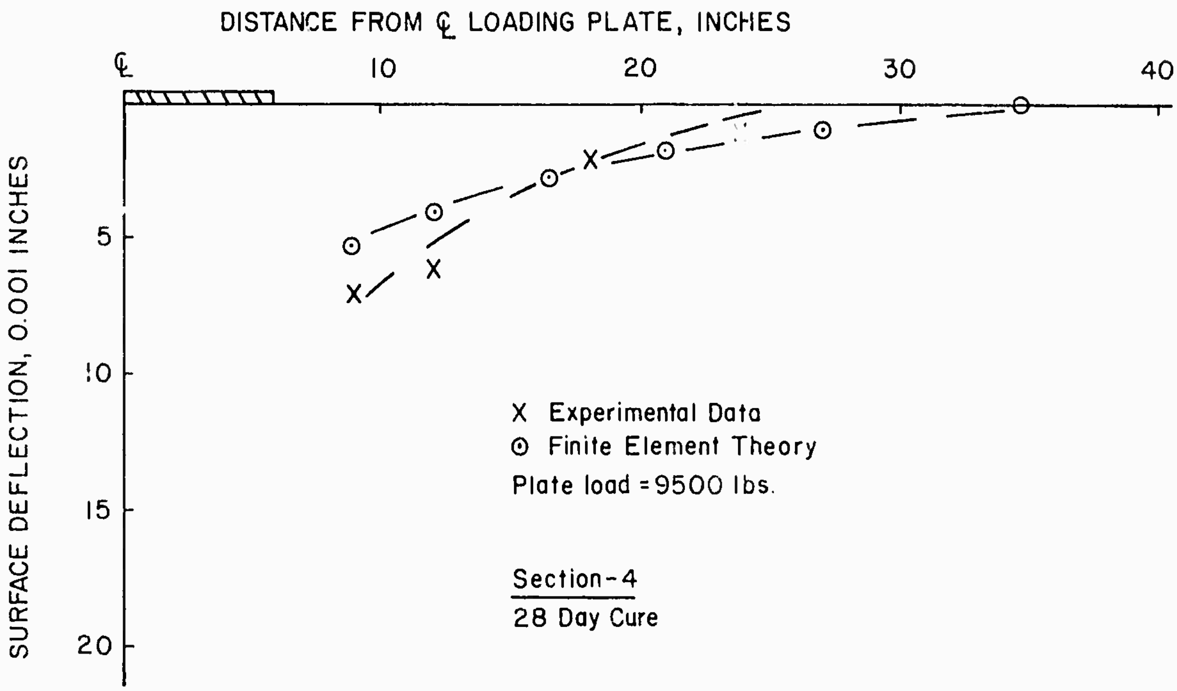


**Figure 39.** Dynamic surface deflection profile for section 4.

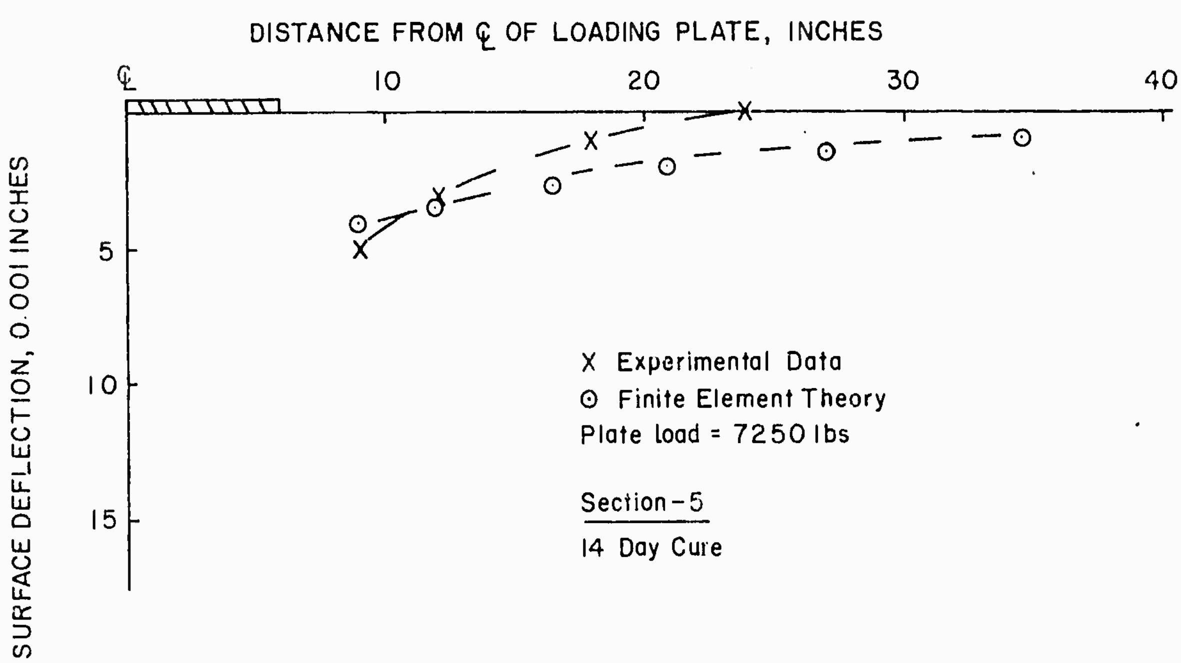


**Figure 40.** Dynamic surface deflection profile for section 5 (7250-lb plate load).

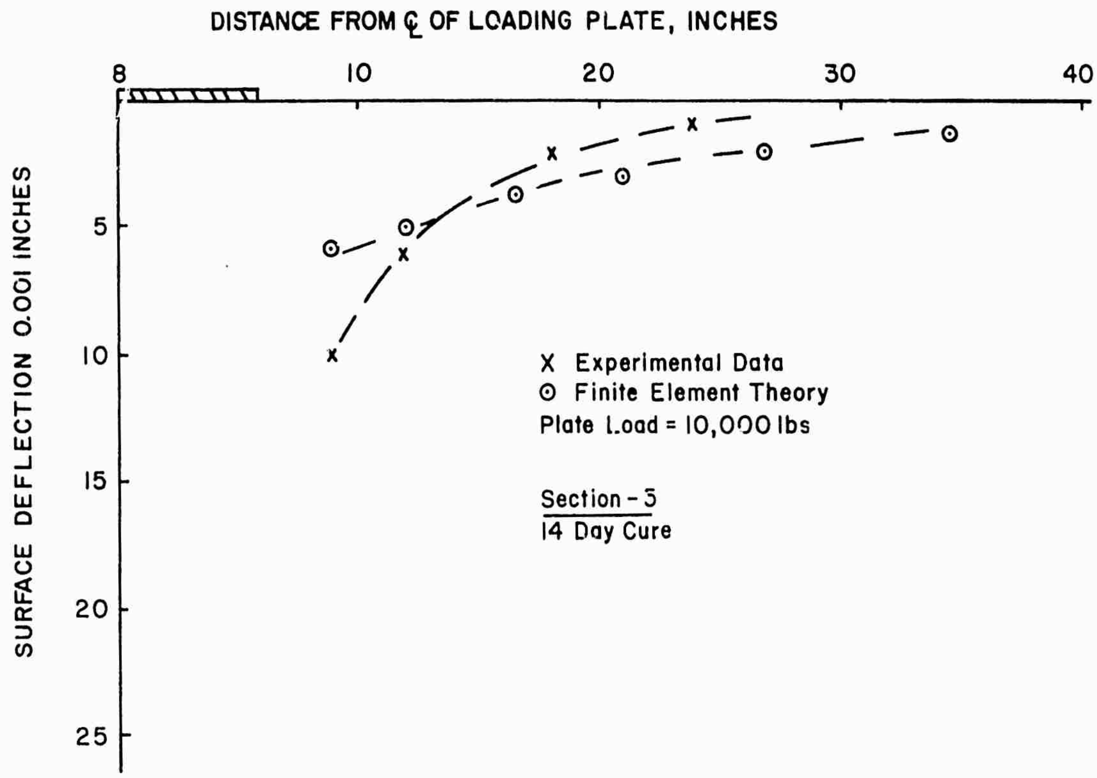


**Figure 41.** Dynamic surface deflection profile for section 5 (10,000-lb plate load).

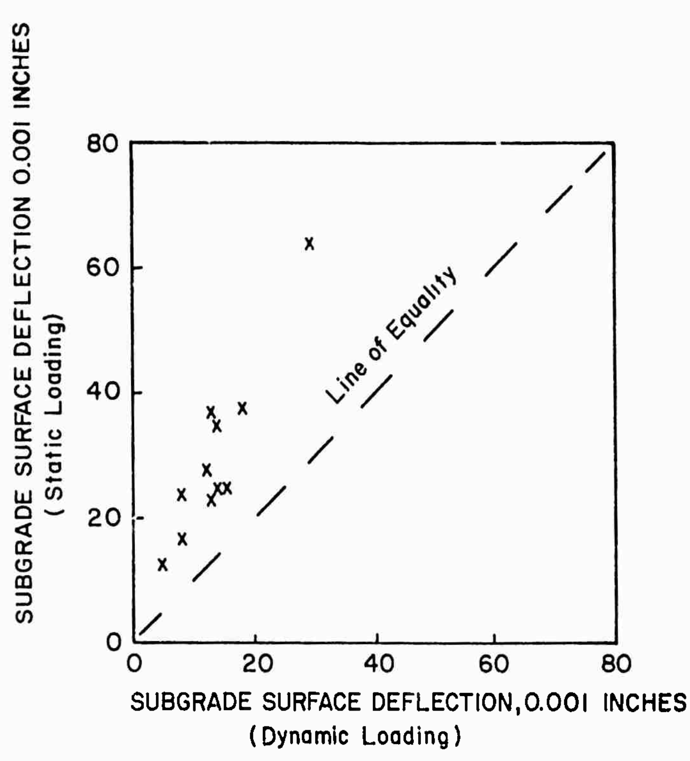


**Figure 42.** Comparison of static and dynamic subgrade surface deflection.

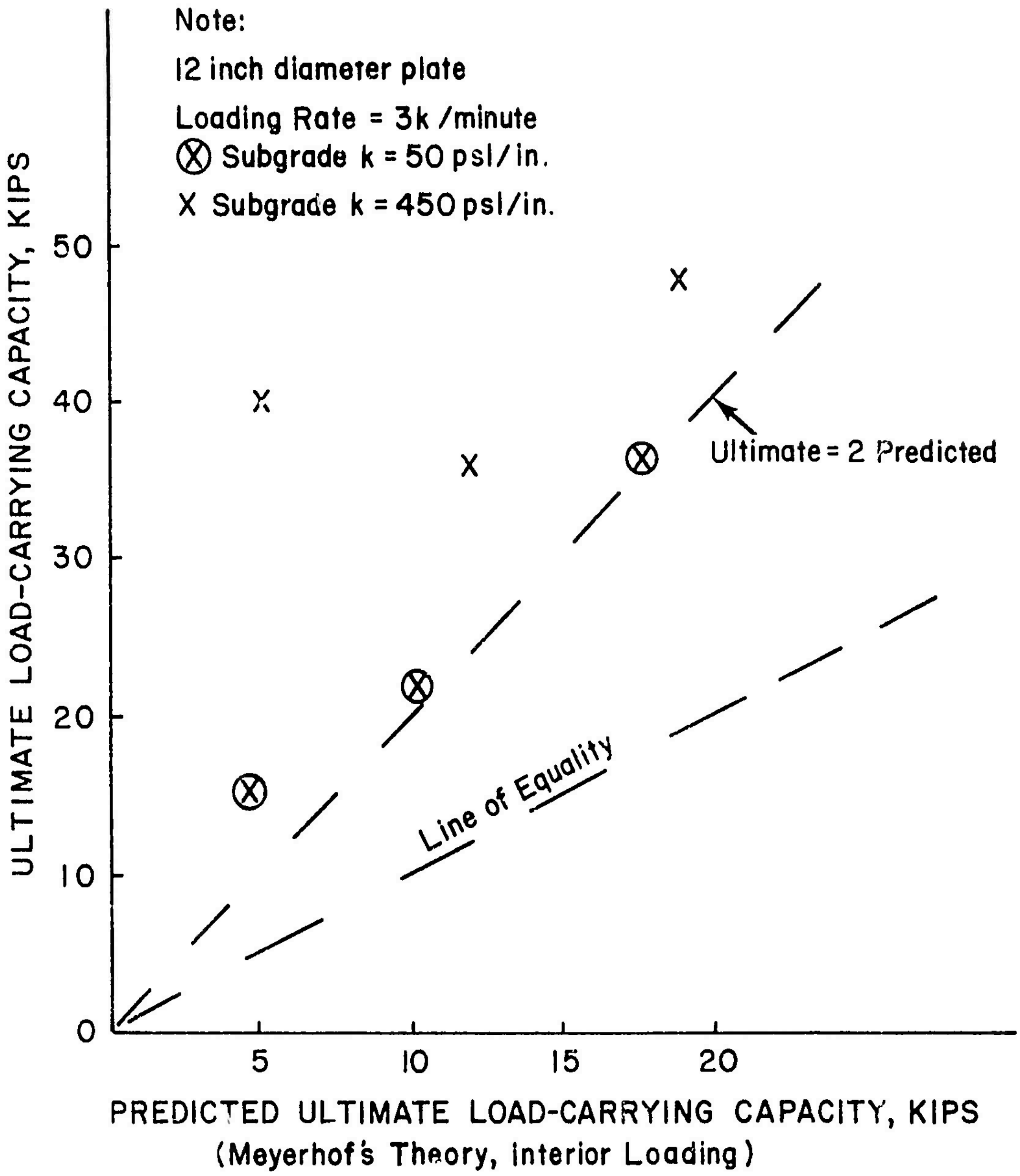


Figure 43. Comparison of predicted ultimate load-carrying capacity and plate-loading data.

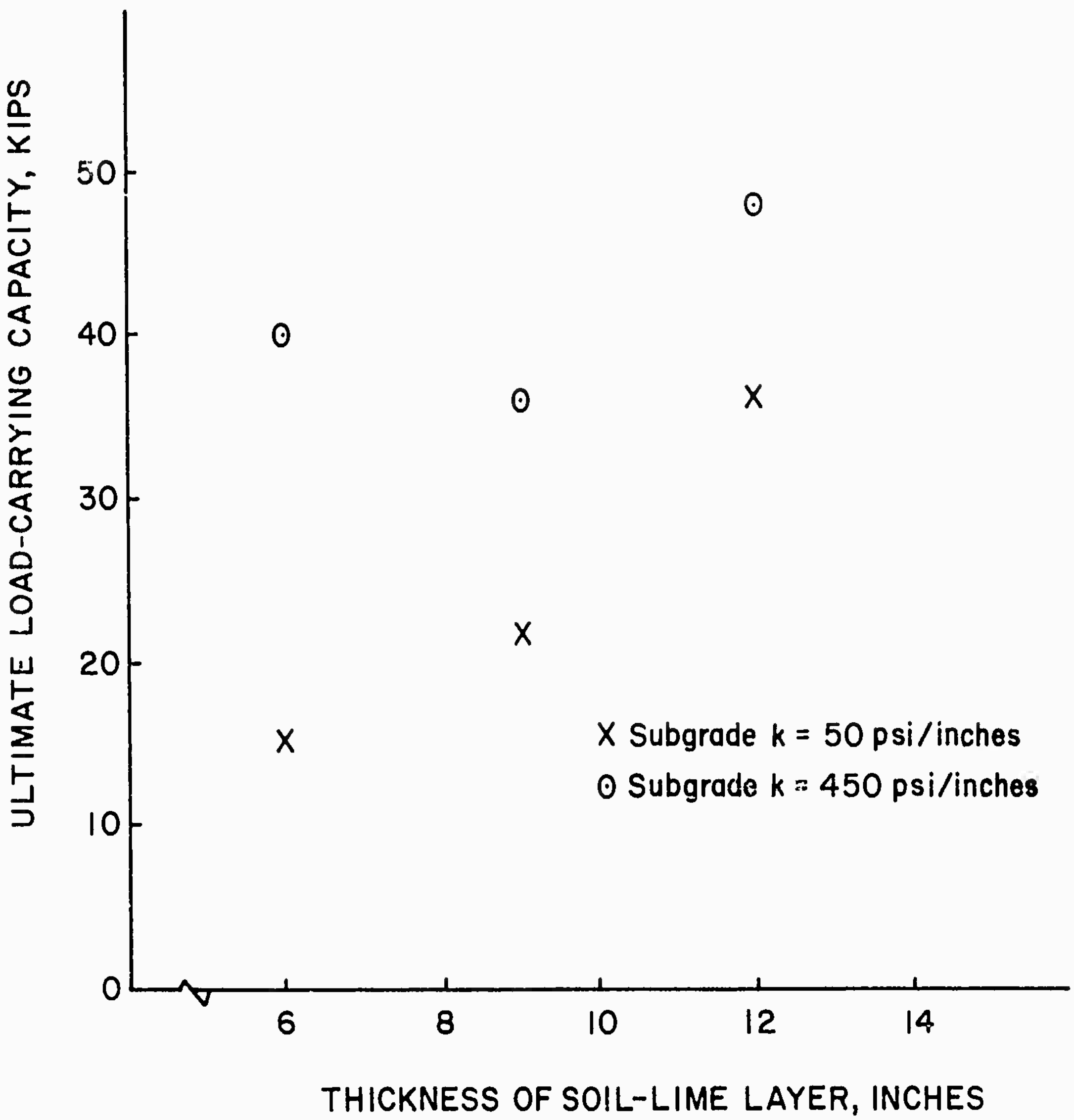


**Figure 44.** Effect of thickness for soft and stiff subgrades.

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