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16. Abstract

This report summarizes the evaluation of six experimental fly ash stabilized base and subgrade test sites on selected Texas Highways. The sites, each consisting of several test sections, were analyzed using the dual parametric approach on field measured Dynaflect and Benkleman Beam deflection data.

Five out of six sites exhibited significant increases in structural support capability with time. Strong relationships were found in variations of lime-fly ash percentages to changes in dual parametric results. Variations in fly ash percentages seemed to dominate the changes in parameters more than variations in lime content.

The study corroborates the viability of fly ash as a partial replacement for lime in stabilized bases and subgrades.

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EVALUATION OF FLY ASH TEST SITES
USING STATIC AND DYNAMIC
DEFLECTION SYSTEMS

by

Gary W. Raba

Dallas N. Little, P.E.

W.B. Ledbetter, P.E.

Shah M. Alam

Research Report 240-3
Research Study 2-9-79-240

Sponsored by

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Texas Transportation Institute
Texas A&M University
College Station, Texas 77843

FOREWORD

The information contained herein was developed on Research Study 2-9-79-240 titled "Fly Ash Experimental Projects" in a cooperative research program with the Texas State Department of Highways and Public Transportation and the U.S. Department of Transportation, Federal Highway Administration.

This report was taken from a Master of Science thesis by Gary W. Raba titled "Evaluation of Lime-Fly Ash Stabilized Bases and Subgrades Using Static and Dynamic Deflection Systems" (December 1982, Texas A&M University).

This is the third report on this study. The first two reports are:

240-1 "Analysis of Fly Ashes Produced in Texas" January 1981

240-2 "Construction of Fly Ash Test Sites and Guidelines for Construction" October 1981

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Acknowledgement is also extended to Koninklijke/Shell-Laboratorium, Amsterdam (Shell Research, N.V.) for the use of the BISTRO computer program.

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

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1. INTRODUCTION

1.1 Background

A pavement evaluation scheme that can accurately predict the structural integrity of a pavement system is highly desirable. This method should also provide the necessary results with a minimum of energy and capital expenditure. Utilizing the framework of multi-layered elastic theory and the magnitude of stresses, strains and displacements, and their respective distress criteria, pavement performance and structural integrity can be analyzed. However, these stresses and strains have the disadvantage of not being easily measured in the field.

Measurements of pavement surface deflection and curvature at the road surface can be considered in lieu of stress and strain measurements. The stiffness or component moduli, thickness, load intensities and the overall structural integrity of the pavement influence the deflection of a pavement system and its curvature under load.

The measurement of the load deflection response of a pavement has been shown to indicate pavement performance (1). More importantly it can be an effective tool for pavement analysis and evaluation. Also, by incorporating the layered elastic analysis in conjunction with the load deflection response, the individual pavement components can be classified according to stiffness or component moduli.

Figure 1 illustrates the general concept of a multi-layered elastic system. The analytical solution to the state of stress or

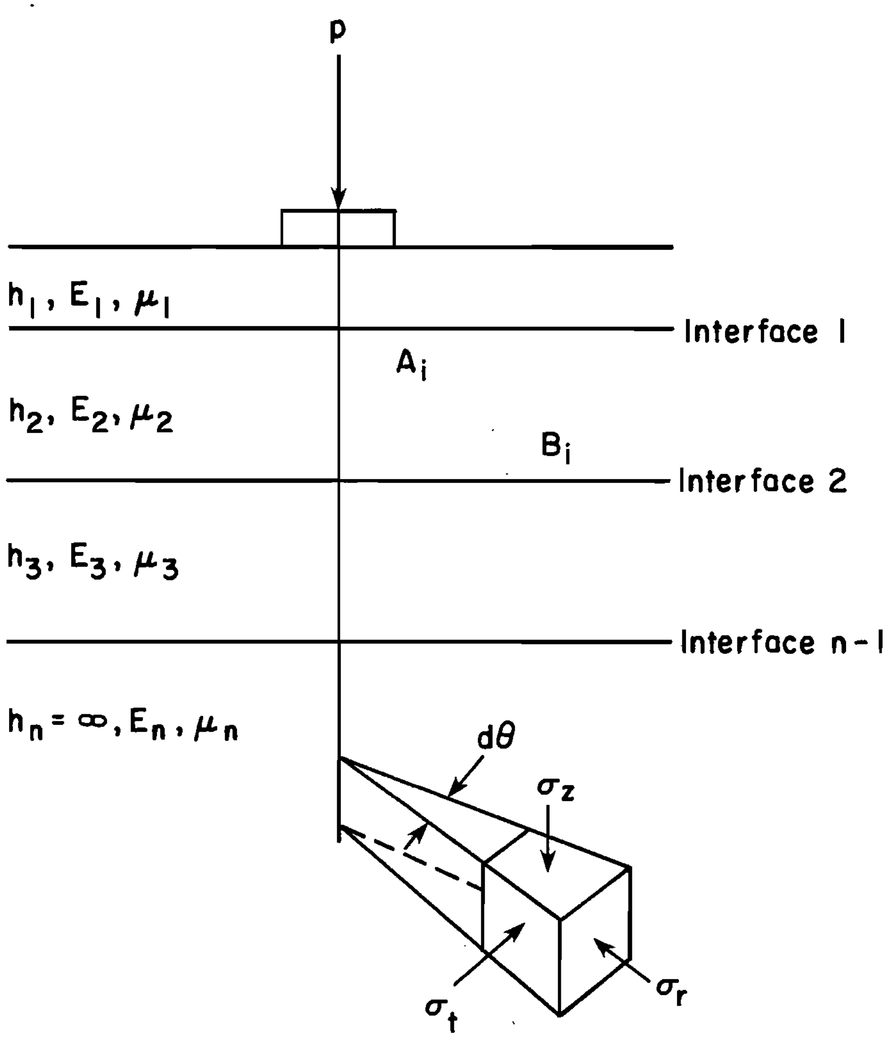


Figure 1. Generalized Multilayered Elastic System

strain has several assumptions. They are: (1) the material properties of each layer are homogeneous, that is, the property at point A_i, is the same as at point B_i; (2) each layer has a finite thickness except for the lower layer, and all are infinite in the lateral directions; (3) each layer is isotropic, that is, the property at a specific point such as A_i is the same in every direction or orientation; (4) full friction is developed between layers at each interface; (5) surface shearing forces are not present at the surface; and (6) the stress solutions are characterized by two material properties for each layer. They are poisson's ratio and elastic modulus, E. Although these items are the more classical assumptions used in most theoretical procedures, recent advances such as the computerized multi-layered Shell BISTRO and BISAR programs have the capability to analyze layered systems without interface friction mobilized and the presence of surface shearing forces.

The results of previous studies (2) have indicated that the maximum total deflection of a pavement system is ineffective in establishing pavement condition. However, by imposing a series of reference points near to and away from the loaded area a deflection profile can be outlined which allows for the calculation of deflection basin parameters.

1.2 OBJECTIVE

The primary objective of this study is to develop a method by which the relative benefit of partial replacement of lime by flyash in stabilized layers of flexible pavement systems maybe analyzed.

With the main objective are the following specific objectives:

1. Evaluate the structural support capability of specially constructed lime-flyash test sections utilizing Dynaflect and Benklemen Beam field measured data.
2. Determine by statistical analysis the effect of lime and flyash percentages on the stiffness of the stabilized layers.
3. Examine the effect of time on lime-flyash stabilized layers.

1.3 SCOPE

This research is continuation of a previous investigation concerned with the field construction of lime-flyash stabilized test sites (3). In this study six of the lime-flyash stabilized test sites located in Texas were analyzed.

1.4 TEST SITES

Test site 1 is located on FM 3378 in Bowie County. This project consists of 10 test sections which used lime-fly ash stabilization in the base layer. Each section consists of an eight inch, four percent lime subbase or a six percent fly ash stabilized subbase covered by an eight inch lime-fly ash stabilized base. The wearing surface is a one course bituminous surface treatment approximately 1/4 inch in thickness. The base course used varying amounts of lime and fly ash to stabilize a low to medium plasticity index (PI) tan, silty clay soil. The typical layout and cross-section are given in Figure A-1, Appendix A. Table 1 lists the soil properties, final stabilization dates and construction control data for each test section.

Test site 2, on US 59 outside of Carthage, Texas, in Panola County, consists of eight inches of lime-fly ash stabilized subgrade

Table 1. Lime-Fly Ash Stabilization Data for Test Site No. 1
(FM 3378 in Bowie County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final Passing No. 4 Sieve[a]	Moisture Content(%)[a]	Field Density (lb/ft^3)	Percent of Laboratory Density[b]
1	8/0	9/17/79	18	73	17	110	101
2	4/4	9/17/79	--	63	15	109	99
3	4/8	9/17/79	19	70	14	112	96
4	4/15	9/25/79	--	81	11	108	95
5	7/0	9/25/79	17	68	14	109	96
6	6/6	9/26/79	--	73	14	109	97
7	6/12	9/26/79	13	77	13	111	97
8	7/18	9/27/79	--	78	14	109	96
9	5/23	9/28/79	14	78	14	113	100
10	6/6	9/27/79	--	84	13	114	100

[a]Average of three or four measurements

[b]As determined by Test Method Tex-114-E

covered by a 12 inch flexible base. The wearing surface is a one course bituminous surface treatment. Ten sections of a low PI tan, silty clay (with sand) were stabilized using different amounts of lime and fly ash. Figure A-2 shows the typical cross section and layout planview of the test sections. Table 2 shows the soil properties along with the final stabilization dates and construction control data for each test section.

Test site 3, is located on the west bound lane of FM 1604 in Bexar County. This test site consists of six test sections which are composed of a six inch flexible base and a two course bituminous surface treatment. The subgrade is a tan, low PI clay silt. Test sections are approximately 800 feet long with a transition zone between each test section. This is depicted in Figure A-3. Figure A-3 also gives a typical section of this project. Table 3 lists all pertinent soil properties, the final construction control data for each test section and the date of construction.

Test site 4, located in Bexar County on FM 1604, contains six test sections. The subgrade is a low PI clay silt and is covered by a 14 inch flexible base. The wearing surface is a two course bituminous surface treatment approximately 1/2 inch in thickness.

Each test section is approximately 800 feet long as seen in Figure A-4, has a transition zone between each section. A typical section can also be seen in Figure A-4. Table 4 contains the actual percentages of lime and fly ash placed in the test sections along with the soil properties, the final dates of construction and the construction control data.

Test site 5 is located on FM 1604 in Bexar County between the

Table 2. Lime-Fly Ash Stabilization Data for Test Site No. 2
(US 59 in Panola County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final % Passing No. 4 Sieve[a]	Moisture Content(%)[a]	Field Density (lb/ft^3)	Percent of Laboratory Density[b]
1	4/0	7/10/79	15	67	11	111	97
2	2/4	7/12/79	--	78	10	112	98
3	2/10	7/20/79	7	89	8	111	97
4	4/8	7/23/79	--	84	8	112	99
5	4/16	7/24/79	18	70	14	108	93
6	0/16	8/02/79	--	74	10	112	98
7	2/24	8/09/79	6	82	10	111	95
8	2/15	8/14/79	--	90	10	111	95
9	0/21	8/16/79	--	80	9	106	92
10	4/4	8/18/79	19	76	12	111	95

[a] Average of three or four measurements

[b] Determined by Test Method Tex-114-E

Table 3. Lime-Fly Ash Stabilization Data for Test Site No.3 (FM 1604 in Bexar County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final % Passing No. 4 Sieve[a]	Moisture Content(%)[a]	Field Density[a] (lb/ft^3)[a]	Percent of Laboratory Density[b]
1	3/6	Dec. 1979	11	72	17	108	100
2	3/0	Dec. 1979	20	72	17	101	93
3	2/5	Dec. 1979	12	81	10	109	93
4	4/0	Dec. 1979	23	68	9	112	101
5	2/8	Dec.1979	22	64	8	113	100
6	0/12	Dec. 1979	5	61	7	116	97

[a]Average of three or four measurements

[b]Determined by Test Method Tex-114-E

Table 4. Lime-Fly Ash Stabilization Data for Test Site No. 4
(FM 1604 in Bexar County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final % Passing No. 4 Sieve[a]	Moisture Content[a] (%)	Field Density (lb/ft^3)[a]	Percent of Laboratory Density[b]
1	4/0	Aug. 1979	27	70	16	109	95
2	3/6	Oct. 1979	22	68	13	104	95
3	3/9	July 1979	20	68	23	104	95
4	0/10	July 1979	8	81	11	112	93
5	1/5	July 1979	15	71	12	105	93
6	2/8	Aug. 1979	18	69	20	103	100

[a] Average of three or four measurements

[b] Determined by Test Method Tex-114-E

San Antonio River and Elmendorf. Six inches of subgrade were stabilized with various lime-fly ash combinations. The treated subgrade consists of a tan, low PI clay silt material and is covered by a 12 inch flexible base. The wearing surface is a two course bituminous surface treatment. The test site is composed of six test sections each approximately 800 feet long. Also, a transition zone approximately 100 feet in length has been built between the sections. A typical section of the west bound lane of FM 1604 is shown in Figure A-5. A summarization of the construction control data and soils properties is given in Table 5.

Test site 8 located on SH 335 in Potter County is constructed of six lime-fly ash soil stabilization test sections. Figure A-6 shows the planview layout of the test sections. Each is approximately 800 feet long. The typical cross section is also illustrated in Figure A-6.

The test sections are composed of a six inch lime-fly ash treated medium to low PI clay covered by 12 inches of flexible base. A two course bituminous surface treatment serves as the wearing surface. Table 6 contains the construction control data the final dates of construction and the soil properties.

Specific construction procedures employed in this investigation are presented in TTI Research Report 240-2 (3).

Table 5. Lime-Fly Ash Stabilization Data for Test Site No. 5 (FM 1604 in Bexar County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final % Passing No. 4 Sieve[a]	Moisture Content(%)[a]	Field Density (lb/ft^3)[a]	Percent of Laboratory Density[b]
1	3/6	Apr. 1980	12	66	13	113	98
2	3/8	Apr. 1980	3	84	14	113	95
3	0/10	Apr. 1980	8	68	12	114	95
4	2/8	Apr. 1980	11	68	12	111	99
5	4/0	Mar. 1980	23	66	16	106	99
6	2/6	Apr. 1980	9	70	12	109	96
7[c]	0/25	--	--	--	--	--	--
8[c]	0/30	--	--	--	--	--	--

[a] Average of three or four measurements

[b] Determined by Test Method Tex-114-E

[c] Stabilization data for Section 7 and 8 not available

Table 6. Lime-Fly Ash Stabilization Data for Test Site No. 8 (SH 335 in Potter County)

Test Section	Lime/Fly Ash Percentage (% by wt.) Actual	Date of Construction	Plasticity Index	Final % Passing No. 4 Sieve[a]	Moisture Content (%)[a]	Field Density (lb/ft^3)[a]	Percent of Laboratory Density[b]
1	3/0	2/26/79	15	60	23	99	96
2	2/4	3/08/79	13	60	20	101	96
3	2/4	3/08/79	12	72	19	103	95
4	2/8	3/07/79	15	70	22	101	95
5	3/6	3/07/79	10	70	24	96	92
6	0/8	3/06/79	18	65	19	103	101

[a] Average of three or four measurements

[b] Determined by Test Method Tex-114-E

2. BACKGROUND

2.1 PAVEMENT DEFLECTION AND PERFORMANCE PARAMETERS

The magnitude of stress and strain within a pavement system has been shown to be representative of pavement performance and, as will be shown, can also be representative of the pavement component moduli.

Since stress and strain distributions within a layered system cannot be easily measured, the measurement of pavement deflection has been universally recognized as an indicator of the pavement's structural capacity.

In 1955, results from the WASHO Road Test established values of 45 and 35 mils as limiting values of allowable maximum deflection under an 18 kip axle for flexible pavements in spring and fall, respectively (4). Following the concept developed at the WASHO Road Test, many other investigators and agencies adopted and established their own limiting deflection criteria (5,6,7). Table 7 summarizes the literature.

However, maximum deflection is not the only indicator of a pavement structural capacity. The shape of the deflection basin, otherwise referred to as the 'deflection profile', can also be representative of the pavement's soundness. The shape of this deflection profile is also representative of a pavement system's load carrying capacity.

Using the measured deflection basin and the pavement curvature, the magnitude of tensile stresses and strains within the pavement system can be indirectly estimated. Previous studies (9-19) have shown that pavement curvature is, in fact, a measure of the structural adequacy of a flexible pavement system, where the tensile strains at

Table 7. Various Limiting Deflection Criteria for Use in Pavement Evaluation [after Hoffman (8)].

Reference	Deflection Criteria	Remarks
WAASHO	Spring $\Delta_{max} = 45$ mils Fall $\Delta_{max} = 35$ mils	Conventional flexible pavements. Deflections measured under 18 K axle.
Hveem	$\Delta_{all} \leq 50$ mils (1) $\Delta_{all} \leq 17$ mils (2)	(1) Surface treatment; (2) AC layer thickness = 4 in. Deflections under 15 K axle. Δ_{all} = allowable maximum deflection
Carneiro	20 mils $\leq \Delta_{max} \leq$ 35 mils	Conventional flexible pavements. Benkleman beam deflections under 18 K axle, 80 psi tire pressure.
Whiffin et al.	20 mils $\leq \Delta_{max} \leq$ 30 mils (1) 5 mils $\leq \Delta_{max} \leq$ 15 mils (2)	(1) Asphalt concrete over granular base. (2) Asphalt concrete over cement treated base. Traffic volume considered. Benkleman beam deflections under 14 K axle, 85 psi tire pressure.
State of California	$\Delta_{all} = f(T_{ac}, N)$	Δ_{all} = Allowable maximum deflection T_{ac} = Thickness of AC layer N = Number of repetitions of a 5 K EWL
Asphalt Institute	$\Delta_{all} = f(DTN, Temp)$	DTN = Design traffic number = average daily 18 K axle loads. Δ_{all} = Allowable maximum deflection (plus two standard deviations).

the bottom of the asphalt-bound layers are a function of the curvature.

Therefore, from a pavement evaluation standpoint, the measured differences between parameters can be used to indicate pavement condition. The pavement's component condition can be also be charted over a period of time. Instruments that measure only maximum deflection have the disadvantage of not being able to directly measure the relative stiffness of the subgrade layer. The deflection profile can be measured by instruments capable of evaluating surface deflections at various points from the load. One excellent deflection measurement device is the Dynaflect. The Dynaflect equipment provides a measure of pavement deflection as well as an indication of the deflection basin's shape. Both can then be translated into parameters describing in-situ moduli of the subgrade and the effective thickness of the overlying pavement layer or layers.

The maximum deflection and deflection basin characteristics are also dependent upon such environmental variables as temperature, moisture and freeze-thaw conditions. Due to the thin bituminous surface course treatments in this study no temperature correction factor was found necessary.

Applied load is also one of the most important performance variances which affect the maximum deflection and deflection profile of the pavement system (20). The increase in pavement deflection due to an increase in the magnitude of the load is influenced by the stress dependency of both subgrade and base elastic moduli.

The relationship between load and deflection can be assumed to be approximately linear for flexible pavement systems, provided the moduli or component layers are assumed to be constant, i.e., stress independent (21). However, for a more realistic analysis, it can be shown that the soil and base support moduli are not constant, but are dependent upon the stress levels applied. It has been shown that the subgrade modulus is dependent upon deviatoric stresses as given by:

$$M_R = K_1 \cdot \sigma_d^n \qquad (1)$$

Where M_R is the resilient modulus of the material,

K_1 is a material constant,

n is a material constant < 1.0 and

σ_d is the deviatoric stress which is equal to $\sigma_1 - \sigma_3$

The variations of deviatoric stresses under a typical pavement subjected to Dynaflect loading are shown in Figure 2.

The moduli of granular base course layers are also dependent upon the stress level. The modulus of resilience of a granular layer is represented by:

$$M_R = K_1' \theta^m \qquad (2)$$

Where M_R is the modulus of resilience,

K_1' and m are material constants and

θ represents the sum of the principle stresses $\sigma_1 + \sigma_3$

In laboratory testing conducted by Monismith et al. (22) the modulus increased considerably with the confining pressure. So long as shear does not occur, the modulus can be approximated by equation (2).

They also found that the resilient properties of granular bases were also affected to a lesser degree by factors such as aggregate density, aggregate gradation (percent passing No. 200 sieve), aggregate type and degree of saturation. At a given stress level, the modulus

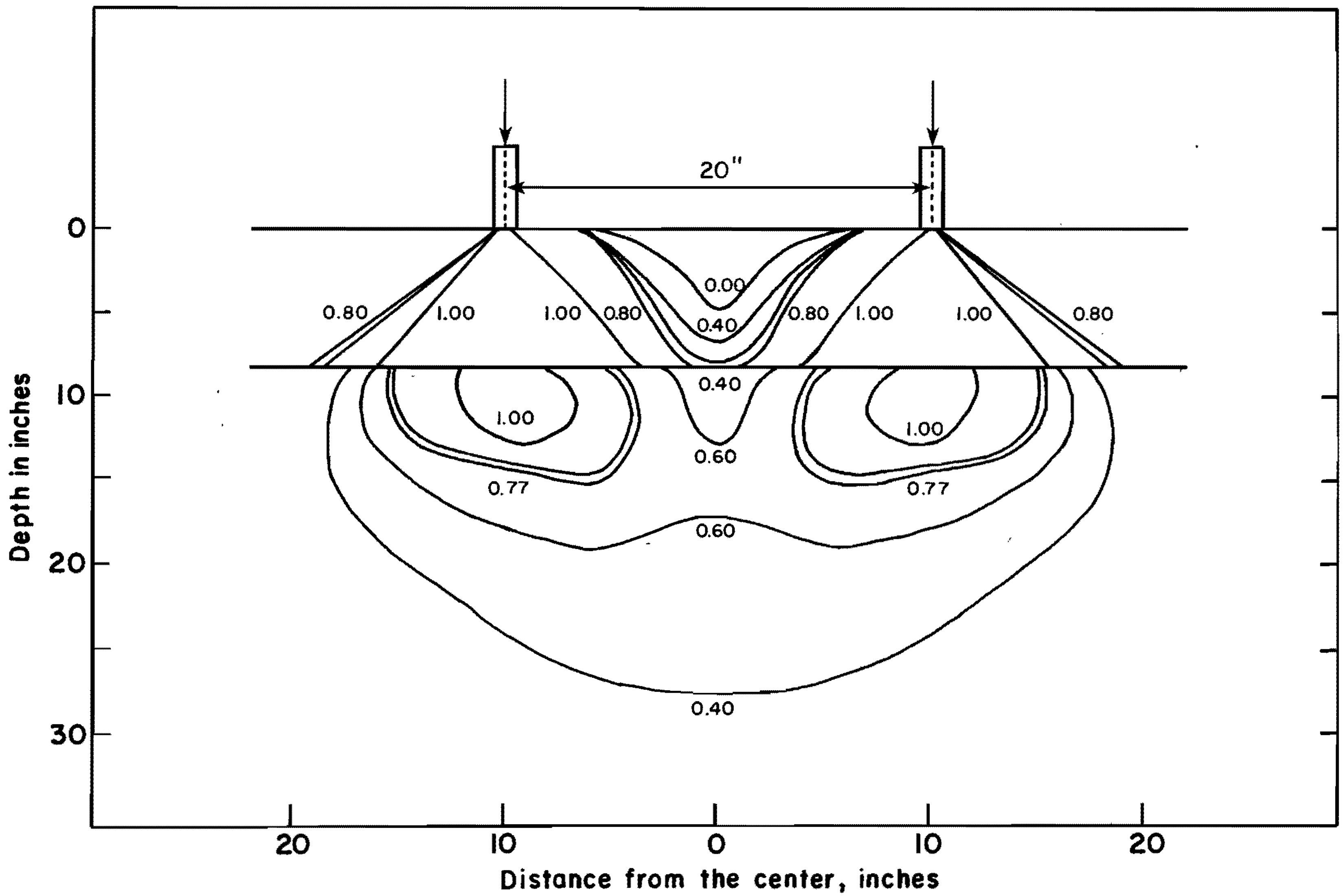


Figure 2. Deviatoric stress contours within a flexible pavement [after Majidzadeh (21)].

increased with increasing density, increasing particle angularity or surface roughness, decreasing fines content and decreasing degree of saturation.

Since the state of stress within the pavement layer is dependent upon the thickness of the pavement structure, deflections and deflection profiles are also affected by the pavement thickness. According to field observations and AASHO Road Test data, pavement deflection is inversely related to thickness H, as given by:

$$d = A\,(1/H)^n \qquad (3)$$

Where d is deflection and

A and n are material constants (n normally ranges from 0.11 to 0.65).

These deflection parameters are also dependent upon the base course thickness which will influence the overall stiffness of the pavement structure. The effect of the base course thickness is more pronounced for pavements constructed on a poor subgrade.

For relatively thicker pavements, the stresses transmitted to the subgrade are small and the soil supporting medium responds as if it has a greater modulus of resilience. Consequently, thicker pavements have lower deflections. The base course, on the other hand, exhibits a lower modulus, with a resultant increase in deflection. The deviatoric stress effect on the subgrade is much larger for thin pavements. The subgrade soil will respond to the load with a much smaller modulus (21). The granular base in this case responds as a material with a higher modulus and will exhibit lower deflections. The calculated subgrade modulus in such structures can be substantially overestimated (23). The opposite is true, however, for granular base courses where the modulus increases with an increase in stress state.

In general, it can be stated that pavement deflection and curvature under load is greatly dependent on the moduli of the pavement layers. In pavement structures approximated by a one-layer system, the deflection is inversely related to the layer modulus. In a multi-layered structure the interrelation of deflection and curvature is somewhat more complex.

Assuming a pavement structure with stress dependent material properties, the interrelation between curvature and deflection is given by the equation:

$$W = \frac{qa}{E_s} F_w \tag{4}$$

Where W is deflection,

q is the load intensity,

a is the radius of the loaded area,

E_s is the subgrade modulus and

F_w is the deflection factor (for one-layer system, $F_w = 1.0$).

The radius of curvature is:

$$(1/R) = \frac{a}{4E_s} F_c \tag{5}$$

Where F_c is the curvature factor and is related to the ratio of E_1/E_2 as well as pavement thickness, H.

Although most asphalt pavement structures cannot be regarded as being homogeneous, the use of equations (3) through (5) are generally applicable for subgrade stress, strain and deflection studies when the modular ratio of the pavement and subgrade is close to unity (20).

this condition is probably best exemplified by conventional flexible granular base/subbase pavement structures having a thin asphaltic concrete surface course.

Normally, when deflection studies for this pavement type are conducted, it is assumed that the pavement portion (above the subgrade) does not contribute any partial deflection component to the total surface deflection. Thus the significant deflection occurs in the subgrade and the one-layer theory applies as follows:

$$t = p + s = s \qquad (6)$$

Where t is the total surface deflection,

p is the deflection within the pavement layer and

s is the deflection within the subgrade.

Generally, as the modular ratio of pavement support ($E_{pavement}/E_{subgrade}$) increases, the load spreadibility increases and shear stresses decrease. Similarly, as the ratio increases, the magnitude of vertical stresses and strains in the subgrade decrease.

The above theoretical conclusions are based on the assumption that the pavement, base and subgrade moduli are stress independent. In reality, the resulting relationships are more complex due to the stress dependency of the pavement component moduli.

The complexity of these interrelations are further compounded by the effects of the geometrical and boundry conditions, such as joints, cracks and physical discontinuities.

2.2 Methodology for Deflection Analysis

2.2.1 Introduction A review of recent literature indicates that there are three distinctive approaches for pavement deflection analysis:

(1) maximum deflection,

(2) deflection basin parameters and

(3) multi-parametric analysis.

2.2.2 Maximum Deflection The maximum deflection approach is used almost entirely as an evaluation of a pavement system's load carrying capability. The interaction between maximum design deflections, allowable 18 kip axle load repetitions and pavement thickness is shown in Figures 3 and 4. Utah (24) uses the effects of seasonal variation on the maximum deflection measurements in their pavement rehabilitation program.

The variability of the maximum deflection as presented by Majidzadeh (21) can be taken into account. By assigning a suitable level of assurance the design deflection parameter can be written as:

$$(W + 2\sigma) \times f \times c$$

Where: W is the average maximum deflection

σ is the standard deviation of the measurement and

f and c are adjustment factors for environment, testing period and seasonal conditions.

However, the maximum deflection does not accurately depict the different performance or strength characteristics of the various layers. Furthermore, the mechanisms associated with pavement failure or the variations associated with the deflection profile are not reflected.

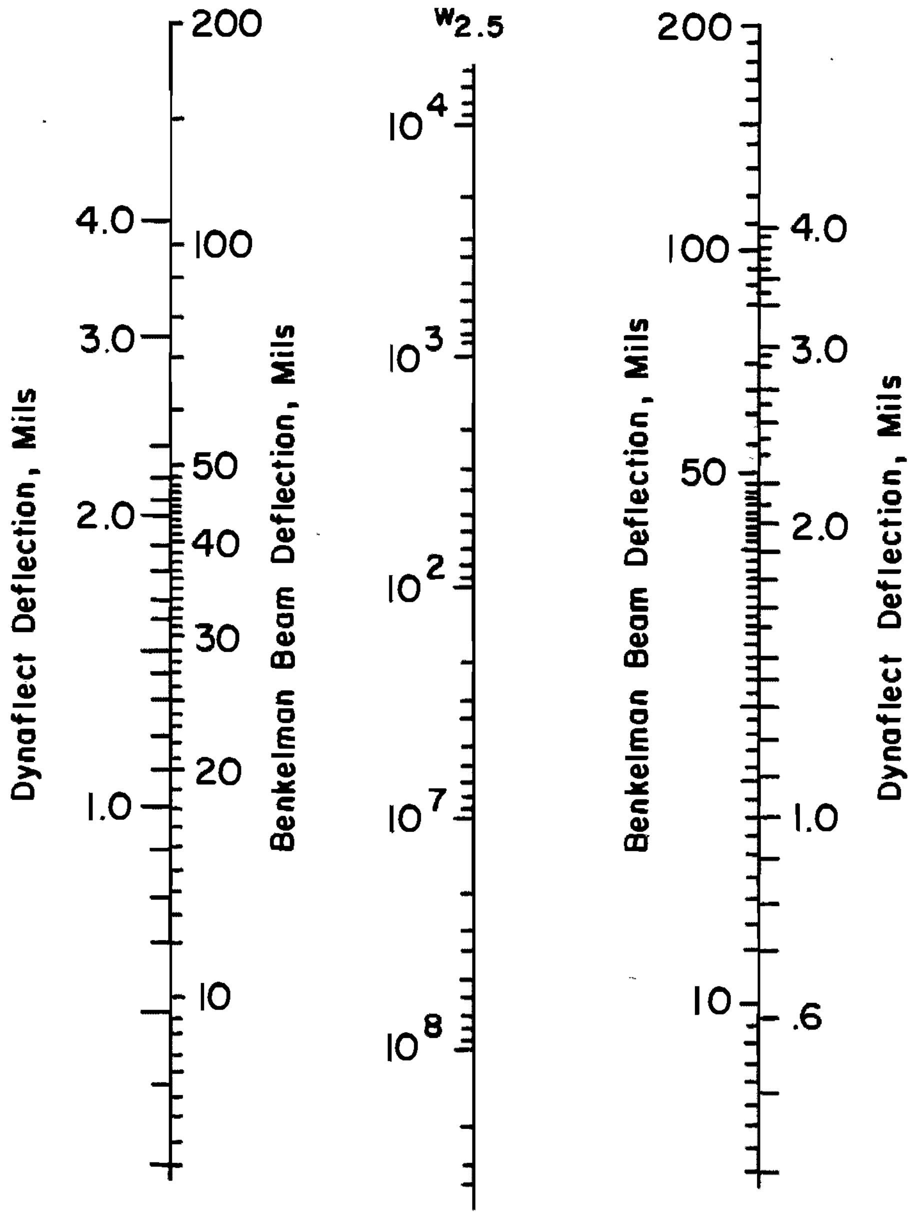


Figure 3. Deflection vs. Life Expectancy (AASHO Road Test) [after Majidzadeh (20)].

$W_{2.5}$= Number of 18 kip axle loads before serviceability level of pavement drops to 2.5 for spring deflections.

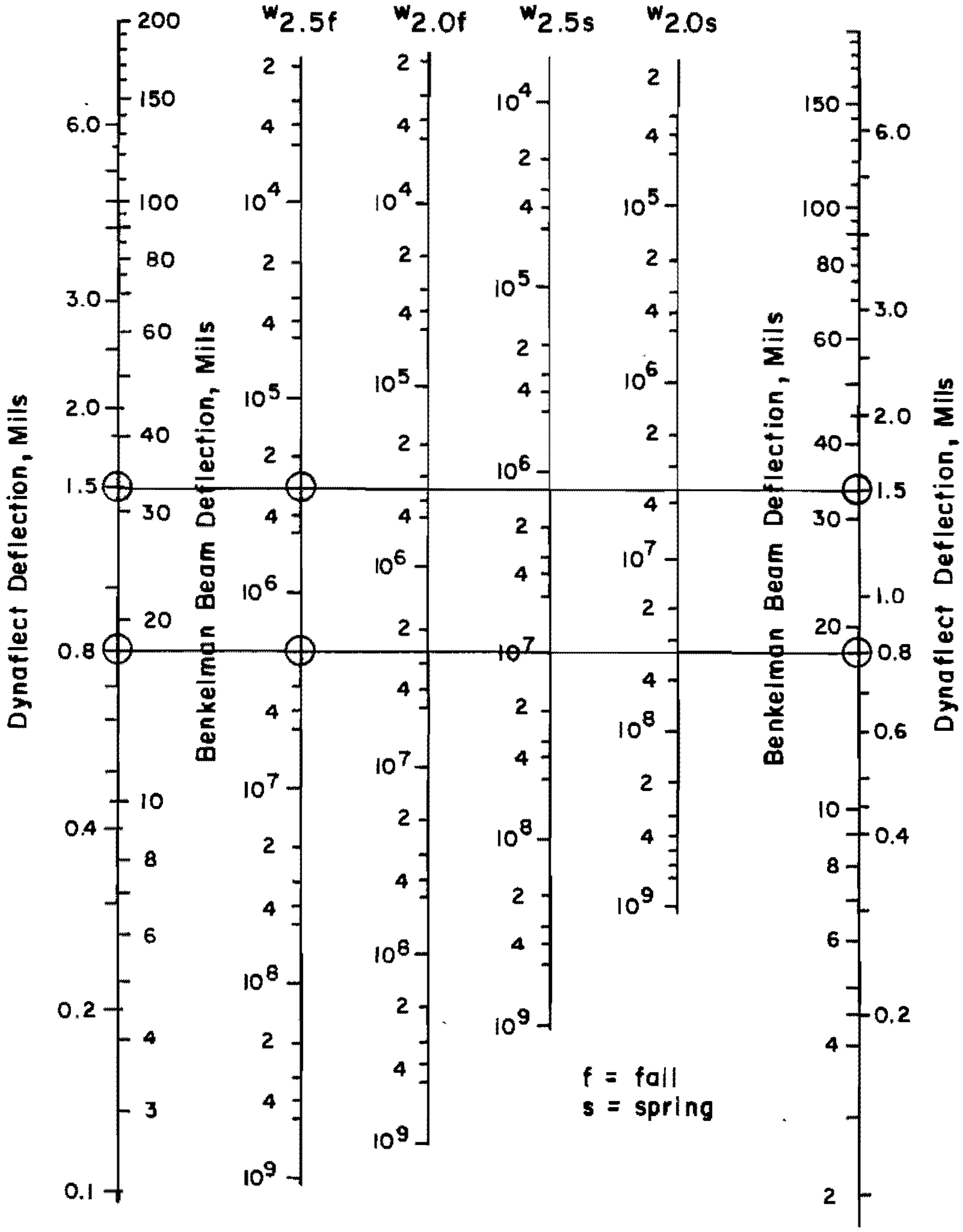
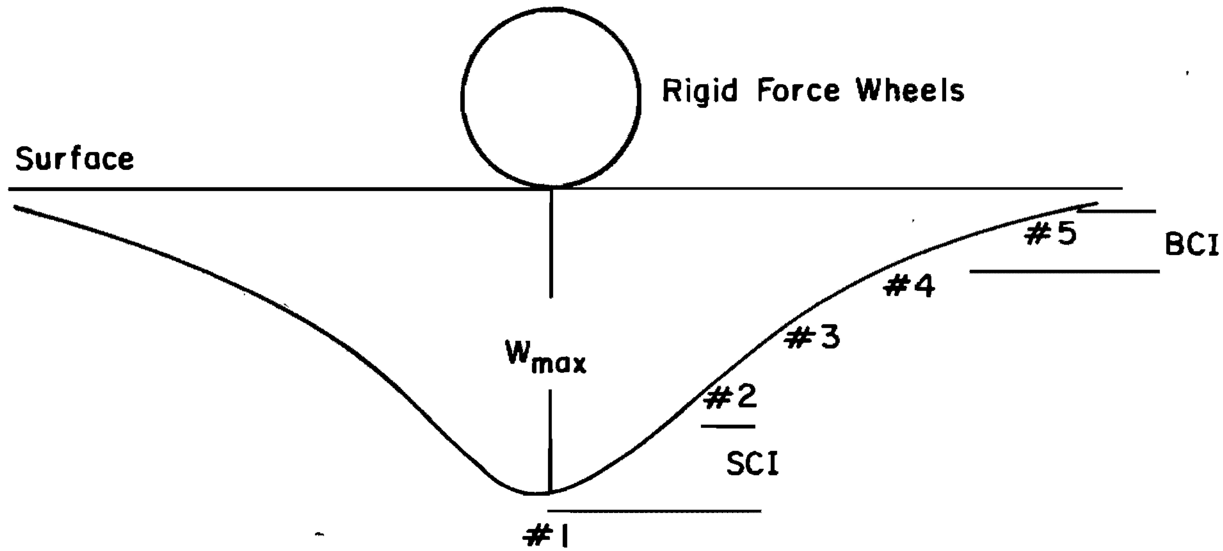


Figure 4. Terminal 18 kip axle loadings and deflections [after Majidzadeh (20)].

2.2.3 Deflection Basin Parameters The Dynaflect deflection basin and associated parameters are depicted in Figure 5. Typically most highway departments use these parameters with their respective criterion for a pavement condition evaluation. This involves comparing the maximum deflection, surface curvature index and base curvature index against tabulated maximum values as shown in Table 8. The particular values shown are those used by the Utah State Department of Highways. According to the Utah recommendations, any base curvature index (BCI) values greater than 0.11 indicate a poor subgrade support condition.

It has been stated that pavement performance analyses have indicated the surface curvature index (SCI) in inversely proportional to the radius of curvature and therefore is a measure of tensile strains in the pavement (23).

The sensitivity of the SCI parameter to changes in modular ratio (E_1/E_2) and the pavement thickness is of particular interest. For relatively thick pavements, SCI decreases with an increase in the modular ratio. For thin pavements it can be shown that the SCI and E_1/E_2 relationship is dependent upon the magnitude of thickness H. In Figures 6 and 7 the relationship between SCI, H and modular ratio is presented. For relatively thick pavements, SCI decreases with an increase in the modular ratio whereas for thin pavements, the SCI and E_1/E_2 relation depends mostly on the magnitude of thickness H. Taking into account the variations of SCI with H and pavement moduli, an increase in thickness reduces the SCI parameter.



S = Spreadability

W_1 = Dynaflect maximum deflection (numerical value of sensor #1)

SCI = surface curvature index (numerical difference of sensors #1 and #2).

BCI = base curvature index (numerical difference of sensors #4 and #5)

$$S = \frac{W_1 + W_2 + W_3 + W_4 + W_5}{5\,W_1} \times 100$$

Figure 5. Deflection basin parametrs as presented by Utah State Department of Highways [after Majidzadeh (21)].

Table 8. Dynaflect Deflection Criteria-- Utah State Department of Highways [after Majidzadeh (23)].

DMD	SCI	BCI	CONDITION OF PAVEMENT STRUCTURE
G.T. 1.25	G.T. 0.48	G.T. 0.11	PAVEMENT AND SUBGRADE WEAK
		L.E. 0.11	SUBGRADE STRONG, PAVEMENT WEAK
	L.E. 0.48	G.T. 0.11	SUBGRADE WEAK, PAVEMENT MARGINAL
		L.E. 0.11	DMD HIGH, STRUCTURE OK
L.E. 1.25	G.T. 0.48	G.T. 0.11	STRUCTURE MARGINAL, DMD OK
		L.E. 0.11	PAVEMENT WEAK, DMD OK
	L.E. 0.48	G.T. 0.11	SUBGRADE WEAK, DMD
		L.E. 0.11	PAVEMENT AND SUBGRADE STRONG

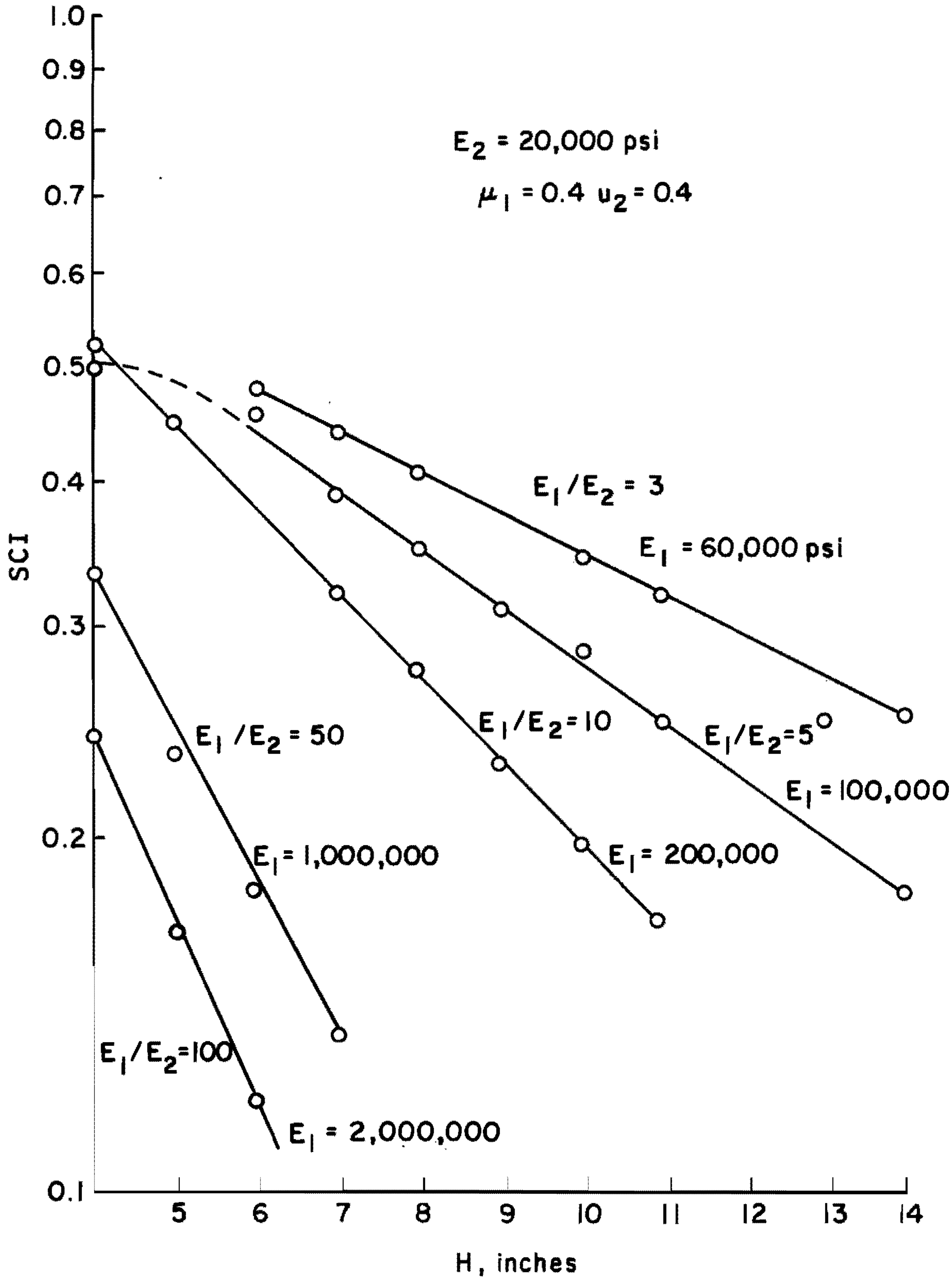


Figure 6. Surface Curvature Index - Thickness relation for various pavement systems [after Majidzadeh (23)].

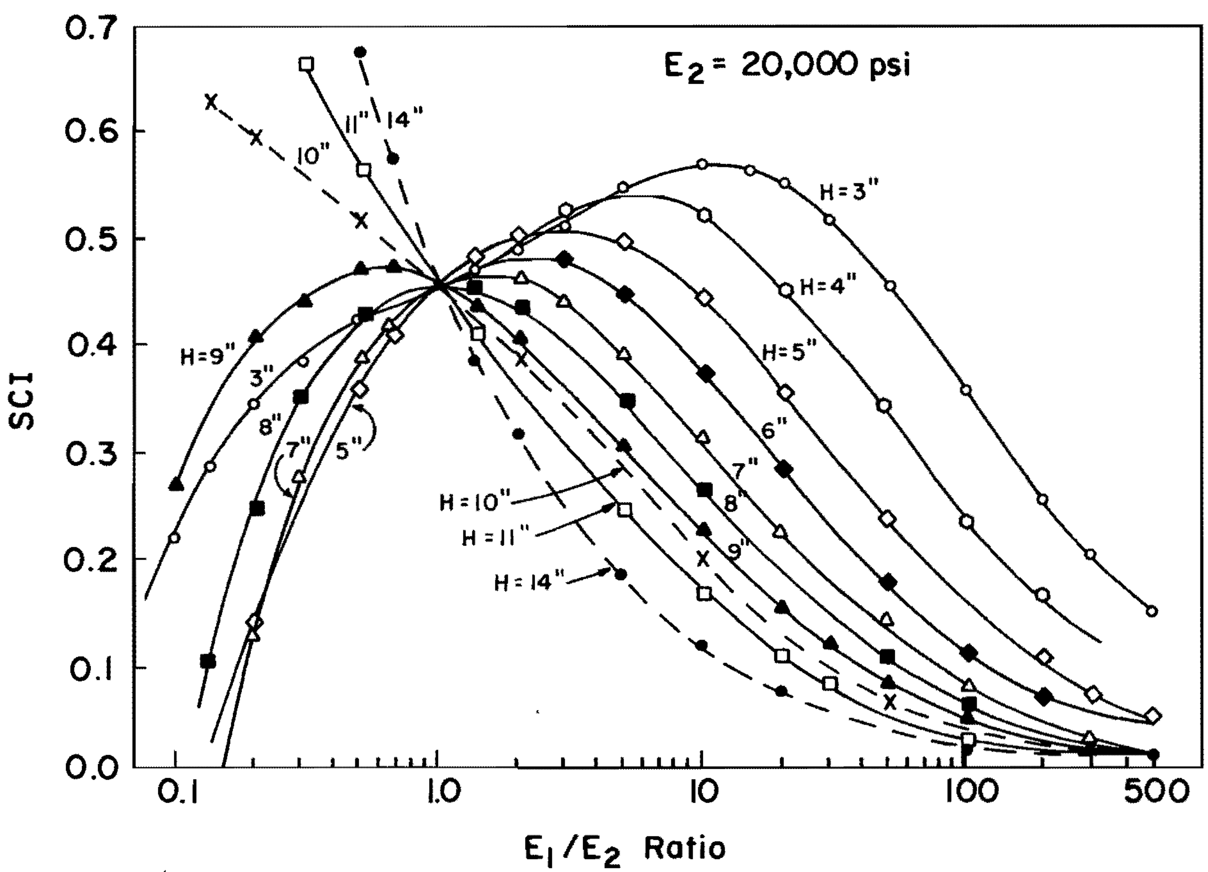


Figure 7. Surface Curvature Index - Modular Ratio Relationship for various pavement systems [after Majidzadeh (23)].

The SCI can also provide information on the magnitude of the strain level within the pavement structure. The tensile strains at the bottom of the pavement layer and the vertical strains in the subgrade are proportional to the surface curvature index. These interrelations are presented in Figures 8 and 9.

The other parameters describing the deflection basin are the base curvature index (BCI) and the fifth sensor reading (W_5).

The BCI is widely accepted as a means of analyzing the subgrade. Field data collected in Ohio (21) have made it possible to establish tolerable ranges for the BCI. Values of 0.05 to 0.11 have been found to be representative of satisfactorily performing pavement systems. Poorly performing roadways respond with 0.15 to 0.20 and greater BCI values.

The base curvature index can also be used to obtain anapproximation of the subgrade modulus. However, the BCI is dependent upon pavement thickness and the modular ratio E_1/E_2. This only holds for pavements structures having good support conditions. The base curvature index has been found to be independent of pavement thickness and surface characteristics (E_1). Majidzadeh has indicated that where the E_1/E_2 ratio does not exceed 50 one might use the BCI as an appropriate measure of the subgrade support value (21).

The third parameter is the spreadibility, S. It is defined as the average deflection expressed as a percentage of the maximum deflection. The spreadibility characterizes the ability of the pavement system to distribute loads, i.e., the slab action of the system. As the magnitude of the spreadibility increases it signifies

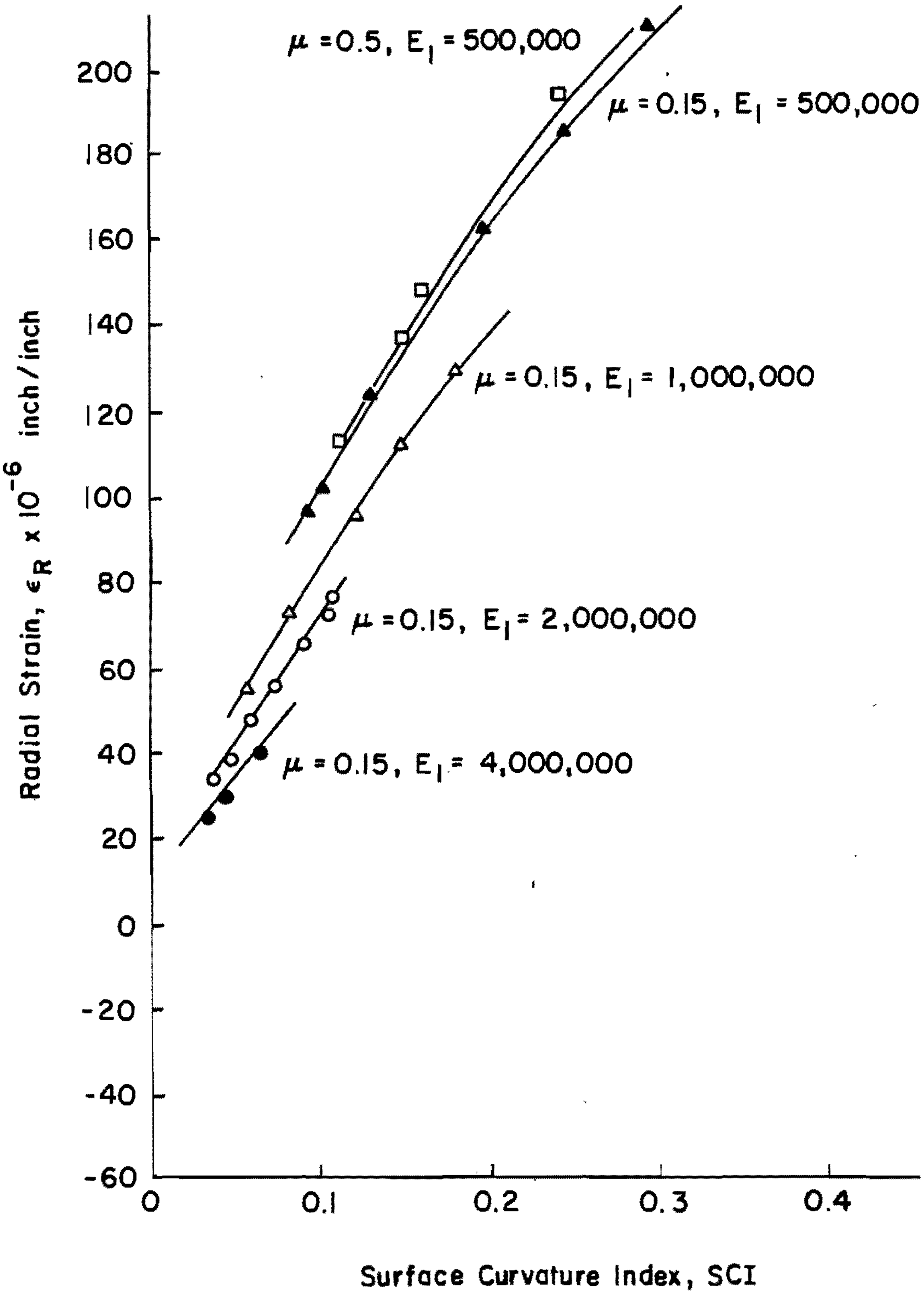


Figure 8. Radial Strain - Surface Curvature Index Relation [after Majidzadeh (23)].

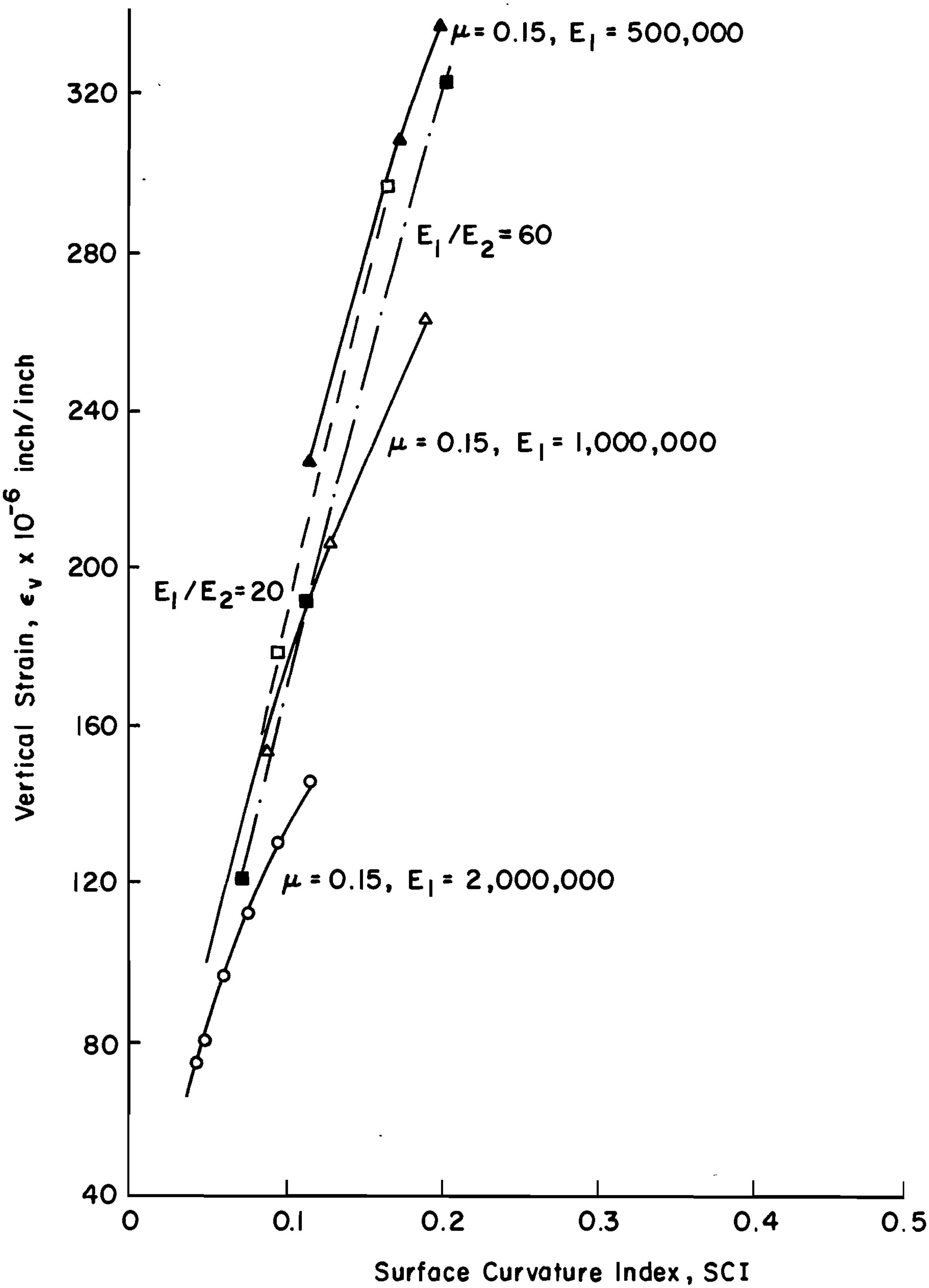


Figure 9. Vertical Strain - Surface Curvature Relation [after Majidzadeh (23)].

a greater capability of the pavement to distribute loads more effectively. If in fact the spreadability does increase then the stresses and strains resulting from the imposed load upon the subgrade are smaller.

The spreadibility parameter is a function of the modular ratio and the thickness of the pavement. As the pavement thickness increases the load distributing capability will increase and as a result spreadibility will increase.

On the other hand, analyses (23) have shown a higher spreadability is not a sufficient requirement for achieving satisfactory pavement conditions. Spreadability is proportional to the modular ratio, rather than the individual values of E_1 and E_2. High spreadibility values may result from a pavement structure consisting of a poor subgrade. When the subgrade modulus is quite high the spreadibility values are proportionally lower.

For very thin pavements, the slab action effectiveness is reduced to a point where the spreadibility concept can no longer be considered a useable concept.

2.2.4 Multiple Parametric Analysis A multiple parametric analysis uses two or more deflection parameters to analyze a layered pavement system. In this study the spreadibility and maximum deflection parameters were used in conjunction with a layered elastic approach.

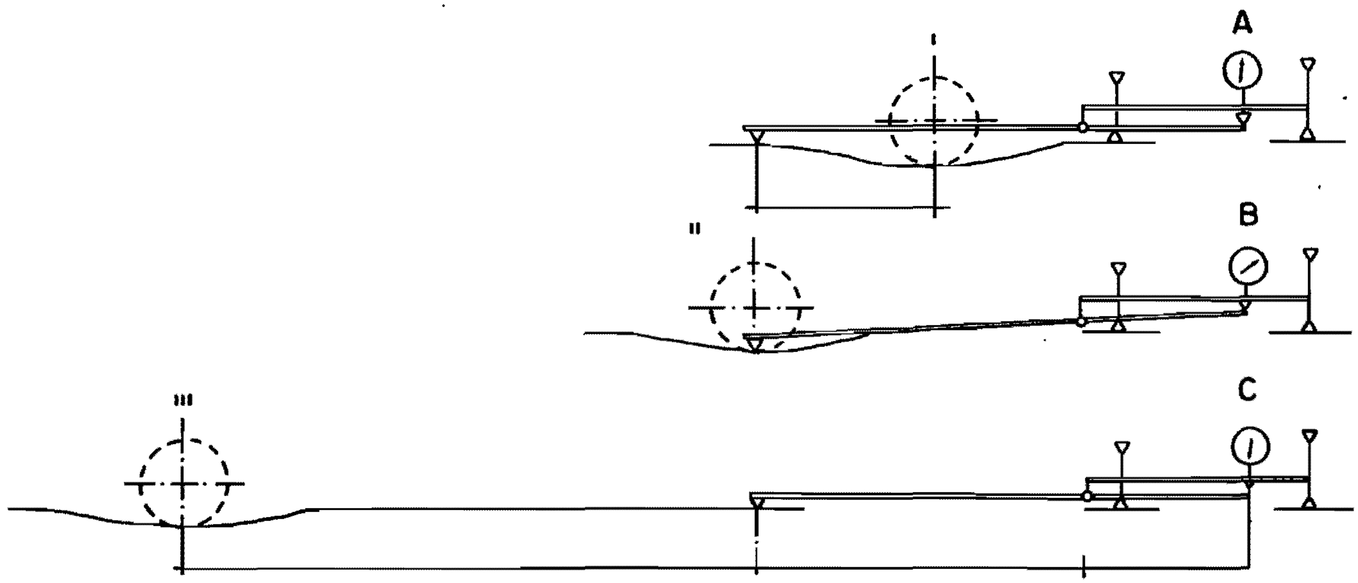
The layered elastic approach is employed to provide the necessary data so that modular values of the subgrade and the effective thickness of the material above the subgrade can be estimated.

The evaluation uses a two layer approximation of the flexible pavement system. This is done by grouping the pavement layers above the subgrade and characterizing them be a 'composite modulus'. In this study the composite modulus of all the test sections evaluated was assumed to be 500 Ksi. Next for the Benkleman beam a 4500 lb. dual wheel load with a radius of contact of 4.23 inches each (two 500 lbs. wheel loads in the case of the Dynaflect) is applied to the subgrade. The maximum deflection and deflections away from the simulated loading center are calculated. The points at which the deflections are calculated are such that they correspond to the recording configurations of the Dynaflect and Benkleman beam. The composite layer above the subgrade is increased in increments of two inches and the resulting maximum deflection and deflections at points from the load are again calculated. As a final step the subgrade elastic modulus is incrementally increased and the previous steps are repeated.

For the Benkleman beam the points where the deflections are requested to be computed are between the dual wheels and at points on a centerline one and three feet behind this point. Figure 10 pictorially depicts these specifications.

The Dynaflect deflections are calculated between the wheels and at distances of 12, 24, 36, and 48 inches behind the loading wheel. This configuration is depicted in Figure 11.

The respective spreadability parameters are calculated from the theoretical deflection data. The effect of the subgrade's elastic



Rebound deflection: wheel moves from II to III or from I to III $d = 2(B-C)$

Total deflection: wheel moves from I to II $d = 2(B-A)$
or from III to II $d = 2(B-C)$

Figure 10. Benkleman Beam in Use

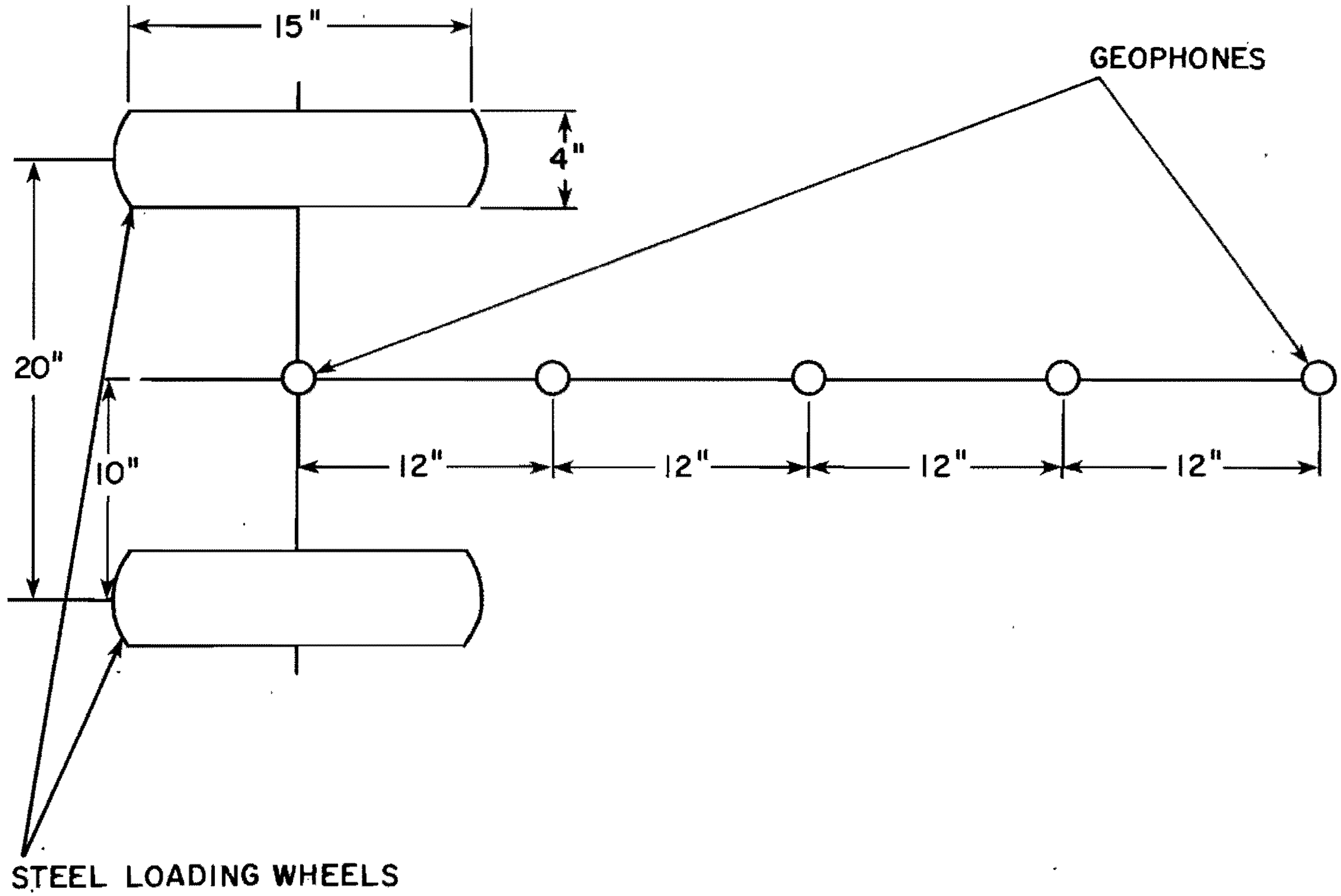


Figure 11.. Position of Dynaflect's Sensors

modulus on the maximum deflection and spreadability can be seen for the Benkleman beam and the Dynaflect in Figures 12 through 14.

For example, in Figure 13 a 4500 lb. dual wheel load is applied to a two inch pavement over a semi-infinite subgrade with an elastic modulus of 7500 psi. The resulting maximum deflection as calculated from the BISTRO program is 0.0256 inch and the spreadability of 57.5. Point A represents the locus of these points.

Point B represents a maximum deflection of 0.0084 inch and a spreadability of 76.5 for a 10 inch pavement. Note that as the pavement thickness increases the maximum deflection decreases and the spreadability increases. The increase in effective thickness follows a path of consistent subgrade modulus, i.e., 7500 psi.

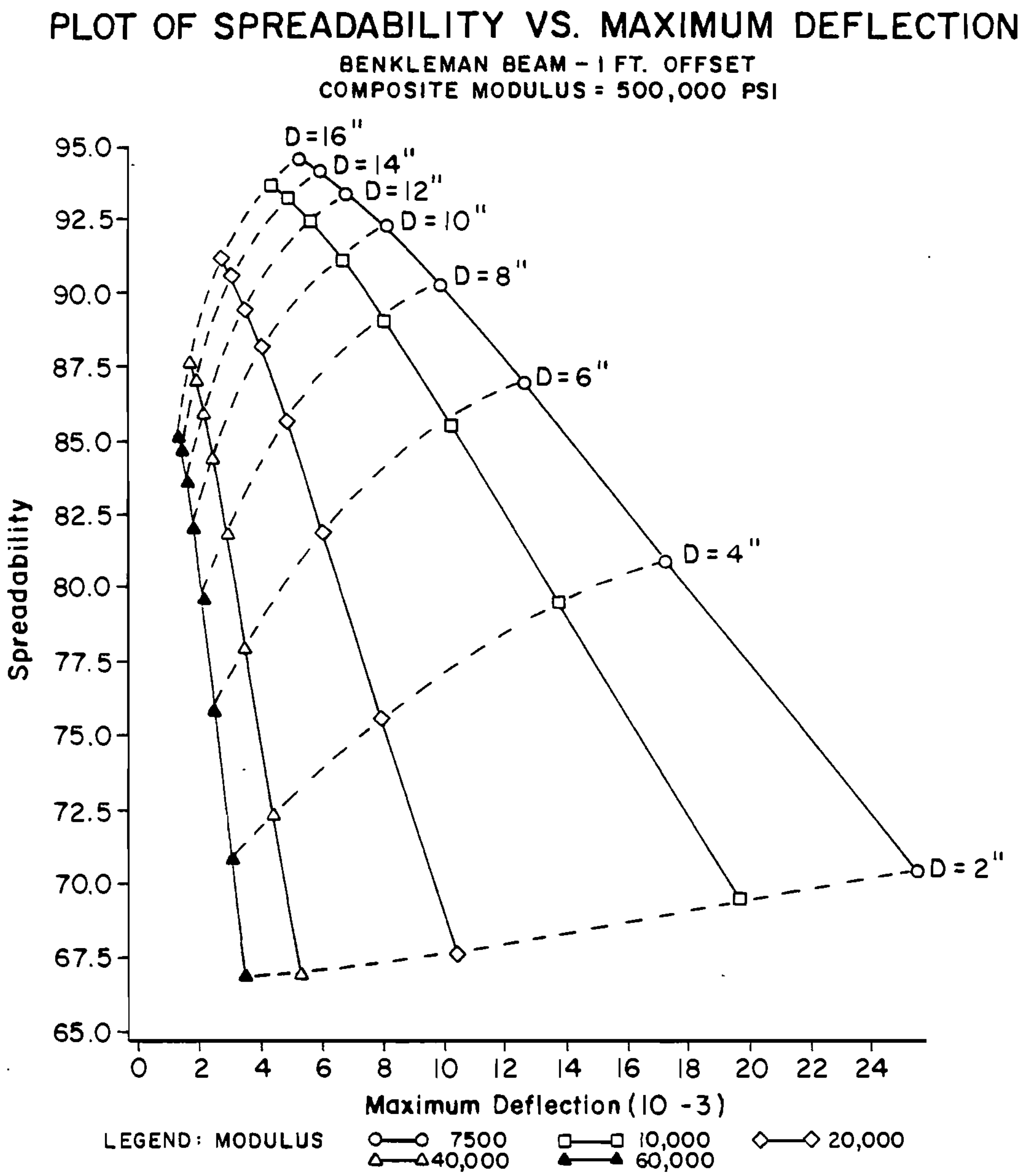


Figure 12. Benkleman beam one foot offset dual parametric chart.

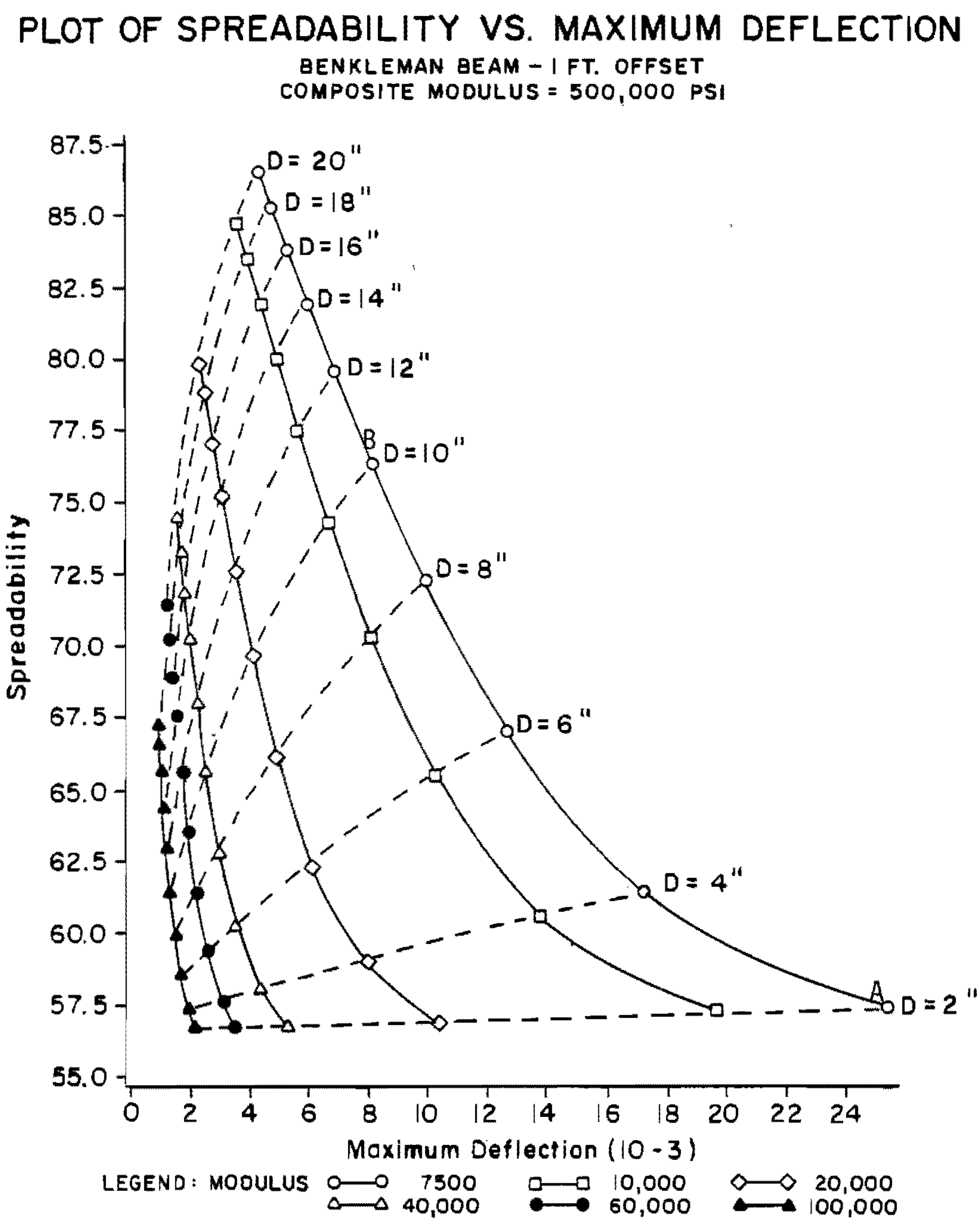


Figure 13. Benkleman beam three foot offset dual parametric chart.

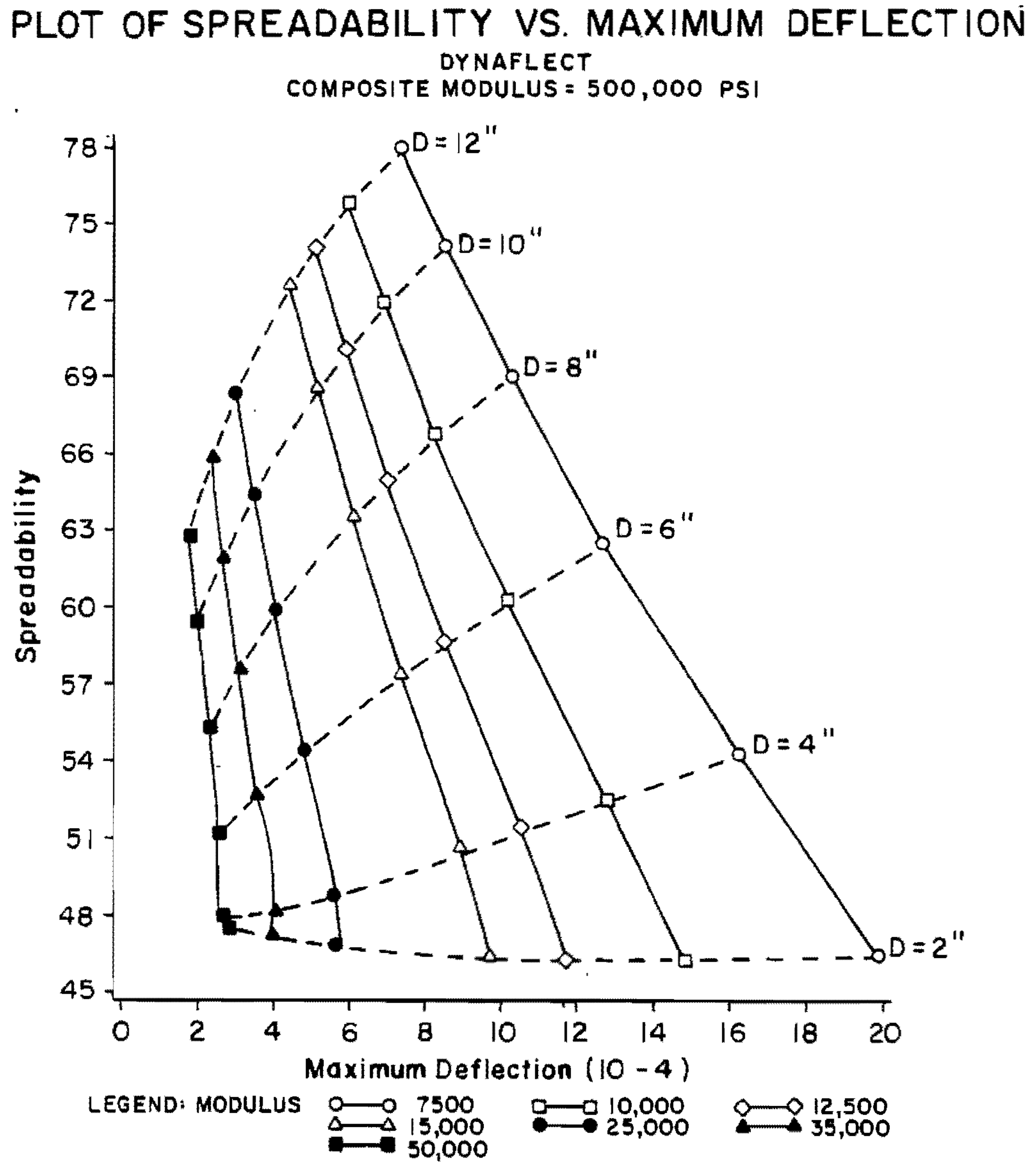


Figure 14. Dynaflect dual parametric chart.

2.3 FLY ASH STABILIZATION

2.3.1 Introduction A problem continually plaguing the pavement design and geotechnical engineer is the ability to deal economically with poor or inadequate subgrade soils. Lime-pozzolan stabilization is currently one method being used to modify soil properties. Conventional methods of lime-pozzolan stabilization deal with the scarifying and mixing of the soil with proper amounts of hydrated lime and a suitable pozzolan. The mixture is then allowed to mellow before final compaction. Stabilization of soils with lime pozzolan results in excellent pavement performance (15,20). A pozzolan, as defined by the American Society for Testing and Materials (ASTM) is a siliceous or aluminosiliceous material that in itself possess little or no cementitous value but in finely divided form and in the presence of moisture will chemically react with alkali and alkaline earth hydroxides at ordinary temperatures to form or assist in forming compounds possessing cementitous properties (25). The most commonly used pozzolan in the U.S. is fly ash. The use of lime-fly ash in soil stabilization dates from about 1934 when a patent was granted on its use as a structural fill material (26).

Lime-fly ash has become increasingly popular because fly ash is a by product in the production of electric power from the burning of coal. Extremely large quantities of fly ash are given in the burning of pulverized coal; this quantity varies from about 80 to 120 pounds per ton of coal (27). It is estimated that more than 90 million tons of fly ash are going to be produced in 1985 (28). Since there is an increasing demand for the utilization or proper disposal of this by product, fly ash may prove a very economical stabilization agent for treatment of road construction materials. However, the use of lime-fly ash stabilization has been somewhat limited. The reasons are 1) it involves the use of two materials, one of which (fly ash) may be quite variable and 2) the process must be accomplished, in most cases, as a two stage process in which the lime is applied, mixed and allowed to mellow before the fly ash is added. This can add from one to two days to the construction schedule for each section constructed.

2.3.2 Lime and Fly Ash Materials Lime is one of the oldest soil stabilizing agents known to man and has been used successfully for improving certain soil properties since the beginning of recorded history. Evidence reveals that lime was used as an effective soil stabilizer during the construction of the Zoser Pyramid (around 3000 BC), where clay, limestone powder and quartz were found to be a filler between stone blocks (29). Another example is the Appian Way which is also referred to as the first lime stabilized road, which has outlasted the Roman Empire (29). Lime stabilization was reintroduced in the early 1920's, but failed initially and was abandoned until the early 1940's.

Lime is a prime example of an active stabilizer, which produces a chemical reaction with the soil or aggregate system. This in turn produces the desirable change in the engineering characteristics of the stabilized soil or aggregate system.

Commercially available limes that are most commonly used are:

1) Hydrated high calcium lime $Ca(OH)_2$,

2) Monohydrate dolomitic lime $Ca(OH)_2 + MgO$,

3) Calcitic quicklime   CaO and

4) Dolomitic quicklime   CaO + MgO.

Fly ash, a by-product of the burning of powdered coal, is collected from the flue gases by either mechanical or electrostatic precipitation devices. Fly ash is composed of spherical solids or hollow amorphous particles of alumina or silica. Other secondary ingrediants such as iron oxide and carbon are also present in fly ash. The color of fly ash is typically black to a light tan and is controlled by the quality of carbon present. The quantity of unburned porous carbon varies depending on the efficiency of the combustion process. Table 9 indicates typical ranges of values for the chemical composition of different classes of fly ash.

The Blaine fineness of quality fly ash typically ranges from 2,000 to 6,000 cm^2/gm which is comparable to that of portland cement which has a fineness of 2,000 to 4,000 cm^2/gm (28).

2.3.3 Mechanisms of Stabilization Generally, there are two separate groups of reactions that take place when lime and fly ash are added to a soil (30, 31, 32). One group of reactions is caused by the lime reacting independently of the fly ash. These are often referred to as lime-soil reactions and consists of: 1) cationic exchange and flocculation - agglomeration, 2) carbonation and 3) pozzolanic reaction. Cationic reaction and flocculation-agglomeration culminate in changes in soil plasticity, workability and swell properties. The soil becomes friable and workability is improved while plasticity and swell properties are reduced. Carbonation is the development of a weak cement due to the carbonation of calcium from the carbon dioxide from the air. The pozzolanic reaction between the

Table 9. Typical Chemical Analyses of Various Fly Ashes after Torrey (28).

Component	Bituminous Ash	Subbituminous Ash	Lignite Ash
Silica, (SiO_2)	49.2	38.4	44.4
Alumina, (Al_2O_3)	23.6	19.0	18.4
Iron Oxide, (Fe_2O_3)	14.7	4.5	5.4
Magnesia, (MgO)	0.8	4.0	4.2
Sulfur Trioxide, (SO_3)	1.0	1.6	1.6
Calcium Oxide, (CaO)	1.0	24.1	18.2
Loss on Ignition, (LOI)	2.7	0.4	0.5

soil particles of reactive soils (33, 34) and lime results in the production of hydrated calcium silicate and aluminate cementing agents.

Practically all fine grained soils display cation exchange and flocculation - agglomeration reactions when treated with lime. These chemical reactions take place immediately after the addition of lime to the soil (30, 35). It is through these reactions that the soil becomes more friable and the plasticity is lowered due to an increase in the plastic limit of the soil (36, 37, 38).

Following this rapid soil improvement is a longer, slower soil improvement termed pozzolanic reaction. In this reaction the lime chemically combines with siliceous and aluminous constituents in the soil to cement the soil particles together. Some confusion exists here. Pozzolanic reaction, sometimes referred to as cementition, a term usually associated with the reaction between portland cement and water in which they combine to form a strong, hard product. The cementing reaction of the lime in soils is much slower. Since there is no rapid cement-water reaction the lime-soil-water reaction is termed pozzolanic because of the extended amount of time necessary for the gradual chemical process by which the calcium hydroxide is combined with the silicates and aluminates in the soil and fly ash.

Two principle components of soil which react with lime in the form of pozzolans are alumina and silica (20). The alumina and silica are dissolved (or partially dissolved) from the clay minerals due to the high alkaline environment produced by the lime. As long as excess lime is in the soil the high alkaline environment will be maintained, thus producing additional dissolved alumina and silica from the clay minerals (39). Recombining the alumina and silica from the clay with additional calcium from the lime forms the complex calcium aluminate silicates which cement the remaining grains together. This is the basis of the strength gaining reactions of lime stabilization methods.As mentioned previously these reactions account for only the lime acting independently of the fly ash.

The second group of reactions is the result of the reaction between the lime and the fly ash. Essentially this complex mechanism stabilizes the material through the formation of hydrated calcium silicates and aluminates (37). These cementing agents all form only after the solubility of the silica and alumina in the fly ash is increased.

It has been discovered that the desired cementitious products form on the surface of the fly ash (40). It follows then that if any more reaction products are to be formed, the necessary calcium must pass through the reacted layer to react with the enclosed pozzolan. Thus, the mechanism of the pozzolanic reaction is one of simultaneous diffusion and chemical reaction of the calcium. Lea (40) has estimated that under normal conditions a pozzolan will not react with more than about 20 percent of its weight of lime within one year.

2.3.4 Factors Affecting Properties of Lime-Fly Ash Stabilized Materials

A satisfactory stabilization technique should be capable of economically imparting strength and durability properties to a soil mixture. However, mixture properties can be affected by many factors. Some of the more important factors involved in lime-fly ash stabilization are illustrated in Figure 15.

The type of lime affects the resulting properties of lime-fly ash stabilized materials. Quicklime is not usually considered unless it can be applied as a slurry. It is limited because it is highly caustic to workers and can also be violently explosive when exposed to water. By-product limes have been successfully used but their variability is great. Also these materials show somewhat of a slower rate of reaction (42).

Dehydrated dolomitic lime is generally not used because it is less effective and produces lower strengths than calcitic and monohydrated dolomitic limes (31, 43).

A controversy arises as to whether calcitic lime or monohydrated dolomitic lime is more effective for use in lime-fly ash stabilization. Studies have shown that monohydrated dolomitic lime is more effective than high calcium lime (44, 45). Both limes produce long term strengths of approximately equal magnitude. Other investigations (43, 45, 46) have found that high calcium limes give higher strengths especially at low lime contents. Thus, it can be said that generally either calcium or monohydrated dolomitic lime is adequate for lime-fly ash stabilization. If possible, laboratory

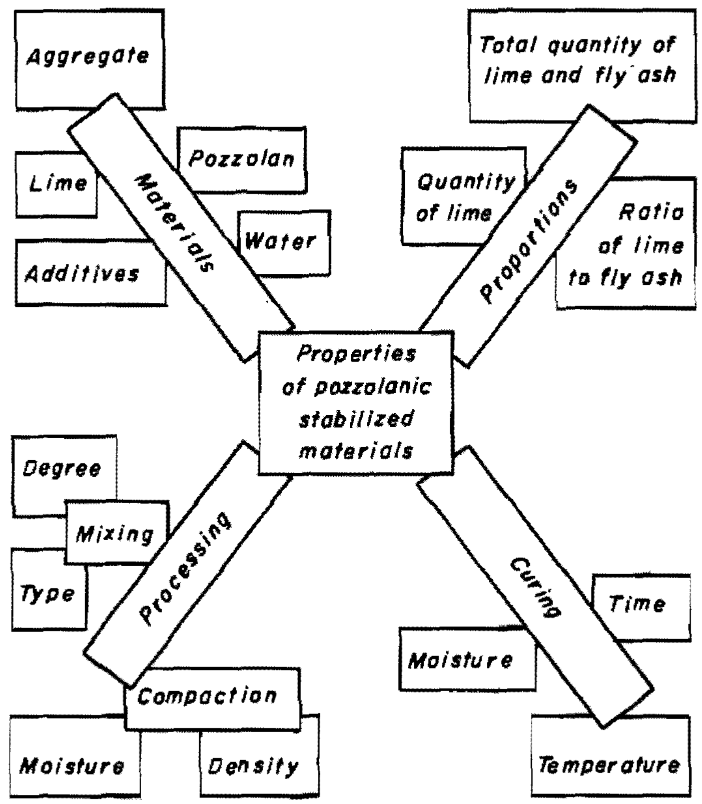


Figure 15. Factors Influencing the properties of Lime-Fly Ash-Soil Mixtures [after Ahlberg (42)].

evaluations should be conducted to indicate the effectiveness of the chosen lime. However, the quality of the fly ash has a much greater influence on the lime-fly ash pozzolanic reaction than does the lime type.

Since fly ash is a by-product and strict controls are not used in its production, great variability can exist among and within fly ash types. Although studies have been conducted to determine methods which will evaluate the reactivity of fly ash (44, 47), problems are being encountered in relating fly ash reactivity to its natural properties. Recognized factors of the pozzolanic reactions in regards to fly ash are: the temperature (Arrhenius effect), nature of the pozzolan, surface area, carbon content, alkali and sulfate content and hydrogen ion concentration (40).

The quality of a lime-fly ash stabilized product depends to a large extent on the materials being stabilized. High clay content, fine grained soils are less desirable for stabilization with lime-fly ash than are silts and more granular sands, gravels and crushed stones. A review of current stabilization practices indicate that lime-fly ash is rarely used to stabilize a fine grained material. This is probably because it is difficult to properly incorporate lime and fly ash with these materials due to their fine grained and plastic nature. It is also economically expensive to produce a lime-fly ash mortar which encases each soil particle due to the fineness of the soil particles. Often fine grained soils will produce similiar strengths with lime alone. In general, well graded aggregates require less lime and fly ash for effective stabilization than do poorly graded aggregates. Angular aggregates normally require slightly more lime-fly ash and produce slightly higher strengths than do rounded aggregates. Basically, Viskochil summed it up in his research on lime-fly ash stabilization--the strength is dependent upon the intimacy of the grain-to-grain contact. Granular soils require about five percent fines (< 0.074 mm) to produce adequate durability in a well graded mix (48). The presence of organic material will interfere with the pozzolanic reaction and is thus quite undesirable. These nonreacting materials prevent contact of the cementitious materials.

The quantity of lime used affects both strength and durability and depends on 1) the percentage of clay and silt present in the material, 2) the total quantity and pozzolanic quality of the fly ash and 3) the quality of the lime. Typically as the clay and silt content increases, an increasing amount of lime is required to react with the clay and silt fraction. Likewise, as the quantity and reactivity of the fly ash increases, the quantity of lime required to react with the fly ash decreases.

The ratio of lime to fly ash also influences strength and durability. However, as shown in Figure 16, there is generally an optimum lime-fly ash ratio for a given soil, lime type and fly ash. A substantial savings may result by choosing the optimum lime to fly ash ratio. Likewise, the total amount of lime and fly ash does affect the strength and durability. As shown in Figure 17 compressive strength increases as the amount of lime and fly ash in the mixture increases. In Figures 16, 17, and 18 the maximum load causing failure is reported as compressive strength, therefore the size of the specimen must be noted in any comparison of the test data.

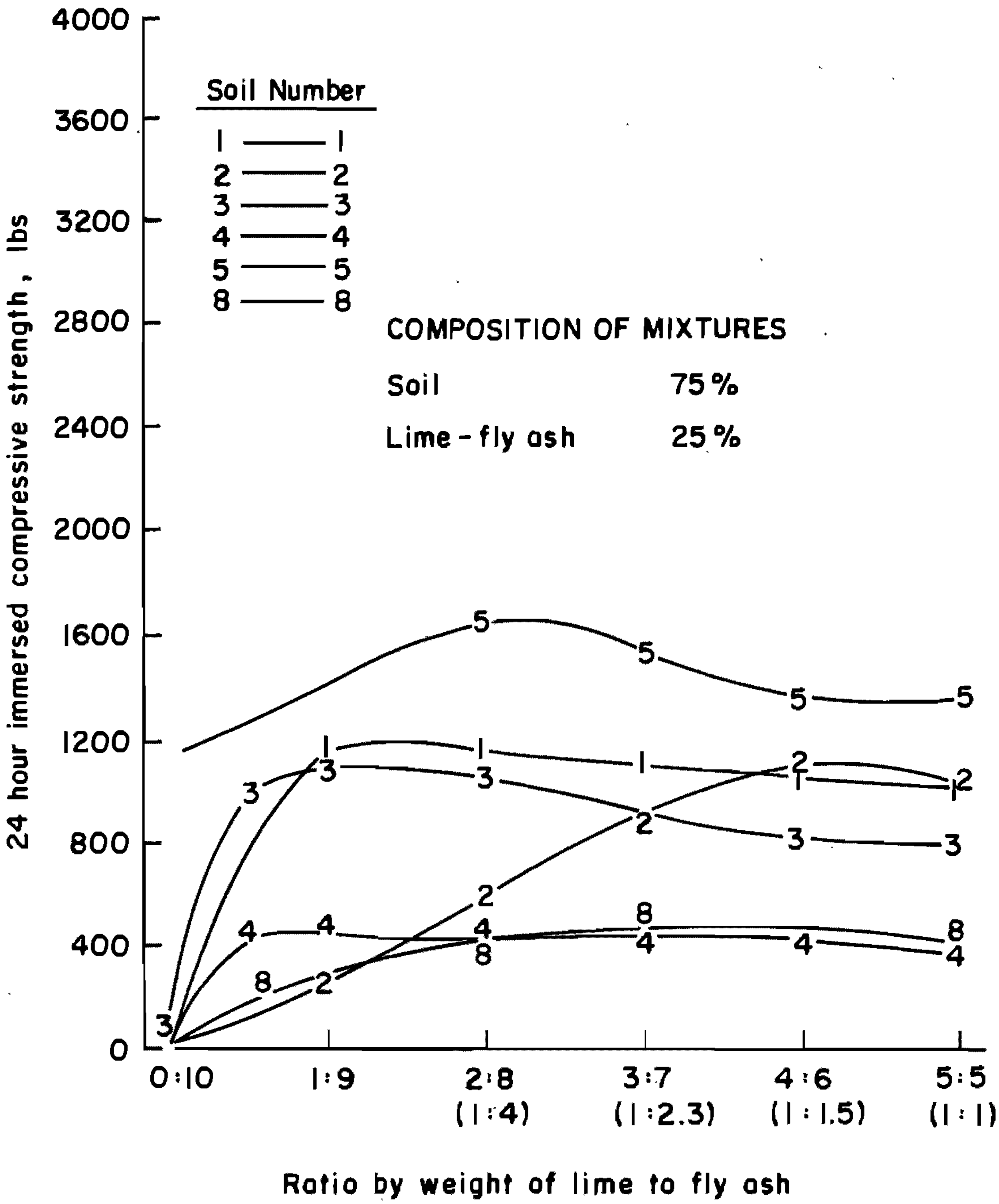


Figure 16. Effect of Variations in the Ratio of Lime to Fly Ash on the 28 Day Compressive Strength of Lime-Fly Ash-Soil Mixtures. The 2" x 2" specimens used were cured at 100% relative humidity and 70°F [after Goecker (46)].

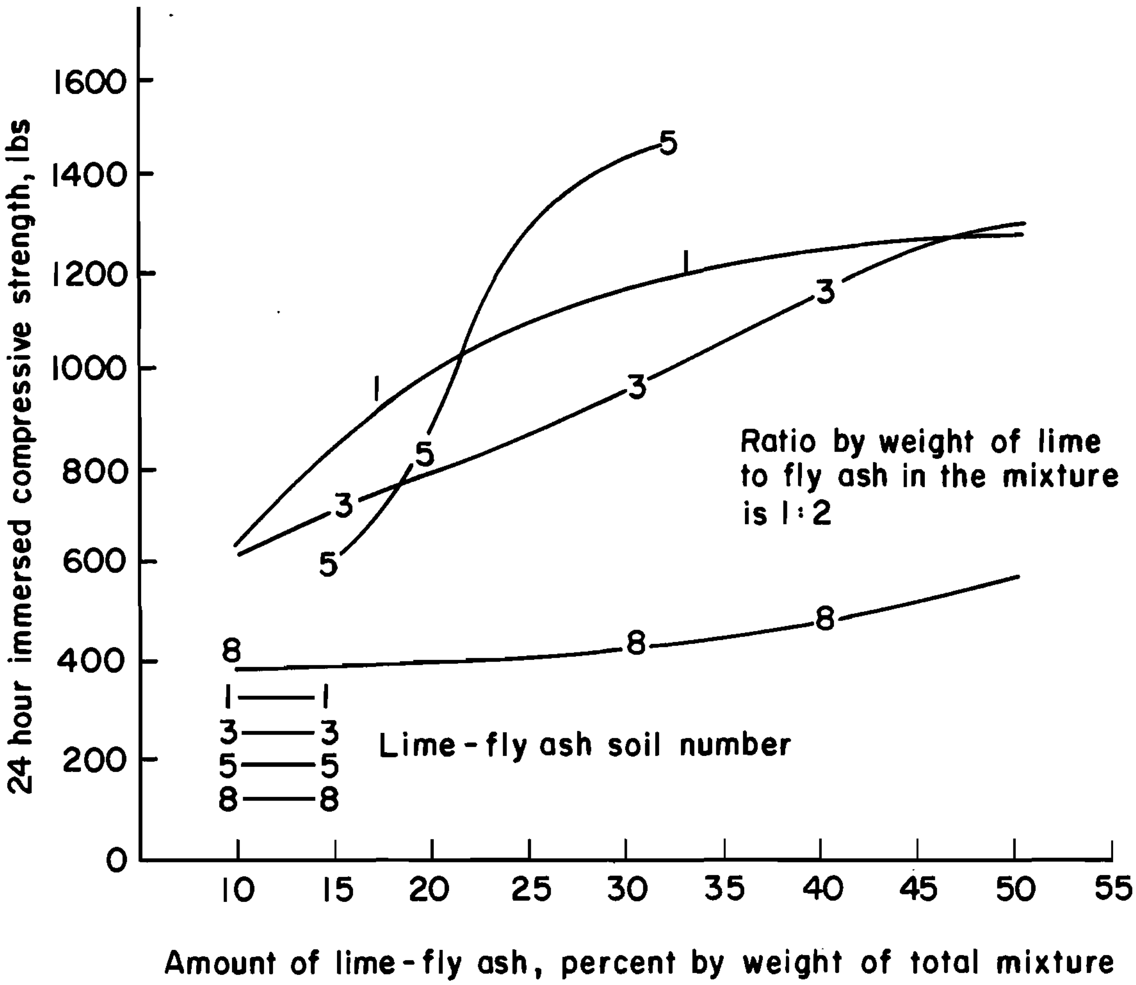


Figure 17. Effect of Variation in the amount of Lime-Fly Ash on the 28 Day Compressive Strengths of Lime-Fly Ash Stabilized fine grained soils. The 2-inch by 2-inch specimen used were cured at near 100% relative humidity and 70°F. [after Goecker (46)].

The optimum moisture content is that which produces the maximum density at a particular compaction effort and is sufficient to complete the pozzolanic reaction. The moisture content that produces maximum density almost always produces maximum strength, although one study (49) indicated that compacting dry of optimum for sand-lime-fly ash mixtures and slightly wet of optimum for clayey soil mixtures produced maximum strength.

The type and quality of processing have been found to affect the properties of the stabilized mixture. As the uniformity and intimacy of the mixture increases, the strength and durability likewise increase. The degree of compaction obtained in the processed material is one of the most important steps in the stabilization process. The main objective of compaction is to insure high density and interparticle contact since these influence strength and durability. The fact that high density insures higher strength and durability has been shown by Viskochil (50) and Hoover (51). Increasing the density by 10 percent over standard compaction (ASTM D 698) doubles the compressive strength. Viskochil (50) also showed evidence that the density is equally a function of the lime-fly ash ratio. It is decreased by higher lime contents because of two factors. First, the lime itself is less dense than soil or fly ash and secondly the lime causes aggregation of the clay which increases the air void matrices.

Similarly, as density in increased, the durability of the lime-fly ash compacted is increased (51).

Proper curing is an extremely important step in the development of lime-fly ash-soil strength and durability. The most important variables involved are curing time, temperature and moisture regime. At ambient temperatures the major portion of strength gain occurs during the first year (42); however, only approximately 50 percent of the ultimate strength is gained during the first month as compared to soil-cement which may gain up to 90 percent of ultimate strength during the first month of curing (40). The effect of curing and temperature on compressive strength is vividly shown in Figure 18. Strength development temporarily ceases at temperatures below about 40-50 deg. F (42). Strength development may actually become dormant during the winter months. A sufficiently long curing period is required for stabilized base courses so that adequate strength and durability can be developed before traffic is applied.

The moisture regime during the curing period is important. Optimum moisture will provide sufficient moisture for the pozzolanic reaction. However, if during the initial stages of the curing period, evaporation of this moisture does occur, the pozzolanic reaction is retarded and ultimate stength and durability will be adversely affected (52). Field curing methods include periodic sprinkling, sealing with a liquid bituminous material, or placing a moist soil cover or water proof cover over the treated and compacted lime-fly ash material.

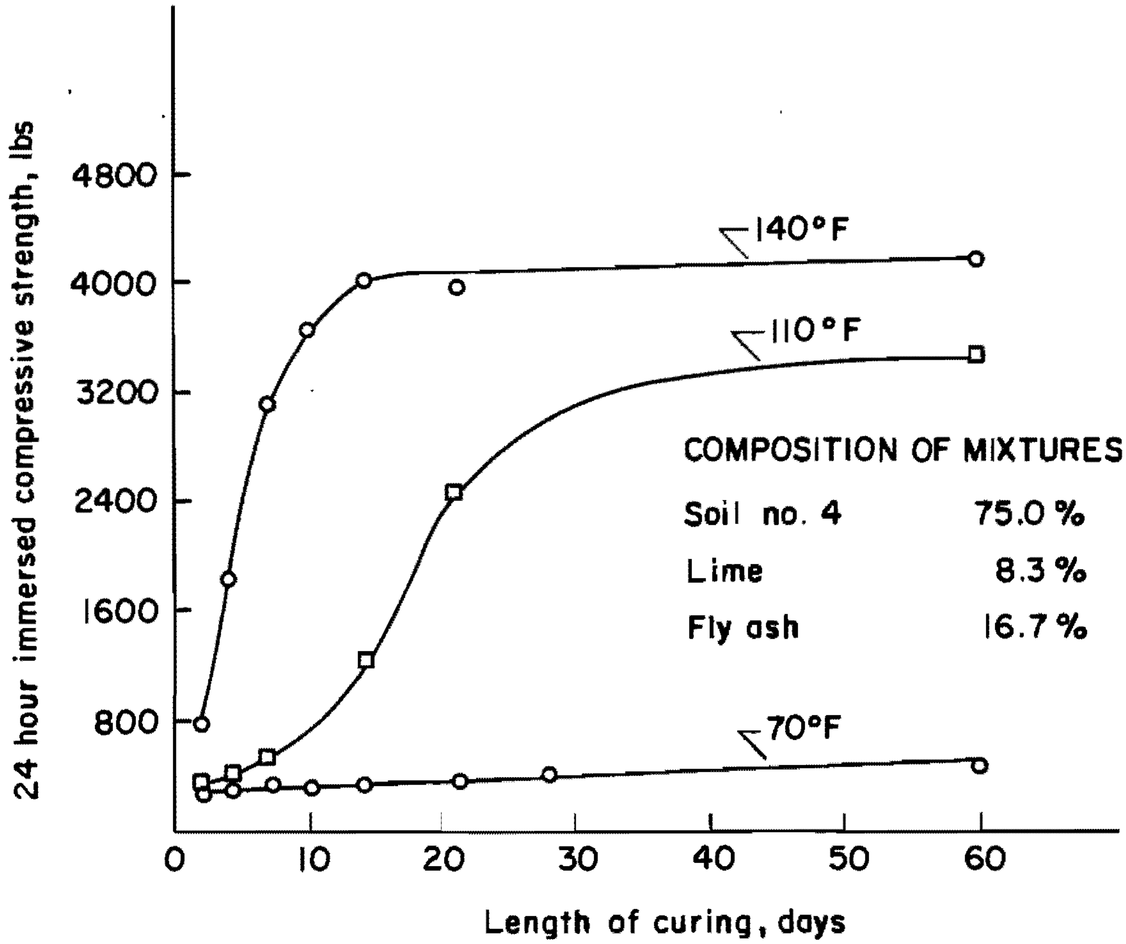


Figure 18. Relationship between compressive strength and curing temperatures of specimens of a lime-flyash stabilized silty soil. The 2-inch by specimen were cured at near 100% relative humidity after Goecker (46) .

3. TEST SITE EVALUATION PROCEDURES

3.1 General

The effectiveness of the lime-fly ash stabilization technique can be determined from quantitively interpreting surface deflection basin parameters and dual parametric results.

The conventional basin parameters, for both the Benkleman beam and the Dynaflect, are calculated and compared for each test site. For the Dynaflect these conventional parameters include; 1) maximum deflection, 2) spreadibility, 3) surface curvature index, 4) base curvature index and 5) last sensor reading. The parameters used from the Benkleman beam surveys are; 1) maximum deflection, 2) offset deflection readings and 3) spreadibility.

Particular interest are paid to the basin parameters which measure the response of the lower layers in the pavement system. Recognition of those test sections which appear to be benefiting from the lime-fly ash stabilization have been made by comparing the deflection parameters against the control section's basin parameters. The control section in each test site is the secion in which lime was the only stabilization agent.

Material characterizations have been made from the spreadibility and maximum deflection for both Benkleman beam and Dynaflect field deflection surveys. The advantage of the dual parametric approach is that it separately evaluates the subgrade and the overlying pavement layers. The results indicate two things - the stiffness of the subgrade (as indicated by the resilient modulus) and the effective thickness of the pavement layers. By using these results one can identify which layers of the pavement system are increasing or decreasing in load carrying capability with respect to the other.

The movement of the effective thickness and resilient modulus over a two year period have been monitored and then classified. The classification system indicates the structural development of specific pavement layers.

The statistical approaches that are used are a general linear model regression and an analysis of variance test. The general linear model (GLM) estimates and tests hypotheses about linear models. If we assume that the model is correct hypotheses can be tested, confidence regions can be assigned and significance probabilities can be computed.

The analysis of variance (ANOVA) is a statistical technique that we used to study the variability of experimental data. For example, one might observe that using different percentages of lime or fly ash in subgrade stabilization results in different dual parametric results. The difference in dual parametric results is the variability. One can use analysis of variance to see if such factors as the lime or fly ash percentages contribute to that variability.

3.2 Analysis of Test Site 1, FM 3378, Bowie Co.

3.2.1 Dynaflect Analysis Table 10 shows the lime and fly ash combinations and the actual age of the deflection data. Tables B-1 through B-3, Appendix B, contain the average of the Dynaflect deflection measurements taken at each test section and the computed spreadibility values. Each value represents the average of 20 field measured deflection values. Since this site involves both stabilized base and subgrade material, the deflection parameters of primary importance are the SCI and the BCI. Tables 11 through 13 list the deflection basin parameters in chronological order for easier comparison.

First consider the maximum deflection. As can be seen from the three Tables different patterns are present.

In regard to the maximum deflection (W_1), the total pavement structure is increasing in load carrying capacity if W_1 decreases with time. The opposite holds true for the spreadibility. With time, S should increase in value.

Only sections 1, 3 and 10 show a continual reduction in the maximum deflection. Section 2 is shown holding about constant after one year's time. All other sections, 4 through 9, show an increase in W_1 with time. This would indicate that the pavement system as a whole, all layers included, are losing their load carrying capacity. But, as previously pointed out, this measurement does not delineate the differences due to the performance of the various layers. To account for this inefficiency, the other parameters should be examined and a conclusion reached which is based upon all of the parameters.

Table 10. Lime-Fly Ash Combinations and Age of Deflection Survey for Site No. 1

Section	Lime/Fly Ash Base[a]	Lime/Fly Ash Subgrade	AC Age (months)	1 yr. Age (months)	2 yr. Age (months)
1	8/0	4/0	3	12	22
2	4/4	4/0	3	12	22
3	4/8	4/0	3	12	22
4	4/15	4/0	3	12	22
5	7/0	4/0	3	12	22
6	6/6	0/6	3	12	22
7	6/11	0/6	3	12	22
8	7/18	0/6	3	12	22
9	5/23	0/6	3	12	22
10	6/6	0/6	3	12	22

[a] weight percentages

Table 11. Dynaflect Deflection Basin Parameters- Site No. 1 (3 month survey).

Test Section	Lime/Fly Ash (base)[a]	Lime/Fly Ash (subgrade)[a]	W_1	SCI	BCI	S	W_5
1	8/0	4/0	.523	.140	.054	58	.151
2	4/4	4/0	.574	.159	.053	55	.139
3	4/8	4/0	.874	.214	.079	62	.312
4	4/15	4/0	.763	.147	.075	67	.311
5	7/0	4/0	1.067	.248	.106	62	.369
6	6/6	0/6	1.149	.396	.079	52	.264
7	6/11	0/6	1.056	.348	.080	50	.189
8	7/18	0/6	.964	.333	.073	51	.202
9	5/23	0/6	.696	.314	.089	53	.232
10	6/6	0/6	.994	.505	.115	51	.319

[a] weight percentages

Table 12. Dynaflect Deflection Basin Parameters- Site No. 1 (1 year survey).

Test Section	Lime/Fly Ash[a] (base)	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	8/0	4/0	.438	.099	.065	65	.154
2	4/4	4/0	.526	.146	.066	58	.137
3	4/8	4/0	.761	.145	.091	69	.329
4	4/15	4/0	.718	.128	.087	69	.307
5	7/0	4/0	.830	.161	.110	68	.329
6	6/6	0/6	.965	.270	.097	57	.264
7	6/11	0/6	.853	.203	.115	57	.189
8	7/18	0/6	.718	.146	.096	62	.204
9	5/23	0/6	.804	.165	.109	63	.244
10	6/6	0/6	1.144	.294	.132	59	.316

[a] weight percentages

Table 13. Dynaflect Deflection Basin Parameters- Site No.1 (22 month survey).

Test Section	Lime/Fly Ash[a] (base)	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	8/0	4/0	.434	.086	.060	65	.151
2	4/4	4/0	.527	.131	.059	58	.141
3	4/8	4/0	.721	.108	.088	71	.326
4	4/15	4/0	.790	.118	.086	68	.316
5	7/0	4/0	.895	.155	.116	67	.345
6	6/6	0/6	1.047	.312	.105	55	.259
7	6/11	0/6	.959	.227	.124	57	.211
8	7/18	0/6	.746	.179	.096	60	.206
9	5/23	0/6	.860	.193	.102	61	.258
10	6/6	0/6	1.120	.267	.124	59	.316

[a] weight percentages

Next, to look at another parameter that depicts the integrity of the pavement system as a whole, the spreadibility is examined.

The spreadibility values show the same thing as did the W_1 measurements. Section 3 is the only section showing improvement. Sections 1, 2 and 10 show a constant magnitude, while the rest show a decrease in load distribution.

As indicated by Tables 11, 12 and 13, the pavement's surface structural integrity, as indicated by the SCI, is on an average, half of the initial value. It's interesting to note that the sections which utilized only lime show a continual decrease in SCI. But since any pavement failure cannot be related entirely to the pavement surface, the evaluation of the BCI is desirable.

During the time period from the just completed construction stage to the one year age, all sections showed an increase in BCI. It is after this one year period that different responses become apparent. Though some sections exhibit a slight increase or decrease in BCI they effectively remain constant.

In evaluating the W_5 readings various patterns are seen. A majority of the sections either remain about the same or increase slightly in the initial year. After this initial year, the sections using fly ash in the prepared subgrade show, as a whole, less deflection.

Figures 19 and 20 depict the Dynaflect dual parametric measurements for the test sections involved in test site 1. The data are shown on two figures for clarity. Tables 14 and 15 summarize the data for the figures. All sections show either a gradual increase or decrease in resilient modulus. The decrease in resilient modulus in

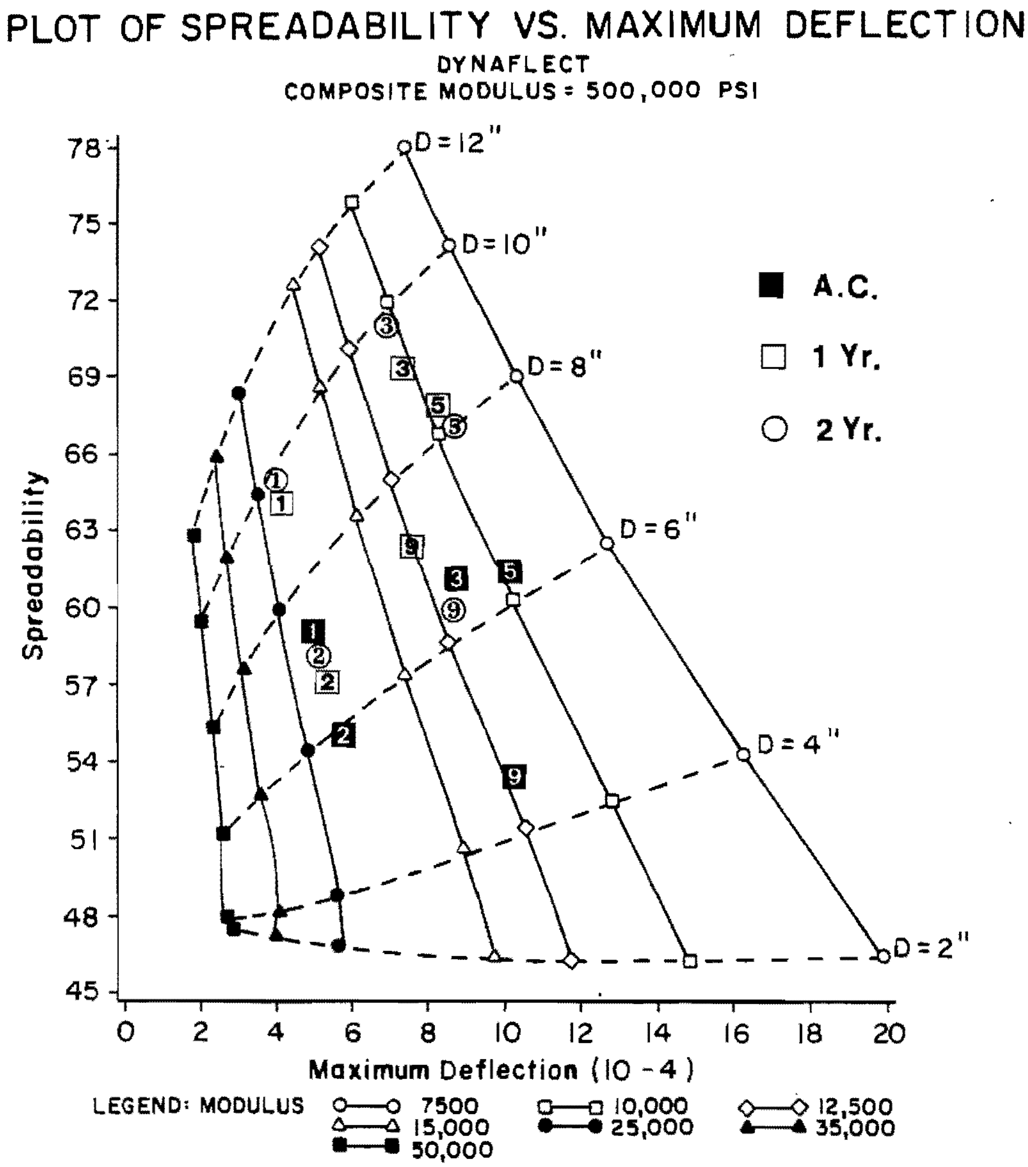


Figure 19. Dynaflect dual parametric chart-- Site 1, Sections 1, 2, 3, 5 and 9.

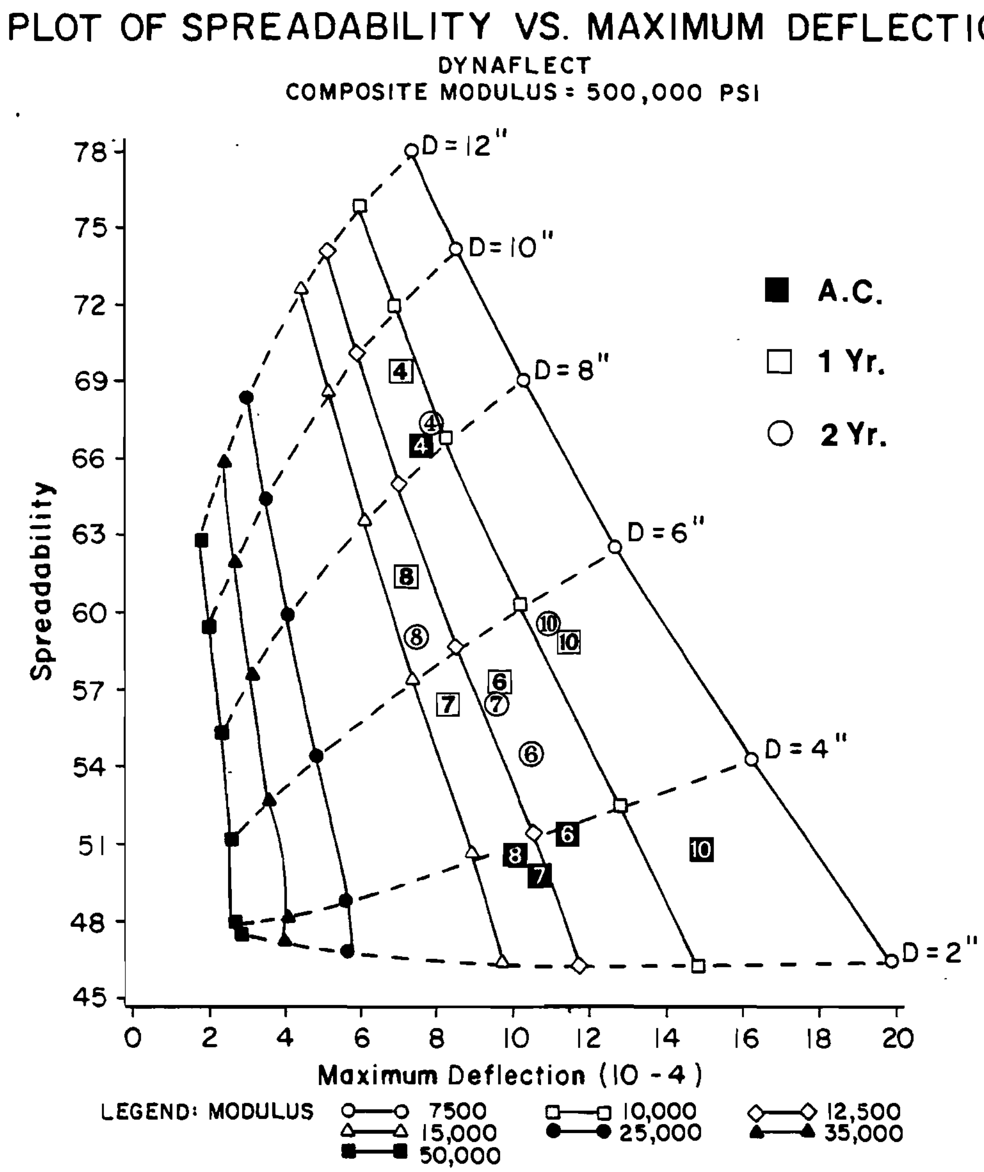


Figure 20. Dynaflect dual parametric chart--Site 1, Sections 4, 6, 7, 8 and 10.

Table 14. Dual Parametric Effective Depth Results - Dynaflect Site No. 1

Section	Effective Depth (in.) D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	7	9.5	9.75
2	6	6.75	7
3	7	9	9.75
4	8.25	9.25	8.50
5	6.5	8.5	8
6	4	5.5	5
7	3.5	5.5	5.25
8	4	7	6.5
9	4.5	7	6.5
10	3.5	5.5	5.5

Table 15. Dual Parametric Resilient Modulus Results - Dynaflect Site No. 1

	Resilient Modulus (Ksi)		
Section	E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	22.5	22.5	22.5
2	22.5	22.5	22.5
3	11.5	11	11
4	11.5	11	10.5
5	10	10	9.5
6	11.5	11.5	11.5
7	13	13	12
8	14	14	14
9	12.5	12	12
10	9	9.5	9.5

sections 4 and 7 is only a 1,000 psi loss and is not too significant. The only test section to show an increase in stiffness was section 10. It's interesting to note that sections 7 and 10, stabilized with the same lime-fly ash combination, gave quite different initial results.

In general, all of the test sections show a movement of up and to the left on the dual parametric charts. This would indicate that both the stiffness and effective thickness are increasing.

To statistically test the significance of the lime and fly ash combinations a general linear regression model was developed and the results are given in Table 16. The significance level at which either lime or fly ash can be concluded to be an important factor on the resulting change in the dual parametric results is listed in this table. From these results it can be seen that no real significance can be inferred from the data as to the effect of the lime or fly ash percentage on the change in resilient modulus of the subgrade.

Another statistical evaluation was made using the analysis of variance test. The results are shown in Table 17. At the 0.10 significance level both lime and fly ash prove to be significant in the change in effective thickness and the subgrade's stiffness over the two year study period.

3.2.2 Benkleman Beam Analysis This analysis contrasts with the previous one involving the Dynaflect by a major factor. Instead of several deflection mesurements being made only two were taken. The first measurement, W_1, was obtained from between the dual wheels and the second reading was one ft. and, at a later time 3 ft., back from the inside wheel. This would translate into a 16.23 and 40.23 inch

Table 16. General Linear Model Regression Analysis- Site No. 1

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from the AC Stage to the 2 Yr. Stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	2.58	2.25	0.10
Fly Ash	2.82	2.25	0.10

Testing the Significance of Lime and Fly Ash Percentages on the Change in Elastic Modulus from the AC Stage to the 2 Yr. Stage

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.24	*	*
Fly Ash	0.23	*	*

Note: Asterisks indicate negligible values of statistical parameters

Table 17. Analysis of Variance Statistical Test- Site No. 1

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from AC Stage to the 2 Yr. Stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	2.58	2.25	0.10
Fly Ash	2.31	2.04	0.10

Test for the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from AC Stage to 2 Yr. Stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.24	*	*
Fly Ash	0.27	*	*

Note: Asterisks indicate negligible values of statistical parameters

radial distance from the W_1 measurement. The average of approximately 20 field measurements are presented in Tables B-4 through B-6, Appendix B.

The maximum deflection values show no apparent pattern. Although in the initial stages the maximum deflection decreases in value, a reversal generally occurs as the pavement structure ages. In two sections (4 and 6) the maximum deflections in the second year indicate that the load supporting capabilities have increased.

The W_2 values can add more information as to which section or sections are improving in load carrying capability.

The type of stabilizer is seen not to affect the resulting W_2 response. However, section 9 which used a larger percentage of fly ash in the base course has a W_2 measurement from two to three times larger than the other sections at the end of the first year.

This same initial increase with a reduction after the first year is also seen in the spreadibility values. From Tables B-4, through B-6 it can be seen that all but two sections show quite an increase in S, but begin to decrease in the second year. Note too that all sections have S values at the end of the first year that are lower than when initially constructed.

Figures 21 through 23 display the data presented in Tables B-4 through B-6 graphically on the dual parametric charts. From these graphs, the effective depth and the resilient moduli were determined and are presented in Tables 18 through 19. As can be seen in Table 19, the modulus values increased from the as constructed stage to the end of the first year in all but section 10. From dual parametric chart of Figure 22, section 8 showed the greatest increase, going from

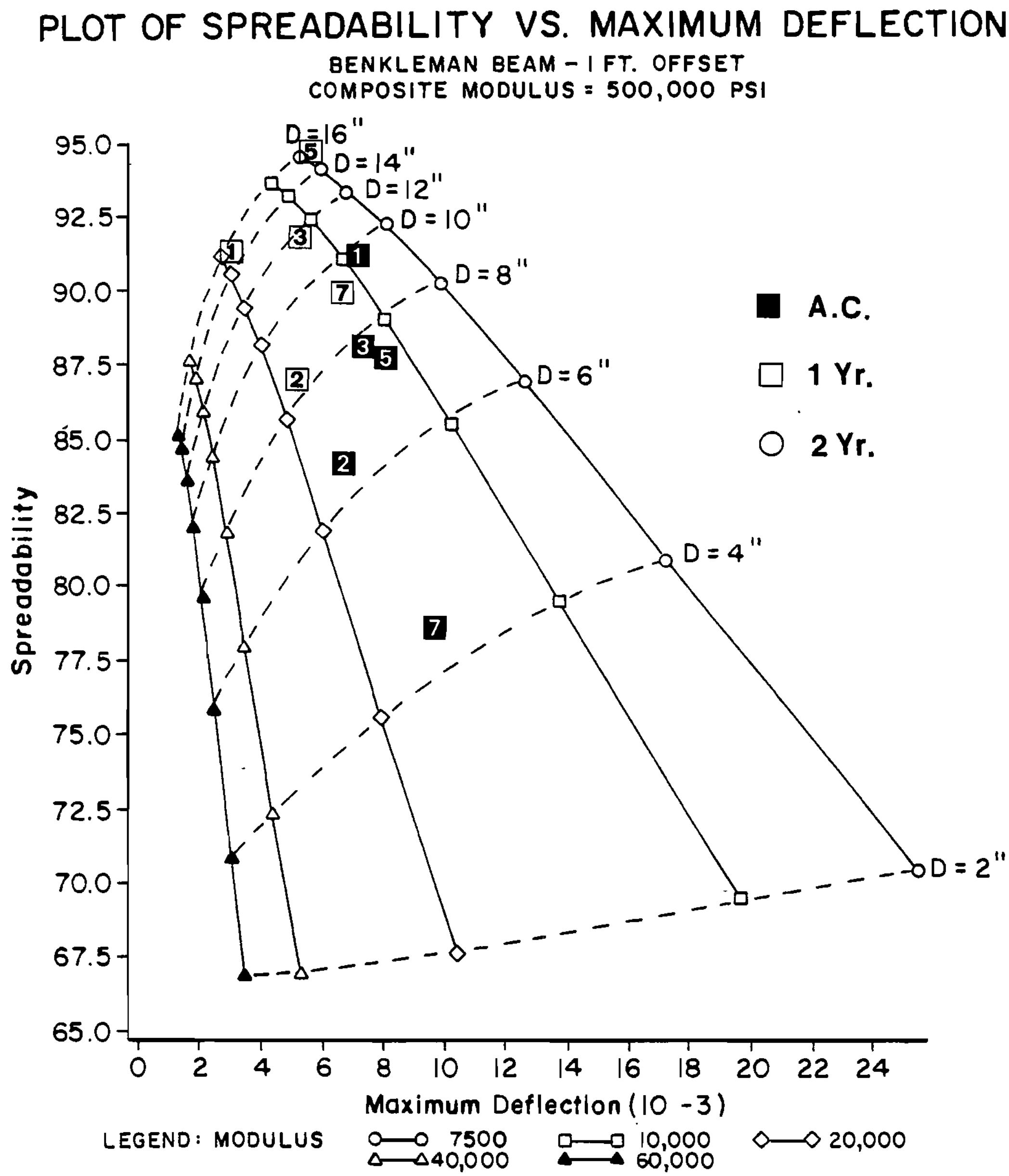


Figure 21. Benkleman beam dual parametric chart--Site 1, Sections 1, 2, 3, 5 and 7.

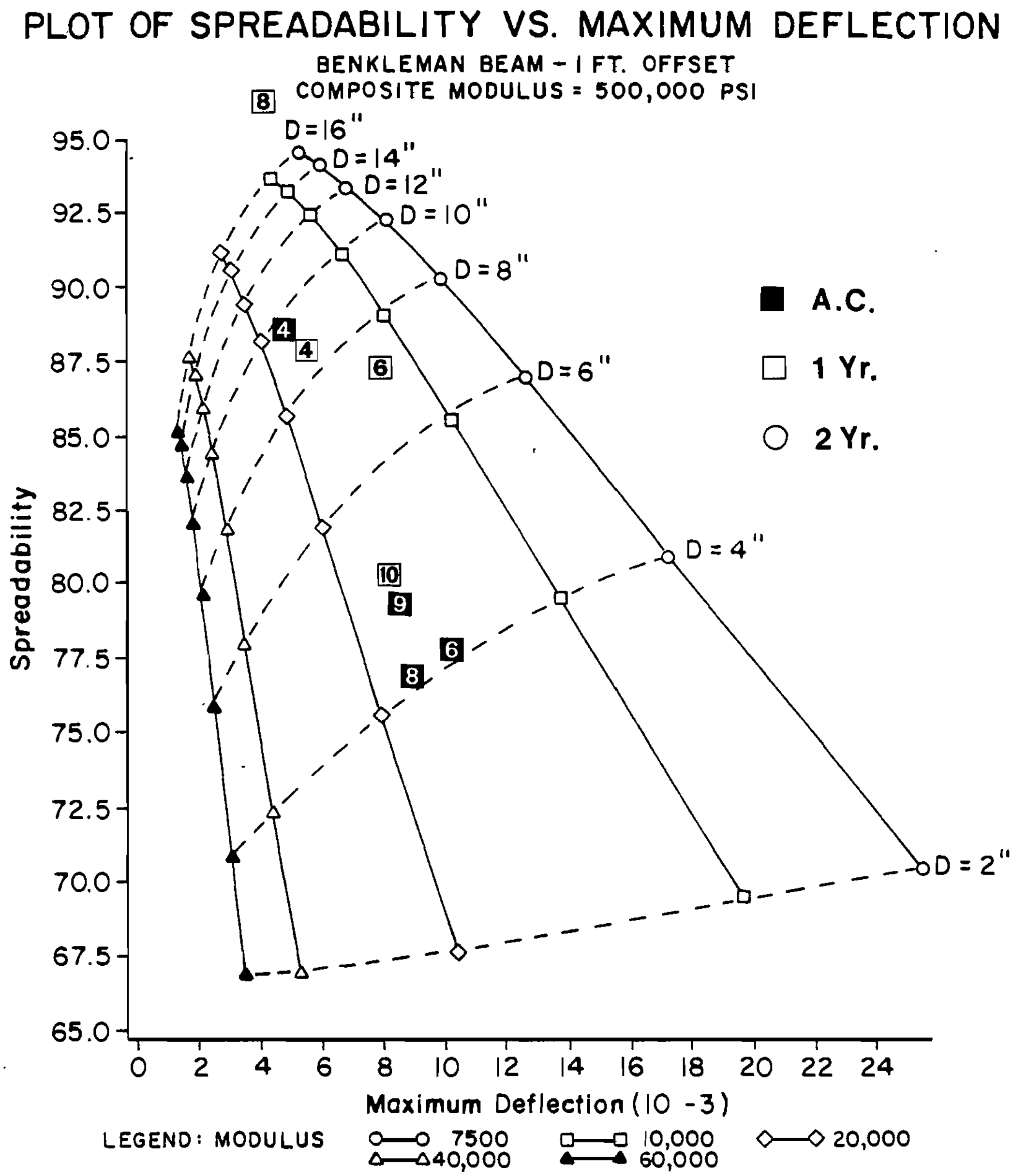


Figure 22. Benkleman beam dual parametric chart--Site 1, Sections 4, 6, 8, 9 and 10.

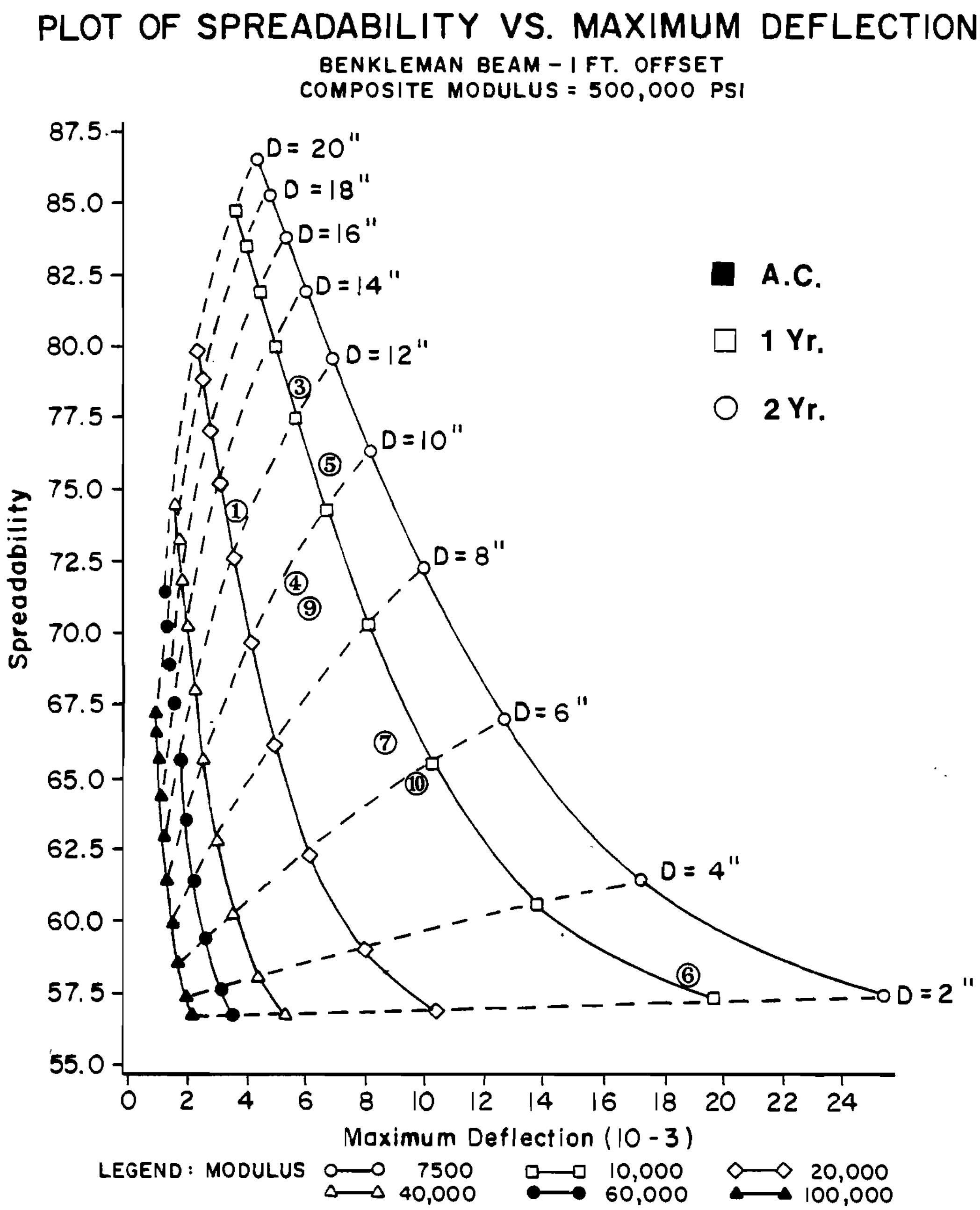


Figure 23. Benkleman beam dual parametric chart--Site 1, Sections 1, 2, 3, 4, 5, 6, 7 and 10.

an effective thickness of 4 inches to 24 inches in one year's time. In evaluating the deflection parameters for the period between the end of the first year to the beginning of the second year one should be cautious. The deflection recording configuration for the two year data used the three foot offset which results in different spreadibility parameters being calculated.

By the end of the second year the control section reflects the greatest effective thickness. The resilient modulus values indicate that the lime-fly ash stabilization technique produces a slightly lower resilient modulus than does lime stabilization. The pattern shown at the end of the first year's time indicates the effectiveness of the lime-fly ash stabilization process. Only sections 5 and 8 show a significant difference.

From these two tables it is apparent that the lime-fly ash stabilization process does result in a final product equivalent, if not superior to, lime stabilization.

Table 18. Dual Parametric Effective Depth Results - Benkleman Beam Site No. 1

Section	Effective Depth (in.) D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	9.5	15	13
2	6.5	8.5	--
3	7.5	12	12.5
4	9	9	9.5
5	7	18	10.5
6	4	7	3
7	4.5	9	7
8	4	24	--
9	4.75	--	9
10	7	5	6

Table 19. Dual Parametric Resilient Moduli Results - Benkleman Beam Site No. 1

	Resilient Modulus (Ksi)		
Section	E_{AC}	E_1	E_2
1	9.5	18	20
2	16	17	--
3	12	11	10
4	17	17	15
5	11	7	9
6	15	13	9
7	15	12	14
8	16	7	--
9	16	--	15
10	5	15	11

3.3 Analysis of Test Site 2, US59, Panola Co.

3.3.1 Dynaflect Analysis The respective lime-fly ash combinations used in the test sections are presented in Table 20. Also shown are the ages of the deflection surveys. The deflection data received from the dynaflect field surveys are presented in Tables B-7 through B-9, Appendix B. Each measurement represents the average of at least 20 field deflection measurements. The deflection basin parameters for each test section at the indicated time period are given in Tables 21 through 23.

Using the W_1 parameter as an indicator of the overall stiffness of the pavement system, it is seen that section 4 shows the least total deflection. Note that, except for section 9, all the sections show a very much reduced maximum deflection in comparison to section 1--the control section. Also note that the sections using a large percentage of fly ash show a larger percent decrease in W_1 for the second year than those using four to eight percent fly ash.

The spreadibility values listed show no signs of a deteriorating pavement system. The SCI parameters indicate the pavement system's upper layers are improving in load carrying capability. The BCI values either remain about the same or decrease. The control section shows the largest BCI reading. The section containing the largest percentage of fly ash has about the same final BCI parameter as section 2 containing a 2/4 lime-fly combination. The W_5 readings show that sections 1 through 4 appear to have reached their maximum value. However, sections 6 through 9 show a gradual decrease in W_5 with time.

Table 20. Lime- Fly Ash Combinations and Age of Deflection Surveys- Site No. 2.

Test Section	Lime/Fly Ash [a] (subgrade)	Age at Time of Test: As Const. (months)	1 Yr (months)	2 Yrs (months)
1	4/0	4	13	24
2	2/4	4	13	24
3	2/10	4	13	24
4	4/8	4	13	24
5	4/16	4	13	24
6	0/16	4	13	24
7	2/24	4	13	24
8	2/15	4	13	24
9	0/21	4	13	24
10	4/4	4	13	24

[a] Weight Percentages

Table 21. Dynaflect Deflection Basin Parameters- Site No.2 (3 month survey).

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.932	.200	.094	67	.384
2	2/4	.689	.202	.066	57	.193
3	2/10	.619	.141	.065	63	.209
4	4/8	.511	.127	.058	60	.153
5	4/16	.548	.121	.059	65	.209
6	0/16	.915	.204	.089	64	.335
7	2/24	.829	.180	.100	67	.332
8	2/15	.802	.161	.099	66	.300
9	0/21	-	-	-	-	-
10	4/4	-	-	-	-	-

[a] Weight Percentage

Table 22. Dynaflect Deflection Basin Parameters- Site No.2 (13 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.792	.197	.064	62	.282
2	2/4	.601	.179	.054	58	.174
3	2/10	.524	.136	.048	62	.182
4	4/8	.445	.120	.049	60	.131
5	4/16	.519	.136	.048	62	.181
6	0/16	.814	.236	.072	60	.262
7	2/24	.754	.173	.081	64	.277
8	2/15	.739	.200	.075	61	.243
9	0/21	1.274	.340	.112	61	.413
10	4/4	-	-	-	-	-

Table 23. Dynaflect Deflection Basin Parameters- Site No. 2 (24 month survey).

Test Section	Lime/Fly Ash [a] (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.808	.209	.087	63	.281
2	2/4	.593	.167	.066	59	.178
3	2/10	.534	.131	.049	63	.184
4	4/8	.422	.111	.045	60	.131
5	4/16	.477	.123	.035	63	.190
6	0/16	.690	.189	.057	61	.242
7	2/24	.630	.138	.062	66	.253
8	2/15	.646	.159	.072	63	.232
9	0/21	1.132	.264	.088	64	.435
10	4/4	-	-	-	-	-

[a] Weight Percentage

The Dynaflect data are presented graphically in Figures 24 and 25 for the nine test sections. The effective thickness and resilient modulus for each section, as taken from the dual parametric charts, are presented in Tables 24 and 25. From Table 24, it can be easily recognized how the parameters are responding in relation to one another. The most important item to note in Table 24 is how the sections compare to the control section, i.e., section 1 containing the 4/0 lime-fly ash combination. Section 7 is the most improved section using a 2/24 lime-fly ash combination. As seen in Table 25 the resilient modulus values range from 8,000 psi to 25,000 psi.

To test the significance of the lime and fly ash percentages in the stabilization process linear regression and an analysis of variance tests were performed.

The GLM test results with respect to the change in effective thickness are presented in Table 26. As seen from these results the lime-fly ash percentages significanty affect the change in effective thickness at the 0.005 significance level. However, in testing the lime-fly ash percentages with respect to the change in the resilient modulus over a two year period a 0.250 significance level was found.

The same data were used in the ANOVA test and the results are presented in Table 27. In reference to this table the ANOVA results indicate that both the lime and fly ash percentages are significant in the development of the effective thickness. Of the two variables the lime is shown to have the greater effect. The lime and fly ash percentages are found to be significant at a level of 0.250 and 0.100, respectively.

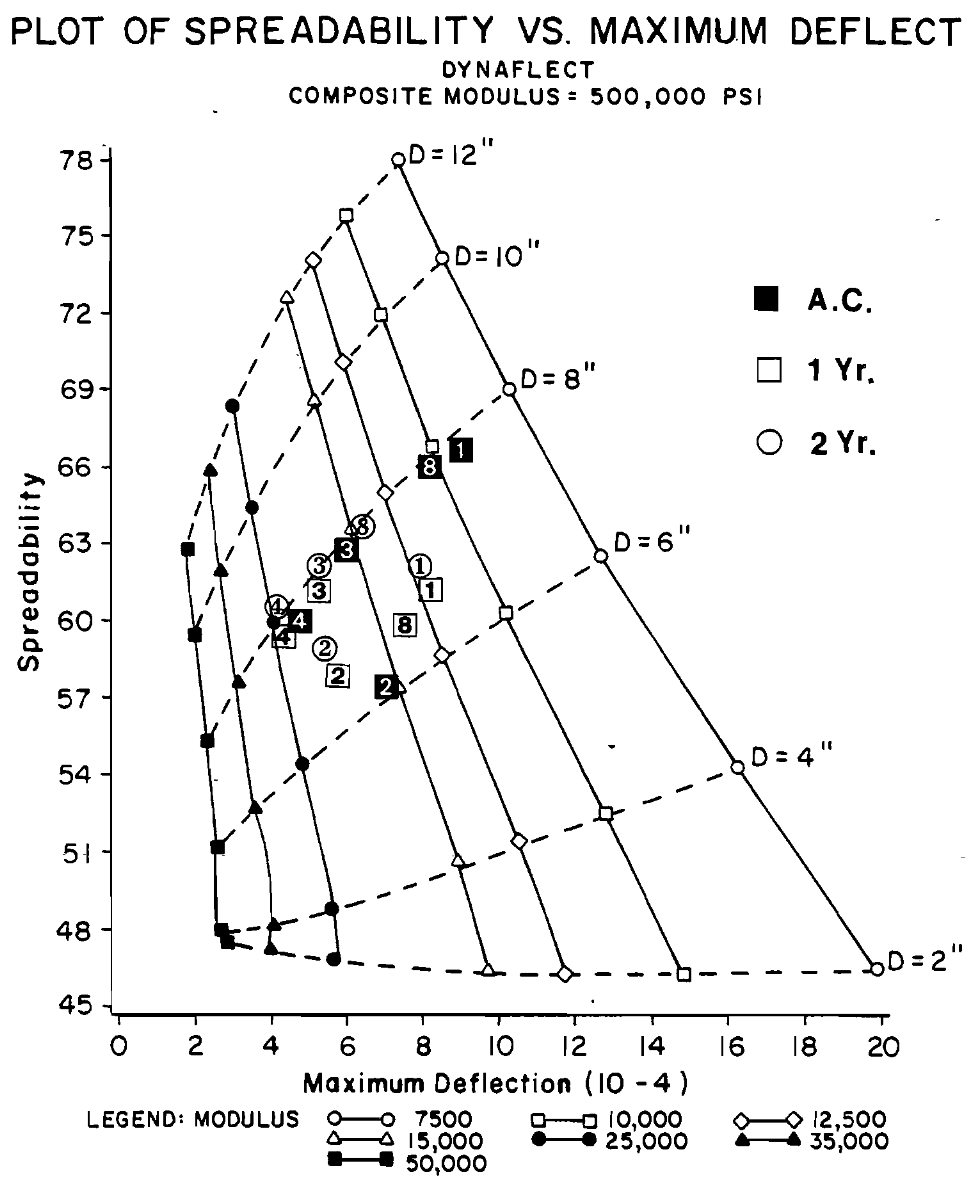


Figure 24. Dynaflect dual parametric chart--Site 2, Sections 1, 2, 3, 4 and 8.

PLOT OF SPREADABILITY VS. MAXIMUM DEFLECTION
DYNAFLECT
COMPOSITE MODULUS = 500,000 PSI

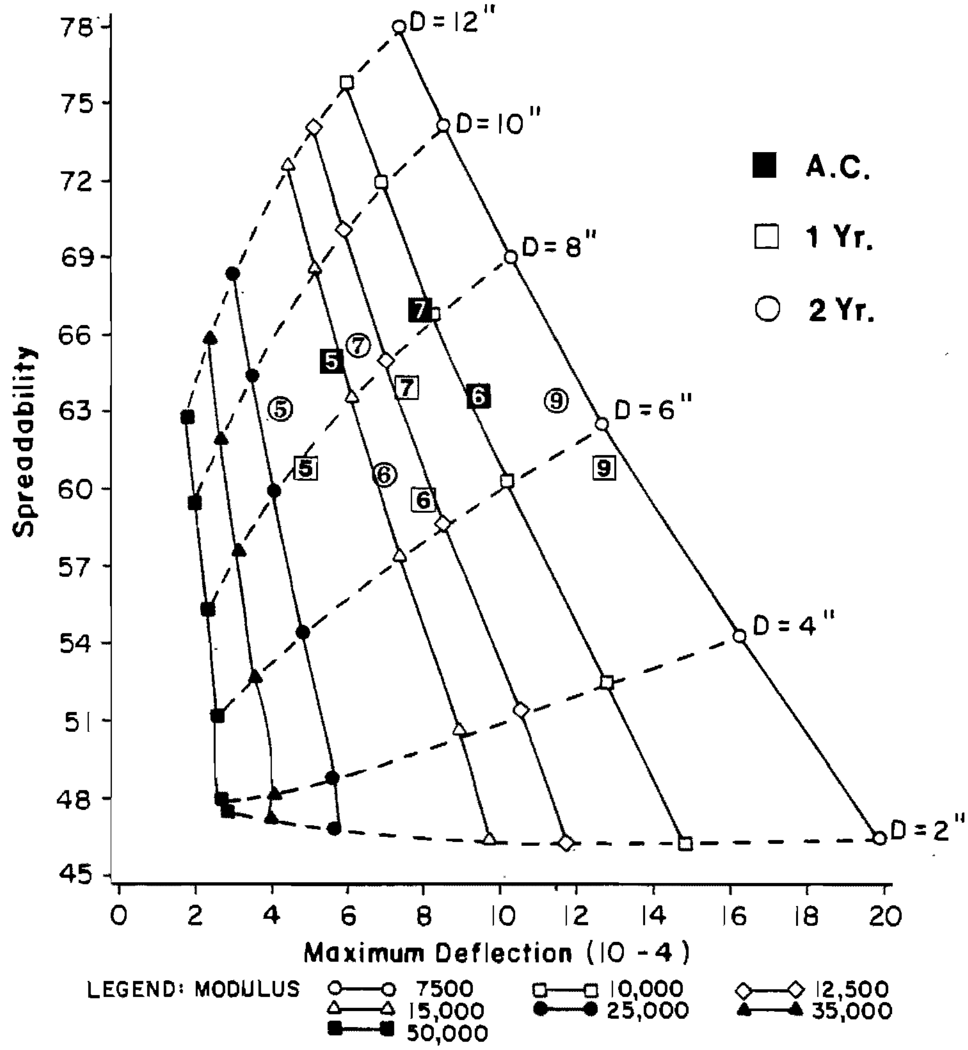


Figure 25. Dynaflect dual parametric chart--Site 2, Sections 5, 6, 7 and 9.

Table 24. Dynaflect Dual Parametric Effective Depth Results-
Site No. 2

Test Section	Lime/Fly Ash[a] (subgrade)	Effective Depth (in.)		
		D_{AC}	$D_{1\ yr.}$	$D_{2\ yr.}$
1	4/0	7.75	7	7
2	2/4	6	7	6.5
3	2/10	7.5	8	8
4	4/8	7.5	8	8
5	4/16	8.5	8	8.5
6	0/16	7	6.5	7
7	2/24	8	7.5	8.5
8	2/15	8	7	8
9	0/21	-	5.5	6.5
10	4/4	-	-	-

[a] Weight Percentage

Table 25. Dynaflect Dual Parametric Resilient Modulus Results-Site No. 2

Test Section	Lime/ Fly Ash[a] (subgrade)	Resilient Modulus (psi) E_{AC}	$E_{1\ yr.}$	$E_{2\ yr.}$
1	4/0	9000	12000	12000
2	2/4	15000	20000	20000
3	2/10	15000	20000	20000
4	4/8	20000	25000	25000
5	4/16	15000	20000	20000
6	0/16	10000	12500	15000
7	2/24	10000	12500	14000
8	2/15	11000	13000	14000
9	0/21	-	8000	8000
10	4/4	-	-	-

[a] Weight Percentage

Table 26. General Linear Model Regression Analysis- Site No. 2 (Based on Dynaflect Dual Parametric Results)

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from the AC stage to the 2 yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	26.17	7.21	0.005
Fly Ash	10.07	4.66	0.005

Testing the Significance of the Lime and Fly Ash Percentages on the Change in Resilient Modulus from the AC stage to the 2 yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	1.98	1.50	0.250
Fly Ash	1.92	1.45	0.025

Table 27. Analysis of Variance Statistical Test Results- Site No. 2

Testing for the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from AC stage to 2 yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	26.17	7.21	0.005
Fly Ash	16.02	4.44	0.005

Testing for the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from AC stage to 2 yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	1.98	1.50	0.250
Fly Ash	2.14	2.08	0.100

In summary, the Dynaflect deflection data and the dual parametric charts generally show that the fly ash used in the test sections proved significant in the development of the subgrade's stiffness. This finding is supported by the statistical analyses.

3.3.2 <u>Benkleman Beam Analysis</u> Tables of the average deflections for the nine test sections are presented in Tables B-10 through B-12, Appendix B. The data represent deflection measurements taken over the two year study period. Sections 2, 3 and 4 show a slight increase in W_1 but yet did not exceed the as constructed measurement. Section 7 shows an increase in W_1 for the second year, but it results in a maximum deflection of only 11 percent higher than when constructed.

It should be noted that a two inch overlay was placed on the test site in the 2 year interim period. Thus, none of the measurements are comparable to the first year's deflection data. But, information taken during the first year will provide some indication of the beneficial aspects of lime-fly ash stabilization.

It is apparent from the tables that the stabilization process has in part, been responsible for the decrease in W_2. The total pavement system's relative stiffness is increasing in all but section 3. Note that section 5 greatly increased in stiffness in the initial phase and the similarity that exists between section 1 and sections 3, 6 and 7.

Figures 26 and 27 display the W_1 and S values, previously presented in Tables B-10 through B-12, graphically on the dual parametric charts. From these charts the effective thickness and the resilient modulus values were interpolated and are summarized in

Tables 28 and 29. Referring to Table 29 containing the resilient modulus values a comparison can be made among the test sections. The first item to note is the identical behavior of sections 1 and 7. The second item that needs mentioning is the 5,000 to 6,000 psi gain in resilient modulus of sections 2, 3 and 8. Note that section 3 (2/10) and section 8 (2/15) have similar moduli in both deflection recording configurations (2 and 3 ft. offsets). The data presented in Table 28 indicate that in all instances the effective thickness of the pavement layers are increasing in magnitude.

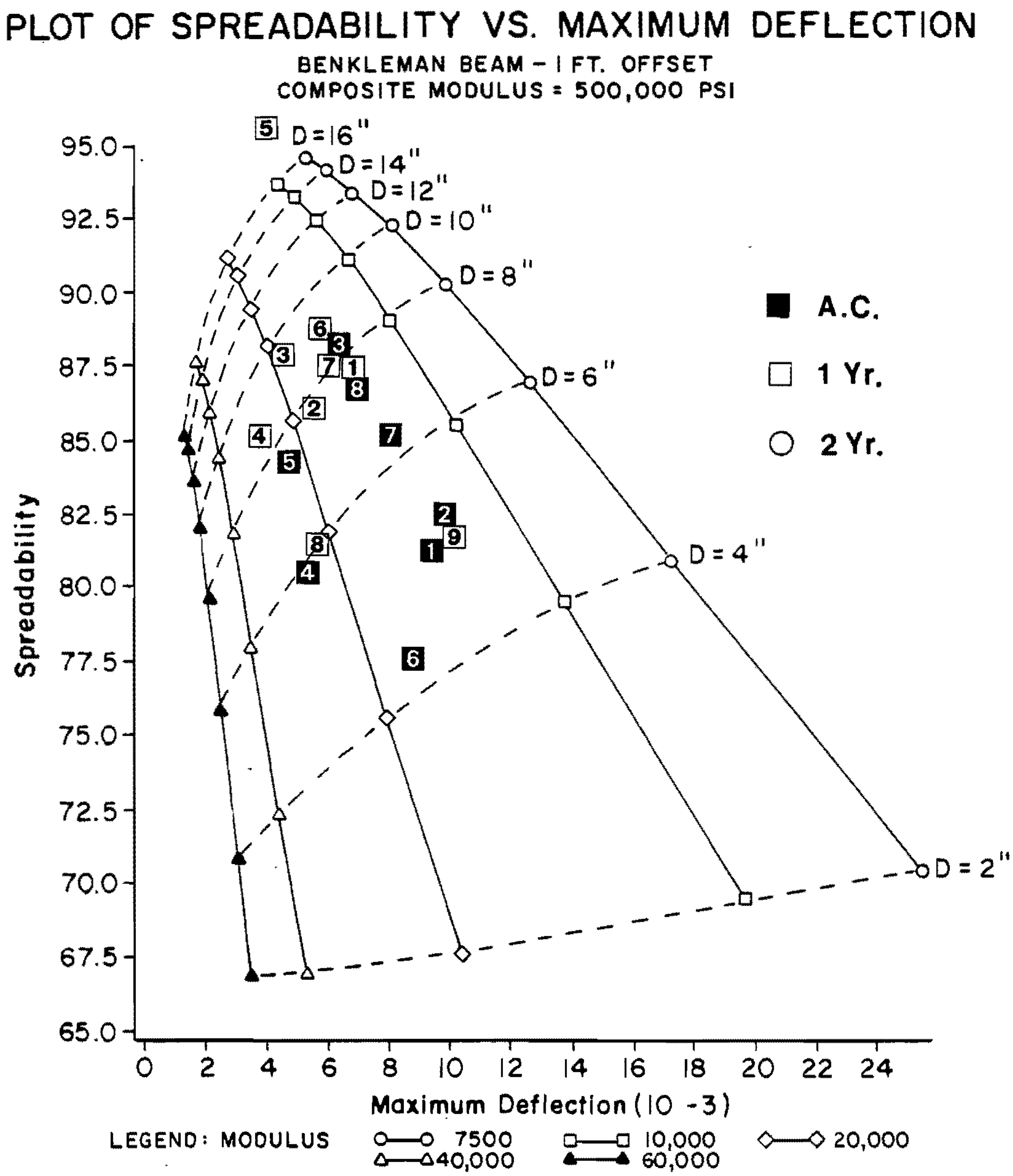


Figure 26. Benkleman beam dual parametric chart--Site 2, Sections 1, 2, 3, 4, 5, 6, 7, 8 and 9.

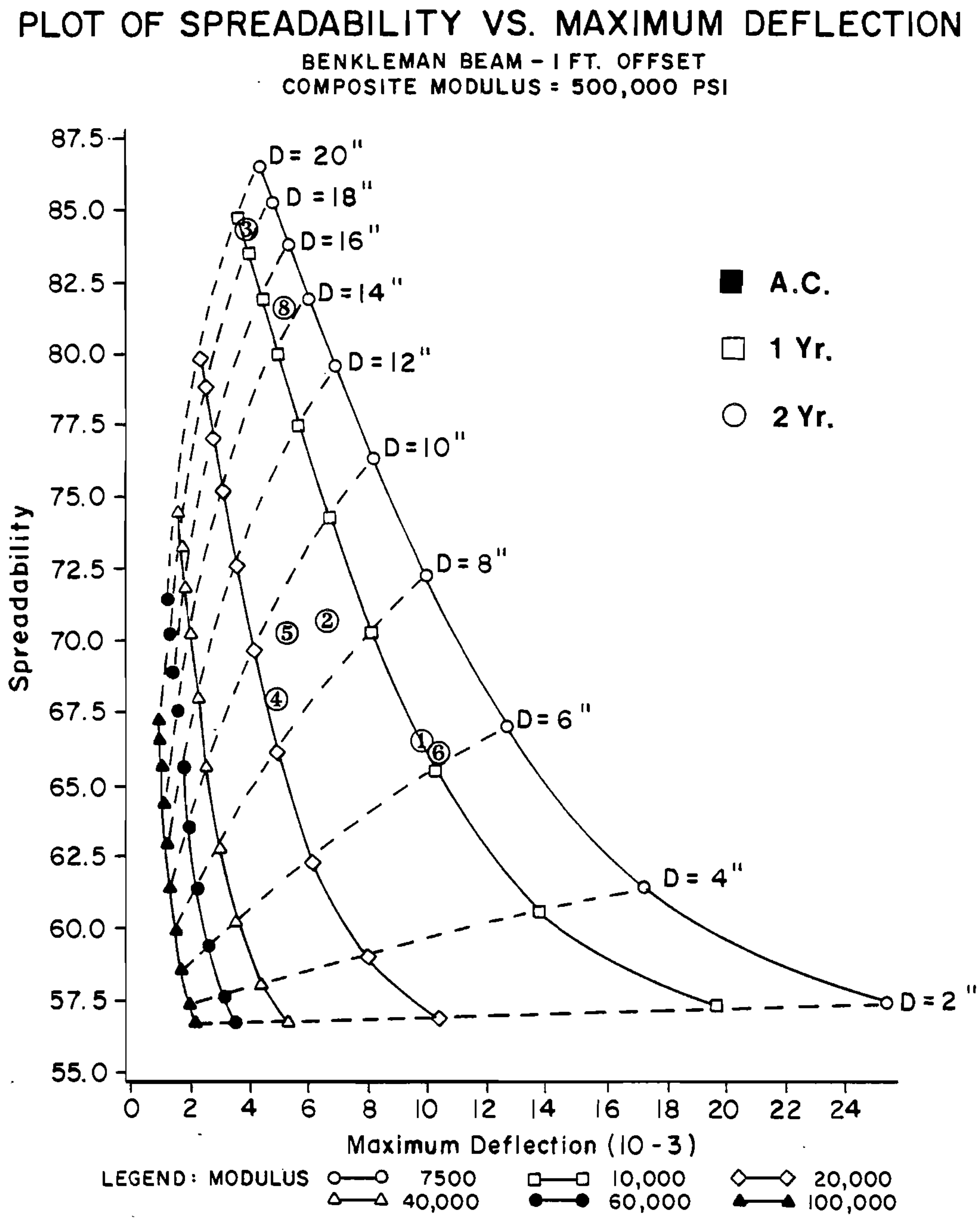


Figure 27. Benkleman beam dual parametric chart (3 ft. offset)--Site 2, Sections 1, 2, 3, 4, 5, 6 and 8.

Table 28. Benkleman Beam Dual Parametric Effective Depth Results- Site No. 2

Test Section	Lime/Fly Ash [a] (subgrade)	Effective Depth (in.) D_{AC}	$D_{1\ yr.}$	$D_{2\ yr.}$
1	4/0	5	8	6.5
2	2/4	5.5	8	9
3	2/10	8	9	19
4	4/8	6	8.5	9
5	4/16	7.5	23	9.5
6	0/16	4.5	9	6
7	2/24	6.5	8	10
8	2/15	7	6	14
9	0/21	-	5.5	13
10	4/4	-	-	-

[a] Weight Percentage

Table 29. Benkleman Beam Dual Parametric Resilient Modulus Results- Site No. 2

Test Section	Lime/Fly Ash[a] (subgrade)	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ yr.}$	$E_{2\ yr.}$
1	4/0	15000	15000	10000
2	2/4	12000	18000	13000
3	2/10	15000	20000	10000
4	4/8	25000	25000	20000
5	4/16	25000	7000	16000
6	0/16	17000	15000	10000
7	2/24	15000	15000	7000
8	2/15	15000	20000	8500
9	0/21	-	12000	7000
10	4/4	-	-	-

[a] Weight Percentage

3.4 Analysis of Test Site 3, FM 1604, Bexar Co.

3.4.1 Dynaflect Analysis The respective lime-fly ash combinations that were used in the test sections are presented in Table 30. Also shown are the age of the pavement sections at the time the deflection surveys were conducted. Tables B-13 through B-15 in Appendix B contain the average of about 20 field measurements taken during the two year study period. The deflection basin parameters are listed in Tables 31 through 33. Comparing the W_1 values listed, it would appear that section 4 has the greatest load carrying capability. A point of interest is that even without any lime section 6 has a lower maximum deflection than two sections using both lime and fly ash (sections 1 and 2).

Also presented are the spreadibility values. From these data it is seen that sections 4 and 6 are superior. Note too that except for section 3, the S values are basically equal throughout the test site. This would indicate that all of the pavement sections within the test site possess about the same load distributing characteristics. This is in contrast to the W_1 measurements which had indicated that section 4 was clearly the most structurally sound.

To evaluate the top layers the SCI parameters are considered. These values indicate that the top layers are improving in stiffness. The similarities between sections 4 and 6 are still present. Something that is of interest is that although sections 1, 2, 3 and 5 had initial increases in SCI the subsequent year brought about a reduction in magnitude. This reduction was such that at the 2 year period the SCI values were still less than the initial values.

Table 30. Lime-Fly Ash Combinations and Age of Deflection Surveys- Site No.3

Test Section	Lime/Fly Ash[a] (subgrade)	Age at time of Test		
		As Const. (months)	1 Yr (months)	2 Yrs (months)
1	3/6	7	12	19
2	3/10	7	12	19
3	2/5	7	12	19
4	4/0	7	12	19
5	2/8	7	12	19
6	0/12	7	12	19

a Weight Percentage

Table 31. Dynaflect Deflection Basin Parameters- Site No.3 (7 month survey).

Test Section	Lime/Fly Ash [a] (subgrade)	W_1	SCI	BCI	S	W_5
1	3/6	.693	.168	.043	61	.247
2	3/10	.725	.176	.059	59	.217
3	2/5	.579	.131	.056	62	.190
4	4/0	.539	.124	.038	66	.231
5	2/8	.599	.137	.060	61	.193
6	0/12	.662	.137	.051	64	.241

[a] Weight Percentage

Table 32. Dynaflect Deflection Basin Parameters- Site No. 3 (12 month survey).

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	3/6	.574	.173	.039	56	.158
2	3/10	.597	.186	.039	53	.132
3	2/5	.502	.134	.036	57	.130
4	4/0	.460	.119	.028	61	.164
5	2/8	.515	.142	.035	57	.143
6	0/12	.532	.110	.043	63	.177

[a] Weight Percentage

Table 33. Dynaflect Deflection Basin Parameters- Site No. 3 (19 month survey).

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	3/6	.579	.136	.049	61	.176
2	3/10	.609	.148	.055	57	.150
3	2/5	.491	.103	.046	61	.144
4	4/0	.445	.081	.036	67	.178
5	2/8	.519	.114	.041	61	.156
6	0/12	.556	.084	.057	67	.202

[a] Weight Percentage

In checking the attribute of the lower layers the BCI parameters are referenced. From Tables 31 through 33 it can be seen that although all test sections initially experienced a decrease in magnitude, with time an increase in magnitude occured.

The dual parametric method using the spreadibility and maximum deflection parameters is presented in Figures 28 and 29. From these figures the effective thickness of the pavement layer and the resilient modulus of the subgrade were interpolated and are presented in Tables 34 and 35. In this analysis sections 4 and 6 are still the most improved of the six test sections. Only section 5 has a lower effective thickness at the 2 year period in relation to the initial AC measurement. One other item of importance is the fact that section 6 is the only section that did not experience a decrease in effective thickness between the as constructed and one year period. However, when the resilient moduli are compared it is apparent that section 6 shows distress in the subgrade. Except for sections 1 and 6, all of the other sections have the same resilient modulus at the end of the two year period. Of these sections only section 2 has shown to be improving with time.

The results of the GLM regression procedure are shown in Table 36. No statistical significance can be found up to a level of 0.250 investigating the effect of the percentages on the change in effective thickness. The results from the ANOVA investigating the change in the resilient modulus shows that a significance level greater than 0.250 was found for the lime percentage, Table 37. The results from the ANOVA procedure also shows that a significance level of 0.250 was found for the variation in fly ash percentages.

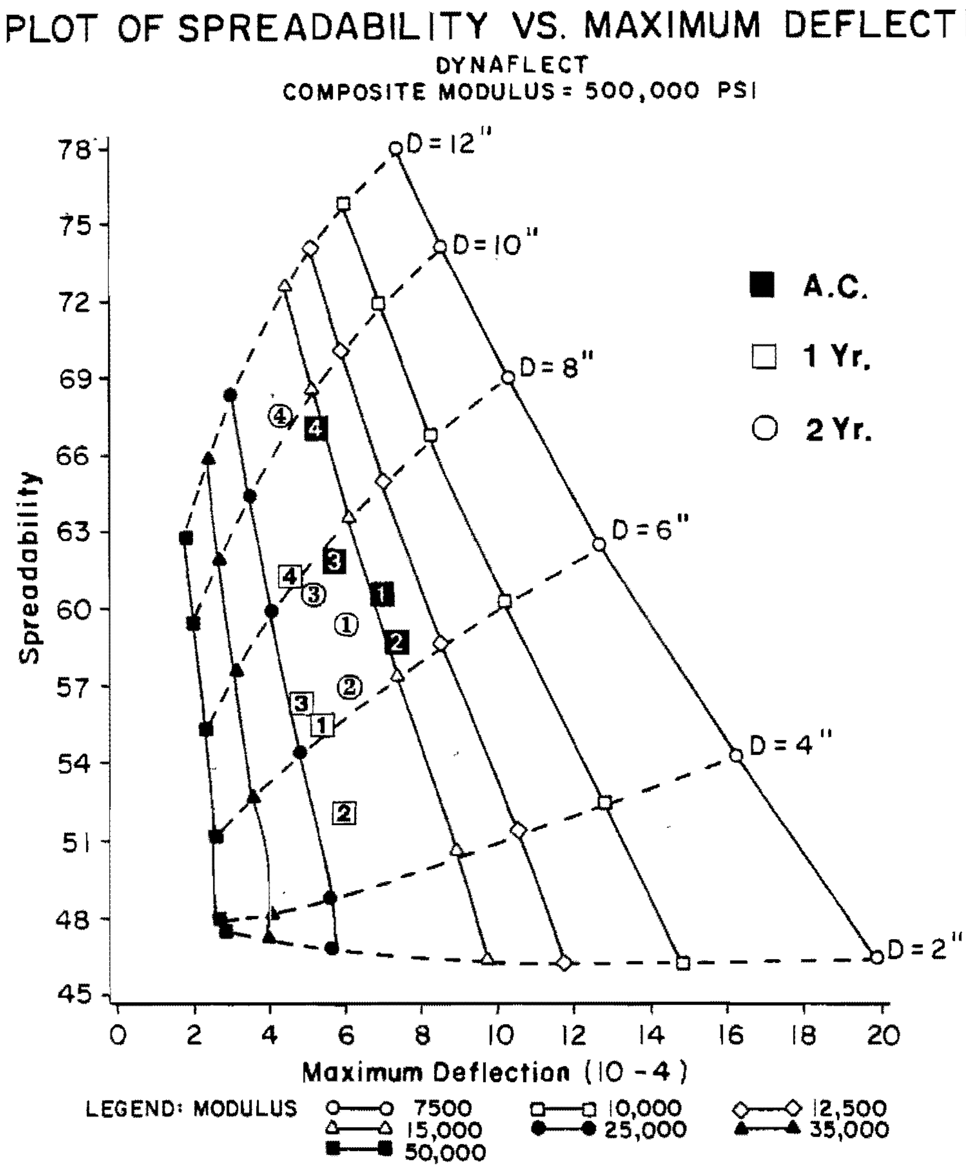


Figure 28. Dynaflect dual parametric chart--Site 3, Sections 1, 2, 3 and 4.

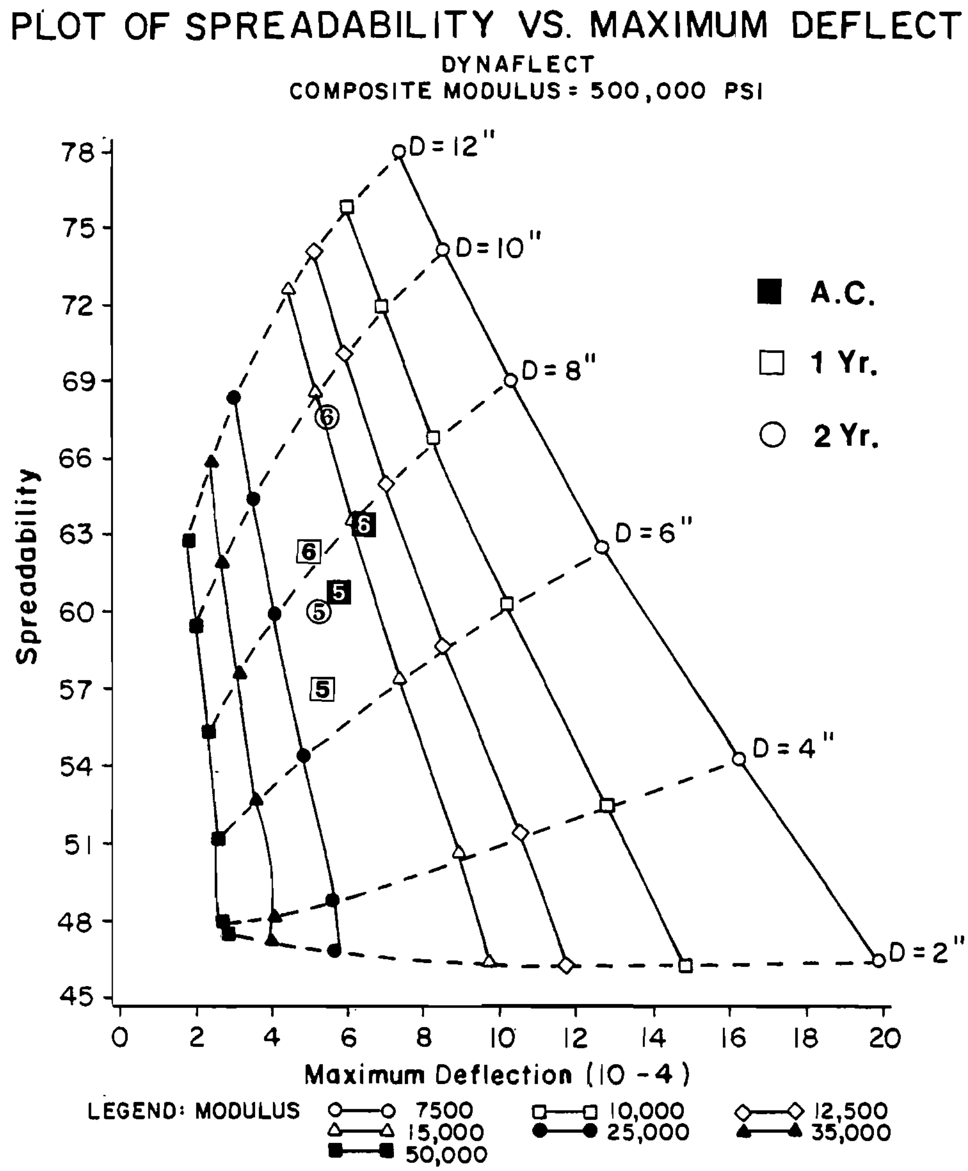


Figure 29. Dynaflect dual parametric chart--Site 3, Sections 5 and 6.

Table 34. Dynaflect Dual Parametric Effective Depth Results-Site No. 3

Test Section	Lime/Fly Ash [a] (subgrade)	Effective Depth (in.)		
		D_{AC}	$D_{1\ yr.}$	$D_{2\ yr.}$
1	3/6	7	6	7
2	3/10	6.5	5	6.5
3	2/5	7.5	6.5	8
4	4/0	9	8	10
5	2/8	7.5	6.5	6.5
6	0/12	8	8	9

[a] Weight Percentage

Table 35. Dynaflect Dual Parametric Resilient Modulus Results-
Site No. 3

Test Section	Lime/Fly Ash[a] (subgrade)	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ yr.}$	$E_{2\ yr.}$
1	3/6	15000	20000	16000
2	3/10	15000	20000	20000
3	2/5	16000	24000	20000
4	4/0	15000	24000	20000
5	2/8	16000	23000	20000
6	0/12	15000	17000	15000

[a] Weight Percentage

Table 36. General Linear Model Regression Analysis- Site No. 3

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from AC stage to 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	1.09	*	>0.25
Fly Ash	0.90	*	>0.25

Testing the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from AC stage to 2 Yr. stage

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.07	*	>0.25
Fly Ash	2.04	1.56	0.25

Note: Asterisks indicate negligible values of statistical parameters

Table 37. Analysis of Variance Statistical Test- Site No. 3

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth From AC stage to 2 Yr. Stage

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	1.09	*	>0.25
Fly Ash	1.01	*	>0.25

Testing the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from AC stage to 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.07	*	>0.25
Fly Ash	0.86	*	>0.25

Note: Asterisks indicate negligible values of statistical parameters

3.4.2 <u>Benkleman Beam Analysis</u> The field deflection measurements are given in Tables B-16 and B-17, Appendix B. Note that no as constructed data were obtained and the two year data was obtained with the three foot offset configuration. Thus, no comparison is possible between spreadibility values from one year stage to the two year data as the maximum deflection increases in all but one section. Section 4, a 4/0 lime-fly ash combination, shows the only real reduction in W_1. The only significant increase in the maximum deflection occurs in section 6 which experienced a two-fold increase in magnitude.

Since the dual parametric charts are a function of the spreadibility they too cannot be compared against one another. The dual parametric results are presented in Figures 30 and 31. The resulting resilient modulus values and effective thicknesses interpolated from these figures are shown in Tables 38 and 39. The resilient modulus results show that with either measuring configuration the sections using a conservative lime to fly ash combination (sections 1, 3 and 5) ultimately have the highest resilient modulus values. Even the all fly ash section (section 6) has either an equivalent or greater resilient modulus result when compared to the control section (section 4), indicating excellent performance to date.

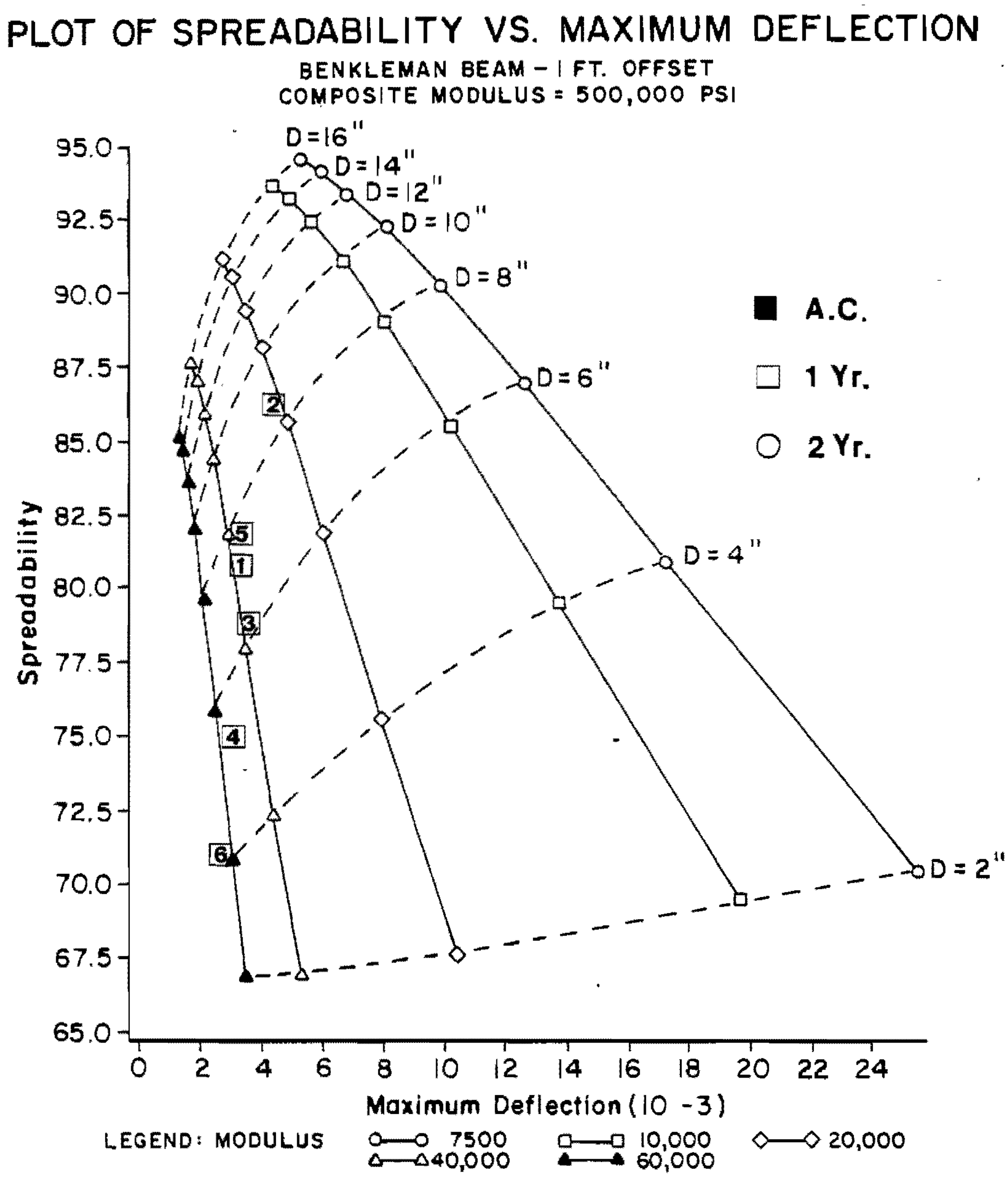


Figure 30. Benkleman beam dual parametric chart--Site 3, Sections 1, 2, 3, 4, 5 and 6.

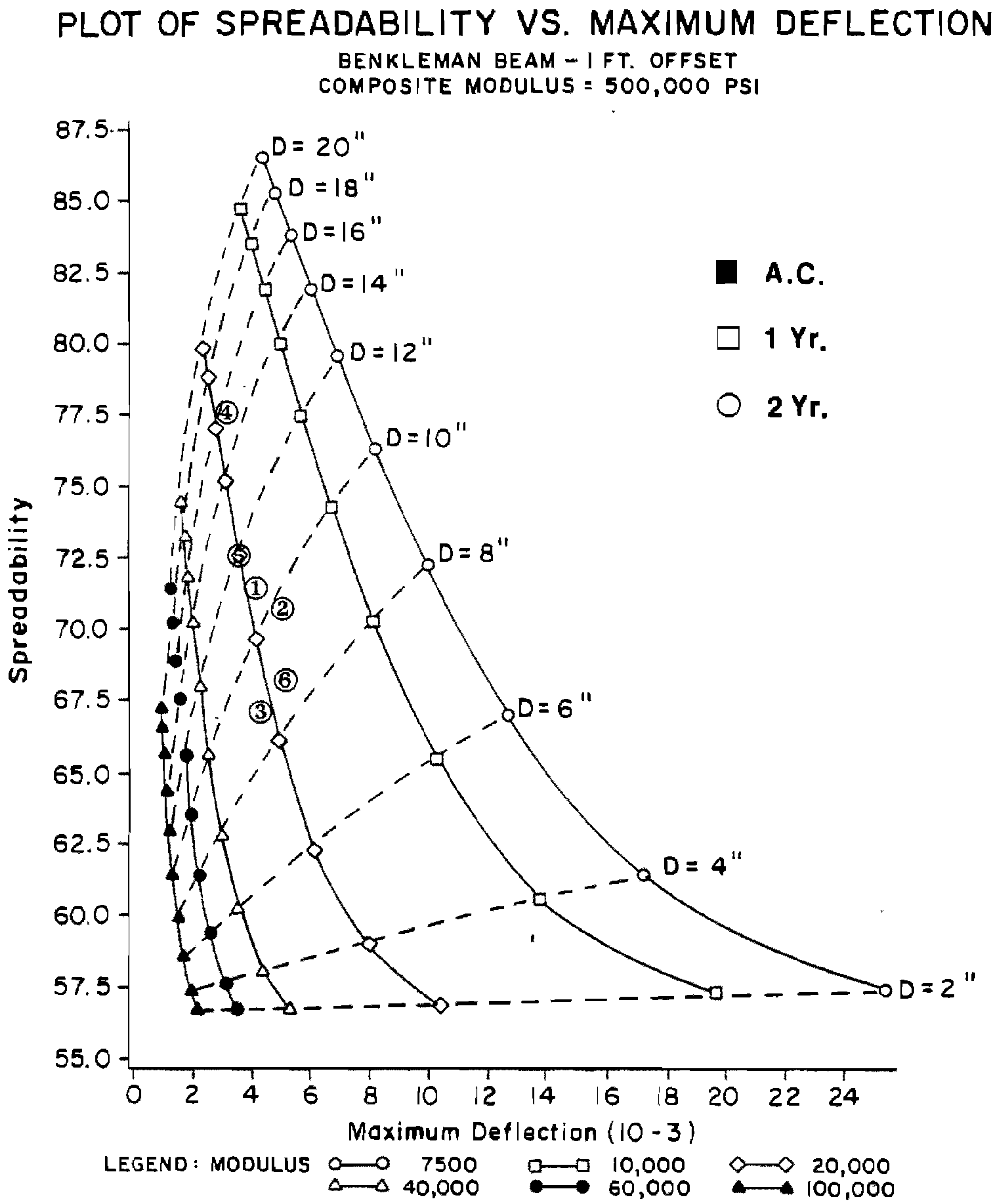


Figure 31. Benkleman beam dual parametric chart (3 ft. offset)--Site 3, Sections 1, 2, 3, 4, 5 and 6.

Table 38. Benkleman Beam Dual Parametric Effective Depth Results
Site No.3

Test Section	Lime/Fly Ash [a] (subgrade)	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	3/6	-	7	11
2	3/10	-	9	10
3	2/5	-	6	9
4	4/0	-	5	16
5	2/8	-	8	12
6	0/12	-	4	8.5

[a] Weight Percentage

Table 39. Benkleman Beam Dual Parametric Resilient Modulus Results- Site No. 3

Test Section	Lime/Fly Ash [a] (subgrade)	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	3/6	-	40000	20000
2	3/10	-	20000	17000
3	2/5	-	40000	20000
4	4/0	-	50000	17000
5	2/8	-	40000	20000
6	0/12	-	60000	17000

[a] Weight Percentage

3.5 Analysis of Test Site 4, FM 1604, Bexar Co.

3.5.1 Dynaflect Analysis The lime fly ash combinations for each test section along with the actual age of the pavement systems are presented in Table 40. All of the field deflection data was reduced and the average of no less than 20 field measurements are shown in Tables B-18 through B-20, Appendix B. The deflection basin parameters are listed in Tables 41 through 43, which show the control section (section 1) to consistently have the lowest W_1 reading. However, all of the test sections show approximately a two-fold increase in maximum deflection during the first year. It is only during the second year that an increase is recognized. Also, despite the fact that sections 1 and 2 are among the lowest in W_1 readings they, and section 5, show an increasing trend in maximum deflection.

The spreadibility parameters indicate that all of the pavement systems increased in rigidity during the initial year and have remained about the same up to the end of the second year.

The SCI parameters indicate that the top of the pavement section dramatically increased in rigidity in the two year period of this study. To determine what attributes the stabilization process contributed to the prepared subgrade layer note that the BCI values dramatically increased in the first year in all but sections 1 and 2. The second year shows all sections responding with an increase in BCI. From these data alone it would appear that sections 1 and 5 benefited most from the lime-fly ash stabilization. In the second year sections 3, 4 and 6 seem to remain intact while sections 1, 2 and 5 lost some of their rigidity.

Table 40. Lime-Fly Ash Combinations and Age of Deflection Surveys- Site No. 4.

Test Section	Lime/Fly Ash[a] (subgrade)	Age at time of Test As Const. (months)	1 yr (months)	2 Yr (months)
1	4/0	6	12	24
2	3/6	6	12	24
3	3/9	6	12	24
4	0/10	6	12	24
5	1/5	6	12	24
6	2/8	6	12	24

[a] Weight Percentage

Table 41. Dynaflect Deflection Basin Parameters - Site No. 4 (6 month survey).

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.988	.386	.064	54	.288
2	3/6	1.054	.447	.064	51	.266
3	3/9	1.200	.512	.017	48	.265
4	0/10	1.498	.712	.011	47	.303
5	1/5	1.273	.597	.012	47	.287
6	2/8	1.255	.522	.015	51	.308

[a] Weight Percentage

Table 42. Dynaflect Deflection Basin Parameters- Site No. 4 (12 month survey)

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.493	.058	.027	75	.264
2	3/6	.537	.080	.035	69	.235
3	3/9	.634	.130	.038	64	.246
4	0/10	.792	.206	.043	60	.274
5	1/5	.686	.181	.030	62	.264
6	2/8	.643	.108	.033	69	.292

[a] Weight Percentage

Table 43. Dynaflect Deflection Basin Parameters- Site No. 4 (24 month survey).

Test Section	Lime/Fly Ash[a] (subgrade)	W_1	SCI	BCI	S	W_5
1	4/0	.530	.080	.030	73	.260
2	3/6	.570	.108	.043	67	.232
3	3/9	.594	.134	.044	64	.225
4	0/10	.769	.213	.052	60	.254
5	1/5	.703	.215	.034	59	.242
6	2/8	.624	.124	.044	68	.271

[a] Weight Percentage

The dynaflect deflection basin parameters W_1 and S are plotted on the dual parametric chart, Figure 32. The interpreted effective depth and resilient moduli for each section are presented in Tables 44 and 45. All sections experienced an increase in effective depth. It is the second year, however, which differentiates those sections responding the most to the stabilization process. Sections 3, 4 and 6 seem to remain intact while sections 1, 2 and 5 lose some of their effectiveness. The resilient modulus of the subgrade (Table 45) indicates an almost identical gain for sections 4 and 6. Also, each test section using fly ash exhibits a higher resilient modulus than the control section. Even section 4, which used only fly ash (without any lime) has a higher second year reading.

A linear regression and an analysis of variance were made on data taken from the dual parametric charts. Table 46 contains the test results for the GLM. The lime percentages were found to be significant in the changes experienced by the effective thickness parameter. The effects of flyash percentages were observed to be insignificant. The effect of the percentages on the change in resilient modulus is also given in Table 46 for the GLM evaluation. The results show that neither the lime or fly ash percentages are significant to a 0.250 level of significance.

The ANOVA results, Table 47, show that the variations in the lime and fly ash percentages are significant at a level of 0.005. The results of the evaluation testing the variations of the stabilization agents on the change in resilient modulus shows that no significance can be found below a 0.250 level.

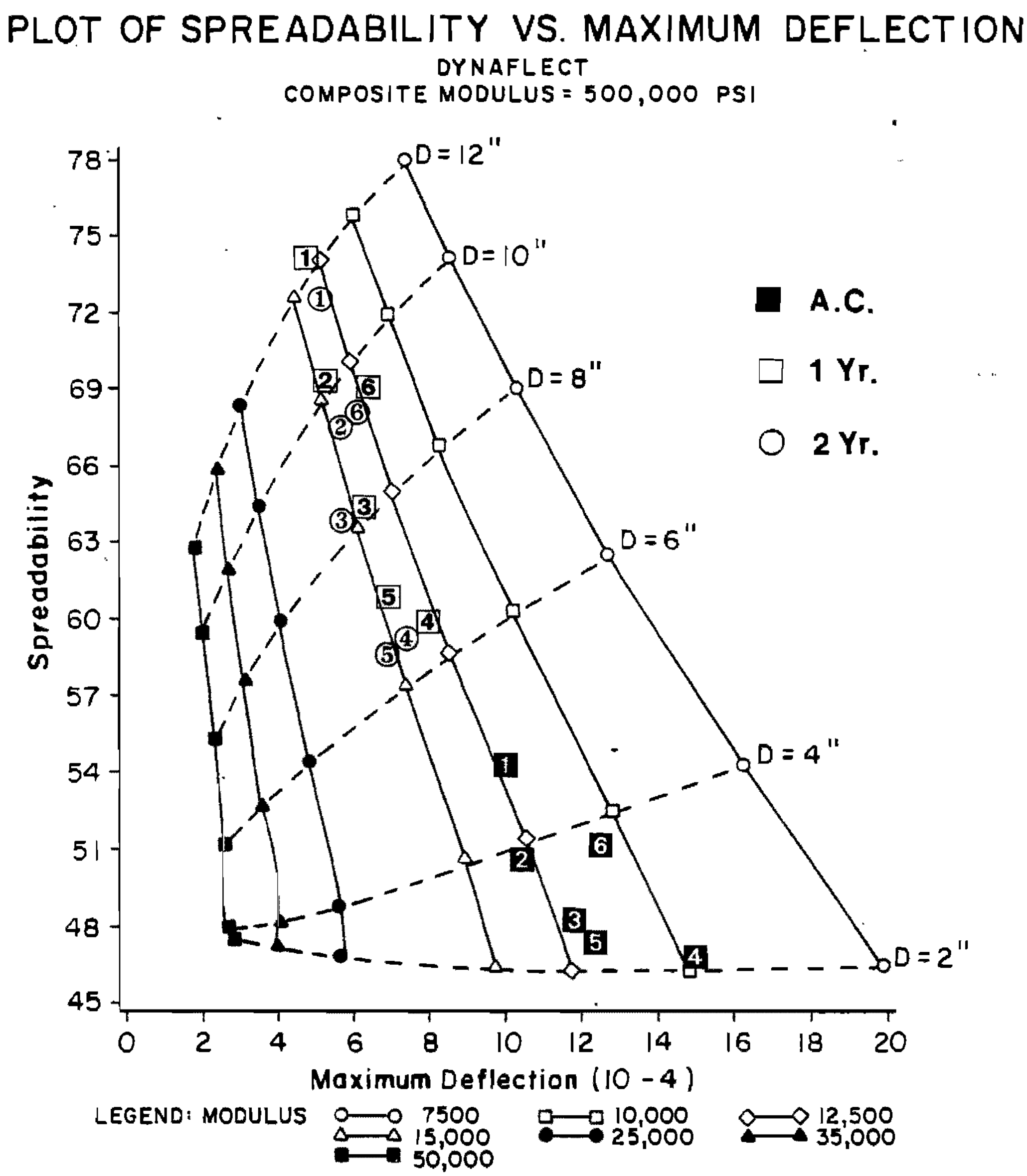


Figure 32. Dynaflect dual parametric chart--Site 4, Sections 1, 2, 3, 4, 5 and 6.

Table 44. Dynaflect Dual Parametric Effective Depth Results-
Site No. 4

Test Section	Lime/Fly Ash[a] (subgrade)	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	4/0	5	12	11
2	3/6	4	10	9
3	3/9	3	8	8
4	0/10	2	6.5	6.5
5	1/5	2.5	7	6.5
6	2/8	3.5	9	9

[a] Weight Percentage

Table 45. Dynaflect Dual Parametric Resilient Modulus Results-Site No. 4.

Test Section	Lime/Fly Ash[a] (subgrade)	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	4/0	12500	12500	12500
2	3/6	12500	15000	15000
3	3/9	12000	15000	15000
4	0/10	10000	12500	13000
5	1/5	11500	15000	15000
6	2/8	11000	12500	13000

[a] Weight Percentage

Table 46. General Linear Model Regression Analysis- Site No. 4

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	8.70	6.52	0.005
Fly Ash	0.15	*	>0.25

Testing the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.71	*	>0.25
Fly Ash	0.01	*	>0.25

Note: Asterisks indicate negligible values of statistical parameters

Table 47. Analysis of Variance Statistical Test- Site No. 4

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	8.70	6.52	0.005
Fly Ash	6.99	6.07	0.005

Testing the Significance of Lime And Fly Ash Percentages on the Change in Resilient Modulus from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.72	*	>0.25
Fly Ash	0.58	*	>0.25

Note: Asterisks indicate negligible values of statistical parameters

3.5.2 Benkleman Beam Analysis The benkleman beam field deflection data are presented in Tables B-21 through B-23, Appendix B. The actual age of the pavement systems are also listed in these tables. Each deflection value represents an average of at least 20 field measurements. The maximum deflections are seen to decrease for all but one section during the first year. The second year indicates that sections 3 and 6 are not losing their load carrying capability. One should be concerned with sections 1, 2 and 5. These sections reflect a greater deflection than when just constructed. An interesting fact is the amount of decrease that sections 4 and 6 display during the first year. Section 4 had a decrease in the first year of 22 thousandths of an inch, but yet used no lime in the stabilization process. The spreadibility values would indicate that section 4 is gradually losing some of its rigidity. While during the just constructed phase it was among the highest in S magnitude, by the end of the first year it possessed the least spreadibility value.

The benkleman beam deflection basin parameters S and W_1 are plotted in Figures 33 and 34. The resulting effective thickness and resilient modulus values are listed in Tables 48 and 49. From the effective depth results it can be seen that the control section exhibited the greatest effective thickness. Note too that the all fly ash section is shown to only decrease in effective depth during the first year. Section 3, a 3/9 lime-fly ash combination, shows the greatest increase in the first year. The resilient modulus values in Table 49 show the control section to lose half of its initial value while the all fly ash section gain almost a two-fold increase.

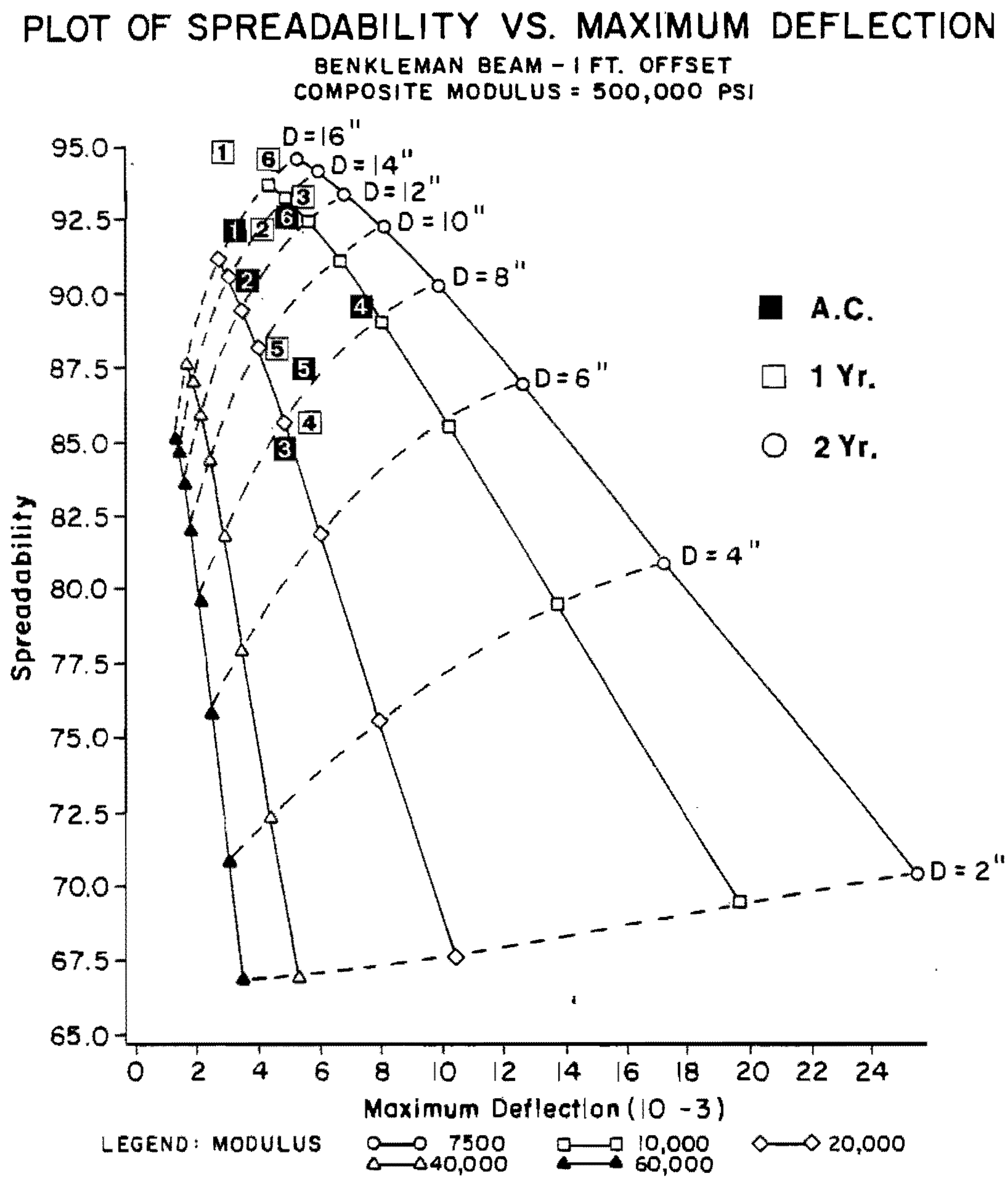


Figure 33. Benkleman beam dual parametric chart--Site 4, Sections 1, 2, 3, 4, 5 and 6.

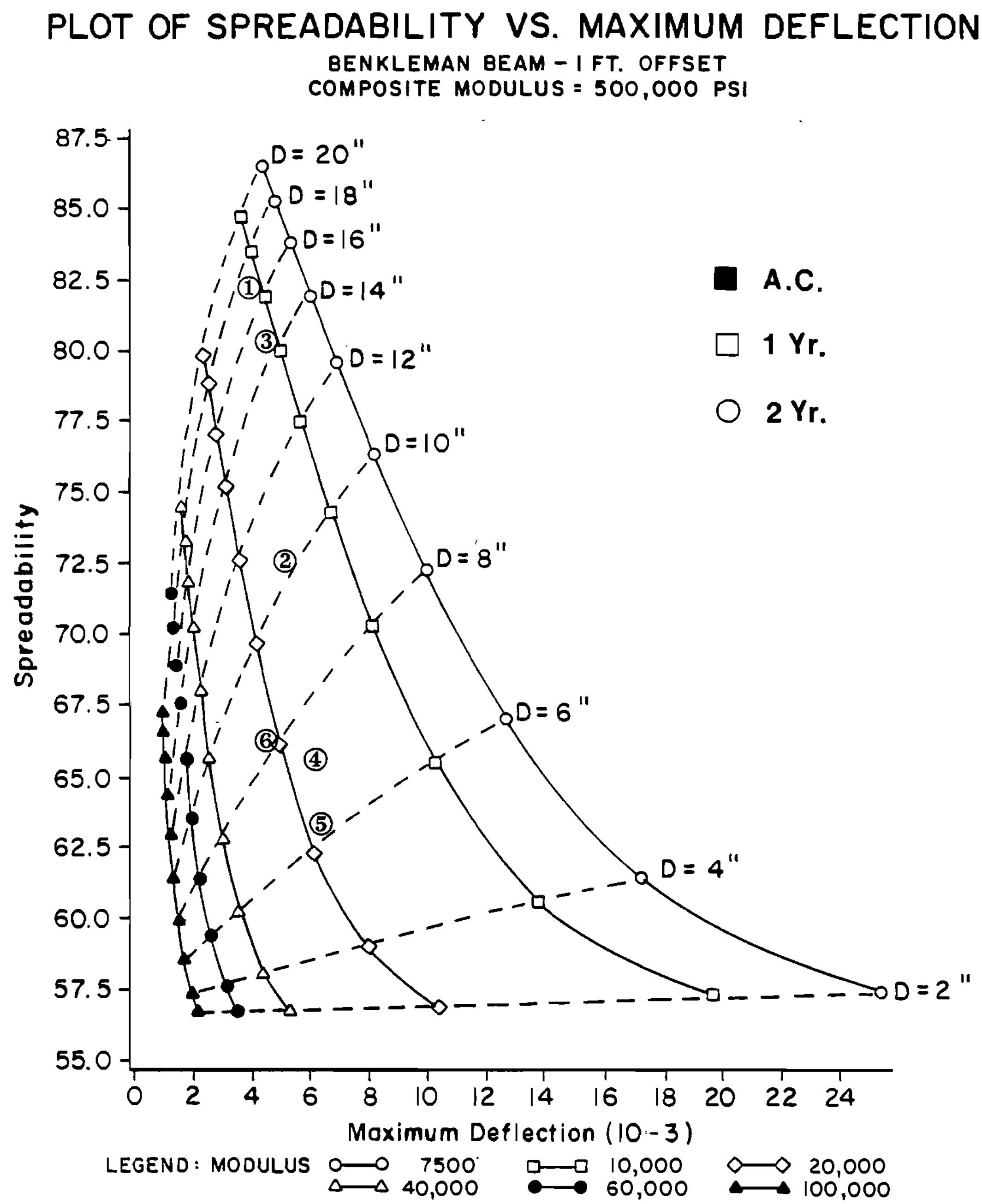


Figure 34. Benkleman beam dual parametric chart (3 ft. offset)--Site 4, Sections 1, 2, 3, 4, 5 and 6.

Table 48. Benkleman Beam Dual Parametric Effective Depth Results- Site No. 4

Test Section	Lime/Fly Ash [a] (subgrade)	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	4/0	16	20	17
2	3/6	12	13	12.5
3	3/9	7	15	15
4	0/10	9	7.5	7
5	1/5	8	9.5	6
6	2/8	13	18	8

[a] Weight Percentage

Table 49. Benkleman Beam Dual Parametric Resilient Modulus Results- Site No. 4.

Test Section	Lime/Fly Ash (subgrade)	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	4/0	17000	8000	11000
2	3/6	18000	15000	16000
3	3/9	20000	10000	10000
4	0/10	10000	18000	16000
5	1/5	17000	18000	19000
6	2/8	11000	7500	20000

[a] Weight Percentage

3.6 Analysis of Test Site 5, FM 1604, Bexar County

3.6.1 Dynaflect Analysis The lime-fly ash combinations that were used in a six inch stabilized subgrade are presented in Table 50. The average of approximately 20 field deflection measurements are presented in Tables B-24 through B-27, see Appendix B. From these deflection measurements the calculated deflection basin parameters are presented in Tables 51 through 53.

The test sections all show a lower maximum deflection at the end of the second year in comparison to the W_1 measurement taken in the first year. The 0/25 weight percentage lime-fly ash combination produces the highest maximum deflection reading over the two year study period. Note that section 7 is the only section that does not exhibit a decreasing maximum deflection in the second year. The sections showing the least maximum deflection at the end of the second year are sections 1 and 2. Although the 0/10 and 0/30 weight percentage lime-fly ash combinations produce roughly the same maximum deflection, the 0/10 weight percentage lime-fly ash combination would be more economical. The 3/0 and 3/8 weight percentage lime-fly ash combinations (sections 1 and 2, respectively) produced results equivalent to the control section.

Section 3 is the only section to show an initial decrease in spreadibility. However, during the second year this same section developed enough rigidity so that the value was on par with the other sections. In contrast to W_1, the S value of the all lime section was the highest at the end of the second year. Sections 1 and 4 gave either the same, or close to, the same measurements for the as

Table 50. Lime-Fly Ash Combinations and Age of Deflection Surveys- Site No. 5

Test Section	Lime/Fly Ash[a] (subgrade)	Age at time of Test: As Const. (months)	1 Yr (months)	2 Yr (months)
1	3/6	8	16	21
2	3/8	12	16	21
3	0/10	12	16	21
4	2/8	8	16	21
5	4/0	8	16	21
6	2/6	8	16	21
7	0/25	12	16	21
8	0/30	12	16	21

[a] Weight Percentage

Table 51. Dynaflect Deflection Basin Parameters- Site No. 5 (8 and 12 month surveys).

Test Section	Lime/Fly Ash [a]	W_1	SCI	BCI	S	W_5
1	3/6	.506	.128	.025	62	.185
2	3/8	.523	.114	.037	60	.162
3	0/10	.673	.158	.044	59	.205
4	2/8	.549	.144	.024	62	.204
5	4/0	.518	.135	.025	62	.193
6	2/6	.637	.220	.034	53	.162
7	0/25	.943	.231	.069	56	.231
8	0/30	.712	.186	.049	58	.209

[a] Weight Percentage

Table 52. Dynaflect Deflection Basin Parameters- Site No. 5 (16 month survey).

Test Section	Lime/Fly Ash [a]	W_1	SCI	BCI	S	W_5
1	3/6	.499	.092	.044	69	.213
2	3/8	.476	.098	.046	65	.170
3	0/10	.570	.130	0	48	0
4	2/8	.557	.117	.042	68	.239
5	4/0	.488	.098	.038	69	.220
6	2/6	.634	.174	.049	59	.193
7	0/25	.815	.174	.079	60	.232
8	0/30	.617	.141	.055	62	.207

[a] Weight Percentage

Table 53. Dynaflect Deflection Basin Parameters- Site No. 5 (21 month survey).

Test Section	Lime/Fly Ash	W_1	SCI	BCI	S	W_5
1	3/6	.496	.090	.037	66	.202
2	3/8	.463	.101	.032	62	.159
3	0/10	.565	.126	.045	63	.198
4	2/8	.549	.121	.037	65	.225
5	4/0	.504	.100	.037	67	.213
6	2/6	.594	.161	.042	59	.186
7	0/25	.830	.202	.069	59	.240
8	0/30	.609	.139	.048	62	.215

[a] Weight Percentage

constructed, one year and two year deflection surveys. This similarity is also seen in the SCI parameters. Sections 1, 2 and 5 had the lowest SCI values at the end of the two year study period.

All sections (except section 7) experienced a stiffening of the upper layer in the first year and remained relatively constant for the second year. In reference to the BCI and W_5 measurements, it can be seen that most sections exhibited some type of distress development in the first year of the study. During the second year BCI measurements showed a decreasing trend. The W_5 measurements showed a decreasing trend for sections 1 through 6. However, the decrease was quite small. The increases in W_5 for sections 7 and 8 were also small. Basically, the test sections changed very little in their W_5 responses during the second year. Section 8 showed a steady response for the test period which was always around 200 mils. This was very similar to the control section (section 5).

Using the same S and W_1 measurements from Tables 51 through 53 the dual parametric charts were prepared (Figures 35 and 36). By interpolating the effective thickness and the resilient modulus values a pavement layer characterization was made (Table 54 and 55). Sections 3, 5 and 8 showed a two inch gain in effective thickness during the study period. Although the effective thickness in section 8 was not as large as in section 3, it showed an increasing trend while section 3 signaled a decreasing trend. The only poorly performing section has been section 6 which was earlier shown to be the least desirable of the all fly ash sections with respect to the deflection basin parameters. The change in resilient modulus showed no apparent pattern in the first year. Section 3 exhibited the

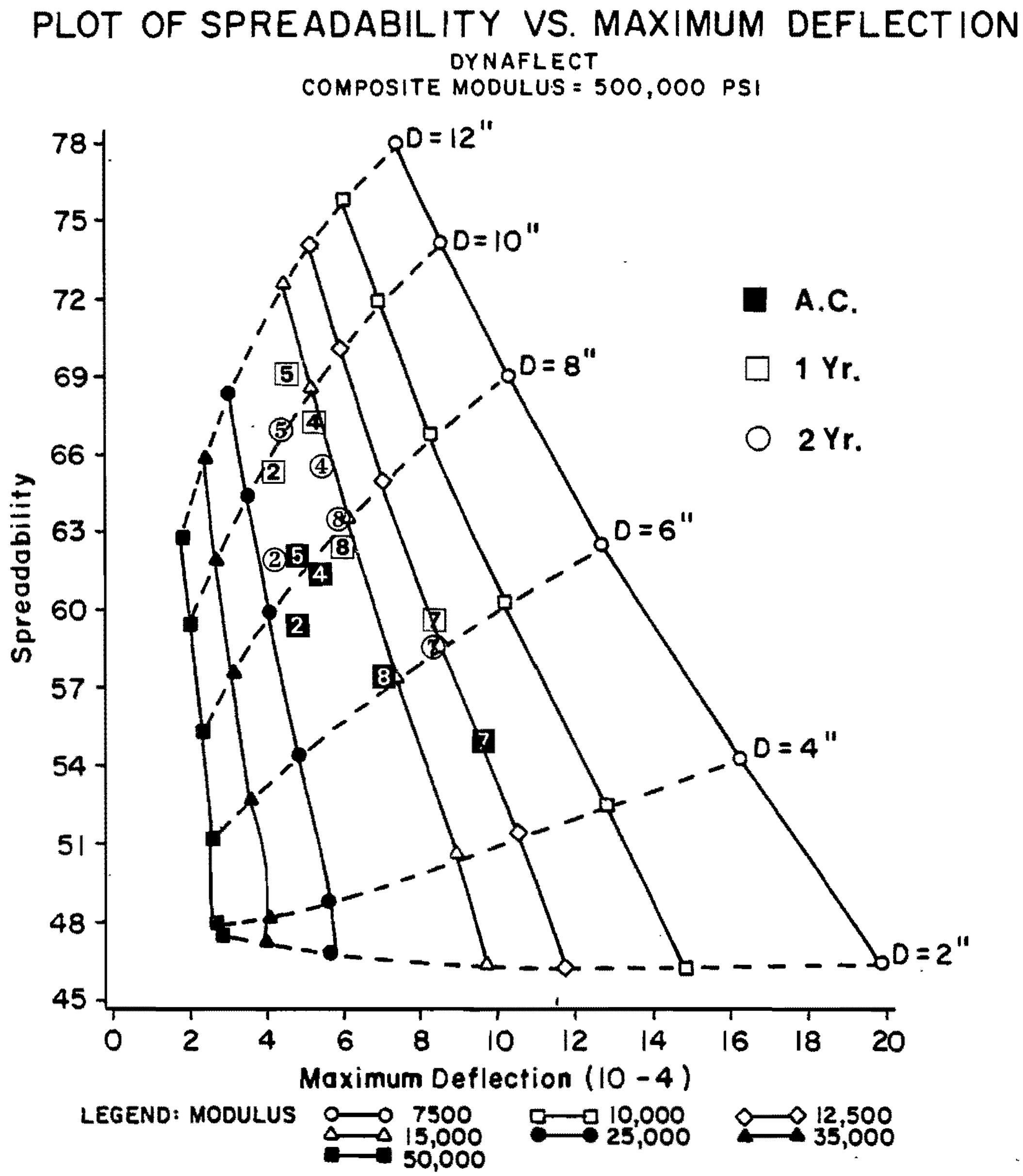


Figure 35. Dynaflect dual parametric chart- Site 5, Sections 2, 4, 5, 7 and 8.

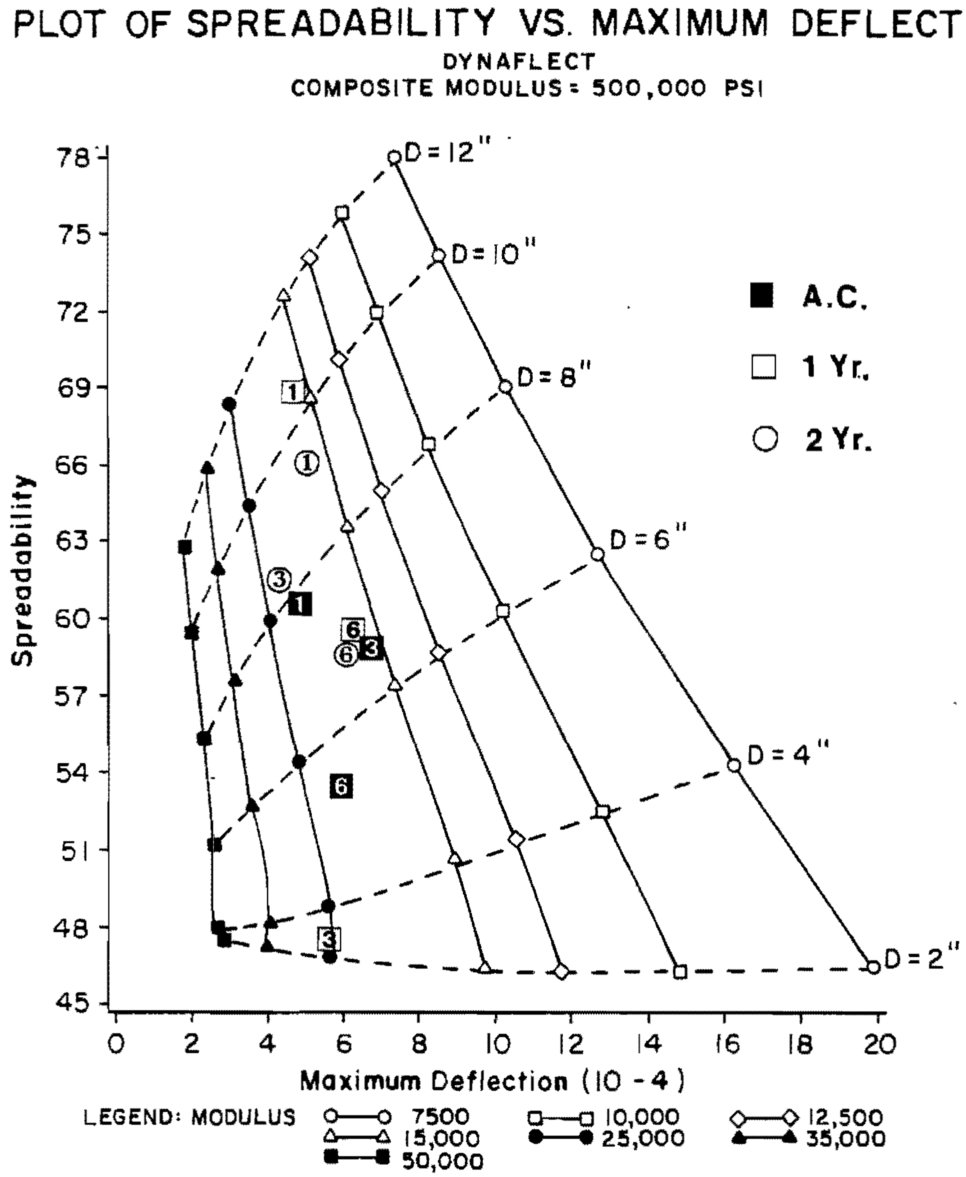


Figure 36. Dynaflect dual parametric chart--Site 5, Sections 1, 3 and 6.

Table 54. Dynaflect Dual Parametric Effective Depth Results
Site No. 5

Test Section	Lime/Fly Ash[a]	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	3/6	8	10	9
2	3/8	7.5	9	8
3	0/10	6.5	3	8.5
4	2/8	7.5	9.5	9
5	4/0	8	10.5	10
6	2/6	5	7	7
7	0/25	5	6.5	6
8	0/30	6	7.5	8

[a] Weight Percentage

Table 55 Dynaflect Dual Parametric Resilient Modulus Results Site No. 5

Test Section	Lime/Fly Ash[a]	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	3/6	21000	15000	17000
2	3/8	21000	20000	21000
3	0/10	15000	25000	21000
4	2/8	20000	15000	16000
5	4/0	20000	17000	17000
6	2/6	20000	17000	18000
7	0/25	12500	12500	12500
8	0/30	15000	15000	16000

[a] Weight Percentage

largest increase in resilient modulus. Section 7 showed a constant 12,500 psi resilient modulus. Section 8 was constant for the first year, then peaked at 16,000 psi at the end of the second year. Only sections 4 and 7 exhibited a lower resilient modulus value than the control section during the two year study period. The sections which showed an initial decrease in resilient modulus (sections 1, 4, 5 and 6) are unique from the previously evaluated test sites in that the two year resilient modulus results have not exceeded the as constructed measurements.

The results of the GLM and ANOVA investigations are presented in Tables 56 and 57, respectively. The results of the GLM testing for the change in the effective thickness indicate significance in both variations of the lime and fly ash percentages. The level of significance was found to be 0.025 and 0.100, respectively. Similiar results were obtained when the change in the resilient modulus was accounted for by the percentages of the lime and fly ash. The significance levels were found to be 0.005 and 0.025, respectively. The ANOVA results indicate significance levels of 0.005 for both lime and fly ash and for both models.

3.6.2 Benkleman Beam Analysis The measured deflections from the field surveys are presented in Tables B-28 through B-31 in Appendix B. It should be noted that a 10 ft. beam was used for the measurements taken at the end of the first year (two year data). This only effected the offset readings and the spreadibility parameters for the evaluation.

Table 56. General Linear Regression Model- Site No. 5

Testing the Significance of Lime and Fly Ash Percentages on the Change in Effective Depth from AC stage to 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	4.82	4.08	0.025
Fly Ash	3.91	2.46	0.100

Testing the Significance of Lime and Fly Ash Percentages on the Change in Elastic Modulus from AC stage to 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	7.70	6.30	0.005
Fly Ash	6.15	4.08	0.025

Table 57. Analysis of Variance Statistical Analysis-Site No. 5

Testing the Significance of Lime and Fly Ash Percentages on the change in Effective Depth from AC stage to 2 Yr. stage. (Based on Dynaflect Dual Parametric Results)

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	7.70	6.30	0.005
Fly Ash	7.59	5.21	0.005

Testing the Significance of Lime and Fly Ash Percentages on the Change in Resilient Modulus from AC stage to 2 Yr. stage. (Based on Dynaflect Dual Parametric Results)

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	7.70	6.30	0.005
Fly Ash	7.59	5.21	0.005

The maximum deflections indicated that in the second year some deterioration was occuring. After increasing a tolerable amount during the first year a two-fold increase in W_1 followed at the end of the second year. The W_2 measurements suggest that the lower layers of the pavement system are increasing in strength and rigidity. A set of comparable two year data would be more helpful in depicting the long term attributes, but is not available. The spreadibility values listed show that only sections 2 and 3 remained relatively stable in the initial aging period. These same sections are also the only ones to possess a greater S value at the two year stage than section 5, the control section.

The dual parametric charts are presented in Figures 37 and 38. Resilient modulus and effective thickness values taken from these charts are listed in Tables 58 and 59. Note that the first year's dual parametric results for section 6 and 7 are not listed. These points were below the range of the dual parametric chart. However, the general pattern is apparent--they are decreasing in magnitude with time. All indications from Table 59 are that the majority of the test sections showed some amount of increase in resilient modulus. The exceptions are sections 2, 3, 6 and 7. The effective thickness measurements for some test sections have been either increasing, while others have been either decreasing, or remaining the same. The control section reduced its magnitude by a factor of four while sections 2 and 3 remained constant.

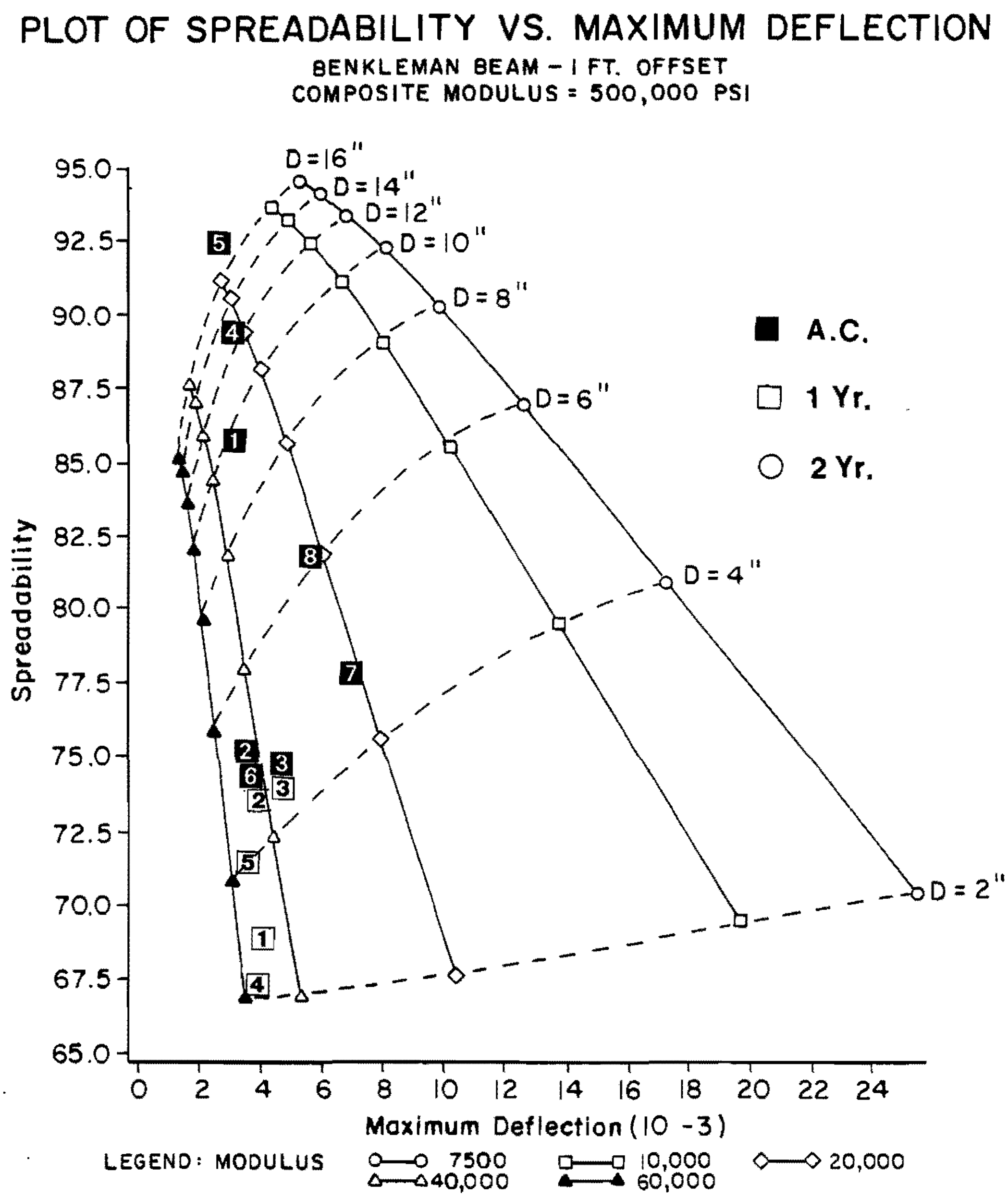


Figure 37. Benkleman beam dual parametric chart--Site 5, Sections 1, 2, 3, 4, 5, 6, 7 and 8.

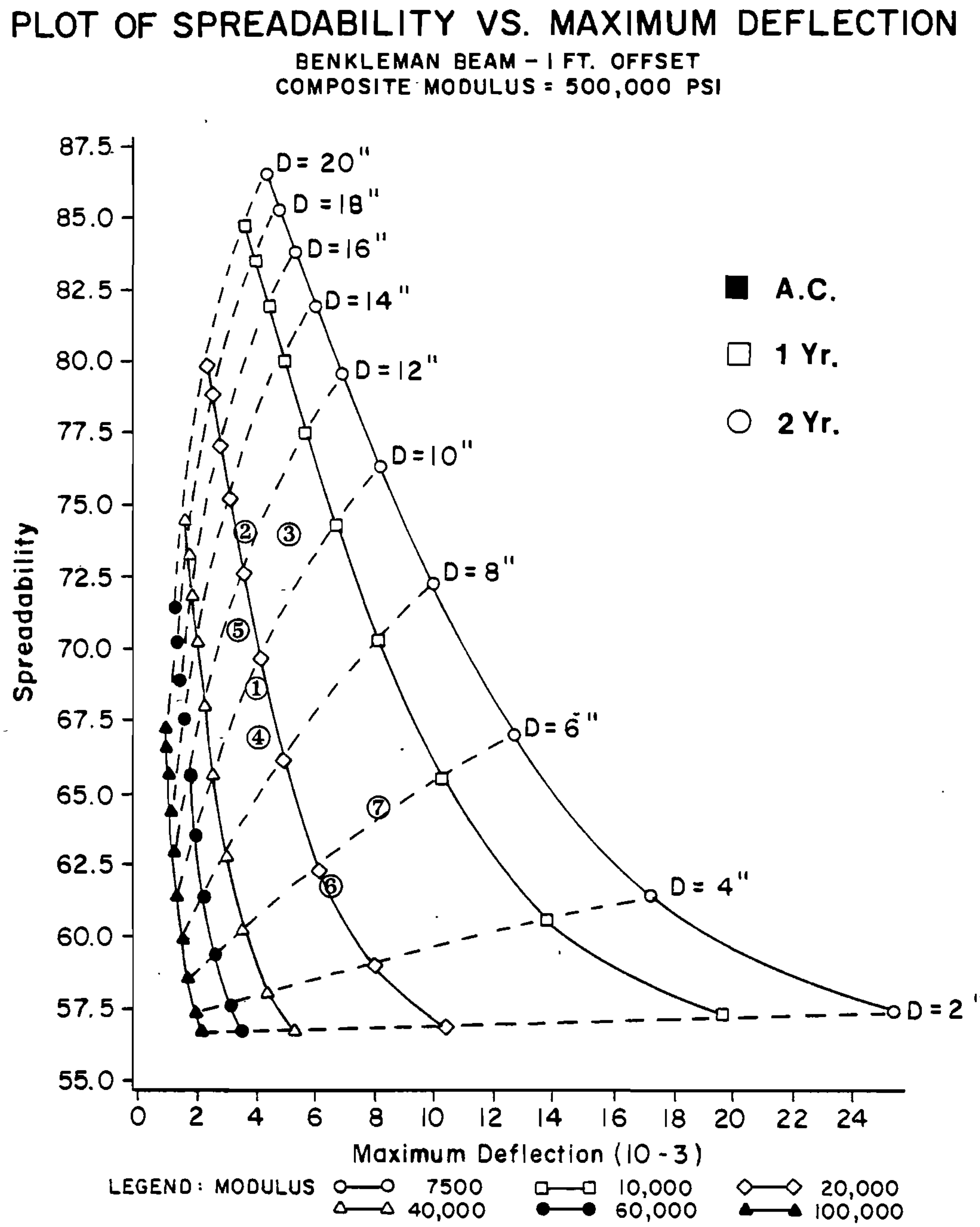


Figure 38. Dynaflect dual parametric chart (3 Ft. offset)--Site 5, Sections 1, 2, 3, 4, 5, 6 and 7.

Table 58. Benkleman Beam Dual Parametric Effective Depth Results- Site No. 5

Test Section	Lime/Fly Ash[a]	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	3/6	9.5	3	10
2	3/8	5	5	12
3	0/10	4.5	4.5	11
4	2/8	12	2	9
5	4/0	18	4	11
6	2/6	4.5	-	6
7	0/25	4.5	-	6
8	0/30	6	-	6

[a] Weight Percentage

Table 59. Benkleman Beam Dual Parametric Resilient Modulus Results- Site No. 5

Test Section	Lime/ Fly Ash [a]	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	3/6	30000	50000	20000
2	3/8	40000	45000	20000
3	0/10	35000	35000	15000
4	2/8	20000	60000	25000
5	4/0	18000	55000	25000
6	2/6	35000	-	20000
7	0/25	20000	-	15000
8	0/30	21000	-	12000

[a] Weight Percentage

3.7 Analysis of Test Site 8, SH 335, Potter County

3.7.1 Dynaflect Analysis The various lime-fly ash combinations corresponding to the individual test sections are listed in Table 60. Also shown are the actual pavement system's age. The average deflection basin measurements are listed in Tables B-32 through B-34, Appendix B. The deflection basin parameters are listed in Tables 61 through 63.

The maximum deflections show that all sections have experienced a gradual decrease in magnitude. Section 6 had a lower W_1 response than the control section at the end of the second year. The spreadibility values indicate that sections 1 and 6 have been the two least supportive pavement systems. Section 2 has been the most capable of distributing an imposed load. The first year's spreadibility values would indicate some inactivity of the pozzolanic reaction between the lime-fly ash-soil mixtures.

The surface curvature index parameters indicate that either fly ash alone, or in combination with lime has been more effective in reducing the SCI than lime alone. The SCI parameters for section 2 reflect a greater overall rigidity in the upper layers than the rest of the test sections. The BCI parameters indicate a deteriorating mechanism had occured during the first year of service. The measurements taken at the end of the first year show a final value that exceeds the initial or as constructed values. Section 2, in particular, showed a catastrophic increase in BCI during the second year. The W_5 values show that the control section has exhibited the lowest response to the imposed loading. Despite this fact, every

Table 60. Lime-Fly Ash Combinations and Age of Deflection Surveys- Site No. 8.

Test Section	Lime/Fly Ash[a]	Age at Time of Test: As Const. (months)	1 Yr (months)	2 Yr. (months)
1	3/0	6	12	26
2	2/4	6	12	26
3	2/4	6	12	26
4	2/8	6	12	26
5	3/6	6	12	26
6	0/8	6	12	26

[a] Weight Percentage

Table 61. Dynaflect Deflection Basin Parameters- Site No. 8 (6 month survey).

Test Section	Lime/Fly Ash [a]	W_1	SCI	BCI	S	W_5
1	3/0	.720	.229	.069	52	.143
2	2/4	.705	.205	.065	57	.188
3	2/4	.779	.206	.075	59	.226
4	2/8	.783	.210	.068	58	.214
5	3/6	.786	.221	.069	55	.186
6	0/8	.793	.258	.071	52	.164

[a] Weight Percentage

Table 62. Dynaflect Deflection Basin Parameters- Site No. 8 (12 month survey).

Test Section	Lime/Fly Ash[a]	W_1	SCI	BCI	S	W_5
1	3/0	.636	.226	.054	51	.137
2	2/4	.672	.225	.044	55	.193
3	2/4	.763	.255	.055	56	.218
4	2/8	.712	.231	.051	56	.211
5	3/6	.682	.224	.053	55	.184
6	0/8	.684	.249	.057	52	.158

[a] Weight Percentage

Table 63. Dynaflect Deflection Basin Parameters- Site No. 8 (26 month survey).

Test Section	Lime/Fly Ash [a]	W_1	SCI	BCI	S	W_5
1	3/0	.523	.130	.072	60	.146
2	2/4	.520	.099	.256	73	.187
3	2/4	.632	.124	.089	65	.212
4	2/8	.590	.115	.079	65	.207
5	3/6	.550	.105	.088	65	.179
6	0/8	.527	.111	.082	63	.160

[a] Weight Percentage

section in the test site has a two year measurement which is lower in magnitude than when constructed.

Figures 39 and 40 depict the deflection data on the dual parametric charts. The effective thickness as well as the resilient modulus values were interpolated from these figures and are listed in Tables 64 and 65. The effective thickness values, Table 64, show the two extreme lime-fly ash combinations to be equivalent. However the gain in the second year for section 6 exceeded that of section 1. Sections 1, 5 and 6 were the only sections to display an increase in effective thickness. The fact that the measured effective thickness responses of sections 2 and 3 (both 2/4 weight percentage lime-fly ash combinations) are almost identical over the two year study period supports the reproducibility of this aspect of the dual parametric approach. However, this reproducibility was not found in the resilient modulus values.

Results from the GLM and ANOVA can be found in Tables 66 and 67. With respect to the change in effective depth, significance levels of >0.250 and 0.250 were found for the lime and fly ash percentages in the GLM test, respectively. Accounting for the change in effective thickness, the GLM test produced significance levels of 0.050 and 0.250 for in the lime and fly ash percentages.

With respect to the change in the resilient modulus, the ANOVA results are found in (Table 67) reveal significance levels of >0.250 and 0.250, respectively. Accounting for the change in the effective thickness results in significance levels of 0.050 for both stabilizing agents.

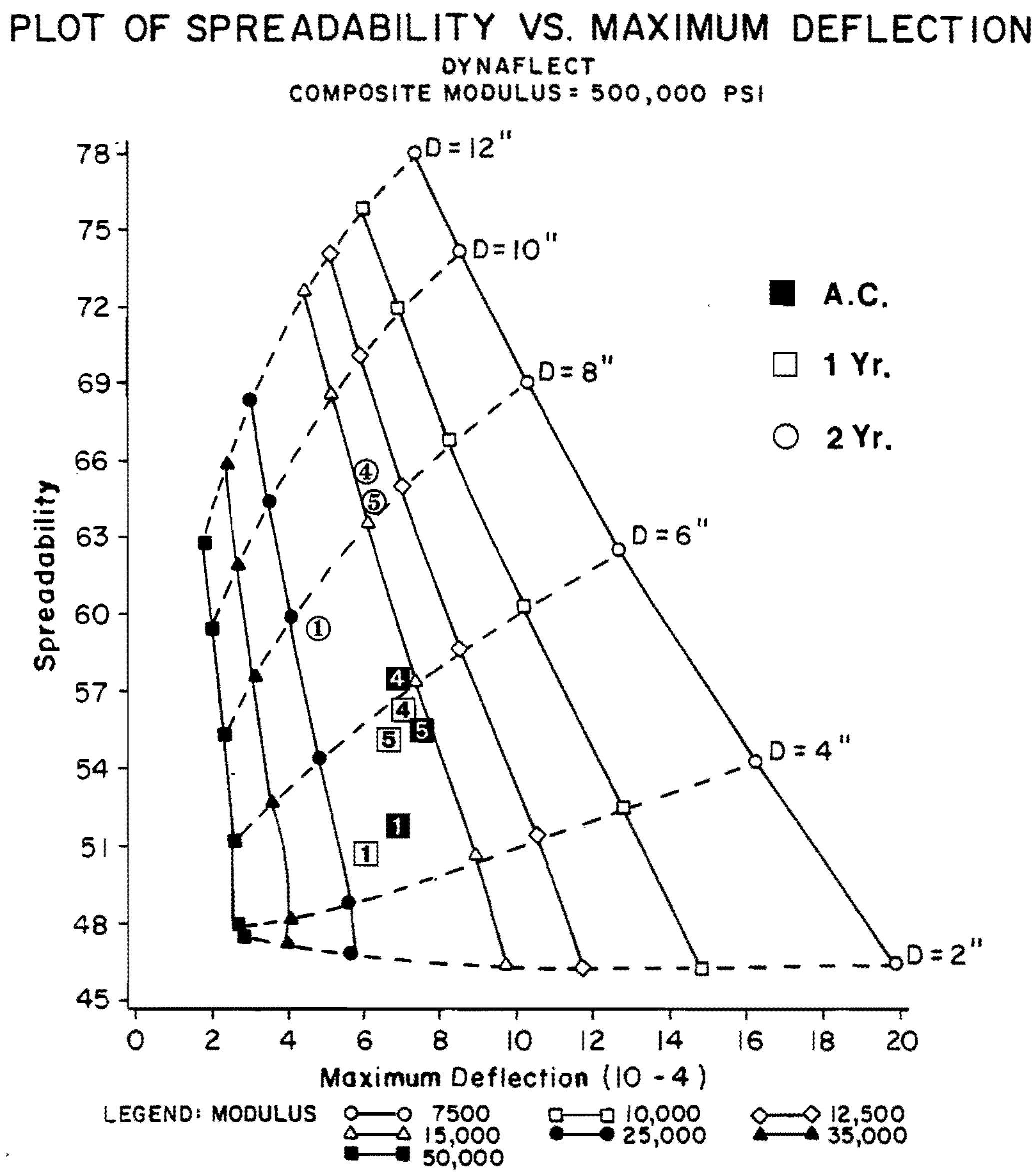


Figure 39. Dynaflect dual parametric chart--Site 8, Sections 1, 4 and 5.

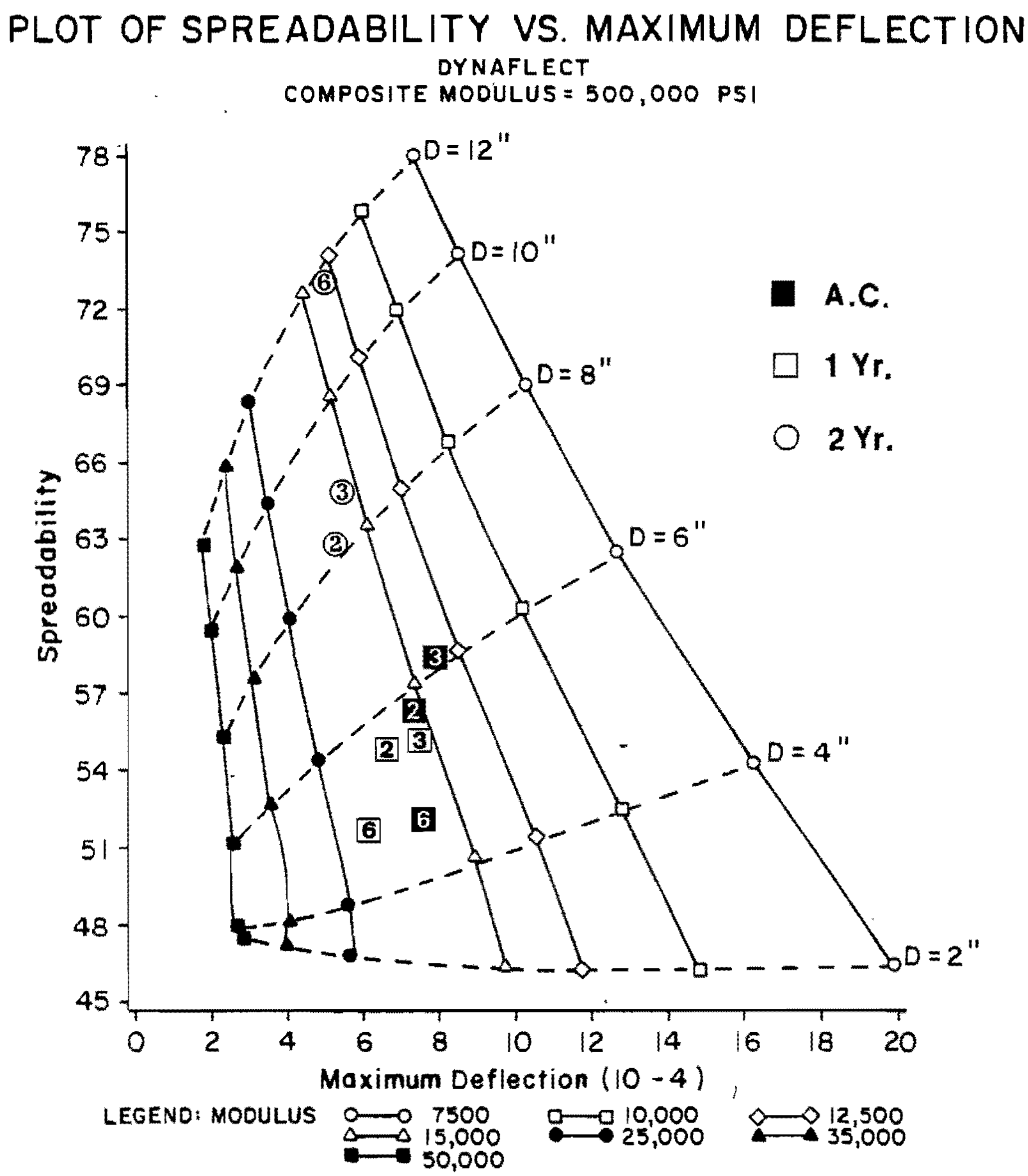


Figure 40. Dynaflect dual parametric chart--Site 8, Sections 2, 3 and 6.

Table 64. Dynaflect Dual Parametric Effective Depth Results-
Site No. 8

Test Section	Lime/Fly Ash[a] Ratio	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	3/0	4.5	5	7.5
2	2/4	6	5.5	8
3	2/4	6	5.5	9
4	2/8	6	5.5	9
5	3/6	5	5.5	8
6	0/8	4.5	5	12

[a] Weight Percentage

Table 65. Dynaflect Dual Parametric Resilient Modulus Results-
Site No. 8

Test Section	Lime/Fly Ash[a] Ratio	Resilient Modulus(psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	3/0	20000	22000	22000
2	2/4	16000	20000	20000
3	2/4	13500	16000	16000
4	2/8	15000	16000	15000
5	3/6	15000	20000	15000
6	0/8	16000	21000	12500

[a] Weight Percentage

Table 66. General Linear Model Regression Analysis- Site No. 8

Testing the Significance of Lime and Fly Ash Percentages in the Change in Effective Depth from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.55	*	>0.25
Fly Ash	2.04	1.55	0.250

Testing the Significance of Lime and Fly Ash Percentages in the Change in Resilient Modulus from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	4.28	2.85	0.050
Fly Ash	1.76	1.55	0.250

Note: Asterisks indicate negligible values of statistical parameters

Table 67. Analysis of Variance Statistical Test- Site No. 8

Testing the Significance of Lime and Fly Ash Percentages in the Change in Effective Depth from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	0.55	*	>0.25
Fly Ash	1.73	1.55	0.250

Testing the Significance of Lime and Fly Ash Percentages in the Change in Resilient Modulus from the AC stage to the 2 Yr. stage.

Source	Computed F Value	Tabulated F Value	Significance Level (α)
Lime	4.28	3.93	0.050
Fly Ash	3.39	3.42	0.050

Note: Asterisks indicate negligible values of statistical parameters

3.7.2 Benkleman Beam Analysis The average of the field deflections data are given in Tables B-35 through B-37, Appendix B.

Section 6 exhibited the only true decline in W_1 with time. Section 4 exhibited the only increase in maximum deflection during the first year of the two year study. For the second year the W_1 measurements were less than or about equal to the first year's measurement. From the W_2 measurements it is seen that only section 3 exhibited an increase during the first year. Despite this increase, the second year W_2 measurements are lower than the as constructed or the one year responses. Section 3 was the only section to show an increase in spreadibility. The second year's spreadibility values showed an increasing trend. Comparison of the control section to the rest of the test sections show it to have some of the lowest S values. Note that section 6, the all fly ash section, had a final spreadibility value that is greater than the control section.

Figures 41 through 43 represent the dual parametric charts for the benkleman beam configurations. The dual parametric results are presented in Tables 68 and 69. The general trend for the effective thickness has been a reduction in the initial year. Interestingly enough the as constructed and one year effective thickness data are almost identical for all but section 3. Even the two year data were not too different among the test sections. In comparing the control section to the section using only fly ash it is seen that at the as constructed stage they were equal.

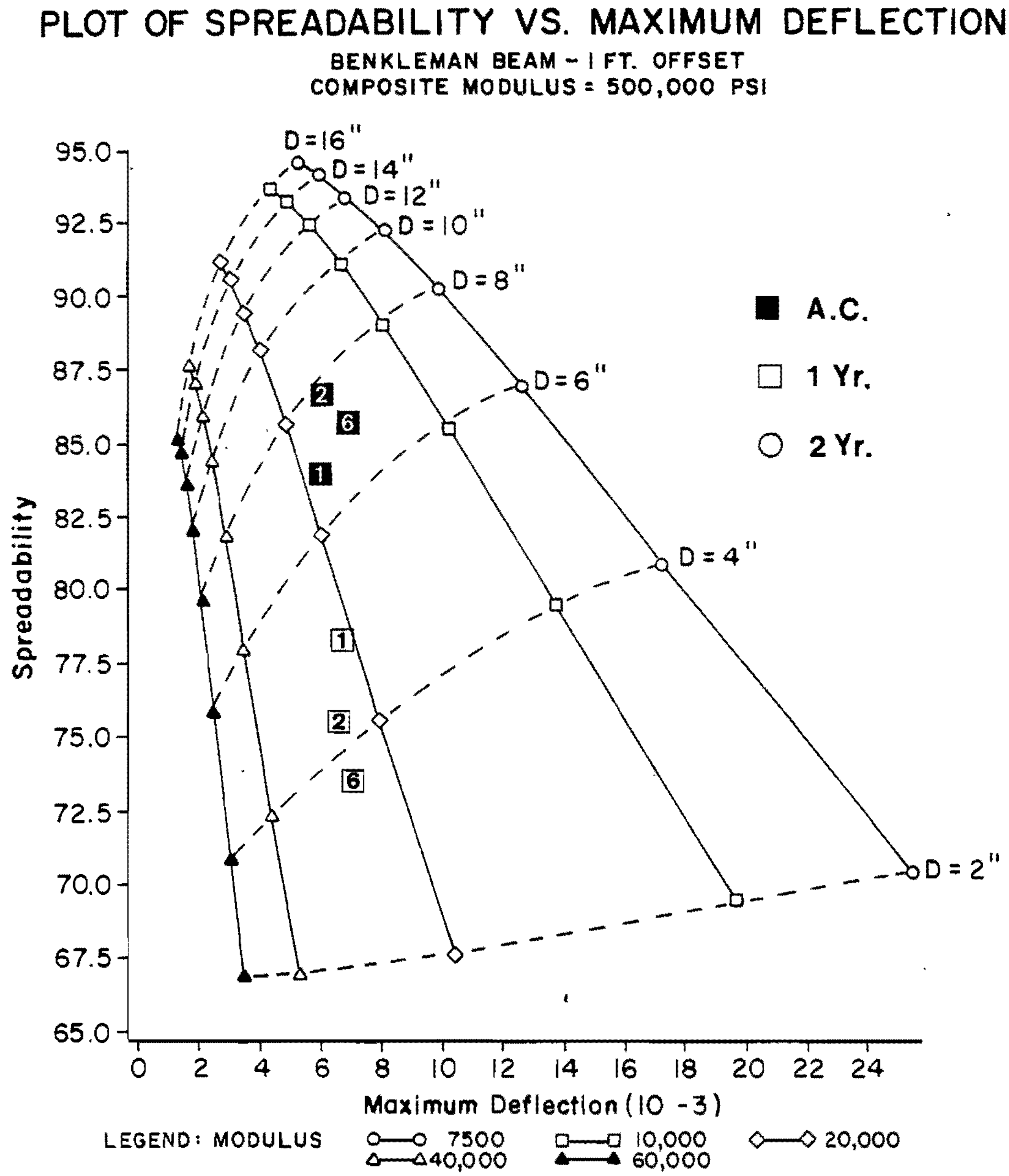


Figure 41. Benkleman beam dual parametric chart--Site 8, Sections 1, 2 and 6.

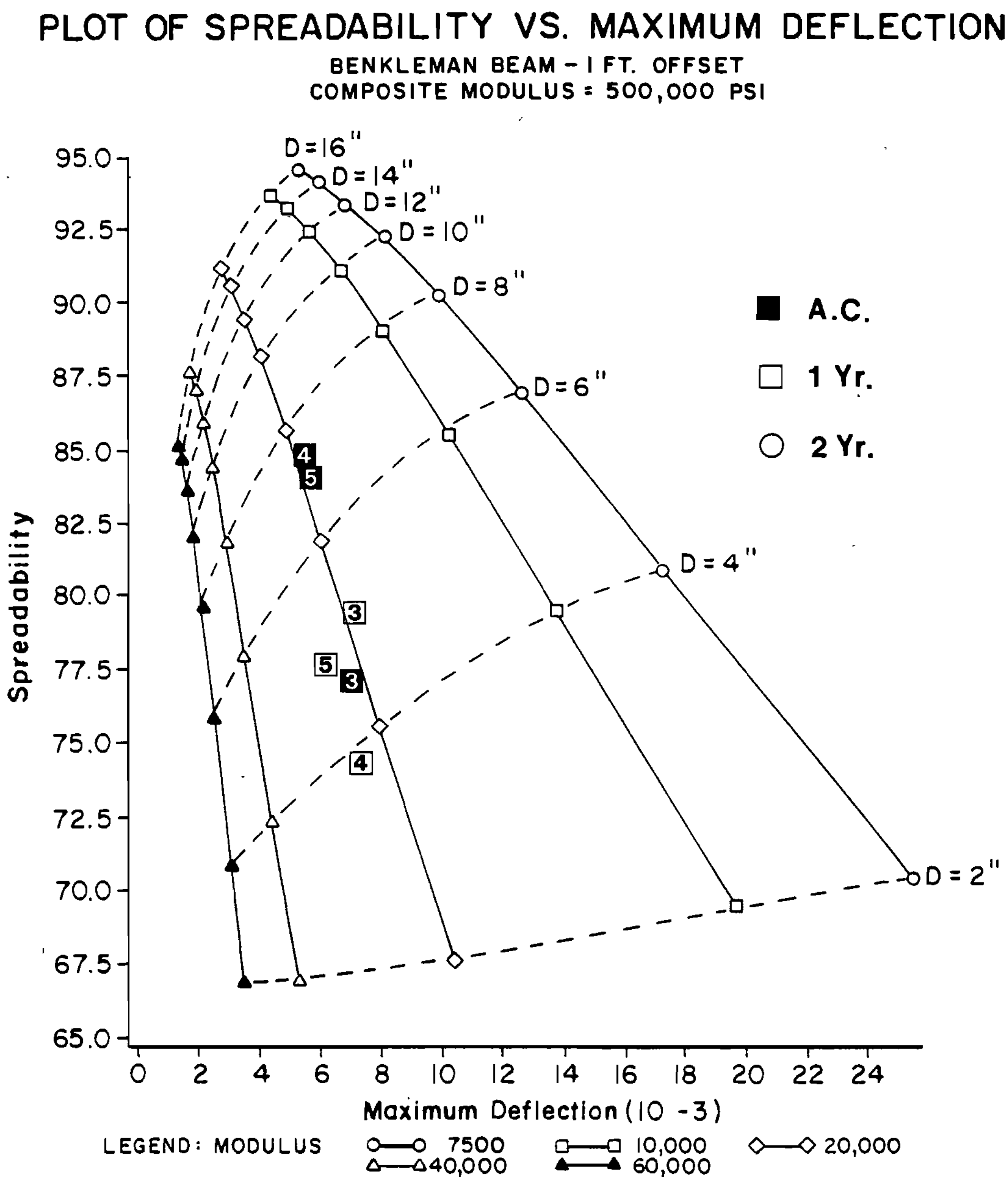


Figure 42. Benkleman beam dual parametric chart--Site 8, Sections 3, 4 and 5.

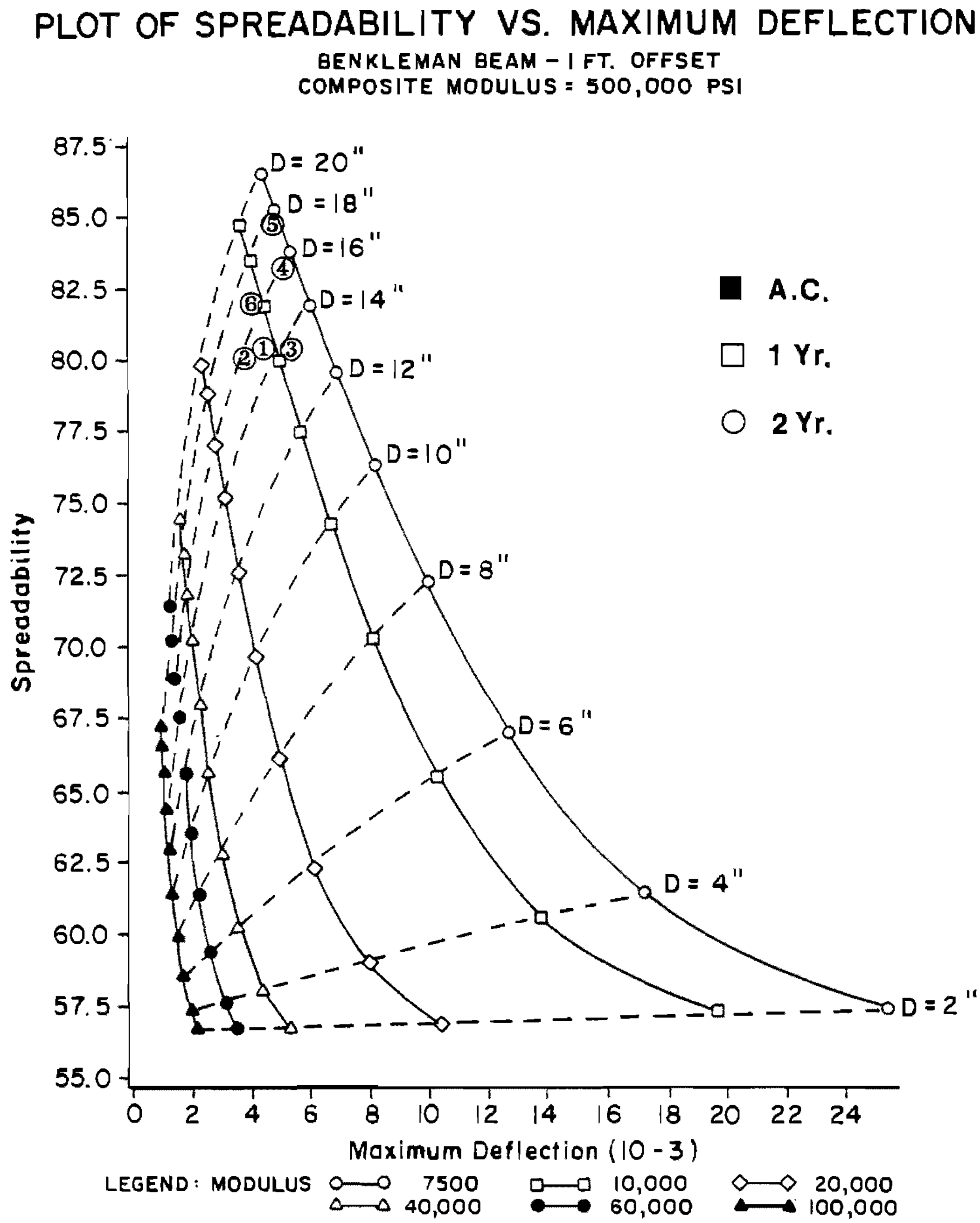


Figure 43. Benkleman beam dual parametric chart (3 ft. offset)--Site 8, Sections 1, 2, 3, 4, 5 and 6.

Table 68. Benkleman Beam Dual Parametric Effective Depth Results Site No. 8

Test Section	Lime/Fly Ash Ratio[a]	Effective Depth (in.)		
		D_{AC}	$D_{1\ Yr.}$	$D_{2\ Yr.}$
1	3/0	7	5	6
2	2/4	8	4	6
3	3/4	5	5	6
4	2/8	7	4	7
5	3/6	7	5	8
6	0/8	7	4	7

[a] Weight Percentage

Table 69. Benkleman Beam Dual Parametric Elastic Modulus Results-
Site No. 8

Test Section	Lime/Fly Ash[a] Ratio	Resilient Modulus (psi)		
		E_{AC}	$E_{1\ Yr.}$	$E_{2\ Yr.}$
1	3/0	20000	20000	28000
2	2/4	16000	28000	28000
3	2/4	22000	20000	28000
4	2/8	20000	22000	22000
5	3/6	20000	28000	20000
6	0/8	15000	28000	28000

[a] Weight Percentage

The resilient modulus values in Table 69 for sections 1 and 6 still showed similarities at the end of the second year. Note too that section 3, after experiencing an initial decrease, then an increase, had the same resilient modulus at the end of the second year as did sections 1 and 3. Section 5 was the only section to show a decreasing trend in resilient modulus with time.

4. OVERALL ANALYSIS

<u>4.1 Dynaflect Dual Parametric Results</u> Table 70 contains an explanation for each classification according to the fluctuation of the effective thickness and resilient modulus. Table 71 summarizes all the results. Five of the six test sites exhibited (the exception was Test Site 4) beneficial results from the lime-fly ash stabilization technique. Test sites 2 and 3 had test sections showing greater development in stiffness in the subgrade than in the upper layers. Three of the sections in Test site 3 gained stiffness in the lower layers more quickly than the upper layers (sections 2, 3 and 5). The corresponding lime-fly ash weight percentage combinations are 3/10, 2/5 and 2/8, respectively. In test site 2, the control section and the fly ash section were classified as IV and V, respectively. The control section and all fly ash section in test site 3 were designated as class II.

A reason for the anomolous behavior in Test Site 4 is not known at this time.

However, it should be noted that while typically a lime-fly ash combinations of 4 to 6 percent lime and 10 to 15 percent fly ash are used in highly plastic clays, the percentages used in test site 4 were much lower than this. Perhaps a slight increase in the lime percentage would have proved beneficial.

<u>4.2 Statistical Analysis</u> Table 72 highlights the results from the GLM procedure. The interaction source (L*FA) is particulary interesting because it signifies the importance of the lime and fly

Table 70. Decription of the Group Classifications

Classification Group	Description
I,II	Indicates that the total pavement system is increasing in load carrying capability.
III	Upper layers of the pavement system are increasing in stiffness at a greater rate than the subgrade.
IV,V	Signifies that the lower layers have greater stiffness increase than the upper layers.
VI	Indicates that the total pavement system is remaining constant in its load carrying capability.
VII	Indicates the the Lime-Fly Ash stabilization is not contributing to the pavement system.
VIII	Indicates that the total pavement system is losing its load carrying capability.
IX	Indicates that the lower layers are decreasing in stiffness at a greater rate than the upper layers.

Table 71. Dynaflect Group Classification based on Change in dual parametric results. Two year study period. (* indicates control section, _ indicates all fly ash section).

Test Site \ Group Classification	I D↑, E=	II D↑, E↑	III D↑, E↓	IV D↓, E↑	V D=, E↑	VI D=, E=	VII D↓, E↓	VIII D↓, E=	IX D=, E↓
1	2*, 5*, <u>8</u>	1*, <u>10</u>	3*, 4*, <u>6</u> <u>7</u>, <u>9</u>						
2		2, 3, 4, 5 7		1*, 8	<u>6</u>				
3		1, 4*, <u>6</u>		2	3, 5				
4		1*					2, 3, <u>4</u>, 5 6		
5		<u>7</u>, <u>3</u>		1, 2, 4, 5* <u>8</u>					
8		1*, 2, 3, 4	5, <u>6</u>						

Table 72. General Linear Model Results for All Dynaflect Dual Parametric Results.

Testing the Significance of the Lime and Fly Ash Percentages on the Change in Effective Depth.

Source	Computed F Value	Tabulated F Value	Significance Level
Lime	1.82	1.77	0.100
Fly Ash	2.92	2.37	0.005
L*FA	2.75	2.39	0.025

Testing the Significance of the Lime and Fly Ash Percentages on the Change in Resilient Modulus.

Source	Computed F Value	Tabulated F Value	Significance Level
Lime	1.82	1.77	0.100
Fly Ash	2.92	2.37	0.005
L*FA	2.75	2.39	0.025

ash interaction. The interaction being the relationship that the variations of the lime and fly ash percentages have in common with respect to the change in either dual parametric result. These GLM results indicate that the lime-fly ash percentages are significant. Note too that the dual parametric results, as reflected by the classifications listed in Table 71, indicate a significant and substantial increase in each parameter with time.

The ANOVA results are presented in Table 73. The statistical data indicates that the dual parametric results are significantly influenced by the variations in fly ash percentages, more so than the variations in lime content.

4.3 Benkleman Beam Dual Parametric Results Table 74 contains the classification information for the dual parametric changes in the first year. According to this table test sites 1, 2, 4, 5 and 8 contain test sections which experienced more stiffness development in the lower layers than in the upper layers. This included section 10 containing a 6/6 lime-fly ash combination for test site 1. Test sections 1, 2, 4 and 5 in test site 5 showed the same patterns of stiffness development. These sections correspond to lime-fly ash combinations of 3/6, 3/8, 2/8 and 4/0, respectively. Test site 8 had four section (2, 4, 5 and 6) which exhibited an increasing resilient modulus with time. The corresponding lime-fly ash combinations are 2/4, 2/8, 3/6 and 0/8, respectively. The subgrade's stiffness in the control section of test site 8 (a 3/0 lime-fly ash combination) remained somewhat constant but the upper layers of the pavement lost their ability to distribute stresses.

4.4 Comparison of Testing Methods To compare the deflection

Table 73. Analysis of Variance Results for All Dynaflect Dual Parametric Results.

Testing the Significance of the Lime and Fly Ash Percentages on the Change in Effective Depth.

Source	Computed F Value	Tabulated F Value	Significance Level
Lime	1.82	1.72	0.100
Fly Ash	2.84	2.37	0.005
L*FA	2.95	2.52	0.025

Testing the Significance of the Lime and Fly Ash Percentages on the Change in Resilient Modulus.

Source	Computed F Value	Tabulated F Value	Significance Level
Lime	0.33	*	*
Fly Ash	1.32	1.24	0.250
L*FA	1.79	1.33	0.250

Note: Asterisks indicate negligible values of statistical parameters

Table 74. Benkleman beam Group Classification Based on Change in dual parametric results. One year study period. (* indicates control section, _ indicates all fly ash section).

Test Site \ Group Classification	I D↑, E=	II D↑, E↑	III D↑, E↓	IV D↓, E↑	V D=, E↑	VI D=, E=	VII D↓, E↓	VIII D↓, E=	IX D=, E↓
1		1*, 2*	3*, 5* $\underline{6}$, $\underline{7}$, $\underline{8}$	$\underline{10}$		4*			
2	1*, 4, 7	2, 3	5, $\underline{6}$	8					
3									
4		5	1*, 2, 3 6	$\underline{4}$					
5				1, 4, 5*	2	$\underline{3}$			
8			2, 4, 5 $\underline{6}$						

measurement methods, the group classifications based on the change in the dual parametric results were be used (Table 75). These group classifications are based on the first year results. Table 74 shows only test sections 4 (test site 1) and 3 (test site 5) had not shown any improvement in either dual parametric parameter. Comparing the group classifications for test site 1 in Tables 74 through 75 reveals that, except for a few cases, the effective thickness characterization is essentially the same for both deflection systems. Sections 4 and 10 are the exceptions. However, the deflection measurement methods do show different trends for the resilient modulus parameter. Since the loading applications are different for the two methods the trends noted should be expected. Only sections 3 and 10 agree between the two methods in regards to the trend shown for the resilient modulus. The dynaflect's results appear to be more conservative, indicating as a whole a constant or decreasing stiffness. Only section 3 (site 1) had the same classification for both deflection systems.

The group classifications for sites 2, 4, 5 and 8 did not show any apparent pattern that would suggest that one deflection system would result in either an opposite or a similar dual parametric characterization from the other. The data presented in Tables 74 and 75 do show that there are some instances of agreement between the deflection systems. This includes sections 2, 3 and 8 (test site 2), section 5 (test site 4) and sections 2 and 4 (test site 8).

Table 75. Dynaflect Group Classification based on Change in dual parametric result. One year study period. (* indicates control section, _ indicates all fly ash section).

Test Site \ Group Classification	I D↑, E=	II D↑, E↑	III D↑, E↓	IV D↓, E↑	V D=, E↑	VI D=, E=	VII D↓, E↓	VIII D↓, E=	IX D=, E↓
1	1*, 2*, 5*, $\underline{6}$, $\underline{7}$, $\underline{8}$	$\underline{10}$	3*, 4*, $\underline{9}$						
2		2, 3, 4		1*, 5, $\underline{6}$, 7, 8					
3				1, 2, 3, 4, 5	6				
4	1*	2, 3, $\underline{4}$, 5, 6							
5	7, 8		1, 2, 4, 5*, 6	$\underline{3}$					
8		1*, 5, $\underline{6}$		2, 3, 4					

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the evaluation of the Dynaflect and Benkleman beam data, of the six test sites (containing 44 test sections) the following conclusions were drawn. These conclusions should not be generalized beyond the limits of this research.

1. The dual parametric approach was adequate in structurally evaluating a two-layered pavement system.
2. Dual parametric results indicated that lime-fly ash stabilization provided a viable alternative to lime stabilization of selected bases and subbases. Five of the six test sites exhibited beneficial results from lime-fly ash stabilization techniques, and significant increases with time in the parameters were observed. The effective depth and resilent modulus results of site 1 indicates a distinct trend where 'flyash only' sections in the subgrade exhibited lower structural support capability than sections stabilized with both lime and flyash.
3. The variations in lime-fly ash percentages were observed to significantly influence changes in the dual parametric results.
4. Some anomolies were found in the dual parametric results of Test Site No. 4 which may have been due to the low percentages of stabilizing agents employed.
5. The variations in fly ash percentages influenced the changes in the parameters than did variations in lime content.
6. Comparison of the two deflection measurement methods, namely Dynaflect and Benkleman beam, generally indicated different

trends for resilient modulus parameters derived from the analysis. Some instances of agreement between the two deflection systems were observed in test sites 2, 4 and 8.

7. Dynaflect results appeared more conservative, indicating constant or decreasing stiffnesses as a whole. It is possible that the Dynaflect imposed too light a load on the test sections, which varied from 16 inches to 20 inches in thickness, to obtain results comparable to the Benkleman beam.

5.2 Recommendations

Based on conclusions reached in this study, the following recommendations are made:

1. Consideration of soil properties and mix design procedure at the test sites is believed to be important in completing the evaluation process. Such a study, when correlated with the dual parametric results, will be useful in defining lime-fly ash ratios for optimum fly ash utilization based on long term performance.
2. Unconfined compressive test results on core samples from the experimental sites should be analyzed to quantitatively check the dual parametric results of the deflection systems.
3. Utilize the available deflection data of these test sites and dual parametric results of the report to devise a method for predicting Elastic Moduli and deflections of generalized lime-fly ash bases and subgrades.
4. Continue to monitor all test sites and observe performance with time.

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Appendix A TYPICAL LAYOUT - LAYOUT OF TEST SITES AND TEST SECTIONS

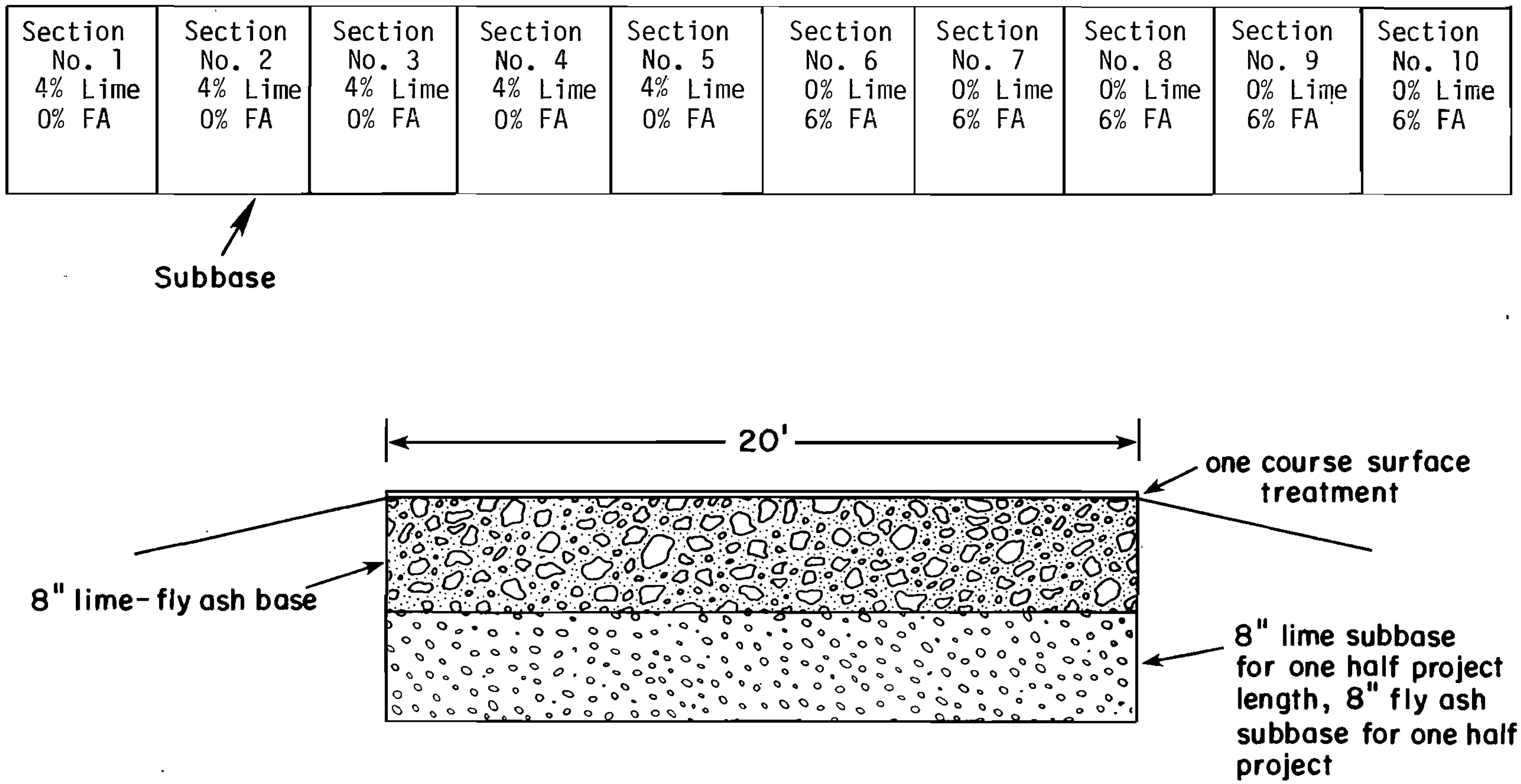

Figure A-1 Typical layout planview and section of FM 3378 in Bowie County, Texas (Site No. 1)

Section No. 1	Section No. 2	Section No. 3	Section No. 4	Section No. 5	Section No. 6	Section No. 7	Section No. 8	Section No. 9	Section No. 10
4% Lime	2% Lime	2% Lime	4% Lime	4% Lime	0% Lime	2% Lime	2% Lime	0% Lime	4% Lime
0% FA	4% FA	10% FA	8% FA	16% FA	16% FA	24% FA	15% FA	21% FA	4% FA

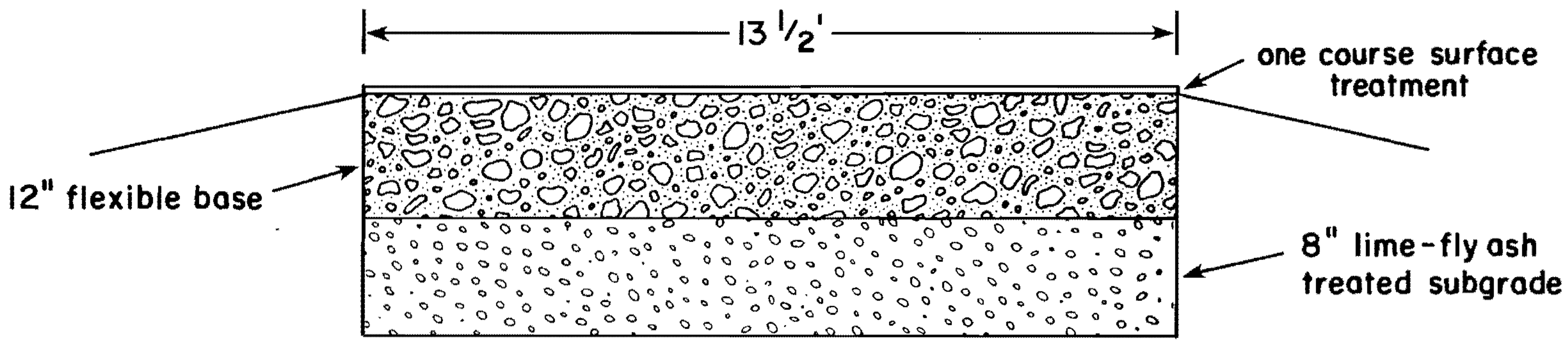

Figure A-2 Typical planview and section of US 59 in Panola County, Texas (Site No. 2).

Section No. 1 3% Lime 6% FA	TRANSITION	Section No. 2 3% Lime 10% FA	TRANSITION	Section No. 3 2% Lime 5% FA	TRANSITION	Section No. 4 4% Lime 0% FA	TRANSITION	Section No. 5 2% Lime 8% FA	TRANSITION	Section No. 6 0% Lime 12% FA

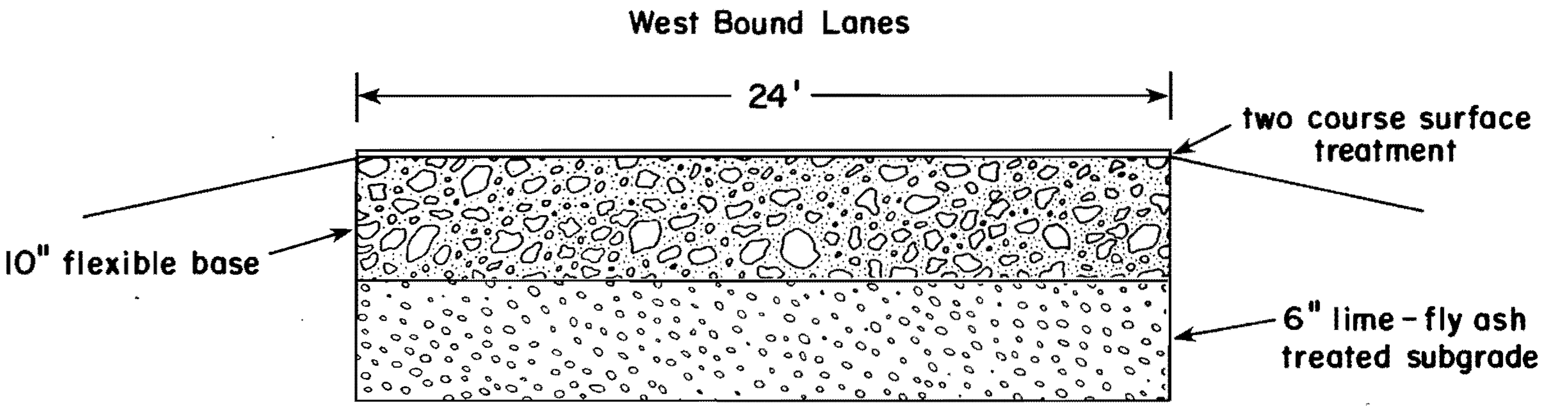

Figure A-3 Typical planview layout and section of west bound lane on FM 1604 in Bexar County, Texas (Site No.3).

Section No. 1 4% Lime 0% FA	TRANSITION	Section No. 2 3% Lime 6% FA	TRANSITION	Section No. 3 3% Lime 9% FA	TRANSITION	Section No. 4 0% Lime 10% FA	TRANSITION	Section No. 5 1% Lime 5% FA	TRANSITION	Section No. 6 2% Lime 8% FA

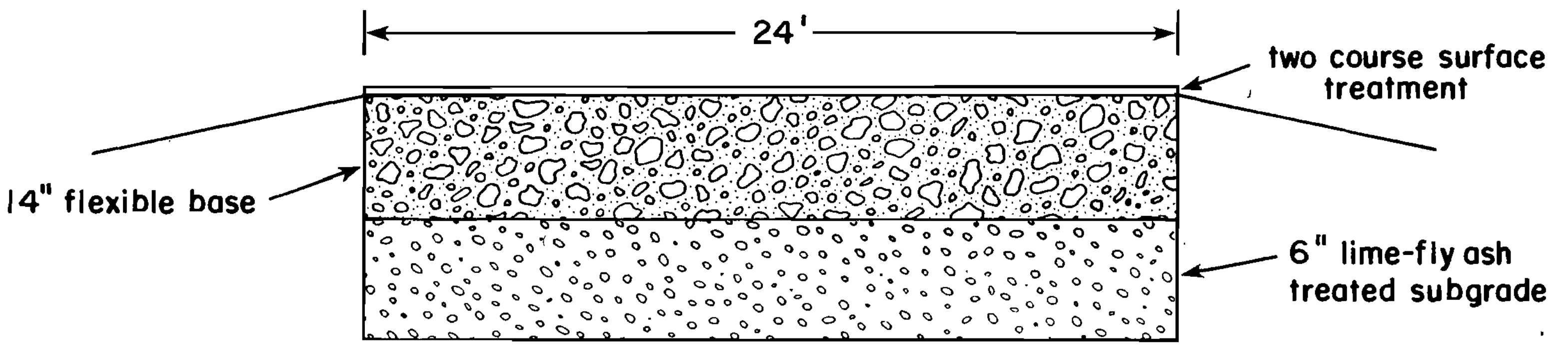

Figure A-4 Typical planview layout and section of west bound lane of FM 1604 in Bexar County, Texas (Site No. 4).

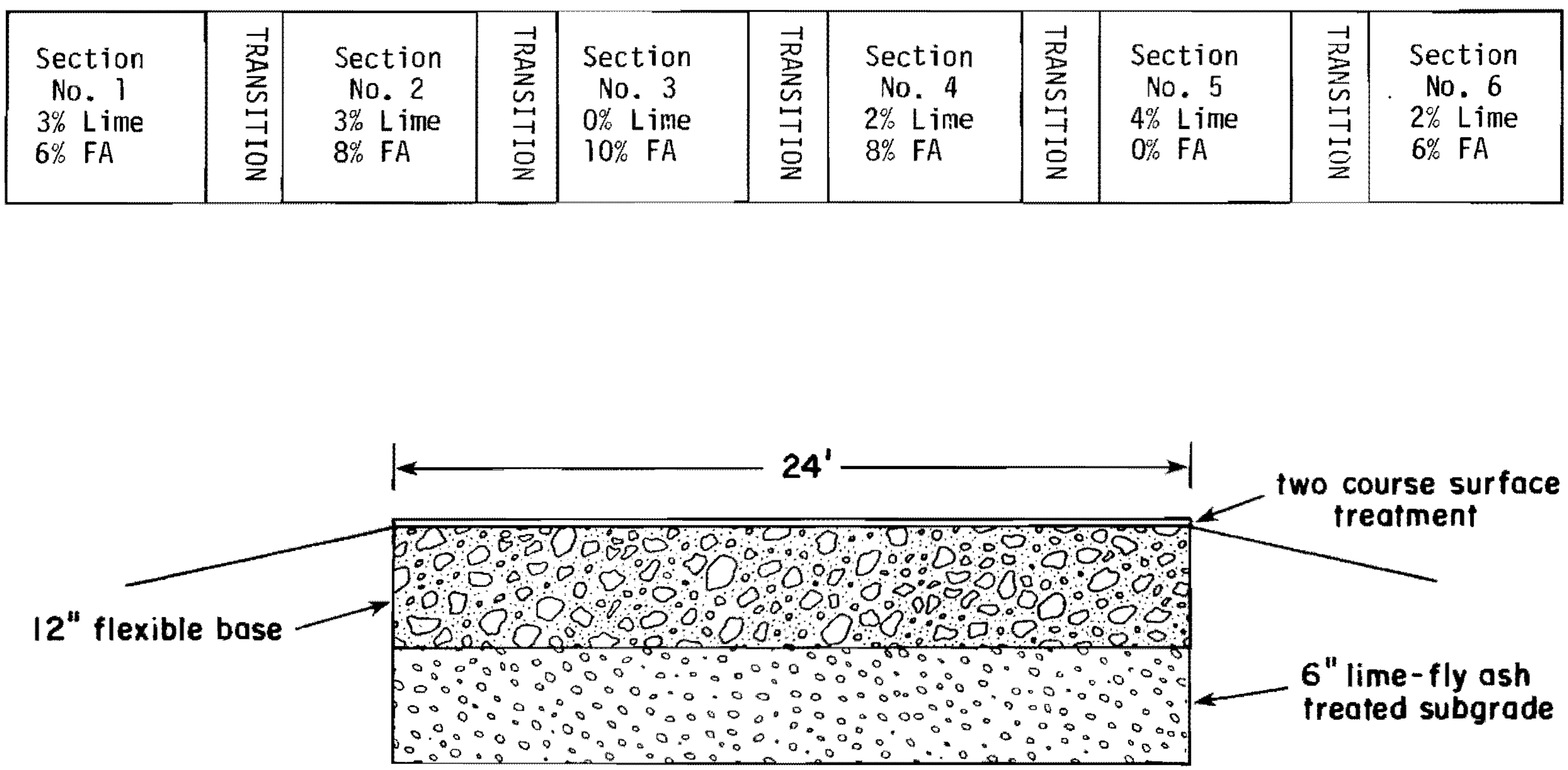

Figure A-5 Typical planview layout and section of west bound lane on FM 1604 in Bexar County, Texas (Site No.5).

Section No. 2 2% Lime 4% FA	TRANSITION	Section No. 3 2% Lime 4% FA	TRANSITION	Section No. 4 2% Lime 8% FA	TRANSITION	Section No. 5 3% Lime 6% FA	TRANSITION	Section No. 6 0% Lime 8% FA	TRANSITION	Section No. 1 3% Lime 0% FA

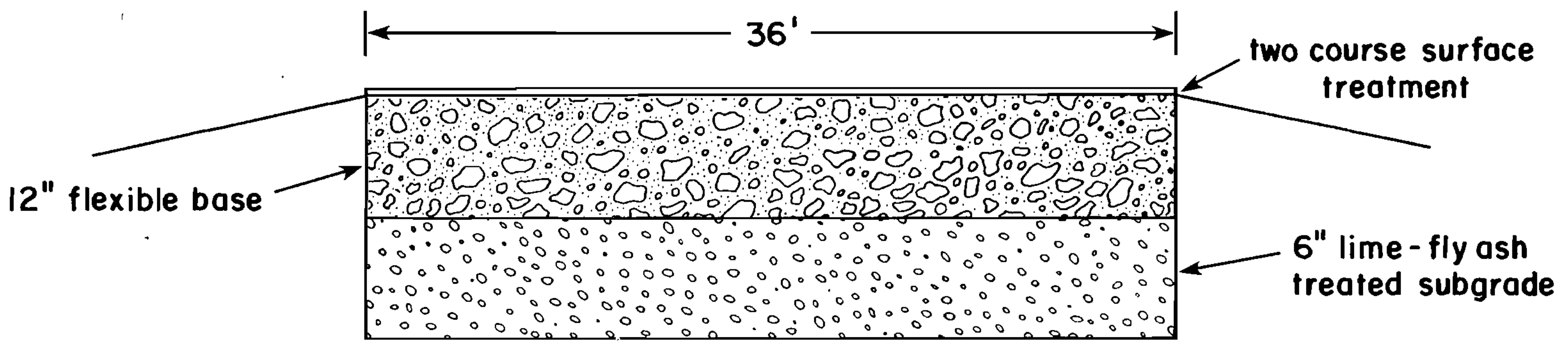

Figure A-6 Typical planview layout and section on SH 335 in Potter County, Texas (Site No.8).

Appendix B DEFLECTION MEASUREMENTS FOR TEST SITES

Table B-1 Deflection Measurements Based on the Dynaflect Results-Site No. 1 (3 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.523	.383	.265	.205	.151	58
2	.574	.415	.266	.192	.139	55
3	.874	.660	.492	.391	.312	62
4	.763	.616	.468	.386	.311	67
5	1.067	.819	.599	.475	.369	62
6	1.149	.753	.448	.343	.264	52
7	1.056	.708	.402	.269	.189	50
8	.964	.631	.378	.275	.202	51
9	1.010	.696	.436	.321	.232	53
10	1.499	.994	.592	.434	.319	51

Note: Deflections noted are in mil.

Table B-2 Deflection Measurements Based on Dynaflect Results- Site No. 1 (12 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.438	.339	.267	.219	.154	65
2	.526	.380	.272	.203	.137	58
3	.761	.616	.510	.420	.329	69
4	.718	.590	.482	.394	.307	69
5	.830	.669	.546	.439	.329	68
6	.965	.695	.485	.361	.264	57
7	.853	.650	.434	.304	.189	57
8	.718	.572	.414	.300	.204	62
9	.804	.639	.471	.353	.244	63
10	1.144	.850	.595	.448	.316	59

Note: Deflections noted are in mil.

Table B-3 Deflection Measurements Based on Dynaflect Results-
Site No. 1 (22 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.434	.348	.266	.211	.151	65
2	.527	.396	.272	.200	.141	58
3	.721	.613	.489	.414	.326	71
4	.790	.672	.492	.402	.316	68
5	.895	.740	.564	.461	.345	67
6	1.047	.735	.479	.364	.259	55
7	.959	.732	.476	.335	.211	57
8	.746	.567	.399	.302	.206	60
9	.860	.667	.457	.360	.258	61
10	1.120	.853	.564	.440	.316	59

Note: Deflections noted are in mil.

Table B-4 Deflection Measurements Based on the Benkleman Beam Results - Site No. 1 (3 month survey).

Test Section	W_1	W_2	S
1	.0075	.0062	91
2	.0074	.0051	85
3	.0077	.0058	88
4	.0055	.0042	88
5	.0087	.0065	87
6	.0110	.0063	78
7	.0107	.0063	79
8	.0095	.0052	77
9	.0093	.0055	80
10	.0205	.0174	92

Note: Deflections noted are in inch

Table B-5 Deflection Measurements Based on the Benkleman Beam Results - Site No. 1. (12 month survey).

Test Section	W_1	W_2	S
1	.0035	.0029	91
2	.0055	.0041	87
3	.0057	.0048	92
4	.0059	.0045	88
5	.0061	.0056	96
6	.0079	.0059	87
7	.0068	.0054	90
8	.0044	.0042	98
9	.0041	.0045	105
10	.0089	.0055	81

Note: Deflections noted are in inch

Table B-6 Deflection Measurements Based on the Benkleman Beam Results - Site No. 1 (2 year survey).

Test Section	W_1	W_2	S
1	.0041	.0020	74
2	--	--	--
3	.0061	.0035	79
4	.0060	.0026	72
5	.0072	.0037	76
6	.0186	.0032	59
7	.0083	.0027	66
8	--	--	--
9	.0062	.0026	71
10	.0097	.0029	65

Note: Deflections noted are in inch

Table B-7 Deflection Measurements Based on Dynaflect Results-
Site No. 2 (4 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.923	.732	.578	.478	.384	67
2	.689	.487	.343	.259	.193	57
3	.619	.478	.361	.274	.209	63
4	.511	.384	.271	.211	.153	60
5	.548	.427	.327	.268	.209	65
6	.915	.711	.547	.424	.335	64
7	.829	.649	.522	.432	.332	68
8	.802	.641	.504	.399	.300	66

Note: Deflections noted are in mil.

Table B-8 Deflection Measurements based on Dynaflect Results-
Site No. 2 (13 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.792	.595	.452	.346	.282	62
2	.601	.422	.310	.228	.174	58
3	.524	.388	.300	.230	.182	62
4	.445	.325	.242	.180	.131	60
5	.519	.383	.295	.229	.181	62
6	.814	.578	.442	.334	.262	60
7	.754	.581	.459	.358	.277	64
8	.739	.539	.425	.318	.243	61
9	1.274	.934	.714	.525	.413	61

Note: Deflections noted are in mil.

Table B-9 Deflection Measurements Based on Dynaflect Results-
Site No. 2 (24 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.808	.599	.472	.368	.281	63
2	.593	.426	.319	.244	.178	59
3	.534	.403	.316	.233	.184	63
4	.422	.311	.231	.176	.131	60
5	.477	.354	.262	.225	.190	63
6	.690	.501	.373	.299	.242	61
7	.630	.492	.382	.315	.253	66
8	.646	.487	.374	.304	.232	63
9	1.132	.868	.649	.523	.435	64

Note: Deflections noted are in mil.

Table B-10 Deflection Measurements Based on the Benkleman Beam Results- Site No. 2 (4 month survey)

Test Section	Lime/Fly Ash (subgrade)	W_1	W_2	S
1	4/0	.0092	.0059	82
2	2/4	.0096	.0058	84
3	2/10	.0067	.0051	88
4	4/8	.0058	.0037	82
5	4/16	.0050	.0034	84
6	0/16	.0087	.0049	78
7	2/24	.0081	.0057	85
8	2/15	.0070	.0051	86
9	0/21	-	-	-
10	4/4	-	-	-

Note: Deflections noted are in inch

Table B-11 Deflection Measurements Based on the Benkleman Beam Results- Site No. 2 (13 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	W_2	S
1	4/0	.0068	.0051	88
2	2/4	.0059	.0047	86
3	2/10	.0049	.0037	88
4	4/8	.0043	.0030	85
5	4/16	.0041	.0038	96
6	0/16	.0061	.0047	89
7	2/24	.0064	.0048	88
8	2/15	.0059	.0037	81
9	0/21	.0101	.0066	83
10	4/4	-	-	-

Note: Deflections noted are in inch

Table B-12 Deflection Measurements Based on Benkleman Beam Results Site No. 2 (24 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	W_2	S
1	4/0	.0101	.0033	66
2	2/4	.0066	.0027	71
3	2/10	.0041	.0028	84
4	4/8	.0048	.0017	68
5	4/16	.0052	.0021	70
6	0/16	.0106	.0033	66
7	2/24	.0090	.0049	77
8	2/15	-	-	-
9	0/21	-	-	-
10	4/4	-	-	-

Note: Deflections noted are in inch

Table B-13 Deflection Measurements Based on Dynaflect Results-
Site No. 3 (7 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.693	.525	.374	.290	.247	61
2	.725	.549	.377	.276	.217	59
3	.579	.448	.327	.246	.190	62
4	.539	.415	.332	.269	.231	66
5	.599	.462	.332	.253	.193	61
6	.662	.525	.386	.292	.241	64

Note: Deflections noted are in mil.

Table B-14 Deflection Measurements Based on Dynaflect Results-
Site No. 3 (12 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.574	.401	.279	.197	.158	56
2	.597	.411	.266	.171	.132	53
3	.502	.368	.252	.166	.130	57
4	.460	.341	.254	.192	.164	61
5	.515	.373	.262	.178	.143	57
6	.532	.422	.312	.220	.177	63

Note: Deflections noted are in mil.

Table B-15 Deflection Measurements Based on Dynaflect Results-Site No. 3 (19 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.579	.443	.330	.225	.176	61
2	.609	.461	.320	.205	.150	57
3	.491	.388	.288	.190	.144	61
4	.445	.364	.292	.214	.178	67
5	.519	.405	.296	.197	.156	61
6	.556	.472	.367	.258	.202	67

Note: Deflections noted are in mil.

Table B-16 Deflection Measurements Based on Benkleman Beam Results-Site No. 3 (12 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/6	.0038	.0023	80
2	3/10	.0048	.0035	87
3	2/5	.0036	.0019	76
4	4/0	.0032	.0016	75
5	2/8	.0035	.0022	81
6	0/12	.0026	.0011	71

Note: Deflections noted are in inch

Table B-17 Deflection Measurements Based on Benkleman Beam Results-Site No. 3 (19 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/6	.0042	.0018	71
2	3/10	.0051	.0021	71
3	2/5	.0043	.0015	67
4	4/0	.0030	.0017	78
5	2/8	.0038	.0017	72
6	0/12	.0054	.0020	69

Note: Deflections noted are in inch

Table B-18 Dynaflect Deflection Measurements Based on Dynaflect Results- Site No. 4 (6 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.988	.602	.442	.352	.288	54
2	1.054	.607	.420	.330	.266	51
3	1.200	.688	.457	.282	.265	48
4	1.498	.786	.595	.314	.303	47
5	1.237	.676	.473	.299	.287	47
6	1.255	.733	.589	.323	.308	51

Note: Deflections noted are in mil.

Table B-19 Dynaflect Deflection Measurements Based on Dynaflect Results- Site No. 4 (12 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.493	.435	.354	.291	.264	75
2	.537	.457	.349	.270	.235	69
3	.634	.504	.374	.284	.246	64
4	.792	.586	.417	.317	.274	60
5	.686	.505	.373	.294	.264	62
6	.643	.535	.413	.325	.292	69

Note: Deflections noted are in mil.

Table B-20 Dynaflect Deflection Measurements Based on Dynaflect Results- Site No. 4 (24 month survey)

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.530	.450	.382	.298	.260	73
2	.570	.462	.363	.275	.232	67
3	.594	.460	.360	.269	.225	64
4	.769	.556	.413	.306	.254	60
5	.703	.488	.366	.276	.242	59
6	.624	.500	.403	.315	.271	68

Note: Deflections noted are in mil.

Table B-21 Deflection Measurements Based on Benkleman Beam Results- Site No. 4 (6 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	W_2	S
1	4/0	.0037	.0031	92
2	3/6	.0041	.0033	90
3	3/9	.0055	.0038	85
4	0/10	.0076	.0060	90
5	1/5	.0060	.0045	88
6	2/8	.0053	.0045	93

Note: Deflections noted are in inch

Table B-22 Deflection Measurements Based on Benkleman Beam Results- Site No. 4 (12 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	W_2	S
1	4/0	.0030	.0027	95
2	3/6	.0044	.0037	92
3	3/9	.0053	.0045	93
4	0/10	.0058	.0041	85
5	1/5	.0051	.0039	88
6	2/8	.0047	.0042	95

Note: Deflections noted are in inch

Table B-23 Deflection Measurements Based on Benkleman Beam Results Site No. 4 (24 month survey).

Test Section	Lime/Fly Ash (subgrade)	W_1	W_5	S
1	4/0	.0039	.0025	82
2	3/6	.0049	.0022	72
3	3/9	.0046	.0028	80
4	0/10	.0066	.0021	66
5	1/5	.0065	.0017	63
6	2/8	.0047	.0015	66

Note: Deflections noted are in inch

Table B-24. Deflection Measurements Based on Dynaflect Results-Site No. 5. (8 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.506	.378	.281	.210	.185	62
4	.549	.405	.303	.288	.204	62
5	.518	.383	.289	.218	.193	62
6	.637	.417	.284	.196	.162	53

Note: Deflections noted are in mil.

Table B-25 Deflection Measurements Based on Dynaflect Results-
Site No. 5 (12 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1						
2	.523	.409	.274	.199	.162	60
3	.673	.515	.341	.249	.205	59
7	.943	.712	.447	.300	.231	56
8	.712	.526	.359	.258	.209	58

Note: Deflections noted are in mil.

Table B-26 Deflection Measurements Based on Dynaflect Results-Site No. 5 (16 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.499	.407	.348	.257	.213	69
2	.476	.378	.307	.216	.170	65
3[a]	.570	.440	.348	0	0	48
4	.557	.440	.365	.281	.239	68
5	.488	.390	.330	.258	.220	69
6	.634	.460	.343	.242	.193	59
7	.815	.641	.451	.311	.232	60
8	.617	.476	.358	.262	.207	62

Note: Deflections noted are in mil.

[a] W_4 and W_5 readings of site 3 recorded in error.

Table B-27 Deflection Measurements Based on Dynaflect Results-Site No. 5 (21 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.496	.406	.302	.239	.202	66
2	.463	.362	.260	.191	.159	62
3	.565	.439	.322	.243	.198	63
4	.549	.428	.329	.262	.225	65
5	.504	.404	.314	.250	.213	67
6	.595	.434	.301	.228	.186	59
7	.830	.628	.428	.309	.240	59
8	.609	.470	.342	.263	.215	62

Note: Deflections noted are in mil.

Table B-28 Deflection Measurements Based on Benkleman Beam Results Site No. 5 (9 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/6	.0035	.0025	86
4	2/8	.0033	.0026	89
5	4/0	.0032	.0027	92
6	2/6	.0048	.0024	75

Note: Deflections recorded are in inch

Table B-29 Deflection Measurements Based on Benkleman Beam Results Site No. 5 (12 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
2	3/8	.0039	.0020	76
3	0/10	.0048	.0024	75
7	0/25	.0075	.0042	78
8	0/30	.0058	.0036	81

Note: Deflections noted are in inch

Table B-30 Deflection Measurements Based on Benkleman Beam Results Site No. 5 (16 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/6	.0039	.0015	63
2	3/8	.0038	.0018	74
3	0/10	.0049	.0016	74
4	2/8	.0038	.0013	67
5	4/0	.0034	.0014	71
6	2/6	.0063	.0015	62
7	0/25	.0078	.0022	64
8	0/30	-	-	-

Note: Deflections noted are in inch

Table B-31 Deflection Measurements Based on Benkleman Beam Results- Site No. 5 (21 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/6	.0047	.0007	57
2	3/8	.0085	.0021	62
3	0/10	.0095	.0024	63
4	2/8	.0088	.0021	62
5	4/0	.0081	.0032	70
6	2/6	.0111	.0019	59
7	0/25	.0141	.0081	92
8	0/30	.0092	.0027	65

Note: Deflections noted are in inch

Table B-32 Deflection Measurents Based on Dynaflect Results-
Site No. 8 (6 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.720	.491	.303	.212	.143	52
2	.705	.500	.350	.253	.188	57
3	.779	.573	.408	.301	.226	59
4	.783	.573	.403	.282	.214	58
5	.786	.565	.362	.255	.186	55
6	.793	.535	.335	.235	.164	52

Note: Deflections noted are in mil.

Table B-33 Deflection Measurements Based on Dynaflect Results-Site No. 8 (12 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.636	.410	.254	.191	.137	51
2	.672	.447	.308	.237	.193	55
3	.763	.508	.354	.273	.218	56
4	.712	.481	.337	.262	.211	56
5	.682	.458	.313	.237	.184	55
6	.684	.435	.288	.215	.158	52

Note: Deflections noted are in mil.

Table B-34 Deflection Measurements Based on Dynaflect Results-Site No. 8 (26 month survey).

Test Section	W_1	W_2	W_3	W_4	W_5	S
1	.523	.393	.285	.218	.146	60
2	.520	.421	.331	.443	.187	73
3	.632	.508	.386	.301	.212	65
4	.590	.475	.371	.286	.207	65
5	.550	.445	.343	.267	.179	65
6	.527	.416	.313	.242	.160	63

Note: Deflections noted are in mil.

Table B-35 Deflection Measurements Based on Benkleman Beam Results-Site No. 8 (6 month survey)

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/0	.0063	.0043	84
2	2/4	.0063	.0046	87
3	2/4	.0067	.0037	78
4	2/8	.0056	.0039	85
5	3/6	.0056	.0038	84
6	0/8	.0072	.0051	85

Note: Deflections noted are in inch

Table B-36 Deflection Measurements Based on Benkleman Beam Results Site No. 8 (12 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/0	.0067	.0038	78
2	2/4	.0065	.0033	75
3	2/4	.0070	.0042	80
4	2/8	.0073	.0036	75
5	3/6	.0057	.0032	78
6	0/8	.0069	.0033	74

Note: Deflections noted are in inch

Table B-37 Deflection Measurements Based on Benkleman Beam Results Site No. 8 (24 month survey).

Test Section	Lime/Fly Ash	W_1	W_2	S
1	3/0	.0052	.0031	80
2	2/4	.0047	.0029	80
3	2/4	.0053	.0031	80
4	2/8	.0054	.0035	83
5	3/6	.0054	.0038	85
6	0/8	.0050	.0032	82

Note: Deflections noted are in inch