LOUISIANA EXPERIMENTAL BASE PROJECT

FINAL REPORT

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ABSTRACT

The Louisiana Experimental Base Project is a research study evaluating the design/performance characteristics of three types of base courses as incorporated into comparable flexible pavement systems on a full-scale test road. Fourteen different test sections were constructed to evaluate the study variables, which included base course type (soil cement, stabilized sand clay gravel, and asphaltic concrete), design life (5, 10, and 15 years), and surface thickness (3 1/2 and 5 1/2-inches). The test road is U.S. Route 71, south of the city of Alexandria.

Fundamental engineering properties of paving materials were determined in the laboratory using a variety of tensile and compressive tests utilizing repeated loading techniques. Layer moduli were also determined from field deflection tests using Dynaflect data. Field monitoring of serviceability and structural number were compared to trends derived from AASHTO equations. Measured values of serviceability decline, cracking, and rutting were also compared to predicted values using the VESYS IIIA program.

Vehicle load equivalency factors were evaluated using Weigh-In-Motion data collected during the course of the study. Structural layer coefficients for design were examined in terms of commonly specified materials' properties and in terms of fundamental materials' properties. Resilient moduli were measured and compared to R-value results at typical optimum moisture contents for sand, silts, and clay soils.

Observations are made regarding materials' properties determination for design, distress mechanisms affecting performance, and performance prediction models.

METRIC CONVERSION FACTORS*

To Convert from	To	Multiply by
	Length	
foot	meter (m)	0.3048
inch	millimeter (mm)	25.4
yard	meter (m)	0.9144
mile (statute)	kilometer (km)	1.609
	Area	
square foot	square meter (m^2)	0.0929
square inch	square centimeter (cm^2)	6.451
square yard	square meter (m^2)	0.8361
	Volume (Capacity)	
cubic foot	cubic meter (m^3)	0.02832
gallon (U.S. liquid)**	cubic meter (m^3)	0.003785
gallon (Can. liquid)**	cubic meter (m^3)	0.004546
ounce (U.S. liquid)	cubic centimeter (cm^3)	29.57
	Mass	
ounce-mass (avdp)	gram (g)	28.35
pound-mass (avdp)	kilogram (kg)	0.4536
ton (metric)	kilogram (kg)	1000
ton (short, 2000 lbs)	kilogram (kg)	907.2
	Mass per Volume	
pound-mass/cubic foot	kilogram/cubic meter (kg/m^3)	16.02
pound-mass/cubic yard	kilogram/cubic meter (kg/m^3)	0.5933
pound-mass/gallon (U.S.)**	kilogram/cubic meter (kg/m^3)	119.8
pound-mass/gallon (Can.)**	kilogram/cubic meter (kg/m^3)	99.78
	Temperature	
deg Celsius (C)	kelvin (K)	$t_K=(t_C+273.15)$
deg Fahrenheit (F)	kelvin (K)	$t_K=(t_F+459.67)/1.8$
deg Fahrenheit (F)	deg Celsius (C)	$t_C=(t_F-32)/1.8$

*The reference source for information on SI units and more exact conversion factors is "Metric Practice Guide" ASTM E 380.

**One U.S. gallon equals 0.8327 Canadian gallon.

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INTRODUCTION

During the mid 1960's the Louisiana Department of Transportation adopted the design procedure established for flexible pavements at the AASHTO Road Test at Ottawa, Illinois. Subsequently, this Department's research engineers verified that in general this design procedure could be applied to the soil types, traffic loadings, and environmental conditions of Louisiana.

Limited funds, materials shortages, and the advent and appeal of new pavement design concepts render "general" verification of the above mentioned design procedure inadequate. The Department's design engineers need to know more precisely the accuracy of their design predictions as reflected by actual performance of flexible pavements. This research project is intended to provide the needed information by facilitating a comparison of section design and performance.

The trend of developing fundamental engineering properties for highway paving materials such as are used in other types of construction brings with it the need to explore new test methods, new expressions of strength and to provide comparisons to currently specified strength indices. Performance predictors other than those described by AASHTO design relationships need to be studied using appropriate materials strength data to compare predicted versus actual performance. This research study will provide the basis for determining the fundamental engineering characteristics of each material using a variety of approaches and for making performance comparisons which should indicate the potential for use of such methods.

SCOPE

The scope of the research project is as follows:

1. Determine the accuracy of vehicle load equivalency factors used to compute design loads.

2. Determine representative fundamental materials' properties and compare the magnitudes of these properties as determined by a variety of laboratory and field tests, such as field-measured surface deflections.

3. Determine layer equivalencies for three different base course materials by inference from observations at some common level of performance.

4. Determine the accuracy of predictors of pavement performance using the AASHTO design relationships, as well as other predictive models based on fundamental materials' properties.

EXPERIMENTAL BASE CONSTRUCTION AND TEST PROGRAM

The methodology involved the design and construction of 18 full-scale test sections on an actual highway project. The test sections represent various combinations of base course type, base course and surface course thicknesses, and design life.

The experimental sections are located on Route U.S. 71 and Route U.S. 167 in central Louisiana. Figure 1 is a map depicting the facility between the communities of Meeker and Chambers in Rapides Parish. The location of the facility represents a compromise between the low wetlands of south Louisiana and the slight hills in the northern part of the state. The terrain at the test site is flat and affords poor drainage. Subgrade material is a relatively uniform, fine-grained soil. The mean variation in air temperature at the test site in 1977 was from 39°F to 84°F. The annual rainfall was typically 55.6 inches.

The road is a major rural route, accommodating a moderate volume of mixed (automobile and truck) traffic. A vehicle count station is located on the highway near the test sections and provides historical and current traffic counts. Weigh-In-Motion (WIM) load frame-transducer assemblies were installed in the outside lane immediately north of the test area. The WIM analyzes the traffic stream with regard to vehicle type, number of axles, weight (per axle and total), axle spacing and speed. Independent vehicle classification at the test site was accomplished manually at selected intervals.

The test facility was constructed as part of a larger project to upgrade the route to a four-lane highway. The experimental section was thus surveyed, planned, bid upon, and constructed as part of the overall project. The contractor completed placement of embankment and an overlying select soil in 1975. In the summer of 1976, the experimental base courses and the surface course were completed. Fourteen test sections were thus built with certain experimental

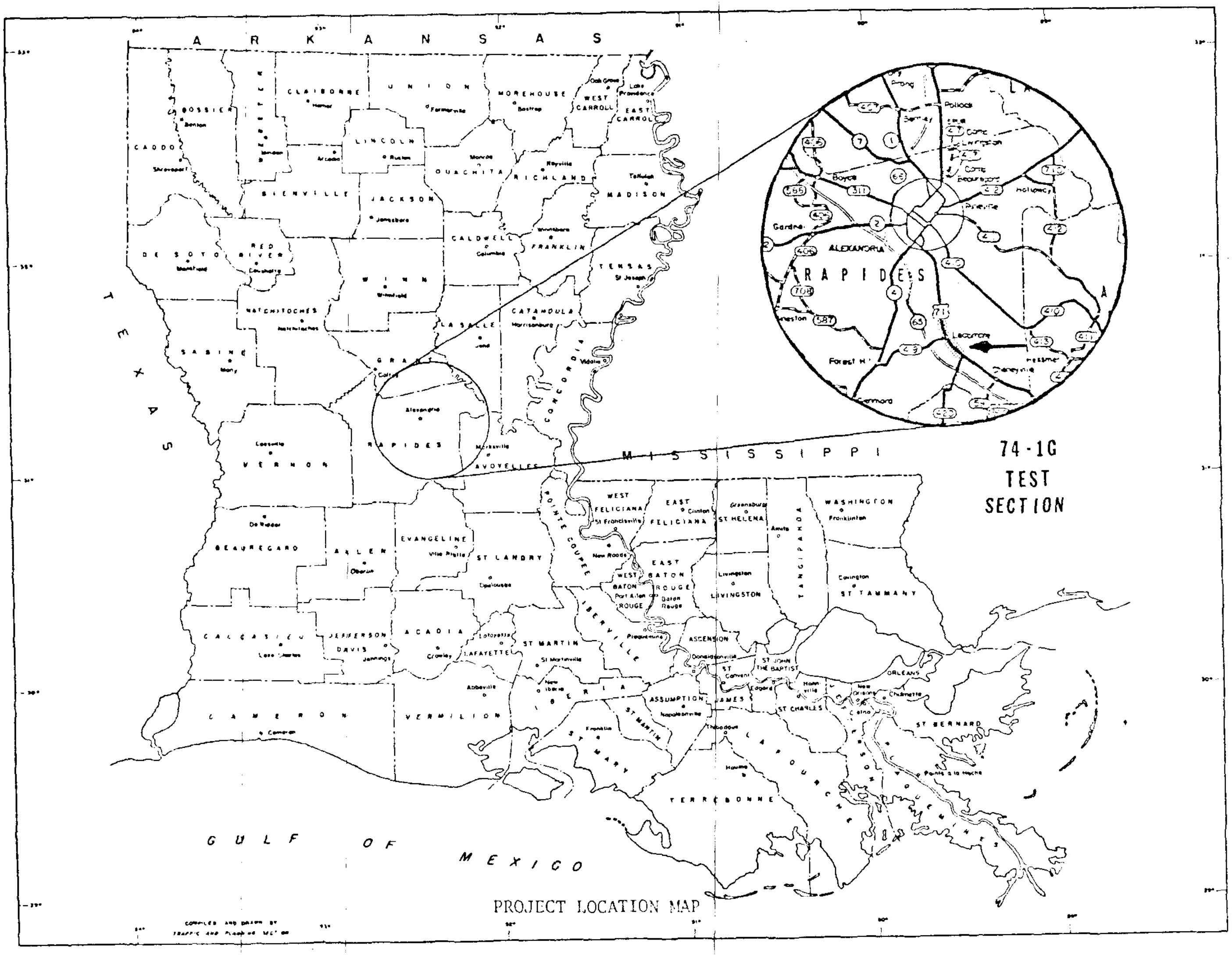


FIGURE 1

Project Location Map

report. Four additional control sections were built for reference--one at each end of the experimental area and two within the research area. The roadway was opened to traffic in August, 1976.

Construction of the test sections was closely monitored, materials were sampled and roadway tests conducted in accordance with a construction sampling and testing program, Table 12, Appendix A. The program outlined in Table 12 is designed to determine laboratory properties of the component road materials and to define the as-built properties of the ensuing (road) structure. A discussion of construction monitoring and materials sampling is included as Appendix B.

Changes in the as-built properties of the experimental sections were determined through a measurements program, Table 13, Appendix A. The purpose of this was to provide an evaluation of materials, traffic, and road performance in pursuit of design verification.

In Table 12 the Asphalt Institute is listed as an agency to perform tests in this research project. The Asphalt Institute determined certain fundamental properties of the materials comprising the experimental sections and at the same time was considering using these properties with a design procedure such as "PDMAP" or "VESYS" to forecast the distress-related life of the test sections.

Louisiana Technical University also provided support to the experimental base course project. The University conducted indirect tensile and fatigue tests on base and surface course materials from the test sections. These laboratory test results were used with layered theory analysis programs and actual road performance results to verify or develop design procedures.(1)

(1)Numbers refer to cited references.

EXPERIMENTAL DESIGN

Basis of Experimental Design

The present theory pertinent to the effect of materials and layer thickness on performance is not fully adequate. To provide the needed answers to the development of theory and obtain some type of empirical answers to the structural road design problems, field experiments are needed. The experiments involve the construction of test sections of specified materials and thickness. The performance of these sections must then be compared in order to seek the needed design concepts.

The designer of these experiments faces several major problems, the foremost being the cost of building the test sections. This means a minimum of replications. By replication it is meant that more than one test section is constructed under a given set of specified variables.

A second major problem concerns the time required to obtain results. An experimenter does not have time to wait a prolonged period before analyzing and interpreting the experimental data. The alternative is to design thinner sections and extrapolate to thicker sections. One of the basic purposes of road test section experiments is to develop a basis for extrapolation.

The other major problem in experimental design is the impracticability of using balanced factorial designs. As an example, consider Material A which provides a useful road life of 25 years for a given thickness. Material B, for the same thickness, may last only half as long. Thus using the same thickness for each material would lead to extended waiting time for some materials before final results become available. The alternative is to select certain levels of performance in terms of design life (5, 10, and 15 years, for example). The problem then would be to assign thicknesses for each material which would provide the desired performance goal.

Experimental Design

On the basis of the above philosophy and the definition of the major objective of the study, an experiment was designed to provide a set of thickness levels for each material. Specifically, the experiment was designed to include two factors relative to the pavement structure and one factor relative to average daily load in terms of design life. The main elements of this experimental design are shown in Table 1. It includes asphaltic concrete surface thickness at two levels (3.5 inches and 5.5 inches), base course material at two levels (asphaltic concrete or black base and cement stabilized soil), and design life at three levels (5, 10, and 15 years). The levels of thickness for each base course material were determined using AASHTO design procedures and the following Louisiana coefficients for material components:

Surface course	0.44	
Base course	0.34	for asphaltic concrete
	0.15	for cement stabilized soil
	0.18	for cement stabilized sand clay gravel

The coefficients for asphaltic materials were adjusted as follows in 1979 to better relate design values to field Marshall properties [2]: surface course = 0.40, base course = 0.33.

TABLE 1

EXPERIMENTAL DESIGN

Surface Thickness	3 1/2"			5 1/2"		
Design Life	Base Type			Base Type		
	Black Base	Soil-Cement	Stab. S.C.G.	Black Base	Soil-Cement	Stab. S.C.G.
5 years	6"	12"	10"	3"	6"	6"
10 years	7 1/2"	15"		4 1/2"	9"	
15 years	11"	20"		8"	16"	

The levels of the pavement surface thickness were chosen because these are the predominant levels encountered in Louisiana. The levels of design life were chosen so as to provide adequate data points for detection of failure of design life (or some transformed function of design life) and its relationship to the two pavement surface thicknesses.

All combinations (2 x 2 x 3) of the factor levels were constructed at a single site without replications. However, all sections refer to two-lane sections. Analysis and evaluation of performance will be for outer lane sections only.

In addition to the 12 sections discussed above, two additional sections of cement stabilized sand clay gravel were constructed under each level of surface course at a single level of design life (5 years). Only limited comparison is anticipated from these sections.

The above 14 experimental test sections were supplemented with four control sections, one at each end of the project and two within the experimental area. These control sections were composed of 5-1/2 inches of surface course over 7 inches of asphaltic concrete base course and 6 inches of soil-cement.

Figure 2 is a profile of the various test sections. Each test section is approximately 550 feet long with a transition of 50 feet between each test section. The 10- and 15-year design sections were placed in random order. However, the 5-year design sections (sections 9 through 14) were grouped together to allow an overlay of the entire segment should early manifestation of distress become evident in any or all of these sections.

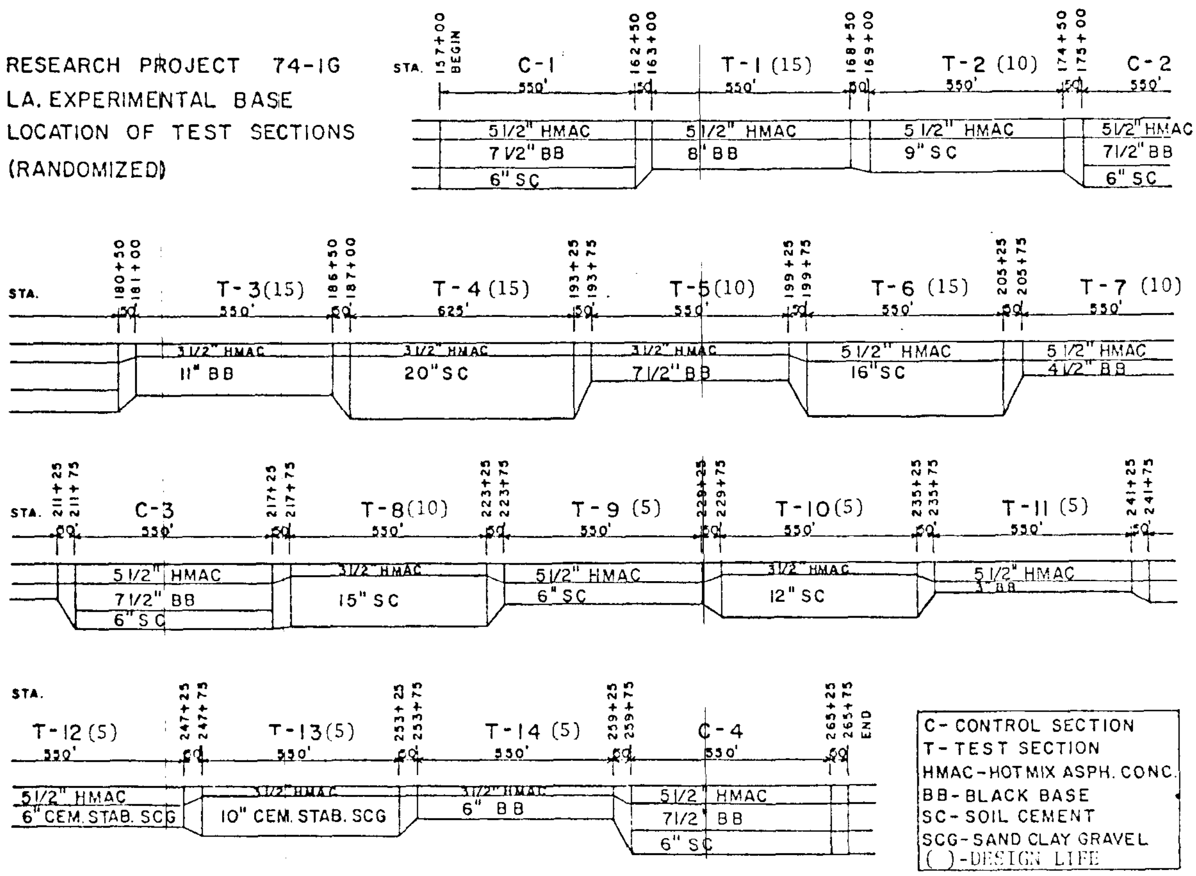


FIGURE 2

Design Profile of Test Sections

Analysis Using Experimental Design

In the recommended experimental design, each of the two base course materials has its own set of thickness levels. In essence, the thickness of each material can be considered as a controlled variable and hence should be assumed to possess no error except that of achieving the specified thickness. The random error will be in the performance response of the experimental sections.

It is anticipated that the recommended design would yield experimental data similar to those shown in Figure 3. In the figure the ordinate is plotted on the log scale and the thickness of the base course on arithmetic scale. Furthermore, the ordinate scale represents the number of years required for the performance index to decline to some specified value. This performance index could be psi, rutting, cracking, deflection or some other manifestation of distress. In the example shown, the ordinate represents design life or life to some terminal measure of performance.

If performance is regressed on thickness, and if it is assumed that the lines will be approximately parallel (and this can be done by choosing scales which will linearize and produce parallelism among performance-thickness regression lines), then it is possible to compare thickness of the two material types for equivalent performance. The performance equations would be of the following form:

$$\log P = a_{BB} + bT_{BB}$$

and

$$\log P = a_{SC} + bT_{SC}$$

For equivalent performance

$$T_{SC} = \frac{a_{BB} - a_{SC}}{b} + T_{BB}$$

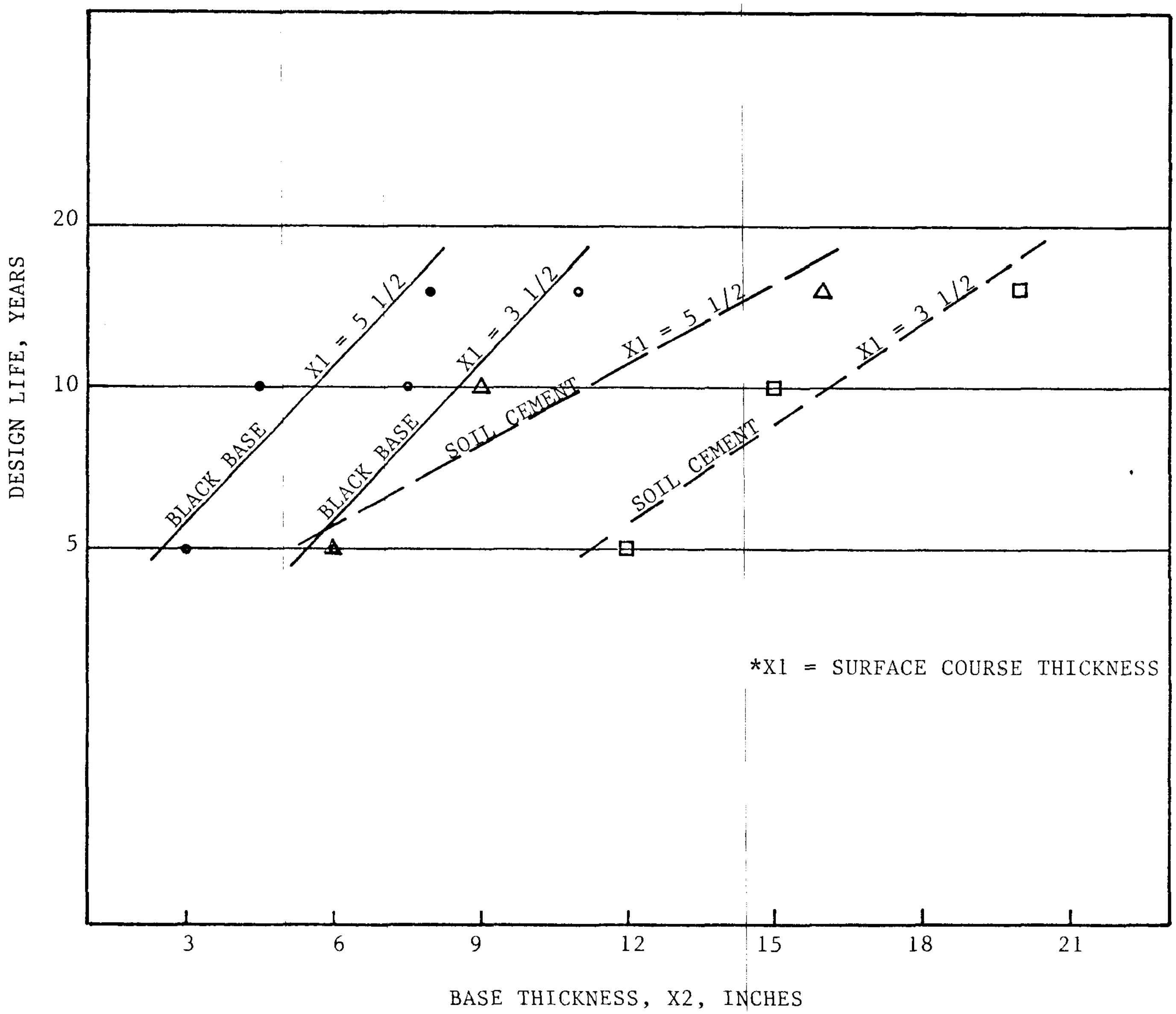


FIGURE 3

Relationship Between Base Thickness
And Design Life

Thus there is a linear relationship between the thicknesses of the material required for equivalent performance.

One of the shortcomings of the recommended design is that it will not be possible to detect the nonlinearity between the surface thickness levels and performance. A minimum of three levels of a factor are required to detect such nonlinearity of the regression of performance on thickness.

DETERMINATION OF TRAFFIC VOLUME AND LOAD

Sampling Scheme

In order to assimilate information on the response of the various pavement test sections to traffic loading through time, it was necessary to develop a scheme for the monitoring of both traffic weight and traffic volume throughout the test area.

The basic plan involved the use of a semi-automated Weigh-In-Motion (WIM) system built by Unitech, Inc. of Austin, Texas, and a Streeter-Amet model 401 vehicle classifier. These two pieces of equipment were operated simultaneously for seven 24-hour days each quarter at a site just north of the northernmost test section. Manual classification counts were eventually substituted for the vehicle classifier due to equipment inaccuracies.

Operations were planned so that 24-hour weight and volume data was available for a seven-day "typical week" each calendar quarter. This provided data which was summarized and/or separated to yield yearly, quarterly, monthly, weekly, daily, and hourly traffic information as needed for the period 1976-1983.

Traffic Sampling Equipment Description

The WIM system basically includes a series of load cells and detector loops installed in the roadway and an instrumented trailer containing electronic measuring and recording equipment. Two "loop detectors" a fixed distance apart provide a speed trap for computing speed. Actuation of a third detector loop, the vehicle presence detector, initiates the weighing and dimensioning program and relates succeeding axles to the transducers. The wheel load transducers containing the aforementioned load cells are powered by signal conditioning equipment, and their output is amplified so that each signal is compatible with an analog-to-digital converter. Each scale voltage signal is sampled 1,200 times per second by the analog-to-digital converter as it is switched between scale outputs by the multiplexer. The digital data is processed by a digital controller and the results may be typed and/or recorded on magnetic tape.

A correlation study completed by the Department in 1975 compared WIM data from 173 trucks, varying in weight and numbers of axles and traveling at various speeds with weight data obtained from permanent weight enforcement scales located nearby. The study showed the WIM system to be capable of a quite acceptable $\pm$ 5 percent total weight accuracy when properly installed, operated and maintained.

PAVEMENT PERFORMANCE MEASUREMENTS

Pavement performance was monitored using the Mays Ride Meter and Chloe devices to measure serviceability indices and the Dynaflect and Benkelman beam devices to measure surface deflections, in conjunction with standard crack mapping and rutting measurements to characterize pavement distress.

Dynaflect, Benkelman Beam Measurements

Dynaflect deflection measurements were made as often as necessary to characterize seasonal changes in deflection levels. Tests were conducted at 100-foot intervals, primarily in the outside wheelpath of the outside lane.

The Benkelman beam was used to provide a correlation with Dynaflect deflections and to establish representative deflection levels under a full-scale, standard 18-kip axle load. Figure 4 depicts correlations of Dynaflect and Benkelman beam deflections from this study and from other agencies.

Structural Evaluation by Means of Visual Condition Survey

Visual condition surveys were conducted to determine the occurence of fatigue cracking and permanent deformation (rutting) in the project. Rut depths were measured with the standard (four-foot) AASHO Road Test A-Frame Rut Depth Device depicted in Figure 5. Measurements were taken at 50-foot intervals in alternate wheelpaths of the outside lane and measured to the nearest tenth of an inch. Cracks and patches were measured with regard to frequency and dimensions.

Additional distress signs such as stabilized base course pumping and wearing of the surface course were also noted.

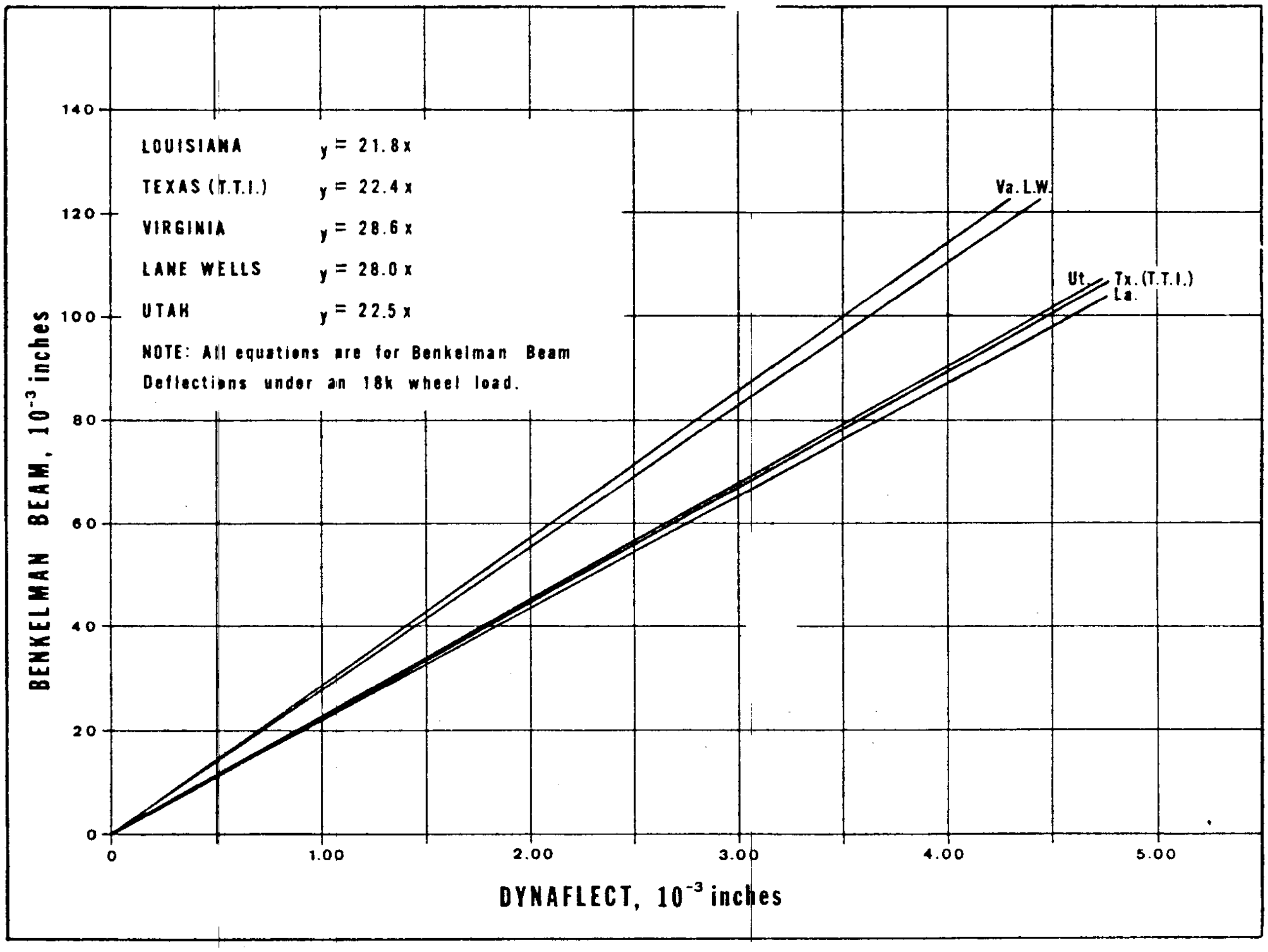


FIGURE 4

Benkelman Beam - Dynaflect Correlations

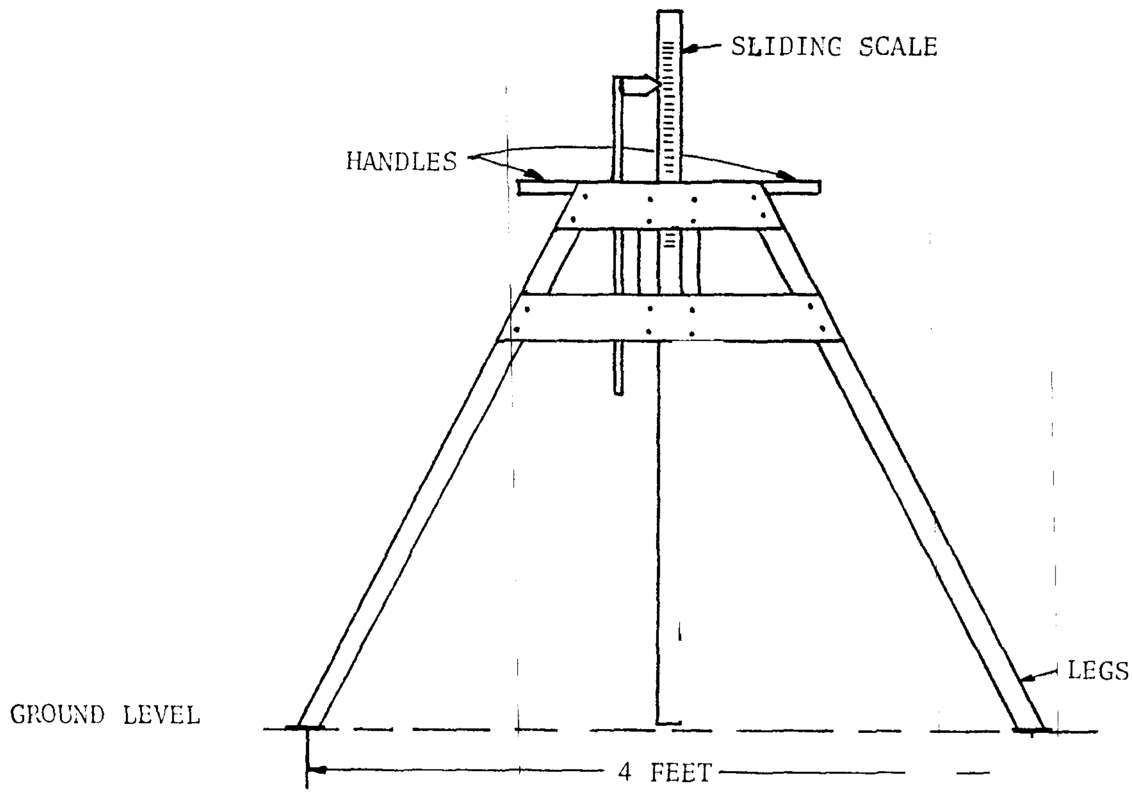


FIGURE 5

AASHO Road Test A-Frame Rut Depth Device

Functional Evaluation by Means of the Chloe Profilometer

The Chloe Profilometer provides a measure of road roughness as it sums the variation in road profile over a given distance. The resulting parameter is termed slope variance and has been used with measurements of cracking, patching, and rutting in the AASHO Road Test equations to express the Present Serviceability Index (P.S.I.) for each test section. P.S.I. is a numerical index which can range between 0 and 5 and denotes the relative manner in which a pavement functions under traffic.

The Chloe Profilometer, as described in the Federal Highway Administration's "Chloe Profilometer Operating and Servicing Instructions," is essentially two units; the trailer unit, which carries the transducing mechanism, and the electronic computer indicator. The electronic computer indicator accepts information from the transducer, performs a computation on it and then indicates the results. The slope transducer, carried at the rear of the 20-foot trailer, is comprised of two 8-inch wheels mounted 9 inches on centers, a roller contact on an upright arm fastened at the pivot point between the wheels, and a printed circuit switch with 29 active segments. The transducer provides a continual measure of the angle between the bar connecting the slope wheels and the arbitrary reference of the trailer unit. A slotted disc-photocell combination, attached to one of the carriage wheels, produces a command to sample pulses at 6-inch intervals of highway travels.

At each 6-inch interval, sample pulses are produced through the 29 active segments. The computer squares these segments and also accumulatively sums the numbered segments, the squares and the number of 6-inch intervals traveled (number of samples). Standard forms are used to record the data accumulation as well as the subsequent calculations.

Functional Evaluation by Means of the Mays Ride Meter (M.R.M.)

The M.R.M. records road roughness as reflected by movement of the vehicle's axle with respect to its chassis. A transmitter attached to the differential collects this movement information and feeds it forward to a portable recorder. Quantitative and qualitative roughness measurements are presented on a strip chart produced by the recorder.

M.R.M. measurements are reported in terms of Present Serviceability Index (P.S.I. has been defined as a "numerical index ranging from 0.0 to 5.0 of the ability of a pavement in its present condition to serve traffic.") A perfectly smooth pavement would have a P.S.I. of 5.0. A pavement so rough as to be impassable would have a P.S.I. of 0.0.

More specifically, a numerical-adjective description of P.S.I. is as follows:

4.1 - 5.0	Very Good
3.1 - 4.0	Good
2.1 - 3.0	Fair
1.1 - 2.0	Poor
0.0 - 1.0	Very Poor

Serviceability index (S.I.) values reported herein were determined using a correlation between the Mays Ride Meter and the University of Texas (General Motors) Surface Dynamics Profilometer, which in turn relates to actual panel ratings.

Serviceability index data determined from Chloe Profilometer measurements are designated S.I.(Chloe), and S.I. data which was determined from Mays Meter output and then converted to Chloe S.I. is designated as Chloe from Mays Meter.

LAB TESTING OF LABORATORY-PREPARED SAMPLES AND FIELD CORES

Fundamental engineering properties of the construction materials used to construct the eighteen test sections were determined through a cooperative testing program involving the Materials Research Laboratory of Louisiana Tech University, the Asphalt Institute, and the Research and Development Section of the Louisiana DOTD.

Static and resilient indirect tensile tests were conducted by the Materials Research Laboratory on laboratory-prepared specimens to obtain estimates of modulus, Poisson's ratio, tensile stress, tensile strain and cycles to failure for the wearing, binder, and black base materials as well as for soil cement and cement stabilized sand-clay gravel layers. A repetitive (fatigue) testing program was also conducted to investigate the fundamental resilient properties of field cores representative of in-service conditions.

The LADOTD test program involved static, indirect tensile test evaluations of a variety of mixture variables, including material type, asphalt content, gradation, compactive effort, compaction temperature, and age. All specimens for Louisiana Tech and DOTD were prepared at the DOTD research lab according to a randomized preparation plan.

The Asphalt Institute test program consisted of dynamic modulus determinations for both asphaltic laboratory specimens and field cores, flexural fatigue evaluations of laboratory prepared asphaltic specimens, and repeated load triaxial compression testing of select and embankment soils. California bearing ratio values were also determined for the select and embankment soils.

Table 2 contains a listing of typical materials' properties determined during the materials characterization phases of the study.

TABLE 2

TYPICAL MATERIALS PROPERTIES

ASPHALTIC CONCRETE (AC-40)

Plant Mix Properties - LA.DOTD

Course	Marshall Stability (lbs.)	% A.C.	% Air Voids
Wearing	1810	4.5	9.0
Binder	1517	4.3	7.4
Base	1355	3.9	7.6

Fundamental Properties (25°C) - Asphalt Institute

Course	Dynamic E* (1Hz) x 10^5 psi Lab Molded	Dynamic E* (1Hz) x 10^5 psi Field Cores	Initial Stiffness, Flexural Fatigue E x 10^5 psi
Wearing	4.2	5.0	3.6
Binder	4.8	5.8	4.0
Base	4.5	4.3	3.9

Fundamental Properties (25°C) - Materials Research Lab

Course	Indirect Tensile, Resilient E x 10^5 psi Lab Molded	Indirect Tensile, Resilient E x 10^5 psi Field Cores	Poisson's Ratio
Wearing	4.8	5.7	0.25
Binder	4.6	5.4	0.27
Base	4.0	5.1	0.26

TABLE 2 (CONT'D)

TYPICAL MATERIALS PROPERTIES

Cement Stabilized Base - Field Cores

Soil Cement (8-10% Cement)

	Mean	Range
28 Day Compressive (psi)	638	275 - 1224
Elastic Modulus (tensile) psi x 10^5	4.5	1.9 - 8.8
Poisson's Ratio	0.25	0.11 - 0.40
Elastic Modulus (compression) psi x 10^5	3.7*	1.1 - 6.3
Stabilized Sand/Clay - Gravel		
28 Day Compressive (psi)	356	184 - 528
Elastic Modulus (tensile) psi x 10^5	3.5	1.0 - 4.4
Poisson's Ratio	0.25	0.15 - 0.50
Elastic Modulus (compression) psi x 10^5	2.6	1.7 - 3.2

* Combination of
8% Cement, E = 3.1 x 10^5 psi and
10% Cement, E = 4.2 x 10^5 psi

Select Soils

	PI	R-Value	CBR	Mr x 10^5	Poisson's Ratio
A-2-4(0) Sand	6	60	31	20	0.35
A-2-6(0) Sa/Cl/Lo	10	20	25	10	0.35

Embankment Soils

	PI	R-Value	CBR	Mr x 10^5	Poisson's Ratio
A-6(9) Si/Cl	14	22	17	11	0.35
A-7-6(20) Hv/Cl	40	5	7	5	0.35

The reader is referred to a series of five reports by Hadley which discuss the materials' test study design, the comprehensive laboratory-based material characterizations with analysis of variance, and applications of determined materials' properties to special finite element modeling and to several deterministic design and analysis programs. A portion of Hadley's summary report has been included as Appendix E.

The special finite element evaluations of the indirect tensile, the beam, the unconfined compression, and the confined triaxial test configurations led to the development of mathematical algorithms for computing resilient layer moduli.

Comparisons between theoretical and finite element solutions indicate that the values of modulus of elasticity, tensile stresses and strains can be over or underestimated using theoretical equations. Hadley modeled a variety of boundary conditions (flexible, rigid, with friction, frictionless) for the indirect tensile test and found that Hondros' equations underestimated tensile stresses by 14 to 60 percent. After modeling the beam test to include shear stresses in addition to bending stresses, conventionally computed values were found to represent an underestimate of from 17 to 20 percent. Similarly, by modeling the frictional resistance between the loading platten and test specimen, the unconfined compression test was found to underestimate moduli values by approximately 38 percent. Tests of significance were used to illustrate the fact that modulus values computed from compression and tension tests using the algorithms were not significantly different. Typical values of conventionally computed moduli and moduli computed using the algorithms are listed in Table 3.

TABLE 3

CORRECTED LAYER MODULI BASED ON IMPROVED MATERIALS CHARACTERIZATION EQUATIONS

Unconfined Compression Test / Beam Test

	Dynamic Modulus - psi x 10^5				Resilient Modulus - Lab psi x 10^5	
	Lab, E*	Ec*	Field Cores E*	Ec*	E	Ec
Wearing	4.2	5.1	5.0	5.5	3.6	5.6
Binder	4.8	5.7	5.8	6.3	4.0	6.2
Base	4.5	5.4	4.3	5.1	3.9	6.1

Indirect Tensile
Resilient Modulus
(psi x 10^5)

	LAB		FIELD	
	E	Ec	E	Ec
Wearing	4.8	5.8	5.7	6.8
Binder	4.6	5.5	5.4	6.5
Base	4.0	4.8	5.1	6.1

(Field Cores)

	Indirect Tensile		Unconfined Compression	
	Modulus E psi x 10^5			
	E	Ec	E	Ec
Soil Cement	4.5	5.4	3.7	5.1
Cement Stabilized SC/G	3.5	4.0	2.6	3.5

Ec = Corrected Modulus

ANALYSIS OF FIELD DATA

Traffic Data

Traffic data in terms of volume and equivalent 18,000-pound single axle loadings (EAL) were determined by two independent sources and methods as a part of the study. The Department's Traffic Section provided traffic volume (ADL) from tube counters and subsequently calculated EAL using historical relationships between traffic volume and load. For research purposes, the cumulative axle weighings from Weigh-In-Motion (WIM) data were combined with visual ADT and visual classification counts to produce daily, yearly, and a total accumulated traffic load history. A comparison of the ADT data may be found in Table 4.

TABLE 4

ANNUAL AVERAGE DAILY TRAFFIC COUNTS

Year	Traffic & Planning	Research & Development
1976	7,989	7,122
1977	8,439	8,448
1978	10,061	9,048
1979	9,458	9,030
1980	8,854	8,194
1981	9,453	8,424
1982	9,438	8,620
1983	Not Available	8,512

Vehicle weight equivalency factors were derived from WIM data by calculating the total of single, tandem, and tridum axle loadings for a given truck class and expressing this total load as a ratio of the number of trucks weighed. The result is a factor which typifies the EAL contribution of a single truck of a given class across the actual population of loaded and unloaded vehicles. The load factors developed in this study generally agree with 1982 loadometer factors developed for Louisiana, as indicated in Table 5. The factor for the 3-S-3 vehicle was found to be much lower than existing loadometer data; therefore, consideration should be given to revising this category for design purposes.

A comparison of accumulated load since construction, Table 6, indicates a favorable comparison between projected data from the Traffic Section and research data which represents many thousands of axle weighings over a seven-year period. Two reasons for the close similarity of the total load data are (1) the similarity of equivalency factors between the two data sources for the 3-S-2 vehicle, and (2) the heavy contribution to total load of that truck class, approximately 80 percent.

TABLE 5

VEHICLE LOAD EQUIVALENCY FACTORS

Vehicle Type	Traffic & Planning Factor* (W-4 Tables)	Computed Factor (WIM DATA)
Passenger Car	.0004	
Pickup	.0036	
2 axle, 4 tire	.0227	.36
2 axle, 6 tire	.2216	.36
3 axle	.4227	.76
2-S-1	.6274	.76
2-S-2	.9101	.92
3-S-1	.9101	.92
3-S-2	1.1186	1.36
3-S-3	3.7533	1.51

*As used in 1982

TABLE 6

SUMMATION OF EQUIVALENT 18-KIP LOADINGS SINCE CONSTRUCTION

	Σ EAL	
Year	T&P	WIM
1976	64,481	80,000
1977	259,940	290,000
1978	513,140	510,000
1979	743,455	760,000
1980	985,487	1,010,000
1981	1,270,807	1,220,000
1982	1,356,472	1,440,000
1985.5		1,660,000**

**Load can be estimated from the following equation; $\Sigma L = 0.2189\ (\text{Date} - 2.07) - 432.509$

Where ΣL = Total 18-kip equivalent loads since opening to traffic

Date = Year + fraction, i.e. 1985.5 = Through June, 1985

PAVEMENT PERFORMANCE

Distress -- Cracking, Rutting

Surface distress in the form of cracking and rutting was measured at six-month intervals from 1976 to 1985. Cracking was characterized as AASHTO class I, II, or III fatigue cracks occurring in the wheelpaths for the full-depth asphaltic concrete sections and shrinkage cracking, which reflected through the wearing and binder course layers of sections with cement stabilized bases. Tabulations of cracking and rutting for each section by date may be found in Table 14, Appendix C.

All of the full-depth asphaltic concrete sections (1,3,5,7,11,14) developed Class I fatigue cracks for 100 percent of their length within seven years of construction. Table 7 is a cracking summary which indicates the cummulative 18-kip axle loads between opening to traffic and the occurence of 50 and 100 percent cracking by crack type. Similarly, the total load to 0.3 inches and greater of rutting is indicated. Patching was required on several sections and is expressed in square feet.

After the Class I cracks had become well defined in the wheelpaths, each full-depth asphalt test section was cored in the outside wheelpath and at the center of the lane to determine the depth of wheelpath cracking. Assumptions used to define crack propagation in elastic-layered theory models usually indicate that fatigue cracking begins at the bottom of the asphalt layer and then progresses upward to the pavement surface. Surprisingly, however, the cracking indicated by the cores was confined to the wearing/binder course layers, where asphalt stripping was also found. The location and extent of the stripping found in the cores is indicated in Figures 6-10. In some instances the cores taken outside of the wheelpath at the center of the lane also contained stripping in the same zone, indicating that the stripping probably preceded cracking in the wheelpath.

TABLE 7

CRACKING, PATCHING, RUTTING
TOTAL EQUIVALENT AXLE LOAD X 10^6

Section	FATIGUE CRACKING				Rutting		PATCHING (SQ.FT.)	
	Class I		Class II					
No.	50%	100%	10%	20%	0.3"	>0.3"	50-100	>100
C-1	1.4	1.9	1.4	1.7				
C-2	1.4	1.6			1.4			
C-3	1.4	1.7			1.2			
C-4	1.4	1.9			1.4			
T-1	1.3	1.7	1.4	1.9	1.4			
T-3	1.3	1.7	1.5		1.2	1.9	1.6	
T-7	1.3	1.7	1.3	1.6	1.2	1.4		1.6
T-5	1.2	1.2	1.4	1.7	0.7	1.3		
T-11	1.3	1.7						
T-14	1.4	1.7			1.4			

Section	BLOCK CRACKING (Linear Feet)			PATCHING (SQ.FT.)	
No.	100	500	1000	50-100	>100
T-6	1.1	1.3			
T-4	0.6	1.0	1.2		
T-2	1.0	1.3		1.6	
T-8	0.4	0.5	0.8	1.3	
T-9	0.6	1.2			
T-10	0.4	0.6	1.0		
T-12	1.1	1.4			
T-13	0.4	0.7	1.2		

The extent to which stripping contributed to cracking and rutting is not specifically known; however, a relative determination can be made by comparing the generally better performance of test sections 11 and 14 to the performance of other full-depth asphalt sections as indicated in Table 7. The test sections on the north end, of the project (T-11,14) did not experience stripping, as indicated in Figures 6-10. A review of construction records indicated that none of the asphaltic concrete mixes used on the project contained antistrip additives. The test sections on the north end of the experiment where no stripping was found were paved with wearing course materials produced at a different plant than the remainder of the project, although under the same mix specifications.

As a result, test sections 11 and 14, which were five-year designs, experienced no Class II cracking and less rutting than test sections 1,3,5,and 7, which were 15- and 10-year designs. On two of the full-depth asphalt sections (T-3, a 15-year design, and T-7, a 10-year design) rutting and cracking progressed to a severity which required some patching in the outside wheelpaths after approximately eight years. This means that the occurence of stripping had a greater influence on performance than additional asphaltic concrete thickness or higher structural numbers. The close association of stripping and distress was an unfortunate development in light of the study objective of relating design to performance.

The four control sections which are essentially full-depth asphalt pavements constructed over a 6.5-inch cement stabilized working table, performed similarly to the other full-depth sections, although rutting and cracking were not as severe or extensive.

Test sections which contained cement stabilized bases experienced longitudinal and transverse cracking (block pattern), which reflected through the asphaltic concrete surface. Three cracking categories in Table 7 for test sections 2,4,6,8,10,12, and 13 are expressed in total linear feet (100, 500 and 1000 ft.), with the total load indicated prior to reaching each respective level of cracking.

FIGURE 6

Location of Asphalt Stripping

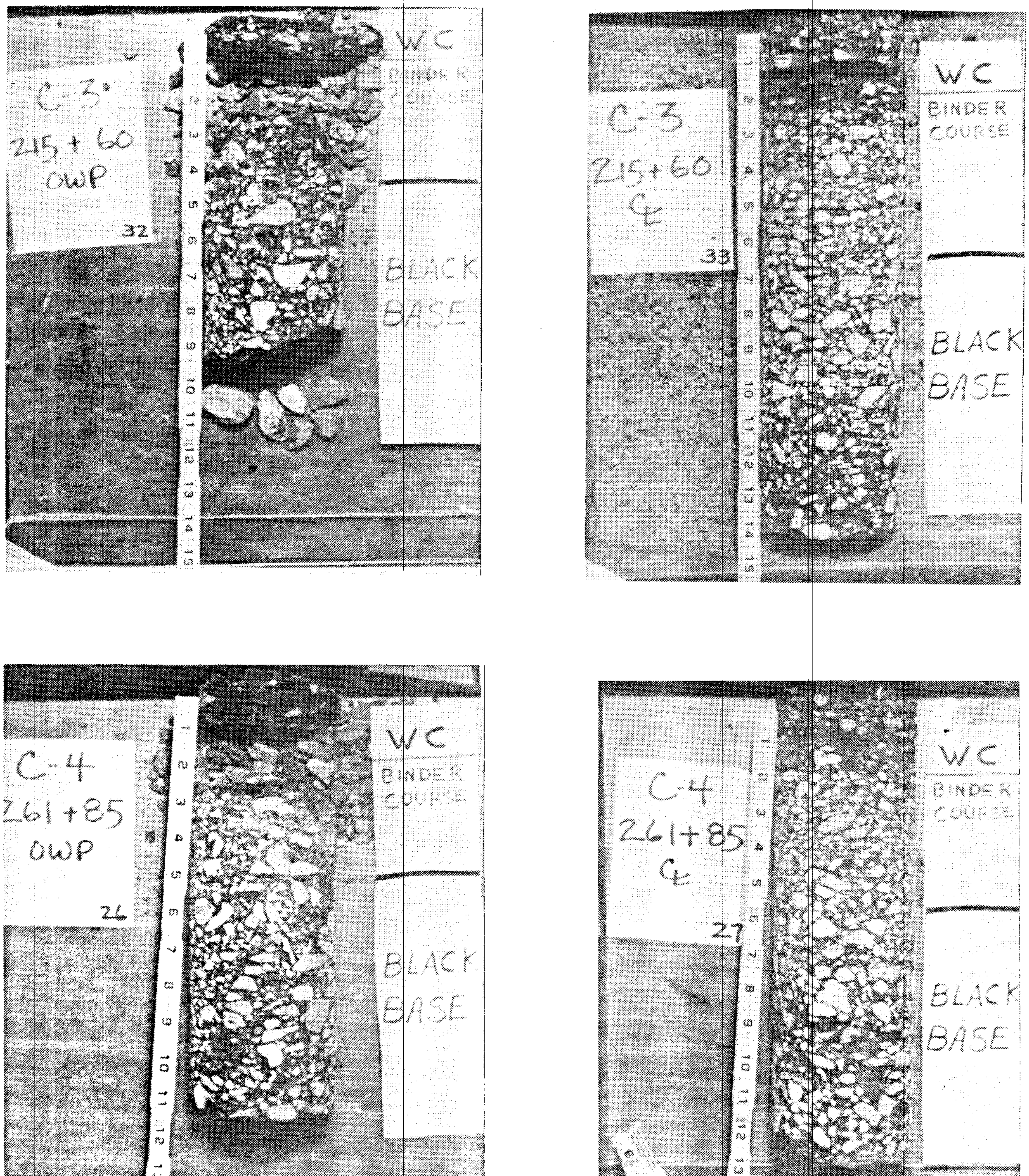


FIGURE 7

Location of Asphalt Stripping

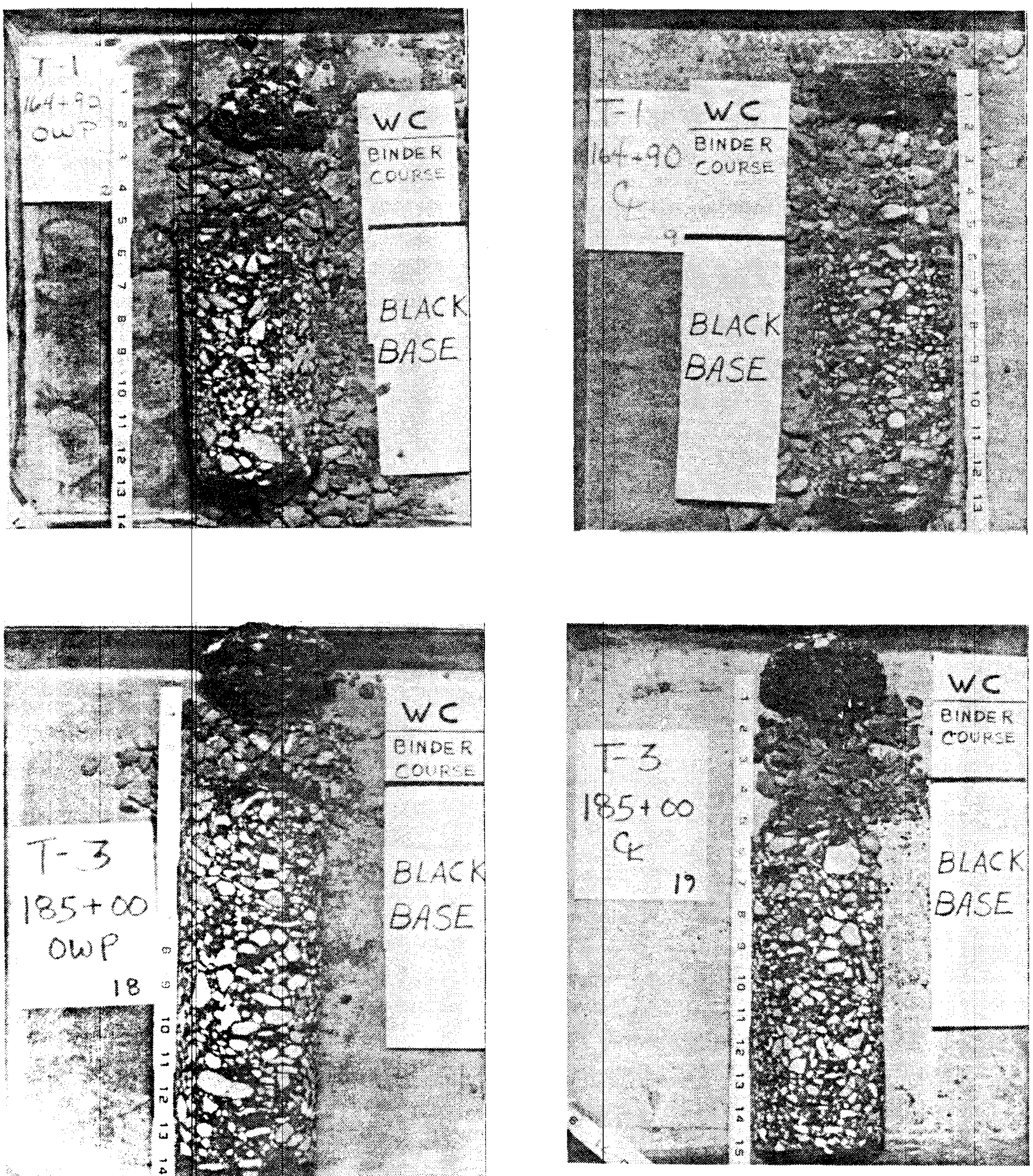


FIGURE 8

Location of Asphalt Stripping

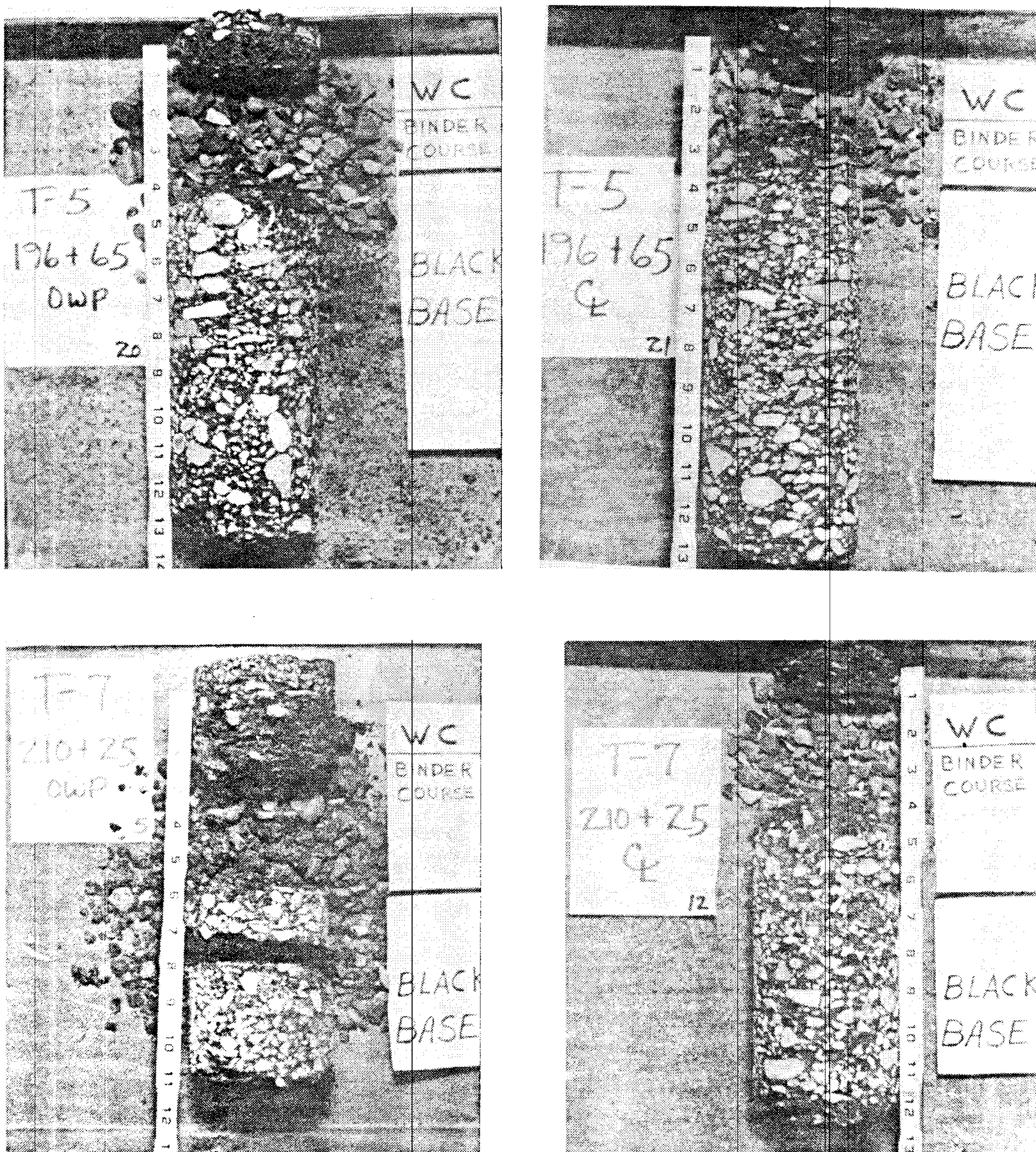


FIGURE 9

Location of Asphalt Stripping

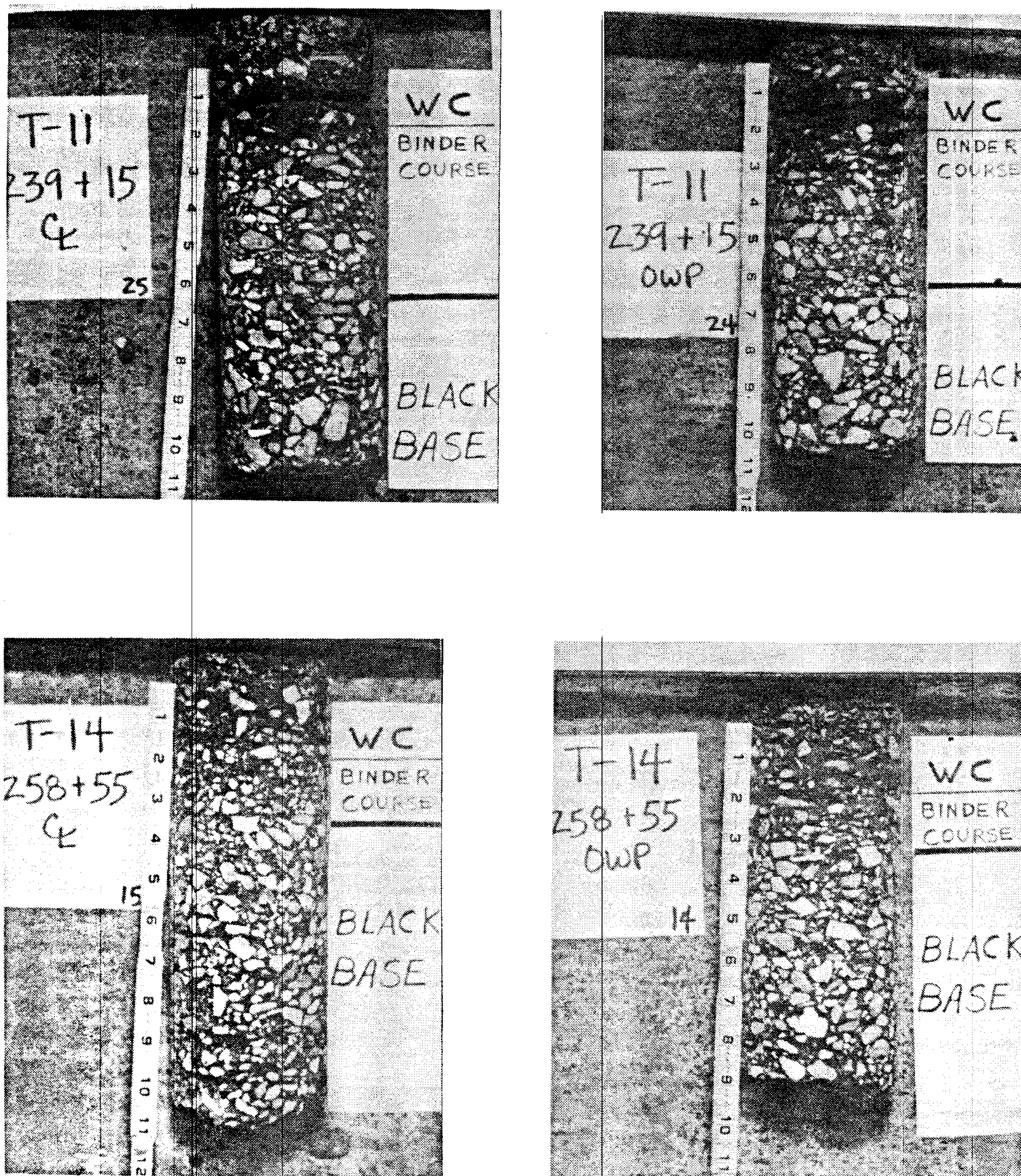


FIGURE 10

Location of Asphalt Stripping

Sections have been paired by design life, with the section containing thicker asphaltic concrete surfacing listed first in each pair. These sections (6,2,9,12) containing thicker surfacing experienced less total reflective cracking across all design levels, as indicated in Table 8, and, as will be shown later, the sections experienced less reduction in serviceability for equivalent loading. Figure 11 provides a pictorial comparison of reflective cracking for test sections 2 and 8.

Performance problems associated with reflective cracking were a result of loss of load transfer across the cracks under repeated heavy loads, which eventually caused differential settlement of the blocks, in excess of 0.5 inches in some instances. Pumping of fines through the cracks and severe ravelling of the asphaltic concrete surfacing along the cracks accelerated the deterioration process. Coring operations indicated a low recovery rate of good field cores in the thicker cement-treated base sections, where bases ranging from 12 to 20 inches were constructed in layers by in-place stabilization. The field cores contained a variety of cracks, laminations, compaction planes and layer separations, indicating a nonuniform base course. Test sections where in-place stabilization was accomplished in one lift (6-9 inches) and where the asphaltic concrete surfacing thickness was almost equal to that of the cement stabilized soil generally exhibited the best performance.

Test sections 12 and 13 contained a cement stabilized mixture of sand, clay, and gravel and were constructed to approximately equal structural numbers; one section with a thicker surfacing, one with a thicker base. Again, the pavement section with the thicker surfacing (base 30 percent thicker than surface) performed noticeably better than the section where the base was twice as thick as the surface in terms of cracking (severity and extent) and resulting serviceability loss.

TABLE 8

SURFACING TO BASE THICKNESS RATIO VS EXTENT OF BLOCK CRACKING

Sec#	HMAC / CTB	Ratio	Total (feet)
6	5.2 / 16.7	1:3.2	812
4	3.6 / 19.2	1:5.3	1595
2	6.4 / 9.0	1:1.4	933
8	4.0 / 15.3	1:3.8	2621
9	5.5 / 6.2	1:1.1	803
10	5.3 / 12.2	1:2.3	2390
12	6.1 / 7.7	1:1.3	769
13	4.6 / 9.8	1:2.1	1604
C1	12.4 / 7.4	1:0.60	50
C2	14.5 / 6.5	1:0.45	0
C3	13.6 / 6.5	1:0.48	0
C4	12.8 / 6.8	1:0.53	0

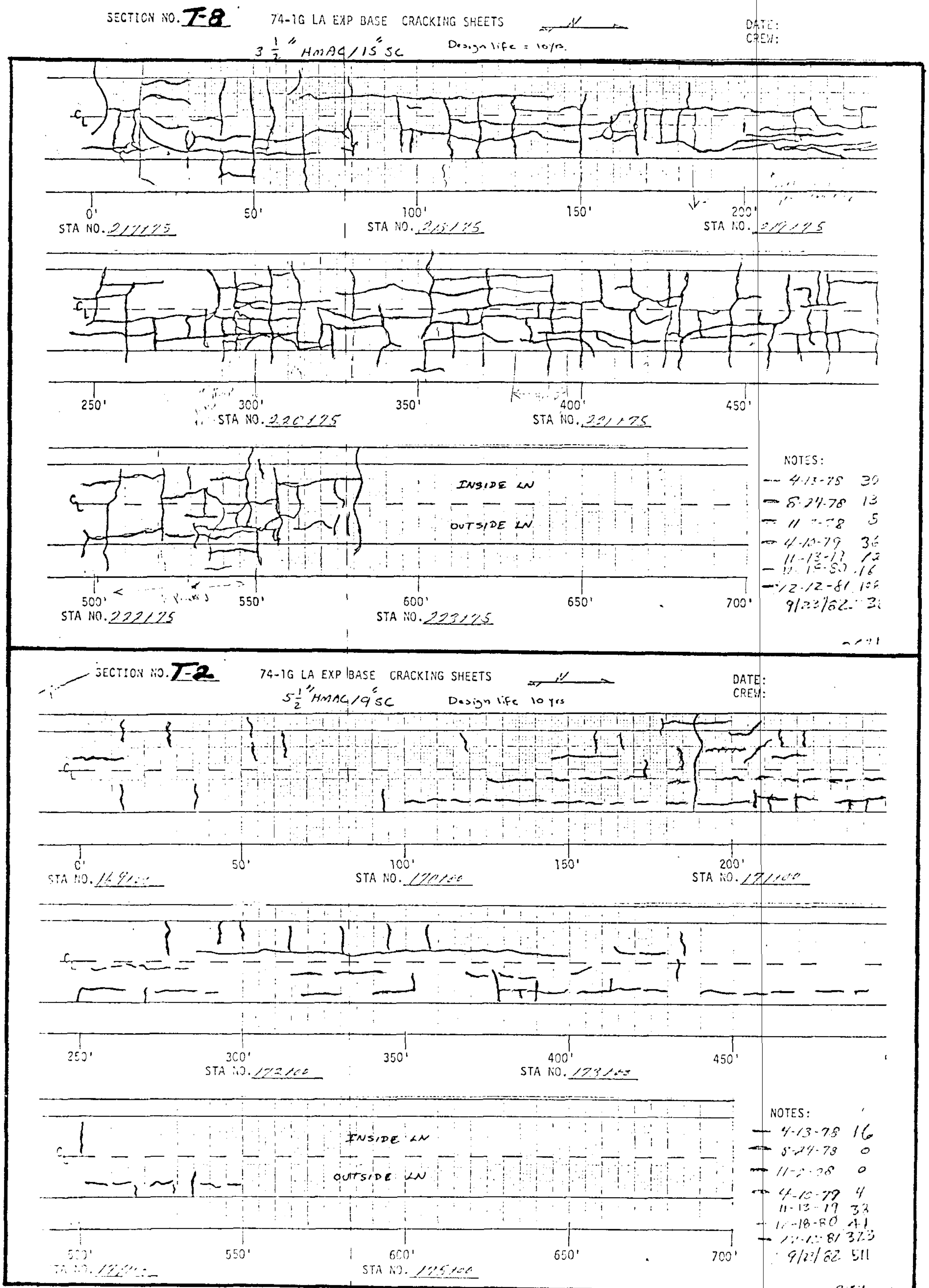


FIGURE 11

Shrinkage-Block Type Cracking
Soil Cement Base

A comparison of deterministically developed performance and distress predictions with performance and distress measurements taken on this project was accomplished as a part of a contract research study conducted by the Materials Research Laboratory located at Louisiana Tech University in Ruston, La. (1) The FHWA-developed Vesys IIIA predictive program was used to predict cracking, rutting and serviceability loss for each of the eighteen pavement sections in this study.

Results of the comparison of predicted versus actual cracking were complicated by the initiative of fatigue cracking in the wearing/binder course layer, as opposed to the expected crack development at the bottom of the black base layer. The VESYS IIIA program predicted almost instantaneous cracking in some black base test sections and no fatigue cracking for 20 years in others. Wheelpath cracking was actually initiated near the wearing-binder course interface where asphalt stripping began. Since the stripping was not accounted for in the original materials properties test values, agreement between actual and predicted cracking was not expected.

Another researcher, Khosla (3), investigating the predictive capabilities of VESYS IIIA, recently reported that the program was a good predictor of cracking indices, except for sections which were susceptible to shrinkage (reflection) cracking or which contained thick bituminous layers. All of the test sections in the experimental base study fall into one or the other of these two categories.

Rutting data provided a more favorable comparison of predicted and measured test values using the VESYS IIIA model, as presented in Figures 12-15. On two soil cement sections (T-8, T-10) measured, rutting values were higher than predicted due to the extreme surface distortion which developed as a result of block cracking settlement.

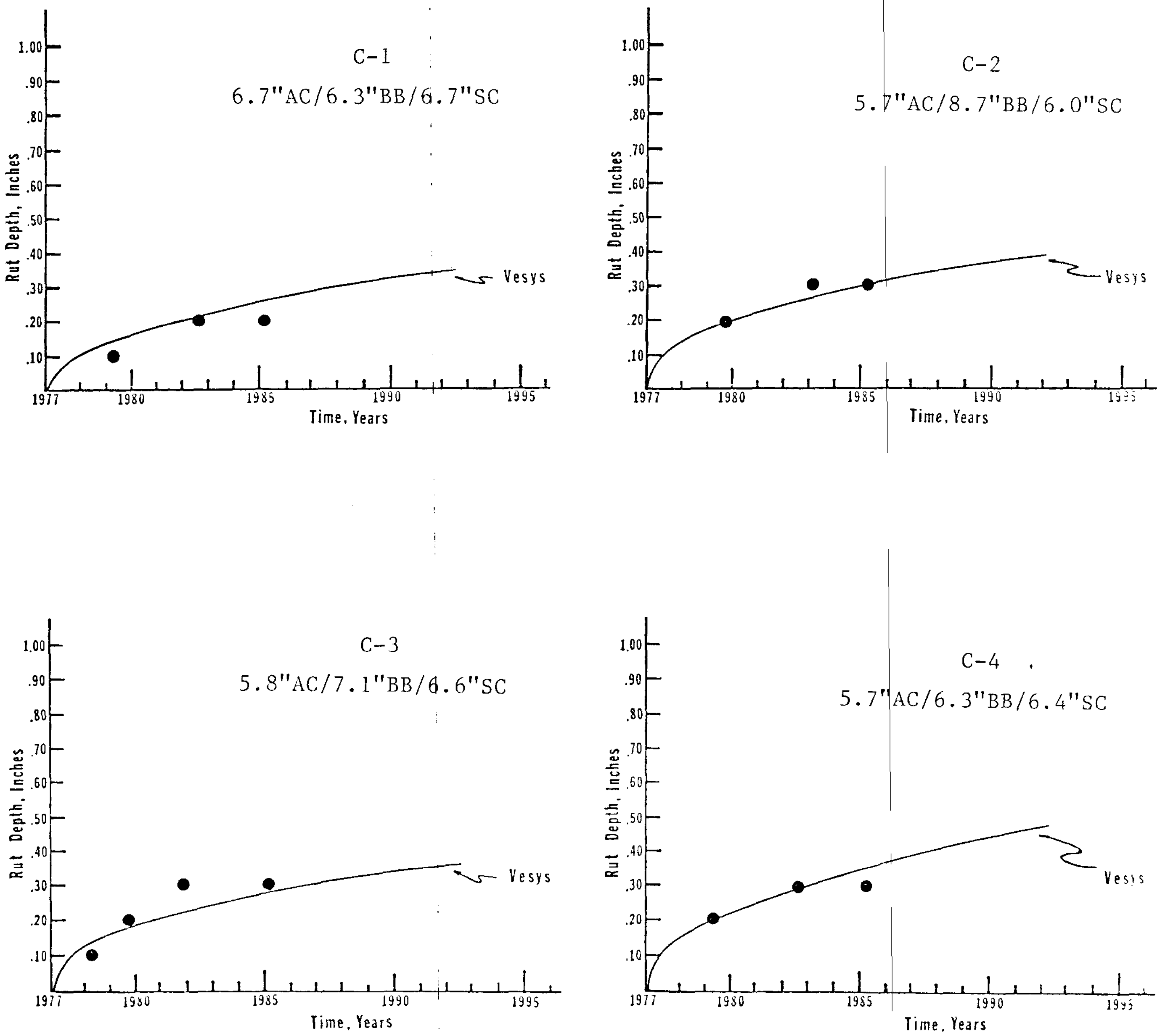


FIGURE 12

Measured vs. Predicted Rutting
Control Sections
(20-year Designs)

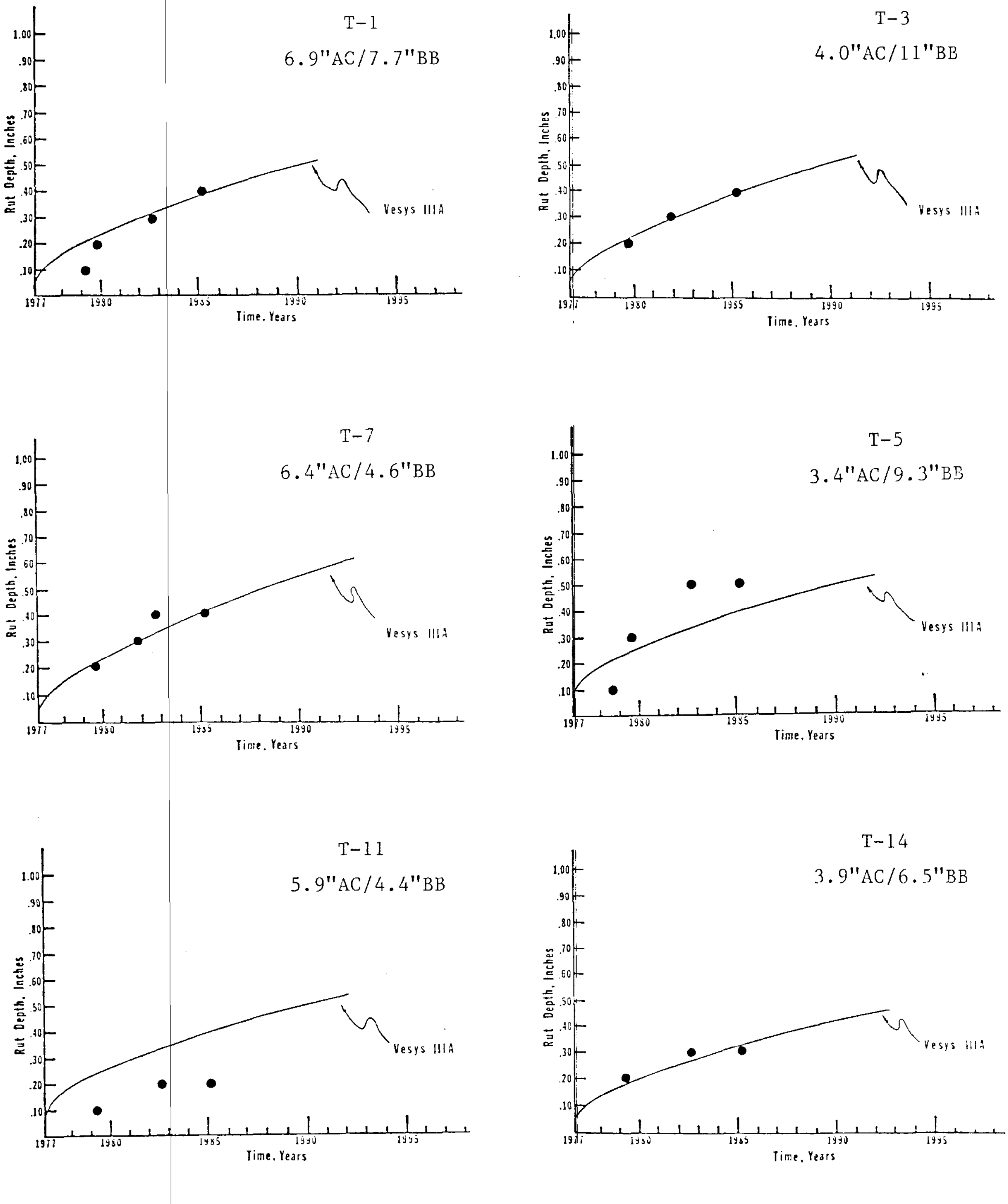


FIGURE 13

Measured vs. Predicted Rutting
Full-Depth Asphaltic Concrete
(15-year, 10-year, 5-year Designs)

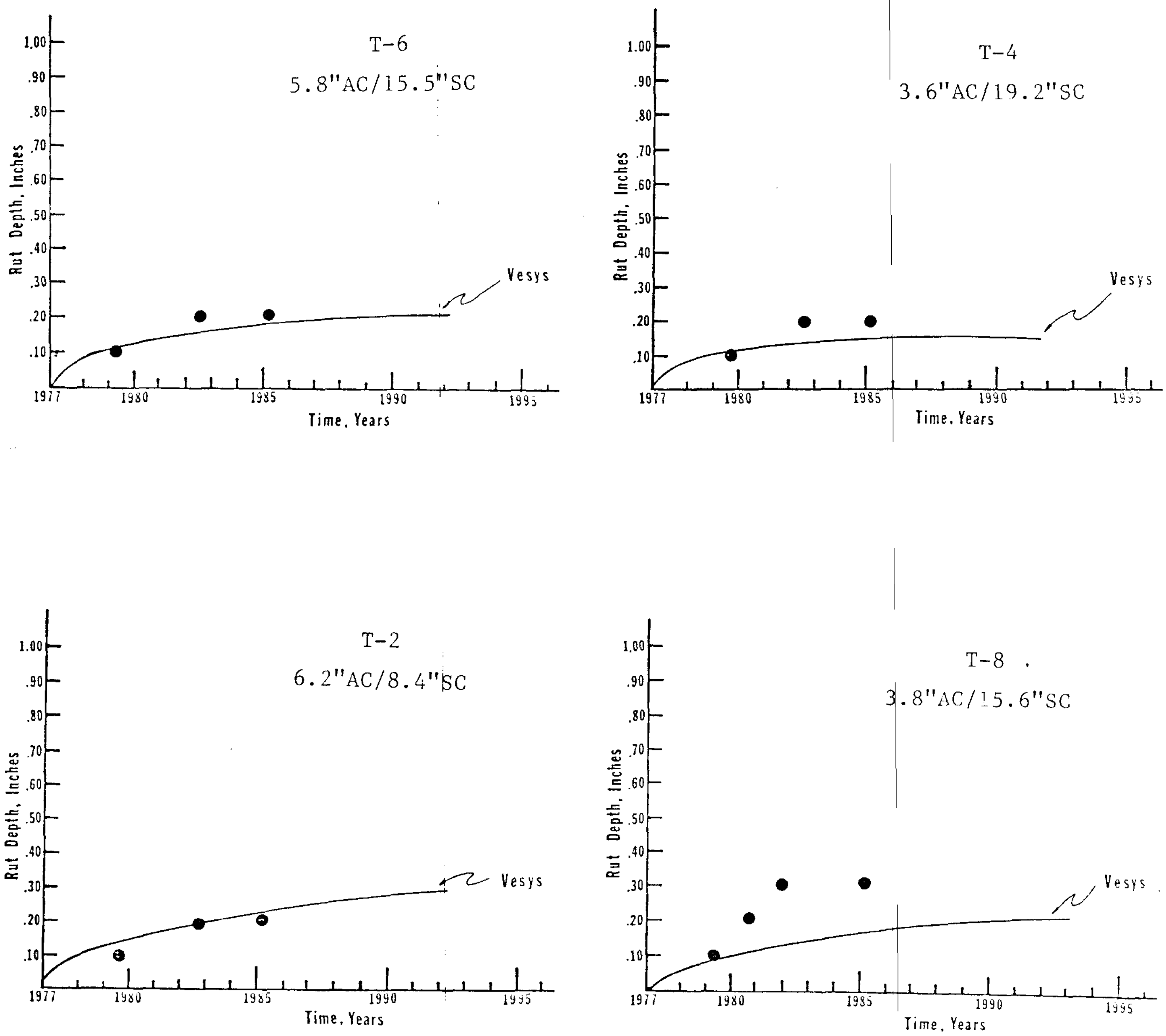


FIGURE 14

Measured vs. Predicted Rutting
Cement Stabilized Base
(15-year and 10-year Designs)

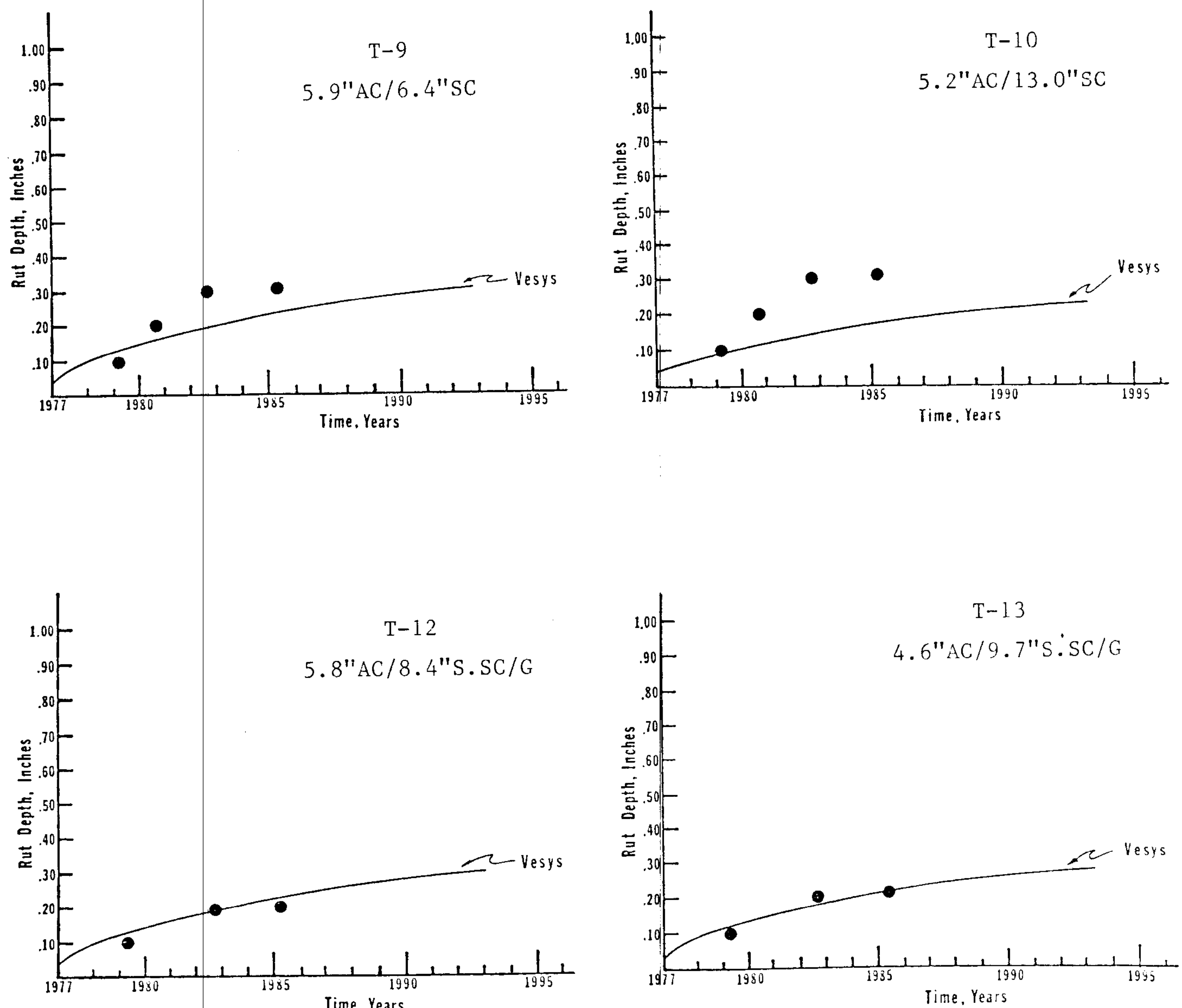


FIGURE 15

Measured vs. Predicted Rutting
Cement Stabilized Base
(5-year Designs)

The program correctly predicted less rutting for the four control sections (which contain a cement stabilized soil layer between the asphaltic concrete pavement and the embankment) than for normal full, depth pavements of similar thickness (T-1, T-3). The rigid subbase layer apparently served to minimize rutting, even though located 13 to 15 inches below the asphaltic concrete surface.

Overall, the VESYS IIIA rutting model appears to be a usable predictor based on the mix properties used in this study; however, the contribution of stripping in the upper pavement layers to total rutting is unknown. Another factor to be considered is that actual truck tire pressures could have been greater than the 70 to 90 psi values input to the program, which would increase the magnitude of predicted values.

Performance--Pavement Ride, Serviceability

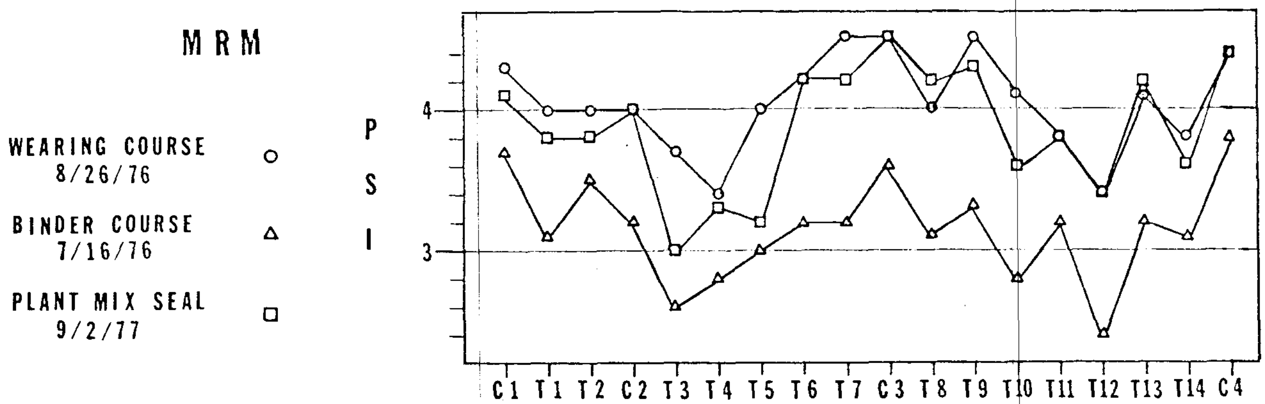
Serviceability Index (S.I.) values were determined using the Mays Ride Meter for the binder course, wearing course, and plant mix seal (friction course) on each test section, as indicated in Figure 16. Test section smoothness improved by approximately 0.5 S.I. upon addition of the wearing course. Even so, the variation in S.I. between the roughest and smoothest sections was 3.3 to 4.5. The addition of a 3/4-inch thick plant mix seal provided no improvement in rideability.

A comparison of S.I. (MRM) and S.I. (CHLOE) is also presented in Figure 16 for roughness tests conducted on the wearing course. The S.I. (MRM), which is based on a correlation with a General Motors Surface Dynamics Profilometer, indicates a slightly smoother ride than the Chloe Profilometer, although the two devices appear to track each other across the range of ride levels built in the eighteen pavement sections. A regression equation which relates the two S.I. data sets follows:

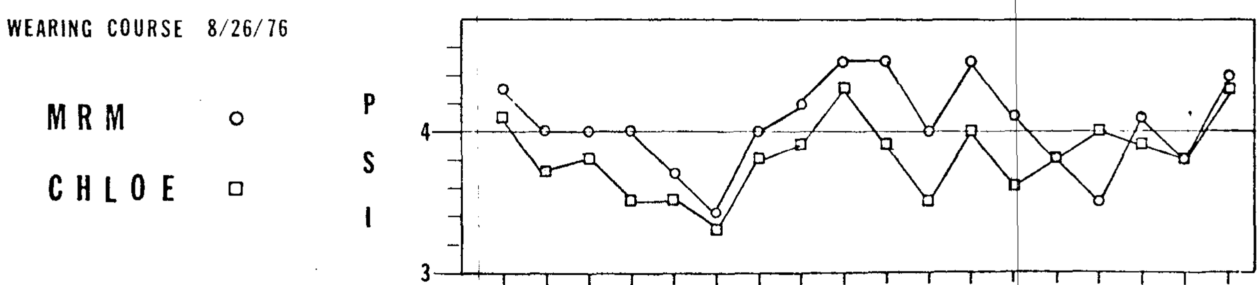
$$\text{CHLOE} = 1.142 + 0.653\ \text{Mays}$$

Within six years of placement, the plant mix seal lost bond to the wearing course and began to peel off, primarily in the wheelpaths. The effect on pavement ride was so negative that by 1983 all field measurements of serviceability decline were discontinued.

Serviceability index values, which were measured with the Mays Ride Meter and then converted to Chloe S.I., are compared to predicted S.I. values in Figures 17-20 using the VESYS IIIA program and the AASHTO equation for flexible pavement design. (4) The VESYS IIIA program predicted S.I. decline most accurately on cement stabilized base sections where block cracking deterioration did not accelerate surface roughness.



Present Serviceability Indices Chart for the Binder, Wearing and Plant Mix Seal Courses on Each Section



A Comparison of Chloe P.S.I. and Mays Ride Meter P.S.I. on the Wearing Course Layer

FIGURE 16

Serviceability Index - Mays Meter, Chloe (As Constructed)

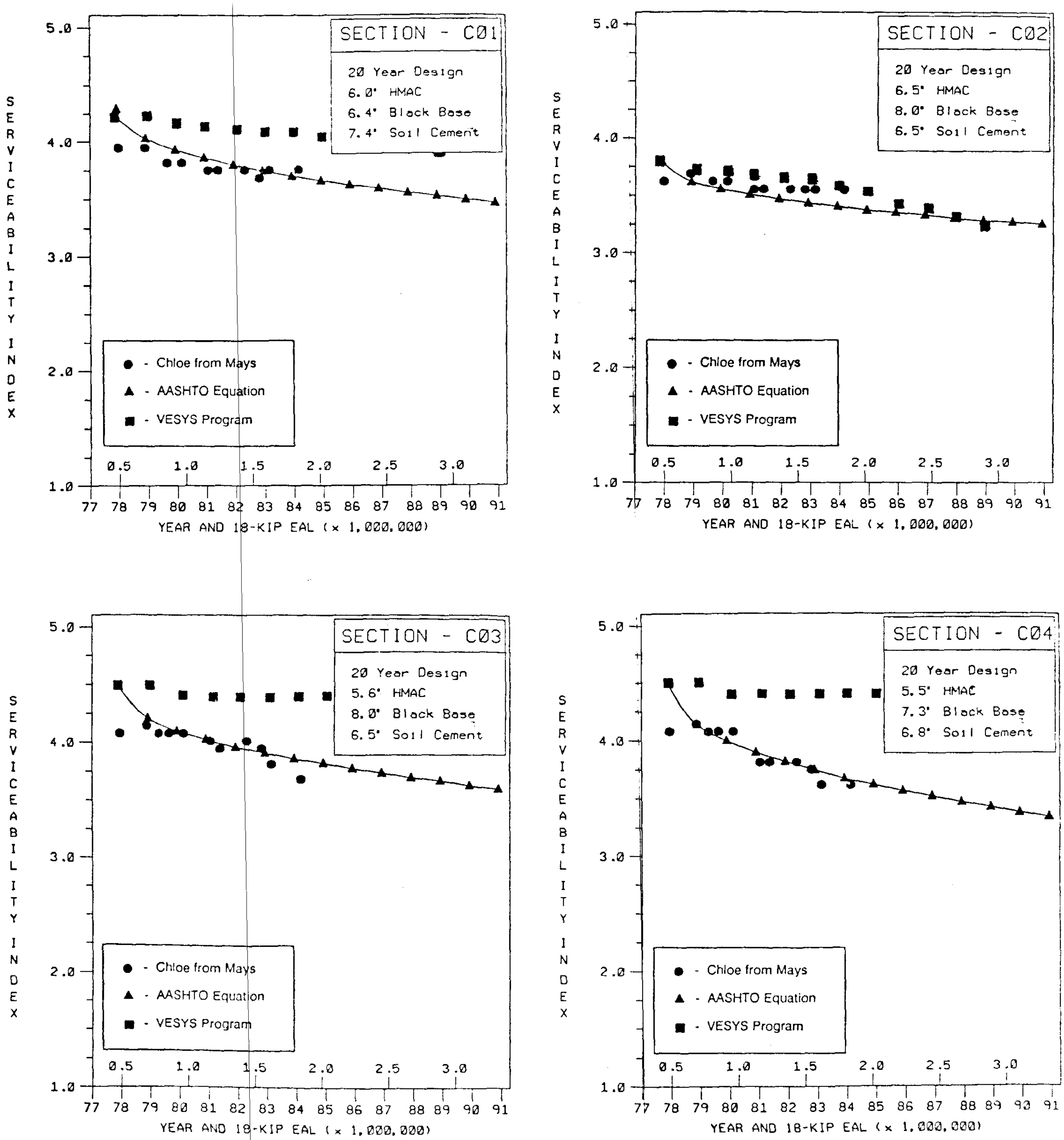


FIGURE 17

Measured vs Predicted Serviceability Index - Control Sections

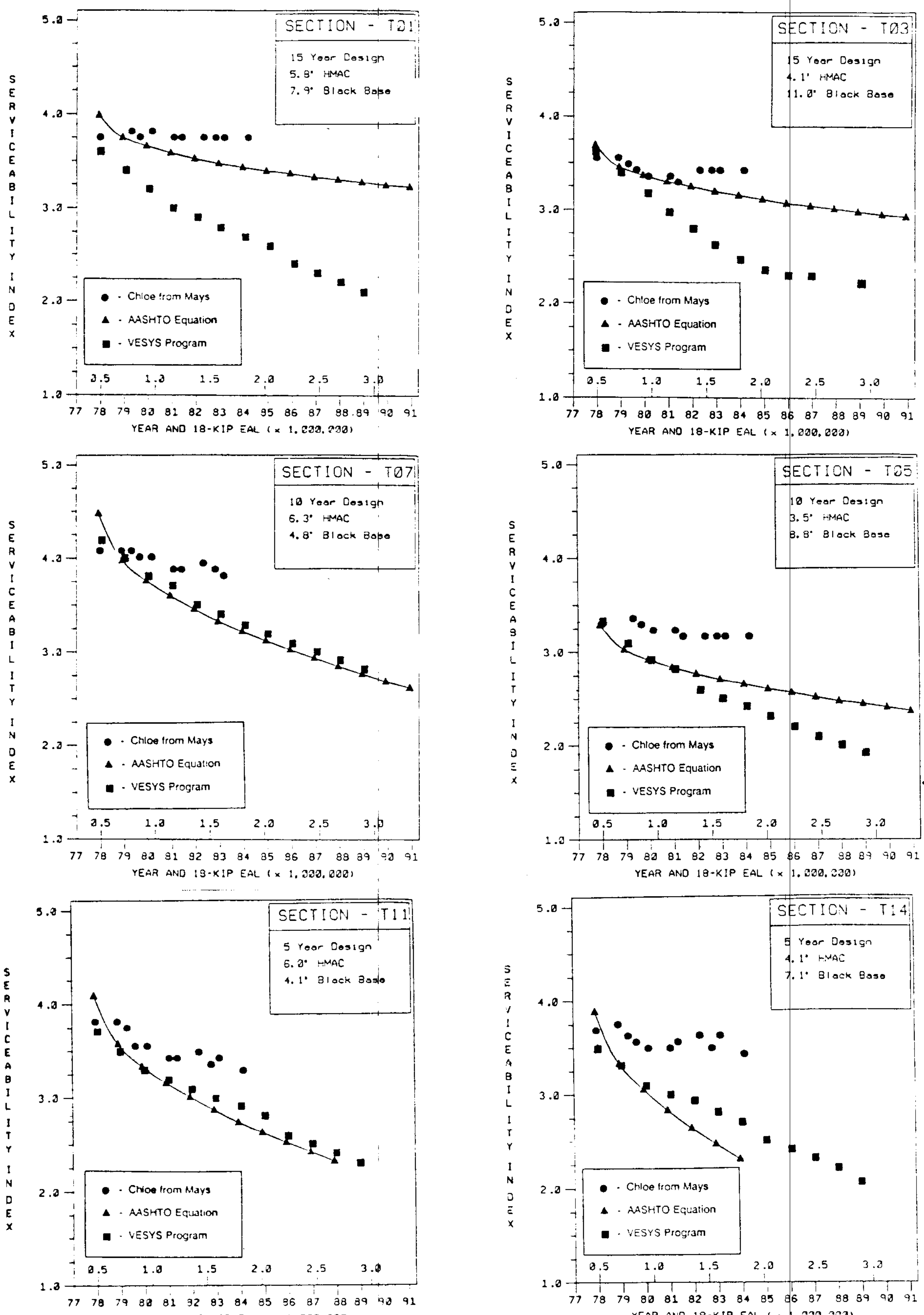


FIGURE 18

Measured vs Predicted Serviceability Index - Full-Depth Asphaltic Concrete

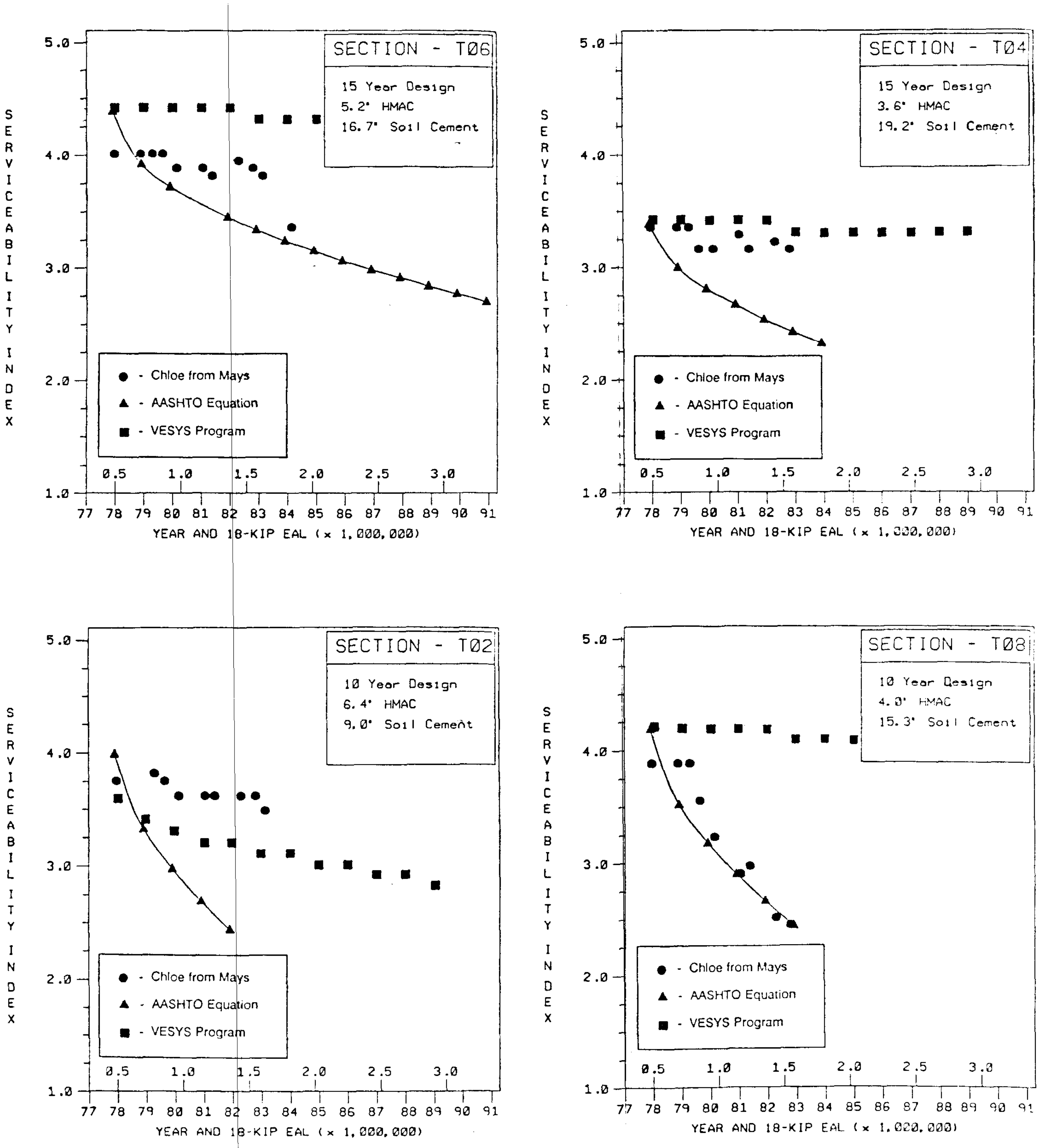


FIGURE 19

Measured vs Predicted Serviceability Index
Cement Stabilized Base (10-and 15-year Designs)

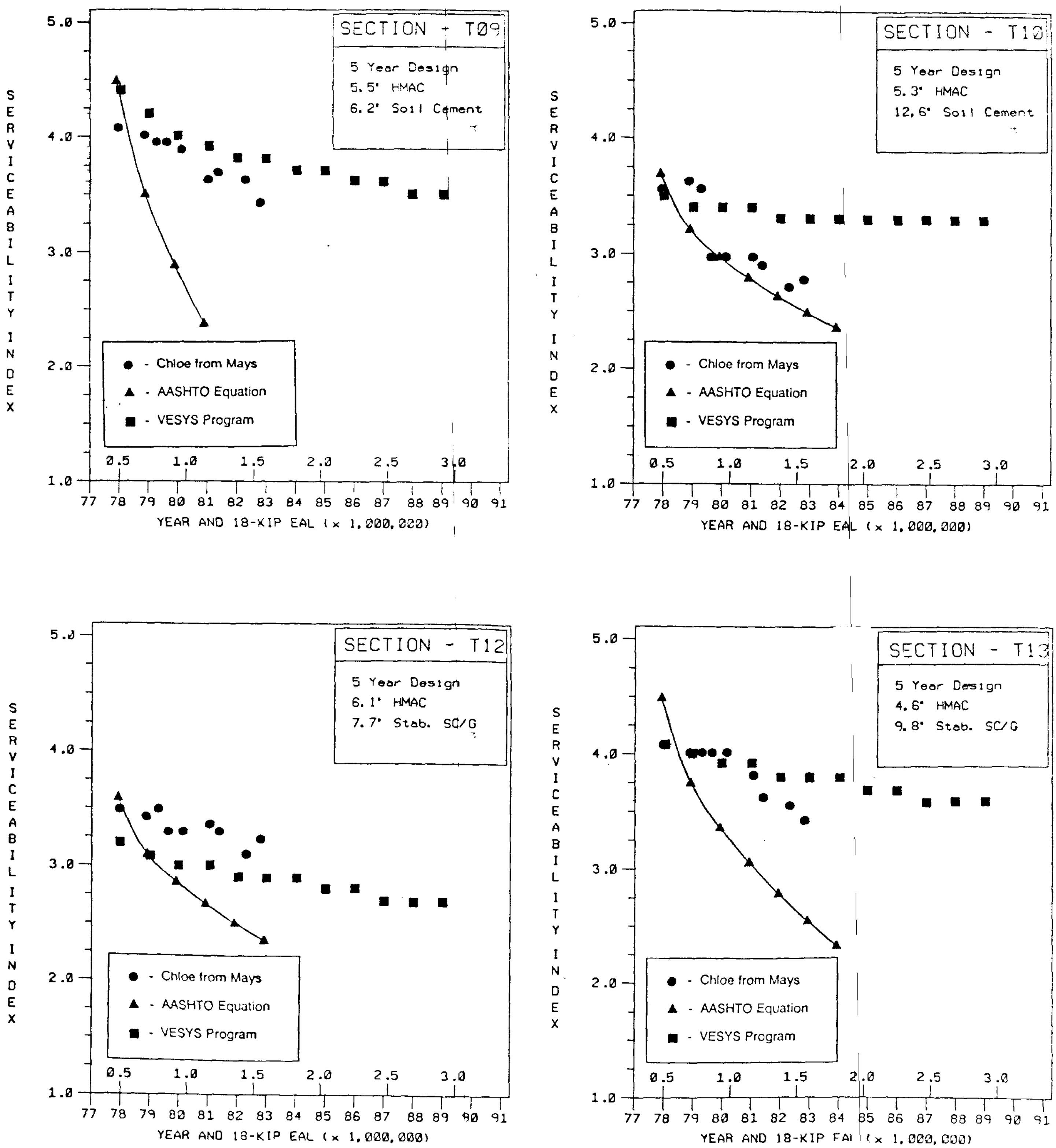


FIGURE 20

Measured vs Predicted Serviceability Index - Cement Stabilized Base (5-year Designs)

It should be noted that the S.I. decline curves account for only six years of performance data spanning approximately 1.7×10^6 equivalent axle loads. During the measurement period, only two sections reached the terminal serviceability level of 2.5; therefore, the comparisons do not indicate the shape of a completed S.I. decay curve.

The AASHTO flexible design equation was found to be conservative and was generally a better predictor of S.I. decline on full-depth asphaltic concrete sections than the VESYS IIIA program. In most pavements, measured serviceability decline is very flat during the initial years of loading, producing a generally convex curve. The AASHTO model is generally concave in shape and therefore may not properly depict the accelerated S.I. decline which pavements experience in their final years of service.

STRUCTURAL ANALYSIS

AASHTO Structural Number - Dynaflect Deflections

Deflection tests were conducted with the Dynaflect device on each pavement layer, beginning with the completed clay embankment. The primary objective of these tests was to characterize the as-constructed strength of each layer. A comparison of as-built strengths and design strengths was facilitated by translating measured Dynaflect deflections into structural numbers, SN, as defined in the AASHTO Flexible Pavement Design system.

This method of deflection analysis was developed through a combination of two-layer linear elastic theory and AASHO-Louisiana flexible pavement design theory. Layered theory provided the ability to individually characterize the strengths of the embankment layer (E_S) and the pavement layer (E_1). A Louisiana Department of Transportation and Development research study (4) provided an E_1-SN relationship in a pavement evaluation chart, which was used to evaluate the strength contribution of each layer upon construction.

An example of the use of the pavement evaluation chart to calculate the strength contributions of individual pavement layers is presented in Figure 21. In this example, as each new pavement layer is added, the total system deflection decreases and the spreadability increases, resulting in an increase in load carring ability, expressed as SN.

The strength contribution of the embankment (E_S) in Figure 21 also increased as additional pavement layers were constructed. This increase is thought to be principally due to a moisture loss in the subgrade soil as deflection evaluations progressed from the wet to the dry season. The confining effect of each added pavement layer on the subgrade may also have contributed to the gain in subgrade strength indicated by the deflections.

AS CONSTRUCTED DEFLECTION TEST DATA-- TEST SECTION # 10 (STA. 230+75)

	PAVEMENT LAYER	LAYER THICKNESS	DATE OF TEST	AGE AT TEST
1	Embankment		9-22-75	1 day
2	Selected Soil	8.8"	10-7-75	5 days
3	Cement Stabilized Soil	12.3"	4-15-76	9 days
4	Cement Stabilized Soil	12.3"	4-26-76	20 days
5	HMAC Binder Course	2.9"	6-15-76	5 days
6	HMAC Wearing Course	1.9"	7-15-76	16 days

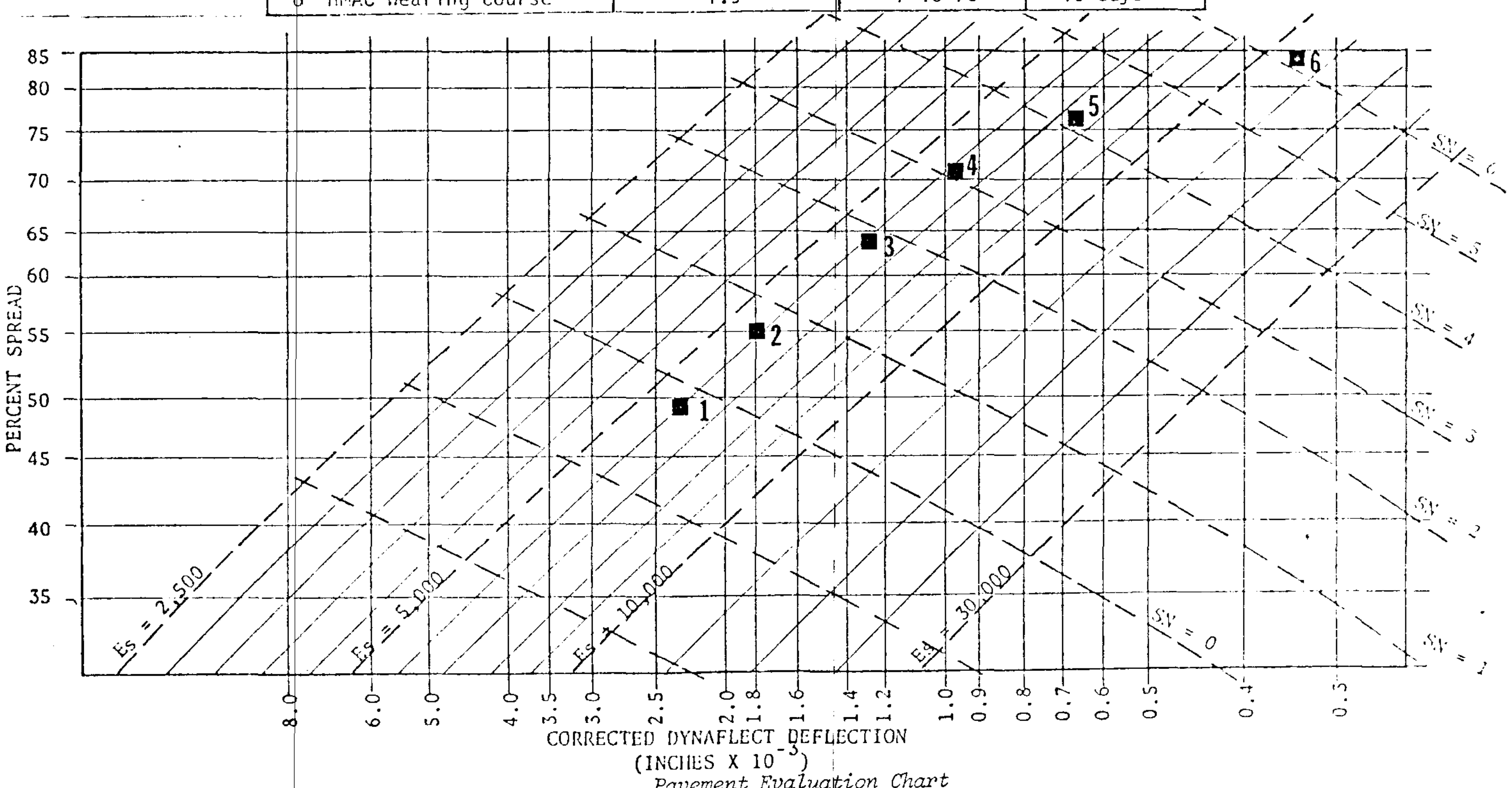


FIGURE 21

Pavement Evaluation Chart
Structural Number, Subgrade Modulus

It is interesting to note the sensitivity of the pavement evaluation chart of Figure 21 in indicating the SN-related strength gain with time of the 12.3-inch soil-cement base course.

Figures 22 - 25 depict the variation in field measured structural numbers and their decrease with time and accumulated traffic load. The theoretical decline in structural number with load, which was calculated by iteration using the AASHTO equation, is also indicated for comparison. It should be noted in the general form of the AASHTO formula:

$$\frac{4.2 - Pt}{4.2 - 1.5} = \left[0.40 + \frac{1094}{SN+1} \right] \left[\log Wt - \left[9.36 \log (SN+1) - 0.20 \right] \right]$$

where Pt = serviceability at end of time t

SN = structural number

Wt = axle load applications at end of time t

that SN is an expression of initial load carrying capability which remains fixed as Wt increases and Pt decreases to some terminal value.

The point of origin of the theoretical SN decline curve in Figures 22 - 25 is the SN value calculated using actual layer thicknesses and the following AASHTO strength coefficients: asphaltic concrete surface = 0.44, asphaltic concrete base = 0.34, cement stabilized soil = 0.15, cement stabilized sand-clay-gravel = 0.18.

To facilitate a model for project-specific SN decline with corresponding Pt decline, the first term in the equation, which represents the rate of change in serviceability has been rearranged as follows:

$$\frac{Po - Pt}{Po - 1.5}$$

where Po = initial serviceability at time zero.

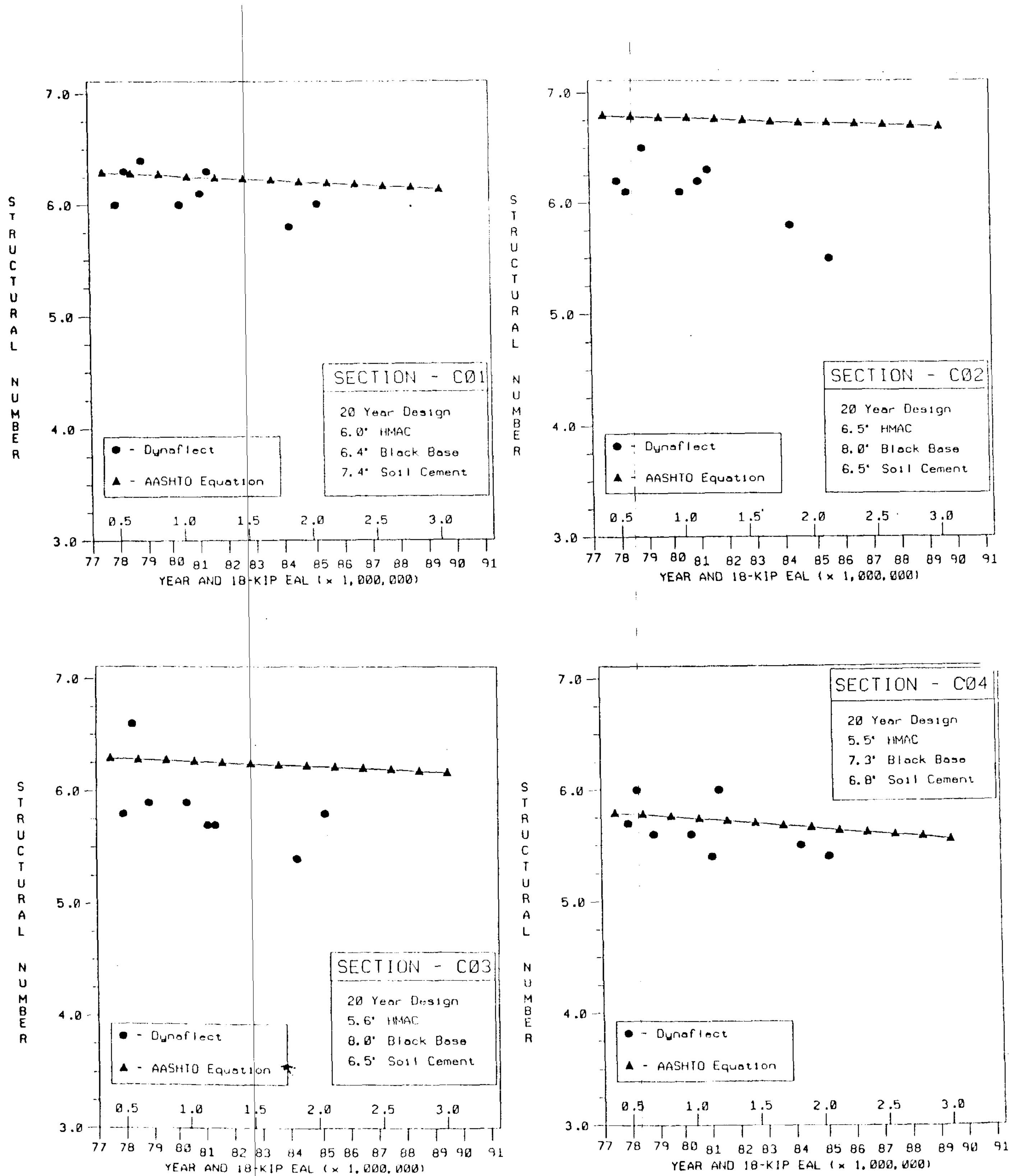


FIGURE 22

Measured vs Predicted Structural Number
Control Sections

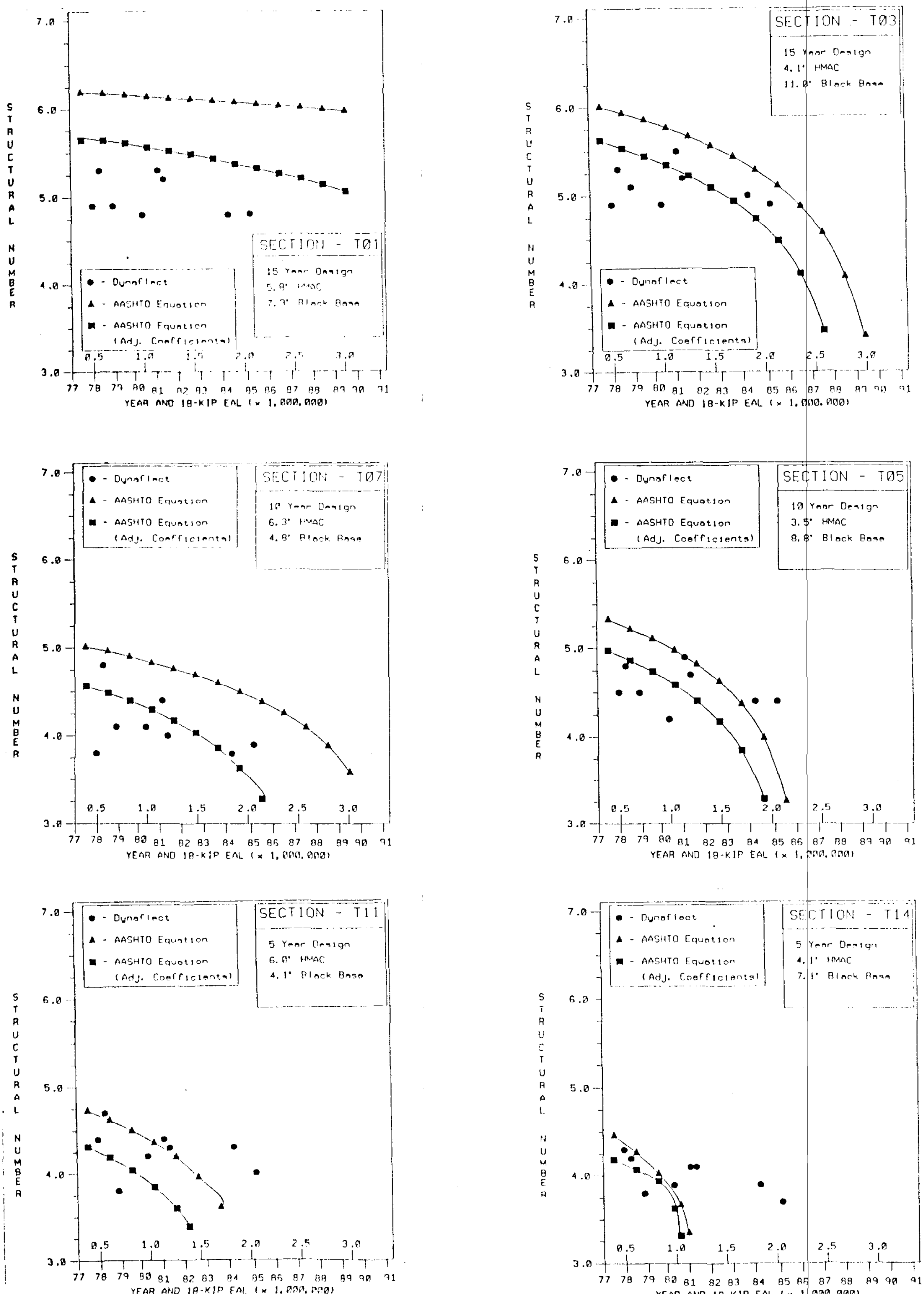


FIGURE 23
Measured vs Predicted Structural Number -
Full-Depth Asphaltic concrete

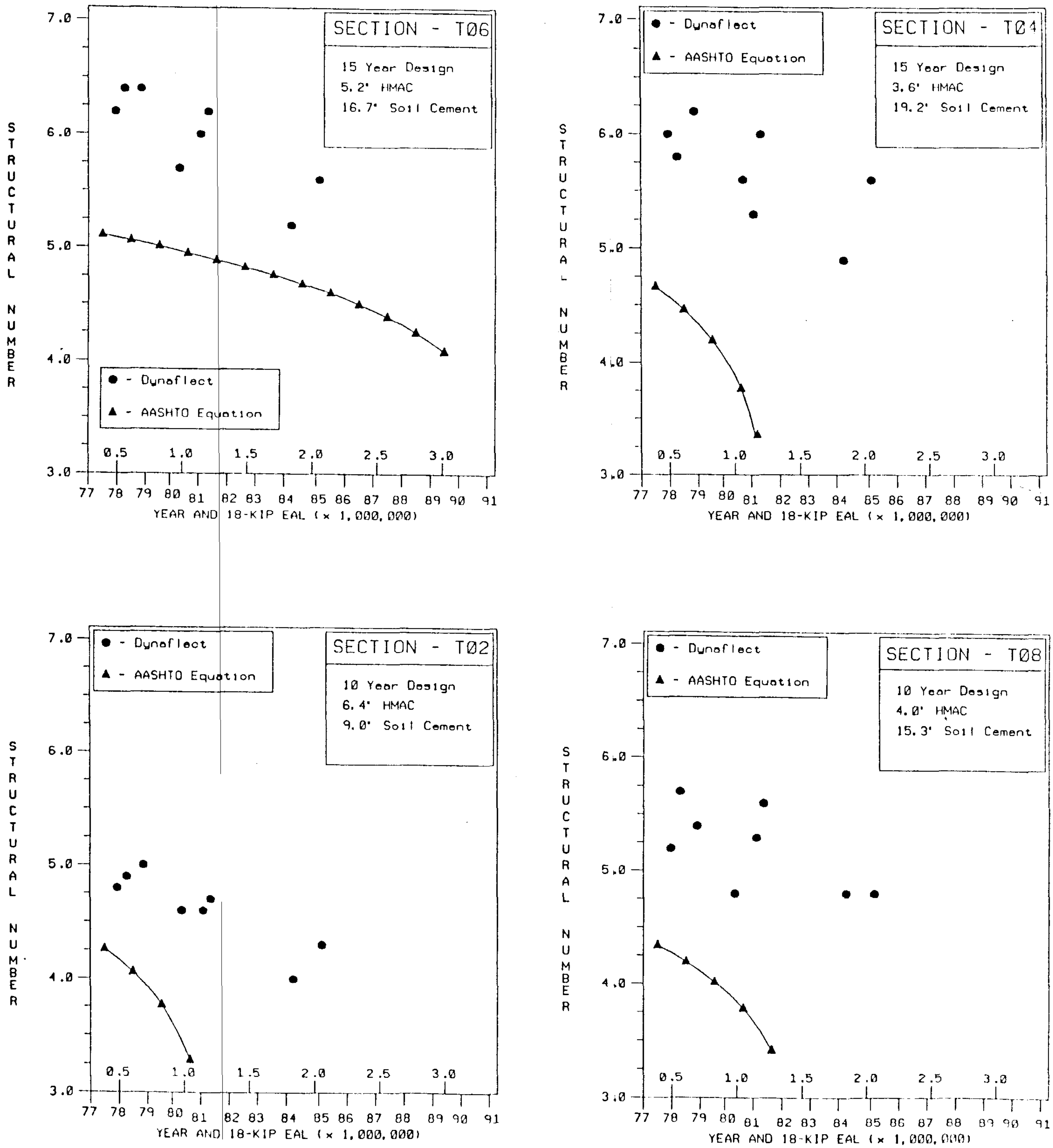


FIGURE 24

Measured vs Predicted Structural Number - Cement Stabilized Base (10-and 15-year Designs)

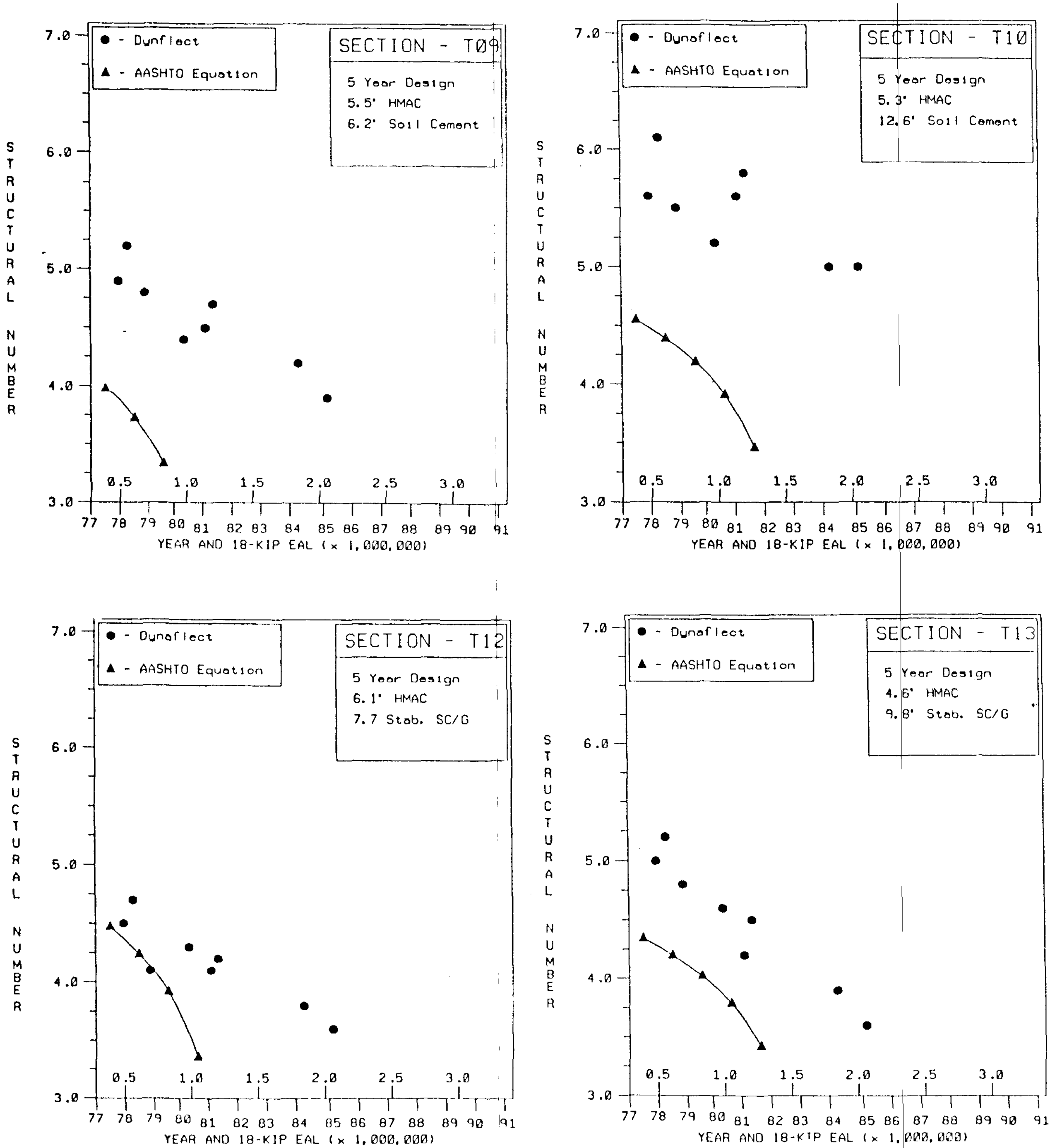


FIGURE 25

Measured vs Predicted Structural Number - Cement Stabilized Base (5-year Designs)

Using an iterative procedure, successively smaller SNs are then calculated as serviceability decreases and load increases. The procedure provides a mechanism for modeling pavement systems which weaken under accumulated truck loading and as a result become rougher.

Field measured SN values for test sections containing cement stabilized bases are typically larger than predicted values due to the deflection-reducing effect of these very stiff layers. The apparently large SN values were not indicative of the performance of these test sections, however, since cracking and surface distortion caused rapid serviceability declines under repeated heavy loads. Deflections measured at cracks are usually higher, producing lower structural numbers; therefore, sampling location becomes critical when attempting to characterize the strength of this type of pavement.

The SN decay curves for the full-depth asphalt test sections contain two theoretical curves, one representing the previously listed AASHTO strength coefficients and the other representing the coefficient values currently used by Louisiana: asphaltic concrete wearing = 0.40, asphaltic concrete binder = 0.36, asphaltic concrete base = 0.33. The design values, which were adjusted downward in 1980 (2) to better reflect field Marshall properties, apparently also better agree with deflection-based strength measurements.

Figure 26 provides a comparison of Dynaflect-measured versus calculated SN values plotted against maximum tensile strain at the bottom of the asphalt base course for each of the full-depth asphalt test sections. Strain values were calculated using the Bisar program (5) at an asphalt mat temperature of 77°F. The field-measured, deflection-based values provide a better fit with load response indicators than SN values computed from coefficient constants.

A similar relationship for test sections with cement treated bases was developed by plotting SN versus maximum compressive strains at the surface of the embankment, as indicated in Figure 27. The relationship between section strength indicators and calculated pavement response is apparently less predictable for cement treated bases than for asphaltic concrete bases.

Results of the resilient indirect tensile test indicate that flawed soil cement samples failed after 1 to 500 cycles, whereas unflawed samples obtained only several feet away withstood 300,000 to 1,000,000 cycles of loading. Extreme strength variations of this nature undoubtedly affect strength characterizations made from surface deflection testings as well as from laboratory testing.

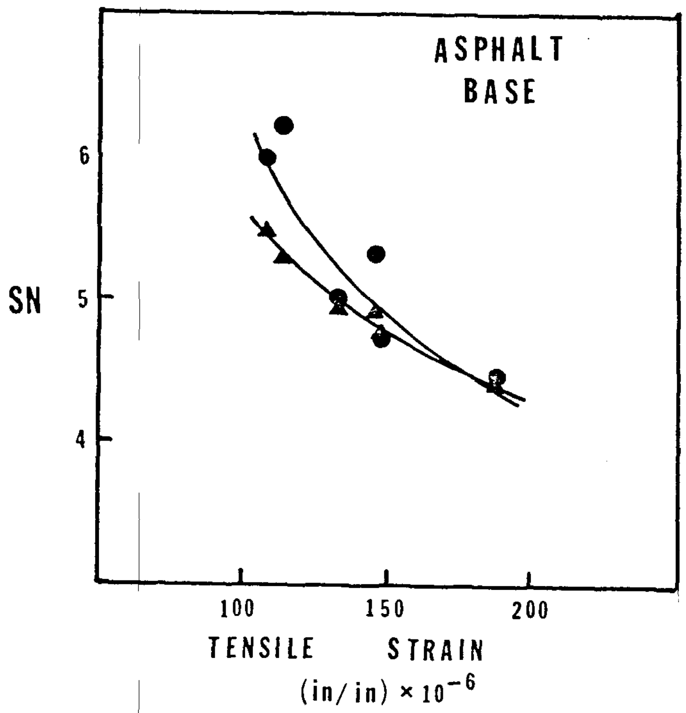


FIGURE 26

SN vs. Maximum Tensile Strain
Full Depth Asphaltic Concrete Sections

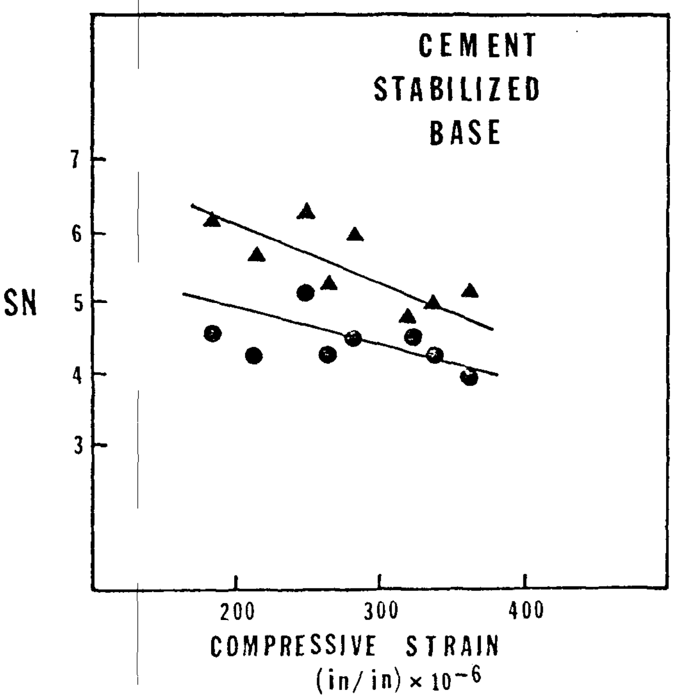


FIGURE 27

SN vs. Maximum Compressive Strain
(Embankment)
Cement Stabilized Base Sections

Layer Moduli - Non Destructive Field Tests

Determination of in situ moduli was accomplished by inputing Dynaflect deflections into a computer program entitled FPEDD1 (a flexible pavement structural evaluation system based on dynamic deflections) which was developed by Uddin and others.(6) The program is designed to analyze deflection basins to predict layer moduli of up to four layers. The in situ moduli are then corrected by equivalent linear analysis to account for nonlinearity of granular layers and cohesive subgrades, in addition to a temperature correction for the modulus of asphaltic concrete.

Three points in time were selected to examine magnitude and change in layer moduli with load: 1978--0.5 x 10^6 EAL, 1980--0.8 x 10^6 EAL, and 1984--1.5 x 10^6 EAL. Figure 28 depicts the average trends in moduli change with time for the wearing-binder course, asphaltic concrete base, cement stabilized soil base, select soil, and embankment layers. Moduli of asphaltic concrete layers have been corrected to 77°F (25°C). Table 16 of Appendix D provides a listing of layer moduli for the three dates by section number.

As depicted in Figure 28, the Dynaflect deflection FPEDD1 analysis was successful in indicating the pavement layers which rapidly weakened under load. The wearing-binder course layer (where cores indicated that fatigue cracking began) experienced the most dramatic decline in layer modulus. The base course materials did not experience a rapid decrease in stiffness and the select and embankment soil layers remained virtually unchanged over 1.5 x 10^6 equivalent axle loads.

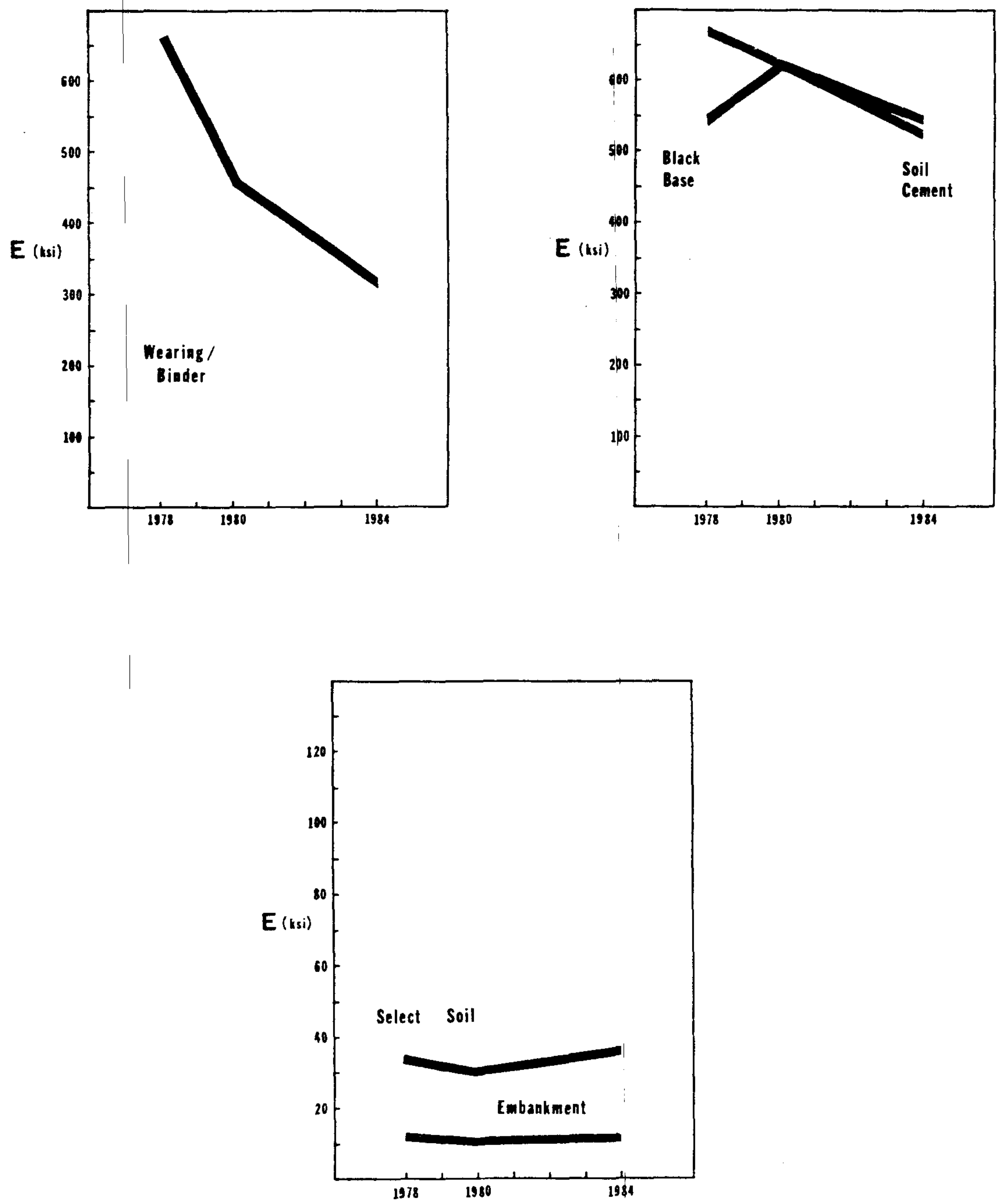


FIGURE 28

Field-Measured Layer Moduli
Dynaflect Device

Comparison of Laboratory and Field Measured Moduli Values

A comparison of elastic modulus values measured in the laboratory to modulus values measured in the field using the Dynaflect is presented in Table 9. The two sources of moduli determination are in general agreement, indicating that with the appropriate analysis techniques the Dynaflect device can be used to estimate in situ moduli of up to four pavement layers. A recent comparison (7) of Dynaflect and Falling Weight Deflectometer data used to predict layer moduli has indicated that both devices are capable of producing moduli suitable for design purposes.

Another method of estimating in situ embankment modulus from Dynaflect deflections is illustrated in Figure 29. The previously described pavement evaluation chart does not require the use of a computer and may be used to estimate embankment strength (as well as in-place pavement structural number) in the field. The embankment modulus values for five field evaluations from 1978 through 1985 are listed in Table 10.

TABLE 9

FIELD MEASURED VS LABORATORY MEASURED
Modulus of Elasticity, E X 10^5 PSI

Layer	Dynaflect (FPEDD1)	Indirect Tensile Test* E	Indirect Tensile Test* E corrected**
Wearing	6.5	5.7	6.8
Binder	6.5	5.4	6.5
Base	5.7	5.1	6.1
Soil Cement	6.4	4.5	5.4
Stab SC/G	5.8	3.5	4.0
Select	0.37	0.20	-
Embankment	0.13	0.11	-

* Field Cores
** Algorithms

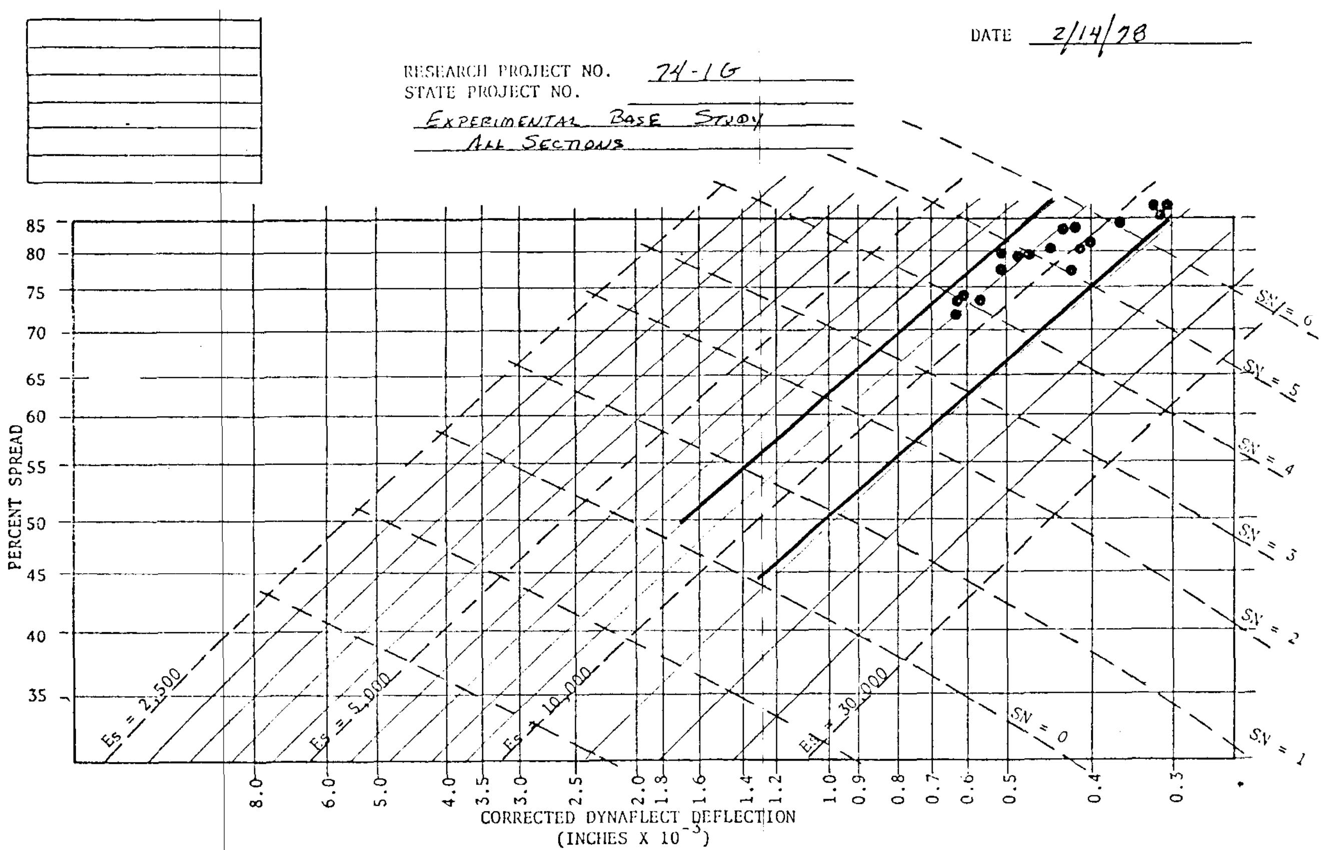


FIGURE 29

Embankment Modulus from Pavement Evaluation Chart

Prior to construction of the embankment, the northern end of the 2-mile experimental base project (Sections T-9 through C-4) was higher in elevation than the southern end, where various quantities of fill material were required. Soil borings taken on the northern end of the project indicated A-7-6 clays, which were the native soils, while the middle to southern end contained some silty clays and silty clay loams, which covered the deeper layers of clay soil. Results from the pavement evaluation chart, Table 10, reflect generally higher and less uniform Mr values in the filled sections, indicating slightly better support than afforded by the A-7-6 soils.

TABLE 10

DYNAFLECT-DETERMINED EMBANKMENT RESILIENT MODULUS
(Mr X 10^3 PSI)

	11/2/78	3/24/80	2/17/83	2/2/84	2/21/85
C-1	13	9	10	8	9
T-1	9	7	7	6	7
T-2	12	8	8	7	8
C-2	14	10	11	8	12
T-3	9	7	8	6	7
T-4	15	11	11	8	10
T-5	9	7	7	5	6
T-6	13	9	11	7	9
T-7	9	8	8	6	7
C-3	11	9	10	7	9
T-8	20	9	10	8	8
T-9	9	7	7	6	7
T-10	9	9	10	8	8
T-11	9	8	8	7	8
T-12	9	8	8	7	8
T-13	9	8	9	8	9
T-14	9	8	8	7	7
C-4	10	9	10	9	9

AASHTO DESIGN APPLICATIONS

Soil Resilient Moduli and R-Value

The material property used to characterize roadbed soil for pavement design in the new AASHTO design guide is the resilient modulus (Mr). Where equipment for performing the resilient modulus test is not available, conversions for R-value and other soils tests are provided.

The resilient moduli of select and embankment soils were measured across a range of moisture contents, as indicated in Figure 30. Moduli are shown to increase quickly as soil moisture decreases for all soil types, from sand to heavy clays. The silts, silty clays, and heavy clays appear to provide similar potential maximum (15,000 psi) and minimum (5,000) resilient moduli values, depending on in situ moisture. Sands developed a greater potential for support, with sandy loams and sandy clay loam soils indicating the highest potential (30,000 to 40,000 psi) at lower moisture levels.

Soil R-value test results for each soil type are paired with the resilient moduli selected from Figure 30 at the optimum moisture content of each of the following soil types:

		O.M.C.	Mr	R-Value
Sand	A-2-4(0)	11.4	19	60
Sa/Lo	A-2-6(0)	11.7	16	21
Sa/Cl/Lo	A-2-6(2)	13.6	14	21
Silt	A-4(8)	16.4	10	25
Si/Cl	A-6(12)	18.3	8	18
Hv/Cl	A-7-6(20)	28.5	5	<5

The resulting Mr - R-value relationship is indicted in Figure 31 along with correlations from other studies[4,8,9]

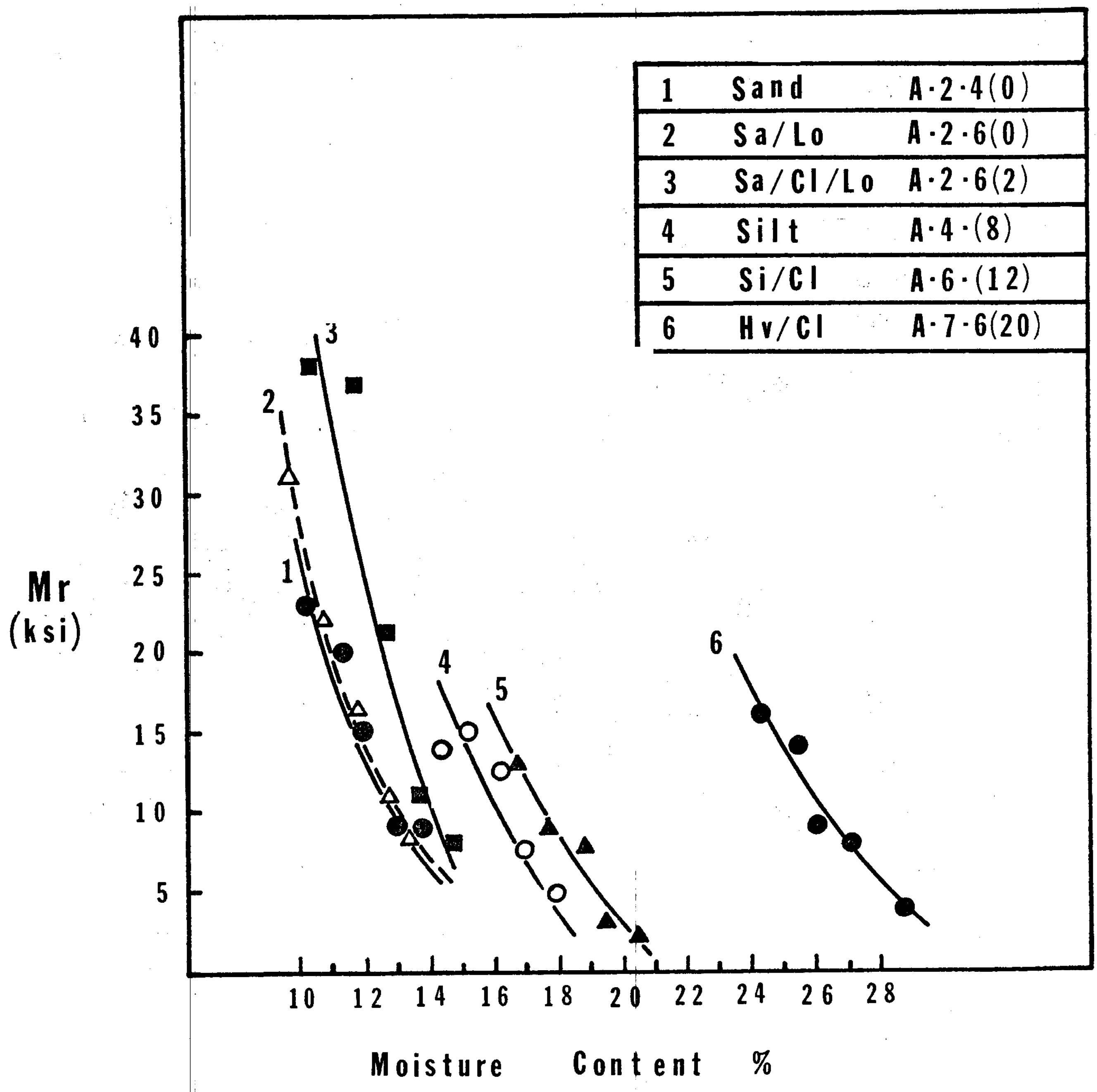


FIGURE 30

Soil Resilient Moduli vs. Moisture Content

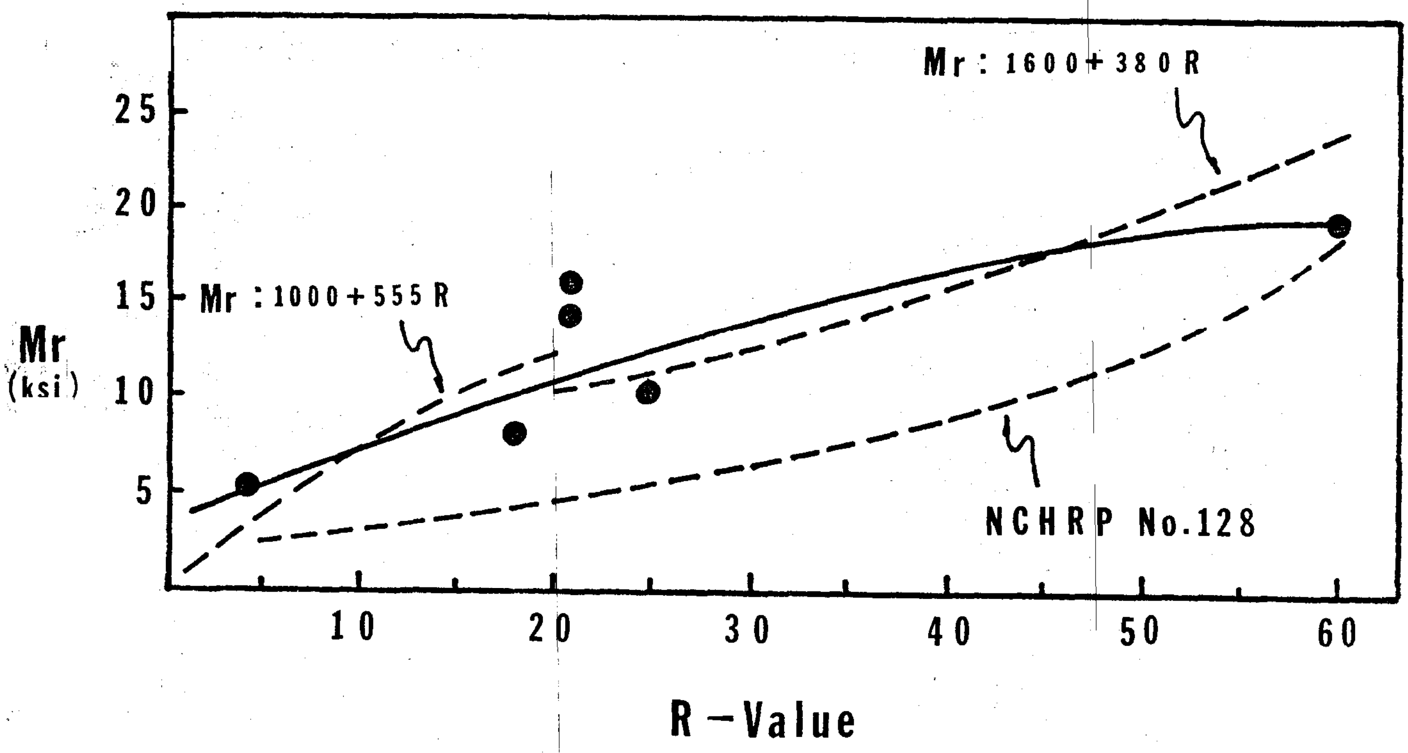


FIGURE 31

Soil Resilient Modulus <u>*vs*</u>*. R-value*

The test data from the Louisiana Experimental Base Study is in general agreement with the theoretical relationship, as follows:

For R-value between 0 and 20	$Mr = 1000 + 555R$
R-value greater than 20	$Mr = 1600 + 380R$

R-values associated with the select soil layer were approximately 20 to 25. Using the relationship between R-value and structural layer coefficient in the 1986 AASHTO Design guide, a value of approximately 0.04 is indicated. This is the value currently used by Louisiana in flexible pavement design.

Actual insitu moisture contents of embankment soils were measured using nuclear moisture-density sensors lowered in one-foot increments down to eight feet below the embankment surface. Soil mositure content,Figure 32, generally increased with depth beyond the four-foot level and typically ranged from 26 to 32 percent. Applying the 26 percent moisture value (representing the top four feet of embankment) to the clay soil Mr curve in Figure 30, a modulus value of 11,000 psi results as previously indicated in Table 9.

Design input values for embankment resilient moduli (from lab tests) were successfully estimated in this study using two independent methods: (1) laboratory R-values with appropriate correlations and (2) Dynaflect deflection data, in conjunction with either a computer program which estimates layer moduli using basin fitting techniques or a nomograph used to estimate embankment moduli in the field.

Layer Strength Coefficients for Asphaltic Concrete and Cement Stabilized Methods

Design strength coefficients computed using typical fundamental materials' properties from this study and currently specified properties representing materials in this study are listed in Table 11. Corresponding design values currently in use by LADOTD are also indicated.

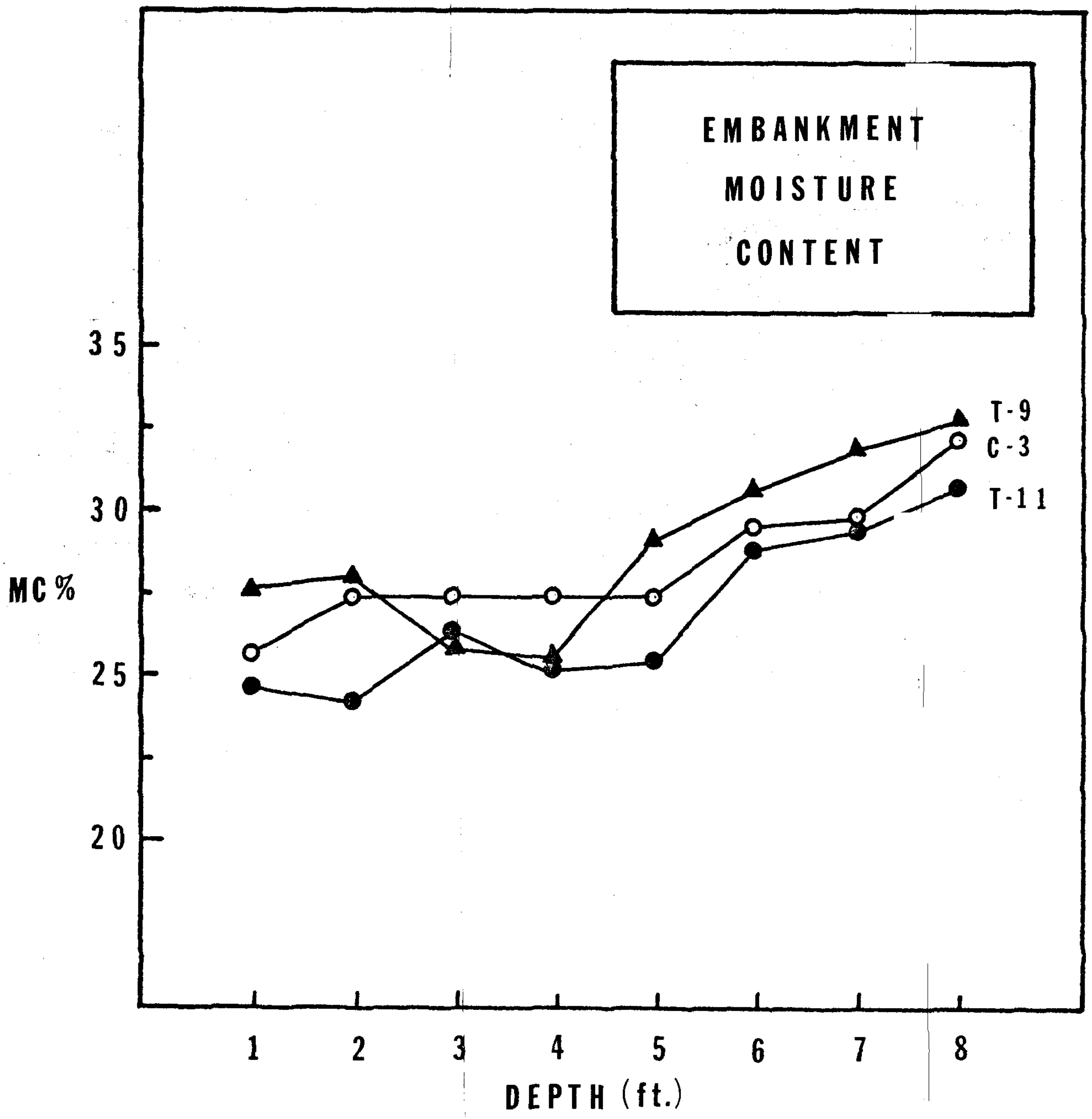


FIGURE 32

Moisture Content of Embankment With Depth

TABLE 11

DESIGN LAYER COEFFICIENTS

AASHTO FLEXIBLE PAVEMENT DESIGN

Er (Indirect Tensile) ($psi \times 10^5$)		$\bar{C}$*	Marshall Stability	$\bar{C}$*
Wearing mix	4.8	0.44	1800	0.42
Binder mix	4.6	0.44	1500	0.37
Base mix	4.0	0.34	1400	0.28
			Currently in Use	
Wearing mix			1700	0.40
Binder mix			1400	0.36
Base mix			800	0.33
			Compressive Strength (psi)	C
Soil cement	5.4	0.14	633	0.19
Stabilized sand clay gravel	4.0	0.08	356	0.14
			Currently in Use	
Soil cement			300+	0.15
Stabilized sand clay gravel			500+	0.18

*Based on relationships in the AASHTO Pavement Design Guide

Asphaltic concrete materials were somewhat stiffer and developed correspondingly higher Marshall Stabilities than are currently specified for flexible pavements. No need for adjusting design coefficients for asphaltic concrete materials was indicated in this study.

The soil cement materials generally exceeded a 300+ psi design compressive strength: however, resilient moduli determined from indirect tensile testing indicate a structural coefficient of 0.14, very near the 0.15 currently used. The cement stabilized sand clay gravel, on the other hand, developed far less compressive strength than designed for and would receive a design coefficient of only 0.08 based on fundamental properties. Field cores were difficult to obtain for both base types due to cracking and laminations between compacted layers. Generally, performance of the cement stabilized base was associated with these failure planes rather than with the magnitude of compressive strengths or layer moduli. For this reason, the use of higher design coefficients is not advised for this type of base, even where modulus values are higher due to layer stiffness.

RESEARCH OBJECTIVES ACCOMPLISHED

1. Fundamental engineering properties of each paving material were measured and compared using a variety of (tensile and compressive) laboratory tests all utilizing repeated loading applied to both laboratory-molded samples and field cores. The engineering properties of each material were also determined from field deflection tests to provide a comparison to laboratory measured values.

2. The Mays Ride Meter and Chloe profilometer were correlated to facilitate measurement of serviceability indices. The Dynaflect and Benkelman beam devices were correlated to enhance deflection-based field data.

3. Field-measured serviceability and pavement strength indices were compared to the theoretical relationships from AASHTO Design theory. An iterative procedure was utilized to compute theoretical change in structural number (SN) with load to provide a comparison to SN decline measured with the Dynaflect device.

4. Measured values of serviceability decline, cracking and rutting were compared to predicted values using the VESYS IIIA program.

5. The magnitudes of vehicle load equivalency factors used in Louisiana for flexible pavement design were compared to factors derived from Weigh-In-Motion data. Actual and estimated total 18-kip equivalent axle load data were also compared.

6. Structural layer coefficients for design were examined in light of the standard specified materials' properties achieved and by using measured fundamental engineering properties. A common level of performance (such as terminal serviceability) was not attained due to the rapidly deteriorating surface friction course; therefore, layer equivalency factors could not be verified on the basis of

this type of information.

7. Changes in layer moduli with time were measured using surface deflections to provide insight into which layers experience the greatest degree of weakening under load and environment.

8. Observations were made concerning:
   1) The effects of stripping of asphaltic concrete on performance and expected life, 2) the benefits of providing a stiff, cement stabilized layer between an embankment and full-depth asphaltic concrete pavement to minimize rutting, 3) optimum thickness ratios of asphaltic concrete surfacing to cement stabilized base to minimize reflective shrinkage cracking, and 4) strength variations attributed to in-place cement stabilization.

9. Resilient moduli and R-values for a wide range of soil types were correlated and compared to theoretical relationships found in the 1986 "AASHTO GUIDE FOR DESIGN OF PAVEMENT STRUCTURES." Tubes placed ten feet into the embankment provided a means of measuring embankment moisture variation with depth and by season. In situ moisture values were used to indicate appropriate resilient moduli for design applications from lab test results.

10. Reported separately is a series of six reports by Hadley(1) describing a study conducted by the Materials Research Laboratory, Louisiana Tech University, which is the phase of the Louisiana Experimental Base Study dealing with materials testing and data applications. The reports describe a very comprehensive, laboratory-based materials characterization study, with analysis of variance, applications of test data to special finite element modeling (of the lab tests) and to several deterministic design and analysis programs (summarized in Appendix E).

SUMMARY OF FINDINGS AND CONCLUSIONS

1. The vehicle load equivalency factors currently used in Louisiana-AASHTO flexible pavement design (W-4 Tables) compared closely with factors determined from Weigh-In-Motion data during this study. The currently used factor for 3-S-3's however, was found to be high by approximately 200 percent and the factor for 3-S-2's was found to be low by approximately 20 percent.

2. It was concluded from an evaluation of field cores that fatigue cracking occurring in the wheelpaths of full-depth asphaltic concrete test sections was confined to the wearing, binder course layers. Moisture susceptibility, also referred to as stripping, originating at the interface of these two layers was a contributor to the load-related cracking and rutting. The absence of antistrip additives and the use of a high void content mix contributed to the stripping problem.

3. Two of the full-depth asphaltic concrete sections which were five-year designs and which did not experience stripping out-performed thicker test sections which did contain stripping in terms of severity of fatigue cracking, patching, and rutting. The occurrence of stripping had a greater influence on performance than additional section thickness or higher structural numbers.

4. The cement stabilized base sections with the largest ratios of asphaltic concrete surfacing to base thickness developed less severe and less extensive reflective cracking across all design levels. Test sections where in-place stabilization was accomplished in one lift (6 to 9 inches) and where the thickness of surfacing was nearly equal to the base thickness generally performed best.

5. Performance of the cement stabilized base sections was associated with shrinkage cracking, failure planes, and laminations between

compacted layers rather than with the magnitude of strength parameters measured away from cracks.

6. The VESYS IIIA program was a good predictor of rutting and a fair predictor of serviceability decline, but a poor predictor of cracking. The program correctly predicted less rutting for full-depth asphaltic concrete sections which also contained a cement stabilized working table than for those with an unstabilized subbase. VESYS IIIA was generally a better predictor of serviceability decline than the AASHTO flexible design equation on cement stabilized base sections, except for those which developed the most reflective cracking (the program does not consider added roughness resulting from reflective cracking).

7. The AASHTO model for serviceability decline was found to be a better predictor for full-depth asphaltic concrete than VESYS IIIA. The AASHTO equation is initially conservative but because of its generally concave shape, the model may not properly predict the accelerated serviceability decline which pavements experience in their final years of service.

8. Louisiana's new design layer coefficient values for asphaltic concrete, adjusted in 1980, agree better with deflection-based strength parameters. The adjustment represented an attempt to better relate Marshall properties achieved in the field to design factors.

9. Using the data analysis techniques described in the report, Dynaflect data can be used to estimate AASHTO structural numbers (SN) which reflect in situ pavement and conditions. The field measured SN values for full-depth asphaltic concrete sections related better to pavement response indicators (stress and strain) than SN values calculated from

strength coefficient constants. Cement stabilized base sections were more difficult to characterize due to the ranges of stiffness measured when testing between cracks or at cracks.

10. Dynaflect data can be used to determine layer moduli reflecting in situ pavement conditions with the appropriate data analysis techniques. The technique used in this study correctly identified the layers experiencing stripping (wearing, binder layers) as having the most rapid rate of decline in layer modulus with time.

11. A comparison of laboratory determined moduli and Dynaflect deflection-based, field-measured layer moduli has indicated a close similarity in magnitude of determined values. Field-measured values using the Dynaflect are therefore suitable for input in pavement design.

12. Comparisons of theoretical and finite element analysis solutions indicate that laboratory-determined values of modulus of elasticity and tensile stresses and strains can be over or underestimated using the theoretical mathematical formulas commonly used to calculate these values.

13. Soils test data for the R-value and resilient modulus testing in this study closely track the suggested conversion relationships proposed in the 1986 AASHTO Design Guide and other cited references. The test data did not compare as well with a similar relationship found in NCHRP publication #128.

RECOMMENDATIONS

1. Weigh-In-Motion equipment should be utilized to verify loadometer results and to increase sample sizes for design input factors. Factors for 3-S-3's and 3-S-2's from sites other than the Experimental Base Study should be evaluated in light of the findings of this study.

2. All asphaltic concrete mixes should be examined for moisture susceptibility and appropriate action taken prior to paving when stripping potential is indicated.

3. Designs of flexible pavements placed over cement stabilized bases should provide asphaltic concrete surfacing equal to or of a greater thickness than the base where repeated heavy loads are anticipated.

4. Consideration should be given to improving the predictive capabilities of VESYS IIIA by enhancing the cracking prediction portion of the program.

5. Research is needed to utilize the Dynaflect device in evaluating typical layer moduli ranges of in-service pavements and embankments to develop appropriate input to the new AASHTO Design Guides.

6. It is recommended that R-value tests and field deflection tests be utilized to estimate soil resilient moduli for design input.

7. Layer design strength coefficients currently in use by the department should not be adjusted at this time. The as-built strength and performance of cement stabilized sand clay gravel bases should be studied further.

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APPENDIX A

TESTING AND MEASUREMENT PROGRAMS

Construction Sampling and Testing Program

Post - Construction Sampling and Testing Program

TABLE 12

CONSTRUCTION SAMPLING AND TESTING PROGRAM

Material and Test	Test Method	Sampling Time	Sampling Size	Sampling Frequency	Testing Time	Testing Agency
SUBGRADE SOIL						
Moisture Density	LDH TR 415-66	Before	15 lb.	Three Minimum/Section	Before	Department
R-value	Calif. 301-F	During	15 lb.	Three Minimum/Section	After	Department
Resilient Modulus, M_R	A1	During	300 lb.	One/Project	After	Asphalt Institute
In-place Density & Moist.	LDH TR 401-67	Not Applicable	Not Applicable	Three Minimum/Section	During	Department
Mechanical & Physical Anal.	LDH TR 407-69	Before	30 lb.	Two Minimum/Section	Before	Department
SOIL CEMENT BASE						
Mix Design	LDH TR 432-71	Before	80 lb.	As Required for Design	Before	Department
Modulus of Elasticity	Indirect Tensile Test	During			After	Department
Poissons Ratio	Indirect Tensile Test	During			After	Department
Fatigue Properties	Repeated Indirect Tensile Test	During			After	Department
Tensile Strength	Indirect Tensile Test	During			After	Department
In-place Density	LDH TR 401-67	Not Applicable	Not Applicable	3 Random Tests/Section	During	Department
Thickness	LDH TR 602-67	Not Applicable	Not Applicable	3 Random Tests/Section	During	Department
ASPHALT BASE & SURFACE						
Mix Design	LDH 303-64	Before	75 lb. agg. 2 qt. A.C. 1 gal. can mineral filler	As Required for Design	Before	Department
~~Modulus of Elasticity~~	~~Indirect Tensile Test~~	~~During~~	~~300 lb. each aggregate~~ 3 gal. asphalt	~~One/Project~~ One/Project	~~After~~	~~Department~~

TABLE 12 (CONTINUED)

CONSTRUCTION SAMPLING AND TESTING PROGRAM

Material and Test	Test Method	Sampling			Testing	
		Time	Size	Frequency	Time	Agency
Fatigue Properties	Repeated Indirect Tensile	During	500 lb. each aggregate 5 gal. asphalt	One/Project One/Project	After	Department
Tensile Strength	Indirect Tensile	During	150 lb. aggregate 4 lb. filler 8 lb. asphalt	One/project One/Project One/Project	After	Department
In-place Density	Troxler Nuclear Method (Back-scatter Air Gap)	Not Applicable	Not Applicable	Three Minimum/Section Five Cores/Section	During	Department
Thickness	LDH TR 602-67	Not Applicable	Not Applicable	Three Minimum/Section Five Cores/Section	During	Department
CEMENT STABILIZED SAND-CLAY-GRAVEL						
Mix Design	LDH TR 432-71	Before	36 lb. cement 300 lb. aggregate	As Required for Design	Before	Department
Modulus of Elasticity	Indirect Tensile Test	During			After	Department
Poissons Ratio	Indirect Tensile Test	During			After	Department
Tensile Strength	Indirect Tensile Test	During			After	Department
Fatigue Properties	Repeated Indirect Tensile Test	During			After	Department
In-place Density & Moisture	LDH TR 401-67	Not Applicable	Not Applicable	Three Minimum/Section	During	Department
Thickness	LDH TR 602-67	Not Applicable	Not Applicable	Three Minimum/Section	During	Department
ASPHALT CEMENT						
Specific Gravity, 77°F	AASHO T228-68	Before	1 qt. for all tests	Routine Number	Before	Department
Specific Gravity, 60°F	AASHO T228-68	Before		Routine Number	Before	Department

TABLE 12 (CONTINUED)

CONSTRUCTION SAMPLING AND TESTING PROGRAM

Material and Test	Test Method	Sampling Time	Sampling Size	Sampling Frequency	Testing Time	Testing Agency
Wt. per gallon, 60°F	AASHO T228-68	Before		Routine Number	Before	Department
Flash Point, C.O.C., °F	AASHO T48-68	Before		Routine Number	Before	Department
Viscosity						
Saybolt Furol Sec. @ 275°F	AASHO T72/102	Before		Duplicate	Before	Department
Absolute @ 140°F, Poises	AASHO T202	Before		Duplicate	Before	Department
~~Pen. @39.2°F, 200g, 60 sec.~~	~~AASHO T49~~	~~Before~~		~~Duplicate~~	~~Before~~	~~Department~~
Pen. @77°F, 100g, 60 sec.	AASHO T49	Before		Duplicate	Before	Department
Duct. @39.2°F, 5cm/min,cm	AASHO T51	Before		Duplicate	Before	Department
Thin Film Oven Test	AASHO T179-68	Before		Duplicate	Before	Department
Loss % @ 325°F, 5 hrs.		Before		Duplicate	Before	Department
Pen. of Residue @ 77°F		Before		Duplicate	Before	Department
Residue Pen. % of Orig.		Before		Duplicate	Before	Department
Duct. of Residue @ 77°F		Before		Duplicate	Before	Department
Pen. of Residue @ 32°F						
Solubility in CS 2%	AASHO T44			Routine Number	Before	Department
Homogeniety Test	AASHO T102			Routine Number	Before	Department
Mixing Temperature	AASHO T72/102			Routine Number	Before	Department
	AASHO T202			Routine Number	Before	Department

TABLE 12 (CONTINUED)

CONSTRUCTION SAMPLING AND TESTING PROGRAM

Material and Test	Test Method	Sampling			Testing	
		Time	Size	Frequency	Time	Agency
PORTLAND CEMENT						
Autoclave Expansion	ASTM C151	Before	1 gal. of each sample of all tests	Routine Number	Before	Department
Time of Setting by Vicat Needle	ASTM C191	Before		Routine Number	Before	Department
Air Content	ASTM C185	Before		Routine Number	Before	Department
Compressive Strength	ASTM C109	Before		Routine Number	Before	Department
Fineness by Air Permeability Apparatus	AASHO T163	Before		Routine Number	Before	Department
Normal Consistency	ASTM C187	Before		Routine Number	Before	Department

TABLE 13

POST CONSTRUCTION MEASUREMENTS PROGRAM

	Program Title and Description	Measurement Schedule
I.	Undisturbed subgrade cores, four-inch thinwall tubing - three samples from each, thickest and thinnest section of each base type - three samples from control. For basement soil characteristics. Coring for Programs I, II and III will be combined.	1. Initial measurements six months after construction. 2. Run special test in second or third year for wet and dry seasons.
II.	Four-inch core samples of base and surfacing for thickness, density, modulus of elasticity and (asphalt mixes only) creep test. Three cores from each section. One additional set may be needed from the asphalt base sections depending on the outcome of development work on the creep test procedure.	1. Cores six months after construction. 2. Repeat in a subsequent year if rutting occurs.
III.	Cores samples for tensile strength tests.	
IV.	Sample for fatigue properties of surfacing and base (asphalt mixes only). Roadway samples may be re-required if performance indicates that original lab tests were unconservative in predicting fatigue cracking.	1. Beam samples in a subsequent year if needed.
V.	Visual observations, cracking maps and condition survey.	1. Initial measurements within first three months after construction. 2. Routine: once per year. 3. Special: as needed, at more frequent intervals.

VI. Deflection measurements: 1. Deflection using Benkelman Beam in outer wheel path. 2. Deflection using Dynaflect. 3. Texas Surface Curvature 4. Read thermocouples each time deflections are taken. (Surface temperature plus two-inch interval of depth, for 5 1/2-inch surfacing and thichest base - in hot mix only).	1. Initial measurements within one month after construction. 2. Routine: two times per year during March and August. 3. Benkelman Beam tests will be dropped as soon as correlation with Dynaflect is established for this project - about 2 years.
VII. Profilometer measurements: 1. Chloe Profilometer with calculated PSI. 2. Mays Ride Meter. 3. BPR Roughometer.	1. Initial measurements within one month after construction. 2. Chloe Profilometer will be dropped as soon as correlation with Mays Ride Meter is established. 3. Special: as needed, at more frequent interval if conditions are changing rapidly.
VIII. Rut Depth Measurements using AASHO Road Test A - frame rut depth device or similar.	1. Same as Chloe Profilometer schedule, except if special conditions indicate more frequent tests.
IX. Traffic count.	1. Once a year to establish growth pattern. 2. More frequent intervals to establish large seasonal variation if needed. 3. Continuous count from permanent station adjacent to project.

X. Truck weight study. 1. Classify and record loads in motion by a system of transducers.	1. Continuing traffic evaluation by loop detector and weigh-in-motion tranducers applied in outer lane. 2. Check seasonal variation of agricultural traffic.

APPENDIX B

CONSTRUCTION AND MATERIALS SAMPLING

Asphaltic Concrete Materials

Cement Stabilized Base

Select Soil

Embankment Soil

ASPHALTIC CONCRETE

Most of the asphaltic concrete for this experimental project was produced at the contractor's Hot Wells batch plant, located approximately thirty miles northeast of the job site. The following listing is an average representation of the percent passing aggregate gradation, the asphalt content, the percent crushed aggregate, the plant batch temperatures, and the field compaction temperatures for all three mix types produced:

Sieve	Type 5-A Base	Type 3 Binder	Type 3 Wearing
1 inch	100	100	100
3/4 inch	93	100	100
1/2 inch	81	89	88
No. 4	49	57	61
No. 10	34	43	48
No. 40	21	26	26
No. 80	11	13	13
No. 200	5	8	6
Asphalt, %	4.0	4.4	4.6
Crushed, %		75	83
Plant Temp., °F	315	320	320
Road Temp., °F	280	300	300

All mix types were composed of siliceous sands and chert gravel. A mineral filler (silica dust) comprised part of the aggregate gradation for the binder and wearing course mixes (2 and 3 percent, respectively). The asphalt cement was a Lion Oil (El Dorado, Arkansas) AC-40 grade with typical properties as shown in Table 9 in Appendix F.

Construction of the asphaltic concrete portion of the experimental project occurred during 1976, as follows:

Type 5-A Base - April 19 through June 2, 1976
Type 3 Binder - June 7 through June 10, 1976
Type 3 Wearing - June 28 and June 29, 1976

The various design thicknesses of eight of the ten black base sections were constructed in two equal lifts with approximately 0.04 gsy tack coat between lifts; section 7's base (4 1/2 inches) and section 11's base (3 inches) were constructed in a single lift. All binder course section design in excess of 2 inches were constructed (and tacked) in two equal lifts. On June 29, 1976, the contractor switched batch plants to complete the wearing course from test section 10 though C-1.

All plant production during the construction period was sampled and tested in accordance with normal state practices. Table 10 in Appendix F lists by construction date the following mix properties:

Marshall Stability
Gradation
Asphalt Content
Percent Crushed (binder and wearing course mixes)

Cores were taken in each section at three locations in the outside wheel path of the outside lane to determine as-constructed section thicknesses. The cores were also used for density verification.

In an effort to establish the relationship between section life and fundamental material properties, raw materials used in the production of each mix ty[e (base, binder, and wearing) were sent to the Asphalt Institute. These materials will be used to reproduce mix specimens which will be tested for modulus of elasticity, tensile strength, and fatigue properties. Another part of this effort was the shipment to the Institute of 56 full depth hot mix cores (4 from each black base section and 2 from each other section). These cores were taken in early July of 1977, approximately ten months after the roadway had been open to traffic. The Asphalt Institute determined in-place modulus of elasticity from the cores.

SOIL-CEMENT AND CEMENT-STABILIZED SAND CLAY GRAVEL

Testing done on the soil-cement material included classification for soil groups (same materials as the select soils), nuclear moistures and densities on the in-place material at the roadway site, laboratory design tests for optimum cement percentages, thickness measurements, and the taking of 7-day and 28-day cores. The physical cores were tested for compressive strengths and cement contents. One basic flaw with many of the cores was that the top 1 to 3 inches of the core either broke or separated due to compaction laminations. Also in the deep soil-cement sections (20, 16, 15 and 12 inches) which were cut-in in two separate layers, the cores came apart at the junction between the layers. Some cores showed 1/2 to 2 inches of raw soil between top and bottom layers which did not have the cement cut-in with the soil.

The 10-inch cement-stabilized sand clay gravel section was troubled in cut-in and compaction attempts during a medium to heavy rain. Compaction equipment had to be removed from the section because of the soupy condition of the roadway. Core attempts in this section produced only four fair cores out of 25 attempts.

SELECT SOIL

The select soil on the Louisiana Experimental Base job was placed according to plans with very few problems. The select soil was composed of sand A-2-4(0), avg. P.I. = 6; a sandy loam A-2-4(0), P.I. = 9; and a sandy clay loam A-2-6(0), avg. P.I. = 11. Testing performed on this material included soil classification, field nuclear moisture and density, R-values, and modulus of elasticity. The modulus of elasticity testing was done by the Asphalt Insitute.

EMBANKMENT

Tests were run on the embankment material for classification into soil groups and for R-values. The natural material in the southern part of the project (Sta. 157+00 to Sta. 202+00) was composed of silts A-4-(8), average plasticity index (P.I.) = 4; silty loams A-4-(8), avg. P.I. = 2; and silty clay loams A-6-(9), avg. P.I. = 14. The northern part (Sta. 202+00 to Sta. 265+25) consisted of silty clay A-6-(12), avg. P.I. = 20; medium silty clay A-7-6(18), avg. P.I. = 34; and heavy clays A-7-6(20, avg. P.I. = 41 or heavy clay A-7-6(20), avg. P.I. = 40. Nuclear moistures and densities were run on the embankment material in place. Undisturbed cores of the embankment materials were taken and sent to the Asphalt Insititute for resilient modulus testing.

APPENDIX C

PAVEMENT DISTRESS MEASUREMENTS --CRACKING, RUTTING

TABLE 14

PAVEMENT DISTRESS MEASUREMENTS

74-1G CRACKING DATA

DATE	CRACK	C01	C02	C03	C04	T01	T02	T03	T04	T05	T06	T07	T08	T09	T10	T11	T12	T13	T14
04/13/78	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						16		25				305		244			128	
08/24/78	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE								26				131		146			190	
11/02/78	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						0		10				57		46			25	
04/10/79	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						4		54		4		368	131	211		11	121	
11/13/79	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						38		134		31		129	36	215		6	152	
01/01/81	TYPE I																		
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						41		235		48		163	86	230		53	117	
12/12/81	TYPE I	100	70		40	220		405		550		485				30			
	TYPE II																		
	TYPE III																		
	PATCHING																		
	SHRINKAGE						323		736		281		1088	306	930		151	447	
09/23/82	TYPE I	535	280	585	245	675		502		491	115	485			25	775	33		293
	TYPE II											140			5				
	TYPE III																		
	PATCHING												40						
	SHRINKAGE				50		511		385		448		380	244	368		548	424	
02/03/84	TYPE I	245	750	515		205		193		59		130				295			807
	TYPE II					120		150		255		155							
	TYPE III																		
	PATCHING						63	50				263							
	SHRINKAGE	50																	

TABLE 14 (CONTINUED)

PAVEMENT DISTRESS MEASUREMENTS

74-1G CRACKING DATA

DATE	CRACK	C01	C02	C03	C04	T01	T02	T03	T04	T05	T06	T07	T08	T09	T10	T11	T12	T13	T14
02/21/85	TYPE I	220			815														
	TYPE II	200				85													
	TYPE III																		
	PATCHING																		
	SHRINKAGE																		
TOTALS	TYPE I	1100	1100	1100	1100	1100	0	1100	0	1100	115	1100	0	0	25	1100	33	0	1100
	TYPE II	200	0	0	0	205	0	150	0	255	0	295	0	0	5	0	0	0	0
	TYPE III	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	PATCHING	0	0	0	0	0	63	50	0	0	0	263	40	0	0	0	0	0	0
	SHRINKAGE	50	0	0	50	0	933	0	1605	0	812	0	0	803	2390	0	769	1604	0

TABLE 15

RUTTING DATA

74-1G RUTTING DATA

DATE	CO1	CO2	CO3	CO4	TO1	TO2	TO3	TO4	TO5	TO6	TO7	TO8	TO9	T10	T11	T12	T13	T14
12/06/77	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
04/13/78	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11/02/78	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
04/10/79	0.1	0.0	0.1	0.2	0.1	0.0	0.2	0.0	0.2	0.1	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.2
08/23/79	0.1	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
11/13/79	0.1	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
02/01/80	0.1	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
09/01/80	0.1	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
12/12/81	0.1	0.2	0.3	0.2	0.2	0.1	0.3	0.1	0.3	0.2	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.2
09/23/82	0.2	0.2	0.3	0.3	0.3	0.2	0.3	0.2	0.5	0.2	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.3
02/01/83	0.2	0.3	0.3	0.3	0.3	0.2	0.3	0.2	0.5	0.2	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.3
02/03/84	0.2	0.3	0.3	0.3	0.3	0.2	0.3	0.2	0.5	0.2	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.3
02/21/85	0.2	0.3	0.3	0.3	0.4	0.2	0.4	0.2	0.5	0.2	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.3

APPENDIX D

FIELD-MEASURED LAYER THICKNESS AND LAYER MODULI

TABLE 16

FIELD MEASURED LAYER MODULI AND LAYER THICKNESS

SECTION YEAR	WEARING/ BINDER COURSE	BLACK BASE	SOIL CEMENT	SELECT SOIL	EMBANKMENT SOIL
C01 THICKNESS	(6.7 ")	(6.3 ")	(6.7 ")	(")	
1978	591,400	659,500	741,500		17,250
1980	592,900	530,000	860,200		13,950
1984	358,500	497,900	684,700		18,260
C02 THICKNESS	(5.7 ")	(8.7 ")	(6.0 ")	(")	
1978	591,400	452,900	680,000		20,120
1980	696,500	487,500	675,300		15,480
1984	269,400	485,800	643,000		20,200
C03 THICKNESS	(5.8 ")	(7.1 ")	(6.6 ")	(")	
1978	591,400	489,100	860,200		14,910
1980	691,100	458,300	696,200		14,580
1984	345,600	442,100	693,400		17,560
C04 THICKNESS	(5.7 ")	(6.3 ")	(6.4 ")	(")	
1978	570,000	731,200	665,300		13,820
1980	441,700	553,300	860,200		14,270
1984	497,700	376,500	636,700		17,890
T01 THICKNESS	(6.9 ")	(7.7 ")	(")	(14.9 ")	
1978	437,800	712,400		37,400	14,260
1980	434,800	860,200		32,300	11,080
1984	310,100	752,400		36,700	14,120
T02 THICKNESS	(6.2 ")	(")	(8.4 ")	(8.9 ")	
1978	493,600		685,900	46,000	16,640
1980	395,900		640,000	35,200	12,760
1984	200,000		530,100	41,300	14,150
T03 THICKNESS	(4.0 ")	(11.1 ")	(")	(12.7 ")	
1978	860,200	376,200		41,300	15,120
1980	457,500	680,800		31,000	11,410
1984	345,600	596,300		42,800	14,720
T04 THICKNESS	(3.6 ")	(")	(19.2 ")	(3.3 ")	
1978	781,200		452,800	47,800	17,800
1980	506,200		365,700	34,700	12,970
1984	289,500		347,700	45,600	15,720
T05 THICKNESS	(3.4 ")	(9.3 ")	(")	(15.6 ")	
1978	508,100	530,100		34,200	13,470
1980	374,100	647,600		29,400	10,570
1984	345,600	466,500		37,000	12,740
T06 THICKNESS	(5.8 ")	(")	(15.5 ")	(6.4 ")	
1978	736,100		582,500	44,400	15,650
1980	458,100		529,800	37,000	13,320
1984	282,300		300,600	43,700	16,030

TABLE 16 (CONTINUED)

FIELD MEASURED LAYER MODULI AND LAYER THICKNESS

SECTION	YEAR	WEARING/ BINDER COURSE	BLACK BASE	SOIL CEMENT	SELECT SOIL	EMBANKMENT SOIL
T07	THICKNESS	(6.4 ")	(4.6 ")	(")	(16.2 ")	
	1978	408,700	620,500		33,200	12,550
	1980	369,500	530,100		31,500	11,200
	1984	249,000	618,900		34,200	14,300
T08	THICKNESS	(3.6 ")	(")	(15.6 ")	(9.0 ")	
	1978	860,200		362,000	37,500	13,980
	1980	525,700		269,700	34,900	12,010
	1984	426,000		290,200	40,800	14,060
T09	THICKNESS	(5.9 ")	(")	(6.4 ")	(14.2 ")	
	1978	860,200		860,200	36,900	12,680
	1980	390,400		860,200	33,800	11,550
	1984	283,800		530,100	30,200	10,730
T10	THICKNESS	(5.2 ")	(")	(13.0 ")	(7.8 ")	
	1978	696,400		692,900	35,300	13,250
	1980	457,600		663,500	35,500	13,260
	1984	516,200		525,300	37,200	12,810
T11	THICKNESS	(5.9 ")	(4.4 ")	(")	(17.6 ")	
	1978	600,900	607,800		26,300	9,950
	1980	576,400	851,700		32,200	12,770
	1984	497,700	761,200		35,300	13,870
T12	THICKNESS	(5.8 ")	(")	(8.4 ")	(10.3 ")	
	1978	781,400		626,200	33,800	12,350
	1980	345,500		634,000	36,300	12,410
	1984	221,000		530,100	37,500	13,030
T13	THICKNESS	(4.6 ")	(")	(9.7 ")	(14.6 ")	
	1978	756,000		652,700	33,600	12,490
	1980	489,300		530,100	35,200	12,100
	1984	292,700		447,300	34,700	14,470
T14	THICKNESS	(3.9 ")	(6.5 ")	(")	(13.7 ")	
	1978	633,500	530,100		31,900	11,300
	1980	351,700	718,900		30,000	10,900
	1984	327,000	530,100		37,200	13,180
AVG	THICKNESS	(5.3 ")	(7.2 ")	(10.2 ")	(11.8 ")	
	1978	653,250	570,980	655,183	37,114	13,678
	1980	475,272	631,840	632,075	33,500	12,022
	1984	336,539	552,770	513,267	38,157	13,852

APPENDIX E

FUNDAMENTAL ENGINEERING PROPERTIES OF CONSTRUCTION MATERIALS

By

Dr. W. O. Hadley

Excerpts from a Summary Report

SUMMARY OF FINDINGS

The significant conclusions reached from this study are summarized as follows:

Material characterization.

1) The soil cement construction procedure used in the construction of the experimental base project apparently does not provide a uniform product. In fact, the procedure apparently resulted in a number of flaws in the various cement stabilized base layers. Evidence of this condition was obtained from two separate coring operations to obtain specimens throughout the length of the Louisiana Experimental Base Project. In these two operations only 125 good cores were obtained from a total of 341 core locations (a recovery rate of 37%). From this information it can be postulated that over one half of soil cement areas would have internal flaws. Some flaws encountered in the cement stabilized material included laminations, cracks, compaction planes, cutter planes, flushing (or migration of cement to a flawed area) and clay balls.

2) There are two levels (or populations in statistical terms) of soil cement base materials existing at the Louisiana Experimental Base Project. One level of soil cement was considered to be clear of major flaws, while the second level was composed of those soil cement materials with major flaws. The clear and flawed soil cement materials were found to have the same basic behavioral response (i.e. modulus and Poisson's ratio values are similar) but drastically different performance characteristics (i.e. consideration of fatigue in producing fracture distress mode).

3) Comparative studies of the fatigue results for laboratory prepared soil-cement specimens and the flawed soil-cement field cores indicate that (1) the laboratory fatigue data was not significantly different than the field core fatigue results; and (2) the laboratory fatigue data could very well be representative of a majority of the soil-cement material in place at the Louisiana Experimental Base Project. The low recovery rate of good (or clear) field cores and the high moisture contents found in the cement-stabilized base layers are

in comparable fundamental material properties such as modulus, E, and fatigue parameters for the beam and indirect tensile tests.

13) In the finite element model study of the unconfined compression test, Poisson's ratio and aspect ratio exhibited significant effects on the maximum compressive stress and maximum tensile strains developed in the test specimen as well as the modulus estimates obtained from stress-deformation results. Equations which included the effects of boundary loading conditions were developed for estimating maximum compressive stress, maximum tensile strain, as well as for estimating modulus for three different strain measurement techniques (i.e. center strain, mid-level deformation and total deformation).

14) From statistical comparisons of modulus values obtained from the equations developed for the unconfined compression and indirect tensile tests, it was found that there were no significant differences in either the means or variances for the modulus values From this it can be suggested that the modulus determined from the unconfined compression test is essentially the same as that value obtained from the indirect tensile test for soil-cement and asphaltic materials.

15) The practical equality of modulus values (based on consideration of boundary loading conditions) for soil cement and asphaltic materials obtained from the unconfined compression test, the indirect tensile tests, as well as the beam test, further indicate that the modulus of these materials is essentially the same in tension (i.e. beam and indirect tensile tests) and compression (i.e. unconfined compression test). These results provide basic supportive evidence of the reasonableness of the recent trend toward the use of elastic layered analyses for pavement structures. It appears that layered elastic theory, as well as finite element theory, can be used to design, evaluate or investigate various pavement structures without having to resort to bimodular considerations.

16) A finite element mechanistic evaluation of the confined triaxial test produced a modulus adjustment algorithm which provided fundamental (adjusted for boundary loading conditions) modulus

estimates from measured (apparent) modulus values. It is believed that the algorithm eliminated and/or reduced the influence of Poisson's ratio and confining pressure on measured modulus values.

Applications of Multilayer Elastic Theory - Supplemental Studies.

17) The utilization of fundamental engineering property values (i.e. modulus and Poisson's ratio) and characterization techniques, developed within this study, to provide input for the elastic layer program BISAR yielded estimates of Benkleman beam and dynaflect deflections for the test sections within reasonable agreement with corresponding measured deflection values. Based upon this observation, it appears that with appropriate material characterization efforts elastic layered theory can be used in an evaluation and design of both flexible and composite pavement structures.

18) It appears that an essentially linear relationship existed between Benkelman beam and dynaflect deflections and that the relationship originated at the origin of the plot. From this effort, it was established that the factor relating the Benkelman beam deflection to the dynaflect deflection was 20.0 (i.e. BB = dyn). This factor was remarkably close to the Louisiana factor of 21.8; however, it should be noted that the LADOTD relationship is based upon dynamic deflections adjusted to a common temperature of 60°F, whereas the factor of 20.0 presented here was based on raw dynaflect deflections regardless of asphalt temperature.

19) Regression equations were developed which are capable of predicting load equivalency factors for varying fundamental property values (i.e. E), thicknesses, layers and loads. The load variable appeared to be the most important effect in establishing equivalencies for layers other than asphalt layers. The most important effect in establishing equivalency factors for asphalt and black base layers was the modulus of the appropriate asphaltic material. In addition, fundamental properties (i.e. modulus) and thicknesses were found to have a significant effect on equivalency factors; therefore, mix design and construction variables can exert a strong influence on load

equivalency values.

20) The results of the finite element model study of the effects of discontinuities (i.e. cracks) on stresses and strains illustrate that there was a drastic change in strain response of cracked pavements when compared with corresponding uncracked models. This was particularly true when a crack is introduced at the edge of the wheel load area. For cracks located beyond a distance of 24 inches from the load centerline, there was little or no change in the strain response in the pavement layers. Increases in strain response of a pavement caused by cracking depended upon layer thickness, material properties, and position of that particular layer in the structural section.

21) A technique utilizing Mohr's circle was developed to allow for estimation of cohesion, C, and angle of internal friction from resilient confined triaxial test results. The resulting C - data was used to estimate the confining pressure conditions for the select and embankment soils for all test sections of the Louisiana Experimental Base Project. The select and embankment modulus values were established considering the appropriate confining pressures.

22) Since the development of longitudinal cracking along the wheel path was apparently not predictable by VESYS IIIA, then a technique for estimating this type of surface cracking, continuing development of rutting and accompanying pavement surface distortion was developed. This phenomenon (i.e. cracking due to rutting) would probably be initiated at the pavement surface and extend downward. Any available moisture would accumulate and percolate downward through this type of crack. Oxidation, exposure to sun rays and other environmental effects would directly affect any exposed portions of the surface cracks. These environmental problems would not exist for a fatigue crack which is generally believed to begin at the bottom of a layer and migrate upwards.

<u>Performance Comparisons--Deterministic Predictions vs. Field Results</u>

23) The VESYS IIIA mechanistic analysis package can adequately predict rut depths over a short period of time; however,

the cracking predictions could not be evaluated since no significant amount of fatigue cracking had developed at the Louisiana Experimental Base Project. Comparisons between measured and predicted PSI values illustrated that the VESYS IIIA model was adequate for short periods of evaluation. The accuracy of the PSI predictions over long time periods may be questionable because consideration would not be given to slope variance related to shrinkage and rut distortion cracking.

24) The PDMAP mechanistic analysis package does not provide the capabilities available in the VESYS IIIA package. PDMAP is limited in that the fatigue and rutting models must be developed from inservice pavements. As a result a variety of sections would have to be evaluated over a long time frame in order to develop the necessary models for a range of design conditions. Attempts to develop these models from layered theory were unsuccessful.

25) The combination of indirect tensile test data and consideration of appropriate temperature effects can produce material characterization information which result in short term VESYS IIIA predictions compatible with measured field results.

26) The use of a terminal surface cracking prediction technique based on rutting distortion (for black base sections) or combined shrinking and rutting distortion (for soil cement sections) may be appropriate to complement the VESYS IIIA analysis package. Additional performance data should provide substantiation for its utilization.

27) If surface cracking prediction are included as a complementary part of a VESYS IIIA evaluation, it is believed that better long-term performance predictions could be developed for flexible (i.e. with asphalt surface and base layers) as well as composite pavements (i.e. those including soil cement bases and asphalt layers).