ARIZONA DEPARTMENT OF TRANSPORTATION

REPORT NUMBER: FHWA/ AZ - 80/157

ENVIRONMENTAL FACTOR DETERMINATION FROM IN-PLACE TEMPERATURE AND MOISTURE MEASUREMENTS UNDER ARIZONA PAVEMENTS

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September 1980
Final Report

Prepared for:
U.S. Department of Transportation
Federal Highway Administration
Arizona Division

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Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/AZ-80/157		

4. Title and Subtitle	5. Report Date
Environmental Factor Determination from in-place Temperature and Moisture Measurements under Arizona Pavements	August, 1980
	6. Performing Organization Code
	8. Performing Organization Report No.
7. Author(s) George B. Way	
9. Performing Organization Name and Address Arizona Department of Transportation Research Section 206 South 17th Ave Phoenix, Az 85007	10. Work Unit No. (TRAIS)
	11. Contract or Grant No. HPR-1-18(157)
12. Sponsoring Agency Name and Address Arizona Department of Transportation	13. Type of Report and Period Covered
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Thirty-seven sites were selected and monitored for five years for temperature, moisture and deflection. From this monitoring, plus detailed materials sampling and testing at each site, a myriad of information was found. In general, subgrade materials do become wetter after the surface paving is completed. Base materials experience long term drying and densification. Significant freezing takes place in the higher elevations of Arizona. Deflection measurements using the dynaflect indicated little if any correlation to temperature; however, a very good correlation was found with percent cracking divided by AC pavement thickness. In addition, dynaflect deflections seem to be influenced the most by the subgrade deflection which is directly related to the weight percent moisture content. Bound surface layers reduce the subgrade deflection; but as cracking takes place, the deflection increases until it reaches the subgrade value. Plate bearing deflections indicate the worth of unbound bases whereas the dynaflect does not.

In terms of pavement performance, the AASHTO interium guidelines for design do not appear to have either increased the actual pavement life or improved design reliability. On the average, AASHTO designs do perform for 20 years; however, a very large variance in pavement performance exists.

17. Key Words	18. Distribution Statement
Temperature, Moisture, Deflection Cracking, Performance, Asphaltic Concrete, Concrete	No restrictions - this report is available to the public through NTIS, Springfield, VA 22161

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

ABSTRACT

Thirty-seven sites were selected and monitored for five years for temperature, moisture and deflection. From this monitoring, plus detailed materials sampling and testing at each site, a myriad of information was found. In general, subgrade materials do become wetter after the surface paving is completed. Base materials experience long term drying and densification. Significant freezing takes place in the higher elevations of Arizona. Deflection measurements using the dynaflect indicated little if any correlation to temperature; however, a very good correlation was found with percent cracking divided by AC pavement thickness. In addition, dynaflect deflections seem to be influenced the most by the subgrade deflection which is directly related to the weight percent moisture content. Bound surface layers reduce the subgrade deflection; but as cracking takes place, the deflection increases until it reaches the subgrade value. Plate bearing deflections indicate the worth of unbound bases whereas the dynaflect does not.

In terms of pavement performance, the AASHTO interim guidelines for design do not appear to have either increased the actual pavement life or improved design reliability. On the average, AASHTO designs do perform for 20 years; however, a very large variance in pavement performance exists.

FORWARD AND MISCELLANEOUS THOUGHTS

This report represents a compilation of a very large amount of data. To analyze and interpret this data, the author has relied on statistical inference. Reported correlations had to be not only significant but also reasonable in an engineering sense. In other words, reasonable cause and effect had to exist before a correlation was accepted. The author avoided using multiple regression analysis because of a fear that an unreasonable but perfectly good correlation might arise; therefore, it is possible that other researchers might want to investigate multiple relationships.

In order to achieve a readable report, no laboratory test numbers are given in the text. Generally, the AASHTO Test Procedure was followed except in a few cases. The following list gives the test number for each test mentioned in the report.

LABORATORY TESTS

AASHTO TEST NUMBER	TEST NAME
T 87	Dry Preparation of Disturbed Soil
T 88	Hydrometer
T 89	Liquid Limit
T 90	Plastic Limit and Plasticity Index
T 99	Proctor Moisture Density
T 100	Specific Gravity of Soils
T 190	R Value Test
T 203	Auger Borings
T 204	Density of Soil In-Place by the Drive Cylinder Method
T 205	In-Place Desity, Rubber Balloon, Volumeter
T 207	Thin Walled Tube Sampling of Soils
T 222	Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components
T 224	Correction for Coarse Particles in the Soil Compaction Test

AASHTO TEST NUMBER	TEST NAME
T 49	Penetration of Bituminous Materials
T 72	Saybolt Viscosity
T 179	Effect of Heat and Air on Asphalt Materials (Thin-Film Oven Test)
T 201	Kinematic Viscosity of Asphalts
T 202	Absolute Viscosity of Asphalts
T 209	Maximum Specific Gravity of Bituminous Paving Mixtures
T 22	Compressive Strength of Cylinder Concrete
T 176	Sand Equivalent

ARIZONA TEST NUMBER	TEST NAME
Arizona 209	Moisture Content, Oven Dry
Arizona 210a	Specific Gravity and Absorption of Coarse Aggregates
Arizona 236	Ph and Minimum Resistivity of Soils

OTHER RESEARCH TESTS

Viscosity at 77°F	-	Micro-Viscosity Test
% Asphaltenes, N, A1, A2 and P	-	Rostler Asphalt Analysis
Vanadium	-	Atomic Absorption Test
Permeability	-	Constant Head

The author wishes to thank a number of people who contributed advise and help in the preparation and conduction of this project. They are:

Douglas Forstie, Senior Laboratory Engineer
Harold "Knobby" Walsh, Geotechnical Engineer
Marshall Christenson, Materials Technician
Bobby Rogers, Research Technician
Walter Chase, Materials Technician
Eugene Poluianov, Materials Technician
Pavement Evaluation (Dave Allocco in particular)
District IV, Holbrook, Materials Staff (Frank Adair in particular)
District V, Flagstaff, Materials Staff (Art Norte in particular)
Rowan Peters, Assistant Engineer of Materials
Edward Burgin, Assistant Engineer, Contracts and Specifications
John Eisenberg, Management Analysis Statistics
Gene R. Morris, Engineer of Research

To each of them, thank you for being there when the volumeter froze solid; the soil auger broke (just about every time); the plate bearing jack stuck just after digging a hole in the pavement; construction ripped up the Lupton conduit -- twice; the dynaflect wouldn't work; soil samples were lost. These are referred to as field problems and, although they did occur, in retrospect, they represent that ingredient which made this project challenging and fun.

Some may argue that these sites do not represent the entire state. Sites were selected for safety, convenience and different environments. In addition, and most importantly in five years of monitoring, which included traffic control for each and every deflection and moisture reading, not one accident occurred.

In summary, this has been a tough report to write, primarily because of the volume of data. Since so much information was collected on each site (about 20 ring binders worth) the author suggests that these 37 sites become a set available to future work and continue to be monitored on a less frequent (annual) basis. In addition, the future construction activities be kept track of for future investigations.

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ENVIRONMENTAL FACTOR DETERMINATION FROM IN-PLACE TEMPERATURE AND MOISTURE MEASUREMENTS UNDER ARIZONA PAVEMENTS - PHASE II

INTRODUCTION

Arizona, at present, uses the AASHTO Interim Guide (1) equations to determine thickness of structural layers (flexible and rigid) for new highway construction. Inputs to these equations are based on past experience and have been empirically developed. In the future, Arizona intends to adopt a systems approach to design of both flexible and rigid pavements. Various systems for design and management have been proposed by Monismith, Hudson and Deacon (2), NCHRP 139 (3) and HRB Special Report 126 (4) to name a few. All of these systems call for several inputs including traffic, environment, construction, structural capacity and maintenance schedule. The long term, five-year objective of this study was to quantify the environmental inputs and relate them to structural characteristics.

In order to meet the objective, a plan of measurement and monitoring over the five-year period was adopted. The execution of this plan involved three phases.

1. Selection of suitable highway projects and specific sites representative of the different climatological zones, soils and traffic. Such projects also cover a large time span of construction from 1941 to 1975. Specific history about each project would be researched to determine what was built and when.

2. After site selection, sampling would be conducted at each site to determine in-place properties, index properties and materials characteristics. Samples were taken as part of five separate operations spanning several years. These included:

   A. Installation of aluminum tubes for nuclear depth moisture installation. Soil samples for moisture, grading and plasticity index (PI).

   B. Drive sampling with a Dames and Moore sampler to obtain samples for in-place density, moisture and shear test.

   C. Installation of thermistors and soil moisture wafers at recording sites to record temperature and moisture. In addition, in-place densities were taken with a volumeter and soil samples for moisture, PI and gradation were also taken.

D. Plate bearing tests were conducted at most sites on the surface, base and subgrade layers. In addition, a set of dynaflect deflections were performed immediately before the plate bearing test. In-place density of the base and subgrade layer was conducted with a volumeter. Following this, soil samples were taken to be tested for PI, grading, proctor moisture density, specific gravity, sand equivalent hydrometer analysis, R value, pH, resistivity and permeability.

E. Cores of asphaltic concrete were removed to obtain the modulus of resiliency of the AC at various temperatures. Also, asphalt content and asphalt properties such as penetration at 77° F, absolute viscosity @ 140° F, microvisosity at 77° F and Rostler analysis were determined.

3. In addition to the single point in time measurements and samples, long-term pavement performance measurements were taken. These included dynaflect deflection, Mays ride meter roughness, rut depth and cracking and patching. Also, changes in moisture were monitored with both the nuclear and the soil moisture wafer devices. Temperature measurements were automatically recorded every other hour on a daily basis.

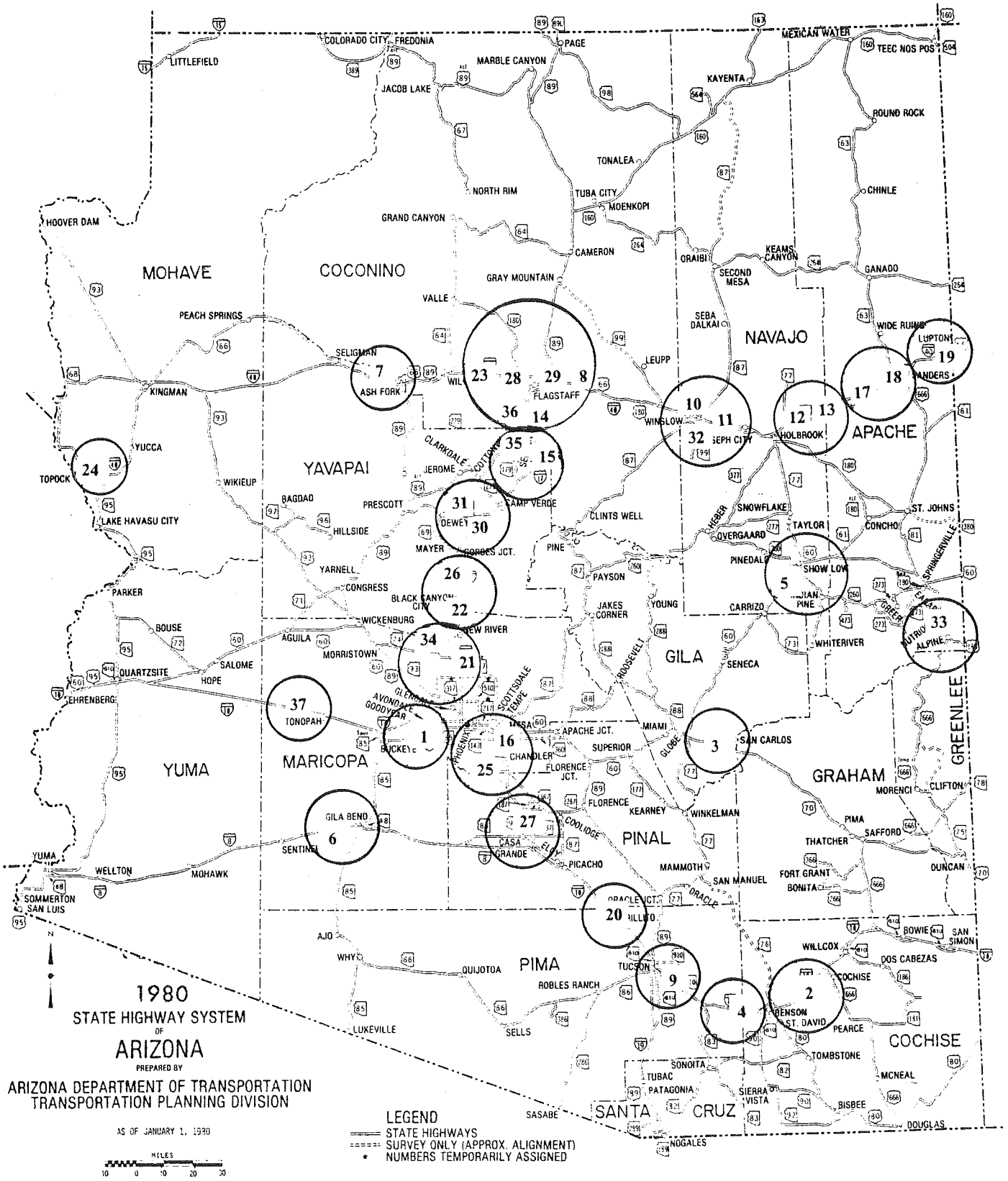
The above information will be fully displayed and reviewed. From this, the analysis section will extract the meaning of the above data in terms for predictive models and expected performance.

SITE SELECTION

Sites were selected during Phase I (5) to accommodate either the nuclear depth moisture or continuous recording facilities. Phase I (5) describes the entire temperature and moisture installation procedure. Figure 1 gives a map showing the 37 selected sites. Table 1 gives detailed information about each site. Sites are listed in chronological order by date of construction. Each site is numbered and this number will be used in all tables and figures where site information is given. Table 1 describes those features which are thought to be fixed or unchanging with time after construction. These include:

Site number
Site name
Route number
Direction
Mile post (MP) location
Station
Project number
Date built of first project
Elevation in feet and meters
Geological area
Topography, vegetation and drainage
Date built of all major construction activities
Thickness in inches of all structural layers above subgrade

FIGURE 1


1980
STATE HIGHWAY SYSTEM
OF
ARIZONA
PREPARED BY
ARIZONA DEPARTMENT OF TRANSPORTATION
TRANSPORTATION PLANNING DIVISION
AS OF JANUARY 1, 1980
MILES
10 0 10 20 30
LEGEND
STATE HIGHWAYS
SURVEY ONLY (APPROX. ALIGNMENT)
* NUMBERS TEMPORARILY ASSIGNED
MOHAVE
COCONINO
YAVAPAI
NAVAJO
APACHE
GILA
GREENLEE
YUMA
MARICOPA
PINAL
GRAHAM
PIMA
COCHISE
SANTA CRUZ
LITTLEFIELD
COLORADO CITY
FREDONIA
PAGE
MARBLE CANYON
JACOB LAKE
MEXICAN WATER
TEEC NOS POS
KAYENTA
ROUND ROCK
TONALEA
NORTH RIM
TUBA CITY
MOENKOPI
CHINLE
HOOVER DAM
GRAND CANYON
CAMERON
ORAIBI
KEAMS CANYON
SECOND MESA
GANADO
GRAY MOUNTAIN
VALLE
PEACH SPRINGS
SEBA DALKAI
WIDE RUINS
LUPTON
SELIGMAN
ASH FORK
FLAGSTAFF
LEUPP
KINGMAN
WINSLOW
HOLBROOK
ANDERS
YUCCA
TOPOCK
WIKIEUP
JEROME
CLARKDALE
COTTONWOOD
SEDONA
CAMP VERDE
BAGDAD
PRESCOTT
DEWEY
MAYER
CORDES JCT.
CLINTS WELL
SNOWFLAKE
TAYLOR
CONCHO
ST. JOHNS
SPRINGERVILLE
LAKE HAVASU CITY
HILLSIDE
YARNELL
PINE
PAYSON
OVERGAARD
PINEDALE
SHOW LOW
PARKER
CONGRESS
BLACK CANYON CITY
WICKENBURG
NEW RIVER
JAKES CORNER
YOUNG
CARRIZO
WHITERIVER
BOUSE
AGUILA
MORRISTOWN
SENECA
ALPINE
QUARTZSITE
SALOME
HOPE
EHRENBERG
TONOPAH
GLENDALE
AVONDALE
GOODYEAR
SCOTTSDALE
TEMPE
MESA
APACHE JCT.
MIAMI
SUPERIOR
SAN CARLOS
BUCKEYE
PHOENIX
CHANDLER
FLORENCE JCT.
FLORENCE
KEARNEY
WINKELMAN
MORENCI
CLIFTON
GILA BEND
SENTINEL
CASA GRANDE
COOLIDGE
PICACHO
MAMMOTH
SAN MANUEL
PIMA
SAFFORD
THATCHER
FORT GRANT
BONITA
DUNCAN
YUMA
WELLTON
MOHAWK
SOMMERTON
SAN LUIS
ORACLE JCT.
ORACLE
AJO
WHY
QUIJOTOA
TUCSON
ROBLES RANCH
WILLCOX
BOWIE
SAN SIMON
DOS CABEZAS
COCHISE
BENSON
ST. DAVID
PEARCE
LUKEVILLE
SELLS
TOMBSTONE
SONOITA
TUBAC
PATAGONIA
SIERRA VISTA
MCNEAL
BISBEE
SASABE
NOGALES
DOUGLAS
1
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Layers are abbreviated and mean the following:

ACFC - Plant mixed seal coat or plant mixed asphaltic concrete finishing course or plant mixed asphaltic concrete friction course.
Seal Coat - Layer consisting of an asphaltic emulsion plus chips
AC - Plant mixed asphaltic concrete
PCCP - Concrete
BTB - Bituminous treated base, also plant mixed
CTB - Cement treated base
AB - Aggregate base
SM - Select material
SGS - Subgrade seal

TABLE I

SITE GENERAL DESCRIPTION

SITE NUMBER AND NAME EXACT LOCATION AND PROJECT NUMBER	DATE BUILT	ELEV. FT. (METERS)	GEOLOGY	TOPOGRAPHY VEGETATION DRAINAGE	DATE BUILT	THICKNESS IN INCHES*									
						ACFC	SEAL COAT	AC	PCCP	BTB	BSB	CTB	AB	SM	SGS
1. Avondale	1936	994	Qs	Flat	1936			2.0	6.0						
Buckeye — 91st Ave.		(303)	Alluvial	Irrigated land on both	12/1956		.3	2.0					4.0	8.0	
I 40 EB, MP 183.75			Sediments with some	sides of road; no curb	8/1974	.5		1.5							
STA 1235+00 S371 (1)			caliche	slow infiltration	11/1977	.5									
2. Sybil Rd. T.I.	1940	4010	QTs Old Valley	Rolling Hills	1940						2.0				
I 10 WB, MP 311		(1222)	Fill Deposit	Desert Grass	7/1960		.3	3.0					3.0	9.0	
STA 283+26 - I 10-6 (9)				Slow Infiltration	10/1975	.5		1.3							
3. Cutter	1941	3209	Qs	Flat	1941						2.0		3.0		6.0
US 70, MP 258.65		(978)	Alluvial	Desert Salt Bush	1964		.3								
STA 84+72			sediments	Creosote Bush	1970		.3								
FLH 13A-1				Slow Infiltration	1974		.3								
4. Benson	1942	4186	Qs	Flat	1942						2.0		3.0	12.0	
I 10 WB, MP 300.07		(1276)	Alluvial	Desert	6/1965	.5		4.0							
STA 2159			Sediments with some	Grass	9/1975	.5		1.5							
SNFA I 8C-D (4)			caliche	Slow Infiltration											
5. Show Low	1946	6384	Ks	Rolling Hills	1946		.3	4.0					4.0	15.5	
US 60, MP 337.74		(1946)	Limestone	Ponderosa Pine	1972		.3								
STA 21+50			shale & limestone	Slow Infiltration	1975		.3								
FH 30 A-3															
6. Gila Bend	5/1955	720	Qfp	Flat	1955			2.0					4.0	4.0	
I 8 EB, MP 112.8		(223)	Alluvial	Desert Salt Bush	1970	.8		4.0							
STA 489+70			sediment some caliche	Creosote Bush	1972		.3								
FI-56 (3)				Slow Infiltration	1976	.5									
7. Ash Fork	1956	5139	QTb	Flat	1/1956						2.5		4.0	11.0	
I 40 WB, MP 144.54		(1567)	Basalt & Andesite	Short grass	11/1964	.5		1.5		1.5					
ING 80 (18)			Clay Cap with	Slow Infiltration	1969		.3								
STA 190+43			Rock beneath												
8. Winona	7/1969	6160	Qb Basalt, Cinders and	Foot Hills Pinyon	1957		.3				4.0		3.0	9.0	
I 40 WB, MP 212.60		(1876)	Silt Mixed	Juniper	7/1969	.8		3.5							
STA 821+15 - I 40-4 (69)				Slow Infiltration	6/1977	.5		2.0							

*1 INCH = .0254 METERS

TABLE I (Con't.)

SITE NUMBER AND NAME EXACT LOCATION AND PROJECT NUMBER	DATE BUILT	ELEV. FT. (METERS)	GEOLOGY	TOPOGRAPHY VEGETATION DRAINAGE	DATE BUILT	THICKNESS IN INCHES*									
						ACFC	SEAL COAT	AC	PCCP	BTB	BSB	CTB	AB	SM	SGS
9. Wilmot Rd. T.I. I 10 EB, MP 270 STA 474+97 — I 10-5 (12)	12/1957	2790 (850)	Qw Younger Alluvial Deposit Intermittent Stream Channel	Flat Valley Floor Desert — Creosote & Salt Bush Very Slow Infiltration	12/1957		.3	2.5					4.0	9.0	
10. Winslow I 40 EB, MP 260.21 STA 286 I 40-4 (90)	7/1958	4897 (1493)	Qs Alluvial sediment	Flat, one mile from Little Colorado River; Short grass Slow Infiltration	7/1958		.3	3.5		3.0				15.0	
					9/1971	.5		1.3							
					6/1975		.3								
11. Minnetonka I 40 EB, MP 261.78 STA 369+00	7/1958	4884 (1489)	Sand and Clay		7/1958		.3	3.5		3.0				15.0	
					9/1971			2.0							
					6/1975		.3								
12. Dead River #1 Navajo Co. Line to Pinta I 40 EB, MP 316.68 STA 500+00 - I 40-5 (38)	7/1960	5568 (1697)	Trc Dessicated Clay Shales of Chinle Formation very Expansive	Flat Upper Plateau Short Grass Slow Infiltration	7/1960		.3	4.0				6.0		4.0	
					11/1973	.5		2.0							
13. Dead River #2 I 40 EB, MP 317.06 STA 520+00	7/1960	5610 (1710)			7/1960		.3	4.0				6.0		4.0	
					11/1973	.5		3.5							
14. Flagstaff N.B. I 17 NB, MP 337 STA 7215+80	8/1960	6958 (2121)	Pk Kaibab Limestone	Rolling Hills Ponderosa Pines Slow Infiltration	8/1960						1.0		6.0	10.0	12.0
					9/1966		.3	3.5							
					1974	.5									
15. Sedona T.I. I 17 NB, MP 298.54 STA 5171+42 — I 17-2 (20)	8/1960	3814 (1163)	QTs Older Valley Fill	Foothills to Mogollon Rim Pinyon-Juniper Very Slow Infiltration	8/1960		.3	4.0		3.8			3.0	15.0	6.0
16. Tempe US 60, MP 172 STA 37+00 Non F 022-3 (61)C	1/1961	1188 (362)	TKs sedimentary conglomerate	Curb & gutter, park with Eucalytus trees on one side Slow Infiltration	1/1961	.5		2.5					6.0	6.0	
					6/1974	.5									

*1 INCH = .0254 METERS

TABLE I (Con't.)

SITE NUMBER AND NAME EXACT LOCATION AND PROJECT NUMBER	DATE BUILT	ELEV. FT. (METERS)	GEOLOGY	TOPOGRAPHY VEGETATION DRAINAGE	DATE BUILT	THICKNESS IN INCHES*									
						ACFC	SEAL COAT	AC	PCCP	BTB	BSB	CTB	AB	SM	SGS
17. Crazy Creek #1 I 40 EB, MP 322.72 STA 819+50—I 40-5 (14)	9/1961	5599 (1707)	TRc Dessiccated Clay Shales of Chinle Formation very Expansive	Flat Upper Plateau Short Grass Slow Infiltration	9/1961		.3	4.0				6.0		6.0	
					10/1975	.5		2.50							
18. Crazy Creek #2 I 40 EB, MP 323.78 STA 875+00	9/1961	5636 (1718)			9/1961		.3	4.0				6.0		6.0	
					10/1975	.5		3.0							
19. Lupton I 40 WB, MP 359 STA 2871+50 I 40-5 (29)	1/1963	6184 (1885)	Jsr Sandstone	Mountains rushing up out of flat plateau Pinyon & Juniper High Infiltration	1/1963		.3	4.0				6.0		21.0	
					10/1975	.5		2.0							
20. Marana T.I. I 10 EB, MP 235 STA 429+85—I 10-4 (27)	7/1963	1950 (594)	Qs Younger Alluvial Deposit Surficial Valley Fill. Undiffertiated	Flat Valley Floor Irrigated Land Slow Infiltration	7/1963		.3	3.0				6.0	4.0	5.0	
21. Upper Deer Valley I 17 NB, MP 223 STA 1170+47—I 17-1 (56)	7/1964	1639 (500)	Qp Younger Alluvial Deposit. Flood Plan and Playas	Flat Valley Floor Desert Creosote & Salt Bush Slow Infiltration	7/1964	.5		3.0					4.0	22.0	
22. Agua Fria River I 17 NB, MP 243 STA 2215+—I 17-1 (59)	8/1964	2000 (610)	QTs Older Valley Fill	Foothills. Leading to Plateau Desert Palo Verde & Cacti Slow Infiltration	8/1964	.5		3.5					4.0	24.0	
23. Bellemont I 40 EB, MP 186.5 STA 1689+30—I40-3 (22)	10/1965	7139 (2176)	QTb Basalt, Cinders and Clay Mixed	Rolling. Ponderosa Pine Slow Infiltration	10/1965					2.0			4.0	9.0	6.0
24. Topock I 40 WB, MP .42 STA 23+85 I 40-1 (8)	6/1966	488 (149)	gm Granite Gneiss	Rolling Hills Desert Salt Bush Creosote Bush Slow Infiltration	6/1966	.5		3.5					4.0	9.0	

*1 INCH = .0254 METERS

TABLE I (Con't.)

SITE NUMBER AND NAME EXACT LOCATION AND PROJECT NUMBER	DATE BUILT	ELEV. FT. (METERS)	GEOLOGY	TOPOGRAPHY VEGETATION DRAINAGE	DATE BUILT	THICKNESS IN INCHES*									
						ACFC	SEAL COAT	AC	PCCP	BTB	BSB	CTB	AB	SM	SGS
25. Williams Field T.I. I 10 EB, MP 160 STA 788+27—I 10-3 (56)	12/1966	1209 (367)	Qfp Alluvial Fan and Rock Sediment	Flat Valley Floor Irrigated Land Slow Infiltration	12/1966	.5		3.5					4.0	15.0	
26. Sunset Point I 17 NB, MP 251.41 STA 2660—I 17-1 (55)	4/1967	3354 (1022)	QTb Basic Volcanic and Intrusive Rocks	Flat Plateau Chaparral Very Slow Infiltration	4/1967			5.0		3.0			2.0	17.0	6.0
27. Casa Grande T.I. I 10 EB, MP 196 STA 2684+08—I 10-3 (59)	3/1968	1486 (438)	Qs Younger Alluvial Deposit. Surficial Valley Fill. Undiffertiated	Flat Valley Floor Irrigated Land Slow Infiltration	3/1968	.5		5.5					4.0	18.0	
28. Woody Mtn. T.I. I 40 EB, MP 192.85 STA 2032— I 40-3 (32)	10/1968	7086 (2160)	Qb Basalt, Cinders and Clay Mixed	Rolling. Ponderosa Pine Slow Infiltration	10/1968				8.0			6.0			6.0
29. Cosnino I 40 EB, MP 206.42 STA 494+76 I 40-4 (69)	6/1969	6496 (1980)	Pk Kaibab Limestone	Rolling Hills Ponderosa Pine & Juniper Slow Infiltration	6/1969				8.0			6.0			5.0
30. Cherry Rd. T.I. I 17 NB, MP 275 STA 3884+16—I 17-2 (62)	10/1969	4442 (1354)	QTb Basic Volcanic and Intrusive Rocks	Rolling Hills Chaparral Very Slow Infiltration	10/1969			5.3		4.7			4.0	21.0	6.0
					7/1976	.5									
31. Cienga Creek SR 169, MP 14.58 STA 770+00—S 447 (2)	8/1971	4460 (1359)	QTb Basalt	Plateau cut by washes Oak and chaparral Very Slow Infiltration	8/1971	G & D									
32. Winslow Bypass I 40 EB, MP 257.78 STA 3215+00—I 40-4 (61)	9/1971	4861 (1482)	Qs River Sand	Flat Upper Plateau Short Grass Slow Infiltration		G & D									

*1 INCH = .0254 METERS

TABLE I (Con't.)

SITE NUMBER AND NAME EXACT LOCATION AND PROJECT NUMBER	DATE BUILT	ELEV. FT. (METERS)	GEOLOGY	TOPOGRAPHY VEGETATION DRAINAGE	DATE BUILT	THICKNESS IN INCHES*									
						ACFC	SEAL COAT	AC	PCCP	BTB	BSB	CTB	AB	SM	SGS
33. Alpine US 180, MP 425.5 STA 1220+00 EDA F051-2 (9)	9/1973	8061 (2458)	Ts Sandstone Silt & Gravel	Mountainous Ponderosa Pine Slow Infiltration	9/1973			6.8						7.0	
34. Lake Pleasant SR 74, MP 23.44 STA 1238+S 434 (3)	10/1973	1693 (516)	Qfp Alluvial Fan and Rock Sediment Caliche	Rolling Hills. Desert Creosote and Salt Bush Very Slow Infiltration	1973	G & D									
35. Kachina Blvd. I 17 SB, MP 334.57 STA 7086+50—I 17-2 (35)	7/1974	6780 (2067)	Pk Kaibab Limestone	Rolling Mogollon Rim Ponderosa Pine Slow Infiltration	7/1974				8.0			6.0			6.0
36. Flagstaff Airport T.I. I 17 SB, MP 337 STA 7215+80—I 17-2 (35)	7/1974	6958 (2121)	Pk Kaibab Limestone	Rolling Hills Ponderosa Pine Slow Infiltration	7/1974				8.0			6.0			6.0
37. Tonopah I 10 EB, MP 100.67 STA 5309+06—I 10-2 (13)	2/1975	1098 (335)	Qfp Alluvial Fan and Rock Sediment	Flat Valley Floor Desert Creosote & Salt Brush Slow Infiltration	2/1975	.5		125						5.0	

*1 INCH = .0254 METERS

SURFACING THICKNESS AND FREQUENCY

Considering all the layers above the subgrade to be structural layers, several trends can be observed.

TRENDS IN AC STRUCTURAL LAYERS

YEARS	AVERAGE TOTAL INCHES OF THICKNESS ABOVE SUBGRADE	AVERAGE AC THICKNESS INCHES	AVERAGE PERCENTAGE AC OF TOTAL
1940-1946	16	2.5	16
1955-1958	16	3.3	21
1960-1961	21	3.9	19
1963-1966	24	3.6	15
1967-1969	34	8.0	24
1973-1975	16	9.7	61
1 Inch = .0254 Meters			

From 1940 to 1958, AC new construction remained relatively constant. About 1958, the interstate program began in earnest. Initially, both base and surfacing courses increased slightly. In 1963, the AASHTO interim design guidelines were adopted. Although AC surface thickness increased dramatically from 1963 to 1969, base course thickness also increased, thus keeping the ratio of AC to total about 20 percent. Since, 1970, base thickness has declined rapidly to the point where the newer sections are virtually full depth, with very thick (more than 10 inches of AC) AC surfacings becoming quite common.

Concrete new construction from 1968 to 1974 remained constant with a section of 8 inches PCCP, 6 inches CTB and 6 inches SGS being virtually a standard. Since 1976, concrete sections became thicker with 11 or more inches of PCCP with no CTB or SGS.

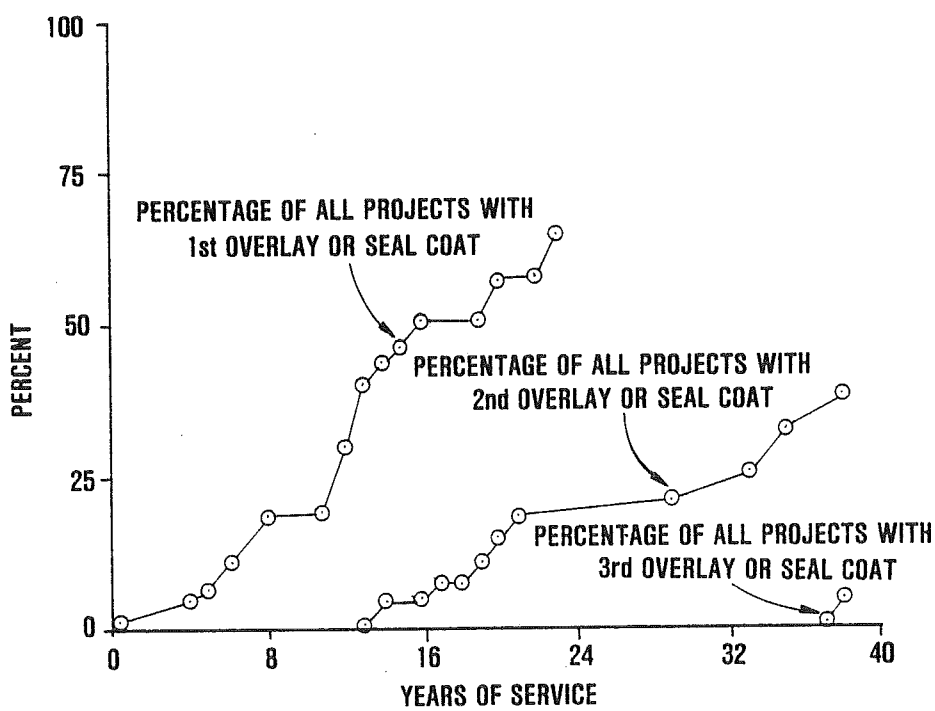
Overlay frequency for AC pavements is described below:

FREQUENCY OF OVERLAYS

YEARS OF ORIGINAL CONSTRUCTION	NO. OF PROJ.	NUMBER OF OVERLAYS	NUMBER OF SEAL COATS	TOTAL NUMBER OF OVERLAYS & SEAL COATS	AVERAGE CUMULATIVE THICKNESS OF OVERLAY & SEAL COAT
1936-1955	6	8	6	14	3.8
1956-1960	6	7	2	9	3.3
1960-1963	5	3	0	3	6.0
1964-1966	5	1	0	1	2.5
1967-1975	5	1	0	1	.5
1 Inch = .0254 Meters					

The figure below describes the historical process of overlaying or seal coating.

Generally, one-half of all AC projects were overlaid or seal coated by the 16th year. Of these projects, about one-half had received a second overlay or seal coat by the 33rd year.



AS CONSTRUCTED MATERIALS

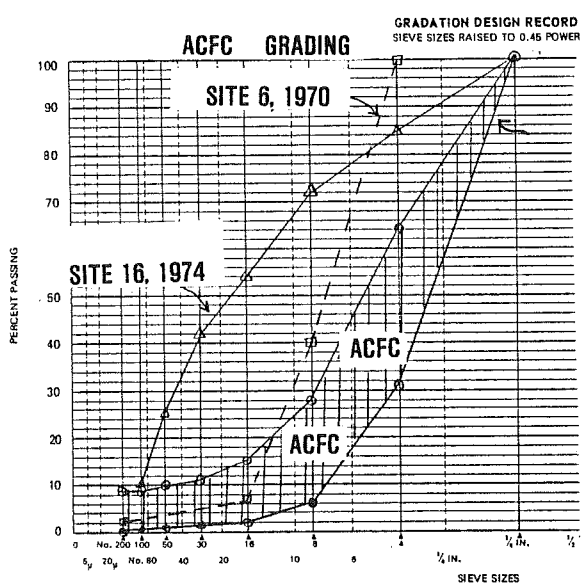
Old construction files were examined to find a description of the materials used in each layer. Appendix A contains the following information by material type in chronological order, cross-referenced to the site number. The table below is a summary of what is in Appendix A.

APPENDIX A CONTENT

MATERIAL TYPE	BY TABLES A-1 THRU A-24			
	GRADING	ASPHALT CONTENT AND PROPERTIES	COMPACTION DATA PI AND SAND EQUIVALENT	% CEMENT SLUMP % AIR 28 DAY STRENGTH
ACFC	A-1	A-2	--	--
Seal Coat	A-3	A-4	--	--
AC	A-5	A-6	A-7	--
PCCP	A-8	--	--	A-9
BTB	A-10	A-11	A-12	--
CTB	A-13	--	A-14	--
AB	A-15	--	A-16	--
SM	A-17	--	A-18	--
SGS	A-17	--	A-18	--
SUBGRADE	A-19	--	A-20	--

Information on these tables has been condensed into eleven graphs on Figure 2. From this figure, it can be seen that a wide variety of materials has been incorporated into the highway structure. Generally, surfacing materials such as ACFC or seal coats tend to be uniformly graded chips, whereas AC, BTB, PCCP, CTB and AB are well graded. All these layers are made up of very clean material with the PI generally non-plastic (NP). Select and subgrade seal are also well graded, but have PI's up to 9. Subgrades varied from somewhat well graded to very fine. Virtually all subgrades had more than 68 percent passing the #4 sieve. PI's ranged from NP to 52.

FIGURE 2



ACFC 1961-1977

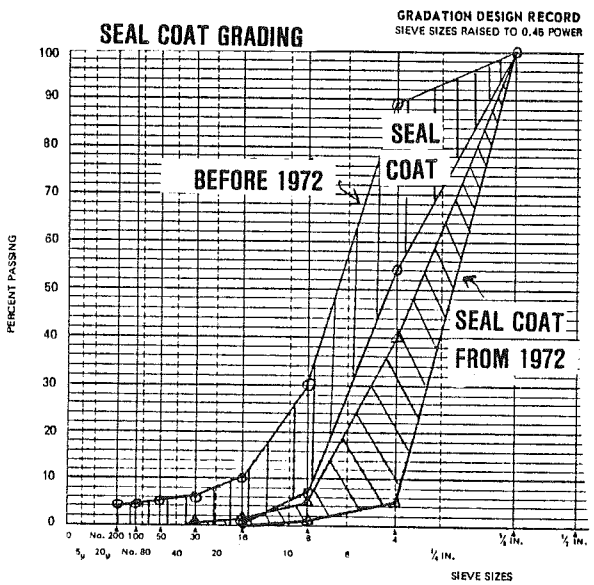
% ASPHALT 4.2-7.2

ASPHALT 85/100
AR 4000
AR 2000

% ASPHALT FOR CINDERS

7, 1964
10,1971 } 11.0

FIGURE 2



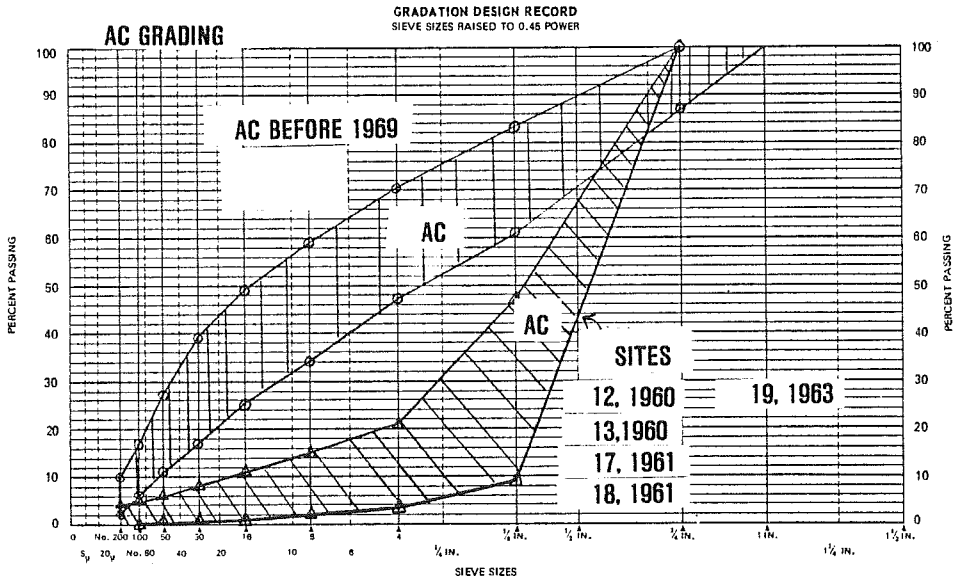
SEAL COATS 1956-1975

EMULSIONS GRADE B
RS-2
RS-K
SS-1H
CRS-2H

ASPHALT AR 2000

EMULSION APPLICATION
.35 - .40 GAL/YD²

CHIP APPLICATION
18-25 LBS/YD²



1955-1968
% ASPHALT 3.5-7.9

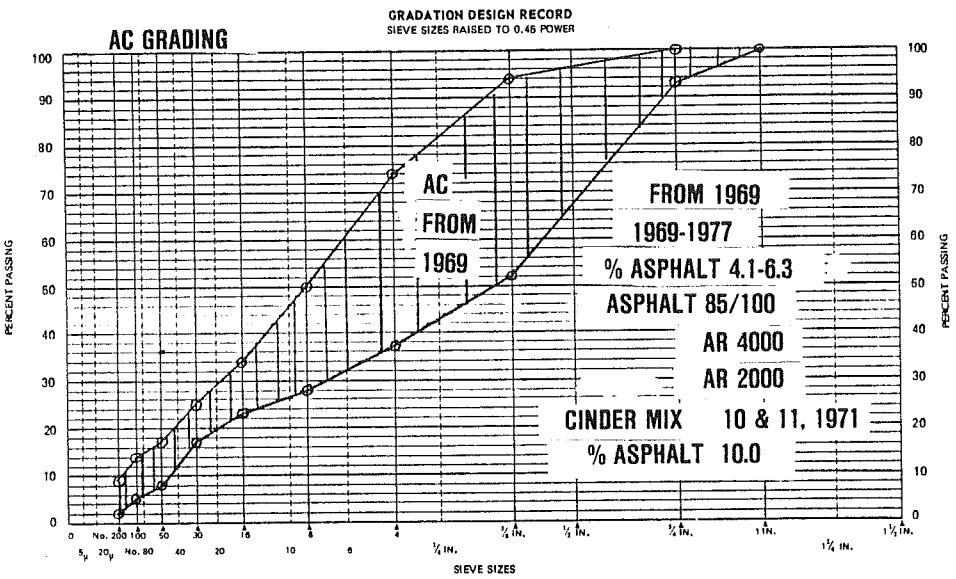
ASPHALT SC-6, 85/100
120/150, 200/3000

CINDER MIX
% ASPHALT 11.3

SITES 12, 13, 1960
17 & 18, 1961
19 , 1963

% ASPHALT 3.1-3.9
ASPHALT 85/100

FIGURE 2


GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
AC GRADING
AC
FROM
1969
FROM 1969
1969-1977
% ASPHALT 4.1-6.3
ASPHALT 85/100
AR 4000
AR 2000
CINDER MIX 10 & 11, 1971
% ASPHALT 10.0
PERCENT PASSING
SIEVE SIZES

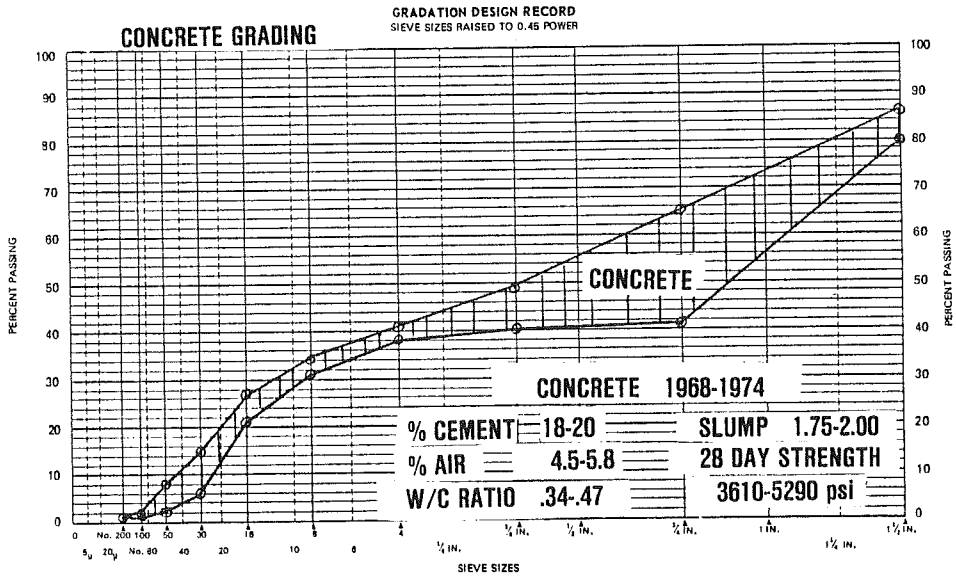
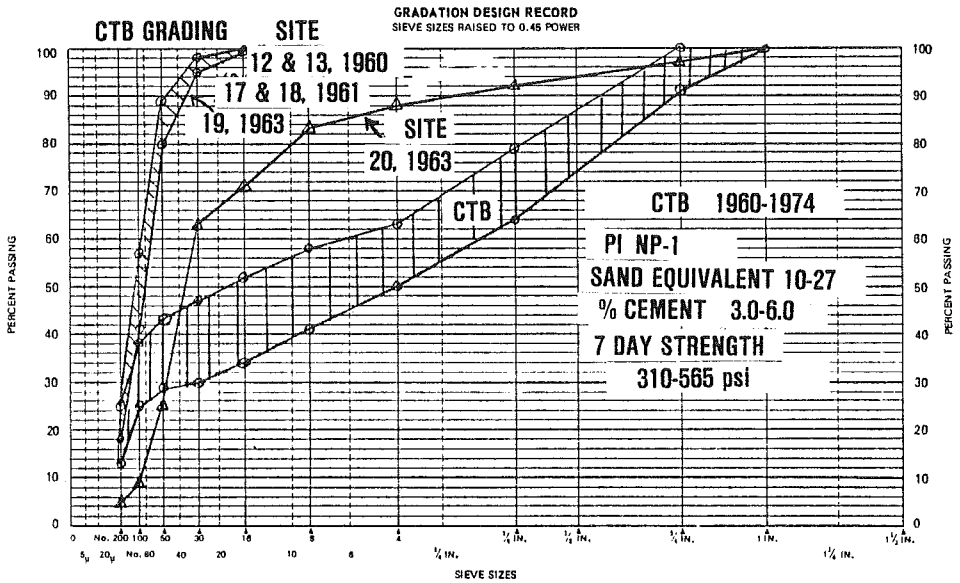

GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
CONCRETE GRADING
CONCRETE
CONCRETE 1968-1974
% CEMENT 18-20
SLUMP 1.75-2.00
% AIR 4.5-5.8
28 DAY STRENGTH
W/C RATIO .34-.47
3610-5290 psi
PERCENT PASSING
SIEVE SIZES

FIGURE 2


GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
CTB GRADING SITE
12 & 13, 1960
17 & 18, 1961
19, 1963
SITE
20, 1963
CTB
CTB 1960-1974
PI NP-1
SAND EQUIVALENT 10-27
% CEMENT 3.0-6.0
7 DAY STRENGTH
310-565 psi
PERCENT PASSING
SIEVE SIZES

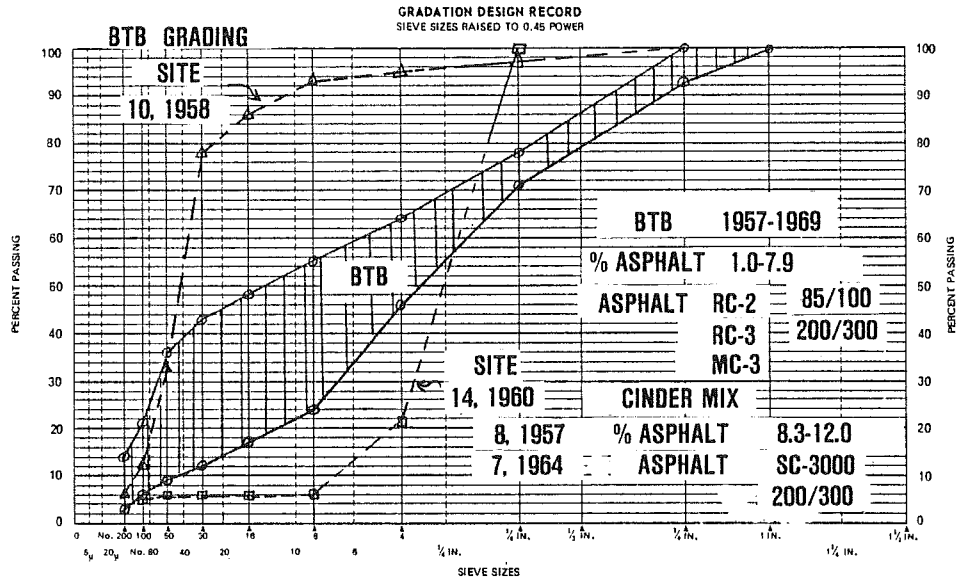
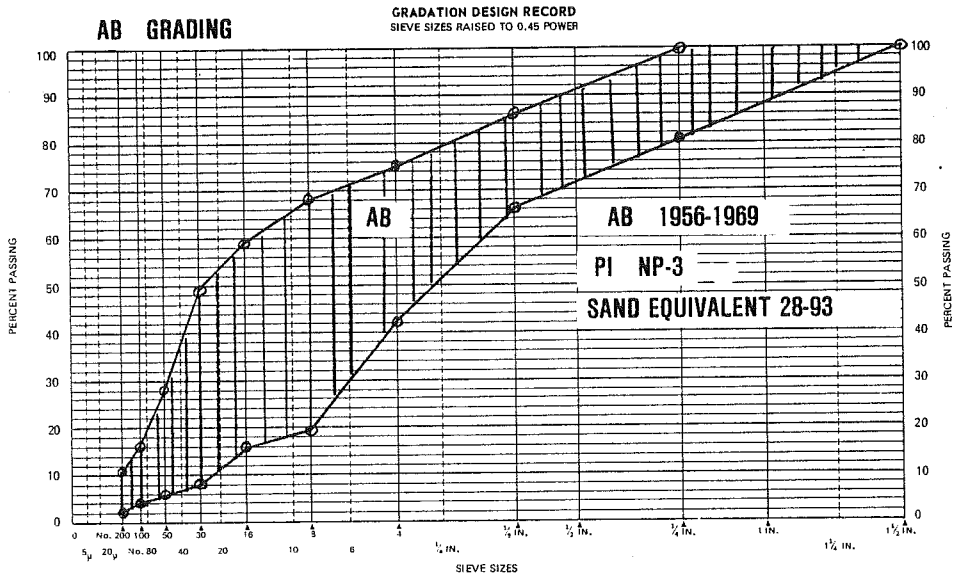

GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
BTB GRADING
SITE
10, 1958
BTB
BTB 1957-1969
% ASPHALT 1.0-7.9
ASPHALT RC-2 85/100
RC-3 200/300
MC-3
SITE
14, 1960
8, 1957
7, 1964
CINDER MIX
% ASPHALT 8.3-12.0
ASPHALT SC-3000
200/300
PERCENT PASSING
SIEVE SIZES

FIGURE 2


AB GRADING
GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
AB
AB 1956-1969
PI NP-3
SAND EQUIVALENT 28-93
PERCENT PASSING
SIEVE SIZES

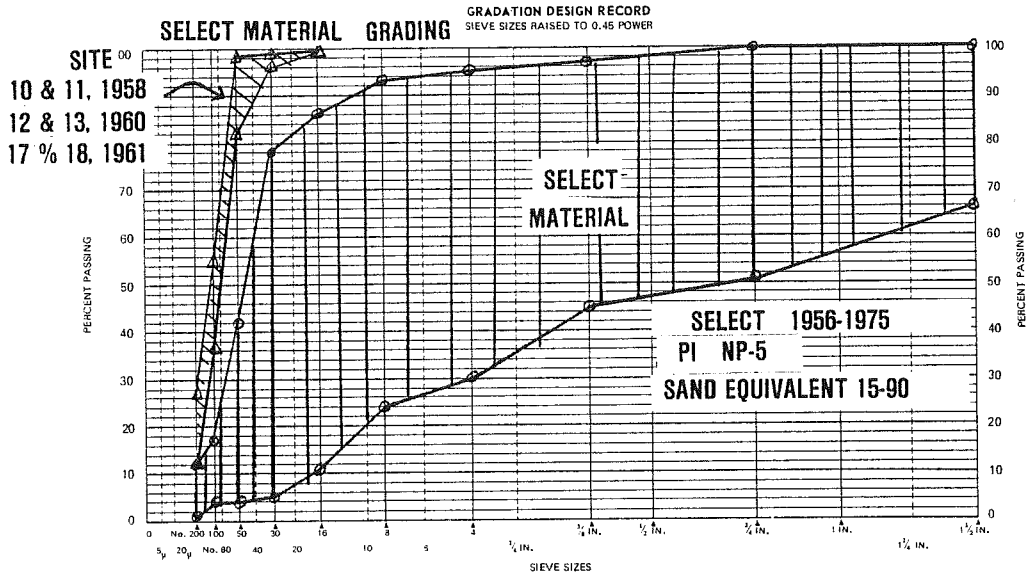
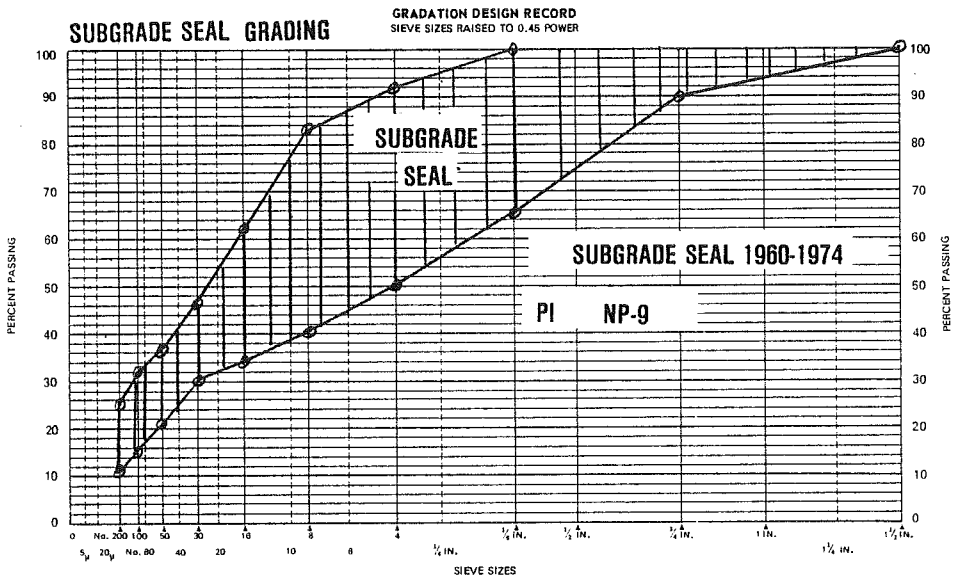
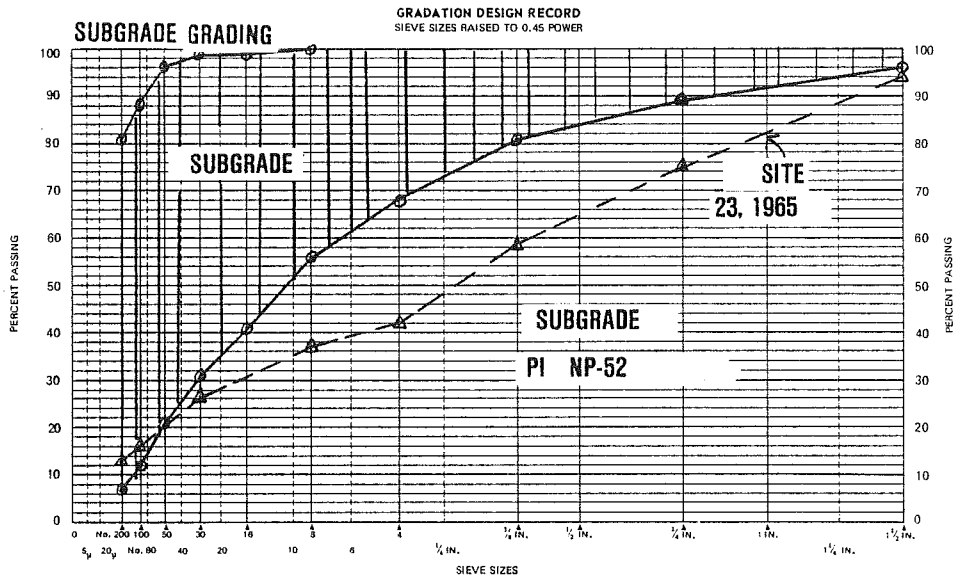

SELECT MATERIAL GRADING
GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
SITE 00
10 & 11, 1958
12 & 13, 1960
17 % 18, 1961
SELECT MATERIAL
SELECT 1956-1975
PI NP-5
SAND EQUIVALENT 15-90
PERCENT PASSING
SIEVE SIZES

FIGURE 2


SUBGRADE SEAL GRADING
GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
SUBGRADE
SEAL
SUBGRADE SEAL 1960-1974
PI NP-9
PERCENT PASSING
SIEVE SIZES


SUBGRADE GRADING
GRADATION DESIGN RECORD
SIEVE SIZES RAISED TO 0.45 POWER
SUBGRADE
SITE
23, 1965
SUBGRADE
PI NP-52
PERCENT PASSING
SIEVE SIZES

GEOMETRICS

Figure 3 contains a series of drawings depicting the cross sectional view descriptive of each site's geometry - cut, fill, urban or side hill cut.

FIGURE 3

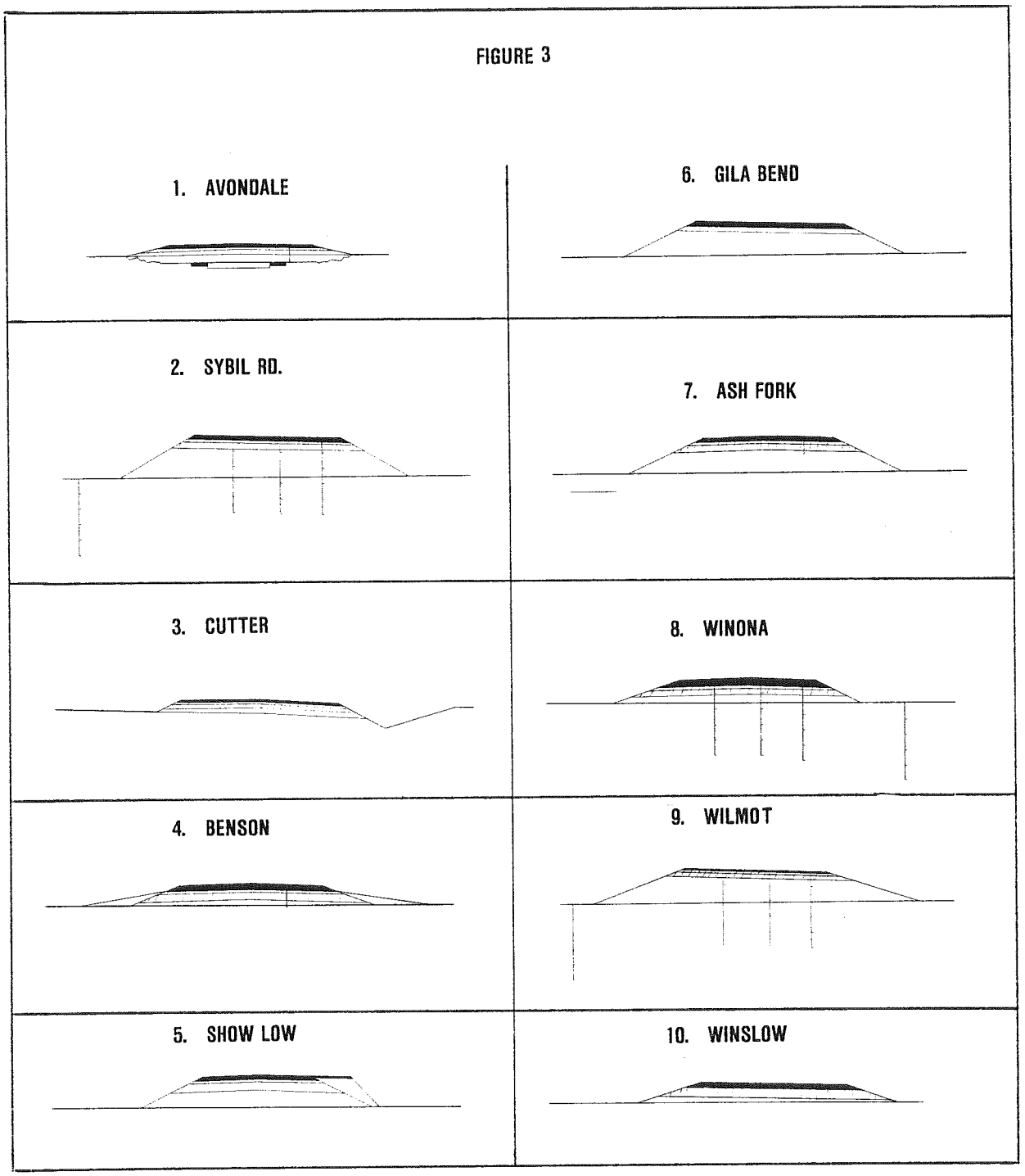
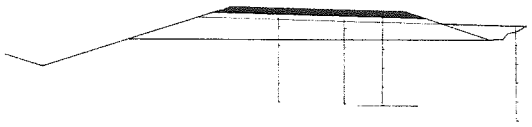
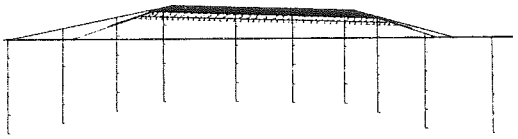


FIGURE 3

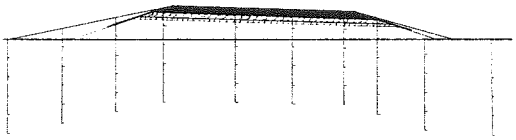
11. MINNETONKA



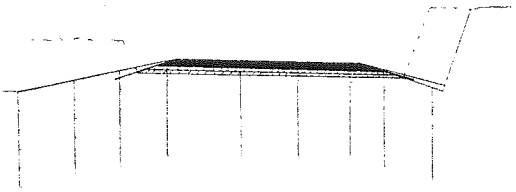
17. CRAZY CREEK 1



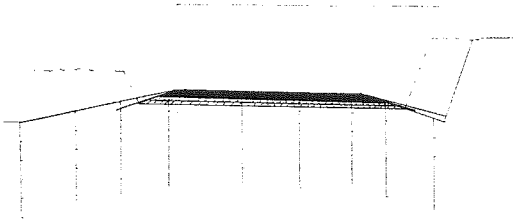
12. DEAD RIVER 1



18. CRAZY CREEK 2

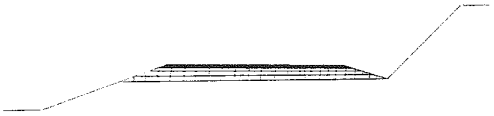


13. DEAD RIVER 2

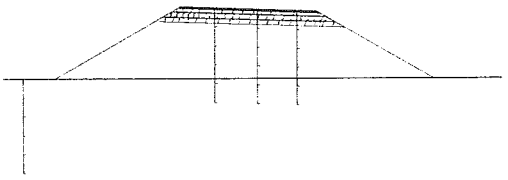


19. LUPTON

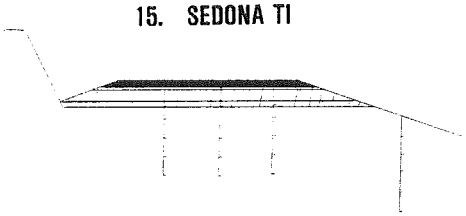
14. FLAGSTAFF NB



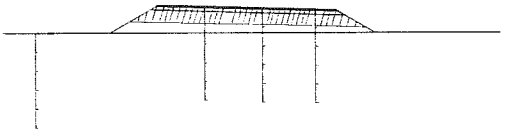
20. MARANA T.I.



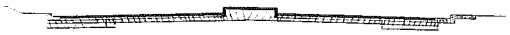
15. SEDONA TI



21. UPPER DEER VALLEY



16. TEMPE



22. AGUA FRIA RIVER

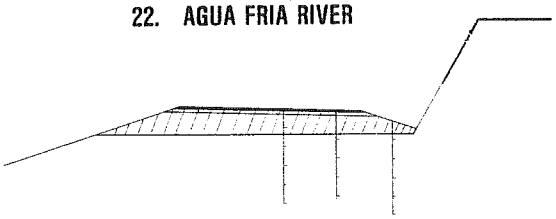
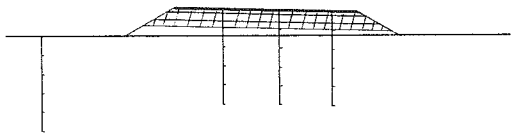


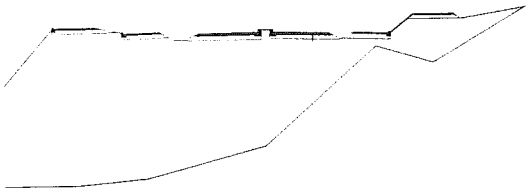
FIGURE 3

23. BELLEMONT

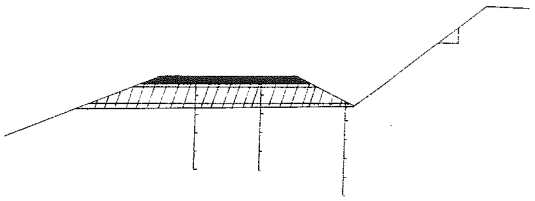


29. COSNINO

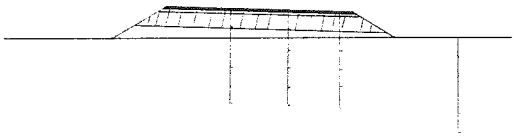
24. TOPOCK



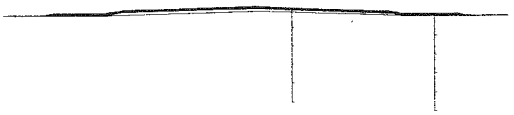
30. CHERRY RD. T.I.



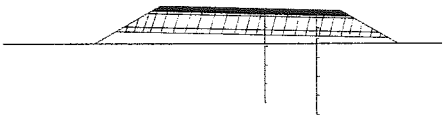
25. WILLIAMS FIELD T.I.



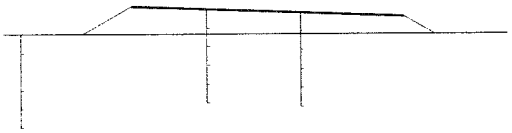
31. CIENAGA CREEK



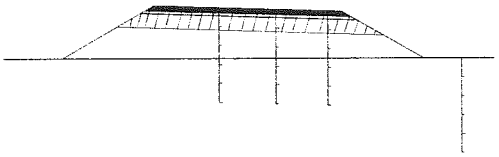
26. SUNSET POINT



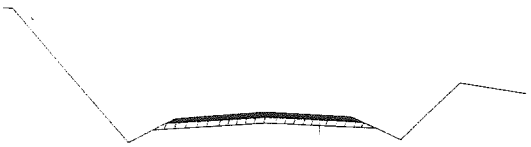
32. WINSLOW BYPASS



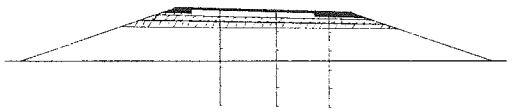
27. CASA GRANDE T.I.



33. ALPINE



28. WOODY MTN. T.I.



34. LAKE PLEASANT

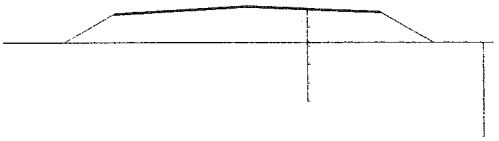
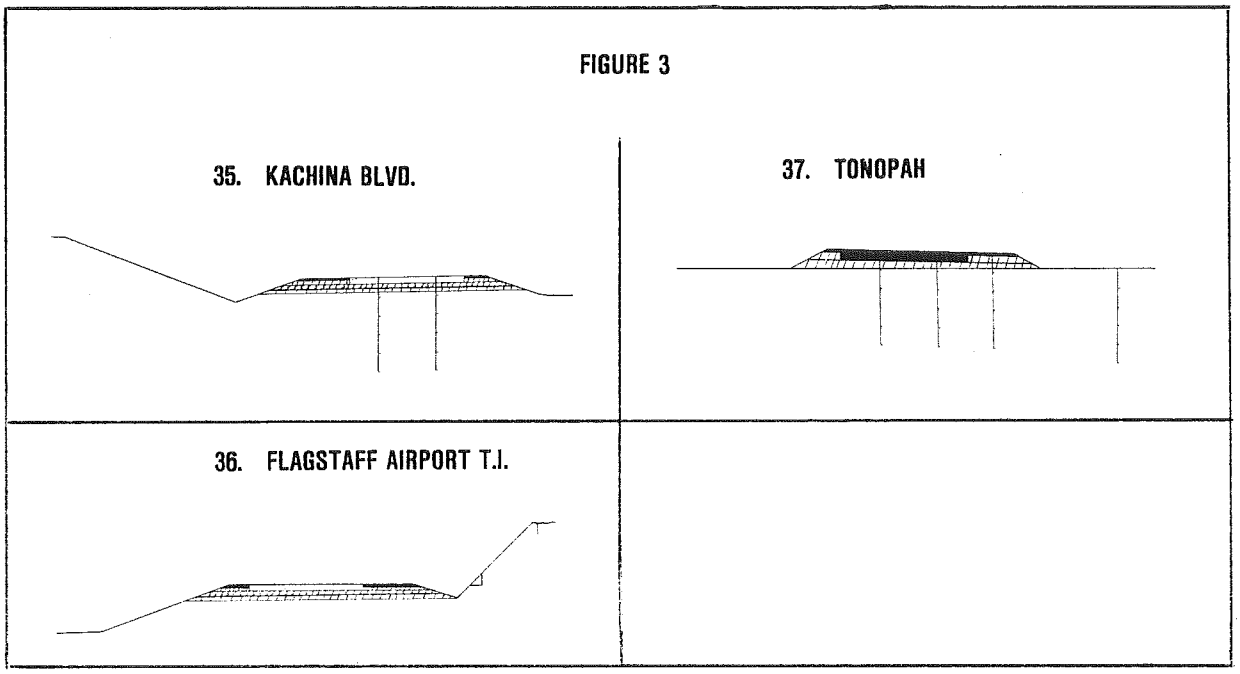


FIGURE 3



TRAFFIC

The 18 kip single axle equivalent one-way traffic for each site was determined. Many sites have very similar traffic loadings, due to their being on the same route within close proximity to one another and the lack of virtually any additional on or off traffic. Table 2 gives the current 1978 ADT, percent trucks and 18 kip loading for 1978. In addition, the cumulative 18 kip loading between activities is given for each site. As an example, for site #1 the 18 kip cumulative traffic from the first event (1936) to the second event (12/1956) is given in the event #1 column. Event #2 represents the 12/1956 to 8/1974 cumulative 18 kip traffic and the event #3 column represents the cumulative 18 kip traffic from 8/1974 to 3/1979. The cumulative traffic was derived by developing a master curve for yearly equivalent single axle 18 kip loads as shown below on Figure 4.

By knowing the 18 kip traffic in 1978, it was possible to apply the derived factor for any year and calculate the 18 kip loading for that year. In this way, the cumulative traffic could be calculated for any time period. Since 1948, the 18 kip loading has doubled about every eight years.

FIGURE 4

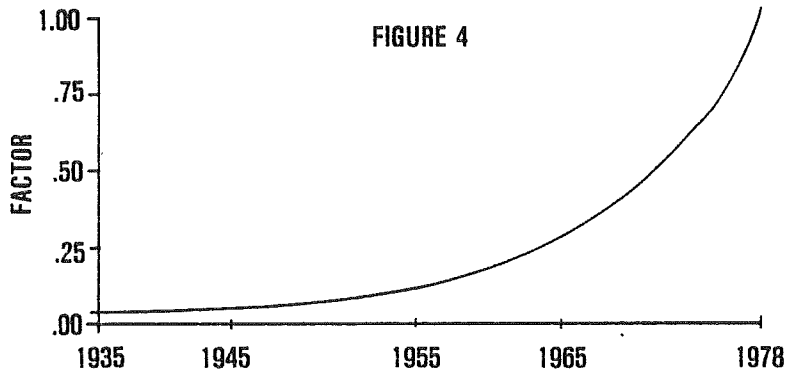


TABLE 2

TRAFFIC

1978; ONE WAY

SITE NO.	ADT	% TRUCKS	18 kips	CUMULATIVE 18 kip EVENT #1-2	EVENT #2-3	EVENT #3-4	EVENT #4-5
1	8,000	45.0	333,000	555,000	2,282,000	1,082,000	
2	4,562	35.4	234,000	505,000	1,950,000	606,000	
3	1,458	47.0	29,000	96,000	61,000	65,000	96,000
4	6,032	34.3	245,000	649,000	1,243,000	635,000	
5	835	50.0	18,000	140,000	34,000	47,000	
6	4,816	41.7	186,000	803,000	185,000	492,000	333,000
7	3,857	37.4	272,000	488,000	480,000	1,749,000	
8	5,777	33.8	288,000	917,000	1,423,000	288,000	
9	7,915	35.3	353,000	3,042,000			
10	5,817	35.3	308,000	1,445,000	582,000	767,000	
11	5,817	35.3	308,000	1,445,000	582,000	767,000	
12	5,057	34.2	286,000	1,662,000	950,000		
13	5,057	34.2	286,000	1,662,000	950,000		
14	3,680	33.8	139,000	204,000	545,000	462,000	
15	4,125	33.8	156,000	1,400,000			
16	16,500	33.0	247,000	1,365,000	820,000		
17	5,058	34.2	286,000	2,002,000	512,000		
18	5,058	34.2	286,000	2,002,000	512,000		
19	5,058	34.2	286,000	1,897,000	512,000		
20	8,525	34.8	395,000	3,227,000			
21	6,688	30.1	187,000	1,473,000			
22	5,760	30.1	159,000	1,252,000			
23	6,510	36.4	287,000	2,077,000			
24	3,258	39.5	230,000	1,666,000			
25	10,570	35.3	417,000	2,858,000			
26	5,760	30.1	159,000	1,090,000			
27	5,760	32.0	240,000	1,849,000			
28	5,410	36.4	287,000	1,834,000	287,000		
29	5,778	33.8	288,000	1,840,000			
30	4,630	33.8	200,000	924,000	372,000		
31	175	36.0	3,000	3,000			
32	0	0	0				
33	578	50.0	43,000	172,000			
34	0	0	0				
35	3,680	33.8	139,000	461,000			
36	3,680	33.8	139,000	461,000			
37	4,335	26.8	186,000	483,000			

FIELD SAMPLE TEST RESULTS

During the course of developing sites and conducting plate bearing tests, samples of the in-place materials were taken and tested in a manner consistent with previously reported construction test results in Appendix A. Accordingly, Appendix B contains the test results of field samples as shown below.

This set of data was developed to help understand the meaning of moisture, deflection and performance measurements; however, by knowing the materials properties at the time of construction and at some time in the future, it is possible to test for the influence of time on the materials.

APPENDIX B CONTENT

MATERIAL TYPE	BY TABLES B-1 THROUGH B-14		
	GRADING	ASPHALT CONTENT AND PROPERTIES	COMPACTION DATA PI AND SAND EQUIVALENT AND R VALUE
AC	--	B-1	B-2
BTB	--	B-3	B-4
CTB	--	--	B-5
AB	B-6	--	B-7 & 8
SM	B-9	--	B-10 & 11
SGS	B-9	--	B-10 & 11
SUBGRADE	B-12	--	B-13 & 14

ASPHALT AGING

Due to asphalts biodegradable nature, it ages and changes properties with continued exposure to sunlight, heat and oxygen. By knowing the asphalts penetration at 77°F before construction (Table A-6) and the penetration at some future time (Table B-1), it was possible to derive the ratio of aged to unaged penetration and plot this relationship on Figure 5.

As can be seen, the penetration falls to one-half its original value in about one year, and by two years it is one-fourth its original value. From two years on, the rate of aging slows, so that it takes about eight years to reach one-eighth of the original value. A similar aging curve was derived for the absolute viscosity at 140° F, as shown on Figure 6. For viscosity, the aging rate is much faster than for penetration because the viscosity is free to increase without limit, whereas penetration is bound to a zero penetration. With this rapid aging asphaltic concrete pavements in Arizona would tend to become very stiff.

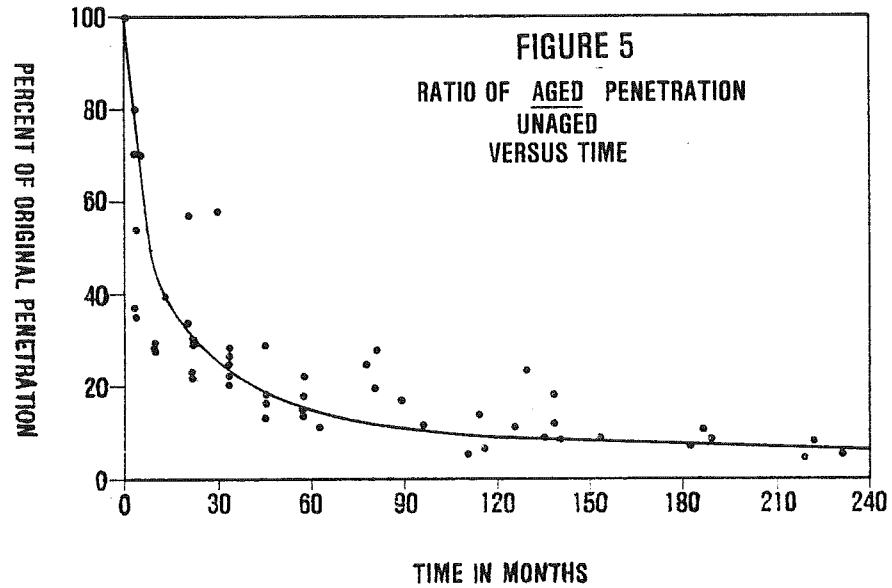
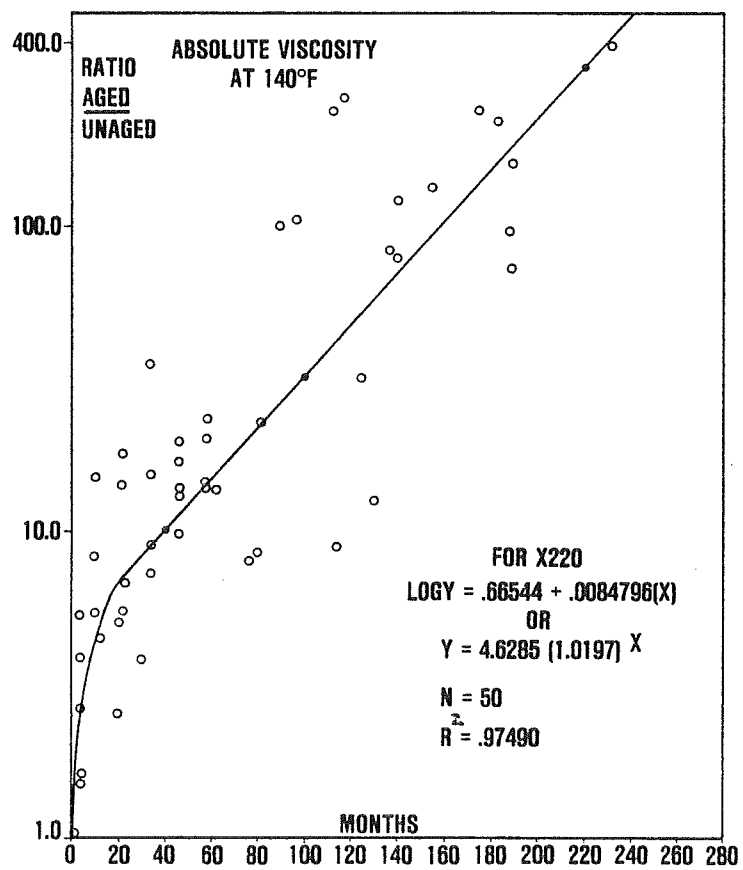


FIGURE 6

TIME VERSUS RATE OF CHANGE

OF VISCOSITY AT 140°F

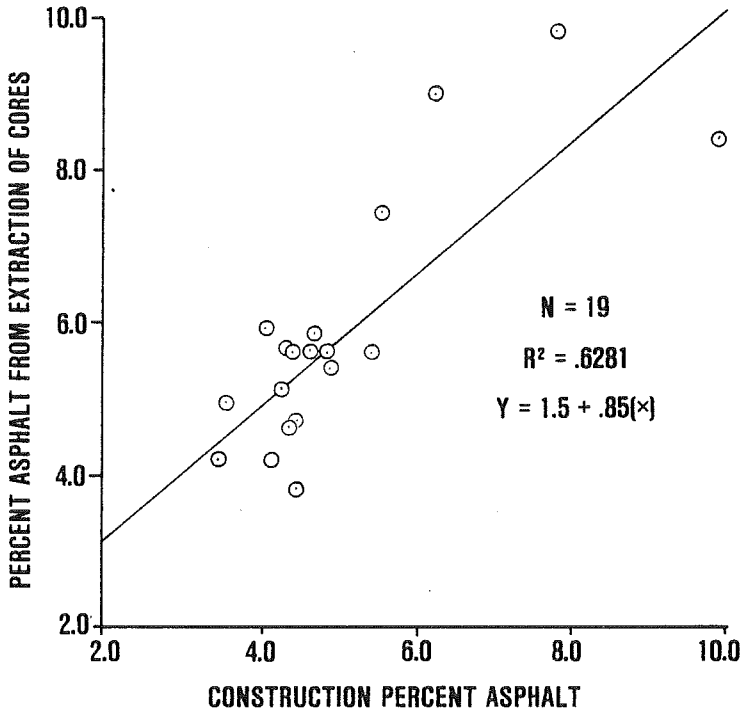


ASPHALT CONTENT

By knowing the asphalt content at construction (Table A-7) and the field asphalt (Table B-2), it was possible to determine the relationship below (Figure 7).

The extracted cores tended to contain more asphalt than at construction. This could be due to either the extraction test or applications of tack coat and flush coats, which have gradually increased the total asphalt content.

FIGURE 7



ASPHALTIC CONCRETE DENSITY, VOIDS AND COMPACTION

In reviewing the construction files, it was very difficult to obtain both the field density and the maximum theoretical density (Rice method). However, laboratory Hveem densities (T.I. compactor) for most project mixes was available. As an estimate, the T.I. densities were plotted versus the construction field density (Figure 8).

As can be seen, the field density is generally four pounds per cubic foot (64 kg/m) less than the T.I. compactor density. Applying this relationship to Table A-7, it was possible to calculate the most likely field density, as indicated by starred (*) values. Likewise, it was possible to estimate the maximum theoretical densities (MTD) by plotting the measured MTD from Table B-2 versus the T.I. compactor density (Table A-7) as shown in Figure 9.

FIGURE 8

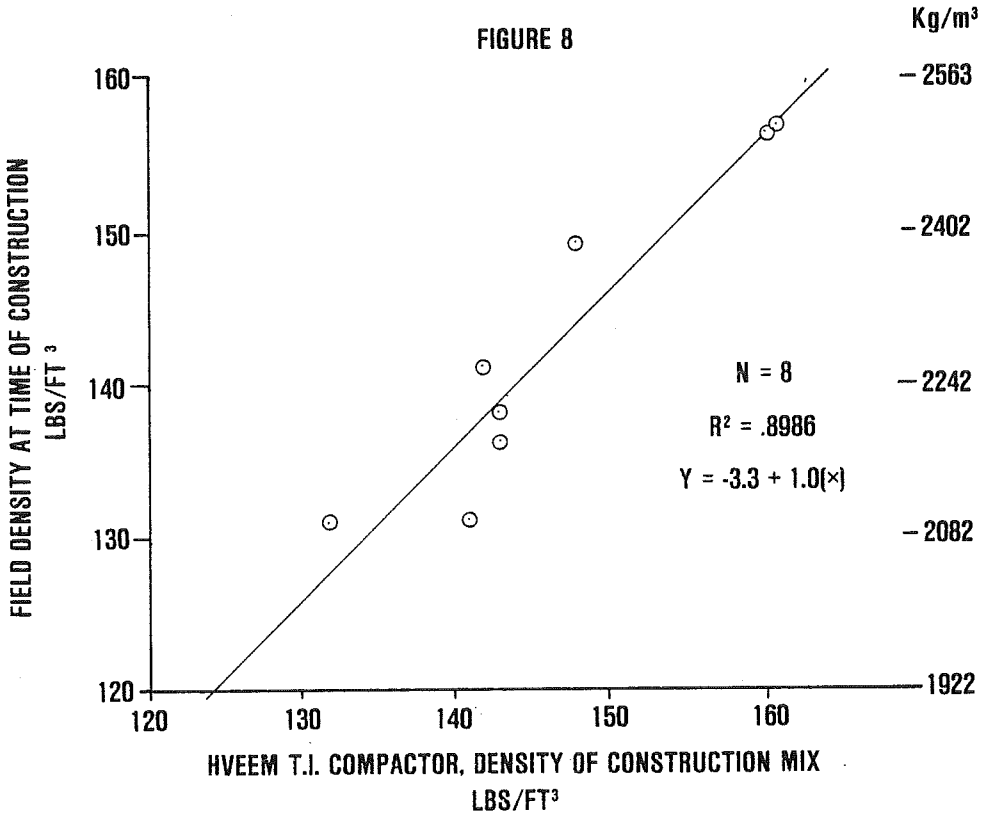
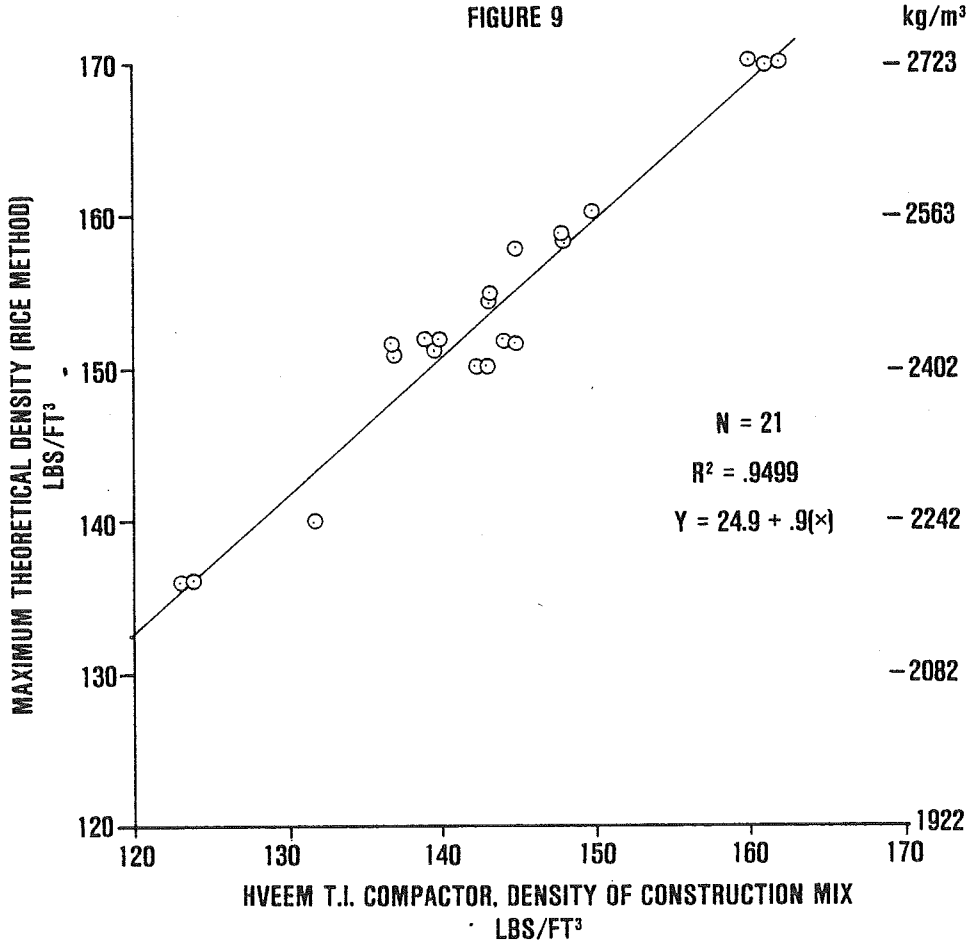


FIGURE 9



Generally the MTD is ten (160) to twelve (192) pounds per cubic foot (kg/m^3) heavier than the T.I. compactor density. By applying the Figure 9 relationship, it was possible to calculate the expected MTD for all projects with a T.I. compaction density. Figure 10 shows the density of AC after traffic has compacted it compared to the construction density.

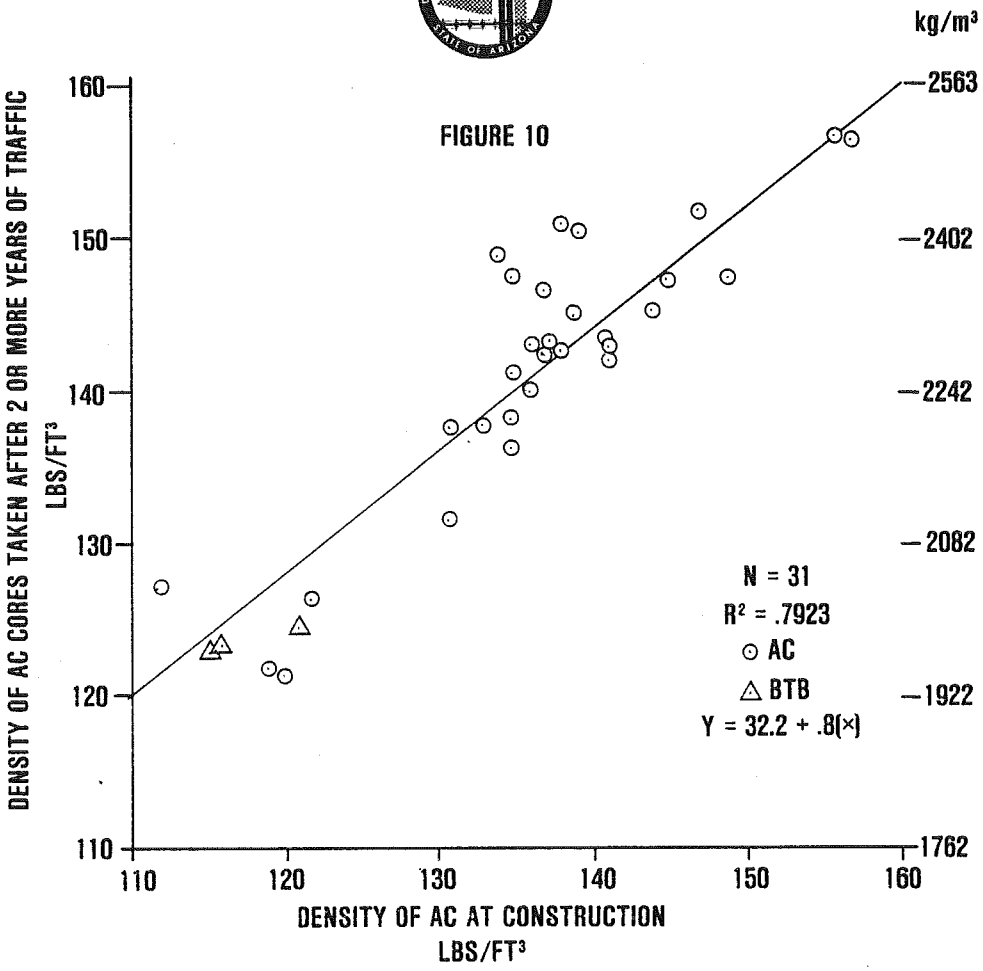
The density of the AC after two or more years of traffic increases dramatically for those very low density mixes, but very little for high density mixes. The explanation for this is that low density mixes are cinder mixes. This type of aggregate is a very rough surfaced material, which does not readily compact. Higher density mixes are combinations of virtually all crushed aggregate and sand. These mixes can be readily compacted to a very high density at time of construction due to the aggregate interlock. Additional densification of these high density mixes is virtually impossible, since the aggregate would begin to break down thus reducing the density.

Air voids are an important consideration to the design of AC mixes. Beginning in 1969, compaction controls were placed on AC so that at the time of construction the compaction would be 92 percent with respect to the MTD. In this way air voids would be approximately eight percent at time of construction. After traffic, additional densification would take place, thus lowering the air voids to a design value of 4-6 percent. Of the 15 projects built since 1969, seven of them were constructed with less than eight percent voids and these projects have attained the 4-6 percent air voids design target under traffic. Table 3 shows the degree of success in attaining the 4-6 design value.

TABLE 3

NUMBER OF PROJECTS AT VARIOUS AIR VOID LEVELS

	1956-1968 NO AC COMPACTION CONTROL		1969-1977 AC COMPACTION CONTROL	
	VOIDS AT CONSTRUCTION	VOIDS AFTER MORE THAN 2 YEARS	VOIDS AT CONSTRUCTION	VOIDS AFTER MORE THAN 2 YEARS
0-2.0	0	4	0	2
2.1-4.0	0	1	0	0
4.1-6.0	0	2	2	3
6.1-8.0	1	6	6	5
8.1-10.0	11	6	3	2
>10.0	7	0	4	2



Evidently mixes built since 1969 have not experienced the degree of compaction gain as those built before 1969. This is probably a direct result of the compaction control specification. With projects now required to achieve at least 92 percent compaction at time of construction, the level of air voids at construction has been reduced. In addition, contractors have found new methods of achieving higher degrees of compaction with fewer roller passes by using big rollers and vibratory rollers. This emphasis on compaction at construction has led to an AC product in the field which does not generally experience much additional traffic compaction. More importantly, the 4-6 percent design air void level has been achieved only about 20 percent of the time.

Interestingly, the voids filled at time of construction had gradually tended to increase as shown below on Table 4.

TABLE 4

VOIDS FILLED WITH ASPHALT

	AVERAGE	HIGH	LOW
1955-1961	46.2	55.0	39.4
1962-1969	53.4	64.5	42.0
1970-1975	57.2	64.1	48.7

BITUMINOUS TREATED BASE (BTB) DENSITY, VOIDS AND COMPACTION

By using Figures 8 and 9, it was possible to estimate the field density at the time of construction and the MTD value for the BTB materials shown in Table A-12. The MTD was calculated according to the Figure 9 equation; however, 20 lbs/ft^3 (320 kg/m^3) were added to the calculated value to give a value more closely in line with site 23/1965 and the value obtained from the specific gravity of the aggregate and asphalt. The four BTB density tests shown on Figure 10 indicate that the long term densification of BTB is very similar to AC. Generally, the BTB mixes had air voids of about 22 percent at construction, and about 19 percent after two or more years of traffic.

CEMENT TREATED BASE

Reviewing Tables A-14 and B-5, it can be seen that the field dry density at time of construction averaged 98 percent of the proctor maximum density. Using this value, it was possible to estimate the four missing field densities. After two or more years under traffic, the field dry densities average about 11 lbs/ft^3 (176 kg/m^3) heavier, or about 106 percent of the proctor maximum density. Only site 28, Woody Mountain, showed considerable decompaction. Field moisture at time of construction averaged 94 percent of the proctor optimum moisture, and this value was used to estimate the remaining four moistures. After many years in the field, CTB moisutres averaged 119 percent of the field moistures at construction, or about 2 percent wetter by weight.

AGGREGATE BASE (AB), SELECT MATERIAL (SM), SUBGRADE SEAL (SGS) AND SUBGRADE

Tables A-16, A-18, and A-20 indicated many missing proctor and field density-moisture data at time of construction. As an estimating process, those sites where actual proctor construction data could be found were used to verify the hypothesis that field samples taken at a later date could represent the proctor density and moisture at the time of construction. Figure 11 shows the proctor dry density from sample locations taken off of Tables B-8, B-11 and B-14 compared to the actual constructin proctor dry density.

As can be seen, the dashed regression line is very close to the line of equality, thus it is reasonable to assume that the sampled proctor dry density can be used to estimate the construction value. This was done, and the starred (*) values in Tables A-16, A-18 and A-20 represent the estimated construction values. A similar approach was taken for the proctor optimum moisture, which gave the following results:

$N=18$
$R^2=.5989$

OPTIMUM % MOISTURE = 1.15 + .87 (SAMPLED OPTIMUM % MOISTURE) @ CONSTRUCTION

Although the correlation is not as good, the slope of the regression line places it very close to the line of equality; thus, the sampled proctor optimum percent moisture was used to estimate the proctor percent moisture at time of construction as indicated by the starred values (*) in Tables A-16, A-18 and A-20. To estimate the field density at time of construction, the construction proctor dry density was plotted on Figure 12 versus the field dry density for AB, SM and SGS. As can be seen, field densities were about two #/ft^3 (32 kg/m^3) heavier than the proctor, or about 101 to 103 percent compaction. This was expected since historically construction specifications have called for AB, SM and SGS acceptance compaction of 100 percent or more. A similar approach was taken for subgrade densities. Results of this analysis indicated the following:

N=10
R^2 =.8733

SUBGRADE FIELD DRY DENSITY @ CONSTRUCTION = 1.71 + .96 (SUBGRADE PROCTOR MAXIMUM DRY DENSITY @ CONSTRUCTION)

This corresponds to a field density about three #/ft^3 (48 kg/m^3) less than the proctor or about a 97 percent compaction. Again this agrees very closely to the subgrade compaction specification of 95 percent or more of the proctor maximum density.

FIGURE 11

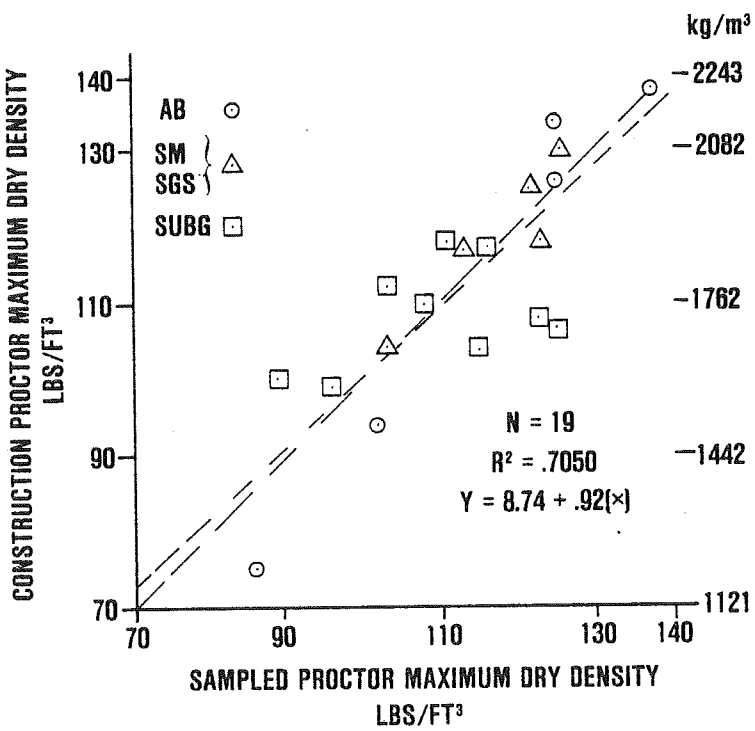
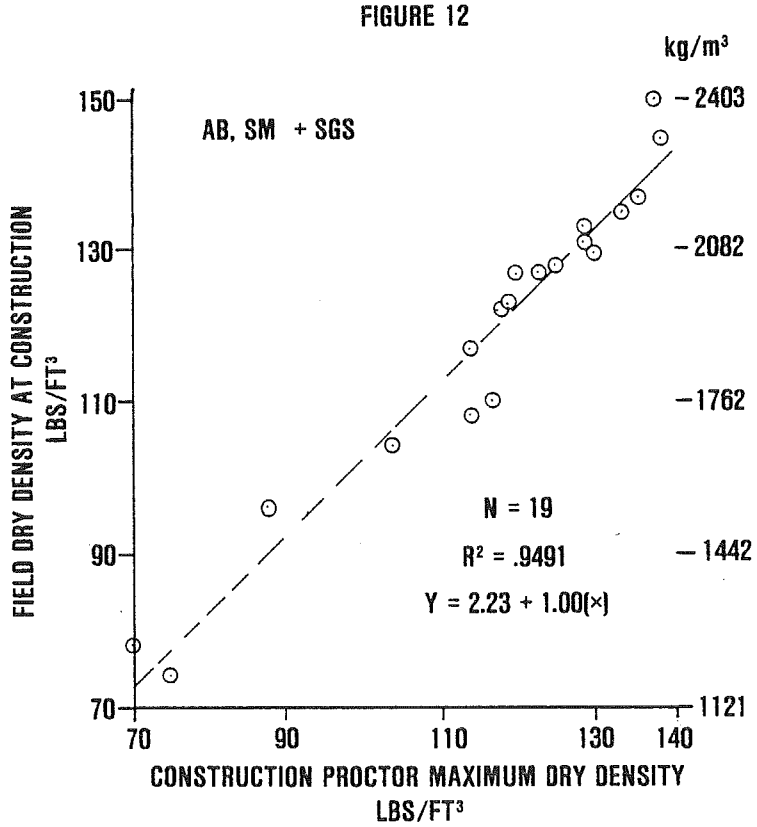


FIGURE 12



A similar approach was taken to estimate the field moisture at time of construction for the base and subgrade materials. Results of this study gave the following:

FIELD PERCENT MOISTURE

@ CONSTRUCTION = A + B*(PROCTOR OPTIMUM PERCENT MOISTURE @ CONSTRUCTION)

BASE	SUBGRADE
AB-SM-SGS	
N = 19	N = 10
R^2 = .7916	R^2 = .1716
A = -3.3	A = .5
B = 1.0	B = .16

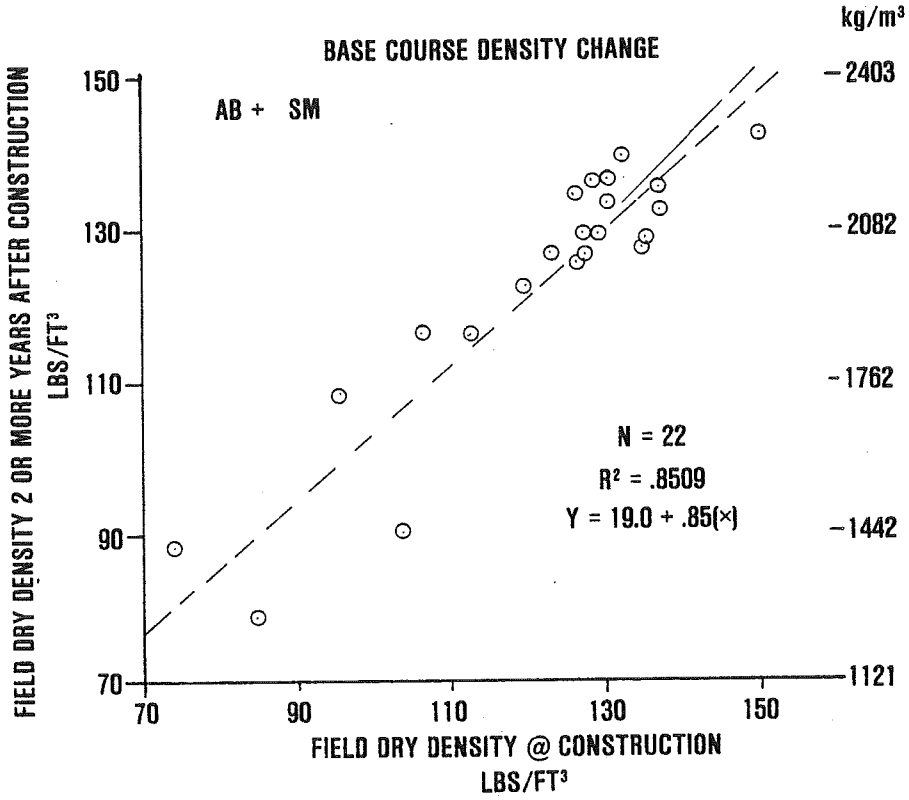
For base materials, the field moisture at construction was generally about 3.3 percent dryer than the proctor optimum moisture. This relationship was surprising since there is no specification control on base course moisture content. The subgrade estimate has considerable scatter; however, the equation does predict field moistures of 1.5 percent to over 6.0 percent dryer than optimum. Even though the correlation is poor, the equation was used to estimate field moisture since no other estimating process was available.

By using the above estimating process, field dry densities and moisture content at time of construction were calculated from the proctor maximum dry density and optimum moisture and are displayed in Tables A-16, A-18 and A-20 with starred values (*). Since field density and moisture at construction and at a later date are available, it was possible to examine the long term changes in density and moisture. During this analysis, it became apparent that AB and SM should be grouped together, and SGS and subgrade should be grouped together since they tended to change both density and moisture in a similar manner. The reason for this is that AB and SM historically have been very similar in grading. Likewise SGS has tended to be closer to the subgrade than the SM grading.

As Figure 13 shows, for AB-SM with dry densities below about 125 $\#/ft^3$, the after construction dry densities increased raising the percent compaction up to about 110 percent of the maximum proctor dry density. AB-SM above about 125 $\#/ft^3$ needed to decrease in density, reducing the compaction to about 99 percent of the proctor maximum dry density. This relationship is very similar to the AC densification under traffic. That is, the low density bases are aggregates with a very rough surface which do not readily compact. Under traffic these materials slowly compact, thus raising their density. The higher density bases are composed of virtually all crushed aggregate, which readily

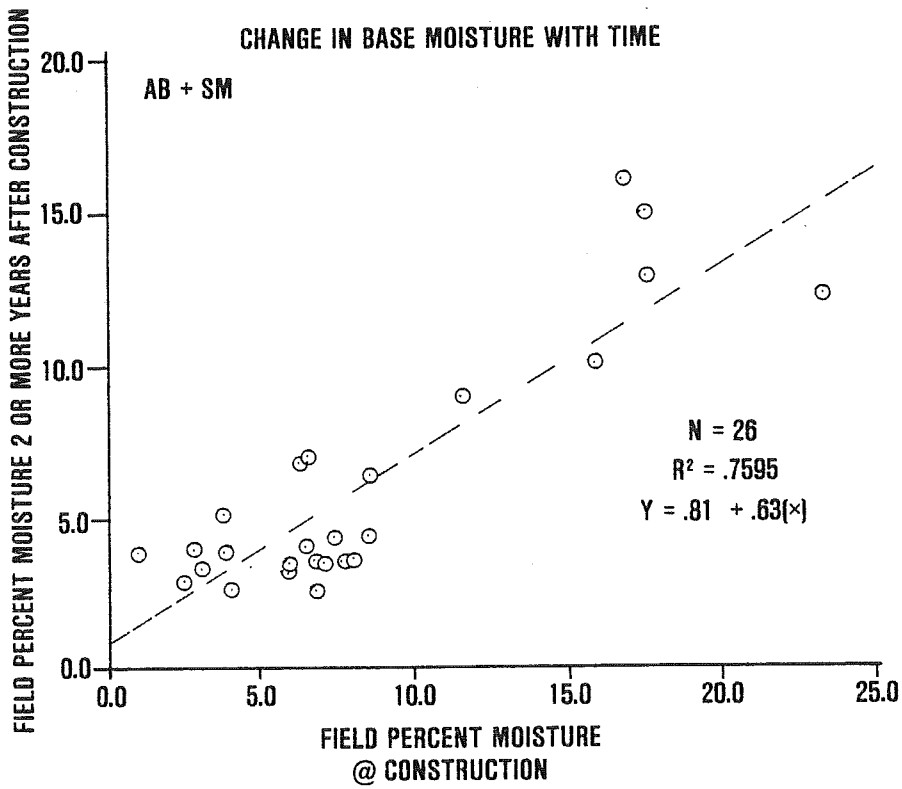
compact under heavy loading; however, keeping this aggregate together or increasing its density without a lubricating agent such as apshalt is very difficult.

FIGURE 13



The AB-SM moisture content changed with time as shown on Figure 14.

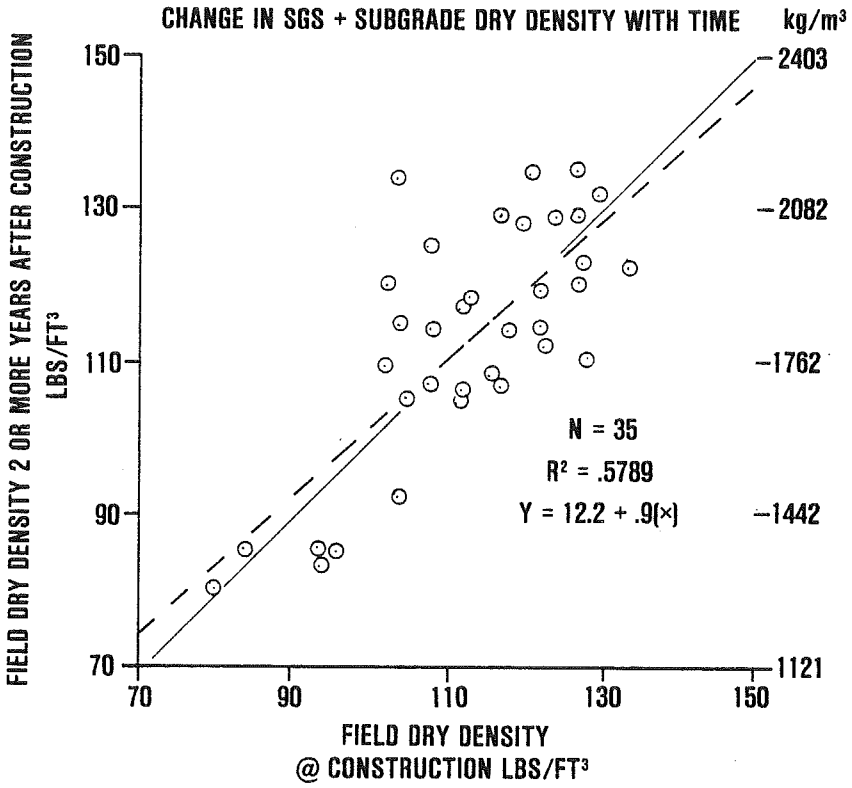
FIGURE 14



As can be seen, the AB-SM tend to become dryer with time. Since at construction their moisture content is generally about 3.0 percent dryer than optimum, the long term moisture condition is from 3.0 percent to as much as 10 percent below optimum.

Figure 15 shows the change in subgrade dry density with time.

FIGURE 15

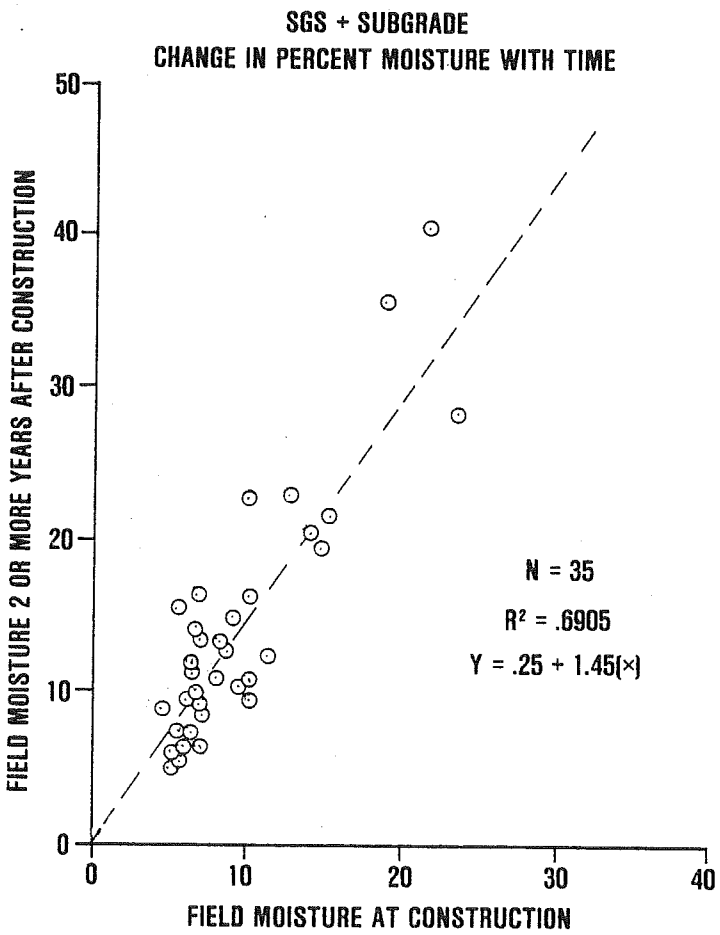


The SGS-SUBGRADE material tended to densify in a manner similar to the base material. That is, subgrades originally compacted below about 120 #/ft^3 increased in density, whereas those above 120 #/ft^3 tended to slightly decrease. Just as importantly, it can be seen that a large number of SGS and subgrades decreased in density over time. To appreciate this fact, Table 5 shows the comparison of dry densities for both base and subgrade materials. Although the proportion of base and subgrade materials decreasing in density is very close, the SGS and subgrade materials have experienced a greater degree of decompaction.

TABLE 5	AB-SM	SGS-SUBGRADE
NUMBER OF SITES	22	35
PERCENT OF SITES BELOW CONSTRUCTION DENSITY	45%	49%
AVERAGE PERCENT OF PROCTOR MAX. DENSITY FOR SITES BELOW CONSTRUCTION DENSITY	97.1%	90.8%

Figure 16 shows the changes in SGS and subgrade moisture content with time. Of the 35 observations, only 4 showed lower moisture two years or more after construction. The average loss in moisture for these 4 cases was only .6 of the percent below the construction moisture. Without a doubt, subgrade moistures do increase with time and those subgrades with high optimum moisture values experience the greatest amount of moisture increase.

FIGURE 16



Another way of seeing this is to compare the two year post construction field moisture to the optimum moisture for both base and subgrade cases.

	AB-SM N = 26	SGS-SUBGRADE N = 35
AVERAGE PERCENT OF OPTIMUM MOISTURE	47.9%*	88% **

* NO FIELD MOISTURES ABOVE OPTIMUM
**10 FIELD MOISTURES ABOVE OPTIMUM

To conduct a pavement design, the subgrade materials are characterized by the Hveem R-value test at 300 psi to determine the soil support. Generally the R-value density and moisture are considered to represent the worst condition. During the field sampling phase, a sufficient quantity of base and subgrade were sampled in order to perform an R-value test. Figure 17 is a plot of the R-value dry density versus the proctor T-99 dry density for both base and subgrade materials. Essentially, the T-99 proctor maximum density is equal to the Hveem R-value 300 psi dry density. Figure 18 shows the relationship between the T-99 proctor optimum moisture and the R-value 300 psi moisture. Again the Hveem R-value moisture is essentially equal to the proctor optimum moisture. An examination of the relationship between the proctor maximum density and the optimum moisture (Figure 19) showed that the two values are strongly correlated. Considerable attempts were made to see if R value correlated with grading, dry density or moisture content.

FIGURE 17

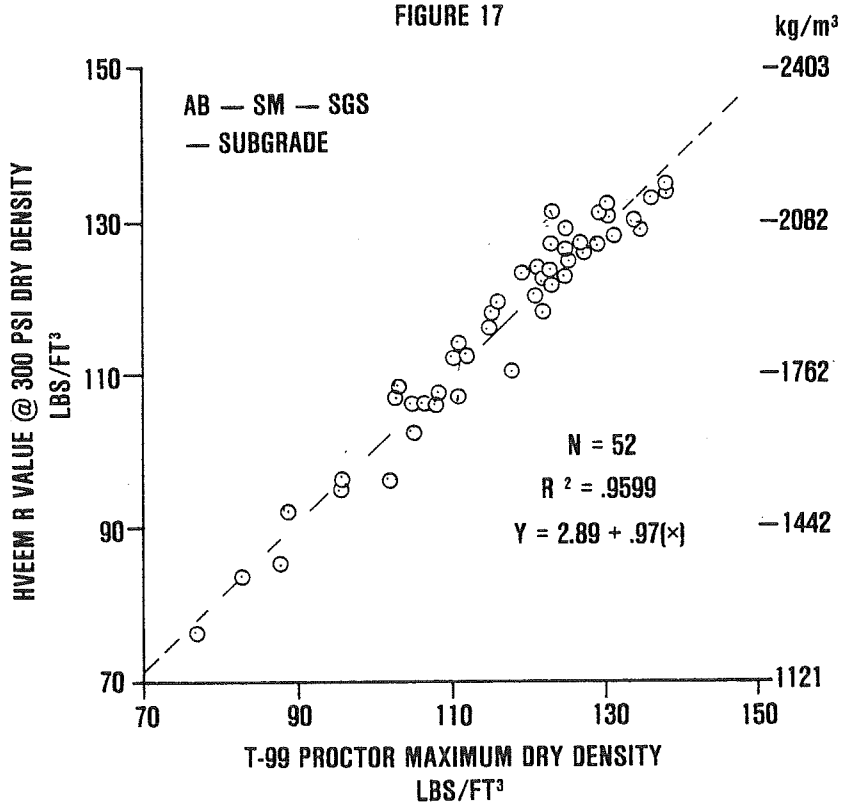


FIGURE 18

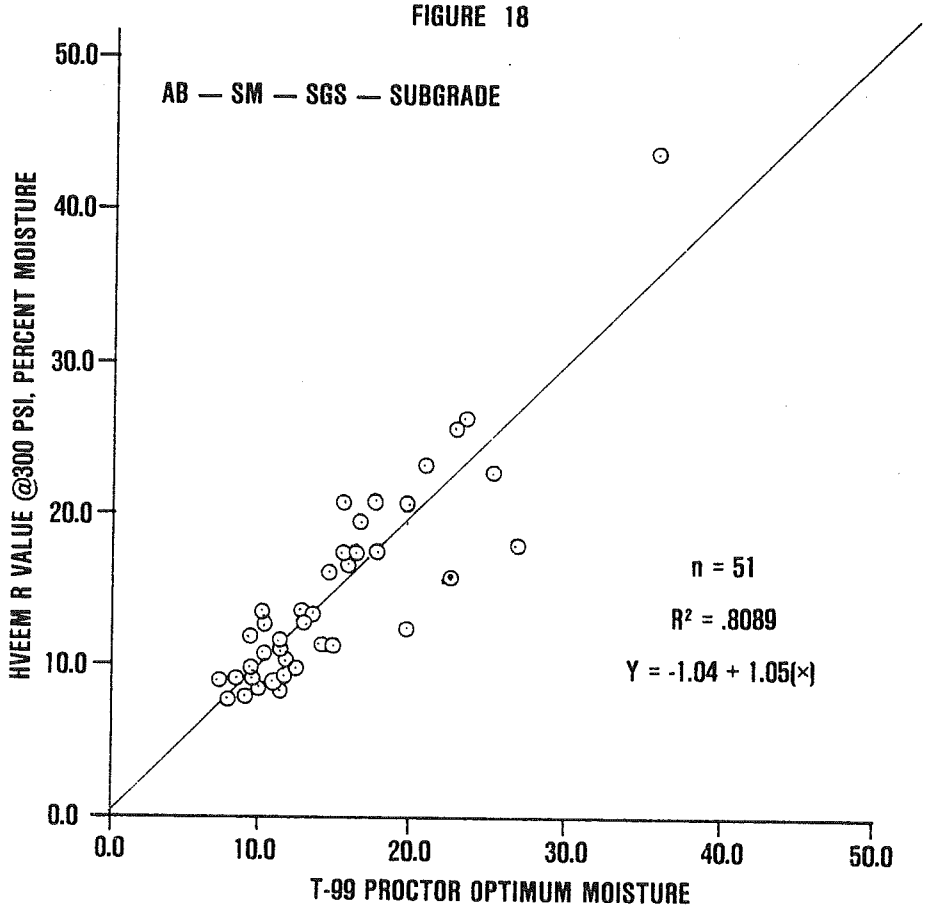
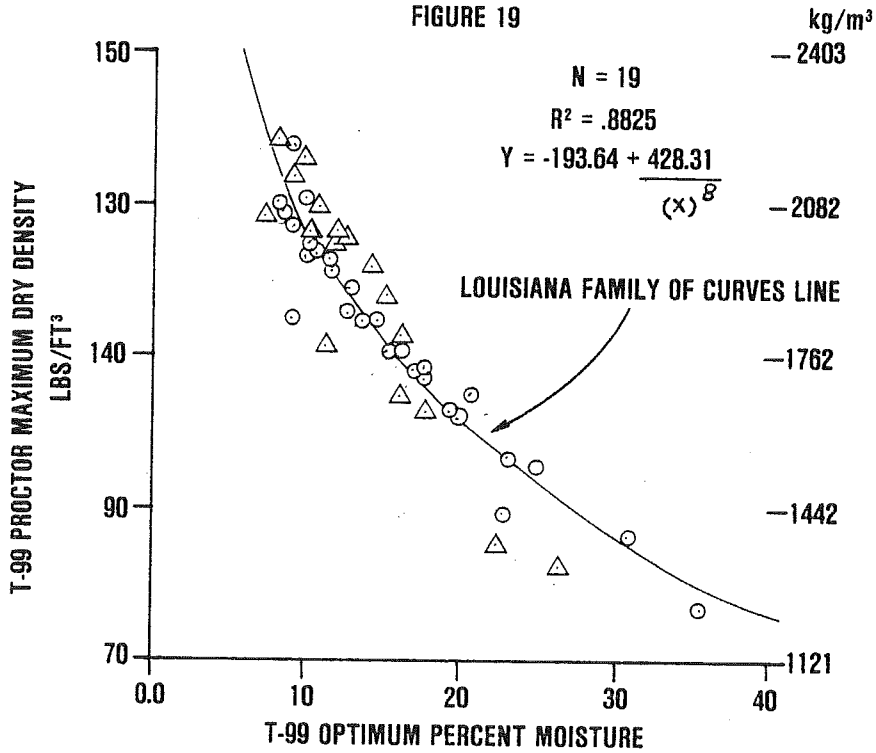


FIGURE 19



Tables B-12, B-13 and B-14 were used to correlate grading, clay content and moisture content to R value. Historically, ADOT has kept a computer file of all R value tests. At present, 2500 test results are in this file. From the file data, a correlation between R value and the variables PI and percent passing the 200 sieve (-200) has been created, Figure 20. Using this figure, it is possible to estimate the R value from the PI and -200s. The correlation between the predicted and actual R value was quite good.

ACTUAL R VALUE @ 300 PSI = -2.95 + 1.11 (PREDICTED R VALUE)

N = 30 R^2 = .8081

The above correlation set a standard by which other variables could be examined. For this study, several individual variables were examined for their correlation to R value. Table 6 gives results of this work.

TABLE 6

R VALUE AT 300 PSI IS VARIABLE Y

VARIABLE X	EQUATION FORM	# OF OBSERVATIONS N	CORRELATION R^2	A	B
PASS 2 MICRON (% CLAY)	$Y = A + B*(X)$	30	.7025	76.91	-2.43
PASS 200 (-200)	$Y = A + B*(X)$	30	.6906	87.38	-1.07
PI	$Y = A + B*(X)^{-1/4}$	30	.6740	-25.11	107.57
PROCTOR OPTIMUM % MOISTURE	$Y = A + B*(X)^{-1/2}$	30	.3216	-38.86	319.32
PROCTOR MAXIMUM DENSITY	$Y = A + B*(X)$	30	.2669	-66.44	.99
SAND EQUIVALENT	$Y = A + B\sqrt{X}$	30	.7424	-13.61	16.43

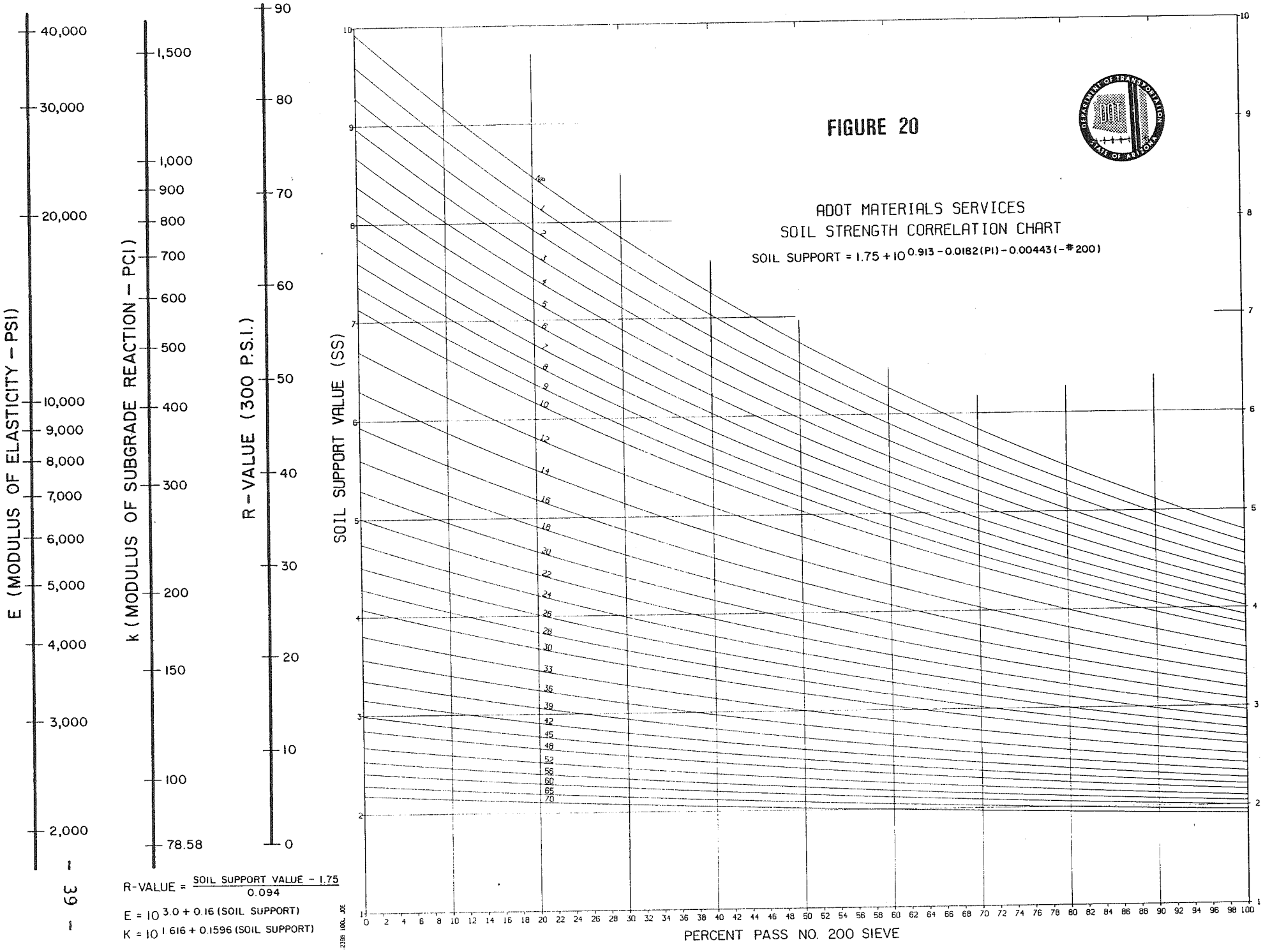


FIGURE 20
ADOT MATERIALS SERVICES
SOIL STRENGTH CORRELATION CHART
SOIL SUPPORT = 1.75 + 10^0.913 - 0.0182(PI) - 0.00443(-#200)
E (MODULUS OF ELASTICITY - PSI)
40,000
30,000
20,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
k (MODULUS OF SUBGRADE REACTION - PCI)
1,500
1,000
900
800
700
600
500
400
300
200
150
100
78.58
R-VALUE (300 P.S.I.)
90
80
70
60
50
40
30
20
10
0
SOIL SUPPORT VALUE (SS)
PERCENT PASS NO. 200 SIEVE
R-VALUE = SOIL SUPPORT VALUE - 1.75 / 0.094
E = 10^3.0 + 0.16 (SOIL SUPPORT)
K = 10^1.616 + 0.1596 (SOIL SUPPORT)

As can be seen in Table 6, the R value test appears to be better related to gradation and in particular tests that describe the clay content of the material. Interestingly, both the -200 and PI are very well related to the minus 2 micron (clay) material as shown on Table 7. This relationship would help explain the reason why both -200 and PI relate so well to the R value.

TABLE 7

Y VARIABLE IS -2 MICRON (CLAY)

X VARIABLE	EQUATION FORM	N	R^2	A	B
-200	Y = A+BX	33	.8435	-2.87	.40
PI	Y = A+BX	29	.7412	-2.61	1.13

In summary, this section has dealt with changes in materials properties, compaction or configuration with time. All layers of materials have experienced changes with time. Many of these long term changes are predictable. In the next section, short term day to day, month to month changes in temperature, moisture and deflection will be examined.

MONITORING OF TEMPERATURE, MOISTURE AND DEFLECTION

TEMPERATURE

In the Phase I report (5), the temperature installation and measurement process was fully described. Essentially, thermistors were used to measure the temperature of various layers of material. Temperature measurements were taken every other hour at 14 selected sites ranging in elevation from 488 feet (149) meters) to 8063 feet (2458 meters) above sea level. Over a three year duration, approximately 700,000 temperature readings were automatically recorded. It would be impossible to display each individual reading within this report. To suitably handle the large body of information, tables and figures of meaningful values have been assembled. Appendix C contains monthly temperature tables for various depths of surface, 3 inch (.0762 m), 9 inch (.2286 m), 15 inch (.3810 m), 24 inch (.6096 M) and 30 inch (.7620 m). Table 1C gives the average monthly temperature, Table 2C shows the average monthly high temperature and Table 3C is the average monthly low temperature. Since designers are often interested in extreme conditions, Table 8 shows the average monthly low temperature for January and Table 9 shows the average monthly high temperature for July by depth.

As can be seen, temperature is generally related to elevation. Those locations below 3500 feet (1067 m) are very warm in the summer. Asphalt at the surface of the pavement would experience temperatures above 140°F (60°C) on a regular basis during the summer. Above 3500 feet (1067 m), temperatures drop off considerably. For these higher elevations, freezing temperatures are common. Table 10 shows the average annual number of freeze thaw cycles per depth, and Table 11 gives the average total hours of freezing.

TABLE 8

JANUARY AVERAGE MONTHLY LOW TEMPERATURE, ^{o}F

FOR 1975, 1976, 1977, 1978

SITE	ELEV	AIR	SURFACE	3 INCH	9 INCH	15 INCH	24 INCH	30 INCH
24	488	37	46	48	52	59	58	60
6	720	38	48	50	53	57	57	61
1	994	36	47	49	51	56	56	58
16	1188	35	46	46	50	57	55	57
3	3209	28	41	42	45	55	52	53
4	4186	30	37	39	41	46	48	49
10	4897	19	27	27	43	52	41	47
7	5139	21	22	29	38	37	42	40
19	6184	15	20	27	34	39	40	39
5	6384	18	15	21	32	41	35	36
29	6496	17	26	26	31	36	36	34
14	6958	15	18	26	30	34	35	35
36	6958	15	21	29	34	39	39	34
33	8061	10	12	21	30	31	32	33

1 FOOT = .3048 METERS; $^{o}F = 32 + 1.8\ (^{o}C)$

TABLE 9

JULY AVERAGE MONTHLY HIGH TEMPERATURE, ^{o}F

FOR 1975, 1976, 1977, 1978

SITE	ELEV	AIR	SURFACE	3 INCH	9 INCH	15 INCH	24 INCH	30 INCH
24	488	109	168	144	139	114	101	101
6	730	109	167	145	141	115	107	101
1	994	107	146	128	124	111	106	101
16	1188	104	145	128	121	110	102	100
3	3209	100	139	127	120	109	100	100
4	4186	96	132	120	113	100	99	90
10	4897	94	120	112	101	95	93	83
7	5138	92	105	90	93	91	82	83
19	6184	92	103	96	88	86	83	80
5	6384	86	107	95	80	80	80	73
29	6496	84	99	90	85	77	74	71
14	6958	81	96	85	80	69	66	70
36	6958	81	92	81	79	72	68	66
33	8063	79	91	79	75	70	65	64

1 FOOT = .3048 METERS; $^{o}F = 32 + 1.8\ (^{o}C)$

TABLE 10

AVERAGE ANNUAL NUMBER OF FREEZE THAW CYCLES: 1975, 1976, 1977, 1978

1 INCH = .0254 M ; 1 FOOT = .3048 M

SITE	ELEV	1/4"	3"	9"	15"	24"	30"
24	488'	2	0	0	0	0	0
6	730	2	0	0	0	0	0
1	994	2	0	0	0	0	0
16	1188	3	0	0	0	0	0
3	3209	30	15	4	1	0	0
4	4186	49	18	10	3	1	0
10	4897	78	70	37	8	6	0
7	5138	65	37	25	9	4	0
19	6184	39	24	12	11	5	0
5	6384	143	67	39	15	6	0
29	6496	56	46	32	21	10	0
14	6958	89	68	28	20	7	0
36	6958	121	91	24	8	6	0
33	8063	126	114	25	14	5	1

TABLE 11

AVERAGE ANNUAL TOTAL HOURS OF FREEZING OR BELOW FOR 1975, 1976, 1977, 1978

SITE	ELEV	1/4"	3"	9"	15"	24"	30"
24	488'	7	0	0	0	0	0
6	730	6	0	0	0	0	0
1	994	8	0	0	0	0	0
16	1188	10	0	0	0	0	0
3	3209	126	89	62	15	0	0
4	4186	310	189	126	32	10	0
10	4897	880	832	490	248	72	0
7	5138	652	380	330	211	61	0
19	6184	412	318	286	212	77	0
5	6384	2410	2147	1573	615	96	0
29	6496	980	692	396	315	81	0
14	6958	1160	715	468	416	91	0
36	6958	1914	1464	276	60	55	0
33	8063	2212	2020	1222	568	394	15

1 INCH = .0254 M, 1 FOOT = .3048 M

Considerable freezing can and does take place in Arizona. For elevation above 3500 feet (1067 m) and freezing index above 100, pavement distress associated with freezing and thawing should be anticipated and considered in the design process.

The above tables should serve as a good guide to existing pavement temperatures. Estimates of future temperatures can be drawn from these tables.

MOISTURE

Unlike temperature, which is reasonably predictable from year to year, moisture has always been thought of as very unpredictable. Two approaches to monitoring moisture are nuclear depth moisture and moisture cells. The Phase I report (5) describes these devices and their installation. It was intended that the nuclear depth moisture gauge would monitor changes in layers of soil at least 2 feet (.61M) in thickness, wheras the moisture cells or wafers would monitor discrete layers such as pavement, base course and top of subgrade.

Appendix D, and Tables 4D, 5D, 6D and 7D give the weight percent moisture measured by both methods for four locations, which include the shoulder or median, distress lane, travel lane and passing lane. Readings at depths of 2 (.05M), 8 (.20M), 14 (.36M), 24 (.61M), 48 (.1.22M), 72 (1.83M), 96 (.2.44M) and 120 (3.05M) inches are shown. The 2, 8, 14 and some 24 inch values were determined from samples and readings of the Bouyoucous moisture meter. Some 24 inch readings, plus all 48, 72, 96 and 120 inch depths, were taken with a Troxler nuclear depth moisture gauge. To determine the weight percent moisture, it was necessary to relate meter reading to percent moisture from soil samples. The process of calibration was described in the first report (5). During the course of the project it became apparent that a correction in the calculated moisture derived from nuclear

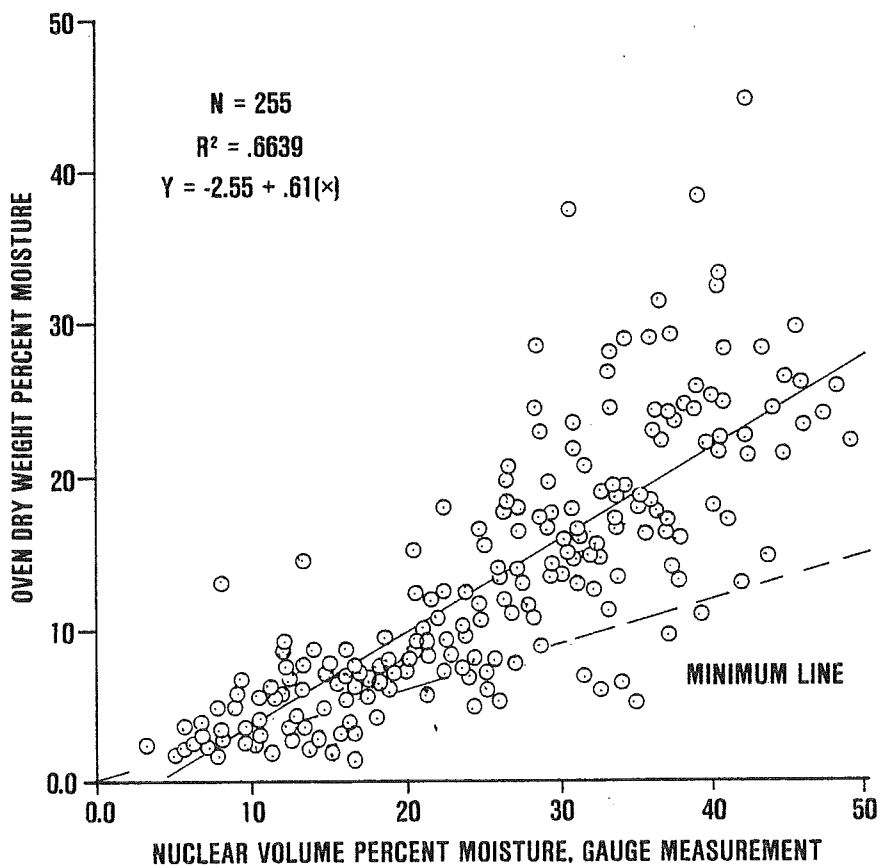
readings was necessary. Figure 21 is a plot of weight percent moisture versus the initial nuclear reading for all sites, derived from Table D-1, Appendix D. As this figure demonstrates, there is a relationship between the two values; however, the slope of the line is rather steep. If this line were used to derive in place weight percent moisture from nuclear readings, rather large values would be calculated. To avoid this, a conservative approach was taken by plotting the minimum line. This line represents a very conservative estimate of the expected change in moisture for change in nuclear readings. An initial formula for all nuclear depth readings was derived in the following fashion:

> Given travel lane aluminum tube installation, the 2 foot initial nuclear reading = 30.0 and the 2 foot oven dry moisture = 15.0% by weight.

The relationship would be:

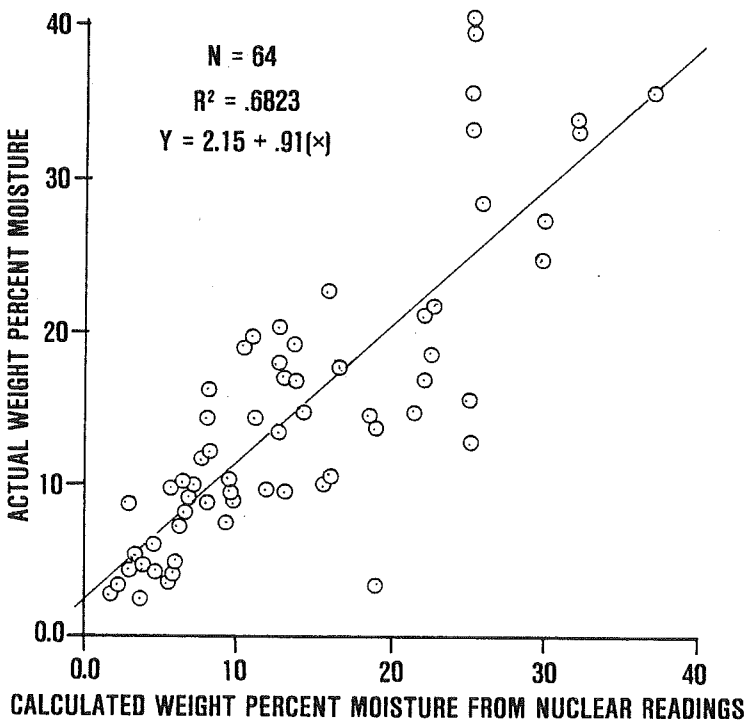
> % Oven Dry = A + B (Nuclear reading) moisture, where B = .3 = slope of the very conservation moisture line.

FIGURE 21



In this way, if a future reading of 30.0 were obtained with the nuclear device, the calculated percent moisutre would be 15.0%, just as it was initially. In addition, as nuclear readings changed, the rate of change of the weight percent moisture would be 3 percent by weight for each 10 percent by nuclear for all cases. In this way, all site and depth values could be unbiasedly compressed. To check the validity of this method, soil samples were taken at various times at the two and four foot depths and tested for percent moisture and plotted against the calculated values. Figure 22 was derived from this data, as shown on Table D-2.

FIGURE 22



This figure shows good correlation between the calculated weight percent moisture and the actual moisture. Considering the fact that samples were taken two to ten feet away from the aluminum tubes, this is extremely good agreement. The slope of line is very close to the line of equality. This approach of using a conservative approach to the slope of the moisture line seems to have worked even better than expected, since the correlation line does not appear to be unduly biased.

Given Tables 4D, 5D, 6D and 7D, it was possible to examine the moisture content of the embankment and subgrade layers. Table 12 shows the average weight percent moisture content for various depths.

TABLE 12

OVERALL GRAND AVERAGE OF PERCENT MOISTURE BY WEIGHT: ALL NUCLEAR SITES: ALL READINGS

	UNCOVERED SOIL NEXT TO HIGHWAY	DISTRESS LANE	TRAVEL LANE	PASSING LANE
2 Feet	9.9	13.8	12.4	11.4
4 Feet	11.1	16.1	16.5	14.8
6 Feet	12.4	15.0	17.1	15.7
8 Feet	11.2	14.5	17.4	13.2
10 Feet	15.4	16.9	15.3	15.1

As can be seen, the average moisture content under the highway is considerably greater than the normal soil moisture next to the highway. All nuclear sites were located on interstate highways with transverse slopes of .015 ft/ft or .020 ft/st and the two foot (.6m) moisture content decreases across the road (distress, travel and passing lane) as the elevation increases. Interestingly, three out of four of the highest moisture contents occurred in the travel lane at the four, six and eight foot (1.2, 1.8, 2.4 m) depths. The average moisture contents at various depths on Tables 4D, 5D, 6D and 7D are probably representative of an equilibrium moisture condition. If this is true, how much do they vary? Table 13 gives the average standard deviation of moisture content by depth and location. This value was derived by adding the variances and dividing by the number of cases to create an average variance, which became the standard deviation by taking the square root.

As expected, the uncovered soil experiences greater fluctuation in moisture than the covered locations under the highway.

TABLE 13

OVERALL AVERAGE STANDARD DEVIATION OF PERCENT MOISTURE BY WEIGHT; ALL NUCLEAR SITES, ALL READINGS

	UNCOVERED SOIL NEXT TO HIGHWAY	DISTRESS LANE	TRAVEL LANE	PASSING LANE
2 Feet	1.3	.9	.8	.8
4 Feet	1.0	.9	.8	.5
6 Feet	.8	.8	.8	.7
8 Feet	.8	.5	.8	.6
10 Feet	.6	.5	.7	.6

With the knowledge of an equilibrium moisture content expressed by the five-year average and the variability, two questions arise. Can the equilibrium moisture for the travel lane at various depths be predicted and can the seasonal moisture content above or below the equilibrium be estimated? The travel lane was selected to derive a prediction equation because it generally experiences the largest proportion of loads, most serious distress and greatest need for maintenance. In predicting the various layers moisture content, some information about the location and the materials properties will be needed. Historically, ADOT has kept design and construction files of what materials were used to build the highway. Generally, this information is expressed as the -200, PI, calculated R value or soil support value for most miles of highway within the ADOT system. In addition to this, the highway elevation (Figure 23), average annual rainfall (Figure 24), climatic zone (Figure 25) and freezing index (Figure 26) exist as possible additional variables. From the elevation, rainfall and climatic zone, a regional factor value representative of the effect of climate on

FIGURE 23

GENERALIZED RELIEF

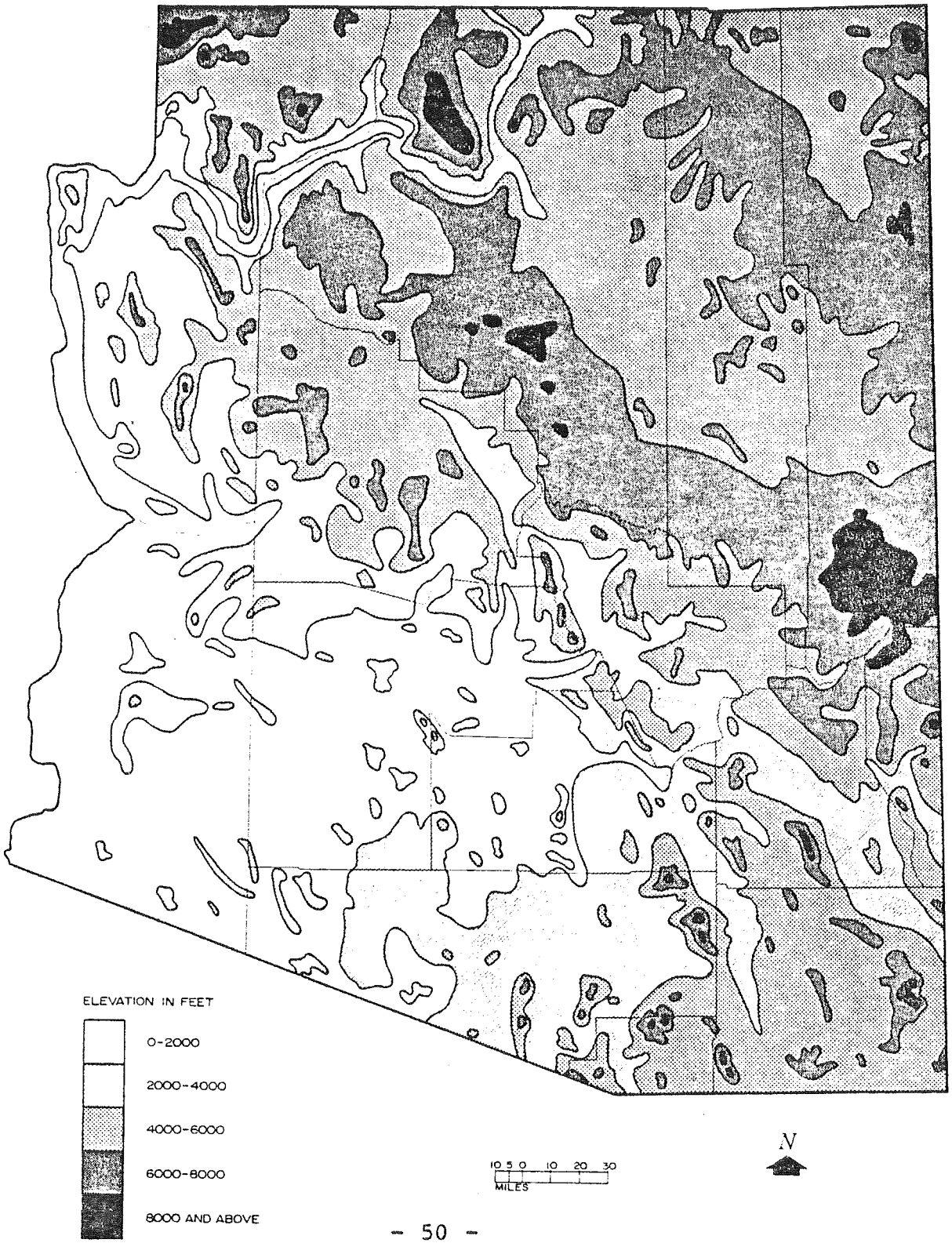
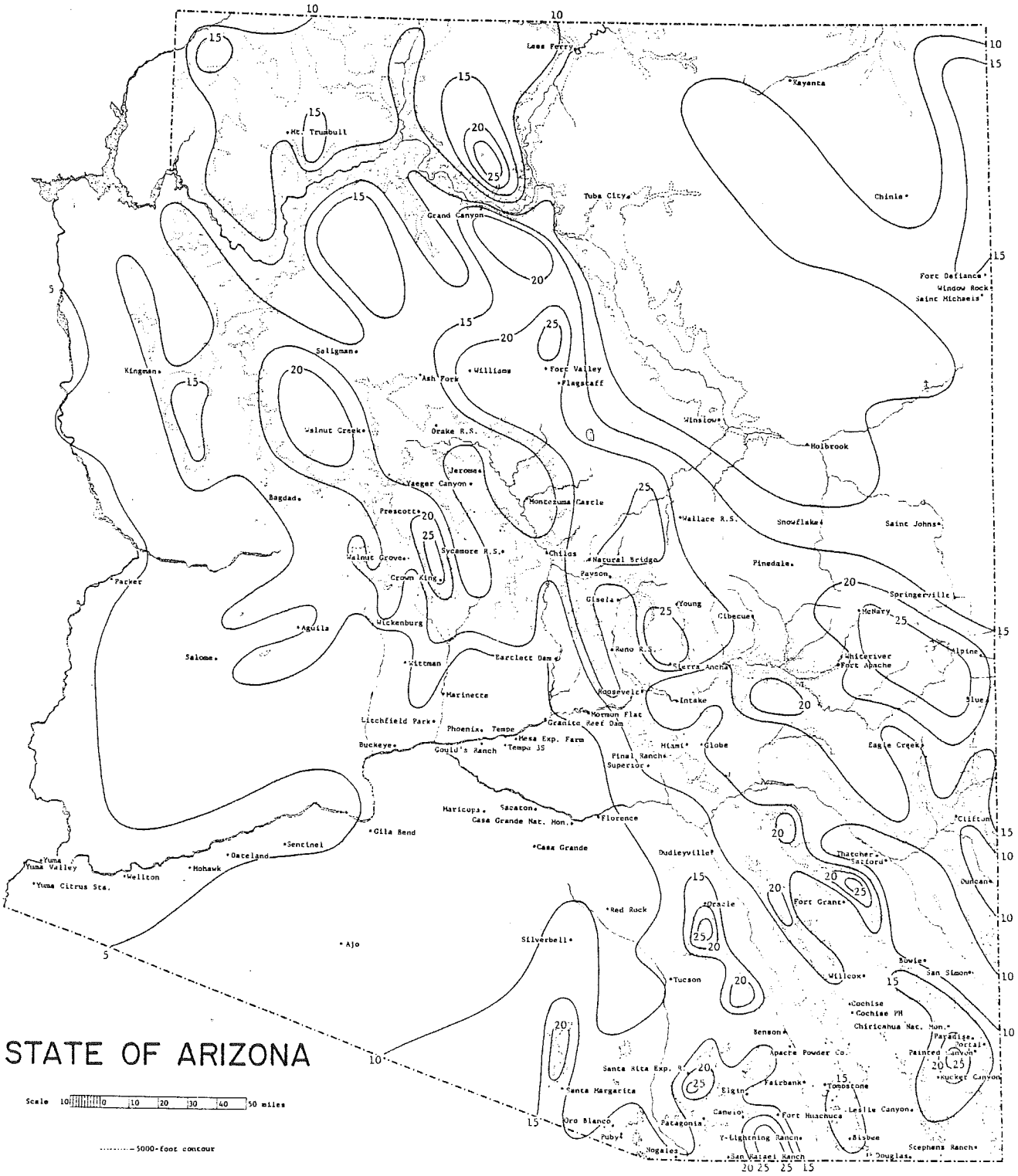
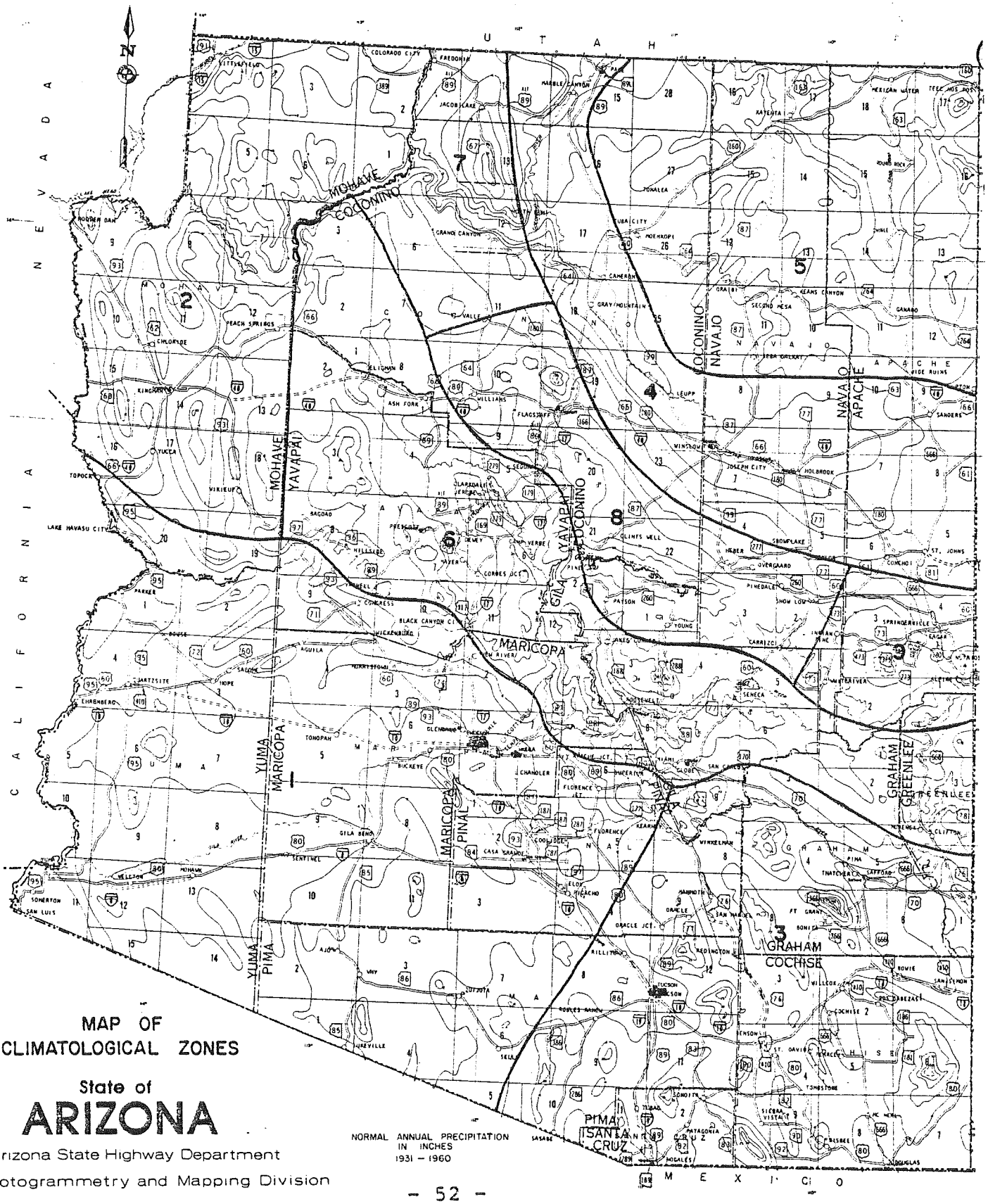


FIGURE 24



Average annual precipitation (in inches) in Arizona.

FIGURE 25



MAP OF
CLIMATOLOGICAL ZONES

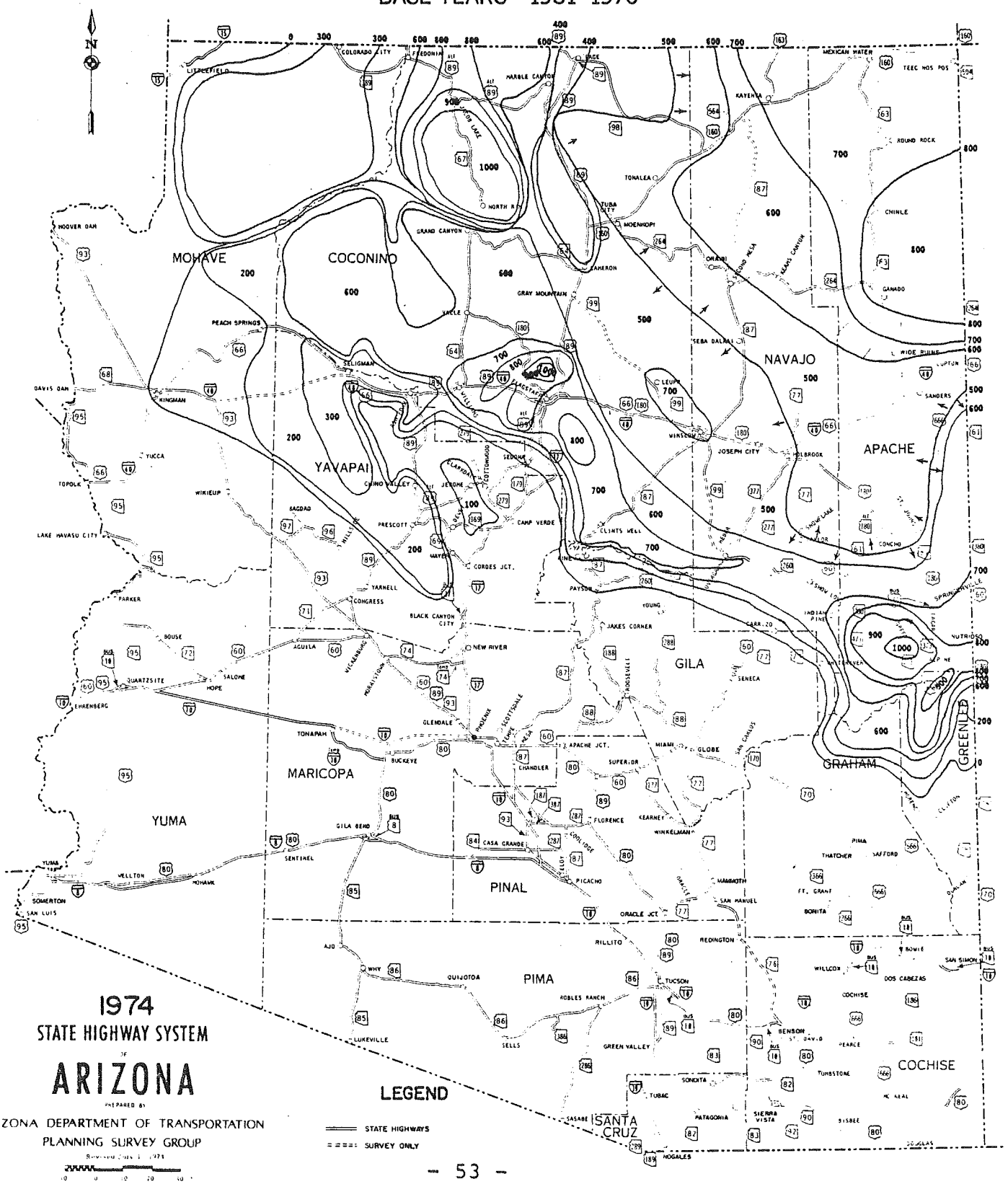
State of
ARIZONA

Arizona State Highway Department
Photogrammetry and Mapping Division

NORMAL ANNUAL PRECIPITATION
IN INCHES
1931 – 1960

FIGURE 26

FREEZING INDEX
MAP OF ARIZONA
BASE YEARS 1931-1970



highway performance is also calculated. Generally, the subgrade location will occur within the two-foot (.6m) to four-foot (1.2m) depth below the pavement surfacing. By using Table 6D for the two-foot (.6m) and four-foot (.12m) depths, it was possible to accumulate the equilibrium moisture for virtually all the subgrades as well as the various other factos, including tbe -200, PI, elevation, average annual rainfall, regional factor, soil support, and several other factors which would not normally be known, including T-99 proctor optimum moisture, liquid limit, plastic limit and sand equivalent. Table 14 gives the correlation between the quilibrium moisture and the above variables. Many of the variables represent material index properties that routinely would not be known. That is, T-99 proctor maximum density and optimum moisture would not normally be known for most soils in Arizona. The term regional factor divided by soil support was created since this value would be known for virtually all miles of highways in Arizona. Fortunately, this interaction value, which combines several factors, including rainfall, elevation, climate zone, PI, -200 and/or R value at 300 psi, gave the highest correlation to the long-term equilibrium moisture. Such a value can serve as an excellent predictor of subgrade soil moisture under Arizona highways. Thus, the equilibrium moisture content can be predicted using readily available file data.

TABLE 14

Y = Equilibrium Moisture = Dependent Variable

X = Independent Variable

X	FUNCTION	# OF OBSERVATIONS	R^2	A	B
Regional Factor/ Soil Support	Y=A+BX	31	.6471	6.83	14.40
T-99 Proctor Max. Density	Y=A+BX	26	.6434	52.97	-.35
T-99 Optimum Moisture	Y=A+BX	26	.6243	2.47	.71
PI	Y=A+BX	31	.5265	9.09	.35
Estimated % Moisture @ Const.	Y=A+BX	28	.5058	4.85	.89
R Value % Moisture @ 300 psi	Y=A+BX	26	.5044	4.35	.51
Sand Equivalent	Y=A+B/X	18	.5043	3.03	30.54
Soil Support	Y=A+BX	31	.4811	25.24	-2.05
Liquid Limit	Y=A+BX	25	.4545	4.59	.29
-2 Micron-% Clay	Y=A+BX	28	.4141	7.77	.44
-200	Y=A+BX	31	.4055	5.82	.19
Regional Factor	Y=A+BX	31	.2486	6.55	3.02
R Value @ 300 psi Average Annual Inches Rainfall	Y=A+BX	31	.2097	6.31	.55
Average Yearly Temperature	Y=A+BX	31	.1324	26.56	-.2333
Elevation	Y=A+BX	31	.1162	9.26	.0009
% Cracking	Y=A+BX	31	.0670	11.99	.0678

Using the regional factor, soil support ratio (RF/SS) additional layers wer examined to determine their relationship. Results of this work gave the following.

TABLE 15

Y = Equilibrium Moisture = Dependent Variable

X = Regional Factor/Soil Support

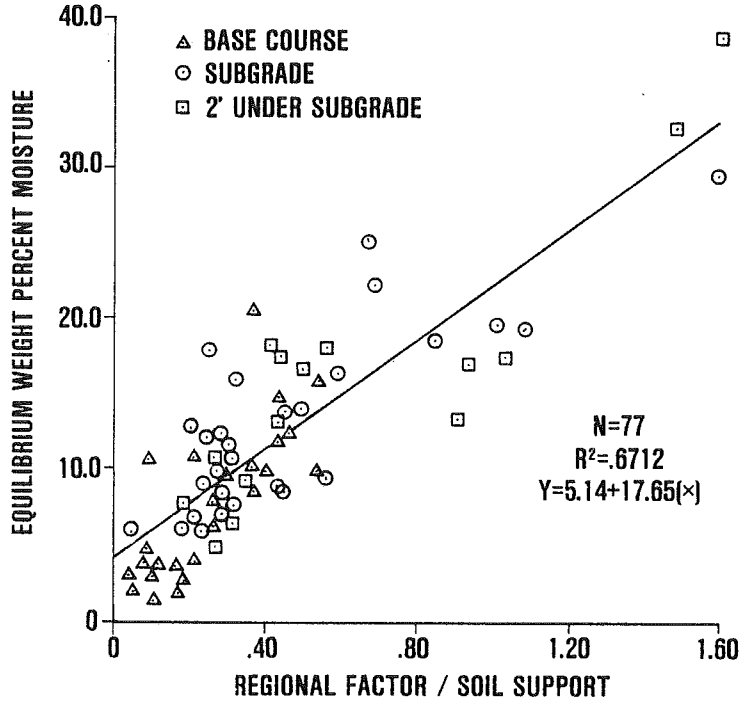
Y = A+B(X)

Y	N	R^2	A	B
Base Course Moisture	27	.5409	2.56	24.36
Subgrade moisture	31	.6471	6.83	14.40
Two Feet Under Subgrade	19	.6668	5.88	17.81
General Equation All Three Depths	77	.6712	5.14	17.65

Base course layers included aggregate base, select material, subgrade seal and cement-treated base. Their PI and -200 values were used to estimate the soil support value.

FIGURE 27

EQUILIBRIUM MOISTURE VS. RATIO
OF REGIONAL FACTOR TO SOIL SUPPORT



As can be seen, the generalized equation can be used just as well for all soil and aggregate materials from the top of the base course to two feet below subgrade for the travel lane. In-place, long-term equilibrium moisture can be estimated or predicted from the ratio of regional factor to soil support.

To predict the moisture variability, a similar approach was taken. The same set of variables was examined and found to have extremely small correlation values; generally less than .03, or virturally random. The only exception to this was percent cracking, which gave the following for subgrade materials:

$$N = 29 \qquad R^2 = .2858$$

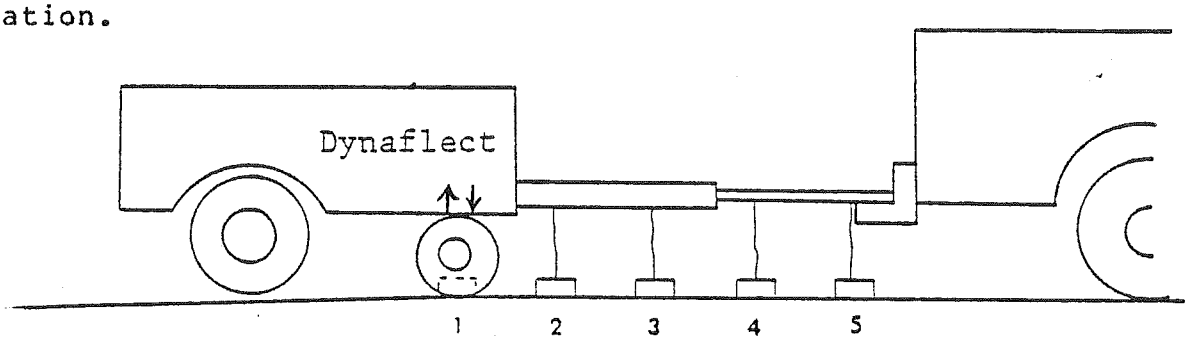
$$\text{Equilibrium Moisture Variance} = .83 + .03 \left(\text{Average Percent Cracking}\right)$$

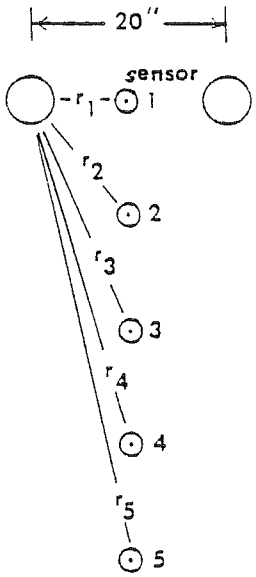
Considering the rather low correlation, it would be acceptable to use Table 13 as a reasonalbe prediction of moisture variability. For subgrade materials, moisture changes would be expected to fluctuate ± 2.5 percent away from the equilibrium value. For base courses, the variability would be slightly higher, at about ± 3.0 percent moisture.

To more fully understand the role that base and subgrade moisture plays in highway performance, detailed deflection studies have been performed at each site.

DEFLECTION

Deflection measurements were taken at each site at two to three month intervals. The location of these measurements are shown below in relation to the moisture and temperature measuring location.





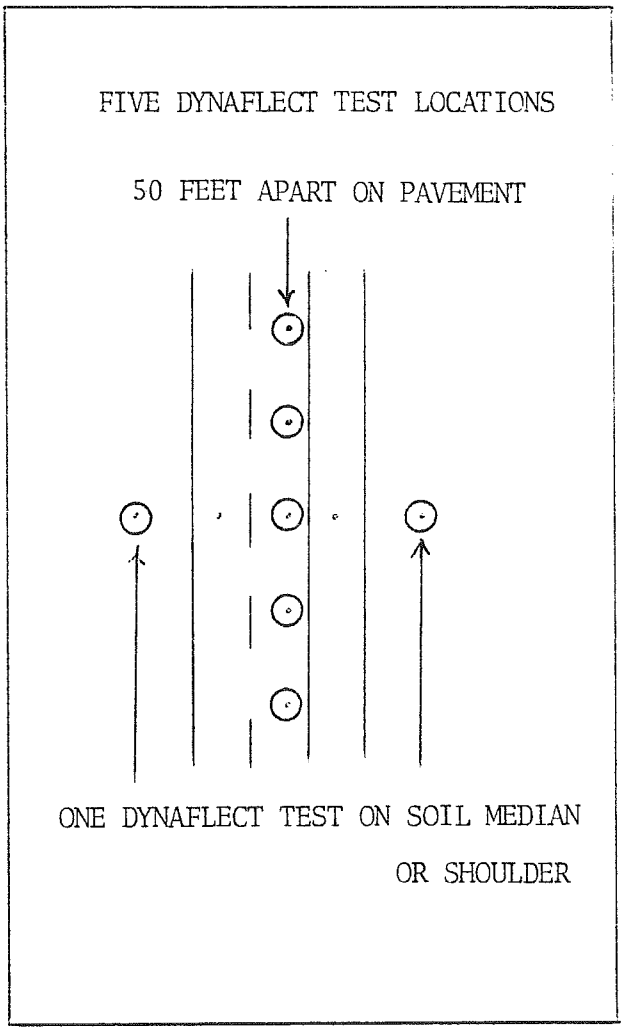
r_1 = 10.0"

r_2 = 15.6"

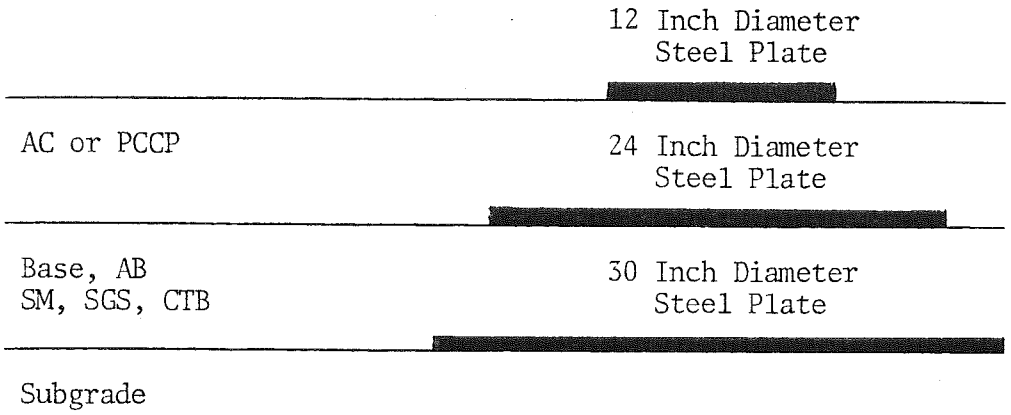
r_3 = 26.0"

r_4 = 37.4"

r_5 = 49.0"



To insure that the same location was tested each time, markers were painted on paved surfaces and stakes were placed on soil and triangulated to benchmarks. In 1975 and 1976, plate bearing tests were performed along with dynaflect tests at most sites. Plate bearing tests were performed as shown below.



To accomplish this testing, test pits were dug in the pavement. At the same time, in-place densities were taken using the volumeter. In addition, samples were taken for moisture content, grading, PI, sand equivalent, R value, proctor T-99 density and moisture, resistivity, soluble salts. Virtually all this test information is reflected in Appendix B.

DYNAFLECT DEFLECTION

Appendix E contains results of all dynaflect deflection testing on Tables 1E through 37E. Tables are in order of increasing site number; thus, Table 1E is for site number one. Table 10E is not shown since all deflections were taken at site 11, approximately one-half mile (805m) away. Temperatures for site 11 were recorded at site 10. For those sites where deflections were taken on both the travel lane and the adjacent median or shoulder, two tables are given. As an example, Table 21E shows deflections for the travel lane, where as Table 21E-M shows deflections for the adjacent median. Each table shows the site number, name, average number of readings take at each time and thickness of layers. In addition, the date tested, air temperature and surface temperature are shown. The pavement mid-depth temperature, as well as approximate depth, was determined by inspecting the large temperature history file and represents either the exact temperature for recording sites or an interpolated temperature for neighboring sites. Dynaflect deflections are shown in thousandths of an inch. That is a tabled value of 1.00 equals .001" (.0254mm) of deflection; likewise, .05 equals .00005" (.0013 mm). Deflection measurements were taken at the standard locations; however, the distances represented are the radial distance from the center of the load wheel to the center of the geophone. During 1975 and 1976,

special tests were performed at most sites. This special test involved placing a geophone as close to the force wheel as possible, which equaled a 4" (.10m) radial distance. The dynaflect illustration graphically depicts the geophone configuration. The deflections shown represent the average of either four or five readings and are the true mechanical readings without correction factors of any kind. The 10" (.25 m) coefficient of variation values, shown as percentages, represent the 10" (.25 m) standard deviation divided by the average 10" (.25m) deflection and represent the degree of variability in the four or five readings. At the bottom of each table, the average, standard deviation and coefficient of variation are calculated for all measurements.

ANALYSIS OF DYNAFLECT DEFLECTIONS

Arizona has taken thousands of dynaflect deflection measurements as part of its Pavement Inventory process. Deflections are routinely corrected for temperature using the Asphalt Institute method. This method involves obtaining average air temperatures for five days previous to the day of test. This method has meant considerable additional work and delays in trying to obtain this information. To help reduce the work, data collected as part of this study were examined to see if just surface temperature could be used to predict the Asphalt Institute average temperature. As part of this study, the previous five-day average temperature values were obtained and the Asphalt Institute average temperature routinely calculated. Three separate correlations were performed which included:

1.	ADOT measured mid-depth pavement temperature (ADOT)	VS.	Measured surface temperature (ST)
2.	Asphalt Institute average pavement temperature (AI)	VS.	ADOT measured mid-depth pavement temperature (ADOT)
3.	Asphalt Institute average pavement temperature (AI)	VS.	Measured surface temperature (ST)

In all, 442 values were involved in each correlation; results of these correlations are shown below.

Equation Y = A+BX

	Y	X	# OF OBSERVATIONS	R^2	A	B
1.	ADOT	ST	442	.9727	.73	.94
2.	AI	ADOT	442	.9593	3.21	.97
3.	AI	ST	442	.9398	7.01	.88

As expected, the surface temperature is most closely related to the actual mid-depth temperature. Although surface temperature is not as closely related to the Asphalt Institute average pavement temperature, the correlation is very good and can be used to replace the Ashalt Institute procedure which is very time consuming.

In addition to correcting for temperature, Arizona has used the temperature correction chart presented in the Asphalt Institute Overlay Manual to correct or adjust the dynaflect deflection. The chart presently in use is called Figure III-4 Temperature Adjustment Factors for Benkelman Beam deflection. Since this chart was created from Benkelman Beam deflection at various temperatures, it is possible it may not be well suited to the Dynaflect. These two deflection devices are quite different in both the time of loading and magnitude of load as shown below.

	Live Load In Pounds	Time of Loading In Miles per Hour
Benkelman Beam	18,000	1-2
Dynaflect	1,000	55

Since the modulus of asphalt-bound layers and soil subgrades generally are related not only to temperature but also load size and/or time of loading, it is indeed possible that the temperature correction chart for the dynaflect deflection would

be different from the Benkelman Beam. To test this hypothesis, actual dynaflect deflections were plotted versus temperature for all locations and the correlation was determined. Table 16 shows results of this work by increasing site number, grouped by asphalt-bound layers, concrete and soil. As can be seen, only three asphaltic concrete sites out of 29, or 10 percent, show a significant correlation to temperature. Concrete and soil locations showed no significant correlation to temperature as would be expected. It would appear that a temperature correction for dynaflect deflections is not necessary. Since an argument can be made for some temperature correction, a suggested relationship would be the following:

Adjustment Factor = 1.175 - .0025 (Mean Pavement Temperature)

NOTE: FACTOR EQUALS 1.00 AT 70°F.

This formula was derived by calculating the average slope for asphaltic concrete pavements on Table 16.

TABLE 16

DYNAFLECT DEFLECTION IN MILS OF AN INCH
VERSUS
ADOT PAVEMENT MID-DEPTH TEMPERATURE

Y = A+BX
Y = Dynaflect Deflection
X = ADOT Mid-Depth Temperature

Asphaltic Concrete

SITE #	N	R^2	A	B
1	11	.1617	.9	+.0059
2	8	.0789	1.0	+.0022
3	12	.1753	1.9	+.0091
4	7	.5711	1.0	+.0020
5	12	.0424	1.8	+.0043
6	14	.8106*	.1	+.0044
7	10	.0189	1.7	+.0054
8	14	.0599	2.2	-.0051
9	15	.0266	1.3	+.0022
11	14	.0990	1.8	+.0067
12	12	.7066*	.6	+.0068
13	12	.5871*	.3	+.0061
14	6	.1287	1.0	+.0018
15	15	.0281	1.2	-.0015
16	11	.0485	.9	-.0005
17	5	.7506	1.8	-.0091
18	5	.2542	1.3	-.0031
19	5	.4446	.9	+.0021
20	15	.0034	1.2	-.0009
21	15	.0955	1.3	-.0024
22	15	.2290	1.4	+.0039
23	15	.2290	1.4	+.0105
24	12	.0469	.8	-.0007
25	15	.0955	1.3	-.0024
26	15	.2379	.6	+.0058
27	16	.0001	1.3	-.0030
30	14	.1846	.3	+.0030
33	11	.2856	.6	+.0080
37	12	.2727	.2	+.0026
CONCRETE				
28	12	.1975	.3	+.0097
29	13	.3009	.9	-.0056
35	14	.4236	.1	+.0053
36	14	.026	.2	+.0019
Deflection on Top of Soil				
20	11	.0732	2.4	-.0086
21	10	.1791	2.0	-.0045
25	10	.2589	2.3	-.0093
31	13	.0031	1.7	+.0012
32	10	.2443	1.3	+.0092
34	12	.0010	.7	+.0005

*Significant Correlation at the .05 level.

During this project, several sites were overlayed, thus affording the opportunity to examine whether such overlays do indeed reduce the deflection. Oftentimes, it is suggested that deflections be taken just before and just after overlay. Table 17 gives results of this type of analysis, except that besides the before and after deflection, an average deflection before and after overlay

TABLE 17

BEFORE AND AFTER OVERLAY DEFLECTIONS

SITE #		OVERALY THICKNESS	10% DEFLECTION JUST BEFORE AND AFTER	N	AVERAGE DEFLECTION 10"	16"	26"	37"	49"
18	Before	---	.83	5	1.09	.89	.62	.45	.30
18	After	3.5"	.59	6	.73	.62	.49	.38	.29
17	Before	---	.86	5	1.27	.99	.70	.52	.36
17	After	3.0"	.80	6	.95	.81	.63	.48	.37
19	Before	---	1.07	5	1.06	.88	.63	.47	.34
19	After	2.5"	.64	6	.78	.67	.56	.43	.33
4	Before	---	1.19	7	1.12	.68	.38	.22	.15
4	After	2.0"	.74	6	.84	.62	.34	.22	.15
2	Before	---	1.58	8	1.21	.60	.23	.13	.09
2	After	1.8"	1.16	7	1.12	.68	.28	.14	.10
6	Before	---	.33	8	.44	.33	.22	.16	.11
6	After	.5"	.34	6	.42	.31	.20	.14	.10

was also computed. As can be seen, the deflection did go down after overlay. Correlation of overlay thickness to reduction in deflection and after overlay showed that the reduction factor derived from the average deflection before and after overlay were far superior to the point values just before and after overlay as shown below.

$$\text{\% Reduction in Deflection} = \frac{\text{Deflection Before - Deflection After Overlay}}{\text{Deflection Before}} \times 100$$

$Y = A+BX$ $\quad$ Y = % Reduction $\quad$ X = Overlay Thickness In Inches

Assumed that for $X = 0$, $Y = 0$

	N	R^2	A	B
% Reduction From 10" Deflection Just Before and After Overlay	6	.3651	4.43	7.59
% Reduction From Average 10" Deflection Before and After Overlay	6	.8485*	- .44	9.33

Reviewing the average deflection data indicated that the neighboring geophones were related to each other. Deflection test results were grouped according to the surfacing of AC, soil and concrete. The 10 inch (.25m) geophone was correlated to the 16 inch (.41m) geophone, likewise the 16 inch (.41m) was correlated to the 26 inch (.66m) geophone and so on. Table 18 shows results of this correlation. These results strongly indicate that the shape of the deflection basin is a function of the type of structure, particularly the surface structure.

The first geophone or 10" (.25m) geophone would appear to be a primary importance to understanding the meaning of the dynaflect deflections. Examining the soil site deflection data indicated that the 10" (.25m) geophone was most closely correlated to the top two foot (.61m) weight percent moisture. The following relationship was found.

$$\text{10" DEFLECTION IN MILS OF AN INCH} = 1.1627 + .0361 * \begin{bmatrix} \text{Weight} \\ \text{\% Moisture} \end{bmatrix}$$

$$N = 6; \quad R^2 = .5693$$

Although the correlation is not significant the resultant equation represented a first approximation. Reviewing those existing AC sites it became evident that those sites #3, 5, 7, 8, 9, 23 experiencing 30 or more percent cracking had 10" (.25m) deflection at or above the predicted subgrade deflection. Thus these additional six sites with surfacing were included with the soil sites giving the relationship below.

$$\text{10" DEFLECTION IN MILES OF AN INCH} = 1.2323 + .0368 * \begin{bmatrix} \text{WEIGHT} \\ \text{\% MOISTURE} \end{bmatrix}$$

$$N = 12: \quad R^2 = .6020^*$$

The above relationship is significant at the .05 confidence level and is very close to the first equation, thus indicating that the second equation can be used as an estimate of soil or subgrade dynaflect 10" (.25m) deflection.

TABLE 18

CORRELATION BETWEEN NEIGHBORING DYNAFLECT GEOPHONES; AVERAGE DEFLECTION AT EACH SITE Y = A + BX

1 INCH = .0254 METERS

Y	X	N	R^2	A	B
ASPHALTIC CONCRETE SITES					
16"	10"	26	.8804**	.1580	.5244
26"	16"	26	.7313**	.0517	.5097
37"	26"	26	.8926**	-.0263	.7039
49"	37"	26	.9706**	-.0157	.7510
SOIL SITES					
16"	10"	6	.6163	-.1491	.6068
26"	16"	6	.8874*	-.1625	.7613
37"	26"	6	.9796**	-.0725	.8391
49"	37"	6	.9916**	-.0381	.8777
CONCRETE SITES					
16"	10"	4	.9998**	-.0136	.9458
26"	16"	4	.9969**	.0211	.8141
37"	26"	4	.9976**	-.0004	.8106
49"	37"	4	.9962**	.0054	.7709

After evaluating the deflection on soil or soil like structures the AC sites were examined. Attempts were made to correlate the 10 inch (.25) dynaflect deflection to many variables including equilibrium weight percent moisture, regional factor divided by soil support, inches of AC pavement, inches of base, percent cracking and total thickness of AC and base. The 10" (.25m) deflection appeared to be most closely related to the thickness of AC and the percent cracking. Using these two variables together created the highest correlation giving the following relationship.

$$\text{10" DEFLECTION IN MILS OF} = 1.1954 + .6164 * \text{L06}\left[\frac{\text{\% CRACKING} + 1}{\text{INCHES OF AC PAVEMENT}}\right] \qquad N = 26; \quad R^2 = .6630\ **$$

This correlation is significant at the .01 confidence level. The equation form indicates that as the percent cracking of a given thickness of pavement increases, the deflection also increases. This increase continues until the subgrade deflection is reached at which time the deflection remains relatively constant. The above relationships indicate that the 10 inch (.25m) dynaflect deflection is related to the structure, subgrade condition, pavement thickness and percent cracking. Many correlations using the base course thickness were tried, however, the base course did not appear to improve the correlation. This would indicate that the dynaflect is somewhat insensitive to the reduction in deflection caused by the base course. Another possiblility is that the reduction in deflection due to the base course is so small in comparison to the subgrade and pavement contribution that the dynfalect is incapable of detecting it. The role of the base course on deflection will be explored in the plate bearing analysis.

The development of the soil subgrade and AC pavement deflection equations makes it possible to observe the potential reduction in subgrade deflection due to the slab effect of the AC surfacing. By calculating the subgrade deflection from each AC sites equilibrium weight percent moisture and the surface deflection on top of the uncracked AC for each site, it was possible to calculate the percent reduction in subgrade deflection and relate it to the thickness of AC. The following relationship was found.

$$\text{PERCENT REDUCTION IN SUBGRADE} = 30.0951 + 3.8986 * \left[\text{INCHES OF AC SURFACING WHERE AC } 2.5''\right] \qquad N = 26 \quad R^2 = .5877**$$

Thus as the uncracked AC thickness increases the subgrade deflection would continue to decrease. Extrapolation above or below the limits of AC thickness reported here would be dangerous since it appears only reasonable to believe that an AC thickness decreases the surfacing effect in reduction would tend to go to zero. Likewise as the AC thickness increases the author would expect to see the percent reduction tend to flatten out.

PLATE BEARING ANALYSIS

Plate bearing tests were conducted in December, 1975 and May to November, 1796. The December, 1975 tests were performed by using a very large (60,000 pound, 267 kn) load frame mounted on an oversize mobil trailer. This work was primarily done to determine the suitability of the equipment to perform tests on the highway. Historically, it has been used to test bridge foundations. After two weeks of testing it became evident that such a large piece of equipment would be too cumbersome to use in a highway testing application. With this in mind, separate plate testing equipment was purchased and modified to perform tests in a pit. Modification included welding three large nuts to the 24 inch (.61m) and 30 inch (.76m) plates. Threaded rods with flat plate washers were assembled in such a way that they could be threaded into the large size plates. In this way it was possible to place the plates into the pits and still be able to read the deflection dials at surface level. A special steel outrigger was attached to the rear of a standard size dump truck. The dump truck was filled with cold patching material, which was used to refill the pit.

The testing operation involved the following steps:

1. Close off the travel lane.

2. Select 4 foot (1.22m) by 4 foot (1.22m) test pit site.

3. Perform dynaflect tests at center of the pit site.

4. Read moisture and/or temperature gauges.

5. Perform 12 inch (.30m) plate bearing test on the surface of the highway.

6. Jackhammer out the surfacing.

7. Perform 24 inch (.61m) plate bearing test on top of the base course.

8. Perform an in place volumeter density test and take samples for moisture content.

9. Excavate base and remove samples for laboratory tests.

10. Continue excavation to subgrade level and repeat operation, except subgrade tested with a 30 inch (.76m) plate.

11. Refill the hole with remaining subgrade, base and surfacing materials. Use cold patch to finish top two to four inches (.05 to.10m).

For purposes of safety it was mandatory that the hole be refilled by the end of the work day.

Results of the December 1975 testing are shown in Appendix F, on tables 6F-1 to 34F-1. The table numbers coincide with the site numbers. The 1976 test results are also shown in Appendix F., Tables 2F-2 to 37F-2. Each table gives the site number, name, layers and thickness, date of test, surface temperature. Deflections at each load are shown as well as the rebound deflection. For this case rebound deflection refers to the dial reading after the live load was completely removed. The R2 value is the correlation between the deflection and the live load psi value for a simple linear equation of Y=A + BX. B refers to the slope of the line in terms of psi divided by inches of deflection. For purposes of examining the deflection it was assumed that the best fit line passed through the origin and had a slope of B. Besides the plate bearing results, the dynaflect deflection at the center of the pit is also shown.

Initially, plate bearing deflections were examined in a manner similar to the dynaflect analysis. Those sites where soil or subgrade material were tested with a 30 inch (.76m) plate were correlated to other variables such as moisture, dry density, etc. To do this the 10 psi (69 k Pa) deflection was calculated by dividing the slope B into 10 psi (69 k Pa). Attempts were made to correlate these values to several variables. As in the case of the dynaflect the plate bearing deflection correlated better with weight percent moisture at the time of test. The relationship is the following:

$$\text{SOIL OR SUBGRADE 30'' PLATE BEARING DEFLECTION AT 10 psi} = .0263 + .0023 * \begin{bmatrix} \text{WEIGHT PERCENT} \\ \text{MOISTURE} \end{bmatrix}$$

N = 34 $\qquad R^2 = .3943*$

Since it was not possible to perform dynaflect tests on top of the subgrade, the previously determined relationship between subgrade weight percent moisture and dynaflect deflection was used to estimate the 10 inch (.25m) dynaflect deflection and correlate it to the 30 inch (.76m) plate bearing deflection at 10 psi (69 k Pa). The relationship is the following:

$$\text{SOIL OR SUBGRADE DEFLECTION OF 30'' PLATE BEARING DEFLECTION AT 10 psi} = .0318 + .0537 * \begin{bmatrix} \text{CALCULATED + ACTUAL} \\ \text{DYNAFLECT 10'' DEFLECTION} \\ \text{AT EQUIVALENT MOISTURE} \end{bmatrix}$$

N = 33 $\qquad R^2 = .3750*$

To obtain this value both the calculated dynaflect deflection and tested values were used. For those sites where there was a very large difference (greater than 15 percent), between the average moisture and the moisture at time of test, these three tests were removed from the correlation. The reason for the removal stemmed from the uncertainty about whether the same exact site was tested on the soil by both the plate bearing and dynaflect.

Both the plate bearing and dynaflect deflections were related to the subgrade moisture content and equations were developed for both tests. Correlating these two relationships gave the following:

$$\text{SOIL OR SUBGRADE DEFLECTION OF 30" PLATE BEARING DEFLECTION AT 10 psi} = -.0502 + .0622 * \left[\text{DYNAFLECT 10" DEFLECTION ON SUBGRADE}\right]$$

This equation is very similar to the one found in the previous correlation using actual data.

Besides the subgrade deflection, AC surfacing deflection was also examined. The 12 inch (.30m) plate bearing deflection at 80 psi (552 kPa) was correlated to the 10" dynaflect deflection on top of the AC. The relationship below was found.

$$\text{12" PLATE BEARING DEFLECTION ON TOP OF AC AT 80 psi} = .0107 + .0188 * \left[\text{10" DYNAFLECT DEFLECTION ON TOP OF AC IN MILS OF AN INCH}\right]$$

$$N = 34 \qquad R^2 = .5793^{**}$$

This relationship would indicate that both the dynaflect and plate bearing tests are measuring the same degree of support capability on top of the AC surface. Reviewing the plate bearing data, the magnitude of the 12 inch (.30m) surface deflection appeared to be related better to the percent cracking, inches of AC ratio than to other comparible variables. The relationship, as shown below, was not as strong as the dynaflect correlation, nor as significant.

$$\text{12" PLATE BEARING DEFLECTION ON TOP OF AC AT 80 psi} = .0312 + .0099 * \left[\frac{\text{\% CRACKING} + 1}{\text{INCHES OF AC}}\right]$$

$$N = 32 \qquad R^2 = .3448$$

The plate bearing evidently is not as sensitive to surfacing effects or bound layer effects as the dynaflect. Indeed, the purpose of performing the plate bearing test was to sort out layer effects on deflection.

Besides the above dynaflect tests it would be important to examine how much each layer contributed to reducing the plate bearing subgrade deflection. Since different size plates were used to test each layer it was necessary to convert a 30 inch (.76m) plate to a 24 inch (.61m) and 12 inch (.30m) plate deflection. To do this the 1975 plate deflection using different size plates was used to determine the effect of plate size. According to Yoder (10) effect of plate size is related to the unit load on a plate for a constant amount of deflection and the perimeter over area ratio (P/A) by the following formula.

$$\text{UNIT LOAD ON A PLATE AT CONSTANT DEFLECTION} = n + m * \left[\frac{\text{Perimeter of plate}}{\text{Area of plate}}\right]$$

Using ths formula approach it was possible to determine the unit load at the .050 inch (.0013) deflection for each of the three plate sizes shown on tables 6F-1 to 37F-1 and solve for n and m. Solutions for each site are different, however, the average n and m values were the following:

n = -50 m = 673

Since the P/A ratio is a constant for each plate, the following values would be obtained for a constant deflection of .050 inches (.0013m) and the corresponding unit load would be as follows:

Plate Diameter	Unit Load	B or Slope of Line; Unit Load / .050"	Factor
12"	174	3480	2.81
18"	100	2000	2.50
24"	62	1240	1.55
30"	40	800	1.00

The table above shows that as the plate diameter increases in size, the unit load necessary to obtain the same deflection decreases. Likewise, the slope of the unit load/deflection line, previously called B, decreases. Using these calculated values it was possible to create a set of factors such that given the slope of the 30 inch (.76) plate it would be possible to estimate the equivalent slope on the same material if a different (smaller) size plate were employed. With this knowledge it was possible to estimate the reduction in plate bearing deflection that can be attributed to those layers above the subgrade. Table 19 shows these reductions for each site. Layers are ordered from the top layer just below the plate to the bottom layer just above subgrade or base. As can be seen, most base and surfacing layers reduced the subgrade deflection. Evidently for sites 24, 25 and 37 the base material was no better able to reduce deflection than the subgrade. Thus, the base was equal in worth to the subgrade. All three sites are located in the desert region and the supporting subgrades are of good quality. Indeed, site 24 has the highest slope B value of 510. Likewise, some surfacings have a negative effect on reducing deflection. For sites 14, 17 and 20 the surfacing layers above the base did not reduce the subgrade deflection more than the base. Both site 17 and 20 have CTB base layers which by themselves reduced the deflection substantially. Site 14 has considerable cracking of the surface and a very thick section of base material on a rather low deflection subgrade. With this screening in mind an attempt was made to determine how much each layer contributed to reducing the subgrade deflection. No reliable correlations could be determined so the following table of ranges of reduction was developed.

Layers	Range of Thickness	Range of Reduction in subgrade deflection
AB, SM, SGS	7" - 31"	19 - 71
CTB, SM, SGS	10" - 27"	13 - 87
AC, AB, CTB, SM	16.3"- 33.8"	35 - 83
AC, AB, SM, SGS	11.9"- 33.3"	59 - 92
PccP, CTB, SM	18.0"- 20.0"	93 - 99

TABLE 19

Reduction Subgrade Deflection due to the base course and surfacing as determined by plate bearing test.

Site	Thickness and type of base	% Reduction due to base	Thickness and type of surfacing	% Reduction due to base & Surfacing
2	3" AB, 9" SM	54	7.1"AC	92
3	3" AB, 6" SGS	50	2.9"AC	72
4	3" AB, 12" SM	56	8.5"AC	86
5	4" AB, 15.5" SM	39	4.9"AC	81
6	4" AB, 4" SM	68	7.6"AC	86
7	4" AB, 11" SM	66	6.3"AC	79
8	3" AB, 9" SM	44	8.6"AC	64
9	4" AB, 9" SM	25	3.0"AC	61
11	15" SM	47	9.1"AC	83
12	6" CTB, 4" SM	65	6.8"AC	83
14	6" AB, 10" SM, 12" SGS	38	5.3"AC	36
15	3" AB, 15" SM, 6" SGS	46	8.1"AC	80
17	6" CTB, 6" SM	83	7.3"AC	35
19	6" CTB, 21" SM	58	6.8"AC	84
20	6" CTB, 5" SM	87	3.3"AC, 4" AB	82
21	4" AB, 22" SM	58	3.5"AC	87
22	4" AB, 24" SM	40	4.0"AC	88
23	4" AB, 9" SM, 6" SGS	49	2.0"AC	85
24	4" AB, 9" SM	-6	4.0"AC	63
25	4" AB, 15" SM	-72	4.0"AC	59
26	2" AB, 17" SM, 6" SGS	70	8.0"AC	88
27	4" AB, 18" SM	58	6.0"AC	88
28	6" CTB, 6" SGS	13	8.0" PccP	96
29	6" CTB, 5" SGS	34	8.0"PccP	93
30	4" AB, 21" SM, 6" SGS	71	10.5" AC	90
33	7" SM	19	6.8" AC	74
35	6" CTB, 6"SGS	76	8.0" PccP	99
37	5" SM	-4	13.0" AC	71

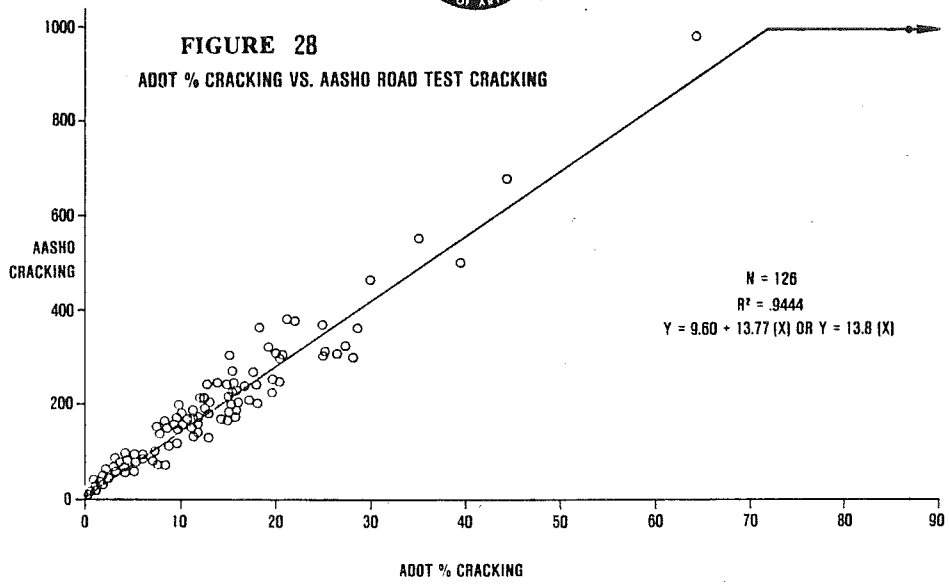
This table shows that substantial reduction in subgrade deflection can be atributed to the base course according to the plate bearing test. It should be recalled that the dynaflect shows little reduction in subgrade deflection due to the base course. This is probably an outgrowth of the mode of testing. In the plate bearing test, greater and greater static loads are applied and the deflection measured. During this process the substantial frictional forces present in the unbound base are mobilized thus giving the appearance of strength. Most of this strength or stiffness is a direct result of confinement and friction. During the dynaflect test, a steady oscillating load, smaller than the plate bearing is applied. This load is probably too small and too transient to mobilize very much of the internal frictional resistance present in the unbound base course. As such, very little reduction in subgrade deflection is attributed to the base course when the dynaflect is used.

The table would also indicate that concrete and CTB structures are capable of achieving more reduction in subgrade deflection than other structures. Interestingly the placing of AC on top of CTB can actually appear as detrimental since the reduction in subgrade deflection is reduced. If this is true it would strongly indicate that the final surfacing in such a system should be as thin as possible, since it is not adding to the support capability already achieved.

In the dynaflect testing the ratio of percent cracking to inches of AC appeared to be strongly related to the magnitude of the deflection. For plate bearing tests this relationship was not found to significantly correlate. This is further evidence of the mobilization of frictional resistance in the plate bearing test and lack of it in the dynaflect test. Since the dynaflect loading is applied to a very small area it is dependent upon the surfaces ability to distribute the stress. As the surface cracks up it begins to look more and more like the unbound base, thus more of the deflection comes from the subgrade. In the plate bearing test, as the amount of cracking is reached the surface begins to look like the base, however, the plate bearing test can mobilize enough frictional resistance even in unbound materials to indicate a reduced deflection. To understand more fully the meaning of the above, performance measurements will be examined.

PERFORMANCE

During the course of this study surface distress data on each site was collected. These measurements included the percent cracking, roughness and rut depth. Percent cracking was determined by estimating the amount of cracking by comparing visual observation to a set of standard photos. This method has been reported in several references (6) (6) (8) and the data is displayed in Table 1G. Roughness was measured using a Mays Ride Meter and is reported in inches of roughness per mile on Table 2G. Rut depth was measured using a four foot straight edge and is reported in Table 3G. From these three measurements it was possible to calculate the present serviceability index (PSI) for each site. To accomplish this the Mays Ride Meter data was



related to the slope variance by taking the Arizona ride index (Figure 16)and setting it equal to PSI. Other terms in the PSI equation were set to zero thus slope variance could be estimated for Arizona's roughness data. In report (8) a method was developed to relate AASHTO road test cracking to Arizona's percent cracking. Figure 28 shows results of this work and gives a relationship in which percent cracking multiplied by 13.8 equals AASHTO lineal feet plus class 2 and 3 area of cracking in 1000 square feet (92.9m2) of pavement. To do this 26 photos were examined and both the AASHTO cracking and percent cracking were determined for each photo. Using the AASHTO slope variance and cracking relationships it was possible to calculate the PSI. Table 4G shows the calculated PSI of each site. Generally, performance is considered the rate of change of surface distress as a function of traffic loading. Table 5G gives the relationship between PSI, cracking, roughness, rut depth and traffic in terms of cumulative 18 kip (80kn)loads. The values shown represent the slope of the distress traffic correlation.

The larger the number (positively or negatively) the faster the rate of change and the quicker an undesirable level of distress will be reached. Besides site number there is a years of service field which indicats the particular structure under study. Those sites built 30 or 40 years ago represent rather complex structures and only one period of service was selected. Generally seal coats and ACFC's were not considered to have much influence on performance. Some sites due to their complex structure, history or lack of change in a particular distress measurement could not be given a performance slope value. The slopes on Table 5G do not by themselves truly represent performance. Rather the number of years to a particular level of distress is the best measure of performance since this represents the target the designer and builder were using to create the particular structure. Table 6G gives the number of years to various levels of distress. The levels selected were:

Percent cracking	=	20 percent
Roughness	=	256 inches/mile
Rut Depth	=	.50 inch
PSI	=	2.5

The years shown in parenthesis are the extrapolated number of years to a given distress condition, however, those very large values have been reduced to more realistic values. All extrapolations of more than 40 years (approximate length of time under study) were reduced to 40 years. Reduced values were used in all future calculations. The average years to the selected levels of distress are:

20 percent cracking	20 years
256 roughness	25 "
.5" rut depth	27 "
2.5 PSI	19 "

Those highways built since 1962 used the AASHTO interim guidelines (1) and were built to last approximately 20 years before a PSI of 2.5 would be reached. Sites number 19 through 22, 24 through 30, 33 and 35 through 37 were designed and constructed by the AASHTO interim guidelines (1) to last 20 years. The table below gives a score of how closely these projects will come to the 20 year target.

PROJECTS DESIGNED BY AASHTO DESIGN GUIDELINES

	DESIGN + CONSTRUCTION PERFORMANCE TARGET IN YEARS	NUMBER OF PROJECTS THAT REACHED 2.5 PSI WITHIN TARGET	LESS THAN 20 YEARS	MORE THAN 20 YEARS
	20 ± 1	1	1	
	20 ± 2	3	2	1
	20 ± 3	5	4	
	20 ± 4	6	5	
	20 ± 5	6	5	
	20 ± 6	7	6	
	20 ± 7	8	7	
	20 ± 8	9	8	
	20 ± 9	10	9	
	20 ± 10	11	10	
TOTAL	0 to 110	15	12	3

As can be seen, approximately 50 percent of the projects will fall between 13 and 22 years, 75 percent will fall between 10 and 22 years and all projects will last somewhere between 9 and 40 years.

As can be seen, approximately 50 percent of the projects will fall between 13 and 22 years, 75 percent will fall between 10 and 22 years and all projects will last somewhere between 9 and 40 years.

This analysis assumed that years of performance less than 20 was acceptable, however, another arguement would be that those highways designed by AASHTO should last 20 or more years. If this approach were taken, only three projects out of 15 or 20 percent would last beyond 20 years. What about those projects designed and built before AASHTO? How would their performance compare to those built after AASHTO?

PROJECTS DESIGNED AND BUILT BEFORE AASHTO

DESIGN & CONSTRUCTION PERFORMANCE TARGET IN YEARS	NUMBER OF PROJECTS THAT REACHED 2.5 PSI WITHIN TARGET	LESS THAN 20 YEARS	MORE THAN 20 YEARS
20 ± 1	1		1
20 ± 2	3	2	
20 ± 3			
20 ± 4			
20 ± 5			
20 ± 6	4	3	
20 ± 7	7	6	
20 ± 8	8	7	
20 ± 9			
20 ± 10	10	8	1
TOTAL 12 to 174	14	8	6

Sites 1 through 5, 7, 9 and 12 through 18 were used to create the above table. Interestingly, 50% of the projects fall between 13 and 21 years which is very close to the AASHTO design and 75 percent fell between 10 and 21 years again very close to the AASHTO performance. Those projects lasting more than 20 years represented 43 percent of the total or about double that obtained using AASHTO. From just this cursory analysis, it would appear that pre and post AASHTO usage has had little impact on performance. This is not to say AASHTO is bad, indeed such performance figures presented here might represent the best that is possible under any system. Since the AASHTO method is dependent upon traffic loading, is there any relationship between years of performance and level of loading? In this project four levels of 18 kip (80kn) loading occurred as shown on Table 20. The average number of years for each form of distress is shown in the table below for the four levels of 18 kip (80kn) loading.

TABLE 20

1978 18 kip LOADING	AVERAGE NUMBER OF YEARS OF PERFORMANCE TO LEVEL OF DISTRESS # OF SITES	% CRACKING	ROUGHNESS	RUT DEPTH	PSI
100,000	3	24	33	33	31
100-200,000	9	30	28	24	22
200-300,000	13	20	21	31	17
300,000+	4	12	26	29	18

Sites number 6, 8, 10, 11, 23, 31, 32 and 34 were not considered since they represent overlays built since 1963, stage construction or grade and drain projects. These same sites were not considered in the previous analysis. This very rough analysis would indicate that as traffic goes up the year to 20 percent cracking, 256 inches/mile roughness, .5 inch rut depth and 2.5 PSI go down.

Besides traffic it is possible that some systematic change in performance has taken place with time. To investigate this possibility the year that a site was constructed was examined in relation to its performance. Sites were grouped into several categories as shown below.

YEAR BUILT	AVERAGE NUMBER OF YEARS OF PERFORMANCE TO LEVEL OF DISTRESS # OF SITES	% CRACKING	ROUGHNESS	RUT DEPTH	PSI
1940-1957	4	12	33	35	28
1958-1962	7	20	23	37	19
1963-1968	12	23	27	26	19
1969+	6	40*	22	14	17

* ONLY ONE SITE

The above numbers are consistent with material property trends examined early in the report. Surfacing thickness has increased with time and in addition the voids filled with asphalt have increased with time (Table 4). The net result of this would be to increase the cracking performance life and decrease the rut depth performance life. For average PSI and roughness the average time to distress is becoming shorter.

Time trends are indicative of systematic changes outside the influence of the design equations. Such actions as use of drum dryer plants, reduction in hot plant temperature, change in the asphalt specifications, concrete highways and testing may also be contributing to the observed time trends.

Besides traffic and time, location of the site in terms of region, soil support and subgrade moisture might also be of importance. The table below shows the average effect of these three factors on performance.

AVERAGE NUMBER OF YEARS OF PERFORMANCE TO UNACCEPTABLE LEVEL OF DISTRESS

REGIONAL FACTOR		# OF SITES	% CRACKING	ROUGHNESS	RUT DEPTH	PSI
0 - 1.7	Desert	10	23	29	27	19
1.8 - 2.7	Transition	12	20	29	32	28
2.8+	Mountains	6	24	17	23	13

AVERAGE NUMBER OF YEARS OF PERFORMANCE TO UNACCEPTABLE LEVEL OF DISTRESS

SOIL SUPPORT	# OF SITES	% CRACKING	ROUGHNESS	RUT DEPTH	PSI
0 - 3.3	3	24	24	30	19
3.4 - 6.6	14	22	26	27	19
6.7 - 10.0	11	20	26	31	22

AVERAGE NUMBER OF YEARS OF PERFORMANCE TO UNACCEPTABLE LEVEL OF DISTRESS

SUBGRADE-MOISTURE	# OF SITES	% CRACKING	ROUGHNESS	RUT DEPTH	PSI
0 - 10.0	11	25	31	27	24
10.0 - 18.0	10	19	22	29	17
18.1 +	6	20	25	31	20

Regional factor, soil support and equilibrium moisture are all very interrelated as shown in this report. Separating them out individually does not indicate any special trends, which reinforces their interrelationship. Evidently time has had the greatest systematic influence on performance. Traffic seems to be next in influence. Subgrade moisture and soil support are also important. Subgrade moisture above 10.0 percent generally reduces the percent cracking, roughness and PSI life.

This review of performance was not meant to develop a new method of design or construction but rather to demonstrate to the reader what kind of performance really occurs. Some of the relative relationships observed are examples of the complexity of relating performance to any variable or group of variables. Undoubtedly, the design and construction of highways has been an evolutionary process, with a mixture of engineering, economics and marketing. Historically, engineers have found solace in the fact that problem highways can always be overlayed thus solving the problem. In reality, highway problems can, and often are, rooted in the subgrade materials an the environment.

DISCUSSION

This project has looked at a myriad of factors that influence the highway's performance. A variety of relationships have been found which should be of use to the highway designer and builder. During this project a large number of miles of highway were intensively observed, monitored and tested. Throughout this process a set of impressions about highway design, construction and performance have been accumulated. Highway design in Arizona was and is an empirical process which currently uses the AASHTO interim guidelines (1). These guidelines are based on the premise of a 20 year design life. As has been shown, designing and building a highway that lasts 20 years before a 2.5 PSI is reached is very difficult and occurs probably less than 50 percent of the time. It would be easy to blame AASHTO, however, even pre-AASHTO projects performed similarly. It would be easy to blame the designers, however, most of today's highways in Arizona have been built in a rush. When the interstate program began, the size of Arizona's program exploded. The main concern was quantity of miles of highway. Only very recently has it been possible to even determine the level of performance. It would be easy to blame the construction operation, but here again the rapid growth of highways and the tremendous changes in specifications as well as contractor capability has also had a very large influence. As an observation, it appears that the real problem resides in the process of feedback. No means existed then or now to translate experience, good or bad, from the field back to the designers and on to the builders. Not that such communication does not exist, rather it has oftentimes been shunted in the belief that one side is right and other side is wrong or that no problem existed. Now is an excellent time to examine the design, construction process in depth. New methods of testing materials and design are currently available and were demonstrated at the 4th International Conference on Pavement Design (9). It is suggested that these methods be given an opportunity.

FINDINGS

The following finding have been drawn from this project.

1. Asphalt ages rapidly in Arizona. The percent of penetration decreases 50 percent in one year, 75 percent in two years and reaches a relatively stable level within three years. Viscosity increases in a similar manner.

2. Field asphalt content is generally more than the amount at construction, due to the application of tack coats, flush coats and seal coats.

3. The field density of AC mixes at time of construction is generally four pounds per cubic foor (64 kg/m^3) less than the Hveem T.I. laboratory compactor density.

4. The maximum theoretical density of AC mixes is generally ten to twelve pounds per cubic foot (160 to 192 Kg/m^3) heavier than the Hveem T.I. compactor density.

7. AC mixes designed since 1969 were to be in the 4-6 percent air void level after two years of traffic. Only about 20 percent of the projects sampled in this study actually achieved the 4-6 percent level, the bulk of them having voids greater than 6 percent.

8. BTB mixes performed similarly to AC mixes in terms of densification. Generally the BTB mixes had air voids of about 22 percent after two or more years of traffic.

9. CTB mixes densified under traffic reaching about 106 percent of the maximum proctor density. The moisture of the CTB increased after construction to approximately 2 percent wetter by weight two or more years after construction.

10. At time of construction the unbound base course (AB, SM, SGS) has a density which is about 102 percent of the maximum proctor density. After two or more years under traffic, the base course density increase, such that low density materials increase to about 110 percent of the proctor maximum density, whereas high density materials increase to about 99 percent. This is very consistent with the densification of AC mixes.

11. At time of construction the base course moisture is about 3.3. percent dryer than the optimum moisture. With time the AB and SM base course layers generally become dryer. Thus after two or more years under traffic the base course can be from 3.0 to 10.0 percent dryer than optimum.

12. Subgrade densities are about 97 percent of the proctor maximum dry density at time of construction. After two or more years under traffic subgrades generally slightly increase in density, although a large number of projects decreased in density to about 91 percent of the proctor maximum density.

13. Subgrade moisture at time of construction is generally about 1 to 6 percent dryer than optimum. Two or more years under traffic the field moisture increased dramatically above the construction moisture. The average subgrade moisture was about 90 percent of optimum, with 10 sites with moisture above optimum. Subgrade moistures do indeed increase with time.

14. The Hveem R value density at 300 psi is virtually the same as the proctor maximum dry density of subgrade soil. In addition, the R value moisture is virtually the same as the proctor optimum moisture.

15. The T-99 proctor maximum dry density and optimum percent moisture are very well related and the relationship in Arizona is virtually identical to the Louisiana family of curves line.

16. The R value test is related to the -2 micron (clay) fraction of a soil sample. In addition the -200 and PI are very well related to the -2 micron material. This helps explain why both the -200 and PI are so well related to the R value.

17. The temperature of the air, pavement surface and layers under the surface were determined. It was found that these temperatures are very well related to elevation. In addition, considerable freezing and freeze thaw occurs in Northern Arizona.

18. Moisture of the subgrade under the highway is generally more (as much as 4 to 5 percent by weight) than the natural soil moisture.

19. Subgrade moisture varies less over time than the natural moisture.

20. Subgrade moisture can be predicted reasonably well from the ratio of regional factors to soil support or the T-99 proctor maximum density.

21. The pavement surface temperature can be used to estimate the pavement mid-depth temperature.

22. The dynaflect deflection of AC surfaces does not appear to be very well related to the temperature of the AC.

23. AC overlays do reduce the dynaflect deflection, however, the use of the average deflection taken over several time periods will reveal this reduction much better.

24. Neighboring geophones used in the dynaflect deflection are very well related to each other. These relationships would strongly indicate that the dynaflect deflection basin is related to the type of structure.

25. The deflection of the first dynaflect geophone (10 inch geophone) is related to the subgrade moisture. Better than any of the other four geophones.

26. The 10 inch geophone deflection when taken on top of an AC pavement is related to the ratio of the percent cracking divided by the inches of AC thickness.

27. The dynaflect evidently does not register the effect of the unbound base. This could be due to the dynaflect size of loading or area of loading. In this sense the dynaflect does not mobilize sufficient stress in the base course to indicate its true value in reducing deflection.

28. The percent reduction in subgrade dynaflect deflection decreases as a function of the AC surfacing thickness.

29. The deflection of a 30 inch plate on subgrade was found to be related to the moisture content. From this, the 30 inch plate bearing and dynaflect deflection of the subgrade were related to each other.

30. 12 inch plate bearing deflections on top of an AC pavement are very well related to the 10 inch geophone dynaflect deflection on top of the AC.

31. The plate bearing test, unlike the dynaflect, indicated a very large reduction in subgrade deflection due to the unbound base course. This is reasonable since the plate bearing plate size and load are of sufficient magnitude to mobilize considerable stress in the base layer.

32. Those projects designed using the AASHTO road test equations do not appear to be performing any differently than those designed and built before the AASHTO road test. In general 50 percent of those projects designed and built after AASHTO road test took 13 to 22 years to reach the 2.5 PSI level. 75 percent of these projects reached the 2.5 PSI level between 10 and 22 years. Pre-AASHTO projects performed in a similar manner.

33. Although traffic is an important part of the AASHTO design, large difference in performance still have occurred due to different traffic levels.

34. Systematic evolutionary changes in the highway specifications, construction techniques and materials over time have all contributed to a change in performance. These actions are outside of the purview of the AASHTO design equiations.

35. Regional factor, soil support and equilibrium subgrade moisture all contribute to different levels of performance and are interrelated. Moisture contents above 10.0 percent can contribute to shortening the life of the pavement.

CONCLUSION

This project was created to determine temperature and moisture conditions in Arizona pavements. Several valuable relationships have been found which should be of use to designers, builders and maintenance forces. Methods have been determined to estimate both temperature and moisture.

Regional factor and soil support were both found to be interrelated to the future equilibrium moisture content. Since moisture content is likewise related to the in place dry density as well as the subgrade deflection it takes on great importance. Thus equilibrium subgrade moisture can be said to be the environmental factor since it constitutes the net results of the forces of nature and man. Historically, ADOT has sampled the top six inches (.15m) of the subgrade soil as subgrade. In this study the top two to four feet (.61 to 1.22m) acted more as the true subgrade.

Unbound base course materials have historically been of good quality. Typically, they have densified and dried out under traffic. Such base courses represent material that can readily benefit by the introduction of the binder material which would further reduce deflection by both the plate bearing ad dynaflect test.

AC and concrete surface courses can reduce the subgrade deflection, however, their long term performance can be seriously impaired if they are improperly designed. Historically, AC mixes have become richer in asphalt and lower in voids filled with asphalt. The results has been a shift from cracking to rutting distress. This might not be bad if relatively thin AC pavements (5 inches or less) were being built, however, when very thick AC pavemets (10 inches or more) are being built it could be quite damaging. Concrete pavements generally experience little or no rutting and very little cracking, therefore, most of their distress is derived from roughness. Concrete pavements are rough and become rougher all too soon. Evidently the independent slabs are moving in relation to each other. Reduction in the movement would be of great help in improving the performance of these pavements.

Overall the AASHTO design equation has done little better in providing 20 year pavements than the pre-AASHTO equation. This lends credence to the belief that forces outside of the design equation can seriously reduce future performance. These outside elements involve specifications, construction and contractor capability.

Generally one half of all AC pavement are overlaid or seal coated by the 16th year. Of these projects about one half receive a second overlay or seal coat by the 33rd year.

RECOMMENDATIONS

It is recommended that:

* The environmental factor be used to estimate the future pavement subgrade moisture and dry density. These values be run as part of the R value test as the most likely future worst condition.

* The subgrade be redefined to mean the top two feet of the subgrade soil. Efforts should be made to incorporate the highest density material possible within this layer, since this will reduce the long term moisture content.

* Compaction levels for the subgrade be raised to 100 percent of T-99 proctor as this also will increase density thus reducing the moisture.

* The dynaflect deflection not be corrected for temperature or if a temperature correction is desirable for other reasons, use the equation in the report.

* Freezing index, freeze thaw cycles and depth of frost penetration should be considered in the design and testing process. In particular aggregates and mixes should be tested to determine their suitability to the action of freezing and thawing. In addition for those regions of the state experiencing freezing and thawing this should be considered in the AC stripping test.

* Recycling of pavements has become quite popular, however, the underlying unbound base courses offer a clean, durable and well graded material suitable for either cement treated, lime fly ash treated, bituminous treated or emulsion treated base. Use of this material for this purpose would improve the overall support capabilities of the structure.

* AC designs should be matched to the thickness of the pavement. For thin pavements (less than 5 inches) high binder content and low voids filled are desirable, however, for thick pavements (more than 5 inches), lower binder content and lower voids filled or structures containing both BTB and AC is recommended.

* Concrete pavements become rough too quickly. ADOT should continue to examine both design and construction techniques in particular some form of joint control which could be used to reduce the roughness. If this is not possible and only aggregate interlock can be used to keep joints from moving, the aggregate should be of a quality to resist the action of freeze thaw as well as other forces.

* The AASHTO design method as practiced in Arizona should be totally reexamined.

* The greatest impact on highway performance has come from outside the design equation. Review of specifications, construction and contract administration should take place to determine inconsistencies. A practical example would be the relationship betwen R value and the proctor maximum density. Altghough the R value test is supposedly emulating the worst possible field condition it has the same dry density as the proctor maximum density. For subgrade soils only 95 percent of the proctor maximum density is needed to meet specifications and considered good, even though it is 5 percent below the R value test density which is considered the worst condition. Another example would be designing for the worst soil support and then allowing materials of this quality to be used, rather than designing to improve subgrade quality.

* New design methods and laboratory tests using principles of mechanics have been presented at the 4th International Conference on Pavement Design. These methods should be given a trial application on future projects to see if they are capable of improving the reliability of pavement design and construction.

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