NCAT Report 90-02

EVALUATION OF BITUMINOUS PAVEMENTS FOR HIGH PRESSURE TRUCK TIRES

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INTRODUCTION

In recent years several states in the United States have experienced premature rutting of hot mix asphalt (HMA) pavements due to increased traffic loads and/or increased truck tire pressures. Current pavement designs are based on 18,000 lb-axle loads and contact pressure of 75 psi. Recent legislation has increased the legalized loads. There are no laws governing the maximum tire inflation pressure.

Recent surveys in Illinois and Texas indicate that the tire pressures have increased substantially. Tire pressures averaged 96 psi with a maximum of 130 psi in the Illinois survey. Texas survey showed an average of 110 psi with a maximum of 155 psi. Recent studies in Texas also indicate that a tire inflation pressure of 90 psi can cause very high pressures (over 200 psi) near the tire shoulder region.

The Pennsylvania Department of Transportation's first major asphalt pavement rutting was experienced on Interstate 70 in Washington County during early summer of 1986. Additional cases of rutting have occurred since that time. A special provision for designing the HMA mixtures for heavy duty pavements was developed by the Pennsylvania Department of Transportation (PennDOT) and implemented in 1987. However, there was a need to evaluate several heavy duty pavements constructed in the past with and without the special provision so that the pavement properties (materials, mixture design, construction and post construction) which typify good and bad pavements could be identified. This will facilitate further changes to PennDOT's current material specifications, mix design, and construction procedures to cope with the increased truck loads and/or tire pressures. This will also identify which items contained in the special provisions are significantly more effective in minimizing or eliminating premature rutting.

HISTORY AND BACKGROUND

About 57 percent or 25,000 miles of interstate highways in the United States have HMA surface. Premature rutting of heavy duty HMA pavements was first experienced as a major problem in WASHTO (Western Association of State Highway & Transportation Officials) states during the late 1970s. WASHTO members discussed this problem in 1983 and issued a report in 1984 (*1*). Before this time most states in the eastern United States including Pennsylvania had not experienced any significant rutting. First major incidence of rutting was reported in 1984 in Virginia and Florida. Pennsylvania, Tennessee and New Jersey Turnpike experienced rutting for the first time in 1986. Illinois had experienced some rutting on 3-inch HMA overlays over PCC pavements since 1979. However, the problem in Illinois became more serious in 1984.

All these developments made the asphalt paving technologists wonder as to why the rutting is being experienced now when in the past the HMA pavements had an excellent performance record on heavy duty highways. The change in scenario can be attributed to the following factors as a minimum:

- Increased truck volume
- Increased truck loading
- Increased tire pressure
- Increased number of HMA overlays over existing PCC pavements
- Channelized traffic during construction

The following FHWA highway statistics (2) for a typical rural interstate highway in the United States for the period 1970 to 1984 are of interest.

	Increase during 1970 to 1984
No. of total vehicles	20%
No. of trucks	49%
No. of 18-kip equivalent single axle loads (ESALs)	126%

During these 14 years, the number of total vehicles including cars increased by 20 percent. However, the significant thing is that the number of trucks (90 percent of which have 5 or more axles) increased by 49 percent during this period. Even more significant than this is that the number of ESALs (equivalent 18-kip single axle load applications per day) increased by 126 percent. That is an average increase of 9 percent per year. What it means is that the load anticipated in 20 years was applied to the pavement in 8-10 years' time in many cases. In other words, the design life of the HMA pavement was reduced from 20 to 8-10 years.

Along with the increased truck loading increased tire pressures were also experienced in recent years. During AASHTO Road Tests in the 1950's the tire pressure was 75-80 psi which formed the basis for pavement analytical distress models. However, recent surveys in Illinois and Texas show that the average tire pressures have increased significantly. In Illinois' study, the average tire pressure was 96 psi and the maximum was 130 psi. In Texas' study, the average tire pressure was 110 psi and the maximum was 155 psi.

Not only has tire pressure increased but its nonuniform distribution over the contact area has compounded the problem (3). Figure 1 shows an actual pressure print of a loaded tire. Lighter color areas indicate higher pressure compared to gray areas. The pressure in the shoulder region of the tire is more than twice the inflation pressure. This non-uniform pattern of pressure has resulted from stiffer sidewall and shoulder design of the tires. Incidence of a dual rut associated with the tandem tires can be attributed partly to this changed tire design.

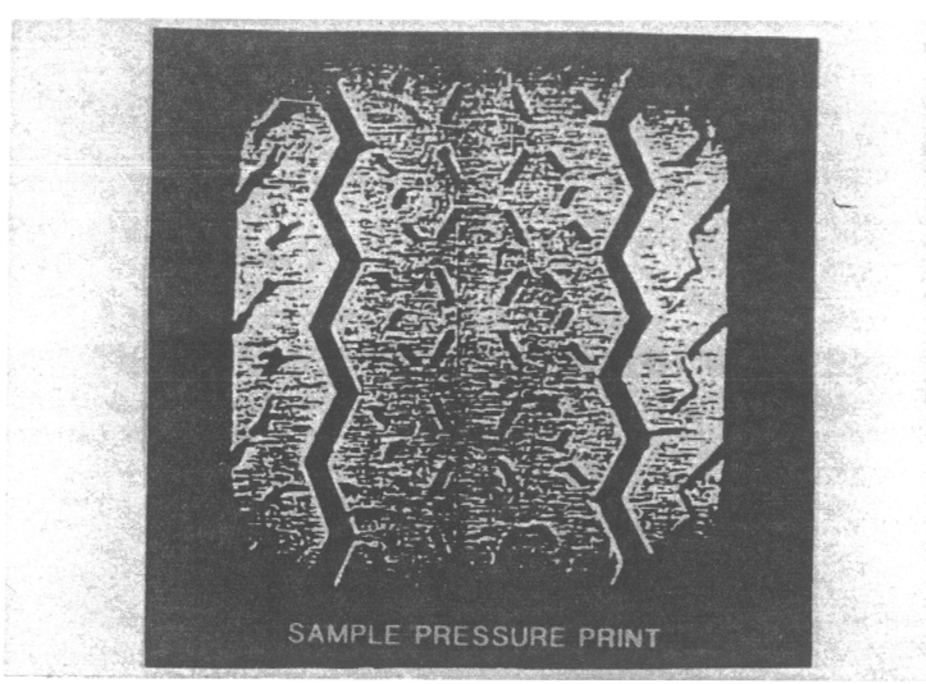

Figure 1. Pressure Print of a Loaded Tire

There was so much concern about this changing scenario that the AASHTO organized a National Workshop on High Pressure Truck Tires which was held in Austin, Texas in February 1987. The workshop concluded that the load changes remain outside the influence of the highway engineers. However, design and construction of HMA pavements could be adapted to meet the changing conditions.

It is quite possible that we will see a further increase in the tire pressures. Increased tire pressures reduce the rolling resistance and, thus, reduce fuel costs for the trucking industry. Tire technology already exists to design and manufacture tires having higher inflation pressures. The U.S. Navy currently uses tires on their fighter planes that are inflated to 400 psi. An increase in the number of axles, say from five to seven, is envisioned. This increase in number of axles will reduce pavement stresses and thus will be a relief to the highway design engineer. This is already being considered at the national level.

Many PCC pavements of interstate and primary highways which were designed for 20 years are deteriorating before the design life. Recent years have seen increased HMA overlays over these existing PCC pavements. Illinois' study (*4*) has shown that excessive shear deformation occurs in HMA overlays underlain by PCC pavements. Channelized traffic can also induce rutting just after construction especially if the mat is still hot. Figure 2 shows two-way bumper to bumper traffic on two recently overlaid lanes of an interstate highway while the other two lanes are under construction.

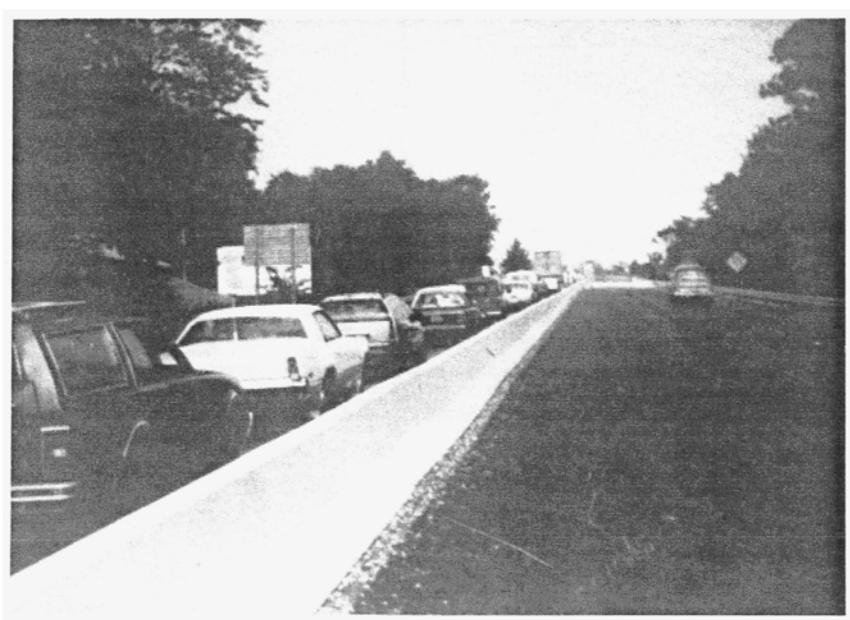

Figure 2. Channelized Two-Way Traffic During Construction

As mentioned earlier, PennDOT's first major HMA pavement rutting was experienced on Interstate 70 in Washington County during early summer of 1986 (Figures 3 and 4). Additional cases of rutting were reported later. The Department developed a special provision for designing the HMA mixtures for heavy duty pavements to minimize rutting (*5*). This special provision was made applicable in 1987 to the wearing, binder and base course mixtures used on the main line, ramps and cross-overs of all Interstate highways, and other highways carrying more than 20,000 ADT (average daily traffic) or more than 1,000 daily 18 tip equivalent single axle load (ESAL) applications. The salient features contained in the special provision are:

1. Use of larger size aggregate for binder and base course mixtures. Previously, AASHTO No. 57 (1 inch to No. 4) aggregate was generally used. Use of AASHTO No. 467 (1-1/2 inch to No. 4) was recommended.

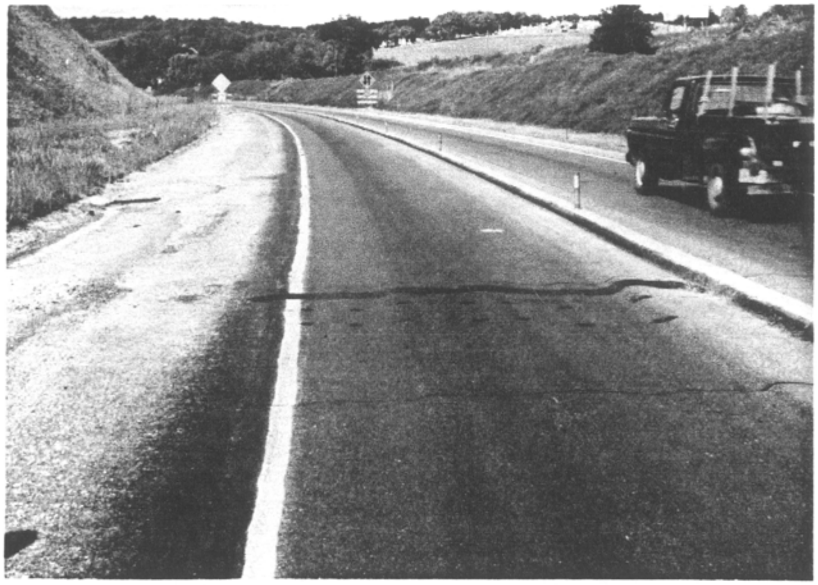

Figure 3. Rutting on 1-70 Washington County (1986)

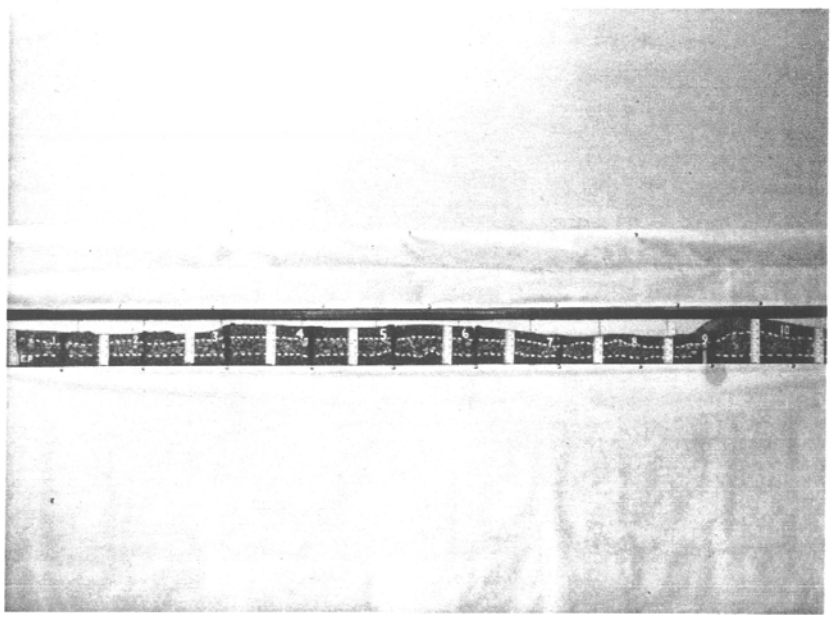

Figure 4. View of the Pavement Slab from I-70

2. More angular coarse aggregate. Previously, the gravel coarse aggregate in the binder course was required to have at least 50 percent with one fractured face. This was increased to at least 85 percent with two or more fractured faces which was the same as the wearing course.
3. At least 75 percent manufactured sand in the fine aggregate. Research in the past has shown that the manufactured sand is generally angular and its incorporation in the mix resists rutting.
4. At least 4 percent minus No. 200 fraction in the mixture to stiffen the asphalt binder. The maximum permissible percentages of minus No. 200 in the job-mix formula was kept the same as previously specified (6 and 5 percent for wearing and binder mixtures, respectively).

5. Marshall specimens to be made with 75 blows on each side.
6. Minimum voids in the mineral aggregate (VMA) was kept the same as previously specified (16 and 13 percent for wearing and binder mixtures, respectively).
7. Marshall stability to be 2,150 lb. minimum for all courses. Flow of 6 to 16 was kept the same.

Of the 34 total pavements evaluated in this research project, seven pavements were constructed using the preceding special provision for heavy duty pavements.

Based on the recommendations of the PennDOT/industry task force on rutting, the special provisions were revised in October 1988 as follows:

1. Marshall stability at 140°F not less than 2000 lbs. in the job-mix formula and not less than 1800 lbs. during production,
2. Percentage of unfilled voids to be 4.0 and 4.5 percent for wearing and binder mixes, respectively, for the reviewed job-mix formula. Voids in plant compacted Marshall specimen not to exceed the master range of 3.0 to 6.0 percent for wearing, and 3.5 to 6.5 percent for binder mixes. Compliance for this criteria is acceptable if 90 percent of the determinations per project are within the stated tolerance band,
3. The binder course mix gradation revised so that 100 percent passes 2" sieve, 95 to 100 percent passes 1 1/2" sieve, 85 to 95 percent 1" sieve and 40-65 percent passes 1/2" sieve in the job-mix formula.
4. The requirement for minimum minus 200 content of 4 percent was deleted, and
5. The wearing course should not be placed if it will be subjected to high temperatures and channelized traffic for extended periods of time until all binder courses have been completed.

OBJECTIVES

The primary objective of this research project was to evaluate 34 in-service heavy duty pavements to identify the material properties, mix design parameters, pavement construction properties, and pavement in-service properties which are responsible for the premature rutting (permanent deformation) of some HMA pavements. A threshold analysis was performed to determine if the measured parameters have a threshold value which separates acceptable and unacceptable performance. The preceding information will be helpful to PennDOT in revising the specifications for heavy duty HMA pavements.

DATA COLLECTION, SAMPLING AND TESTING PLAN

Thirty-four (34) heavy duty pavements encompassing poor to excellent performance in terms of rutting were identified by Penn DOT. All pavements except two (Sites 5 and 31) met PennDOT's criteria for heavy duty pavements mentioned earlier.

A special effort was made in selecting pavements which had the best and most readily available test data, specifically projects constructed under the Department's restricted performance specifications. This was done to facilitate the correlation of as-constructed pavement properties with rutting.

Data Collection

An attempt was made to collect the following data on all projects from the Engineering District Offices (District Materials and Construction Engineers) and the Materials Testing Division (MTD) of the PennDOT's Bureau of Construction and Materials.

1. State Route (S.R.) Number and stations

2. Geographical location
3. Average climatic conditions
4. General topography (including grades)
5. Average daily traffic (ADT) including the percentage of trucks, and 18-kip equivalent single axle loads (ESAL) per day.
6. Details of underlying pavement structure such as, type (flexible or rigid, base course, binder course, wearing course), thickness, material type and condition.
7. Details of new construction or overlay such as, thickness and type of bituminous concrete base course (BCBC), leveling or scratch course, binder course and wearing course.
8. Dates of construction for various courses in all lanes and the prevailing weather renditions.
9. Traffic management during construction such as, the time and duration when certain lanes or segments of the roadway were subjected to two-way or increased traffic intensity, use of barrier or curb, extent of channelization, cross-overs, and how soon road opened to traffic.
10. Job-mix formula (JMF) including the Marshall design data at various asphalt contents used for determining the optimum asphalt content, aggregate sources and types (stone, gravel, slag, manufactured or natural fine aggregates, crush count, etc.), and asphalt grade used.
11. Construction data including daily plant reports to determine the following:
    (a) Construction dates and prevailing weather conditions
    (b) Type of mix plant
    (c) Compaction equipment and rolling procedures
    (d) Daily mix and Marshall test data such as, extraction results, hot bin gradation, specimen specific gravity, theoretical maximum specific gravity, percent air voids, percent VMA, stability and flow
    (e) Penetration and viscosity of asphalt cement used
12. Special features of the project such as, long steep grades, and intersections or exits causing frequent slowing or stopping of traffic.
13. RPS (Restricted Performance Specification) data showing the mix composition and percent compaction obtained in all lots and sublets of the project along with the delineation (starting and ending stations) of lots, and location of sublot samples (particularly core samples)
14. Quality assurance (QA) sample test data

Complete data could not be obtained for some pavements which are very old and, therefore, records are not available.

Sampling and Testing Plan

Eleven 6-inch diameter cores were to be obtained from a <u>representative</u> one lane mile segment (travel lane) of each project (Figure 5). However, Penn DOT decided to select the worst segment (maximum rutting) of the project for taking these cores. On some projects, considerations for sight distance and safety precluded coring the worst sites. All cores were taken during spring of 1989. Five cores numbered C1 -C5 were obtained at random locations from the inside wheel track of this segment. These five cores from each project (total 170 cores) were tested as follows:

1. Thickness of layers (all cores);
2. Bulk specific gravity (all cores);
3. Theoretical maximum specific gravity;
4. Extraction - asphalt content and gradation (all cores);
5. Recovered coarse aggregate (retained on No. 4 sieve) - crush count (one core); and
6. Recovered fine aggregate - particle shape and texture (determined in terms of b percent void content using the National Aggregate Association procedure given in Appendix A).

Five additional 6-inch diameter cores (C7-C11) were obtained across the pavement two feet center to center at the worst (maximum rutting) location of the selected segment as shown in Figure 5. The testing program for these cores is shown in Figure 6. Essentially, the following tests were run:

1. Bulk specific gravity of layers (all cores);
2. Static unconfined creep test (two cores); and
3. Bulk specific gravity, stability and flow tests were run on two specimens each recompacted by three compaction methods: (a) Gyratory testing machine (GTM), (b) Rotating base, slanted foot mechanical Marshall compactor, and (c) Static base conventional mechanical Marshall compactor.

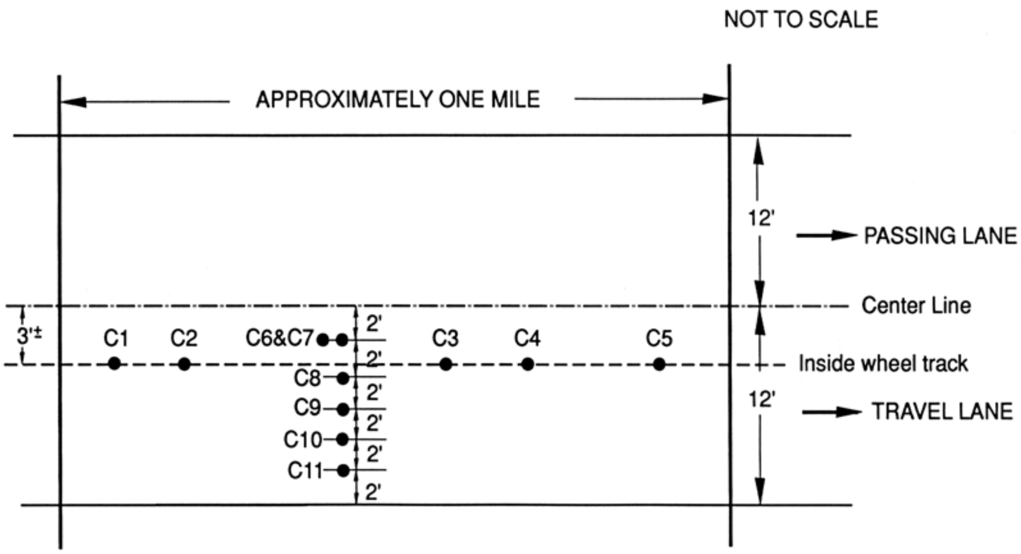

Figure 5. Core Sampling Plan

The thickness of all layers in Cores C7-C11 was accurately measured before sawing the layers. These thicknesses were used to obtain the profiles of the underlying layers once the surface profile was established. Bulk specific gravity of all layers in these cores was obtained to determine the in-place voids in the total mix (VTM) at each location.

Two cores were subjected to unconfined static creep test using the Shell method using the MTS machine shown in Figure 7. The cores were heated to 104°F for two hours and then loaded in the environmental chamber (104°F) of the MTS for testing. Teflon disks, approximately 1/1 6 inches thick by 6 inches diameter, were placed at the ends of the sample to reduce the effects of friction. The samples were then preloaded to 30 lbs. for two minutes. At the end of the preload the load was increased to 425 lbs. and held constant for one hour and the vertical deformation of the sample was recorded continuously. After one hour the load was removed. The rebound was not recorded.

After the creep tests on two cores all five cores were warmed to crumble the mix and then reheated to 275°F for compaction. Two 4-inch diameter specimens each were compacted using the three compaction methods mentioned earlier. The Gyratory testing machine (U.S. Corp of Engineers) or GTM shown in Figure 8 was used. The machine was set at 120 psi (typical of today's truck tire pressures), one degree angle, and 300 revolutions. Past experience has shown that this compactive effort provides a specimen having density approximately equal to that observed in the field after several years of traffic. During recompaction of the mixture a Gyratory Shear index (GSI) is determined from a printout of the mix strain. Past studies have also shown that the GSI correlates very well with rutting.

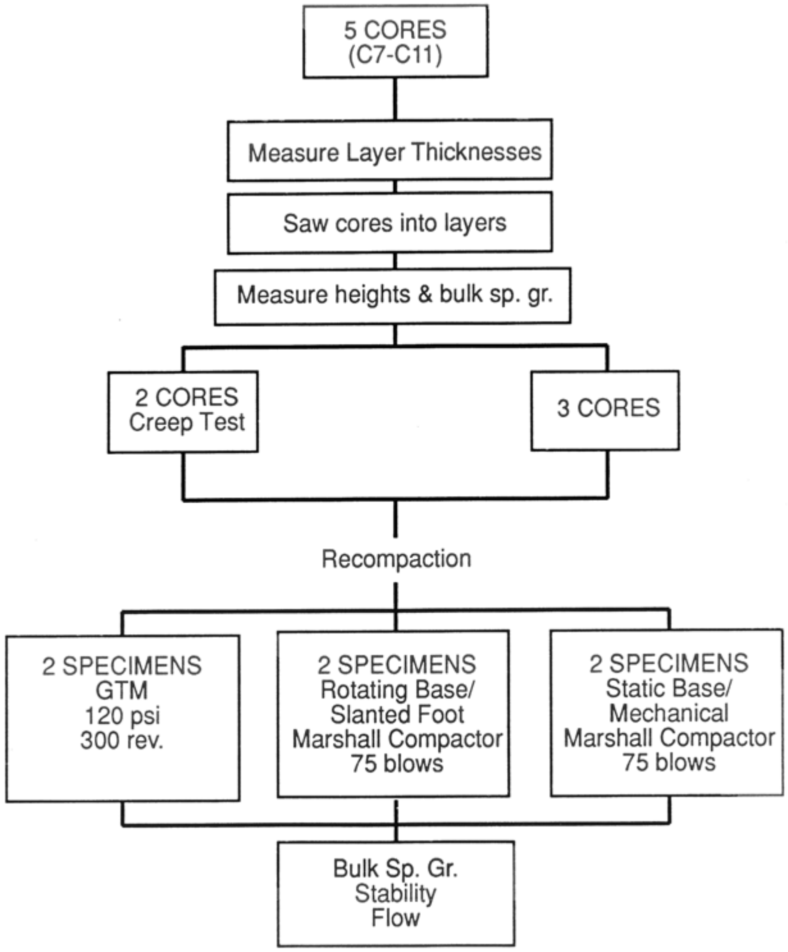

Figure 6. Flow Chart for Testing Cores C7-C11

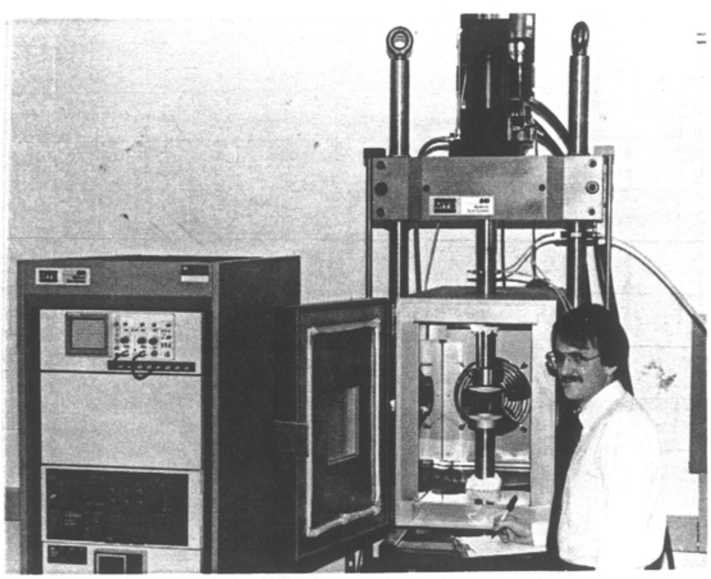

Figure 7. MTS Machine Used for Static Creep Tests

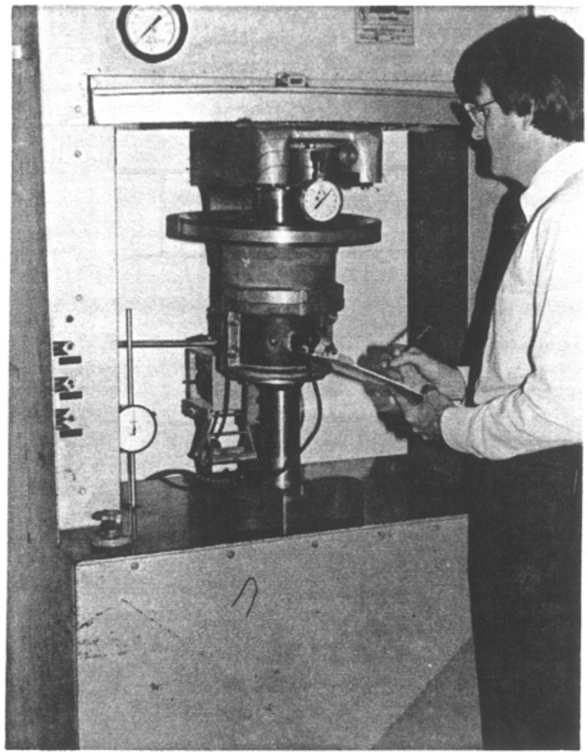

Figure 8. Gyratory Testing Machine (GTM)

Two 4-inch diameter specimens were compacted in a rotating base, slanted foot mechanical Marshall compactor (Figure 9). Every time the hammer is lifted the base supporting the mold assembly rotates 110° (Figure 10). The foot of the hammer has a 10 slant. The combination of rotating base and slant in the foot provides a kneading action during compaction, and generally results in higher densities compared to the conventional static base mechanical Marshall hammer. Seventy five blows were applied on each face of the specimen.

Two 4-inch diameter specimens were compacted (75 blows per face) in a conventional static base mechanical Marshall compactor (Figure 9) used by most states including Pennsylvania.

All six specimens compacted by the three methods were tested for bulk specific gravity, voids in the total mix (VTM), Marshall stability and flow at 140°F.

One core (C6) was taken beside Core C7 as shown in Figure 5. Aged asphalt cement was recovered by the Abson method from this core and tested for penetration at 77°F and viscosity at 140°F.

Rut Depth Measurements

A transverse surface profile of the lane adjacent to Cores C7-C11 (Figure 5) was obtained at the time cores were obtained in spring of 1989. A 12-foot level straight edge was intended to be used (Figure 11) to measure offsets at 1-foot intervals across the lane to obtain the pavement surface profile (including the cross slope or superelevation) and measure surface rut depths. Cores taken transversely across the pavement were used to help determine the amount of rutting in the top layer and the underlying layer(s). This was done by drawing a profile of the layers using the core layer thicknesses. The amount of rutting was determined for the top layer by subtracting the rut depth in the second layer from the rut depth in the top layer. The rut depth in the second layer was determined by subtracting the rut depth in the third layer from the rut depth in the second

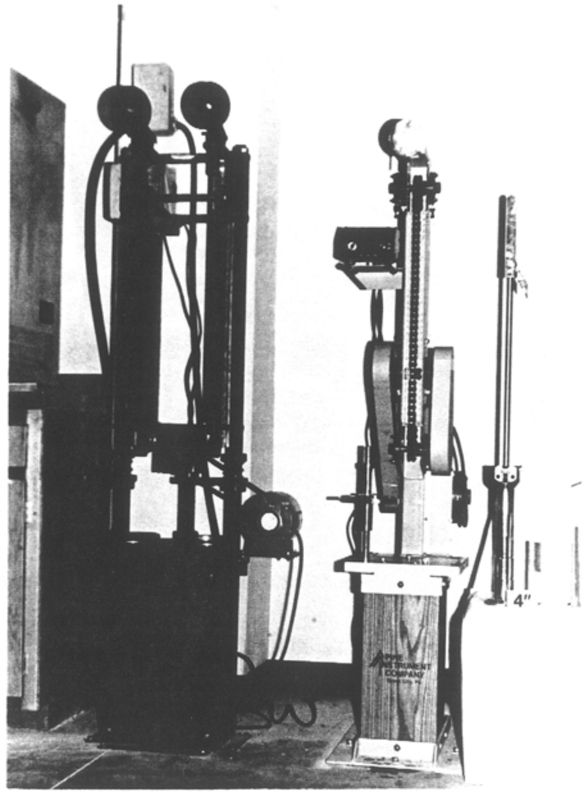

Figure 9. Rotating Base (Left) and Static Base (Right) Mechanical Marshall Compactors

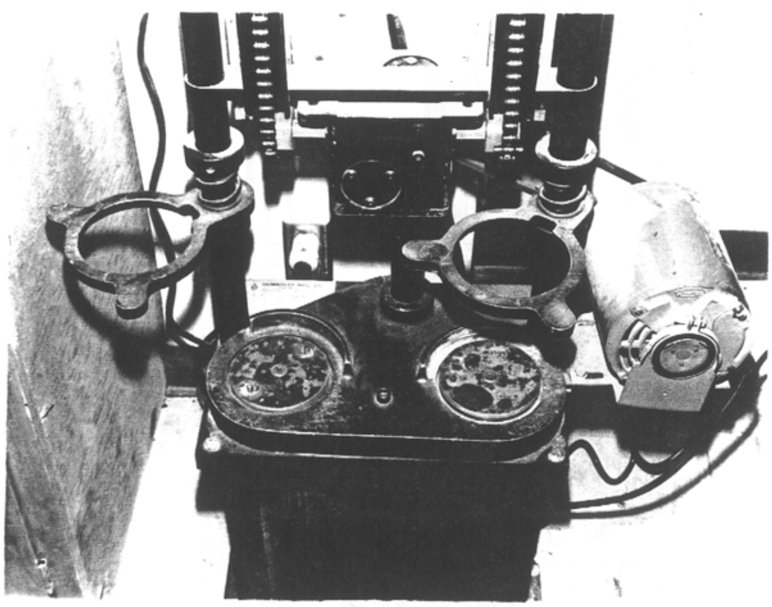

Figure 10. Closeup of Rotating Base Slanted Foot Marshall Compactor

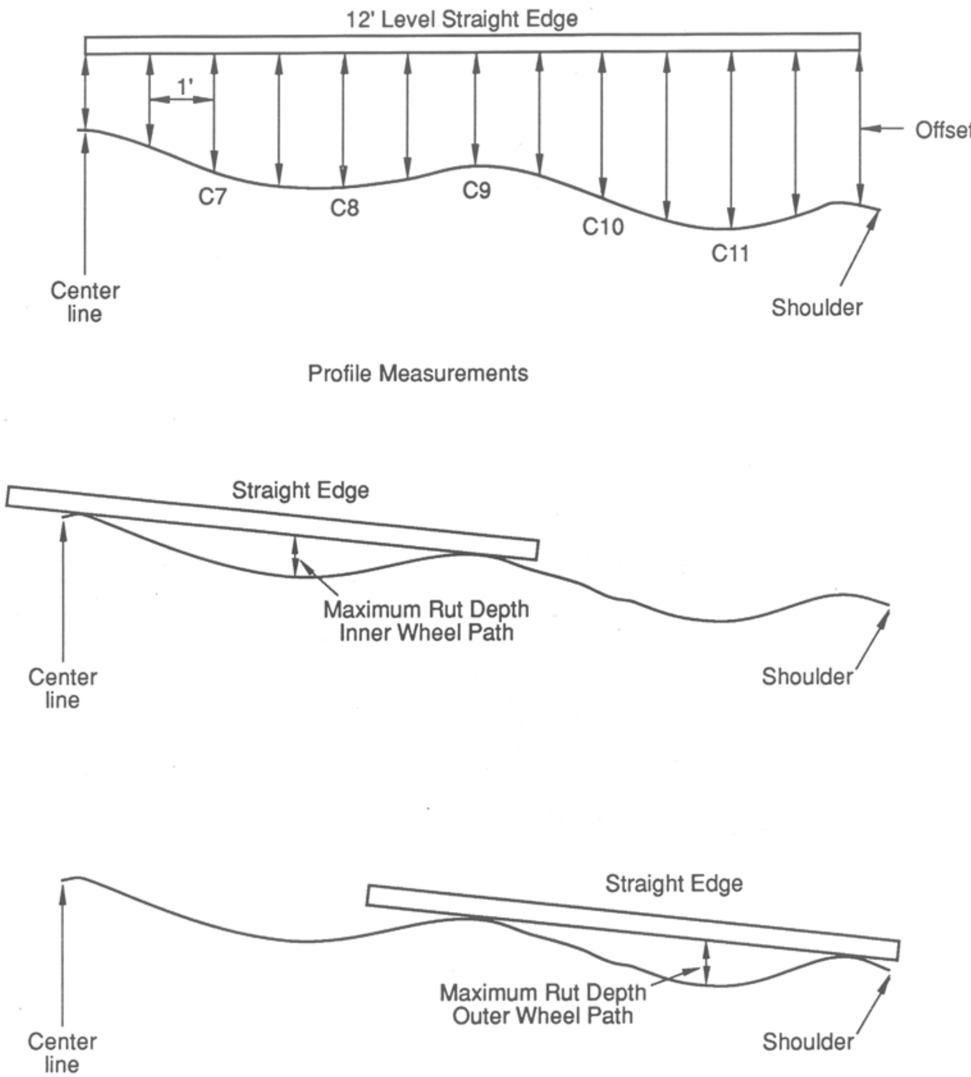

Figure 11. Profile and Rut Depth Measurements

layer. These rut depth values for a given layer were then correlated to the mixture properties of the same layer to insure a meaningful correlation.

It was determined that a taut level string line was used in lieu of the straight edge as planned. Since the string line sags and gives inaccurate surface profile and rut depths, it was decided to re-measure the transverse surface profile using a transverse profilograph device (Figure 12). The profilograph device consists of a 14-foot straight reference beam/track which supports and guides a recorder. A fresh chart is installed on the recorder’s drum, the recorder’s sensing wheel is lowered to contact the pavement, the felt tip marker is adjusted in height as well as pressure, and the recorder is manually rolled along the beam across the lane. The resulting recording displays the vertical pavement profile (including its ruts) across the pavement. Horizontal distances are recorded to the scale of 1 foot = 1 inch. The beam was leveled so that the cross slope or super-elevation was also displayed. Revised surface profiles were obtained during the summer of 1990 (about 1 1/4 year later than the core sampling).

Transverse profilographs were taken at the worst site (where Cores C7-C11 were taken) and at another site more representative of the project, preferably within 500 feet of the worst site.

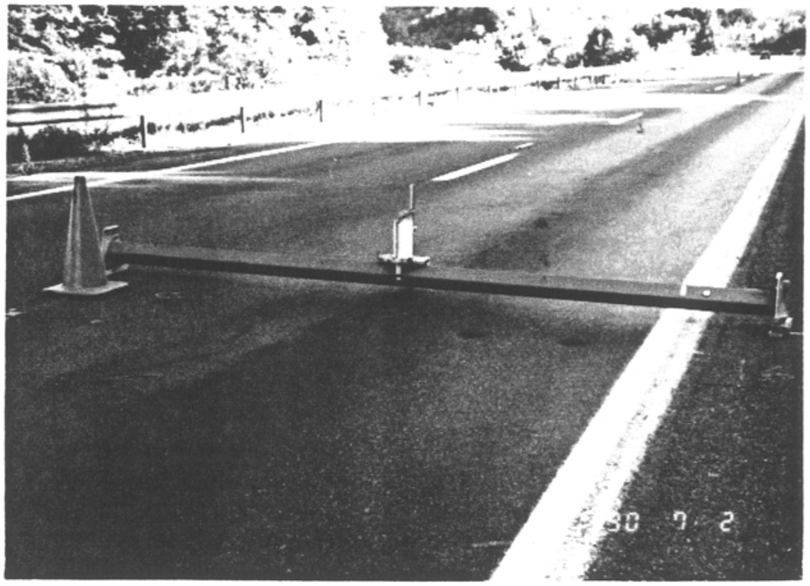

Figure 12. Transverse Profilograph Device

PROJECT DETAILS AND TEST DATA

Project Location Details

Originally, the plan was to evaluate 35 sites. However, Site #21 was deleted by PennDOT and, therefore, there is no data for this site in this report. The locations of the remaining 34 project sites scattered across Pennsylvania are shown in Figure 13. Table 1 gives the location details such as Engineering District, county, State Route (SR) number, Legislative Route (LR) number, section number, segment or milepost. This table also gives the year of construction of the last HMA overlay, whether the HMA is underlain by PCC pavement or not, and the condition of the pavement based on the maximum surface rut depth determined by the transverse profilograph. The pavement condition rating was subjectively determined for each pavement as follows:

Max. Rut Depth (inch)	Age of the Overlay, Years	Rating
0 - 1/8	---	Excellent (E)
1/8 - 1/4	>3	Excellent (E)
1/8 - 1/4	#3	Good (G)
1/4 - 3/8	>3	Good (G)
1/4 - 3/8	#3	Fair (F)
3/8 - 3/4	>3	Fair (F)
3/8 - 3/4	#3	Poor (P)
> 3/4	---	Poor (P)

The preceding subjective rating proved to be fairly reasonable on subsequent rut depth/traffic load data analyses which will be discussed later.

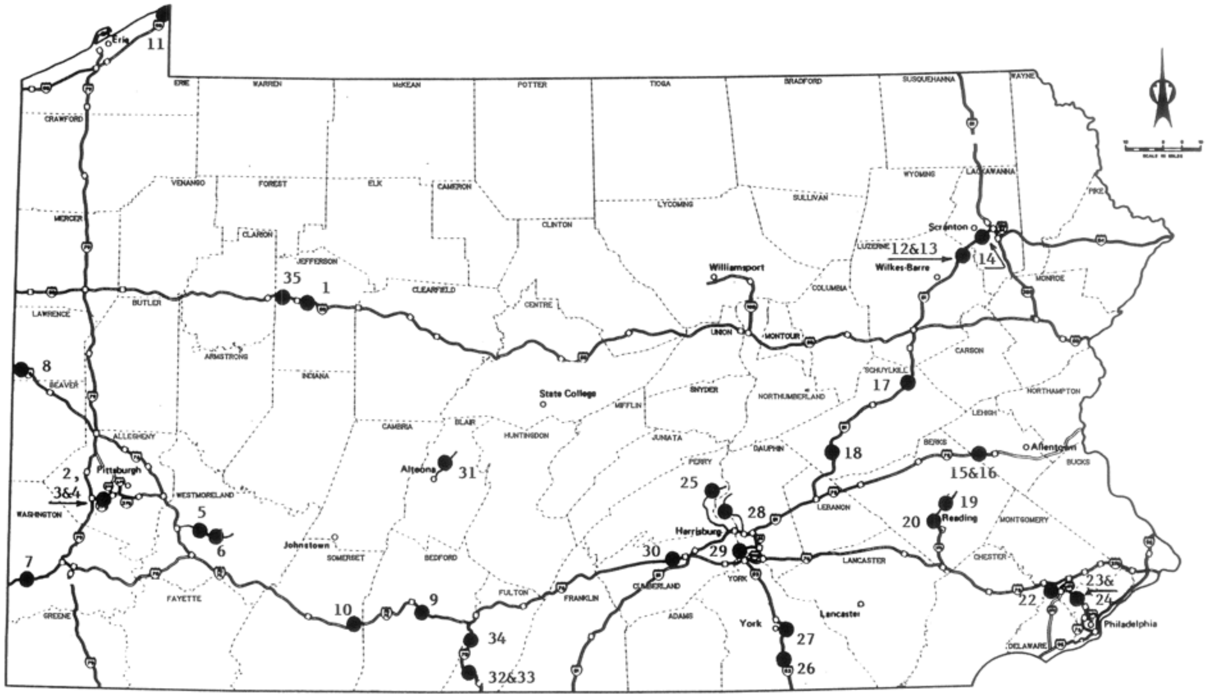

Figure 13. Project Sites Location Map

Only four of the 34 projects did not have PCC pavements underneath the HMA overlay. The age of the HMA overlays as of 1990 summer ranged from two to 19 years. Of the 34 projects, ten were excellent, nine were good, 12 were fair, and 3 were poor based on the rating discussed earlier. There was an assignable cause for the poor performance of Project 11 on Interstate 90. The HMA overlay was placed on a seal coat which was tack coated excessively and, therefore, provided a slip plane for rutting to occur. Therefore, Site #11 was removed from the data base for statistical analysis.

Traffic and Climatological Data

Table 2 gives the traffic data such as average daily traffic (ADT), percentage of trucks, HMA overlay age as of 1990, and total 18-kip equivalent single axle loads (TESALs) applied to the HMA overlay as of 1990. Since the data on ESALs per day was available only for the current year, a traffic growth rate of 10 percent per year was assumed to calculate the total cumulative ESALs applied to the pavement since construction. According to FHWA statistics the average increase in ESALs per year is about 9 percent for a typical rural interstate highway in the United States. It has been shown in other studies that the amount of rutting is dependent upon the total cumulative ESALs.

The average daily traffic ranged from 5925 to 41,000 vehicles per day, and the ESALs ranged from 440 to 9288 per day. Of the 34 projects, only two (Projects 5 and 31) did not meet PennDOT's criteria for heavy duty pavements which includes all interstate highways and other highways carrying more than 20,000 ADT or more than 1,000 ESALs per day. The projects included ten sites on interstate highways, three sites on Pennsylvania Turnpike, and three sites each on heavily traveled Schuylkill Expressway near Philadelphia and Pittsburgh Parkway in Pittsburgh. The remaining sites on primary highways also carried large volumes of traffic. The total estimated traffic carried by the pavements in this study ranged from less than one million ESALs to over 30 million ESALs.

Table 1. Project Sites Location Details

Site No.	District	County	SR	LR	Section	Segment Begin	Segment End	Age	Year Const.	PCC Overlay	Condition*
1	10	Jefferson	80	1009	203	775	835	4	1985	Y	E
2	11	Allegheny	279	765	3117	30	51	3	1986	Y	F
3	11	Allegheny	279	765	---	4	30	5	1984	Y	F
4	11	Allegheny	279	765	---	4	30	5	1984	Y	E
5	12	Westmoreland	30	787	123	260	301	3	1986	Y	F
6	12	Westmoreland	30	787	02R	310	340	4	1985	Y	G
7	12	Washington	70	798	03R	75	104	3	1986	Y	P
8	Turnpike	Lawerance	76	---	---	MP .6	MP 4	18	1971	Y	E
9	Turnpike	Bedford	70/76	---	---	MP 129	MP 163	19	1970	Y	F
10	Turnpike	Somerset	70/76	---	---	MP 124	MP 129	9	1980	Y	G
11	1	Erie	90	797	03M	360	420	2	1987	N	P
12	4	Luzerne	81	1005	09M	1694	1771	3	1986	Y	E
13	4	Luzerne	81	1005	09M	1694	1771	3	1986	Y	G
14	4	Lackawana	81	1005	04M	1820	1851	4	1985	Y	G
15	5	Lehigh	78	443	32M	435	475	3	1986	Y	G
16	5	Lehigh	78	443	32M	436	480	4	1985	Y	F
17	5	Schuykill	81	1005	021	1330	1390	3	1986	Y	G
18	5	Schuykill	81	1005	19M	1074	1095	7	1982	N	F

*E = Excellent, G = Good, F = Fair, and P Poor

Table 1. Project Sites Location Details (Continued)

Site No.	District	County	SR	LR	Section	Segment Begin	Segment End	Age	Year Const.	PCC Overlay	Condition*
19	5	Berks	222	157	37M	490	510	5	1984	Y	G
20	5	Berks	222	6150	13M	240	250	4	1985	Y	E
22	6	Montgomery	76	769	100	3270	3315	5	1984	Y	F
23	6	Montgomery	76	769	300	3380	3391	4	1985	Y	F
24	6	Montgomery	76	769	420	3285	3345	2	1987	Y	E
25	8	Perry	11	195	524	280	320	14	1975	Y	E
26	8	York	83	---	813	2	75	2	1987	Y	E
27	8	York	83	287	---	134	221	5	1984	Y	F
28	8	Dauphin	22	1	003	100	150	2	1987	Y	E
29	8	Cumberland	11	708	072	790	811	2	1987	N	F
30	8	Cumberland	11	34	633	510	541	6	1983	Y	P
31	9	Blair	220	55	37M	480	510	3	1986	Y	F
32	9	Fulton	70	267	015	1554	1654	2	1987	Y	G
33	9	Fulton	70	267	015	1554	1654	2	1987	Y	F
34	9	Bedford	70	267	007	1554	1530	4	1985	Y	G
35	10	Jefferson	80	1009	204	834	884	2	1987	Y	E

*E = Excellent, G = Good, F = Fair, and P Poor

Table 2. Traffic and Climatological Data

Site No.	SR	PCT Trucks	ADT	ESASls Per Day	Pavement Age	Total ESALs (x10EE6)	Traffic Survey Direction	Average Yearly Temp (F)
1	80	40	7655	2765	4	3.87	WB	49.5
2	279	9	39250	2100	3	2.31	NB	50.4
3	279	9	41000	2214	5	3.71	NB	50.4
4	279	9	41000	2214	5	3.71	SB	50.4
5	30	5	16000	440	3	0.48	WB	47.6
6	30	4	21173	471	4	0.66	EB	47.6
7	70	32	9759	2803	3	3.08	WB	50.1
8	76	29	17069	4935	18	17.88	EB	48.6
9	70/76	33	28018	9288	19	34.31	EB	50.5
10	70/76	35	26395	9251	9	23.53	EB	48.0
11	90	43	7227	2775	2	3.05	EB	49.5
12	81	22	14476	2653	3	2.91	NB	50.6
13	81	22	14476	2647	3	2.91	SB	50.6
14	81	19	21629	3483	4	2.46	NB	50.6
15	78	35	11259	3280	3	3.60	WB	52.0
16	78	37	10210	3280	4	4.59	EB	52.0
17	81	23	8120	1522	3	1.67	NB	47.9
18	81	22	7085	1323	7	2.84	NB	51.2
19	222	12	20014	1690	5	2.83	NB	52.0
20	222	9	21180	1195	4	1.67	NB	52.0
22	76	8	28808	1529	5	2.56	NB	53.7
23	76	7	37477	1680	4	2.35	WB	53.7
24	76	7	37477	1680	2	1.29	EB	53.7
25	11	20	6892	1109	14	3.39	BOTH	50.7
26	83	22	11025	1907	2	1.46	NB	53.3
27	83	16	22272	2653	5	4.44	NB	53.3
28	22	12	23296	825	2	0.63		50.7
29	11	3	23775	441	2	0.34	NB	53.0
30	11	29	7856	2025	6	3.90	NB	53.0
31	220	9	8562	542	3	0.60	BOTH	50.7
32	70	37	5925	1982	2	1.52	EB	51.3
33	70	37	5925	1978	2	1.52	WB	51.3
34	70	20	7648	1330	4	1.86	EB	51.3
35	80	44	6187	2477	2	1.90	EB	49.5
Average:		21.2	18121	2426	4.9	4.41		50.9
Std Dev:		12.5	11139	1962	4.1	6.88		1.7

Average yearly temperatures for all project sites are also given in Table 2. These are based on the data from the U.S. Weather Bureau for the nearest weather station. The average yearly temperature ranged from 47.6°F to 53.7°F, which is a very narrow range.

Mix Design Data

Tables 3 through 6 give the mix design data obtained from the job-mix formula (JMF) of the wearing course (Layer 1) selected gradation, number and the binder course (Layer 2). The data includes asphalt content, of blows/face used, specimen specific gravity, maximum specific gravity, percent VTM (voids in total mix), percent VMA (voids in the mineral aggregate), percent VFA (voids filled with asphalt), Marshall stability and flow. An ID-2W mix (a dense graded wearing course mix with 1/2 inch top size) was used in Layer 1 of all projects except Projects 28 and 29 which used an ID-3W mix (a dense graded wearing course mix with 3/4 inch top size).

Similarly, an ID-2B (a dense graded binder course mix with 1 inch topside) was used in Layer 2 except Projects 11 and 12 which used a BCBC mix (a dense graded base course mix with 1 inch top size similar to ID-2B except the BCBC mixture generally has a lower asphalt content). Projects 19, 22 and 34 used an ID2-W mixture as the second layer and Project 25 used a special binder mix (top aggregate size of 2 inches). The 14-year old HMA overlay of Project 25 was rated excellent which is possibly due to the large stone mix used in the binder course. Large stone mix is defined in this study as the mix containing maximum aggregate size greater than one inch.

Wearing Course Mix (Layer 1): Asphalt content ranges from 5.0 to 8.75 percent depending on the aggregates used. Complete aggregate gradations are not given in the tables, only the percentages of material passing 1/2", No. 8 and No. 200 which are considered critical sieve sizes, are given. For ID-2W mixes, the percentage passing No. 8 and No. 200 ranges from 35 to 50 and 3.0 to 6.0, respectively. All mixes were designed using the Marshall method. The number of blows/face used was 50 for 24 projects, 65 for three projects (Pennsylvania Turnpike), and 75 for seven projects. The following mix design data for the Layer 1 mixture is of interest:

	Average	Range
VTM or Air Voids, %	3.6	2.8 to 4.5
VMA, %	16.6	14.5 to 22.4
VFA, %	78.5	73.9 to 83.9
Stability, lbs.	2514	2019 to 3666
Flow, 0.01 inches	10.9	8 to 15

The average VTM is below the midpoint of the 3-5 percent range generally recommended for the mix design. Only seven of the 34 projects had design VTM equal to or more than 4.0 percent. Stability values are generally very high, and the flow values are within the acceptable range of 6 to 16.

Binder course mix (Layer 2): Excluding the ID-2W courses, the asphalt content for the Layer 2 mixes ranged from 4.0 to 5.2 percent. The percentages passing 1/2", No. 8 and No. 200 sieves for the binder courses ranged from 42 to 69, 19 to 30, and 2.5 to 5.0, respectively. The Layer 2 in Projects 19, 22 and 34 consisted of ID-2W wearing course mixtures and were excluded from the ranges listed. The number of blows/face used was 50 for 21 projects, and 75 for seven projects with no data being available for 6 projects. The following mix design data for Layer 2 mixtures is of interest:

Table 3. Project Job-Mix Formulas (Mix Composition) Wearing Course (Layer 1)

Site	SR	Mix Type	Design Asphalt Content	Percent Passing 1/2"	No. 8	No. 200
1	80	ID2W	6.20	100	47	5.5
2	279	ID2W	6.40	100	43	5.0
3	279	ID2W	7.20	100	48	4.0
4	279	ID2W	6.70	100	43	4.0
5	30	ID2W	6.50	100	42	5.0
6	30	ID2W	6.50	100	43	5.0
7	70	ID2W	5.90	100	44	5.0
8	76	ID2W	6.50	100	45	5.0
9	70/76	ID2W	8.75	100	45	3.0
10	70/76	ID2W	6.90	100	45	4.5
11	90	ID2W	7.20	100	50	4.0
12	81	ID2W	6.60	100	48	4.5
13	81	ID2W	6.60	100	48	4.5
14	81	ID2W	6.60	100	45	5.0
15	78	ID2W	6.40	100	45	5.0
16	78	ID2W	6.40	100	45	5.0
17	81	ID2W	6.20	100	48	4.0
18	81	ID2W	5.70	100	42	5.0
19	222	ID2W	6.20	100	45	5.0
20	222	ID2W	6.60	100	45	5.0
22	76	ID2W	5.70	100	43	5.2
23	76	ID2W	5.80	100	43	5.2
24	76	ID2W	6.00	100	43	4.5
25	11	ID2W	---	---	---	--
26	83	ID2W	6.10	100	40	4.5
27	83	ID2W	6.00	100	38	5.0
28	22	ID3W	5.20	80	35	5.0
29	11	ID3W	5.00	76	37	5.0
30	11	ID2W	6.40	100	40	4.0
31	220	ID2W	6.30	100	40	5.0
32	70	ID2W	6.00	100	45	5.0
33	70	ID2W	6.00	100	45	5.0
34	70	ID2W	6.30	100	45	6.0
35	80	ID2W	6.10	100	47	5.0
Average:			6.15	95.8	42.6	4.63
Std Deviation:			1.237	17.46	8.08	0.969

"--- " = Data Not Available

Table 4. Project Job-Mix Formulas (Mix Composition) Binder Course (Layer 2)

				Percent Passing		
Site	**SR**	**Mix Type**	**Design Asphalt Content**	**1/2"**	**No. 8**	**No. 200**
1	80	ID2B	4.6	69	25	4.5
2	279	ID2B	4.2	57	27	5.0
3	279	ID2B	5.0	69	30	4.0
4	279	ID2B	5.0	67	30	4.0
5	30	ID2B	4.8	64	25	4.0
6	30	ID2B	4.6	62	25	4.4
7	70	ID2B	4.5	63	25	4.0
8	76	ID2B	---	---		--
9	70/76	ID2B		---	---	--
10	70/76	ID2B	---	---		--
11	90	BCBC	4.6	66	28	--
12	81	BCBC	4.7	61	30	5.0
13	81	ID2B	4.7	61	30	5.0
14	81	ID2B	5.2	65	30	4.0
15	78	ID2B	4.3	63	30	4.0
16	78	ID2B	4.5	61	28	5.0
17	81	ID2B	4.5	55	30	4.0
18	81	ID2B	4.5	60	29	4.0
19	222	ID2W	6.2	---	---	5.0
20	222	ID2B	4.5	56	25	4.0
22	76	ID2W	4.5	63	26	4.0
23	76	ID2B	4.5	63	26	4.0
24	76	ID2B	4.1	57	30	4.0
25	11	SP B	4.0	42	19	2.5
26	83	ID2B	4.2	55	28	4.0
27	83	ID2B	4.5	62	25	4.0
28	22	ID2B		---	---	--
29	11	ID2B	4.5	63	27	4
30	11		---	---	---	--
31	220	ID2B	---	---	---	--
32	70	ID2B	4.6	55	30	4.0
33	70	ID2B	4.6	55	30	4.0
34	70	ID2W	6.3	100	45	6.0
35	80	ID2B	4.4	63	30	4.5
Average:			4.66	62.1	28.3	4.26
Std Deviation:			0.508	9.21	4.20	0.613

"--- " = Data Not Available

Table 5. Project Marshall Mix Design Data - Wearing Course (Layer 1)

Site	SR	Mix Type	Heavy Duty	# Blows	Specimen Sp. Gr.	Max. Sp. Gr.	% VTM	% VMA	% VFA	Stab. (lbs)	Flow (units)
1	80	ID2W	N	50	2.413	2.501	3.5	16.1	78.3	2765	10
2	279	ID2W	N	50	2.332	2.404	3.0	16.0	81.3	2650	11
3	279	ID2W	N	50	2.289	2.355	2.8	16.1	82.6	2350	11
4	279	ID2W	N	50	2.288	2.371	3.5	16.6	78.9	2075	10
5	30	ID2W	Y	75	2.313	2.385	3.0	16.3	81.6	2322	10
6	30	ID2W	N	50	2.323	2.397	3.1	15.4	79.9	2198	10
7	70	ID2W	Y	75	2.310	2.396	3.6	15.7	77.1	2600	9
8	76	ID2W	N	65	2.322	2.391	2.9	17.7	83.6	2658	11
9	70/76	ID2W	N	65	2.192	2.274	3.6	22.4	83.9	3666	11
10	70/76	ID2W	N	65	2.334	2.406	3.0	18.6	83.9	2783	12
11	90	ID2W	N	50	2.273	2.343	3.0	16.9	82.2	2167	8
12	81	ID2W	N	50	2.350	2.435	3.5	16.3	78.5	3087	10
13	81	ID2W	N	50	2.350	2.435	3.5	16.3	78.5	3087	10
14	81	ID2W	N	50	2.345	2.420	3.1	16.5	81.2	2667	15
15	78	ID2W	N	50	2.386	2.462	3.1	16.4	81.1	2347	12
16	78	ID2W	N	50	2.334	2.421	3.6	17.7	79.7	2019	12
17	81	ID2W	N	50	2.321	2.410	3.7	16.1	77.0	3540	12
18	81	ID2W	N	50	2.440	2.536	3.8	16.6	77.1	2160	11
19	222	ID2W	N	50	2.381	2.483	4.1	18.0	77.2	2350	11
20	222	ID2W	N	50	2.305	2.399	3.9	16.4	76.2	3000	13
22	76	ID2W	N	50	2.397	2.479	3.3	15.3	78.4	2098	11
23	76	ID2W	N	50	2.397	2.479	3.3	15.3	78.4	2098	11
24	76	ID2W	Y	75	2.386	2.467	3.3	15.3	78.4	2098	11
25	11	ID2W	N	50	---	---	---	---	---	--	--
26	83	ID2W	Y	75	2.387	2.486	4.0	16.4	75.6	2820	14
27	83	ID2W	N	50	2.373	2.469	3.9	15.4	T4.T	2066	11
28	22	ID3W	N	50	---	---	3.8	15.7	75.8	2500	9
29	11	ID3W	N	50	2.432	2.523	3.6	14.5	75.2	2378	11
30	11	ID2W	N	50	2.341	2.433	3.8	16.9	77.5	2450	11
31	220	ID2W	N	50	2.361	2.459	4.0	18.5	78.4	2667	11
32	70	ID2W	Y	75	2.413	2.519	4.2	16.1	73.9	2201	10
33	70	ID2W	Y	75	2.413	2.519	4.2	16.1	73.9	2201	10
34	70	ID2W	N	50	2.365	2.476	4.5	19.0	76.3	2187	11
35	80	ID2W	Y	75	2.387	2.486	4.0	16.3	75.5	2700	10
Average:					2.352	2.438	3.55	16.63	78.54	2513.8	10.91
Std. Deviation:					0.052	0.058	0.431	1.436	2.815	411.10	1.33

"--- " Data Not Available

Table 6. Project Marshall Mix Design Data - Binder Course (Layer 2)

Site	SR	Mix Type	Heavy Duty	# Blows	Specimen Sp. Gr.	Max. Sp. Gr.	% VTM	% VMA	% VFA	Stab. (lbs)	Flow (units)
1	80	ID2B	N	50	2.481	2.582	3.9	14.1	72.3	2018	11
2	279	ID2B	N	50	2.415	2.516	4.0	12.2	67.2	2700	9
3	279	ID2B	N	50	2.322	2.394	3.0	12.4	75.8	2183	10
4	279	ID2B	N	50	2.332	2.394	2.6	12.4	79.0	2200	12
5	30	ID2B	Y	75	2.410	2.490	3.2	13.7	76.6	3100	13
6	30	ID2B	N	50	2.400	2.500	4.0	13.7	70.8	2490	10
7	70	ID2B	Y	75	2.401	2.488	3.5	13.3	73.7	3100	9
8	76	ID2B	N	---	---	---	---	---	---	---	--
9	70/76	ID2B	N	---	---	---	---	---	---	---	---
10	70/76	ID2B	N	---	---	---	---	---	---	---	---
11	90	BCBC	N	50	2.306	2.407	4.2	13.9	69.8	1908	10
12	81	BCBC	N	50	2.446	2.522	3.0	13.2	77.3	3050	14
13	81	ID2B	N	50	2.446	2.522	3.0	13.2	77.3	3050	14
14	81	ID2B	N	50	2.393	2.490	3.9	14.2	72.5	2283	12
15	78	ID2B	N	50	2.455	2.552	3.8	13.7	72.3	2400	12
16	78	ID2B	N	50	2.438	2.529	3.6	13.9	74.1	2200	12
17	81	ID2B	N	50	2.372	2.453	3.3	13.0	74.6	2850	12
18	81	ID2B	N	50	2.512	2.603	3.5	14.3	75.5	1733	11
19	222	ID2W	N	50	2.381	2.483	4.1	18.0	77.2	2350	11
20	222	ID2B	N	50	2.456	2.558	4.0	14.3	72.0	2008	12
22	76	ID2W	N	50	2.511	2.616	4.0	14.3	71.9	1696	11
23	76	ID2B	N	50	2.579	2.686	4.0	14.2	71.8	1696	11
24	76	ID2B	Y	75	2.560	2.667	4.0	14.2	71.8	1696	11
25	11	SP B	N	50	---	---	---	---	---	---	--
26	83	ID2B	Y	75	2.513	2.615	3.9	13.2	70.5	2357	13
27	83	ID2B	N	50	2.499	2.600	3.9	13.8	71.7	1477	12
28	22	---	---	---	---	---	---	---	---	---	--
29	11	ID2B	N	50	2.429	2.530	4.0	14.2	71.8	1517	10
30	11	ID2B	---	---	---	---	---	---	---	---	--
31	220	---	---	---	---	---	---	---	---	---	--
32	70	ID2B	Y	75	2.409	2.520	4.4	13.6	67.6	2467	14
33	70	ID2B	Y	75	2.409	2.520	4.4	13.6	67.6	2467	14
34	70	ID2W	N	50	2.365	2.476	4.5	19.0	76.3	2187	11
35	80	ID2B	Y	75	2.484	2.588	4.0	13.6	70.6	2685	11
Average (ALL):					2.434	2.530	3.77	13.97	72.96	2291	11.6
Std. Deviation (ALL):					0.067	0.073	0.47	1.41	3.11	479	1.42
Excluding ID-2W Mixes											
Average:					2.436	2.530	3.71	13.58	72.68	2318	11.6
Std. Deviation:					0.069	0.758	0.48	0.61	3.16	502	1.53

"--- " Data Not Available

	Average	Range
VTM, %	3.7	2.6 to 4.4
VMA, %	13.6	12.2 to 14.3
VFA, %	72.7	67.2 to 79.0
Stability, lbs.	2318	1477 to 3100
Flow, 0.01 inches	11.6	9 to 14

The average VTM is less than 4.0 percent. The average stability value of 2318 lbs., although satisfactory, is lower than that of the wearing courses. The average flow value is slightly higher than the wearing courses. Normally, it is desirable to have a stiffer binder course mix than wearing course mix to minimize rutting.

Construction Data

Table 7A gives the project construction data on percent air voids or VTM, asphalt content, and the material passing 1/2", No. 8 and No. 200 sieves. The projects constructed under the RPS (restricted performance specification) or end result specifications are identified in this table. RPS data obtained by the PennDOT central laboratory on mix composition (loose mixtures) and density (cores) at the time of construction has been reported for these projects. PennDOT's quality assurance test data (if available) or contractor's daily test data was used for non-RPS projects. The table gives the values of mean, standard deviation, and conformal index (Cl) for various properties.

The conformal index (Cl) expresses deviations from the JMF and indicates the relative target miss and affords the opportunity to evaluate mixes of different JMFs (Q). It is calculated as follows:

$$CI = \sqrt{\frac{\sum (x-T)^2}{n}}$$

where,

CI = Conformal index
x = Individual measurement
T = Target established by JMF
n = Sample size

A review and analysis of CI data obtained on loose mixture samples at the time of construction indicates the following:

1. Generally, the asphalt content was deficient from the JMF asphalt content for the wearing course, with the average CI values being -0.11 for the wearing and 0.00 for the binder. Although CI values are always positive, minus values have been assigned to indicate that the average values were lower than the corresponding JMF values.
2. The percent of material passing the No. 8 sieve was generally higher than the JMF value for both wearing and binder courses, with average CI values of +0.35 and +2.47, respectively. These values indicate that the problem of JM F target miss was serious for binder mixes.

Table 7A. Project Construction Data - All Courses

Site	Layer	SR	Mix Type	RPS	% Voids Avg	% Voids SD	Asphalt Content, % JMF	Asphalt Content, % Avg	Asphalt Content, % SD	Asphalt Content, % CI	Passing 1/2 inch JMF	Passing 1/2 inch Avg	Passing 1/2 inch SD	Passing 1/2 inch CI	Passing #8 JMF	Passing #8 Avg	Passing #8 SD	Passing #8 CI	Passing #200 JMF	Passing #200 Avg	Passing #200 SD	Passing #200 CI
1	1	80	ID2W	Y	6.86	1.224	6.2	6.15	0.170	-0.179	100	100.0	0.000	0.000	47	46.9	2.688	-2.693	5.5	5.71	0.784	0.813
1	2	80	ID2B	Y	6.55	1.290	4.5	4.58	0.205	0.206	69	71.3	3.358	4.094	25	27.1	1.917	2.847	4.5	4.81	0.552	0.633
2	1	279	ID2W	Y	6.60	1.363	6.4	6.02	0.113	-0.395	100	100.0	0.000	0.000	43	44.7	2.582	2.933	5.0	6.39	0.335	1.365
2	2	279	ID2B	Y	4.39	1.430	5.0	4.20	0.413	-0.415	57	55.6	8.558	-8.672	27	25.9	3.562	-3.728	5.0	5.30	0.778	0.841
3	1	279	ID2W	Y	4.54	1.231	7.2	7.02	0.209	-0.275	100	100.0	0.000	0.000	48	46.4	1.989	-2.550	4.0	4.69	0.589	0.904
3	2	279	ID2B	N	---	---	4.0	4.92	0.382	0.390	69	76.6	5.886	9.613	30	29.6	2.417	-2.450	4.0	4.08	0.293	4.091
4	1	279	ID2W	Y	5.16	1.387	6.7	6.55	0.148	-0.210	100	100.0	0.000	0.000	43	44.1	1.583	1.971	4.0	3.69	0.674	-0.741
4	2	279	ID2B	N	---	---	4.0	4.26	0.361	0.823	67	66.4	14.691	-14.704	30	30.8	6.493	6.542	4.0	5.46	0.801	1.666
5	1	30	ID2W	Y	5.01	1.396	6.5	6.42	0.181	-0.200	100	100.0	0.000	0.000	42	42.3	2.048	2.064	5.0	5.75	0.707	1.031
5	2	30	ID2B	Y	4.89	1.054	4.0	4.73	0.253	0.261	64	69.9	5.827	8.270	25	27.1	1.880	2.842	4.0	4.29	0.546	0.618
5	3	30	ID2B	N	---	---	4.0	4.28	0.090	0.426	---	57.3	10.143	12.138	---	23.7	3.091	3.367	---	3.67	0.419	0.535
6	1	30	ID2W	Y	4.75	1.458	6.5	6.53	0.346	0.347	100	100.0	0.000	0.000	43	40.4	1.615	-2.291	5.0	5.60	0.689	0.915
6	2	30	ID2B	Y	5.38	0.906	4.4	4.81	0.306	0.369	62	67.2	5.574	7.609	25	25.8	2.352	2.491	4.4	4.51	0.663	0.672
7	1	70	ID2W	Y	6.28	1.990	5.9	6.13	0.191	0.297	100	100.0	0.000	0.000	44	43.5	4.450	-4.478	5.0	5.43	0.463	0.634
7	2	70	ID2B	Y	5.32	1.364	4.0	4.41	0.355	0.367	63	66.0	6.844	7.457	25	28.0	2.993	4.238	4.0	4.46	0.358	0.580
8	1	76	ID2W	N	---	---	6.5	6.08	0.040	-0.422	100	100.0	.0.000	0.000	45	43.0	0.894	-2.191	5.0	6.30	0.283	1.330
8	3	76	ID2B	N	---	---	---	4.58	0.312	---	---	73.4	4.716	---	---	30.8	2.993	---	---	4.98	0.467	--
9	1	70/76	ID2W	N	---	---	8.8	7.96	0.265	-0.881	100	100.0	0.000	0.000	45	45.0	2.098	2.098	3.0	5.18	0.483	2.233
9	2	70/76	ID2B	N	---	---	---	4.78	0.271	---	---	69.6	3.007	---	---	28.2	1.167	---	---	5.32	0.893	--
10	1	70/76	ID2W	N	---	---	6.9	6.20	0.179	-0.723	100	100.0	0.000	0.000	45	45.2	1.721	1.732	4.5	6.78	0.615	2.361
10	2	70/76	ID2B	N	---	---	---	4.26	0.186	---	---	76.6	11.893	---	---	33.8	4.956	---	---	5.10	0.555	--
11	1	90	ID2W	Y	3.97	1.408	7.2	7.04	0.141	-0.213	100	100.0	0.000	0.000	50	48.7	2.749	-3.055	4.0	4.48	0.973	1.085
11	2	90	BCBC	N	---	---	4.6	4.48	0.194	-0.228	66	81.0	3.033	15.304	28	30.4	2.728	3.633	---	6.36	0.344	--
11	3	90	BCBC	N	---	---	4.6	3.92	0.337	-0.759	---	77.8	1.600	11.908	---	29.2	2.227	2.530	---	6.40	0.772	--
12	1	81	ID2W	Y	4.95	1.473	6.6	6.40	0.272	-0.335	100	100.0	0.000	0.000	48	49.3	3.608	3.819	4.5	5.51	0.796	1.287
12	2	81	BCBC	Y	2.43	1.038	4.7	4.76	0.398	0.403	61	63.1	8.211	8.481	30	31.1	3.844	3.987	5.0	5.46	0.625	0.774

"--- " Data Not Available

Table 7A. Project Construction Data - All Courses (Continued)

					% Voids		Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	Layer	SR	Mix Type	RPS	Avg	SD	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI
13	1	81	ID2W	Y	5.52	1.785	6.6	6.39	0.227	-0.308	100	100.0	0.000	0.000	48	49.8	2.364	2.983	4.5	6.49	1.580	2.541
13	2	81	ID2B	Y	3.10	1.389	4.7	4.77	0.348	0.355	61	63.9	6.892	7.470	30	33.2	3.561	4.787	5.0	5.35	0.834	0.903
14	1	81	ID2W	Y	4.58	1.854	6.6	6.58	0.218	-0.219	100	100.0	0.000	0.000	45	45.5	2.920	2.956	5.0	5.81	0.882	1.199
14	2	81	ID2B	Y	4.95	1.620	5.2	5.13	0.196	-0.208	65	68.6	4.355	5.645	30	27.7	1.763	-2.876	4.0	5.23	0.831	1.486
15	1	78	ID2W	Y	5.36	1.453	6.4	6.26	0.115	-0.178	100	100.0	0.000	0.000	45	45.4	2.500	2.541	5.0	5.52	0.622	0.244
15	2	78	ID2B	Y	5.41	1.142	4.3	4.41	0.205	-0.233	63	65.1	4.644	5.099	30	32.2	3.407	4.033	4.0	5.31	0.796	1.529
16	1	78	ID2W	Y	5.88	1.770	6.4	6.19	0.162	-0.265	100	100.0	0.000	0.000	45	43.9	1.730	-2.045	5.0	6.31	0.306	1.344
16	2	78	ID2B	Y	4.60	1.765	4.5	4.40	0.171	-0.198	61	67.2	4.079	7.447	28	29.7	1.814	2.481	5.0	5.40	0.864	0.952
17	1	81	ID2W	Y	7.39	1.530	6.2	5.90	0.253	-0.392	100	100.0	0.000	0.000	48	47.7	3.002	-3.017	4.0	4.95	0.541	1.093
17	2	81	ID2B	Y	5.07	1.351	4.5	4.53	0.390	0.391	55	58.8	5.887	7.007	30	33.5	3.722	5.109	4.0	4.60	0.658	0.887
18	1	81	ID2W	Y	5.32	1.042	5.7	5.72	0.240	2.392	100	100.0	0.000	0.000	42	41.2	1.796	-6.419	5.0	5.07	0.793	2.252
18	2	81	ID2B	Y	3.09	1.239	4.5	4.56	0.187	0.203	60	69.9	4.211	11.088	29	28.9	1.886	-1.949	4.0	5.30	0.619	1.487
20	1	222	ID2W	Y	6.66	1.833	6.6	6.29	0.422	-0.526	100	100.0	0.000	0.000	45	42.0	1.512	-3.359	5.0	5.43	0.647	0.776
20	2	222	ID2B	Y	2.80	1.571	4.5	4.59	0.273	0.288	56	60.7	5.836	7.488	25	29.2	3.278	5.292	4.0	4.76	0.769	1.082
20	3	222	LEVEL	N	---	---	---	3.17	1.836	---	---	---	---	---	---	21.2	11.47	---	---	---	---	--
22	1	76	ID2W	N	---	---	5.7	5.60	0.228	-0.249	100	100.0	0.000	0.000	43	43.4	1.020	1.095	5.2	7.14	0.605	2.032
22	2	76	...	N	---	---	4.5	4.42	0.117	-0.141	---	73.0	3.286	10.526	26	24.8	2.400	-2.683	4.0	4.64	0.413	0.762
22	3	76	ID2W	N	---	---	5.40	0.237	---	---	100.0	0.000	---		---	45.0	1.095	---	---	5.84	0.539	--
23	1	76	ID2W	Y	4.87	2.412	5.8	5.58	0.227	-0.314	100	100.0	0.000	0.000	43	43.2	3.184	3.189	5.2	5.34	1.339	1.346
23	2	76	ID2B	Y	---	---	4.5	4.15	0.314	-0.348	---	62.9	6.066	6.067	26	27.1	2.368	2.602	4.0	4.48	0.654	0.810
23	3	76	ID2B	N	---	---	---	3.78	0.177	0.280	---	---	---	---	---	26.0	1.155	1.155	---	---	---	--
24	1	76	ID2W	Y	6.36	1.838	6.0	5.92	0.274	-0.284	100	100.0	0.000	0.000	43	46.0	8.391	8.920	4.5	5.61	0.458	1.199
24	2	76	ID2B	N	---	---	4.1	3.75	0.440	-0.559	63	56.0	5.385	-4.503	30	33.5	2.894	5.208	4.0	5.68	0.503	1.749
24	3	76	ID2B	N	---	---	---	4.75	0.892	---	---	---	---	---	---	27.5	8.827	---	---	---	---	--
25	1	11	ID2W	N	---	---	---	6.04	0.250	---	---	100.0	0.000	---	---	42.6	1.960	---	---	5.42	0.909	--
25	2	11	SP B	N	---	---	4.0	3.70	0.490	-0.574	63	46.2	7.782	-8.843	19	20.2	3.187	3.406	2.5	3.38	0.160	0.894

"--- " Data Not Available

Table 7A. Project Construction Data - All Courses (Continued)

					% Voids		Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	Layer	SR	Mix Type	RPS	Avg	SD	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI
26	1	83	ID2W	Y	6.83	1.512	6.1	6.07	0.323	-0.325	100	100.0	0.000	0.000	40	39.8	3.848	-3.856	4.5	5.22	0.874	1.132
26	2	83	ID2B	Y	5.38	1.097	4.2	4.48	0.405	-0.491	57	61.7	7.916	10.349	28	29.0	2.944	3.109	4.0	5.72	0.581	1.818
26	3	83	---	Y	8.77	3.287	---	4.46	0.233	---	---	---	---	---	---	26.4	2.216	---	---	---	---	--
27	1	83	ID2W	Y	3.74	0.772	6.0	5.73	0.254	-0.368	100	100.0	0.000	0.000	38	37.1	1.792	-2.000	5.0	5.27	0.569	0.631
27	2	83	ID28	Y	3.79	0.953	4.5	4.42	0.350	-0.358	62	61.4	6.576	-6.601	25	25.3	2.333	2.353	4.0	4.50	0.697	0.860
28	1	22	ID3W	Y	5.75	1.413	5.2	5.38	0.293	0.315	---	---	---	---	35	37.0	4.534	4.955	5.0	3.92	0.708	-1.239
28	3	22	ID2W	Y	6.42	1.285	---	6.25	0.198	0.319	---	100.0	0.000	0.000	---	39.8	3.387	3.391	---	4.82	0.498	0.531
29	1	11	ID3W	Y	4.71	0.338	5.0	5.17	0.415	0.447	100	100.0	0.000	0.000	37	36.0	3.000	-3.162	5.0	5.05	0.608	0.610
30	1	11	ID2W	Y	5.88	1.279	6.4	6.11	0.136	-0.317	100	100.0	0.000	0.000	40	39.4	1.769	-1.884	4.0	5.08	0.833	1.368
31	1	220	ID2W	Y	6.93	1.082	6.3	6.45	0.218	0.265	100	100.0	0.000	0.000	40	41.7	4.723	5.008	5.0	5.22	0.953	0.977
32	1	70	ID2W	Y	6.93	0.786	6.0	6.00	0.229	0.229	100	100.0	0.000	0.000	45	46.5	3.395	3.715	5.0	5.40	0.331	0.519
32	2	70	ID2B	Y	6.45	1.350	4.6	4.49	0.395	-0.231	55	55.3	3.518	1.369	30	30.3	2.654	0.985	4.0	3.65	0.524	-0.402
33	1	70	ID2W	Y	6.83	1.434	6.0	5.86	0.236	-0.274	100	100.0	0.000	0.000	45	47.7	2.883	3.931	5.0	5.96	0.872	1.294
33	2	70	ID2B	Y	---	---	4.6	4.65	0.296	0.300	55	57.1	5.411	5.797	30	30.1	6.742	6.742	4.0	4.51	0.525	0.728
34	1	70	ID2W	Y	6.90	1.586	6.3	6.30	0.161	0.161	100	100.0	0.000	0.000	45	49.8	2.935	5.646	6.0	6.73	1.030	1.264
34	4	70	ID2B	Y	5.24	1.788	---	4.69	0.399	0.410	---	55.2	5.199	5.203	---	28.4	3.957	5.189	---	4.10	0.688	1.297
35	1	80	ID2W	Y	7.08	0.964	6.1	6.06	0.123	-0.128	100	100.0	0.000	0.000	47	45.5	2.463	-2.884	5.0	5.96	0.410	1.047
35	2	90	ID2B	Y	7.17	1.115	4.4	4.15	0.239	-0.281	63	64.0	5.013	5.112	30	28.4	2.173	2.711	4.5	4.48	0.582	-0.545
Layer 1																						
	Average:				5.79	1.407	6.33	6.18	0.219	-0.115	100	100	0.0	0.0	43.8	43.95	2.701	0.347	4.77	5.562	0.699	1.111
	Std Deviation:				1.005	0.4051	0.630	0.499	0.080	0.5324	0.0	0.0	0.0	0.0	3.28	3.496	1.359	3.590	0.546	0.759	0.275	0.758
Layer 2																						
	Average:				4.751	1.275	4.50	4.531	0.295	0.004	61.9	66.12	5.848	4.319	27.6	30.05	3.041	2.469	4.20	4.992	0.606	1.036
	Std Deviation:				1.315	0.233	0.451	0.428	0.098	0.375	4.35	9.932	2.786	7.150	2.72	6.585	1.274	2.919	0.527	0.819	0.179	0.846

“--- ” Data Not Available

3. The percentage of minus 200 material in the "produced mix" was mostly higher than the "designed mix" for both courses with average CI values of +1.11 and +1.04, respectively. There is a need for closer control of the minus 200 material during mix production. If the minus 200 material tends to be consistently excessive during the production the job-mix formula should be revised to incorporate higher amounts of minus 200 as long as the mix meets the specified mix design criteria.
4. The percentage of material passing the 1/2" sieve for the binder course had a CI value of +4.32 indicating that the "produced mix" was finer than the "designed mix." For better resistance to rutting, it is desirable to have a higher percentage of material retained on 1/2" sieve.

Generally accepted CI values for the materials passing 1/2", No. 8 and No. 200 sieves are ±7, ±4 and ±1 percent (based on 3 to 5 samples), respectively. However, the overall data from this project is skewed on the excessive side which is not desirable. Ideally, projects must have variations on both positive and negative sides.

The statistical analysis of VTM (voids in total mix) data obtained at the time of construction in HMA pavement is as follows:

	Wearing Course	Binder Course
Number of Projects	29	19
Mean	5.79	4.75
Standard Deviation	1.01	1.32
95% Confidence Limits	3.8 - 7.8	2.1 - 7.4

The data indicates that the level of compaction in both layers was generally acceptable. Lower voids (about one percent) were achieved in the binder course than in the wearing course.

Table 7B gives the construction dates (seasons) and the types of traffic just after construction such as one-way and two-way.

Longitudinal Cores (C1-C5) Test Data

As mentioned in the sampling and testing plan five 6-inch diameter cores (Cl -C5) were taken at random locations longitudinally within a one mile long segment of the project. These cores were taken in the inside wheel track as shown in Figure 5. An additional core (C6) was taken to recover the aged asphalt cement and test its consistency.

Tables 8, 9 and 10 give the following core test data:

1. Asphalt content (JMF, mean, standard deviation, and CI);
2. Passing 1/2" (JMF, mean, standard deviation, and CI);
3. Passing No. 8 (JMF, mean, standard deviation, and CI);
4. Passing No. 200 (JMF, mean, standard deviation, and CI);
5. Layer thickness (mean and standard deviation);
6. Percent VTM (mean and standard deviation);
7. Penetration (77°F) and viscosity (140°F) in poises of recovered asphalt cement;
8. Percent fractured face count of recovered coarse aggregate (retained on No. 4 sieve);
9. Percent void content in the recovered fine aggregate (determined by the National Aggregate Association procedure to quantify particle shape and texture);
10. Percent natural sand in the fine aggregate (based on JMF); and
11. Type of manufactured sand (such as dolomite and sandstone).

Table 7B. Project Construction Dates and Traffic Control

Site	Construction Dates	Traffic	Condition	Maximum Surface Rut Depth	Average Surface Rut Depth	Maximum Rate of Rutting	Average Rate of Rutting
1	SUMMER	NONE	E	0.076	0.150	0.039	0.076
2	SUMMER	NONE	F	0.593	0.325	0.390	0.214
3	SPRING	N/A	F	0.243	0.400	0.126	0.208
4	SPRING	N/A	E	0.551	0.125	0.286	0.065
5	FALL	2-WAY	F	0.312	N/A	0.450	N/A
6	SUMMER	2-WAY	G	0.262	0.300	0.322	0.369
7	FALL	N/A	P	0.556	0.650	0.317	0.370
8	SPRING	N/A	E	N/A	N/A	N/A	N/A
9	FALL	N/A	F	0.683	0.700	0.117	0.120
10	FALL	N/A	G	0.371	0.250	0.076	0.052
12	SUMMER	N/A	E	0.186	0.100	0.109	0.059
13	SUMMER	N/A	G	0.168	0.200	0.098	0.117
14	SUMMER	N/A	G	0.411	0.350	0.262	0.223
15	SPRING	2-WAY	G	0.300	0.200	0.158	0.105
16	SUMMER	2-WAY	F	0.343	0.400	0.160	0.187
17	SUMMER	1-WAY	G	0.144	0.200	0.111	0.155
18	SPRING	1-WAY	F	1.627	0.650	0.965	0.386
19	SUMMER	1-WAY	G	0.393	0.350	0.234	0.208
20	SUMMER	1-WAY	E	0.212	0.225	0.164	0.174
22	N/A	I-WAY	F	0.487	0.400	0.304	0.250
23	FALL	N/A	F	0.329	0.400	0.215	0.261
24	SUMMER	N/A	E	0.117	0.100	0.104	0.088
25	SUMMER	N/A	E	0.333	0.225	0.181	0.122
26	SUMMER	N/A	E	0.060	0.000	0.050	0.000
27	SPRING	N/A	F	0.795	0.550	0.377	0.261
28	FALL	N/A	E	0.308	0.200	0.157	0.157
29	SPRING	N/A	F	0.317	0.350	0.544	0.600
30	SUMMER	N/A	p	1.664	1.300	0.843	0.658
31	SUMMER	N/A	F	0.200	0.250	0.258	0.323
32	SUMMER	N/A	G	1.038	0.200	0.842	0.162
33	FALL	N/A	F	0.232	0.300	0.188	0.243
34	FALL	N/A	G	0.183	0.275	0.134	0.202
35	SUMMER	NONE	E	0.036	0.000	0.026	0.000

N/A = Data not available

Table 8. Project Core Test Data (C1-C5) - Mix Composition - All Courses

				Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	**Layer**	**SR**	**Mix Type**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**
1	1	80	ID2W	6.2	5.7	0.134	-0.475	100	100.0	0.000	0.000	47	48.2	1.924	2.098	5.5	6.5	0.207	0.978
1	2	80	ID2B	4.6	4.1	0.279	-0.523	69	69.6	4.980	4.494	25	25.2	2.168	1.949	4.5	4.9	0.498	0.626
1	3	80	BCBC	3.3	3.3	0.321	0.535	---	---	---	---	---	26.7	1.155	1.915	---	---	---	--
2	1	279	ID2W	6.4	6.3	0.152	-0.195	100	100.0	0.000	0.000	43	45.4	3.782	4.147	5.0	4.7	0.687	-0.693
2	2	279	ID2B	4.2	5.0	0.367	-0.865	57	77.6	3.362	20.818	27	33.6	1.949	6.826	5.0	4.1	0.297	0.977
3	1	279	ID2W	7.2	6.7	0.130	-0.494	100	100.0	0.000	0.000	48	49.8	2.168	2.646	4.0	4.7	0.192	0.701
3	2	279	ID2B	5.0	4.9	0.427	-0.390	69	76.6	6.580	11.261	30	29.6	2.702	-2.449	4.0	4.1	0.327	0.303
4	1	279	ID2W	6.7	5.8	0.192	-0.897	100	100.0	0.000	0.000	43	43.2	3.962	3.550	4.0	6.0	0.518	2.092
4	2	279	ID2B	5.0	4.3	0.404	-0.823	67	66.4	16.426	-14.704	30	30.8	7.259	6.542	4.0	5.5	0.896	1.666
5	1	30	ID2W	6.5	6.3	0.148	-0.224	100	100.0	0.000	0.000	42	42.8	1.095	1.265	5.0	6.9	0.587	1.971
5	2	30	ID2B	4.8	4.9	0.303	0.297	64	70.6	3.362	7.253	25	28.4	1.817	3.768	4.0	5.1	0.383	1.171
6	1	30	ID2W	6.5	6.4	0.209	-0.241	100	100.0	0.000	0.000	43	39.4	1.517	-2.933	5.0	6.5	0.336	1.491
6	2	30	ID2B	4.6	4.6	0.287	0.250	62	76.0	13.565	18.276	25	28.5	5.568	5.958	4.4	5.1	0.602	0.893
6	3	30	L-ID2W	---	7.6	---	---	---	100.0	---	---	---	61.8	---	---	---	7.0	---	--
7	1	70	ID2W	5.9	6.0	0.277	0.261	100	100.0	0.000	0.000	44	41.0	1.581	-3.317	5.0	6.0	0.455	1.098
7	2	70	ID2B	4.5	6.0	---	---	63	76.7	---	---	25	35.4	---	---	4.0	6.5	---	--
7	3	70	L-ID2W	---	7.4	---	---	---	100.0	---	----	---	45.2	---	---	---	3.5	---	--
8	1	76	ID2W	6.5	6.1	0.045	-0.422	100	100.0	0.000	0.000	45	43.0	1.000	-2.191	5.0	6.3	0.316	1.330
8	2	76	L	---	7.3	---	---	---	100.0	---	---	---	56.5	---	---	---	13.5	---	--
8	3	76	ID2B	---	4.6	0.349	---	---	73.4	5.273	---	---	30.8	3.347	---	---	5.0	0.522	--
8	4	76	FJ	---	6.5	---	---	---	100.0	---	---	---	68.3	---	---	---	11.7	---	--
9	1	70/76	ID2W	8.8	8.0	0.297	-0.833	100	100.0	0.000	0.000	45	45.0	2.345	2.098	3.0	5.2	0.540	2.233
9	2	70/76	ID2B	---	4.8	0.303	---	---	69.6	3.362	---	---	28.2	1.304	---	---	5.3	0.998	---
9	3	70/76	ID2W	---	5.7	---	---	---	100.0	---	---	---	49.7	---	---	---	9.5	---	---

“--- ” Data Not Available

Table 8. Project Core Test Data (C1-C5) - Mix Composition - All Courses (Continued)

				Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	**Layer**	**SR**	**Mix Type**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**
10	1	70/76	ID2W	6.9	6.2	0.192	-0.701	100	100.0	0.000	0.000	45	45.2	1.924	1.732	4.5	6.8	0.687	2.361
10	2	70/76	ID2B	---	4.3	0.207	---	---	76.6	13.297	---	---	33.8	5.541	---	---	5.1	0.620	--
11	1	90	ID2W	7.2	6.8	0.297	-0.447	100	100.0	0.000	0.000	50	42.8	2.588	-7.563	4.0	6.4	0.764	2.534
11	2	90	BCBC	4.6	4.5	0.217	---	66	81.0	3.391	---	28	30.4	3.050	---	---	6.4	0.385	---
11	3	90	BCBC	4.6	3.9	0.377	0.000	66	77.8	1.789	---	28	29.2	2.490	---	---	6.4	0.863	---
12	1	81	ID2W	6.6	6.3	0.182	-0.377	100	100.0	0.000	0.000	48	52.2	3.033	5.000	4.5	5.4	0.592	1.044
12	2	81	BCBC	4.7	4.8	0.421	0.385	61	64.8	4.764	5.710	30	33.8	3.271	4.796	5.0	5.4	0.400	0.537
12	3	81	L-ID2B	---	6.6	---	---	---	100.0	---	---	---	44.0	---	---	---	6.2	---	--
13	1	81	ID2W	6.6	6.1	0.207	-0.496	100	100.0	0.000	0.000	48	51.0	3.162	4.123	4.5	6.1	0.329	1.666
13	2	81	ID2B	4.7	4.6	0.332	-0.313	61	61.8	7.887	7.099	30	32.4	2.702	3.406	5.0	5.7	0.563	0.846
13	3	81	ID2B	---	4.4	---	---	---	58.4	---	---	---	26.8	---	---	---	4.2	---	--
13	4	81	ID2W	---	6.5	---	---	---	100.0	---	---	---	46.3	---	---	---	7.5	---	--
14	1	81	ID2W	6.6	6.4	0.122	-0.228	100	100.0	0.000	0.000	45	47.0	2.915	3.286	5.0	6.7	0.804	1.828
14	2	81	ID2B	5.2	5.0	0.230	-0.261	65	66.6	3.578	3.578	30	30.0	2.345	2.098	4.0	5.2	0.838	1.449
14	3	81	ID2W	---	---	---	---	---	100.0	---	---	---	43.0	---	---	---	3.4	---	---
14	4	81	ID2W	---	---	---	---	---	100.0	---	---	---	44.4	---	---	---	4.7	---	---
14	5	81	ID2B	---	3.6	0.306	---	---	---	---	---	---	---	---	---	---	---	---	---
15	1	78	ID2W	6.4	6.3	0.270	-0.279	100	100.0	0.000	0.000	45	47.0	2.345	2.898	5.0	6.5	0.691	1.660
15	2	78	ID2B	4.3	3.9	0.487	-0.605	63	60.8	9.576	-8.843	30	27.8	4.658	-4.712	4.0	5.9	0.826	2.076
16	1	78	ID2W	6.4	5.9	0.164	-0.502	100	100.0	0.000	0.000	45	46.8	2.049	2.569	5.0	7.5	0.451	2.572
16	2	78	ID2B	4.5	4.2	0.356	-0.424	61	72.0	3.536	5.916	28	30.8	3.899	4.472	5.0	5.8	0.907	1.153
17	1	81	ID2W	6.2	5.8	0.114	-0.452	100	100.0	0.000	0.000	48	48.6	1.673	1.612	4.0	5.1	0.100	1.104
17	2	81	ID2B	4.5	4.1	0.635	-0.672	55	63.2	10.378	12.385	30	31.0	4.416	4.074	4.0	4.5	0.434	0.665
18	1	81	ID2W	5.7	5.5	0.230	-0.316	100	100.0	0.000	0.000	42	42.8	1.789	1.789	5.0	5.9	0.667	1.113
18	2	81	ID2B	4.5	4.6	0.219	0.205	60	72.8	2.387	12.977	29	30.6	1.342	2.000	4.0	5.2	0.719	1.344
18	3	81	BCBC	---	4.5	---	---	---	68.0	---	---	---	35.8	---	---	---	8.4	---	--

"--- " Data Not Available

Table 8. Project Core Test Data (C1-C5) - Mix Composition - All Courses (Continued)

				Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	**Layer**	**SR**	**Mix Type**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**	**JMF**	**Avg**	**SD**	**CI**
19	1	222	ID2W	6.2	5.9	0.250	---	100	100.0	0.000	0.000	45	44.8	1.643	-1.483	5.0	6.8	0.311	1.802
19	2	222	ID2W	6.2	6.1	0.164	---	---	100.0	0.000	---	---	60.6	4.159	16.04	5.0	7.7	0.647	2.722
19	3	222	ID2B	---	4.2	0.356	---	---	62.0	9.028	---	---	25.8	3.701	---	---	4.0	1.496	--
20	1	222	ID2W	6.6	6.5	0.230	-0.249	100	100.0	0.000	0.000	45	46.6	1.517	2.098	5.0	6.7	0.416	1.779
20	2	222	ID2B	4.5	4.1	0.415	-0.531	56	55.8	9.039	-8.087	25	24.0	2.236	-2.236	4.0	3.5	0.568	-0.686
20	3	222	L-ID2B	---	6.2	---	---	---	100.0	---	---	---	46.6	---	---	---	5.6	---	--
22	1	76	ID2W	5.7	5.6	0.255	-0.249	100	100.0	0.000	0.000	43	43.4	1.140	1.095	5.2	7.1	0.677	2.032
22	2	76	---	4.5	4.4	0.130	-0.141	63	73.0	3.674	10.526	26	24.8	2.683	-2.683	4.0	4.6	0.462	0.762
22	3	76	ID2W	---	5.4	0.265	---	---	100.0	0.000	---	---	45.0	1.225	---	---	5.8	0.602	--
23	1	76	ID2W	5.8	5.6	0.288	-0.293	100	100.0	0.000	0.000	43	47.4	0.548	4.427	5.2	5.2	0.497	0.445
23	2	76	ID2B	4.5	4.4	0.365	-0.355	63	75.0	8.124	14.893	26	27.4	4.159	3.975	4.0	4.9	1.408	1.572
23	3	76	ID2B	4.5	4.4	0.299	-0.287	63	72.0	4.967	10.886	26	26.8	3.594	3.202	4.0	4.6	0.377	0.705
24	1	76	ID2W	6.0	5.6	0.100	-0.134	100	100.0	0.000	0.000	43	48.2	2.588	5.692	4.5	6.9	0.503	1.797
24	2	76	ID2B	4.1	3.8	0.688	-0.902	57	59.4	9.154	8.591	30	32.4	3.912	7.294	4.0	5.4	0.559	1.505
24	3	76	ID2B	4.1	3.6	0.403	-0.942	57	54.0	5.292	-9.220	30	30.8	2.754	5.315	4.0	5.8	0.768	1.895
25	1	11	ID2W	---	6.0	0.279	---	---	100.0	0.000	---	---	42.6	2.191	---	---	5.4	1.016	---
25	2	11	SP B	4.0	3.7	0.548	---	42	46.2	8.701	---	19	20.2	3.564	---	2.5	3.4	0.179	---
26	1	83	ID2W	6.1	5.7	0.148	-0.440	100	100.0	0.000	0.000	40	41.6	1.673	2.191	4.5	5.6	0.378	1.113
26	2	83	ID2B	4.2	4.3	0.370	0.341	55	69.0	6.000	14.993	28	28.8	3.493	3.225	4.0	5.6	0.904	1.811
27	1	83	ID2W	6.0	5.6	0.217	-0.210	100	100.0	0.000	0.000	38	37.2	1.483	-1.549	5.0	5.7	0.623	0.926
27	2	83	ID2B	4.5	4.3	0.207	-0.303	62	67.6	5.857	7.668	25	27.4	2.302	3.162	4.0	4.8	1.408	1.515
28	1	22	ID3W	5.2	5.3	0.114	0.190	100	100.0	0.000	0.000	35	36.2	2.280	2.366	5.0	3.8	0.572	-1.323
28	2	22	LEVEL	---	5.2	---	---	---	100.0	---	---	---	40.5	---	---	---	4.4	---	---
28	3	22	ID2W	---	5.9	---	---	---	100.0	---	---	---	40.3	---	---	---	6.5	---	---
28	4	22	ID28	---	3.9	---	---	---	59.4	---	---	---	27.9	---	---	---	4.4	---	---

"--- " Data Not Available

Table 8. Project Core Test Data (C1-C5) - Mix Composition - All Courses (Continued)

				Asphalt Content, %				Passing 1/2 inch				Passing #8				Passing #200			
Site	Layer	SR	Mix Type	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI	JMF	Avg	SD	CI
29	1	11	ID3W	5.0	4.7	0.356	-0.452	100	100.0	0.000	0.000	37	35.6	2.702	-2.793	5.0	5.1	0.410	0.392
29	2	11	ID2B	4.5	6.7	---	---	63	---	---	---	27	---	---	---	4.0	---	---	--
30	1	11	ID2W	6.4	5.7	0.658	0.000	100	100.0	0.000	0.000	40	41.4	1.140	-1.732	4.0	5.9	1.521	2.304
30	2	11	ID2B	---	5.4	---	---	---	81.0	---	---	---	37.4	---	---	---	5.3	---	--
31	1	220	ID2W	6.3	6.2	0.346	-0.326	100	100.0	0.000	0.000	40	45.4	5.857	7.523	5.0	5.8	1.489	1.575
31	2	220	ID2B	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
31	3	220	LEVEL	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
32	1	70	ID2W	6.0	5.6	0.370	-0.504	100	100.0	0.000	0.000	45	46.8	0.837	1.949	5.0	6.8	0.228	1.792
32	2	70	ID2B	3.9	4.5	0.305	0.307	55	63.8	6.340	10.469	30	29.6	0.894	-0.894	4.0	4.3	0.167	0.307
33	1	70	ID2W	6.0	5.7	0.114	-0.279	100	100.0	0.000	0.000	45	49.0	1.871	2.504	5.0	7.5	0.522	2.504
33	2	70	1112B	3.9	4.4	0.554	0.527	55	60.4	6.387	7.861	30	33.0	5.848	5.273	4.0	5.6	0.763	1.758
34	1	70	ID2W	6.3	5.9	0.195	-0.400	100	100.0	0.000	0.000	45	50.0	2.550	5.495	6.0	6.9	1.677	1.770
34	2	70	ID2W	5.6	5.8	0.000	0.000	100	100.0	---	---	45	50.3	---	---	6.0	7.0	---	---
34	3	70	LEVEL	---	6.8	---	---	---	100.0	---	---	---	48.1	---	---	---	7.3	---	---
34	4	70	ID2B	---	5.2	---	---	---	80.2	---	---	---	36.9	---	---	---	4.8	---	---
35	1	80	ID2W	6.1	5.7	0.158	-0.245	100	100.0	0.000	0.000	47	48.0	1.012	4.911	5.0	0.6	0.402	-1-234
35	2	80	ID2B	3.9	4.0	0.381	0.690	63	67.6	5.413	6.678	30	30.0	3.317	2.966	4.5	4.9	0.828	0.841
35	3	80	BCBC	---	4.0	0.255	0.460	---	---	---	---	---	32.6	3.782	4.266	---	---	---	---
Layer 1																			
	Average:			6.33	6.00	0.22	-0.34	100.0	100.0	0.00	0.00	43.85	44.07	2.11	1.68	4.77	5.91	0.59	1.36
	Std Deviation:			0.631	0.541	0.109	0.235	0.000	0.000	0.000	0.000	3.286	6.508	1.024	3.097	0.546	1.247	0.357	0.963
Layer 2																			
	Average:			4.57	4.75	0.34	-0.18	62.11	72.55	6.63	7.26	28.26	32.60	3.35	3.12	4.26	5.46	0.65	1.12
	Std Deviation:			0.495	0.807	0.149	0.463	9.211	12.724	3.714	8.324	4.204	8.485	1.500	4.151	0.613	1.698	0.310	0.694

“--- ” Data Not Available

Table 9. Project Core Test Data (C1-C5) - Thickness, Voids, and Abson Recovery - All Courses

Site	Layer	SR	Mix Type	Layer Thickness		% Voids			Recovered Asphalt	
				Avg	SD	Avg	SD	Min	PEN (0.1 mm) (77F)	VISC (Poises) (140F)
1	1	80	ID2W	2.4050	0.015	3.96	0.754	3.20	27	12782
1	2	80	ID2B	2.4550	0.025	5.20	1.241	3.80	33	8376
1	3	80	BCBC	---	---	8.70	1.072	8.20	---	--
2	1	279	ID2W	1.5630	0.117	1.91	0.296	1.51	39	13475
2	2	279	ID2B	2.3130	0.212	1.46	0.251	1.10	45	7273
3	1	279	ID2W	1.4500	0.082	1.44	0.404	1.00	46	5678
3	2	279	ID2B	2.6570	0.231	2.38	1.054	1.60	45	5799
4	1	279	ID2W	1.5880	0.144	2.40	0.361	1.90	35	9728
4	2	279	ID2B	2.8130	0.348	1.54	0.428	0.90	---	--
5	1	30	ID2W	1.4500	0.230	3.12	0.563	2.50	54	5571
5	2	30	ID2B	2.7250	0.220	1.82	0.740	1.10	64	3974
6	1	30	ID2W	1.4500	0.142	2.96	1.494	1.20	38	19498
6	2	30	ID2B	2.8130	0.125	3.53	1.401	2.30	71	12373
6	3	30	L-ID2W	---	---	---	---	---	---	--
7	1	70	ID2W	2.3390	0.010	2.52	0.549	1.90	45	6577
7	2	70	ID2B	---	---	---	---	---	35	10673
7	3	70	L-ID2W	---	---	---	---	---	---	--
8	1	76	ID2W	1.4750	0.050	0.92	0.256	0.50	51	7869
8	2	76	L	---	---	---	---	---	---	--
8	3	76	ID2B	2.0250	0.140	1.00	0.228	0.70	31	25206
8	4	76	FJ	---	---	---	---	---	---	--
9	1	70/76	ID2W	0.8000	0.400	1.92	0.444	1.20	40	10607
9	2	70/76	ID2B	1.8000	0.190	1.98	0.303	1.70	44	8215
9	3	70/76	ID2W	---	---	---	---	---	---	--
10	1	70/76	ID-2W	0.9875	0.160	0.61	0.503	0.04	44	7921
10	2	70/76	ID2B	2.4625	0.540	3.98	1.252	2.70	36	15388
11	1	90	ID2W	1.3125	0.153	-0.28	0.726	-1.00	46	5573
11	2	90	BCBC	3.9625	0.912	3.74	1.977	2.00	20	24867
11	3	90	BCBC	4.6000	0.596	6.70	0.608	6.00	---	--

“--- ” Data Not Available

Table 9. Project Core Test Data (C1-C5) - Thickness, Voids, and Abson Recovery - All Courses (Continued)

Site	Layer	SR	Mix Type	Layer Thickness		% Voids			Recovered Asphalt	
				Avg	SD	Avg	SD	Min	PEN (0.1 mm) (77F)	VISC (Poises) (140F)
12	1	81	ID2W	1.4375	0.088	4.92	1.501	2.80	31	15253
12	2	81	BCBC	5.1250	1.180	1.98	0.729	1.00	58	3705
12	3	81	L-ID2B	---	---		---	---	---	---
12	4	81	---	---	---	---	---	---	---	---
13	1	81	ID2W	1.1375	0.156	2.44	0.422	1.90	47	9493
13	2	81	ID2B	6.4000	0.978	0.18	1.156	-1.10	65	3977
13	3	81	ID2B	---	---	---	---	---	---	---
13	4	81	ID2W	---	---	---	---	---	---	---
14	1	81	ID2W	1.4000	0.137	2.28	0.694	1.30	46	9440
14	2	81	ID2B	4.2500	0.500	4.04	1.426	3.00	47	7356
14	3	81	ID2W	---	---	---	---	---	---	---
14	4	81	ID2W	---	---	---	---	---		
14	5	78	ID2B	4.5	0.500	4.10	1.217	3.30	---	---
15	1	78	ID2W	1.4125	0.105	2.98	0.963	1.80	57	5105
15	2	78	ID2B	1.4625	0.240	3.74	1.278	2.10	54	5613
16	1	78	ID2W	1.3125	0.342	3.94	0.688	3.10	40	8858
16	2	78	ID2B	1.4375	0.250	3.88	1.879	2.60	49	5709
17	1	81	ID2W	1.3625	0.284	3.14	0.709	2.10	41	15623
17	2	81	ID2B	1.9375	0.225	4.00	1.500	2.20	54	6248
18	1	81	ID2W	1.5250	0.137	2.32	0.589	1.70	48	5856
18	2	81	ID2B	2.6500	0.137	1.70	0.412	1.40	49	5855
18	3	81	BCBC	---	---	---	---	---	---	---
19	1	222	ID2W	---	---	4.28	1.542	2.70	25	18664
19	2	222	ID2W	1.5250	0.056	3.64	0.805	2.70	---	---
19	3	222	ID2B	2.0000	0.088	4.56	2.721	1.90	23	22032
20	1	222	ID2W	---	---	2.72	0.356	2.40	37	11262
20	2	222	ID2B	---	---	2.10	1.321	0.80	48	6000
20	3	222	L-ID2B	---	---	---	---	---	---	---
22	1	76	ID2W	0.9875	0.143	3.06	0.699	2.20	31	17964
22	2	76	---	2.4375	0.212	1.44	0.559	0.50	68	4007
22	3	76	ID2W	---	---	---	---	---	---	---

"--- " Data Not Available

Table 9. Project Core Test Data (C1-C5) - Thickness, Voids, and Abson Recovery - All Courses (Continued)

Site	Layer	SR	Mix Type	Layer Thickness		% Voids			Recovered Asphalt	
				Avg	SD	Avg	SD	Min	PEN (0.1 mm) (77F)	VISC (Poises) (140F)
23	1	76	ID2W	1.8125	0.543	2.64	0.598	2.30	42	7401
23	2	76	ID2B	2.3375	0.503	1.66	0.850	0.50	44	6100
23	3	76	ID2B	---	---	---	---	---	---	---
24	1	76	ID2W	1.3125	0.221	3.82	1.320	2.60	34	11280
24	2	76	ID2B	2.2750	0.656	2.58	1.580	1.10	37	9357
24	3	76	ID2B	---	---	---	---	---	---	---
25	1	11	ID2W	1.3125	0.313	3.64	1.443	2.10	52	7018
25	2	11	SP B	3.5250	0.399	4.68	2.134	1.30	53	7004
26	1	83	ID2W	1.5750	0.068	7.40	0.274	7.00	45	6643
26	2	83	ID2B	2.5375	0.169	5.44	1.798	4.30	45	5661
27	1	83	ID2W	1.5750	0.288	2.50	0.381	2.00	38	9690
27	2	83	ID2B	2.1875	0.165	2.90	0.784	1.80	45	8165
28	1	22	ID3W	1.9750	0.071	5.10	0.755	4.60	44	5051
28	2	22	LEVEL	0.8875	0.052	---	---	---	---	---
28	3	22	ID2W	---	---			---	38	9037
28	4	22	ID2B	---	---	---	---	---	---	---
29	1	11	ID3W	1.9875	0.413	2.60	0.354	2.20	41	7752
29	2	11	ID2B	---	---	---	---	---	18	109172
30	1	11	ID2W	1.5630	0.063	2.92	1.126	2.10	59	5753
30	2	11	ID2B	---	---	---	---	---	56	7904
31	1	220	ID2W	---	---	5.86	1.590	4.00	42	5701
31	2	220	ID2B	---	---	---	---	---	35	14262
31	3	220	LEVEL	---	---	---	---	---		
32	1	70	ID2W	1.3750	0.198	4.38	1.410	3.00	35	14262
32	2	70	ID2B	1.6750	0.401	2.48	0.988	1.40	55	5729
33	1	70	ID2W	1.3000	0.326	4.90	0.822	3.80	44	8583
33	2	70	ID2B	1.5250	0.185	3.58	1.642	1.90	40	9740

"--- " Data Not Available

Table 9. Project Core Test Data (C1-C5) - Thickness, Voids, and Abson Recovery - All Courses (Continued)

Site	Layer	SR	Mix Type	Layer Thickness Avg	Layer Thickness SD	% Voids Avg	% Voids SD	% Voids Min	Recovered Asphalt PEN (0.1 mm) (77F)	Recovered Asphalt VISC (Poises) (140F)
34	1	70	ID2W	1.5000	0.000	2.54	0.230	2.40	38	19498
34	2	70	ID2W	---	---	---	---	---	---	---
34	3	70	LEVEL	---	---	---	---	---	---	---
34	4	70	ID2B	---	---	---	---	---	---	---
35	1	80	ID2W	---	---	5.82	1.126	4.37	37	8378
35	2	80	ID2B	---	---	5.94	1.043	4.88	49	6295
35	3	80	BCBC	---	---	5.22	0.801	4.00	---	---
Layer 1										
	Average:			1.49	0.18	3.17	0.76	2.31	41.74	9994.32
	Std Deviation:			0.342	0.130	1.544	0.423	1.389	7.690	4335.142
Layer 2										
	Average:			2.62	0.35	3.02	1.13	1.84	46.97	11544.72
	Std Deviation:			1.188	0.290	1.384	0.510	1.221	12.319	18932.15

"--- " Data Not Available

Table 10. Project Core Test Data (C1-C5) - Aggregate Properties

Site	Layer	SR	Mix Type	Coarse Aggregate PCT Total Mix	Coarse Aggregate Crush Count	Fine Aggregate PCT Total Mix	Fine Aggregate PCT Voids Total	Fine Aggregate PCT Nat'l Sand in F.A.	Fine Aggregate Man'f. Sand Type
1	1	80	ID2W	45.2	98	54.8	46.0	0.0	DO
1	2	80	ID2B	71.2	100	28.8	44.6	0.0	DO
2	1	279	ID2W	48.0	86	52.0	44.5	25.0	LS
2	2	279	ID2B	66.5	100	33.5	43.5	25.1	LS
3	1	279	ID2W	32.3	86	67.7	43.9	100.0	---
3	2	279	ID2B	61.5	---	38.5	---	100.0	---
4	1	279	ID2W	48.3	86	51.7	42.6	74.9	LS
4	2	279	ID2B	61.5	---	38.5	43.0	100.0	---
5	1	30	ID2W	49.7	87	50.3	44.5	25.0	CS
5	2	30	ID2B	72.9	100	27.1	43.i	25.1	CS
6	1	30	ID2W	49.6	82	50.4	44.2	50.0	CS
6	2	30	ID2B	70.9	100	29.1	43.0	50.2	CS
7	1	70	ID2W	48.2	66	51.8	43.5	24.5	CS
7	2	70	ID2B	66.6	100	33.4	---	24.3	CS
8	1	76	ID2W	47.6	94	52.4	41.8	100.0	---
9	1	70/76	ID2W	---	100	---	---	---	LS
10	1	70/76	ID2W	41.2	---	58.8	39.9	0.0	SS
11	1	90	ID2W	---	96	---	42.2	---	---
12	1	81	ID2W	42.3	100	57.7	42.7	25.0	SS
12	2	81	BCBC	68.6	100	31.4	44.2	0.0	SS
13	1	81	ID2W	42.3	100	57.7	44.6	25.0	SS
13	2	81	ID2B	68.6	100	31.4	43.4	0.0	SS
14	1	81	ID2W	49.7	100	50.3	46.0	0.0	SS
14	2	81	ID2B	77.7	100	22.3	44.5	0.0	SS
15	1	78	ID2W	48.9	87	51.1	42.8	0.0	LS-DO
15	2	78	ID2B	61.7	100	38.3	44.2	0.0	LS-DO
16	1	78	ID2W	45.2	83	54.8	44.5	0.0	LS-DO
16	2	78	ID2B	70.2	---	29.8	42.8	50.0	LS-DO

"--- " Data Not Available

Table 10. Project Core Test Data (C1-C5) - Aggregate Properties (Continued)

Site	Layer	SR	Mix Type	Coarse Aggregate PCT Total Mix	Coarse Aggregate Crush Count	Fine Aggregate PCT Total Mix	Fine Aggregate PCT Voids Total	Fine Aggregate PCT Nat'l Sand in F.A.	Fine Aggregate Man'f. Sand Type
17	1	81	ID2W	42.0	---	58.0	42.6	100.0	---
17	2	81	ID2B	65.3	---	34.7	---	100.0	---
18	1	81	ID2W	56.2	100	43.8	---	0.0	DO
18	2	81	ID2B	69.8	100	30.2	---	0.0	DO
19	1	222	ID2W	53.4	100	46.6	45.3	0.0	LS-DO
19	2	222	ID2W	53.4	100	46.6	---	0.0	--
20	1	222	ID2W	55.0	100	45.0	44.9	0.0	LS
20	2	222	ID2B	73.6	100	26.4	44.6	0.0	LS
22	1	76	ID2W	52.0	---	48.0	---	50.0	---
22	2	76	---	71.0	---	29.0	---	44.8	---
23	1	76	ID2W	52.0	---	48.0	---	50.0	---
23	2	76	ID2B	71.0	---	29.0	---	44.8	---
24	1	76	ID2W	57.4	92	42.6	43.6	20.0	DO
24	2	76	ID2B	74.5	---	25.5	43.9	20.0	DO
25	1	11	ID2W	---	---	---	---	---	SS
25	2	11	SP B	75.0	100	25.0	46.5	0.0	SS
26	1	83	ID2W	48.2	100	51.8	---	0.0	SS
26	2	83	ID2B	63.4	100	36.6	46.8	0.0	SS
27	1	83	ID2W	---	---	---	---	---	---
27	2	83	ID2B	---	---	---	---	---	---
28	1	22	ID3W	52.8	100	47.2	46.1	0.0	SS
28	2	22	LEVEL	---	---	---	---	---	SS
29	1	11	ID3W	---	---	---	---	---	---
29	2	11	ID2B	---	---	---	---	---	---
30	1	11	ID2W	---	---	---	---	---	---
31	1	220	ID2W	46.8	---	53.2	---	0.0	LS
31	2	220	ID2B	---	---	---	---	---	---

"--- " Data Not Available

Table 10. Project Core Test Data (C1-C5) - Aggregate Properties (Continued)

Site	Layer	SR	Mix Type	Coarse Aggregate		Fine Aggregate			
				PCT Total Mix	Crush Count	PCT Total Mix	PCT Voids Total	PCT Nat'l Sand in F.A.	Man'f. Sand Type
32	1	70	ID2W	46.4	100	53.6	44.8	0.0	LS
32	2	70	ID2B	65.0	100	35.0	45.0	0.0	LS
33	1	70	ID2W	46.4	100	53.6	44.8	0.0	LS
33	2	70	ID2B	65.0	100	35.0	45.0	0.0	LS
34	1	70	ID2W	51.0	100	49.0	46.3	0.0	LS-DO
34	2	70	ID2W	69.9	100	30.1	---	0.0	LS-DO
35	1	80	ID2W	53.4	---	46.6	45.3	0.0	DO
35	2	80	ID2B	66.3	100	33.7	45.5	0.0	DO
Layer 1									
	Average:			48.27	93.46	51.73	44.06	23.90	
	Std Deviation:			5.218	8.631	5.177	1.536	33.060	
Layer 2									
	Average:			68.04	100.00	31.96	44.33	23.37	
	Std Deviation:			5.218	0.000	5.218	1.148	33.244	

DO=dolomite LS=limestone CS=calcareous SS=sandstone

"--- " Data Not Available

Mix composition: Mix composition was determined by extracting core samples. Generally the asphalt content measured from the cores was deficient from the JMF asphalt content for both wearing and binder courses with the average CI values equal to -0.34 and -0.18, respectively. The percentage of material passing No. 8 sieve was also generally higher than the JMF values for both wearing and binder courses with the average CI values equal to 1.68 and 3.12, respectively. As expected, these values are higher than those obtained on loose mixes at the time of construction because some degradation takes place under roller, under subsequent traffic, and from coring and sawing operations. The percentage of minus 200 was also significantly higher than the JMF values for both courses with the average CI values equal to +1.36 and +1.12, respectively. The percentage of material passing the 1/2" sieve in case of the binder course had a CI value of +7.26 indicating that the produced mix was significantly finer than the “designed mix” although some degradation had taken place due to reasons previously mentioned. The average percentage passing 1/2" sieve (72.6 percent) of the “produced mix” exceeded the average percentage passing 1/2" (62.1 percent) of the “designed mix” by 10.5 percent.

Voids in total mix (VTM): The statistical analysis of VTM data (Table 9) obtained by testing cores Cl through C5 is as follows:

	Wearing Course	Binder Course
Number of Projects	34	27
Mean	3.17	3.02
Standard Deviation	1.54	1.38
95% Confidence Limits	0.0 - 7.4	0.3 - 5.9

The average VTM values in both courses are very low. According to past experience MA pavements approach the potential for rutting when the VTM is 3 percent or less. Since these are average values obviously there are many projects which have VTM less than 3 percent. It should be noted that the average mix design VTM values were 3.6 and 3.7 percent, respectively for wearing and binder courses as reported earlier. Generally, the HMA pavement is densified by the traffic to an optimum level during the first three years in service. Further examination of VTM data obtained on projects which were in service for three or more years (at the time of coring in 1989) reveals even lower values for projects 3 or more years in age. These older projects have average values of 2.61 and 2.85 percent, respectively for wearing and binder courses. Thus, the VTM data indicates that the Pennsylvania HMA mixtures are compacted by traffic generally to a higher degree than that provided by laboratory compaction. Therefore, the laboratory compaction effort needs to be increased.

Recovered asphalt cement properties: Aged asphalt cement was recovered from Core C6 and tested for penetration at 77°F and viscosity (poises) at 140°F. The data is given in Table 9. The statistical analysis of data is as follows:

	Wearing Course		Binder Course	
	Penetration	Viscosity	Penetration	Viscosity
Number of Projects	34	34	29	29
Mean	41.7	9,994	47.0	11,544
Standard Deviation	7.7	4,335	12.3	18,932
95% Confidence Limits	25 - 59	5,051 - 19,498	18 - 71	3,705 - 109,172

The recovered asphalt cement test data appears reasonable considering the age of the pavement ranged from two to 19 years (at the time of coring in 1989). An unusually hard asphalt cement was encountered in the binder course of Project 29 (Route 11 - Camp Hill Bypass). Surprisingly, the asphalt cements in the wearing courses of the three oldest projects (Projects 8, 9 and 25) did not age much in spite of their ages ranging from 14 to 18 years.

Recovered aggregate properties: Table 10 gives the following data on recovered from Cores C1-C5:

1. Coarse aggregate percentage in total mix and its fractured face count;
2. Fine aggregate percentage in total mix;
3. Percentage of natural sand in the fine aggregate;
4. Type of manufactured sand, if used ; and
5. Fine aggregate particle shape and texture obtained in terms of percentages of void content using the National Aggregate Association (NAA) method. High void contents indicate angular and rough textured fine aggregate particles.

The percentage of coarse aggregate in the wearing course (Layer 1) ranged from 32 to 57 percent averaging 48 percent. The fractured face count of the coarse aggregate in the wearing course ranged from 66 percent (gravel) to 100 percent (stone) averaging 93 percent. The percentage of coarse aggregate in the binder course (Layer 2) ranged from 53 to 78 percent averaging 68 percent. All coarse aggregates in the binder course for which data is available are 100 percent crushed stone aggregates.

The percentage of fine aggregate in the wearing course (Layer 1) mix ranged from 43 to 68 percent averaging 52 percent. The percentage of natural sand in total fine aggregate ranged from 0 to 100 percent averaging 24 percent in the wearing course. The percentage of fine aggregate in the binder course (Layer 2) ranged from 22 to 47 percent averaging 32 percent. The percentage of natural sand in total fine aggregate ranged from O to 100 percent averaging 23 percent in the binder course.

Transverse Core (C7-C11) Test Data

Tables 11 and 12 give the following test data obtained on five transverse cores (C7-C11) from each project:

1. M (voids in total mix): Individual cores, average, minimum, and lower 20th percentile.
2. Average VTM, VMA and VFA of specimens prepared by recompaction using 3 compaction procedures: gyratory (GTM), mechanical Marshall with rotating base and slanted foot, and conventional mechanical Marshall with static base.
3. GSI (Gyratory Shear Index) obtained during recompaction in the gyratory compactor. GSI is believed to be an indicator of the rutting potential of HMA mixes.
4. Average Permanent deformation (inch/inch) of the core specimens measured by creep test. Values obtained after 15, 30, 45 and 60 minutes under a constant load are included.

The results of statistical analysis of the preceding cores C7-C11 data is given at the bottom of Tables 11 and 12. The following observations are made.

1. Average VTM values of 3.75 and 3.63 percent, respectively, for wearing and binder courses are higher than those obtained from cores Cl through C5 sampled longitudinally. This can be attributed to the location of cores - all Cl through C5 cores were taken in the inside wheel track (where most densification occurs) whereas cores C7 through Cl 1 were taken transversely across the pavement including areas other than wheel tracks. Cores C7-C11 were taken at a location where the most rutting had occurred.

   When pavements undergo a shearing failure the voids can increase as the aggregate particles slide up and over one another. Pavements that exhibit plastic flow are undergoing a shear type failure and the air voids across the pavement change. The voids

Table 11. Project Core Test Data (C7-C11) - Void Contents - All Courses

Site	Layer	SR	Mix Type	% In-Place Voids								Recompacted Voids %		
				C7	C8	C9	C10	C11	Avg	Min	20th Pct'l	Gyratory Comp	Rotating Base	Static Base
1	1	80	ID2W	5.19	4.99	4.59	5.31	5.15	5.05	4.59	4.81	2.8	1.6	2.5
1	2	80	ID2B	5.06	7.53	6.02	6.37	7.45	6.49	5.06	5.62	2.6	3.2	3.7
1	3	80	BCBC	8.41	7.91	8.18	8.41	8.18	8.22	7.91	8.04	4.0	5.0	5.0
2	1	279	ID2W	1.97	2.26	2.39	2.05	2.09	2.15	1.97	2.01	2.5	2.3	3.2
2	2	279	ID2B	2.54	2.08	3.00	2.46	1.83	2.38	1.83	2.00	1.3	1.7	2.1
3	1	279	ID2W	1.40	1.35	1.35	1.35	1.35	1.36	1.35	1.34	0.8	1.4	1.5
3	2	279	ID2B	2.10	2.22	3.31	1.72	2.98	2.47	1.72	1.91	0.9	0.8	1.4
4	1	279	ID2W	6.66	5.20	5.58	5.24	5.04	5.54	5.04	4.99	4.2	2.2	2.5
4	2	279	ID2B	2.94	3.39	2.62	1.65	2.78	2.68	1.65	2.14	1.3	1.1	1.5
5	1	30	ID2W	2.52	2.40	1.68	2.02	1.30	1.98	1.30	1.56	1.2	1.6	1.6
5	2	30	ID2B	1.30	1.67	1.79	1.71	1.71	1.64	1.30	1.47	0.6	0.7	1.0
6	1	30	ID2W	4.86	3.48	3.35	3.48	3.52	3.74	3.35	3.21	2.3	2.1	2.5
6	2	30	ID2B	4.69	3.84	4.08	3.07	3.11	3.76	3.07	3.18	1.2	2.3	2.4
6	3	30	LEVEL	---	4.27	4.14	4.14	4.74	4.32	4.14	4.08	-0.6	---	-0.6
7	1	70	ID2W	2.50	2.34	4.42	2.13	1.50	2.58	1.50	1.66	1.5	---	1.1
7	2	70	ID2B	3.68	4.45	3.03	2.26	2.59	3.2 0	2.26	2.47	1.0	1.4	1.5
7	3	70	LEVEL	3.22	0.00	3.35	2.96	2.96	2.50	0.00	1.32	0.5	---	1.0
8	1	76	ID2W	0.63	1.21	4.35	1.00	1.88	1.81	0.63	0.56	3.7	1.0	1.5
8	2	76	LEVEL	6.21	4.79	7.29	6.94	-3.36	4.37	-3.36	0.65	3.8	---	3.7
8	3	76	ID2B	4.67	4.59	6.86	5.71	5.03	5.37	4.59	4.58	3.8	2.9	3.5
8	4	76	FJ	3.57	3.52	4.71	4.30	---	4.03	3.52	3.54	0.4	---	0.8

"--- " Insufficient Material for Testing

Table 11. Project Core Test Data (C7-C11) - Void Contents - All Courses (Continued)

Site	Layer	SR	Mix Type	% In-Place Voids C7	C8	C9	C10	C11	Avg	Min	20th Pct'l	Recompacted Voids % Gyratory Comp	Rotating Base	Static Base
9	1	70/76	ID2W	2.61	2.09	2.99	1.75	1.92	2.27	1.75	1.84	4.9	----	4.9
9	2	70/76	ID2B	2.42	2.98	3.06	1.59	1.59	2.33	1.59	1.73	2.0	2.0	1.9
9	3	70/76	ID2W	6.29	4.88	4.64	4.68	4.96	5.09	4.64	4.52	2.9	1.5	1.9
10	1	70/76	ID2W	1.00	0.71	2.17	0.71	0.92	1.10	0.71	0.59	2.1	---	1.5
10	2	70/76	ID2B	4.10	4.41	5.60	3.98	4.45	4.51	3.98	3.97	2.2	2.1	3.2
11	1	90	ID2W	2.32	1.27	3.80	2.07	1.73	2.24	1.27	1.43	-0.1	0.2	0.3
11	2	90	BCBC	7.58	8.31	7.98	6.48	7.17	7.50	6.48	6.90	3.3	2.7	3.4
11	3	90	BCBC	8.35	8.88	9.57	8.23	9.57	8.92	8.23	8.38			
12	1	81	ID2W	6.81	5.86	5.90	5.70	6.36	6.13	5.70	5.74	2.8	2.2	2.5
12	2	81	BCBC	1.27	0.91	1.19	2.14	1.55	1.41	0.91	1.02	1.5	1.9	1.7
12	3	81	LEVEL	6.03	6.31	6.68	6.85	4.48	6.07	4.48	5.28	3.0	1.6	2.2
13	1	81	ID2W	3.33	2.34	2.71	2.71	2.30	2.68	2.30	2.33	3.2	---	3.2
13	2	81	ID2B	0.76	0.08	0.52	0.32	0.84	0.50	0.08	0.24	1.7	1.0	1.4
13	3	81	ID2B	4.22	1.97	5.56	5.56	4.89	4.44	1.97	3.19	4.0	4.1	4.4
13	4	81	ID2W	---	2.47	---	6.10	8.28	5.62	2.47	3.15	1.5	---	1.9
14	1	81	ID2W	3.05	2.68	2.81	2.68	3.05	2.85	2.68	2.70	1.7	1.2	1.2
14	2	81	ID2B	3.92	5.14	4.70	4.04	3.76	4.31	3.76	3.82	2.0	1.7	1.8
14	3	81	ID2W	4.68	5.17	3.68	6.08	---	4.90	3.68	4.06	---	---	--
14	4	81	ID2W	---	1.81	2.35	4.61	---	2.92	1.81	1.68	---	---	--
15	1	78	ID2W	2.78	2.37	2.82	2.98	1.39	2.47	1.39	1.93	-2.1	-2.2	-1.4
15	2	78	ID2B	5.40	4.28	5.25	4.62	4.16	4.74	4.16	4.27	7.1	6.4	6.9

"--- " Insufficient Material for Testing

Table 11. Project Core Test Data (C7-C11) - Void Contents - All Courses (Continued)

				% In-Place Voids								Recompacted Voids %		
Site	Layer	SR	Mix Type	C7	C8	C9	C10	C11	Avg	Min	20th Pct'l	Gyratory Comp	Rotating Base	Static Base
16	1	78	ID2W	-5.18	3.02	4.44	5.79	3.18	4.32	3.02	3.30	3.7	1.3	3.1
16	2	78	ID2B	5.01	4.19	5.40	5.87	3.91	4.88	3.91	4.19	1.9	2.0	3.2
17	1	81	ID2W	4.59	3.81	3.85	3.15	4.35	3.95	3.15	3.48	2.8	2.6	3.4
17	2	81	ID2B	2.70	3.15	5.81	2.78	6.69	4.23	2.70	2.65	2.7	2.8	3.8
18	1	81	ID2W	2.32	3.46	2.72	2.64	2.60	2.75	2.32	2.39	2.8	1.6	--
18	2	81	ID2B	3.13	0.08	1.55	2.16	1.62	1.71	0.08	0.78	0.8	0.2	2.1
18	3	81	BCBC	3.66	3.01	2.68	---	4.59	3.49	2.68	2.78	2.0	1.6	2.5
19	1	222	ID2W	5.04	6.46	5.97	3.83	4.48	5.16	3.83	4.26	2.8	1.8	2.3
19	2	222	ID2W	4.73	5.53	4.29	3.57	4.49	4.52	3.57	3.92	3.6	2.2	3.0
19	3	222	ID2B	6.55	3.80	4.96	3.80	4.26	4.67	3.80	3.71	3.0	2.0	2.8
20	1	222	ID2W	3.43	2.93	4.48	2.51	3.73	3.42	2.51	2.78	9.2	8.0	--
20	2	222	ID2B	3.25	2.31	3.33	4.15	2.66	3.14	2.31	2.55	3.0	3.0	4.1
20	3	222	LEVEL	6.75	5.66	6.55	---	---	6.32	5.66	5.83	1.3	---	---
22	1	76	ID2W	4.96	3.87	7.09	4.15	4.07	4.83	3.87	3.71	1.5	0.6	1.5
22	2	76	---	1.07	0.69	0.31	0.27	0.84	0.64	0.27	0.35	0.6	0.2	0.6
22	3	76	ID2W	2.11	2.30	2.19	2.54	2.42	2.31	2.11	2.17	0.4	0.4	0.5
23	1	76	ID2W	2.27	4.95	5.15	0.32	3.21	3.18	0.32	1.50	1.5	1.8	1.6
23	2	76	ID2B	2.71	2.63	3.89	3.66	5.11	3.60	2.63	2.75	0.9	2.3	1.5
23	3	76	ID2B	5.75	2.06	---	---	4.49	4.10	2.06	2.52	1.9	2.4	2.2
24	1	76	ID2W	7.39	2.00	1.64	2.36	2.72	3.22	1.64	1.24	1.0	1.2	1.3
24	2	76	ID2B	3.15	2.20	2.50	1.21	1.33	2.08	1.21	1.39	1.8	1.8	1.7

"--- " Insufficient Material for Testing

Table 11. Project Core Test Data (C7-C11) - Void Contents - All Courses (Continued)

Site	Layer	SR	Mix Type	% In-Place Voids C7	C8	C9	C10	C11	Avg	Min	20th Pct'l	Recompacted Voids % Gyratory Comp	Rotating Base	Static Base
25	1	11	ID2W	3.00	2.59	4.02	2.22	5.50	3.47	2.22	2.36	3.5	2.7	3.1
25	2	11	ID2B	2.96	2.84	2.84	2.52	4.08	3.05	2.52	2.54	3.4	3.4	3.7
26	1	83	ID2W	7.14	5.24	5.97	3.53	5.32	5.44	3.53	4.34	3.9	1.8	1.5
26	2	83	ID2B	6.48	4.50	5.70	4.31	5.51	5.30	4.31	4.55	1.9	3.4	2.8
27	1	83	ID2W	3.96	3.06	2.61	3.51	2.57	3.14	2.57	2.64	1.1	----	1.7
27	2	83	ID2B	3.78	2.81	2.38	2.61	1.36	2.59	1.36	1.86	0.5	0.6	1.5
28	1	22	ID3W	4.66	5.89	4.67	4.42	11.79	6.29	4.42	3.66	2.1	1.2	1.6
28	2	22	LEVEL	8.05	6.60	7.72	8.75	6.52	7.53	6.52	6.72	3.2	---	2.2
28	3	22	ID2W	2.59	3.29	4.15	1.52	---	2.89	1.52	1.95	1.8	---	1.3
28	4	22	ID2B	7.10	6.00	3.53	7.77	---	6.10	3.53	4.54	-2.1	-1.1	-0.6
29	1	11	ID3W	5.51	3.33	5.63	4.92	2.94	4.47	2.94	3.41	3.1	2.7	3.0
29	2	11	ID2B	5.92	3.12	3.61	3.94	3.53	4.02	3.12	3.10	-0.3	0.6	1.9
30	1	11	ID2W	1.73	1.81	2.10	1.98	2.22	1.97	1.73	1.80	0.9	0.7	0.8
30	2	11	ID2B	2.39	3.59	2.91	2.47	2.15	2.70	2.15	2.23	1.2	1.4	1.9
31	1	220	ID2W	8.60	8.43	6.42	8.35	6.71	7.70	6.42	6.82	3.1	2.7	4.2
31	2	220	ID2B	8.99	7.51	4.80	6.75	6.47	6.90	4.80	5.62	1.4	1.5	2.2
32	1	70	ID2W	5.52	3.06	3.77	3.69	3.85	3.98	3.06	3.21	2.2	1.7	2.1
32	2	70	ID2B	3.12	2.52	2.80	3.12	3.04	2.92	2.52	2.70	2.0	2.6	2.1
33	1	70	ID2W	7.05	5.56	6.12	7.29	6.61	6.53	5.56	5.94	1.8	1.0	1.3
33	2	70	ID2B	5.96	4.44	3.28	3.36	4.24	4.26	3.28	3.35	2.1	1.8	2.2

"--- " Insufficient Material for Testing

Table 11. Project Core Test Data (C7-C11) - Void Contents - All Courses (Continued)

				% In-Place Voids								Recompacted Voids %		
Site	**Layer**	**SR**	**Mix Type**	**C7**	**C8**	**C9**	**C10**	**C11**	**Avg**	**Min**	**20th Pct'l**	**Gyratory Comp**	**Rotating Base**	**Static Base**
34	1	70	ID2W	4.81	2.89	3.42	3.79	3.42	3.67	2.89	3.07	1.7	1.6	1.7
34	2	70	ID2W	8.31	6.19	6.96	8.51	6.56	7.31	6.19	6.43	1.6	1.1	1.9
34	3	70	LEVEL	3.23	4.09	5.24	5.18	5.20	4.59	3.23	3.83	0.8	---	0.6
34	4	70	ID2B	7.50	4.48	3.89	4.40	4.68	4.99	3.89	3.79	3.5	2.5	3.8
35	1	80	ID2W	6.29	5.77	5.73	6.70	5.97	6.09	5.73	5.75	2.6	1.9	2.5
35	2	80	ID2B	6.19	6.23	6.80	6.23	6.88	6.47	6.19	6.18	3.1	2.9	3.6
35	3	80	BCBC	5.83	6.30	5.99	6.14	6.10	6.07	5.83	5.92	3.2	2.8	3.9
Layer 1														
	Average:			4.15	3.49	4.02	3.42	3.67	3.75	2.86	3.01	2.44	1.74	2.04
	Std Deviation:			2.018	1.745	1.531	1.851	2.152	1.624	1.559	1.564	1.761	1.511	1.160
Layer 2														
	Average:			4.17	3.74	4.01	3.69	3.52	3.83	2.77	3.10	2.00	1.96	2.46
	Std Deviation:			4.173	3.741	4.009	3.694	3.518	3.827	2.769	3.095	1.997	1.963	2.459

"--- " Insufficient Material for Testing

Table 12. Project Core Test Data (C7-C11) - VMA/VF/Creep - All Courses

				VMA			VFA				Creep (0.0001 inch)			
Site	**Layer**	**SR**	**Mix Type**	**GTM**	**FOT**	**Pine**	**GTM**	**ROT**	**Pine**	**GSI**	**60 Min**	**45 Min**	**30 Min**	**15 Min**
1	1	80	ID2W	14.7	13.7	14.4	81.1	88.1	82.9	1.26	6.98	6.89	6.84	6.65
1	2	80	ID2B	12.1	12.6	13.0	78.3	74.8	71.7	1.13	4.56	4.52	4.47	4.36
1	3	80	BCBC	11.5	12.4	12.4	65.1	60.0	59.9	1.07	17.25	16.75	15.96	14.66
2	1	279	ID2W	16.2	16.0	16.8	84.4	85.7	81.0	1.10	9.38	9.33	9.12	8.82
2	2	279	ID2B	9.8	10.2	10.5	86.7	83.5	80.3	1.23	13.52	13.29	13.05	12.56
3	1	279	ID2W	13.5	14.0	14.1	94.3	90.1	89.3	1.50	9.00	8.92	8.72	8.42
3	2	279	ID2B	10.4	10.3	10.8	91	92.8	87.4	1.10	11.36	11.00	10.50	9.67
4	1	279	ID2W	15.2	13.5	13.7	72.4	83.8	82.0	1	10.39	10.29	10.13	9.94
4	2	279	ID2B	10.1	9.9	10.3	86.9	88.9	85.7	1.24	12.15	12.00	11.75	11.20
5	1	30	ID2W	14.7	15.1	15.1	91.7	89.1	89.1	1.68	10.41	10.31	10.22	10.05
5	2	30	ID2B	10.8	10.9	11.2	94.1	93.6	91.0	1.32	15.3	14.90	14.15	13.20
6	1	30	ID2W	14.5	14.3	14.7	84.0	85.6	82.9	1.60	12.05	11.9	11.70	11.40
6	2	30	ID2B	11.0	11.9	12.1	88.8	80.7	79.7	1.30	5.95	5.89	5.80	5.64
6	3	30	LEVEL	16.8	---	16.7	103.5	---	103.8	1.66	14.95	14.75	14.35	13.75
7	1	70	1 D2W	13.8	---	13.5	92.0	---	89.5	1.55	14.40	14.30	14.00	13.50
7	2	70	ID2B	15.0	15.3	15.4	93.3	91.1	50.3	1.28	12.20	12.10	12.00	11.70
7	3	70	LEVEL	17.2	---	17.5	96.9	---	94.6	1.48	13.80	13.60	13.40	12.90
8	1	76	ID2W	14.5	12.2	12.6	74.7	91.5	88.3	1.00	15.70	15.70	15.50	15.20
8	2	76	LEVEL	19.6	---	19.5	80.8	---	81.2	1.36	62.75	62.60	62.30	61.70
8	3	76	ID2B	14.6	13.8	14.3	73.8	78.7	75.5	1.10	20.40	20.40	20.10	19.80
8	4	76	FJ	13.6	---	16.0	98.8	---	95.2	1.45	16.10	16.10	16.00	15.70
9	1	70/76	ID2W	22.1	---	22.1	77.9	---	77.7	1.05	36.70	36.60	36.50	36.20
9	2	70/76	ID2B	13.5	13.5	13.4	85.2	85.1	86.4	1.05	15.00	14.80	14.50	14.00
9	3	70/76	ID2W	16.3	15.1	15.4	82.2	90.1	87.9	1.23	16.70	16.70	16.60	16.50

"--- " Insufficient Material for Testing

Table 12. Project Core Test Data (C7-C11) - VMA/VF/Creep - All Courses (Continued)

				VMA			VFA				Creep (0.0001 inch)			
Site	**Layer**	**SR**	**Mix Type**	**GTM**	**FOT**	**Pine**	**GTM**	**ROT**	**Pine**	**GSI**	**60 Min**	**45 Min**	**30 Min**	**15 Min**
10	1	70/76	ID2W	14.7	---	14.1	85.8	---	89.7	1.24	31.45	31.40	31.30	30.95
10	2	70/76	ID2B	12.5	12.5	13.4	82.7	83.0	76.5	1.19	7.66	7.46	7.22	6.80
11	1	90	ID2W	14.9	15.2	15.3	100.8	98.6	97.8	1.57	12.50	12.30	12.10	11.60
11	2	90	BCBC	13.6	13.0	13.7	75.9	79.6	75.3	1.05	15.30	15.10	14.80	14.30
11	3	90	BCBC	---	---	---	---	---	---	---	14.90	14.80	14.60	14.20
12	1	81	ID2W	15.9	15.3	15.6	82.2	85.6	83.8	1.20	9.56	9.49	9.41	9.17
12	2	81	BCBC	10.8	11.2	10.9	86.4	82.9	84.8	1.25	3.65	3.60	3.54	3.41
12	3	81	LEVEL	12.0	10.7	11.3	74.6	84.8	80.2	1.23	13.10	13.00	12.90	12.60
13	1	81	ID2W	15.6	---	15.6	79.4	---	79.4	1.21	8.86	8.81	8.60	8.33
13	2	81	ID2B	11.2	10.5	10.9	84.9	90.9	86.8	1.33	3.90	3.87	3.81	3.69
13	3	81	ID2B	13.3	13.3	13.6	69.5	69.1	67.4	1.00	3.86	3.83	3.79	3.68
13	4	81	ID2W	15.5	---	16.9	90.5	---	88.8	1.48	---	---	---	---
14	1	81	ID2W	15.3	14.9	14.9	89.2	91.8	91.7	1.39	9.49	9.38	9.25	8.99
14	2	81	ID2B	12.4	12.1	12.2	83.8	86.2	85.4	1.35	4.83	4.70	4.51	4.19
14	3	81	ID2W	---	---	---	---	---	---	---	9.18	9.09	8.99	8.75
14	4	81	ID2W	---	---	---			---	---	22.8	22.7	22.5	22.4
14	5	81	ID2B	---	---	---	---	---	---	1.00	16.4	16.15	15.7	14.8
15	1	78	ID2W	12.1	12.0	12.7	117.4	118.5	111.6	1.21	11.31	11.24	11.18	10.99
15	2	78	ID2B	15.7	15.1	15.5	54.9	57.8	55.8	1.22	11.06	11.00	10.78	10.41
16	1	78	ID2W	16.2	14.1	15.7	78.3	91.1	80.3	1.38	10.94	10.83	10.62	10.27
16	2	78	ID2B	11.5	11.6	12.7	83.2	82.8	74.7	1.31	9.65	9.48	9.30	8.98
17	1	81	ID2W	15.9	15.7	16.5	82.7	83.8	79.2	1.11	10.30	10.20	10.00	9.72
17	2	81	ID2B	11.0	11.1	12.0	75.7	74.6	68.2	1.17	4.09	4.01	4.01	3.97
17	3	81	LEVEL	---	---	---	---	---	---	---	6.44	6.4	6.37	6.19

“--- ” Insufficient Material for Testing

Table 12. Project Core Test Data (C7-C11) - VMA/VF/Creep - All Courses (Continued)

				VMA			VFA				Creep (0.0001 inch)			
Site	**Layer**	**SR**	**Mix Type**	**GTM**	**FOT**	**Pine**	**GTM**	**ROT**	**Pine**	**GSI**	**60 Min**	**45 Min**	**30 Min**	**15 Min**
18	1	81	ID2W	14.5	13.8	14.4	89.2	94.3	89.7	1.67	9.42	9.32	9.22	8.99
18	2	81	ID2B	12.1	11.7	13.3	93.7	98.1	84.1	1.30	7.93	7.85	7.70	7.42
18	3	81	BCBC	12.6	12.2	13.0	83.9	86.8	80.6	1.22	26.35	26.00	25.40	24.30
19	1	222	ID2W	16.6	15.8	16.1	83.3	88.3	86.1	1.23	10.74	10.67	10.57	10.33
19	2	222	ID2W	17.8	16.6	17.3	79.8	86.5	02.7	1.12	7.14	7.06	7.01	6.85
19	3	222	ID2B	13.3	12.4	13.1	77.3	83.6	78.7	1.11	13.40	13.30	13.10	12.60
20	1	222	ID2W	15.7	14.6	---	82.0	89.4	---	1.06	9.99	9.89	9.82	9.65
20	2	222	ID2B	12.9	12.9	14.0	76.8	76.8	70.3	1.05	4.26	4.25	4.24	4.20
20	3	222	LEVEL	16.1	---	---	91.7	---	---	---	19.45	19.40	19.20	18.85
22	1	76	ID2W	14.8	14.1	14.8	89.7	95.4	90.0	1.46	16.95	16.65	16.25	15.40
22	2	76	---	11.7	11.4	11.8	95.3	97.8	94.5	1.37	7.07	6.99	6.81	6.53
22	3	76	ID2W	13.7	13.8	13.9	97.4	96.9	96.4	1.88	9.20	9.11	8.86	8.51
23	1	76	ID2W	14.6	14.8	14.7	89.9	88.2	89.2	1.64	9.17	9.06	8.85	8.43
23	2	76	ID2B	11.9	13.1	12.4	92.5	82.6	89.2	1.07	5.44	5.36	5.27	5.09
23	3	76	ID2B	12.9	13.3	13.2	85.0	82.2	83.0	1.92	14.08	13.58	12.94	11.87
24	1	76	ID2W	14.5	14.6	14.7	93.2	91.9	91.4	1.78	13.47	13.33	13.30	12.99
24	2	76	ID2B	11.5	11.4	11.4	84.2	84.6	85.2	1.56	---	---	---	---
25	1	11	ID2W	17.3	16.6	17.0	80.0	83.9	81.6	1.12	15.30	15.20	14.90	14.55
25	2	11	SP B	12.1	12.1	12.4	71.7	71.7	70.0	1.33	4.97	4.85	4.64	4.33
26	1	83	ID2W	17.0	15.1	14.8	76.9	90.0	90.2	1.03	7.06	7.05	6.96	6.78
26	2	83	ID2B	12.4	13.7	13.3	85.0	76.1	78.5	1.10	7.13	7.07	7.00	6.83
27	1	83	ID2W	14.4	---	14.8	92.0	---	84.7	1.65	10.25	10.10	9.89	9.51
27	2	83	ID2B	11.0	11.2	12.0	95.9	94.3	87.1	1.58	12.70	12.50	12.20	11.65

"--- " Insufficient Material for Testing

Table 12. Project Core Test Data (C7-C11) - VMA/VF/Creep - All Courses (Continued)

				VMA			VFA				Creep (0.0001 inch)			
Site	**Layer**	**SR**	**Mix Type**	**GTM**	**FOT**	**Pine**	**GTM**	**ROT**	**Pine**	**GSI**	**60 Min**	**45 Min**	**30 Min**	**15 Min**
28	1	22	ID3W	14.2	13.4	13.8	85.3	90.9	88.4	1.25	5.31	5.27	5.22	5.10
28	2	22	LEVEL	15.1	---	14.2	78.5	---	84.4	1.04	10.10	10.00	9.96	9.83
28	3	22	ID2W	14.8	---	14.4	88.2	---	91.0	1.28	11.60	11.50	11.45	11.20
28	4	22	ID2B	7.3	8.2	8.7	---	---	---	1.27	9.75	9.52	9.15	8.52
29	1	11	ID3W	14.2	13.8	14.1	78.4	80.6	78.6	1.14	6.42	6.35	6.23	6.02
29	2	11	ID2B	15.7	16.5	17.6	102.2	96.2	89.4	1.36	19.95	19.75	19.30	18.45
30	1	11	ID2W	14.1	13.9	14.1	93.8	95.3	94.0	1.56	11.97	11.85	11.64	11.29
30	2	11	ID2B	14.2	14.3	14.8	91.4	90.4	87.1	1.43	12.25	12.10	11.90	11.49
31	1	220	ID2W	16.4	16.1	17.3	80.9	82.9	76.0	1.09	11.90	11.85	11.70	11.53
31	2	220	ID2B	13.4	13.6	14.2	89.7	88.7	84.5	1.34	9.75	9.68	9.54	9.34
32	1	70	ID2W	14.4	14.0	14.3	84.8	87.8	85.2	1.73	7.24	7.20	7.08	6.92
32	2	70	ID2B	11.3	11.8	11.4	82.3	77.9	81.4	1.48	10.05	9.98	9.78	9.46
33	1	70	ID2W	14.0	13.3	13.5	86.8	92.1	90.6	1.58	9.63	9.58	9.51	9.36
33	2	70	ID2B	11.4	11.1	11.5	81.7	83.5	80.7	1.48	15.50	15.45	15.20	14.80
34	1	70	ID2W	14.9	14.8	14.9	88.7	88.9	88.5	1.57	13.35	13.30	13.15	12.85
34	2	70	ID2W	14.6	14.3	14.9	89.2	92.0	07.5	1.23	19.10	19.10	19.00	18.85
34	3	70	LEVEL	16.8	---	16.6	95.3	---	96.7	1.69	49.70	49.50	49.25	48.60
34	4	70	ID2B	15.7	14.8	16.0	78.5	82.9	76.0	1.08	16.25	16.05	15.70	15.10
35	1	80	ID2W	15.0	14.4	15.0	82.8	86.7	83.1	1.33	6.87	6.79	6.76	6.63
35	2	80	ID2B	12.6	12.5	13.1	75.5	76.7	72.5	1.07	5.48	5.40	5.35	5.29
35	3	80	BCBC	12.0	11.6	12.6	73.6	76.1	69.5	1.10	6.31	6.18	5.99	5.71
Layer 1														
	Average:			15.19	14.45	15.02	86.06	90.00	86.89	1.35	11.90	11.80	11.65	11.37
	Std Deviation:			1.589	1.059	1.663	8.266	6.735	6.704	0.240	6.179	6.172	6.162	6.118
Layer 2														
	Average:			12.73	12.50	13.15	84.65	84.44	81.48	1.26	11.27	11.14	10.95	10.61
	Std Deviation:			2.160	1.715	2.089	8.592	8.517	7.902	0.148	10.120	10.099	10.043	9.941

"--- " Insufficient Material for Testing

in the wheel path can increase due to the shearing forces. Previous work at NCAT (*10*) showed that rutting was related to low air voids. However, the low void content did not always occur exactly in the wheel paths. As a result the 20th percentile air void content (80 percent higher and 20 percent lower) from voids obtained across the pavement lane were utilized in correlations with rutting. The results indicated that the use of the 20th percentile air void content was reasonable when compared to the use of the average or minimum air void content.

The average lowest 20th percentile VTM values are 3.01 and 3.10 percent, respectively, for wearing and binder courses, and are very close to the average values obtained from cores Cl through C5. As discussed earlier in case of the test data from cores Cl -C5, these values of WM are considered low and will increase the potential for rutting.

2. The average percentages of VTM obtained in recompacted specimens are as follows:

Compactor	Wearing Course	Binder Course
Gyratory	2.44	2.00
Marshall Rotating Base	1.74	1.96
Marshall Static Base	2.04	2.46

It is significant to note that the Marshall compactor with rotating base and slanted foot gave the highest density (least VTM) for both wearing and binder courses and thus can be used to obtain near maximum potential compaction of mixes which is likely to be achieved under two-three years' traffic. Surprisingly, the gyratory compactor gave the least density (lower than the conventional Marshall method using static base) for the wearing course. However, the gyratory compactor had a significant edge over the conventional static base mechanical Marshall compactor in case of binder course mixes containing larger aggregates (1 - 1 1/2 inches maximum size). This indicates that the gyratory compaction is more effective in densifying the mix when the maximum aggregate size is increased. Based on the preceding data it appears that the mechanical Marshall compactor with rotating base and slanted foot should be used for both wearing and binder course mixes to minimize the potential of over-asphalting mixes designed for heavy duty pavements and high pressure truck tires.

3. Average VMA values obtained for wearing and binder courses using the three compaction procedures are also considered on the low side.
4. Average GSI (gyratory shear index) values were 1.35 and 1.26 for wearing and binder courses, respectively. Whereas a value of 1.00 is considered ideal to prevent rutting, values up to 1.20 may be acceptable. Therefore, both average values are on the high side and indicate a high potential for rutting.
5. Maximum 60-minute permanent deformation values (creep test at 104°F) for wearing and binder courses were observed to be close: 11.90 and 11.27 x 10^{-4} inch/inch, respectively. No reliable deformation threshold values are available in the literature.

Rut Measurement Data

As discussed in detail earlier, surface profiles were obtained adjacent to cores C7-C11 (worst location) and at another site within 500 feet during the summer of 1990. The profile of underlying layers were drawn by using the core layer thicknesses. Complete profiles of the 34 projects are shown in Figures 14 through 47. The rut depths do not appear to be too pronounced on the rutted pavements because the profiles include cross slopes or superelevations. However, a close examination indicates in which layer(s) rutting has occurred.

Table 13 gives the maximum surface rut depth at the worst location (termed "new" surface rut depth in the table because it was obtained in 1990) and the corresponding maximum rut depths in

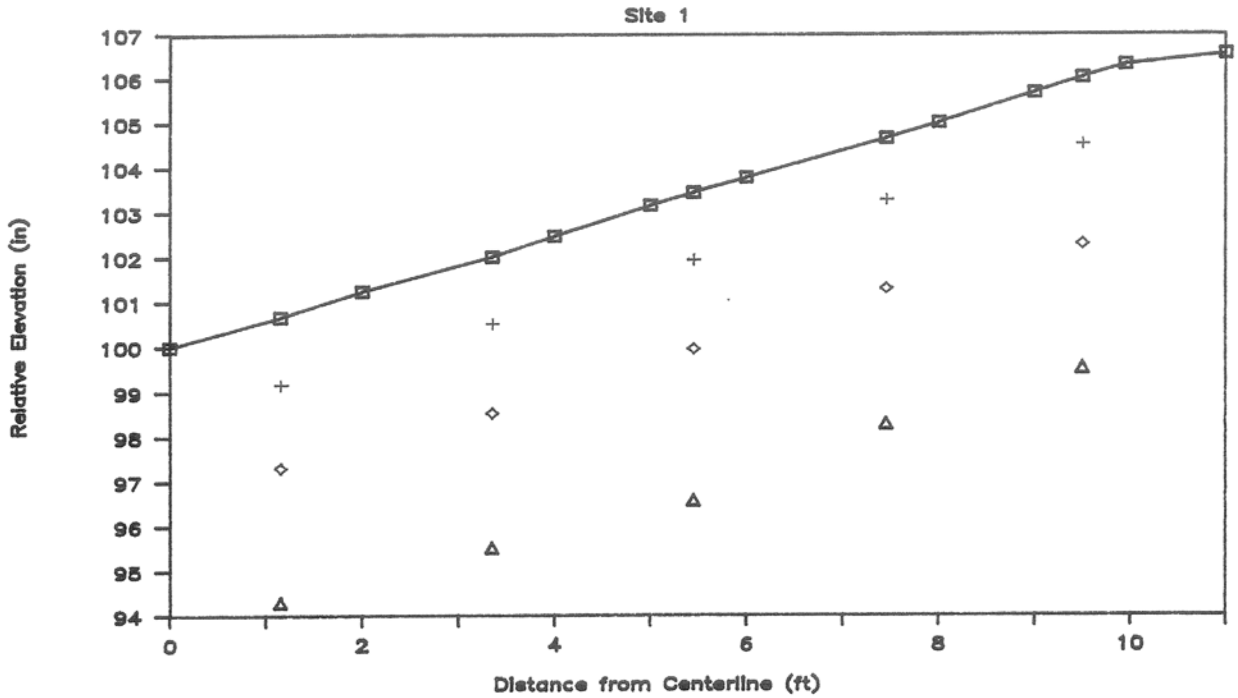

Figure 14. Transverse Profile of Project 1

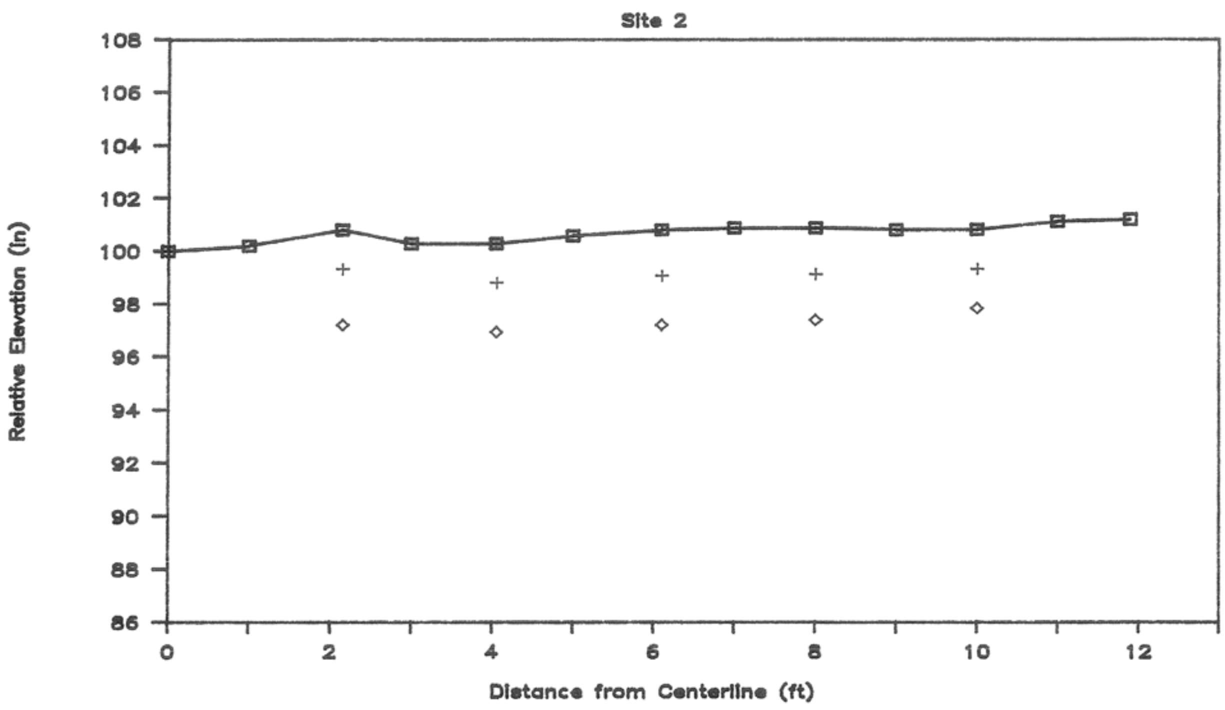

Figure 15. Transverse Profile of Project 2

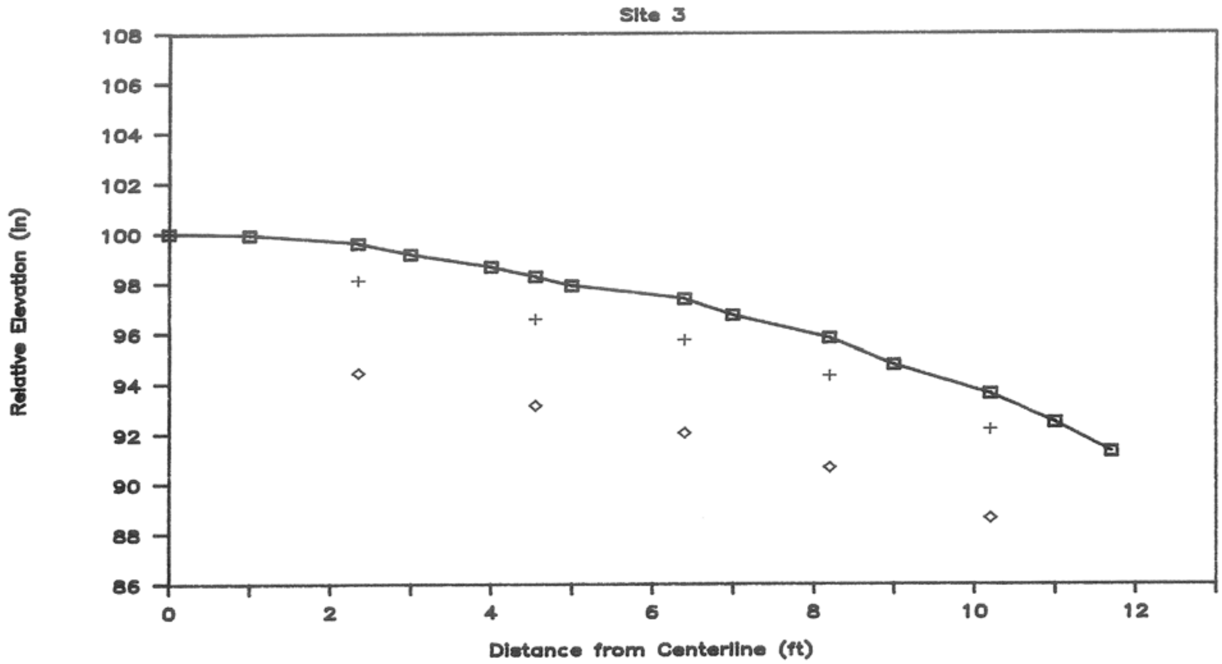

Figure 16. Transverse Profile of Project 3

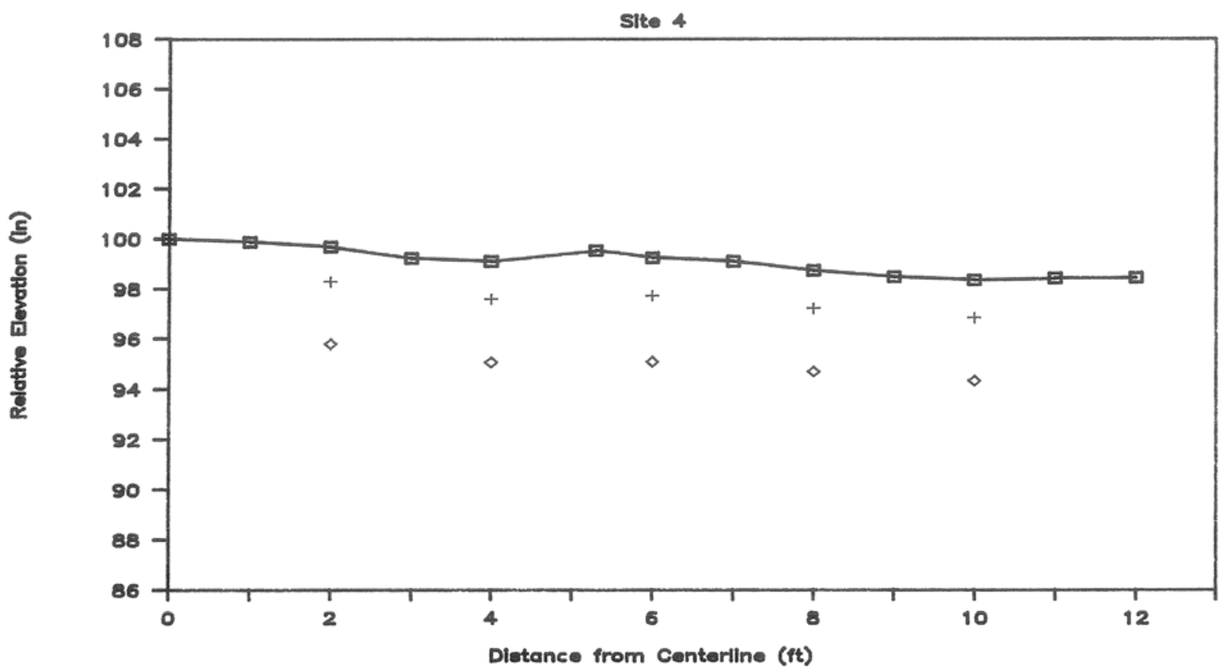

Figure 17. Transverse Profile of Project 4

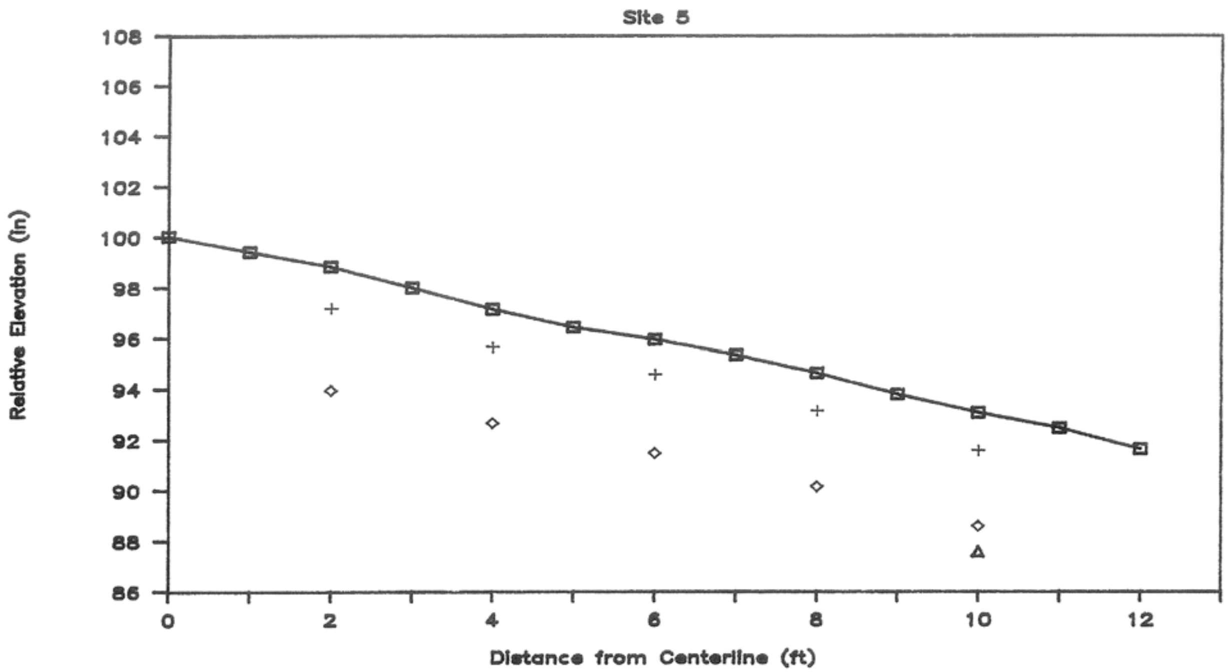

Figure 18. Transverse Profile of Project 5

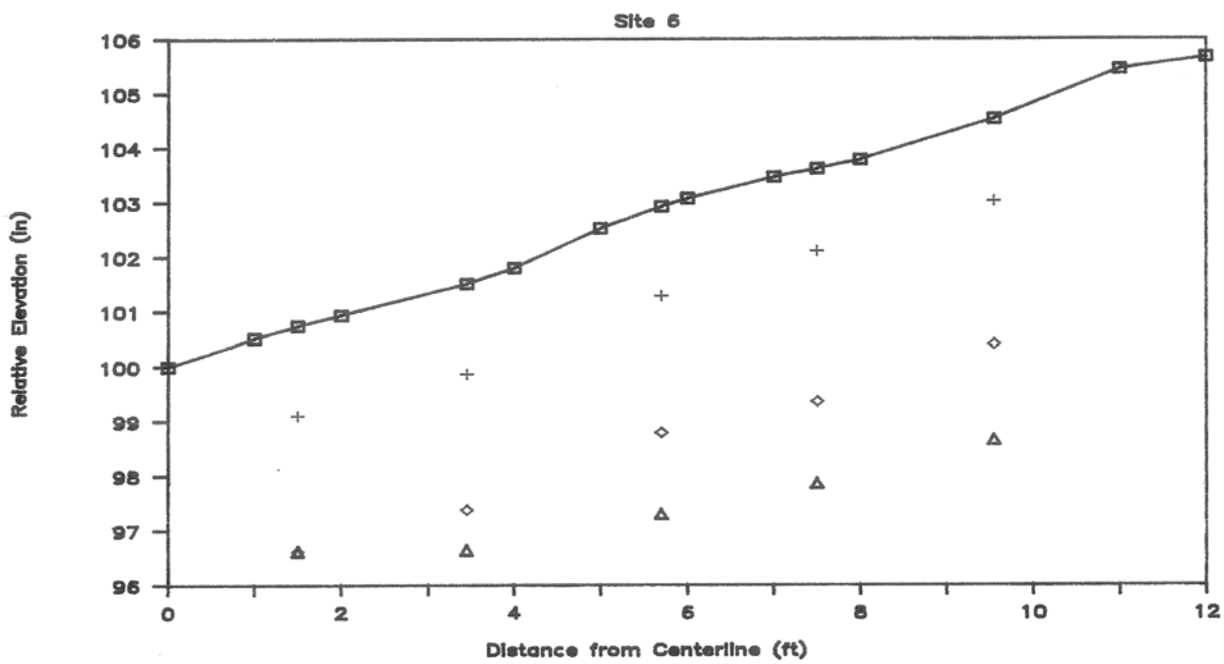

Figure 19. Transverse Profile of Project 6

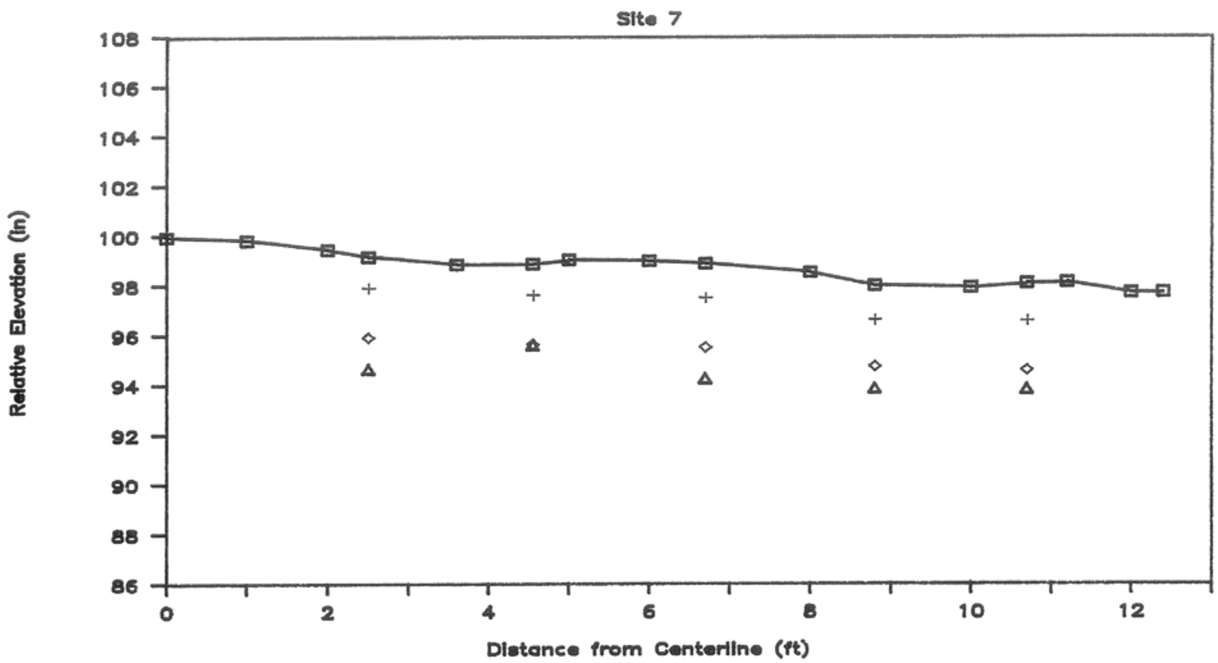

Figure 20. Transverse Profile of Project 7

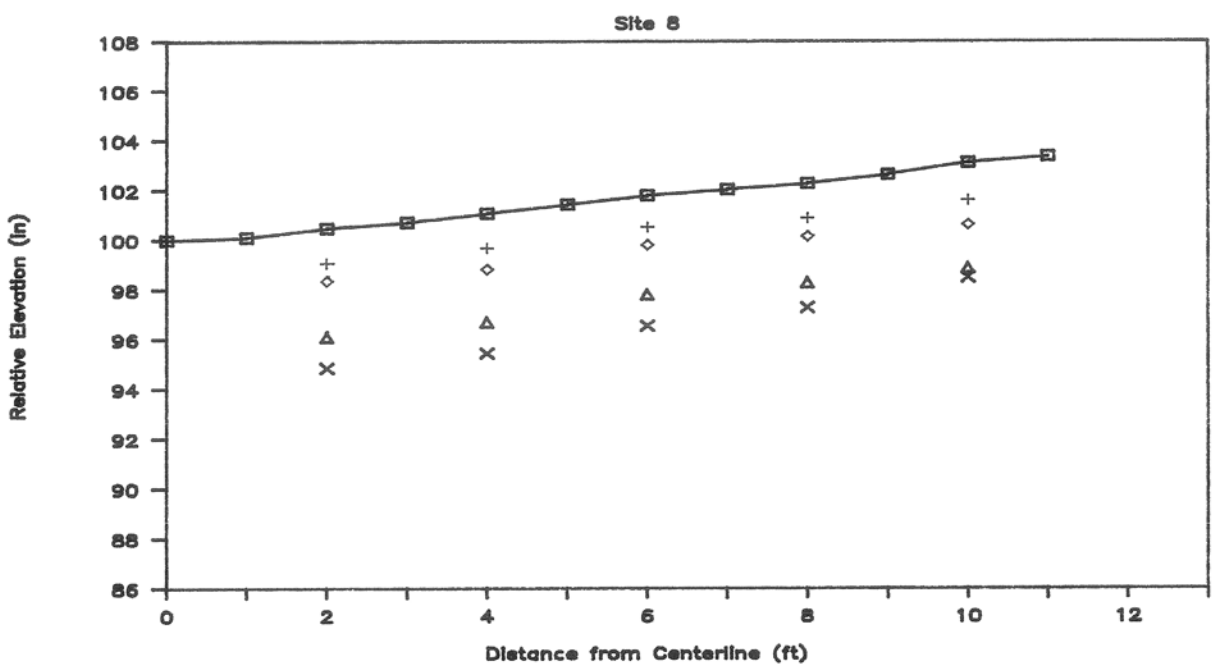

Figure 21. Transverse Profile of Project 8

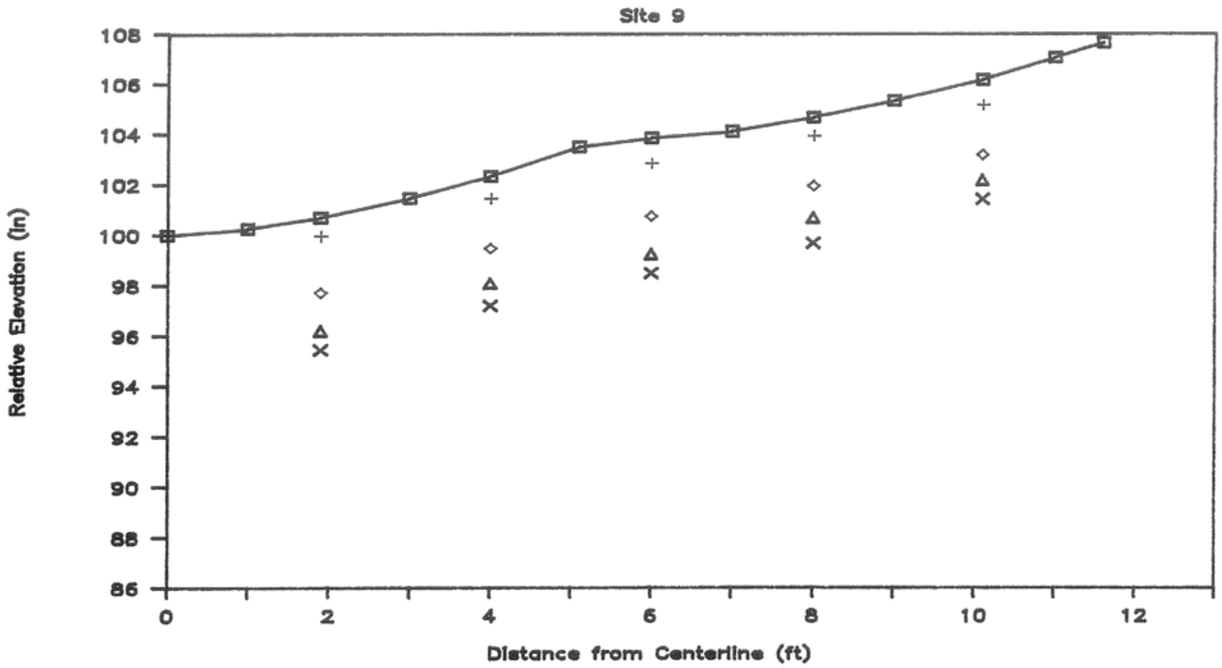

Figure 22. Transverse Profile of Project 9

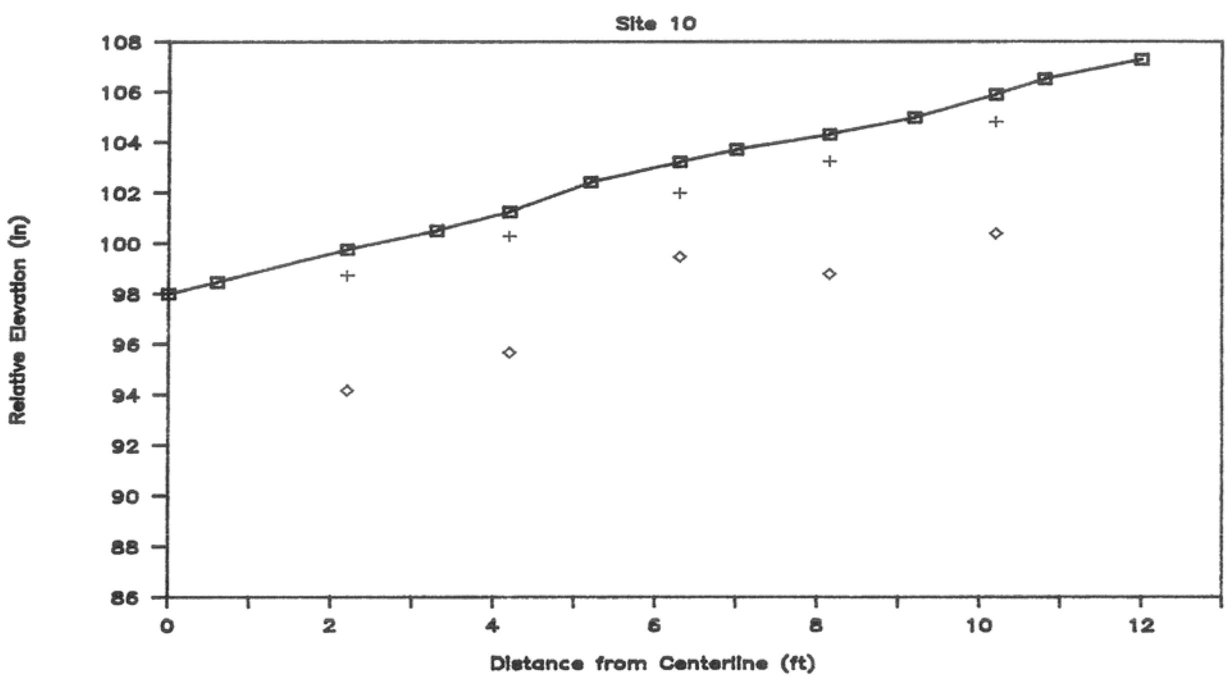

Figure 23. Transverse Profile of Project 10

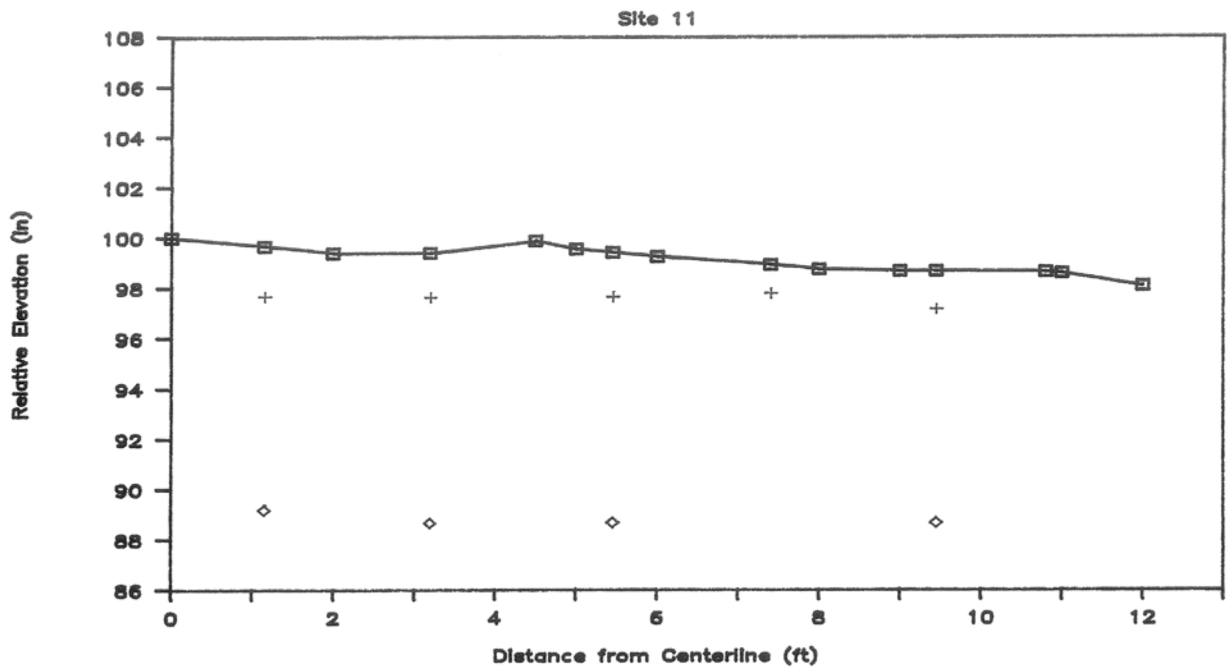

Figure 24. Transverse Profile of Project 11

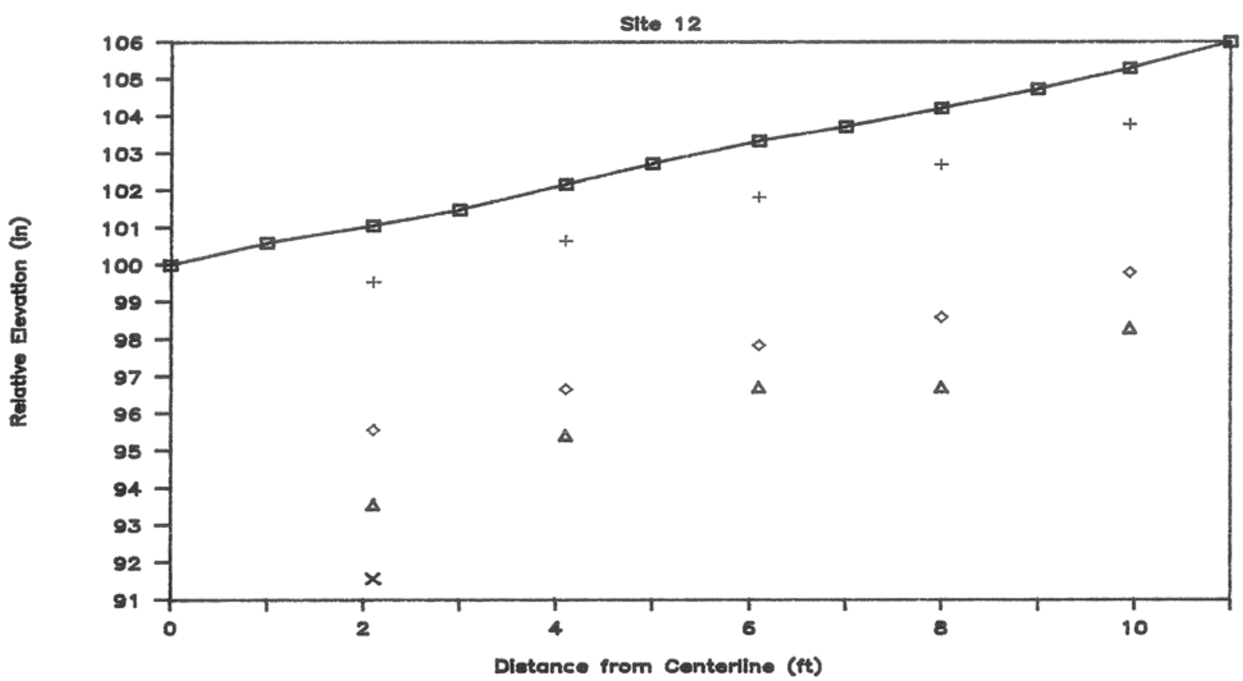

Figure 25. Transverse Profile of Project 12

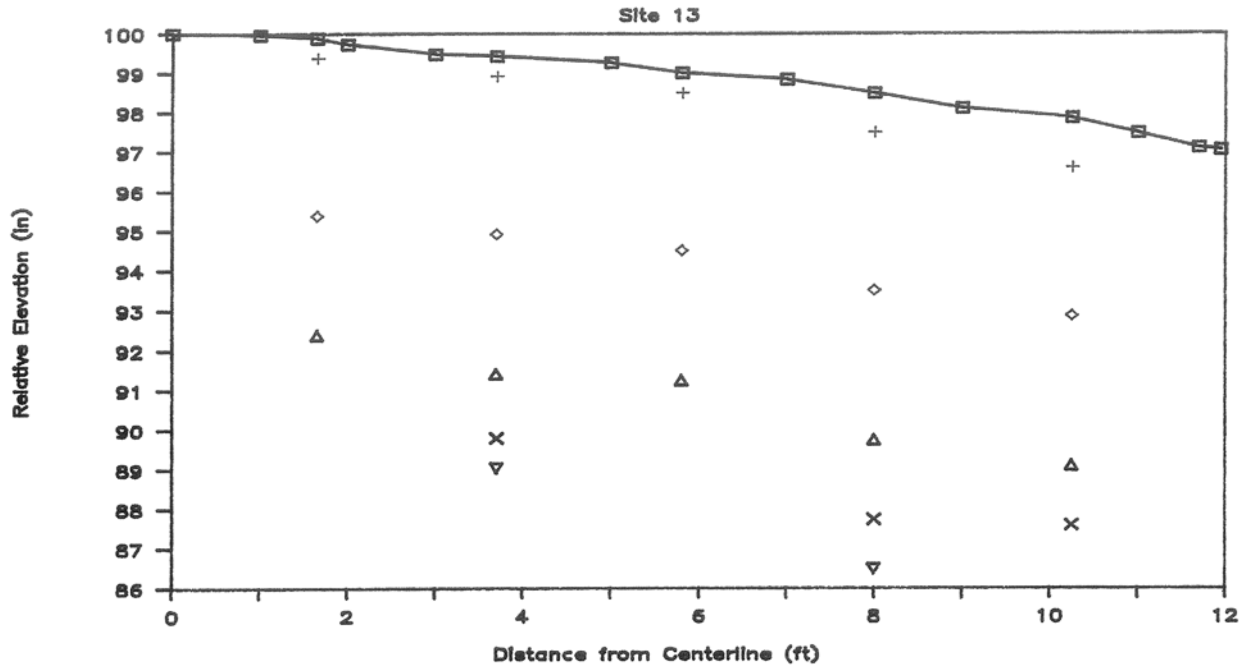

Figure 26. Transverse Profile of Project 13

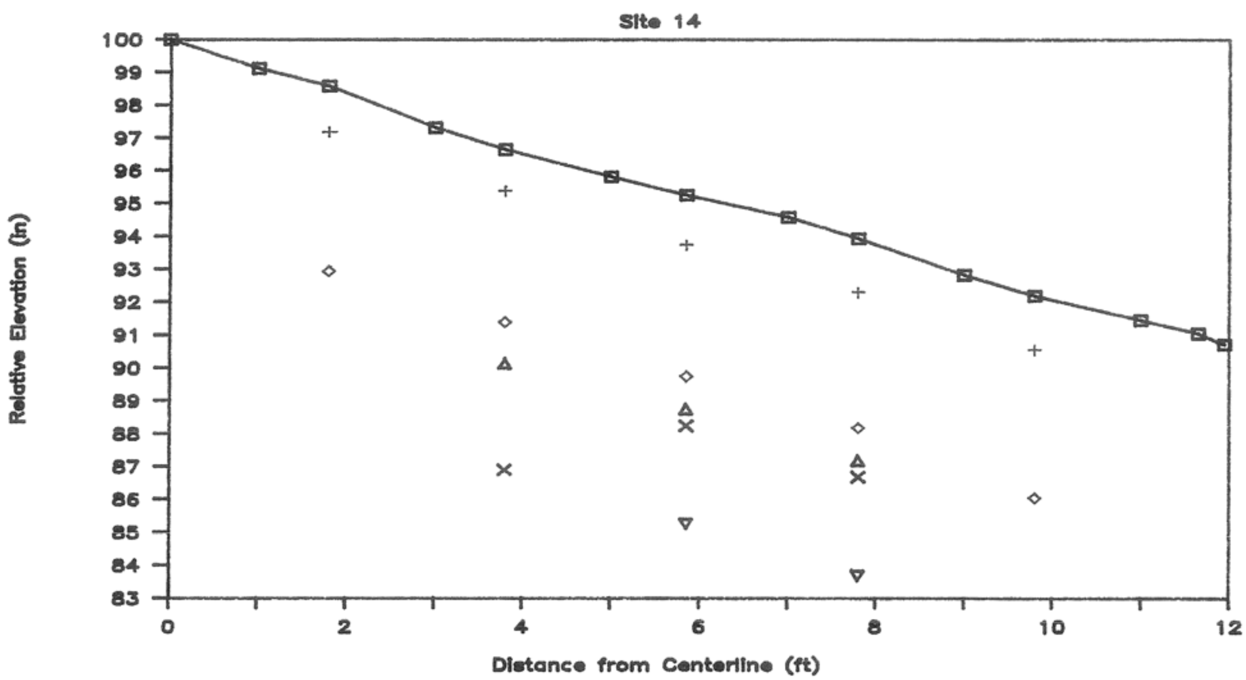

Figure 27. Transverse Profile of Project 14

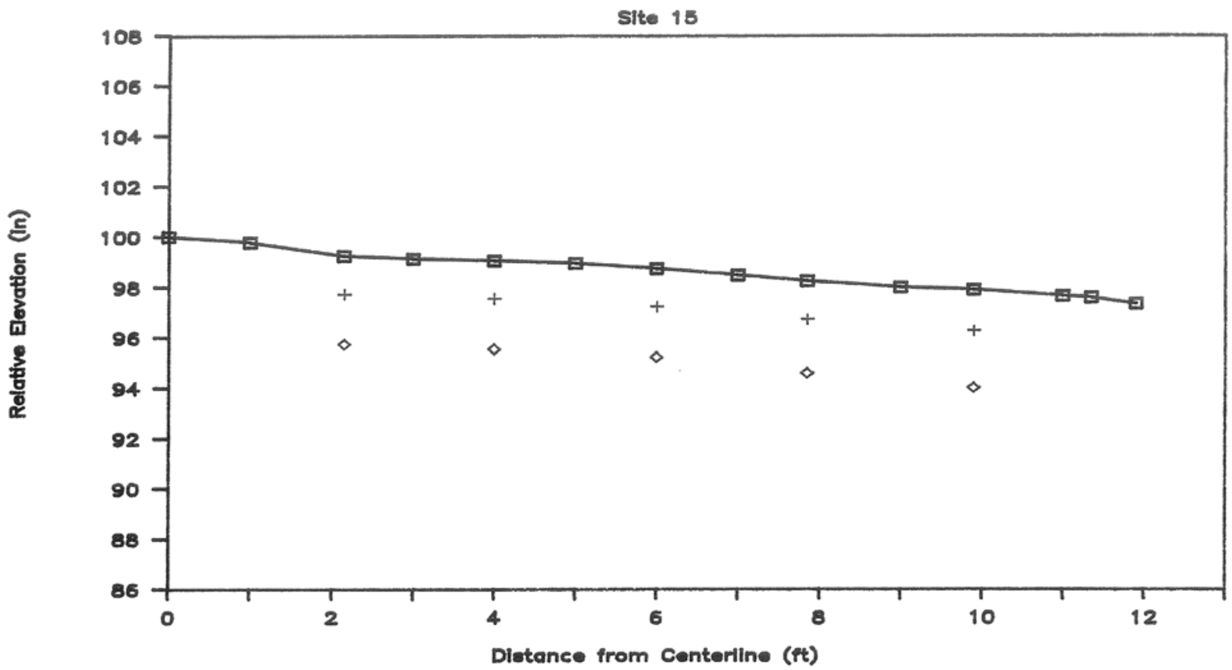

Figure 28. Transverse Profile of Project 15

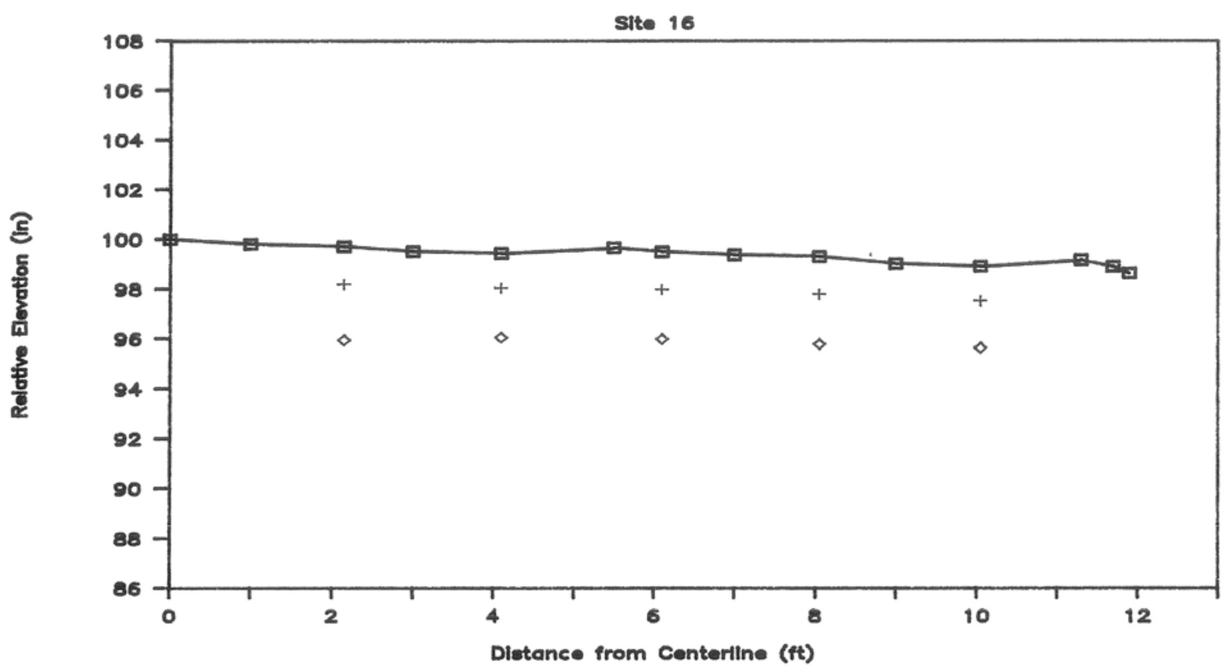

Figure 29. Transverse Profile of Project 16

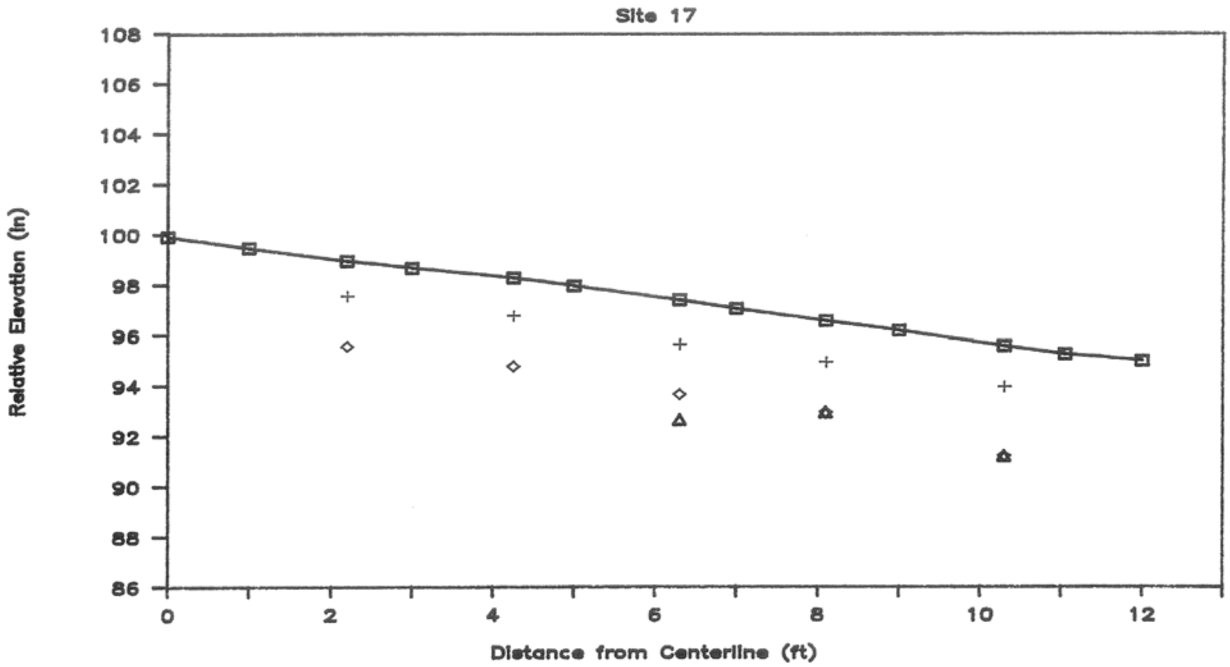

Figure 30. Transverse Profile of Project 17

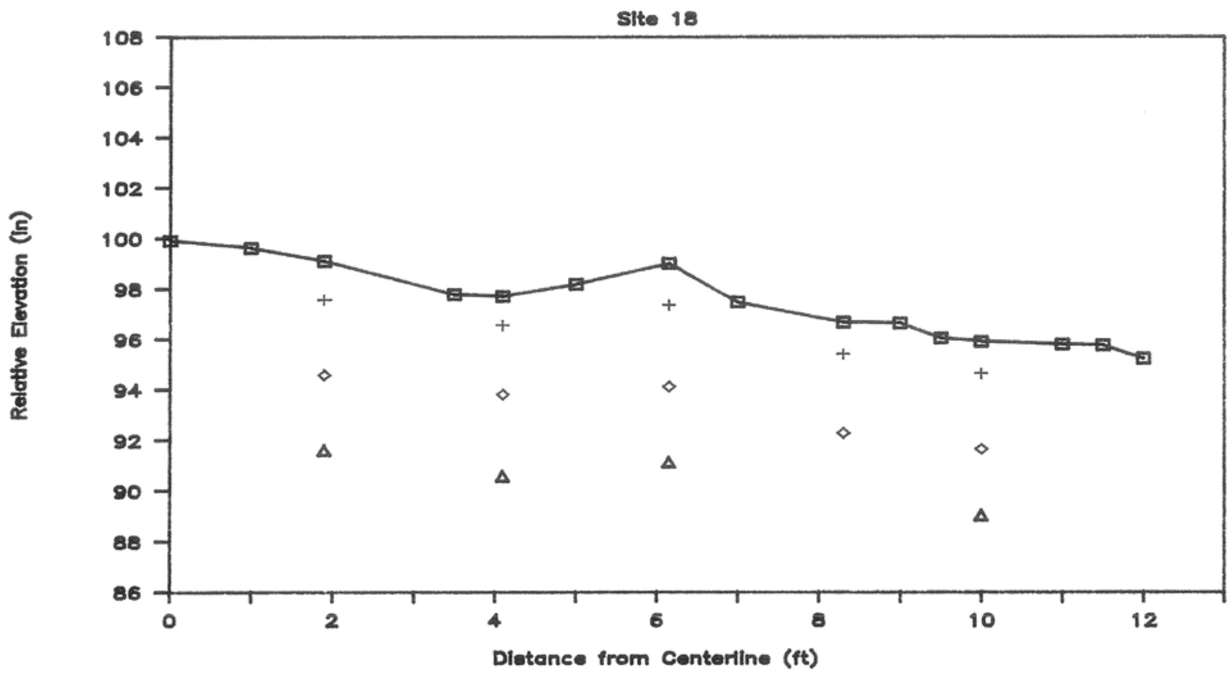

Figure 31. Transverse Profile of Project 18

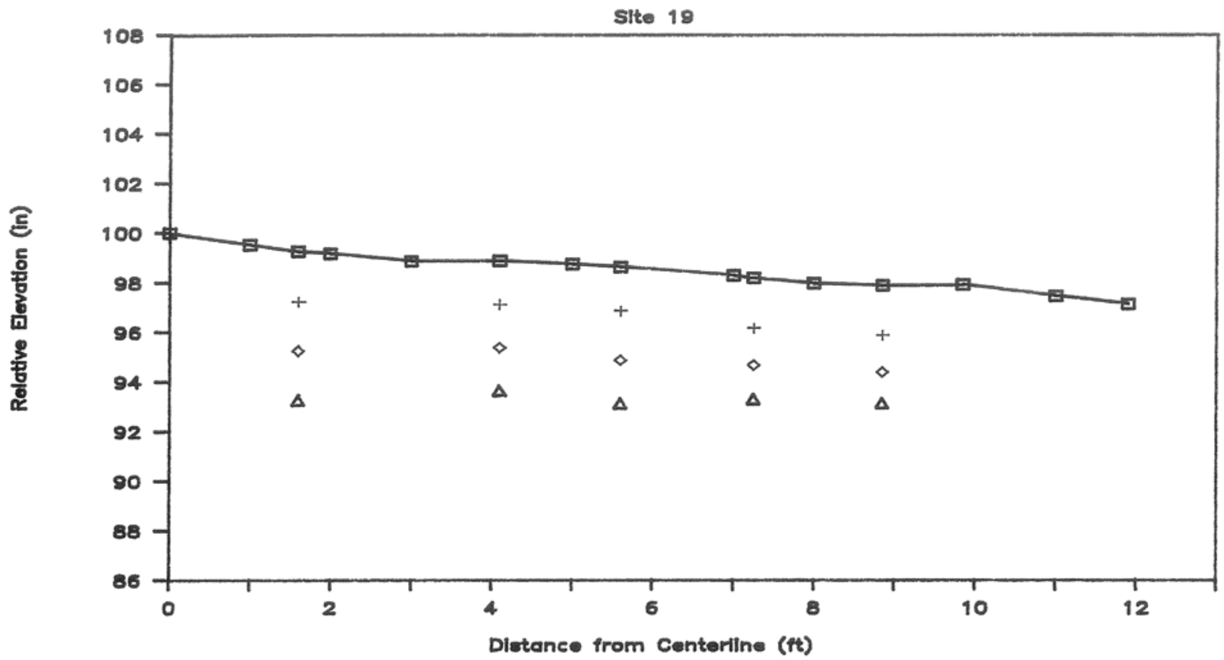

Figure 32. Transverse Profile of Project 19

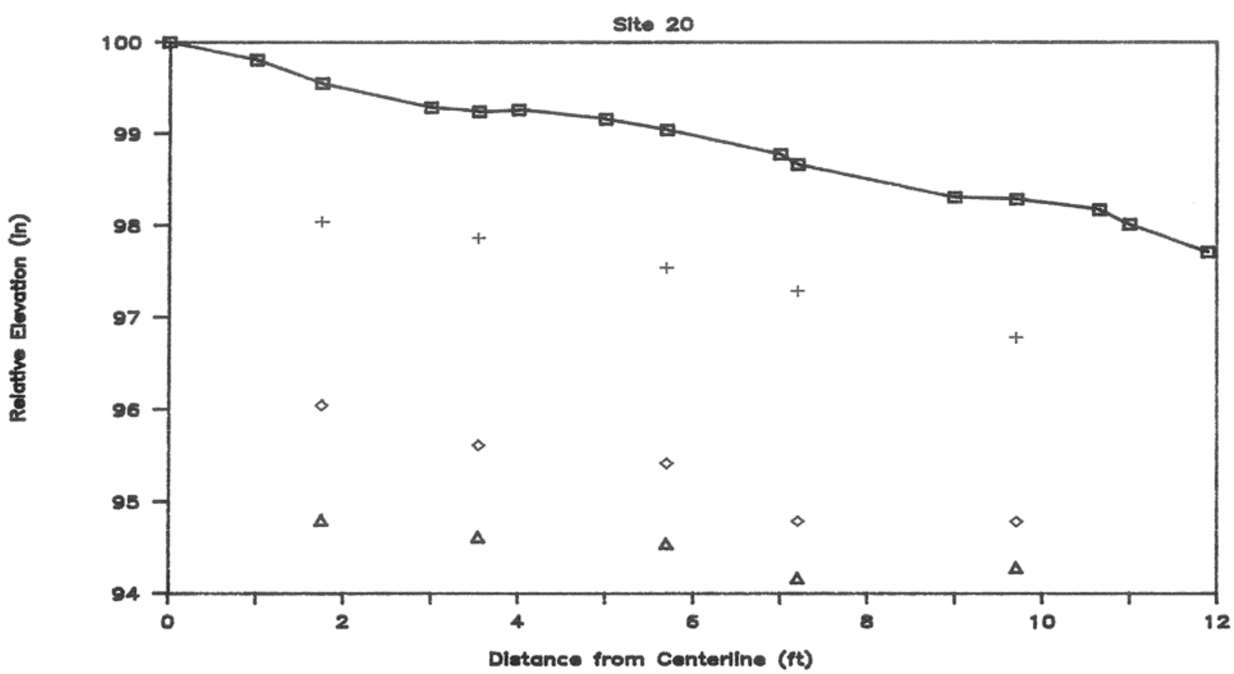

Figure 33. Transverse Profile of Project 20

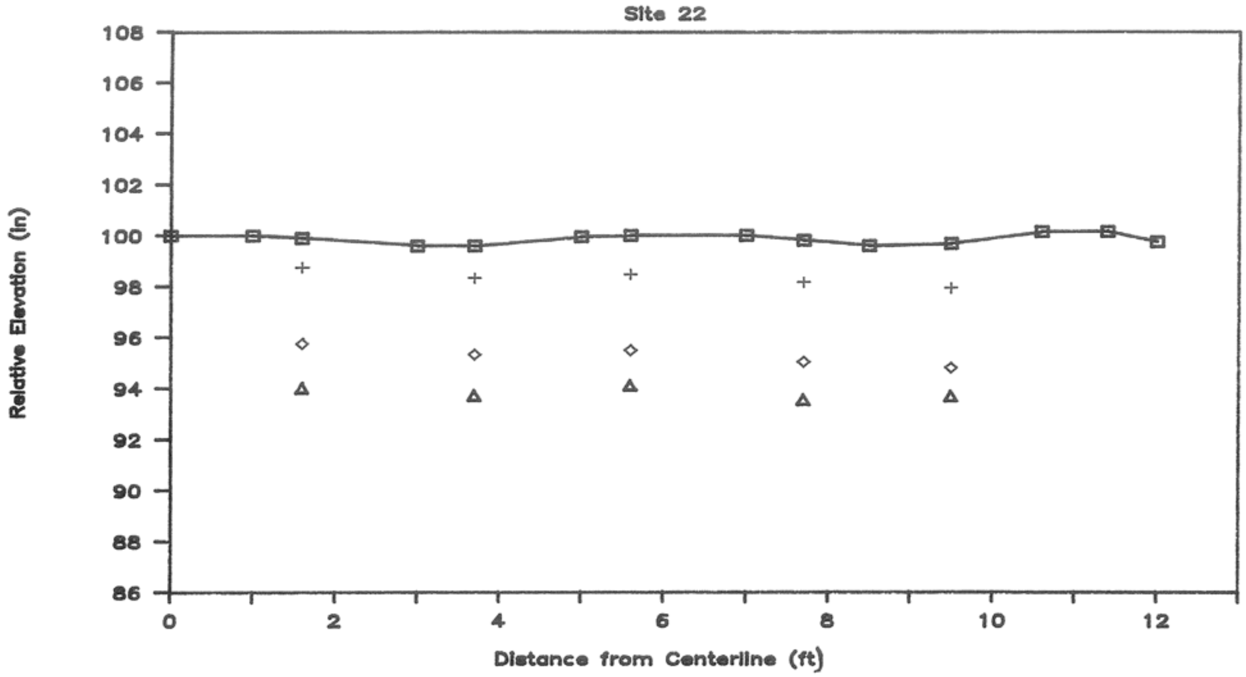

Figure 34. Transverse Profile of Project 22

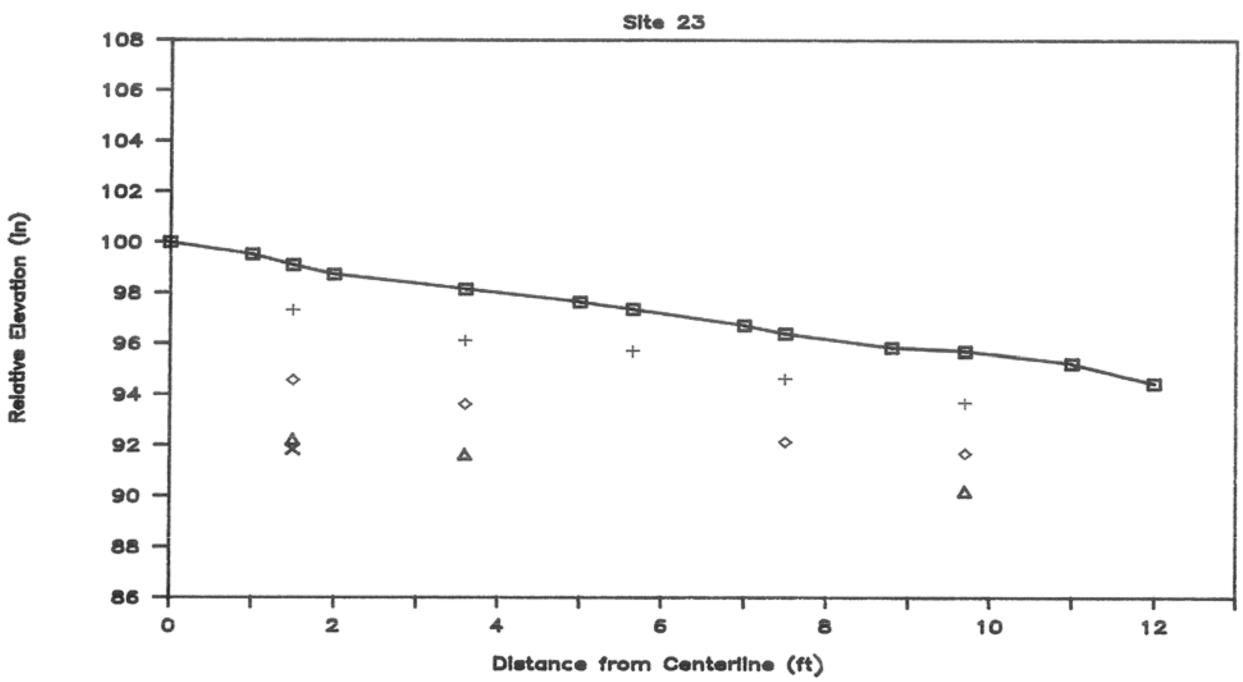

Figure 35. Transverse Profile of Project 23

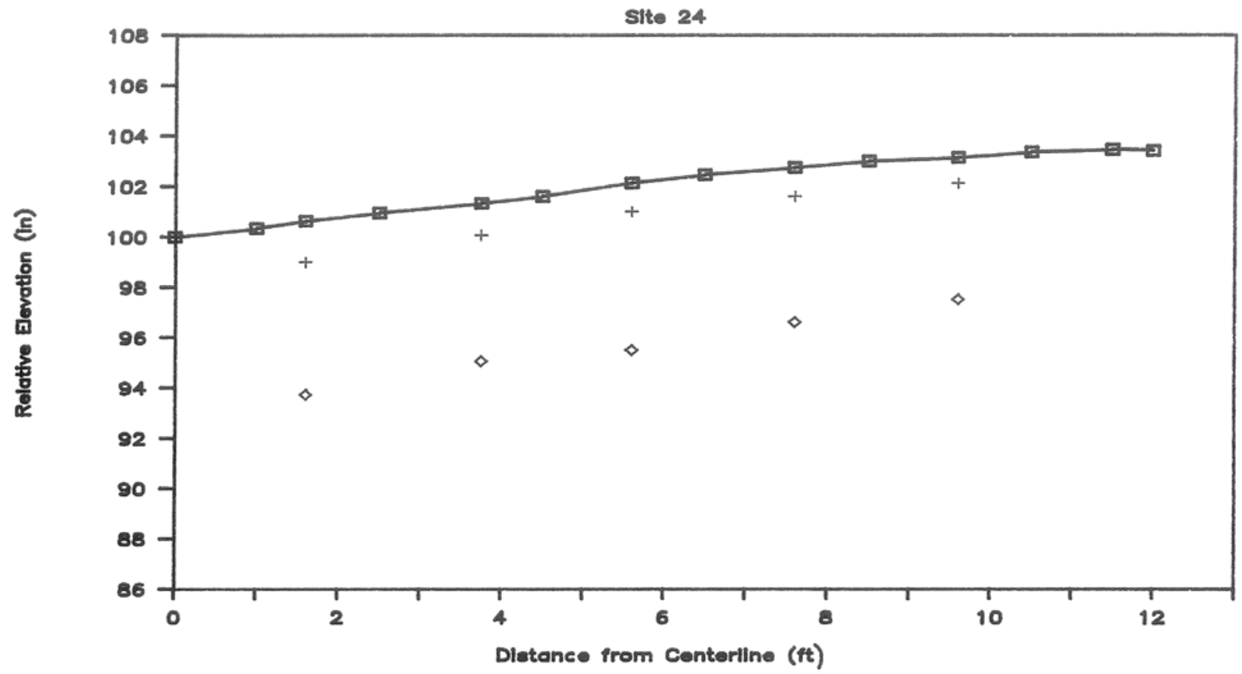

Figure 36. Transverse Profile of Project 24

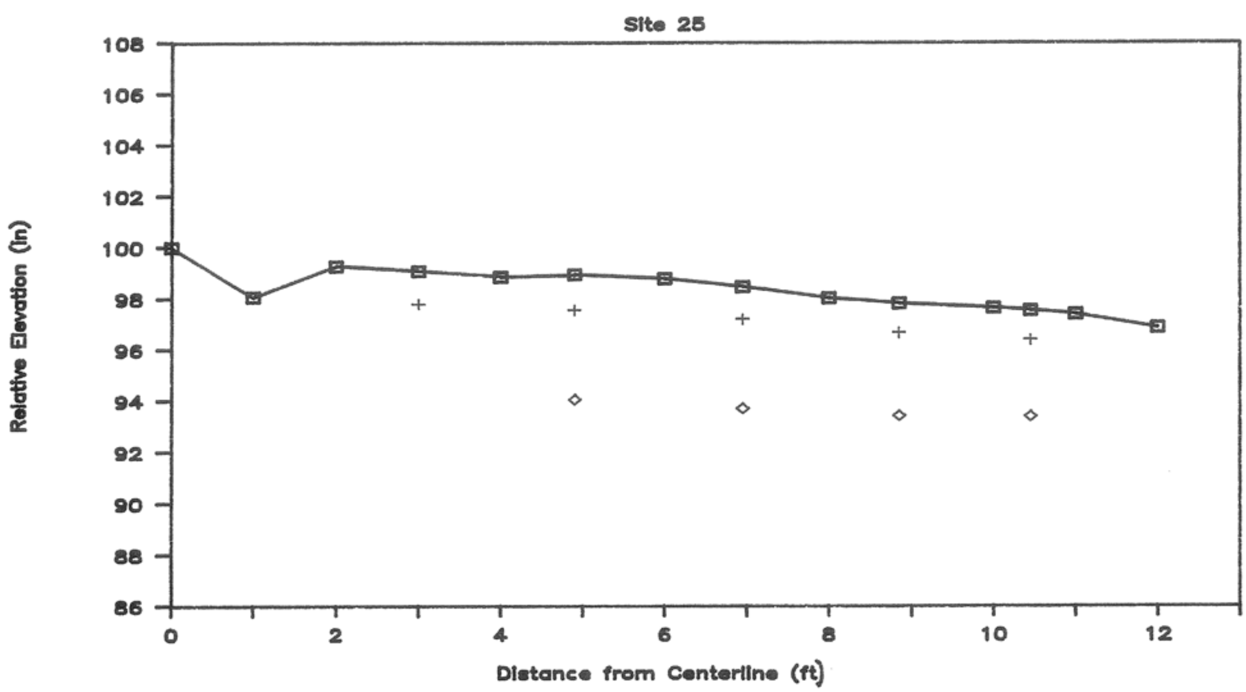

Figure 37. Transverse Profile of Project 25

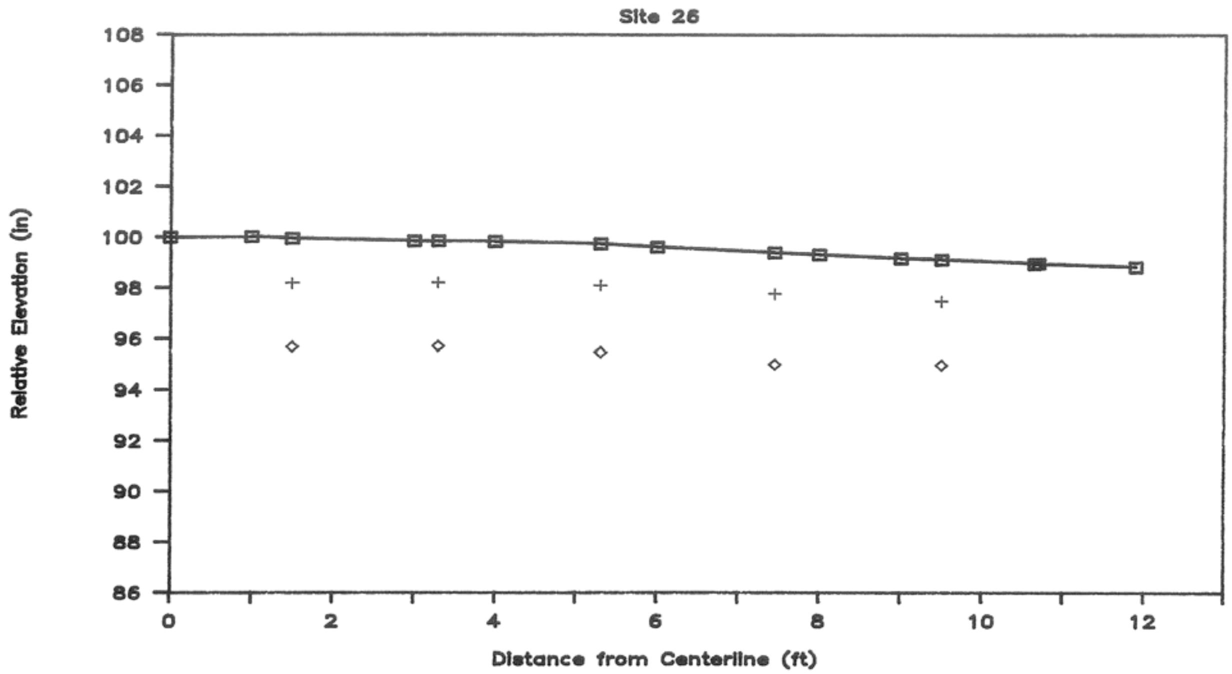

Figure 38. Transverse Profile of Project 26

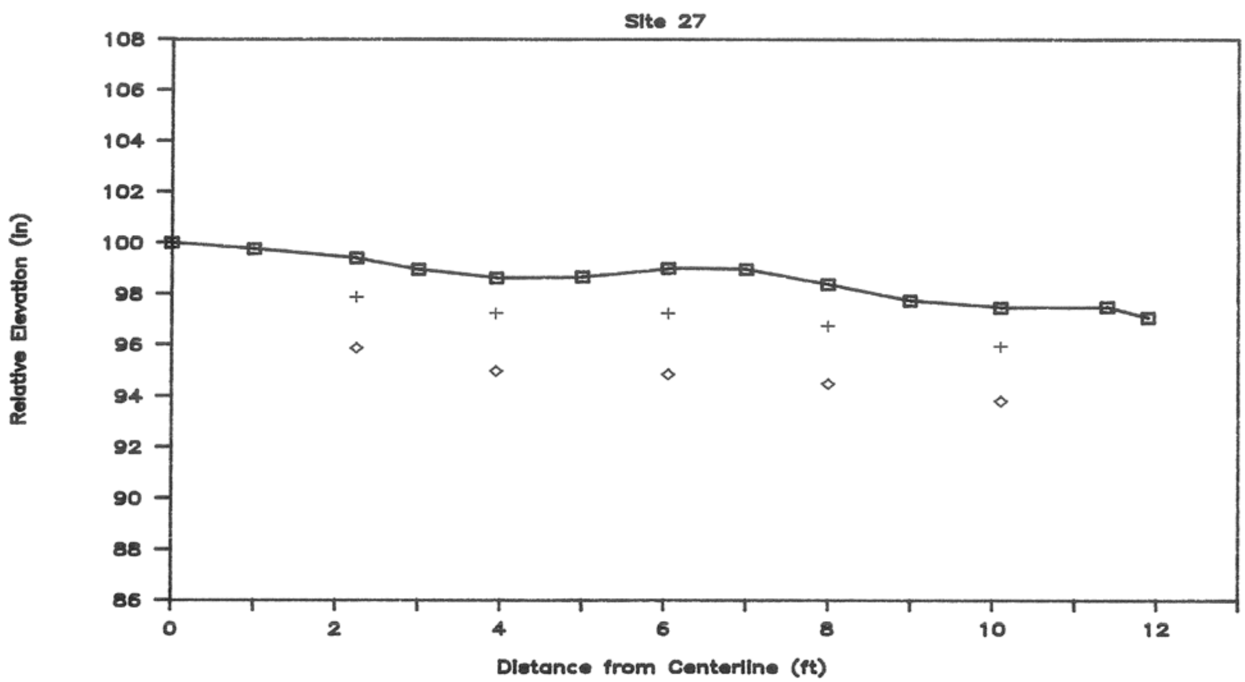

Figure 39. Transverse Profile of Project 27

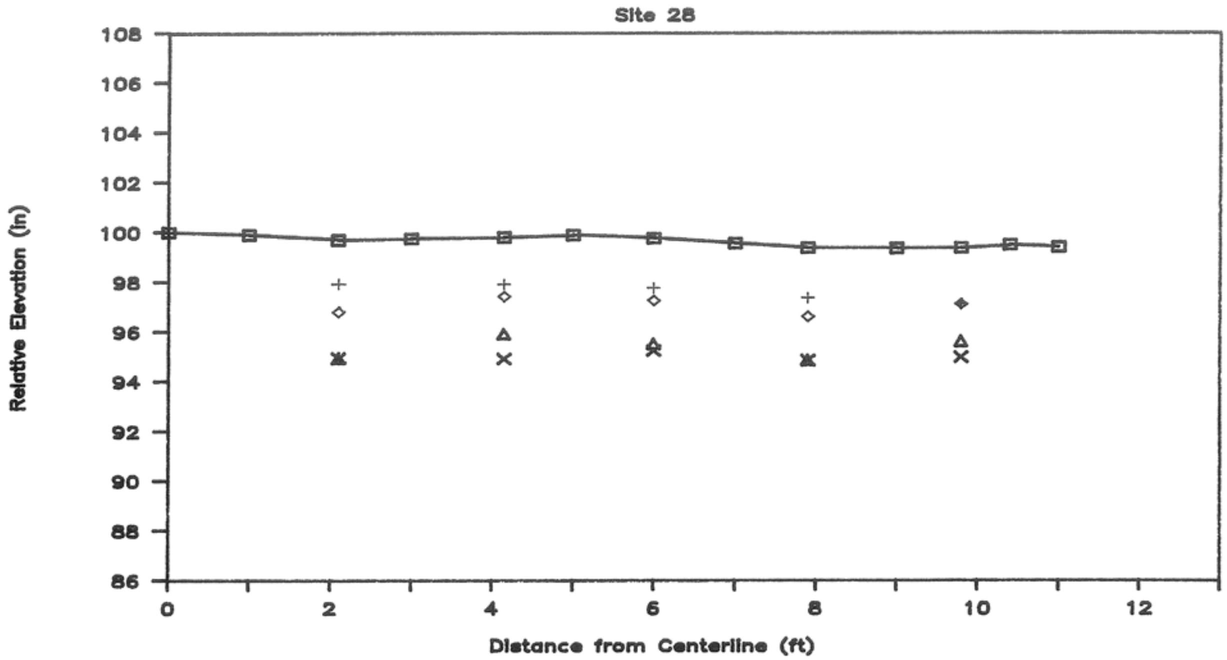

Figure 40. Transverse Profile of Project 28

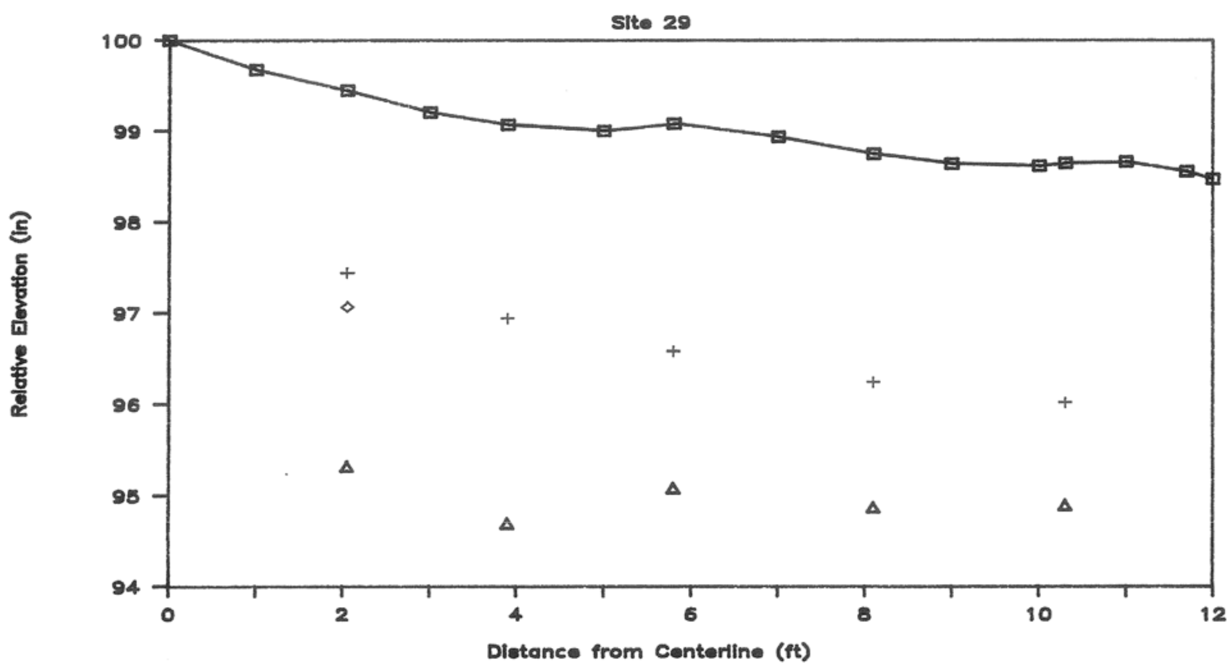

Figure 41. Transverse Profile of Project 29

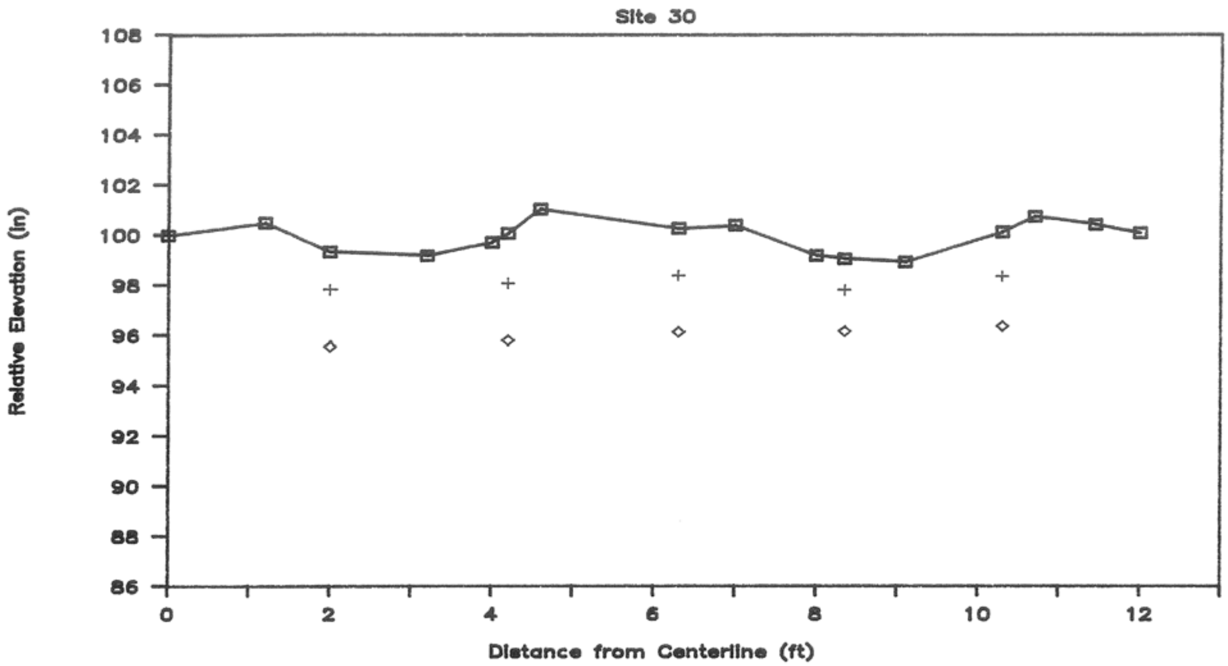

Figure 42. Transverse Profile of Project 30

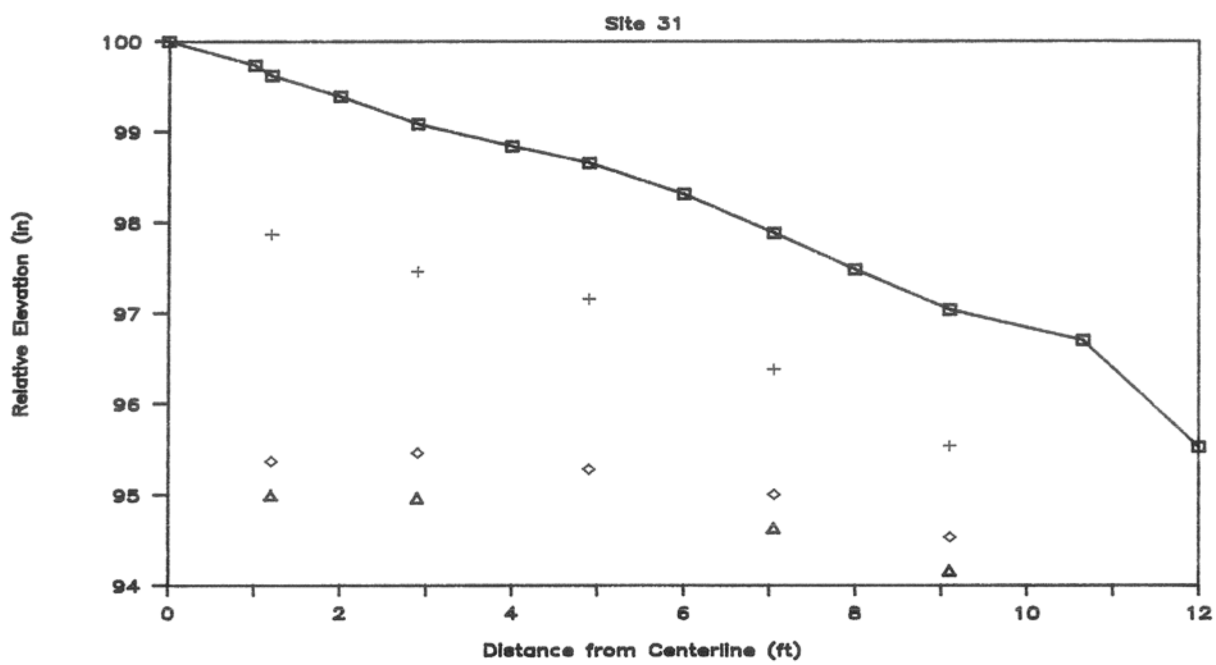

Figure 43. Transverse Profile of Project 31

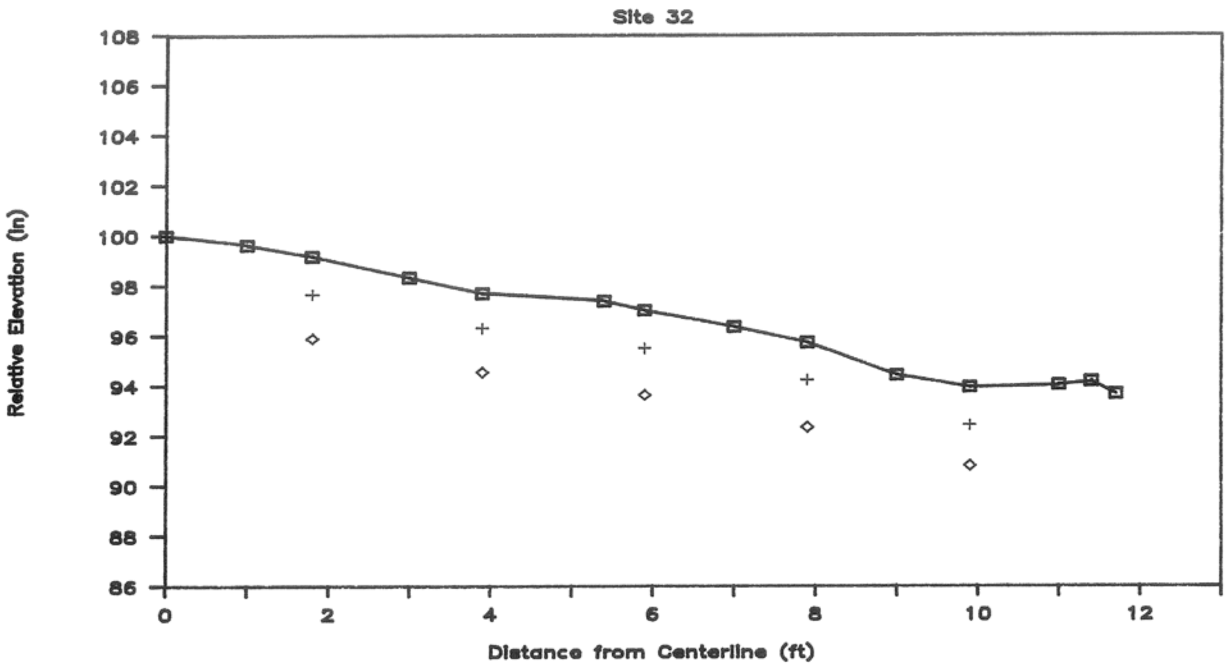

Figure 44. Transverse Profile of Project 32

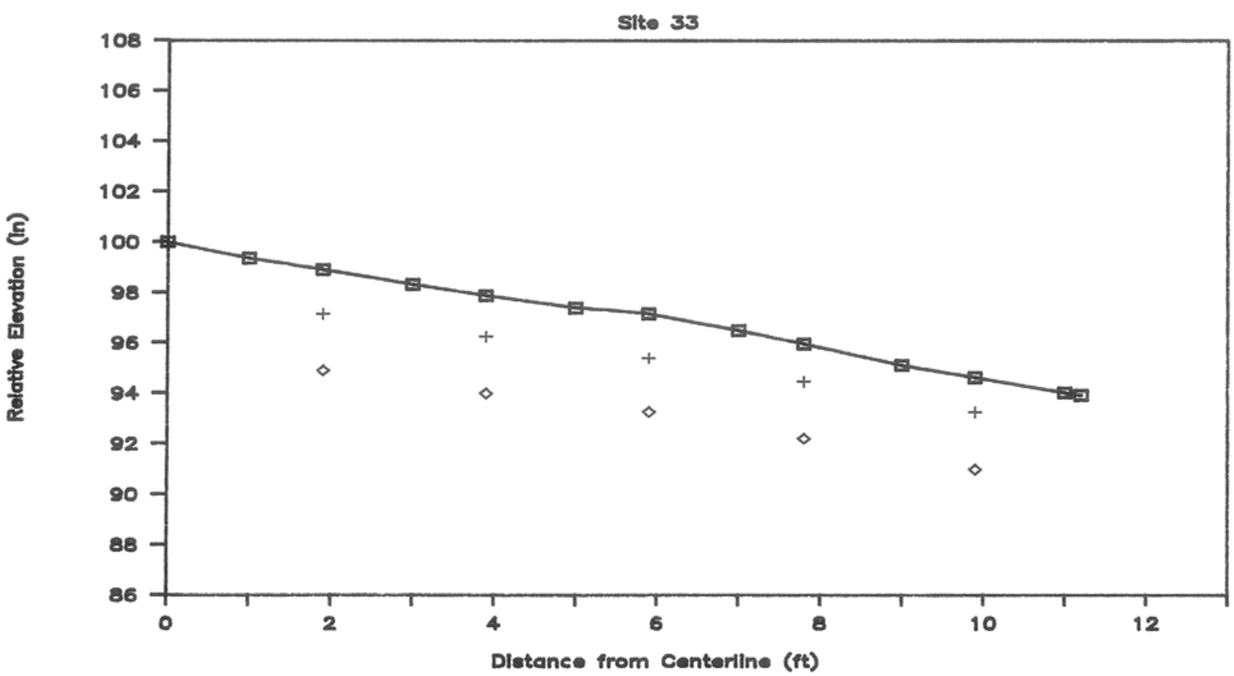

Figure 45. Transverse Profile of Project 33

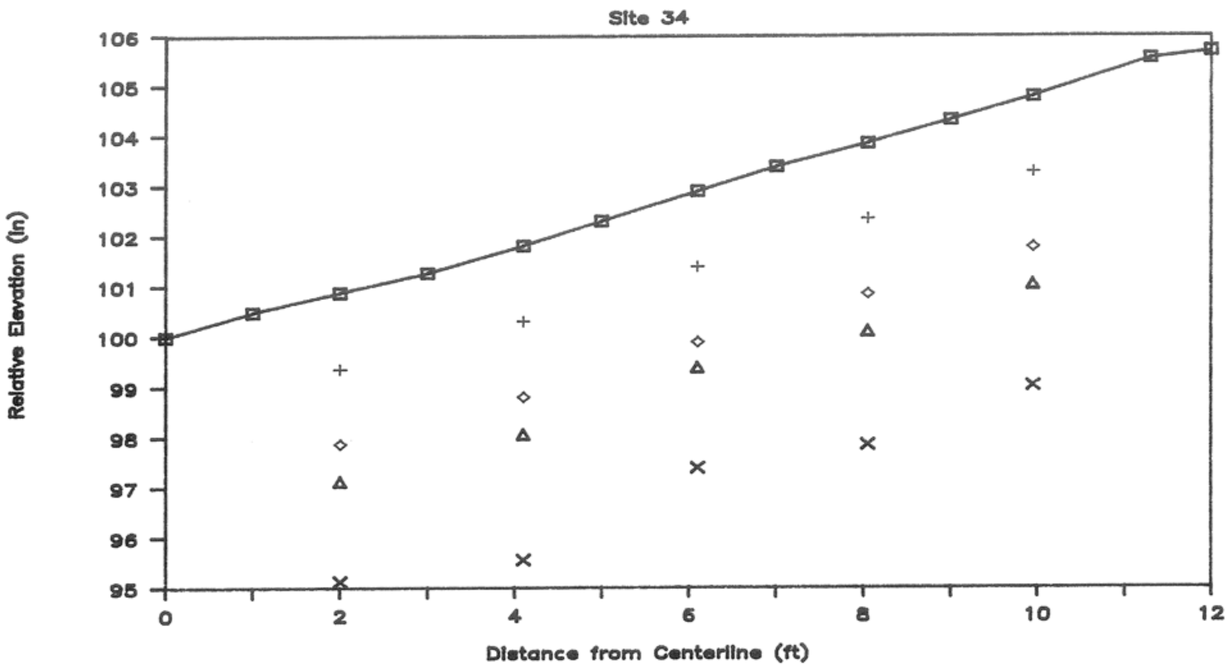

Figure 46. Transverse Profile of Project 34

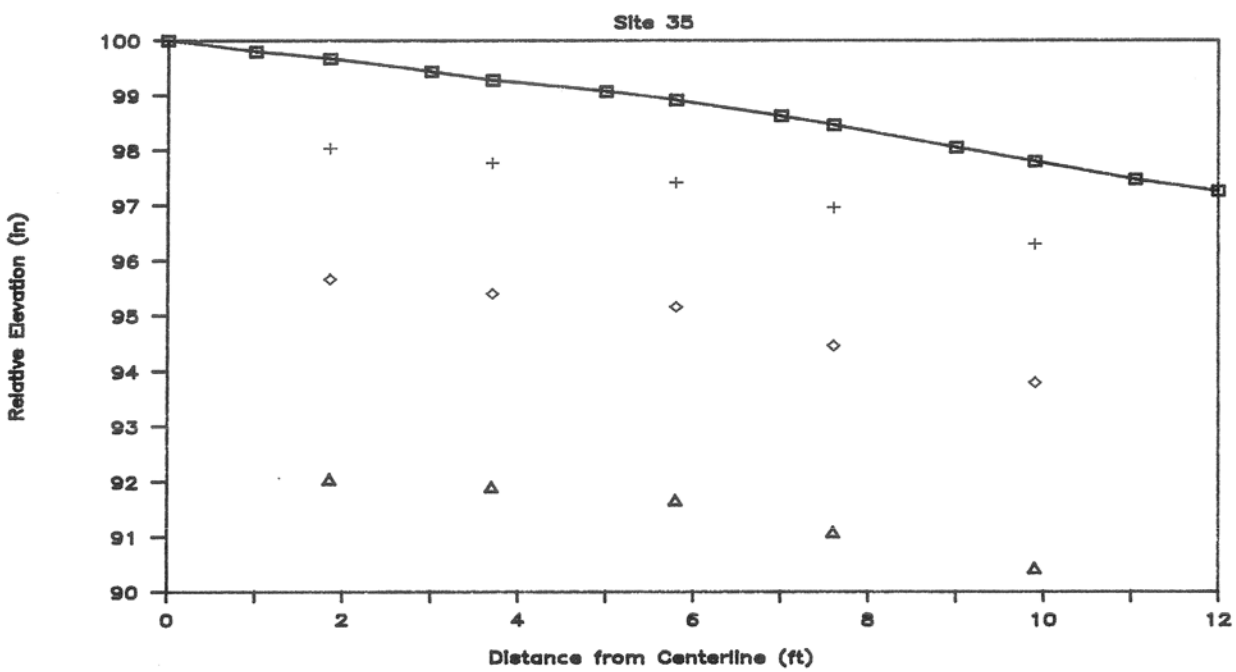

Figure 47. Transverse Profile of Project 35

Table 13. Rut Depth Data (C7-C11) in Inches

Site No.	County	SR	Layer No.	New Surface Rut Depth (in)	New Max. Rut in Layer (in)	Surface Rut Depth (500 ft) (in)	Pavement Age	Subjective Rating	Total ESALs (x10EE6)	Rut Depth 500 ft/ SQRT (TESALs)
1	Jefferson	80	1	0.076	0.076	0.150	4	E	3.87	0.076
1	Jefferson	80	2	---	0.000	---	---	---	---	---
1	Jefferson	80	3	---	0.000	---	---	---	---	---
2	Allegheny	279	1	0.593	0.196	0.325	3	F	2.31	0.214
2	Allegheny	279	2	---	0.397	---	---		---	---
3	Allegheny	279	1	0.243	0.000	0.400	5	F	3.71	0.208
3	Allegheny	279	2	---	0.243	---	---	---	---	---
4	Allegheny	279	1	0.551	0.122	0.125	5	E	3.71	0.065
4	Allegheny	279	2	---	0.429	---	---	---	---	---
5	Westmoreland	30	1	0.312	0.052	N/A	3	F	0.48	---
5	Westmoreland	30	2	---	0.260	---	---	---	---	---
6	Westmoreland	30	1	0.262	0.015	0.300	4	G	0.66	0.369
6	Westmoreland	30	2	---	0.000	---	---	---	---	---
6	Westmoreland	30	3	---	0.246	---	---	---	---	---
7	Washington	70	1	0.556	0.416	0.650	3	P	3.08	0.370
7	Washington	70	2	---	0.000	---	---	---	---	---
7	Washington	70	3	---	0.140	---	---	---	---	---
8	Lawerance	70	1	N/A	N/A	N/A	18	E	17.88	---
8	Lawerance	70	2	---	N/A	---	---	---	---	---
8	Lawerance	70	3	---	N/A	---	---	---	---	---
8	Lawerance	70	4	---	N/A	---	---	---	---	---
9	Bedford	70/76	1	0.683	0.628	0.700	19	F	34.31	0.120
9	Bedford	70/76	2	---	0.064	---	---	---	---	---
9	Bedford	70/76	3	---	0.006	---	---	---	---	---
9	Bedford	70/76	4	---	-0.109	---	---	---	---	---

Table 13. Rut Depth Data (C7-C11) in Inches (Continued)

Site No.	County	SR	Layer No.	New Surface Rut Depth (in)	New Max. Rut in Layer (in)	Surface Rut Depth (500 ft) (in)	Pavement Age	Subjective Rating	Total ESALs (x10EE6)	Rut Depth 500 ft/ SQRT (TESALs)
10	Somerset	70/76	1	0.371	0.262	0.250	9	G	23.53	0.052
10	Somerset	70/76	2	---	0.109	---	---	---	---	---
11	Erie	90	1	0.550	0.526	1.650	---	P	---	---
11	Erie	90	2&3	---	0.024	---	---	---	---	---
12	Luzerne	81	1	0.186	0.114	0.100	3	E	2.91	0.059
12	Luzerne	81	2	---	0.000	---	---	---	---	---
12	Luzerne	81	3	---	0.072	---	---	--	---	---
12	Luzerne	81	4	---	0.000	---	---	---	---	---
13	Luzerne	81	1	0.168	0.110	0.200	3	G	2.91	0.117
13	Luzerne	81	2	---	0.000	---	---	---	---	---
13	Luzerne	81	3	---	0.058	---	---	---	---	---
14	Lackawana	81	1	0.411	0.306	0.350	4	G	2.46	0.223
14	Lackawana	81	2	---	0.105	---	---	---	---	---
14	Lackawana	81	3	---	0.000	---	---	---	---	---
15	Lehigh	78	1	0.300	0.197	0.200	3	G	3.60	0.105
15	Lehigh	78	2	---	0.103	---	---	---	---	---
16	Lehigh	78	1	0.343	0.343	0.400	4	F	4.59	0.187
16	Lehigh	78	2	---	0.000	---	---	---	---	---
17	Schuykill	81	1	0.144	0.144	0.200	3	G	1.67	0.155
17	Schuykill	81	2	---	0.000	---	---	---	---	---
18	Schuykill	81	1	1.627	0.705	0.650	7	F	2.84	0.386
18	Schuykill	81	2	---	0.384	---	---	---	---	---
18	Schuykill	81	3	---	0.538	---	---	---	---	---

Table 13. Rut Depth Data (C7-C11) in Inches (Continued)

Site No.	County	SR	Layer No.	New Surface Rut Depth (in)	New Max. Rut in Layer (in)	Surface Rut Depth (500 ft) (in)	Pavement Age	Subjective Rating	Total ESALs (x10EE6)	Rut Depth 500 ft/ SQRT (TESALs)
19	Berks	222	1	0.393	0.184	OP350	5	G	2.83	0.208
19	Berks	222	2	---	0.209	---	---	---	---	---
19	Berks	222	3	---	0.000	---	---	---	---	---
20	Berks	222	1	0.212	0.212	0.225	4	E	1.67	0.174
20	Berks	222	2	---	0.000	---	---	---	---	---
20	Berks	222	3	---	0.000	---	---	---	---	---
22	Montgomery	76	1	0.487	0.145	0.400	5	F	2.56	0.250
22	Montgomery	76	2	---	0.000	---	---	---	---	---
22	Montgomery	76	3	---	0.342	---	---	---	---	---
23	Montgomery	76	1	0.329	0.102	0.400	4	F	2.35	0.261
23	Montgomery	76	2	---	0.227	---	---	---	---	---
23	Montgomery	76	3	---	0.000	---	---	---	---	---
24	Montgomery	76	1	0.117	0.112	0.100	2	E	1.29	0.088
24	Montgomery	76	2	---	0.005	---	---	---	---	---
25	Perry	11	1	0.333	0.068	0.225	14	E	3.39	0.122
25	Perry	11	2	---	0.265	---	---	---	---	---
26	York	83	1	0.060	0.060	0.000	2	E	1.46	0.000
26	York	83	2	---	0.000	---	---	---	---	---
27	York	83	1	0.795	0.436	0.550	5	F	4.44	0.261
27	York	83	2	---	0.359	--	---	---	---	---
28	Dauphin	22	1	0.308	0.308	0.200	2	E	0.63	0.252
28	Dauphin	22	2	---	0.000	---	---	---	---	---
28	Dauphin	22	3	---	0.000	---	---	---	---	---
28	Dauphin	22	4	---	0.000	---	---	---	---	---
28	Dauphin	22	5	---	0.000	---	---	---	---	---

Table 13. Rut Depth Data (C7-C11) in Inches (Continued)

Site No.	County	SR	Layer No.	New Surface Rut Depth (in)	New Max. Rut in Layer (in)	Surface Rut Depth (500 ft) (in)	Pavement Age	Subjective Rating	Total ESALs (x10EE6)	Rut Depth 500 ft/ SQRT (TESALs)
29	Cumberland	11	1	0.317	0.239	0.350	2	F	0.34	0.600
29	Cumberland	11	2	---	0.078	---	---	---	---	---
30	Cumberland	11	1	1.664	1.001	1.300	6	P	3.90	0.658
30	Cumberland	11	2	---	0.663	---	---	---	---	---
31	Blair	220	1	0.200	0.140	0.250	3	F	0.60	0.323
31	Blair	220	2	---	0.060	---	---	---	---	---
31	Blair	220	3	---	0.000	---	---	---	---	---
32	Fulton	70	1	1.038	0.467	0.200	2	G	1.52	0.162
32	Fulton	70	2	---	0.571	---	---	---	---	---
33	Fulton	70	1	0.232	0.185	0.300	2	F	1.52	0.243
33	Fulton	70	2	---	0.047	---	---	---	---	---
34	Bedford	70	1	0.183	0.183	0.275	4	G	1.86	0.202
34	Bedford	70	2	---	0.000	---	---	---	---	---
34	Bedford	70	3	---	0.000	---	---	---	---	---
34	Bedford	70	4	---	0.000	---	---	---	---	---
35	Jefferson	80	1	0.036	0.036	0.000	2	E	1.90	0.000
35	Jefferson	80	2	---	0.000	---	---	---	---	---
35	Jefferson	80	3	---	0.000	---	---	---	---	---
Layer 1										
	Average:			0.427	0.245	0.367	5.0		4.45	0.210
	Std Deviation:			0.376	0.218	0.334	4.2		6.98	0.149
Layer 2										
	Average:			N/A	0.143	N/A	N/A		N/A	N/A
	Std Deviation:			N/A	0.183	N/A	N/A		N/A	N/A

all layers for each project. It should be mentioned again that on some projects considerations for sight distance and safety precluded coring and measuring rut depths at the worst location. The table also contains the maximum surface rut depth at the other location within 500 feet of the worst location. This is supposed to represent the project segment evaluated. Therefore, rut depths (profile) were measured at two locations only on each site.

Statistical analysis of the maximum surface rut depth (inch) data is as follows:

	Surface Rut Depth		Rut in Each Layer Worst Location	
	Worst Location	500 ft	Layer 1	Layer 2
Number of Projects	34	34	29	29
Mean	0.43	0.37	0.24	0.14
Standard Deviation	0.38	0.33	0.22	0.18

Maximum surface rut depth at the worst site on all projects ranged from 0.04 inch (Site #35) to 1.66 inches (Site #30), averaging 0.43 inch. Although the average surface rut depth obtained at the 500 feet location was slightly lower than those obtained at the worst location, there are significant differences between the two on many individual projects. The average rut depth in the wearing course is 0.10 inch greater than in the binder.

As shown in Table 13 there are several projects where the underlying layers contributed significantly to the total surface rut depth. Fifteen poor to fair projects can be broken down into three general categories as follows:

Type	Project Nos.
Projects in which the underlying layers contributed significantly (in addition to the wearing course) towards the total surface rut depth	2, 7, 18, 22, 23, 27, 29, 30, 31, and 33 (Ten Projects)
Projects in which the underlying layers were primarily responsible for the total surface rut depth.	3 and 5 (Two Projects)
Projects where most rutting contributed by the wearing course only	9, 11 and 16 (Three Projects)

Profiles of the underlying layers were obtained by subtracting the layer thickness of the transverse cores from the surface profile. The cores were taken at two-foot intervals at only <u>one</u> location (worst site). The thickness of layers is not always the same across the pavement when constructed. Therefore, the rut depth data for individual layers reported in Table 13 can only be considered approximate and, therefore, the preceding categorization is not absolute. However, it appears that in a majority of cases the underlying layers (including the binder course) contributed to the surface rut depth.

Table 13 also gives the total 18-kip equivalent single axle loads (TESALs) carried by the HMA overlays as of 1990. The last column gives the values of surface rut depth at 500 feet divided by the square root of TESALs. This will be discussed later.

Visual Observations of Project Sites and Notes

All sites were visited, inspected visually, and photographed to document the general terrain and conditions in the vicinity of the coring site (Cores C7-C11). Figure 48 shows a typical view of coring locations C7-C11. Table 14 summarizes specific observation notes taken and refers to the corresponding photographs (Figures 49 through 83). Unless it is indicated otherwise photographs of each project were taken from a point ahead of the site where transverse cores (C7-C11) were taken.

Figure 48. Typical View of C7-C11 Coring Site

Table 14. Visual Observations and General Notes

Site No.	County	SR	Max. Surface Rut Depth (500')	Pavement Age (1990)	Subjective Rating	Figure (Photo) No.	Notes
1	Jefferson	80	0.150	4	E	49	1-90 (W. B) site on long up grade E. of Hazen exit, excellent job, in-place voids 4.0% in wearing and 5.2% in binder, no natural sand in both courses.
2	Allegheny	279	0.325	3	F	50	Pittsburgh Parkway (W.B.), site on long up grade, rutting gets worse going uphill away from the coring site, middle lane (of the 3 W.B. lanes) has more rutting due to construction. Binder course has #467 coarse aggregate, in-place voids 1.9% in wearing and 1.5% in binder course, 25% natural sand in fine aggregate of both courses, according to rut profiles the binder course is the major contributor to rutting surface.
3	Allegheny	279	0.400	5	F	51	Pittsburgh Parkway (E.B.), site on long up grade, more rutting in the middle lane (of the 3 E.B. lanes) because of traffic backup before the exit. ID-2 wearing has absorptive gravel aggregate (water absorption more than 2.5%), in-place voids 1.4% in wearing and 2.4% in binder course, 100% natural sand used as fine aggregate in both courses, according to rut profiles the binder course is the main contributor to surface rutting.
4	Allegheny	279	0.312	5	E	52	Pittsburgh Parkway (W.B.), just across from Site #3 above, however different mix in the wearing course, rutting occurring in 2 lanes of the 4 W.B. lanes, site on long up grade, in-place voids 2.4% in wearing and 1.5% in binder course, 75% natural sand in wearing course and 100% natural sand in the binder course, according to rut profiles the binder course is the major contributor to surface rutting.
5	Westmoreland	30	0.300	3	F	53,54	Route 30 (E.B. lanes), site on long up-grade, 75-blow Marshall design (3.0% voids), this section had 2-way hot weather traffic after construction, mix ruts on long up grades and when traffic is channelized, in-place voids 3.1% in wearing and 1.8% in binder course, 25% natural sand in the fine aggregate of both courses, according to the rut profile the binder course is the major contributor to surface rutting.
6	Westmoreland	30	0.300	4	G	55	Route 30 (W.B. lanes), site on long up grade, 50-blow Marshall design (3.1% voids), this section had 2-way traffic but only during winter, in-place voids 3.0% in wearing and 3.5% in binder course, 50% natural sand in the fine aggregate of both courses, according to rut profiles rutting appears to be in the leveling course (Layer 3).
7	Washington	70	0.650	3	P	56	1-70 (E.B. lanes) near W. VA. border between Exits 2 and 3, long steep up grade, cores taken from area which had rutted earlier, milled off and replaced in 1997, wearing and binder designed with 75 blows (3.5-3.6% voids), was a semi-heavy duty design, in-place voids 2.3% in wearing course, 24% natural sand in the fine aggregate of both courses.
8	Lawrence	76 (T.P.)	0.180	18	E	---	Very old Pennsylvania Turnpike section, maximum surface rut depth of 0.180" was measured in 1989, now the pavement has been milled off and overlayed, 65-blow Marshall design, in-place voids 0.9% in wearing course of this 18 year old overlay.

Table 14. Visual Observations and General Notes (Continued)

Site No.	County	SR	Max. Surface Rut Depth (500')	Pavement Age (1990)	Subjective Rating	Figure (Photo) No.	Notes
9	Bedford	70/76 (T.P.)	0.700	19	F	57	Very old Pennsylvania Turnpike section, mild up grade on curve, rutting primarily in slow lane, minimal rutting in the passing lane after 19 years, 65-blow Marshall design, in-place voids 1.9% in wearing and 2.0% in binder course of this 19-year old overlay.
10	Somerset	70/76 (T.P.)	0.250	9	G	---	Pennsylvania Turnpike, site on long up grade, cores taken from the truck (3rd) lane, which was added nine years ago (10" Bit. Conc. Base course, 2" binder and 1" wearing course), 65-blow Marshall design, in-place voids of 0.6% in wearing and 4% in binder course, no natural sand in the wearing course, according to rut profile some rutting has taken place in the binder course as well.
11	Erie	90	1.650	3	P	58	1-90 (W.B. lanes) near N.Y. border, coring site on a slight down grade, excessive rutting in the traffic lane, no significant rutting in the passing lane; old wearing course had been milled off, a seal coat was applied and then a heavy tack coat applied before placing the new wearing course; possible slippage due to tack coat; daily 50-blow Marshall data shows air voids less than 3.0% consistently, in-place voids almost 0% in wearing and 3.7% in old binder course.
12	Luzerne	81	0.100	3	E	59	I-81 (N.B. lanes), site on very long up grade, mix design same as Site #13 (same contract), binder course had low air voids (more than 97% of T.M.D.) at the time of construction, in-place voids 4.9 % in wearing and 2.0% in binder course, 25% natural send in fine aggregate of wearing course and none in binder course.
13	Luzerne	81	0.200	3	G	60	I-81 (S.B. lanes), site on down grade, 24' wide paver used, some segregation problems experienced, in-place voids 2.4% in wearing and 0.4% in binder course, 25% natural sand in the fine aggregate of wearing and none in binder course.
14	Lackawana	81	0.350	4	G	61	1-81 (S.B. lanes), core site on long up grade and also on curve, in-place voids 2.3% in wearing and 4.0% in binder course, 100% manufactured sand in both courses.
15	Lehigh	78	0.200	3	G	62	1-78 (W.B. lanes) just across from Site #16, core site on a down grade, asphalt content same as Site #16, slow lane had channelized traffic and is flushing in wheel paths, in-place voids 3.0% in wearing and 3.7% in binder course, 100% manufactured sand in both courses.
16	Lehigh	78	0.400	4	F	63	1-78 (E.B. lanes), site on a relatively flat area, no flushing, 50-blow Marshall design, in-place voids 3.9% in both courses, no natural sand in wearing course, 50% natural sand in the fine aggregate of binder course.
17	Schuykill	81	0.200	3	G	64	I-81 (S.B. lanes), site on mild up grade, no flushing, Mix looks dry (possibly due to gravel aggregate), sawed & scaled joints look good, some water seeping out from center line joint, in-place voids 3.1% in wearing and 4.0% in binder course, 100% natural sand in both courses.

Table 14. Visual Observations and General Notes (Continued)

Site No.	County	SR	Max. Surface Rut Depth (500')	Pavement Age (1990)	Subjective Rating	Figure (Photo) No.	Notes
18	Schuykill	81	0.650	7	F	65, 66	I-81 (N.B. lanes), site on a very long up grade north of Ravine, excessive rutting in the slow lane, minimal rutting in the passing lane or S.B. lanes going down hill, no flushing, 100% flexible pavement, rutting 1.627" at the worst site, in-place voids (average) 2.3% in wearing course and 1.7% in binder course, no natural sand in both courses, according to rut profiles significant rutting has taken place in binder course and base courses as well.
19	Berks	222	0.350	5	G		Route 222 (E.B. lane), 3-lane highway in rural area, relatively flat terrain, rutting near the intersections approaching 1/2", in-place voids 4.3% in wearing and 3.6% in binder course, 100% manufactured sand in both courses, according to rut profiles both binder and wearing courses am contributing to rutting.
20	Berks	222	0.225	4	E	67	Route 222, 4-lane divided highway with N.J. barrier, inside the Town of Reading, relatively flat terrain, 100% crushed limestone in both binder and wearing courses, in-place voids 2.7% in wearing and 2.1% in binder course, 100% manufactured sand in both courses.
22	Montgomery	76	0.400	5	F	68	Schuykill Expressway (Section 100) E.B. lance, mild up grade, Just off Pennsylvania Turnpike in Philadelphia, 4-lanes with tall median divider, some rutting in the passing lane as well, fast moving traffic, average in-place voids 3.1% in wearing and 1.4% in binder course, 50% natural sand in the fine aggregate of wearing and 45% in the binder course mix.
23	Montgomery	76	0.400	4	F	69	Schuykill Expressway (Section 300), E.B. lanes, relatively flat terrain, average in place voids 2.6% in wearing and 1.7% in binder course, 50% natural sand in the fine aggregate of wearing and 45% in the binder course mix, according to rut profiles binder course is the major contributor to the surface rutting.
24	Montgomery	76	0.100	2	E	70	Schuykill Expressway (Section 420), W.B. lance, 3-lanes W.B., coring site in outside lane which is merging lane and carries relatively less traffic, very slight up grade, rutting in the middle lane estimated to be 1/4% average in-place voids 3.9% in wearing course and 2.6% in binder course, 20% natural sand in the fine aggregate of both courses, heavy duty mix design.
25	Perry	11	0.225	14	E	71	Route 11 near Amity Hall, 3-lane highway, almost level terrain, has a special binder course (42% pass. 1/2") above PCC pavement, ideal pavement with minimal rutting after 14 years, in-place voids 3.6% in wearing and 4.7% in binder course, 100% manufactured sand in the binder mix, according to rut profiles binder course has contributed most to the surface rutting.
26	York	83	0.000	2	E	72	1-83 near Mile Post 5, mostly flat terrain, heavy duty mix design, surface appeared black with no aggregate exposed in 1989, looks excellent so far, in-place voids 7.4% in wearing and 5.4% in binder, 100% manufactured sand in both courses, heavy duty mix design.

Table 14. Visual Observations and General Notes (Continued)

Site No.	County	SR	Max. Surface Rut Depth (500')	Pavement Age (1990)	Subjective Rating	Figure (Photo) No.	Notes
27	York	83	0.550	5	F	73	I-83 between Harrisburg and York (S.B. lanes), site on a long up grade, rutting primarily in slow lane, gravel aggregate mix looks dry on the edge of slow lane and in the passing lane, in-place voids 2.5% in wearing and 2.9% in binder course, according to rut profiles binder course is also contributing significantly to the surface rutting.
28	Dauphin	22	0.200	2	E	74,75	Route 22 in Dauphin Boro, 3-lane highway, almost level terrain, ID-3 wearing course with AC-30 asphalt cement and significant amount of +1/2" aggregate, some segregation visible, in-place voids 5.1 % in ID-3, 100 % manufactured sand in mix.
29	Cumberland	11	0.350	2	F	76	Route 11 (Camp Hill By-pass) S.B. lanes, site on a mild long up grade, ID-3 wearing course flushing badly, rutting more pronounced near intersections, in-place voids 2.6% in ED-3 course.
30	Cumberland	11	1.300	6	P	77,78	Route 11 (between Mechanicsburg and Carlisle), W.B. lanes, site on a flat terrain, excessive rutting and shoving in slow lanes, unfilled core holes were getting smaller due to plastic flow of the mix inwards, heavy truck traffic slowing down to get on Pennsylvania Turnpike, in-place voids 2.9% in wearing course, according to rut profiles the binder course is also contributing significantly to the surface rutting.
31	Blair	220	0.230	3	F	79	Route 220, site on a mild up grade, rutting is more pronounced near the intersections as reported later, in-place voids 5.9% in wearing course, 100% manufactured sand in the fine aggregate of wearing course.
32	Fulton	70	0.200	2	G	80	1-70 (E.B. lanes), site on a long steep up grade, minimal rutting downhill, heavy duty mix design (4.2% voids), in-place voids 4.4% in wearing and 2.5% in binder course, 100% manufactured sand in both courses, according to the nit profiles the binder course is a major contributor to the surface rutting.
33	Fulton	70	0.300	2	F	81	1-70 (W.B. lanes), site on a long mild up grade, heavy duty mix same as Site #32, minus 200 content during production higher than Site #32, in-place voids 4.9% in wearing and 3.6% in binder course, 100% manufactured sand in both courses.
34	Bedford	70	0.275	4	G	82	1-70 (just south of Pennsylvania Turnpike), site on a down grade, no flushing, mix ruts near intersections, in-place voids 2.5% in wearing course, 100% manufactured sand in both courses.
35	Jefferson	80	0.000	2	E	83	1-90 (W.B. lanes), 2 miles E. of Corsica Exit, core site on a moderate up grade, these W.B. lanes did not have 2-way traffic (built after E.B. lanes), even E.B. lanes do not show any significant rutting, appears like an excellent job so far, in-place voids 5.9% in wearing and 5.9% in binder course, 100% manufactured sand in both courses, heavy duty mix design.

Figure 49. Project 1, I-80 (W.B.)

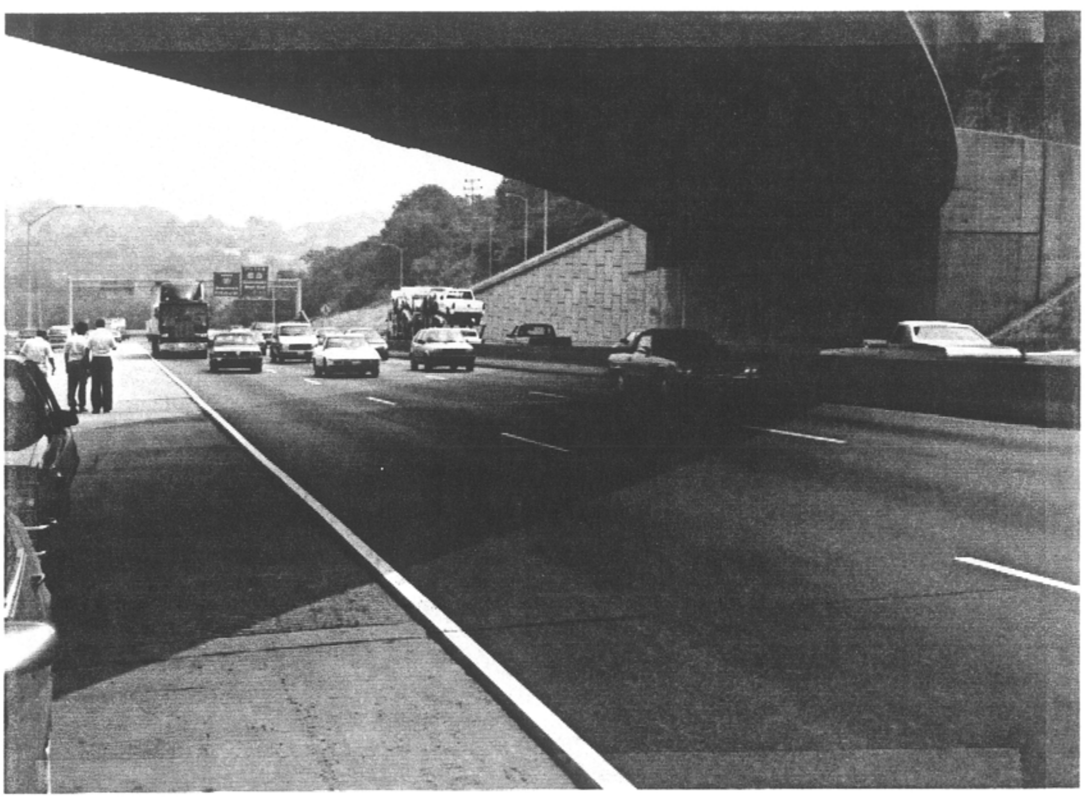

Figure 50. Project 2, Pittsburgh Parkway (W.B.)

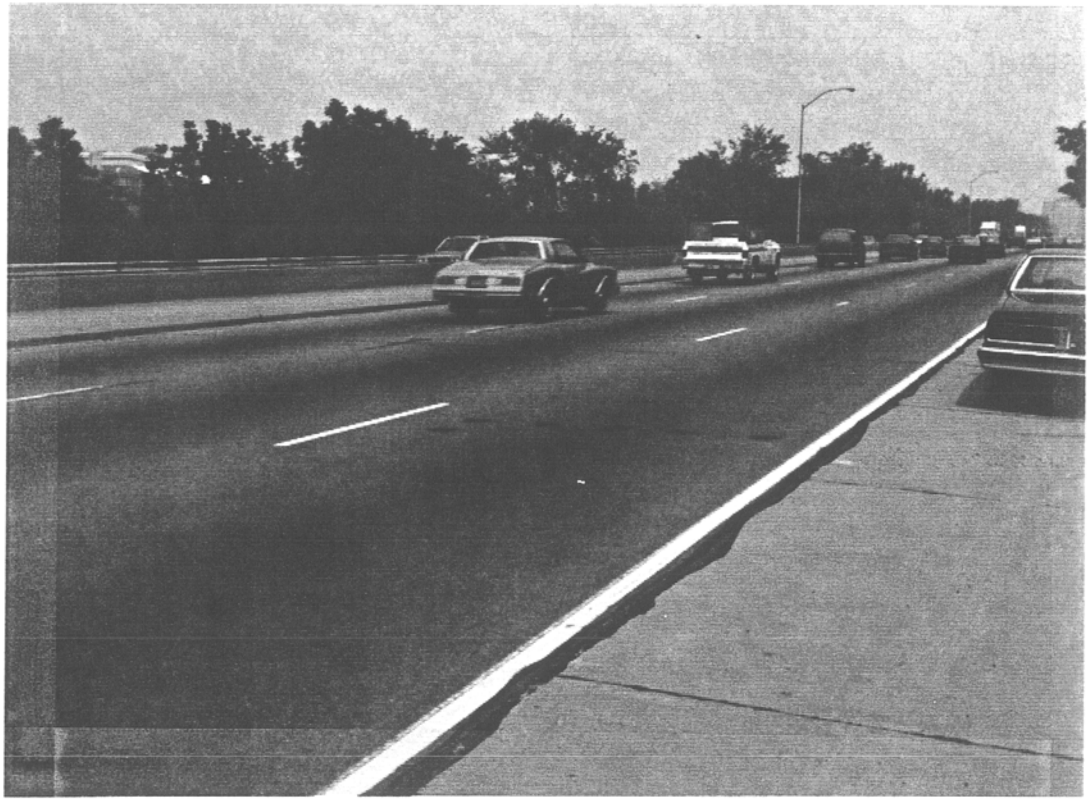

Figure 51. Project 3, Pittsburgh Parkway (E.B.)

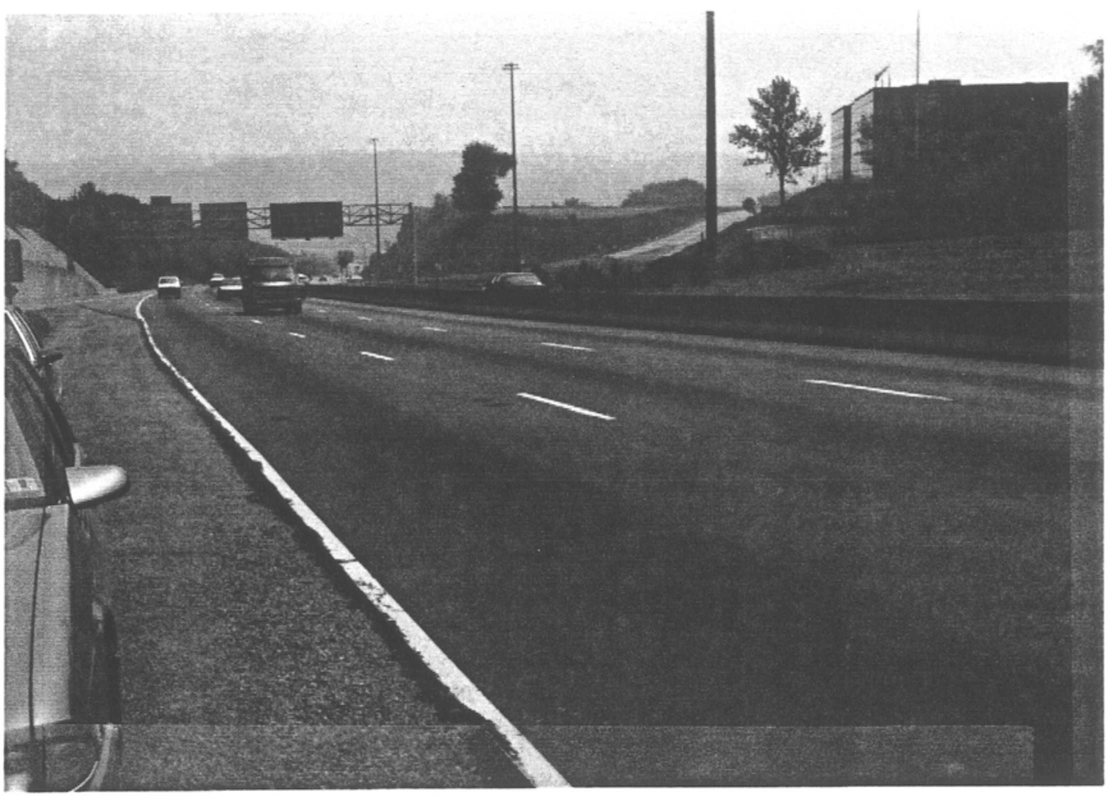

Figure 52. Project 4, Pittsburgh Parkway (W.B.)

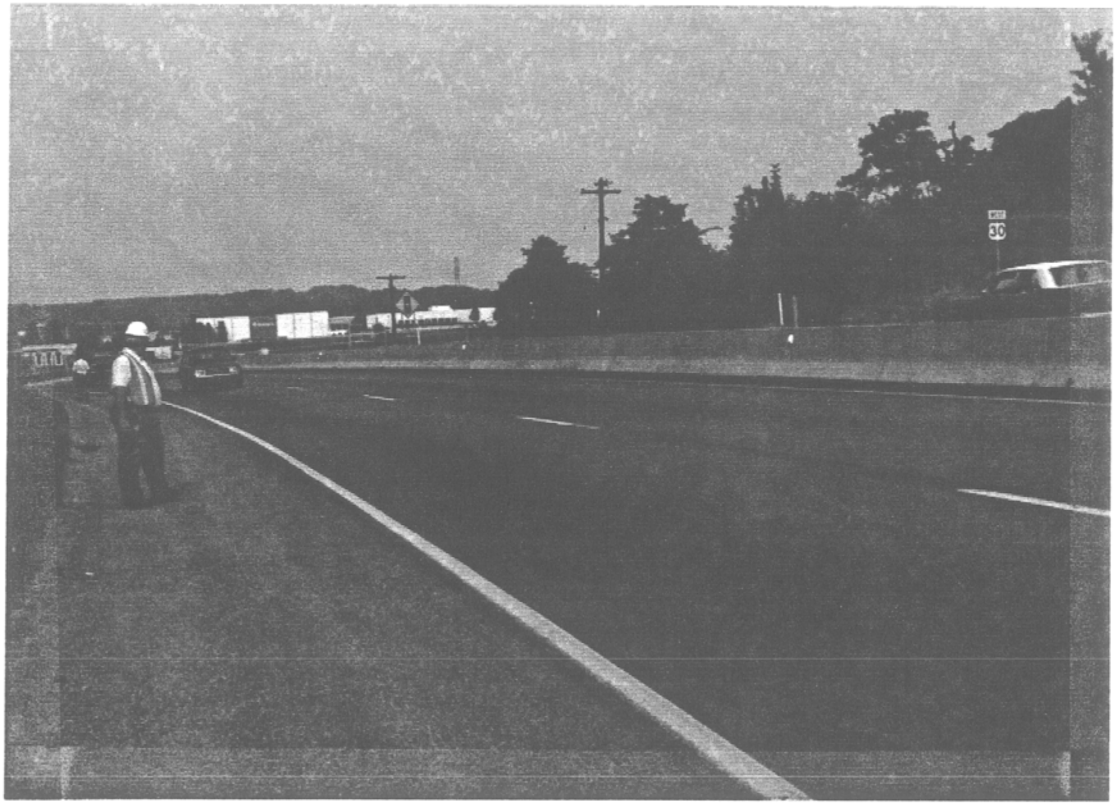

Figure 53. Project 5, Route 30 (E.B.)

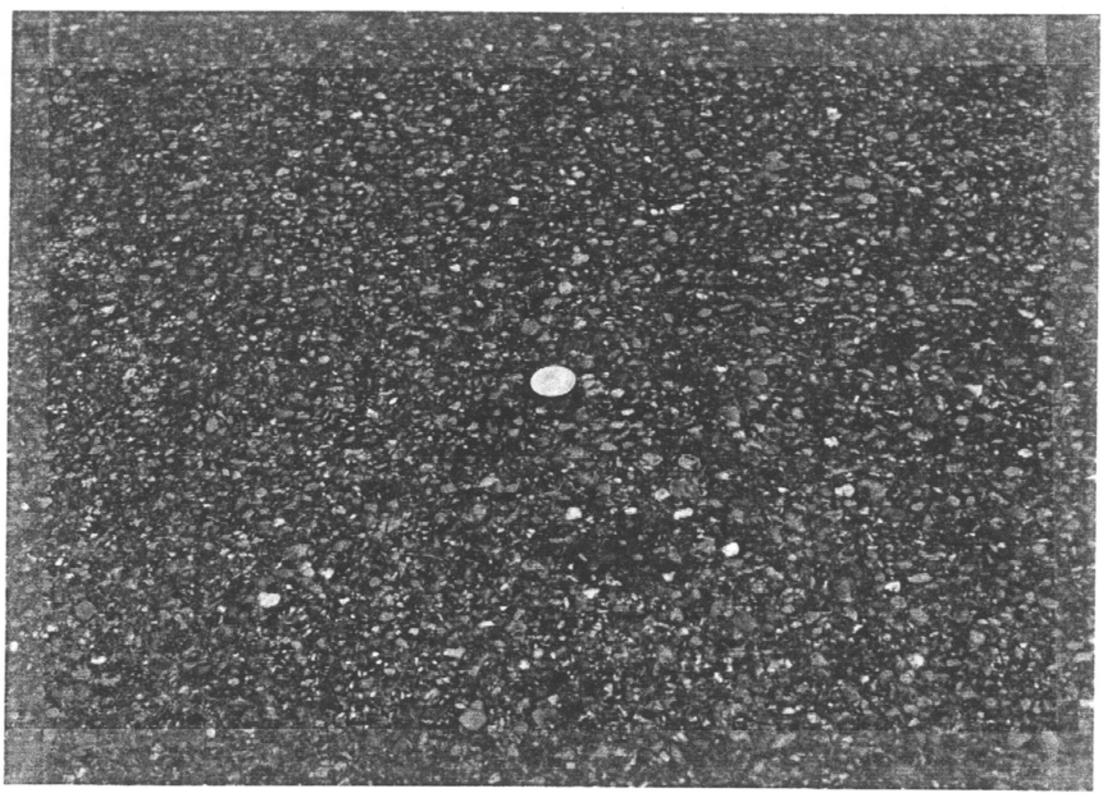

Figure 54. Project 5, Route 30 (E.B.), Closeup of ID-3 Texture

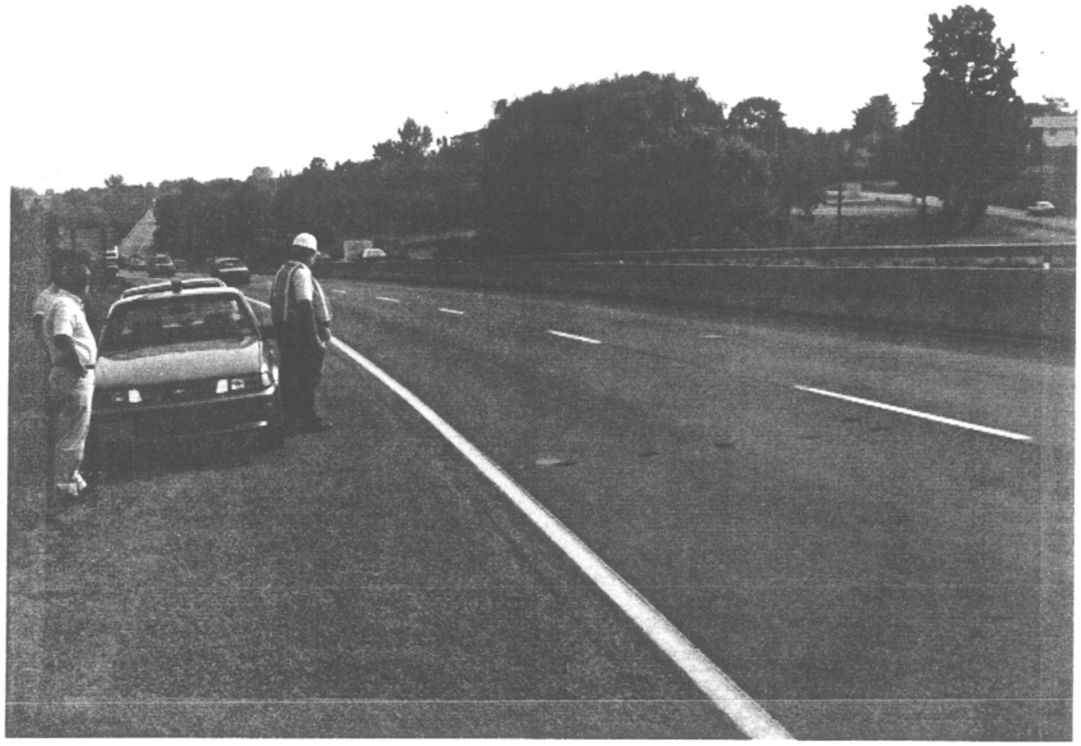

Figure 55. Project 6, Route 30 (W.B.)

Figure 56. Project 7, I-70 (E.B.)

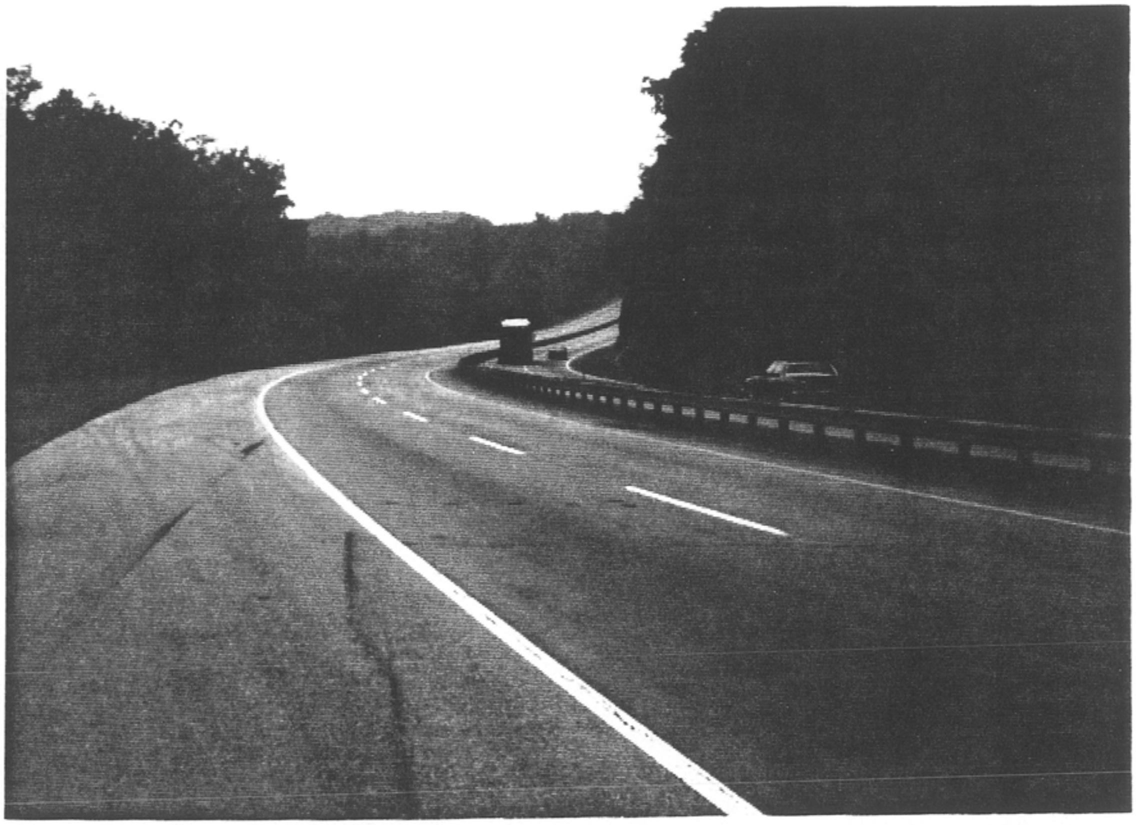

Figure 57. Project 9, Pennsylvania Turnpike (Bedford County)

Figure 58. Project 11, I-90 (W.B.) Near N.Y. Border

Figure 59. Project 12, I-81 (N.B.) Luzerne County

Figure 60. Project 13, I-81 (S.B.) Luzerne County

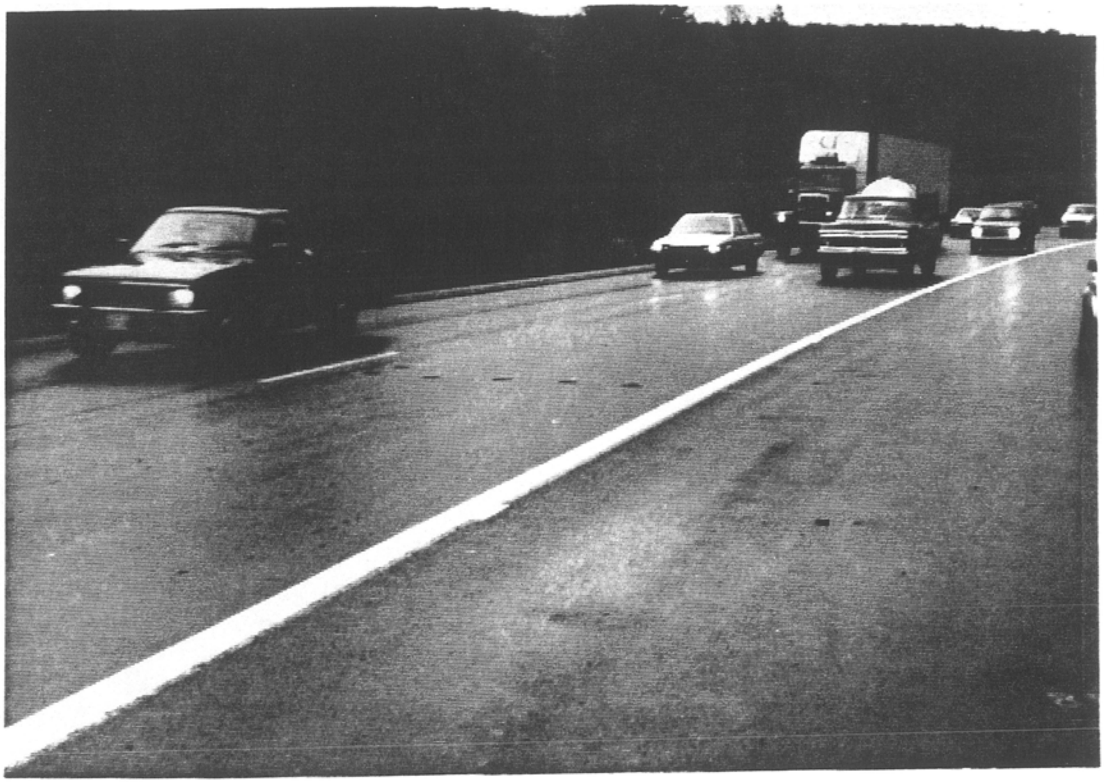

Figure 61. Project 14, I-81 (S.B.) Lackawana County

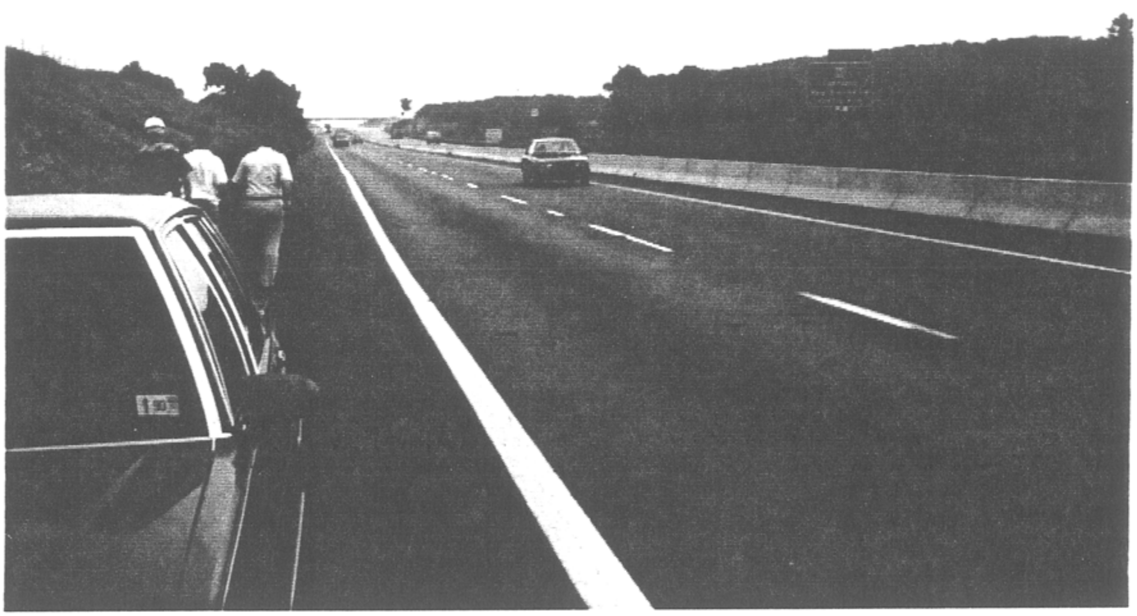

Figure 62. Project 15, I-78 (W.B.) Lehigh County

Figure 63. Project 16, I-78 (E.B.) Lehigh County

Figure 64. Project 17, I-81 (S.B.) Schuyikill County

Figure 65. Project 18, I-81 (N.B.) Schuylkill County

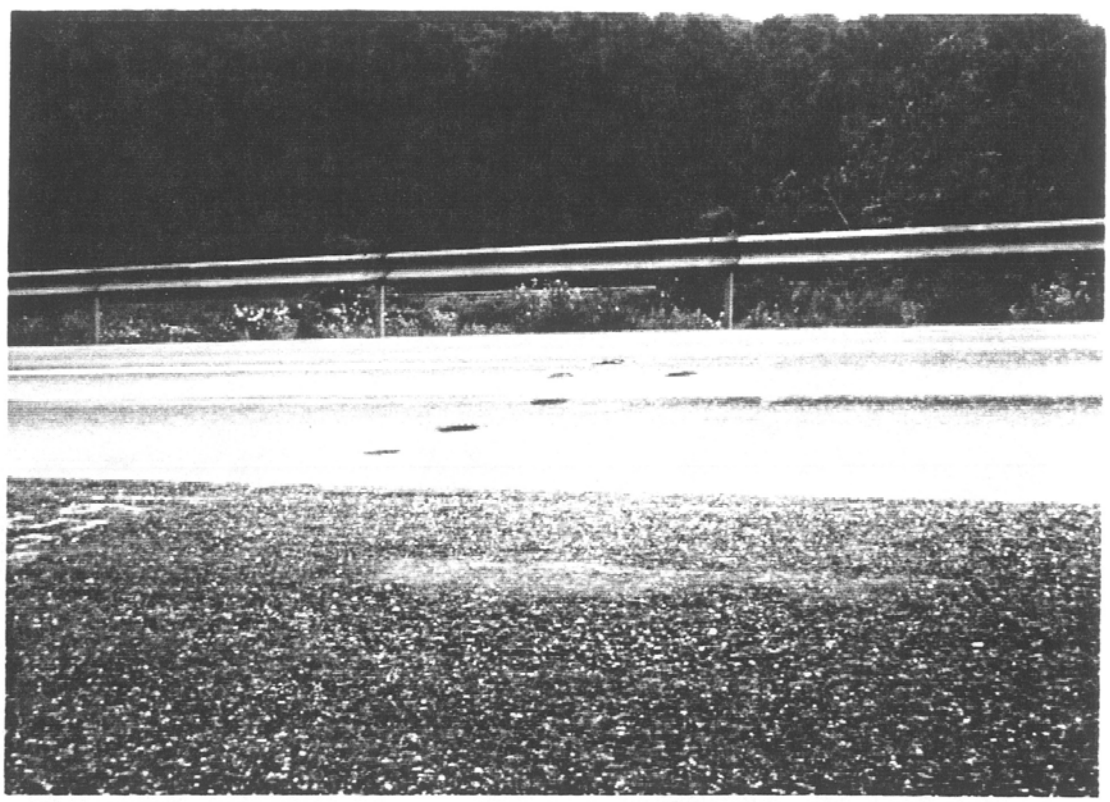

Figure 66. Project 18, I-81 (N.B.) Schuyikill County; View from Low Angle Showing Rutting

Figure 67. Project 20, Route 222, Berks County

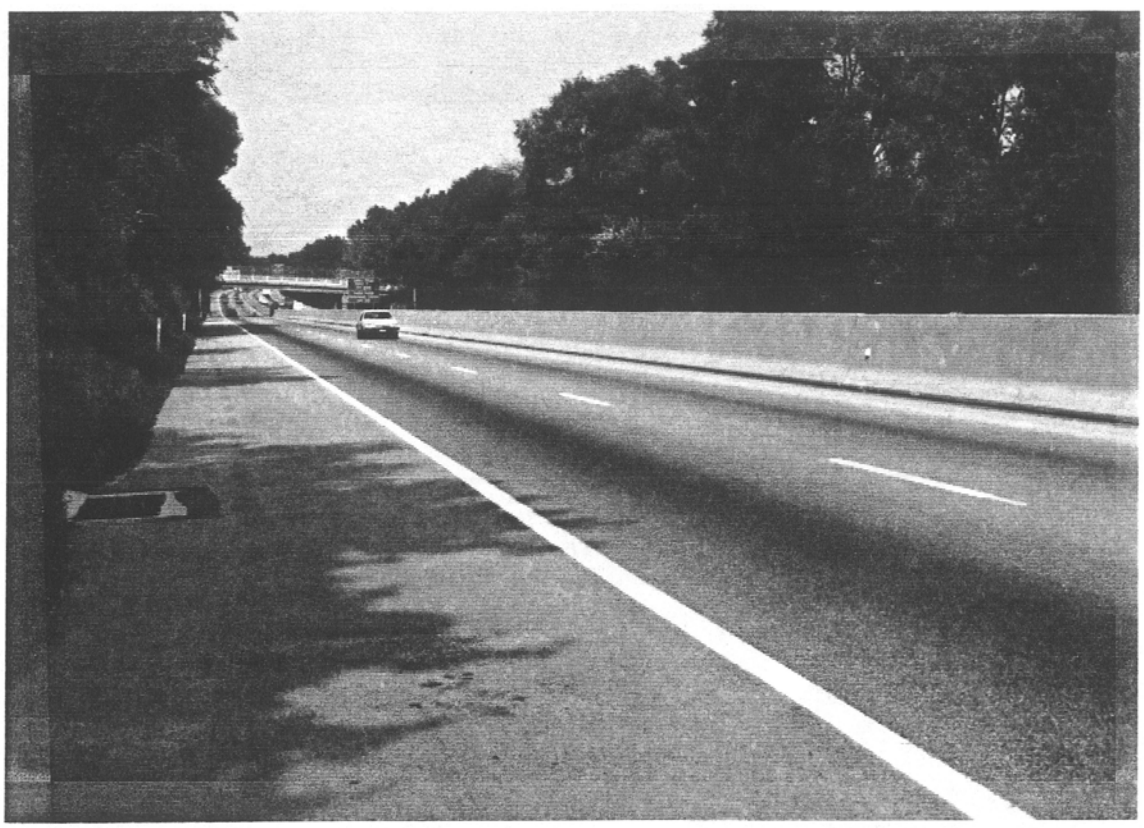

Figure 68. Project 22, Schuylkill Expressway (Section 100)

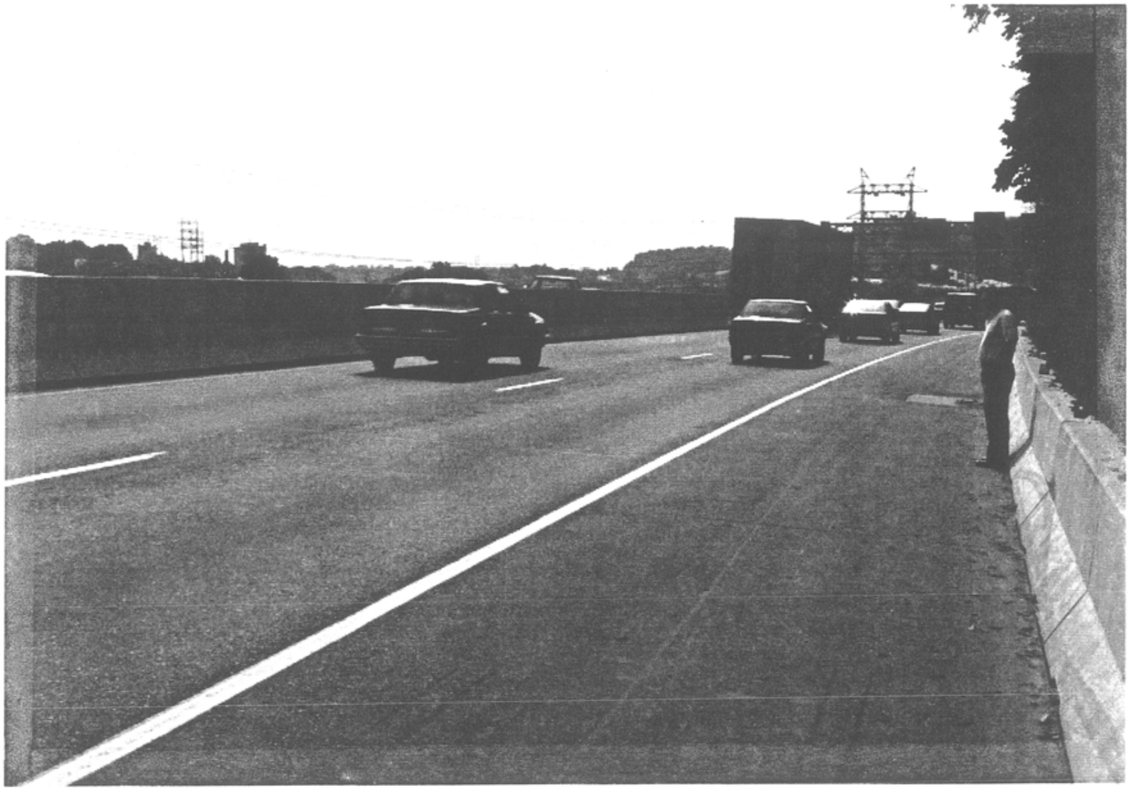

Figure 69. Project 23, Schuylkill Expressway (Section 300)

Figure 70. Project 24, Schuylkill Expressway (Section 420)

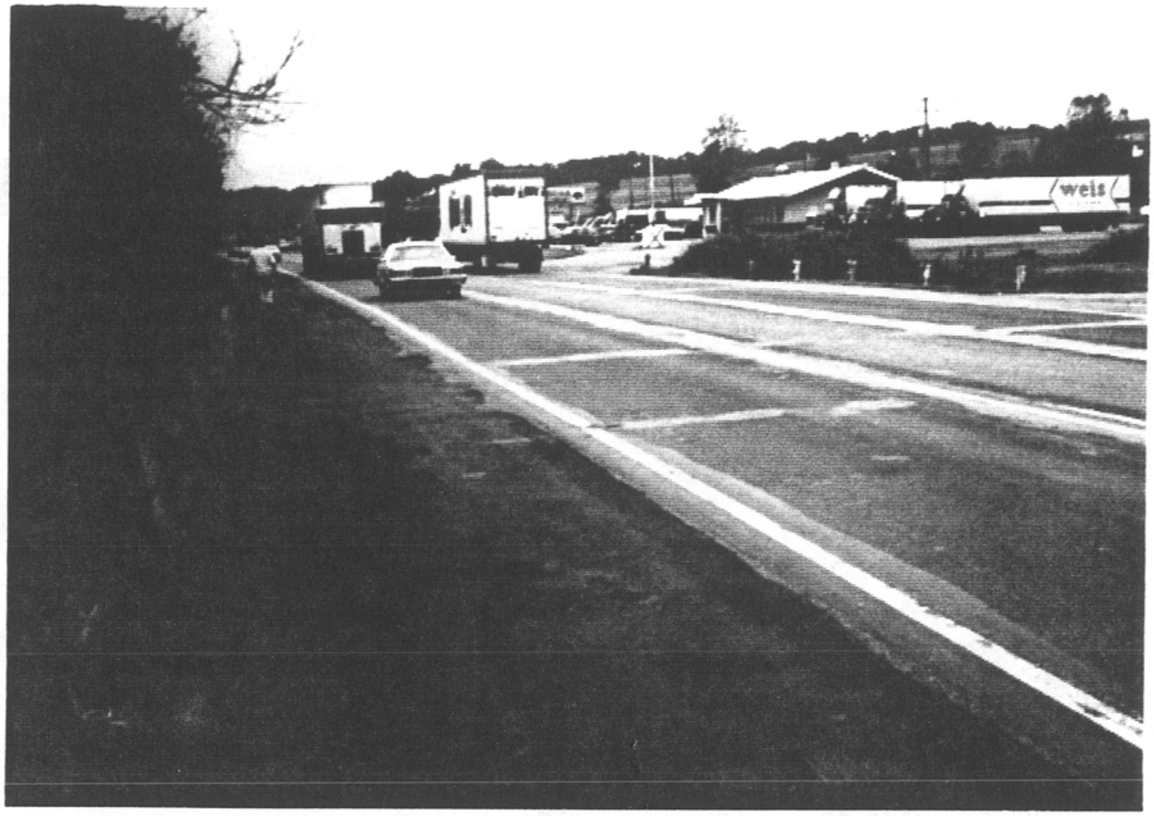

Figure 71. Project 25, Route 11, Perry County

Figure 72. Project 26, I-83, near M.P. 5, York County

Figure 73. Project 27, I-83 (S.B.) York County

Figure 74. Project 28, Route 22, Dauphin Boro

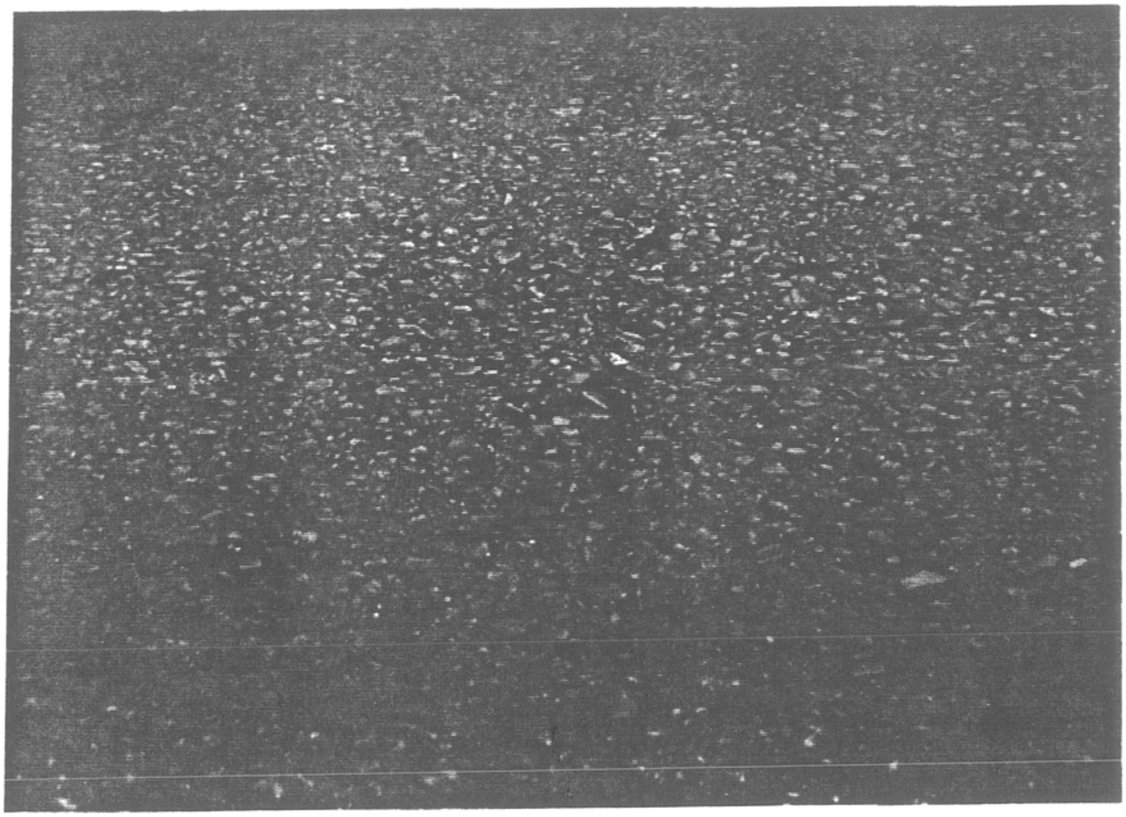

Figure 75. Project 28, Route 22, Dauphin Boro (Close-up of ID-3 Surface)

Figure 76. Project 29, Route 11, Camp Hill By-Pass

Figure 77. Project 30, Route 11 (Between Mechanicsburg and Carlisle)

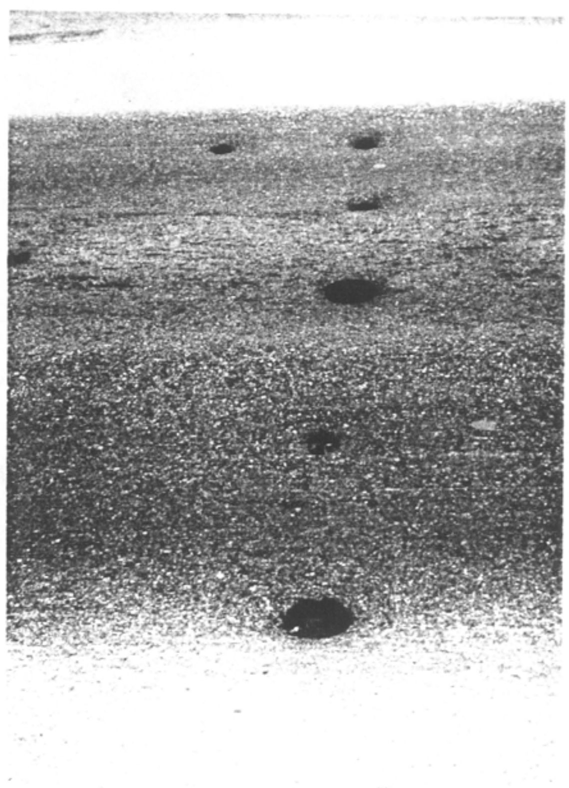

Figure 78. Project 30, Route 11 (Showing Core Holes Closing from Plastic Flow)

Figure 79. Project 31, Route 220, Blair County

Figure 80. Project 32, I-70 (E.B.), Fulton County

Figure 81. Project 33, I-70 (W.B.), Fulton County

Figure 82. Project 34, I-70, Bedford County

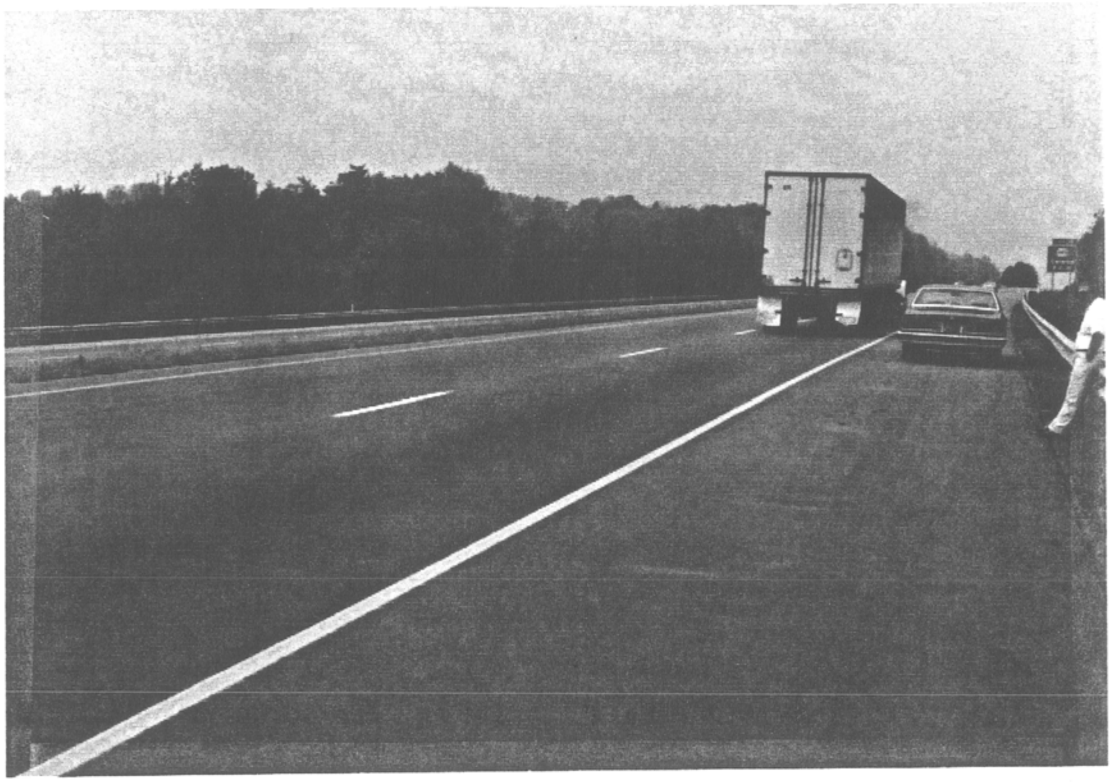

Figure 83. Project 35, I-80 (W.B.), Jefferson County

STATISTICAL ANALYSIS OF DATA AND DISCUSSION OF RESULTS

Independent Variables

Five broad categories of 60 independent variables covering the general design, construction, and post construction data for each pavement, were selected to determine the effect these variables might have on rutting. The five categories with the number of independent variables in parenthesis are:

1. General variables (2);
2. Mix design variables (10);
3. Construction variables (9);
4. Post construction longitudinal variables (16); and
5. Post construction transverse variables (23).

A brief discussion of these independent variables and their anticipated effect on rutting is given below.

General variables. The two general independent variables selected for study were average yearly temperature and total traffic loadings in 18 kip ESALs. It was anticipated that both factors would have the same general effect on rutting with an increase in either leading to an increase in rut depth.

Mix design variables. The ten mix design variables investigated included the mix composition (asphalt content and the percent passing No. 8 and No. 200 sieves) and the Marshall mix design properties of VTM, VMA, stability, flow, and the number of blows per side utilized during laboratory compaction. In addition, the relationship between stability and flow as expressed by the stability/flow ratio, and the bearing capacity was also investigated. The bearing capacity of the HMA mix was determined by the formula developed by Metcalf (7) as follows:

$$Bearing\ \ Capacity\ \ (\psi)\ -\ \frac{Stability}{Flow}\times\frac{120-Flow}{100}$$

It was anticipated that low VTM, low stability, low stability/flow ratio, low bearing capacity, and low blows per side would generally lead to increased rutting. High flow and high asphalt content were expected to lead to increased rutting. Very high or very low VMA were also expected to lead to increased rutting.

Construction variables. Nine construction variables were selected for review. They are VTM, asphalt content (AC), the percent of the material passing the 1/2", No. 8 and No. 200 sieves, and the conformal index (C1) for the AC and percent passing the 1/2", No. 8 and No. 200 sieves. It was anticipated that lower VTM, higher percent passing the 1/2", No. 8 and No. 200 sieves (finer mix), and higher AC would result in increased rutting. The conformal indexes were investigated to determine the effect of quality control on performance.

Table 7B gives the construction season and the construction traffic condition for each site. Very little data was available of the maximum and minimum daily temperatures during construction. Therefore, the paving dates were utilized in an attempt to relate weather to pavement performance. The construction dates are by season with construction during the months of April through May being spring, May through August being summer and August through October being fall. Only 13 of 34 pavements had information on traffic control during construction. If there was no channelized traffic on the fresh mat the pavements were classified as none. If construction traffic was switched from the passing lane to the traffic lane or vice versa then the pavement was classified as one-way traffic. If the traffic in both lanes in one direction were moved to the other lanes with two-way traffic then the pavements were classified as two-way.

Post construction longitudinal variables. These sixteen variables were obtained from cores C1-C8 sampled longitudinally and were selected to represent the average properties of the pavement over the section investigated. The variables investigated were VTM (average, minimum and lower 20th percentile), asphalt content (AC), the percent of the material passing the 1/2", No. 8 and No. 200 sieves, the recovered asphalt penetration and viscosity, the percent crushed particles in the coarse aggregate, the percent natural sand in the fine aggregate, and the percent voids in the fine aggregate (indicator of particle shape and texture). The conformal index for the AC and the percent passing the 1/2", No. 8 and No. 200 sieves were also investigated. It was anticipated that percent natural higher percent crushed particles, higher voids in the fine aggregate and lower sand in the fine aggregate would result in decreased rutting. It was also expected that the harder the recovered asphalt cement the lower the incidence of rutting. The other variables were expected to have the same influence as previously stated.

Post construction transverse variables. In general, the difference in the transverse samples and the longitudinal samples are the location where the samples were obtained. The transverse samples (cores C7-C11) were obtained across the traffic lane at the location with the severest rutting in the test section. After the voids analysis (VTM) and creep test, the transverse cores were subjected to a recompaction analysis where the cores were recompacted to give an indication of the original mix properties of laboratory compacted samples. The samples were recompacted using three compactive efforts. The three compactive efforts were 300 revolutions, one degree gyration angle and 120 psi pressure on the GTM, and 75 blows per side using two different automatic Marshall hammers. One Marshall hammer utilized a static base and the other a rotating base with a slanted foot compaction hammer. The recompacted variables investigated were VTM, VMA, stability, flow, GSI, stability/flow ratio and bearing capacity. It was expected that high creep strain and high GSI would indicate an increased incidence of rutting. The other properties would behave as previously discussed.

Dependent Variables

The dependent variable selected for analysis was rut depth. The difficulty encountered centered on selecting the best method to express rut depth. From the rut depth and core measurements obtained, three distinct possibilities existed for expressing the rut depth. The three possibilities were the maximum rut depth at the surface, the rut depth occurring in each layer and the average surface rut depth of the test section. The maximum surface rut depth and the rut depth in each layer were available at the location of cores C7-C11 (worst location). The sum of the rut depths in each layer adds to the maximum rut depth at the surface. The average rut depth at the surface was obtained within 500 feet of cores C7-C11. Layer thickness measurements in this vicinity were not available.

It is a well established fact that traffic affects rutting in pavements. The total estimated traffic experienced by the pavements in this study ranged from less than 1 million ESALs to over 30 million ESALs. By dividing the rut depth by some funtilon of traffic the pavements could be normalized to a rate of rutting and two pavements with 1/2 inch ruts of differing age (for example) could be compared based on this rate of rutting. Figure 84 shows the generally accepted model of the relationship between rut depth and traffic for a given pavement (8). The initial densification for a rutted pavement follows a direct relationship with traffic, however,

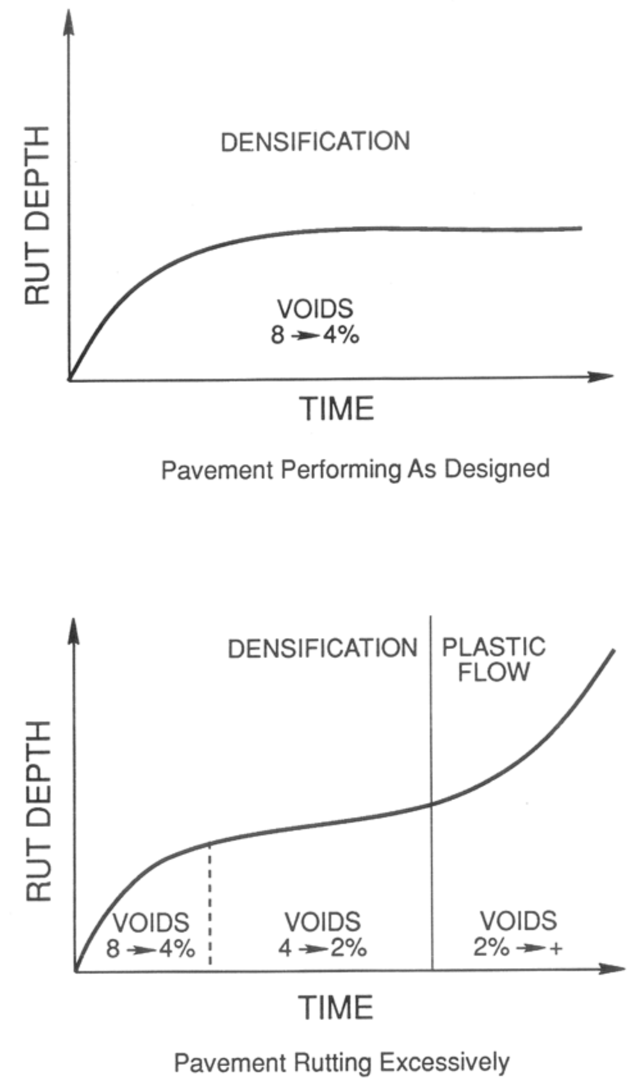

Figure 84. General Model for Pavement Rutting

after initial densification the rate of rutting decreases with an increase in traffic until a condition of plastic flow occurs and the rate of rutting again increases. Previous work at NCAT (8, 9), has shown that expressing the rate of rutting as a function of the square root of total traffic better models pavement behavior when compared to other expressions for the rate of rutting.

Six different methods of expressing the dependent variable of rut depth were initially utilized in the preliminary analysis. The six different methods utilized were: maximum rut depth at the surface (worst location), the rut depth in each layer (worst location), the average maximumsurface rut depth (within 500 feet of the worst site), and each of the above variables divided by the square root of the total traffic expressed as million 18-kip ESALs (TESALs). In general, the average maximum surface rut depth divided by the square root of traffic gave the best correlations with the majority of the independent variables investigated. Figure 85 is a histogram showing all sites with increasing magnitude of this parameter (average maximum rut depth divided by the square root of traffic). The sites have been labeled E (excellent), G (good), F (fair) and P (poor) based on the subjective performance rating discussed earlier. It can be seen that a value of 0.2 for this parameter generally divides E and G sites from F and P sites. Therefore, this value of 0.2 can reasonably be considered as a threshold value above which pavements are expected to develop undesirable amount of rutting. This value also agrees with similar values established by Parker and Brown for Alabama highways (8). The average maximum surface rut depth (within 500 feet of the worst location) divided by the square root of traffic was utilized as the dependent variable for statistical analysis for the general, mix design, construction, and post construction longitudinal independent variables. The maximum surface rut depth (worst location) divided by the square root of traffic (obtained at cores C7-C11) was utilized as the dependent variable for the post construction transverse independent variables.

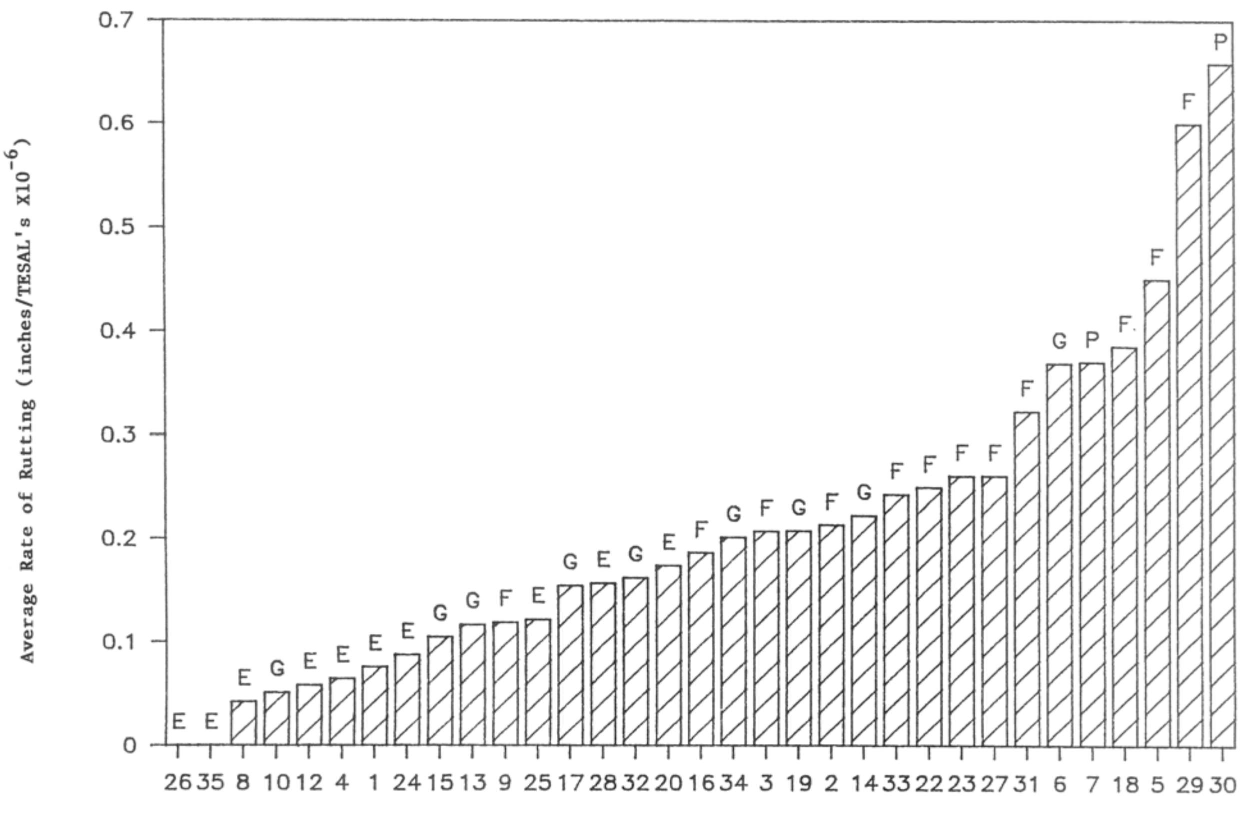

Figure 85. Histogram of Pavement Rating and Rate of Rutting

Ideally, the best correlation of the independent variables pertaining to a specific layer should be obtained with the rut depth in that layer and not with the surface rut depth which can be affected by other layer(s). However, this was not generally the case. One possible reason is that the rut depths in individual layers could not be measured accurately because of (a) imperfections at the points where cores C7-C11 were taken, and (b) cores were taken at 2-foot intervals.

Regression Analysis

All of the data available pertaining to the independent and dependent variables were input into a data base and analyzed using the software package SAS by the SAS Institute Inc., Cary N.C.

The data was analyzed using correlation analysis, linear regression analysis methods and stepwise multiple variable analysis methods. The objective was to identify the independent variables which significantly affect rutting, and to establish their threshold values, if possible. The results of the analysis for the different categories of independent variables are discussed below.

Tables 15 through 18 show the results of the correlation analysis performed on the five categories of independent variables. The tables give Pearson's correlation coefficient (R) in the second column for the dependent variable average surface rut depth/square root traffic. Column 3 gives the R value for the other dependent variables (listed in Column 4) giving the best or highest correlation coefficient. An R value of 1.00 would mean the variables are perfectly correlated and an R value of 0.00 would mean no relationship exists between the two variables. A positive number indicates a direct relationship and a negative number an inverse relationship. The first observation that can be made about the data is the poor correlation between the individual independent variables and rutting. Rutting is a complex phenomenon and it is doubtful that any one independent variable alone could predict rutting with any degree of confidence. Pavement material characteristics for each layer were correlated to either the total rut depth at the surface or to a rate of rutting occurring at the surface. All of the rut depth appearing at the surface can not be explained by the material properties of a single pavement layer. Developing a model to combine the material properties from each layer to predict the total rut depth at the surface was outside the scope of this project. Moreover, the dependent variable (rut depth) was measured at two locations only on each project.

Figure 86 shows the average percent of the total rut depth occurring in each layer for the two, three and four layer pavements investigated. The results show than on average only 60 percent of the total rut observed can be attributed or explained by the properties of the surface mix and only 40 percent by the properties of the binder mix. These averages are approximate, however, this fact alone would lower the R-values obtainable to below a level that is generally regarded as significant even if the properties of each layer completely explained the rutting occurring in that layer.

Additionally, within each layer one bad property (for example, excessive asphalt content) can nullify other good properties (such as, 0 percent natural sand and 100 percent crush content). There are also numerous interactions between the properties.

In general, the correlation coefficients are too low to be of much use. However, from the plots of the data, some general trends can be observed and threshold values identified. The significant correlations (*R* $ 0.5) from the correlation analysis for the independent variables will be briefly discussed below.

Table 15. Summary of Correlation Analysis for General and Mix Design Variables

Independent Variable	Correlation with Rut Depth/SQRT TESALs R Value	Best Correlation	
		R Value	Dependent Variable
		Layer 1	
Temperature	0.1576	0.3585	Rut in Layer/SQRT TESALs
ESALs	-0.2762		
VTM	0.0415 *	0.3984 *	Rut in Layer/SQRT TESALs
VMA	-0.2308	0.3162 *	Rut in Layer
Stability	-0.3167	0.3992 *	Max Surface/SQRT TESALs
Flow	-0.1384 *	-0.1370 *	Max Surface/SQRT TESALs
Stability/Flow	-0.2127 *	-0.2945 *	Max Surface/SQRT TESALs
Bearing Capacity	-0.1996 *	-0.2797 *	Max Surface/SQRT TESALs
# Blows Per Side	-0.1916		
Passing #8 Sieve	-0.4905		
Passing #200 Sieve	0.0973		
Asphalt Content	-0.2979		
		Layer 2	
Temperature	0.1576	0.2301	Max Surface Rut Depth
ESALs	-0.2762		
VTM	0.0727	0.0921 *	Max Surface/SQRT TESALs
VMA	0.0949		
Stability	-0.1941	-0.2662	Max Surface Rut Depth
Flow	-0.3746	-0.4226 *	Avg Surface Rut Depth
Stability/Flow	0.0370	-0.1584	Max Surface Rut Depth
Bearing Capacity	0.0573	-0.1454	Max Surface Rut Depth
# Blows Per Side	-0.0287	-0.2344	Rut in Layer
Passing #8 Sieve	-0.1922		
Passing #200 Sieve	-0.1142		
Asphalt Content	0.0313		

* = Reverse Relationship

Table 16. Summary of Correlation Analysis for Construction Variables

Independent Variable	Correlation with Rut Depth/SQRT TESALs R Value	Best Correlation	
		R Value	Dependent Variable
		Layer 1	
Voids Total Mix	-0.3207		
Asphalt Content	-0.2289 *	-0.2519 *	Max Surface/SQRT TESALs
Passing #8 Sieve	-0.5258		
Passing #200 Sieve	-0.1231 *	-0.1817 *	Max Surface/SQRT TESALs
Passing 1/2" Sieve	N/A	N/A	
Asphalt Content CI	0.3953	0.6268	Max Surface/SQRT TESALs
Passing #8 Sieve CI	-0.2249	-0.2731	Avg Surface Rut Depth
Passing #200 Sieve CI	0.0187	0.2741	Avg Surface Rut Depth
Passing 1/2" Sieve CI	N/A	N/A	
		Layer 2	
Voids Total Mix	-0.1988	-0.3110	Avg Surface Rut Depth
Asphalt Content	0.2300		
Passing #8 Sieve	-0.1245		
Passing #200 Sieve	-0.2096 *	-0.2573 *	Rut in Layer/SQRT TESALs
Passing 1/2" Sieve	0.1082	0.2320	Avg Surface Rut Depth
Asphalt Content CI	-0.0321 *	-0.3174 *	Rut in Layer
Passing #8 Sieve CI	-0.2390	-0.2986	Max Surface/SQRT TESALs
Passing #200 Sieve CI	-0.1067 *	-0.2524 *	Max Surface/SQRT TESALs
Passing 1/2" Sieve CI	0.1169	-0.3332 *	Rut in Layer/SQRT TESALs

* = Reverse Relationship

Table 17. Summary of Correlation Analysis for Post Construction Longitudinal Variables (C1-C6)

Independent Variable	Correlation with Rut Depth/SQRT TESALs R Value	Best Correlation R Value	Best Correlation Dependent Variable
Layer 1			
Asphalt Content	-0.2434	-0.3184	Rut in Layer/ SQRT TESALs
Asphalt Content CI	0.3193		
Passing 1/2" Sieve	N/A	N/A	
Passing 1/2" Sieve CI	N/A	N/A	
Passing #8 Sieve	-0.6039	-0.6812	Avg Surface Rut Depth
Passing #8 Sieve CI	-0.4792		
Passing #200 Sieve	0.1556	0.1865	Rut in Layer/SQRT TESALs
Passing #200 Sieve CI	0.1030	0.3542	Rut in Layer
Penetration	0.3197	0.3537	Avg Surface Rut Depth
Viscosity	-0.1497	-0.1686	Avg Surface Rut Depth
% Crushed Faces	-0.1500	0.1613 *	Rut in Layer/SQRT TESALs
% Nat’l Sand	-0.1276 *	-0.3976 *	Rut in Layer/SQRT TESALs
Voids in Fine Agg	0.1448 *		
Avg VTM	-0.1856	-0.3817	Avg Surface Rut Depth
Min VTM	-0.1710	-0.3544	Avg Surface Rut Depth
20th Pct’l VTM	-0.2133	-0.3998	Avg Surface Rut-Depth
Layer 2			
Asphalt Content	0.6195		
Asphalt Content CI	0.1416	0.1858	Rut in Layer/sqrt Tesal's
Passing 1/2" Sieve	0.2806		
Passing 1/2" Sieve CI	0.1590	0.3139	Rut in Layer
Passing #8 Sieve	0.1289	0.1344	Avg Surface Rut Depth
Passing #8 Sieve CI	-0.0250	0.1428	Rut in Layer
Passing #200 Sieve	0.0783	-0.1937 *	Rut in Layer/SQRT TESALS
Passing #200 Sieve CI	0.0041	-0.0720 *	Rut in Layer/SQRT TESALs
Penetration	0.0060	0.1465	Max Surface Rut Depth
Viscosity	0.4219 *		
% Crushed Faces	-0.1037	0.1463 *	Rut in Layer
% Nat’l Sand	-0.0549 *	-0.1689 *	Rut in Layer/SQRT TESALs
Voids in Fine Agg	-0.4507		
Avg VTM	-0.3094	-0.3505	Avg Surface Rut Depth
Min VTM	-0.3099	-0.3157	Rut in Layer
20th Pct’l VTM	-0.3421		

* = Reverse Relationship

Table 18. Summary of Correlation Analysis for Post Construction Transverse Variables (C7-C11)

Independent Variable	Correlation with Rut Depth/SQRT TESALs R Value	Best Correlation	
		R Value	Dependent Variable
		Layer 1	
Avg VTM	-0.1813	-0.4309	Avg Surface Rut Depth
Min VTM	-0.1268	-0.3638	Avg Surface Rut Depth
20th Pct'l VTM	-0.1260	-0.3568	Avg Surface Rut Depth
GTM VTM	-0.3733	-0.4457	Avg Surface Rut/SQRT TESALs
Rotating VTM	-0.1979	-0.4251	Avg Surface Rut Depth
Static VTM	-0.2072		
GTM VMA	-0.2778	-0.3150	Avg Surface Rut/SQRT TESALs
Rotating VMA	-0.1286	-0.1338	Rut in Layer
Static VMA	-0.1697	0.1710	Rut in Layer
GSI	0.4823		
Creep	-0.1856	0.2878	Rut in Layer
GTM Stability	-0.3371	-0.3742	Max Surface Rut/SQRT TESALs
GTM Flow	0.5134	0.5736	Rut in Layer
GTM Stab/Flow	-0.4861		
GTM Bearing Capacity	-0.4859		
Static Stability	-0.5266		
Static Flow	0.3928	0.4959	Rut in Layer
Static Stab/flow	-0.5221		
Static Bearing Cap	-0.5108		
Rotating Stability	-0.5130	-0.5361	Max Surface Rut/SQRT TESALs
Rotating Flow	0.3770	0.5509	Rut in Layer
Rotating Stab/Flow	-0.4974		
Rotating Bearing Cap	-0.4884		

* = Reverse Relationship

Table 18. Summary of Correlation Analysis for Post Construction Transverse Variables (C7-C11) (Continued)

Independent Variable	Correlation with Rut Depth/SQRT TESALs R Value	Best Correlation	
		R Value	Dependent Variable
		Layer 2	
Avg VTM	-0.3347	-0.3925	Max Surface Rut Depth
Min VTM	-0.1995	-0.2877	Max Surface Rut Depth
20th Pct'l VTM	-0.2749	-0.3504	Max Surface Rut Depth
GTM VTM	-0.4863	-0.6227	Avg Surface Rut/SQRT TESALs
Rotating VTM	-0.4569	-0.5658	Avg Surface Rut/SQRT TESALs
Static VTM	-0.3564	-0.4107	Avg Surface Rut/SQRT TESALs
Gtm VMA	0.0691 *	0.2891 *	Avg Rut Depth
Rotating VMA	0.0891 *	0.3321 *	Avg Surface Rut/SQRT TESALs
Static VMA	0.1342 *	0.3827 *	Avg Surface Rut/SQRT TESALs
GSI	0.4688		
Creep	-0.0204 *	0.0893	Rut in Layer
GTM Stability	-0.2138		
GTM Flow	0.1961	0.2299	Max Surface Rut Depth
GTM Stab/Flow	-0.2417	-0.2475	Max Surface Rut Depth
GTM Bearing Capacity	-0.2348	-0.2407	Max Surface Rut Depth
Static Stability	-0.3388		
Static Flow	-0.1858	-0.2845 *	Avg Surface Rut/SQRT TESALs
Static Stab/Flow	-0.1179	0.1557 *	Rut in Layer/SQRT TESALs
Static Bearing Cap	-0.0830	0.1642 *	Rut in Layer/SQRT TESALs
Rotating Stability	-0.2788	-0.3053	Max Surface Rut Depth
Rotating Flow	0.1759	0.1823	Max Surface Rut/SQRT TESALs
Rotating Stab/Flow	-0.3053		
Rotating Bearing Cap	-0.2958		

* = Reverse Relationship

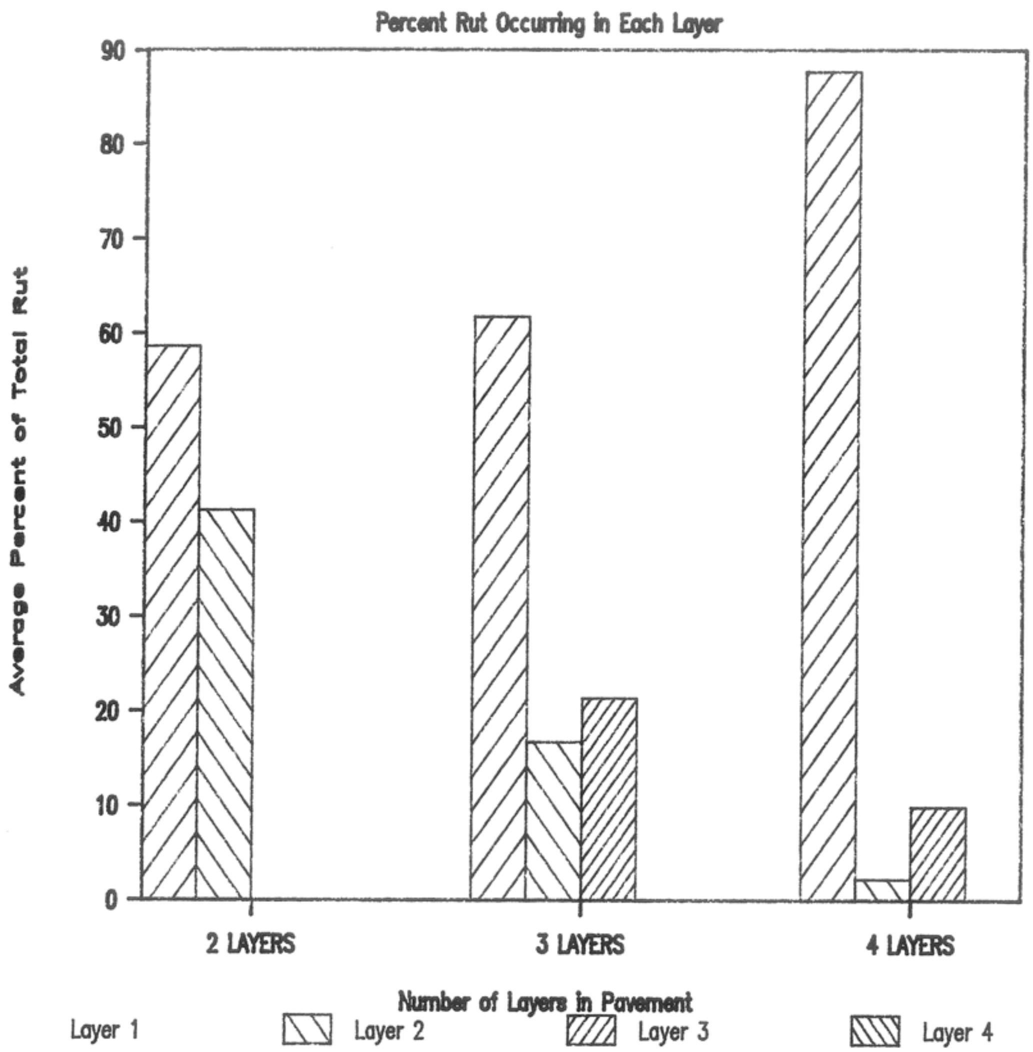

Figure 86. Percentage of the Average Total Rut Depth Occurring in Each Layer of the Pavement

General variables. The average daily temperature did not correlate well with rutting giving an R value of 0.16. The R value is positive and this indicates a slight trend for an increase in rut depth with an increase in temperature. This increase in rutting with an increase in temperature was as expected, however the small range in temperatures within Pennsylvania could help explain the poor correlation. The rate of traffic loading in ESALs proved inconclusive in predicting rutting. Some slight trend of increased rutting with an increase in traffic was expected, however this was not the case with an R value of -0.28. A trend or a good correlation with traffic would have meant that traffic alone and not mix properties controlled rutting to a great extent.

Mix design variables. The mix design properties of VTM (R= 0.04), VMA (R= -0.23), stability (R=-0.32), and flow (R=-0.14) showed either poor correlations or reverse trends with rutting. The results of the correlation analysis for all ten of the mix design variables are shown in Table 15. One likely reason for this poor correlation is the difference between the mix "as designed" and the mix "as placed" in terms of not only mix composition but also compacted density. It has been noted from previous discussions that the mixes in the field are compacted to a higher density after traffic than that produced in the laboratory. Previous work at NCAT (9) has shown this high in-place density to be a major cause of premature rutting. Obviously, differences in density will affect most of the mix design variables such as VTM, VMA, stability and flow which affect the pavement performance. Figures 87 and 88 show graphically the results of the difference in the average in-place (C1-C5) unit weight and the mix design unit weight for the wearing and binder mixes. The in-place unit weight exceeded the mix design unit weight by one lb. or more 75 percent of the time, was within one lb. ten percent of the time and less than the mix design 15 percent of the time for the wearing mixes. For the binder mixes, the in-place unit weight exceeded the mix design unit weight by one lb. or more 50 percent of the time, was within one lb. ten percent of the time and less than 40 percent of the time.

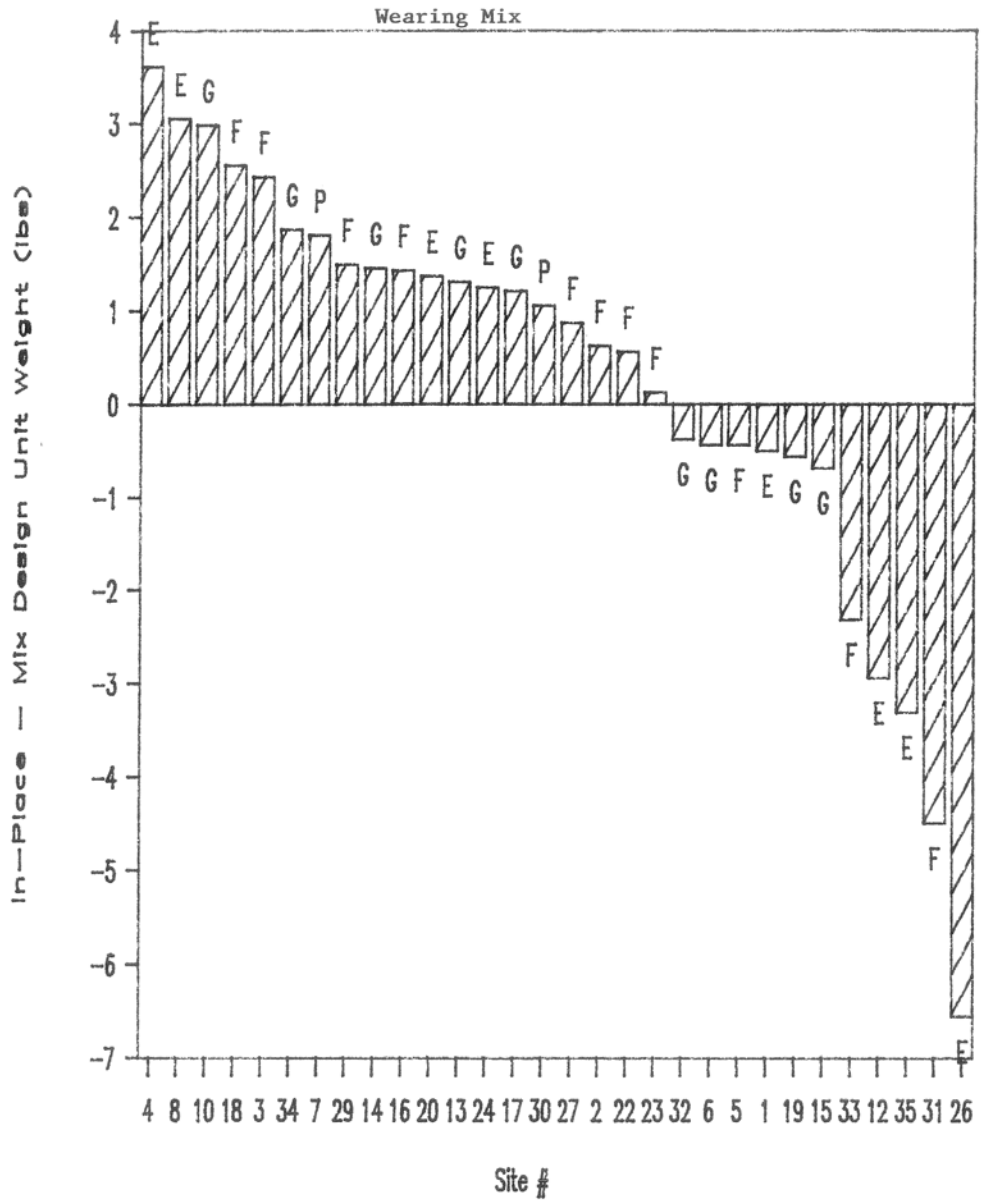

Figure 87. Comparison of In-Place and Mix Design Unit Weights, Wearing

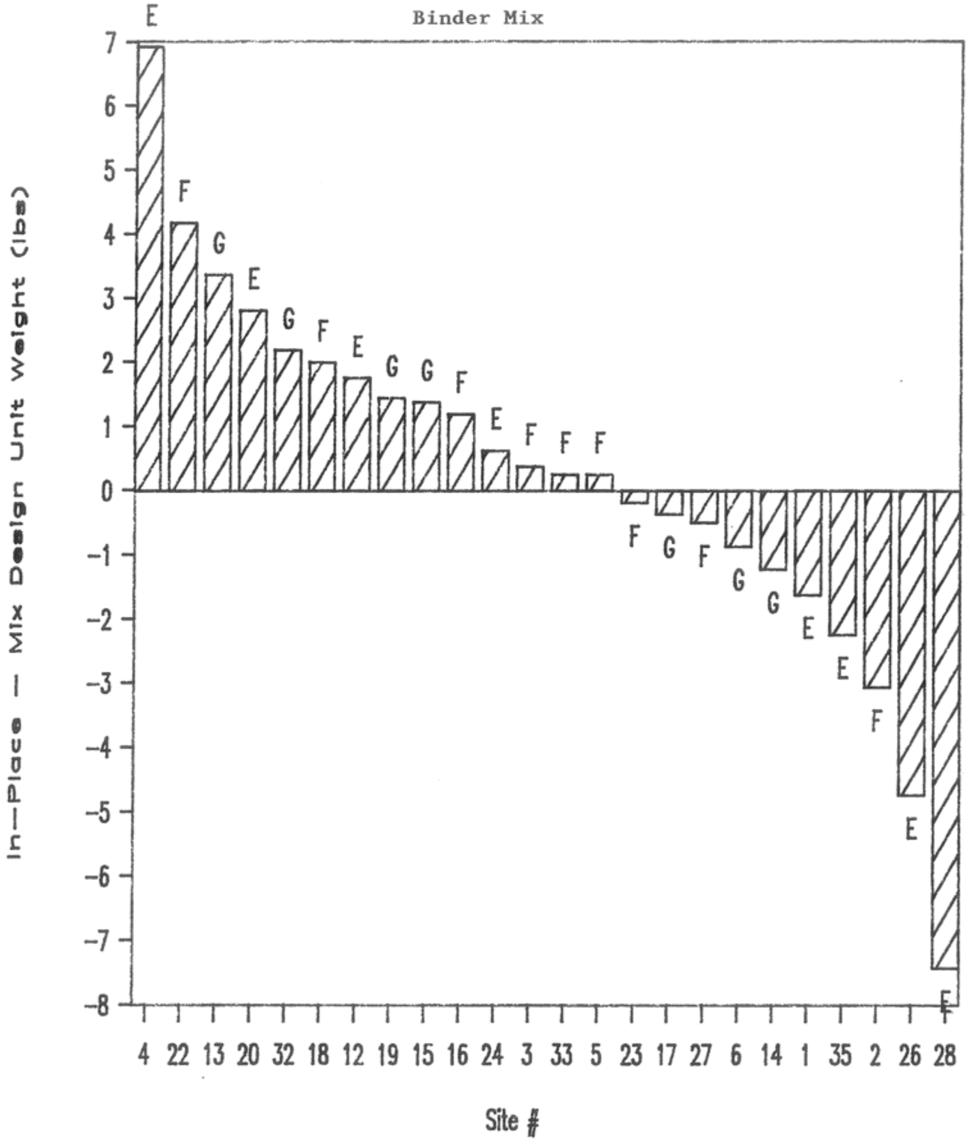

Figure 88. Comparison of In-Place and Mix Design Unit Weights, Binder

Figures 89 and 90 show the comparisons of the mix design construction, and post construction in-place (C1-C6) asphalt contents for the wearing and binder mixes, respectively. For the wearing mixes, 24 percent of the in-place asphalt contents were more than 0.4 percent below the mix design asphalt content and 76 percent were within ±0.4 percent of the mix design asphalt content. However, only two of the 33 mixes were above the mix design asphalt content and both of them were within 0.1 percent of the design asphalt content. Similar trends were noted for the construction data as shown in these two figures.

The “as placed” mixes are also finer than the “designed” mixes determined from both the in-place cores (C1-C5) and the construction data for both the wearing and binder mixes. Figures 91-94 show the difference between the mix design values and the as placed values for percent passing the No. 8 and No. 200 sieves, respectively, for the wearing and binder mixes. The results show the mixes to generally fall within the specification limits. However, only a small percentage of the mixes are coarser than the mix design value.

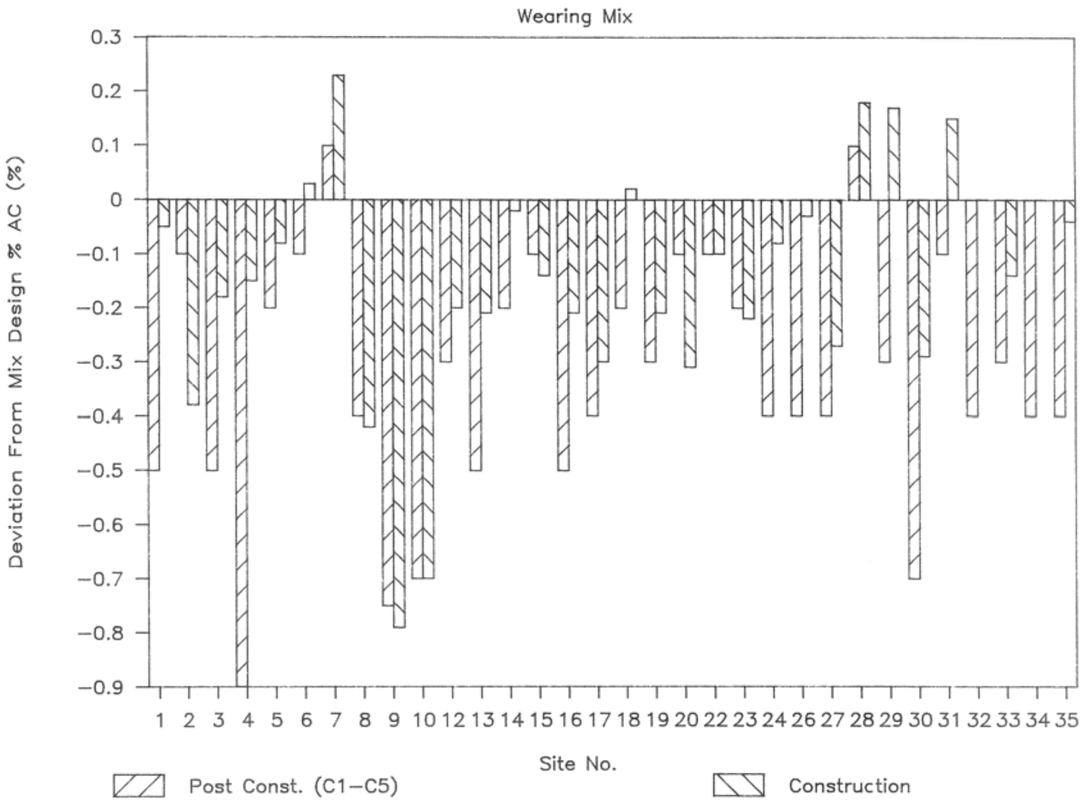

Figure 89. Comparison of In-Place and Construction Asphalt Content to Mix Design Asphalt Content, Wearing

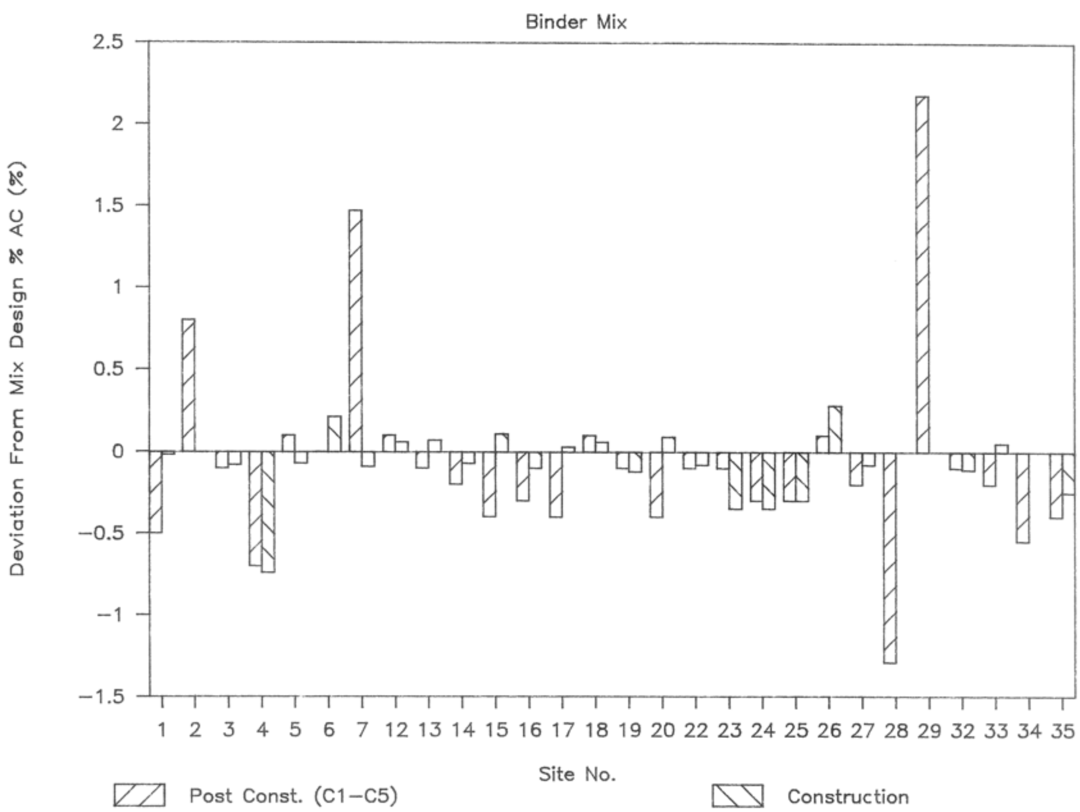

Figure 90. Comparison of In-Place and Construction Asphalt Content to Mix Design Asphalt Content, Binder

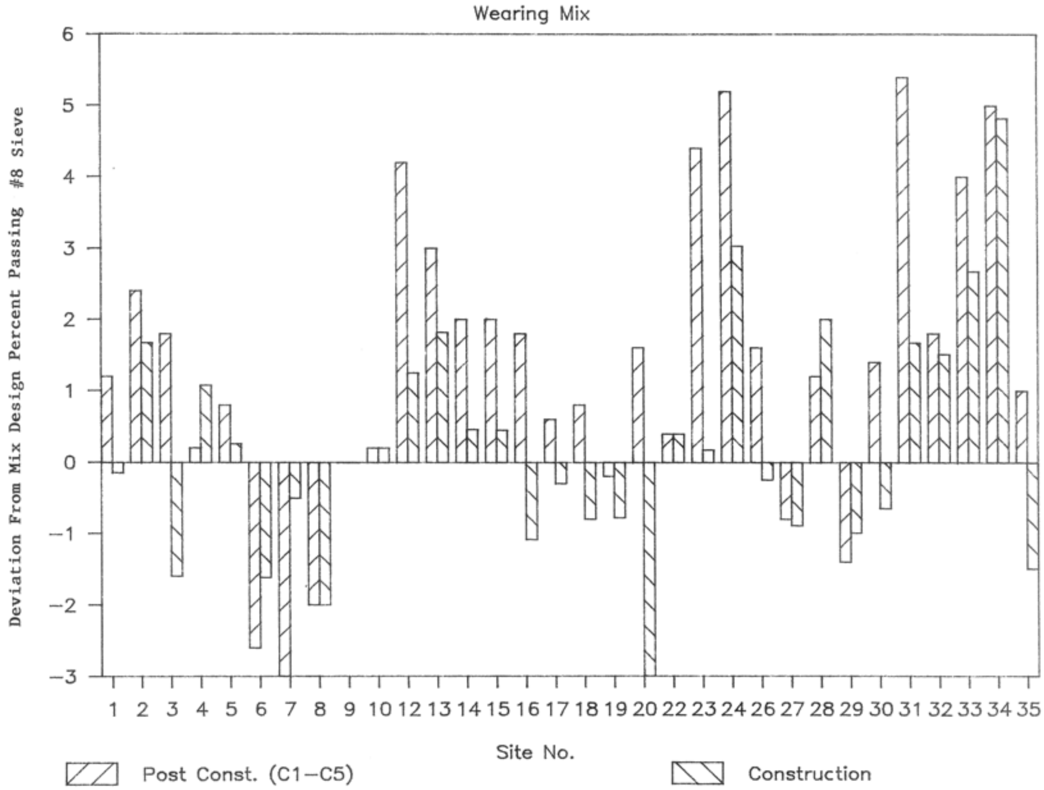

Figure 91. Comparison of In-Place and Construction to Mix Design Percent Passing No. 8 Sieve, Wearing

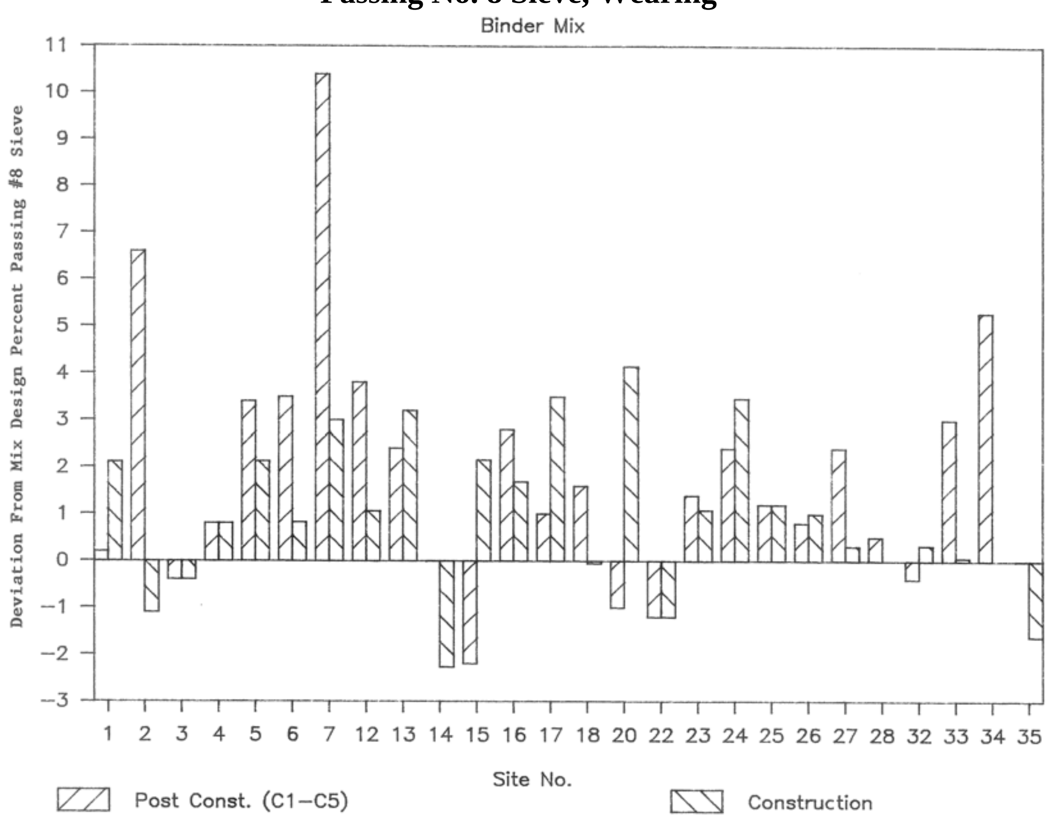

Figure 92. Comparison of In-Place and Construction to Mix Design Percent Passing No. 8 Sieve, Binder

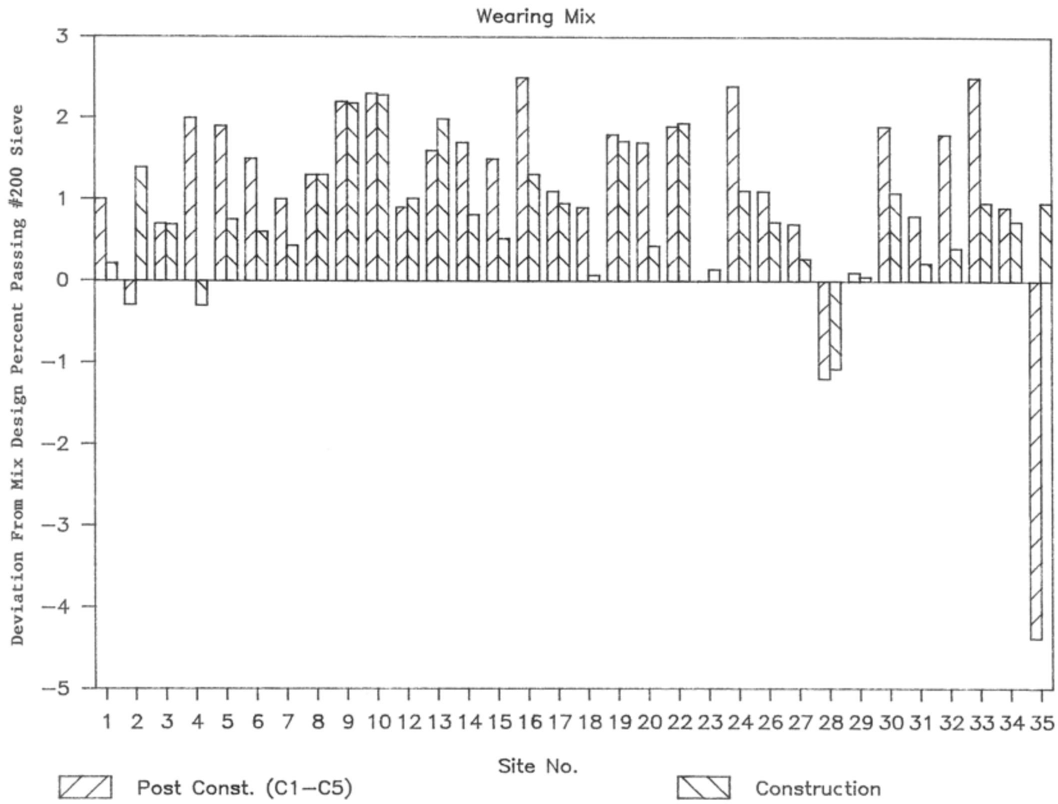

Figure 93. Comparison of In-Place and Construction to Mix Design Percent Passing No. 200 Sieve, Wearing

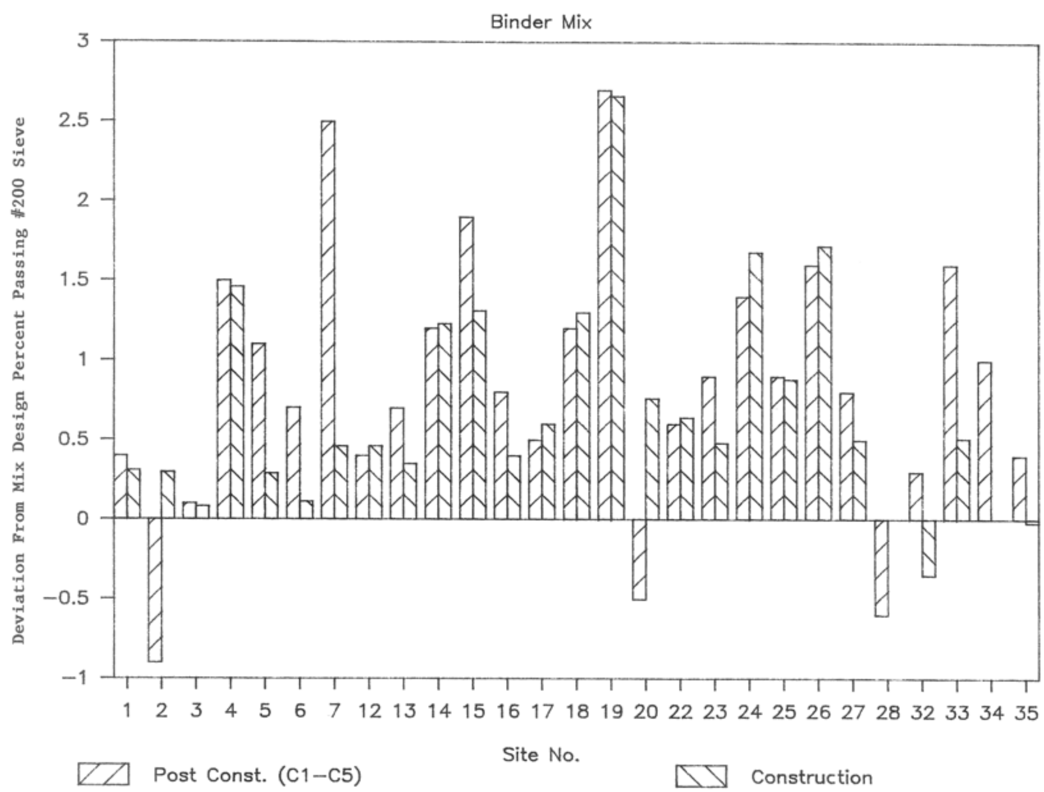

Figure 94. Comparison of In-Place and Construction to Mix Design Percent Passing No. 200 Sieve, Binder

From this data it appears that the mixes "as placed" have less asphalt cement, are somewhat finer, and have higher minus 200 content than the mixes "as designed." In addition, the in-place unit weights after traffic are exceeding the mix design unit weight. From the above discussion it is believed that the mix design compactive effort is inadequate. This change in the mix "as placed" could account for the poor correlations between mix design variables and rutting. Because of the change in mix composition, the recompacted mix properties were investigated to determine trends and threshold values of the Marshall mix properties.

To test the assumption that the mix design compactive effort is inadequate, an analysis of variance (ANOVA) was performed on the unit weights and VTMs from the mix design data, the in-place after traffic (Cl -C5) data, and the recompacted data (GTM, rotating base and static base). The in-place data contained the lowest 20th percentile VTM and highest 80th percentile unit weight of cores C7-C11. The ANOVA was performed on all pavements with 4 or more years of traffic to insure that the expected initial densification by traffic was complete.

Tables 19 and 20 show the results of the ANOVA and Duncan's multiple range test for the voids total mix (VTM) and unit weight for the wearing and binder mixes, respectively. The results show there is a significant difference in the means at the 95 percent confidence level for both the VTMs and unit weights for the wearing and binder mixes. Duncan's multiple range test is utilized to determine the sets of means which are significantly different. Duncan's test showed the mix design unit weight and VTM to be the lowest and highest respectively and significantly different from the other four variables investigated for both the wearing and binder mixes. The in-place unit weight was significantly different from and fell between the mix design and recompacted variables for both wearing and binder mixes. The in-place (C1-C5) VTM was lower and significantly different from the mix design VTM for both wearing and binder mixes, however, the in-place VTM was not significantly different from the each of the recompacted variables. The inconsistency between the unit weights and VTMs is caused by incomplete or "unbalanced" mix design VTM and unit weight data.

Table 19. Summary of ANOVA and Duncan's Test on Unit Weight and Voids Total Mix for Wearing Mixes

Source	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Unit Weight				
Total	83	150.7		
Model	4	95.7	23.9	34.36
Error	79	55.0	0.70	
Voids Total Mix				
Total	84	78.8		
Model	4	36.3	9.08	17.10
Error	80	42.5	0.53	

Duncan's Multiple Range Test alpha = 0.05					
	Unit Weight			**Voids Total Mix**	
Duncan's* Grouping	**Compactor**	**Mean (pcf)**	**Duncan's* Grouping**	**Compactor**	**Mix Mean (%)**
D	Rotating	1.437	A	Rotating	-0.954
C	Static	0.703	B	Static	-0.376
C	GTM	0.205	B & C	GTM	-0.056
B	In-Place	-0.432	C	In-Place	0.193
A	Mix Design	-1.756	D	Mix Design	1.073

* Variables with the same letter are not significantly different

Table 20. Summary of ANOVA and Duncan's Test on Unit Weight and Voids Total Mix for Binder Mixes

Source	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Unit Weight				
Total	80	176.3		
Model	4	95.0	23.7	22.21
Error	76	81.3	1.07	
Voids Total Mix				
Total	84	89.1		
Model	4	39.1	9.78	15.65
Error	80	50.0	0.62	

Duncan's Multiple Range Test alpha = 0.05					
Duncan's* Grouping	Unit Weight		Duncan's* Grouping	Voids Total Mix	
	Compactor	Mean (pcf)		Compactor	Mix Mean (%)
D	Rotating	1.030	A	Rotating	-0.647
D	GTM	0.976	A	GTM	-0.594
C	Static	0.130	A & B	Static	-0.074
B	In-Place	-0.792	B	In-Place	0.357
A	Mix Design	-1.964	C	Mix Design	1.326

* Variables with the same letter are not significantly different

The results of the ANOVA and Duncan's test show that the in-place unit weights are significantly exceeding the mix design unit weights and the in-place VTMs are significantly lower than the mix design VTMs. This higher in-place density could be caused by inadequate laboratory compactive effort and/or a change in the mix between design and placement. Compacting samples of the plant produced mixture and making mix-adjustments on the basis of the test results could help prevent the low in-place voids. Using the correct compaction hammer could solve the first problem. It appears that the rotating base, slanted foot Marshall compactor gives near maximum potential compaction level likely to be achieved only after two-three years' traffic.

Construction variables. Table 16 shows the results of the correlation analysis performed on nine construction variables. Again the correlation coefficients are low with the best correlation being with the material passing the No. 8 sieve followed by the VTM with R values of -0.53 and -0.32, respectively for the wearing course. The percent passing the No. 8 sieve showed up as significant in other models and will be discussed in detail later.

Since it is difficult to quantify the construction dates and construction traffic control, only qualitative evaluations can be made. The construction seasons given in Table 16 were related to the rate of rutting (rut depth divided by the square root of traffic in million ESALs). The results are shown in Figure 95 which indicates that pavements placed during the spring had slightly higher rates of rutting than pavements placed later in the year. The reason for this increased rate of rutting for pavements placed during the spring could be more hot weather traffic being placed on the new pavement without sufficient time for the asphalt cement to oxidize and harden where pavements placed later in the year would go through a winter before being trafficked during hot weather.

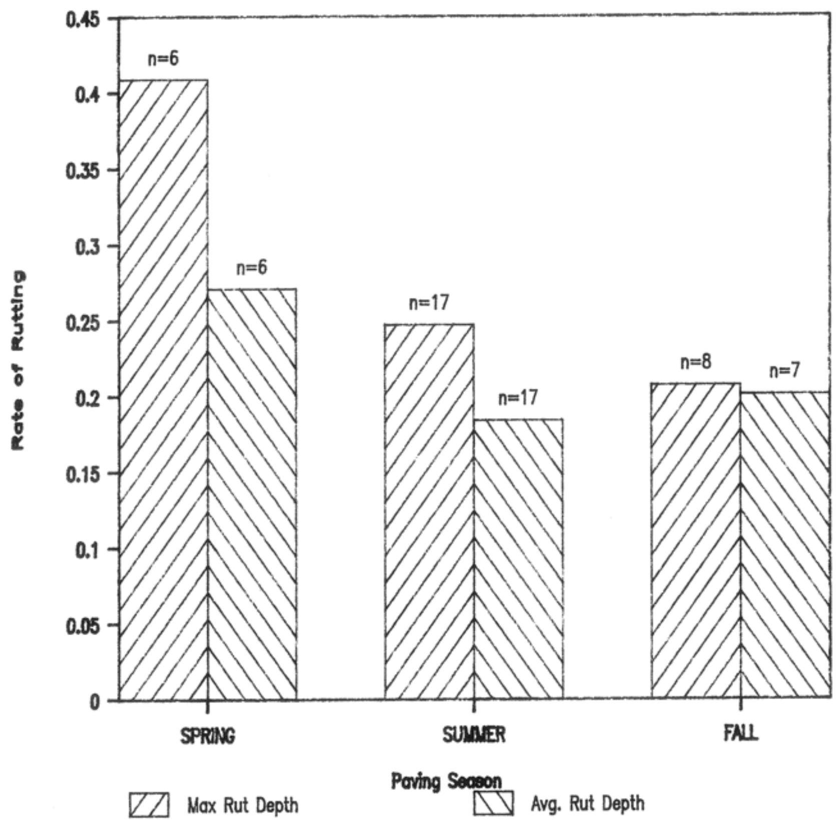

Figure 95. Paving Season Versus Rate of Rutting

Figure 96 shows that channelized construction traffic (one-way or two-way) just after paving increases the rut depth. Such traffic is more critical for mixes which are marginally resistant to rutting.

Post construction longitudinal variables. The results of the correlation analysis for the sixteen post construction longitudinal variables are shown in Table 17. Significant correlations were found between the percent passing the No. 8 sieve and rutting (R = -0.60) for the wearing mixes and between asphalt content and rutting (R= 0.62) for the binder mixes. Figure 97 shows the relationship between the percent passing the No. 8 sieve and rutting for wearing mixes. The plot shows an increase in rutting with a decrease in the amount of material passing the No. 8 sieve for the wearing mixes with an R-square of 0.22. This trend was not repeated for the binder mixes. For the binder mixes an increase in asphalt content leads to an increase in rutting with an R-square of 0.35 as shown in Figure 98. The percent voids in the fine aggregate also showed a somewhat significant trend (R= -0.45) for the binder mixes. In other words, more angular and rough textured fine aggregate tended to reduce rutting. VTM (particularly the lower 20th percentile) also showed a trend in both layers. High VTM values are associated with low rut depths. Surprisingly better correlations are indicated between VTMs and average surface rut depth (excluding the traffic) for both layers.

Post construction transverse variables. Table 18 shows the results of the correlation analysis performed on the post construction transverse variables. Significant correlations with rutting were found between GTM flow (R= 0.51), static base Marshall stability (R= -0.53), static base stability/flow ratio (R= -0.52), rotating base stability/flow ratio (R= 0.50), static base bearing capacity (R= -0.51) and rotating base Marshall stability (R= -0.51) for the wearing mixes. Other less significant correlations (R 10.5 I) between rutting and the wearing mix properties of GSI (R = 0.48), stability/flow ratio for the GTM (R = 0.49), bearing capacity (R = -0.49) for the GTM,

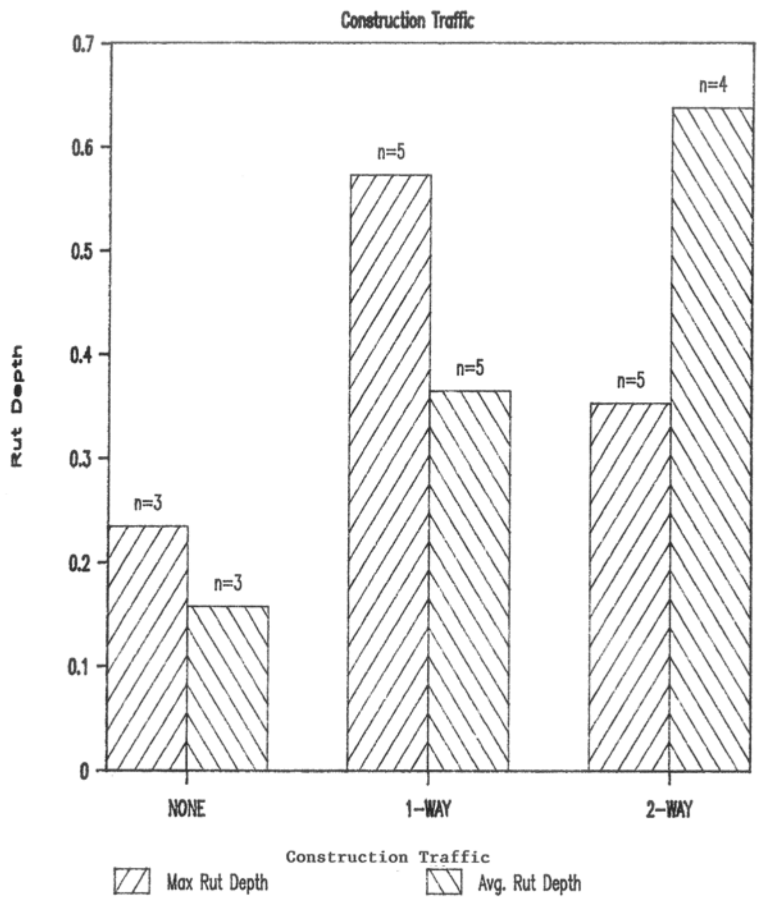

Figure 96. Construction Traffic Versus Rut Depth

and bearing capacity (R = -0.49) for rotating base compaction were found. The GSI of the binder mix had an R value of 0.47.

Figures 99 and 100 show the relationship between GSI and rutting (at the worst site) for the wearing and binder mixes respectively. The plots show an increase in rutting with an increase in GSI with R-square values of 0.21 for the wearing mixes and 0.22 for the binder mixes.

Figures 101 and 102 show the relationship between recompacted stability and flow using static base compaction for the wearing mixes. Similar trends were seen with the other two compaction methods and will not be mentioned in detail. Figure 101 shows a decrease in rutting with an increase in stability, with an R-square of 0.28 for the wearing mix. Figure 102 shows the relationship between rutting and flow, with an R-square of 0.15. As the flow increases the rutting also increases. The GTM flow had the best R-square (0.26) of the three compactive efforts. Figures 103 and 104 show the relationship between static base stability/flow ratio and bearing capacity respectively with rutting for the wearing mixes. Both parameters show a decrease in rutting with an increase in the parameter. The relationships have an R-square of 0.27 for stability/flow ratio and 0.26 for bearing capacity.

Again, better correlations are indicated between in-place VTMs and absolute surface rut depths (excluding the traffic) for both layers.

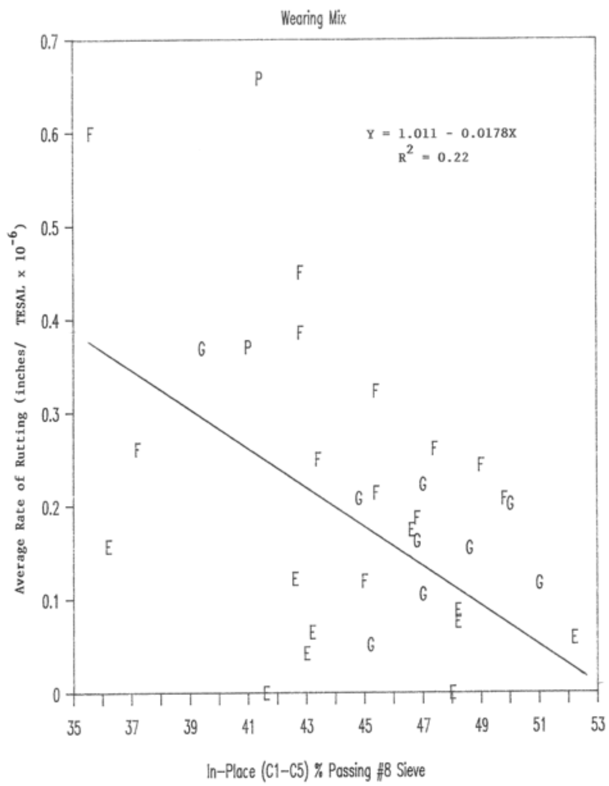

Figure 97. In-Place Percent Passing No. 8 Sieve Versus Average Rate of Rutting, Wearing

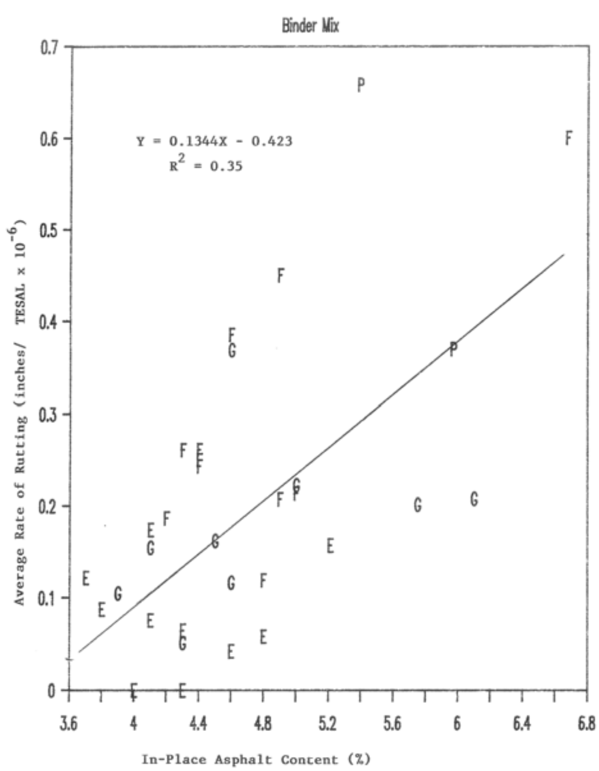

Figure 98. In-Place Asphalt Content Versus Average Rate of Rutting, Binder

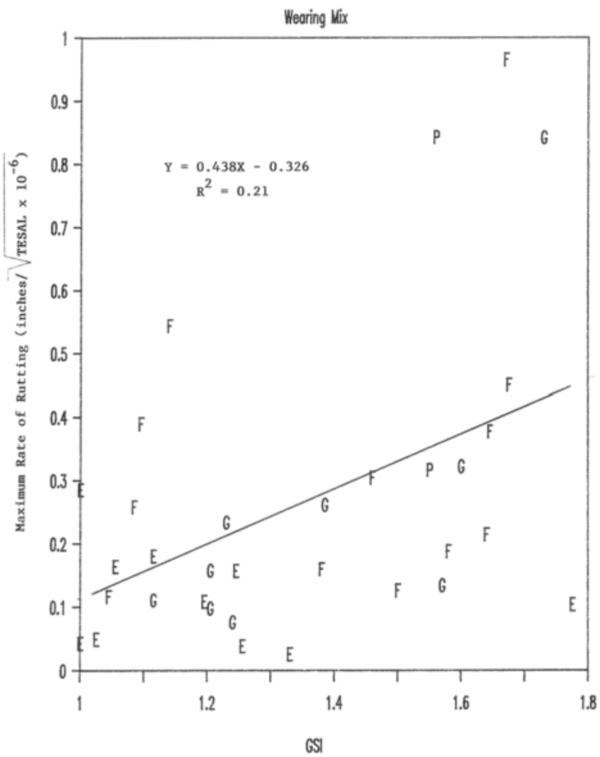

Figure 99. Recompacted GSI Versus Maximum Rate of Rutting, Wearing

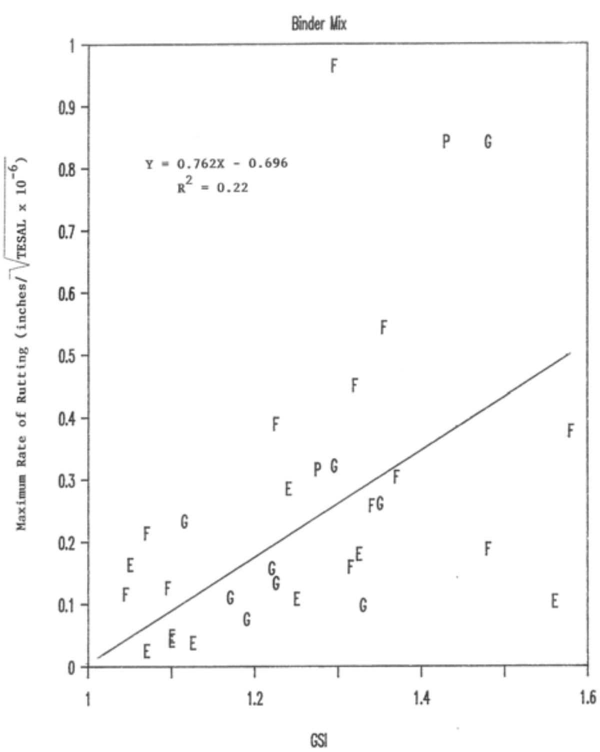

Figure 100. Recompacted GSI Versus Maximum Rate of Rutting, Binder

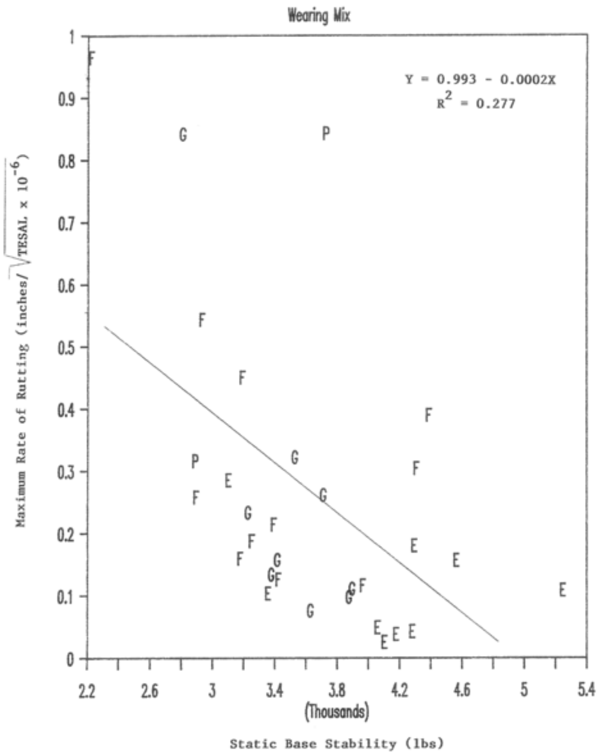

Figure 101. Static Base Stability Versus Maximum Rate of Rut, Wearing

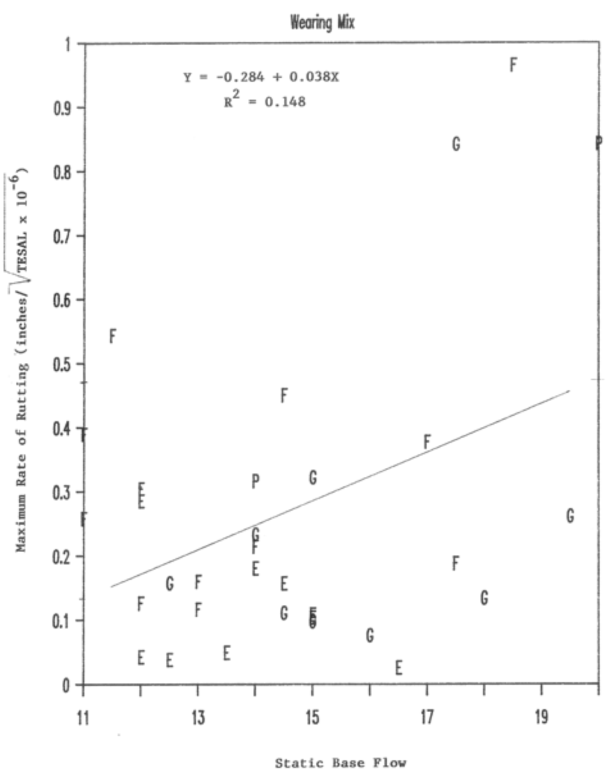

Figure 102. Static Base Flow Versus Maximum Rate of Rut, Wearing

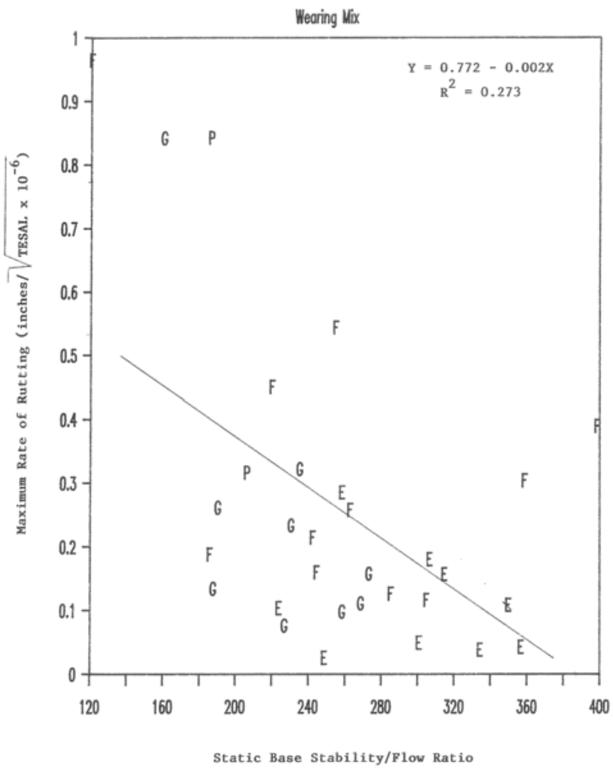

Figure 103. Static Base Stability/flow Ratio Versus Maximum Rate of Rutting, Wearing

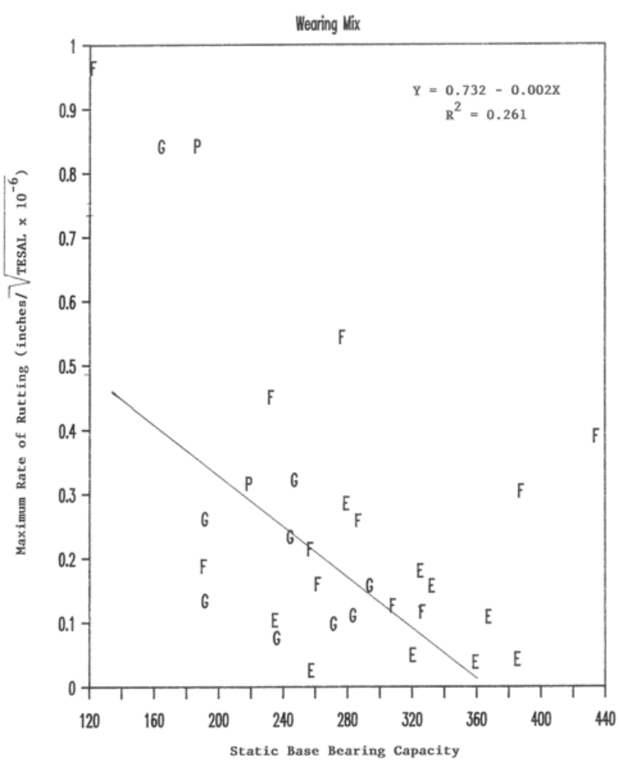

Figure 104. Static Base Bearing Capacity Versus Maximum Rate of Rutting, Wearing

Threshold Analysis

Threshold values were identified for mix design variables and post construction variables. The various threshold values were determined for the above parameters by examining plots of the percent of fair to poor pavements occurring at greater than or less than a given value of that parameter. A change in slope of the line indicates an increase or decrease in the occurrence of fair to poor pavements giving a threshold value. If a change in slope was not very apparent, then the values corresponding to about 10 percent fair/poor sites were considered.

Mix design variables. Threshold values for the mix design variables of VTM, VMA, flow and number of blows per side (obtained from the approved job-mix formula) were not readily apparent from plots of the data. Threshold values for mix design stability, stability/flow ratio and bearing capacity were determined for both the wearing and binder mixes. Figures 105 and 106 show the relationship between mix design stability and rutting for the wearing and binder mixes, respectively. The relationships show a decrease in rutting with an increase in mix design stability with an R-square of 0.10 for the surface and 0.04 for the binder. Figure 107 shows the percent of fair to poor pavements with a mix design Marshall stability greater than the given value for both the wearing and binder mixes. The slope of the curves change at 2800 lbs. for both mixes indicating a threshold value of 2800 lbs. below which the incidence of fair to poor pavements increases.

Figures 108 and 109 show the relationship between the mix design stability/flow ratio and rutting for wearing and binder mixes, respectively. The relationship shows a slight decrease in rutting with an increase in the ratio. The relationships have very low R-square values (0.04 and 0.00) respectively. Figure 110 shows the relationship between the mix design stability/flow ratio and the occurrence of fair to poor pavements for both the wearing and binder mixes. The slopes of the lines show threshold values of 250 for the wearing and 275 for the binder mix. Mixes with design stability/flow ratios below these threshold values would show an increased likelihood of rutting. Some highway agencies have minimum specification requirements for mix design stability/flow ratio.

Figures 111 and 112 show the relationship between the mix design bearing capacity and rutting for the wearing and binder mixes respectively. The relationship shows a slight decrease in rutting with an increase in the mix design bearing capacity. The relationships have very low R-square values (0.04 and 0.00) respectively. Figure 113 shows the relationship between the bearing capacity and the occurrence of fair to poor pavements for both the wearing and binder mixes. The slopes of the lines show threshold values of 275 for both wearing and binder mixes. Mixes with bearing capacities below this threshold value would show an increased likelihood of rutting.

It appears that mix design stability/flow ratios are slightly better than the bearing capacity to indicate potential rutting based on the data evaluated in this study. However, due to the poor correlations and the difference between the “as designed” and “as built” mix properties, threshold values from mix design variables could be misleading.

Construction variables. All of the threshold values identified for the construction variables were also identified for the post construction variables. Selecting threshold values based on parameters not subjected to traffic loadings could lead to misleading conclusions and, therefore, are not reported.

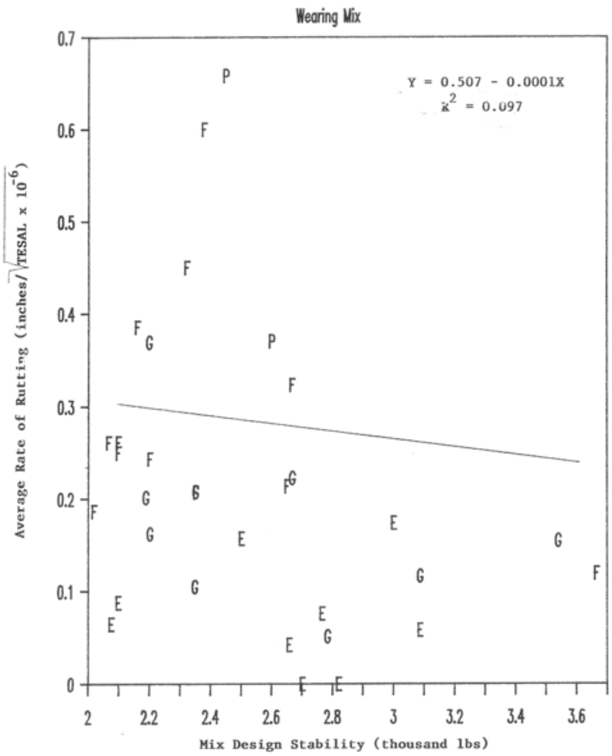

Figure 105. Mix Design Stability Versus Average Rate of Rutting, Wearing

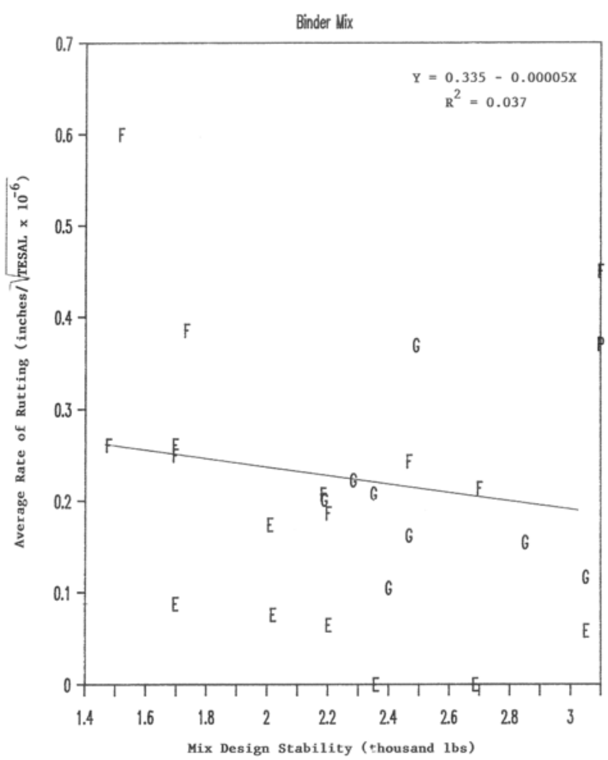

Figure 106. Mix Design Stability Versus Average Rate of Rutting, Binder

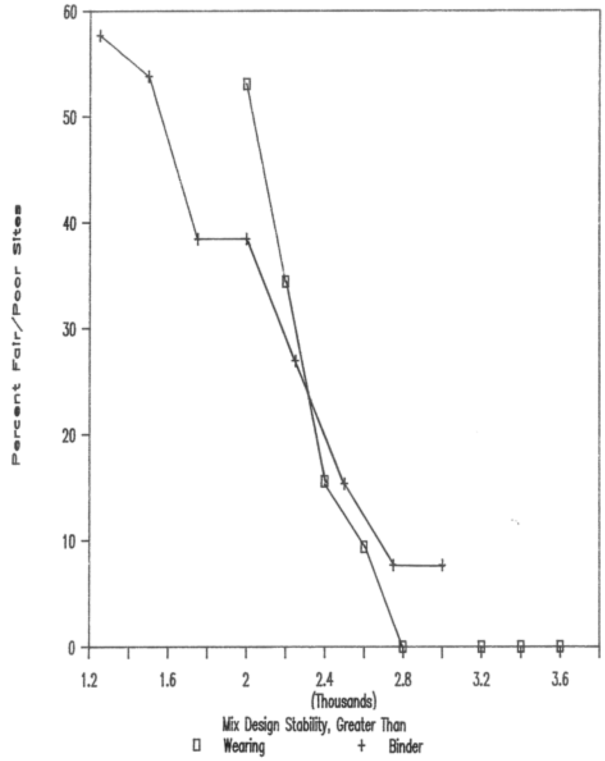

Figure 107. Mix Design Stability Versus Percent Fair to Poor Pavements

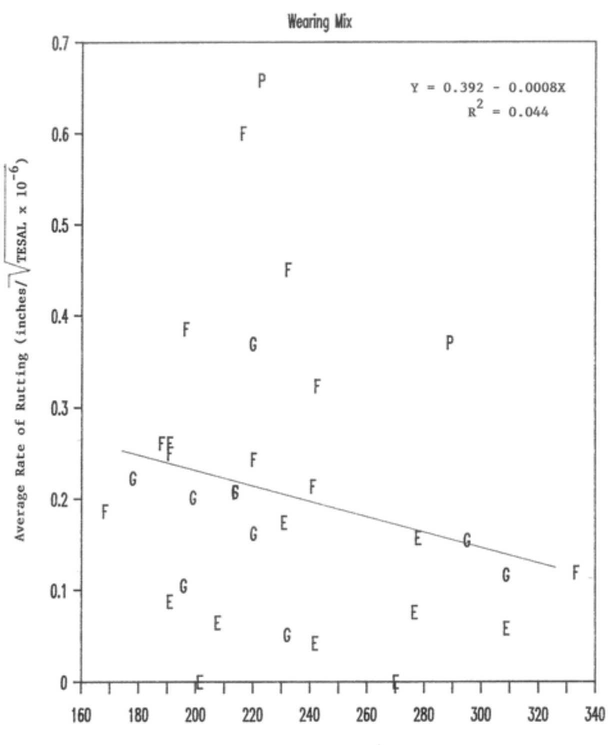

Figure 108. Mix Design Stability/Flow Ratio Versus Average Rate of Rutting, Wearing

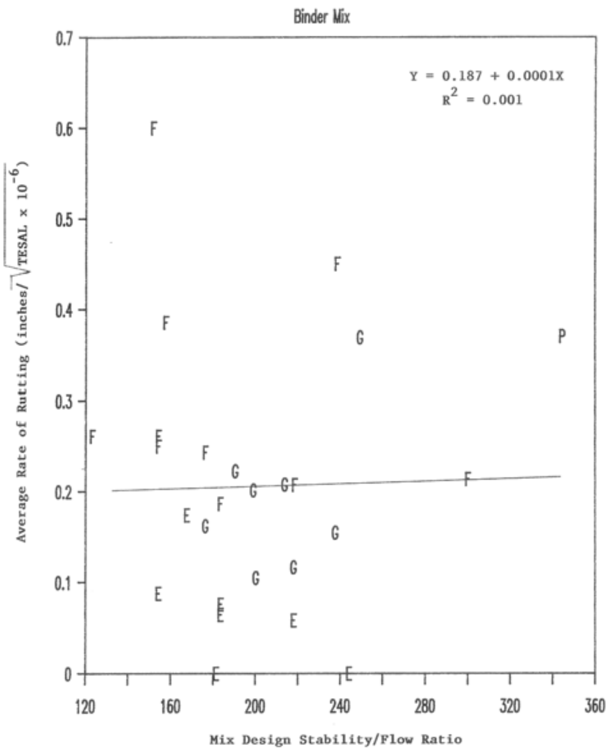

Figure 109. Mix Design Stability/Flow Ratio Versus Average Rate of Rutting, Binder

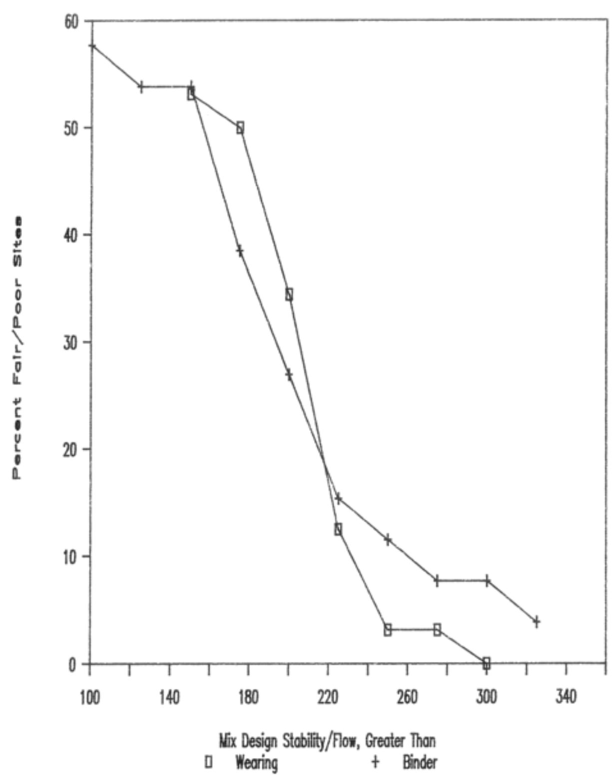

Figure 110. Mix Design Stability/Flow Ratio Versus Percent Fair to Poor Pavements

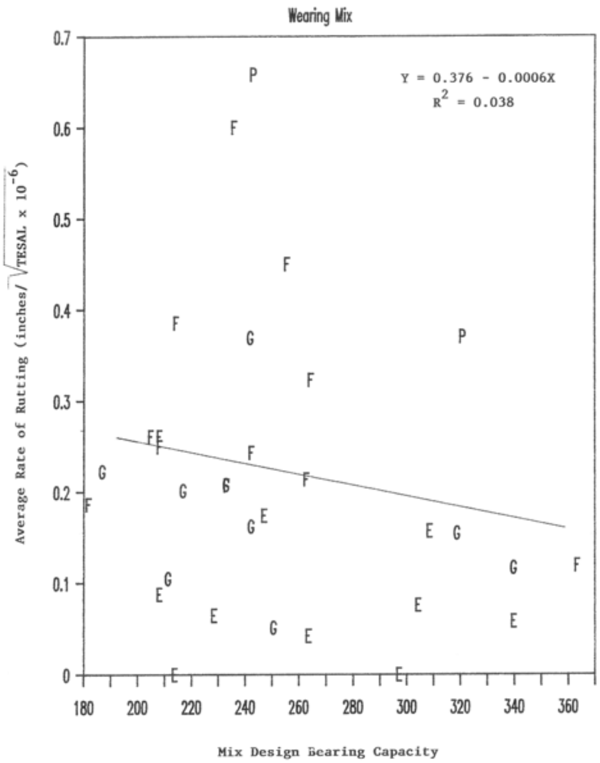

Figure 111. Mix Design Bearing Capacity Versus Average Rate of Rutting, Wearing

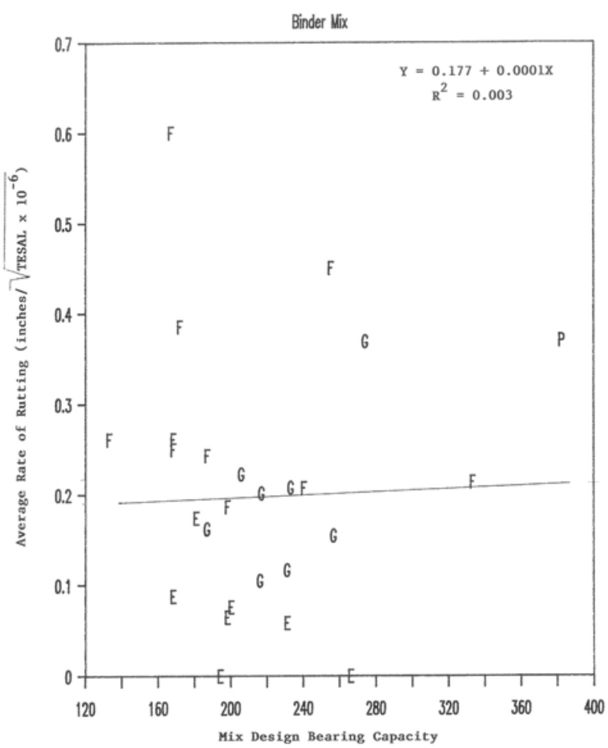

Figure 112. Mix Design Bearing Capacity Versus Average Rate of Rutting, Binder

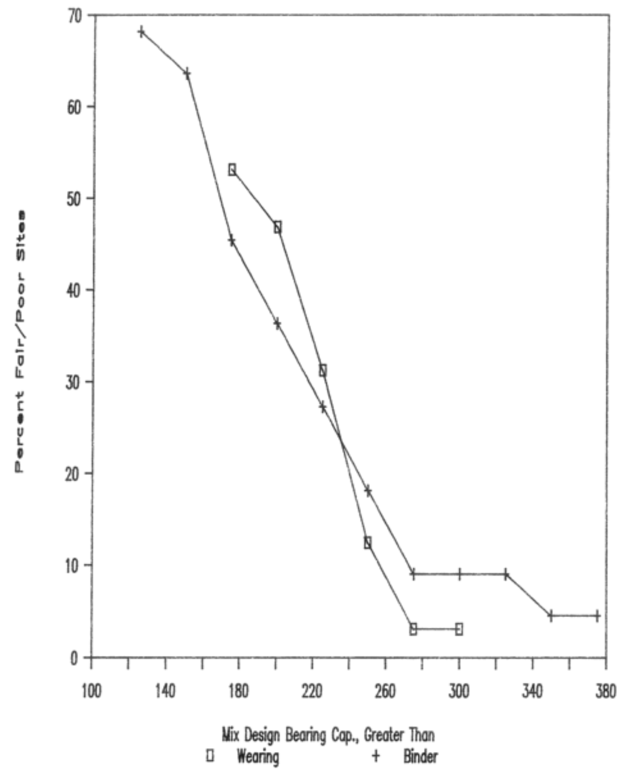

Figure 113. Mix Design Bearing Capacity Versus Percent Fair to Poor Pavements

Post construction longitudinal variables. Threshold values were identified for in-place VTM, percent natural sand in the fine aggregate and the percent passing the No. 8 and No. 200 sieves for the post construction longitudinal variables. Figures 114 and 115 show the relationship between the average in-place VTM and rutting for the wearing and binder mix, respectively. The R-square (for this relationship is 0.04 for the wearing mix and 0.10 for the binder, which is too low to be useful. However, a trend of an increase in rutting with a decrease in VTM is evident. Figure 116 shows the percent of pavements rated fair to poor at greater than a given air void content versus rutting for the wearing and binder mixes. The plot shows a change in the slope of the line at 3.0 percent VTM for the wearing mixes and 2.0 percent VTM for the binder mixes. Below these threshold values the occurrence of fair to poor pavements increased.

Figures 97 and 117 show the relationship between the percent passing the No. 8 sieve and rutting for the wearing and binder mixes respectively. The R-square for this relationship is 0.22 for the wearing and 0.01 for the binder. Figure 118 and 119 show the relationship between the percent passing the No. 8 sieve and the percent of fair to poor pavements for the wearing and binder mixes. From a review of these plots in conjunction with Figures 97 and 117 (particularly the rate of rutting of 0.2 inches per square root of million ESALs) it appears that mixes with between 45-50 percent passing the No. 8 sieve for wearing mixes and 25-30 percent passing for binder mixes generally performed the best. However, it must be realized that the gradation obtained from extracting pavement cores is generally finer than the mix produced at the plant because of degradation resulting from compaction during construction, coring and sawing operations.

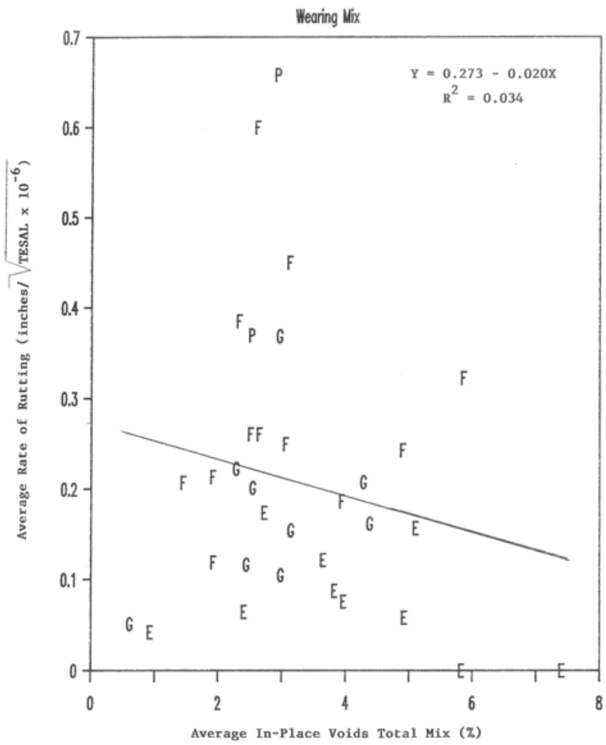

Figure 114. Average In-Place VTM Versus Average Rate of Rutting, Wearing

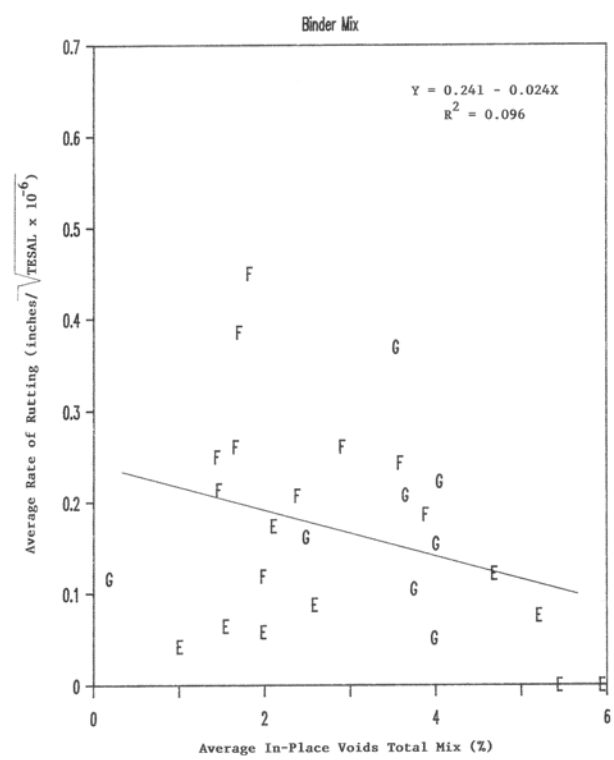

Figure 115. Average In-Place VTM Versus Average Rate of Rutting, Binder

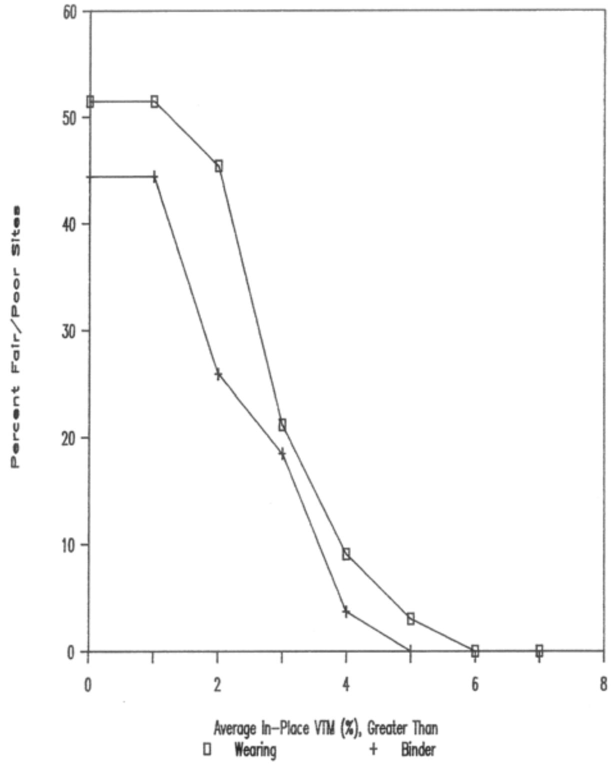

Figure 116. Average In-Place VTM Versus Percent Fair to Poor Pavements

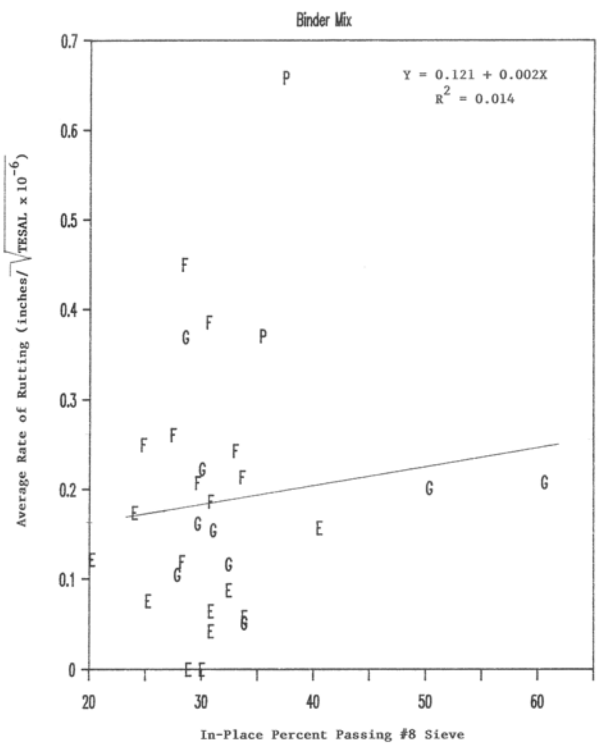

Figure 117. In-Place Percent Passing No. 8 Sieve Versus Average Rate of Rutting, Binder

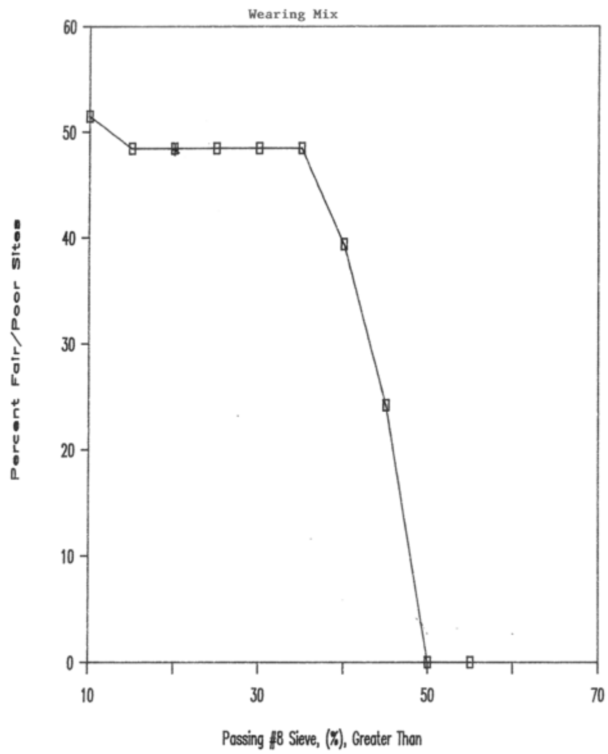

Figure 118. In-Place Percent Passing No. 8 Sieve Versus Percent Fair to Poor Pavements, Wearing

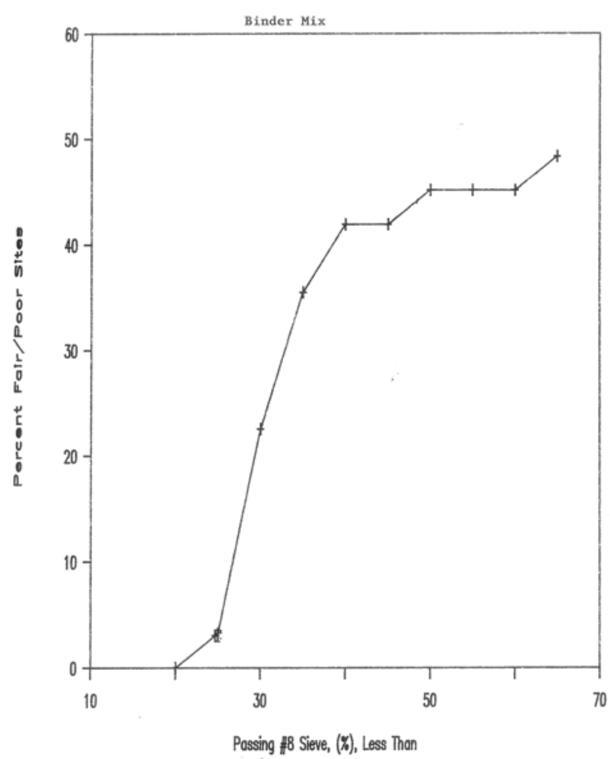

Figure 119. In-Place Percent Passing No. 8 Sieve Versus Percent Fair to Poor Pavements, Binder

Figures 120 and 121 show the relationship between the percent passing the No. 200 sieve and rutting for the wearing and binder mixes respectively. The R-square values are 0.02 and 0.01 respectively which are much too low to be useful. Figure 122 shows the relationship between the percent passing the No. 200 sieve and the percent of fair to poor pavements. The plot shows that the slope of the line changes at 5 percent for the wearing mixes and 4 percent for the binder mixes with an increase in the occurrence of fair to poor pavements occurring above these limits. However, the spread of data is too small and insufficient to recommend these values as threshold values.

Figure 123 shows the relationship between the percent natural sand in the fine aggregate and the percent fair to poor pavements. It appears that mixes with less than 20 percent natural sand in the fine aggregate contained fewer fair to poor pavements than mixes with over 20 percent natural sand in the fine aggregate. Ten pavements (Nos. 1, 14, 15, 18, 19, 20, 26, 32, 33 and 34) had no natural sand in both wearing and binder course mixes. Of these ten pavements, eight were good to excellent and two were fair in performance.

Post construction transverse variables. Threshold values were found for GSI, lower 20th percentile VTM, and the recompacted properties of VMA, VTM, stability, stability/flow ratio and bearing capacity for both the wearing and binder mixes. Similar results were found for the different recompactive efforts utilized so only the static base samples will be discussed because static base compaction was used by PennDOT for mix design and production control.

Figures 99 and 100 show the relationship between GSI and rutting for the wearing and binder mixes respectively. The relationship has an R-square value of 0.21 for the wearing mix and 0.22 for the binder mixes. The graphs show a definite trend of increased rutting with an increase in GSI. Figure 124 shows the relationship between GSI and the percent of fair to poor pavements. The plot shows a significant increase in the percentage of fair to poor pavements when the GSI is above 1.2 for both wearing mixes and binder mixes. This trend agrees with previous work at NCAT (*10*) which shows that mixes with a GSI of 1.0 will be stable, mixes with a GSI of 1.1 to 1.3 will rut moderately and mixes with a GSI of over 1.3 will rut severely. Average GSI values of 1.35 and 1.26 for wearing and binder courses, respectively, obtained in this study are on the high side and indicate potential for rutting.

Figures 125 and 126 show the percent of fair to poor pavements with VMAs greater than the given value for the wearing and binder mixes respectively. The data shows that the percentage of fair to poor pavements increases when the recompacted VMA falls below 15 percent for the wearing mixes and below 12 to 13 percent for the binder mixes.

Figures 101 and 127 show the relationship between recompacted stability and rutting for the wearing and binder mixes. The relationship has an R-square value of 0.28 for the wearing mixes and 0.12 for the binder mixes. Figure 128 shows the relationship between stability and the percent of fair to poor pavements. The results show an increase in the percent of poor to fair pavements when the recompacted stability drops below 3400 lbs. for wearing mixes and 3600 lbs. for binder mixes.

Figures 102 and 129 show the relationship between recompacted flow and rutting for wearing and binder mixes, respectively. The relationships have an R-square value of 0.15 for the wearing mixes and 0.05 for the binder mixes. A threshold value for flow could not be determined.

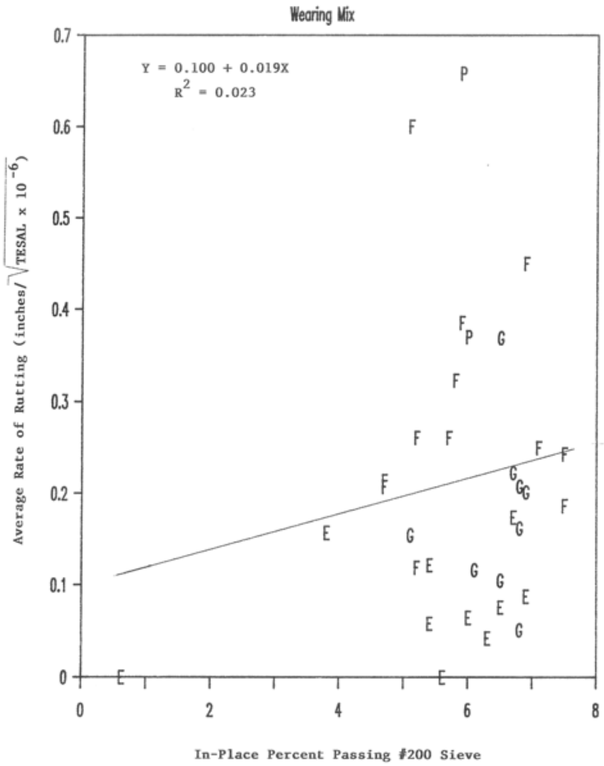

Figure 120. In-Place Percent Passing No. 200 Sieve Versus Average Rate of Rutting, Wearing

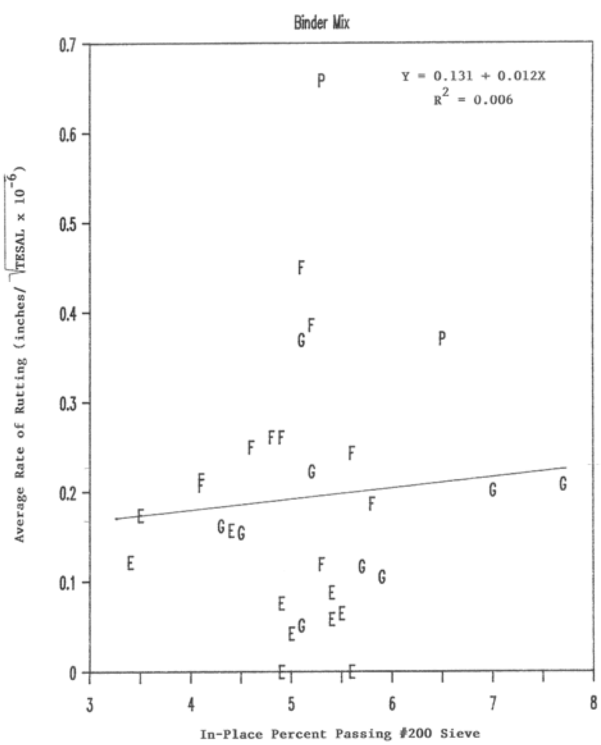

Figure 121. In-Place Percent Passing No. 200 Sieve Versus Average Rate of Rutting, Binder

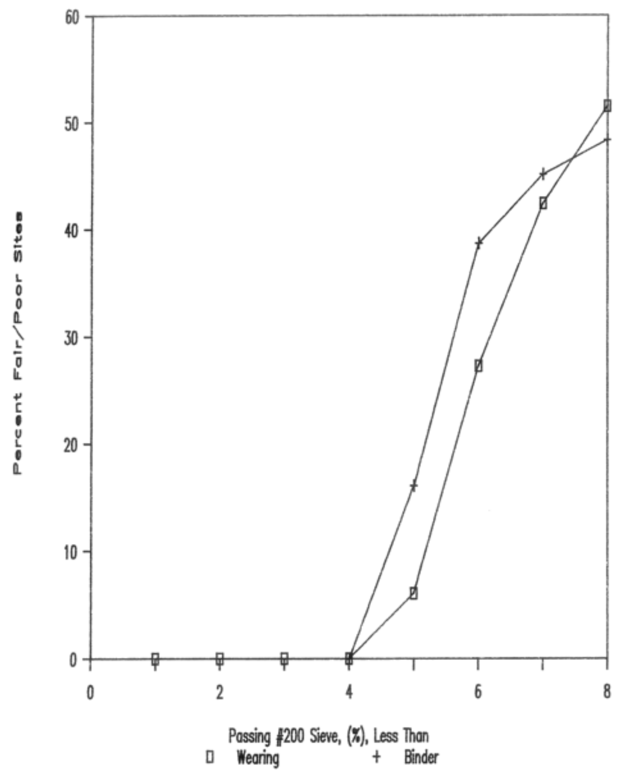

Figure 122. In-Place Percent Passing No. 200 Sieve Versus Percent Fair to Poor Pavements

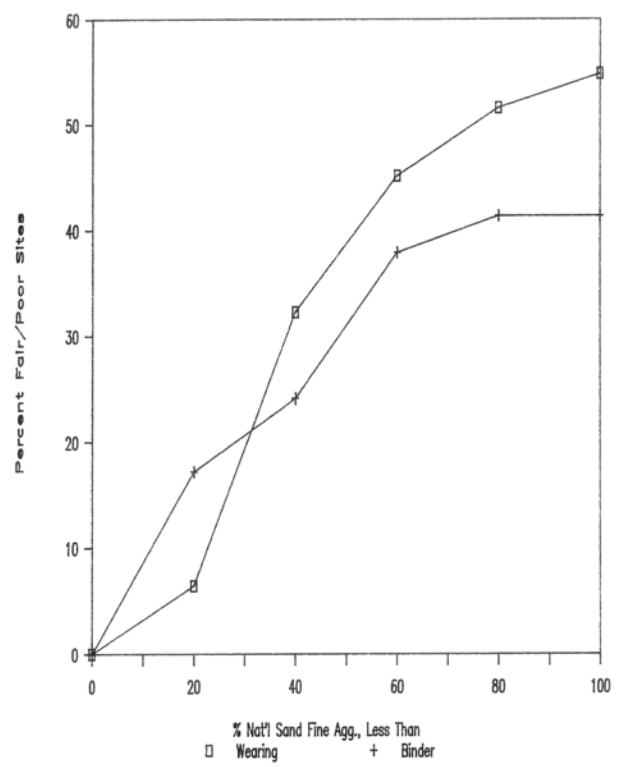

Figure 123. Percent Natural Sand in Fine Aggregate Versus Percent Fair to Poor Pavements

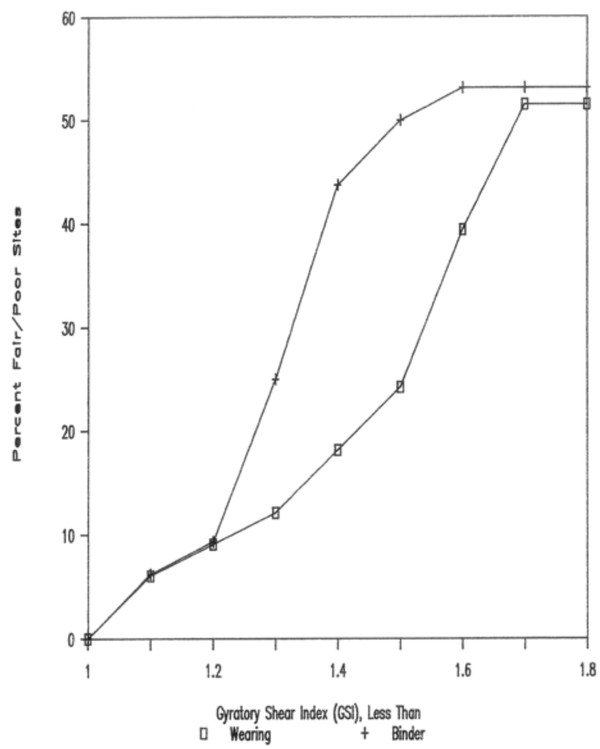

Figure 124. Recompacted GSA Versus Percent Fair to Poor Pavements

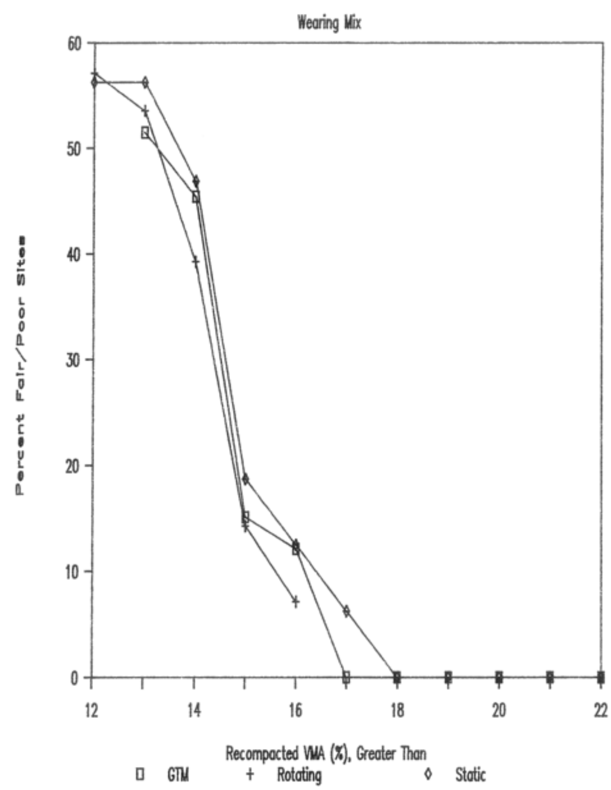

Figure 125. Recompacted VMA Versus Percent Fair to Poor Pavements, Wearing

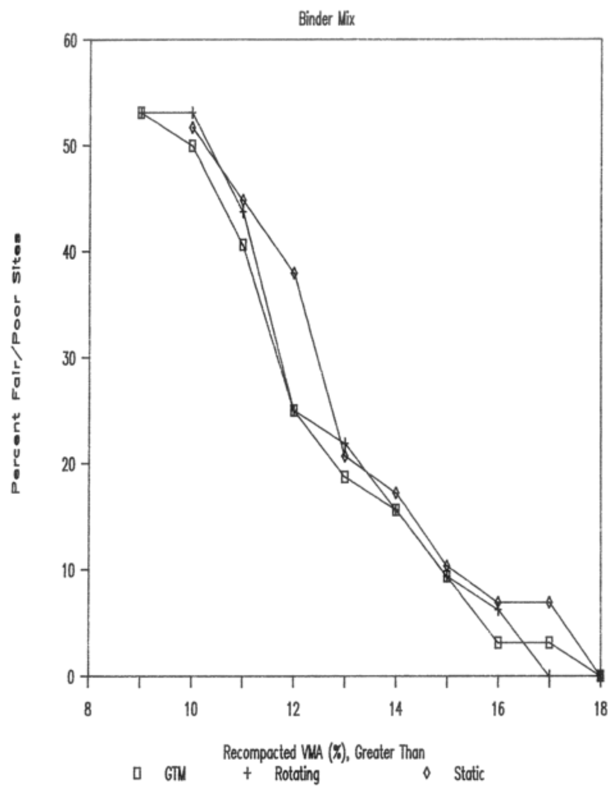

Figure 126. Recompacted VMA Versus Percent Fair to Poor Pavements, Binder

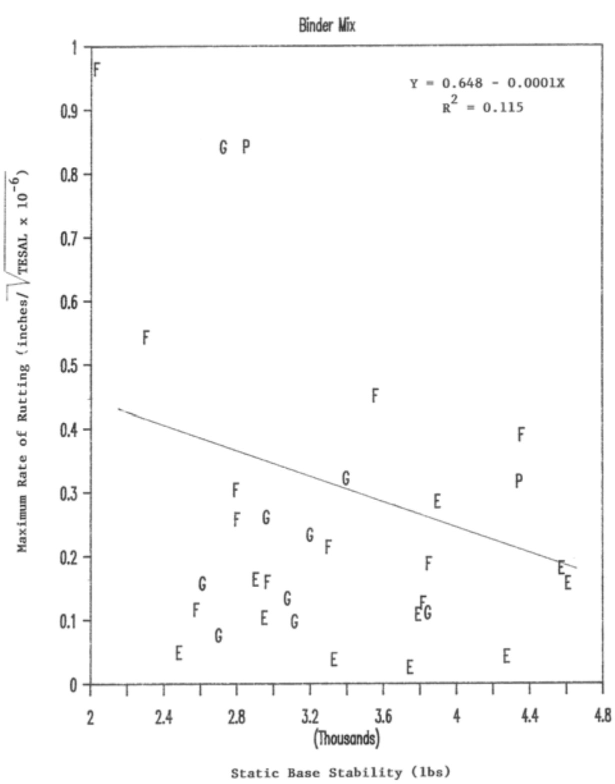

Figure 127. Static Base Stability Versus Maximum Rate of Rutting, Binder

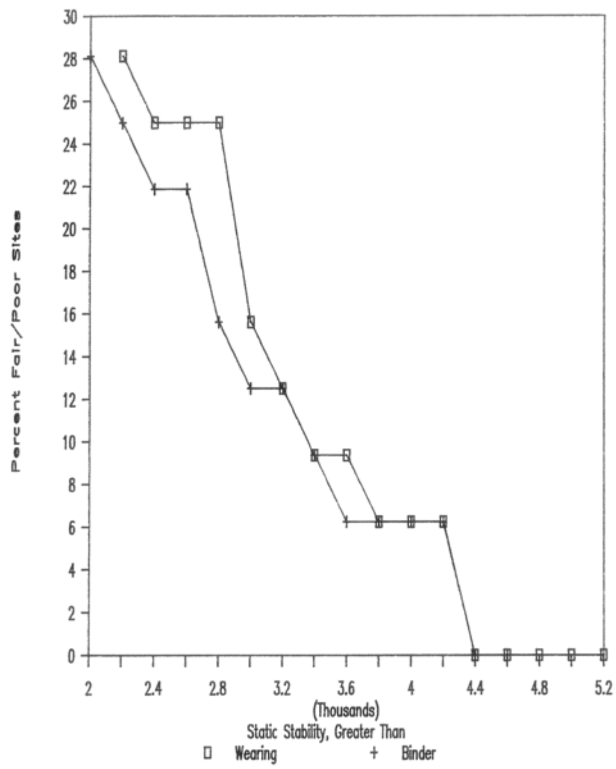

Figure 128. Static Base Stability Versus Percent Fair to Poor Pavements

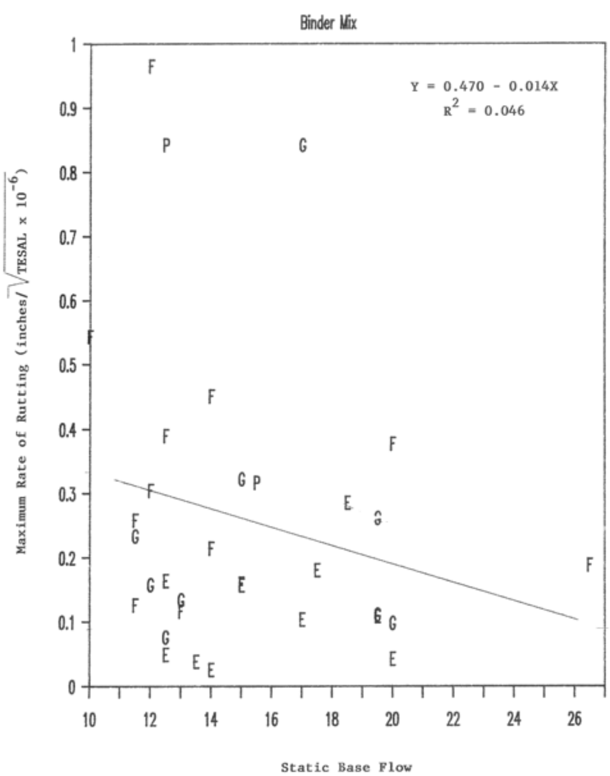

Figure 129. Static Base Flow Versus Maximum Rate of Rutting, Binder

Figures 103 and 130 show the relationship between recompacted stability/flow ratio and rutting for the wearing and binder mixes, respectively. The relationship has an R-square value of 0.27 for the wearing mixes and 0.01 for the binder mixes. Figure 131 shows the relationship between stability/flow ratio and the percent of fair to poor pavements. The results show an increase in the percent of poor to fair pavements when the recompacted stability/flow ratio drops below 280 for wearing mixes and 260 for binder mixes.

Figures 104 and 132 show the relationship between recompacted bearing capacity and rutting. The relationships have an R-square value of 0.26 for the wearing mixes and 0.01 for the binder mixes. Figure 133 shows the relationship between bearing capacity and the percentage of fair to poor pavements. The results show that the percentage of fair to poor pavements increases when the recompacted bearing capacity drops below 300 for the wearing and below 280 for binder mixes.

Stepwise Regression Analysis

Rutting appears to be a complex phenomenon in which no one parameter is able to predict rut depth with an acceptable level of significance as evidenced by the low correlation coefficients reported. The stepwise procedure for selection of single regressor variables, which when retained stepwise in a multiple linear regression equation, are most correlated to the dependent variable. Two stepwise procedures were utilized to analyze the groups of independent variables. The dependent variable utilized is the average surface rut depth divided by the square root of traffic.

The two stepwise procedures utilized were the forward and backward methods. In the forward selection procedure, the single variable which is most correlated to the dependent variable in a step is added to the multiple regression equation until no variables remain that, when added to the model, reduce the deviations sum of squares at a 0.5 significance level. In the backward procedure, the single variable which is least correlated to the dependent variable in a step is deleted from the multiple regression equation. The procedure stops when all variables remaining in the model are significant at the 0.1 level. It should be noted that the R-square values for individual independent variables reported in the stepwise regression analysis may be different than those reported earlier. The stepwise regression procedure requires balanced data (no missing values) for every level of each factor in the model. Therefore, only sites with complete data for the variables selected for the model were analyzed.

Design variables. The stepwise procedure for the mix design variables is summarized in Table 21. All 10 mix design variables predicted rutting with an R-square of 0.33 for the wearing mixes and 0.43 for the binder mixes. For the wearing mixes, the percent passing the No. 8 sieve, number of blows per side, stability and flow made a significant contribution to the model (R-square = 0.30) with the percent passing the No. 8 sieve making the largest single contribution (R-square = 0.24).

For the binder mixes, the forward stepwise procedure identified VMA, flow, percent AC, and the percent passing the No. 8 and No. 200 sieves, as making significant contributions to the model with an R-square of 0.40. The backward procedure selected the percent AC and percent passing the No. 8 sieve as significant with an R-square of 0.22. From this information it is evident that the mix design parameters do not do a good job of predicting rutting especially when the “as placed” mix is significantly different from the “as designed” mix as discussed earlier.

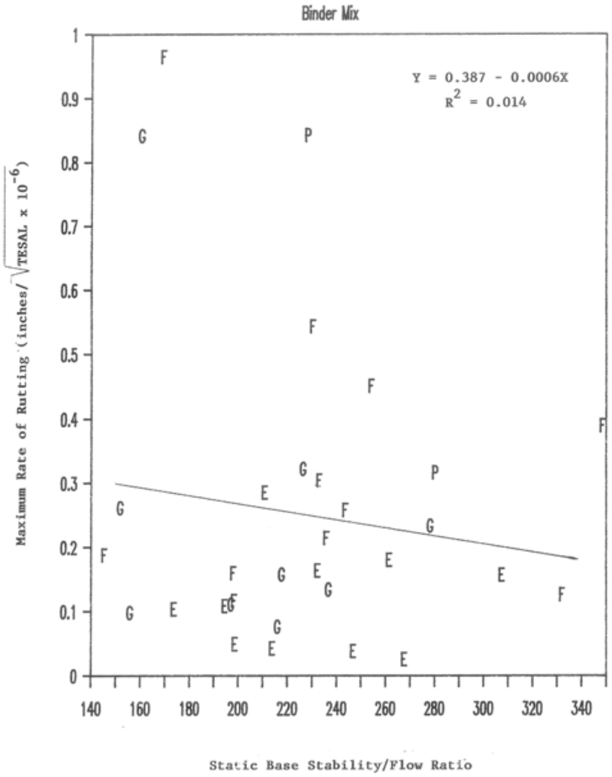

Figure 130. Static Base Stability/Flow Ratio Versus Maximum Rate of Rutting, Binder

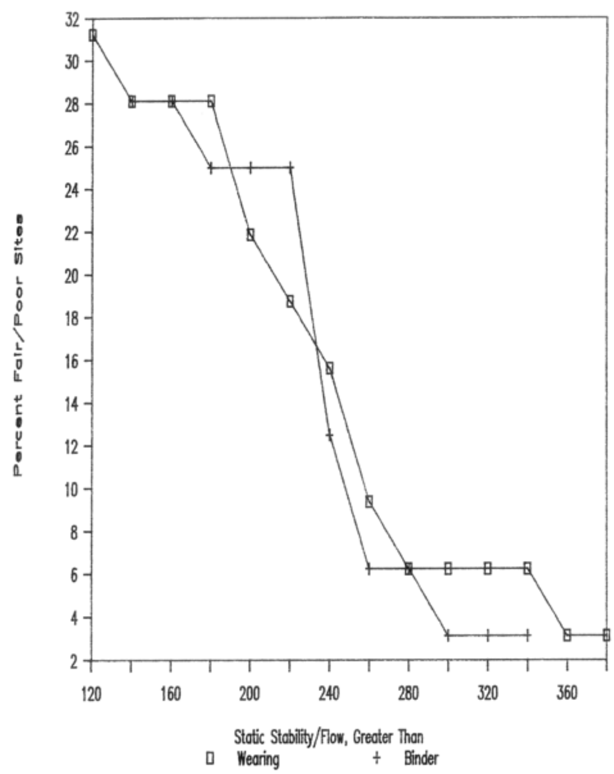

Figure 131. Static Base Stability/Flow Ratio Versus Percent Fair to Poor Pavements

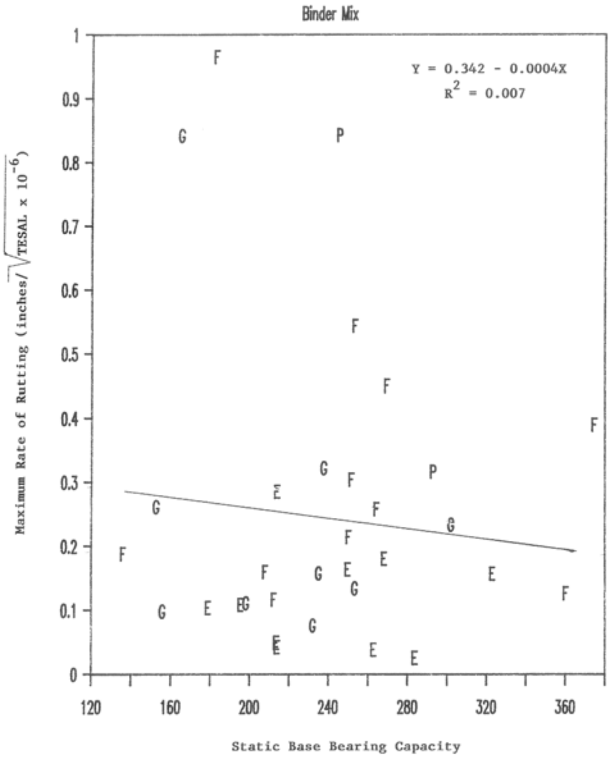

Figure 132. Static Base Bearing Capacity Versus Maximum Rate of Rutting, Binder

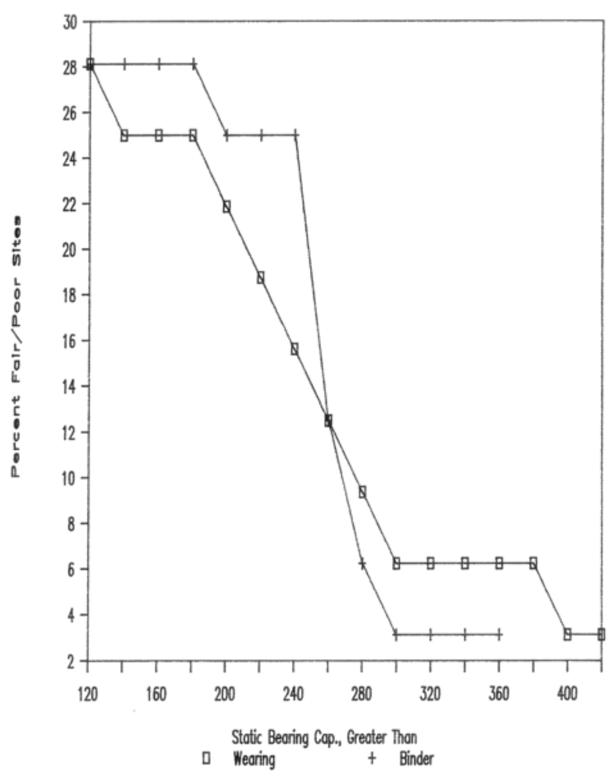

Figure 133. Static Base Bearing Capacity Versus Percent Fair to Poor Pavements

Table 21. Summary of Stepwise Regression Analysis for Mix Design Variables

Forward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.33)				
1	Passing #8	1	0.2445	0.2445
2	# Blows	2	0.0280	0.2725
3	Stability	3	0.0190	0.2915
4	Flow	4	0.0124	0.3040
Binder Mix (All Variables R-square = 0.43)				
1	Flow	1	0.1382	0.1382
2	Passing #200	2	0.0469	0.1852
3	VMA	3	0.0666	0.2517
4	Passing #8	4	0.0792	0.3308
5	Asphalt Cement	5	0.0726	0.4035

Backward Selection Procedure

Step	Variable Removed	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.33)				
1	VTM	9	0.0001	0.3273
2	Passing #200	8	0.0011	0.3262
3	% Asphalt Cement	7	0.0049	0.3213
4	Stability	6	0.0032	0.3181
5	VMA	5	0.0050	0.3131
6	Stability/Flow	4	0.0062	0.3068
7	Bearing Capacity	3	0.0165	0.2903
8	Flow	2	0.0178	0.2725
9	# Blows	1	0.0280	0.2445
Binder Mix (All Variables R-square = 0.43)				
1	VTM	9	0.0010	0.4285
2	# Blows	8	0.0039	0.4246
3	Bearing Capacity	7	0.0063	0.4183
4	Stability/Flow	6	0.0041	0.4141
5	Stability	5	0.0107	0.4035
6	Passing #200	4	0.0317	0.3718
7	VMA	3	0.0582	0.3136
8	Flow	2	0.0937	0.2198

Construction variables. The stepwise analysis of the construction variables was limited to the four independent variables VTM, AC and the percent passing the No. 8 and No. 200 sieves. The conformal indexes were left out of this portion of the analysis due to the difficulty in determining the usefulness of results from conformal indexes. The results of the stepwise analysis are summarized in Table 22. The remaining construction variables predicted rutting with an R-square of 0.34 for the wearing and 0.47 for the binder mixes. The R-square values are still low, however they are higher than the mix design variables. For the wearing mixes the stepwise procedures identified the variables percent passing the No. 8 and No. 200 sieve and the VTM as having a significant effect on rutting with an R-square value of 0.34. The percent passing the No. 8 sieve was identified as having the most significant contribution to rutting with an R-square of 0.28. For the binder mixes all four variables made significant contributions to the model with an R-square of 0.47. The variable AC was identified as making the least contribution to the model.

Table 22. Summary of Stepwise Regression Analysis for Construction Variables

Forward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.34)				
1	Passing #8	1	0.2787	0.2787
2	Passing #200	2	0.0385	0.3172
3	VTM	3	0.0272	0.3444
Binder Mix (All Variables R-square = 0.47)				
1	Passing #8	1	0.1895	0.1895
2	VTM	2	0.0788	0.2684
3	Passing #200	3	0.1678	0.4362
4	% AC	4	0.0346	0.4708

Backward Selection Procedure

Step	Variable Removed	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.34)				
1	% AC	3	0.0001	0.3444
2	VTM	2	0.0272	0.3172
3	Passing #200	1	0.0385	0.2787
Binder Mix (All Variables R-square = 0.47)				
1	% AC	3	0.0346	0.4362

Table 23. Summary of Stepwise Regression Analysis for Post Construction Longitudinal Variables

Forward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.55)				
1	Passing #8	1	0.4373	0.4373
2	Passing #200	2	0.0341	0.4714
3	Crushed Particles	3	0.0206	0.4920
4	% Natural Sand	4	0.0122	0.5042
5	Avg. VTM	5	0.0161	0.5203
6	% AC	6	0.0189	0.5392
Binder Mix (All Variables R-square = 0.64)				
1	% AC	1	0.2435	0.2435
2	Passing #8	2	0.0873	0.3308
3	Crushed Particles	3	0.0811	0.4119
4	Avg. VTM	4	0.0857	0.4976
5	Viscosity	5	0.1008	0.5985
6	Passing #200	6	0.0369	0.6354

Backward Selection Procedure

Step	Variable Removed	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.55)				
1	Passing #200	7	0.0043	0.5494
2	Viscosity	6	0.0160	0.5335
3	Penetration	5	0.0044	0.5290
4	% AC	4	0.0205	0.5086
5	Avg. VTM	3	0.0270	0.4816
6	% Natural Sand	2	0.0185	0.4631
7	Crushed Particles	1	0.0258	0.4373
Binder Mix (All Variables R-square = 0.64)				
1	Penetration	7	0.0015	0.6382
2	% Natural Sand	6	0.0028	0.6354
3	Passing #200	5	0.0369	0.5985
4	Crushed Particles	4	0.0550	0.5434

Post construction longitudinal variables. The results of the stepwise analysis for the post construction longitudinal variables are shown in Table 23. Again, the conformal indexes were removed from the model prior to analysis. The analysis shows that the eight post construction longitudinal variables of AC, passing the No. 8 and No. 200 sieves, penetration, viscosity, percent crushed particles in the coarse aggregate, percent natural sand in the fine aggregate and average VTM predict rutting with an R-square value of 0.55 for the wearing mixes and 0.64 for the binder mixes.

For the wearing mixes the stepwise procedure identified all of the variables except the penetration and viscosity of the recovered asphalt as contributing significantly to the model with a combined R-square of 0.54. The percent passing the No. 8 sieve made the largest single contribution to the model with an R-square of 0.44. For the binder mixes the AC, percent passing the No. 8 sieve, percent crushed particles, average VTM, viscosity, and percent passing the No. 200 sieve contributed significantly to the model with an R-square of 0.64. The variables AC, percent passing the No. 8 sieve, average VTM and viscosity contributed to the model with an R-square of 0.54 based on backward selection procedure. It is interesting to note that AC contributed significantly to rutting in binder mixes and not in wearing mixes. This indicates that the binder mixes in Pennsylvania need to be made relatively leaner and, therefore, stiffer to resist rutting.

Post construction transverse variables. With the exception of GSI and creep the post construction transverse variables have been included in other models. Rather than repeating the analysis, the transverse variables were included with the longitudinal variables to create a new data set. The new data set included the variables that could be performed during mix production quality control to determine if a quality control test program utilizing recompacted samples of the produced mix could predict rutting. The variables were divided into three groups for analysis based on the recompactive method utilized. These three groups included GTM recompaction, static base recompaction and rotating base recompaction. The variables selected were AC, average in-place VTM, percent passing the No. 8 and No. 200 sieves, recovered asphalt penetration and viscosity, percent crushed particles, percent natural sand in the fine aggregate, creep, and the recompacted properties of stability, flow, stability/flow ratio, bearing capacity, VTM and VMA. GSI was included in the GTM recompacted model. The results are shown in Tables 24 through 26 for GTM, static base and rotating base recompaction variables, respectively.

GTM variables. The model for predicting rutting based on the sixteen GTM variables has an R-square of 0.68 for the wearing mixes and 0.95 for the binder mixes. The results of the stepwise procedure is shown in Table 24. For the wearing mixes the forward procedure selected the percent passing the No. 8 sieve, GSI, average VTM, creep, VMA, and the percent passing the No. 200 sieve as all contributing significantly to the model with an R-square of 0.64. The backward procedure selected the variables of the percent passing the No. 8 sieve and the stability/flow ratio as the only significant variables with an R-square of 0.51.

The forward procedure for the binder mixes selected all of the variables as significant with the exception of penetration and flow with an R-square of 0.97. The backward procedure selected the variables of percent passing the No. 200 sieve, viscosity, percent crushed particles, average in-place VTM, GTM VTM, GTM VMA, creep, stability, and stability/flow ratio. The backward model has an R-square of 0.93.

Table 24. Summary of Stepwise Regression Analysis for GTM Recompacted Variables

Forward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.68)				
1	Passing #8	1	0.4373	0.4373
2	GSI	2	0.1219	0.5592
3	Avg. VTM	3	0.0206	0.5798
4	Creep	4	0.0308	0.6106
5	VMA	5	0.0254	0.6360
6	Passing #200	6	0.0103	0.6463
Binder Mix (All Variables R-square = 0.95)				
1	GTM VTM	1	0.4158	0.4158
2	Viscosity	2	0.1022	0.5180
3	Avg. VTM	3	0.0600	0.5780
4	Passing #8	4	0.0511	0.6290
5	Creep	5	0.0616	0.6906
6	Bearing Capacity	6	0.0437.	0.7344
7	Crushed Particles	7	0.0474	0.7818
8	GTM VMA	8	0.0287	0.8105
9	Stability	9	0.0565	0.8670
10	Passing #200	10	0.0657	0.9326
11	Stab/Flow	11	0.0173	0.9499
12	% AC	12	0.0091	0.9589
13	GSI	13	0.0095	0.9684

Table 24. Summary of Stepwise Regression Analysis for GTM Recompacted Variables (Continued)

Step	Variable Entered	Number In	Partial R-square	Model R-square
Forward Selection Procedure				
Wearing Mix (All Variables R-square 0.68)				
1	Stability	15	0.0000	0.6769
2	Avg. VTM	14	0.0000	0.6769
3	Flow	13	0.0005	0 6764
4	Penetration	12	0.0003	0:6761
5	Passing #200	11	0.0019	0.6742
6	Passing #8	10	0.0029	0.6713
7	GSI	9	0.0068	0 6646
8	Viscosity	8	0.0030	0:6616
9	% Natural Sand	7	0.0069	0.6547
10	Crushed Particles	6	0.0074	0.6473
11	GTM VMA	5	0.0398	0.6075
12	GTM VTM	4	0.0134	0.5942
13	Creep	3	0.0413	0.5528
14	Avg. VTM	2	0.0405	0.5123
Binder Mix (All Variables R-square = 0.95)				
1	Flow	14	0.0000	0.9450
2	% Natural sand	13	0.0003	0.9447
3	GSI	12	0.0013	0.9434
4	Penetration	11	0.0007	0.9589
5	% AC	10	0.0091	0.9499
6	Passing #8	9	0.0074	0.9425
7	Bearing Capacity	8	0.0169	0.9257

Table 25. Summary of Stepwise Regression Analysis for Static Base Recompacted Variables

Forward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.72)				
1	Passing #8	1	0.4378	0.4378
2	Stability	2	0.1606	0.5986
3	Avg. VTM	3	0.0356	0.6342
4	Static VMA	4	0.0557	0.6899
5	Natural Sand	5	0.0140	0.7039
Binder Mix (All Variables R-square = 0.97)				
1	Static VTM	1	0.3970	0.3970
2	Viscosity	2	0.1794	0.5764
3	Passing #8	3	0.0553	0.6317
4	Crushed particles	4	0.0448	0.6765
5	Avg. VTM	5	0.0424	0.7189
6	Static VMA	6	0.0473	0.7662
7	Flow	7	0.0472	0.8134
8	Creep	8	0.0403	0.8537
9	Passing #200	9	0.0299	0.8836
10	Natural Sand	10	0.0381	0.9217

Backward Selection Procedure

Step	Variable Entered	Number In	Partial R-square	Model R-square
Wearing Mix (All Variables R-square = 0.72)				
1	Stability	13	0.0000	0 7217
2	Bearing Capacity	12	0.0000	0:7217
3	Crushed Particles	11	0.0001	0.7216
4	Viscosity	10	0.0002	0.7214
5	Creep	9	0.0003	0.7211
6	Penetration	8	0.0009	0.7201
7	% AC	7	0.0028	0.7173
8	Passing #200	6	0.0046	0.7127
9	Static VMA	5	0.0036	0.7091
10	% Natural sand	4	0.0170	0.6922
11	Flow	3	0.0309	0.6612
Binder Mix (All Variables R-square = 0.97)				
1	% AC	13	0.0013	0.9652
2	Passing #8	12	0.0010	0.9642
3	Penetration	11	0.0044	0.9598
4	% Natural sand	10	0.0065	0.9534

Table 26. Summary of Stepwise Regression Analysis for Rotating Base Recompacted Variables

Step	Variable Entered	Number In	Partial R-square	Model R-square
Forward Selection Procedure				
Wearing Mix (All Variables R-square = 0.73)				
1	Passing #8	1	0.4393	0 4393
2	Stability	2	0.1333	0:5726
3	Rotating VMA	3	0.0544	0.6270
4	Average VTM	4	0.0649	0.6919
5	% Natural Sand	5	0.0144	0.7063
6	Rotating VTM	6	0.0087	0.7151
Binder Mix (All Variables R-square = 0.93)				
1	Rotating VTM	1	0.3970	0.3970
2	Viscosity	2	0.1794	0.5764
3	Passing #8	3	0.0553	0.6317
4	Crushed Particles	4	0.0448	0.6765
5	Average VTM	5	0.0424	0.7189
6	Rotating VMA	6	0.0473	0.7662
7	Creep	7	0.0343	0.8005
8	Stability	8	0.0380	0.8385
9	Stability/Flow	9	0.0617	0.9002
10	Penetration	10	0.0160	0.9162
11	% Natural Sand	11	0.0130	0.9292

Static base variables. The model for predicting rutting based on the fifteen static base variables has an R-square of 0.72 for the wearing mixes and 0.97 for the binder mixes. The results of the stepwise procedure for the static base variables are shown in Table 25. For the wearing mixes the forward procedure selected the percent passing the No. 8 sieve, the percent natural sand in the fine aggregate, average in-place VTM, VMA, and stability as all contributing significantly to the model with an R-square of 0.70. The backward procedure selected the variables of the percent passing the No. 8 sieve, VMA, average in-place WM, and the stability/flow ratio as the significant variables with an R-square of 0.66.

The forward procedure for the binder mixes selected all of the variables as significant with the exception of AC, stability and stability/flow ratio with an R-square of 0.92. The backward procedure selected all of the variables as significant with the exception of the percent passing the No. 8 sieve, AC, penetration, and the percent natural sand in the fine aggregate. The backward model has an R-square of 0.95.

Rotating base variables. The model for predicting rutting based on the fifteen rotating base variables has an R-square of 0.73 for the surface mixes and 0.93 for the binder mixes. The results of the stepwise procedure for the rotating base variables are shown in Table 26. For the wearing mixes the forward procedure selected the percent passing the No. 8 sieve, the percent natural sand in the fine aggregate, recompacted VTM, the average in-place VTM, VMA, and stability as all contributing significantly to the model with an R-square of 0.71. The backward procedure selected the variables of the percent passing the No. 8 sieve, average in-place VTM, VMA and the stability/flow ratio as the significant variables with an R-square of 0.67.

The forward procedure for the binder mixes selected all of the variables as significant with the exception of AC, percent passing the No. 200 sieve, and bearing capacity with an R-square of 0.93. The backward procedure selected all of the variables as significant with the exception of penetration, flow, percent natural sand, percent passing the No. 8 and No. 200 sieve, and percent AC. The backward model has an R-square of 0.87.

The above analysis shows that many variables contribute to rutting and that no one variable adequately predicts rut depths. Many of the variables utilized above, such as recovered penetration and viscosity, contribute to rutting. However, they can not be controlled or predicted during design and construction. A meaningful model to predict rutting would contain variables that both significantly contribute to the model and can be controlled during design and/or construction. Eight variables were selected to represent mix properties that are controllable during design and construction. The mix design variables were not utilized because they were not representative of the mix “as placed.” These eight variables are the 20th percentile VTM from cores C7-C11 to represent the mix design VTM; the VMA calculated from recompacted samples to represent mix design VMA; the percent passing the 1/2, No. 8 and No. 200 sieves and the percent crushed faces from the in-place cores (Cl -C5); the recompacted flow to represent the mix design flow; and the mix design stability. For the GTM samples, the GSI was included.

Two models for each mix type, wearing and binder, were developed for each of the three compaction methods. The first model utilized all eight (9 for the GTM) variables. The second model only utilized those variables that contributed significantly to the model.

GTM model. The model for predicting rutting based on all of the GTM variables has an R-square of 0.56 for the wearing mixes and 0.62 for the binder mixes. For the wearing mixes, the stepwise procedure identified the percent passing the No. 8 sieve, the recompacted flow and the GSI as contributing significantly with an R-square of 0.42 as shown in Figure 134. For the binder mix all variables except recompacted flow were significant with an R-square of 0.62 as shown in Figure 135.

Rotating base model. The eight selected variables for predicting rutting has an R-square of 0.38 b for the wearing mix and 0.49 for the binder mix. The stepwise procedures selected all variables as significant except recompacted flow for the wearing mix with an R-square of 0.37. The model is shown in Figure 136. For the binder mix the 20th percentile VTM, the percent crushed faces and the percent passing the 1/2" and the No. 8 sieves contributed significantly with an R-square of 0.48 as shown in Figure 137.

Static base model. The model for predicting rutting using the static base variables had an R-square of 0.37 for the wearing mix and 0.63 for the binder mix. The stepwise procedure identified the recompacted flow, percent crushed faces, and the percent passing the No. 8 sieve and No. 200 sieves as significant. The model using these four variables has an R-square of 0.34 and is shown in Figure 138. The model for the binder mix (Figure 139) has an R-square of 0.52 and contains the 20th percentile VTM, VMA, and percent passing the 1 /2" and the No. 8 sieves.

Only one variable appeared in each model for the wearing mixes. This variable was the percent passing the No. 8 sieve. For the binder mixes, the WM and the percent passing the l/2" and the No. 8 sieves appeared in each model. This would seem to indicate that the gradation is one of the most important parameters to control in preventing rutting. However, the gradations for the

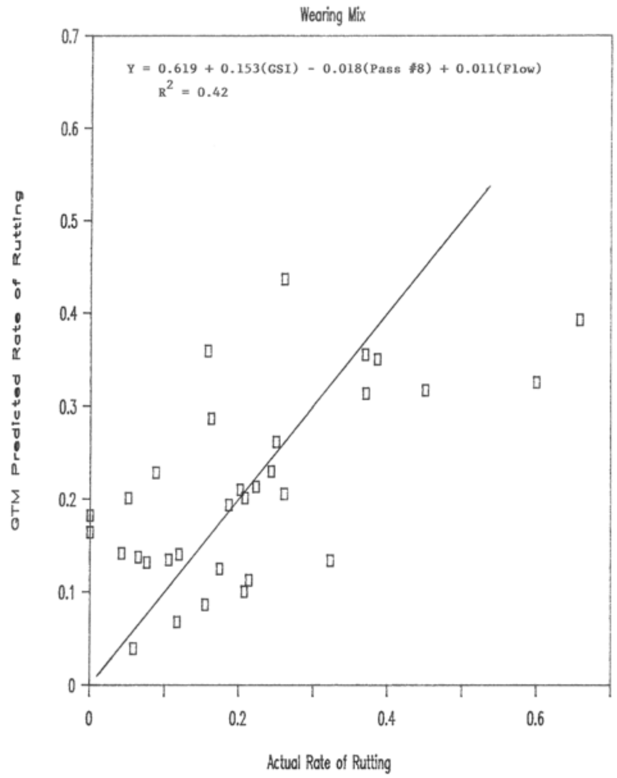

Figure 134. Rate of Rutting Model for GTM Compaction, Wearing

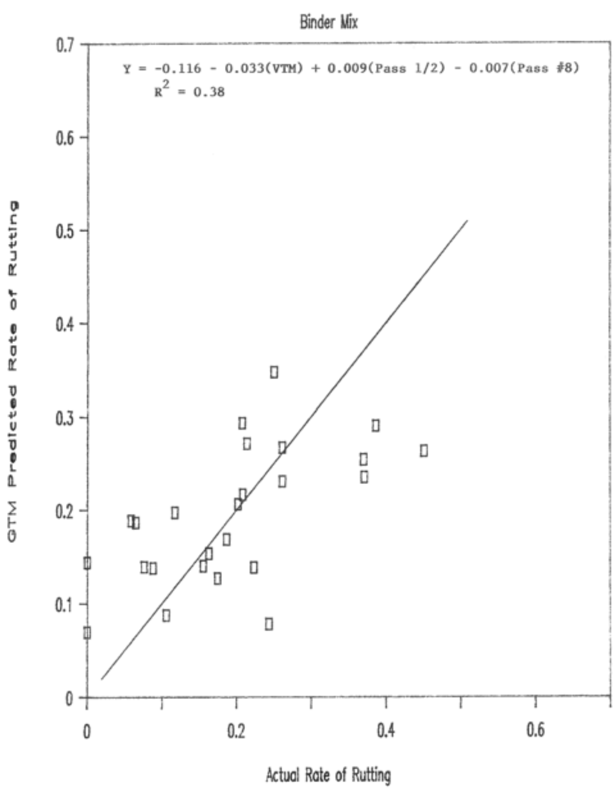

Figure 135. Rate of Rutting Model for GTM Compaction, Binder

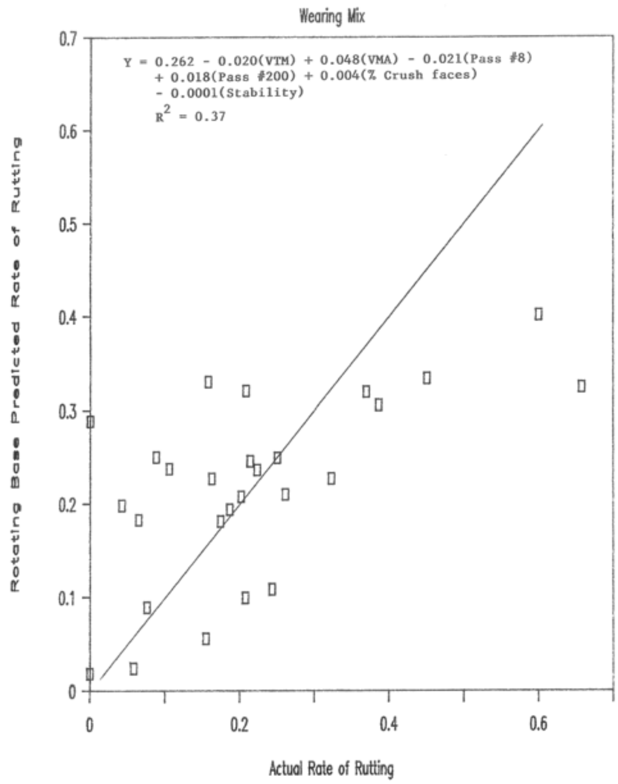

Figure 136. Rate of Rutting Model for Rotating Base Compaction, Wearing

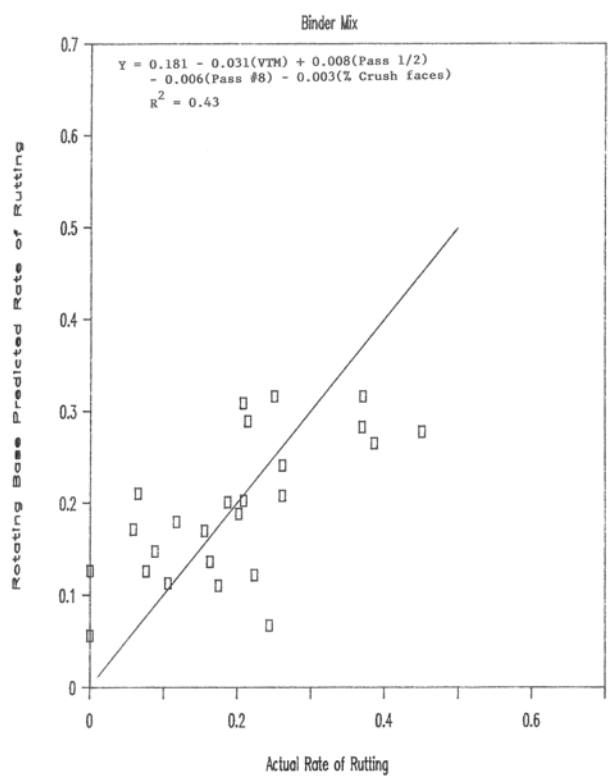

Figure 137. Rate of Rutting Model for Rotating Base Compaction, Binder

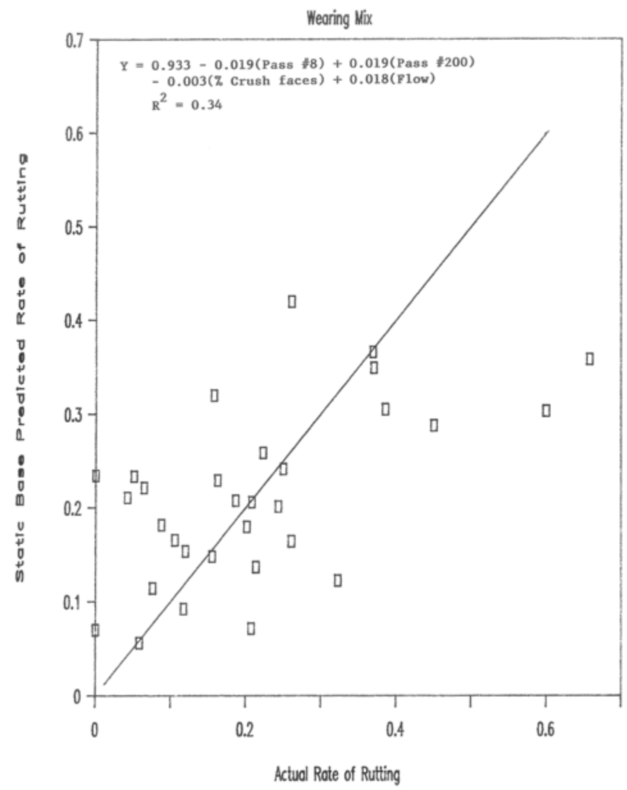

Figure 138. Rate of Rutting Model for Static Base Compaction, Wearing

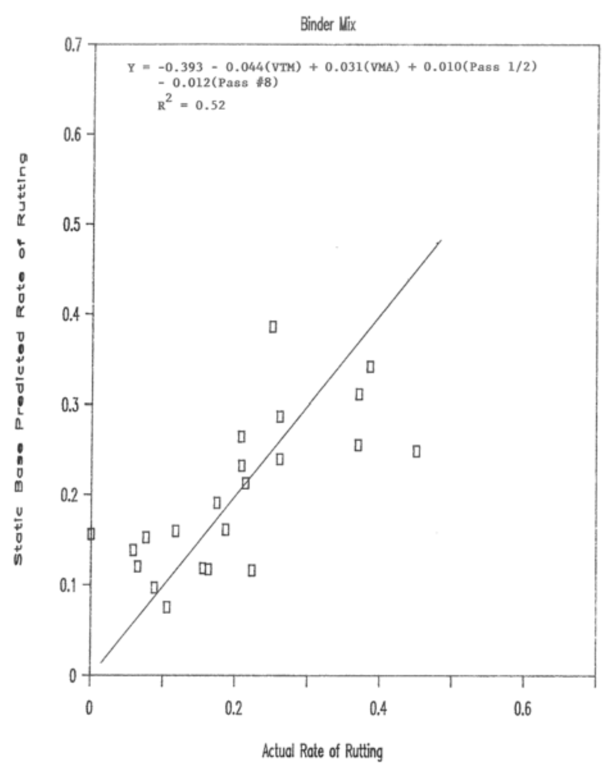

Figure 139. Rate of Rutting Model for Static Base Compaction, Binder

mixes were very similar, especially for the No. 8 sieve and therefore, the effect of this variable could be overstated in the model. The models are applicable over the ranges of the test data for the indicated variables and should not be extrapolated to other mixes or levels of traffic.

Summary. Obviously, rutting is a complex phenomenon as evidenced by the many independent variables selected by the stepwise procedure as significantly contributing to rutting. Each selected variable contributes significantly to rutting and, therefore, must be considered while designing the HMA mix and controlling HMA construction quality. Ideally, a simple, end-result test method capable of determining rutting potential is needed which can be used to design the HMA mix in the laboratory and control its quality on a daily basis in the field. Until such a test method is available it is prudent to use specifications for mix composition, mix design, and construction quality control, which are based on significant independent variables and their respective threshold values, to minimize the rutting problem.

Heavy Duty Specifications

Seven of the 34 projects evaluated were constructed using the heavy duty specifications implemented by PennDOT in 1987. These seven projects (Sites #5, 7, 24, 26, 32, 33 and 35) have been in service for only two-three years. The subjective rating and the average rate of rut development occurring in these heavy duty pavements are as follows:

Site #	Rating	Avg. Rut Depth/SQRT TESALs
4	F	N/A*
7	P	0.211
24	E	0.078
26	E	0.000
32	G	0.132
33	F	0.197
35	E	0.000
	Average	0.103

*Overlaid prior to profilometer testing

Four of the seven heavy duty pavements were rated good to excellent. Sites #5 and #33 received fair ratings and Site #7 a poor rating. The low ratings of Sites #5 and #7 could be attributed to low mix design air void contents (3.0 and 3.2 percent for Site #5 and 3.6 and 3.5 percent for Site #7 in the wearing and binder mixes, respectively). It should be noted that the heavy duty mix specifications were subsequently revised to require mix design air voids of not less than 4.0 and 4.5 percent for wearing and binder mixes, respectively. Site #32, which received a “good” rating contained 1.8 percent more minus 200 as placed than designed in the wearing mix. Site #33, which received a “fair” rating, had excessive minus 200 in both the wearing and binder mix (2.5 and 1.6 percent more than the JMF, respectively). The remaining three sites are rated as excellent.

The average rate of rutting of the heavy duty pavements is 0.103 inches per square root million ESALS for all seven sites, and 0.053 when Sites #5, 7 and 33 are excluded due to their low mix design VTMs. Both of these values are well below the threshold value of 0.20 determined earlier. It should be noted that the rate of rutting is also less than the average rate of rutting of 0.167 which includes poor to excellent sites.

The preceding data and discussion indicate that the current PennDOT heavy duty specifications have minimized the rutting problem. Further changes in the specifications, especially the HMA mix production quality control, based on the results of this study are expected to improve the resistance of PennDOT HMA mixes to rutting induced by high pressure truck tires and high traffic volumes.

SUMMARY AND CONCLUSIONS

This research project was undertaken to evaluate 34 in-service heavy duty pavements across Pennsylvania to identify the material properties, mix design parameters, pavement construction properties, and pavement in-service properties which are responsible for the premature rutting (permanent deformation) of some HMA pavements. Of the 34 projects, ten were excellent, nine were good, 12 were fair, and three were poor based on a subjective rating system which was validated in this study.

Traffic, mix design, and construction data was collected for all projects. The total estimated traffic carried by the pavements (ranging in age from two to 19 years) ranged from less than one million ESALS to over 30 million ESALS.

Eleven 6-inch diameter cores were taken from each project to determine the VTM (voids in total mix), creep (permanent deformation), mix composition (asphalt content and gradation), fractured face count of coarse aggregate, particle shape and texture of fine aggregate, and recovered asphalt penetration and viscosity. The cores were also reheated and compacted using three compaction methods: gyratory testing machine (GTM), rotating base-slanted foot mechanical Marshall compactor, and static base mechanical Marshall compactor. Recompacted specimens were tested for VTM, Marshall stability and flow.

Transverse surface profiles of the pavement were obtained at two locations: worst site and a representative site within 500 feet of the worst site. Maximum surface rut depth and the rut depth in individual layers were determined from the surface profile and the thickness of the layers measured from transverse sets of cores. The maximum surface rut depth at the worst location on all projects ranged from 0.04 inch to 1.66 inch. In a majority of cases the underlying layers in conjunction with the wearing course contributed to the surface rut depth.

Mix Design. The number of blows/face used was 50 for 24 projects, 65 for three projects (Turnpike), and 75 for 7 projects in designing the wearing mixes. Only seven projects of 34 projects had wearing mix design VTM equal to or greater than 4.0 percent. Of 26 binder mixes, 12 mixes had VTM equal to or greater than 4.0 percent. This indicates that both the wearing and binder mixes were designed closer to the minimum VTM value of the 3-5 percent range used by PennDOT.

Construction. Construction data indicates that the percentage of minus 200 material was generally higher in the “produced mix” compared to the “designed mix” for both wearing and binder mixes. In the case of the binder mixes, the percentage of material passing 1/2" and No. 8 sieve was also generally higher in the “produced mix” compared to the “designed mix” indicating that the “produced mix” was finer.

In-Service Properties. Excessive minus 200 in both wearing and binder mixes, and excessive material passing 1/2" and No. 8 sieves in binder mixes as reported during construction (testing of loose mixtures) was confirmed by the core test data. Average in-place VTMs in the wearing and binder courses were determined to be 3.2 and 3.0 percent which are significantly lower than lower than the mix design VTMs. This indicates that the laboratory compactive effort was inadequate and/or excessive fines created during construction filled the voids. Obviously, there are many projects which have VTMS lower than 3 percent. According to past experience HMA pavements approach the potential for rutting when the VTM is 3 percent or less.

Of the three compactors used to recompact the mix from the pavements, Marshall compactor with rotating base and slanted foot gave the highest density (least VTM) for both wearing and binder mixes. This compactor is recommended for use by PennDOT to obtain the near maximum potential compaction of mixes which is likely to be achieved in heavy duty pavements subjected to high pressure truck tires.

Average GSI (gyratory shear index) values of 1.35 and 1.26 for wearing and binder courses, respectively, are on the high side and indicate potential for rutting. Whereas a value of 1.00 is considered ideal to prevent rutting, values up to 1.20 may be acceptable.

Statistical Analysis

Some 60 independent variables covering the general design, construction and post construction data for each pavement were selected to determine their effect on rutting. The dependent variable selected for analysis was rut depth in inches divided by square root of total traffic in million ESALS. A threshold value of 0.2 for this dependent variable was determined in this study. Pavements are expected to develop undesirable amounts of rutting if this value is exceeded.

All data pertaining to the 60 independent variables and the dependent variable was analyzed using correlation analysis, linear regression analysis, and stepwise multiple variable analysis methods.

Since rutting is a complex phenomenon, no one independent variable alone could predict rutting with any degree of confidence. However, the following significant trends were observed and threshold values identified.

Mix Composition and Design. Rutting potential increased as (a) minus 200 content increased, (b) fractured face count of coarse aggregate decreased, (c) percentage of natural sand in the fine aggregate increased, (d) percentage of asphalt content increased, (e) Marshall mix design stability decreased, (f) mix design stability/flow ratio decreased, and (g) mix design bearing capacity of mix decreased. Threshold values to control the rutting are as follows:

	Wearing Mix	Binder Mix
Percent natural sand in the fine aggregate	Less than 20%	Less than 20%
Marshall mix design stability, lbs.	Above 2800	Above 2800
Design stability/flow ratio	Above 250	Above 275
Design bearing capacity	Above 275	Above 275

Threshold values of Marshall mix design stability, design stability/flow ratio, and design bearing capacity are considered high. These values (obtained from the job-mix formula) cannot be used because the “as placed” mixes were generally significantly different from the “as designed” mixes. Optimum pavement performance was generally observed when the percentage of material passing No. 8 sieve was 45-50 for wearing mixes, and 25-30 for binder mixes. The indicated maximum percentage of natural sand in the fine aggregate is 20. It is reasonably close to the present specification requirement of 25 percent which is considered adequate.

In-Service Properties. Rutiing potential increased as (a) in-place VTM decreased, (b) gyratory shear index (GSI) increased, (c) recompacted VMA decreased, (d) recompacted VTM decreased, (e) recompacted stability decreased, (f) recompacted stability/flow ratio decreased, and (g) recompacted bearing capacity decreased. Threshold values to control the rutting are as follows:

	Wearing Mix	Binder Mix
Average in-place VTM	Above 3.0%	Above 2.0%
GSI	Below 1.2	Below 1.2
Recompacted VMA	Above 15%	Above 12%
Static base recompacted stability, lbs.	Above 3400	Above 3600
Static base recompacted stability/flow	Above 280	Above 260
Static base recompacted bearing capacity	Above 300	Above 280

Again, the threshold values of stability, stability/flow and bearing capacity cannot be used because these were obtained on aged, recompacted mixtures.

RECOMMENDATIONS

Data from this research project indicates that the current PennDOT heavy duty specifications have minimized the rutting potential of HMA pavements in Pennsylvania. However, the following recommendations are made to improve and optimize the resistance of PennDOT HMA mixes to rutting induced by high pressure truck tires and increasing traffic volumes.

Materials

1. Coarse aggregate retained on No. 4 sieve. Continue to use at least 85 percent of particles with two or more fractured faces for wearing and binder courses.
2. Fine aggregate. Continue to use at least 75 percent manufactured sand in the fine aggregate for both wearing and binder courses. Encourage use of 100 percent manufactured sand if possible.
3. Size. Although limited data is available from this project to justify increasing the maximum size of aggregate for wearing and binder courses, it is prudent to do so based on nationwide experience. Use 1 1/2 inches maximum aggregate size (at least 5 percent retained on 1 inch) for binder courses. Encourage increased use of ID-3 wearing course (3/4 inch maximum aggregate size).

Mix Design

1. Mechanical Marshall compactor with rotating base and slanted foot (75 blows/face) gave the highest density (least air void content), for both wearing and binder courses, compared to the gyratory testing machine (GTM) and conventional static base Marshall compactor (75 blows/face). Rotating base/slanted foot Marshall compactor should be used, at least in the central laboratory, to obtain near maximum potential compaction of mixes which will likely to be achieved after two-three years’ traffic. This will minimize the potential over-asphalting of mixes designed for heavy duty pavements and high pressure truck tires.
2. Design mixes with at least 4.0 percent air voids when using rotating base/slanted foot Marshall compactor.
3. Current specifications for VMA, stability and flow appear adequate.

Mix Production Quality Control

1. Binder course mixes "as placed" were generally finer than mixes "as designed." On average, the percentages passing 1/2", No. 8 and No. 200 exceeded the job-mix formula values by 4.3, 2.5 and 1.0 percent, respectively. Wearing course mixes "as placed" have 1.1 percent (average) higher minus 200 than the job-mix formula. Better mix gradation control is necessary. If the quality control charts or historical RPS data indicate that the values are consistently on the high or low side of the JMF, the mix design should be revised to incorporate production gradation. If all RPS lots on a project get 100 percent payment, it does not mean necessarily that the mix is satisfactory from a rutting standpoint because the mix could consistently be finer than designed and still meet the specification requirements.
2. Air void content in laboratory compacted samples of "produced mix" is more important than that of the "designed mix." If the produced mix contains excessive minus 200 material, its air void content will be lower than the designed mix and, therefore, potential for rutting will increase. It appears from the preliminary review of the test data that the percentage of air void content obtained from the daily compacted Marshall specimens should be made a pay item in the RPS specifications in lieu of minus 200 material. A control of air void content will indirectly control the amount of minus 200 material in the mix. Some states use this approach because air void content of the daily compacted Marshall specimens is the most important parameter affecting rutting. Air void content should not be allowed to fall below 3.0 percent.
3. There is some indication of increased rutting potential if the freshly laid wearing course is subjected to high temperatures and channelized traffic for extended periods of time. Project construction traffic control should be planned in advance to minimize this effect. Whenever possible, the wearing course should not be placed until all binder courses have been completed.

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The opinions, findings, and conclusions expressed here are those of the authors and not necessarily those of the Pennsylvania Department of Transportation, the Federal Highway Administration, and the National Center for Asphalt Technology (NCAT) of Auburn University. This report does not constitute a standard, specification or regulation.

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APPENDIX

NAA METHOD FOR PARTICLE SHAPE & TEXTURE

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March 10, 1989

Proposed Method of Test for Particle Shape and Texture
of Fine Aggregate Using Uncompacted Void Content

1. Scope

1.1 This method covers the determination of the loose uncompacted void content of a fine aggregate for use as a measure of its angularity and texture.

1.2 Procedures are included for the measurement of void content using either a graded sand or through the use of several individual size fractions for void content determinations.

1.2.1 Graded Sample (Method A) -- Consists of 190 grams of a standard sand grading which can be obtained from the individual sieve fractions in a typical fine aggregate sieve analysis.

1.2.2 Individual Size Samples (Method B) -- Consists of 190 grams each of three fine aggregate size fractions: (1) No. 8 to No. 16; (2) No. 16 to No. 30; and (3) No. 30 to No. 50. For Method B each size is tested separately.

2. Summary

2.1 A 100 cm3 cylinder is filled with fine aggregate of prescribed gradation by allowing the sample to flow through a funnel from a fixed height into the calibrated cylinder. The cylinder is struck off and weighed. Uncompacted void content is calculated as the difference between the cylinder volume and the absolute volume of the measured weight of fine aggregate collected in the cylindrical container. It is calculated using the bulk dry specific gravity of the sand. Two runs are made on each sample and the results are averaged.

2.1.1 For the graded sample (Method A) the void content so determined is used directly.

2.1.2 For the individual size fractions (Method B), the mean void content percent is calculated using the void content results from tests of each of the three individual size fractions: No. 8 to 16, No. 16 to 30, and No. 30 to 50.

3. Significance and Use

3.1 This procedure provides a numerical result in terms of percent void content determined under standardized conditions which correlates with the particle shape and texture properties of a fine aggregate. An increase in void content by this procedure indicates greater angularity and rougher texture. Lower void content results are associated with more rounded smooth sands.

3.2 The void content determined on the standard graded sample is not directly comparable with the average void content of the three individual size fractions from the same sample tested separately. single size particles have higher void contents than graded samples. Therefore, use either one method or the other as a measure of shape and texture; and identify which method is applicable with respect to reported data.

3.2.1 The graded sample (Method A) is most useful for a quick test which indicates the particle shape properties of a graded fine aggregate.

3.2.2 Obtaining and testing individual size fractions (Method B) is more time consuming than the graded sample.

3.2.3 Generally, the bulk dry specific gravity of the sand, graded as received, is used for calculating the void content. Occasionally, if the mineralogy of the size fractions varies markedly it may be necessary to determine the specific gravity of the size fraction used.

3.3 Void content information from either of the two procedures will be useful as an indicator of properties such as: the mixing water demand of portland cement concrete; in asphaltic concrete the effect of the fine aggregate on stability and voids in the mineral aggregate; or the stability of the fine aggregate phase of a base course aggregate.

4. Applicable Documents

4.1 ASTM Standards

4.1.1 Method C 128 for fine aggregate specific gravity

4.1.2 Method C 136 for sieve analysis of aggregate

4.1.3 Method C 117 for Minus No. 200 in aggregate.

5. Apparatus

5.1 Funnel -- The lateral surface of the right frustum of a cone sloped 60 ± 4° from the horizontal with an opening of 0.375 ± 0.025 in. (9.5 ± 0.6 mm) in diameter. The funnel shall be smooth on the inside and at least 1.5 in. (38 mm) high[1]. It shall have a volume of at least 200 cm^3 or shall be provided with a supplemental container to provide the required volume.

5.2 Funnel stand -- A support capable of holding the funnel firmly in position with its axis collinear with the axis of the measure and the funnel opening 4.5 ± 0.1 in. (114 ± 3 mm) above the top of the cylinder. A suitable arrangement is shown in Figure 1.

5.3 Measure -- A right cylinder of approximately 100 cm^3 capacity having an inside diameter of 1.52 ± 0.05 in. (38.6 ± 1.3 mm) and an inside height of approximately 3.37 in. (85.6 mm) made of drawn copper water tube meeting ASTM Specification B 88 Type M[2] or equally rigid material. The bottom of the measure shall be at least 0.25 in. (6.3 mm) thick, shall be firmly sealed to the tubing, and shall be provided with means for aligning the axis of the cylinder with that of the funnel.

5.4 Pan -- A shallow metal or plastic pan of sufficient size to contain the funnel stand. The purpose of the pan is to catch and retain sand grains that overflow the measure during filling or strike off.

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[1] Pycnometer top C 9455 sold by Hogentogler and Co., Inc., 9515 Gerwig, Columbia, Maryland 21045, 301-381-2390. Appears to be satisfactory, except that the size of the opening may have to be enlarged slightly, and any burrs or lips that are apparent should be removed by light filing or sanding.

[2] Type M copper drain, waste and vent pipe should have outside and inside diameters of approximately 1.63 (41.4 mm) and 1.52 (38.6 mm) inches, respectively.

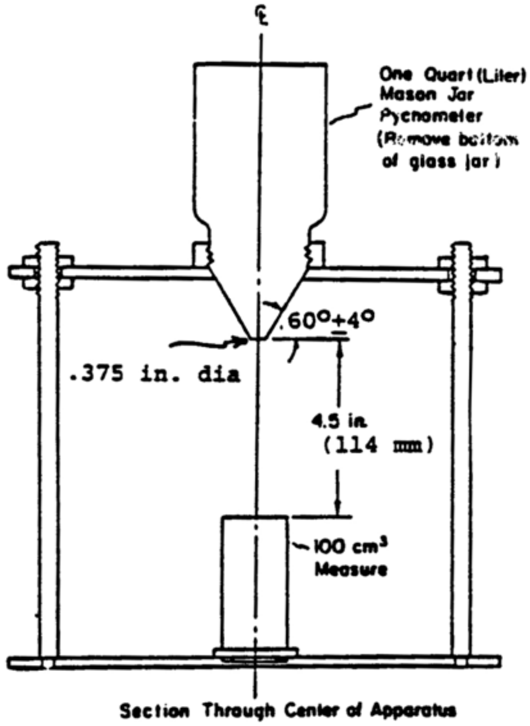

Figure 1

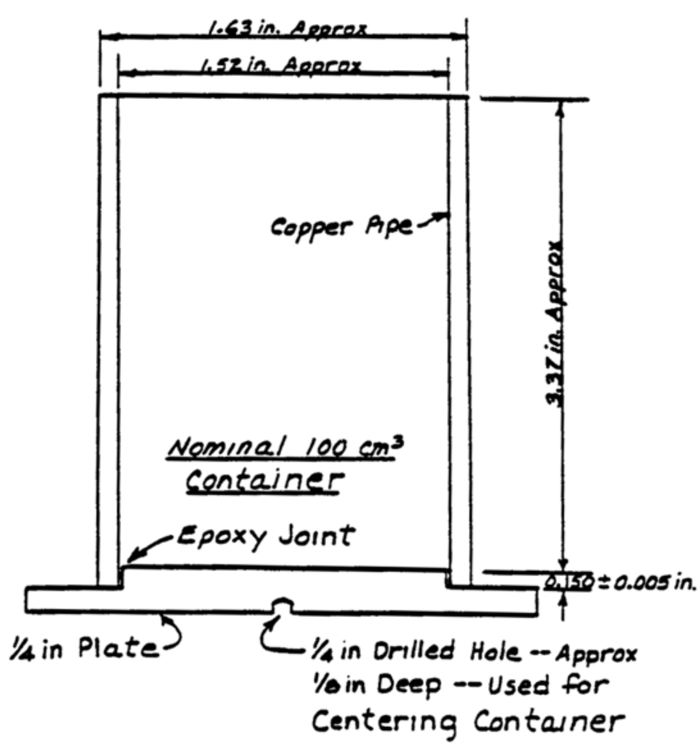

Figure 2

5.5 A metal spatula about 4 in. (100 mm) long with sharp straight edges. The end shall be cut at a right angle to the edges. The straight edge of the spatula is used to strike off the fine aggregate.

5.6 Scale or balance capable of weighing the measure and its content to ± 0.1 g.

6. Calibration of Measure

6.1 Weigh the dry, empty measure with a flat, glass plate slightly larger than its diameter and with the top edge of the container lightly coated with grease. Fill the measure with water at a temperature of 65 to 75°F (18 to 24°C). Place the glass plate on the measure, being sure that no air bubbles remain. Dry the outer surfaces of the measure and determine the combined weight of measure, glass plate, grease and water.

6.2 Calculate the volume of the measure as follows:

$$V = \frac{W}{0.998}$$

where,

V = volume of cylinder in cm^3
W = net weight of water in grams

7. Sampling

7.1 The sample(s) used for this test shall be obtained from a completed sieve analysis of aggregate by Method C 136 after washing as required in ASTM C 117. Maintain the necessary size fractions obtained from one or more sieve analyses in a dry condition in separate containers for each size.

8. Preparation of Test Samples

8.1 Method A - Graded Sample -- weigh out and combine the following quantities of dry sand from each of the sizes:

Individual Size Fraction	Weight, g
No. 8 to No. 16	44
No. 16 to No. 30	57
No. 30 to No. 50	72
No. 50 to No. 100	17
	190

The tolerance on each of these weights is ± 0.2 g. Mix the test sample until it appears homogenous.

8.2 Method B - Individual Size Samples -- Prepare a separate 190 g sample of dry fine aggregate for each of the following size fractions:

Individual Size Fraction	Weight, g
No. 8 to No. 16	190
No. 16 to No. 30	190
No. 30 to No. 50	190

The tolerance on each of these weights is ± 1 g. Do not mix these samples together. Each size is tested separately.

9. Procedure

9.1 If the sand has become moist, dry the sand to the constant weight in accordance with Method C 136 and cool to room temperature.

9.2 Mix the test sample until it appears homogeneous. Using a finger to block the opening, pour the test sample into the funnel. Center the funnel over the measure, remove the finger, and allow the sample to fall freely into the measure.

9.3 After the funnel empties, remove excess sand from the measure by a single pass of the spatula with the blade vertical using the straight part of its edge in light contact with the top of the measure. Until this operation is complete, exercise care to avoid vibration or disturbance that could cause compaction of the fine aggregate in the measure. Brush adhering grains from the outside of the measure and weigh the measure and contents to the nearest 0.1 g. Retain all sand grains.

Note 1 -- After strike-off the measure may be tapped lightly to compact the sample to make it easier to transfer the measure to scale or balance without spilling any of the sample.

9.4 Collect the sample from the retaining pan and measure, and repeat the procedure again.

9.5 For each run record the weight of the container and sand. Also record the weight of the empty measure.

10. Calculation

10.1 Calculate the uncompacted voids f or each determination as follows:

$$U = \frac{V - \frac{W}{G}}{V} \times 100$$

where,
V = volume of measure in cm^3
W = net weight of f ine aggregate in measure (Gross weight minus the weight of the empty measure)
G = bulk dry specific gravity of fine aggregate measured in accordance with Method C 128, Test for Specific Gravity and Absorption of Fine Aggregate.
U = uncompacted voids, percent.

Note 2 -- For most aggregate sources the fine aggregate specific gravity does not vary much from sample to sample or from size to size in the minus No. 8 fraction. Therefore, unless there is reason to believe that the specific gravity of individual sizes is appreciably different, it is intended that the value used in this calculation may be from a routine specific gravity test of an asreceived grading of the fine aggregate. If significant variation between different samples is expected then specific gravity should be determined on material from the same field sample from which the uncompacted void content sample wAs derived. Normally the asreceived gradation can be tested for specific gravity, particularly if the No. 8 to No. 100 size fraction represents more than 50 percent of the as-received grading. However, it may be necessary to test the graded No. 8 to No. 100 sizes f or specif ic gravity f or use with the graded void sample (Method A) or the individual size fractions for use with the individual size method (Method B). A difference in specific gravity of 0.05 will change the calculated void about one percent.

10.2 For the Graded Sample (Method A) calculate the average uncompacted voids for the two determinations and report the result as U_G.

10.3 For the Individual Size Fractions (Method B) calculate:

10.3.1 First, the average uncompacted voids for the two determinations made on each of the three size-fraction samples:

U_l = Uncompacted Voids, No. 8 - 16, percent

U_2 = Uncompacted Voids, No. 16 - 30, percent

U_3 = Uncompacted Voids, No. 30 - 50, percent

10.3.2 Second, the mean uncompacted voids (U_m) including the results for all three sizes:

$$U_m = \frac{U_1 + U_2 - U_3}{3}$$

11. Report

11.1 For the Graded Sample (Method A) report:

11.1.1 The Uncompacted voids (U_G) in percent to the nearest onetenth of a percent.

11.1.2 The Specific gravity value used in the calculation and whether it was determined on: (a) another sample from the same source, (b) as-received gradation from this sample, or (c) regraded sand from this sample.

11.2 For the <u>Individual Size Fractions</u> (Method B) report the following percent voids to the nearest one-tenth of a percent:

11.2.1 Uncompacted Voids for size fractions No. 8-16 (U_l) , No. 16-No. 30 (U_2), and No. 30-No. 50 (U_3).

11.2.2 Mean Uncompacted Voids (U_m).

11-2.3 Specific gravity value(s) used in the calculations, and whether the specific gravity value(s) were determined on: (a) another sample from the same source (b) as-received gradation from this sample, or (c) individual size fractions from this sample.

12. Precision

12.1 <u>Within Laboratory</u> -- Analysis of within laboratory data from sixteen laboratories which made void content tests on independent samples of three similar sources of rounded sands, graded in accordance with the graded standard sand in C 778, resulted in a within laboratory standard deviation (1S) of 0.13 percent voids for repeat determinations on the same sample. Differences greater than 0.37 percent voids between duplicate tests on the same sample by the same operator should occur by chance less than 5 percent of the time (D2S limit).

12.2 Multi-Laboratory -- Analyses of data from sixteen laboratories which made void content tests on independent samples of three similar sources of rounded sands, graded in accordance with the graded standard sand in C 778, resulted in a multilaboratory standard deviation (1S) of 0.33 percent voids. Since this value includes random variance due to the difference in samples, the standard deviation for multi-laboratory tests on the same sample should be lower. Differences greater than 0.93 percent voids between tests in two different labs should occur by chance less than 5 percent of the time (D2S limit) for these rounded sands.

12.3 Additional precision data is needed for tests of sands having different levels of angularity and texture tested in accordance with both procedures included in this Method.

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