in ESAL's per year is about 9°A 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 ESAL's.

The average daily traffic ranged from 5925 to 41,000 vehicles per day, and the ESAL's ranged from 440 to 9288 per day. Of the 34 projects, only 2 (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 ESAL's per day. The projects included 10 sites on interstate highways, 3 sites on Pennsylvania Turnpike, and 3 sites each on heavily travelled 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 1 million ESAL's to over 30 million ESAL's.

Average yearly temperatures for all project sites are also given in able 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) and the binder course (Layer 2). The data includes asphalt content, selected gradation, number of blows/face used, specimen specific gravity, maximum specific gravity, % VTM (voids in total mix), % VMA (voids in the mineral aggregate), % VFA (voids filled with asphalt), Marshall stability and flow. An ID-2W mix (a dense graded wearing course mix with 1/2" 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" top size).

Similarly, an ID-2 B (a dense graded binder course mix with I "topside) was used in Layer 2 except Projects 11 and 12 which used a BCBC mix (a dense graded base course mix with 1' 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"). 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 3 projects (Pennsylvania Turnpike), and 75 for 7 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 7 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 6to 16.

Binder course mix (Layer 2): Excluding the ID-2W courses, the asphalt content forthe 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 7 projects with no data being available for 6 projects. The following mix design data for layer 2 mixtures is of interest:

	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 % air voids or VTM, asphalt content, and the material passing 1/2", No. 8 and No. 200 sieves. The projects constructed under the RPS (restrioted 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 (CI) for various properties.

The conformal index (CI) expresses deviations from the JMF and indicates the relative target miss and affords the opportunity to evaluate mixes of different JMF's (6). 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.

3. The percentage of minus 200 material in the "produced mix" was mostly higher than the "designed mix" for both courses with average CI valuesof+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 1-way and 2-way.

Longitudinal Cores (C1-C5) Test Data

As mentioned in the sampling and testing plan five six-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 Fig. 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 mre test data:

1. Asphalt content (JMF, mean, standard deviation, and Cl)
2. Passing 1/2" (JMF, mean, standard deviation, and Cl)
3. Passing No. 8 (JMF, mean, standard deviation, and Cl)
4. Passing No. 200 (JMF, mean, standard deviation, and Cl)
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)
11. Type of manufactured sand (such as dolomite and sandstone)

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 #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 & 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 (I/TM): 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 HMA 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
Range	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 2 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 the aggregate recovered from Cores Cl -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
5. Fine aggregate particle shape & 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 O 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. VTM (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 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% higher and 20% 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 VTM 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 & 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 2-3 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" 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 ft 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 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 ft. 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	34	34
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 (10 Projects)
Projects in which the underlying layers were primarily responsible for the total surface rut depth.	3 and 5 (2 Projects)
Projects where most rutting contributed by the wearing course only	9, 11 and 16 (3 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 2-foot intervals at only one 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 (TESAL's) carried by the HMA overlays as of 1990. The last column gives the values of surface rut depth at 500 ft divided by the square root of TESAL's. 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 atypical 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.'

STATISTICAL ANALYSIS OF DATA AND DISCUSSION OF RESULTS

Independent Variables

Five broad categories of sixty (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).
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 ESAL's. 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 #8 and #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:

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

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", #8 and #200 sieves, and the conformal index (Cl) for the AC and percent passing the 1/2", #8 and #200 sieves. It was anticipated that lower VTM, higher percent passing the 1/2", #8 and #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 1-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 2-way.

Post construction longitudinal variables. These sixteen variables were obtained from cores C1-C6 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 inch, #8 and #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, #8 and #200 sieves were also investigated. It was anticipated that higher percent crushed particles, higher voids in the fine aggregate and lower percent natural 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, 1 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 ESAL's to over 30 million ESAL's. By dividing the rut depth by some function 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, 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 maximum surface rut depth (within 500 ft. of the worst site), and each of the above variables divided by the square root of the total traffic expressed as million 18-kip ESAL's (TESAL's). 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 labelled 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.

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 2,3 and 4 layer pavements investigated. The results show than on average only 60% of the total rut observed can be attributed or explained by the properties of the surface mix and only 40% 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, O°A natural sand and 10OOA 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 ($|R| \geq 0.5$) from the correlation analysis for the independent variables will be briefly discussed below.

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 ESAL'S 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= O.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 (Cl -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 1 pound or more 75% of the time, was within 1 pound 10% of the time and less than the mix design 15% of the time for the wearing mixes. For the binder mixes, the in-place unit weight exceeded the mix design unit weight by one pound or more 50°A of the time, was within 1 pound 10% of the time and less than 40% of the time.

Figures 89 and 90 show the comparisons of the mix design construction, and post construction in-place (Cl -C6) asphalt contents for the wearing and binder mixes, respectively.

For the wearing mixes, 24% of the in-place asphalt contents were more than 0.4% below the mix design asphalt content and 76% were within ± 0.4% of the mix design asphalt content. However, only 2 of the 33 mixes were above the mix design asphalt content and both of them were within 0.1 % 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 (Cl -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 #8 and #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.

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 VTM'S 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% confidence level for both the VTM'S 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 (Cl -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 VTM's is caused by incomplete or "unbalanced" mix design VTM and unit weight data.

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 VTM'S are significantly lower than the mix design VTM's. 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 2-3 years traffic.