Subbase Permeability and Pavement Performance

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This project demonstrated that open-graded, permeable, subbase materials can be designed that provide adequate constructibility and pavement support as well as good internal drainage at a competitive cost. Five types of subbases ranging from an impermeable cement stabilized material to a very permeable, uniformly graded, crushed aggregate were evaluated during this 7-yr research project. Testing indicated that the open-graded subbases had adequately high permeabilities, but the dense-graded subbase permeability was unsatisfactorily low. Deflection measurements, indicating the relative strength of the pavement structure in each subbase section, were made using a falling weight deflectometer. The lowest pavement deflections were found in the aggregate cement section. Deflections in the asphalt treated permeable material (PA No. 2B aggregate) and high permeability aggregate sections were approximately equal to each other and slightly higher than those measured in the aggregate cement section. The highest deflections were measured in the PA No. 2A aggregate section. The results of this evaluation infer that dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbases all influence joint efficiency measurements. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but is influenced by a combination of factors in the pavement structure and environment during testing. Roughness measurements showed smooth pavements can be built on all five subbase types. After 7 yr, no significant difference in riding characteristics exists among the five subbase sections.

The Pennsylvania Department of Transportation (PennDOT) specification for crushed, densely graded subbase (PA No. 2A aggregate) was developed over a number of years as a result of extensive testing, evaluation, and field performance monitoring. The gradation specifications were chosen to provide

- The necessary strength and stability to support construction equipment, the pavement, and subsequent traffic;
- Drainage; and
- A material that could be manufactured with adequate quality control at reasonable cost (*1*).

The specification was developed as a compromise that would best meet the above criteria.

Numerous problems of premature pavement and shoulder distress were reported to be a result of excess water in the pavement system, and questions were raised as to whether PA No. 2A subbase was adequate to provide sufficient drainage for the pavement system. Laboratory permeability tests of the PA No. 2A subbase indicate a range in the coefficient of permeability of 2.8 to 0.028 ft per day (0.001 to 0.00001 cm/s). This range of permeability represents a very slow to nearly impermeable material. Field permeability of the subbase is varied because of in situ gradation variations at the same job site and appears to be higher in some cases than indicated by the laboratory tests. Field permeability tests conducted as part of a nationwide study by L. K. Moulton and Roger Seals of West Virginia University on PennDOT subbase indicate permeabilities in the range of 280 to 28 ft per day (0.1 to 0.01 cm/s) (*2*).

This experimental project was originally devised to demonstrate the feasibility of providing good construction and pavement support as well as good internal drainage at a competitive cost to PA No. 2A subbase. An additional, long-term objective of this research project was to determine the significance of the permeability of subbase materials on pavement performance. The subbases were chosen to represent a range in permeability from impermeable to very permeable.

A more detailed discussion of the construction and material testing of the subbases on this project may be found in PennDOT's Interim Report (*1*) and Final Report (*3*) on Research Project No. 79-3. Some information included in those earlier reports is again presented in this report.

Based on positive interim results obtained since this research project began, PennDOT has changed its specifications to require open-graded subbase (OGS) as an interlayer between rigid pavements and PA No. 2A aggregate subbase.

SITE DETAILS

The project field site is located on PA Traffic Route 66 (Pennsylvania Legislated Route 203) and U.S. Traffic Route 422 (Pennsylvania Legislated Route 1037) in Armstrong County, Pennsylvania. These routes have approximately 10,000 average daily traffic (ADT) with 7 percent trucks. A location map showing the project location is shown in Figure 1.

Five sections of base/subbase materials representing a range of permeability conditions from impermeable to very permeable were constructed. Following are listed the five material sections. (They are also shown by number in Figure 2.)

1. Aggregate cement,
2. Asphalt treated permeable material,
3. PA No. 2B aggregate subbase,
4. High permeability subbase, and
5. PA No. 2B aggregate.

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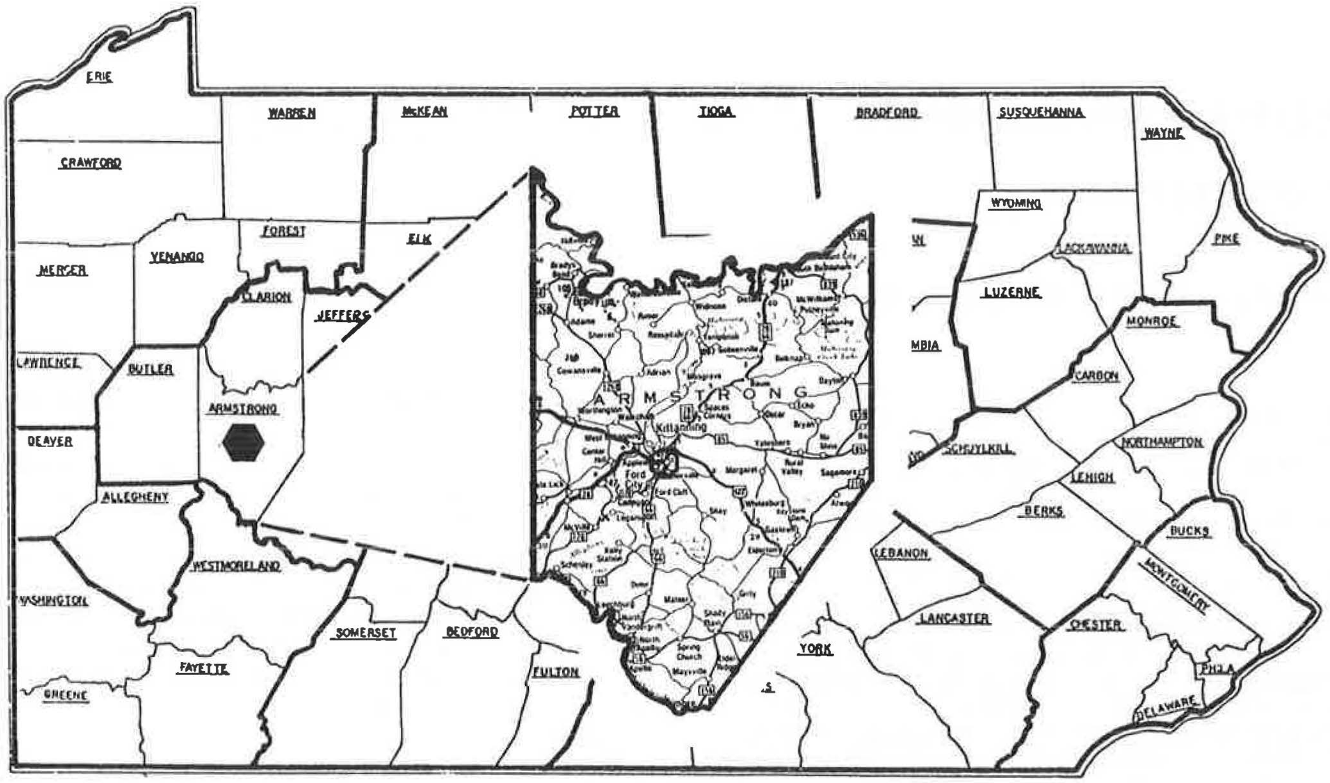

FIGURE 1 Project site location.

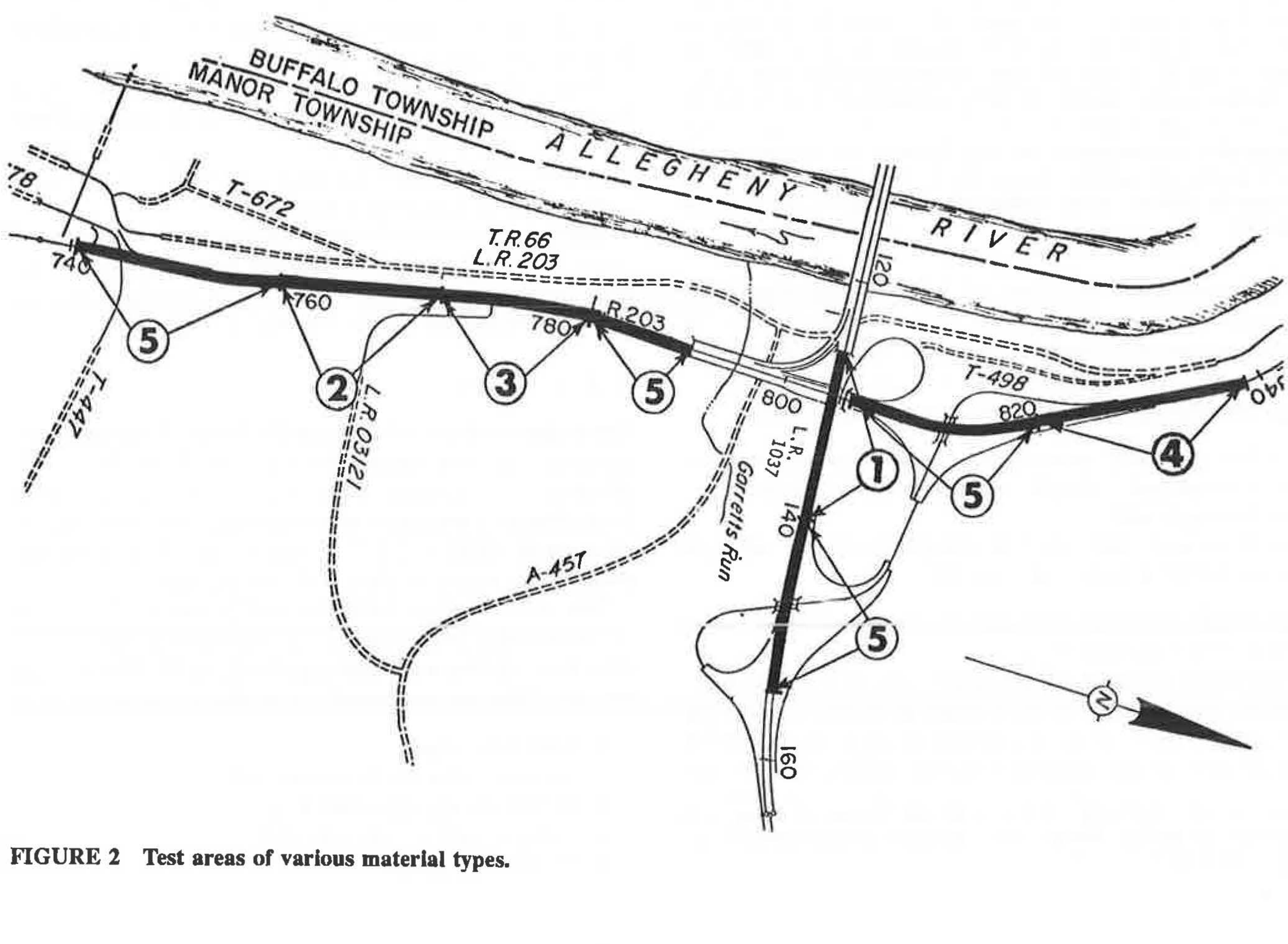

FIGURE 2 Test areas of various material types.

The standard design used for the control sections included placement of 10 in. of reinforced concrete pavement (RCP) on 13 in. of dense-graded aggregate subbase (PA No. 2A subbase). Graphite-coated round dowel bars were placed at each transverse joint. The dowels were 1¼ in. in diameter, 18 in. long, and spaced 1 ft apart. In the experimental sections, other materials were placed as an interlayer between the PA No. 2A subbase and the RCP, as shown in the pavement cross sections (Figures 3 through 7). The total thickness of the experimental interlayer and the subbase was 13 in. for all sections. Each experimental section was between 1,000 and 1,700 ft long and was constructed in adjacent sections in both the northbound and southbound lanes of the four-lane divided highway.

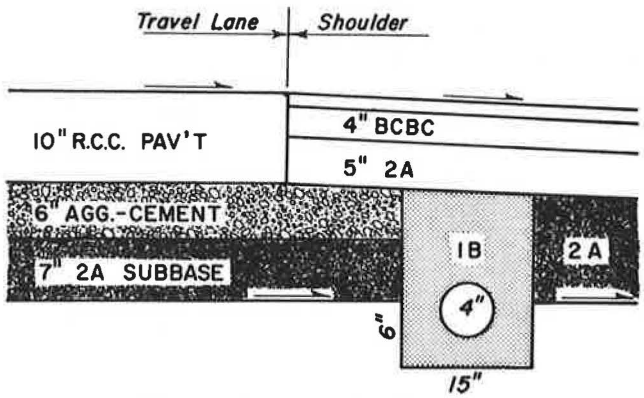

FIGURE 3 Pavement cross section (aggregate cement test area).

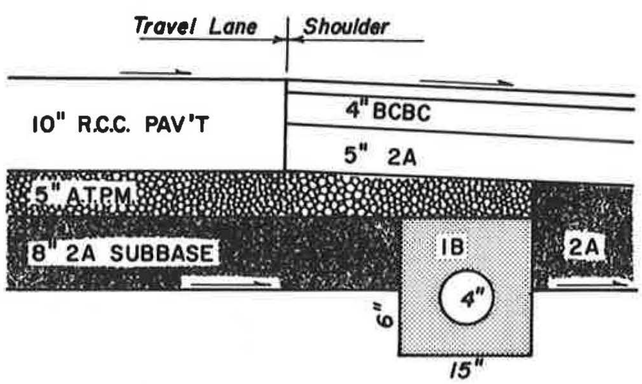

FIGURE 4 Pavement cross section (asphalt treated permeable material test area).

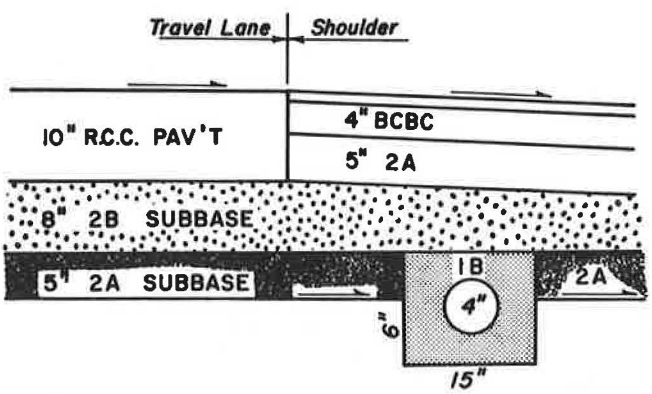

FIGURE 5 Pavement cross section (2B aggregate test area).

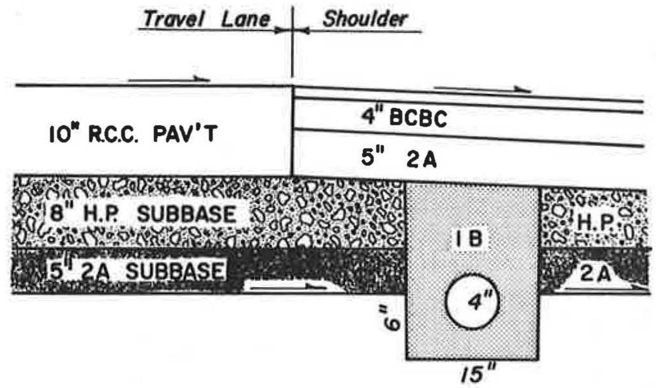

FIGURE 6 Pavement cross section (high permeability test area).

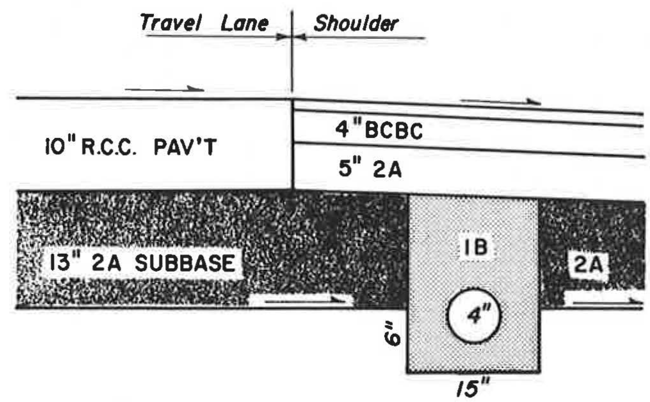

FIGURE 7 Pavement cross section (2A aggregate test area).

MATERIAL PROPERTIES

Laboratory Testing

Laboratory testing was done to determine the particle size distribution curves, the maximum dry densities, and the corresponding permeabilities of each of the five material types. California bearing ratio (CBR) testing was also performed to determine the relative difference in stability between the PA No. 2A aggregate subbase and the high permeability material. All aggregate material was a crushed glacial sand and gravel that was shipped from Davison Sand and Gravel's Tarrtown Flats source.

The specified gradation limits (dashed lines) are shown along with the actual particle size distribution curves in Figures 8 through 12, respectively, for the aggregate cement gradation, the asphalt treated permeable material (ATPM), the unstabilized PA No. 2B (2B) gradation, the high permeability (HP) gradation, and the PA No. 2A (2A) gradation. The ATPM aggregate gradation is equivalent to the 2B gradation. The 2A and the HP materials are both well-graded, but the HP material has coarser fragments than the 2A material throughout its entire particle size distribution. The 2B gradation band is narrow, and this material is uniformly sized. The 2B gradation is comparable to the AASHTO 57 gradation.

Laboratory densities, porosities, and permeabilities are tabulated in Table 1. A source specific gravity of 2.61 was used for all calculations.

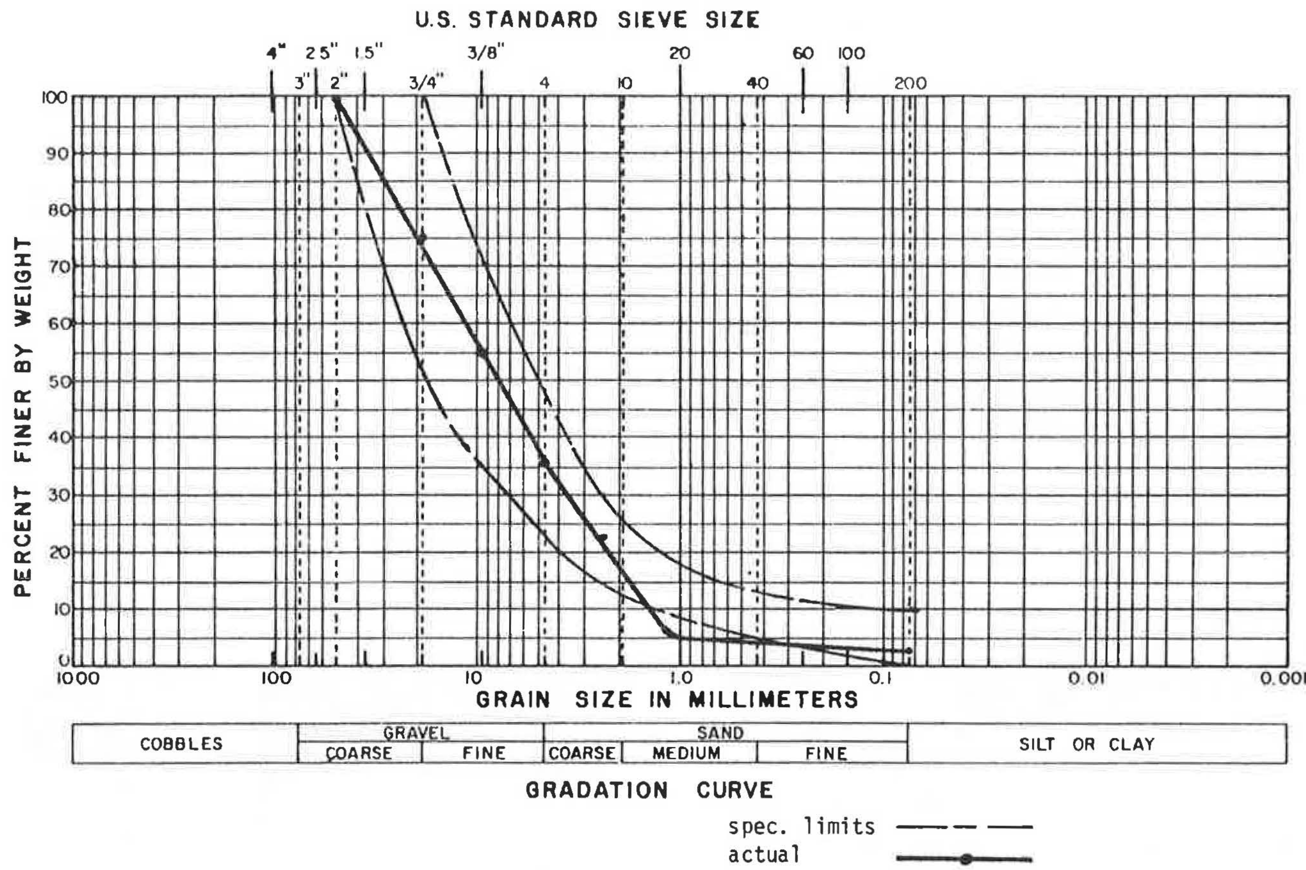

FIGURE 8 Aggregate cement (6 percent) gradation curve: aggregate only.

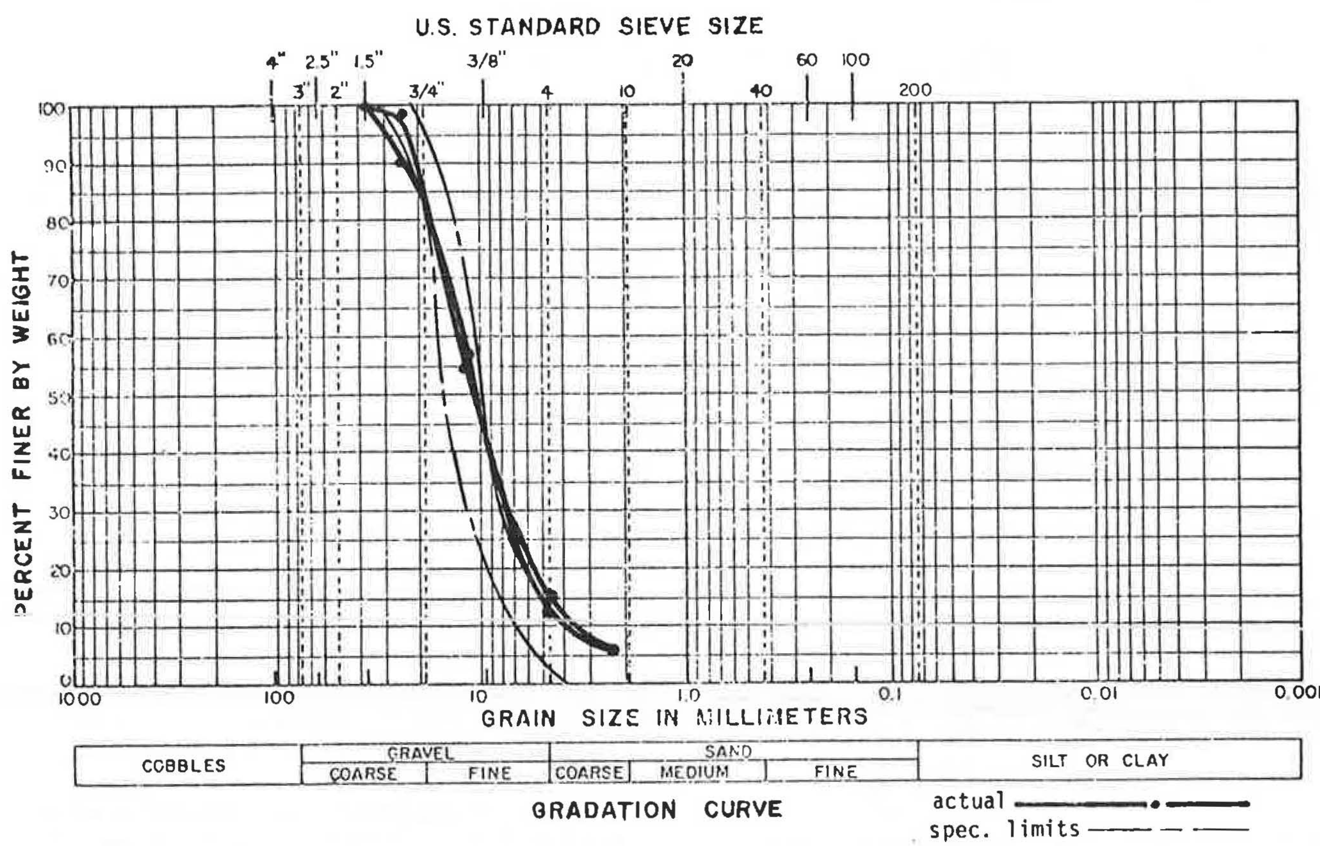

FIGURE 9 Asphalt treated permeable material gradation curve.

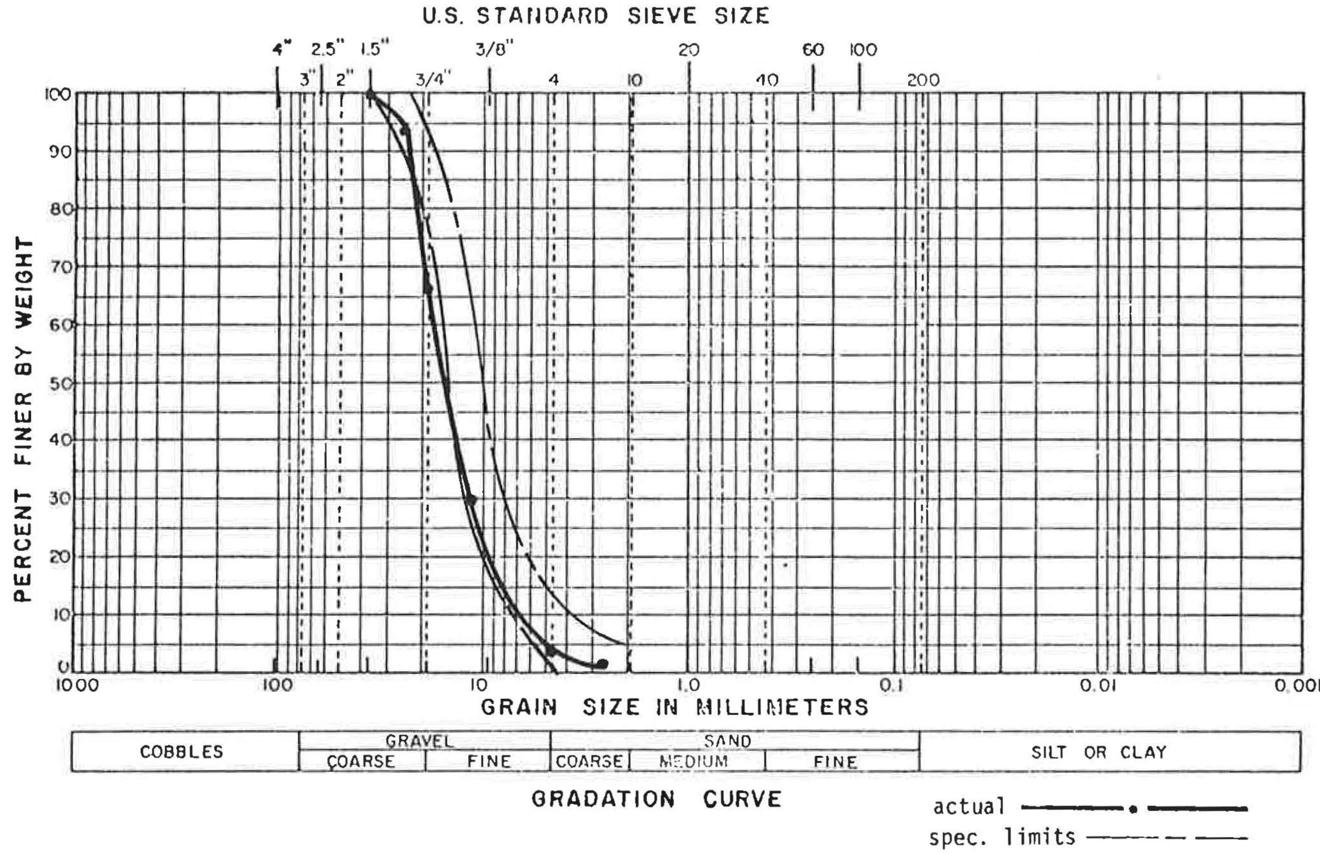

FIGURE 10 PA No. 2B gradation curve.

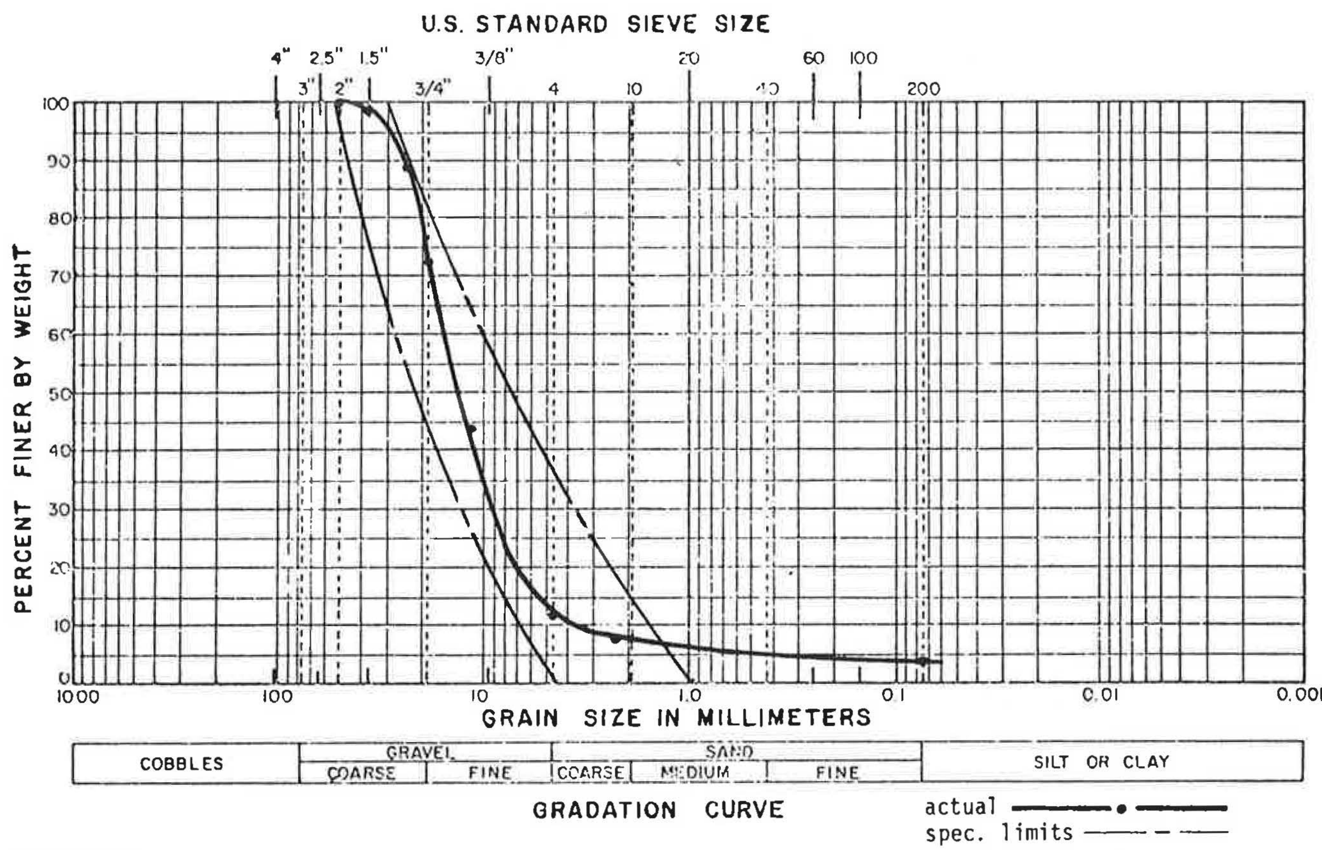

FIGURE 11 High permeability gradation curve.

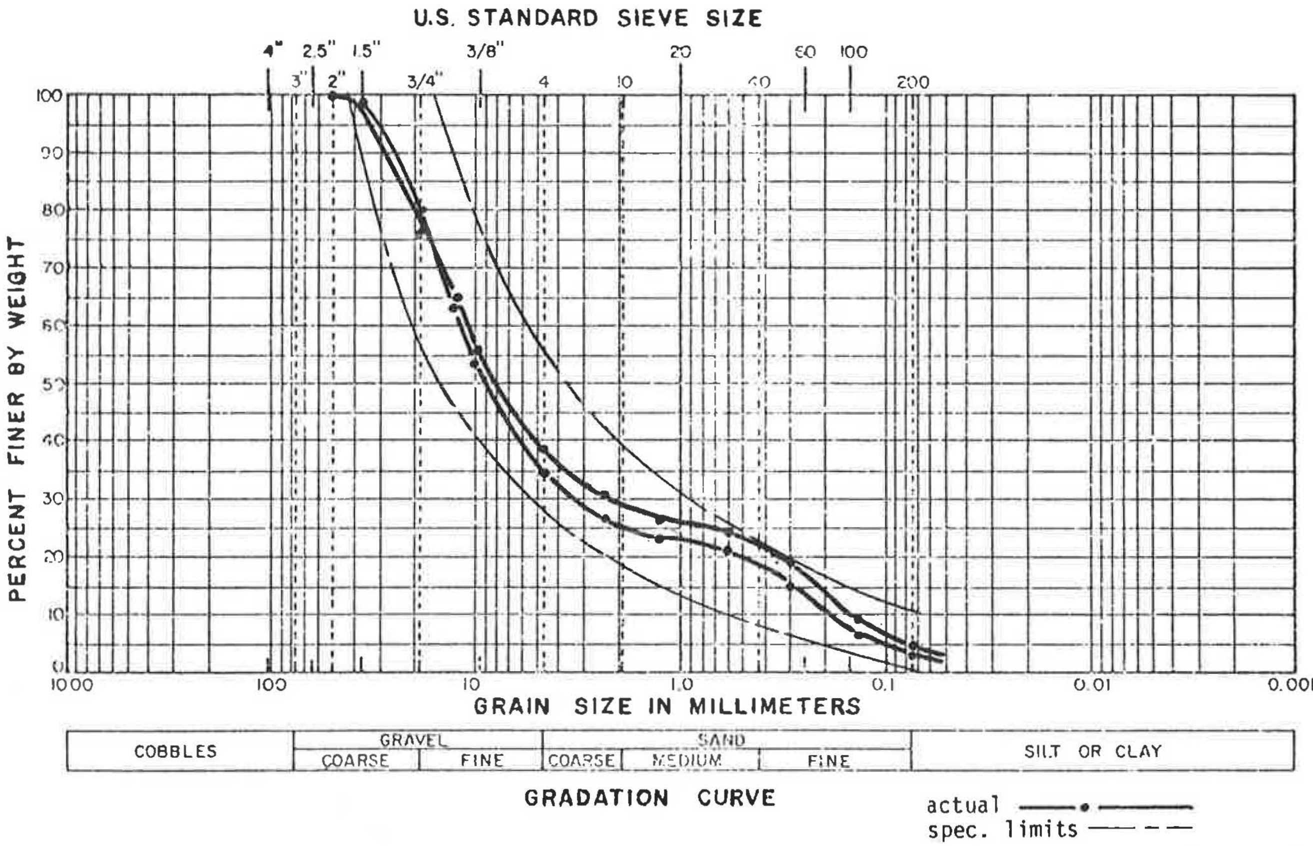

FIGURE 12 PA No. 2A gradation curve.

TABLE 1 LAB AND FIELD SUBBASE MATERIAL PROPERTIES

Subbase Type	Laboratory Permeability K ft./day	Field Permeabilities K1 ft./day	Field Permeabilities K2 ft./day	Lab. $d_{max.}$ (pcf)	Field $d_{max.}$ (pcf)	Lab. $n_{min.}$ (%)	Field $n_{min.}$ (%)	Field e
Aggregate Cement	2.83×10^{-4} (1)	(7)	(7)	138.1	138.1 (4)	16	16 (4)	0.19 (4)
ATPM	6.519×10^{3} (2)	5.39×10^{3}	6.07×10^{3}	112.7	106.9	31	33	0.51
2B Aggregate	2.15×10^{4} (2)	7.74×10^{3}	2.39×10^{4}	102.9	93.2 (5)	37	43 (5)	0.75 (5)
H.P.	1.81×10^{4} (2)	1.73×10^{4}	1.78×10^{4}	110.0	100.0	32	39	0.63
2A Aggregate	1.22 (3)	3.97×10^{1} (6)	1.79×10^{1} (6)	124.9	125.4	23	23	0.30

(1) Triaxial test permeability
(2) Fabricated falling head test
(3) Standard constant head permeameter
(4) Data obtained from mix design testing
(5) Data derived from field concrete design data
(6) Due to limitations of test equipment, field permeability measurements in 2A Aggregate may not be accurate
(7) No measurements because permeabilities were below the lower testing capabilities of the testing equipment

K = Permeability
K1 & K2 - Permeabilities in orthogonal (90 degrees apart) directions
d = Dry density
n = porosity
e = void ratio

Naturally, the stabilized aggregate cement base material had the highest maximum density, lowest porosity, and slowest permeability. The well-graded 2A subbase material (control) had the next highest maximum density, a low porosity, and a slow permeability. The ATPM and HP materials had intermediate maximum densities, porosities, and permeabilities. The unstabilized 2B material had the lowest maximum density, highest porosity, and fastest permeability. Figure 13 depicts the relationship between porosity (maximum density) and permeability.

CBR testing showed that the 2A subbase had values ranging between 80 percent and 85 percent; the HP material had values around 60 percent. Even though the relative stability of the more permeable HP material was notably less than the 2A material, it was substantial and should provide satisfactory performance when confined.

Field Testing

Field permeability testing was performed using the field permeability test device (FPTD) developed by Lyle Moulton and Roger Seals at West Virginia University. Field permeability test results are shown in Table 1. All test sections were located in the southbound lanes. In-place density measurements were made with a Troxler nuclear gauge at the FPTD test locations. Because of the high void ratio and unconfined instability of the 2B aggregate, field densities were not obtained in this material; however, a density was approximated from laboratory design

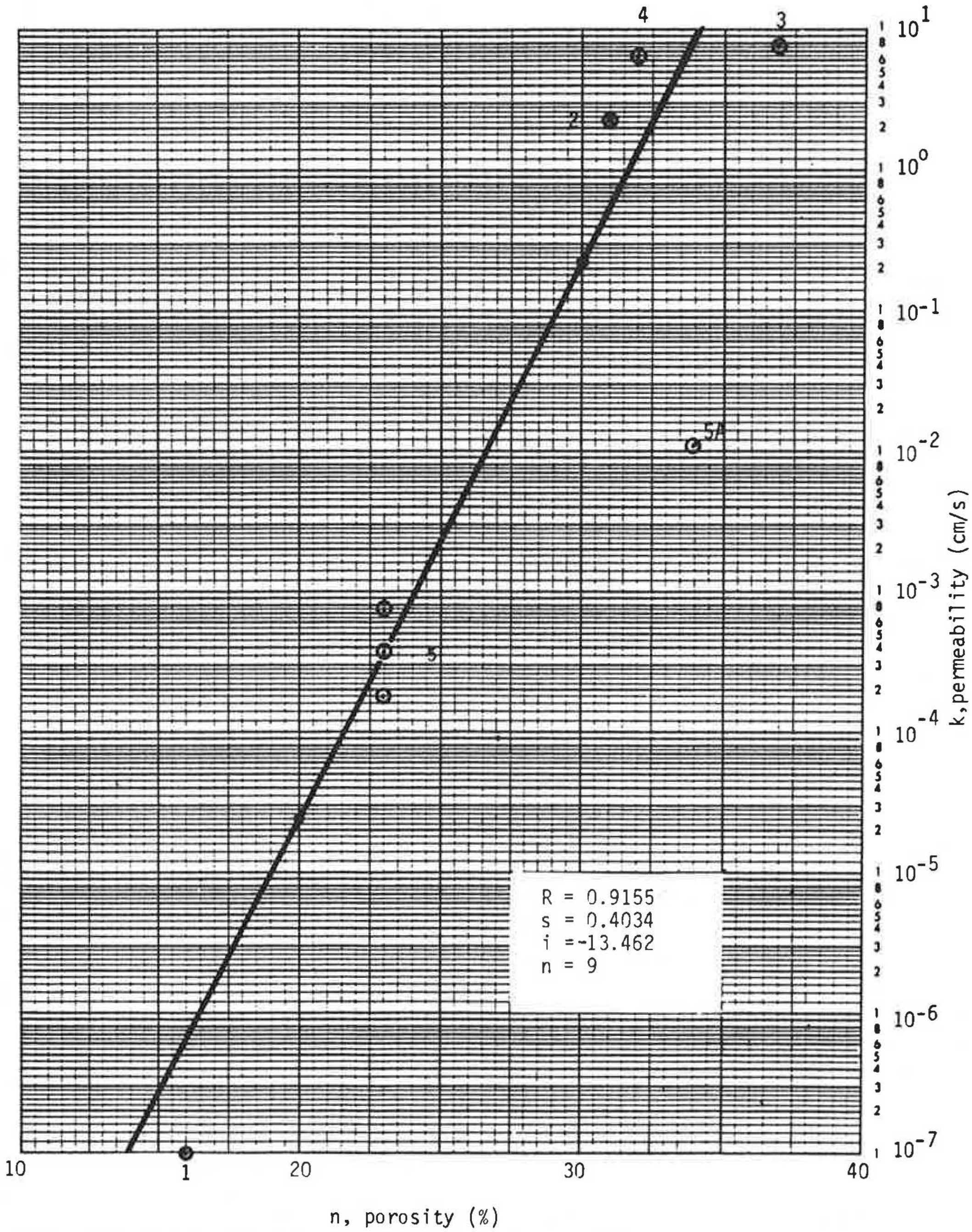

FIGURE 13 Relationship between lab porosity and permeability.

data. Void ratios (*e*) and porosities (*n*) were calculated using a source specific gravity of 2.61. These field density and porosity data are also listed in Table 1.

CONSTRUCTION DETAILS

The entire job, which included the experimental sections, was bid on a competitive basis. The unit prices received from the selected contractor for the different base/subbase materials are shown in Table 2.

TABLE 2 SUBBASE UNIT PRICES

		Interlayer Only	Interlayer and Subbase
1.	Aggregate Cement (6-inch)	$10.00/s.y.	$13.50/s.y.
2.	Asphalt Treated Permeable Material (5-inch)	$ 5.40/s.y.	$ 9.40/s.y.
3.	PA #2B Aggregate (8-inch)	$ 4.30/s.y.	$ 6.80/s.y.
4.	High Permeability (8-inch)	$ 4.30/s.y.	$ 6.80/s.y.
5.	PA #2A Aggregate (8-inch)	$ 4.00/s.y.	$ 6.50/s.y.

The five sections of different base/subbase materials were constructed in July of 1980 without major difficulties or delays, even though the contractor was unfamiliar with some of these materials.

The 10-in.-thick RCP was placed in August 1980 by conventional fixed form methods without incident. A 24-ft-wide (two lanes) pavement was placed monolithically.

The pavement base drain system was typical throughout the test sections. The longitudinal trench was excavated 13 in. (the depth of the subbase) away from the travel lane/shoulder edge of the pavement in both the northbound and southbound directions on tangent sections. The trenches were located on the low side of the pavement on superelevated horizontal curve sections. A perforated plastic drain pipe was then placed, and the trench was backfilled with PA No. 1B crushed aggregate ("pea" gravel). The plastic drain was $4^5/_8$ in. in diameter, semicircular, and was outletted through the slope or into drop inlets. Outlet spacings ranged from 100 to 600 ft and were typically on the order of 300 ft. In all cases, the experimental permeable layer was brought into immediate contact with the PA No. 1B trench backfill material to ensure a continuous flow path. It is crucial for free-draining bases to always be outletted with positive drainage systems.

More detailed information regarding the construction and material testing performed on this project may be found in PennDOT's Research Project No. 79-3 Interim Report (*1*).

PAVEMENT ROUGHNESS MEASUREMENTS AND CONDITION SURVEY

Roughness measurements were made at various times during this evaluation with the Mays ride meter. These measurements represent the response of the vehicle to road roughness and indicate both the relative paving control afforded by the various bases in the initial measurements and the pavement performance under traffic loadings in the longer term measurements. The average pavement serviceability indices (PSI) for the pavements over the subbase sections are listed in Table 3.

TABLE 3 AVERAGE PAVEMENT SERVICEABILITY INDICES

	Subbase Type	Initial 1980	1981	1986	1987
1.	Aggregate Cement	4.0	4.1	3.0	3.4
2.	Asphalt Treated Permeable Material	4.1	4.2	3.3	3.6
3.	PA #2B Aggregate	3.9	4.0	3.2	3.3
4.	High Permeability	3.8	4.0	3.2	3.4
5.	PA #2A Aggregate	3.8	4.0	3.2	3.6

There were no extreme differences among the PSI values measured for each section at any time during this evaluation. The average PSI values of all the sections were slightly higher at the 1-yr testing point. The PSI values of all the sections were lower in 1986 than at the 1-yr testing point but were slightly higher again during the 1987 testing than during the 1986 testing. The slight increase in pavement rideability in the first year can be explained by the wearing of the surface texture and seating of the pavement under traffic.

The initial roughness measurements indicate that relatively smooth pavements can be constructed on all five subbase types included in this study. The comparable roughness measurements obtained in each section during each testing cycle do not indicate any significant influence of subbase type on short-term pavement riding characteristics. If any of these subbases fail to provide adequate pavement support in the future, the pavement distresses which result will certainly adversely affect the rideability of the pavement.

A walking visual survey of the entire pavement over the experimental subbase sections revealed that the pavement was generally in extremely good condition. One transverse crack was found between the 2A aggregate and HP sections. This crack was probably caused by the discontinuity in subbase support, because it occurred in the transition zone from one type of subbase to another.

The only other significant cracking was in the aggregate cement base section. Approximately 25 percent of the slabs in this section had mid-point transverse cracks. These cracks were probably shrinkage cracks. They occurred in this section and not in the other sections as a result of the different frictional characteristics of the slab and subbase in the aggregate cement section at the slab/base interface. As the slabs moved in the aggregate cement section during concrete curing, the frictional forces at the slab/base interface on the slabs over the aggregate cement resisted slab movement and caused cracks to occur in the slabs.

No noticeable faulting exists. There were some occasional minor (1- to 2-in.) spalls at joints. The joints were initially sealed, but because of sealant failure, debris accumulated in some of the slab joints. As the slabs expanded with increased temperature, the incompressible debris material in the joints prevented sufficient slab expansion to alleviate the thermal stresses in the slabs. These stresses most likely caused the joint spalling.

DEFLECTION TESTING

PennDOT's falling weight deflectometer (FWD) was used to measure the deflections of the pavement slabs over each subbase section evaluated in this report. The deflection data not only indicated the deflection characteristics of rigid pavement slabs over each subbase type, but were also used to calculate joint efficiencies and determine possible underlying void locations. The analysis programs used with PennDOT's FWD to determine some of the results presented in this report may be found in the Final Report of PennDOT Research Project No. 85-23 entitled "Pavement Evaluation Procedure Utilizing the Falling Weight Deflectometer" (*4*).

The deflection testing results obtained on this project indicated that pavement temperature levels significantly influence the magnitude of deflection of jointed, rigid pavements when loaded repeatedly with the same or similar loads.

Some differences exist between pavement temperatures measured manually with handheld thermometers and those measured with PennDOT's FWD during this project's deflection testing. Usually, the manually measured and FWD measured temperatures were close. There appeared to be no definite pattern relating the differences in the two types of temperature measurements. At this time, it is not known what caused the discrepancy between the two sets of temperature readings.

The average loaded slab deflections measured at a 25-cm load drop height in 1986 and 1987 are presented in Table 4. To provide data less affected by joint "locking," average loaded slab deflection values, which exclude measurements taken when slab temperatures measured with PennDOT's FWD were above 70°F, are shown in Table 4 in parentheses. As indicated in Table 4, there was not a considerable difference in the deflection readings measured in the same sections in 1986 and 1987.

As expected, the deflection data indicate that the aggregate cement subbase is the strongest. The deflection data also indicate that the ATPM, 2B, and HP subbases all have relatively equal strengths and the 2A subbase was markedly the weakest subbase type tested.

As mentioned earlier, some differences were found between manually measured and FWD measured pavement temperatures. Usually, the two temperature readings were close. Using Spring 1987 data, a linear regression of the deflections measured at the corner of the loaded slab at each joint tested versus pavement temperature was done for each subbase section. The resulting slopes of the regression lines are presented in Table 5. All slopes are negative, indicating that deflections in jointed rigid pavements constructed over each subbase type decrease as temperatures increase. Although the preciseness of the pavement temperatures may be slightly questionable, the difference

TABLE 4 AVERAGE LOADED SLAB DEFLECTIONS

	Subbase Type	1986 Loaded Slab Deflection (10^{-6} inches) *	**	Spring, 1987 Loaded Slab Deflection (10^{-6} inches) *	**
1.	Aggregate Cement	5421	(----)	5555	(5555)
2.	Asphalt Treated Permeable Material	6372	(6560)	6556	(11465)
3.	PA #2B Aggregate	7380	(----)	6370	(7382)
4.	High Permeability	7140	(7266)	7192	(10976)
5.	PA #2A Aggregate	19642	(18650)	9943	(28668)

*Averages not in parenthesis include measurements taken at all pavement temperatures.

**Averages in parenthesis only include measurements taken when pavement temperatures were less than or equal to 70 degrees F. Temperatures were measured by PennDOT's FWD.

TABLE 5 LINEAR REGRESSION SLOPES, PAVEMENT TEMPERATURE VERSUS LOADED SLAB DEFLECTION

	Subbase Type	Linear Regression Slope (10^{-6} inches/F**)
1.	Aggregate Cement	-16
2.	Asphalt Treated Permeable Material	-295
3.	PA #2B Aggregate	-225
4.	High Permeability	-375
5.	PA #2A Aggregate	-795

*Deflections were measured during Spring, 1987.

**Pavement temperatures were measured with PennDOT's FWD.

in linear regression slopes calculated between the aggregate cement section and the ATPM, 2B, and HP sections is very large, with the linear regression slopes of the latter mentioned sections being 14 to 23 times greater than that calculated in the aggregate cement section. This indicates a marked difference in the deflection behavior of pavements over stiff subbases compared to that of less stiff subbases.

It is logical that deflections in pavements over stiff subbases will be relatively less affected by pavement joint locking than deflections in pavements over weak subbases. By comparing the slopes of the linear regression lines for the various subbase material types, it becomes apparent that the pavement deflections measured in the aggregate cement section decreased less with temperature increase than deflections measured in the other sections. The aggregate cement base already exhibited the lowest average total deflection. Conversely, the dense-graded 2A section had the highest average total deflections and the greatest regression line slope or rate of change in deflection as temperatures increased.

VOID DETECTION TESTING

Included in the software used to analyze deflection data with PennDOT's FWD is a program that determines if it is probable that a void exists under the pavement at the tested joint. The void detection analysis requires repeated deflections to be measured at a joint using three different loads. The loads used on this project were approximately 6,000 lb, 9,000 lb, and 12,000 lb.

A summary of the void detection analysis results obtained on this project during Spring 1987 deflection testing is presented in Table 6. To minimize the use of data skewed by locked pavement joints, only deflections measured when pavement temperatures were equal to or less than 70°F, as measured with PennDOT's FWD, are included in the void detection results shown in Table 6.

TABLE 6 VOID DETECTION SUMMARY

S	Subbase Type	Joint No.	Testing Year	Probability that Void Exists? Approach Slab	Leave Slab
1	Aggregate Cement	5	1987	No	No
1	Aggregate Cement	13	1987	No	No
2	Asphalt Treated Permeable Material	9	1986	No	No
2	Asphalt Treated Permeable Material	9	1987	Yes	Yes
2	Asphalt Treated Permeable Material	16	1986	No	No
3	PA #2B Aggregate	2	1987	No	No
4	High Permeability	15	1986	Yes	No
4	High Permeability	15	1987	No	No
4	High Permeability	18	1986	No	No
5A	PA #2A Aggregate	4	1986	Yes	Yes
5A	PA #2A Aggregate	16	1986	Yes	Yes
5A	PA #2A Aggregate	16	1987	Yes	Yes
5A	PA #2A Aggregate	18	1986	Yes	Yes
5A	PA #2A Aggregate	18	1987	Yes	Yes

Table 6 indicates that all joints in the 2A aggregate sections that were tested for voids probably had voids under both the approach and leave slabs. None of the other subbase sections exhibited a tendency to have voids at joints. During the 1987 testing, a probable void was found under a joint in the ATPM section. In 1986, a probable void was located under the approach slab of one joint in the HP section, but this determination is questionable. The data indicating whether a void exists were near the borderline of the criteria for determining the probability of a void existing. In 1987, no void was detected at the same joint and slab. The data were slightly on the other side of the criteria for determining the probability of void existence under a slab.

The gradation and poor drainage characteristics of 2A subbase as compared to ATPM, 2B subbase, and HP subbase make it more susceptible to void formation due to the buildup of pore pressures and the pumping of fine material. The aggregate cement base is probably too impermeable and too rigid to readily allow void formation.

JOINT EFFICIENCY

The joint efficiency results used in this evaluation were calculated using FWD deflection data. Listed in Table 7 are the average joint efficiencies calculated for FWD tested joints in each subbase section during the Spring of 1987. FWD measurements used to calculate the average joint efficiencies were not purged because of high pavement temperatures. For uniformity in sampling, only measurements made at an FWD load drop height of 25 cm were included in the Table 7 data. Thus, the impact force on each joint whose efficiency is included in Table 7 was approximately the same.

TABLE 7 AVERAGE JOINT EFFICIENCIES (25-cm LOAD DROP HEIGHT), SPRING 1987 TESTING

	Subbase Type	1986 Average Joint Efficiency (%)	1987 Average Joint Efficiency (%)
1.	Aggregate Cement	73	70
2.	Asphalt Treated Permeable Material	66	76
3.	PA #2B Aggregate	62	49
4.	High Permeability	37	54
5.	PA #2A Aggregate	94	83

As indicated in Table 7, the average joint efficiencies calculated in the more open-graded HP and 2B aggregate sections are less than the other sections, but the joints in the impermeable, stable, aggregate cement section had lower efficiencies than those in the 2A aggregate sections. The variation in average joint efficiencies presented in Table 7 implies that joint efficiency, as calculated by PennDOT's FWD, may vary with subbase type. With the information presented in Table 7, the correlating factors between joint efficiency and subbase type cannot be definitely determined.

Certainly, a major factor determining the efficiency of pavement joints is the effectiveness of the load transfer devices constructed at the joints. The round dowel bars placed at each transverse joint were $1^1/_4$ in. in diameter, 18 in. long, and spaced 1 ft apart. Since identical load transfer devices and construction techniques were used for all the joints in all sections of this project, it is reasonable to expect little or no difference in the average joint efficiencies of the various sections as a result of the load transfer devices.

The formula currently being used to calculate joint efficiencies with PennDOT's FWD is (*4*)

$$\textit{Joint Efficiency} = \frac{\textit{Deflection of Nonloaded Slab} \times 100}{\textit{Deflection of Loaded Slab}}$$

In 1958, Teller and Cashell reported that dowel looseness, caused by coatings used to prevent bond, concrete air or water voids, concrete shrinkage during hardening, and wear of the concrete surrounding the dowel by repetitive loading, is a significant variable affecting joint efficiency (*5*, p. 26). They also stated that they consider dowel looseness to include all conditions preventing dowels from offering full load resistance (*5*, p. 16). In the same report (*5*, p. 26), Teller and Cashell stated,

> It is obvious that a dowel or dowel system does not begin to function at maximum efficiency until all looseness is taken up by the deflecting pavement on the loaded side of the joint farther than would be necessary if initial looseness were not present.

All dowels in all sections constructed on this project had a graphite bond breaker applied to the dowels in the field. The relatively small traffic loadings should not have caused excessive concrete wear. Probably, most of the existing dowel looseness was built into the pavement and should be approximately the same for all sections.

Teller and Cashell stated further (*5*, p. 17),

> It is apparent that the load-transfer system is much more effective when the slab-end deflection is relatively large. . . . It is of interest that once the play and looseness of the system is taken up the effectiveness is relatively high. . . .

During the Spring 1987 FWD testing, pavements over the 2A subbase had the highest deflections. The dowels in the 2A sections were deflected beyond the amount necessary to "take up" dowel looseness farther and for a greater percentage of their total deflections than dowels in the other subbase sections. This allowed the load transfer devices in the 2A aggregate sections to work at maximum efficiency during a larger portion of the deflections of the slabs, resulting in a higher overall average percent joint efficiency for the joints in the 2A aggregate sections.

The ATPM section, 2B section, and HP section all had similar average loaded slab deflection magnitudes. They all would have deflected past the amount necessary to take up dowel looseness approximately the same amount, so the factor influencing the joints in the ATPM section to have a higher joint efficiency than those in the 2B aggregate and HP sections was probably not dowel looseness. Because other factors affect pavement performance and were apparently influencing joint efficiency calculations in the other subbase sections, it was realized that dowel looseness was probably not the only factor causing higher joint efficiency results in the 2A aggregate section.

It was decided that additional FWD testing should be done to further define the factors influencing joint efficiency measurements. This additional FWD testing took place during the Fall of 1987. During this testing, the loads and drop heights of the FWD were adjusted so that approximately equal deflections were obtained in each of the subbase sections. This should have reduced the effect of dowel looseness on joint efficiency results since pavement slabs in each subbase section would be deflected past the amount required to take up dowel looseness the same amount. The results of the Fall 1987 joint efficiency testing are shown in Table 8.

The two types of subbases on this project which were bound with cementitious materials are aggregate cement and ATPM. The 2A aggregate subbase is not a bound material but compacts very tightly because of the amount of fine material included in its gradation. The HP and 2B aggregate subbases are not bound and do not have the amount of fine material in their gradations that the 2A subbase does. Thus, they do not compact as tightly as the 2A aggregate subbase does.

During the equal deflection testing, joint efficiency results in the unbound 2A aggregate section were similar to those obtained in the two bound subbase sections. This inferred dowel looseness probably did play a part in causing higher joint efficiency results in the 2A aggregate section during the Spring 1987 testing.

Even with the equal deflections imparted on the pavement, there remained a significant difference between the joint efficiencies measured in the relatively open-graded 2B and HP subbase sections and the 2A aggregate and bound subbase sections. This may be explained by considering the bound subbases and the tightly compacted 2A aggregate subbase to deflect in more of a beam-like manner than the 2B and HP subbases. The more beam-like subbase deflections would allow the subbase under the immediately adjacent nonloaded slab to deflect when a load is imparted on a slab. The higher deflections in the 2A aggregate and bound subbases would cause a lack of pavement support, resulting in higher deflections in the nonloaded slabs over the 2A aggregate and the bound subbases. Higher deflections in nonloaded slabs result in higher joint efficiencies. Hence, if all other factors influencing joint efficiency are equal, lower joint efficiencies should be found in pavements over bound or tightly compacted subbases than over unbound, relatively loosely compacted, more open-graded subbases.

Pavement temperature is also believed to affect the magnitude of deflections in adjacent nonloaded slabs and, hence, joint efficiency calculations. Two sets of measurements, approximately equal in deflection magnitude, were made on 2 different days in the ATPM section. There was approximately 12°F difference in the average pavement temperatures measured during testing in the ATPM section on these 2 days. The average joint efficiency calculated during the higher temperature deflection testing was approximately 11 percent higher than that calculated during the lower temperature deflection

TABLE 8 AVERAGE JOINT EFFICIENCIES, APPROXIMATELY EQUAL DEFLECTIONS IN EACH SECTION, FALL 1987 TESTING

	Subbase Type	Average Pavement Temperature During Testing (Degrees F*)	Average Loaded Slab Deflection (10^{-6} inches)	Average Joint Efficiency (%)
1.	Aggregate Cement	68	12,230	70
** 2a.	Asphalt Treated Permeable Material	57	13,620	81
** 2b.	Asphalt Treated Permeable Material	45	12,760	70
3.	PA #2B Aggregate	70	12,390	47
4.	High Permeability	65	13,630	52
5.	PA #2A Aggregate	68	12,680	75

*Pavement temperatures were measured with PennDOT's FWD.

**For comparison purposes, two sets of equal deflection measurements were obtained on two different days in the Asphalt Treated Permeable Material section.

testing. This indicates the significant effect pavement temperatures have on deflections in adjacent nonloaded slabs and the joint efficiencies that are calculated using those deflections. Increased joint locking at higher pavement temperatures causes relatively larger deflections in adjacent nonloaded slabs which results in higher joint efficiencies.

Another factor which could affect joint efficiency results is the magnitude of the load applied during the FWD deflection testing. If higher loads are applied during testing, the pavement will be deflected farther past the amount necessary to take up dowel looseness, allowing load transfer devices to work at maximum efficiency during a larger percentage of the total slab deflection. As discussed earlier in this report, higher joint efficiencies will result. The effect different pavement loadings have on joint efficiency should be kept in mind when interpreting FWD joint efficiency data.

As discussed in this report, joint efficiency appears to be affected by dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbase. It is not presumed that these are the only factors that influence joint efficiency. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but to be influenced by a combination of factors in the pavement structure and environment during testing.

It should be kept in mind that the FWD joint efficiency calculation method of dividing the nonloaded slab deflection by the loaded slab deflection does not necessarily indicate only the load transfer occurring across the slabs or even how well the pavement's load transfer devices are performing. As discussed in this report, if some subbases do not deflect in a beam-like manner as much as others, they will support nonloaded slabs better. Lower joint efficiencies will be determined at joints in these better supported pavements. Lower joint efficiencies influenced by the beam-like deflection characteristics of subbases do not indicate a pavement support problem or that load transfer devices are working less effectively.

It is realized that PennDOT's FWD joint efficiency formula is not the only available formula for calculating load transfer efficiency. A load transfer measuring formula that takes into account the variety of factors influencing joint efficiency could make joint efficiencies calculated by a FWD more useful to engineers determining required joint rehabilitation.

PENNDOT'S OPEN-GRADED SUBBASE

PennDOT now uses open-graded subbase (OGS) as an interlayer between rigid pavements and 2A aggregate. OGS was developed to provide an aggregate layer under pavements that is more free draining than 2A aggregate, does not "pump" fines, and is capable of supporting pavement loads. To optimize manufacturing, construction stability, and permeability of the OGS material, it was decided to make the OGS permeability approximately 500 ft per day (0.18 cm/s). Field measurements indicate a 500 ft/day to 1,500 ft per day (0.18 to 0.53 cm/s) permeability in OGS.

The gradation curve for OGS is shown in Figure 14. The OGS gradation is in between the gradations of the high permeability and 2B aggregate subbases mentioned in this report.

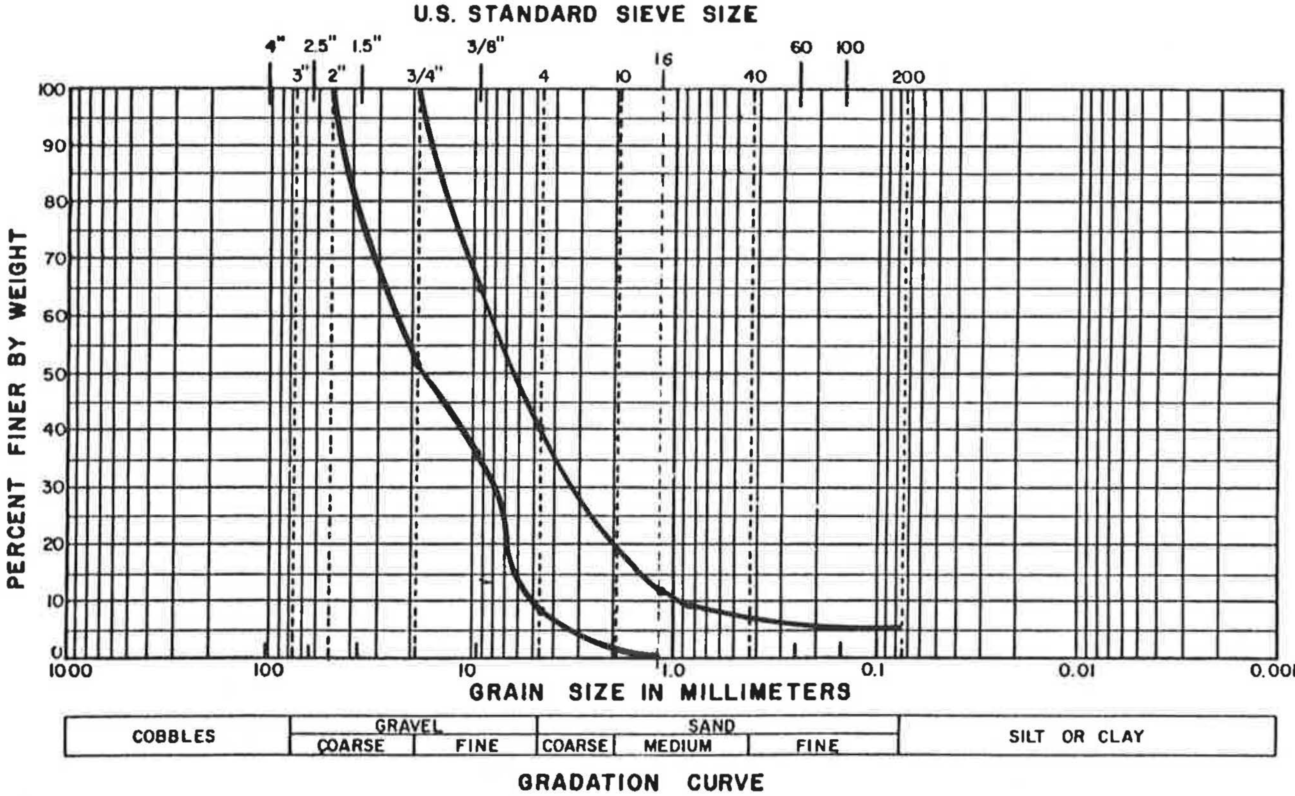

FIGURE 14 PA No. OGS gradation curve.

As a result of the gradation, most aggregate suppliers can produce OGS with one straight crusher run and do not have to blend different sizes of aggregates. This provides fewer OGS production steps which allows lower costs.

OGS has a greater tendency to segregate during handling and placing operations than does denser-graded subbase. Because of this, some contractors are having trouble meeting gradation requirements for OGS when random sampling is done on material placed on grade. PennDOT is considering looking at another point of sampling OGS to determine if it meets gradation requirements. This other point of testing will be earlier in the placing operation where the effects of segregation due to handling have not been introduced. It is believed that the fine end of the gradation band is not critical from a confined stability standpoint and that this portion of the matrix will drop to the bottom inch of the OGS layer, probably during the construction sequence of first hydraulic loading, anyway. However, these fines will probably improve the compactibility and construction stability of the material.

Contractors have demonstrated on a few major jobs their ability to achieve good compaction and stability of the OGS and successfully pave on it using both slip-form and fixed-form techniques.

CONCLUSIONS

- Subbase material with significantly high permeabilities (three or more orders of magnitude) can be produced with adequate quality control at a competitive cost. The contract price for the original standard design of 13 in. of 2A subbase was \$6.50/sq yd. The substitution of 8 in. of 2B or HP material as an interlayer on top of 5 in. of subbase increased the comparable cost to \$6.80/sq yd—about 5 percent. The substitution of 5 in. of ATPM on top of 8 in. of subbase increased the comparable cost to \$9.40/sq yd—about 45 percent over the cost of 13 in. of subbase. This ATPM cost increase would not be as great in a flexible pavement design where a higher structural coefficient and lesser required thickness, as compared to the standard unbound aggregate subbase, would be used.
- Adequate stability to support construction equipment was provided by the more porous, open-graded base materials. All sections of various bases were constructed without major difficulties or delays even though the contractor was unfamiliar with some of these materials. Pavement roughness measurements on the new reinforced concrete pavement indicated that the stabilized aggregate cement and ATPM sections had PSI values 0.2 to 0.3 higher than the unstabilized or unbounded sections. The PSI values of the pavement in the unstabilized open-graded materials sections were approximately equal to the sections with the previous 2A dense graded subbase. These roughness comparisons were similar after 15 mo, 6 yr, and 7 yr of service life with only 0.2 to 0.3 variation in PSI among the sections during each respective testing.
- The three open-graded materials had adequately high permeabilities, but the permeability of the 2A subbase was unsatisfactorily low. The more porous, open-graded ATPM, 2B, and HP materials exhibited field permeabilities on the order of 2.8×10^3 ft per day (10^0 cm/s), while the standard 2A subbase had measured permeabilities of 28 ft per day to 0.28 ft per day (10^{-2} cm/s to 10^{-4} cm/s). Excellent relationships existed between measured laboratory and field permeabilities for the

same materials. Field testing results indicated that permeabilities measured in two orthogonal directions at the same location generally were not significantly different. Permeabilities varied by as much as one order of magnitude within a material section because of segregation resulting from placement practices. The more fines that exist in the material or the more "gap-graded" the material is, the greater the propensity for segregation to occur.

- Visual surveys of pavement surface conditions indicate that all sections are in extremely good condition, albeit the truck traffic frequencies are relatively small. Only minor joint spalling and no faulting were noted.
- The results of this evaluation infer that dowel looseness, pavement temperature, loading magnitude, and the extent of beam-like behavior exhibited by underlying subbases all influence joint efficiency measurements. Joint efficiency appears not to be controlled by one or even a few factors at a particular site but is affected by and influenced by a combination of factors in the pavement structure and environment during testing.
- Average total deflection measurements, indicating relative strengths of the pavement sections, show the aggregate cement section to have the lowest deflections, while the deflections in the ATPM, 2B aggregate, and HP sections were approximately equal to each other but slightly higher than the deflections measured in the aggregate cement section. The 2A section had markedly the highest total deflections. These data indicate that the open-graded subbases should out-perform the dense-graded 2A material from a structural standpoint under the same loading conditions.
- An assessment of the probability of voids existing under pavement joints made from FWD deflection results indicates that voids probably already exist under all the joints tested in the 2A dense-graded aggregate sections. There was no strong tendency shown in the data obtained during this study to indicate voids frequently exist in the other subbase material sections. These data support the achievement of one of PennDOT's main objectives by switching to open-graded materials, that is, reducing the pumping of fines, which creates voids and ultimately causes loss of support, faulting, and deterioration of the slabs at the joints.
- PennDOT has changed its specifications and standards to require the use of open-graded subbase (OGS) interlayers immediately beneath rigid pavements. This change was based on the early results of this project. The intermediate range (7 yr) results relating the performance of dense-graded and open-graded subbases continue to support PennDOT's decision to use OGS.

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