Civil Engineering, University of Illinois at Urbana-Champaign, under the sponsorship of the Federal Highway Administration, U.S. Department of Transportation. George Ring III of the Federal Highway Administration was the project monitor.

The contents of this paper reflect my views, and I am responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

REFERENCES

1. N.C. Phu and M. Ray. L'Erodabilité des Materiaux de la Couche de Fondation et de la Couche de Forme des Chaussées en Beton. Chaussees en Beton, Laboratoire des Ponts et Chaussées, Paris, 1979.
2. B.J. Dempsey, S.H. Carpenter, and M.I. Darter. Improving Subdrainage and Shoulders of Existing Pavements. FHWA, Final Rept., 1980.
3. The AASHO Road Test: Report 7. HRB, Special Rept. 61G, 1962.
4. E.J. Yoder. Pumping of Highway and Airfield Pavements. Purdue Univ., West Lafayette, IN, Joint Highway Research Project, 1946.
5. H.R. Cedergren. Methodology and Effectiveness of Drainage Systems for Airport Pavements. U.S. Army, Construction Engineering Research Laboratory, Champaign, IL, Tech. Rept. C-13, 1974.
6. W. Gulden. Investigation into the Causes of Pavement Faulting on the Georgia Interstate System. Office of Materials and Tests, Georgia Department of Transportation, Atlanta, Interim Rept. 2, 1974.
7. D.L. Spellman, J.H. Woodstrom, and B.F. Neal. Faulting of Portland Cement Concrete Pavements. Materials and Research Department, State of California, Sacramento, Res. Rept. M&R 635167-2, 1972.
8. R.G. Packard. Design Considerations for Control of Joint Faulting of Undoweled Pavements. Proc., International Conference on Concrete Pavement Design, Purdue Univ., West Lafayette, IN, 1977.
9. J.H. Woodstrom. Improved Base Design for Portland Cement Concrete Pavements. Proc., International Conference on Concrete Pavement Design, Purdue Univ., West Lafayette, IN, 1977.
10. S.H. Carpenter, M.I. Darter, and B.J. Dempsey. A Pavement Moisture Accelerated Distress (MAD) User's Manual. FHWA, 1980, Vol. 2.
11. J.P. Zaniewski. Economic Considerations of Faulting and Cracking in Rigid Pavement Design. TRB, Transportation Research Record 602, 1976, pp. 1-8.
12. E.J. Barenberg, C.L. Bartholomew, and M. Herring. Pavement Distress Identification and Repair. U.S. Army, Construction Engineering Research Laboratory, Champaign, IL, Tech. Rept. P-6, 1973.
13. R.E. Frost. Correcting Pavement Pumping by Mudjacking--Indiana. HRB, Res. Rept. 1-D, 1945, pp. 13-54.
14. C.A. Hogentozler. Report of Investigation of the Economic Value of Reinforcement in Concrete Roads. Proc., HRB, Vol. 6, 1926.
15. S.A. LaCoursiere, M.I. Darter, and S.A. Smiley. Performance of Continuously Reinforced Concrete Pavements in Illinois. FHWA, Rept. FHWA-IL-UI-112, 1978.
16. B.J. Dempsey, M.I. Darter, and S.H. Carpenter. Improving Subdrainage and Shoulders of Existing Pavements--State of the Art. FHWA, Interim Rept., 1980.
17. S.H. Carpenter, M.I. Darter, and B.J. Dempsey. A Pavement Moisture Accelerated Distress (MAD) Identification System. FHWA, 1980, Vol. 1.
18. S.H. Carpenter, M.I. Darter, and B.J. Dempsey. Evaluation of Pavement Systems for Moisture-Accelerated Distress. TRB, Transportation Research Record 705, 1979, pp. 7-13.
19. M. Ray. A European Synthesis on Drainage, Subbase Erodibility, and Load Transfer in Concrete Pavements. Proc., 2nd International Conference on Concrete Pavement Design, Purdue Univ., West Lafayette, IN, 1981.
20. H.L. Ahlberg and E.J. Barenberg. The University of Illinois Pavement Test Track--A Tool for Evaluating Highway Pavements. HRB, Highway Research Record 13, 1963, pp. 1-21.
21. B.J. Dempsey and Q.L. Robnett. Influence of Precipitation, Joints, and Sealing on Pavement Drainage. TRB, Transportation Research Record 705, 1979, pp. 13-23.
22. M.I. Darter and M.B. Snyder. Development of a Nationwide Concrete Pavement Evaluation System. NCHRP, Interim Rept., 1980.
23. M.I. Darter, S.A. LaCoursiere, and S.A. Smiley. Structural Distress Mechanisms in Continuously Reinforced Concrete Pavements. TRB, Transportation Research Record 715, 1979, pp. 1-7.
24. Le Transfer de Charge aux Joints Transversaux de Retrait-Flexion et la Conception des Chaussées en Beton. Chaussées en Beton, Laboratoire Centrale des Ponts et Chaussées, Paris, 1979.

Subbase Permeability and Pavement Performance

GARY L. HOFFMAN

Problems of premature pavement and shoulder distress in Pennsylvania have been attributed to excess water in the standard, dense-graded subbase. An experimental project was constructed to demonstrate the feasibility of providing good support and good internal drainage to the single layer of subbase with a two-layer system at a competitive cost. An additional long-term objective of the project was to determine the significance of the permeability of subbase layer materials on pavement performance. Five types of subbases, ranging from a very impermeable cement-stabilized material to a very permeable and uniformly graded crushed aggregate, were incorporated into the project. The study documented the manufacturing of the various materials and the associated unit costs. Base materials with permeabilities 3 orders of magnitude more than that of the standard subbase were placed for only a 5 percent increase in cost. The ability of the contractor to handle, place, and pave on the various subbases was evaluated. In-place permeabilities were measured with the field permeability test device developed by L.K. Moulton of West Virginia University to statistically determine permeability by variation within a material section and between the five material sections. Initial pavement roughness measurements were made on the concrete pavement in each experimental sec-

tion, and these measurements indicated no loss in construction quality with the use of the more permeable interlayers. Underdrain system outlets were instrumented so the rates at which the various subbase materials carry off infiltrated surface water could be determined throughout the long-term evaluation.

The current Pennsylvania Department of Transportation (PennDOT) specification for crushed-aggregate subbase (PA 2A) 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 (a) the necessary strength and stability to support construction equipment, the pavement, and subsequent traffic; (b) drainage; and (c) a material that could be manufactured with adequate quality control at reasonable cost. The specification was developed as a compromise that would best meet the above criteria.

Several problems of premature pavement and shoulder distress have been attributed to excess water in the pavement system, and questions have been raised as to the adequacy of the current PA 2A subbase to provide sufficient drainage for the pavement system. Laboratory permeability tests of the 2A subbase indicate a range in the coefficient of permeability of 1×10^{-3} to 1×10^{-5} 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 seems to be higher in some cases than indicated by the laboratory tests. Recent field permeability tests conducted as part of a nationwide study by L.K. Moulton and Roger Seals of West Virginia University (1) on PennDOT subbase indicate permeabilities in the range of 10^{-1} to 10^{-2} cm/s.

This experimental construction project was devised to demonstrate the feasibility of providing good support and good internal drainage at a competitive cost to our current subbase. These facts were recently demonstrated in a similar project in Kentucky (2). An additional long-term objective of the project was to determine the significance of the permeability of subbase layer materials on pavement performance. The subbases were to represent a range in permeability from impermeable to very permeable.

The project field site was located on traffic route 66 (Legislative Route 203) in Armstrong County, Pennsylvania. Five sections of base/subbase materials representing a range of permeability conditions from impermeable (aggregate-cement) to very permeable (PA 2B aggregate) were constructed. The standard design was the placement of 25 cm of reinforced concrete pavement (RCP) on 33 cm of PA 2A densely graded aggregate subbase (control section). In the experimental sections, other materials were placed as an interlayer between the PA 2A subbase and the RCP as shown in the pavement cross sections (Figures 1-5). The total thickness of the experimental interlayer and the subbase was 33 cm for all sections. Each experimental section was between 304 and 509 m long and was constructed in adjacent sections in both the northbound and southbound lanes of the four-lane divided highway.

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. All aggregate material was a glacial sand and gravel that was shipped from Davison Sand and Gravel's Tarrtown Flats source.

The specified gradation limits for the standard PA 2A gradation, the high-permeability (HP) gradation, and the unstabilized PA 2B gradation are shown in Figure 6. The PA 2A and the HP materials are both well graded, but the HP material has coarser fragments than the PA 2A throughout its entire particle size distribution. The PA 2B gradation band is narrow, and this material is uniformly sized. The actual distribution curve of the unstabilized 2B material is close to the coarser side of the specified gradation band, whereas the distribution curve for the 2B material used in the stabilized asphalt-treated permeable material (ATPM) mix is close to the finer side of the specified band.

Laboratory densities, porosities, and permeabilities are given in Table 1. A source specific gravity of 2.61 was used for all calculations. Naturally, the stabilized aggregate-cement base material had the highest maximum density, lowest porosity, and slowest permeability--of the order of 10^{-7} cm/s. The well-graded standard PA 2A subbase material (control) had the next highest maximum density, a low porosity, and a slow permeability--of the order of 10^{-4} cm/s. The ATPM and HP material had intermediate maximum densities, porosities, and permeabilities, of about 2 and 6 cm/s, respectively. The unstabilized PA 2B material had the lowest maximum density, highest porosity, and fastest permeability--nearly 8 cm/s.

The laboratory permeabilities, except for the aggregate-cement, were determined by standard constant-head test equipment and by falling-head and constant-head testing with a piece of equipment that was fabricated in-house. Because the aggregate-cement was nearly impermeable, its permeability was determined by using core samples set up in a triaxial testing device. Laboratory permeability testing indicated that the specified HP gradation included unnecessary fines. The material finer than approximately the 2.00-mm (U.S. No. 10 standard) sieve size readily migrated within the HP gradation matrix under the imposed hydraulic conditions. This migration of about 5 percent of the material appreciably decreased the measured permeabilities with time as the test progressed.

Field Testing

Field permeability testing was performed by using the field permeability test device (FPTD) developed by Lyle Moulton and Roger Seals at West Virginia University. Schematic diagrams of the water source subsystem and the plate/probe subsystem for this FPTD are shown in Figures 7 and 8. This field testing provided information to statistically compare (a) the laboratory permeabilities to field permeabilities, (b) the variability of permeabilities within a given material section, (c) the variability of permeabilities among the various sections, and (d) the differences in permeabilities measured in two orthogonal directions (90° apart) at the same test location.

Field permeability measurements were recorded at only one location in the PA 2A subbase because of difficulties in obtaining legitimate results at a number of other locations in this material section. A field permeability of 1×10^{-2} cm/s was calculated for one direction and of about 6×10^{-3} cm/s in the orthogonal direction at this one location. Much difficulty was encountered in establishing flow through this layer without the occurrence of piping immediately beneath the plate. It was believed that the field permeabilities of the 2A material (10^{-4} cm/s from laboratory testing) for the most part were near or below the lower testing capability of the FPTD equipment. The recorded measurements at this one location may have been erroneous also or they may have been factual, indicating a more permeable zone. No field tests were attempted in the impermeable aggregate-cement base layer after the difficulty that was encountered in the moderately slow-permeability 2A material.

Testing the ATPM, the HP material, and the PA 2B coarse aggregate was relatively successful and provided interesting statistical data, which will be discussed in the next section. Tests were made at different locations in each of these sections, and multiple measurements were taken in each of two (K_1 and K_2) orthogonal directions. For the most part, the 45° arrow halfway between these two rows of probes was aligned with the fall line of the grade. The test locations in the ATPM, the HP, and the 2B sections were randomly chosen to provide both longitudinal and transverse coverage of the section material. 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 data. Void ratios (e) and porosities (n) were calculated by using a source specific gravity of 2.61. These field density and porosity data are listed in Table 2.

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 as follows (1 in = 2.5 cm; $1.00/yd² = $0.83/m²):

Section	Interlayer Only ($/yd²)	Interlayer and Subbase ($/yd²)
1, aggregate-cement (6 in)	10.00	13.50
2, ATPM (5 in)	5.40	9.40
3, PA 2B aggregate (8 in)	4.30	6.80
4, HP (8 in)	4.30	6.80
5, PA 2A subbase (8 in)	4.00	6.50

The five sections of different base/subbase materials were constructed in July 1980 without major difficulties or delays even though the contractor was unfamiliar with some of these materials. The 25-cm-thick RCP pavement was placed in August 1980 by conventional fixed-form methods without incident. A 7-m-wide (two lanes) pavement was poured monolithically.

The pavement base drain system was typical throughout all the test sections and was constructed in accordance with recently implemented design and material changes. The longitudinal trench was excavated 33 cm (the depth of the subbase) away from the travel lane and shoulder edge of the pavement in both the northbound and southbound directions on tangent sections. The perforated plastic drain pipe was then placed, and the trench was backfilled with PA 1B crushed aggregate (pea gravel). The plastic drain pipe was 12 cm in diameter, semicircular, and outlets were through the slope or into drop inlets. Outlet spacings ranged from 30 to 182 m and were typically of the order of 91 m. In all cases, the experimental permeable layer was brought into intimate contact with the PA 1B trench backfill mate-

Figure 1. Aggregate-cement test area.

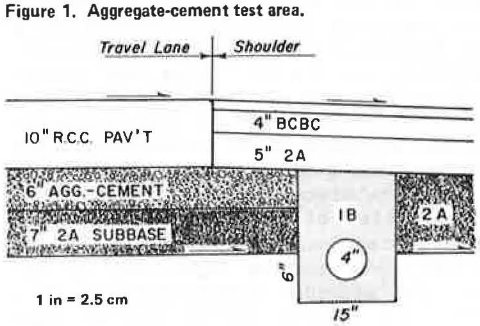


Figure 2. Asphalt-treated permeable material test area.

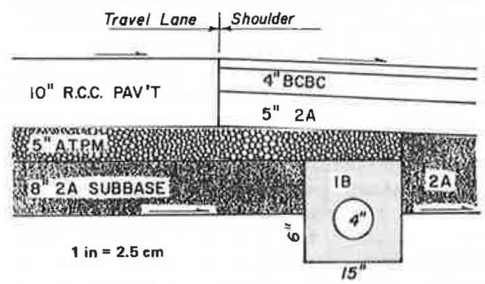


Figure 3. 2B aggregate test area.

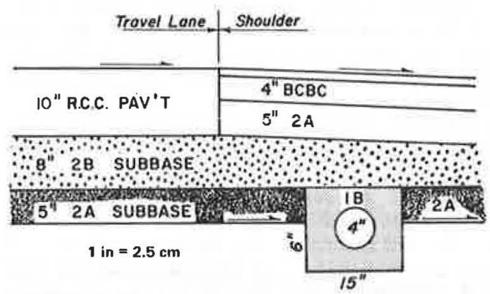


Figure 4. High-permeability test area.

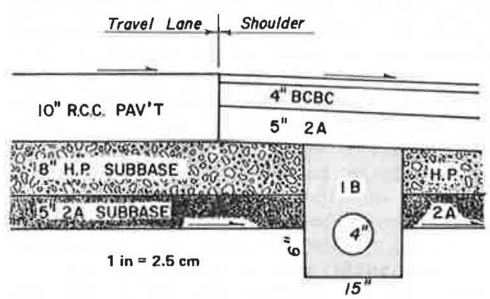
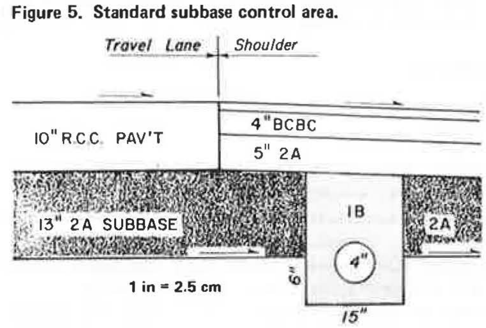


Figure 5. Standard subbase control area.



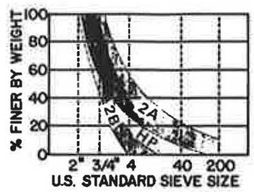


Figure 6. Aggregate gradation bands.

Table 1. Laboratory test data for material types.

Material	$\delta_{d\ max}$ (pcf)	n_{min} (%)	K (cm/s)
Aggregate-cement	138.1	16	1.0×10^{-7}[a]
2A subbase	124.9	23	1.8×10^{-4}[b]
			3.7×10^{-4}[b]
			7.4×10^{-4}[c]
	106.9[d]	34	1.1×10^{-2}[e]
ATPM	112.7	31	2.3×10^{0}[c]
			2.4×10^{0}[e]
HP	110.0	32	6.4×10^{0}[c]
2B subbase	102.9	37	7.6×10^{0}[c]

Note: 1 pcf = 16 kg/m^3.
[a]Triaxial-test permeability.
[b]Standard constant-head permeameter.
[c]Fabricated falling-head test.
[d]Indicates δ_d minimum.
[e]Fabricated constant-head test.

rial to ensure a continuous flow path.

To periodically measure outflow from the base drain system in the various base/subbase sections, 14 accessible outlets were selected for monitoring. Point measurements will be made at various time intervals after rainstorms at these locations. A tipping bucket with a continuously recording counter was installed at the outlet at the downslope end of the ATPM section. This tipping bucket will provide data to determine flow-rate changes with time after rainstorms. A standard weather bureau rain gage was also installed at the site to better predict the infiltration of surface water into the pavement system from these particular rainstorms.

RESULTS

Pavement roughness measurements made on the new reinforced concrete pavement are listed in Table 3 and indicate the paving control afforded by the various bases. The average pavement serviceability indices (PSIs) for the stabilized aggregate-cement and ATPM base sections were 0.2 to 0.3 higher than the unstabilized aggregate base sections. This tends to indicate that these stabilized bases provided a more stable and/or uniform platform on which to pave. The PSI averages for the unstabilized aggregate sections ranged from 3.81 to 3.87, an insignificant difference. This indicates that there was little difference in the contractor's ability to provide a high-quality pavement surface on the PA 2B, the HP, and the PA 2A (control) material sections. Comments regarding the difficulty in maintaining a uniform grade on the PA 2B base appear unfounded when these roughness measurements are compared. PSI measurements made one year after construction indicated the same roughness relationship among the sections as the initial measurements.

A detailed visual inspection of the pavement surface made 15 months after construction revealed that most of the transverse and longitudinal joints were well sealed and in excellent condition. The hot poured sealant was cracked, debonded, or removed on a small percentage of transverse joints. No spalling, faulting, or cracking was encountered.

Statistical methods were applied to the permeability data to calculate the means and deviations of the calculated permeabilities within each test section, to compare the average field permeabilities with the average permeabilities determined in the laboratory, to compare permeabilities in the K_1 and K_2 orthogonal directions at the same location, and to relate the average field permeability with the average material porosity.

The field permeability means and standard deviations for each base material section are given below

Figure 7. Schematic of fresh and salt water subsystem.

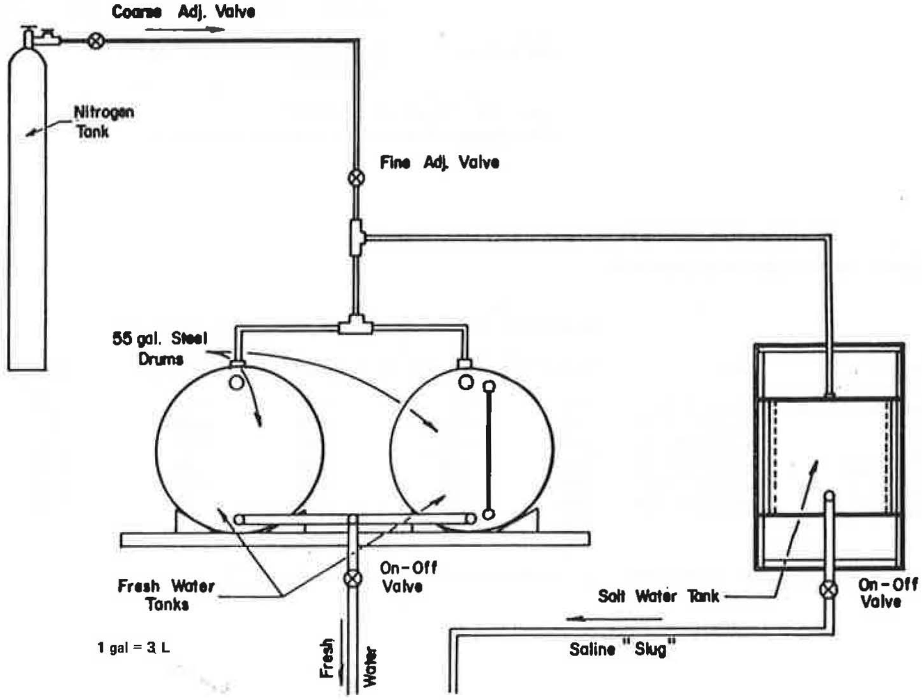


Figure 8. Schematic of plate and probe subsystem.

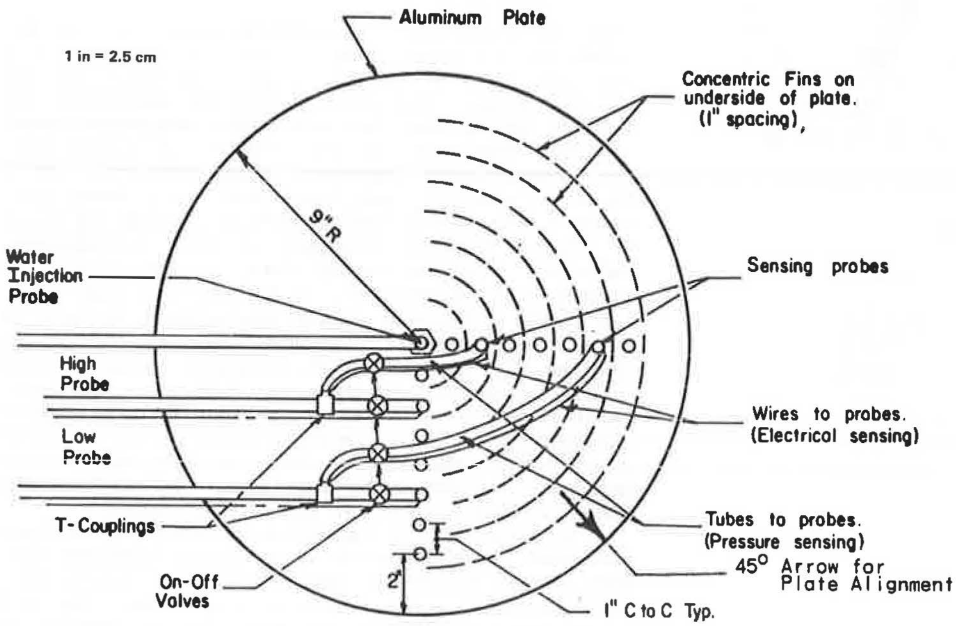


Table 2. Field density and porosity data.

Material	Station	Offset (ft)[a]	δ_w	w(%)	δ_d(pcf)	e	n(%)
Aggregate-cement	Data obtained from mix design testing			–	138.1	0.19	16
2A subbase	731+75	12 R	134.3	6.1	126.7	0.29	22
	738+50	12 L	133.5	6.4	125.5	0.30	23
	739+00	20 R	133.5	7.1	124.6	0.31	23
	754+45	10 R	133.5	7.0	124.8	0.30	23
Avg					125.4		23
ATPM	758+44	26 R	107.8	–	107.8	0.49	33
	768+20	14 R	101.0	–	101.0	0.59	37
	770+85	10 R	106.5	–	106.5	0.51	34
	772+10	19 R	112.1	–	112.1	0.43	30
Avg					106.9		33
HP subbase	829+44	8 R	102.0	7.5	94.5	0.72	42
	829+90	15 R	114.0	7.5	106.5	0.53	35
	830+35	7 R	106.0	6.9	99.1	0.64	39
	831+00	18 R	109.3	7.3	102.0	0.60	38
	833+70	6 R	104.2	6.5	97.7	0.67	40
Avg					100.0		39
2B subbase	Data derived from field concrete design data			–	93.2	0.75	43

Note: 1 ft = 0.3 m; 1 pcf = 16 kg/m^3.

[a]R = right side of pavement; L = left side of pavement.

Table 3. Initial roughness measurements.

		Northbound or Eastbound		Southbound or Westbound		
Material	Station	Travel Lane	Passing Lane	Travel Lane	Passing Lane	Section Avg
Aggregate-cement	125+95-140+09	3.96	4.00	4.02	3.95	3.98
ATPM	758+00-772+50	3.88	4.16	4.14	4.08	4.07
PA 2B	772+50-782+50	3.78	4.00	3.95	3.76	3.87
HP material	822+50-839+75	3.84	3.79	3.83	3.78	3.81
Control A	745+00-758+00	3.93	3.70	3.76	3.84	3.81
Control B	806+00-821+00	3.98	3.62	3.78	3.93	3.83
Avg[a]		3.90	3.88	3.91	3.89	

[a]Average of northbound or eastbound lanes, 3.89; average of southbound or westbound lanes, 3.90.

for both the K_1 and K_2 orthogonal directions:

Material	K_1	K_2
2A	$\bar{X} = 1.4 \times 10^{-2}$	$\bar{X} = 6.3 \times 10^{-3}$
	$S = 0.2 \times 10^{-2}$(?)	$S = 0.1 \times 10^{-3}$(?)
ATPM	$\bar{X} = 1.9$	$\bar{X} = 2.1$
	$S = 1.1$	$S = 1.3$
HP	$\bar{X} = 6.1$	$\bar{X} = 6.3$
	$S = 3.7$	$S = 3.0$
2B	$\bar{X} = 2.7$	$\bar{X} = 8.4$
	$S = 0.7$	$S = 2.3$

The questionable field measurements in the standard PA 2A material (control) indicated the slowest permeability, although the aggregate-cement would obviously have been slower if field measurements had been attainable. The average permeabilities of ATPM, HP, and the PA 2B material were similar and nearly 1000 times more permeable than the dense-graded PA 2A subbase. The means and standard deviations were also very similar for both the K_1 and K_2 orthogonal directions in the ATPM and HP materials. This indicated that, when the average data in these sections are considered, the direction of testing at any test location made little difference. However, there was a significant difference between the means and standard deviations for the K_1 and K_2 directions in the 2B material, thus indicating that directional differences in material permeabilities existed. The HP material exhibited the largest deviation of permeabilities, which indicated the greatest variability between test locations in the same material test section. This variation could be explained by the fact that the 2.00-mm sieve size material was unstable within the total HP matrix. With the segregation of these fines, as noted during laboratory permeability testing and during transport by the producer, varied in-place gradations and hence permeabilities existed in both transverse and longitudinal directions.

The average field and laboratory permeabilities for each material type were compared to determine the relative performance of the FPTD against standard laboratory methods. The correlation coefficient ($R = 0.996$; $m = 0.911$; $i = -0.03$; $n = 4$) for the data shown below indicated a significant correlation at the 99 percent confidence level:

Material	Laboratory Avg (cm/s)	Field Avg (cm/s)
2A	4.3×10^{-4}	1.0×10^{-2}(?)
ATPM	2.4	2.0
HP	6.4	6.2
2B	7.6	6.6

The average permeabilities determined in the field with the FPTD compared very favorably with the average laboratory permeabilities for each of the four material types that were evaluated.

Field permeabilities were compared with the corresponding material porosities calculated from nuclear gauge densities at the test locations. These data showed a significant relationship between permeability and porosity: generally, the greater the porosity, the higher the permeability. The relationship is significant at the 95 percent level of confidence for all points listed above. This good relationship existed in part because all the aggregates came from the same source.

CONCLUSIONS

Base material with significantly higher permeabilities (3 or more orders of magnitude) than the standard PA 2A can be manufactured with adequate quality control at a competitive cost. The contract price for the original standard design of 33 cm of subbase was \$6.50/yd² (\$5.44/m²). The substitution of 20 cm of PA 2B or HP material as an interlayer on top of 12 cm of subbase increased the comparable cost to \$6.80/yd² (\$5.69/m²), about 5 percent more. The substitution of 12 cm of ATPM on top of 20 cm of subbase increased the comparable cost to \$9.40/yd² (\$7.86/m²)--about 45 percent over the cost of 33 cm of subbase. This ATPM cost increase would not be so great in a flexible pavement design where a higher structural coefficient and a decreased required thickness, as compared with the standard 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 0.2-0.3 higher than those of the unstabilized or unbounded sections. The PSI values in the unstabilized open-graded materials were equal to or slightly better than the standard PA 2A dense-graded subbase now specified by PennDOT. These same roughness relationships existed after 15 months of service life.

The three open-graded materials had adequately high permeabilities, but the permeability of the PA 2A subbase was unsatisfactorily low. The more porous ATPM, PA 2B, and HP materials exhibited field permeabilities of the order of 10^0 cm/s, whereas the standard PA 2A subbase had measured permeabilities of 10^{-2} 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 gradation segregation resulting from placement practices. The more fines that existed in the material or the more "gap"-graded the material was, the greater was the propensity for this segregation to occur.

Porous material gradations used for drainage interlayers should have minimal material passing the 2.00-mm sieve size. The minus 2.00-mm sieve size material in the HP gradation did not add to the material stability and migrated through the coarser matrix. Even as much as 5 percent of this fine, migrating material could significantly alter the permeability at various locations in the base layer; moreover, it could clog the longitudinal pavement base drain system.

The pavement was performing satisfactorily in all the various base layer sections one year after construction. No distinct differences in pavement performance due to the improved base drainage were evident at that time. Pavement and drainage system performance will continue to be evaluated until a relationship has been determined.

ACKNOWLEDGMENT

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REFERENCES

1. L.K. Moulton and R.K. Seals. Determination of the In Situ Permeability of Base and Subbase Courses. FHWA, Rept. FHWA-RD-79-88, May 1979.
2. E.B. Drake. Kentucky Highway Project (KY55). Kentucky Department of Transportation, Frankfort, 1979.

Pavement Drainage in Seasonal Frost Area, Ontario

J.B. MACMASTER, G.A. WRONG, AND W.A. PHANG

During the last two decades, full-width granular construction filter courses; improved ditching, trenches, and drains; and axle-load controls have all been implemented on Ontario highways. In spite of this, pavement damage during late winter and spring continues to be a problem for the Ontario Ministry of Transportation and Communications. This paper illustrates how this problem is compounded in seasonal frost areas. During warm winter days, melt-water from deicing salts enters the partly thawed base. Trapped there by frozen subbases and shoulders, it creates differential heaving during subsequent freezing periods. Two experiments carried out to explore the problem of pavement edge cracking are described briefly. These tests include the use of plastic pavement edge skirts and partial-width paved shoulders. The success and practicality of the paved shoulders prompted the Ministry to use them on a continuing basis. The Ministry has been using plastic pipe pavement edge drains since 1978 to improve the drainage of rigid pavements. Details are given on how the drains are placed with trenchless plows; an innovative and very successful installation technique. The Ministry's limited use of open-graded drainage layers is touched on briefly. In the area of preventive maintenance, the discussion centers on preliminary studies on the use of primed and surface-treated shoulders as waterproofing measures. Routing and sealing of cracks has also become a significant feature of the Ministry's program in upgrading the performance of pavements and prolonging the life of overlays.

It is generally acknowledged by highway agencies that, of all the environmental factors that adversely affect the performance of pavements, water is the most significant. Excess moisture in granular base and subbase layers leads to high pore pressures under the dynamic loading of traffic. These high pore pressures tend to overcome the frictional forces between the granular particles and cause a reduction in the bearing capacity of the base. This in turn causes an increase in stress in the wearing course.

Cedergren and Godfrey (1) claim that inadequate drainage of excess moisture in the structural section leads to premature damage of the pavement. Ratios of damage caused by traffic impacts on pavements with free water versus those with little or none and the tests that determined them are as follows: Western Association of State Highway Officials (WASHO) Road Test (2), up to 70 000:1; American Association of State Highway Officials (AASHO) Road Test (3), up to 40:1 (Cedergren analysis of Liddle data); University of Illinois Circular Test Track (4), 100:1 to 200:1 (Cedergren analysis of test data).

In Ontario, as in other seasonal frost areas, weakening of the pavement occurs during the spring when the subgrade begins to thaw. The thaw period can spread over several weeks, particularly in northern Ontario where the frost has penetrated more than 3.0 m (10 ft) below the surface. However, this spring effect may, in the southern parts of Ontario, be repeated several times during the winter, since the base layer is subjected to periodic thaw cycles caused by deicing salts.

In the past 15-20 years, designers have developed various changes in the physical structure of pavements and in the materials used in highway construction. This reflects the attempt to improve the performance of roadways, in part by maintaining a low moisture content in pavement structures.

Subdrains have been installed at various locations within the right-of-way to intercept water that might otherwise enter the base and subbase. Pipes have been located below ditchlines to lower high water tables in the subgrade and cut slopes. Drains have also been placed in the outer edges of the shoulders to help drain the pavement structure when it was impossible to provide side ditches of adequate depth. Such treatments, although effective, were limited in scope and constructed to deal with specific problem locations.

In the early 1960s, the Ontario Ministry of Transportation and Communications (MTC) switched from a core, or earth shoulder, design to full-width granular construction in order to provide lateral drainage of the pavement structure. At the same time, the specifications of the base and subbase materials being used in Ontario were being altered (made more dense) to achieve greater stability to cope with the increasing volume and weight of traffic loads. However, while the densities and bearing capacities of these aggregates were increasing, the permeability values were decreasing, thus reducing the effectiveness of the full-width lateral drainage.

In more recent years, some highway drainage experts have vigorously promoted the use of an open-graded drainage layer within the pavement structure. These layers of materials that exhibit high permeability may be constructed full width and daylighted at the side slopes or designed to outlet into a collector drain installed beneath the shoulder. Cedergren (5) advocates that such a drainage layer have laboratory permeability rates of 6000 m/day (20 000 ft/day) in areas where frost penetrates below the depth of the drainage layer. He cautions, however, that actual field values are likely to be of the order of 2100-3000 m/day (7000-10 000 ft/day).

In this report, experiments are described that demonstrate the pavement structure drainage problem in a seasonal frost area of Ontario and the steps that are being taken to avoid or reduce the damaging effects. These include limited use of very permeable drainage layers but principally relate to provision of partly paved shoulders and pavement edge pipe drains.

SOURCES OF WATER

Although it is known that excess moisture adversely affects the performance of roadways in any climate, pavements in seasonal frost areas are subject to additional stresses at certain times of the year (6).