EVALUATION OF ASPHALT MIX PERMEABILITY

FINAL REPORT
FOR
MLR-90-2

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Highway Division

Iowa Department of Transportation

Evaluation of Asphalt Mix Permeability

Final Report
for
MLR-90-2

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8. ABSTRACT

Efforts to eliminate rutting on the Interstate system have resulted in 3/4" aggregate mixes, with 75 blow Marshall, 85% crushed aggregate mix designs. On a few of these projects paved in 1988-1989, water has appeared on the surfaces. Some conclusions have been reached by visual on-sight investigations that the water is coming from surface water, rain and melting snow gaining entry into the surface asphalt mixture, then coming back out in selected areas. Cores were taken from several Interstate projects and tested for permeability to investigate the surface water theory that supposedly happens with only the 3/4" mixtures. All cores were of asphalt overlays over portland cement concrete, except for the Clarke County project which is full depth AC.

The testing consisted of densities, permeabilities, voids by high pressure airmeter (HPAM), extraction, gradations, A.C. content and film thicknesses. Resilient modulus, indirect tensile and retained strengths after freeze/thaw were also done. All of the test results are about as expected. Permeabilities, the main reason for testing, ranged from 0.00 to 2.67 ft. per day and averages less than 1/2 ft. per day if the following two tests are disregarded.

One test on each binder course came out to 15.24 ft/day, and a surface course at 13.78 ft/day but are not out of supposedly problem projects.

9. KEY WORDS	**10. NO. OF PAGES**
Rutting, Permeability, Voids	36

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INTRODUCTION

Efforts to eliminate rutting on heavy traffic asphalt pavements have resulted in 75 blow Marshall mix designs with 3/4 inch coarse gradations of 85% crushed materials on Interstate acc resurfacing projects. On three of these projects paved in 1988 and 1989, water has been observed exiting through the recently paved surface. On-site investigations consisting of coring, trenching through the shoulder at the pavement edge, and visual observation gave some small indication that the water may be surface water that has penetrated the pavement through voids in the asphalt mix and has gravitated through the pavement voids until the right conditions of pressure, permeability, and surface cracks exist to allow the water to exit through the pavement surface.

The possibility of any highly permeable asphalt pavement layers is prompting concern about future performance problems related to asphalt stripping and/or damage due to freeze/thaw action when these layers are saturated with water.

OBJECTIVE

The objective of this study is to evaluate the permeability of cores taken from Interstate paving projects which have exhibited surface water as well as investigate several projects which do not exhibit this action. Evaluation of voids, extractions, gradations, indirect tensile strength, resilient modulus, and physical changes from freeze/thaw action will be conducted.

PROCEDURE

Four-inch diameter cores, through the entire thickness of asphalt paving, were removed from the following projects.

County	Project No.	Milepost	Station	Direction	Lane
Clarke	IR-35-2(216)33--12-20	38.0-43.0	500	S	DR
Decatur	IR-35-1(54)00--12-27	0.0-7.3	125	S	DR
Harrison	IR-29-5(58)77--12-43	75.7-90.5	1620	N	DR
Mills	IR-29-2(32)34--12-65	38.6-43.6	745	N	DR
Polk	IR-35-4(59)92--12-77	92.5-98.7	415	S	DR
Woodbury	IR-29-6(87)126--12-97	128.1-141.4	1200	N	DR

Ten cores were taken from each project in the outside wheel path of the driving lane at 50 ft. intervals.

The cores were divided by sawing to separate surface and binder layers. The following tests were performed on the specimens taken from each project.

Specimens Tested Per Project

	Surface	Binder
Density	All	All
Permeability	3	3
Voids (HPAM)	3	3
Resilient Modulus (MR) & Indirect Tension (TI)	3	3
Condition by Freeze/Thaw	3	3
TI & MR After Freeze/Thaw	3	3
Extraction & Gradation	1	1

Three additional cores were obtained from the centerline on Decatur and Clarke County projects to be tested for voids and density.

Specific procedures for freeze/thaw and permeability testing were developed.

A falling head permeameter was built by machining parts out of heavy, clear plastic components. A simple schematic of this device is shown in Appendix "B". The sides of the cores had to be sealed to stop any water from escaping and going out the sides or edges of the cores. Colored water was used. A plastic paint called "noryde" was used to seal the sides of the cores. The paint did not penetrate the cores filling any voids. Experimenting was done on cores other than the ones included in this research.

Each core was left in the permeability machine for 120 minutes. The water temperature and testing temperatures were done at 77°F. The amount of water that permeated the cores was then calculated according to the formulas found in the procedure in Appendix B.

Resilient modulus testing was conducted on six cores from each layer of each project. Data and test parameters is included in Appendix "A". A summary of the data is included in Table II. Three of the cores tested for each mix were also subjected to 50 cycles of freeze and thaw and retested. The data also is included in Table II and Appendix "A".

Indirect tensile testing was done on three cores from each layer of each project. The formula for indirect tensile strength was calculated as per the formula shown in Appendix B.

Some cores on which resilient modulus testing was conducted were also used for indirect tensile. The resilient modulus is considered a nondestructive test. This is the reason for using some of the cores for resilient modulus, then also for indirect tensile testing and then were further used for extraction and gradation analysis.

The cores tested for indirect tensile were extracted and the aggregate gradation and asphalt content was determined. The gradations, AC content and calculated film thicknesses are shown in Tables I and III.

The aggregate type and percent of each aggregate in each layer for each project is shown in Table IV.

Table I shows core densities, AC contents, permeabilities and high pressure air meter voids for specific cores for each project and layer.

RESULTS/OBSERVATIONS

There appears to be no obvious visual correlation of air voids and permeability. A California study and also a Georgia study both concluded that permeability of mixes increases drastically at 8% of voids in the California study and 10% of voids in the Georgia study. At this void level, the water permeability in the California voids study allowed 200 MM/min. to 1.3 MM/min. in the Georgia voids study.

Only in a few cases were high pressure air meter voids measured on the same cores that were tested for permeability. For Clarke County, binder Table I showed 15.24 ft/day permeability with 8.7% voids (core 10). Core 6 had 8.6% voids with a permeability of only 0.41 ft/day. There is no sound explanation for this except there may have been some aggregate size segregation in core 10 that is not visible that contained interconnected voids which allowed more water to pass through. Perhaps this research shows that asphalt cement concrete, properly mixed and placed, is not waterproof like most people believe.

The summary showing resilient modulus and indirect tensile averages are shown in Table II. Test data for individual cores is shown in Appendix A.

No obvious visual correlations are evident in respect to the resilient modulus on any one mix in respect to being tested at 50 or 75 lbs. Even the Harrison County project, the test results, after 50 cycles of freeze/thaw, were not consistently more or less than the tests before the freeze/thaw on the same cores. The resilient modulus was quite high on the Mills County binder course mixture. In this mix, the high pressure air meter voids were the lowest of any of the mixtures. This indicates that the lower the voids the higher the resilient modulus and indirect tensile, which consistently may not be the case.

The same cores could not be used for indirect tensile testing before and after the 50 cycles of freeze/thaw due to destruction of the cores in the indirect tensile test.

The percent of retained strength for each particular mixture (layer) ranged from 70.1% lowest to 86.1% the highest, disregarding the Decatur binder which was 40.2%. There seems to be no explanation for this low retained result which is based on the tests of three cores in Table II.

Extraction and gradation was done on the cores after the indirect tensile tests. The gradations, AC contents and the AC film thicknesses were calculated. The results are shown in Tables I and III. Generally, the film thicknesses calculate lower than shown in the assurance samples due to the gradation changes.

The aggregate in a mix generally breaks down and becomes finer as it is being handled and mixed. For example, the aggregate will break down from handling before going through the plant. The plant mixing will usually further generate more minus #50, #100, #200 materials. The compaction process also breaks the aggregate down a small amount more, as shown by research done by Lowell Zearly in 1982 "Effect of Compaction on the Aggregate Gradation of Asphalt Concrete." This research studied field roller and also laboratory compaction effects on gradation. Laboratory compaction breakdown was again illustrated in MLR-86-7, "Effect of Compaction on the Aggregate Fracturing of Asphalt Concrete" by R. W. Monroe.

The extractions and gradations in this research were done on cored samples. The results shown in Table III reflect the gradations from the plant mix breakdown, compaction breakdown, and cutting of particles in the coring process which all reflect in finer gradations on the larger aggregate to the #200 material. All of the breakdown causes more aggregate surface area, resulting in lower calculated film thicknesses. This must be kept in mind when using gradations from cored samples for referee or verification tests.

Filler bitumen ratios on the data here in Table III in Decatur surface 7.9/4.3 = 1.84 which is quite high and is not indicative of the project mixture. The filler bitumen for the Woodbury binder, for example, 4.6 ÷ 4.3 = 1.07, which is quite a variance between these two mixtures. Filler bitumen ratio specifications are based on cold feed gradations and tank stick A.C. measurements.

Following are some observations and information as to the source of water on the surfaces that prompted this investigation. Water appeared on the surfaces of the Decatur and Clarke County projects. No explanation is given for the Clarke County project. The Clarke County project is a full depth asphalt project, but still has longitudinal centerline and transverse temperature related cracks which can allow water to come up from the subbase.

The Decatur County project was investigated to a much larger extent. Originally, cores were drilled when water started appearing on the new asphalt surface near centerline. A report of the findings was made at the time the first six cores were taken by F. E. Neff on September 6, 1989, copy included in Appendix C. Mr. Neff stated that core #5 taken at a 1/4 point, Station 230+00 southbound lane, was cut through the 8" pcc and through the 2 1/2" of acc base under the pcc. Water seeped into the core hole from beneath the pcc. Core #6 was cut at the same station, but on centerline.

Mr. Neff's observations here which he describes in more detail in the report in the appendix indicates that in core hole #6 water came in from beneath the pcc and in 10 minutes filled the core hole to a 12" depth. The core hole was cleaned of water several times and the water kept coming in. Side shoulder underdrains were in place, but no water was running out, although the drain were wet.

This Decatur County project ultimately showed severe longitudinal segregation in very narrow strips on the bottom of the surface course. These narrow strips would allow some water to migrate longitudinally, but not transversely to any degree.

During May of 1990 (approximate time) maintenance forces cut seven transverse joints in the overlay in an attempt to drain the trapped water from the mat. The joint configuration used is similar to Detail "A" on Road Standard RH-50, copy is in Appendix C. The depth of cut extended through the acc down to the top of the pcc. The joints were cut from shoulder to shoulder at Stations 220, 225, 230, 235, 240, 245, and 250. I have observed some of these joints from time to time when in the area, but have not seen any water. There have been no other reports that I know of that these joints are draining any water away.

CONCLUSIONS/RECOMMENDATIONS

This investigation has been worth the effort due to the large amount of data collected. Data from test results on samples out of constructed projects most often times does not correlate well with design parameters due to small inherent differences in the product that develops between design and placement in the aggregate gradations, voids, A.C. contents and densities.

There is little correlation from one project or layer to the other in respect to resilient modulus, indirect tensile, permeability, filler bitumen ratios, calculated film thicknesses, percent retained strengths after freeze/thaw to draw any absolute conclusions except that the water is coming up from beneath the portland cement concrete pavement in the case of the Decatur County project and from beneath the full depth asphalt pavement in the case of the Clarke County project.

These projects are all quite successful considering the truck traffic they are carrying, <u>without rutting</u>. Rutting was a problem before we went to these coarse, harsh 3/4" mixtures.

ACKNOWLEDGEMENTS

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3. L. Zearley. Effect of Compaction on the Aggregate Gradation of Asphalt Concrete. June 1982.

4. R. Monroe. Effect of Compaction on the Aggregate Fracturing of Asphalt Concrete. MLR-86-7, June 1986.

Table Titles

Table I
Core Permeabilities, Densities, and Voids

Clarke Co.
IR-35-2(216)33

	Core	Density	A.C. %	Perm. Ft/Day	Southbound Lane Core No.	Voids % HPM
Binder	6	2.279	4.8	0.41	6	8.6
	9	2.354	4.8	0.58	9	4.4
	10	2.261	4.8	15.24	10	8.7
					C_L	6.9
Surface	1	2.315	5.1	1.62	2	5.4
	5	2.340	5.1	0.00	7	6.9
	11	2.343	5.1	0.00	10	7.5
					C_L	6.3

Decatur County
IR-35-1(54)00

Southbound Lane

	4	2.336	4.3	0.41	4	6.1
Binder	10	2.330	4.3	0.71	10	7.2
	11	2.352	4.3	0.18	11	5.8
					C_L	6.5
Surface	7	2.374	4.3	0.02	4	6.8
	9	2.356	4.3	0.03	5	6.4
	10	2.377	4.3	0.0025	8	8.6
					C_L	8.1

Harrison County
IR-29-5(58)77

Northbound Lane

Binder	4	2.326	4.3	0.07	4	3.9
	6	2.342	4.3	0.82	6	4.7
	7	2.336	4.3	2.67	8	4.0
Surface	3	2.316	4.4	13.40	2	6.3
	8	2.334	4.4	0.25	5	7.4
	10	2.313	4.4	1.98	6	5.7

Table I (cont'd)
Mills County
IR-29-2(32)34

	Core	Density	A.C. %	Perm. Ft/Day	Core No.	Voids % HPM
					Northbound Lane	
Binder	2	2.381	4.4	.0056	2	2.2
	4	2.395	4.4	.0056	4	1.9
					7	2.4
Surface	5	2.333	4.7	0.89	1	9.2
	7	2.326	4.7	0.90	4	7.9
	8	2.323	4.7	0.88	10	9.3

Polk Co.
IR-35-4(59)92

					Southbound Lane	
Binder	A		4.4	---	3	8.5
	B		4.4	---	7	8.6
	C		4.4	0.03	9	8.0
Surface	4	2.371	3.9	0.44	4	6.2
	6	2.418	3.9	0.005	6	3.8
					10	5.7

Woodbury County
IR-29-6(87)126

					Northbound Lane	
Binder	5	2.397	4.3	0.021	4	4.1
	8		4.3	0.050	6	5.9
	10		4.3	0.63	9	4.2
Surface	4	2.389	4.4	0.038	3	5.5
	10	2.382	4.4	0.14	6	5.1
					7	5.6

Table II
Summary of
Resilient Modulus, Indirect Tensile,
and Retained Strength Averages

	Resilient Modulus		Indirect Tensiles		
	50 lbs.	75 lbs.	Before F & T	After F & T	% Ret.
Clarke County					
Binder	240,000	220,000	107.5	83.1	77.3
Surface	280,000	270,000	125.4	90.7	72.3
Decatur County					
Binder	280,000	240,000	121.8	49.0	40.2
Surface	570,000	570,000	167.8	126.9	75.6
Harrison County					
Binder	330,000	280,000	176.9	139.7	79.0
After F & T	340,000	350,000			
Surface	350,000	370,000	159.4	135.6	85.1
After F & T	360,000	340,000			
Mills County					
Binder	1,160,000	1,070,000	291.6	246.8	84.6
Surface	510,000	530,000	154.3	127.8	82.9
Polk County					
Binder	300,000	280,000	120.1	103.4	86.1
Surface	380,000	490,000	157.1	110.1	70.1
Woodbury County					
Binder	380,000	340,000	195.2	140.0	71.7
Surface	370,000	350,000	182.6	134.3	73.5

Table III
Extracted Gradations, Asphalt Cement
and A.C. Films

	Percent Passing											%	A.C. Film Mic.
	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	A.C.	Cores
Clarke Co.													
Binder		100	93	74	43	29	22	17	12	9.0	7.8	4.8	6.8
Surface		100	93	78	51	31	23	18	12	9.6	8.1	5.1	7.3
Decatur Co.													
Binder	100	99	93	75	43	28	21	15	9.9	7.7	7.0	4.3	7.0
Surface		100	87	61	41	30	23	18	12	9.4	7.9	4.3	5.7
Harrison Co.													
Binder		100	95	77	49	35	26	20	12	8.2	6.4	4.3	7.0
Surface		100	92	79	54	34	25	19	12	7.7	5.7	4.4	6.2
Mills Co.													
Binder		--	100	96	65	51	41	31	18	9.4	6.8	4.4	5.7
Surface		100	95	80	61	41	28	20	11	8.0	6.4	4.7	7.3
Polk Co.													
Binder		100	95	75	45	30	22	17	11	7.6	6.4	4.4	7.0
Surface		100	93	74	44	29	22	16	11	7.5	6.2	3.9	6.8
Woodbury Co.													
Binder	100	98	93	84	52	33	25	20	13	7.5	4.6	4.3	8.0
Surface		100	91	74	58	43	32	24	15	8.6	5.5	4.4	7.2

Table IV
Aggregate Types and Percent
in Each Mixture

Clark Co.				
Binder	15% RAP		76% Limestone	9% Sand
Surface			85% Limestone	15% Sand
Decatur Co.				
Binder			85% Limestone	15% Sand
Surface		33% Quartzite	52% Limestone	15% Sand
Harrison Co.				
Binder	30% RAP		64% Limestone	6% Sand
Surface	38% RAP	58% Quartzite		4% Sand
Mills Co.				
Binder			85% Limestone	15% Sand
Surface	25% Granite		60% Limestone	15% Conc.Sand
Polk Co.				
Binder	15% RAP		74.5% Limestone	10.5% Sand
Surface		30% Quartzite	55% Limestone	15% Sand
Woodbury Co.				
Binder	15% RAP	32% Quartzite	53% Limestone	
Surface	32% Cr. Gravel	53% Quartzite		15% Sand

Appendix A

Individual Core Density, Indirect Tensile and Data

CLARKE COUNTY

	Core No.	Average Density Mainline	Average Density C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	5			109.1	
	8			99.3	
	11			114.1	
	Avg.	2.293	2.285	107.5	83.1
Surface	1			115.9	
	5			129.8	
	11			130.5	
	Avg.	2.336	2.331	125.4	90.7

DECATUR COUNTY

	Core No.	Average Density Mainline	Average Density C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	5			134.3	
	6			103.0	
	11			128.1	
	Avg.	2.329	2.289	121.8	49.0
Surface	4			154.7	
	5			194.8	
	8			154.0	
	Avg.	2.439	2.302	167.8	126.9

HARRISON COUNTY

	Core No.	Average Density Mainline	Average Density C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	4			164.5	
	6			189.1	
	7			177.0	
	Avg.	2.362	-----	176.9	139.7
Surface	3			161.0	
	8			165.6	
	10			151.6	
	Avg.	2.324	-----	159.4	135.6

Individual Core Density,
Indirect Tensile and Data

MILLS COUNTY

	Core No.	Average Density Mainline	C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	3			301.8	
	6			284.7	
	10			288.3	
	Avg.	2.378	-----	291.6	246.8
Surface	5			151.3	
	7			153.5	
	8			158.1	
	Avg.	2.322	-----	154.3	127.8

POLK COUNTY

	Core No.	Average Density Mainline	C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	3			159.2	
	7			148.7	
	9			52.3	
	Avg.	2.367	-----	120.1	103.4
Surface	5			168.9	
	8			165.5	
	9			136.8	
	Avg.	2.396	-----	157.1	110.1

WOODBURY COUNTY

	Core No.	Average Density Mainline	C_L Core	Indirect Tensile PSI	Average Indirect Tensile PSI After F & T of 3 Cores
Binder	5			175.3	
	8			183.2	
	10			227.0	
	Avg.	2.398	-----	195.2	140.0
Surface	4			197.7	
	8			180.2	
	9			169.8	
	Avg.	2.372	-----	182.6	134.3

Individual Core Density,
Resilient Modulus and Data

CLARKE COUNTY
Binder

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
B-1	2.327	0.21E6	127.4	0.20E6	198.1
2	2.267				
3	2.287	0.28E6	87.3	0.26E6	146.2
4	2.323	0.24E6	89.6	0.25E6	134.6
5	2.238	0.21E6	141.8	0.20E6	224.5
6	2.279	0.25E6	115.3	0.22E6	192.9
7	2.271				
8	2.229				
9	2.354	0.22E6	106.6	0.21E6	163.1
10	2.261				
11	2.292				
12*	2.281				
13*	2.299				
Avg.	2.285	0.24E6	111.3	0.22E6	176.6

Individual Core Density,
Resilient Modulus and Data

CLARKE COUNTY
Surface

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
S-1	2.315				
2	2.366				
3	2.338	0.26E6	54.7	0.25E6	86.8
4	2.336	0.28E6	46.2	0.24E6	78.3
5	2.340				
6	2.328	0.28E6	52.3	0.29E6	77.8
7	2.351	0.29E6	47.5	0.30E6	71.4
8	2.317				
9	2.350	0.31E6	44.5	0.28E6	71.9
10	2.309	0.28E6	50.8	0.23E6	91.7
11	2.343				
12*	2.356				
13*	2.305				
Avg.	2.335	0.28E6	49.3	0.27E6	

*Centerline Cores

Individual Core Density,
Resilient Modulus and Data

DECATUR COUNTY
Binder

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
B-1	2.356	0.31E6	52.3	0.26E6	92.1
2	2.337	0.28E6	52.6	0.23E6	95.8
3	2.347	0.34E6	43.4	0.28E6	79.1
4	2.336	0.32E6	48.9	0.28E6	86.6
5	2.309	0.30E6	45.9	0.26E6	83.7
6	2.206	0.13E6	126.6	0.11E6	219.6
7	2.345				
8	2.355				
9	2.351				
10	2.330				
11	2.352				
12	2.250				
13	2.327				
Avg.	2.323	0.28E6	61.6	0.24E6	109.5

Core B-12 and B-13 are centerline cores

Individual Core Density,
Resilient Modulus and Data

DECATUR COUNTY
Surface

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
S-1	2.367	0.47E6	33.1	0.46E6	45.5
2	2.357	0.51E6	29.2	0.51E6	45.7
3	2.366	0.66E6	22.5	0.66E6	34.2
4	2.336				
5	2.357				
6	2.283				
7	2.374	0.60E6	24.0	0.59E6	37.2
8	2.336				
9	2.356	0.58E6	24.7	0.58E6	37.1
10	2.377	0.61E6	24.0	0.62E6	36.0
11	2.327				
12	2.268				
13	2.318				
14	2.319				
Avg.	2.339	0.57E6	26.3	0.57E6	39.3

Cores S-12 thru S-14 are centerline cores

Individual Core Density,
Resilient Modulus and Data

HARRISON COUNTY
Binder

Core No.	Density	Resilient Modulus 50 lbs. PSI	Resilient Modulus 50 lbs. HDEF	Resilient Modulus 75 lbs. PSI	Resilient Modulus 75 lbs. HDEF	Resilient Modulus After 50 Cycles of Freeze & Thaw 50 lbs. PSI	Resilient Modulus After 50 Cycles of Freeze & Thaw 50 lbs. HDEF	Resilient Modulus After 50 Cycles of Freeze & Thaw 75 lbs. PSI	Resilient Modulus After 50 Cycles of Freeze & Thaw 75 lbs. HDEF
B-1	2.327	0.27E6	117.7	0.25E6	200.1				
B-2	2.383	0.31E6	115.6	0.31E6	180.9	0.32E6	117.2	0.33E6	172.1
B-3	2.365	0.31E6	74.8	0.31E6	115.7				
B-4	2.360	0.46E6	56.9	0.42E6	96.0				
B-5	2.364	0.26E6	96.1	0.23E6	163.3	0.28E6	92.6	0.28E6	137.6
B-6	2.352								
B-7	2.386								
B-8	2.355	0.39E6	73.4	0.18E6	108.7	0.43E6	68.5	0.43E6	101.8
Avg.	2.362	0.33E6	71.4	0.28E6	144.1	0.34E6	92.8	0.35E6	137.1

Individual Core Density,
Resilient Modulus and Data

HARRISON COUNTY
Surface

Core No.	Density	Resilient Modulus 50 lbs. PSI	Resilient Modulus 50 lbs. HDEF	Resilient Modulus 75 lbs. PSI	Resilient Modulus 75 lbs. HDEF	Resilient Modulus After 50 Cycles of Freeze & Thaw 50 lbs. PSI	Resilient Modulus After 50 Cycles of Freeze & Thaw 50 lbs. HDEF	Resilient Modulus After 50 Cycles of Freeze & Thaw 75 lbs. PSI	Resilient Modulus After 50 Cycles of Freeze & Thaw 75 lbs. HDEF
S-1	2.316	0.34E6	50.1	0.36E6	68.1	0.32E6	52.6	0.29E6	83.1
S-2	2.321								
S-3	2.316	0.32E6	51.5	0.32E6	74.1				
S-4	2.326	0.35E6	45.8	0.39E6	66.4	0.39E6	40.9	0.36E6	66.1
S-5	2.312								
S-6	2.342	0.34E6	50.2	0.36E6	72.5				
S-7	2.336	0.36E6	40.7	0.42E6	54.4	0.37E6	39.4	0.36E6	60.4
S-8	2.334	0.36E6	45.4	0.36E6	69.5				
S-9	2.324	Cracked Core							
S-10	2.313								
Avg.	2.324	0.35E6	47.3	0.37E6	67.5	0.36E6	44.3	0.34E6	69.9

All R/M answers are an average of two readings per core (20 cycles)

Individual Core Density,
Resilient Modulus and Data

MILLS COUNTY
Binder

Core No.	Density	Resilient 50 lbs. PSI	HDEF	Modulus 75 lbs. PSI	HDEF
B-1					
2	2.381	1.07E6	26.9	0.99E6	44.1
	2.350	1.42E6	24.1	1.36E6	37.6
4	2.395	1.07E6	26.5	0.90E6	49.0
5	2.381	1.08E6	28.6	1.22E6	38.3
6	2.386	1.27E6	25.1	0.97E6	49.4
7	2.388				
8	2.372				
9	2.391	1.10E6	29.8	1.00E6	51.5
10	2.354				
Avg.	2.378	1.16E6	26.8	1.07E6	45.0

*Note high indirect tensile results compared to all others

Individual Core Density,
Resilient Modulus and Data

MILLS COUNTY
Surface

Core No.	Density	Resilient 50 lbs. PSI	HDEF	Modulus 75 lbs. PSI	HDEF
S-1	2.309				
2	2.300	0.46E6	38.5	0.54E6	50.4
3	2.325	0.49E6	40.9	0.51E6	59.8
4	2.333	0.51E6	34.5	0.53E6	51.5
5	2.333	0.54E6	37.8	0.57E6	55.8
6	2.333	0.58E6	33.6	0.57E6	53.0
7	2.326				
8	2.323				
9	2.311	0.47E6	39.2	0.46E6	62.8
10	2.324				
Avg.	2.322	0.51E6	37.4	0.53E6	55.6

Individual Core Density,
Resilient Modulus and Data

POLK COUNTY
Binder

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
B-1	2.395	0.29E6	81.0	0.28E6	122.7
2	2.381	0.37E6	59.6	0.31E6	120.4
3	2.382				
4*	*2.284				
5	2.390	0.35E6	64.6	0.34E6	102.0
6	2.376				
7	2.352	0.21E6	101.8	0.21E6	153.0
8	2.381	0.31E6	77.1	0.27E6	134.7
9	2.361	0.29E6	69.5	0.29E6	115.7
10	2.368				
Avg.	2.367	0.30E6	75.6	0.28E6	121.8

*Core #4 has a mud pocket

Individual Core Density,
Resilient Modulus and Data

POLK COUNTY
Surface

Core No.	Density	Resilient 50 lbs. PSI	Resilient 50 lbs. HDEF	Modulus 75 lbs. PSI	Modulus 75 lbs. HDEF
S-1	2.392	0.51E6	30.6	0.57E6	42.0
2	2.415	0.34E6	48.0	0.40E6	60.6
3	2.400	0.42E6	38.3	0.54E6	44.0
4	2.371				
5	2.423				
6	2.418	0.44E6	34.2	0.55E6	38.7
7	2.374				
8	2.385	0.29E6	56.0	0.45E6	51.0
9	2.368	0.30E6	60.6	0.27E6	96.1
10	2.412				
Avg.	2.396	0.38E6	44.6	0.49E6	55.4

Individual Core Density,
Resilient Modulus and Data

WOODBURY COUNTY
Binder

Core No.	Density	Resilient 50 lbs. PSI	HDEF	Modulus 75 lbs. PSI	HDEF
B-1	2.396	0.38E6	38.5	0.35E6	57.5
2	2.399	0.27E6	54.2	0.23E6	88.3
3	2.406	0.42E6	37.5	0.40E6	54.8
4	2.402				
5	2.397	0.37E6	37.2	0.34E6	56.7
6	2.389	0.39E6	34.2	0.34E6	54.4
7	2.410	0.46E6	39.9	0.37E6	51.4
8	2.406				
9	2.401				
10	2.374				
Avg.	2.398	0.38E6	38.6	0.34E6	60.5

Individual Core Density,
Resilient Modulus and Data

WOODBURY COUNTY
Surface

Core No.	Density	Resilient 50 lbs. PSI	HDEF	Modulus 75 lbs. PSI	HDEF
S-1	2.384	0.56E6	24.2	0.54E6	37.5
2	2.353	0.35E6	41.0	0.25E6	86.1
3	2.362				
4	2.389	0.43E6	30.3	0.36E6	54.1
5	2.372	0.44E6	31.3	0.41E6	49.1
6	2.379				
7	2.365				
8	2.368	0.36E6	36.6	0.39E6	50.2
9	2.362	0.50E6	28.3	0.39E6	52.7
10	2.382				
Avg.	2.372	0.37E6	32.0	0.35E6	55.0

Appendix B

INDIRECT TENSILE STRENGTH

Indirect Tensile Strength $(S_t) = \dfrac{2P}{\pi\ td}$

Where: S_t = tensile strength (psi)
P = maximum load (pounds)
t = specimen thickness (inches)
d = specimen diameter (inches)

RESILIENT MODULUS

Test Parameters: 77 ± 1°F
90° rotation @ 20 cycles ea.
Frequency .33 hz
Load Time 0.1 sec.
Tested @ 50 lb. & 75 lb.

11

Gravitational Water and Seepage

11.1. NATURE OF GRAVITATIONAL FLOW IN SOIL. The flow of gravitational water in soil is caused by the action of gravity which tends to pull the water downward to a lower elevation. It is similar in many respects to the free flow of water in a conduit or an open channel in that it is attributable to the gravitational pull which acts to overcome certain resistances to movement or flow of the water. Such resistances are due mainly to friction or drag along the surfaces of contact between the water and the conduit in free flow and to friction and viscous drag along the sidewalls of the pore spaces in the case of flow through soils. In hydraulics, gravity flow of water may be either laminar or turbulent in character, its nature depending on the velocity of flow and on the size, shape, and smoothness of the sides of the conduit or channel. In the study of gravitational flow in soils, we are primarily interested in the laminar type of flow, since the velocity of ground water rarely, if ever, becomes high enough to produce turbulence in the sense in which it is used here.

11.2. CHARACTERISTICS OF LAMINAR FLOW. Laminar flow is said to exist when all particles of water move in parallel paths and the lines of flow are not braided or intertwined as the water moves forward. The quantity of water flowing past a fixed point in a stated period of time is equal to the cross-sectional area of the water multiplied by the average

velocity of flow. This relationship may be expressed by the formula

$$Q = Av \qquad (11\text{-}1)$$

in which Q is the volume of flow per unit of time, such as cubic feet per day or cubic centimeters per minute; A is the cross-sectional area of flowing water, in square feet or square centimeters; and v is the velocity of flow, in feet per day or centimeters per minute.

11.3. HYDRAULIC GRADIENT. The driving force which causes water to flow may be represented by a quantity known as the *hydraulic gradient*. This is defined as the drop in head divided by the distance in which the drop occurs. It may be expressed by the relation

$$i = \frac{h}{d} \qquad (11\text{-}2)$$

in which i is the hydraulic gradient; h is the drop in head; and d is the distance in which the drop occurs.

For example, if an open channel is 5000 ft long and drops 50 ft in that distance, the hydraulic gradient is 50/5000 or 0.01.

11.4. DARCY'S LAW. The general relationship between hydraulic gradient and the character and velocity of flow is indicated in the diagram of Fig. 11-1. As the hydraulic gradient is increased through zones I and II, the flow remains laminar and the velocity increases in linear proportion to the gradient. At the boundary between zones II and III the flow breaks from laminar to turbulent and the proportional relationship between velocity and gradient no longer prevails.

Under decreasing hydraulic gradient, the flow remains turbulent through zones III and II and does not resume the laminar characteristic until the boundary between zones II and I is reached. Here the relationship between velocity and gradient again becomes linear and coincides with that for an increasing gradient.

The gravitational flow of water in soil is represented by the curve in zone I of Fig. 11-1 and we may write the equation

$$v = ki \qquad (11\text{-}3)$$

in which k is a proportionality constant.

By substituting this expression for v in Eq. (11-1), we obtain the relation

$$Q = Aki \qquad (11\text{-}4)$$

This relationship is general and may be applied to any situation in

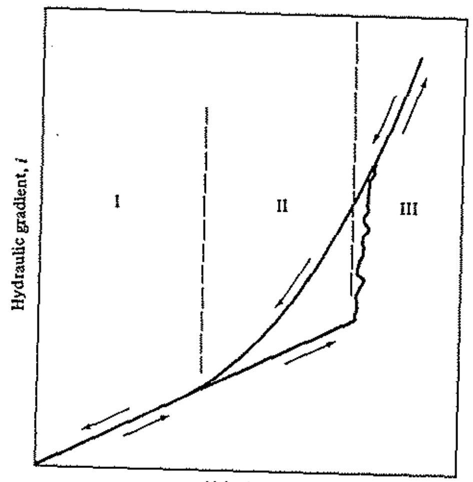


Fig. 11-1. Relationship between hydraulic gradient, velocity, and type of flow.

which the flow is laminar in character. It is an expression of Darcy's law applied to the flow of gravitational water through soil. A more general statement of the Darcy law is necessary in connection with the flow of other fluids through other types of porous media, but the foregoing statement is sufficient for the purpose of this discussion.

Let us assume that we have a conduit in which a mass of soil is placed in such a manner that all of the water flowing through the conduit must flow through the soil, as illustrated in Fig. 11-2. Since practically all the resistance to flow in this case is caused by the mass of soil, the value of the proportionality constant in Eq. (11-4) depends on the characteristics of the soil which influence the flow of water through its pores. The equation indicates that the quantity of water flowing through a given cross-sectional area of soil is equal to a constant multiplied by the hydraulic gradient.

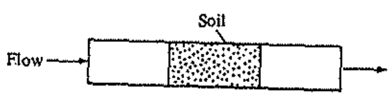


Fig. 11-2. Hypothetical flow through soil.

11.5. COEFFICIENT OF PERMEABILITY. The constant k in Eq. (11-4) is known as the *coefficient of permeability* or, more recently, as the *coefficient of hydraulic conductivity*. It constitutes an important property of soil, and its value depends largely on the size of the void spaces, which in turn depends on the size, shape, and state of packing of the soil grains. A clayey soil with very fine grains will have a very much lower permeability coefficient than will a sand with relatively coarse grains, even though the void ratio and the density of the two soils may be nearly the same. The reason is the greater resistance offered by the very much smaller pores or flow channels in the fine-grained soil through which the water must pass as it flows under the influence of a hydraulic gradient. From this standpoint, we may say that the coefficient of permeability is independent of the void ratio or density when we are comparing soils of different textural characteristics. On the other hand, when we consider the same soil in two different states of density, the permeability is dependent on the void ratio, since the soil grains are brought into closer contact by the process of compaction and densification. The pore spaces are reduced in size, and resistance to flow is increased.

11.6. VELOCITY OF APPROACH OF WATER. Attention is directed to the fact that, in the application of the Darcy law and Eq. (11-4), the cross-sectional area A is the area of the soil including both solids and void spaces. Obviously, the water cannot flow through the solids, but must pass only through the void spaces. Therefore, the velocity ki in Eq. (11-4) is a factitious velocity at which the water would have to flow through the whole area A in order to yield the quantity of water Q which actually passes through the soil. This factitious velocity is referred to as the "velocity of approach" or the "superficial velocity" of the water just before entering, or after leaving, the soil mass.

A dimensional analysis of Eq. (11-4) indicates that the coefficient of permeability k has the dimensions of a velocity, that is, a distance divided by time. Therefore, permeability is sometimes defined as "the superficial velocity of water flowing through soil under unit hydraulic gradient."

11.7. COEFFICIENT OF PERCOLATION. If the actual velocity of flow through the pores of the soil is considered, then the corresponding area which must be used in writing the flow equation is the area of the pore spaces cut by a typical cross section of the soil. The Darcy equation for this case is

$$Q = A_v k_p i \qquad (11\text{-}5)$$

in which A_v is the area of pore spaces in a soil cross section; and k_p is a proportionality constant.

The product $k_p i$ in this case is equal to the average actual velocity of the water through the soil pores. Since the area of the pores in any cross section will always be less than the total area, it is obvious that this actual velocity will always be greater than the velocity of approach. The proportionality constant k_p is called the *coefficient of percolation*, and it always has a greater value than the coefficient of permeability for any given soil.

11.8. RELATION BETWEEN COEFFICIENTS OF PERMEABILITY AND PERCOLATION. The distinction between the two flow coefficients should be clearly understood by the student. The coefficient of percolation refers to the average actual velocity of water flowing through the actual pore area of the soil; whereas, the coefficient of permeability refers to a factitious velocity of flow through the total area of solids plus pore spaces, as pointed out in Section 11.6. Since, as a rule, the total area of soil is more conveniently determined in gravitational flow problems, the permeability coefficient is used more often than the percolation coefficient.

The area of the pore spaces in a typical cross section of soil is equal to the total area multiplied by the porosity. It therefore follows that the coefficient of permeability of the soil is equal to the coefficient of percolation multiplied by the porosity. Thus,

$$A_v = nA \tag{11-6}$$

By substituting this value of A_v in Eq. (11-5) and setting the result thus obtained equal to the expressions for Q given by Eq. (11-4), we get

$$Aki = nAk_p i \tag{11-7}$$

from which

$$k = nk_p \tag{11-8}$$

11.9. APPLICATIONS OF PERMEABILITY CHARACTERISTICS OF SOIL. There are numerous types of problems in connection with engineering projects which require knowledge of the permeability characteristics of the soil involved, such as computations of seepage through earth dams and levees and losses from irrigation ditches. Estimates of pumpage-capacity requirements for unwatering cofferdams or excavations below a water table are familiar examples of such problems. The spacing and depth of underdrains for lowering the water table under a road or runway in order to improve subgrade stability or for draining waterlogged agricultural land is another type of problem in which the permeability of the soil is of

paramount importance. Also, the rate of settlement of a structure resting on a soil foundation is a function of the rate at which water moves through and out of the foundation soil.

11.10. MEASUREMENT OF PERMEABILITY CHARACTERISTICS OF SOIL. Several methods of measuring permeability characteristics of soils are available. Some methods involve laboratory procedures on disturbed or undisturbed samples, and others are adapted to determination of the permeability of the soil in place below a water table. Each of these procedures has advantages which are important in different types of problems; and the method which is most feasible and appropriate for the particular problem in hand should be chosen. For example, in studying the seepage through a rolled earth dam, it would be appropriate to make a laboratory type of test on a sample of the soil to be used which would be compacted to the same density as in the prototype structure. On the other hand, a field test of the soil in place would be more appropriate in the case of studies relating to the unwatering of an excavation. In every case, the objective should be to determine the permeability of the soil in its natural or normal operating condition or to do so as nearly as is possible. Furthermore, soils in nature are frequently nonisotropic with respect to flow; that is, the coefficient of permeability in the vertical direction may differ considerably from that in the horizontal direction. If this condition exists, it may be necessary to measure the permeability in both directions.

11.11. TEST WITH CONSTANT-HEAD PERMEAMETER. A laboratory test which is particularly adapted to determination of the coefficient of perme-

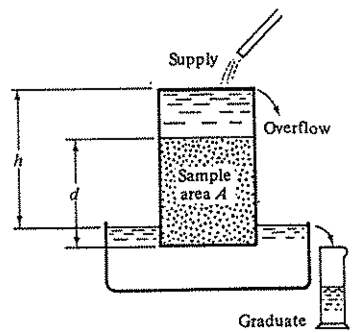


Fig. 11-3. Constant-head permeameter.

ability of relatively coarse-grained soils is one in which the hydraulic gradient is held constant throughout the testing period. A typical arrangement of apparatus for this test, called a constant-head permeameter, is shown in Fig. 11-3.

In the conduct of the test, all the water passing through the soil sample in a measured period of time is collected, and the quantity is measured. This quantity of water and the appropriate dimensions of the apparatus and the soil sample are substituted in Eq. (11-4), and a value of the permeability coefficient is obtained.

EXAMPLE 11-1. A soil sample in a constant-head permeameter is 6 in. in diameter and 8 in. long. The vertical distance from headwater to tailwater is 11 in. In a test run, 766 lb of water passes through the sample in 4 hr 15 min. Determine the coefficient of permeability.

SOLUTION: From this test, $h = 11$ in. and $d = 8$ in.; and

$$i = \frac{h}{d} = 1.375$$

Also, $A = 28.27$ sq in. $= 0.196$ sq ft and

$$Q = \frac{766}{62.4 \times 255} = 0.048 \text{ cfm}$$

Substituting these values in Eq. (11-4), we obtain

$$0.048 = 0.196 \times k \times 1.375$$

from which

$$k = 0.178 \text{ fpm}$$

In computing the value of the permeability coefficient from data obtained in a test of this type, as in all permeability problems, it is important to keep the computations dimensionally correct. A relatively easy and sure way to do this is to decide in advance the units in which the coefficient of permeability is desired. Then reduce the values of Q and A to those units before making the computation. In the preceding example, Q and A were reduced to feet and minutes and the resulting value of k was expressed in feet per minute. Since the hydraulic gradient is a dimensionless quantity, the units of h and d are not important, provided the same units are used for both distances.

11.12. TEST WITH FALLING-HEAD PERMEAMETER. Another laboratory test, which is more appropriate in the case of fine-grained soils, is called a

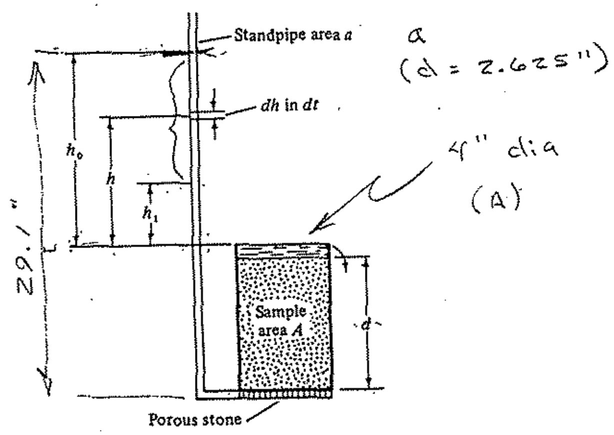


Fig. 11-4. Falling-head permeameter.

variable-head permeameter or *falling-head permeameter.* A typical arrangement of apparatus for this test is shown in Fig. 11-4. In the conduct of the test, the water passing through the soil sample causes water in the standpipe to drop from h_0 to h_1 in a measured period of time t_1. The head on the sample at any time t between the start and finish of the test is h; and, in any increment of time dt, there is a decrease in head equal to dh. From these facts, the following relationships may be written:†

$$k \frac{h}{d} A = -a \frac{dh}{dt} \tag{11-9}$$

Then

$$k \frac{A}{d} \int_0^{t_1} dt = -a \int_{h_0}^{h_1} \frac{dh}{h} \tag{11-10}$$

from which

$h_0 = 29.1 - d$

$$k = \frac{ad}{At_1} \log_e \frac{h_0}{h_1} \tag{11-11}$$

EXAMPLE 11-2. A sample of clay soil, having a cross-sectional area

†The minus sign in Eq. (11-9) is appropriate because the head decreases with elapsed time.

of 78.5 sq cm and a height of 5 cm, is placed in a falling-head permeameter in which the area of the standpipe is 0.53 sq cm. In a test run, the head on the sample drops from 80 cm to 38 cm in 1 hr 24 min 18 sec. What is the coefficient of permeability of the soil?

SOLUTION: From this test, $a = 0.53$ sq cm; $t_1 = 1$ hr 24 min 18 sec = 84.3 min; $h_0 = 80$ cm; $d = 5$ cm; and $A = 78.5$ sq cm.

Substitution in Eq. (11-11) gives†

$$k = \frac{0.53 \times 5}{78.5 \times 84.3} \log_e \frac{80}{38} = 0.000299 \text{ cm/min}$$

11.13. EFFECT OF AIR IN PORES. The permeability of the soil sample in either of the two laboratory tests just described may be affected appreciably by pocketed bubbles of air in the soil pores. Attempts should be made to eliminate entrapped air from the sample by passing water through it for a considerable period of time before a test run is made. Also, since difficulty may be encountered if dissolved air is released from the permeating water and trapped in the pores as the water passes through the soil, it is advisable to use air-free or distilled water as the permeate. Furthermore, since water tends to absorb air as it cools and to release dissolved air as it warms up, the temperature of the permeating water should preferably be somewhat higher than that of the soil sample. This precaution not only will prevent air from being released in the soil, but may assist in removing entrapped air in the pores since the water will be cooled as it passes through the soil and will have a tendency to absorb air.

11.14. EFFECT OF VISCOSITY OF WATER. The coefficient of permeability is primarily influenced by the size and shape, or tortuousness, of the soil pores and by the roughness of the mineral particles of the soil. However, it is also affected by the viscosity of the permeating water. Since the viscosity of water is a function of its temperature, it may be advisable in some cases to correct the laboratory-measured permeability coefficient for temperature difference between that of the laboratory water and that of the water which will flow through the prototype structure. For example, laboratory measurements of permeability may be made at a room temperature of say 80°F, whereas it is known that the temperature of the seepage water through the prototype structure will be in the neighborhood of 50°F. The coefficient determined in the laboratory may be too high in this case because the viscosity of 80° water is less than of 50° water.

A correction factor for the permeability coefficient with water at

†$\log_e N = 2.303 \log_{10} N$.

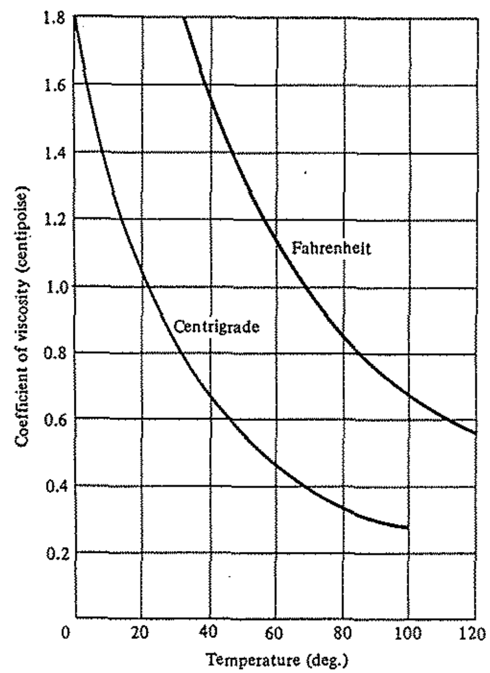


Fig. 11-5. Correction factor to permeability with water at 20.20°C.

various temperatures may be determined on the basis of the relationship between temperature and the coefficient of viscosity for water. The unit of the coefficient of viscosity in the metric system is the dyne-second per square centimeter and is called the *poise*. The coefficient of viscosity of water at 20.20°C (68.36°F) is 0.01 poise or 1 centipoise. The curves in Fig. 11-5 show the values of this coefficient, in centipoises, for a range of temperatures on both the centigrade and Fahrenheit scales.

Since the coefficient of permeability is inversely proportional to the viscosity of the permeating water and directly proportional to its temperature, the coefficient of viscosity can be used as a correction factor by which the permeability determined at one temperature can be reduced to that at the base temperature of 20.20°C.

Appendix C

To Office: District #5 Office | Date: September 7, 1989

Attention: Pete Tollenaere | Ref: 435.2402

From: F. E. Neff

Office: District #5 Materials

Subject: Research Coring on I-35 Decatur Co.

Coring conducted 9-6-89 by F. E. Neff and C. Proper, District #5 Materials, with District #5 Materials core machine. Cores delivered to Pete Tollenaere's office 9-7-89. The core holes were left open by direction of Pete Tollenaere. Results of core length, material and presence of water, note below. All cores taken in southbound inside (passing) lane.

Core #1: Sta. 239+00+ 14" Lt. of CL
Thickness 17", 4 3/4" AC surface, 8½" concrete
4" asphalt base
Note: Hole checked approximately 5 hours after coring, had filled about 1/2 full of water.

Core #2: Sta. 239+00+ on 1/4 point 5'+ Lt. of CL
Thickness 17", 4" AC surface, 8½" concrete
4½" asphalt base. Little to no water in this hole when checked.

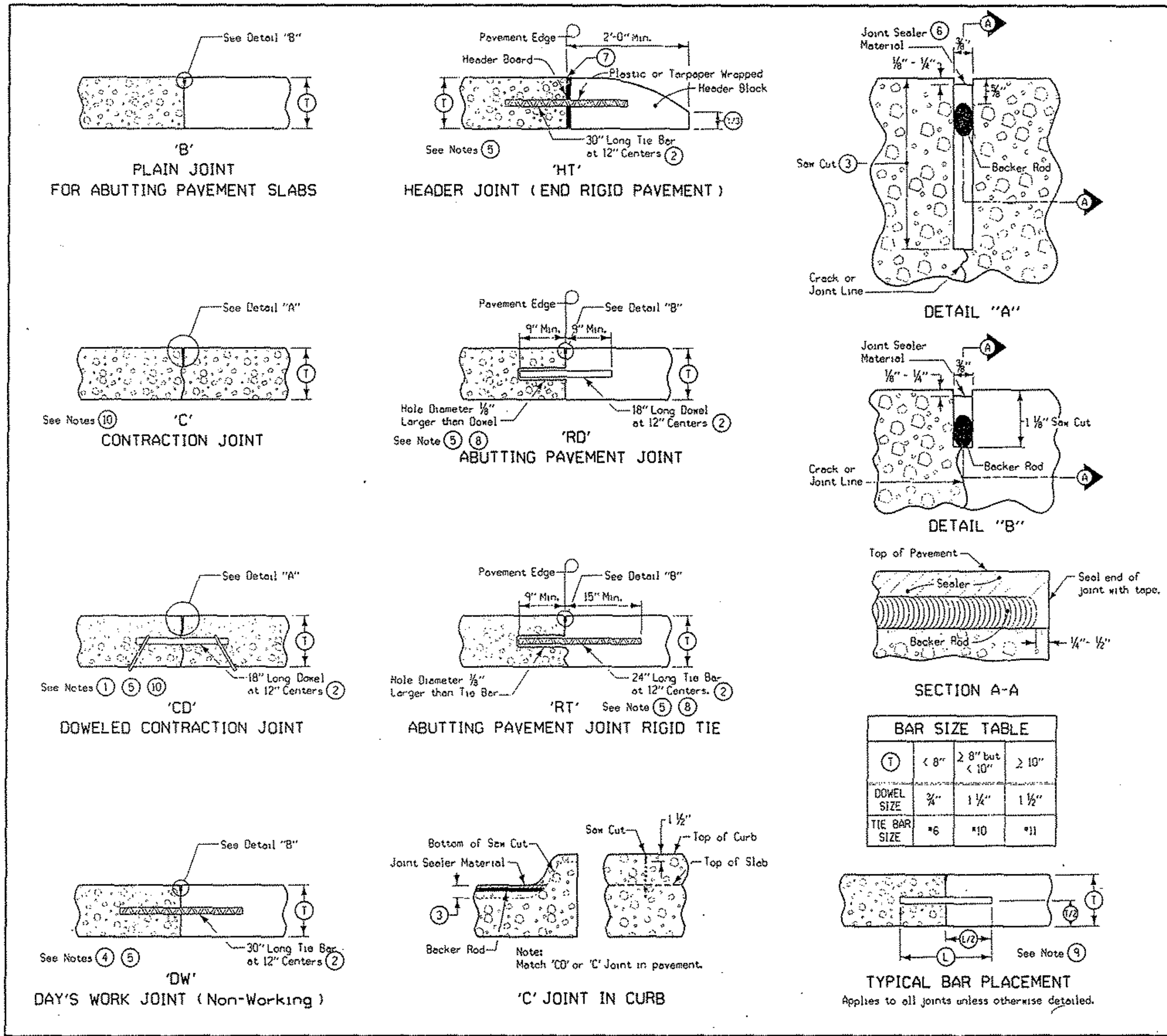
Core #3: Sta. 241+75+ 4" Lt. of CL
Thickness 16", 5½" AC Surface, 8" concrete
2½" asphalt base. Base core unable to remove in tact. No water at time of coring, did not check later.

Core #4: Sta. 241+75+ on 1/4 point
Thickness 15", 5¾" AC surface, 7¼" concrete
2½" asphalt base. Unable to remove base core but broken through to determine thickness.

Core #5: Sta. 230+00+ on 1/4 point
Thickness 16", 5½" AC surface, 8" concrete
2½" asphalt base. Unable to remove base core but broken out to determine thickness. After coring, water seeping in slow, unable to tell for sure if between base and concrete or from under base.

Core #6: Sta. 230+00+ on CL
Thickness 16", 5½" AC surface, 8½" concrete
2¼" asphalt base. Unable to remove base core but broken out to determine thickness. As concrete core was removed and water removed from hole, observed water running into the hole quite fast. Appeared to be coming from between base and conrete, but could have been coming up around outside of base core. Removed base, cleaned hole and within ten (10) minutes the hole had 12 or more inches of water. Cleaned hole again and again and water returned quite fast. Shoulder drain outlet on right shoulder at this location was found to have some water in it but not running.

FEN/ed
cc: T. McDonald
S. Moussalli
P. Chapman



BAR SIZE TABLE

T	< 8"	≥ 8" but < 10"	≥ 10"
DOWEL SIZE	3/4"	1 1/4"	1 1/2"
TIE BAR SIZE	#6	#10	#11

GENERAL NOTES:

All materials and construction features used in the construction of pavement joints shall conform to the requirements of current Standard Specifications. Refer to other appropriate Standard Road Plans and project plans for additional information. Alternate methods for construction of joints may be submitted to the Engineer for consideration.

Dowels for the 'CD' joint shall be properly positioned by the use of an approved support assembly.

Tie bars shall be held in place by approved devices or methods approved by the Engineer. Bars placed after concrete slab is poured shall be installed prior to vibration of pavement slab.

Epoxy coat all bars (smooth and tie bars), see "Pavement Reinforcement" in the current Standard Specifications.

The joints as detailed hereon shall not be measured for payment. The construction detailed hereon including the furnishing of the dowels, dowel assemblies, and joint filler material shall be considered incidental to PCC paving, unless noted otherwise.

SPECIAL NOTES:

1. The free moving ends of dowel support assembly shall be placed alternately across joints.
2. Refer to Bar Size Table.
3. Depth of saw cut shall be T/3 except 'C' joint shall be T/4.
4. 'DW' Joint shall be located at the midpoint between future 'C' or 'CD' Joints, with a tolerance of ±2 feet. Do not place at a location of a future 'C' or 'CD' Joint.
5. Bars in Transverse Joints shall be placed so that no bar will be closer than 6" to any Longitudinal Joint (centerline or laneline). The distance to the first bar from edge of pavement will vary from 6" to 12" depending upon pavement width.
6. Silicone Joint Sealer shall be used for Interstate pavement, and associated work. Refer to the current Standard Specification on "Sealing Joints".
7. Edge with 1/4" tool for length of joint indicated if formed; edging not required when cut with diamond blade saw. Remove header block and board when second slab is poured. The joint shall be cut and sealed as shown in Detail 'B'.
8. Placement of dowels or tie bars shall be in accordance with the current Standard Specification on "Placing Reinforcement". The method of anchoring bars into existing pavement shall be as approved by the Engineer as set forth in appropriate Materials Instructional Memorandums.
9. When tieing into old pavement, (T) represents the depth of sound Portland Cement Concrete.
10. Unless otherwise specified, transverse contraction joints in mainline pavement shall be 'CD' when (T) is greater or equal to 8". 'C' when (T) is less than 8".

REVISION: Changed "Backer Rope" to "Backer Rod". Revised general notes.

NO.	DATE
3	01-07-92

Iowa Department of Transportation
Highway Division

STANDARD ROAD PLAN RH-50

Donald L. East — 01-03-92
RECOMMENDED BY ROAD DESIGN ENGINEER — DATE

George F. Sisson — 01-03-92
APPROVED BY DEPUTY DIRECTOR DEVELOPMENT — DATE

JOINTS
(TRANSVERSE CONTRACTION)