

GEORGIA INSTITUTE OF TECHNOLOGY
OFFICE OF CONTRACT ADMINISTRATION
SPONSORED PROJECT INITIATION

No action
B. J. G.

Date: 7/27/78

Project Title: A Study of Factors Affecting Crushed Stone Base Course Performance

Project No: E-20-639

Project Director: Dr. R. D. Barksdale

Sponsor: Georgia Department of Transportation

4/31/83
7/31/82
4/20/82
June 30 - 1978

Agreement Period: From 7/17/78 Until 7/16/80

Type Agreement: GDOT No. 7603

Amount: \$140,591 (GDOT E-20-639)
31,441 (GIT E-20-313)
\$172,032 TOTAL

Reports Required: Quarterly Progress Reports, Final Project Report, Executive Summary

Sponsor Contact Person (s):

Technical Matters

Wouter Gulden, P.E., Chief
Pavement and Physical Research Branch
Georgia Department of Transportation
Research and Development Bureau
Office of Materials and Research
15 Kennedy Drive
Forest Park, GA 30050

Contractual Matters

(thru OCA)

Mr. Hugh Tyner, Chief
Research and Development Bureau
Georgia Department of Transportation
15 Kennedy Drive
Forest Park, GA 30050

Defense Priority Rating: N/A

Assigned to: Civil Engineering (School/Laboratory)

COPIES TO:

Project Director
Division Chief (EES)
School/Laboratory Director
Dean/Director - EES
Accounting Office
Procurement Office
Security Coordinator (OCA)
Reports Coordinator (OCA)

Library, Technical Reports Section
EES Information Office
EES Reports & Procedures
Project File (OCA)
Project Code (GTRI)
Other

SPONSORED PROJECT TERMINATION SHEET

Date 7/6/83

Project Title: A Study of Factors Affecting Crushed Stone Base Course Performance

Project No: E-20-639

Project Director: Dr. R. D. Barksdale

Sponsor: Georgia DOT

Effective Termination Date: 1/31/83

Clearance of Accounting Charges: 1/31/83

Grant/Contract Closeout Actions Remaining:

None

- Final Invoice ~~and Closing Documents~~ 5/27/83
- Final Fiscal Report
- Final Report of Inventions
- Govt. Property Inventory & Related Certificate
- Classified Material Certificate
- Other _____

Assigned to: Civil Engineering (School/~~Laboratory~~)

COPIES TO:

- | | | |
|---------------------------------------|---------------------------------|--------------------------|
| Administrative Coordinator | Research Administrative Network | |
| Research Property Management | Research Security Services | EES Public Relations (2) |
| Accounting | Reports Coordinator (OCA) | Computer Input |
| Procurement/EES Supply Services | Legal Services (OCA) | Project File |
| | Library | Other <u>Barksdale</u> |

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

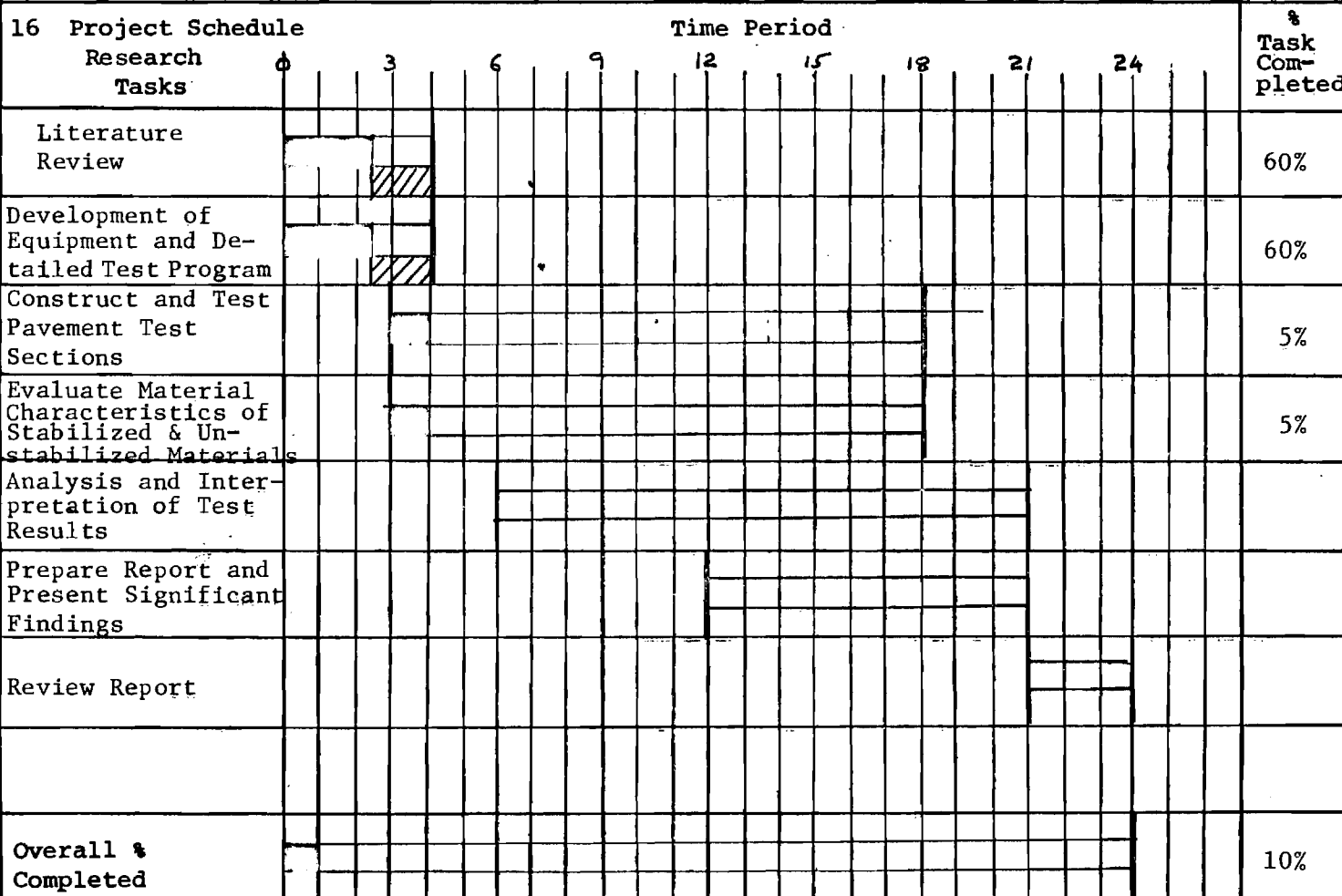
Date of Report
 October 13, 1978

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>1</u> From <u>7/17/1978</u> To <u>9/30/1978</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q. L. Robnett, Co-Director Warren F. Bailey, Jr., Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 10%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended				
11 Total GDOT/FHWA \$140,591 Ga. Tech \$31,441	12 Current Fiscal Year \$1875.95	13 Total to Date \$1875.95	% 0	14 Current Fiscal Year \$1875.95	% 0	15 Report Quarter \$1875.95



Approved Schedule
 Work Completed Schedule
 Projected Completion Schedule

17 Progress This Quarter (By Task)

Literature Review: Well advanced. A considerable body of relevant literature has been assembled and assessed.

Development of Equipment and Detailed Test Program: Assembly and calibration of equipment on hand almost complete. Plans for loading system partially complete; fabrication in machine shop under way for items already detailed. Test pit raised concrete floor has been almost completed.

18 Work Planned for Next Quarter

1. Completion of design and fabrication of repetitive loading system..!
2. Acquisition of materials, blending, and filling test pit to required densities.
3. Simultaneously with (2) above, placing of instrumentation in test pit.
4. Calibration of test pit instrumentation.
5. Evaluation of material characteristics.

19 Significant Technical Information, Recommendations, Implementation

NONE

20 Problems

NONE

Report Prepared by, A

Richard D. Barksdale
Signature

Richard D. Barksdale
Name

Project Director
Title

E-20639

GEORGIA INSTITUTE OF TECHNOLOGY
ATLANTA, GEORGIA 30332

SCHOOL OF
CIVIL ENGINEERING

TELEPHONE:
(404) 894-2284

January 8, 1979

Mr. Warren F. Bailey, Jr.
Research Engineer
Georgia Dept. of Transportation
Office of Materials and Research
15 Kennedy Drive
Forest Park, Georgia 30050

Dear Warren:

Enclosed are seven copies of Quarterly Progress Report Number 2 for Georgia DOT Research Project 7603, "A Study of Factors Affecting Crushed Stone Base Course Performance". If you have any questions concerning this quarterly progress report or the progress of the project in general, please contact either myself or Dr. Robnett.

Sincerely,

Richard D. Barksdale, P.E.
Professor

RDB:vc
Enclosures (7)

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report
 January 5, 1979

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>2</u> From <u>10/1/78</u> To <u>12/31/78</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q. L. Robnett, Co-Director Warren F. Bailey, Jr., Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 24%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized			Funds Expended			
11 Total GDOT/FHWA \$140,591 Ga. Tech \$31,441	12 Current Fiscal Year	13 Total to Date \$15,077	%	14 Current Fiscal Year \$14,828.70	%	15 Report Quarter \$14,828.70 (8.6%)*
			8.8		8.6	

16 Project Schedule Research Tasks	Jan. '78 Time Period												% Task Completed		
	1	2	3	4	5	6	7	8	9	12	18	21		24	
Literature Review	[Approved Schedule]														100%
Development of Equipment & Detailed Test Program	[Approved Schedule]			[Projected Completion Schedule]											90%
Construct and Test Pavement Test Sections			[Approved Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	10%
Evaluate Material Characteristics of Stabilized & Un-Stabilized Materials			[Approved Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	[Projected Completion Schedule]	10%
Analysis and Interpretation of Test Results						[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	0
Prepare Report and Present Significant Findings										[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	[Approved Schedule]	0
Review Report													[Approved Schedule]	[Approved Schedule]	0
Overall % Completed	[Approved Schedule]												10%		

Approved Schedule
 Work Completed Schedule
 Projected Completion Schedule
 *Based on GADOT/FHWA and Georgia Tech Matching Funds

17 Progress This Quarter (By Task)

- a. Literature Review: Completed. A considerable body of relevant literature has been assembled and assessed.
- b. Development of Equipment and Detailed Test Program: Loading equipment and reaction assembly designed. Loading system has been fabricated. Test pit raised concrete floor has been completed and storage areas constructed. The facility will be ready for testing by February 1, 1979. Standard subgrade soil has been selected and first portion of testing program firmed up at meeting with Georgia DOT.
- c. Design and Calibration of Pressure Cells and Bison Strain Coils: Two slightly different pressure cells have been designed and final fabrication was completed. A switching box for the Bison coils has been designed and constructed. Bison coils have been wired and calibration is being completed.
- d. Repeated Load Triaxial Tests: The apparatus for performing repeated load triaxial tests has been set up and calibrated.

18 Work Planned for Next Quarter

1. Complete assembly of repetitive loading system.
2. Fill test pit to required densities and begin testing.
3. Simultaneously with (2) above, place strain coils and pressure cells in test pit.
4. Complete calibration of pressure cells for pit.
5. Begin evaluation of material characteristics using repeated load test apparatus.

19 Significant Technical Information, Recommendations, Implementation

No significant technical information has been developed to date.

20 Problems

No important problems have developed to date.

21 Report Prepared by *A /*

Richard D. Barksdale

Project Director

Signature

Name

Title

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report
 April 27, 1979

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>3</u> From <u>01/01/79</u> To <u>03/31/79</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q. L. Robnett, Co-Director Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 38 %	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized	Funds Expended												
11 Total GDOT/FHWA \$140,591 (Ga. Tech) (\$31,441)	<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <th style="width: 15%;">12 Current Fiscal Year</th> <th style="width: 15%;">13 Total to Date</th> <th style="width: 15%;">%</th> <th style="width: 15%;">14 Current Fiscal Year</th> <th style="width: 15%;">%</th> <th style="width: 20%;">15 Report Quarter</th> </tr> <tr> <td></td> <td>34,515.44 (1,348.00**)</td> <td>24.5</td> <td>34,515.44 (21%)*</td> <td>24.5</td> <td>18,428.91 (10.7%)*</td> </tr> </table>	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter		34,515.44 (1,348.00**)	24.5	34,515.44 (21%)*	24.5	18,428.91 (10.7%)*
12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter								
	34,515.44 (1,348.00**)	24.5	34,515.44 (21%)*	24.5	18,428.91 (10.7%)*								

16 Project Schedule Research Tasks	Jan. '78 Time Period												% Task Completed		
	1	2	3	4	5	6	7	8	9	12	18	21		24	
Literature Review	[Dotted]														100%
Development of Equipment & Detailed Test Program	[Dotted]														100%
Construct and Test Pavement Test Sections			[Dotted]										20%		
Evaluate Material Characteristics of Stabilized & Un-Stabilized Materials			[Dotted]						[Hatched]						15%
Analysis and Interpretation of Test Results				[Dotted]									0		
Prepare Report and Present Significant Findings										[Dotted]				0	
Review Report												[Dotted]		0	
Overall % Completed	[Dotted]												20%		

Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

*Based on GADOT/FHWA and Georgia Tech Matching Funds
 ** Spent on Ga. Tech Matching Funds

17 Progress This Quarter

1. Test apparatus has been fabricated and checked out
2. All instrumentation for the test pit has been fabricated and calibrated.
3. Developed detailed plan for placing instrumentation and filling of test pit.
4. Test pit has been filled to top of base course

18. Work Planned for Next Quarter

1. Complete filling test pit to required densities and testing one test section.
2. Fill test pit with full instrumentation and begin testing second section.
3. Evaluation of material characteristics using repeated load test apparatus.
4. Begin evaluating test section results.

19. Significant Technical Information, Recommendations, Implementation

No significant technical information has been developed to date.

20. Problems

No important problems have developed to date.

21. Report Prepared by

SIGNATURE

Richard D. Barksdale

NAME

Project Director

TITLE

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

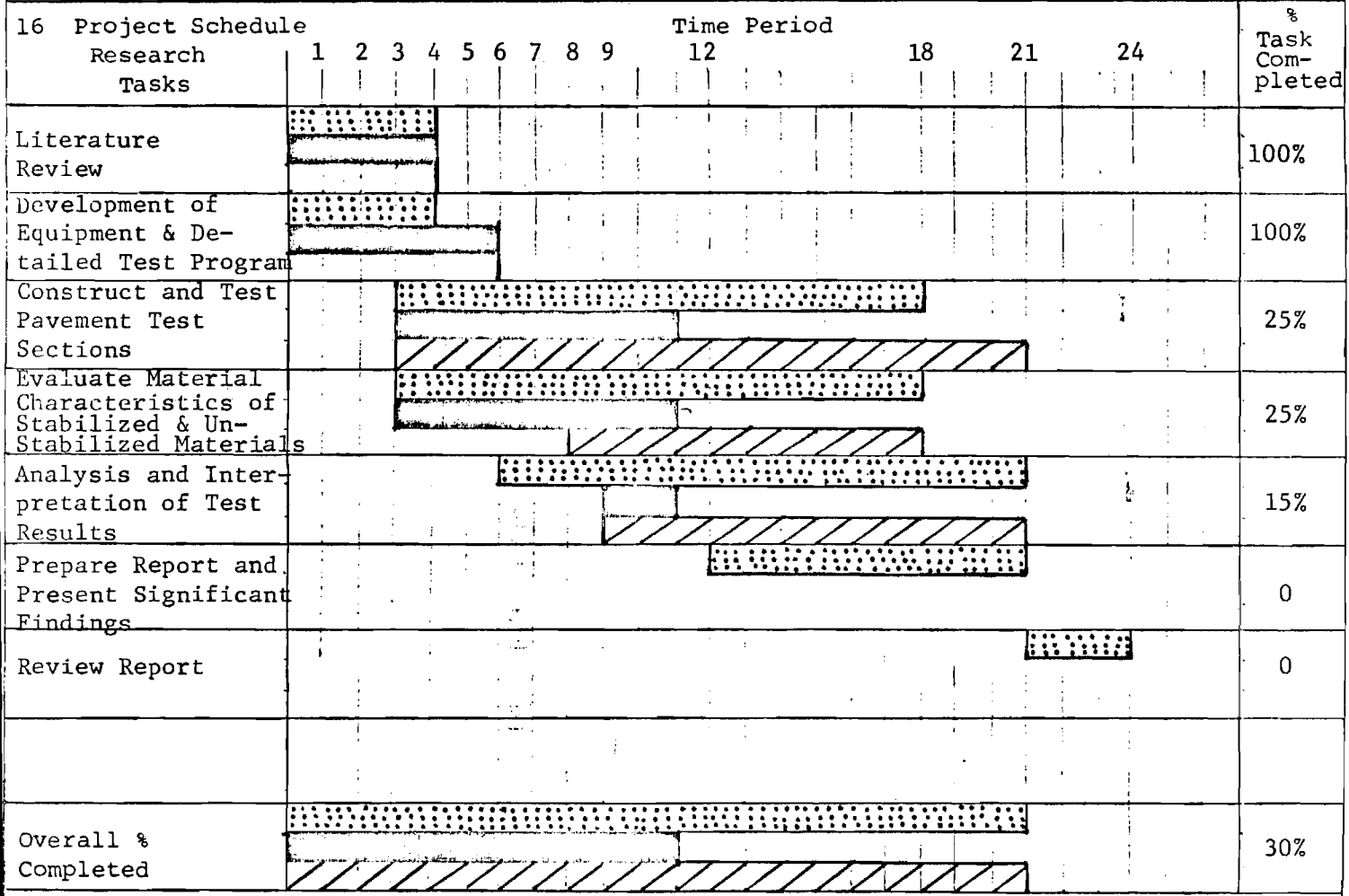
Date of Report
 August 10, 1979

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>4</u> From <u>04/01/79</u> To <u>06/30/79</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 55%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended			
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$54,301.20	% 	14 Current Fiscal Year \$19,785.76	15 Report Quarter \$19,785.76



Approved Schedule
 Work Completed Schedule
 Projected Completion Schedule

17 Progress This Quarter (By Task)

- Task 3. Test section with 12 in. crushed stone base and 3.5 in. AC surfacing was tested to about 1.6 million load repetitions with no sign of structural distress.
- Task 4. Resilient properties of compacted subgrade were evaluated.
- Task 5. The data from the test pavement to date has been reduced and is currently being analyzed.

18 Work Planned for Next Quarter

- 1. Test to failure both the 12 inch and 8 inch thick base sections.
- 2. Evaluate the properties of the crushed stone base course material.
- 3. Begin filling the pit for the next test series.

19 Significant Technical Information, Recommendations, Implementation

- 1. The 3.5 in. AC surfacing with 12 inch crushed stone base has under carefully controlled conditions performed very good.

20 Problems

No significant problems.

21 Report

Signature

Richard D. Barksdale

Name

Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

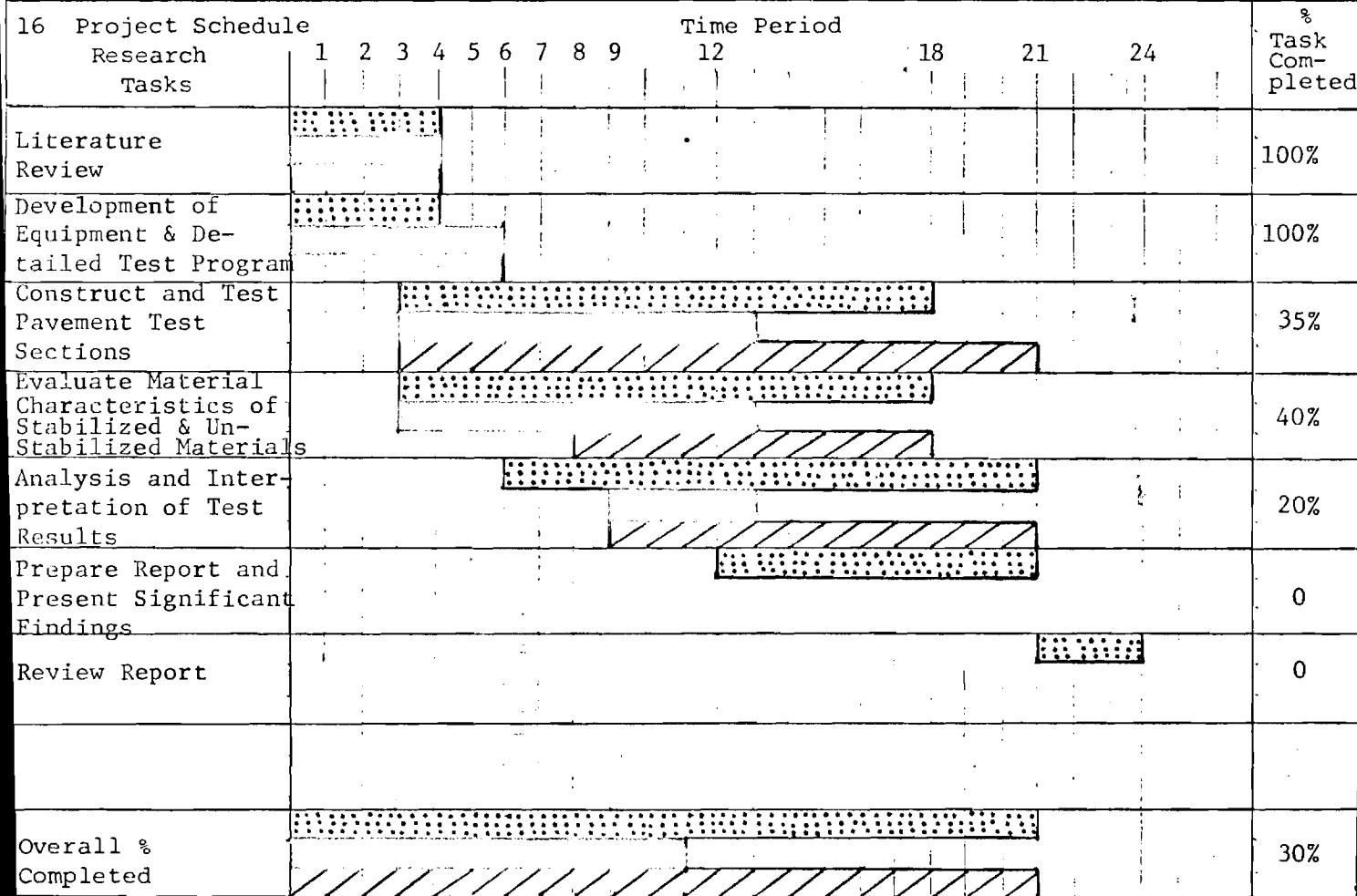
Date of Report
 August 10, 1979

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>5</u> From <u>07/01/79</u> To <u>09/30/79</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 67%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended				
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$70,913.16	%	14 Current Fiscal Year \$36,397.72	%	15 Report Quarter \$16,611.96



[Dotted] Approved Schedule [Solid] Work Completed Schedule [Hatched] Projected Completion Schedule

17 Progress This Quarter (By Task)

- Task 3. Test sections with 8 in. and 12 in. crushed stone base and 3.5 in. AC surfacing were tested to failure based on preliminary failure criteria of 1/2 in. of surface rutting.
- Task 4. Resilient properties of compacted subgrade were evaluated.
- Task 5. The data from the test pavement to date has been reduced and analyzed.

18 Work Planned for Next Quarter

1. Fill the test pit for the next test series which will be two full depth asphalt concrete sections.
2. Evaluate the properties of the crushed stone base course material.
3. Begin load testing of the full depth sections.

19 Significant Technical Information, Recommendations, Implementation

1. Both the 3.5 in. AC pavements with 8 in. and 12 in. crushed stone bases have under carefully controlled laboratory conditions, performed very good.

20 Problems

No significant problems.

21 Report Prepared by A. A.

Richard D. Barksdale

Director

Signature

Name

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

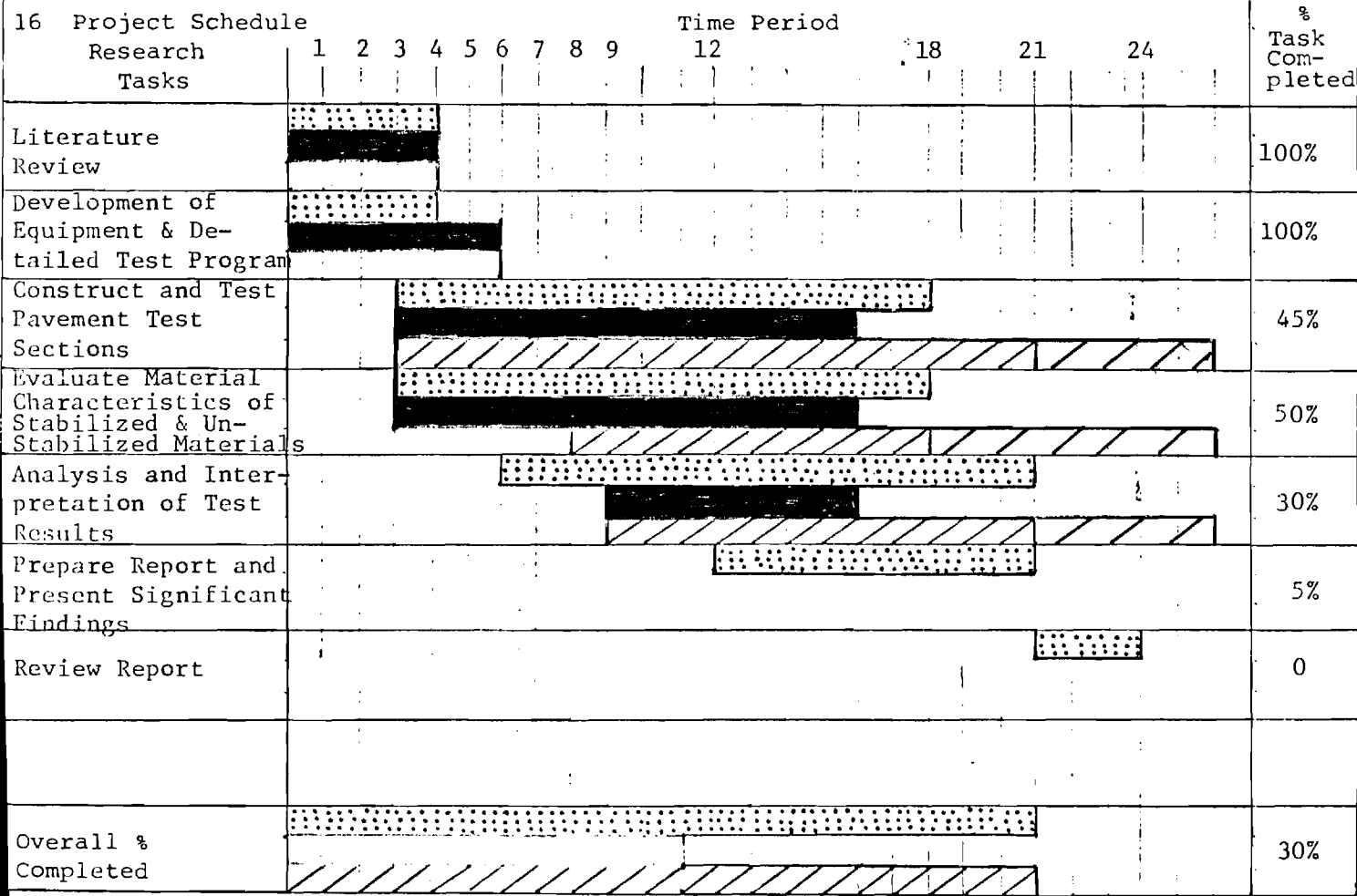
Date of Report
 January 28, 1980

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>6</u> From <u>10/01/79</u> To <u>12/31/79</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 77%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized				Funds Expended			
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$80,283.00	%	14 Current Fiscal Year \$25,981.47	%	15 Report Quarter \$ 9,369.51	
			57		18		



Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. The test section with an 8 in. crushed stone base was failed; the new subgrade was placed and instrumentation prepared for the full-depth asphalt concrete test sections. A second loading system was constructed so that the two sections in the pit could be simultaneously tested.

Task 4. Resilient properties of compacted subgrade soil were analyzed.

Task 5. The data from the 8 in. thick test section was reduced and analyzed.

18 Work Planned for Next Quarter

1. Place the full depth asphalt concrete for the second test series.
2. Begin load testing of the full depth sections.
3. Complete the evaluation of the properties of the crushed stone base course material used for the first series of tests.

19 Significant Technical Information, Recommendations, Implementation

1. Both the 3.5 in. PC pavements with 8 in. and 12 in. crushed stone bases have under carefully controlled laboratory conditions, performed very good.

20 Problems

No significant problems.

21 Report Prepared by

RDB

Signature

Richard D. Barksdale

Name

Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

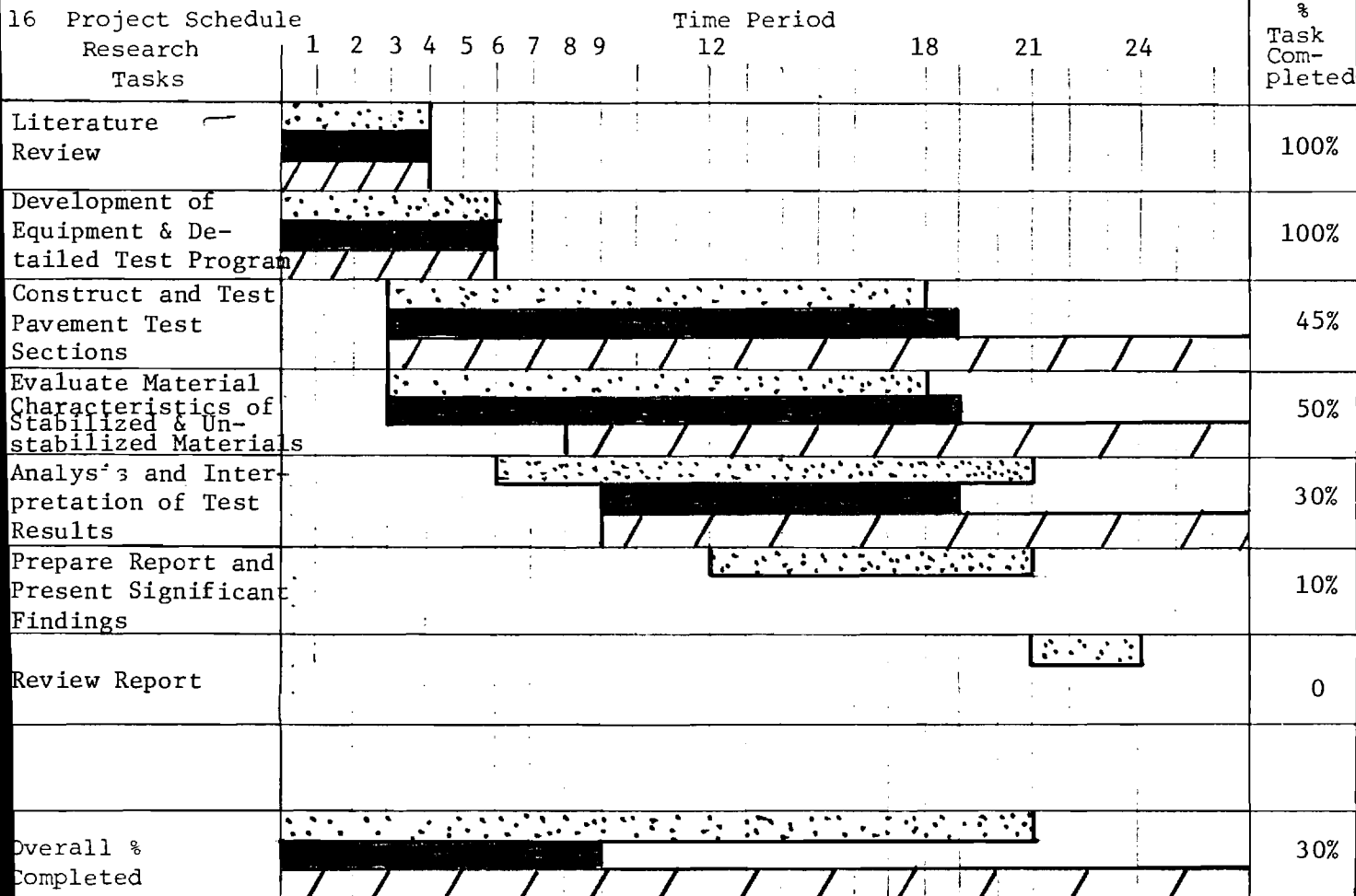
Date of Report
 April 30, 1980

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>7</u> From <u>1/01/80</u> To <u>3/31/80</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q. L. Robnett, Co-Director Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date July 16, 1980	8 % Time Expended 80	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended		
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$95,151	%	14 Current Fiscal Year \$40,849
				15 Report Quarter \$14,868

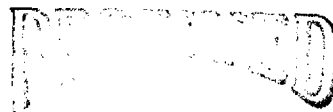


Approved Schedule
 Work Completed Schedule
 Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. The second test section (full-depth asphalt concrete) was constructed. One end was tested to failure in rutting at 10,000 repetitions. Extraction tests showed the asphalt content to be 5.9% which explains the early failure. The full-depth asphalt section was removed and reconstruction begun on a similar one hopefully with an acceptable asphalt content. Operation of both loading systems simultaneously was checked out and found to be working nicely.

Task 4. The resilient and plastic properties of the crushed stone were evaluated.



NOV 2 1980

OFFICE OF CONTRACT
ADMINISTRATION

18 Work Planned for Next Quarter

Task 3. Complete reconstruction of the full-depth asphalt pavement; test third section to failure.

Task 4. Finish evaluation of the crushed stone base. Begin fatigue testing of the asphalt concrete surfacing mix.

Task 5. Begin evaluation of the full-depth asphalt concrete sections.

19 Significant Technical Information, Recommendations, Implementation

A 0.75 in. rut depth was attained in a full-depth section after 10,000 repetitions. In comparison, 2 to 3 million repetitions were required in conventional sections constructed with an asphalt concrete surfacing having an asphalt content of 4.7%.

20 Problems

Bad asphalt (5.9% A.C.) was obtained from the plant which made it necessary to replace the full-depth asphalt concrete sections.

21 Report Prepared by

Signature

Dr. R. D. Barksdale

Name

Project Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report
 July 26, 1980




1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>8</u> From <u>4/01/80</u> To <u>6/30/80</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date June 30, 1981	8 % Time Expended 67%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended				
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$111,726 (\$ 11,209)	% 79 (36)	14 Current Fiscal Year \$57,424	% 36	15 Report Quarter \$16,575

16 Project Schedule Research Tasks	Time Period												% Task Com- pleted
	2	4	6	8	10	12	16	20	24	28	32	36	
Literature Review	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	100%
Development of Equipment & De- tailed Test Program	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	Approved	100%
Construct and Test Pavement Test Sections	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	50%
Evaluate Material Characteristics of Stabilized & Un- stabilized Materials	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	60%
Analysis and Inter- pretation of Test Results	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	40%
Prepare Report & Present Significant Findings	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	15%
Review Report	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	0
Overall % Completed	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	Projected	45%

 Approved Schedule
  Work Completed Schedule
  Projected Completion Schedule

17 Progress This Quarter (By Task)

- Task 3. The third test section (full-depth asphalt concrete) was constructed and is currently being tested. Both testing systems are being used to load each end to failure simultaneously. Projected failure by rutting appears to be occurring at approximately 8×10^5 repetitions for the 6.5 in. thick full-depth asphalt concrete section and 5×10^5 repetitions for the 9.5 in. full-depth section.
- Task 4. The elastic and plastic properties of the crushed stone base were evaluated. Fatigue tests were begun on the Modified B asphalt concrete mix.

18 Work Planned for Next Quarter

- Task 3. Complete testing of the current full-depth asphalt pavements; construct and begin testing the next test pavement.
- Task 4. Evaluate properties of the "optimum" crushed stone base. Continue fatigue testing of the asphalt concrete Modified B mix.
- Task 5. Evaluate the full-depth asphalt concrete sections which have been tested to failure.

19 Significant Technical Information, Recommendations, Implementation

20 Problems

1 Report Prepared by RAK by AN

Signature

Dr. R. D. Barksdale

Name

Project Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

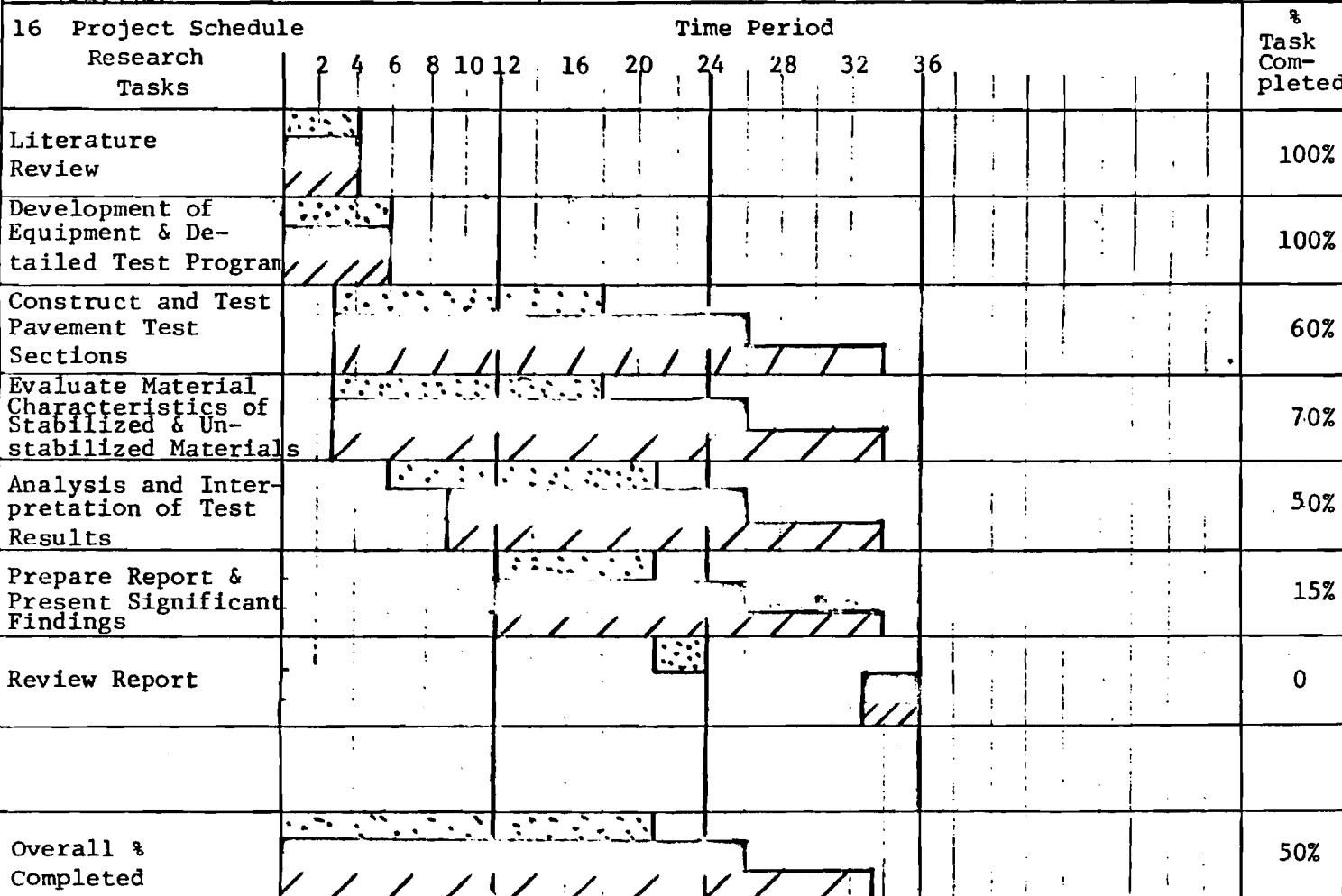
Date of Report
 July 26, 1980

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>9</u> From <u>7/01/80</u> To <u>9/30/80</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date June 30, 1981	8 % Time Expended 75%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized				Funds Expended			
11 Total GDOT/FHWA \$140,591 (Ga. Tech \$31,441)	12 Current Fiscal Year \$75,000	13 Total to Date \$115,582 (\$ 11,209)	% 82 (36)	14 Current Fiscal Year \$61,280	%	15 Report Quarter \$ 3,856	



Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

17 Progress This Quarter (By Task)

- Task 3. The third test section (full-depth asphalt concrete) was tested to failure (rutting controlled). Both testing systems are being used to load each end to failure simultaneously. Failure by rutting occurred at approximately 1.4×10^6 repetitions for the 6.5 in. thick full-depth asphalt concrete section and 7×10^5 repetitions for the 9.5 in. full-depth section. Construction was begun on the fourth test section which should be completed near the end of October.
- Task 4. Elastic and plastic properties of the standard crushed stone base were evaluated. Fatigue tests were performed on the Modified B asphalt concrete mix.
- Task 5. The full-depth sections tested to failure were evaluated.

18 Work Planned for Next Quarter

- Task 3. Complete construction of the fourth test section which consists of one section with 3.5 in. AC and 8 in. crushed stone and one section with 7 in. of AC. Test this section to failure.
- Task 4. Evaluate elastic and rutting properties of the "optimum" crushed stone base.

19 Significant Technical Information, Recommendations, Implementation

20 Problems

The asphalt plant has not been running Modified B base on a regular basis. This has slowed considerably completion of the test sections.

21 Report Prepared by / /

Signature

Dr. R. D. Barksdale

Name

Project Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report

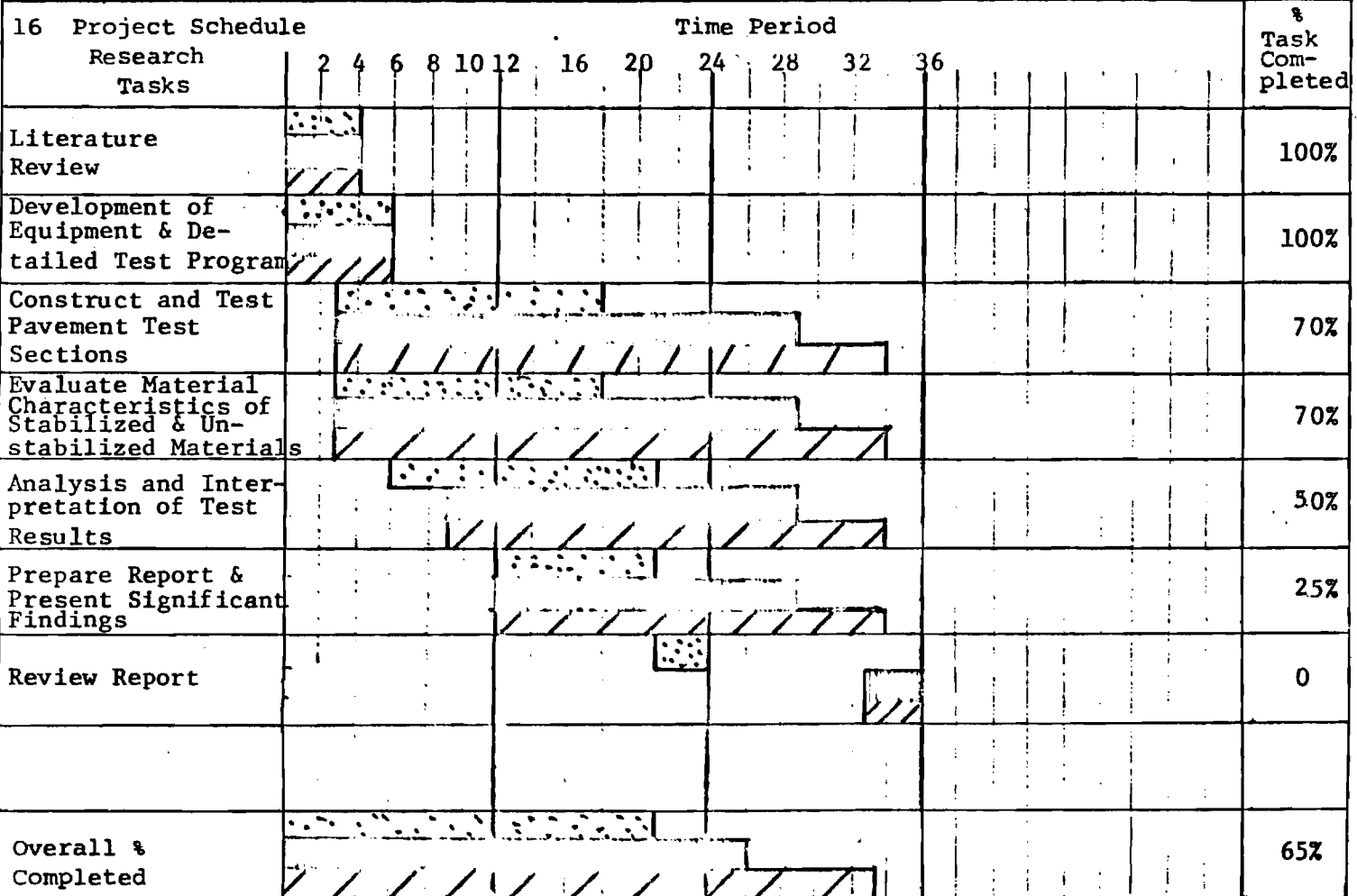
Jan. 25, 1981

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>10</u> From <u>9/01/80</u> To <u>12/31/80</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date June 30, 1981	8 % Time Expended 80%	9 Schedule Status <input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On	10 Sufficiency of Funds Sufficient <input checked="" type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended			
11 Total	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	15 Report Quarter
GDOT/FHWA \$140,591 (Ga. Tech \$31,441)		\$119,293 (21,175)	85 (36)	\$65,023	\$3,743.00



Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. The fourth test section has been constructed. One end is 3.5 in. A.C. and 8.0 in. crushed stone and one end is 8.0 in. of A.C. Testing of this section began January 5, 1981.

Task 4. Marshall properties of mixes being laid were determined.

Task 5. Analysis of results continued.

18 Work Planned for Next Quarter

Task 3. Complete testing of the current full-depth asphalt and crushed stone sections; construct and begin testing the next test pavement (hopefully).

Task 4. Evaluate properties of the "optimum" crushed stone base. Continue fatigue testing of the asphalt concrete Modified B mix.

Task 5. Evaluate the current sections being tested.

19 Significant Technical Information, Recommendations, Implementation

A presentation was made on January 22, 1981 to the Pavement Design Committee concerning the research project.

20 Problems

21 Report Prepared by

Dr. R. D. Barksdale

Project Director

Signature

Name

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report

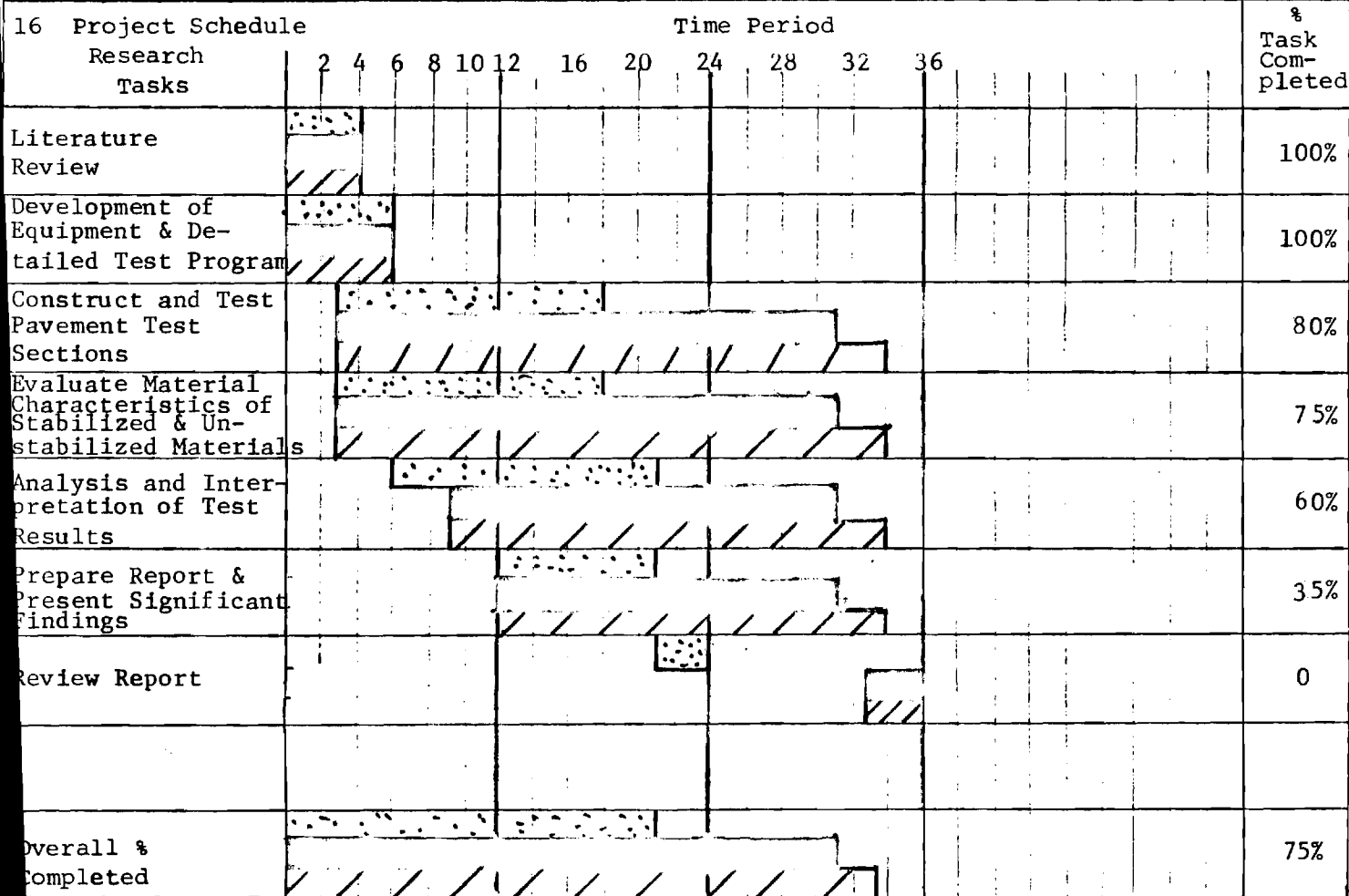
May 7, 1981




<p>1 Project No. State/Agency</p> <p>7603 E20-639</p>	<p>2 Project Title</p> <p>A Study of Factors Affecting Crushed Stone Base Course Performance</p>	<p>3 Quarterly Report No. <u>11</u></p> <p>From <u>1/1/81</u></p> <p>To <u>3/31/81</u></p>
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<p>4 Research Agency</p> <p>School of Civil Engineering Georgia Institute of Technology</p>	<p>5 Project Director(s)</p> <p>Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director</p>
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<p>6 Starting Date</p> <p>July 17, 1978</p>	<p>7 Completion Date</p> <p>June 30, 1981</p>	<p>8 % Time Expended</p> <p>90%</p>	<p>9 Schedule Status</p> <p><input type="checkbox"/> Ahead <input checked="" type="checkbox"/> Behind <input type="checkbox"/> On</p>	<p>10 Sufficiency of Funds</p> <p>Sufficient <input checked="" type="checkbox"/> Insufficient</p>
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Funds Authorized		Funds Expended			
<p>11 Total</p> <p>GDOT/FHWA \$140,591 (Ga. Tech \$31,441)</p>	<p>12 Current Fiscal Year</p>	<p>13 Total to Date</p> <p>\$131,418 (\$30,151)</p>	<p>%</p> <p>93 (96)</p>	<p>14 Current Fiscal Year</p> <p>\$77,148</p>	<p>15 Report Quarter</p> <p>\$12,125 (\$12,799.41)</p>



 Approved Schedule
  Work Completed Schedule
  Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. The fourth test section was tested to failure. One end is 3.5 in. A.C. and 8.0 in. crushed stone and one is 8.0 in. of A.C. Testing of this Section began January 5, 1981. The crushed stone did significantly better than the A.C. section with both sections failing in rutting. Construction of the fifth test section was begun.

Task 4. Marshall properties of mixes being laid were determined.

Task 5. Analysis of results continued.

18 Work Planned for Next Quarter

Task 3. Complete construction of the current crushed stone sections; begin testing the next test pavement.

Task 4. Evaluate properties of the "optimum" crushed stone base. Continue fatigue testing of the asphalt concrete Modified B mix.

Task 5. Evaluate the current sections being tested.

19 Significant Technical Information, Recommendations, Implementation

A presentation was made on January 22, 1981 to the Pavement Design Committee concerning the research project.

20 Problems

21 Report Prepared by Dr. R. D. Barksdale

signature

Dr. R. D. Barksdale

Name

Project Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

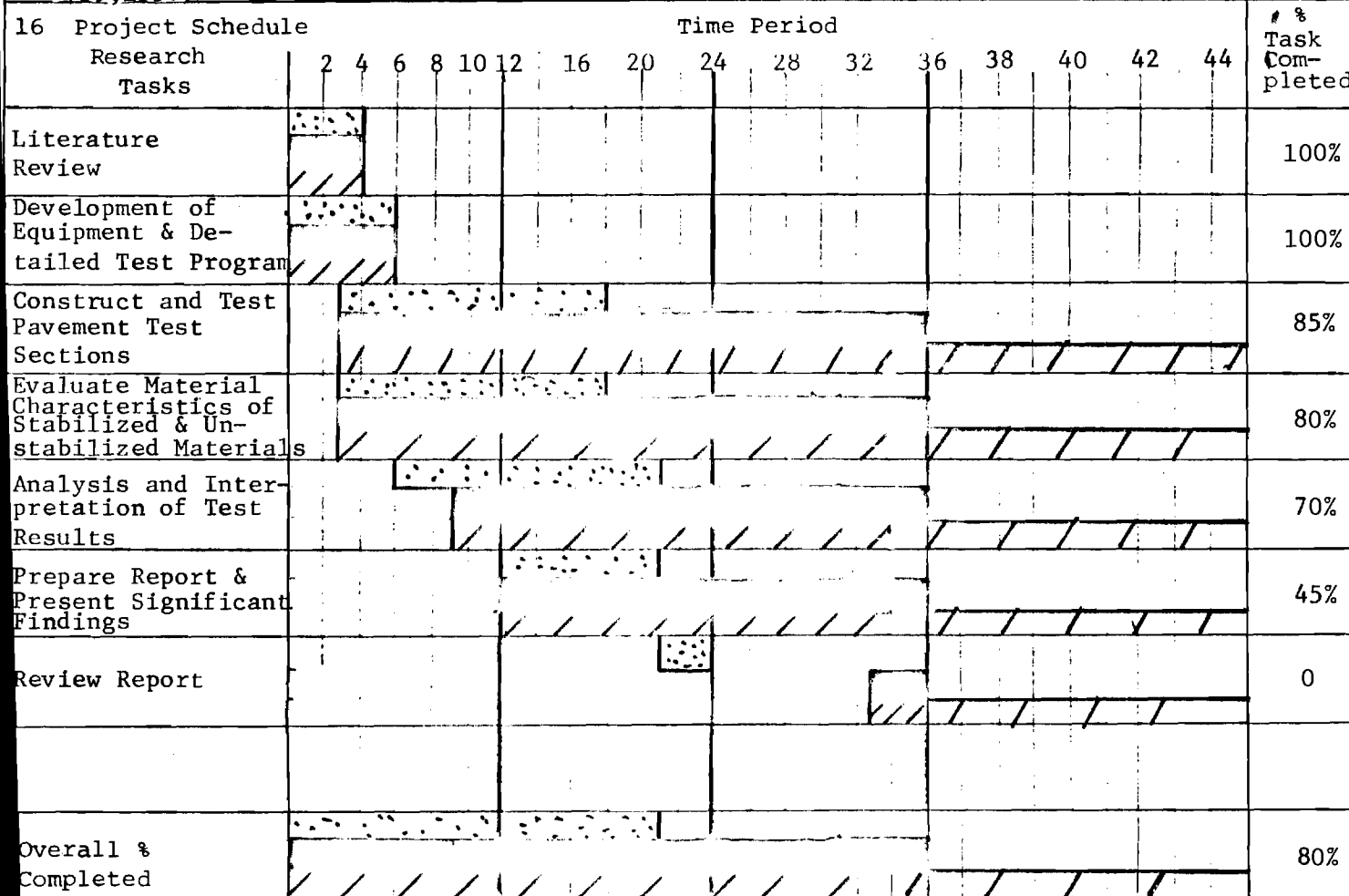
Date of Report
 July 31, 1981

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. <u>12</u> From <u>4/1/81</u> To <u>6/30/81</u>
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date April 30, 1982	8 % Time Expended 80%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended				
11 Total	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter
GDOT/FHWA \$168,588. (Ga. Tech \$39,155.)		\$136,262. (\$31,150.)	81 (80)	\$81,992.	49	\$4844 (\$1290)



Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

17 Progress This Quarter (By Task)

- Task 3. Construction of the fifth test section was completed. The south end consists of 3.5 in. of AC overlying an 8 in. crushed stone base having a fine gradation. The north end consists of 3.5 in. AC overlying an 8 in. crushed stone base having a coarse "optimum" gradation. The asphalt was obtained from the Forest Park Plant of Ashland-Warren rather than Lithia Springs which was used for the other sections.
- Task 4. Marshall properties of mixes being laid were determined; Rutting and Beam Fatigue asphalt test samples were prepared.
- Task 5. Analysis of results continued.

18 Work Planned for Next Quarter

- Task 3. Test to failure the current crushed stone sections; begin construction of the final sections.
- Task 4. Evaluate properties of the "optimum" crushed stone base. Continue fatigue testing of the asphalt concrete Modified B mix.
- Task 5. Evaluate the current sections being tested.

19 Significant Technical Information, Recommendations, Implementation

After about 100,000 repetitions both sections have rutted approximately 8/32 in.

20 Problems

No significant problems to report

21 Report Prepared by / /

Dr. R.D. Barksdale
Signature

Dr. R.D. Barksdale

Name

Project Director

Title

E 20 639

GEORGIA INSTITUTE OF TECHNOLOGY
ATLANTA, GEORGIA 30332

SCHOOL OF
CIVIL ENGINEERING

TELEPHONE:
(404) 894- 2284

October 27, 1981

Mr. Wouter Gulden
Research Engineer
Georgia Dept. of Transportation
Office of Materials & Research
15 Kennedy Drive
Forest Park, Georgia 30050

Dear Wouter:

Enclosed are ten copies of Quarterly Progress Report No. 13 for Georgia DOT Research Project 7603, "A Study of Factors Affecting Crushed Stone Base Course Performance". If you have any questions concerning this quarterly progress report or the progress of the project in general, please contact me.

Sincerely,

Richard D. Barksdale, P.E.
Professor

RDB:vc

Enclosures (10)

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

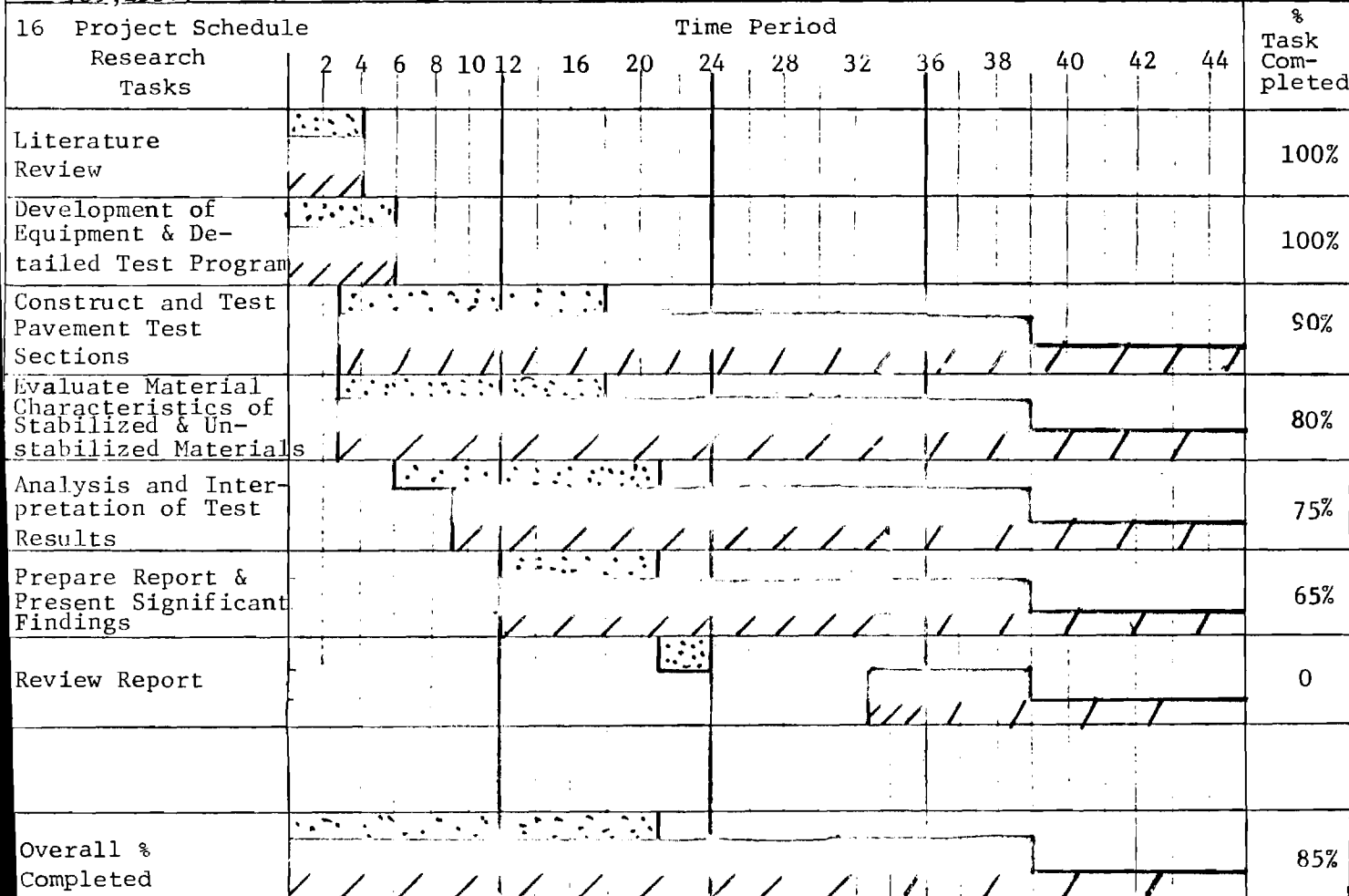
Date of Report
 Oct. 27, 1981

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. 13 From 7/1/81 To 9/30/81
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date April 30, 1982	8 % Time Expended 86%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended				
11 Total GDOT/FHWA \$168,588. (Ga. Tech \$39,155.)	12 Current Fiscal Year	13 Total to Date \$145,292 (\$31,150.)	% 86 (80)	14 Current Fiscal Year \$91,021	% 54	15 Report Quarter \$9,029



[Dotted] Approved Schedule [Hatched] Work Completed Schedule [Hatched] Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. Testing of the fifth test section was completed. Construction of the sixth test section was almost completed. The north end consists of 3.5 in. of AC overlying an 8 in. crushed stone base having a standard gradation. The subgrade is stabilized with 7% cement to give a strength of about 160 psi. The south end consists of 3.5 in. AC overlying an 8 in. crushed stone base having a standard gradation. A six inch cement stabilized crushed stone underlies the granular material.

Task 4. Fatigue testing was continued.

Task 5. Analysis of results continued. Write-up of the literature section was begun.

18 Work Planned for Next Quarter

Task 3. Complete and test to failure (hopefully) the current composite sections.

Task 4. Evaluate triaxial elastic and plastic properties of the "optimum" crushed stone base. Continue fatigue testing of the asphalt concrete Modified B mix.

Task 5. Evaluate the current sections being tested.

19 Significant Technical Information, Recommendations, Implementation

No significant information to report at this time.

20 Problems

No significant problems to report.

21 Report Prepared by *R. D. Barksdale*

Signature

R. D. Barksdale

Name

Project Director

Title

RESEARCH QUARTERLY PROGRESS REPORT
GEORGIA DEPARTMENT OF TRANSPORTATION

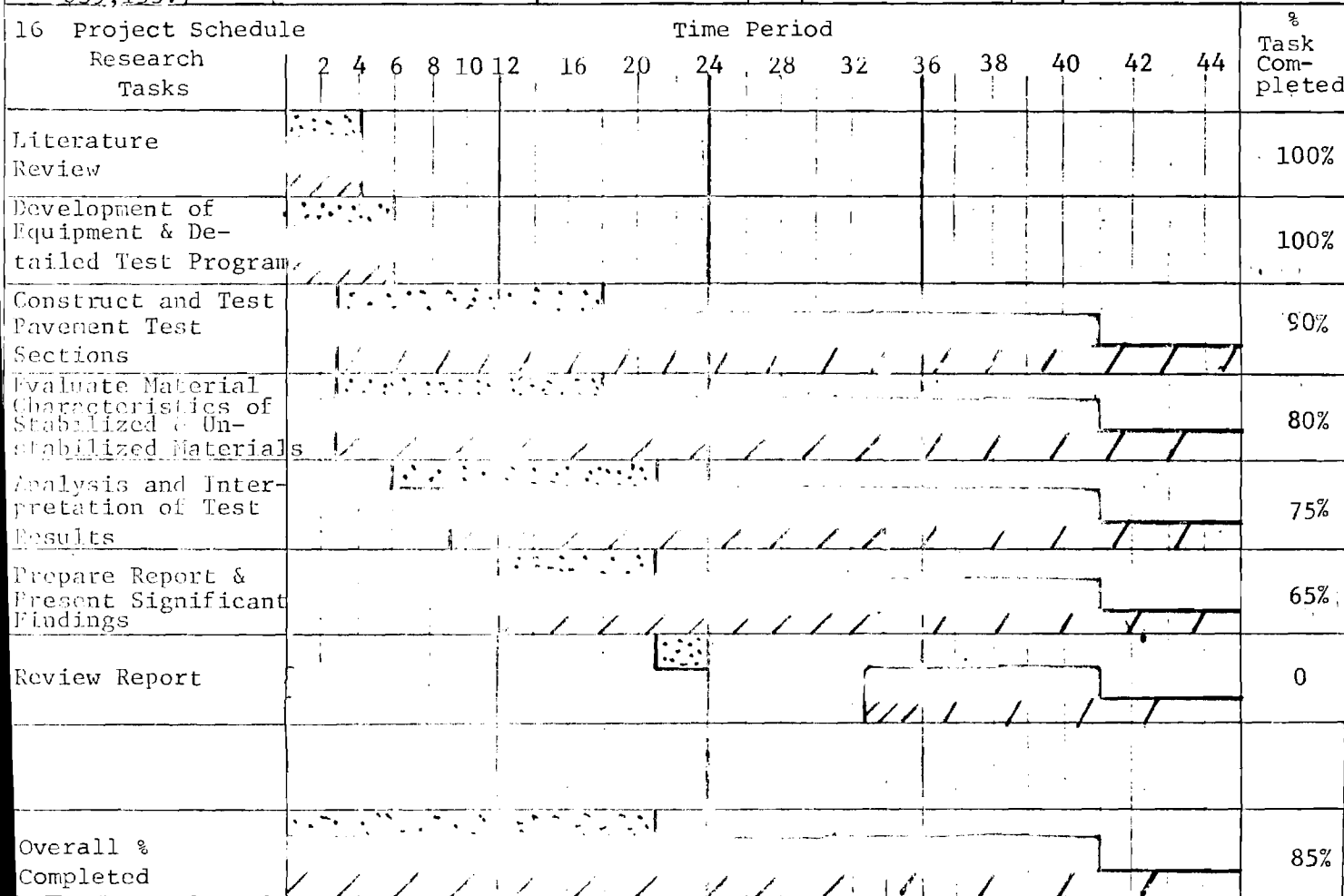
Date of Report
 January 26, 1982

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. 14 From Oct. 1, 1981 To Dec. 31, 1981
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date April 30, 1982	8 % Time Expended 90%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended		
11 Total GDOT/FHWA \$168,588. (Ga. Tech \$39,155.)	12 Current Fiscal Year	13 Total to Date \$159,013 ((\$39,155))	%	14 Current Fiscal Year \$22,656 54
		15 Report Quarter \$9,034 (\$5,196)		



Approved Schedule

 Work Completed Schedule

 Projected Completion Schedule

17 Progress This Quarter (By Task)

Task 3. Construction of Test Sections 11 and 12 was completed; testing was begun and as of 1/26/82 approximately 1,500,000 load repetitions has been applied to each section. Sections 11 and 12 consist of a 3.5 in. AC surfacing, 8 in. crushed stone base, and 6 in. cement stabilized subbase or subgrade. Little rutting has occurred and these sections are performing quite good; they will probably take more than 3×10^6 repetitions of load.

Task 4. Fatigue testing was continued; rutting tests were begun.

Task 5. Data is being tabulated. The data for Sections 9 and 10 was reduced; literature write-up and test description was essentially finished.

18 Work Planned for Next Quarter

Task 3: Complete Test Sections 11 and 12.

Task 4: Concentrate on rutting tests of AC; complete granular base material tests.

Task 5: Reduce data for Sections 11 and 12; write report.

19 Significant Technical Information, Recommendations, Implementation

The two inverted sections presently being tested are performing quite well.

20 Problems

No significant problems to report.

21 Report Prepared by

Dr. R. D. Barksdale

Professor/Project Director

Signature

Name

Title

RESEARCH QUARTERLY PROGRESS REPORT
 GEORGIA DEPARTMENT OF TRANSPORTATION

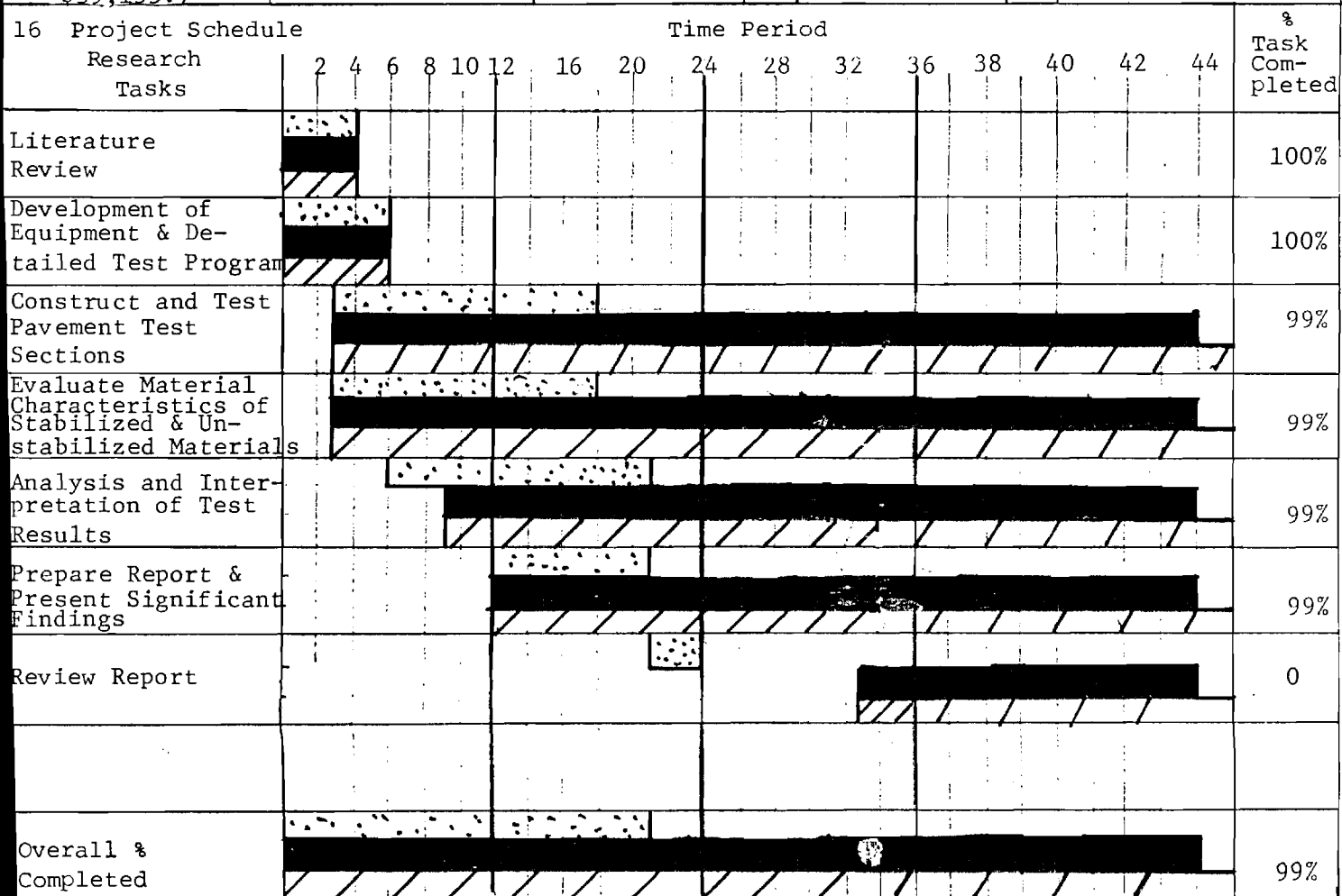
Date of Report
 May 15, 1982

1 Project No. State/Agency 7603 E20-639	2 Project Title A Study of Factors Affecting Crushed Stone Base Course Performance	3 Quarterly Report No. 15 From 1/1/82 To 3/31/82
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4 Research Agency School of Civil Engineering Georgia Institute of Technology	5 Project Director(s) Dr. Richard D. Barksdale, Director Dr. Q.L. Robnett, Co-Director Mr. Wouter Gulden, Co-Director
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6 Starting Date July 17, 1978	7 Completion Date October 31, 1982	8 % Time Expended 85%	9 Schedule Status <input type="checkbox"/> Ahead <input type="checkbox"/> Behind <input checked="" type="checkbox"/> On	10 Sufficiency of Funds <input checked="" type="checkbox"/> Sufficient <input type="checkbox"/> Insufficient
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Funds Authorized		Funds Expended			
11 Total GDOT/FHWA \$168,588. (Ga. Tech \$39,155.)	12 Current Fiscal Year	13 Total to Date \$166,457 (\$39,155)	% 99 (100)	14 Current Fiscal Year \$34,786	15 Report Quarter \$12,130



Approved Schedule Work Completed Schedule Projected Completion Schedule

17 Progress This Month (By Task)

Tasks 3, 4, 5 and 6 were essentially completed during this quarter.

18 Work Planned for Next Month

Submit draft copy of report in May.

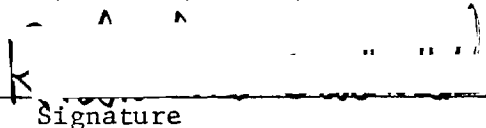
19 Significant Technical Information, Recommendations, Implementation.

Refer to draft copy of report.

20 Problems

No significant problems to report.

21 Report Prepared by



Dr. R. D. Barksdale

Project Director

Signature

Name

Title

E 20-627

FOR REVIEW ONLY

Report

EFFECTING CRUSHED STONE
PERFORMANCE

Prepared for

Department of Transportation
State of Georgia

Cooperation With

Department of Transportation
Highway Administration

September, 1982



INSTITUTE OF TECHNOLOGY
UNIVERSITY SYSTEM OF GEORGIA
SCHOOL OF CIVIL ENGINEERING
ATLANTA, GEORGIA 30332



FOR REVIEW ONLY

Contract Research
GDOT Research Project No. 7305
Georgia Tech Research Project No. E20-639

DRAFT REPORT

A STUDY OF FACTORS AFFECTING CRUSHED STONE BASE
COURSE PERFORMANCE

by

Richard D. Barksdale
Professor of Civil Engineering

and

H. A. Todres
Instructor

School of Civil Engineering
Georgia Institute of Technology
Atlanta, Georgia 30332

Prepared for

Department of Transportation
State of Georgia

In Cooperation With

U.S. Department of Transportation
Federal Highway Administration

September, 1982

"The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the State of Georgia or the Federal Highway Administration. This report does not constitute a standard, specification or regulation".

EXECUTIVE SUMMARY

The purpose of the full-scale pavement tests was to study the feasibility of replacing full-depth and deep strength asphalt concrete pavements with flexible pavements utilizing relatively thick layers of crushed stone base. A total of twelve test sections were constructed and tested to failure (one of these sections was only tested close to failure) by applying a repeated, dynamic loading having a magnitude of 6,500 lbs. (30 kN). To simulate traffic wander and prevent a localized punching type failure, the stationary circular loading was applied in a primary load position and six supplementary positions located symmetrically around the edge of the primary position. In all sections an asphalt concrete B or B-modified binder was used for the entire asphalt concrete thickness. The properties of all materials were evaluated using dynamic testing procedures.

The pavement sections tested were as follows: (1) five crushed stone base sections, (2) five full-depth asphalt concrete sections, and (3) two inverted sections. All the crushed stone base sections and the inverted sections had an asphalt concrete thickness of 3.5 in. (89 mm). The crushed stone bases had either an 8 or 12 in. (203-508 mm) thickness. Three crushed stone base gradations were studied approximately defining the limits of crushed stone bases produced in the Atlanta area. The full-depth sections consisted of either 6.5, 7.0, or 9.0 in. (165, 178, 229 mm) of asphalt concrete binder placed directly on the subgrade. The two composite sections consisted of 8 in. (203 mm) of unstabilized crushed stone sandwiched between 3.5 in. (89 mm) of asphalt concrete above and 6 in. (152 mm) of cement stabilized material below.

The results of this study showed that it was feasible to test in the laboratory full-scale sections to failure by subjecting them to large numbers of repeated loadings. By carefully controlling the quality of construction and environmental factors, direct comparisons of performance were obtained between different structural sections.

Test Sections

All tests were performed in an environmentally controlled room at a temperature varying from 78 to 80°F (25.5 - 26.7°C). The unstabilized crushed stone base in all sections was constructed by blending three different sizes of crushed stone together in a pugmill to minimize

segregation. Crushed stone base sections were compacted to or slightly above 100% of AASHTO T-180 density. In the inverted sections because of the presence of a rigid working platform, a crushed stone base density of 105% of AASHTO T-180 was obtained for the same compaction effort used in the other crushed stone bases.

A uniform, silty sand subgrade 50 in. (1270 mm) thick was used in all tests. The subgrade was compacted to an average of 94% ($\sigma = 1.5\%$) of AASHTO T-99 density at a moisture content of 20.4% ($\sigma = 0.5\%$) which was 2% above the optimum value. This material had an average Georgia DOT volume change of 38.5% and classified as a III-B embankment material.

Two hybrid air-over-oil cyclic loading systems were designed and constructed to test to failure the pavement sections. The two testing systems were used to apply over 17 million load repetitions during the study. These testing systems were found to be quite reliable and relatively easy to maintain considering the large number of repetitions applied.

Instrumentation

The test sections were fully instrumented to give important information concerning the amount and distribution of permanent deformation within each layer, resilient displacement, resilient strain and vertical stress on the subgrade. Bison-type strain sensors were used to measure permanent deformations and resilient strains. Vertical stresses were measured using small, diaphragm type pressure cells. Resilient surface displacements were measured with LVDT's. The results from this instrumentation gave valuable information concerning the mechanisms of pavement performance.

Summary of Findings

The findings were developed using the results of 12 test sections carefully integrated together with the findings from field studies presented in Chapters II and VI. Field studies of particular applicability to this investigation on crushed stone bases were conducted in North Carolina and Virginia on Piedmont soils and at Lake Wales, Florida on a sand subgrade. A detailed discussion of the findings is given in Chapter V and VI. Significant conclusions reached from this study are summarized as follows:

1. The 8 and 12 in. (203-305 mm) crushed stone base pavements withstood between 0.64 and 2.9 million wheel loadings, respectively. The crushed stone base was covered with only a 3.5 in. (89 mm) thick surfacing of asphalt concrete binder. These results clearly show that crushed stone base sections having thin asphalt concrete surfacings can successfully withstand large numbers of heavy loadings. These findings are supported by the Lake Wales, Florida test pavement results. At Lake Wales a 1.5 in. (38 mm) asphalt concrete surfacing underlain by varying thicknesses of limerock base withstood 2.6 million load repetitions.

2. The test results for the conditions of this study indicate the crushed stone base was more resistant to rutting than the asphalt concrete at a similar depth in the pavement structure. Admittedly, the asphalt concrete was probably marginally acceptable. Nevertheless other studies also indicate that under favorable conditions crushed stone can be less susceptible to rutting than asphalt concrete.
3. Good crushed stone base course performance is attributed to (1) a uniformly high level of compaction (100% of T-180 density), (2) no segregation of the base, (3) a thin asphalt concrete surfacing, and (4) only 4 to 5% of the base passed the No. 200 sieve.
4. For the variation studied, crushed stone gradation was found not to significantly influence performance. Of significance, however, was the fact that all three gradations had only 4 to 5% passing the No. 200 sieve and a 1 to 2 in. (25 to 51 mm) top maximum size crushed aggregate.
5. Performance of the tests appeared to be controlled more by the quality of the asphalt concrete surfacing than the crushed stone base. In this laboratory study the crushed stone base quality could be closely controlled whereas the quality of the in-place asphalt concrete was considerably more variable.
6. Test results indicate an increase in asphalt content from about 5 to 5.25% approximately doubled the rutting in a layer of asphalt concrete.
7. For the reasonably firm sand subgrade investigated, the minimum required structural thickness above the subgrade to limit rutting to about 0.07 to 0.1 in. (1.9-2.5 mm) is about 13 to 15 in. (330-381 mm) for 2 million 6.5 kip (29 kN) loadings. In general however, a safety factor with respect to rutting of 1.25 is recommended as shown in Fig. 117. The standard penetration resistance of the subgrade was about 7 blows per foot (23 blows/meter).
8. In a well constructed section about one-half of the total permanent deformation occurring in a uniform subgrade occurred in the upper 12 in. (305 mm). Further, as the quality of the material in the base and surfacing increased, the relative amount of rutting in the subgrade increased and was as high as 50 to 60% of the total.
9. The crushed stone base sections were just as effective inch for inch as the full-depth asphalt concrete sections in reducing subgrade rutting.
10. For the conditions of the tests, 1.6 in. (46 mm) of crushed stone replaced 1.0 in. (25 mm) of asphalt concrete base. Refer to Chapter VI for a discussion of these results.

11. In the crushed stone base sections about 70% of the total permanent strain occurred in the top of the crushed stone. This finding suggests that improved performance may be obtained by taking special precautions in constructing the upper portion of a deep granular base. More relaxed requirements in the lower portion of the base are not, however, recommended.
12. For pavements having structural numbers less than about 4.5, the AASHTO Interim Design Guide underpredicts the life of both crushed stone base and deep strength sections by factors up to 10 or more (Fig. 115).

Recommendations

The following design recommendations were developed from this study. Refer to Chapter VI for a detailed discussion of these recommendations.

1. Engineered crushed stone base pavements can be constructed that perform as well as deep strength asphalt concrete sections. In the future crushed stone base sections should be considered as a viable alternative to deep strength construction.
2. If pavements are constructed having relatively thin asphalt concrete surfacings and deep crushed stone bases, their use should be gradually phased in. During construction of the early deep granular base pavements a concentrated effort should be made to develop better techniques to minimize segregation and obtain high, uniform densities. This information should then be incorporated into subsequent projects.
3. In design 1 in. (25 mm) of asphalt concrete base should be replaced by 2.0 in. (51 mm) of engineered crushed stone base for pavement thicknesses less than 16 in. (406 mm). For an asphalt concrete AASHTO Base course coefficient of 0.30, the corresponding crushed stone base coefficient is 0.18 using the proposed substitution ratio of 1.7.
4. At the present time, a maximum crushed stone base thickness is recommended of 16 to 18 in. (406 - 457 mm). Layered elastic theory adjusted to approximately agree with observed test section response indicates greater depths of crushed stone give little additional benefit. Also, the North Carolina sections which performed well had maximum thicknesses of crushed stone of about this magnitude.
5. An engineered crushed stone base should be covered by the minimum amount of asphalt concrete necessary to insure stability and provide the required structural number. A minimum thickness of asphalt concrete insures maximum flexibility of both the section and the asphalt concrete. For AASHTO structural numbers varying from 4 to 6 asphalt

concrete surfacing thicknesses are recommended varying from 4 to 8 in. (102-203 mm). Tentative recommendations are given in Table 20. Both the Georgia Tech tests and the North Carolina studies indicate use of deep crushed stone bases and a substitution ratio of 1.7 is justified. Of course to insure satisfactory performance the other recommendations given below should also be followed.

6. A structural section sufficiently thick must be used to protect the subgrade. In general a crushed stone base structural section designed by the AASHTO Interim Guide will provide adequate protection. Table 21 and Fig. 117 provide a guide for required thickness of pavement to protect the subgrade.
7. Well constructed crushed stone base or deep strength sections having AASHTO structural numbers less than about 4.5 can withstand considerably greater numbers of load repetitions than predicted by the AASHTO Interim Design Guide. A conservative correction to the design guide results is given by equation (1) and in Fig. 116. Therefore, lighter, more flexible pavement sections perform extremely well. This finding suggests the alternative approach of designing a relatively light section to withstand the design traffic using equation (1) and Figs. 115 and 116 as a guide. If the pavement does not last the full design life an overlay can always be placed. Although somewhat similar to stage construction, this approach differs significantly in basic design philosophy: in the proposed approach the section is designed to withstand the full traffic loading using an admittedly light, flexible section, while in stage construction the section is purposely designed to fail. This approach should be tried on a limited basis on carefully selected projects such as temporary roads. Certainly the outstanding performance of the light limerock sections at Lake Wales, Florida supports this approach. Using a relatively light, flexible section certainly has considerable economic advantages over a heavy, less flexible one.
8. Proper base construction is considered to be the single most important factor in achieving a high level of performance from a crushed stone base. The construction operation should be conducted such that segregation is minimized and a uniform density is achieved equal to or greater than 100% of AASHTO T-180. Pugmilling of the stone is recommended; stockpiling of the aggregate at the site should not be permitted. Further the recommendation is made that the base should not be left exposed for more than 5 working days.
9. Excellent performance was obtained from the base course which had the relatively narrow range in gradations shown in Fig. 119. This stone was well-graded and had a 1.0 to 2 in. (25-51 mm) top size and 4 to 5% fines. From these results a base course

stone should have the minimum amount of fines practical with 8 to 9% being the upper allowable limit. The top size should be at least 1 to 2 in. (25 - 51 mm).

More research is needed concerning the most effective maximum top size. Some research indicates a larger top size gives better performance. In this study the finer gradation base section had a longer fatigue life than the coarse gradation section. Rutting however was less in the coarse gradation base section.

10. Special attention should be given to the field inspection program for a crushed stone base. Careful, thorough inspection of the base course on a full-time basis is required by a senior inspector. Further, a bonus/penalty system should be implemented to insure a uniform, high density of the crushed stone base. It should be remembered that the weaker 10 to 15% of the area will determine the performance of the pavement.
11. The recommendation is also made that the desirability of proof-rolling the base with a 50 ton pneumatic-tired roller be studied in the field. Proof-rolling is common practice in the construction of military airfields. Proof-rolling would help to detect localized weak areas that could easily be missed by density testing.
12. All weak subgrades should be proof-rolled by a loaded dump truck (a loaded pan would be an alternative). All weak areas should, if possible, be undercut and replaced with compacted, high quality structural fill or crushed stone. Further, the inspector should carefully watch for locally weak spots by observing loaded pans and other heavy equipment during subgrade construction in other areas.
13. If weak subgrades are present that cannot be undercut, consideration should be given to using an inverted section. If an inverted section is selected a high level of stabilization should be used to insure adequate subgrade protection. Refer to Chapter VI for a detailed discussion of inverted sections. A base course coefficient of 0.18 to 0.20 is recommended for the stabilized layer if overlain by a thin crushed stone base.

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CHAPTER I INTRODUCTION

In the 1960's and throughout the early 1970's deep lift and full-depth asphalt concrete pavements gained wide popularity throughout the United States. This general trend started as the result of full-depth asphalt concrete pavements showing good performance at the AASHTO Road Test. However, because of the rapidly rising cost of all petroleum products, full-depth and deep lift asphalt concrete construction is now rapidly becoming prohibitive in cost. Future pavement designs must therefore utilize alternate structural sections which can both perform satisfactorily and are less expensive than full-depth designs. Unstabilized crushed stone has been used for many years for base and subbase construction in both Georgia and also throughout the United States. During this time a considerable amount of general experience has been gained concerning the performance of granular bases. Further, two comprehensive laboratory studies using repeated load testing techniques have been performed for the Georgia Department of Transportation on both unstabilized and stabilized base materials^(1,2). Design base coefficients were developed in these studies by extending the laboratory test results to field conditions using layered theory. Full-scale test sections have not, however, been used to validate and extend the laboratory and theoretical results.

The rising costs of petroleum products dictates the use of more low-energy intensive materials such as unstabilized crushed stone compared with full-depth or deep strength asphalt concrete. An important need therefore exists for formally studying the performance of granular bases in full-scale test sections taking advantage of recent advances in materials technology and instrumentation. Use of a crushed stone base offers an attractive alternative to full-depth and deep lift asphalt concrete construction. In the mid-1960's a number of base course studies were conducted, primarily to evaluate deep lift or full-depth asphalt construction. As a result, little information was gained concerning how variables affected the performance of crushed stone bases.

One approach for studying the relative influence of selected variables on base course performance is by the use of full-scale laboratory pavement sections. Using this approach a relatively large number of sections can be studied under closely controlled conditions at a minimum cost. A primary concern is the *relative* influence of base course variables. By carefully designing and instrumenting test sections, the influence can be minimized of factors such as variable subgrade support and materials variability, which is always an important problem in full-scale test pavements.

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1. Barksdale, R. D., REPEATED LOAD TEST EVALUATION OF BASE COURSE MATERIALS, Georgia Institute of Technology, Georgia DOT Project 7002, Atlanta, Georgia, 1972.
 2. Barksdale, R. D., and Miller, J., DEVELOPMENT OF EQUIPMENT AND TECHNIQUES FOR EVALUATING FATIGUE AND RUTTING CHARACTERISTICS OF ASPHALT CONCRETE MIXES, Georgia Institute of Technology, Georgia DOT Project 7305, Atlanta, Georgia, 1977.

Therefore, in studying the *relative* performance of unstabilized bases use of full-scale laboratory test sections offers an excellent approach for maximizing the amount of information that can be obtained while minimizing the cost and time involved. The results of such a study provides considerable technical knowledge concerning cost-effective structural designs utilizing low-energy intensive crushed stone. As a result, design sections can be developed which minimize the use of stabilizing agents which undoubtedly will become more expensive in the future. Further, the data gained from such test studies can be readily extended through suitable theories to many other conditions not tested in the laboratory. A laboratory study of this type, however, does not *alleviate* the need for constructing at least a limited number of field test sections.

Objectives of the Study

The general objective of this research study is to investigate selected variables which influence the design and performance of unstabilized, crushed stone bases. In specific the objectives of this study are as follows:

1. Experimentally evaluate using full-scale laboratory test sections the fatigue and rutting behavior of selected pavement sections constructed with granular bases and relatively thin asphalt concrete surfacings.
2. Determine how the following base course variables influence pavement performance: (a) thickness, (b) gradation, and (c) aggregate size.
3. Relate performance of unstabilized crushed stone bases with that of full-depth asphalt concrete and develop design recommendations.
4. Evaluate the potential advantages of placing an unstabilized crushed stone base over a cement stabilized layer.
5. Determine the influence of subgrade saturation on performance of the base course.

Research Plan

To accomplish the objectives of this study, twelve full-scale pavement sections were tested in a special facility under closely controlled environmental conditions. In the past, most full-scale pavements constructed in the laboratory have not been tested to failure because of the time and special testing apparatus required. In such tests resilient deflection has usually been used as a criteria for comparing performance. In this study each section was repetitively loaded to failure (or close to failure) by a 6.5 kip (29 kN) uniform, circular loading. The load was systematically moved to prevent a punching failure. Loading the pavement to a large number of repetitions served the important purpose of defining failure and the associated failure mechanisms. This information was quite valuable for theoretically extending the results to other structural sections.

All tests were performed in an existing test facility 8 ft. by 12 ft. in plan and 7 ft. deep (2.4 x 3.6 x 1.5 m). Temperature was controlled at

78 to 80°F (27.6 to 28.8°C) throughout the test. In addition, a complete series of laboratory tests including fatigue and rutting were performed to characterize the materials for theoretically predicting pavement behavior.

Seven of the pavement sections were loaded to over one million repetitions, and five sections to over two million repetitions. As a result, considerably more time was required to complete the project than originally planned. Because of the large number of load repetitions applied, however, the experimental findings are considered to be more representative of field loading conditions and hence of greater overall value.

Pavements tested consisted of five conventional sections having crushed stone bases, five full-depth asphalt concrete sections, and two inverted sections. The inverted sections consisted of a crushed stone base sandwiched between a lower cement stabilized layer and an upper asphalt concrete layer. Conventional sections were tested having two thicknesses of base and three base gradations. The results of full-scale field tests and layered theory are used together with the results of this study to develop practical design recommendations including recommended AASHTO base course coefficients. A general summary of the sections tested and the results is presented in Table 1.

TABLE 1. CONSTRUCTION AND PERFORMANCE SUMMARY OF PAVEMENT SECTIONS TESTED.

Section Number	Asphalt Concrete Thickness (in.)	Crushed Stone Thickness (in.)	Repetitions to Failure	Failure Mode(1)	Comments
1	3.5	12.0	3,000,000 3,500,000	Fatigue/ Rutting	Tested to 2.4 million reps.; Failure Extrapolated
2	3.5	8.0	1,000,000	Rutting	
3	9.0	NONE	10,000	Rutting (1 in.)	Bad Asphalt: AC Content 5.9% Flow: 15.4 Stab.: 1870 lbs.; Y _D = 145.1 pcf
4	6.5	NONE	10,000	Rutting (1 in.)	
5	9.0	NONE	130,000	Rutting	Rutting Failure Primarily in AC
6	6.5	NONE	440,000	Rutting	
7	7.0	NONE	150,000	Rutting	Direct Comparison of Crushed Stone and Full- Depth
8	3.5	8.0	550,000	Rutting	
9	3.5	8.0	2,400,000	Fatigue	$\delta_p = 0.28$ in. @ 2.9 million
10	3.5	8.0	2,900,000	Fatigue	$\delta_p = 0.34$ in. @ 2.9 million
11	3.5	8.0	3,600,000	Fatigue/ Rutting	6.0 in. Soil Cement Subbase
12	3.5	8.0	4,400,000	Fatigue/ Rutting	6.0 in. Cement Stabilized Stone Subbase

NOTE: 1. A fatigue failure is defined as Class 2 cracking; a rutting failure is defined as a 0.5 in. (12 mm) rut depth.

2. Coarse Gradation Base.
3. Fine Gradation Base.

CHAPTER II LITERATURE REVIEW

Introduction

Because of the high cost of petroleum products, unstabilized crushed stone bases when properly constructed offer an attractive alternative to full-depth and deep strength asphalt concrete construction. Only a limited number of well-documented field studies are presently available comparing the performance of pavements having bituminous and crushed stone bases. Unfortunately, serious deficiencies in granular base design were present, for example, at the AASHTO and Brampton Road Tests. These comprehensive studies have been summarized elsewhere [1] and are not discussed in detail in this chapter. Common deficiencies in granular base design include (1) use of a very thin base underlain by an inferior gravel subbase, (2) the gravel subbase was often frost susceptible, and (3) the granular layers in many experimental roads were compacted to 100% of AASHTO T-99 density rather than 100% or more of T-180 density. Several of these deficiencies were present, for example, at the AASHTO, Brampton and Colorado Test Roads [1,9].

Base course materials are usually compared using either the AASHTO Interim Guide [2] base course coefficients or else are compared in terms of a base course equivalency. The base course equivalency is expressed as the thickness of unstabilized stone required to replace 1 in. (25 mm) of bituminous base. The ratio of the AASHTO base course coefficient of the asphalt base to the crushed stone is therefore equivalent to the number of inches of crushed stone that replaces 1 in. (25 mm) of asphalt concrete. Using either method for comparing bases, it should be remembered that the equivalencies vary with many factors and hence are far from being a true constant

In the present study 12 pavement sections were relatively rapidly loaded to failure in a closely controlled environment. Undoubtedly differences exist between the present study and actual field conditions including environmental effects, construction technique, material variability and quality control. As a result available field base performance data should be rationally integrated together with the results of the present study to develop realistic design recommendations. To develop a basic understanding of performance under actual environmental conditions the results of a number of field studies involving granular bases are therefore reviewed in this chapter.

Piedmont Residual Soils

McGhee [3] has summarized the findings of three experimental pavements constructed on Piedmont residual soils in Virginia. These studies have shown that stabilization of the upper 6 in. (152 mm) of a micaceous silty and silty clay Piedmont residual soil with 10% cement by volume is beneficial in improving performance and reducing elastic deflections. Forty to 60% of the elastic deformation occurring in the pavement sections on one test pavement was found to be caused by a 6 to 6.5 in. (152-165 mm) thick layer of select borrow placed over the cement treated subgrade. The select borrow had an average CBR of 33. Another study showed improved performance when the select borrow material was not placed between the cement treated subgrade and crushed stone base.

Under heavy traffic conditions 4 in. (102 mm) of a lean asphalt concrete mix was found to be approximately equivalent to 6 in. (152 mm) of untreated crushed stone. The granular base section had a PSI rating of 4.26 and Benkelman beam deflection of 0.034 in. (0.9 mm) compared with corresponding values of 4.17 and 0.019 in. (0.5 mm) for the bituminous base sections. The unstabilized crushed stone base section, however, had a crack factor⁽¹⁾ of 52 compared with 21 for the bituminous base section. The unstabilized base section was rated as having very minor cracking and good performance; the bituminous base section was rated as having excellent performance. Because of the greater amount of cracking in the granular base, the base course equivalency for the granular base was probably a little greater than the 1.5 value reported. However, the overall cost of the section having the crushed stone base was, at the time of construction in about 1971, 9% less than the bituminous base section.

Of interest is the finding that the inverted pavement section having a cement treated subgrade and unstabilized crushed stone base demonstrated good performance over a 5 year period. This section had the lowest overall cost and was judged to be the most cost effective design. The inverted design utilized a high level of cement stabilization corresponding to a cement content of about 9 to 10% by weight in the 6 in. (152 mm) stabilized layer of subgrade.

Granular Highway Bases and Subbases

General

The field performance of granular bases tend to vary considerably more than either bituminous or cement treated bases [1]. The observed variation in performance is due to an unstabilized base being considerably more affected than stabilized materials by such variables as magnitude of wheel load, material type, compaction density, frost susceptibility, grain size, gradation and angularity.

Field compaction tests on crushed stone bases described by Nichols and James [4] have shown that compaction to 100% of AASHTO T-99 density failed to give densities nearly as high as could be readily attained in the field with a conventional vibratory roller. Further, the average increase in density in going from 30 to 50 coverages of the vibratory roller caused an average increase in density of 3.4% at the 40 sites studied. Test strips such as reported to be used by Ohio might be utilized at the beginning of a project to establish minimum allowable compaction densities [4]. If test strips are used however, the minimum allowable density should not be less than 100% of AASHTO T-180 density [1].

-
1. Crack factor is obtained by assigning a factor of 15 units and 20 units for each incidence of longitudinal and alligator cracking, respectively, in a 1000 ft. (305 m) segment.

AASHTO Road Test

The results of the AASHTO Road Test [5] are relatively well-known and will only be briefly summarized. At the AASHTO Road Test most of the rutting was found to occur in the asphalt concrete surfacing (42%) and sand gravel subbase (41%) indicating the importance of using high quality materials in not only the surfacing and base, but also in the subbase. Only 9% of the rutting occurred in the base and 8% in the subgrade. The densities of all the layers tended to increase beneath the outer wheel path. Densities beneath the inner path tended to increase in all layers except the subbase where a definite tendency existed for a decrease in density. McNaughton and Rand [6] found in Maine that gravel bases may either increase or decrease in density with time.

The significant effect which magnitude of the wheel loading has on the relative performance of materials is clearly shown from the results of the AASHTO Road Test (Fig. 1). The unstabilized aggregate base-subbase sections clearly showed significantly better performance with increasing magnitude of wheel load. For a 12 kip (53 kN), single axle loading, 3 in. and 1.9 in. (76 and 48 mm) of crushed stone were required to replace 1 in. (25 mm) of bituminous and cement treated base, respectively. For a 30 kip (130 kN) single axle loading, 1.3 in. and 1.2 in., respectively were required to replace 1 in. (25 mm) of bituminous and cement treated materials. These comparisons are valid for the conditions existing at the AASHTO Road Test including: (1) 1×10^6 wheel load applications applied in only two years; (2) granular materials compacted to only 100% of T-99 density; (3) a frost susceptible, sand-gravel subbase; and (4) the other material properties summarized in Reference [1].

New Jersey Experimental Pavement Project

Nine experimental test pavements were constructed in 1964 on Route I-80 and I-95 in New Jersey [7]. After eight years of service a detailed study [7] indicated all sections were performing satisfactorily after 1.2 million equivalent 18 kip (80 kN) single axle loads; these sections have continued to perform satisfactorily until the present time (1981). To eliminate pavement subgrade support as a variable in the study, the pavement sections were constructed in rock cut areas. The structural sections used are summarized in Table 2, and the gradations of the unstabilized stone bases are given in Table 3.

The performance of the nine test sections is summarized in Table 4. After 1.2 million equivalent 18 kip (80 kN) single axle loadings, best performance was shown by the following sections: (1) the deep strength bituminous section with 10 in. (254 mm) of bituminous material (Section 2), (2) an inverted section having a 4 in. (102 mm) dense graded stone base overlying an 8 in. (203 mm) cement treated base, and (3) Section 9 which had both a 6 in. (152 mm) dry bound macadam base and quarry processed stone subbase. Of interest is the fact that Section 9, the only thinner asphalt concrete section (6 in. of AC) exhibiting above average performance, used an unstabilized layer of dry bound macadam overlying quarry processed stone. Both of these materials had a coarser gradation (Table 3) than conventional graded aggregate bases.

Section 3 also showed above average performance. This section consisted of 4 in. (102 mm) of AC, 4 in. (102 mm) of bituminous base and 6 in. (152 mm)

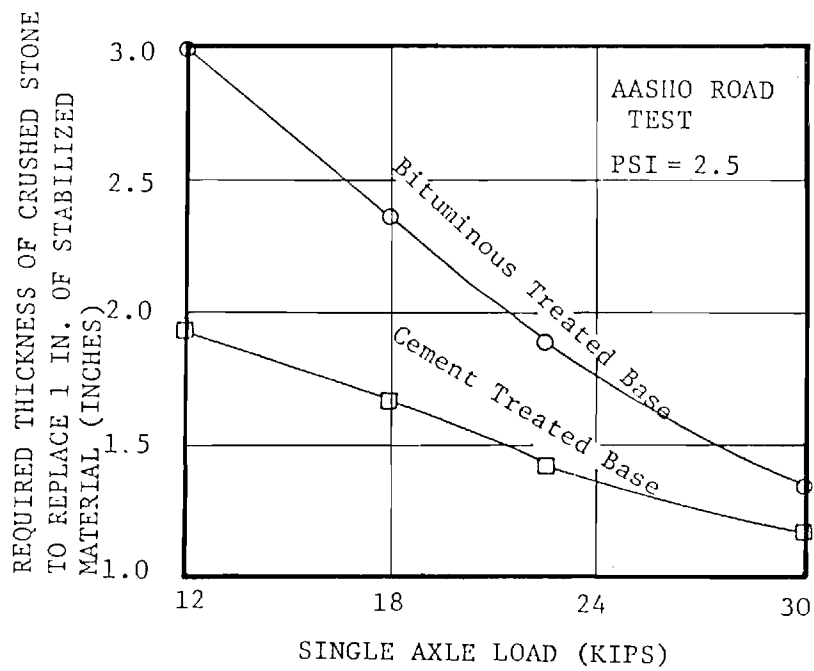


FIGURE 1. RELATIVE PERFORMANCE OF SPECIAL BASES AT AASHTO ROAD TEST AS A FUNCTION OF AXLE LOAD.

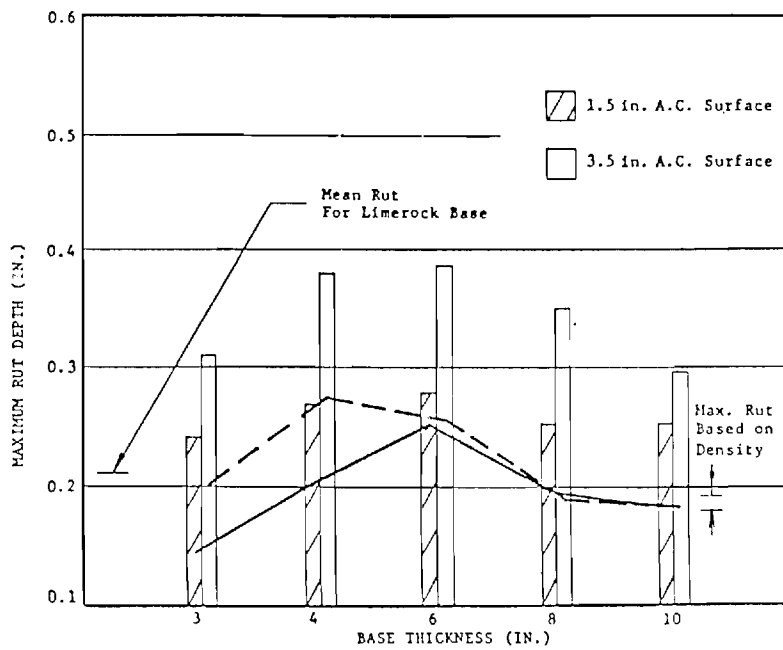


FIGURE 2. RUTTING IN LIMEROCK BASE SECTIONS AT LAKE WALES [8].

TABLE 2. STRUCTURAL SECTIONS USED IN NEW JERSEY EXPERIMENTAL PAVEMENT PROJECT.

Section	Construction				Rut ⁽¹⁾ (in.)	Crack ⁽¹⁾ (%)	R ⁽²⁾	PSI	Perf. ⁽³⁾ Rating	Rank
	Surface (in.)	Base 1 (in.)	Base 2 (in.)	Subbase (in.)						
1	4 AC	6 DBM	6 QPS	14 G	0.26	78	97	2.9	178	5
2	4 AC	6 BB	6 QPS	14 G	0.34	22	86	3.3	132	2
3	4 AC	4 BB	6 QPS	16 G	0.34	48	120	2.8	168	4
4	4 AC	6 BGB	6 QPS	14 G	0.53	62	87	3.1	200	7
5	4 AC	6 PM	6 QPS	14 G	0.29	95	86	3.1	197	6
6	4 AC	6 DGA	6 QPS	14 G	0.32	82	108	2.8	197	6
7	2 AC	4 BB	4 DGA	8CTB ⁽⁴⁾	0.32	22	108	2.8	137	3
8	2 AC	4 BB	6 DBM	6QPS ⁽⁴⁾	0.21	-	144	2.4	-	-
9	2 AC	4 BB	6 DBM	6QPS ⁽⁴⁾	0.24	5	115	2.7	104	1

- Notes:
1. Average rutting and cracking in inside lanes, east and west bound traffic.
 2. Average roughness, east and west bound inside lanes (Roughometer - units per mile).
 3. Cumulative % failed considering equally rutting, cracking and PSI; Rutting failure taken as 0.45 in; Example Sect. 1: $0.26/0.45 \times 100 + 78 + \{100 - (2.9/5.0) \times 100\} = 178$.
 4. A 12 in. gravel subbase is located beneath.
 5. Notation Used: AC = Asphalt Concrete; DBM = Dry Bound Macadam;
 QPS = Quarry Processed Stone; G = Gravel; BB = Bituminous Base;
 BGB = Bituminous Gravel Base; PM = Penetration Macadam;
 DGA = Dense Graded Aggregate; CTB = Cement Treated Base

TABLE 3. SUMMARY OF BASE COURSE GRADATIONS FOR NEW JERSEY
EXPERIMENTAL PAVEMENT.

Stone	Gradation											
	3-1/2	3	2-1/4	2	1-1/4	3/4	3/8	1/4	No. 4	No. 30	No. 40	No. 200
Quarry Processed	-	-	-	100	-	55-90	-	-	25-60	-	15-30	5-12
Macadam (Dry Bound)												
Stone Screenings	100	85-100	0-45	-	0-5	-	-	-	-	-	-	-
	-	-	-	-	-	100	-	40-100	-	25-65	-	5-20
Penetration Macadam(1)	100	85-100	0-45	-	0-5	-	-	-	-	-	-	-
Dense-Graded Stone	-	-	-	-	100 ⁽²⁾	80	-	68 ⁽³⁾	50	-	21	11
Bituminous Base												
Stone	-	-	-	-	100 ⁽²⁾	55-90	-	-	25-60	-	15-30	5-12
Gravel	-	-	-	-	100 ⁽²⁾	65-100	50-90	-	37-75	-	12-30	3-8

1. Large aggregate is 2-1/2 in. size and choke aggregate is 5/8 in. or 1/2 in. size. An 85-100 or 100-120 penetration grade asphalt cement is used.
2. 100% passing the 1-1/2 in. sieve.
3. 68% passing the 1/3 in. sieve.

of quarry processed stone. Section 6 had the thinnest bituminous surfacing used in the experiment. This section consisted of 4 in. (102 mm) of asphalt concrete overlying 6 in. (152 mm) of dense-graded stone and 6 in. (152 mm) of quarry processed stone. Baker and Quinn [7] ranked this section poorest; the ranking given in Table 4 also indicates poor performance with this section being next to last.

Summary. Important findings from this study related to granular bases are as follows: (1) the inverted pavement showed excellent performance under service conditions. The ranking given by Baker and Quinn was highest; Table 4 places this section as third best with not much difference between it and the second ranked deep asphalt concrete section, (2) the dry bound stone macadam performed better than the dense-graded stone base exhibiting the smallest rut depths of all sections. The dry bound macadam had a 3.5 in. (89 mm) top size and less than 5% passing the 3/4 in. (19 mm) sieve. The stone screenings used to choke the dry bound macadam had a 3/4 in. (19 mm) top size and 5 to 20% passing the No. 200 sieve. In comparison, the dense-graded aggregate base had a similar gradation to the stone used at the AASHTO Road Test: the top size was 1.5 in. (38 mm) with 11% fines. The coarser quarry processed stone may have performed better than the dense-graded aggregate although more data is needed.

Section 4, which had a total of 10 in. (254 mm) of asphalt concrete, showed below average performance and the largest amount of rutting. This section had 6 in. (152 mm) of bituminous stabilized gravel base, 6 in. (152 mm) of quarry processed stone and a 14 in. (356 mm) gravel subbase. Generally thinner asphalt concrete thicknesses gave lower rut depths. Sections 8 and 9, which had a dry macadam stone base showed the smallest rut depths. Both these sections had a relatively thin asphalt concrete surfacing (6 in. or 152 mm).

Surface cracking, which was limited to the upper lift, was related to hardening of the asphalt as defined by penetration. Traffic loading apparently prevented visible cracks from appearing on the surface in the truck lanes. Since the more lightly trafficked passing lane cracked, healing of the cracks due to repeated loading in the heavy traffic lanes apparently occurred.

Florida Experimental Pavements

Experimental pavements have been constructed and carefully monitored by the Florida DOT at Palm Beach, Lake Wales and Marianna [8]. Limerock bases are frequently used in Florida and were included in the Palm Beach and Lake Wales experimental pavements. Unstabilized limerock bases typically have about 25 to 40% material finer than the No. 200 sieve. The limerock is compacted following Florida DOT specifications to a minimum of 98% of AASHTO T-180 density. Limerock bases tend to increase in stiffness with age possibly due to cementing [8]. These bases, however, are also quite susceptible to water that enters the pavement; as a result rapid deterioration develops if the pavement cracks allowing water to enter [1,8].

Lake Wales. The Lake Wales experimental road consisted of either 1.5 or 3.0 in. (38 or 76 mm) of asphalt concrete surfacing overlying a 3 to 10 in. (76 - 254 mm) thick base constructed of either sand asphalt or limerock.

TABLE 4. PERFORMANCE OF NEW JERSEY EXPERIMENTAL PAVEMENTS⁽¹⁾.

Rank	Section	Construction ⁽⁶⁾					Avg. ⁽²⁾ Rut (in.)	Crack ⁽³⁾ Avg. (%)	PSI (Avg.)	Perfor- mance Rating ⁽⁴⁾	SN ⁽⁷⁾	Remaining Loading (x10 ⁶)
		AC Surf. (in.)	Bit. Base (in.)	Stone Base (in.)	Stone Base (in.)	Subbase (in.)						
1	9	2	4	6 DBM	6 QPS	12 G	0.24	5	2.7	104	4.52	6.5
2	2	4	6	6 QPS	-	14 G	0.34	22	3.3	132	5.26	8.6
3	7	2	4	4 DGA	8 CTB	12 G	0.32	22	2.8	137	5.24	16.2
4	8	2	4	6 DBM	6 QPS	12 G	0.21	-(5)	2.4	≤148 ⁽⁵⁾	6.0	7.3
5	3	4	4	6 QPS	-	16 G	0.34	48	2.8	168	4.48	4.0
6	1	4	-	6 DBM	6 QPS	14 G	0.26	78	2.9	178	4.42	4.2
7	5	4	-	6 PM	6 QPS	14 G	0.29	95	3.1	197	5.26	3.5
7	6	4	-	6 DGA	6 QPS	14 G	0.32	82	2.8	197	4.42	2.1
8	4	4	6	6 QPS	-	14 G	0.53	62	3.1	200	5.26	5.8

- Notes:
1. Based on Ninth Interim Report Data.
 2. Average of truck lane, both wheel paths and both east and west sections.
 3. Average of inside lane for both east and west directions.
 4. Performance rating is the sum (expressed as a percent) of rutting deterioration, cracking and PSI loss.
Rut failure is taken as 0.45 in.; Example (Sect. 9), Performance Rating = $0.24/0.45 \times 100 + 5 + (1.0 - 2.7/5.0) \times 100 = 104$
 5. Cracking data on Section 8 was not given; Performance rating is estimated.
 6. Notation Used: DBM = dry bound macadam; QPS = Quarry processed stone;
G = Gravel; DGA = Dense graded aggregate;
CTB = Cement treated base; PM = Penetration macadam
 7. AASHTO Structural Number using New Jersey layer equivalents; Soil Support = 6; Regional Factor = 1.25

Approximately 2.5 million equivalent 18 kip (80 kN) single axle loadings have been applied to the north bound traffic lane. The limerock base had an average of 81% passing the No. 10 sieve, 62.6% passing the No. 40 sieve and 35.0% passing the No. 200 sieve. The limerock was compacted between 97 and 101.5% of AASHTO T-80 maximum density. The optimum moisture content of the limerock was 11.1% compared with a field placement moisture of 10.1%. The sand asphalt consisted of 50% of a local, rounded "blow" sand and 50% screenings stabilized with 7.7% asphalt. All of the sand passed the No. 10 sieve and 2% passed the No. 200 sieve.

Potts, et al. [8] estimated that about 20 to 25% of the rutting occurred in the 1.5 in. (38 mm) surfacing, and 75 to 80% in the limerock or sand asphalt base section. In all sections having a 3 in. (76 mm) asphalt concrete surface, about 35% of the total rutting occurred in this layer. These percentages, however, appear to neglect rutting caused by shear distortion.

The sand asphalt base sections rutted about 50% more than the limerock base sections (maximum rut depths of 0.43 in., 11 mm, compared with 0.30 in., 8 mm). Almost identical levels of rutting were also observed at the Palm Beach Test Road for similar sections but considerably lighter traffic. Slightly more rutting occurred at Lake Wales in the limerock base sections having 3.0 in. (76 mm) of asphalt concrete compared with 1.5 in. (38 mm) as shown in Fig. 2. This is true for all base thicknesses indicating the asphalt concrete surface mix used at Lake Wales was more susceptible in the warm climate to rutting than the limerock base placed under a thin covering of asphalt concrete.

Surface cracking of the sand asphalt sections was also greater than for the limerock base sections (Fig. 3). Further, the limerock base sections having 1.5 in. (38 mm) of asphalt concrete surface exhibited no more (and perhaps even less) cracking than the sections having a 3.0 in. (76 mm) asphalt concrete surfacing. The Present Serviceability Index (PSI) of the sand asphalt sections after 2.5 million equivalent single axle loadings was about equal to or slightly higher than the limerock bases. This difference in performance was attributed by Potts, et al. to the initial difference in the pavement profile before traffic loading associated with differences in construction techniques.

Palm Beach. The Palm Beach test pavement consisted of a 1.5 in. (38 mm) thick asphalt concrete surfacing having a mean Marshall stability of 860 lbs. (3.8 kN) placed over either sand asphalt, shell or limerock bases varying from 3 to 9 in. (76-229 mm) in thickness. A total of 120,000 equivalent 18 kip (80 kN) single axle loads (both directions) were applied during 9 years. At the lightly trafficked Palm Beach Test Road after nine years the PSI of all sections except two having sand asphalt bases was greater than 4.0. Rutting was the only difference in performance. Rutting in the limerock base section was about 0.3 in. (8 mm) compared with 0.4 in. (10 mm) in the higher stability sand asphalt base sections and 0.5 in. (13 mm) in the shell base sections.

Colorado Experimental Base Project

A series of twenty full-scale test sections each 450 ft. (13 m) in length were constructed in Colorado in 1965 and monitored through 1978 to evaluate base course performance [9]. During twelve years of service,

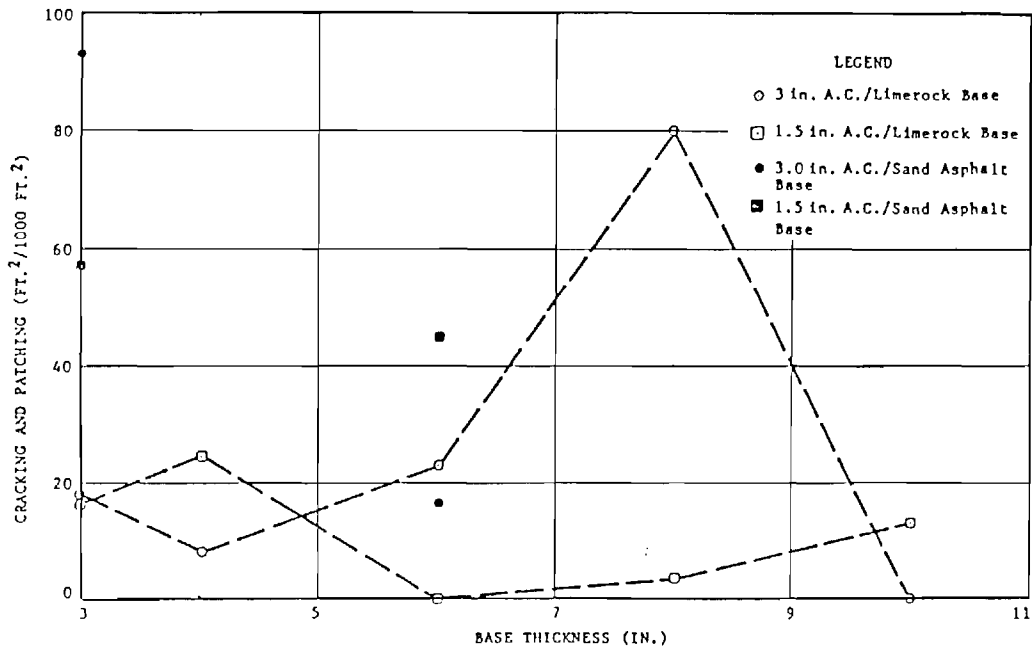


FIGURE 3. MAXIMUM CRACKING AND PATCHING OBSERVED ON LIMEROCK AND SAND ASPHALT SECTIONS AT LAKE WALES [8].

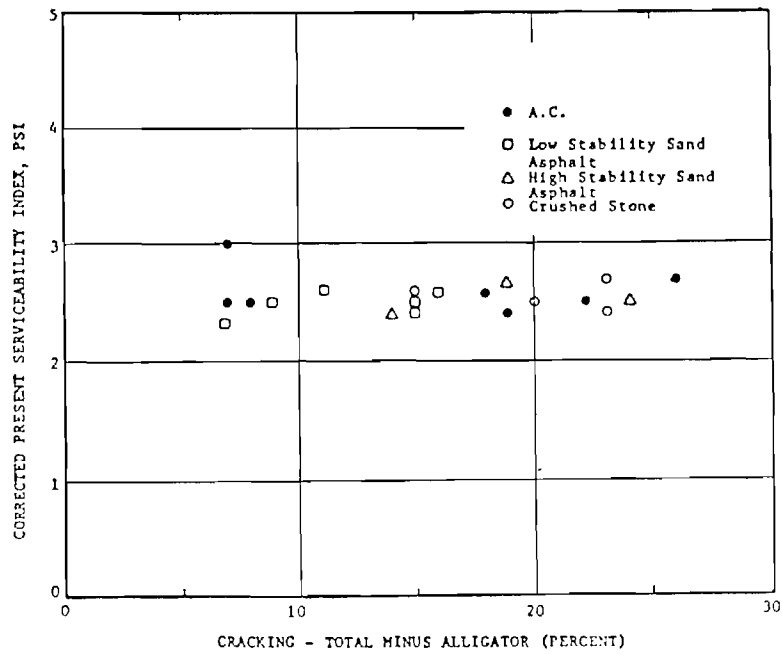


FIGURE 4. EFFECT OF CRACKING ON PRESENT SERVICEABILITY INDEX - COLORADO BASE STUDY [9].

only 140,000 equivalent 18 kip (80 kN) axle loads were applied to the pavement. Structural designs evaluated include two full-depth sand-asphalt bases, one full-depth asphalt concrete base, and one design having an untreated gravel base and a subbase of varying thickness. Full-depth asphalt concrete and sand asphalt sections had thicknesses varying from 5.5 to 9.4 in. (140-216 mm). The conventional sections had an average asphalt concrete thickness of 2.6 in. (66 mm), a 4 in. (102 mm) gravel base and 14 in. (356 mm) gravel subbase. Sections were constructed on both an A-7-6 and A-6 subgrade.

The asphalt concrete surface and base mix had an asphalt content of 5.6%. The well-graded coarse aggregate fraction of the surface course had at least 60% crushed stone with a minimum of two crushed faces. An uncrushed gravel was used in the asphalt concrete base with this mix being slightly coarser than the surface. The sand asphalt had 8% asphalt and 3 to 6% air voids. Marshall stability of the low stability mix is 451 lb. (2 kN) and the high stability mix 770 lbs. (3.4 kN); mineral filler was used to achieve the higher stability mix. The sand asphalt was placed at an average of 98.6 to 98.9% of the 50 blow Marshall density and the asphalt concrete at 99%. The 1/2 in. (12 mm) maximum size uncrushed gravel used in the thin base had 9% passing the No. 200 sieve. The 3 in. (76 mm) maximum size, uncrushed gravel subbase had 5% passing the No. 200 sieve. The average density of the base and subbase was 101.7 and 102.5%, respectively of the AASHTO T-99 density. Gravel densities varied from 133 to 136.7 pcf (20.9-21.5 kN/m³) for one standard deviation.

Performance. Typical test results are summarized in Figs. 4 and 5. A statistical study indicated no practical difference in PSI between the asphalt concrete, sand asphalt and unstabilized gravel base sections after 12 years (Fig. 4). Rutting in the full-depth bituminous sections was about twice that in the unstabilized gravel base sections (Fig. 5). After 7 years, little difference in alligator cracking existed between the full-depth asphalt and unstabilized gravel base sections. During the seventh to ninth years, however, more alligator cracking occurred in the gravel base sections than in the full-depth bituminous sections. The full-depth asphalt concrete sections placed on the A-7-6 subgrade had cracks over an average of 22% of the area compared with an average of 32% for the unstabilized crushed stone base. Alligator cracking was most important in the unstabilized base sections while transverse and longitudinal cracking was greater in the full-depth sections. No significant difference in alligator cracking existed between the full-depth bituminous sections.

After 12 years of service the PSI values for both the asphalt concrete sections and the granular base/subbase sections were about the same. Also a significant amount of cracking was present in both the bituminous (22%) and granular base (32%) sections. The unstabilized base sections had an average of 0.13 in. (3 mm) of rutting compared to 0.19 in. (48 mm) of rutting in the full-depth bituminous sections. From considerations of cracking, Shook and Kallas [9] recommend base equivalencies of 3.4 to 4.25 for unstabilized base/subbase design for the materials used in the study. Considering the PSI ratings were the same, these ratios are considered too high. Due to the light traffic, the relatively thick granular base sections would very likely have undergone about the same rutting and cracking had a thinner subbase been used. For the light traffic conditions of the study, use of a crushed gravel compacted to 100% of T-180 maximum density would have significantly improved performance. For these conditions considerably higher substitution ratios would be appropriate.

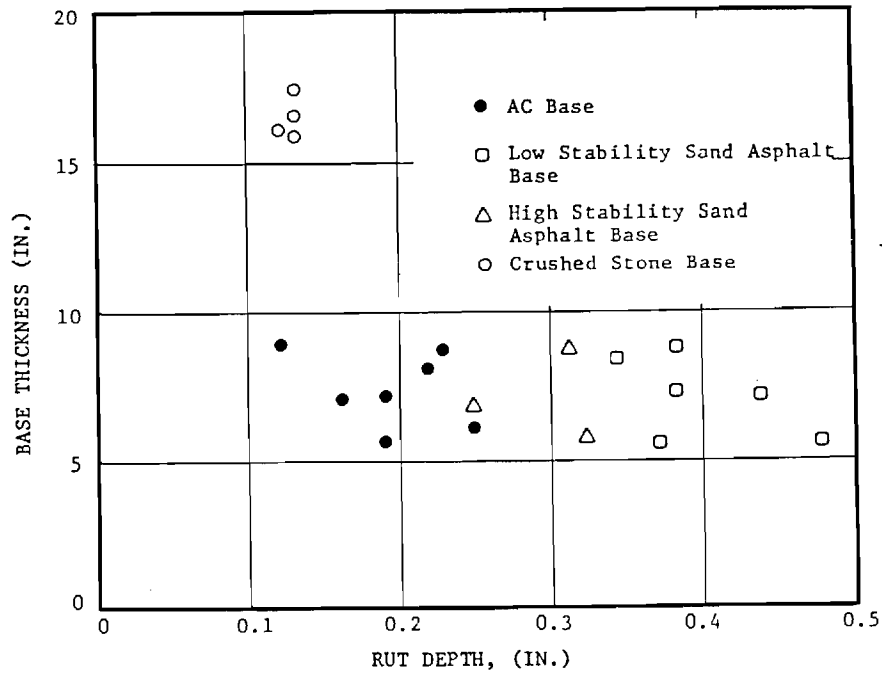


FIGURE 5. INFLUENCE OF BASE TYPE AND THICKNESS ON RUT DEPTH [9].

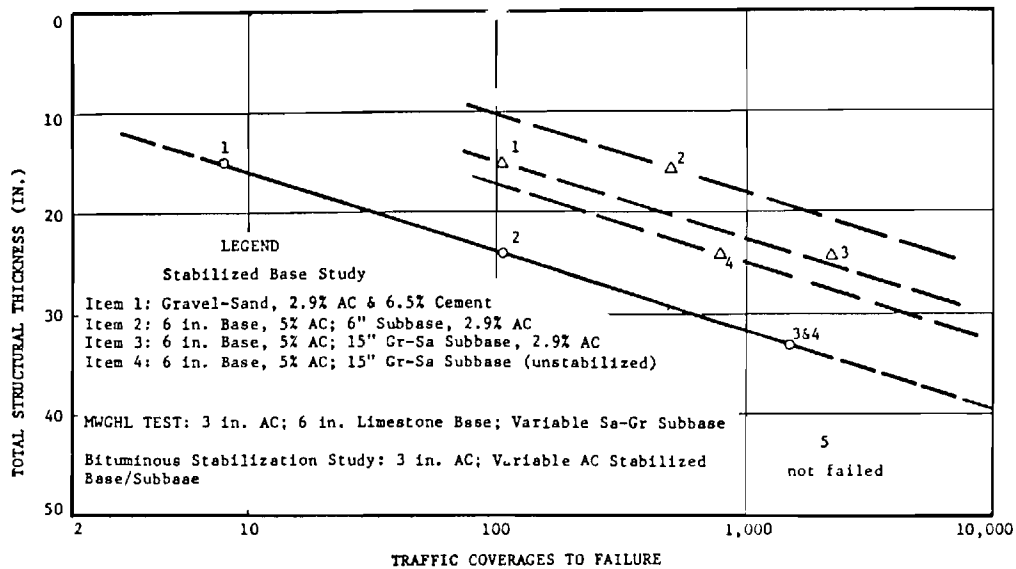


FIGURE 6. EFFECT OF STRUCTURAL THICKNESS AND TYPE CONSTRUCTION ON COVERAGES TO FAILURE - 360,000 LB., 12-WHEEL ASSEMBLY, 100 PSI TIRE PRESSURE [12].

only 140,000 equivalent 18 kip (80 kN) axle loads were applied to the pavement. Structural designs evaluated include two full-depth sand-asphalt bases, one full-depth asphalt concrete base, and one design having an untreated gravel base and a subbase of varying thickness. Full-depth asphalt concrete and sand asphalt sections had thicknesses varying from 5.5 to 9.4 in. (140-216 mm). The conventional sections had an average asphalt concrete thickness of 2.6 in. (66 mm), a 4 in. (102 mm) gravel base and 14 in. (356 mm) gravel subbase. Sections are constructed on both an A-7-6 and A-6 subgrade.

The asphalt concrete surface and base mix had an asphalt content of 5.6%. The well-graded coarse aggregate fraction of the surface course had at least 60% crushed stone with a minimum of two crushed faces. An uncrushed gravel was used in the asphalt concrete base with this mix being slightly coarser than the surface. The sand asphalt had 8% asphalt and 3 to 6% air voids. Marshall stability of the low stability mix is 451 lb. (2 kN) and the high stability mix 770 lbs. (3.4 kN); mineral filler was used to achieve the higher stability mix. The sand asphalt was placed at an average of 98.6 to 98.9% of the 50 blow Marshall density and the asphalt concrete at 99%. The 1/2 in. (12 mm) maximum size uncrushed gravel used in the thin base had 9% passing the No. 200 sieve. The 3 in. (76 mm) maximum size, uncrushed gravel subbase had 5% passing the No. 200 sieve. The average density of the base and subbase was 101.7 and 102.5%, respectively of the AASHTO T-99 density. Gravel densities varied from 133 to 136.7 pcf (20.9-21.5 kN/m³) for one standard deviation.

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Granular Airfield Bases

Crushed Stone and Bituminous Stabilization

Burns and Ahlvin [10] describe the results of accelerated, full-scale field tests using heavy aircraft loadings conducted at the Waterways Experiment Station. These tests compared full-depth bituminous sections of varying quality with granular base sections having a 3 in. (76 mm) asphalt concrete surfacing. The test sections were loaded to failure under a C-5A multiple-wheel, main gear loading. All sections were placed over 36 in. (0.91 m) of heavy clay subgrade (CBR=4), which in turn was constructed on a lean clay subgrade also having a CBR of 4.

The full-depth bituminous sections were 15 and 24 in. (381 - 610 mm) in thickness and constructed with varying quality bituminous layers. The unstabilized granular sections had 12 to 39 in. (305 - 991 mm) thick granular layers with a 3 in. (76 mm) thick asphalt concrete surfacing. The granular sections consisted of a 6 in. (152 mm) thick crushed limestone base constructed over a gravelly sand subbase of varying thickness. The gravelly sand passed the 1.5 in. (38 mm) sieve with about 50% passing the No. 4 sieve, and 1% passing the No. 200 sieve.

In one series of tests, a 360,000 lb. (1602 kN), 12 wheel assembly was used having a 100 psi (689 kN/m²) tire pressure. The comparison of coverages to failure of the bituminous and granular base sections is shown in Fig. 6. One bituminous section consisted of a 12 in. (305 mm) gravelly sand base stabilized with 2.9% asphalt (Item 1) underlying a 3 in. (76 mm) asphalt concrete surfacing. The granular base MWHGL section had a 3 in. (76 mm) asphalt concrete surfacing, a 6 in. (152 mm) crushed stone base and 15 in. (381 mm) of gravelly sand subbase. The corresponding base course equivalency for 100 coverages to failure was about 1.75 for the test conditions. For a 75 kip (334 kN) single wheel loading having a 290 psi (2000 kN/m²) tire pressure, the corresponding base course equivalency about 1.9 for similar construction. Item 3 consists of 9 in. (231 mm) of high quality asphalt concrete overlying 15 in. (381 mm) of bituminous stabilized gravelly sand. Comparing this section with the standard unstabilized MWHGL sections having the deep gravelly sand subbase gives a base course equivalency of about 2.0. Further, a base course equivalency of about 2.2 is obtained when the high quality, full-depth asphalt concrete construction (Item 2) is compared with the unstabilized base section having the deep gravelly sand subbase.

Of practical interest is the finding that the high quality, full-depth asphalt concrete section has a base course equivalency of 2.2; the lower quality full-depth AC section utilizing both crushed stone and gravelly sand materials has an equivalency of 2.0; and the full-depth bituminous section having a gravelly sand bituminous base has an equivalency of 1.75. All of these bituminous sections are compared with unstabilized granular base sections having a deep gravelly sand subbase, 6 in. (152 mm) crushed stone base and a 3 in. (76 mm) asphalt concrete surfacing. Because of the presence of the thick, lower quality gravelly sand subbase in the unstabilized sections, the most valid base course equivalency for materials of comparable quality is about 1.75 to 1.9 for the heavy aircraft loadings.

Stabilization of the gravelly sand subbase with 2.9% asphalt gives a base course equivalency of 1.75 for the unstabilized material. Further,

changing the bottom 12 in. (305 mm) of Item 1 from uncrushed gravelly sand at 2.9% asphalt content to a crushed limestone at 4.5% asphalt content results in the number of coverages to failure increasing from 98 to 424 under the 12-wheel gear loading. These results illustrate the importance of using where practical a (1) high level of stabilization and (2) high quality materials.

Inverted Sections

Introduction

Successful use of cement stabilized subbase layers overlain by unstabilized granular bases was reported by Johnson [11] in New Mexico and at the Waterways Experiment Station by Grau [12]. Use of inverted sections in Virginia having a cement stabilized subgrade have already been described. Advantages of inverted construction include: (1) better compaction of unstabilized materials placed over the stabilized layers; this layer acts as a working platform where soft subgrades are encountered, (2) optimum use is made of unstabilized crushed stone, and (3) elimination or significant reduction in reflection cracking since the cement treated layer is placed deep in the section. Inverted sections make optimum use of the tensile strength of the cement treated subbase or subgrade in bridging over a weak subgrade through "beam" action. Optimum use is also made of the excellent compressive characteristics of unstabilized aggregate since it is placed in the upper part of the pavement structure where stresses are compressive (Fig. 7). For relatively high levels of cement stabilization required for durability and fatigue resistance, some shrinkage cracking develops in the cement treated layer. Therefore the inverted section should not trap as much water in the granular base as might be expected.

New Mexico

The "inverted" or "upside down" sections were first successfully used in New Mexico on urban projects having weak subgrades; two test sections were subsequently built which included this type construction [11]. Use of inverted pavement sections in New Mexico apparently first began in 1954 when several badly broken concrete pavements near Albuquerque were overlaid with 6 in. (152 mm) of unstabilized granular base and 2 in. (51 mm) of asphalt concrete. After six years of heavy traffic, no reflection cracking or significant amount of rutting had developed.

Following these early successful uses of inverted sections, two experimental roads were constructed in New Mexico in about 1960. A typical inverted section used on the test sections constructed on U.S. 64 North of Santa Fe consisted of a 3 in. (76 mm) asphalt concrete surfacing, 6 in. (152 mm) granular base and a 6 in. (152 mm) granular subbase treated with 4% cement. A 1 in. (25 mm) maximum size aggregate was used in both the untreated and cement treated layer. The untreated base was compacted to 100% (average) of Modified Proctor maximum density. The asphalt concrete was compacted to 100% of the 75 blow Marshall maximum density.

Shortly after construction, surface cracking was only visible in sections where the cement stabilized layers were immediately below the surface. Roughness measurements showed the inverted section to be the smoothest, although a significant difference in roughness was not observed between sections. Sections having treated base course materials immediately

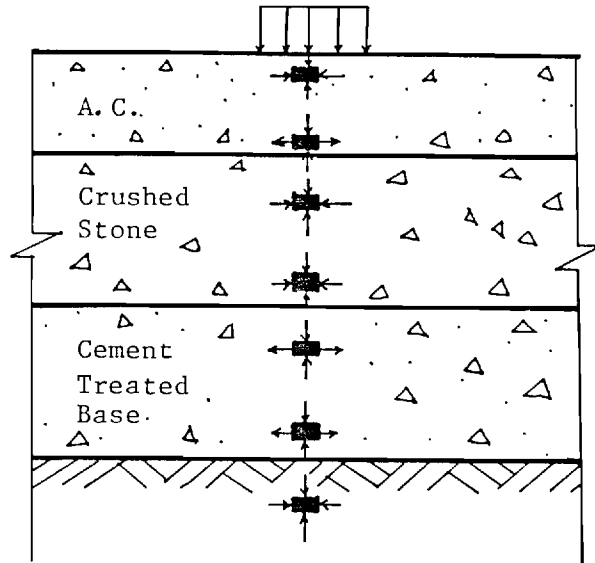


FIGURE 7. STATE OF STRESS IN AN INVERTED SECTION.

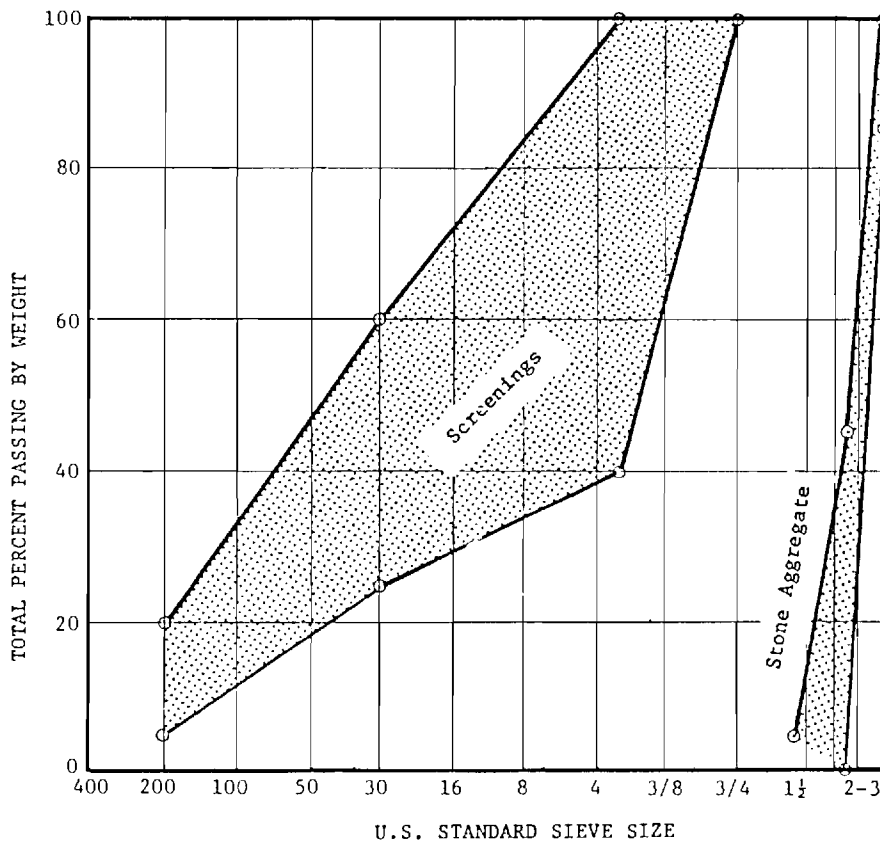


FIGURE 8. GRADATION OF STONE AGGREGATE AND SCREENINGS USED IN NEW JERSEY EXPERIMENTAL ROAD MACADAM [72].

below the surfacing were slightly rougher than others. The inverted section also had the smallest Benkelman beam deflection. As expected, the largest Benkelman beam deflections were observed in the conventional unstabilized sections. Longterm performance information was not given on these sections.

Eight test sections were also constructed on the Roads Forks-East project in New Mexico including two inverted sections. The inverted sections consisted of a 6 in. (152 mm) untreated granular base (PI=6) having a 1 in. (25 mm) maximum size aggregate. A 6 in. (152 mm) thick subbase treated with 3% cement was located beneath the base. One inverted section had a 3 in. (76 mm) asphalt concrete surfacing and 9 in. (229 mm) untreated subbase; the other inverted section had a 1.5 in. (38 mm) asphalt concrete surfacing and a 3-1/2 in. (89 mm) untreated subbase below the treated subbase.

After one year of heavy traffic no significant differences in performance were observed between the inverted and conventional pavement sections. Neither longitudinal nor transverse cracking of the mainline pavement was evident in the inverted sections; some longitudinal cracking was, however, observed on the shoulder. One section with a 6 in. (152 mm) cement stabilized base immediately below the surface had transverse and longitudinal cracking on both the inner edge of the pavement and on the shoulders. Rutting in all sections was between 0.12 and 0.25 in. (3-6 mm). The higher strength inverted section had 0.12 in. (3 mm) of rutting while the other inverted section had 0.25 in. (6 mm).

Macadam Bases and Open-Graded Drainage Layers

Introduction

Stromberg [13] has found in Maryland that on poor subgrades a water bound macadam performed significantly better than a dense-graded aggregate base which performed relatively badly. The water bound macadam (2-1/2 to 3 in., 64-76 mm, max. size stone) typically had approximately 50% passing the 1 in. (25 mm) sieve and 4 to 8% fines. In comparison, the dense graded aggregate had about 90% passing the 3/4 in. (19 mm) sieve and 7 to 15% fines. Baker and Quinn [7] found in New Jersey that a dry bound macadam base having stone up to 3-1/2 in. (89 mm) in size performed significantly better than a crushed limestone similar to that used at the AASHTO Road Test. Wisconsin and Delaware have also apparently recognized that dry and waterbound macadam bases tend to perform better than regular bases since they have assigned base course coefficients for use in the AASHTO Interim Guide [2] of 0.15 to 0.20 for macadam bases as compared with values of 0.1 to 0.14 for crushed gravel and crushed rock bases.

Possible factors contributing to the good performance of the dry and water bound macadam bases include: (1) A crushed stone up to 3 in. (76 mm) in size having a minimum number of point contacts was used that was larger in top size than dense-graded aggregate base stone, (2) Since the coarse stone is placed and vibrated before finer choker material is added to the top, the finer material added should fill the voids and not, to any significant degree, get between the contact points of the larger particles, (3) The macadam bases tend to have a coarser gradation than graded aggregate bases, and (4) The choker material used results in a relatively small percent fines in the finished water bound macadam layer. These findings together with those from WES [10,12] suggest improved performance

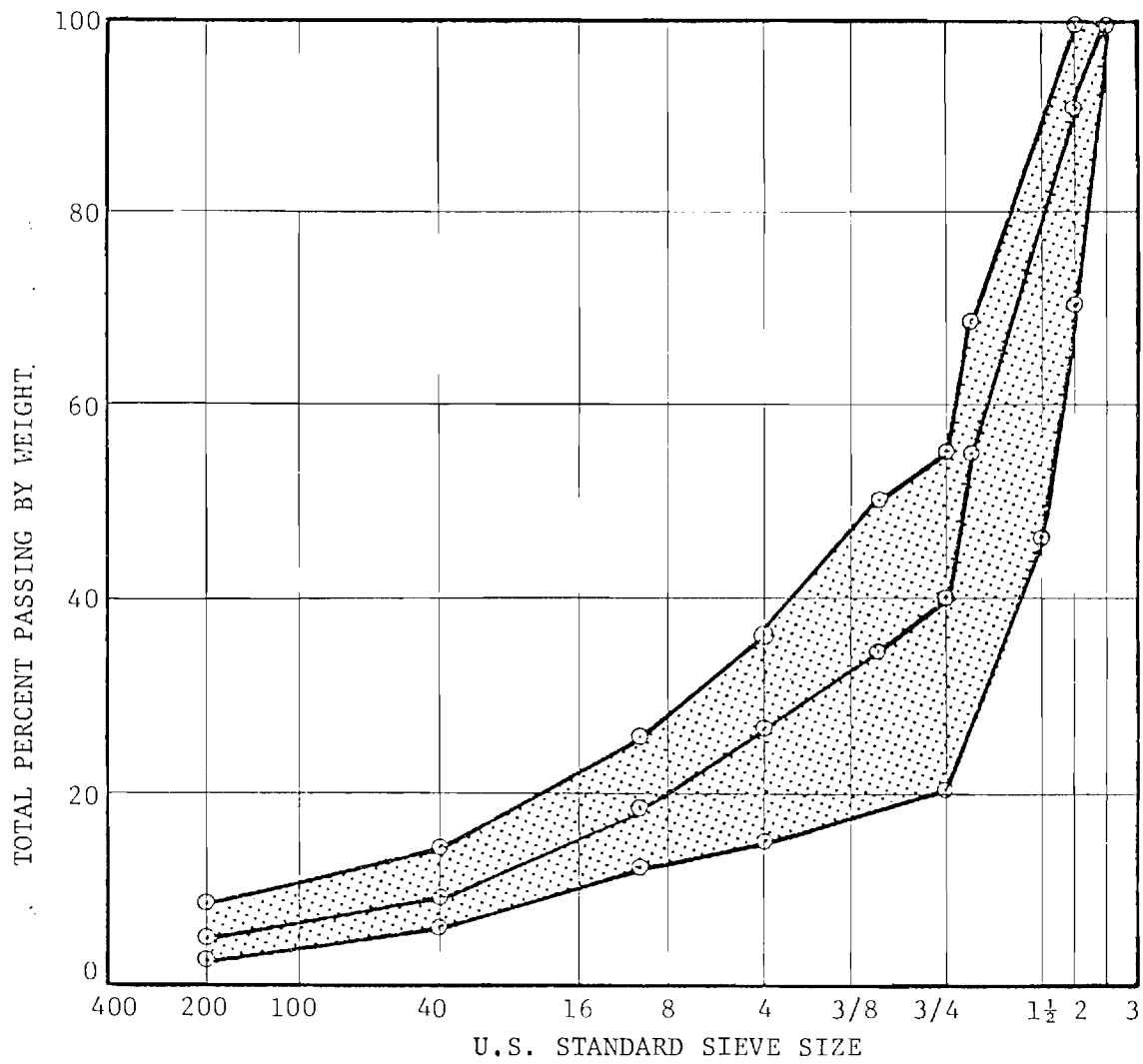


FIGURE 9. EXTREMES AND AVERAGE GRADATION OF DRY BOUND MACADAM USED IN MARYLAND [13].

of a crushed stone base can be obtained using a relatively large top size stone (2-3 in., 51-76 mm, max. size) having a coarse gradation and less than 6 to 8% fines. Repeated load triaxial test results presented by Barksdale [14] agree with this concept.

New Jersey Dry Bound Macadam

In New Jersey dry bound macadam bases have been constructed using the (1) vibrating method, and (2) rolling method of construction. Since more hand work is required using the rolling method, the vibrating method is of most interest. The macadam stone gradations used in the experimental road in New Jersey [7] are given in Fig. 8, and Fig. 9 gives the combined macadam gradation used in Maryland [13] for one pavement. Dry bound macadam bases have not been constructed by the New Jersey DOT since about 1965. As a result, engineers personally familiar with this type construction could not be located. The following description of dry bound stone macadam base construction is taken from the standard specifications of the New Jersey Department of Transportation.

Dry bound macadam bases constructed in lifts 4 in. (102 mm) or more thick use a 2.5 in. (64 mm) dia. stone; a 1.5 in. (38 mm) stone must be used in thinner lifts. Before placing the coarse aggregate base, an inverted choke stone layer 1 in. (25 mm) in thickness is placed on the subgrade using a stone spreader. The thin inverted layer is not, however, compacted until after the base has been placed. For base courses greater than 5 in. (127 mm) in total thickness, the vibrating method of construction is used; thinner bases are constructed using either the vibrating or rolling method.

Vibrating Method. After the choke stone layer is placed, the large stone is spread using an aggregate spreader. Two to five passes of a vibratory roller is used to key the aggregate together. The base course is then rolled longitudinally using either a 10 ton, three-wheel roller or with two or three-axle, tandem rollers weighing 8 or more tons. Rolling starts at the edges and progresses toward the center. On super elevated curves longitudinal rolling starts at the low side and progresses toward the high side. Where practical, diagonal rolling is also performed.

After rolling, the voids are filled with stone screenings. The screenings are applied using a spreader in three applications. Approximately 50% of the total quantity of dry screenings is placed in the first application and 25% in the second and third. After each application, the stone is worked into the surface using a single pass of the vibrator. After three applications, any voids visible on the surface are filled by hand spreading, brooming and rolling. The surface is then rolled again with either the three-wheel or tandem rollers until no movement of the base is visible under the action of the roller.

Rolling Method. The rolling method is used for only lifts less than 5 in. (127 mm) in thickness. Using this method the keying and compaction of the lifts composed of large aggregate is accomplished using three wheel, and two-axle or three axle tandem rollers. A stone spreader is then used to place a thin lift of dry screenings on the surface which is broomed into

the visible voids. The screenings are placed so as to neither cover nor form a layer over the coarse stone. Screenings that do not properly work into the voids are removed. In areas where all voids are not completely filled, additional screenings are added and swept into the voids. After filling all voids, the surface is rolled in the same manner as for the vibrating method of construction.

Iowa Dry Bound Macadam

In several rural counties of Iowa a number of low volume roads have been recently constructed using an open-graded crushed limestone, dry bound macadam base [15]. The base is usually covered with either a surface treatment, asphalt prime and chip seal, or about 2 in. (51 mm) of asphalt concrete surfacing. In some instances, the base has been left exposed for a year before the surfacing is applied. During this time the surface is kept smooth by blading and light applications of calcium chloride. The open-graded stone is laid full width of the section on a dense-graded stone filter layer. A steel wheel vibratory roller is used to compact all the granular layers.

The crushed stone used in the dry bound macadam is generally taken directly from the primary crusher. This material has a maximum size of 4 in. (102 mm) and less than 30% finer than the No. 8 sieve. After placing the open-graded base, a 2 to 3 in. (51-76 mm) thick layer of finer graded choke stone is placed on top to stabilize the open-graded stone and give a smooth working and/or riding surface.

In some instances a variation is used of the construction sequence described above. Following this modified procedure, the stone is separated at the quarry in a 1 in. (25 mm) sieve. The coarser fraction is used for the open-graded free draining base while the finer fraction is used for the choke stone layer.

Open-Graded Drainage Layers

Several free-draining, granular bases and subbases have been successfully constructed in the United States [16,17]. Unstabilized, very open-graded aggregate layers have generally been placed below the structural section to act as a drainage layer. In some instances the open-graded aggregate has been stabilized with asphalt for use as either a drainage layer or a combined base and drainage layer [16].

An open-graded, unstabilized granular base was constructed at the Kansas City International Airport. The very open-graded base consisted of a No. 4 stone covered with only 2.5 in. (64 mm) of asphalt concrete [17]. This open-graded base was placed over a layer of well-compacted, dense graded limestone. Unfortunately, performance information is not available on this section.

The problems associated with constructing open-graded, unstabilized layers, and the required construction techniques are of interest if larger top size, coarser bases are used in the future as base courses. Both the Commonwealth of Kentucky and the Pennsylvania Department of Transportation have constructed unstabilized drainage layers using a No. 57 stone [16]. The description given of construction on Kentucky State Route 55 of an unstabilized drainage layer has been given by Drake [16] and is as follows:

"The drainage blanket was rolled, compacted, or seated with a 10 ton tandem steel wheel roller and because it was unstable, it is neither practical nor possible to maintain any type of traffic over the unprotected blanket. It was very difficult to hold the stone in place and to the proper section during the placing of the bituminous concrete base overlay.

The contractor wisely selected to use a Barber Green Paver that was truck mounted which gave excellent traction. It was essential to operate a roller out ahead of the paver to shape the stone and improve the stability by aggregate interlock in order to allow the bituminous concrete delivery trucks to back into and unload the paver. By utilizing crushed limestone, it was practical to construct the drainage blanket with conventional hauling and paving equipment. It was necessary that the trucks use much caution when operating on the blanket. It was necessary to allow the first course of bituminous concrete to cure out adequately in order to gain strength for a period of from three to seven days, depending upon the temperature". And "By allowing the bituminous concrete to cure out properly, the second and third courses were constructed in a most conventional manner".

From the above description, the conclusion is reached that considerable caution is required in using open-graded aggregate similar to the No. 57 stone used in Kentucky. As expected, the surface of such materials is quite unstable and readily moves around and shoves. Use of a larger top size aggregate and choke stone as has traditionally been done in macadam construction would certainly help to stabilize the surface.

Subgrade Support Effects

General

Field results indicate poor subgrades and water have a very significant detrimental effect on pavement performance [1,13,18]. A comprehensive study conducted in Maryland [13] indicates pavements constructed on poor subgrades have a wide variation in performance, but in general perform relatively poorly. These poor subgrade soils usually have CBR values less than 8 to 12 and AASHTO Classifications of A-2-6, A-2-7, A-4, A-5, A-6, A-7, and A-7-6. Further, dense graded aggregate bases constructed on poor subgrades perform poorly, but water bound macadam bases perform quite well. Use of asphalt concrete surfacings greater than 4 in. (102 mm) thick above dense graded aggregate bases result in no significant improvement in performance when placed over poor subgrades [13].

Even under optimum field conditions a pavement constructed on a very good subgrade probably would not have the long life predicted by the Interim Guide [2] using high soil support values. In applying the Interim Guide to excellent subgrade conditions, possibly an effective limiting subgrade soil support value should be used, primarily because of the

practical limitations of the materials placed above the subgrade. For the conditions at the New Jersey experimental study [7], a limiting subgrade soil support value of approximately 6 to 7 ($18 \leq \text{CBR} \leq 32$) appears to exist [1]. These pavements were constructed in rock cuts and hence had an infinitely high subgrade strength. Some of the deterioration observed in New Jersey is attributed to hardening of the asphalt concrete surfacing; lack of drainage of water into the subgrade may also have been a factor [18]. The hypothesis of a limiting subgrade strength tends to be supported by the existence of an effective 18 in. (457 mm) limiting total thickness of deep granular sections used in Maine [6].

Piedmont Residual Soils

The resilient micaceous silty sands, sandy silts, and clayey silts of the Piedmont are well known as poor subgrades [1,3]. In Virginia the soil support values for these resilient subgrades have been found to be actually lower than indicated by CBR tests [3]. To give predicted lives similar to those observed, a soil support value approximately equal to 3 is required which corresponds to a CBR value of about 2.8. Measured CBR values varied from a minimum of 3 up to 11. These and other experimental results [1,3,13] indicate that the lower measured values of subgrade strength should probably be used for design purposes, particularly for the weak, resilient subgrades of the Piedmont Province.

Conclusions

With sufficient care pavements can be successfully constructed using relatively thin asphalt concrete surfacings as proven by the Lake Wales, Virginia and WES experimental pavements. Performance of a granular base is greatly affected by many factors including (1) quality of aggregate including gravel compared to crushed stone, (2) aggregate size, gradation, and angularity, (3) level of compaction, (4) magnitude of wheel loading, (5) aggregate segregation, and (6) level of field inspection. As a result of these and other factors, the variation of performance of pavements having unstabilized granular bases and/or subbases in general is greater than for asphalt concrete bases.

Full-scale test pavement results indicate rutting in pavements constructed with granular bases should, if good practices are followed, be no greater than full-depth asphalt concrete sections. Under favorable conditions such as the warm climate of the southeast, rutting in granular base sections could even be less than in full-depth sections. Additional care is, however, required in constructing unstabilized granular bases including using the proper gradation, achieving a high level of density and minimizing aggregate segregation during construction.

Base course equivalencies and base course coefficients are often misused being applied to conditions for which they are not valid. These equivalency factors must therefore be associated with two specific base materials and specific conditions of design and construction. The base course equivalency determined from well constructed test pavements is typically that 1.0 in. (25 mm) of black base is equivalent to 1.6 to 2.5 in. (41 - 64 mm) of granular base. Under unfavorable conditions such as existed in Colorado the equivalency can be greater; unfavorable conditions at the

Colorado site included low density, use of an uncrushed gravel base/subbase and light loading.

Inverted pavement sections consist of an unstabilized granular base sandwiched between an asphalt concrete surfacing and cement stabilized granular subbase or cement stabilized subgrade. Optimum use of the crushed stone is thus made in an inverted section since it is both confined and primarily subjected to compression. Inverted sections have been successfully used in New Mexico, Virginia and at the Waterways Experiment Station. Inverted construction appears to offer promise particularly for construction of pavements over weak subgrades. Inverted construction certainly deserves further study in Georgia, particularly for use over the micaceous silty sand subgrades of the Piedmont Province.

Three experimental pavement studies in Virginia show that a relatively high level of cement stabilization (10% by volume) of the subgrade improves performance of pavements constructed on micaceous Piedmont soils. Use of a select borrow of local material (CBR=33) placed over a cement stabilized micaceous Piedmont subgrade is not beneficial to pavement performance.

Macadam bases having a coarse gradation and large top size aggregate have been found to perform better than conventional graded aggregate bases on weak subgrades. Dry bound macadam bases are presently being constructed in Iowa for low volume county roads and previously were constructed in the northeastern states.

Pavements constructed over poor subgrades such as the resilient micaceous silty sands of the Piedmont Province often perform below expectations. For design probably the lower limit of the observed CBR values should be used in the Interim Guide design method. In many instances the pavement may even perform as if the soil is weaker than the lowest observed CBR value. In rock cuts an upper limiting value of subgrade support appears to exist associated with a limiting CBR value of about 18 to 32.

CHAPTER III MATERIAL PROPERTIES

Introduction

The material properties of the components of a pavement structure have a significant influence on performance. Therefore, to document the properties of the materials used in the twelve test sections extensive standard and dynamic tests were performed as a part of this study. Dynamic tests included fatigue, rutting and bending modulus of the asphalt concrete, and rutting and resilient modulus tests of the subgrade and crushed stone materials.

Asphalt Concrete

The asphalt concrete used in all sections was either a modified-B or B-binder for the full-depth of asphalt. These mixes were obtained from either the Lithia Springs or Forest Park plant of APAC-Georgia, Inc. An AC-20 viscosity grade asphalt cement was used in the mix obtained from the Shell Oil Company (Trumbull Products, Atlanta, Ga.). The typical physical properties of the asphalt cement are given in Table 5.

Physical Properties

Fifth blow Marshall mix designs for the asphalt concrete mixes used in this study were prepared by the Georgia Department of Transportation, Office of Materials and Research. A summary of the asphalt concrete mixes used in this study are given in Figs. 10 through 13. The type mix and test sections for which the mix applied is indicated on each figure.

The properties of the asphalt concrete placed in each section are summarized in Table 6 including gradation, asphalt content, unit weight and air voids. The air voids were approximately determined using an average unit weight and representative theoretical maximum specific gravity. These physical properties were determined by the Office of Materials and Research using 4 in. (100 mm) dia. cores taken from the pavement after failing the section. Before testing was begun on each section, the density of the asphalt concrete was generally determined by a Georgia DOT technician using a nuclear gage; these densities as a percent of the Marshall design density are also given in Table 6.

TABLE 5. ASPHALT CEMENT PROPERTIES.

Property	Sections					
	1,2	3,4	5,6	7,8	9,10	11,12
Manufacturer	Shell	Fulco	Shell	Shell	Shell	Shell
Viscosity at 140°F (poises)	1875	1942	1867	1838	1917	1985
Viscosity at 275°F (poises)	368	390	405	379	387	393
Penetration (77°F, 100g, 5s)	63	61	70	65	68	63
Flash Point (Cleveland open cup) (°F)	587	600	615	615	550	590
Specific Gravity @ 60°F	1.022	1.020	1.023	1.024	1.031	1.037
Tests on Thin Film Residue						
a) Viscosity @ 140°F, 30 cm Hg. (poises)	4866	5028	5516	4925	5985	5492
b) Ductility @ 77°F, 5 cm/min. (cm)	135+	150	150	150	150	150
c) Ductility @ 60°F, 5 cm/min. (cm)	8.1	9.9	11.0	10.3	11.0	13.0

FIGURE 10. ASPHALT CONCRETE B-MODIFIED MIX DESIGN APPLICABLE TO SECTIONS 1 THROUGH 4.

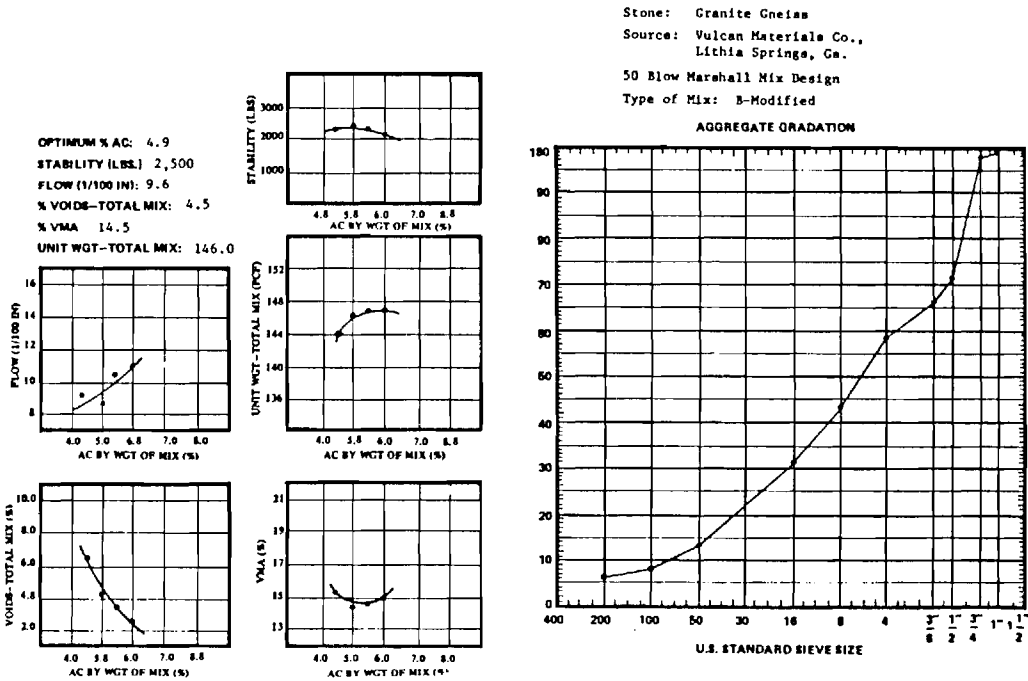


FIGURE 11. ASPHALT CONCRETE L-MODIFIED MIX DESIGN APPLICABLE TO SECTIONS 5 THROUGH 8.

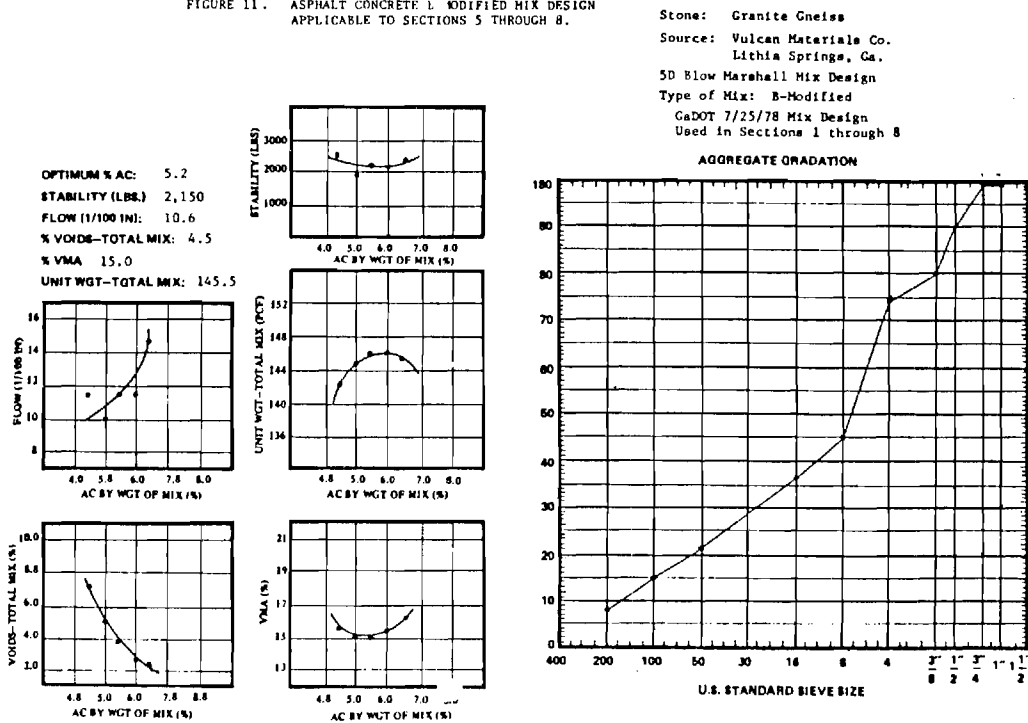


FIGURE 12. ASPHALT CONCRETE B-MIX DESIGN
 APPLICABLE TO SECTIONS 9 AND 10).

Agg. Type: Dixie Lime and Stone
 Source: Mountain View, Ga.
 50 Blow Marshall Mix Design
 Type of Mix: B-Mix
 CaDOT 11/19/74 Mix Design
 Used in Sections 9 and 10.

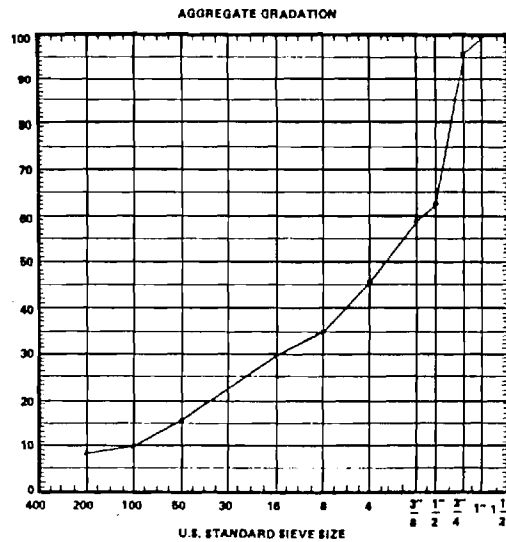
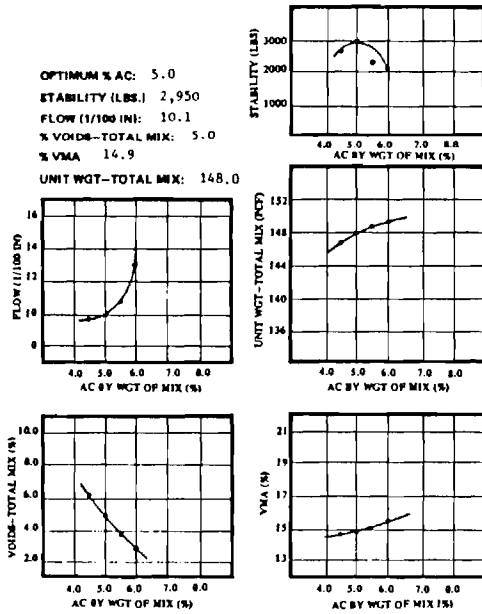


FIGURE 13. ASPHALT CONCRETE B-MIX DESIGN APPLICABLE TO SECTIONS 11 and 12.

Stone: Granite Gneiss
 Source: Vulcan Materials Co.
 Lithia Springs, Ga.
 50 Blow Marshall Mix Design
 Type of Mix: B-Mix
 CaDOT 7/1/81

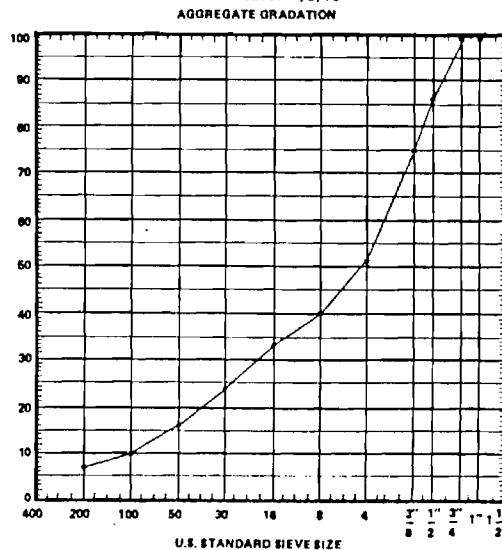
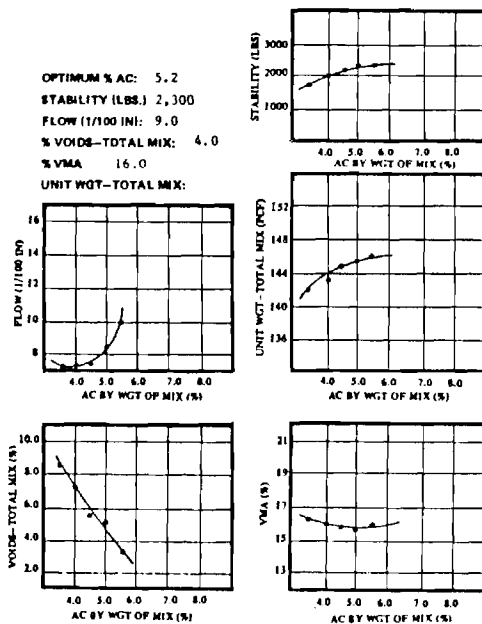


TABLE 6. NUCLEAR DENSITIES AT TIME OF CONSTRUCTION AND EXTRACTION DATA FOR CORES
TAKEN SUBSEQUENT TO SECTION FAILURE.

Section	Cumulative % Passing (by wt.)								Asphalt Content (%)	Unit Weight (pcf)	Air Voids (%)	Number of Cores	Construction Density (Nuclear) % Blow Marshall
	Sieves												
	1"	3/4"	1/2"	3/8"	4	8	50	200					
1 & 2	100	99.5		72		42		4	4.61	143.7	5.8	8	
3 & 4	97	97		71		40	21	8	5.9	145.1	3.8 ⁽¹⁾	1	91.3
5		100	89	73	53	41	16	6	5.71	145.8	3.6 ⁽¹⁾	4	97.2
6		100	86	70	49	38	15	6	5.17	144.8	5.0 ⁽¹⁾	6	96.1
7	100	99.7	96	85	60	46	17	6	5.60	145.9	3.7 ⁽¹⁾	8	
8	100	99.4	82	70	51	41	18	8	4.23	144.2	6.7 ⁽¹⁾	6	
9 & 10		100	76	62	48	38	22	7	4.9	148.0	5.2 ⁽²⁾	4	98.6
11 & 12												7	99.2

(1), (2): Based on theoretical S.G.
(Taken from Ga. DOT Design Analysis
Sheets).

Fatigue Properties

Sample preparation and testing procedures of the fatigue and rutting tests performed on asphalt concrete specimens have been described in detail elsewhere [20,22]. The fatigue tests were conducted on 3 in. by 3 in. by 20 in. long (76 x 76 x 508 mm) asphalt concrete beams. The asphalt concrete beams were placed on a rubber subgrade having a modulus of subgrade reaction of 116 pci (31.5 MN/m³). A 0.15 sec. duration, 1 Hz, repeated load was applied at the center of the beam until failure. All tests were conducted in an environmental chamber at a temperature of 80°F ± 1°F (26.7°C ± 0.6°C).

A series of fatigue tests was performed on a laboratory prepared B-modified asphalt concrete mix at 4.3, 4.8 and 5.3% asphalt content. The beams were prepared at 100% of the 50 blow Marshall density. Bending modulus of the beam as a function of repetitions to failure and asphalt content is given in Fig. 14 and also Table 7. The relationship between number of repetitions to failure and applied load is given in Fig. 15. Fig. 16 gives the relationship between number of repetitions to failure and tensile strain for the B-modified mix.

Rutting Properties

The rutting properties of the asphalt concrete were evaluated by subjecting 4 in. (102 mm) dia. by 8 in. (203 mm) high, cylindrical asphalt concrete specimens to 100,000 repetitions of loading. For comparison purposes most of the tests were conducted at 95°F (35°C), although some tests were conducted at 80°F (26.7°C). The tests were conducted in an environmental chamber using a confining pressure of 5 psi (34 kN/m²) and a deviator stress of 25 psi (172 kN/m²) [20].

Results of rutting tests performed on specimens prepared from the asphalt concrete used in selected test sections are given in Table 8. cylinders were prepared from the actual concrete mix used in selected lifts of the test sections. The cylinders were made at 100% of the 50% below Marshall density by reheating the asphalt concrete several days after the construction of the test section was completed.

Crushed Stone Base

Physical Properties

The crushed stone used for all unstabilized bases was a granitic gneiss obtained from the Norcross Quarry of Vulcan Materials Co. The physical properties of the granitic gneiss are summarized in Table 9. To minimize segregation three sizes of crushed stone (No. 5, No. 57 and No. 810) were stockpiled and blended together during construction of each crushed stone base. A detailed description of base construction is given in Chapter IV. The gradation of each of the three crushed stone bases is given in Table 10. The "standard" crushed stone base material was prepared by blending together in a pugmill 20% of the No. 5 size stone, 25% of No. 57 stone and 55% of No. 810 stone. The standard gradation stone was used in Test Sections 1, 2, 8, 11 and 12.

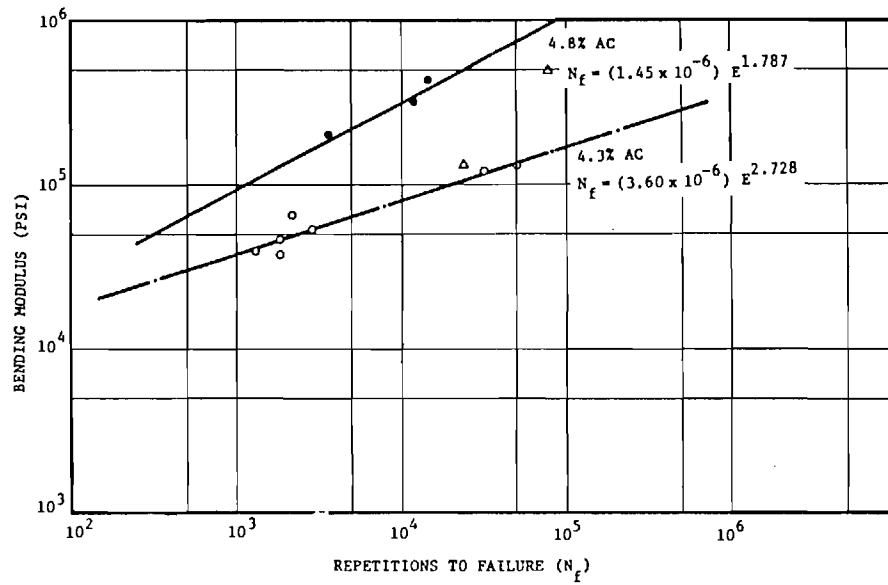


FIGURE 14. B-MODIFIED MIX: REPETITIONS TO FAILURE AS A FUNCTION OF BEAM BENDING MODULUS AND ASPHALT CONTENT.

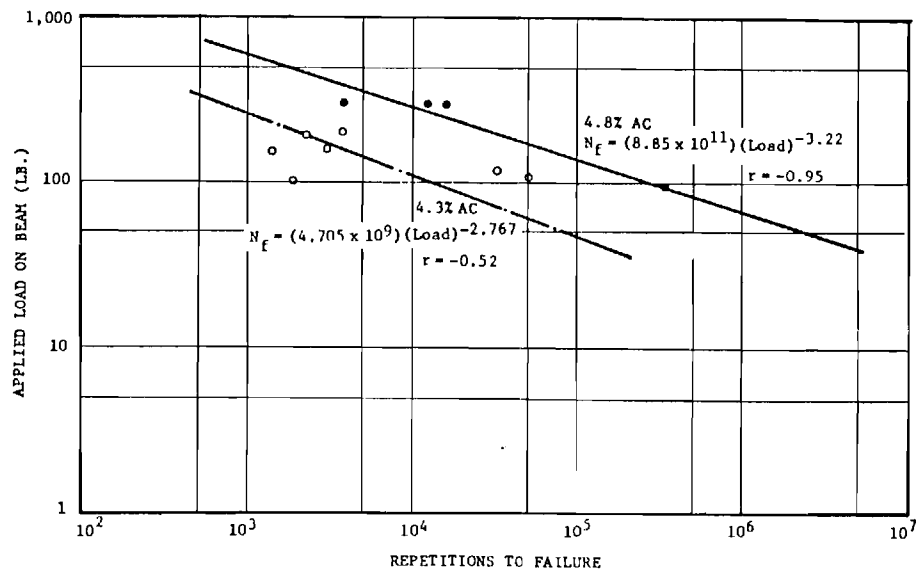


FIGURE 15. B-MODIFIED MIX: REPETITIONS TO FAILURE AS A FUNCTION OF APPLIED LOAD AND ASPHALT CONTENT.

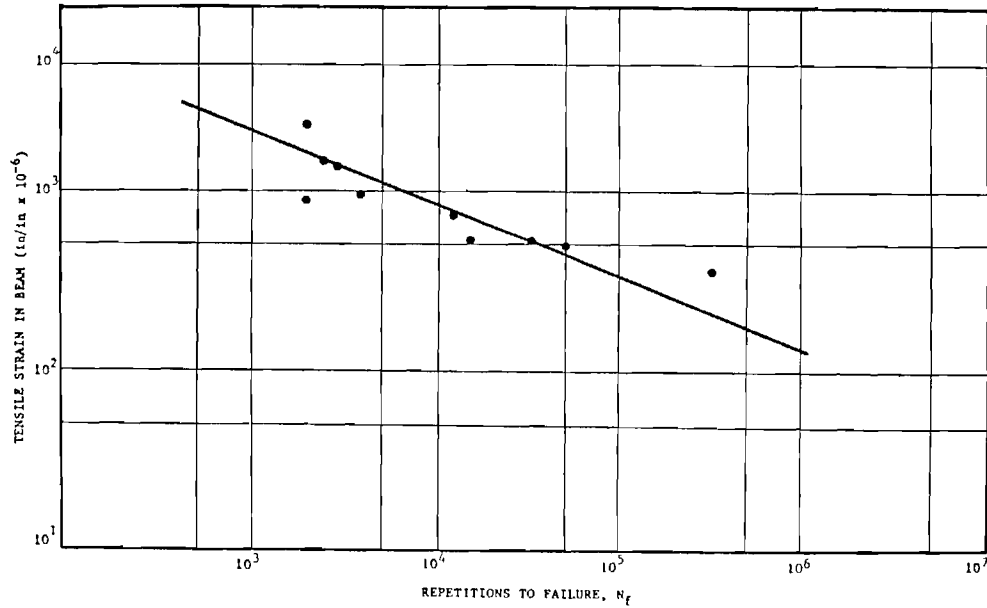


Fig 16: Fatigue Life of Beam as a Function of Tensile Strain (B-Modified Misc.).

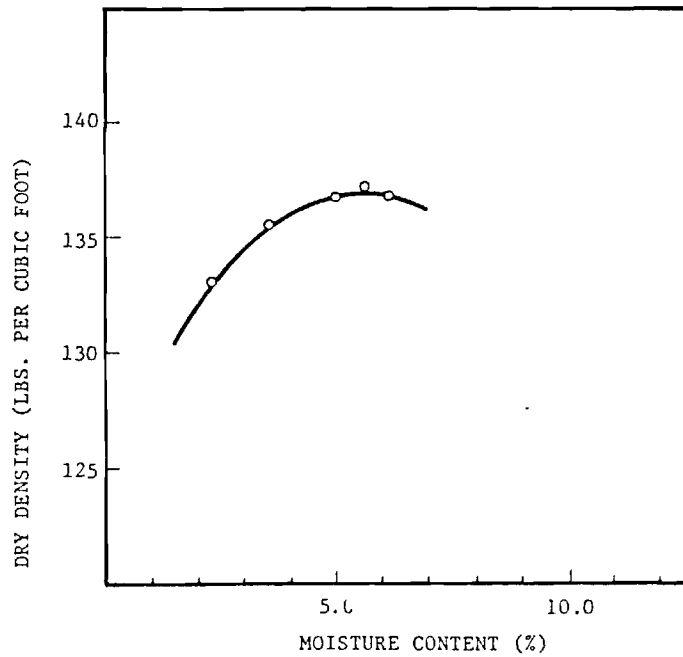


FIGURE 17. MOISTURE-DENSITY CURVE FOR STANDARD BASE COURSE (T-180 COMPACTION).

TABLE 7. BEAM FATIGUE TESTS ON LABORATORY-PREPARED B-MODIFIED MATERIAL.

Specimen No.	Asphalt Content (%)	Load (lb)	Tensile Strain (μ in/in)	Modulus of Elasticity (psi)	Reps. to Failure
MB2	4.8	100	300	208,407	323,000
MB3	4.8	300	500	457,544	16,000
MB4	4.8	300	680	303,656	13,000
MB7	4.8	300	884	214,021	4,000
MB9	4.3	150	1550	40,170	1,450
MB 10	4.3	100	867	50,068	2,000
MB 11	4.3	160	1280	56,508	3,220
MB 12	4.3	110	464	132,657	53,800
MB 13	4.3	200	2110	39,075	2,070
MB 14	4.3	120	511	130,992	33,940
MB 15	4.3	198	1340	70,628	2,400
MB 16	5.3	100	308	508,398	85,000
MB 17	5.3	200	815	138,908	26,744

Table 8. Results of Rutting Tests on Asphalt Concrete Used in Sections 7 and 8

Sample No.	Deviator Stress (psi)	Confining Pressure (psi)	Resilient Strain ($\mu\epsilon$)						Permanent Strain ($\mu\epsilon$)					
			Repetitions						Repetitions					
			1	10	100	1,000	10,000	100,000	1	10	100	1,000	10,000	100,000
4	25	5	870	750	675	600	500	625	5,600*	5,700*	6,000*	6,300*	7,500*	8,300*
5	25	5	500	575	500	375	300	150	250	275	950	1,250	2,200	2,250
7	25	5	425	425	375	325	275	275	750	775	925	1,000	1,250	1,250

* Very high values possibly due to an initial seating deflection rather than load.

TABLE 9. TYPICAL PROPERTIES OF NORCROSS GRANITE-GNEISS
CRUSHED STONE.

Aggregate Description	Granitic Gneiss
Georgia DOT Class Aggregate	B
Bulk Specific Gravity	2.73
Bulk Specific Gravity (SSD)	2.74
Apparent Specific Gravity	2.77
Sand Equivalent	85
Absorption (%)	0.61
L. A. Abrasion(%)	50
Percent Sand Loss	0.6
Mag. Sulfate Soundness Loss (%)	1.0

TABLE 10. GRADATIONS OF NORCROSS MATERIALS.

Sieve	Cumulative % Passing, By Weight										
	1-1/2"	1"	3/4"	1/2"	3/8"	4	8	10	30	60	200
No. 5	100	96	37	5	2						
No. 57	100	98	82	43	20	3	2				
No. 810					100	77	59	56	35	19	8

Test Section 10 used a fine gradation base course approximating the Villa Rica gradation. This gradation was obtained by using 45% of the No. 57 stone and 55% of No. 810 stone. Section 9 had a coarse gradation base that approximately followed the 2 in. (51 mm) Fuller gradation curve. The coarse gradation base was obtained by blending 20% of the No. 5 stone, 12% of the No. 57 stone and 52% of the No. 810 stone. In addition the following non-standard stone sizes were required to obtain the coarse gradation: (1) 2% of stone passing the 2 in. (51 mm) sieve and retained on the 1.5 in. (38 mm) sieve, and (2) 14% of stone passing the 1.5 in. (38 mm) sieve and retained on the 1 in. (25 mm) sieve. The crushed stone gradations used are given in Table 11 together with the Villa Rica and 2 in. (51 mm) Fuller gradation curves for comparison.

The AASHTO T-180 maximum dry density and corresponding optimum moisture content are summarized in Table 12 for the three gradations tested. Fig. 17 gives the moisture density curve for the standard gradation base material.

Rutting and Resilient Modulus

The repeated load triaxial test consisted of subjecting a 6 in. (152 mm) dia. by 14 in. (356 mm) high, cylindrical specimen of crushed stone to 100,000 repetitions of a deviator stress. During the test the specimen was placed inside a triaxial cell and subjected to a constant confining pressure. The test method and specimen preparation for this test have been described in detail elsewhere [19].

The resilient moduli of the three gradations of crushed stone tested are shown in Figs. 18 through 20 as a function of bulk stress σ_θ ($\sigma_\theta = \sigma_1 + 2\sigma_3$) for the standard, coarse and fine gradations, respectively. For the gradations tested relatively little difference existed between the resilient modulus for the three gradations. The earlier work of Barksdale [19] is in agreement with this finding.

The plastic strain response of the crushed stone having the standard gradation is shown in Figs. 21 through 24. The plastic strain responses of the coarse and fine gradation base materials are given in Figs. 25 and 26, respectively.

Subgrade

The subgrade used beneath all pavement sections was a micaceous silty sand. This soil was obtained from I-575 between Stations 208+00 and 220+00, near North Booth Road, Cobb County, Georgia. The basic physical properties of the silty sand averaged for seven tests are given in Table 13. The silty sand subgrade classified as a III-B embankment material following the Georgia DOT Classification System. The average volume change was 31.3% and the clay content was 20%. The average maximum dry density from seven tests was 105 pcf (16.5 kN/m³) and optimum water content was 18.5%.

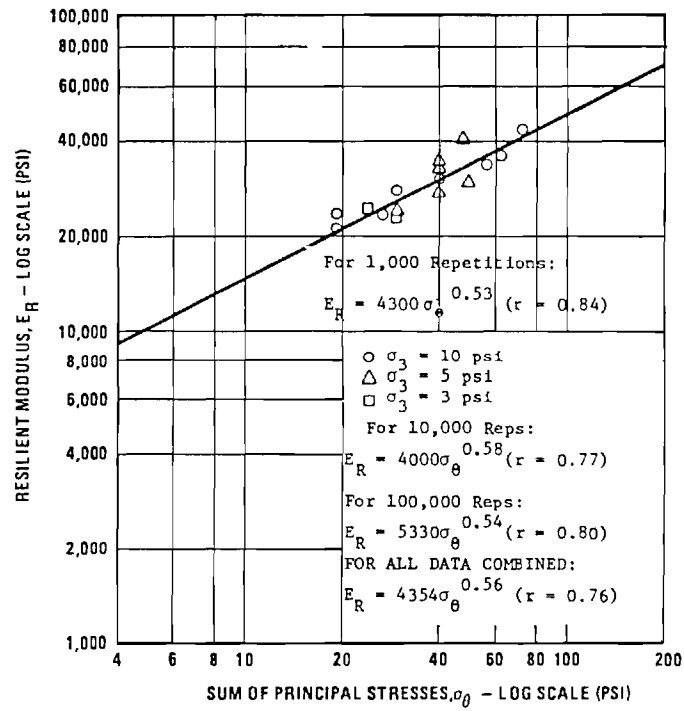


FIG. 18, 19, 20: INFLUENCE OF STRESS STATE ON RESILIENT MOODULUS

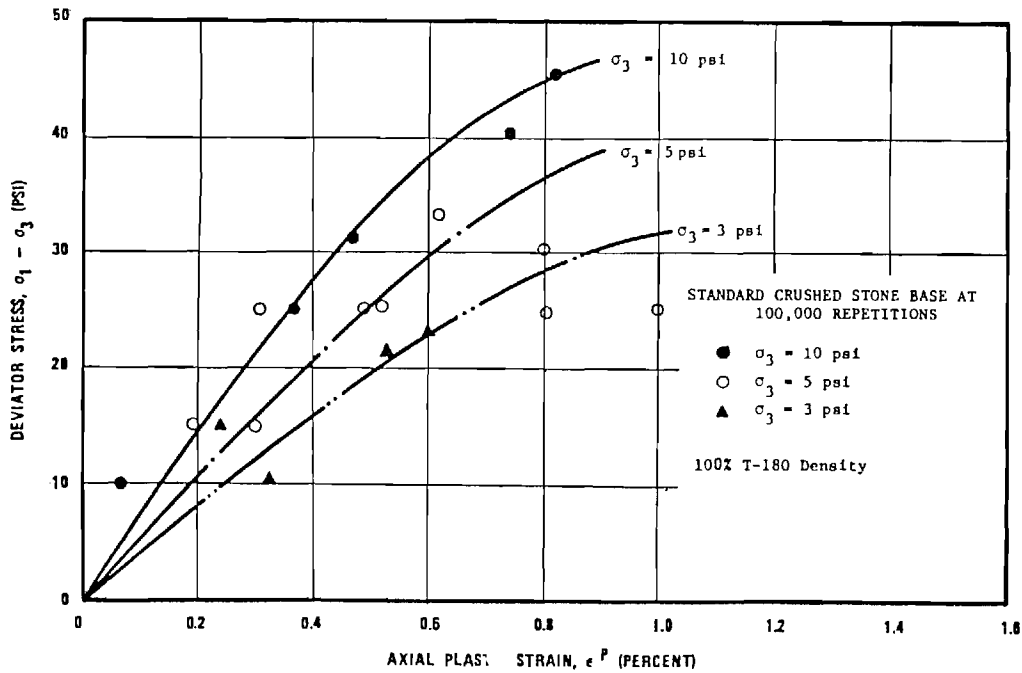


FIGURE 21. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION CRUSHED STONE BASE.

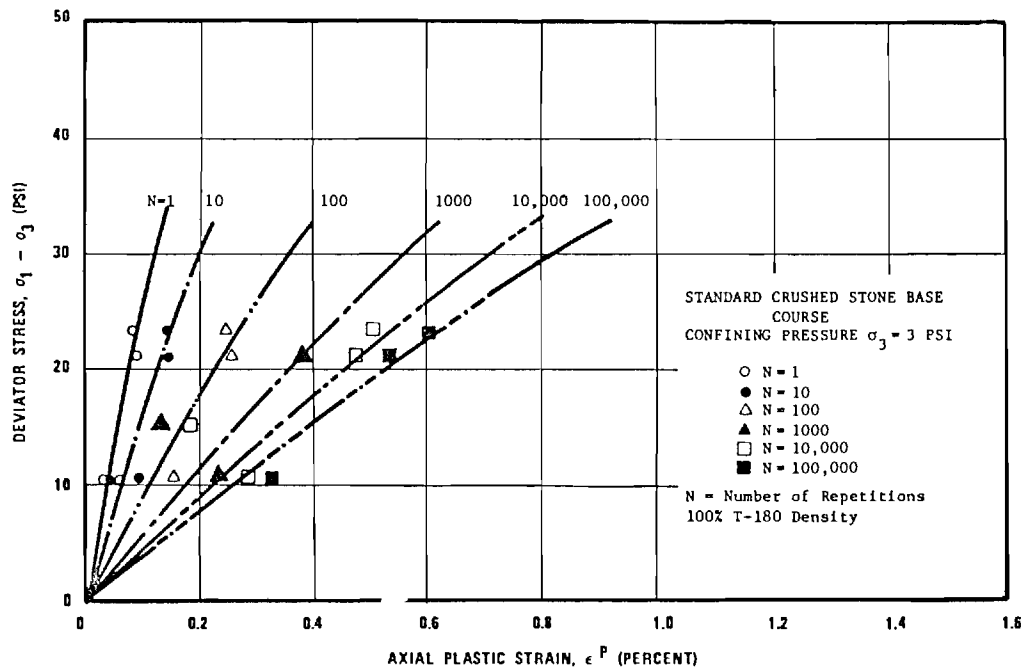


FIGURE 22. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION BASE COURSE.

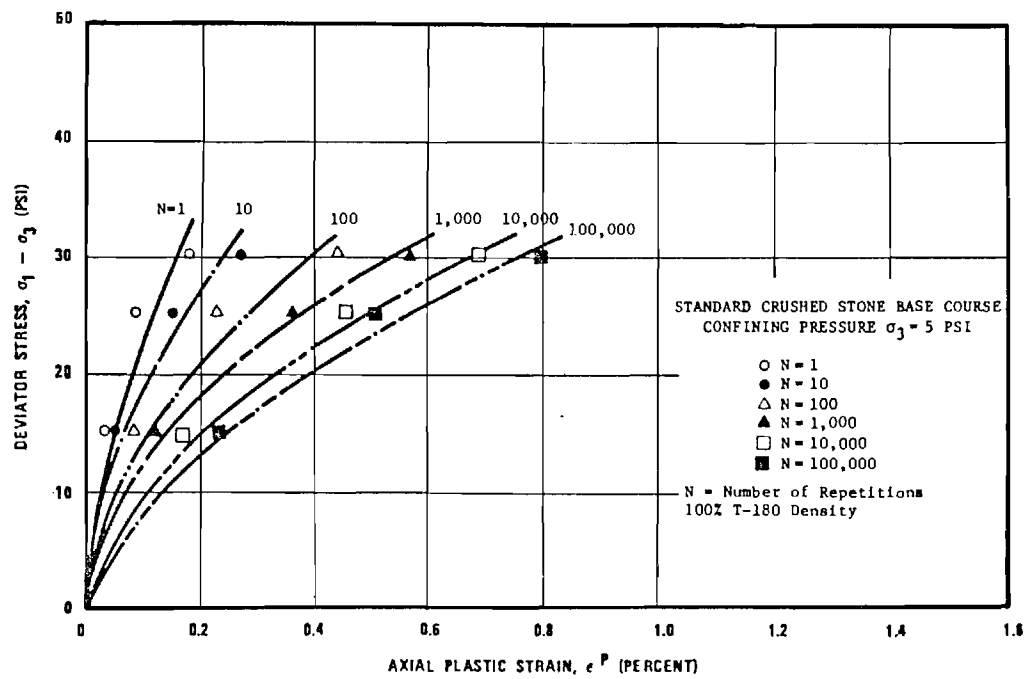


FIGURE 23. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION CRUSHED STONE BASE.

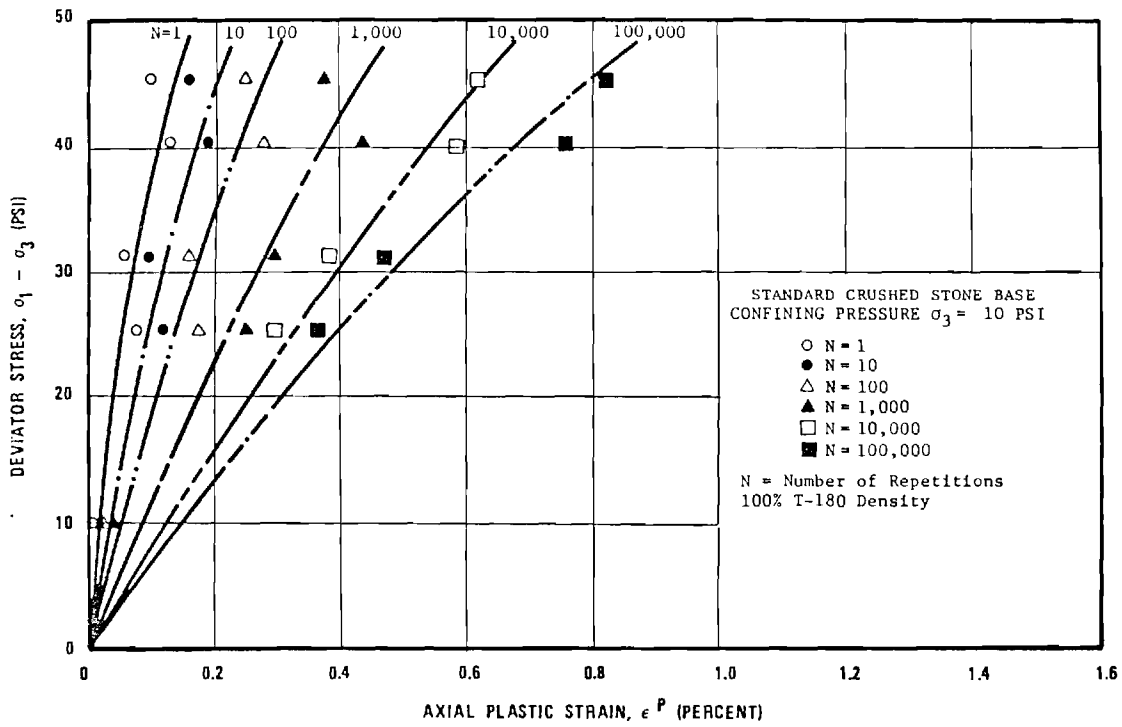


FIGURE 24 AND 25. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION CRUSHED STONE BASE.

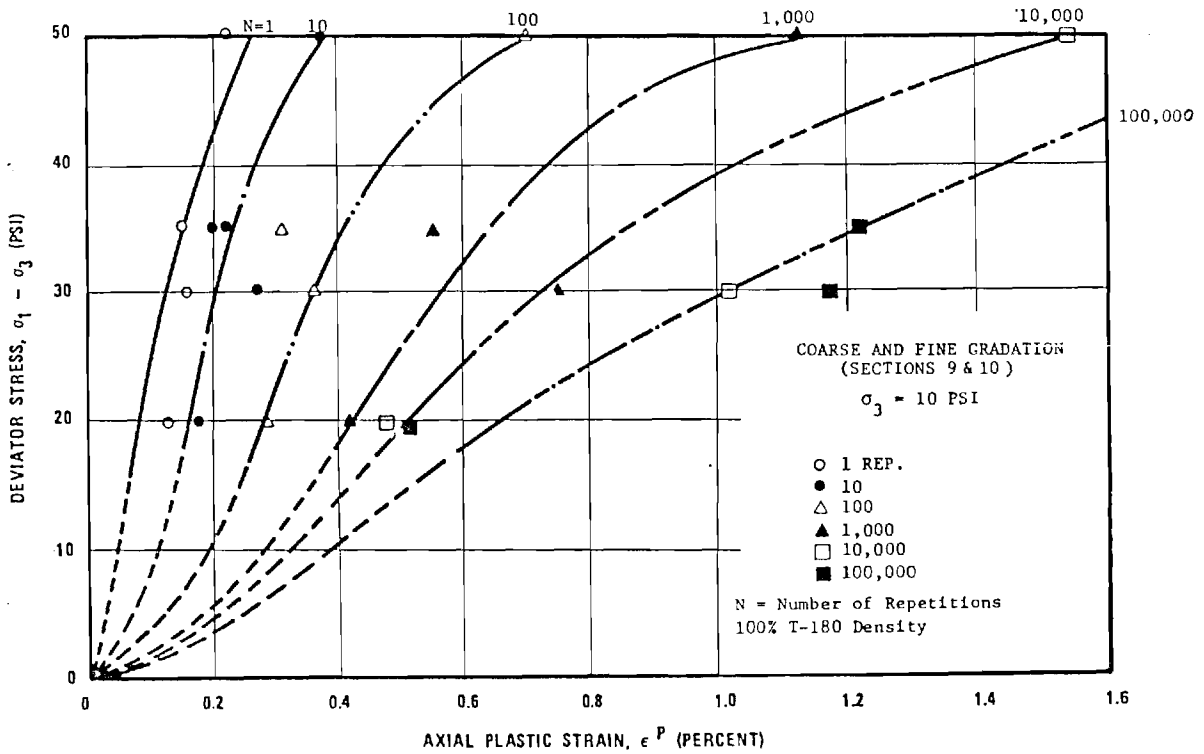


FIGURE 26. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN COARSE AND FINE GRADATION CRUSHED STONE BASE.

TABLE 11. CRUSHED STONE BASE COURSE GRADATIONS.

Sieve	Cumulative % Passing, By Weight										
	1-1/2"	1"	3/4"	3/8"	4	8	10	50	60	100	200
Standard	100	98	83	61	43		31	13			4
"Fine"	100	99	92	64	44		31		11		4
"Coarse"	98	83	69		40	31	29		10		4
Villa Rica	100		92				30		12		5
2" Fuller	98	82	69		39	31	29		12		4

TABLE 12. MAXIMUM DRY DENSITIES AND OPTIMUM MOISTURE CONTENTS OF BASE COURSE.

Material Gradation	Maximum Dry Density		Optimum Moisture Content (%)
	(pcf)	(kg/m ³)	
Standard	137.0	2,217	5.7
Fine	139.5	2,257	6.8
Coarse	141.5	2,289	6.8

TABLE 13. MICACEOUS SILTY SAND SUBGRADE PROPERTIES.

Sieve Size	Cumulative % Passing, By Weight
1-1/2"	100
# 10	99
# 40	85
# 60	70
# 200	39
Total Clay: 20%	
Liquid Limit: S.I.C.	
Plasticity Index: N.P.	
Volume Change: Swell 30.3%	
Shrinkage 1.0%	
Total 31.3	
T-99 Proctor Test Results	
Maximum Dry Density: 105 pcf	
Optimum Moisture Content: 18.5%	
Note: Average of seven sets of tests	

Dynamic Response

The dynamic properties of the remoulded subgrade soil were evaluated using the repeated load triaxial test. A detailed description of sample preparation and testing is given elsewhere [21]. Repeated load triaxial tests were performed on the silty sand subgrade for densities varying from about 93 to 95% of AASHTO T-99, and at moisture contents from about 1% below to about 2% above optimum.

Typical test results showing the resilient modulus as a function of deviator stress, moisture content and confining pressure are given in Figs. 27 through 29. Generally the resilient modulus was between 700 and 1100 psi (4820 - 7580 kN/m²) which indicates a quite resilient subgrade. The resilient modulus was found to be essentially independent of moisture content for moisture contents from about 17.4 to 20.3%, and only slightly effected by deviator stress for values greater than about 2 psi (13.8 kN/m²).

The plastic strain response of the silty sand is given in Figs. 30 through 32. Approximately 80% of the plastic strain occurred after only 100 load repetitions. Further, an increase in moisture content from optimum to 2% above optimum caused a significant increase in permanent deformation (Fig. 31).

Cement Stabilized Crushed Stone Subbase

Sections 11 and 12 were inverted sections having a cement stabilized subbase and crushed stone base (Table 1). Section 11 was constructed with a soil-cement subbase. To construct this layer 5% of Type I Portland cement was added to the silty sand used as a subgrade beneath all the sections. The design for the soil cement subbase by the Office of Materials and Research gave a maximum dry density of 107 pcf (16.8 kN/m³) and an optimum moisture content of 18%. Standard Proctor specimens prepared during construction of the subbase had an average unconfined compressive strength of 214 psi (1474 kN/m²). The seven day unconfined compressive strength was 71% of the 28 day value. The unconfined stress-strain curves for the soil cement subbase are given in Fig. 33.

The cement stabilized subbase used in Section 12 was constructed by adding 4.5% of Type I Portland cement to the standard gradation crushed stone base material. The cement stabilized stone was placed at a water content of 6% at a density of about 138 pcf (21.7 kN/m³). The 28 day average unconfined compressive strength of 6 in. (152 mm) dia. by 12 in. (305 mm) high cylindrical specimens stored in a moisture room was 1146 psi (7896 kN/m²); at 7 days the unconfined compressive strength was 63% of the 28 day value. The unconfined stress-strain curves are given in Fig. 34 for the cement stabilized crushed stone subbase material.

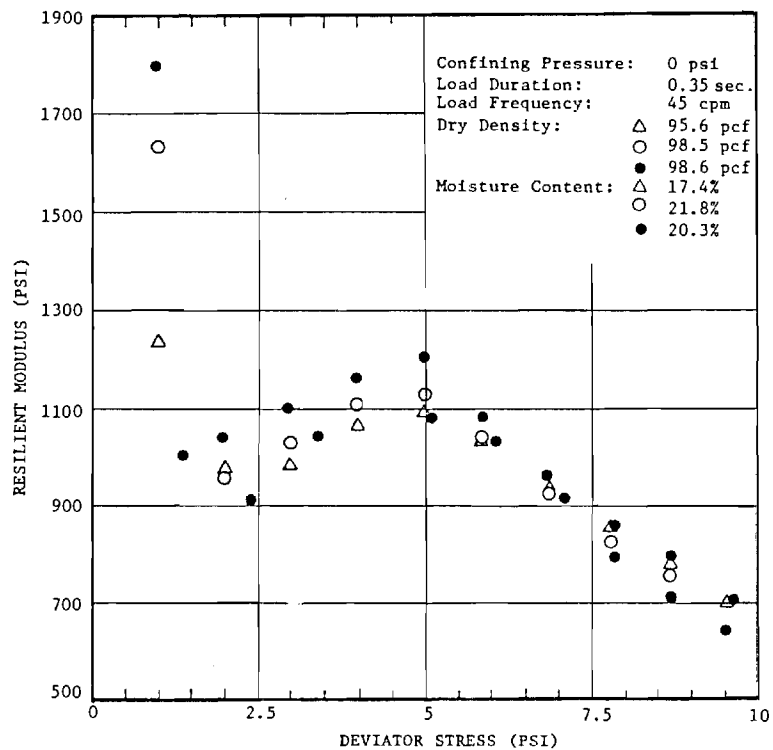


FIGURE 27. INFLUENCE OF DEVIATOR STRESS AND MOISTURE CONTENT ON RESILIENT MODULUS OF MICACEOUS SILTY SAND (CONFINING PRESSURE = 0 PSI).

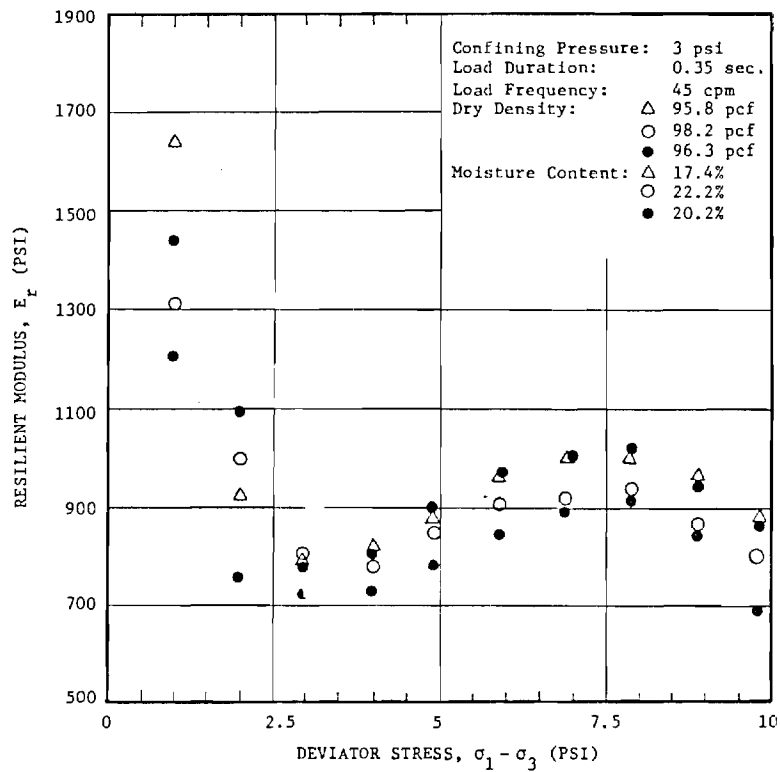


FIGURE 28. INFLUENCE OF DEVIATOR STRESS AND MOISTURE CONTENT ON RESILIENT MODULUS OF MICACEOUS SILTY SAND (CONFINING PRESSURE = 3 PSI).

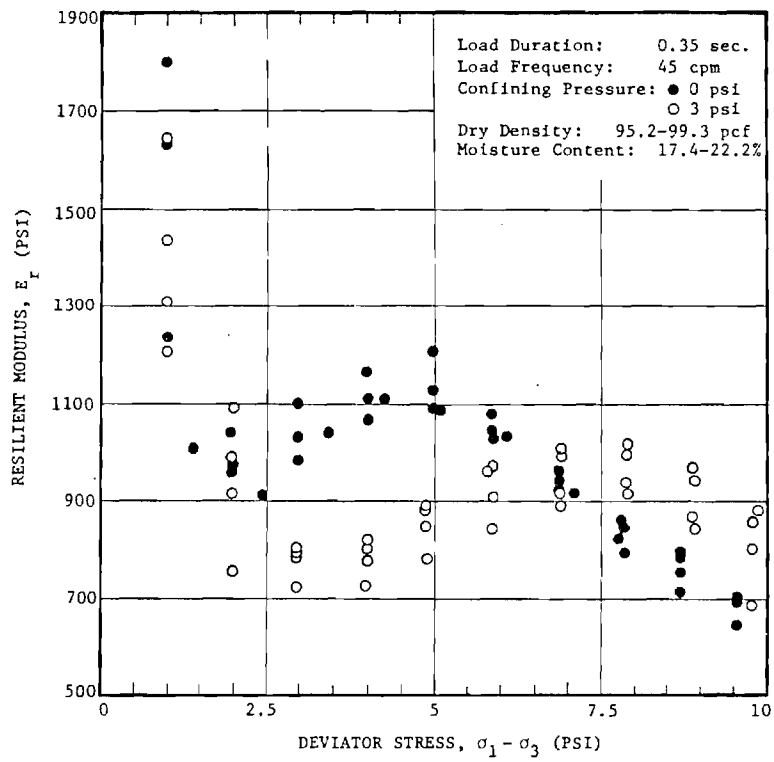


FIGURE 29. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON RESILIENT MODULUS FOR MICACEOUS SILTY SAND - SUMMARY OF ALL RESULTS.

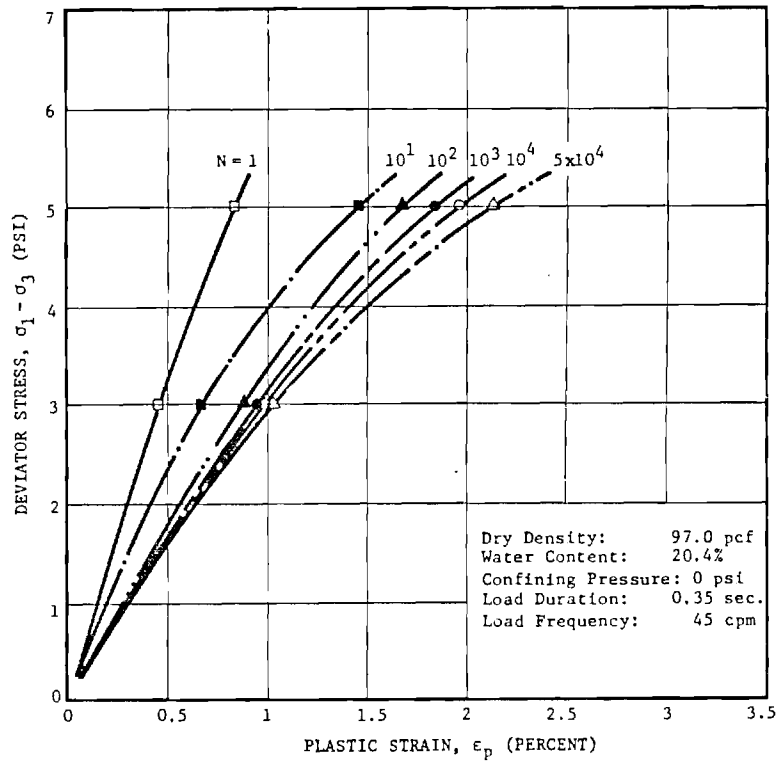


FIGURE 30. INFLUENCE OF LOAD CYCLES AND DEVIATOR STRESS ON PLASTIC STRAIN IN MICACEOUS SILTY SAND.

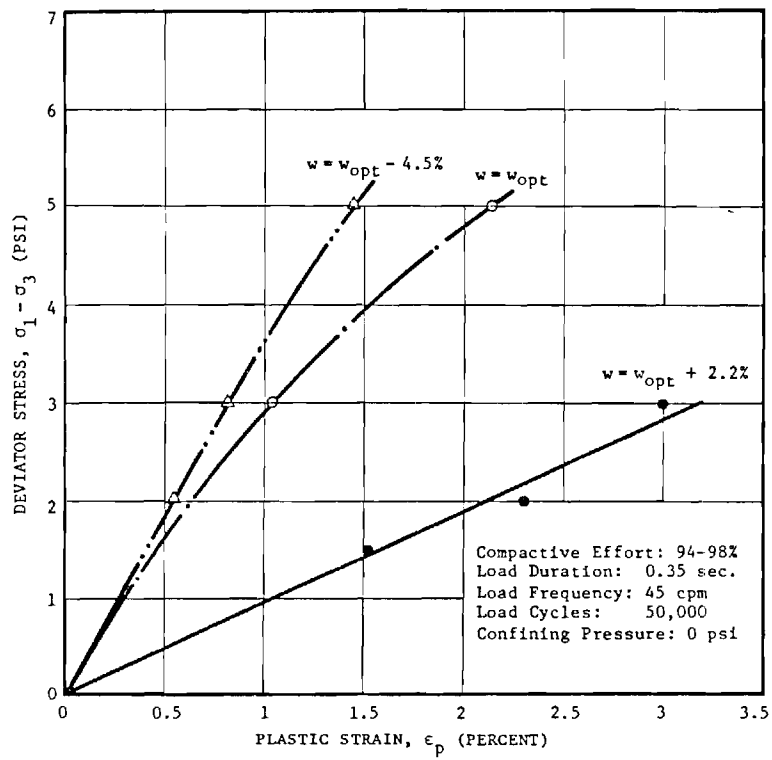


FIGURE 31. INFLUENCE OF DEVIATOR STRESS AND WATER CONTENT ON PLASTIC STRAIN FOR MICACEOUS SILTY SAND - 50,000 LOAD CYCLES.

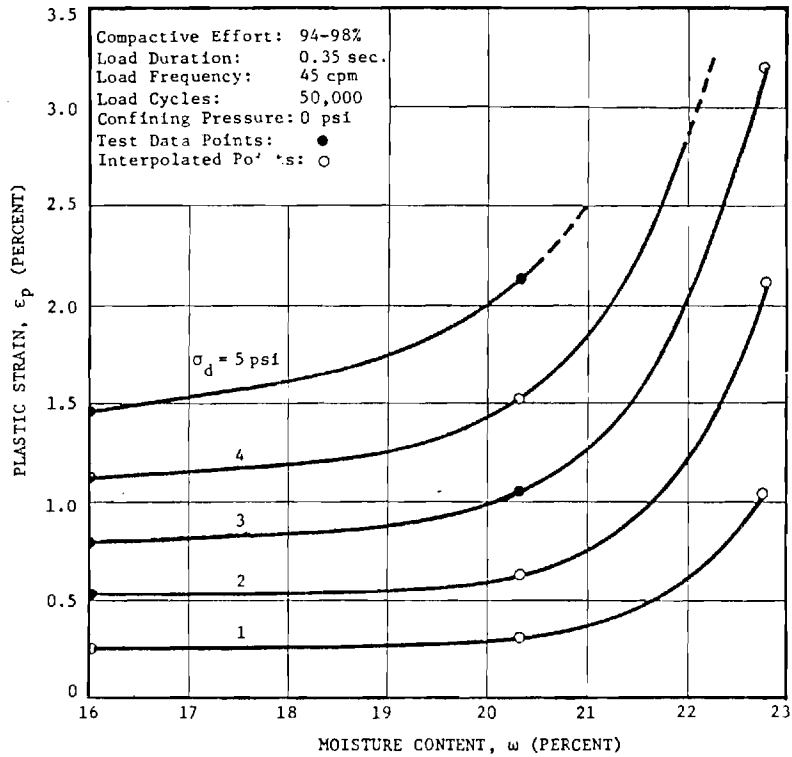


FIGURE 32. INFLUENCE OF WATER CONTENT AND DEVIATOR STRESS ON PLASTIC STRAIN FOR MICACEOUS SILTY SAND - 50,000 LOAD CYCLES.

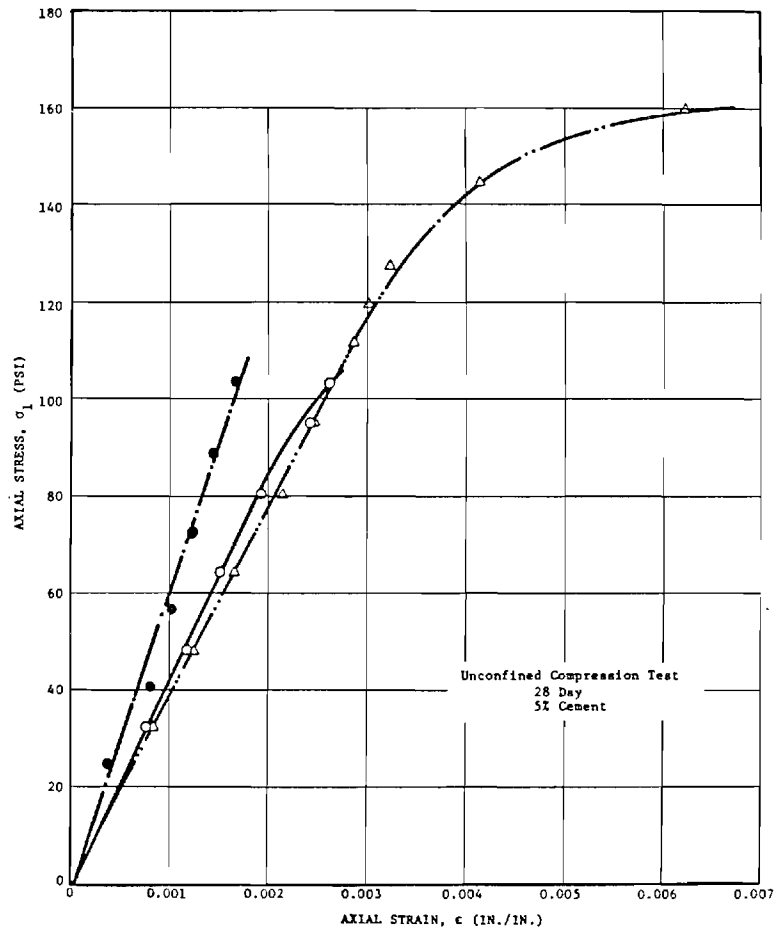


FIGURE 33. UNCONFINED COMPRESSION STRESS-STRAIN RESPONSE OF CEMENT TREATED SILTY SAND SUBBASE.

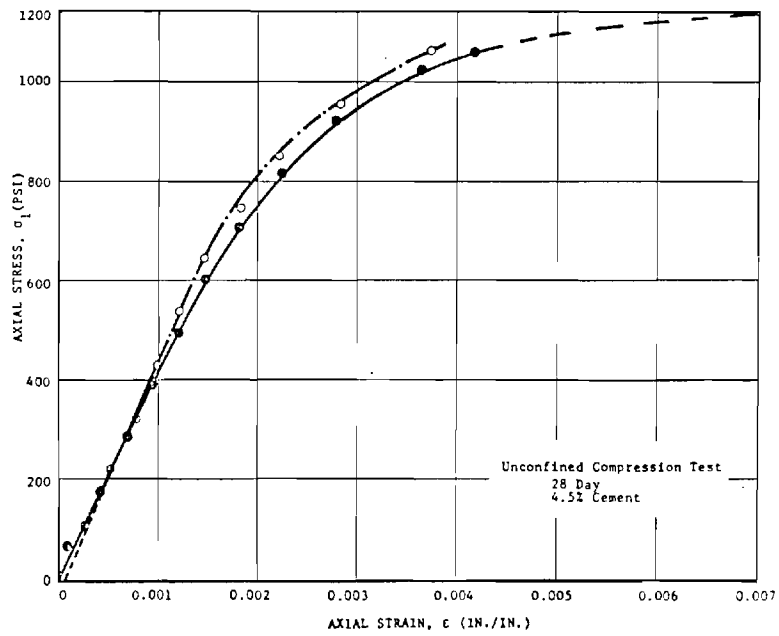


FIGURE 34. UNCONFINED COMPRESSION STRESS-STRAIN RESPONSE OF CEMENT STABILIZED CRUSHED STONE SUBBASE.

CHAPTER IV TEST SECTION CONSTRUCTION, INSTRUMENTATION AND LOADING

Introduction

A total of twelve test sections (Table 1) were constructed and tested to failure (or near failure) in the test facility. All tests were performed in an environmentally controlled room at a temperature varying from 78 to 80°F (25.5–26.7°C). The pit in which the tests were performed is 8 ft. (2.4 m) wide, 12 ft. (3.7 m) long and 5 ft. (1.5 m) deep. To study a maximum number of base variables a different structural section was constructed in each end of the pit to give two tests for each complete pit filling. Emphasis was placed during construction of the test sections on achieving uniform material properties and meeting Georgia Department of Transportation material specifications.

A modified B binder was used for Sections 1 through 8, and a B binder for Sections 9 through 12. All asphalt concrete was obtained from APAC Georgia Inc.-McDougald Warren Division. The modified B binder used in Sections 1 through 8 was from the Lithia Springs Plant. The B binder used in Sections 9 and 10 was from Forest Park, and in 11 and 12 from Lithia Springs. The change in type of mix and plant became necessary because the Georgia Department of Transportation shifted from the use of Modified B to B binder during the course of the project. Also, during the latter part of the project the Lithia Springs plant had a low volume of asphalt production.

The unstabilized crushed stone base course was constructed by blending different sizes of crushed stone together in a pugmill to minimize segregation. The crushed stone base course used in sections 1, 2, 8, 9 and 10 was compacted to 100% of AASHTO T-180 density. A cement treated soil subgrade was used in Section 11 and a cement stabilized subbase in Section 12. Because of the presence of the rigid working platform in these sections, a base density of 105% of AASHTO T-180 was obtained using the same compaction effort as used in previous sections.

The basic dynamic loading applied to the pavement was 6,500 lbs. (28.9 kN) applied uniformly over a circular area having a diameter of approximately 9.1 in. (231 mm). To decrease the time required to fail the pavements, the loading was increased to 7,500 lbs. (33.4 kN) after 2×10^6 load repetitions except in Sections 11 and 12 for which the magnitude of load was not increased. To prevent a localized, punching type failure from occurring during the test, the repeated loading was applied at a primary load position and six secondary positions located around the edge of the primary position.

Test Facility

General Test Facility

The tests were conducted in a special test facility located in the Geotechnical/Materials Laboratory, School of Civil Engineering. The test facility is shown in Fig. 35 including test pit, reaction frame and loading system. The test sections were constructed in a pit 8 ft. (2.4 m)

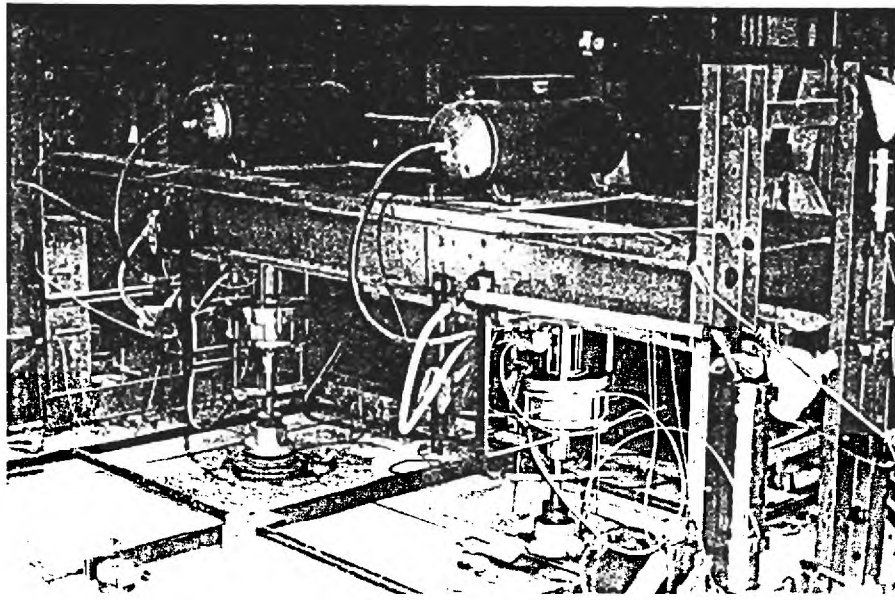


FIGURE 35. GENERAL VIEW OF FACILITY.

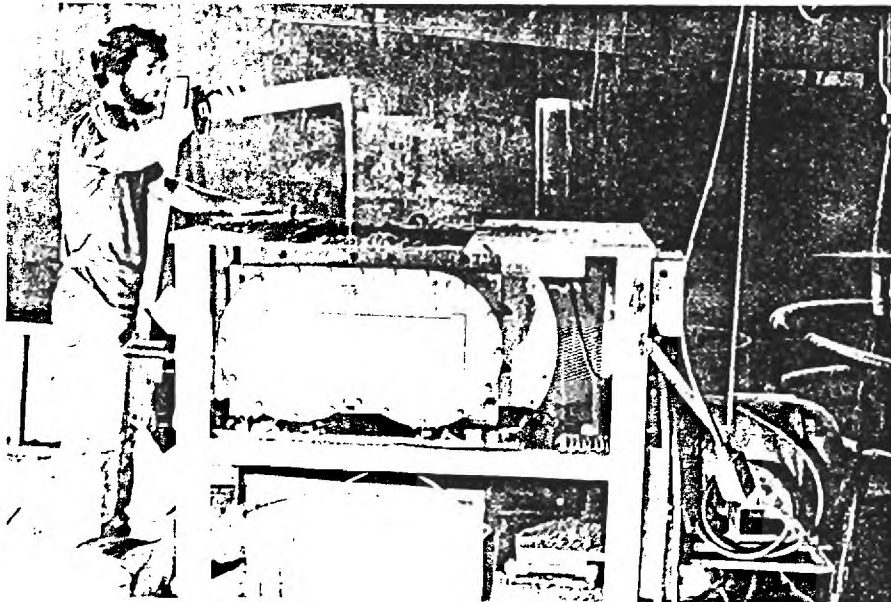


FIGURE 36. BARBER-GREENE PUGMILL USED TO MIX SOIL AND CRUSHED STONE.

by 12 ft. (3.6 m) in plan and 5 ft. (1.5 m) in depth. The top of the test pit is about 6 in. (152 mm) above the surrounding floor.

A constant temperature of 78 to 80°F (25.5–26.7°C) was maintained during the tests using a control system. The temperature control system consisted of a thermostat which controlled a large heater, and two air conditioning units. To prevent excessive heat loss or infiltration, the room was sealed and insulated. This system was capable of maintaining the required temperature in the test facility except when the outside air temperature was above 95°F (35°C), or below about 20°F (7°C). When these extremes in temperature were exceeded, testing was discontinued. Both air temperature and the temperature at the center of the asphalt concrete were monitored throughout the test by means of thermometers.

The walls and floor of the pit consist of 6 in. (152 mm) thick reinforced concrete. A false concrete bottom was constructed in the pit for this study to reduce the total depth from 8 ft. (2.4 m) to 5 ft. (1.5 m). Using the false bottom significantly decreased the volume of subgrade material required for each filling. Storage bins were constructed along the walls of the room to hold the materials during emptying and filling operations. A small belt conveyor, 1/2 ton crane and forklift truck were used for material handling.

Reaction Frame

A reaction frame extended horizontally across the pit in the long direction about 3 ft. (0.9 m) above the surface of the pavement (Fig. 35). For most load positions used in the load sequence, the center of the load was offset from the centerline of the reaction frame. Because of this eccentricity, a relatively large torsional moment was applied to the reaction frame by the 6,500 lb. (28.9 kN) load. Therefore, two wide flange sections spaced 23.5 in. (597 mm) apart were used to reduce the effects of torsion caused by the eccentric pavement loading.

The reaction frame consisted of two 12 ft. (3.7 m) long wide flange beams (10 x 8 x 33 lbs/ft.) stiffened by a channel cover plate (7 x 2-1/2 x 9.8 lbs/ft.) welded to the top of each beam. The horizontal reaction frame was supported on each end by two upright channels (9 x 2-1/2 x 13.4 lb/ft.) welded at the bottom to a heavy horizontal angle. The reaction frame was fastened together with 1 in. (25 mm) dia. steel bolts for easy disassembly and removal from the immediate vicinity of the pit during construction of each section. The end support frames were attached to the concrete walls of the pit by 7/8 in. (22 mm) dia. bolts retained in the pit wall by Rawl multi-calk anchors (No. 9135).

The loading system was attached to the load frame by means of a 1 in. (25 mm) thick, horizontally oriented thrust plate 26 in. by 33 in. (660 x 838 mm) in size. The horizontal thrust plate was attached using 1 in. (25 mm) dia. bolts to the bottom of the wide flange beams. Three 1/2 in. by 3 in. (13 x 76 mm) steel stiffeners 18 in. (454 mm) in length were welded to the top of the plate. Rapid positioning of the loading system at the seven fixed load locations was achieved using seven sets of bolt holes in the thrust plate made to support the load system. One load system and thrust plate was used on each end of the pit.

Test Section Construction

Introduction

All test sections were constructed using the same standardized procedures which were found to give consistent, reproducible results. After testing to failure, each section was completely removed from the pit and new sections were constructed from the bottom of the subgrade up. Only the silty sand subgrade soil was reused; after each test the subgrade soil was removed from the pit, stored, remixed and then placed and recompactd in the pit. Each filling of the pit required the movement of about 30 tons of material. Because of the small size of the pit, a significant amount of the material movement was accomplished by hand.

A general summary of the test sections constructed is given in Table 1. A detailed description of the materials used in the test sections is given in Chapter III.

Subgrade

To achieve uniform thickness and density of the soil, the pit was partitioned into six segments, and the required amount of soil added to each segment to give a compacted thickness of 2.0 in. (51 mm). The subgrade soil was placed at the optimum moisture content of 20.5% and compacted to 102.7 pcf (16.1 kN/m³) corresponding to 95% of the AASHTO T-99 density. The silty sand subgrade was brought in from stockpiles and temporarily stored in plywood bins constructed along the inside walls of the test facility. The existing water content of each batch of soil was measured using a Speedy Moisture Meter⁽¹⁾. The required amount of soil was then weighed out by placing it in a specially constructed, 10 ft.³ (0.3 m³) bottom dump bucket placed on a 1,000 lb. (4.5 kN platform scale). A 1/2 ton (4.5 kN) overhead crane was used to move the bucket.

The soil was mixed in 150 lb., 0.67 kN batches in a small 1/8 yd³ (0.1 m³) Barber-Green Pugmill (Fig. 36). During mixing the required quantity of water was added to bring the moisture content up to optimum. After mixing for about 1-1/2 to 2 min., the soil was discharged from the bottom of the pugmill into a wheelbarrow and dumped in one of the partitioned areas in the pit.

Twelve batches of soil were required to construct each 2 in. (51 mm) thick lift. After placing, the soil was leveled and the partitions removed. Compaction was achieved in 5 passes of a Jay 12 vibratory waker compactor (Fig. 37). A pneumatic tamper having a square foot was also used to obtain compaction in the corners and along the sides of the pit. Before placing each new lift, the surface was scarified with a rake to give good bond between lifts.

Density Control. Immediately after compaction of each lift, a spring loaded, static penetrometer was used to carefully check the lift for proper density; if necessary soft areas were recompactd before the next lift was constructed. The density of the compacted subgrade was determined

1. A calibration was used between the Speedy Moisture Meter readings and the 104°C (219°F) oven moisture content.

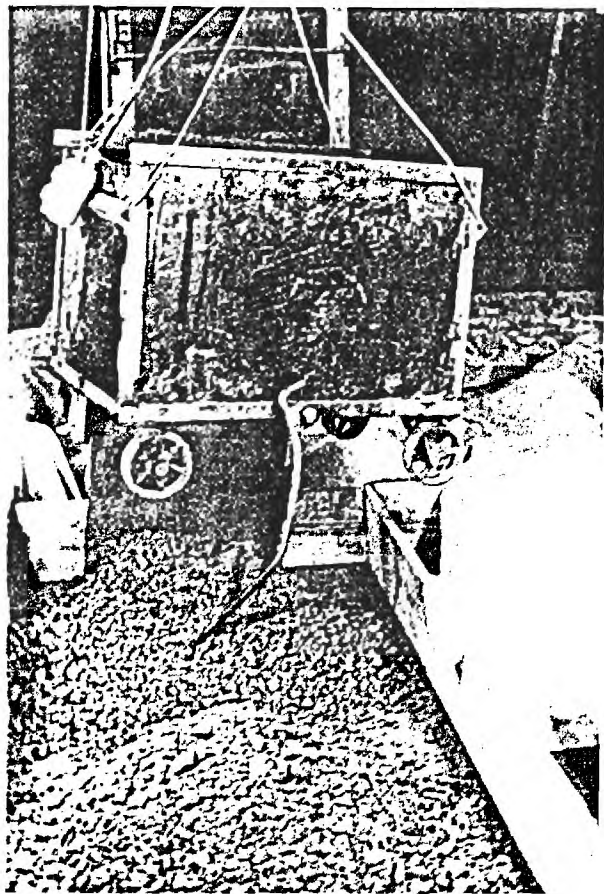


FIGURE 38.

BOTTOM DUMP BUCKET USED TO PLACE SOIL, CRUSHED STONE AND ASPHALT CONCRETE.

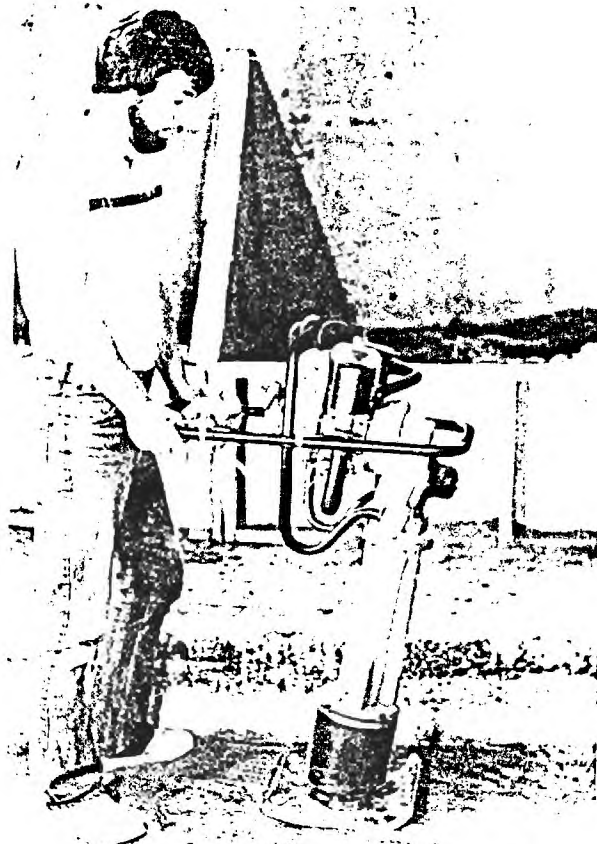


FIGURE 37.

WACKER COMPACTOR USED TO OBTAIN 95% OF T-99 IN SILTY SAND SUBGRADE.

using a thin wall, drive tube sampler. The average subgrade density of all test sections is 98.7 pcf (1597 kg/m³) which is 96.1% of the T-99 maximum dry density. After testing, dynamic cone penetrometer [20] readings were also taken in the subgrade. All sections had a uniform cone penetration resistance approximately equivalent to a standard penetration resistance of 7 to 8 blows per foot.

Protection Against Moisture Loss. Considerable caution was taken during construction to protect the subgrade from moisture loss due to evaporation. The exposed soil was covered with two 4 mil thick polyethylene sheets whenever possible during construction. After completing work each day, the surface was very lightly sprinkled with water. Two sheets of plastic were then placed over the surface and covered with wet burlap.

Crushed Stone Base

As discussed in Chapter 3, the standard crushed stone base was constructed by blending No. 5, No. 57 and No. 810 crushed stone together to give the gradation shown in Table 11. The three sizes of stone were stockpiled outside and brought into the test facility using a forklift truck. The forklift was fitted with a specially constructed tripping bucket attached to the front.

All mixing, handling, and moisture control procedures used for base construction were similar to those previously described for the subgrade. Each crushed stone size was weighed out and placed in the pugmill. The moisture content was adjusted to optimum and the stone blended. The crushed stone was discharged from the pugmill into the bottom dump bucket, and moved to the pit by means of the overhead crane. The stone was bottom-dumped from the bucket into the proper partitioned compartment of the pit (Fig. 38). Use of separate size stone, pugmilling and bottom dumping resulted in a very uniform, homogeneous blend having a minimum amount of segregation after placement.

The crushed stone base was placed in approximately 2 in. (51 mm) lifts. Actual lift thickness depended upon the number of lifts and the total base thickness. Compaction was achieved using 5 to 7 passes of the Jay 12 vibrating plate compactor (Fig. 39).

Density Control. The sand replacement method was used to determine the density of Test Sections 1 and 2. This method caused excessive disturbance of the crushed stone base. The density of all subsequent sections having unstabilized granular bases was therefore determined using a nuclear density gage. The nuclear density tests were performed by an inspector from the Georgia Department of Transportation. The average density of all sections (except 11 and 12) is 100.1% of the AASHTO T-180 maximum dry density of the crushed stone bases. The unstabilized crushed stone bases used in Sections 11 and 12 were constructed over a rigid cement stabilized layer. Apparently as a result, the density obtained in the crushed stone in these bases was significantly greater than the other granular base sections. The density of the inverted sections was 105.0% of the T-180 maximum dry density.

TABLE 14A: CRUSHED STONE BASE COURSE DENSITIES.

Section	Density (% of T-180)
1 & 2	100.4
3 & 4	N/A
5 & 6	N/A
7 & 8	100.0
9 & 10	100.0
11 & 12	105.0



FIGURE 39. JAY-12 VIBRATING PLATE COMPACTOR USED TO OBTAIN 100% OF T-180 DENSITY IN CRUSHED STONE BASE.

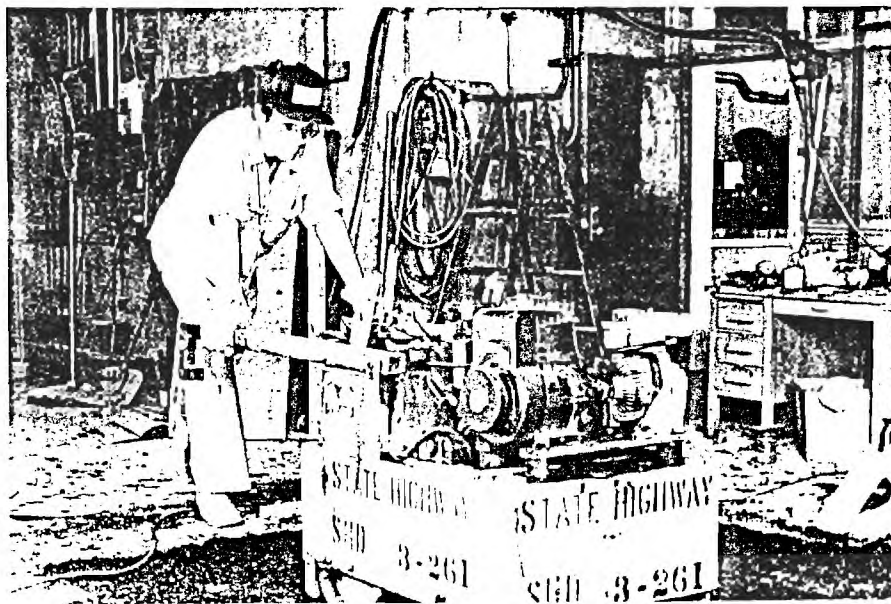


FIGURE 40. DENSIFICATION OF ASPHALT CONCRETE USING MAINTENANCE ROLLER OPERATED DRY.

Priming. The base (or the subgrade when full-depth asphalt concrete sections were used) was primed with a cut-back RC-70. The RC-70 was placed in a pressure vessel and heated to about 150°F (65°C), and then sprayed through a nozzle by applying a 30 psi (205 kN/m²) air pressure to the tank. Using this procedure a thin, uniform coating of RC-70 was obtained over the surface. To protect the surface from dust as the prime coat cured, a wood frame covered with polyethylene was placed over the top of the pit.

Asphalt Concrete

As previously discussed either a Georgia Department of Transportation Modified B or B binder was used for the full thickness of the asphalt concrete layer. The asphalt concrete was transported from the plant to the test facility in an enclosed plywood box. The box had 2 in. (51 mm) of styrofoam insulation to prevent excessive heat loss during transportation. Further, the insulated box was filled with 5,000 lb. (22 kN) of asphalt concrete, which was approximately three times the amount required for the single lift constructed from each load. The additional asphalt concrete mass helped maintain the temperature in the box during transportation and was wasted after each lift. At the time of delivery to the test facility, the temperature of the asphalt was between 290 and 300°F (143 to 149°C).

The pit was temporarily partitioned into six compartments as done during placement of the subgrade and crushed stone. The hot asphalt concrete was quickly shoveled from the box into the bottom dump bucket placed on platform scales. After filling with the correct weight of asphalt concrete, the bucket was rapidly moved to the correct location in the pit and dumped. The asphalt was quickly covered with three to five layers of cardboard insulation to prevent heat loss during placement of the remaining asphalt concrete. As the final compartments were filled, the asphalt concrete was leveled in each previously filled compartment using shovels and lutes. After preliminary leveling, the partitions were removed and the asphalt concrete cut into 3/4 in. (19 mm) wide slots left by the partitions. The asphalt concrete surface was then quickly leveled. Rolling was immediately performed using a small two wheel, vibratory roller used for maintenance by the Georgia Department of Transportation (Fig 40). Rolling was carried out in both the longitudinal and transverse directions of the pit until further rolling produced no noticeable indentation of the surface. Placing and rolling the asphalt concrete took about 20 to 25 minutes using a crew of approximately ten men. Where possible all lifts were placed on the same day; in this case a tack coat was not used between layers. Marshall specimens, beams and cylinders were made for future testing from asphalt concrete saved from each lift (except for sections 1 through 4).

Before loading the test sections, the asphalt concrete was allowed to cure for at least one week. After testing sections 3 and 4 which had bad asphalt concrete, the density of the asphalt concrete in subsequent sections was determined before testing using a nuclear moisture-density gage to insure Georgia DOT density requirements were met. After failure of the test sections, the asphalt concrete was cored and the asphalt content, density and gradation was determined by the Georgia DOT (Table 6).

Cyclic Loading

Loading Sequence and Magnitude

To prevent a localized punching failure from occurring during the test, the repeated loading was applied at a primary load position and six secondary positions located symmetrically around the edge of the primary position (Fig. 41). In all tests load was applied in the ratio of five repetitions at the primary position to each repetition applied at any individual secondary position. The basic pattern was to apply 100,000 repetitions at the primary position and 20,000 repetitions to each secondary position; this number, however, was reduced in the early phases of most tests; in a few tests greater numbers of load repetitions were applied in the latter phases of testing (Table 14).

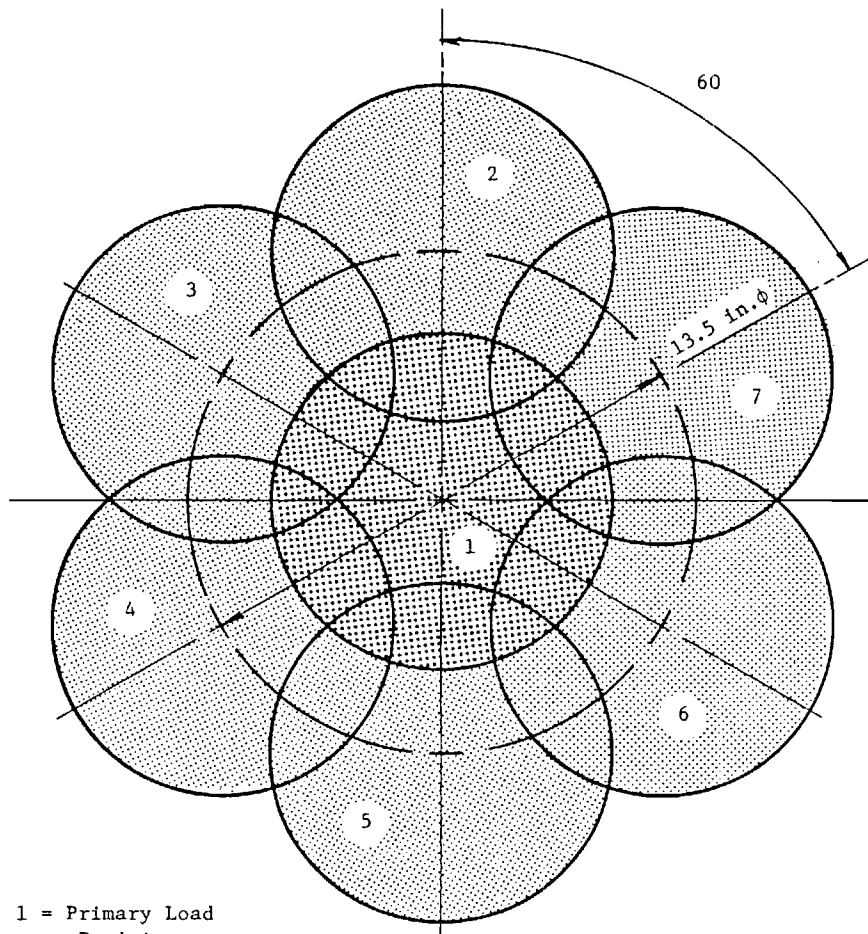
The pavement was subjected to a cyclic loading of 6,500 lbs. (28.9 kN) up to 2×10^6 repetitions. The load was applied over a circular area having a dia. of approximately 9.1 in. (231 mm). By applying the load to a water-filled, rubber bladder a pressure of approximately 100 psi (689 kN/m^2) was applied uniformly to the pavement surface. A load pulse of 0.17 sec. duration was applied throughout the tests at a frequency of 70 to 90 pulses per minute.

To decrease the time required to fail strong pavement sections, the loading was increased to 7,500 lbs. (33.4 kN) after 2×10^6 repetitions. In Sections 11 and 12 the load was maintained at 6,500 lbs. (28.9 kN) throughout the test. In this series, however, 200,000 load repetitions were applied at the primary load position in the latter stages of testing; use of this pattern of loading greatly increased the number of repetitions that could be applied during a given 24 hour day.

Cyclic Loading System

The dynamic loading was applied to the pavement system using a hybrid air-over-oil loading system. A general schematic of the cyclic testing system is shown in Fig. 42, and details of load actuator is shown in Fig. 43. The air-over-oil system was developed to apply to the pavement a large load in a short period of time (0.17 sec.). The applied pulse simulates a slowly moving heavy wheel loading. In the hybrid load system oil is sandwiched between a small 4 in. (102 mm) dia. aluminum, free floating piston on the top and a large 12 in. (305 mm) dia. aluminum piston on the bottom. Air pressure applied to the top of the small piston was transmitted undiminished to the large lower piston giving a large force out. A push rod transmitted this force from the lower piston to the loading bladder which rests on top of the pavement (Fig. 44). To develop the repeated loading, air was cyclically applied to the top of the upper cylinder.

The free-floating aluminum piston was located in the small upper cylinder on top of the oil. Only a small air space was left between the piston and top of the cylinder to minimize response time. The piston was guided in the cylinder by means of two sets of nylon wear rings. The free-floating piston was used in most tests to separate the air and hydraulic oil. The system was also found to work reasonably well without a physical separation between the air and oil, provided a baffle was



1 = Primary Load
Position
2-7 = Secondary Load
Positions

FIGURE 41. LOAD POSITIONS USED IN TEST SECTION LOADING.

TABLE 14. SUMMARY OF TEST SECTION LOADING HISTORY.

Test Section	Sequence 1			Sequence 2			Sequence 3			Sequence 4			Sequence 5		
	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		Position
From	To	From		To	From		To	From		To	From		To		
1	0	10,000	10,000	10,000	3,350,578	100,000	-	-	-	-	-	-	-	-	
2	0	10,000	10,000	10,000	1,891,000	100,000	-	-	-	-	-	-	-	-	
3	0	10,000	10,000	-	-	-	-	-	-	-	-	-	-	-	
4	0	10,000	10,000	-	-	-	-	-	-	-	-	-	-	-	
5-8(3)	0	2,600	1,000	2,600	20,200	2,000	20,200	26,600	4,000	26,600	554,800	20,000	554,000	1,032,000	40,000
9	0	222,000	30,000	222,000	1,117,000	100,000	1,117,000	3,080,000	200,000	-	-	-	-	-	
10	0	101,967	30,000	101,967	1,117,000	100,000	1,117,000	3,080,000	200,000	-	-	-	-	-	
11-12(4)	0	60,000	25,000	60,000	280,000	100,000	280,000	4,000,000	200,000	-	-	-	-	-	

1. Applied load was 6,500 lb. (28.9 kN) unless indicated otherwise.
2. Five repetitions were applied at the primary position for every repetition applied at an individual secondary position. Then one-fifth this number of repetitions was applied to each secondary position; this sequence was thereafter repeated.
3. Tests were terminated after the following number of load repetitions: Section 5 - 1,389,896; Section 6 - 1,228,183; Section 7 - 1,032,000; Section 8 - 1,081,000.
4. Tests were terminated after the following number of load repetitions: Section 11 - 3,200,000; Section 12 - 4,000,000.

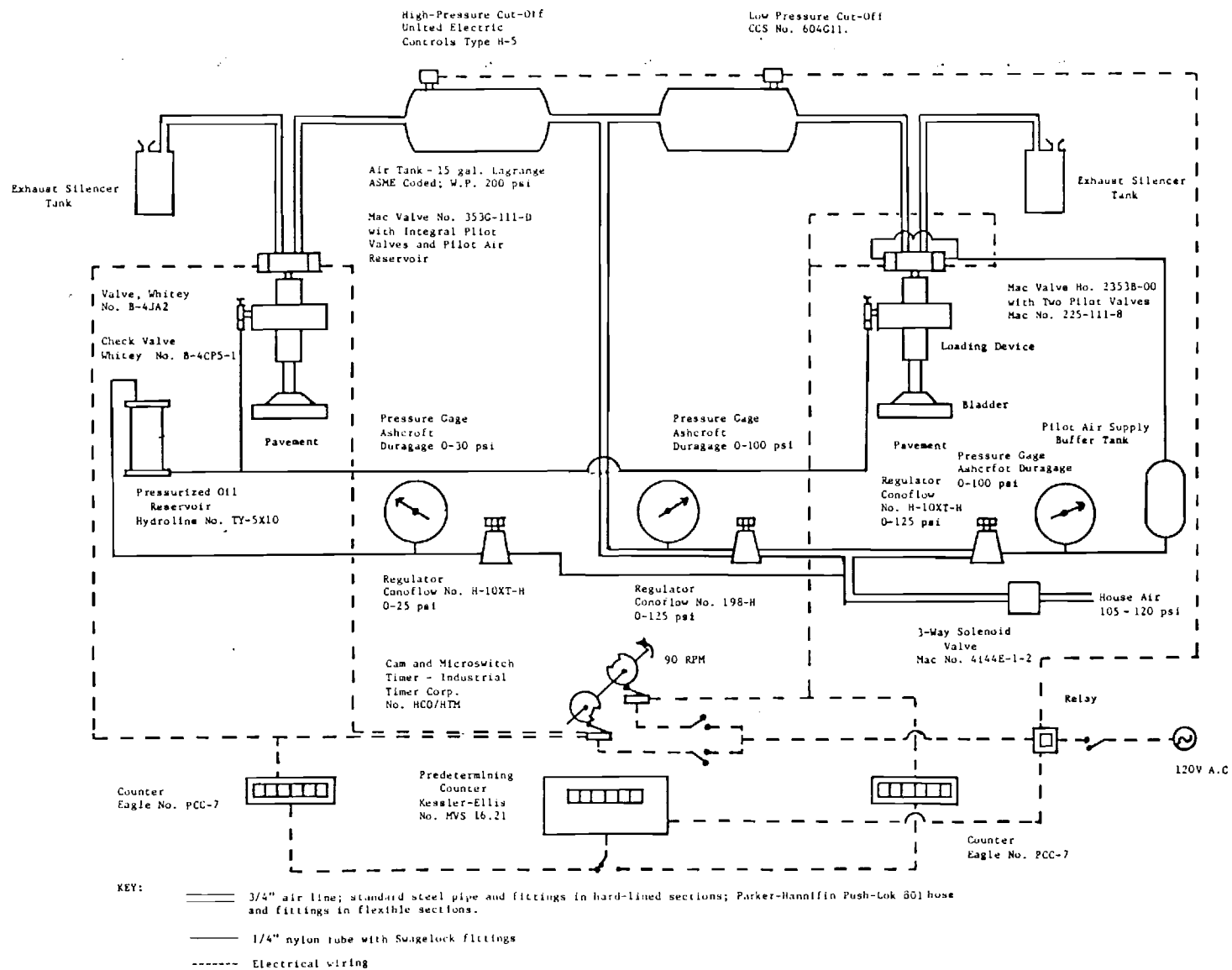


FIGURE 42. GENERAL SCHEMATIC OF DYNAMIC TESTING SYSTEM.

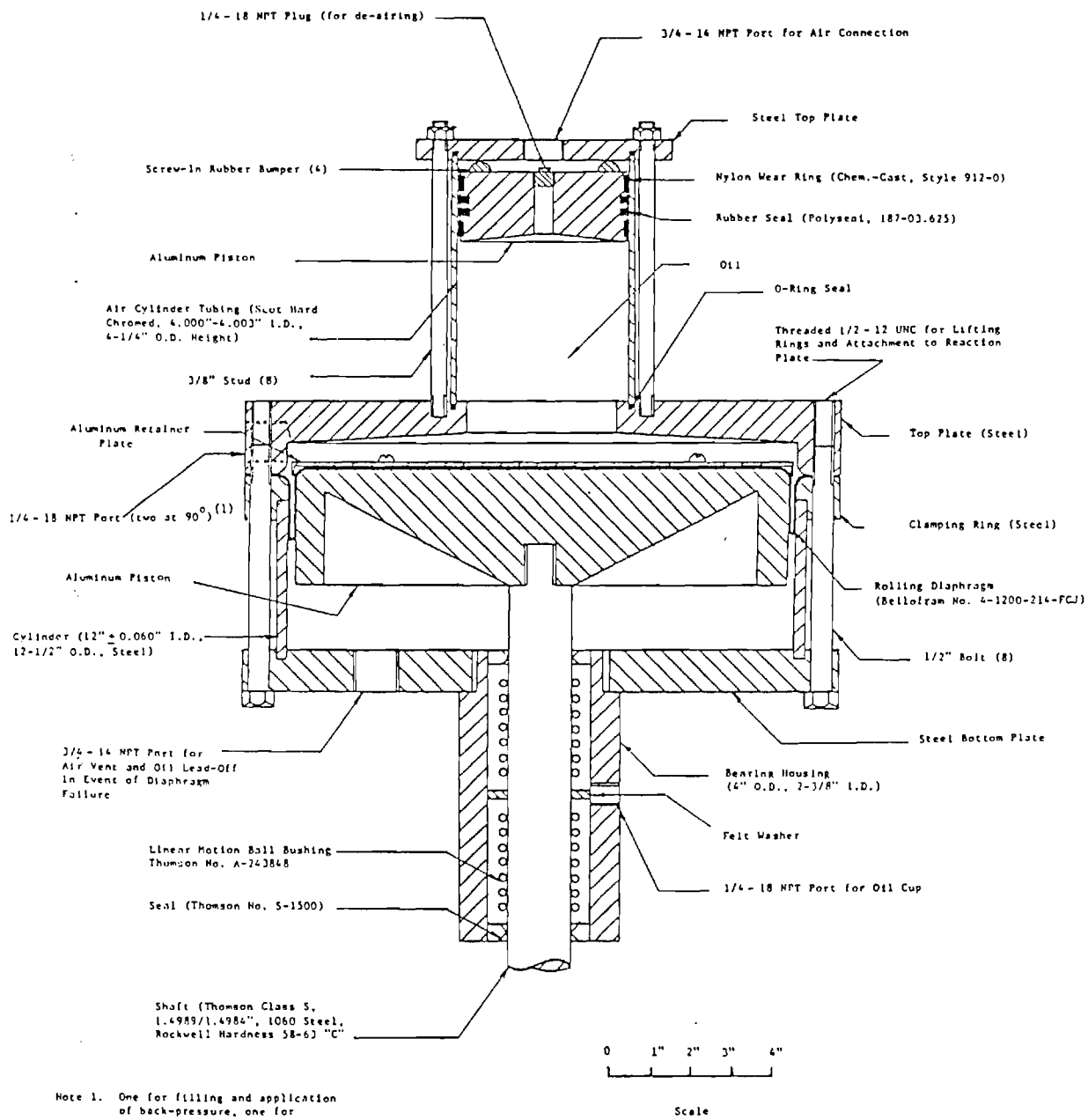


FIGURE 43. SECTIONAL VIEW OF LOADING DEVICE.

installed at the air inlet. The large lower piston was sealed to the walls of the cylinder using a rolling diaphragm. A 1 in. (25 mm) push rod transmitted the load from the lower piston to the load bladder. The push rod was guided in the vertical direction using two linear motion Thomson ball bushings located in a housing below the large cylinder.

A constant seating loading was maintained between load applications to prevent a shock loading. The seating load was applied by maintaining a small head of hydraulic fluid on the cylinder through a pressurized oil reservoir (Fig. 42). In addition to maintaining a constant seating load, the oil reservoir kept the cylinder filled with oil during the test. An electronic clock timer was initially used to automatically shut the loading system off after the correct number of pulses were applied at each load position; subsequently a predetermining counter was used to stop the test. A low pressure sensor was installed in the air lines to turn the system off in the event the house air pressure dropped below a preset value.

Cyclic Load. A repeated load was applied to the pavement by cycling the air pressure on top of the small, upper piston. The cyclic air pressure was controlled by means of a 3/4 in. (19 mm) dia. spool valve operated by two small solenoid pilot valves. The solenoid valves were opened and closed by sending an electrical signal to them using a multiple cam operated microswitch timer. Both load systems were controlled by using this timer. A buffer air tank was placed in the system before the valves to prevent fluctuations of air pressure during loading. Pressure regulators were used to maintain the desired pressure.

Loading Bladder. Load from the piston was applied to the top of a water-filled, circular bladder (Fig. 44) resting on the pavement surface. The bladder essentially consisted of a thin steel ring covered with a rubber diaphragm the top and bottom. Upon application of load to the top of the bladder through a rigid, circular loading plate, the concentric steel ring lifted up from the pavement surface giving a loaded area having a dia. of about 9.1 in. (231 mm). The pressure created in the bladder by the applied load was transmitted through the water to the surface of the pavement as a uniform, circular loading. For a 6,500 lb. (28.9 kN) load the corresponding uniform vertical pressure was about 100 psi (689 kN/m²). To prevent failure of the rubber diaphragms on the bladder due to abrasion, two diaphragms were used on both the top and bottom. The diaphragms were 1/16 in. (1.6 mm) thick neoprene having a single ply of nylon reinforcement.

Load Actuator Movement. The load actuator had four internally threaded hollow steel rods extending upward from the top of the cylinder. These rods were used to bolt the cylinder to the flat thrust plate attached to the reaction frame. Movement of the load cylinder from one fixed load position to another was easily accomplished by a special carriage which hung from the reaction beams. The carriage rolled along the reaction beam on four wheels, and temporarily supported the load cylinder during movement. The load cylinder was supported by nylon bearings on two steel rods on the carriage. These rods were perpendicular to the reaction beams. Use of the carriage thus permitted easy movement of the load cylinder in two orthogonal directions. After positioning, cap screws were used to securely fasten the load cylinder to the thrust plate.

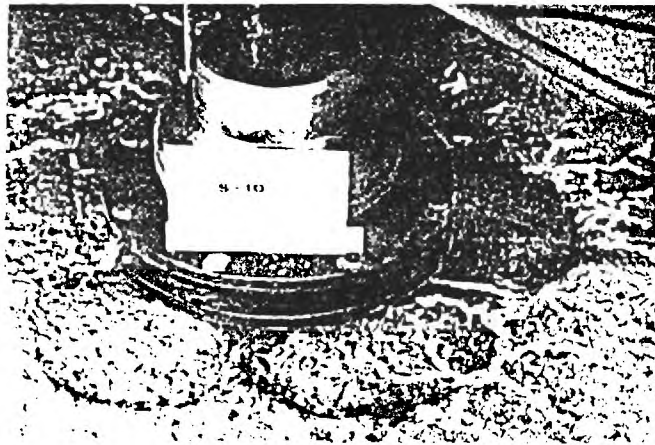


FIGURE 44. RUBBER BLADDER AND LOAD FOOT USED IN ALL TESTS.

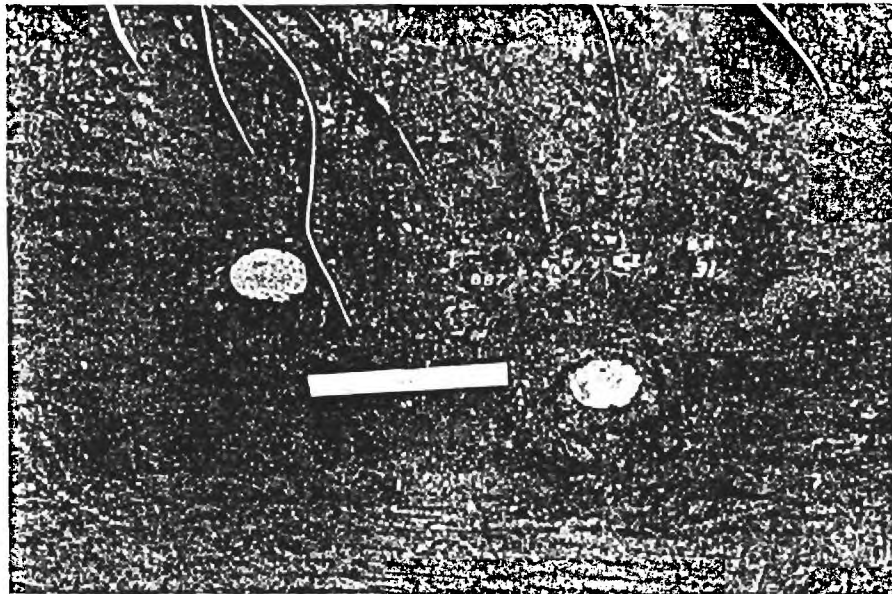


FIGURE 45. DIAGRAM TYPE PRESSURE CELLS AND STRAIN SENSORS USED IN TEST SECTIONS.

Instrumentation

The test sections were extensively instrumented to define the response of the pavement system. Bison type strain sensors were installed to measure both resilient and permanent deformations throughout the pavement section. Small diaphragm type pressure cells were used to measure vertical stress. Pressure cells and strain sensors placed on top of the subgrade are illustrated in Fig. 45. Resilient surface deflections were measured using linear variable differential transducers (LVDT's). Finally, permanent deformations of the pavement surface were measured from a string line using a metal scale. All instrumentation was carefully calibrated using the general procedures described in this section.

Strain Sensors. Both resilient and permanent deformations within the pavement were measured between pairs of Bison-type strain sensors. These strain sensors physically consisted of a thin, acrylic disc 1 to 4 in. (25 to 100 mm) in dia. having 200 to 400 windings of a small dia. copper wire. The strain sensors work on the induction principal and are excited by a Bison Soil Strain Box (Model 4101A). Relative movement between coils induces a change in current which is detected by the Bison box. Static movements were read directly from the box while transient movements were routed through the Bison box to a strip chart recorder. To reliably select and obtain readings from different pairs of coils, a switching box was used for each section to permit changing quickly between pairs of coils. Calibration was performed with all coils wired through the switching boxes. The strain sensors, which do not have any physical connection between coils, have proved in a number of different experiments to be reliable, rugged and relatively inexpensive.

A typical layout of strain sensors for both a granular base section and a full-depth asphalt concrete section is given in Figs. 46 and 47. Each of the sections tested had between 20 and 24 strain sensors. Generally 1 in. (25 mm) dia. sensors were used in the asphalt concrete, 2 in. (51 mm) dia. sensors in the crushed stone and 4 in. (102 mm) dia. sensors in the subgrade. The sensors were placed in vertical stacks with their flat side oriented horizontally. Using this instrumentation only displacement was actually measured between either vertical or horizontal pairs of coils. The average strain occurring between coils was determined by dividing the measured displacement by the distance between coils. Since the coils were closely spaced, the strain determined in this way was sufficiently close for practical purposes to the actual maximum strain occurring between coils.

Two primary vertical stacks of coils were placed between the bottom and surface of the pit (Figs. 46 and 47). Secondary coil stacks were placed in the surface and base course between the primary stacks. The main purpose of the secondary stacks of coils was to measure the relative horizontal tensile and compressive movements occurring in the surface and base course. Several special pairs of 1 in. (25 mm) dia. coils were also placed in the subgrade just below the interface to measure vertical strains in Test Sections 5 through 11.

Calibration. The strain sensors were calibrated before installation in each test section using either a micrometer stand or calibration rack. The calibration rack has 23 fixed positions in which a sensor can be easily positioned. One sensor was placed in a fixed location on the

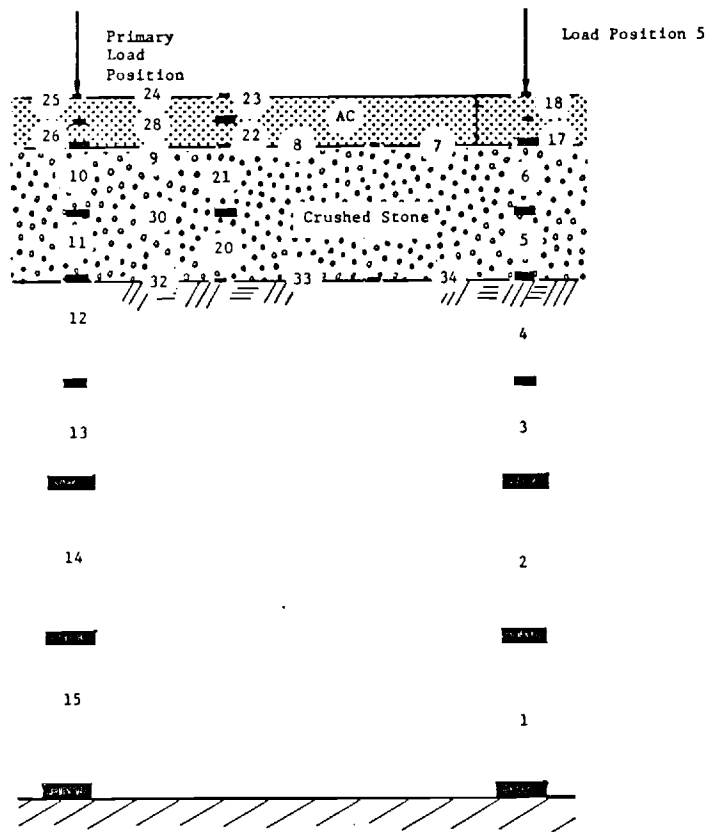


FIGURE 46. BISON STRAIN SENSOR LAYOUT USED IN SECTION 8 - CRUSHED BASE.

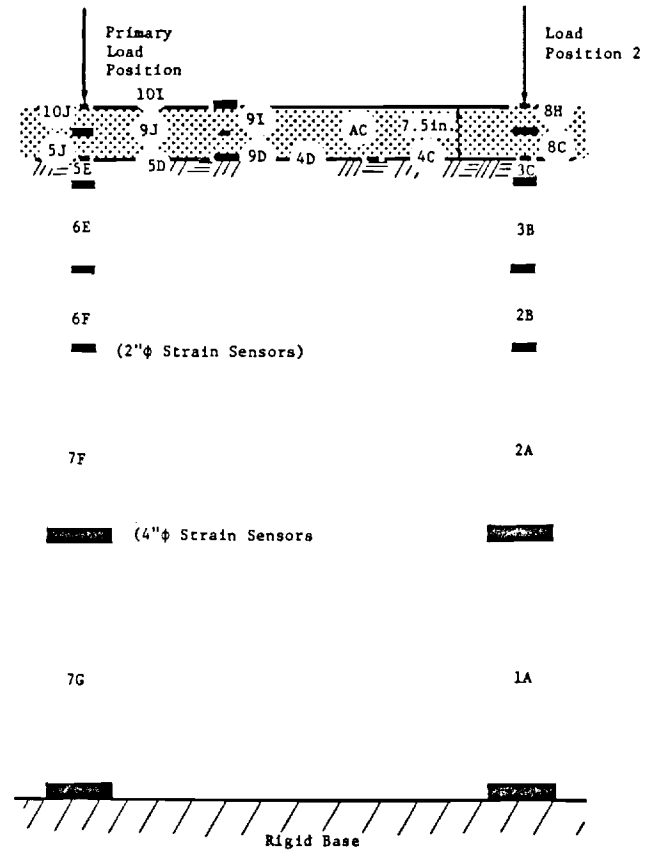


FIGURE 47. BISON STRAIN SENSOR LAYOUT USED IN SECTION 7 - FULL DEPTH AC.

rack while the other sensor was moved progressively from one fixed position to the next with an output reading being taken on the box at each position. Since the distance between every combination of positions was known, distance measurements were not required during calibration.

All calibrations for dynamic displacement were made in a micrometer calibration stand. The micrometer calibration stand had two movable jigs in which a pair of sensors were fastened at the desired distance apart. The distance between coils was then changed using a micrometer having 0.0001 in. (0.0025 mm) divisions, and the corresponding output was read in a strip chart recorder.

Calibration readings for at least six displacements were obtained on the strain box (or strip chart recorder) for each initial strain coil separation distance. Coil calibration was carried out at several initial coil separations due to nonlinearity of the output. After the sensors were positioned in the test sections, the separation between the coils was determined using the static mode of operation and interpolating between calibration curves. Calibrations for static displacement (i.e., displacement due to a stationary constant load and also permanent deformation) were generally made using the calibration rack. Static displacement readings were taken for permanent deformation when the pavement was unloaded, and for a constant load condition where the load was not repeated. Strain sensor calibration was carried out with the loading bladder in the proper position when it would influence the readings.

Pressure Cells

Two and 2.5 in. (51 and 64 mm) dia. diaphragm type pressure cells described in detail elsewhere [32] were used in the test sections to measure vertical stress. These gages satisfied the pressure cell design criteria summarized by Brown [33]. Diaphragm type strain gages were used which measured both maximum radial and tangential strains occurring when the diaphragm deflected. The maximum strains were combined to give maximum cell sensitivity for small applied pressures.

The two diaphragm pressure cells used during the investigation are

1. Initially a 2 in. (51 mm) dia., 0.22 in. (5.6 mm) thick titanium cell was used to reduce problems with corrosion. The active diaphragm thickness varied from 0.055 in. to 0.078 in. (.4 to 1.9 mm) depending upon the stiffness of the material in which it was embedded.
2. During the study the pressure cell design was changed to a 2.5 in. (64 mm) dia., 0.30 in. (7.6 mm) thick cell made from aluminum. This larger pressure cell design was easier to machine. The active diaphragm thickness of this cell varied from 0.090 in. to 0.135 in. (2.3 to 3.4 mm). The active diaphragm dia. of both cells was 1.25 in. (32 mm).

All pressure cells were coated for corrosion protection with an epoxy conformal coating (Magnolia Plastics, Magnobond 3900). Before the cells were dipped in this coating, the surface was sand-blasted and degreased with MEK.

Output from the pressure cells was sent through a Budd or BLH switching and balancing unit to either a Hewlett-Packard Model 321 DC recorder or Model 320 DC recorder. When the DC recorder was used excitation was supplied by a 6 volt dry cell battery. A variable potentiometer was used to maintain the excitation throughout the test at either 4 or 5v (+ 0.010v). The output from the pressure cells was amplified by a Neff Model 122 DC amplifier with filtering capability.

Calibration. Calibration was carried out in a steel calibration tank about 17 in. (430 mm) in dia. and 8 in. (200 mm) in height. Subgrade soil was placed in the tank at the same moisture content and density used in the test pit. As the soil was being placed in the calibration tank, the pressure cell was carefully embedded in the soil at a depth of about 4 in. (100 mm). A rubber membrane was then placed over the soil and the top fastened on the calibration tank. Air pressure was applied to the top of the membrane in small increments over the range of pressure expected to develop in the pit, and the corresponding output from the cell recorded. The air pressure was measured using a test gage and controlled with a sensitive regulator. Before calibration the first time, each cell was preconditioned by applying at least 50 cycles of stress up to 60 psi (413 kN/m²).

Selected cells were dynamically calibrated by applying a 0.17 sec. pulse to the cell while placed within the soil tank. Dynamic calibration was accomplished by filling the space above the top of the rubber membrane with water, and applying a cyclic air pressure to the top of the water. For the two pressure cells used in this study, the dynamic response was found to be within about $\pm 2\%$ of the static response. Therefore the pressure cell output did not have to be corrected for dynamic effects.

The cross-sensitivity of selected cells was also determined by placing the cell in the same vessel without soil so that an equal pressure was applied to the cell. Considering the response of the pressure cell in soil to be correct, the cross-sensitivity of the pressure cell was determined by dividing for a given output signal the difference in pressures for the two conditions (cell embedment in air and water) by the pressure in soil, and multiplying by 100 to convert to percent. The cross-sensitivity of the two pressure cells was found to be about 16%. No significant difference in cross-sensitivity was observed between the 2.0 and 2.5 in. (51 and 64 mm) dia. pressure cells.

Displacement

Resilient displacements across the pavement surface were measured using 5 D.C. linear variable differential transformers (LVDT's). The LVDT's were excited with a 24v D.C. power supply, with the output being recorded on a Hewlett-Packard Model 320 D.C. strip chart recorder. The LVDT's were precisely calibrated by placing each transducer in a stand and applying known displacements using a micrometer. Periodically the calibrations were checked by inserting a calibration block of known thickness and comparing the observed recorder pen movement with the calculated movement using the calibration constant for the LVDT obtained from the micrometer calibration.

Permanent surface rutting was determined by periodically measuring with a metal scale the distance from a reference string line to the surface of the pavement. Readings to the nearest 1/32 in. (0.8 mm) were taken on both sides of the load in the short direction of the pit. As previously discussed, resilient and permanent displacements occurring within the pavement were obtained using the Bison strain sensors.

Conclusions

Twelve full-scale test sections were carefully constructed over a relatively weak silty sand subgrade 50 in. (1270 mm) thick. A completely new section was constructed for each test including the subgrade. All test sections except 3 and 4 met Georgia DOT construction specifications. Sections 3 and 4 were later found to be constructed using an asphalt concrete having an excessively high asphalt content that resulted in a premature rutting failure of the asphalt concrete during subsequent testing. The granular bases used in the conventional sections were compacted to 100% of AASHTO T-180 density. To minimize segregation, different sizes of stone were blended together just before placement. The blended aggregate was bottom dumped to minimize segregation.

Two hybrid air-over-oil cyclic loading systems were designed and constructed (including the reaction frame) to test to failure the pavement sections. The two testing systems were used to apply over 17 million load repetitions during the study. These testing systems were found to be quite reliable and relatively easy to maintain considering the large number of repetitions applied.

The test sections were fully instrumented to give important information concerning the amount and distribution of permanent rutting within each layer, resilient displacement, resilient strain and vertical stress on the subgrade. This response information was later used in Chapter 6 to help extend the results to sections having different geometries and loading conditions than used in this study.

CHAPTER V TEST RESULTS

Introduction

The purpose of testing to failure full-scale pavement sections in the laboratory was to study the feasibility of replacing full-depth and deep strength asphalt concrete with crushed stone base pavements utilizing relatively thick layers of unstabilized aggregate. Therefore, for comparison purposes tests were performed on both full-depth asphalt concrete sections and also crushed stone base pavement sections. A detailed description of the materials used was given in Chapter III, and construction, instrumentation and testing of the pavement sections were described in Chapter IV.

Definition of Fatigue and Rutting Failure

Fatigue. A fatigue failure was defined for purposes of this study as the initiation of CLASS 2 cracking. Class 2 cracking is defined [5] as the stage where cracks have connected together to form a grid type pattern. Cracking was found in general to progress relatively rapidly over the loaded area. Therefore, this somewhat general definition of a fatigue failure was in practice adequate (refer for example to Fig. 98 for two stages of crack development). Only fine hairline cracks which often were hard to see developed in the sections undergoing fatigue failures. Testing was terminated before wider cracks could develop because of the large number of load repetitions required to cause failure of the sections tested.

Rutting. A rutting failure was defined *for comparison purposes* as a 0.50 in. (12.7 mm) rut depth measured from a fixed string line. Admittedly smaller rut depths might frequently be used for design by the Georgia DOT. The 0.5 in. (12.7 mm) rut depth was the average of the rut depths measured at completion of testing at the primary load position and at completion of testing at the sixth secondary load position. Also, since the asphalt concrete sections often underwent relatively large amounts of rutting during the early stages of loading, the rut depth occurring during the first 1,000 repetitions was not included in the average.

The average value of rutting was used since an important amount of creep and plastic flow of the asphalt concrete and base materials was found to occur as the load was moved back and forth between the primary and secondary load positions. When the load was applied at the primary load position, the underlying material flowed out laterally and then upward from beneath the load causing a bulge to appear around the sides of the loaded area. As the secondary positions were loaded, the center area went up, and the loaded area moved downward. As a result, a given point within the general loaded area was cyclically going up and down depending upon the physical location of the load.

Materials

The asphalt concrete used in all sections was composed for the entire depth of either a Georgia DOT Modified B or B-binder. The asphalt concrete was compacted to approximately 99 % of the maximum 50 blow Marshall density. The crushed stone base course aggregate used in this study was obtained from the Norcross Quarry of Vulcan Materials Company. The stone used in the conventional crushed stone base sections was compacted to slightly in excess of 100% of AASHTO T-180 density. The two inverted sections had a crushed stone base overlying either a 6 in. (152 mm) layer of cement treated subgrade soil (Section 11) or cement stabilized crushed stone (Section 12). The crushed stone base density in these two sections using the same level of effort was about 3.6% higher than in the other granular bases.

The subgrade consisted of a loose silty sand having a standard penetration resistance of about 8 blows per foot. The depth of the subgrade was 50 in. (1270 mm). Before constructing each section⁽¹⁾, the subgrade was removed and replaced in 2 in. (51 mm) lifts at an average of 96% of AASHTO T-99 density.

Structural Sections

The pavement sections tested were as follows (Table 1): (1) five crushed stone base sections, (2) five full-depth asphalt concrete sections, and (3) two inverted sections. All crushed stone base and inverted sections had an asphalt concrete thickness of 3.5 in. (89 mm). The crushed stone bases used with the conventional pavements were either 8 or 12 in. (203- 508 mm) thick. The following three crushed stone base gradations were used in the investigation: (1) a standard gradation close to the average gradation used in the Atlanta area, (2) a fine gradation, and (3) a coarse gradation. The coarse and fine gradation were only used in Sections 9 and 10, respectively. The base course gradations were given in Table 11.

The full-depth sections consisted of either 6.5, 7.0, or 9.0 in. (165, 178, 229 mm) of asphalt concrete binder; these sections rested directly on the subgrade. The two composite sections consisted of 8 in. (203 mm) of unstabilized crushed stone sandwiched between 3.5 in. (89 mm) of asphalt concrete above and 6 in. (152 mm) of cement stabilized material below. One inverted pavement (Section 11) had a cement treated silty sand subbase, while the other inverted pavement (Section 12) had a cement stabilized crushed stone subbase.

Loading

As discussed in Chapter 4, a repeated loading was applied to the pavement surface of 6,500 lbs. (29 kN) exerting a uniform contact pressure

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1. The lower 4 ft. (1.2 m) of the subgrade from Sections 3 and 4 was reused in Sections 5 and 6 because it was not damaged during the 10,000 load repetitions applied to Sections 3 and 4.

of 100 psi (689 kN/m²). Sections 9 through 12 were tested to more than 2 million load repetitions. After 2 million repetitions, the load applied to Sections 9 and 10 was increased to 7,500 lbs. (33 kN) to reduce the time to failure. In Sections 11 and 12 the load was maintained at 6,500 lbs. (29 kN) after 2 million repetitions, but the number of load repetitions applied at each location was doubled. To simulate traffic wander the stationary circular loading was applied in a primary load position and six supplementary positions located symmetrically around the edge of the primary position as shown in Fig. 41.

Performance

The results obtained from repeatedly loading the sections to failure are summarized in this chapter. Because of the large quantity of experimental data collected during this study, only the most relevant information is presented.

A general summary of the experimental results is given in Table 15. Table 15 serves as a representative summary of the detailed response information obtained from these tests. The two test sections described in each subsequent section were constructed at the same time. Therefore, these companion sections should have similar material characteristics.

Pavement Performance

Sections 1 and 2

Test Sections 1 and 2 consisted of 3.5 in. (89 mm) of modified B-binder overlying a crushed stone base 12.0 and 8 in. (305 and 203 mm) thick, respectively. The modified B asphalt concrete mix design is given in Fig. 10; this mix was obtained from APAC-Georgia, Inc.'s Lithia Springs plant. The standard crushed stone base gradation was used in Test Sections 1 and 2.

Rutting. Test Section 1 was tested to 2.4 million load applications before terminating the test. An extrapolation of the test results indicates a rutting failure (0.5 in., 25 mm) rut depth would have occurred at about 3.0 to 3.5 million repetitions. Section 2 failed in rutting after 1 million load repetitions. The variation of rut depth under the center of the primary load position with number of load repetitions is shown for Sections 1 and 2 in Figs. 48 and 49, respectively.

By the completion of testing for Sections 1 and 2, a limited amount of fatigue cracking had developed. These cracks usually disappeared when the load was positioned above them, only to reappear again when the load was moved to another location. Healing and the reappearance of these cracks was a common occurrence in Sections 1 and 2. The extent of cracking, however, was never any greater than shown in Fig. 50. Since Section 1 had developed a limited amount of cracking after 2 million repetitions, the postulation is made that by the estimated failure of 3 to 3.5 million repetitions cracking would have developed sufficiently

TABLE 15. DETAILED SUMMARY OF RESILIENT TEST SECTION RESPONSE.

Sect.	RESILIENT SURFACE DEFLECTION (IN.)																			
	10,000 Repetitions				100,000 Repetitions				1,000,000 Repetitions				Termination							
	Q ⁽¹⁴⁾	1	2	3	4	Q ⁽¹⁴⁾	1	2	3	4	Q ⁽¹⁴⁾	1	2	3	4	Q ⁽¹⁴⁾	1	2	3	4
1	-	0.011	-	0.015	0.0069	-	0.020	0.020	0.001	0.0055	-	0.026	0.062	0.012	0.093	-	0.052 ⁽³⁾	0.044	0.020	0.011
2	-	0.030	0.019	0.010	0.0045	-	0.029	0.015	0.008	0.0022	-	0.035	0.024	0.014	0.004	-	0.045 ⁽⁴⁾	0.025	0.015	0.0014
3	PREMATURE FAILURE - EXCESSIVE ASPHALT CONTENT																			
4	PREMATURE FAILURE - EXCESSIVE ASPHALT CONTENT																			
5	-	0.010 ⁽¹³⁾	0.011	0.005	0.006	-	0.011	0.014	0.008	0.004	-	0.014	0.012	0.008	0.005	-	0.011 ⁽⁹⁾	0.012	0.007	0.008
6	-	0.014 ⁽¹²⁾	0.014	0.007	0.008	-	0.016	0.024	0.015	0.010	-	-	-	-	-	-	0.016 ⁽¹⁰⁾	0.024	0.015	0.010
7	-	0.014	0.017	0.0088	0.0064	-	0.018	0.022	0.015	0.003	-	0.013	0.016	0.010	0.0058	-	0.016 ⁽⁵⁾	0.022	0.015	0.0059
8	-	0.016	0.01	0.012	0.0054	-	0.015	0.022	0.013	0.0015	-	0.0151	0.025	0.013	0.0079	-	0.015 ⁽⁶⁾	0.025	0.013	0.0079
9	-	0.013	0.017	0.0096	0.0080	-	0.016	0.023	0.012	0.009	0.029	0.016 ⁽⁷⁾	0.026	0.015	0.007	-	0.019 ⁽⁷⁾	0.035	0.015	0.0099
10	-	0.016	0.022	0.010	0.008	-	0.015	0.020	0.010	0.008	0.037	0.016	0.023	0.012	0.009	-	0.028 ⁽⁸⁾	0.034	0.020	0.014
11	0.026	0.005	0.006	0.003	0.002	-	0.003	0.005	0.003	0.002	-	0.001	0.003	0.002	0.001	-	0.004 ⁽¹⁾	0.007	0.002	0.001
12	0.017	0.004	0.005	0.002	-	-	0.004	0.005	0.003	0.002	-	0.003	0.006	0.002	0.002	-	0.003 ⁽¹²⁾	0.004	0.002	0.002

NOTES: 1. Resilient surface deflections are average values. For tests 1 and 2 load positions 1, 2, 3 and 4 correspond to radial distances of 7, 12, 16.5, and 21 in. from the centerline; for tests 5 through 12 they correspond to distances of 11, 10, 14.5, and 19 in.

2. Average horizontal tensile strain for the first 100,000 load repetitions.

3. Resilient surface deflections at 2,125,555 load repetitions.

4. Resilient surface deflections at 1,899,950 load repetitions.

5. 110,000 repetitions.

6. 1,081,000 repetitions.

7. 2,280,000 repetitions.

8. 3,080,000 repetitions.

9. 200,000 repetitions.

10. 350,000 repetitions.

11. 3,600,000 repetitions.

12. 4,400,000 repetitions.

13. Initial sequence of readings taken at about zero repetitions.

14. Deflections were not taken under the bladder; any set of readings with a value at the centerline corresponds to a special rigid plate load.

15. Resilient strain of 460×10^{-6} in./in. at 1,000 reps. and 633×10^{-6} in./in. at 126,000 reps.

16. Very small strains were observed in the cement stabilized layer.

TABLE 15. DETAILED SUMMARY OF RESILIENT TEST SECTION RESPONSE (continued).

RESILIENT STRAIN RESPONSE								
Sect.	Horiz. Tensile Strain ($\times 10^{-6}$)		Vert. Stress (psi)		Vertical Strain ($\times 10^{-3}$)			
	Bot. A.C.	Bot. Stone	Top Stone	Top Subgrade	AC	Top Stone	Bot. Stone	Top Subgrade
1	465	597		3.4	-	-	-	1.7
2	674	754		-	11.0	21.3	-	13.1
3								
4								
5	319	-	-	8.7	0.85	-	-	1.38
6	460 ⁽¹⁵⁾	-	-	12.6	-	-	-	1.50
7	410	-	-	12.9	0.65	-	-	2.2
8	300	375	-	11.9	-	0.56	.11	1.85
9	280	1,030	62	11.1	-	0.56	0.34	1.75
10	400	1,025	54	6.8	-	0.62	0.40	2.50
11	340	54 ⁽¹⁶⁾	-	3.3	-	0.73	0.37	0.39
12	260	22 ⁽¹⁶⁾	-	3.4	-	0.76	0.42	0.34

74b

to result in a combined rutting/fatigue failure of this section. This postulation is in agreement with the results of subsequent sections which failed in fatigue.

Figs. 48 and 49 nicely illustrate the up and down movement of the surface beneath the primary load position due to lateral shear flow of the material. A number of the points of minimum deflection (i.e., the rut depth immediately after loading the secondary positions) were not measured for Sections 1 and 2 because the importance of shear flow and creep were not fully realized at the time; these points are therefore frequently missing on the figures.

The surface deflection profile at selected numbers of load repetitions is shown for Sections 1 and 2 in Figs. 51 and 52, respectively. Loading the pavement at the six secondary load positions was reasonably effective in preventing a punching failure from occurring at the primary position. Nevertheless, some punching did occur as indicated by the deflection profile. Since rutting in a prototype pavement also occurs over a relatively narrow width, some punching also occurs in the field, although it is probably somewhat less than in the test sections. In this study the performance of the crushed stone base sections was directly compared with full-depth asphalt concrete sections for similar load, environmental and subgrade conditions. Therefore, the absolute degree of punching experienced should not significantly influence the test results.

Distribution of Permanent Deformation. The variation of permanent deformation in Sections 1 and 2 with depth and number of load repetitions is summarized in Table 16 and Figs. 53 and 54. These results were obtained from the strain sensors placed throughout the pavement structure.

In Section 1 the majority (59%) of the total permanent deformation occurred in the 3.5 in. (89 mm) asphalt concrete layer; 21% occurred in the 12 in. (305 mm) crushed stone base; and 20% in the subgrade. In Section 2 the crushed stone base thickness was reduced to 8 in. (203 mm). Use of the thinner base caused an important influence on the relative distribution of permanent deformation through the section: (1) the relative amount of permanent deformation in the subgrade almost doubled going from 20 to 39%; (2) relative permanent deformation in base slightly increased going from 21% to 27%; (3) finally, permanent deformation in the asphalt concrete surfacing dropped from 59% to 34%. Thus presence of the thinner base tended to shift some of the rutting from the asphalt concrete to the subgrade.

Resilient Response. The resilient (dynamic) response of Sections 1 and 2 is summarized in Table 15. For Section 1 the measured horizontal tensile strain in the bottom of the asphalt concrete was 465×10^{-6} in./in., the dynamic vertical stress on the subgrade was 3.4 psi (23.4 kN/m²), and the vertical strain at the top of the subgrade was 1.7×10^{-3} in./in. The typical variation of resilient surface displacement for Sections 1 and 2 is given in Figs. 55 and 56. The resilient surface displacement in general gradually increased with load repetitions after an abrupt initial increase. The variation for Section 2 of resilient vertical

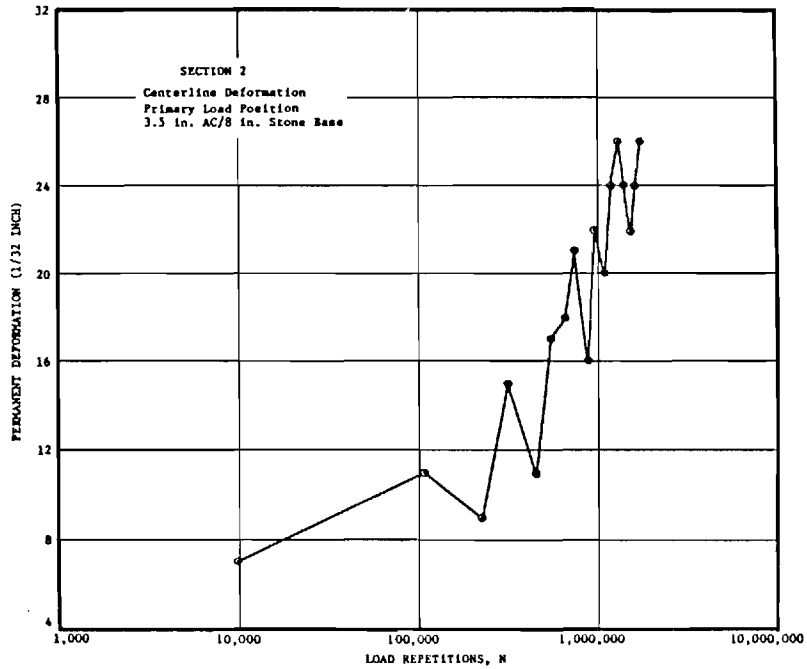


FIGURE 48. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 2.

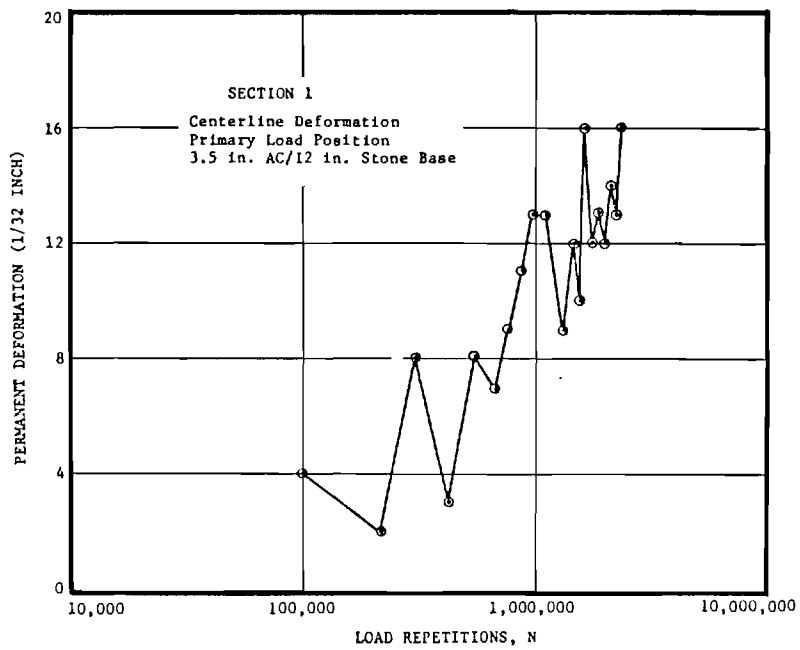
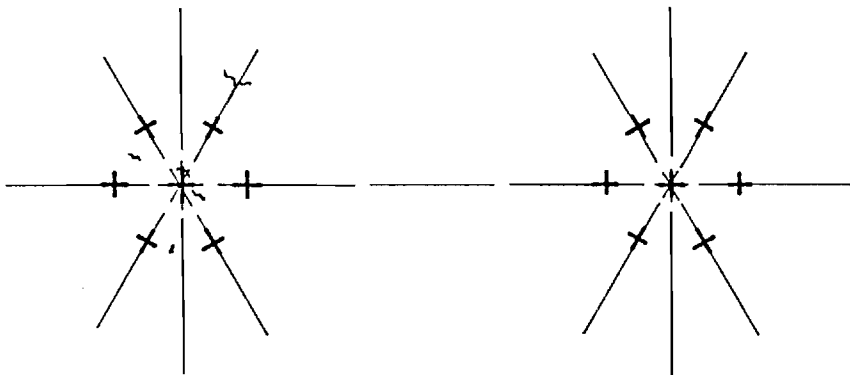


FIGURE 49. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 1.



(a) Section 2 (N = 1,200,000)

(b) Section 1 (N = 2,400,000)

FIGURE 50. SURFACE CRACKING OBSERVED IN TEST SECTIONS 1 AND 2 AT INDICATED NUMBER OF LOAD REPETITIONS.

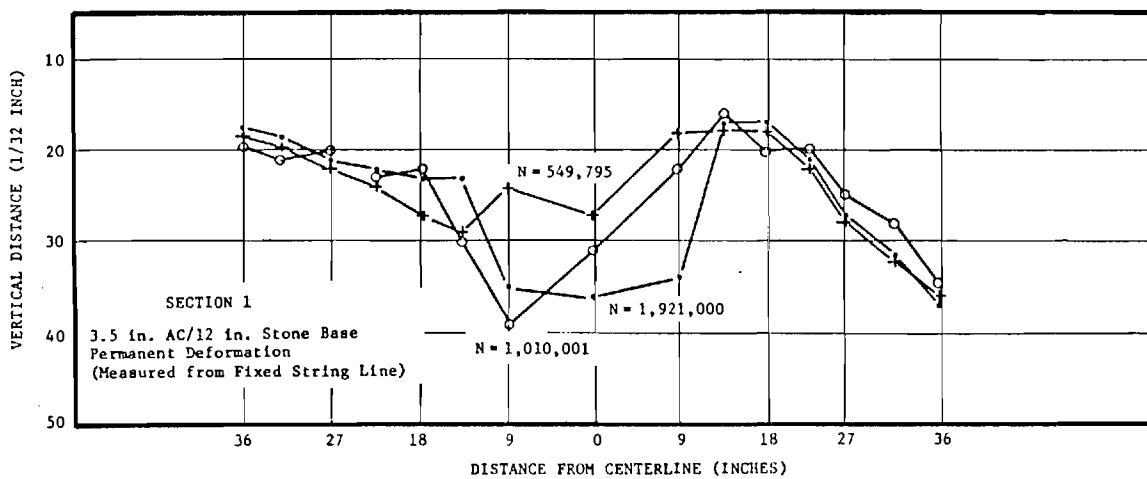


FIGURE 51. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 1.

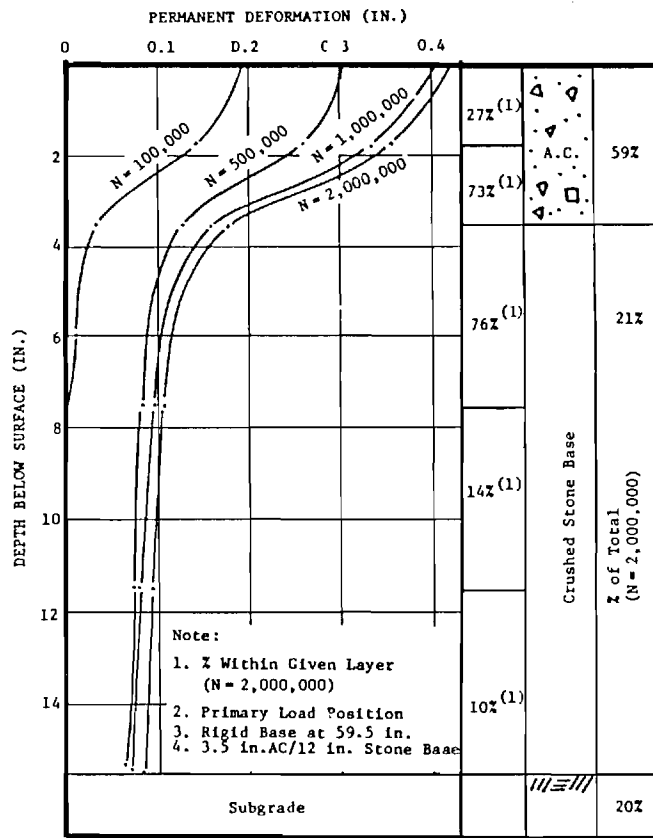


FIGURE 53. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 1.

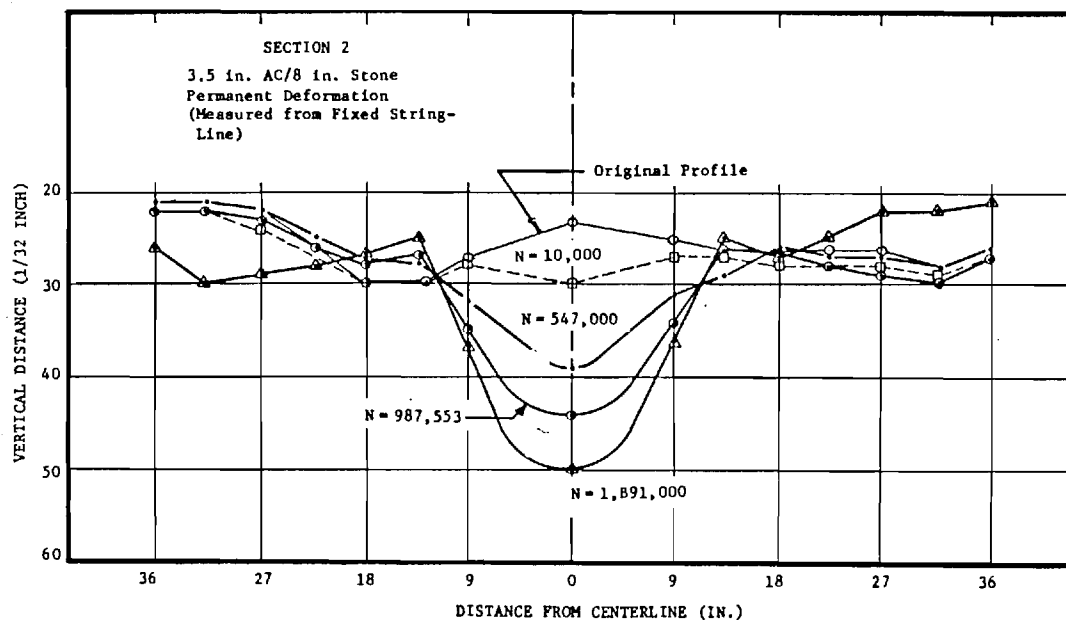


FIGURE 52. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 2.

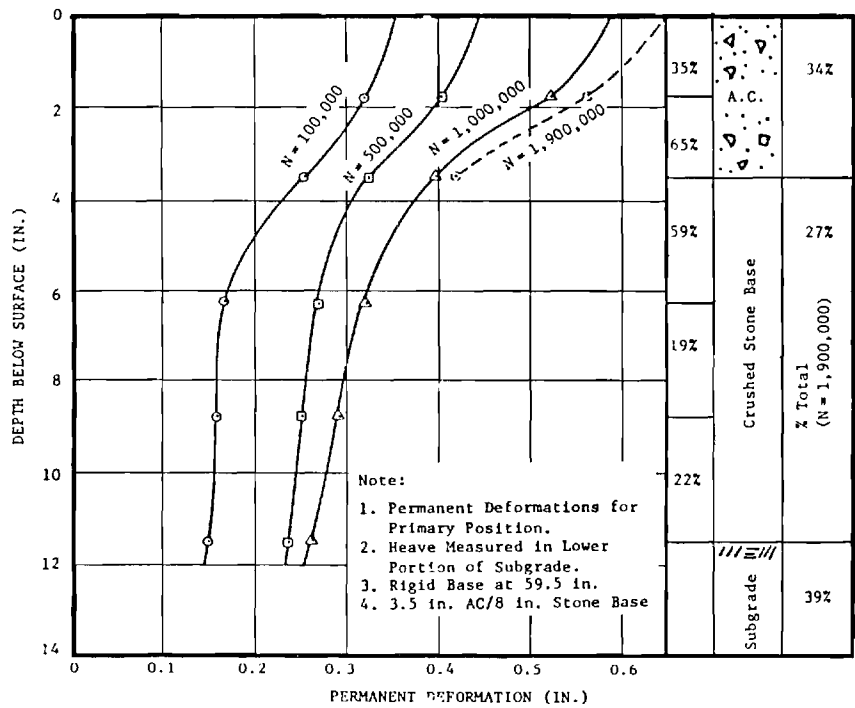


FIGURE 54. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 2.

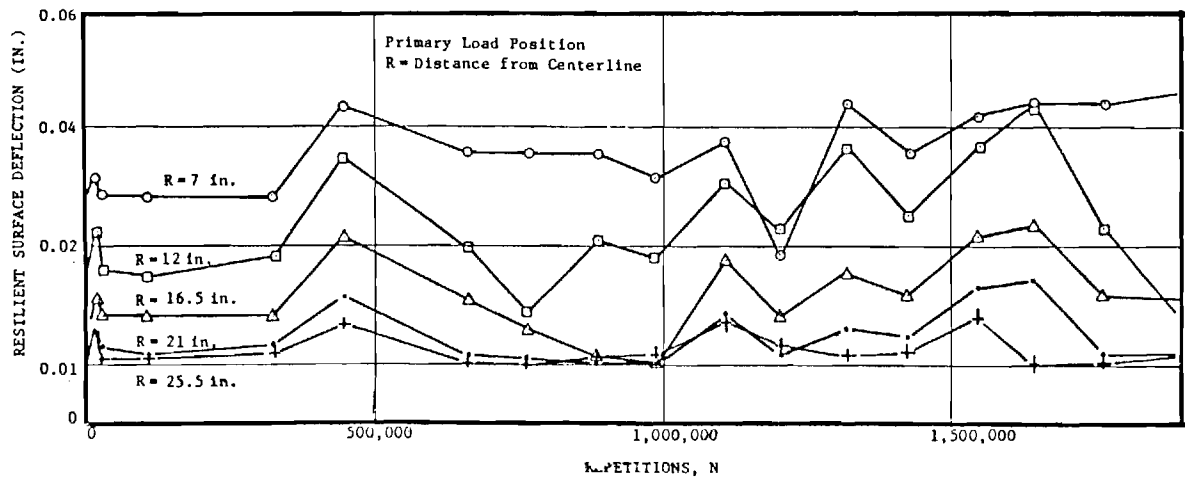


FIGURE 55. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 1.

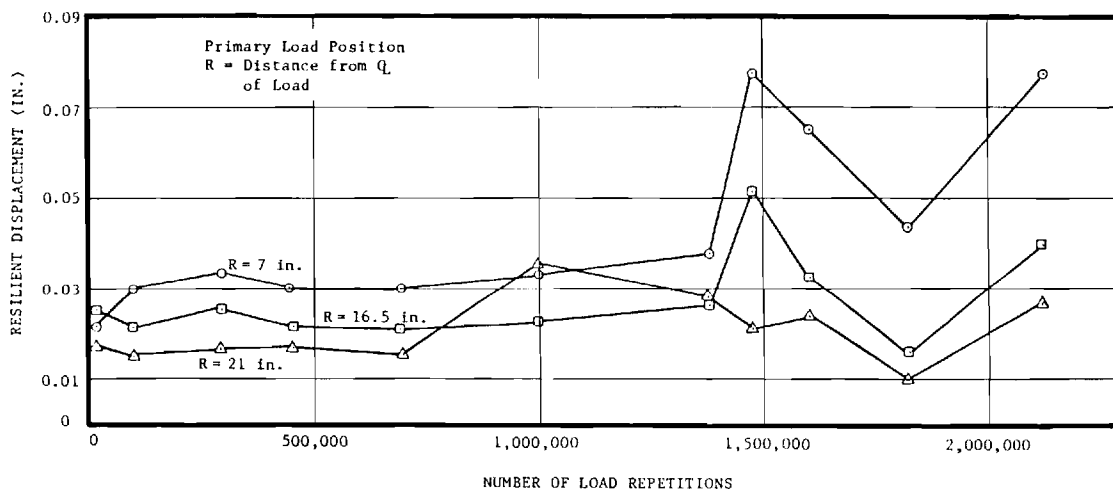


FIGURE 56. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 2.

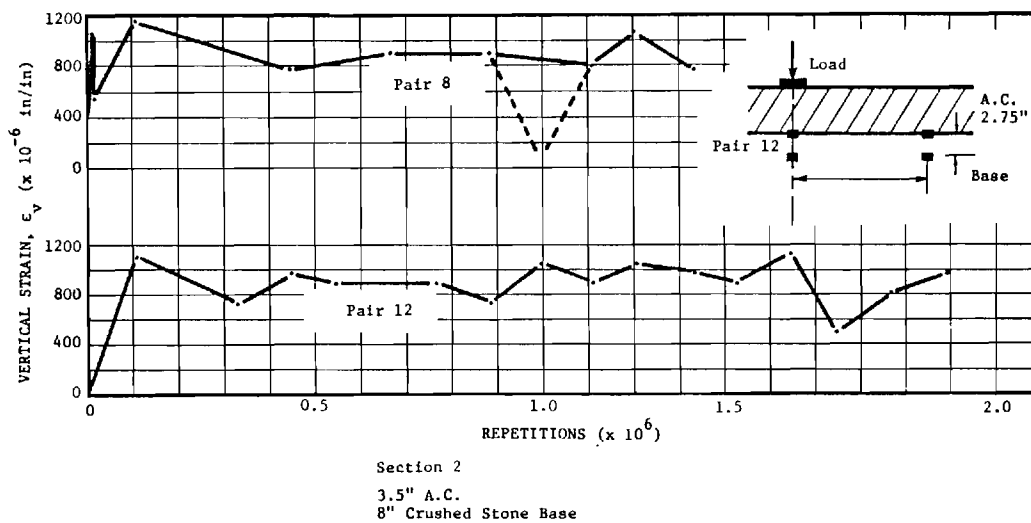


FIGURE 57. DYNAMIC VERTICAL STRAIN ON SUBGRADE AS A FUNCTION OF LOAD REPETITIONS - SECTION 2.

TABLE 16. DISTRIBUTION OF PERMANENT DEFORMATION IN SECTION 1 AND 2
AS A FUNCTION OF LOAD REPETITION.

Section	N ($\times 10^6$)	Surface			Base				Subgrade	
		Top(1)	Bottom(1)	Total(2)	Top(1)	Middle(1)	Bottom(1)	Total(2)	Top(1)	Total(2)
1	0.1	25	75	82	78	6	16	18	heave	heave
	0.5	25	75	58	66	18	16	21	46	21
	1.0	26	74	61	71	16	13	22	13	16
	2.0	28	72	59	76	13	11	21	18	20
2	0.1	35	65	28	81	8	11	31	66	41
	0.5	34	66	28	59	24	17	20	58	53
	1.0	34	66	32	59	17	24	23	62	45
	1.9	35	65	34	59	17	24	27	67	39

- Notes: 1. Percent of total permanent deformation occurring in the *layer*.
2. Percent of total permanent deformation occurring in the pavement structure.

displacement on top of the base with load repetitions is given in Fig. 57, and horizontal resilient displacements within the base in Fig. 58.

Sections 3 and 4

Sections 3 and 4 were full-depth sections consisting of 9.0 and 6.5 in. (229 and 165 mm), respectively of B-modified asphalt concrete. Both of these sections underwent 1 in. (25 mm) of rutting after only 10,000 load repetitions. Because of the premature failure, extensive response information is not presented. The results of tests performed on asphalt concrete cores taken from these sections were previously summarized in Table 6. Although a density of 99% of the maximum Marshall value was obtained, the asphalt content of these sections was about 5.9% compared with a mix design value of 5.2%. Although the gradation was slightly out of the allowable range in the 1 in. (25 mm) sieve by 3.1%, and on the No. 8 screen by 4%, gradation is not considered to be a significant factor in rutting of this mix. The early failure of these two sections is attributed primarily to high asphalt content.

Sections 5 and 6

Sections 5 and 6 also consisted of full-depth, modified-B asphalt concrete binder having thicknesses of 9.0 (229 mm) and 6.5 in. (165 mm), respectively. The asphalt concrete was obtained from the Lithia Springs plant of APAC-Georgia, Inc. As just discussed, Sections 3 and 4 failed in rutting after only 10,000 load repetitions. Upon removal of the asphalt concrete from these sections, the subgrade was found to be in excellent condition with essentially all of the rutting having occurred in the asphalt concrete. Therefore, only the upper 8 in. (203 mm) of the subgrade was removed and replaced before construction of Sections 5 and 6.

Rutting. Section 5 underwent a rutting failure after 130,000 load repetitions, and Section 6 after 580,000 load repetitions. The variation of rut depth beneath the center of the primary load position for Section 5 is shown in Fig. 59 and for Section 6 in Fig. 60. Figs. 61 and 62 show the deflection profile at selected numbers of load repetitions. Figure 63 shows pictorially the condition of the two sections after 1 million repetitions.

Surface cracking was not observed in Section 5, which consisted of 9.0 in. (229 mm) of asphalt concrete. Section 6, the thinner 6.5 in. (165 mm) thick full-depth section, did at the termination of the test have 5 small hairline cracks as shown in Fig. 64. The rut depth at this time was 0.75 in. (19 mm). The primary failure mode of this section was, however, clearly a rutting failure.

The average asphalt content obtained from six cores taken from these sections was 5.3% (one core had a reported asphalt content of 7.14% which was not included). The average density of the seven cores was 99% of the Marshall maximum density. The relatively early failure of these

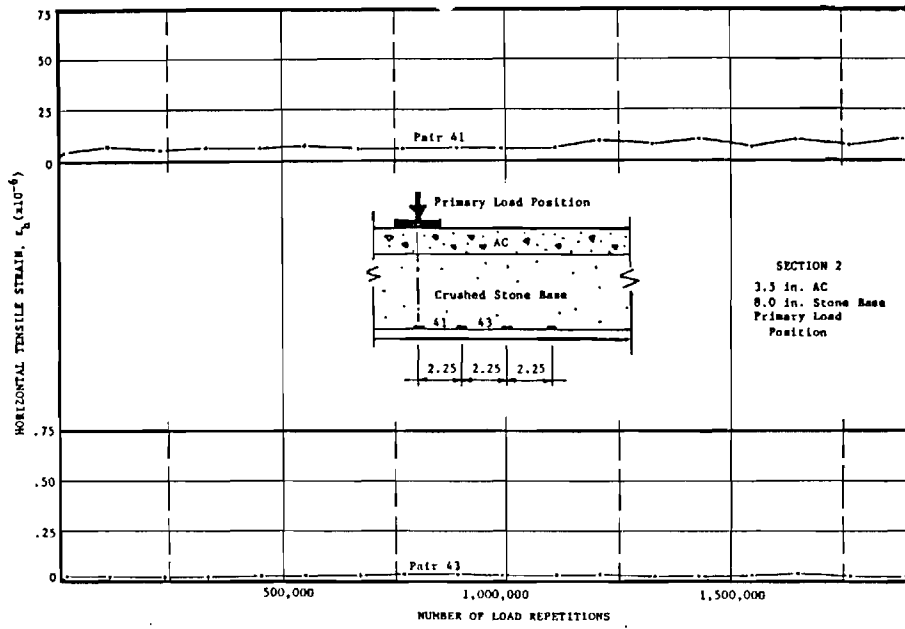


FIGURE 58. VARIATION OF RESILIENT HORIZONTAL STRAIN IN BOTTOM OF CRUSHED STONE BASE WITH NUMBER OF LOAD REPETITIONS - SECTION 2.

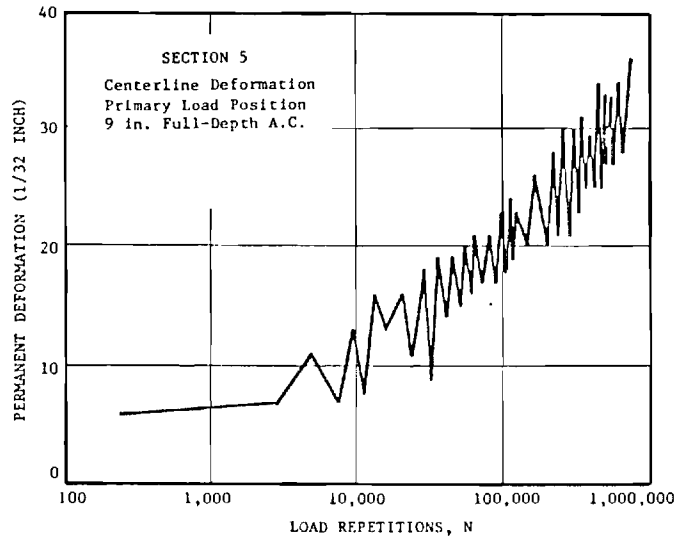


FIGURE 59. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 5.

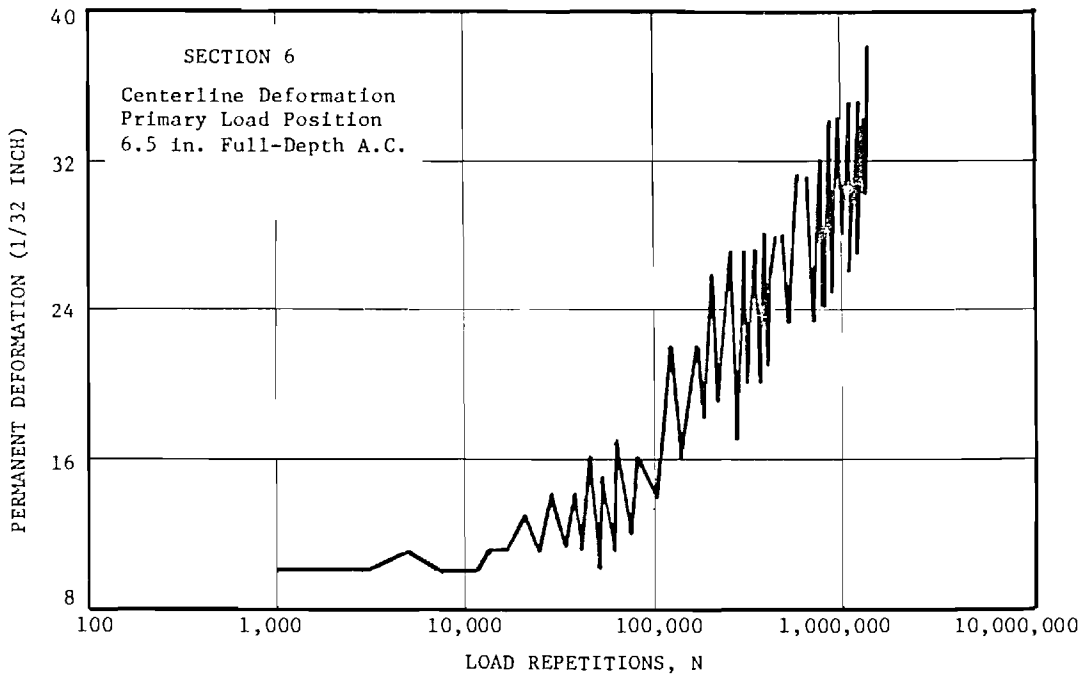


FIGURE 60. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 6.

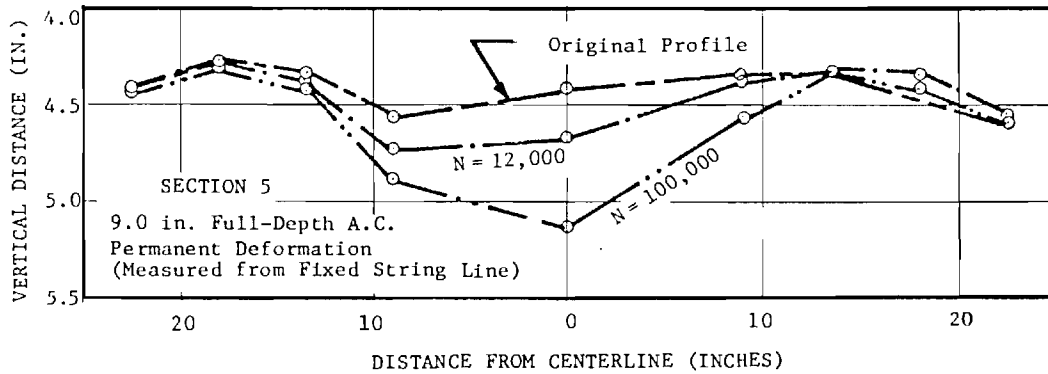


FIGURE 61. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 5.

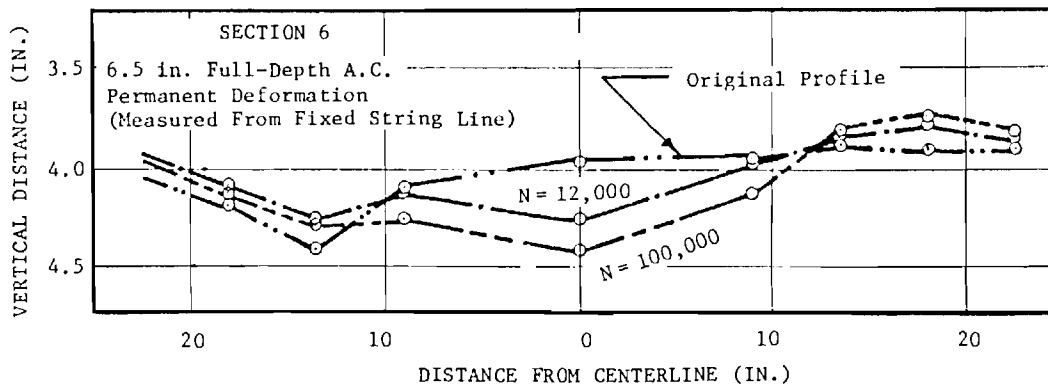
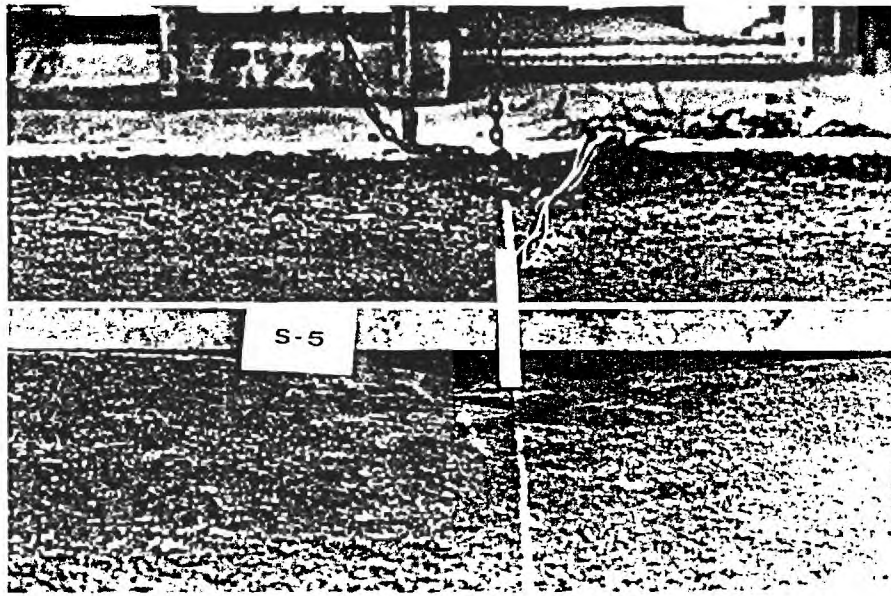
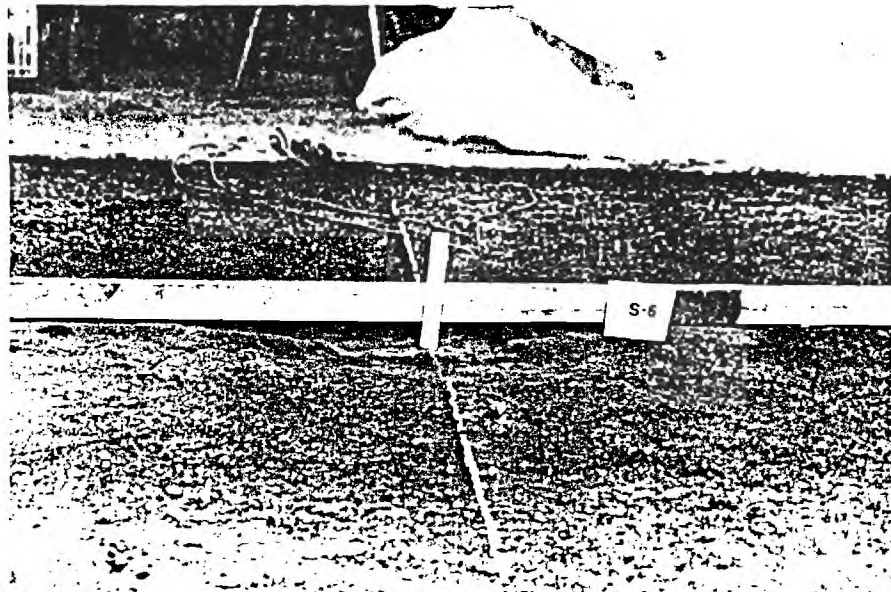


FIGURE 62. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 6.



(a) Section 5 After 200,000 Repetitions: 9 in. Full-Depth AC.



(b)

FIGURE 63. SURFACE PROFILE OF SECTION 5 AND 6 AFTER TEST TERMINATION.

test sections in rutting appears to be due to (1) application of the heavy loading in a reasonably concentrated pattern, (2) a slightly high asphalt content, and (3) the relatively thin asphalt concrete layer resulted in important contributions of rutting from the somewhat weak subgrade.

Distribution of Permanent Deformation. The distribution of permanent deformation for Section 5 is shown in Fig. 65 and for Section 6 in Fig. 66. In both sections permanent deflection in the asphalt concrete decreased from the surface downward almost linearly with depth. For the relatively thin asphalt concrete layers used in these sections, permanent deformation in the subgrade accounted for about one-half of the total rutting. Further, most of the subgrade rutting occurred in the upper 12 in. (305 mm). Fig. 66 shows the distribution of permanent deformation for load position 5 and not the primary load position. Load position 5 is shown because sufficient information was not available to give results for position 1 because of problems with several strain sensors.

Resilient Response. The resilient response of Sections 5 and 6 is summarized in Table 15. For Section 5 the measured horizontal tensile strain in the bottom of the asphalt concrete was about 319×10^{-6} in./in. and the vertical strain at the top of the subgrade was 1.38×10^{-3} in./in. The variation with number of load repetitions of resilient horizontal tensile strain in the bottom of the asphalt concrete and vertical strain on top of the subgrade is shown in Fig. 67. These results suggest a gradual increase in resilient horizontal tensile strain in the bottom of the asphalt concrete until rut depths of about 0.75 to 1.0 in. (19-25 mm). After this the tensile strain appeared to decrease perhaps due to micro-cracking in the asphalt concrete. Vertical strain on the top of the subgrade increased with number of load repetitions to approximately failure as also shown in Fig. 67. The resilient vertical pressure measured on top of the subgrade was 8.7 psi (60.0 kN/m²). This relatively high magnitude of vertical stress appears realistic since deformation in the subgrade was large.

For Section 6 the resilient horizontal tensile strain in the bottom of the asphalt concrete varied from 460×10^{-6} in./in. at 1,000 repetitions to 633×10^{-6} at 126,000 repetitions. The resilient vertical strain on top of the subgrade was 1.41×10^{-3} in./in., and the dynamic vertical pressure on top of the subgrade was 10.9 psi (75.1 kN/m²).

The variation of resilient surface displacement for Sections 5 and 6 is given as a function of number of load repetitions in Figs. 68 and 69, respectively. The initial resilient deflection just outside the edge of the loaded area was 0.0096 in. (0.24 mm) for Section 5 and 0.014 in. (0.36 mm) for Section 6. These resilient deflections tended to increase at least up to failure showing a general weakening of the pavement.

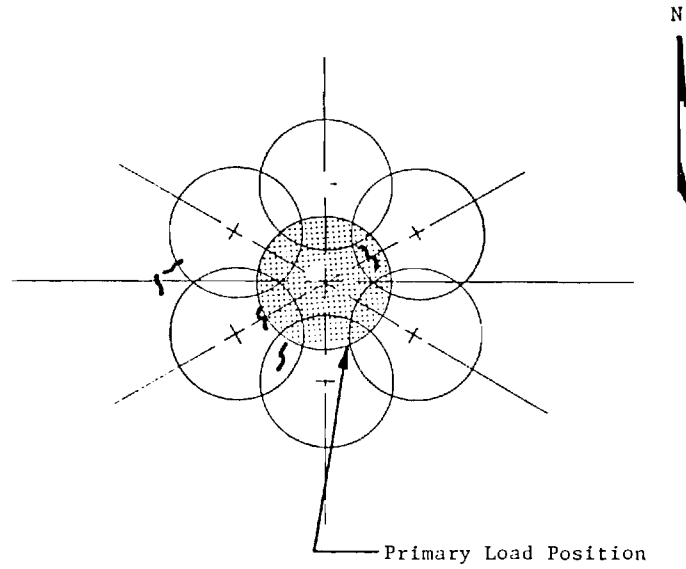


FIGURE 64. CRACK PATTERN IN SECTION 6 AT COMPLETION OF TEST: 6.5 IN. FULL-DEPTH A.C. SECTION.

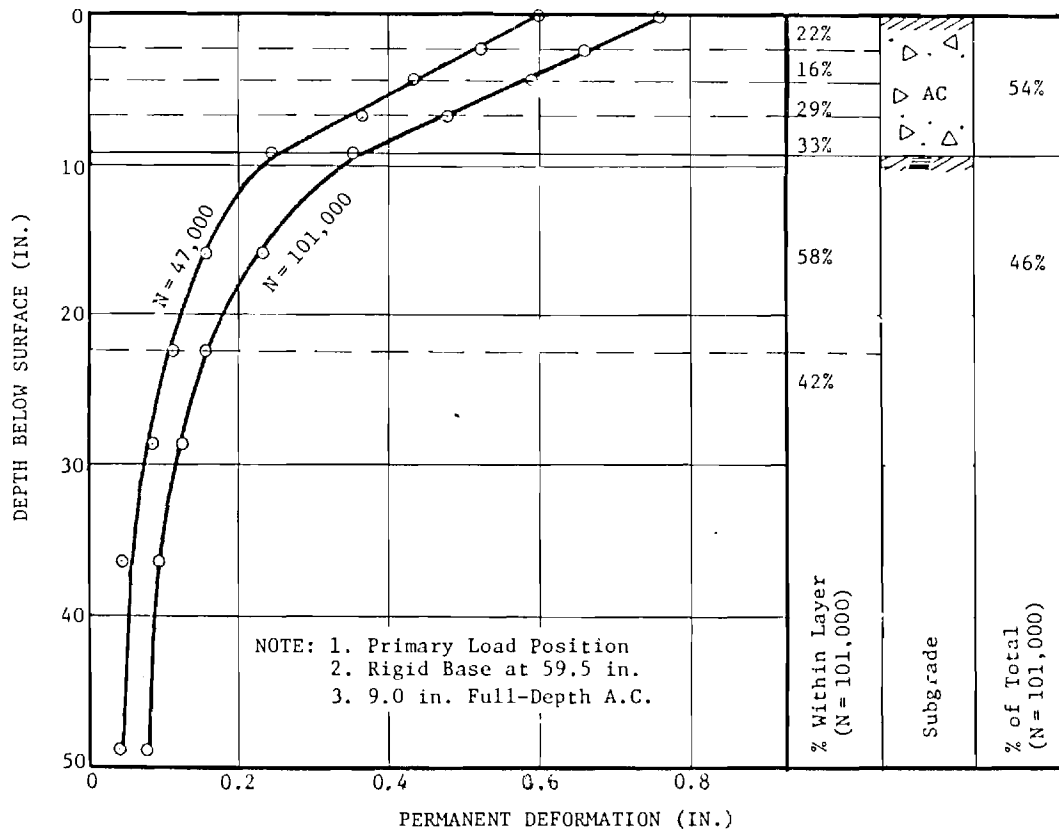


FIGURE 65. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 5.

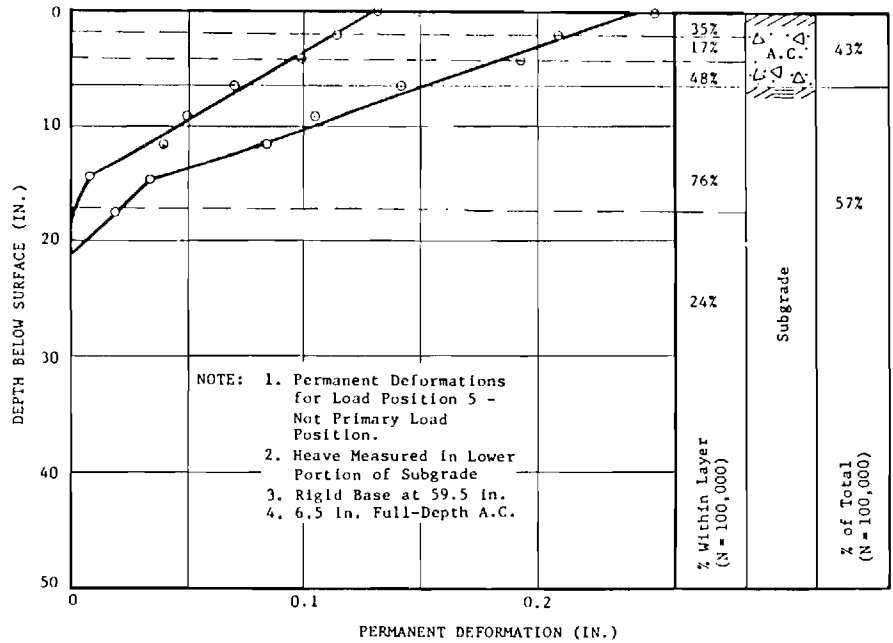


FIGURE 66. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 6.

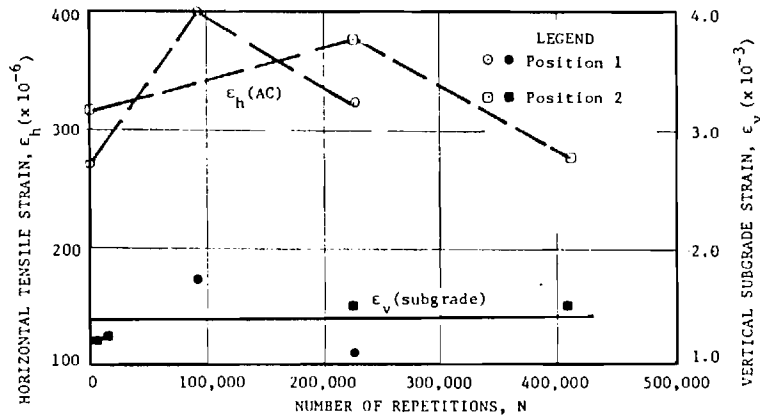


FIGURE 67. VARIATION OF RESILIENT HORIZONTAL STRAIN IN BOTTOM OF AC AND VERTICAL STRAIN IN TOP OF SUBGRADE WITH NUMBER OF LOAD REPETITIONS - SECTION 5.

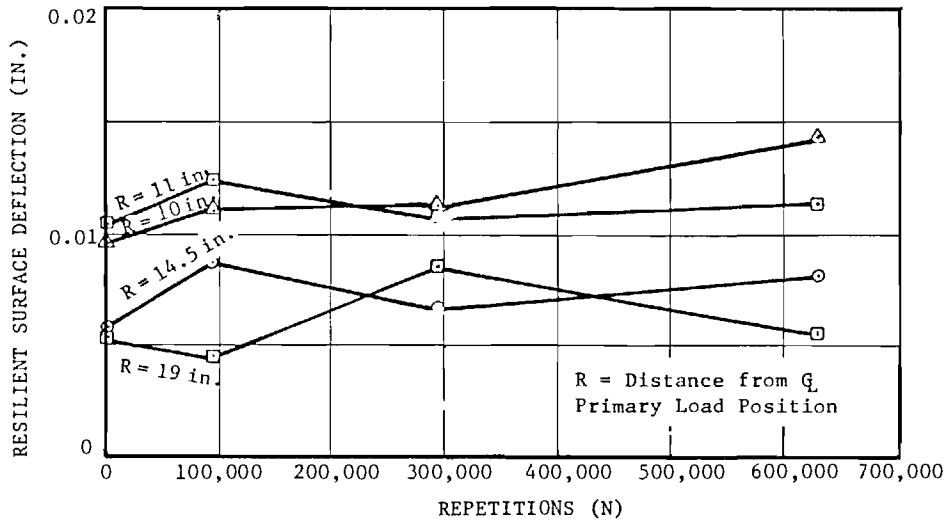


FIGURE 68. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 5.

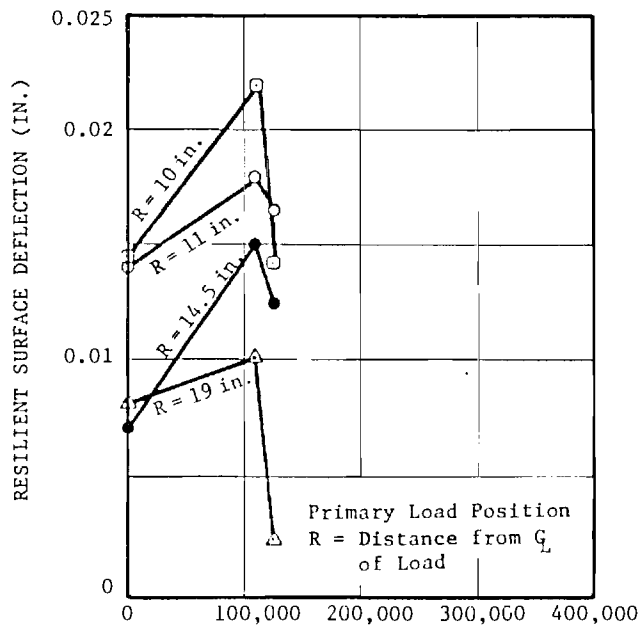


FIGURE 69. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 6.

Sections 7 and 8

Sections 7 and 8 were constructed to directly compare the performance of a full-depth asphalt concrete pavement (Section 7) with a crushed stone base pavement (Section 8). Section 7 consisted of 7.0 in. (178 mm) of full-depth asphalt concrete. Section 8 consisted of 3.5 in. (89 mm) of asphalt concrete overlying an 8 in. (203 mm) crushed stone base. The 1.5 in. (38 mm) maximum size crushed stone base had the standard gradation consisting of 83% passing the 3/4 in. (19 mm) sieve, 48% passing the No. 4 sieve, 16% passing the No. 40 sieve, and 4.8% passing the No. 200 sieve.

Sections 7 and 8 were constructed at the same time (i.e., during one filling of the pit) to minimize the influence of differences in the subgrade and asphalt concrete used in the sections. The modified-B binder used in these sections was obtained from the Lithia Springs plant of APAC-Georgia, Inc. The asphalt concrete used to cover the crushed stone base in Section 8 was from the same batch and was placed at the same time as the asphalt concrete in the upper two lifts of the full-depth section. Therefore, in these two sections the material properties of the subgrade and the upper two lifts of asphalt concrete for purposes of analysis should have essentially the same physical properties.

Rutting. The full-depth asphalt concrete pavement (Section 7) failed in rutting after 150,000 load repetitions with a rut depth of 0.5 in. (12 mm) having developed. The crushed stone pavement (Section 8) failed in rutting after 550,000 load repetitions. In Section 8, 2.3 in. (58 mm) of crushed stone replaced 1 in. (25 mm) of asphalt concrete base from Section 7. Since the crushed stone section withstood 3.7 times more load repetitions, the base course equivalency for similar performance observed in these experiments was considerably less than 2.3.

The variation of rut depth beneath the center of the primary load position for Section 7 is shown in Fig. 70 and for Section 8 in Fig. 71. Rut depth accumulation at a distance of 9 in. (229 mm) from the center of the primary load position is given in Figs. 72 and 73 for these two sections. Figs. 74 and 75 show the deflection profile at selected numbers of load repetitions. Fig. 76 pictorially compares the surface condition of the two sections after 1 million repetitions. Surface cracking was not observed in either Section 7 or Section 8.

The average asphalt content obtained from five cores taken from these sections was 5.1%. Three cores had asphalt contents reported to vary from 3 to 3.6%. Since the cores tested were only about 1.5 in. (38 mm) thick, the low asphalt contents were assumed to be erroneous and not included in the average. The possibility does exist that the asphalt content in Section 7 may have been slightly higher than Section 8. Any significant difference in asphalt content between sections, however, is not considered probable because all the asphalt came from the same batch (for two lifts) and from the same plant and job for the other two lifts in the full-depth section. The more probable explanation is due to experimental error in the recovery tests performed on small specimens. The average density of the five cores was 144.9 pcf (22.7 kN/m³) which was 99% of the maximum Marshall density.

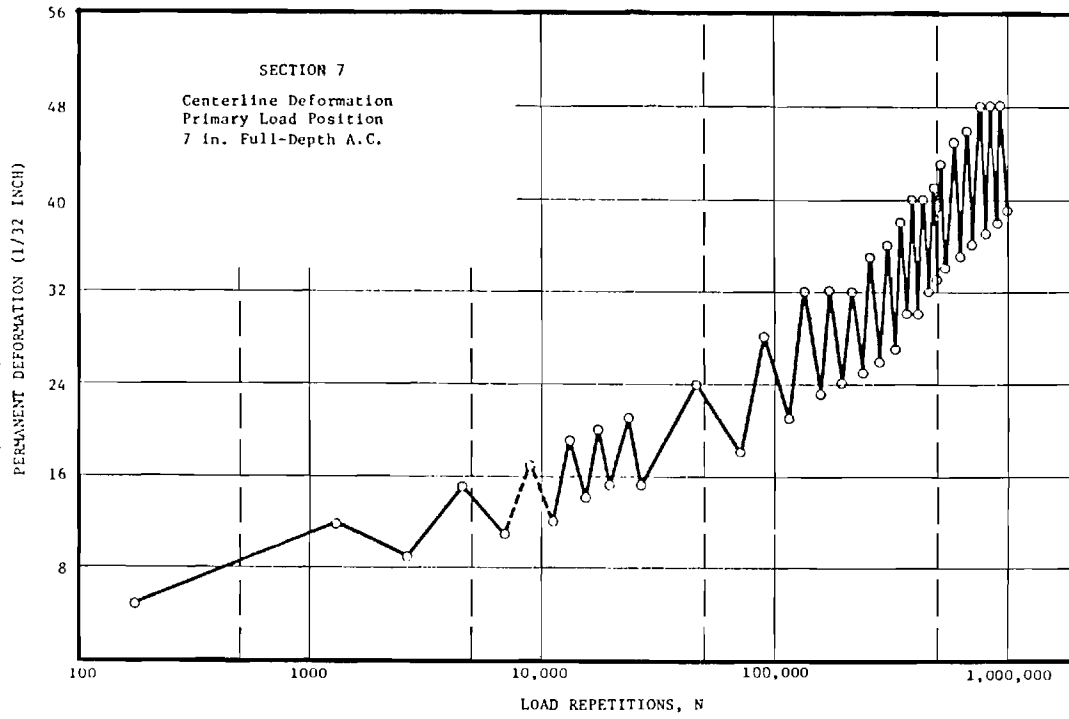


FIGURE 70. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 7.

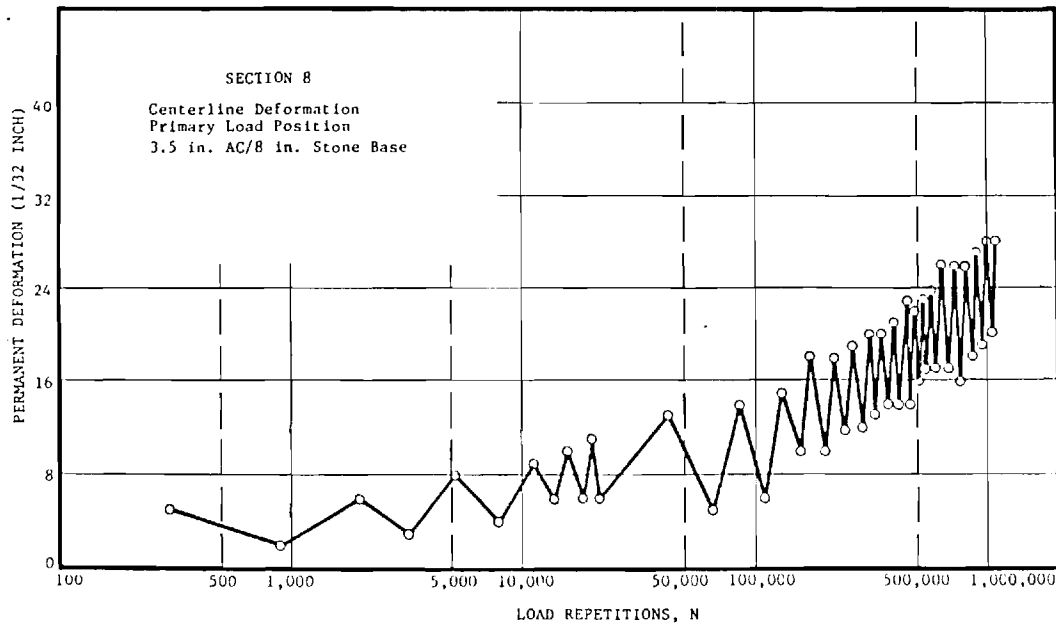


FIGURE 71. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 8.

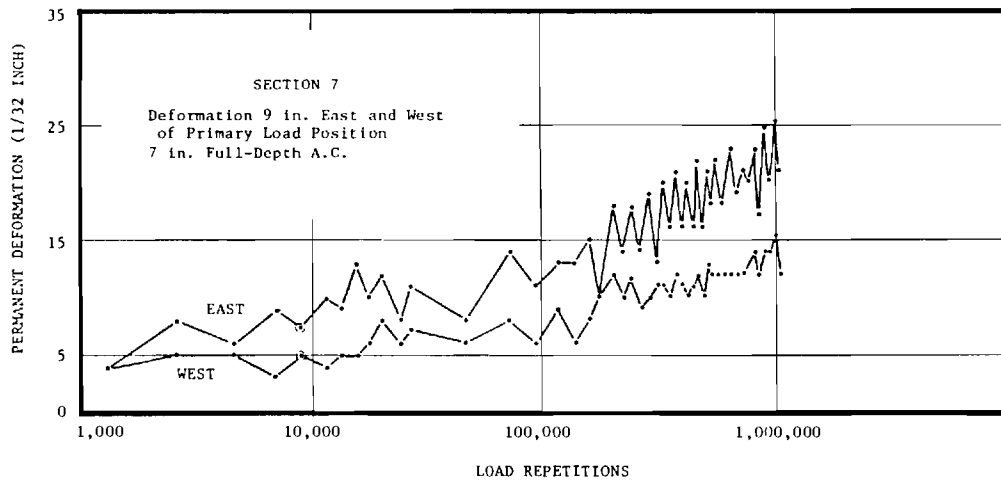


FIGURE 72. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 7.

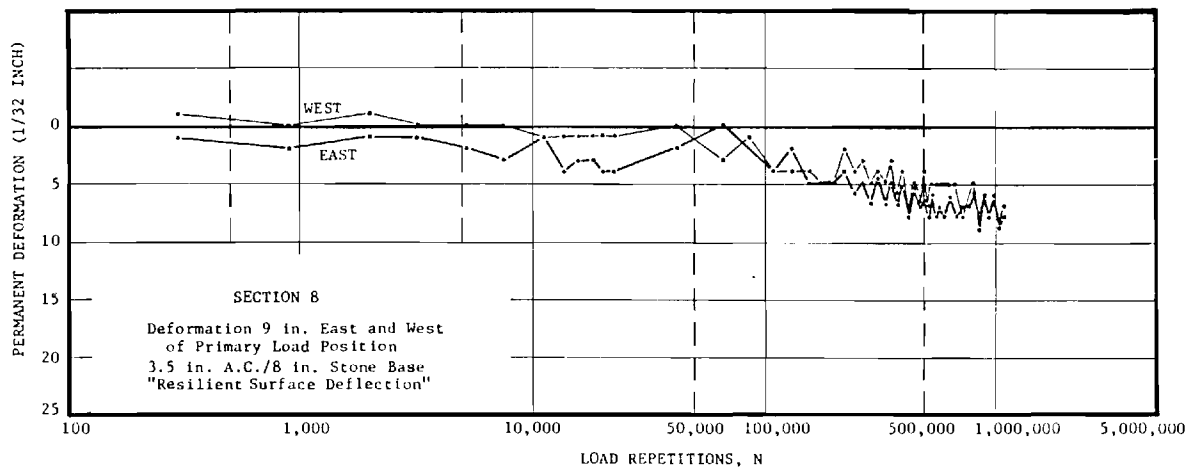


FIGURE 73. VARIATION OF DEFORMATION 9 IN. FROM CENTERLINE WITH NUMBER OF LOAD REPETITIONS - SECTION 8.

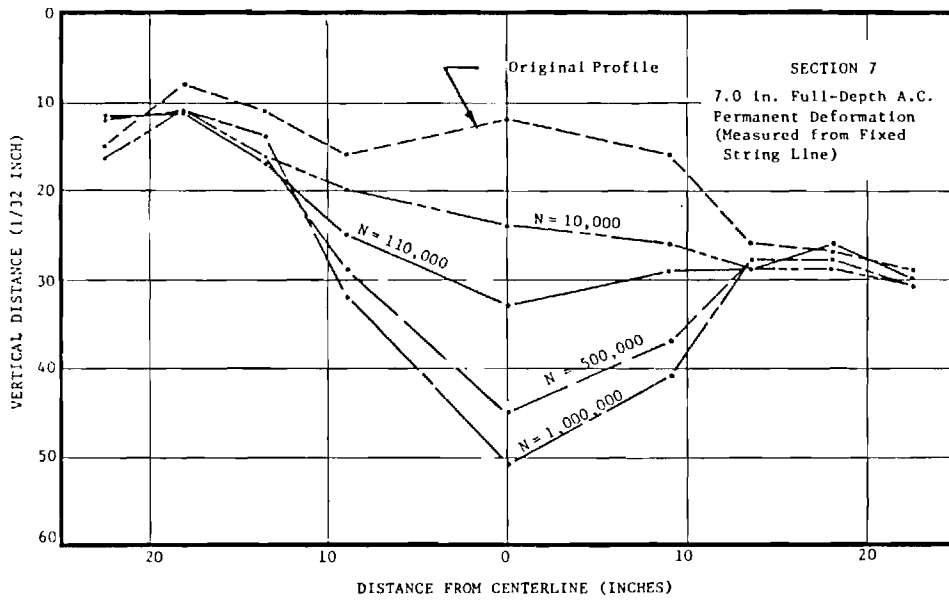


FIGURE 74. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 7.

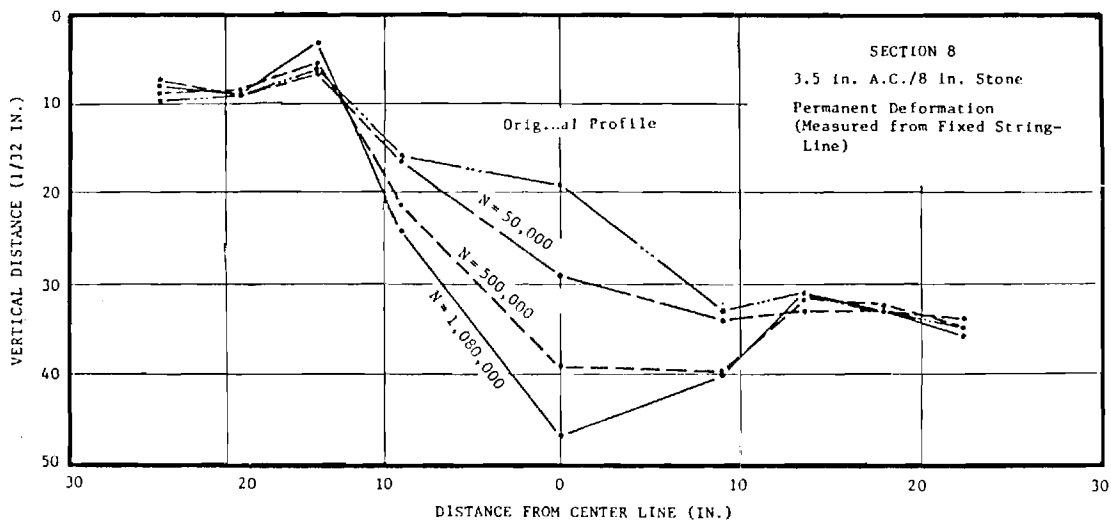
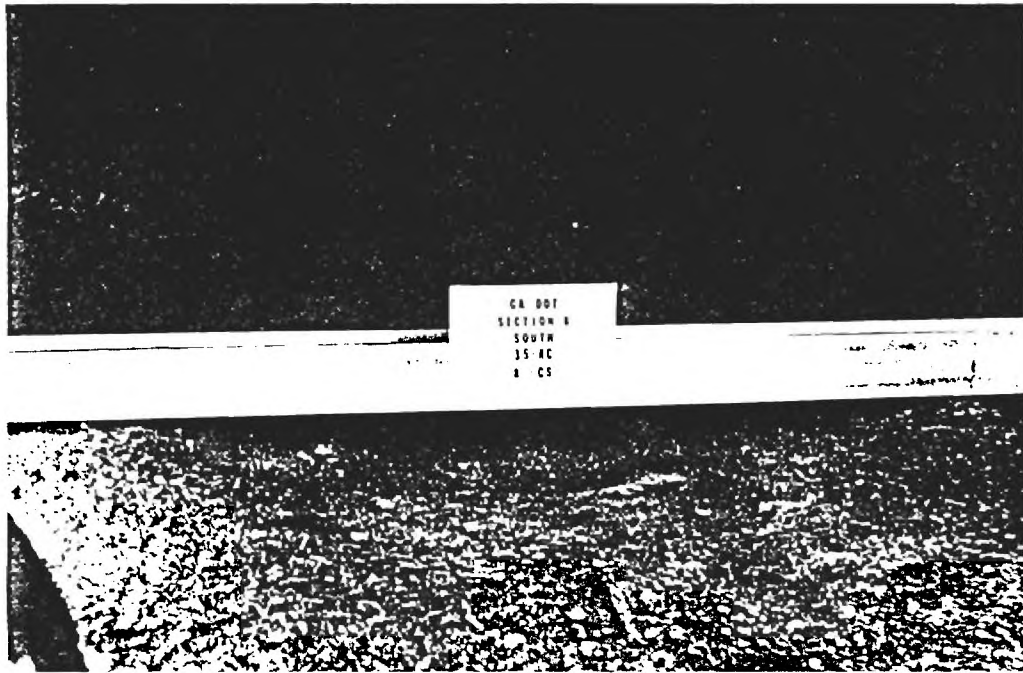
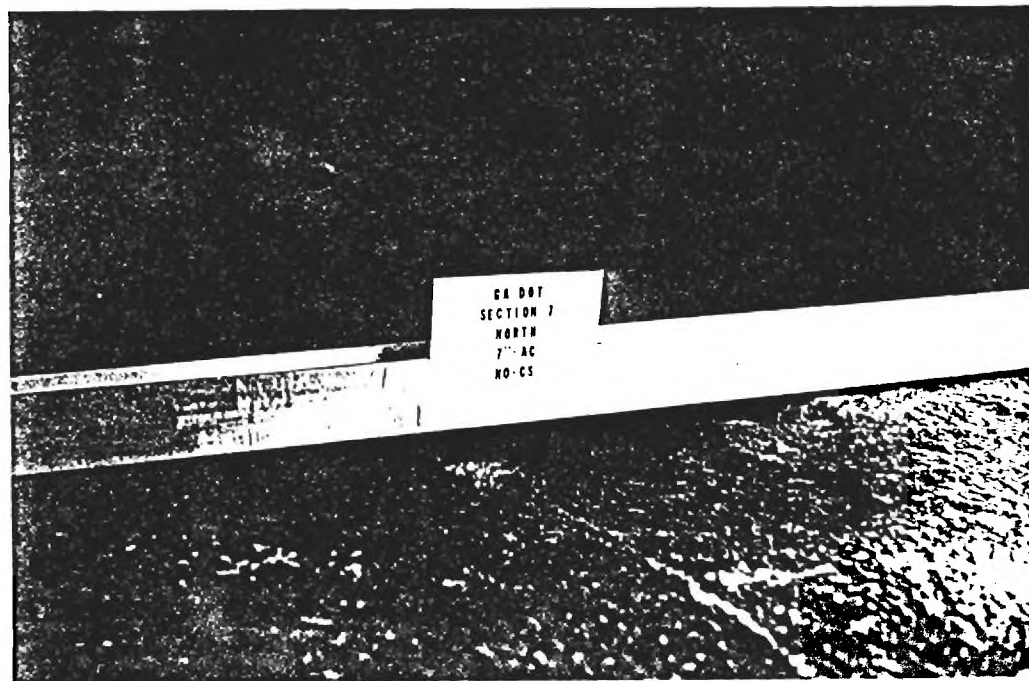


FIGURE 75. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 8.



(a) Section 7: 7 in. Full-Depth AC (110,000 Repetitions)



(b) Section 8: 3.5 in./8 in. Stone (1 Million Repetitions)

FIGURE 76. SURFACE DEFLECTION PROFILE AFTER TEST TERMINATION.

The relatively early failure of the full-depth section in rutting compared with the crushed stone base section appears to be due to (1) application of the heavy loading in a reasonably concentrated pattern, (2) important contributions of rutting from the weak subgrade beneath the relatively thin, full-depth asphalt concrete (the subgrade was located closer to the surface than the crushed stone section and apparently as a result had a larger contribution to rutting), and (3) the full-depth section *may* have had a slightly higher asphalt content than the crushed stone section.

Distribution of Permanent Deformation. The distribution of permanent deformation for Sections 7 and 8 is compared in Fig. 77 after approximately 300,000 load repetitions. Once again, in both sections permanent deformation in the asphalt concrete decreased almost linearly with depth. In the full-depth asphalt concrete section 67% of the total permanent deformation occurred in the asphalt concrete and 33% in the subgrade. In the crushed stone section, 55% of the permanent deformation occurred in the 3.5 in. (89 mm) thick asphalt concrete surfacing, 10% in the crushed stone and 35% in the subgrade. In the asphalt concrete layers of both sections the permanent deformation was approximately equally distributed between the top and bottom half of the asphalt concrete layer. Also of considerable interest is the finding that at equal depths more permanent rutting occurred in the upper part of the subgrade beneath the full-depth asphalt concrete section than in the crushed stone section (refer to Fig. 77).

Resilient Response. A general summary of the resilient response of all test sections is given in Table 15. The response of Section 7 is directly compared with Section 8 in Table 17 for selected numbers of load repetitions. The full-depth pavement (Section 7) had for approximately the first 100,000 load repetitions a typical measured horizontal tensile strain in the bottom of the asphalt concrete of about 410×10^{-6} in./in. and a vertical strain at the top of the subgrade of about 2.2×10^{-3} in./in. In comparison, in the crushed stone base section the measured horizontal tensile strain in the bottom of the asphalt concrete was about 300×10^{-6} in./in. and the vertical strain at the top of the subgrade was about 1.85×10^{-3} in./in. The resilient vertical pressure measured at the top of the subgrade was about 12.9 psi (88.9 kN/m²) for Section 7 and 11.9 psi (82.0 kN/m²) for Section 8.

For Section 7 the variation as a function of number of load repetitions of the resilient horizontal tensile strain in the asphalt concrete at selected locations is shown in Fig. 78; variations of vertical strain in the top of the subgrade and in the asphalt concrete are shown in Fig. 79. Similar relationships for Section 8 for strain are presented in Figs. 80 through 82. Once again, the resilient strains observed in Sections 7 and 8 indicate a gradual increase with load repetitions in resilient horizontal and vertical strain in the bottom of the asphalt concrete at least until about failure.

TABLE 17. SUMMARY COMPARISON BETWEEN FULL-DEPTH AC SECTION 7 (7AC) AND CRUSHED STONE SECTION 8 (3.5 AC/8CS)⁽¹⁾.

Repetition	Section Number	Resilient δ_v ⁽²⁾ (in.)	Avg. Rut Depth ⁽⁵⁾ (in.)	Vert. ⁽³⁾ Subgrade Strain (in./in. x 10 ⁻³)	Horiz. Tensile ⁽⁴⁾ Strain in AC (in./in. x 10 ⁻⁶)	Vert. Stress @ Top Subgrade
1,000	7(7AC)	0.0126	0.30	1.90	420	12.2
	8(3.5/8)	0.0112	0.12	1.73	270	11.7
10,000	7(7AC)	0.0126	0.45	2.08	410	13.6
	8(3.5/8)	0.0166	0.21	1.77	270	12.1
110,000	7(7AC)	0.0158	0.78	2.43	372	15.2
	8(3.5/8)	0.0148	0.33	1.94	295	12.6
500,000	7(7AC)	0.0204	1.16	2.79	268	16.1
	8(3.5/8)	0.0154	0.59	2.37	330	12.9

- Note:
1. Sections 7 and 8 were constructed at the same time using the same asphalt concrete.
 2. Resilient vertical surface deflection at load position 1 located 11 in. (279 mm) from the center of the load (6.45 in., 164 mm from the edge of the load).
 3. Vertical strain in top of subgrade immediately beneath loading.
 4. Horizontal tensile strain in bottom of AC immediately beneath loading.
 5. Average measured rut depth beneath the primary load position; not corrected for rutting occurring before 1,000 repetitions.

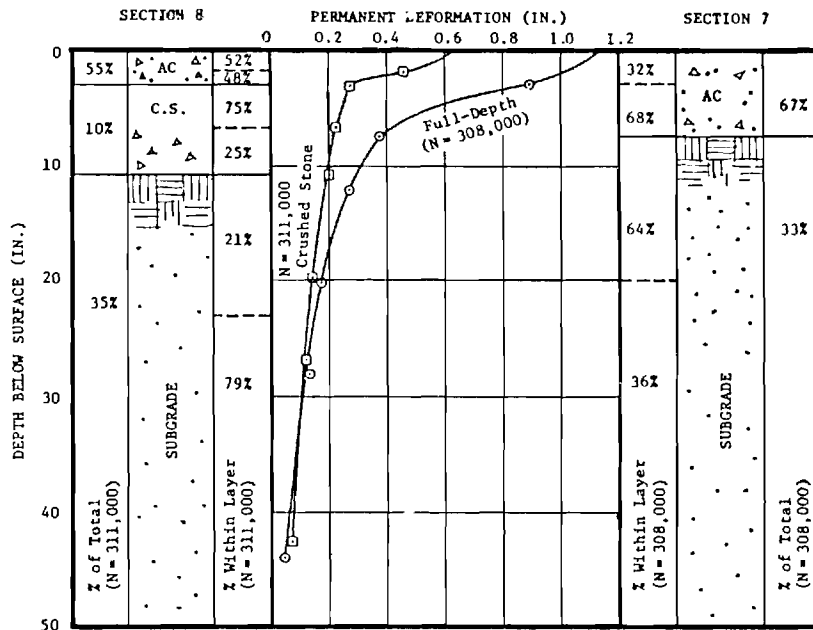


FIGURE 77. COMPARISON OF DISTRIBUTION OF PERMANENT DEFORMATION IN FULL-DEPTH SECTION 7 AND CRUSHED STONE SECTION 8.

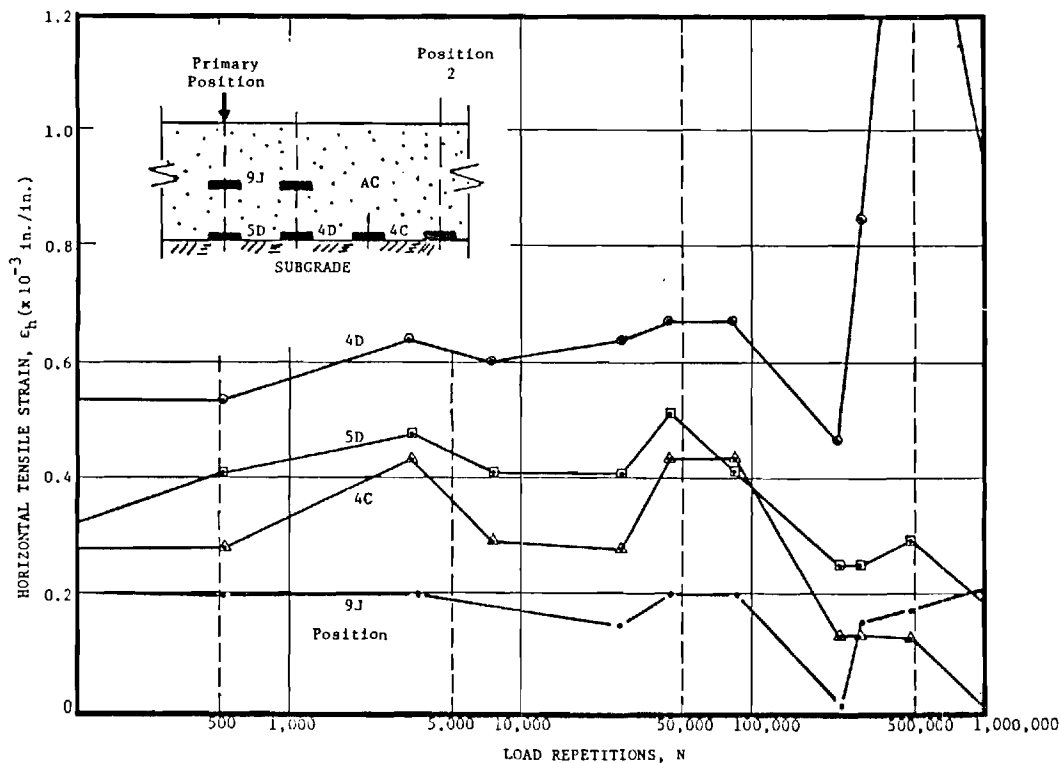


FIGURE 78. VARIATION OF RESILIENT HORIZONTAL STRAIN WITH LOAD REPETITIONS WITHIN THE ASPHALT CONCRETE - SECTION 7.

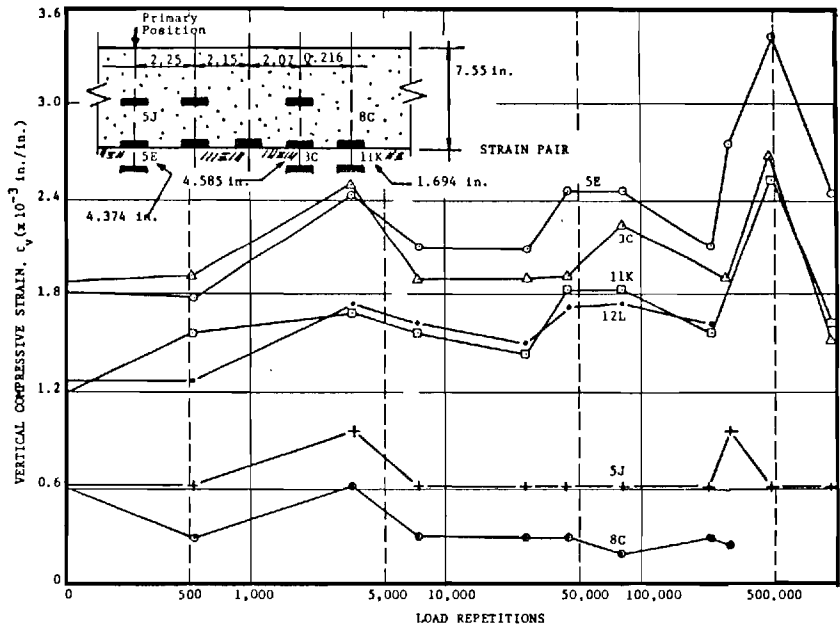


FIGURE 79. VARIATION OF VERTICAL RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 7.

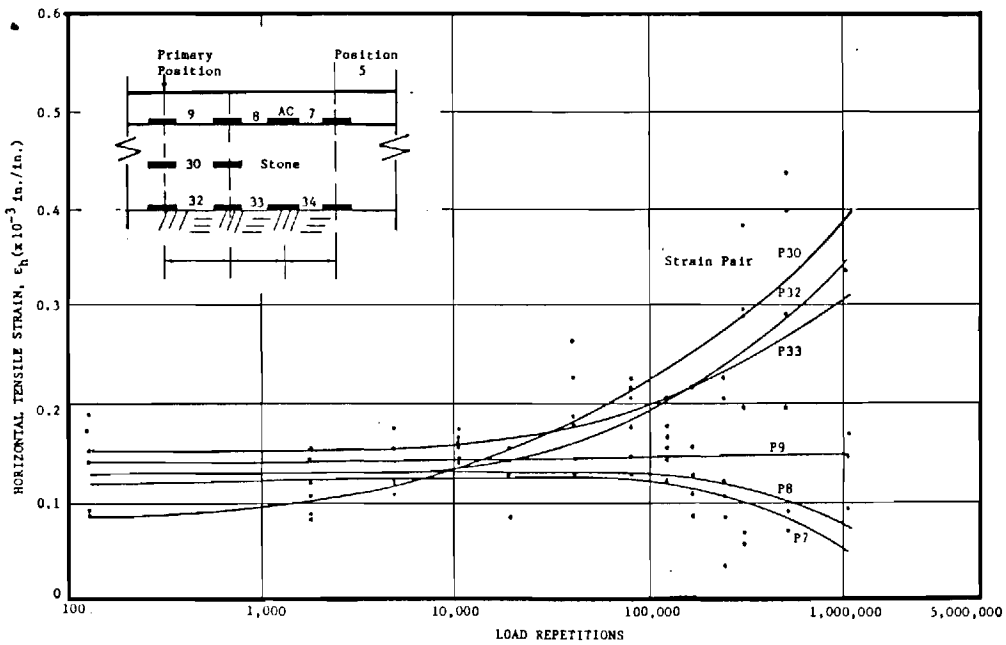


FIGURE 80. VARIATION OF RESILIENT HORIZONTAL STRAIN WITH LOAD REPETITIONS - SECTION 8

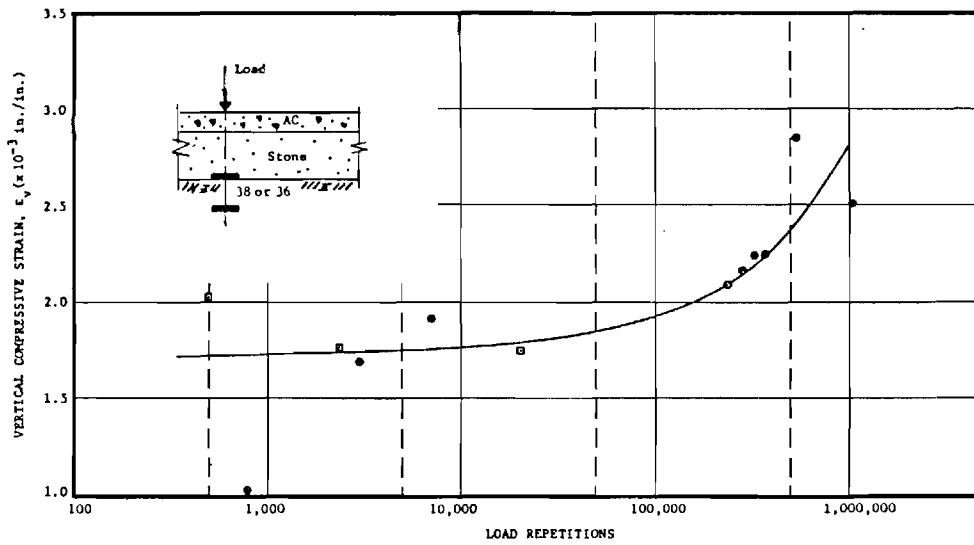


FIGURE 81. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD - SECTION 8.

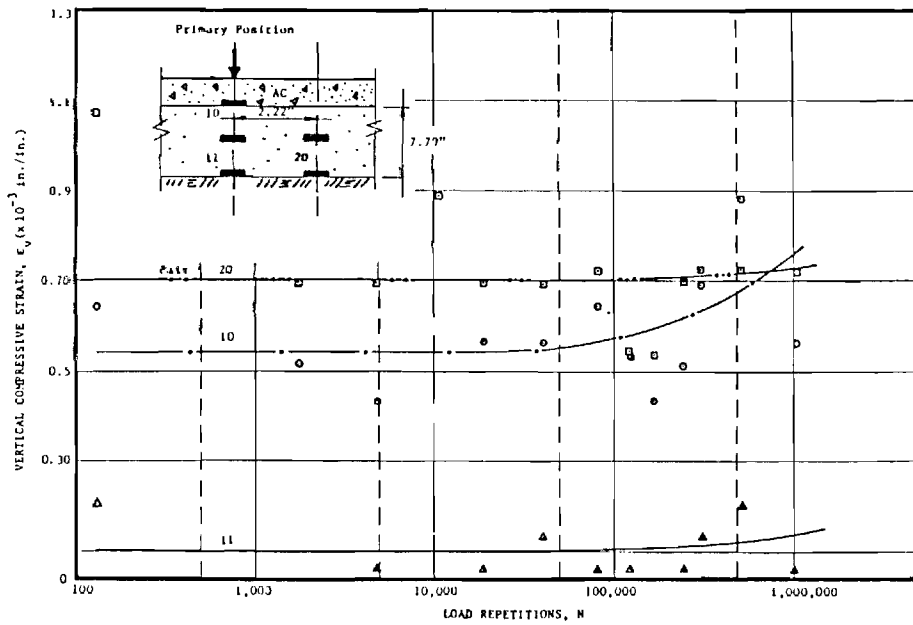


FIGURE 82. VARIATION OF RESILIENT VERTICAL STRAIN WITHIN CRUSHED STONE WITH NUMBER OF LOAD REPETITIONS - SECTION 8.

Sections 9 and 10

Sections 9 and 10 were constructed to compare use of a coarser and finer gradation crushed stone base than the standard gradation used in Sections 1, 2 and 8. Sections 9 and 10, which were identical except for the gradation of the crushed stone base, consisted of 3.5 in. (89 mm) of asphalt concrete B-binder and 8 in. (203 mm) of crushed stone base. The asphalt concrete B-binder was obtained from the Forest Park plant of APAC-Georgia, Inc.

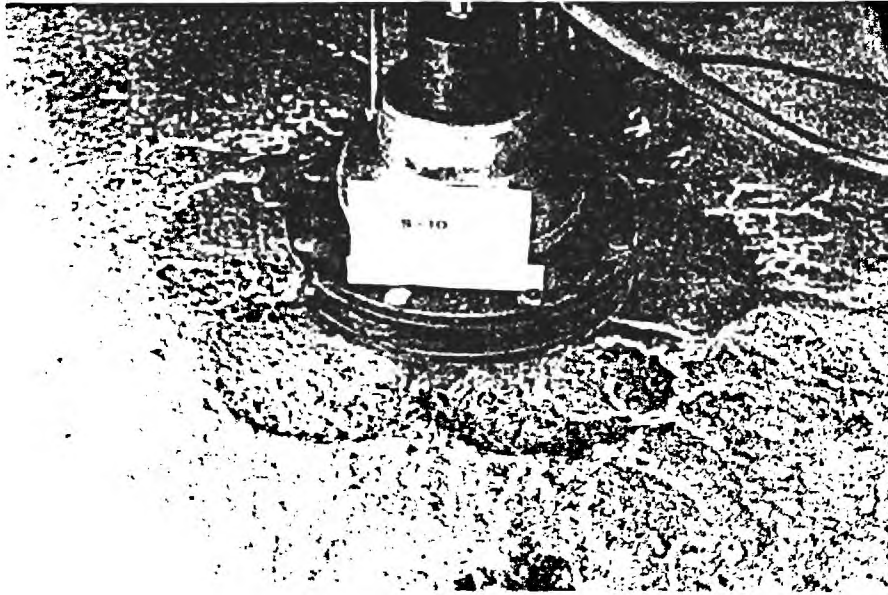
Section 9 was the coarse gradation crushed stone base pavement having a 2 in. (51 mm) top size, 83% passing the 1 in. (25 mm) sieve, 60% passing the 3/4 in. (19 mm) sieve, 40% passing the No. 4, 15% passing the No. 40 sieve and 4.2% passing the No. 200 sieve. The finer gradation, 1.5 in. (38 mm) crushed stone used in Section 10 had 99% passing the 1 in. (25 mm) sieve, 92% passing the 3/4 in. (19 mm) sieve, 44% passing the No. 4 sieve, 15% passing the No. 40 and 4.4% passing the No. 200 sieve. Therefore, the difference in these two crushed stone gradations is only between the 2 in. (51 mm) and No. 4 sieve size; below the No. 4 sieve the gradation is essentially the same.

At 2 million repetitions the loading applied to the pavement was increased from 6,500 lbs. (28.9 kN) to 7,500 lbs. (33.4 kN) for the remainder of the test. An oil spill occurred in Section 10 at 546,400 load repetitions that covered load positions 3 through 6 for a short period of time. Apparently the oil spill did not significantly effect the life of this section.

Fatigue. Both Sections 9 and 10 failed by fatigue cracking of the asphalt concrete B-binder. Section 9, which had the coarse gradation base, failed in fatigue after approximately 2.4 million load repetitions. Cracking began in this section after approximately 2 million load repetitions and rapidly propagated leading to the initiation of Class 2 cracking considered as a failure condition in this study.

Section 10, which had the fine gradation crushed stone base, failed in fatigue (initiation of Class 2 cracking) after approximately 2.9 million load repetitions. The cracking patterns present in Sections 9 and 10 after 2.9 million repetitions are compared in Fig. 83. Almost all of the cracking occurred outside of the loaded area. In both sections only hairline cracks developed which were hard to see. Therefore to illustrate the pattern, the cracks were painted white to be visible in the pictures.

The measured resilient horizontal tensile strain in the bottom of the asphalt concrete beneath the primary load in Section 9 was 270×10^{-6} in./in. and in Section 10 was 390×10^{-6} in./in. Nevertheless, Section 10 which had the higher resilient tensile strain performed better with respect to fatigue than Section 9.



(a) Section 10: 3.5 in./8.0 in. (Fine Gradation)



(b) Section 9: 3.5 in./8.0 in. (Coarse Gradation)

FIGURE 83. FATIGUE CRACKING IN SECTIONS 9 AND 10 AFTER ONE MILLION LOAD REPETITIONS.

rutting. The coarse crushed stone base pavement (Section 9) rutted 0.28 in. (7.2 mm) after 2.8 million load repetitions. The fine gradation crushed stone base pavement (Section 10) rutted 0.34 in. (8.7 mm) after 2.8 million load repetitions. Therefore, the section having the larger top size, coarser base stone underwent somewhat less rutting. The variation with load repetitions of rut depth beneath the center of the primary load position for Section 9 is shown in Fig. 84 and for Section 10 in Fig. 85. Figs. 86 and 87 show the deflection profile of these sections at selected numbers of load repetitions.

Distribution of Permanent Deformation. The distribution of permanent deformation in Sections 9 and 10 is compared in Fig. 88 after 3 million load repetitions. Permanent deflection through the asphalt concrete for both sections decreased from the surface downward. For these sections, in contrast to the previous sections, the variation of permanent deflection through the asphalt concrete was not linear. Although surface rutting was greater in Section 10 which had the finer gradation base, the relative distribution of rutting in the two sections was almost identical (i.e., the shape of the pavement deformation profiles shown in Fig. 88 were similar). In both sections about 20% of the total permanent deformation occurred in the asphalt concrete, 12% in the crushed stone base and 68% in the subgrade. The surfacing and base of these sections were, based on the large number of repetitions required to fail the sections, high quality and hence did not undergo large amounts of permanent deformation. As a result a large relative amount of the total permanent deformation occurred in the subgrade which was not adequately protected by the 11.5 in. (292 mm) thick structural section.

In contrast to the full-depth pavement (Section 7), in Sections 9 and 10 about 70% of the permanent deformation occurred in the top of the asphalt concrete and only 30% in the bottom. More permanent deformation also occurred in the top portion of the crushed stone layer (64%) than in the bottom (36%). These findings suggest that the distribution of rutting is affected not only by the thickness of the layers but also by the quality of materials.

Resilient Response. A general summary of the resilient response of all test sections including 9 and 10 is given in Table 15. The typical resilient strain response for Section 9 as a function of load repetitions is shown in Figs. 89 through 93 and for Section 10 in Figs. 94 through 97.

The horizontal tensile strain measured in the bottom of the asphalt concrete was 280×10^{-6} in./in. in Section 9 which had the large top size crushed stone base and 400×10^{-6} in Section 10 which had the smaller top size crushed stone base. Similar tensile strains were determined from two different strain sensor pairs in each section. Therefore the measured horizontal tensile strains should indicate the true relative tensile strains that existed in these sections.

The horizontal tensile strain in the bottom of the crushed stone obtained from the strain sensor readings in Section 9 was 1.08×10^{-3} in./in., and the vertical compressive strain in the bottom of the stone

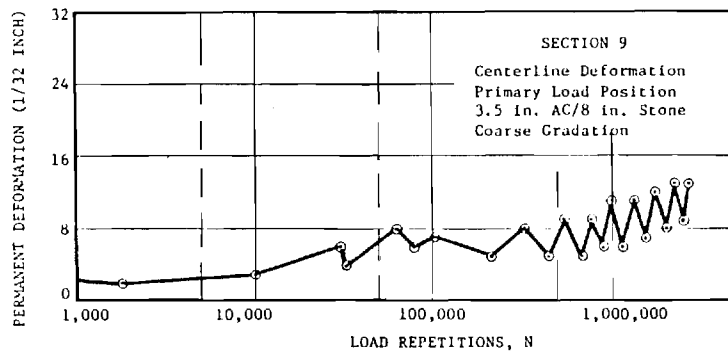


FIGURE 84. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 9.

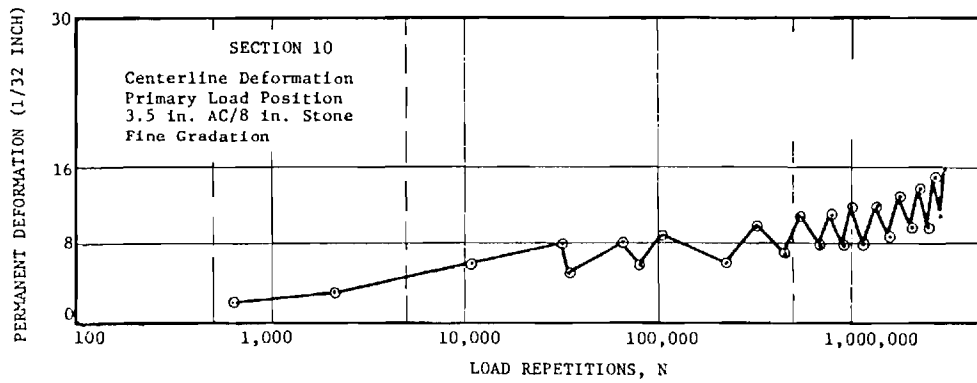


FIGURE 85. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 10.

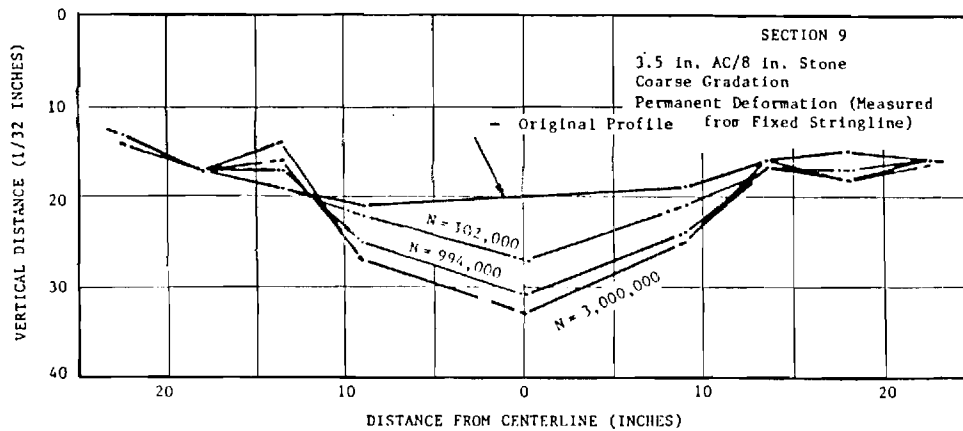


FIGURE 86. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 9.

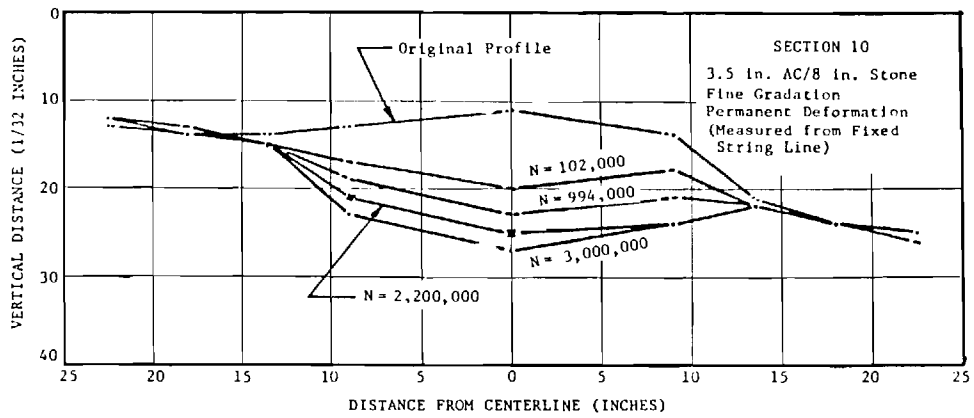


FIGURE 87. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 10.

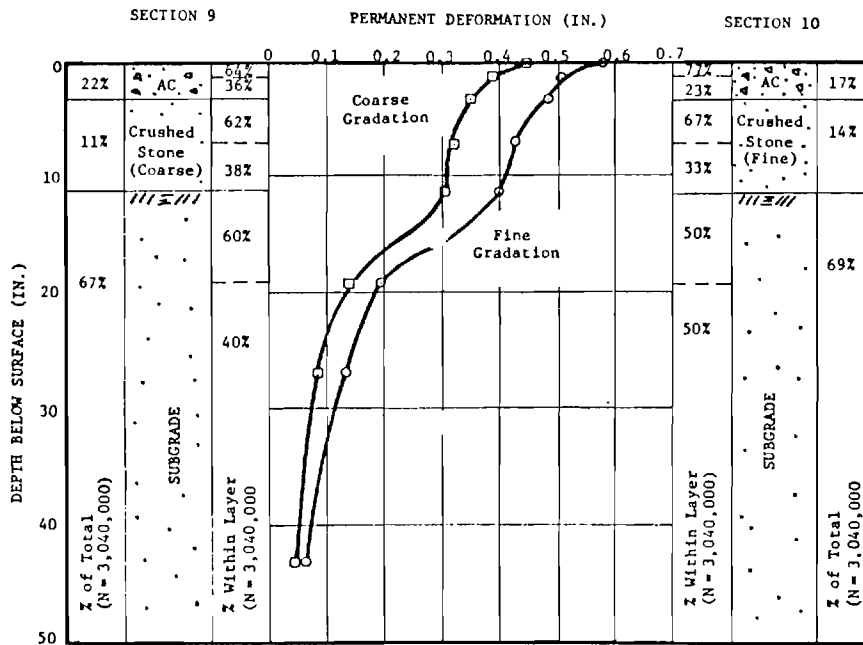


FIGURE 88. COMPARISON OF DISTRIBUTION OF PERMANENT DEFORMATION IN COARSE AND FINE GRADATION CRUSHED STONE BASE PAVEMENTS - SECTIONS 9 AND 10.

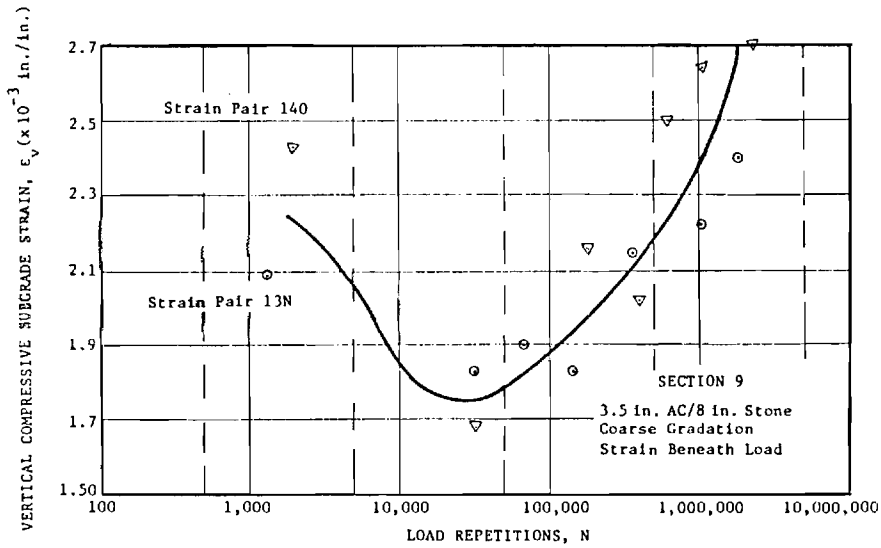


FIGURE 89. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 9.

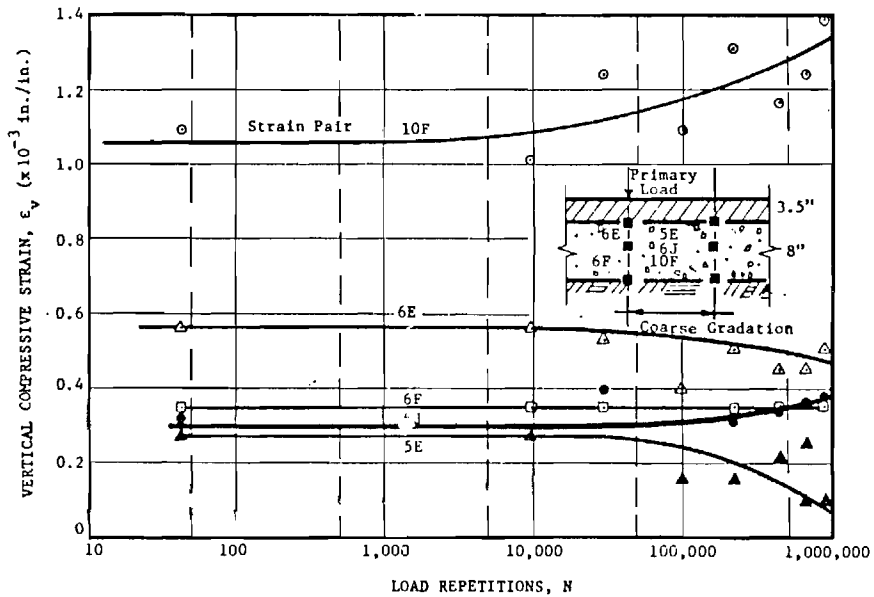


FIGURE 90 AND 91. VARIATION OF RESILIENT VERTICAL STRAIN IN CRUSHED STONE BASE WITH LOAD REPETITIONS FOR LOAD IN PRIMARY POSITION - SECTION 9.

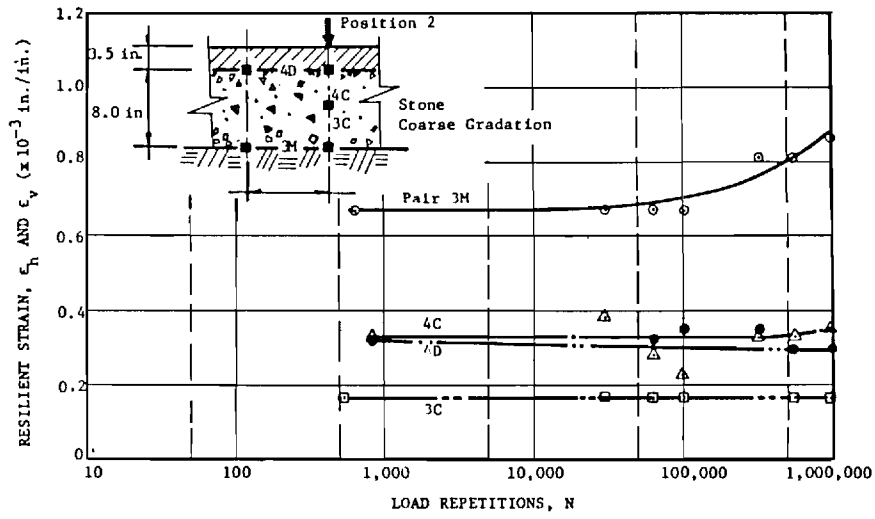


FIGURE 92 AND 93. VARIATION OF RESILIENT HORIZONTAL AND VERTICAL STRAINS WITH LOAD REPETITIONS FOR LOADING AT POSITION 2 - SECTION 9.

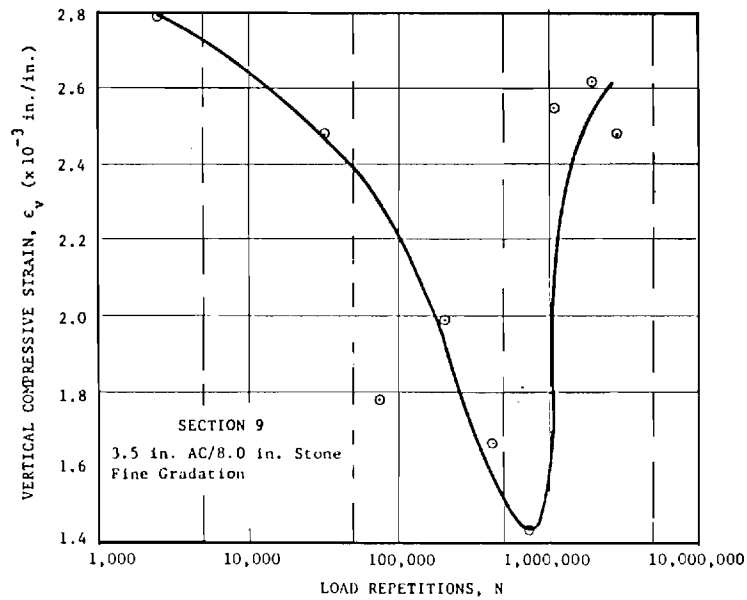


FIGURE 94 & 95. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 10.

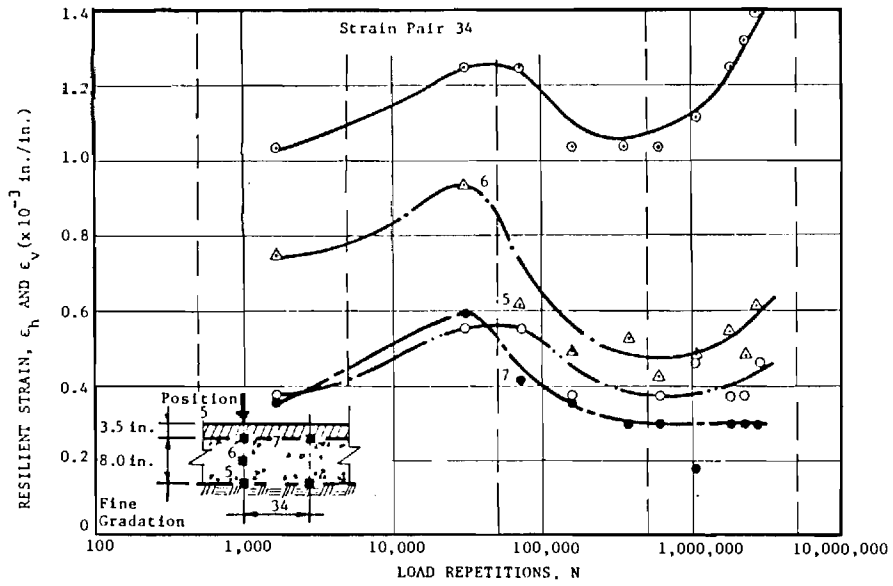


FIGURE 96. VARIATION OF RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 10.

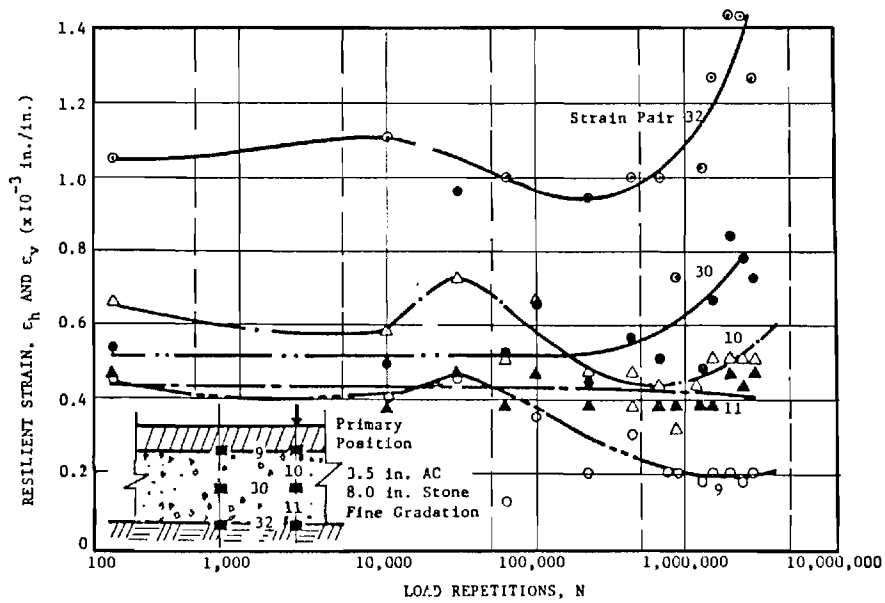


FIGURE 97. VARIATION OF RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 10.

was about 0.34×10^{-3} in./in. In Section 10 the horizontal tensile strain in the bottom of the crushed stone was 1.0×10^{-3} in./in., and the vertical strain in the bottom of the crushed stone was about 0.40×10^{-3} in./in.

The measured vertical strain on top of the subgrade for Section 9 was 1.75×10^{-3} in./in. with a corresponding vertical stress of 11.1 psi (76.5 kN/m^2). In Section 10 the vertical subgrade strain was 2.5×10^{-3} in./in., and the subgrade stress was 6.8 psi (44.9 kN/m^2). The measured vertical subgrade stress of 6.8 psi (44.9 kN/m^2) is believed to be low since it is considerably less than the vertical stresses measured on the subgrade for the other conventional crushed stone base sections of similar thickness.

Section 10 underwent more rutting than Section 9. Based on the strain sensor measurements, about 75% of the increase in rutting in Section 10 compared with Section 9 occurred in the subgrade, with the remaining 25% taking place in the base. Of significance is the finding that the measured vertical resilient strains in both the base and subgrade were larger in Section 10 than Section 9. Also the vertical strain in the bottom of the crushed stone in Section 9 was 0.34×10^{-3} in./in. compared with about 0.40×10^{-3} in./in. for Section 9. These results substantiate the finding that the finer gradation stone was slightly more susceptible to rutting. Also the smaller size stone apparently did not protect the subgrade as much as the coarse crushed stone base.

Sections 11 and 12

Sections 11 and 12 were inverted pavements having a 3.5 in. (89 mm) asphalt concrete surfacing, 8 in. (203 mm) crushed stone base and a 6 in. (152 mm) cement stabilized layer beneath the base. In Section 11 the cement stabilized subbase was constructed by adding 5% Portland cement (by weight) to the silty sand subgrade soil used in all test sections. This subbase had a 28 day unconfined compressive strength of 200 psi (1400 kN/m^2). The cement stabilized subbase used in Section 12 was constructed by adding 4.5% cement to the crushed stone base having the standard gradation. The 28 day unconfined compressive strength of this material was 1,200 psi ($8,300 \text{ kN/m}^2$). The asphalt concrete B-binder used in these sections was obtained from the Lithia Springs plant of APAC-Georgia, Inc.

The load applied to Sections 11 and 12 was 6,500 lbs. (28.9 kN) throughout the test. The number of load repetitions applied at each position was doubled after 480,000 repetitions. As a result instead of applying a maximum of 100,000 load repetitions at the primary load position, 200,000 repetitions were applied during each subsequent sequence of loading. Doubling the number of load repetitions reduced the time required to fail the section since the load did not have to be manually moved as many times. This modified load procedure was particularly severe from the standpoint of rutting.

Fatigue. Section 11 failed in fatigue (initiation of Class 2 cracking) in the asphalt concrete B-binder after approximately 3.6 million repetitions. Section 12 failed in fatigue cracking after about 4.4 million repetitions. The cracking patterns present in Sections 11 and 12 are compared in Fig. 98 after 3.6 million load repetitions. Once again, almost all of the cracks occurred outside the loaded area. Only hairline cracks developed which were quite hard to see.

The horizontal tensile strain measured in the bottom of Section 11, which had the cement stabilized subgrade, was 340×10^{-6} in./in. In contrast Section 12, which had the cement stabilized crushed stone layer and performed better than Section 11, had a horizontal tensile strain in the bottom of the asphalt concrete of 260×10^{-6} in./in. These results suggest the cement stabilized base may have been slightly more effective in reducing tensile strain in the asphalt concrete than the less rigid, cement treated subgrade.

Rutting. Section 11, which had the cement stabilized subgrade, had a rut depth of about 0.5 in. (13 mm), after 3.6 million load repetitions. Therefore Section 11 was a balanced design failing in fatigue and rutting at about the same number of load repetitions. Section 12 had a rut depth of about 0.44 in. (11 mm) after 3.6 million load repetitions. This section also had a reasonably balanced design since it too simultaneously approached a fatigue and rutting failure.

The variation of rut depth with load repetitions beneath the center of the primary position is shown in Fig. 99 for Section 11 and in Fig. 100 for Section 12. Figs. 101 and 102 show the deflection profile at selected numbers of load repetitions.

Distribution of Permanent Deformation. The distribution of permanent deformation that occurred in Sections 11 and 12 is compared in Fig. 103. This figure shows that the magnitude and distribution of permanent deformation in Sections 11 and 12 were essentially the same although the total surface rut depth in Section 11 was slightly greater than for Section 10. In these two inverted sections about 70% of the rutting occurred in the thin, 3.5 in. (89 mm) thick asphalt concrete layer. Twelve to 18% of the permanent deformation occurred in the unstabilized crushed stone base and about 12% in the subgrade. Although only a small portion of the total permanent deformation occurred in the unstabilized base, most of it developed in the upper half of this layer. As expected, essentially no permanent deformation occurred in the cement stabilized layer in either section.

Of practical significance is the finding that the cement stabilized layer was quite effective in reducing rutting in the subgrade. In crushed stone base Sections 9 and 10, which showed excellent performance, about 68% of the total permanent deformation occurred in the subgrade. In contrast, only about 12% of the total permanent deformation occurred in the subgrade of inverted Sections 11 and 12. The structural thickness of the inverted sections were, however, 6 in. (152 mm) greater than the conventional sections; hence some of the reduction in subgrade rutting was due to a greater structural thickness above the subgrade. The



(a) Section 12: 3.5 in. AC/8 in. Stone/6 in. C.S.B.



(b) Section 11: 3.5 in. AC/8 in. Stone/6 in. S.C.

FIGURE 98. FATIGUE CRACKING AND SURFACE DEFLECTION PROFILE OF SECTIONS 11 AND 12 AFTER 3.8 MILLION LOAD REPETITIONS.

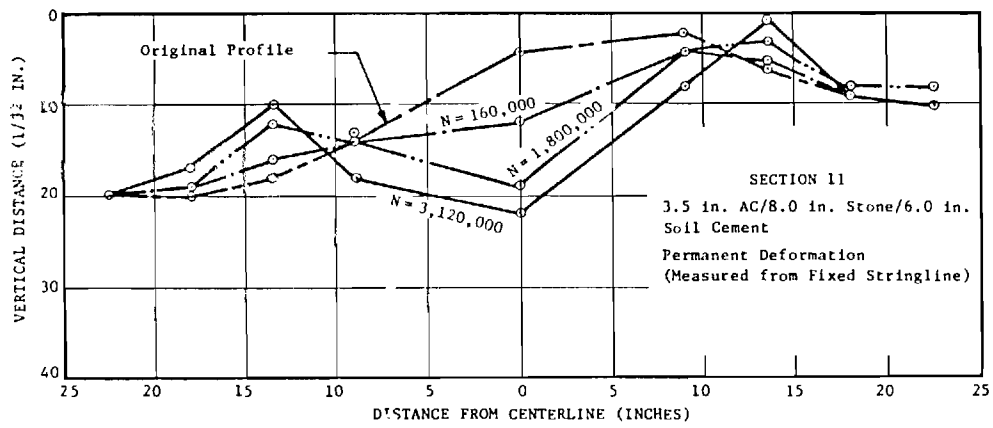


FIGURE 101. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 11.

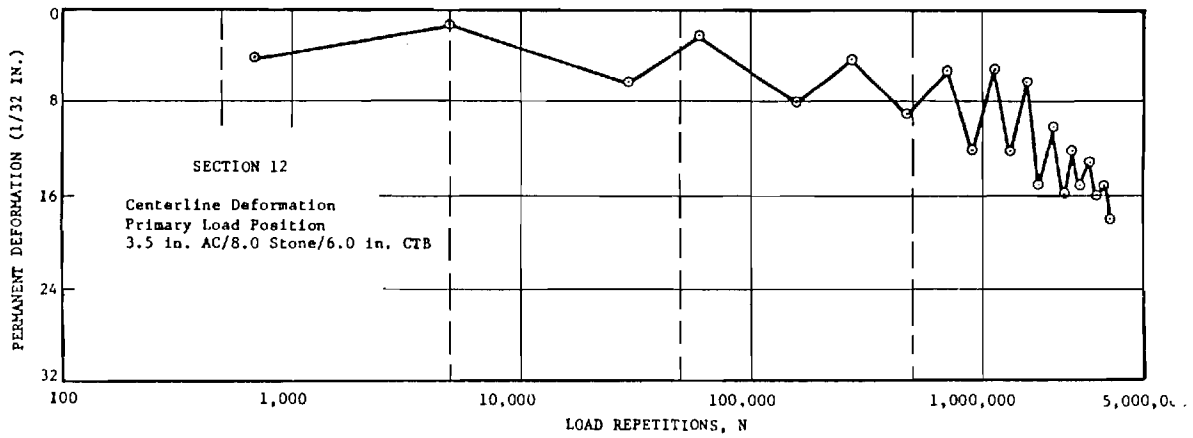


FIGURE 100. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

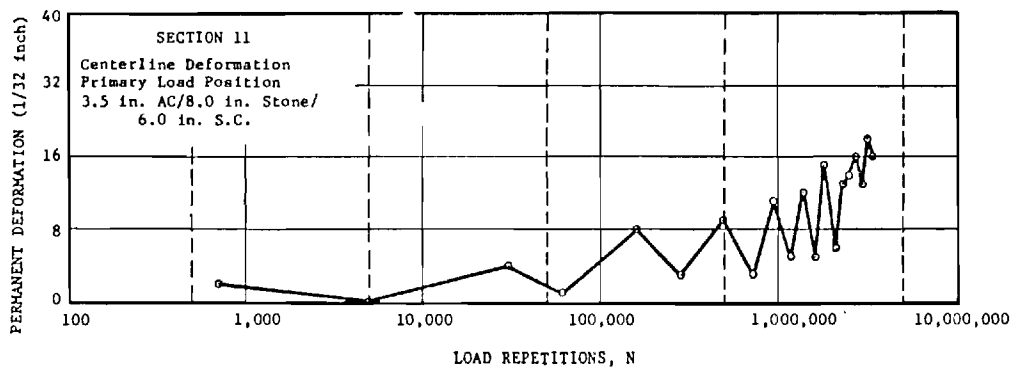


FIGURE 99. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 11.

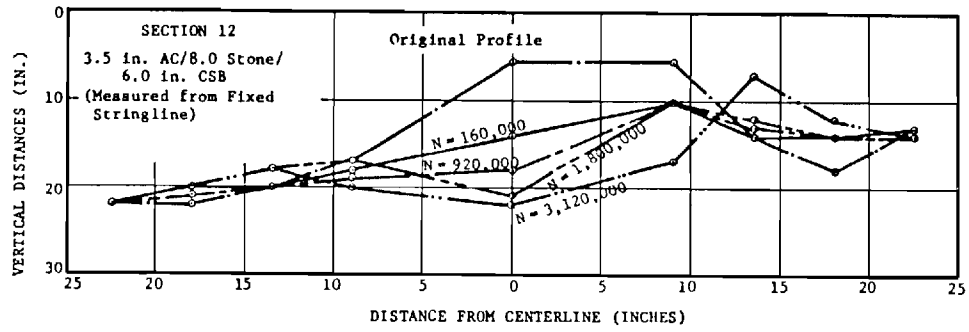


FIGURE 102. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 12.

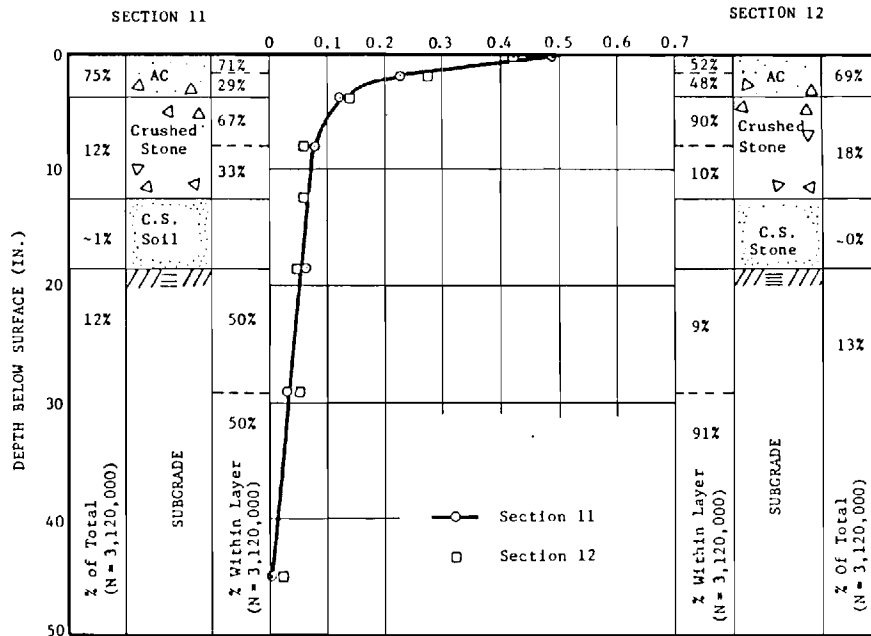


FIGURE 103. COMPARISON OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTIONS 11 & 12.

majority of the reduction in rutting however, was due to the beneficial effect of the cement stabilized layer which resulted in a significant reduction in vertical stress on the subgrade.

The presence of the stiff inverted layer apparently caused the amount of rutting occurring in the B-binder asphalt concrete surfacing to increase from about 20% of the total permanent deformation in Sections 9 and 10 to 75% in Sections 11 and 12. Permanent deformation decreased almost linearly with depth in the asphalt concrete.

Resilient Response. A general summary of the resilient response of all sections including 11 and 12 is given in Table 15. As previously discussed the horizontal tensile strain measured in the bottom of the asphalt concrete was 340×10^{-6} in./in. in Section 11 and 260×10^{-6} in./in. in Section 12. Tensile strains occurring in the bottom of the cement treated layer of both sections were too small to measure with the strain sensors. The typical resilient response for Section 11 as a function of load repetitions is shown in Figs. 104 through 106, and for Section 12 in Figs. 107 through 109.

In Section 11 the dynamic vertical strain in the bottom of the crushed stone beneath the load was 0.37×10^{-3} in./in. and the horizontal tensile strain at the interface between the crushed stone base and cement treated soil subbase was 54×10^{-6} in./in. In Section 12 the dynamic vertical strain in the bottom of the crushed stone beneath the load was 0.42×10^{-3} , and horizontal strain at the interface between the crushed stone and the cement-stabilized layer was 22×10^{-6} in./in. Apparently some slip may have occurred between the crushed stone and cement treated layer as indicated by the strain sensors.

The measured dynamic vertical strain on top of the subgrade was 0.39×10^{-3} in./in. (1) in Section 11 and 0.34×10^{-3} in./in. in Section 12. The average measured vertical stress on top of the subgrade was 3.3 psi (22.7 kN/m^2) for Section 11 and 2.6 to 4.2 psi ($17.2 - 28.9 \text{ kN/m}^2$) for Section 12.

The dynamic vertical stress measured on top of the subgrade for the inverted sections was considerably less than measured in crushed stone Sections 9 and 10. Measured dynamic strains on top of the subgrade, however, were less in Sections 9 and 10 than 11 and 12 which is somewhat surprising. Nevertheless, permanent deformation of the subgrade was less in the inverted sections than in the conventional crushed stone base sections.

Summary

All crushed stone base sections were constructed on a weak subgrade and had a 3.5 in. (89 mm) asphalt concrete B-binder or modified B binder

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1. Another strain pair in Section 11 indicated an average vertical strain of 491×10^{-6} in./in. This coil pair was felt to not be functioning properly and hence these readings were not used.

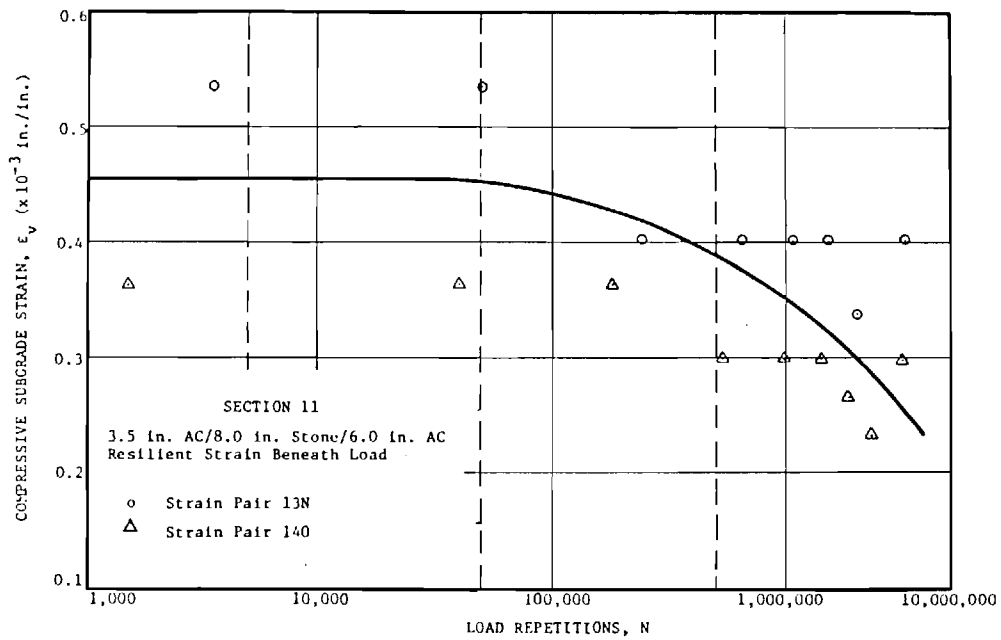


FIGURE 104. VARIATION OF RESILIENT VERTICAL STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 11.

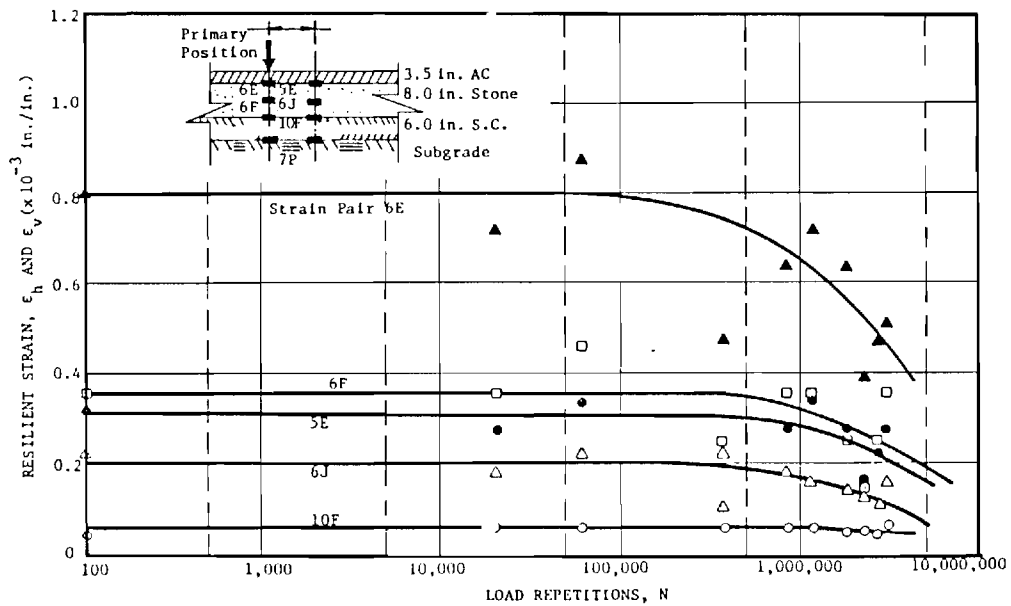


FIGURE 105. VARIATION OF RESILIENT VERTICAL STRAIN WITH LOAD REPETITIONS - SECTION 11.

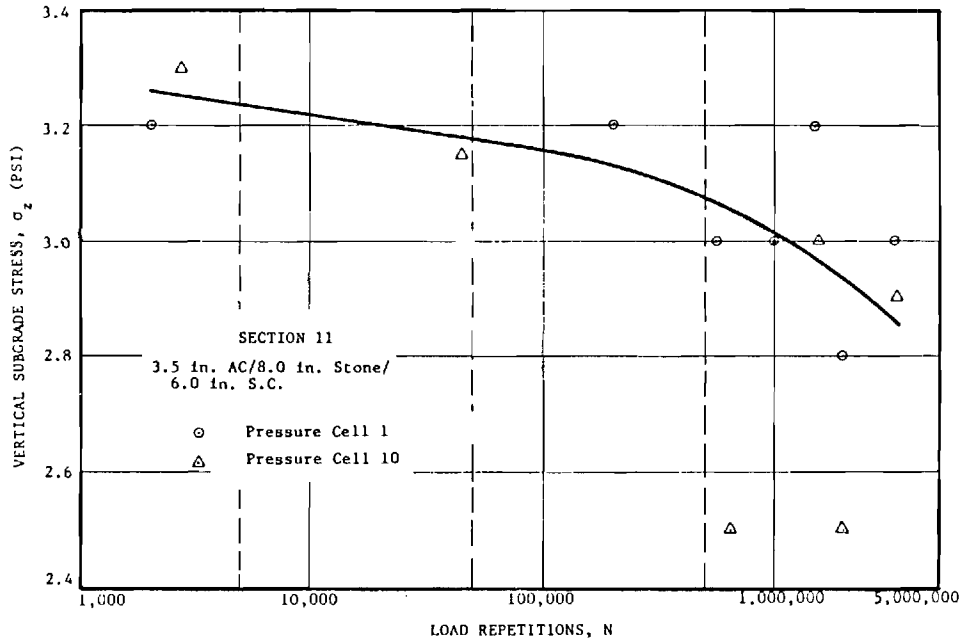


FIGURE 106. VARIATION OF VERTICAL SUBGRADE STRESS BENEATH LOAD WITH LOAD REPETITIONS - SECTION 11.

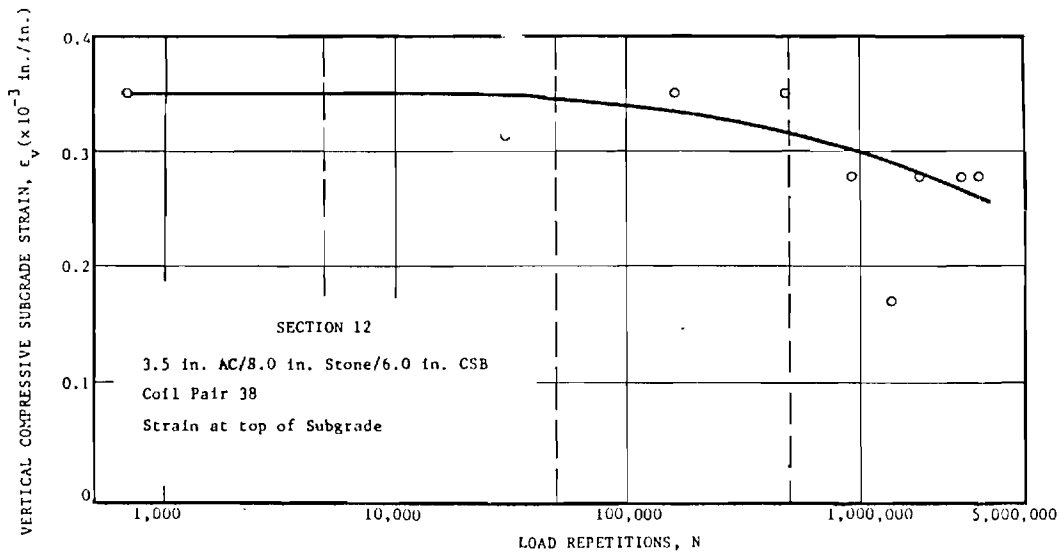


FIGURE 107. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

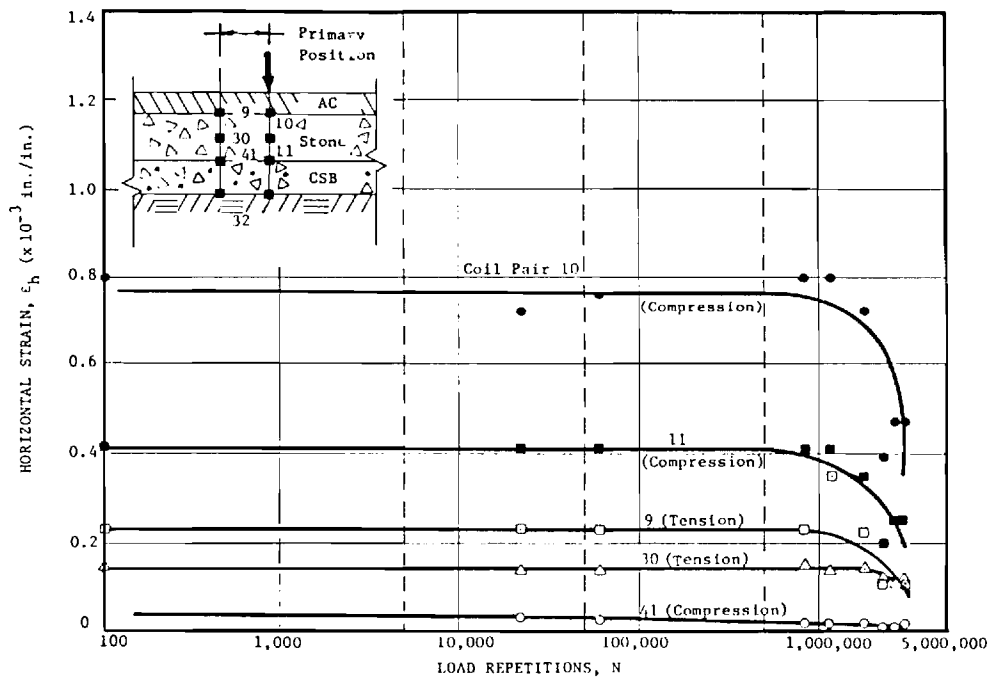


FIGURE 108. VARIATION OF HORIZONTAL STRAIN WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

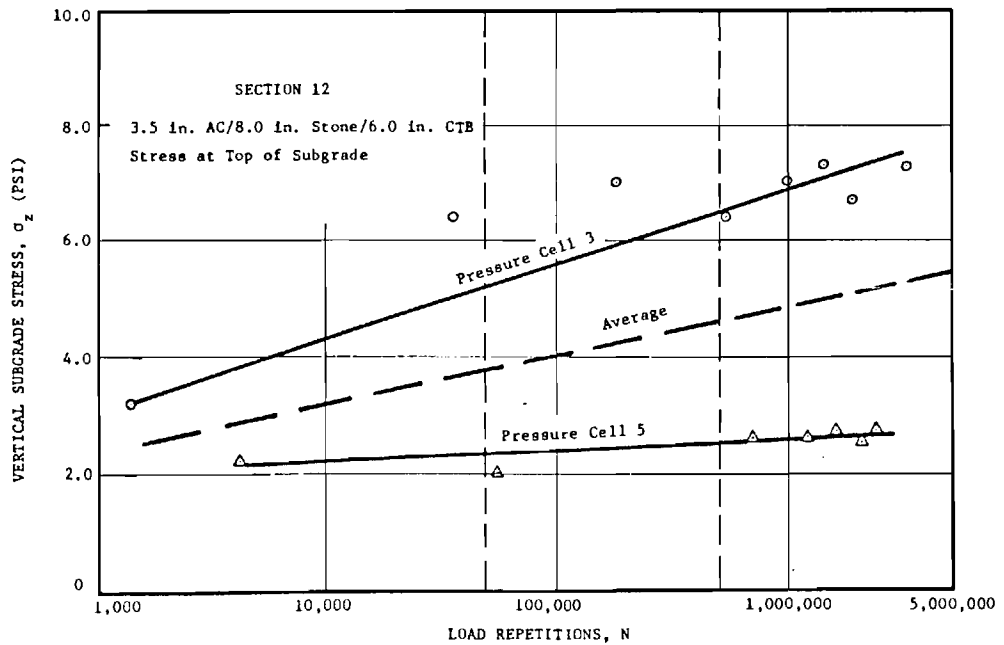


FIGURE 109. VARIATION OF RESILIENT VERTICAL SUBGRADE STRESS BELOW LOAD WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

surfacing. Increasing the crushed stone base thickness from 8 to 12 in. (203-305 mm) increased the life of the pavement by a factor of approximately three. The unstabilized crushed stone base sections studied all performed quite well, showing better performance than the full-depth asphalt concrete sections. In a direct comparison of full-depth asphalt concrete pavement (Section 7) with a crushed stone pavement (Section 8), the crushed stone base pavement withstood 500,000 load repetitions before undergoing a rutting failure while the full-depth asphalt concrete section withstood only 150,000 repetitions. Thus the crushed stone base was quite resistant to rutting compared with asphalt concrete even at the test temperature of 78 to 80°F (25.6-26.7°C). These findings indicate that if the quality of construction of the crushed stone bases achieved in this study can be duplicated in the field, crushed stone base can be effectively used to replace a significant amount of asphalt concrete in flexible pavements without a sacrifice in performance.

Inverted pavements having a 6 in. (152 mm) cement treated soil subbase (Section 11) and a cement stabilized crushed stone subbase (Section 12) also demonstrated excellent performance. The performance of Section 1, which had a 12 in. (305 mm) crushed stone base, however, was approximately comparable to the inverted section having the cement stabilized soil subbase (Section 11).

The tests conducted during this study were short-term and hence did not include environmental effects. Compared with crushed stone base sections, environmental factors should tend to reduce the service life of the inverted sections, and make rutting at higher summer temperatures a more important factor in the full-depth asphalt concrete sections.

CHAPTER VI DISCUSSION

Introduction

The purpose of this study was to evaluate the use of crushed stone bases as an alternative to the deep strength asphalt concrete construction presently used by the Georgia DOT in flexible pavements. Twelve⁽¹⁾ full-scale test sections were tested to failure to determine if crushed stone can be successfully used to replace asphalt concrete in the base course. With the exception of Test Sections 3 and 4, between 150,000 and 4.4 million repetitions were applied to each section (Table 1). All tests were conducted in a constant temperature environment at 78 to 80°F (25.6 - 26.7°C) over a period of two to four months for each pit filling.

In this study the performance of crushed stone bases was directly compared with full-depth asphalt concrete construction. No attempt was made in these tests to include environmental factors such as moisture, temperature changes, temperature gradients and the effects of ultraviolet light. The results of the Brampton Test Road [1,23], however, have shown the very significant effect which the environment has on pavement performance. Further, other differences undoubtedly also exist between the present study and prototype pavements including construction technique, level of quality control and material variability. Therefore, to take these factors into consideration the findings of this study are integrated together with the results from previous full-scale experimental pavement sections to give a valid basis upon which to design and construct pavements having thick crushed stone bases.

Test Section Findings

General Comparison

Figure 110 gives a general graphical summary of the performance of the sections tested in this study. Sections 3 and 4 were not included in this summary since they failed prematurely by rutting due apparently to a high asphalt content. Both rutting and fatigue failures occurred in the tests. Sections 1, 11 and 12 failed in a combined fatigue/rutting mode. Sections 2 through 8 failed in rutting, and Sections 9 and 10 failed in fatigue. A fatigue failure was defined as the initiation of Class 2 cracking. A rutting failure was defined as an average rut depth of 0.5 in. (12 mm) measured from a fixed string line; the rutting occurring during the first 1,000 repetitions was not included. Of significance is the finding that the sections surviving over 2 million repetitions failed in fatigue, or else were close to a fatigue failure. In contrast, sections having relatively short lives failed in rutting. Thus, these test results nicely illustrate the importance of preventing excessive rutting in either the

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1. Section 1 was tested to 2.4 million load repetitions; failure would have occurred at about 3 to 3.5 million repetitions.

asphalt concrete or crushed stone base.

Influence of Base Thickness. The effect of thickness of crushed stone base on pavement life is illustrated in Fig. 110. Section 1, which had a 12 in. (305 mm) crushed stone base would have undergone a combined fatigue/rutting failure after approximately 3.0 to 3.5 million repetitions. Section 2, which had an 8 in. (203 mm) base and was a companion section to Section 1, failed in rutting after 1.2 million load repetitions. Thus, an increase in base thickness of 4 in. (102 mm) increased the pavement life by a factor of almost 3. The AASHTO Road Test results also indicated a similar significant beneficial effect of a small increase in base course thickness [1]. As the base thickness becomes greater, however, the beneficial effect of increasing thickness probably decreases (refer to the analytical study in Appendix A).

Influence of Crushed Stone Base Gradation. Excellent performance was obtained from the granular base pavements having both the coarse (Section 9) and fine gradations (Section 10). Both these sections failed in fatigue compared with a rutting failure at a lower number of repetitions in the other two 8 in. (203 mm) crushed stone base pavements (Sections 2 and 8). The fatigue life of the fine gradation base section was about 20% greater than the fatigue life of the coarse base section. On the other hand, rutting in the fine gradation base section was 21% greater than in the section having a coarse gradation granular base. These differences are considered reasonably minor considering the possible variation. The somewhat limited test results indicate for the relatively narrow range of gradations tested, gradation has a reasonably minor influence on performance *provided* the section is compacted to 100% of T-180 density and little segregation is allowed to occur. These results also indicate the extremes in crushed stone base gradation presently produced in the Atlanta area should perform well under conditions similar to those for the present tests. It should be remembered, however, that all three crushed stone base gradations had a 1 to 2 in. (25 - 51 mm) top size, 40 to 44% passing the No. 4 sieve and 4 to 5% fines.

Influence of Asphalt Content. The approximate effect of asphalt content on permanent deformation is illustrated in Fig. 111. The results presented in this figure were developed considering the permanent deformation occurring in the upper 3.5 in. (89 mm) of both full-depth and crushed stone base sections at 10,000 load repetitions. Because the permanent deformations in most sections were small at 10,000 repetitions, this figure gives only a general indication of the influence of asphalt content on rutting. The results do illustrate that permanent deformation in the B and B-modified binder mixes used in this study was very sensitive to asphalt content. An increase in asphalt content from 5 to 5.25% approximately doubled the amount of permanent deformation; increasing the asphalt content from 5 to 5.5% caused the permanent deformation to increase by a factor of about 3. The full-scale tests indicate that asphalt contents greater than 5.0 to 5.25% should not be used to avoid excessive rutting.

Permanent Subgrade Deformation. The same resilient micaceous silty sand subgrade was used beneath all test sections. As previously discussed, the

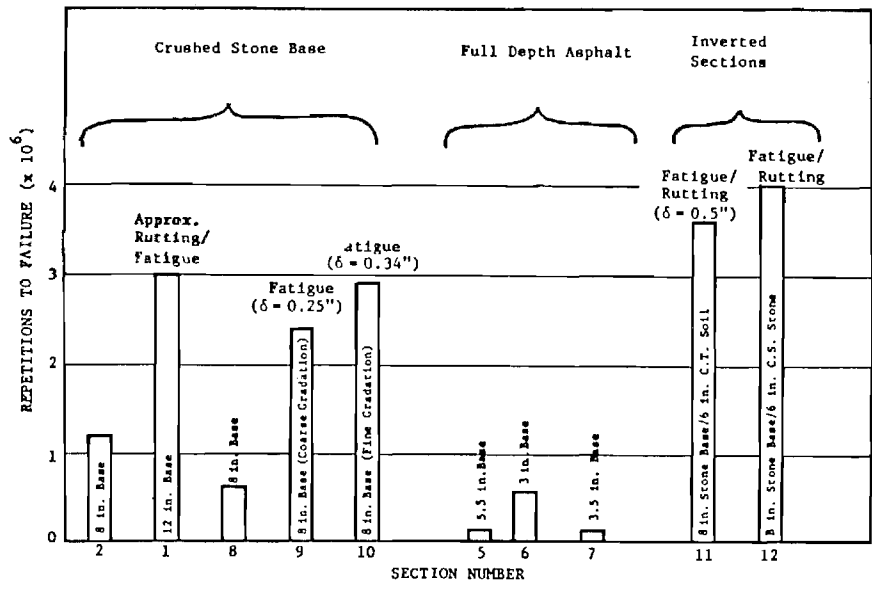


FIGURE 110. SUMMARY OF TEST SECTION RESULTS.

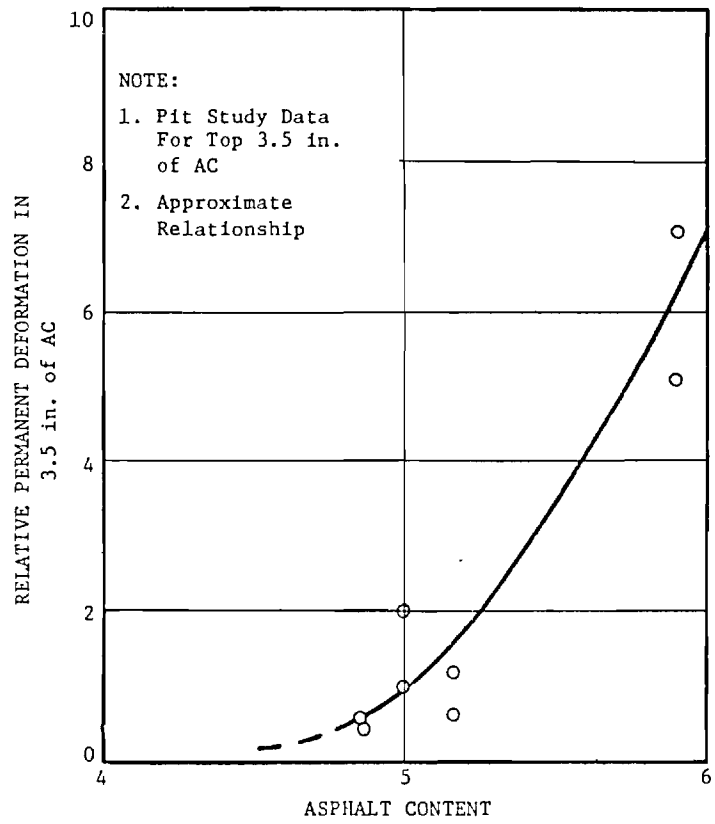


FIGURE 111. APPROXIMATE RELATIVE RUTTING IN ASPHALT CONCRETE AS A FUNCTION OF ASPHALT CONTENT - B AND B-MODIFIED BINDER.

subgrade was removed and recompacted at the same density and moisture content for each test. The resilient modulus of the subgrade was about 3,000 psi (2.07×10^4 kN/m²). This estimate is based on the measured resilient deflection of the test sections considering the resilient modulus obtained from the repeated load triaxial test.

Fig. 112 shows the variation of permanent deformation in this weak subgrade as a function of the base thickness and number of load repetitions. This figure shows that an increase in base thickness causes a decrease in permanent subgrade deformation. As the base thickness increases, however, the rate of decrease in permanent subgrade deformation becomes less. Of practical significance is the finding that the full-depth asphalt concrete sections appeared to be no more effective in reducing subgrade rutting per inch of base than the unstabilized crushed stone base.

For fair to poor construction approximately 30 to 40% of the permanent deflection can, based on the test results, be expected to occur in the subgrade. For a design rut depth of 0.25 in. (6.3 mm), the allowable permanent deformation in the subgrade would be 0.075 to 0.1 in. (1.9 - 2.5 mm). From Fig. 112 the required minimum total structural thickness is about 13 in. (330 mm) to withstand 2 million 6.5 kip (28.9 kN) wheel load, and about 15 in. (381 mm) to withstand 3 million repetitions.

Base Course Coefficients. The full-scale laboratory test results (Fig. 110) show excellent performance can be obtained from pavement sections having relatively thin asphalt concrete surfacings and properly constructed crushed stone bases. In fact, the crushed stone base sections out-performed the full-depth sections in every test series. In turn, inverted Sections 11 and 12 out-performed the crushed stone base sections. The good performance of the inverted sections compared to the crushed stone base sections is not surprising since the inverted sections were stronger and more expensive. The inverted sections had a structural number of 4.0 compared with structural numbers of 2.7 to 3.2 for the crushed stone base sections. The AASHTO Interim Guide [2] structural coefficients used in all calculations and figures are given in Table 18.

A base course coefficient of about 0.19 was obtained for the crushed stone sections from this test series assuming the asphalt concrete had a base course coefficient of 0.30. The corresponding base course equivalency was that 1.6 in. (46 mm) of crushed stone replaced 1 in. (25 mm) of asphalt concrete base in this investigation. The base course coefficients were determined by averaging the results from (1) the most efficient full-depth asphalt concrete pavement (Section 6) with the most efficient crushed stone base pavements (Sections 9 and 10) and (2) good performing full-depth Section 8 with crushed stone Sections 1 and 2 (refer to Fig. 113).

The required depth of crushed stone base to give equal performance to full-depth Section 4 was obtained by conservatively extrapolating crushed stone base test Sections 9 and 10 results to smaller base thicknesses. Admittedly other comparisons could readily give base course equivalencies and coefficients for the crushed stone considerably higher than presented above. However, since none of the full-depth asphalt concrete sections failed in fatigue, very likely the asphalt concrete in the full-depth sections was perhaps more susceptible to rutting than in prototype pavements.

TABLE 18. AASHTO Design Coefficients Used For Analysis and Obtained From Full-Scale Laboratory Tests.

Material	AASHTO Design Coefficients, a_1		Amount of Crushed ⁽²⁾ Stone Required to Replace 1 in of Base	Structural Number of Test Sections (1)
	Design ⁽¹⁾	Experimental ⁽²⁾		
Asphalt Concrete				
4.5 in	0.44	0.44 (assumed)	-	-
4.5 in	0.3	0.3 (assumed)	1.8	2.7 - 3.3
Crushed Stone	0.18	0.19	1.0	2.7 - 3.2
Cement Treated Subgrade	0.22	0.13	0.8	3.9
Cement Stabi- lized Stone	0.2	0.15	0.9	4.0

1. Used in analysis of test section results, for asphalt concrete thickness less than 4.5 in, a_1 was taken as 0.44.
2. Obtained from Georgia Tech Test Sections

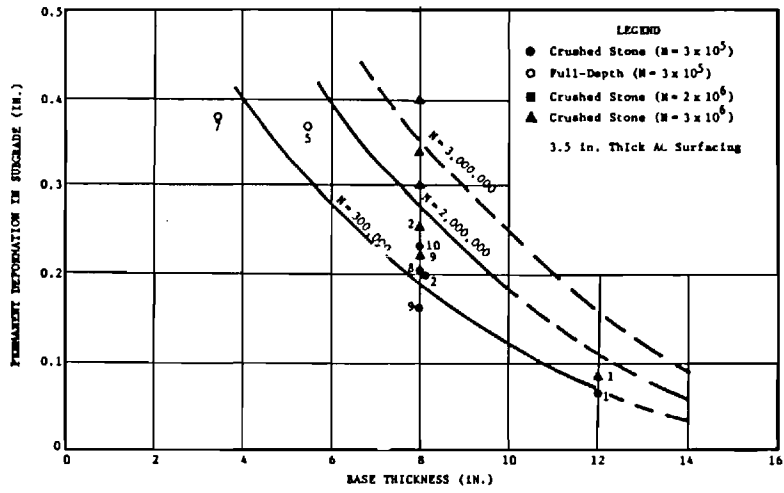


FIGURE 112. EFFECT OF BASE THICKNESS AND NUMBER OF LOAD REPETITIONS ON SUBGRADE RUTTING - CRUSHED STONE BASE AND FULL-DEPTH AC TEST RESULTS.

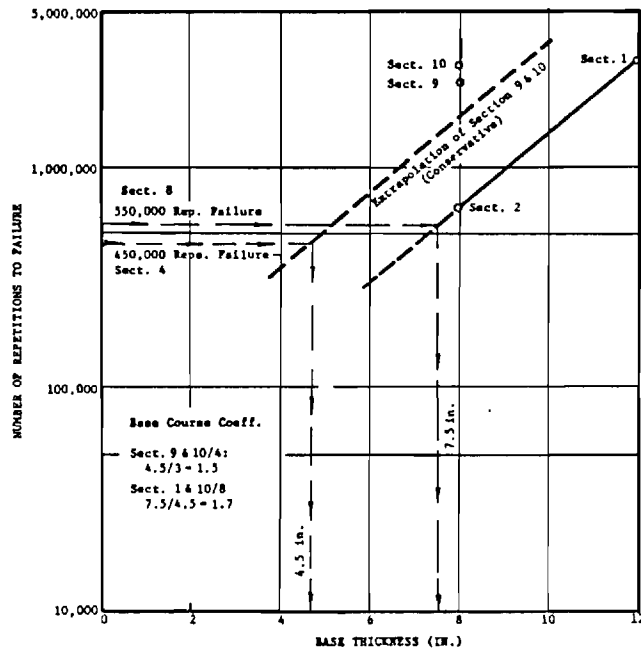


FIGURE 113. EXTRAPOLATION OF TEST RESULTS FOR CALCULATING BASE EQUIVALENCIES.

Therefore more optimistic interpretations of the results are not considered prudent without additional verification.

In this study the crushed stone base material was blended, pugmilled, and bottom dumped so that segregation and variation in gradation were practically non-existent. Also, the crushed stone base was uniformly compacted to between 100 and 101% of AASHTO T-180 density. As a result the laboratory sections had little variation within the crushed stone, and represented the limiting, optimum condition that can be achieved in the field. Finally, the crushed stone had only 4 to 5% passing the No. 200 sieve.

In contrast, a light maintenance roller was used to compact the asphalt concrete. Also, the asphalt plant was run at about 315°F (157°C) to conserve energy. As a result, the asphalt concrete was laid at approximately 290 to 295°F (143 - 146°C) which was about the minimum laying temperature that would give the required density for these conditions. All full-depth asphalt concrete sections failed in rutting even though an asphalt concrete density was achieved that met Georgia DOT specifications (typically the asphalt concrete had 96 to 100% of Marshall maximum density).

In summary, this study in general probably compared crushed stone at its best with somewhat marginally acceptable asphalt concrete. Nevertheless, excellent performance was obtained from the crushed stone base. For the conditions existing in the tests a very conservative base course equivalency was that 1.6 in. (41 mm) of crushed stone was required to replace 1.0 in. (25 mm) of asphalt concrete base.

The longest pavement life was obtained from the inverted sections which also had the greatest structural number. In relative terms of the crushed stone base Sections 9 and 10, however, the cement stabilized stone and cement treated soil inverted sections were found to have base course coefficients of only 0.16 and 0.14, respectively compared to about 0.18 for the crushed stone. Therefore, based on the laboratory tests, the inverted sections appear at least in general not to be as efficient as the crushed stone sections. Further, the influence of a possible gradual deterioration of the cement stabilized or cement treated subbase used in the inverted sections was of course not considered in the study and hence was not reflected in the base course coefficients. Of practical significance however, is the finding that the inverted sections both (1) did an excellent job of protecting the subgrade and (2) provided a strong working platform which permitted obtaining exceptionally high crushed stone base densities. Finally the inverted sections test results indicate that the limiting strength of the materials above a certain level can control performance. In these tests, however, environmental factors such as frost action were not present.

Rutting. Under the heavy 6,500 lb. (28.9 kN) repeated loading, the B and modified B-binder were more susceptible to rutting at the same depth than the three gradations of crushed stone used in these experiments. At high summer temperatures the asphalt concrete would be considerably more susceptible to rutting than at the test temperature of 78 to 80°F (25.6 - 26.7°C). The loading used in this study was slightly more severe with respect to rutting in the asphalt concrete than a 4,500 lb. (20 kN) dual wheel loading.

In the full-depth and inverted asphalt concrete sections the permanent strain (and hence rutting) was approximately constant with depth in the asphalt concrete. In the granular base sections having a 3.5 in. (89 mm) thick asphalt concrete surfacing, the distribution of permanent strain (and hence rutting) in the asphalt concrete varied with section. Section 9 had the coarse gradation crushed stone base and Section 10 had the fine gradation crushed stone base. In Sections 9 and 10 less rutting occurred in the lower part of the thin asphalt concrete surfacing than in the upper portion. In the other granular base sections more rutting occurred in the bottom of the asphalt concrete.

These experimental results indicate the distribution of rutting within the asphalt concrete is dependent upon both the type and quality of construction; probably the properties of the asphalt concrete have the most effect. A knowledge of the occurrence and distribution of rutting in all layers is important in developing mechanistic methods for estimating rutting in pavement systems.

In all test sections studied having an unstabilized granular base (including the inverted sections), most of the permanent strain within the base occurred in the upper portion of the crushed stone base. For 7 tests, an average of 73% of the permanent strain occurred in the upper portion of the crushed stone base and 23% in the lower portion. These important results suggest that improved pavement performance might be obtained by taking special precautions in constructing the upper part of a granular base course. More relaxed requirements in the lower portion are not, however, recommended.

For the conventional sections having an 8 in. (203 mm) unstabilized crushed stone base, an average of 52% of the total permanent deformation occurred in the subgrade. Sections 9 and 10, which had the coarse and fine gradation bases, performed extremely well. The amount of permanent deformation occurring in the subgrade of these sections was much higher than in the other sections, and was 68% of the total. The increased relative amount of rutting in the subgrade apparently occurred because the upper layers (particularly the asphalt concrete) exhibited a high resistance to rutting in these sections. Hence the relative resistance of the subgrade was less than for the other sections. Further, the permanent deformation in the subgrade developed in these two sections at greater depths than in the other granular base sections. In contrast, in Section 1 which had a 12 in. (305 mm) crushed stone base, only 20% of the total permanent deformation occurred in the subgrade. Therefore the additional 4 in. (102 mm) of crushed stone was quite effective in reducing the relative amount of permanent deformation in the subgrade. Section 1, however, did not perform as well as Sections 9 and 10 considering the difference of 4 in. (102 mm) in base thickness.

Now consider relative percentages of permanent deformation occurring in the full-depth asphalt concrete sections. In the 9.0 in. (289 mm) full-depth Section 5, 46% of the total deformation occurred in the subgrade. In the companion 6.5 in. (165 mm) thick full-depth Section 6, 57% of the total amount of rutting occurred in the subgrade. On the average 67% of the total subgrade deformation occurred in the upper 12 in. (305 mm) of the subgrade.

A comparison of the permanent deformation occurring in the subgrade of a crushed stone pavement (Section 8) with a full-depth pavement (Section 7) at similar depths below the surface is shown in Fig. 114. The granular base section had 8 in. (203 mm) of crushed stone while the full-depth section had a 7.0 in. (178 mm) thickness of asphalt concrete. Of considerable practical interest is the finding that the permanent deformation occurring in the subgrade beneath the crushed stone base section was about 30% less than the permanent deformation occurring beneath the same level in the companion full-depth section. These results indicate that crushed stone base sections constructed in this study were at least as effective in protecting the subgrade from rutting as full-depth sections of similar thickness.

The effect on permanent subgrade deformation of base thickness and number of load repetitions is shown in Fig. 112. The relationship shown in Fig. 112 was developed by combining the results of both the crushed stone base and full-depth test sections. For 2 million repetitions of a 6,500 lb. (29 kN) load, approximately 15 in. (381 mm) of structural section is required to limit subgrade rutting to 0.05 in. (1.3 mm), and a 12.5 in. (318 mm) thick structural section would limit subgrade rutting to about 0.1 in. (2.5 mm). The subgrade studied for which this relationship was developed was silty sand compacted to about 94% of AASHTO T-99 density at about 2% above optimum. The corresponding standard penetration resistance of the subgrade was 7 to 8. Weaker subgrades of course would be more susceptible to rutting.

Design

Crushed Stone

AASHTO Interim Design Guide. The performance of the Georgia Tech test sections is compared in Fig. 115 with 17 field test sections constructed in North Carolina [24]. This figure gives as a function of the structural number (SN) the ratio of actually applied load repetitions to the number of repetitions predicted by the AASHTO Interim Guide for a Present Serviceability Index (PSI) of 2.5. The actual Present Serviceability Rating (PSR) of each section is given in parentheses by the section. In predicting the AASHTO design life a subgrade support value of 2.0 was used. In the Georgia Tech tests a regional factor of 0.8 was used to consider that environmental effects were not included in the tests. A regional factor of 1.8 was used for the North Carolina tests. The base course coefficients used are given in Table 18.

The fact that the North Carolina test sections were all constructed on Piedmont subgrade soils and had service lives varying from 10 to 15 years is of relevance to this study. These tests therefore serve as a valuable full-scale extension of the laboratory tests conducted as a part of this study. The North Carolina test pavements included crushed stone base sections, deep strength sections and sections that fall between these two extremes. All the sections have Present Serviceability Ratings greater than 2.5 except 3 sections; the PSI values of all sections are greater than 3.25. The last PSR and PSI ratings were made in 1977. At that time approximately 252 to 950 thousand 18 kip (80 kN) equivalent single axle

TABLE 18. AASHTD Design Coefficients Used For Analysis and Obtained From Full-Scale Laboratory Tests.

Material	AASHTD Design Coefficients, a_1		Amount of Crushed ⁽²⁾ Stone Required to Replace 1 in of Base	Structural Number of Test Sections (1)
	Design ⁽¹⁾	Experimental ⁽²⁾		
Asphalt Concrete				
4.5 ⁱⁿ	0.44	0.44 (assumed)	-	-
4.5 ⁱⁿ	0.3	0.3 (assumed)	1.8	2.7 - 3.3
Crushed Stone	0.18	0.19	1.0	2.7 - 3.2
Cement Treated Subgrade	0.22	0.13	0.8	3.9
Cement Stabi- lized Stone	0.2	0.15	0.9	4.0

1. Used in analysis of test section results, for asphalt concrete thickness less than 4.5 in, a_1 was taken as 0.44.
2. Obtained from Georgia Tech Test Sections

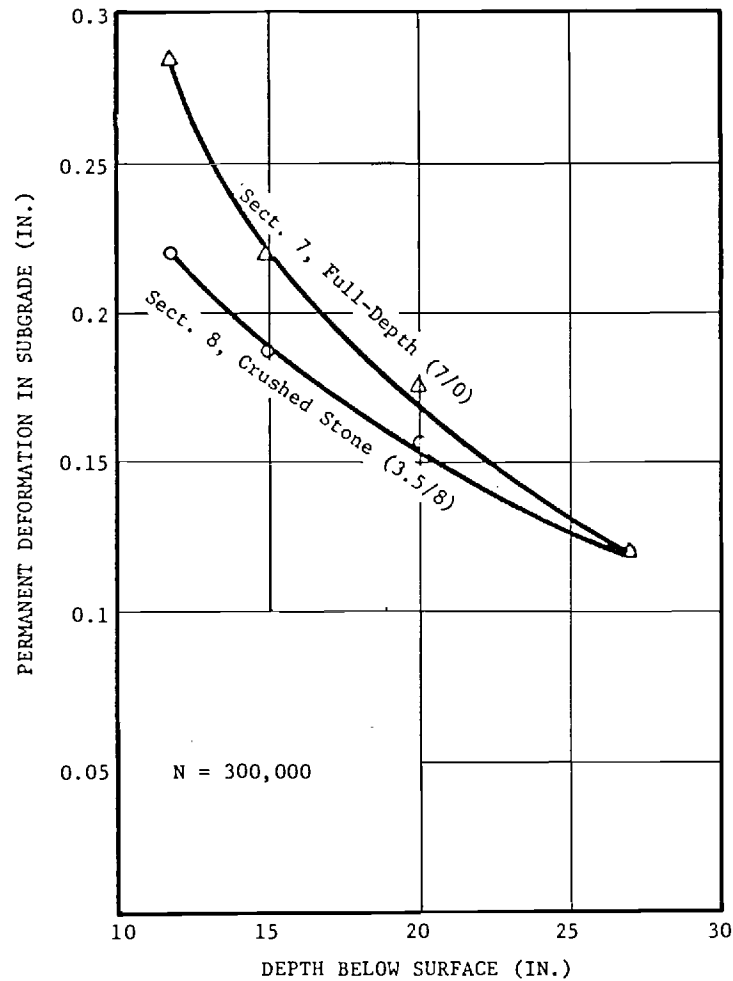


FIGURE 114. COMPARISON OF PERMANENT DEFORMATION IN SUBGRADE OF CRUSHED STONE BASE AND FULL-DEPTH ASPHALT CONCRETE SECTIONS - 300,000 LOAD REPETITIONS.

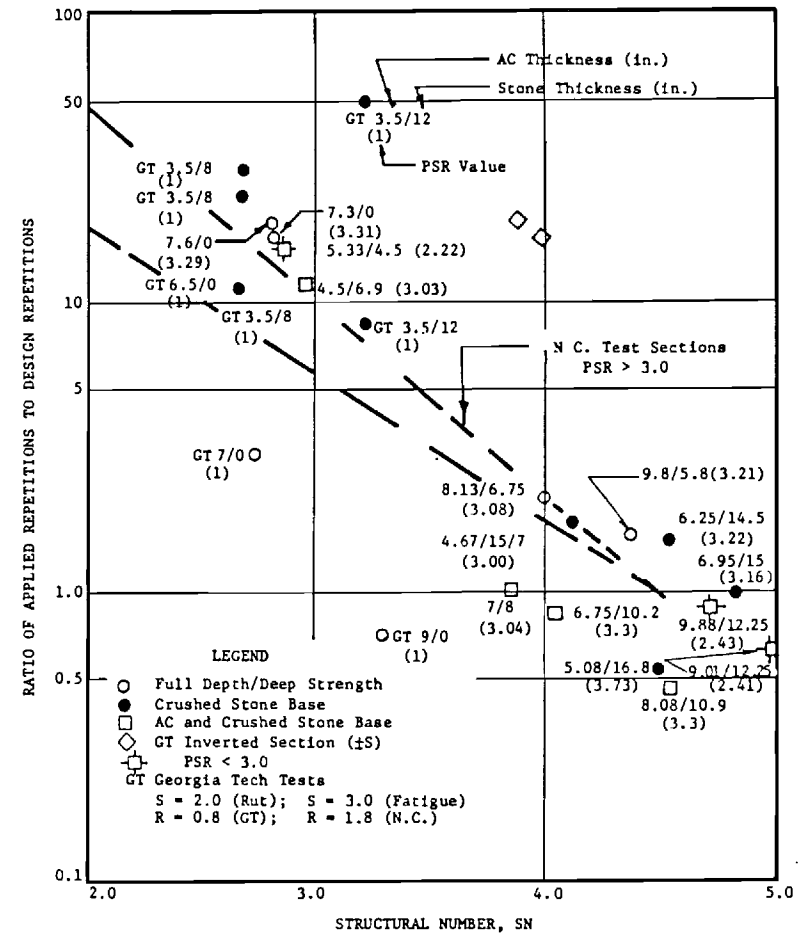


FIGURE 115a. COMPARISON OF GEORGIA TECH AND NORTH CAROLINA EXPERIMENTAL RESULTS WITH AASHTO INTERIM GUIDE DESIGN - STONE BASE COEFFICIENT OF 0.14.

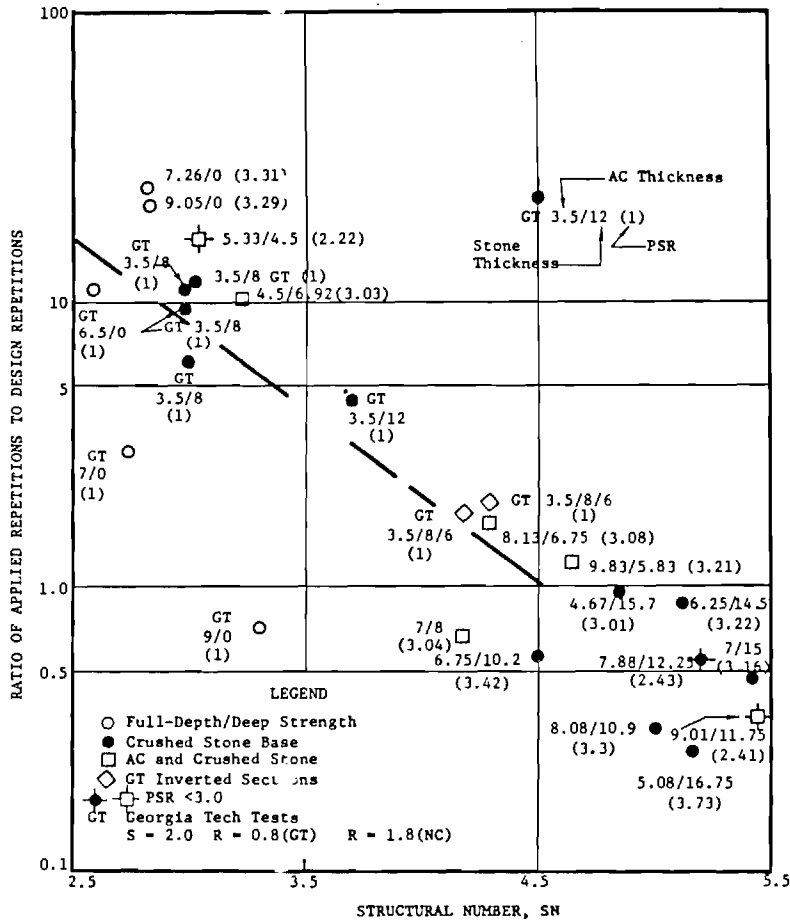


FIGURE 115b. COMPARISON OF GEORGIA TECH AND NORTH CAROLINA EXPERIMENTAL RESULTS WITH AASHI. INTERIM DESIGN GUIDE - STONE BASE COEFFICIENT OF 0.18.

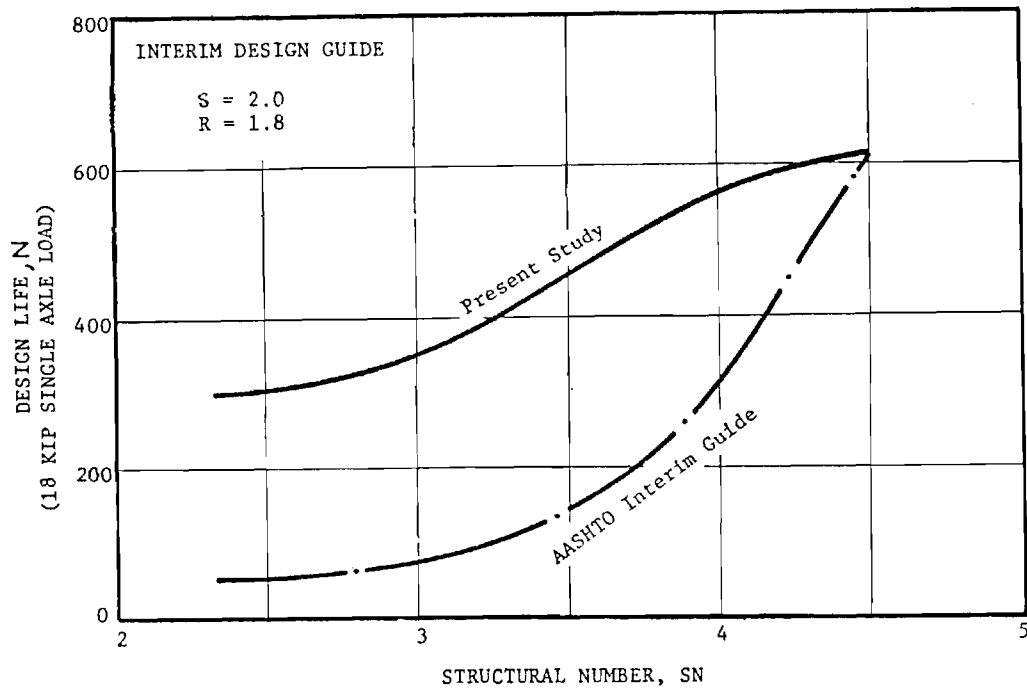


FIGURE 115c. COMPARISON OF PAVEMENT LIFE USING AASHTO INTERIM GUIDE AND PRESENT STUDY FINDINGS.

loadings had been applied to these pavements (Table 19).

Fig. 115 clearly shows that for well constructed pavement sections having structural numbers less than about 4.5, the AASHTO Interim Guide as applied in this study is overly conservative. This finding was true for all crushed stone and asphalt concrete base sections in both the Georgia Tech and North Carolina studies. Based on these experimental results the following empirical correction is proposed for modifying the calculated number of equivalent 18 kip (80 kN) single axle loads for structural numbers SN between 2.0 and 4.5:

$$N' = C_1 \times N \quad (1)$$

where: N' = corrected number of equivalent 18 kip (80 kN) single axle loads the pavement can withstand (PSI = 2.5)

N = number of equivalent single axle loads determined from the AASHTO Interim Guide (PSI = 2.5)

C_1 = empirical correction factor

The correction factor, based on the most conservative data from both studies is as follows:

$$C_1 = 10^{-0.5 SN + 2.25} \quad (2)$$

where C_1 is the correction factor and SN is the structural number of the section. The correction factor, C_1 is also given in Fig. 116 as a function of the structural number SN.

Consider the problem where the design traffic loading is known and a structural number is required. First estimate a design structural number and determine the correction term C_1 from either equation (2) or Fig. 116. Now multiply the actual traffic loading N by the correction factor C_1 to obtain the adjusted traffic loading N' . Enter the adjusted traffic loading in the AASHTO Interim design chart to determine the required structural number SN. Compare the calculated value of SN with the assumed value, and repeat the calculations if the two values differ by more than about 0.25.

Base Course Equivalency. Considering all available information including the findings presented in the literature survey, the North Carolina field test sections and the Georgia Tech study, the recommendation is made that 1.7 in. (51 mm) of crushed stone be used in the base to replace 1.0 in. (25 mm) of asphalt concrete base. In terms of the AASHTO Interim Design Method, the crushed stone base equivalency is numerically equal to the asphalt concrete base course coefficient divided by the crushed stone base course coefficient. For an AASHTO asphalt concrete base course coefficient of 0.30, the corresponding crushed stone base coefficient would be 0.18 for the proposed base course equivalency of 1.7. These equivalencies should be used for total pavement depths less than 16 in. (406 mm); for greater thicknesses a coefficient of 0.14 is recommended. The equivalency between crushed stone and asphalt concrete is dependent upon many factors including

TABLE 19. SUMMARY OF NORTH CAROLINA TEST SECTION STUDY [24].

Test Section	Open Traffic	YEAR 1977				DESIGN THICKNESS						SN ⁽²⁾	Design ⁽³⁾ N (P _t = 2.5)	
		Age	Total Traffic ⁽¹⁾	PSR	PSI	Asphalt Concrete			Crushed Stone Base	STBC Soil	Surface Course			
						Wearing	Binder	Base			AC%			Voids
T ₁	6/65	12	660	2.43	3.62	1.88	-	6.0	12.25		5.57	6.44	5.20	1200
T ₂	6/65	12	660	3.08	3.76	1.88	-	6.25	6.75		5.91	7.61	4.28	400
T ₃	8/64	12.8	629	2.41	3.68	1.88	-	7.13	11.75		6.88	.03	5.45	1800
T ₄	9/62	14.8												
T ₅	9/62	14.8												
T ₇	11/65	11.6	765	3.01	3.79	2.0	2.67	-	15.67		7.07	7.8	4.85	800
T ₁₁	10/63	13.7	934								5.69	7.0		
T ₁₇	4/65	12.2	953	3.13	3.25	1.88	-	7.50	-	14.67	5.76	8.54	-	
T ₁₈	4/65	12.7	953	2.71	3.76	2.50		8.0	-	9.0	6.8	5.46	-	
T ₂₂	11/65	11.6	305	3.30	3.54	2.75	-	5.33	10.92		6.82	8.45	5.01	980
T ₂₃	11/65	11.6	305	3.42	3.74	1.58	-	5.17	10.25		7.48	7.11	4.50	530
T ₂₄	7/64	12.9	332	3.73	3.67	1.83	3.25	-	16.75		7.24	5.93	5.17	1200
T ₂₅	6/65	12	832	3.16	3.56	2.25	-	4.70	15.0		7.35	8.26	5.42	1800
T ₂₆	6/65	12	832	3.21	3.76	2.25	-	7.58	5.83		7.32	5.77	4.63	650
T ₂₇	3/64	13.2	892	3.22	3.71	2.08	-	4.17	14.50		6.72	7.12	5.11	1000
T ₂₈	8/65	11.8	252											
T ₂₉	8/65	11.8	252	3.04	3.76	2.25	-	4.75	8.0		7.16	7.54	4.17	380
T ₃₀	3/67	10.2	757	2.22	3.47	2.58	2.75	-	4.50		6.83	5.20	3.04	45
T ₃₁	3/67	10.2	757	3.31	3.68	2.42	2.50	2.34	-		6.85	4.32	2.81	30
T ₃₂	3/67	10.2	757	3.29	3.64	1.92	2.50	2.91	-		6.86	5.57	2.83	35
T ₃₃	3/67	10.2	757	3.03	3.83	2.17	2.33	-	6.92		6.46	4.87	3.23	70

- NOTES: 1. Approximate number of equivalent 18 kip axle loads (in thousands) applied to inside lane; projected from 4 to 6 years of measurements using an assumed growth rate of 2.5% each year.
2. The structural number was determined using the AASHTO Interim Design Guide Coefficients given in Table 18.
3. AASHTO Interim Guide design life using a Regional Factor of 1.8 and Soil Support Value of 2.0.

(1) magnitude of wheel loading, (2) level of compaction, (3) uniformity of the base which is influenced by construction and inspection practices, (4) segregation of crushed stone, (5) thickness of structural layers, and (6) aggregate size and undoubtedly gradation, angularity and durability. Because of the influence of these and other variables, the additional recommendations presented subsequently for crushed stone bases should be followed to obtain satisfactory performance.

The Georgia Tech study indicated a conservative crushed stone base course equivalency of 1.6 for high quality crushed stone bases constructed over a micaceous silty sand Piedmont subgrade. The North Carolina field test sections also constructed on Piedmont soils had an equivalency of about this magnitude. Further, the studies comparing high quality crushed stone and asphalt concrete bases summarized in Chapter II indicate base course equivalencies varying from 1.6 to 2.5. Therefore the recommendation of a base course equivalency of 1.7 is supported by a considerable amount of field test results in addition to the results of the present study.

Asphalt Concrete Thickness. High quality, engineered crushed stone bases should be protected by the maximum thicknesses of well constructed asphalt concrete given in Table 20. These recommendations are made considering the results of the Florida, North Carolina and Georgia Tech studies. Particular weight was given to the North Carolina study in developing the table.

The results of the North Carolina, Lake Wales, Florida and Georgia Tech studies show that pavements having relatively thin asphalt concrete surfacings can, if properly constructed, successfully withstand heavy traffic loadings. Further, crushed stone bases constructed following the recommendations given subsequently should be no more susceptible to rutting than asphalt concrete. In fact, both the Georgia Tech and Florida studies indicate that particularly for high summer temperatures rutting can actually be greater in the asphalt concrete than crushed stone. About half the rutting in asphalt concrete pavements constructed in Georgia occurs at temperatures above about 95°F (35°C). Therefore use of crushed stone base sections may indeed prove beneficial in limiting rutting.

The results of the Florida and Georgia Tech studies show that thin sections having small structural numbers can exhibit exceptionally good performance compared with that predicted by the AASHTO Interim Design Guide. The postulation is presented that the good performance is at least partly due to overall flexibility of the pavement structure. Many years ago Hveem [25] realized that thinner asphalt concrete surfacings could withstand greater deflections. Early fatigue studies by Monismith and Deacon [26] showed thin layers of asphalt concrete are more flexible than thick ones. A detailed analysis of the AASHTO Road Test results by Barksdale [19] also showed that asphalt concrete becomes more flexible as the layer thickness decreases. The good performance of surface treated crushed stone base pavements is further evidence of this principle.

Therefore in designing crushed stone base pavements for optimum performance, the thickness of the surfacing should be made no greater than necessary to protect the crushed stone base. In the Lake Wales, Florida study a 1.5 in. (38 mm) asphalt concrete surfacing withstood 2.5 million

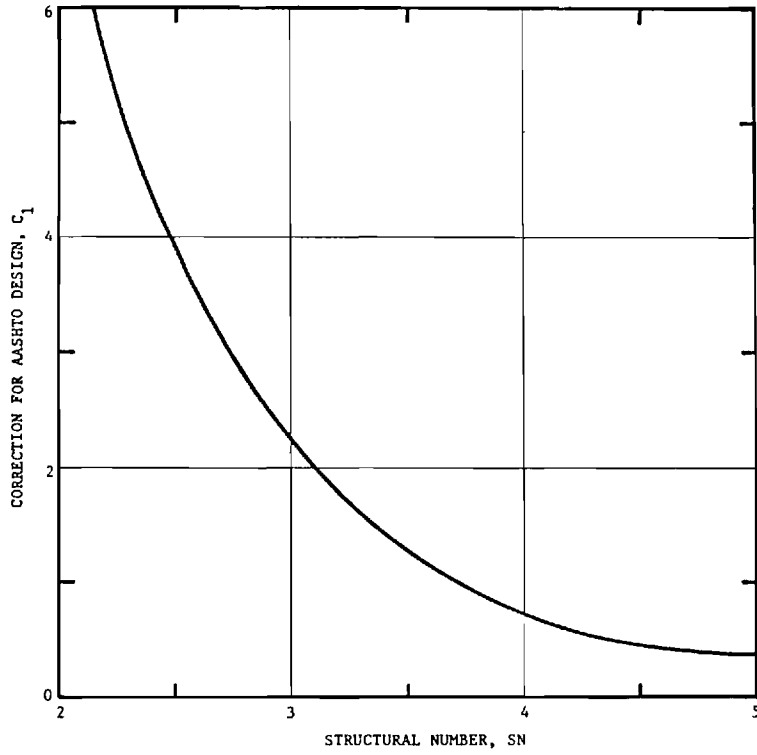


FIGURE 116. CORRECTION FACTOR C_1 FOR AASHTO INTERIM DESIGN GUIDE.

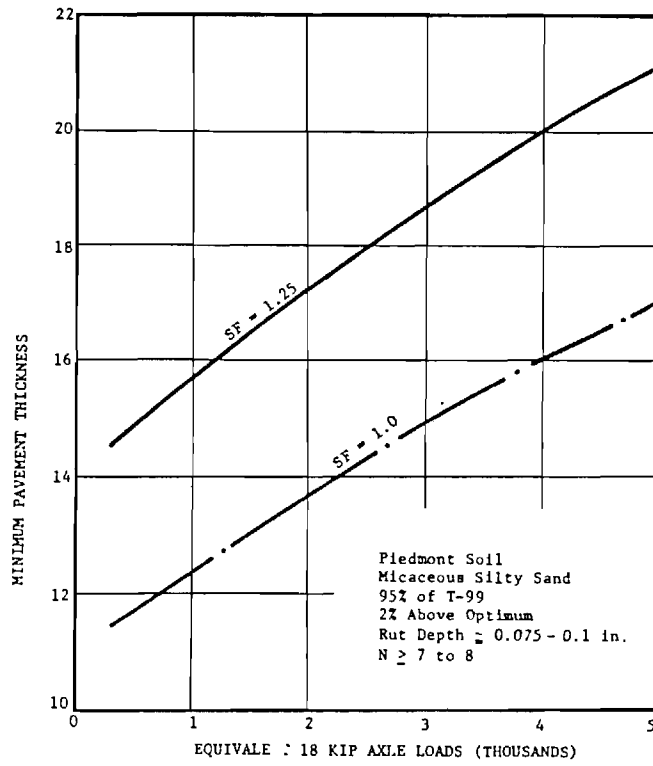


FIGURE 117. MINIMUM TOTAL PAVEMENT THICKNESS TO LIMIT SUBGRADE RUTTING.

TABLE 20. RECOMMENDED MAXIMUM THICKNESS OF ASPHALT CONCRETE IN CRUSHED STONE BASE CONSTRUCTION.

AASHTO Structural Number	Asphalt Concrete Thickness (in.)
<4	≤ 4.0
4 to 5	3.5 - 6.5
5 to 6	5 - 8

equivalent 18 kip (80 kN) axle loadings. As many as 4 million 6.5 kip (29 kN) wheel loadings were applied to a 3.5 in. (89 mm) surface during the present study. Further, military airfield pavements have successfully used thin asphalt concrete surfacings for many years under heavy aircraft loadings. These examples illustrate that even quite thin asphalt concrete surfacings can be successfully used in granular base pavements.

An important need presently exists to establish the optimum thickness of asphalt concrete for use over a crushed stone base pavement. This optimum depth must be determined using full-scale test pavements. Until such test results are available, the recommended maximum asphalt concrete thicknesses given in Table 20 serve as a general guide for design. Future studies may show that even smaller thicknesses of asphalt concrete surfacing are desirable.

Limiting Structural Thickness

The minimum total structural thickness is given in Table 21 for crushed stone base sections to limit permanent deformation in the subgrade to less than about 0.075 to 0.1 in. (1.9 - 2.5 mm). Use of a safety factor of 1.25 in general is recommended to insure adequate protection of weak, Piedmont subgrades. The recommendations given in Table 21 are valid if the subgrade is reasonably good. In specific the results were developed for a resilient silty sand subgrade having a density of at least 94% of AASHTO T-99 density and a standard penetration resistance of 7 to 8 blows per foot. On poor subgrades alternate design sections or greater structural thickness are required.

Using a safety factor of 1.25 gives required structural thicknesses slightly less than the thickness of the crushed stone base sections used in North Carolina which have performed extremely well. Finally, the recommended thicknesses to insure subgrade protection are in reasonable agreement with the results obtained at AASHTO for no subgrade rutting (Fig. 118).

In general, use of the AASHTO Interim Guide design method gives structural section thicknesses greater than required to protect the subgrade even using a safety factor of 1.25. Limiting structural thicknesses should certainly be considered in designing light pavements to be constructed in stages.

Crushed Stone Base Construction. The crushed stone base sections tested in this study significantly outperformed the full-depth asphalt concrete sections. However, granular base sections used in the North Carolina study demonstrated about the same performance as the deep strength sections using a base course equivalency of about 2.0. The markedly good performance of the crushed stone base sections in the present study is attributed to the use of a crushed stone having only 4 to 5% fines and the use of optimum construction techniques and quality control procedures in constructing the crushed stone bases. To insure a similar high level of performance for prototype pavements, construction procedures and specifications should be modified, and new innovative construction techniques developed. In summary

TABLE 21. MINIMUM RECOMMENDED TOTAL STRUCTURAL THICKNESS
TO PROTECT SUBGRADE FROM RUTTING.

Equivalent 18 kip Single Axle Loadings	Minimum Structural ⁽¹⁾ Thickness (inches)	
	SF = 1.0	SF = 1.25
300,000	11.5	14.5
2,000,000	13.5	17
3,000,000	15	19
5,000,000	17	21

1. The subgrade should have a density equal to or greater than 95% of AASHTO T-99 density and/or a standard penetration resistance of about 7 to 8 blows per foot.

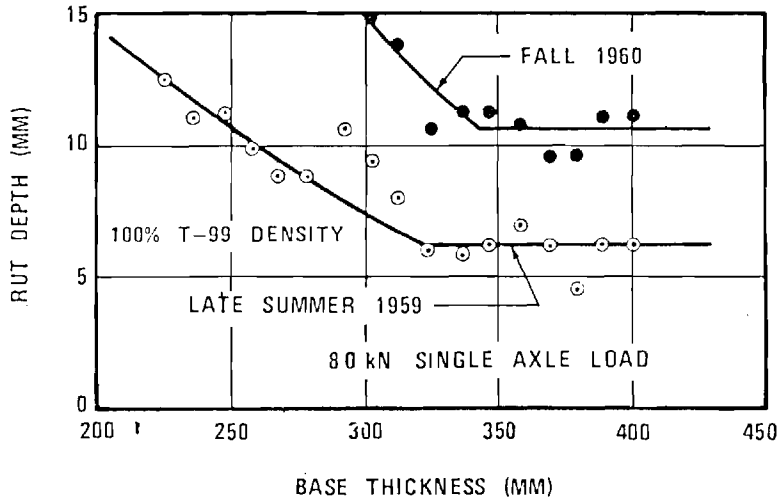


FIGURE 118. RUT DEPTH AS A FUNCTION OF BASE THICKNESS FROM AASHTO ROAD TEST.

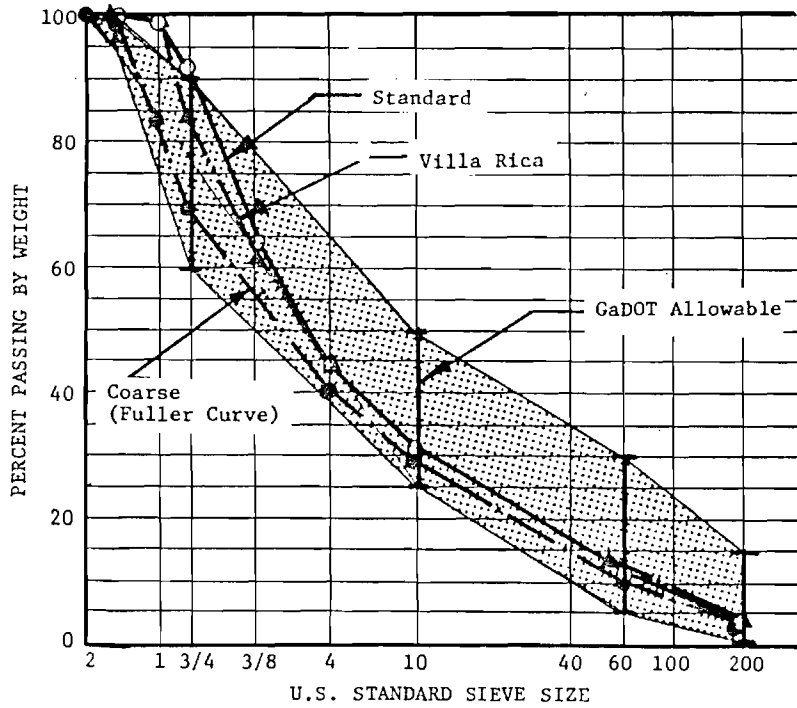


FIGURE 119. COMPARISON OF CRUSHED STONE GRADATIONS TESTED WITH GEORGIA DOT ALLOWABLE VARIATION.

the entire construction-inspection sequence must be conducted so as to give a truly "engineered" crushed stone base.

The outstanding performance of the crushed stone base can be primarily attributed to the following two important factors: (1) pugmilling the crushed stone and bottom dumping it so as to eliminate, as nearly as practical, segregation of the stone, and (2) uniformly compacting the crushed stone to a minimum of 100% of T-180 density. Certainly concentrated effort should be placed in the field on satisfying these two important requirements. The following recommendations are given in an attempt to improve present field construction practices:

1. Pugmilling. The crushed stone should be pugmilled to achieve a uniform blend. The cost of pugmilling should, for example, be less than the addition of lime to an asphalt concrete mix.
2. Stockpiling. The crushed stone should not be stockpiled at the site unless it is pugmilled afterwards.
3. Base Protection. After placing the crushed stone with a spreader, the surface should not be left exposed for more than 5 working days unless it is covered in an approved manner.
4. Base Density. Optimum base performance is obtained by placing a crushed stone base at the maximum practical density. Therefore the contractor should be strongly encouraged to achieve a high base course density. This can perhaps be best achieved by developing a specification that places a stiff penalty for obtaining densities less than 100% T-180. Perhaps a bonus might be given for attaining higher densities than 100% of T-180 and also for achieving uniformity of construction at acceptable densities.
5. Inspection. In general an increase in in-place variability tends to occur as the cost of a material decreases. Certainly the in-place variability of crushed stone bases is greater than for asphalt concrete. Therefore special attention should be given to inspecting crushed stone base construction. An adequate number of senior level technicians should be used to give full time inspection to base construction.

Maximum Crushed Stone Base Thickness

The results of an extensive parametric study of stress, strain, and deflection variations in crushed stone base, inverted and full-depth asphalt concrete sections are presented in Appendix A. These results were developed by first determining material properties that gave the best overall fit of the observed response of each type section. The theoretical results presented in Appendix A indicate that as the crushed stone base increases in thickness the tensile strain in the asphalt concrete becomes less at a decreasing rate. After a crushed stone base thickness of 16 to 18 in. (406 - 457 mm) is reached, little additional benefit appears theoretically

to be gained from the standpoint of fatigue by a further increase in crushed stone base thickness. An increase in thickness of asphalt concrete, however, is quite effective in causing a significant decrease in theoretical tensile strain. It must be remembered, however, that increases in asphalt concrete thicknesses apparently cause the surfacing to become more brittle and hence undergo a fatigue failure at a lower strain.

The designer must therefore select an optimum combination of asphalt concrete and crushed stone base thickness to maximize fatigue life. Unfortunately at the present time sufficient information does not exist, as previously discussed, on effect of asphalt concrete thickness on fatigue life to fully understand the complex interaction between fatigue life and asphalt concrete thickness. The tentative recommendation is made that a crushed stone base be no greater than 16 to 18 in. (406 - 457 mm) in thickness. After reaching this thickness, the structural number should be increased, if required, by increasing the asphalt concrete thickness. To achieve maximum flexibility the asphalt concrete surfacing should at the same time be made as thin as practical and still obtain the required structural number. Table 20 can be used as a tentative guide in selecting asphalt concrete thickness for flexible, granular base pavements.

Crushed Stone Gradation. The three gradations of crushed stone base used in this study all demonstrated good performance. This grading band is compared in Fig. 119 with present Georgia DOT crushed stone base grading specifications. The fine gradation base section gave the best overall performance of any 8 in. (203 mm) crushed stone section. The coarse graded crushed stone gradation base had slightly less rutting but failed in fatigue at 2.4 million repetitions compared with 2.9 million for the fine gradation base. Of significance is the fact that all three gradations had less than 5% passing the No. 200 sieve. Based on these findings all three gradations can give good performance provided segregation is minimized and a high level of densification obtained.

Proof-Rolling

Proof-rolling offers an excellent method for evaluating the in-situ behavior of either the subgrade or the crushed stone base. Proof-rolling is common in private consulting practice, and is also conducted on military airfield pavements.

Any subgrade which may be weak should be proof-rolled with a loaded dump truck (an alternate would be to use a loaded pan). A qualified inspector should carefully observe the proof-rolling operation watching for excessive subgrade deflections. The inspector should also observe pans and other heavy construction equipment moving over the subgrade along the route even in areas considered to be good. All detected weak areas should be undercut and replaced with well compacted structural fill, crushed stone or other suitable material.

Serious consideration should also be given to proof-rolling the crushed stone base. Proof-rolling should be accomplished using at least 3 to 6 passes of a loaded, 50 ton pneumatic tired roller operating at a

high tire pressure. The recommendation is presented that proof-rolling be tried in the field on a limited basis to evaluate its effectiveness. In a well-constructed crushed stone base almost three-fourths of the permanent deformation occurs in the upper portion of the base. Therefore, proof-rolling could be carried out in the top half of the base if the economics of proof-rolling justifies an increase in the level of effort expended.

Inverted Sections

The two inverted sections tested both demonstrated excellent performance; these pavement sections withstood more repetitions of load than any other section which was tested to failure. The inverted sections, however, had a relatively large structural thickness. The structural number of the inverted sections⁽¹⁾ was about 3.9 compared with 2.7 and 3.2 for the 8 and 12 in. (203 and 305 mm), respectively, crushed stone base sections. Because of the larger structural thickness the inverted sections did not perform as efficiently as did the crushed stone base sections. As a result the AASHTO base course coefficients for the inverted sections were approximately 0.15 and 0.13 for the cement stabilized crushed stone and the cement treated subgrade soil, respectively.

Elastic layered theory (Appendix A) indicates that when used in inverted sections, crushed stone bases up to about 24 in. (610 mm) in thickness have little effect in reducing the horizontal tensile strain in the bottom of the asphalt concrete (Appendix A). Further, either with or without the presence of the cement stabilized layer relatively little rutting would occur on the subgrade for structural thicknesses greater than those given in Table 21 provided the subgrade is reasonably competent. These factors therefore help to explain the relatively low base course coefficients observed in the cement stabilized layer of the inverted sections. Further, probably the crushed stone base should only be made thick enough to reduce reflection cracking or else omitted altogether.

These findings tend to verify the hypothesis that with depth the base course coefficient of any material tends to decrease. *This simply means a strong and/or deep structural section becomes less efficient and failure is ultimately controlled by the quality of the material in the upper levels.* For conventional crushed stone base sections on a reasonably good subgrade, this depth appears to be about 12 to 15 in. (305 - 381 mm) for 3,000,000 equivalent 18 kip, single axle loadings. The quality of the subgrade of course varies significantly throughout Georgia.

Cement stabilization of a weak subgrade does improve the subgrade and gives a strong working platform upon which the base can be constructed. Indeed a higher density of the crushed stone can be obtained when a cement stabilized layer is present, although the higher density apparently did not significantly increase resistance to rutting (compare rutting in Sections 9 and 10 with that in Sections 11 and 12).

1. Recall that the structural numbers used in this chapter are those based on the base course coefficients given in Table 18 and not those determined during this research.

Good performance of cement treated subgrades has been observed in Virginia [3] when placed directly beneath the asphalt concrete. Inverted sections constructed in Virginia have demonstrated somewhat variable performance; McGhee [27] believes good performance of inverted sections is related to good drainage. A positive mini-drain collection system having a positive outlet [28] or conventional lateral drains should therefore be used to remove water from the base of inverted sections constructed where water is expected to constitute a problem.

In areas having poor subgrade conditions which cannot be undercut the use should be considered of inverted sections or composite asphalt concrete-cement stabilized subbase sections. The subgrade should be stabilized with about 10% cement by weight to insure a strong, durable structural layer, or else a cement stabilized crushed stone subbase should be used. Such a layer will indefinitely reduce the stress reaching the subgrade to an acceptable level and minimize future problems in the subgrade.

Conclusions

The five crushed stone base sections tested in this study performed better than the full-depth asphalt concrete sections. Both fatigue and rutting failures occurred in the crushed stone base sections, while the full-depth asphalt concrete sections failed in only rutting. The complete response of these sections was measured including both resilient and long-term strains, and dynamic subgrade stresses. These experimental findings have contributed significant information concerning the mechanistic behavior of pavements constructed on resilient subgrades such as those found in the Piedmont Geologic Province.

Both the results of this study and full-scale field test results presented in the literature show crushed stone base pavements can be constructed having excellent performance. To insure good performance, more caution, however, must be exercised in constructing granular bases compared to deep strength asphalt concrete sections. Special precautions required in constructing crushed stone bases include (1) minimizing segregation (2) attaining a uniform product, and (3) achieving a high level of density at least equal to 100% of AASHTO T-180. Finally, the crushed stone bases which demonstrated good performance all had 4 to 5% passing the No. 200 sieve.

Both the results of this study and also the Lake Wales, Florida and North Carolina test roads indicate that the AASHTO Interim Guide significantly underestimates the number of load repetitions a pavement can withstand for structural numbers less than about 4.5. Further, 1.0 in. (25 mm) of asphalt concrete base can be replaced by 1.7 in. (51 mm) of high quality crushed stone meeting the requirements presented in this chapter. To maximize fatigue life the crushed stone base sections should be covered with the minimum practical thickness of asphalt concrete. Even for sections having large structural numbers the asphalt concrete surfacing should be kept less than 7.0 to 8.0 in. (178-203 mm) in thickness. Further, the crushed stone base should have a maximum thickness of 16 to 18 in. In conclusion, for optimum performance of crushed stone base pavements, the section must be kept as flexible as possible by minimizing the thickness of asphalt concrete.

CHAPTER VII SUMMARY AND RECOMMENDATIONS

Test Study

Experiment Design

The purpose of the full-scale pavement tests was to study the feasibility of replacing full-depth and deep strength asphalt concrete pavements with flexible pavements utilizing relatively thick layers of crushed stone base. A total of twelve test sections were constructed and tested to failure (one of these sections was only tested close to failure) by applying a repeated, dynamic loading having a magnitude of 6,500 lbs. (30 kN). To simulate traffic wander and prevent a localized punching type failure, the stationary circular loading was applied in a primary load position and six supplementary positions located symmetrically around the edge of the primary position. In all sections an asphalt concrete B or B-modified binder was used for the entire asphalt concrete thickness. The properties of all materials were evaluated using dynamic testing procedures.

The pavement sections tested were as follows: (1) five crushed stone base sections, (2) five full-depth asphalt concrete sections, and (3) two inverted sections. All the crushed stone base sections and the inverted sections had an asphalt concrete thickness of 3.5 in. (89 mm). The crushed stone bases had either an 8 or 12 in. (203-508 mm) thickness. Three crushed stone base gradations were studied approximately defining the limits of crushed stone bases produced in the Atlanta area. The full-depth sections consisted of either 6.5, 7.0, or 9.0 in. (165, 178, 229 mm) of asphalt concrete binder placed directly on the subgrade. The two composite sections consisted of 8 in. (203 mm) of unstabilized crushed stone sandwiched between 3.5 in. (89 mm) of asphalt concrete above and 6 in. (152 mm) of cement stabilized material below.

The results of this study showed that it was feasible to test in the laboratory full-scale sections to failure by subjecting them to large numbers of repeated loadings. By carefully controlling the quality of construction and environmental factors, direct comparisons of performance were obtained between different structural sections.

Test Sections

All tests were performed in an environmentally controlled room at a temperature varying from 78 to 80°F (25.5-26.7°C). The unstabilized crushed stone base in all sections was constructed by blending three different sizes of crushed stone together in a pugmill to minimize

segregation. Crushed stone base sections were compacted to or slightly above 100% of AASHTO T-180 density. In the inverted sections because of the presence of a rigid working platform, a crushed stone base density of 105% of AASHTO T-180 was obtained for the same compaction effort used in the other crushed stone bases.

A uniform, silty sand subgrade 50 in. (1270 mm) thick was used in all tests. The subgrade was compacted to an average of 94% ($\sigma = 1.5\%$) of AASHTO T-99 density at a moisture content of 20.4% ($\sigma = 0.5\%$) which was 2% above the optimum value. This material had an average Georgia DOT volume change of 38.5% and classified as a III-B embankment material.

Two hybrid air-over-oil cyclic loading systems were designed and constructed to test to failure the pavement sections. The two testing systems were used to apply over 17 million load repetitions during the study. These testing systems were found to be quite reliable and relatively easy to maintain considering the large number of repetitions applied.

Instrumentation

The test sections were fully instrumented to give important information concerning the amount and distribution of permanent deformation within each layer, resilient displacement, resilient strain and vertical stress on the subgrade. Bison-type strain sensors were used to measure permanent deformations and resilient strains. Vertical stresses were measured using small, diaphragm type pressure cells. Resilient surface displacements were measured with LVDT's. The results from this instrumentation gave valuable information concerning the mechanisms of pavement performance.

Summary of Findings

The findings were developed using the results of 12 test sections carefully integrated together with the findings from field studies presented in Chapters II and VI. Field studies of particular applicability to this investigation on crushed stone bases were conducted in North Carolina and Virginia on Piedmont soils and at Lake Wales, Florida on a sand subgrade. A detailed discussion of the findings is given in Chapter V and VI. Significant conclusions reached from this study are summarized as follows:

1. The 8 and 12 in. (203-305 mm) crushed stone base pavements withstood between 0.64 and 2.9 million wheel loadings, respectively. The crushed stone base was covered with only a 3.5 in. (89 mm) thick surfacing of asphalt concrete binder. These results clearly show that crushed stone base sections having thin asphalt concrete surfacings can successfully withstand large numbers of heavy loadings. These findings are supported by the Lake Wales, Florida test pavement results. At Lake Wales a 1.5 in. (38 mm) asphalt concrete surfacing underlain by varying thicknesses of limerock base withstood 2.6 million load repetitions.

2. The test results for the conditions of this study indicate the crushed stone base was more resistant to rutting than the asphalt concrete at a similar depth in the pavement structure. Admittedly, the asphalt concrete was probably marginally acceptable. Nevertheless other studies also indicate that under favorable conditions crushed stone can be less susceptible to rutting than asphalt concrete.
3. Good crushed stone base course performance is attributed to (1) a uniformly high level of compaction (100% of T-180 density), (2) no segregation of the base, (3) a thin asphalt concrete surfacing, and (4) only 4 to 5% of the base passed the No. 200 sieve.
4. For the variation studied, crushed stone gradation was found not to significantly influence performance. Of significance, however, was the fact that all three gradations had only 4 to 5% passing the No. 200 sieve and a 1 to 2 in. (25 to 51 mm) top maximum size crushed aggregate.
5. Performance of the tests appeared to be controlled more by the quality of the asphalt concrete surfacing than the crushed stone base. In this laboratory study the crushed stone base quality could be closely controlled whereas the quality of the in-place asphalt concrete was considerably more variable.
6. Test results indicate an increase in asphalt content from about 5 to 5.25% approximately doubled the rutting in a layer of asphalt concrete.
7. For the reasonably firm sand subgrade investigated, the minimum required structural thickness above the subgrade to limit rutting to about 0.07 to 0.1 in. (1.9- 2.5 mm) is about 13 to 15 in. (330- 381 mm) for 2 million 6.5 kip (29 kN) loadings. In general however, a safety factor with respect to rutting of 1.25 is recommended as shown in Fig. 117. The standard penetration resistance of the subgrade was about 7 blows per foot (23 blows/meter).
8. In a well constructed section about one-half of the total permanent deformation occurring in a uniform subgrade occurred in the upper 12 in. (305 mm). Further; as the quality of the material in the base and surfacing increased, the relative amount of rutting in the subgrade increased and was as high as 50 to 60% of the total.
9. The crushed stone base sections were just as effective inch for inch as the full-depth asphalt concrete sections in reducing subgrade rutting.
10. For the conditions of the tests, 1.6 in. (46 mm) of crushed stone replaced 1.0 in. (25 mm) of asphalt concrete base. Refer to Chapter VI for a discussion of these results.

11. In the crushed stone base sections about 70% of the total permanent strain occurred in the top of the crushed stone. This finding suggests that improved performance may be obtained by taking special precautions in constructing the upper portion of a deep granular base. More relaxed requirements in the lower portion of the base are not, however, recommended.
12. For pavements having structural numbers less than about 4.5, the AASHTO Interim Design Guide underpredicts the life of both crushed stone base and deep strength sections by factors up to 10 or more (Fig. 115).

Recommendations

The following design recommendations were developed from this study. Refer to Chapter VI for a detailed discussion of these recommendations.

1. Engineered crushed stone base pavements can be constructed that perform as well as deep strength asphalt concrete sections. In the future crushed stone base sections should be considered as a viable alternative to deep strength construction.
2. If pavements are constructed having relatively thin asphalt concrete surfacings and deep crushed stone bases, their use should be gradually phased in. During construction of the early deep granular base pavements a concentrated effort should be made to develop better techniques to minimize segregation and obtain high, uniform densities. This information should then be incorporated into subsequent projects.
3. In design 1 in. (25 mm) of asphalt concrete base should be replaced by 2.0 in. (51 mm) of engineered crushed stone base for pavement thicknesses less than 16 in. (406 mm). For an asphalt concrete AASHTO Base course coefficient of 0.30, the corresponding crushed stone base coefficient is 0.18 using the proposed substitution ratio of 1.7.
4. At the present time, a maximum crushed stone base thickness is recommended of 16 to 18 in. (406 - 457 mm). Layered elastic theory adjusted to approximately agree with observed test section response indicates greater depths of crushed stone give little additional benefit. Also, the North Carolina sections which performed well had maximum thicknesses of crushed stone of about this magnitude.
5. An engineered crushed stone base should be covered by the minimum amount of asphalt concrete necessary to insure stability and provide the required structural number. A minimum thickness of asphalt concrete insures maximum flexibility of both the section and the asphalt concrete. For AASHTO structural numbers varying from 4 to 6 asphalt

concrete surfacing thicknesses are recommended varying from 4 to 8 in. (102 - 203 mm). Tentative recommendations are given in Table 20. Both the Georgia Tech tests and the North Carolina studies indicate use of deep crushed stone bases and a substitution ratio of 1.7 is justified. Of course to insure satisfactory performance the other recommendations given below should also be followed.

6. A structural section sufficiently thick must be used to protect the subgrade. In general a crushed stone base structural section designed by the AASHTO Interim Guide will provide adequate protection. Table 21 and Fig. 117 provide a guide for required thickness of pavement to protect the subgrade.
7. Well constructed crushed stone base or deep strength sections having AASHTO structural numbers less than about 4.5 can withstand considerably greater numbers of load repetitions than predicted by the AASHTO Interim Design Guide. A conservative correction to the design guide results is given by equation (1) and in Fig. 116. Therefore, lighter, more flexible pavement sections perform extremely well.

This finding suggests the alternative approach of designing a relatively light section to withstand the design traffic using equation (1) and Figs. 115 and 116 as a guide. If the pavement does not last the full design life an overlay can always be placed. Although somewhat similar to stage construction, this approach differs significantly in basic design philosophy: in the proposed approach the section is designed to withstand the full traffic loading using an admittedly light, flexible section, while in stage construction the section is purposely designed to fail. This approach should be tried on a limited basis on carefully selected projects such as temporary roads. Certainly the outstanding performance of the light limerock sections at Lake Wales, Florida supports this approach. Using a relatively light, flexible section certainly has considerable economic advantages over a heavy, less flexible one.
8. Proper base construction is considered to be the single most important factor in achieving a high level of performance from a crushed stone base. The construction operation should be conducted such that segregation is minimized and a uniform density is achieved equal to or greater than 100% of AASHTO T-180. Pugmilling of the stone is recommended; stockpiling of the aggregate at the site should not be permitted. Further the recommendation is made that the base should not be left exposed for more than 5 working days.
9. Excellent performance was obtained from the base course which had the relatively narrow range in gradations shown in Fig. 119. This stone was well-graded and had a 1.0 to 2 in. (25 - 51 mm) top size and 4 to 5% fines. From these results a base course

stone should have the minimum amount of fines practical with 8 to 9% being the upper allowable limit. The top size should be at least 1 to 2 in. (25 - 51 mm).

More research is needed concerning the most effective maximum top size. Some research indicates a larger top size gives better performance. In this study the finer gradation base section had a longer fatigue life than the coarse gradation section. Rutting however was less in the coarse gradation base section.

10. Special attention should be given to the field inspection program for a crushed stone base. Careful, thorough inspection of the base course on a full-time basis is required by a senior inspector. Further, a bonus/penalty system should be implemented to insure a uniform, high density of the crushed stone base. It should be remembered that the weaker 10 to 15% of the area will determine the performance of the pavement.
11. The recommendation is also made that the desirability of proof-rolling the base with a 50 ton pneumatic-tired roller be studied in the field. Proof-rolling is common practice in the construction of military airfields. Proof-rolling would help to detect localized weak areas that could easily be missed by density testing.
12. All weak subgrades should be proof-rolled by a loaded dump truck (a loaded pan would be an alternative). All weak areas should, if possible, be undercut and replaced with compacted, high quality structural fill or crushed stone. Further, the inspector should carefully watch for locally weak spots by observing loaded pans and other heavy equipment during subgrade construction in other areas.
13. If weak subgrades are present that cannot be undercut, consideration should be given to using an inverted section. If an inverted section is selected a high level of stabilization should be used to insure adequate subgrade protection. Refer to Chapter VI for a detailed discussion of inverted sections. A base course coefficient of 0.18 to 0.20 is recommended for the stabilized layer if overlain by a thin crushed stone base.

Additional Research

Use of crushed stone bases together with relatively thin asphalt concrete surfacings has the potential for significant cost savings over deep strength sections. To achieve maximum economy without a sacrifice in performance, optimum thicknesses of asphalt concrete and crushed stone base must be combined for given traffic loadings and subgrade conditions. At the present time an understanding is just beginning to be developed of the complex behavior of both asphalt concrete and crushed stone bases. To take full economic advantage of crushed stone base sections the following additional research needs to be performed:

1. The effect of asphalt concrete thickness on performance desperately needs to be studied. Field experience in Georgia suggests that a greater asphalt concrete thickness does not necessarily mean better performance. On the other hand, the results of the Georgia Tech, Florida and North Carolina studies show that relatively thin asphalt concrete surfacings can give excellent performance.

A study is needed of the performance of full-scale sections having different asphalt concrete thicknesses. Such a study could be achieved by (1) investigating existing pavements or (2) constructing in the field (or laboratory) full-scale test pavements and testing them to failure. These pavements could be tested to failure by either natural truck traffic or else using the loading systems developed in this study. Weigh station pavements and parking areas are very suitable for such full-scale tests.

2. Improved techniques need to be studied for constructing crushed stone bases including: (1) minimizing segregation, (2) obtaining a high density, and (3) proof-rolling. Also utilization of 2 to 3 in. (51-76 mm) top size crushed stone (such as macadam) in base course construction also needs to be investigated in both the laboratory and field.
3. Finally, the results of the past research studies conducted by Georgia Tech on crushed stone, asphalt concrete and pavement performance need to be integrated together into a workable flexible pavement design procedure.

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COMPUTER STUDY OF LAYERED SYSTEMS

Introduction

A multi-layer computer program CHEV5L [32] was used to perform a sensitivity study of selected pavement designs utilizing materials similar to those used in the full-scale study. Material parameters were used in the theoretical analysis that gave theoretical deflection, strain and stress responses of the test sections approximately comparable to those observed in the test sections. A trial and error iterative procedure was used to determine the "best fit" material properties. Laboratory evaluated material properties were first used in the theory, and the material parameters were then modified until reasonable agreement was obtained with the observed response. This calibration was thus used to develop realistic values for the material parameters. These parameters were used to analyze a wide range of pavement designs and hence extended at least approximately the test section results to other conditions.

The CHEV5L program was used in this study [32]. This nonlinear, elastic layered program calculates the complete stress-strain and vertical displacement response within a five-layer pavement system. The five layer system consists of four upper layers of finite thickness and infinite horizontal extent resting on a semi-infinite half space. Each layer was assumed to be isotropic but the program can handle nonlinear material properties. The loading was assumed to be circular, exerting a uniform pressure on the surface. The effect of dual wheel loads can readily be obtained by superposition.

Calibration Study

To develop realistic material properties that gave computed results in good agreement with measured values, 27 computer runs were made. Vertical stress at the top of the subgrade, surface displacement, and tangential strain at the bottom of the asphaltic layer were used to assess the accuracy of each calibration run. It was found to be relatively simple to adjust the material property parameters to give good agreement with measured values of surface displacement and tangential strain at the bottom of the asphalt concrete. Computed vertical strains at the top of the subgrade were usually substantially less than the measured values, with better agreement being obtained for the inverted sections.

Several thicknesses of asphalt concrete and crushed stone base were used in the calibration. Therefore it was possible to approximately determine how the modulus of elasticity varied with layer thickness. The best match material properties are shown in Figure A-1. Fig. A-1 indicates that the average modulus of the crushed stone layer becomes greater with thickness (about 14,000 psi for a 6 in. thick base and 17,500 for a 24 in. stone base). In contrast the average modulus of the asphalt concrete appeared to decrease significantly with thickness (750,000 psi for a 4 in. thick AC surfacing to 400,000 for 9 in.). The elastic moduli for the other materials were considered to be constant with thickness and are given in Table A-2.

Theoretical Analysis of Pavement Designs

A parametric study involving 48 computer runs was performed to determine the sensitivity of surface deflection, vertical stress at the top of the subgrade and tangential tensile strain at the bottom of the asphalt concrete to the structural pavement design. A poor and good subgrade support condition was assumed for each structural design. The "poor" subgrade was assumed to have a modulus of elasticity of 2,800 psi, and the "good" subgrade a modulus of elasticity of 5,600 psi. This subgrade was assumed to be very deep. The pavement was subjected to a dual wheel load of 9,000 lb. with a tire pressure of 85 psi.

The inverted sections were assumed to have cement stabilized subbase. Separate analyses were not made for the soil cement subbase since theoretically little difference was found between the two type subbases.

The results for crushed stone base pavements are shown in Figs. A-2 through A-7, for full-depth asphalt concrete pavements in Figs. A-8 through A-10, and for inverted sections in Figs. A-11 through A-16.

Discussion of Results

At the present time the recommendation is made that the results given in Figures A-2 through A-16 should be used for *general* comparisons of similar type designs. In other words the figures give an indication of the effect of increasing the thickness of the crushed stone base, increasing the thickness of asphalt concrete or changing the stiffness of the subgrade for similar type construction. The figures should not be used to compare crushed stone base designs with full-depth asphalt concrete designs.

Crushed Stone Base. Tangential tensile strain at bottom of asphalt decreases with increasing thickness of asphalt concrete and base. Increasing thickness of asphalt concrete is by far the most significant effect. Subgrade quality affects tensile strain mainly when the asphalt concrete layer is thin; for example, when the asphalt is 3.5 in. (90 mm) thick, tensile strains are increased by about 40% when going from a good to a poor subgrade. However, for asphalt concrete thicknesses of 6.5 in. (165 mm) and 9 in. (230 mm), tensile strains in the asphalt concrete are essentially unchanged for the two values of subgrade modulus.

The vertical stress on subgrade decreases with increasing thickness of asphalt concrete and base. Subgrade quality has a significant effect with vertical stresses on the good subgrade being about 25% higher than on the poor subgrade. Maximum surface deflection decreases with increasing thickness of asphalt concrete and base. Subgrade quality has a major effect with poor subgrades showing deflections about 50% higher than good ones.

Full-Depth Asphalt Concrete. The tangential tensile strain at the bottom of the asphalt concrete decreases with increasing thickness of asphalt concrete and improving subgrade quality. About 0.7 in. (18 mm) of additional asphalt concrete thickness compensates the tangential strain for a change from a "good" to "poor" subgrade. The vertical stress at the top of the subgrade decreases with increasing thickness of asphalt concrete and increases with improving subgrade quality. About 5 in. (125 mm) of additional asphalt concrete thickness compensates for a change from "good" to "poor" subgrade.

Maximum surface deflection decreases with increasing asphalt concrete thickness and with improving subgrade quality. About 11 in. (280 mm) of additional asphalt concrete compensates when going from a "good" to a "poor" subgrade.

Inverted Section. The tangential tensile strain at bottom of asphalt for base thicknesses greater than about 4 in. (100 mm) only decreases very slightly for increasing thicknesses of either the cement treated subbase or the crushed stone base. As the base thickness decreases, however, tensile strain decreases greatly reaching a minimum value at zero base thickness. Based on these theoretical results, using a crushed stone base between the two treated materials does not reduce the tensile strain in the asphalt concrete. Presence of the crushed stone would help to reduce and/or prolong development of reflection cracking on the surfacing.

Use of the cement stabilized subbase does cause an important reduction in stress on the subgrade for all designs. Vertical stresses on the "good" subgrade are about 50% higher than on the "poor" subgrade. Maximum surface deflection decreases with the stabilized subbase thickness and with crushed stone base thickness. However, a crushed stone base thickness of about 3 in. (75 mm) is a critical value: below that base thickness deflections remain either approximately constant or actually decrease somewhat with decreasing thickness. Of course crushed stone base thicknesses less than about 6 in. (150 mm) are not practical.

TABLE A-1. COMPARISON OF MEASURED AND THEORETICAL STRESSES, STRAINS AND DEFLECTIONS USING FINAL MATERIAL PARAMETERS.

Pavement Design	Vertical Stress on Subgrade (psi)		Tangential Strain at Bottom of AC ($\mu\epsilon$)		Surface Displacement (mil)	
	Typical Measured	Theoretical	Typical Measured	Theoretical	Typical Measured	Theoretical
9" (235 mm) Full-Depth AC	8.7	2.5	308	280	14	17
3.5" (90 mm) AC on 8" (200 mm) Crushed Stone Base	6.8 - 11.2	4.0	270-390	352	23	29
3.5" (90 mm) AC on 8.7" (220 mm) Crushed Stone Base on 5.8" (147 mm) Cement Treated Crusher Run	3.5	2.1	272	262	17	15
3.7" (94 mm) AC on 8.9" (226 mm) Crushed Stone Base on 6.0" (150 mm) Cement Treated Subgrade	3.2	2.5	324	276	26	19

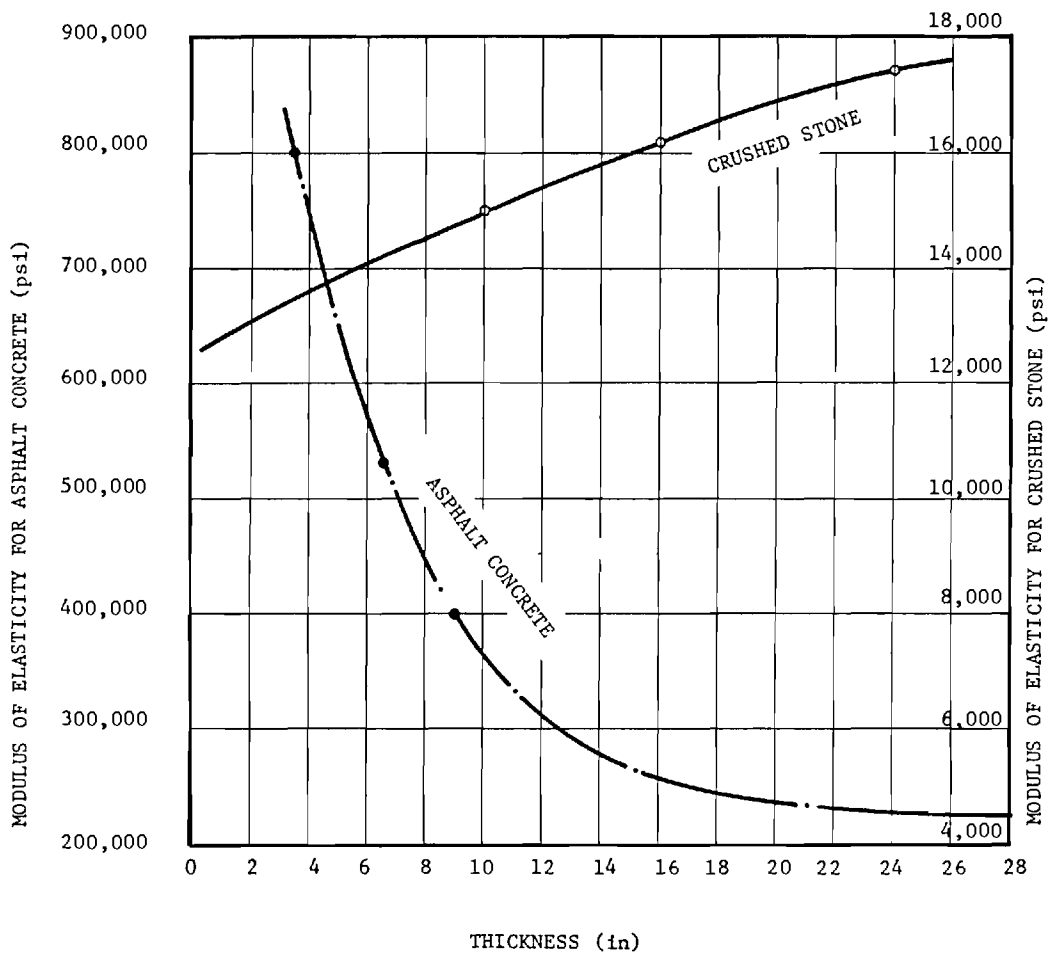


FIGURE A.1: VARIATION OF MODULUS OF ELASTICITY WITH DEPTH FOR CRUSHED STONE AND ASPHALT CONCRETE

TABLE A-2. MATERIAL PROPERTIES.

Material	In-Situ Unit Wt.		Poisson's Ratio	Modulus of Elasticity	
	(pcf)	(kg/m ³)		(psi)	(MPa)
Asphalt Concrete	145	2,345	.3	See Figure A	
Crushed Stone Base	140	2,265	.35	See Figure A	
Subgrade	125	2,020	.35	2,800	19.29
Cement Treated Crusher Run	145	2,345	.2	100,000	689.0
Soil Cement	126	2,040	.2	15,000	103.4

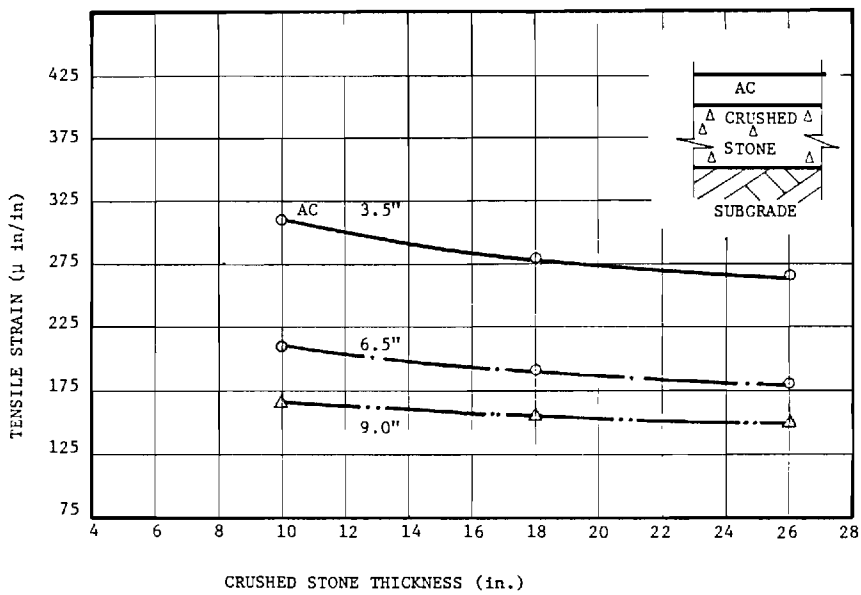


FIGURE A-2: CRUSHED STONE BASE: GOOD SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF BASE AND SURFACING THICKNESS.

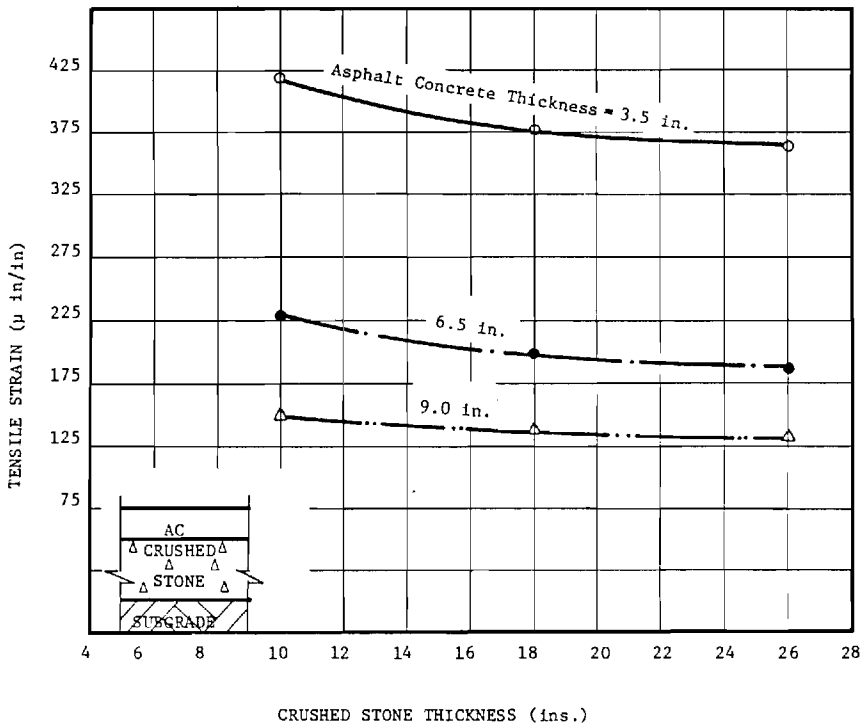


FIGURE A-3: CRUSHED STONE BASE: POOR SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF BASE AND SURFACING THICKNESS

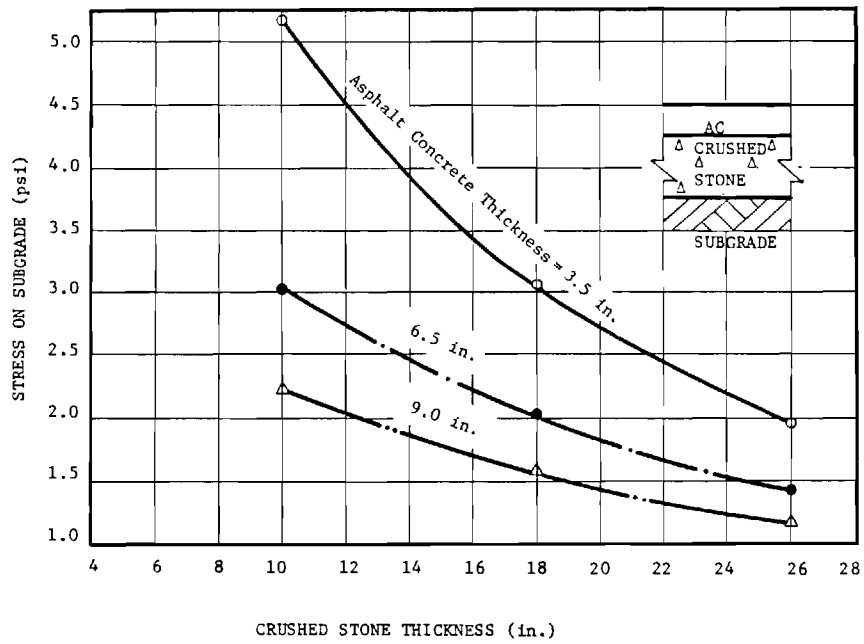
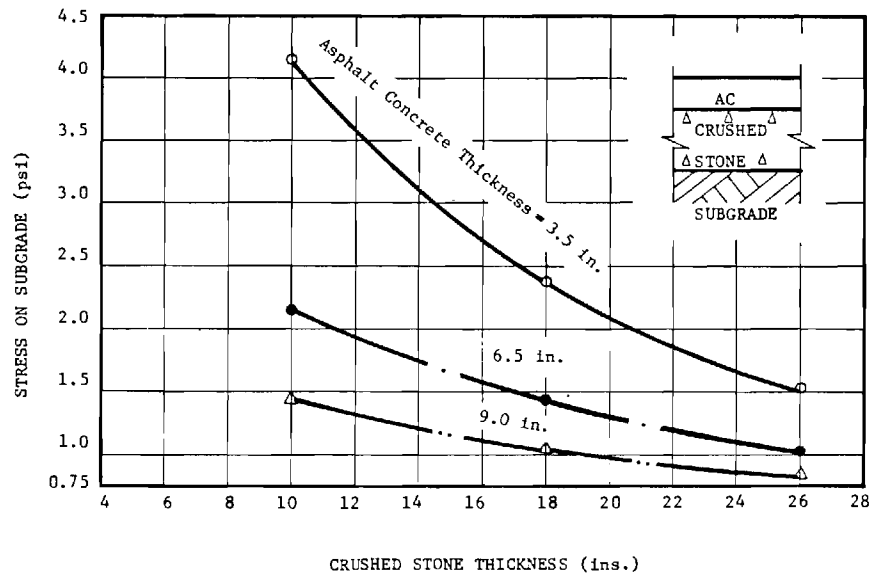


FIGURE A.4: CRUSHED STONE BASE: GOOD SUBGRADE VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF BASE AND SURFACING THICKNESS



FIGURES A.5: CRUSHED STONE BASE: POOR SUBGRADE. VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF BASE AND SURFACING THICKNESS

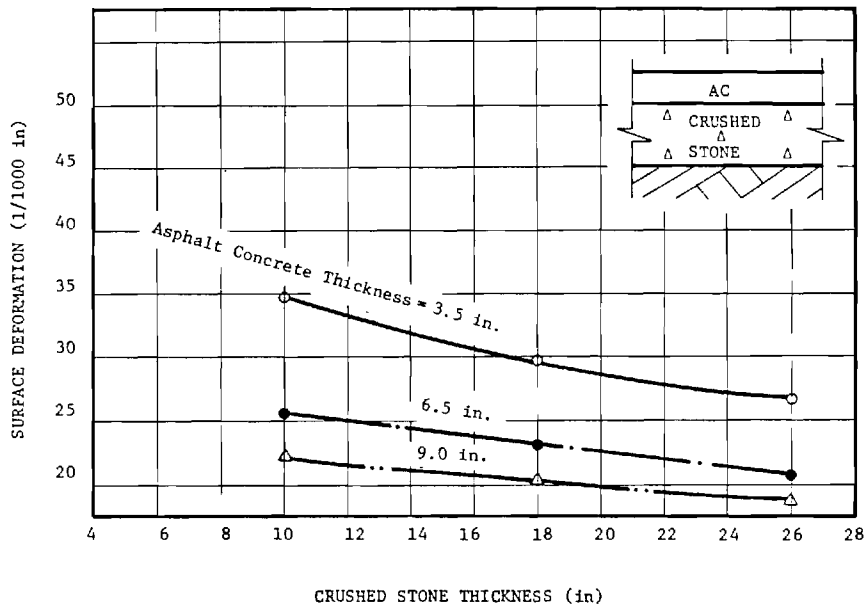


FIGURE A.6: CRUSHED STONE BASE: GOOD SUBGRADE MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF BASE AND SURFACE THICKNESS

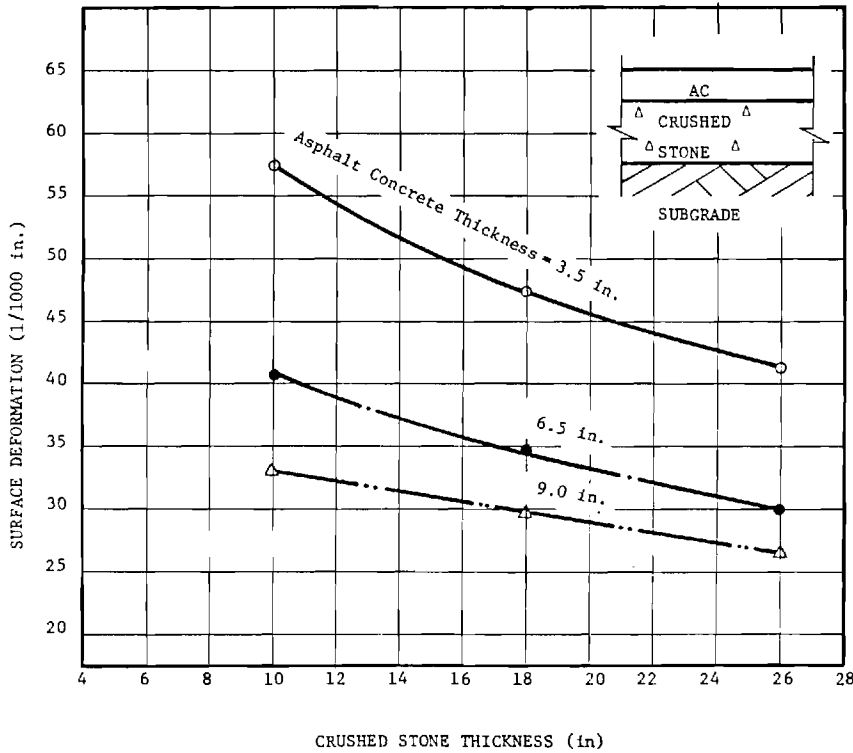


FIGURE A.7: CRUSHED STONE BASE: POOR SUBGRADE MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF BASE AND SURFACING THICKNESS

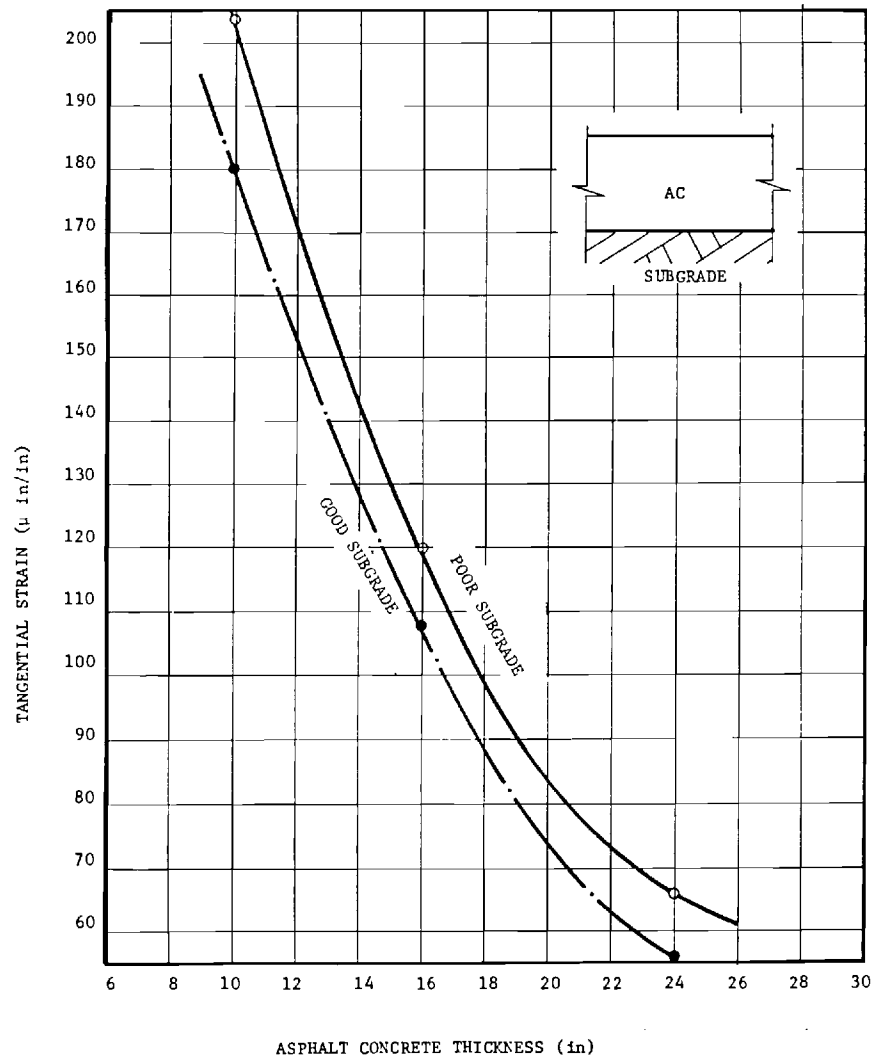


FIGURE A.8: FULL DEPTH ASPHALT: TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY

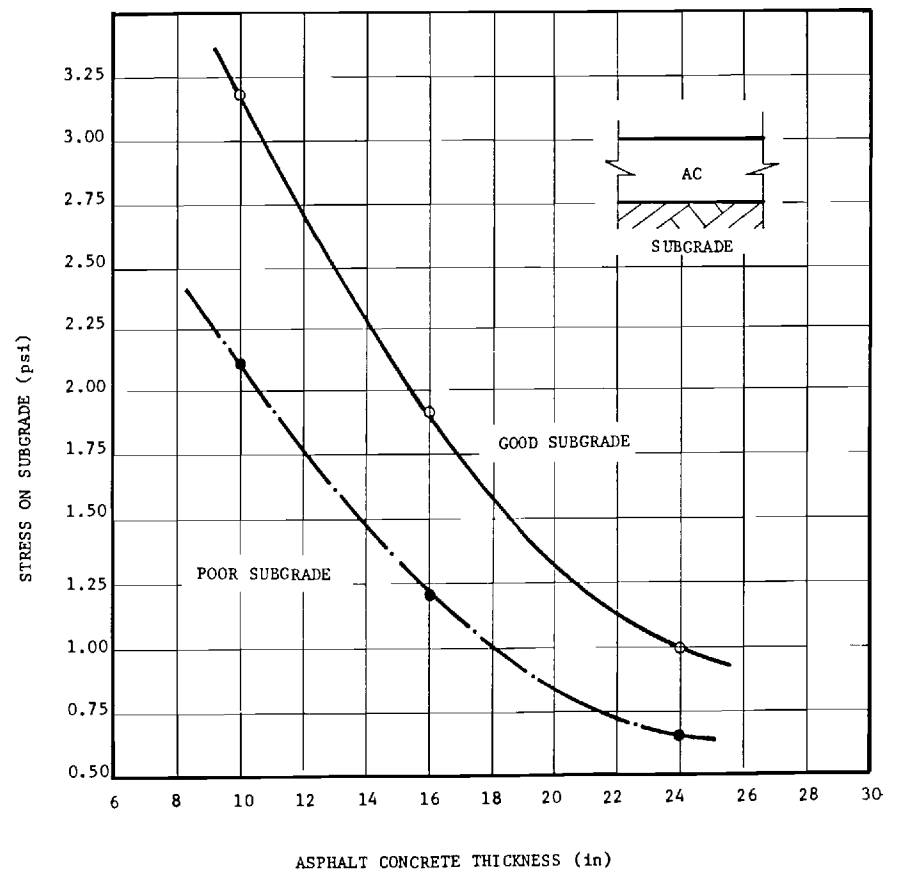


FIGURE A.9: FULL DEPTH ASPHALT: VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY.

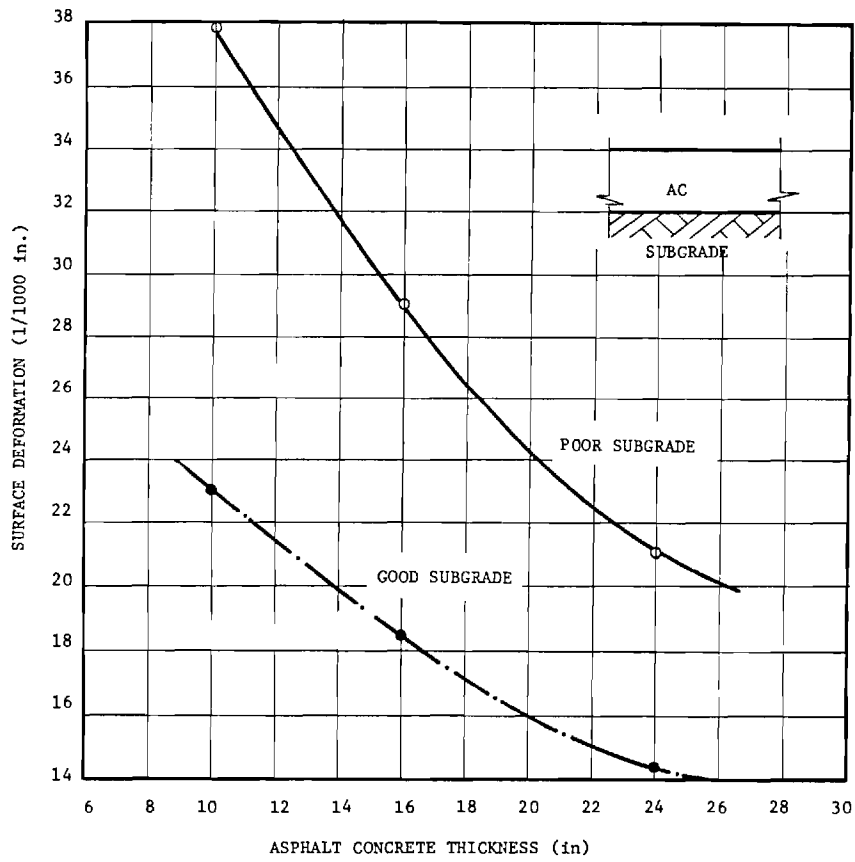


FIGURE A.10: FULL DEPTH ASPHALT: MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY

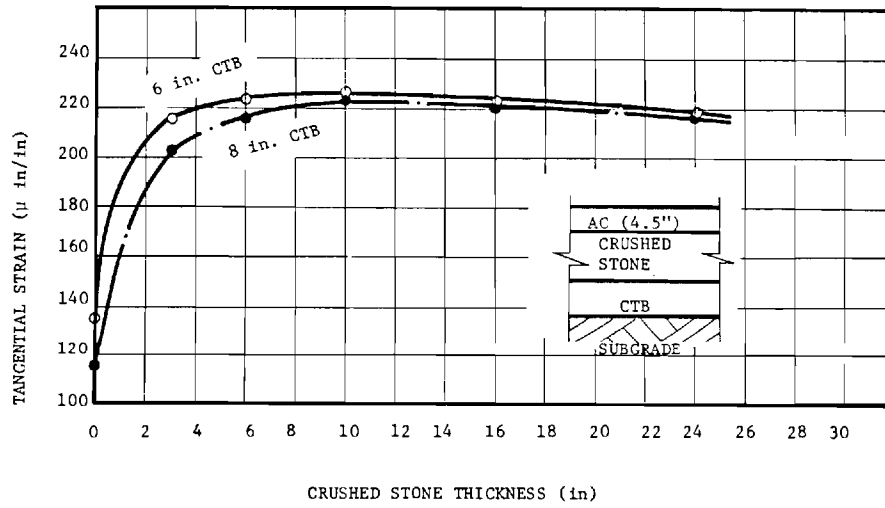


FIGURE A.11: INVERTED CONSTRUCTION: GOOD SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF CRUSHED STONE AND CEMENT TREATED BASE THICKNESS

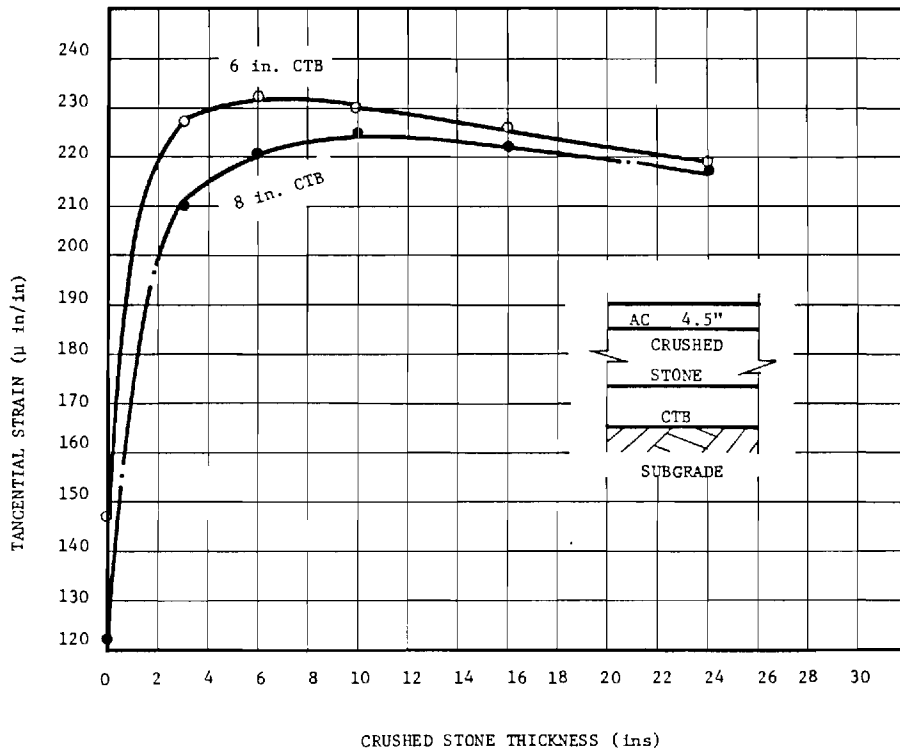


FIGURE A.12: INVERTED CONSTRUCTION. POOR SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF CRUSHED STONE AND CEMENT TREATED BASE THICKNESS

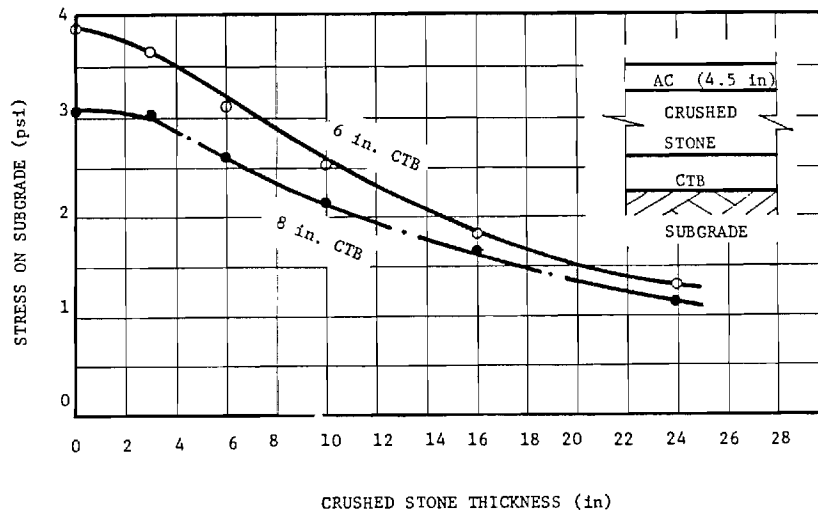
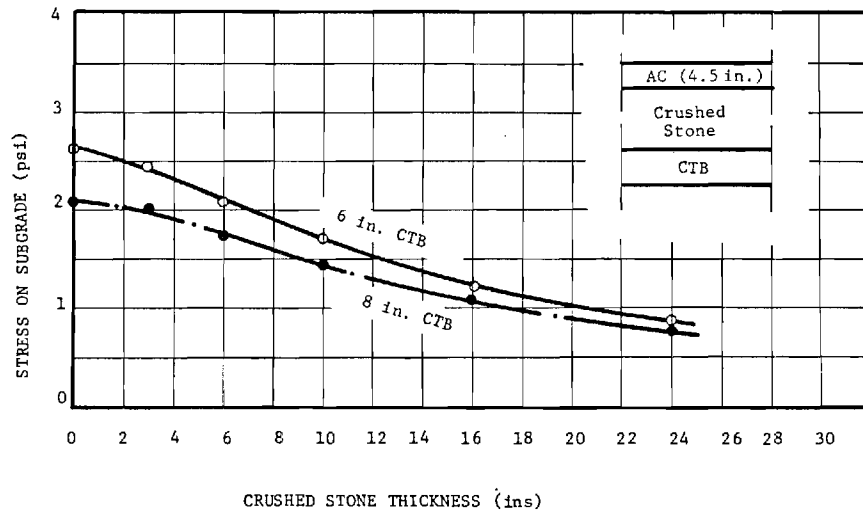


FIGURE A.13: INVERTED CONSTRUCTION. GOOD SUBGRADE. VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS



FIGURES A.14: INVERTED CONSTRUCTION: POOR SUBGRADE. VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS

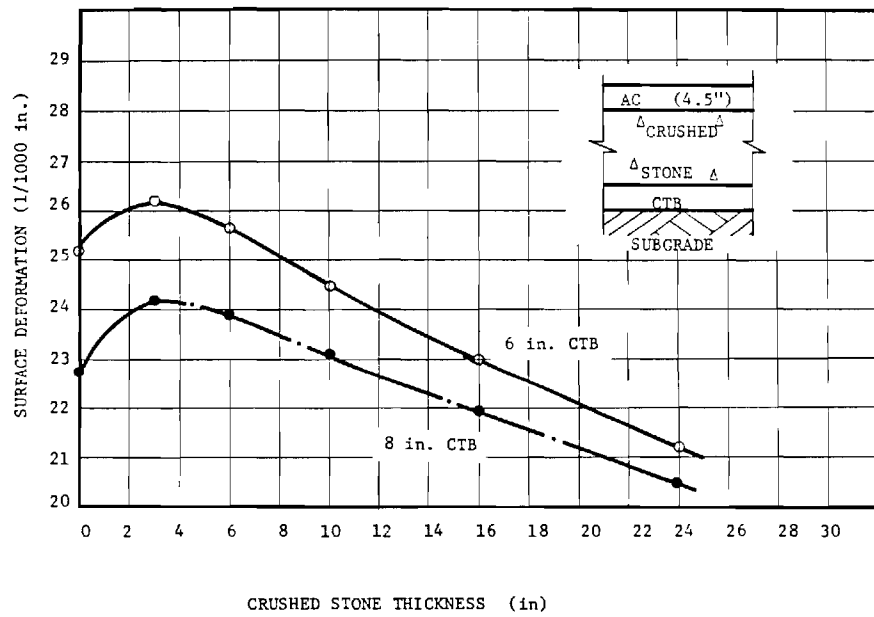


FIGURE A.15: INVERTED CONSTRUCTION: GOOD SUBGRADE MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS

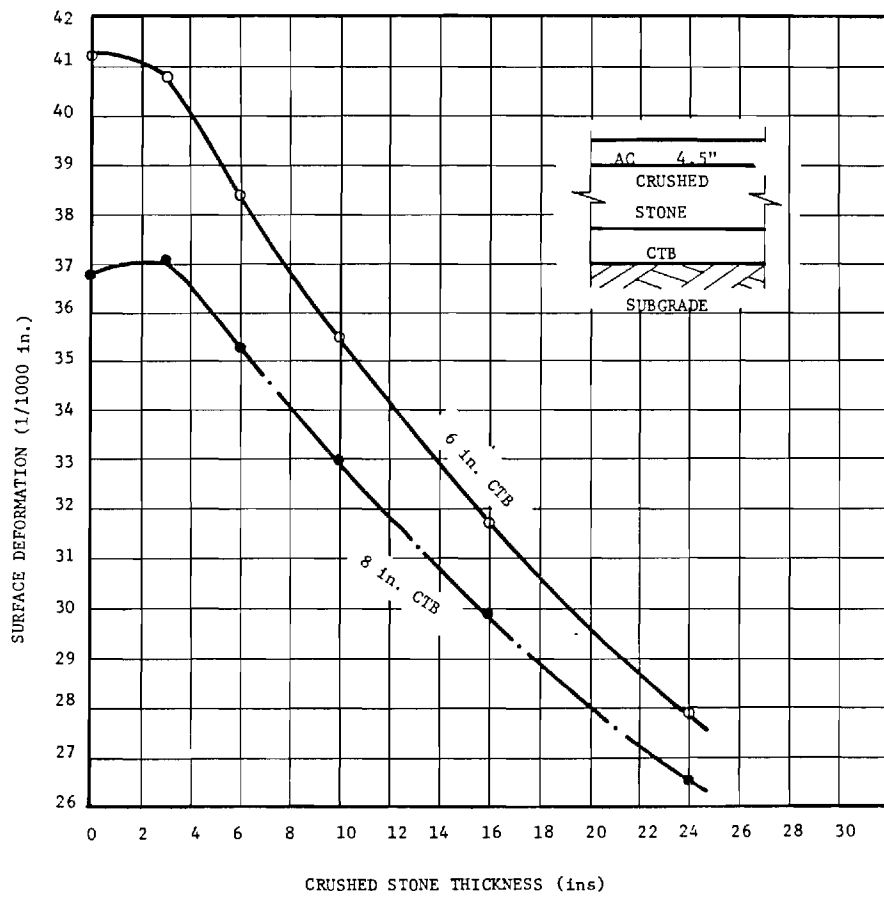


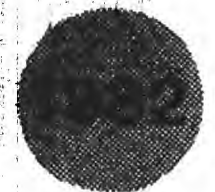
FIGURE A.16: INVERTED CONSTRUCTION: POOR SUBGRADE. MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS

CONTRACT RESEARCH
GDOT RESEARCH PROJECT NO. 7603
FINAL REPORT



A STUDY OF FACTORS
AFFECTING CRUSHED STONE
BASE COURSE PERFORMANCE

SCHOOL OF CIVIL ENGINEERING
GEORGIA INSTITUTE OF TECHNOLOGY



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16. Abstract <p>Twelve full-scale pavement sections were tested to failure in a special laboratory facility under closely controlled environmental conditions. Seven of the pavement sections were loaded to over one million repetitions, and five sections to over two million repetitions. A 6.5 kip (29 kN) uniform, circular loading was applied to the surface, and systematically moved to prevent a punching failure. A complete series of laboratory tests including fatigue and rutting were performed to characterize the materials.</p> <p>Pavements tested consisted of five conventional sections having crushed stone bases, five full-depth asphalt concrete sections, and two inverted sections. The inverted sections consisted of a crushed stone base sandwiched between a lower cement stabilized layer and an upper asphalt concrete layer. Conventional sections were tested having two thicknesses of base and three base gradations.</p> <p>The crushed stone base sections were found to give excellent performance when covered with a 3.5 in. (89 mm) thick asphalt concrete surfacing. The AASHTO Interim Guide significantly underestimated the number of repetitions this type pavement, when properly constructed, can withstand.</p>					
17. Key Words Base Course Coefficients, Failure, Fatigue, Pavements, Pavement Design, Pavement Performance, Rutting			18. Distribution Statement		
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"The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the State of Georgia or the Federal Highway Administration. This report does not constitute a standard, specification or regulation".

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CHAPTER I INTRODUCTION

In the 1960's and throughout the early 1970's deep lift and full-depth asphalt concrete pavements gained wide popularity throughout the United States. This general trend started as the result of full-depth asphalt concrete pavements showing good performance at the AASHTO Road Test. However, because of the rapidly rising cost of all petroleum products, full-depth and deep lift asphalt concrete construction is now rapidly becoming prohibitive in cost. Future pavement designs must therefore utilize alternate structural sections which can both perform satisfactorily and are less expensive than full-depth designs. Unstabilized crushed stone has been used for many years for base and subbase construction in both Georgia and also throughout the United States. During this time a considerable amount of general experience has been gained concerning the performance of granular bases. Further, two comprehensive laboratory studies using repeated load testing techniques have been performed for the Georgia Department of Transportation on both unstabilized and stabilized base materials [14,20]. Design base coefficients were developed in these studies by extending the laboratory test results to field conditions using layered theory. Full-scale test sections have not, however, been used to validate and extend the laboratory and theoretical results.

The rising costs of petroleum products dictates the use of more low-energy intensive materials such as unstabilized crushed stone compared with full-depth or deep strength asphalt concrete. An important need therefore exists for formally studying the performance of granular bases in full-scale test sections taking advantage of recent advances in materials technology and instrumentation. Use of a crushed stone base offers an attractive alternative to full-depth and deep lift asphalt concrete construction. In the mid-1960's a number of base course studies were conducted, primarily to evaluate deep lift or full-depth asphalt construction. As a result, little information was gained concerning how variables affected the performance of crushed stone bases.

One approach for studying the relative influence of selected variables on base course performance is by the use of full-scale laboratory pavement sections. Using this approach a relatively large number of sections can be studied under closely controlled conditions at a minimum cost. A primary concern is the *relative* influence of base course variables. By carefully designing and instrumenting test sections, the influence can be minimized of factors such as variable subgrade support and materials variability, which is always an important problem in full-scale test pavements.

Therefore, in studying the *relative* performance of unstabilized bases use of full-scale laboratory test sections offers an excellent approach for maximizing the amount of information that can be obtained while minimizing the cost and time involved. The results of such a study provides considerable technical knowledge concerning cost-effective structural designs utilizing low-energy intensive crushed stone. As a result, design sections can be developed which minimize the use of stabilizing agents which undoubtedly will become more expensive in the future. Further, the data gained from such test studies can be readily extended through suitable theories to many other conditions not tested in the laboratory. A laboratory study of this type, however, does not *alleviate* the need for constructing at least a limited number of field test sections.

Objectives of the Study

The general objective of this research study is to investigate selected variables which influence the design and performance of unstabilized, crushed stone bases. In specific the objectives of this study are as follows:

1. Experimentally evaluate using full-scale laboratory test sections the fatigue and rutting behavior of selected pavement sections constructed with granular bases and relatively thin asphalt concrete surfacings.
2. Determine how the following base course variables influence pavement performance: (a) thickness, (b) gradation, and (c) aggregate size.
3. Relate performance of unstabilized crushed stone bases with that of full-depth asphalt concrete and develop design recommendations.
4. Evaluate the potential advantages of placing an unstabilized crushed stone base over a cement stabilized layer.
5. Determine the influence of subgrade saturation on performance of the base course.

Research Plan

To accomplish the objectives of this study, twelve full-scale pavement sections were tested in a special facility under closely controlled environmental conditions. In the past, most full-scale pavements constructed in the laboratory have not been tested to failure because of the time and special testing apparatus required. In such tests resilient deflection has usually been used as a criteria for comparing performance. In this study each section was repetitively loaded to failure (or close to failure) by a 6.5 kip (29 kN) uniform, circular loading. The load was systematically moved to prevent a punching failure. Loading the pavement to a large number of repetitions served the important purpose of defining failure and the associated failure mechanisms. This information was quite valuable for theoretically extending the results to other structural sections.

All tests were performed in an existing test facility 8 ft. by 12 ft. in plan and 7 ft. deep (2.4 x 3.6 x 1.5 m). Temperature was controlled at

78 to 80°F (27.6 to 28.8°C) throughout the test. In addition, a complete series of laboratory tests including fatigue and rutting were performed to characterize the materials for theoretically predicting pavement behavior.

Seven of the pavement sections were loaded to over one million repetitions, and five sections to over two million repetitions. As a result, considerably more time was required to complete the project than originally planned. Because of the large number of load repetitions applied, however, the experimental findings are considered to be more representative of field loading conditions and hence of greater overall value.

Pavements tested consisted of five conventional sections having crushed stone bases, five full-depth asphalt concrete sections, and two inverted sections. The inverted sections consisted of a crushed stone base sandwiched between a lower cement stabilized layer and an upper asphalt concrete layer. Conventional sections were tested having two thicknesses of base and three base gradations. The results of full-scale field tests and layered theory are used together with the results of this study to develop practical design recommendations including recommended AASHTO base course coefficients. A general summary of the sections tested and the results is presented in Table 1.

TABLE 1. CONSTRUCTION AND PERFORMANCE SUMMARY OF PAVEMENT SECTIONS TESTED.

Section Number	Asphalt Concrete Thickness (in.)	Crushed Stone Thickness (in.)	Repetitions to Failure	Failure Mode(1)	Comments
1	3.5	12.0	3,000,000 3,500,000	Fatigue/ Rutting	Tested to 2.4 million reps.; Failure Extrapolated
2	3.5	8.0	1,000,000	Rutting	
3	9.0	NONE	10,000	Rutting (1 in.)	Bad Asphalt: AC Content 5.9% Flow: 15.4 Stab.: 1870 lbs.; $\gamma_D = 145.1$ pcf
4	6.5	NONE	10,000	Rutting (1 in.)	
5	9.0	NONE	130,000	Rutting	Rutting Failure Primarily in AC
6	6.5	NONE	440,000	Rutting	
7	7.0	NONE	150,000	Rutting	Direct Comparison of Crushed Stone and Full- Depth
8	3.5	8.0	550,000	Rutting	
9	3.5	8.0 ⁽²⁾	2,400,000	Fatigue	$\delta_p = 0.28$ in. @ 2.9 million
10	3.5	8.0 ⁽³⁾	2,900,000	Fatigue	$\delta_p = 0.34$ in. @ 2.9 million
11	3.5	8.0	3,600,000	Fatigue/ Rutting	6.0 in. Soil Cement Subbase
12	3.5	8.0	4,400,000	Fatigue/ Rutting	6.0 in. Cement Stabilized Stone Subbase

NOTE: 1. A fatigue failure is defined as Class 2 cracking; a rutting failure is defined as a 0.5 in. (12 mm) rut depth.

2. Coarse Gradation Base.

3. Fine Gradation Base.

CHAPTER II LITERATURE REVIEW

Introduction

Because of the high cost of petroleum products, unstabilized crushed stone bases when properly constructed offer an attractive alternative to full-depth and deep strength asphalt concrete construction. Only a limited number of well-documented field studies are presently available comparing the performance of pavements having bituminous and crushed stone bases. Unfortunately, serious deficiencies in granular base design were present, for example, at the AASHTO and Brampton Road Tests. These comprehensive studies have been summarized elsewhere [1] and are not discussed in detail in this chapter. Common deficiencies in granular base design include (1) use of a very thin base underlain by an inferior gravel subbase, (2) the gravel subbase was often frost susceptible, and (3) the granular layers in many experimental roads were compacted to 100% of AASHTO T-99 density rather than 100% or more of T-180 density. Several of these deficiencies were present, for example, at the AASHTO, Brampton and Colorado Test Roads [1,9].

Base course materials are usually compared using either the AASHTO Interim Guide [2] base course coefficients or else are compared in terms of a base course equivalency. The base course equivalency is expressed as the thickness of unstabilized stone required to replace 1 in. (25 mm) of bituminous base. The ratio of the AASHTO base course coefficient of the asphalt base to the crushed stone is therefore equivalent to the number of inches of crushed stone that replaces 1 in. (25 mm) of asphalt concrete. Using either method for comparing bases, it should be remembered that the equivalencies vary with many factors and hence are far from being a true constant.

In the present study 12 pavement sections were relatively rapidly loaded to failure in a closely controlled environment. Undoubtedly differences exist between the present study and actual field conditions including environmental effects, construction technique, material variability and quality control. As a result available field base performance data should be rationally integrated together with the results of the present study to develop realistic design recommendations. To develop a basic understanding of performance under actual environmental conditions the results of a number of field studies involving granular bases are therefore reviewed in this chapter.

Piedmont Residual Soils

McGhee [3] has summarized the findings of three experimental pavements constructed on Piedmont residual soils in Virginia. These studies have shown that stabilization of the upper 6 in. (152 mm) of a micaceous silt and silty clay Piedmont residual soil with 10% cement by volume is beneficial in improving performance and reducing elastic deflections. Forty to 60% of the elastic deformation occurring in the pavement sections on one test pavement was found to be caused by a 6 to 6.5 in. (152-165 mm) thick layer of select borrow placed over the cement treated subgrade. The select borrow had an average CBR of 33. Another study showed improved performance when the select borrow material was not placed between the cement treated subgrade and crushed stone base.

Under heavy traffic conditions 4 in. (102 mm) of a lean asphalt concrete mix was found to be approximately equivalent to 6 in. (152 mm) of untreated crushed stone. The granular base section had a PSI rating of 4.26 and Benkelman beam deflection of 0.034 in. (0.9 mm) compared with corresponding values of 4.17 and 0.019 in. (0.5 mm) for the bituminous base sections. The unstabilized crushed stone base section, however, had a crack factor⁽¹⁾ of 52 compared with 21 for the bituminous base section. The unstabilized base section was rated as having very minor cracking and good performance; the bituminous base section was rated as having excellent performance. Because of the greater amount of cracking in the granular base, the base course equivalency for the granular base was probably a little greater than the 1.5 value reported. However, the overall cost of the section having the crushed stone base was, at the time of construction in about 1971, 9% less than the bituminous base section.

Of interest is the finding that the inverted pavement section having a cement treated subgrade and unstabilized crushed stone base demonstrated good performance over a 5 year period. This section had the lowest overall cost and was judged to be the most cost effective design. The inverted design utilized a high level of cement stabilization corresponding to a cement content of about 9 to 10% by weight in the 6 in. (152 mm) stabilized layer of subgrade.

Granular Highway Bases and Subbases

General

The field performance of granular bases tend to vary considerably more than either bituminous or cement treated bases [1]. The observed variation in performance is due to an unstabilized base being considerably more affected than stabilized materials by such variables as magnitude of wheel load, material type, compaction density, frost susceptibility, grain size, gradation and angularity.

Field compaction tests on crushed stone bases described by Nichols and James [4] have shown that compaction to 100% of AASHTO T-99 density failed to give densities nearly as high as could be readily attained in the field with a conventional vibratory roller. Further, the average increase in density in going from 30 to 50 coverages of the vibratory roller caused an average increase in density of 3.4% at the 40 sites studied. Test strips such as reported to be used by Ohio might be utilized at the beginning of a project to establish minimum allowable compaction densities [4]. If test strips are used however, the minimum allowable density should not be less than 100% of AASHTO T-180 density [1].

1. Crack factor is obtained by assigning a factor of 15 units and 20 units for each incidence of longitudinal and alligator cracking, respectively, in a 1000 ft. (305 m) segment.

AASHTO Road Test

The results of the AASHTO Road Test [5] are relatively well-known and will only be briefly summarized. At the AASHTO Road Test most of the rutting was found to occur in the asphalt concrete surfacing (42%) and sand gravel subbase (41%) indicating the importance of using high quality materials in not only the surfacing and base, but also in the subbase. Only 9% of the rutting occurred in the base and 8% in the subgrade. The densities of all the layers tended to increase beneath the outer wheel path. Densities beneath the inner path tended to increase in all layers except the subbase where a definite tendency existed for a decrease in density. McNaughton and Rand [6] found in Maine that gravel bases may either increase or decrease in density with time.

The significant effect which magnitude of the wheel loading has on the relative performance of materials is clearly shown from the results of the AASHTO Road Test (Fig. 1). The unstabilized aggregate base-subbase sections clearly showed significantly better performance with increasing magnitude of wheel load. For a 12 kip (53 kN), single axle loading, 3 in. and 1.9 in. (76 and 48 mm) of crushed stone were required to replace 1 in. (25 mm) of bituminous and cement treated base, respectively. For a 30 kip (130 kN) single axle loading, 1.3 in. and 1.2 in., respectively were required to replace 1 in. (25 mm) of bituminous and cement treated materials. These comparisons are valid for the conditions existing at the AASHTO Road Test including: (1) 1×10^6 wheel load applications applied in only two years; (2) granular materials compacted to only 100% of T-99 density; (3) a frost susceptible, sand-gravel subbase; and (4) the other material properties summarized in Reference [1].

New Jersey Experimental Pavement Project

Nine experimental test pavements were constructed in 1964 on Route I-80 and I-95 in New Jersey [7]. After eight years of service a detailed study [7] indicated all sections were performing satisfactorily after 1.2 million equivalent 18 kip (80 kN) single axle loads; these sections have continued to perform satisfactorily until the present time (1981). To eliminate pavement subgrade support as a variable in the study, the pavement sections were constructed in rock cut areas. The structural sections used are summarized in Table 2, and the gradations of the unstabilized stone bases are given in Table 3.

The performance of the nine test sections is summarized in Table 4. After 1.2 million equivalent 18 kip (80 kN) single axle loadings, best performance was shown by the following sections: (1) the deep strength bituminous section with 10 in. (254 mm) of bituminous material (Section 2), (2) an inverted section having a 4 in. (102 mm) dense graded stone base overlying an 8 in. (203 mm) cement treated base, and (3) Section 9 which had both a 6 in. (152 mm) dry bound macadam base and quarry processed stone subbase. Of interest is the fact that Section 9, the only thinner asphalt concrete section (6 in. of AC) exhibiting above average performance, used an unstabilized layer of dry bound macadam overlying quarry processed stone. Both of these materials had a coarser gradation (Table 3) than conventional graded aggregate bases.

Section 3 also showed above average performance. This section consisted of 4 in. (102 mm) of AC, 4 in. (102 mm) of bituminous base and 6 in. (152 mm)

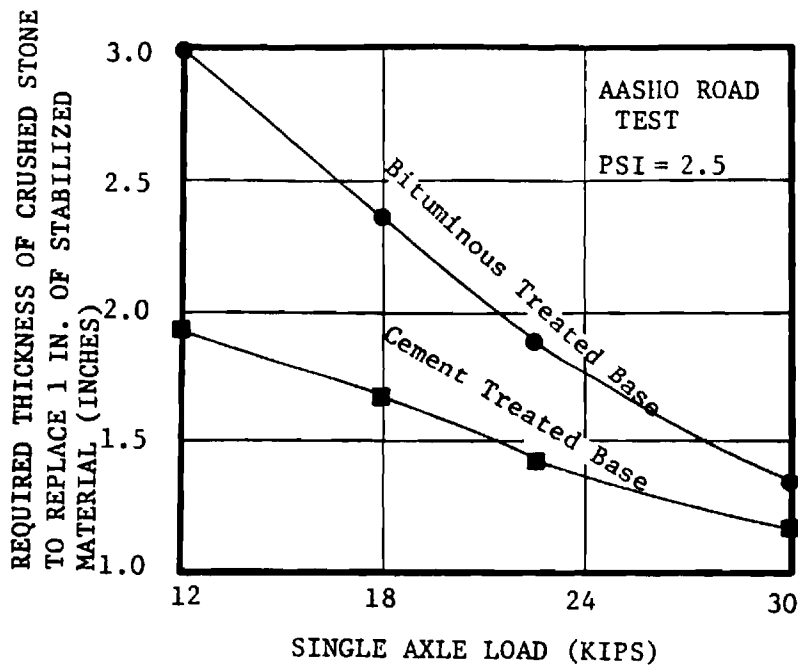


FIGURE 1. RELATIVE PERFORMANCE OF SPECIAL BASES AT AASHTO ROAD TEST AS A FUNCTION OF AXLE LOAD .

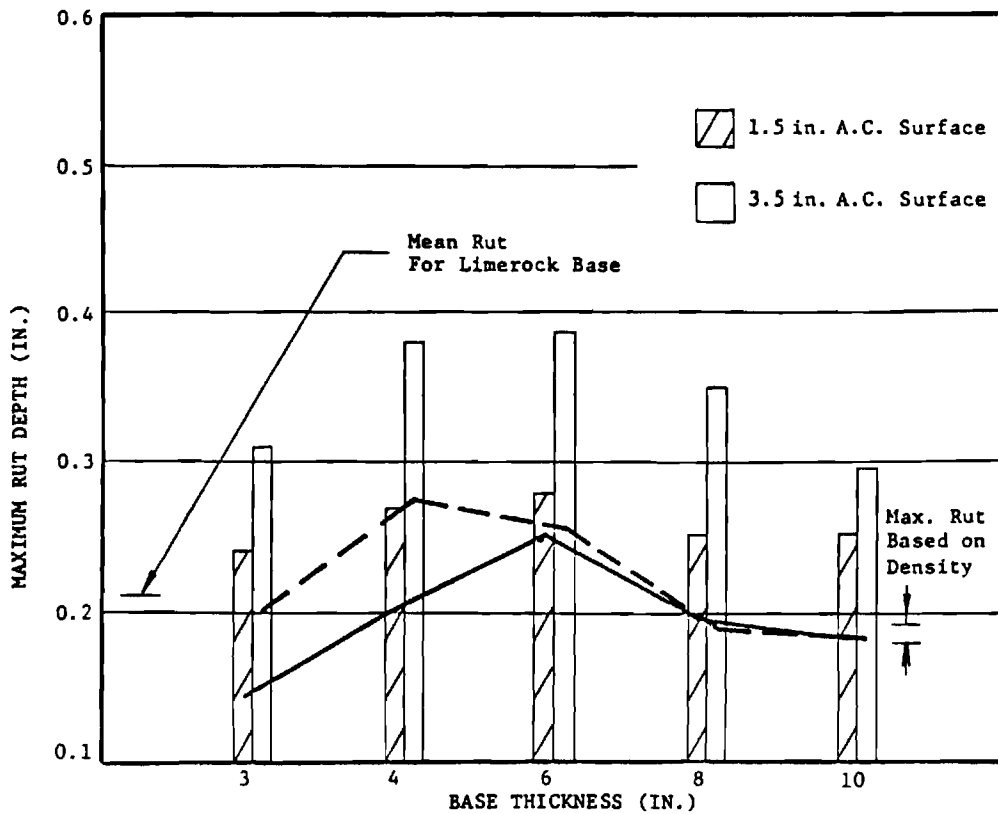


FIGURE 2. SURFACE RUTTING IN LIMEROCK BASE SECTIONS AT LAKE WALES [8] .

TABLE 2. STRUCTURAL SECTIONS USED IN NEW JERSEY EXPERIMENTAL PAVEMENT PROJECT

Section	Construction				Rut ⁽¹⁾ (in.)	Crack ⁽¹⁾ (%)	R ⁽²⁾	PSI	Perf. (3) Rating	Rank
	Surface (in.)	Base 1 (in.)	Base 2 (in.)	Subbase (in.)						
1	4 AC	6 DBM	6 QPS	14 G	0.26	78	97	2.9	178	5
2	4 AC	6 BB	6 QPS	14 G	0.34	22	86	3.3	132	2
3	4 AC	4 BB	6 QPS	16 G	0.34	48	120	2.8	168	4
4	4 AC	6 BGB	6 QPS	14 G	0.53	62	87	3.1	200	7
5	4 AC	6 PM	6 QPS	14 G	0.29	95	86	3.1	197	6
6	4 AC	6 DGA	6 QPS	14 G	0.32	82	108	2.8	197	6
7	2 AC	4 BB	4 DGA	8CTB ⁽⁴⁾	0.32	22	108	2.8	137	3
8	2 AC	4 BB	6 DBM	6QPS ⁽⁴⁾	0.21	-	144	2.4	-	-
9	2 AC	4 BB	6 DBM	6QPS ⁽⁴⁾	0.24	5	115	2.7	104	1

- Notes:
1. Average rutting and cracking in inside lanes, east and west bound traffic.
 2. Average roughness, east and west bound inside lanes (Roughometer - units per mile).
 3. Cumulative % failed considering equally rutting, cracking and PSI; Rutting failure taken as 0.45 in; Example Sect. 1: $0.26/0.45 \times 100 + 78 + \{100 - (2.9/5.0) \times 100\} = 178$.
 4. A 12 in. gravel subbase is located beneath.
 5. Notation Used: AC = Asphalt Concrete; DBM = Dry Bound Macadam; QPS = Quarry Processed Stone; G = Gravel; BB = Bituminous Base; BCB = Bituminous Gravel Base; PM = Penetration Macadam; DGA = Dense Graded Aggregate; CTB = Cement Treated Base

TABLE 3. SUMMARY OF BASE COURSE GRADATIONS FOR NEW JERSEY EXPERIMENTAL PAVEMENT.

Stone	Gradation											
	3-1/2	3	2-1/4	2	1-1/4	3/4	3/8	1/4	No. 4	No. 30	No. 40	No. 200
Quarry Processed	-	-	-	100	-	55-90	-	-	25-60	-	15-30	5-12
Macadam (Dry Bound)												
Stone	100	85-100	0-45	-	0-5	-	-	-	-	-	-	-
Screenings	-	-	-	-	-	100	-	40-100	-	25-65	-	5-20
Penetration Macadam(1)	100	85-100	0-45	-	0-5	-	-	-	-	-	-	-
Dense-Graded Stone	-	-	-	-	100 ⁽²⁾	80	-	68 ⁽³⁾	50	-	21	11
Bituminous Base												
Stone	-	-	-	-	100 ⁽²⁾	55-90	-	-	25-60	-	15-30	5-12
Gravel	-	-	-	-	100 ⁽²⁾	65-100	50-90	-	37-75	-	12-30	3-8

1. Large aggregate is 2-1/2 in. size and choke aggregate was 5/8 in. or 1/2 in. size. An 85-100 or 100-120 penetration grade asphalt cement was used.
2. 100% passing the 1-1/2 in. sieve.
3. 68% passing the 1/3 in. sieve.

of quarry processed stone. Section 6 had the thinnest bituminous surfacing used in the experiment. This section consisted of 4 in. (102 mm) of asphalt concrete overlying 6 in. (152 mm) of dense-graded stone and 6 in. (152 mm) of quarry processed stone. Baker and Quinn [7] ranked this section poorest; the ranking given in Table 4 also indicates poor performance with this section being next to last.

Summary. Important findings from this study related to granular bases are as follows: (1) the inverted pavement showed excellent performance under service conditions. The ranking given by Baker and Quinn was highest; Table 4 places this section as third best with not much difference between it and the second ranked deep asphalt concrete section, (2) the dry bound stone macadam performed better than the dense-graded stone base exhibiting the smallest rut depths of all sections. The dry bound macadam had a 3.5 in. (89 mm) top size and less than 5% passing the 3/4 in. (19 mm) sieve. The stone screenings used to choke the dry bound macadam had a 3/4 in. (19 mm) top size and 5 to 20% passing the No. 200 sieve. In comparison, the dense-graded aggregate base had a similar gradation to the stone used at the AASHO Road Test: the top size was 1.5 in. (38 mm) with 11% fines. The coarser quarry processed stone may have performed better than the dense-graded aggregate although more data is needed.

Section 4, which had a total of 10 in. (254 mm) of asphalt concrete, showed below average performance and the largest amount of rutting. This section had 6 in. (152 mm) of bituminous stabilized gravel base, 6 in. (152 mm) of quarry processed stone and a 14 in. (356 mm) gravel subbase. Generally thinner asphalt concrete thicknesses gave lower rut depths. Sections 8 and 9, which had a dry macadam stone base showed the smallest rut depths. Both these sections had a relatively thin asphalt concrete surfacing (6 in. or 152 mm).

Surface cracking, which was limited to the upper lift, was related to hardening of the asphalt as defined by penetration. Traffic loading apparently prevented visible cracks from appearing on the surface in the truck lanes. Since the more lightly trafficked passing lane cracked, healing of the cracks due to repeated loading in the heavy traffic lanes apparently occurred.

Florida Experimental Pavements

Experimental pavements have been constructed and carefully monitored by the Florida DOT at Palm Beach, Lake Wales and Marianna [8]. Limerock bases are frequently used in Florida and were included in the Palm Beach and Lake Wales experimental pavements. Unstabilized limerock bases typically have about 25 to 40% material finer than the No. 200 sieve. The limerock is compacted following Florida DOT specifications to a minimum of 98% of AASHTO T-180 density. Limerock bases tend to increase in stiffness with age possibly due to cementing [8]. These bases, however, are also quite susceptible to water that enters the pavement; as a result rapid deterioration develops if the pavement cracks allowing water to enter [1,8].

Lake Wales. The Lake Wales experimental road consisted of either 1.5 or 3.0 in. (38 or 76 mm) of asphalt concrete surfacing overlying a 3 to 10 in. (76 - 254 mm) thick base constructed of either sand asphalt or limerock.

TABLE 4. PERFORMANCE OF NEW JERSEY EXPERIMENTAL PAVEMENTS. ⁽¹⁾

Rank	Section	Construction ⁽⁶⁾					Avg. Rut ⁽²⁾ (in.)	Crack ⁽³⁾ Avg. (%)	PSI (Avg.)	Performance Rating ⁽⁴⁾	SN ⁽⁷⁾	Remaining Loading (x 10 ⁶)
		AC Surf. (in.)	Bit. Base (in.)	Stone Base (in.)	Stone Base (in.)	Subbase (in.)						
1	9	2	4	6 DBM	6 QPS	12 G	0.24	5	2.7	104	4.52	6.5
2	2	4	6	6 QPS	-	14 G	0.34	22	3.3	132	5.26	8.6
3	7	2	4	4 DGA	8 CTB	12 G	0.32	22	2.8	137	5.24	16.2
4	8	2	4	6 DBM	6 QPS	12 G	0.21	-(5)	2.4	≤148 ⁽⁵⁾	6.0	7.3
5	3	4	4	6 QPS	-	16 G	0.34	48	2.8	168	4.48	4.0
6	1	4	-	6 DBM	6 QPS	14 G	0.26	78	2.9	178	4.42	4.2
7	5	4	-	6 PM	6 QPS	14 G	0.29	95	3.1	197	5.26	3.5
7	6	4	-	6 DGA	6 QPS	14 G	0.32	32	2.8	197	4.42	2.1
8	4	4	6	6 QPS	-	14 G	0.53	62	3.1	200	5.26	5.8

- Notes:
1. Based on Ninth Interim Report Data.
 2. Average of truck lane, both wheel paths and both east and west sections.
 3. Average of inside lane for both east and west directions.
 4. Performance rating is the sum (expressed as a percent) of rutting deterioration, cracking and PSI loss.
Rut failure is taken as 0.45 in.; Example (Sect. 9), Performance Rating = $0.24/0.45 \times 100 + 5 + (1.0 - 2.7/5.0) \times 100 = 104$
 5. Cracking data on Section 8 was not given; Performance rating is estimated.
 6. Notation Used: DBM = dry bound macadam; QPS = Quarry processed stone;
G = Gravel; DGA = Dense graded aggregate;
CTB = Cement treated base; PM = Penetration macadam
 7. AASHO Structural Number using New Jersey layer equivalents; Soil Support = 6; Regional Factor = 1.25

Approximately 2.5 million equivalent 18 kip (80 kN) single axle loadings have been applied to the north bound traffic lane. The limerock base had an average of 81% passing the No. 10 sieve, 62.6% passing the No. 40 sieve and 35.0% passing the No. 200 sieve. The limerock was compacted between 97 and 101.5% of AASHTO T-180 maximum density. The optimum moisture content of the limerock was 11.1% compared with a field placement moisture of 10.1%. The sand asphalt consisted of 50% of a local, rounded "blow" sand and 50% screenings stabilized with 7.7% asphalt. All of the sand passed the No. 10 sieve and 2% passed the No. 200 sieve.

Potts, et al. [8] estimated that about 20 to 25% of the rutting occurred in the 1.5 in. (38 mm) surfacing, and 75 to 80% in the limerock or sand asphalt base section. In all sections having a 3 in. (76 mm) asphalt concrete surface, about 35% of the total rutting occurred in this layer. These percentages, however, appear to neglect rutting caused by shear distortion.

The sand asphalt base sections rutted about 50% more than the limerock base sections (maximum rut depths of 0.43 in., 11 mm, compared with 0.30 in., 8 mm). Almost identical levels of rutting were also observed at the Palm Beach Test Road for similar sections but considerably lighter traffic. Slightly more rutting occurred at Lake Wales in the limerock base sections having 3.0 in. (76 mm) of asphalt concrete compared with 1.5 in. (38 mm) as shown in Fig. 2. This is true for all base thicknesses indicating the asphalt concrete surface mix used at Lake Wales was more susceptible in the warm climate to rutting than the limerock base placed under a thin covering of asphalt concrete.

Surface cracking of the sand asphalt sections was also greater than for the limerock base sections (Fig. 3). Further, the limerock base sections having 1.5 in. (38 mm) of asphalt concrete surface exhibited no more (and perhaps even less) cracking than the sections having a 3.0 in. (76 mm) asphalt concrete surfacing. The Present Serviceability Index (PSI) of the sand asphalt sections after 2.5 million equivalent single axle loadings was about equal to or slightly higher than the limerock bases. This difference in performance was attributed by Potts, et al. to the initial difference in the pavement profile before traffic loading associated with differences in construction techniques.

Palm Beach. The Palm Beach test pavement consisted of a 1.5 in. (38 mm) thick asphalt concrete surfacing having a mean Marshall stability of 860 lbs. (3.8 kN) placed over either sand asphalt, shell or limerock bases varying from 3 to 9 in. (76 - 229 mm) in thickness. A total of 120,000 equivalent 18 kip (80 kN) single axle loads (both directions) were applied during 9 years. At the lightly trafficked Palm Beach Test Road after nine years the PSI of all sections except two having sand asphalt bases was greater than 4.0. Rutting was the only difference in performance. Rutting in the limerock base section was about 0.3 in. (8 mm) compared with 0.4 in. (10 mm) in the higher stability sand asphalt base sections and 0.5 in. (13 mm) in the shell base sections.

Colorado Experimental Base Project

A series of twenty full-scale test sections each 450 ft. (13 m) in length were constructed in Colorado in 1965 and monitored through 1978 to evaluate base course performance [9]. During twelve years of service,

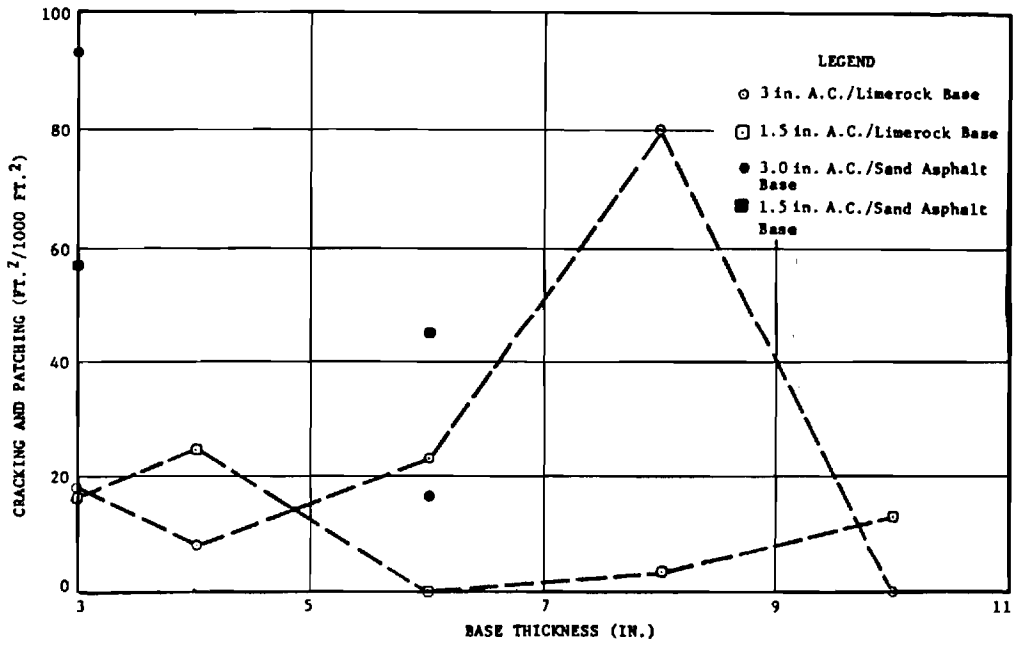


FIGURE 3. MAXIMUM CRACKING AND PATCHING OBSERVED ON LIMEROCK AND SAND ASPHALT SECTIONS AT LAKE WALES [8]

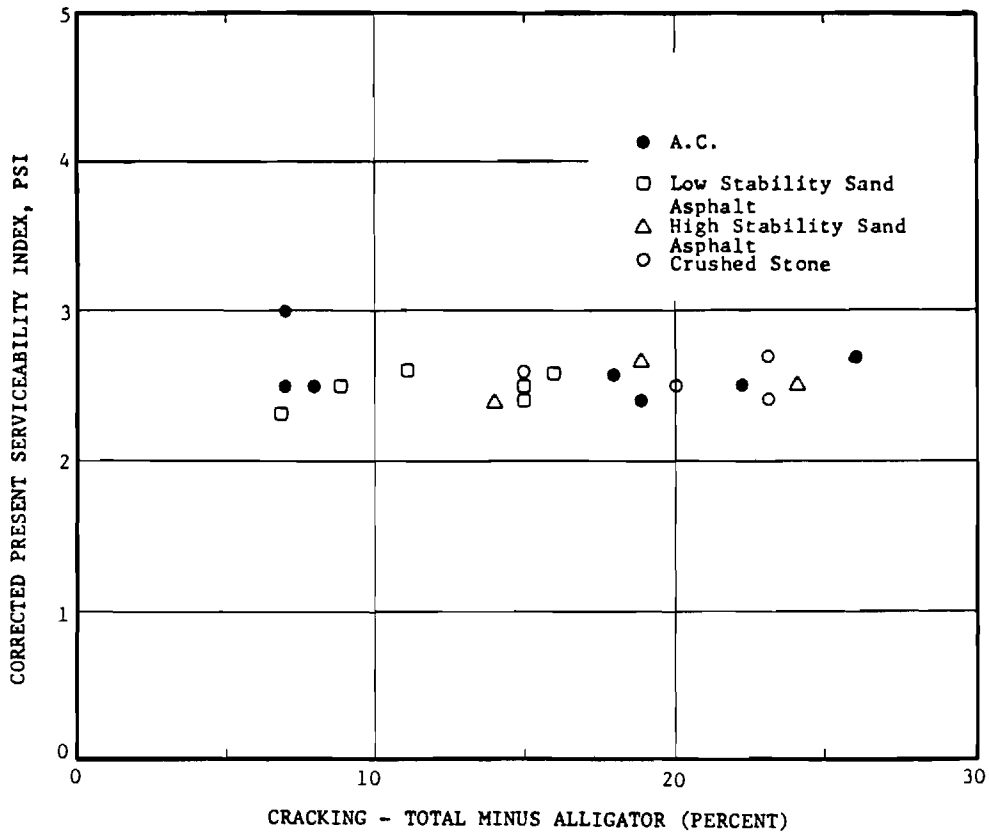


FIGURE 4. EFFECT OF CRACKING ON PRESENT SERVICEABILITY INDEX - COLORADO BASE STUDY [9].

only 140,000 equivalent 18 kip (80 kN) axle loads were applied to the pavement. Structural designs evaluated include two full-depth sand-asphalt bases, one full-depth asphalt concrete base, and one design having an untreated gravel base and a subbase of varying thickness. Full-depth asphalt concrete and sand asphalt sections had thicknesses varying from 5.5 to 9.4 in. (140-216 mm). The conventional sections had an average asphalt concrete thickness of 2.6 in. (66 mm), a 4 in. (102 mm) gravel base and 14 in. (356 mm) gravel subbase. Sections were constructed on both an A-7-6 and A-6 subgrade.

The asphalt concrete surface and base mix had an asphalt content of 5.6%. The well-graded coarse aggregate fraction of the surface course had at least 60% crushed stone with a minimum of two crushed faces. An uncrushed gravel was used in the asphalt concrete base with this mix being slightly coarser than the surface. The sand asphalt had 8% asphalt and 3 to 6% air voids. Marshall stability of the low stability mix is 451 lb. (2 kN) and the high stability mix 770 lbs. (3.4 kN); mineral filler was used to achieve the higher stability mix. The sand asphalt was placed at an average of 98.6 to 98.9% of the 50 blow Marshall density and the asphalt concrete at 99%. The 1/2 in. (12 mm) maximum size uncrushed gravel used in the thin base had 9% passing the No. 200 sieve. The 3 in. (76 mm) maximum size, uncrushed gravel subbase had 5% passing the No. 200 sieve. The average density of the base and subbase was 101.7 and 102.5%, respectively of the AASHTO T-99 density. Gravel densities varied from 133 to 136.7 pcf (20.9 - 21.5 kN/m³) for one standard deviation.

Performance. Typical test results are summarized in Figs. 4 and 5. A statistical study indicated no practical difference in PSI between the asphalt concrete, sand asphalt and unstabilized gravel base sections after 12 years (Fig. 4). Rutting in the full-depth bituminous sections was about twice that in the unstabilized gravel base sections (Fig. 5). After 7 years, little difference in alligator cracking existed between the full-depth asphalt and unstabilized gravel base sections. During the seventh to ninth years, however, more alligator cracking occurred in the gravel base sections than in the full-depth bituminous sections. The full-depth asphalt concrete sections placed on the A-7-6 subgrade had cracks over an average of 22% of the area compared with an average of 32% for the unstabilized crushed stone base. Alligator cracking was most important in the unstabilized base sections while transverse and longitudinal cracking was greater in the full-depth sections. No significant difference in alligator cracking existed between the full-depth bituminous sections.

After 12 years of service the PSI values for both the asphalt concrete sections and the granular base/subbase sections were about the same. Also a significant amount of cracking was present in both the bituminous (22%) and granular base (32%) sections. The unstabilized base sections had an average of 0.13 in. (3 mm) of rutting compared to 0.19 in. (48 mm) of rutting in the full-depth bituminous sections. From considerations of cracking, Shook and Kallas [9] recommend base equivalencies of 3.4 to 4.25 for unstabilized base/subbase design for the materials used in the study. Considering the PSI ratings were the same, these ratios are considered too high. Due to the light traffic, the relatively thick granular base sections would very likely have undergone about the same rutting and cracking had a thinner subbase been used. For the light traffic conditions of the study, use of a crushed gravel compacted to 100% of T-180 maximum density would have significantly improved performance. For these conditions considerably higher substitution ratios would be appropriate.

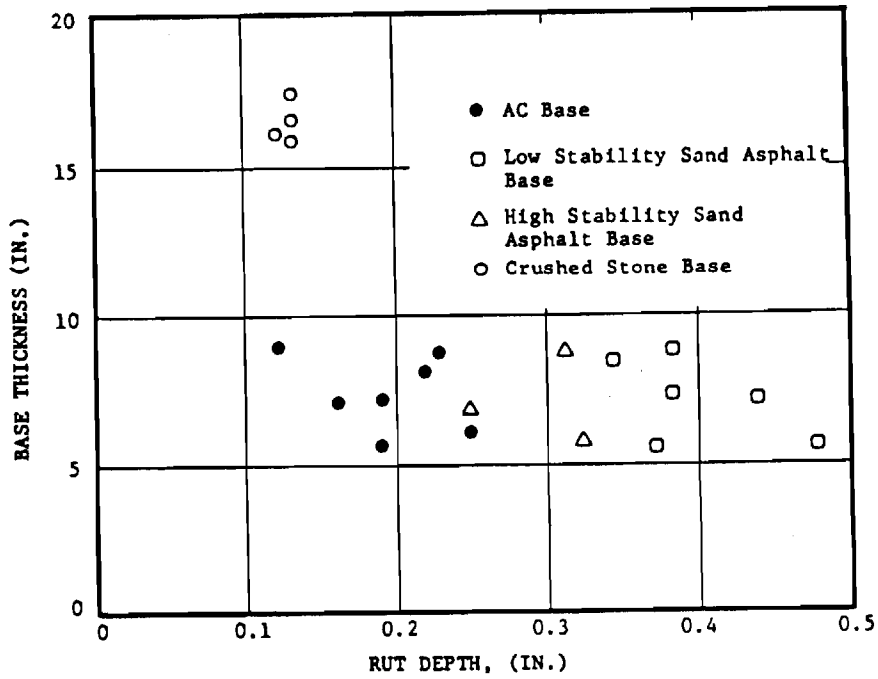


FIGURE 5. INFLUENCE OF BASE TYPE AND THICKNESS ON RUT DEPTH [9]

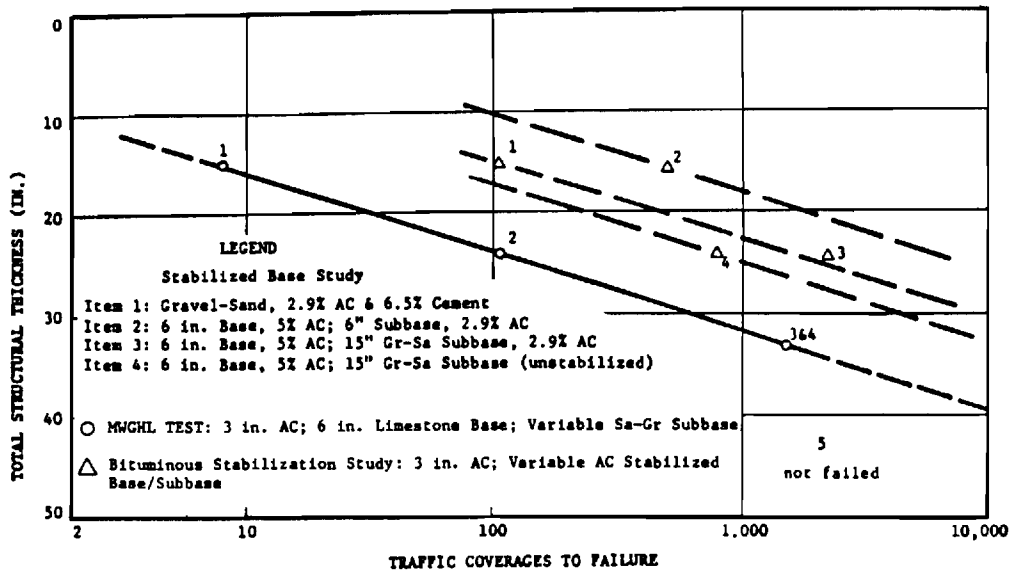


FIGURE 6. EFFECT OF STRUCTURAL THICKNESS AND TYPE CONSTRUCTION ON COVERAGES TO FAILURE - 360,000 LB., 12-WHEEL ASSEMBLY, 100 PSI TIRE PRESSURE [12].

only 140,000 equivalent 18 kip (80 kN) axle loads were applied to the pavement. Structural designs evaluated include two full-depth sand-asphalt bases, one full-depth asphalt concrete base, and one design having an untreated gravel base and a subbase of varying thickness. Full-depth asphalt concrete and sand asphalt sections had thicknesses varying from 5.5 to 9.4 in. (140 - 216 mm). The conventional sections had an average asphalt concrete thickness of 2.6 in. (66 mm), a 4 in. (102 mm) gravel base and 14 in. (356 mm) gravel subbase. Sections are constructed on both an A-7-6 and A-6 subgrade.

The asphalt concrete surface and base mix had an asphalt content of 5.6%. The well-graded coarse aggregate fraction of the surface course had at least 60% crushed stone with a minimum of two crushed faces. An uncrushed gravel was used in the asphalt concrete base with this mix being slightly coarser than the surface. The sand asphalt had 8% asphalt and 3 to 6% air voids. Marshall stability of the low stability mix is 451 lb. (2 kN) and the high stability mix 770 lbs. (3.4 kN); mineral filler was used to achieve the higher stability mix. The sand asphalt was placed at an average of 98.6 to 98.9% of the 50 blow Marshall density and the asphalt concrete at 99%. The 1/2 in. (12 mm) maximum size uncrushed gravel used in the thin base had 9% passing the No. 200 sieve. The 3 in. (76 mm) maximum size, uncrushed gravel subbase had 5% passing the No. 200 sieve. The average density of the base and subbase was 101.7 and 102.5%, respectively of the AASHTO T-99 density. Gravel densities varied from 133 to 136.7 pcf (20.9 - 21.5 kN/m³) for one standard deviation.

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Granular Airfield Bases

Crushed Stone and Bituminous Stabilization

Burns and Ahlvin [10] describe the results of accelerated, full-scale field tests using heavy aircraft loadings conducted at the Waterways Experiment Station. These tests compared full-depth bituminous sections of varying quality with granular base sections having a 3 in. (76 mm) asphalt concrete surfacing. The test sections were loaded to failure under a C-5A multiple-wheel, main gear loading. All sections were placed over 36 in. (0.91 m) of heavy clay subgrade (CBR = 4), which in turn was constructed on a lean clay subgrade also having a CBR of 4.

The full-depth bituminous sections were 15 and 24 in. (381 - 610 mm) in thickness and constructed with varying quality bituminous layers. The unstabilized granular sections had 12 to 39 in. (305 - 991 mm) thick granular layers with a 3 in. (76 mm) thick asphalt concrete surfacing. The granular sections consisted of a 6 in. (152 mm) thick crushed limestone base constructed over a gravelly sand subbase of varying thickness. The gravelly sand passed the 1.5 in. (38 mm) sieve with about 50% passing the No. 4 sieve, and 1% passing the No. 200 sieve.

In one series of tests, a 360,000 lb. (1602 kN), 12 wheel assembly was used having a 100 psi (689 kN/m²) tire pressure. The comparison of coverages to failure of the bituminous and granular base sections is shown in Fig. 6. One bituminous section consisted of a 12 in. (305 mm) gravelly sand base stabilized with 2.9% asphalt (Item 1) underlying a 3 in. (76 mm) asphalt concrete surfacing. The granular base MWHGL section had a 3 in. (76 mm) asphalt concrete surfacing, a 6 in. (152 mm) crushed stone base and 15 in. (381 mm) of gravelly sand subbase. The corresponding base course equivalency for 100 coverages to failure was about 1.75 for the test conditions. For a 75 kip (334 kN) single wheel loading having a 290 psi (2000 kN/m²) tire pressure, the corresponding base course equivalency was about 1.9 for similar construction. Item 3 consists of 9 in. (231 mm) of high quality asphalt concrete overlying 15 in. (381 mm) of bituminous stabilized gravelly sand. Comparing this section with the standard unstabilized MWHGL sections having the deep gravelly sand subbase gives a base course equivalency of about 2.0. Further, a base course equivalency of about 2.2 is obtained when the high quality, full-depth asphalt concrete construction (Item 2) is compared with the unstabilized base section having the deep gravelly sand subbase.

Of practical interest is the finding that the high quality, full-depth asphalt concrete section has a base course equivalency of 2.2; the lower quality full-depth AC section utilizing both crushed stone and gravelly sand materials has an equivalency of 2.0; and the full-depth bituminous section having a gravelly sand bituminous base has an equivalency of 1.75. All of these bituminous sections are compared with unstabilized granular base sections having a deep gravelly sand subbase, 6 in. (152 mm) crushed stone base and a 3 in. (76 mm) asphalt concrete surfacing. Because of the presence of the thick, lower quality gravelly sand subbase in the unstabilized sections, the most valid base course equivalency for materials of comparable quality is about 1.75 to 1.9 for the heavy aircraft loadings.

Stabilization of the gravelly sand subbase with 2.9% asphalt gives a base course equivalency of 1.75 for the unstabilized material. Further,

changing the bottom 12 in. (305 mm) of Item 1 from uncrushed gravelly sand at 2.9% asphalt content to a crushed limestone at 4.5% asphalt content results in the number of coverages to failure increasing from 98 to 424 under the 12-wheel gear loading. These results illustrate the importance of using where practical a (1) high level of stabilization and (2) high quality materials.

Inverted Sections

Introduction

Successful use of cement stabilized subbase layers overlain by unstabilized granular bases was reported by Johnson [11] in New Mexico and at the Waterways Experiment Station by Grau [12]. Use of inverted sections in Virginia having a cement stabilized subgrade have already been described. Advantages of inverted construction include: (1) better compaction of unstabilized materials placed over the stabilized layers; this layer acts as a working platform where soft subgrades are encountered, (2) optimum use is made of unstabilized crushed stone, and (3) elimination or significant reduction in reflection cracking since the cement treated layer is placed deep in the section. Inverted sections make optimum use of the tensile strength of the cement treated subbase or subgrade in bridging over a weak subgrade through "beam" action. Optimum use is also made of the excellent compressive characteristics of unstabilized aggregate since it is placed in the upper part of the pavement structure where stresses are compressive (Fig. 7). For relatively high levels of cement stabilization required for durability and fatigue resistance, some shrinkage cracking develops in the cement treated layer. Therefore the inverted section should not trap as much water in the granular base as might be expected.

New Mexico

The "inverted" or "upside down" sections were first successfully used in New Mexico on urban projects having weak subgrades; two test sections were subsequently built which included this type construction [11]. Use of inverted pavement sections in New Mexico apparently first began in 1954 when several badly broken concrete pavements near Albuquerque were overlaid with 6 in. (152 mm) of unstabilized granular base and 2 in. (51 mm) of asphalt concrete. After six years of heavy traffic, no reflection cracking or significant amount of rutting had developed.

Following these early successful uses of inverted sections, two experimental roads were constructed in New Mexico in about 1960. A typical inverted section used on the test sections constructed on U.S. 64 North of Santa Fe consisted of a 3 in. (76 mm) asphalt concrete surfacing, 6 in. (152 mm) granular base and a 6 in. (152 mm) granular subbase treated with 4% cement. A 1 in. (25 mm) maximum size aggregate was used in both the untreated and cement treated layer. The untreated base was compacted to 100% (average) of Modified Proctor maximum density. The asphalt concrete was compacted to 100% of the 75 blow Marshall maximum density.

Shortly after construction, surface cracking was only visible in sections where the cement stabilized layers were immediately below the surface. Roughness measurements showed the inverted section to be the smoothest, although a significant difference in roughness was not observed between sections. Sections having treated base course materials immediately

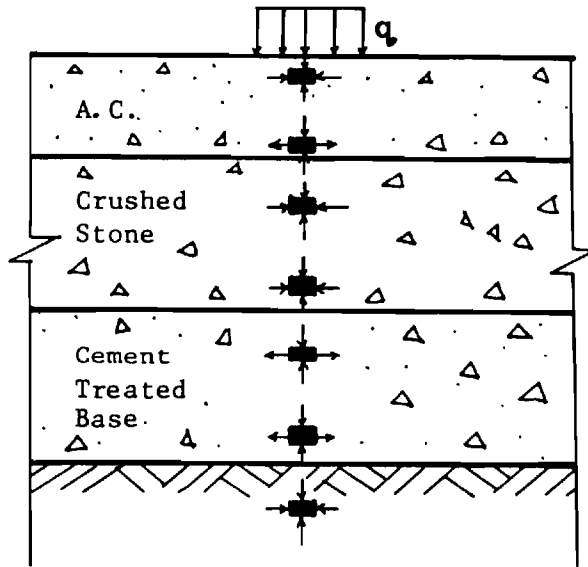


FIGURE 7. STATE OF STRESS IN AN INVERTED SECTION.

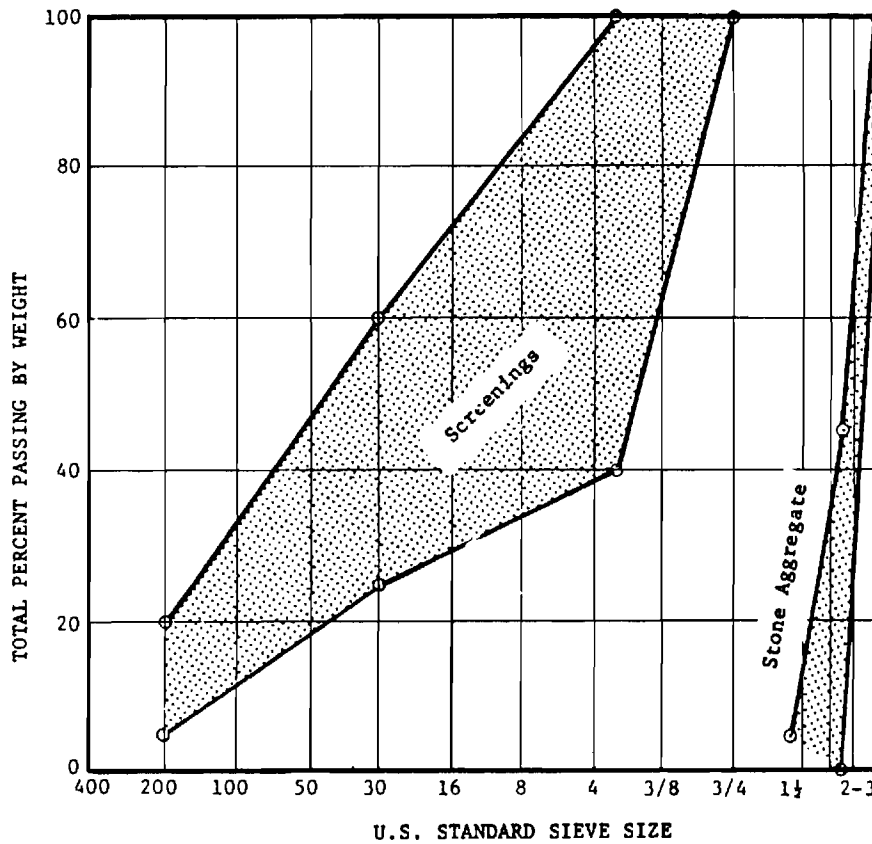


FIGURE 8. GRADATION OF STONE AGGREGATE AND SCREENINGS USED IN NEW JERSEY EXPERIMENTAL ROAD MACADAM [7]

below the surfacing were slightly rougher than others. The inverted section also had the smallest Benkelman beam deflection. As expected, the largest Benkelman beam deflections were observed in the conventional unstabilized sections. Longterm performance information was not given on these sections.

Eight test sections were also constructed on the Roads Forks-East project in New Mexico including two inverted sections. The inverted sections consisted of a 6 in. (152 mm) untreated granular base (PI=6) having a 1 in. (25 mm) maximum size aggregate. A 6 in. (152 mm) thick subbase treated with 3% cement was located beneath the base. One inverted section had a 3 in. (76 mm) asphalt concrete surfacing and 9 in. (229 mm) untreated subbase; the other inverted section had a 1.5 in. (38 mm) asphalt concrete surfacing and a 3-1/2 in. (89 mm) untreated subbase below the treated subbase.

After one year of heavy traffic no significant differences in performance were observed between the inverted and conventional pavement sections. Neither longitudinal nor transverse cracking of the mainline pavement was evident in the inverted sections; some longitudinal cracking was, however, observed on the shoulder. One section with a 6 in. (152 mm) cement stabilized base immediately below the surface had transverse and longitudinal cracking on both the inner edge of the pavement and on the shoulders. Rutting in all sections was between 0.12 and 0.25 in. (3 - 6 mm). The higher strength inverted section had 0.12 in. (3 mm) of rutting while the other inverted section had 0.25 in. (6 mm).

Macadam Bases and Open-Graded Drainage Layers

Introduction

Stromberg [13] has found in Maryland that on poor subgrades a water bound macadam performed significantly better than a dense-graded aggregate base which performed relatively badly. The water bound macadam (2-1/2 to 3 in., 64 - 76 mm, max. size stone) typically had approximately 50% passing the 1 in. (25 mm) sieve and 4 to 8% fines. In comparison, the dense graded aggregate had about 90% passing the 3/4 in. (19 mm) sieve and 7 to 15% fines. Baker and Quinn [7] found in New Jersey that a dry bound macadam base having stone up to 3-1/2 in. (89 mm) in size performed significantly better than a crushed limestone similar to that used at the AASHTO Road Test. Wisconsin and Delaware have also apparently recognized that dry and waterbound macadam bases tend to perform better than regular bases since they have assigned base course coefficients for use in the AASHTO Interim Guide [2] of 0.15 to 0.20 for macadam bases as compared with values of 0.1 to 0.14 for crushed gravel and crushed rock bases.

Possible factors contributing to the good performance of the dry and water bound macadam bases include: (1) A crushed stone up to 3 in. (76 mm) in size having a minimum number of point contacts was used that was larger in top size than dense-graded aggregate base stone, (2) Since the coarse stone is placed and vibrated before finer choker material is added to the top, the finer material added should fill the voids and not, to any significant degree, get between the contact points of the larger particles, (3) The macadam bases tend to have a coarser gradation than graded aggregate bases, and (4) The choker material used results in a relatively small percent fines in the finished water bound macadam layer. These findings together with those from WES [10,12] suggest improved performance

of a crushed stone base can be obtained using a relatively large top size stone (2-3 in., 51-76 mm, max. size) having a coarse gradation and less than 6 to 8% fines. Repeated load triaxial test results presented by Barksdale [14] agree with this concept.

New Jersey Dry Bound Macadam

In New Jersey dry bound macadam bases have been constructed using the (1) vibrating method and (2) rolling method of construction. Since more hand work is required using the rolling method, the vibrating method is of most interest. The macadam stone gradations used in the experimental road in New Jersey [7] are given in Fig. 8, and Fig. 9 gives the combined macadam gradation used in Maryland [13] for one pavement. Dry bound macadam bases have not been constructed by the New Jersey DOT since about 1965. As a result, engineers personally familiar with this type construction could not be located. The following description of dry bound stone macadam base construction is taken from the standard specifications of the New Jersey Department of Transportation.

Dry bound macadam bases constructed in lifts 4 in. (102 mm) or more thick use a 2.5 in. (64 mm) dia. stone; a 1.5 in. (38 mm) stone must be used in thinner lifts. Before placing the coarse aggregate base, an inverted choke stone layer 1 in. (25 mm) in thickness is placed on the subgrade using a stone spreader. The thin inverted layer is not, however, compacted until after the base has been placed. For base courses greater than 5 in. (127 mm) in total thickness, the vibrating method of construction is used; thinner bases are constructed using either the vibrating or rolling method.

Vibrating Method. After the choke stone layer is placed, the large stone is spread using an aggregate spreader. Two to five passes of a vibratory roller is used to key the aggregate together. The base course is then rolled longitudinally using either a 10 ton, three-wheel roller or with two or three-axle, tandem rollers weighing 8 or more tons. Rolling starts at the edges and progresses toward the center. On super elevated curves longitudinal rolling starts at the low side and progresses toward the high side. Where practical, diagonal rolling is also performed.

After rolling, the voids are filled with stone screenings. The screenings are applied using a spreader in three applications. Approximately 50% of the total quantity of dry screenings is placed in the first application and 25% in the second and third. After each application, the stone is worked into the surface using a single pass of the vibrator. After three applications, any voids visible on the surface are filled by hand spreading, brooming and rolling. The surface is then rolled again with either the three-wheel or tandem rollers until no movement of the base is visible under the action of the roller.

Rolling Method. The rolling method is used for only lifts less than 5 in. (127 mm) in thickness. Using this method the keying and compaction of the lifts composed of large aggregate is accomplished using three wheel, and two-axle or three axle tandem rollers. A stone spreader is then used to place a thin lift of dry screenings on the surface which is broomed into

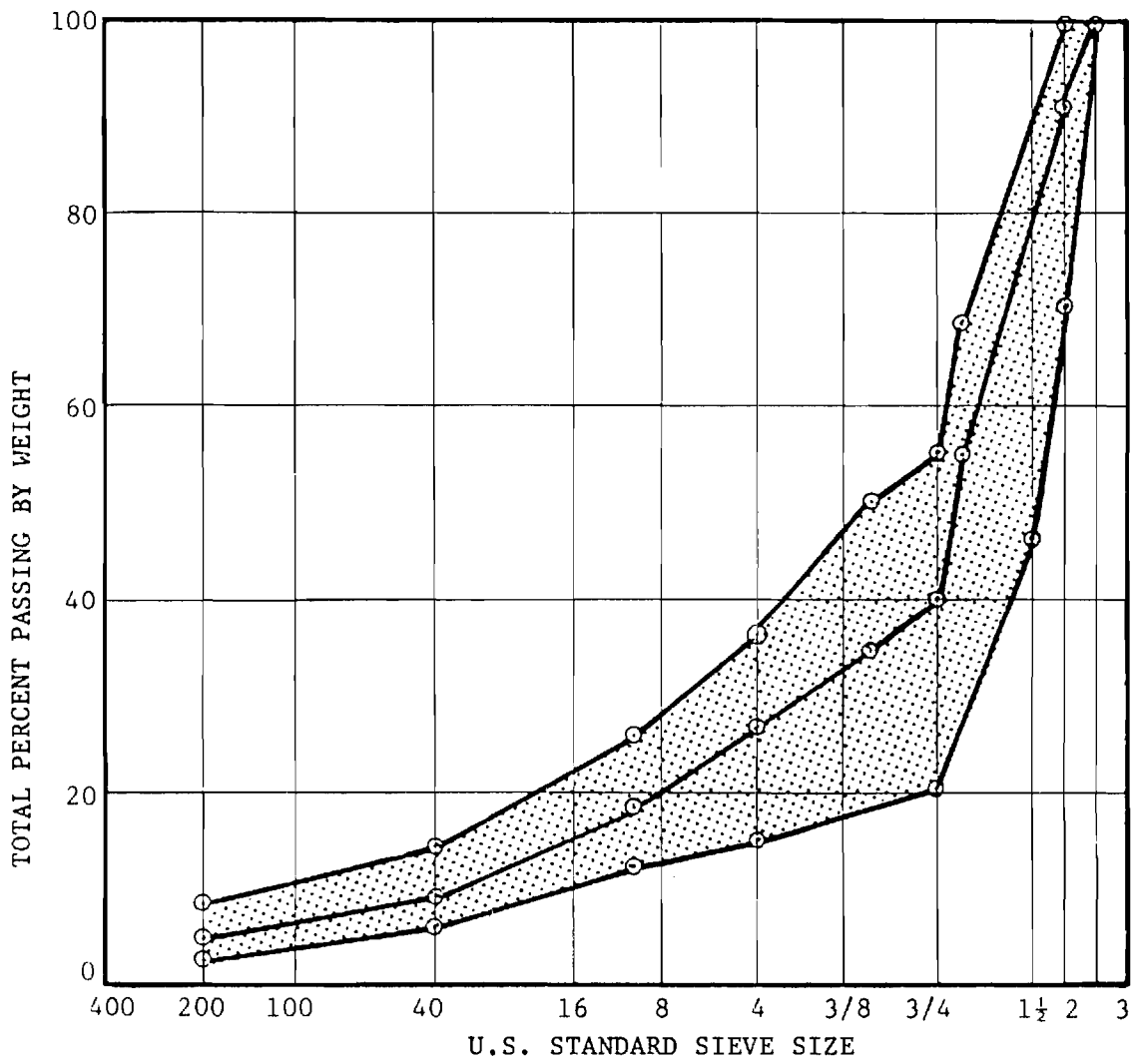


FIGURE 9. EXTREMES AND AVERAGE GRADATION OF DRY BOUND MACADAM USED IN MARYLAND [13].

the visible voids. The screenings are placed so as to neither cover nor form a layer over the coarse stone. Screenings that do not properly work into the voids are removed. In areas where all voids are not completely filled, additional screenings are added and swept into the voids. After filling all voids, the surface is rolled in the same manner as for the vibrating method of construction.

Iowa Dry Bound Macadam

In several rural counties of Iowa a number of low volume roads have been recently constructed using an open-graded crushed limestone, dry bound macadam base [15]. The base is usually covered with either a surface treatment, asphalt prime and chip seal, or about 2 in. (51 mm) of asphalt concrete surfacing. In some instances, the base has been left exposed for a year before the surfacing is applied. During this time the surface is kept smooth by blading and light applications of calcium chloride. The open-graded stone is laid full width of the section on a dense-graded stone filter layer. A steel wheel vibratory roller is used to compact all the granular layers.

The crushed stone used in the dry bound macadam is generally taken directly from the primary crusher. This material has a maximum size of 4 in. (102 mm) and less than 30% finer than the No. 8 sieve. After placing the open-graded base, a 2 to 3 in. (51-76 mm) thick layer of finer graded choke stone is placed on top to stabilize the open-graded stone and give a smooth working and/or riding surface.

In some instances a variation is used of the construction sequence described above. Following this modified procedure, the stone is separated at the quarry on a 1 in. (25 mm) sieve. The coarser fraction is used for the open-graded free draining base while the finer fraction is used for the choke stone layer.

Open-Graded Drainage Layers

Several free-draining, granular bases and subbases have been successfully constructed in the United States [16,17]. Unstabilized, very open-graded aggregate layers have generally been placed below the structural section to act as a drainage layer. In some instances the open-graded aggregate has been stabilized with asphalt for use as either a drainage layer or a combined base and drainage layer [16].

An open-graded, unstabilized granular base was constructed at the Kansas City International Airport. The very open-graded base consisted of a No. 4 stone covered with only 2.5 in. (64 mm) of asphalt concrete [17]. This open-graded base was placed over a layer of well-compacted, dense graded limestone. Unfortunately, performance information is not available on this section.

The problems associated with constructing open-graded, unstabilized layers, and the required construction techniques are of interest if larger top size, coarser bases are used in the future as base courses. Both the Commonwealth of Kentucky and the Pennsylvania Department of Transportation have constructed unstabilized drainage layers using a No. 57 stone [16]. Construction on Kentucky State Route 55 of an unstabilized drainage layer has been described by Drake [16] and is as follows:

"The drainage blanket was rolled, compacted, or seated with a 10 ton tandem steel wheel roller and because it was unstable, it is neither practical nor possible to maintain any type of traffic over the unprotected blanket. It was very difficult to hold the stone in place and to the proper section during the placing of the bituminous concrete base overlay.

The contractor wisely selected to use a Barber Green Paver that was truck mounted which gave excellent traction. It was essential to operate a roller out ahead of the paver to shape the stone and improve the stability by aggregate interlock in order to allow the bituminous concrete delivery trucks to back into and unload the paver. By utilizing crushed limestone, it was practical to construct the drainage blanket with conventional hauling and paving equipment. It was necessary that the trucks use much caution when operating on the blanket. It was necessary to allow the first course of bituminous concrete to cure out adequately in order to gain strength for a period of from three to seven days, depending upon the temperature". And "By allowing the bituminous concrete to cure out properly, the second and third courses were constructed in a most conventional manner".

From the above description, the conclusion is reached that considerable caution is required in using open-graded aggregate similar to the No. 57 stone used in Kentucky. As expected, the surface of such materials is quite unstable and readily moves around and shoves. Use of a larger top size aggregate and choke stone as has traditionally been done in macadam construction would certainly help to stabilize the surface.

Subgrade Support Effects

General

Field results indicate poor subgrades and water have a very significant detrimental effect on pavement performance [1,13,18]. A comprehensive study conducted in Maryland [13] indicates pavements constructed on poor subgrades have a wide variation in performance, but in general perform relatively poorly. These poor subgrade soils usually have CBR values less than 8 to 12 and AASHTO Classifications of A-2-6, A-2-7, A-4, A-5, A-6, A-7, and A-7-6. Further, dense graded aggregate bases constructed on poor subgrades perform poorly, but water bound macadam bases perform quite well. Use of asphalt concrete surfacings greater than 4 in. (102 mm) thick above dense graded aggregate bases result in no significant improvement in performance when placed over poor subgrades [13].

Even under optimum field conditions a pavement constructed on a very good subgrade probably would not have the long life predicted by the Interim Guide [2] using high soil support values. In applying the Interim Guide to excellent subgrade conditions, possibly an effective limiting subgrade soil support value should be used, primarily because of the

practical limitations of the materials placed above the subgrade. For the conditions at the New Jersey experimental study [7], a limiting subgrade soil support value of approximately 6 to 7 ($18 \leq \text{CBR} \leq 32$) appears to exist [1]. These pavements were constructed in rock cuts and hence had an infinitely high subgrade strength. Some of the deterioration observed in New Jersey is attributed to hardening of the asphalt concrete surfacing; lack of drainage of water into the subgrade may also have been a factor [18]. The hypothesis of a limiting subgrade strength tends to be supported by the existence of an effective 18 in. (457 mm) limiting total thickness of deep granular sections used in Maine [6].

Piedmont Residual Soils

The resilient micaceous silty sands, sandy silts, and clayey silts of the Piedmont are well known as poor subgrades [1,3]. In Virginia the soil support values for these resilient subgrades have been found to be actually lower than indicated by CBR tests [3]. To give predicted lives similar to those observed, a soil support value approximately equal to 3 is required which corresponds to a CBR value of about 2.8. Measured CBR values varied from a minimum of 3 up to 11. These and other experimental results [1,3,13] indicate that the lower measured values of subgrade strength should probably be used for design purposes, particularly for the weak, resilient subgrades of the Piedmont Province.

Conclusions

With sufficient care pavements can be successfully constructed using relatively thin asphalt concrete surfacings as proven by the Lake Wales, Virginia and WES experimental pavements. Performance of a granular base is greatly affected by many factors including (1) quality of aggregate including gravel compared to crushed stone, (2) aggregate size, gradation, and angularity, (3) level of compaction, (4) magnitude of wheel loading, (5) aggregate segregation, and (6) level of field inspection. As a result of these and other factors, the variation of performance of pavements having unstabilized granular bases and/or subbases in general is greater than for asphalt concrete bases.

Full-scale test pavement results indicate rutting in pavements constructed with granular bases should, if good practices are followed, be no greater than full-depth asphalt concrete sections. Under favorable conditions such as the warm climate of the southeast, rutting in granular base sections could even be less than in full-depth sections. Additional care is, however, required in constructing unstabilized granular bases including using the proper gradation, achieving a high level of density and minimizing aggregate segregation during construction.

Base course equivalencies and base course coefficients are often misused being applied to conditions for which they are not valid. These equivalency factors must therefore be associated with two specific base materials and specific conditions of design and construction. The base course equivalency determined from well constructed test pavements is typically that 1.0 in. (25 mm) of black base is equivalent to 1.6 to 2.5 in. (41 - 64 mm) of granular base. Under unfavorable conditions such as existed in Colorado the equivalency can be greater; unfavorable conditions at the

Colorado site included low density, use of an uncrushed gravel base/subbase and light loading.

Inverted pavement sections consist of an unstabilized granular base sandwiched between an asphalt concrete surfacing and cement stabilized granular subbase or cement stabilized subgrade. Optimum use of the crushed stone is thus made in an inverted section since it is both confined and primarily subjected to compression. Inverted sections have been successfully used in New Mexico, Virginia and at the Waterways Experiment Station. Inverted construction appears to offer promise particularly for construction of pavements over weak subgrades. Inverted construction certainly deserves further study in Georgia, particularly for use over the micaceous silty sand subgrades of the Piedmont Province.

Three experimental pavement studies in Virginia show that a relatively high level of cement stabilization (10% by volume) of the subgrade improves performance of pavements constructed on micaceous Piedmont soils. Use of a select borrow of local material (CBR=33) placed over a cement stabilized micaceous Piedmont subgrade is not beneficial to pavement performance.

Macadam bases having a coarse gradation and large top size aggregate have been found to perform better than conventional graded aggregate bases on weak subgrades. Dry bound macadam bases are presently being constructed in Iowa for low volume county roads and previously were constructed in the northeastern states.

Pavements constructed over poor subgrades such as the resilient micaceous silty sands of the Piedmont Province often perform below expectations. For design probably the lower limit of the observed CBR values should be used in the Interim Guide design method. In many instances the pavement may even perform as if the soil is weaker than the lowest observed CBR value. In rock cuts an upper limiting value of subgrade support appears to exist associated with a limiting CBR value of about 18 to 32.

CHAPTER III MATERIAL PROPERTIES

Introduction

The material properties of the components of a pavement structure have a significant influence on performance. Therefore, to document the properties of the materials used in the twelve test sections extensive standard and dynamic tests were performed as a part of this study. Dynamic tests included fatigue, rutting and bending modulus of the asphalt concrete, and rutting and resilient modulus tests of the subgrade and crushed stone materials.

Asphalt Concrete

The asphalt concrete used in all sections was either a modified-B or B-binder for the full-depth of asphalt. These mixes were obtained from either the Lithia Springs or Forest Park plant of APAC-Georgia, Inc. An AC-20 viscosity grade asphalt cement was used in the mix obtained from the Shell Oil Company (Trumbull Products, Atlanta, Ga.). The typical physical properties of the asphalt cement are given in Table 5.

Physical Properties

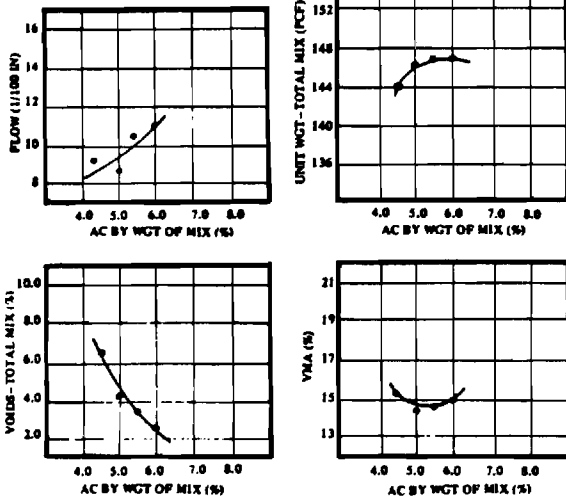
Fifty blow Marshall mix designs for the asphalt concrete mixes used in this study were prepared by the Georgia Department of Transportation, Office of Materials and Research. A summary of the asphalt concrete mixes used in this study are given in Figs. 10 through 13. The type mix and test sections for which the mix applied is indicated on each figure.

The properties of the asphalt concrete placed in each section are summarized in Table 6 including gradation, asphalt content, unit weight and air voids. The air voids were approximately determined using an average unit weight and representative theoretical maximum specific gravity. These physical properties were determined by the Office of Materials and Research using 4 in. (100 mm) dia. cores taken from the pavement after failing the section. Before testing was begun on each section, the density of the asphalt concrete was generally determined by a Georgia DOT technician using a nuclear gage; these densities as a percent of the Marshall design density are also given in Table 6.

TABLE 5. ASPHALT CEMENT PROPERTIES.

Property	Sections					
	1,2	3,4	5,6	7,8	9,10	11,12
Manufacturer	Shell	Fulco	Shell	Shell	Shell	Shell
Viscosity at 140°F (poises)	1875	1942	1867	1838	1917	1985
Viscosity at 275°F (poises)	368	390	405	379	387	393
Penetration (77°F, 100g, 5s)	63	61	70	65	68	63
Flash Point (Cleveland open cup) (°F)	587	600	615	615	550	590
Specific Gravity @ 60°F	1.022	1.020	1.023	1.024	1.031	1.037
Tests on Thin Film Residue						
a) Viscosity @ 140°F, 30 cm Hg. (poises)	4866	5028	5516	4925	5985	5492
b) Ductility @ 77°F, 5 cm/min. (cm)	135+	150	150	150	150	150
c) Ductility @ 60°F, 5 cm/min. (cm)	8.1	9.9	11.0	10.3	11.0	13.0

Stone: Granite Gneiss
 Source: Vulcan Materials Co., Lithia Springs, Ga.
 OPTIMUM % AC: 4.9
 STABILITY (LBS.) 2,500
 FLOW (1/100 IN): 9.6
 % VOIDS-TOTAL MIX: 4.5
 % VMA 14.5
 UNIT WGT-TOTAL MIX: 146.0



50 Blow Marshall Mix Design
 Type of Mix: B-Modified
 AGGREGATE GRADATION

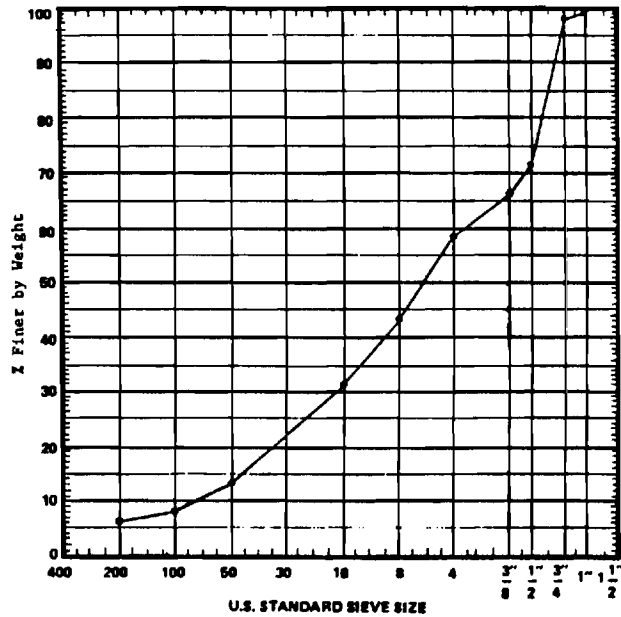
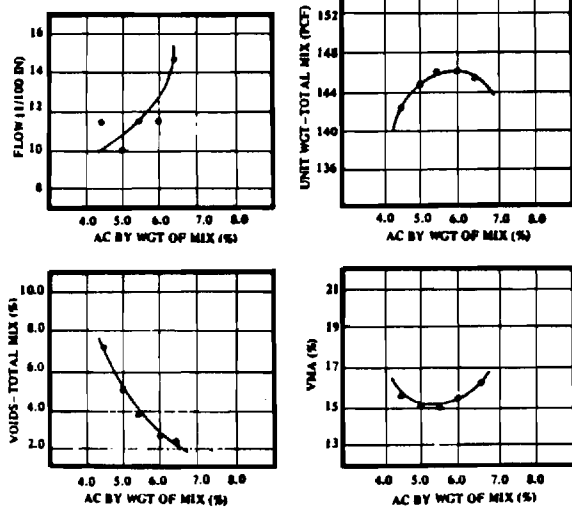


FIGURE 10. ASPHALT CONCRETE B-MODIFIED MIX DESIGN APPLICABLE TO SECTIONS 1 THROUGH 4.

Stone: Granite Gneiss
 Source: Vulcan Materials Co., Lithia Springs, Ga.
 OPTIMUM % AC: 5.2
 STABILITY (LBS.) 2,150
 FLOW (1/100 IN): 10.6
 % VOIDS-TOTAL MIX: 4.5
 % VMA 15.0
 UNIT WGT-TOTAL MIX: 145.5



50 Blow Marshall Mix Design
 Type of Mix: B-Modified
 GaDOT 7/25/78 Mix Design
 Used in Sections 1 through 8

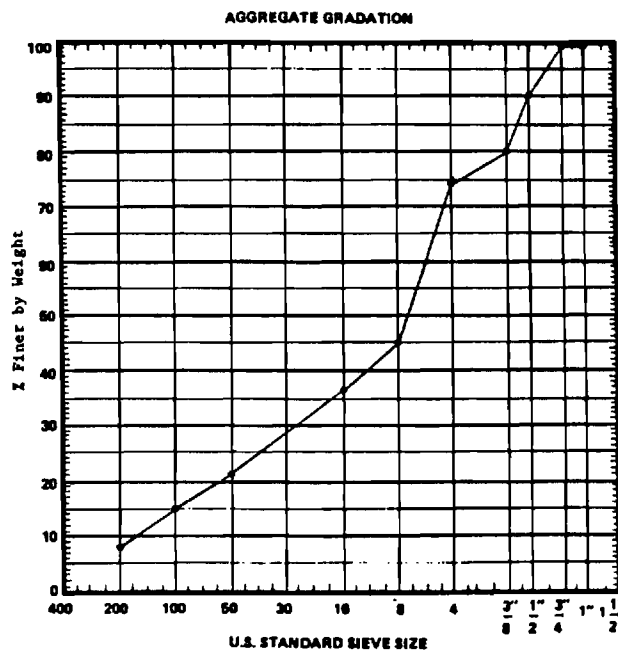


FIGURE 11. ASPHALT CONCRETE B-MODIFIED MIX DESIGN APPLICABLE TO SECTIONS 5 THROUGH 8.

50 Blow Marshall Mix Design
 Type of Mix: B-Mix
 GaDOT 11/19/74 Mix Design
 Used in Sections 9 and 10.

Agg. Type: Dixie Lime and Stone
 Source: Mountain View, Ga.

OPTIMUM % AC: 5.0
 STABILITY (LBS.) 2,950
 FLOW (1/100 IN): 10.1
 % VOIDS-TOTAL MIX: 5.0
 % VMA 14.9
 UNIT WGT-TOTAL MIX: 148.0

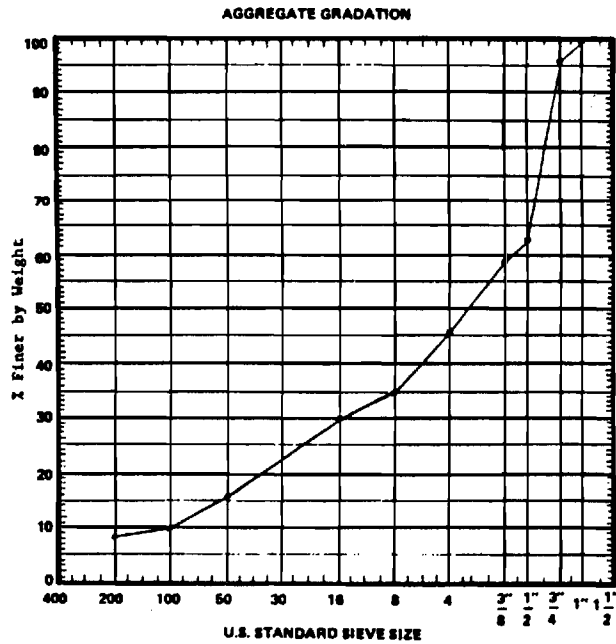
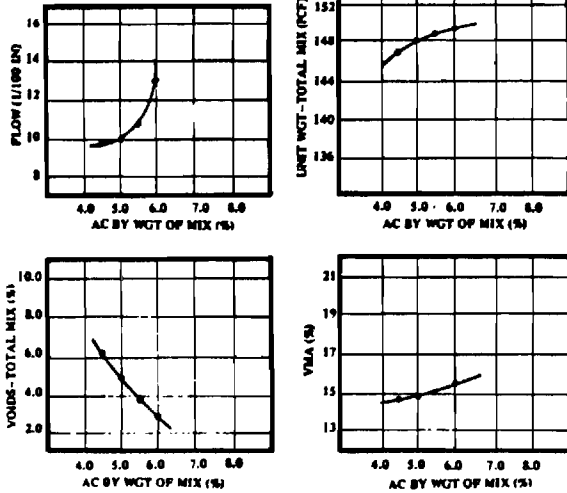
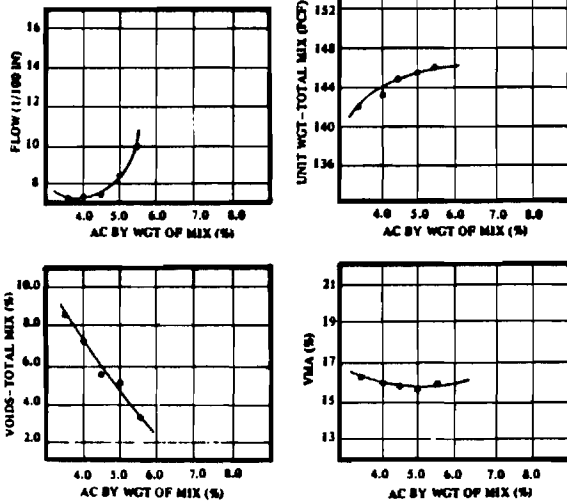


FIGURE 12. ASPHALT CONCRETE B-MIX DESIGN APPLICABLE TO SECTIONS 9 AND 10.

Stone: Granite Gneiss
 Source: Vulcan Materials Co.
 Lithia Springs, Ga.

OPTIMUM % AC: 5.2
 STABILITY (LBS.) 2,300
 FLOW (1/100 IN): 9.0
 % VOIDS-TOTAL MIX: 4.0
 % VMA 16.0
 UNIT WGT-TOTAL MIX:



50 Blow Marshall Mix Design
 Type of Mix: B-Mix
 GaDOT 7/1/81

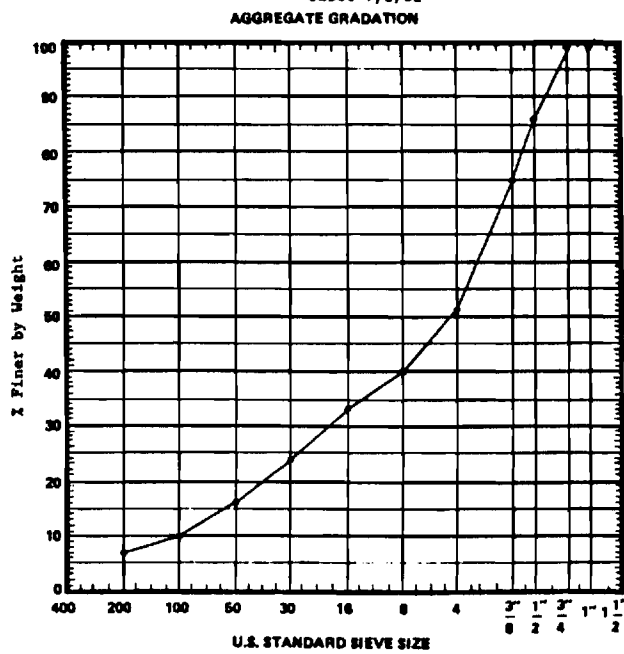


FIGURE 13. ASPHALT CONCRETE B-MIX DESIGN APPLICABLE TO SECTIONS 11 AND 12.

TABLE 6. NUCLEAR DENSITIES AT TIME OF CONSTRUCTION AND EXTRACTION DATA FOR CORES TAKEN SUBSEQUENT TO SECTION FAILURE.

Section	Cumulative % Passing (by wt.)								Asphalt Content (%)	Unit Weight (pcf)	Air Voids (%)	Number of Cores	Construction Density (Nuclear) % 50 Blow Marshall
	Sieves												
	1"	3/4"	1/2"	3/8"	4	8	50	200					
1 & 2	100	99.5		72		42		4	4.61	143.7	5.8	8	
3 & 4	97	97		71		40	21	8	5.9	145.1	3.8 ⁽¹⁾	1	91.3
5		100	89	73	53	41	16	6	5.71	145.8	3.6 ⁽¹⁾	4	97.2
6		100	86	70	49	38	15	6	5.17	144.8	5.0 ⁽¹⁾	6	96.1
7	100	99.7	96	85	60	46	17	6	5.60	145.9	3.7 ⁽¹⁾	8	
8	100	99.4	82	70	51	41	18	8	4.23	144.2	6.7 ⁽¹⁾	6	
9 & 10		100	76	62	48	38	22	7	4.9	148.0	5.2 ⁽²⁾	4 7	98.6
11 & 12													99.2

(1), (2): Based on theoretical S.G.
(Taken from Ga. DOT Design Analysis Sheets).

Fatigue Properties

Sample preparation and testing procedures of the fatigue and rutting tests performed on asphalt concrete specimens have been described in detail elsewhere [20,22]. The fatigue tests were conducted on 3 in. by 3 in. by 20 in. long (76 x 76 x 508 mm) asphalt concrete beams. The asphalt concrete beams were placed on a rubber subgrade having a modulus of subgrade reaction of 116 pci (31.5 MN/m³). A 0.15 sec. duration, 1 Hz, repeated load was applied at the center of the beam until failure. All tests were conducted in an environmental chamber at a temperature of 80°F ± 1°F (26.7°C ± 0.6°C).

A series of fatigue tests was performed on a laboratory prepared B-modified asphalt concrete mix at 4.3, 4.8 and 5.3% asphalt content. The beams were prepared at 100% of the 50 blow Marshall density. Bending modulus of the beam as a function of repetitions to failure and asphalt content is given in Fig. 14 and also Table 7. The relationship between number of repetitions to failure and applied load is given in Fig. 15. Fig. 16 gives the relationship between number of repetitions to failure and tensile strain for the B-modified mix.

Rutting Properties

The rutting properties of the asphalt concrete were evaluated by subjecting 4 in. (102 mm) dia. by 8 in. (203 mm) high, cylindrical asphalt concrete specimens to 100,000 repetitions of loading. For comparison with previous work [20] the tests were conducted at 95°F (35°C). The tests were conducted in an environmental chamber using a confining pressure of 5 psi (34 kN/m²) and a deviator stress of 25 psi (172 kN/m²) [20].

Results of rutting tests performed on specimens prepared from the asphalt concrete used in selected test sections are given in Table 8. cylinders were prepared from the actual concrete mix used in selected lifts of the test sections. The cylinders were made at 100% of the 50 blow Marshall density by reheating the asphalt concrete several days after the construction of the test section was completed.

Crushed Stone Base

Physical Properties

The crushed stone used for all unstabilized bases was a granitic gneiss obtained from the Norcross Quarry of Vulcan Materials Co. The physical properties of the granitic gneiss are summarized in Table 9. To minimize segregation three sizes of crushed stone (No. 5, No. 57 and No. 810) were stockpiled and blended together during construction of each crushed stone base. A detailed description of base construction is given in Chapter IV. The gradation of each of the three crushed stone bases is given in Table 10. The "standard" crushed stone base material was prepared by blending together in a pugmill 20% of the No. 5 size stone, 25% of No. 57 stone and 55% of No. 810 stone. The standard gradation stone was used in Test Sections 1, 2, 8, 11 and 12.

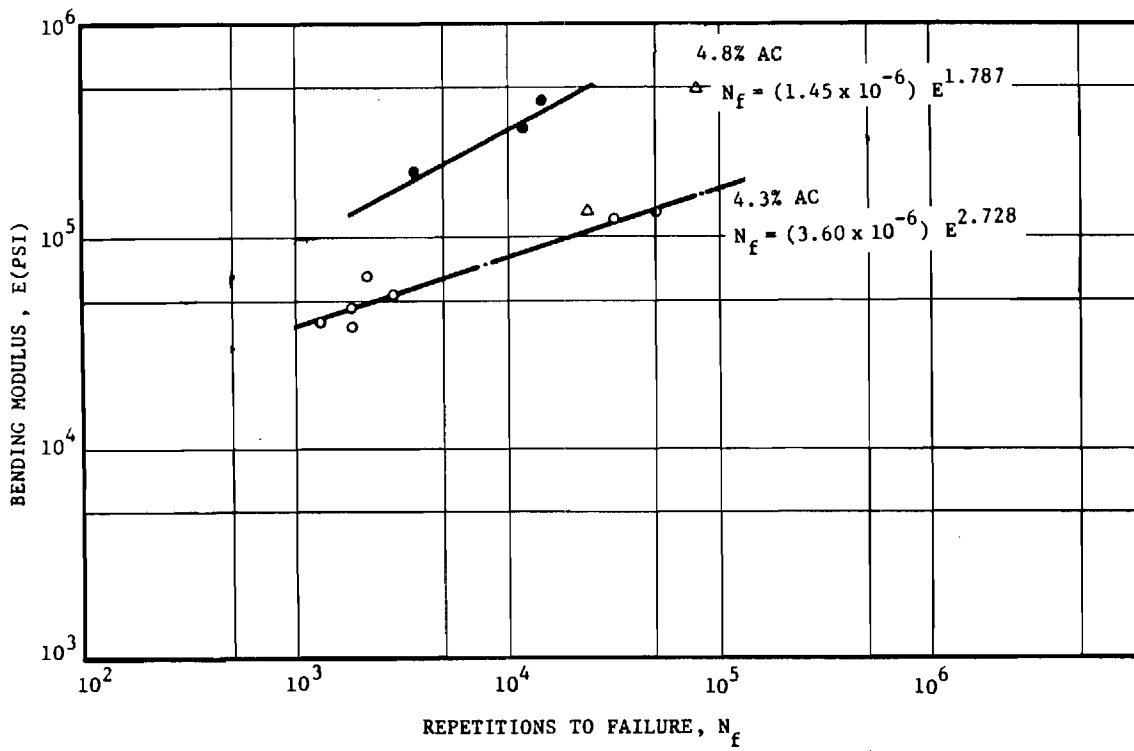


FIGURE 14. B-MODIFIED MIX: REPETITIONS TO FAILURE AS A FUNCTION OF BEAM BENDING MODULUS AND ASPHALT CONTENT .

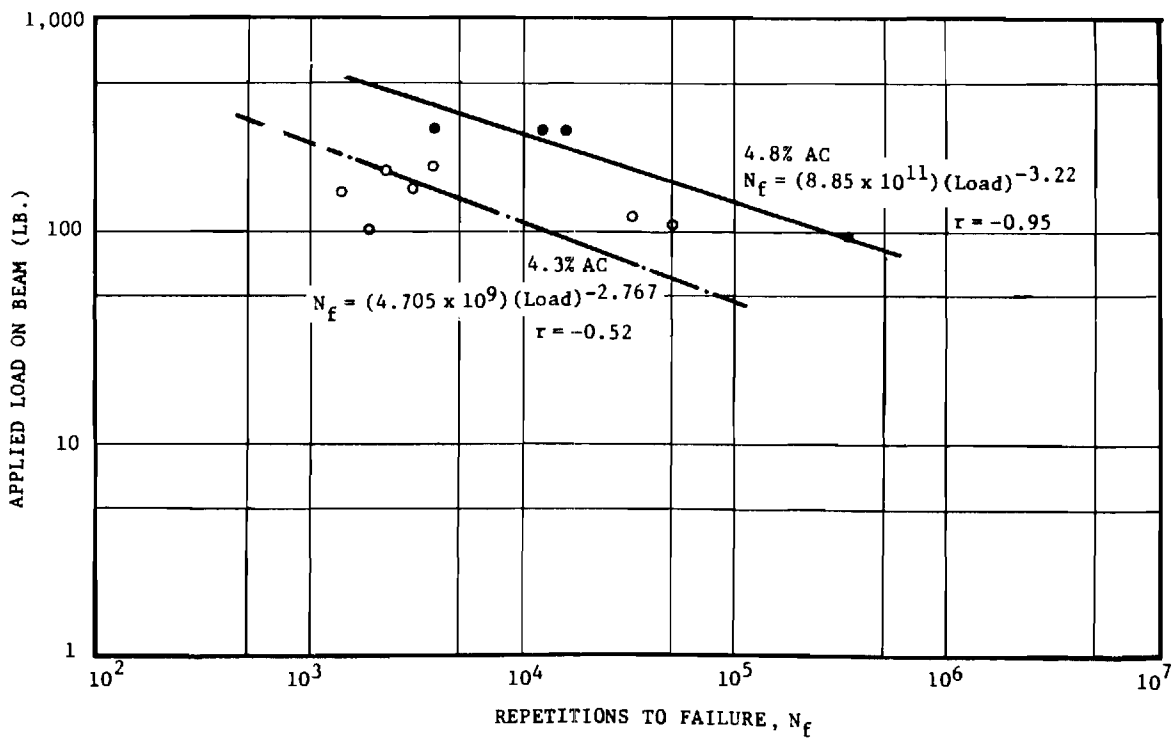


FIGURE 15. B-MODIFIED MIX: REPETITIONS TO FAILURE AS A FUNCTION OF APPLIED LOAD AND ASPHALT CONTENT.

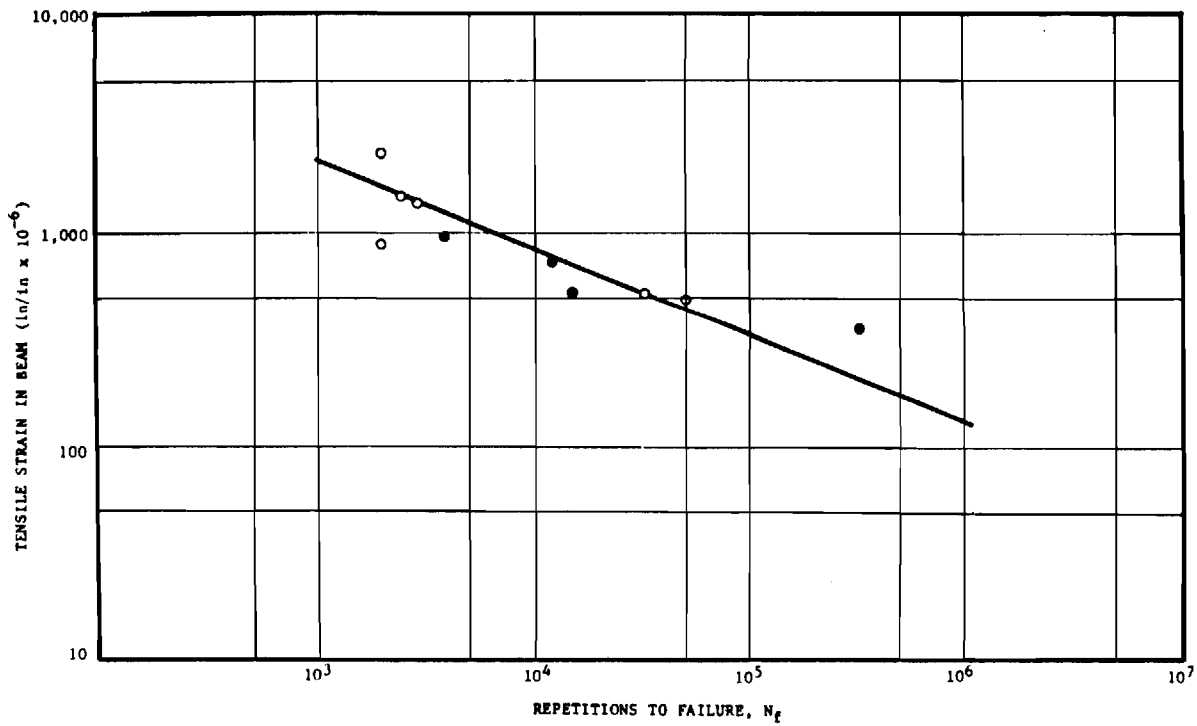


FIGURE 16. FATIGUE LIFE OF BEAM AS A FUNCTION OF TENSILE STRAIN - B-MODIFIED MIX.

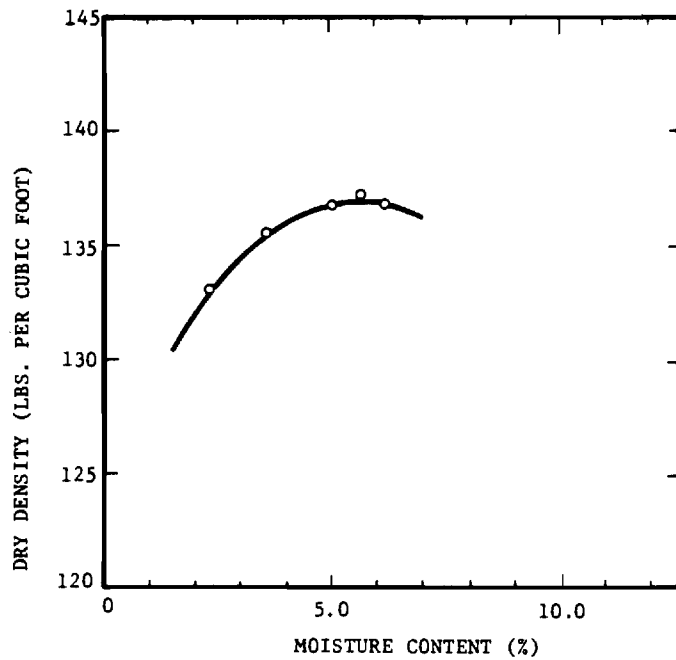


FIGURE 17. MOISTURE-DENSITY CURVE FOR STANDARD BASE COURSE (T-180 COMPACTION).

TABLE 7. BEAM FATIGUE TESTS ON LABORATORY-PREPARED B-MODIFIED MATERIAL.

Specimen No.	Asphalt Content (%)	Load (lb)	Tensile Strain (μ in/in)	Modulus of Elasticity (psi)	Reps. to Failure
MB2	4.8	100	300	208,407	323,000
MB3	4.8	300	500	457,544	16,000
MB4	4.8	300	680	303,656	13,000
MB7	4.8	300	884	214,021	4,000
MB9	4.3	150	1550	40,170	1,450
MB 10	4.3	100	867	50,068	2,000
MB 11	4.3	160	1280	56,508	3,220
MB 12	4.3	110	464	132,657	53,800
MB 13	4.3	200	2110	39,075	2,070
MB 14	4.3	120	511	130,992	33,940
MB 15	4.3	198	1340	70,628	2,400
MB 16	5.3	100	308	508,398	85,000
MB 17	5.3	200	815	138,908	26,744

TABLE 8. RESULTS OF RUTTING TESTS ON ASPHALT CONCRETE USED IN SECTIONS 7 AND 8 -
TEST AT 95°F (35°C).

Sample No.	Deviator Stress (psi)	Confining Pressure (psi)	Resilient Strain ($\mu\epsilon$)						Permanent Strain ($\mu\epsilon$)					
			Repetitions						Repetitions					
			1	10	100	1,000	10,000	100,000	1	10	100	1,000	10,000	100,000
4	25	5	870	750	675	600	500	625	5,600*	5,700*	6,000*	6,300*	7,500*	8,300*
5	25	5	500	575	500	375	300	150	250	275	950	1,250	2,200	2,250
7	25	5	425	425	375	325	275	275	750	775	925	1,000	1,250	1,250

* Very high values possibly due to an initial seating deflection rather than load.

TABLE 9. TYPICAL PROPERTIES OF NORCROSS GRANITE-GNEISS CRUSHED STONE.

Aggregate Description	Granitic Gneiss
Georgia DOT Class Aggregate	B
Bulk Specific Gravity	2.73
Bulk Specific Gravity (SSD)	2.74
Apparent Specific Gravity	2.77
Sand Equivalent	85
Absorption (%)	0.61
L. A. Abrasion(%)	50
Percent Sand Loss	0.6
Mag. Sulfate Soundness Loss (%)	1.0

TABLE 10. GRADATIONS OF NORCROSS CRUSHED STONE AGGREGATES.

Sieve	Cumulative % Passing, By Weight										
	1-1/2"	1"	3/4"	1/2"	3/8"	4	8	10	30	60	200
No. 5	100	96	37	5	2						
No. 57	100	98	82	43	20	3	2				
No. 810					100	77	59	56	35	19	8

Test Section 10 used a fine gradation base course approximating the Villa Rica gradation. This gradation was obtained by using 45% of the No. 57 stone and 55% of No. 810 stone. Section 9 had a coarse gradation base that approximately followed the 2 in. (51 mm) Fuller gradation curve. The coarse gradation base was obtained by blending 20% of the No. 5 stone, 12% of the No. 57 stone and 52% of the No. 810 stone. In addition the following non-standard stone sizes were required to obtain the coarse gradation: (1) 2% of stone passing the 2 in. (51 mm) sieve and retained on the 1.5 in. (38 mm) sieve, and (2) 14% of stone passing the 1.5 in. (38 mm) sieve and retained on the 1 in. (25 mm) sieve. The crushed stone gradations used are given in Table 11 together with the Villa Rica and 2 in. (51 mm) Fuller gradation curves for comparison.

The AASHTO T-180 maximum dry density and corresponding optimum moisture content are summarized in Table 12 for the three gradations tested. Fig. 17 gives the moisture density curve for the standard gradation base material.

Rutting and Resilient Modulus

The repeated load triaxial test consisted of subjecting a 6 in. (152 mm) dia. by 14 in. (356 mm) high, cylindrical specimen of crushed stone to 100,000 repetitions of a deviator stress. During the test the specimen was placed inside a triaxial cell and subjected to a constant confining pressure. The test method and specimen preparation for this test have been described in detail elsewhere [19].

The resilient moduli of the three gradations of crushed stone tested are shown in Figs. 18a and 18b as a function of bulk stress σ_θ ($\sigma_\theta = \sigma_1 + 2\sigma_3$) for the standard, coarse and fine gradations, respectively. For the gradations tested relatively little difference existed between the resilient modulus for the three gradations. The earlier work of Barksdale [19] is in agreement with this finding.

The plastic strain response of the crushed stone having the standard gradation is shown in Figs. 19 through 22. The plastic strain response of the coarse gradation base material is given in Fig. 23.

Subgrade

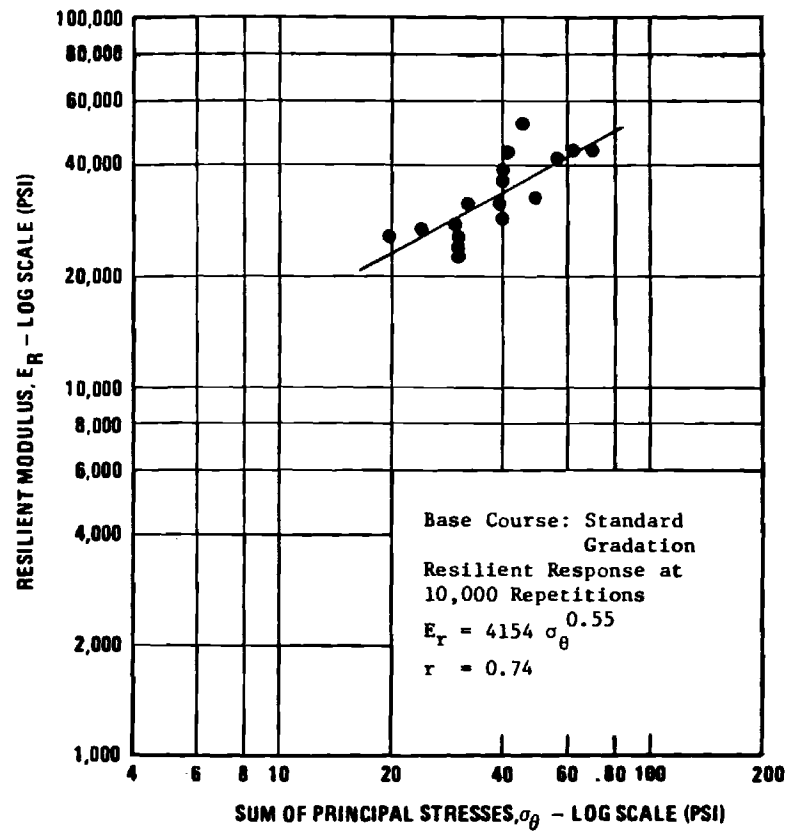
The subgrade used beneath all pavement sections was a micaceous silty sand. This soil was obtained from I-575 between Stations 208+00 and 220+00, near North Booth Road, Cobb County, Georgia. The basic physical properties of the silty sand averaged for seven tests are given in Table 13. The silty sand subgrade classified as a III-B embankment material following the Georgia DOT Classification System. The average volume change was 31.3% and the clay content was 20%. The average maximum dry density from seven tests was 105 pcf (16.5 kN/m³) and optimum water content was 18.5%.

TABLE 11. CRUSHED STONE BASE COURSE GRADATIONS.

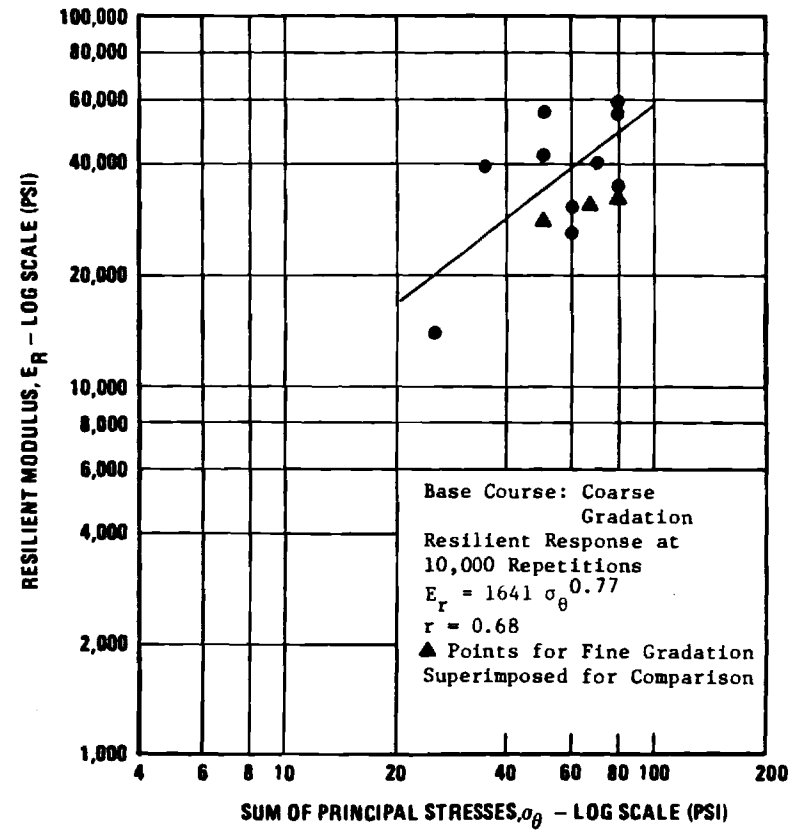
Sieve	Cumulative % Passing, By Weight										
	1-1/2"	1"	3/4"	3/8"	4	8	10	50	60	100	200
Standard	100	98	83	61	43		31	13			4
"Fine"	100	99	92	64	44		31		11		4
"Coarse"	98	83	69		40	31	29		10		4
Villa Rica	100		92				30		12		5
2" Fuller	98	82	69		39	31	29		12		4

TABLE 12. MAXIMUM DRY DENSITIES AND OPTIMUM MOISTURE CONTENTS OF BASE COURSE [AASHTO T-180].

Material Gradation	Maximum Dry Density		Optimum Moisture Content (%)
	(pcf)	(kg/m ³)	
Standard	137.0	2,217	5.7
Fine	139.5	2,257	6.8
Coarse	141.5	2,289	6.8



(a) Standard Gradation



(b) Coarse and Fine Gradation

FIGURE 18. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS - CRUSHED STONE BASE MATERIAL.

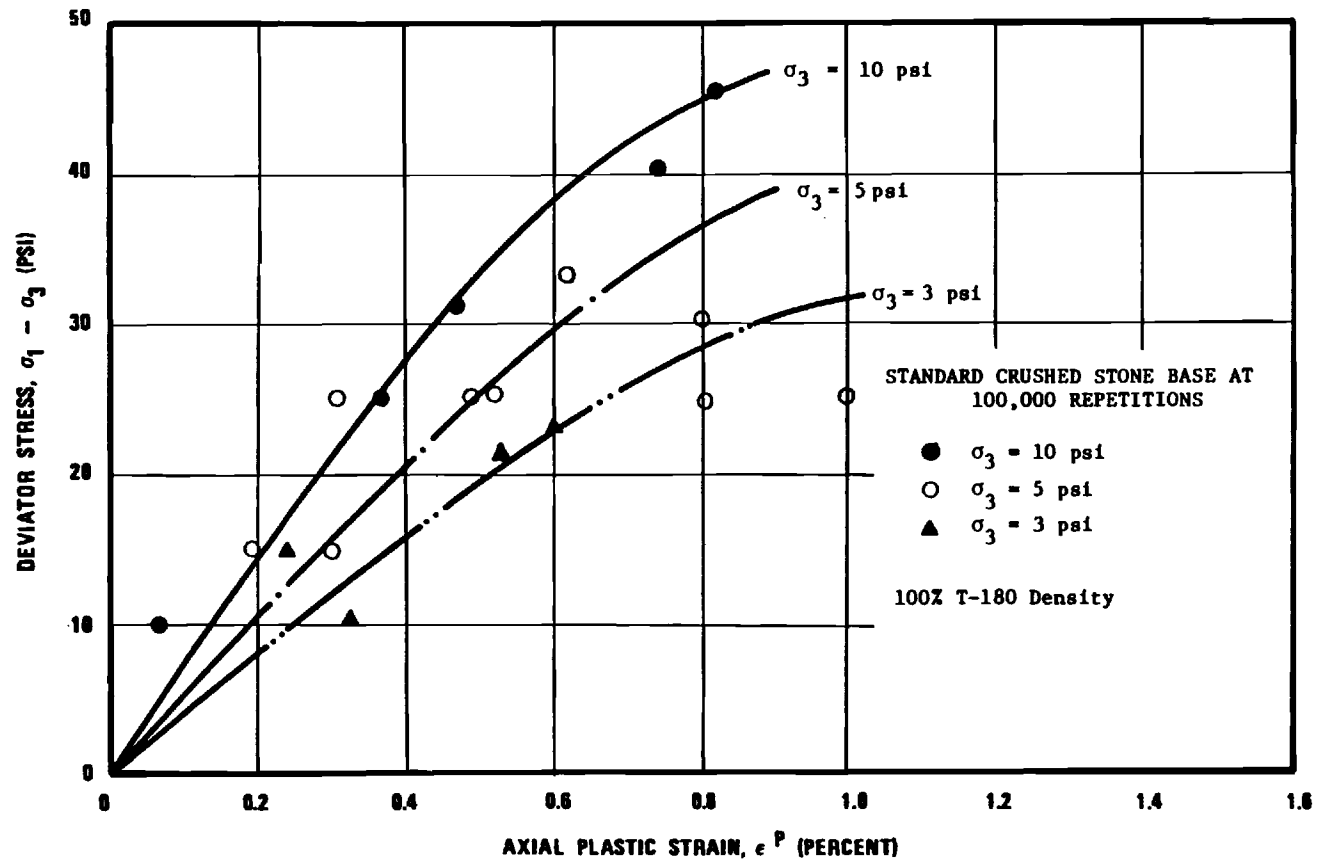


FIGURE 19. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION CRUSHED STONE BASE

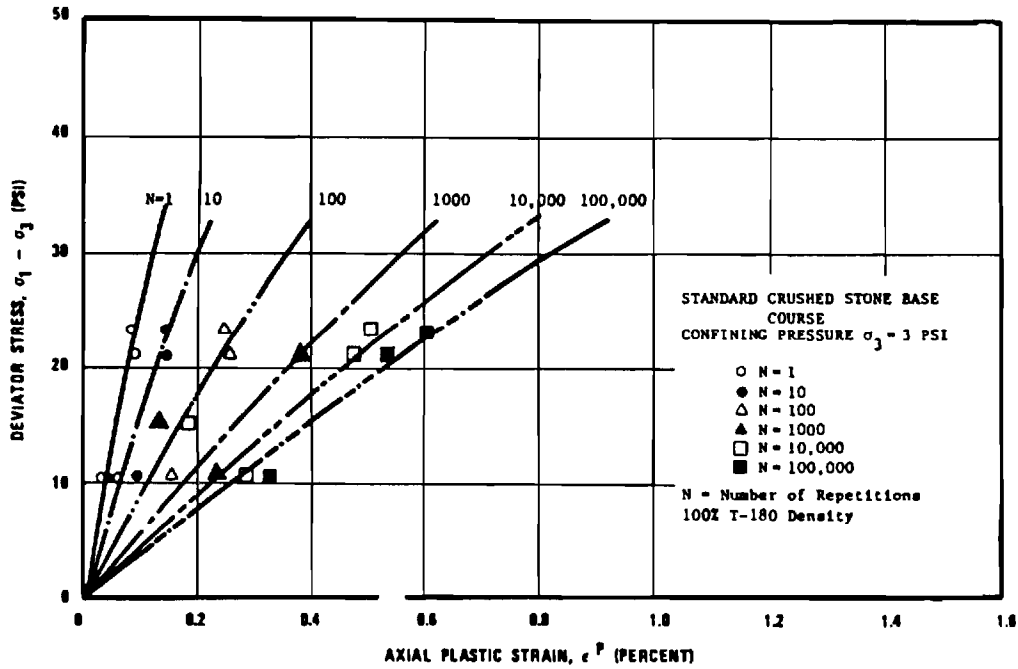


FIGURE 20. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION BASE COURSE.

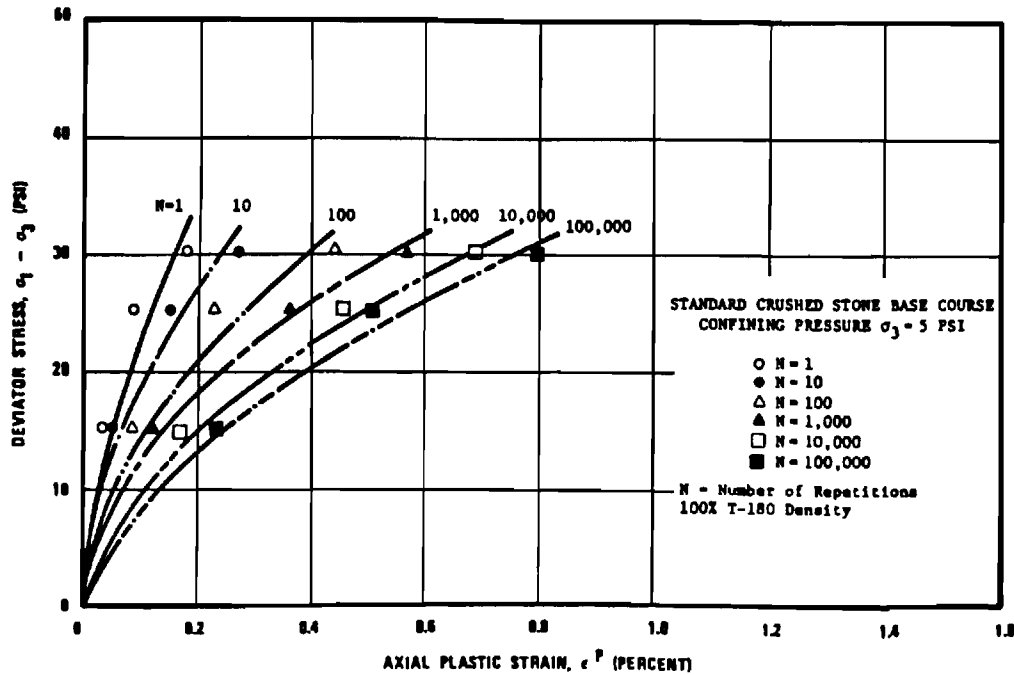


FIGURE 21. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON STANDARD GRADATION CRUSHED STONE BASE

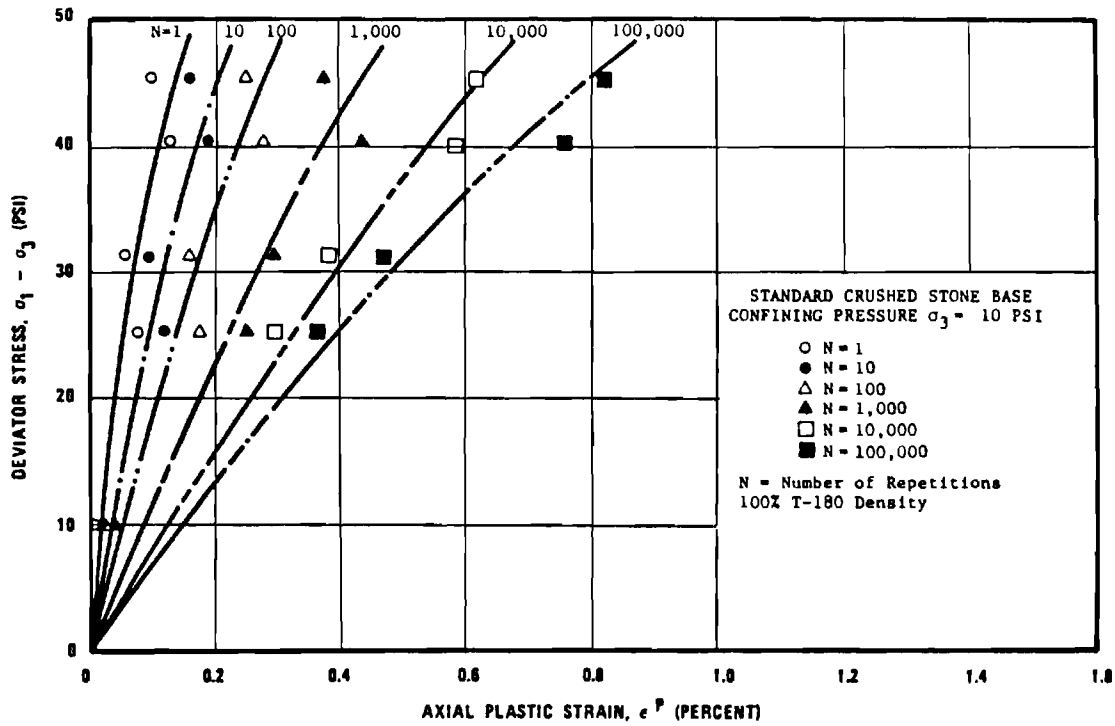


FIGURE 22. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN STANDARD GRADATION CRUSHED STONE BASE

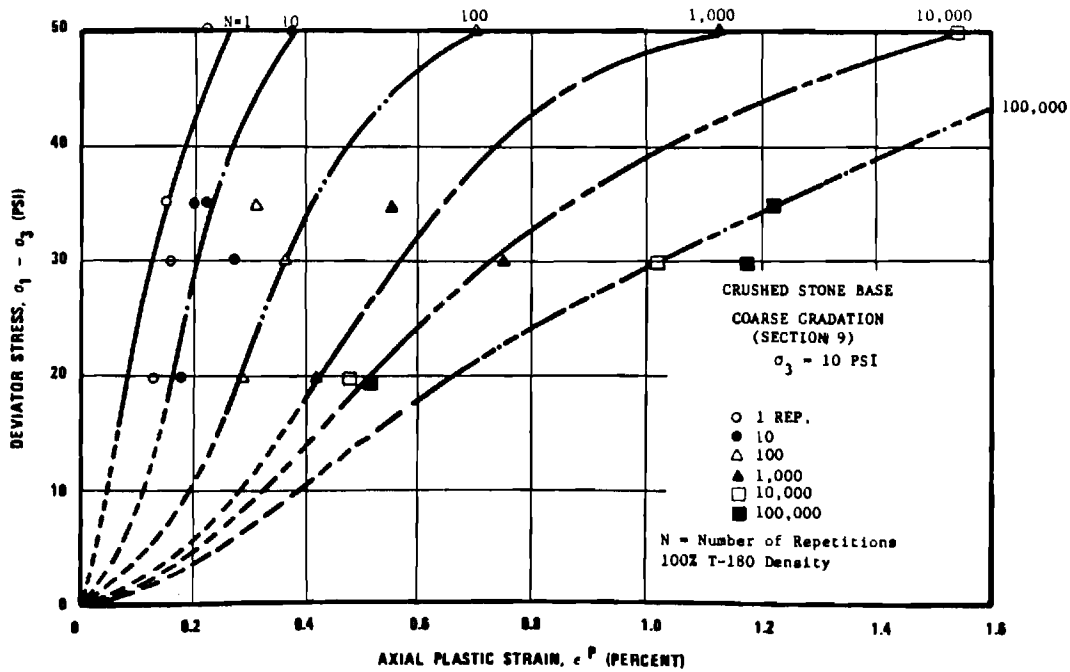


FIGURE 23. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON PLASTIC STRAIN IN COARSE GRADATION CRUSHED STONE BASE .

TABLE 13. MICACEOUS SILTY SAND SUBGRADE PROPERTIES.

Sieve Size	Cumulative % Passing, By Weight
1-1/2"	100
# 10	99
# 40	85
# 60	70
# 200	39
Total Clay: 20% Liquid Limit: S.I.C. Plasticity Index: N.P.	
Volume Change: Swell 30.3% Shrinkage 1.0% Total 31.3	
T-99 Proctor Test Results (1) Maximum Dry Density: 105 pcf Optimum Moisture Content: 18.5%	

1. Average of seven tests of soil taken along the embankment route; one test on the soil used in the pit indicated an optimum moisture content of 20.5% and a maximum dry density of 108 pcf.

Dynamic Response

The dynamic properties of the remoulded subgrade soil were evaluated using the repeated load triaxial test. A detailed description of sample preparation and testing is given elsewhere [21]. Repeated load triaxial tests were performed on the silty sand subgrade for densities varying from about 93 to 95% of AASHTO T-99, and at moisture contents from about 1% below to about 2% above optimum.

Typical test results showing the resilient modulus as a function of deviator stress, moisture content and confining pressure are given in Figs. 24 through 26. Generally the resilient modulus was between 700 and 1100 psi (4820 - 7580 kN/m²) which indicates a quite resilient subgrade. The resilient modulus was found to be essentially independent of moisture content for moisture contents from about 17.4 to 20.3%, and only slightly effected by deviator stress for values greater than about 2 psi (13.8 kN/m²).

The plastic strain response of the silty sand is given in Figs. 27 through 29. Approximately 80% of the plastic strain occurred after only 100 load repetitions. Further, an increase in moisture content from optimum to 2% above optimum caused a significant increase in permanent deformation (Fig. 28).

Cement Stabilized Crushed Stone Subbase

Sections 11 and 12 were inverted sections having a cement stabilized subbase and crushed stone base (Table 1). Section 11 was constructed with a soil-cement subbase. To construct this layer 5% of Type I Portland cement was added to the silty sand used as a subgrade beneath all the sections. The design for the soil cement subbase by the Office of Materials and Research gave a maximum dry density of 107 pcf (16.8 kN/m³) and an optimum moisture content of 18%. Standard Proctor specimens prepared during construction of the subbase had an average unconfined compressive strength of 214 psi (1474 kN/m²). The seven day unconfined compressive strength was 71% of the 28 day value. The unconfined stress-strain curves for the soil cement subbase are given in Fig. 30.

The cement stabilized subbase used in Section 12 was constructed by adding 4.5% of Type I Portland cement to the standard gradation crushed stone base material. The cement stabilized stone was placed at a water content of 6% at a density of about 138 pcf (21.7 kN/m³). The 28 day average unconfined compressive strength of 6 in. (152 mm) dia. by 12 in. (305 mm) high cylindrical specimens stored in a moisture room was 1146 psi (7896 kN/m²); at 7 days the unconfined compressive strength was 63% of the 28 day value. The unconfined stress-strain curves are given in Fig. 31 for the cement stabilized crushed stone subbase material.

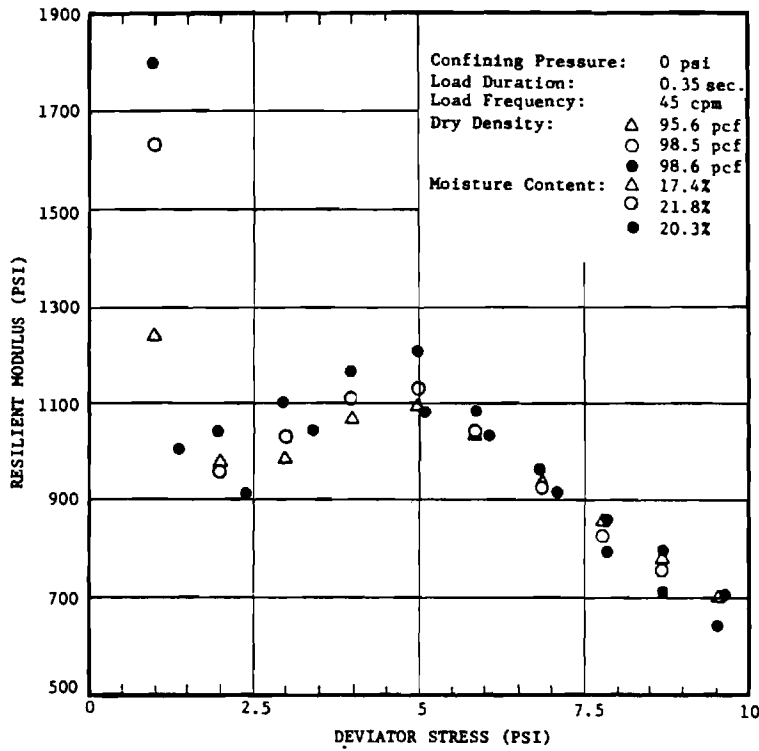


FIGURE 24. INFLUENCE OF DEVIATOR STRESS AND MOISTURE CONTENT ON RESILIENT MODULUS OF MICACEOUS SILTY SAND (CONFINING PRESSURE = 0 PSI) .

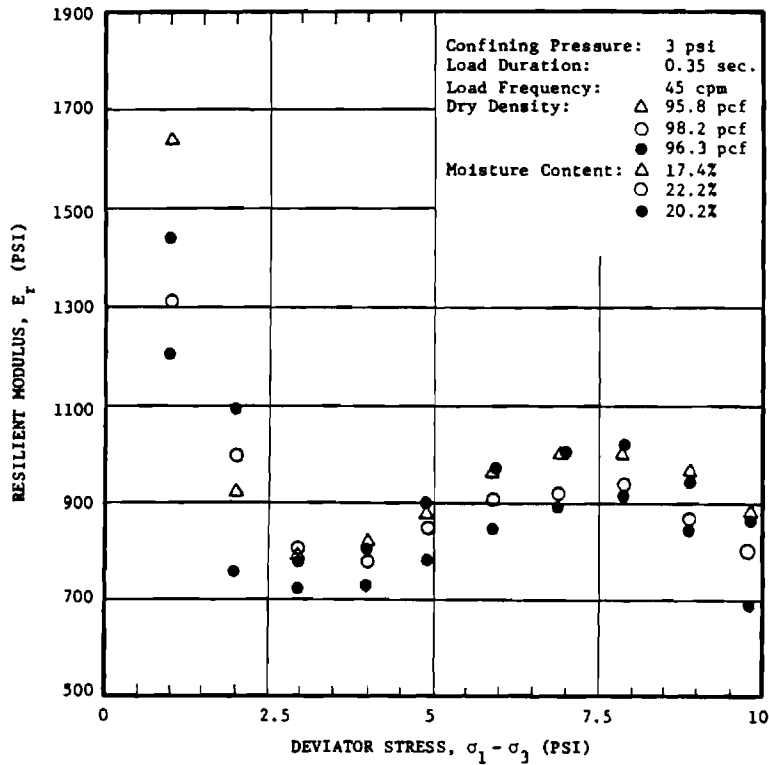


FIGURE 25. INFLUENCE OF DEVIATOR STRESS AND MOISTURE CONTENT ON RESILIENT MODULUS OF MICACEOUS SILTY SAND (CONFINING PRESSURE = 3 PSI)

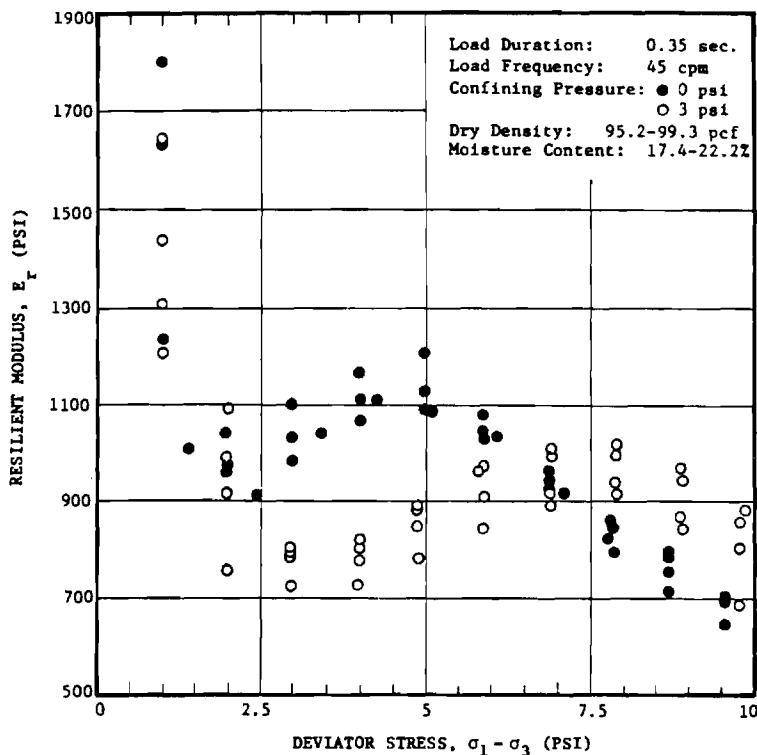


FIGURE 26. INFLUENCE OF DEVIATOR STRESS AND CONFINING PRESSURE ON RESILIENT MODULUS FOR MICACEOUS SILTY SAND - SUMMARY OF ALL RESULTS

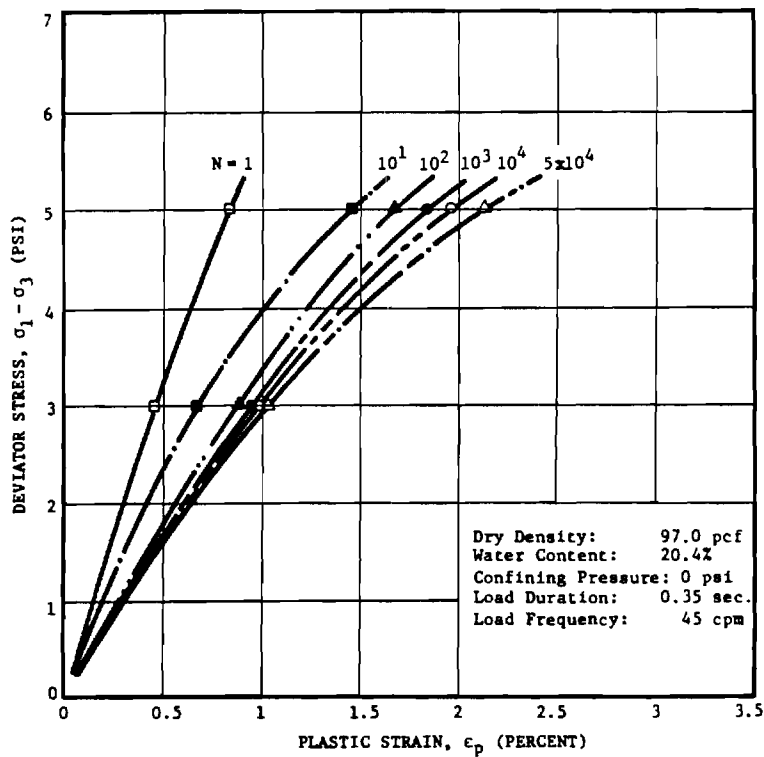


FIGURE 27. INFLUENCE OF LOAD CYCLES AND DEVIATOR STRESS ON PLASTIC STRAIN IN MICACEOUS SILTY SAND .

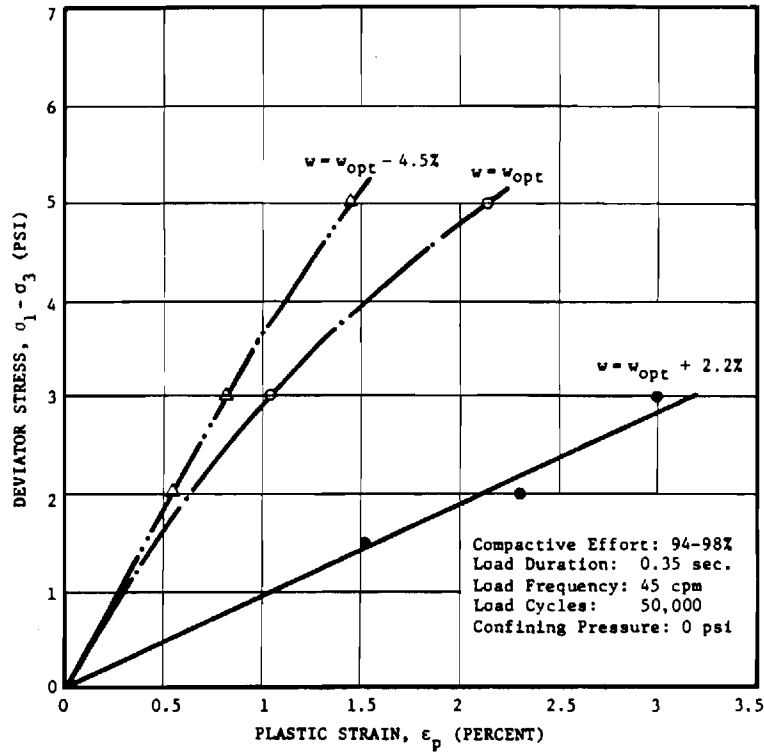


FIGURE 28. INFLUENCE OF DEVIATOR STRESS AND WATER CONTENT ON PLASTIC STRAIN FOR MICACEOUS SILTY SAND - 50,000 LOAD CYCLES.

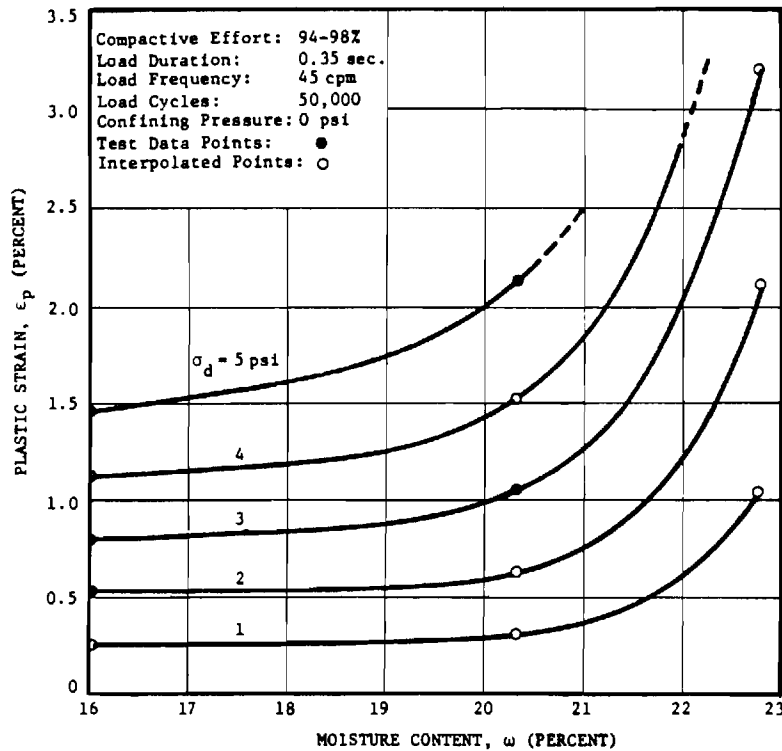


FIGURE 29. INFLUENCE OF WATER CONTENT AND DEVIATOR STRESS ON PLASTIC STRAIN FOR MICACEOUS SILTY SAND - 50,000 LOAD CYCLES.

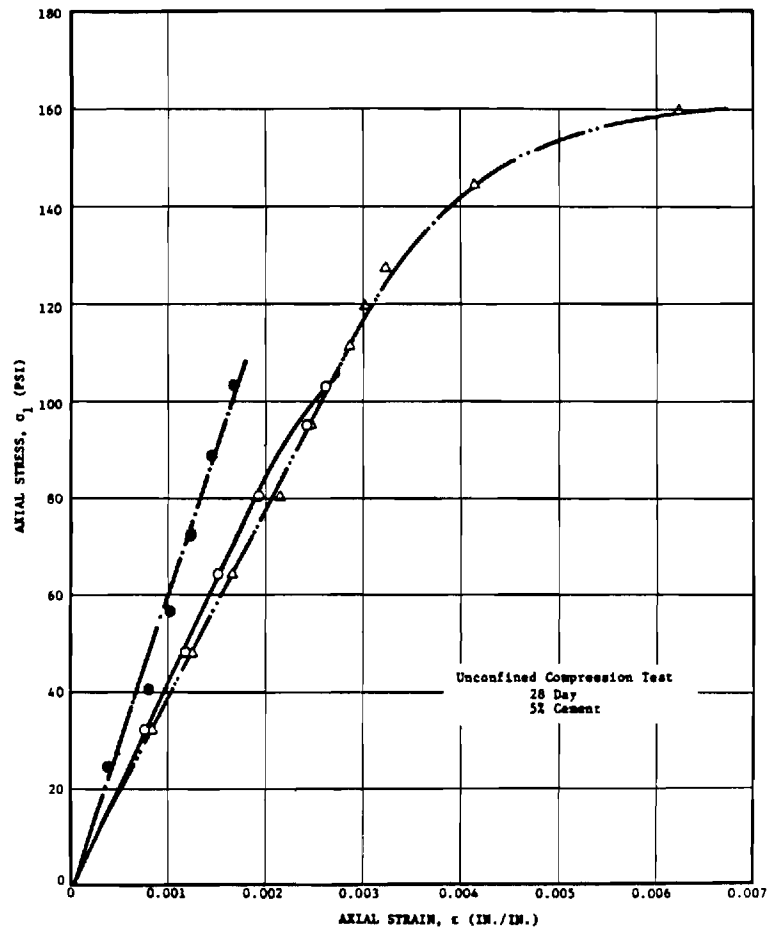


FIGURE 30. UNCONFINED COMPRESSION STRESS-STRAIN RESPONSE OF CEMENT TREATED SILTY SAND SUBGRADE.

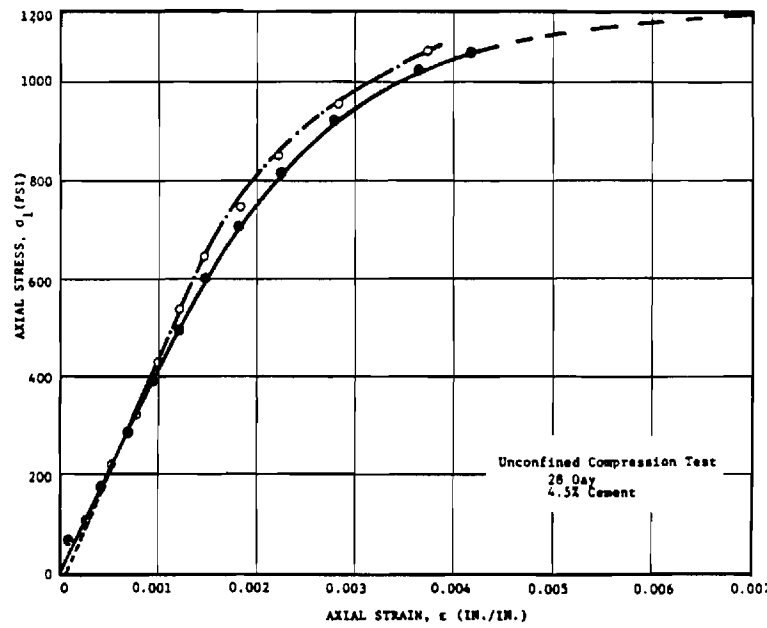


FIGURE 31. UNCONFINED COMPRESSION STRESS-STRAIN RESPONSE OF CEMENT STABILIZED CRUSHED STONE SUBBASE .

CHAPTER IV TEST SECTION CONSTRUCTION, INSTRUMENTATION AND LOADING

Introduction

A total of twelve test sections (Table 1) were constructed and tested to failure (or near failure) in the test facility. All tests were performed in an environmentally controlled room at a temperature varying from 78 to 80°F (25.5 - 26.7°C). The pit in which the tests were performed is 8 ft. (2.4 m) wide, 12 ft. (3.7 m) long and 5 ft. (1.5 m) deep. To study a maximum number of base variables a different structural section was constructed in each end of the pit to give two tests for each complete pit filling. Emphasis was placed during construction of the test sections on achieving uniform material properties and meeting Georgia Department of Transportation material specifications.

A modified B binder was used for Sections 1 through 8, and a B binder for Sections 9 through 12. All asphalt concrete was obtained from APAC Georgia Inc.-McDougald Warren Division. The modified B binder used in Sections 1 through 8 was from the Lithia Springs Plant. The B binder used in Sections 9 and 10 was from Forest Park, and in 11 and 12 from Lithia Springs. The change in type of mix and plant became necessary because the Georgia Department of Transportation shifted from the use of Modified B to B binder during the course of the project. Also, during the latter part of the project the Lithia Springs plant had a low volume of asphalt production.

The unstabilized crushed stone base course was constructed by blending different sizes of crushed stone together in a pugmill to minimize segregation. The crushed stone base course used in sections 1, 2, 8, 9 and 10 was compacted to 100% of AASHTO T-180 density. A cement treated soil subgrade was used in Section 11 and a cement stabilized subbase in Section 12. Because of the presence of the rigid working platform in these sections, a base density of 105% of AASHTO T-180 was obtained using the same compaction effort as used in previous sections.

The basic dynamic loading applied to the pavement was 6,500 lbs. (28.9 kN) applied uniformly over a circular area having a diameter of approximately 9.1 in. (231 mm). To decrease the time required to fail the pavements, the loading was increased to 7,500 lbs. (33.4 kN) after 2×10^6 load repetitions except in Sections 11 and 12 for which the magnitude of load was not increased. To prevent a localized, punching type failure from occurring during the test, the repeated loading was applied at a primary load position and six secondary positions located around the edge of the primary position.

Test Facility

General Test Facility

The tests were conducted in a special test facility located in the Geotechnical/Materials Laboratory, School of Civil Engineering. The test facility is shown in Fig. 32 including test pit, reaction frame and loading system. The test sections were constructed in a pit 8 ft. (2.4 m)

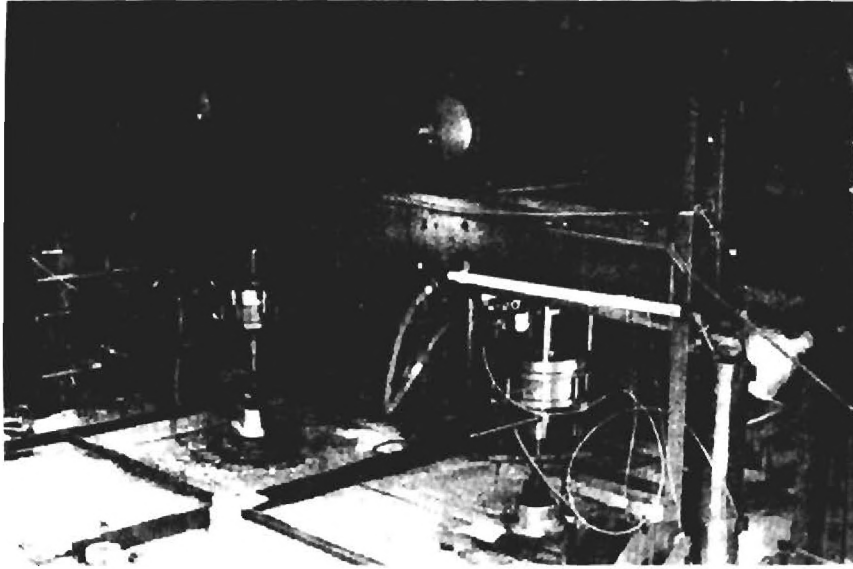


FIGURE 32. GENERAL VIEW OF PIT TEST FACILITY

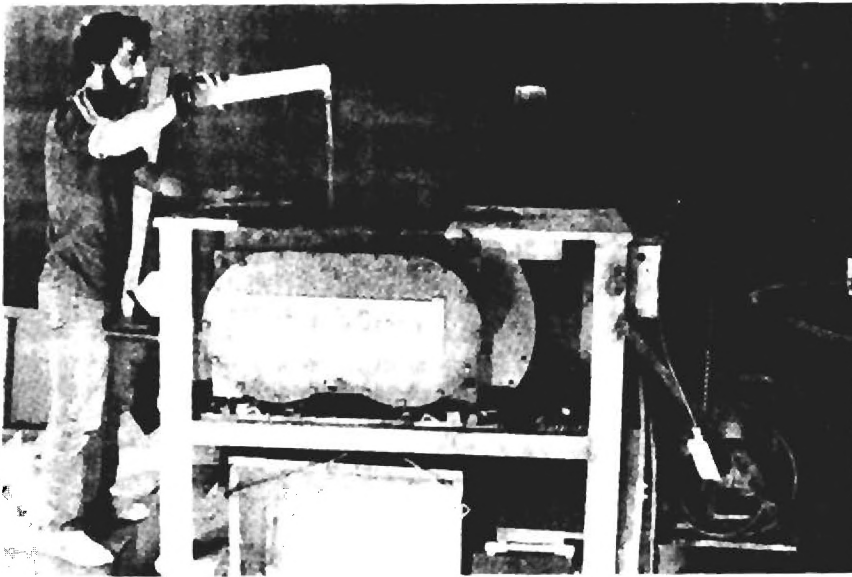


FIGURE 33. BARBER-GREENE PUGMILL USED TO MIX SOIL AND CRUSHED STONE.

by 12 ft. (3.6 m) in plan and 5 ft. (1.5 m) in depth. The top of the test pit is about 6 in. (152 mm) above the surrounding floor.

A constant temperature of 78 to 80°F (25.5 - 26.7°C) was maintained during the tests using a control system. The temperature control system consisted of a thermostat which controlled a large heater, and two air conditioning units. To prevent excessive heat loss or infiltration, the room was sealed and insulated. This system was capable of maintaining the required temperature in the test facility except when the outside air temperature was above 95°F (35°C), or below about 20°F (7°C). When these extremes in temperature were exceeded, testing was discontinued. Both air temperature and the temperature at the center of the asphalt concrete were monitored throughout the test by means of thermometers.

The walls and floor of the pit consist of 6 in. (152 mm) thick reinforced concrete. A false concrete bottom was constructed in the pit for this study to reduce the total depth from 8 ft. (2.4 m) to 5 ft. (1.5 m). Using the false bottom significantly decreased the volume of subgrade material required for each filling. Storage bins were constructed along the walls of the room to hold the materials during emptying and filling operations. A small belt conveyor, 1/2 ton crane and forklift truck were used for material handling.

Reaction Frame

A reaction frame extended horizontally across the pit in the long direction about 3 ft. (0.9 m) above the surface of the pavement (Fig. 32). For most load positions used in the load sequence, the center of the load was offset from the centerline of the reaction frame. Because of this eccentricity, a relatively large torsional moment was applied to the reaction frame by the 6,500 lb. (28.9 kN) load. Therefore, two wide flange sections spaced 23.5 in. (597 mm) apart were used to reduce the effects of torsion caused by the eccentric pavement loading.

The reaction frame consisted of two 12 ft. (3.7 m) long wide flange beams (10 x 8 x 33 lbs/ft.) stiffened by a channel cover plate (7 x 2-1/2 x 9.8 lbs/ft.) welded to the top of each beam. The horizontal reaction frame was supported on each end by two upright channels (9 x 2-1/2 x 13.4 lb/ft.) welded at the bottom to a heavy horizontal angle. The reaction frame was fastened together with 1 in. (25 mm) dia. steel bolts for easy disassembly and removal from the immediate vicinity of the pit during construction of each section. The end support frames were attached to the concrete walls of the pit by 7/8 in. (22 mm) dia. bolts retained in the pit wall by Rawl multi-calk anchors (No. 9135).

The loading system was attached to the load frame by means of a 1 in. (25 mm) thick, horizontally oriented thrust plate 26 in. by 33 in. (660 x 838 mm) in size. The horizontal thrust plate was attached using 1 in. (25 mm) dia. bolts to the bottom of the wide flange beams. Three 1/2 in. by 3 in. (13 x 76 mm) steel stiffeners 18 in. (454 mm) in length were welded to the top of the plate. Rapid positioning of the loading system at the seven fixed load locations was achieved using seven sets of bolt holes in the thrust plate made to support the load system. One load system and thrust plate was used on each end of the pit.

Test Section Construction

Introduction

All test sections were constructed using the same standardized procedures which were found to give consistent, reproducible results. After testing to failure, each section was completely removed from the pit and new sections were constructed from the bottom of the subgrade up. Only the silty sand subgrade soil was reused; after each test the subgrade soil was removed from the pit, stored, remixed and then placed and recompacted in the pit. Each filling of the pit required the movement of about 30 tons of material. Because of the small size of the pit, a significant amount of the material movement was accomplished by hand.

A general summary of the test sections constructed is given in Table 1. A detailed description of the materials used in the test sections is given in Chapter III.

Subgrade

To achieve uniform thickness and density of the soil, the pit was partitioned into six segments, and the required amount of soil added to each segment to give a compacted thickness of 2.0 in. (51 mm). The subgrade soil was placed at the optimum moisture content of 20.5% and compacted to 102.7 pcf (16.1 kN/m³) corresponding to 98% of the AASHTO T-99 density. The silty sand subgrade was brought in from stockpiles and temporarily stored in plywood bins constructed along the inside walls of the test facility. The existing water content of each batch of soil was measured using a Speedy Moisture Meter⁽¹⁾. The required amount of soil was then weighed out by placing it in a specially constructed, 10 ft.³ (0.3 m³) bottom dump bucket placed on a 1,000 lb. (4.5 kN) platform scale. A 1/2 ton (4.5 kN) overhead crane was used to move the bucket.

The soil was mixed in 150 lb. (0.67 kN) batches in a small 1/8 yd³ (0.1 m³) Barber-Green Pugmill (Fig. 33). During mixing the required quantity of water was added to bring the moisture content up to optimum. After mixing for about 1-1/2 to 2 min., the soil was discharged from the bottom of the pugmill into a wheelbarrow and dumped in one of the partitioned areas in the pit.

Twelve batches of soil were required to construct each 2 in. (51 mm) thick lift. After placing, the soil was leveled and the partitions removed. Compaction was achieved in 5 passes of a Wacker (Fig. 34) or a Jay 12 (Fig. 36) compactor. A pneumatic tamper having a square foot was also used to obtain compaction in the corners and along the sides of the pit. Before placing each new lift, the surface was scarified with a rake to give good bond between lifts.

Density Control. Immediately after compaction of each lift, a spring loaded, static penetrometer was used to carefully check the lift for proper density; if necessary soft areas were recompacted before the next lift was constructed. The density of the compacted subgrade was determined

1. A calibration was used between the Speedy Moisture Meter readings and the 105°C (221°F) oven moisture content.

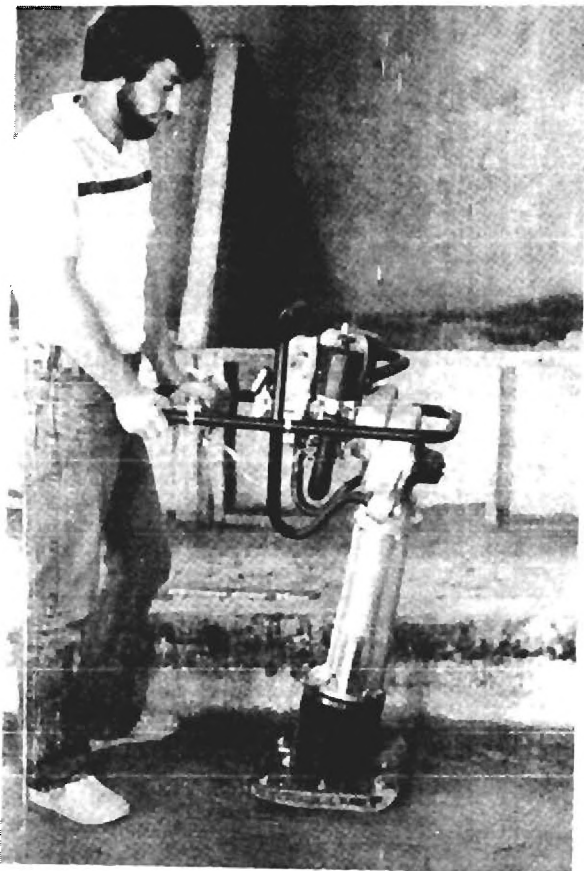


FIGURE 34. WACKER COMPACTOR USED TO OBTAIN 98% OF T-99 IN SILTY SAND SUBGRADE.

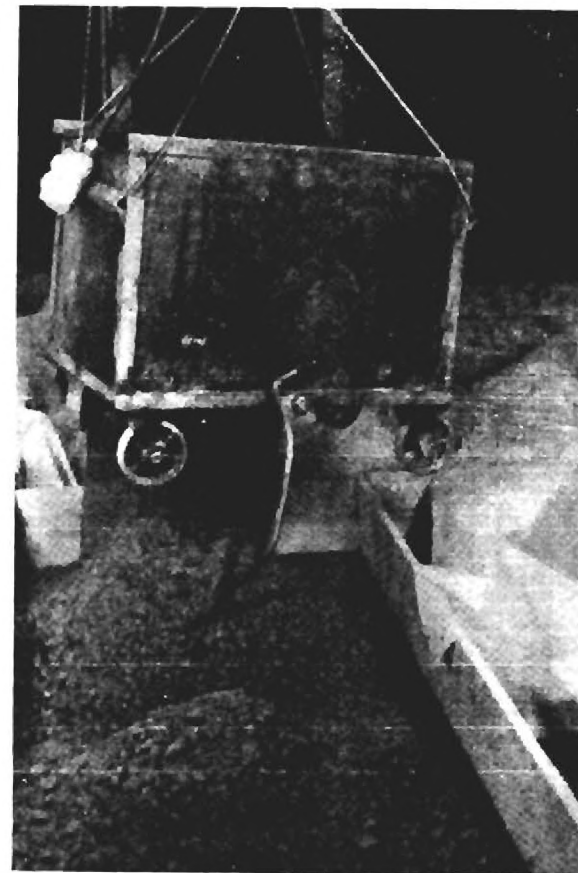


FIGURE 35. BOTTOM DUMP BUCKET USED TO PLACE SOIL, CRUSHED STONE AND ASPHALT CONCRETE.

using a thin wall, drive tube sampler. The average subgrade density of all test sections is 98 pcf (15.4 kN/m^3) which is 96.1% of the T-99 maximum dry density. After testing, dynamic cone penetrometer [20] readings were also taken in the subgrade. All sections had a uniform cone penetration resistance approximately equivalent to a standard penetration resistance of 7 to 8 blows per foot.

Protection Against Moisture Loss. Considerable caution was taken during construction to protect the subgrade from moisture loss due to evaporation. The exposed soil was covered with two 4 mil thick polyethylene sheets whenever possible during construction. After completing work each day, the surface was very lightly sprinkled with water. Two sheets of plastic were then placed over the surface and covered with wet burlap.

Crushed Stone Base

As discussed in Chapter 3, the standard crushed stone base was constructed by blending No. 5, No. 57 and No. 810 crushed stone together to give the gradation shown in Table 11. The three sizes of stone were stockpiled outside and brought into the test facility using a forklift truck. The forklift was fitted with a specially constructed tripping bucket attached to the front.

All mixing, handling, and moisture control procedures used for base construction were similar to those previously described for the subgrade. Each crushed stone size was weighed out and placed in the pugmill. The moisture content was adjusted to optimum and the stone blended. The crushed stone was discharged from the pugmill into the bottom dump bucket, and moved to the pit by means of the overhead crane. The stone was bottom-dumped from the bucket into the proper partitioned compartment of the pit (Fig. 35). Use of separate size stone, pugmilling and bottom dumping resulted in a very uniform, homogeneous blend having a minimum amount of segregation after placement.

The crushed stone base was placed in approximately 2 in. (51 mm) lifts. Actual lift thickness depended upon the number of lifts and the total base thickness. Compaction was achieved using 5 to 7 passes of the Jay 12 vibrating plate compactor (Fig. 36).

Density Control. The sand replacement method was used to determine the density of Test Sections 1 and 2. This method caused excessive disturbance of the crushed stone base. The density of all subsequent sections having unstabilized granular bases was therefore determined using a nuclear density gage. The nuclear density tests were performed by an inspector from the Georgia Department of Transportation. The average density of all sections (except 11 and 12) is 100.1% of the AASHTO T-180 maximum dry density of the crushed stone bases. The unstabilized crushed stone bases used in Sections 11 and 12 were constructed over a rigid cement stabilized layer. Apparently as a result, the density obtained in the crushed stone in these bases was significantly greater than the other granular base sections. The density of the inverted sections was 105.0% of the T-180 maximum dry density. Base course density test results are summarized in Table 14.

TABLE 14. CRUSHED STONE BASE COURSE DENSITIES.

Section	Density (% of T-180)
1 & 2	100.4 (1)
3 & 4	N/A
5 & 6	N/A
7 & 8	100.0 (2)
9 & 10	100.0 (2)
11 & 12	105.0 (2)

1. Density measured by sand cone method.
2. Density measured by nuclear gage.



FIGURE 36. JAY-12 VIBRATING PLATE COMPACTOR USED TO OBTAIN 100% OF T-180 DENSITY IN CRUSHED STONE BASE.

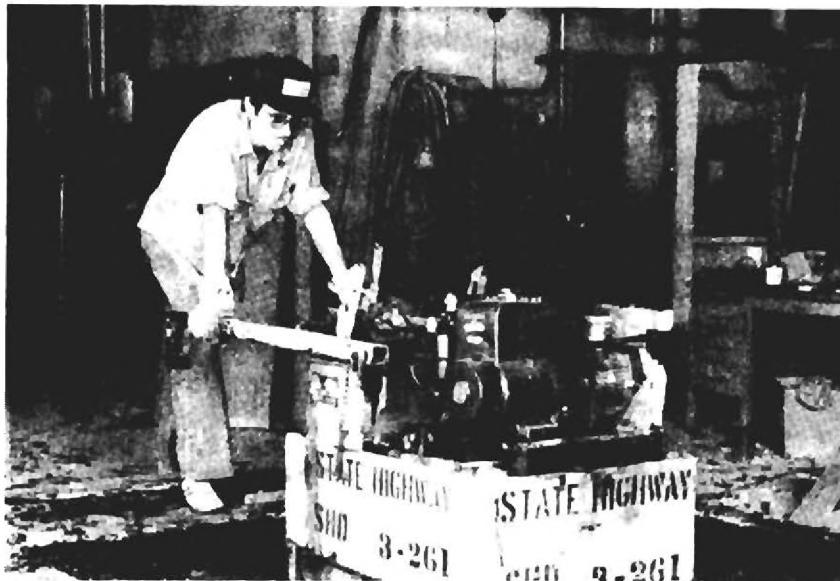


FIGURE 37. DENSIFICATION OF ASPHALT CONCRETE USING MAINTENANCE ROLLER OPERATED DRY.

Priming. The base (or the subgrade when full-depth asphalt concrete sections were used) was primed with a cut-back RC-70. The RC-70 was placed in a pressure vessel and heated to about 150°F (65°C), and then sprayed through a nozzle by applying a 30 psi (205 kN/m²) air pressure to the tank. Using this procedure a thin, uniform coating of RC-70 was obtained over the surface. To protect the surface from dust as the prime coat cured, a wood frame covered with polyethylene was placed over the top of the pit.

Asphalt Concrete

As previously discussed either a Georgia Department of Transportation Modified B or B binder was used for the full thickness of the asphalt concrete layer. The asphalt concrete was transported from the plant to the test facility in an enclosed plywood box. The box had 2 in. (51 mm) of styrofoam insulation to prevent excessive heat loss during transportation. Further, the insulated box was filled with 5,000 lb. (22 kN) of asphalt concrete, which was approximately three times the amount required for the single lift constructed from each load. The additional asphalt concrete mass helped maintain the temperature in the box during transportation and was wasted after each lift. At the time of delivery to the test facility, the temperature of the asphalt was between 290 and 300°F (143 to 149°C).

The pit was temporarily partitioned into six compartments as done during placement of the subgrade and crushed stone. The hot asphalt concrete was quickly shoveled from the box into the bottom dump bucket placed on platform scales. After filling with the correct weight of asphalt concrete, the bucket was rapidly moved to the correct location in the pit and dumped. The asphalt was quickly covered with three to five layers of cardboard insulation to prevent heat loss during placement of the remaining asphalt concrete. As the final compartments were filled, the asphalt concrete was leveled in each previously filled compartment using shovels and lutes. After preliminary leveling, the partitions were removed and the asphalt concrete cut into 3/4 in. (19 mm) wide slots left by the partitions. The asphalt concrete surface was then quickly leveled. Rolling was immediately performed using a small two wheel, vibratory roller used for maintenance by the Georgia Department of Transportation (Fig 37). Rolling was carried out in both the longitudinal and transverse directions of the pit until further rolling produced no noticeable indentation of the surface. Placing and rolling the asphalt concrete took about 20 to 25 minutes using a crew of approximately ten men. Where possible all lifts were placed on the same day; in this case a tack coat was not used between layers. Marshall specimens, beams and cylinders were made for future testing from asphalt concrete saved from each lift (except for sections 1 through 4).

Before loading the test sections, the asphalt concrete was allowed to cure for at least one week. After testing sections 3 and 4 which had bad asphalt concrete, the density of the asphalt concrete in subsequent sections was determined before testing using a nuclear moisture-density gage to insure Georgia DOT density requirements were met. After failure of the test sections, the asphalt concrete was cored and the asphalt content, density and gradation was determined by the Georgia DOT (Table 6).

Cyclic Loading

Loading Sequence and Magnitude

To prevent a localized punching failure from occurring during the test, the repeated loading was applied at a primary load position and six secondary positions located symmetrically around the edge of the primary position (Fig. 38). In all tests load was applied in the ratio of five repetitions at the primary position to each repetition applied at any individual secondary position. The basic pattern was to apply 100,000 repetitions at the primary position and 20,000 repetitions to each secondary position; this number, however, was reduced in the early phases of most tests; in a few tests greater numbers of load repetitions were applied in the latter phases of testing (Table 15).

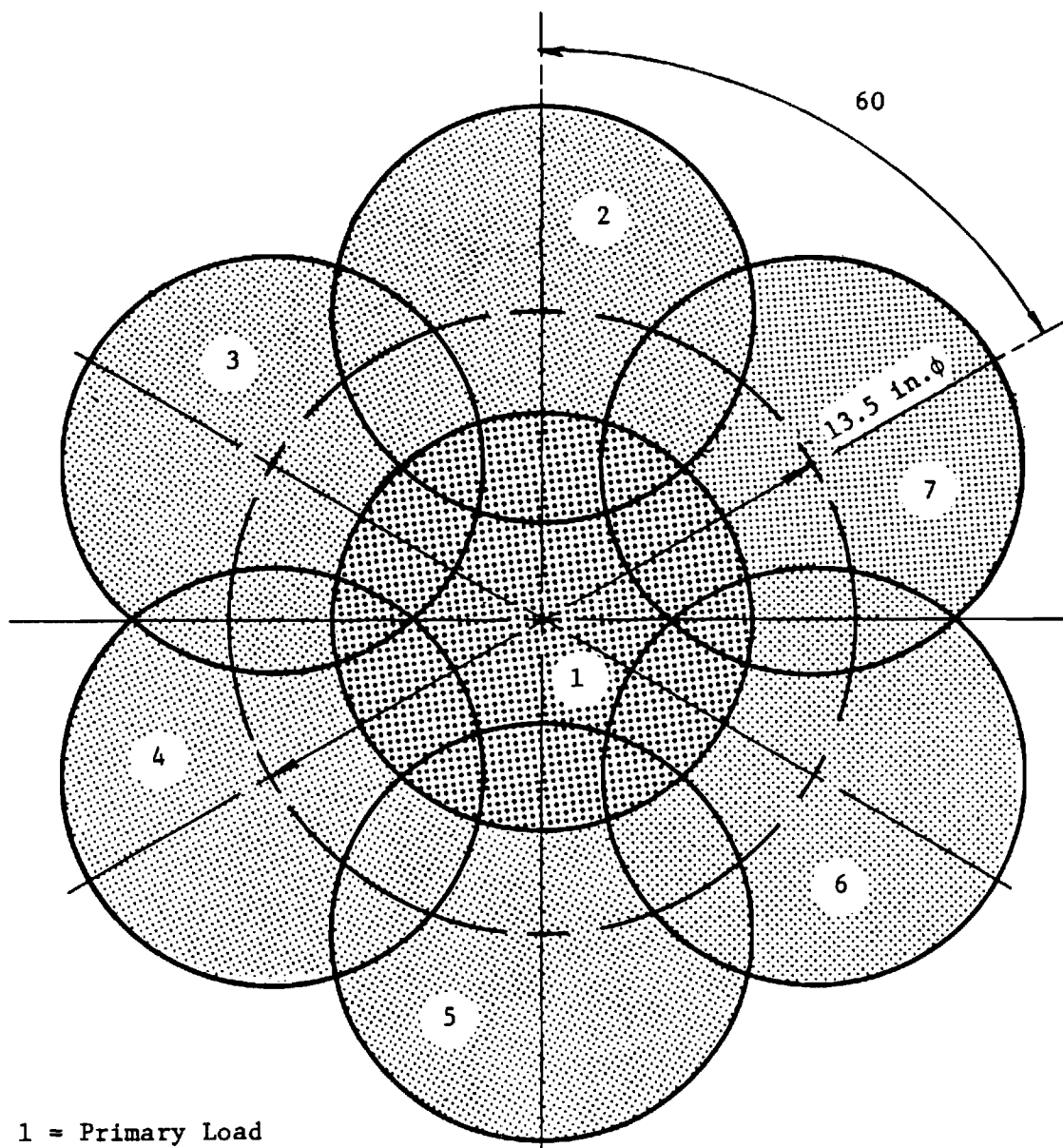
The pavement was subjected to a cyclic loading of 6,500 lbs. (28.9 kN) up to 2×10^6 repetitions. The load was applied over a circular area having a dia. of approximately 9.1 in. (231 mm). By applying the load to a water-filled, rubber bladder a pressure of approximately 100 psi (689 kN/m^2) was applied uniformly to the pavement surface. A load pulse of 0.17 sec. duration was applied throughout the tests at a frequency of 70 to 90 pulses per minute.

To decrease the time required to fail strong pavement sections, the loading was increased to 7,500 lbs. (33.4 kN) after 2×10^6 repetitions. In Sections 11 and 12 the load was maintained at 6,500 lbs. (28.9 kN) throughout the test. In this series, however, 200,000 load repetitions were applied at the primary load position in the latter stages of testing; use of this pattern of loading greatly increased the number of repetitions that could be applied during a given 24 hour day.

Cyclic Loading System

The dynamic loading was applied to the pavement system using a hybrid air-over-oil loading system. A general schematic of the cyclic testing system is shown in Fig. 39, and details of load actuator is shown in Fig. 40. The air-over-oil system was developed to apply to the pavement a large load in a short period of time (0.17 sec.). The applied pulse simulates a slowly moving heavy wheel loading. In the hybrid load system oil is sandwiched between a small 4 in. (102 mm) dia. aluminum, free floating piston on the top and a large 12 in. (305 mm) dia. aluminum piston on the bottom. Air pressure applied to the top of the small piston was transmitted undiminished to the large lower piston giving a large force out. A push rod transmitted this force from the lower piston to the loading bladder which rests on top of the pavement (Fig. 41). To develop the repeated loading, air was cyclically applied to the top of the upper cylinder.

The free-floating aluminum piston was located in the small upper cylinder on top of the oil. Only a small air space was left between the piston and top of the cylinder to minimize response time. The piston was guided in the cylinder by means of two sets of nylon wear rings. The free-floating piston was used in most tests to separate the air and hydraulic oil. The system was also found to work reasonably well without a physical separation between the air and oil, provided a baffle was



1 = Primary Load
Position
2-7 = Secondary Load
Positions

FIGURE 38. LOAD POSITIONS USED IN TEST SECTION LOADING.

TABLE 15. SUMMARY OF TEST SECTION LOADING HISTORY.

Test Section	Sequence 1			Sequence 2			Sequence 3			Sequence 4			Sequence 5		
	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position	Repetitions		No. @ (2) Primary Position
From	To	From		To	From		To	From		To	From		To		
1	0	10,000	10,000	10,000	3,350,578	100,000	-	-	-	-	-	-	-	-	
2	0	10,000	10,000	10,000	1,891,000	100,000	-	-	-	-	-	-	-	-	
3	0	10,000	10,000	-	-	-	-	-	-	-	-	-	-	-	
4	0	10,000	10,000	-	-	-	-	-	-	-	-	-	-	-	
5-8(3)	0	2,600	1,000	2,600	20,200	2,000	20,200	26,600	4,000	26,600	554,800	20,000	554,000	1,032,000	40,000
9	0	222,000	30,000	222,000	1,117,000	100,000	1,117,000	3,080,000	200,000	-	-	-	-	-	
10	0	101,967	30,000	101,967	1,117,000	100,000	1,117,000	3,080,000	200,000	-	-	-	-	-	
11-12(4)	0	60,000	25,000	60,000	280,000	100,000	280,000	4,500,000	200,000	-	-	-	-	-	

1. Applied load was 6,500 lb. (28.9 kN) unless indicated otherwise.
2. Five repetitions were applied at the primary position for every repetition applied at an individual secondary position. Then one-fifth this number of repetitions was applied to each secondary position; this sequence was thereafter repeated.
3. Tests were terminated after the following number of load repetitions: Section 5 - 1,389,896; Section 6 - 1,228,183; Section 7 - 1,032,000; Section 8 - 1,081,000.
4. Tests were terminated after the following number of load repetitions: Section 11 - 3,200,000; Section 12 - 4,500,000.

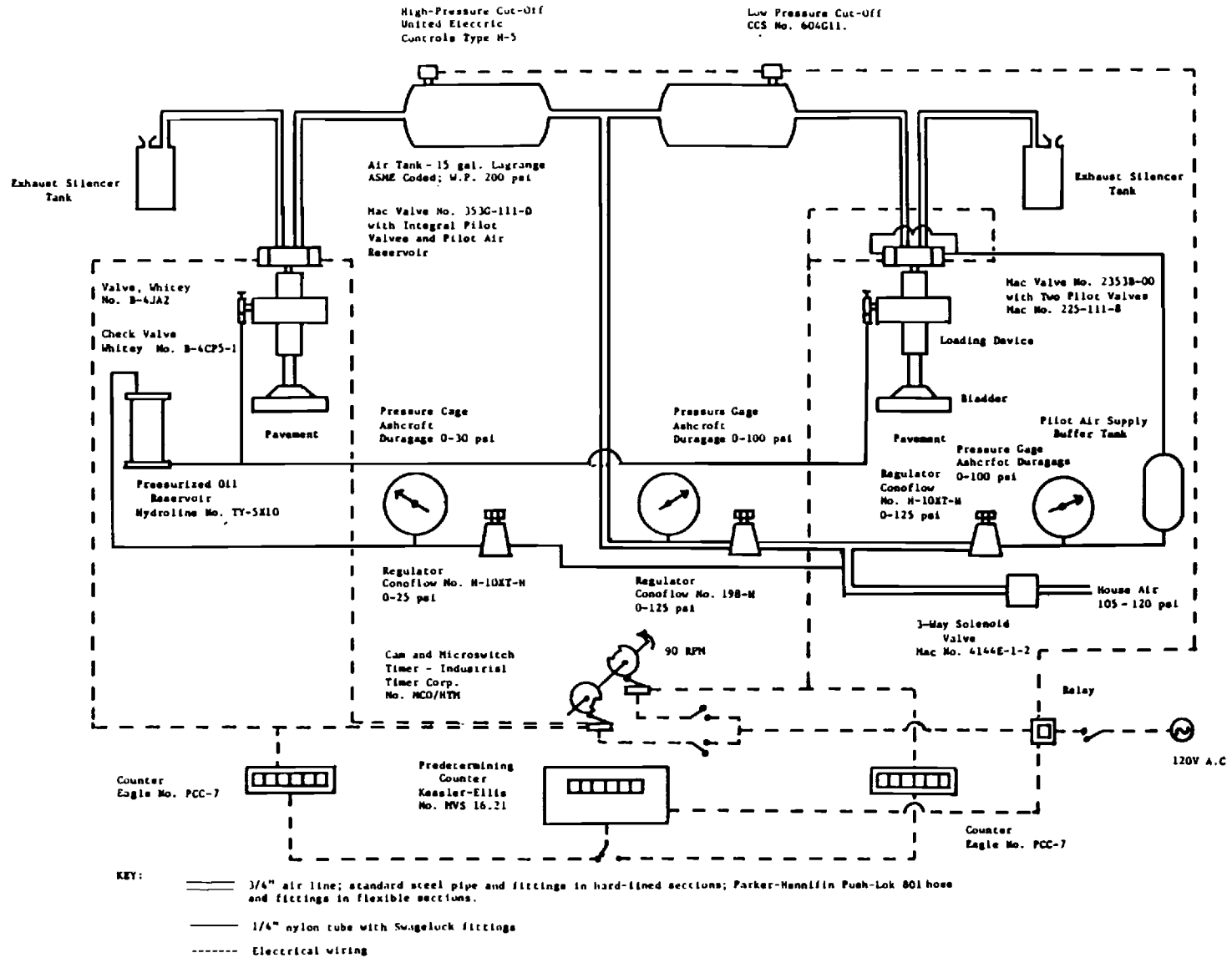


FIGURE 39. GENERAL SCHEMATIC OF DYNAMIC TESTING SYSTEM.

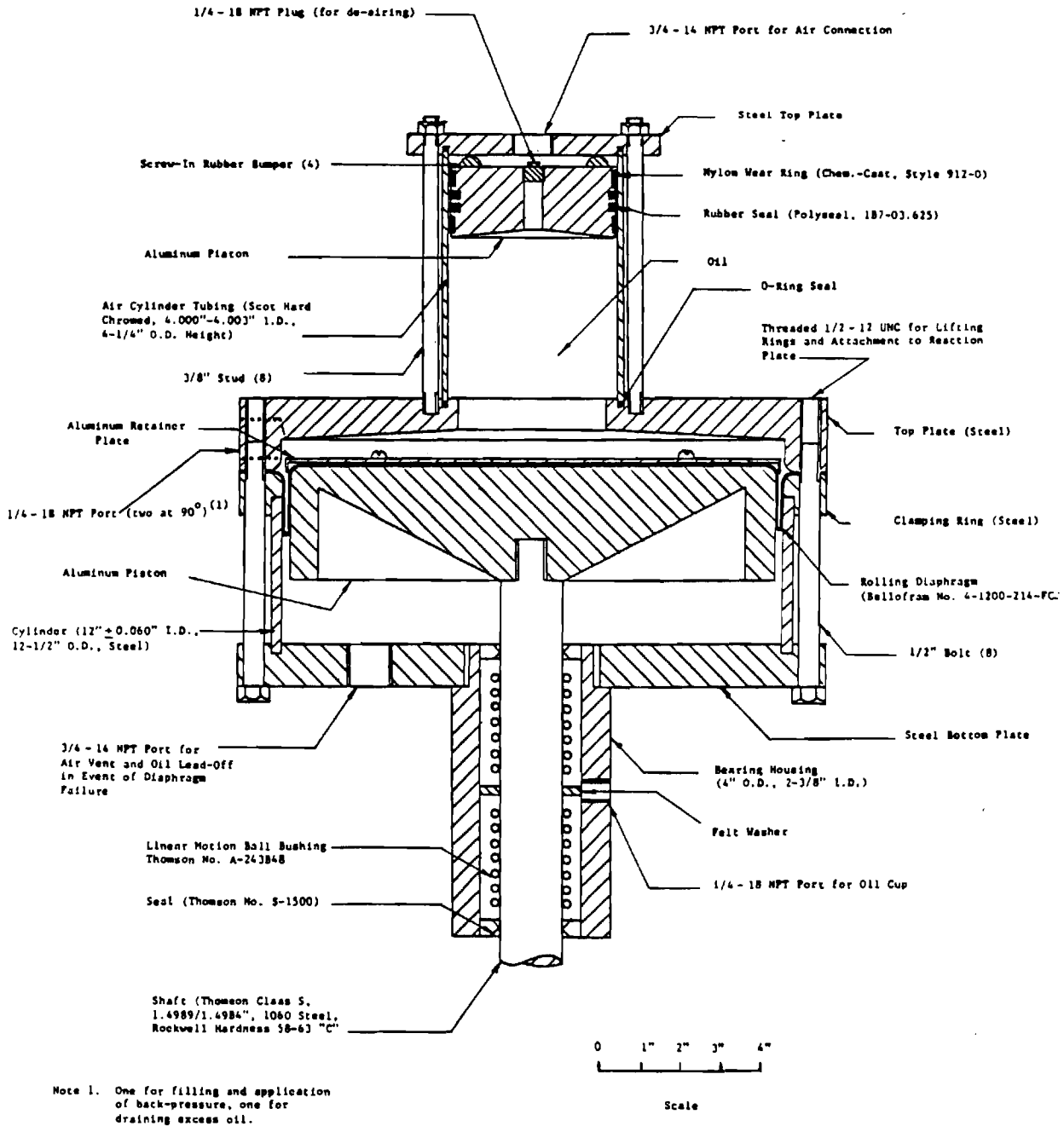


FIGURE 40. SECTION VIEW OF HYBRID AIR OVER OIL LOADING DEVICE.



FIGURE 41. RUBBER BLADDER AND LOAD FOOT USED IN ALL TESTS.

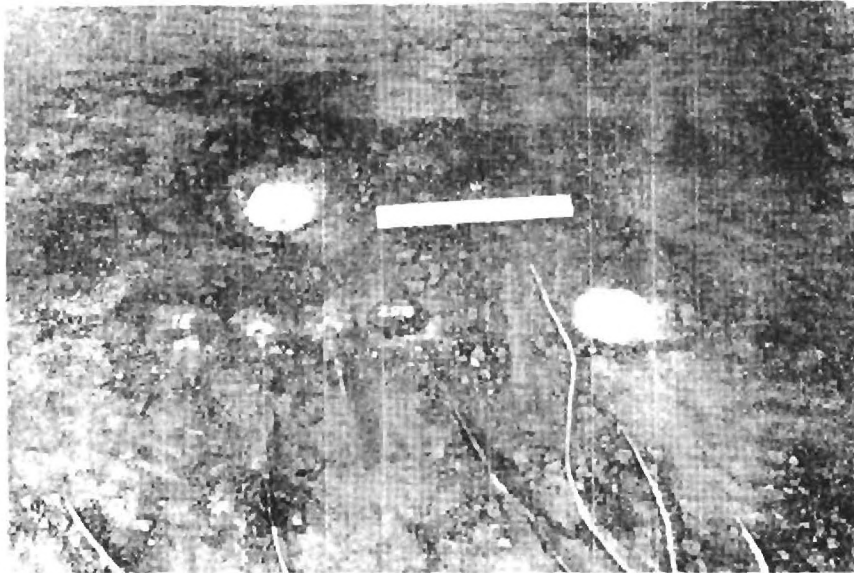


FIGURE 42. DIAPHRAGM TYPE PRESSURE CELLS AND STRAIN SENSORS USED IN TEST SECTIONS.

installed at the air inlet. The large lower piston was sealed to the walls of the cylinder using a rolling diaphragm. A 1 in. (25 mm) dia. push rod transmitted the load from the lower piston to the load bladder. The push rod was guided in the vertical direction using two linear motion Thomson ball bushings located in a housing below the large cylinder.

A constant seating loading was maintained between load applications to prevent a shock loading. The seating load was applied by maintaining a small head of hydraulic fluid on the cylinder through a pressurized oil reservoir (Fig. 39). In addition to maintaining a constant seating load, the oil reservoir kept the cylinder filled with oil during the test. An electronic clock timer was initially used to automatically shut the loading system off after the correct number of pulses were applied at each load position; subsequently a predetermining counter was used to stop the test. A low pressure sensor was installed in the air lines to turn the system off in the event the house air pressure dropped below a preset value.

Cyclic Load. A repeated load was applied to the pavement by cycling the air pressure on top of the small, upper piston. The cyclic air pressure was controlled by means of a 3/4 in. (19 mm) dia. spool valve operated by two small solenoid pilot valves. The solenoid valves were opened and closed by sending an electrical signal to them using a multiple cam operated microswitch timer. Both load systems were controlled by using this timer. A buffer air tank was placed in the system before the valves to prevent fluctuations of air pressure during loading. Pressure regulators were used to maintain the desired pressure.

Loading Bladder. Load from the piston was applied to the top of a water-filled, circular bladder (Fig.41) resting on the pavement surface. The bladder essentially consisted of a thin steel ring covered with a rubber diaphragm on the top and bottom. Upon application of load to the top of the bladder through a rigid, circular loading plate, the concentric steel ring lifted up from the pavement surface giving a loaded area having a dia. of about 9.1 in. (231 mm). The pressure created in the bladder by the applied load was transmitted through the water to the surface of the pavement as a uniform, circular loading. For a 6,500 lb. (28.9 kN) load the corresponding uniform vertical pressure was about 100 psi (689 kN/m²). To prevent failure of the rubber diaphragms on the bladder due to abrasion, two diaphragms were used on both the top and bottom. The diaphragms were 1/16 in. (1.6 mm) thick neoprene having a single ply of nylon reinforcement.

Load Actuator Movement. The load actuator had four internally threaded hollow steel rods extending upward from the top of the cylinder. These rods were used to bolt the cylinder to the flat thrust plate attached to the reaction frame. Movement of the load cylinder from one fixed load position to another was easily accomplished by a special carriage which hung from the reaction beams. The carriage rolled along the reaction beam on four wheels, and temporarily supported the load cylinder during movement. The load cylinder was supported by nylon bearings on two steel rods on the carriage. These rods were perpendicular to the reaction beams. Use of the carriage thus permitted easy movement of the load cylinder in two orthogonal directions. After positioning, cap screws were used to securely fasten the load cylinder to the thrust plate.

Instrumentation

The test sections were extensively instrumented to define the response of the pavement system. Bison type strain sensors were installed to measure both resilient and permanent deformations throughout the pavement section. Small diaphragm type pressure cells were used to measure vertical stress. Pressure cells and strain sensors placed on top of the subgrade are illustrated in Fig. 42. Resilient surface deflections were measured using linear variable differential transducers (LVDT's). Finally, permanent deformations of the pavement surface were measured from a string line using a metal scale. All instrumentation was carefully calibrated using the general procedures described in this section.

Strain Sensors. Both resilient and permanent deformations within the pavement were measured between pairs of Bison-type strain sensors. These strain sensors physically consisted of a thin, acrylic disc 1 to 4 in. (25 to 100 mm) in dia. having 200 to 400 windings of a small dia. copper wire. The strain sensors were on the induction principle and are excited by a Bison Soil Strain Box (Model 4101A). Relative movement between coils induces a change in current which is detected by the Bison box. Static movements were read directly from the box while transient movements were routed through the Bison box to a strip chart recorder. To reliably select and obtain readings from different pairs of coils, a switching box was used for each section to permit changing quickly between pairs of coils. Calibration was performed with all coils wired through the switching boxes. The strain sensors, which do not have any physical connection between coils, have proved in a number of different experiments to be reliable, rugged and relatively inexpensive.

A typical layout of strain sensors for both a granular base section and a full-depth asphalt concrete section is given in Figs. 43 and 44. Each of the sections tested had between 20 and 24 strain sensors. Generally 1 in. (25 mm) dia. sensors were used in the asphalt concrete, 2 in. (51 mm) dia. sensors in the crushed stone and 4 in. (102 mm) dia. sensors in the subgrade. The sensors were placed in vertical stacks with their flat side oriented horizontally. Using this instrumentation only displacement was actually measured between either vertical or horizontal pairs of coils. The average strain occurring between coils was determined by dividing the measured displacement by the distance between coils. Since the coils were closely spaced, the strain determined in this way was sufficiently close for practical purposes to the actual maximum strain occurring between coils.

Two primary vertical stacks of coils were placed between the bottom and surface of the pit (Figs. 43 and 44). Secondary coil stacks were placed in the surface and base course between the primary stacks. The main purpose of the secondary stacks of coils was to measure the relative horizontal tensile and compressive movements occurring in the surface and base course. Several special pairs of 1 in. (25 mm) dia. coils were also placed in the subgrade just below the interface to measure vertical strains in Test Sections 5 through 11.

Calibration. The strain sensors were calibrated before installation in each test section using either a micrometer stand or calibration rack. The calibration rack has 23 fixed positions in which a sensor can be easily positioned. One sensor was placed in a fixed location on the

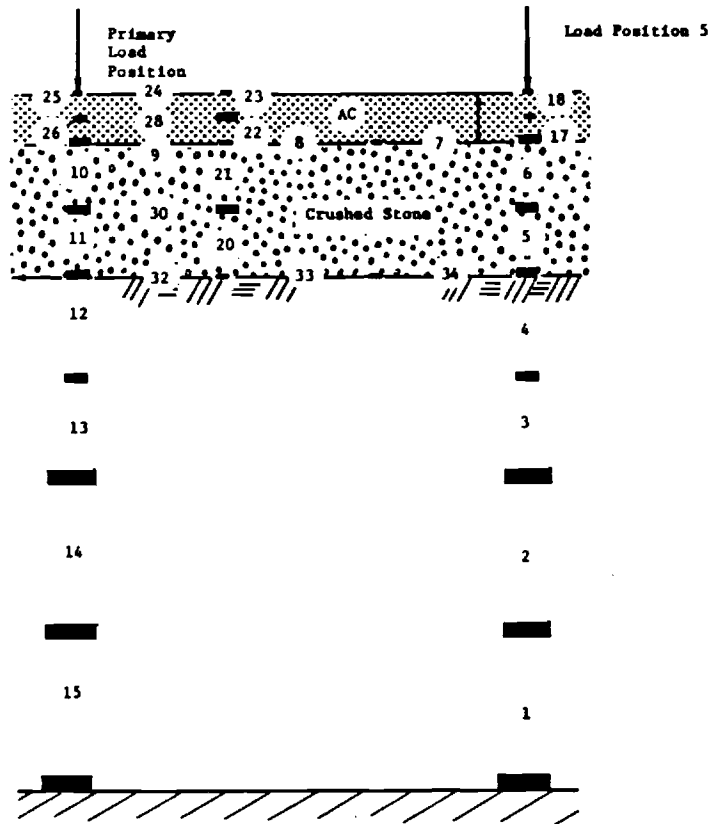


FIGURE 43. BISON STRAIN SENSOR LAYOUT USED IN SECTION 8 - CRUSHED STONE BASE

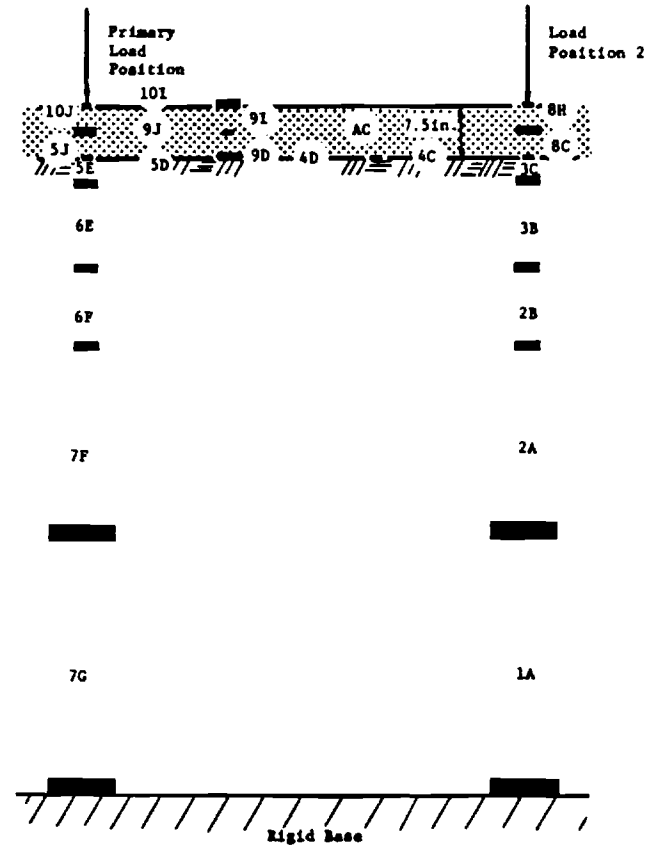


FIGURE 44. BISON STRAIN SENSOR LAYOUT USED IN SECTION 7- FULL-DEPTH A.C.

rack while the other sensor was moved progressively from one fixed position to the next with an output reading being taken on the box at each position. Since the distance between every combination of positions was known, distance measurements were not required during calibration.

All calibrations for dynamic displacement were made in a micrometer calibration stand. The micrometer calibration stand had two movable jigs in which a pair of sensors were fastened at the desired distance apart. The distance between coils was then changed using a micrometer having 0.0001 in. (0.0025 mm) divisions, and the corresponding output was read on a strip chart recorder.

Calibration readings for at least six displacements were obtained on the strain box (or strip chart recorder) for each initial strain coil separation distance. Coil calibration was carried out at several initial coil separations due to nonlinearity of the output. After the sensors were positioned in the test sections, the separation between the coils was determined using the static mode of operation and interpolating between calibration curves. Calibrations for static displacement (i.e., displacement due to a stationary constant load and also permanent deformation) were generally made using the calibration rack. Static displacement readings were taken for permanent deformation when the pavement was unloaded, and for a constant load condition where the load was not repeated. Strain sensor calibration was carried out with the loading bladder in the proper position when it would influence the readings.

Pressure Cells

Two and 2.5 in. (51 and 64 mm) dia. diaphragm type pressure cells described in detail elsewhere [32] were used in the test sections to measure vertical stress. These gages satisfied the pressure cell design criteria summarized by Brown [33]. Diaphragm type strain gages were used which measured both maximum radial and tangential strains occurring when the diaphragm deflected. The maximum strains were combined to give maximum cell sensitivity for small applied pressures.

The two diaphragm pressure cells used during the investigation are

1. Initially a 2 in. (51 mm) dia., 0.22 in. (5.6 mm) thick titanium cell was used to reduce problems with corrosion. The active diaphragm thickness varied from 0.055 in. to 0.078 in. (.4 to 1.9 mm) depending upon the stiffness of the material in which it was embedded.
2. During the study the pressure cell design was changed to a 2.5 in. (64 mm) dia., 0.30 in. (7.6 mm) thick cell made from aluminum. This larger pressure cell design was easier to machine. The active diaphragm thickness of this cell varied from 0.090 in. to 0.135 in. (2.3 to 3.4 mm). The active diaphragm dia. of both cells was 1.25 in. (32 mm).

All pressure cells were coated for corrosion protection with an epoxy conformal coating (Magnolia Plastics, Magnobond 3900). Before the cells were dipped in this coating, the surface was sand-blasted and degreased with MEK.

Output from the pressure cells was sent through a Budd or BLH switching and balancing unit to either a Hewlett-Packard Model 321 DC recorder or Model 320 DC recorder. When the DC recorder was used excitation was supplied by a 6 volt dry cell battery. A variable potentiometer was used to maintain the excitation throughout the test at either 4 or 5v ($\pm 0.010v$). The output from the pressure cells was amplified by a Neff Model 122 DC amplifier with filtering capability.

Calibration. Calibration was carried out in a steel calibration tank about 17 in. (430 mm) in dia. and 8 in. (200 mm) in height. Subgrade soil was placed in the tank at the same moisture content and density used in the test pit. As the soil was being placed in the calibration tank, the pressure cell was carefully embedded in the soil at a depth of about 4 in. (100 mm). A rubber membrane was then placed over the soil and the top fastened on the calibration tank. Air pressure was applied to the top of the membrane in small increments over the range of pressure expected to develop in the pit, and the corresponding output from the cell recorded. The air pressure was measured using a test gage and controlled with a sensitive regulator. Before calibration the first time, each cell was preconditioned by applying at least 50 cycles of stress up to 60 psi (413 kN/m^2).

Selected cells were dynamically calibrated by applying a 0.17 sec. pulse to the cell while placed within the soil tank. Dynamic calibration was accomplished by filling the space above the top of the rubber membrane with water, and applying a cyclic air pressure to the top of the water. For the two pressure cells used in this study, the dynamic response was found to be within about $\pm 2\%$ of the static response. Therefore the pressure cell output did not have to be corrected for dynamic effects.

The cross-sensitivity of selected cells was also determined by placing the cell in the same vessel without soil so that an equal pressure was applied to the cell. Considering the response of the pressure cell in soil to be correct, the cross-sensitivity of the pressure cell was determined by dividing for a given output signal the difference in pressures for the two conditions (cell embedment in air and water) by the pressure in soil, and multiplying by 100 to convert to percent. The cross-sensitivity of the two pressure cells was found to be about 16%. No significant difference in cross-sensitivity was observed between the 2.0 and 2.5 in. (51 and 64 mm) dia. pressure cells.

Displacement

Resilient displacements across the pavement surface were measured using 5 D.C. linear variable differential transformers (LVDT's). The LVDT's were excited with a 24v D.C. power supply, with the output being recorded on a Hewlett-Packard Model 320 D.C. strip chart recorder. The LVDT's were precisely calibrated by placing each transducer in a stand and applying known displacements using a micrometer. Periodically the calibrations were checked by inserting a calibration block of known thickness and comparing the observed recorder pen movement with the calculated movement using the calibration constant for the LVDT obtained from the micrometer calibration.

Permanent surface rutting was determined by periodically measuring with a metal scale the distance from a reference string line to the surface of the pavement. Readings to the nearest 1/32 in. (0.8 mm) were taken on both sides of the load in the short direction of the pit. As previously discussed, resilient and permanent displacements occurring within the pavement were obtained using the Bison strain sensors.

Conclusions

Twelve full-scale test sections were carefully constructed over a relatively weak silty sand subgrade 50 in. (1270 mm) thick. A completely new section was constructed for each test including the subgrade. All test sections except 3 and 4 met Georgia DOT construction specifications. Sections 3 and 4 were later found to be constructed using an asphalt concrete having an excessively high asphalt content that resulted in a premature rutting failure of the asphalt concrete during subsequent testing. The granular bases used in the conventional sections were compacted to 100% of AASHTO T-180 density. To minimize segregation, different sizes of stone were blended together just before placement. The blended aggregate was bottom dumped to minimize segregation.

Two hybrid air-over-oil cyclic loading systems were designed and constructed (including the reaction frame) to test to failure the pavement sections. The two testing systems were used to apply over 17 million load repetitions during the study. These testing systems were found to be quite reliable and relatively easy to maintain considering the large number of repetitions applied.

The test sections were fully instrumented to give important information concerning the amount and distribution of permanent rutting within each layer, resilient displacement, resilient strain and vertical stress on the subgrade. This response information was later used in Chapter 6 to help extend the results to sections having different geometries and loading conditions than used in this study.

CHAPTER V TEST RESULTS

Introduction

The purpose of testing to failure full-scale pavement sections in the laboratory was to study the feasibility of replacing full-depth and deep strength asphalt concrete with crushed stone base pavements utilizing relatively thick layers of unstabilized aggregate. Therefore, for comparison purposes tests were performed on both full-depth asphalt concrete sections and also crushed stone base pavement sections. A detailed description of the materials used was given in Chapter III, and construction, instrumentation and testing of the pavement sections were described in Chapter IV.

Definition of Fatigue and Rutting Failure

Fatigue. A fatigue failure was defined for purposes of this study as the initiation of CLASS 2 cracking. Class 2 cracking is defined [5] as the stage where cracks have connected together to form a grid type pattern. Cracking was found in general to progress relatively rapidly over the loaded area. Therefore, this somewhat general definition of a fatigue failure was in practice adequate (refer for example to Fig. 92 for two stages of crack development). Only fine hairline cracks which often were hard to see developed in the sections undergoing fatigue failures. Testing was terminated before wider cracks could develop because of the large number of load repetitions required to cause failure of the sections tested.

Rutting. A rutting failure was defined *for comparison purposes* as a 0.50 in. (12.7 mm) rut depth measured from a fixed string line. Admittedly smaller rut depths might frequently be used for design by the Georgia DOT. The 0.5 in. (12.7 mm) rut depth was the average of the rut depths measured at completion of testing at the primary load position and at completion of testing at the sixth secondary load position. Also, since the asphalt concrete sections often underwent relatively large amounts of rutting during the early stages of loading, the rut depth occurring during the first 1,000 repetitions was not included in the average.

The average value of rutting was used since an important amount of creep and plastic flow of the asphalt concrete and base materials was found to occur as the load was moved back and forth between the primary and secondary load positions. When the load was applied at the primary load position, the underlying material flowed out laterally and then upward from beneath the load causing a bulge to appear around the sides of the loaded area. As the secondary positions were loaded, the center area went up, and the loaded area moved downward. As a result, a given point within the general loaded area was cyclically going up and down depending upon the physical location of the load.

Materials

The asphalt concrete used in all sections was composed for the entire depth of either a Georgia DOT Modified B or B-binder. The asphalt concrete was compacted to approximately 99% of the maximum 50 blow Marshall density. The crushed stone base course aggregate used in this study was obtained from the Norcross Quarry of Vulcan Materials Company. The stone used in the conventional crushed stone base sections was compacted to slightly in excess of 100% of AASHTO T-180 density. The two inverted sections had a crushed stone base overlying either a 6 in. (152 mm) layer of cement treated subgrade soil (Section 11) or cement stabilized crushed stone (Section 12). The crushed stone base density in these two sections using the same level of effort was about 3.6% higher than in the other granular bases.

The subgrade consisted of a loose silty sand having a standard penetration resistance of about 8 blows per foot. The depth of the subgrade was 50 in. (1270 mm). Before constructing each section⁽¹⁾, the subgrade was removed and replaced in 2 in. (51 mm) lifts at an average of 98% of AASHTO T-99 density.

Structural Sections

The pavement sections tested were as follows (Table 1): (1) five crushed stone base sections, (2) five full-depth asphalt concrete sections, and (3) two inverted sections. All crushed stone base and inverted sections had an asphalt concrete thickness of 3.5 in. (89 mm). The crushed stone bases used with the conventional pavements were either 8 or 12 in. (203-508 mm) thick. The following three crushed stone base gradations were used in the investigation: (1) a standard gradation close to the average gradation used in the Atlanta area, (2) a fine gradation, and (3) a coarse gradation. The coarse and fine gradation were only used in Sections 9 and 10, respectively. The base course gradations were given in Table 11.

The full-depth sections consisted of either 6.5, 7.0, or 9.0 in. (165, 178, 229 mm) of asphalt concrete binder; these sections rested directly on the subgrade. The two composite sections consisted of 8 in. (203 mm) of unstabilized crushed stone sandwiched between 3.5 in. (89 mm) of asphalt concrete above and 6 in. (152 mm) of cement stabilized material below. One inverted pavement (Section 11) had a cement treated silty sand subbase, while the other inverted pavement (Section 12) had a cement stabilized crushed stone subbase.

Loading

As discussed in Chapter 4, a repeated loading was applied to the pavement surface of 6,500 lbs. (29 kN) exerting a uniform contact pressure

-
1. The lower 4 ft. (1.2 m) of the subgrade from Sections 3 and 4 was reused in Sections 5 and 6 because it was not damaged during the 10,000 load repetitions applied to Sections 3 and 4.

of 100 psi (689 kN/m²). Sections 9 through 12 were tested to more than 2 million load repetitions. After 2 million repetitions, the load applied to Sections 9 and 10 was increased to 7,500 lbs. (33 kN) to reduce the time to failure. In Sections 11 and 12 the load was maintained at 6,500 lbs. (29 kN) after 2 million repetitions, but the number of load repetitions applied at each location was doubled. To simulate traffic wander the stationary circular loading was applied in a primary load position and six supplementary positions located symmetrically around the edge of the primary position as shown in Fig. 38.

Performance

The results obtained from repeatedly loading the sections to failure are summarized in this chapter. Because of the large quantity of experimental data collected during this study, only the most relevant information is presented.

A general summary of the experimental results is given in Tables 16 and 17. Table 16 serves as a representative summary of the detailed response information obtained from these tests. The two test sections described in each subsequent section were constructed at the same time. Therefore, these companion sections should have similar material characteristics.

Pavement Performance

Sections 1 and 2

Test Sections 1 and 2 consisted of 3.5 in. (89 mm) of modified B-binder overlying a crushed stone base 12.0 and 8 in. (305 and 203 mm) thick, respectively. The modified B asphalt concrete mix design is given in Fig. 10; this mix was obtained from APAC-Georgia, Inc.'s Lithia Springs plant. The standard crushed stone base gradation was used in Test Sections 1 and 2.

Rutting. Test Section 1 was tested to 2.4 million load applications before terminating the test. An extrapolation of the test results indicates a rutting failure (0.5 in., 25 mm) rut depth would have occurred at about 3.0 to 3.5 million repetitions. Section 2 failed in rutting after 1 million load repetitions. The variation of rut depth under the center of the primary load position with number of load repetitions is shown for Sections 1 and 2 in Figs. 45 and 46, respectively.

By the completion of testing for Sections 1 and 2, a limited amount of fatigue cracking had developed. These cracks usually disappeared when the load was positioned above them, only to reappear again when the load was moved to another location. Healing and the reappearance of these cracks was a common occurrence in Sections 1 and 2. The extent of cracking, however, was never any greater than shown in Fig. 47. Since Section 1 had developed a limited amount of cracking after 2 million repetitions, the postulation is made that by the estimated failure of 3 to 3.5 million repetitions cracking would have developed sufficiently

TABLE 16. DETAILED SUMMARY OF RESILIENT TEST SECTION RESPONSE.

Sect.	RESILIENT SURFACE DEFLECTION (IN.)																						
	10,000 Repetitions				100,000 Repetitions				1,000,000 Repetitions				Termination										
	q _v ⁽¹⁶⁾	1	2	3	4	q _v ⁽¹⁴⁾	1	2	3	4	q _v ⁽¹⁴⁾	1	2	3	4	q _v ⁽¹⁴⁾	1	2	3	4			
1	-	0.011	-	0.015	0.0069	-	0.020	0.020	0.001	0.0055	-	0.026	0.062	0.012	0.093	-	0.051 ⁽³⁾	0.044	0.020	0.011			
2	-	0.030	0.019	0.010	0.0045	-	0.029	0.015	0.006	0.0022	-	0.035	0.024	0.014	0.004	-	0.045 ⁽⁴⁾	0.025	0.015	0.0014			
3						PREMATURE FAILURE - EXCESSIVE ASPHALT CONTENT																	
4																							
5	-	0.010 ⁽¹³⁾	0.011	0.005	0.006	-	0.011	0.014	0.008	0.004	-	0.014	0.012	0.008	0.005	-	0.011 ⁽⁹⁾	0.012	0.007	0.006			
6	-	0.016 ⁽¹³⁾	0.014	0.007	0.006	-	0.016	0.024	0.013	0.010	-	-	-	-	-	-	0.016 ⁽¹⁰⁾	0.024	0.013	0.010			
7	-	0.014	0.017	0.0088	0.0064	-	0.018	0.022	0.015	0.003	-	0.013	0.016	0.010	0.0058	-	0.016 ⁽⁵⁾	0.022	0.013	0.0059			
8	-	0.016	0.020	0.012	0.0054	-	0.015	0.022	0.013	0.0033	-	0.0151	0.025	0.013	0.0079	-	0.015 ⁽⁶⁾	0.025	0.013	0.0079			
9	-	0.015	0.017	0.0096	0.0080	-	0.016	0.023	0.012	0.009	0.029	0.016 ⁽⁷⁾	0.026	0.015	0.007	-	0.017 ⁽⁷⁾	0.035	0.013	0.0099			
10	-	0.016	0.022	0.010	0.006	-	0.015	0.020	0.010	0.008	0.037	0.016	0.021	0.012	0.009	-	0.023 ⁽⁸⁾	0.034	0.020	0.014			
11	0.026	0.003	0.006	0.003	0.002	-	0.003	0.005	0.003	0.002	-	0.001	0.003	0.002	0.001	-	0.004 ⁽¹¹⁾	0.007	0.002	0.001			
12	0.017	0.004	0.005	0.002	-	-	0.004	0.005	0.003	0.002	-	0.003	0.006	0.002	0.002	-	0.003 ⁽¹²⁾	0.004	0.002	0.002			

NOTES: 1. Resilient surface deflections are average values. For tests 1 and 2 load positions 1, 2, 3 and 4 correspond to radial distances of 7, 12, 16.5, and 21 in. from the centerline; for tests 5 through 12 they correspond to distances of 11, 10, 14.5, and 19 in.

2. Average horizontal tensile strain for the first 100,000 load repetitions.

3. Resilient surface deflections at 2,125,555 load repetitions.

4. Resilient surface deflections at 1,099,950 load repetitions.

5. 110,000 repetitions.

6. 1,081,000 repetitions.

7. 2,280,000 repetitions.

8. 3,080,000 repetitions.

9. 200,000 repetitions.

10. 350,000 repetitions.

11. 3,600,000 repetitions.

12. 4,400,000 repetitions.

13. Initial sequence of readings taken at about zero repetitions.

14. Deflections were not taken under the bladder; any set of readings with a value at the centerline corresponds to a special rigid plate load.

15. Resilient strain of 460×10^{-6} in./in. at 1,000 reps. and 633×10^{-6} in./in. at 126,000 reps.

16. Very small strains were observed in the cement stabilized layer.

TABLE 17. DETAILED SUMMARY OF RESILIENT TEST SECTION RESPONSE
(CONTINUED) .

RESILIENT STRAIN RESPONSE								
Sect.	Horiz. Tensile Strain ($\times 10^{-6}$)		Vert. Stress (psi)		Vertical Strain ($\times 10^{-3}$)			
	Bot. A.C.	Bot. Stone	Top Stone	Top Subgrade	AC	Top Stone	Bot. Stone	Top Subgrade
1	465	597	—	3.4	-	-	-	1.7
2	674	754	—	-	11.0	21.3	-	13.1
3	—	—	—	—	—	—	—	—
4	—	—	—	—	—	—	—	—
5	319	-	-	8.7	0.85	-	-	1.38
6	460(15)	-	-	12.6	-	-	-	1.50
7	410	-	-	12.9	0.65	-	-	2.2
8	300	375	-	11.9	-	0.56	.11	1.85
9	280	1,080	62	11.1	-	0.56	0.34	1.75
10	400	1,025	54	6.8	-	0.62	0.40	2.50
11	340	54 ⁽¹⁶⁾	-	3.3	-	0.73	0.37	0.39
12	260	22 ⁽¹⁶⁾	-	3.4	-	0.76	0.42	0.34

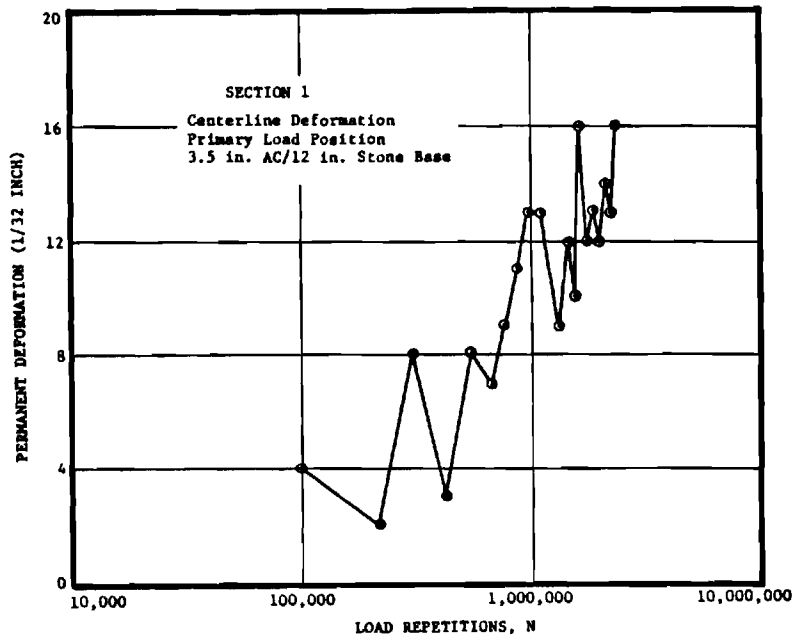


FIGURE 45. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 1 .

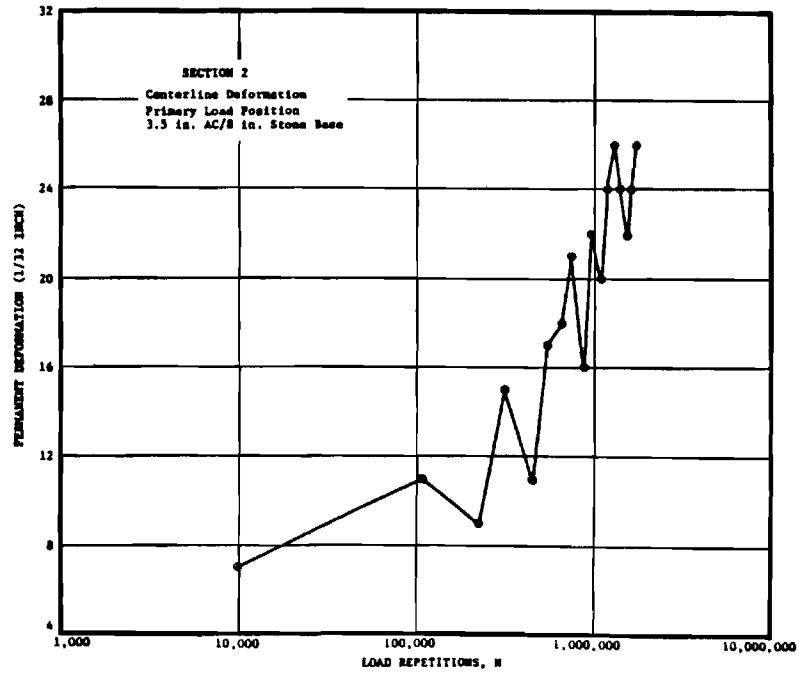


FIGURE 46. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 2 .

to result in a combined rutting/fatigue failure of this section. This postulation is in agreement with the results of subsequent sections which failed in fatigue.

Figs. 45 and 46 nicely illustrate the up and down movement of the surface beneath the primary load position due to lateral shear flow of the material. A number of the points of minimum deflection (i.e., the rut depth immediately after loading the secondary positions) were not measured for Sections 1 and 2 because the importance of shear flow and creep were not fully realized at the time; these points are therefore frequently missing on the figures.

The surface deflection profile at selected numbers of load repetitions is shown for Sections 1 and 2 in Figs. 48 and 49, respectively. Loading the pavement at the six secondary load positions was reasonably effective in preventing a punching failure from occurring at the primary position. Nevertheless, some punching did occur as indicated by the deflection profile. Since rutting in a prototype pavement also occurs over a relatively narrow width, some punching also occurs in the field, although it is probably somewhat less than in the test sections. In this study the performance of the crushed stone base sections was directly compared with full-depth asphalt concrete sections for similar load, environmental and subgrade conditions. Therefore, the absolute degree of punching experienced should not significantly influence the findings.

Distribution of Permanent Deformation. The variation of permanent deformation in Sections 1 and 2 with depth and number of load repetitions is summarized in Table 18 and Figs. 50 and 51. These results were obtained from the strain sensors placed throughout the pavement structure.

In Section 1 the majority (59%) of the total permanent deformation occurred in the 3.5 in. (89 mm) asphalt concrete layer; 21% occurred in the 12 in. (305 mm) crushed stone base; and 20% in the subgrade. In Section 2 the crushed stone base thickness was reduced to 8 in. (203 mm). Use of the thinner base caused an important influence on the relative distribution of permanent deformation through the section: (1) the relative amount of permanent deformation in the subgrade almost doubled going from 20 to 39%; (2) relative permanent deformation in base slightly increased going from 21% to 27%; (3) finally, permanent deformation in the asphalt concrete surfacing dropped from 59% to 34%. Thus presence of the thinner base tended to shift some of the rutting from the asphalt concrete to the subgrade.

Resilient Response. The resilient (dynamic) response of Sections 1 and 2 is summarized in Table 17. For Section 1 the measured horizontal tensile strain in the bottom of the asphalt concrete was 465×10^{-6} in./in., the dynamic vertical stress on the subgrade was 3.4 psi (23.4 kN/m^2), and the vertical strain at the top of the subgrade was 1.7×10^{-3} in./in. The typical variation of resilient surface displacement for Sections 1 and 2 is given in Figs. 52 and 53. The resilient surface displacement in general gradually increased with load repetitions after an abrupt initial increase. The variation for Section 2 of resilient vertical

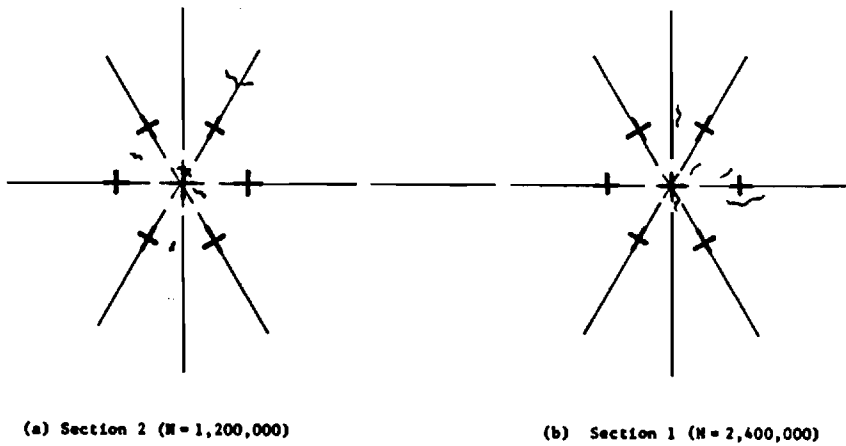


FIGURE 47. SURFACE CRACKING OBSERVED IN TEST SECTIONS 1 AND 2 AT INDICATED NUMBER OF LOAD REPETITIONS.

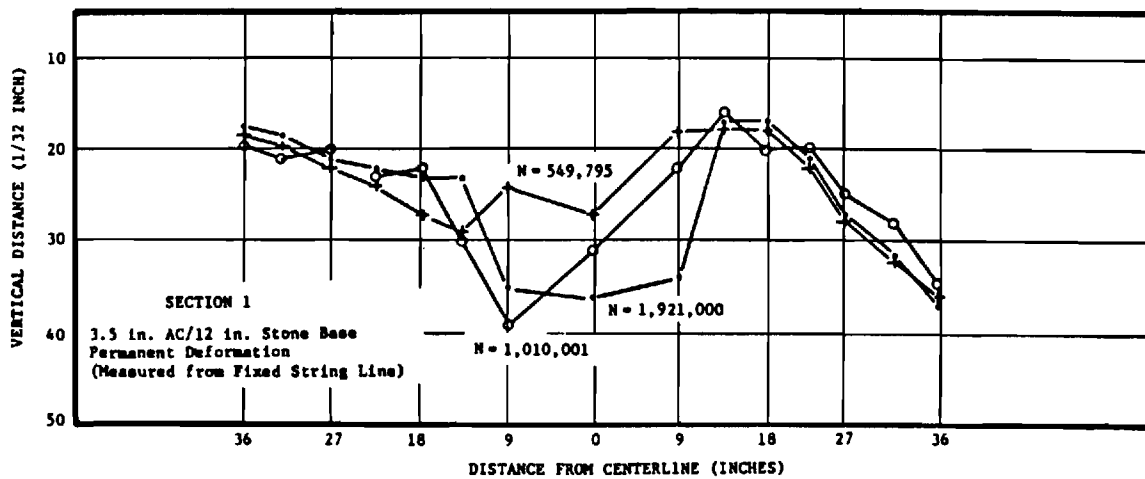


FIGURE 48. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 1.

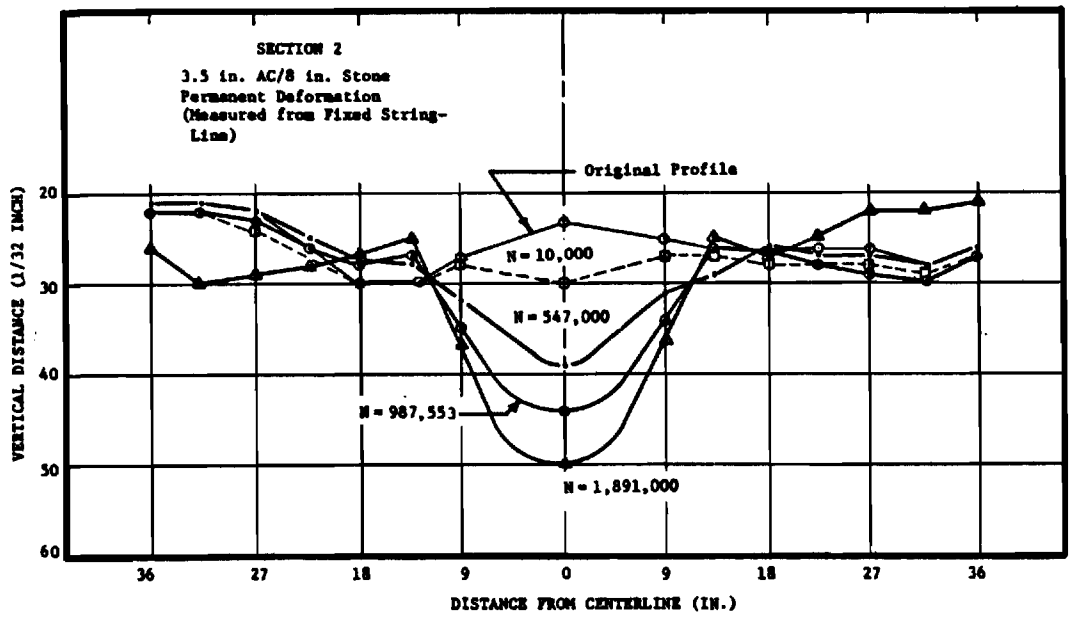


FIGURE 49. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 2.

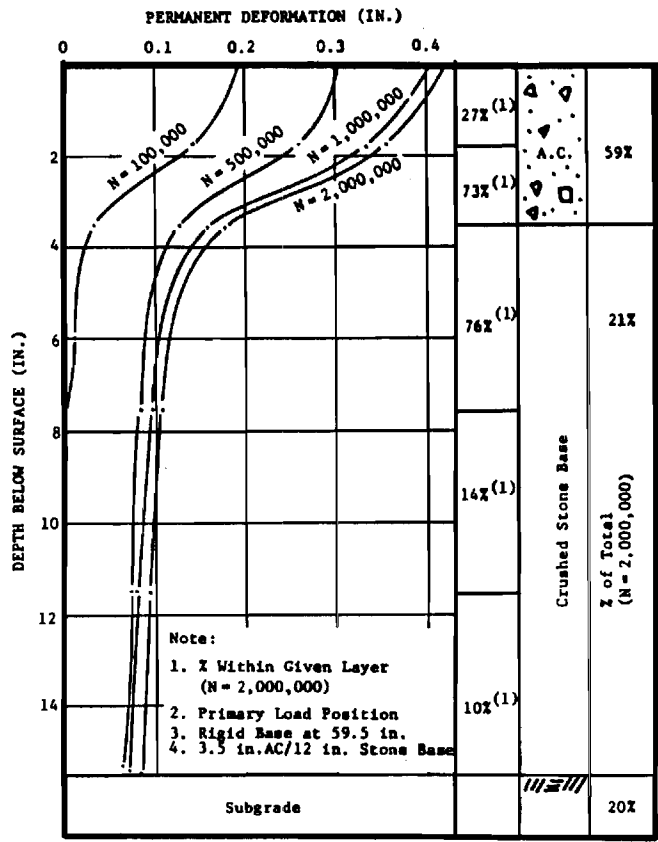


FIGURE 50. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 1.

TABLE 18. DISTRIBUTION OF PERMANENT DEFORMATION IN SECTIONS 1 AND 2 AS A FUNCTION OF LOAD REPETITION .

Section	N (x 10 ⁶)	Surface			Base				Subgrade	
		Top(1)	Bottom(1)	Total(2)	Top(1)	Middle(1)	Bottom(1)	Total(2)	Top(1)	Total(2)
1	0.1	25	75	82	78	6	16	18	heave	heave
	0.5	25	75	58	66	18	16	21	46	21
	1.0	26	74	61	71	16	13	22	13	16
	2.0	28	72	59	76	13	11	21	18	20
2	0.1	35	65	28	81	8	11	31	66	41
	0.5	34	66	28	59	24	17	20	58	53
	1.0	34	66	32	59	17	24	23	62	45
	1.9	35	65	34	59	17	24	27	67	39

- Notes: 1. Percent of total permanent deformation occurring in the *layer*.
 2. Percent of total permanent deformation occurring in the pavement structure.

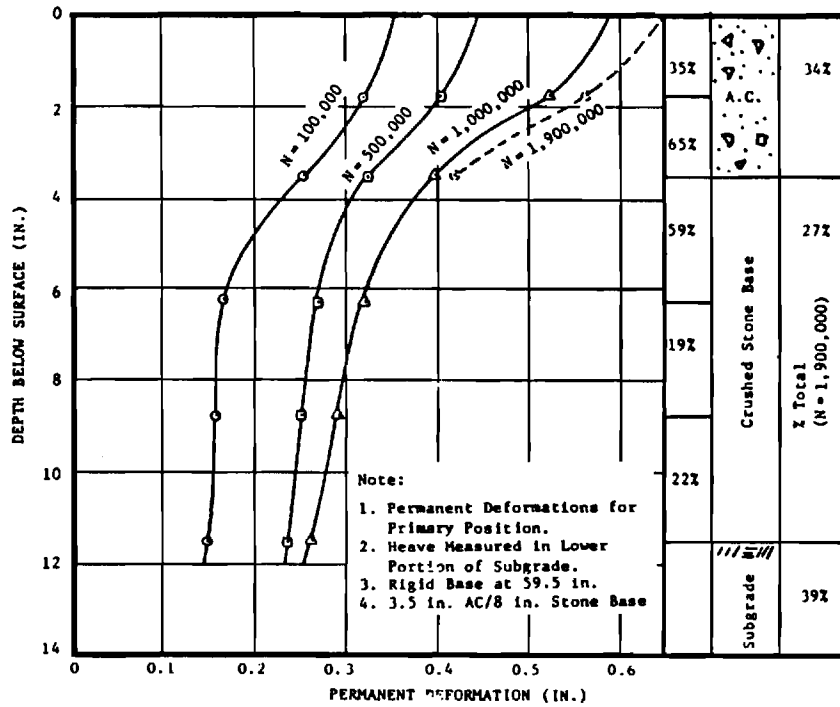


FIGURE 51. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 2.

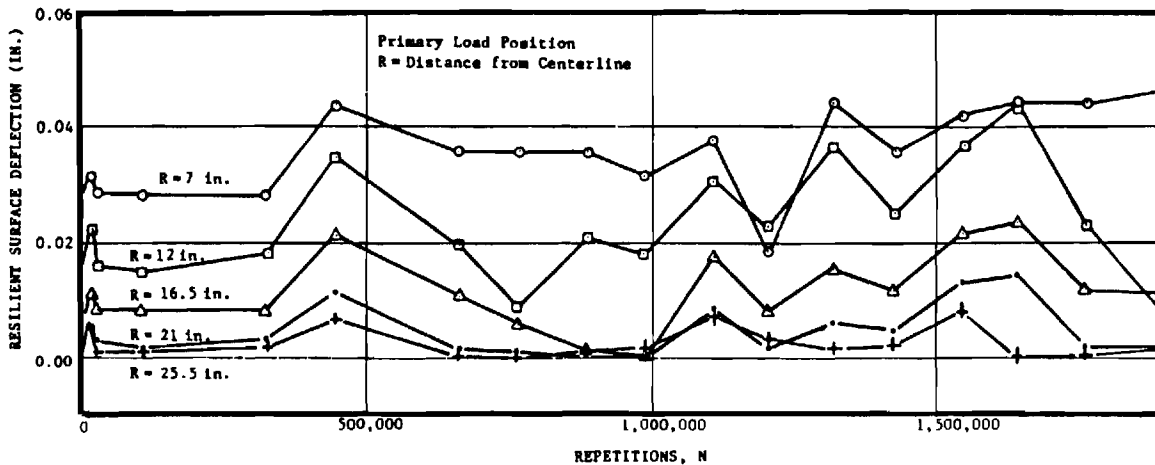


FIGURE 52. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 1 .

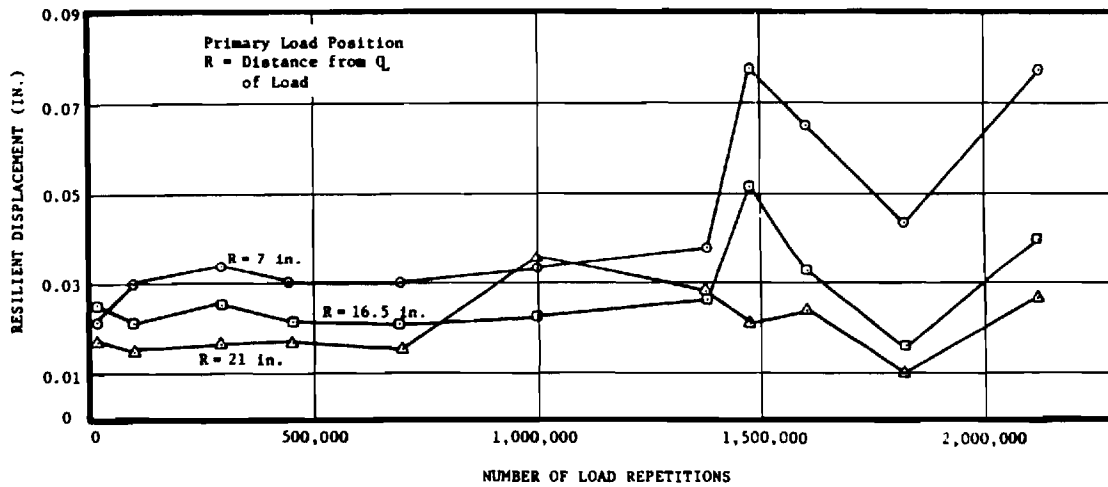


FIGURE 53. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 2 .

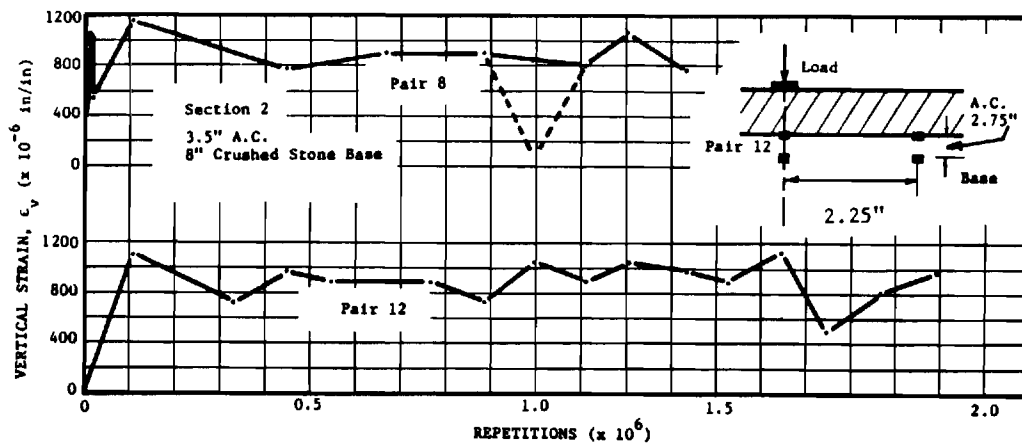


FIGURE 54. DYNAMIC VERTICAL STRAIN ON SUBGRADE AS A FUNCTION OF LOAD REPETITIONS - SECTION 2

displacement on top of the base with load repetitions is given in Fig. 54, and horizontal resilient displacements within the base in Fig. 55.

Sections 3 and 4

Sections 3 and 4 were full-depth sections consisting of 9.0 and 6.5 in. (229 and 165 mm), respectively of B-modified asphalt concrete. Both of these sections underwent 1 in. (25 mm) of rutting after only 10,000 load repetitions. Because of the premature failure, extensive response information is not presented. The results of tests performed on asphalt concrete cores taken from these sections were previously summarized in Table 6. Although a density of 99% of the maximum Marshall value was obtained, the asphalt content of these sections was about 5.9% compared with a mix design value of 5.2%. Although the gradation was slightly out of the allowable range on the 1 in. (25 mm) sieve by 3.1%, and on the No. 8 screen by 4%, gradation is not considered to be a significant factor in rutting of this mix. The early failure of these two sections is attributed primarily to high asphalt content.

Sections 5 and 6

Sections 5 and 6 also consisted of full-depth, modified-B asphalt concrete binder having thicknesses of 9.0 (229 mm) and 6.5 in. (165 mm), respectively. The asphalt concrete was obtained from the Lithia Springs plant of APAC-Georgia, Inc. As just discussed, Sections 3 and 4 failed in rutting after only 10,000 load repetitions. Upon removal of the asphalt concrete from these sections, the subgrade was found to be in excellent condition with essentially all of the rutting having occurred in the asphalt concrete. Therefore, only the upper 8 in. (203 mm) of the subgrade was removed and replaced before construction of Sections 5 and 6.

Rutting. Section 5 underwent a rutting failure after 130,000 load repetitions, and Section 6 after 580,000 load repetitions. The variation of rut depth beneath the center of the primary load position for Section 5 is shown in Fig. 56 and for Section 6 in Fig. 57. Figs. 58 and 59 show the deflection profile at selected numbers of load repetitions. Figure 60 shows pictorially the condition of the two sections after 1 million repetitions.

Surface cracking was not observed in Section 5, which consisted of 9.0 in. (229 mm) of asphalt concrete. Section 6, the thinner 6.5 in. (165 mm) thick full-depth section, did at the termination of the test have 5 small hairline cracks as shown in Fig. 61. The rut depth at this time was 0.75 in. (19 mm). The primary failure mode of this section was, however, clearly a rutting failure.

The average asphalt content obtained from six cores taken from these sections was 5.3% (one core had a reported asphalt content of 7.14% which was not included). The average density of the seven cores was 99% of the Marshall maximum density. The relatively early failure of these

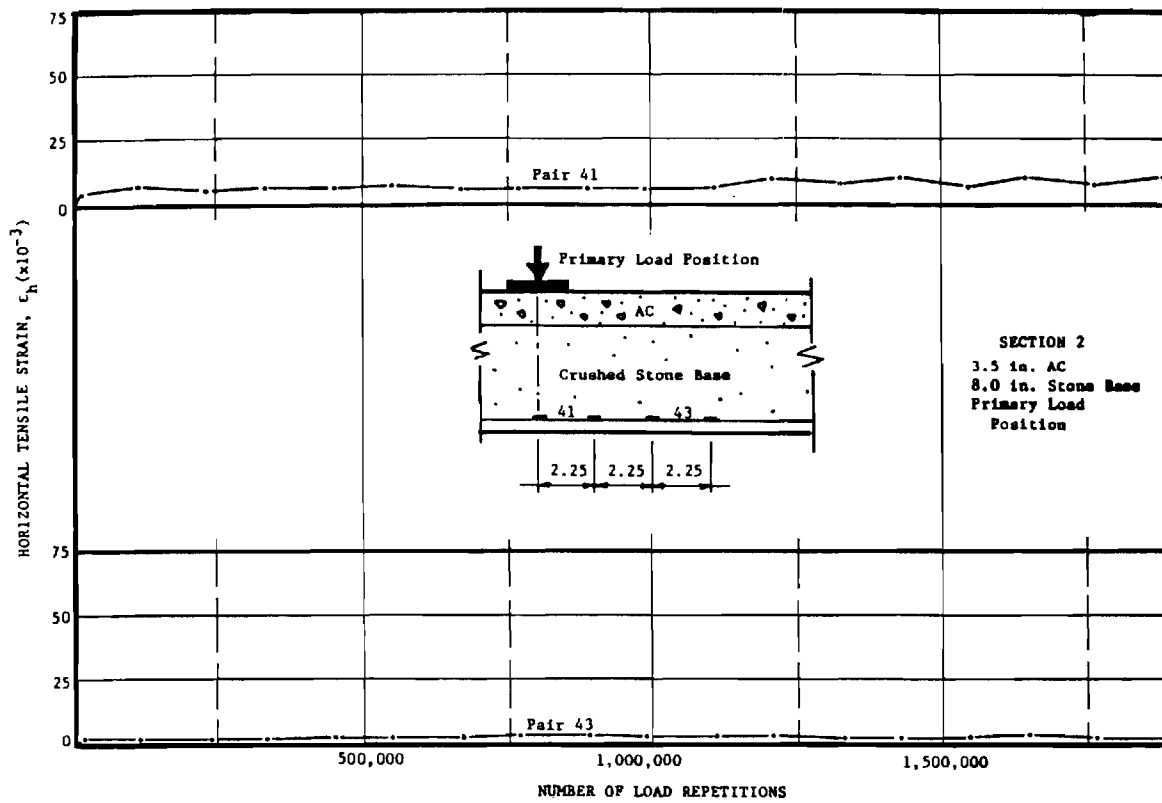


FIGURE 55. VARIATION OF STATIC HORIZONTAL STRAIN IN BOTTOM OF CRUSHED STONE BASE WITH NUMBER OF LOAD REPETITIONS - SECTION 2.

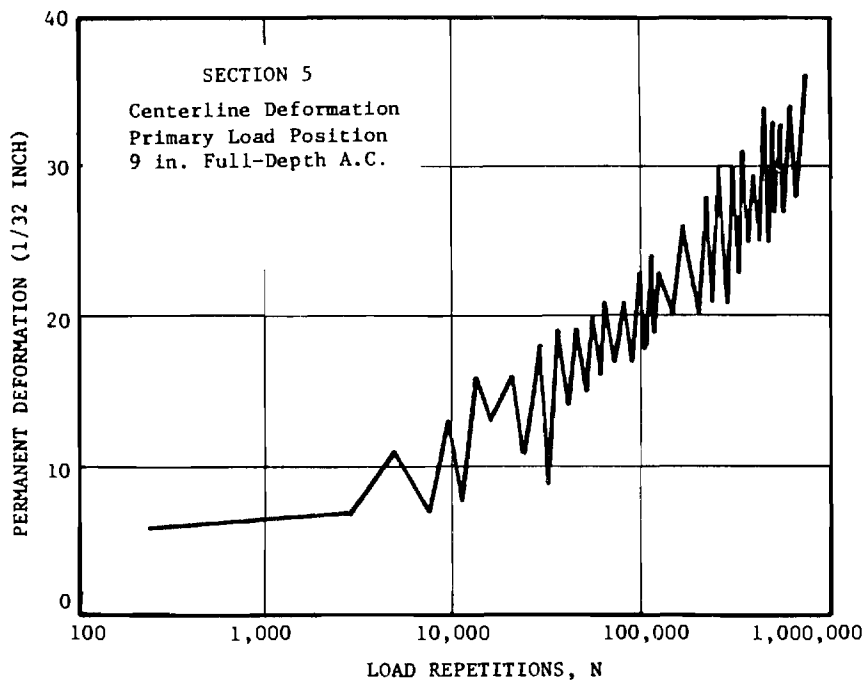


FIGURE 56. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 5

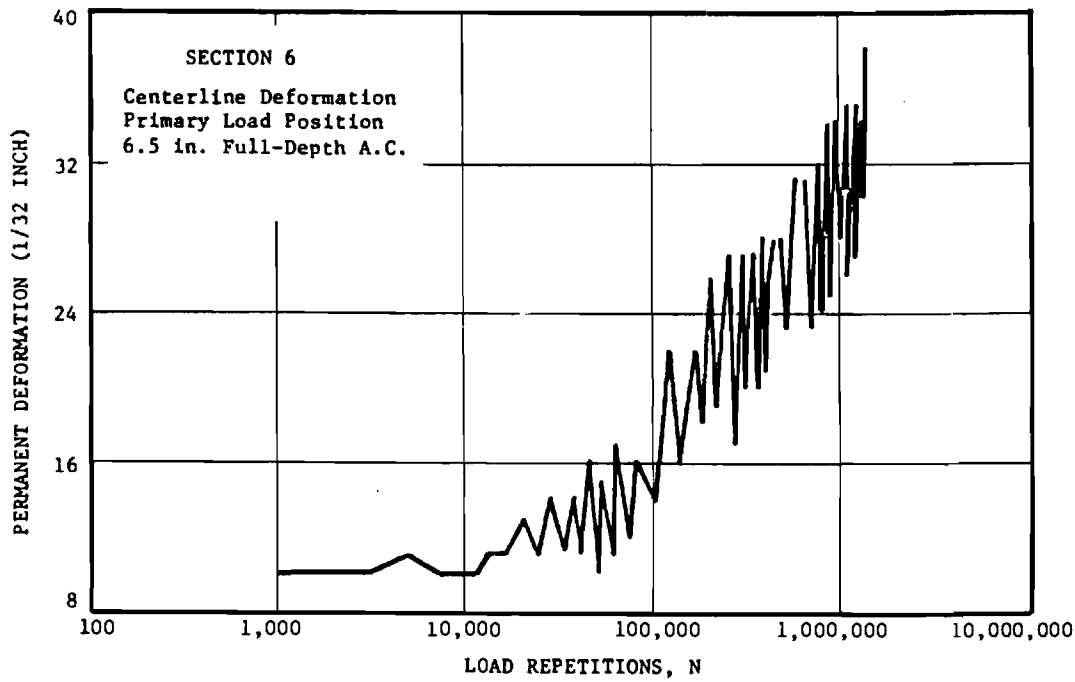


FIGURE 57. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 6

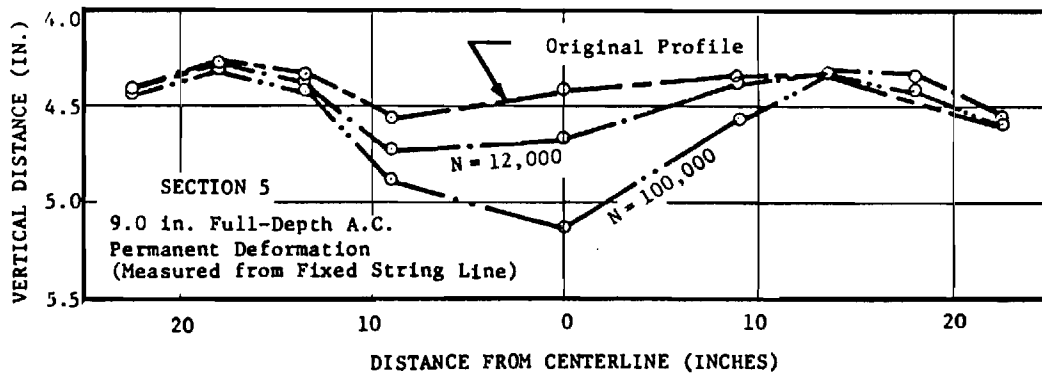


FIGURE 58. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 5.

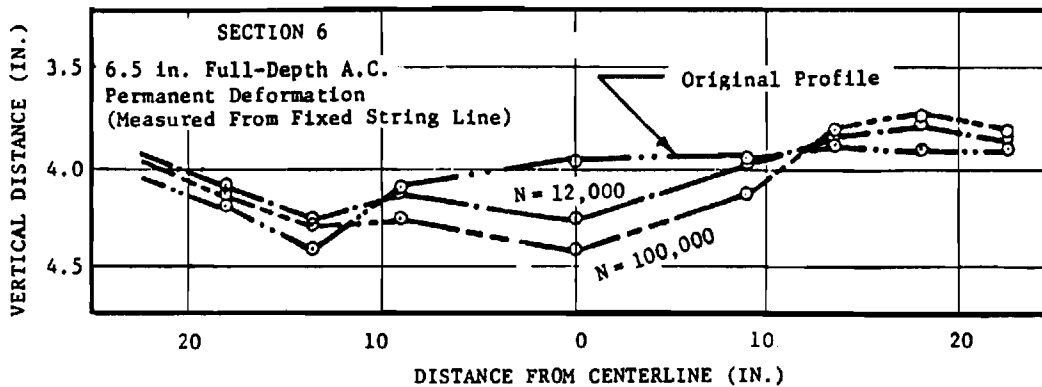
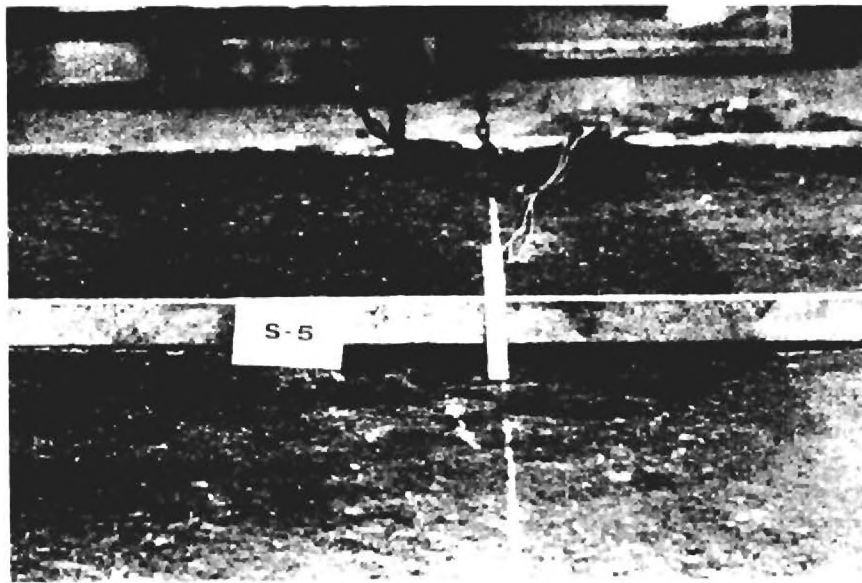
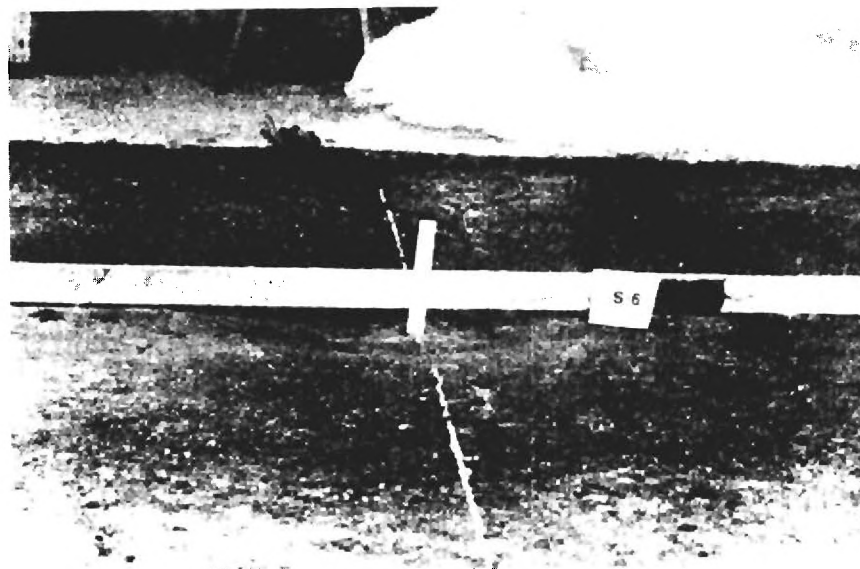


FIGURE 59. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 6.



(a) Section 5: 9 in. Full-Depth A.C.



(b) Section 6: 6.5 in. Full-Depth A.C.

FIGURE 60. SURFACE PROFILE OF SECTION 5 AND 6 AFTER 1.3 MILLION REPETITIONS.

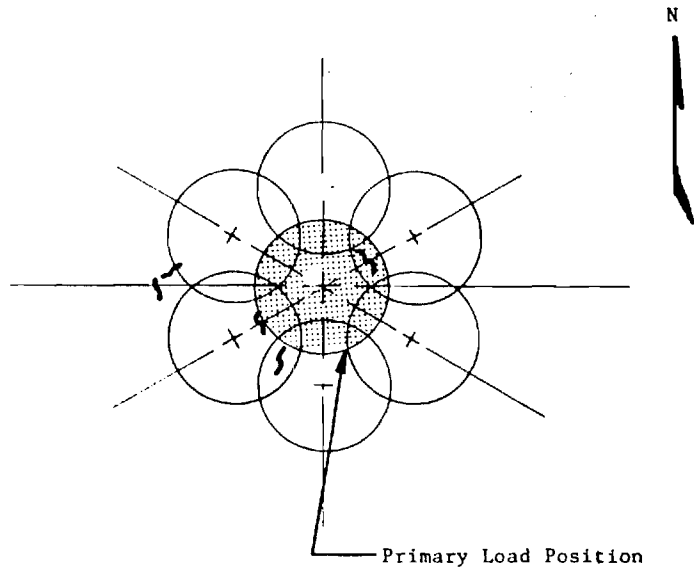


FIGURE 61. CRACK PATTERN IN SECTION 6 AT COMPLETION OF TEST:
6.5 IN. FULL-DEPTH A.C. SECTION

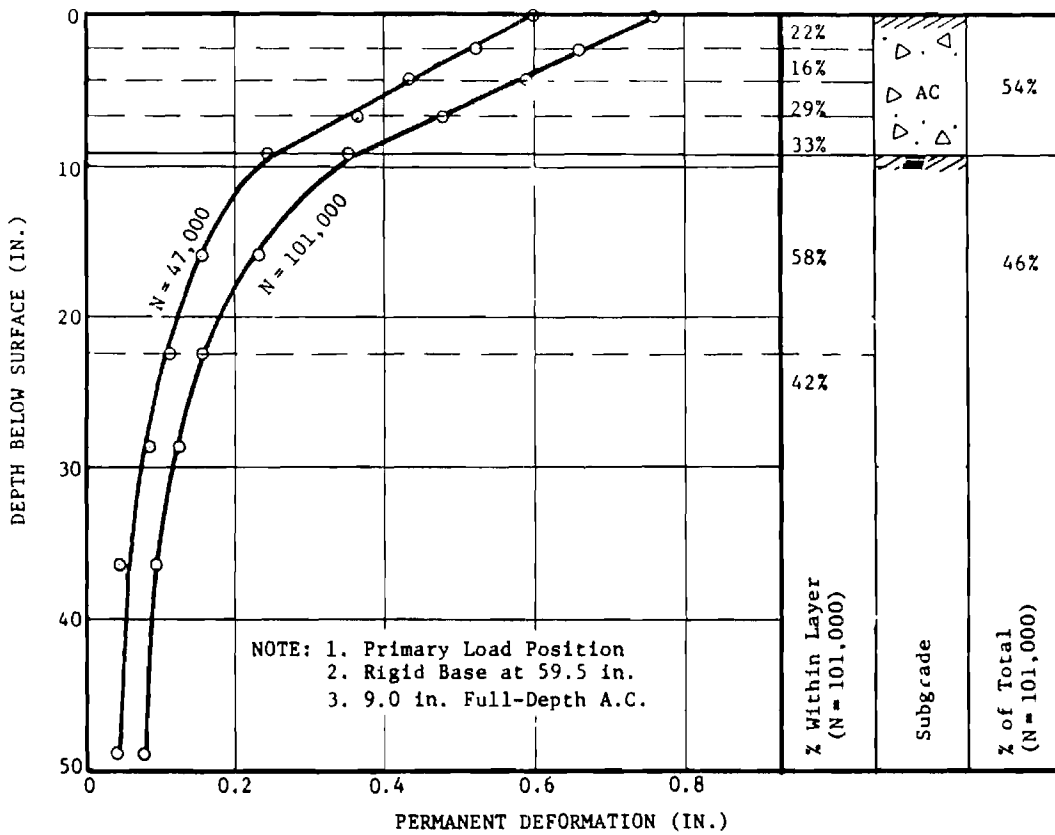


FIGURE 62. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 5.

test sections in rutting appears to be due to (1) application of the heavy loading in a reasonably concentrated pattern, (2) a slightly high asphalt content, and (3) the relatively thin asphalt concrete layer resulted in important contributions of rutting from the somewhat weak subgrade.

Distribution of Permanent Deformation. The distribution of permanent deformation for Section 5 is shown in Fig. 62 and for Section 6 in Fig. 63. In both sections permanent deflection in the asphalt concrete decreased from the surface downward almost linearly with depth. For the relatively thin asphalt concrete layers used in these sections, permanent deformation in the subgrade accounted for about one-half of the total rutting. Further, most of the subgrade rutting occurred in the upper 12 in. (305 mm). Fig. 63 shows the distribution of permanent deformation for load position 5 and not the primary load position. Load position 5 is shown because sufficient information was not available to give results for position 1 because of problems with several strain sensors.

Resilient Response. The resilient response of Sections 5 and 6 is summarized in Table 17. For Section 5 the measured horizontal tensile strain in the bottom of the asphalt concrete was about 319×10^{-6} in./in. and the vertical strain at the top of the subgrade was 1.38×10^{-3} in./in. The variation with number of load repetitions of resilient horizontal tensile strain in the bottom of the asphalt concrete and vertical strain on top of the subgrade is shown in Fig. 64. These results suggest a gradual increase in resilient horizontal tensile strain in the bottom of the asphalt concrete until rut depths of about 0.75 to 1.0 in. (19-25 mm). After this the tensile strain appeared to decrease perhaps due to micro-cracking in the asphalt concrete. Vertical strain on the top of the subgrade increased with number of load repetitions to approximately failure as also shown in Fig. 64. The resilient vertical pressure measured on top of the subgrade was 8.7 psi (60.0 kN/m²). This relatively high magnitude of vertical stress appears realistic since deformation in the subgrade was large.

For Section 6 the resilient horizontal tensile strain in the bottom of the asphalt concrete varied from 460×10^{-6} in./in. at 1,000 repetitions to 633×10^{-6} at 126,000 repetitions. The resilient vertical strain on top of the subgrade was 1.41×10^{-3} in./in., and the dynamic vertical pressure on top of the subgrade was 10.9 psi (75.1 kN/m²).

The variation of resilient surface displacement for Sections 5 and 6 is given as a function of number of load repetitions in Figs. 65 and 66, respectively. The initial resilient deflection just outside the edge of the loaded area was 0.0096 in. (0.24 mm) for Section 5 and 0.014 in. (0.36 mm) for Section 6. These resilient deflections tended to increase at least up to failure showing a general weakening of the pavement.

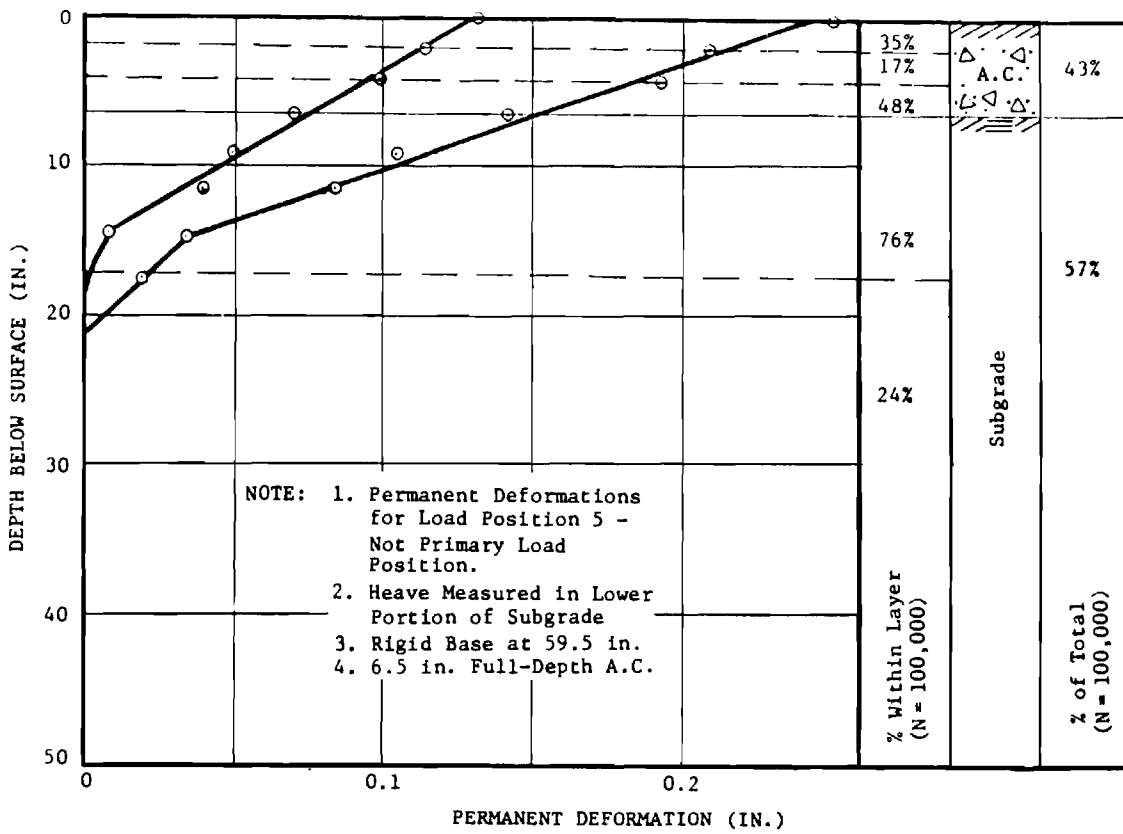


FIGURE 63. DISTRIBUTION OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTION 6.

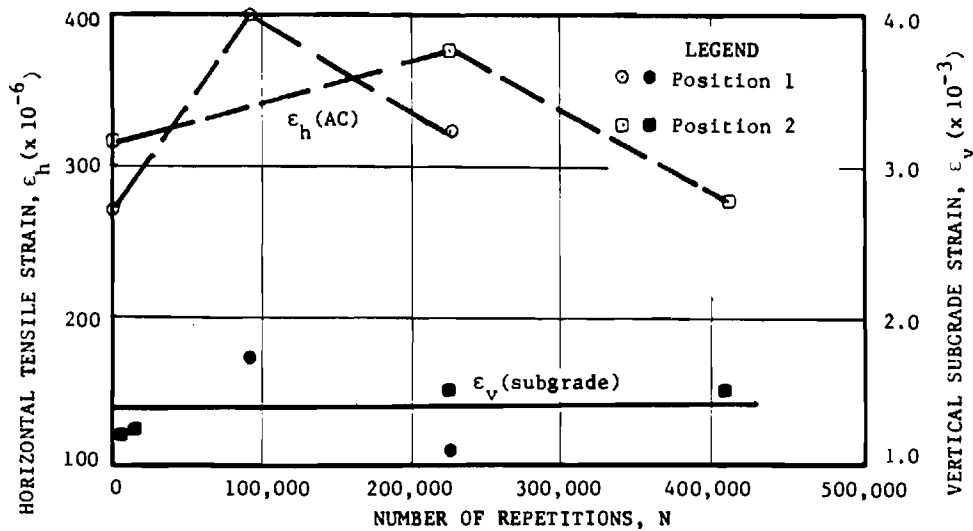


FIGURE 64. VARIATION OF RESILIENT HORIZONTAL STRAIN IN BOTTOM OF A.C. AND VERTICAL STRAIN IN TOP OF SUBGRADE WITH NUMBER OF LOAD REPETITIONS - SECTION 5 .

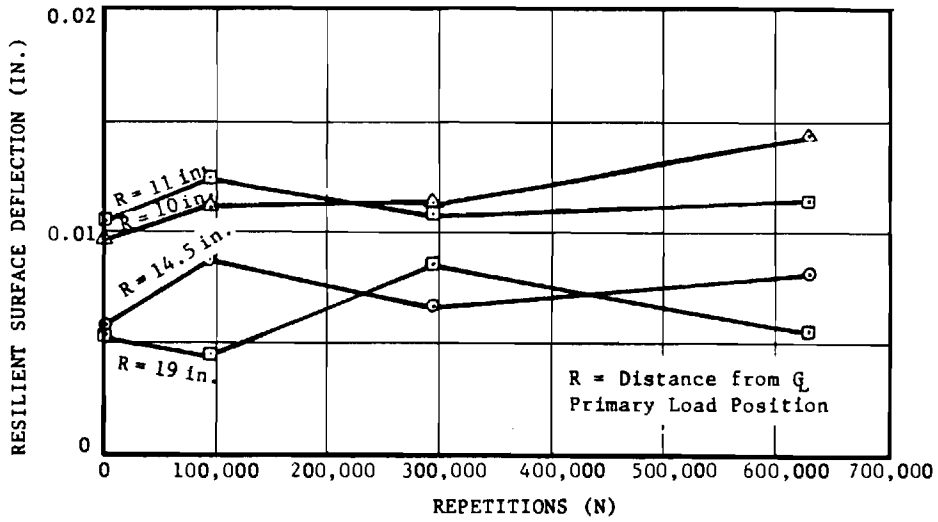


FIGURE 65. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 5 .

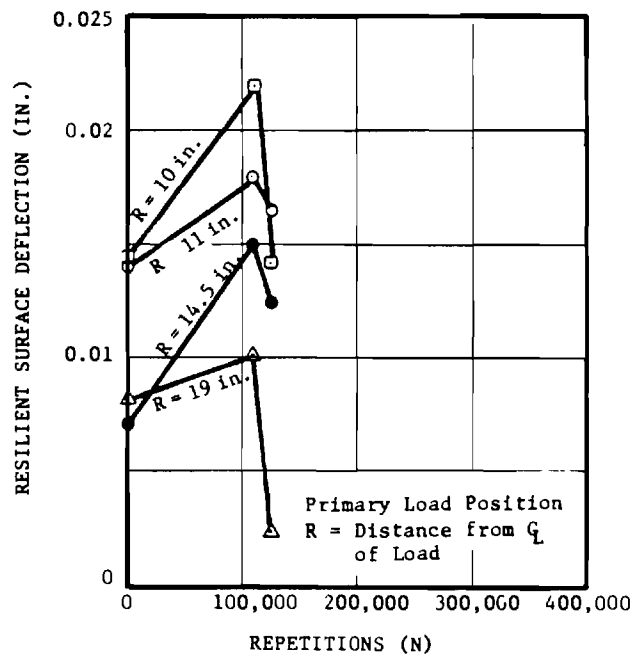


FIGURE 66. VARIATION OF RESILIENT SURFACE DEFLECTION WITH LOAD REPETITIONS - SECTION 6 .

Sections 7 and 8

Sections 7 and 8 were constructed to directly compare the performance of a full-depth asphalt concrete pavement (Section 7) with a crushed stone base pavement (Section 8). Section 7 consisted of 7.0 in. (178 mm) of full-depth asphalt concrete. Section 8 consisted of 3.5 in. (89 mm) of asphalt concrete overlying an 8 in. (203 mm) crushed stone base. The 1.5 in. (38 mm) maximum size crushed stone base had the standard gradation consisting of 83% passing the 3/4 in. (19 mm) sieve, 48% passing the No. 4 sieve, 16% passing the No. 40 sieve, and 4.8% passing the No. 200 sieve.

Sections 7 and 8 were constructed at the same time (i.e., during one filling of the pit) to minimize the influence of differences in the subgrade and asphalt concrete used in the sections. The modified-B binder used in these sections was obtained from the Lithia Springs plant of APAC-Georgia, Inc. The asphalt concrete used to cover the crushed stone base in Section 8 was from the same batch and was placed at the same time as the asphalt concrete in the upper two lifts of the full-depth section. Therefore, in these two sections the material properties of the subgrade and the upper two lifts of asphalt concrete for purposes of analysis should have essentially the same physical properties.

Rutting. The full-depth asphalt concrete pavement (Section 7) failed in rutting after 150,000 load repetitions with a rut depth of 0.5 in. (12 mm) having developed. The crushed stone pavement (Section 8) failed in rutting after 550,000 load repetitions. In Section 8, 2.3 in. (58 mm) of crushed stone replaced 1 in. (25 mm) of asphalt concrete base from Section 7. Since the crushed stone section withstood 3.7 times more load repetitions, the base course equivalency for similar performance observed in these experiments was considerably less than 2.3.

The variation of rut depth beneath the center of the primary load position for Section 7 is shown in Fig. 67 and for Section 8 in Fig. 68. Rut depth accumulation at a distance of 9 in. (229 mm) from the center of the primary load position is given in Figs. 69 and 70 for these two sections. Figs. 71 and 72 show the deflection profile at selected numbers of load repetitions. Fig. 73 pictorially compares the surface condition of the two sections after 1 million repetitions. Surface cracking was not observed in either Section 7 or Section 8.

The average asphalt content obtained from five cores taken from these sections was 5.1%. Three cores had asphalt contents reported to vary from 3 to 3.6%. Since the cores tested were only about 1.5 in. (38 mm) thick, the low asphalt contents were assumed to be erroneous and not included in the average. The possibility does exist that the asphalt content in Section 7 may have been slightly higher than Section 8. Any significant difference in asphalt content between sections, however, is not considered probable because all the asphalt came from the same batch (for two lifts) and from the same plant and job for the other two lifts in the full-depth section. The more probable explanation is due to experimental error in the recovery tests performed on small specimens. The average density of the five cores was 144.9 pcf (22.7 kN/m³) which was 99% of the maximum Marshall density.

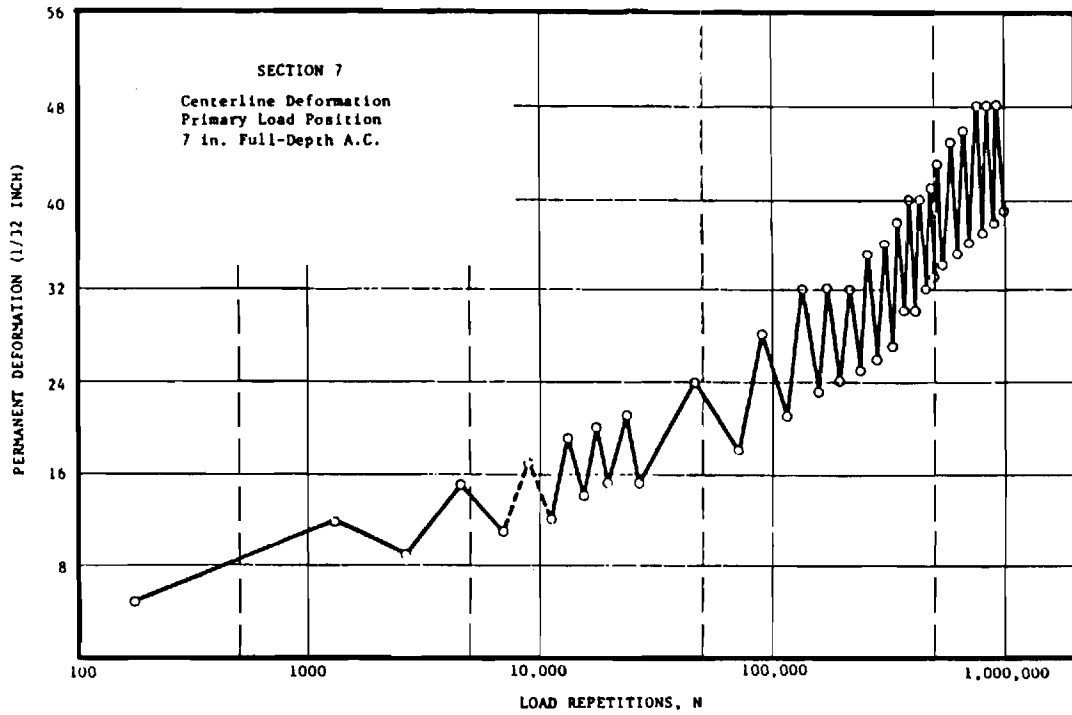


FIGURE 67. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 7 .

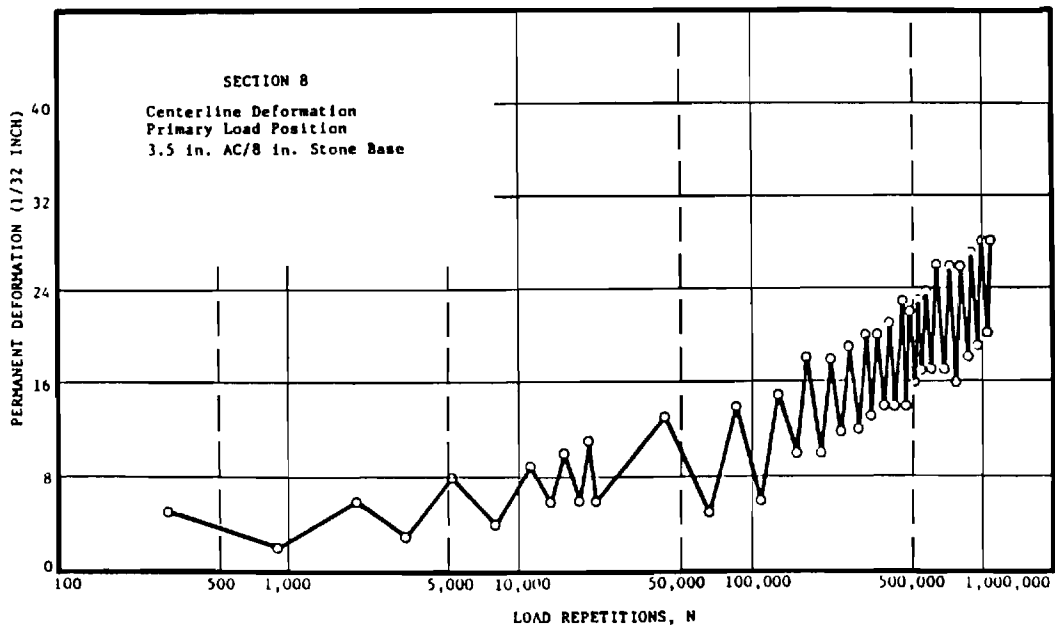


FIGURE 68. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 8

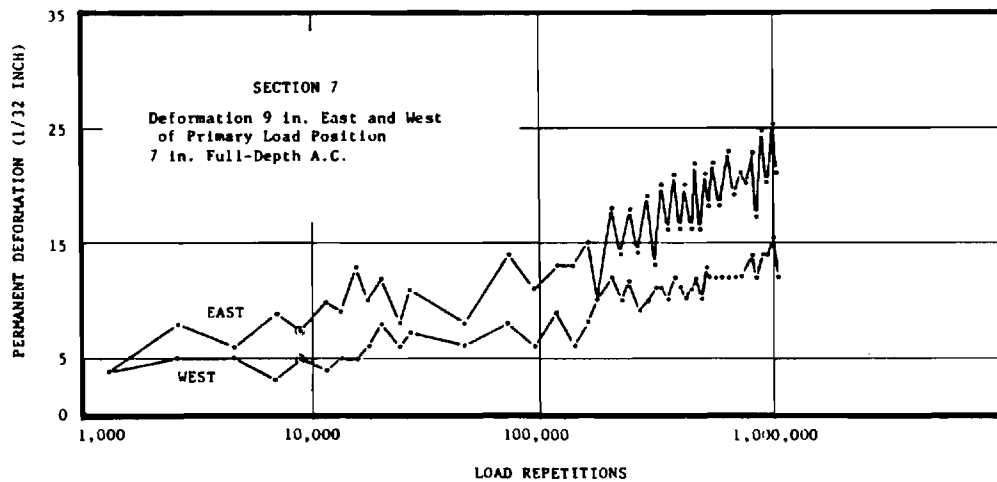


FIGURE 69. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 7

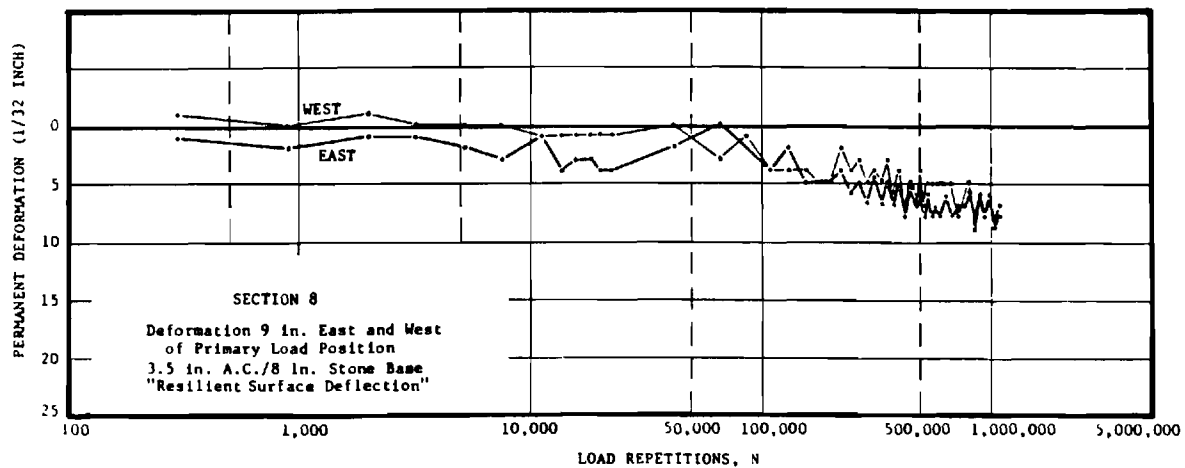


FIGURE 70. VARIATION OF DEFORMATION 9 IN. FROM CENTERLINE WITH NUMBER OF LOAD REPETITIONS - SECTION 8 .

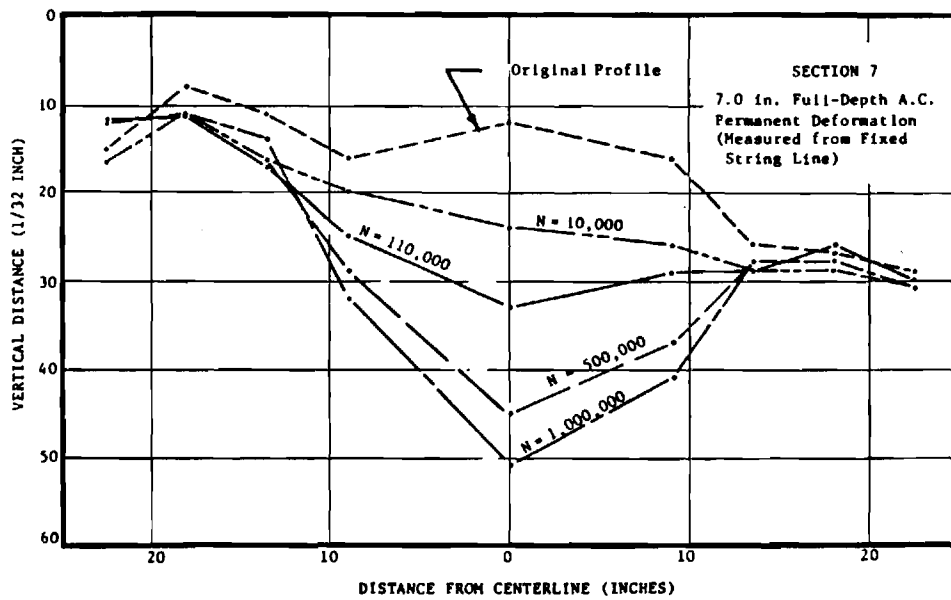


FIGURE 71. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 7.

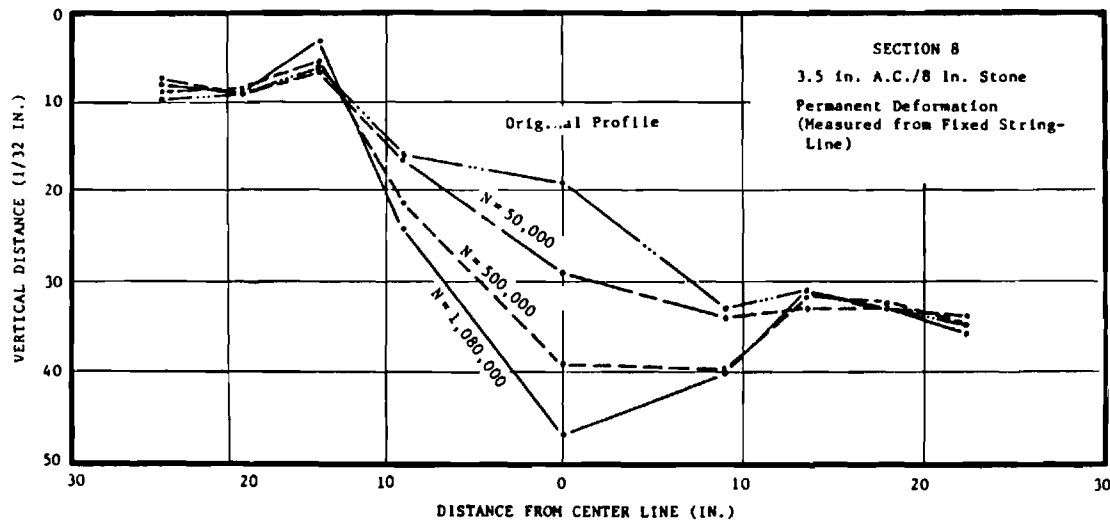
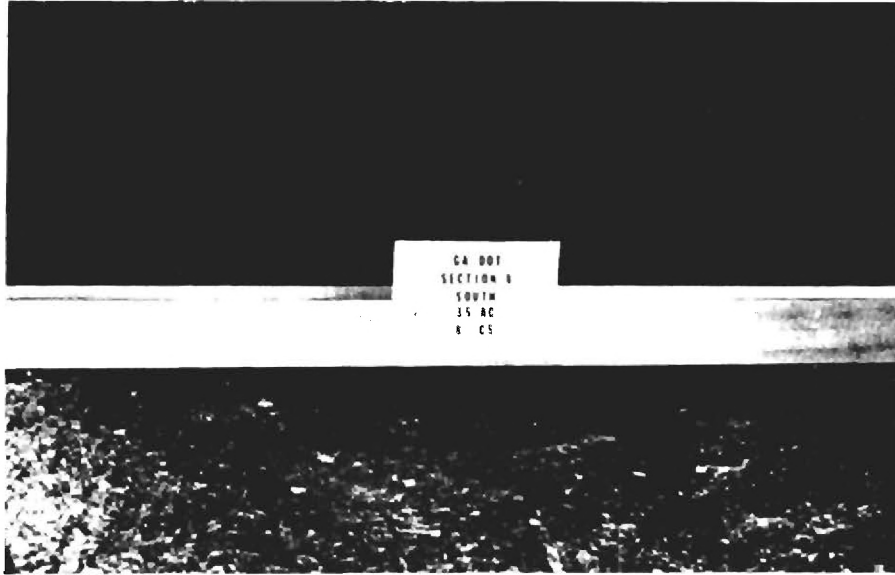
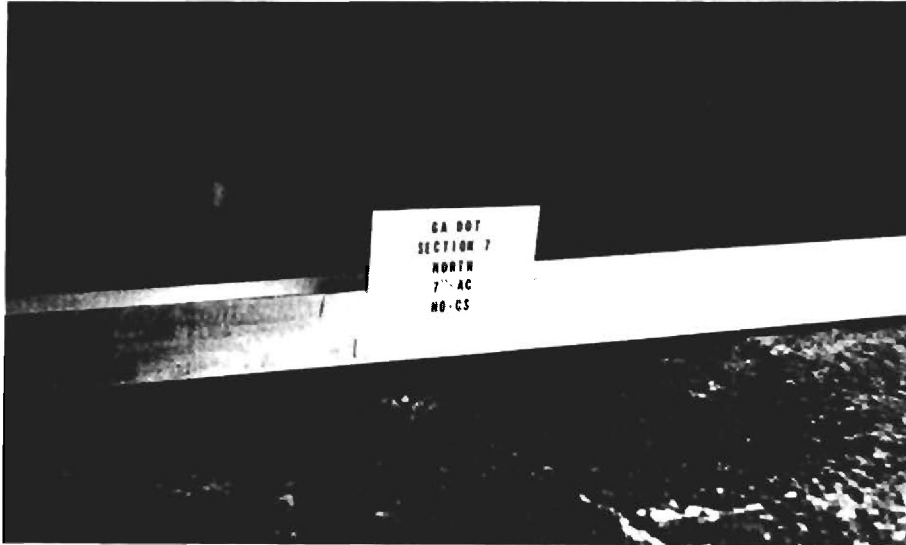


FIGURE 72. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 8.



(a) Section 7: 7 in. Full-Depth A.C. (Note: Failure was at 150,000 Repetitions).



(b) Section 8: 3.5 in./8 in. Stone (Note: Failure was at 550,000 Repetitions).

FIGURE 73. SURFACE DEFLECTION PROFILE AFTER 1 MILLION REPETITIONS.

The relatively early failure of the full-depth section in rutting compared with the crushed stone base section appears to be due to (1) application of the heavy loading in a reasonably concentrated pattern, (2) important contributions of rutting from the weak subgrade beneath the relatively thin, full-depth asphalt concrete (the subgrade was located closer to the surface than the crushed stone section and apparently as a result had a larger contribution to rutting), and (3) the full-depth section *may* have had a slightly higher asphalt content than the crushed stone section.

Distribution of Permanent Deformation. The distribution of permanent deformation for Sections 7 and 8 is compared in Fig. 74 after approximately 300,000 load repetitions. Once again, in both sections permanent deformation in the asphalt concrete decreased almost linearly with depth. In the full-depth asphalt concrete section 67% of the total permanent deformation occurred in the asphalt concrete and 33% in the subgrade. In the crushed stone section, 55% of the permanent deformation occurred in the 3.5 in. (89 mm) thick asphalt concrete surfacing, 10% in the crushed stone and 35% in the subgrade. In the asphalt concrete layers of both sections the permanent deformation was approximately equally distributed between the top and bottom half of the asphalt concrete layer. Also of considerable interest is the finding that at equal depths more permanent rutting occurred in the upper part of the subgrade beneath the full-depth asphalt concrete section than in the crushed stone section (refer to Fig. 74).

Resilient Response. A general summary of the resilient response of all test sections is given in Table 17. The response of Section 7 is directly compared with Section 8 in Table 19 for selected numbers of load repetitions. The full-depth pavement (Section 7) had for approximately the first 100,000 load repetitions a typical measured horizontal tensile strain in the bottom of the asphalt concrete of about 410×10^{-6} in./in. and a vertical strain at the top of the subgrade of about 2.2×10^{-3} in./in. In comparison, in the crushed stone base section the measured horizontal tensile strain in the bottom of the asphalt concrete was about 300×10^{-6} in./in. and the vertical strain at the top of the subgrade was about 1.85×10^{-3} in./in. The resilient vertical pressure measured at the top of the subgrade was about 12.9 psi (88.9 kN/m²) for Section 7 and 11.9 psi (82.0 kN/m²) for Section 8.

For Section 7 the variation as a function of number of load repetitions of the resilient horizontal tensile strain in the asphalt concrete at selected locations is shown in Fig. 75; variations of vertical strain in the top of the subgrade and in the asphalt concrete are shown in Fig. 76. Similar relationships for Section 8 for strain are presented in Figs. 77 through 79. Once again, the resilient strains observed in Sections 7 and 8 indicate a gradual increase with load repetitions in resilient horizontal and vertical strain in the bottom of the asphalt concrete at least until about failure.

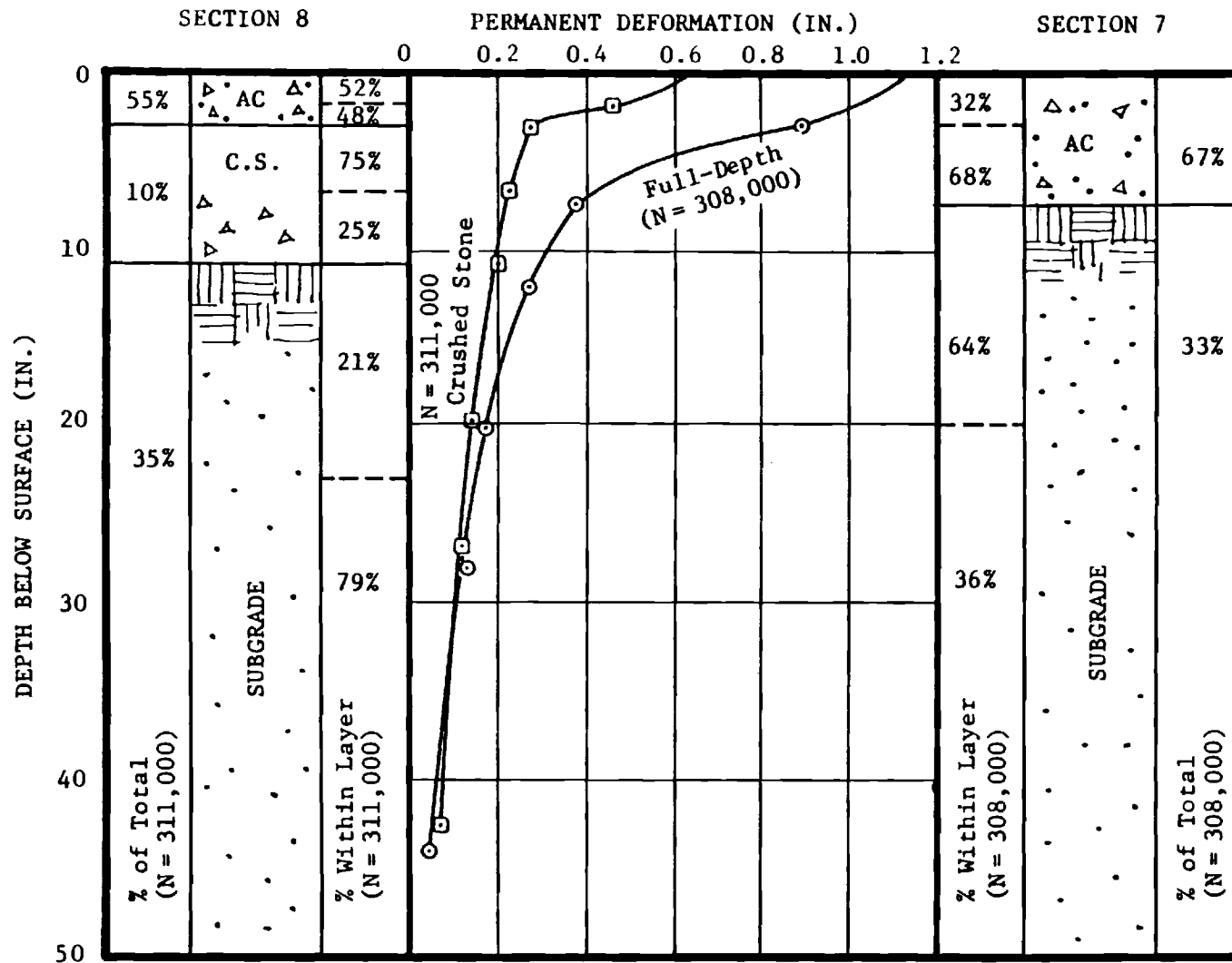


FIGURE 74. COMPARISON OF DISTRIBUTION OF PERMANENT DEFORMATION IN FULL-DEPTH SECTION 7 AND CRUSHED STONE SECTION 8.

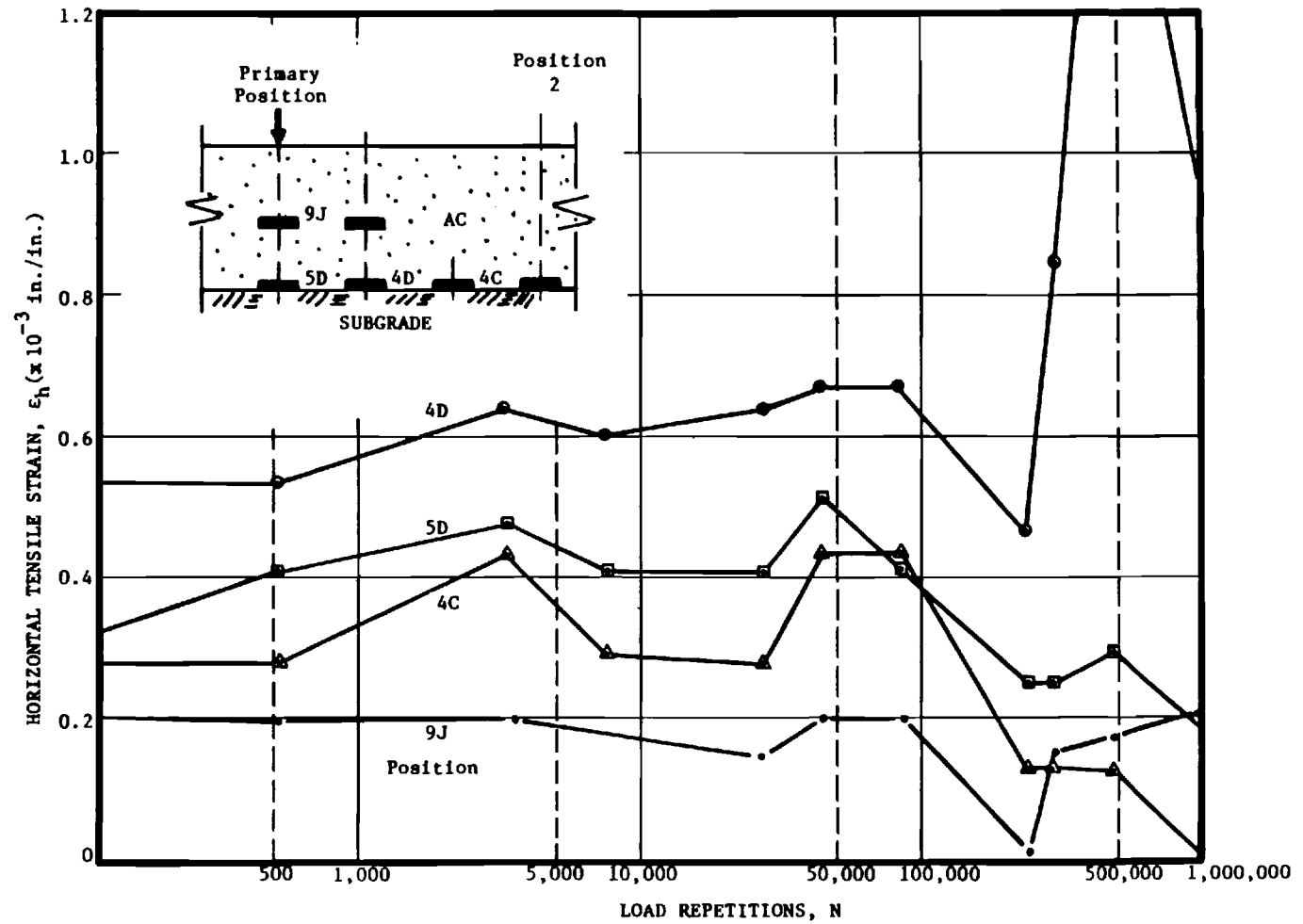


FIGURE 75. VARIATION OF RESILIENT HORIZONTAL STRAIN WITH LOAD REPETITIONS WITHIN THE ASPHALT CONCRETE - SECTION 7.

TABLE 19. SUMMARY COMPARISON BETWEEN FULL-DEPTH AC SECTION 7 (7AC) AND CRUSHED STONE SECTION 8 (3.5 AC/8CS)(1).

Repetition	Section Number	Resilient δ_v (2) (in.)	Avg. Rut Depth(5) (in.)	Vert. (3) Subgrade Strain (in./in. x 10 ⁻³)	Horiz. Tensile(4) Strain in AC (in./in. x 10 ⁻⁶)	Vert. Stress @ Top Subgrade
1,000	7(7AC) 8(3.5/8)	0.0126 0.0112	0.30 0.12	1.90 1.73	420 270	12.2 11.7
10,000	7(7AC) 8(3.5/8)	0.0126 0.0166	0.45 0.21	2.08 1.77	410 270	13.6 12.1
110,000	7(7AC) 8(3.5/8)	0.0158 0.0148	0.78 0.33	2.43 1.94	372 295	15.2 12.6
500,000	7(7AC) 8(3.5/8)	0.0204 0.0154	1.16 0.59	2.79 2.37	268 330	16.1 12.9

- Note:
1. Sections 7 and 8 were constructed at the same time using the same asphalt concrete.
 2. Resilient vertical surface deflection at load position 1 located 11 in. (279 mm) from the center of the load (6.45 in., 164 mm from the edge of the load).
 3. Vertical strain in top of subgrade immediately beneath loading.
 4. Horizontal tensile strain in bottom of AC immediately beneath loading.
 5. Average measured rut depth beneath the primary load position; not corrected for rutting occurring before 1,000 repetitions.

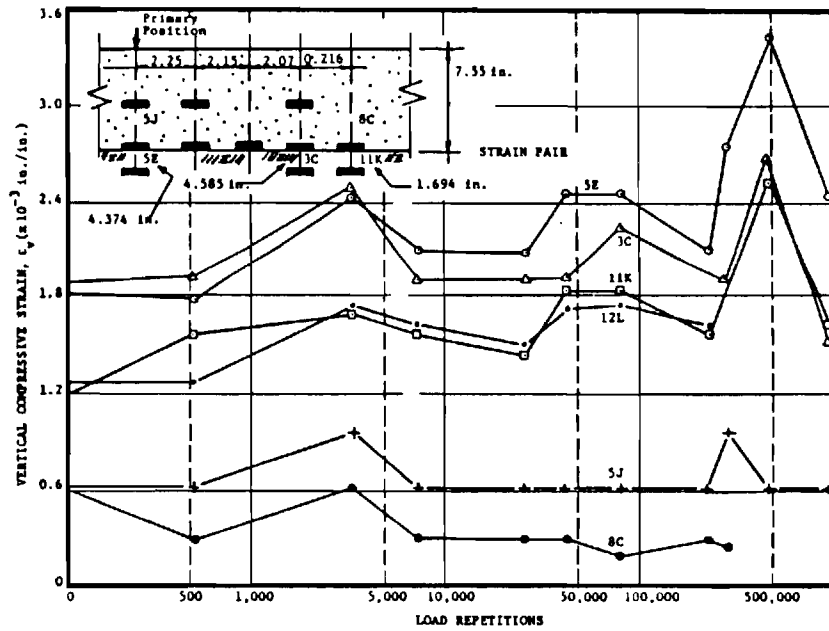


FIGURE 76. VARIATION OF VERTICAL RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 7 .

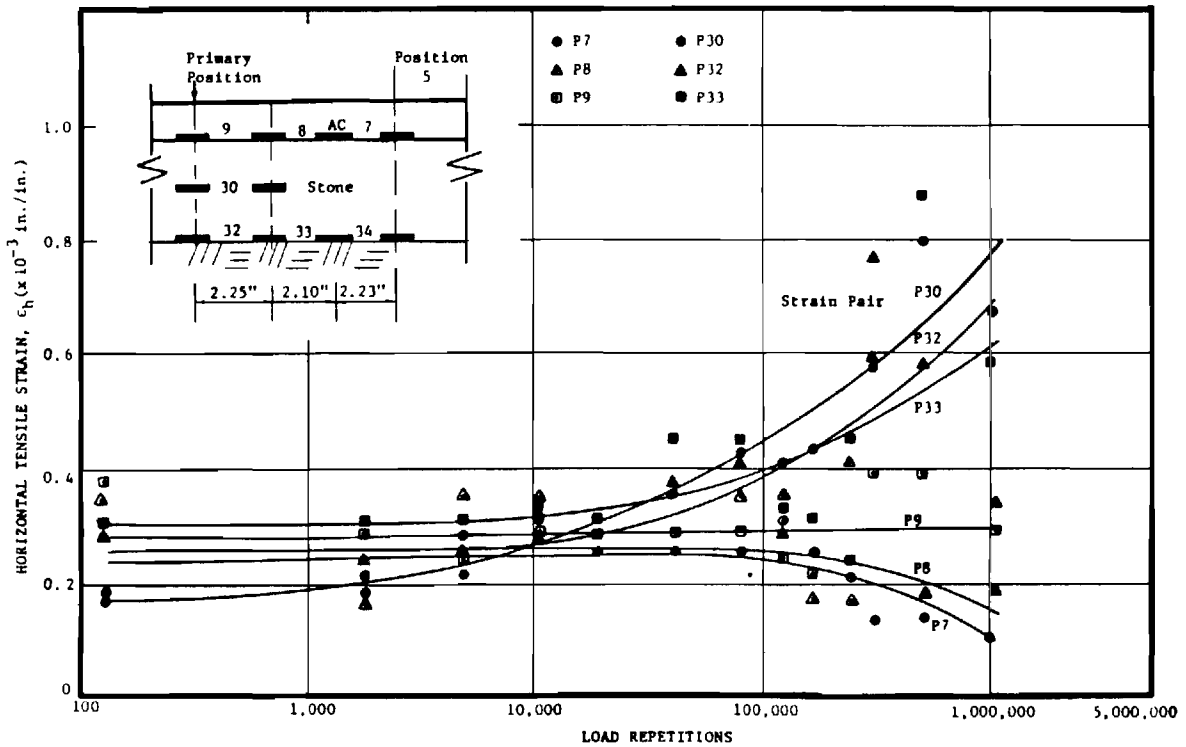


FIGURE 80. VARIATION OF RESILIENT HORIZONTAL STRAIN WITH LOAD REPETITIONS - SECTION 8

FIGURE 77. VARIATION OF RESILIENT HORIZONTAL STRAIN WITH LOAD REPETITIONS - SECTION 8.

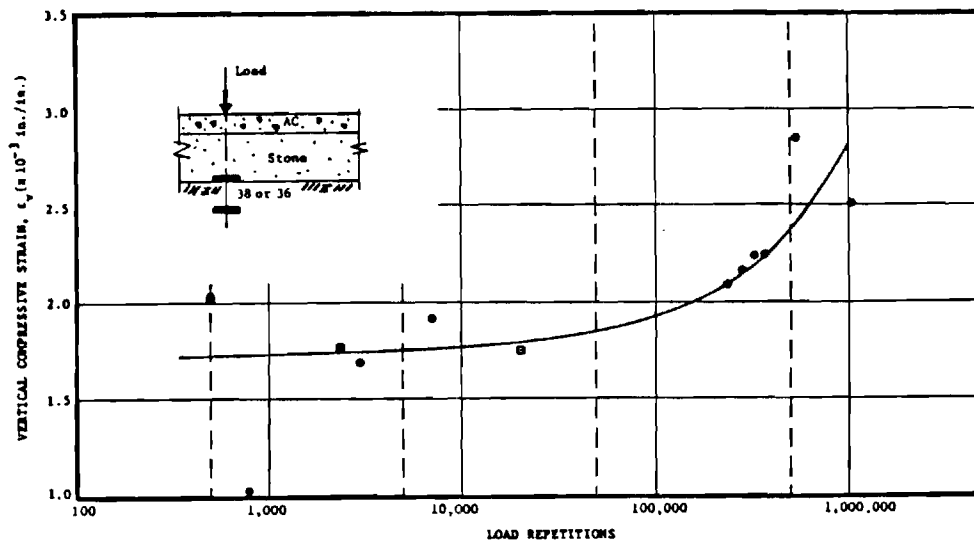


FIGURE 78. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD - SECTION 8 .

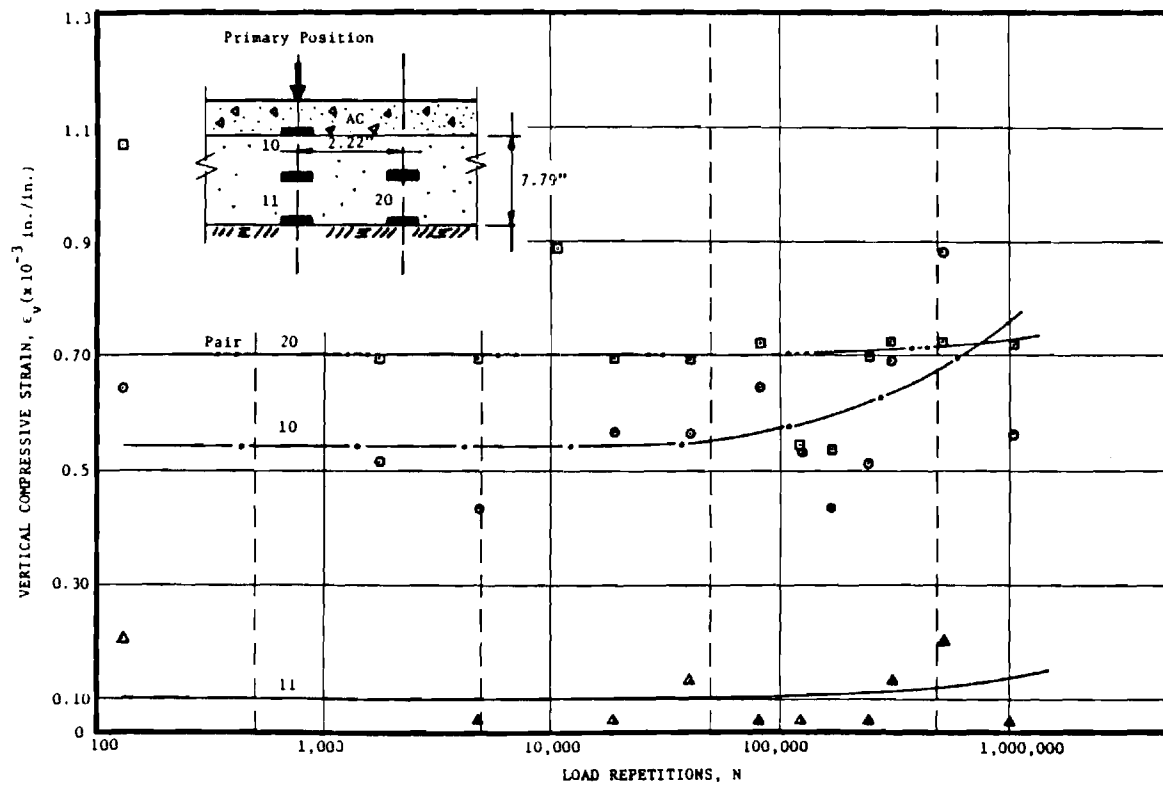


FIGURE 79. VARIATION OF RESILIENT VERTICAL STRAIN WITHIN CRUSHED STONE WITH NUMBER OF LOAD REPETITIONS - SECTION 8 .

Sections 9 and 10

Sections 9 and 10 were constructed to compare use of a coarser and finer gradation crushed stone base than the standard gradation used in Sections 1, 2 and 8. Sections 9 and 10, which were identical except for the gradation of the crushed stone base, consisted of 3.5 in. (89 mm) of asphalt concrete B-binder and 8 in. (203 mm) of crushed stone base. The asphalt concrete B-binder was obtained from the Forest Park plant of APAC-Georgia, Inc.

Section 9 was the coarse gradation crushed stone base pavement having a 2 in. (51 mm) top size, 83% passing the 1 in. (25 mm) sieve, 60% passing the 3/4 in. (19 mm) sieve, 40% passing the No. 4, 15% passing the No. 40 sieve and 4.2% passing the No. 200 sieve. The finer gradation, 1.5 in. (38 mm) crushed stone used in Section 10 had 99% passing the 1 in. (25 mm) sieve, 92% passing the 3/4 in. (19 mm) sieve, 44% passing the No. 4 sieve, 15% passing the No. 40 and 4.4% passing the No. 200 sieve. Therefore, the difference in these two crushed stone gradations is only between the 2 in. (51 mm) and No. 4 sieve size; below the No. 4 sieve the gradation is essentially the same.

At 2 million repetitions the loading applied to the pavement was increased from 6,500 lbs. (28.9 kN) to 7,500 lbs. (33.4 kN) for the remainder of the test. An oil spill occurred on Section 10 at 546,400 load repetitions that covered load positions 3 through 6 for a short period of time. Apparently the oil spill did not significantly effect the life of this section.

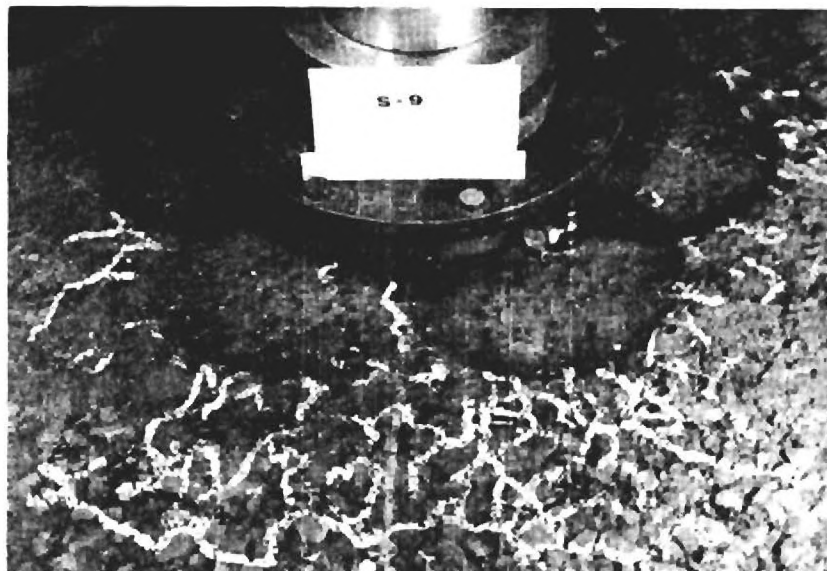
Fatigue. Both Sections 9 and 10 failed by fatigue cracking of the asphalt concrete B-binder. Section 9, which had the coarse gradation base, failed in fatigue after approximately 2.4 million load repetitions. Cracking began in this section after approximately 2 million load repetitions and rapidly propagated leading to the initiation of Class 2 cracking considered as a failure condition in this study.

Section 10, which had the fine gradation crushed stone base, failed in fatigue (initiation of Class 2 cracking) after approximately 2.9 million load repetitions. The cracking patterns present in Sections 9 and 10 after 2.9 million repetitions are compared in Fig. 80. Almost all of the cracking occurred outside of the loaded area. In both sections only hairline cracks developed which were hard to see. Therefore to illustrate the pattern, the cracks were painted white to be visible in the pictures.

The measured resilient horizontal tensile strain in the bottom of the asphalt concrete beneath the primary load in Section 9 was 270×10^{-6} in./in. and in Section 10 was 390×10^{-6} in./in. Nevertheless, Section 10 which had the higher resilient tensile strain performed better with respect to fatigue than Section 9.



(a) Section 10: 3.5 in. A.C./8.0 in. Stone (Fine Gradation)



(b) Section 9: 3.5 in. A.C./8.0 in. Stone (Coarse Gradation)

FIGURE 80. FATIGUE CRACKING IN SECTIONS 9 AND 10 AFTER TWO MILLION LOAD REPETITIONS.

Rutting. The coarse crushed stone base pavement (Section 9) rutted 0.28 in. (7.2 mm) after 2.8 million load repetitions. The fine gradation crushed stone base pavement (Section 10) rutted 0.34 in. (8.7 mm) after 2.8 million load repetitions. Therefore, the section having the larger top size, coarser base stone underwent somewhat less rutting. The variation with load repetitions of rut depth beneath the center of the primary load position for Section 9 is shown in Fig. 81 and for Section 10 in Fig. 82. Figs. 83 and 84 show the deflection profile of these sections at selected numbers of load repetitions.

Distribution of Permanent Deformation. The distribution of permanent deformation in Sections 9 and 10 is compared in Fig. 85 after 3 million load repetitions. Permanent deflection through the asphalt concrete for both sections decreased from the surface downward. For these sections, in contrast to the previous sections, the variation of permanent deflection through the asphalt concrete was not linear. Although surface rutting was greater in Section 10 which had the finer gradation base, the relative distribution of rutting in the two sections was almost identical (i.e., the shape of the pavement deformation profiles shown in Fig. 85 were similar). In both sections about 20% of the total permanent deformation occurred in the asphalt concrete, 12% in the crushed stone base and 68% in the subgrade. The surfacing and base of these sections were, based on the large number of repetitions required to fail the sections, high quality and hence did not undergo large amounts of permanent deformation. As a result a large relative amount of the total permanent deformation occurred in the subgrade which was not adequately protected by the 11.5 in. (292 mm) thick structural section.

In contrast to the full-depth pavement (Section 7), in Sections 9 and 10 about 70% of the permanent deformation occurred in the top of the asphalt concrete and only 30% in the bottom. More permanent deformation also occurred in the top portion of the crushed stone layer (64%) than in the bottom (36%). These findings suggest that the distribution of rutting is affected not only by the thickness of the layers but also by the quality of materials.

Resilient Response. A general summary of the resilient response of all test sections including 9 and 10 is given in Table 17. The typical resilient strain response for Section 9 as a function of load repetitions is shown in Figs. 86 through 88 and for Section 10 in Figs. 89 through 91.

The horizontal tensile strain measured in the bottom of the asphalt concrete was 280×10^{-6} in./in. in Section 9 which had the large top size crushed stone base and 400×10^{-6} in Section 10 which had the smaller top size crushed stone base. Similar tensile strains were determined from two different strain sensor pairs in each section. Therefore the measured horizontal tensile strains should indicate the true relative tensile strains that existed in these sections.

The horizontal tensile strain in the bottom of the crushed stone obtained from the strain sensor readings in Section 9 was 1.08×10^{-3} in./in., and the vertical compressive strain in the bottom of the stone

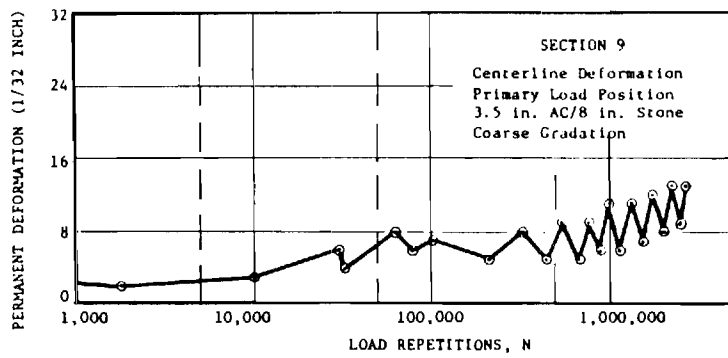


FIGURE 81. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 9 .

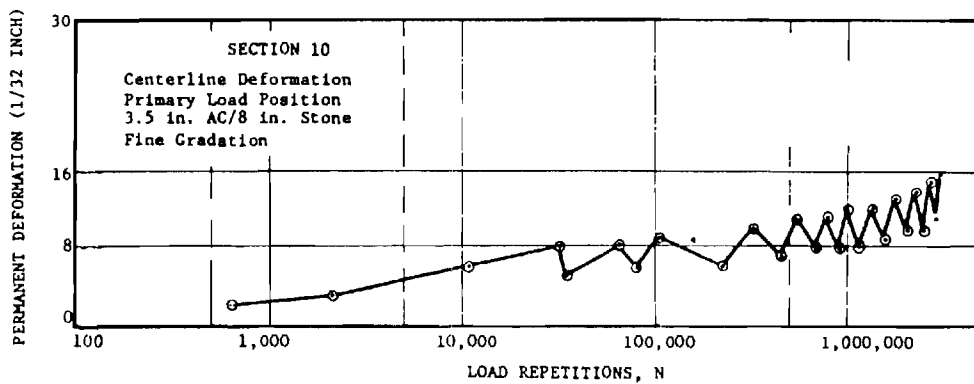


FIGURE 82. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 10.

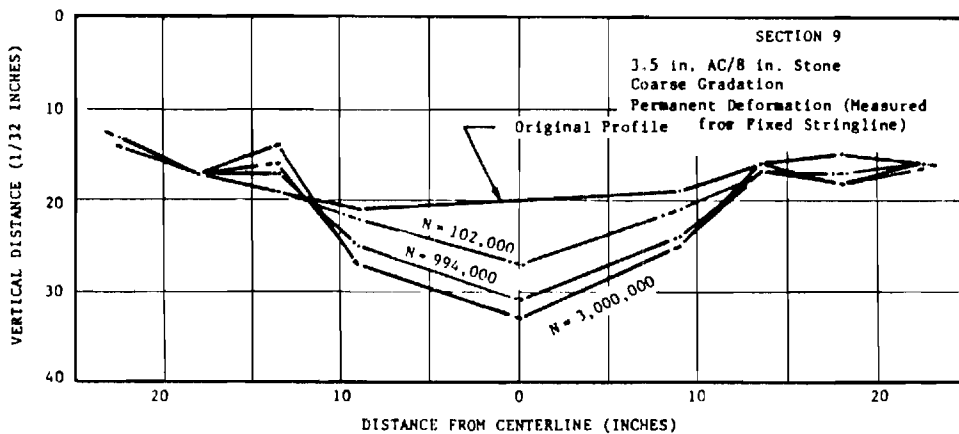


FIGURE 83. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 9 .

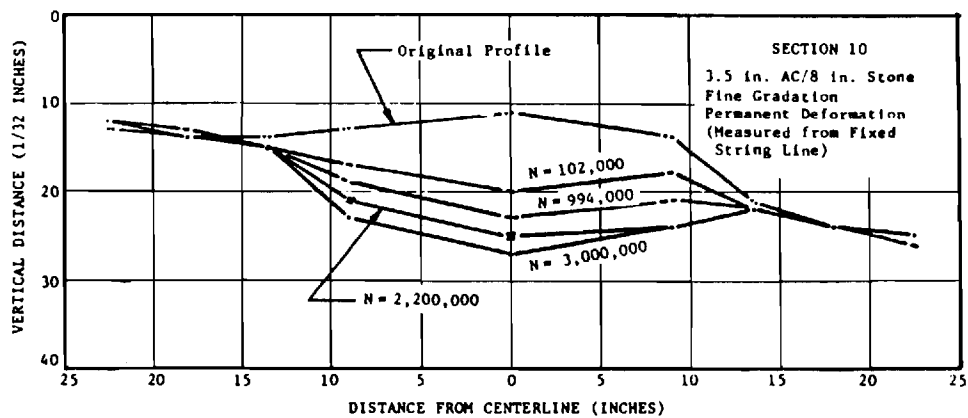


FIGURE 84. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 10 .

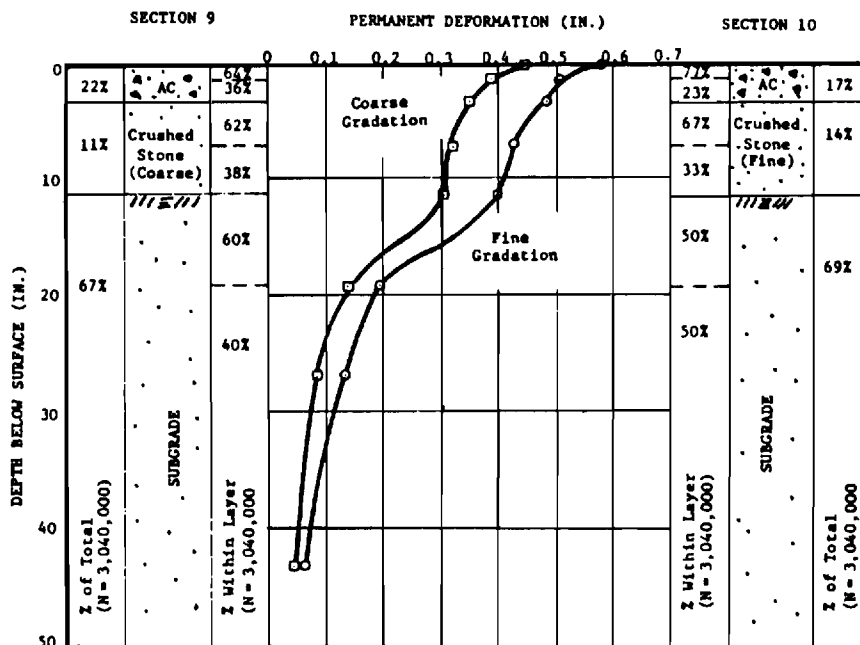


FIGURE 85. COMPARISON OF DISTRIBUTION OF PERMANENT DEFORMATION IN COARSE AND FINE GRADATION CRUSHED STONE BASE PAVEMENTS - SECTIONS 9 AND 10.

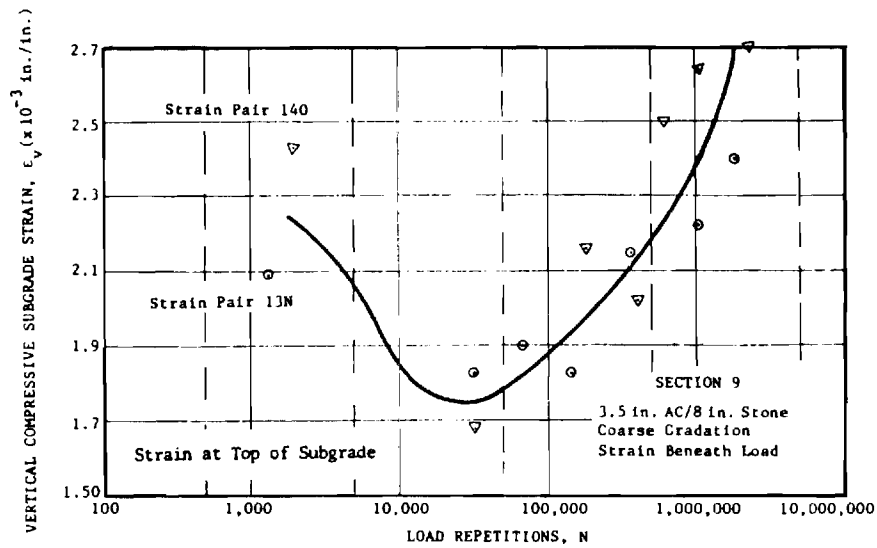


FIGURE 86. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 9 .

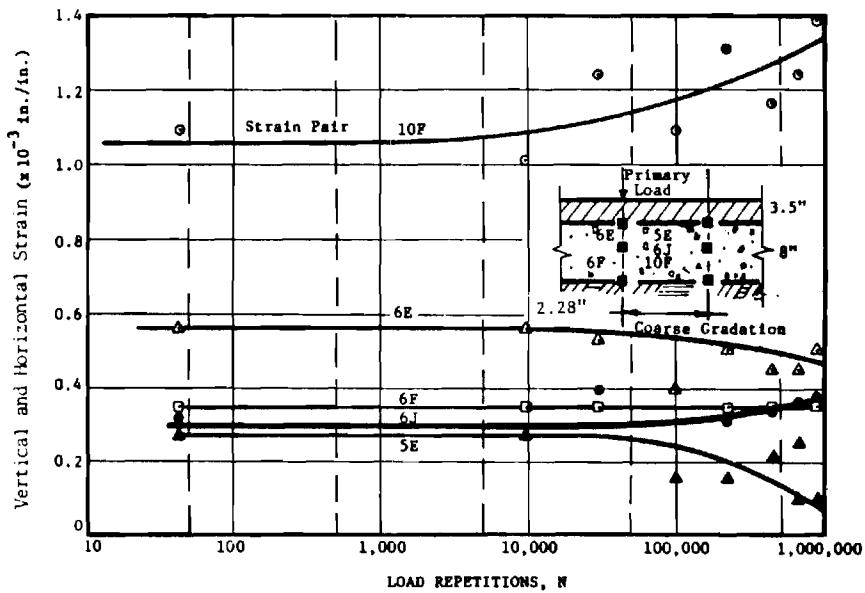


FIGURE 87. VARIATION OF RESILIENT HORIZONTAL AND VERTICAL STRAIN IN CRUSHED STONE BASE WITH LOAD REPETITIONS FOR LOAD IN PRIMARY POSITION - SECTION 9.

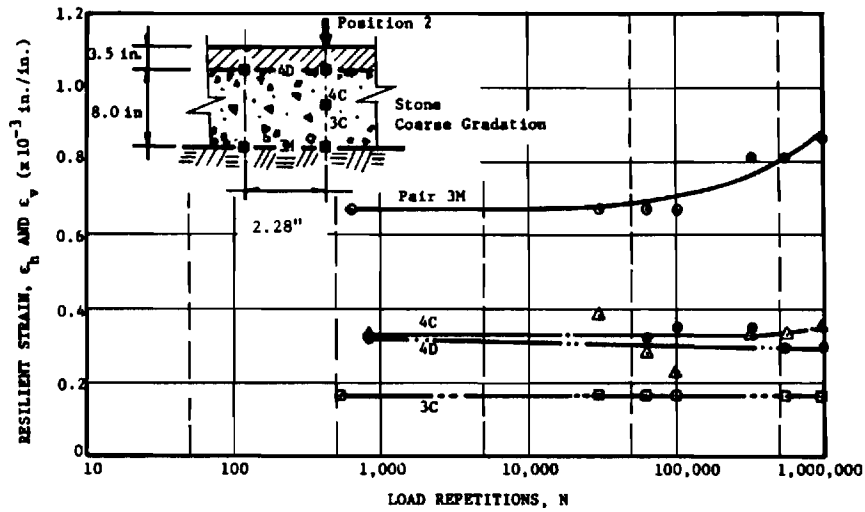


FIGURE 88. VARIATION OF RESILIENT HORIZONTAL AND VERTICAL STRAINS WITH LOAD REPETITIONS FOR LOADING AT POSITION 2 - SECTION 9 .

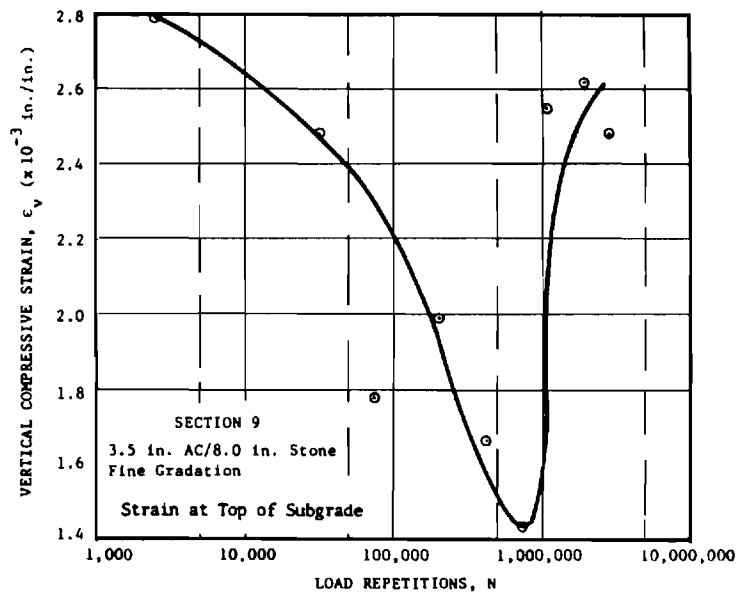


FIGURE 89. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 10.

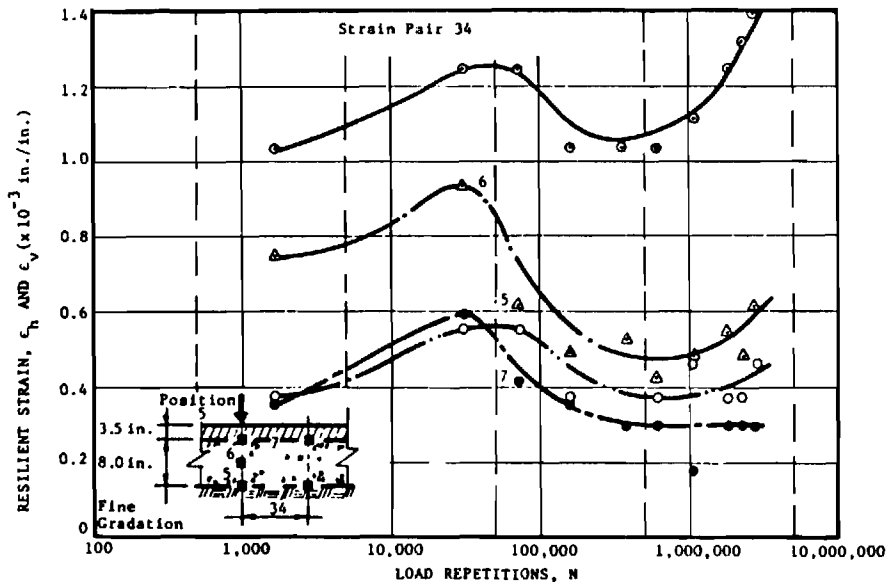


FIGURE 90. VARIATION OF RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 10 .

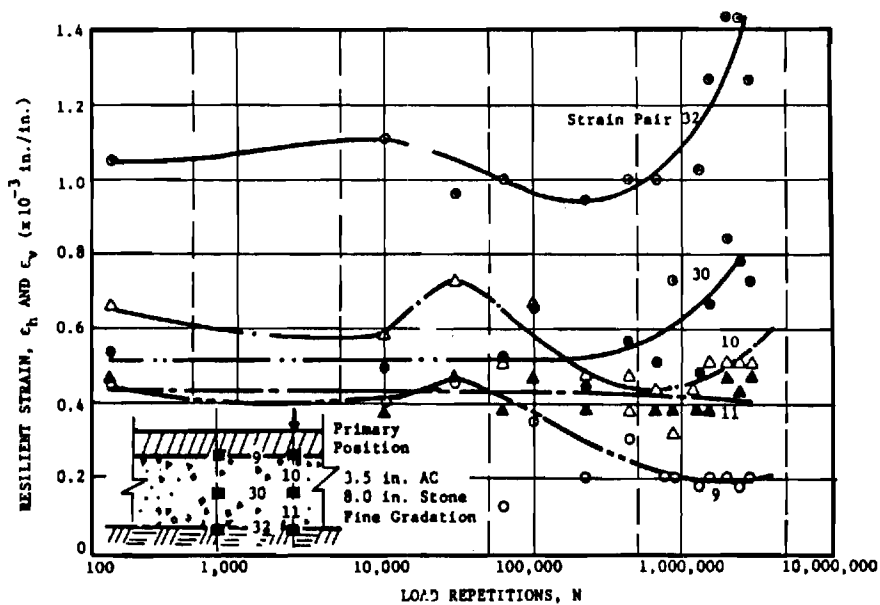


FIGURE 91. VARIATION OF RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 10.

was about 0.34×10^{-3} in./in. In Section 10 the horizontal tensile strain in the bottom of the crushed stone was 1.0×10^{-3} in./in., and the vertical strain in the bottom of the crushed stone was about 0.40×10^{-3} in./in.

The measured vertical strain on top of the subgrade for Section 9 was 1.75×10^{-3} in./in. with a corresponding vertical stress of 11.1 psi (76.5 kN/m^2). In Section 10 the vertical subgrade strain was 2.5×10^{-3} in./in., and the subgrade stress was 6.8 psi (44.9 kN/m^2). The measured vertical subgrade stress of 6.8 psi (44.9 kN/m^2) is believed to be low since it is considerably less than the vertical stresses measured on the subgrade for the other conventional crushed stone base sections of similar thickness.

Section 10 underwent more rutting than Section 9. Based on the strain sensor measurements, about 75% of the increase in rutting in Section 10 compared with Section 9 occurred in the subgrade, with the remaining 25% taking place in the base. Of significance is the finding that the measured vertical resilient strains in both the base and subgrade were larger in Section 10 than Section 9. Also the vertical strain in the bottom of the crushed stone in Section 9 was 0.34×10^{-3} in./in. compared with about 0.40×10^{-3} in./in. for Section 9. These results substantiate the findings that the finer gradation stone was slightly more susceptible to rutting. Also the smaller size stone apparently did not protect the subgrade as much as the coarse crushed stone base.

Sections 11 and 12

Sections 11 and 12 were inverted pavements having a 3.5 in. (89 mm) asphalt concrete surfacing, 8 in. (203 mm) crushed stone base and a 6 in. (152 mm) cement stabilized layer beneath the base. In Section 11 the cement stabilized subbase was constructed by adding 5% Portland cement (by weight) to the silty sand subgrade soil used in all test sections. This subbase had a 28 day unconfined compressive strength of 200 psi (1400 kN/m^2). The cement stabilized subbase used in Section 12 was constructed by adding 4.5% cement to the crushed stone base having the standard gradation. The 28 day unconfined compressive strength of this material was 1,200 psi ($8,300 \text{ kN/m}^2$). The asphalt concrete B-binder used in these sections was obtained from the Lithia Springs plant of APAC-Georgia, Inc.

The load applied to Sections 11 and 12 was 6,500 lbs. (28.9 kN) throughout the test. The number of load repetitions applied at each position was doubled after 480,000 repetitions. As a result instead of applying a maximum of 100,000 load repetitions at the primary load position, 200,000 repetitions were applied during each subsequent sequence of loading. Doubling the number of load repetitions reduced the time required to fail the section since the load did not have to be manually moved as many times. This modified load procedure was particularly severe from the standpoint of rutting.

Fatigue. Section 11 failed in fatigue (initiation of Class 2 cracking) in the asphalt concrete B-binder after approximately 3.6 million repetitions. Section 12 failed in fatigue cracking after about 4.4 million repetitions. The cracking patterns present in Sections 11 and 12 are compared in Fig. 92 after 3.6 million load repetitions. Once again, almost all of the cracks occurred outside the loaded area. Only hairline cracks developed which were quite hard to see.

The horizontal tensile strain measured in the bottom of Section 11, which had the cement stabilized subgrade, was 340×10^{-6} in./in. In contrast Section 12, which had the cement stabilized crushed stone layer and performed better than Section 11, had a horizontal tensile strain in the bottom of the asphalt concrete of 260×10^{-6} in./in. These results suggest the cement stabilized base may have been slightly more effective in reducing tensile strain in the asphalt concrete than the less rigid, cement treated subgrade.

Rutting. Section 11, which had the cement stabilized subgrade, had a rut depth of about 0.5 in. (13 mm), after 3.6 million load repetitions. Therefore Section 11 was a balanced design failing in fatigue and rutting at about the same number of load repetitions. Section 12 had a rut depth of about 0.44 in. (11 mm) after 3.6 million load repetitions. This section also had a reasonably balanced design since it too simultaneously approached a fatigue and rutting failure.

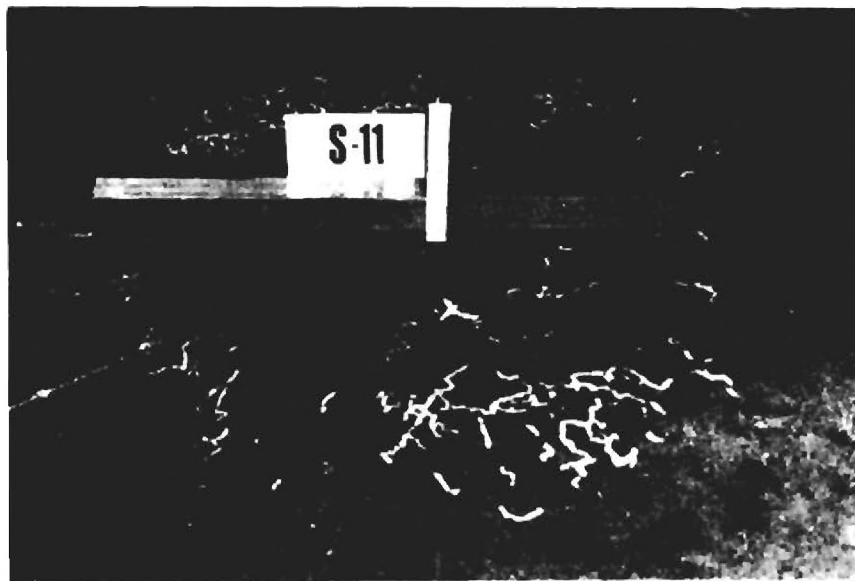
The variation of rut depth with load repetitions beneath the center of the primary position is shown in Fig. 93 for Section 11 and in Fig. 94 for Section 12. Figs. 95 and 96 show the deflection profile at selected numbers of load repetitions.

Distribution of Permanent Deformation. The distribution of permanent deformation that occurred in Sections 11 and 12 is compared in Fig. 97. This figure shows that the magnitude and distribution of permanent deformation in Sections 11 and 12 were essentially the same although the total surface rut depth in Section 11 was slightly greater than for Section 10. In these two inverted sections about 70% of the rutting occurred in the thin, 3.5 in. (89 mm) thick asphalt concrete layer. Twelve to 18% of the permanent deformation occurred in the unstabilized crushed stone base and about 12% in the subgrade. Although only a small portion of the total permanent deformation occurred in the unstabilized base, most of it developed in the upper half of this layer. As expected, essentially no permanent deformation occurred in the cement stabilized layer in either section.

Of practical significance is the finding that the cement stabilized layer was quite effective in reducing rutting in the subgrade. In crushed stone base Sections 9 and 10, which showed excellent performance, about 68% of the total permanent deformation occurred in the subgrade. In contrast, only about 12% of the total permanent deformation occurred in the subgrade of inverted Sections 11 and 12. The structural thickness of the inverted sections were, however, 6 in. (152 mm) greater than the conventional sections; hence some of the reduction in subgrade rutting was due to a greater structural thickness above the subgrade. The



(a) Section 12: 3.5 in. AC/8 in. Stone/6 in. C.S.B.



(b) Section 11: 3.5 in. AC/8 in. Stone/6 in. S.C.

FIGURE 92. FATIGUE CRACKING AND SURFACE DEFLECTION PROFILE OF SECTIONS 11 AND 12 AFTER 3.8 MILLION LOAD REPETITIONS.

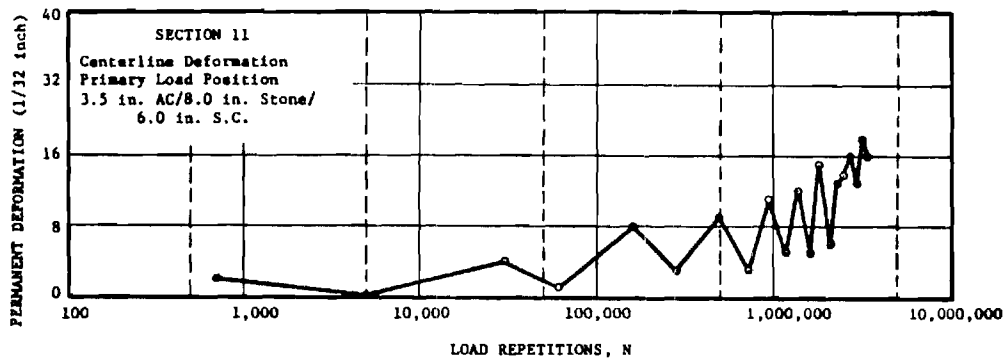


FIGURE 93. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 11 .

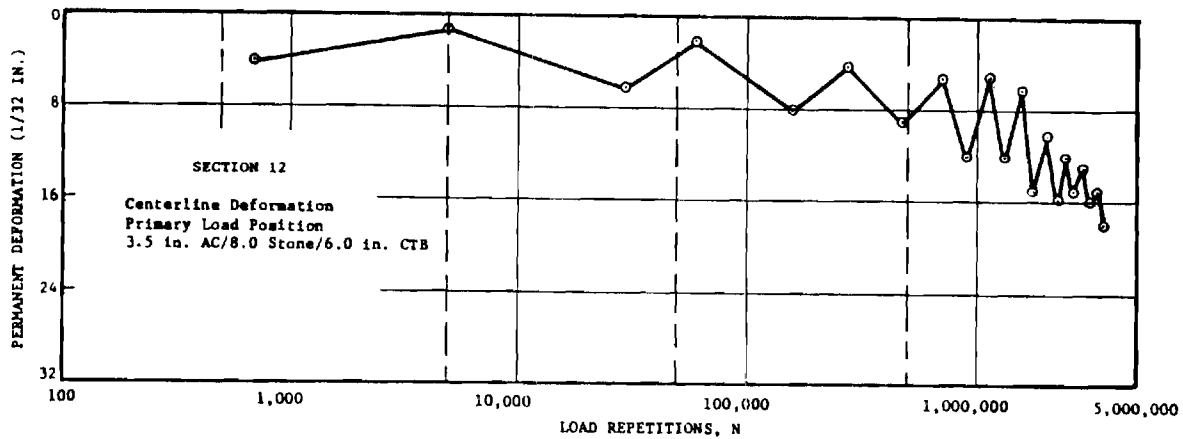


FIGURE 94. VARIATION OF CENTERLINE DEFORMATION WITH NUMBER OF LOAD REPETITIONS - SECTION 12 .

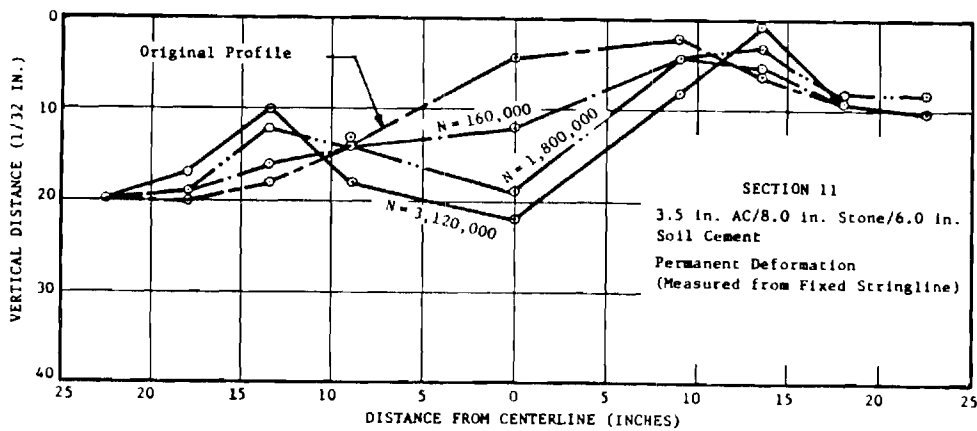


FIGURE 95. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 11.

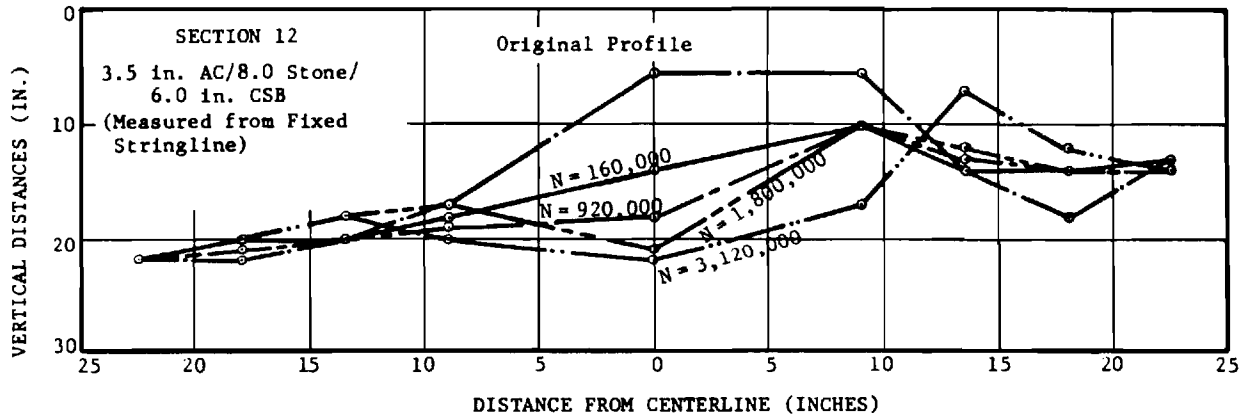


FIGURE 96. VARIATION OF SURFACE PROFILE WITH LOAD REPETITIONS - SECTION 12.

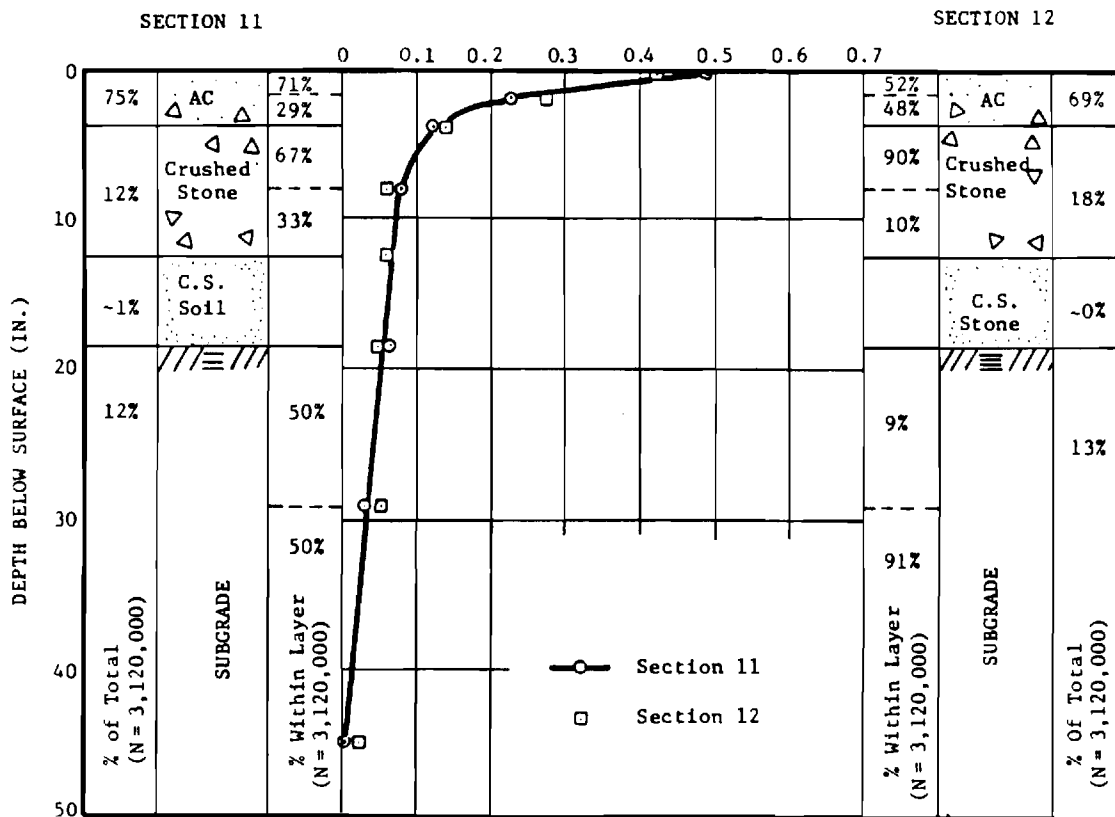


FIGURE 97. COMPARISON OF PERMANENT DEFORMATION WITHIN PAVEMENT STRUCTURE - SECTIONS 11 AND 12 .

majority of the reduction in rutting however, was due to the beneficial effect of the cement stabilized layer which resulted in a significant reduction in vertical stress on the subgrade.

The presence of the stiff inverted layer apparently caused the amount of rutting occurring in the B-binder asphalt concrete surfacing to increase from about 20% of the total permanent deformation in Sections 9 and 10 to 75% in Sections 11 and 12. Permanent deformation decreased almost linearly with depth in the asphalt concrete.

Resilient Response. A general summary of the resilient response of all sections including 11 and 12 is given in Table 17. As previously discussed the horizontal tensile strain measured in the bottom of the asphalt concrete was 340×10^{-6} in./in. in Section 11 and 260×10^{-6} in./in. in Section 12. Tensile strains occurring in the bottom of the cement treated layer of both sections were too small to measure with the strain sensors. The typical resilient response for Section 11 as a function of load repetitions is shown in Figs. 98 through 100, and for Section 12 in Figs. 101 through 103.

In Section 11 the dynamic vertical strain in the bottom of the crushed stone beneath the load was 0.37×10^{-3} in./in. and the horizontal tensile strain at the interface between the crushed stone base and cement treated soil subbase was 54×10^{-6} in./in. In Section 12 the dynamic vertical strain in the bottom of the crushed stone beneath the load was 0.42×10^{-3} , and horizontal strain at the interface between the crushed stone and the cement-stabilized layer was 22×10^{-6} in./in. Apparently some slip may have occurred between the crushed stone and cement treated layer as indicated by the strain sensors.

The measured dynamic vertical strain on top of the subgrade was 0.39×10^{-3} in./in.⁽¹⁾ in Section 11 and 0.34×10^{-3} in./in. in Section 12. The average measured vertical stress on top of the subgrade was 3.3 psi (22.7 kN/m^2) for Section 11 and 2.6 to 4.2 psi ($17.2 - 28.9 \text{ kN/m}^2$) for Section 12.

The dynamic vertical stress measured on top of the subgrade for the inverted sections was considerably less than measured in crushed stone Sections 9 and 10. Measured dynamic strains on top of the subgrade, however, were less in Sections 9 and 10 than 11 and 12 which is somewhat surprising. Nevertheless, permanent deformation of the subgrade was less in the inverted sections than in the conventional crushed stone base sections.

Summary

All crushed stone base sections were constructed on a weak subgrade and had a 3.5 in. (89 mm) asphalt concrete B-binder or modified B binder

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1. Another strain pair in Section 11 indicated an average vertical strain of 491×10^{-6} in./in. This coil pair was felt to not be functioning properly and hence these readings were not used.

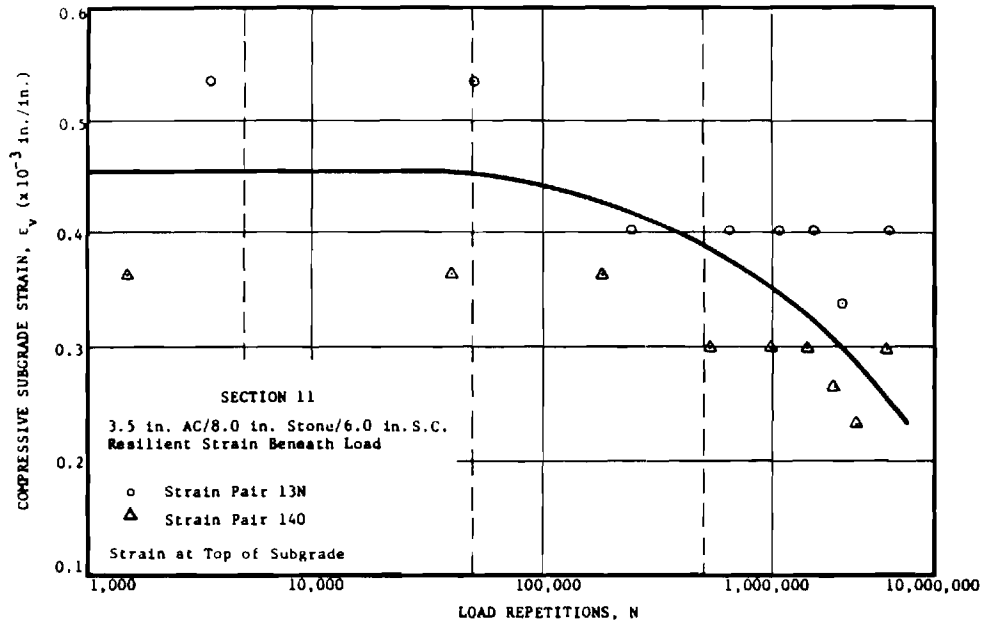


FIGURE 98. VARIATION OF RESILIENT VERTICAL STRAIN BENEATH LOAD WITH LOAD REPETITIONS - SECTION 11.

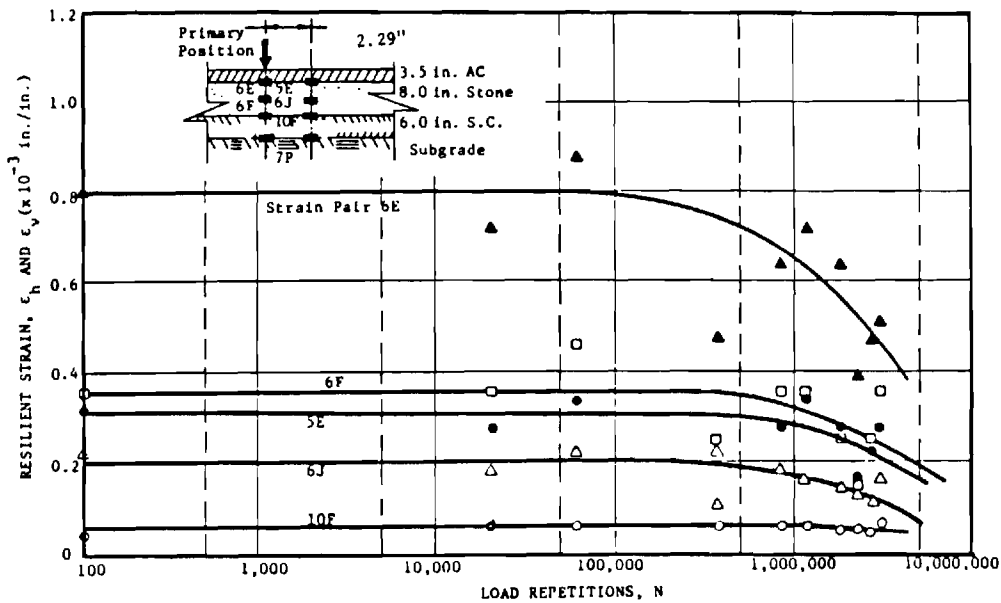


FIGURE 99. VARIATION OF RESILIENT STRAIN WITH LOAD REPETITIONS - SECTION 11 .

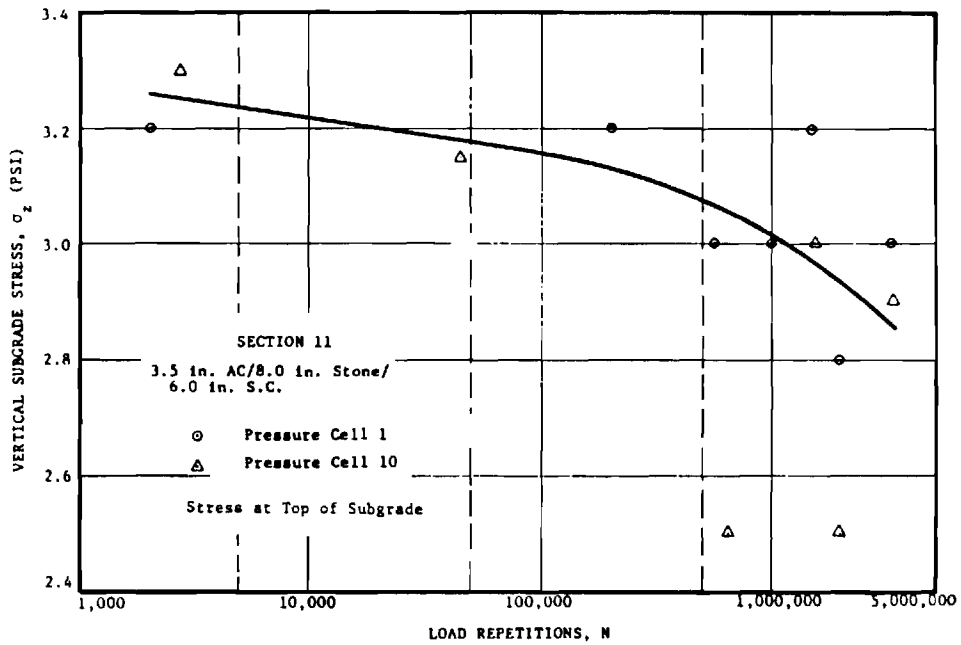


FIGURE 100. VARIATION OF VERTICAL SUBGRADE STRESS BENEATH LOAD WITH LOAD REPETITIONS - SECTION 11.

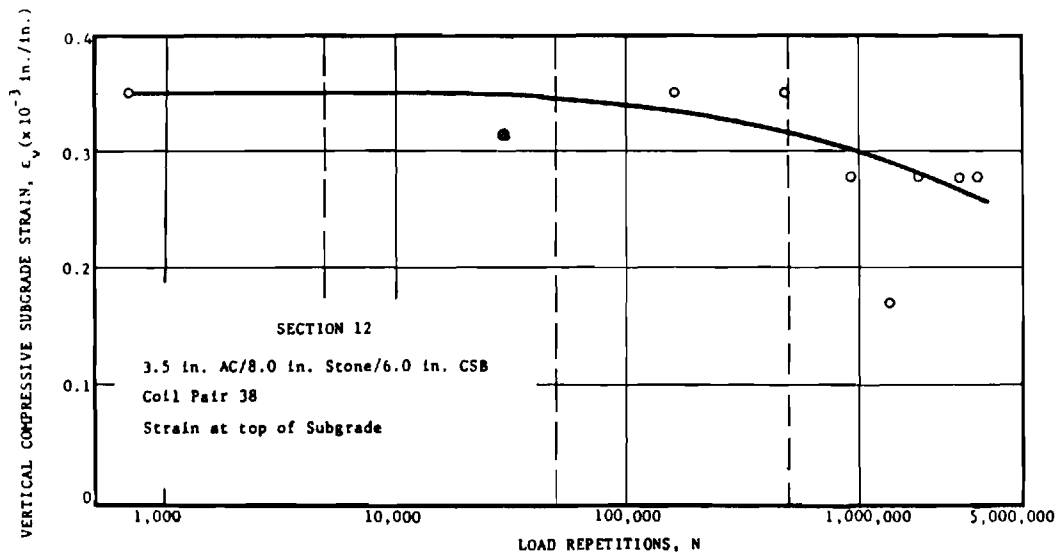


FIGURE 101. VARIATION OF RESILIENT VERTICAL SUBGRADE STRAIN BENEATH LOAD WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

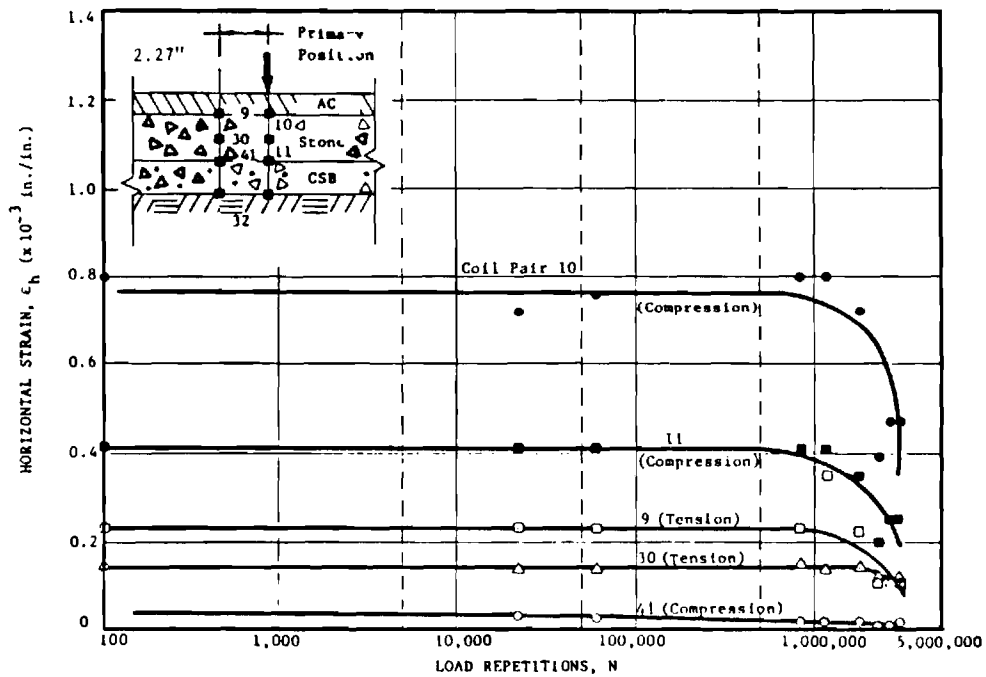


FIGURE 102. VARIATION OF RESILIENT STRAIN WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

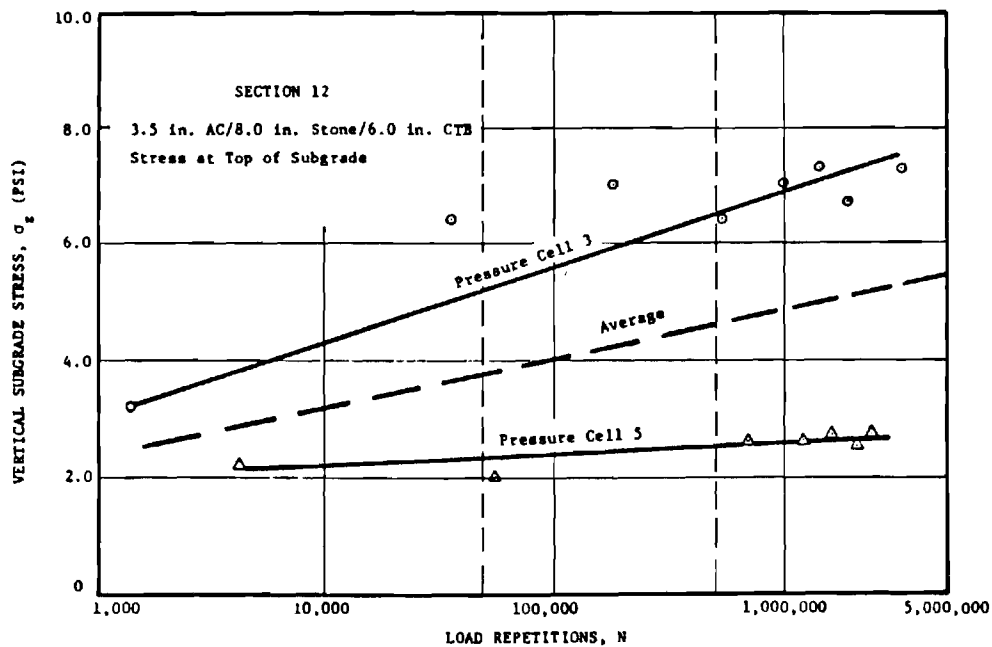


FIGURE 103. VARIATION OF RESILIENT VERTICAL SUBGRADE STRESS BELOW LOAD WITH NUMBER OF LOAD REPETITIONS - SECTION 12.

surfacing. Increasing the crushed stone base thickness from 8 to 12 in. (203 - 305 mm) increased the life of the pavement by a factor of approximately three. The unstabilized crushed stone base sections studied all performed quite well, showing better performance than the full-depth asphalt concrete sections. In a direct comparison of full-depth asphalt concrete pavement (Section 7) with a crushed stone pavement (Section 8), the crushed stone base pavement withstood 500,000 load repetitions before undergoing a rutting failure while the full-depth asphalt concrete section withstood only 150,000 repetitions. Thus the crushed stone base was quite resistant to rutting compared with asphalt concrete even at the test temperature of 78 to 80°F (25.6 - 26.7°C). These findings indicate that if the quality of construction of the crushed stone bases achieved in this study can be duplicated in the field, crushed stone base can be effectively used to replace a significant amount of asphalt concrete in flexible pavements without a sacrifice in performance.

Inverted pavements having a 6 in. (152 mm) cement treated soil subbase (Section 11) and a cement stabilized crushed stone subbase (Section 12) also demonstrated excellent performance. The performance of Section 1, which had a 12 in. (305 mm) crushed stone base, however, was approximately comparable to the inverted section having the cement stabilized soil subbase (Section 11).

The tests conducted during this study were short-term and hence did not include environmental effects. Compared with crushed stone base sections, environmental factors should tend to reduce the service life of the inverted sections, and make rutting at higher summer temperatures a more important factor in the full-depth asphalt concrete sections.

CHAPTER VI DISCUSSION

Introduction

The purpose of this study was to evaluate the use of crushed stone base as an alternative to the deep strength asphalt concrete construction presently used by the Georgia DOT in flexible pavements. Twelve⁽¹⁾ full-scale test sections were tested to failure to determine if crushed stone can be successfully used to replace asphalt concrete in the base course. With the exception of Test Sections 3 and 4, between 150,000 and 4.4 million repetitions were applied to each section (Table 1). All tests were conducted in a constant temperature environment at 78 to 80°F (25.6 - 26.7°C) over a period of two to four months for each pit filling.

In this study the performance of crushed stone bases was directly compared with full-depth asphalt concrete construction. No attempt was made in these tests to include environmental factors such as moisture, temperature changes, temperature gradients and the effects of ultraviolet light. The results of the Brampton Test Road [1,23], however, have shown the very significant effect which the environment has on pavement performance. Further, other differences undoubtedly also exist between the present study and prototype pavements including construction technique, level of quality control and material variability. Therefore, to take these factors into consideration the findings of this study are integrated together with results from previous full-scale experimental pavement sections to give a valid basis upon which to design and construct pavements having thick crushed stone bases.

Test Section Findings

General Comparison

Figure 104 gives a general graphical summary of the performance of the sections tested in this study. Sections 3 and 4 were not included in this summary since they failed prematurely by rutting due apparently to a high asphalt content. Both rutting and fatigue failures occurred in the tests. Sections 1, 11 and 12 failed in a combined fatigue/rutting mode. Sections 2 through 8 failed in rutting, and Sections 9 and 10 failed in fatigue. A fatigue failure was defined as the initiation of Class 2 cracking. A rutting failure was defined as an average rut depth of 0.5 in. (12 mm) measured from a fixed string line; the rutting occurring during the first 1,000 repetitions was not included. Of significance is the finding that the sections surviving over 2 million repetitions failed in fatigue, or else were close to a fatigue failure. In contrast, sections having relatively short lives failed in rutting. Thus, these test results nicely illustrate the importance of preventing excessive rutting in either the

1. Section 1 was tested to 2.4 million load repetitions; failure would have occurred at about 3 to 3.5 million repetitions.

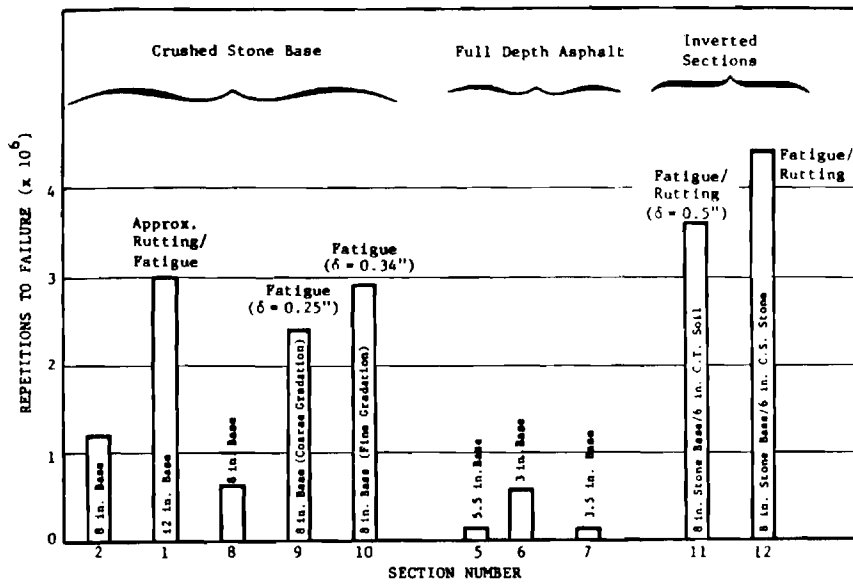


FIGURE 104. SUMMARY OF TEST SECTION RESULTS.

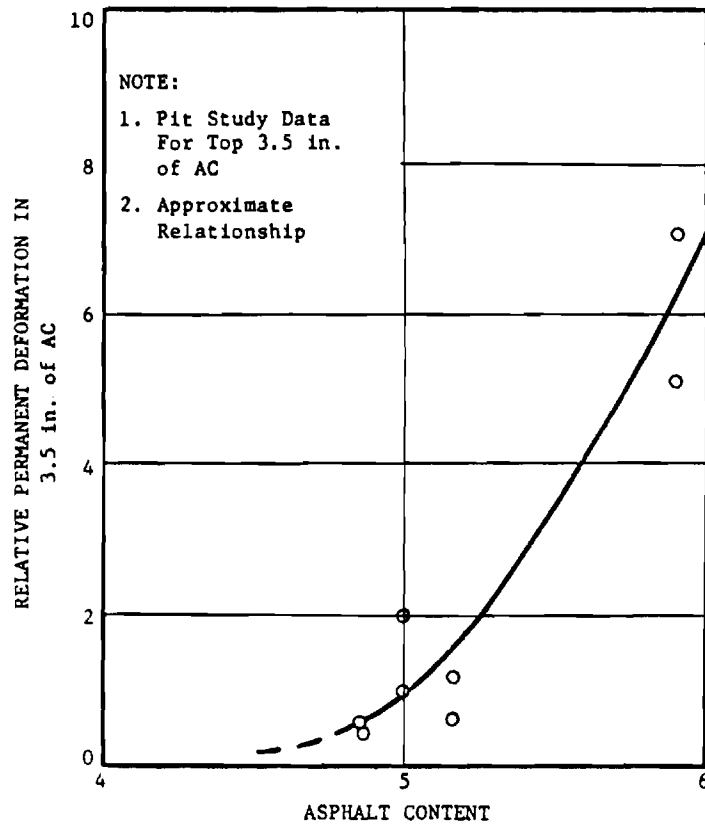


FIGURE 105. APPROXIMATE RELATIVE RUTTING IN ASPHALT CONCRETE AS A FUNCTION OF ASPHALT CONTENT - B AND B-MODIFIED BINDER .

asphalt concrete or crushed stone base.

The stone base was carefully blended together, pugmilled, and compacted in a standard manner for each filling. Likewise the subgrade soil was pugmilled, and the moisture content and density were carefully controlled during each pit filling. Both observations during placement and laboratory tests (Table 6) indicate more variation occurred in the asphalt concrete density and asphalt content than for the other components of the pavement. Contributing to this problem was the fact that 3 months or more elapsed between pit fillings. Also two different asphalt plants and asphalt mixes were used due to unavoidable circumstances. Therefore, in comparing the performance of various sections, the most valid comparisons are in general felt to be between sections constructed during the same filling of the pit. Companion sections that were constructed at the same time are grouped together in Table 1. Asphalt concrete properties of the sections are shown in Table 6.

Crushed stone base Sections 2, 8, 9 and 10 all had an 8 in. (203 mm) stone base and 3.5 in. (89 mm) of asphalt concrete surfacing. Sections 9 and 10 failed in fatigue at 2.4 and 2.9 million repetitions of load, respectively. Sections 2 and 8 failed in rutting at 1 and 0.55 million repetitions, respectively. Although sections 9 and 10 had the coarse and fine gradation stone base, respectively, it is not felt that base gradation accounted for the difference in performance. Possibly the higher density of the asphalt concrete surfacing (Table 6) on Sections 9 and 10 compared with 2 and 8 account for the better performance, and the difference in failure mechanisms. The asphalt content of sections 2 and 8 were, however, lower than sections 9 and 10.

Influence and Base Thickness. The effect of thickness of crushed stone base on pavement life is illustrated in Fig. 104. Section 1, which had a 12 in. (305 mm) crushed stone base would have undergone a combined fatigue/rutting failure after approximately 3.0 to 3.5 million repetitions. Section 2, which had an 8 in. (203 mm) base, and was a companion section to Section 1, failed in rutting after 1.0 million load repetitions. Thus, an increase in base thickness of 4 in. (102 mm) increased the pavement life by a factor of almost 3. Companion sections are used to make this comparison. The AASHTO Road Test results also indicated a similar significant beneficial effect of a small increase in base course thickness [1]. As the base thickness becomes greater, however, the beneficial effect of increasing thickness probably decreases (refer to the analytical study in Appendix A).

Influence of Crushed Stone Base Gradation. Excellent performance was obtained from the granular base pavements having both the coarse (Section 9) and fine gradations (Section 10). Both these sections failed in fatigue compared with a rutting failure at a lower number of repetitions in the other two 8 in. (203 mm) crushed stone base pavements (Sections 2 and 8). The fatigue life of the fine gradation base section was about 20% greater than the fatigue life of the coarse base section. On the other hand,

rutting in the fine gradation base section was 21% greater than in the section having a coarse gradation granular base. These differences are considered reasonably minor considering the possible variation. The somewhat limited test results indicate for the relatively narrow range of gradations tested, gradation has a reasonably minor influence on performance provided the section is compacted to 100% of T-180 density and little segregation is allowed to occur. These results also indicate that the extremes in crushed stone base gradation presently produced in the Atlanta area should perform well under conditions similar to those for the present tests. It should be remembered, however, that all three crushed stone base gradations had a 1 to 2 in. (25 - 51 mm) top size, 40 to 44% passing the No. 4 sieve and 4 to 5% fines.

Influence of Asphalt Content. The approximate effect of asphalt content on permanent deformation is illustrated in Fig. 105. The results presented in this figure were developed considering the permanent deformation occurring in the upper 3.5 in. (89 mm) of both full-depth and crushed stone base sections at 10,000 load repetitions. Because the permanent deformations in most sections were small at 10,000 repetitions, this figure gives only a general indication of the influence of asphalt content on rutting. The results do illustrate that permanent deformation in the B and B-modified binder mixes used in this study was very sensitive to asphalt content. An increase in asphalt content from 5 to 5.25% approximately doubled the amount of permanent deformation; increasing the asphalt content from 5 to 5.5% caused the permanent deformation to increase by a factor of about 3. The full-scale tests indicate that asphalt contents greater than 5.0 to 5.25% should not be used to avoid excessive rutting in B and modified B mixes.

Sections 5 and 6 were full-depth asphalt concrete companion sections that both failed in rutting. Section 5 which consisted of 9 in. (229 mm) of asphalt concrete failed at a lower number of repetitions than Section 6 which consisted of 6 in. (152 mm) of asphalt concrete (Table 1). This difference in performance may have been due to an apparent higher asphalt content in the thicker section (A.C. = 5.71%) than the thinner section (A.C. = 5.17%).

Also, sections 6 and 7, both full-depth asphalt concrete sections, failed at 440,000 and 150,000 repetitions, respectively (Table 1). These were not companion sections and hence were constructed at different times. Section 6 was 0.5 in. (13 mm) thinner than Section 7. Apparently the observed difference in performance was due to the higher asphalt content of Section 7 (A.C. = 5.6%) than Section 6 (A.C. = 5.17%).

Permanent Subgrade Deformation. The same resilient micaceous silty sand subgrade was used beneath all test sections. As previously discussed, the subgrade was removed and recompacted at the same density and moisture content for each test. The resilient modulus of the subgrade was about 3,000 psi (2.07×10^4 kN/m²). This estimate is based on the measured resilient deflection of the test sections considering the resilient modulus obtained from the repeated load triaxial test.

Fig. 106 shows the variation of permanent deformation in this weak subgrade as a function of the base thickness and number of load repetitions. This figure shows that an increase in base thickness causes a decrease in permanent subgrade deformation. As the base thickness increases, however, the rate of decrease in permanent subgrade deformation becomes less. Of practical significance is the finding that the full-depth asphalt concrete sections appeared to be no more effective in reducing subgrade rutting per inch of base than the unstabilized crushed stone base.

Base Course Coefficients. The full-scale laboratory test results (Fig. 104) show excellent performance can be obtained from pavement sections having relatively thin asphalt concrete surfacings and properly constructed crushed stone bases. In fact, the crushed stone base sections out-performed the full-depth sections in every test series. In turn, inverted Sections 11 and 12 out-performed the crushed stone sections. The good performance of the inverted sections compared to the crushed stone base sections is not surprising since the inverted sections were stronger and more expensive. The inverted sections had a structural number of about 4.2 compared with structural numbers of 3.0 to 3.7 for the crushed stone base sections. The AASHTO Interim Guide [2] structural coefficients used in all calculations and figures are given in Table 20.

A base course coefficient c_f of about 0.18 was obtained for the crushed stone sections from this test series assuming the asphalt concrete had a base course coefficient of 0.30. The corresponding base course equivalency was that 1.7 in. (43 mm) of crushed stone replaced 1 in. (25 mm) of asphalt concrete base in this investigation. The base course coefficients were determined by averaging the results from (1) the most efficient full-depth asphalt concrete pavement (Section 6) with (2) the good performing crushed stone Sections 1 and 2 (refer to Fig. 107). Although the asphalt concrete and stone sections were not companion sections, this was felt to be the most valid comparison although the asphalt content and asphalt concrete densities of the sections were not the same (Table 6).

The required depth of crushed stone base to give equal performance to full-depth Section 6 was obtained by conservatively extrapolating crushed stone base test Sections 1 and 2 results to smaller base thicknesses. Admittedly other comparisons could readily give base course equivalencies and coefficients for the crushed stone considerably different than presented above. However, since none of the full-depth asphalt concrete sections failed in fatigue, very likely the asphalt concrete in the full-depth sections was perhaps more susceptible to rutting than in prototype pavements. Therefore more optimistic interpretations of the results are not considered prudent without additional verification.

In this study the crushed stone base material was blended, pugmilled, and bottom dumped so that segregation and variation in gradation were practically non-existent. Also, the crushed stone base was uniformly compacted to between 100 and 101% of AASHTO T-180 density. As a result the laboratory sections had little variation within the crushed stone, and represented the limiting, optimum condition that can be achieved in the field. Finally, the crushed stone had only 4 to 5% passing the No. 200 sieve.

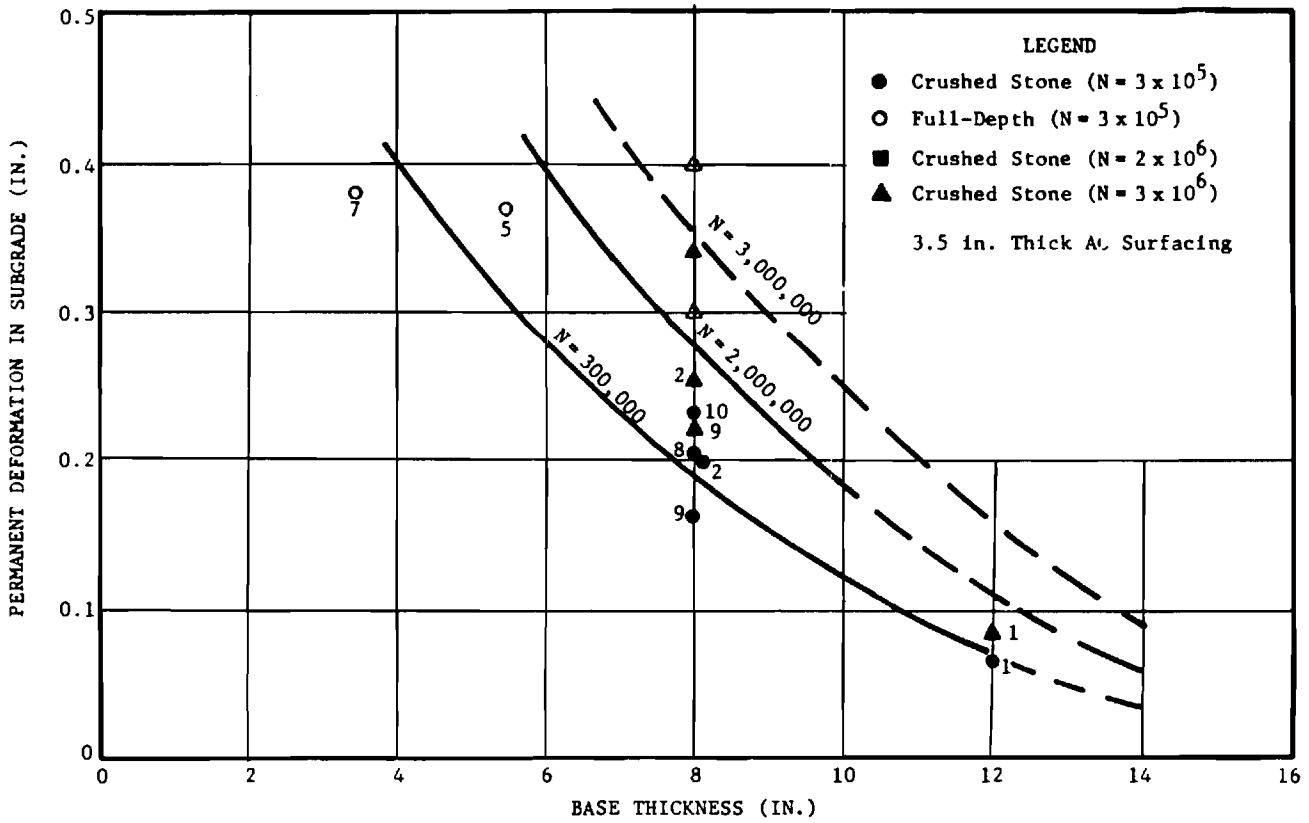


FIGURE 106. EFFECT OF BASE THICKNESS AND NUMBER OF LOAD REPETITIONS ON SUBGRADE RUTTING - CRUSHED STONE BASE AND FULL-DEPTH AC TEST RESULTS .

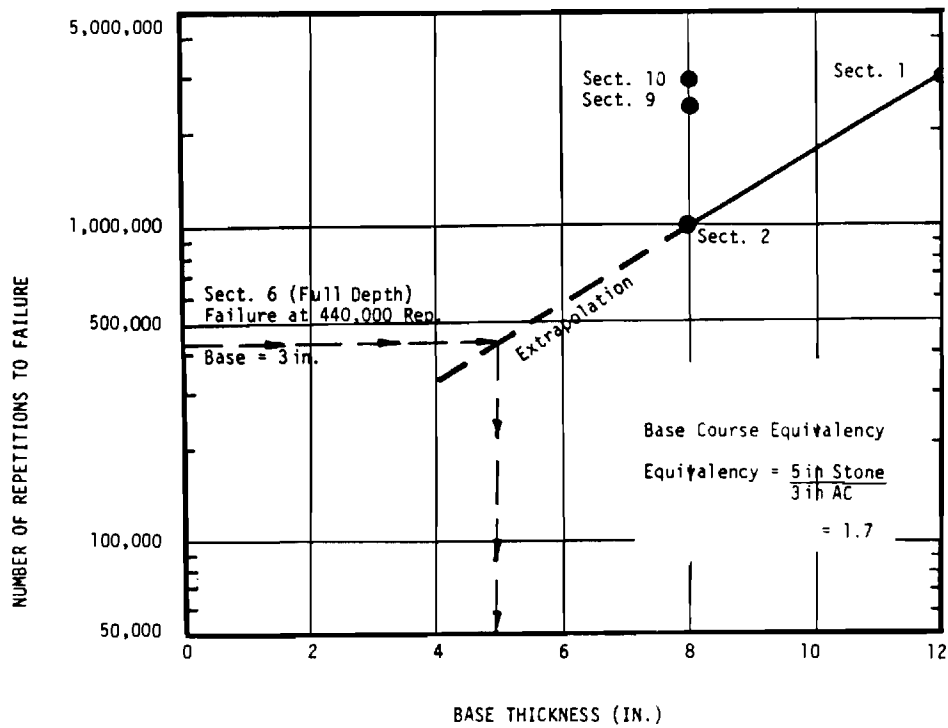


FIGURE 107. EXTRAPOLATION OF TEST RESULTS FOR CALCULATING BASE EQUIVALENCIES.

TABLE 20. AASHTO DESIGN COEFFICIENTS USED FOR ANALYSIS AND OBTAINED FROM FULL-SCALE LABORATORY TESTS .

Material	AASHTO Design Coefficients, a_1		Amount of Crushed ⁽²⁾ Stone Required to Replace 1 in of Base	Structural Number of Test Sections (1)
	Design ⁽¹⁾	Experimental ⁽²⁾		
Asphalt Concrete				2.6-3.2
<4.5 in	0.44	0.44 (assumed)	-	-
>4.5 in	0.3	0.3 (assumed)	1.7	3.2
Crushed Stone	0.18	0.18	1.0	3.0-3.7
Cement Treated Subgrade	0.20	0.14	0.8 ⁽³⁾	4.2
Cement Stabilized Stone	0.22	0.16	0.9 ⁽³⁾	4.3

1. Based on the design base coefficients and used in analysis of test section results; for asphalt concrete thickness less than 4.5 in., a_1 was taken as 0.44.

2. Obtained from Georgia Tech Test Sections.

In contrast, a light maintenance roller was used to compact the asphalt concrete. Also, the asphalt plant was run at about 315°F (157°C) to conserve energy. As a result, the asphalt concrete was laid at approximately 290 to 295°F (143-146°C) which was about the minimum laying temperature that would give the required density for these conditions. All full-depth asphalt concrete sections failed in rutting even though an asphalt concrete density was achieved that met Georgia DOT specifications (typically the asphalt concrete had 96 to 100% of Marshall maximum density. Probably a higher asphalt content in the full-depth sections compared to the crushed stone sections explains this difference in performance (Table 6).

In summary, this study in general probably compared crushed stone at its best with somewhat marginally acceptable asphalt concrete. Nevertheless, excellent performance was obtained from the crushed stone base. For the conditions existing in the tests a conservative base course equivalency was that 1.7 in. (43 mm) of crushed stone was required to replace 1.0 in. (25 mm) of asphalt concrete base.

The longest pavement life was obtained from the inverted sections which also had the greatest structural number. In relative terms of the crushed stone base Sections 9 and 10, however, the cement stabilized stone and cement treated soil inverted sections were found to have base course coefficients of only 0.16 and 0.14, respectively compared to about 0.18 for the crushed stone. The apparent low coefficients of the cement stabilized layer was due to its relatively deep depth in the pavement section. Had this layer been placed higher up in the pavement, the coefficients would have been higher. The influence of a possible gradual deterioration of the cement stabilized or cement treated subbase used in the inverted sections was of course not considered in the study and hence was not reflected in the base course coefficients. Of practical significance however, is the finding that the inverted sections both (1) withstood the greatest number of load repetitions, (2) protected the subgrade, and (3) provided a strong working platform which permitted obtaining exceptionally high crushed stone base densities (Table 14). Finally the inverted section test results indicate that the limiting strength of the materials above a certain level can control performance. In these tests, however, environmental factors such as frost action were not present.

Rutting. Under the heavy 6,500 lb. (28.9 kN) repeated loading, the B and modified B-binder were more susceptible to rutting at the same depth than the three gradations of crushed stone used in these experiments. At high summer temperatures the asphalt concrete would be considerably more susceptible to rutting than at the test temperature of 78 to 80°F (25.6 - 26.7°C). The loading used in this study was slightly more severe with respect to rutting in the asphalt concrete than a 4,500 lb. (20 kN) dual wheel loading.

In the full-depth and inverted asphalt concrete sections the permanent strain (and hence rutting) was approximately constant with depth in the asphalt concrete. In the granular base sections having a 3.5 in. (89 mm) thick asphalt concrete surfacing, the distribution of permanent strain

(and hence rutting) in the asphalt concrete varied with section. Section 9 had the coarse gradation crushed stone base and Section 10 had the fine gradation crushed stone base. In Sections 9 and 10 less rutting occurred in the lower part of the thin asphalt concrete surfacing than in the upper portion. In the other granular base sections more rutting occurred in the bottom of the asphalt concrete.

These experimental results indicate the distribution of rutting within the asphalt concrete is dependent upon both the type and quality of construction; probably the properties of the asphalt concrete have the most effect. A knowledge of the occurrence and distribution of rutting in all layers is important in developing mechanistic methods for estimating rutting in pavement systems.

In all test sections studied having an unstabilized granular base (including the inverted sections), most of the permanent strain within the base occurred in the upper portion of the crushed stone base. For 7 tests, an average of 73% of the permanent strain occurred in the upper portion of the crushed stone base and 23% in the lower portion. These important results suggest that improved pavement performance might be obtained by taking special precautions in constructing the upper part of a granular base course. More relaxed requirements in the lower portion are not, however, recommended.

For the conventional sections having an 8 in. (203 mm) unstabilized crushed stone base, an average of 52% of the total permanent deformation occurred in the subgrade. Sections 9 and 10, which had the coarse and fine gradation bases, performed extremely well. The amount of permanent deformation occurring in the subgrade of these sections was much higher than in the other section, and was 68% of the total. The increased relative amount of rutting in the subgrade apparently occurred because the upper layers (particularly the asphalt concrete) exhibited a high resistance to rutting in these sections. Further, the permanent deformation in the subgrade developed in these two sections at greater depths than in the other granular base sections. In contrast, in Section 1 which had a 12 in. (305 mm) crushed stone base, only 20% of the total permanent deformation occurred in the subgrade. Therefore the additional 4 in. (102 mm) of crushed stone was quite effective in reducing the relative amount of permanent deformation in the subgrade. Section 1, however, did not perform as well as Sections 9 and 10 considering the difference of 4 in. (102 mm) in base thickness.

Now consider relative percentages of permanent deformation occurring in the full-depth asphalt concrete sections. In the 9.0 in. (229 mm) full-depth Section 5, 46% of the total deformation occurred in the subgrade. In the companion 6.5 in. (165 mm) thick full-depth Section 6, 57% of the total amount of rutting occurred in the subgrade. On the average 67% of the total subgrade deformation occurred in the upper 12 in. (305 mm) of the subgrade. Probably the rutting problems observed in the full-depth asphalt concrete sections were due to the asphalt content being above the 50 blow Marshall optimum (Table 6); the crushed stone base sections had an asphalt content below optimum.

A comparison of the permanent deformation occurring in the subgrade of a crushed stone pavement (Section 8) with a full-depth pavement (Section 7) at similar depths below the surface is shown in Fig. 108. The granular base section had 8 in. (203 mm) of crushed stone while the full-depth section had a 7.0 in. (178 mm) thickness of asphalt concrete. Of considerable practical interest is the finding that the permanent deformation occurring in the subgrade beneath the crushed stone base section was about 30% less than the permanent deformation occurring beneath the same level in the companion full-depth section. These results indicate that crushed stone base sections constructed in this study were at least as effective in protecting the subgrade from rutting as full-depth sections of similar thickness.

The effect on permanent subgrade deformation of base thickness and number of load repetitions is shown in Fig. 106. The relationship shown in Fig. 106 was developed by combining the results of both the crushed stone base and full-depth test sections. For fair construction approximately 40% of the permanent deflection can, based on the test results, be expected to occur in the subgrade. For a design rut depth of 0.25 in. (6.3 mm), the allowable permanent deformation in the subgrade would be about 0.1 in. (2.5 mm). From Fig. 106 the required minimum total structural thickness of surfacing and base is about 16 in. (406 mm) to withstand 2 million 6.5 kip (28.9 kN) wheel loads, and about 17 in. (332 mm) to withstand 3 million repetitions. The subgrade studied for which this relationship was developed was silty sand compacted to about 94% of AASHTO T-99 density at about or slightly above optimum. The corresponding standard penetration resistance of the subgrade was 7 to 8. Weaker subgrades of course would be more susceptible to rutting.

Design

Crushed Stone

AASHTO Interim Design Guide. The performance of the Georgia Tech test sections is compared in Figs. 109 and 110 with 17 field test sections constructed in North Carolina [24]. These figures give as a function of the structural number (SN) the ratio of actually applied load repetitions to the number of repetitions predicted by the AASHTO Interim Guide for a Present Serviceability Index (PSI) of 2.5. The actual Present Serviceability Rating (PSR) of each section is given in parentheses by the section. In predicting the AASHTO design life a subgrade support value of 2.0 was used. In the Georgia Tech tests a regional factor of 0.8 was used to consider that environmental effects were not included in the tests. A regional factor of 1.8 was used for the North Carolina tests. A stone base course coefficient of 0.18 and 0.14 was used in Figs. 109 and 110, respectively; the other coefficients are given in Table 20.

The North Carolina test sections were all constructed on Piedmont subgrade soils and had service lives varying from 10 to 15 years. These tests therefore serve as a valuable full-scale extension of the laboratory tests conducted as a part of this study. The North Carolina test pavements

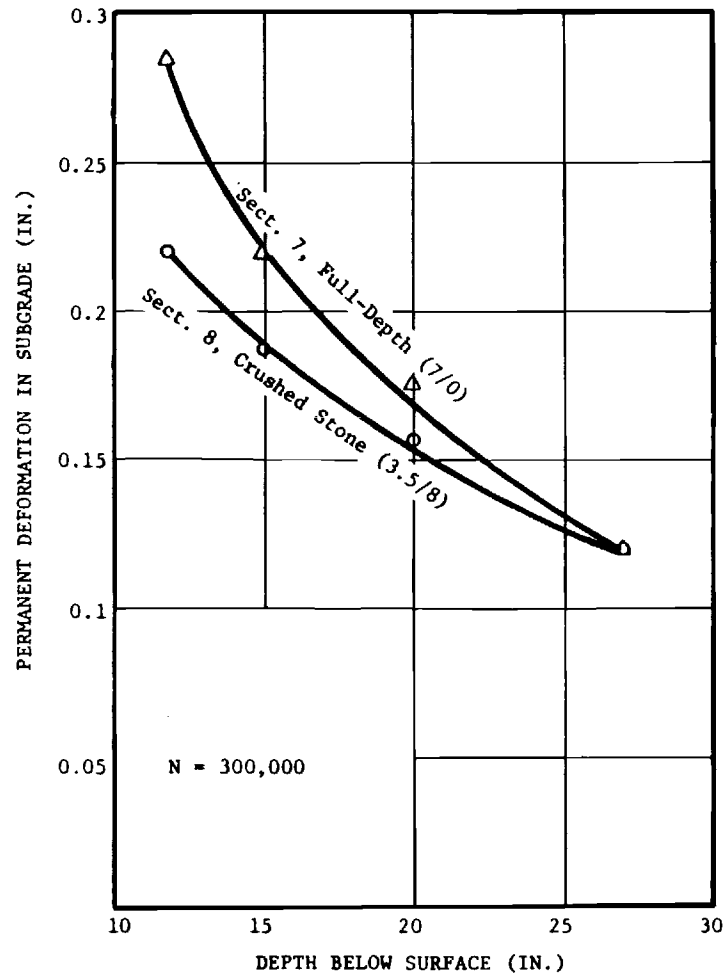


FIGURE 108. COMPARISON OF PERMANENT DEFORMATION IN SUBGRADE OF CRUSHED STONE BASE AND FULL-DEPTH ASPHALT CONCRETE SECTIONS - 300,000 LOAD REPETITIONS.

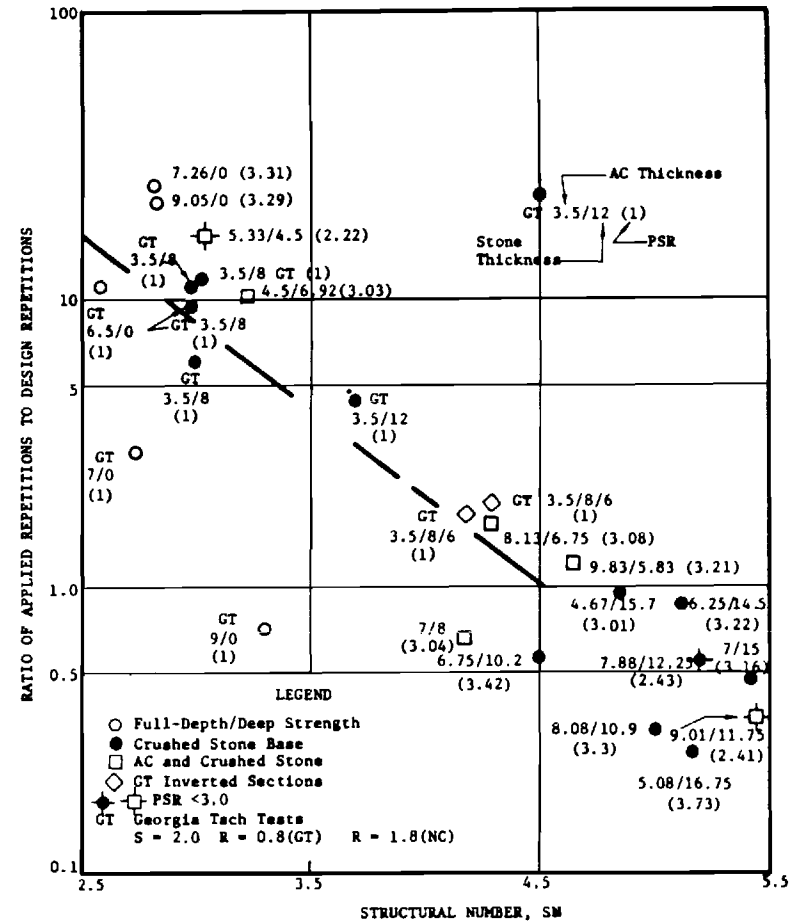


FIGURE 109. COMPARISON OF GEORGIA TECH AND NORTH CAROLINA EXPERIMENTAL RESULTS WITH AASHTO INTERIM DESIGN GUIDE-STONE BASE COEFFICIENT OF 0.18.

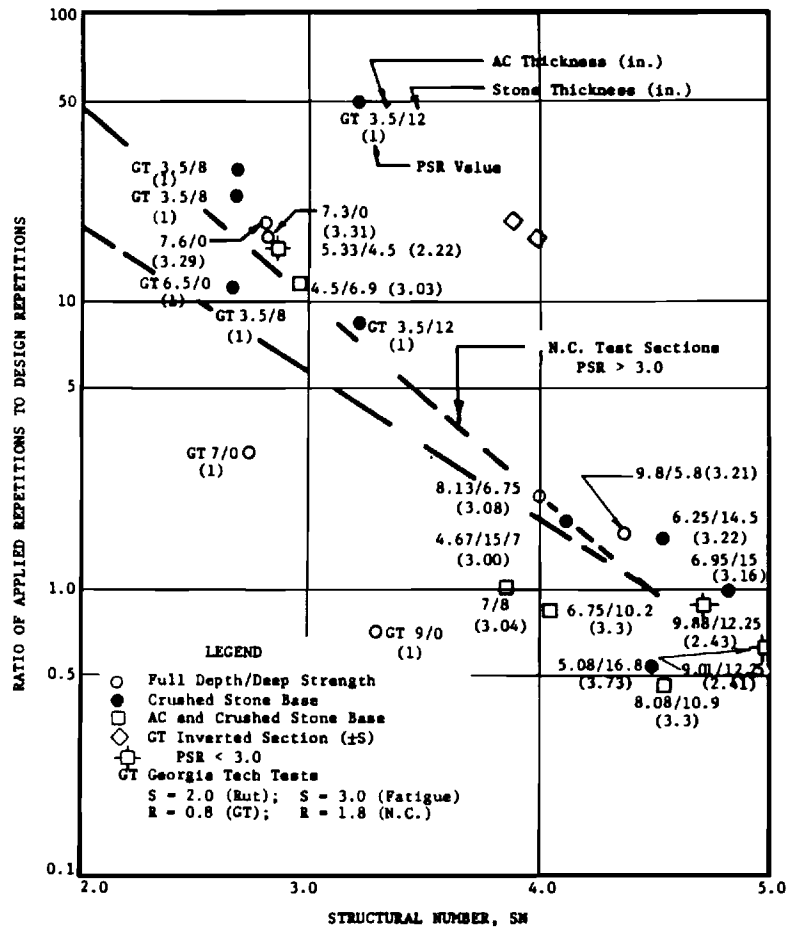


FIGURE 110. COMPARISON OF GEORGIA TECH AND NORTH CAROLINA EXPERIMENTAL RESULTS WITH AASHTO INTERIM GUIDE DESIGN-STONE BASE COEFFICIENT OF 0.14 .

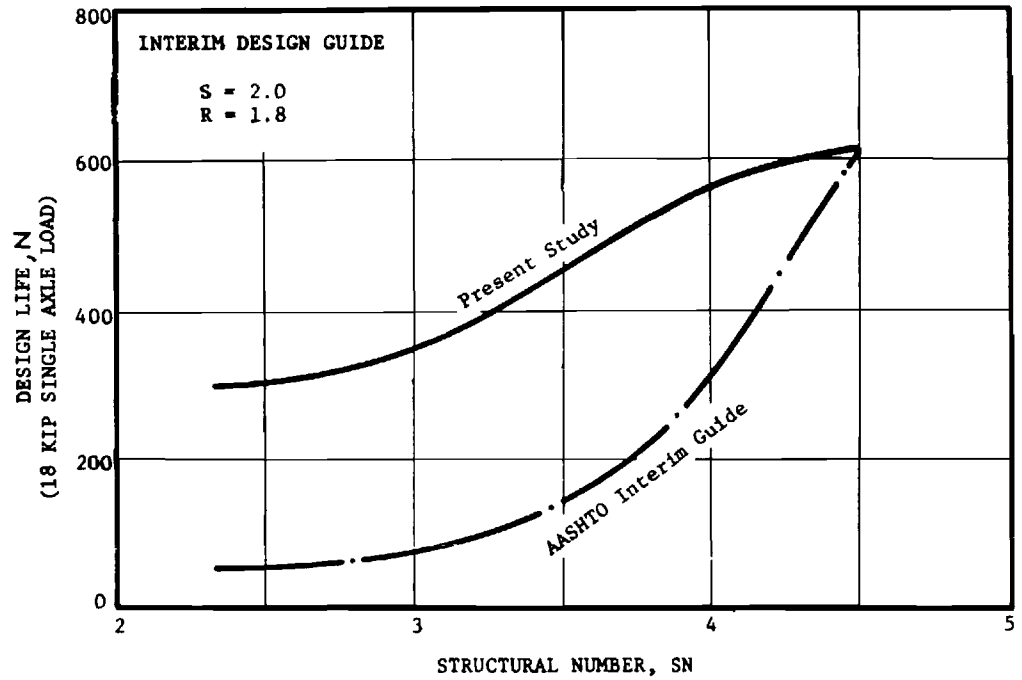


FIGURE 111. COMPARISON OF PAVEMENT LIFE USING AASHTO INTERIM GUIDE AND PRESENT STUDY FINDINGS .

included crushed stone base sections, deep strength sections and sections in between these extremes. All sections except three have Present Serviceability Ratings greater than 2.5; the PI values of all sections are greater than 3.25. The last PSR and PSI ratings were made in 1977. At that time approximately 252 to 950 thousand 18 kip (80 kN) equivalent single axle loadings had been applied to these pavements (Table 21).

Figs. 109-111 clearly show that for well constructed pavement sections having structural numbers less than about 4.5, the AASHTO Interim Guide as applied in this study is overly conservative. This finding was true for almost all crushed stone and asphalt concrete base sections in both the Georgia Tech and North Carolina studies. Based on these experimental results the following empirical correction is proposed for modifying the calculated number of equivalent 18 kip (80 kN) single axle loads for structural numbers SN between 2.0 and 4.5; data are not available for extrapolating the results beyond SN = 4.5:

$$N' = C_1 \times N \quad (1)$$

where: N' = corrected number of equivalent 18 kip (80 kN) single axle loads the pavement can withstand (PSI = 2.5)

N = number of equivalent single axle loads determined from the AASHTO Interim Guide (PSI = 2.5)

C_1 = empirical correction factor

The correction factor, based on the most conservative data from both studies is as follows:

$$C_1 = 10^{-0.445 SN + 2} \quad (2)$$

where C_1 is the correction factor and SN is the structural number of the section. The correction factor, C_1 is also given in Fig. 112 as a function of the structural number SN for $2 \leq SN \leq 4.5$. Data are not available for extrapolating beyond a structural number of 4.5.

Consider the problem where the design traffic loading is known and a structural number is required. First estimate a design structural number and determine the correction term C_1 from either equation (2) or Fig. 112. Now divide the actual traffic loading N by the correction factor C_1 to obtain the adjusted traffic loading N' . Enter the adjusted traffic loading in the AASHTO Interim design chart to determine the required structural number SN. Compare the calculated value of SN with the assumed value, and repeat the calculations if the two values differ by more than about 0.25.

TABLE 21. SUMMARY OF NORTH CAROLINA TEST SECTION STUDY .

Test Section	Open Traffic	YEAR 1977				DESIGN THICKNESS					Surface Course		SN ⁽²⁾	Design ⁽³⁾ N (P _t = 2.5)
		Age	Total Traffic (1)	PSR	PSI	Asphalt Concrete			Crushed Stone Base	STBC Soil	AC %	Voids		
						Wearing	Binder	Base						
T ₁	6/65	12	660	2.43	3.62	1.88	-	6.0	12.25		5.57	6.44	5.20	1200
T ₂	6/65	12	660	3.08	3.76	1.88	-	6.25	6.75		5.91	7.61	4.28	400
T ₃	8/64	12.8	629	2.41	3.68	1.88	-	7.13	11.75		6.88	.03	5.45	1800
T ₄	9/62	14.8												
T ₅	9/62	14.8												
T ₇	11/65	11.6	765	3.01	3.79	2.0	2.67	-	15.67		7.07	7.8	4.85	800
T ₁₁	10/63	13.7	934								5.69	7.0		
T ₁₇	4/65	12.2	953	3.13	3.25	1.88	-	7.50	-	14.67	5.76	8.54	-	
T ₁₈	4/65	12.2	953	2.71	3.76	2.50	-	8.0	-	9.0	6.8	5.46	-	
T ₂₂	11/65	11.6	305	3.30	3.54	2.75	-	5.33	10.92		6.82	8.45	5.01	980
T ₂₃	11/65	11.6	305	3.42	3.74	1.58	-	5.17	10.25		7.48	7.11	4.50	530
T ₂₄	7/64	12.9	332	3.73	3.67	1.83	3.25	-	16.75		7.24	5.93	5.17	1200
T ₂₅	6/65	12	832	3.16	3.56	2.25	-	4.70	15.0		7.35	8.26	5.42	1800
T ₂₆	6/65	12	832	3.21	3.76	2.25	-	7.58	5.83		7.32	5.77	4.63	650
T ₂₇	3/64	13.2	892	3.22	3.71	2.08	-	4.17	14.50		6.72	7.12	5.11	1000
T ₂₈	8/65	11.8	252											
T ₂₉	8/65	11.8	252	3.04	3.76	2.25	-	4.75	8.0		7.16	7.54	4.17	380
T ₃₀	3/67	10.2	757	2.22	3.47	2.58	2.75	-	4.50		6.83	5.20	3.04	45
T ₃₁	3/67	10.2	757	3.31	3.68	2.42	2.50	2.34	-		6.85	4.32	2.81	30
T ₃₂	3/67	10.2	757	3.29	3.64	1.92	2.50	2.91	-		6.86	5.57	2.83	35
T ₃₃	3/67	10.2	757	3.03	3.83	2.17	2.33	-	6.92		6.46	4.87	3.23	70

- NOTES: 1. Approximate number of equivalent 18 kip axle loads (in thousands) applied to inside lane; projected from 4 to 6 years of measurements using an assumed growth rate of 2.5% each year.
2. The structural number was determined using the AASHTO Interim Design Guide Coefficients given in Table 19.
3. AASHTO Interim Guide design life using a Regional Factor of 1.8 and Soil Support Value of 2.0.

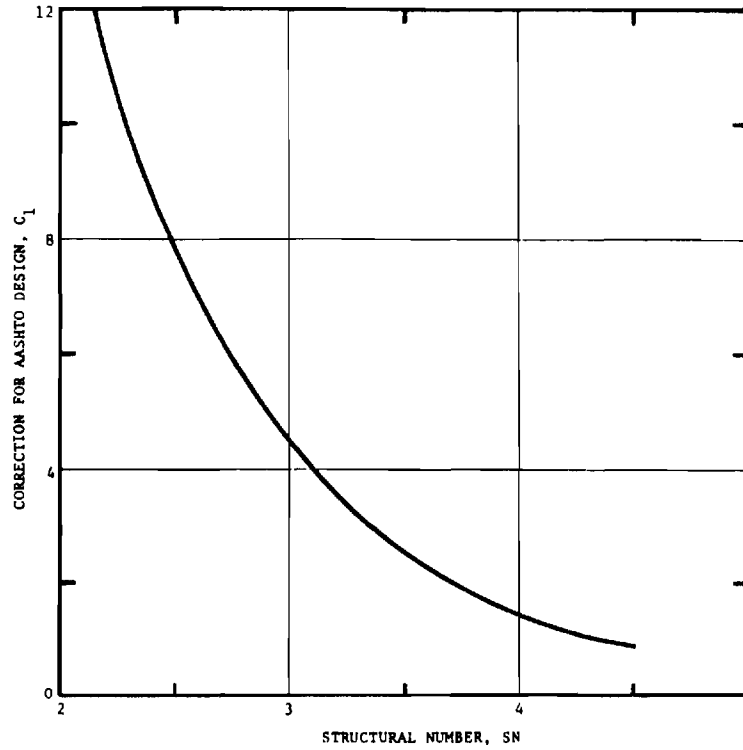


FIGURE 112. CORRECTION FACTOR C_1 FOR AASHTO INTERIM DESIGN GUIDE.

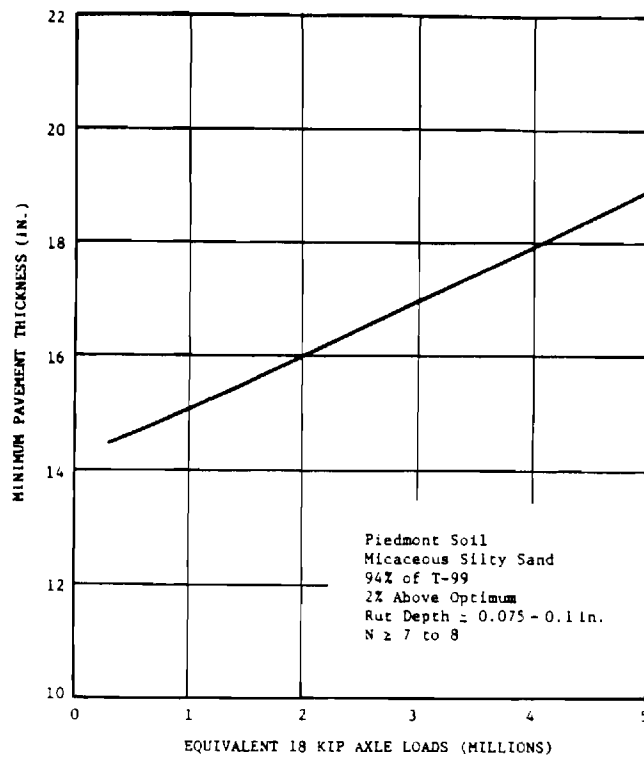


FIGURE 113. MINIMUM TOTAL PAVEMENT THICKNESS TO LIMIT SUBGRADE RUTTING.

Base Course Equivalency. Considering all available information including the findings presented in the literature survey, the North Carolina field test sections and the Georgia Tech study, the recommendation is made that 1.7 in. (51 mm) of crushed stone be used in the base to replace 1.0 in. (25 mm) of asphalt concrete base. In terms of the AASHTO Interim Design Method, the crushed stone base equivalency is numerically equal to the asphalt concrete base course coefficient divided by the crushed stone base course coefficient. For an AASHTO asphalt concrete base course coefficient of 0.30, the corresponding crushed stone base coefficient would be 0.18 for the proposed base course equivalency of 1.7. These equivalencies should be used for total pavement depths less than 15 in. (381 mm); for greater thicknesses a coefficient of 0.14 is recommended. The equivalency between crushed stone and asphalt concrete is dependent upon many factors including (1) magnitude of wheel loading, (2) level of compaction, (3) uniformity of the base which is influenced by construction and inspection practices, (4) segregation of crushed stone, (5) thickness of structural layers, and (6) aggregate size and undoubtedly gradation, angularity and durability. Because of the influence of these and other variables, the additional recommendations presented subsequently for crushed stone bases should be followed to obtain satisfactory performance.

The Georgia Tech study indicated a crushed stone base course equivalency of 1.7 for high quality crushed stone bases constructed over a micaceous silty sand Piedmont subgrade. The North Carolina field test sections also constructed on Piedmont soils had an equivalency of about this magnitude. Further, the studies comparing high quality crushed stone and asphalt concrete bases summarized in Chapter II indicate base course equivalencies varying from 1.6 to 2.5. Therefore the recommendation of a base course equivalency of 1.7 is supported by a considerable amount of field test results in addition to the results of the present study.

Asphalt Concrete Thickness. For optimum pavement performance engineered crushed stone bases should be protected by the asphalt concrete thicknesses given in Table 22. These recommendations are made considering the results of the Florida, North Carolina and Georgia Tech studies. Particular weight was given to the North Carolina study in developing the table. Table 22 indicates, for example, that between 5 and 8 in. (127-203 mm) of asphalt concrete could be used for sections having a structural number of 5.5.

The results of the North Carolina, Lake Wales, Florida and Georgia Tech studies show that pavements having relatively thin asphalt concrete surfacings can, if properly constructed, successfully withstand heavy traffic loadings. Further, crushed stone bases constructed following the recommendations given subsequently should be no more susceptible to rutting than asphalt concrete. In fact, both the Georgia Tech and Florida studies indicate that particularly for high summer temperatures, rutting can actually be greater in the asphalt concrete than crushed stone. About half the rutting in asphalt concrete pavements constructed in Georgia occurs at temperatures above about 95°F (35°C). Therefore use of crushed stone base sections may indeed prove beneficial in limiting rutting.

TABLE 22. RECOMMENDED MAXIMUM THICKNESS OF ASPHALT CONCRETE IN CRUSHED STONE BASE CONSTRUCTION.

AASHTO Structural Number	Asphalt Concrete Thickness (in.)
<4	≤4.0
4 to 5	3.5 - 6.5
5 to 6	5 - 8

TABLE 23. MINIMUM RECOMMENDED TOTAL STRUCTURAL THICKNESS (SURFACING AND BASE) TO PROTECT SUBGRADE FROM RUTTING.

Equivalent 18 kip Single Axle Loading	Minimum Structural ⁽¹⁾ Thickness (inches)
	SF = 1.0
300,000	14.5
2,000,000	16
3,000,000	17
5,000,000	19

1. The subgrade should have a density equal to or greater than 95% of AASHTO T-99 density and/or a standard penetration resistance of about 7 to 8 blows per foot; water content at or slightly above optimum.

The results of the Florida and Georgia Tech studies show that thin sections having small structural numbers can exhibit exceptionally good performance compared with that predicted by the AASHTO Interim Design Guide. The postulation is presented that the good performance is at least partly due to overall flexibility of the pavement structure. Many years ago Hveem [25] realized that thinner asphalt concrete surfacings could withstand greater deflections. Early fatigue studies by Monismith and Deacon [26] showed thin layers of asphalt concrete are more flexible than thick ones. A detailed analysis of the AASHTO Road Test results by Barksdale [19] also showed that asphalt concrete becomes more flexible as the layer thickness decreases. The good performance of surface treated crushed stone base pavements is further evidence of this principle.

Therefore in designing crushed stone base pavements for optimum performance, the thickness of the surfacing should be made no greater than necessary to protect the crushed stone base. In the Lake Wales, Florida study a 1.5 in. (38 mm) asphalt concrete surfacing withstood 2.5 million equivalent 18 kip (80 kN) axle loadings. As many as 4.4 million 6.5 kip (29 kN) wheel loadings were applied to a 3.5 in. (89 mm) surface during the present study. Further, military airfield pavements have successfully used thin asphalt concrete surfacings for many years under heavy aircraft loadings. These examples illustrate that even quite thin asphalt concrete surfacings can be successfully used in granular base pavements.

An important need presently exists to establish the optimum thickness of asphalt concrete for use over a crushed stone base pavement. This optimum depth must be determined using full-scale test pavements. Until such test results are available, the recommended maximum asphalt concrete thicknesses given in Table 22 serve as a general guide for design. Future studies may show that even smaller thicknesses of asphalt concrete surfacing are desirable.

Limiting Structural Thickness

The minimum total structural thickness is given in Table 23 for crushed stone base sections to limit permanent deformation in the subgrade to less than about 0.1 in. (1.9-2.5 mm). The recommendations given in Table 23 are valid if the subgrade is reasonably good. In specific the results were developed for a resilient silty sand subgrade having a density of at least 94% of AASHTO T-99 density and a standard penetration resistance of 7 to 8 blows per foot. The subgrade was at or slightly above optimum moisture content. On poor subgrades alternate design sections, greater structural thickness or remedial work is required as discussed subsequently.

Using Table 23 gives required structural thicknesses slightly less than the thickness of the crushed stone base sections used in North Carolina which have performed extremely well. Finally, the recommended thicknesses to insure subgrade protection are in reasonable agreement with the results obtained at AASHTO for no subgrade rutting (Fig. 114).

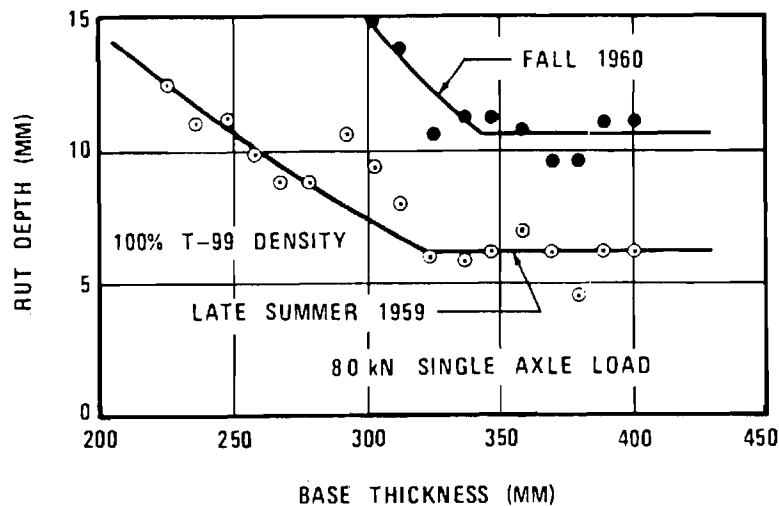


FIGURE 114. RUT DEPTH AS A FUNCTION OF BASE THICKNESS FROM AASHTO ROAD TEST.

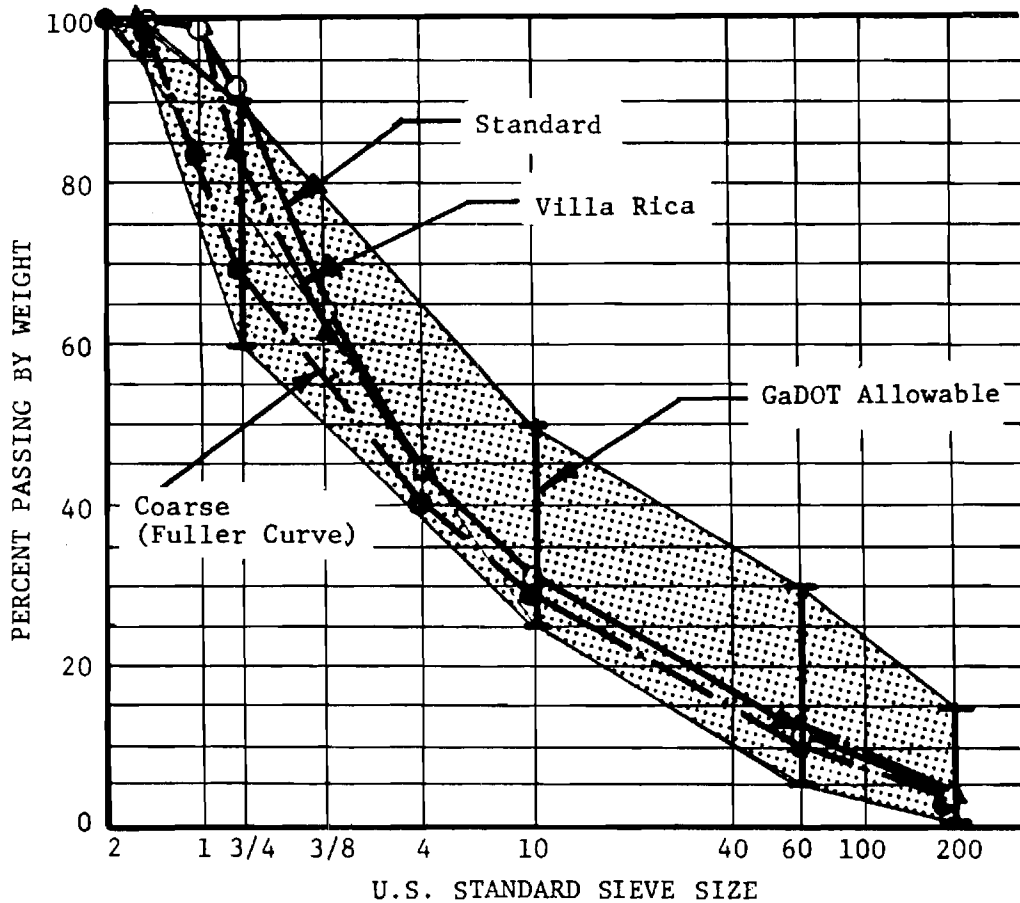


FIGURE 115. COMPARISON OF CRUSHED STONE GRADATIONS TESTED WITH GEORGIA DOT ALLOWABLE VARIATION.

For design of flexible crushed stone base pavements the AASHTO Interim Guide should be used together with Table 22 and Table 23. In general, use of the AASHTO Interim Guide design method gives structural section thicknesses greater than required to protect the subgrade. Limiting structural thicknesses should certainly be considered in designing light pavements to be constructed in stages.

Crushed Stone Base Construction. The markedly good performance of the crushed stone base sections in the present study is attributed to the use of a crushed stone having only 4 to 5% fines and the use of optimum construction techniques and quality control procedures in constructing the crushed stone bases. To insure a similar high level of performance for prototype pavements, construction procedures and specifications should be modified, and new innovative construction techniques developed. In summary the entire construction-inspection sequence must be conducted so as to give a truly "engineered" crushed stone base.

The outstanding performance of the crushed stone base can be primarily attributed to the following two important factors: (1) pugmilling the crushed stone and bottom dumping it so as to eliminate, as nearly as practical, segregation of the stone, and (2) uniformly compacting the crushed stone to a minimum of 100% of T-180 density. Certainly concentrated effort should be placed in the field on satisfying these two important requirements. The following recommendations are given in an attempt to improve present field construction practices:

1. Pugmilling. The crushed stone should be pugmilled to achieve a uniform blend. The cost of pugmilling should, for example, be less than the addition of lime to an asphalt concrete mix.
2. Stockpiling. The crushed stone should not be stockpiled at the site unless it is pugmilled afterwards.
3. Base Protection. After placing the crushed stone with a spreader, the surface should not be left exposed for more than 7 working days unless it is covered in an approved manner.
4. Base Density. Optimum base performance is obtained by placing a crushed stone base at the maximum practical density. Therefore the contractor should be strongly encouraged to achieve a high base course density. This can perhaps be best achieved by developing a specification that places a stiff penalty for obtaining densities less than 100% T-180. Perhaps a bonus might be given for attaining higher densities than 100% of T-180 and also for achieving uniformity of construction at acceptable densities.
5. Inspection. In general an increase in in-place variability tends to occur as the cost of a material decreases. Probably the in-place variability of crushed stone bases is greater than

for asphalt concrete. Therefore special attention should be given to inspecting crushed stone base construction. An adequate number of senior level technicians should be used to give full time inspection to base construction.

Maximum Crushed Stone Base Thickness

The results of an extensive parametric study of stress, strain, and deflection variations in crushed stone base, inverted and full-depth asphalt concrete sections are presented in Appendix A. These results were developed by first determining material properties that gave the best overall fit to the observed response of each type section. The theoretical results presented in Appendix A indicate that as the crushed stone base increases in thickness the tensile strain in the asphalt concrete becomes less at a decreasing rate. After a crushed stone base thickness of 16 to 18 in. (406 - 457 mm) is reached, little additional benefit appears theoretically to be gained from the standpoint of fatigue by a further increase in crushed stone base thickness. An increase in thickness of asphalt concrete, however, is quite effective in causing a significant decrease in theoretical tensile strain. It must be remembered, however, that increases in asphalt concrete thicknesses apparently cause the surfacing to become more brittle and hence undergo a fatigue failure at a lower strain than in thin layers.

The designer must therefore select an optimum combination of asphalt concrete and crushed stone base thickness to maximize fatigue life. Unfortunately at the present time sufficient information does not exist, as previously discussed, on effect of asphalt concrete thickness on fatigue life to fully understand the complex interaction between fatigue life and asphalt concrete thickness. The tentative recommendation is made that a crushed stone base be no greater than 16 to 18 in. (406 - 457 mm) in thickness. After reaching this thickness, the structural number should be increased, if required, by increasing the asphalt concrete thickness. To achieve maximum flexibility the asphalt concrete surfacing should at the same time be made as thin as practical and still obtain the required structural number. Table 22 can be used as a tentative guide in selecting asphalt concrete thickness for flexible, granular base pavements.

Crushed Stone Gradation. The three gradations of crushed stone base used in this study all demonstrated good performance. This grading band is compared in Fig. 115 with present Georgia DOT crushed stone base grading specifications. The fine gradation base section gave the best overall performance of any 8 in. (203 mm) crushed stone section. The coarse graded crushed stone base had slightly less rutting but failed in fatigue at 2.4 million repetitions compared with 2.9 million for the fine gradation base. Of significance is the fact that all three gradations had less than 5% passing the No. 200 sieve. Based on these findings all three gradations can give good performance provided segregation is minimized and a high level of densification obtained.

Proof-Rolling

Proof-rolling offers an excellent method for evaluating the in-situ behavior of either the subgrade or the crushed stone base. Proof-rolling is common in private consulting practice, and is also conducted on military airfield pavements.

Any subgrade which may be weak should be proof-rolled with a loaded dump truck (an alternate would be to use a loaded pan). A qualified inspector should carefully observe the proof-rolling operation watching for excessive subgrade deflections. The inspector should also observe pans and other heavy construction equipment moving over the subgrade along the route even in areas considered to be good. All detected weak areas should be undercut and replaced with well compacted structural fill, crushed stone or other suitable material.

Serious consideration should also be given to proof-rolling the crushed stone base. Proof-rolling should be accomplished using at least 3 to 6 passes of a loaded, 50 ton pneumatic tired roller operating at a high tire pressure. The recommendation is presented that proof-rolling be tried in the field on a limited basis to evaluate its effectiveness. In a well-constructed crushed stone base almost three-fourths of the permanent deformation occurs in the upper portion of the base. Therefore, proof-rolling might be carried out in the top half of the base.

Inverted Sections

The soil cement inverted section withstood 3.6 million repetitions of load before failing at about the same time in both fatigue and rutting. The cement stabilized crushed stone inverted section failed similarly after about 4.4 million repetitions. Both sections therefore exhibited excellent performance with the soil-cement section undergoing only 18% fewer repetitions than the cement stabilized crushed stone.

The inverted sections, however, had a relatively large structural thickness. The structural number of the inverted sections⁽¹⁾ was about 4.2 compared with 3.0 and 3.7 for the 8 and 12 in. (203 and 305 mm), respectively, crushed stone base sections. Because of the larger structural thickness the inverted sections did not perform as efficiently as did the crushed stone base sections. As a result the AASHTO base course coefficients for the inverted sections were approximately 0.16 and 0.14 for the cement stabilized crushed stone and the cement treated subgrade soil, respectively.

Elastic layered theory (Appendix A) indicates that when used in inverted sections, crushed stone bases up to about 24 in. (610 mm) in thickness have little effect in reducing the horizontal tensile strain in the bottom of the asphalt concrete (Appendix A). Further, either with or without the presence of the cement stabilized layer relatively little rutting would occur on the subgrade for structural thicknesses greater than those given in Table 23 provided the subgrade is reasonably competent.

1. Recall that the structural numbers used in this chapter are those based on the base course coefficients given in Table 20 and not those determined during this research.

These factors therefore help to explain the relatively low base course coefficients observed in the cement stabilized layer of the inverted sections. Further, probably the crushed stone base should only be made thick enough to reduce reflection cracking or else omitted altogether.

These findings tend to verify the hypothesis that with depth the base course coefficient of any material tends to decrease. *This simply means a strong and/or deep structural section becomes less efficient and failure is ultimately controlled by the quality of the material in the upper levels.* For conventional crushed stone base sections on a reasonably good subgrade, this depth of total pavement section appears to be about 17 in. (432 mm) for 3,000,000 equivalent 18 kip, single axle loadings. The quality of the subgrade of course varies significantly throughout Georgia.

Cement stabilization of a weak subgrade does improve the subgrade and gives a strong working platform upon which the base can be constructed. Indeed a higher density of the crushed stone can be obtained when a cement stabilized layer is present, although the higher density apparently did not significantly increase resistance to rutting (compare rutting in Sections 9 and 10 with that in Sections 11 and 12).

The soil cement had a 28-day unconfined compressive strength of only about 200 psi (1380 kN/m²) at a 5% cement level. In contrast, the cement stabilized crushed stone section had a 28-day strength of about 1200 psi (8270 kN/m²) at a cement level of 4.5%. For the laboratory conditions under which the tests were performed, the soil cement section thus had a sufficient level of strength throughout the test to perform almost as well as the much higher strength cement stabilized section. In the field, however, a soil cement having an unconfined compressive strength of only 200 psi (1380 kN/m²) would not be expected to have the necessary durability to withstand a large number of wet-dry and freeze-thaw cycles.

Good performance of cement treated subgrades has been observed in Virginia [3] when placed directly beneath the asphalt concrete. Inverted sections constructed in Virginia have demonstrated somewhat variable performance; McGhee [27] believes good performance of inverted sections is related to good drainage. A positive mini-drain collection system having a positive outlet [28] or conventional lateral drains should therefore be used to remove water from the base of inverted sections constructed where water is expected to constitute a problem.

In areas having poor subgrade conditions which cannot be undercut the use should be considered of inverted sections or composite asphalt concrete-cement stabilized subbase sections. The subgrade should be stabilized with about 10% cement by weight to insure a strong, durable structural layer, or else a cement stabilized crushed stone subbase should be used. Such a layer will indefinitely reduce the stress reaching the subgrade to an acceptable level and minimize future problems in the subgrade. Good performance has been observed in Virginia when subgrades are stabilized at a level of 10% cement by volume.

Conclusions

The five crushed stone base sections tested in this study performed better than the full-depth asphalt concrete sections. Both fatigue and rutting failures occurred in the crushed stone base sections, while the full-depth asphalt concrete sections failed in only rutting. The complete response of these sections was measured including both resilient and long-term strains, and dynamic subgrade stresses. These experimental findings have contributed significant information concerning the mechanistic behavior of pavements constructed on resilient subgrades such as those found in the Piedmont Geologic Province.

Both the results of this study and full-scale field test results presented in the literature show crushed stone base pavements can be constructed having excellent performance. To insure good performance, more caution, however, must be exercised in constructing granular bases compared to deep strength asphalt concrete sections. Special precautions required in constructing crushed stone bases include (1) minimizing segregation, (2) attaining a uniform product, and (3) achieving a high level of density at least equal to 100% of AASHTO T-180. Finally, the crushed stone bases which demonstrated good performance all had 4 to 5% passing the No. 200 sieve.

Both the results of this study and also the Lake Wales, Florida and North Carolina test roads indicate that the AASHTO Interim Guide significantly underestimates the number of load repetitions a pavement can withstand for structural numbers less than about 4.5. Further, 1.0 in. (25 mm) of asphalt concrete base can be replaced by 1.7 in. (51 mm) of high quality crushed stone meeting the requirements presented in this chapter. This substitution corresponds to a base course coefficient of 0.18 for a corresponding asphalt concrete base course coefficient of 0.30. This substitution can be used up to a total pavement thickness of 15 in. (381 mm); for greater depths a base coefficient of 0.14 is recommended. To maximize fatigue life the crushed stone base sections should be covered with the minimum practical thickness of asphalt concrete. Even for sections having large structural numbers the asphalt concrete surfacing should be kept less than 7.0 to 8.0 in. (178-203 mm) in thickness. Further, the crushed stone base should have a maximum thickness of 16 to 18 in. In conclusion, for optimum performance of crushed stone base pavements, the section must be kept as flexible as possible by minimizing the thickness of asphalt concrete.

CHAPTER VII SUMMARY AND RECOMMENDATIONS

Test Study

Experiment Design

The purpose of the full-scale pavement tests was to study the feasibility of replacing full-depth and deep strength asphalt concrete pavements with flexible pavements utilizing relatively thick layers of crushed stone base. A total of twelve test sections were constructed and tested to failure (one of these sections was only tested close to failure) by applying a repeated, dynamic loading having a magnitude of 6,500 lbs. (30 kN). To simulate traffic wander and prevent a localized punching type failure, the stationary circular loading was applied in a primary load position and six supplementary positions located symmetrically around the edge of the primary position. In all sections an asphalt concrete B or B-modified binder was used for the entire asphalt concrete thickness. The properties of all materials were evaluated using dynamic testing procedures.

The pavement sections tested were as follows: (1) five crushed stone base sections, (2) five full-depth asphalt concrete sections, and (3) two inverted sections. All the crushed stone base sections and the inverted sections had an asphalt concrete thickness of 3.5 in. (89 mm). The crushed stone bases had either an 8 or 12 in. (203-305 mm) thickness. Three crushed stone base gradations were studied approximately defining the limits of crushed stone bases produced in the Atlanta area. The full-depth sections consisted of either 6.5, 7.0, or 9.0 in. (165, 178, 229 mm) of asphalt concrete binder placed directly on the subgrade. The two composite sections consisted of 8 in. (203 mm) of unstabilized crushed stone sandwiched between 3.5 in. (89 mm) of asphalt concrete above and 6 in. (152 mm) of cement stabilized material below.

The results of this study showed that it was feasible to test in the laboratory full-scale sections to failure by subjecting them to large numbers of repeated loadings. By carefully controlling the quality of construction and environmental factors, direct comparisons of performance were obtained between different structural sections.

Test Sections

All tests were performed in an environmentally controlled room at a temperature varying from 78 to 80°F (25.5 - 26.7°C). The unstabilized crushed stone base in all sections was constructed by blending three different sizes of crushed stone together in a pugmill to minimize

segregation. Crushed stone base sections were compacted to or slightly above 100% of AASHTO T-180 density. In the inverted sections because of the presence of a rigid working platform, a crushed stone base density of 105% of AASHTO T-180 was obtained for the same compaction effort used in the other crushed stone bases.

A uniform, silty sand subgrade 50 in. (1270 mm) thick was used in all tests. The subgrade was compacted to an average of 98% ($\sigma = 1.5\%$) of AASHTO T-99 density at a moisture content of 20.4% ($\sigma = 0.5\%$) which was 2% above the optimum value. This material had an average Georgia DOT volume change of 38.5% and classified as a III-B embankment material.

Two hybrid air-over-oil cyclic loading systems were designed and constructed to test to failure the pavement sections. The two testing systems were used to apply over 22 million load repetitions during the study. These testing systems were found to be quite reliable and relatively easy to maintain considering the large number of repetitions applied.

Instrumentation

The test sections were fully instrumented to give important information concerning the amount and distribution of permanent deformation within each layer, resilient displacement, resilient strain and vertical stress on the subgrade. Bison-type strain sensors were used to measure permanent deformations and resilient strains. Vertical stresses were measured using small, diaphragm type pressure cells. Resilient surface displacements were measured with LVDT's. The results from this instrumentation gave valuable information concerning the mechanisms of pavement performance.

Summary of Findings

The findings were developed using the results of 12 test sections carefully integrated together with the findings from field studies presented in Chapters II and VI. Field studies of particular applicability to this investigation on crushed stone bases were conducted in North Carolina and Virginia on Piedmont soils and at Lake Wales, Florida on a sand subgrade. A detailed discussion of the findings is given in Chapters V and VI. Significant conclusions reached from this study are summarized as follows:

1. The 8 and 12 in. (203 - 305 mm) crushed stone base pavements withstood between 0.64 and 2.9 million wheel loadings, respectively. The crushed stone base was covered with only a 3.5 in. (89 mm) thick surfacing of asphalt concrete binder. These results clearly show that crushed stone base sections having thin asphalt concrete surfacings can successfully withstand large numbers of heavy loadings. These findings are supported by the Lake Wales, Florida test pavement results. At Lake Wales a 1.5 in. (38 mm) asphalt concrete surfacing underlain by varying thicknesses of limerock base withstood 2.6 million load repetitions.

2. The test results for the conditions of this study indicate the crushed stone base was more resistant to rutting than the asphalt concrete at a similar depth in the pavement structure. Admittedly, the asphalt concrete was probably marginally acceptable, and had a higher asphalt content than optimum. Nevertheless other studies also indicate that under favorable conditions crushed stone can be less susceptible to rutting than asphalt concrete.
3. Good crushed stone base course performance is attributed to (1) a uniformly high level of compaction (100% of T-180 density), (2) no segregation of the base, (3) a thin asphalt concrete surfacing, and (4) only 4 to 5% of the base passed the No. 200 sieve.
4. For the variation studied, crushed stone gradation was found not to significantly influence performance. Of significance, however, was the fact that all three gradations had only 4 to 5% passing the No. 200 sieve and a 1 to 2 in. (25 to 51 mm) top maximum size crushed aggregate.
5. Performance of the tests appeared to be controlled more by the quality of the asphalt concrete surfacing than the crushed stone base. In this laboratory study the crushed stone base quality could be closely controlled whereas the quality of the in-place asphalt concrete was considerably more variable.
6. Test results indicate an increase in asphalt content from about 5 to 5.25% approximately doubled the rutting in a layer of modified-B or B binder; optimum was about 4.9-5.0% which is close to the 50 blow Marshall optimum.
7. For the reasonably firm silty sand subgrade investigated, the minimum required structural thickness above the subgrade to limit rutting to about 0.1 in. (2.5 mm) is about 16 in. (406 mm) for 2 million 6.5 kip (29 kN) loadings. The subgrade was compacted to 98% of AASHTO T-99 density at or slightly above optimum moisture; the SPT value was 7 blows per foot.
8. In a well constructed section about one-half of the total permanent deformation occurring in a uniform subgrade developed in the upper 12 in. (305 mm). Further, as the quality of the material in the base and surfacing increased, the relative amount of rutting in the subgrade increased and was as high as 50 to 60% of the total.
9. The crushed stone base sections were just as effective inch for inch as the full-depth asphalt concrete sections in reducing subgrade rutting.
10. For the conditions of the tests, 1.7 in. (46 mm) of crushed stone replaced 1.0 in. (25 mm) of asphalt concrete base. Refer to Chapter VI for a discussion of these results.

11. In the crushed stone base sections about 70% of the total permanent strain occurred in the top of the crushed stone. This finding suggests that improved performance may be obtained by taking special precautions in constructing the upper portion of a deep granular base. More relaxed requirements in the lower portion of the base are not, however, recommended.
12. For pavements having structural numbers less than about 4.5, the AASHTO Interim Design Guide underpredicts the life of both crushed stone base and deep strength sections by factors up to 10 or more (Fig. 112).

Recommendations

The following design recommendations were developed from this study. Refer to Chapter VI for a detailed discussion of these recommendations.

1. Engineered crushed stone base pavements can be constructed that perform as well as deep strength asphalt concrete sections. In the future crushed stone base sections should be considered as a viable alternative to deep strength construction where reasonably good subgrades are present.
2. If pavements are constructed having relatively thin asphalt concrete surfacings and deep crushed stone bases, their use should be gradually phased in. During construction of the early deep granular base pavements a concentrated effort should be made to develop better techniques to minimize segregation and obtain high, uniform densities. This knowledge should then be incorporated into subsequent projects.
3. In design 1 in. (25 mm) of asphalt concrete base should be replaced by 1.7 in. (51 mm) of engineered crushed stone base for pavement thicknesses less than 15 in. (381 mm). For an asphalt concrete AASHTO Base course coefficient of 0.30, the corresponding crushed stone base coefficient is 0.18 using the substitution ratio of 1.7.
4. At the present time, a maximum crushed stone base thickness is recommended of 16 to 18 in. (406 - 457 mm). Layered elastic theory adjusted to approximately agree with observed test section response indicates greater depths of crushed stone give little additional benefit. Also, the North Carolina sections which performed well had maximum thicknesses of crushed stone of about this magnitude.
5. An engineered crushed stone base should be covered by the minimum amount of asphalt concrete necessary to insure stability and provide the required structural number. A minimum thickness of asphalt concrete insures maximum flexibility of both the section and the asphalt concrete. For AASHTO structural numbers varying from 4 to 6 asphalt

concrete surfacing thicknesses are recommended varying from 3.5 to 8 in. (89 - 203 mm). Tentative recommendations are given in Table 20. Both the Georgia Tech tests and the North Carolina studies indicate use of deep crushed stone bases and a substitution ratio of 1.7 is justified. Of course to insure satisfactory performance the other recommendations given below should also be followed.

6. A structural section sufficiently thick must be used to protect the subgrade. In general a crushed stone base structural section designed by the AASHTO Interim Guide will provide adequate protection. Table 23 and Fig. 113 provide a guide for required thickness of pavement to protect the a fair silty sandy subgrade not excessively wet.
7. Well constructed crushed stone base or deep strength sections having AASHTO structural numbers less than about 4.5 can withstand considerably greater numbers of load repetitions than predicted by the AASHTO Interim Design Guide. A conservative correction to the design guide results is given by equation (1) and in Fig. 112. Therefore, lighter, more flexible pavement sections perform extremely well.
This finding suggests the alternative approach of designing a relatively light section to withstand the design traffic using equation (1) and Figs. 111 and 112 as a guide. If the pavement does not last the full design life an overlay can always be placed. Although somewhat similar to stage construction, this approach differs significantly in basic design philosophy: in the proposed approach the section is designed to withstand the full traffic loading using an admittedly light, flexible section, while in stage construction the section is purposely underdesigned. This approach should be tried on a limited basis on carefully selected projects such as temporary roads. Certainly the outstanding performance of the light limerock sections at Lake Wales, Florida supports this approach. Using a relatively light, flexible section certainly has considerable economic advantages over a heavy, less flexible one.
8. Proper base construction is considered to be the single most important factor in achieving a high level of performance from a crushed stone base. The construction operation should be conducted such that segregation is minimized and a uniform density is achieved equal to or greater than 100% of AASHTO T-180. Pugmilling of the stone is recommended; stockpiling of the aggregate at the site should not be permitted. Further the recommendation is made that the base should not be left exposed for more than 7 working days.
9. Excellent performance was obtained from the base courses which had the relatively narrow range in gradations shown in Fig. 115. This stone was well-graded and had a 1.0 to 2 in. (25 - 51 mm) top size and 4 to 5% fines. From these results a base course

stone should have the minimum amount of fines practical with 8 to 9% being the upper allowable limit. The top size should be at least 1 to 2 in. (25 - 51 mm).

More research is needed concerning the most effective maximum top size. Some research indicates a larger top size gives better performance. In this study the finer gradation base section had a longer fatigue life than the coarse gradation section. Rutting however was less in the coarse gradation base section.

10. Special attention should be given to the field inspection program for a crushed stone base. Careful, thorough inspection of the base course on a full-time basis is required by a senior inspector. Further, a bonus/penalty system might be implemented to encourage uniform, high density of the crushed stone base. It should be remembered that the weaker 10 to 15% of the area will determine the performance of the pavement.
11. The recommendation is also made that the desirability of proof-rolling the base with a 50 ton pneumatic-tired roller be studied in the field. Proof-rolling is common practice in the construction of military airfields. Proof-rolling would help to detect localized weak areas that could easily be missed by density testing.
12. All subgrades should be proof-rolled by a loaded dump truck (a loaded pan would be an alternative). All weak areas should, if possible, be undercut and replaced with compacted, high quality structural fill or crushed stone. Further, the inspector should carefully watch for locally weak spots by observing loaded pans and other heavy equipment during subgrade construction in other areas.
13. If weak subgrades are present that cannot be undercut, a deep granular base crushed stone section should not be used. Instead, a deep strength asphalt concrete section, or inverted or composite cement stabilized subbase section should be considered. Excellent performance was obtained from the inverted sections studied. Theory and experience in Virginia indicate that an asphalt concrete-cement stabilized subgrade or crushed stone subbase section will give good performance.

Additional Research

Use of crushed stone bases together with relatively thin asphalt concrete surfacings has the potential for significant cost savings over deep strength sections. To achieve maximum economy without a sacrifice in performance, optimum thicknesses of asphalt concrete and crushed stone base must be combined for given traffic loadings and subgrade conditions. At the present time an understanding is just beginning to be developed of the complex behavior of both asphalt concrete and crushed stone bases. To take full economic advantage of crushed stone base sections the following additional research needs to be performed:

1. The effect of asphalt concrete thickness on performance desperately needs to be studied. Field experience in Georgia suggests that a greater asphalt concrete thickness does not necessarily mean better performance. On the other hand, the results of the Georgia Tech, Florida and North Carolina studies show that relatively thin asphalt concrete surfacings can give excellent performance.

A study is needed of the performance of full-scale sections having different asphalt concrete thicknesses. Such a study could be achieved by (1) investigating existing pavements or (2) constructing in the field (or laboratory) full-scale test pavements and test them to failure. These pavements could be tested to failure by either natural truck traffic or else using the loading systems developed in this study. Weigh station pavements and parking areas are very suitable for such full-scale tests.

2. Improved techniques need to be studied for constructing crushed stone bases including: (1) minimizing segregation, (2) obtaining a high density, and (3) proof-rolling. Also utilization of 2 to 3 in. (51-76 mm) top size crushed stone (such as macadam) in base course construction also needs to be investigated in both the laboratory and field.
3. Finally, the results of the past research studies conducted by Georgia Tech on crushed stone, asphalt concrete and pavement performance need to be integrated together into a workable flexible pavement design procedure.

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APPENDIX A

COMPUTER STUDY OF LAYERED SYSTEMS

COMPUTER STUDY OF LAYERED SYSTEMS

Introduction

A multi-layer computer program was used to perform a sensitivity study of selected pavement designs utilizing materials similar to those used in the full-scale study. Material parameters were used in the theoretical analysis that gave theoretical deflection, strain and stress responses of the test sections approximately comparable to those observed. A trial and error iterative procedure was used to determine the "best fit" material properties. Laboratory evaluated material properties were first used in the theory, and the material parameters were then modified until reasonable agreement was obtained with the observed response. This calibration was thus employed to develop realistic values for the material parameters. These parameters were used to analyze a wide range of pavement designs, and hence extended at least approximately the test section results to other conditions.

The CHEV5L program was used in this study [32]. This nonlinear, elastic layered program calculates the complete stress-strain and vertical displacement response within a five-layer pavement system. The five layer system consists of four upper layers of finite thickness and infinite horizontal extent resting on a semi-infinite half space. Each layer was assumed to be isotropic, but the program can handle nonlinear material properties. The loading was assumed to be circular, exerting a uniform pressure on the surface. The effect of dual wheel loads can readily be obtained by superposition.

Calibration Study

To develop realistic material properties that gave computed results in good agreement with measured values, 27 computer runs were made. Vertical stress at the top of the subgrade, surface displacement, and tangential strain at the bottom of the asphalt layers were used to assess the accuracy of each calibration run. It was found to be relatively simple to adjust the material property parameters to give good agreement with measured values of surface displacement and tangential strain at the bottom of the asphalt concrete. Computed vertical strains at the top of the subgrade were usually substantially less than the measured values, with better agreement being obtained for the inverted sections (Table A-1).

Several thicknesses of asphalt concrete and crushed stone base were used in the calibration. Therefore it was possible to approximately determine how the modulus of elasticity varied with layer thickness. The best match material properties are shown in Figure A-1. Fig. A-1 indicates that the average modulus of the crushed stone layer becomes slightly greater with thickness (about 15,000 psi for a 6 in. thick base and 16,000 for a 12 in. stone base). In contrast the average modulus of the asphalt concrete appeared to decrease significantly with thickness (525,000 psi for a 6.5 in. thick AC surfacing to 400,000 psi for 9 in.). The elastic moduli for the other materials were considered to be constant with thickness and are given in Table A-2.

Theoretical Analysis of Pavement Designs

A parametric study involving 48 computer runs was performed to determine the sensitivity of surface deflection, vertical stress at the top of the subgrade and tangential tensile strain at the bottom of the asphalt concrete to the structural pavement design. A poor and good subgrade support condition was assumed for each structural design. The "poor" subgrade was assumed to have a modulus of elasticity of 2,800 psi, and the "good" subgrade a modulus of elasticity of 5,600 psi. This subgrade was assumed to be very deep. The pavement was subjected to a dual wheel load of 9,000 lb. with a tire pressure of 85 psi.

The inverted sections were assumed to have cement stabilized subbase. Separate analyses were not made for the soil cement subbase since theoretically little difference was found between the two type subbases.

The results for crushed stone base pavements are shown in Figs. A-2 through A-7, for full-depth asphalt concrete pavements in Figs. A-8 through A-10, and for inverted sections in Figs. A-11 through A-16.

Discussion of Results

At the present time the recommendation is made that the results given in Figures A-2 through A-16 should be used for *general* comparisons of similar type designs. In other words the figures give an indication of the effect of increasing the thickness of the crushed stone base, increasing the thickness of asphalt concrete or changing the stiffness of the subgrade for similar type construction. The figures should not be used to compare crushed stone base designs with full-depth asphalt concrete designs.

Crushed Stone Base. Tangential tensile strain at bottom of asphalt decreases with increasing thickness of asphalt concrete and base. Increasing thickness of asphalt concrete is by far the most significant effect. Subgrade quality affects tensile strain mainly when the asphalt concrete layer is thin; for example, when the asphalt is 3.5 in. (90 mm) thick, tensile strains are increased by about 40% when going from a good to a poor subgrade. However, for asphalt concrete thicknesses of 6.5 in. (165 mm) and 9 in. (230 mm), tensile strains in the asphalt concrete are essentially unchanged for the two values of subgrade modulus.

The vertical stress on subgrade decreases with increasing thickness of asphalt concrete and base. Subgrade quality has a significant effect with vertical stresses on the good subgrade being about 25% higher than on the poor subgrade. Maximum surface deflection decreases with increasing thickness of asphalt concrete and base. Subgrade quality has a major effect with poor subgrades showing deflections about 50% higher than good ones.

Full-Depth Asphalt Concrete. The tangential tensile strain at the bottom of the asphalt concrete decreases with increasing thickness of asphalt concrete and improving subgrade quality. About 0.7 in. (18 mm) of additional asphalt concrete thickness compensates the tangential strain for a change from a "good" to "poor" subgrade. The vertical stress at the top of the subgrade decreases with increasing thickness of asphalt concrete and increases with improving subgrade quality. About 5 in. (125 mm) of additional asphalt concrete thickness compensates for a change from "good" to "poor" subgrade.

Maximum surface deflection decreases with increasing asphalt concrete thickness and with improving subgrade quality. About 11 in. (280 mm) of additional asphalt concrete compensates when going from a "good" to a "poor" subgrade.

Inverted Section. The tangential tensile strain at bottom of asphalt for base thicknesses greater than about 4 in. (100 mm) only decreases very slightly for increasing thicknesses of either the cement treated subbase or the crushed stone base. As the base thickness decreases, however, tensile strain decreases greatly reaching a minimum value at zero base thickness. Based on these theoretical results, using a crushed stone base between the two treated materials does not reduce the tensile strain in the asphalt concrete. Presence of the crushed stone would help to reduce and/or prolong development of reflection cracking on the surfacing.

Use of the cement stabilized subbase does cause an important reduction in stress on the subgrade for all designs. Vertical stresses on the "good" subgrade are about 50% higher than on the "poor" subgrade. Maximum surface deflection decreases with increasing stabilized subbase thickness, and with increasing crushed stone base thickness. However, a crushed stone base thickness of about 3 in. (75 mm) is a critical value: below that the base thickness deflections remain either approximately constant or actually decrease somewhat with decreasing thickness. Of course crushed stone base thicknesses less than about 6 in. (150 mm) are not practical.

TABLE A-1. COMPARISON OF MEASURED AND THEORETICAL STRESSES, STRAINS AND DEFLECTIONS USING FINAL MATERIAL PARAMETERS.

Pavement Design	Vertical Stress on Subgrade (psi)		Tangential Strain at Bottom of AC ($\mu\epsilon$)		Surface Displacement (mil)	
	Typical Measured	Theoretical	Typical Measured	Theoretical	Typical Measured	Theoretical
9' (235 mm) Full-Depth AC	8.7	2.5	308	280	14	17
3.5" (90 mm) AC on 8" (200 mm) Crushed Stone Base	6.8 - 11.2	4.0	270-390	352	23	29
3.5" (90 mm) AC on 8.7" (220 mm) Crushed Stone Base on 5.8" (147 mm) Cement Treated Crusher Run	3.5	2.1	272	262	17	15
3.7" (94 mm) AC on 8.9" (226 mm) Crushed Stone Base on 6.0" (150 mm) Cement Treated Subgrade	3.2	2.5	324	276	26	19

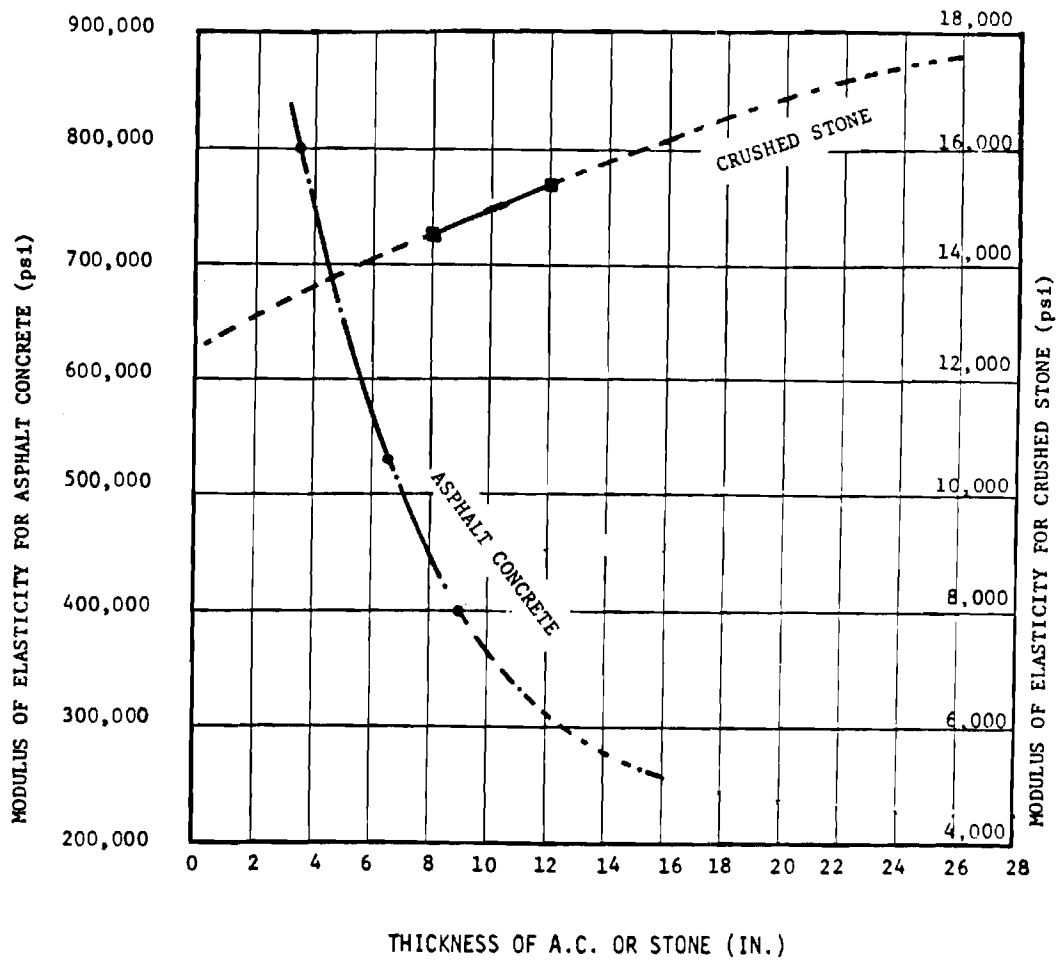


FIGURE A.1. VARIATION OF MODULUS OF ELASTICITY WITH DEPTH FOR CRUSHED STONE AND ASPHALT CONCRETE.

TABLE A-2. MATERIAL PROPERTIES.

Material	In-Situ Unit Wt.		Poisson's Ratio	Modulus of Elasticity	
	(pcf)	(kg/m ³)		(psi)	(MPa)
Asphalt Concrete	145	2,345	.3	See Figure A	
Crushed Stone Base	140	2,265	.35	See Figure A	
Subgrade	125	2,020	.35	2,800	19.29
Cement Treated Crusher Run	145	2,345	.2	100,000	689.0
Soil Cement	126	2,040	.2	15,000	103.4

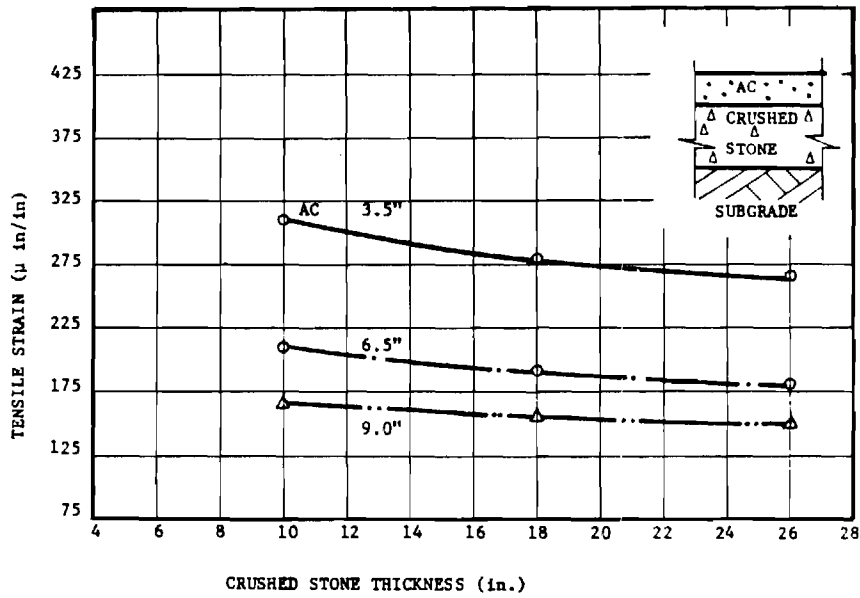


FIGURE A.2. CRUSHED STONE BASE: GOOD SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF BASE AND SURFACING THICKNESS.

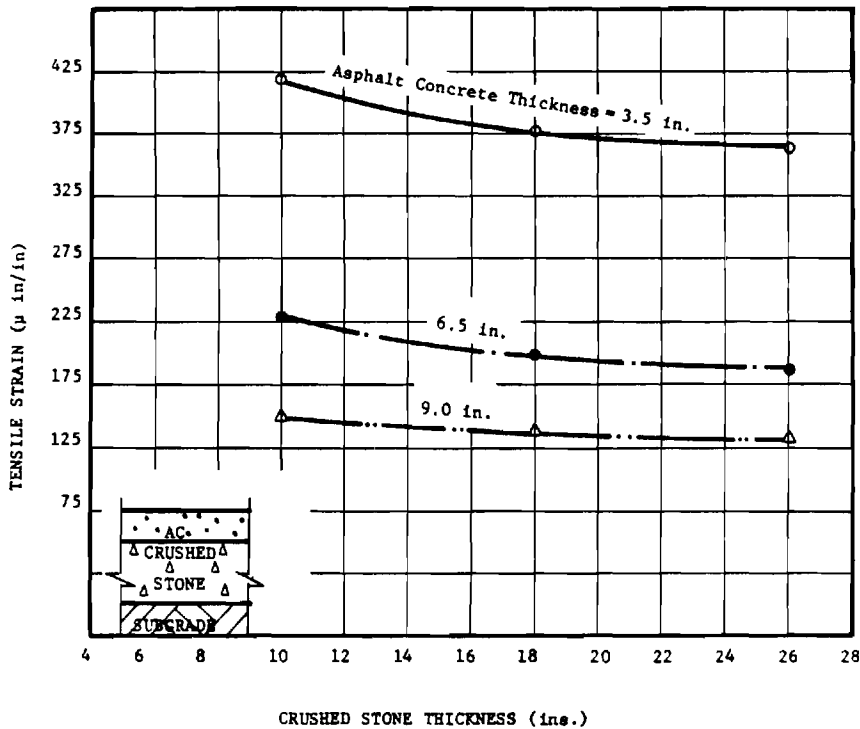


FIGURE A.3. CRUSHED STONE BASE: POOR SUBGRADE. TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF BASE AND SURFACING THICKNESS.

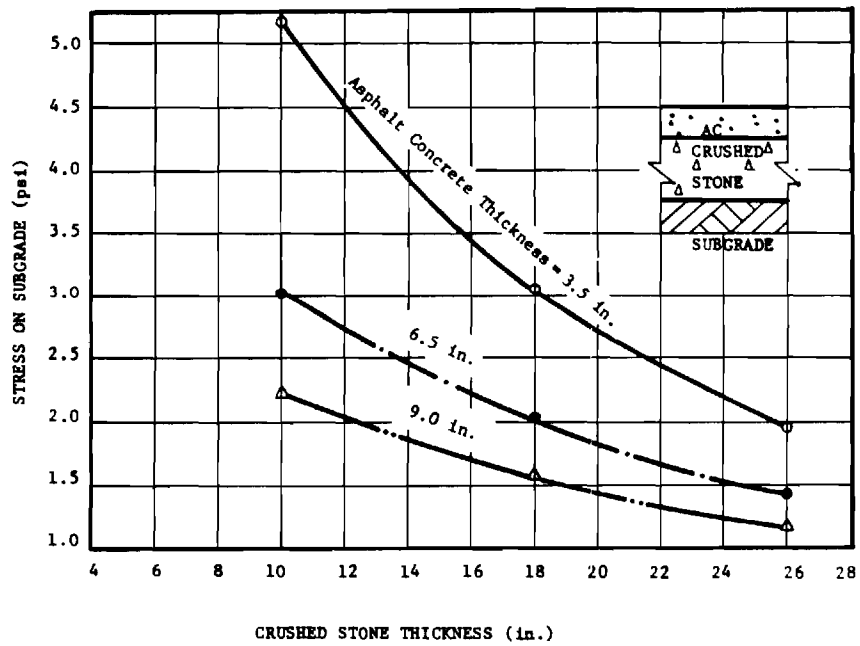


FIGURE A.4. CRUSHED STONE BASE: GOOD SUBGRADE-VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF BASE AND SURFACING THICKNESS.

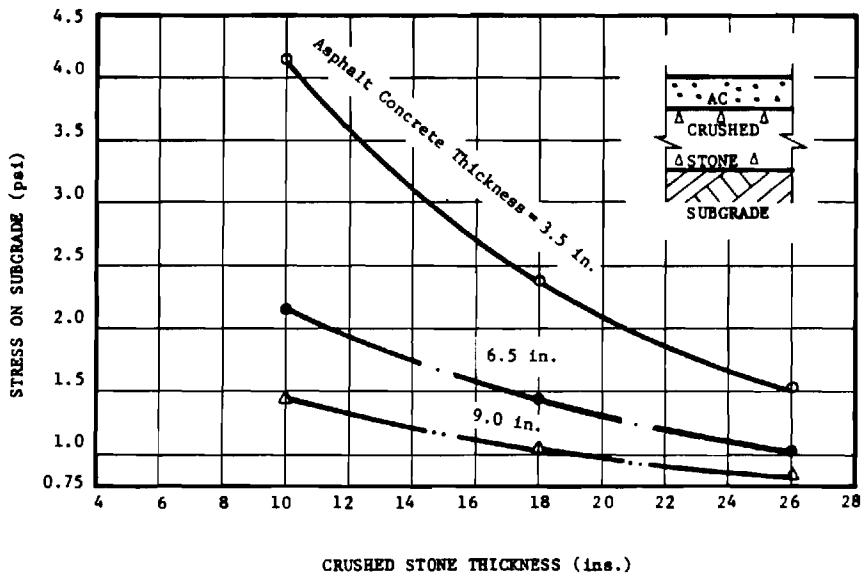


FIGURE A.5. CRUSHED STONE BASE: POOR SUBGRADE - VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF BASE AND SURFACING THICKNESS.

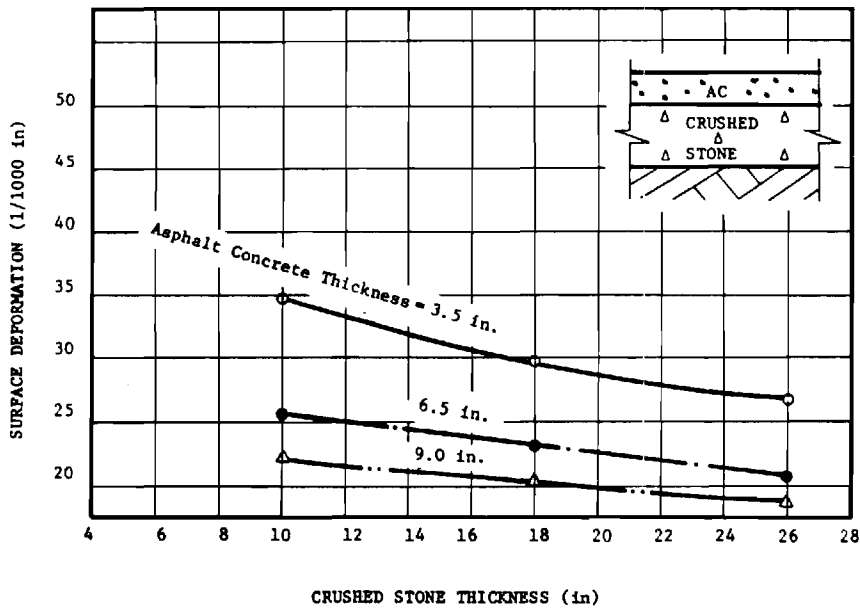


FIGURE A.6. CRUSHED STONE BASE: GOOD SUBGRADE-MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF BASE AND SURFACE THICKNESS.

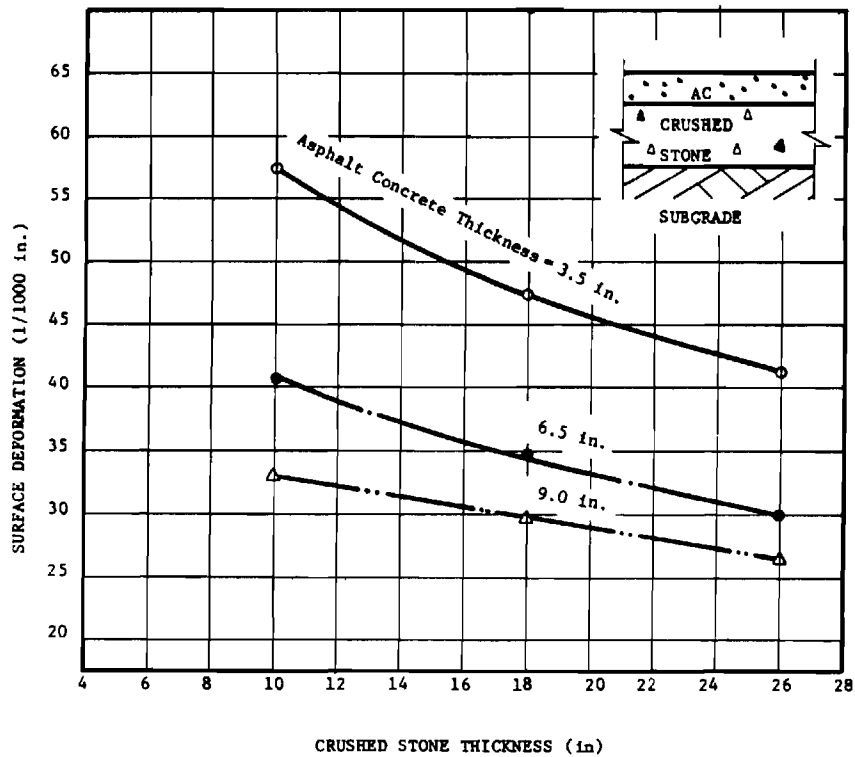


FIGURE A.7. CRUSHED STONE BASE: POOR SUBGRADE-MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF BASE AND SURFACING THICKNESS.

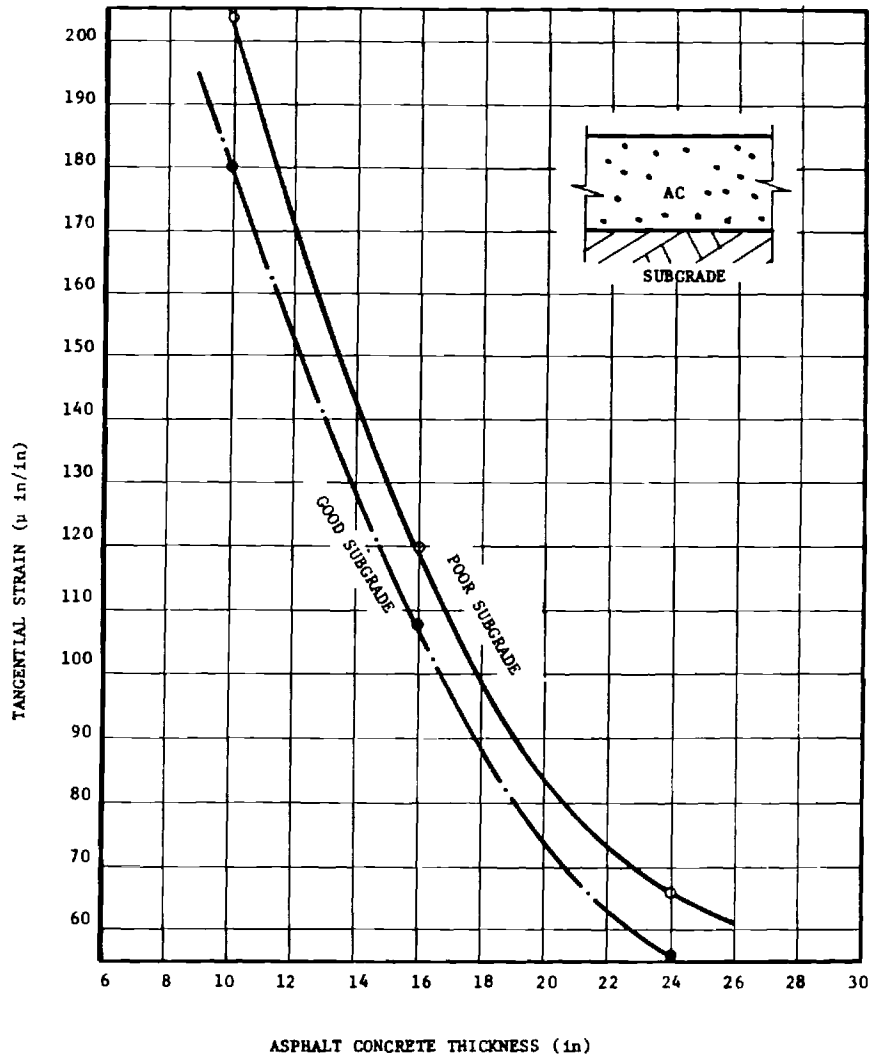


FIGURE A.8. FULL-DEPTH ASPHALT: TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY.

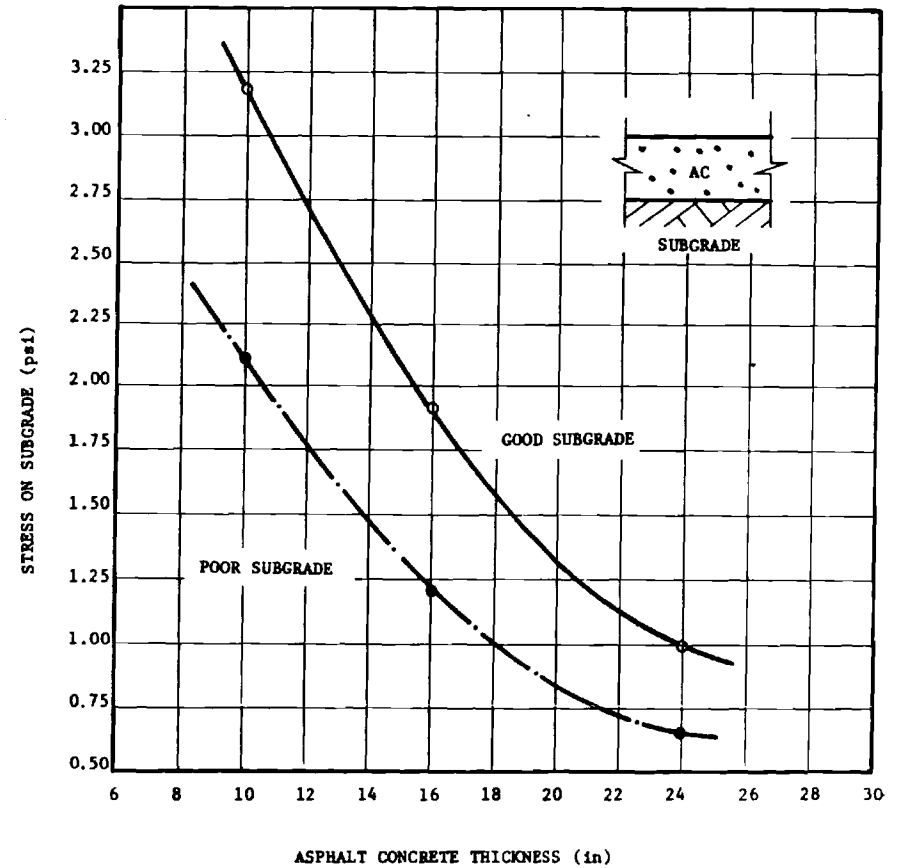


FIGURE A.9. FULL-DEPTH ASPHALT: VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY.

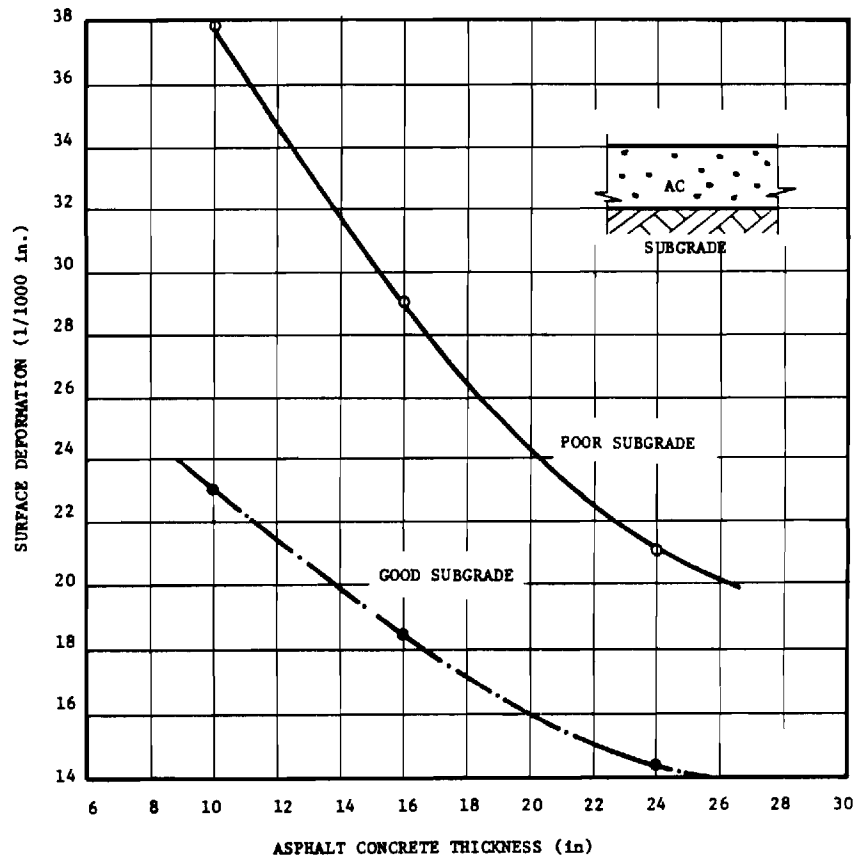


FIGURE A.10. FULL-DEPTH ASPHALT: MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF ASPHALT CONCRETE THICKNESS AND SUBGRADE QUALITY.

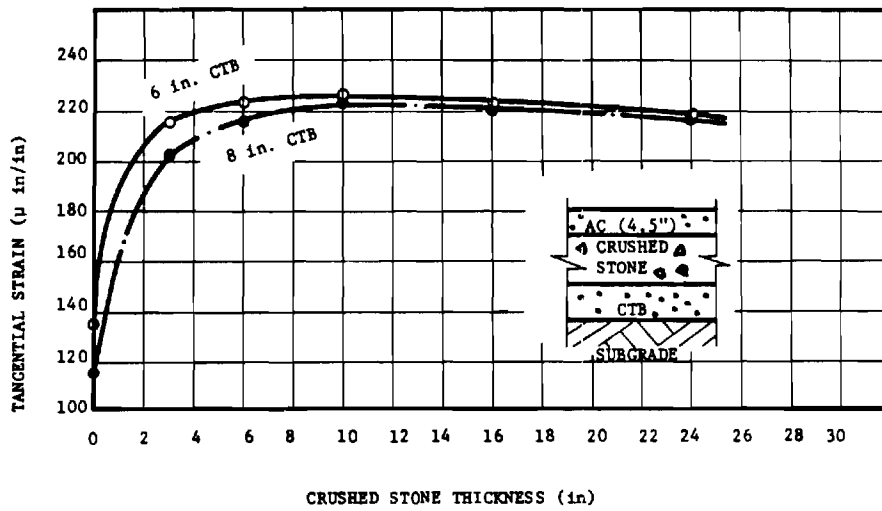


FIGURE A.11. INVERTED CONSTRUCTION: GOOD SUBGRADE — TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF CRUSHED STONE AND CEMENT TREATED BASE THICKNESS.

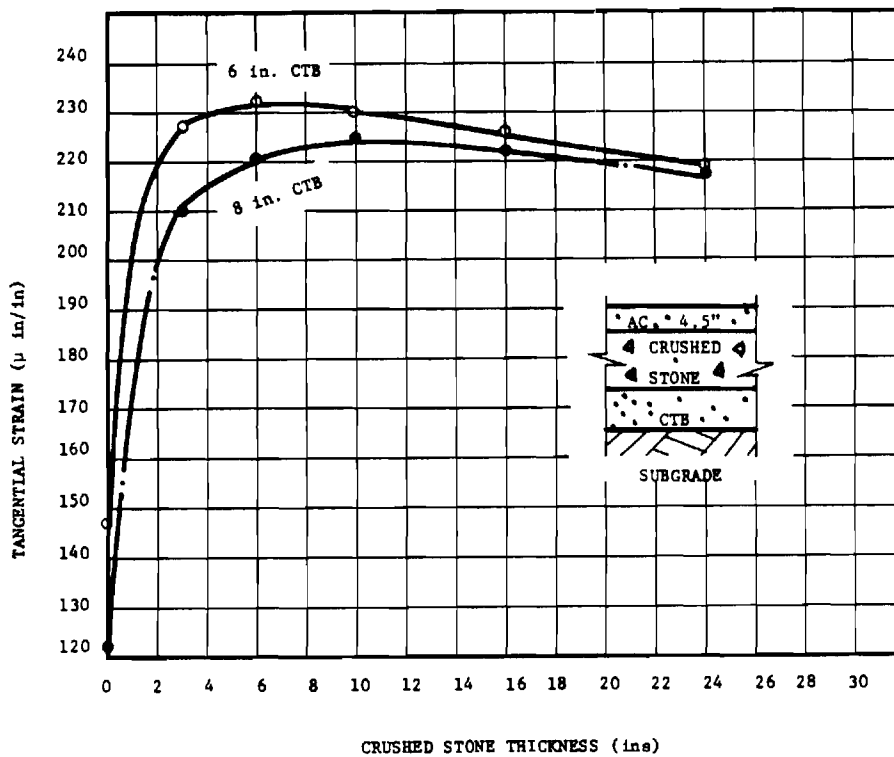


FIGURE A.12. INVERTED CONSTRUCTION: POOR SUBGRADE — TANGENTIAL STRAIN AT BOTTOM OF ASPHALT CONCRETE AS A FUNCTION OF CRUSHED STONE AND CEMENT TREATED BASE THICKNESS.

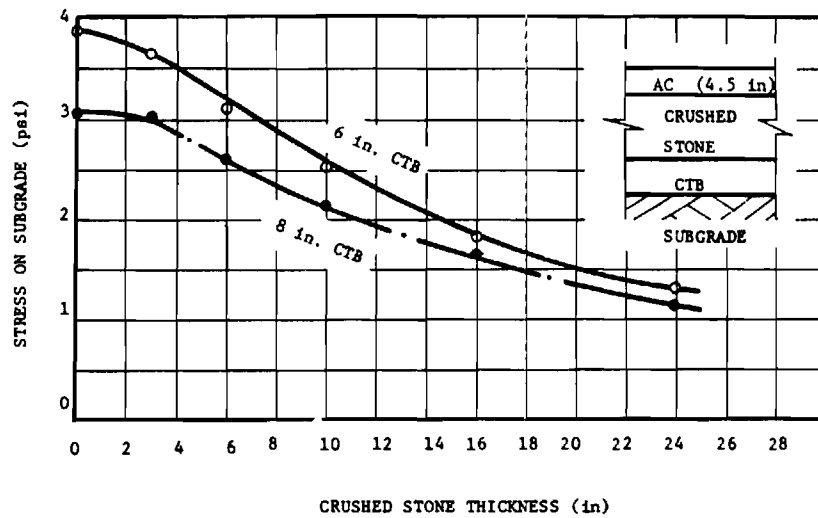


FIGURE A.13. INVERTED CONSTRUCTION: GOOD SUBGRADE — VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS.

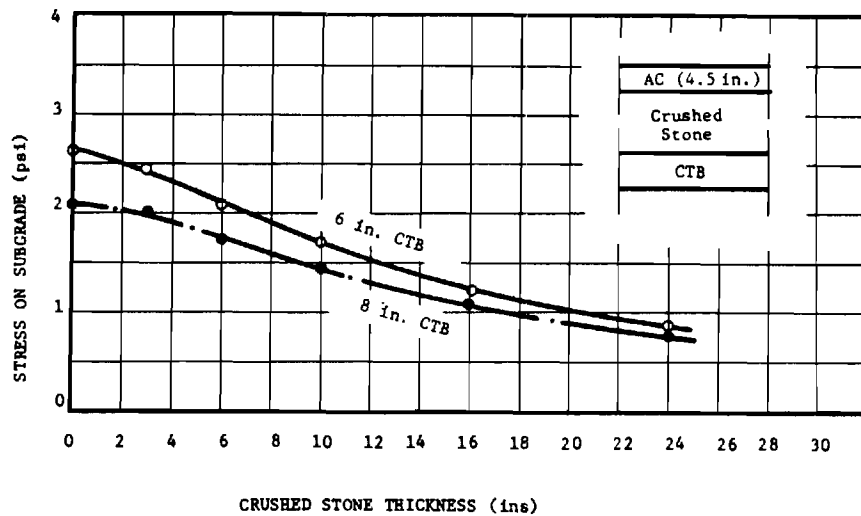


FIGURE A.14. INVERTED CONSTRUCTION: POOR SUBGRADE — VERTICAL STRESS AT TOP OF SUBGRADE AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS.

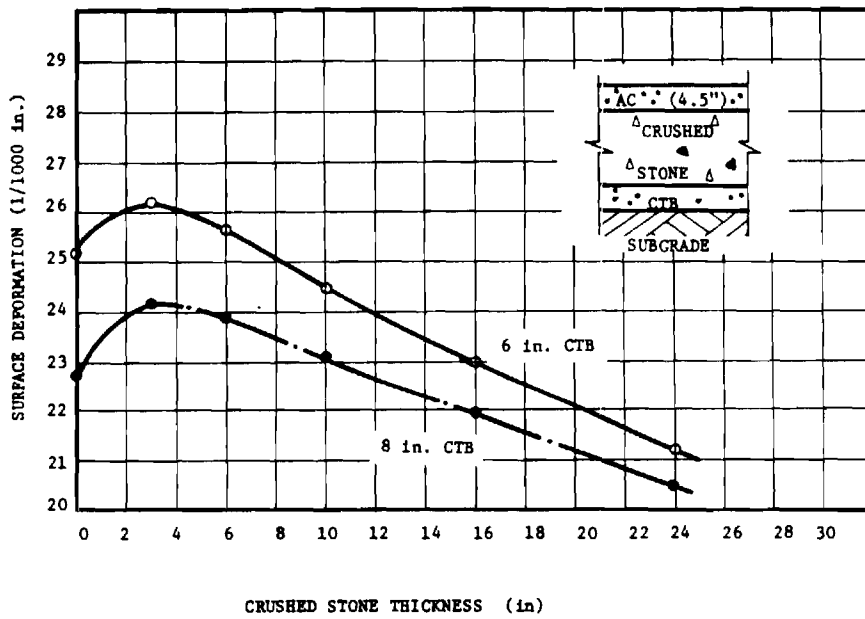


FIGURE A.15. INVERTED CONSTRUCTION: GOOD SUBGRADE-MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS.

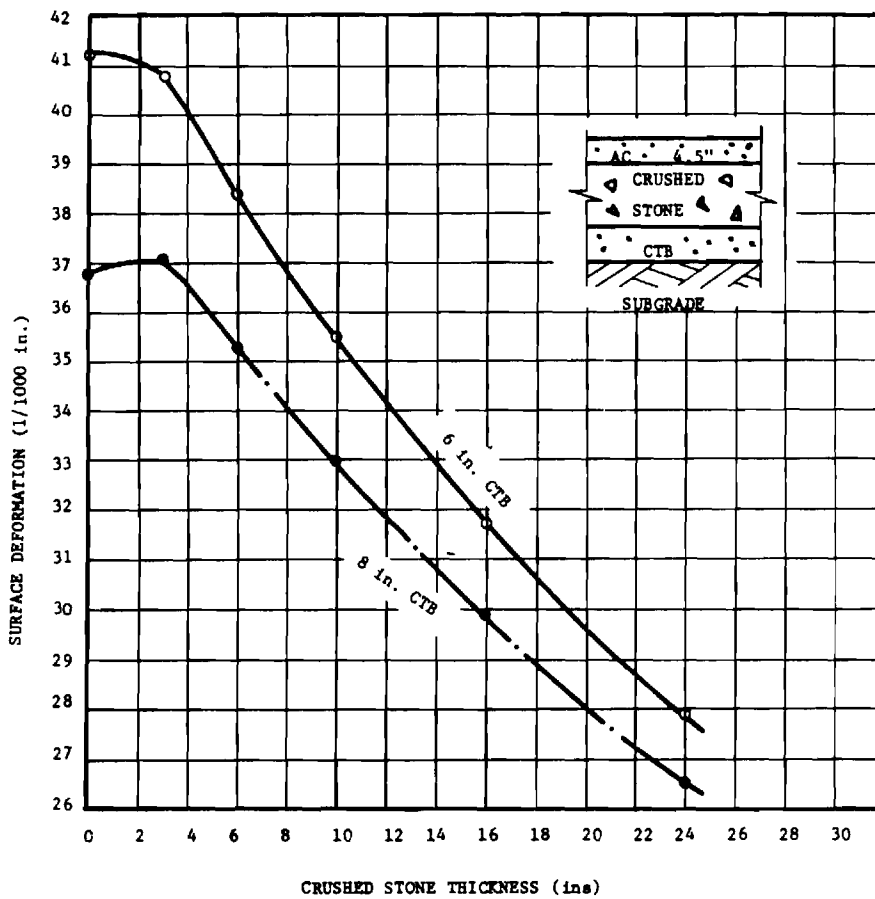


FIGURE A.16. INVERTED CONSTRUCTION: POOR SUBGRADE — MAXIMUM SURFACE DEFORMATION AS A FUNCTION OF CRUSHED STONE AND CTB THICKNESS.