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Report No. FHWA-RD-78-109

TECHNIQUES FOR REHABILITATING PAVEMENTS WITHOUT OVERLAYS – A SYSTEMS ANALYSIS

Vol. 2 Appendixes

September 1977

Final Report

Prepared for

FEDERAL HIGHWAY ADMINISTRATION

Offices of Research & Development

Washington, D. C. 20590

FOREWORD

These reports present the results of research conducted by the Texas Transportation Institute, Texas A&M University, for the Federal Highway Administration (FHWA), Office of Research, under contract DOT-FH-11-9142. This research study was part of FCP Project 5D, "Structural Rehabilitation of Pavement Systems." There are two volumes: Volume 1 describes the analysis used to determine the feasibility of various innovative techniques for rehabilitating pavements without using thick overlays and Volume 2 contains the appendixes of related information.

The research included the development of a systems decision analysis computer program which used utility theory to simultaneously consider seventeen different decision criteria. From this, a total of nineteen techniques demonstrated the capability of solving certain problems better than currently used techniques for rehabilitation.

Copies of the report are being distributed by memorandum to individual researchers. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

NOTICE

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16. Abstract

The objective of the study was to determine the feasibility of a variety of innovative techniques for rehabilitating pavements without using thick overlays and to develop these techniques to the point where they could be implemented. A total of 39 potential techniques were identified. To determine their feasibility a systems decision analysis computer program was developed which utilizes utility theory to simutaneously consider 17 different decision criteria under four main attributes (Cost, Performance, Energy and Impact). Using this utility decision analysis program, a total of 19 techniques demonstrated the capability of solving certain problems better than currently used techniques. Included in the 19 promising techniques are a) use of rejuvinating agents for flexible pavements, b) horizontally-bored sleeper slab and joint restoration for rigid pavements, c) precast joint assemblies for rigid pavements, d) change the location of lane markings, and e) reworked surface of flexible pavement.

For other specific problems the program indicated presently used techniques are better than any of the potential techniques. And finally, the program revealed that 15 potential techniques did not show any promise at this time. Suggestions for further development, implementation, and research have been made.

The report is presented in two Volumes. Vol. 1 contains the analysis and Vol. 2 contains the appendixes. Vol. 1 is Report No. FHWA RD-78-108 and Vol. 2 is Report No. FHWA RD-78-109.

17. Key Words

Pavement, Rehabilitation, Repair, Patching, Systems Analysis, Flexible, Rigid, Joints, Prestressing

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PREFACE

This report summarizes the results obtained from a research project conducted for the Structures and Applied Mechanics Division, Office of Research, Federal Highway Administration, under Contract No. DOT-FH-9142.

The work was conducted by a team of professional researchers and staff personnel of the Texas Transportation Institute. William B. Ledbetter and Robert L. Lytton served as co-principal investigators. The report was organized and compiled by William B. Ledbetter. Primary responsibility for preparation of the various chapters and appendixes were:

Chapter and Appendix	Responsible Person
1, 14	Entire Team
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App. I	N. F. Rhodes

The report is published in two volumes. Volume 1 contains the analysis (chapters 1 through 15) while volume 2 contains the appendixes (A through I).

Valuable supportive direction was given by Donald Ader, who performed most of the computer programming; D. J. Teague, who analyzed data and assisted in computer work, Lynette Kuykendall who keypunched and verified data; and Doris Christensen, Barbara Hodge and Loretta Rother, who typed the rough drafts of the manuscript. Final typing and publication of the report was under the direction of Louis J. Horn, Associate Research Editor for the Institute.

Technical supervision and management was under the direction of Mr. Richard A. McComb of the Federal Highway Administration. Mr. McComb's reviews and suggestions were instrumental in keeping the project headed toward meeting the objectives.

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APPENDIX A. ANALYSIS OF STATE QUESTIONNAIRES ON PAVEMENT MAINTNEANCE AND REHABILITATION

Introduction

To gain pertinent pavement maintenance and rehabilitation information from all of the state transportation agencies a questionnaire was developed and, through the auspices of the ASSHTO operating subcommittee on maintenance, sent to each state's maintenance engineer. Ideas were solicited for new and innovative techniques for rehabilitating pavements. Those received were incorporated into the project and evaluated. Additionally several items of information were solicited which would be valuable in the conduct of the research project. These items are summarized in the following paragraphs.

Survey of Highway Surface Types

Each state was polled to find out what percentage of each classification of highway (interstate, primary, etc.,) was rigid, flexible, or other (Table A-1). Note the wide variation between states (as expected) and the overall average values; 54% rigid and 46% flexible on interstate, 28% rigid and 72% flexible on primary, 10% rigid and 81% flexible on secondary. This attests to the general belief that flexible type pavements can be constructed more economically than rigid pavements and thus are more widely used on the lower highway types.

Survey of Cost Information

Three questions were posed; what is your average construction cost per lane mile (1.6 km), what is your state maintenance budget, and what is your average annual maintenance cost per lane mile (1.6 km)? The results are tabulated in Tables A-2, A-3, and A-4. Here are exhibited very wide differences in costs between states. For example, on the interstate average construction costs vary from as little as $100,000 per lane mile (1.6 km) in Ohio to as high as $2,500,000 per lane mile (1.6 km) in Delaware, with an average cost of $892,000 per lane mile (1.6 km). The average maintenance budget for the states is $41,000,000 (Table A-3) and the average main-

Table A-1. Tabulation of Highway Surfaces of the different States by Highway Classification and by Pavement Type

	Interstate		Primary		Secondary			Farm to Market		
	Rigid	Flex	Rigid	Flex	Rigid	Flex	Other	Rigid	Flex	Other
Alabama	36	64	1	99	1	99	0	0	100	0
Alaska	0	0	0	100	0	50	50	0	10	90
Arizona	7	93	1	99	1	99	0	1	99	0
Arkansas	100	0	65	35	6	94	0	0	100	0
California	62	38	29	71	2	98	0	0	100	0
Colorado	50	50	2	98	0	100	0	0	0	0
Connecticut	37	63	64	36	94	6	0	0	0	0
Delaware	100	0	85	15	80	20	0	15	85	0
Florida	27	73	2	98	0	100	0	0	0	0
Georgia	50	50	5	95	1	99	0	1	99	0
Hawaii	40	60	2	98	0	100	0	0	0	0
Idaho	12	88	2	98	0	100	0	0	100	0
Illinois	99	1	96	4	0	0	0	2	98	0
Indiana	90	10	58	42	0	0	0	0	0	0
Iowa	90	10	76	24	1	1	98	4	10	86
Kansas	40	60	9	91	1	49	50	1	2	97
Kentucky	70	30	5	95	0	100	0	0	100	0
Louisiana	93	7	18	82	5	95	0	3	97	0
Maine										
Maryland	61	39	20	80	7	93	0	0	0	0
Massachusetts	1	99	0	100	0	100	0	0	0	0
Michigan	70	30	60	40	75	25	0	0	0	0
Minnesota	60	40	18	82	75	25	0	0	0	0
Mississippi	68	32	19	81	1	99	0	0	0	0
Missouri	99	1	49	51	6	4	90	6	4	90
Montana	10	90	99	1	0	100	0	0	100	0
Nebraska	100	0	14	86	14	86	0	0	0	0
Nevada	7	93	1	99	0	100	0	0	100	0
New Hampshire	0	100	8	92	1	99	0	0	0	0
New Jersey	50	50	30	70	0	0	0	0	0	0
New Mexico	24	76	1	99	0	91	9	0	44	56
New York										
North Carolina										
North Dakota	95	5	98	2	0	100	0	0	100	0
Ohio	97	3	50	50	0	100	0	0	0	0
Oklahoma	30	70	30	70	30	70	0	1	99	0
Oregon	30	70	2	98	1	99	0	0	0	0
Pennsylvania	93	7	84	16	30	70	0	5	95	0
Rhode Island	100	0	23	77	4	90	0	0	0	0
South Carolina	43	57	2	98	1	99	0	1	99	0
South Dakota	90	10	10	90	2	98	0	0	100	0
Tennessee	20	80	2	98	0	0	0	0	0	0
Texas	50	50	20	80	2	98	0	1	99	0
Utah	15	85	1	99	1	99	0	0	100	0
Vermont	0	100	0	100	0	100	0	0	100	0
Virginia	35	65	4	96	0	66	34	0	0	0
Washington	35	65	30	70	0	100	0	0	100	0
West Virginia										
Wisconsin	100	0	27	73	4	96	0	0	0	0
Wyoming	22	78	0	100	0	100	0	0	0	0
Ontario	0	0	11	89	0	62	38	0	27	73
Average Values	54	46	28	72	10	81	9	2	80	18
Standard Deviation	34	34	31	30	23	30	23	3	36	38

Highway Classification; Percent of Highway Surface

Table A-2. Tabulation of Average Construction Cost per Lane Mile* and Percent Annual Increase as Reported by States

	Interstate ($1,000)	Primary ($1,000)	Secondary ($1,000)	Farm - Market ($1,000)	Overall Annual Percent Increase
Alabama	1,175	146	146	30	20
Alaska		250	150	100	10
Arizona	800	212	188	88	8
Arkansas	1,300	425	100	100	6
California	625	375	235	75	8
Colorado					
Connecticut	1,200	1,000	990		6
Delaware	2,500	1,000	400	50	6
Florida	570	219	98		8
Georgia	1,500	400	200	100	8
Hawaii					
Idaho	100	60	30	30	7
Illinois	2,600	750	400	250	7
Indiana	406	360	207	63	30
Iowa					
Kansas	2,000	300			6
Kentucky	1,375	750	400	250	7
Louisiana	250	240		70	
Maine					
Maryland	2,250	1,450	670		15
Massachusetts	500	500	170		10
Michigan	500	300	250		10
Minnesota	825	160	42	25	7
Mississippi	568	360	316		
Missouri	462	300	120	120	5
Montana	250	175	150		10
Nebraska	275	253	111	90	8
Nevada	250	120	15	15	12
New Hampshire	500	375	300	100	8
New Jersey	1,700	1,000	450	320	6
New Mexico	1,750	450	300	50	
New York					
North Carolina					
North Dakota	200	125	75	40	5
Ohio	100	90	40		5
Oklahoma	300	177	83	35	
Oregon	1,500	500	325	250	10
Pennsylvania	1,000	1,000	450	450	9
Rhode Island					
South Carolina	500	100	23	23	5
South Dakota	410	320	110	55	12
Tennessee					
Texas	280	150	102	71	
Utah	1,265	160	100	40	10
Vermont	543	324	159	65	
Virginia	1,500	375	200		5
Washington	467	287	132	55	
West Virginia					
Wisconsin	300	200	75		7
Wyoming	700	280	240	200	10
Ontario					
Average Values	892	391	219	107	8
Standard Deviation	667	310	192	102	3

* 1 mi = 1.6 km

Table A-3. Tabulation of State Maintenance Budget and Percent of Maintenance Budget Expended by Categories

	Maintenance Budget ($1,000,000)	Percent of Maintenance Budget Expended: Roadway Surface	Roadway Shoulder	Roadside and Drainage	Bridges	Traffic Services	Adim.	Other
Alabama	29	24	9	8	18	10	0	31
Alaska								
Arizona	20	21	1	15	1	14	11	37
Arkansas	38	30	4	24	2	4	11	25
California	160	15	3	32	3	17	13	17
Colorado		18	3	12	2	18	2	45
Connecticut	39	18	4	25	5	8	12	26
Delaware	11	30	15	20	10	15	10	0
Florida	72	8	12	30	6	11	28	5
Georgia	40	22	24	15	6	8	2	23
Hawaii	9	17	13	19	8	17	22	4
Idaho	23	38	1	6	2	36	7	0
Illinois	80	22	7	26	10	5	12	18
Indiana	60	13	12	11	3	18	0	43
Iowa	37	9	11	10	4	13	27	26
Kansas	49	35	4	15	6	17	6	17
Kentucky	83	19	5	19	18	14	23	2
Louisiana	65	29	8	17	11	23	8	4
Maine								
Maryland	32	16	16	17	8	16	14	13
Massachusetts	32	4	3	14	4	22	22	31
Michigan	55	7	6	9	5	6	18	49
Minnesota	57	18	8	22	4	16	2	30
Mississippi	21	23	9	33	4	13	10	8
Missouri	100	54	4	11	3	10	5	8
Montana	21	25	24	7	2	33	7	2
Nebraska	24	51	4	9	8	15	3	0
Nevada		41	16	6	1	12	8	15
New Hampshire	14	26	8	19	13	20	0	14
New Jersey	23	53	0	10	6	11	8	12
New Mexico	14	23	5	7	2	11	16	36
New York								
North Carolina								
North Dakota	12	69	3	3	5	10	9	1
Ohio	94	14	7	7	6	7	38	21
Oklahoma	25	17	18	27	4	32	0	3
Oregon	51	14	3	15	8	11	5	44
Pennsylvania	100	26	9	12	2	9	18	24
Rhode Island	10	30	15	5	15	15	20	0
South Carolina	41	17	7	28	5	21	8	22
South Dakota	13	21	22	10	2	10	7	27
Tennessee	42	19	10	34	3	10	16	8
Texas	150	37	10	30	2	21	0	0
Utah	20	37	2	14	1	14	4	27
Vermont	14	18	3	6	7	10	9	47
Virginia	89	18	11	0	6	19	19	33
Washington	47	22	6	21	10	12	7	22
West Virginia								
Wisconsin	34	16	16	21	10	0	7	30
Wyoming	16	38	4	10	2	12	9	2
Ontario	20	22	12	15	4	14	0	33
Average Values	41	24	8	16	6	15	11	20
Standard Deviation	31	14	6	8	4	7	7	14

Table A-4. Tabulation of Average Annual Maintenance Cost per Lane Mile and Percent Annual Increase in Cost per Lane Mile as Reported by the States

	Average Annual Maintenance Cost/Lane Mile	Percent Annual Increase in Cost/Lane Mile
Alabama	$ 1,041	17
Alaska	10,000	
Arizona	1,300	6
Arkansas	1,800	12
California	2,700	8
Colorado	1,650	11
Connecticut	3,140	2
Delaware	700	
Florida	3,300	13
Georgia	1,000	8
Hawaii	3,100	6
Idaho	1,965	7
Illinois	1,800	11
Indiana	2,100	
Iowa	935	9
Kansas	1,700	
Kentucky	1,400	6
Louisiana	1,700	7
Maine		
Maryland	1,800	15
Massachusetts	2,300	10
Michigan	1,841	4
Minnesota	2,000	7
Mississippi	855	6
Missouri	1,400	9
Montana	1,300	10
Nebraska	1,100	8
Nevada	1,184	12
New Hampshire	2,200	18
New Jersey	3,100	9
New Mexico	1,340	15
New York		
North Carolina		
North Dakota	875	5
Ohio	1,386	5
Oklahoma	1,500	
Oregon	2,140	14
Pennsylvania	1,897	10
Rhode Island	2,000	
South Carolina	506	7
South Dakota		
Tennessee	1,525	10
Texas	1,305	6
Utah	1,380	16
Vermont	2,135	11
Virginia	3,350	9
Washington	2,645	6
West Virginia		
Wisconsin	1,400	8
Wyoming	1,100	10
Ontario	1,100	9
Average Values	1,694	9.2
Standard Deviation	728	3.6

*Omitted from average

1 mi = 1.6 km

tenance cost per lane mile (1.6 km) is $1,694. Another interesting statistic generated in these three questions was the average annual increase in the construction costs (8%) and maintenance costs (9.2%). These increases reflect the inflationary trends experienced in the United States during the last few years.

As the cost data varied so widely (standard deviations almost equaling averages) an attempt was made to stratify the data to see if any trends would by established. States were divided into two groups using four methods; (a) according to truck population (1), (b) according to how much maintenance costs were incurred, (c) according to the freeze zone (2), and (d) according to soil type (3). The results of these groupings are given in Table A-5. While the averages did change somewhat according to the method of grouping selected, for the most part the scatter was not significantly reduced. One improved statistic was for maintenance cost per lane mile (1.6 km) according to how much maintenance costs were incurred. Those states spending less than the overall average maintenance costs ($1,694) per lane mile (1.6 km) had an average cost of $1,057, while those states spending more than the overall average had an avercost of $2,266 per lane mile (1.6 km) or more than twice as much. This dramatically demonstrates the wide divergence in maintenance costs per lane mile (1.6 km) between states. Another improved statistic was the percentage of rigid pavement on the interstates as a function of truck population. Those states with the larger truck population averaged 60% rigid pavement while those states with the smaller truck population averaged only 36% rigid pavement (on the interstate). The remainder of the items analyzed in this manner showed little or no improvement by such stratification (in terms of reduced data scatter). The facts are that factors other than the four utilized here to analyze the data are strongly influencing costs. These other factors would probably include such things as interactive effects of climate and soil, traffic conditions, level of pavement service (very important), support of the state legislatures toward providing highways for the traveling public, and method of accounting for and reporting of maintenance costs. This last reason may explain a great deal of the data scatter as accounting methods vary widely from state-to-state.

Table A-5. Summary of Cost Data from Questionnaire by Truck Population, Maintenance Cost, Climatic Conditions, and Soil Conditions (4)

Major Categories (Units)	Average Values		Twenty five states with larger truck population (1)		Twenty five states with smaller truck population		States spending less than 1800 cost/lane mile (2		States spending more than 1800 cost/lane mile		Non freeze zone		Freeze zone		Widespread to medium expansive clay		Medium to non existant expansive clay	
	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation	Average	Standard Deviation
Maintenance Budget and Categories																		
Budget (Sx106)	41	31	60	32	24	12	36	33	50	33	52	44	27	25	61	40	42	43
Roadway Surface(%)	24	14	21	11	29	15	29	15	21	11	21	7	27	17	22	15	26	14
Roadway Shoulder(%)	8	6	9	5	7	6	10	7	6	4	8	6	8	6	9	7	8	6
Roadside and Drainage(%)	16	8	19	8	15	8	16	9	18	7	22	8	14	6	19	9	16	8
Bridges(%)	6	4	7	5	5	3	6	5	6	3	6	6	6	3	5	5	6	3
Traffic Services(%)	15	7	15	6	15	8	15	7	15	8	15	6	15	8	17	10	15	6
Administration(%)	11	7	13	8	10	5	10	7	11	7	14	7	9	6	15	15	10	7
Other(%)	20	14	22	15	20	15	17	13	24	16	17	13	23	15	17	14	23	14
Maintenance Cost																		
Cost/Lane Mile($)	1694	720	1724	785	1732	781	1057	393	2266	10	1623	927	1800	620	1487	546	1808	816
Annual Increase(%)	9	4	10	4	9	3	9	3	568	4	9	4	9	4	7	4	11	7
Road Surfaces																		
Interstate																		
Rigid(%)	54	33	60	28	36	37	52	35	50	28	51	28	48	29	51	34	50	28
Flex(%)	46	34	40	28	64	33	48	36	50	29	49	27	52	33	49	37	50	33
Primary																		
Rigid(%)	28	31	26	27	26	34	26	34	24	28	30	27	24	33	60	32	47	34
Flex(%)	72	30	74	25	74	38	74	34	76	27	70	29	76	39	40	28	53	38
Secondary																		
Rigid(%)	10	24	12	22	10	26	9	19	15	28	15	25	13	27	10	20	10	26
Flex(%)	81	30	66	40	87	27	78	37	79	31	70	38	75	42	78	38	74	27
Other(%)	9	23	22	32	3	11	13	29	6	16	15	34	12	25	12	18	16	11
Farm-Market																		
Rigid(%)	2	3	5	2	1	4	2	4	1	1	9	4	7	5	2	3	3	2
Flex(%)	80	46	69	43	86	29	73	40	78	41	63	36	70	29	74	39	79	40
Other(%)	18	38	26	37	13	29	25	36	21	41	28	39	23	38	24	38	18	41
Construction Cost/Lane Mile																		
Interstate	950	717	780	647	932	710	802	653	792	627	1078	703	797	658	772	767	966	657
Primary	405	306	323	188	448	374	298	236	467	348	426	360	387	282	382	224	410	340
Secondary	230	198	179	108	253	230	172	114	273	238	231	167	230	217	192	124	238	216
Farm-Market	163	82	95	77	101	84	84	64	116	94	79	60	137	122	127	88	108	111
% Increase	8	5	10	7	8	3	9	4	9	6	8	5	9	5	9	10	7	4

1 mi = 1.6 km

Summary of Distress Data for Flexible Pavements

The states were polled concerning the types of distress experienced on their various highway classifications. Their detailed responses, in terms of the most-prevalent and second-most-prevalent, are tabulated in Tables A-6 and A-7. A summary of the most prevalent and the second most prevalent distress types are given in Tables A-8 and A-9. Table A-10 summarizes the combined (most) prevalent and second-most prevalent) distress types occurring in flexible pavements. On interstate and primary highways rutting is the most frequently occurring distress type, followed closely by cracking. Thus potential rehabilitation techniques, to be of widespread general applicability, should address the problems of rutting and cracking.

Summary of Distress Data for Rigid Pavements

In the same manner as discussed previously the states were questioned concerning distress types on their rigid pavements. Their detailed responses are tabulated in Tables A-11 and A-12, and summarized in Tables A-13, A-14, and A-15. From Table A-15 the most frequently occurring distress conditions are roughness and spalled joints. If spalled joints and spalling (which might have been confusing) are combined, then spalling would be the most frequently occurring distress condition. Other frequently occurring distress conditions involve joints in most cases (faulting, failed joint, and pumping). Thus the joint problem is seen as a most serious one and rehabilitation techniques are needed to solve this vexing problem in a more satisfactory manner.

Development and Construction Costs for Good Rehabilitation Techniques

To gain some insight on the maximum permissible development and construction costs that could be tolerated for a "good" rehabilitation technique, the states were asked their "opinion" as to what they thought (a) the maximum development cost a contractor would be willing to gamble on and (b) the maximum construction cost per lane mile (1.6 km) a state transportation agency would be willing to pay for a "good" rehabilitation technique. The results of this opinion survey are given in Figures A-1 and A-2. The numbers on each abscissa were given in the questions posed! This was

Table A-6. Tabulation of Most Prevalent Distress Type for Flexible Pavements By Highway Classification

	Distress Type Code by Highway Classification			
	Interstate	Primary	Secondary	Farm-Market
Alabama	LC	LC	LC	LC
Alaska	AC	AC	AC	AC
Arizona	MC	MC	RO	
Arkansas		RA	RA	RA
California	TC	TC	MC	MC
Colorado	RA	RA	RA	RA
Connecticut	MC	MC	MC	
Delaware		LC	MC	AC
Florida	RU	RU	RU	
Georgia	TC	TC	TC	TC
Hawaii	MC	MC	MC	
Idaho	MC	TC	RU	
Illinois			AC	AC
Indiana			TC	RU
Iowa	RU	RU		
Kansas	S	S	S	S
Kentucky	RU	RU	RO	AC
Louisiana	RU	RU	RU	RU
Maine				
Maryland	RU	RU	MC	
Massachusetts	RU	RU	MC	C
Michigan	TC	RU		
Minnesota	TC	MC	MC	
Mississippi	MC	MC	MC	
Missouri	MC	MC	P	P
Montana	FL	MC	MC	
Nebraska		RU	BF	BF
Nevada	RU	RU	AC	AC
New Hampshire	LC	TC	RO	
New Jersey	MC	MC	P	P
New Mexico	FL	FL	FL	P
New York				
North Carolina				
North Dakota	RU	RU	RU	RU
Ohio	LC	LC	LC	
Oklahoma	TC	TC	TC	TC
Oregon	RU	TC	BF	BF
Pennsylvania	P	P	P	P
Rhode Island	LC	LC	LC	
South Carolina	LC	LC	MC	MC
South Dakota	RO	RO	RO	RA
Tennessee	RU	RU		
Texas	TC	TC	AC	P
Utah	RU	RU	RU	RU
Vermont	TC	TC	TC	P
Virginia	RU	MC	MC	AC
Washington	TC	LC	AC	AC
West Virginia				
Wisconsin	RU	TC	MC	
Wyoming	RU	RU	RA	
Ontario		AC	RO	RO

FLEXIBLE PAVEMENT DISTRESS TYPE CODE

TC Transverse Crack
LC Longitudinal Crack
MC Multiple Crack
AC Alligator Crack
RA Raveling
RU Rutting
FL Flushing
RO Roughness
P Patching
BF Base Failure
C Corrugations
S Shrinkage
LS Low Skid Number

Table A-7. Tabulation of Second Most Prevalent Distress Type for Flexible Pavements by Highway Classification

	Distress Type coded by Highway Classification			
	Interstate	Primary	Secondary	Farm-Market
Alabama	RU	RU	RU	RU
Alaska		TC	TC	PA
Arizona	TC	TC	RU	
Arkansas		TC	RU	AC
California	LC	LC	AC	AC
Colorado	RU	RU	RU	RU
Connecticut	AC	AC	AC	
Delaware		RU	AC	RA
Florida	MC	AC	P	
Georgia	LC	LC	LC	LC
Hawaii	RA	RA	RA	
Idaho		FL	FL	FL
Illinois			RU	RU
Indiana			RA	P
Iowa	MC	AC		
Kansas	RU	RU	RU	RU
Kentucky	RO	RO	MC	RO
Louisiana	PH	PH	PH	PH
Maine				
Maryland	MC	RO	RO	
Massachusetts	LC	LC	RA	RO
Michigan	MC	MC		
Minnesota	MC	RA	RA	
Mississippi	RU	RU	RU	
Missouri	TC	TC	RA	RA
Montana	MC	RU	P	
Nebraska		BF	RA	RA
Nevada	LC	LC	LC	
New Hampshire	TC	RO	AC	
New Jersey	RA	P	RO	RO
New Mexico	TC	TC	TC	MC
New York				
North Carolina				
North Dakota	MC	MC	MC	MC
Ohio	AC	AC	AC	
Oklahoma	RU	RU	RU	RU
Oregon	TC	BF	TC	TC
Pennsylvania	RU	RU	RU	RU
Rhode Island				
South Carolina	TC	TC	RA	RA
South Dakota	AC	AC	AC	AC
Tennessee	RA	RA		
Texas	LS	LS	LS	LS
Utah	RO	MC	P	P
Vermont	LC	LC	LC	RA
Virginia	LC	P	P	
Washington	LC	AC	RA	RA
West Virginia				
Wisconsin	RA	RU	RU	
Wyoming	FL	P	P	
Ontario		RU	TC	MC

FLEXIBLE PAVEMENT DISTRESS TYPE CODE

TC Transverse Crack
LC Longitudinal Crack
MC Multiple Crack
AC Alligator Crack
RA Raveling
RU Rutting
FL Flushing
RO Roughness
P Patching
BF Base Failure
C Corrugations
S Shrinkage
LS Low Skid Number

Table A-8. Summary of Most Prevalent Flexible Pavement Distress Type as Reported by State Agencies

Distress Type	Interstate		Primary		Secondary		Farm-Market	
	Highway Classification							
	No.[a]	%	No.[a]	%	No.[a]	%	No.[a]	%
Longitudinal Crack	5	10.0	7	15.9	3	6.4	0	0.0
Alligator Crack	1	2.0	2	4.5	5	10.6	7	25.0
Multiple Crack	7	14.0	9	20.5	14	29.9	2	7.1
Transverse Crack	7	14.0	7	15.9	4	8.5	2	7.1
Raveling	1	2.0	2	4.5	3	6.4	3	10.7
Rutting	14	28.0	13	29.5	6	12.8	5	17.8
Flushing	2	4.0	1	2.3	1	2.1	0	0.0
Roughness	1	2.0	1	2.3	5	10.6	0	0.0
Patching	10	20.0	1	2.3	3	6.4	5	17.9
Base Failure	0	0.0	0	0.0	2	4.2	2	7.2
Corrugations	0	0.0	0	0.0	0	0.0	1	3.6
Shrinkage	2	4.0	1	2.3	1	2.1	1	3.6
Total	50	100.0	45	100.0	47	100.0	28	100.0

[a]Number of states naming the indicated distress type as the most prevalent.

Table A-9. Summary of Second Most Prevalent Flexible Pavement Distress Type as Reported By State Agencies

Distress Type	Highway Classification							
	Interstate		Primary		Secondary		Farm-Market	
	No.[a]	%	No.[a]	%	No.[a]	%	No.[a]	%
Longitudinal Crack	8	21.7	3	7.7	3	7.3	1	3.6
Alligator Crack	2	5.4	5	12.9	5	12.2	3	10.7
Multiple Crack	8	21.7	4	10.2	3	7.3	3	10.7
Transverse Crack	5	13.4	7	18.0	4	9.8	1	3.5
Raveling	4	10.8	3	7.7	7	17.1	7	25.1
Rutting	6	16.2	10	5.7	9	22.0	6	21.4
Shrinkage	0	0.0	0	0.0	0	0.0	0	0.0
Flushing	1	2.7	1	2.5	1	2.4	1	3.6
Roughness	2	5.4	3	7.7	3	7.3	3	10.7
Pot Hole	1	2.7	1	2.5	1	2.4	1	3.6
Base Failure	0	0.0	2	5.1	0	0.0	0	0.0
Totals	37	100.0	39	100.0	36	100.0	26	100.0

[a]Number of states naming the indicated distress type as the second most prevalent.

Table A-10. Summary of Combined Flexible Pavement Distress Type as Reported by State Agencies

Distress Type	Interstate No.[a]	Interstate %	Primary No.[a]	Primary %	Secondary No.[a]	Secondary %	Farm-Market No.[a]	Farm-Market %
Longitudinal Crack	13	14.9	10	11.6	6	6.8	1	1.9
Alligator Crack	3	3.4	7	8.1	10	11.4	10	18.5
Multiple Crack	15	17.2	13	15.2	17	19.3	5	9.2
Transverse Crack	12	13.8	14	16.2	8	9.1	3	5.5
Raveling	5	5.7	5	5.8	10	11.4	8	14.8
Rutting	20	23.2	23	26.7	15	17.0	11	20.3
Flushing	3	3.4	2	2.4	2	2.3	1	1.9
Roughness	3	3.4	4	4.6	8	9.1	3	5.5
Patching	10	11.5	4	4.6	8	9.1	7	12.9
Base Failure	0	0.0	2	2.4	2	2.3	2	3.8
Corrugations	0	0.0	0	0.0	0	0.0	1	1.9
Shrinkage	2	2.3	1	1.2	1	1.1	1	1.9
Pot Hole	1	1.2	1	1.4	1	1.1	1	1.9
Totals	87	100.0	74	100.0	79	100.0	54	100.0

(Highway Classification: Interstate, Primary, Secondary, Farm-Market)

[a]Number of states naming the indicated distress type as either the most prevalent or second most prevalent.

Table A-11. Tabulation of Most Prevalent Distress Type for Rigid Pavements by Highway Classification

	Distress Type Code by Highway Classification			
	Interstate	Primary	Secondary	Farm-Market
Alabama	LC	LC		LC
Alaska				
Arizona	RO	RO		
Arkansas	PU	PU	SD	
California	FA	FA	TC	TC
Colorado	SC	SC		
Connecticut	RO	RO	RO	
Delaware	TC	SJ	SJ	
Florida	PU	FA		
Georgia	FA	FA		
Hawaii				
Idaho	FA	LC		
Illinois	FJ	FJ		
Indiana	FA	FA	FA	
Iowa	SJ	FA		
Kansas	SP	SP	SP	
Kentucky	JD	JD		
Louisiana	BU	BU	BU	BU
Maine				
Maryland	FJ	FJ	FJ	
Massachusetts	BU	BU	MC	
Michigan	FJ	FJ		
Minnesota	FJ	SJ		
Mississippi	SD	SD	SD	
Missouri	RO	RO	RO	RO
Montana	SD			
Nebraska	SJ	SJ	LC	
Nevada	SP	SP		
New Hampshire		RO	RO	
New Jersey	CP	SD	SD	P
New Mexico	PU	PU		
New York				
North Carolina				
North Dakota	SJ	FA		
Ohio	SJ	SJ		
Oklahoma	SJ	SJ	SJ	SJ
Oregon	C	BF	BF	BF
Pennsylvania	SD	SD	SD	SD
Rhode Island	BU	BU	BU	
South Carolina	FA	LC	LC	LC
South Dakota	FJ	FJ		
Tennessee	RO	RO		
Texas	MC	MC	CP	CP
Utah	SP	RO	RO	
Vermont				
Virginia	RO	FA		
Washington	FA	FA	FA	FA
West Virginia				
Wisconsin	SP	SP	FA	
Wyoming	SJ			
Ontario		TC		

RIGID PAVEMENT DISTRESS TYPE CODE

RO Roughness
PU Pumping
FJ Failed Joint
SC Surface Crack
SJ Spalled Joint
TC Transverse Crack
C Corrugations
FA Faulting
SP Spalling
JD Joint Deterioration
SD Surface Deterioration
CP Cracked Panel
P Patching
LC Longitudinal Crack
MC Multiple Crack
G Grooving
BU Blowups

Table A-12. Tabulation of Second Most Prevalent Distress Type for Rigid Pavements By Highway Classification

	Distress Type Code by Highway Classification			
	Interstate	Primary	Secondary	Farm-Market
Alabama	SJ	SJ		
Alaska				
Arizona	FA			
Arkansas	SJ	SP	FA	
California	TC	TC	SJ	SJ
Colorado	RO	RO		
Connecticut	SJ	SJ	SJ	
Delaware	SJ	SD	SD	
Florida	FA	SP		
Georgia	PU	PU		
Hawaii				
Idaho	LC	FA		
Illinois	FA	BU		
Indiana	PU	PU	PU	
Iowa	SD	SJ		
Kansas				
Kentucky	CP	CP		
Louisiana	SP	SP	SP	SP
Maine				
Maryland	RO	RO	RO	
Massachusetts	TC	TC	FJ	
Michigan	TC	TC		
Minnesota	SD	RO		
Mississippi	TC	TC	TC	
Missouri	PU	PU	PU	PU
Montana	FA			
Nebraska	G	G	SJ	
Nevada	SJ	SJ		
New Hampshire		SP	SP	
New Jersey	SD	P	RO	RO
New Mexico	MC	MC		
New York				
North Carolina				
North Dakota	FA	LC		
Ohio	PU	PU		
Oklahoma	LC	LC	LC	LC
Oregon	FJ	LC	LC	LC
Pennsylvania	RO	RO	RO	RO
Rhode Island				
South Carolina	SP	SD	SD	SD
South Dakota	SJ	SJ		
Tennessee	SP	SP		
Texas				
Utah	RO	CP	CP	
Vermont				
Virginia	PU	BU	SJ	
Washington	MC	MC	LC	LC
West Virginia				
Wisconsin	BU	BU	RO	
Wyoming	FJ			
Ontario		FJ		

RIGID PAVEMENT DISTRESS TYPE CODE

RO Roughness
PU Pumping
FJ Failed Joint
SC Surface Crack
SJ Spalled Joint
TC Transverse Crack
C Corrugations
FA Faulting
SP Spalling
JD Joint Deterioration
SD Surface Deterioration
CP Cracked Panel
P Patching
LC Longitudinal Crack
MC Multiple Crack
G Grooving

Table A-13. Summary of Most Prevalent Rigid Pavement Distress Type as Reported by State Agencies

Distress Type	Interstate		Primary		Secondary		Farm-Market	
	No.[a]	%	No.[a]	%	No.[a]	%	No.[a]	%
Roughness	5	11.9	5	11.3	4	18.1	1	10.0
Pumping	4	9.5	4	9.5	0	0.0	0	0.0
Failed Joint	5	11.8	4	9.5	1	4.5	0	0.0
Surface Crack	1	2.4	1	2.4	0	0.0	0	0.0
Spalled Joint	6	14.3	6	14.1	2	9.1	0	0.0
Transverse Crack	1	2.4	1	2.3	0	0.0	1	10.0
Corrugations	1	2.4	0	0.0	0	0.0	1	10.0
Faulting	5	11.9	7	16.5	2	9.1	0	0.0
Spalling	4	9.5	2	4.7	1	4.5	0	0.0
Joint Deterioration	1	2.4	1	2.3	0	0.0	0	0.0
Blow Up	2	4.8	2	4.7	2	9.1	1	10.0
Surface Deterioration	3	7.1	3	9.1	4	18.1	1	10.0
Cracked Panels	1	2.4	0	0.0	1	4.5	1	10.0
Patching	1	2.4	0	0.0	0	0.0	1	10.0
Longitudinal Crack	1	2.4	3	9.1	2	9.1	1	10.0
Multiple Crack	1	2.4	1	2.4	1	4.5	1	10.0
Base Failure	0	0.0	1	2.4	1	4.5	1	10.0
Grooving	0	0.0	0	0.0	1	4.5	0	0.0
Total	42	100.0	41	100.0	22	100.0	10	100.0

Highway Classification: Interstate, Primary, Secondary, Farm-Market.

[a]Number of states naming the indicated distress type as the most prevalent.

Table A-14. Summary of Second Most Prevalent Rigid Pavement Distress Type as Reported by State Agencies

Distress Type	Interstate		Primary		Secondary		Farm-Market	
	Highway Classification							
	No.[a]	%	No.[a]	%	No.[a]	%	No.[a]	%
Roughness	4	11.7	4	11.3	4	20.0	2	20.0
Pumping	4	11.7	3	8.6	1	5.0	1	10.0
Failed Joint	2	5.9	1	2.9	1	5.0	0	0.0
Surface Crack	0	0.0	0	0.0	0	0.0	0	0.0
Spalled Joints	4	11.7	3	8.6	3	15.0	1	10.0
Transverse Crack	4	11.7	4	11.3	1	5.0	0	0.0
Corrugations	0	0.0	0	0.0	0	0.0	0	0.0
Faulting	5	14.7	1	2.9	1	5.0	0	0.0
Spalling	3	8.9	5	14.3	2	10.0	1	10.0
Joint Deterioration	0	0.0	0	0.0	0	0.0	0	0.0
Blow Ups	1	3.0	3	8.6	0	0.0	0	0.0
Surface Deterioration	3	8.9	2	5.7	2	10.0	1	10.0
Cracked Panel	1	3.0	2	5.7	1	5.0	0	0.0
Patch	0	0.0	1	2.9	0	0.0	0	0.0
Longitudinal Crack	2	5.9	3	8.6	3	15.0	3	30.0
Multiple Crack	2	5.9	2	5.7	0	0.0	0	0.0
Base Failure	0	0.0	0	0.0	0	0.0	0	0.0
Grooving	1	3.0	1	2.9	1	5.0	1	10.0
Total	34	100.0	35	100.0	20	100.0	10	100.0

[a]Number of states naming the indicated distress type as the second most prevalent.

Table A-15. Summary of Combined Rigid Pavement Distress Type as Reported by State Agencies

Distress Type	Interstate		Primary		Secondary		Farm-Market	
	No.[a]	%	No.[a]	%	No.[a]	%	No.[a]	%
Roughness	9	11.5	9	11.8	8	20.0	3	18.9
Pumping	8	10.3	7	9.3	1	2.5	1	6.2
Failed Joint	7	9.0	5	6.6	2	5.0	0	0.0
Surface Crack	1	1.3	1	1.3	0	0.0	0	0.0
Spalled Joint	10	12.8	9	11.8	5	12.5	1	6.2
Transverse Crack	5	6.4	5	6.6	1	2.5	1	6.2
Corrugations	1	1.3	0	0.0	0	0.0	1	6.2
Faulting	10	12.8	8	10.5	3	7.5	0	0.0
Spalling	7	9.0	7	9.3	3	7.5	1	6.2
Joint Deterioration	1	1.3	1	1.3	0	0.0	0	0.0
Surface Deterioration	6	7.7	5	6.6	6	15.0	2	12.7
Cracked Panel	2	2.6	2	2.6	1	2.5	0	0.0
Patching	1	1.3	1	1.3	0	0.0	1	6.2
Longitudinal Crack	3	3.8	6	7.9	6	15.0	3	18.8
Multiple Crack	3	3.8	3	3.9	1	2.5	1	6.2
Grooving	1	1.3	1	1.3	0	0.0	0	0.0
Blow Up	3	3.8	5	6.6	2	5.0	1	6.2
Base Failure	0	0.0	1	1.3	1	2.5	0	0.0
Total	78	100.0	71	100.0	40	100.0	16	100.0

Highway Classification: Interstate, Primary, Secondary, Farm-Market.

[a]Number of states naming the indicated distress type as either the most prevalent or the second most prevalent.

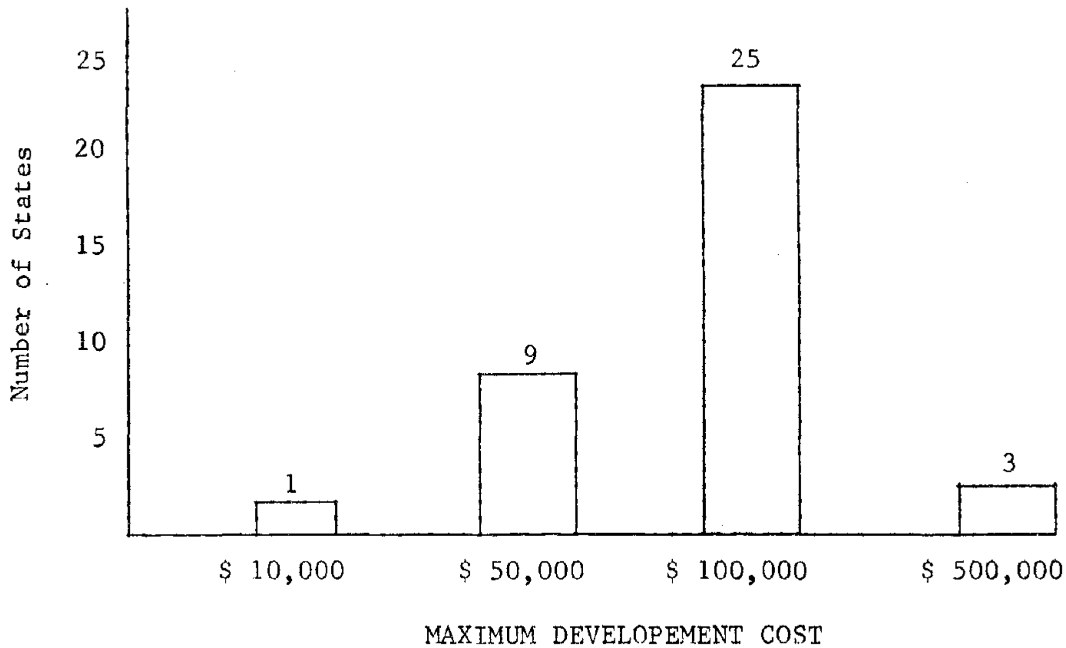

Figure A-1. States Opinion on Maximum Developement Cost a Contractor Would Pay for Rehabilitation Equipment.

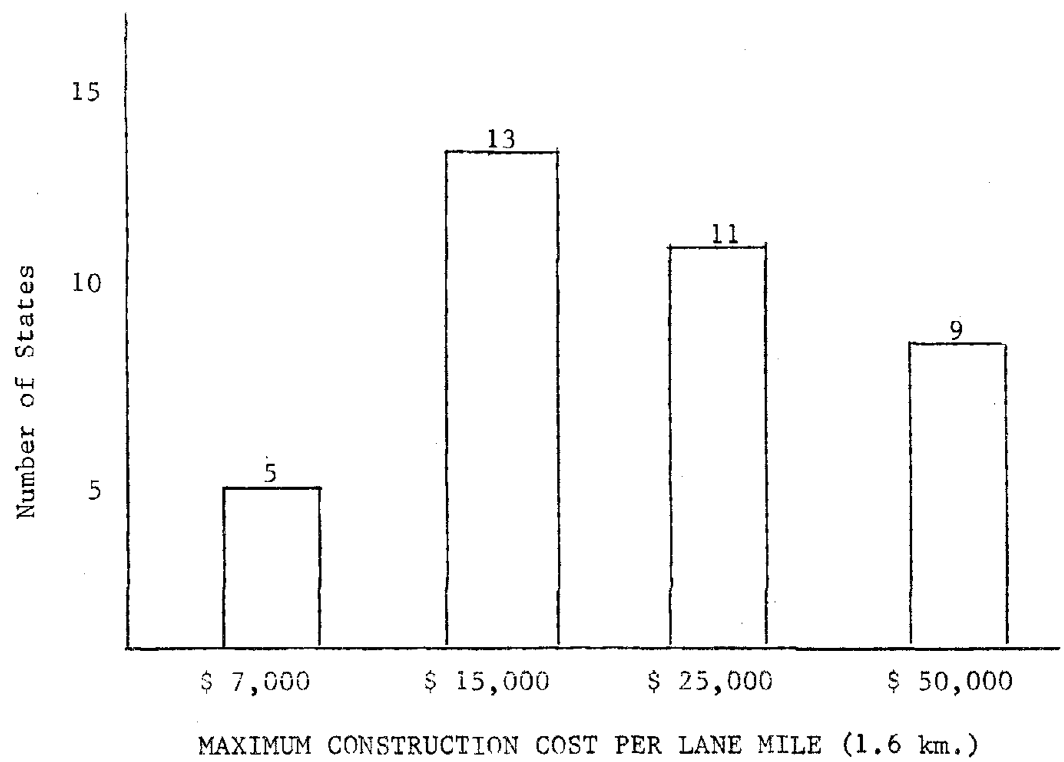

Figure A-2. States Opinion on Maximum Construction Cost Per Lane Mile for Good Rehabilitation Technique.

done to provide some guidance and help with data analysis. Unfortunately they also assuredly influenced the opinions - perhaps to the point where the results are meaningless. What is the upper limit on development cost for a new piece of rehabilitation equipment? Perhaps, the answer is obvious; not so high that the investor does not have a good chance of receiving a minimum attractive rate of return on his investment!

The amount of capital involved - in itself - is probably not a major consideration. If a need exists and a profit can be made, capital generally will be forthcoming. But an acceptable rate-of-return is required! Also, as the future is often almost impossible to predict with sufficient accuracy to justify the risk, the pay back period for the investment becomes an important consideration. The higher the development cost, probably the longer the pay back period and thus the greater the risk. From this aspect, development cost is an important consideration.

With all these disclaimers then, most state officials believe (no doubt influenced by our numbers) development costs in excess of $100,000 for a piece of rehabilitation equipment would be too high.

Even though suggested numbers were supplied for maximum construction costs, the data should reflect the vast experience of the responders. Here a wide variety of opinions exist with 9 states feeling that $50,000 per lane mile (1.6 km) could be justified for a top technique that would restore the pavement to its original condition, while 29 states felt $50,000 would be too high. This gives some valuable guidance on the value of potential techniques that involve major construction costs.

Closure

The questionnaire resulted in some extremely valuable information in this important area of pavement maintenance and rehabilitation. Practices differ widely throughout the United States as they are influenced by many different and extremely complex factors. This analysis shows the extent and nature of the rehabilitation problem in the United States and indicates the vast amounts of money that are required to maintain our investment in our nationwide network of highways.

APPENDIX B. REHABILITATION CONCEPT INDEX AND DISPOSITION

The 92 rehabilitation concepts considered in this project are listed in Table B-1. Each concept was given a numerical classification number. To make every effort to obtain all potential ideas, every person contacted was urged to submit any idea he might have, no matter how implausible he might feel it to be. Furthermore, no mention was made of the constraints on the projects limiting the scope of the investigation. As a result, some 30 ideas were considered to be either outside the scope of the project or too implausible to be considered further. The specific reasons for dropping each of these 30 ideas are enumerated following Table B-1.

Table B-1. REHABILITATION CONCEPT INDEX AND DISPOSITION

Classification Number	Concept	Disposition
101	Sulfur Injected Edge Beam	Long Edge Strengthening
102	Injection of Bentonite/Kerosene Mix	Discarded
103	Injection of High Viscosity Fluid Between PCC and Base	Stab. Sublayers in Place
104	Injection of Silicate Material	Stab. Sublayers in Place
105	Underfilling Joints With Sealant	Stab. Sublayers in Place
106	Drying/Sealing With Microwave	Surf. Rehab. of Flex. Pvts.
107	Microwave Heating With Layer of Absorbent Material in Pavement Structure	Surf. Rehab. of Flex. Pvts.
108	Use Polysulfide Foam as Joint Filler	Repair and Replacement of Joints
109	Replace Existing Joints With Waterstop System	Repair and Replacement of Joints
110	Pressure Injection of Sulfur into Subbase	Discarded
111	Use Gas to Drive Moisture From Subbase	Discarded
112	Install Pavement Side Drains to Remove Water From Subbase	Discarded
113	Use Cross Linkable Hydrocarbon to Remove Moisture from Subbase	Discarded
114	Injection of Expansive Foam to Drive Moisture from Subbase	Discarded
115	Injection of Silicone Rubber Between Subbase and Pavement to Form Continuous Moisture Barrier	Discarded

B-1 cont.

Classification Number	Concept	Disposition
116	Drying of Pavement Structure by Microwave	Discarded
117	Injection of Fluid Material into Void Area in Subbase	Stab. Sublayers in Place
118	Preconstruction Moisture Proofing Subbase at Proposed Joints	Discarded
119	Heavy Rolling to Increase Density of Surface, Subbase or Subgrade	Surf. Rehab. of Flex. Pvt.
120	Reworking Surface	Surface Rehab. of Flex. Pvt.
121	Heating Asphaltic Pavement Surface	Surf. Rehab. of Flex. Pvt.
122	Use Rejuvenating Agents to Restore Properties of Pavement Surface	Surf. Rehab. of Flex. Pvt.
123	Remove Moisture from Sub-grade by Electro-Osmosis	Stab. Sublayers in Place
124	Post Hole Piles	Stab. Sublayers in Place
125	Construct Additional Relief Joints	Repair and Replacement of Joints
126	Seal Joints in Pavement	Repair and Replacement of Joints
127	Replace Pavement	Use of Precast Elements
128	Adhesive Injection into Subbase and Subgrade	Stab. Sublayers in Place
129	Break Up Rigid Pavement and Inject Adhesive	Crack Repair of Rigid Pvt.
130	Heating and Planing Flexible Pavement	Surface Rehab. of Flex. Pvt.
131	Injection of Subsealing Material into Selected Pavement Layer	Stab. Sublayers in Place

B-1 cont.

Classification Number	Concept	Disposition
132	Pressure Grout Void Areas in Subbase or Subgrade	Stab. Sublayers in Place
133	Use Ultrasonics to Densify Pavement Structure	Surf. Rehab. of Flex. Pvt.
134	Patching and Glueing	Localized Rehab. Tech.
135	Freezing Flexible Pavement Surface	Discarded
136	Replace Pavement Surface	Discarded
137	Vibrate Pavement to Increase Density	Surf. Rehab. of Flex. Pvt.
138	Shave Pavement Surface	Surf. Rehab. of Flex Pvt.
139	Irradiation of Asphaltic	Surf. Rehab. of Flex Pvt.
140	Stabilization of Pavement Structure	Stab. of Sublayers in Place
141	Seal the Surface of Broken Pavement	Surf. Rehab. of Flex Pvt.
142	Prestress Existing Pavement	Prestressing Existing Rigid Pvt.
143	Joint Repair by Welding or Installing Flat Jacks	Discarded
144	Seal Cracks in Rigid or Flexible Pavements	Crack Repair of Rigid Pvt.
145	Install Key Lock Joints in Rigid Pavements	Repair and Repl. of Joints
146	Repair Holes with a Screw in Plug Patch	Discarded
147	Use Skewed Joints	Discarded
148	Replace Joints with Post Tensioned Precast Unit	Prestressing Existing Rigid Pvt.
149	Use on Aramid Material in Patching	Localized Rehab. Tech.

Table B-1 cont.

Classification Number	Concept	Disposition
150	Use a Shape Charge to Remove or Break Up Paving	Discarded
151	Use Water Repellent Materials in the Pavement Structure	Discarded
152	Replace Joint Sealant with Self Leveling Material	Repair and Repl. of Joints
153	Use Rubber or Polyvinyl Chloride Water Stop in Joint Repair	Repair and Repl. of Joints
154	Use Chemically Prestressed Concrete	Discarded
155	Use Zero Slump Concrete, Vibrator-Rolled as Subbase Surface with Asphaltic Concrete	Discarded
156	Reverse the Direction of Cross Slope	Geometric Revisions
157	Use of Lignin to Rejuvinate Asphaltic Concrete Pavement and Seal Cracks	Surf. Rehab. of Flex. Pvt.
158	Use of Petroset AT and Rock Binder	Surf. Rehab. of Flex. Pvt.
159	Use Polymer Impregnated Concrete in Patching or Slab Replacement	Surf. Rehab. of Flex. Pvt.
160	Saw Pavement Joints at an Angle When Repairing Failed Joints	Discarded
161	Spray and Place Kevlar into Rutted Wheel Path or Other Pavement Depressions, Then Level Pavement Surface	Discarded
162	Use Polyester Resins With Aggregate as Patching Materials	Surf. Restoration of Rigid Pvt.
163	Replace Local Concrete Failure Due to Subgrade, with Prestressed Unit	Prestressing Existing Rigid Pvt

Table B-1 cont.

Classification Number	Concept	Disposition
164	Use Epoxy Asphalt as Patching Material	Localized Rehab. Tech.
165	Use European Asphalt Roofing Material that is Welded to Roof	Localized Rehab. Tech.
166	Use Polyurethane Sealants for Joints	Repair and Repl. of Joints
167	Use of High Absorbing Cornstarch Type Material to Dry Up Wet Soils	Discarded
168	Pick Up Segments of Pavements, Repair Segment and Replace Pavement	Discarded
169	Move Centerline Stripe 1 Foot Each Way	Geometric Revisions
170	Impose Seasonal Load Restrictions	Discarded
171	Implement Preventive Maintenance Program to Provide Regularly Scheduled Maintenance	Promising Tech. for Rehab.
172	Replace Wheel Paths-Possibly With Rails	Surf. Rehab. of Flex Pvt.
173	Implace Wicks Into Sublayer to Remove Moisture	Discarded
174	Use Conductive Asphalt Mixes Containing Coke	Surf. Rehab. of Flex Pvt.
175	Inplace Incapsulation With Strong and/or Impermeable Membrane	Discarded
176	Place Drainage Pipes Under Pavement From the Sides and Use the Same Pipes to Inject Stabilization Material	Stab. Sublayers in Place
177	Inject Lime into Subgrade	Stab. Sublayers in Place
178	Install Vertical Sand Columns as Dry Wells and Then Stabilize Columns	Stab. Sublayers in Place

Table B-1 cont.

Classification Number	Concept	Disposition
179	Proof Rolling by Temporarily Increasing the Legal Load Limit	Discarded
180	Reverse Traffic Flow	Discarded
181	Use Heated Aggregate to Heat Asphalt Surface, Remove Aggregate and Roll the Asphalt	Surf. Rehab. of Flex Pvt.
182	Use Open Graded Emulsions	Surf. Rehab. of Flex Pvt.
183	Stress Relieving Interface Between Pavement Overlay with Overflex	Surf. Rehab. of Flex Pvt.
184	Use of Carbon Black and Other Asphalt Fillers	Discarded
185	Use Continuously Mixed Concrete in Construction Practice to Improve Uniformity	Discarded
186	Seal Joints with Super Moduli Orthotropic Sealants	Repair and Replac. of Failed Joints
187	Reduce Wheel Load Limit	Discarded
188	Make CRCP Jointed Pavement	Crack Repair of Rigid Pvt.
189	Scarify Rigid Pavement Surface and Place Thin Layer of Latex Overlay	Surface Restoration of Rigid Pvt.
190	Asphalt Rubber Stress Absorbing Membrane	Surf. Rehab. of Flex Pvt.
191	Wide Expansion Joints over Concrete Load Transfer Beams at 1000ft intervals	Repair and Replac. Failed Joints
192	Vee Load Transfer Device	Repair and Replac. Failed Joints

1. Rehabilitation Classification Numbers 102, 111, 112, 113, 114, 116, 167, and 173 involving various aspects of draining or forcing water from underlying layers

These ideas were dropped from further consideration for the following reasons:

1. They involve drainage which is specifically excluded from the work statement on this project.

2. They involve extremely complex aspects of soils and drainage which would require extensive expenditures of time and money to fully evaluate, thus diluting efforts on other items more directly applicable to the project objective.

Drainage is recognized as one of the most important factors influencing pavement performance and, hence, plays an important role in any rehabilitation concept. But, for the purposes of this project, drainage must be assumed to be adequate by either currently available methods or by anticipated results of other research.

2. Rehabilitation Classification Number 110 Pressure Injection of Sulphur into Subbase

This idea involves the use of a plentiful material, sulphur, which has been shown to have excellent binding properties. However when the sulphur, heated to around 150°C in order to be in liquid form, comes in contact with a subbase at ambient temperature it immediately solidifies and thus does not penetrate. As discussed by Meyer, et al. (B 1), the subbase must be heated and dried out for the sulphur to be used successfully. This would involve removing the surface layers and heating the subbase which would be expensive, time consuming, and energy consuming. Thus this idea was dropped from further consideration on this project.

3. Rehabilitation Classification Number 115 Injection of Silicone Rubber between Subbase and Pavement to Form Continuous Moisture Barrier

This idea was dropped from further consideration because existing techniques for forming a moisture barrier, using low viscosity asphalt, work very well (see section on Stabilizing Subbases in Place) and silicone rubber is more expensive.

4. Rehabilitation Classification Numbers 118, 136, 147, 151, 155, and 185, involving various new Construction Activities.

These ideas involve some potentially innovative techniques for new construction (or major reconstruction). They were dropped from the list of possibilities because the scope of this research was limited to potential techniques to be used for rehabilitating existing pavements.

5. Rehabilitation Classification Number 135 Surface Reworking by Freezing.

This idea has been dropped from the list of possibilities for the following reasons:

1. Low heat diffusivity of bituminous pavements and relatively small ΔT would tend to make this a slow process.

2. Not an efficient use of energy.

3. Other techniques considered for surface removal without heating (milling machines) will essentially do the same job, are faster and more efficient, and equipment is already available.

6. Rehabilitation Classification Number 143 Joint Repair by Welding or Installing Flat Jacks.

This idea has been dropped from the list of possibilities for the following reasons:

1. It involves expensive, time consuming, and energy consuming equipment and procedures, the results of which are doubtful unless new technology is developed. Laser welding of PCC is possible but the long term effects of such procedures are unknown. For example, at around 350°C PCC starts to lose its hydration water and the bonds are reduced significantly. Flat jacks require considerable time to install and are expensive.

2. Several more promising techniques for joint repair have been developed in this research project.

7. Rehabilitation Classification Number 146 Repair Hole With a Screw-In Plug Patch

Repair of a rigid or flexible pavement with this technique would be very difficult if not impossible. Preparation of the pavement to receive this type of patch would have to be performed with a specially developed machine that would be self leveling. Additionally, the pavement which would receive the patch would have to be sound and fairly thick to insure adequate load transfer. Casting of the patch out of conventional asphalt concrete could be cast to form the patch with some difficulty. The weight of the patch would make handling in the field difficult.

The fit of the patch in the pavement would have to be fairly secure otherwise a bonding aid would have to be used to insure that cracking would not continue. A "snug" fit probably would not be possible.

This idea has been dropped from the list of possibilities.

8. Rehabilitation Classification Number 150 Use of a Shape Charge to Remove or Break Up Paving

Shape charges to fracture or cut strong materials has been successfully employed by the military. Large scale soil removal operations have also been explored using explosives. Therefore this idea would be based on some available information and data from which to make an engineering assessment of its potential. However, the use of explosives involves a significant potential safety and environmental pollution hazard, especially in a populated area where the majority of the innovative rehabilitation techniques will be applicable.

Therefore, this idea has been dropped from the list of possibilities.

9. Rehabilitation Classification Number 154 Chemically Prestressed Concrete

This concept is dropped as not worthy of detailed investigation for the following reasons:

1. No method of chemically prestressing concrete exists for practical application.

2. There is no known record of a chemically prestressed concrete application in the construction industry.

3. There is no reported activity at the present in the development of chemically prestressed concrete.

10. Rehabilitation Classification Number 160 Saw Pavement Joints at an Angle (to the vertical) When Repairing Failed Joints

This idea was dropped from detailed consideration on the project for the following reasons:

1. No justification could be found for sawing the concrete at an angle. The potential for spalling is enhanced, the load transfer remains essentially the same as if the saw cut were vertical, and such a cut would be more expensive.

2. Vertical cuts have performed quite satisfactory.

3. Potential techniques utilizing vertical cuts look promising (see the section on Repair and Replacement of Failed Joints).

11. Rehabilitation Classification Number 161 Use Kevlar Spray followed by Level up on Surface.

This idea has been dropped from the list of possibilities for the following reasons:

1. Kevlar is a very strong fiber used for parachutes and aerospace

load slings, but is not now available in the form of a spray.

2. Cost of material, in any form, is now much too high for highway application (7 to 8 $/lb.).*

12. Rehabilitation Classification Number 168 - Pick Up Segments of Pavement, Repair Sublayers and Replace Pavement

This idea was dropped from the list of possibilities for the following reasons:

1. The only type of pavement with sufficient strength to be picked up in any significant size is PCC. And any PCC which needs rehabilitation will already be cracked or spalled at the joints. Thus the rehabilitated pavement, at best, will be only of marginal quality.

2. The equipment required to pick up a major segment of pavement, e.g., 12 ft by 15 ft,** without damage does not presently exist. Therefore, substantial developmental costs will be incurred.

3. Once developed, any satisfactory machine capable of picking up these pavement segments will be large and difficult to transport, and will consume significant quantities of energy to perform its function.

4. The locations where conditions conducive to this approach exist are infrequent and widely scattered. Therefore, only a very limited number of machines would be manufactured. This will result in excessive mobilization and use charges for the equipment.

13. Rehabilitation Classification Number 170 Impose Seasonal Load Restriction

This idea was dropped from further consideration on this project because the determination of the overall utility of the concept would

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* 1 lb = 0.454 kg
** 1 ft = 0.305 m

be practically impossible to estimate with any acceptable accuracy as every condition where this technique could be considered would be unique. Also the political and enforcement problems associated with the idea would be difficult to assess. The idea has merit, especially during the spring thaw in northern states on their secondary roads, and perhaps, should be considered in a separate study.

14. Rehabilitation Classification Number 175 Inplace Incapsulation with Strong and/or Impermeable Membrane

This idea was dropped from detailed consideration on the project because it involves extensive reworking of the pavement structure which is as costly and energy consumptive as total reconstruction. Such extensive a measure involves more than simple rehabilitation and thus was beyond the scope of the study.

15. Rehabilitation Classification Number 179 Proof Rolling By Temporarily Increasing The Legal Load Limit.

This idea has been dropped from the list of possibilities for the following reasons:

1. Harmful effects of application of this concept probably would outweigh any beneficial results because densification would tend to be localized and control would be very difficult to achieve.

2. Legal and administrative problems.

16. Rehabilitation Classification Number 180 Reverse Traffic Flow

This idea involves the reversal of traffic flow to correct joint faulting. Investigation of this idea revealed that, in Wisconsin, a jointed, two-lane, PCC pavement which was faulted was incorporated into a four lane divided highway with the result that one lane of the faulted pavement was subjected to a reversal of traffic flow. According to Karl Dunn the faulting did diminish, only to be followed by faulting in the opposite direction. Thus the result was a temporary correction at best.

This idea was dropped from the possibilities for the following reasons:

1. The correction will, in all probability, be temporary only.

2. The places where such a concept could be used will be extremely scarce, as traffic safety, geometric constraints, and tradition would deter any major effort to reverse the flow of traffic.

17. Rehabilitation Classification Number 184 Use Carbon Black and Other Fillers in Asphalt.

This idea has been dropped from the list of possibilities for the following reasons:

This concept is actually not a rehabilitation technique in itself but rather a way of modification and improvement of the binder used for asphalt concrete. As such it becomes a way of improving standard overlays or the replacement materials used in other rehabilitation techniques such as items 120 and 176.

18. Rehabilitation Classification Number 187 Reduce Wheel Load Limits

The idea was dropped from further consideration for the following reason:

While the reduction of wheel load limits can significantly prolong the life of a given pavement structure such a reduction involves political and social considerations beyond the scope and intent of this research study. To compare the overall utility, or value, of this concept with concepts involving restoration of pavements under existing wheel load limits, a much more complex decision criteria would have to be developed.

This concept is an excellent one and perhaps should be investigated as part of a separate study in order to provide decision makers with the full cause and effect relationship between wheel load limits and the cost of a highway system.

Reference

B 1. Meyer, A. H., Ledbetter, W. B., Layman, A. H., and Saylak, D., "Reconditioning Heavy-Duty Freeways in Urban Areas," Final Report on NCHRP Project 14-4, Transportation Research Board, NAS, in Press.

APPENDIX C. HIGHWAY MAINTENANCE COSTS

Introduction

Annual highway maintenance budgets for the various states range from a low of $9,000,000 (Hawaii) to a high of $160,000,000 (California), with an average of $41,000,000. Annual maintenance costs are on the order of $1,000 to $4,000 per lane mile ($600 to $2400 per km) (C 1). Thus, it is apparent that highway maintenance expenditures have become a significant portion of the total money expended on our highway system.

Maintenance costs for the pavement are typically of the order of 30 to 50 percent of the total highway maintenance budget. Expenditures for mowing, vegetation control, drainage, traffic services, snow and ice control, and administration are the other areas of a state's maintenance program which require relatively large expenditures.

In an attempt to define pavement maintenance cost, four states were contacted and maintenance cost information obtained (C 2 - C 9). A summary of state-wide pavement maintenance costs by maintenance activity is shown in Table C-1 for Arizona, California, Nevada and North Dakota. These costs are for the 1976 fiscal year (March - April 1976). Low and high costs represent the range of unit costs for individual districts within each state. Also given are average costs for each maintenance activity. In addition, productivity, crew size, and equipment requirements for the individual maintenance activities are given in the table.

Flexible Pavements

A review of Table C-1 indicates that a wide variety of pavement maintenance activities have been defined. For purposes of establishing maintenance cost information for flexible pavements, these activities have been condensed to the following items:

1. Fog Seal - Partial Width
2. Fog Seal - Full Width

TABLE C-1. Highway Maintenance Cost for Various Maintenance Activities

State	No.	Descriptive Title	Unit Costs 1976: Range		Avg.	Std.	unit of measure	Productivity	Crew and Equipment
AZ	101	Hand Patch with Premix	70.24	280.97	112.39		yd^3	6-24 man-hr/yd^3	3 men, 1 truck, 1 loader
AZ	102	Level with Premix	25.92	51.84	34.56		yd^3	.933 - 1.867 man-hr/yd^3	7 men, 3 trucks, 1 loader, 1 roller, 1 distributor, 1 grader, 1 pickup
AZ	103	Fill Cracks	2.54	5.07	3.38		gal	0.20 - 0.40 man-hr/gal	5 men, 1 truck, 1 distributor, 1 compressor, 1 pickup
AZ	104	Spot Seal Patching	33.48	59.51	42.85		yd^3	1.50 - 2.67 man-hr/yd^3	6 men, 2 trucks, 1 loader, 1 distributor, 1 aggre. spreader
AZ	105	Surface/Base Replacement	21.91	36.51	27.38		yd^3	0.96 - 1.60 man-hr/yd^3	6 men, 3 trucks, 2 loaders, 1 roller, 1 distributor, 1 grader, 1 pickup
AZ	106	Seal Coating (Major)	.143	.258	.184		yd^3	.0019 - .0034 man-hr/yd^2	15 men, 5 trucks, 1 loader, 2 rollers, 1 aggre. spreader, 1 broom, 2 pickups
AZ	107	Seal Coating (Minor)	.180	.541	.270		yd^3	.0034 - .102 man-hr/yd^2	9 men, 3 trucks, 1 loader, 1 roller, 1 distributor, 1 aggre, spreader, 1 broom, 1 pickup
AZ	108	Flush Coating	.0578	0.866	.069		yd^3	.0010 - .00068 man-hr/yd^2	9 men, 3 trucks, 1 loader, 2 spreader boxes, 1 broom, 2 pickups
AZ	109	Spot Flush Coating	.75	1.31	.95		gal	.034 - .060 man-hr/gal	6 men, 1 truck, 1 loader, 1 spreader box, 1 distributor, 1 broom, 1 pickup
CA	01-011	Machine Digout & Repair			38.72		ton (Removed)	1.81 man-hr/ton of mat. removed	2 men, 1 truck, 1 motor grader, 1 backhoe
CA	01-021	Machine placed surface 0-50 tons per day			29.94		ton	0.270 man-hr/ton of mix	2 men, 1 truck, 1 motor grader
CA	01-022	Machine placed surface 50 - 100 tons per day			24.52		ton	0.163 man-hr/ton of mix	2 men, 1 truck, 1 motor grader
CA	01-023	Machine placed surface 100+ tons per day			16.23		ton	0.063 man-hr/ton of mix	2-3 men, 1 truck, 1 motor grader
CA	01-024	Laydown surfacing			23.00		ton	0.200 man-hr/ton of mix	8 men, trucks, laydown machine, roller
CA	01-031	Hand placed surface patch			83.89		ton	2.7 man-hr/ton of mix	2-3 men, trucks, roller
CA	01-032	Pot Hole patching-hand			92.18		100 lb of petrolastic	6.04 man-hr/100 lb container	2 men, 1 truck, 1 kettle
CA	01-033	Hand Digout & Repair			79.97		ton		
CA	01-034	Hand Digout & Repair			82.63		ton	3.45 man-hr/ton of mat. replaced	3 men, trucks
CA	01-041	Crack & Joint Fill-Petrolastic			58.37		100 lb of petrolastic	2.2 man-hr/100 lb container	2-5 men, trucks, kettle
CA	01-042	Crack & Joint Fill-Other			6.41		gal	.33 man-hr/gal	5 men, trucks, kettle
CA	01-51	Rock Seal to 6 ft Spread			32.78		ton of aggr.	.40 man-hr/ton of rock	5 men, trucks, kettles, roller
CA	01-052	Rock Seal Over 6 ft Spread			30.35		ton of aggr.	.25 man-hr/ton of rock	6 men, trucks, kettles, roller
CA	01-053	Sand Seal-Mechanical Spreader			28.89		ton of aggr.	.195 man-hr/ton of sand	4 men, trucks, kettles, drag broom
CA	01-983	Liquid Seal			147.08		ton of asphalt		
CA	02-011	Mudjacking			7.28		yd^2	0.290 man-hr/yd^2	5 men, trucks, mudjack pump, air compressor, drilling tools
CA	02-012	Remove & Replace PCC			26.22		yd^3		
CA	02-021	Hand Placed Surfacing			13.48		ton	3.12 man-hr/ton of mat. placed	2-3 men, trucks, steel roller
CA	02-022	Machine Placed Surfacing			13.25		ton	.15 man-hr/ton of mat. placed	2-8 men, trucks, motor grader or laydown machine, roller, kettle
CA	02-041	Air Cleaning Joint or Crack			.0393		lin ft	.002 man-hr/lin ft	2 men, truck, air compressor
CA	02-042	Crack & Joint Fill-Petrolastic			47.99		100 lb of petrolastic	2.00 man-hr/100 lb container	2-5 men, trucks, kettle
CA	02-043	Crack & Joint Fill-Other			4.77		gal	0.23 man-hr/gal	5 men, trucks, kettle
NV	101.01	Base & Surface Repair	5.60	37.02	17.35	17.5	yd^3	.833 man-hr/yd^3	8 men, 3 trucks, 1 loader, 1 roller, 1 m. grader, 1 water truck, 1 dist. 1 pavement cutter
NV	101.02	Surface Patching-Premix(hand)	67.80	163.77	123.60	137.46	yd^3	2.0 man-hr/yd^3	4 men, 2 trucks, 1 loader, 1 tilt bed trailer
NV	101.03	Surface Patching-Premix(Machine)	25.09	34.36	27.96	26.70	yd^3	.588 man-hr/yd^3	5 men, 2 trucks, 1 loader, 1 roller, 1 m. grader, 1 asphalt dist. 1 tilt bed trailer
NV	101.04	Surface Patching-Spot Seal	.17	.58	.23	.15	yd^2	.006 man-hr/yd^2	3 men, 2 trucks, 1 distributor
NV	101.05	Seal Coat - Sand	.06	.17	.14	.09	yd^2	.0021 man-hr/yd^2	10 men, 4 trucks, 1 loader, 1 dist., 1 chip spreader, 1 broom, 1 tilt bed trailer, 1 pickup
NV	101.06	Seal Coat - Flush	.05	.11	.06	.05	yd^2	.00138 man-hr/yd^2	7 men, 2 trucks, 1 loader, 1 distributor, 1 tilt bed trailer, 1 pickup
NV	101.07	Crack Filling	.21	.53	.36	.43	lb	.022 man-hr/lb of filler	5 men, 2 trucks, 1 asphalt kettle, 1 air compressor
NV	101.08	Heater Planing	.18	2.02	.32	.32	yd^2	.01818 man-hr/yd^2	5 men, 2 trucks, 1 loader, 1 m. grader, 1 heater planer, 1 tilt bed trailer
NV	101.09	Seal Coat - Chips	.15	.27	.23	.20	yd^2	.00267 man-hr/yd^2	14 men, 5 trucks, 1 loader, 1 roller, 1 broom, 1 chip spreader, 1 dist., 2 pickups
NV	111.01	Temporary Patching of PCC Pavements				106.26	yd^3	9.0 man-hr/yd^3	5 men, 2 trucks, 1 pickup
NV	111.02	Permanent Patching of PCC Pavements				371.25	yd^3	28.56 man-hr/yd^3	8 men, 3 trucks, 1 loader, 1 concrete saw, 1 air compressor, 2 trailers, 1 vibrator, 1 pkup
NV	111.05	Joint Sealing				1.20	lb	0.100 man-hr/lb of filler	5 men, 1 truck, 1 air compressor, 1 asphalt kettle, 1 pickup
NV	111.06	Expansion Joint Repair		44.97	44.97	6.79	lin ft	0.625 man-hr/lin ft	5 men, 1 truck, 1 concrete saw, 1 air compressor, 1 pickup
ND	411	Hand Patching	46.59	76.30	55.34	60.62	yd^3	5-7 man-hr/yd^3	5 men, 2 trucks, 1 distributor, 1 loader at stockpile
ND	412	Spot Sealing	.21	.36	.26	.40	yd^2	0.015 to 0.03 man-hr/yd^2	3 men, 2 trucks, 1 distributor
ND	414	Crack Pouring	.92	2.23	1.18	1.32	gal	0.10 to 0.25 man-hr/gal	4 men, 1 truck, 1 distributor, 1 pickup
ND	416	Blading Gravel Rd.	3.50	13.76	4.83	4.50	mi		
ND	421	Blade Patch & Level	17.66	26.15	22.35	23.15	yd^3	0.5 to 0.9 man-hr/yd^3	7 men, 4 trucks, 1 distributor, 1 motor grader, 1 or 2 rollers
ND	422	Seal Coat	.126	.272	.21	.28	yd^2		17-23 men, 8 trucks, 1 loader, 2 dists., 1 spreader box, 2 rollers, 1 broom, 1 tankcar heater, 1 pkup
ND	424	Light Reseal	.033	.064	.06	.13	yd^2	.0028 man-hr/yd^2	10 men, 5 trucks, 1 dist. 1 tank car heater, 1 roller, 1 loader, 1 broom
ND	431	Shoulder Patching	19.29	47.98	29.01	30.00	yd^3	2.4 - 3.4 man-hr/yd^3	14-18 men, 3-5 trucks, 1 dist., 1-2 motor graders, 2 rollers, 1 mixer, 1 loader
ND	433	Shoulder Seal with Aggregate	.16	.22	.27	.20	yd^2	.01 man-hr/yd^2	7-9 men, 2 distributors, 1 tank car heater, 1 broom, 1 roller, 3 trucks
ND	435	Shoulder Seal without Aggregate	.10	.15	.11	.45	yd^2	.01 man-hr/yd^2	4-5 men, 2 distributors, 1 tank car heater, 1 broom

Metric Conversions:

$1\ yd^2 = 0.84\ m^2$ 1 ton = 907 kg 1 mi = 1609 m

$1\ yd^3 = 0.76\ m^3$ 1 ft = 0.0305 m

1 gal = 3.79 litre 1 lb = 0.45 kg

3. Chip Seal - Partial Width
4. Chip Seal - Full Width
5. Surface Patch - Hand Method
6. Surface Patch - Machine Method
7. Digout and Repair - Hand Method
8. Digout and Repair - Machine Method
9. Crack Pouring
10. Asphalt Concrete Overlay (C 10 - C 11).

A general description for each activity has been prepared and is shown in Table C-2 together with average, low, and high unit costs for these activities. The reported suggested costs are the authors best estimate of representative unit costs for the stated maintenance activity (Table C-2). Figure C-1, which is a comparison of unit costs for the 4 states surveyed, was used as an aid in the analysis of the data and determination of representative average unit costs. The wide range of reported unit costs for this condensed list of activities is due in part to:

1. Different crew sizes utilized in the various states
2. Different equipment requirements for various states
3. Differences in maintenance work activity as defined by various states
4. Variety of traffic conditions under which maintenance is performed
5. Type of facility on which maintenance activities are performed
6. Amount of work performed per lane mile.

Maintenance unit costs information has been converted to costs per yd^2 of total pavement surface area treated and cost per lane mile. Figure C-2, which graphically represents pavement area treated for a lane mile of highway, was used to estimate the amount of area typically treated. The suggested average unit cost for the maintenance activity (Table C-2) was utilized together with the expected range in cost to prepare Figures C-3 to C-12. These figures graphically illustrate the cost per yd^2 or cost per lane mile of rehabilitated pavement. Thus, costs are represented in terms of percent of total pavement surface area treated. Adjustments,

TABLE C-2. Unit Cost for Flexible Pavement Maintenance Operations

Descriptive Title	General Description	State	No.	Reported Average Unit Cost, Dollars	Suggested Costs, Dollars			
					Avg.	Low	High	unit of measure
Fog Seal - Partial Width	Light application of diluted emulsion or a proprietary material over a partial lane.	AZ	109	$0.095/yd^2$	.095	.075	.131	yd^2
Fog Seal - Full Width	Light application of diluted emuslion or a proprietary material over a full lane width in a continous section.	AZ CA NV ND	108 01:983 101.06 435	$0.069/yd^2$ $0.06/yd^2$ $0.06/yd^2$ $0.11/yd^2$	.06	.05	.11	yd^2
Chip Seal - Partial Width	Application of Asphalt and cover aggregate to a limited area.	AZ CA NV ND	104 01-051 101.05 412	$0.36/yd^2$ $0.41/yd^2$ $0.23/yd^2$ $0.26/yd^2$	.35	.23	.41	yd^2
Chip Seal - Full Width	Application of asphalt and cover aggregate to a full lane width in a continous section.	AZ CA NV ND	106 01-054 101.09 422	$0.18/yd^2$ $0.24/yd^2$ $0.23/yd^2$ $0.21/yd^2$	.21	.18	.24	yd^2
Surface Patch-Hand Method	Application of a Permix material to the surface of the pavement by hand method.	AZ CA NV	102 01-031 101.02	$34.56/yd^3$ $147.00/yd^3$ $123.60/yd^3$	130.00	60.00	170.00	yd^3
Surface Patch-Machine Method	Application of a Premix material to the surface of the pavement with machine	AZ CA CA CA CA NV ND	102 01-021 01-022 01-023 01-024 101.03 421	$34.56/yd^3$ $52.50/yd^3$ $43.00/yd^3$ $28.50/yd^3$ $40.40/yd^3$ $27.96/yd^3$ $22.35/yd^3$	28.00	20.00	40.00	yd^3
Digout & Repair Hand Method	Removal & repair of limited areas by use of hand tools.	AZ CA ND	101 01-034 411	$112.39/yd^3$ $145.00/yd^3$ $55.34/yd^3$	110.00	40.00	160.00	yd^3
Digout & Repair Machine Method	Removal & repair of limited areas by use of mechanized equipment.	AZ CA NV	105 01-011 101.01	$27.38/yd^3$ $68.00/yd^3$ $17.35/yd^3$	25.00	10.00	70.00	yd^3
Crack Pouring	Pouring cracks in flexible pavement with asphalt material (may include cleaning with compressed air & covering with sand.	AZ CA CA NV ND	103 01-041 01-042 101.07 414	3.38/gal 4.83/gal 6.41/gal 3.00/gal 1.18/gal	3.25	1.10	6.50	gal
Asphalt Concrete Overlay	Application of an asphalt concrete overlay usually less than about 2 inches.	TX US		21.00*/ton 15.12*/ton	31.00	23.00	43.00	yd^3

*Cost per ton **Metric Conversions:** 1 yd^2 = 0.84 m^2 1 ton = 907 kg

1 yd^3 = 0.76 m^3

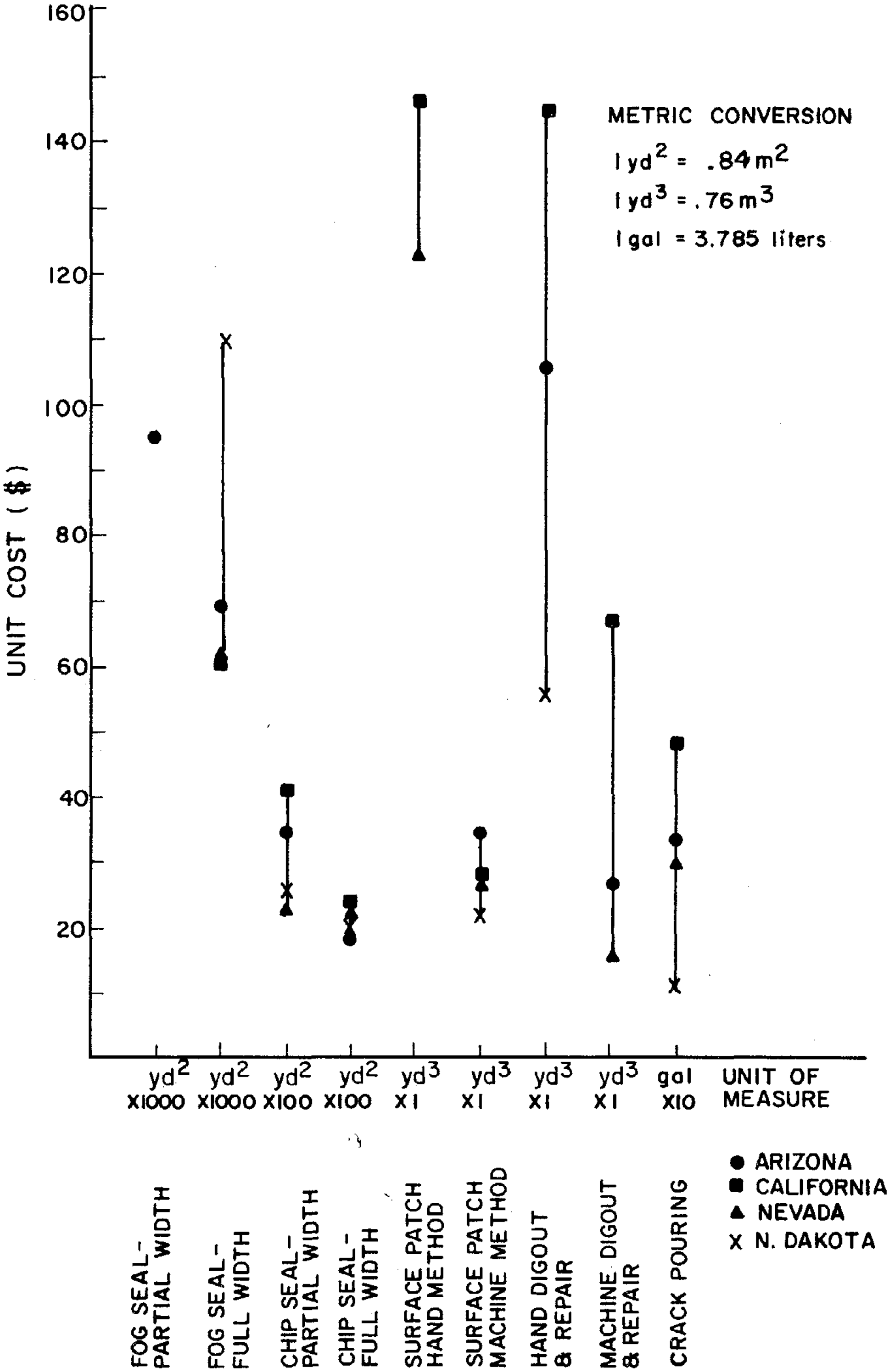

Figure C-1. Ranges in flexible pavement unit costs.

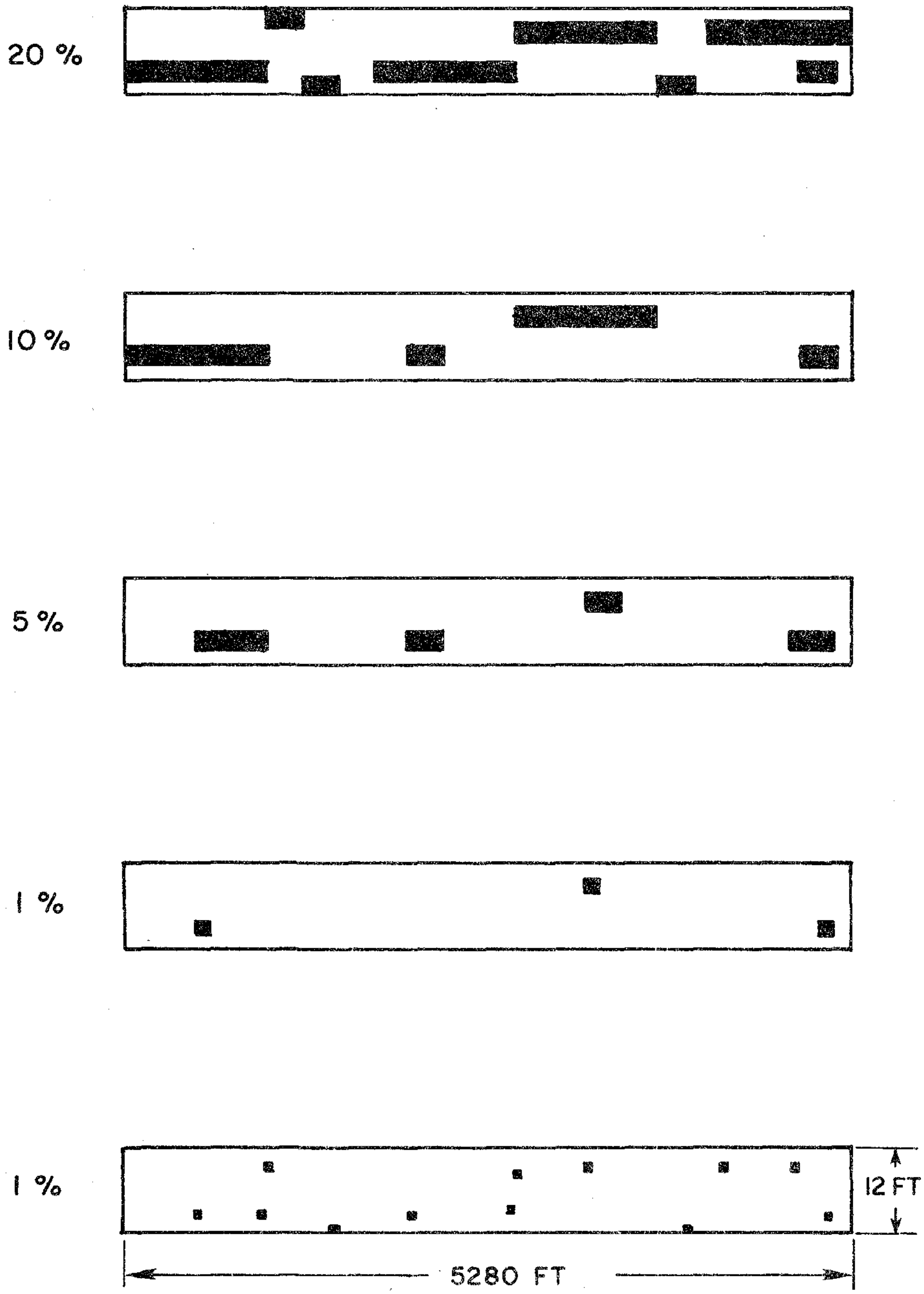

Figure C-2. Graphic representation of area to be repaired.
(Metric conversion: 1 ft = 0.3 m)

based on experience, were utilized to alter average unit costs when very small or very large areas of the pavement were treated with a particular maintenance activity. For information purposes, the approximate pavement condition has been estimated in each case, in terms of distress type, distress severity, and distress extent. This condition is in the form of deduct points, as described in reference (C-12). The overall pavement condition is represented by a number from 0 to 100, where 100 represents a distress-free pavement. The deduct points are to be subtracted from 100 (or some existing initial value less than 100) and thus the more extensive the maintenance the higher the deduct points. These deduct points are shown on each figure.

To prepare this cost figure for each maintenance activity, representative average costs were selected from Table C-2, keeping in mind the range of reported costs. Each activity's costs are described in the following paragraphs.

Fog Seal - Partial Width. Costs for various percentages of the pavement surface area treated are shown in Figure C-3. An average unit cost of \$0.095 per yd^2 (\$0.11 per m^2) is the basis for the relationship represented in Figure C-3.

Fog Seal - Full Width. Costs for various pavement conditions are shown in Figure C-4. An average cost of \$0.06 per yd^2 (\$0.07 per m^2) was used as the basis for the relationship. For pavements that are severely raveled (30 or more deduct points), the amount of applied emulsion was increased 0.02 gal per yd^2 (0.09 litre per m^2). Since material costs represent 20% of the total unit cost of this operation (C 2), an increase in cost of \$0.01 per yd^2 (\$0.012 per m^2) resulted. Likewise, costs representing treatments for lightly raveled pavement were reduced by \$0.01 per yd^2 (\$0.01 per m^2).

Chip Seal - Partial Width. Costs for various percentages of the pavement surface area treated are shown in Figure C-5. An average unit costs of \$0.35 per yd^2 (\$0.42 per m^2) is the basis for the relationship represented in this Figure.

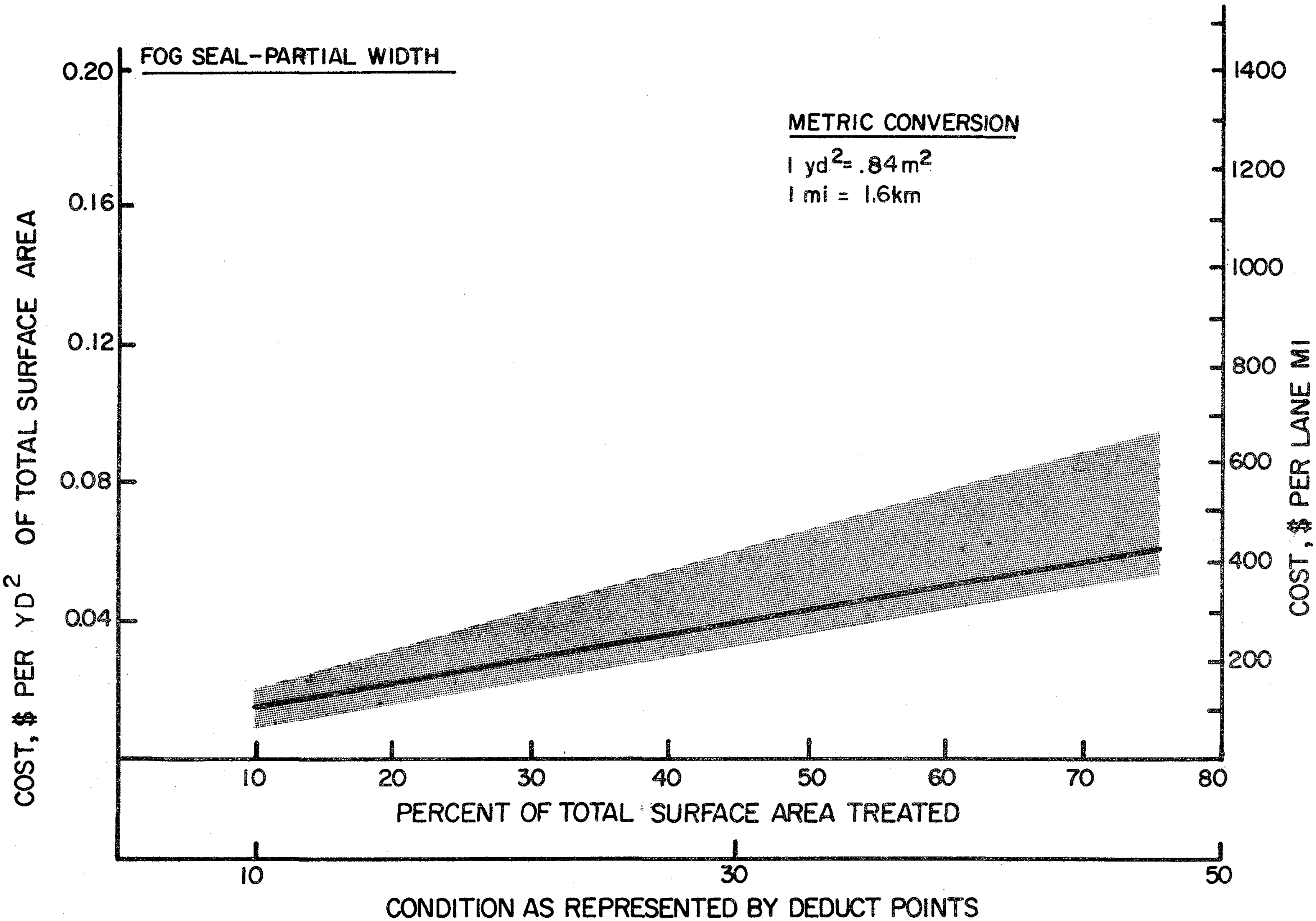

Figure C-3. Representative maintenance costs for partial width fog seal.

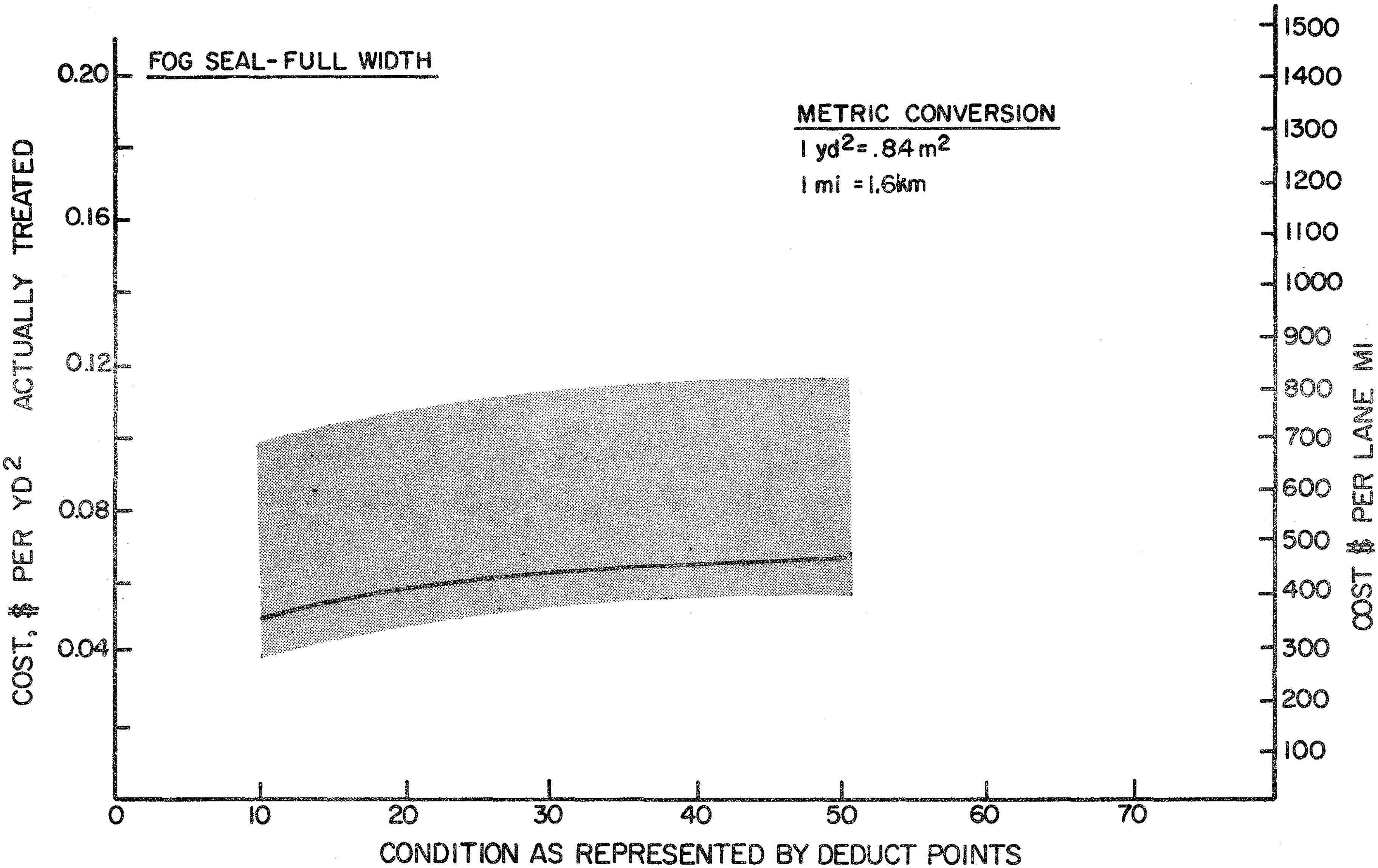

Figure C-4. Representative maintenance costs for full width fog seal.

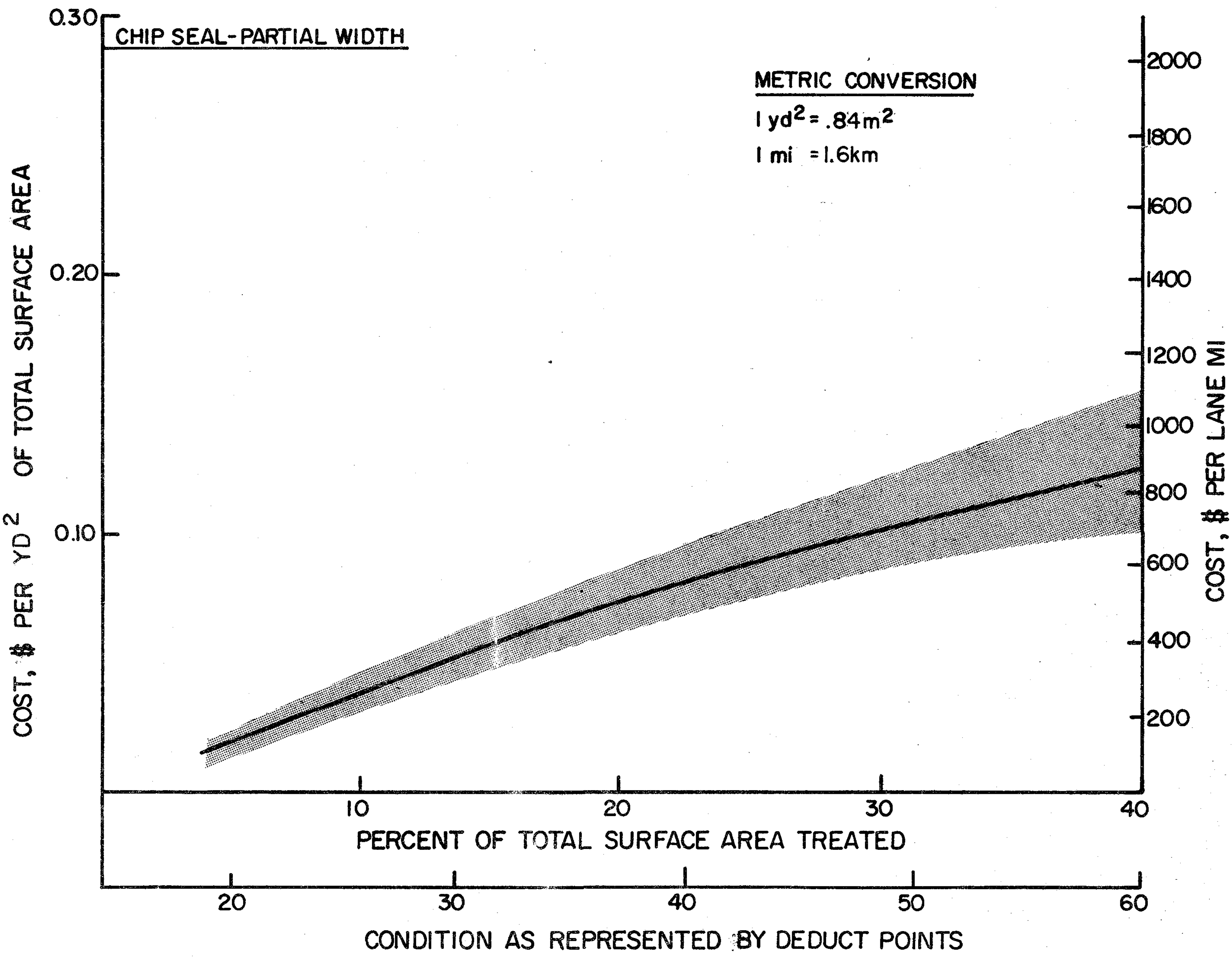

Figure C-5. Representative maintenance costs for partial width chip seal.

Chip Seal - Full Width. Costs for various pavement conditions are shown in Figure C-6. An average unit cost of \$0.21 per yd^2 (\$0.25 per m^2) was used as the basis for this relationship. An appropriate cost adjustment was made for pavements requiring additional asphalt due to their surface condition. Some states placing chip seals under contract have experienced unit costs of the order of \$0.35 per yd^2 to \$0.40 per yd^2 (\$.42 to \$0.48 per m^2).

Surface Patch - Hand Method. Costs for various percentages of the pavement surface area treated are shown in Figure C-7. An average unit cost of \$130 per yd^3 (\$170 per m^3) of material placed was used as the basis for this relationship, assuming a one in. (2.5 cm) thick patch.

Surface Patch - Machine Method. Costs for various percentages of the pavement surface area treated are shown in Figure C-8. An average unit cost of \$28 per yd^3 (\$37 per m^3) of material placed was used as the basis for this relationship, assuming a one in. (2.5 cm) thick patch.

Digout and Repair - Hand Method. Costs for various percentages of the pavement surface area treated are shown in Figure C-9. An average unit cost of \$110 per yd^3 (\$144 per m^3) of material placed was used for this relationship assuming a six in. (15 cm) thick patch.

Dig Out and Repair - Machine Method. Costs from various percentages of the pavement surface area treated are shown in Figure C-10. An average unit cost of \$25.00 per yd^3 (\$32.70 per m^3) of material placed was used as the basis for the relationship assuming a six in. thick patch (15 cm).

Crack Pouring. Costs for the pouring of various amounts of cracks (represented by linear ft (0.3 m) of cracks per 100 foot (30.5 m) length of a 12 ft (3.7 m) lane are shown in Figure C-11. An average unit cost of \$3.25/gal (\$.85/litre) of crack sealing material placed was used as the basis for this relationship. It was assumed that 1 gal (3.785 litres) of liquid would be utilized to pour 50 lineal ft (15 m) of cracks.

Asphalt Concrete Overlay. Costs for the placement of various thicknesses of asphlat concrete are shown on Figure C-12. An average unit cost of \$16/ton or \$31.30/yd^3 (\$.018/kg or \$40.90/m^3) was used as the basis for this relationship. Overlay costs of the order of \$20/ton (\$.022/kg) are

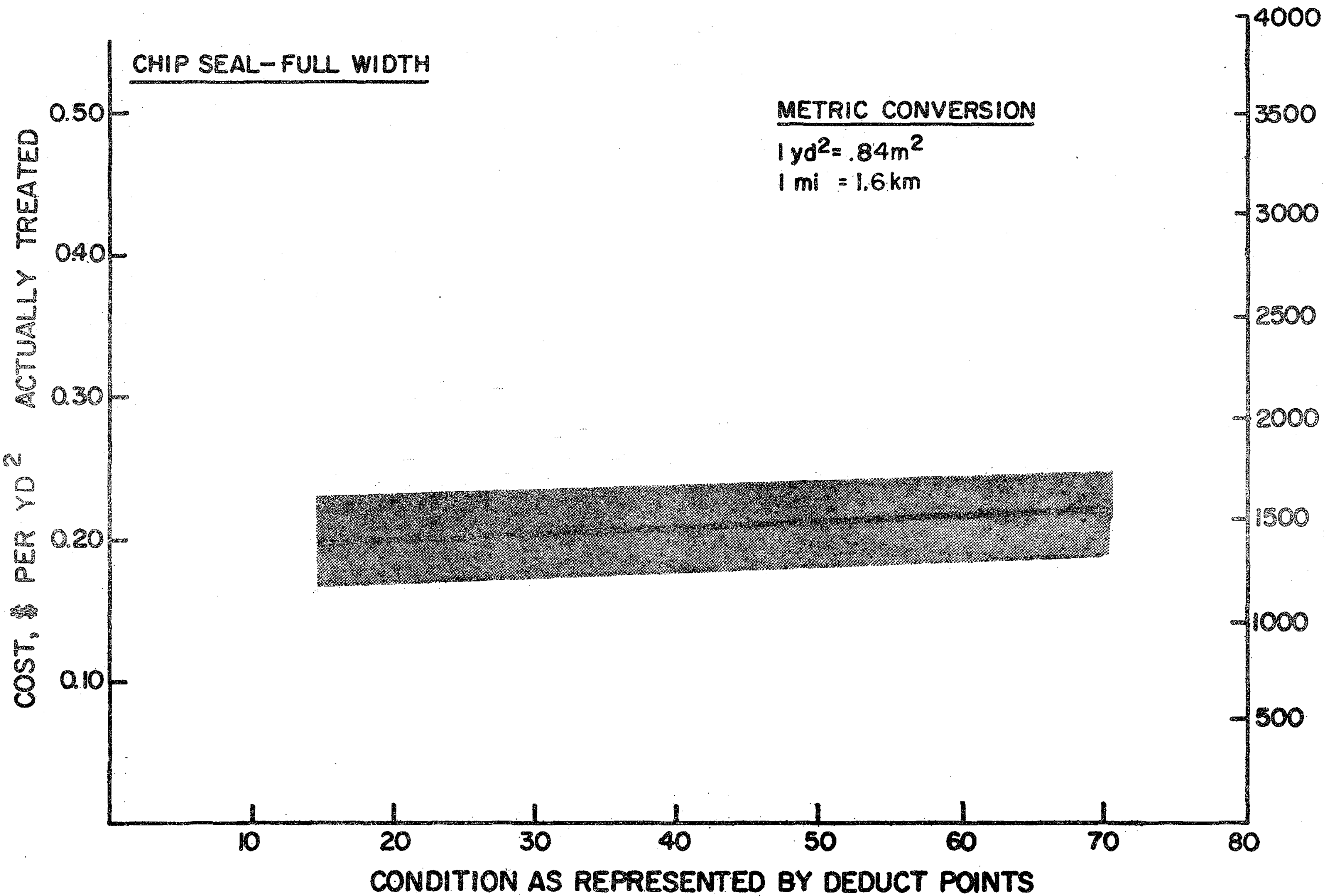

Figure C-6. **Representative maintenance costs for full width chip seal.**

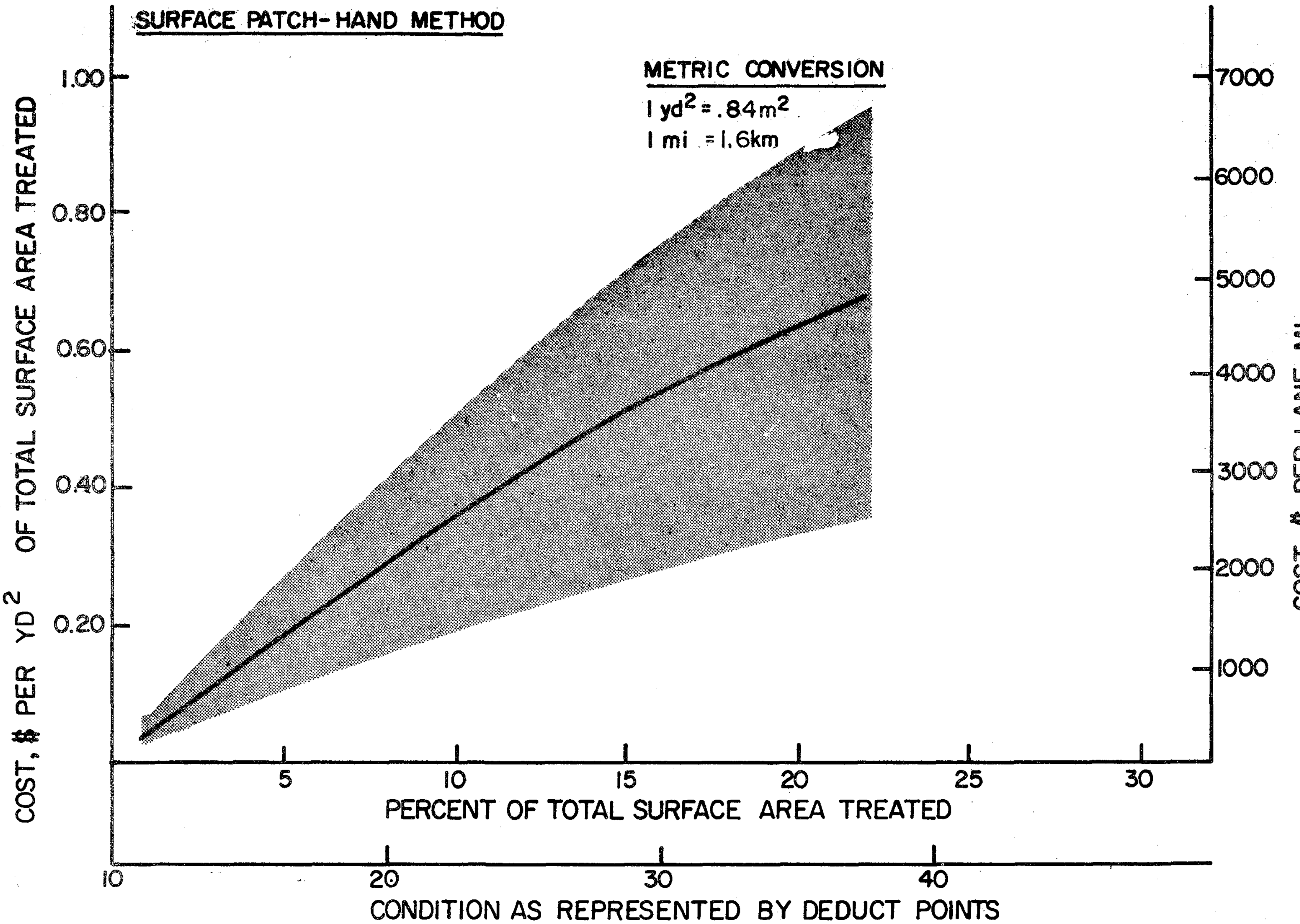

Figure C-7. Representative maintenance costs for hand surface patch.

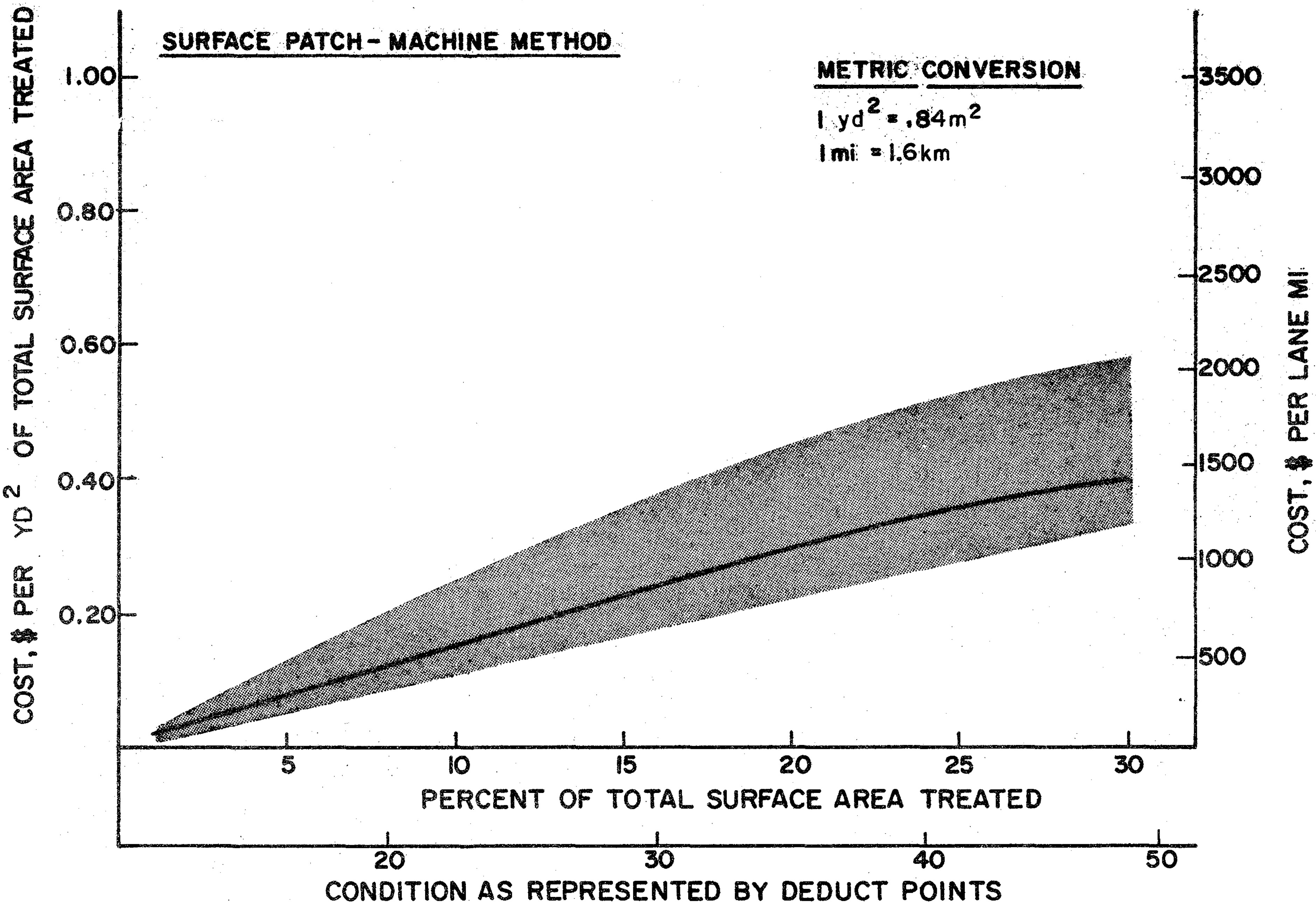

Figure C-8. Representative maintenance costs for machine surface patch.

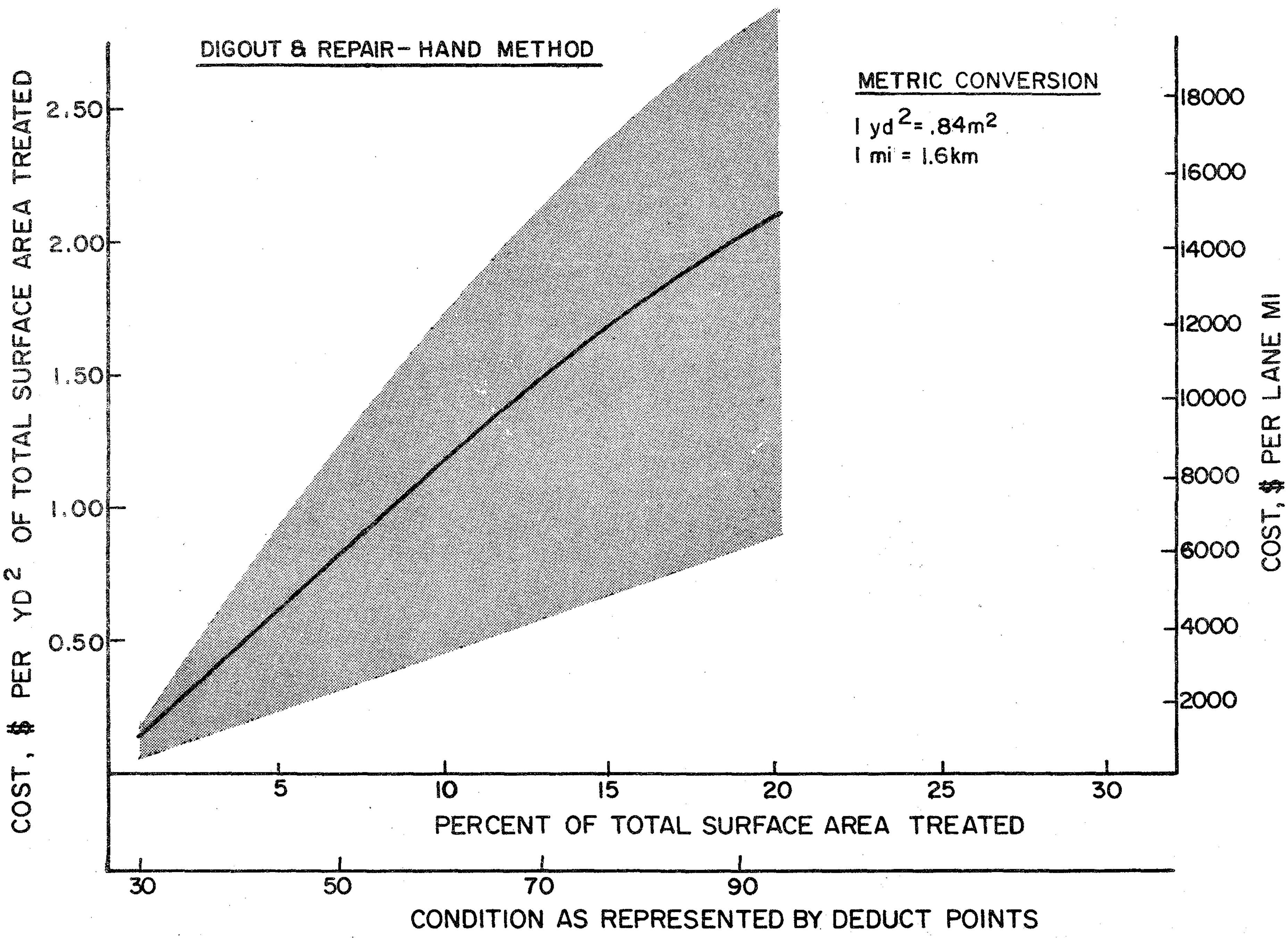

Figure C-9. Representative maintenance costs for hand digout and repair.

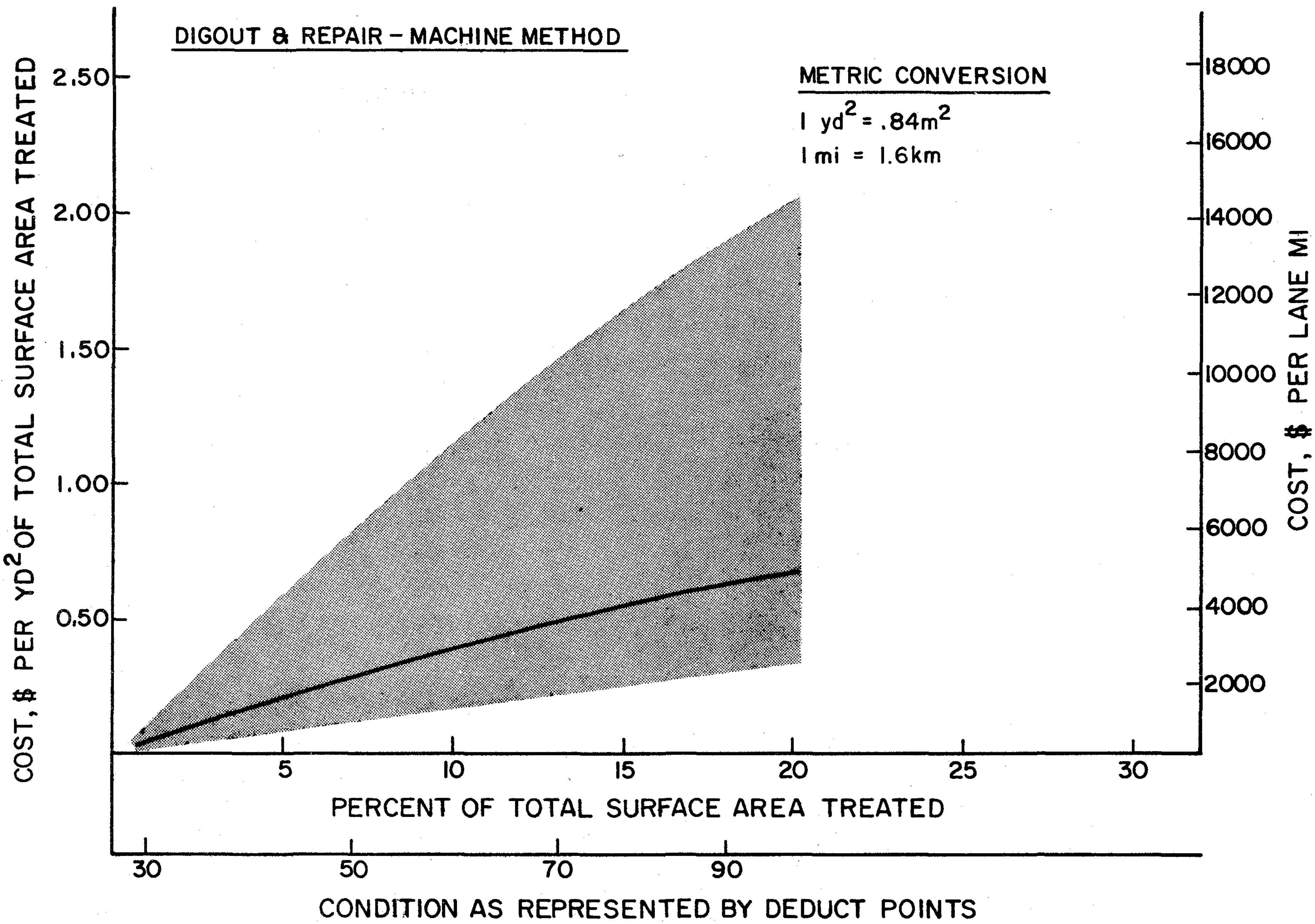

Figure C-10. Representative maintenance costs for machine digout and repair.

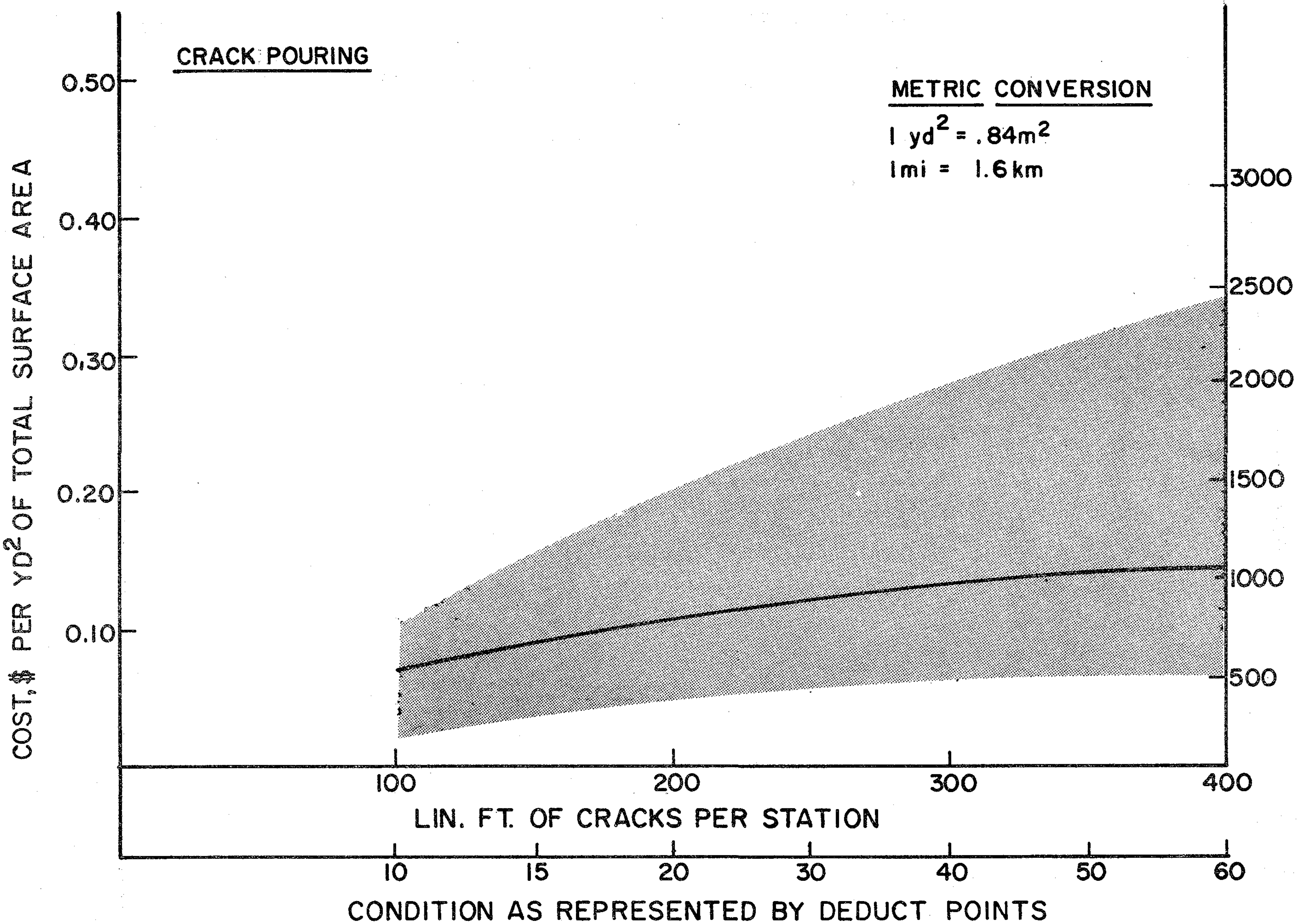

Figure C-11. Representative maintenance costs for crack pouring.

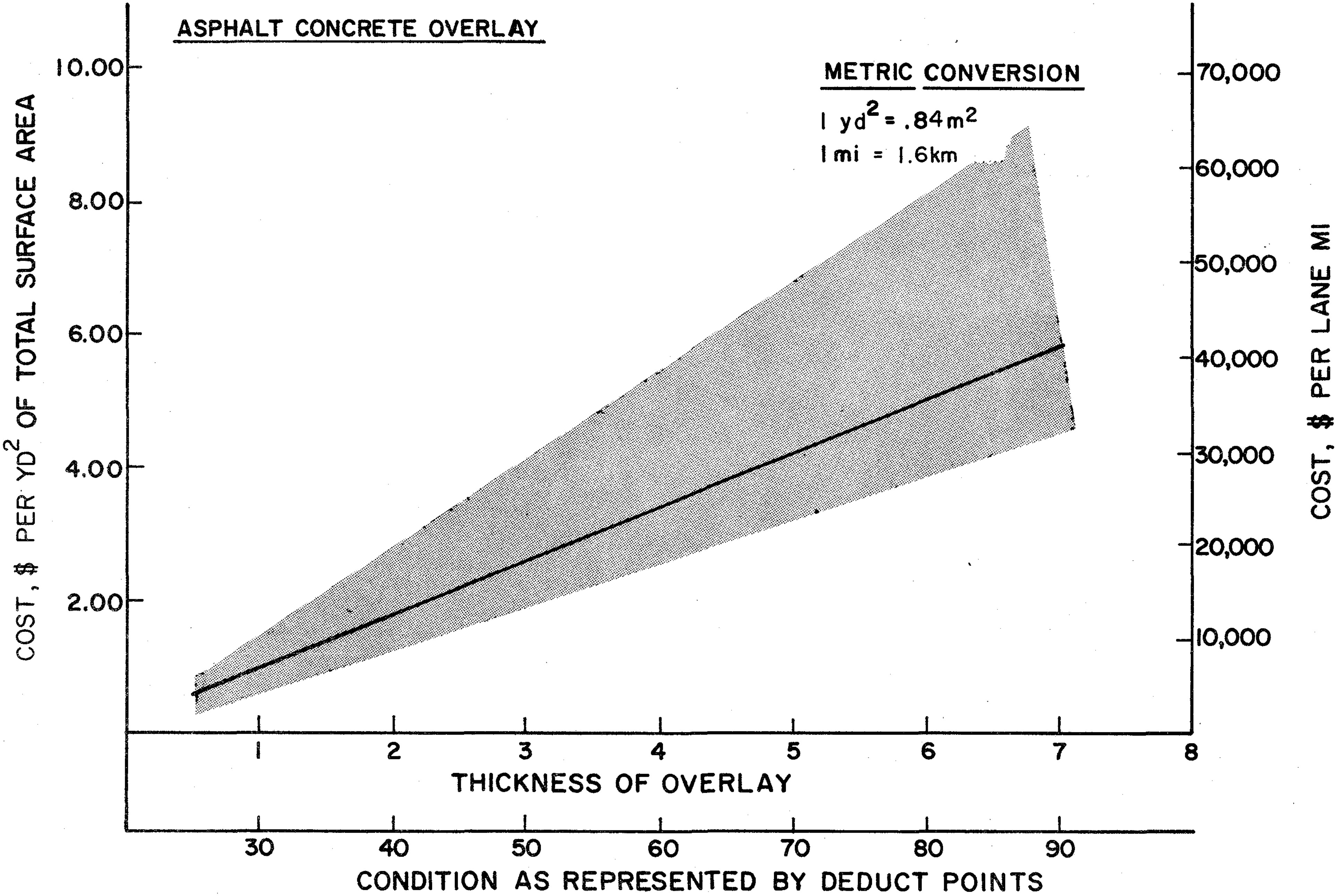

Figure C-12. Representative maintenance costs for asphalt concrete overlay.

not unusual in many parts of the United States.

Rigid Pavements. For the purpose of establishing maintenance cost information for rigid pavements, the following five rigid pavement maintenance activities have been defined:

1. Mudjacking
2. Temporary Patching
3. Permanent Patching
4. Joint Sealing
5. Expansion Joint Repair

Unit cost information is shown in Table C-3. Again the wide range of values can be attributed to many of the previously enumerated reasons, and suggested unit costs are offered.

Discussion of Flexible Pavement Costs

A summary of information contained on the previous Tables and Figures is shown in Table C-4 for ten maintenance and rehabilitation activities. These costs are based on the data obtained from four states and the assumption stated in the discussion. If the reader has need of determining maintenance costs for activities other than those listed on Table C-4, it will be necessary to obtain data from a state, county or city performing that activity.

The reader is reminded that the maintenance activities described in this report are normally performed on pavements with certain specific types of distress. For example, fog seals and chip seals are popular maintneance or rehabilitation activities that are used to correct raveling pavements. Typical types of pavement distress and maintenance activities associated with maintenance of these types of distress are shown on Table C-5. The reader is refined to reference (C 13) for a detailed description of the distress types

TABLE C-3. Unit Cost for Rigid Pavement Maintenance Operations

Maintenance Activity				Reported Average Unit Costs, Dollars	Suggested Unit Cost, Dollars			
Descriptive Title	General Description	State	No.		Avg.	Low	High	unit of measure
Mudjacking	Drilling Holes and Pumping concrete slurry under slab to fill the voids and raise the slab to grade.	CA	02-011	$7.28/yd^2$	7.25			yd^2
Temporary Patching	Patch with Bituminous material.	CA NV	02-021	$25.50/yd^3$ $106.26/yd^3$	80	20	160	yd^3
Permanent Patching	Patch with P.C.C.	NV	111.02	$371.25/yd^3$	375			yd^3
Joint Sealing	Cleaning joint, pour joint and apply sand as required.	CA CA NV	02-042 02-043 111.05	5.75/gal 4.77/gal 10.00/gal.	7.00	5.00	12.00	gal
Expansion Joint Repair	Cut along distressed area, clean out area, place filler material.	NV	111.06	6.79/lin ft	6.75	5.00	40.00	lin ft

Metric Conversions:

$1\ yd^2 = 0.84\ m^2$

$1\ yd^3 = 0.76\ m^3$

1 gal = 0.26 litre

1 ft = 0.305 m

TABLE C-4. Representative Costs for Maintenance and Rehabilitation Activities

Maintenance Activity	Cost Dollars * Per		Percent of Total Pavement Area Treated
	Sq Yd	Lane Miles	
Fog Seal - Partial Width	0.045	320	50 percent
Fog Seal - Full Width	0.06	420	100 percent
Chip Seal - Partial Width	0.06	420	15 percent
Chip Seal - Full Width	0.21	1500	100 percent
Surface Patch - Hand Method	0.10	700	2.5 percent 1 inch thick
Surface Patch - Machine Method	0.08	560	10 percent 1 inch thick
Digout & Repair - Hand Method	0.25	1760	2 percent 4 inches thick
Digout & Repair - Machine Method	0.20	1400	5 percent 6 inches thick
Crack Pouring	0.12	850	250 lin. ft. Per Station
Asphalt Concrete Overlay	1.90	13,400	100 percent 2 inches thick

*Costs are for square yards of total pavement surface maintained. For example, surface patching by the hand method may have been applied over only 5 percent of total pavement surface area, yet costs reported are for the total pavement area maintained or one mile of pavement.

Metric Conversions:

$1\ yd^2 = 0.84\ m^2$

1 mi = 1609 m

1 in. = 0.024 m

1 ft = 0.305 m

TABLE C-5. Maintenance Activities Associated with Flexible Pavement Distress

Types of Distress	Maintenance Activity	
Rutting	Surface Patch - Hand Surface Patch - Machine	Asphalt Concrete Overlay
Raveling	Fog Seal - Partial Width Fog Seal - Full Width	Chip Seal - Partial Width Chip Seal - Full Width
Flushing (Bleeding)	Overlay Chip Seal - Full Width	
Corrugations	Surface Patch - Hand Surface Patch - Machine Asphalt Cone - Overlay	Digout & Repair - Hand Digout & Repair - Machine
Alligator Cracking	All Maintenance Operations could be used	
Longitudinal Cracking	Fog Seal - Partial Width Fog Seal - Full Width Crack Pouring	Chip Seal - Partial Chip Seal - Full Width Asphalt Cone - Overlay
Transverse Cracking	Crack Pouring Chip Seal - Fill Width	Asphalt Concrete Overlay
Patching	Surface Patch - Hand Surface Patch - Machine Chip Seal - Full Width	Digout & Repair - Hand Digout & Repair - Machine Asphalt Concrete Overlay
Failures	Surface Patch - Hand Surface Patch - Machine Asphalt Cone Overlay	Digout & Repair - Hand Digout & Repair - Machine

Conclusions

From a review of Tables C-1 - C-4 and Figures C-1 - C-12 the following observations are made:

1. Hand digout and repair techniques are very expensive. A one inch overlay can be placed at the same cost as performing hand digout and repair over about 8 percent of the pavement surface area.

2. A chip seal can be placed full-width at the same cost of performing hand digout and repair over about 2 percent of the pavement surface.

3. Full-width fog seals can be performed as economically as partial-width fog seals which cover 70 percent of the pavement surface area.

4. Full-width fog seals can be performed as economically as partial-width chip which cover 70 percent of the pavement surface area.

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C 2. Personal correspondence with John Hellmer, Maintenance Systems Engineer, State of California, Department of Transportation, Division of Highways, April, 1977.

C 3. Personal correspondence with Ray Zink, Maintenance Engineer, North Dakota, State Highway Department, April 13, 1977.

C 4. Personal correspondence with Kenneth J. Davis, Assistant Maintenance Engineer, State of Nevada, Department of Highways, April 29, 1977.

C 5. Personal correspondence with Samuel F. Lanford, Assistant State Engineer, Maintenance Section Arizona Department of Transportation, Highway Division, February 23, 1977.

C 6. "Performance Standards" Arizona Department of Transportation, Highway Division, Maintenance Section, July 1, 1976.

C 7. "Manual of Instructions," Vol. II "Programing and Scheduling," State of California, Department of Transportation, Maintenance, July, 1976.

C 8. "Maintenance Management System - Manual of Instruction," State of Nevada, Department of Highways, July, 1973.

C 9. "Maintenance Manual," North Dakota, State Highway Department, Maintenance Division.

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C 11. "Bid Price Trends on Federal Aid Highway Contracts," Engineering New Record, December 23, 1976.

C 12. Epps, J. A. et al., "The Development of Maintenance Management Tools for use by the Texas State Department of Highways and Public Transportation," Research Report 151-4F, Texas Transportation Institute, September, 1976.

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APPENDIX D. A REVIEW OF PRESTRESSED CONCRETE PAVEMENT CONSTRUCTION PRACTICES AND SELECTED VARIABLES AFFECTING THE APPLICATION OF PRESTRESSING AS A REHABILITATION METHOD

Concrete pavements have been used extensively throughout the world as a "high-type" of pavement. But the service lives that have been obtained with concrete pavements have been generally less than that obtained for other concrete structures (D 2). New research information has shown that this is particularly true for some of the newer concrete paving techniques in popular use today.

The most widely used concrete pavement types to date are as follows:

1. Jointed Unreinforced
2. Jointed Reinforced Concrete
3. Continuously Reinforced Concrete (used extensively during the last 10 to 15 years with the first significant construction built in Indiana in 1938 (D 36).

The problems associated with concrete pavements appear to be primarily joint and crack related distress. Joint distress can stem from several mechanisms among which are joint lockup, lack of adequate load transfer between slabs and subgrade movement. Cracking type of distress can be caused by numerous mechanisms. Lack of adequate flexural strength to resist fatigue and/or overloading is one of the principal causative mechanisms. Additionally, shrinkage and curling stresses in concrete pavements can cause or contribute to cracking.

Both joint distress and the various kinds of cracking that can occur in concrete pavements are very important in that they will generally lead to pavement deterioration and ultimately to failure if not corrected.

Jointed unreinforced concrete pavements are generally built with contraction/expansion joints placed every 15 to 25 ft (4.6 to 7.6 m) with no steel reinforcement being contained in the slab. These relatively short slabs are designed to allow for adequate expansion and contraction of the slab so that cracking will not occur due to environmental and curing induced stresses. However, the numerous joints that are required can fail and therefore cause the pavement serviceability to deteriorate.

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Jointed reinforced concrete pavements also contain numerous joints but are generally spaced further apart, say on the order of 25 to 100 ft (7.6 to 30.5 m). The reinforcing steel is used to strengthen the slabs against stresses induced by the environment because of their longer length. This reinforcing steel may or may not add to the load carrying capability of the pavement but it is generally positioned so as to add little to the flexural strength. Its primary function is to resist the slab movements induced by environmental and curing stresses. The numerous joints in this type of pavement present the same problem as they do in the unreinforced pavements.

Continuously reinforced concrete pavement (CRCP) differs primarily from the other two types in that its design does not require joints other than for construction stoppages and bridge abutments or other related structures. To allow for a joint-free pavement, longitudinal reinforcing steel in the amounts ranging generally from 0.5 to 0.7 percent of the transverse cross-sectional area is used. This amount of reinforcement is usually adequate to hold the slab together as the pavement slab shrinks after placement. Transverse cracks do occur and are eventually spaced about 6 ft (1.8 m) apart on properly designed and constructed CRCP (D 36). This rigid pavement type has gained wide usage in recent years. In 1958 approximately 80 equivalent two-lane miles, (129 equivalent two-lane kilometers) of CRCP had been built or was under contract. By 1971 about 10,000 equivalent two-lane miles, (16,000 equivalent two-lane kilometers) had been constructed or under contract.

Joint-free CRCP was major advancement in pavement design but a recent study in Texas (D 42) has indicated significant distress in some of these pavements.

The rehabilitation of CRCP as well as all rigid pavement types is expensive. Not only is rehabilitation expensive but maintenance/

rehabilitation alternatives are limited. The most frequently used maintenance method in use today for resurfacing these pavements is asphalt concrete overlays, most of which are only marginally effective due to reflective cracking caused by the underlying concrete layer.

Review of Prestressed Concrete Pavements

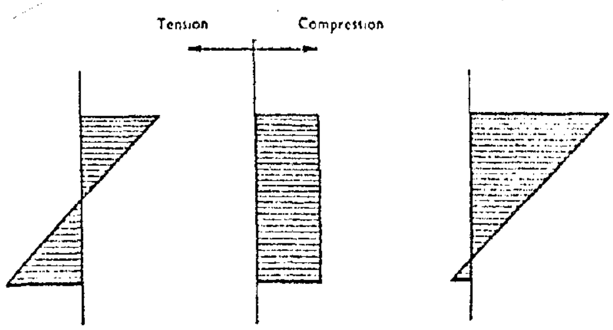
After creating a possibly overly dismal picture of current concrete pavement practice, it is time to introduce an old but paradoxically new concrete pavement type which may overcome some of the difficulties observed with the previously mentioned three types. This "other" type is prestressed concrete pavement. Prestressed concrete pavement appears to be an attractive alternative in two ways. First, it may be a suitable alternate method for new rigid pavement highway construction. Second, existing prestressing methods may possibly be applied as a way to rehabilitate existing rigid pavements. The remainder of this paper will be devoted to a general review of existing rigid pavements which were constructed using prestressing methods and a review of some of the topics which should be examined in order to utilize prestressing as a rehabilitation method.

Prestressing a concrete pavement adds a compressive stress to the pavement cross section which is cumulative with the normal flexural strength of the concrete. This allows for a greater stress range in the flexural zone of the pavement and may be thought of as increasing the concrete tensile strength without increasing the modulus of elasticity of the concrete. Conventional concrete pavement design differs in that only the flexural strength of the unreinforced concrete can be utilized for load support (D 2). Figure D-1 shows how prestressing affects a concrete pavement subjected to an applied bending moment. Prestressing new pavements can provide the following (D 24):

1. Elimination of a large percentage of transverse cracks.
2. Reduction or elimination of cracks in the road surface which can result in a reduction of moisture in the pavement foundation.
3. Decreased pavement thickness.

Presently, there are three methods which can be used to apply prestressing to a concrete pavement:

1. Pretensioned steel. Steel strands are pulled to prescribed tension between anchors placed prior to concrete placement. The strands are cut near the ends and at joints after the concrete



A B A+B

A. Stress Distribution Across at Concrete Section Due to Applied Bending Moment.

B. Stress Distribution Due to a Uniform Prestress.

A+B. Resultant, Actual Stress Distribution Showing a Stress Shift.

From Reference D3

Figure D-1. Stress Shift Caused By Prestressing.

has attained required strength.

2. Post-tensioned steel. Horizontal strands or bars are coated or enclosed in tubes, unstressed before concrete is cast. After concrete strength development the steel is tensioned by jacking against the concrete end faces. Steel has been placed longitudinally, longitudinally and transversely, or diagonally.
3. Poststressed concrete. Plain concrete slabs are cast between anchors and compressed by jacks or wedges at ends and in transverse joints. No tendons are used longitudinally, but transverse post-tensioning is sometimes used in conjunction with the poststressing operation (D 24).

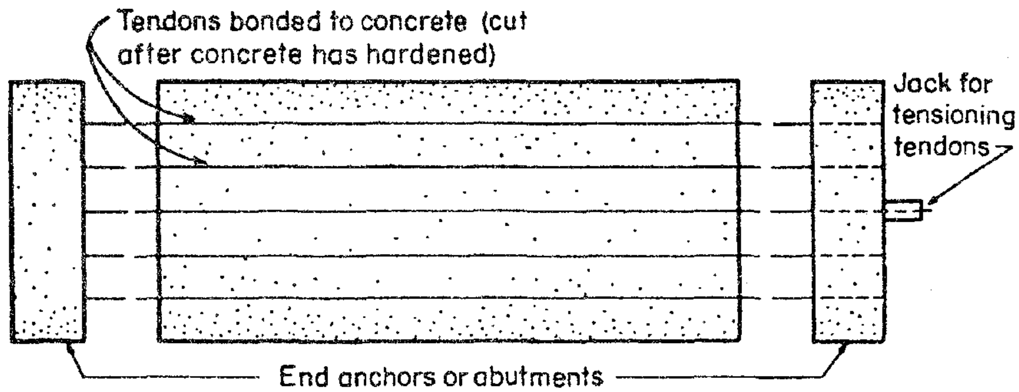
Figures D-2, D-3, and D-4 are idealized plan views of typical prestressed pavements. Figure D-2 show two pretensioning methods with the longitudinal pretensioning being the more common for this prestressing technique. Figure D-3 shows two post-tensioning methods, again with new longitudinal placement pattern being the more common. Figure D-4 shows a typical poststressing method.

The basic requirements to be met in prestressed pavement design are the following:

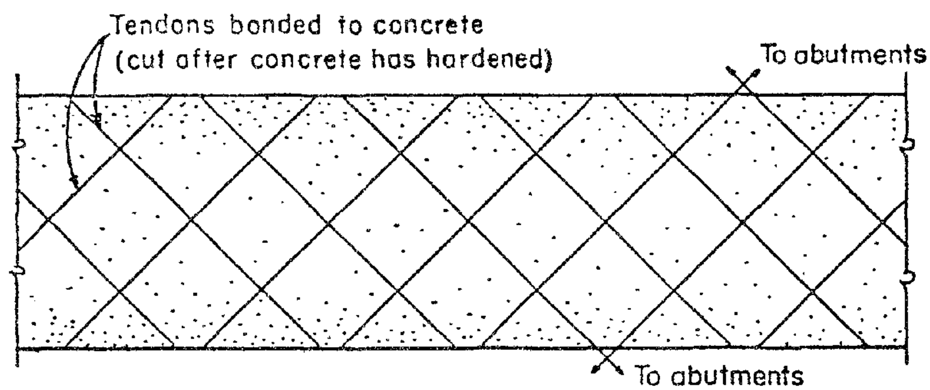
1. The section thickness must be adequate for the imposed stresses and climate variations.
2. The number of joints should be reduced by making the slab as long as possible, consistent with economy and construction needs.
3. At the joints, provision must be made to permit substantial longitudinal movement, sustain adequate load transfer, and protect the foundation (D 24).

As can be readily observed, prestressed pavements offer several potential advantages over the more conventional rigid pavement types. Even though the first major prestressed pavement was constructed at Orly Airport in Paris in 1946, this type of pavement has not gained widespread usage. A review of the literature shows that a minimum of sixty to seventy prestressed pavement projects have been completed throughout the world since 1946. It is interesting to note that most of these projects were located in Europe.

A relatively brief summary of these projects as described in the literature would be informative in several respects. First, a more



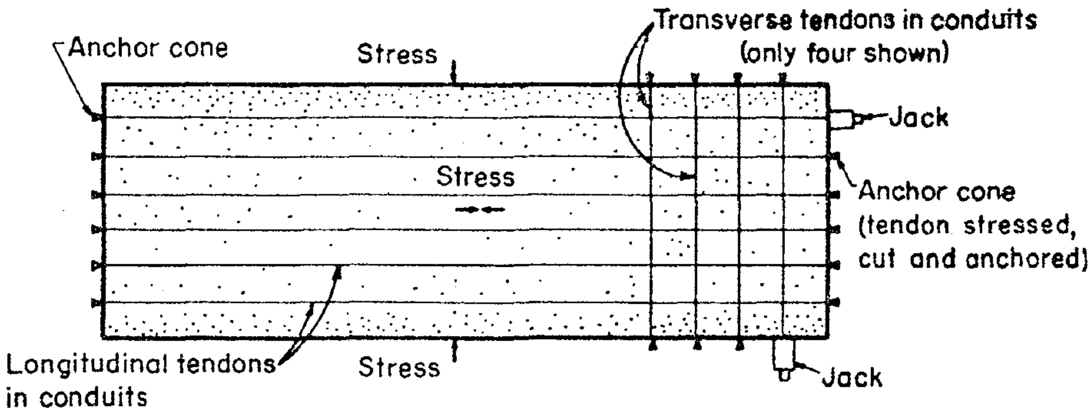
Longitudinal Pretensioning



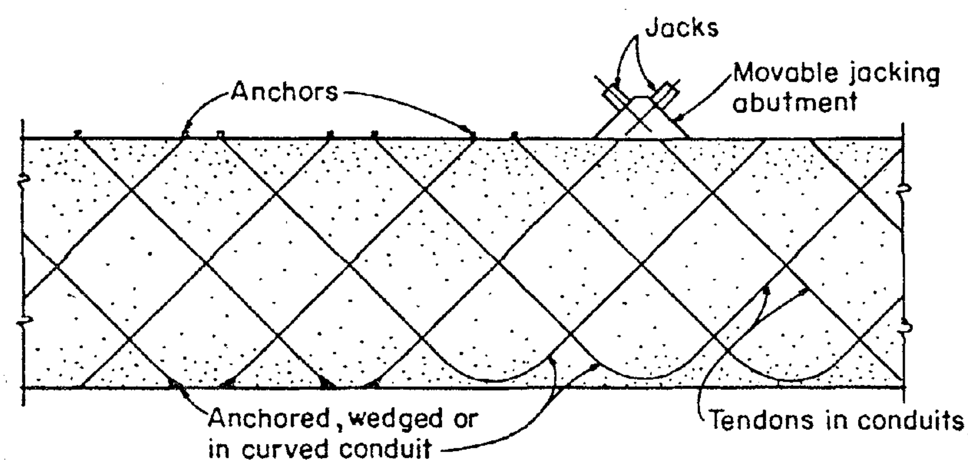
Diagonal Pretensioning

From Reference D 54

Figure D-2. Two Pretensioning Methods.



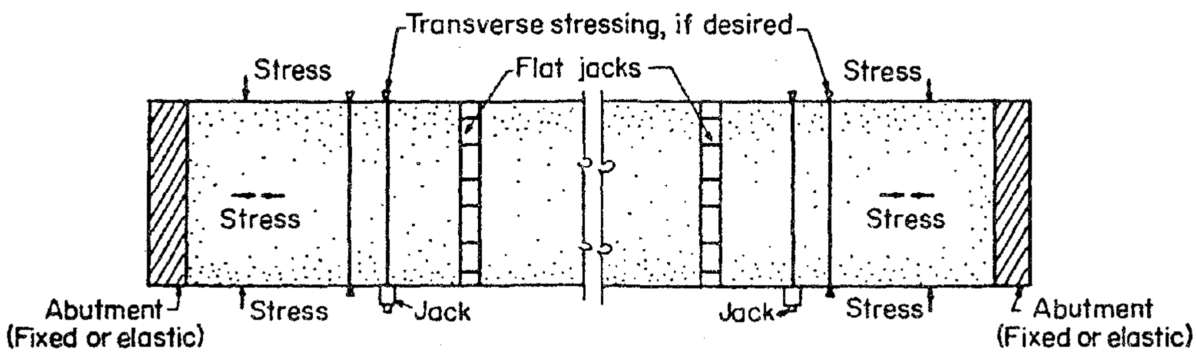
Longitudinal Post-Tensioning



Diagonal Post-Tensioning

From Reference D 54

Figure D-3. Two Post-Tensioning Methods.



From Reference D 54

Figure D-4. Typical Longitudinal Poststressing Method.

TABLE D-1. Chronological Summary of Known Prestressed Pavement Projects

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1946	Luzancy, France	Road-Bridge Approach	61 76	18 18	7.9 @ CL 6.3 @ edge	300 300	240 240	Post-tensioned diagonally with cables at 45^0 to CL	Unk	Good as of 1960	3, 13, 16, 54
1946	Orly Aerodrome, France	Runway	1380 (total length of individual precast slabs)	197	6.3	476	476	Precast 39" square slabs post-tensioned transversally with cables and restrainted longitudinally with abutments	Concrete pavement on waterproof kraft paper on 4" compacted gravel	Good as of 1958	3, 4, 37, 54, 57
1947	Brussels Airport Belgium	Runway	243	164	6.3	Unk	Unk	Similar to system used at Orly but both longitudinal & transverse post-tensioning was used	Unk	Good as of 1956	37, 54
1949	Esbly, France	Road	164	18	6.0	228	228	Post-tensioned diagonally with cables at 45^0 to CL	Unk	Good as of 1960	3, 13, 54, 57
1949	London Airport, Great Britain	Runway	360	120	6.5	550	550	Precast square slabs post-tensioned transversally with cables and restrainted longitudinally with abutments	Concrete pavement on bituminous paper on sand on gravel or brick earth	Good as of 1956	3, 37, 54
1950	Crawley, Sussex (Great Britain)	Road	404	24	6.0	212	23	Post-tensioned with diagonal cables at $18\frac{1}{2}^0$ to CL	Unk	Good as of 1955	3, 54
1951	Schiphol Airport, Netherlands	Airfield Pavement	5/92' slabs 5/136' slabs	136 136	5.5 5.5	510-570 510-570	510-570 510-570	Longitudinal: Post tensioned with cables Transverse: Post-tensioned with cables	Concrete pavement on 0.08" bitumen on 2" concrete on 18-24" compacted sand on clay subgrade	Good as of 1956	37, 54, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1951	Wexham Springs,	Road	110	10	6.0	190	0	Longitudinal: Post-tensioned with cables	Unk	Good as of 1960	3, 13, 54
	Buckinghamshire	Road	130	10	6.0	90	0		Unk	Good as of 1960	
	(Great Britain)	Road	190	11.5	6.0	245	0		Unk	Good as of 1960	
1951	John Laing's, Ltd. Great Britain	Road	3000 (15/180' slabs)	12	10.0	410	0	Longitudinal: Post-tensioned with cables	Unk	Unk	54
			(2/150' slabs)	14 2/3	6.0	300	0				
1951	St. Leonards, Hampshire (Great Britain)	Road	1200 (3/400' slabs)	24	6.0	280	13	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Unk	Poor as of 1955	3, 54
1952	Basildon, Essex (Great Britain)	Road	660 (4/165' slabs)	18	6.0	280	0	Longitudinal: Post-tensioned with cables	Unk	Good as of 1960	3, 13, 54
1952	Woolwich, Great Britain	Road	3350	18-24	6.0	250	28	Post-tensioned diagonally with cables	Unk	Good as of 1960	3, 13, 54
1953	Orly Aerodrome, France	Runway	1410 (intermediate jacking locations every 360')	82	7.1	Variable, 256 initially	256	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned cables	Concrete pavement on kraft paper on 1½" of sand on 8" compacted hoggin	Good as of 1958	3, 4, 37, 39, 54, 57

1 ft = 0.305 m., 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1953	Boug-Servas, France	Road	984 (intermediate jacking locations every 195')	23	4.7	215 (Final) 740 (Initial)	Experimental Variation	Longitudinal: Post-stressed with flat jacks Transverse: Part post-tensioned cables, part reinforcing rods	Concrete pavement on waterproof paper on prepared subgrade	Unk	3, 7, 39, 54, 57
1953	Heidenheim, Germany	Road	365 365 365	28 28 28	6.0 6.0 6.0 @ CL 8.0 @ edge	300 240 374	128 43 125	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables Post-tensioned diagonally with cables at 30^{0} to CL	Unk	Fair as of 1955 Transverse cracks Good as of 1955	3, 54
1953	Patuxent River Naval Air Station, Maryland	Experimental Pavement	500	12	7.0	690 initially, under testing varied between 600 to 0	Under testing varied between 240 to 0	Longitudinal: Post-tensioned with strands Transverse: Post-tensioned with strands	Concrete pavement on building paper on 1" sand layer on 4" sand-clay layer on 2' sand-clay and gravel subbase For this system, expected coefficient of friction between 0.60 and 0.72	NA	6, 9, 15, 54, 57
1954	Australia	Runway (Overlay)	27.5	27.5	2.2	Unk	Unk	Unk	Unk	Unk	3, 37
1954	Port Talbot, S. Wales (Great Britain)	Road	1500 (5/300' slabs)	22	6.0	220-320	Varied between 0 to 35	Post-tensioned longitudinally and diagonally with cables. One slab jacked	Unk	Unk	3, 13, 54

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1954	South Benfleet, Essex (Great Britain)	Road	330	20	4.0	550	50	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned with wires	Unk	Good as of 1960 Although reinstatement of longitudinal compression was required	3, 13, 54
1954	Maison Blanche Airfield, Algiers Algeria	Runway	8000 (intermediate jacking locations every 1000')	197	7.1	1000, min. 250 psi at lowest temperature, 1250 psi at max. temperature	250	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned cables	Concrete pavement on waterproof kraft paper on 1" beach sand on bitumen coating on three 4" layers of graded material on subgrade	Excellent as of 1963	4, 10, 37, 40, 54
		Taxiway	7700 (intermediate jacking locations every 1000')	82	7.1	1000, min. 250 psi at lowest temperature, 1250 psi at max. temperature	250				
1954	Speyer, Germany	Road	558	20	8.0	85-341	0	Longitudinal: Post-tensioned with cables	Unk	Unk	54
1954	Montabaur, Germany	Road	460	25	3.4	460	460	Longitudinal: Post-tensioned with steel bars Transverse: Post-tensioned with steel bars	Unk	Unk	54

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1954	Margelstetten, Germany	Road	788	25	6.0	299	29-142	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned	Unk	Unk	54
		Road	394	25	6.0	370	128	Post-tensioned diagonally	Unk	Unk	54
1954	Vienna Airport, Austria	Airfield Pavement	656	197	8.0	213	107	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Concrete pavement on 3/4" sand layer on 23" subbase	Unk	54
1955 (circa)	Bingen, Germany	Road	3280 (individual slabs 492')	30 (approx.)	6.3	700	210	Longitudinal and Transverse: Post-tensioned strands encased in metal tubes	Concrete pavement on two layers of talc-coated paper on 5.9" thick bituminous base on 15.7" sand and gravel subbase. Estimated coefficient of friction = 0.35	Good as of 1960	13, 17, 18, 57
1955	Naz, Switzerland	Road	1640 (flat jacks placed every 229')	8.2	4.7	840	56	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned with single strand cables	Unk	Good as of 1960	10, 13, 54, 57
			1640 (wedges placed every 393')	18	4.7	980	0	Longitudinal: Post-stressed with wedges Transverse: None - although light reinforcing steel was utilized	Unk	Good as of 1960	

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1956	San Antonio International Airport, San Antonio, Texas	Taxiway Overlay (Placed over 6" PCC pavement)	80 80	75 75	4.0 4.0	425 175	425 175	Longitudinal and Transverse: Post-tensioned strands in metal conduit	Concrete overlay on kraft paper on asphalt concrete level-up on existing 6" PCC pavement. Assumed max. f between overlay and building paper to be 1.2	Good as of 1959	9, 38, 54
1956	Finningly, Great Britain	Airfield Pavement	200	200	6.0	250	250	Precast slabs 30'x 9' post-tensioned in both directions with cables	Unk	Unk	54
1956	Vienna, Austria	Road	427	24	8.0	228	107	Longitudinal: Post-tensioning Transverse: Post-tensioning	Unk	Unk	54
1957	Wolfsburg, Germany	Road	5906	30	6.3	356-498	185	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Unk	Unk	54
1957	Moricken-Brunegg Switzerland	Road	6575 (200', 400' 600' slabs)	18	5.0	840-1330	0	Longitudinal: Post-stressed with wedges	Unk	Unk	54, 57
1957	Cesena, Italy	Road	1641	24.5	3.2	355 (@ 68°F)	355 (@ 68°F)	Longitudinal: Pre-tensioned with steel cables Transverse: Pre-tensioned with steel cables	Unk	Unk	54, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1957	Sharonville, Ohio	Experimental Pavement	500	25 (2/ 12.5' slabs)	9.0	400	Variable	Longitudinal and Transverse: Post-tensioned steel bars in steel conduits	Concrete slabs on waterproofed paper on ½" layer of sand on variable thickness subbase	Slabs loaded to failure in experimental project	9, 39, 54, 57
			500	25 (2/ 12.5' slabs)	9.0	200	Variable				
1957	Pittsburgh, Pennsylvania	Experimental Highway Pavement (built by Jones & Laughlin Steel Co.)	530 (30',100', 400' slabs)	12	5.0	450	0	Longitudinal: Post-tensioned strands in metal conduit	Concrete pavement on 1" layer of sand on 6" granular subbase. Tests indicate coefficient of friction approx. 0.7	NA	1, 21, 39, 54, 57
1958	Gatwick Airport, London (Great Britain)	Airfield Pavement	290	230	5.0	300-350	250	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Concrete pavement on waxed paper on 3" concrete layer	Unk	54
1958	Strosshof, Austria	Road	164	37	6.0	235	142	Longitudinal: Post-stressed Transverse: Post-tensioned	Unk	Unk	54

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1958	Osaka City, Japan	Road	197	18	6.0	495	108	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with bars	Unk	Unk	54
		Road	134	36	6.0	426	158	Post-tensioned diagonally with cables at 30° to CL	Unk	Unk	54
1958	Melsbroek Airport, Belgium	Taxiway	1150	75 (18/4' panels)	4.0	620 (center area) 810, 935 (edge area)	470	Longitudinal: Pre-tensioned strands in individual 4'x39' precast panels. Transverse: Precast panels connected transversally with post-tensioned, grouted cables.	Concrete pavement on 1' layer of compacted sand	Good as of 1960	10, 13, 54
1958	Woodbourne, New Zealand	Airfield Pavement	450 (3/150' slabs)	150	6.0	305	305	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Unk	Unk	54
1958-59	Anif-Saltzburg, Austria	Road	2625 (6 wedge jacking joints ~425' on centers)	25	6.3	1138	0	Longitudinal: Post-stressed with wedges	One blowup occurred on a similar prestressed pavement constructed at the same time. Pavement thickness was 7.9" instead of 6.3"	Unk	54, 57
1959	Hopsten, Germany	Runway	9842	98	5.5	165	165	Longitudinal: Post-tensioned with cables Transverse: Post-tensioned with cables	Concrete pavement on double layer of paper on 3/4" fine sand on 17" stone subbase	Unk	13, 54

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1959	Biggs Air Force Base, El Paso, Texas	Taxiway	1500 (3/500' slabs)	75 (3/ 25' slabs)	9.0	350	150	Longitudinal: Post-tensioned tendons in flexible metal conduit Transverse: Post-tensioned tendons in rigid metal conduit	Concrete pavement on polyethylene sheeting on ½" sand layer on 12" stabilized aggregate base. Estimated coefficient of friction = 0.75	Good to fair as of 1976. Pavement was in good condition but serious joint problems were continuing	8, 9, 54, 57
1959	Vienna Airport, Austria	Runway	3200 (8/400' slabs)	147.5 (6/ 24.7' slabs)	6.0	654	142	Longitudinal: Pre-tensioned wire (227 psi) + Post-stressing with flat jacks (427 psi) Transverse: Post-tensioned cables in conduits	Concrete pavement on paper on ¼" of sand asphalt on gravel base. Coefficient of friction less than 0.5	Unk	11, 54
		Taxiway	3609 (10/361' slabs)	74 (3/ 24.7 slabs)	6.0	654	142			Unk	
		Apron	1425 (4 slabs of variable length)	172 (7/ 24.7 slabs)	6.0	227	142	Longitudinal: Pre-tensioned wire Transverse: Post-tensioned cables in conduits		Unk	
1959	Melsbroek Airport, Belgium	Runway	11,100 (intermediate jacking locations every 1082')	148	7.1 (9.8" at intermediate jacking locations	1650 (with pavement temperature @ 90°F as measured during June, 1961)	Unk	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned with cables	Concrete pavement on 3/4" layer of sand on 7.9" base course on sandy silt subgrade	Some cracking occurred at low temperatures and two compression failures at active joints occurred in 1962 & 1963	10, 13, 54, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1959	Dietersheim, Germany	Road	2953 (6/492' slabs)	25	6.3	455-654	213	Longitudinal: Post-tensioned Transverse: Post-tensioned	Unk	Unk	54, 57
1960	Between Zwartberg and Meeuwen, Belgium	Road	11,484 (26/443' slabs)	26	3.2, 4.0, 4.7	284,427,569 (above prestress are design values - actual prestressing values varied greatly)	107, 273	Longitudinal: Post-stressed with flat jacks Transverse: Post-tensioned with steel strands	Concrete pavement on kraft paper on 3/4" sand layer on cut-back asphalt sprayed on compacted silty sand subgrade	During 1961 blow-up occurred in 2 of the 3.2" slabs. Another slab failed in 1962 and joint faulting occurred in 1963.	13, 54, 57
1960	Boudry, Switzerland	Road	4265	34.5	6.0	None at 50°F	0	Longitudinal: Post-stressed with flat jacks	Unk	Unk	54
1960	The Hague, Netherlands	Road	328	28.5	4.7	384	0	Longitudinal: Pre-tensioned with strands	Unk	Unk	54
1960	Lemoore Naval Air Station, California	Taxiway	512	75	6.0 @ CL 9.0 @ Edges	365	225	Longitudinal and Transverse: Post-tensioned tendons in steel conduits	Concrete pavement on two layers of polyethylene sheeting on soil cement base. Estimated coefficient of friction = 0.3.	Unk	9, 54

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1960	Gatwick Airport, London (Great Britain)	Apron	132 (9/15' slabs)	150	6.5	100+	100-	Longitudinal and Transverse: Post-tensioned steel bars wrapped with impregnated fabric to prevent bond	Concrete pavement on 1" sand layer on 4" of lean concrete on clay subgrade	Unk	19, 54
1961	Maison Blanche Airfield, Algiers Algeria	Runway	7700 (jacking intervals every 645')	148	6.4	1700, minimum 255 psi throughout slab	142, minimum throughout slab	Longitudinal: Post-stressed with flat jacks and elastic abutments Transverse: Post-tensioned wire strands	Concrete pavement on impervious paper on 1" layer of sea sand on 12" thick subbase on silty clay subgrade	Good as of 1963	40
1961	Winthorpe, Nottinghamshire (Great Britain)	Road	1060 (active joints 280' from each end) 1060 (active joints 280' from each end)	26 26	5.0 7.0	1200, minimum 100 residual prestress at lowest temperature	0	Longitudinal: Post-stressed with jacks Transverse: No transverse prestressing utilized	Concrete pavement on two sheets of polyethylene with a slip additive sandwiched between two sheets of building paper on 4" of lean concrete on 12" of limestone on silty sand subgrade. Coefficient of friction approx. 1.0.	Good as of 1967 although pavement restressing has been required each year to account for creep losses.	22, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1964	Lier, Belgium	Industrial Factory Road	1450 (active joint 287' from each abutment)	19'8"	4.7	1850, initial (Effective prestress temperature dependent. Over 5 year period ranged from approx. 40 psi @ 34°F to 1800 psi @ 100°F)	0	Longitudinal: Post-stressed with screw jacks	Concrete pavement on bond breaker consisting of two sheets of polyethylene between two sheets of kraft paper on 1" sand layer on 5.9" thick cement stabilized base	Good as of 1968 although a 1/8" longitudinal crack occurred approx. 3½ years after construction in the middle 2/3 of the road	34, 57
1965	Portugal	Runway	13,100 (42/312' slabs)	197 (8/ 24.6' slabs)	6.0	341	Unk	Longitudinal: Post-tensioned tendons Transverse: Post-tensioned tendons	Concrete pavement on two layers of paper on soil cement base	Unk	23
		Taxiway	10,500 (33/318' slabs)	98 (4/ 24.6 slabs)	6.0	341	Unk				
1971	Milford, Delaware	Road	300	14	6.0	238	0	Longitudinal: Post-tensioned with teflon coated 7-wire strands Transverse: No prestressing utilized although some conventional steel was placed	Unk	Unk	57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1971	Dulles International Airport	Road	3200 (consists of six slabs ranging from 400' to 760' in length)	24	6.0	200 (applied at ends of slab)	0	Longitudinal: Post-tensioned wire strands Transverse: No prestressing applied but No. 3 or No. 4 bars placed @ 30" centers	Concrete pavement on two layers of polyethylene sheeting on a 6" cement stabilized subbase	Good as of 1976. Although several joints had been removed and replaced. Some transverse cracking of the slabs has been observed	41, 57
1972	US 222, Berks County, Pennsylvania	Road	500	24	6.0	204 (design stress after losses at the ends of the pavement)	0	Longitudinal: Post-tensioned with 7-wire strand in steel tubing or sheathed with polypropylene. Transverse: No prestressing utilized but No. 3 or No. 4 bars were placed every 30"	Concrete pavement on two layers of polyethylene sheeting on sand wept over a 3" bituminous concrete base on a 9" subbase	Good as of 1976. Although one transverse crack and one oval shaped crack had occurred.	48, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

TABLE D-1 (Continued)

Date	Location	Type Facility	Length (ft)	Width (ft)	Thickness (in)	Prestress Amount Longitudinal (psi)	Prestress Amount Transverse (psi)	Prestressing System	Friction Reducing Layers & Sublayers	Pavement Condition	References (D)
1973	Route 114, Cumberland County, Pennsylvania	Road	13,232 (23/600' slabs nomial)	24	6.0	244 (design stress after losses at the ends of the pavement)	0	Longitudinal: Post-tensioned with 7-wire strand in polypropylene conduit	Concrete pavement on two layers of polyethylene on sand wept over 6" bituminous base on 6" unstabilized subbase	Good as of 1976. Serviceability Index rating was 3.9. Narrow transverse crack was observed in 2 of the 23 slabs. Repairs had been made at 2 of the 19 joints.	53, 57

1 ft = 0.305 m, 1 inch = 25.4 mm, 1 PSI = 6.89 kPa

general knowledge of the state-of-the-art of prestressed pavement construction could be ascertained. Second, from this collection of background material, enough key information could be made available to conduct a proper analysis into the rehabilitation possibilities of prestressing methods.

Table D-1 is a chronological listing of 58 separate completed prestressed pavement projects. To the extent possible, the information includes project location, date of construction, type of facility, project length, width, and thickness, prestress amount and type of prestress system, and pavement performance.

The following observations partially summarize the material contained in the table:

1. Of the 58 projects reviewed, a minimum of 64 separate pavements were constructed. They can be classified as 36 roads, 11 runways, 7 taxiways, 7 miscellaneous airfield pavements, and 3 experimental. This results in 56% of the prestressed pavements reviewed being roads, 39% airfield pavements, and 5% experimental. No project was found which utilized prestressing techniques for repair of rigid pavements already in service.

2. Chronologically, the number of projects can be broken out as follows:

Year	No. of Projects Constructed
1946	2
1947	1
1949	2
1950	1
1951	4
1952	2
1953	4
1954	8
1955	2
1956	3
1957	5
1958	6
1959	5
1960	5

1961	2
1964	1
1965	1
1971	2
1972	1
1973	1

A weighted average of the above data indicates that the mean construction year occurred between 1956 and 1957. Averaging only the beginning and ending years (1946 and 1973) provides an average year of 1960. This is an indicator that the number of new prestressed pavements has been decreasing in recent years rather than increasing or simply that fewer are being reported in the literature.

3. The number of reported prestressed pavements, ordered from high to low, are listed below by the countries in which they were constructed:

Country	No. of Projects
Great Britain	13
United States	10
Germany	8
Austria	5
Belgium	5
France	5
Switzerland	3
Algeria	2
Netherlands	2
Australia	1
Italy	1
Japan	1
New Zealand	1
Portugal	1

The majority of prestressed pavement projects were constructed in Europe although a number have been constructed in the United States.

4. Of the projects reviewed, the number of types of prestressing systems and prestressing amounts are summarized as follows:

Prestressing System and Number		Average Prestress Amount
A. Pretensioned:		
Longitudinal	5	508 psi (3503 kPa)
Transverse	1	355 psi (2448 kPa)
*Diagonal	0	-

B. Post-Tensioned:		
Longitudinal	33	319 psi (2200 kPa)
Transverse	39	197 psi (1358 kPa)
*Diagonal	8	-
C. **Poststressed:		
Longitudinal	19	821 psi (5661 kPa)
Transverse	0	-
*Diagonal	0	-

It is apparent from the above data that post-tensioning systems have been the most requently used. For these systems the average amount of prestress used is 319 psi (2200 kPa) longitudinally and 197 psi (1358 kPa) transversally. The next most commonly used system is poststressing. The literature only revealed poststressing in the longitudinal direction with an average applied prestress of 821 psi (5661 kPa). This value represents an average upper amount and can vary significantly with increasing or decreasing pavement temperatures. The total number of pretensioned pavements is somewhat distorted to the low side due to the fact some of the pavements which were poststressed or post-tensioned were composed of precast, pretensioned units. But, for those pretensioned pavements reported, longitudinal and transverse prestress values of 508 psi (3503 kPa) were calculated, respectively.

5. A summary of the various project average lengths, widths, and thicknesses are as follows:

Type Facility	Avg. Length (ft)	Avg. Width (ft)	Avg. Thickness (in.)
Road	1988 (606m)	23 (7m)	5.8 (14.7cm)
Runway	5634 (1717m)	150 (46m)	6.4 (16.3cm)
Taxiway	3590 (1094m)	79	6.0 (15.2cm)
Misc. Airfield	537 (164m)	171 (52m)	6.1 (15.5cm)
Experimental	677 (206m)	16 (5m)	7.0 (17.8cm)

The averages shown for the above data are informative but can be deceiving. For example, the lengths of roads range from 61 to 13,232

*Note 1: Diagonal prestressing amounts were separated into longitudinal and transverse components.

**Note 2: Average represents upper limit of values reported.

ft, (19 to 4033m) widths range from 8.2 to 37 ft, (2.5 to 11.3m) and thickness range from 3.2 to 8.0 in. (8.1 to 20.3cm).

Selective Examination of Variables Affecting Prestressing Techniques For Use In Pavement Rehabilitation

The information contained in the literature is particularly informative in more carefully examining prestressing methods as a possible rehabilitation technique. A number of the factors of importance in the design of new prestressed pavements are also important when considering prestressing in rehabilitation of existing pavements. Three of these factors considered to be of the most importance are coefficient of friction between the slab and its underlying layer, expected distress mechanisms of prestressed pavements and material properties of in-service concrete pavements.

In addition to the problems common to new prestressed construction and rehabilitation are others unique to rehabilitation. Probably the most important of these is that of how to install the prestressing tendons in the pavement.

Slab Friction

In this research effort the problem of overcoming slab friction was identified as a major consideration in applying prestressing methods to existing rigid pavements. Timms (D 12) has noted that a 50 percent reduction in pavement friction could result in a 30 to 40 percent reduction in the required prestressing force.

The force that resists movement of pavement on its supporting foundation is complex. It is a combination of friction and cohesion and resembles the direct-shear test used in geotechnical engineering. Since the subgrade and base materials differ from one area to another, the approach taken in the rehabilitation study is to treat the force as friction. For comparison purposes, the coefficient of friction will be reported whenever possible.

The first two laws governing friction were first put forth by Leonardo daVinci and the third by Coulomb in 1785. These three laws are (D 25):

1. Frictional force is proportional to the normal force.
2. Frictional force is independent of the area of contact, and
3. Frictional force is independent of the sliding speed.

The first two laws are generally accepted as being true, the third law is not known as being true for all cases (D 20, D 25). The coefficient of friction depends on many factors such as surface smoothness, available moisture and temperature. There are no theories that adequately treat dry friction and thus the laws of friction are empirical laws which are based principally on observations. Friction between lubricated surfaces lends itself to a more theoretical approach.

The kind of friction acting on pavements may be considered to be dry friction. Although if slip additives are applied or injected beneath pavements or if significant moisture is present, this could partially change to the lubricated type of friction. For dry friction, there can be two types of coefficients used in the following general formula:

$$f = \mu N$$

where f = frictional force
μ = coefficient of friction
N = normal force

These two coefficients occur for a static case and for a kinetic (sliding) case. Kinetic coefficients of friction are often referred to in the literature as the "steady" or "sliding" coefficient of friction. Some of the kinetic coefficient of friction values reported actually continue to increase slightly as movement of the slab progresses. But, it is helpful to delineate between the static and kinetic condition of friction whenever possible. Discussion of slab friction will be primarily in terms of these (static and kinetic) coefficient of friction values. These values will act as a common denominator between the various experiments and tests reviewed.

It should be recognized that what is generally considered to be "true" coefficient of friction values may not always be directly applicable to pavement slabs. Many of the experiments reviewed showed that the coefficients of friction were often significantly influenced by the shearing strength of the layer underlying the slab. The resulting slab displacements often occurred in this underlying layer - not between the slab and the layer. This should be remembered while reading the following information obtained from the literature.

Slab movements of any concrete pavement are restrained to some extent by the friction between the slab and the subgrade support layer. For relatively short pavement slabs this kind of restraint does not appear to be of great significance. For much longer slabs, particularly for prestressed concrete pavement, support layer restraint commonly referred to as "subgrade restraint" can be of great significance. Subgrade restraint can lead to induced compressive stresses in a slab when it expands and to induced tensile stresses when the slab attempts to contract (D 14). Since concrete is weakest in tension, any potential tensile stresses in the slab can cause problems for a pavement. Subgrade friction tensile forces can be additive to load induced tensile stresses and will tend to counteract any prestressing applied to the pavement. These observations are partially depicted in Figure D-5.

It is reasonable that if the coefficient of friction between the slab and its underlying layers is high enough and cannot be reduced, prestressing methods would be ineffective in either new construction or in rehabilitation applications. Therefore, realistic ranges of slab friction values should be obtained to determine how effective rehabilitation prestressing techniques may be. First, an examination of friction values obtained from non-friction reducing construction is appropriate.

The earliest friction testing results in the literature were conducted by the U. S. Office of Public Roads and was first reported by Goldbeck (D 47) in 1917 and again in 1924 (D 51). The testing program was performed at the Arlington Experimental Farm belonging to the U. S. Department of Agriculture. Concrete slabs 6 ft (1.8 m) long by 2 ft (0.6 m) wide and 6 in. (15.2 cm) thick were cast on various types of subbases. The subbases ranged from smooth clay to 3 in. (7.6 cm) broken stone.

Force was measured with a spring dynamometer and applied to the test slabs by two men using a light steel rail as a lever. Displacements were measured by use of a Berry strain gage.

Table D-2 shows some of the results obtained in this testing program. In this table at a displacement of 0.050 in. (0.127 cm), the slabs were sliding and had reached their maximum friction values. Thus, the coefficients of friction shown at this displacement may be considered to be the static coefficients. As one might expect, the coefficient of friction for the 3 in. (7.6 cm) broken stone was the largest of all the subbase types but surprisingly was not much larger than those reported for the clay subbase at a 0.050 in. (0.127 cm) displacement.

Teller and Bosley (D 50) in 1930 reported the results of additional test slabs constructed by the Bureau of Public Roads in 1926. The test slabs were also 6 ft (1.8 m) long by 2 ft (0.6 m) wide and 6 in. (15.2 cm)

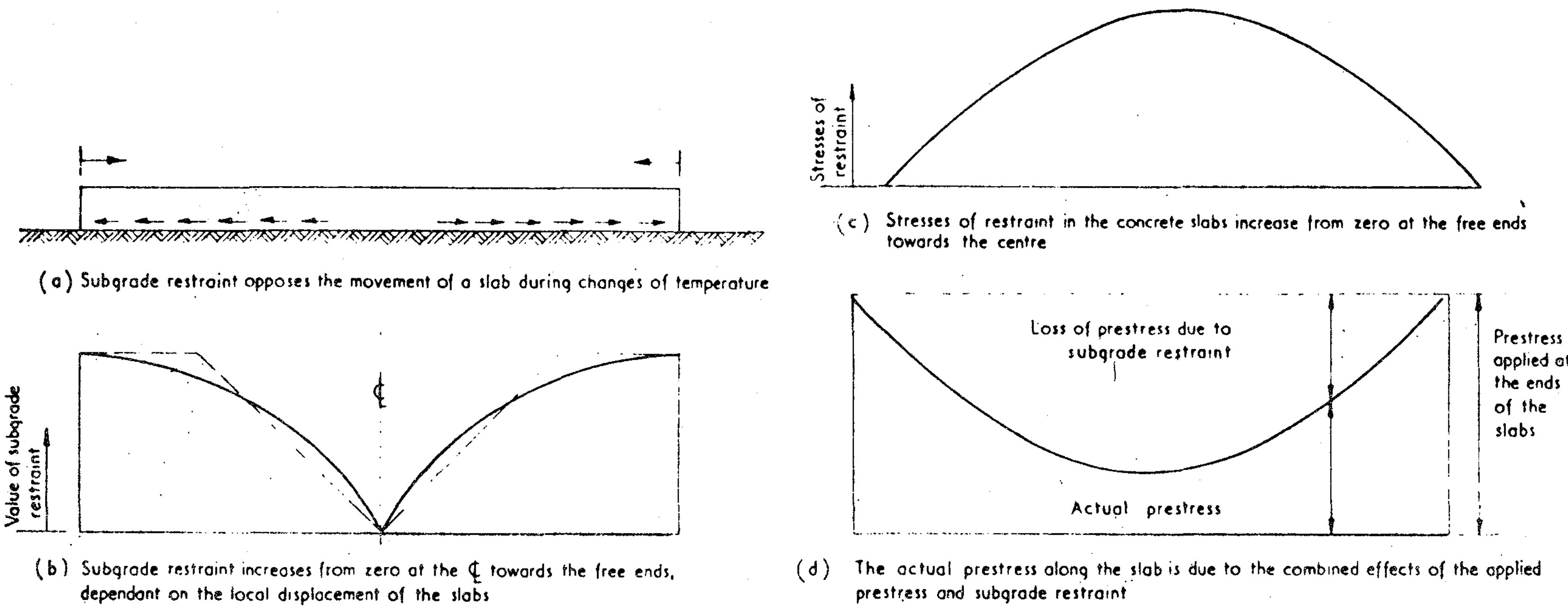


FIGURE D-5. Effect of Subgrade Restraint on the Prestress in a Concrete Slab.

From Reference <u>D14</u>

thick. Coefficient of friction values were obtained for these slabs on varying subgrade conditions which ranged from dry to frozen. Table D-3 shows the coefficient of friction (static) for the first test conducted on each subgrade and the slab displacement occurring at this maximum friction value. The information contained in Table D-3 indicates that the wet subgrade produces the lowest coefficient of friction and a frozen subgrade can have an extremely high value.

The fact that the wettest subgrade exhibited the lowest friction may be of possible value in prestressing existing pavements. Water may be considered for use as an injected lubricant between the slab and underlying pavement layers prior to prestressing such pavements.

Teller and Sutherland (D 49) reported in 1935 the results of friction tests conducted on 4 ft (1.2 m) by 4 ft (1.2 m) concrete slabs placed on a compacted silty loam subgrade. The thicknesses for these slabs varied from 2 to 8 in. (5.1 to 20.3 cm). Testing of these slabs involved moving them forward and backward through a displacement of 0.040 in. (0.102 cm) several times until the subgrade resistance became stabilized. In the same manner displacements of 0.070 and 0.100 in. (0.178 and 0.254 cm) were accomplished. The initial displacement level of 0.040 in. (0.102 cm) was chosen because this was felt by the authors to represent actual field movements for a 40 ft (12.2 m) slab.

It was concluded that the coefficient of friction (kinetic) values decrease with increasing slab thicknesses. This can be seen in Table D-4. The coefficient of friction (kinetic) reported for the 2 in. (5.1 cm) thick slab is approximately 1.4 times greater than for that reported for the 8 in. (20.3 cm) slab at the 0.040 in. (0.102 cm) displacement level. Additionally, Teller and Sutherland noted from their tests that the resistance to slab movement will be greater for the first movement of a new pavement than it will after the concrete has expanded and contracted a number of times.

A possible explanation for the coeffieient of friction values increasing with decreasing slab thickness may be due to the influence of the silty loam Mohr's envelope. Assuming that the Mohr envelope for this soil has a cohesion intercept and increased with a small angle of ϕ, the thin slabs tested should be expected to have, relatively, higher coefficients of friction than for thicker slabs tested on the same soil.

TABLE D-2. Coefficient of Friction Values for Various Subbase Types and Displacements as Reported by Goldbeck

Subbase Type	Coefficient of Friction @ 0.001 in. Displacement	@ 0.010 in. Displacement	@ 0.050 in. Displacement
Level Clay	0.55	1.30	2.07
Uneven Clay	0.57	1.29	2.07
Loam	0.34	1.18	2.07
Level Sand	0.69	1.24	1.38
3/4 in. Gravel	0.52	1.10	1.26
3/4 in. Broken Stone	0.44	0.92	1.09
3 in. Broken Stone	1.84	1.78	2.18

1 in. = 2.54 cm

From Reference D47

TABLE D-3. Maximum Coefficient of Friction Values and Associated Displacements for Four Subgrade Conditions

Subgrade Condition	Maximum Coefficient of Friction	Displacement at the Maximum Coefficient of Friction (in.)
Dry	2.0	0.05
Damp	2.5	0.04
Wet	1.7	0.12
Frozen	>8.5	>0.06

1 in. = 2.54 cm

Data From Reference D50

More specifically, by increasing the slab weight (or normal stress), the resulting shear strength of the soil would increase less rapidly.

Sparkes (D 55) in 1939 published the results of friction tests conducted at the Road Research Laboratory in Great Britain. The results were reported for concrete slabs placed on a clinker base course. The slab sizes were 4 ft (1.2 m) by 2 ft (0.6 m) and 7 ft (2.1 m) by 3.5 ft (1.1 m). The load was applied to the test slabs by the use of springs and horizontal displacements were recorded with dial gauges.

The results were reported in terms of slab resistance to movement (psf) versus slab movement. To convert this to coefficient of friction values, the unit weight of the concrete in the slabs is assumed to be 150 pcf (2403 kg/m^3). Making the necessary calculations results in coefficient of friction (static) values of 3.7 for the 4 ft by 2 ft (1.2 m) by 0.6 m) slab and 3.1 for the 7 ft by 3.5 ft (2.1 m by 1.1 m) slab both taken at a displacement of 0.065 in (0.165 cm). These values were obtained on the first loading cycle of the two slabs. Sparkes noted that subsequent cycles resulted in considerably less force to produce a given displacement.

Scott (D3) in 1955 observed that other research work done at the Road Research Laboratory found slab friction values of 1.25 to 2.0 when using a layer of sand between the slab and the base.

Highway Research Board Special Report 78 (D 54) reports that values of slab friction in the range of 1.5 have been found for slabs placed directly on granular subbases.

Fribert (D2) reported in 1955 that the coefficient of sliding friction is 1.5 or more on rough subgrades and between 1.0 and 1.5 on sandy, even subgrades.

In 1954, Friberg (D 56) reported an average coefficient of friction for a 100 ft (30.5 m) pavement slab. This slab was 20 ft (6.1 m) wide with a 9 in. - 7 in. - 9 in. (22.9 cm - 17.8 cm - 22.9 cm) thickness cross section on paper over a sand-loam subgrade. The maximum slab friction value reported was approximately 1.5. An interesting quote from Frigerg's paper is: "Frictional coefficients for long slabs may be smaller than for short slabs, as they are also smaller for increasing thickness, ..." Given that this statement is correct, then the magnitude of this potential

TABLE D-4. Coefficient of Friction Values For Varying Slab Thicknesses and Displacements on a Silty Loam Subgrade

Slab Thickness (in.)	Coefficient of Friction					
	0.01 (in.)	0.02 (in.)	0.03 (in.)	0.04 (in.)	0.07 (in.)	0.10 (in.)
8	0.8	1.2	1.5	1.8	2.1	2.2
6	0.9	1.3	1.6	2.0	2.4	2.5
4	1.1	1.5	1.8	2.2	2.8	3.1
2	1.3	1.7	2.1	2.5	3.3	3.5

1 in. = 2.54 cm

From Reference D49

decrease in slab friction could be of interest in prestressing "conventional" pavements.

Friberg reported on another pavement located near Stilesville, Indiana. This pavement was constructed as CRCP with a length of 1310 ft placed directly on a clayey silt subgrade. The subgrade had an average Liquid Limit of 33 and Plasticity Index of 12.

By using an equation and measured movements for two separate temperature changes, Friberg calculated the range of coefficient of friction values which could be expected at the ends of this pavement. These values were estimated to range between 0.3 to 1.5. Figure D-6 shows the actual and calculated movements for a contraction temperature change of 19°F (11°C) and an expansion temperature change of 23°F (13°C) for this particular Stilesville pavement.

The values of coefficients of friction so computed will depend on the actual stresses and strains in the CRCP pavement and are certainly affected by the expected cracking in these kinds of pavements. But, the range of frictional values presented should include the "actual" value for the Stilesville pavement.

Cholnoky (D 6) in 1956 presented the results of data collected on four test slabs. These concrete test slabs were 3 ft by 8 ft by 7 in. (0.9 m by 2.4 m by 17.8 cm) and were constructed and tested prior to the prestressed pavement installation at Patuxent Naval Air Station in Maryland. The "pulling" test conducted on the four slabs produced the results shown in Table D-5. Figure D-7 shows details of the test setup.

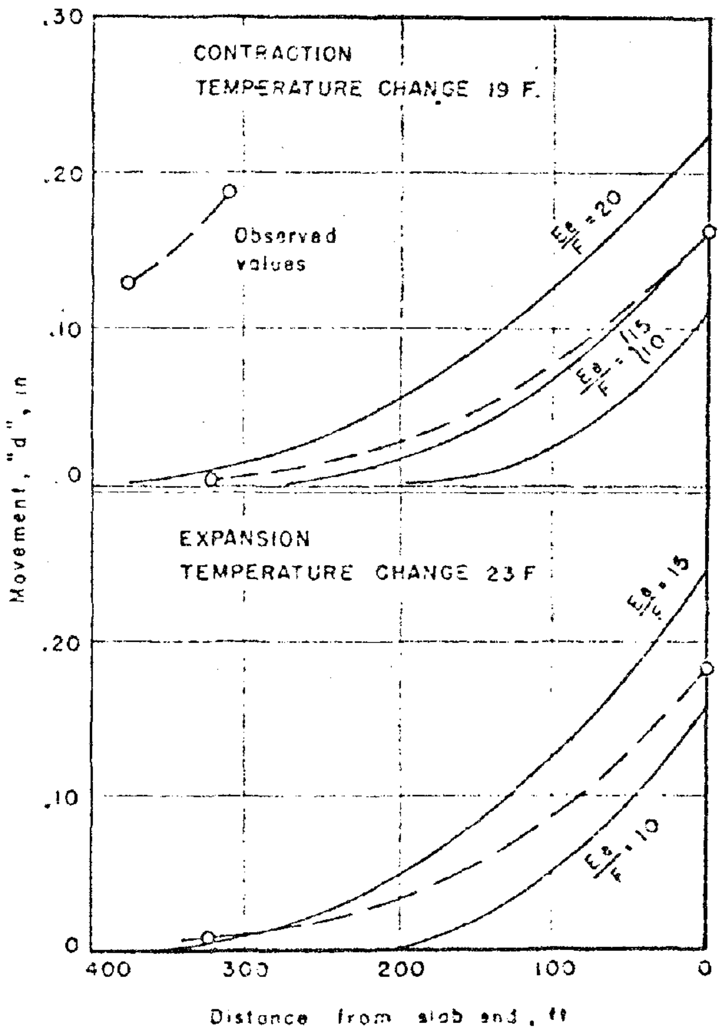
TABLE D-5. Friction Tests Conducted Prior Prestress Pavement Construction At Patuxent Naval Air Station

Slab No.	Coefficient of Friction At Failure	Coefficient of Friction During Sliding	Layers Below Test Slabs
1	0.72	0.60	1" of sand covered with one layer of building paper.
2	1.13	0.55	Two layers of building paper.
3	0.77	0.63	Two layers of copper-clad Sisalkraft paper, with copper faces turned together with powdered soapstone between.
4	*5.15	1.10	The prepared base directly, with no intermediate device.

*2nd measurement was 2.45 (D57).

From Reference D6

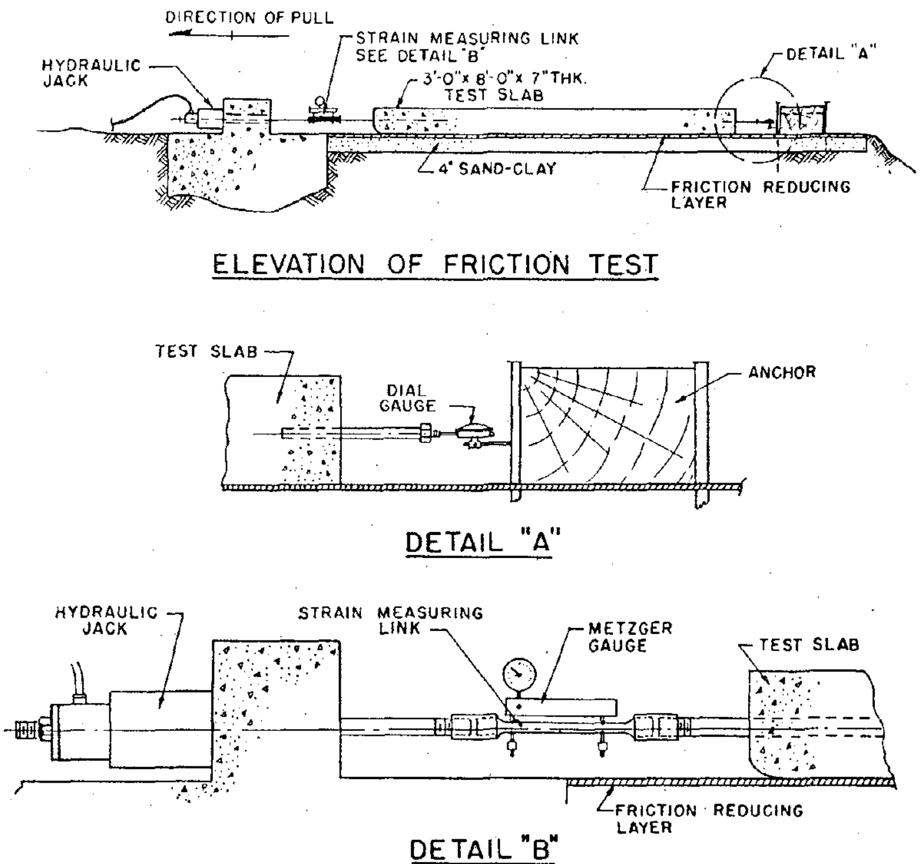
1 in. = 2.54 cm



1 ft = 0.3m
1 in = 2.54cm

From Reference D56

FIGURE D-6. Actual And Calculated Slab Movements At Stilesville For 19°F (11°C) and 23°F (13°C) Temperature Change.



1 in. = 2.54 cm

From Reference D6

FIGURE D-7. Friction Test Setup Utilized By Cholnoky.

Up until the time of this testing, no coefficient of friction values of the size shown for Slab No. 4 had been observed in any of the literature.

Yoder and Witczak (D 26) in their pavement design textbook make the following quote: "Most engineers use an average f equal to 1.5 for making stress calculations (in pavements)."

In a 1960 testing program conducted by the Louisiana Department of Highways (D 52), coefficient of friction (static) values were determined for concrete test slabs on a soil cement base. Three different subgrade surface cover treatments were evaluated but only one is of direct interest. This treatment was the application of 0.3 gal. per sq. yd. of asphalt emulsion to the soil cement surface as a curing membrane.

The test slab utilized was 9 ft (2.7 m) wide and 12 ft (3.7 m) long by 10 in. (25.4 cm) thick. The force was applied to the slab by use of a 100 ton (8.9 KN) hydraulic jack and slab movement was measured with Ames dials. The loading rate was 4,000 pounds (17,793 N) every 15 minutes until continuous sliding of the test slab occurred.

The concrete for the test slab was placed directly on the asphalt emulsion curing membrane. The soil cement was produced by stabilizing the subgrade with 12 percent cement by weight.

When tested under load, the slab moved 0.042 in. (0.107 cm) before sliding occurred. When sliding did occur the resulting coefficient of friction (static) was about 5.1. After this initial release of the slab, slab movement could be maintained with only 10,000 pounds (44,482 N) of force which equates to an approximate coefficient of friction (kinetic) of 0.8.

Stott (D 14) reported in 1961 the results of extensive laboratory testing of a small concrete slab on various underlying materials. The slab utilized in this testing program was 5 ft (1.5 m) by 4 ft (1.2 m) by 6 in. (15.2 cm) thick. The force was applied to slab by use of an electrically-driven screw and displacement was measured by dial and strain gauges. The rate of movement could be varied between 0 and 1/2 in. (1.27 cm) per hour which is similar to movements experienced by actual pavement slabs. Figure D-8 shows a typical force versus displacement loop utilizing this system. In Table D-6, the "force of restraint" is taken as the force required to move the slab divided by the area of the base of the slab, and the "coefficient of restraint" is this same force divided by the weight of the slab. Thus the coefficient of restraint is reported in lieu of coefficient of friction.

The results contained in Table D-6 show that for a concrete test slab cast directly on one of the sharp sands an initial (static) coefficient of restraint value of 1.05 was obtained. The "steady" (kinetic) coefficient of restraint following several cycles reduced this to 0.66. The results ot similar tests are also shown in Table D-6 but these other systems generally employed the use of paper or some other smooth material on which to cast the slab.

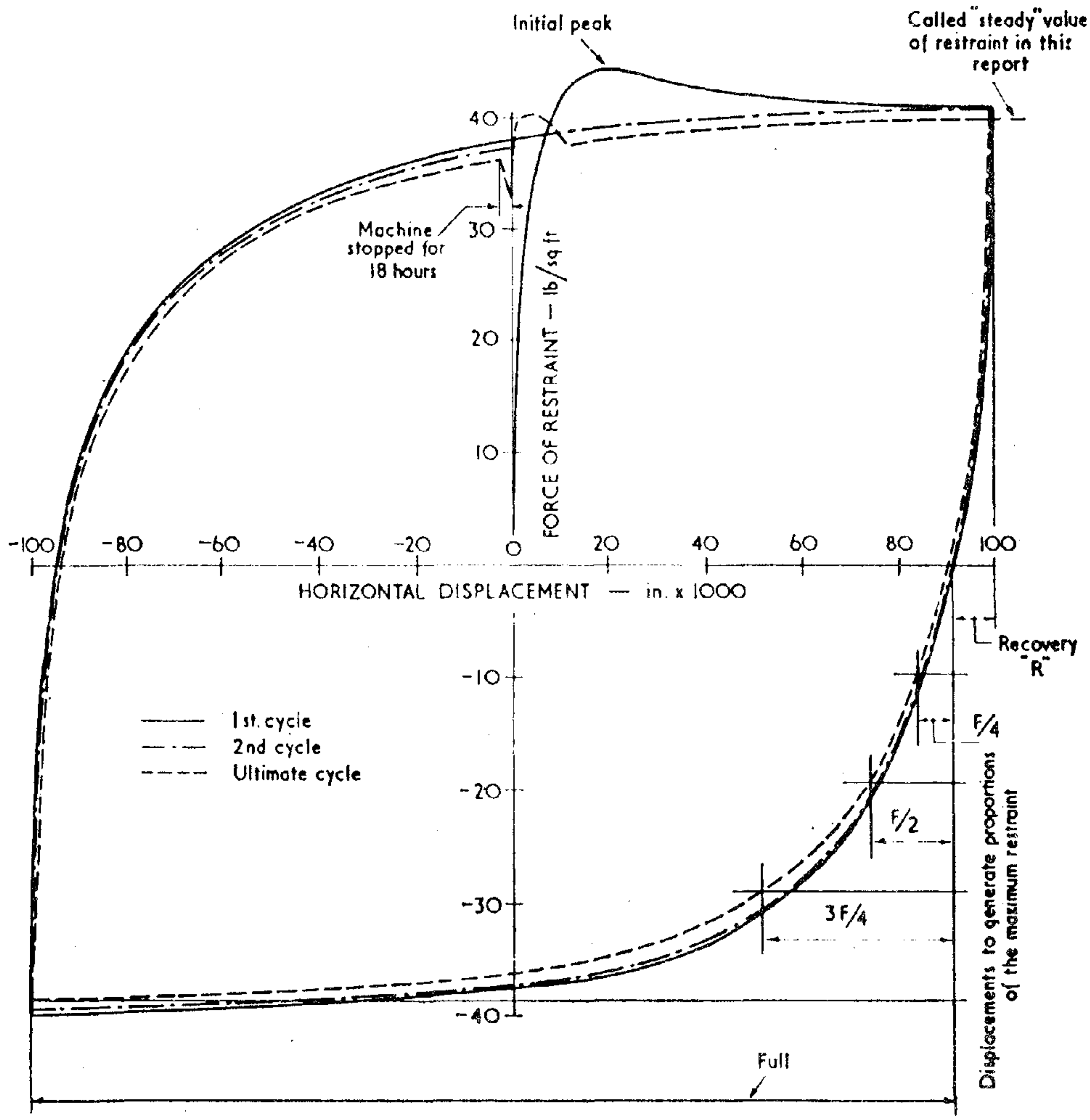
High-pressure lubricating oil approximately 1/8 in. (0.32 cm) thick was also placed on a smooth mortar base onto which the concrete slab was placed. Both the "initial" (static) and "steady" (kinetic) coefficient of restraint values reported were relatively low and ranged from 0.37 for "initial" and 0.33 to 0.49 for "steady" restraint conditions. Although these coefficients are low, the oil quickly squeezed out from underneath the slab. It is of interest to keep this experiment in mind while searching for materials to consider for injecting under existing rigid pavements.

Stott also tested a number of bitumens not reported in Table D-6. Bitumens were tested which had penetration values of 65, 100, 300 and 500. Thicknesses ranged from 0.115 to 0.275 in. (0.292 to 0.698 cm). This testing was accomplished to determine how bitumen induced restraint would depend on the following:

(a) grade of bitumen
(b) temperature of bituminous layer
(c) thickness of bituminous layer
(d) rate of slab movement

Figure D-9 shows some of the results of the testing performed. These curves indicate the following:

(a) restraint is directly proportional to rate of slab movement
(b) restraint is inversely proportional to thickness of the bituminous layer (after adjustment to a common test speed)
(c) softer bitumens give less restraint than harder materials (after adjustment for differences in test speed and thickness)
(d) restraint offered by bitumen increases significantly with decreasing temperature (roughly doubles for each 5.4°F (3°C) drop in temperature)



1 in = 2.54cm
1 psf = 47.88 Pa

From Reference D14

FIGURE D-8. Typical Force/Displacement Curbes.

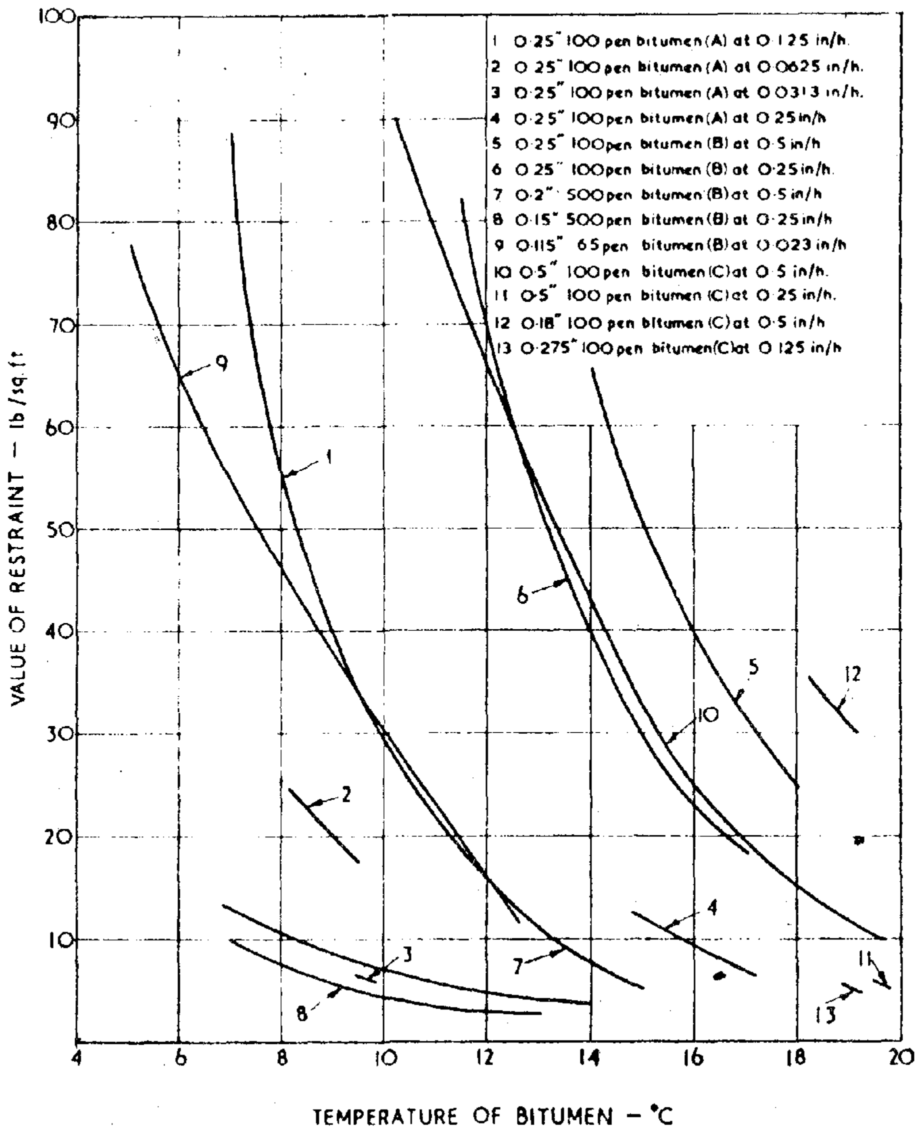
TABLE D-6. Slab-Moving Test Results for Materials Other Than Bitumens

Material under test	"Initial peak" restraint		"Steady" value of restraint after several displacements			
	Coeff. of restraint	Force of restraint (lb/sq.ft)	Coeff. of restraint* (from test with 6in slab only)	Force of restraint (lb/sq.ft) (from test with 6in slab only)	Formula of force (from test with slab and extra wts) (lb/sq.ft)	Displacement to generate ¾ max. restraint (in)
Sand and aggregates—						
Sharp sand A	0·74	50	0·62	42	10 + 0·47W	0·035
Sharp sand B	—	—	0·69	46	9 + 0·57W	0·065
Dune sand	0·78	54	0·69	48	—	0·035
Gravel	0·75	51	0·75	51	—	0·15
Limestone chippings	—	—	0·56	37	6·5 + 0·46W	0·12
(The above results are from 1in thick layers covered with concreting paper before casting the concrete)						
Sharp sand B (from ¼in thick layer covered with concreting paper)	—	—	0·64	43	3 + 0·59W	0·060
Sharp sand B (from 1in thick layer and concrete cast directly on the sand)	1·05	79	0·66	50	4·5 + 0·6W	0·50
Smooth mortar base	0·49	33	0·38	25	0 + 0·38W	0·007
Waterproof paper on smooth mortar base	0·90	60	0·65	43	10 + 0·48W	0·015
Hessian-backed paper on smooth mortar base	0·74	57	0·60	46	—	0·040
Polythene—						
Polythene on sand (slab *placed*, not cast, on to polythene. Sliding between concrete and polythene)	—	—	0·43	29	—	0·025
Polythene on sand (slab *cast* on to polythene. Sliding between polythene and sand)	—	—	0·55	38	4·5 + 0·44W	0·035
Polythene on smooth mortar base	Impracticable due to rapid wear					
Paraffin wax on smooth mortar base	1·11	74	0·17 to 0·34	11 to 22	2 + 0·14W	0·080
High-pressure lubricating oil (cardium compound D) on smooth mortar base	0·37	28	0·33 to 0·49	25 to 37	—	0·007
Asphalts—						
¾in thick asphalt composed of 6 per cent of Shelphalt by weight to Thames Valley sand	0·86	67	0·3 to 0·6	23 to 46	0 + 0·3W	0·030
Ditto, but with 10 per cent of Shelphalt by weight	0·64	48	Similar results to previous asphalt			

* Coefficient of restraint is the slab-moving force divided by the area of the base of the slab.

1 in. = 2.54 cm
1 psf = 47.88 Pa

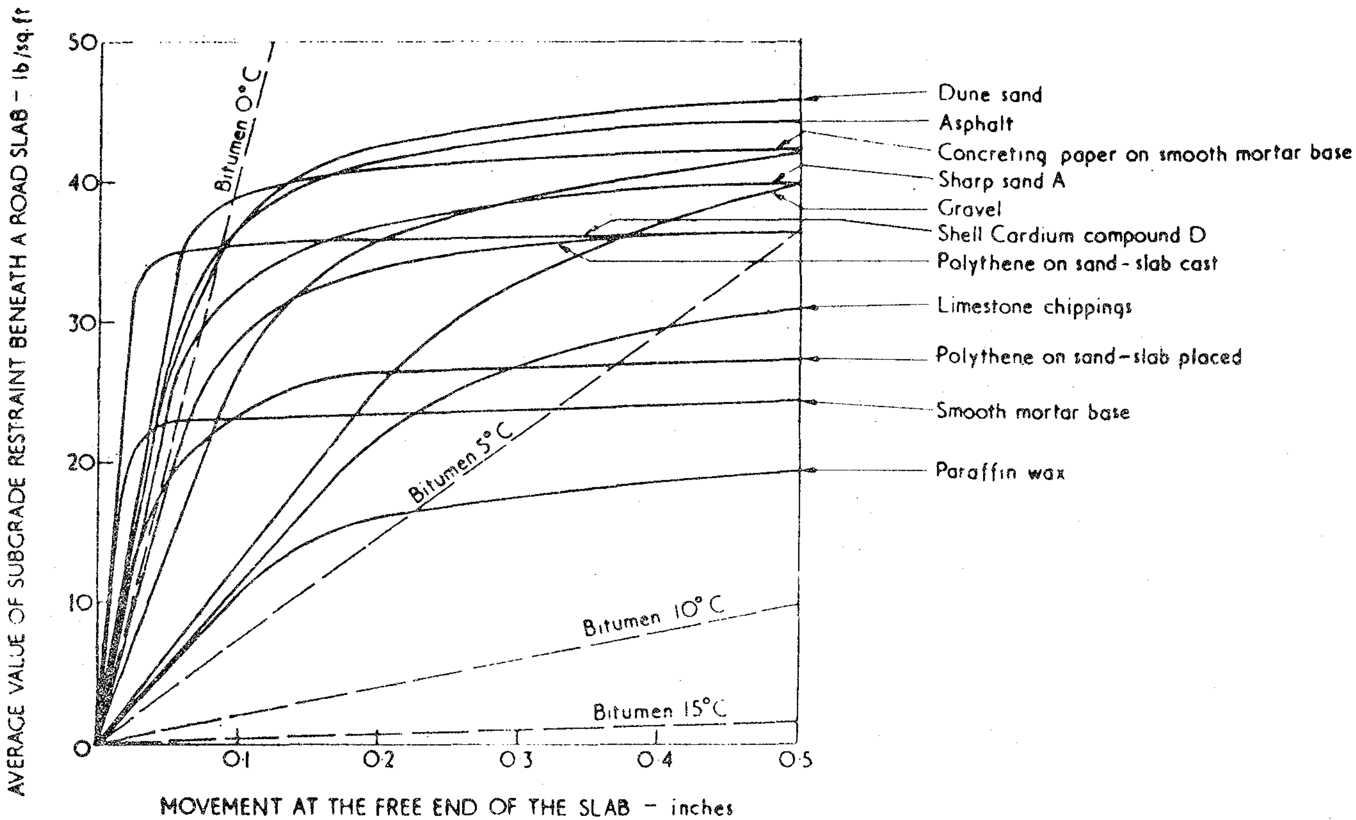
From Reference D14



1 in. = 2.54cm
1 psf = 47.88 Pa

From Reference D14

FIGURE D-9. Restraint Versus Temperature For Different Speeds, Thicknesses, and Bitumens.



1 in. = 2.54cm
1 psf = 47.88 Pa

From Reference D14

FIGURE D-10. Average Value of Subgrade Restraint Versus Movement at The Free End of A 6 In. Thick Slab.

TABLE D-7. Movement of Free Ends of Concrete Pavement Slabs

Slab length (ft)	Movement (inches) of each free end for a temperature change of				
	5deg C	10deg C	15deg C	20deg C	25deg C
20	0·006	0·012	0·018	0·024	0·030
40	0·012	0·024	0·036	0·048	0·060
60	0·018	0·036	0·054	0·072	0·090
100	0·030	0·060	0·090	0·120	0·150
250	0·075	0·15	0·225	0·30	0·375
500	0·15	0·30	0·45	0·60	0·75

1 ft = 0.3m
1 in. = 2.54cm

From Reference D14

Additionally, it was noted that increasing the vertical force on the slab did not affect restraint. Thus one can conclude that increasing the thickness of pavement will not increase the restraint induced by a bituminous layer. It was also noted that the softer bitumens tested tended to flow out from beneath the slab.

Stott also added that restraint will vary throughout the length of a pavement slab. Restraint induced stresses will be zero at the slab ends and a maximum at the center of the slab. Therefore an approximate value of average restraint can be defined over half of a slab. The average subgrade restraint is plotted versus movement of the slab free end in Figure D-10 for the various materials tested. Companion data is also shown in Table D-7 which contains the calculated theoretical maximum movements of the free ends of concrete pavement slabs due to temperature changes taken over five such temperature changes and six slab lengths. Utilizing both Figure D-10 and Table D-7, comparisons can be made of the most promising restraint reducing materials. For example from Table D-7, the free end of a 500 ft (152 m) slab is calculated to move approximately 0.45 (1.14 cm) inches at 15°C (59°F) temperature change. Applying this movement value to Figure D-10 reveals that 100 penetration bitumen also at 15°C (59°F) offers the lowest value of restraint. However, it should be noted that restraint induced by bitumen at 5°C (41°F) significantly exceeds several other friction reducing materials tested for slab free end movements exceeding 0.20 in. (0.51 cm).

Comparing data in Table D-7 with data presented by Friberg (D 56), a rough check can be made on potential slab movement. Using a 500 ft (152 m) slab length (about 1/3 the length of the Stilesville slab) and a temperature change of 10°C (18°F), Table D-7 indicates the free end movement of a slab to be 0.30 in. (0.76 cm). Figure D-6 indicates the actual free end movement for the Stilesville 1,310 ft (399 m) CRCP slab to be approximately 0.17 in. (0.43 cm) for a 19°F (11°C) change in temperature. These two values are significantly different. Part of the difference can be explained by the fact that CRCP pavements are designed to crack and hence some of this cracking would tend to adsorb part of the slab movement. This would decrease the total movement of a free slab end. Sublayer restraint would also reduce in-service slab movements.

Data presented by Teller and Sutherland (D 49) allow for an additional check on potential slab movements. A 40 ft (12.2 m) long, 20 ft (6.1 m) wide and 6 in. (15.2 cm) thick concrete slab was constructed on a compacted silty loam subgrade and contained one transverse and one longitudinal joint. The total expansion and contraction of this slab was measured over a 5 year period. Typical values of total length changes for this slab ranged from -0.035 in. (-0.089 cm) (contraction) at 30°F (-1.1°C) or movement at one end of the slab of approximately 0.050 in. (0.127 cm). A calculation of potential temperature movement for this slab disregarding subgrade and other restraints can be made by using the following formula:

$$\text{Total Slab Movement} = e\ L\ \Delta T$$

where

e = coefficient of thermal expansion (per °F)

L = length of slab (inches)

ΔT = temperature change (°F)

So, using $e = 0.000005/°\text{F}$ ($0.000009/°\text{C}$), $L = 40$ ft. (12.2 m), and $\Delta T = 70°\text{F}$ (38.9°C), we obtain

$$\text{Total Slab Movement} = (0.000005)\ (40\text{x}12)\ (70) = 0.168 \text{ in. (or 0.427 cm)}$$

For the movement at one slab end, divide the total movement in half which results in a movement of 0.08 in. (0.213 cm). Therefore, the theoretical movement is approximately 1.7 times greater than the actual movement for the given temperature range. However, it should be noted that the transverse joints may have decreased the actual slab movement.

Chastain and Burke (D 5), Velz and Carsberg (D 43) presented information on slab movements measured on in-service pavements. Chastian and Burke obtained data on Illinois test pavements, Velz and Carsberg on Minnesota test pavements. Tables D-8 and D-9 show summaries of these measurements. From Table D-8 it is observed that typical slab end movements for a 20 ft (6.1 m) long slab ranges from 0.01 in. (0.025 cm) to 0.03 in. (0.076 cm) for varying seasonal air temperature changes. For 100 ft long slabs, movements ranging from 0.04 in. (0.102 cm) to 0.09 in. (0.229 cm) occurred. Table D-9 shows similar data for the Minnesota slabs with lengths ranging from 15 to 60 ft (4.6 to 18.3 m) with various expansion joint spacings. Movements for these slabs are slightly higher than for the measurements made in Illinois.

TABLE D-8. Measured and Calculated Seasonal Slab Displacements on Experimental Inservice Pavements in Illinois (Field Data from Reference D 5)

Length (ft.)	Slab ID Number	From/To Dates	*Seasonal Air Temperature Change (°F)	Measured Slab End Displacements (in.)
20	1B	Summer 1952/ Winter 1952-53	46	0.020
20	1B	Summer 1955/ Winter 1956/57	65	0.028
20	1B	Summer 1957/ Winter 1957-58	79	0.035
20	2B	Summer 1952/ Winter 1952-53	27	0.010
20	2B	Summer 1955/ Winter 1956-57	64	0.030
20	2B	Summer 1957/ Winter 1957-58	71	0.025
100	3	Summer 1952/ Winter 1952-53	25	0.092
100	3	Summer 1955/ Winter 1956-57	58	0.080
100	3	Summer 1957/ Winter 1957-58	[illegible]	0.052
100	6	Summer 1952/ Winter 1952-53	21	0.060
100	6	Summer 1955/ Winter 1956-57	62	0.075
100	6	Summer 1957/ Winter 1957-58	39	0.045
100	7	Summer 1952/ Winter 1952-53	24	0.065
100	7	Summer 1955/ Winter 1956-57	60	0.072
100	7	Summer 1957 Winter 1957-58	39	0.042

*Temperature changes shown are diffferences between the average air temperatures at the time the joint openings were measured.

1 in. = 2.54 cm, 1 ft = 0.305 m, $1^{o}F = 1.8\ (^{o}C)+32^{o}$

TABLE D-9. Measured Seasonal Slab Displacements on Experimental Inservice Pavements in Minnesota Data Obtained From Reference D43

*Slab Length (ft)	Expansion Joint Interval (ft)	**Slab End Displacements Measured Between February and August 1948 (in.)
15	120	0.030
15	420	0.026
15	795	0.028
15	5260	0.028
20	120	0.052
20	400	0.032
20	800	0.046
20	5260	0.042
25	400	0.036
25	800	0.043
25	5260	0.056
30	120	0.075
30	420	0.056
30	810	0.055
30	5260	0.068
60	120	0.050

*Distance between contraction joints.

**Average concrete temperature for February was 40°F and for August was 92°F resulting in a seasonal change of 72°F.

1 ft = 0.3 m
1 in. = 2.54 cm
$1^{\circ}F = 1.8(^{\circ}C)+32^{\circ}$

The amount of movement to be expected in typical in-service slabs is quite important for two reasons. One, by knowing cyclic slab movements, a better evaluation of existing friction tests can be made to arrive at typical in-service coefficient of friction values; and two, it could be that in-service pavements may have experienced large enough temperature and moisture induced cyclic movements to be experiencing coefficient of friction values more in the sliding or kinetic range rather than in the static range. Thus, from the data just described (D 5, D 43, D 49), it is reasonable to assume that in-service pavements of lengths ranging from 20 ft (61. m) to over 100 ft (30.5 m) in length can be expected to move at about 0.05 in. or more. This will be somewhat dependent upon the subgrade restraint. Thus, it is not unreasonable to assume that many in-service pavements with the possible exception of CRCP have reached their maximum friction values and are now operating at somewhat lower friction levels than when first constructed. With this background information on slab movements, additional data from friction tests reported in the literature will be more informative.

In 1964 Timms (D 12) conducted a series of tests to determine the effect of various friction reducing mediums. The concrete slabs used in this testing program were 6 ft by 6 ft by 5 in. (1.8 by 1.8 m by 12.7 cm) thick. The slabs were cast in place on the following types of sublayers:

1. Subgrade soil consisting of micaceous clay loam referred to as "plastic soil".

2. Granular subbase consisting of material grading and plasticity requirements for Federal Highway projects.

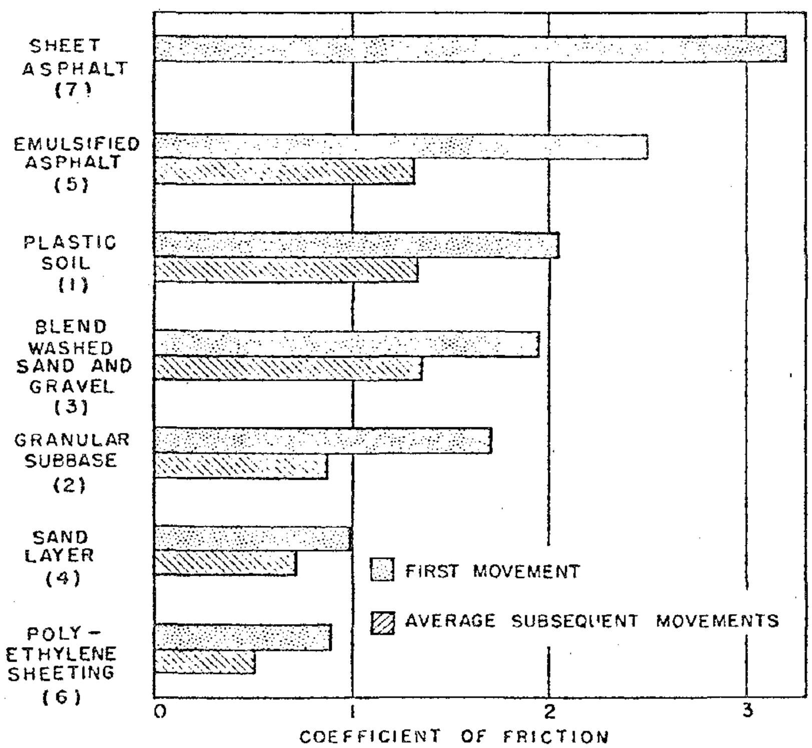
3. Granular subbase consisting of washed sand and gravel.

4. Granular subbase same as (2), with a 1 in. (2.54 cm) thick sand layer covered by one-ply building paper.

5. Granular subbase same as (2), covered with a layer of emulsified sand asphalt approximately 1 in. (2.54 cm) thick.

6. Granular subbase same as (2), with a thin leveling course of sheet asphalt covered by a double layer of polyethylene sheeting which contained a special friction reducing additive, and

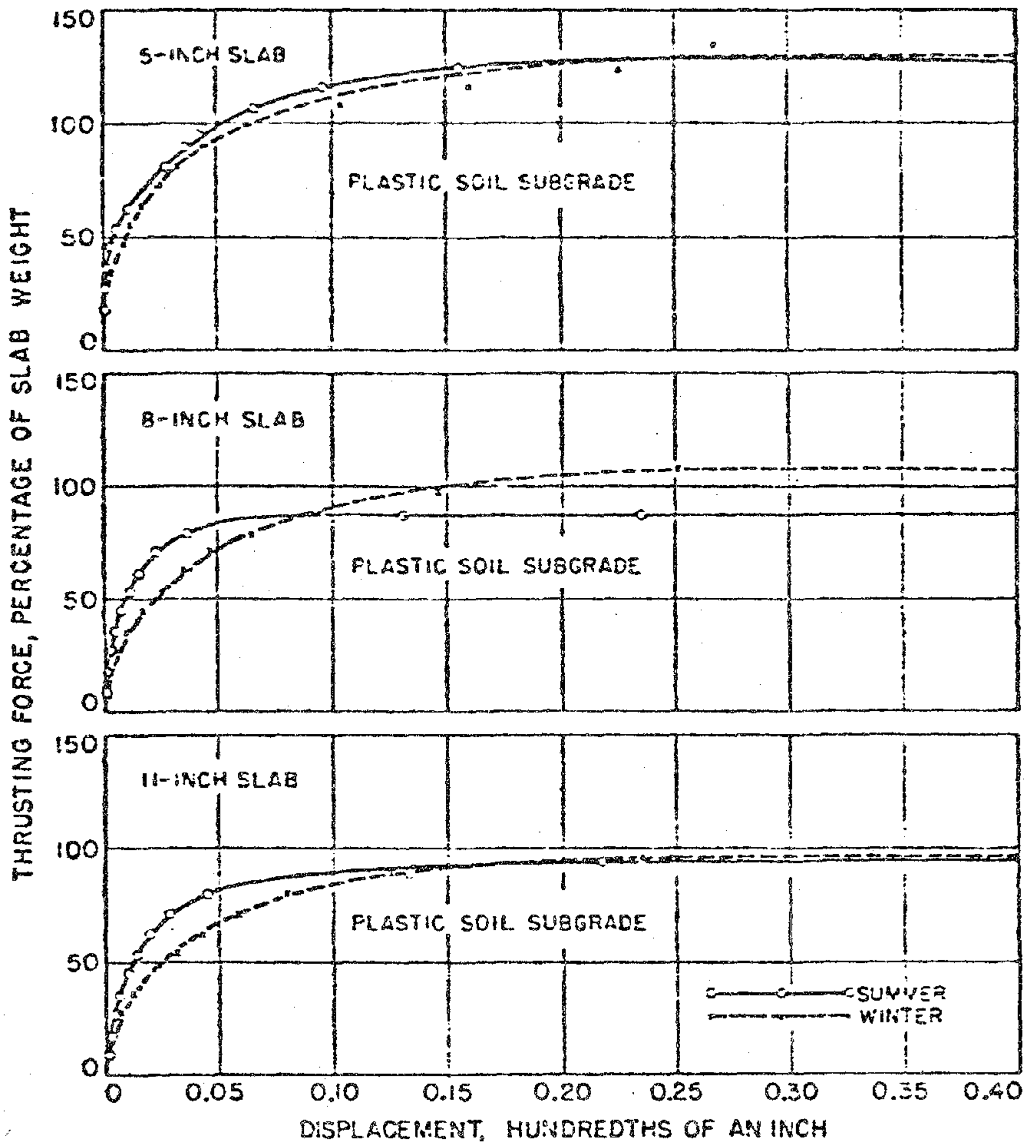
7. Granular subbase same as (2), covered by a layer of sheet asphalt approximately 1/2 in. (1.27 cm) thick.



From Reference D12

FIGURE D-11. Summary of Coefficients of Friction For 5 In. Slabs.

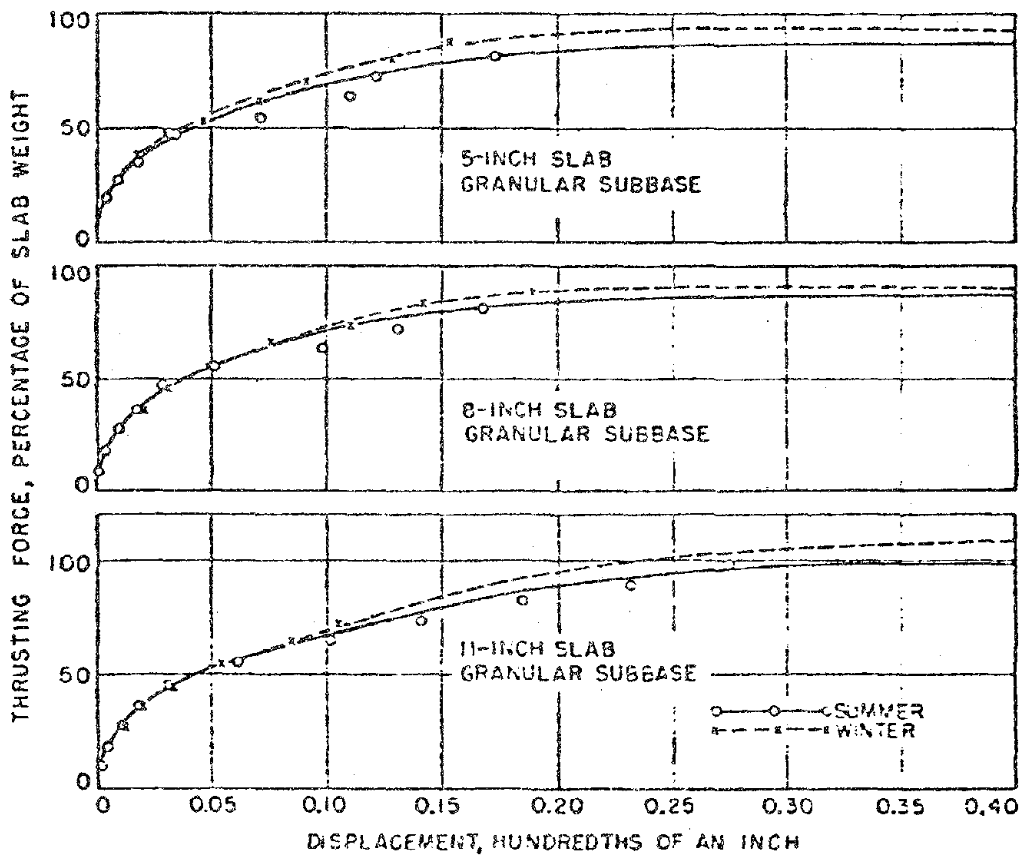
1 in = 2.54 cm



1 in. = 2.54cm

From Reference D12

FIGURE D-12. Force-Displacement Curves For Concrete Slabs On Plastic Soil.



1 in. = 2.54cm

From Reference D12

FIGURE D-13. Force-Displacement Curves For Concrete Slabs On Granular Subbase.

The force applied to the 6 ft (1.8 m) square slabs was done so with a hydraulic jack through a 3 ft - 4 in. long channel bearing plate. The horizontal displacement was measured with a micrometer dial.

The summary of the results from this testing program is shown in Figure D-11. The numbers shown in this figure under the various layer types correspond to the numbers used to describe these layers earlier. Layer 7 (sheet asphalt on granular subbase) is observed to have the highest initial (static) coefficient of friction and is slightly higher than 3.0. Layer 6 (double layer of polyethylene sheeting with additive on granular subbase) is seen to have both the lowest initial (static) and average subsequent (kinetic) coefficient of friction and are approximately 0.9 and 0.5, respectively. It is of special interst to note that both Layers 1 and 2, which are typical of standard sublayers under existing concrete pavements, had initial (static) coefficient of friction values of about 2.0 and 1.7, respectively. Average subsequent (kinetic) coefficient of friction values for these two layers were about 1.3 and 0.9.

Additionally, tests were conducted in this experiment to evaluate both the effect on increasing slab thickness and seasonal effects. For Layer 1 (plastic subgrade), as the equivalent slab thickness increased from 5 in. to 11 in. (12.7 to 27.9 cm), the coefficient of friction tended to decrease. In the free sliding (kinetic) range, the force required to move the slabs in winter were generally higher than for the summer. This can be observed in Figure D-12. Moisture contents in the top 1/2 in. (1.27 cm) of the soil for summer and winter were 22 and 25 percent, respectively. For Layer 2 (granular subbase), the coefficient of friction tends to remain the same or increase slightly for increasing pavement thickness and there is little difference between winter and summer values. This can be seen in Figure D-13.

Venkatasubramanian (D 35) conducted friction studies with small concrete test slabs placed on smooth to very rough base surfaces. He investigated the amount of restraint and coefficient of friction values produced by moving these slabs across various surfaces. What is unique about this work is the types of bases used. One series of tests were conducted on a macadam base which can have an extremely rough texture.

The purpose of Venkatasubramanian's work was to investigate friction in "bonded" concrete pavement slabs. Venkatasubramanian stated that

European highway engineers were not certain that utilizing a minimum coefficient of friction is an advantage in Highway construction and that a high coefficient of friction may help to distribute stresses in pavements induced by expansion and contraction.

The test slabs used to investigate friction in Venkatasubramanian's experiment were either 4 ft by 2 ft (1.2 m by 0.6 m) or 3 ft by 1 ft (0.9 m by 0.3 m) in size and 4 or 6 in. (10.2 or 15.2 cm) thick. These test slabs were tested within a tank which was 4.5 ft by 2.5 ft by 1.0 ft (1.37 m by 0.76 m by 0.3 m) deep. A hydraulic load cell was used to measure the force applied to the test slab. Horizontal movement was measured with four dial gages set at each corner of the slab. Figure D-14 shows a generalized plan view of the testing scheme.

Several base course frictional conditions were examined in this experiment. Of the ten slabs tested, only five of these were cast directly on the prepared bases inside the testing tank. It is these five slabs in which we are interested and for convenience are labeled 1 through 5 and are described as follows:

Slab 1: 3 ft by 1 ft by 6 in. (0.9 m by 0.3 m by 15.2 cm), on compacted damp sand base

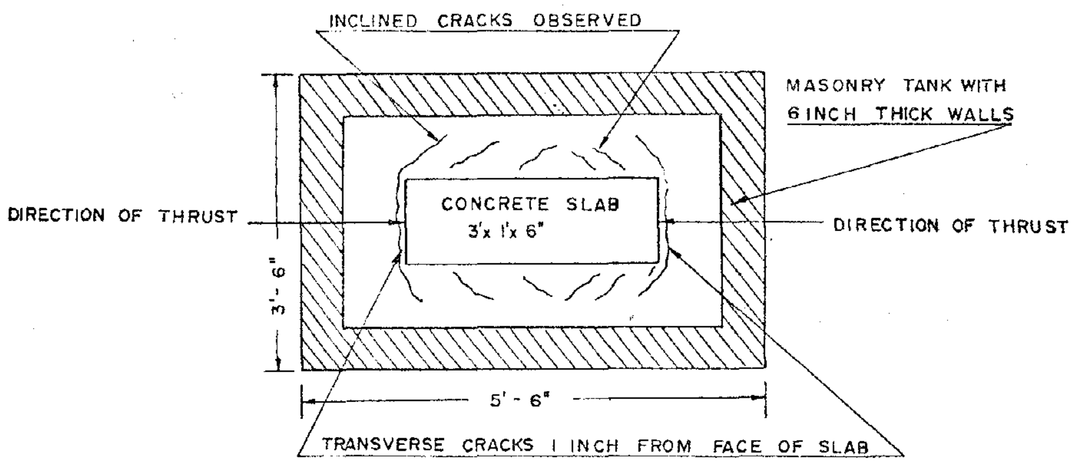
Slab 2: 4 ft by 2 ft by 4 in. (1.2 m by 0.6 m by 10.2 cm), on saturated water-bound macadam base

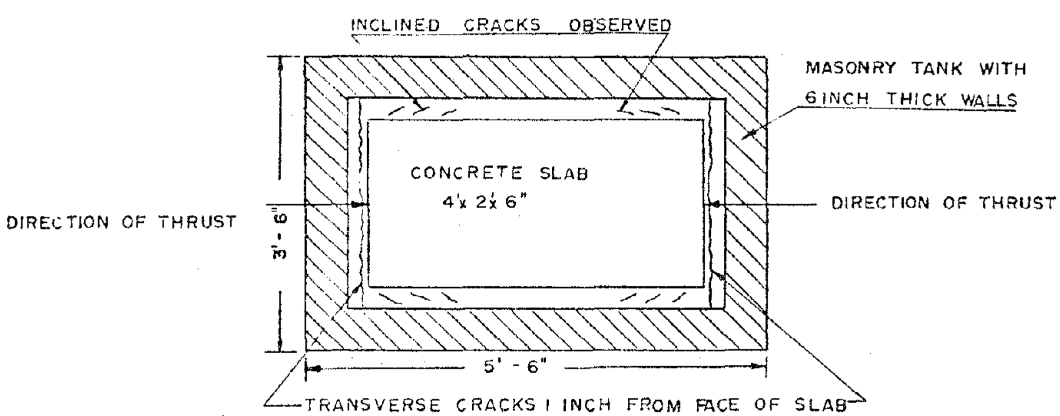
Slab 3: 3 ft by 1 ft by 6 in. (0.9 m by 0.3 m by 15.2 cm), on saturated water-bound macadam base

Slab 4: 4 ft by 2 ft by 7 in. (1.2 m by 0.6 m by 15.2 cm), on dry water-bound macadam base

Slab 5: 3 ft by 1 ft by 7 in. (0.9 m by 0.3 m by 15.2 cm), on dry water-bound macadam base

For Slab 1, the sand base was comprised of a locally available river sand. For Slabs 2 through 5, the water-bound macadam base was composed of three layers. The first of these was a 2 in. (5.1 cm) thick layer of compacted sand on which a 6 in. (15.2 cm) thick handpacked laterite stone subbase was placed. On top of this was placed a 4 in. (10.2 cm) thick water-bound macadam base which utilized 1 1/2 in. (3.8 cm) broken stones. Disintegrated gravel was placed around the broken stones as a filler material. The surface of the macadam was finished by wire brushing which

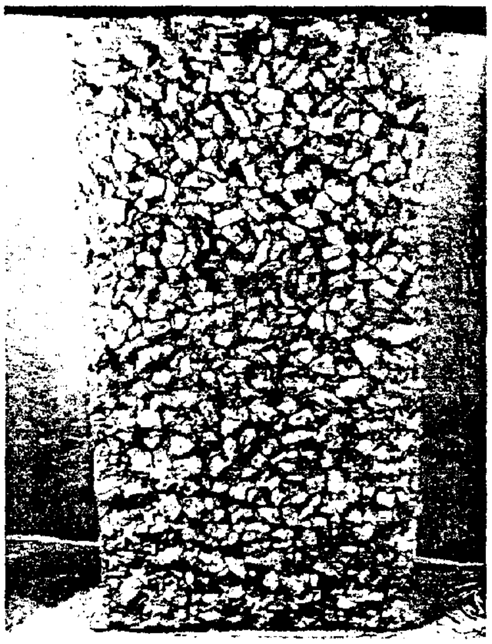




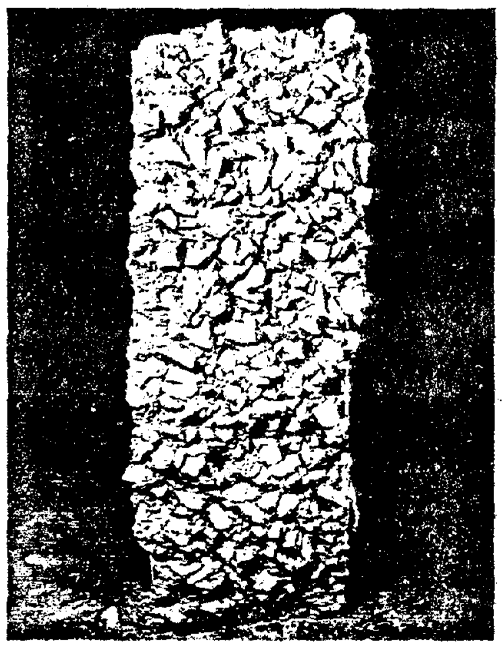
From Reference D35

1 ft = 0.3m
1 in. = 2.54cm

FIGURE D-14. Generalized Plan View of Testing Apparatus Used By Venkatasubramanian



Underside of 4-ft by 2-ft by 4-in. slab bonded to saturated water-bound macadam base after test.



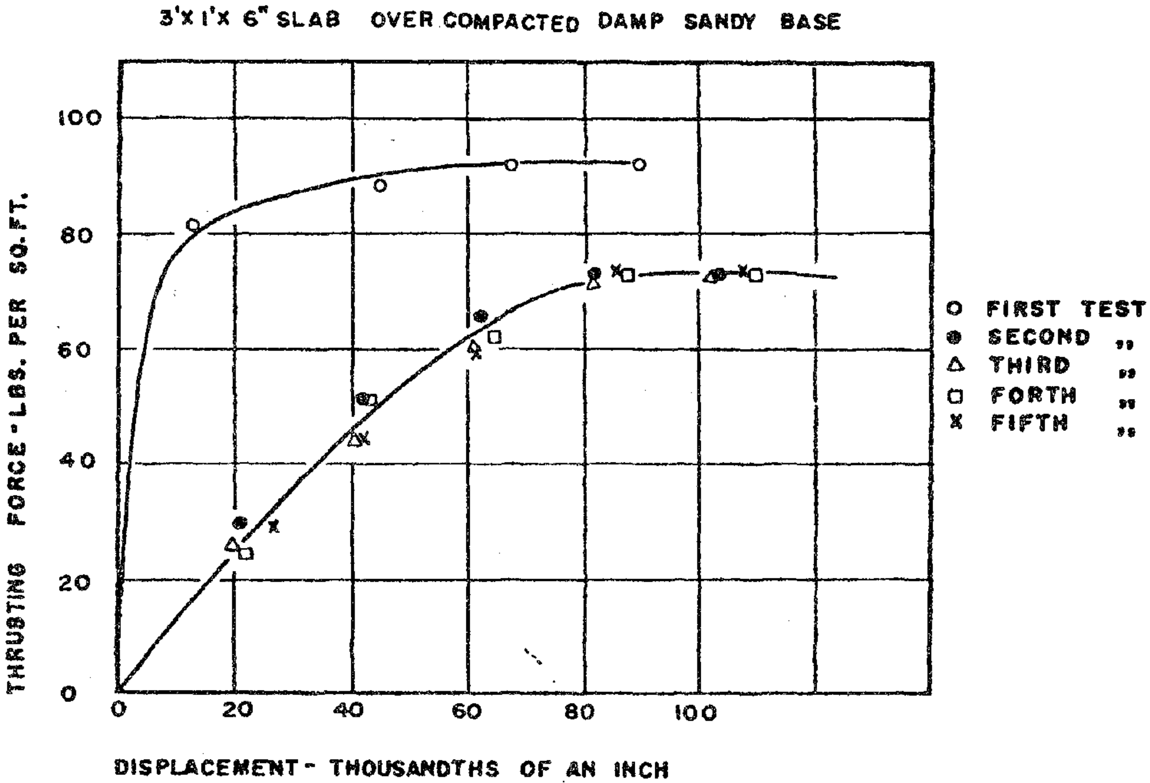
Underside of 3-ft by 1-ft by 6-in. slab bonded to saturated water-bound macadam base after test.

Underside of 4-ft by 2-ft by 6-in. slab bonded to water-bound macadam base (dry) after test.

Underside of 3-ft by 1-ft by 6-in. slab bonded to water-bound macadam base (dry) after test.

FIGURE D-15. Undersides of Concrete Test Slabs Placed on Macadam Base.
1 inch = 25.4 mm
1 ft = 0.305 m

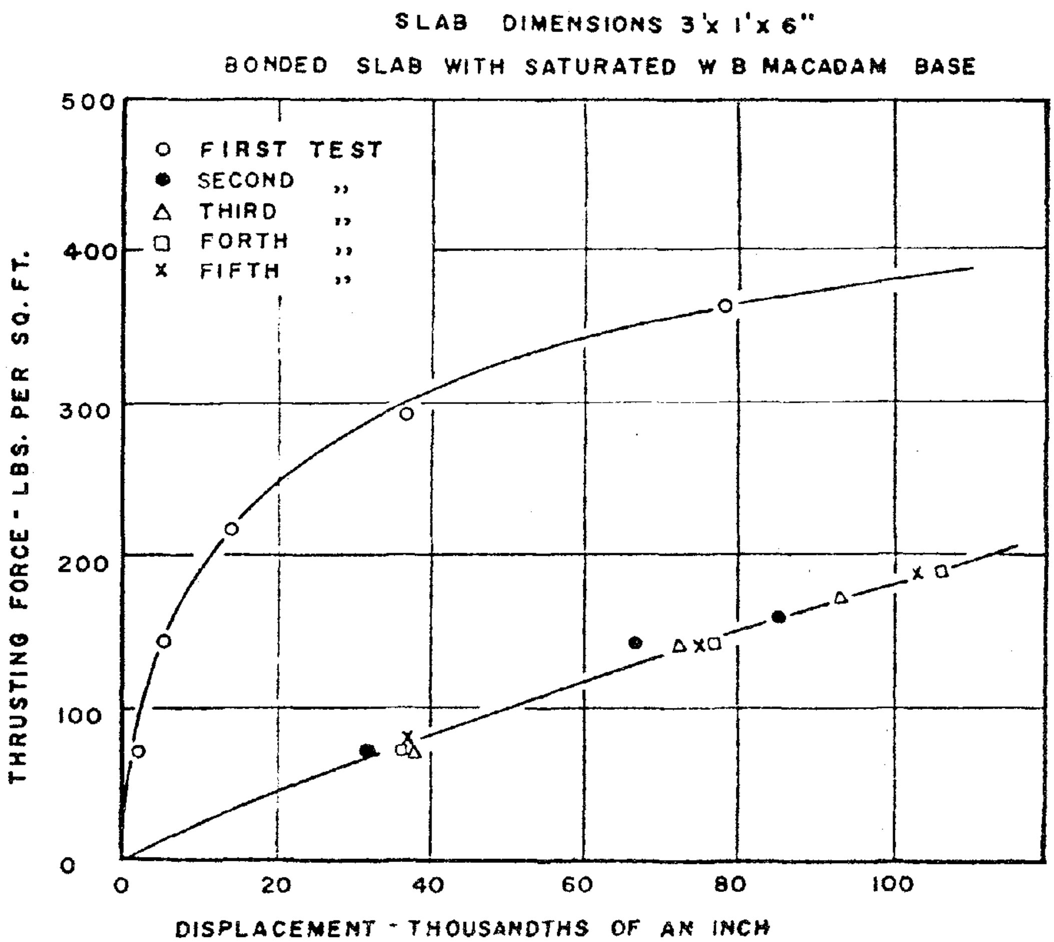
From Reference D35



1 ft = 0.3m
1 m = 2.54cm
1 psf = 47.88 Pa

From Reference D35

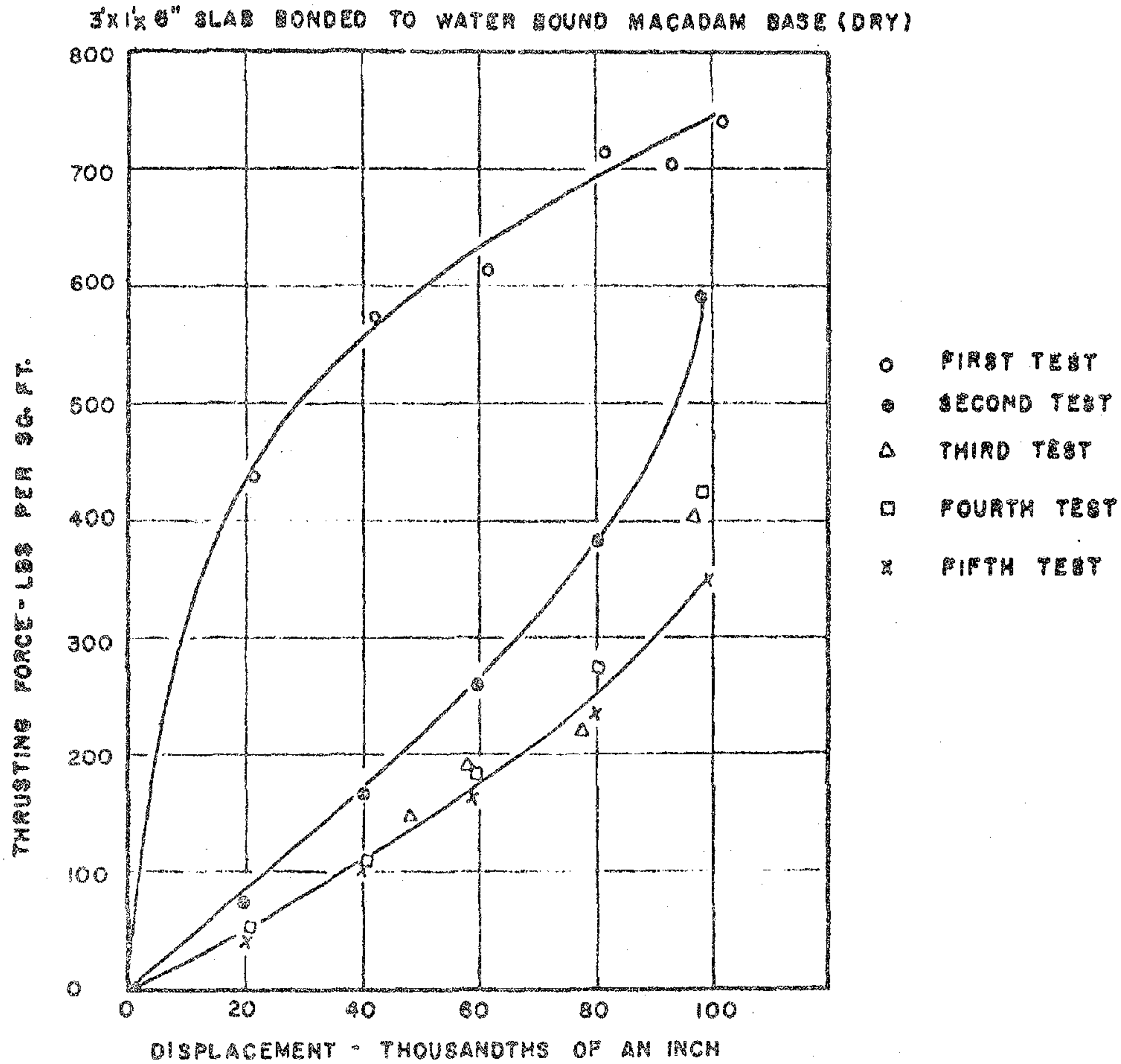
FIGURE D-16. Thrusting Force (Restraint) Versus Displacement For A 3' x1' x6" (0.9m x 0.3m x 15.2cm) Slab Over Damp Sand Base.



1 ft = 0.3m
1 in = 2.54cm
1 psf = 47.88 Pa

From Reference D35

FIGURE D-17. Thrusting Force (Restraint) Versus Displacement For A 3' x1' x6" (0.9m x 0.3m x 15.2cm) Slab Over Saturated Macadam Base.



1 ft = 0.3m
1 in = 2.54cm
1 psf = 47.88 Pa

From Reference D35

FIGURE D-18. Thrusting Force (Restraint) Versus Displacement For A 3' x1' x6" (0.9m x 0.3m x 15.2cm) Slab Over Dry Macadam Base.

removed loose material prior to slab placement. The construction of the macadam was accomplished in such a manner to create a large amount of friction between the slab and the base. Figure D-15 shows the underside of both the 4 ft 2 ft (1.2 m by 0.6 m) and 3 ft by 1 ft (0.9 m by 0.3 m) slabs which were placed on the macadam base. Notice in this figure the large amount of angular macadam stone clinging to the underside of the slab.

Figure D-16, D-17 and D-18 show partial results from the testing program. In Figure D-16, a plot of "thrusting force" (or restraint) versus displacement is plotted for Slab 1 on a damp, sandy base. Figures D-17 and D-18 are the same type of plots for Slabs 3 and 5, respectively. All three figures exhibit the same trend which is that more force is required to overcome friction during the initial (static) forcing cycle. For Slabs 1 and 3, the force required to cause movement is approximately constant for the four subsequent forcing cycles. Recall that Slabs 1 and 3 were placed on damp sand and saturated macadam base, respectively. For Slab 5, placed on a dry macadam base, the force versus displacement curves become essentially constant after the third forcing cycle.

Values of subgrade restraint (Venkatasubramanian calls restraint "thrusing force") are summarized in Table D-10 and D-11. These values are significant in that coefficient of friction values may be obtained from this data.

In Table D-10, restraint values were taken from Venkatasubramanian's plots (D 35) at a displacement of 0.05 in. (0.127 cm). The restraint values for the slabs placed on the dry water-bound macadam are the highest. A displacement value of 0.05 in. (0.127 cm) was chosen to approximately match that displacement value used in Table 9. And because of information contained in Reference D49 and Discussed earlier. Table D-11 is a summary of similar data obtained at the Road Research Laboratory (D 44). The restraint values shown in this table for a rough subgrade taken at a 0.05 in. (0.127 cm) displacement are approximately equal to those obtained by Venkatasubramanian for a saturated macadam base.

The restraint values shown in Table D-10 for a dry macadam base should represent the extreme case of frictional restraint to be expected in existing in-service pavements. Probably, the "rough subgrade" value of 300 psf (14.36 KPa) in Table D-11 and the static restraint values for Slabs 2 and 3 in Table D-10 are more realistic upper limits. This should

TABLE D-10. Values of Subgrade Restraint For Selected Slabs Data Obtained From Reference D35

Slab Number, Size, and Base	Restraint (Thrusting Force) For 1st Cycle @ 0.05 in. Movement (psf) (Static)	Restraint (Thrusting Force) for Subsequence Cycles @ 0.05 in. Movement (psf) (Kinetic)
Slab 1: 3'x1'x6" on compacted damp sand base	90	75
Slab 2: 4'x2'x4" on saturated water-bound macadam base	330	90
Slab 3: 3'x1'x6" on saturated water-bound macadam base	325	100
Slab 4: 4'x2'x6" on dry water-bound macadam base	560	∿ 180
Slab 5: 3'x1'x6" on dry water-bound macadam base	600	∿ 180

1 ft = 0.3m

1 in. = 2.54cm

1 psf = 47.88 Pa

TABLE D-11. Estimated Values of Subgrade Restraint and Displacements For Various Subgrades

Type of Subgrade	Maximum Restraint of Subgrade (psf)	Displacement Needed to Produce Slipping (in.)
Very smooth subgrade of completed sand or gravel. Smooth subgrade covered with waterproof paper.	100	0.01
Moderately smooth subgrade of compacted sand, gravel or clinker. Rough subgrade covered with waterproof paper.	200	0.03
Rough subgrade	300	0.05

1 in. = 2.54cm

1 psf = 47.88 Pa

From Reference D44

be a reasonable assumption since normal subbase preparation in the United States does not utilize rough surface techniques such as that obtained with the macadam type of sublayers.

Subbase preparation is discussed in a 1969 PCA summary presented by the ACI (D 45) which revealed the results of a nationwide concrete pavement practice survey. These results indicated that 68 percent of the states have an optional requirement which allows the use of automatic, self-propelled fine grading machines for subbase preparation. The survey also showed that at least 36 percent of the states do not allow the operation of trucks on the prepared subgrade thus precluding rutting and nonuniform compaction of the subgrade surface. Thus, relatively smooth subbases and subgrades can be expected in much of the construction in the United States.

The restraint values previously discussed are informative but a presentation of coefficient of friction values would round out this examination of Venkatasubramanian's experiment. Knowing the size and depth of the test slab and the restraint value, the coefficient of friction can be calculated by using the following formula:

$$\mu = f/N$$

where

μ = coefficient of friction

f = subbase/subgrade restraint

N = normal force (weight of slab)

For example, in Table D-10, a maximum restraint value of 600 psf (28.73 KPa) is shown for a 0.05 in. (0.127 cm) displacement for Slab 5 on a dry macadam base. The coefficient of friction (static) can be calculated as follows:

$$\mu = f/N = \frac{(600 \text{ psf})\ (3 \text{ ft})\ (1 \text{ ft})}{(150 \text{ pcf})\ (3 \text{ ft})\ (1 \text{ ft})\ (0.5 \text{ ft})} = 8.0$$

Therefore the coefficient of friction (static) at the specific displacement is equal to 8.0.

Table D-12 gives the coefficient of friction values (static and kinetic) as obtained by Venkatasubramanian for the five slabs previously discussed. For Slabs 2 and 3, which are considered as upper limit values for typical in-service pavements, the averaged coefficient of friction values at 0.1 in. (0.254 cm) for these two slabs are 6.5 (static) and

TABLE D-12. Coefficients of Friction (Static and Kinetic) For Five Test Slabs On Two Types of Bases at Various Displacements Data Obtained From Reference D35

	Coefficient of Friction					
	0.02 in.		0.06 in.		0.10 in.	
Slab Number, Size, and Base	Static	Kinetic	Static	Kinetic	Static	Kinetic
Slab 1: 3'x1'x6" on compacted damp sand base	1.17	0.34	1.26	0.86	1.28	1.00
Slab 2: 4'x2'x4" on saturated water-bound macadam base	4.70	0.52	7.30	2.10	7.81	3.33
Slab 3: 3'x1'x6" on saturated water-bound macadam base	3.47	0.69	4.72	1.67	5.27	2.50
Slab 4: 4'x2'x6" on dry water-bound macadam base	3.48	0.55	8.68	2.44	10.44	4.58
Slab 5: 3'x1'x6" on dry water-bound macadam base	6.11	0.70	8.70	2.43	10.14	4.86

1 ft = 0.3m

1 in. = 2.54cm

2.9 (kinetic). Although, displacements of 0.1 in. (0.254 cm) are probably on the lower end of the range of those expected for existing rigid pavements after receiving prestressing as a rehabilitation technique.

For example, to illustrate the range of displacements expected, assume a 200 ft (61 m) long segment of rigid pavement is post-tensioned in a rehabilitation effort. Also assume two seasonal temperature changes of 50°F (27.8°C) and 100°F (55.6°C). Disregarding subgrade friction, moisture changes and using a concrete coefficient of thermal expansion of 0.000005/°F (0.000009/°C), this results in movements at the end of the slab of approximately 0.3 in. and 0.6 in. (0.8 cm and 1.5 cm), respectively due to temperature changes alone.

Other references (D 12, D 14) have shown that the coefficient of friction stabilizes to approximately a constant value usually between displacements of 0.05 and 0.2 in. (0.127 and 0.508 cm). Therefore, the coefficient of friction values presented by Venkatasubramanian (D 35) and shown in Table D-12 can probably be expected to increase. Thus, in the range of displacements expected for prestressed rehabilitated pavements, the upper limit of expected coefficient of friction (static and kinetic) can be expected to be in the range of about 7 to 10 for the static case and 3 to 5 for the kinetic case.

The highest coefficient of friction values found in the literature were obtained in a testing program conducted by the PCA (D 46) in 1971. In this testing program a number of different friction reducing methods were utilized on cement-treated subbases. Summaries of the complete testing program may be found in Reference Nos. D46 and D57. Of more direct interest was the tests performed on the cement-treated subbase (CTB) without the use of any friction reducing layers.

These test were conducted on 4 ft by 4 ft by 7 in. thick (1.2 m by 1.2 m by 15.2 cm) concrete slabs which were cast directly on the CTB. Crushed limestone was used as aggregate for the CTB in three different gradations which produced smooth, medium, and rough surface textures. The gradations used to produce these different textures are shown in Table 13. One set of tests were run on each of the three CTB gradations and all were covered with 0.2 gallon per square yard of bituminous curing compound. The bitumen used was Shell SS-1 asphalt emulsion. One additional test was run on the medium aggregate gradation utilizing CTB without the curing compound. The average compressive strength of the CTB as determined by cores was 3050 psi (21,030 KPa) and was attained by adding 6 percent by weight of cement to the crushed limestone.

The slabs were loaded through a load cell and movement was measured with an electronic transducer. The loading rate was approximately 200 pounds (889.6 N) per minute. The results of the load tests showed that extremely high coefficient of friction values resulted from this testing program. For both the bituminous coated and uncoated CTB, the static coefficient of friction could not be overcome with the testing equipment available and this varied between test slabs. The resulting values are shown in Table D-14, Item No. 13.

It appears that the CTB became bonded to the concrete slabs in both of the cases examined. The effect the bituminous curing compound may have had in causing this bonding for the high coefficient of friction values reported is not known. In any case, the coefficient of friction values are so high that existing concrete pavements placed on CTB should be eliminated from consideration for receiving prestressed rehabilitation at the present time. A summarized PCA survey (D 45) of current nationwide paving practice published in 1969 shows that at least 10 states allow only the use of cement treated subbases in concrete pavement construction. Thus, a number of existing pavements may be unsuitable for receiving prestressing as a rehabilitation method.

With the information summarized in Table D-14, an indication of the values of coefficient of friction that will be encountered on existing concrete pavements can be roughly estimated. An average of all static coefficient of friction values in the table excluding only the values obtained by the PCA for cement-treated bases results in a value of 2.9. If the values obtained by Venkatasubramanian are also excluded because of the extreme roughness of the bases tested, the average static value lowers to about 2.2. Using the same averaging procedure for kinetic coefficient of friction, these values are 1.3 and 1.1, respectively. It is expected that many existing in-service concrete pavements will have coefficient of friction values somewhere between 2.9 to 1.1. This is due to the fact that many jointed pavements experience seasonal temperature and moisture induced movements which will tend to lower friction values from the static

TABLE D-13. Crushed Limestone Gradations Used for Cement-Treated Subbases

	Percent Passing for Each Surface Texture of CTS		
Sieve Size	Smooth	Medium	Rough
1 1/2 in.			100
3/4 in.			67
1/2 in.		100	
3/8 in.		66	
No. 4	100	0	
No. 10	48		
No. 18	38		
No. 35	20		
No. 60	13		
No. 140	9		
No. 200	7		
0.005	1		

1 in. = 2.54cm

From Reference D46

TABLE D-14. Summary of Coefficient of Friction Values for Pavements/Test Slabs Without Use of Friction Reducing Methods

Author	(Ref. No.)	Date	Concrete Pavement/Slab Placed on Following	*Coefficient of Friction
1. Goldbeck	(D 47)	1917	Clay subgrade	2.1 (Static)
			Loam subgrade	2.1 (Static)
			Sand layer	1.4 (Static)
			3/4 in. gravel subbase	1.3 (Static)
			3/4 in. broken stone	1.1 (Static)
			3 in. broken stone	2.2 (Static)
2. Teller and Bosley	(D 50)	1930	Dry subgrade	2.0 (Static)
			Damp subgrade	2.5 (Static)
			Wet subgrade	1.7 (Static)
			Frozen subgrade	>8.5 (Static)
3. Sparkes	(D 55)	1939	Clinker base	3.1-3.7 (Static)
4. Friberg	(D 56)	1954	Subgrade paper on sand-loam subgrade	~1.5 (Unk)
			Clayey silt subgrade	Probable range between 0.33 and 1.67
5. Friberg	(D 2)	1955	Rough subgrade	1.5 (Kinetic)
			Sandy, even subgrade	1.0-1.5 (Kinetic)
6. Stott	(D 3)	1955	Sand layer	1.2-2.0 (Unk)
7. Cholnoky	(D 6)	1956	Base course	5.2 (Static) 1.1 (Kinetic)
8. Watson	(D 52)	1960	Soil cement base with asphalt emulsion curing membrane	5.1 (Static) 0.8 (Kinetic)
9. Stott	(D 14)	1961	Sharp sand	1.0 (Static) 0.7 (Kinetic)
10. HRB S.R.78	(D 54)	1963	Granular subbase	1.5 (Unk)
11. Timms	(D 12)	1964	Micaceous clay loam subgrade	2.0 (Static) 1.3 (Kinetic)
			Granular subbase	1.7 (Static) 0.9 (Kinetic)

*Parenthesis indicate type of friction, either static or kinetic.

TABLE D-14. Summary of Coefficient of Friction Values for Pavements/Test Slabs Without Use of Friction Reducing Methods - (CONTINUED)

	Author	(Ref. No.)	Date	Concrete Pavement/Slab Placed on Following	*Coefficient of Friction
12.	Venkatasubramanian (all values taken at 0.1 in. displacement)	(D 35)	1966	Compacted damp sand base	1.28 (Static) 1.00 (Kinetic)
				Saturated water-bound macadam base	7.81 (Static) 3.33 (Kinetic)
				Dry water-bound macadam base	10.14 (Static) 4.86 (Kinetic)
13.	PCA	(D 57)	1971	Cement-treated base coated with bituminous curing compound: Smooth Surface Medium Surface Rough Surface	 >13.5 (Static) >44.0 (Static) >51.0 (Static)
				Cement-treated base uncoated: Medium Surface	 >8.0 (Static)

*Parenthesis indicate type of friction, either static or kinetic.

TABLE D-15. Coefficient of Friction for Selected Materials

Material	Coefficient of Friction (Includes Both Static and Kinetic)
Sand	0.49 to 1.03
Polyethylene Sheets	0.3 to 0.8
Polyethylene Sheet Over Sand Layer	0.55 to 0.8
Bitumen	Depends on Condition
Oil	0.33 to 0.49

From Reference D57

to that approaching the kinetic situation. In any case, for planning purposes, a maximum coefficient of friction value of about 3.0 and minimum value of about 0.1 should be anticipated. Although, it should also be recognized that for some existing concrete pavements the coefficient of friction will approach those as determined by Venkatasubramanian.

For typical coefficient of friction values for prestressed pavements incorporating some type of friction reducing layer, refer to Table D-1 or the recently completed report by the PCA for the FHWA (D 57). A good summary of typical ranges presented in the PCA report is shown as Table D-15.

It has been noted from the literature reviewed that thin sand layers have been frequently used in new prestressed pavement projects as a friction reducing layer. Timms (D 12) points out that these sand layers could contribute to pumping under the relatively thin prestressed pavements. Both edge pumping and pumping through any open transverse cracks could occur.

Before concluding this section on friction, it is appropriate to briefly examine new types of potential friction reducing layers. These materials could possibly be used in either new prestressed pavement construction or as friction reducers in rehabilitating existing concrete pavements with prestressing techniques.

For example, Bowden and Tabor (D-27) examined several types of friction reducing materials which include carbon, graphite, Teflon, glass and rubber. It was noted that carbon, and especially graphite, has a low coefficient of friction. For hard, non-graphitic carbon surfaces, it was found that the coefficient of steel on carbon and of carbon on carbon is approximately 0.16. With graphite the coefficient was found to be about 0.1.

Teflon was also found to be an excellent friction reducing material. The friction of Teflon on Teflon is comparable to ice on ice. This coefficient of friction is about 0.04 which is quite low. Additionally, Teflon maintains this low coefficient of friction over a range of temperatures from at least 20°C (68°F) to 200°C (392°F).

Bowden and Tabor also observed that glass on glass has a coefficient of friction of about 0.9 and that the coefficient for rubber on steel can be about 4.0. Others have found that for a wide range of solids sliding on rubber the coefficient of friction is about 1.0. The observed coefficients of friction for these materials do not encourage their use.

There are many possible materials that could be used for reducing friction in prestressing applications. Further examination graphite should be considered as well as water injection or possibly a combination of both.

Distress of Prestressed Pavements

In attempting to apply prestressing techniques to existing concrete pavements, the problem of prestressed related distress should be examined. Conceivably, prestressing may in some cases cause more damage to a pavement as opposed to doing nothing at all.

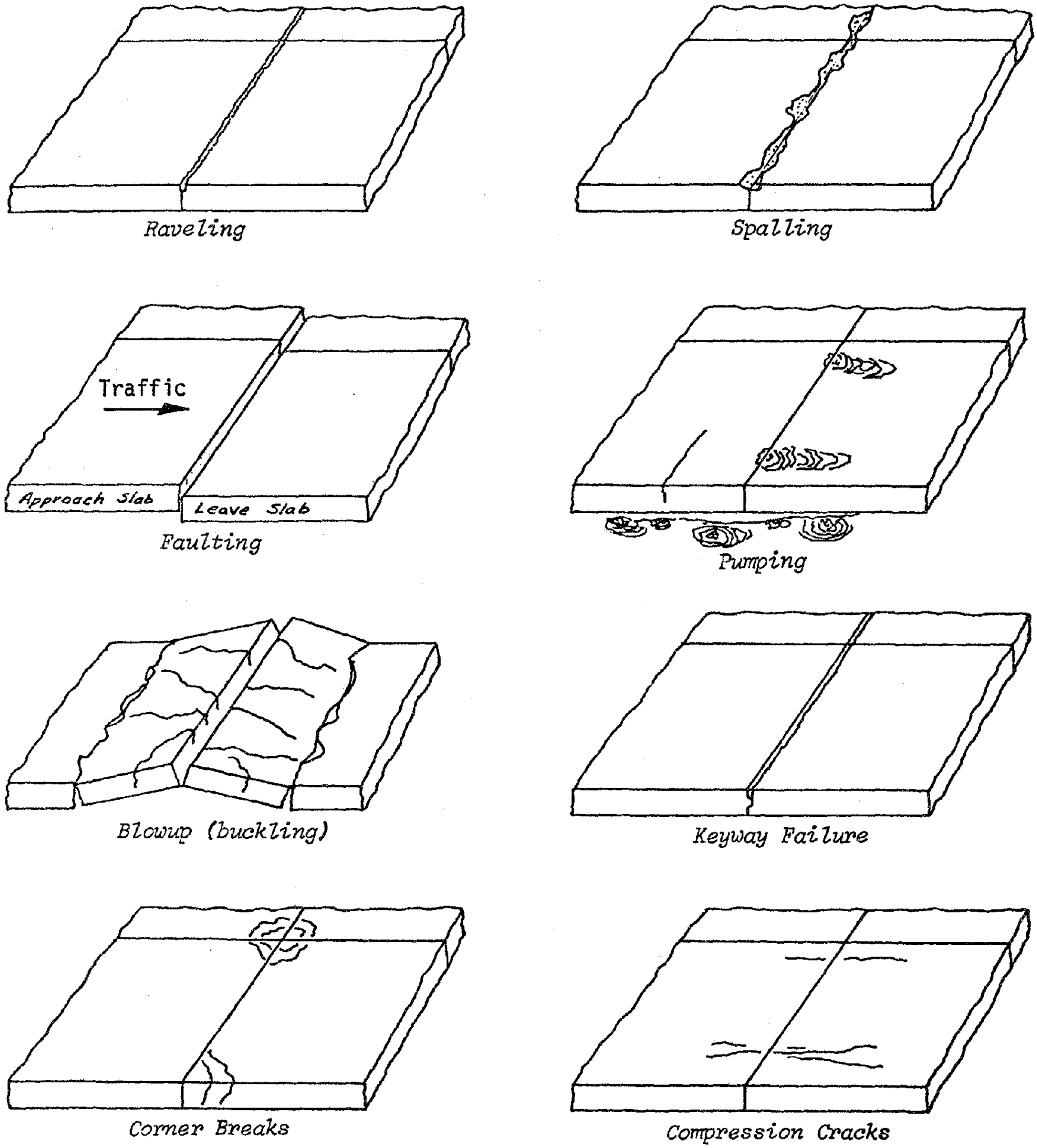
To get a "feel" for this kind of distress, a review of available distress information on existing prestressed pavement projects should be informative. Table D-19 partially attempts to review distress but this section will go into more detail. Individual projects will be reviewed in a chronological sequence. For each project, four possible types of distress can be summarized. These are as follows:

1. Cracking
2. Blowups
3. Faulting
4. Other types of joint related problems

Although mainly used to illustrate distress at joints, Figure D-19 is generally descriptive of the types of distress which affect rigid pavements.

For each project reviewed, the location, date of construction and distress types will be underlined. For applicable references for each project listed refer to Table D-1. The following projects are reviewed:

1. Heidenhein, Germany. This road was constructed in 1953. The pavement slabs were 365 ft (111 m) long by 28 ft (8.5 m) wide with a 6.0 in. (15.2 cm) depth. Two slabs with these dimensions were constructed and both were post-tensioned with cables in the longitudinal and transverse directions. One slab had prestressing amounts of 300 psi (2068 KPa) longitudinally and 240 psi (1655 KPa) transversally. The other companion slab had 240 psi (1655 KPa) longitudinally and 43 psi (296 KPa)



From Reference D28

FIGURE D-19. Typical Rigid Pavement Distress transversally. Transverse cracking was reported at intervals for both of these slabs in 1955. No other distress information was provided.

2. Biggs Air Force Base, El Paso, Texas. A prestressed taxiway was constructed at this air base in 1959. The taxiway has a total length of 1500 ft (427 m) which was segmented into 3/500 ft (3/152 m) slabs. The width was 75 ft (23 m) segmented into 3/25 ft (3/7.6 m) paving lanes. All slabs had a total depth of 9.0 in. (22.9 cm). In the longitudinal direction the slabs were post-tensioned with tendons in flexible metal conduit and prestressed to 350 psi (2413 KPa) Transversally, the slabs were post-tensioned with tendons in rigid metal conduit and prestressed to 150 psi (1034 KPa). Transverse cracks occurred during construction at the ends of the taxiway. These were identified as shrinkage cracks and were due to rapid temperature changes during the early curing period. Serious joint performance problems have been encountered on this project. A detailed account of these joint problems is contained in the recent PCA report (D 57). This material is repeated here, in part, for convenience. "Initial construction required two 15-ft. 4-in. intermediate gap slabs between the 500-ft. prestressed slabs and two 9-ft. 8-in. joint slab at the ends of the pavement. One, 1/4 x 1-1/2-in. contraction joint was provided in the center two joint slabs. Polyurethane foam filled, 1-1/2-in. wide expansion joints were constructed in the joint slabs at the ends of the section. The contraction joints at the intermediate gap slabs opened about 1 in. These joints were then filled with polyurethane foam in combination with a polysulfide rubber joint sealer. Because of continuing joint performance problems, the sealer was removed and the joint was filled with concrete.

Forces due to restrainted temperature expansion caused spalling and crushing of the intermediate and end joint slabs. Removal and replacement of the 8-ft. wide and 5-ft. wide sections of these slabs was initiated in 1960. Two expansion joints were provided in each of the intermediate slabs and one in each of the end slabs.

One, approximately 1 1/2-in. wide opening was observed in each of the end joint slabs when the pavement was inspected in March, 1962. No spalling or crushing of the gap slab was observed at that time, but distress was noted in the cork asphalt joint filler and sealant. During an October, 1963 inspection, it was observed that none of the transverse joints had retained their sealant.

Premolded neoprene joint sealants were placed into the transverse joints during 1964. The uncompressed premolded seal width was about 1 1/2-in. The material was compressed to about 1 in. at the time of pavement inspection during July, 1976. The top of the premolded sealant was squeezed from the joint and had been cut by aircraft traffic.

3. Melsbrock Airport, Belgium. A poststressed runway 11,100 ft (3383 m) long and 148 ft (45 m) wide was constructed in 1959. The pavement thickness was 71. in. (18.0 cm) and the pavement was poststressed longitudinally with flat jacks every 1082 ft (330 m). From available data a prestress value of 1650 psi (11,377 KPa) was recorded at a pavement temperature of 90°F (32.2°C) in June, 1961. Transversally, the pavement was post-tensioned with wire strands, but this prestressing amount is not available. During December, 1961, cracks appeared at a temperature of 27°F (-2.8°C). Restressing was done to close these cracks. Also two compression failures were reported at the active joints. These joint failures occurred in 1962 and 1963, respectively, on days with no wind and intense sunlight. The pavement surface temperature exceeded 120°F (48.9°C) at the time of the compression failures. Additionally, it is believed that steep temperature gradients caused additional prestress at the pavement surface.

4. Between Zwartberg and Meeuwen, Belgium. This road was constructed in 1960 and is 11,484 ft (3500 m) long (26/443 ft slabs) and 26 ft (7.9 m) wide. Three thicknesses were used: 3.2, 4.0, and 4.7 in. (8.1, 10.2, and 11.9 cm). Longitudinally, the slabs were poststressed with flat jacks and design prestressing values were 284, 427, and 529 psi (1958, 2944, and 3657 KPa), respectively for increasing thickness. Transversally, the slabs were post-tensioned with steel strands at prestressing design levels of 107 psi (738 KPa) for the 3.2 (8.1 cm) and 4.0 in. (10.2 cm) slabs and 273 psi (1882 KPa) for the 4.7 in. (11.9 cm) slabs. The variation in longitudinal prestress due to temperature changes was measured to be 30 to 40 psi per °F (373 to 497 KPa/°C) during the spring of 1960. Numerous cracks occurred by October, 1960. Restressing was applied in October, 1960, and again during the spring of 1961. An unspecified time later during that that same year, blowups occurred in two of the 3.2 in. (8.1 cm) thick slabs

at 104°F (40°C). The prestress amount was measured one day after the blowups at 2800 psi (19,306 KPa) five slabs away from the location of one of the failures. Each 443 ft (135 m) slab contracted 1.06 in. (2.69 cm) due to another restressing accomplished during December, 1961. The prestress applied at this time was 1000 psi (6895 KPa) at 41°F (5°C). During April, 1962, the temperature rose from 32°F (0°C) to 73°F (22.8°C) with the result that a blowup occurred in another slab (thickness unknown). A concrete stress of 2700 psi (18,616 KPa) was measured five slabs away. Additional cracks formed after the April, 1962, blowup. The pavement was restressed for the winter condition during September, 1962. More cracks formed after a sharp temperature drop during that same month. Joint faulting of 0.8 in. (2.03cm) was observed between two slabs during January, 1963, after a temperature drop to 14°F (-10°C). At several other active jacking joints, the slabs had lifted from the grade for a distance of 20 ft (6.1 m) from the joint. Another 3.2 in. (8.1 cm) thick slab experienced a blowup 16 ft (4.9 m) from the nearest joint in March, 1963. During May, 1963, a blowup again occurred in one of the slabs which had experienced a previous blowup (thickness unknown). This blowup also occurred 16 ft (4.9 m) from the nearest joint. The temperature at the time of this blowup was 95°F (35°C).

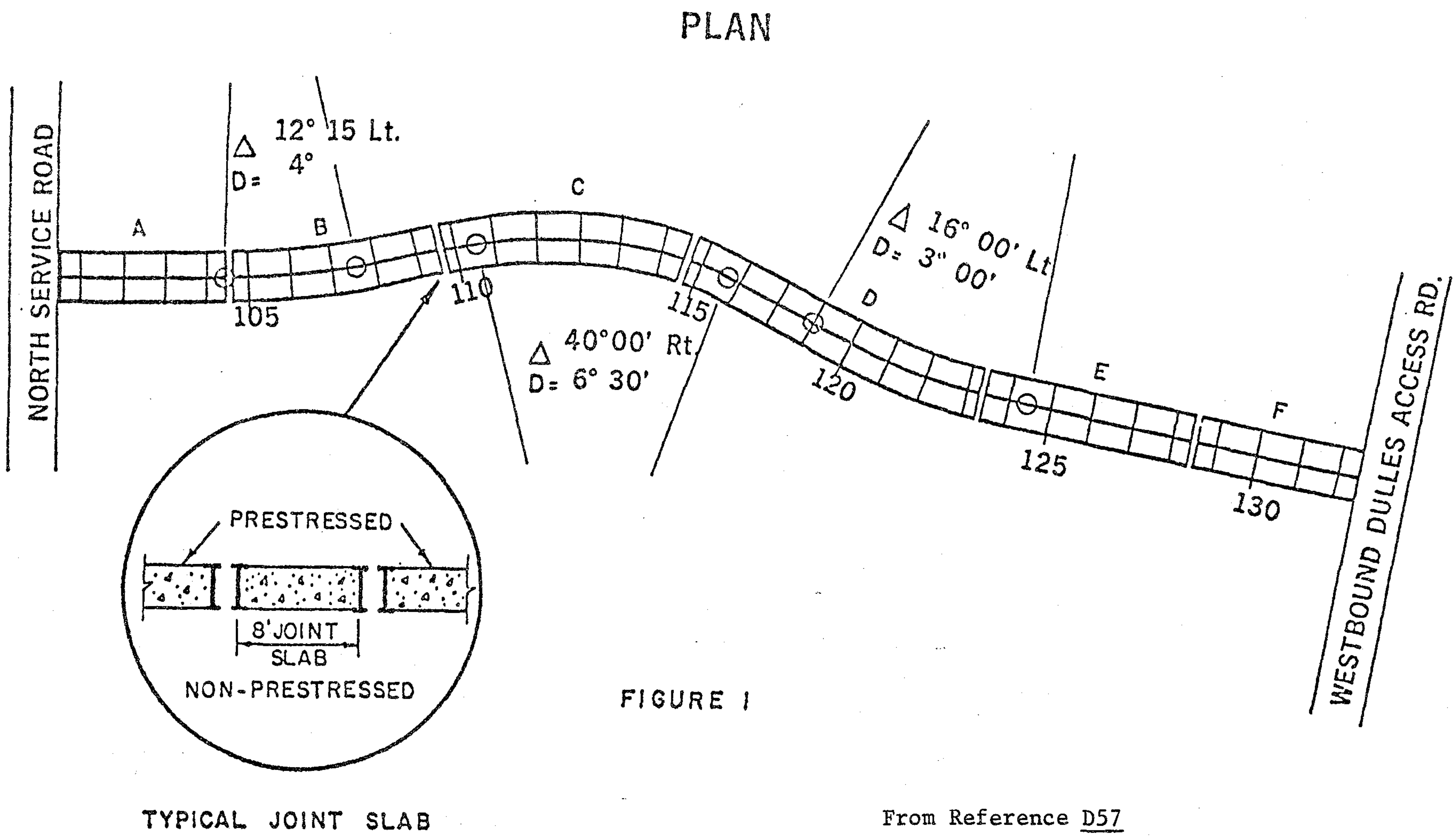
5. Lier, Belgium. This industrial factory road was constructed in 1964 and is 1450 ft (442 m) long, approximately 20 ft (6.1 m) wide, and 4.7 in. (11.9 cm) thick. Longitudinally, the pavement was poststressed with screw jacks at an initial prestress of 1850 psi (12,756 KPa). This value of prestress varied over a 5 year period ranging from 40 psi (276 KPa) at 34°F (1.1°C) to 1800 psi (12,411 KPa) at 100°F(38°C). No transverse prestressing was utilized. Active jacking joints were provided 287 ft (84.5 m) from each abutment. At an age of 3 1/2 years a 1/8 in. (0.32 cm) longitudinal crack appeared along 2/3 of the length of the road. No transverse cracks occurred. Apparently, the longitudinal crack is not considered as presenting any significant problems.

6. Dulles International Airport, Virginia. This road was constructed in 1971. Its length is 3200 ft (975 m) which consists of six slabs ranging from 400 to 760 ft (122 to 232 m) long. The road is 24 ft (7.3 m) wide and 6.0 in. (15.2 cm) thick. Longitudinally, the slabs were post-tensioned with wire strands. A prestress of 200 psi (1379 KPa) was applied at the ends

of all slabs. Transversally, no prestressing was applied but No. 3 or No. 4 bars were placed at 30 in. (76.2 cm) centers. Transverse cracks have occurred in several of the slabs. The first crack occurred in the 760 ft (232 m) slab shortly after construction when the concrete temperature dropped more than 40°F (4.4°C). However, as of 1976, none of the transverse cracks show any significant surface spalling. Joint distress has been encountered on this project. Several of the joints were removed and replaced after the gap concrete (placed after post-tensioning was completed) separated from the I-beam. This was reported to be due to inadequacies in the reinforcing design details of the gap concrete and to freezing of the dowel bars. Figures 20 and 21 show the layout and details of the joints installed on this project.

7. Route 222 Bypass, Kutztown, Pennsylvania. This road was constructed in 1972, is 500 ft (152 m) long, 24 ft (7.3 m) wide, and 6.0 in. (15.2 cm) thick. Longitudinally, the pavement is post-tensioned with 7-wire strand in steel tubing or sheathed with polypropylene. The design prestress amount was 204 psi (1407 KPa). No transverse prestressing was utilized but No. 3 or No. 4 bars were placed every 30 in. (76.2 cm). A transverse crack was first reported about one year after construction 300 ft (91.4 cm) from the east end of the slab and extends across both traffic lanes. The crack was approximately 0.05 in. (0.127 cm) wide at the slab surface. An oval shaped crack about 15 ft (4.6 m) long occurred in the outside traffic lane about 50 ft (15.2 m) from the west and of the slab. The cause of this crack was an overload during shoulder construction. Neither one of the reported cracks were spalling as of a July, 1976, inspection.

8. Route 114, Cumberland County, Pennsylvania. This road was constructed in 1973 and is 13,232 ft (4033 m) long (23/600 ft slabs), 24 ft (7.3 m) wide and 6.0 in. (15.2 cm) thick. Longitudinally, the pavement is post-tensioned with 7-wire strand in polypropylene conduit. Design prestress amount was 244 psi (1682 KPa). Transversally, no prestressing was applied. A transverse crack was observed in July, 1976, in two of the pavement slabs. There was no observed spalling at either crack. Joint repairs have been made on two of the nineteen joints. At one of the two joints repaired, the female beam separated from the slab concrete for a short distance. It is speculated that the male and female beam interlocked



From Reference D57

FIGURE D-20. Layout of Prestressed Concrete Pavement at Dulles International Airport, Virginia.
1 ft = 0.305 m

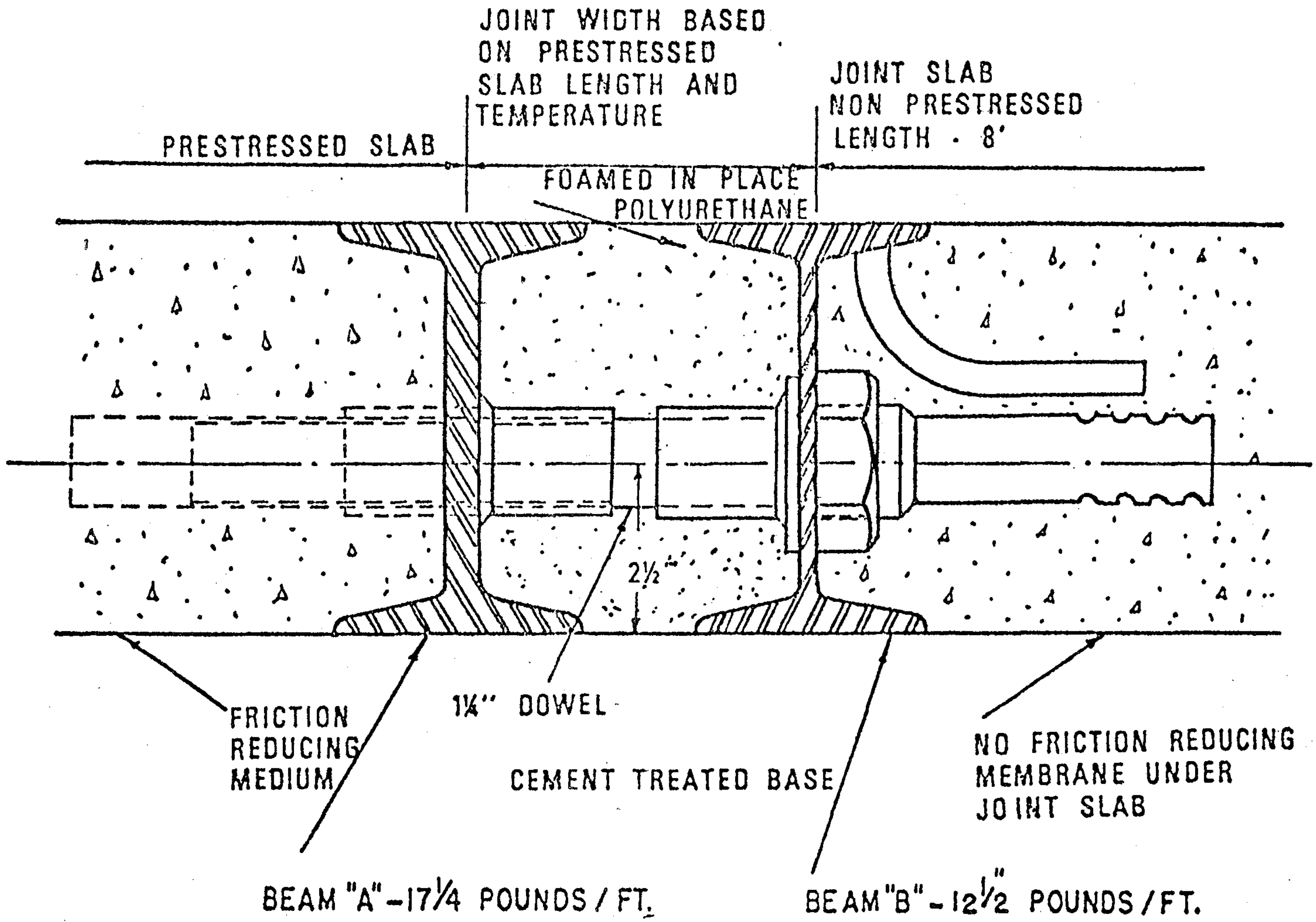


FIGURE D-21. Joint Detail. From Reference D57

1 in = 25.4 mm
1 ft = 0.305 m
1 lb = 0.454 kg

during pavement contraction. Repair at the second joint was due to gap concrete failure at a male beam which had interlocked with the female beam. Additionally, shoulder distress was observed along many of the slabs. Open interfaces with shoulder drop-offs in excess of 2 in. (5.1 cm) have occurred. The shoulders are constructed of an aggregate base covered with a double bituminous surface treatment.

Briefly summarizing the distress observed on the eight projects:

1. Heidenheim, Germany (Post-tensioned)
   - Transverse cracking, cause unknown.
2. Biggs Air Force Base, El Paso, Texas (Post-tensioned)
   - Transverse cracking, due to shrinkage cracking after temperature drop during early curing stage.
   - Joint distress, due apparently to excessive joint movement and inadequacy of joint sealants to accommodate movement.
3. Melsbrock Airport, Belgium (Poststressed)
   - Cracking (presumably transverse), occurred at low temperatures.
   - Compressive failures at active joints, occurred at high pavement temperatures.
4. Between Zwartberg and Meeuwen, Belgium (Poststressed)
   - Cracking (presumably transverse), occurred at low temperatures.
   - Blowups and slab lifting, occurred at high temperatures.
   - Joint faulting, occurred at low temperatures.
5. Lier, Belgium (Poststressed)
   - Longitudinal crack, cause unknown but was considered to be minor.
6. Dulles International Airport, Virginia (Post-tensioned)
   - Transverse cracks, at least one occurred shortly after construction when temperature dropped significantly.
   - Joint distress, related to inadequate reinforcement in the gap slabs and freezing of the dowel bars.
7. Route 222 Bypass, Kutztown, Pennsylvania (Post-tensioned)
   - One transverse crack (minor).
   - One oval shaped crack, due to temporary overload.
8. Route 114, Cumberland County, Pennsylvania (Post-tensioned)
   - Two transverse cracks (minor).

- Joint distress, problems occurred with male and female beams interlocking at joints.

For post-tensioned pavements, available information indicates transverse cracking and joint problems are the most common kinds of distress associated with these pavements. The most serious problems appear to occur with joints and these problesm seem to stem primarily from inadequate joint design.

Transverse cracking, various joint problems, and blowups were the most often observed distress types for poststressed pavements. This distress is in general caused by the varying prestress amount which is directly proportional to temperature changes.

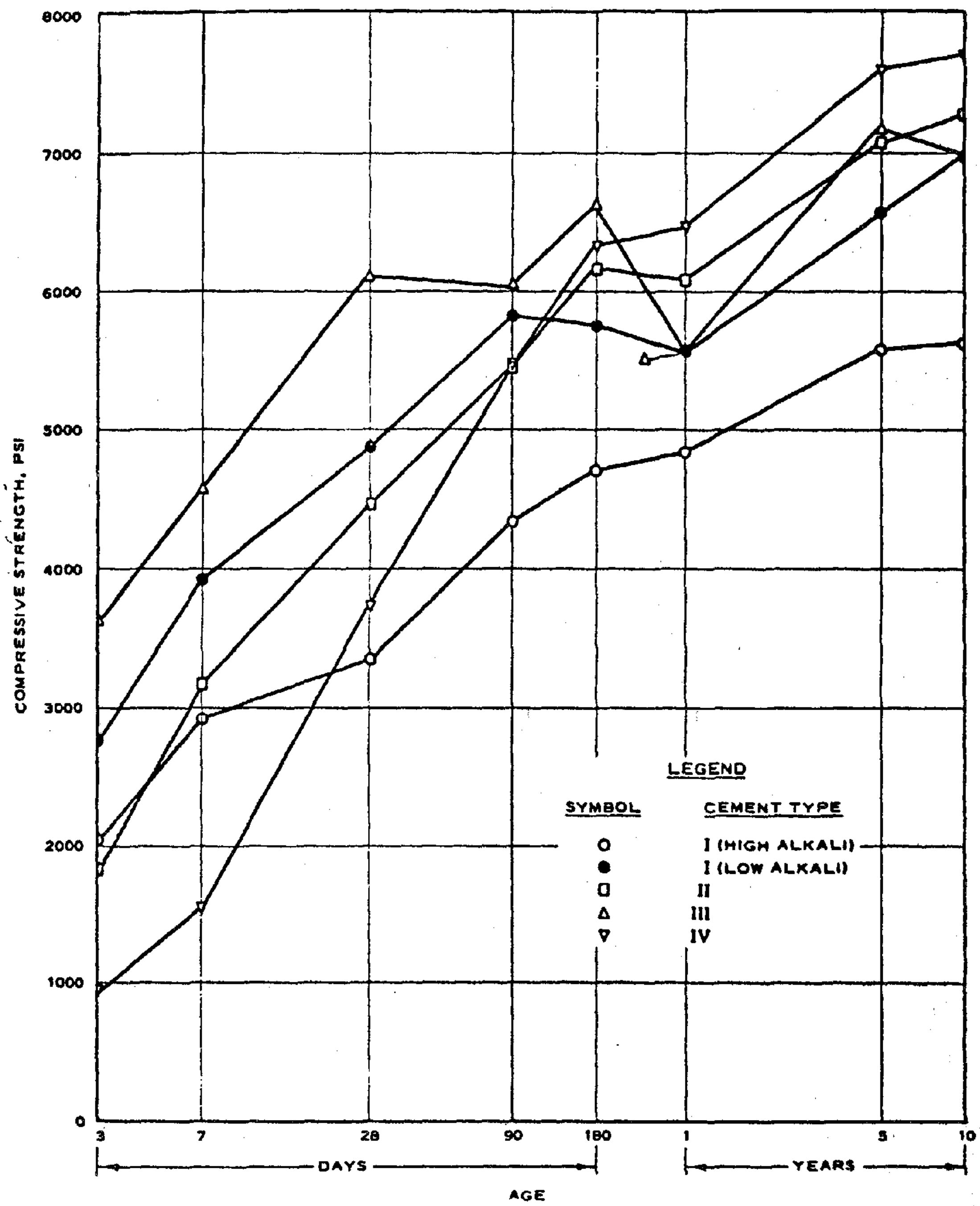
Two recommendations result from this short review of distress. One, if better joint construction methods can be found, then many of the distress problems associated with post-tensioned prestressed pavements can be eliminated. Two, it is not advisable to prestress existing in-service concrete pavements which are significantly faulted -- unless the faulting is corrected prior to prestressing.

Two Material Properties of In-Service Concrete Pavements

This section is specifically oriented toward two concrete material properties: compressive strength and creep. These two variables can have a significant impact on prestressing applied in new pavement construction. The effect of these two variables in applying prestressing to existing rigid pavements should also be examined. One reason is to determine typical in-service compressive strengths of concrete pavements and the other is to see if creep may be significant after the prestressing is applied.

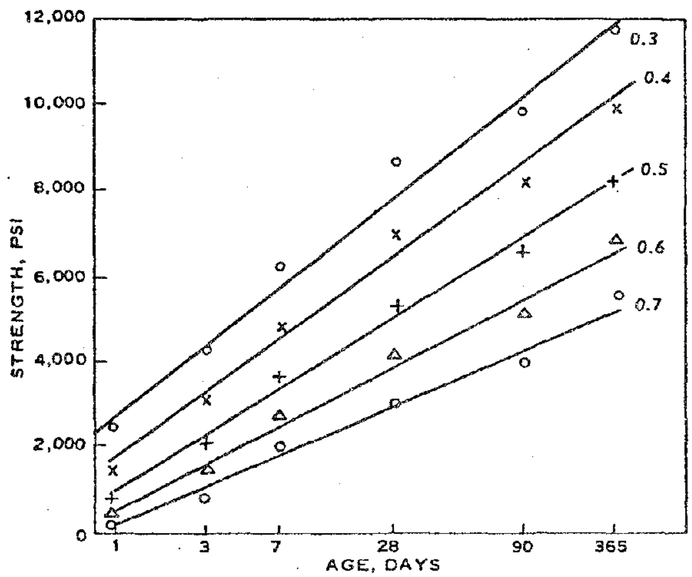
Compressive Strength. A generally accepted rule-of-thumb for concrete is that it gains strength for several years after placement. Of course, numerous factors can affect this strength gain. Such factors as water-cement ratios, curing methods, type of cement are some of the more important ones.

Mather (D 31) showed several examples of how these factors vary with time. Figure D-22 was obtained from a Waterways Experiment Station



From Reference D31

FIGURE D-22 Effect of Cement Characteristics on Strength Development to 10 Years Age With 0.5 Water-Cement Ratio and 6 Percent Entrained Air.
1 PSI = 6.89 kPa



From Plowman and Reference D31

FIGURE D-23 Age-Strength Relation for Various Water-Cement Ratios. 1 PSI = 6.89 kPa investigation and shows how compressive strength increases with concrete age for five different cement types for a concrete with a water-cement ratio of 0.5. As shown in the figure for a Type I (high alkali) cement, the compressive strength increases from 3300 psi (22,754 KPa) at an age of 28 days to approximately 5700 psi (39,302 KPa) at an age of 10 years. Thus, for this specific concrete, a 1.7 increase in compressive strength occurred in about 10 years. The same kind of trend was apparent for the non-air entrained concrete data also presented by Mather.

The influence of water/cement ratios on compressive strength was also described in this reference. Figure D-23 is a plot of data first presented by Plowman and summarized by Mather. This figure shows compressive strength versus concrete age for various levels of water-cement ratios. It is quite apparent that increasing the water-cement ratio significantly reduces both the initial and one year old compressive strength values.

One of the most informative references on concrete compressive strength increasing with age is by Washa and Wendt (D 29) reporting on work conducted at the University of Wisconsin. Concrete properties were measured over a period of up to 50 years on laboratory produced samples. The testing was conducted primarily on concrete cylinders made in either 1910, 1923, or 1937.

Table D-16 provides the basic information on the laboratory samples made in each of the three test series. This table shows that the cement used in the 1937 series is more typical of today's fine ground cements which results in higher amounts of C_3S - tricalcium silicate. Thus the 1937 series is of the greatest interest.

The 1937 series samples were moist cured for 28 days before being placed outdoors for the remaining length of the test period. Either 6 by 12 in. (15.2 by 30.5 cm) or 6 by 18 in. (15.2 by 45.7 cm) sized cylinders were used in the testing program. The results of the compressive strength testing performed can be seen in Figure D-24. The data plotted in this figure show that the 1937 series samples increased in compressive strength up to the 10-year point then decreased slightly from 10 years to 25 years of age. An increase by a factor of approximately 1.7 to 1.8 occurred from the initial 28 day compressive to the 10-year value. This is about the

TABLE D-16. Comparison of Compressive Strengths of Concretes at Different Ages With The compound Compositions of The Constituent Cements.

From Reference D29

Test series	Coarse aggregate	Mix proportions by Vol.	Mix proportions by Wt.	Cement content, Sk/yd[a]	Water cement By Vol.	Water cement By Wt.	Method of molding	Cement	Compound composition of cement C_3S	C_2S	C_3A	Surface area, cm^2/gr	Compressive strength, psi† at 1 mo	1 yr	5 yr	10 yr	25 yr‡	50 yr
A 1910	Lower Mag. Limestone	1:2:4	1:2.18:3.50	5.75	0.94	0.63	Hand	Atlas	28.9	44.4	11.4	1045	1920	3440	4250	4050	6250	6160
		1:3:6	1:3.27:5.25	4.01	1.36	0.90	Hand		28.9	44.4	11.4	1045	875	1920	2550	3010	3240	3970
B 1923	Average of						Hand	3M	43.7	29.7	11.0	1100	2285	3735	5250	5870	6645	6295
	Janes. Gr.	1:2:4	1:2.4:4.47	5.15	0.76	0.51	Hand	4M	32.8	38.6	14.2	1285	2715	3775	5780	6185	6915	6675
	Lannon Dol.	1:2:4	1:2.4:3.92	5.52	0.81	0.54	Hand	5M	46.5	25.1	14.8	1235	2740	3690	5735	5570	6475	6140
	Red Granite	1:2:4	1:2.4:4.00	5.41	0.77	0.51	Hand	7M	30.4	41.4	13.3	1295	3005	4165	6215	6610	7270	7390
C 1937	Janesville Gravel	1:1.66:3.80	1:2:4.25	5.68	0.74	0.49	Hand	3M7					3990	5195	7510	7865	7585	
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	3M7	53.1	21.5	10.4	1370	5030	6530	8780	8700	8200	
		1:2.09:4.78	1:2.51:5.34	4.73	0.74	0.49	Vib.	3M7					4285	5805	7400	7855	7840	
		1:1.66:3.80	1:2:4.25	5.66	0.74	0.49	Hand	5M7					4525	6015	7540	8025	7885	
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	5M7	52.6	21.0	11.9	1780	6185	8195	9015	10465	9850	
		1:2.09:4.78	1:2.51:5.34	4.75	0.74	0.49	Vib.	5M7					4990	6240	7820	7885	8050	
		1:1.66:3.80	1:2:4.25	5.72	0.72	0.48	Hand	5UM					5150	7395	7940	8525	8070	
		1:1.66:3.80	1:2:4.25	5.84	0.59	0.39	Vib.	5UM	37.5	34.0	4.53	1920	6265	8310	10430	10145	9915	
		1:2.09:4.78	1:2.51:5.34	4.76	0.71	0.47	Vib.	5UM					5130	7325	8860	9015	8250	
		1:1.66:3.80	1:2:4.25	5.67	0.74	0.49	Hand	I					5145	6715	7640	7910	6835	
		1:1.66:3.80	1:2:4.25	5.82	0.60	0.40	Vib.	I	56.5	16.2	13.0	2110	7050	8695	9995	9515	8985	
		1:2.09:4.78	1:2.51:5.34	4.74	0.74	0.49	Vib.	I					5480	7360	8405	8180	7500	

[a]All specimens stored outdoors after moist curing.

Series A test results adjusted to 12-in. (30 cm) cylinder length for purposes of comparison ($S_{12} = 1.05\ S_{18}$). (See Reference 3).

†psi values may be converted to kgf/cm^2 by multiplying by 0.07031.

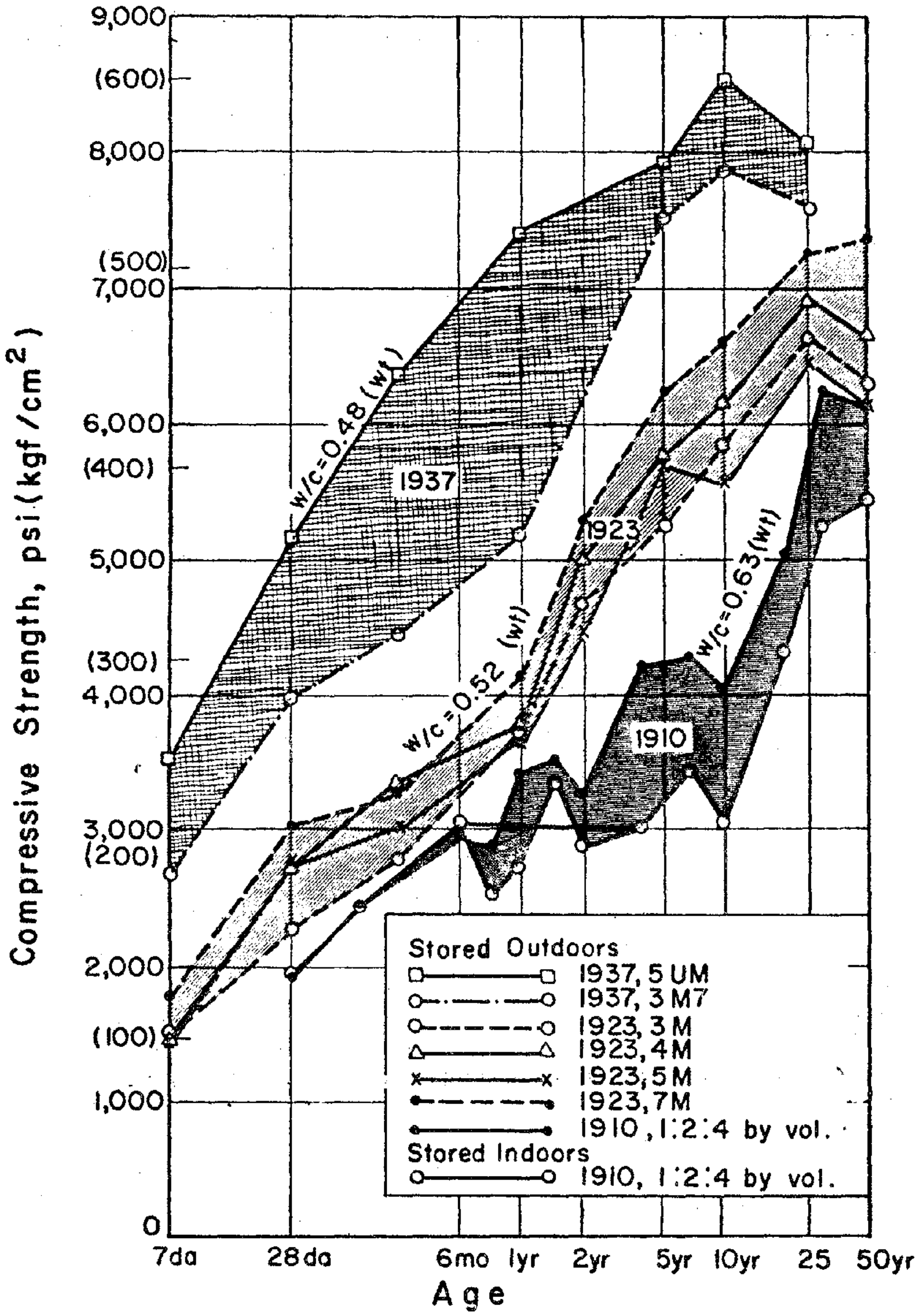
‡Test Series A results at 30 yr.

1 psi = 6.895×10^3 Pa

same increase as reported by Mather for concrete also aged 10 years.

With a rough idea of how concrete compressive strength can increase with age, the next question might be what are typical initial compressive strengths used in pavement concrete. In Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, a minimum compressive strength of 3,500 psi (24,132 KPa is recommended if the mix specification is to be based on strength. Finney (D 45) states that the average compressive strength of pavement concrete at one year of age is consistently between 4,000 and 6,000 psi (27,580 and 41,370 KPa). Finney also states in reviewing a PCA survey conducted in 1969 that 12 states require 28 day compressive strengths fanging from a minimum of 3,000 psi (20,685 KPa) to a maximum of 4,370 psi (30,131 KPa). Additionally, in "Specifications for Concrete Pavements and Concrete Bases" (ACI 617-58) it is stated that an average compressive strength shall not be less than 4,000 psi (27,580 KPa) at 28 days for use in the design of dowels and tie bars. Typical compressive strengths reported for the AASHO Road Test (D 32) ranged from about 2,900 psi (19,996 KPa) at 3 days and over 6,100 (42,060 KPa) psi at 2 years for 1-1/2 in. (3.81 cm) maximum size aggregate.

Typical in-service concrete pavement compressive strengths have been reported in numerous references. Two of these references are by Chastin and Burke (D 5), Velz and Carsberg (D 43). Chastin and Burke (D 5) reported on US 66 concrete pavement built in Illinois in 1952. Utilizing Type 1A cement and a water-cment ratio of 0.48, the average strength of 44 cores was 4,433 psi (30,566 KPa) at an average age of 40 days. Velz and Carsberg (D 43) reported on concrete pavement built in Minnesota in 1940. Using Type 1 cement and a water-cement ratio ranging from 0.51 to 0.54, the average compressive strength at 28 days was 4,536 psi (31,276 KPa) (based on 6 in. modified cubes), at 150 days 5,451 psi (37,585 KPa) (based on 6 in. cores), and at 19 years 7,660 psi (52,816) (based on cores). Thus the compressive strength increase for this project over the 19 year period was 1.7 times the initial 28 day value. The above discussion can be summarized by assuming that the average 28 day compressive strength for existing in-service concrete pavements ranges from 3,500 to 4,500 psi (24,132 to 31,028 KPa). By applying a strength increase factor



From Reference D29

FIGURE D-24. Compressive Strength-Age Relations For Specimens Molded in 1910, 1923 and 1937.

of about 1.7 for pavements 10 years old, expected ranges of compressive strength would be from 6,000 to 7,700 psi (41,370 to 53,092 KPa). For pavements less than 10 years old, these values would be less and for pavements more than 10 years the compressive strengths should not be expected to change significantly from their 10 year values.

Creep. The creep of in-service pavements which receive prestressing is of interest. The amount of creep that occurs after the prestressing is applied will influence the type and amount of prestressing to be used. Creep of "young" concrete has been well defined in the literature but the effect of applying compressive loads to aged concrete is not.

Hanson (D 30) in a 1953 Bureau of Reclamation report described the creep studies which extended over a period of 10 years on concretes from several dams constructed in the Western United States. The specimens tested were sealed after overnight curing and remained sealed for the duration of the testing program. Various specimens obtained from the Shasta Dam in California were initially loaded at an early age and subsequently 7 years later and are therefore of interest. The concrete from this dam utilized Type IV cement and a water-cement ratio of 0.58. No air entrainment was used and the maximum sized aggregate ranged from 3 to 6 in. (7.6 to 15.2 cm). Specimen size was 6 in. by 27 in. (15.2 cm by 66.0 cm) cylinders. For samples loaded at 28 days to 500 psi, (3448 KPa), the elastic plus creep strain was computed to be 520 millionths after 10 years of continuous loading. For identical samples loaded at 7.25 years of age, the elastic plus creep strain was measured at 120 millionths for loadings up to about 2 years.

Troxell, Raphael, and Davis (D 33) also conducted long term creep tests on concrete specimens. The influence of many concrete variables were investigated in this testing program but of particular interest are the creep measurements made on specimens subjected to similar conditions as the ones reported by Hanson (D 30). A group of 4 in. by 14 in. (10.2 cm by 10.2 cm) cylinders with a water-cement ratio of 0.69 and 1-1/2 in. (3.81 cm) macimum size aggregate were moist cured for 28 days then subjected to a 600 psi (4137 KPa) compressive loading. Under loading the samples were exposed to 70°F (21.1°C) air and 70 percent relative humidity. After 10 years of this loading condition creep was reported at a strain value of approximately 750 millionths. This value is about 44 percent higher than the reported by Hanson but several major variables were different - such as water-cement ratio, aggregate size, cement type and others. The important point in this comparison is that creep tests conducted on concretes obtained from dam construction are at least in the same range of values obtained for typical pavement concrete thus providing a rough check on the Shasta Dam specimens which were loaded after approximately 7 years.

Based on the preceding discussion, it is speculated that existing in-service concrete pavements prestressed up to 500 psi (3448 KPa) can be expected to creep at a strain somewhere between 100 to 200 millionths. For a 300 psi (2068 KPa) prestressing value, this strain would range from 50 to 100 millionths. At this prestressing level and without considering subgrade friction or potential joint effects, a 100 ft (30.5 m) slab would move a total of about 0.09 in. (0.229 cm) or 0.045 in. (0.114 cm) at each slab end.

Conclusions

1. The number of new prestressed concrete pavements reported in the literature are declining.

2. Post-tensioning is the most common prestressing technique used in prestressing new concrete pavements.

3. The majority of prestressed pavements reviewed were constructed in either Great Britain, the United States or Germany.

4. With the possible exception of CRCP it may be reasonable to assume that many of the existing in-service concrete pavements have reached and passed maximum subgrade friction values and are now operating at lower friction levels than when initially constructed. This is due to environmentally induced movements.

5. The coefficient of friction for existing in-service concrete pavements which did not receive special subgrade or supporting layer friction reducing methods during construction can be expected to range between 1.1 and 2.9. Existing in-service pavements which did receive friction reducing construction methods can be expected to range between 0.3 and 1.0.

6. Graphite or water could be possible materials to inject underneath existing pavements to reduce friction and hence increase the benefit of prestressing such pavements.

7. Prestressing of existing concrete pavements should not be attempted if frozen subgrade (supporting layer) conditions exist.

8. For post-tensioned pavements, transverse cracking and joint problems are the most common kinds of observed distress. For poststressed pavements, these distress types are transverse cracking, various joint problems and blowups.

9. More emphasis should be placed on the design of joints for prestressed pavements.

10. Existing in-service concrete pavements which are faulted should not be considered for prestressing unless the faulting is corrected pfior to prestressing.

11. Expected compressive strengths for 10 year ole in-service concrete pavements can be expected to range between 6,000 and 7,700 psi (41,370 and 53,092 KPa).

12. Creep of existing concrete pavement after prestressing is applied is not expected to be major consequence. This conclusion is based on the assumption that existing cracks and joints will be adequately prepared to prevent movement prior to prestressing and that significant creep will not occur in aged concrete.

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APPENDIX E. TIME VALUE OF MONEY (INTEREST)

Highway engineers almost always find that the money consequences of any alternative involving construction, rehabilitation, or maintenance occur over a substantial period of time - often many years. Thus the question arises - can we simply add up the various sums of money at various times and obtain a net result? Obviously money today and money tomorrow do not have the same "worth", or "value" in terms of what goods and services they can buy. Factors such as inflation, supply and demand, and individual likes and dislikes affect the value of particular goods and services. But, these factors aside, is there still a time value of money? Again, the obvious answer is yes! One hundred dollars today is worth more than the promise of $100 one year from now. The reason is that you can take the $100 you receive today, invest it in a savings account and receive at least $105 one year from now. The use of money is a valuable asset - so valuable that people are willing to pay to have money available for their use. Money can be rented in roughly the same way one rents an apartment, only with money the charge for its use is called interest instead of rent.

The existence of interest is demonstrated by the continuing offer by banks and savings institutions to pay for the use of people's money, to pay interest. (Currently, a passbook savings account will draw at least 5% interest.)

What about the treatment of interest in economy studies for public works, such as highways? An excellent treatment of this question is given by Grant, et al. (E 1):

> Engineers have not always agreed on the point of view that should be taken toward the treatment of interest in judging the soundness of proposed public works expenditures. Some different viewpoints on this subject have been as follows:
>
> 1. Costs should, in effect, be computed at zero interest rate. The advocates of this viewpoint have generally restricted its application to those public works that were financed out of current taxation rather than by borrowing.
> 2. Costs should be computed, using an interest rate equal to the rate paid on borrowings by the particular unit of government in question. If the proposed public works are to be financed by borrowing, the probable cost of the borrowed money should be used. Otherwise the average cost of money for long-term borrowings should be used.
> 3. Just as in private enterprise, the question of the interest rate to be used in an economy study is essentially the question of

what is a minimum attractive rate of return under the circumstances. Although the cost of borrowed money is one appropriate element in determining the minimum attractive rate of return, it is not the sole element to be considered. In most instances the appropriate minimum attractive rate of return should be somewhat higher than the cost of borrowed money.

Our discussions in Chapters 9 and 11 made it clear that the authors of this book favor the view stated under heading 3. Some further aspects of the case supporting this view are developed following Example 19-1.

Example 19-1. The Effect of the Selection of the Minimum Attractive Rate of Return on a Comparison of Highway Bridge Types

Facts of the Case. In a certain location near the Pacific Ocean, two alternative types of highway bridge are under consideration for the replacement of an existing timber trestle bridge on a state highway in a rural area. The first cost of a steel bridge will be $340,000; the first cost of a concrete arch bridge will be $390,000. Maintenance costs for the steel bridge consist chiefly of painting; the average annual figure is estimated to be $3,000. Maintenance costs on the concrete arch bridge are assumed to be negligible over the life of the bridge. Either bridge has an estimated life of 50 years. The two bridges have no differences in their prospective services to the highway users.

It is evident that in this instance the choice between the two types depends on the assumed interest rate or minimum attractive rate of return. A tabulation of annual costs with various interest rates is as follows:

Interest Rate	Annual Cost		Difference in Annual Cost	
	Steel	Concrete	Favoring Steel	Favoring Concrete
0%	$ 9,800	$ 7,800		$2,000
2%	13,820	12,410		1,410
4%	18,830	18,150		680
5%	21,630	21,360		270
6%	24,570	24,740	$ 170	
8%	30,790	31,880	1,090	
10%	37,290	39,340	2,050	
12%	43,940	46,960	3,020	

If i^* is below 5.6%, the concrete bridge is more economical for this location. If above 5.6%, the steel bridge is more economical.

The Need for Some Minimum Attractive Rate of Return in Economy Studies for Public Works. Examples 9-1 and 19-1 both dealt with economy studies for state highway projects. In general, such projects in the United States have been financed chiefly by current highway-user taxes and have involved little or no public borrowing. This is the field in which the advocates of the 0% interest rate in public works projects were most articulate. It is also a field in which the funds available in any year have been limited by current tax collections, and in which there often have been many desirable projects that could not be constructed because of the limitation on current funds.

Example 19-1 represents a type of decision that usually is made on the level of engineering design rather than on the policy level of determining the order of priority of projects competing for funds. If each authorized project is to be designed to best advantage, it is essential that economy studies be made to compare the various alternative features in the design. If such studies were made at 0% interest, and if the conclusions of the studies were accepted in determining the design, many extra investments would be made that would yield relatively small returns (such as 1% or 2%). These extra investments in the projects actually undertaken would absorb funds that might otherwise have been used for additional highway projects. If the additional projects put off by a shortage of funds should be the ones where the benefits to highway users represented a return of, say, 15%, on the highway investment, it is clearly not in the over-all interest of highway users to have invested funds earning a return of only 2%. In other words, where available funds are limited, the selection of an appropriate minimum attractive rate of return calls for consideration of the prospective returns obtainable from alternative investments. This is as sound a principle in public works as it is in private enterprise.

If the time should ever be reached when economy studies indicate that all the highway funds currently available cannot be used without undertaking a number of highway investments yielding very low returns (such as 2%), a fair conclusion would be that highway user taxes should be lowered. In such a case the alternative investments would be those that might be made by individual taxpayers if taxes should be reduced. Money has a time value to the taxpayers; this is a fact that shoudl be recognized in the use of funds collected from taxpayers.

On the basis of the foregoing comparisons, this report involves the use of interest on money. In 1962, 45% of the state agencies used an interest rate of 0%, 22% used an interest rate between 2 and 3 3/4% and 33% used rates between 4 and 7% (E2). Today, most, if not all, state transportation agencies, use some interest rate in their economics evaluations. In 1959 a value of 7% was suggested as an appropriate value for interest rate to reflect the value of money at that time (E3). More recently (1976), Caltrans has selected 7% as a realistic value to determine the present worth of future dollars (E4). Finally, looking at the average costs to borrow money, as reflected in the municipal bond rate (5%) the public utility bond rate (8%) and the home mortgage rate (9%) reported in Engineering News Record (E5), the 7% interest rate utilized by many agencies for the past 15 years seems appropriate.

Therefore, on this project, an interest rate of 7% has been selected.

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APPENDIX F. UPDATING ESTIMATED REHABILITATION COST INFORMATION

As cost information is obtained from various sources at various times it is necessary to bring these costs to a common time frame. For the purposes of this project, April 1, 1977, has been selected. In order to convert whatever cost figures obtained to a April 1, 1977, (first quarter 1977) estimate, the cost index method is used. In this method:

$$C_c = C_o \left(\frac{I_c}{I_o} \right)$$

Where: C_c = Current estimated total cost

C_o = Total cost at other time "O" (same sized project)

I_c = Current index number (first quarter 77)

I_o = Index number at other time "O"

The index number to use depends upon the type of cost being estimated. Four indices are given from which to choose:

1. The ENR Construction Cost Index (F 1)
2. The ENR Bid Price Trends on Federal - Aid Highway Contracts (F 1, F 2)
3. The ENR Equipment Price Index (F 1)
4. The Cost Trends on Highway Maintenance and Operations (F 2)

The ENR Construction Cost Index (Table F-1) was designed as a general purpose construction cost index to chart basic costs with time. It is a weighted aggregrate index of constant quantities of structural steel, portland cement, lumber, and common labor, valued at $100 in 1913.

The Bid Price Trends on Federal - Aid Highway Contracts is compiled by the Federal Highway Administration as reported by state transportation agencies, (Table F-2). The base year for this index is 1967.

The ENR Equipment Price Index (Table F-3) is compiled from Bureau of Labor statistics and only the January 1977 index is given (for a base year of 1967). To use this index subtract 100 from the 1977 index then divide by 10 to obtain an average yearly percent increase in equipment costs. Then use as shown in the following example.

Equipment: Concrete paver which cost $125,000 in June 1971.

Time change - June 1971 to April 1977 = 5.75 years.

Index from Table 3 = 176.9

Average yearly increase = (176.9 - 100)/ 10 = 7.69%

$$\text{Current Equipment Cost Estimate} = 125{,}000\,[1 + 5.75\,(.0769)]$$
$$= 125{,}000\,(1.442)$$
$$= \$180{,}000$$

The Cost Trends for Highway Maintenance and Operations (Table F-4) are given through 1973 (the latest year available). To update to 1977, an estimate must be made using Figure F-1.

Table F-1. Construction Cost Index History 1903-1976

How ENR builds the Index: 200 hours of common labor at the 20-cities average rate, plus 25 cwt of standard structural steel shapes at the mill price, plus 22.56 cwt (1.128 tons) of Portland cement at the 20-cities average price, plus 1,088 board feet of 2 x 4 lumber at the 20-cities average price.

1903	94	1911	93	1919	198	1927	206	1935	196	1943	290	1951	543
1904	87	1912	91	1920	251	1928	207	1936	206	1944	299	1952	569
1905	91	1913	100	1921	202	1929	207	1937	235	1945	308	1953	600
1906	95	1914	89	1922	174	1930	203	1938	236	1946	346	1954	628
1907	101	1915	93	1923	214	1931	181	1939	236	1947	413	1955	660
1908	97	1916	130	1924	215	1932	157	1940	242	1948	461	1956	692
1909	91	1917	181	1925	207	1933	170	1941	258	1949	477	1957	724
1910	96	1918	189	1926	208	1934	198	1942	276	1950	510	1958	759

1913=100

Monthly

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual Average
1960	812	813	813	815	823	827	829	830	831	830	830	831	824
1961	834	834	834	838	847	850	854	854	854	854	855	855	847
1962	855	858	861	863	872	873	877	881	881	880	880	880	872
1963	883	883	884	885	894	899	909	914	914	916	914	915	901
1964	918	920	922	926	930	935	945	948	947	948	948	948	936
1965	948	957	958	957	958	969	977	984	986	986	986	988	971
1966	988	997	998	1006	1014	1029	1031	1033	1034	1032	1033	1034	1019
1967	1039	1041	1043	1044	1059	1068	1078	1089	1092	1096	1097	1098	1070
1968	1107	1114	1117	1124	1142	1154	1158	1171	1186	1190	1191	1201	1155
1969	1216	1229	1238	1249	1258	1270	1283	1292	1285	1299	1305	1305	1269
1970	1309	1311	1314	1329	1351	1375	1414	1418	1421	1434	1445	1445	1385
1971	1465	1467	1496	1513	1551	1589	1618	1629	1654	1657	1665	1672	1581
1972	1686	1691	1697	1707	1735	1761	1772	1777	1786	1794	1808	1816	1753
1973	1838	1850	1859	1874	1880	1896	1901	1902	1929	1933	1935	1939	1895
1974	1940	1940	1940	1961	1961	1993	2040	2076	2089	2100	2094	2101	2020
1975	2103	2128	2128	2135	2164	2205	2248	2274	2275	2293	2292	2297	2212
1976	2305	2314	2322	2327	2357	2410	2414	2445	2465	2478	2486	2490	3401
1977	2494	2505	2513										

From Reference F 1.

1 board ft = 25.4 mm x 304.8 mm x 304.8 mm

1 in = 25.4 mm, 1CWT = 45.36 kg

Table F-2. Bid Price Trends on Federal-Aid Highway Contracts

Federal Highway Administration Base: 1967 = 100

			Surfacing			Structures					
	Excavation price (yd^3)	Index	PCC price (yd^2)	Bit. conc. price (t)	Combined Index	Rein. steel price (lb)	Struc. steel price (lb)	Struc. conc. price (yd^3)	Combined Index	Highway Bid Price Index	ENR Building Cost Index
1967	0.54	100.0	4.43	6.47	100.0	0.131	0.247	70.30	100.0	100.0	100.0
1970	0.66	121.8	5.42	8.04	123.3	0.163	0.338	92.73	132.2	125.6	124.4
1971	0.67	123.8	6.06	8.54	134.5	0.177	0.348	97.02	138.5	131.7	140.5
1972	0.72	133.4	6.25	9.22	141.9	0.181	0.342	100.17	140.6	138.2	155.2
1973 Av.	.80	147.1	6.87	9.99	154.8	.207	.373	111.83	156.5	152.4	171.3
Q1	0.67	124.7	6.57	9.85	150.3	0.181	0.295	109.34	141.9	137.8	166.3#
Q2	75	138.0	6.36	9.90	148.2	0.193	0.352	113.51	153.4	145.9	168.5#
Q3	81	149.5	7.10	9.61	154.7	0.212	0.422	110.60	162.1	155.1	170.4#
Q4	93	172.7	7.43	10.83	167.7	0.233	0.379	113.51	162.0	167.8	171.8#
1974 Av.	1.00	184.1	8.67	14.74	211.3	.340	.551	136.80	214.5	201.8	179.8
Q1	97	179.7	8.17	13.28	194.6	.281	.459	129.64	190.2	187.4	171.0#
Q2	96	178.0	8.48	15.77	216.8	.342	.555	137.07	215.4	201.4	177.5#
Q3	1.02	187.9	8.82	14.64	212.4	.371	.577	152.57	233.7	209.7	183.2#
Q4	1.03	190.6	9.10	15.18	219.7	.362	.648	130.33	224.1	209.9	183.8#
1975 Av.	1.03	190.6	8.62	15.13	213.8	.297	.554	138.76	210.5	203.8	195.5
Q1	1.02	188.1	9.84	13.95	219.1	.332	.577	140.93	219.7	207.3	187.3#
Q2	1.00	184.9	8.22	14.35	203.2	.320	.542	139.85	213.1	199.3	193.4#
Q3	1.02	188.8	8.49	15.58	215.5	.283	.556	142.13	211.5	203.9	196.9#
Q4	1.10	202.6	9.00	16.41	227.7	.277	.548	131.90	207.9	209.8	199.8#
1976 Av.	1.03	191.2	8.65	15.07	213.7	.257	.493	138.75	198.1	200.4	219.9
Q1	1.04	192.0	7.70	16.28	212.3	.251	.543	133.72	199.3	200.3	202.7#
Q2	1.05	194.3	8.56	14.13	205.5	.242	.510	145.65	203.1	200.4	217.5#
Q3	1.03	191.1	9.18	15.12	219.4	.264	.438	135.28	189.6	199.0	227.6#
Q4	1.01	187.3	9.17	14.76	217.4	.271	.481	141.34	200.4	200.4	231.8#
%chg.											
Q3'76-Q4'76	-2.0	-2.0	-	-1.7	-0.9	+2.9	+9.8	+4.5	+5.7	+0.7	+1.8
Q4'75-Q4'76	-8.2	-7.6	+1.9	-10.1	-4.5	-2.2	-12.2	+7.2	-3.6	-4.5	-16.0

#last month of quarter

From Reference F 1.

1 yd^3 = 0.765 m^3, 1 yd^2 = 0.836 m^2, 1 lb = 0.454 kg

Table F-3. Equipment Price Indexes

Bureau of Labor Statistics, 1967=100

	Jan. 1977	%chg. 10/76-1/77	%chg. 1/76-1/77
All Construction Equipment*	208.8	+3.3	+8.0
Power Cranes, Excavators & equip	210.1	+2.8	+7.1
Crane, hydr., rbbr-tired,12-18tons(a)#.	196.1	+2.2	+7.7
trk-mntd,15-25tons(e)	147.4	+3.2	+3.9
25-50tons(e)	149.2	+1.2	+4.0
cable, trk-mntd,50-100tons(e)	159.2	+3.2	+7.6
crawler,50-100tons(e)	181.5	+2.8	+7.0
Excavators, hydr.(e)	151.1	+2.4	+6.3
Bucket,clamshell,1/4 yd^3	267.6	+6.3	+14.3
dragline,3/4 yd^3	271.7	+3.4	+9.5
Backhoes	180.7	+2.6	+3.7
Scraper,12-18 yd^3	199.6	+1.0	+3.6
20-35 yd^3	213.3	+9.2	+15.2
Grader,115-144 BHP	200.4	+3.5	+7.5
Tractors	215.0	+4.3	+9.2
Wheel, off-highway,250-350 HP	223.4	+6.6	+9.6
375-475 HP(c)	188.8	+1.9	+6.1
Crawler,60-89 net eng HP	185.2	--	+11.0
90-129 net eng HP	214.1	+3.2	+6.3
130-199 net eng HP	228.3	+3.0	+7.0
200- & over net eng HP	226.1	+4.1	+7.8
Shovel-Loader, crawler,90-129 HP	198.3	+2.8	+5.8
rbbr-tired,2 1/2 & under 3 1/2 yd^3(e)	156.1	+2.9	+5.8
rbbr-tired, 5 & under 7 1/2 yd^3(e)	162.0	+3.6	+5.9
Contractors' Off-Highway Truck, 50-ton	217.5	+3.5	+6.5
Roller, tandem	--	--	--
pneumatic	--	--	--
vibratory(d)	--	--	--
Dewatering Pump, 10 m GPH	197.1	+1.5	+.9
90 m GPH	231.4	+1.8	+7.8
Portable Air Compressors	124.1	+3.4	+8.7
Mixers, Pavers, Spreaders	176.9	+2.9	+7.5
Concrete Mix Plant, mobile(c)	158.6	+2.8	+7.1
Truck Mixer, 7 yd^3	172.7	+6.6	+12.8
Slipform Paver(d)	--	--	--
Bituminous Batch Plant, portable(b)	175.7	+1.6	+3.1
Bituminous Spreader	194.9	--	+7.6
Crushing Plant, portable(b)	190.5	+1.8	+6.3
Welding Machines and Equipment	191.0	+1.9	+2.8

(a)Dec. '67=100 (b)Dec. '68=100 (c)Dec. '69=100 (d)Dec. '70=100 (e)Dec. '72=100 Manufacturer to Dealer (first transaction) Construction Equipment Price Indexes by Bureau of Labor Statistics. Department of Labor.
*Excluding welding machinery. #Self-propelled.

From Reference F 1.

1 ton = 907.2 kg
1 yd^3 = 0.765 m^3

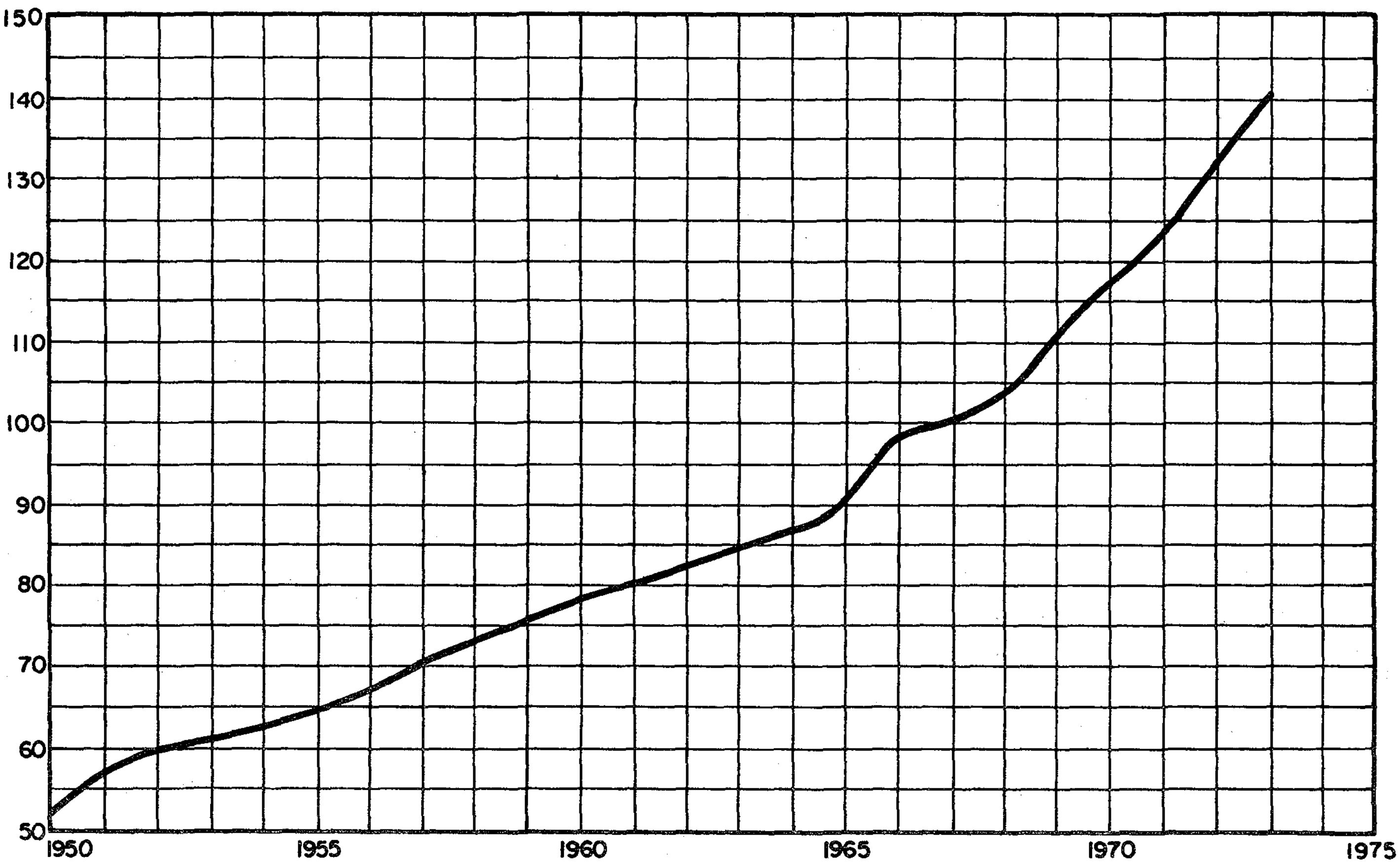
Table F-4. Cost Trends

Highway Maintenance and Operation[1]

1967 = Base Year

Year	Labor	Material	Equipment	Overhead	Total
1950	43.58	74.53	57.66	57.07	51.31
1951	47.76	81.07	64.34	62.23	56.41
1952	51.15	81.99	66.86	65.05	59.28
1953	52.00	82.54	68.76	65.73	60.33
1954	54.89	83.49	70.40	66.42	62.55
1955	55.94	82.80	74.24	67.71	64.09
1956	58.70	86.91	74.06	70.55	66.31
1957	63.20	90.86	75.66	78.22	70.28
1958	65.74	92.27	78.91	81.21	72.90
1959	67.82	92.40	83.15	81.88	75.17
1960	71.02	94.68	86.98	84.19	78.35
1961	73.25	95.18	87.19	85.08	79.82
1962	76.06	96.66	88.76	86.47	82.09
1963	79.46	96.87	89.25	88.05	84.32
1964	81.79	97.48	91.25	89.98	86.35
1965	85.69	99.23	94.23	92.01	89.66
1966	98.02	99.68	96.70	96.23	97.76
1967	100.00	100.00	100.00	100.00	100.00
1968	103.63	102.03	100.42	105.03	102.79
1969	113.71	106.24	104.24	110.86	110.44
1970	122.02	111.03	106.56	116.81	116.78
1971	129.67	117.37	107.93	122.76	122.68
1972	138.21	124.27	119.98	128.71	131.68
1973	148.04	130.42	133.70	134.66	141.75

[1]These data are prepared from the unit cost information submitted each year by State highway departments, and cover both physical maintenance and major traffic service items including snow and ice control. Previous issues of this table used base period 1957-59.



FROM REFERENCE F2.

Figure F-1. Highway Maintenance and Operation Cost Index.

References

F 1. Engineering News Record, March 24, 1977.

F 2. Highway Statistics, 1973, FHWA, 1973.

APPENDIX G. UPDATING USER COST AND ACCIDENT COST INFORMATION

User costs have been obtained from McFarland's data (G 1) which reflects 1972 dollars. Accident costs are based on a 1975 report which reflects 1974 dollars ($1240 per accident) (G 2). The problem is how to update these costs to 1977 dollars. To accomplish this the consumer price index has been selected. (Table G-1). This index reflects the average increase in prices for a number of selected consumer products from 1965 (the selected base year) through 1976. To estimate the index for April 1977 is 186.

Accidents costs, updated to April 1977, are:

$$C_c = \$1240 \left(\frac{186}{154}\right) = 1240 \times 1.21 = \$1500$$

This value has been entered into the computer program as a default value, which means the value does not have to be entered each time a technique is analyzed.

User costs must be entered each time a technique is analysed. To update the McFarland data, as prepared in graphical form, multiply the value obtained from the graph by:

$$\frac{186}{131} = 1.42$$

For example, if user costs per vehicle mile of $0.16 was obtained from the graph, enter $0.16 x 1.42 = $0.23 on the computer input sheet.

TABLE G-1. Consumer Price Index

Year	Index	Percent Rise each Year
1965	100	3.4
1966	103.4	3.0
1967	106.4	4.7
1968	111.5	4.7
1969	118.3	6.1
1970	124.8	5.5
1971	129.0	3.4
1972	133.4	3.4
1973	145.1	8.8
1974	162.8	12.2
1975	174.2	7.0
1976	182.6	4.8

From Reference G 3.

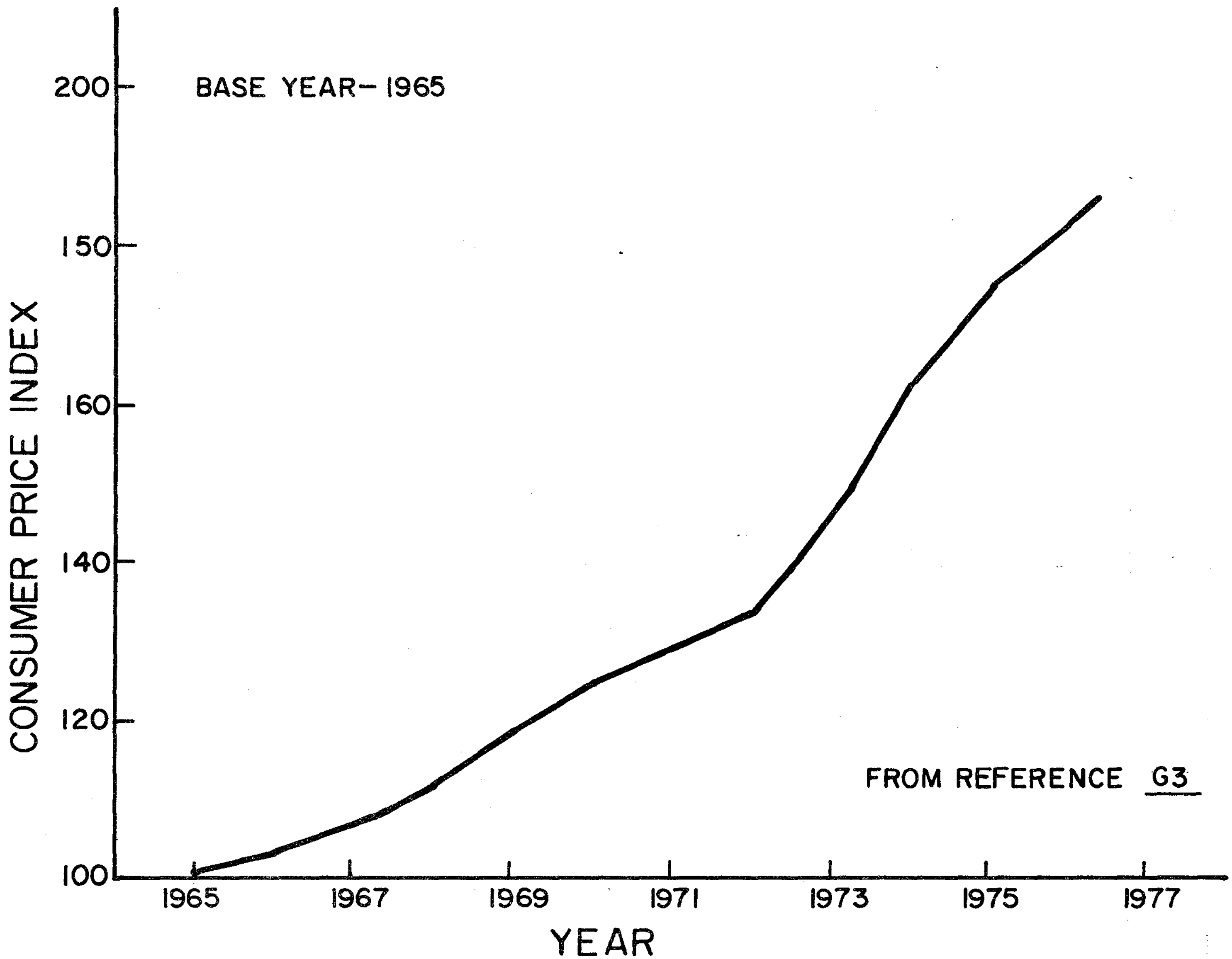


FIG. G-1. Consumer Price Index From 1965 Through 1976

References

G 1. McFarland, W. F., "Benefit Analysis for Pavement Design Systems," Research Report No. 123-13, Texas Transportation Institute, Texas A&M University, 1972.

G 2. Accident Facts, 1975

G 3. Wall Street Journal, Monday, April 4, 1977, p 28.

APPENDIX H. ENERGY REQUIREMENTS ASSOCIATED WITH HIGHWAY MAINTENANCE AND REHABILITATION

Introduction

Transportation of goods and services required 25 percent of the total 90 quadrillion (10^{15}) Btu (95000 quadrillion J) consumed in the United States in 1977 (H 1). This amount increases to 42 percent if the total amount of energy required for 1) the production of raw materials used in transportation vehicles, 2) manufacture of transportation vehicles and 3) the production of materials for construction, rehabilitation and maintenance of transportation facilities is considered.

The information included below defines the energy requirements for operations associated with highway maintenance and rehabilitation. It is estimated that the energy associated with these operations consumes about 1.5 to 2.0% of the total energy consumed in the United States (H 2). Even with this relatively small percent of total energy consumption associated with highway maintenance and rehabilitation, it is none-the-less important that the engineer optimize these operations based on energy requirements just as he presently optimizes his operation based on costs.

Energy Equivalents

A wide variety of equipment and processes are utilized to produce, transport and place materials associated with highway maintenance and rehabiliation activities. Typical equivalencies for a wide variety of fuels associated with these operations are shown in Table H-1. It should be noted that as the density of the petroleum product increases, the energy equivalent increases. Asphalt cement which has a high density

TABLE H-1. Fuel Equivalents

Fuel	Energy Equivalencies
Gasoline	125,000 Btu/gal (H 14)
Kerosene	135,000 Btu/gal (H 14)
Fuel Oil, No. 1 (API 42)	135,000 Btu/gal (H 14)
Fuel Oil, No. 2 (API 35) (diesel)	139,000 Btu/gal (H 14)
Fuel Oil, No. 3 (API 28)	143,000 Btu/gal (H 14)
Fuel Oil, No. 4 (API 20)	148,500 Btu/gal (H 14)
Fuel Oil, No. 5 (API 14)	152,000 Btu/gal (H 14)
Fuel Oil, No. 6 (API 10) (Bunker C)	154,500 Btu/gal (H 14)
Natural Gas	1,000 Btu/ft^3 (H 14)
Propane Gas	91,000 Btu/gal (H 14)
Butane Gas	100,000 Btu/gal (H 14)
Asphalt Cement	158,000 Btu/gal (H 5) 19, 045 Btu/lb.
Coal	11,670 Btu/lb (H 7)
Petroleum Coke	14,470 Btu/lb (H 7)
Lignite	6000 to 9000 Btu/lb

Metric Conversion:

1 Btu/gal = 278.7 J/l

1 Btu/ft^3 = 37.26 J/l

1 Btu/lb = 2324 J/kg

has a relatively large energy equivalent. It also should be noted that asphalt has not been considered as a fuel source but rather as a construction material in this report. Thus if asphalt cement, cutback asphalt or emulsified asphalt are materials utilized as a part of the maintenance or rehabilitation activity; their energy equivalencies as a fuel are not considered (H 14). The potential is there, however.

To aid the reader in conversion from one energy unit to another energy unit the following is offered:

1 kWh	= 3412 Btu
1 hp-hr	= 2547 Btu
1 hp	= 0.7457 kW
1 kWh	= 1.341 hp-hr
1 kW	= 1.341 hp
1 Btu	= 1055 J
1 J	= 0.000948 Btu

A British thermal unit (Btu) is the quantity of heat required to raise the temperature of one pound of water one degree Fahrenheit when water is at or near 39.2°F. A Joule is a unit of work and energy in the SI System. 4.186 Joules are required to raise one gram of water 1°C.

In actual practice, energy is lost when fuel is converted into electrical energy or into horsepower. For example, the following energy conversions are not unlikely:

1. 11,000 Btu to generate 1 kWh

2. 0.06 gal of gasoline to generate 1 brake horsepower hour (bhp-hr)*

*The brake horsepower of an engine is calculated by direct measurement by use of a dynamometer and takes into account system losses.

3. 0.04 gal of diesel fuel to generate 1 brake horsepower hour (bhp-hr)*

Thus, the burning of fuel to generate electricity is about 31 percent efficient. The burning of fuel in engines to obtain power is approximately 34 percent and 46 percent efficient for gasoline and diesel engines, respectively. Additionally, since power equipment is ordinarily not operated at full rated power for a prolonged period of time, adjustments of the order of 67 percent and 75 percent of rated power (relative to continious operation) are normally made for stationary and power vehicles respectively (H 14).

Energy Requirements for Highway Maintenance

Energy requirements for equipment associated with maintenance and rehabilitation, manufacture of materials, production of mixtures, construction operations and individual maintenance and rehabilitation operations are included below.

Equipment. Energy requirements for various types of vehicles and equipment associated with maintenance and rehabilitation are shown on Table H-2 and Table H-3. Table H-2 gives energy requirements for automobiles and trucks while Table H-3 includes various maintenance equipment. Appropriate references are included.

Production and Manufacture. Energy requirements for the manufacture of asphalt products, portland cement, steel and lime are shown on Table H-4. Energy Associated with operations involving the production of aggregates, asphalt concrete and portland cement concrete are shown in Tables H-5, H-6 and H-7 respectively. In some cases different values have been

TABLE H-2. Energy Requirements for Automobile and Truck Operation

Type of Vehicle	Energy Requirements			Ref
	Btu/mi	Btu/hr	Btu/ton mi	
Automobile	7,230			(H 23)
Stationwagon	7,760			(H 23)
Pickup	11,400			(H 24)
Maintenance Trucks - Diesel	26,700	97,300		(H 23)
Maintenance Trucks - Gasoline	26,600	100,000		(H 23)
Maintenance Trucks - 1 ton	15,600			(H 24)
Maintenance Trucks - 2 Axle	27,500			(H 24)
Distributor Truck - Gasoline	31,300			(H 24)
Truck Tractor - Diesel	30,400			(H 24)
Truck - 2 Axle, 6 Tire, Gasoline			11,000	(H 14)
Truck - 3 Axle, Gasoline			4,270	(H 14)
Truck - 3 Axle, Diesel			3,800	(H 14)
Truck - 3 Axle (combination) Gasoline			7,440	(H 14)
Truck - 3 Axle (combination) Diesel			5,840	(H 14)
Truck - 4 Axle (combination) Gasoline			5,040	(H 14)
Truck - 4 Axle (combination) Diesel			3,270	(H 14)
Truck - 5 Axle (combination) Gasoline			2,900	(H 14)
Truck - 5 Axle (combination) Diesel			1,960	(H 14)

Metric Conversion:

1 Btu/mi = 656.1 J/km
1 Btu/hr = 1055 J/hr
1 Btu/ton mi = 0.723 J/kg km

TABLE H-3. Energy Requirements for Miscellaneous Maintenance and Rehabilitation Equipment

Type of Vehicles	Energy Requirement	
	Btu/hr	Ref
Front End Loader - 2 cu yd Diesel	6,950	(H 23)
Front End Loader - 1.5 cu yd Gasoline	5,000	(H 23)
Loader for Aggregates	875,000	(H 14)
Front End Loader, Diesel	222,000	(H 24)
Motor Grader - 23,000 lb Diesel	6,950	(H 23)
Grader, Diesel	375,000	(H 24)
Rollers	625,000	(H 14)
Roller	111,000	(H 24)
Striping Machine, Self Contained	125,000	(H 24)
Hand Striping Machine	62,500	(H 24)
Mower, Roadside	125,000	(H 24)
Mower, Landscape	46,800	(H 24)
Tractor, Farm Type	375,000	(H 24)
Spreader, Self Propelled	338,000	(H 24)
Broom, Mechanical	125,000	(H 24)
Dozer, Track Type	417,000	(H 24)
Crushing/Screening Plant	695,000	(H 24)
Asphalt Paver	626,000	(H 14)

Metric Conversion:
1 Btu/mi = 656.1 J/km
1 Btu/hr = 1055 J/hr
1 cu yd = 0.765 m^3
1 lb = 0.454 kg

TABLE H-4. Energy Associated with Manufacturing

Item	Energy Requirements			Ref
	Btu/gal	Btu/lb	Btu/ton	
Asphalt Cement	2,500	300	600,000	(H 14)
Emulsified Asphalt	2,000	240	480,000*	(H 14)
Cutback Asphalt	2,500	300	600,000**	(H 14)
Portland Cement		3,750	7,500,000	(H 5), (H 14)
Steel, for tiebars, re-bars		10,500	210,000,000	(H 14)
Lime		3,000	6,000,000	(H 14)

*For equal quantities of binder this is equivalent to 740,000 Btu/ton. Assumes 65 percent residual asphalt.

**For equal quantities of binder this is equivalent to 750,000 Btu/ton. Assumes 80 percent residual asphalt.

Metric Conversion:
1 Btu/gal = 278.7 J/l
1 Btu/lb = 2324 J/kg
1 Btu/ton = 1.164 J/kg

TABLE H-5. Energy Associated with Aggregate Production

Product	Operation	Energy Requirement Btu/lb	Btu/ton	Btu/yd^3*	Ref
Crushed Stone	Drilling and shooting	6	12,000	21,000	(H 14)
	Crushing	25.5	51,000	89,500	(H 14)
	Handling (cranes & bulldozers)	3.5	7,000	12,300	(H 14)
	Total	35	70,000	123,000	(H 14)
	Total	26	52,000	91,300	(H 13)
Crushed Gravel	Crushing	17.5	35,000	61,400	(H 14)
	Handling (cranes & bulldozers)	2.5	5,000	8,780	(H 14)
	Total	20	40,000	70,200	(H 14)
Natural or Uncrushed Aggregate	Total	7.5	15,000	26,300	(H 14)

*130 lbs/ft^3 assumed unit weight (2100 km/m^3)

Metric Conversion:

1 Btu/lb = 2324 J/kg

1 Btu/ton = 1.164 J/kg

1 Btu/yd^3 = 1381 J/m^3

TABLE H-6. Energy Associated with Asphalt Concrete Production*

Operation	Energy Requirements			
	Btu/ton of mix	Btu/of operation**	Equivalent gals. of diesel/hr	Equivalent gals. of diesel/ton of mix
Asphalt Heating & Storage	6,400	960,000	6.9	0.046
Loader	4,380	657,000	4.7	0.031
Cold Bins, Vibrators, Belt Feeders	100	15,000	0.1	0.001
Cold Feed Belt Conveyor	250	37,500	0.3	0.002
Cold Feed Total	4,730	710,000	5.1	0.034
Dryer Drive Motor	1,260	188,000	1.3	0.009
Dryer Fuel Pump Blower	1,460	218,000	1.5	0.010
Dryer Exhaust Fan	1,260	189,000	1.4	0.009
Dryer Secondary Dust Collector	800	120,000	0.9	0.006
Dryer Total	4,780	715,000	5.1	0.034
Mixing Plant Hot Elevator	350	53,000	0.4	0.003
Mixing Plant Screening	455	68,300	0.5	0.003
Mixing Plant Asphalt Pump	250	37,500	0.3	0.002
Mixing Plant Mineral Filler Elevator	200	30,000	0.2	0.001
Mixing Plant Pugmill	2,070	310,000	2.2	0.015
Mixing Plant Compressor (Discharge)	200	30,000	0.2	0.001
Mixing Plant Storage Conveyor	400	60,000	0.4	0.003
Mixing Plant Total	3,920	589,000	4.2	0.028
Drying and Heating Aggregate	233,000***	35,000,000	252	1.68
Plant Operation Total	253,000	38,000,000	273	1.82
Paving Machine	4,170	625,000	4.5	0.030
Rollers - 3	12,500	1,880,000	13.5	0.090
Spreading and Compaction Total	16,700	2,500,000	18.02	0.120
Drying and Heating Aggregate	278,000 (H 3)	41,700,000	300	2.00
Drying and Heating Aggregate	278,000 (H 22)	41,700,000	300	2.00
Drying and Heating Aggregate	327,000****	49,000,000	353	2.35
Plant Operation (excluding drying) Lay & Compact	41,700 (H 3)	6,260,000	45	0.300
Plant Operation (excluding drying) Lay & Compact	40,910	6,140,000	44.1	0.30

After references (H 14) except where noted.
*Operating at 67 percent rated power.
** Operating at 150 ton/hr (907 kg/hr).
***5% moisture removed and raise temperature to 300°F (148°C) for a mix which contains 94% by wt. of aggregate.
****Unpublished Illinois source stated in reference (H 14). Data from Illinois quoted in reference (H 14).

Metric Conversion: 1 Btu/ton = 1.164 J/kg; 1 gal/hr = 3.785 l/hr; 1 Btu/hr = 1055 J/hr; 1 gal/ton = 4.173 l/g

TABLE H-7. Energy Associated with Portland Cement Concrete Production

Operation	Energy Requirement			
	Btu ton ton of mix	Btu/yd^3 of mix*	Equivalent gal of diesel per ton of mix	Equivalent gal of diesel per yd^3 of mix
Loader	4380	8870	0.032	0.065
Conveyor	270	550	0.001	0.003
Mixing & Other Plant Operations	1770	3580	0.013	0.026
Total Plant Operation	6420	13,000	0.046	0.094
Placing, Consolidation & Finishing	2590	5240	0.019	0.038

* 150 lb/ft^3 (2400 kg/m^3) assumed unit weight (after reference (H 14))

Metric Conversion:

1 Btu/ton = 1.164 J/kg

1 Btu/yd^3 = 1.381 J/m^3

1 gal/ton = 0.00417 l/kg

1 gal/yd^3 = 4.951 l/m^3

reported and thus the different values are given in the Table. Requirements for miscellaneous construction operations are shown in Table H-8.

Maintenance and Rehabilitation Activities. Energy requirements associated with the performance of routine maintenance and rehabilitation activities is shown in Table H-9. The specific activity for which energy data have been calculated are:

1. Fog Seal - Partial Width
2. Fog Seal - Full Width
3. Chip Seal - Partial Width
4. Chip Seal - Full Width
5. Surface Patch - Hand Method
6. Surface Patch - Machine Method
7. Digout and Repair - Hand Method
8. Digout and Repair - Machine Method
9. Crack Pouring
10. Slurry Seal
11. Asphalt Concrete Overlay.

Energy required for material manufacture, material transportation, mixture production, mixture transportation, mixture placement, and compaction are included in the data reported in Table H-9. Assumptions as to the percent of the pavement area treated with the particular maintenance activity and the thickness or quantity of material applied are identical to those used for estimating maintenance costs. These data were developed based primarily on information obtained from the Arizona Department of Transportation (H 24).

Energy consumption for materials utilized in pavements (in-place) are given on Table H-10. Materials included in the table are asphalt con-

TABLE H-8. Energy Requirements for Miscellaneous Construction Operations

Operation	Energy Requirement				
	Btu/gal	Btu/ton	Btu/yd^3	Equivalent gallons of diesel per	
				ton	yd^3
Spreading and Compaction Granular and Stabilized bases		17,000	30,980	0.122	0.223
Travel Plant Mixing in Windrow		3,000	5,470	0.022	0.039
Blade Mixing		7,820	14,250	0.056	0.103
Central Plant Mixing of Stabilized Base		6,890	12,550	0.050	0.090
Excavation - Earth		39,800	59,100(16)	0.286	
Excavation - Rock		35,500	76,700(16)		
Excavation - Other		39,100	68,700(16)		
Asphalt Distribution, Asphalt Cement	590				
Asphalt Distribution, Cutback Asphalt	445				
Asphalt Distribution, Emulsified Asphalt	145				
Aggregate Spreading for Seal Coats	9.4**				
Rolling Cold Asphalt Mixes	120***				

*135 lb/ft^3 (2160 kg/m^3) assumed unit weight except for excavation items

**9.4 Btu/yd^2

***120 Btu/yd^2 in.

After Reference (H 14) except where noted

Metric Conversion:

1 Btu/gal = 278.7 J/1

1 Btu/ton = 1.164 J/kg

1 Btu/yd^3 = 1.381 J/m^3

1 ton = 907 kg

1 yd^3 = 0.764 J m^3

1 in = 2.54 cm

TABLE H-9. Representative Energy Requirements for Maintenance and Rehabilitation Activities

Maintenance Activity	Energy Requirements: Energy/unit	Btu/yd^3 of area treated	Btu/yd^2 in.	Btu/lane mi*	Btu/yd^2*	Percent of total Pavement area treated & other Assumptions
Fog Seal- Partial Width	10,500 Btu/gal (H 24)	1,050 (H 24)	-	3,700,000 (H 24)	525 (H 24)	50 percent
Fog Seal - Full Width	6,850,000 Btu/lane mi (H 24) 3,300,000 Btu/lane mi (H 14)	970 (H 24) 470 (H 14)	-	6,850,000 (H 24) 3,300,000 (H 14)	970 (H 24) 470 (H 14)	100 percent
Chip Seal - Partial Width	537,000 Btu/yd^3 (H 24)	4,480 (H 24)	-	4,700,000 (H 24)	670 (H 24)	15 percent
Chip Seal - Full Width	14,400,000 Btu/lane mi (H 24) 27,800, 000 Btu/lane mi (H 14)	2,050 (H 24) 3,950 (H 14)	-	14,400,000 (H 24) 27,800,000 (H 14)	2,050 (H 24) 3,950 (H 14)	100 percent
Surface Patch - Hand Method	Data Not Available					2.5 percent 1 in. thick
Surface Patch - Machine Method	1,070,000 Btu/yd^3 (H 24)	29,800 (H 24)	29,800 (H 24)	21,000,000 (H 24)	2,990 (H 24)	10 percent 1 in. thick
Digout & Repair Hand Method	1,600,000 Btu/yd^3 (H 24)	178,000 (H 24)	44,460 (H 24)	25,000,000 (H 24)	3,560 (H 24)	2 percent 4 in. thick
Digout & Repair Machine Method	1,120,000 Btu/yd^3 (H 24)	187,000 (H 24)	31,200 (H 24)	65,800,000 (H 24)	9,350 (H 24)	5 percent 6 in. thick
Crack Pouring	32,400 Btu/lane mi (H 19) 33,500 (H 25)Btu/gal (H 25)	- -	- -	8,500,000 (H 24) 3,900,000 (H 25)	1,220 (H 24) 560 (H 25)	250 lin. ft per station
Slurry Seal	9,400,000 Btu/lane mi (H 19)	1,340 (H 19)	-	9,400,000 (H 19)	1,340 (H 19)	100 percent
Asphalt Concrete Overlay	512,000 Btu/ton (H 14) 533,000 Btu/ton (H 16)	55,600 (H 14) 57,800 (H 16)	27,800 (H 14) 28,900 (H 16)	391,000,000 (H 14) 407,000,000 (H 16)	55,600 (H 14) 57,800 (H 16)	100 percent 2 in. thick

* Energy requirements for yd^2 of total pavement surface maintained. For example, surface patching by the hand method may have been applied over only 5 percent to total pavement surface area, yet energy reported is for the pavement area maintained on one lane mi of pavement.

(H 25) Indicates Reference on which data is based.

Metric Conversion:

1 Btu/gal = 278.7 J/1   1 Btu/mi = 656.1 J/km   1 Btu/ yd^3 = 1.381 J/m^3

1 Btu/ton = 1.164 J/kg   1 Btu/yd^2 = 1263 J/m^2 1 Btu/yd^{2}in = 497 J/m^{2}cm

1 in. = 2.54 cm   1 ft = .305 m

TABLE H-10. Energy Consumption for Pavement Materials In-Place*

Material	Energy Requirement Btu/ton	Btu/yd^3	Btu/yd^2-in.	Ref.
	512,000	1,000,000	27,800	(H 14)
Asphalt Concrete	533,000	1,040,000	29,000	(H 16)
PCC-Jointed Non-Reinforced	1,210,000	2,450,000	68,000	(H 14)
PCC-Jointed Reinforced	1,390,000	2,820,000	78,400	(H 14)
PCC-Continuously Reinforced	1,620,000	3,280,000	91,110	(H 14)
Slurry Seal			1,340**	(H 19)
Chip Seal-Emulsion & Crushed Stone			3,950**	(H 14)
Fog Seal			470**	(H 14)
Crushed Stone Base	236,000	414,000	11,500	(H 14)
	218,000	382,000	10,600	(H 13)
Emulsified Asphalt Base	87,400	562,000	15,600	(H 14)

*Includes energy associated with manufacturing, mixing, hauling, placing and compacting.

**These treatments are not 1 in. in thickness.

Metric Conversion:

1 Btu/ton = 1.164 J/kg

1 Btu/yd^3 = 1.381 J/m^3

1 Btu/yd^{2}in = 497 J/m^{2}cm

1 in. = 2.54 cm

crete, portland cement concrete, slurry seal, chip seal, fog seal, crushed stone base and emulsified asphalt base. The energy consumed includes the energy associated with manufacturing, mixing, hauling, placing, and compacting.

A summary of the data presented on Tables H-9 and H-10 is shown in Table H-11 together with energy requirements per dollar (December 1975) for ten maintenance and rehabilitation activities. If one assumes that the Federal Highway Administration estimate for annual maintenance of highway and roadways is correct (5 billion dollars for 3,800,000 mi (6,100,000 km) of road and furthermore, if it is assumed that on the average 20,000 Btu (21,000,000 J) (See Table H-11) of energy are required for each dollar expended on maintenance, it can be concluded that about 0.1 percent of the total energy consumed in the United States is utilized in highway maintenance operations. This 0.1 percent of the total energy represents 100,000,000,000,000 Btu per year (1.06×10^{17}J) or approximately 15,800,000 bbl of oil per year. The reader is reminded that this neglects the approximately 140,000,000 bbls of asphalt consumed each year as a pavement ingredient.

TABLE H-11. Representative Costs and Energy Requirements for Maintenance and Rehabilitation Activities

Maintenance Activity	Costs Dollars * Per		Energy Requirements*, Btu		Energy required per unit costs, Btu/dollar	Percent of total pavement area treated
	$Yd.^2$	Lane Mi	$Yd.^2$	Lane Mi		
Fog Seal - Partial Width	0.045	320	525	3,700,000	11,700	50 percent
Fog Seal - Full Width	0.06	420	970	6,850,000	16,200	100 percent
Chip Seal - Partial Width	0.06	420	670	4,700,000	11,200	15 percent
Chip Seal - Full Width	0.21	1500	2050	14,400,000	9,800	100 percent
Surface Patch - Hand Method	0.10	700	Data Not Available			2.5 percent 1 in. thick
Surface Patch - Machine Method	0.08	560	2990	21,000,000	37,400	10 percent 1 in. thick
Digout & Repair - Hand Method	0.25	1760	3560	25,000,000	14,200	2 percent 4 in. thick
Digout & Repair - Machine Method	0.20	1400	9350	65,800,000	46,700	5 percent 6 in. thick
Crack Pouring	0.12	850	1220	8,500,000	10,200	250 Lin. ft Per Station
Asphalt Concrete Overlay	1.90	13,400	55,600	391,000,000	29,300	100 percent 2 in. thick

* Costs and energy are for yd^2 of total pavement surface maintained. For example, surface patching by the hand method may have been applied over only 5 percent of total pavement surface area, yet costs and energy reported are for the total pavement area maintained or one lane mi of pavement.

Metric Conversion:
1 yd^2 = 0.836 m^2 1 mi = 1.609 km 1 Btu/yd^2 = 1263 J/m^2 1 Btu/mi = 676.1 J/km
1 Btu = 1056 J 1 in. = 2.54 cm

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APPENDIX I. USER'S INPUT GUIDE

The input for the UDAREM decision analysis computer program (See Ch. 2) is divided into two distinct divisions; utility curve and probability density function (PDF) inputs. The utility curve inputs require a set of points describing each of the seventeen curves. The PDF input, in the form of optimistic, most probable, and pessimistic estimates (O, MP, P), is needed for each decision criterion. There must be an equal number of PDF inputs as there are utility curves. Most of these inputs are relatively simple, but a few require additional information and calculations in order to arrive at the O, MP, and P values.

The computer program presently requires that the input have English units but, with minor changes it will become compatible with SI units. A list of conversions that would be needed for this purpose are:

1 British Thermal Unit (Btu) = 1.055×10^3 joule (J)
1 mile (mi) (U. S. Statute) = 1.609×10^3 metre (m)
1 pound-mass (16m) = 4.535×10^{-1} kilogram (kg)
1 yard^2 (yd^2) = 8.361×10^{-1} metre^2 (m^2)

A sample data collection form is shown on pages I-2 through I-9. Following the form are guide instructions for the user to fill out this form.

REHAB TECHNIQUE ______________________________ CLASS. # ______

TYPE OF ROAD (URBAN, SUBURBAN, OR RURAL) ______________________

PRIMARY DISTRESS APPLICATION ______________________________

A. COST

1. DEVELOPMENT COST (TOTAL $) ________ ________ ________
2. CAPITAL EQUIPMENT COST (TOTAL $) ________ ________ ________
3. CONSTRUCTION COST ($/LANE MILE) ________ ________ ________
4. MAINTENANCE COST ($/LANE MILE) ________ ________ ________
5. USER SAVINGS

Interest Rate (%) ________
(Default) (7.0)

	S.I.			
Serviceability Index -	t	0	0	0
Time Relation	S.I.			
(With Rehab.)	t			
(Time in Yrs)	S.I.			
	t			

a. PRESENT WORTH ACCUMULATED USER COST

METHOD # 1

	S.I.			
Serviceability Index -	t			
Time Relation	S.I.			
(Without Rehab.)	t			
(Time in Yrs)	S.I.			
	t			

	S.I.			
	UC			
User Cost-	S.I.			
	UC			
Serviceability Index	S.I.			
	UC			

		O	MP	P
Average Daily Traffic-	ADT	(3000)	(5000)	(6500)
Time Relation	t	(0)	(0)	(0)
(one way - 24 hr Traffic) (Defaults)	ADT	(4500)	(9000)	(13,000)
	t	(10)	(10)	(10)

METHOD # 2

User Cost (w/o Rehab.)	______	______	______
User Cost (w/ Rehab.)	______	______	______

b. USER COSTS DURING CONSTRUCTION

Percent Cars (%)		______	
Percent Trucks (%)		______	

1. Speed Change Cost

Number of Speed Changes (Default)	(2)		
Basic Speed Change Cost ($/1000 Speed Changes/VEH) (Default)		(10)	
Truck Speed Change Cost Multiplier (Default)			(9)

2. Maneuvering Costs

Percent of Detour Length in Curves (Default)	(10)		
Basic Maneuvering Cost ($/1000 VEH-MILES) (Default)		(22)	
Truck Maneuvering Cost Multiplier (Default)			(6)

3. Roughness Costs

Serviceability Index of Detour (Default)	(3.0)		
Detour Length/Rehab Length Ratio (Default)	(1.0)		

	O	MP	P
4. Present Worth Additional Accident Cost			
Rehabilitation Section Length (mi) (Default)		(1)	
Normal Speed (mph) (Default)		(55)	
Restricted Speed (mph) (Default)		(45)	
# of Lanes Open, One Way, Normal Operation, (1-4) (Default)		(2)	
# of Lanes Open, One Way, During Rehab., (1-4) (Default)		(1)	
# of Lanes Open, One Way, for Detour, (1-3) (Default)		(1)	
Average Cost of Accident ($/Accident) (Default)		$1,500	
B. EXPECTED PERFORMANCE			
6. TIME OF DEVELOPMENT (yr)	______	______	______
7. LIFE (yr)	______	______	______
8. LEVEL OF PAVEMENT SERVICE			
Type of Pavement; Enter One Number: Rigid - 1 or Flexible - 2		______	
a. SERVICEABILITY INDEX - TIME RELATION (WITH REHAB) HAS BEEN INPUTTED IN USER SAVINGS (Decision Criteria No. 5).			
Minimum Value (Default)		(0)	
Maximum Value (Default)		(5)	
Acceptable Life (Default)		(20/10)	

		O	MP	P
b.	SN_{40}	(74/74)	(56/57)	(40/40)
	t	0 (0/0)	0 (0/0)	0 (0/0)
	SN_{40}	(52/54)	(38/42)	(23/22)
Skid Number-Time Relation	t	(5/5)	(5/5)	(5/5)
(Default Rigid/Flexible)	SN_{40}	(48/48)	(36/37)	(18/15)
	t	(10/10)	(10/10)	(10/10)
Minimum Value (Default)			(10)	
Maximum Value (Default)			(80)	
Acceptable Life (Default)			(20/10)	
	t	0	0	0
c.				
	t			
	t			
Minimum Value				
Maximum Value				
Acceptable Life (Default Rigid/Flexible)			(20/10)	
	t	0	0	0
d.				
	t			
	t			

		O	MP	P
Minimum Value			______	
Maximum Value			______	
Acceptable Life (Default Rigid/Flexible)			(20/10)	
	t	0	0	0
e. ____________________	t	______	______	______
____________________	t	______	______	______
Minimum Value			______	
Maximum Value			______	
Acceptable (Default Rigid/Flexible)			(20/10)	
	t	0	0	0
f. ____________________	t	______	______	______
____________________	t	______	______	______
Minimum Value			______	
Maximum Value			______	
Acceptable Life (Default Rigid/Flexible)			(20/10)	
	t	0	0	0
g. ____________________	t	______	______	______
____________________	t	______	______	______

		O	MP	P
Minimum Value			____	
Maximum Value			____	
Acceptable Life (Default Rigid/Flexible)			(20/10)	

		O	MP	P
h. ____	t	0	0	0
____	t	____	____	____
	t	____	____	____
Minimum Value			____	
Maximum Value			____	
Acceptable Life (Default Rigid/Flexible)			(20/10)	

		O	MP	P
i. ____	t	0	0	0
____	t	____	____	____
	t	____	____	____
Minimum Value			____	
Maximum Value			____	
Acceptable Life (Default Rigid/Flexible)			(20/10)	

		O	MP	P
		______	______	______
	t	0	0	0
j. ____________________		______	______	______
____________________	t	______	______	______
		______	______	______
	t	______	______	______

Minimum Value ______

Maximum Value ______

Acceptable Life
(Default Rigid/Flexible) (20/10)

Weights

a. Serviceability Index ______
b. Skid Number ______
c. ______
d. ______
e. ______
f. ______
g. ______
h. ______
i. ______
j. ______

1.0 (Total)

9. TRAFFIC VOLUME-CONSTRUCTION TIME

Urban-1, Suburban-2, or Rural-3 ______
(Enter one number)

Construction Time (Months) ______ ______ ______

Volume-to-Capacity Ratio (V/C) ______

10. EXPECTED REWORKABILITY ______ ______ ______

C. ENERGY

11. USER (ALL REQUIRED DATA PREVIOUSLY RECORDED IN USER SAVINGS, DECISION CRITERION NO. 5)

	O	MP	P
12. REHABILITATION (Btu/sy)	________	________	________
13. MATERIAL (Btu/lb)		________	
Application Rate (lb/sy)	________	________	________
D. IMPACT			
14. SAFETY IMPROVEMENT	________	________	________
15. SAFETY (DURING REHAB.)	________	________	________
16. NOISE (DURING REHAB.)	________	________	________
17. POLLUTION (DURING REHAB.)	________	________	________

A. COST ATTRIBUTE

ALL COSTS SHOULD BE UPDATED TO REFLECT CURRENT COSTS.

1. Development Cost

The total cost incurred from the inception of the idea until the time the technique has been proven applicable.

Such costs include:

a. Feasibility study
b. Engineering, research and development
c. Legal fees including patents, incorporation, etc.
d. Prototype construction and testing
e. Marketing the technique
f. Tooling up for production

INPUT (Total $) ______ O ______ MP ______ P

2. Capital Equipment Cost

The total cost incurred in acquiring all "special" equipment necessary for implementing the technique, and all costs associated with marketing.

Such costs include:

a. Equipment cost, including purchase, transportation, and special handling
b. Equipment storage, insurance safety and security
c. Marketing and management costs

INPUT (Total $) ______ O ______ MP ______ P

3. Construction Cost

These are the total costs per lane mile incurred in applying the technique. Restorative rehabilitation will generally cost more than preventative rehabilitation.

Such costs include:

a. Material

b. Labor

c. Equipment ownership or rental (excluding special equipment)

d. All equipment operating costs including fuel and maintenance

e. Overhead, contingencies, and profit

INPUT ($/lane mile) ________ O ________ MP ________ P

4. Maintenance Cost

The annual cost per lane mile of maintaining the rehabilitated pavement for the life of the technique after the technique has been applied.

INPUT ($/lane mile) ________ O ________ MP ________ P

5. User Savings

The difference in present value of annual costs per lane mile incurred by the user if the pavement was not rehabilitated and if the pavement was rehabilitated. Costs encountered if there were no rehabilitation would include (a) vehicle ownership and operating costs, (b) time delay due to roughness, (c) and accident costs. If the rehabilitation technique was used the costs would include (a) time delay around detours during construction, (b) vehicle ownership and operating costs (presumably less than if no rehabilitation were performed), and (c) accident costs. User SAVINGS consists of the discounted present worth of accumulated user costs for the life of the rehabilitation and user costs during construction which are then analyzed over the life of the rehabilitation. Figure I-1 is provided to show the distribution of input which is required for this decision criteria. The initial input is an interest rate which is used to calculate discounted present worth and annual costs. This interest rate has a default value of

7.0% that will be used unless a different rate is recorded on the input form.

INPUT (%) Interest Rate ________

The Serviceability Index - Time Relation (With Rehabilitation) is the next required input.

This relation will begin at time equal to zero; the time at which the rehabilitation technique is installed.

INPUT

SERVICEABILITY INDEX - TIME RELATION*
(WITH REHABILITATION)

SERVICEABILITY INDEX (SI)	___	___	___
TIME (t=o)	___	___	___
SERVICEABILITY INDEX (SI)	___	___	___
TIME (YR)	___	___	___
SERVICEABILITY INDEX (SI)	___	___	___
TIME (YR)	___	___	___
	0	MP	P

Serviceability Indexes and their associated years are to be selected to represent the probable changes in SI with time. The interest rate and this SI matrix are purposely shown as input data prior to user cost input because they are used later as input for User Energy Savings (Decision Criterion No. 11).

a. Present Worth Accumulated User Cost

This cost, being one of five costs which make up User Savings, can utilize either of two methods for PDF input.

METHOD 1

Method 1 requires the selection of a Serviceability Index - Time Relation (Without Rehabilitation) which will begin at time equal to zero. Time zero denotes the time at which the rehabilitation technique would have been installed.

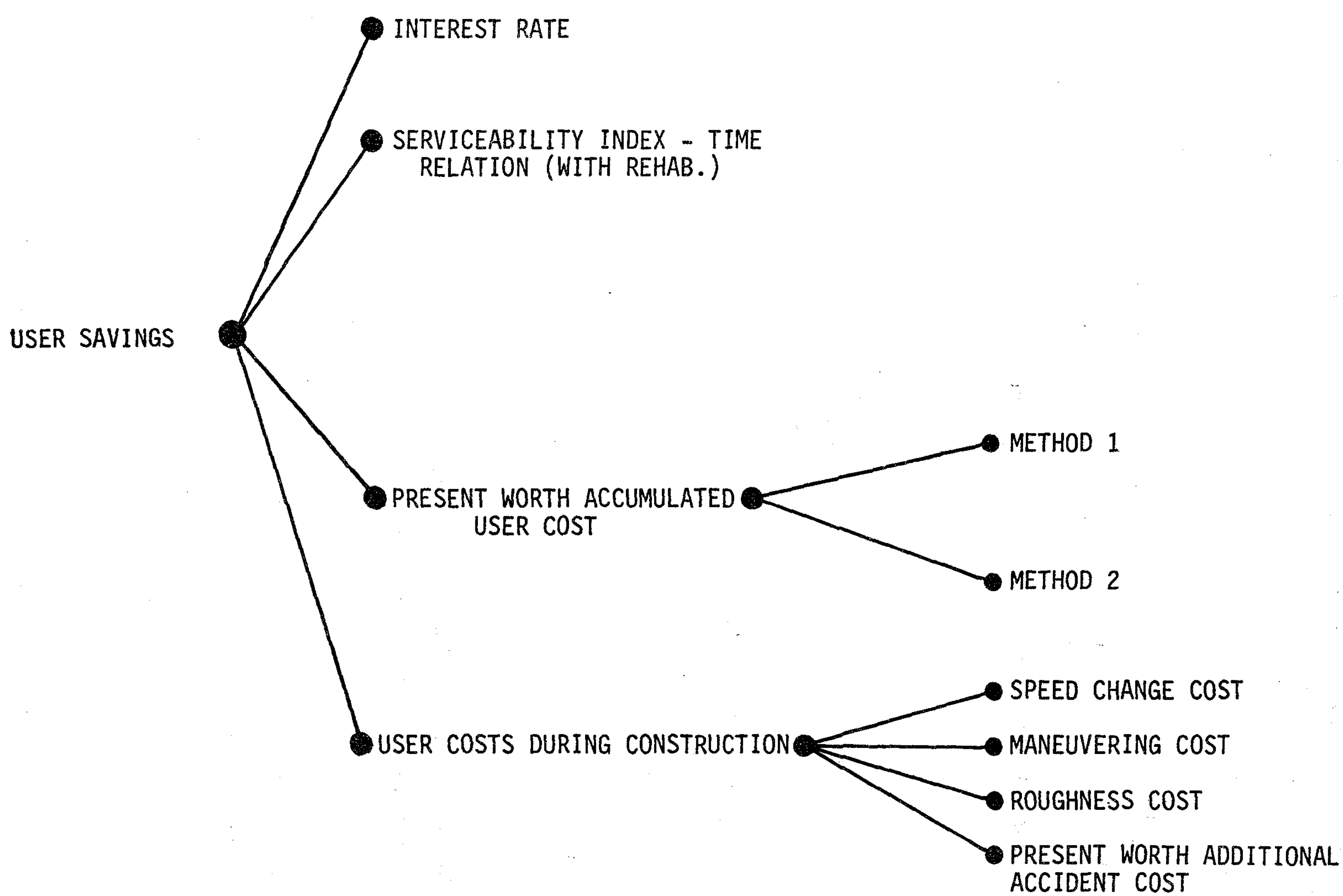


Figure 1-1. Branch Diagram of User Savings Input.

SERVICEABILITY INDEX - TIME RELATION*
(WITHOUT REHABILITATION)

SERVICEABILITY INDEX (SI)	___	___	___
TIME (t=o)	___	___	___
SERVICEABILITY INDEX (SI)	___	___	___
TIME (YR)	___	___	___
SERVICEABILITY INDEX (SI)	___	___	___
TIME (YR)	___	___	___
	0	MP	P

Serviceability Indexes and their associated years are to be selected to represent the probable changes in SI with time.

A relationship between user cost and serviceability index is needed in order to facilitate relating user cost with time. Two alternatives are provided here for the purpose of obtaining user costs. It should be noted here that both alternatives utilize the data provided by McFarland (I-1). The main difference between the two is the manner in which the basic data are handled.

The curves that apply to the first alternative are shown in Fig. I-2 for urban and suburban conditions and Fig. I-3 for rural conditions. These curves offer the capability of obtaining O, MP, and P estimates by a change in volume-to-capacity ratio (v/c).

Figures I-4, I-5, and I-6 apply to the second alternative method in determining user cost as a function of serviceability index. A graph is provided for each type of road - urban, suburban and rural - and on each graph an O, MP and P curve is provided. After the type of road is determined, the graph which corresponds to that type road is used. The graph is entered with serviceability index and the O, MP and P user cost estimates can be read for each curve.

INPUT

USER COST - SERVICEABILITY INDEX RELATION*

SERVICEABILITY INDEX (SI)	___	___	___
USER COST ($/veh-mi)	___	___	___
SERVICEABILITY INDEX (SI)	___	___	___
USER COST ($/veh-mi)	___	___	___

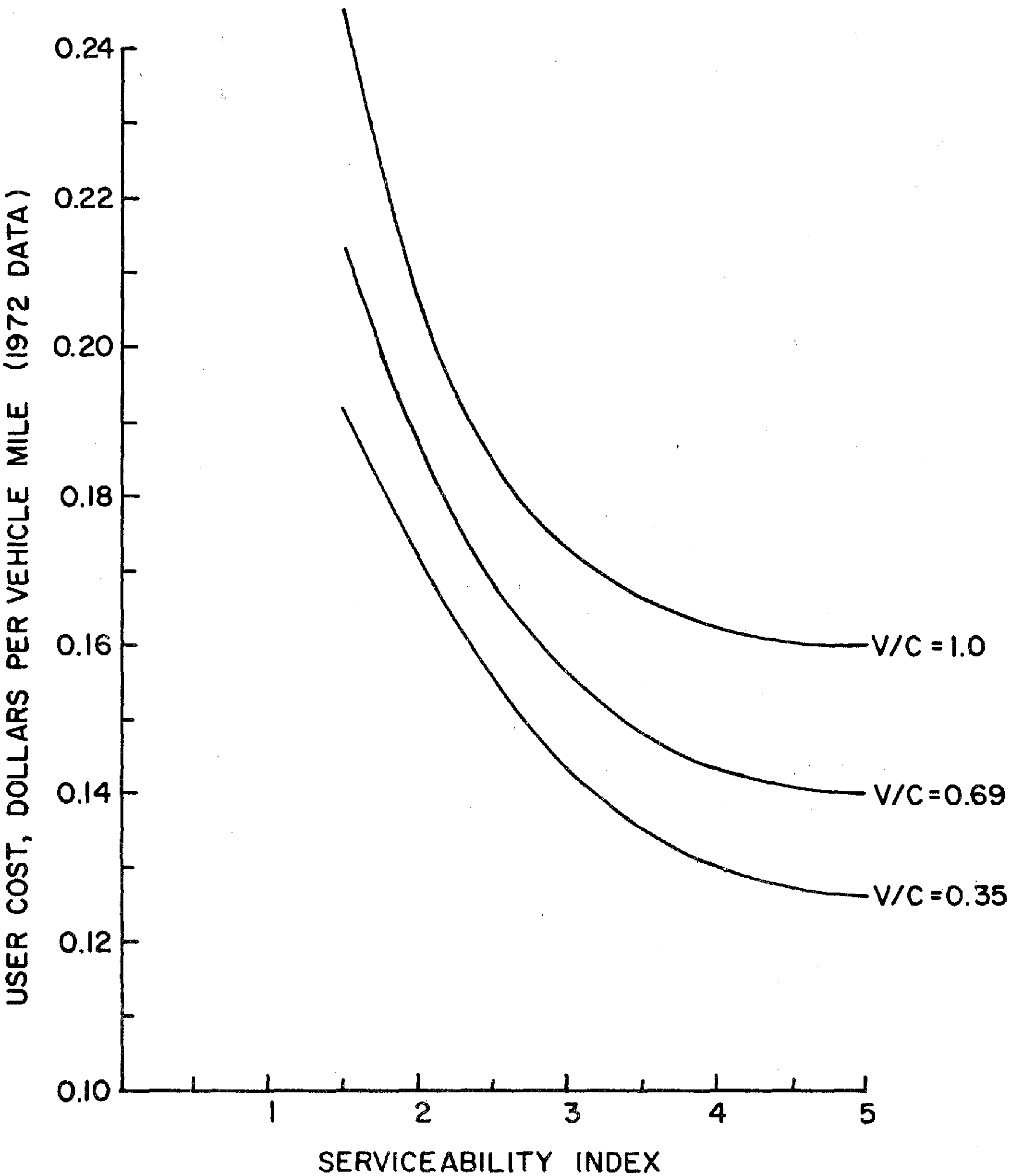


Figure I-2. Variation in Total User Costs With Serviceability Index for Divided Highways in Urban and Suburban Locations.

From Reference I-1

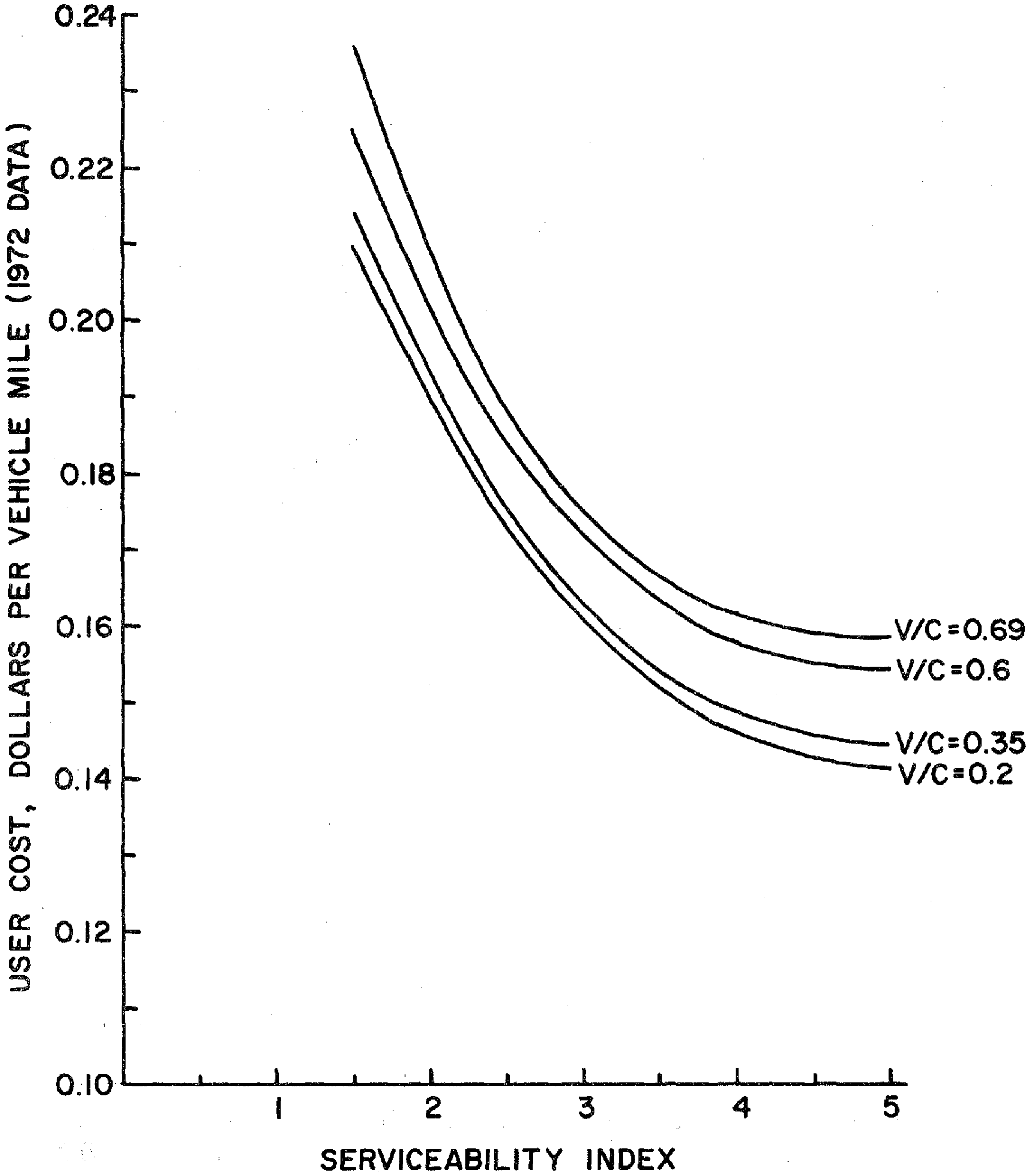


Figure I-3. Variation in Total User Costs With Serviceability Index for Divided Highways in Rural Areas.

From Reference I-1

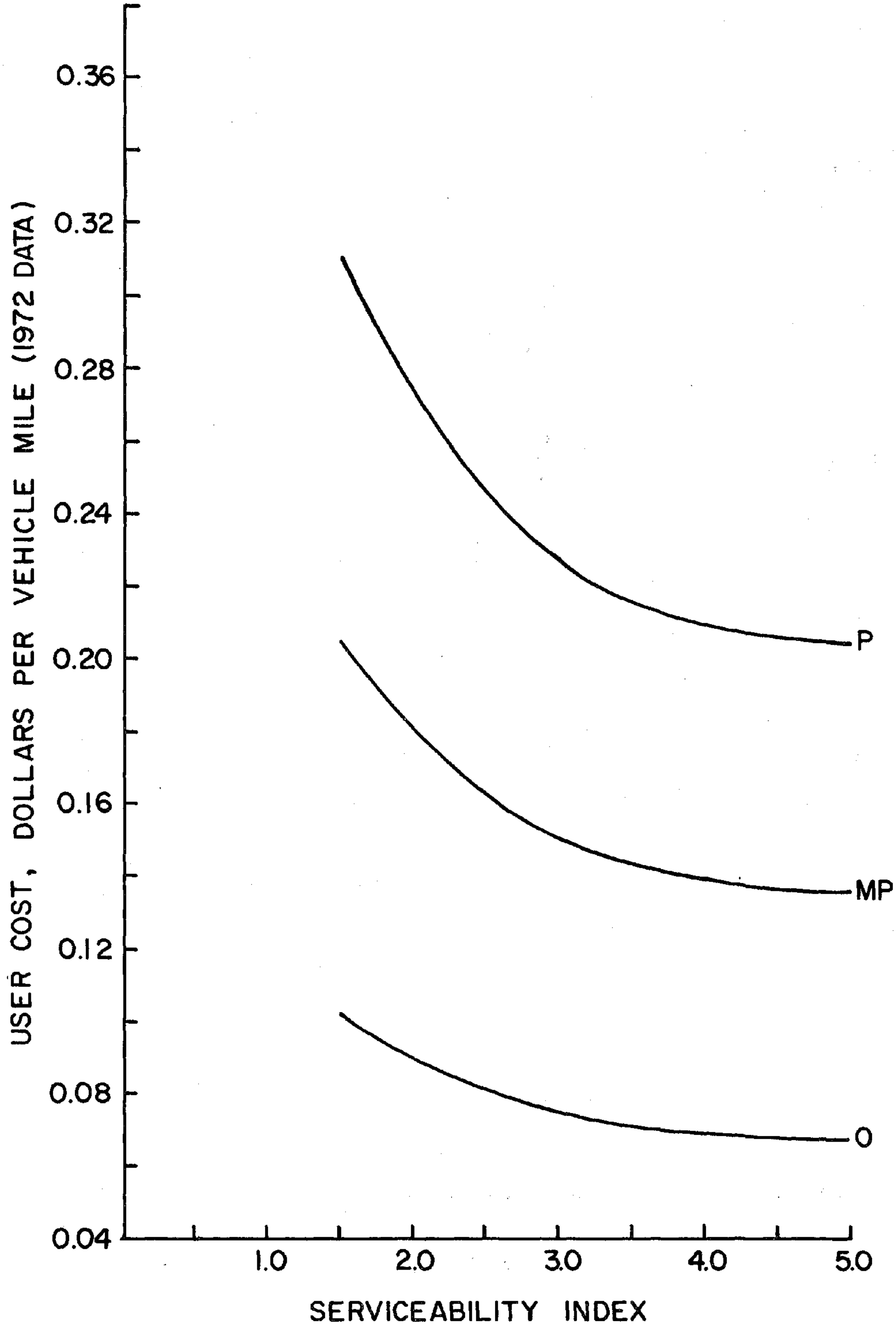


Figure I-4. User Costs Versus Serviceability Index for Urban Conditions.

From Reference I-1

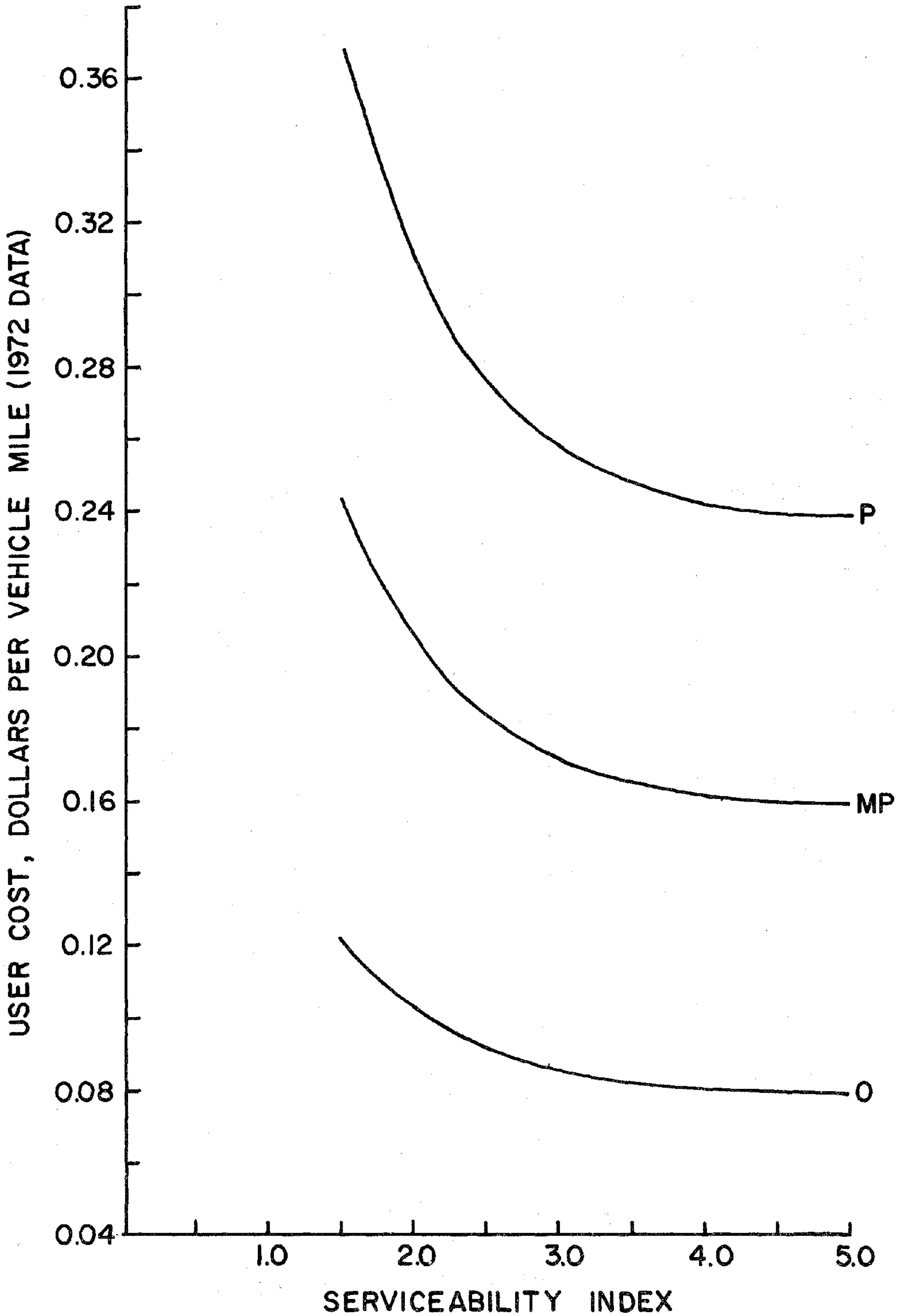


Figure I-5. User Cost Versus Serviceability Index for Suburban Conditions.
From Reference I-1

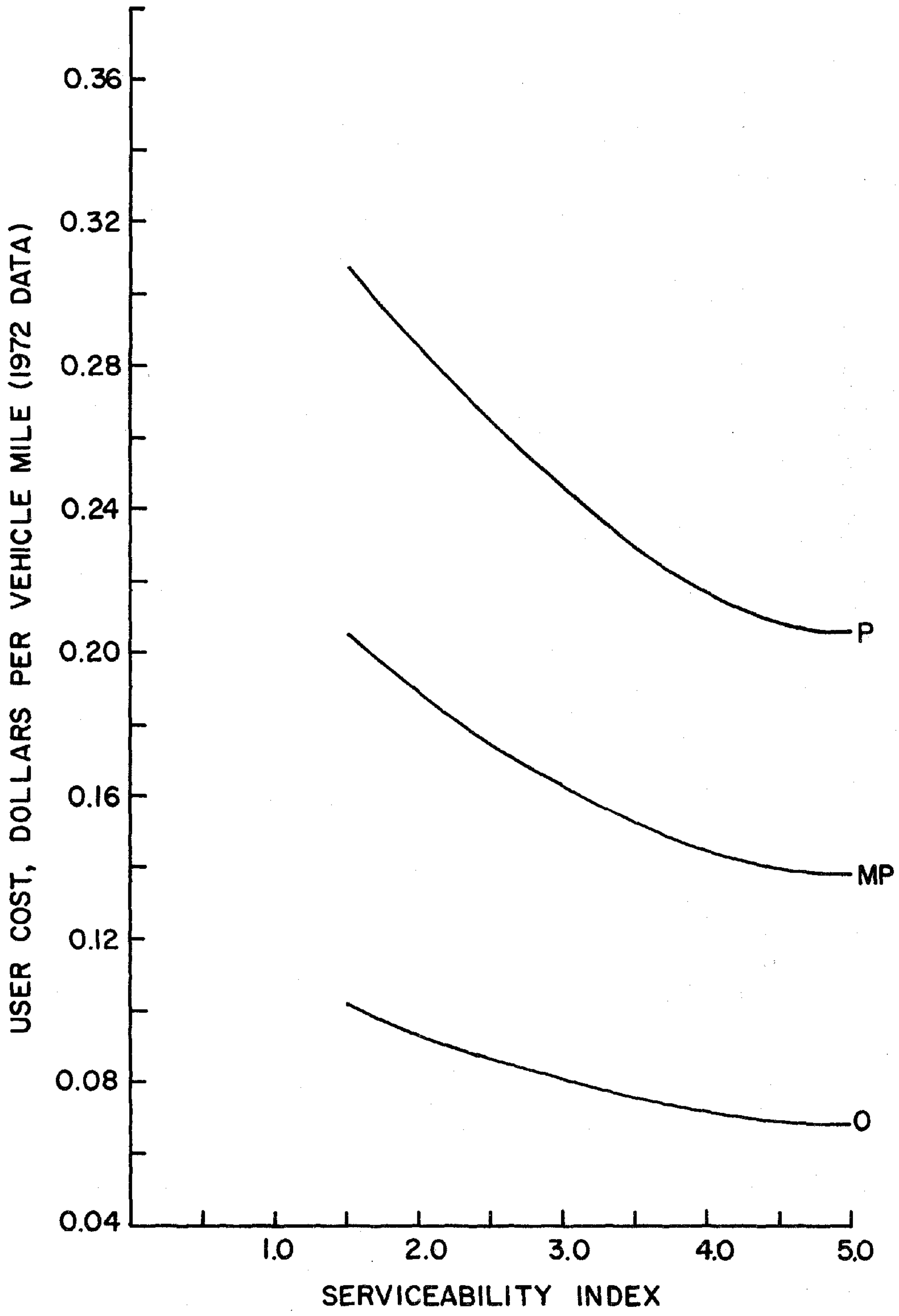


Figure I-6. User Costs Versus Serviceability Index for Rural Conditions.

From Reference I-1

SERVICEABILITY INDEX (SI)	______	______	______
USER COST ($/veh-mi)	______	______	______
	O	MP	P

*NOTE: It should be kept in mind that these costs are related to the expected performance of the rehabilitation technique. User costs and their corresponding Serviceability Indexes are to be selected to describe the change in user costs with change in SI.

Average daily traffic (ADT) is defined as the amount of one-way vehicular traffic per lane for a 24-hour period. Only two points are required of the O, MP and P Average Daily Traffic - Time Relation curves. It is not compulsory that the time in years begin at time equal to zero; the values may be any reasonable estimates of time. Default values are supplied and shown on the input form.

INPUT

	ADT	______ (3000)	______ (5000)	______ (6500)
AVERAGE DAILY TRAFFIC - TIME RELATION (veh/day/lane)	t	______ (0)	______ (0)	______ (0)
(one-way-24 hr. traffic)	ADT	______ (4500)	______ (9000)	______ (13,000)
(DEFAULTS)	t	______ (10)	______ (10)	______ (10)
		O	MP	P

METHOD 2

A second method of inputting user costs is provided for flexibility. This second method allows one to input different values of total user costs as equivalent uniform annual costs (EUAC)---for the life of the pavement (without rehab.) and life of the technique (with rehab.). In determining these costs an interest rate of 7.0% should be used.

INPUT

User Cost (w/o Rehab.) ($/YR) ________ ________ ________

User Cost (w/Rehab.) ($/YR) ________ ________ ________

b. User Costs During Construction

The percent cars and trucks are needed to calculate Speed Change Cost, Maneuvering Costs and Roughness Costs.

INPUT

PERCENT CARS (%) ___

PERCENT TRUCKS (%) ___

1) Speed Change Cost

This is the cost of extra fuel and oil that is consumed when a vehicle slows and then resumes speed. Three inputs are needed to compute this cost, the basic speed change cost, the number of speed changes and the truck speed change cost multiplier. Figure I-7, from Ogelsby (I-2), has been used to determine the speed change cost and Table I-1, from Ogelsby (I-2), has been used to determine the truck multiplier. The number of speed changes is to be determined by the user. Default values are provided for each of these three inputs, if desired by the user.

INPUT

Number of Speed Changes ________ (Default = 2)

Basic Speed Change Cost ($/1000/speed changes/veh) ________

Truck Speed Change Cost Multiplier ________ (Default = 9)

2) Maneuvering Cost

This cost is associated with the travel on curves and around corners due to a detour. Three inputs are required to determine this cost; the percent of the detour route length in curves, the basic maneuvering cost and the truck maneuvering cost multiplier. The percent of the detour route in curves is determined by the user (with the default value of ten provided). The basic maneuvering cost can be

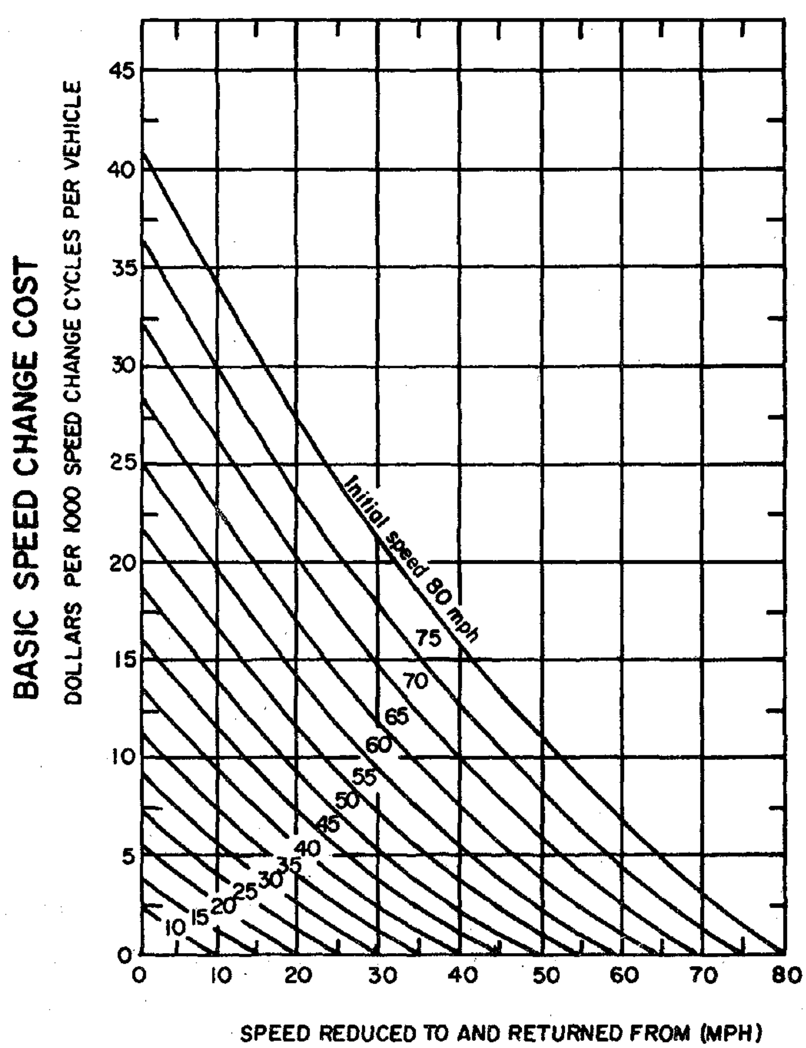


Figure I-7. Passenger Cars--Excess Running Cost of Speed-Change Cycles (Above Cost of Continuing at Initial Speed).

From Reference I-2.

taken from Fig. I-3. Table I-1 is again used to determine the truck maneuvering multiplier. The default values for the basic maneuvering cost and the multiplier are $22 per 1000 VEHICLE-MILES and 6, respectively.

INPUT

Percent of Detour Length in Curves (%)	________
Basic Maneuvering Cost ($/1000 veh-mi)	________
Truck Maneuvering Cost Multiplier	________ (Default = 6)

3) Roughness Cost

This cost is associated with the riding surface quality of the detour route. The serviceability index of the detour is required (with a default value of 3.0 provided). The other input needed is a ratio of the detour route length to the rehabilitation length (with the default value of 1.0 provided).

INPUT

Serviceability Index of Detour (SI)	________ (Default = 3.0)
Detour Length/Rehab. Length Ratio	________ (Default = 1.0)

4) Present Worth Additional Accident Cost

This cost is the present worth of a difference of costs, accident cost associated with a normal road and accident costs associated with a restricted road. A normal road would be one which is void of any rehabilitation construction efforts. A restricted road is one which part or all has been closed and traffic detoured due the rehabilitation work in progress. There are seven inputs required and all are to be determined by the user.

INPUT

Rehabilitation Section Length (mi)	________
Normal Speed (mph)	________
Restricted Speed (mph)	________
No. of Lanes Open, One Way, Normal Operation (1-4)	________

Table I-1. First Approximations of Multipliers to Determine Running Costs for Other Vehicle Classes from Those for Passenger Cars.

From Reference I-2

	Vehicle Class			
Operating Condition	5,000 lb Pickup	12,000 lb Single-Unit Truck	40,000 lb Gasoline-driven Truck (2-S2)	50,000 lb Diesel-driben Truck (3-S2)
Added costs for speed changes	1.15	2.5	9.0	11.5
Added costs for maneuvering curves and corners	1.15	2.2	6.0	6.0

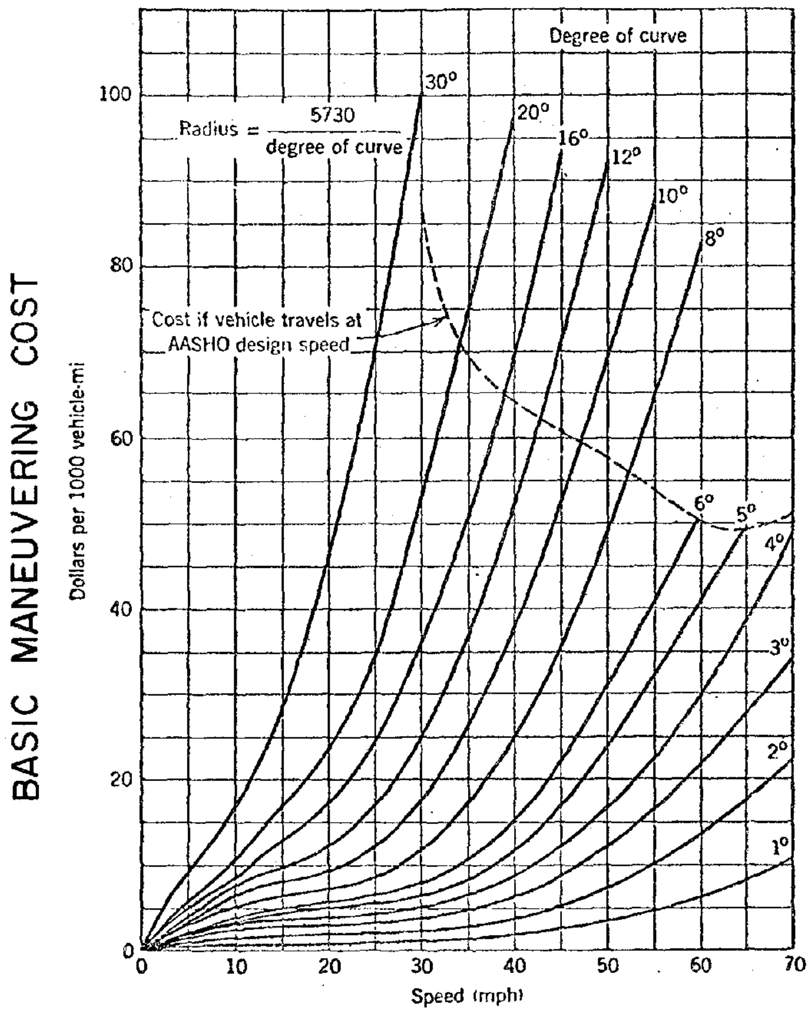


FIGURE I-8. Passenger cars - excess vehicle running cost due to horizontal curves. (Above cost on level tangent). (Source: NCHRP Report 133).

From Reference I-2

No. of Lanes Open, One Way, During Rehab. (1-4) ________

No. of Lanes Open, One Way, for Detour (1-3) ________

Average Cost of Accident ($/Accident) ________

B. EXPECTED PERFORMANCE

6. Time of Development

The time of development is the time in years that will be required to develop an idea to the point where it has been proven applicable.

INPUT (YR)

____	____	____
O	MP	P

7. Life

Expected life is the time in years that a given rehabilitation technique will keep the pavement at an acceptable level of service or distress while requiring no more than periodic minor maintenance. Life of the technique ends when the probability of maintaining an acceptable level of service drops below 50 percent.

INPUT (YR)

____	____	____
O	MP	P

8. Level of Pavement Service

Level of pavement service is defined by the critical performance or distress conditions measured over the life of the technique (See Chapter 1).

This particular decision criterion can vary in size depending upon the number of distress conditions utilized. For each condition to be inputted a maximum and minimum condition value must be assigned. In addition, the "acceptable" life must be selected along with one of the two general pavement types (rigid or flexible). The "acceptable" life is the time in years management and the traveling public expect the pavement to perform in a satisfactory manner. This not necessarily the same as the expected life of the technique. Rather it is an overall pavement life before major revision and/or reconstruction is contemplated. This "acceptable" life is usually determined by experience and is related to the period of time beyond which more funding for additional rehabilitation can be requested for the same section of road without adverse reaction from management or the legislature. Default values for acceptable life have been selected as 10 years for flexible pavements and 20 years for rigid pavements.

Two of the distress conditions are to always be used as input for this decision criterion; the Serviceability Index - Time Relation, and the Skid Number - Time Relation. The Serviceability Index - Time Relation input that is used here is identical to the Serviceability Index - Time Relation (with rehabilitation) that was recorded in User Benefit Cost (Decision Criterion No. 5). In order to assure that the Skid Number - Time Relation input is always utilized default values have been provided. There are two sets of defaults; the first is for rigid pavements and the second is for flexible pavements. Since the input for this decision criterion can vary, only one example is shown to illustrate the type of input that is necessary.

INPUT

Type of Pavement; Rigid-1 or Flexible-2 (1 or 2) ________

Minimum Value ________

Maximum Value ________

Acceptable Life ________ (Default 20/10)

Distress or Performance Condition (DPC)	DPC			
	t	0	0	0
	DPC			

t	______	______	______
DPC	______	______	______
t	______	______	______
	0	MP	P

Weights are assigned to each distress or performance condition in describing Level of Pavement Service. These weights must sum to a total of 1.0.

INPUT

Distress or Performance Condition	Weights
a. Serviceability Index	______
b. Skid Number	______
______	______
______	______
______	______
	1.0 (Total)

9. Traffic Volume Construction Time

Input values for this decision criterion are type of roadway (urban, suburban, or rural), volume/capacity ratio at time of construction (a number from 0 to 1.0) and the estimated construction time for the technique, in months.

INPUT

Urban - 1, Suburban - 2, or Rural - 3 (1, 2 or 3) ______

Volume - To - Capacity Ratio (0-1) ______

Estimated Construction Time (month) ______ ______ ______

0	MP	P

This decision criterion is defined as the ratio between the estimated construction time, and the "tolerable" construction time, based on traffic conditions. The tolerable construction time is calculated in the program from equations developed from data in reference (I-2) and (I-3). These equations are graphically portrayed in Figure I-9. Note the definition of volume/capacity ratio as being the ADT per lane/13,000 (veh/day)! The tolerable construction time (not to be confused with the estimated construction time) is shown on the abscissa of Figure I-9 and comes from the traffic levels shown in Figure I-8.

10. Expected Reworkability

Expected reworkability describes how easily the pavement can continue to be upgraded, added to, or maintained once the rehabilitation technique has been applied. The following qualitative judgement should be used.

Relative Scale

1. Excellent
2. Good
3. Fair
4. Poor
5. Very poor (P < .001 %)
6. Unacceptable expense
7. Totally unacceptable

INPUT

Rating (1-7)	______	______	______
	O	MP	P

C. ENERGY

11. User Energy Savings

This energy is the quantity of energy in Btu/yr./mi that is saved by applying the rehabilitation method. All required data has been previously recorded in User Savings, decision criterion No. 5, and thus no additional input is needed.

12. Rehabilitation Energy

This input involves the energy required to install the technique, and includes such items as transportation, material handling, placing and compaction. It does not include the energy involved in the manufacture of any of the materials utilized (see Decision Criterion No. 13).

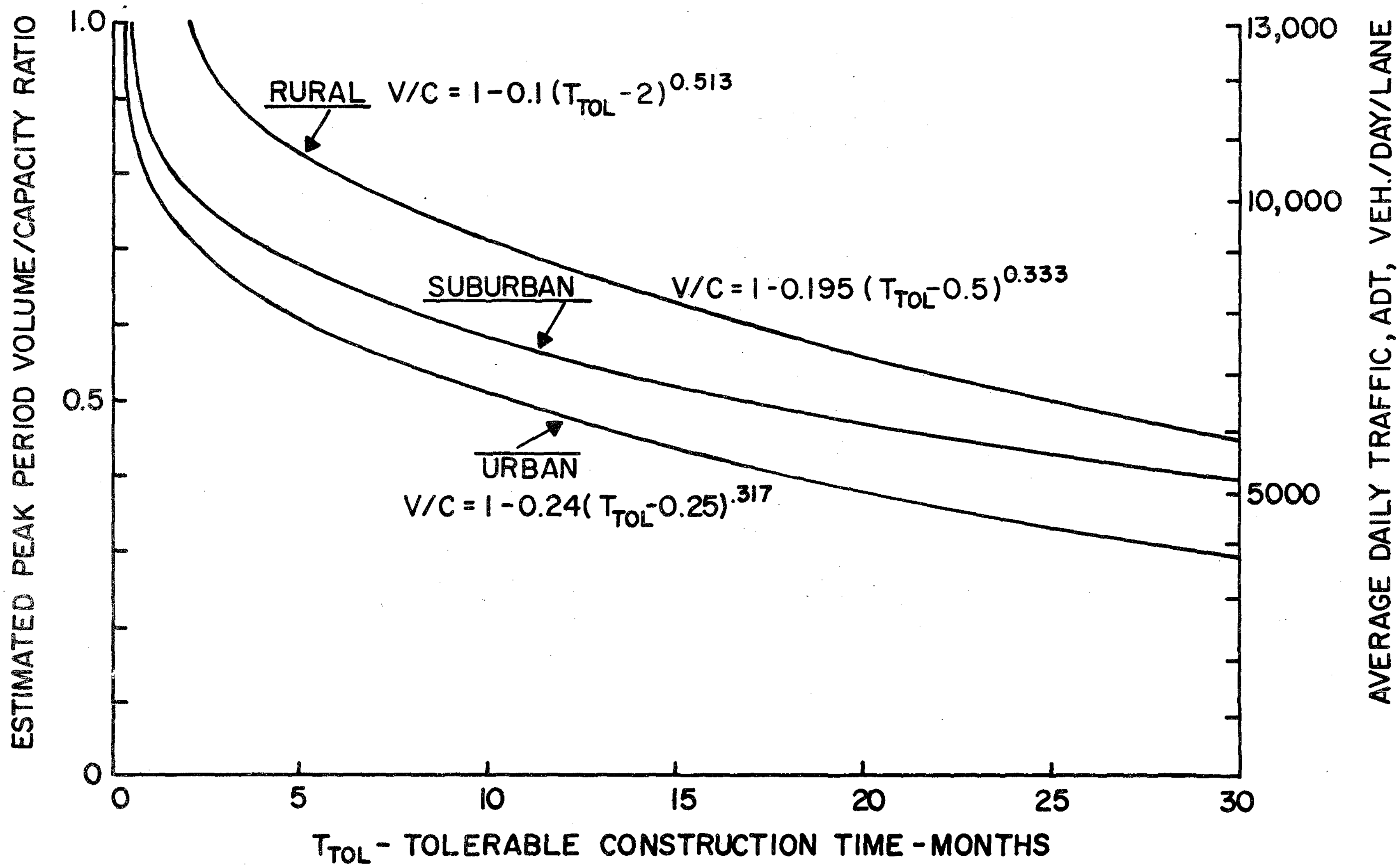


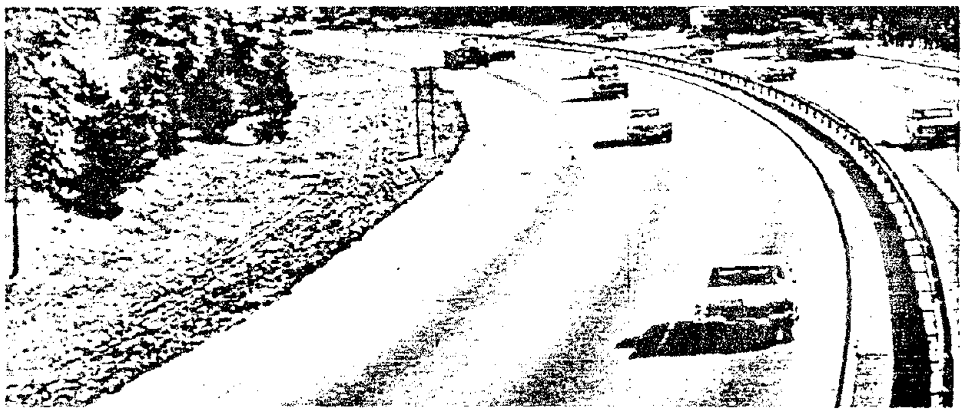
Figure I-9. Traffic Volume - Construction Time Tolerance (Based on Information in Figure I-8).

Figure I-10. Relation Assumed Between Traffic Level of Service and Tolerable Construction Time (T_{TOL}) Level of Service Definitions From Reference I-3 Level of Service Illustrations From Reference I-4.

Level A
Free flow, no restrictions on maneuvering or operating speed

Assumed T_{TOL}

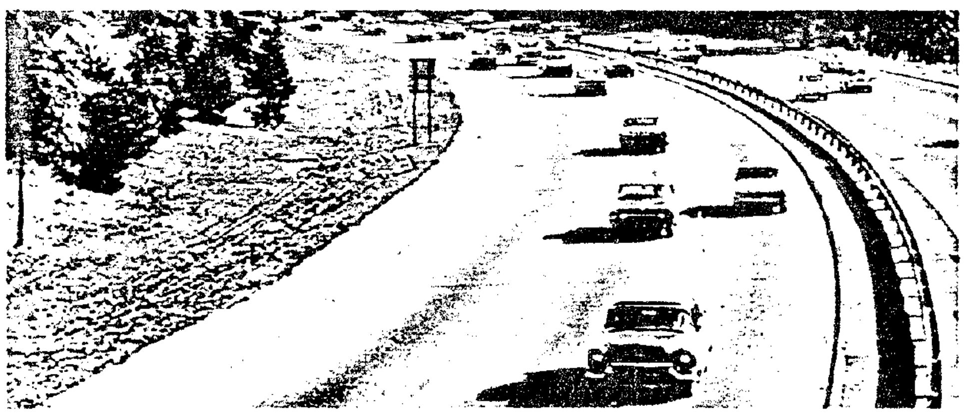
	Months
Rural →	∞
Suburban →	∞
Urban →	∞



Level B
Stable flow, few restrictions

Assumed T_{TOL}

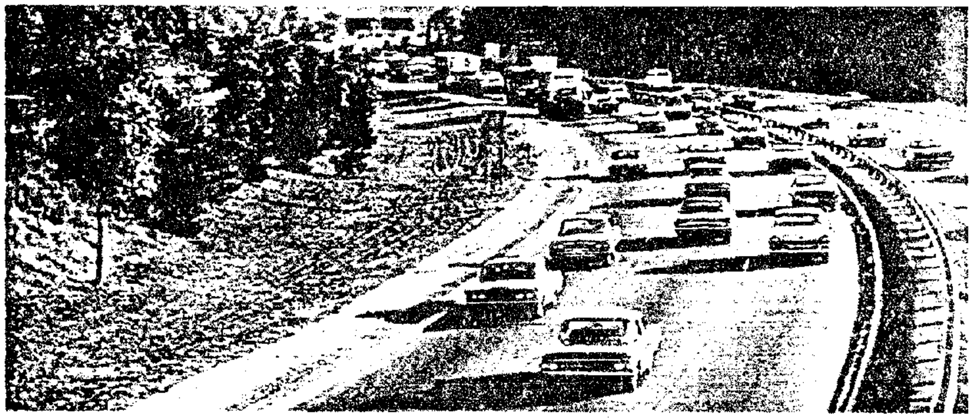
	Months
Rural	40
Suburban	30
Urban	15



Level C
Stable flow,
more restrictions

Assumed T_{TOL}

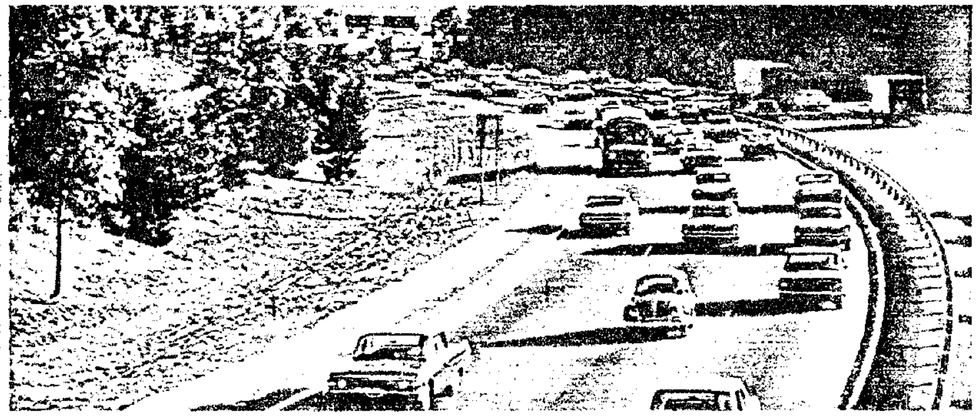
	Months
Rural	25
Suburban	10
Urban	4



Level D
Approaching
unstable flow

Assumed T_{TOL}

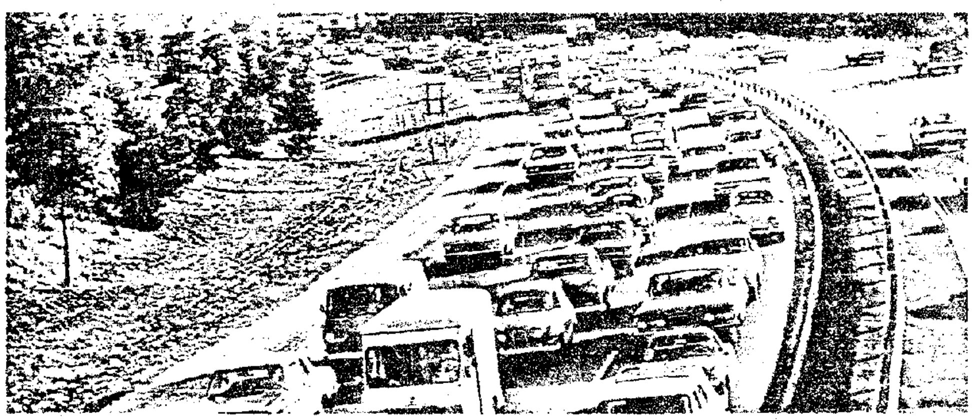
	Months
Rural	8
Suburban	5
Urban	3



Level E
Unstable flow,
some stoppages

Assumed T_{TOL}

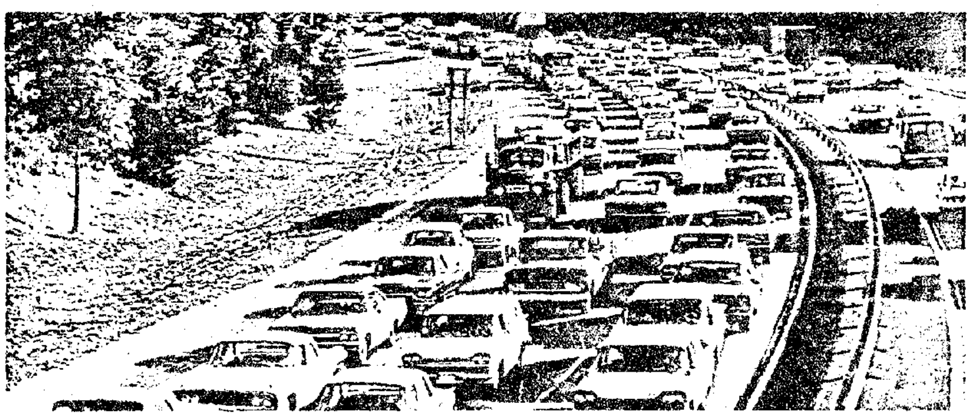
	Months
Rural	3
Suburban	1
Urban	0.5



Level F
Forced flow,
many stoppages

Assumed T_{TOL}

	Months
Rural	2
Suburban	0.5
Urban	0.25



INPUT

Rehabilitation (Btu/yd^2) ________ ________ ________

0 MP P

13. Material Energy

This input involves the energy required to manufacture each material used in the technique. In addition, if the material has an intrinsic energy value (eg. can be utilized as a fuel) this can be included. Whether or not this intrinsic energy is included depends upon the materials suitability as a fuel source. For example, cut-back additives in asphalt are valuable fuels and thus their intrinsic energy should be included. On the other hand many asphalts contain chemically combined sulphur which render them unsuitable for use as a fuel (at least until technology advances significantly). In this case the intrinsic energy value of asphalt should probably not be included.

INPUT

Material (Btu/lb) ________

Application Rate (lb/sy) ________ ________ ________

0 MP P

D. IMPACT

14. Safety Improvement

This is a qualitative measure of 1 to 7 of the degree of improvement in accident hazard due to the application of the technique based on the following qualitative judgement:

1. Excellent
2. Good
3. Fair
4. Poor
5. Unacceptable without some precautions ($P < .001$ %)
6. Unacceptable without extensive precautions ($0.001\% < P < .01\%$)
7. Totally unacceptable ($P > 0.01\%$)

INPUT

Rating (1-7) ________ ________ ________

0 MP P

15. Safety (During Rehabilitation)

This is a qualitative measure of 1 to 7 (See Decision Criterion 14) of the degree of safety to the traveling public and the work force during the application of the rehabilitation technique. Traffic addicent hazard as well as occupational hazards due to chemical, electrical, radioactive, and mechanical energy should be considered.

INPUT

Rating (1-7)	______	______	______
	O	MP	P

16. Environmental Impact - Noise

This is a qualitative measure of 1 to 7 (See Decision Criterion 14) of the expected noise level due to applying the technique. This may be any unusual traffic noise expected because of maneuvering around the rehabilitation activity and any noise level changes due to the rehabilitated pavement surface. The time required to apply the technique will be weighted as well; the longer the rehabilitation construction time, the lower the rating (higher number)

INPUT

Rating (1-7)	______	______	______
	O	MP	P

17. Environmental Impact - Pollution

This is a qualitative measure of 1 to 7 (See Decision Criterion No. 14) of the expected level of pollution due to applying the rehabilitation technique and to any unusual amount of pollution to be expected from re-routing the traffic. Pollution that may result will include solids and gases as well as harmful chemicals that may leach out with time.

The time it takes to apply the technique will increase the amount of traffic - related pollution and will tend to lower the rating (higher number).

INPUT

Rating (1-7)	________	________	________
	O	MP	P

References

I-1. McFarland, W. F., "Benefit Analysis for Pavement Design Systems," Texas Transportation Institute Report 123-13, 1972.

I-2. Oglesby, C. H., Highway Engineering, 3rd Ed., John Wiley & Sons, New York, 1975.

I-3. "Highway Capacity Manual," HRB Special Report 87, Transportation Research Board, NAS, 1966, 397 pp.

I-4. Cappelle, D. G., et al., An Introduction to Traffic Engineering, Institute of Traffic Engineers, 1968.

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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R&D