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# Guide to Developing Performance-Related Specifications for PCC Pavements, Volume III: Appendixes C Through F

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## FOREWORD

This report is the third volume of a four-volume set of reports presenting the results of a study to further the development of performance-related specifications for portland cement concrete pavement construction. Using the recommended performance-related specifications, a highway agency can upon construction predict the performance of a pavement and forecast the pavement's life-cycle costs. The benefits to the agency from using such specifications can be significant. This report contains supporting documentation for the study, including information on the performance prediction models used in the specifications and on the typical variabilities associated with acceptance quality characteristics. This report will be of interest to engineers concerned with quality asssurance, specifications, and construction of concrete pavements.

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Acting Director
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16. Abstract

This study continued the development of performance-related specifications (PRS) for jointed plain concrete pavements (JPCP). Under this research project, the prototype PRS (developed under previous FHWA research) was revised and expanded to make it more practical. Different implementation levels of the PRS were defined (Levels 1, 2, and 3), with the simplified PRS (Level 1) designed to be immediately implementable. This report (four volumes) is offered as a practical guide to developing and using PRS for the acceptance of JPCP lots. It discusses all aspects of the research, including the step-by-step procedures for developing and using PRS, results of demonstrations of the PRS methodology, explanations of supplementary laboratory and field studies, and the user guide for the revised PaveSpec 2.0 PRS demonstration software.

This volume includes supporting documentation for the research study. Appendix C contains the complete results of the literature search that investigated not only typical PRS-related acceptance quality characteristic (AQC) variations, but possible AQC-to-distress indicator relationships. The specific research conducted to investigate the typical AQC variability (including conducted laboratory/field studies) is contained in appendix D. A summary of the laboratory/field studies used to investigate specific AQC-to-distress indicator relationships is presented in appendix E. Appendix E also contains detailed descriptions of each of the distress indicator models used in the revised PaveSpec 2.0 PRS demonstration software. Appendix F presents an annotated bibliography of the literature found to be pertinent to this research project.

This volume is the third in a series. The other volumes are:

| | |
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| FHWA-RD-98-155 | Volume I: Practical Guide, Final Report and Appendix A |
| FHWA-RD-98-156 | Volume II: Appendix B—Field Demonstrations |
| FHWA-RD-99-059 | Volume IV: Appendix G—PaveSpec 2.0 User Guide |


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# SI* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

| Symbol | When You Know | Multiply By | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| in | inches | 25.4 | millimeters | mm |
| ft | feet | 0.305 | meters | m |
| yd | yards | 0.914 | meters | m |
| mi | miles | 1.61 | kilometers | km |
| | **AREA** | | | |
| $in^2$ | square inches | 645.2 | square millimeters | $mm^2$ |
| $ft^2$ | square feet | 0.093 | square meters | $m^2$ |
| $yd^2$ | square yards | 0.836 | square meters | $m^2$ |
| ac | acres | 0.405 | hectares | ha |
| $mi^2$ | square miles | 2.59 | square kilometers | $km^2$ |
| | **VOLUME** | | | |
| fl oz | fluid ounces | 29.57 | milliliters | mL |
| gal | gallons | 3.785 | liters | L |
| $ft^3$ | cubic feet | 0.028 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.765 | cubic meters | $m^3$ |
| NOTE: Volumes greater than 1000 l shall be shown in $m^3$. | | | | |
| | **MASS** | | | |
| oz | ounces | 28.35 | grams | g |
| lb | pounds | 0.454 | kilograms | kg |
| T | short tons (2000 lb) | 0.907 | megagrams (or "metric ton") | Mg (or "t") |
| | **TEMPERATURE (exact)** | | | |
| °F | Fahrenheit temperature | 5(F-32)/9 or (F-32)/1.8 | Celcius temperature | °C |
| | **ILLUMINATION** | | | |
| fc | foot-candles | 10.76 | lux | lx |
| fl | foot-Lamberts | 3.426 | candela/$m^2$ | $cd/m^2$ |
| | **FORCE and PRESSURE or STRESS** | | | |
| lbf | poundforce | 4.45 | newtons | N |
| $lbf/in^2$ | poundforce per square inch | 6.89 | kilopascals | kPa |

## APPROXIMATE CONVERSIONS FROM SI UNITS

| Symbol | When You Know | Multiply By | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| mm | millimeters | 0.039 | inches | in |
| m | meters | 3.28 | feet | ft |
| m | meters | 1.09 | yards | yd |
| km | kilometers | 0.621 | miles | mi |
| | **AREA** | | | |
| $mm^2$ | square millimeters | 0.0016 | square inches | $in^2$ |
| $m^2$ | square meters | 10.764 | square feet | $ft^2$ |
| $m^2$ | square meters | 1.195 | square yards | $yd^2$ |
| ha | hectares | 2.47 | acres | ac |
| $km^2$ | square kilometers | 0.386 | square miles | $mi^2$ |
| | **VOLUME** | | | |
| mL | milliliters | 0.034 | fluid ounces | fl oz |
| L | liters | 0.264 | gallons | gal |
| $m^3$ | cubic meters | 35.71 | cubic feet | $ft^3$ |
| $m^3$ | cubic meters | 1.307 | cubic yards | $yd^3$ |
| | **MASS** | | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.202 | pounds | lb |
| Mg (or "t") | megagrams (or "metric ton") | 1.103 | short tons (2000 lb) | T |
| | **TEMPERATURE (exact)** | | | |
| °C | Celcius temperature | 1.8C + 32 | Fahrenheit temperature | °F |
| | **ILLUMINATION** | | | |
| lx | lux | 0.0929 | foot-candles | fc |
| $cd/m^2$ | candela/$m^2$ | 0.2919 | foot-Lamberts | fl |
| | **FORCE and PRESSURE or STRESS** | | | |
| N | newtons | 0.225 | poundforce | lbf |
| kPa | kilopascals | 0.145 | poundforce per square inch | $lbf/in^2$ |

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

# TABLE OF CONTENTS

APPENDIX C— LITERATURE SEARCH SUMMARY ..... 1

INTRODUCTION ..... 1
CONCRETE STRENGTH ..... 2
Acceptance Method Used in the Prototype PRS ..... 2
Variation of Concrete Compressive Strength ..... 2
Variation of Concrete Flexural Strength ..... 19
Variation of Concrete Split Tensile Strength ..... 21
Within-Test and Between-Batch Variability ..... 22
Accelerated Testing Methods for Concrete Strength ..... 24
Compressive Versus Flexural Strength ..... 27
Strength Type Conversion Equations ..... 28
SLAB THICKNESS ..... 30
ENTRAINED AIR VOID PARAMETERS ..... 35
Variation of Entrained Air Content ..... 37
Relationship Between the Entrained Air Content of Plastic and Hardened Concrete ..... 40
New Technology for Measuring Entrained Air Content ..... 43
INITIAL SMOOTHNESS ..... 43
Variation of Initial Smoothness ..... 43
Sources of Error in Measuring Smoothness ..... 48
Variability Types ..... 50
Variation in Trace Reduction ..... 51
Variation Due to Data Filter Settings ..... 51
Conclusions ..... 53
JOINT SAWCUT DEPTH ..... 55
Variation of Joint Sawcut Depth ..... 56
TIMING OF JOINT SAWING ..... 56
Investigation of Earliest Joint Sawcutting ..... 56
Investigation of Latest Joint Sawcutting ..... 57
Recommended Timing of Joint Sawcutting ..... 57
Timing of Joint Sawing Conclusions ..... 57
PERCENT CONSOLIDATION AROUND DOWELS ..... 57
DOWEL MISALIGNMENT ..... 62
Summary of Dowel Misalignment Studies ..... 62
Factors Affecting Dowel Misalignment ..... 68
Methods of Measuring Misalignment ..... 70
Possible Solution to Misalignment Problems ..... 70
Summary of Dowel Misalignment Tolerances ..... 71
TIE BAR DEPTH DEVIATION (FROM MID-SLAB) AND ALIGNMENT ..... 71
EFFECTIVENESS OF CURING ..... 71

# TABLE OF CONTENTS (continued)


DEPTH OF TINING ..... 71
Variation of Depth of Tining ..... 71
Brief Summary of Potentially Useful Ongoing Research Projects ..... 74
CONCLUSIONS ..... 77

APPENDIX D—TYPICAL ACCEPTANCE QUALITY CHARACTERISTIC VARIABILITY ..... 79

INTRODUCTION ..... 79
DISCUSSION OF VARIANCE ..... 80
Components of Variance ..... 80
Calculating Equivalent Variability ..... 81
IDENTIFICATION OF FIELD PROJECTS ..... 82
CONCRETE STRENGTH ..... 82
Historical Concrete Strength Data ..... 82
Field Investigation of the Typical Concrete Strength Within-Lot Standard Deviation ..... 85
SLAB THICKNESS ..... 112
Field Investigation of the Typical Slab Thickness Within-Lot Standard Deviation ..... 113
Summary of Slab Thickness Variation Results ..... 114
ENTRAINED AIR CONTENT ..... 114
Historical Entrained Air Content Data ..... 117
Field Investigation of the Typical Entrained Air Content Within-Lot Standard Deviation ..... 118
INITIAL SMOOTHNESS ..... 125
Historical Initial Smoothness Data ..... 125
Additional Initial Smoothness Data Used to Investigate the Typical Initial Smoothness Within-Lot Standard Deviation ..... 126
Summary of the Initial Smoothness Variation Results ..... 126
SAWCUT DEPTH ..... 128
Historical Sawcut Depth Data ..... 128
Field Investigation of the Typical Sawcut Depth Within-Lot Standard Deviation ..... 130
Summary of Sawcut Depth Variation Results ..... 130
PERCENT CONSOLIDATION AROUND DOWELS ..... 130
Field Investigation of the Percent Consolidation Around Dowels Within-Lot Standard Deviation ..... 132
Summary of Percent Consolidation Around Dowels Variation Results ..... 132
DEVIATION OF TIE BAR PLACEMENT FROM SLAB MID-DEPTH ..... 134

# TABLE OF CONTENTS (continued)


Field Investigation of the Within-Lot Standard Deviation of the Deviation of Tie Bar Placement From Slab Mid-Depth ..... 134
Summary of the Deviation of Tie Bar Placement From Slab Mid-Depth Variation Results ..... 135
ACCEPTANCE QUALITY CHARACTERISTIC VARIABILITY SUMMARY ..... 136

APPENDIX E—DISTRESS INDICATOR PREDICTION MODELS ..... 139

INTRODUCTION ..... 139
IDENTIFICATION OF PROJECTS ..... 139
SUMMARY OF LABORATORY AND FIELD STUDIES ..... 139
Prediction of 28-day Flexural Strength ..... 139
Effect of Entrained Air Content on Transverse Joint Spalling ..... 147
Effect of Percent Consolidation Around Dowels on Transverse Joint Faulting ..... 156
Tie Bar Depth Versus Load Transfer Efficiency ..... 158
SUMMARY OF CURRENT DISTRESS INDICATOR MODELS ..... 158
Transverse Slab Cracking ..... 161
Transverse Joint Spalling ..... 164
Transverse Joint Faulting ..... 176
Present Serviceability Rating (PSR) ..... 185
International Roughness Index (IRI) ..... 187

APPENDIX F—ANNOTATED BIBLIOGRAPHY ..... 193

REFERENCES ..... 259

# LIST OF FIGURES

Figure No.


1 PCC relative standard deviation of components of variation for compressive strength ..... 6
2 Histogram of 28-day cylinder compressive strength data for a project constructed in Oklahoma (200 pairs of 2 cylinders) ..... 9
3 Histogram of 28-day cylinder compressive strength results—1993 project on I-70 near Vona, Colorado ..... 12
4 Histogram of 28-day concrete compressive strength ..... 15
5 Distribution of PCC compressive strength ..... 15
6 Histogram of 28-day cylinder concrete compressive strength ..... 16
7 Histogram of 7-day cylinder concrete compressive strength ..... 16
8 Histogram of concrete flexural strength ..... 17
9 Histogram of concrete flexural strength ..... 17
10 Repeatability (within a laboratory) standard deviation versus average compressive strength for all programs ..... 23
11 Average repeatability (within a laboratory) and reproducibility (between laboratories) standard deviations versus average compressive strength ..... 23
12 Reproducibility (between laboratories) standard deviation versus average compressive strength for all programs ..... 24
13 Relationship between flexural and compressive strength for various aggregate combinations ..... 29
14 Histogram of thickness data for a project constructed in Oklahoma ..... 32
15 Frequency distribution of concrete pavement thickness—1959 to 1961 Michigan data ..... 33
16 Distribution of PCC Core Thickness ..... 34
17 Histogram of concrete slab thickness ..... 34
18 Histogram of pressure meter entrained air content data for an Oklahoma project ..... 38
19 Plastic versus hardened air content—Plot 1 ..... 41
20 Plastic versus hardened air content—Plot 2 ..... 41
21 Plastic versus hardened air content—Plot 3 ..... 42
22 Plastic versus hardened air content—Plot 4 ..... 42
23 Histogram of section profiles—(1985) U.S. 73, Atchison County, Kansas ..... 44
24 Histogram of section profiles—(1985) U.S. 50—Eastbound, Finney County, Kansas ..... 45
25 Histogram of section profiles—(1985) U.S. 50—Westbound, Finney County, Kansas ..... 45
26 Histogram of section profiles—(1985) I-70—Westbound, Dickinson County, Kansas ..... 46

# LIST OF FIGURES (continued)

Figure No.


27 Histogram of section profiles—(1986) I-70—Eastbound, Dickinson County, Kansas ..... 47
28 Histogram of section profiles—(1990) I-70—Geary County, Kansas (measured with a 0.0-mm blanking band) ..... 47
29 Histogram of section profiles—(1990) I-70—Geary County, Kansas (measured with a 5.1-mm blanking band) ..... 48
30 Histogram of section profiles—(1990) I-470—Shawnee County, Kansas (measured with a 0.0-mm blanking band) ..... 49
31 Histogram of section profiles—(1990) I-470—Shawnee County, Kansas (measured with a 5.1-mm blanking band) ..... 49
32 Profilograph variability as a function of JPCP for mechanical profilographs ..... 54
33 Profilograph variability as a function of JPCP for automated profilographs ..... 55
34 U.S. 69 concrete pavement joint faulting (1992 deflection versus original density) ..... 61
35 Plot of compressive strength ratio versus density ratio ..... 62
36 Effects of dowel misalignment ..... 63
37 Frequency graphs of deviation of dowel bars from mid-depth for five investigated projects ..... 66
38 Distribution of groove-depth compliance measurements ..... 73
39 Example of a laboratory compressive strength development versus maturity relationship ..... 89
40 Example of a laboratory-developed (compressive to flexural strength) interstrength relationship ..... 90
41 Example of a measured early age field compressive strength plotted on the laboratory-developed strength versus maturity curve ..... 91
42 Example of using the laboratory-developed strength versus maturity relationship to extrapolate a 28-day compressive strength from a measured early age compressive strength ..... 92
43 Example of using the laboratory-developed (compressive to flexural strength) interstrength relationship to predict 28-day flexural strengths ..... 93
44 Compressive strength versus maturity curve for Shawano, WI ..... 98
45 Splitting tensile strength versus maturity curve for Shawano, WI ..... 98
46 Flexural strength versus maturity curve for Shawano, WI ..... 99
47 Flexural strength versus compressive strength relationship for Shewano, WI ..... 99
48 Flexural strength versus splitting tensile strength relationship for Shewano, WI ..... 100
49 Compressive strength versus maturity curve for St. Johns, MI ..... 100
50 Splitting tensile strength versus maturity curve for St. Johns, MI ..... 101
51 Flexural strength versus maturity curve for St. Johns, MI ..... 101

# LIST OF FIGURES (continued)

Figure No.


52 Flexural strength versus compressive strength relationship for St. Johns, MI ..... 102
53 Flexural strength versus splitting tensile strength relationship for St. Johns, MI ..... 102
54 Arrhenius maturity versus time (based on laboratory data) for Ottumwa, IA ..... 103
55 Compressive strength versus maturity curve for Ottumwa, IA ..... 103
56 Flexural strength versus maturity curve for Ottumwa, IA ..... 104
57 Flexural strength versus compressive strength relationship for Ottumwa, IA ..... 104
58 Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder f'c → 28-day f'c → 28-day MR) ..... 106
59 Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder ST → 3-day MR → 28-day MR) ..... 106
60 Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder f'c → 28-day f'c → 28-day MR) ..... 108
61 Predicted versus measured 28-day flexural strength (Translation method: 14-day cylinder f'c → 28-day f'c → 28-day MR) ..... 108
62 Predicted versus measured 28-day flexural strength (Translation method: 28-day f'c → 28-day MR) ..... 109
63 Predicted versus measured 28-day flexural strength (Translation method: 3-day core f'c → 28-day f'c → 28-day MR) ..... 109
64 Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder ST → 3-day MR → 28-day MR) ..... 110
65 Predicted versus measured 28-day flexural strength (Translation method: 3-day beam MR → 28-day MR) ..... 110
66 Predicted versus measured 28-day flexural strength (Translation method: 14-day beam MR → 28-day MR) ..... 111
67 Backcalculated flexural strength conversion multiplier for specimens at different ages ..... 113
68 Air content measured on cores at the AASHO Road Test ..... 117
69 Comparison of plastic entrained air content sampled before and after the paver ..... 124
70 Plastic entrained air content versus hardened entrained air content (measured from linear traverse testing of cores) ..... 124
71 Histogram of the measured within-lot initial smoothness standard deviations for profile indices reduced using a 5.1-mm blanking band ..... 129
72 Histogram of the measured within-lot initial smoothness standard deviations for profile indices reduced using a 0.0-mm blanking band ..... 129
73 Results of the cylinder compressive strength versus core compressive strength study ..... 143

# LIST OF FIGURES (continued)

Figure No.


74 Results of the core split tensile strength versus cylinder split tensile strength study ..... 144
75 Relative frequency histogram of cylinder-to-core compressive strength ratios ..... 145
76 Relative frequency histogram of cylinder-to-core splitting tensile strength ratios ..... 146
77 Predicted versus measured transverse joint spalling using the "entrained air content only" model developed under a previous PRS project ..... 152
78 Predicted versus measured transverse joint spalling using the calibrated "entrained air content only" model ..... 153
79 Sensitivity of the calibrated "entrained air content only" spalling model (percentage of joint length spalled versus age for different entrained air content percentages) ..... 154
80 Predicted versus measured transverse joint spalling using the calibrated "entrained air content and void spacing factor" model ..... 155
81 Predicted versus measured transverse joint spalling using the calibrated "void spacing factor only" model ..... 157
82 Plot of load transfer efficiency as a function of percent concrete consolidation ..... 159
83 Load transfer efficiency versus tie bar deviation from slab mid-depth ..... 160
84 Sensitivity of transverse cracking model to shoulder type ..... 165
85 Sensitivity of transverse cracking model to transverse joint spacing ..... 165
86 Sensitivity of transverse cracking model to slab thickness ..... 166
87 Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 1650) ..... 171
88 Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 1000) ..... 171
89 Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 0, Days90 = 10) ..... 172
90 Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 0, Days90 = 150) ..... 172
91 Sensitivity of the JPCP transverse joint spalling base model to age and dowel protection ..... 173
92 Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (liquid sealant) ..... 174
93 Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (preformed sealant) ..... 174
94 Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (silicone sealant) ..... 175
95 Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (no sealant) ..... 175

# LIST OF FIGURES (continued)

Figure No.


96 Sensitivity of the JPCP transverse joint faulting base model to traffic and dowel diameter ..... 182
97 Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and widened lanes ..... 182
98 Sensitivity of the JPCP transverse joint faulting base model (for doweled joints) to traffic and widened lanes ..... 183
99 Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and base permeability ..... 183
100 Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and number of days above 32 °C ..... 184
101 Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and number of wet days ..... 184
102 Sensitivity of the JPCP transverse joint faulting base model to traffic and percent consolidation around dowels (faulting versus ESAL's for different percentages of consolidation around dowels) ..... 185
103 Sensitivity of the JPCP PSR model to transverse joint faulting and transverse slab cracking ..... 187
104 Sensitivity of the JPCP PSR model to transverse slab cracking and transverse joint spalling ..... 188
105 Sensitivity of the JPCP PSR model to transverse joint faulting and transverse joint spalling ..... 188
106 Sensitivity of the JPCP PSR model to transverse slab cracking and longitudinal cracking ..... 189
107 Sensitivity of the JPCP IRI model to age and transverse slab cracking ..... 190
108 Sensitivity of the JPCP IRI model to age and transverse joint faulting ..... 191
109 Sensitivity of the JPCP IRI model to age and transverse joint spalling ..... 191

# LIST OF TABLES


Table No.

1 Coefficient of variation due to in-place strength variation within the structure... 3
2 Magnitude and precision of strength correction factors for converting core strengths into equivalent in-place strengths ..... 3
3 Concrete cylinder 28-day compressive strength variability data from the Illinois Department of Transportation MISTIC database ..... 4
4 Components of 28-day compressive strength variability for seven West Virginia projects ..... 5
5 Summary of 28-day cylinder compressive strength statistics of various projects. 7
6 ACI 1972 recommended standards for 28-day cylinder compressive test COV ... 9
7 Variation of normalized concrete compressive strength with age—moist curing—portland cements ..... 10
8 Variation of normalized concrete compressive strength with age—moist curing—blended cements ..... 10
9 Variation of normalized concrete compressive strength with age—air curing ..... 10
10 Variation of normalized concrete compressive strength with age—outdoor exposure ..... 11
11 Average deviation of 28-day concrete compressive strengths ..... 13
12 Historical concrete compressive strength data published by the State Road Department of Florida—project comparison ..... 14
13 Historical concrete compressive strength data published by the State Road Department of Florida—production source comparison ..... 14
14 Strength parameters for Class A structural concrete—two projects in West Virginia ..... 15
15 1995 28-day cylinder and core compressive strength data ..... 18
16 Typical standard deviations for 28-day PCC cylinder strengths ..... 18
17 Concrete beam 28-day flexural strength variability data from the Illinois Department of Transportation MISTIC Database ..... 19
18 Concrete beam 28-day flexural strength variability data from the Texas Highway Department and the AASHO Road Test ..... 19
19 Typical flexural strength data for different State studies ..... 20
20 Measured flexural strength variations at the Atlanta International Airport ..... 21
21 Proposed guidelines for assessing the level of quality associated with flexural strength testing data ..... 21
22 Compressive strength within-test COV guidelines ..... 24
23 Non-destructive test methods for concrete ..... 25
24 Concrete slab thickness variability data from the Louisiana Department of Transportation and Development and the Texas Highway Department ..... 31
25 1995 Core and GPR thickness data ..... 35

# LIST OF TABLES (continued)

Table No.


26 Standard deviations of core thickness from projects in Ohio, Kansas, and Georgia ..... 35
27 Entrained air content variability data from the Illinois Department of Transportation's MISTIC database ..... 37
28 Plastic concrete air content research data— New York State ..... 39
29 1995 pressure meter entrained air content data collected for three SHA projects 40
30 Project air content standard deviations ..... 40
31 Pavement roughness data obtained with profilographs ..... 51
32 Results of trace reduction by different operators ..... 52
33 Profile index data obtained at different filter settings of electronic profilographs52
34 Relative advantages and disadvantages of nuclear density gauge types ..... 60
35 Summary of vertical rotation measurements ..... 67
36 Summary of horizontal rotation measurements ..... 67
37 Summary of longitudinal alignment ..... 67
38 Dowel bar depth statistics for three projects constructed in South Dakota in 1989 ..... 69
39 Dowel bar average vertical rotation statistics for three projects constructed in South Dakota in 1989 ..... 69
40 Dowel bar horizontal rotation statistics for three projects constructed in South Dakota in 1989 ..... 69
41 Dowel bar longitudinal displacement statistics for three projects constructed in South Dakota in 1989 ..... 69
42 Suggested misalignment tolerances presented in different research studies ..... 72
43 Acceptable misalignment tolerances from different construction specifications .. 72
44 Summary of recommended data sources for investigation efforts ..... 78
45 Data collected at each project evaluated for acceptance quality characteristic variation ..... 83
46 Summary of the measured total within-lot concrete strength variations ..... 96
47 Recommended within-lot concrete strength target standard deviations for different specimens at different ages ..... 97
48 Strength development and interstrength relationship regression equations ..... 105
49 Summary of accuracy indicators for the 28-day flexural strength prediction models ..... 111
50 Recommended concrete strength target standard deviations ..... 113
51 Summary of within-lot slab thickness standard deviations based on drilled core data ..... 115
52 Summary of within-lot slab thickness standard deviations based on ground-penetrating radar data ..... 116

# LIST OF TABLES (continued)

Table No.


53 Comparison of average slab thickness standard deviations for different design thicknesses ..... 116
54 Summary of within-lot plastic entrained air content standard deviations (based on air pressure meter data) ..... 120
55 Summary of within-lot hardened entrained air content standard deviations (linear traverse testing data) ..... 121
56 Summary of within-lot void spacing factor standard deviations (linear traverse testing data) ..... 121
57 Summary of within-lot specific surface area standard deviations (linear traverse testing data) ..... 122
58 Summary of within-lot voids per mm standard deviations (linear traverse testing data) ..... 122
59 Summary of within-lot hardened concrete air void system parameter standard deviations (linear traverse testing data) ..... 123
60 Summary of measured within-lot initial smoothness statistics for profile indices (California profilograph) reduced using a 5.1-mm blanking band ..... 127
61 Summary of measured within-lot initial smoothness statistics for profile indices (California profilograph) reduced using a 0.0-mm blanking band ..... 128
62 Summary of within-lot sawcut joint depth standard deviations ..... 131
63 Summary of within-lot percent consolidation around dowels standard deviations ..... 133
64 Summary of within-lot tie bar deviation from mid-depth standard deviations ... 135
65 Summary of target within-lot standard deviations for PCC pavement acceptance quality characteristics ..... 137
66 Data collected at each project evaluated for the development of new (or calibration of existing) distress indicator models ..... 140
67 Data collected for variables related to percent joint spalling ..... 151
68 Average daytime thermal gradients (based on slab thickness and climatic zone), °F/in ..... 163

# LIST OF ACRONYMS AND ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AC | Asphalt concrete |
| ADT | Average daily traffic |
| AQC | Acceptance quality characteristic |
| ASTM | American Society for Testing and Materials |
| CPF | Composite pay factor |
| EP | Expected pay (curve) |
| ESAL | Equivalent single-axle load |
| FHWA | Federal Highway Administration |
| FTC | Freeze-thaw cycle |
| IRI | International Roughness Index |
| JPCP | Jointed plain concrete pavement |
| LCC | Life-cycle cost |
| $LCC_{CON}$ | As-constructed life-cycle cost |
| $LCC_{DES}$ | As-designed life-cycle cost |
| M&R | Maintenance and rehabilitation |
| MQL | Maximum quality limit |
| OC | Operating characteristic (curve) |
| OTA | Office of Technology Applications (FHWA) |
| PCC | Portland cement concrete |
| PF | Pay factor |
| PI | Profile index |
| PMS | Pavement management system |
| PRS | Performance-related specification |
| PSF | Percentage of sublots failed |
| PSI | Present serviceability index |
| PSR | Present serviceability rating |
| PW | Present worth |
| PWL | Percent within limits |
| PW$ | Present-worth dollars |
| QA | Quality assurance |
| QC | Quality control |
| RQL | Rejectable quality limit |
| SHA | State highway agency |
| USL | Upper specification limit |

# APPENDIX C—LITERATURE SEARCH SUMMARY

## INTRODUCTION

The validity and defensibility of a performance-related specification (PRS) depends on the accurate prediction of pavement performance, which is described in terms of distress indicators (i.e., transverse cracking, transverse joint faulting, transverse joint spalling, and pavement smoothness). The implementation of a PRS requires considerable knowledge of how these distress indicators develop over time, as well as the specific construction and material-related acceptance quality characteristics (AQC's) that influence them. The entire process is made even more difficult by considering the variation of the material- and construction-related AQC's. Therefore, in an attempt to move toward creating an implementable PRS, two of the goals of the current research were identified as the following:

- *Establish the "typical" variations associated with the AQC's to be included in the PRS for concrete pavements*—This information will be used to provide guidance to agencies in setting the target as-designed standard deviations.

- *Establish additional (and improve existing) distress indicator prediction models to be included in the PRS for concrete pavements*—Each of these prediction models must be associated with one or more AQC's that are measurable and under the control of the contractor.

A comprehensive literature search was conducted to facilitate this research. The search was limited to the investigation of the following AQC's identified by the research team:

- Concrete strength.
- Slab thickness.
- Entrained air void parameters.
- Initial smoothness.
- Joint sawcut depth.
- Timing of joint sawing.
- Percent consolidation around dowels.
- Dowel misalignment.
- Tie bar depth deviation (from midslab) and alignment.
- Effectiveness of curing.
- Depth of tining.

Although the literature search identified many general jointed plain concrete pavement (JPCP) PRS documents, the search specifically focused on identifying previous studies in which: (1) the effects of concrete pavement AQC's on pavement performance were

well documented and (2) the variations of concrete pavement AQC's were reported and documented in an unbiased manner.

The literature search consisted of on-line queries of the Transportation Research Information System (TRIS), National Technical Information Service (NTIS), and Engineering Index (EI) Compendex computer databases. Approximately 170 references from these sources were identified, acquired, and carefully reviewed. Annotations were prepared for each pertinent document and are presented in appendix F. This appendix (appendix C) contains a summary of the relevant information obtained from the literature review.

## CONCRETE STRENGTH

### Acceptance Method Used in the Prototype PRS

Concrete strength is one of the four AQC's included in the prototype PRS developed under the previous FHWA PRS study.[1,2,3] The prototype required that flexural strength be measured directly from beam specimens or indirectly from cylinder strength tests and used for the acceptance of the as-constructed portland cement concrete (PCC) slab. The prototype PRS recommended using core testing as an indirect measure of the in situ flexural strength. With this method, cores are taken at some reference maturity equivalent (for example, 3 days at 22 °C). To provide rapid feedback to the contractor, the 28-day flexural strength is then projected from the early age in situ strength data. The inter-strength relationship between flexural and compressive strength would be established for the approved mix design (prior to construction) using project-approved aggregates, cement, and admixtures. The previous PRS study concluded that there is no statistical difference between the compressive strength of cores and cylinders cured under identical curing conditions.[3] The literature search focused on obtaining information related to these processes, as well as the typical standard deviations associated with the different strength sampling methods.

### Variation of Concrete Compressive Strength

A discussion of the overall variation of in-place concrete core strengths in structures was included in a draft report prepared by the American Concrete Institute (ACI) Committee 214.[4] This report referenced findings from a previous study which reported coefficients of variation (COV) for structures as a function of one or many members, and one or many batches of concrete.[5] These findings are presented in table 1. Although these reported COV's are specific to structures, the core COV expected in a PCC pavement is most likely close to the 13.0-percent value associated with *many members* and *many cast-in-place batches.*

The ACI Committee 214 report also includes a section dealing with the conversion of core strengths to equivalent in-place strengths (again, focusing on structures).[4] The report presents an equation that is used to calculate the equivalent compressive

Table 1. Coefficient of variation due to in-place strength variation within the structure.[4]

| Structure Composed of: | | One Member | Many Members |
|---|---|---|---|
| One Batch of Concrete | | 7% | 8% |
| Many Batches of Concrete | Cast-in-place | 12% | 13% |
| | Precast | 9% | 10% |

strength ($f'_{C\,(EQUIVALENT)}$) in terms of measured core compressive strength ($f_{CORE}$) and four correction factors. The correction factors account for the effects of core length-to-diameter ratio ($F_{l/d}$), core diameter ($F_{DIAMETER}$), moisture condition of the core ($F_{MOISTURE}$), and damage sustained during drilling ($F_{DAMAGE}$). The correcting equation is defined in equation 1.

$$f'_{C\,(EQUIVALENT)} = F_{l/d} * F_{DIAMETER} * F_{MOISTURE} * F_{DAMAGE} * f_{CORE} \quad (1)$$

The report also contained a table of guidelines for estimating the specific correction factors required in the equation. These values are shown in table 2.

Table 2. Magnitude and precision of strength correction factors for converting core strengths into equivalent in-place strengths.[4]

| Factor | Mean Value | COV, % |
|---|---|---|
| $F_{l/d}$: l/d ratio | | |
| soaked | $1 - \{0.117 - 4.3 * (10^{-4}) * f_{core}\}(2 - l/d)^2$ | $2.5 * (2 - l/d)^2$ |
| air dried | $1 - \{0.144 - 4.3 * (10^{-4}) * f_{core}\}(2 - l/d)^2$ | $2.5 * (2 - l/d)^2$ |
| $F_{dia}$: core diameter | | |
| 50 mm | 1.06 | 11.8 |
| 100 mm | 1.00 | 0.0 |
| 150 mm | 0.98 | 1.8 |
| $F_{mc}$: core moisture content | | |
| soaked | 1.09 | 2.5 |
| air dried | 0.96 | 2.5 |
| $F_d$: damage due to drilling | 1.06 | 2.5 |

Notes: [1] To obtain the equivalent in-place concrete strength, multiply the observed core strength by the appropriate factor(s) in accordance with equation 1.

[2] $f_{core}$ is expressed in terms of MPa.

[3] Standard *soaked* and *air dried* conditions are specified in American Society for Testing and Materials (ASTM) C 42.

Core data for one pavement project in Indiana were presented in the 1996 Hughes report titled *Variability in Highway Pavement Construction*.[6] This 28-day core compressive strength data consisted of 63 cores taken over a 6-week period. A

statistical analysis of the data resulted in a mean and standard deviation of 35.1 and 6.0 MPa, respectively. The COV was then calculated to be 17.0 percent for this project.

A 1995 Illinois Department of Transportation (IDOT) report presented structural and pavement strength data (contained in IDOT's *MISTIC* database).[7] Both structural and pavement data were included in the report because most of the mix designs were "indistinguishable." Each calculated variability is specific to one mix design, one construction year, and in some cases, one producer. Table 3 contains a summary of the reported concrete cylinder compressive strength variability data.

Table 3. Concrete cylinder 28-day compressive strength variability data from the Illinois Department of Transportation MISTIC database.[7]

| Mix Number | Class | Year | n | Mean, MPa | Std Dev, MPa | COV, % |
|---|---|---|---|---|---|---|
| 1 | pavement | 1992 | 110 | 30.81 | 2.69 | 8.7 |
| 2 | pavement | 1992 | 38 | 28.31 | 3.87 | 13.7 |
| 3 | pavement | 1993 | 16 | 34.70 | 2.69 | 7.8 |
| 4 | pavement | 1992 | 82 | 33.47 | 5.51 | 16.5 |
| 5 | pavement | 1993 QA/QC | 10 | 33.21 | 3.53 | 10.6 |
| 6 | pavement | 1993 Non-QA/QC | 8 | 31.32 | 3.84 | 12.3 |
| 7 | pavement | 1992 Non-QA/QC | 12 | 33.14 | 3.44 | 10.4 |
| 8 | pavement | 1991 Non-QA/QC | 18 | 29.88 | 2.75 | 9.2 |
| 9 | pavement | 1993 QA/QC | 31 | 35.23 | 6.27 | 17.6 |
| 10 | pavement | 1992 | 23 | 34.51 | 2.92 | 8.5 |
| 11 | pavement | 1993 | 15 | 35.11 | 1.69 | 4.8 |
| 12 | pavement | 1992 Non-QA/QC | 28 | 35.88 | 5.58 | 15.5 |
| 13 | structural | 1992 | 29 | 33.07 | 4.63 | 14.0 |
| 14 | structural | 1993 | 23 | 35.20 | 4.01 | 11.4 |
| 15 | structural | 1992 | 94 | 32.76 | 4.61 | 14.0 |
| 16 | structural | 1993 | 30 | 33.27 | 5.73 | 17.2 |
| 17 | structural | 1992 | 110 | 30.81 | 2.69 | 8.7 |
| 18 | structural | Unknown | 40 | 35.80 | 6.34 | 17.7 |
| 19 | structural | Unknown | 11 | 34.58 | 9.16 | 23.9 |
| 20 | structural | 1993 QA/QC | 34 | 29.61 | 4.72 | 16.0 |
| Pavement Data Only Averages | | | | 32.96 | 3.73 | 11.3 |
| Structural Data Only Averages | | | | 33.14 | 5.23 | 15.4 |
| All Data (Pavement and Structural) Averages | | | | 33.03 | 4.33 | 12.9 |

QA/QC = Quality assurance/quality control.

An analysis of the compressive strength data in table 3 shows little difference between the averages of the pavement and structural 28-day cylinder compressive strength means (32.96 MPa for pavement and 33.14 MPa for structural concrete). The

data did show, however, that greater variation (a higher standard deviation) was typically associated with the structural data (average structural standard deviation of 5.23 MPa, compared to an average pavement standard deviation of 3.73 MPa).

A 1972 paper by Darter specifically discussed compressive strength variability.[8] He presented results from a West Virginia study in which the variation from materials, sampling, and testing was investigated for seven different projects. Table 4 contains highlighted data from the West Virginia study.

Table 4. Components of 28-day compressive strength variability for seven West Virginia projects.[8]

| | | Overall | | Testing | | Sampling | | Materials | |
|---|---|---|---|---|---|---|---|---|---|
| Project Number | Mean | Std Dev, MPa | COV, % | Std Dev, MPa | COV, % | Std Dev, MPa | COV, % | Std Dev, MPa | COV, % |
| Paving Concrete | | | | | | | | | |
| 1 | 32.23 | 3.76 | 11.7 | 2.60 | 8.1 | 0.63 | 0 | 2.73 | 8.3 |
| 2 | 25.89 | 2.90 | 11.2 | 2.22 | 8.5 | 0.29 | 0 | 1.86 | 7.1 |
| 3 | 25.65 | 3.96 | 15.5 | 2.19 | 8.5 | — | — | 3.41 | 13.3 |
| 4 | 32.82 | 3.22 | 9.8 | 1.38 | 4.2 | 0.23 | 0 | 2.90 | 8.8 |
| 5 | 32.32 | 5.05 | 16.5 | 4.03 | 12.5 | — | — | 3.76 | 11.7 |
| Structural Concrete | | | | | | | | | |
| 1 | 29.20 | 3.00 | 10.0 | 1.17 | 4.0 | 1.63 | 5.6 | 2.14 | 7.2 |
| 2 | 30.47 | 3.32 | 10.9 | 2.23 | 7.3 | 0.27 | 0 | 2.48 | 8.1 |
| Paving Data Only Averages | | 3.78 | 12.9 | 2.48 | 8.4 | 0.39 | — | 2.93 | 9.8 |
| Structural Data Only Averages | | 3.16 | 10.5 | 1.70 | 5.7 | 0.95 | 2.8 | 2.31 | 7.7 |
| All Data (Pavement and Structural) Averages | | 3.60 | 12.2 | 2.26 | 7.6 | 0.61 | 0.8 | 2.75 | 9.2 |

The data from these seven projects show an average overall project standard deviation of 3.60 MPa and a corresponding overall project average COV of 12.2 percent. Darter noted that the sampling and testing error make up a significant portion of the overall variation, and the materials COV is about three-fourths of the overall COV. Figure 1 illustrates the relationships among the materials, sampling, and testing standard deviations. It is important to note that the standard deviations are not additive; therefore, the sum of the standard deviation shown does not equal the overall standard deviation of the concrete strength.[9]

Similar data was presented in a 1973 report by Di Cocco.[10] He analyzed 28-day cylinder compressive test data from nine projects (the data included concrete placed

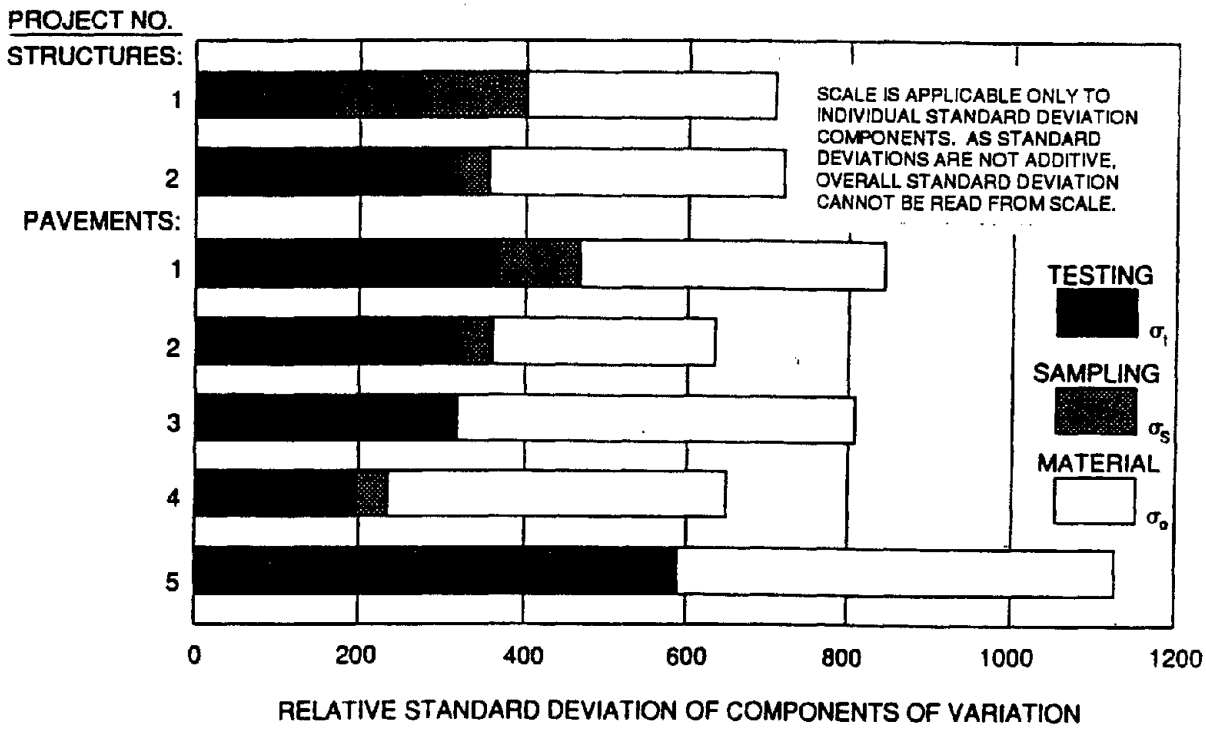



Figure 1. PCC relative standard deviation of components of variation for compressive strength.[9]

using three different mixer types—central mixer, paver, and truck mixer). The results showed an overall standard deviation of 4.98 MPa for the nine projects.

Darter also combined 28-day cylinder compressive strength data from 11 different sources—a total of 56 projects in all. It is important to note that the data are comprised of test values in which *one test* represents the average of three replicate cylinder tests. The data collected for these 56 projects are presented in table 5. The data show an average overall project standard deviation of 3.72 MPa (averages of three cylinder tests) and an average overall project COV of 12.1 percent.

The standards for 28-day cylinder compressive test COV (recommended by the American Concrete Institute [ACI] in 1972) were also presented in the 1972 paper by Darter.[8] These standards are summarized in table 6. A comparison of the summary results from the 56 projects (COV of 12.1 percent) shows that the average observed project quality falls in the "Good" range. In the conclusions of his 1972 report, Darter recommends that a compressive strength COV of 15 percent be used for design.

Neaman and Laguros presented 28-day cylinder compressive strength results from a 12.9-km project in Oklahoma.[11] Two-hundred pairs of cylinders (400 total cylinders) were tested for the project, resulting in an overall 28-day cylinder compressive strength mean and standard deviation of 26.22 and 4.97 MPa, respectively. A histogram of this strength data is presented in figure 2.

Table 5. Summary of 28-day cylinder compressive strength statistics of various projects.[8]

| Project Number | Class | No. of Tests | Mean, MPa | Std. Dev., MPa | COV, % |
|---|---|---|---|---|---|
| 1 | pavement | 536 | 31.19 | 2.73 | 8.8 |
| 2 | pavement | 292 | 33.65 | 3.72 | 11.1 |
| 3 | pavement | 96 | 39.20 | 3.75 | 9.6 |
| 4 | pavement | 192 | 38.11 | 3.90 | 10.2 |
| 5 | pavement | 196 | 35.15 | 3.98 | 11.3 |
| 6 | pavement | 112 | 33.27 | 4.19 | 12.6 |
| 7 | pavement | 258 | 37.71 | 4.60 | 12.2 |
| 8 | pavement | 320 | 36.16 | 4.65 | 12.9 |
| 9 | pavement | 232 | 36.47 | 4.90 | 13.4 |
| 10 | pavement | 176 | 33.97 | 5.00 | 14.7 |
| 11 | pavement | 126 | 34.94 | 5.05 | 14.4 |
| 12 | pavement | 46 | 24.13 | 3.19 | 13.2 |
| 13 | pavement | 12 | 28.90 | 3.52 | 12.2 |
| 14 | pavement | 12 | 28.10 | 2.18 | 7.8 |
| 15 | pavement | 15 | 29.65 | 1.92 | 6.5 |
| 16 | pavement | 14 | 37.48 | 4.38 | 11.7 |
| 17 | pavement | 22 | 28.33 | 3.79 | 13.4 |
| 18 | pavement | 27 | 36.41 | 3.30 | 9.0 |
| 19 | pavement | 31 | 30.34 | 2.89 | 9.5 |
| 20 | pavement | 24 | 30.45 | 2.75 | 9.0 |
| 21 | pavement | 14 | 27.53 | 6.74 | 24.5 |
| 22 | pavement | 25 | 28.46 | 1.99 | 7.0 |
| 23 | pavement | 10 | 26.82 | 3.56 | 13.3 |
| 24 | pavement | 15 | 31.76 | 3.31 | 10.4 |
| 25 | pavement | 13 | 26.73 | 2.68 | 10.0 |
| 26 | pavement | 93 | 29.75 | 3.56 | 12.0 |
| 27 | pavement | 82 | 33.74 | 3.01 | 9.0 |
| 28 | pavement | 845 | 33.45 | 5.12 | 15.3 |

Note: One test usually indicates the average of three replicate cylinders.

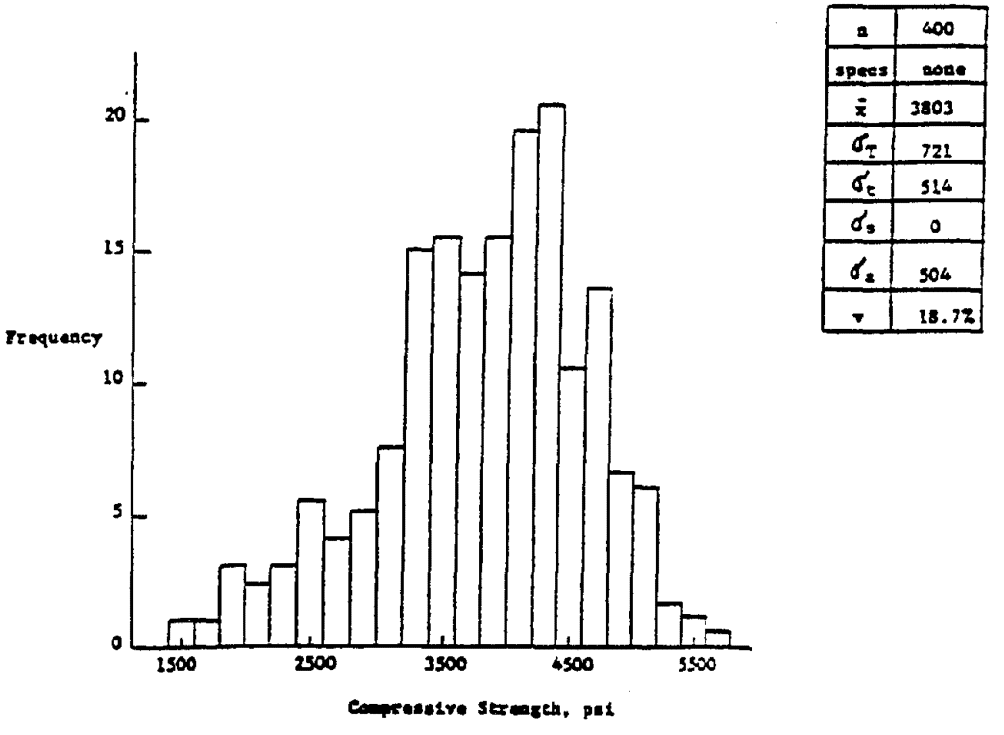
Table 5. Summary of 28-day cylinder compressive strength statistics of various projects (continued).[8]

| Project Number | Class | No. of Tests | Mean, MPa | Std. Dev., MPa | COV, % |
|---|---|---|---|---|---|
| 29 | pavement | 5318 | 31.35 | 4.36 | 13.9 |
| 30 | pavement | 8468 | 29.35 | 4.29 | 14.6 |
| 31 | pavement | 146 | 33.17 | 4.58 | 13.8 |
| 32 | pavement | 39 | 29.79 | 5.01 | 16.8 |
| 33 | pavement | 68 | 15.58 | 2.10 | 13.5 |
| 34 | pavement | 208 | 36.19 | 3.38 | 9.3 |
| 35 | pavement | 861 | 35.71 | 3.72 | 10.4 |
| 36 | pavement | 296 | 40.33 | 10.62 | 25.6 |
| 37 | pavement | 51 | 19.51 | 2.79 | 14.3 |
| 38 | pavement | 44 | 20.13 | 3.19 | 15.8 |
| 39 | pavement | 153 | 26.06 | 2.81 | 10.8 |
| 40 | pavement | 87 | 23.37 | 3.80 | 16.2 |
| 41 | pavement | 81 | 25.30 | 3.09 | 12.2 |
| 42 | pavement | 89 | 24.13 | 2.50 | 10.4 |
| 43 | pavement | 61 | 22.61 | 3.30 | 14.6 |
| 44 | pavement | 19 | 32.41 | 2.09 | 6.4 |
| 45 | pavement | 23 | 41.09 | 2.39 | 5.8 |
| 46 | pavement | 33 | 25.17 | 2.16 | 8.6 |
| 47 | pavement | 74 | 36.34 | 4.62 | 12.7 |
| 48 | pavement | 44 | 30.27 | 4.01 | 13.3 |
| 49 | pavement | 14 | 35.85 | 2.94 | 8.2 |
| 50 | pavement | 28 | 34.68 | 4.41 | 12.7 |
| 51 | pavement | 47 | 17.75 | 2.28 | 12.8 |
| 52 | pavement | 63 | 22.25 | 2.96 | 13.3 |
| 53 | pavement | 63 | 29.18 | 3.40 | 11.6 |
| 54 | pavement | 187 | 35.64 | 4.23 | 11.9 |
| 55 | pavement | 65 | 33.25 | 3.67 | 11.1 |
| 56 | pavement | 90 | 40.55 | 5.12 | 12.6 |
| Average | — | — | 30.87 | 3.72 | 12.1 |

Note: One test usually indicates the average of three replicate cylinders.

Table 6. ACI 1972 recommended standards for 28-day cylinder compressive test COV.[8]

| Class of Operation | Coefficients of Variation for Different Control Standards | | | |
|---|---|---|---|---|
| | Excellent | Good | Fair | Poor |
| Overall variations: | | | | |
| General construction | Below 10.0 | 10.0 to 15.0 | 15.0 to 20.0 | Above 20.0 |
| Laboratory control | Below 5.0 | 5.0 to 7.0 | 7.0 to 10.0 | Above 10.0 |
| Within-test variations: | | | | |
| Field control | Below 4.0 | 4.0 to 5.0 | 5.0 to 6.0 | Above 6.0 |
| Laboratory control | Below 3.0 | 3.0 to 4.0 | 4.0 to 5.0 | Above 5.0 |




1 psi = 6.9 kPa

Figure 2. Histogram of 28-day cylinder compressive strength data for a project constructed in Oklahoma (200 pairs of 2 cylinders).[11]

In a 1992 report, Wood presented tables of data showing normalized compressive strength means and standard deviations for different ages.[12] In addition to age, the report also looked at the influence of curing condition and type of cement. Concrete compressive strength values at 1, 3, and 7 days were normalized by dividing by the corresponding 28-day compressive strength. These results, for the four different curing conditions, are presented in tables 7 through 10.

Table 7. Variation of normalized concrete compressive strength with age—moist curing—portland cements.[12]

| Age at Test | Type of Cement | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | | | II | | | III | | | IV | | |
| | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa |
| Moist Curing | | | | | | | | | | | | |
| 1 day | 46 | 0.18 | 0.07 | 13 | 0.16 | 0.04 | 10 | 0.30 | 0.11 | 11 | 0.13 | 0.03 |
| 3 days | 46 | 0.46 | 0.10 | 3 | 0.35 | 0.02 | 24 | 0.62 | 0.11 | 3 | 0.24 | 0.03 |
| 7 days | 64 | 0.70 | 0.09 | 13 | 0.61 | 0.08 | 28 | 0.81 | 0.08 | 11 | 0.48 | 0.06 |
| 28 days | 64 | 1.0 | — | 13 | 1.0 | — | 28 | 1.0 | — | 11 | 1.0 | — |

Note: Data from 152-mm modified cubes and 152- x 305-mm cylinders are included.

Table 8. Variation of normalized concrete compressive strength with age—moist curing—blended cements.[12]

| Age at Test | Type of Cement | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | IS | | | IP | | | IA | | |
| | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa |
| Moist Curing | | | | | | | | | |
| 1 day | 20 | 0.22 | 0.07 | 15 | 0.10 | 0.03 | 9 | 0.14 | 0.06 |
| 3 days | 20 | 0.42 | 0.09 | 15 | 0.32 | 0.09 | — | — | — |
| 7 days | 20 | 0.62 | 0.10 | 15 | 0.53 | 0.11 | 9 | 0.69 | 0.09 |
| 28 days | 20 | 1.0 | — | 15 | 1.0 | — | 9 | 1.0 | — |

Note: Data from 152-mm modified cubes and 152- x 305-mm cylinders are included.

Table 9. Variation of normalized concrete compressive strength with age—air curing.[12]

| Age at Test | Type of Cement | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | I | | | III | | | IS | | | IP | | |
| | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa |
| Air Curing | | | | | | | | | | | | |
| 1 day | 10 | 0.16 | 0.08 | 6 | 0.24 | 0.08 | 20 | 0.21 | 0.06 | 15 | 0.10 | 0.03 |
| 3 days | 19 | 0.44 | 0.10 | 15 | 0.56 | 0.13 | 20 | 0.40 | 0.08 | 15 | 0.33 | 0.08 |
| 7 days | 19 | 0.67 | 0.09 | 15 | 0.78 | 0.10 | 20 | 0.59 | 0.08 | 15 | 0.54 | 0.10 |
| 28 days | 19 | 1.0 | — | 15 | 1.0 | — | 20 | 1.0 | — | 15 | 1.0 | — |

Note: Data from 152-mm modified cubes and 152- x 305-mm cylinders are included.

Table 10. Variation of normalized concrete compressive strength with age—outdoor exposure.[12]

| Age at Test | Type of Cement | | | | | |
|---|---|---|---|---|---|---|
| | I | | | III | | |
| | No. | Mean, MPa | Std. Dev., MPa | No. | Mean, MPa | Std. Dev., MPa |
| Outdoor Exposure | | | | | | |
| 1 day | — | — | — | — | — | — |
| 3 days | 21 | 0.53 | 0.10 | 21 | 0.69 | 0.15 |
| 7 days | 21 | 0.78 | 0.09 | 21 | 0.88 | 0.12 |
| 28 days | 21 | 1.0 | — | 21 | 1.0 | — |

Note: Data from 152-mm modified cubes.

A large amount of cylinder compressive strength data was collected at a 1993 paving project in Colorado.[13] The project consisted of 25.7 km of paving on I-70 near the town of Vona. A mean and standard deviation of 37.02 and 3.14 MPa, respectively, were calculated for the 436 cylinders tested for 28-day compressive strength. A histogram showing the testing results is presented in figure 3.

An article published by the American Concrete Pavement Association (ACPA) in 1995 discussed the typical 28-day compressive strength standard deviation expected from ready-mix plants.[14] It reported that historical records from past plant operations showed that this standard deviation could be expected to be less than 2.76 MPa for concrete coming from a well-controlled ready-mix operation. In addition, it reported that ready-mix concrete historically has COV's in the 7- to 13-percent range, whereas COV's for central-mix concrete generally fall in the 5- to 12-percent range.

A 1990 report contained a reprint of an article by McMahon et al. that discussed the quality assurance (QA) of PCC pavement.[9] The authors reported the following information related to concrete strength:

- Strength is not always the most important characteristic of concrete quality, but it is the one that is most often measured. It is assumed to be indicative of the water-cement ratio and, accordingly, an indicator of durability. The magnitude of the variability in strength, is, therefore, an indicator of the magnitude of variability of the other characteristics.

- Variability in concrete strengths can be attributed to two other types of variability: (1) Inherent variability in the materials and processes that results from chance causes, which cannot be controlled, and (2) variability from assignable causes that can be controlled.

- The measurement of variability from chance or inherent causes is complicated by the inherent variability of each of the ingredients in the mix, which can interact with the processes of blending, mixing, and placing, and result in a much larger variability in the concrete itself.

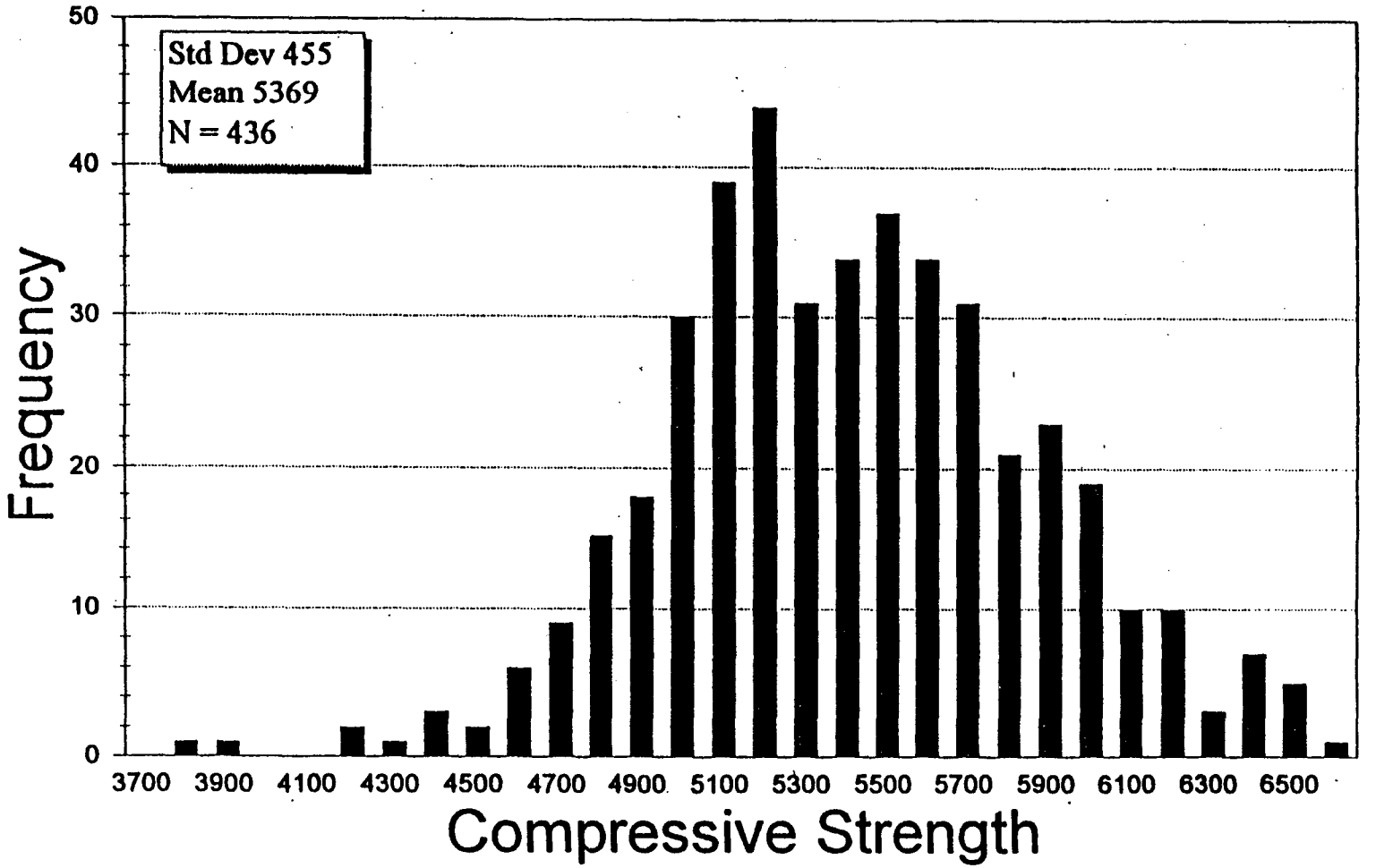



Figure 3. Histogram of 28-day cylinder compressive strength results—1993 project on I-70 near Vona, Colorado.[13]

- The assignable causes of variability are more numerous and more difficult to isolate, but the production of quality concrete is dependent on the reduction of all variables. However, the isolation and restriction of variables can be carried only as far as economic conditions warrant. Under the present state of knowledge, the ultimate uniformity of concrete production cannot be precisely stated. Extensive research will be necessary to isolate variables and to determine the extent to which variation can be reduced.

McMahon et al. present variability data from many different published sources in their article. Table 11 summarizes their results.

Table 11. Average deviation of 28-day concrete compressive strengths.[9]

| Agency | Concrete Type | Data Source | Average Standard Deviation, MPa |
|---|---|---|---|
| Bureau of Public Roads | Paving | Research | 4.03 |
| Bureau of Public Roads | Paving | Historical | 3.26 |
| Bureau of Public Roads | Structural | Historical | 3.97 |
| Virginia Dept of Highways | Structural | Special | 3.22 |
| Virginia Dept of Highways | Paving | Cores | 4.57 |
| Ontario Dept of Highways | Paving | Routine | 3.41 |

Note: The average standard deviation is for all data presented.

Results of an analysis of historical cylinder compressive strengths by the State Road Department of Florida were also presented in the article by McMahon et al.[9] These results included data from two different concrete types (class A and class NS). It was noted that "routine control is normally exercised over class A concrete, whereas class NS concrete is spot checked only occasionally." Tables 12 and 13 contain the results from the Florida study (table 12 presents the results by project, and table 13 presents the results by concrete production source).

A 1966 paper by Steele et al. presented a summary of data collected for two projects in West Virginia.[15] A summary of the 28-day cylinder compressive strength results (reported for both projects) is presented in table 14. The authors of this report point out that although a typical *project* standard deviation could be estimated to be around 3.17 MPa, it could range from 2.41 to 4.48 MPa. A calculated standard deviation for one *lot* may even be higher.

A literature review conducted by ERES Consultants, Inc. for a report titled *Measurement and Specification of Construction Quality* included six different figures showing normal distributions of different types of PCC strength data.[16] These are presented in figures 4 through 9.

Table 12. Historical concrete compressive strength data published by the State Road Department of Florida—project comparison.[9]

| Project Number | Number of Samples | Mean, MPa | Overall Std. Dev., MPa | COV, % | Testing Error, MPa |
|---|---|---|---|---|---|
| 28-day cylinders—class A concrete | | | | | |
| 1[1] | 536 | 31.19 | 2.73 | 8.8 | 1.21 |
| 2 | 292 | 33.65 | 3.72 | 11.1 | 1.43 |
| 3 | 96 | 39.20 | 3.75 | 9.6 | 1.28 |
| 4 | 192 | 38.11 | 3.90 | 10.2 | 1.07 |
| 5 | 196 | 35.15 | 3.98 | 11.3 | 0.33[2] |
| 6 | 112 | 33.27 | 4.19 | 12.6 | 1.35 |
| 7 | 258 | 37.71 | 4.60 | 12.2 | 1.03 |
| 8 | 320 | 36.16 | 4.65 | 12.9 | 1.09 |
| 9 | 232 | 36.47 | 4.90 | 13.4 | 1.32 |
| 10 | 176 | 33.97 | 5.00 | 14.7 | 1.45 |
| 11 | 126 | 34.94 | 5.05 | 14.4 | 1.12 |
| Average[3] | 230 | 35.44 | 4.23 | 13.1 | 1.23 |
| Range | 224 | 5.93 | 1.32 | 5.1 | 0.41 |
| 28-day cylinders—NS concrete | | | | | |
| 1[1] | 50 | 27.72 | 2.74[1] | 9.9 | 115 |
| 2 | 340 | 24.51 | 3.79 | 15.5 | 93 |
| 3 | 240 | 27.62 | 4.00 | 14.5 | 134 |
| 4 | 200 | 23.95 | 4.17 | 17.4 | 122 |
| 5 | 240 | 26.07 | 4.62 | 17.7 | 96 |
| 6 | 196 | 28.90 | 5.03 | 17.4 | 160 |
| 7 | 148 | 29.05 | 5.05 | 17.4 | 92 |
| 8 | 94 | 29.23 | 5.34 | 18.3 | 87 |
| 9 | 108 | 25.21 | 5.34 | 21.2 | 116 |
| 10 | 182 | 25.33 | 5.35 | 21.1 | 113 |
| 11 | 156 | 28.34 | 5.35 | 18.9 | 70 |
| 12 | 224 | 28.81 | 5.69 | 19.8 | 127 |
| 13 | 138 | 27.17 | 6.09 | 22.4 | 161 |
| Average[3] | 178 | 27.07 | 4.81 | 17.8 | 114 |
| Range | 246 | 5.27 | 2.30 | 7.9 | 91 |

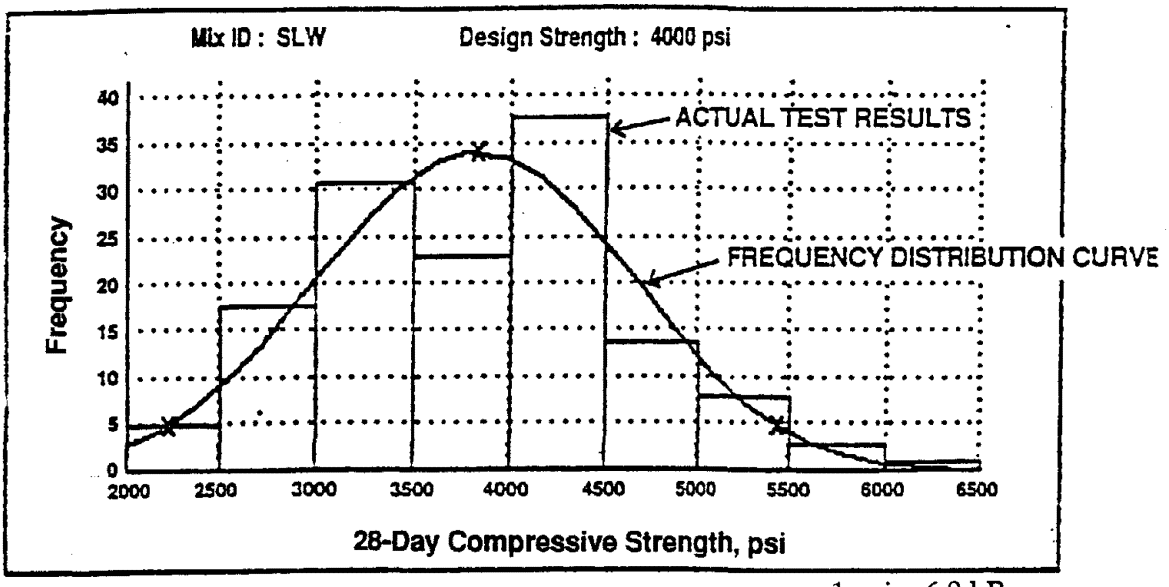
Notes: [1] Values are not included in range calculations.
[2] Statistical outlier—not included in calculation of range or average.
[3] Averages are not weighted and include all values except the outlier.

Table 13. Historical concrete compressive strength data published by the State Road Department of Florida—production source comparison.[9]

| Source | Concrete Class | Number of Samples | Mean Strength, MPa | Pooled Std. Dev., MPa | COV, % |
|---|---|---|---|---|---|
| 1 | A | 480 | 33.09 | 4.09 | 12.4 |
| | NS | 218 | 28.21 | 4.27 | 15.2 |
| 2 | A | 540 | 33.83 | 4.63 | 13.7 |
| | NS | 360 | 29.03 | 4.55 | 15.7 |
| 3 | A | 440 | 34.85 | 4.03 | 11.6 |
| | NS | 308 | 27.79 | 4.03 | 14.5 |

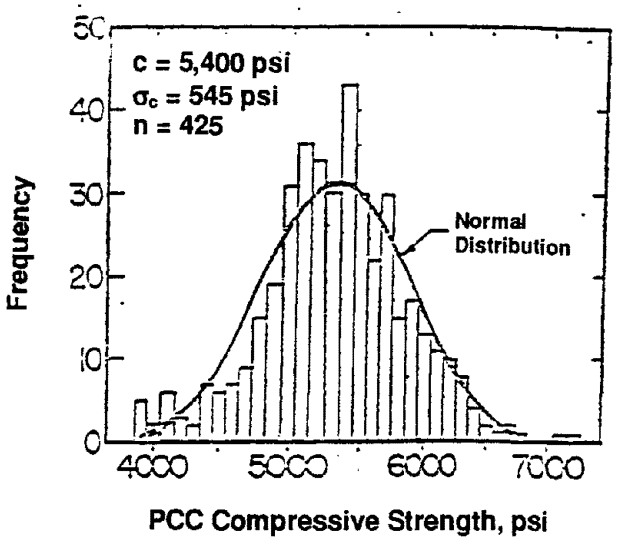
Table 14. Strength parameters for Class A structural concrete—two projects in West Virginia.[15]

| Project | Number of Tests | Avg 28-day Compressive Strength, MPa | Standard Deviation Individuals, MPa |
|---|---|---|---|
| West Virginia—Project A | 190 | 29.20 | 3.00 |
| West Virginia—Project B | 200 | 30.47 | 3.32 |
| Average | — | 29.84 | 3.17 |



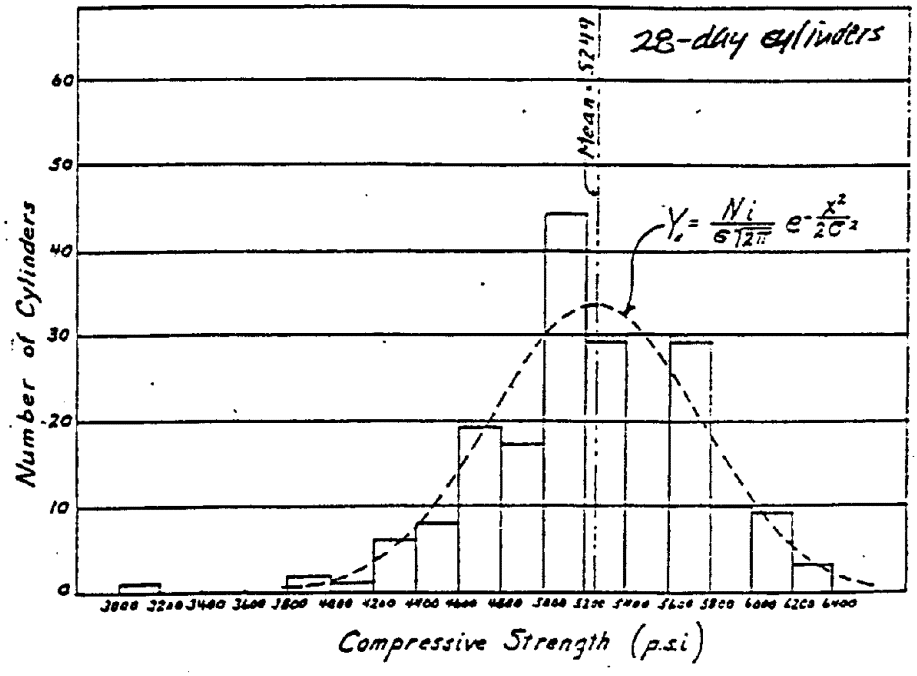

1 psi = 6.9 kPa

Figure 4. Histogram of 28-day concrete compressive strength.[16]



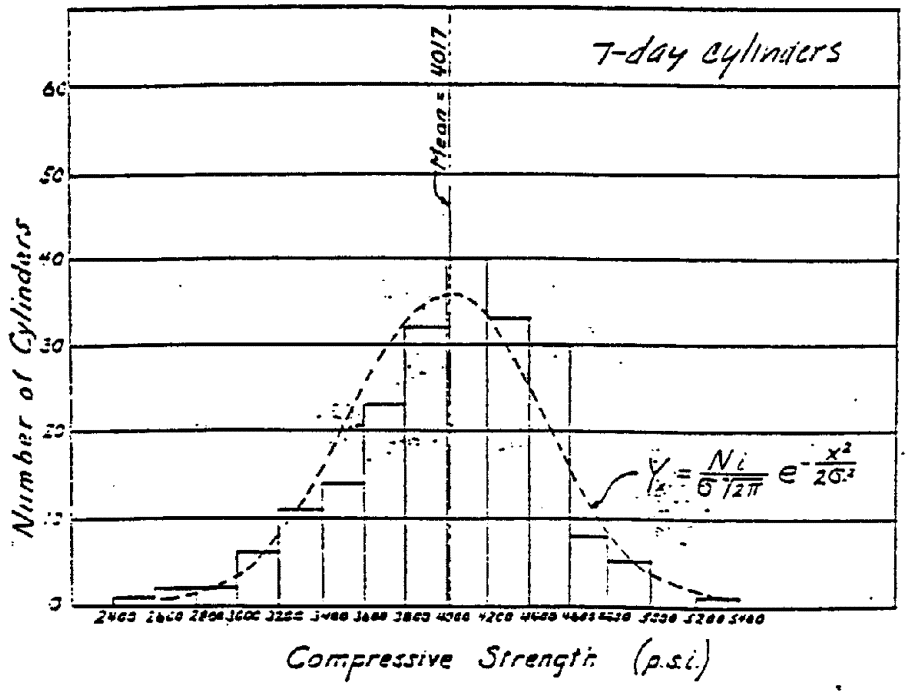

1 psi = 6.9 kPa

Figure 5. Distribution of PCC compressive strength.[16]



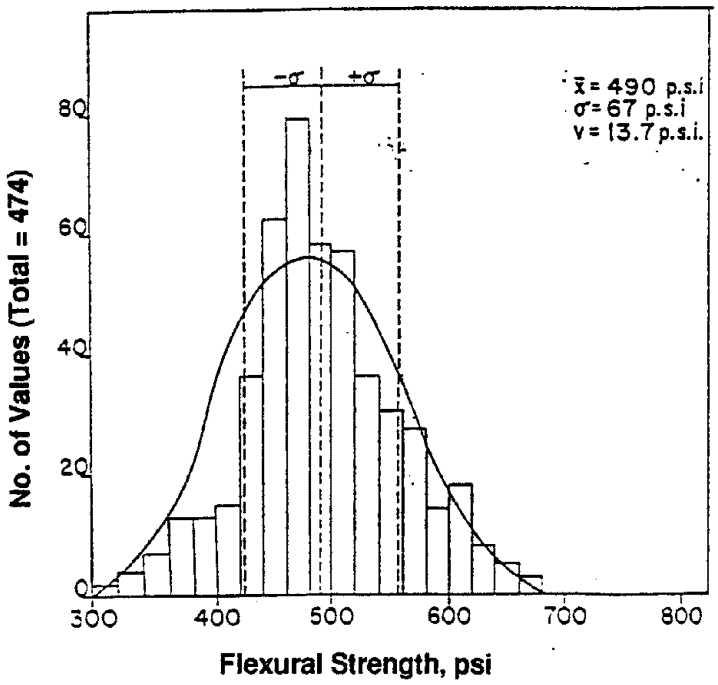

1 psi = 6.9 kPa

Figure 6. Histogram of 28-day cylinder concrete compressive strength.[16]



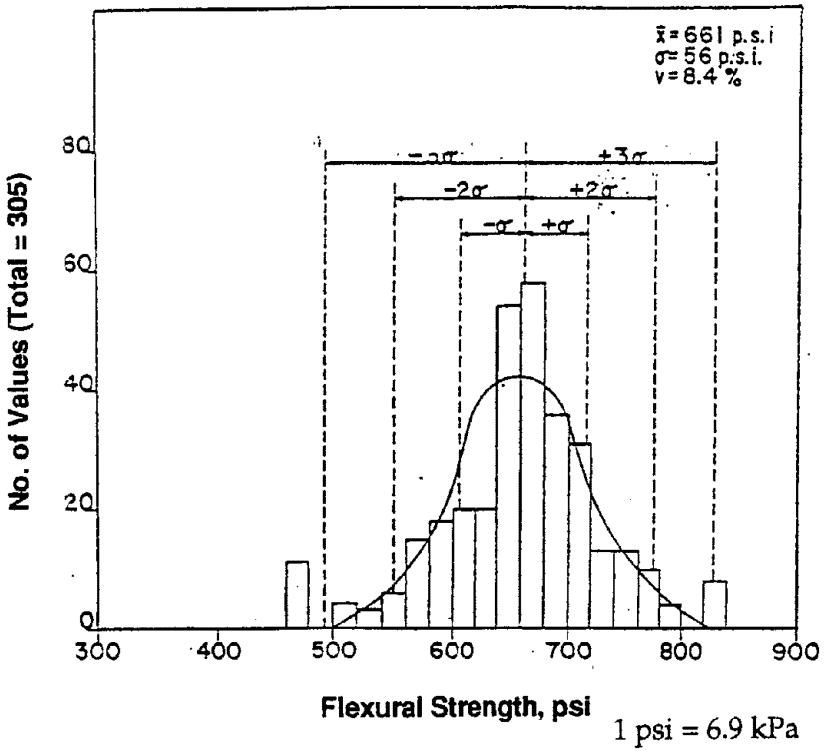

1 psi = 6.9 kPa

Figure 7. Histogram of 7-day cylinder concrete compressive strength.[16]




1 psi = 6.9 kPa

Figure 8. Histogram of concrete flexural strength.[16]




1 psi = 6.9 kPa

Figure 9. Histogram of concrete flexural strength.[16]

Actual concrete strength compressive strength data were also collected for the Federal Highway Administration (FHWA)-sponsored project "Measurement and Specification of Construction Quality."[17] Both 28-day cylinder and core data were collected for three projects (one each for Illinois, Minnesota, and Ohio) constructed during the 1995 construction season. The results of the statistical analysis for both data sets are presented in table 15.

Table 15. 1995 28-day cylinder and core compressive strength data.[17]

| Project Location | Sample Type | No. of Samples | Mean, MPa | Std Dev, MPa | COV, % |
|---|---|---|---|---|---|
| Rochelle, IL | Cylinder | 106 | 37.40 | 5.16 | 13.8 |
| | Core | 30 | 40.12 | 3.76 | 9.4 |
| Mankato, MN | Cylinder | 56 | 36.62 | 3.36 | 9.2 |
| | Core | 30 | 41.26 | 7.38 | 17.9 |
| Bellefontaine, OH | Cylinder | 100 | 29.53 | 2.43 | 8.2 |
| | Core | 31 | 41.45 | 5.32 | 12.8 |
| Cylinder Data Only—Averages | | | | 3.65 | 10.4 |
| Core Data Only—Averages | | | | 5.49 | 13.4 |

In 1996, Hughes compiled a table of typical 28-day PCC cylinder statistics in a report titled *Variability in Highway Pavement Construction*.[6] This table included data from two published State Highway Agency (SHA) reports (from Arizona and Pennsylvania), as well as data from databases and variability studies provided with a completed questionnaire. These data are presented in table 16.

Table 16. Typical standard deviations for 28-day PCC cylinder strengths.[6]

| Source | Year | Class | Mean, MPa | Std Dev, MPa | COV, % |
|---|---|---|---|---|---|
| Georgia | 1993 | Paving | 32.89 | 3.35 | 10.2 |
| New York | 1993 | Gen Structural | 30.43 | 5.21 | 17.3 |
| New York | 1993 | Thin Structural | 33.27 | 5.36 | 16.3 |
| New York | 1993 | Pumping | 34.74 | 5.39 | 15.8 |
| Maine | 1992 | Gen Structural | 35.66 | 4.05 | 11.4 |
| Maine | 1991 | Gen Structural | 34.12 | 4.55 | 13.3 |
| Virginia | 1988 | Gen Structural | 33.40 | 4.55 | 13.6 |
| Virginia | 1988 | High Strength | 39.76 | 4.37 | 11.2 |
| Arizona | 1974 | Paving | 30.16 | 4.15 | 13.8 |
| Pennsylvania | 1972 | Gen Structural | 32.06 | 4.83 | 15.1 |

Note: Standard deviation based on single 28-day cylinders.

Hughes also reported that the New Jersey DOT's typical standard deviation associated with a lot is approximately 2.07 MPa.

### Variation of Concrete Flexural Strength

Flexural strength data from the Illinois Department of Transportation MISTIC database were summarized and presented in a 1995 report.[7] The results are shown in table 17. An analysis of the data shows a typical average beam flexural strength standard deviation of 0.44 MPa and an average COV of 8.1 percent.

Table 17. Concrete beam 28-day flexural strength variability data from the Illinois Department of Transportation MISTIC Database.[7]

| Mix Number | Class | Year | n | Mean, MPa | Std Dev, MPa | COV, % |
|---|---|---|---|---|---|---|
| 1 | Structural | Unknown | 18 | 5.63 | 0.60 | 10.7 |
| 2 | Structural | Unknown | 129 | 5.41 | 0.34 | 6.4 |
| 3 | Structural | Unknown | 13 | 5.35 | 0.39 | 7.3 |
| Structural Data Averages | | | | 5.46 | 0.44 | 8.1 |

Flexural stength variability data were also presented by Darter in a 1972 report.[8] This consisted of data from five Texas Highway Department projects, as well as some data from the original American Association of State Highway Officials (AASHO) Road Test. This information is summarized in table 18.

Table 18. Concrete beam 28-day flexural strength variability data from the Texas Highway Department and the AASHO Road Test.[8]

| Project Number | Number of Tests | Mean, MPa | Std Dev, MPa | COV, % |
|---|---|---|---|---|
| Texas Highway Department Data | | | | |
| 1 | 135 | 4.78 | 0.57 | 11.8 |
| 2 | 305 | 4.56 | 0.39 | 8.4 |
| 3 | 474 | 3.38 | 0.46 | 13.7 |
| 4 | 258 | 2.82 | 0.14 | 4.9 |
| 5 | 270 | 4.67 | 0.45 | 9.6 |
| AASHO Road Test Data | | | | |
| 1 | 11 | 4.55 | 0.35 | 7.7 |
| 2 | 11 | 4.90 | 0.37 | 7.5 |
| Texas Data Only—Averages | | 4.04 | 0.40 | 9.7 |
| AASHO Data Only—Averages | | 4.72 | 0.36 | 7.6 |
| All Project Data—Averages | | 4.23 | 0.39 | 9.1 |

An analysis of the flexural strength data in table 18 shows a typical average beam flexural strength standard deviation of 0.39 MPa and an average COV of 9.1 percent.

Darter does point out that the AASHO data show less variability than the Texas data (probably due to the stringent quality control procedures at the Road Test). In this report, Darter recommends that a flexural strength COV of 15 percent be used for design.[8]

In a 1996 report, Hughes presented typical flexural strength data collected from four different State studies.[6] It is interesting to note that one of these States (Indiana) presented contractor quality control data and State DOT acceptance data for the same project. The collected data are presented in table 19.

Table 19. Typical flexural strength data for different State studies.[6]

| Data Source | Test Type | Mean, MPa | Std. Dev., MPa | COV, % |
|---|---|---|---|---|
| Midwestern State #1 | mid-point loading | 5.63 | 0.72 | 12.7 |
| Midwestern State #2 | third-point loading | 4.69 | 0.48 | 10.4 |
| Pennsylvania | not reported | 5.45 | 0.69 | 12.7 |
| Indiana—Contractor Quality Control Tests | not reported | 4.52* | 0.23* | 5.0 |
| Indiana—State Acceptance Tests | not reported | 4.52* | 0.26* | 5.8 |

* Each "sample value" included in the statistical analysis represents the average of two beam breaks.

A report by Greer contained information regarding the within- and between-batch flexural strength variations in the laboratory and the field.[18] The study analyzed published data from more than 200 mixes (more than 765,000 $m^3$ of concrete) placed at the Atlanta International Airport. In the laboratory, the *within-test* standard deviation represents the amount of variation among flexural strength tests from the same batch of concrete and is an indicator of the quality of the procedures performed by the laboratory personnel. The laboratory *between-batch* variation, however, "indicates the amount of variation between different batches of the same concrete materials." In the field, the *within-test* standard deviation is an indicator of the quality of the testing procedures performed by the field testing personnel. The field *between-batch* variability indicates the quality of the production and control procedures used by the contractor and supplier. The results of the Greer study are presented in table 20.

Greer noted that both the laboratory within-test and between-batch standard deviations for both sources are independent of the measured flexural strength.[18] Therefore, it can be concluded that the standard deviation is more appropriate than the COV when evaluating flexural strength tests. Table 21 contains proposed guidelines for assessing the level of quality associated with flexural strength testing data.[18]

Table 20. Measured flexural strength variations at the Atlanta International Airport.[18]

| Variation Type | Std Dev Range, MPa | Average Std. Dev., MPa |
|---|---|---|
| 28-day laboratory within-test | Not available | 0.23 |
| 28-day laboratory between-batch | Not available | 0.19 |
| 28-day field within-test | 0.22 – 0.28 | 0.24 |
| 28-day field between-batch | 0.18 – 0.36 | 0.24 |
| 3-day field within-test | 0.17 – 0.23 | 0.20 |
| 7-day field between-batch | 0.17 – 0.33 | 0.29 |

Table 21. Proposed guidelines for assessing the level of quality associated with flexural strength testing data.[18]

| Within-Test Standard Deviation (Testing Agency) | Overall Standard Deviation (Contractor) | Level of Quality (Quality Control) |
|---|---|---|
| < 0.28 MPa | < 0.28 MPa | Excellent to Good |
| 0.28 – 0.45 MPa | 0.28 – 0.45 MPa | Good to Fair |
| > 0.45 MPa | > 0.45 MPa | Fair to Poor |

A large amount of data was collected at a 1993 Colorado paving project. The project consisted of 25.7 km of paving on I-70 near the town of Vona. Flexural strength was measured from third-point load testing (modulus of rupture) of 11 sets of 2 beams each. A mean and standard deviation of 4.69 and 0.44 MPa, respectively, were observed from the testing.[13]

An article published by ACPA in 1995 discussed the typical 28-day third-point flexural strength standard deviation expected from ready-mix plants.[14] It reported that historical records from past plant operations showed that this standard deviation could be expected to be less than 0.48 MPa for concrete coming from a well-controlled ready-mix operation.

**Variation of Concrete Split Tensile Strength**

Split tensile strength data were also collected at the 1993 Colorado paving project.[13] Split tensile strength tests were conducted on 39 sets of 2 cylinders each, resulting in an overall mean and standard deviation of 3.10 and 0.26 MPa, respectively.

**Within-Test and Between-Batch Variability**

A report by Gray reported on a study that investigated both the between- and within-laboratory variability.[19] The study looked at testing results obtained in interlaboratory or comparative testing programs that were operated in the three largest metropolitan areas of British Columbia and Alberta, Canada. The study separately analyzed laboratories that were and were not certified by the Canadian Standards Association (CSA). Figure 10 shows linear trends of within-laboratory variability versus average compressive strength for all programs. Figure 11 shows linear trends of average within-laboratory and between-laboratory variability versus average compressive strength. Figure 12 shows linear trends of between-laboratory variability versus average compressive strength for all programs. The following two conclusions were drawn from the study's results:[19]

1. The within-laboratory precision of the standard test method for determining the compressive strength of cylindrical portland cement concrete test specimens varies linearly with the compressive strength of the test concrete. As measured by the 95-percent repeatability limit (within a laboratory), this variability approaches 10 percent of the compressive strength. This implies that on the basis of test error alone, the difference in absolute value of two test results obtained in the same laboratory on the same material can be expected to exceed approximately 10 percent of the average compressive strength of the test concrete about 5 percent of the time, and there is reason to question one or both of the test results only if such a difference is found to be larger than this value.

2. The between-laboratory precision of the standard test method also varies linearly with the compressive strength of the test concrete. As expressed by the 95-percent reproducibility limit (between laboratories), this variability approaches 15 percent of the compressive strength. This implies that on the basis of test error alone (including both the within and between-laboratory components), the difference in absolute value of two test results obtained in different laboratories on the same material can be expected to exceed approximately 15 percent of the average compressive strength of the test concrete about 5 percent of the time, and there is reason to question one or both of the test results only if such a difference is found to be larger than this value.

ACI 214-89 "Recommended Practice for Evaluation of Strength Test Results of Concrete" was reviewed as part of a literature review conducted by ERES Consultants, Inc. for a report titled *Measurement and Specification of Construction Quality*.[16] ACI Guidelines for determining the quality of a laboratory, based on observed within-test COV's for compressive strength, are provided in table 22.

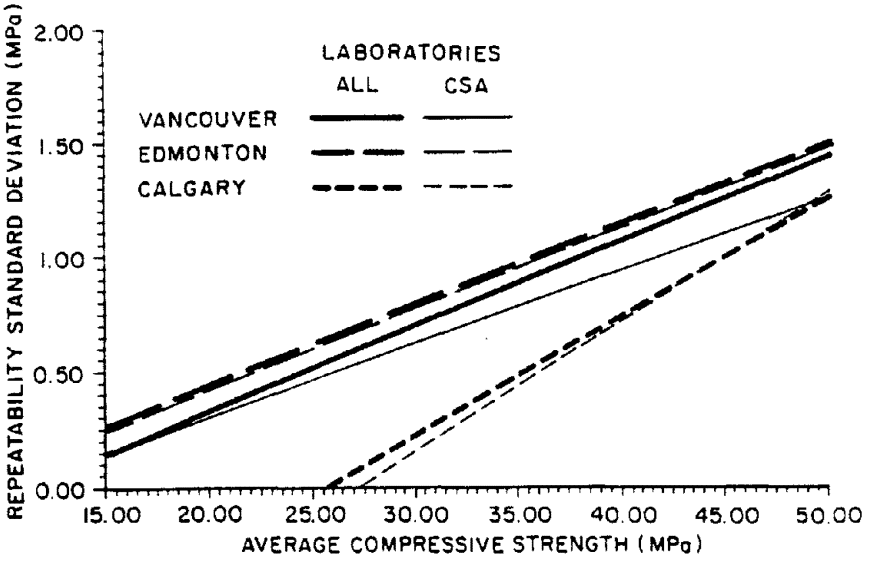



Figure 10. Repeatability (within a laboratory) standard deviation versus average compressive strength for all programs.[19]

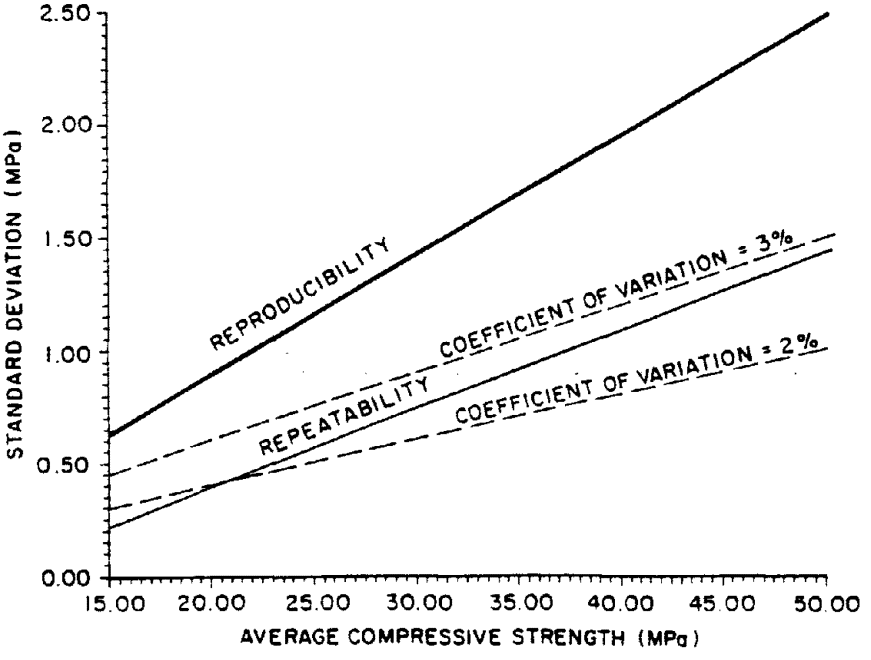



Figure 11. Average repeatability (within a laboratory) and reproducibility (between laboratories) standard deviations versus average compressive strength.[19]

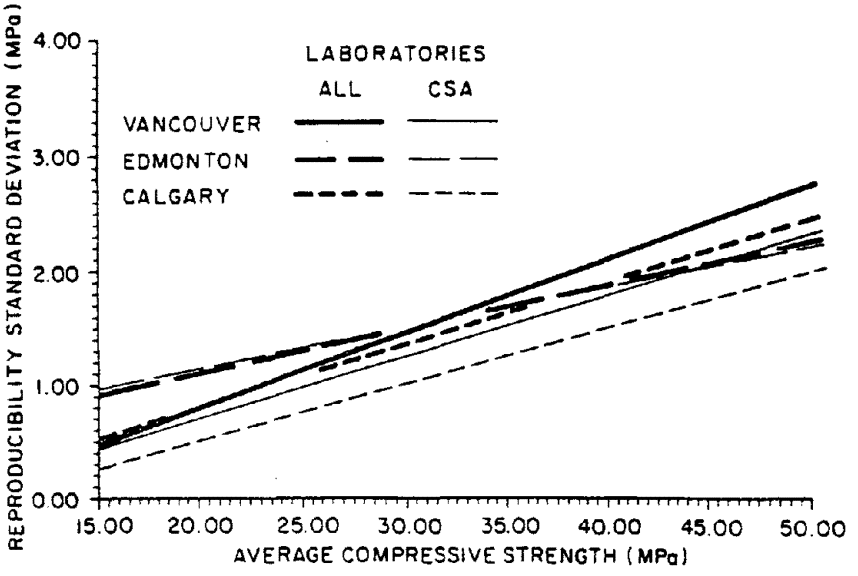



Figure 12. Reproducibility (between laboratories) standard deviation versus average compressive strength for all programs.[19]

Table 22. Compressive strength within-test COV guidelines.[16]

| Within-Test COV* | Laboratory Control |
|---|---|
| Below 3.0 | Excellent |
| 3.0 to 4.0 | Very Good |
| 4.0 to 5.0 | Good |
| 5.0 to 6.0 | Fair |
| Above 6.0 | Poor |

* Variation in compressive strength between replicate samples tested by the same operator.

Other information included in the literature review stated that typical COV values for 7-day flexural strength range from 2.8 to 17.6 percent, while COV's for 28-day flexural strength range from 3.5 to 9.6 percent.[16]

**Accelerated Testing Methods for Concrete Strength**

Much can be learned about nondestructive testing (NDT) methods for concrete strength from the fast-track paving technology. Fast-track techniques are generally used to move up the traffic open date of a newly constructed or repaired PCC

pavement. A number of different techniques have been developed to provide quick and reliable assessments of the concrete strength at early ages. One reviewed ACPA report describes the current state of fast-track paving techniques, including six nondestructive strength assessment methods that have been used.[20] A table outlining the six methods is presented as table 23. Of these six techniques, the most commonly used are the maturity and pulse-velocity methods (both of which are described in more detail in the following sections).

Table 23. Non-destructive test methods for concrete.[20]

| Test Method | Standard | Basic Description | Testing Precision to Baseline Cylinder Strength[2] |
|---|---|---|---|
| Surface Hardness (Swiss Hammer) | ASTM C 805 | Rebound of hammer correlates to surface hardness & compressive strength | ± 40 % |
| Penetration Resistance (Windsor Probe) | ASTM C 803 | Penetration depth of gun-fired probe correlates to surface hardness & compressive strength | ± 20 % |
| Pullout[1] | ASTM C 900 | Force to remove cast-in metal probe correlates to surface compressive strength | ± 15 % |
| Break-off | ASTM C 1150 | Force necessary to break a circular core cast or cut partially into slab correlates to flexural strength | ± 15 % |
| Maturity | ASTM C 1074 | Internal temperature of concrete relates directly to concrete strength | ± 5 % |
| Pulse Velocity | ASTM C 597 | Velocity of sound wave from transducer to receiver through concrete relates to concrete strength | ± 10 % |

Notes: [1] Cap and pull-out (CAPO) variation of pull-out test not approved by ASTM.
[2] These are estimates of precision based on cylinder strength tests made using recommended ASTM procedures. Inaccurate concrete strength characterization by destructive cylinder testing is a common problem.

## Maturity Method

The following is an excerpt from the above-mentioned ACPA report that describes some of the maturity concepts.[20]

> "Maturity testing provides strength evaluation through monitoring of internal concrete temperature in the field. The basis of maturity is that each concrete mix has a unique strength-time relationship. Therefore, a mix will have the same strength at a given maturity no matter what conditions (time or temperature) occur before measurement."

Two methods of maturity were introduced in the report—the *Nurse-Saul* method and the *Arrhenius maturity equation*. In the United States, the Nurse-Saul method is the more

commonly used of the two methods. The Nurse-Saul method calculates maturity as a function of time and temperature using equation 2.

$$M(t) = \Sigma(T_A - T_0)\,\Delta t \qquad (2)$$

where

$M(t)$ = Temperature-time factor, degree-days or degree-hours.
$\Delta t$ = Time interval, days or hours.
$T_A$ = Average concrete temperature during time interval, °C.
$T_0$ = Datum temperature, °C (typically –10 °C).

No specific information on the Arrhenius maturity method was provided; however, the report did refer to ASTM C 1074 as a source for more information on the method.

Estimated strength gain versus maturity is mix-specific. Therefore, laboratory testing is required to accurately assess the as-constructed slab strength. The following excerpt from the ACPA report describes the details of the required laboratory and field maturity procedures.[20]

> "Laboratory testing requires preparation of trial batches using the actual field-mix materials. Technicians must monitor the batch temperature and break cylinders to develop a relationship between the strength criterion and the temperature-time factor. This relationship becomes the calibration curve for evaluating the field concrete strength.
>
> "Field maturity evaluation begins with the embedment of thermocouples of temperature probes in the concrete when practicable after finishing or curing. The probes must connect to either commercially available maturity meters or temperature recorders with an accuracy of 1 °C. Technicians take readings at regular intervals then estimate strength using the temperature-time relationship from the laboratory study."

Pulse-Velocity Method

The pulse-velocity method estimates concrete strength or stiffness by measuring the time required for an ultrasonic wave to pass through the concrete from one transducer to another. This method is also mix-specific; therefore, laboratory testing is required to accurately assess the as-constructed slab strength. The following excerpt from the ACPA report describes the details of the required laboratory and field pulse-velocity procedures.[20]

> "Pulse-velocity testing requires laboratory calibration to produce meaningful field information. Pulse-velocity readings are sensitive to aggregate, water-cement-plus-pozzolan ratio, moisture content, and

concrete consolidation. Therefore, trial batches must contain the same mix materials at similar proportions as the project mix. In the laboratory, technicians take pulse-velocity measurements through a representative number of cast specimens, test the specimens for strength, and plot the results against the pulse-velocity readings to create a calibration curve."

Finally, pulse-velocity measurements are taken in the field following the manufacturer's recommendations for placement of the transducers. Early-age concrete strengths are then estimated using the laboratory-developed calibration curves.

**Compressive Versus Flexural Strength**

Historically, flexural strength testing has been used for acceptance and design; however, today the trend is toward using compressive strength testing for not only quality control, but acceptance as well.

Advantages of Compressive Strength Over Flexural Strength for Acceptance

The following list of the advantages of using compression test cylinders, as opposed to flexural test beams, was compiled by Kosmatka:[21]

- Cylinders are easier to make, handle, transport, and test.
- Construction and inspection personnel are better acquainted with proper casting and testing procedures for cylinders.
- Cylinders are less sensitive to variations in testing procedures.
- Compression tests have a lower coefficient of variation. For example, tests using concretes made with 56 aggregate combinations and 2 different cement contents had an average batch-to-batch COV of 5.73 percent for flexural tests and 3.25 percent for compression tests.
- Some of the most commonly used specifications and guidelines refer to concrete acceptance criteria based on compressive strength.
- Statistical data for ready-mix concrete design are usually available for compressive strength, but not necessarily for flexural strength.
- Durability criteria for mix design is usually based on compressive strength.
- Investigations on low in-place concrete strengths are much easier to perform using drilled cores than sawed beams.

**Strength Type Conversion Equations**

In the current PRS approach, flexural strength is used in the distress indicator models that are dependent on strength. As many SHA's move toward strength acceptance based on compressive or tensile strength testing, it is becoming more important to understand the relationships between different strength types.

Many different strength-type relationships were discussed in Darter's 1972 report.[8] A relationship between modulus of elasticity and compressive strength is presented as equation 3. (Note: Equation 3 was developed in English units.)

$$E_c = 33\, w^{1.5} (f'_c)^{0.5} \quad (3)$$

where

$E_c$ = Static modulus of elasticity of concrete, psi.
$w$ = Air dry weight of concrete at time of test, pcf.
$f'_c$ = Compressive strength of concrete at time of test, psi.

Flexural strength is commonly estimated using an equation of the form shown in equation 4.

$$S = K (f'_c)^{0.5} \quad (4)$$

where

$S$ = Flexural strength (modulus of rupture), MPa.
$K$ = Constant usually between 0.6 and 0.8 MPa.
$f'_c$ = Compressive strength of concrete at time of test, MPa.

As stated, values for K typically range from 0.6 to 0.8 MPa, but can be as low as 0.5 or as high as 0.9 for normal strength concrete. Because of the possible wide range of values for K, estimated correlations are not advised for the design of large projects. However, once established, the correlation for a particular mix can be quite stable.[21]

Kosmatka presented an example of a flexural strength versus compressive strength relationship compiled by Walker, Stanton, and Bloem in 1956.[21] This relationship is presented in figure 13. This chart (compiled from data for various aggregate combinations) demonstrates the range associated with making correlations from compressive to flexural strength. (Note: Figure 13 is presented in both English and metric units.)

Darter presented the results of a 1962 study conducted by the Bureau of Public Roads in which a relationship between flexural strength and modulus of elasticity was developed.[8] This relationship is presented as equation 5. (Note: Equation 5 was developed in English units.)

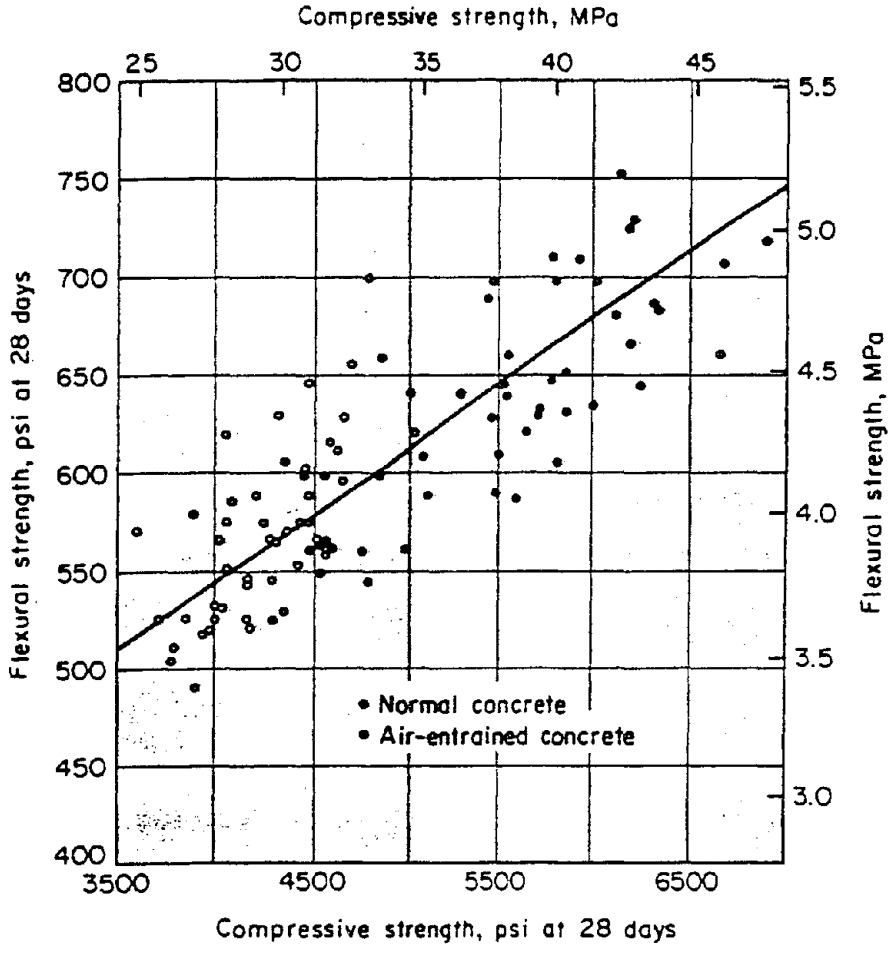



Figure 13. Relationship between flexural and compressive strength for various aggregate combinations.[21]

$$E_c = 14.4\, w^{1.5}\, S^{0.77} \tag{5}$$

where

$E_c$ = Static modulus of elasticity of concrete, psi.
w = Air dry weight of concrete at time of test, pcf.
S = Flexural strength (modulus of rupture), psi.

The same Bureau of Public Roads report also produced a relationship between indirect splitting tensile strength and compressive strength. This relationship is presented as equation 6. (Note: Equation 6 was developed in English units.)

$$T = 1.37\, (f'_c)^{0.69} \tag{6}$$

where

T = Indirect tensile strength, psi.
$f'_c$ = Compressive strength of concrete at time of test, psi.

A 1995 research work plan from Virginia was to focus on determining relationships between compressive strength, tensile strength, flexural strength, and wave propagation.[22] The developed relationships were then to be verified by field testing on construction projects using the same concrete mixtures evaluated in the laboratory. This study was being conducted with the ultimate goal of collecting enough data to develop a performance-related specification for Virginia.

## SLAB THICKNESS

The American Association of State Highway and Transportation Officials (AASHTO) *Quality Assurance Guide Specification* provides interesting insight into the perceived acceptable quality level for thickness.[23] Excerpts from the specification (section QA-501.01 - *Portland Cement Concrete Pavement*) pertaining to thickness include the following:

> "The pavement thickness will be determined from an analysis of measurements made on cores. The total length of the paving lane in linear feet in the highway proper will be divided into sublots of 152.4 m each. The width of the sublot shall be the same as the paving lane.
>
> "A sublot of pavement represented by a core deficient by more than 25.4 mm will not be accepted.
>
> "Cores from the balance of the pavement sublots will be analyzed to determine the average and standard deviation of the pavement thickness. When evaluated in accordance with QA-110.06, the percent within limits (PWL) shall be at least 90 percent.
>
> "Note: The thickness target value (TV) specified by the agency should be the design thickness as specified in the AASHTO guide. It is suggested that the lower specification limit (LSL) be calculated as 'TV – 5 mm.' However, each agency should verify and, if necessary, adjust this tolerance based upon their own research and experience."

Slab thickness variability data were presented by Darter in his 1972 report.[8] This consisted of data from three projects each from the Louisiana Department of Transportation and Development and the Texas Highway Department. The slab thickness data are presented in table 24.

Table 24. Concrete slab thickness variability data from the Louisiana Department of Transportation and Development and the Texas Highway Department.[8]

| Project Number | Mean, mm | Std. Dev., mm | COV, % |
|---|---|---|---|
| Louisiana Department of Transportation and Development Data | | | |
| 1 | 211 | 8 | 3.6 |
| 2 | 234 | 7 | 3.2 |
| 3 | 262 | 7 | 2.6 |
| Texas Highway Department Data | | | |
| 1 | 203 | 6 | 2.9 |
| 2 | 229 | 9 | 3.8 |
| 3 | 254 | 8 | 3.0 |
| Louisiana Data Only—Averages | | 7 | 3.1 |
| Texas Data Only—Averages | | 7 | 3.2 |
| All Project Data—Averages | | 7 | 3.2 |

An analysis of the slab thickness data in table 24 shows identical average project standard deviations of 7 mm for the Louisiana and Texas data. A combination of the data shows a typical average project slab thickness standard deviation of 7 mm and an average project COV of 3.2 percent. Darter makes the point that the standard deviation of concrete slab thickness does not appear to be related to the average thickness for the range of thicknesses (203 to 254 mm). Based on this observation, Darter recommends that a constant standard deviation of 8 mm be used for slab thickness in design.[8]

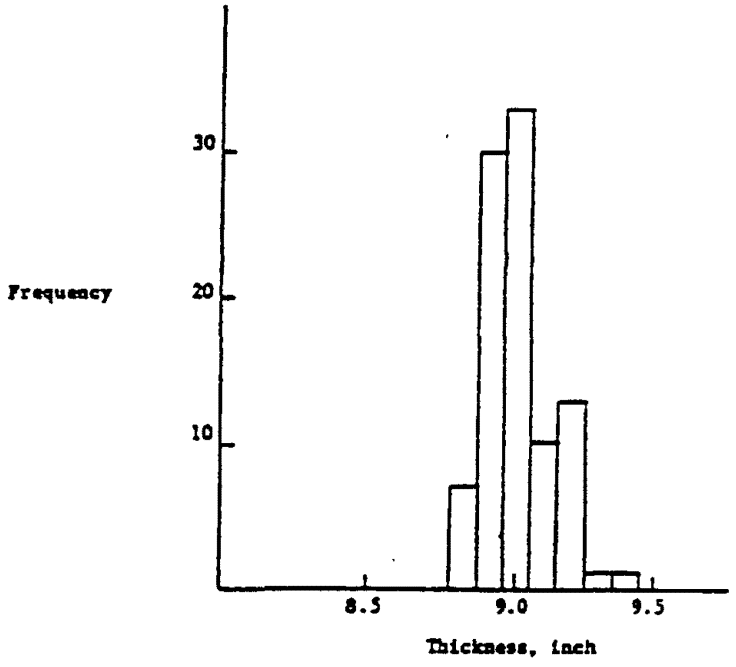
Neaman and Laguros presented thickness sampling results taken from a 12.9-km project in Oklahoma.[11] The design thickness for the project was 229 mm. The observed mean and standard deviation of 95 thickness samples taken at the project were 226 and 3 mm, respectively. These values translate to a COV of 1.1 percent for the project. A histogram of the thickness data is presented in figure 14. (Note: Figure 14 is presented in English units.)

Another histogram (based on historical data) is made up of thickness measurements for 656 cores taken from 15 projects constructed in Michigan from 1959 to 1961.[9] This histogram is included as figure 15. (Note: Figure 15 is presented in English units.) As reported on the figure, the standard deviation of the data was calculated to be 0.28 in (7 mm).

A literature review conducted by ERES for a report titled *Measurement and Specification of Construction Quality* included two different figures showing normal distributions of slab thickness data.[16] These are presented in figures 16 and 17.

| No. | Thickness, in. | f | % | Cum % |
|---|---|---|---|---|
| 1 | 8.8 | 7 | 7.4 | 7.4 |
| 2 | 8.9 | 30 | 31.6 | 39.0 |
| 3 | 9.0 | 33 | 34.8 | 73.8 |
| 4 | 9.1 | 10 | 10.5 | 84.3 |
| 5 | 9.2 | 13 | 13.7 | 98.0 |
| 6 | 9.3 | 1 | 1.0 | 99.0 |
| 7 | 9.4 | 1 | 1.0 | 100.0 |

| | |
|---|---|
| n | 95 |
| specs | 9.0 |
| $\bar{x}$ | 8.9 |
| $\sigma_x$ | 0.1 |
| v | 2.2% |

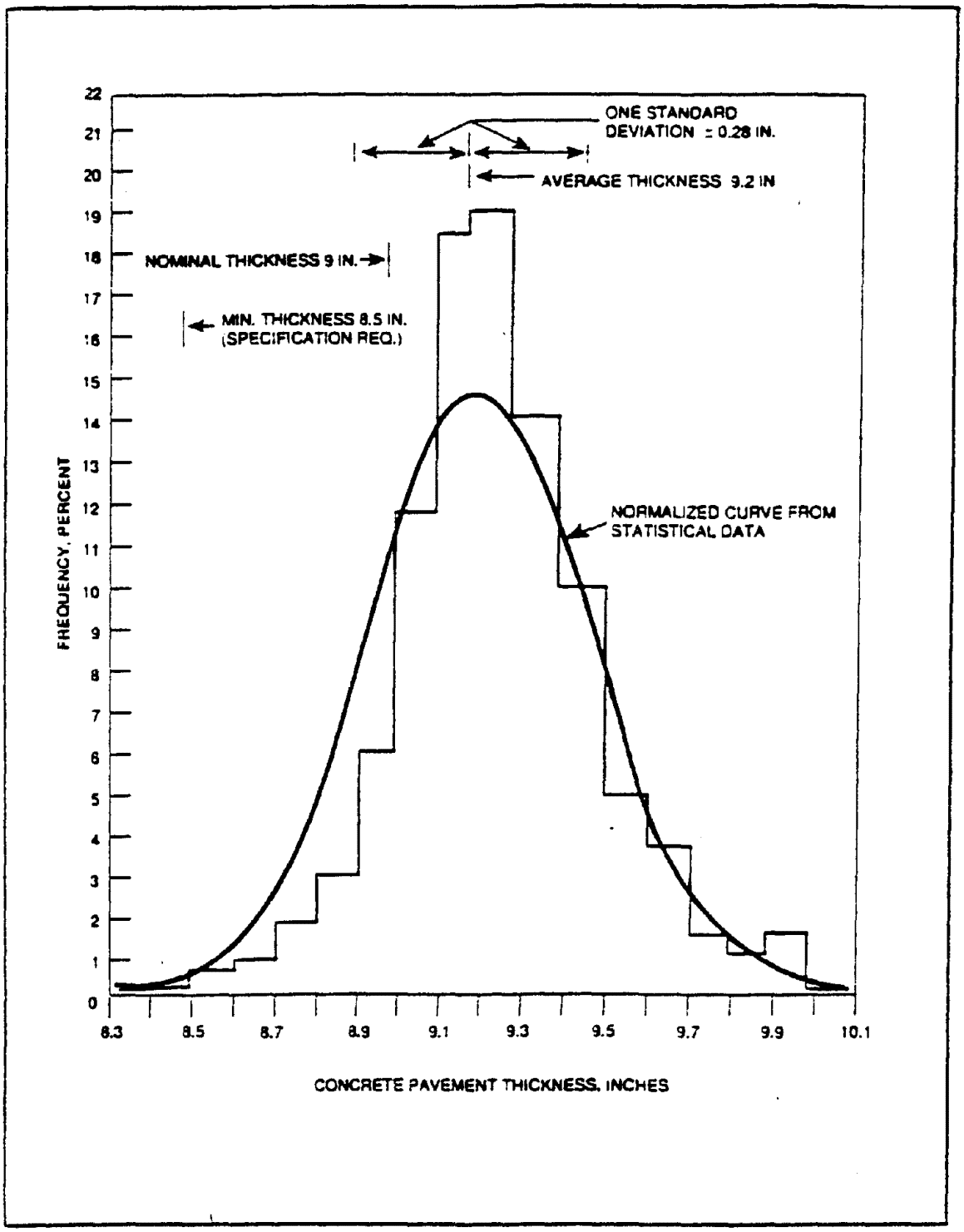



1 in = 25.4 mm

Figure 14. Histogram of thickness data for a project constructed in Oklahoma.[11]

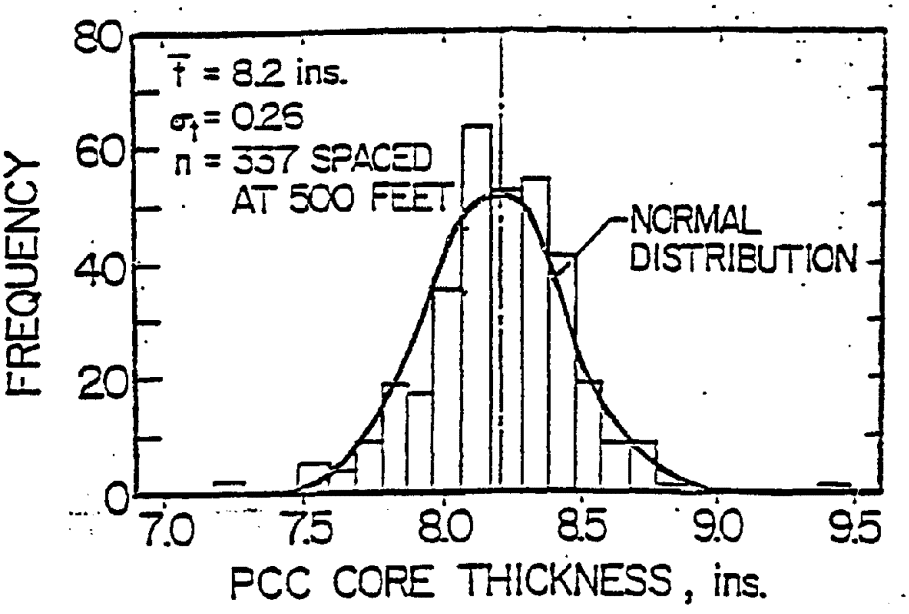
Actual thickness data were also collected for the FHWA-sponsored project "Measurement and Specification of Construction Quality."[17] Both core and ground-penetrating radar (GPR) data were collected for three projects (one each for Illinois, Minnesota, and Ohio) constructed during the 1995 construction season. The results of the statistical analysis for both data sets are presented in table 25.

In the 1996 report titled *Variability in Highway Pavement Construction*, Hughes presented a table of thickness data statistics for projects in three different states (Ohio, Kansas, and Georgia).[6] This information is presented in table 26.



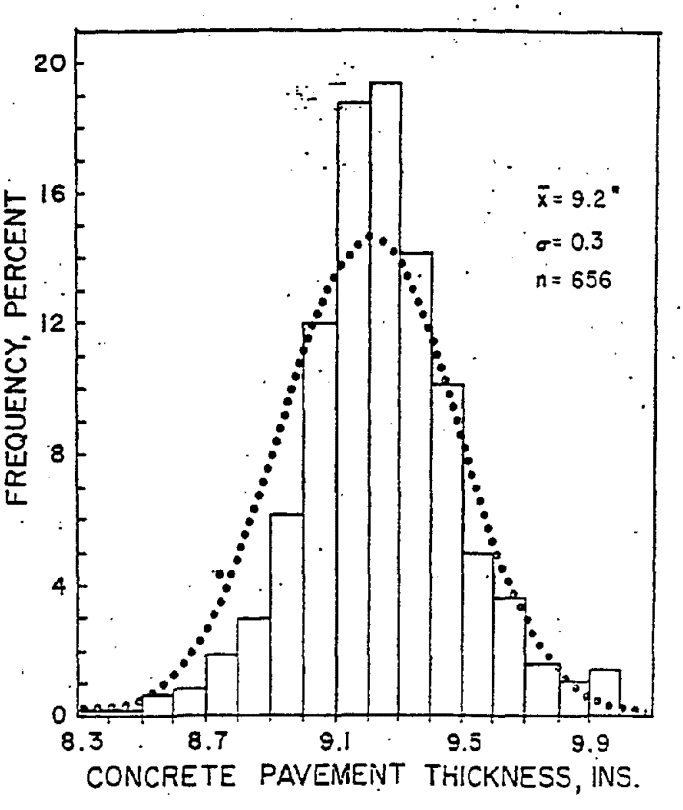

1 in = 25.4 mm

Figure 15. Frequency distribution of concrete pavement thickness—1959 to 1961 Michigan data.[9]




1 in = 25.4 mm
1 ft = 0.305 m

Figure 16. Distribution of PCC Core Thickness.[16]




1 in = 25.4 mm

Figure 17. Histogram of concrete slab thickness.[16]

Table 25. 1995 Core and GPR thickness data.[17]

| Project Location | Sample Type | No. of Samples | Mean, mm | Std. Dev., mm | COV, % |
|---|---|---|---|---|---|
| Rochelle, IL | Cores | 30 | 247 | 6 | 2.5 |
| | GPR | 1450 | 248 | 3 | 1.2 |
| Mankato, MN | Cores | 41 | 208 | 7 | 3.6 |
| | GPR | 1187 | 193 | 12 | 6.3 |
| Bellefontaine, OH | Cores | 30 | 313 | 12 | 3.7 |
| | GPR | 1188 | 305 | 13 | 4.4 |
| Core Data Only—Averages | | | | 8 | 3.3 |
| GPR Data Only—Averages | | | | 9 | 4.0 |

Table 26. Standard deviations of core thickness from projects in Ohio, Kansas, and Georgia.[6]

| Project | Year | Design Thickness, mm | No. of Samples | Average, mm | Std. Dev., mm | COV, % |
|---|---|---|---|---|---|---|
| Ohio #1 | 1993 | 279 | 21 | 279 | 10 | 3.2 |
| Ohio #2 | 1993 | 229 | 79 | 229 | 8 | 3.0 |
| Ohio #3 | 1993 | 229 | 30 | 282 | 8 | 3.3 |
| Kansas #1 | 1994 | 229 | 117 | 234 | 8 | 3.5 |
| Kansas #2 | 1994 | 229 | 196 | 236 | 10 | 4.3 |
| Kansas #3 | 1994 | 229 | 215 | 234 | 10 | 3.9 |
| Kansas #4 | 1993 | 229 | 140 | 234 | 13 | 5.1 |
| Kansas #5 | 1992 | 267 | 83 | 234 | 10 | 3.5 |
| Georgia #1 | 1994 | 254 | 87 | 241 | 5 | 2.1 |
| Georgia #2 | 1993 | 254 | 70 | 277 | 5 | 1.9 |
| Georgia #3 | 1993 | 254 | 60 | 259 | 5 | 2.0 |

## ENTRAINED AIR VOID PARAMETERS

A number of entrained air void parameters influence the durability of in-place concrete. A good explanation of air void parameters and their importance to pavement performance was presented in a report by Flude and Janssen.[24] First, two different types of air voids are found in concrete: natural forming *entrapped* air voids, and *entrained* air voids (entrained air voids are the result of an applied air-entraining agent). Entrapped air voids are typically 1 mm or more in diameter, whereas entrained air

voids are considerably smaller—ranging in size between 10 to 100 μm. Entrained air bubbles are typically spherical in shape, compared to the irregular shapes taken by the entrapped air voids. A number of other measurable air void parameters can be used to estimate the frost-resistant capabilities of a certain concrete mix. These parameters include the following:[24]

- *Air Content*—proportion of the total volume of the concrete that is air voids (expressed as percentage by volume).
- *Spacing Factor*—maximum distance from any point in the cement paste to the boundary of an air void (expressed in millimeters).
- *Specific Surface Area*—total surface area of the air voids divided by the volume of air entrained in the concrete (expressed in suitable units so that the unit of specific surface is a reciprocal length).
- *Void Frequency*—voids per unit length of traverse (i.e., the number of air voids intercepted by a traverse line divided by the length of that line [expressed in the unit of reciprocal length]).
- *Air Void Content*—number of voids per unit volume of concrete (expressed in units of reciprocal length cubed).
- *Philleo Factor*—indication of the concrete's ability to withstand freezing and thawing.
- *Air Void Chord Distribution*—number of chords of a particular length measured over the entire traverse length.
- *Average Chord Length*—average length of the chords of the voids formed by the line of traverse across the voids in the concrete (expressed in units of length).
- *Median Chord Length*—value that separates the air void chord distribution histogram into two parts of equal area.
- *Median Offset*—difference between the average chord length and the median chord length.

Flude and Janssen also investigated the correlation of these air void parameters to the *durability factor*—a variable representing the overall durability of the concrete. Conclusions of the report showed that the spacing factor was the "best single predictor" of the durability factor.[24]

### Variation of Entrained Air Content

A report compiled for the Illinois Department of Transportation in 1995 contained air content variability data.[7] The reported variations were compiled from both structural and pavement data contained in IDOT's MISTIC database. Each calculated standard deviation is specific to one mix design and construction year. Table 27 contains a summary of the reported air content variability data.

Table 27. Entrained air content variability data from the Illinois Department of Transportation's MISTIC database.[7]

| Mix Number | Class | Year | n | Mean, % | Std. Dev., % | COV (%) | Min | Max |
|---|---|---|---|---|---|---|---|---|
| 1 | Pavement | 1993 | 82 | 5.9 | 1.11 | 19.0 | 4.3 | 9.9 |
| 2 | Pavement | 1993 | 28 | 5.9 | 0.75 | 9.1 | 4.5 | 7.5 |
| 3 | Pavement | 1993 | 39 | 6.3 | 0.58 | 9.3 | 5.5 | 7.8 |
| 3 | Pavement | 1992 | 77 | 6.5 | 0.61 | 9.4 | 5.2 | 7.8 |
| 4 | Pavement | 1993 | 36 | 7.2 | 0.68 | 9.3 | 6.0 | 9.2 |
| 4 | Pavement | 1992 | 72 | 7.1 | 0.85 | 11.9 | 5.2 | 9.0 |
| 5 | Structural | 1992 | 269 | 7.8 | 0.95 | 12.2 | 4.8 | 9.9 |
| 5 | Structural | 1993 | 104 | 8.0 | 0.90 | 11.2 | 5.4 | 9.5 |
| 6 | Structural | 1992 | 12 | 5.7 | 0.66 | 11.6 | 5.0 | 7.3 |
| 6 | Structural | 1993 | 10 | 5.7 | 0.38 | 6.7 | 5.0 | 6.0 |
| 6 | Structural | QC/QA | 124 | 5.9 | 0.91 | 15.4 | 3.4 | 8.3 |
| 7 | Structural | Unknown | 38 | 6.4 | 0.87 | 13.5 | 5.0 | 8.0 |
| 8 | Structural | Unknown | 45 | 6.7 | 0.62 | 9.2 | 5.6 | 8.0 |
| 9 | Structural | Unknown | 315 | 6.4 | 0.75 | 11.7 | 5.0 | 8.0 |
| 10 | Structural | Unknown | 13 | 7.1 | 0.75 | 10.4 | 5.7 | 8.0 |
| 11 | Structural | Unknown | 99 | 6.4 | 0.91 | 14.2 | 4.8 | 9.9 |
| 12 | Structural | Unknown | 24 | 6.1 | 0.37 | 6.1 | 5.5 | 6.9 |
| 13 | Structural | Unknown | 12 | 6.2 | 0.60 | 9.7 | 5.5 | 7.6 |
| Pavement Data Only—Averages | | | | 6.48 | 0.76 | 11.33 | 4.3 | 9.9 |
| Structural Data Only—Averages | | | | 6.53 | 0.72 | 10.99 | 3.4 | 9.9 |
| All Data (Pavement and Structural)—Averages | | | | 6.52 | 0.74 | 11.11 | 3.4 | 9.9 |

An analysis of the data in table 27 shows little difference between the pavement and structural air content variability data. This suggests that typical values for within-mix air content standard deviation and coefficient of variation are 0.74 percent and 11.11 percent, respectively.

Neaman and Laguros presented pressure meter air content results from a 12.9-km project in Oklahoma.[11] Two-hundred air content samples were taken at the project. The sample results from the 200 samples showed an overall entrained air content mean and standard deviation of 4.6 and 0.8 percent, respectively.[11] These values translate into a COV of 17.3 percent for the project. A histogram of the air content data is presented in figure 18.

| No. | Air Cont. Range (%) | f | % | Cum % |
|---|---|---|---|---|
| 1 | -3.0 | 4 | 2.0 | 2.0 |
| 2 | 3.0-3.5 | 3 | 1.5 | 3.5 |
| 3 | 3.5-4.0 | 27 | 13.5 | 17.0 |
| 4 | 4.0-4.5 | 76 | 38.0 | 55.0 |
| 5 | 4.5-5.0 | 54 | 27.0 | 82.0 |
| 6 | 5.0-5.5 | 21 | 10.5 | 92.5 |
| 7 | 5.5-6.0 | 6 | 3.0 | 95.5 |
| 8 | 6.0-6.5 | 4 | 2.0 | 97.5 |
| 9 | 6.5- | 5 | 2.5 | 100.0 |
| | | 200 | 100.0 | |

| | |
|---|---|
| n | 200 |
| specs | 3-6% |
| $\bar{x}$ | 4.6 |
| $\sigma_T$ | 0.8 |
| $\sigma_c$ | 0.2 |
| $\sigma_s$ | 0.1 |
| $\sigma_a$ | 0.7 |
| v | 17.6% |

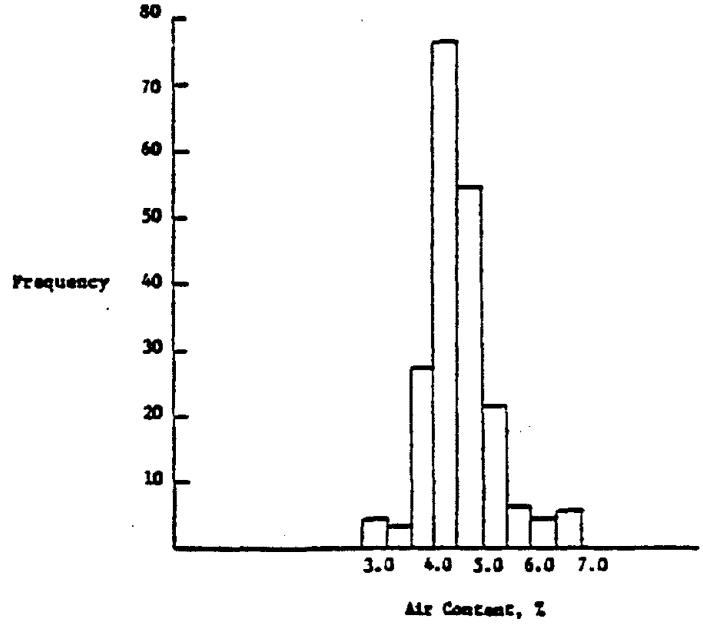



Figure 18. Histogram of pressure meter entrained air content data for an Oklahoma project.[11]

McMahon et al. presented a summary of a statistical analysis of New York State air content variability data from 1973.[9] The analyzed data included air content data collected using both the pressure and Chace air meters. The results of the statistical analysis of the data are presented in table 28.

Table 28. Plastic concrete air content research data— New York State.[9]

| Project | Mixer Type and Use | Observations | Test Method | Testing Variance, % | Sampling Variance, % | Material Variance, % | Standard Deviation, % | Mean, % |
|---|---|---|---|---|---|---|---|---|
| 1 | Central mix | 216 | Pressure | 0.043 | 0.07 | 0.80 | 0.95 | 4.55 |
| | Paving | 216 | Chace | 0.20 | 0.545 | 0.65 | 1.18 | 6.39 |
| 2 | Central mix | 200 | Pressure | 0.126 | 0.04 | 0.50 | 0.82 | 5.91 |
| | Paving | 200 | Chace | 0.22 | 0.134 | 0.70 | 1.026 | 7.42 |
| 3 | E-34 paver | 200 | Pressure | 0.067 | 0.05 | 0.48 | 0.77 | 5.14 |
| | Paving | 200 | Chace | 0.15 | 0.10 | 0.33 | 0.76 | 7.38 |
| 4 | Truck mix | 200 | Pressure | 0.16 | 0.29 | 1.74 | 1.48 | 7.90 |
| | Structural | 200 | Chace | 0.335 | 0.15 | 1.38 | 1.36 | 10.2 |
| 5 | Truck mix | 204 | Pressure | 0.035 | 0.04 | 1.27 | 1.16 | 6.11 |
| | Structural | 204 | Chace | 0.45 | 0.99 | 0.48 | 1.39 | 8.75 |
| 6 | Central mix | 200 | Pressure | 0.06 | 0.105 | 0.34 | 0.71 | 6.18 |
| | Paving | 200 | Chace | 0.256 | 0.137 | 0.40 | 0.89 | 7.39 |
| 7 | E-34 paver | 200 | Pressure | 0.08 | 0.08 | 0.39 | 0.74 | 4.94 |
| | Paving | 200 | Chace | 0.14 | 2.55 | 0.96 | 1.32 | 6.41 |
| 8 | E-34 paver | 200 | Pressure | 0.047 | 0.04 | 0.70 | 0.89 | 4.82 |
| | Paving | 200 | Chace | 0.26 | 0.25 | 1.02 | 1.24 | 6.51 |
| 9 | Truck mix | 200 | Pressure | 0.05 | 0.135 | 2.39 | 1.60 | 5.80 |
| | Paving | 200 | Chace | 0.43 | 0.325 | 1.79 | 1.60 | 8.43 |
| 10 | Truck mix | 200 | Pressure | 0.09 | 0.136 | 1.64 | 1.37 | 6.07 |
| | Structural | 200 | Chace | 0.28 | 0.24 | 1.71 | 1.49 | 6.33 |

A closer analysis of the New York State results presented in table 28 showed the following averages of the 10 project standard deviations (for each reported test method—pressure meter or Chace meter):

Pressure Meter—Average Project Standard Deviation: 1.05 percent
Chace Meter— Average Project Standard Deviation: 1.17 percent

Pressure meter air content data were collected by ERES Consultants for the FHWA-sponsored project "Measurement and Specification of Construction Quality."[16] The data were collected for three projects (one each for Illinois, Minnesota, and Ohio) constructed during the 1995 construction season. The results of the statistical analysis are presented in table 29.

In the 1996 report by Hughes titled *Variability in Highway Pavement Construction*, typical air content standard deviations for different projects are presented.[6] The collected data are presented in table 30.

Table 29. 1995 pressure meter entrained air content data collected for three SHA projects.[16]

| Project Location | No. of Samples | Mean, % | Std. Dev., % | COV, % |
|---|---|---|---|---|
| Rochelle, IL | 43 | 5.24 | 1.25 | 23.9 |
| Mankato, MN | 29 | 5.75 | 0.57 | 9.9 |
| Bellefontaine, OH | 115 | 7.52 | 1.05 | 14.0 |

Table 30. Project air content standard deviations.[17]

| Data Source | Year | Testing Method | Std. Dev., % |
|---|---|---|---|
| Rieth–Riley | 1993 | Pressure meter | 0.53 |
| New Jersey | 1993 | Pressure meter | 0.75 |
| Virginia | 1988 | Pressure meter | 0.74 |
| Arizona | 1974 | Pressure meter | 0.97 |

**Relationship Between the Entrained Air Content of Plastic and Hardened Concrete**

A 1994 study by Nagi and Whiting investigated discrepancies in measurement of concrete air content between plastic and hardened states.[25] As part of the study, the authors gathered information via a literature review and questionnaire. They created a database of studies in which air content was determined in the fresh and hardened states on the same batches of concrete. A laboratory investigation was used to determine the influence of air-entraining admixture, cement alkali level, temperature, and retempering on concrete air contents in both fresh and hardened states. Finally, a field study was conducted to examine the influence of field conditions on air content measurements.

Plots of plastic concrete air content versus hardened concrete air content were included in the report. Each plot contains data from four or five different studies. Because of the volume of data included, four plots were used so that all of the data could be viewed. The four plots are presented in figures 19 through 22.

The results of this study generally found that discrepancies are observed between plastic and hardened air contents whenever both measurements are taken; however, the relationship between the two is inconsistent. In most cases, however, the discrepancies were found to be insignificant. The final conclusion stated by the authors is the following:[25]

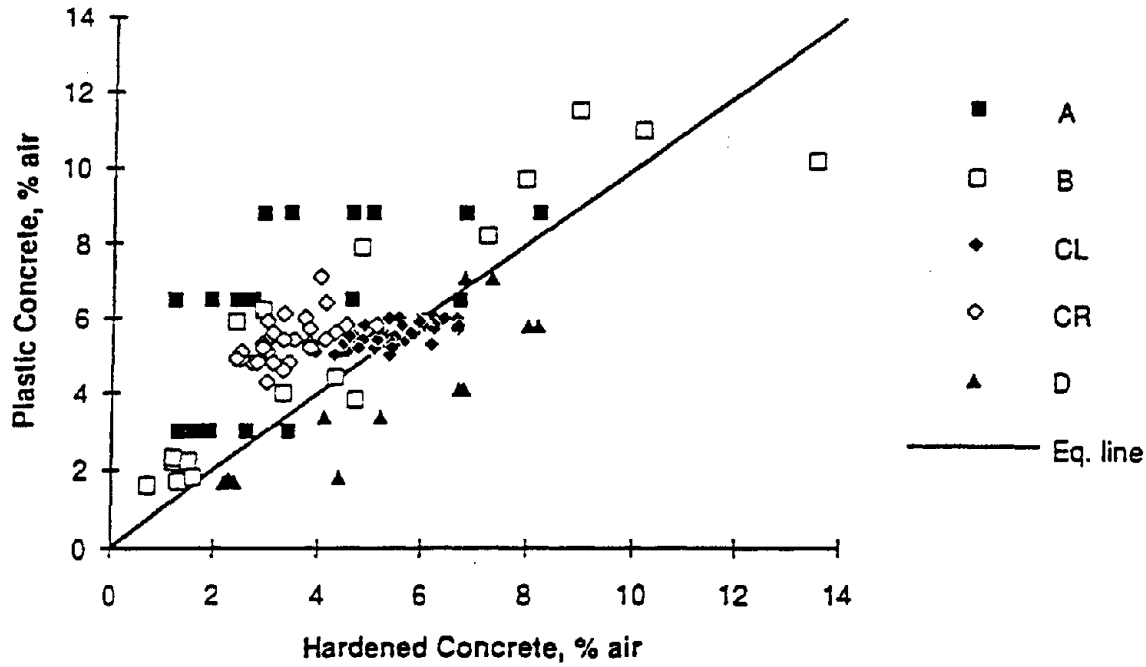



Figure 19. Plastic versus hardened air content—Plot 1.[25]

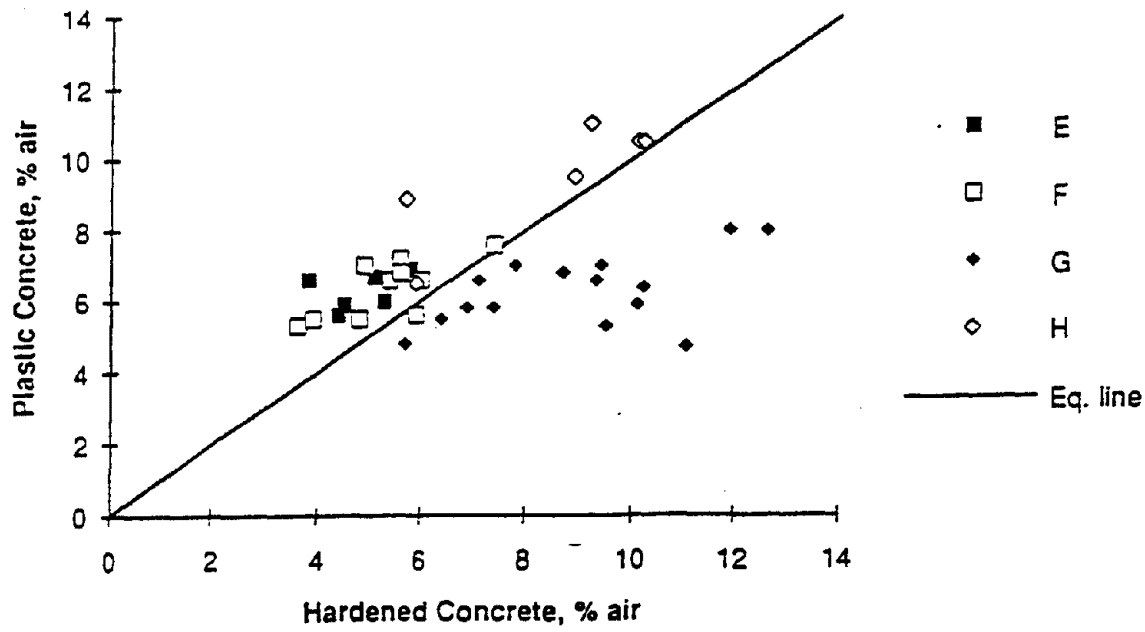



Figure 20. Plastic versus hardened air content—Plot 2.[25]

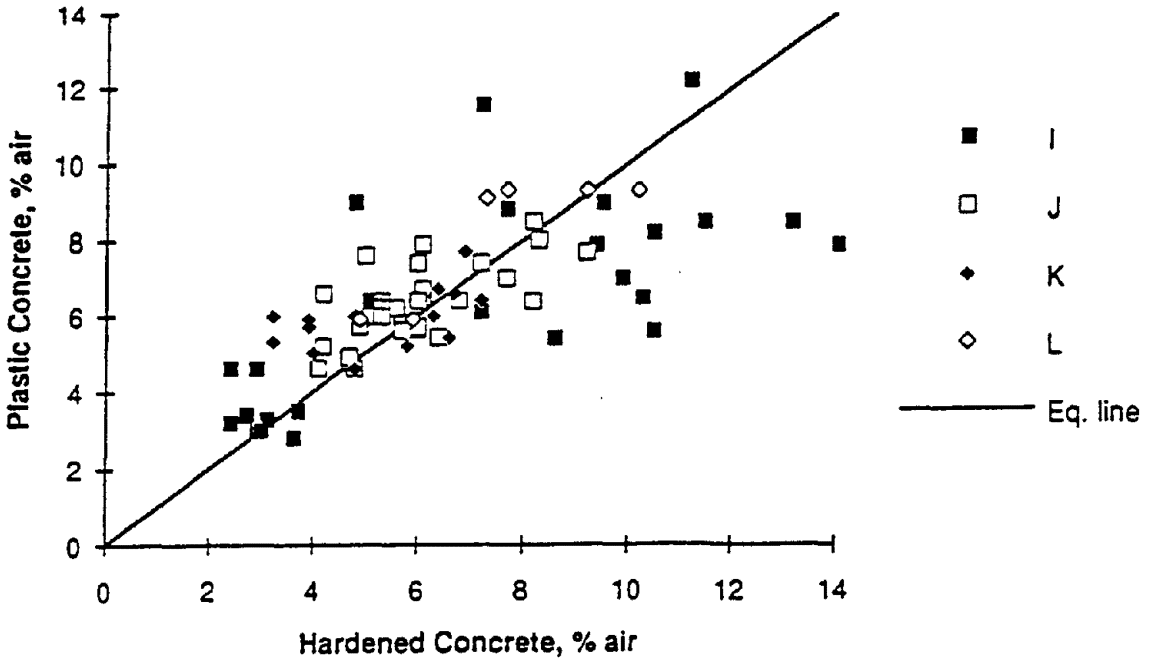



Figure 21. Plastic versus hardened air content—Plot 3.[25]

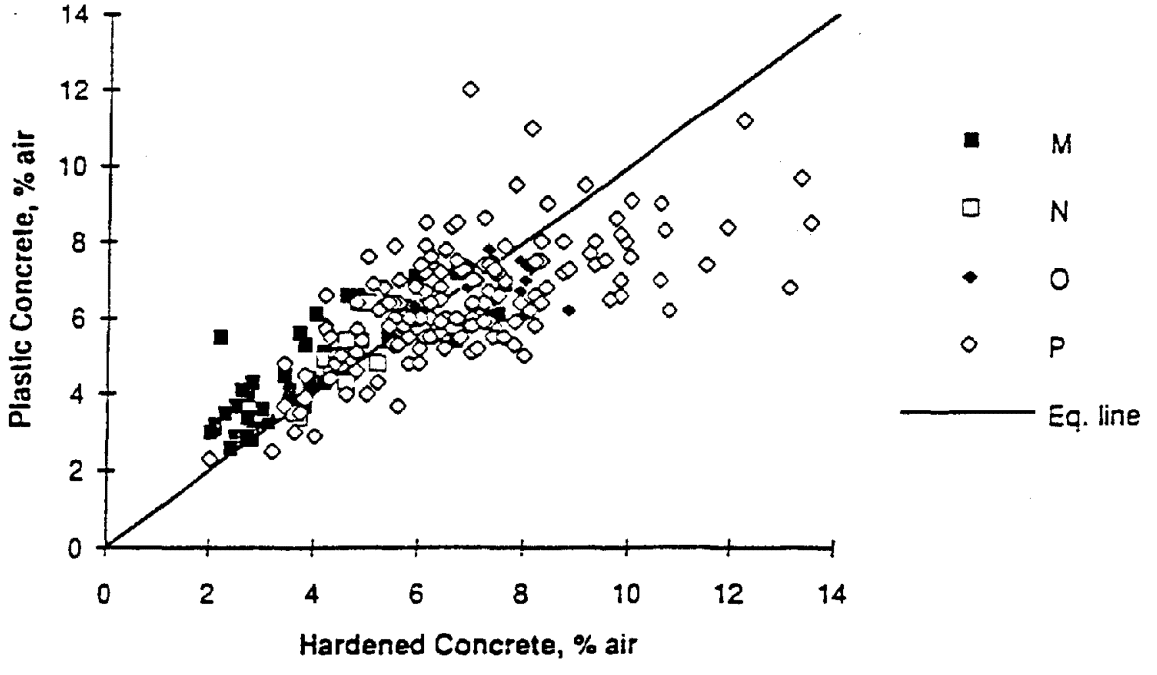



Figure 22. Plastic versus hardened air content—Plot 4.[25]

"In conclusion, if air content measurements are conducted as specified in ASTM test procedures and air-entraining agents are used as specified, discrepancies in air content should not be a problem."

### New Technology for Measuring Entrained Air Content

There is a strong push toward the in situ measurement of entrained air content. Currently, in situ air content is most commonly measured by coring the pavement and analyzing the cores using linear traverse methods. Ideally, new nondestructive methods will take the place of coring.

One reviewed report describes the development of a hand-held device that measures the entrained air content of the in-place slab using fiber optics.[26] A probe called an airtrode is inserted into the surface of the pavement. The airtrode is connected to a silical glass multimode optical fiber that delivers light from a diode laser. The amount of light reflected back into the fiber is directly dependent on the amount and number of air bubbles in the fresh concrete. This information is then used to estimate the entrained air content. The use of this rapid, nondestructive testing method is still in the development stages. At the time of this report, the reliability of the testing method was being tested in the laboratory, as well as on field trials in New York, Pennsylvania, and Minnesota. Results of these reliability tests were not published as part of this report.

Another device, called the *air void analyzer* (AVA), is also currently undergoing much field testing. This device, developed in Denmark in the early 1990's, can be used to determine the amount of air voids and the spacing between them from a sample of fresh concrete. Before the AVA, linear traverse testing of hardened concrete was the only reliable testing method for determining air void characteristics (amount, size, and distribution of entrained air voids). An AVA test is conducted on a 20-cc sample of concrete mortar and takes about 25 minutes to complete. FHWA is continuing to test this device in the field.[27]

## INITIAL SMOOTHNESS

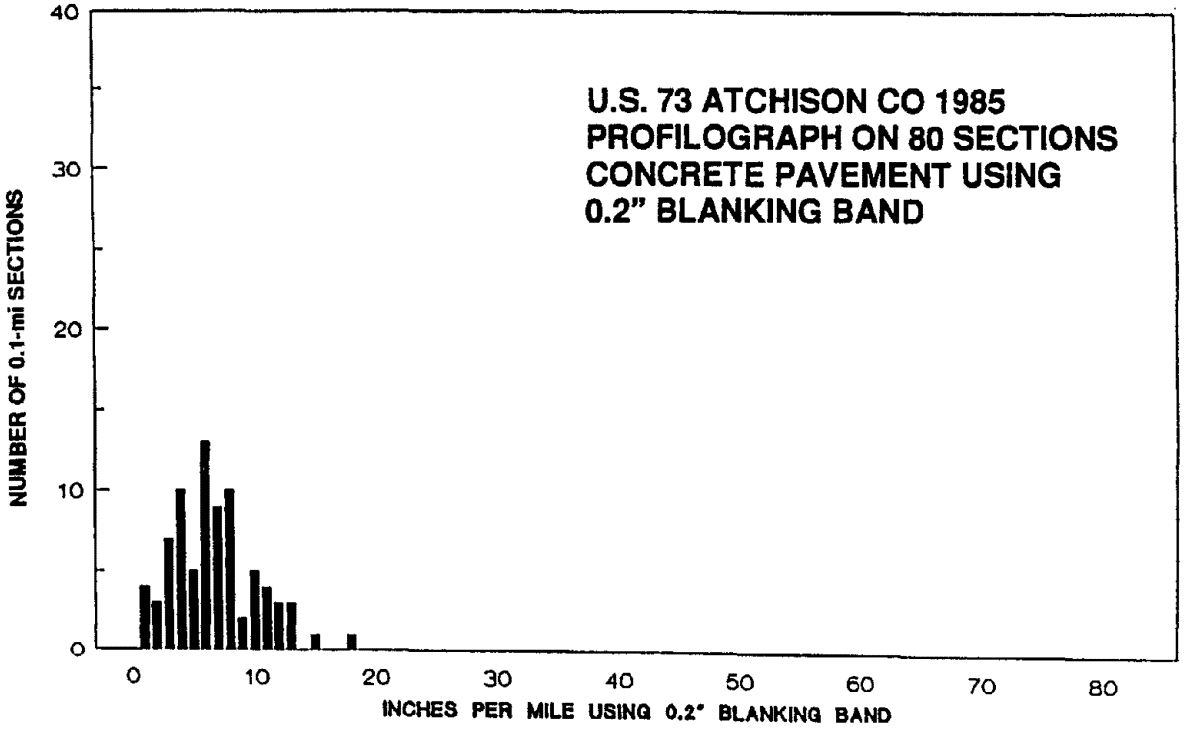
The 1972 Darter report presented a summary of the initial serviceability index data taken at the AASHO Road Test.[8] The data were collected for 224 sections and showed a mean value of 4.71 with a standard deviation of 0.14. Darter suggested that this 0.14 value be considered a minimum value or the "best" obtainable in the field (due to the stringent quality control at the AASHO Road Test). He also suggested that a standard deviation of 0.30 be used for design.

### Variation of Initial Smoothness

Histograms for three different Kansas projects (constructed in 1985 and 1986) were included in a 1992 report by Parcells.[28] The smoothness data collected at all three

projects were measured with a California profilograph and reduced using a 5.1-mm blanking band.

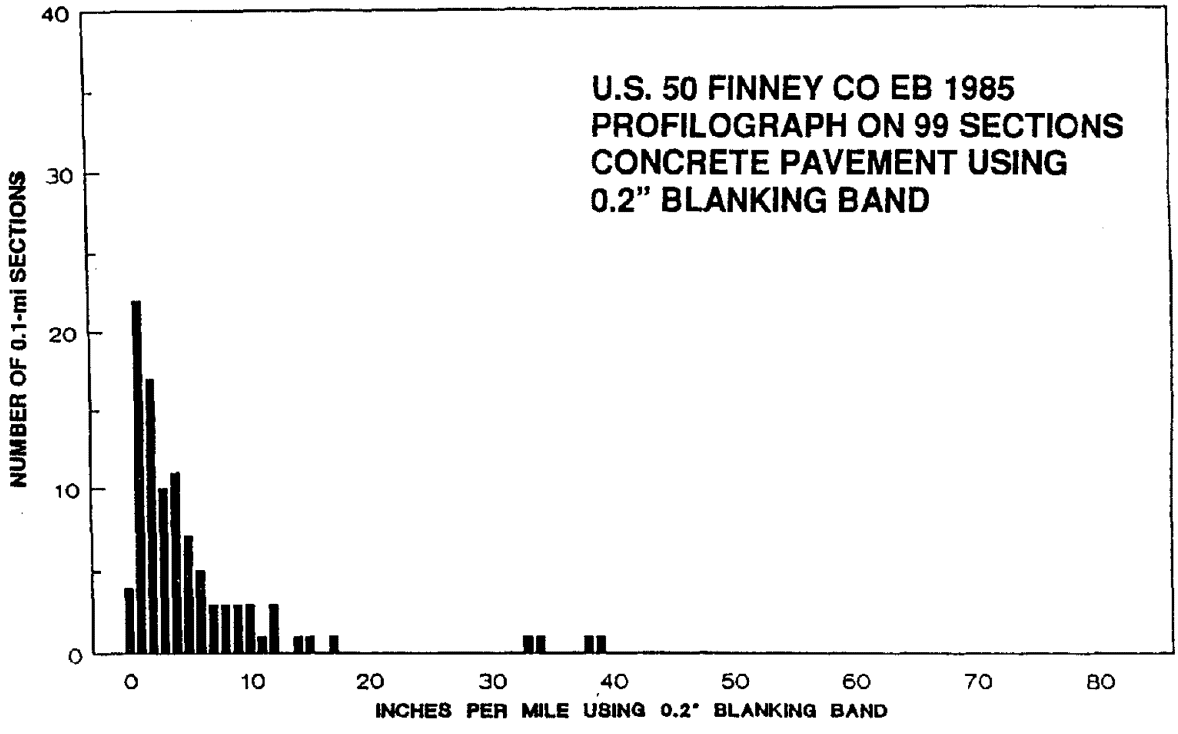
The first project consisted of a two-lane section of U.S. 73 in Atchison County, Kansas. Both lanes were paved in one construction pass. Four profiles were measured from each 0.16-km section (one for each wheelpath in the two lanes). The average of the four measured profiles was calculated and used to represent the profile for each section. Figure 23 shows a histogram of the 80 section profiles calculated for the project.[28] (Note: Figure 23 is presented in English units.)




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

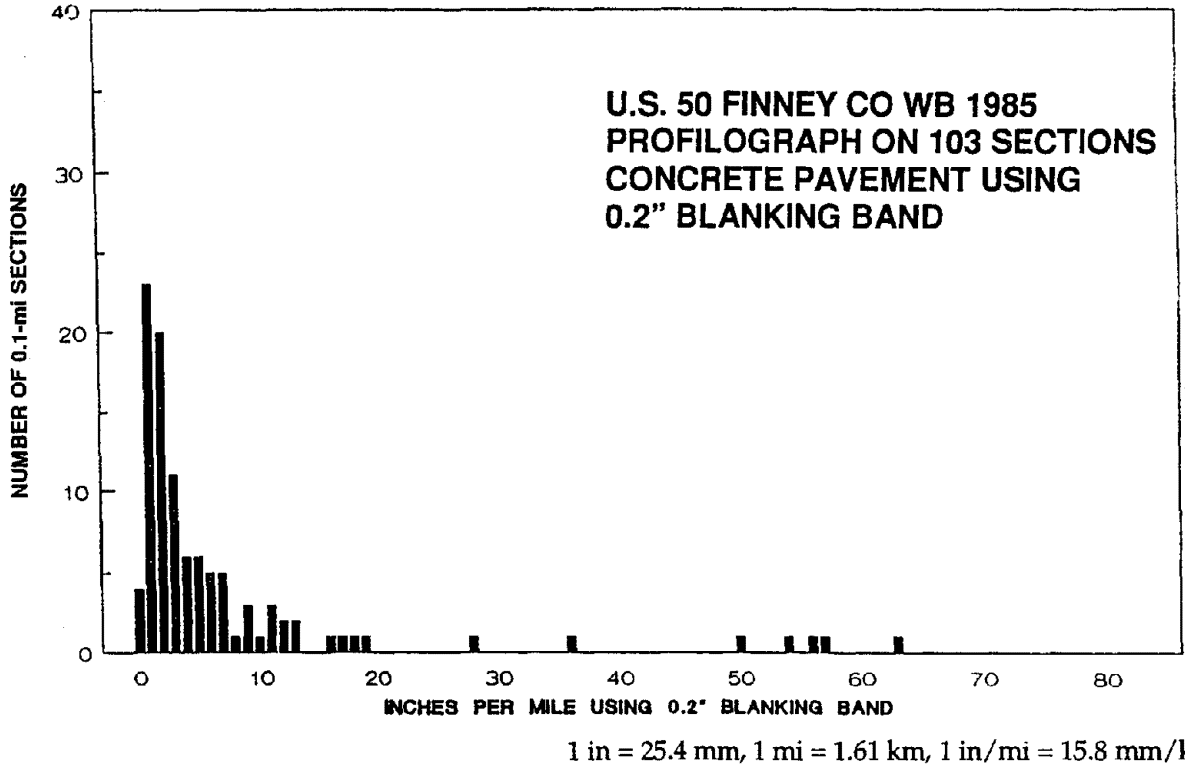
Figure 23. Histogram of section profiles—(1985) U.S. 73, Atchison County, Kansas.[28]

The second project was a two-lane section of U.S. 50 in Finney County, Kansas. Both lanes were paved separately. Two profiles were taken from each lane (one in each wheelpath) for each 0.16-km segment. The average of the two profiles was calculated and used to represent the profile of each section. Figure 24 shows a histogram of the 99 eastbound section profiles. Figure 25 shows a histogram of the 103 westbound section profiles.[28] (Note: Figures 24 and 25 are presented in English units.)




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

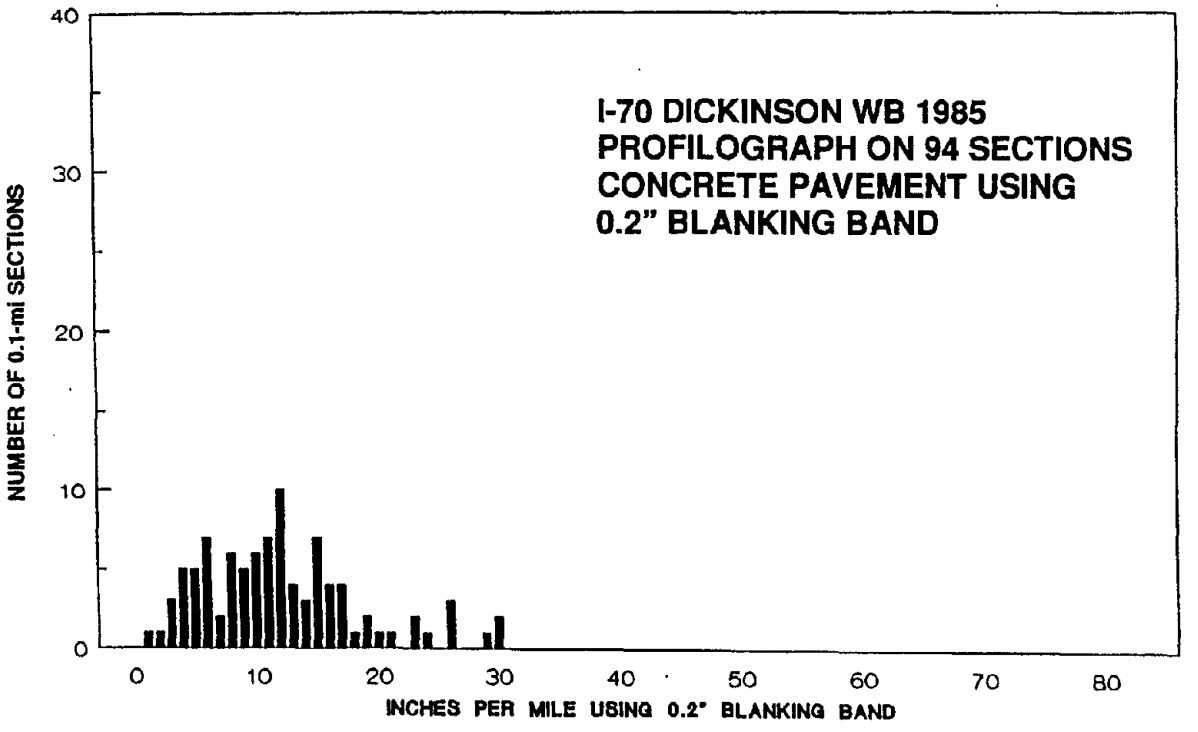
Figure 24. Histogram of section profiles—(1985) U.S. 50—Eastbound, Finney County, Kansas.[28]




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 25. Histogram of section profiles—(1985) U.S. 50—Westbound, Finney County, Kansas.[28]

The third project consisted of the reconstruction of part of I-70 in Dickinson County, Kansas. The westbound lanes were reconstructed in 1985, with the eastbound lanes being completed the following year. The profilograph results for each 0.16-km section represent the average of four traces (one in each wheelpath for two lanes). Figure 26 shows a histogram of the 94 westbound section profiles. Figure 27 shows a histogram of the 84 eastbound section profiles.[28] (Note: Figures 26 and 27 are presented in English units.)

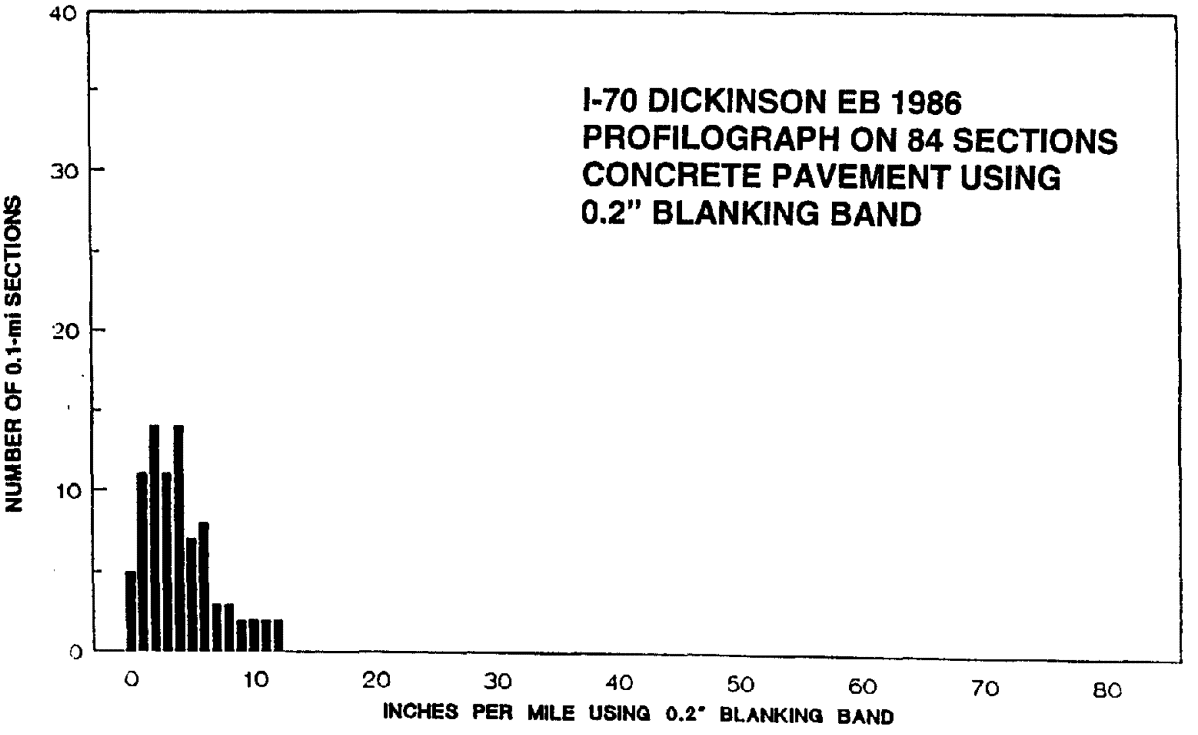



1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 26. Histogram of section profiles—(1985) I-70—Westbound, Dickinson County, Kansas.[28]

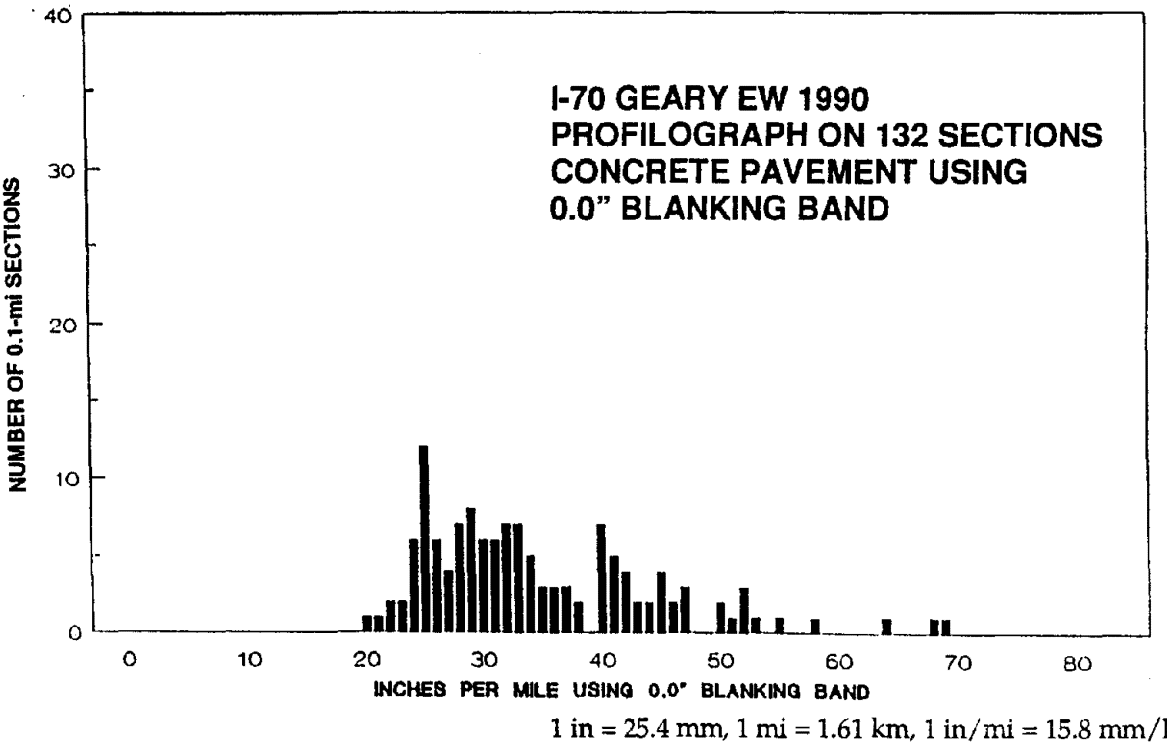
The 1992 report by Parcells also included data showing the comparison of data reduction blanking bands. The initial smoothness data for two different projects were reduced using both 0.0-mm and 5.1-mm blanking bands. The first project consisted of the 1990 reconstruction of part of I-70 in Geary County, Kansas. Smoothness was measured as the average of 3 traces for each of 132 segments. Figures 28 and 29 show histograms of these data reduced using both 0.0-mm and 5.1-mm blanking bands. (Note: Figures 28 and 29 are presented in English units.)

The second project consisted of the 1990 reconstruction of part of I-470 in Shawnee County, Kansas. Smoothness was measured as the average of 3 traces for each of 77



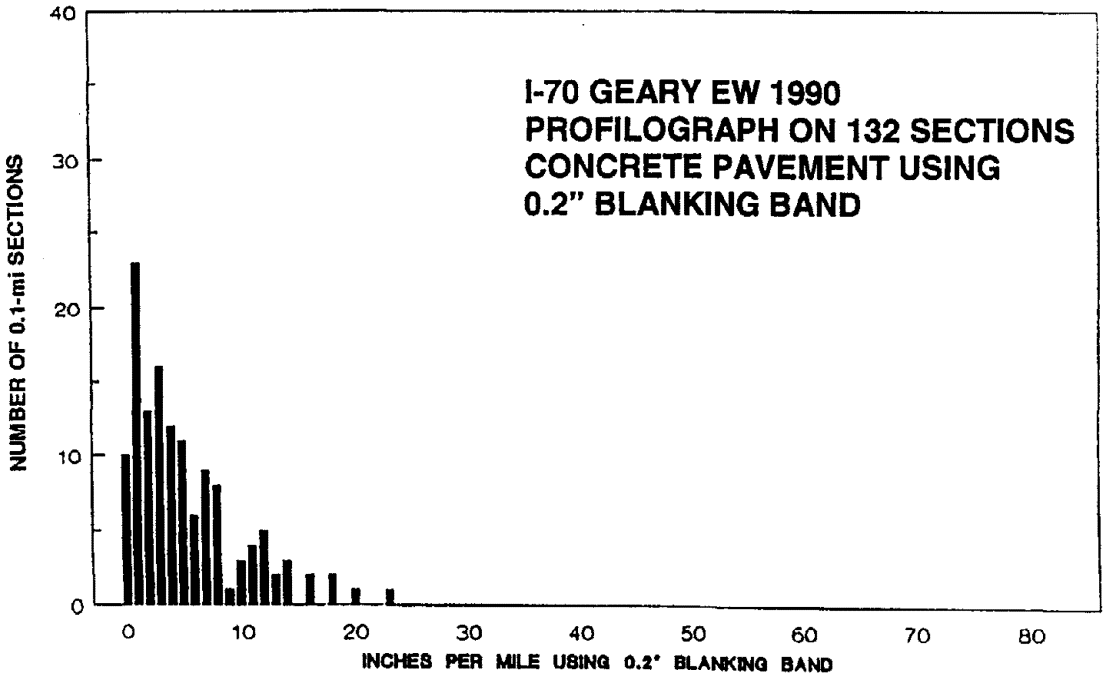

1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 27. Histogram of section profiles—(1986) I-70—Eastbound, Dickinson County, Kansas.[28]




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 28. Histogram of section profiles—(1990) I-70—Geary County, Kansas (measured with a 0.0-mm blanking band).[28]




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 29. Histogram of section profiles—(1990) I-70—Geary County, Kansas (measured with a 5.1-mm blanking band).[28]

segments. Figures 30 and 31 show histograms of these data reduced using both 0.0-mm and 5.1-mm blanking bands. (Note: Figures 30 and 31 are presented in English units.)

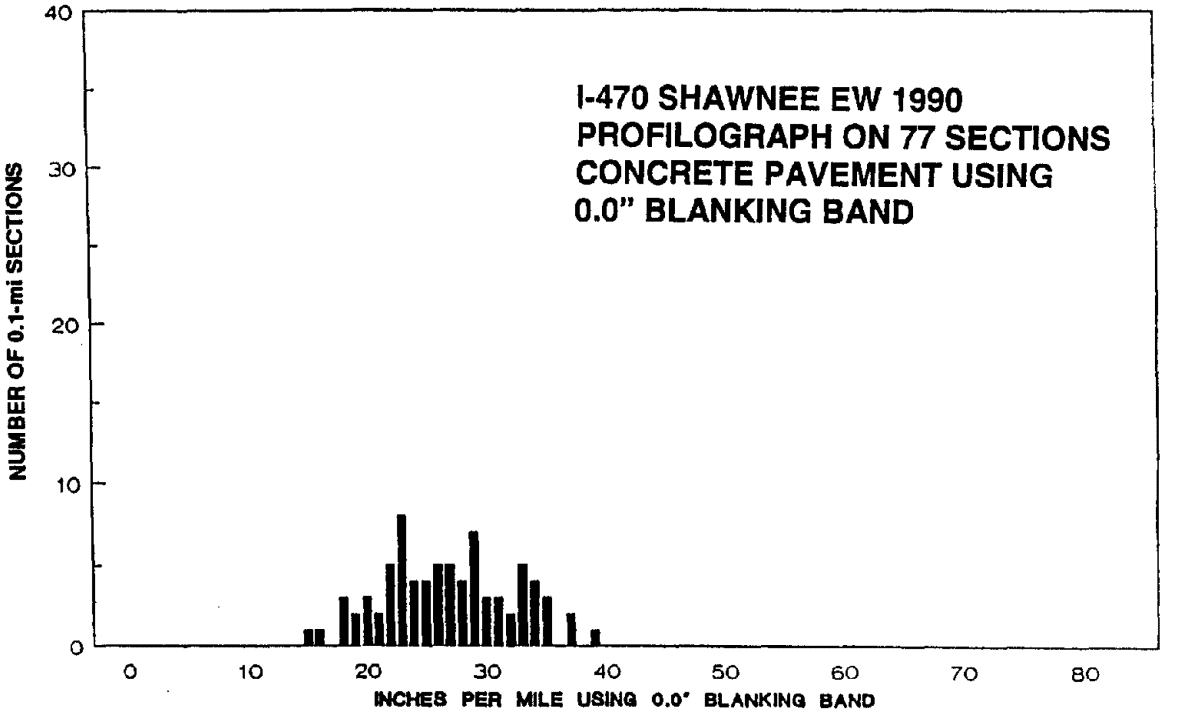
## Sources of Error in Measuring Smoothness

### Profilograph Differences

In a report by Huft, the author noted that the profilograph type influences the testing results.[29] Specifically, the profilograph geometry affects profile measurements. It was also noted that some of the digital filters used with automated profilographs (used to remove spikes caused by extaneous mechanical vibrations) have caused problems in accurately interpreting the pavement profile.

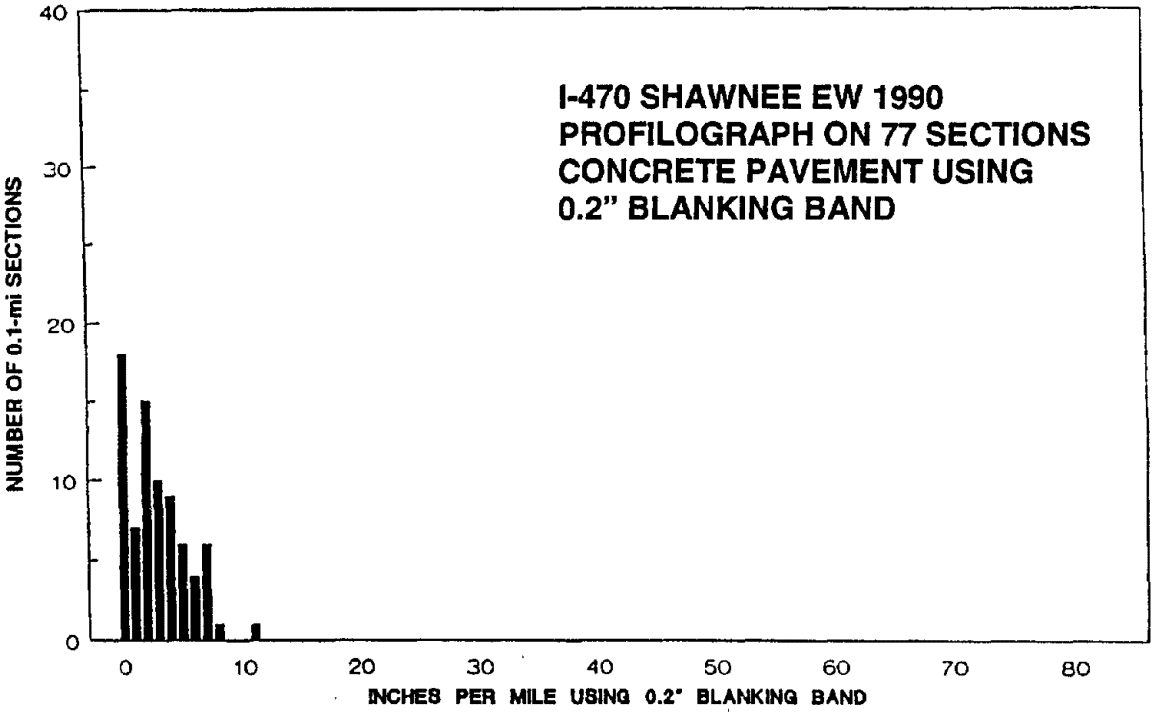
### Roughness Variability During Day

The initial smoothness measured with a profilometer showed an increase in smoothness between morning and afternoon readings of 7 to 10 percent for three different test dates. The report by Scofield et al. reported a rate of decrease in roughness of 32 to 47 mm/km/h.[30]




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 30. Histogram of section profiles—(1990) I-470—Shawnee County, Kansas (measured with a 0.0-mm blanking band).[28]




1 in = 25.4 mm, 1 mi = 1.61 km, 1 in/mi = 15.8 mm/km

Figure 31. Histogram of section profiles—(1990) I-470—Shawnee County, Kansas (measured with a 5.1-mm blanking band).[28]

Environmental Factors

One report stated that "the amount of rainfall within 72 hours of testing" directly influenced the measured smoothness.[31] The roughness (measured in terms of international roughness index [IRI]) was found to increase with an increase in rainfall. Fluctuations in air temperature prior to testing were also found to influence the test results.

**Variability Types**

Between-Device Variability

Scofield et al. performed an experiment in which four different profilographs were tested on two wheelpaths (one smooth and one rough). The test results ranged from 55 to 110 mm/km for the smooth wheelpath and 110 to 174 mm/km for the rough wheelpath.[30]

Between-Operator Variability

The study by Scofield et al. concluded that different operators produced statistically different profile indexes at a significance level of 7 percent.[30]

Within-Device Repeatability

Repeatability is defined as the "closeness of agreement between mutually independent test results obtained from the same wheelpath within the short time intervals by the same operator with the same device." All of the devices investigated in the study were found to be repeatable within 32 mm/km on the rough wheelpath and within 24 mm/km on the smooth wheelpath.[30]

Within-Operator Repeatability

The average computed from the test results produced by the four operators for each device was defined as the repeatability range. The average repeatability range was found to be 12 mm/km for the rough wheelpath and 9 mm/km for the smooth wheelpath. The main results of the 1992 Scofield et al. study are presented in table 31.

Another study investigated the repeatability of IRI measurements by different operators. The COV for IRI ranged from 0.32 to 14.33, with an average COV of 5.23. In addition, the standard deviations for the sections (each average reported is the summary three runs on the same section, by the same driver) ranged from 0.01 to 0.28, with an average standard deviation of 0.12.[31]

Table 31. Pavement roughness data obtained with profilographs.[30]

| Profilograph Type | Operator | Profile Index Readings (mm/km) | | | | | |
|---|---|---|---|---|---|---|---|
| | | Smooth Track | | | Rough Track | | |
| | | Test 1 | Test 2 | Difference | Test 1 | Test 2 | Difference |
| Mechanical 1 | 1 | 63 | 55 | 8 | 158 | 134 | 24 |
| | 2 | 55 | 55 | 0 | 134 | 118 | 16 |
| | 3 | 79 | 63 | 16 | 110 | 110 | 0 |
| | 4 | 63 | 63 | 0 | 110 | 126 | 16 |
| | Mean | 65 | 59 | 6 | 128 | 122 | 14 |
| Mechanical 2 | 1 | 79 | 79 | 0 | 174 | 174 | 0 |
| | 2 | 79 | 95 | 16 | 158 | 174 | 16 |
| | 3 | 87 | 95 | 8 | 142 | 150 | 8 |
| | 4 | 87 | 79 | 8 | 166 | 174 | 8 |
| | Mean | 83 | 87 | 8 | 160 | 168 | 8 |
| Automated 1 | 1 | 79 | 95 | 16 | 134 | 126 | 8 |
| | 2 | 79 | 103 | 24 | 118 | 134 | 16 |
| | 3 | 95 | 103 | 8 | 126 | 134 | 8 |
| | 4 | 87 | 87 | 0 | 118 | 126 | 8 |
| | Mean | 85 | 97 | 12 | 124 | 130 | 10 |
| Automated 2 | 1 | 95 | 103 | 8 | 174 | 174 | 0 |
| | 2 | 110 | 95 | 16 | 142 | 134 | 8 |
| | 3 | 110 | 103 | 8 | 126 | 158 | 32 |
| | 4 | 95 | 87 | 8 | 158 | 134 | 24 |
| | Mean | 103 | 97 | 10 | 150 | 150 | 16 |

### Variation in Trace Reduction

Considerable judgment is used by an operator when reducing a profilograph trace. The variation associated with trace reduction was also investigated by Scofield et al. in 1992.[30] The study consisted of presenting eight different profilograph traces (four from rough pavements and four from smooth pavements) to four different operators. Each operator then interpreted all eight traces. For the smooth wheelpath, all of the operator readings were within three standard deviations of the mean of 60 mm/km. The detailed results of this investigation are included in table 32.[30]

### Variation Due to Data Filter Settings

The 1992 report by Scofield et al. also included an experiment that evaluated the effect of reducing the digital filter setting (used with automated profilographs).[30] The authors investigated two operators and two automated devices at three settings on both rough and smooth pavements. The results from all 48 tests are summarized in table 33.

Table 32. Results of trace reduction by different operators.[30]

| Operator | Reading Replicate | Profile Index Readings (mm/km) for Different Operators | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Profile Trace Number | | | | | | | |
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| 1 | 1 | 79 | 142 | 87 | 126 | 134 | 142 | 182 | 189 |
| | 2 | 47 | 79 | 47 | 95 | 118 | 126 | 158 | 174 |
| Difference | | 32 | 63 | 39 | 32 | 16 | 16 | 24 | 16 |
| 2 | 1 | 55 | 71 | 79 | 103 | 110 | 126 | 150 | 166 |
| | 2 | 47 | 55 | 95 | 110 | 110 | 118 | 253 | 182 |
| Difference | | 8 | 16 | 16 | 8 | 0 | 8 | 103 | 16 |
| 3 | 1 | 24 | 47 | 24 | 39 | 110 | 103 | 126 | 142 |
| | 2 | 32 | 39 | 39 | 47 | 118 | 110 | 142 | 95 |
| Difference | | 8 | 8 | 16 | 8 | 8 | 8 | 16 | 47 |
| 4 | 1 | 24 | 39 | 47 | 47 | 103 | 118 | 150 | 150 |
| | 2 | 24 | 47 | 32 | 39 | 110 | 87 | 103 | 126 |
| Difference | | 0 | 8 | 16 | 8 | 8 | 32 | 47 | 24 |

Table 33. Profile index data obtained at different filter settings of electronic profilographs.[30]

| Filter Setting | Profilograph Type | Operator | Profile Index Readings (mm/km) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Smooth Wheelpath | | | Rough Wheelpath | | |
| | | | Test 1 | Test 2 | Mean | Test 1 | Test 2 | Mean |
| 8000 | Automated 1 | 1 | 79 | 87 | 83 | 150 | 174 | 162 |
| | | 2 | 63 | 71 | 67 | 126 | 126 | 126 |
| | Automated 2 | 1 | 95 | 95 | 95 | 150 | 182 | 166 |
| | | 2 | 103 | 87 | 95 | 150 | 126 | 138 |
| Mean | | | 85 | 85 | 85 | 144 | 152 | 148 |
| 6000 | Automated 1 | 1 | 71 | 55 | 63 | 126 | 134 | 130 |
| | | 2 | 63 | 55 | 59 | 134 | 118 | 126 |
| | Automated 2 | 1 | 63 | 95 | 79 | 150 | 126 | 138 |
| | | 2 | 71 | 79 | 75 | 126 | 142 | 134 |
| Mean | | | 67 | 71 | 69 | 134 | 130 | 132 |
| 4000 | Automated 1 | 1 | 71 | 63 | 67 | 110 | 126 | 118 |
| | | 2 | 55 | 47 | 51 | 95 | 87 | 91 |
| | Automated 2 | 1 | 47 | 47 | 47 | 134 | 95 | 114 |
| | | 2 | 71 | 79 | 75 | 110 | 110 | 110 |
| Mean | | | 61 | 59 | 60 | 113 | 105 | 109 |

The results showed that the variability due to devices was significant at the 1-percent level. An analysis of variance also indicated "significant interactions between operators and road roughness, and between operators and devices." The average difference between operators was found to be 9.6 mm/km for the filter investigation; the average difference between devices was 10.3 mm/km. The difference due to filter setting was almost 25.4 mm/km between each setting level (i.e., 8,000 = 116.8 mm/km, 6,000 = 101.0 mm/km, 4,000 = 83.6 mm/km). The authors found it difficult to determine specific effects from these settings due to strong interactions between operators and road roughness and between operators and devices.[30]

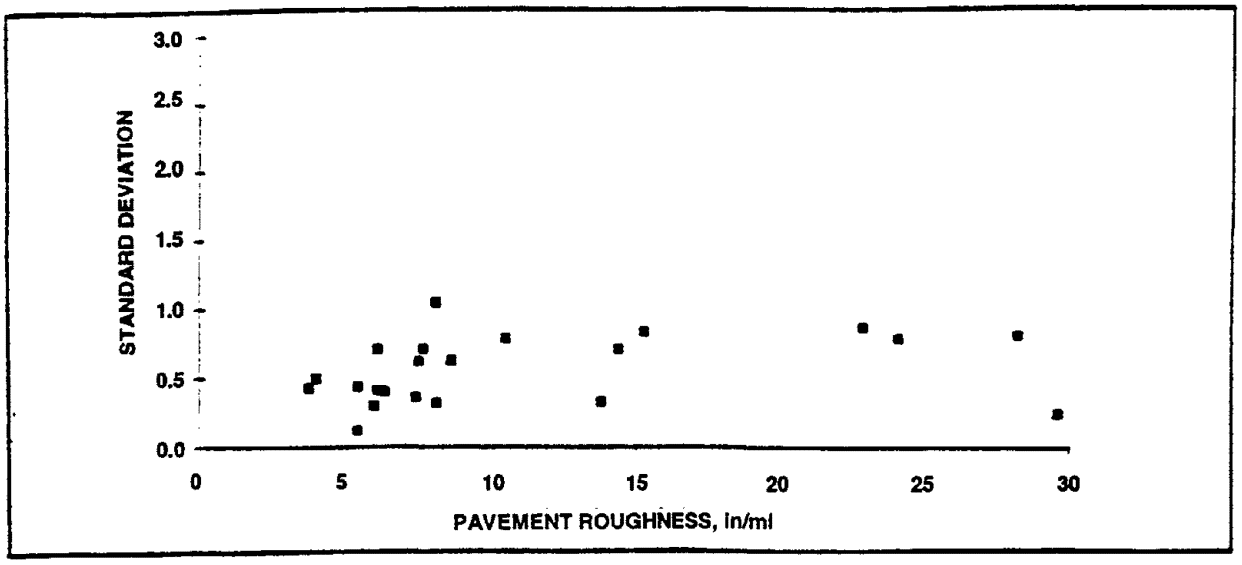
**Conclusions**

The following recommendations were extracted from the 1992 report by Scofield et al.[30]

- The current specifications simply expect too much from the California profilograph.
- The standard of 110 mm/km was established before slip form paving and long before electronic paver controls.
- One standard for all types of roadways does not appear to be appropriate for the wide range of pavement conditions found today.
- Currently, little or no information is available to determine if statistical sampling methods need to be developed to properly assess "true" roughness.
- The change in pavement roughness for both daily cycles and short-term roughness increases is not well-documented.
- There were some strong statistical interactions between some of the variables. These interactions make it difficult to account for the variability present in profilograph testing with limited experimentation such as with one machine or one operator.
- The average repeatability established was approximately 14.8 and 29.7 mm/km. Other studies found trace interpretation repeatability to be approximately 25.4 mm/km.
- Operator interpretation alone may approach a variability of 25.4 mm/km. These results encourage the use of more efficient computerized profilographs.
- As incentive specifications reward contractors for producing ever smoother pavements, consideration should be given to the effects this may have on concrete mix design and resulting concrete quality. Mix designs that promote

smooth pavements may produce surfaces with greater attrition and hence lower skid properties with time. Smooth pavement surfaces should be provided in concert with durable concrete pavements and not in lieu of them.

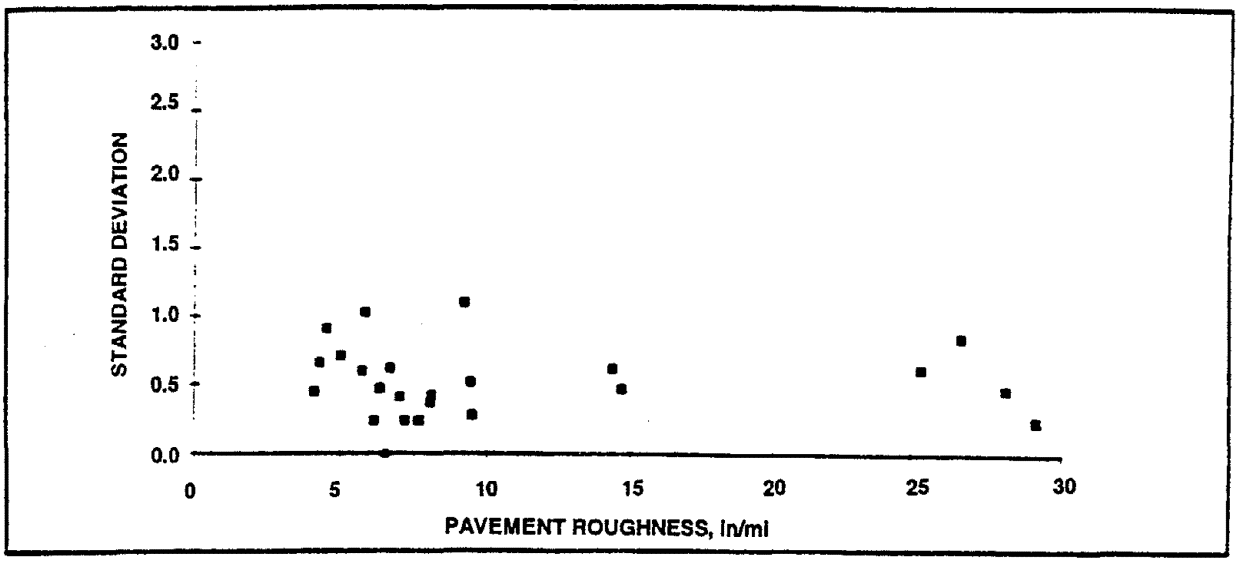
In a 1993 report, Scofield explains that measurement variability has classically been considered to include two components of measurement error: repeatability and reproducibility.[32] Repeatability refers to the "inherent precision of the device," whereas reproducibility refers to the measurement error due to different operators. Since it is difficult to know the exact conditions of testing for data collected under historical research, the variation described in the Scofield report was expressed as the standard deviation without regard to the number of operators. Figures 32 and 33 show plots from this study that illustrate the California profilograph variability as a function of JPC pavements for mechanical and automated profilographs.[32] (Note: Figures 32 and 33 are presented in English units.)




1 in/mi = 15.8 mm/km

Figure 32. Profilograph variability as a function of JPCP for mechanical profilographs.[32]

The results of this study indicated that test variability associated with mechanical profilographs is a function of initial smoothness (i.e., as roughness increases, so does variability). The variability associated with a mechanical profilograph is "generally never less than 8 mm/km, and it is approximately one-tenth the measurement range." This means a pavement with an initial smoothness of 158 mm/km should have a standard deviation of approximately 15.8 mm/km. It was also reported that this variability is a function of pavement smoothness because of trace reduction.[32]




1 in/mi = 15.8 mm/km

Figure 33. Profilograph variability as a function of JPCP for automated profilographs.[32]

The variability of automated profilographs was not found to be a function of smoothness level. The author reports that this variability is typically fixed between 8 and 16 mm/km for all roughness levels.[32]

## JOINT SAWCUT DEPTH

A 1993 study by Zollinger et al. investigated the sawcut depth requirements using fracture mechanics analysis.[33] Historically, a sawcut depth of one-third the pavement thickness (d/3) has been recommended; however, little technical justification has been found to support this guideline. This study focused on determining the appropriate sawcut depth using modified linear fracture mechanics. The results of the study were stated as the following:[33]

> "The reduction of sawcut depth (less than d/3 or d/4) at concrete joints by early-aged sawcut or placement techniques can take advantage of the greater change in moisture and temperature in the concrete at the pavement surface (in comparison to the change at d/3 or d/4) to initiate cracking at the notches at a greater incidence than otherwise. Therefore, the control of cracking of concrete pavement should be improved. Field surveys indicate that under some circumstances (such as cool weather paving conditions) transverse cracks at the sawcut notches may initiate much later after placement. On a preliminary basis, this study indicates that it is reasonable to use notch depths on the order of one inch to initiate cracking at the pavement surface which is significantly less than the traditional d/4 or d/3."

### Variation of Joint Sawcut Depth

A recent ERES study for the Ontario Ministry of Transportation measured the depth of sawing on 40 cores taken across the longitudinal joint.[34] The following data were collected:

| | |
|---|---|
| Target Pavement Thickness | 200 mm |
| Range of Depth Measurements | 33 to 67 mm |
| Average Sawcut Depth | 54 mm |
| Standard Deviation of Sawcut Depth | 8.8 mm |
| Specified Sawcut Depth | 66 mm |

Sawcut depth data for one project using an abrasive sawblade were also reported in a recent FHWA study of sawcutting.[35] Sawcut depth for 22 transverse joints averaged 91 mm, with a standard deviation of 8 mm. For the 22 longitudinal joints measured, sawcut depth averaged 94 mm, with a standard deviation of 6 mm.

## TIMING OF JOINT SAWING

### Investigation of Earliest Joint Sawcutting

An investigation of joint sawcut timing was performed by Okamoto et al. in 1993.[36] Slabs were placed in an exterior area and sawcut to one-third slab depth over a range of time after concrete placement. A 48-kW walk-behind saw operated at about 1500 rpm's was used for making sawcuts. The saw was operated by a technician with limited concrete sawing experience, to minimize operator bias. The diamond blades used made approximately 4-mm-wide sawcuts. The timing of final sawcuts showing no raveling ranged from 3.9 to 12.1 h after concrete placement.[36]

The sawcuts in the study were rated from 1 to 5 based on the severity of edge raveling. The following are the categories:

- 1 = badly spalled - sawcut too early.
- 2 = unacceptable.
- 3 = acceptable if sealant reservoir widening is to be done.
- 4 = good.
- 5 = excellent.

Compressive strength was then selected to be the early sawing criterion. Compressive strength was related to the previously listed sawcut ratings using equation 7.[36]

$$(f_c')^{1/2} = 13.9432 + 5.1931*R + 8.7086*H + 5.4198*G - 0.0263*C \quad (7)$$

where

$f_c'$ = Concrete compressive strength, psi (100 psi = 0.69 MPa).

R = Sawcut rating.
H = Coarse aggregate hardness (soft = 0, hard = 1).
G = Coarse aggregate geometry (rounded = 0, crushed = 1).
C = Cement content, lb/yd$^3$ (500 lb/yd$^3$ = 297 kg/m$^3$).

This equation may then be used to determine required concrete compressive strengths for producing acceptable and good sawcuts.[36]

### Investigation of Latest Joint Sawcutting

Cracks were observed below all transverse joint sawcut notches within 20 h after concrete placement.[36]

### Recommended Timing of Joint Sawcutting

Alternate tests to compressive strength determination were selected to help determine the near limit for sawcutting. NDT methods such as pulse velocity and maturity can be used to monitor in situ concrete pavement compressive strength. Either method can be correlated well with compressive strength; however, these relationships are influenced by changes in aggregate source, cement source, admixture types, and paste volumes.[36]

In order to control cracking and widen the sawcutting window of opportunity, it is recommended that paving start in the early morning hours (or even at night) and should be completed by about 10:00 to 11:00 a.m. Such a paving schedule helps to "minimize top and bottom slab temperature differentials, minimize average slab temperature drops, and maximize concrete strength gains ahead of developing tensile restraint stresses." A good rule of thumb may be to start sawcutting as early as possible and no later than the time when surface cooling starts.[36]

### Timing of Joint Sawing Conclusions

Although some information was located on recommended practices for sawing joints, no data were found regarding the specific relationship between timing of joint sawcutting and performance. Therefore, no useful information regarding the variation of timing of sawing was located.

## PERCENT CONSOLIDATION AROUND DOWELS

Very few reports involving the concrete consolidation level were located. Of those located, none contained much detailed discussion of the variation of consolidation level around joints or its influence on pavement performance. Brief summaries of the reviewed reports are included below.

One report addressed consolidation as a function of distance from vibrator path, seconds of vibration, and speed of paver.[37] However, no details were provided on the

consolidation level around dowel bars. Therefore, the results of this study are not included as part of this summary.

A 1987 report by Tayabji and Whiting presented results indicating that compressive strength sustains a 30-percent loss for every 5-percent decrease in consolidation.[38] The authors stated that reduced service life and poor performance of doweled joints are two effects of inadequate consolidation. Faulting, pumping, and bottom or surface spalling may occur at joints. A laboratory study was conducted that consisted of varying the degree of consolidation from 85 to 100 percent. The laboratory results included the following:

- A 5-percent decrease in consolidation resulted in a 30-percent decrease in compressive strength and a 50-percent reduction in bond strength.
- Overconsolidation had little effect on compressive strength and may improve bond strength due to the displacement of air in air-entrained concrete.

Recommendations were made for using *degree of consolidation* as a means of concrete acceptance. The relationship shown as equation 8 between percent consolidation and compressive strength was recommended by the authors.[38]

$$f_{cx} = 10^{(2-[A_1(100-x)])} \quad (8)$$

where

$f_{cx} = f_{c,x}/f_{c,100}$
$f_{c,x}$ = Compressive strength at $x$-percent consolidation.
$f_{c,100}$ = Compressive strength at 100-percent consolidation.
$A_1$ = Regression constant.

The consolidation compressive strength relationship was also used to establish the relationship between flexural strength and degree of consolidation. The projected service life of a concrete pavement was determined as a function of degree of consolidation. The authors explained that methods exist to relate the effect of degree of consolidation to pavement service life based only on the strength characteristics. Therefore, to develop acceptance plans for concrete consolidation, only the effect of consolidation on strength (and, therefore, on expected pavement service life) is considered. The authors assumed that "if the strength-based acceptance criteria are satisfied, then there will be sufficient assurance that the other characteristics such as steel-concrete bond strength and permeability to chloride ions would be acceptable." A model acceptance plan based on degree of consolidation was developed. The buyer's and seller's risks were set at 5 percent.

In a look at typical consolidation levels, the authors stated that "a mean degree of consolidation of at least 98 percent of the rodded unit weight is achievable on well-

constructed projects."[38] The collected data indicated that average overall standard deviation for consolidation is about 32 kg/m$^3$ (measured with a nuclear gauge). This value is approximately 1.4 percent of the rodded (reference) unit weight. The following criteria were used to develop the model sampling plan:[38]

- Achievable standard deviation for the population of degree of consolidation measurements = 1.5 percent.

- Lower acceptance limit for degree of consolidation measurements = 97 percent. This lower limit should be as high as achievable in the field.

Finally, Tayabji and Whiting looked at measuring concrete consolidation with four different nuclear gauge types: direct transmission, backscatter, consolidation monitoring device (CMD), and twin-probe. A comparison of the four gauges is included in table 34.

A discussion of the relationship between percent consolidation and concrete compressive strength was included in a draft report prepared by ACI Committee 214.[4] Although the report concentrates on "structures," it does provide interesting information on this subject. The report explains that the commonly accepted rule of thumb has been that one can expect a 5-percent decrease in strength for every 1-percent increase in air content. For example, the strength of a mix with 7-percent air content is likely to be 10 percent less than the strength of a mix with 5-percent air.

Another report was referenced in which that author proposed the following expression relating compressive strength to air content:

$$f_{rel} = 10^{-0.035v} \quad (9)$$

where $f_{rel}$ is the relative compressive strength of concrete when the air-free strength is unity, and v is the air content of the concrete in percent.[39]

A report by Gerhardt et al. discussed the use of a nuclear device for measuring concrete density.[40] The report describes a study in which core densities were compared to nuclear density values. The study concluded that the nuclear device is at least as reliable as coring for determining densities. It did, however, recommend that the level of the detector or the radioactive source be specified since the nuclear device is sensitive to the increased density observed at lower layers in a slab.

A report by Heston and Wajokowski presents data from a study in Kansas that investigated the effect of consolidation (density) on transverse joint faulting.[41] The study investigated a section of pavement on U.S. 69 that was constructed in 1979. During construction, 23 test sections were placed with various adjustments to vibrator settings, concrete admixtures, and other features. The concrete density of all test sections was measured at time of construction using a nuclear meter. Seven of the test sections have been surveyed on a yearly basis since 1987. The results have indicated

Table 34. Relative advantages and disadvantages of nuclear density gauge types.[38]

| Gauge Type | Advantages | Disadvantages |
|---|---|---|
| Direct Transmission | • Includes full concrete thickness in measurement.<br>• Little chemical interference.<br>• Widely used in other areas, commercially available.<br>• Fairly easy to calibrate, precision is good.<br>• Can avoid steel by proper gauge positioning. | • Disturbance of concrete; only measures small volume of concrete placed.<br>• Relatively slow measurement.<br>• Difficult to clean gauge completely.<br>• May be difficult to use where reinforcement is congested.<br>• Radiation monitoring required. |
| Backscatter | • Easy to perform. Minimal disturbance of concrete.<br>• Widely used in other areas, commercially available.<br>• Satisfactory precision.<br>• Useful on thin overlays (more sensitive to surface layers).<br>• Facilitates cleanup. | • Insensitive to deep layers.<br>• Volume of influence ill-defined.<br>• Sensitive to chemical effects.<br>• Reinforcing steel and underlying concrete may interfere.<br>• Radiation monitoring required. |
| Twin-Probe | • Depth localization of density.<br>• Long path length included in measurement.<br>• Sensitive to small voids.<br>• Little chemical effect.<br>• Small radiation source.<br>• Good precision. | • Increased disturbance of concrete.<br>• Slow measurement.<br>• Difficult to operate.<br>• Equipment not commercially available.<br>• Radiation monitoring required. |
| CMD | • Large volume of concrete included in measurement.<br>• Adaptable to process control.<br>• No disturbance of concrete.<br>• Satisfactory precision. | • Sensitive to surface layers only.<br>• Measurement value represents average of volume scanned.<br>• Sensitive to chemical effects.<br>• Difficult to calibrate.<br>• Awkward to handle.<br>• Increased radiation hazard.<br>• Needs custom installation for each paver type.<br>• Measurements near pavement edges restricted.<br>• Radiation monitoring required. |

that higher density in the consolidated concrete results in less faulting at doweled joints at a given age. Figure 34 shows a plot from the study supporting this conclusion. (Note: Figure 34 is presented in English units.)

The Road and Traffic Authority's *Concrete Pavement Manual* contains a plot that shows a relationship between compressive strength ratio and density ratio. This plot is contained in figure 35.[42]

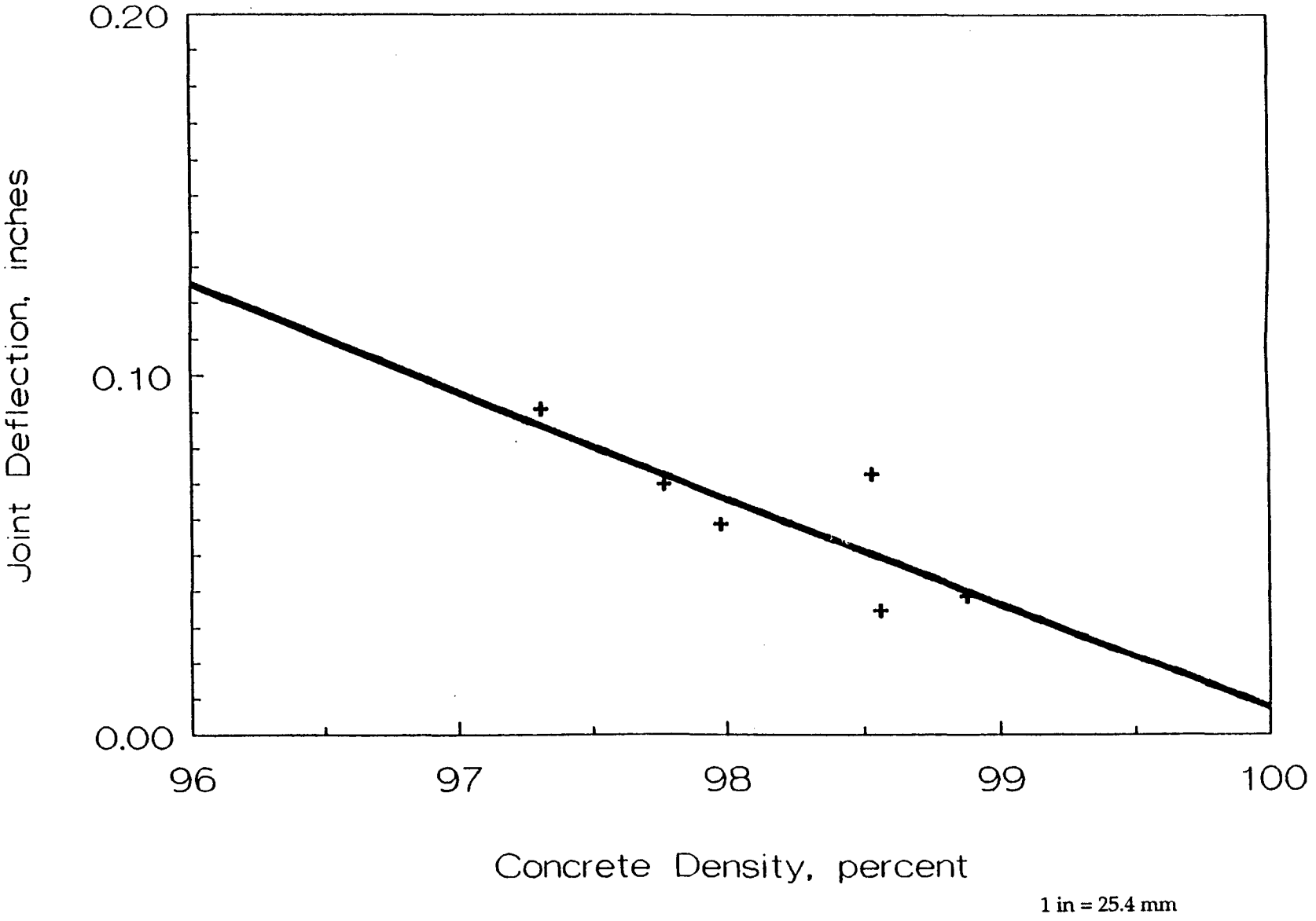



Figure 34. U.S. 69 concrete pavement joint faulting (1992 deflection versus original density).[41]

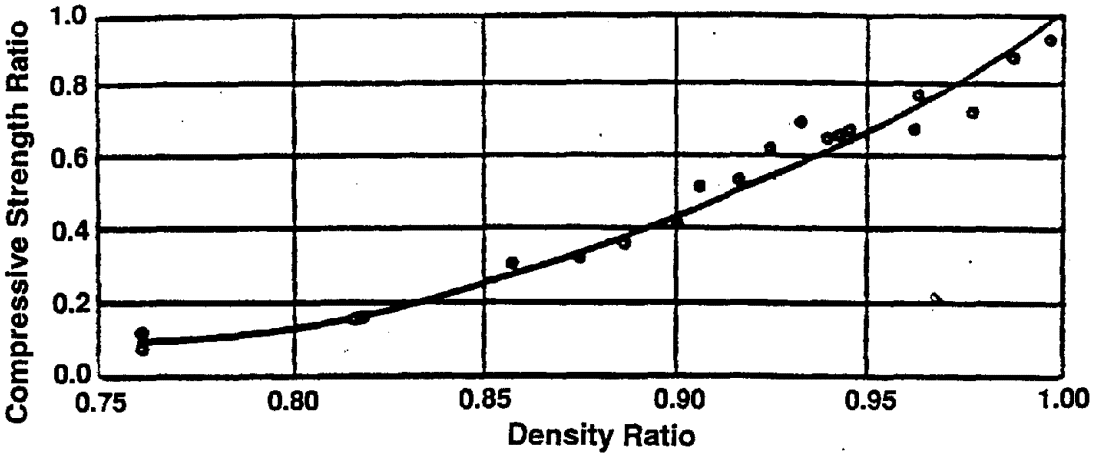



Figure 35. Plot of compressive strength ratio versus density ratio.[42]

## DOWEL MISALIGNMENT

Dowel misalignment is the combination of both misplacement and displacement of dowel bars. Misplacement is the initial incorrect placement of dowels (baskets only). Displacement involves movement of dowels during the paving operation. Both are expressed as percentages, where the percentage is figured by dividing the end-to-end misalignment by the dowel length.[43] Figure 36 explains the components making up dowel misalignment and their effect on pavement performance.[44]

### Summary of Dowel Misalignment Studies

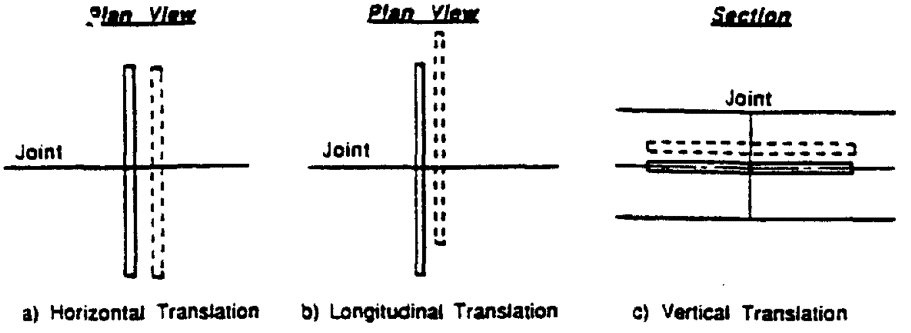
A number of different studies discussed details of dowel misalignment and its effect on pavement performance. A short summary of each is presented below.

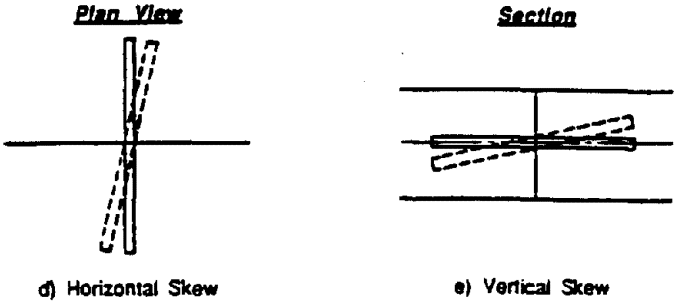
Study #1[45]

Misplacement measurements were taken for 37 joints (728 individual dowels) using simple, but effective, equipment. The accuracy of the equipment used was 0.5 percent vertically and 0.7 percent horizontally. Displacement measurements were collected by equipping 18 dowel bars, 3 each of 6 different assemblies, with motion-sensing transducers to measure the movement during paving. The mean residual displacement (position before placing concrete minus the position after concrete hardens) was measured, and both vertical and horizontal misplacement and displacement data were collected.

*Misplacement Summary*

Results showed that 5 percent of the dowels were misplaced more than 3.5 percent (13 mm) horizontally and 2 percent (6 mm) vertically. These percentages were taken as




| Type of Misalignment | Effect on | | |
|---|---|---|---|
| | Spalling | Cracking | Load Transfer |
| a | - | - | Yes* |
| b | - | - | Yes* |
| c | Yes | - | Yes* |
| d | Yes | Yes | Yes |
| e | Yes | Yes | Yes |

*Effect will depend on amount of translation

Figure 36. Effects of dowel misalignment.[44]

the *attainable* tolerances (not to be confused with the *acceptable* tolerances). Horizontal misplacement was found to be a more serious problem than vertical misplacement. Only 5 percent of the dowel assemblies were misplaced more than 1.5 percent vertically.[45]

Horizontally, about 30 percent of dowel assemblies were misplaced more than 1.5 percent. The authors, therefore, concluded that the measured horizontal misplacement was due to the improper setting of entire assemblies, rather than deviations of individual bars within the assemblies. It may be sufficient to check the assemblies rather than individual bars. The following statistics were compiled for misplacements.[45]

MEAN MISPLACEMENTS (Baskets Only)
Mean Vertical Misplacement: 0.83 percent.
Mean Horizontal Misplacement: 1.16 percent.

MEDIAN MISPLACEMENTS (Baskets Only)
Median Vertical Misplacement: 0.4 percent.
Median Horizontal Misplacement: 0.6 percent.

*Displacement Summary*

VERTICAL DISPLACEMENT RESULTS (Baskets Only)

| | |
|---|---|
| Mean Residual Displacement: | 0.23 percent. |
| Standard Deviation of Residual Displacement: | 0.20 percent. |
| Maximum Displacement: | 0.50 percent (about 2 mm). |

HORIZONTAL DISPLACEMENT RESULTS (Baskets Only)
Data not collected. Assumed to be on the same order as the vertical displacement (about 2 mm).

*Misalignment Tolerance Information*

Ross and Rutkowski stated that recommended tolerances generally range from 1 to 4 percent. This study found displacement to be small compared to misplacement. Therefore, misalignment was approximated using the misplacement data.[45]

Study #2[46]

A 1986 report by Tayabji contains a section that summarizes previous findings related to the measurement of dowel misalignment tolerances. He reported that past recommendations have generally stated an acceptable alignment error of 6.4 mm per 45.7 cm of dowel length. In 1979, the Illinois Department of Transportation specified that any deviation from correct alignment greater than 3.2 mm in 30.5 cm should be corrected before any concrete is placed. For skew, the Georgia DOT uses an allowable tolerance of 9.5 mm per 30.5 cm in both the horizontal and vertical directions. A Pennsylvania DOT study from 1980 was presented. Field data of levels of misalignment were presented. These data consisted of skew and translation measurements in both the horizontal and vertical directions. Data were present for both basket assemblies and implanted dowels. Ten dowels for each placement type were investigated to obtain the field data. These field data were presented with the Pennsylvania DOT-specified acceptable tolerances.

Study #3[47]

This study was conducted "to determine the effectiveness of an automatic dowel bar inserter at properly placing dowel bars in rigid pavements." Data were collected from a project on I-45 (south of Dallas, Texas) in February and March 1987. A radar system was used to evaluate dowel placement along 51 basket assemblies and 52 inserter joints. The radar data were calibrated with 32 cores taken from both the dowel basket and

inserter sections. An analysis of the results showed the following vertical and horizontal misalignments:

*Dowels Placed with Baskets*

Vertical Misalignment

| | |
|---|---|
| Average: | 11.7 mm/m (1.2 percent). |
| Standard Deviation: | 10.0 mm/m (1.0 percent). |

Horizontal Misalignment

| | |
|---|---|
| Average: | 28.3 mm/m (2.8 percent). |
| Standard Deviation: | 25.0 mm/m (2.5 percent). |

*Dowels Placed with an Inserter*

Vertical Misalignment

| | |
|---|---|
| Average: | 7.5 mm/m (0.8 percent). |
| Standard Deviation: | 5.8 mm/m (0.6 percent). |

Horizontal Misalignment

| | |
|---|---|
| Average: | 27.5 mm/m (2.8 percent). |
| Standard Deviation: | 25.8 mm/m (2.6 percent). |

Study #4[48]

This study had two specific objectives: (1) to determine the final position of dowel bars placed by a mechanical implanter and (2) to investigate the effect of misaligned dowels on joint distress in Georgia pavements. Five projects were selected for investigation and comparison. Dowels were placed with mechanical implanters on three projects (projects A, B, and C) and with baskets on two projects (projects D and E). The project investigated a total of 1,044 dowel bars and 261 joints. Figure 37 shows frequency graphs of deviation of dowel bars from mid-depth for all five investigated projects. (Note: Figure 37 is presented in English units.) Table 35 presents the summary of the percent occurrence of vertical rotation measurements for each of the five projects (the specification allowed 14.3 mm of vertical rotation).

Horizontal rotation measurements were measured with cores and a metal detector. The specification requirements allowed 28.6 mm rotation for projects A through D and 14.3 mm on project E. A summary of the horizontal rotation measurements is presented in table 36.

Longitudinal alignment was measured in the study and compared to an acceptable tolerance of ± 25.4 mm. A summary of the longitudinal alignment measurements is presented in table 37.

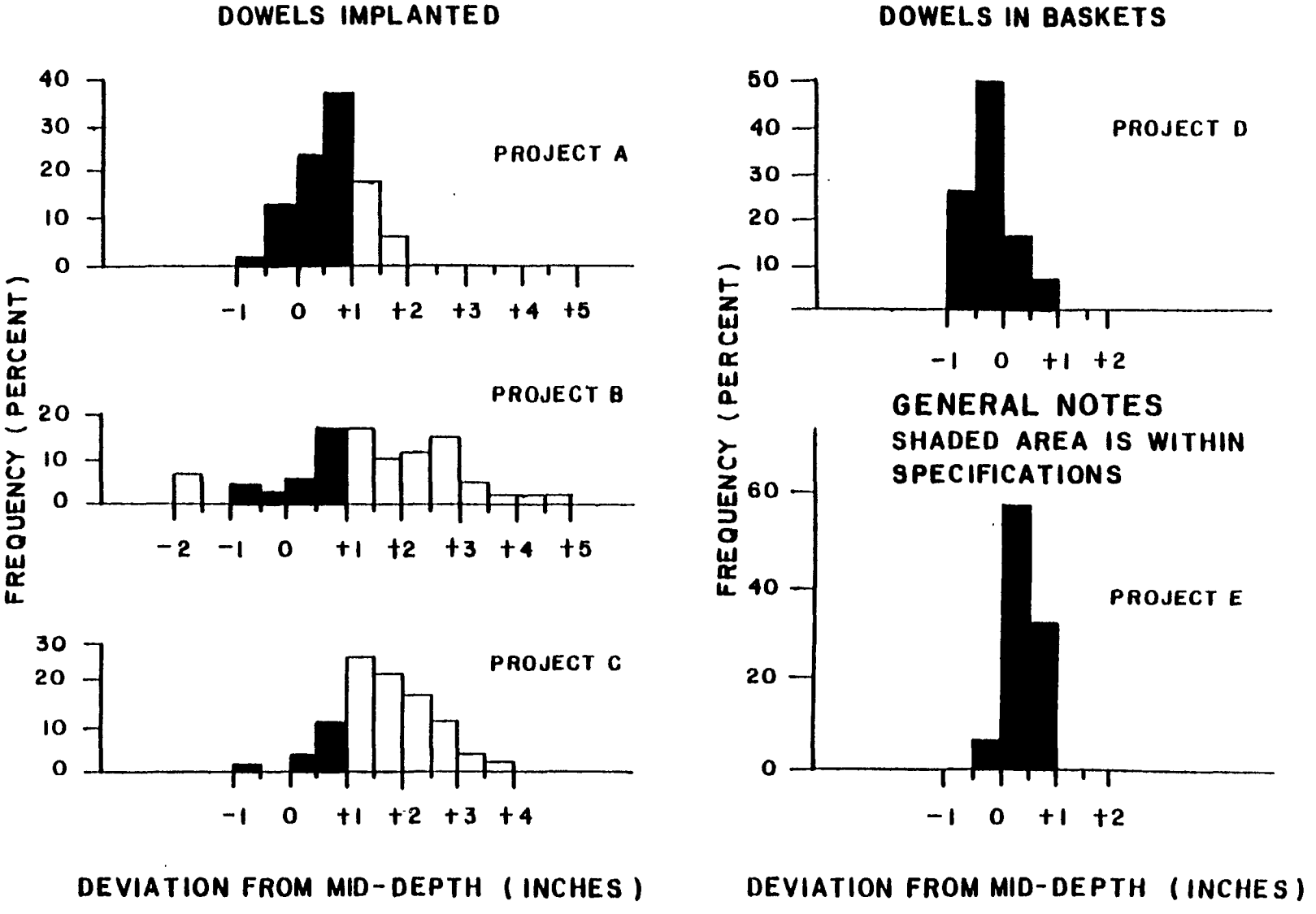



Figure 37. Frequency graphs of deviation of dowel bars from mid-depth for five investigated projects.[48]

Table 35. Summary of vertical rotation measurements.[48]

| Vertical Rotation Range, mm | Percent Occurrence | | | | |
|---|---|---|---|---|---|
| | A | B | C | D | E |
| 0 | 35 | 25 | 20 | 35 | 33 |
| 0 to 6.4 | 63 | 57 | 49 | 82 | 92 |
| 0 to 12.7 | 80 | 83 | 72 | 95 | 100 |
| 0 to 19.1 | 96 | 92 | 87 | 97 | – |
| 0 to 25.4 | 96 | 95 | 93 | 100 | – |

Table 36. Summary of horizontal rotation measurements.[48]

| Horizontal Rotation Range, mm | Percent Occurrence | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | A | | B | | C | | D | | E | |
| | Core | Detector | Core | Detector | Core | Detector | Core | Detector | Core | Detector |
| 0 | 26 | 12 | 12 | 14 | 22 | 9 | 45 | 18 | 36 | 23 |
| 0 to 6.4 | 54 | 29 | 29 | 34 | 37 | 25 | 85 | 56 | 76 | 63 |
| 0 to 12.7 | 72 | 68 | 46 | 60 | 69 | 46 | 95 | 76 | 95 | 90 |
| 0 to 25.4 | 91 | 90 | 75 | 85 | 80 | 78 | 100 | 96 | 100 | 100 |
| 0 to 38.1 | 100 | 96 | 88 | 93 | 93 | 93 | 100 | 99 | 100 | 100 |

Table 37. Summary of longitudinal alignment.[48]

| Length, mm | Percent Occurrence | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | A | | B | | C | | D | | E | |
| | Core | Detector | Core | Detector | Core | Detector | Core | Detector | Core | Detector |
| 0 to 50 | | | 3.4 | 1.1 | | | 1.7 | 1.7 | | |
| 51 to 101 | | | 1.7 | 3.3 | | | 1.7 | 0.6 | | |
| 102 to 151 | 1.9 | 6.6 | 10.2 | 11.1 | 4.3 | 2.9 | – | 1.1 | | |
| 152 to 202 | 22.2 | 21.1 | 22.0 | 16.7 | 23.9 | 18.8 | 3.3 | 2.2 | 16.7 | 11.1 |
| 203 to 254 | 35.2 | 32.2 | 25.4 | 34.4 | 37.0 | 40.6 | 43.3 | 45.6 | 78.6 | 77.8 |
| 255 to 304 | 29.6 | 25.6 | 16.9 | 14.4 | 15.2 | 23.9 | 28.3 | 26.1 | 4.8 | 10.3 |
| 305 to 355 | 9.3 | 11.8 | 10.2 | 13.3 | 17.4 | 10.9 | 10.0 | 11.1 | | 0.8 |
| 356 to 405 | 1.9 | 2.6 | 3.4 | 3.9 | 2.2 | 2.9 | 6.7 | 8.3 | | |
| 406 to 455 | | | | | | | 3.3 | 1.7 | | |
| out of joint | | | 6.8 | 1.7 | | | 1.7 | 1.7 | | |

The Fowler–Gulden report also presented results of a previous study conducted by the Alabama Highway Department in 1967.[48] This Alabama study concluded that misalignments of 6.4 mm or less were insignificant regardless of the plane of misalignment. Also, dowel misalignment errors in the horizontal plane could be as great as 19.1 mm and not affect the load required to produce a 12.7-mm joint opening. However, dowel misalignment errors in the vertical plane could only be as great as

6.4 mm without increasing the load required to produce the same resulting 12.7-mm joint opening.

Study #5[49]

This study investigated the dowel misalignments associated with transverse contraction joints only. Dowel bar alignment error was defined in this report as the "square root of the sum of the squares of the horizontal and vertical errors when the total length of the bar is projected on an end plane." The horizontal and vertical errors are the misalignments in the horizontal and vertical planes, respectively. In a previous study by Grinter, it was suggested that 6.4 mm be the maximum tolerated misalignment based on theoretical calculations. In another study published in 1940, Friberg stated that "the efficiency of the joint and the dowels are seriously impaired by greater misalignments than 10.4 mm/m although the edges of the joints may not fail." Friberg also suggested that "the stresses in the concrete due to misalignment and extreme joint movement may be higher than those caused by traffic loads."

Segner and Cobb described a 1938 study in which Smith and Benham conducted research that indicated misalignment tolerances for 127- and 152-mm-thick pavements. Their results showed that an alignment error of 25.4 mm could occur in a 152-mm-thick pavement without causing spalling. However, for 127-mm-thick pavements, alignment errors of 6.4 mm caused slight spalling.

Results of the study by Segner and Cobb indicated that an alignment error in the vertical or oblique plane is more critical than the horizontal plane—in terms of the load required to produce joint openings of 12.7 mm and 19.1 mm, respectively. Dowel alignment errors in the horizontal plane may be as great as 19.1 mm without requiring an appreciable increase in load to produce joint openings of 12.7 mm and 19.1 mm, respectively. However, dowel bar alignment errors greater than approximately 6.4 mm in the vertical or oblique plane result in a severe increase in the load required to produce a corresponding joint opening. Finally, Segner and Cobb indicated that significant spalling failures may occur at contraction joints when alignment errors approach 25.4 mm in the vertical plane or 76 mm in the horizontal plane.

Study #6[50]

Soriano presents data from three different projects conducted in South Dakota in 1989. Statistics for the three projects concerning the dowel bar depth, vertical rotation, horizontal rotation, and longitudinal displacement are presented in tables 38 through 41.

**Factors Affecting Dowel Misalignment**

In a 1986 report, Tayabji identified factors that affect dowel misalignment for both placement methods (baskets and implanters).[46] For dowels placed on basket assemblies, the following factors were identified:

Table 38. Dowel bar depth statistics for three projects constructed in South Dakota in 1989.[50]

| Project | Average Depth, mm | Std. Dev., mm | Allowable Range, mm* | Number out of Spec./ Number Tested | Percent out of Specification |
|---|---|---|---|---|---|
| A | 108.9 | 9.0 | 99.3 – 115.1 | 46/132 | 34.8 |
| B | 108.2 | 8.4 | 98.4 – 114.2 | 52/160 | 32.5 |
| C | 108.1 | 7.0 | 104.0 – 119.8 | 41/116 | 35.3 |

* Based on projects' average thickness.

Table 39. Dowel bar average vertical rotation statistics for three projects constructed in South Dakota in 1989.[50]

| Project | Average Vertical Rotation, mm | Std. Dev., mm | Allowable Range, mm | Number out of Spec./ Number Tested | Percent out of Specification |
|---|---|---|---|---|---|
| A | 1.1 | 2.4 | 0 – 3.2 | 1/132 | 0.8 |
| B | 2.4 | 7.8 | 0 – 3.2 | 19/160 | 11.9 |
| C | 1.0 | 1.2 | 0 – 3.2 | 0/116 | 0.0 |

Table 40. Dowel bar horizontal rotation statistics for three projects constructed in South Dakota in 1989.[50]

| Project | Avg. Horizontal Rotation, mm | Std. Dev., mm | Allowable Range, mm | Number out of Spec./ Number Tested | Percent out of Specification |
|---|---|---|---|---|---|
| A | 6.6 | 4.6 | 0 – 12.7 | 10/132 | 7.6 |
| B | 10.6 | 9.8 | 0 – 12.7 | 39/172 | 22.7 |
| C | 8.3 | 6.4 | 0 – 12.7 | 21/116 | 18.1 |

Table 41. Dowel bar longitudinal displacement statistics for three projects constructed in South Dakota in 1989.[50]

| Project | Average Longitudinal Displacement, mm | Standard Deviation, mm | Allowable Range, mm | Number out of Spec./ Number Tested | Percent out of Specification |
|---|---|---|---|---|---|
| A | 17.1 | 21.2 | 0 – 25.4 | 25/132 | 18.9 |
| B | 27.2 | 36.0 | 0 – 25.4 | 59/172 | 34.3 |
| C | 22.0 | 25.4 | 0 – 25.4 | 30/116 | 25.9 |

- Basket rigidity.
- Quality control during basket fabrication.
- Care during basket transportation and placement.
- Fastening of basket to subbase.
- Location of saw cut over basket.
- Paving operation (the large roll of concrete ahead of the paver may displace individual dowels or the basket assembly).
- Field inspection during construction.

For dowels placed using mechanical inserters, the following factors affected misalignment:

- Implanting machine operation.
- Strike-off after dowel placement.
- Consolidation (vibration) after dowel placement.
- Location of saw cut over implanted dowels.
- Field inspection during construction.

**Methods of Measuring Misalignment**

A number of different methods have been used to measure dowel misalignment. One method is a radar system. These measurements are influenced by operator experience, quality of calibration, and equipment operation. Test results for 33 measurements showed that the standard deviation for the differences between actual and measured depths is 6.1 mm. The results indicate that there is 95-percent confidence that the actual depth of the dowel bar at a point is within ± 11.9 mm of the estimated depth. Also, there is 95-percent confidence that the actual misalignment of a dowel is within ± 17.0 mm of the estimated misalignment.[44]

In another report, Soriano concluded that the Soiltest CT-4950 Micro-Covermeter could be used to accurately inspect dowel bars.[50]

**Possible Solution to Misalignment Problems**

One report offered a possible solution to cracking and stresses caused by dowels and tie bars.[51] The author recommends the use of square bars with clip-on plastic sheaths.

These sheaths have compressive material on the sides that compensates for horizontal misalignment. When a joint opens, a horizontally misaligned bar does not have to bend; the compressive material simply compresses. Also, by using square bars, the load transfer is maintained. The author cites additional advantages of using square bars, including providing more resistance to bending and avoiding the induction of tensile stresses in the slab due to the shape of the round bars. Also, the plastic sheath provides additional resistance to corrosion.

The questions of vertical misalignment and horizontal and vertical translation still remain. The author avoids discussing these topics by saying they are pretty much avoidable. It was also suggested to use square bars as tie bars by providing some resistance to pull-out. This may be done by manufacturing a dumbbell-shaped bar, or more simply by drilling holes and attaching bolts to the bars.

### Summary of Dowel Misalignment Tolerances

Table 42 contains a summary of *suggested* misalignment tolerances based on observed data from a number of studies. Table 43 presents a summary of the reported *acceptable* misalignment tolerances included in different construction specifications.

## TIE BAR DEPTH DEVIATION (FROM MID-SLAB) AND ALIGNMENT

No interesting information was located regarding the specific relationship between tie bar depth deviation (from mid-slab) or alignment and performance. Also, no useful information regarding the variation of tie bar depth deviation (from mid-slab) was located in the literature search.

## EFFECTIVENESS OF CURING

One interesting report relating the effectiveness of curing to pavement performance was located in the literature search.[52] The report discusses the in situ measurement of surface wave velocity using a spectral-analysis-of-surface-waves (SASW) method. The results showed that this type of seismic measurement is a reliable method for determining PCC stiffness during curing. Therefore, this type of testing may be useful in the future in assessing the degree of curing that has occurred.

## DEPTH OF TINING

A number of reports related to PCC surface texturing were found in the literature search. Summaries of the most useful reports are included in the following sections.

### Variation of Depth of Tining

The variability of tining depth was measured on 9 test pavements and 25 bridge decks in New York.[53] Histograms of the reported tining depths are presented in figure 38. (Note: Figure 38 is presented in English units.)

Table 42. Suggested misalignment tolerances presented in different research studies.

| Data Source, (Reference) | Year | Relevant Placement Type | Vertical Skew, % | Horizontal Skew, % | Vertical Translation, % | Horizontal Translation, % |
|---|---|---|---|---|---|---|
| Study #1 [45] | 1989 | Baskets | X=2.0, SD=NA | X=3.5, SD=NA | NA | NA |
| Study #2 [46] | 1986 | Baskets | X=1.1, SD=1.2 | X=0.7, SD=0.8 | X=2.4, SD=2.1 | X=4.7, SD=2.1 |
| Study #2 [46] | 1986 | Inserter | X=0.6, SD=0.3 | X=1.5, SD=1.2 | X=4.3, SD=1.2 | X=2.2, SD=1.6 |
| Study #3 [47] | 1988 | Baskets | X=1.2, SD=1.0 | X=2.8, SD=2.5 | NA | NA |
| Study #3 [47] | 1988 | Inserter | X=0.8, SD=0.6 | X=2.8, SD=2.6 | NA | NA |

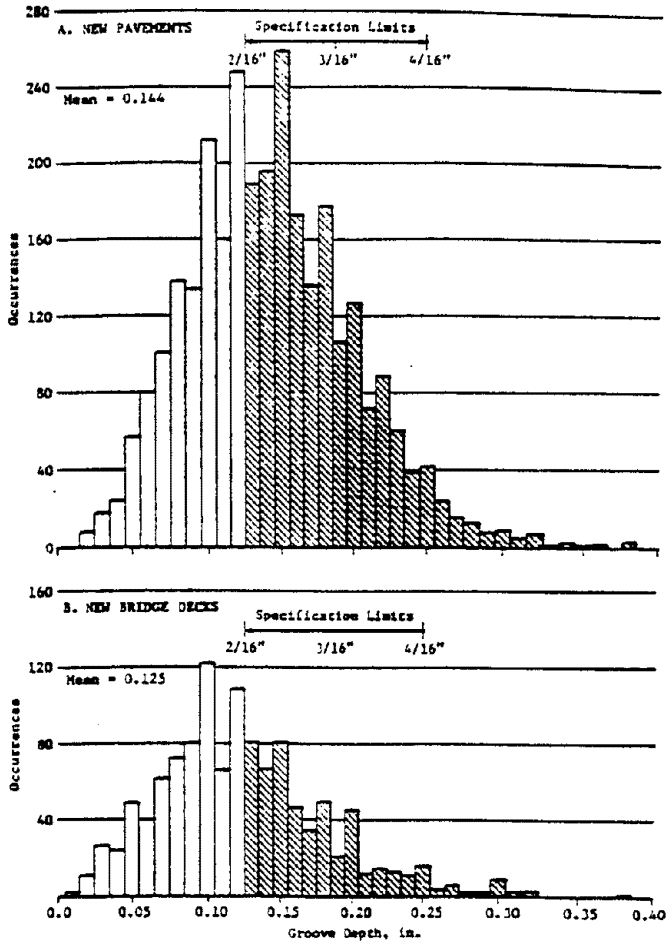
Note: X = mean, SD = standard deviation, NA = not available.



Table 43. Acceptable misalignment tolerances from different construction specifications.

| Data Source, (Reference) | Year | Relevant Placement Type | Vertical Skew, % | Horizontal Skew, % | Vertical Translation, % | Horizontal Translation, % |
|---|---|---|---|---|---|---|
| Alabama—Study #1 [45] | 1967 | Baskets or Inserter | 1.5 | 4.5 | NA | NA |
| Great Britain—Study #1 [45] | 1973 | Baskets or Inserter | 2.5 | 2.5 | NA | NA |
| Illinois—Study #2 [46] | 1979 | Baskets or Inserter | 1.1 (10.4 mm/m) | 1.1 (10.4 mm/m) | NA | NA |
| Georgia—Study #2 [46] | 1986 | Baskets or Inserter | 3.1 (31.3 mm/m) | 3.1 (31.3 mm/m) | NA | NA |
| Pennsylvania—Study #2 [46] | 1986 | Baskets or Inserter | 1.4 (13.9 mm/m) | 1.4 (13.9 mm/m) | 5.6 (± 25.4 mm) | 5.6 (± 25.4 mm) |
| Georgia—Study #4 [48] | 1983 | Inserter | 6.3 (62.5 mm/m) | 3.1 (31.3 mm/m) | 5.6 (± 25.4 mm) | 5.6 (± 25.4 mm) |

Note: NA = not available.




1 in = 25.4 mm

Figure 38. Distribution of groove-depth compliance measurements.[53]

Results of the study showed a mean of all groove-depth measurements to be 3.7 mm for pavements and 3.2 mm for bridge decks.[53] Another part of this study investigated the expected rate of wear of grooved pavements. The results showed that pavement grooves 4.8 mm wide on 19.1-mm centers were found to wear at a mean rate of 0.33 mm per million vehicle passes (measured with a dial depth gauge). Based on this rate, New York State DOT determined that 71 percent of their pavements would retain an acceptable groove depth for 15 years if a minimum tining depth of 4.8 mm was used. The percentage increased to 85 percent if the minimum tining depth was 6.4 mm. Both of these estimations were based on 1977 data.[53]

**Brief Summary of Potentially Useful Ongoing Research Projects**

PCC Texturing Methods[54]

Two probable future PRS quality characteristics include *friction* and *noise from tire-pavement interaction*. Both of these are greatly influenced by the finished pavement surface texture. The purpose of this study is to investigate the effects of surface texture on the long-term performance, skid resistance, and noise of rigid pavements. The constructability, costs, and performance of various texturing methods are being investigated. The following nine different texturing methods were included in the study:

- Transverse tining (25.4-mm spacing).
- Transverse astroturf drag—no tining.
- Longitudinal astroturf drag—random transverse tining.
- Longitudinal astroturf drag—transverse tining (12.7-mm spacing).
- Longitudinal astroturf drag—random transverse saw grooving.
- Longitudinal astroturf drag—transverse tining (25.4-mm spacing).
- Longitudinal astroturf drag—longitudinal saw grooving (19.1-mm spacing).
- Longitudinal astroturf drag—no tining.
- Longitudinal astroturf drag—longitudinal tining (19.1-mm spacing).

Friction was measured on each test section by measuring skid numbers according to ASTM skid testing procedure E 274. Tests were conducted using two different tire types (smooth and ribbed) at three different testing speeds (64, 80, and 105 km/h).

The amount of texture on each test section was measured with the following five different measuring devices:

1. *Texture Van (Laser Van)*—This consists of a television camera attached to a standard van. The camera takes "snapshots" of the pavement (in the wheelpath) approximately every 15.2 m as the van travels at 80 km/h. The output is reported as an average root mean square (RMS).

2. *Outflow Meter*—This method gives an indirect measure of texture. Water is placed in a cylinder with rubber seals on the bottom. The cylinder is placed on the pavement surface. When the test is started, water escapes from the cylinder between the rubber seal and the pavement surface. The time the water takes to escape is the measure of surface texture.

3. *Texture Beam*—This method uses two "texture sensors" to measure the surface texture. The first is a mechanical linear variable differential transformer (LVDT) that measures vertical motion. The second is a "commercial laser stylus with its power supply and signal processor." The output is reported as an RMS value (similar to the texture van).

4. *Sand Patch Method*—This method (using the ASTM procedure E 965) is a volumetric method. The test involves placing 24.6 $cm^3$ of fine sand or glass bead particles at a selected spot on the pavement. The particles are then carefully spread (in a circular motion) so that they are all below the visible peaks of the surface texture. The average diameter of the circular "sand patch" is then used to calculate the average texture depth.

5. *Tire Gauge*—This method involved using a standard tire tread depth gauge with an accuracy of 0.8 mm. The average of five texture depth measurements was taken as a representative measure of texture depth.

Noise was measured on each of the test sections using three different pieces of measuring equipment. Measurements were also taken at three different locations relative to the pavement or car (7.6 m from centerline of the pavement section, inside the vehicle at ear height at the center of the front seat, and near the right rear tire—away from the exhaust pipe).

Conclusions presented in the 1995 interim report include the following:

- The use of the smooth tire over the ribbed tire as a method of acquiring skid numbers is recommended. The smooth tire (ASTM E 524) showed more sensitivity to both microtexture and macrotexture than their corresponding ribbed tire (ASTM E 501).

- Texture depth taken with various texture measuring devices correlated favorably with the smooth-tire skid numbers taken at 64, 80, and 105 km/h, indicating a linear relationship with excellent correlation factors.

- The use of sand patch test method as a texture depth measuring device is highly recommended. Excellent correlations were achieved using the sand patch test method and smooth-tire skid numbers.

- Pavement section 3 (combination of longitudinal astroturf and random transverse tining) and pavement section 8 (longitudinal astroturf) showed the highest and lowest skid numbers, respectively, using both the ribbed and the smooth tire.

- The relationship between the skid number and the speed appeared to be approximately linear for the smooth tire, and not as linear for the ribbed tire.

- Longitudinal macro- and microtexture were the most quiet surfaces based on the sound pressure levels taken at the shoulder, inside the test vehicle, and at the rear tire.

- The combination of burlap drag and uniform 25.4-mm spacing exhibited the highest noise level among all of the test sections with the microphone at the rear-tire position.

Proposed 1994 PCC Surface Texture Field Test[55]

In 1994, a work plan was written for a project that would investigate the depth of transverse and longitudinal tining and evaluate possible non-tining alternatives to minimize noise and maximize skid resistance. The suggested field study proposed the following types of surface textures for investigation:

- Current State standards (if different from below).
- Longitudinal tining—19.1-mm spacing, 3.2-mm and 4.8-mm depth.
- Longitudinal tining—19.1-mm spacing, 3.2-mm and 1.6-mm depth.
- Transverse tining—variable spacing (19.1-mm average spacing), 3.2-mm and 4.8-mm depth, 15.2-cm repeating pattern.
- Transverse tining—variable spacing (19.1-mm average spacing), 1.6-mm depth, 15.2-cm repeating pattern.
- Transverse tining—12.7-mm spacing, 3.2-mm and 4.8-mm depth.
- Transverse tining—12.7-mm spacing, 1.6-mm depth.
- Longitudinal astroturf drag.
- Transverse astroturf drag.

The proposed work plan also included the following list of related studies that were known to be "in development or underway":

- Results of testing the Michigan I-75 exposed aggregate section and the standard tined surface.
- Results of testing the evaluation sections completed in Iowa in late 1993 and previously constructed sections.
- Results of March or April 1994 friction testing of two California experimental noise sites, including longitudinal and transverse texturing (including sand patch texture measurements, if possible).
- Synthesis of noise-related studies for ACPA/Portland Cement Association (PCA)—prepared by Construction Technology Laboratories.

- Colorado was considering the construction of at least one experimental section in 1994.

- Wisconsin was considering a research study on PCC surface texturing. It was to involve construction of some experimental sections and the inclusion of friction testing, and both interior and exterior vehicle noise.

- North Dakota completed some experimental sections in 1993 and planned additional noise study test sections in 1994—including skewed texturing to match the skewed joints.

- Minnesota planned to re-evaluate some of the experimental sections previously constructed.

- A couple of related foreign friction and/or noise studies were underway or recently completed.

- Proposed National Cooperative Highway Research Program (NCHRP) synthesis study.

## CONCLUSIONS

Based on the completed literature search, a laboratory/field work plan was developed to investigate the typical variation of AQC's and the possible development of new (or improvement of old) pavement distress indicator prediction models.

For the investigation of AQC typical variations, only those AQC's being investigated for model development (or incorporated in existing distress indicator models) were nominated for the laboratory/field study. This list was evaluated so that those AQC's for which variability was not well-documented in the literature could be targeted. Based on the literature search results, additional field/laboratory work was recommended to investigate the typical variations of the following AQC's:

- Concrete strength.
- Entrained air content.
- Slab thickness.
- Initial smoothness.
- Percent consolidation around dowels.
- Tie bar depth deviation (from mid-slab).
- Sawcut depth.

The research team first identified needed distress indicator models (refinement of old, or development of new) and their related AQC's. The AQC's identified as having a significant effect on pavement performance were then prioritized based on the practicality of developing a related distress indicator model, the feasibility of

developing such a model under the current research project, and the amount of information obtained from the literature search. Field/laboratory work and the use of existing data for distress indicator model development was recommended for the following AQC's:

- Concrete strength.
- Entrained air content.
- Initial smoothness.
- Consolidation level at doweled joints.

The data sources used for the AQC investigations are summarized in table 44.

Table 44. Summary of recommended data sources for investigation efforts.

| Acceptance Quality Characteristic | Investigation of Typical AQC Variability | | | | Distress Indicator Model Improvement & Development | | | |
|---|---|---|---|---|---|---|---|---|
| | Field Study | Laboratory Study | Existing Data | Results of Ongoing Research | Field Study | Laboratory Study | Existing Data | Results of Ongoing Research |
| Concrete Strength | √ | | | | | √ | | |
| Entrained Air Content | √ | | | | √ | | | |
| Slab Thickness | √ | | √ | | | | | |
| Initial Smoothness | | | √ | | | | | √ |
| Percent Consolidation Around Dowels | √ | | | | √ | | | |
| Tie Bar Depth Deviation (from mid-slab) | √ | | | | | | | |
| Sawcut Depth | √ | | √ | | | | | |

# APPENDIX D—TYPICAL ACCEPTANCE QUALITY CHARACTERISTIC VARIABILITY

## INTRODUCTION

Variation exists in all material- and construction-related acceptance quality characteristics (AQC's). The implementation of performance-related specifications (PRS) requires considerable knowledge about the variability of these AQC's. The goal of this specific investigation was to establish the "typical" variability associated with the AQC's included in the PRS for concrete pavements. The information obtained from this study will provide guidance to agencies when selecting PRS target as-designed variabilities.

A comprehensive literature search was conducted (described in appendix C) to identify previous studies where variations of concrete pavement AQC's were reported and documented in an unbiased manner. The information was evaluated so that those AQC's for which variability was not well-documented could be targeted for field testing and evaluation. In order to establish additional information on typical levels of variation, a field testing program was developed for the identified AQC's. Using the work plan developed for this investigation, data were collected from actual field construction projects. The specific AQC variabilities investigated during this research project include:

- Concrete strength—28-day flexural strength (predicted using maturity and measured directly).
- Slab thickness—measured from cores and ground-penetrating radar (GPR).
- Entrained air content—measured in the plastic concrete using an air pressure meter and in the hardened concrete slab using linear traverse tests of cores.
- Initial smoothness—measured as the initial profile index using a California profilograph.
- Transverse and longitudinal sawcut depth—measured with a ruler prior to any joint sealing.
- Consolidation level at doweled joints—measured using relative density tests of cores.
- Longitudinal tie bar depth—measured using GPR.

All testing was performed in accordance with standards developed by the American Society for Testing and Materials (ASTM). The information collected and analyzed

during this research will help identify allowable tolerances for each of the investigated AQC's. These tolerances are based on the observed variability limits contractors are currently able to achieve. The identified AQC tolerances may then be used by agencies to select appropriate AQC target standard deviations.

A detailed discussion of the field/laboratory investigations (including projects studied, sampling and testing procedures, data analysis procedures, and a summary of results for each of the targeted AQC's) is provided in the following sections.

## DISCUSSION OF VARIANCE

Because the PRS approach determines the performance of a pavement on a lot basis, the most important variability of interest for the procedure is the overall within-lot variability. The procedure assumes that the pavement lot quality is represented by AQC's that are normally distributed. Each AQC is measured by testing randomly selected samples located within the pavement lot. Each sample is the mean of all replicate test values obtained at the randomly selected sample location. All samples representing a given lot are assumed to estimate a distribution of the AQC. For each AQC, the value targeted during construction of the lot (specified by the governing agency) is the mean of the distribution. The tolerance value for each AQC (minimum acceptable value without penalty) allowed by the governing agency was determined during this investigation as the currently achievable within-lot variability. For each AQC studied in this investigation, the total within-lot variability was calculated as the standard deviation of all samples collected within the lot. If replicate values were used to represent the samples, the total within-lot variability was calculated using equation 10.

$$\sigma_{TOTAL\ WITHIN\text{-}LOT} = \sigma_{MEASURED\ WITHIN\text{-}LOT} * n^{0.5} \quad (10)$$

where

$\sigma_{TOTAL\ WITHIN\text{-}LOT}$ = The total within-lot standard deviation for the AQC (materials/process and testing).

$\sigma_{MEASURED\ WITHIN\text{-}LOT}$ = The standard deviation between all representative samples in the lot. Each sample is the mean of all replicate test values obtained at the sample location.

n = Number of replicate test values obtained at each AQC sample location.

### Components of Variance

For each AQC required by the governing agency, the within-lot variability is an estimate of the total variability of the AQC over the entire lot. This total within-lot

variability is comprised of testing variation and materials/process variation. The testing variation, or within-test variability, is a measure of the repeatability of a particular testing method. This variability is calculated as the standard deviation of the test values representing a sample for a random sample location. As long as the standard deviation is independent of the mean, it can be used to represent the variability of the AQC being evaluated. The materials/process variability represents the variation due to actual AQC point-to-point material differences and process control. It is the difference between the total within-lot variability and the within-test variability.

The relationship of the components of variance is shown in equation 11.

$$\sigma^2_{\text{TOTAL WITHIN-LOT}} = \sigma^2_{\text{WITHIN-TEST}} + \sigma^2_{\text{MATERIALS/PROCESS}} \quad (11)$$

where

$\sigma^2_{\text{TOTAL WITHIN-LOT}}$ = The total within-lot variance for the AQC.

$\sigma^2_{\text{WITHIN-TEST}}$ = The estimated variance of the testing method (testing repeatability).

$\sigma^2_{\text{MATERIALS/PROCESS}}$ = The estimated variance of the materials and process (material variance).

**Calculating Equivalent Variability**

When implementing a PRS, it is important that the selected AQC target variabilities correspond with the sampling and testing methods being used during construction. Each of the AQC variabilities recommended in this report represents the variability associated with evaluating individual samples (comprised of one actual test value) within a given lot. If an agency elects to collect replicate test values to represent a sample (for example, three cast cylinders from the same sample location tested for 3-day compressive strength), the variability must be adjusted using equation 12.

$$\sigma_{\text{TARGET}} = \sigma_{\text{RECOMMENDED}} / n^{0.5} \quad (12)$$

where

$\sigma_{\text{TARGET}}$ = The targeted within-lot standard deviation for the AQC.

$\sigma_{\text{RECOMMENDED}}$ = The recommended within-lot standard deviation for the AQC (materials/process and testing).

$n$ = Number of replicate test values obtained at each AQC sample location.

## IDENTIFICATION OF FIELD PROJECTS

To evaluate the variability of construction parameters achievable by the contractor, testing was performed on field construction projects during the 1995 and 1996 construction seasons. The project team established criteria for the field sites as follows:

- Jointed plain air-entrained concrete pavement.
- Located on a highway in a rural or suburban area.
- Construction duration of at least 3 days.
- Located within a 1-day drive of Skokie, IL (because test samples and materials had to be transported to the laboratory facilities).
- Willingness of the State highway agency (SHA) and contractor to allow the project to be included in the study.

The established criteria for the field studies limited the search to northern midwestern States. SHA's were contacted and a testing schedule was planned. As projects were let, close contact with field engineers was required to schedule beneficial field visits to the projects. When possible, the field evaluations were conducted once the contractor had established a smooth, trouble-free paving operation. It was desirable to work with several SHA's to collect data for this investigation. The construction projects selected for evaluation in this study were performed in Illinois, Iowa, Michigan, Minnesota, Nebraska, Ohio, and Wisconsin. A list of the specific projects, along with the construction parameters evaluated, is shown in table 45.

## CONCRETE STRENGTH

The typical field concrete strength variability was evaluated for use in both Level 1 and Level 2 PRS programs. Both PRS levels require strength to be inputted into the distress indicator models as a 28-day flexural strength (third-point loading). For a Level 1 PRS, this required strength value is typically determined directly using 28-day flexural strength data from beams cast in the field and cured under standard laboratory conditions. The Level 2 PRS, however, is being investigated to evaluate the possibility of providing the contractor with a quicker indication of the 28-day flexural strength results so that he may make material or method adjustments sooner. Therefore, methods used to estimate the 28-day flexural strength based on early age flexural, compressive, or split-tensile strength tests were also investigated.

### Historical Concrete Strength Data

As discussed previously, the variance in any AQC is comprised of two sources. The first source of variance is "within-test" variance, commonly expressed as the standard

Table 45. Data collected at each project evaluated for acceptance quality characteristic variation.

| Project | Project Type | Acceptance Quality Characteristic | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 3-day Cyl. Compressive | 3-day Core Compressive | 14-day Cyl. Compressive | 28-day Cyl. Compressive | 28-day Core Compressive | 3-day Cyl. Splitting Tensile | 3-day Beam Flexural | 14-day Beam Flexural | 28-day Beam Flexural | Predicted 28-day Flexural | Slab Thickness by Coring | Slab Thickness by GPR | Plastic—Before Paver | Plastic—After Paver | Hardened Linear Traverse | Joint Sawcut Depth | Consolidation Around Dowels | Longitudinal Tie Bar Depth | Initial Smoothness |
| Rochelle, IL—IL Route 38 | New Construction | | | | √ | √ | | | | | | √ | √ | √ | √ | √ | √ | | | |
| Shawano, WI—Route 29 East | | √ | | | | | √ | | | √ | √ | √ | | √ | √ | √ | √ | | | |
| St. Johns, MI—Route 27 North | | √ | | | | | √ | | | √ | √ | √ | | √ | √ | √ | √ | | | |
| Ottumwa, IA—Route 23 South | | √ | √ | √ | √ | √ | | √ | √ | √ | √ | √ | | √ | √ | √ | √ | √ | √ | √ |
| Omaha, NE—I-80 East & West | | | | | | | | | | | | | | | | | | | √ | |
| Ontario, Canada—Hwy 115 | | | | | | | | | | | | √ | | | | | √ | | | |
| Mankato, MN—U.S. 169 | | | | | √ | √ | | | | | | | √ | | | √ | | | | |
| Bellefontaine, OH—U.S. 33 | | | | | √ | √ | | | | | | | √ | | | √ | | | | √ |
| Des Plaines, IL—Route 58 West | Doweled Joint Patching | | | | | | | | | | | √ | | | | √ | | √ | | |
| Benton Harbor, MI—I-94 West | | | | | | | | | | | | √ | | | | √ | | √ | | |
| Philo, IL—Route 130 South | | | | | | | | | | | | √ | | | | √ | | √ | | |

deviation between test values obtained from the same sample location. The within-test standard deviation indicates the amount of variation among strength tests from the same batch of concrete and is an indicator of the quality of specimen fabrication, handling, curing, and testing procedures (overall testing repeatability). Flexural strength tests for almost 200 laboratory mixes conducted during the expansion of the Atlanta International Airport in the 1970's, and numerous mixes reported in other research studies, indicated average within-test standard deviations of 0.23 MPa and 0.27 MPa, respectively.[18] The within-test standard deviations for the laboratory data were independent of flexural strength. The analysis of field data from the Atlanta International Airport projects indicated average within-test standard deviations ranging from 0.21 to 0.28 MPa with an overall weighted average of 0.24 MPa. The overall average is very close to the 0.23 MPa determined from laboratory mixes.[18] Average within-test standard deviation for the field data was only slightly dependent on flexural strength.

The second source of variance is the "materials/process" variance. This variance cannot be physically measured without incorporating "within-test" variance. Therefore, the total variance, commonly expressed as the "between-test" standard deviation, is measured. The between-test standard deviation indicates the variance between different batches of the same concrete materials (attributed to material and production quality control), as well as the testing repeatability described above. The average between-test standard deviation for laboratory mixes reported for the Atlanta International Airport, and numerous mixes reported in other research studies, was 0.19 MPa.[18] As with the within-test standard deviation, the between-test standard deviation for the laboratory data was generally independent of flexural strength. The analysis of field data from the Atlanta International Airport projects indicated average 7-day between-test standard deviations ranging from 0.17 to 0.33 MPa, with an overall weighted average of 0.29 MPa. The standard deviations at 28 days ranged from 0.18 to 0.36 MPa, with an overall weighted average of 0.24 MPa. The overall average is very close to those determined from laboratory mixes.[18] The average between-test standard deviation for the field data was only slightly dependent on flexural strength.

Theoretically, the between-test (or total within-lot) variance should be greater than the within-test variance as shown by equation 11. However, often the computed between-test standard deviation is less than the calculated within-test standard deviation. For the Atlanta International Airport study of concrete flexural strength data, Greer attributed this observation to the use of averages for the between-test calculations and suggested that it indicated that a significant portion of the variation in flexural strength tests is the result of inherent variations in the test procedure.[18]

Within- and between-test standard deviations were examined in the laboratory in a previous PRS study.[1] The flexural strength within-test standard deviation ranged from 0.02 to 0.31 MPa and averaged 0.16 MPa at 7 days for 18 different mixes. At 14 days, the within-test standard deviation ranged from 0.06 to 0.35 MPa and averaged 0.18 MPa.

At 28 days, the within-test standard deviation ranged from 0.03 to 0.57 MPa and averaged 0.23 MPa at 28 days. On average, the standard deviation slightly increased from 7 to 28 days. Between-test standard deviations for nine replicated mixes were 0.15, 0.18, and 0.25 MPa at 7, 14, and 28 days, respectively. Similar to other studies reported, the between-test and within-test standard deviations are nearly identical. As with the within-test standard deviation, the between-test standard deviation slightly increased with age. This conclusion differs from other studies that have concluded that standard deviations of flexural strength are generally independent of age.

The standard deviation of interest in the PRS is the between-test standard deviation because all samples within each lot are assumed to come from the same population. The literature study indicates that the average between-test standard deviation determined from different laboratory studies ranges from 0.15 to 0.26 MPa and is not significantly influenced by strength magnitude (age). Based on the Atlanta International Airport field study, the average between-test standard deviation ranged from 0.24 MPa at 28 days to 0.29 MPa at 7 days.[18] As reported in the previous PRS research, a recommended reasonable between-test standard deviation for use in the PRS (when flexural strength is directly measured) is 0.24 to 0.28 MPa.

**Field Investigation of the Typical Concrete Strength Within-Lot Standard Deviation**

To verify the recommended variability of 28-day concrete flexural strength for a Level 1 PRS, 152- x 152- x 533-mm beams were molded in the field and transported to the laboratory curing room for a standard 28-day lab moist cure (temperature of 23 °C and 100-percent humidity). The 28-day flexural strength results were directly evaluated for the total within-lot standard deviations after verifying that the standard deviation was independent of the mean. For all the lots evaluated, the total within-lot standard deviations were summarized and the average value was reported as the recommended target standard deviation to be used for a Level 1 PRS.

For a Level 2 PRS, the research team investigated the ability to predict the 28-day flexural strength from earlier age strength data using maturity concepts. With the additional error involved with using early age flexural, compressive, or splitting tensile strength data and indirectly projecting a 28-day flexural strength, the possibility for greater variability was investigated. Early age field-cast beam, core, and cylinder specimens were used with laboratory-developed interstrength relationships to predict the 28-day flexural strength. The predictions were compared with actual field-cast beams tested at 28 days to determine the accuracy of the prediction models. The variability of the predicted flexural strength data was then calculated as described above for the most accurate prediction model.

Field Sampling and Testing Procedures

For all the projects shown in table 45, fresh concrete was sampled at the paver in accordance with ASTM D 172, *Standard Practice for Sampling Freshly Mixed Concrete*. In

general, concrete was sampled from the discharged material in front of the paver and transported by wheelbarrow to the casting location. Specimens were cast in accordance with ASTM C 31, *Standard Practice for Making and Curing Concrete Test Specimens in the Field.* After casting, the samples were kept shaded in a low-traffic area and cured using wet burlap. The burlap was kept moist by occasional wetting until the specimens could be transported and placed in a standard laboratory curing room until testing. During transport to the laboratory facilities, the samples were packed in wet sand or water tanks. In addition to freshly cast specimens, cores were removed from each project in accordance with ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.* After removal, the cores were treated in the same manner as the freshly cast specimens until testing. All strength testing was performed in the laboratory in accordance with the following ASTM specifications:

- Compressive Strength—ASTM C 39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.*
- Splitting Tensile Strength—ASTM C 496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.*
- Flexural Strength—ASTM C 78, *Modulus of Rupture of Cast Beams Using Third-Point Loading.*

Predicting 28-day Flexural Strengths Based on Early Age Concrete Testing

In order to predict the 28-day flexural strength from early age strength data, the concept of concrete maturity was used in conjunction with laboratory strength development and interstrength relationship curves. Throughout the sampling and testing at each site, concrete temperatures were monitored using thermocouples embedded at the approximate center of a cylinder and a beam from each sampling location. A combination of temperature recorders and maturity meters were used to monitor the maturity of the concrete specimens in accordance with ASTM C 1074, *Standard Practice for Estimating Concrete Strength by the Maturity Method.* The method describes two alternative functions for computing the maturity of the concrete from a measured temperature history. The traditional Nurse-Saul function is used to calculate a temperature-time factor, such as °C-Hrs, to represent the maturity of concrete and the Arrhenius function computes an Equivalent Age (Eq. Hr. at 20 °C) for the monitored concrete.

Previous PRS research showed that when the maturity is calculated within laboratory-maintained temperature conditions, both methods result in equal accuracy.[3] However, the Arrhenius equation can better represent the effects of temperature on strength development when wide temperature ranges are expected.[56] Since relatively large temperature fluctuations were recorded for the projects included in this study, the Arrhenius maturity method was chosen.

Maturity, rather than age, is used to estimate the in-place strength because of the effect of curing temperature on the strength development of concrete. For example, the 3-day strength of field-cured concrete may not equal the 3-day strength of laboratory-cured concrete from the same mix. Using maturity instead of age considers the effect of different curing temperatures, which results in more accurate estimations of in-place strength. To minimize errors in calculating the maturity of the field specimens, during transport to the laboratory facilities, the samples were packed in wet sand or water tanks and the temperatures were continuously recorded. Once in the lab, prior to testing, the samples were kept in a moist-curing room where the temperature and humidity are maintained at 23 °C and 100 percent, respectively.

To predict the 28-day flexural strength at a significantly earlier age, the method of developing mix-specific strength interrelationships outlined in previous PRS research should be used.[1] Strength development and interstrength relationship curves (specific to the chosen concrete mix) are developed in the laboratory prior to construction. This can easily be incorporated in the initial stages of a project by the testing laboratory contracted to develop the concrete mixture design. After obtaining adequate amounts of the coarse and fine aggregates, cement, flyash, and admixtures to be used for the production of the field trial concrete pavement, concrete beam and cylinder specimens are cast in the laboratory in accordance with ASTM C 192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*. The maturity of the samples is measured over the curing period and samples are tested for flexural strength (third-point loading) and compressive strength or splitting tensile strength (depending on the chosen strength testing type) at incremental ages of strength development. The number of testing ages should be sufficient to enable the user to predict strength from maturity. As a minimum, it is recommended that tests be conducted at strength development ages of 1, 3, 5, 7, 14, and 28 days. If possible, it is also recommended that testing ages be extended past 28 days to ensure that the field maturity at the time of coring falls within the range from which the maturity prediction equation was developed.

Equations representing the strength development (maturity) and interstrength relationship curves are derived from the average strength test data. Simple regression equations are recommended for predicting strength as a function of Arrhenius maturity. For the regression analysis, dependent and independent variables are used directly and transformed using square root, logarithmic, and inverse transformations. In general, the equation with the highest $R^2$ represents the best interstrength model; however, the derived curve should be plotted against the average strengths to evaluate the accuracy in predicting the desired 28-day strength. Since predicting early age strength values is of less interest, sometimes a curve with a slightly lower $R^2$ provides greater accuracy in predicting the 28-day strength.

After the equations are derived, they can be used to predict 28-day flexural strengths based on early age field strength data. An example of this procedure is illustrated graphically in this section. Figures 39 and 40 show hypothetical laboratory strength

development and interstrength curves (where compressive strength is the chosen strength type). Early age field test data can be plotted on the laboratory strength curve according to the measured field maturity, as shown in figure 41. Invariably, there will be a strength difference between the plotted field data point and the laboratory-developed strength curve. This difference is used to adjust the laboratory 28-day compressive strength from the derived curve equation to a predicted field 28-day compressive strength, as shown in figure 42. Once the 28-day field compressive strength has been extrapolated, the interstrength relationship between compressive and flexural strength can be used to predict the field flexural strength. This step is shown graphically in figure 43.

Concrete Strength Field Data Collection

For two of the new construction projects (Shawano, WI, and St. Johns, MI), the total within-lot variability was evaluated by sampling four cylinders each from five different trucks. Two of the four cylinders from each truck were used to establish 3-day compressive strength, and the remaining two were used to establish 3-day splitting tensile strength (in accordance with ASTM standards). These early age cylinder strengths were used in conjunction with the laboratory-developed strength curves to predict the 28-day flexural strength of the sublots. Two beams were also cast from each of the five trucks. The beams were tested for 28-day flexural strength. These actual 28-day flexural strengths served two purposes in the study. First, the data were used to evaluate the typical variability in 28-day flexural strength currently achievable in the field for use in a Level 1 PRS. Second, the data were used to evaluate the accuracy of the predictions obtained from the early age cylinder strength data. The results are presented in the following section.

The Ottumwa, IA, project (also used as the original shadow field trial under this research project) was divided into three lots. Lot 1 was subdivided into three sublots, and Lots 2 and 3 were each divided into four sublots. From each sublot, the material from two trucks was sampled at two different longitudinal sampling locations. For the purpose of evaluating the total within-lot concrete strength standard deviation, the average strengths at each longitudinal sampling location in a lot (regardless of sublot divisions) were used to calculate the total within-lot standard deviation. Therefore, for this investigation, Lot 1 was comprised of six representative samples and Lots 2 and 3 each had eight representative samples. At each sampling location, six cylinders and six beams were cast. As in Shawano and St. Johns, two cylinders from each truck were used to establish the 3-day compressive strength for that sample. However, additional cylinders were cast to determine the 14- and 28-day compressive strength (two cylinders each) variation. Also, while a 3-day early prediction model is preferred, a 14-day prediction model was investigated to quantify the increase in accuracy in predicting the 28-day flexural strength. The six beams per truck were used to evaluate the variability in flexural strength at 3, 14, and 28 days, as well as to provide a comparison for determining the accuracy of the 28-day prediction models. In addition, two cores were removed from each sublot (one from each longitudinal sampling

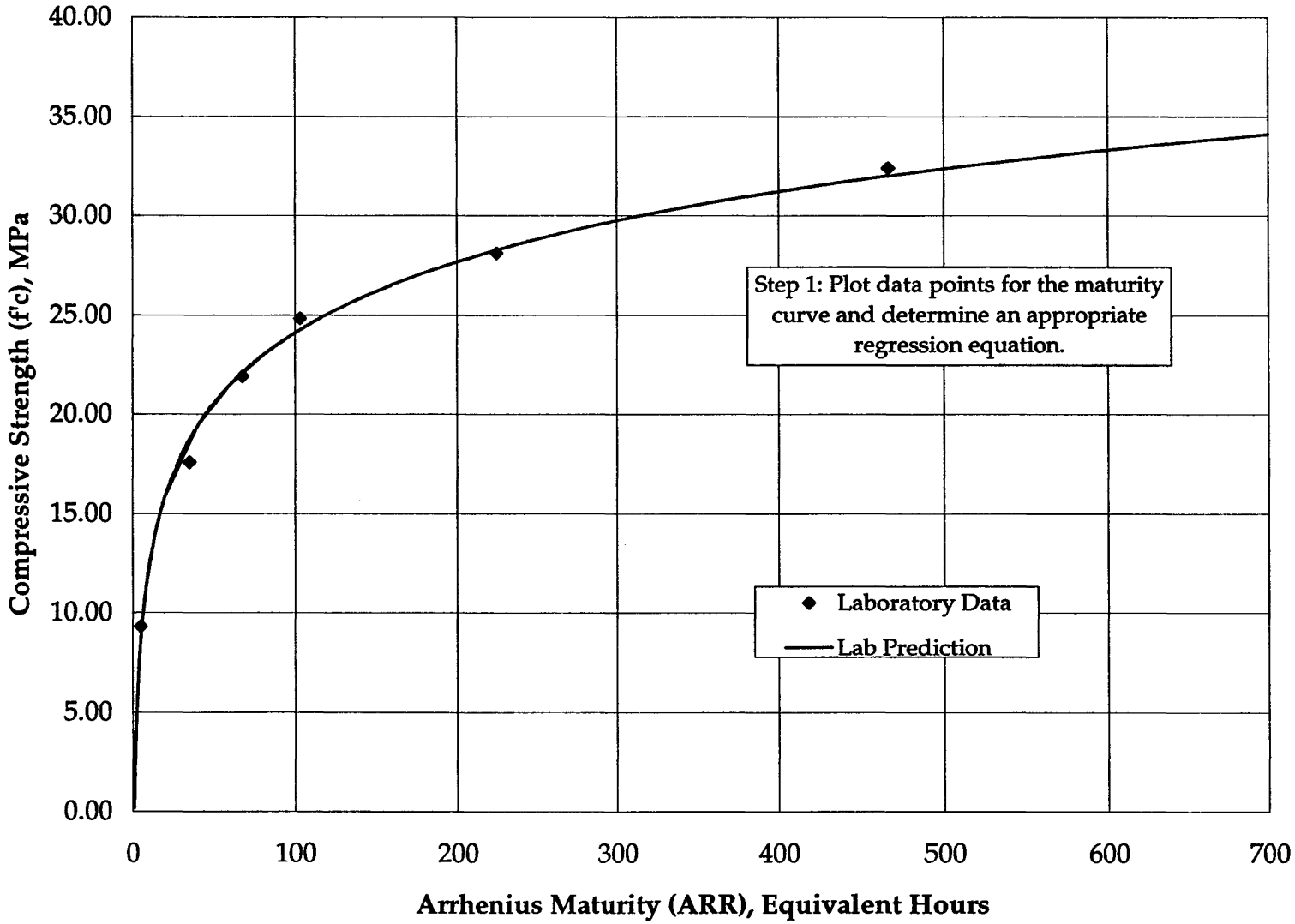



Figure 39. Example of a laboratory compressive strength development versus maturity relationship.

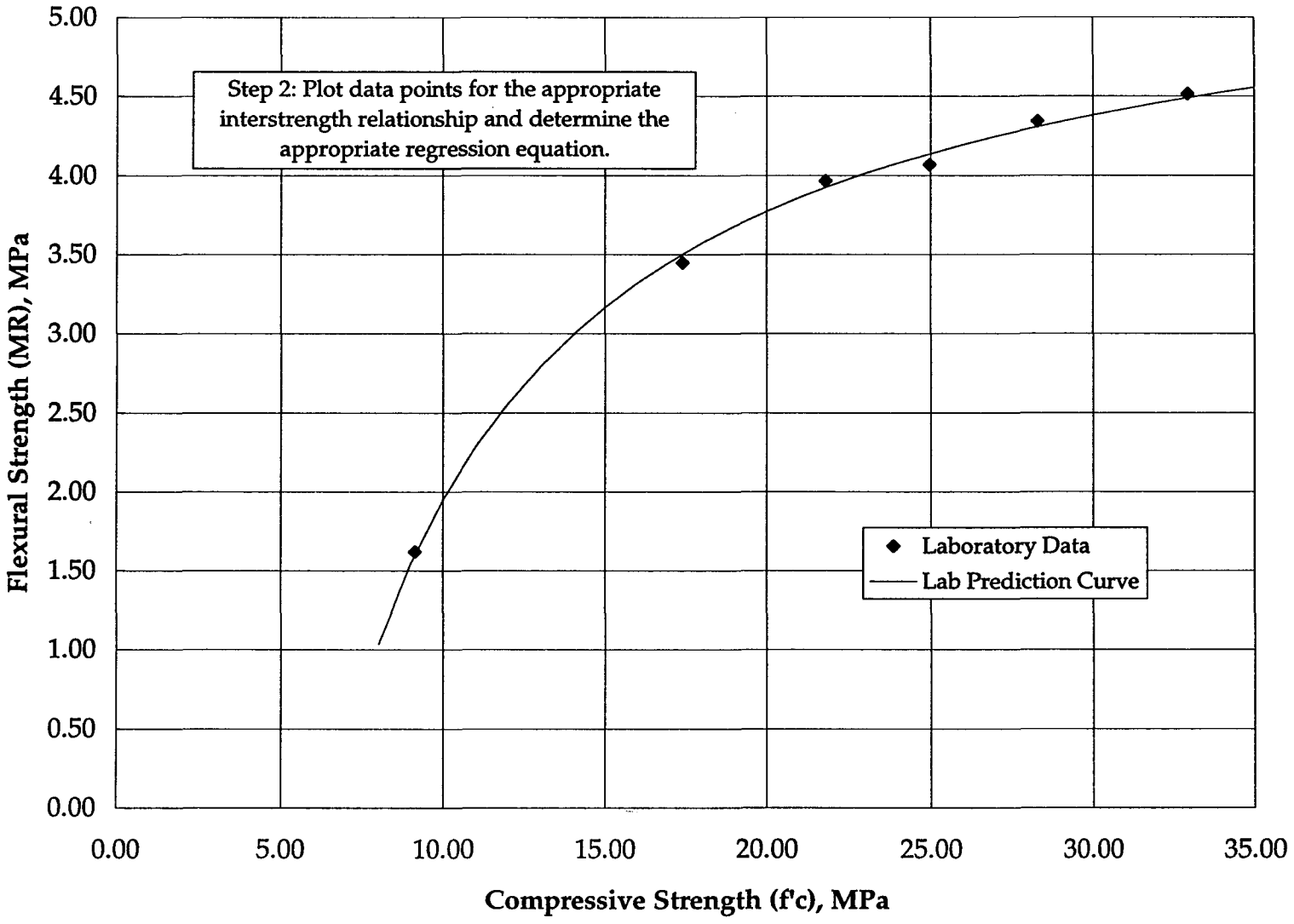



Figure 40. Example of a laboratory-developed (compressive to flexural strength) interstrength relationship.

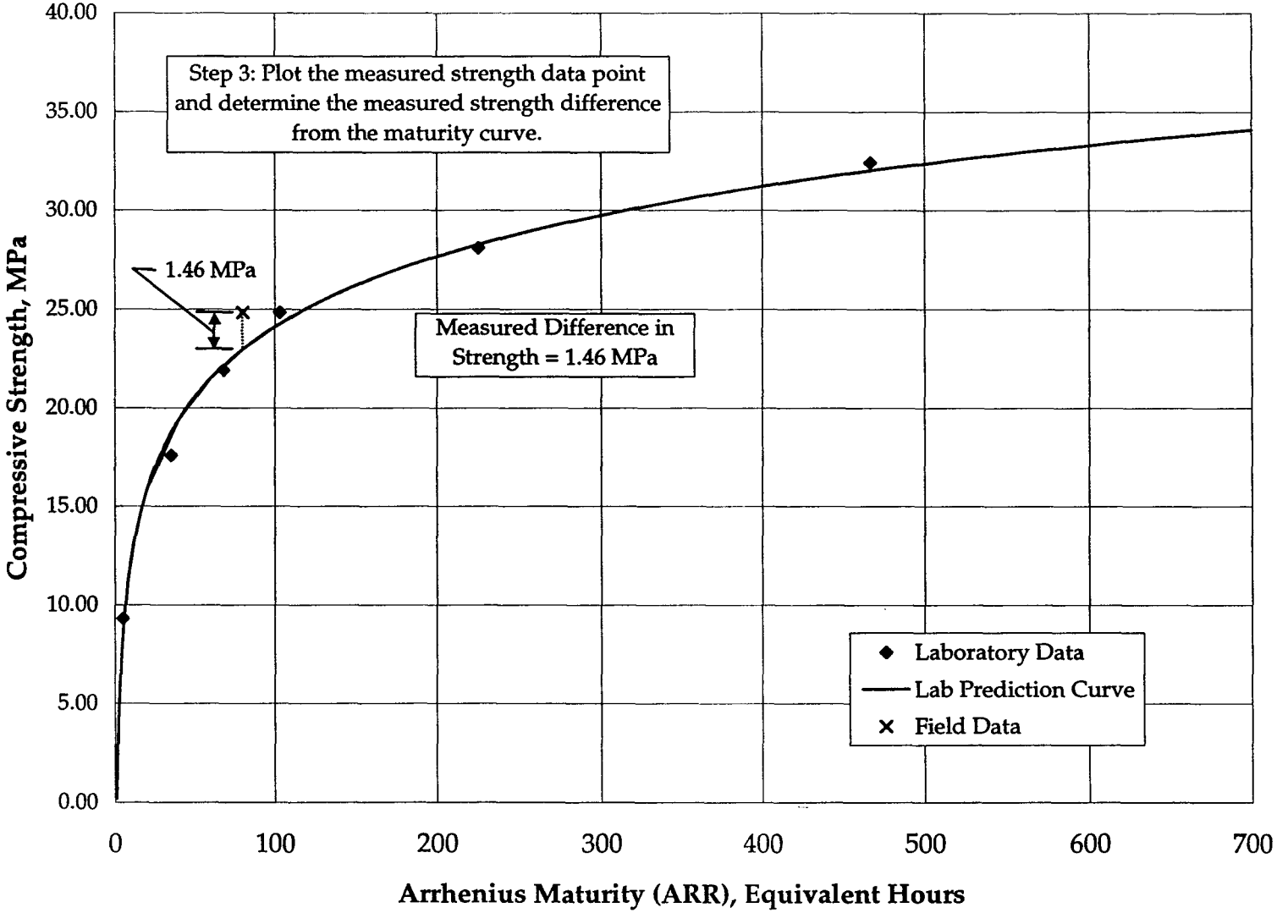



Figure 41. Example of a measured early age field compressive strength plotted on the laboratory-developed strength versus maturity curve.

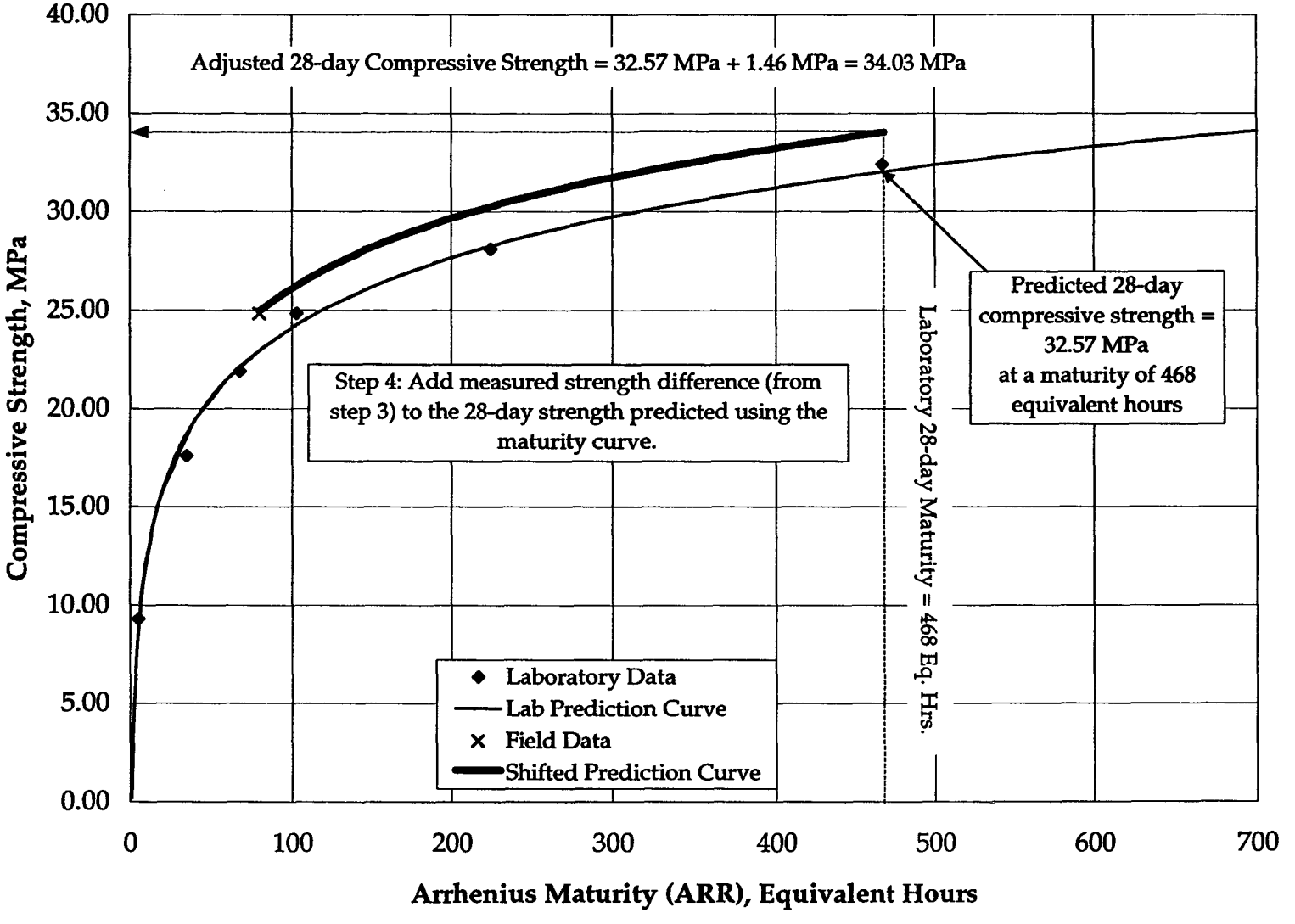



Figure 42. Example of using the laboratory-developed strength versus maturity relationship to extrapolate a 28-day compressive strength from a measured early age compressive strength.

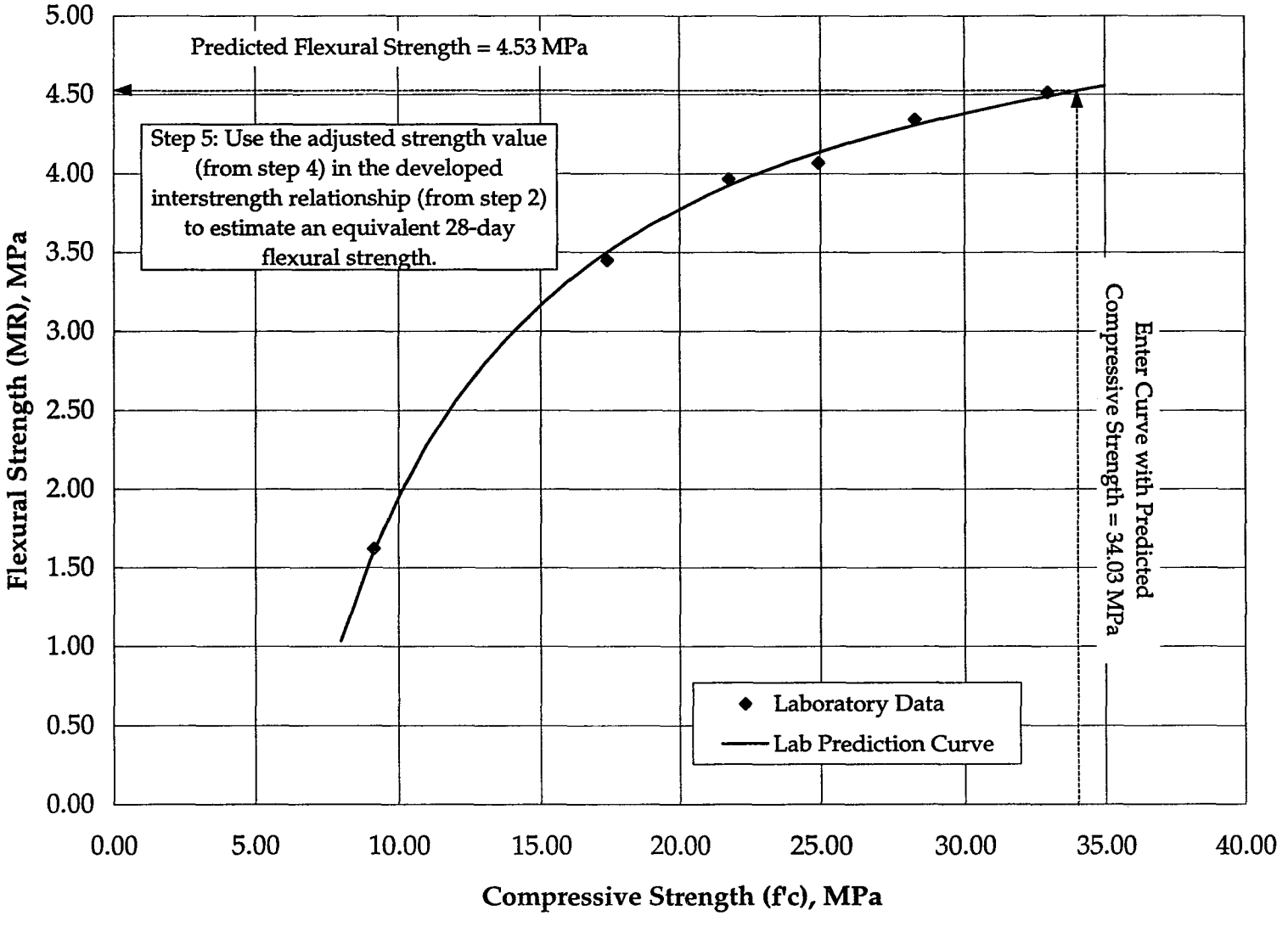



Figure 43. Example of using the laboratory-developed (compressive to flexural strength) interstrength relationship to predict 28-day flexural strengths.

location) to evaluate the feasibility of predicting the 28-day flexural strength from 3-day core compressive strength. Maturity meters were left in the field to monitor thermocouple trees embedded in the pavement. Thermocouples monitored the temperature at the middle of the pavement slab. Two additional cores were removed from each sublot (one from each longitudinal sampling location) to measure the 28-day in situ core compressive strength of the pavement. The within-lot variability of the core compressive strength data was evaluated.

As shown in table 45, additional 28-day strength data were collected from other projects evaluated during this investigation. Both 28-day cylinder and core compressive strength data were obtained from the Rochelle, IL; Mankato, MN; and Bellefontaine, OH, projects. The data were included in the variability study as a point of interest because they were readily available; however, since flexural strengths were not measured at these projects, the collected strength data were not included in the study used to determine typical flexural strength standard deviations.

## Summary of Concrete Strength Variation Results

### *Collection of Strength Variability Data*

Under the current PRS approach, the agency may base performance on any combination of the available AQC's (concrete strength, slab thickness, entrained air content, initial smoothness, and percent consolidation around dowels). The PRS computer software, PaveSpec 2.0, is used to simulate the expected performance and associated life-cycle costs (LCC's) for the pavement. Therefore, the PRS approach requires accurate, achievable target standard deviations for each of the chosen AQC's. Current distress indicator prediction models (used to evaluate the fatigue performance of the concrete pavement) require knowledge of the 28-day flexural strength. The standard deviation of this flexural strength is, therefore, important when calculating the LCC of a particular pavement lot.

The most direct method for measuring the 28-day concrete flexural strength is from beam specimens cast in the field. The specimens are lab-cured for a 28-day period and tested in accordance with ASTM C 78 to obtain the 28-day third-point modulus of rupture. Thus, the as-constructed mean and standard deviation of the 28-day flexural strength can be obtained directly. The field sampling and test methods are already being performed by SHA's to evaluate newly constructed pavements. Therefore, one objective of the current research project was to determine a recommended as-designed target standard deviation for 28-day beam flexural strength testing. For each of the construction project lots investigated, 28-day flexural strengths were directly measured and evaluated to determine the representative total within-lot standard deviation (as described in the section *Discussion of Variance*).

The variability of flexural strength was also evaluated at 3 and 14 days. In addition to the investigation of flexural strength, the variability of 3-, 14-, and 28-day cylinder

and core compressive strengths was evaluated. The variability of 3-day splitting tensile strength was determined based on data collected from two projects (Shawano, WI, and St. Johns, MI).

For each strength type (compressive, splitting tensile, and flexural), the data were summarized into the age/sample categories (e.g., 3-day core, 14-day beam) shown in table 46. For each category, lot statistics, including the total within-lot standard deviation, were summarized. The overall goal of the investigation was to recommend a target as-designed variability for input into the PaveSpec 2.0 program. Therefore, for each strength category, the standard deviations computed for each investigated lot were assumed to be normally distributed. The mean standard deviation was then determined to be the recommended target standard deviation. The results are shown in table 47.

*Determining 28-day Flexural Strengths From Early Age Strength Data*

Currently, there is a move toward computing concrete strength pay factors based on 28-day flexural strengths predicted from early age concrete strength data. Eventually, it is envisioned that the predictions will be made from in situ (and even nondestructive) testing procedures. However, current research efforts focus on basing the predictions on early age strength data obtained using conventional sampling and testing methods.

Curves representing the laboratory strength development and interstrength relationships for each of the three construction projects evaluated (Shawano, WI; St. Johns, MI; and Ottumwa, IA) are shown in figures 44 through 57. Also shown in the figures are the field sample test results for each project. The interstrength relationship curves were developed by plotting different strength measurements (flexural versus compressive or flexural versus splitting tensile) over equivalent maturity intervals. For the Shawano and St. Johns projects, interstrength relationships were plotted over equivalent time intervals. Based on the results of these two projects and because the Ottumwa project was also being investigated as the original field trial, it was necessary to omit the evaluation of the accuracy of the splitting tensile strength prediction for this project. Linear regression equations for the best-fitting curves through the data points were also developed, as shown in table 48.

Once the strength and maturity standards were developed in the laboratory, they were used in the field to predict the 28-day flexural strength for input into the PaveSpec 2.0 program. For the Shawano and St. Johns projects, the flexural strength was predicted from 3-day compressive and 3-day splitting tensile strength test data.

Figures 58 and 59 show the predicted versus measured 28-day flexural strength data for these respective prediction models. In the compressive strength model, the measured 3-day cylinder strengths were first extrapolated to 28-day cylinder strengths using maturity concepts, and then converted to 28-day flexural strengths using the laboratory interstrength relationships provided in figures 44 through 53. For the

Table 46. Summary of the measured total within-lot concrete strength variations.

| Strength Type | Age/ Specimen | Project Lot | No. of Samples per Lot | Replicates per Sample | Lot Average, MPa | Within-Lot Std. Dev., MPa |
|---|---|---|---|---|---|---|
| Compressive | 3-day Cylinder | Shawano, WI | 5 | 2 | 23.10 | 2.07 |
| | | St. Johns, MI | 5 | 2 | 25.96 | 2.25 |
| | | Ottumwa, IA Lot 1 | 6 | 2 | 27.81 | 3.25 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 29.81 | 1.71 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 28.90 | 1.78 |
| | 3-day Core | Ottumwa, IA Lot 1 | 6 | 1 | 30.61 | 1.85 |
| | | Ottumwa, IA Lot 2 | 8 | 1 | 30.49 | 2.00 |
| | | Ottumwa, IA Lot 3 | 8 | 1 | 27.47 | 2.44 |
| | 14-day Cylinder | Ottumwa, IA Lot 1 | 6 | 2 | 38.87 | 2.88 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 36.59 | 2.58 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 36.01 | 1.69 |
| | 28-day Cylinder | Rochelle, IL Lot 1 | 17 | 4 | 33.81 | 7.29 |
| | | Rochelle, IL Lot 2 | 14 | 3 | 41.55 | 4.76 |
| | | Mankato, MN Lot 1 | 15 | 2 | 38.60 | 4.19 |
| | | Mankato, MN Lot 2 | 13 | 2 | 34.28 | 2.63 |
| | | Bellefontaine, OH Lot 1 | 15 | 3 | 29.02 | 3.79 |
| | | Bellefontaine, OH Lot 2 | 15 | 3 | 29.95 | 4.72 |
| | | Ottumwa, IA Lot 1 | 6 | 2 | 42.86 | 1.88 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 41.95 | 2.64 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 39.15 | 2.05 |
| | 28-day Core | Rochelle, IL Lot 1 | 13 | 1 | 36.29 | 7.11 |
| | | Rochelle, IL Lot 2 | 13 | 1 | 46.18 | 3.14 |
| | | Mankato, MN Lot 1 | 13 | 1 | 43.25 | 4.59 |
| | | Mankato, MN Lot 2 | 14 | 1 | 39.48 | 5.49 |
| | | Bellefontaine, OH Lot 1 | 14 | 1 | 40.34 | 3.51 |
| | | Bellefontaine, OH Lot 2 | 14 | 1 | 39.86 | 4.10 |
| | | Ottumwa, IA Lot 1 | 6 | 1 | 41.11 | 2.50 |
| | | Ottumwa, IA Lot 2 | 8 | 1 | 41.56 | 2.86 |
| | | Ottumwa, IA Lot 3 | 8 | 1 | 37.87 | 2.52 |
| Split Tensile | 3-day Cylinder | Shawano, WI | 5 | 2 | 2.40 | 0.39 |
| | | St. Johns, MI | 5 | 2 | 2.59 | 0.39 |
| Flexural | 3-day Beam | Ottumwa, IA Lot 1 | 6 | 2 | 3.88 | 0.43 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 4.14 | 0.17 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 4.17 | 0.27 |
| | 14-day Beam | Ottumwa, IA Lot 1 | 6 | 2 | 4.68 | 0.43 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 4.57 | 0.30 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 4.55 | 0.18 |
| | 28-day Beam | Shawano, WI | 5 | 2 | 5.58 | 0.29 |
| | | St. Johns, MI | 5 | 2 | 5.31 | 0.19 |
| | | Ottumwa, IA Lot 1 | 6 | 2 | 4.75 | 0.24 |
| | | Ottumwa, IA Lot 2 | 8 | 2 | 4.86 | 0.15 |
| | | Ottumwa, IA Lot 3 | 8 | 2 | 4.66 | 0.20 |

Table 47. Recommended within-lot concrete strength target standard deviations for different specimens at different ages.

| Strength Type | Age/Specimen | Number of Lots | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, MPa | | | | |
|---|---|---|---|---|---|---|---|
| | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Compressive | 3-day Cylinder | 5 | 3.13 | 2.21 | 1.81 | 1.56 | 1.40 |
| | 3-day Core | 3 | 2.10 | 1.48 | 1.21 | 1.05 | 0.94 |
| | 14-day Cylinder | 3 | 3.37 | 2.38 | 1.95 | 1.69 | 1.51 |
| | 28-day Cylinder | 3 | 3.10 | 2.19 | 1.79 | 1.55 | 1.39 |
| | 28-day Core | 3 | 2.63 | 1.86 | 1.52 | 1.31 | 1.18 |
| Split Tensile | 3-day Cylinder | 2 | 0.55 | 0.39 | 0.32 | 0.28 | 0.25 |
| Flexural | 3-day Beam | 3 | 0.41 | 0.29 | 0.24 | 0.20 | 0.18 |
| | 14-day Beam | 3 | 0.43 | 0.30 | 0.25 | 0.21 | 0.19 |
| | 28-day Beam | 5 | 0.30 | 0.21 | 0.18 | 0.15 | 0.14 |

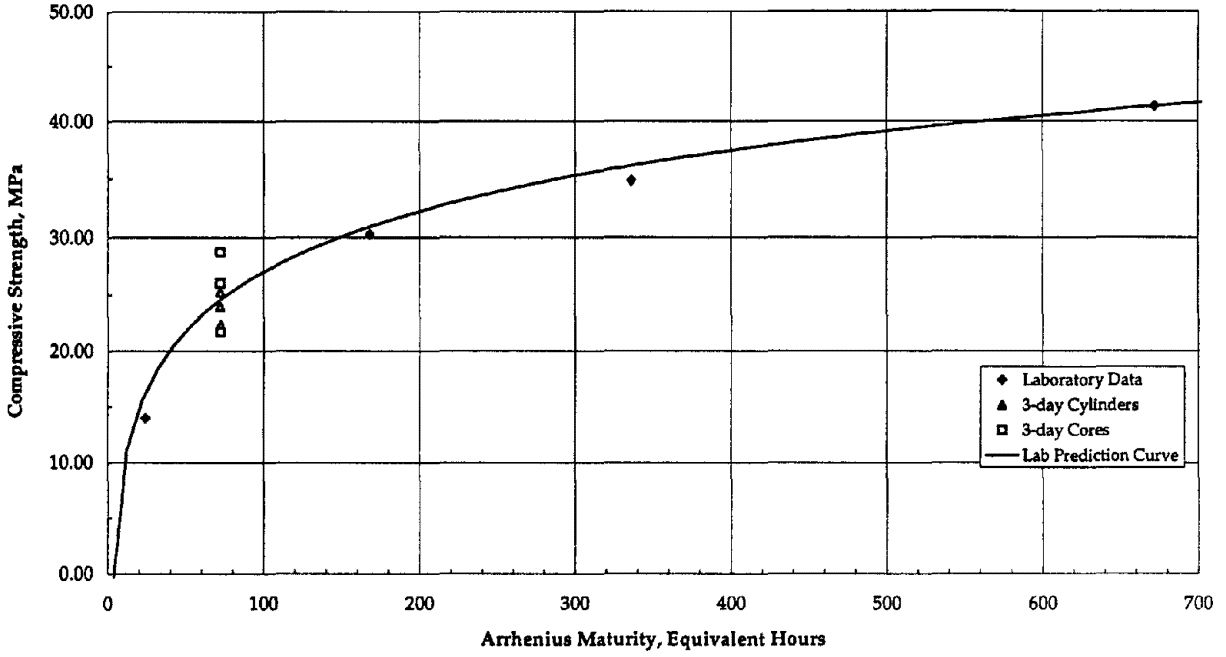



Figure 44. Compressive strength versus maturity curve for Shawano, WI.

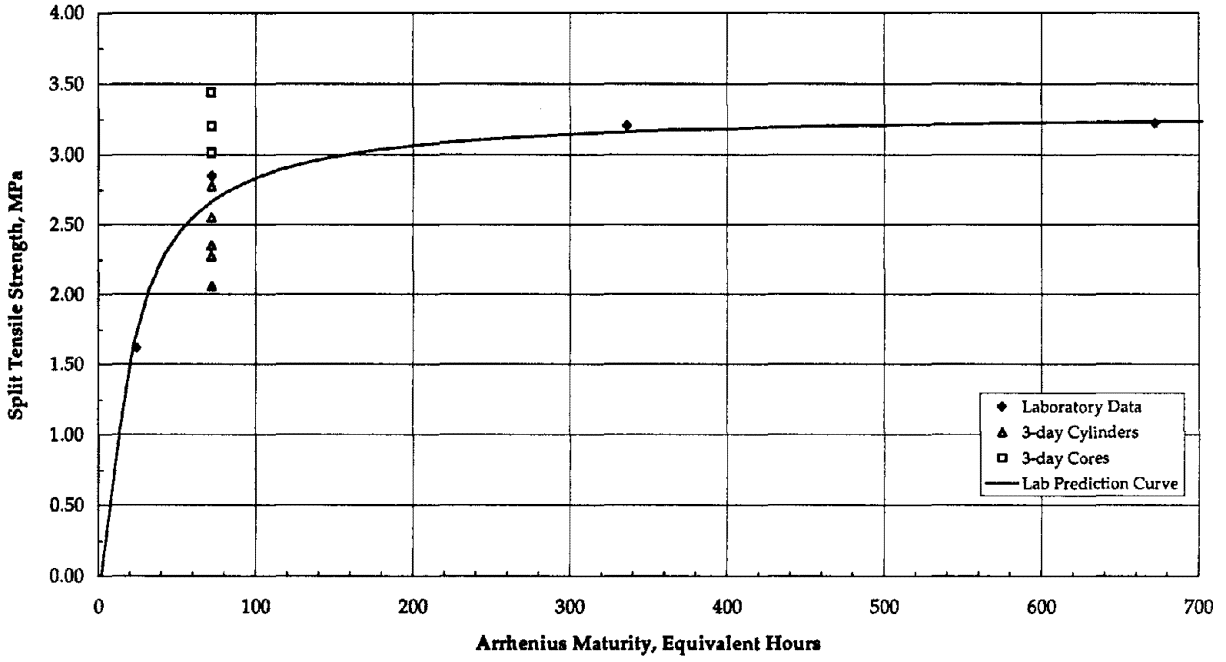



Figure 45. Splitting tensile strength versus maturity curve for Shawano, WI.

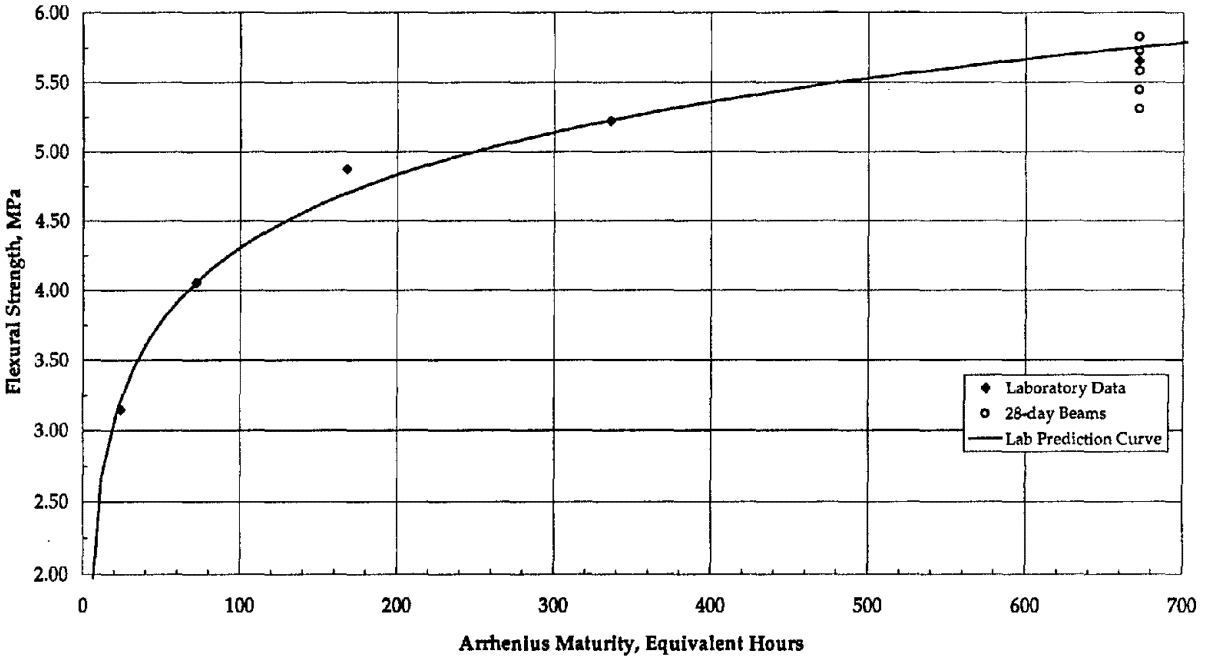



Figure 46. Flexural strength versus maturity curve for Shawano, WI.

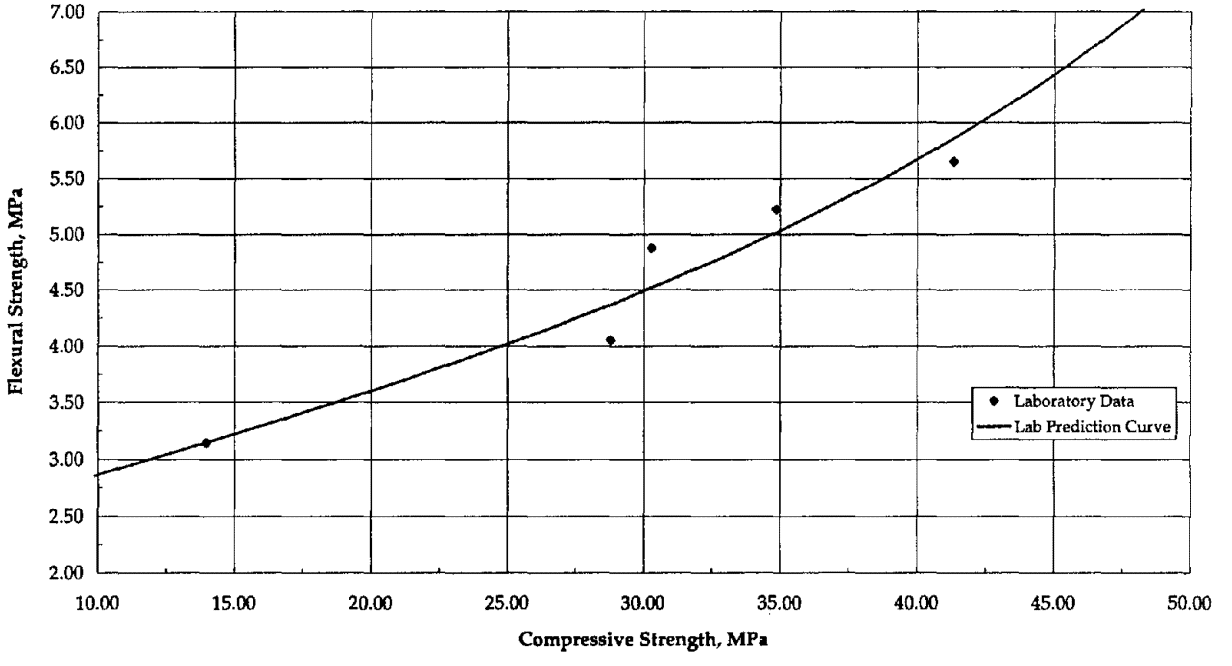



Figure 47. Flexural strength versus compressive strength relationship for Shawano, WI.

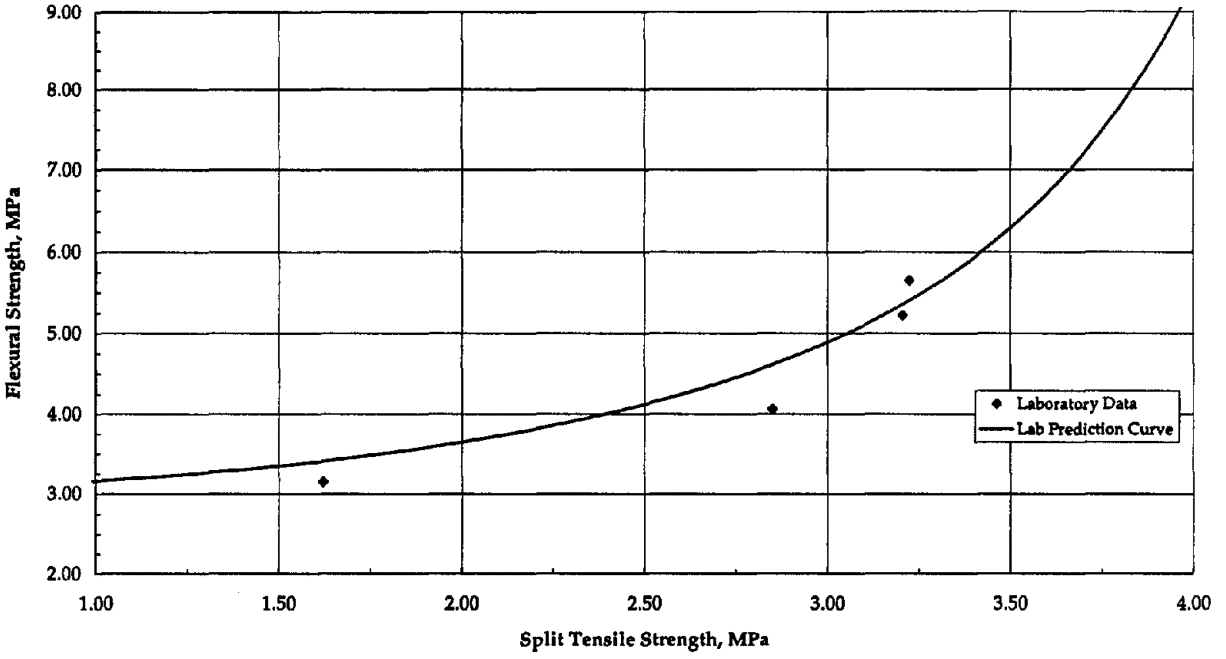



Figure 48. Flexural strength versus splitting tensile strength relationship for Shawano, WI.

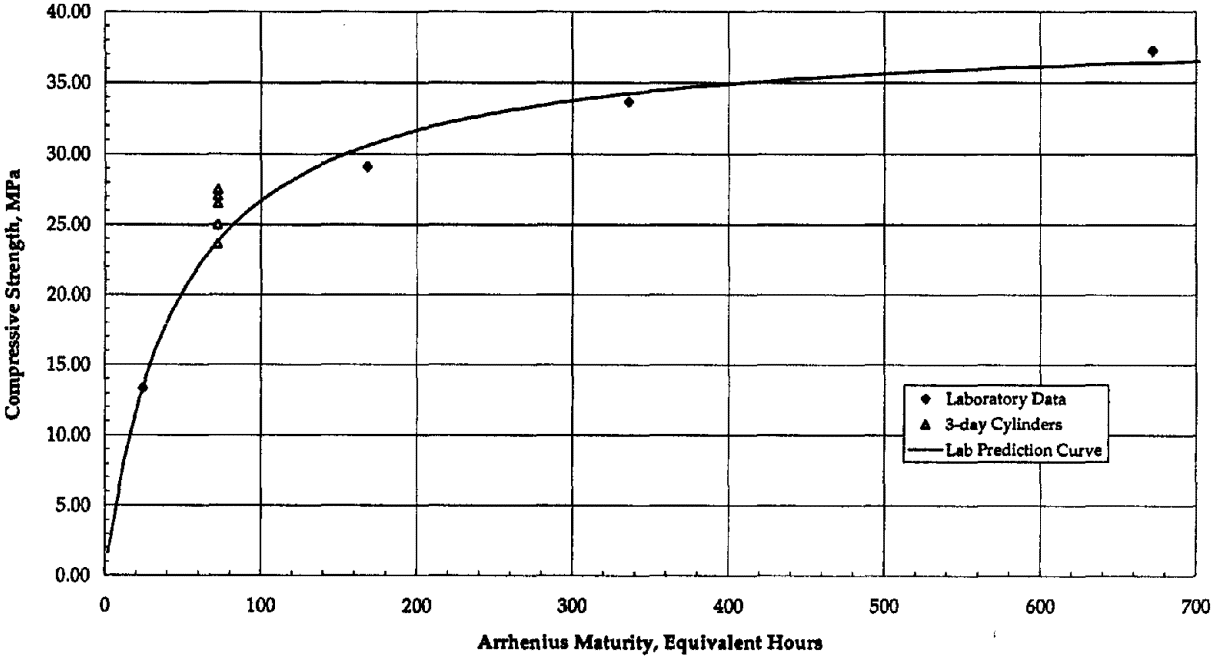



Figure 49. Compressive strength versus maturity curve for St. Johns, MI.

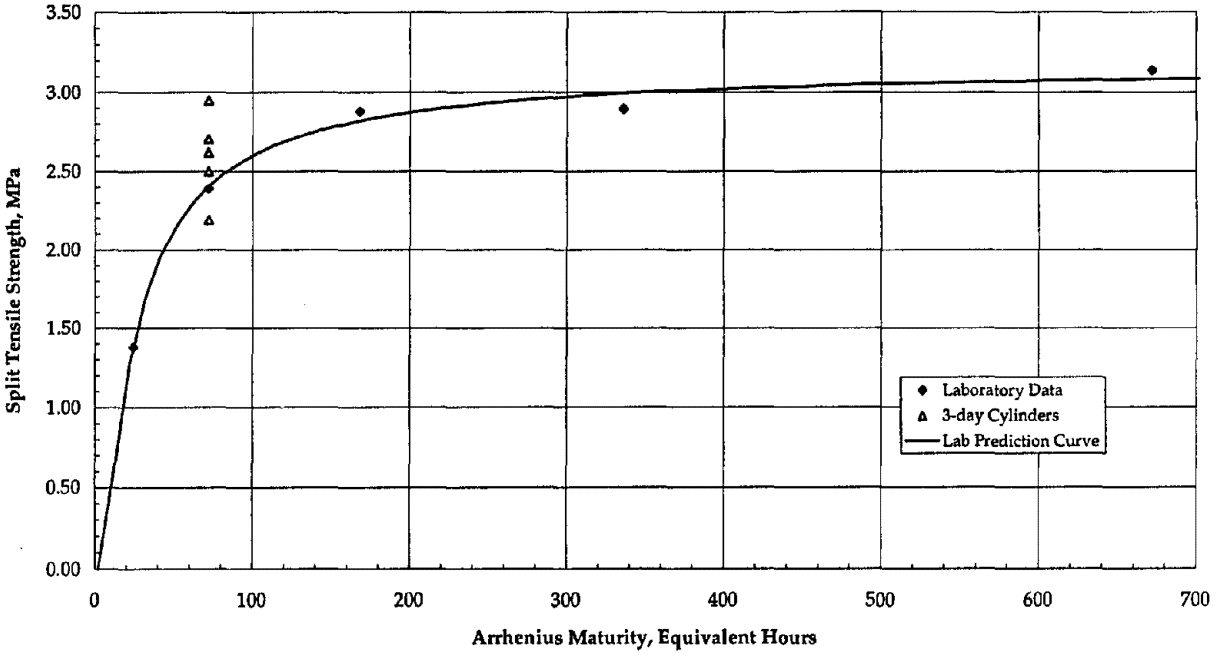



Figure 50. Splitting tensile strength versus maturity curve for St. Johns, MI.

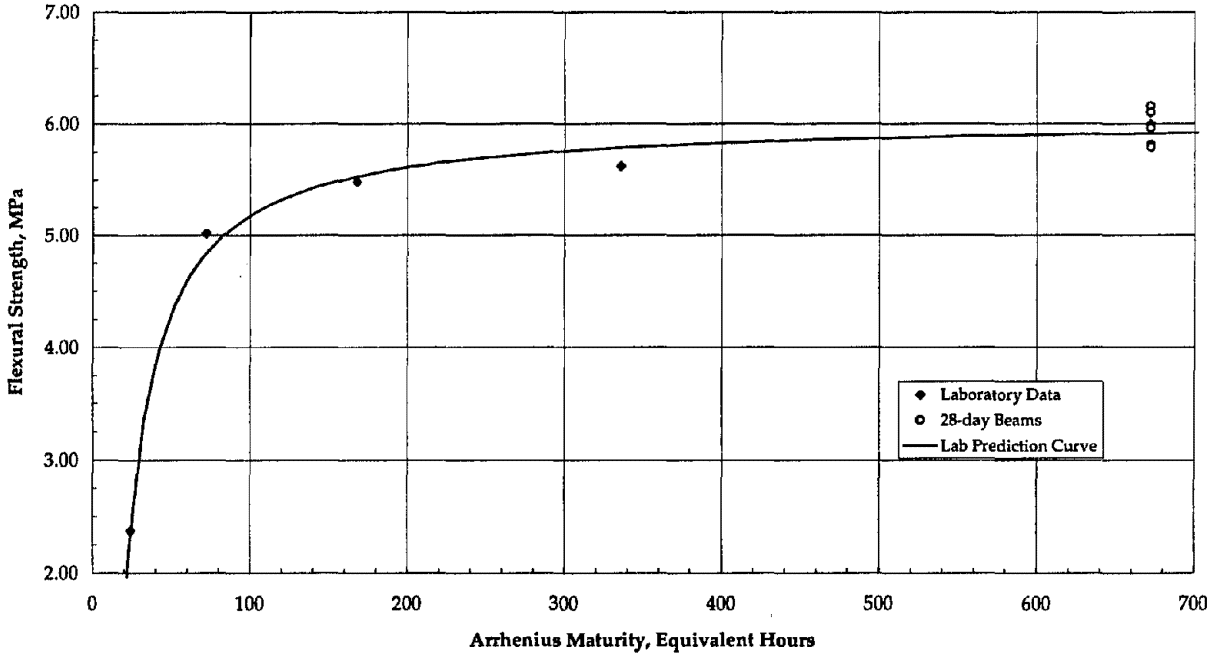



Figure 51. Flexural strength versus maturity curve for St. Johns, MI.

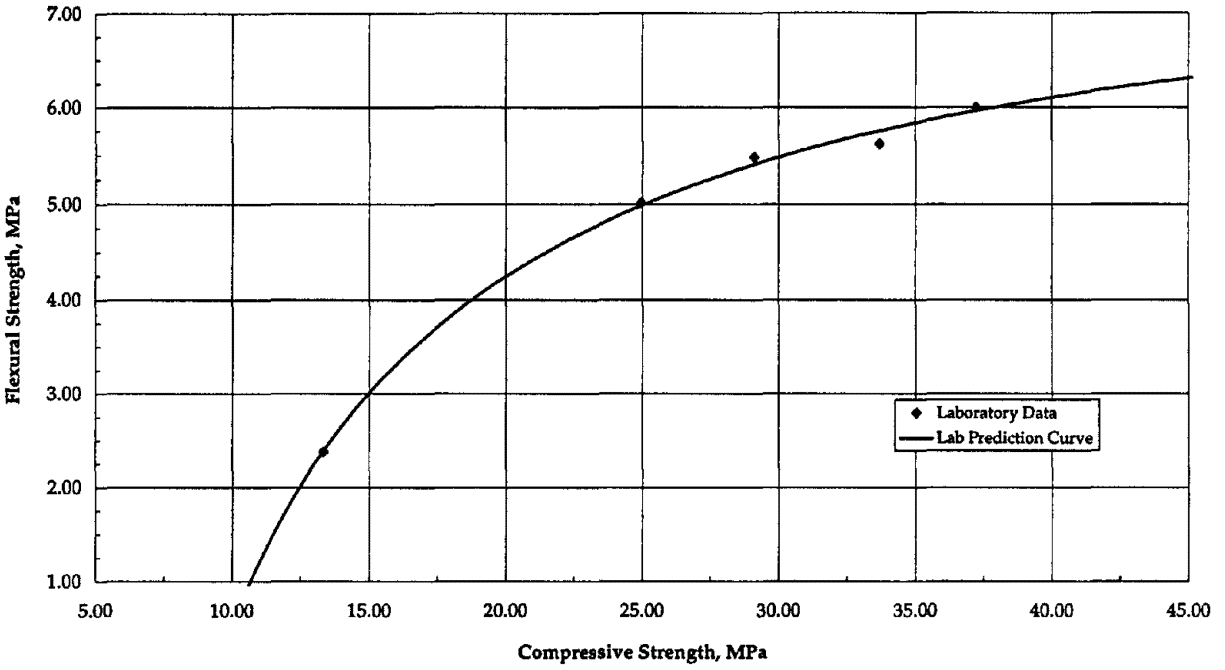



Figure 52. Flexural strength versus compressive strength relationship for St. Johns, MI.

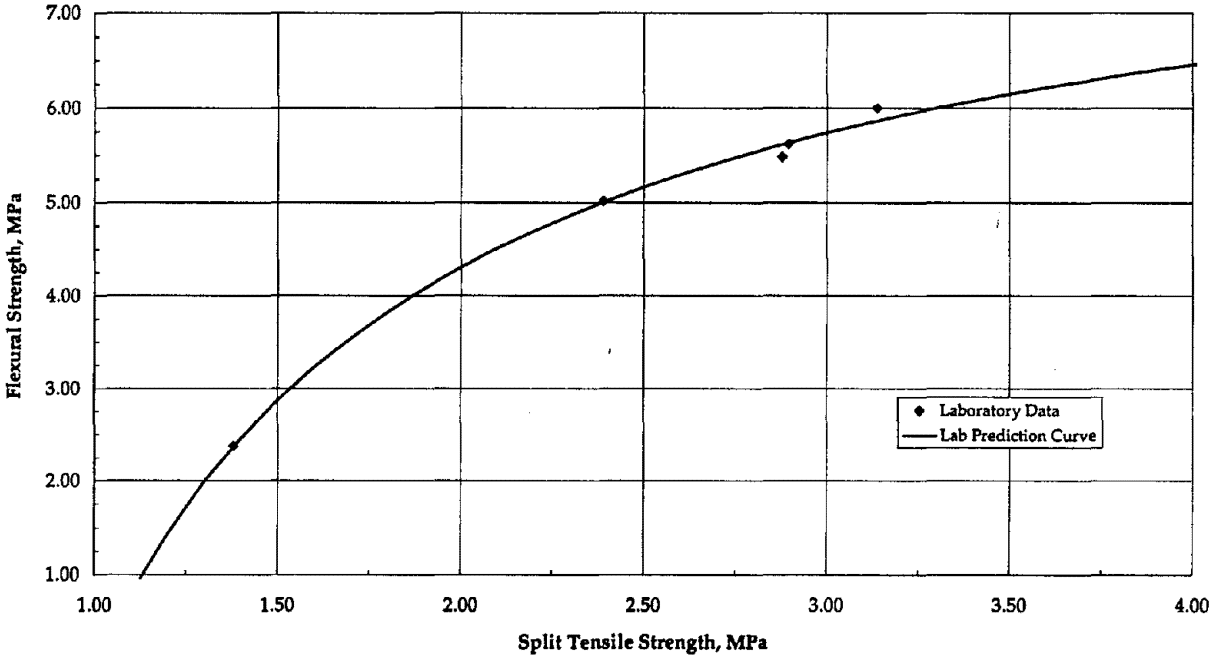



Figure 53. Flexural strength versus splitting tensile strength relationship for St. Johns, MI.

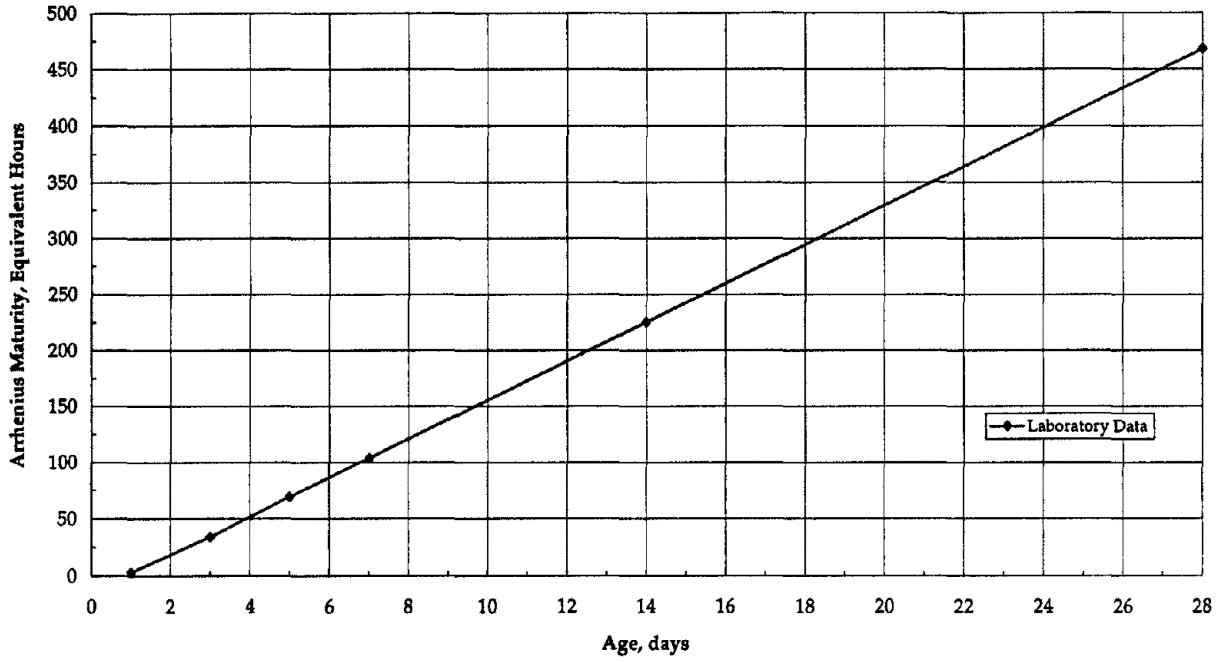



Figure 54. Arrhenius maturity versus time (based on laboratory data) for Ottumwa, IA.

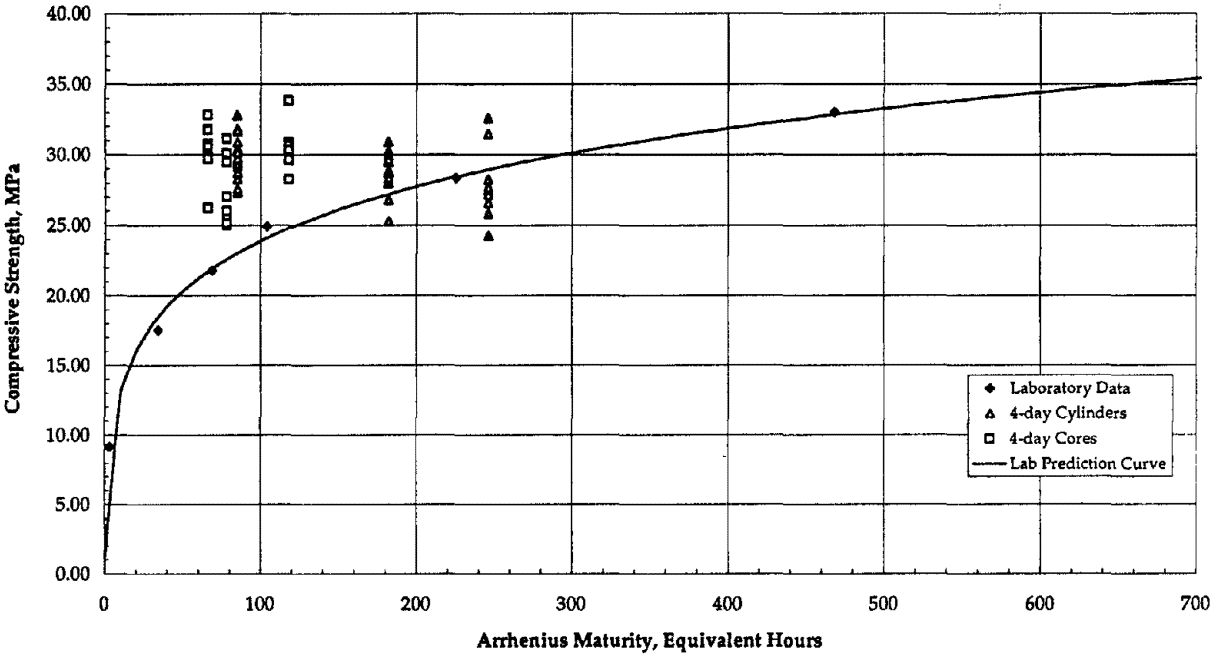



Figure 55. Compressive strength versus maturity curve for Ottumwa, IA.

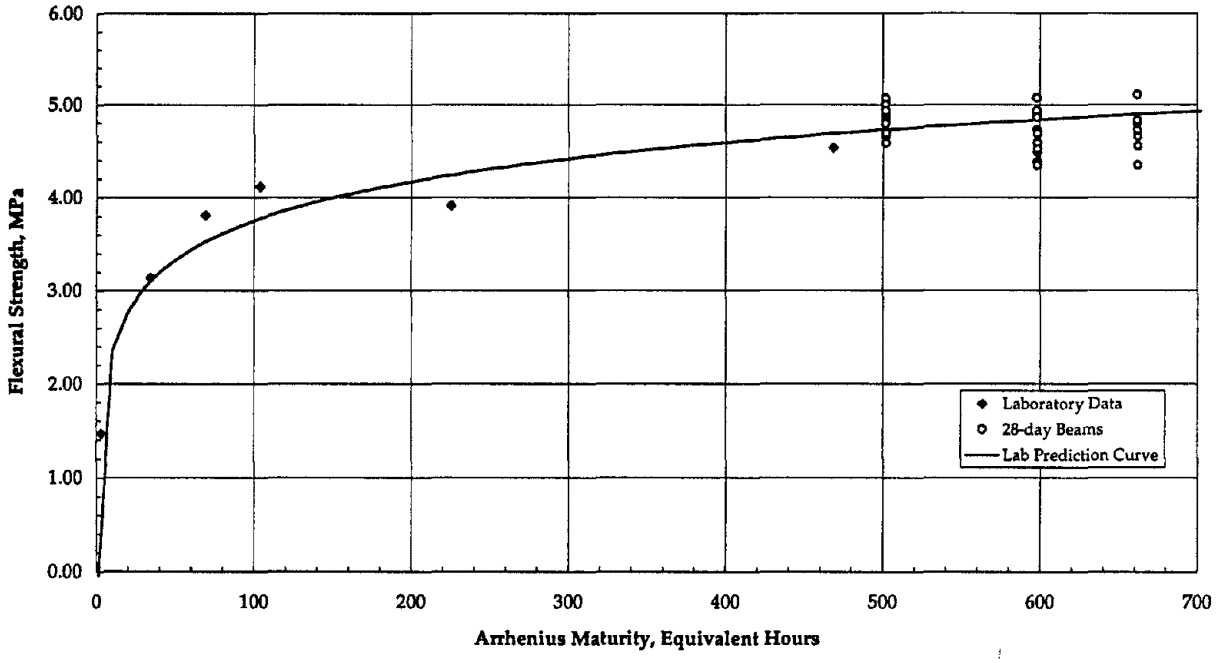



Figure 56. Flexural strength versus maturity curve for Ottumwa, IA.

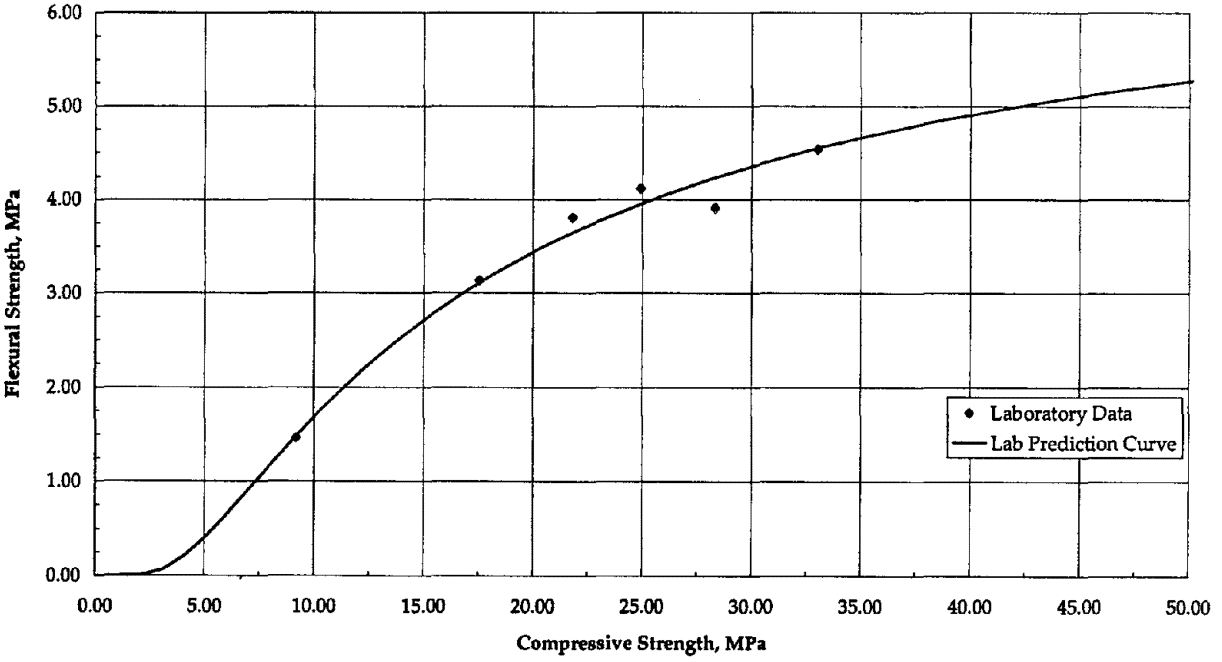



Figure 57. Flexural strength versus compressive strength relationship for Ottumwa, IA.

Table 48. Strength development and interstrength relationship regression equations.

| Project | Model | Dependent Variable | $R^2$ |
|---|---|---|---|
| Shawano, WI | f'c vs. ARR | f'c = 17.328 * log(ARR) – 7.647 | 0.938 |
| | ST vs. ARR | log(ST) = – 6.76 * (1/ARR) + 0.520 | 0.799 |
| | MR vs. ARR | MR = 1.75 * log(ARR) + 0.806 | 0.988 |
| | f'c vs. MR | 1/MR = – 0.055 * sqrt(f'c) + 0.521 | 0.947 |
| | ST vs. MR | $1/MR = – 0.014 * (ST)^2 + 0.330$ | 0.679 |
| St. Johns, MI | f'c vs. ARR | 1/f'c = 1.172 * (1/ARR) + 0.026 | 0.995 |
| | ST vs. ARR | log(ST) = – 8.706 * (1/ARR) + 0.502 | 0.996 |
| | MR vs. ARR | MR = – 86.990 * (1/ARR) + 6.047 | 0.992 |
| | f'c vs. MR | MR = – 74.238 * (1/f'c) + 7.956 | 0.997 |
| | ST vs. MR | MR = – 8.596 * (1/ST) + 8.604 | 0.996 |
| Ottumwa, IA | f'c vs. ARR | sqrt(f'c) = 1.258 * log(ARR) + 2.372 | 0.995 |
| | MR vs. ARR | MR = 1.393 * log(ARR) + 0.965 | 0.939 |
| | f'c vs. MR | log(MR) = – 6.180 * (1/f'c) + 0.845 | 0.988 |

Notes: f'c = compressive strength, MPa.
ARR = Arrhenius equivalent laboratory-cured maturity, equivalent hours.
ST = split tensile strength, MPa.
MR = flexural strength (modulus of rupture), MPa.

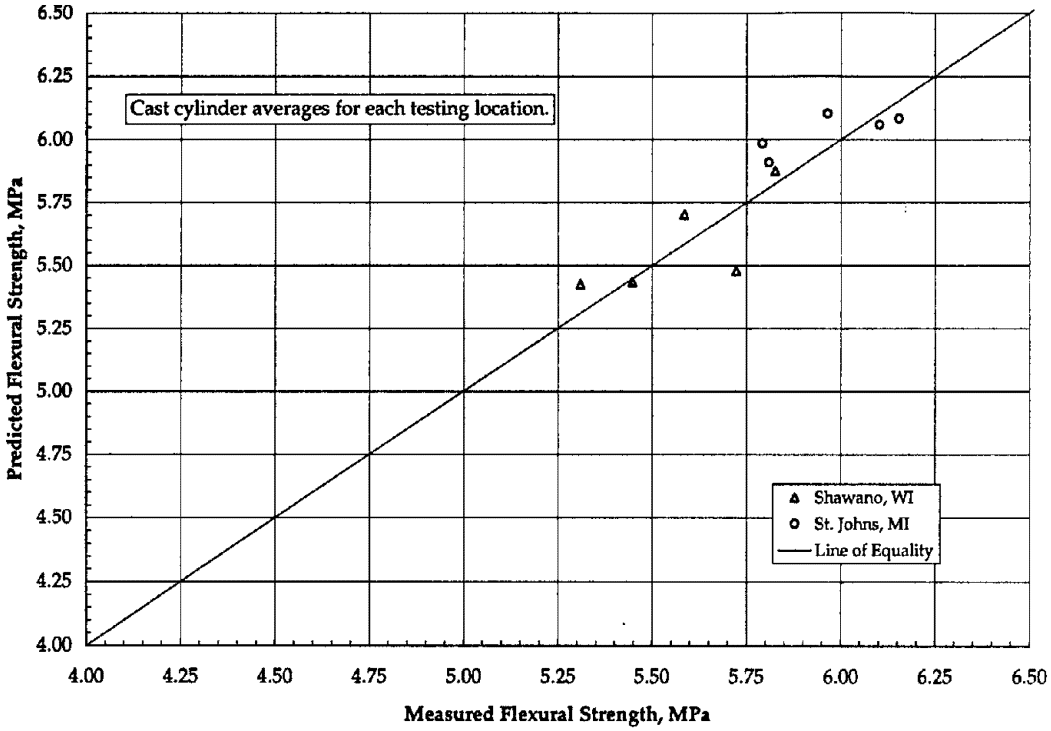



Figure 58. Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder f'c → 28-day f'c → 28-day MR).

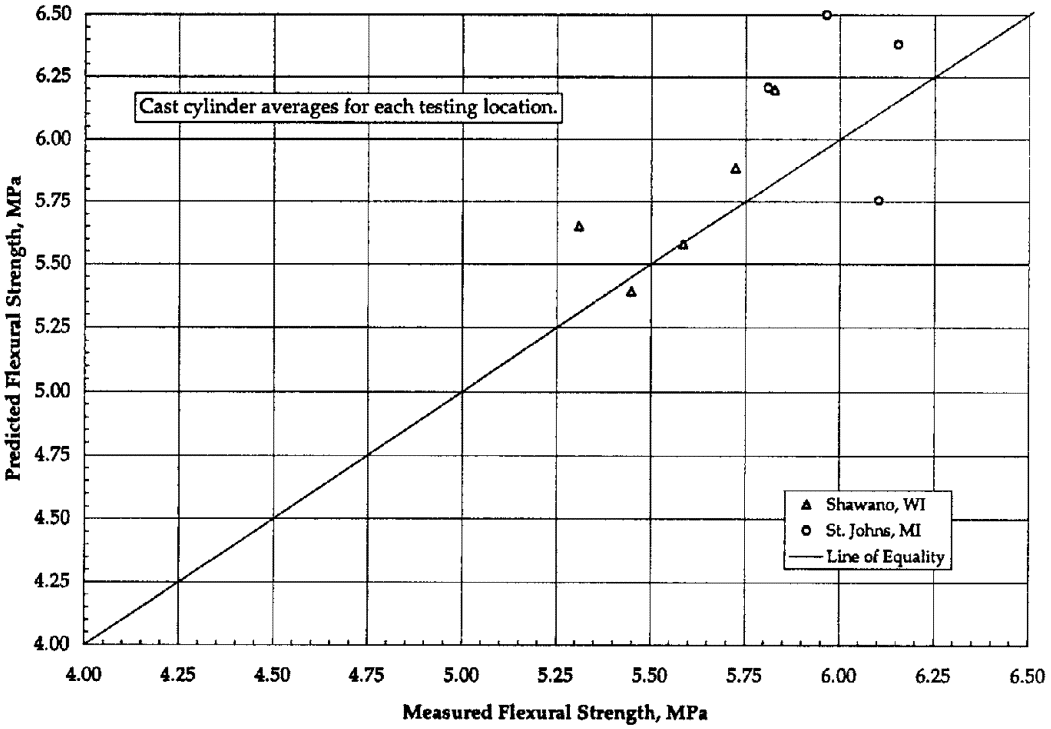



Figure 59. Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder ST → 3-day MR → 28-day MR).

splitting tensile strength model, a better prediction was achieved by first converting the 3-day cylinder strengths to 3-day flexural strengths, and then extrapolating to a 28-day flexural strength using maturity concepts. The accuracy of these two prediction procedures was evaluated by comparing the standard deviations of the differences between predicted and measured values. The core compressive strength model, with a standard deviation of 0.13 MPa, resulted in a higher prediction accuracy than the cylinder splitting tensile strength model, with a standard deviation of 0.36 MPa. Based on these results, attention was focused on compressive strength as the early age strength predictor for the Ottumwa project. Also, because this investigation was being used to evaluate the feasibility of accurately predicting the 28-day flexural strength at an acceptable earlier age, it was decided that two sampling ages would be attempted from which predictions would be made. It makes sense that the closer the sampling gets to the 28-day age, the more accurate the prediction should be. Therefore, it was decided to attempt predictions from 3- and 14-day compressive and flexural strengths. While the splitting tensile strength predictor was not evaluated for the Ottumwa project, early age beams were tested to evaluate the possibility of eliminating the error involved in converting from compressive to flexural strength.

The average predicted 28-day flexural strengths (for all of the different investigated prediction methods) are plotted versus the average measured 28-day flexural strengths in figures 60 through 66. Also, the average percent errors, and the standard deviations of the differences between predicted and measured values for each prediction model evaluated, are summarized in table 49. As shown in table 49, the most accurate prediction of 28-day flexural strength was based on the 28-day compressive strength estimated from a 3-day cylinder compressive strength (this method had the lowest average prediction error of 2.32 percent). It is believed, based on data collected during this and previous PRS studies, that the core compressive strength also accurately predicts the 28-day flexural strength. While the core prediction models slightly overpredicted the 28-day beam flexural strengths (as shown in figure 63), the cores were not necessarily removed from the same batch of concrete from which the beams and cylinders were sampled. After the concrete was sampled for the fabrication of beams and cylinders, the paver blended several batches (or loads) transversely across the pavement width. This incorporates any material variability into the core prediction model. However, based on the study of core-to-cylinder strength ratios (discussed in appendix E), the core models probably predict the flexural strength of the blended batches as accurately as the cylinder models predict the flexural strength of the same batch of concrete.

A brief evaluation of the American Concrete Institute (ACI) conversion equation was also performed. Simply converting a 28-day compressive strength to a 28-day flexural strength using the mix-specific relationships developed in the laboratory resulted in a conversion error of 4.2 percent. For comparison, using the compressive-to-flexural strength conversion suggested by ACI 318 ($MR = 7.5 * [f'c]^{0.5}$), the conversion error was 16 percent.[57] The 7.5 multiplier does not accurately convert compressive to

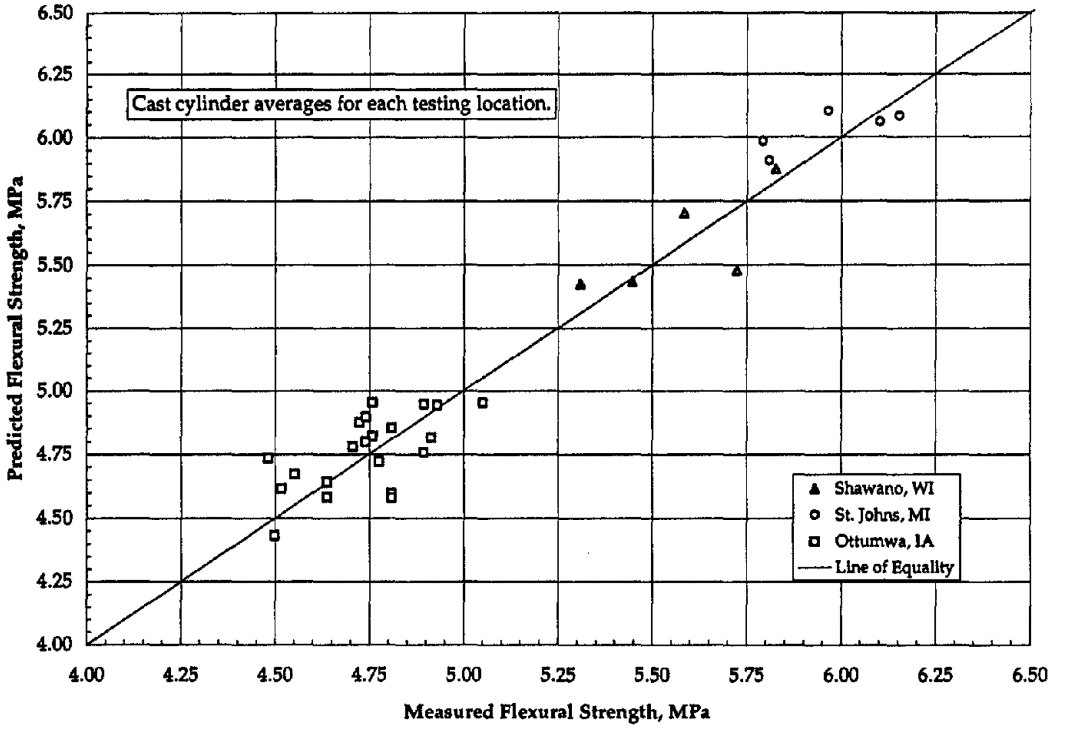



Figure 60. Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder f'c → 28-day f'c → 28-day MR).

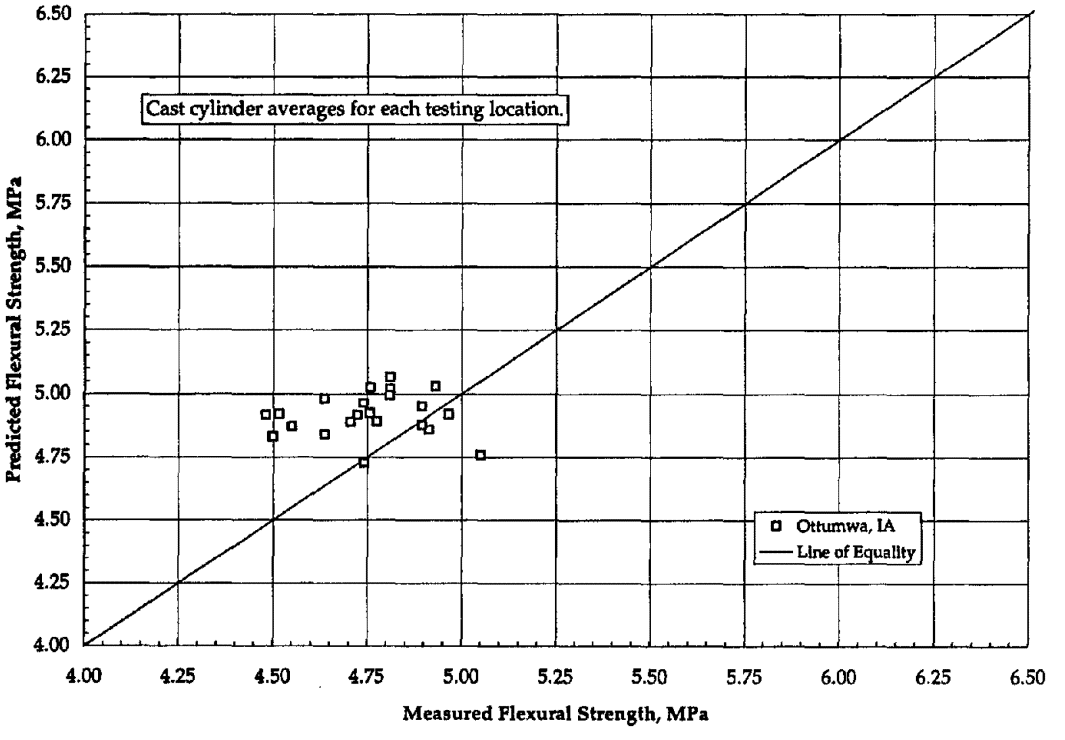



Figure 61. Predicted versus measured 28-day flexural strength (Translation method: 14-day cylinder f'c → 28-day f'c → 28-day MR).

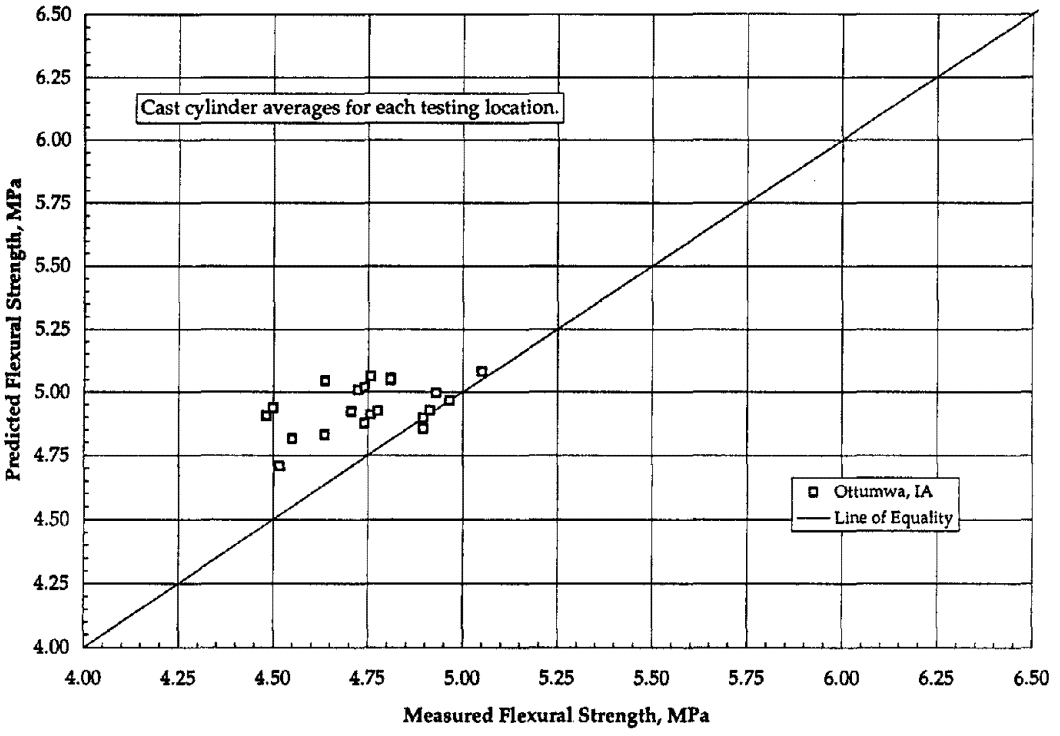



Figure 62. Predicted versus measured 28-day flexural strength (Translation method: 28-day f'c → 28-day MR).

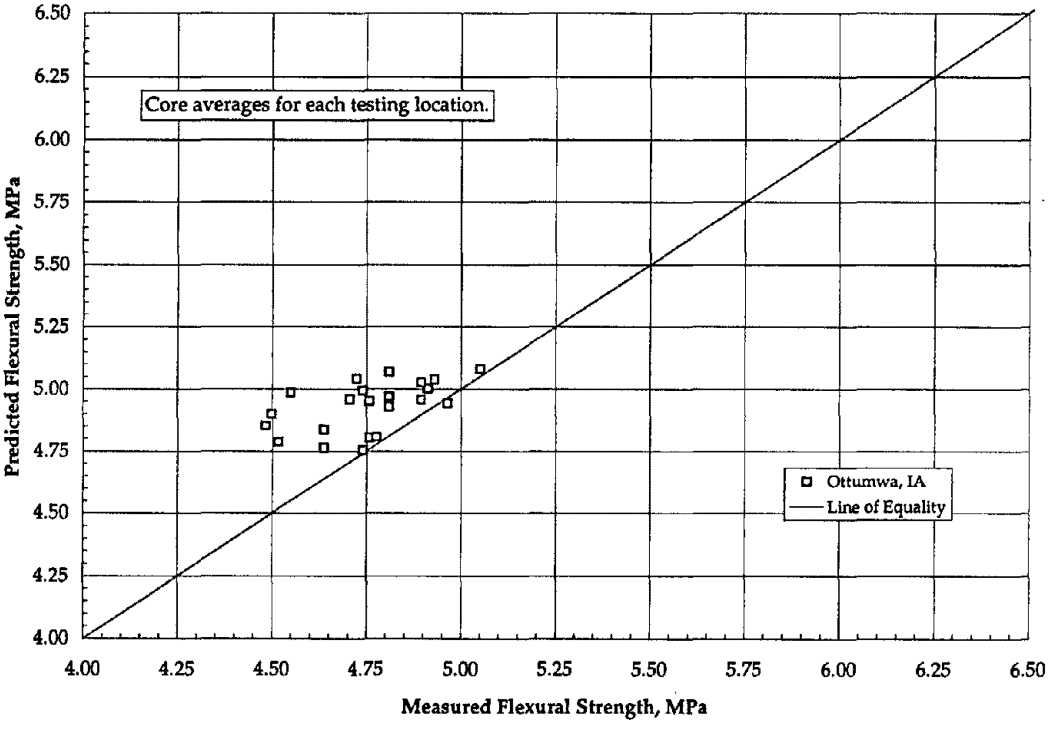



Figure 63. Predicted versus measured 28-day flexural strength (Translation method: 3-day core f'c → 28-day f'c → 28-day MR).

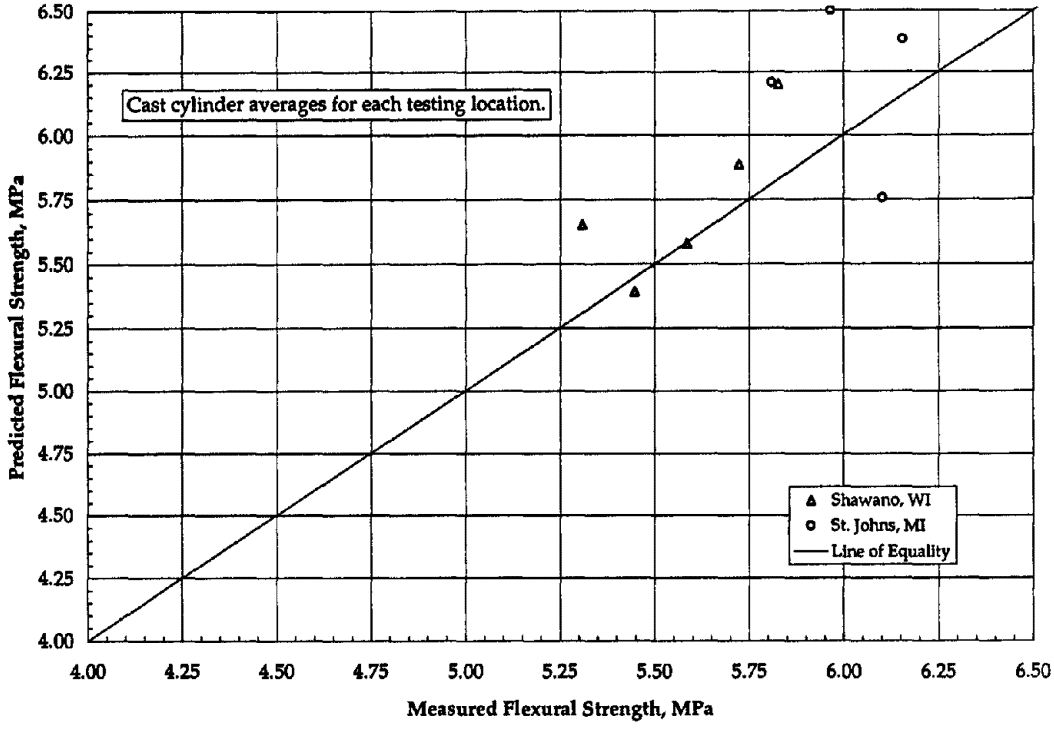



Figure 64. Predicted versus measured 28-day flexural strength (Translation method: 3-day cylinder ST → 3-day MR → 28-day MR).

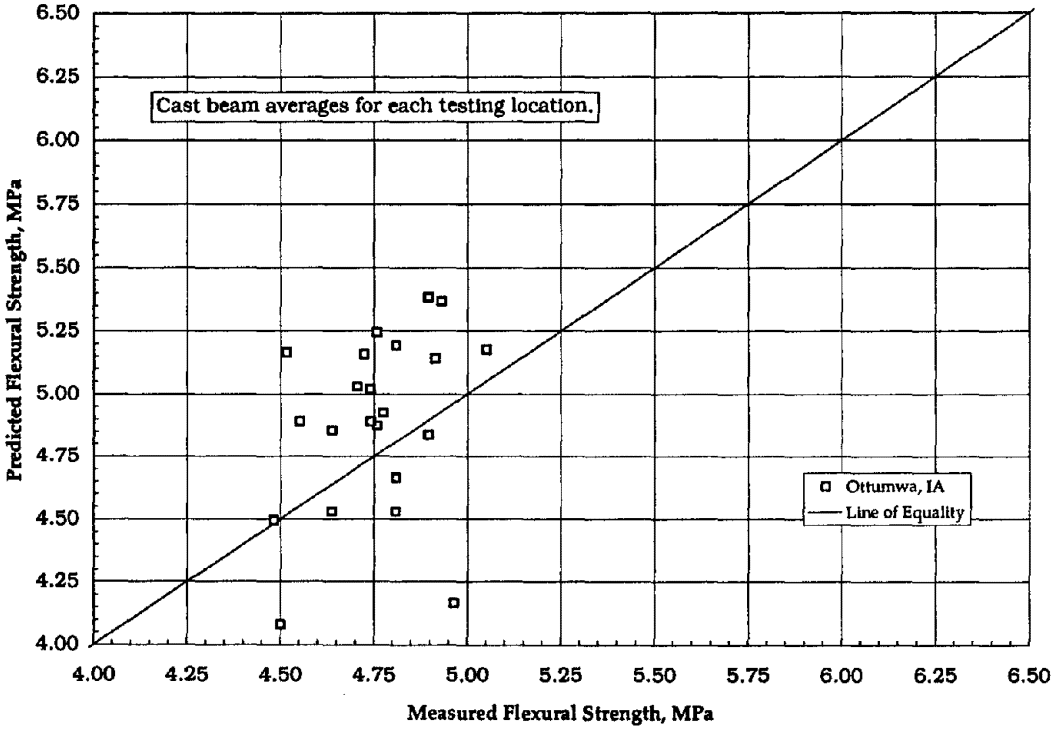



Figure 65. Predicted versus measured 28-day flexural strength (Translation method: 3-day beam MR → 28-day MR).

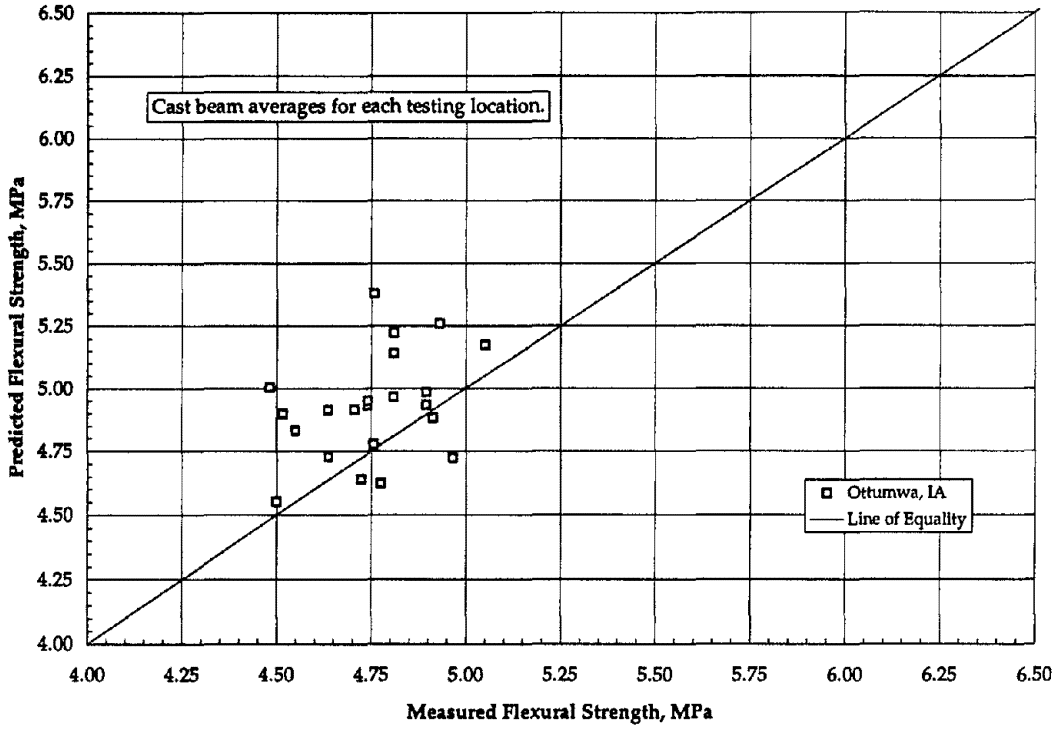



Figure 66. Predicted versus measured 28-day flexural strength (Translation method: 14-day beam MR → 28-day MR).

Table 49. Summary of accuracy indicators for the 28-day flexural strength prediction models.

| Sample Type | Model Translation Method | Standard Deviation of the Difference, MPa | Average Prediction Error, % |
|---|---|---|---|
| Cylinder Compressive Strength | 3-day f'c → 28-day f'c → 28-day MR | 0.12 | 2.32 |
| | 3-day f'c → 3-day MR → 28-day MR | 0.27 | 6.09 |
| | 14-day f'c → 28-day f'c → 28-day MR | 0.17 | 4.29 |
| | 14-day f'c → 14-day MR → 28-day MR | 0.19 | 3.15 |
| | 28-day f'c → 28-day MR | 0.14 | 4.22 |
| Core Compressive Strength | 3-day f'c → 28-day f'c → 28-day MR | 0.13 | 3.80 |
| | 3-day f'c → 3-day MR → 28-day MR | 0.16 | 4.82 |
| Cylinder Split Tensile Strength | 3-day ST → 28-day ST → 28-day MR | 0.39 | 8.21 |
| | 3-day ST → 3-day MR → 28-day MR | 0.36 | 5.88 |
| Beam Flexural Strength | 3-day MR → 28-day MR | 0.34 | 6.35 |
| | 14-day MR → 28-day MR | 0.21 | 4.69 |

flexural strength for all data sets. The multiplier was backcalculated using measured compressive and flexural strength data from specimens sampled from the same batch of concrete during the Ottumwa field investigation. The most accurate multiplier to use in the square root conversion equation is 9.00, as recommended by the Portland Cement Association in the publication *Slab Thickness Design for Industrial Concrete Floors on Grade*.[58] It was found that for samples cured for at least 4 days, age had little effect on the multiplier. Calculated multipliers for beams and cylinders cast in Ottumwa at ages 4, 14, and 28 days were 9.14, 9.11, and 8.94, respectively. Using 9.00 as the multiplier in the equation reduces the error in the 28-day conversion to 2.6 percent. While this prediction error is better than that produced using the mix-specific relationship developed for the project, the mix-specific relationship included data at 1 and 3 days. At very early ages, the data seem to indicate a trend toward a somewhat lower multiplier. Figure 67 contains a plot of the backcalculated multipliers versus testing age for each of the three investigated projects. Using the multiplier backcalculated from early age strength data resulted in an average prediction error of 11 percent.

For a total of 30 sampling locations, the 3-day cylinder compressive strength translation method (3-day cylinder f'c → 28-day f'c → 28-day MR) predicted the 28-day flexural strength with the greatest accuracy. While it was anticipated that better predictions would result from 14-day compressive strength or early age flexural strength (eliminating strength conversion error) data, increased variability in the data inhibits the accuracy of these prediction models. Further work is needed to investigate these discrepancies; however, predictions made from 3-day cylinder compressive strength samples would give the contractor earlier feedback so that alterations could be made as quickly as possible.

Using the most accurate translation method (3-day cylinder f'c → 28-day f'c → 28-day MR), the resulting 28-day flexural strengths were evaluated to recommend a target standard deviation for use in the PRS method. The 28-day flexural strength standard deviations (using the 3-day cylinder f'c → 28-day f'c → 28-day MR translation method) are presented in table 50 for different numbers of replicate specimens per sampling location. (Note: The recommended 28-day flexural strength standard deviations shown in table 50 are approximately equal to those presented in table 47.)

## SLAB THICKNESS

Few reports were found in the literature review that specifically addressed the variation of slab thickness. One report that compiled the results from several field studies did recommend that a point-to-point standard deviation of 8 mm be used in design.[8] For the current research study, the investigation of slab thickness variation consisted of examining cores removed from new pavement and rehabilitation construction projects. Also, because the industry is shifting toward nondestructive testing techniques, thickness data were obtained using GPR equipment. The standard deviations of within-lot GPR thickness measurements were also determined.

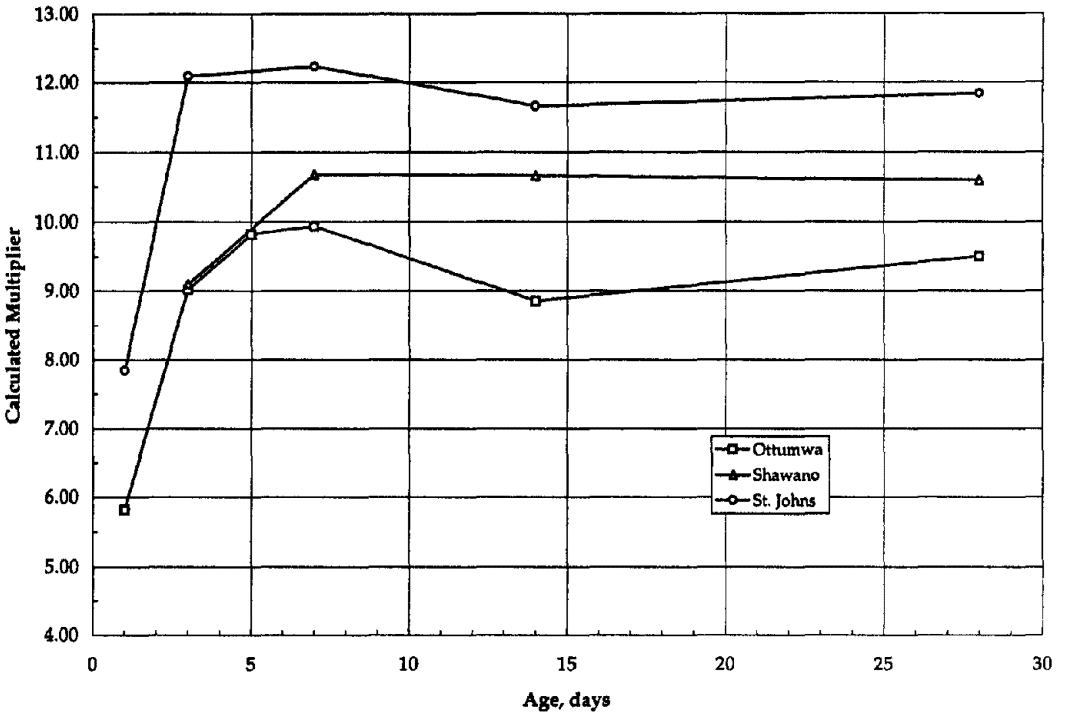



Figure 67. Backcalculated flexural strength conversion multiplier for specimens at different ages.

Table 50. Recommended concrete strength target standard deviations.

| Project Lot | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, MPa | | | | |
|---|---|---|---|---|---|---|
| | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Shawano, WI | 5 | 0.40 | 0.28 | 0.23 | 0.20 | 0.18 |
| St. Johns, MI | 5 | 0.16 | 0.11 | 0.09 | 0.08 | 0.07 |
| Ottumwa, IA Lot 1 | 6 | 0.25 | 0.18 | 0.15 | 0.13 | 0.11 |
| Ottumwa, IA Lot 2 | 8 | 0.11 | 0.07 | 0.06 | 0.05 | 0.05 |
| Ottumwa, IA Lot 3 | 8 | 0.13 | 0.09 | 0.08 | 0.07 | 0.06 |
| Recommended Standard Deviation | | 0.21 | 0.15 | 0.12 | 0.10 | 0.09 |

**Field Investigation of the Typical Slab Thickness Within-Lot Standard Deviation**

For each of the project sites investigated during this study, a significant number of cores were removed to evaluate characteristics such as hardened entrained air content, concrete strength, and percent consolidation around dowel bars. Full-depth cores were removed for these cases, so an evaluation of the slab thickness within-lot variation was completed using the measured thickness of each of these cores. Additional data were obtained from an historical project in Ontario, Canada (Highway 115), and other

research projects (as shown in table 45).[34] For three of the investigated projects (Rochelle, IL; Mankato, MN; and Bellefontaine, OH), thickness was also determined using GPR. Radar traverses were made at distances of 0.46, 1.37, 2.29, and 3.20 m from the centerline of the pavement. Thickness was evaluated at 3.05-m intervals along these traverses for each sublot.

### Summary of Slab Thickness Variation Results

The slab thickness variability evaluated for all projects is summarized by lot in tables 51 and 52 for drilled core and GPR data, respectively. Recommended average within-lot standard deviations are computed using an analysis of variance of the standard deviations obtained from each lot studied. As shown in table 51, for drilled cores without replicates (n = 1), the recommended mean within-lot standard deviation was computed to be 8.03 mm. This standard deviation matches the previous recommendation by Darter for portland cement concrete (PCC) pavement design, as well as values published by Yoder and Witczak for new highway construction.[8,59] Table 53 presents a comparison of the average standard deviations determined under this investigation with typical ranges given by Yoder and Witczak for various pavement thicknesses.

The data obtained by the GPR surveys yield a slightly higher average within-lot standard deviation of 9.5 mm. The higher variability recommended for the GPR as compared to drilled cores is probably due to a significantly higher number of thickness readings within each lot.

Thickness variation is a function of the grade control, base preparation and protection, and elevation control lines for the paver. For the new highway projects studied, these factors are generally tightly controlled by the contractor. For smaller projects, such as city streets, the factors affecting slab thickness may not be controlled as carefully. This may lead to greater variation, as suggested by Yoder and Witczak, who report a typical city street standard deviation range of 14 to 22 mm.[59]

## ENTRAINED AIR CONTENT

In situ entrained air content and associated air void system parameters can affect concrete durability. A model (developed from laboratory test data) for determining the percentage of joints exhibiting spalling/scaling as a function of air void system parameters was established in the preceding PRS study.[3] Two alternatives were recommended for incorporating in situ properties: incorporating air content measured directly from concrete in the plastic state using an air pressure meter, or using air void system parameters measured from linear traverse testing of cores. Commonly, when the air content exceeds minimum recommended percentages, the remaining air void system parameters also exceed recommended minimum standards.

Table 51. Summary of within-lot slab thickness standard deviations based on drilled core data.

| Project | Lot | Lot Average Thickness, mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, mm | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 242 | 23 | 6.1 | 4.3 | 3.5 | 3.0 | 2.7 |
| | 2 | 251 | 15 | 4.3 | 3.0 | 2.5 | 2.1 | 1.9 |
| Shawano, WI | 1 | 266 | 5 | 10.2 | 7.2 | 5.9 | 5.1 | 4.6 |
| St. Johns, MI | 1 | 241 | 5 | 11.3 | 8.0 | 6.6 | 5.7 | 5.1 |
| Ottumwa, IA | 1 | 311 | 12 | 8.3 | 5.9 | 4.8 | 4.1 | 3.7 |
| | 2 | 307 | 16 | 7.2 | 5.1 | 4.2 | 3.6 | 3.2 |
| | 3 | 308 | 16 | 7.3 | 5.2 | 4.2 | 3.6 | 3.3 |
| Ontario, Canada | 1 | 216 | 44 | 10.3 | 7.3 | 6.0 | 5.2 | 4.6 |
| Mankato, MN | 1 | 206 | 21 | 8.1 | 5.7 | 4.7 | 4.1 | 3.6 |
| | 2 | 209 | 20 | 6.8 | 4.8 | 3.9 | 3.4 | 3.0 |
| Bellefontaine, OH | 1 | 319 | 15 | 11.0 | 7.8 | 6.4 | 5.5 | 4.9 |
| | 2 | 307 | 15 | 9.5 | 6.7 | 5.5 | 4.8 | 4.3 |
| Des Plaines, IL | 1 | 237 | 10 | 9.8 | 6.9 | 5.7 | 4.9 | 4.4 |
| Benton Harbor, MI | 1 | 230 | 14 | 5.6 | 4.0 | 3.2 | 2.8 | 2.5 |
| Philo, IL | 1 | 204 | 7 | 4.6 | 3.3 | 2.7 | 2.3 | 2.1 |
| Recommended Target Standard Deviation | | | | 8.0 | 5.7 | 4.6 | 4.0 | 3.6 |

Table 52. Summary of within-lot slab thickness standard deviations based on ground-penetrating radar data.

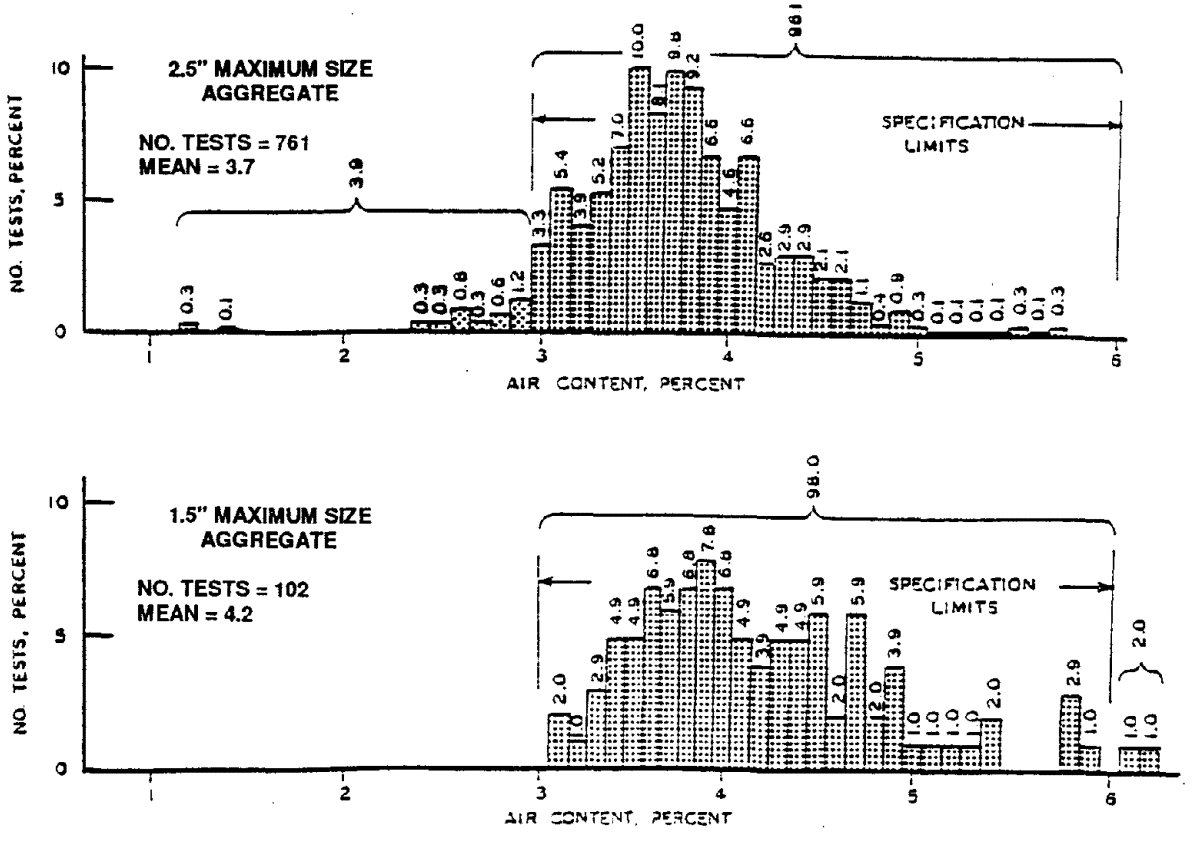
| Project | Lot | Lot Average Thickness, mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, mm | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 246 | 168 | 3.4 | 2.4 | 2.0 | 1.7 | 1.5 |
| | 2 | 249 | 200 | 1.6 | 1.1 | 0.9 | 0.8 | 0.7 |
| Mankato, MN | 1 | 304 | 200 | 10.9 | 7.7 | 6.3 | 5.5 | 4.9 |
| | 2 | 305 | 196 | 11.2 | 7.9 | 6.5 | 5.6 | 5.0 |
| Bellefontaine, OH | 1 | 194 | 201 | 15.1 | 10.6 | 8.7 | 7.5 | 6.7 |
| | 2 | 187 | 200 | 14.7 | 10.4 | 8.5 | 7.4 | 6.6 |
| Recommended Target Standard Deviation | | | | 9.5 | 6.7 | 5.5 | 4.7 | 4.2 |

Table 53. Comparison of average slab thickness standard deviations for different design thicknesses.

| Slab Design Thickness, mm | Standard Deviation Range, mm [10] | Standard Deviation Range Determined Under the Current PRS Study, mm |
|---|---|---|
| 203 | 4.8 – 11.2 | 4.6 – 10.4 |
| 229 | 2.5 – 11.9 | 5.6 – 11.4 |
| 254 | 5.8 – 7.4 | 4.3 – 10.2 |

### Historical Entrained Air Content Data

Very little historical data were readily available on the typical within-lot standard deviation of in situ air void system parameters (air content, void spacing factor, specific surface, voids per mm). However, one interesting source of data on the variation of air content was the American Association of State Highway Officials (AASHO) Road Test, in which cores were retrieved and tested for entrained air content. Hardened and plastic concrete air content data from the Road Test are illustrated in figure 68.[60] Plastic concrete air content data obtained from the Illinois Department of Transportation (IDOT) database are summarized in table 27 (in appendix C). Overall standard deviations from both data sources ranged from 0.32 to 1.13 percent. Although some studies have reported inconsistencies between plastic and hardened air contents, on average, there was not a significant difference in standard deviations measured for these data sets. Average standard deviations measured in the plastic and hardened state were 0.52 and 0.57 percent, respectively. The weighted average for the data was 0.54 percent. There is no significant trend between standard deviation and average air content, suggesting that air content variability can be represented by the standard deviation, rather than by the coefficient of variation.




1 in = 25.4 mm

Figure 68. Air content measured on cores at the AASHO Road Test.[60]

The air content variations determined in the AASHO and IDOT research were computed from a significant number of tests. Presumably, the data cover multiple sublots placed on multiple paving days. However, using the weighted average of 0.54 percent may not be realistic for use in a PRS where smaller lots (comprised of a maximum of one day's production) are of interest. Inherent multi-day material or batching variability is included in the 0.54 percent weighted average. For the PRS, the variation of interest is the within-lot standard deviation.

**Field Investigation of the Typical Entrained Air Content Within-Lot Standard Deviation**

Plastic and hardened air content testing was conducted at new pavement construction projects in order to evaluate the standard deviation of entrained air content representative of individual lots (considered to be one day of paving). Fresh concrete retrieved from before and after the paver was tested at each of the three projects where flexural strength standard deviation was determined. Cores were removed from locations where before- and after-paver plastic air contents were measured. The cores were used to determine the hardened air content properties of the concrete (via linear traverse testing).

Field Sampling and Testing Procedures

To evaluate the variation in the measurement of plastic air content, fresh concrete was sampled at the paver in accordance with ASTM D 172, *Standard Practice for Sampling Freshly Mixed Concrete.* Concrete was sampled from the discharged material in front of the paver (at each randomly selected longitudinal sampling location) and transported by wheelbarrow to the testing location. The air content of the concrete was measured in accordance with ASTM C 231, *Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.* When possible, a sample of concrete was removed from the same location after the paver had distributed, vibrated, and finished the plastic concrete. In general, due to safety concerns (as well as to protect the slip-formed edge), the contractor would not allow research personnel to retrieve the concrete from behind the paver. However, the finishers retrieved the samples by shovel from approximately 0.61 m from the pavement edge. The void left in the finished pavement was replaced with fresh concrete and finished by hand. Because the samples were retrieved from the wheelpath where smoothness was measured, the contractor would not allow an equal number of tests to be performed after the paver. However, enough data were collected so that significant conclusions could be drawn.

The literature review indicated that several studies report a significant difference between air contents measured in the plastic and hardened states. In general, no data were reported on the variability between the plastic and hardened states. Nagi and Whiting investigated possible factors that may affect the discrepancies in entrained air content measurements.[25] Based on their research, it was concluded that, although discrepancies almost always exist, they are not consistent, and if the testing is

performed in accordance with ASTM procedures, the discrepancies should be minimal. To evaluate any differences between the measurement of plastic and hardened air content, cores were sampled at selected plastic concrete points for comparison. Hardened air void system parameters were established in accordance with ASTM C 457, *Standard Test Method for Microscopical Determination of Parameters of the Air Void System in Hardened Concrete*. The linear traverse data were also used to establish the typical within-lot standard deviation of hardened concrete air void system parameters for possible use in the PRS approach.

Summary of Entrained Air Content Results

*Entrained Air Content Standard Deviation*

The typical plastic entrained air content within-lot standard deviations were computed and summarized in table 54. The table includes the results of data obtained from concrete sampled before and after the paver. As with the within-lot concrete strength standard deviations, each entrained air content within-lot standard deviation is computed using all of the representative sample values obtained from the lot. The recommended typical standard deviations are also given as a function of the number of replicate tests used (as discussed previously).

For this investigation, the average standard deviation of unreplicated (n=1) plastic air content measurements taken from concrete before the paver for each lot studied was 0.75 percent. This is slightly higher than the average standard deviation of 0.54 percent reported in the AASHO and IDOT studies. However, when replicate air content measurements are averaged for representative samples, the average standard deviation between samples is 0.92 for the lots studied, which compares favorably with Yoder and Witczak, who reported a typical standard deviation of 0.83 for central plant mix concrete measured using a pressure meter.[59]

When the plastic air content was measured on concrete sampled from behind the paver, the recommended standard deviation was 0.78 percent. In this study, no replicate measurements were taken on concrete sampled after the paver.

Very little published information exists on the variation of hardened air void system parameters (measured using the linear traverse technique on cores removed from the pavement). Therefore, as part of the field investigation, cores were removed from a number of projects to evaluate the variation of each air void system parameter. The resulting recommended total within-lot standard deviations for entrained air content, void spacing factor, specific surface, and voids per mm are presented in tables 55 through 58. Table 59 contains an overall summary of the within-lot standard deviation information (using all of the data collected from all projects) for each air void system parameter.

Table 54. Summary of within-lot plastic entrained air content standard deviations (based on air pressure meter data).

| Sample Location | Project | Lot | Lot Average Entrained Air Content, % | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, % | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Before Paver | Shawano, WI | 1 | 7.25 | 14 | 0.67 | 0.48 | 0.39 | 0.34 | 0.30 |
| | St. Johns, MI | 1 | 5.47 | 29 | 0.59 | 0.42 | 0.34 | 0.30 | 0.27 |
| | Ottumwa, IA | 1 | 7.87 | 3 | 0.40 | 0.29 | 0.23 | 0.20 | 0.18 |
| | | 2 | 7.69 | 4 | 0.30 | 0.21 | 0.17 | 0.15 | 0.13 |
| | | 3 | 8.91 | 4 | 0.45 | 0.32 | 0.26 | 0.23 | 0.20 |
| | Rochelle, IL | 1 | 6.28 | 26 | 0.89 | 0.63 | 0.51 | 0.44 | 0.40 |
| | | 2 | 4.77 | 14 | 1.62 | 1.14 | 0.93 | 0.81 | 0.72 |
| | Mankato, MN | 1 | 6.03 | 15 | 0.50 | 0.36 | 0.29 | 0.25 | 0.22 |
| | | 2 | 5.46 | 14 | 0.51 | 0.36 | 0.29 | 0.25 | 0.23 |
| | Bellefontaine, OH | 1 | 6.83 | 31 | 1.22 | 0.86 | 0.70 | 0.61 | 0.55 |
| | | 2 | 8.11 | 30 | 1.08 | 0.76 | 0.62 | 0.54 | 0.48 |
| Recommended Target Standard Deviation | | | | | 0.75 | 0.53 | 0.43 | 0.37 | 0.33 |
| After Paver | Rochelle, IL | 1 | 5.34 | 5 | 0.78 | 0.55 | 0.45 | 0.39 | 0.35 |
| | Shawano, WI | 1 | 7.08 | 14 | 1.03 | 0.73 | 0.59 | 0.51 | 0.46 |
| | St. Johns, MI | 1 | 4.68 | 29 | 0.53 | 0.38 | 0.31 | 0.27 | 0.24 |
| Recommended Target Standard Deviation | | | | | 0.78 | 0.55 | 0.45 | 0.39 | 0.35 |

Table 55. Summary of within-lot hardened entrained air content standard deviations (linear traverse testing data).

| Project | Lot | Lot Average Entrained Air Content, % | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, % | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 7.46 | 5 | 1.38 | 0.98 | 0.80 | 0.69 | 0.62 |
| Shawano, WI | 1 | 7.66 | 5 | 2.11 | 1.50 | 1.22 | 1.06 | 0.95 |
| St. Johns, MI | 1 | 7.32 | 5 | 1.88 | 1.33 | 1.08 | 0.94 | 0.84 |
| Ottumwa, IA | 1 | 8.20 | 3 | 1.82 | 1.29 | 1.05 | 0.91 | 0.81 |
| | 2 | 9.85 | 4 | 2.56 | 1.81 | 1.48 | 1.28 | 1.15 |
| | 3 | 11.25 | 4 | 2.86 | 2.02 | 1.65 | 1.43 | 1.28 |
| Des Plaines, IL | 1 | 5.54 | 5 | 0.86 | 0.61 | 0.50 | 0.43 | 0.39 |
| Benton Harbor, MI | 1 | 7.35 | 6 | 1.88 | 1.33 | 1.09 | 0.94 | 0.84 |
| Philo, IL | 1 | 5.34 | 5 | 1.74 | 1.23 | 1.00 | 0.87 | 0.78 |
| Recommended Target Hardened Entrained Air Content Std. Dev. | | | | 1.90 | 1.34 | 1.10 | 0.95 | 0.85 |

Table 56. Summary of within-lot void spacing factor standard deviations (linear traverse testing data).

| Project | Lot | Lot Average Void Spacing Factor, mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, mm | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 0.112 | 5 | 0.023 | 0.016 | 0.013 | 0.011 | 0.010 |
| Shawano, WI | 1 | 0.081 | 5 | 0.011 | 0.008 | 0.007 | 0.006 | 0.005 |
| St. Johns, MI | 1 | 0.086 | 5 | 0.014 | 0.010 | 0.008 | 0.007 | 0.006 |
| Ottumwa, IA | 1 | 0.119 | 3 | 0.015 | 0.010 | 0.008 | 0.007 | 0.007 |
| | 2 | 0.102 | 4 | 0.021 | 0.015 | 0.012 | 0.010 | 0.009 |
| | 3 | 0.089 | 4 | 0.015 | 0.010 | 0.008 | 0.007 | 0.007 |
| Des Plaines, IL | 1 | 0.218 | 5 | 0.069 | 0.049 | 0.040 | 0.034 | 0.031 |
| Benton Harbor, MI | 1 | 0.140 | 6 | 0.045 | 0.032 | 0.026 | 0.022 | 0.020 |
| Philo, IL | 1 | 0.259 | 5 | 0.073 | 0.051 | 0.042 | 0.036 | 0.033 |
| Recommended Target Within-Lot Void Spacing Factor Std. Dev. | | | | 0.032 | 0.022 | 0.018 | 0.016 | 0.014 |

Table 57. Summary of within-lot specific surface area standard deviations (linear traverse testing data).

| Project | Lot | Lot Average Specific Surface Area, $mm^2/mm^3$ | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, $mm^2/mm^3$ | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 33.40 | 5 | 3.57 | 2.52 | 2.06 | 1.78 | 1.60 |
| Shawano, WI | 1 | 41.83 | 5 | 7.94 | 5.61 | 4.58 | 3.97 | 3.55 |
| St. Johns, MI | 1 | 34.98 | 5 | 6.56 | 4.64 | 3.79 | 3.28 | 2.93 |
| Ottumwa, IA | 1 | 27.72 | 3 | 4.50 | 3.18 | 2.60 | 2.25 | 2.01 |
| | 2 | 23.82 | 4 | 4.42 | 3.12 | 2.55 | 2.21 | 1.98 |
| | 3 | 25.08 | 4 | 5.06 | 3.58 | 2.92 | 2.53 | 2.26 |
| Des Plaines, IL | 1 | 20.92 | 5 | 6.13 | 4.33 | 3.54 | 3.06 | 2.74 |
| Benton Harbor, MI | 1 | 25.70 | 6 | 7.19 | 5.08 | 4.15 | 3.59 | 3.21 |
| Philo, IL | 1 | 16.03 | 5 | 5.61 | 3.97 | 3.24 | 2.81 | 2.51 |
| Recommended Target Specific Surface Area Std. Dev. | | | | 5.66 | 4.00 | 3.27 | 2.83 | 2.53 |

Table 58. Summary of within-lot voids per mm standard deviations (linear traverse testing data).

| Project | Lot | Lot Average Voids per mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, voids/mm | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Rochelle, IL | 1 | 0.620 | 5 | 0.113 | 0.080 | 0.065 | 0.057 | 0.051 |
| Shawano, WI | 1 | 0.776 | 5 | 0.127 | 0.090 | 0.073 | 0.064 | 0.057 |
| St. Johns, MI | 1 | 0.619 | 5 | 0.052 | 0.037 | 0.030 | 0.026 | 0.023 |
| Ottumwa, IA | 1 | 0.554 | 3 | 0.032 | 0.022 | 0.018 | 0.016 | 0.014 |
| | 2 | 0.566 | 4 | 0.074 | 0.052 | 0.043 | 0.037 | 0.033 |
| | 3 | 0.683 | 4 | 0.100 | 0.071 | 0.058 | 0.050 | 0.045 |
| Des Plaines, IL | 1 | 0.294 | 5 | 0.124 | 0.088 | 0.072 | 0.062 | 0.056 |
| Benton Harbor, MI | 1 | 0.465 | 6 | 0.149 | 0.106 | 0.086 | 0.075 | 0.067 |
| Philo, IL | 1 | 0.199 | 5 | 0.044 | 0.031 | 0.025 | 0.022 | 0.020 |
| Recommended Target Within-Lot Voids per mm Std. Dev. | | | | 0.091 | 0.064 | 0.052 | 0.045 | 0.041 |

Table 59. Summary of within-lot hardened concrete air void system parameter standard deviations (linear traverse testing data).

| Hardened Air Void System Parameter | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample | | | | |
|---|---|---|---|---|---|
| | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Entrained Air Content, % | 1.90 | 1.34 | 1.10 | 0.95 | 0.85 |
| Spacing Factor, mm | 0.032 | 0.022 | 0.018 | 0.016 | 0.014 |
| Specific Surface Area, $mm^2/mm^3$ | 5.66 | 4.00 | 3.27 | 2.83 | 2.53 |
| Voids per mm | 0.091 | 0.064 | 0.052 | 0.045 | 0.041 |

*Comparison of Entrained Air Content Tests Taken Before and After the Paver*

A particularly interesting result of the entrained air content variation research is the relationship between plastic entrained air content measurements taken before and after the paver. The plastic entrained air content was measured before and after the paver at 50 sampling locations. The results indicate that, on average, the consolidation and finishing action of the paver reduces the entrained air content of the plastic concrete by approximately 0.7 percent when measured using an air pressure meter (see figure 69).

*Plastic versus Hardened Entrained Air Content Relationship*

Another interesting result of the entrained air content variability research involved the relationship between plastic and hardened entrained air contents of material taken from approximately the same location. Cores were removed for linear traverse analyses from 26 locations where the plastic concrete air content was measured before the paver. Unfortunately, the plastic concrete was only sampled after the paver from 16 of these locations. The collected data are plotted in figure 70.

According to this data, measuring the entrained air content by linear traverse yields a measurably higher value than that obtained using a plastic concrete pressure meter. On average, the difference for the projects evaluated was 2.4 and 1.4 percent greater than plastic concrete sampled before and after the paver, respectively. This result agrees with previous research which concluded that the hardened entrained air content was 2.0 percent higher than the plastic entrained air content.[61] Mielenz attributed the difference to the dissolution of air from small bubbles and its release into larger bubbles. Various hypotheses have attempted to explain this observation, including an inability of the pressure meter to measure very small entrained air bubbles, addition of retempering water after plastic air tests have been completed, and the use of particularly sensitive air-entraining admixtures. However, Nagi and Whiting performed extensive testing and concluded that retempering and admixture type have little effect on the discrepancy between plastic and hardened air contents.[25] In a paper

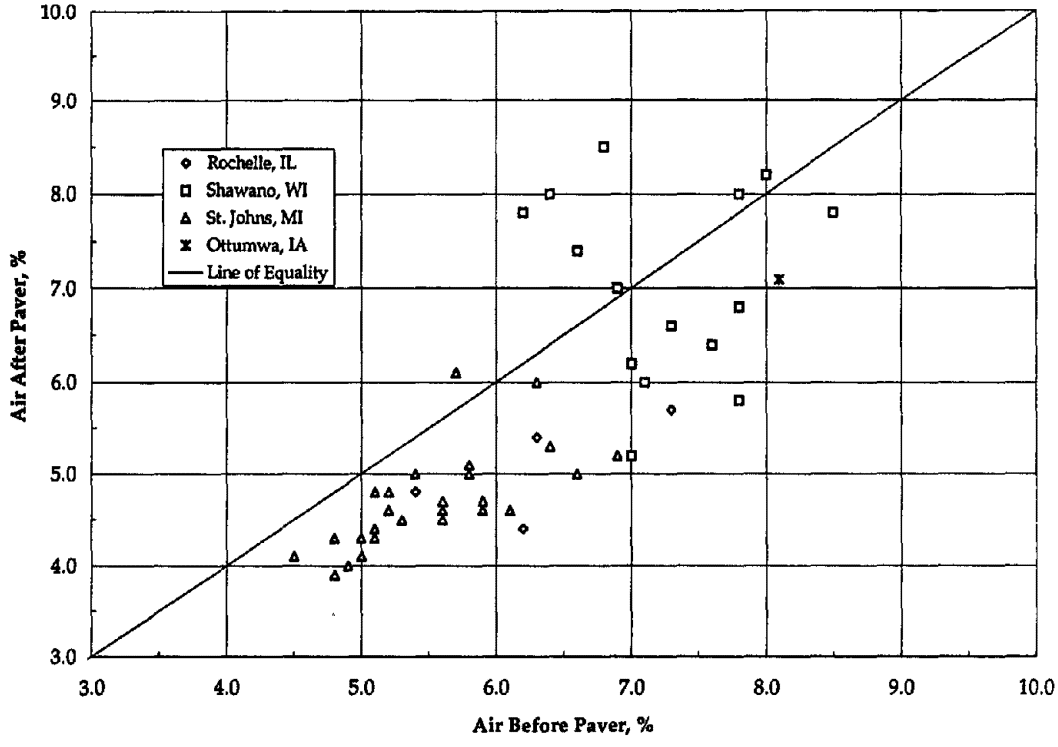



Figure 69. Comparison of plastic entrained air content sampled before and after the paver.

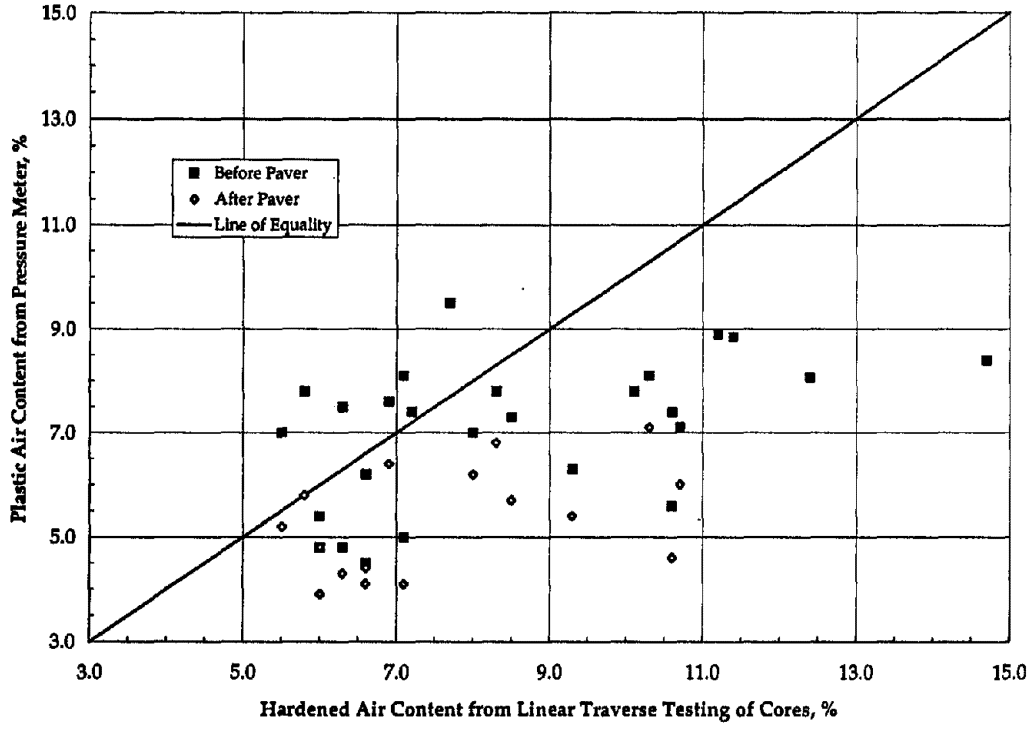



Figure 70. Plastic entrained air content versus hardened entrained air content (measured from linear traverse testing of cores).

published in the *ACI Journal* in 1983, Burg stated that the air content of hardened concrete should be higher than that of fresh concrete.[62] Burg hypothesized that:

> "...air voids determined microscopically include some of the space originally occupied by mixing water and small honeycombs caused by lack of complete consolidation. In addition, the small bubbles have a higher internal pressure and are less compressible in the plastic concrete, indicating a lower than actual pressure meter reading."

## INITIAL SMOOTHNESS

### Historical Initial Smoothness Data

Numerous studies of initial smoothness measuring error and variability were cited in the literature. The construction process, concrete temperatures, curing procedures, subgrade and pavement stiffness, and rainfall can lead to differences in initial pavement smoothness. Variation sources include between-path, between-device, between-operator, within-device (repeatability), and within-operator (repeatability). Under the PRS approach, it is assumed that one device and one operator are used to establish sublot initial smoothness within each wheelpath. The total within-lot variation of interest in a PRS would, therefore, include variation expected from testing different wheelpaths, within-device repeatability, within-operator repeatability, and the actual variation of the pavement smoothness.

One study examined within-device repeatability using four different profilographs and four different operators.[30] Tests were conducted for two different initial smoothness levels. All four devices were repeatable within 24 mm/km and 32 mm/km on the *smoother* and *rougher* wheelpaths, respectively. The replicate root mean square calculated for the smoother wheelpath was 8 mm/km; and for the rougher wheelpath, it was 10 mm/km.

Initial smoothness of concrete pavements is generally assessed using a California-type profilograph. Because of its widespread use and acceptance, the California-type profilograph is recommended for use in the PRS. However, it is important to recognize that there are both manual and automated versions of the California-type profilograph, and the characteristics and variability of each are different.

The within-test variability of noncomputerized (manual) profilographs is a function of the pavement smoothness. Variability increases as smoothness decreases. This increase is attributed to increased subjectivity associated with more active output traces. For new construction, the standard deviation of noncomputerized profilographs generally ranges from about 8 mm/km to about 32 mm/km. An approximate rule of thumb is that the variability is generally about 1/10 of the recorded profile index. For example, if the profile index is 158 mm/km, the corresponding standard deviation is 16 mm/km. Within-operator variability associated with manual trace interpretation and

reduction was also determined. Eight traces were interpreted two times by four different operators. The replicate root mean square calculated for the four operators was 21 mm/km.

The variability of computerized (automated) profilographs does not vary as much with pavement smoothness, presumably because of the more uniform and consistent application of interpretation rules. Generally, the variability of the initial smoothness from computerized systems is between 16 and 32 mm/km.

The variation of interest for the PRS is the within-lot standard deviation of initial pavement smoothness measurements (profile indices). As with the other AQC's studied, this total variability includes within-test (device and operator repeatability) and materials/process (actual deviations in smoothness) variabilities. The overall variability for use in the PRS was calculated as the standard deviation of all representative sample smoothness values recorded within the lot. If replicate measurements are collected (repeated profilometer trace location, such as a wheelpath), the replicate values are averaged into a representative sample value.

### Additional Initial Smoothness Data Used to Investigate the Typical Initial Smoothness Within-Lot Standard Deviation

Additional initial smoothness data (measured with a California profilograph) were obtained for all of the PRS shadow field trial projects, as well as those archived projects used in the comparison of actual to PRS pay adjustments (see chapter 5 of volume II). Data were also included from one additional project used in a recent NCHRP study on pavement smoothness.[63] Data were collected for a total of 11 projects made up of 52 separate lots. The data for 6 projects (33 lots) were reduced using a 5.1-mm blanking band, while the remaining 4 projects (19 lots) were reduced using a 0.0-mm blanking band. The data are analyzed separately for each blanking band setting.

### Summary of the Initial Smoothness Variation Results

A summary of the collected initial smoothness data reduced using a 5.1-mm blanking band is presented in table 60; table 61 contains a summary of the collected initial smoothness data reduced using a 0.0-mm blanking band. Plots showing the distribution of computed within-lot standard deviations for the data reduced using both blanking bands are presented in figures 71 and 72. The recommended average target within-lot standard deviation (for initial profile indices reduced using a 5.1-mm blanking band) was computed to be 27.1 mm/km, while the recommended value for the 0.0-mm blanking band data was 49.9 mm/km.

Table 60. Summary of measured within-lot initial smoothness statistics for profile indices (California profilograph) reduced using a 5.1-mm blanking band.

| Project | Lot Number | Number of Samples | Profile Index Mean, mm/km | Profile Index Standard Deviation, mm/km | Coefficient of Variation, % |
|---|---|---|---|---|---|
| 1 | 1 | 120 | 10.1 | 24.2 | 2.39 |
| | 2 | 39 | 11.7 | 14.6 | 1.24 |
| | 3 | 10 | 20.5 | 25.3 | 1.23 |
| | 4 | 14 | 13.5 | 23.7 | 1.75 |
| | 5 | 11 | 12.7 | 17.7 | 1.40 |
| | 6 | 10 | 28.3 | 28.9 | 1.02 |
| | 7 | 11 | 38.7 | 68.3 | 1.76 |
| | 8 | 13 | 10.9 | 13.9 | 1.27 |
| 2 | 1 | 12 | 43.1 | 26.3 | 0.61 |
| | 2 | 16 | 69.6 | 37.1 | 0.53 |
| | 3 | 16 | 99.6 | 55.6 | 0.56 |
| 3 | 1 | 4 | 82.7 | 19.9 | 0.24 |
| | 2 | 4 | 136.1 | 14.7 | 0.11 |
| | 3 | 4 | 56.1 | 1.6 | 0.03 |
| | 4 | 4 | 56.8 | 12.1 | 0.21 |
| | 5 | 4 | 39.6 | 6.2 | 0.16 |
| 4 | 1 | 4 | 50.4 | 33.9 | 0.67 |
| | 2 | 4 | 53.9 | 20.5 | 0.38 |
| | 3 | 4 | 28.5 | 14.0 | 0.49 |
| | 4 | 4 | 40.1 | 5.8 | 0.14 |
| | 5 | 4 | 76.2 | 17.3 | 0.23 |
| | 6 | 4 | 36.8 | 14.9 | 0.41 |
| | 7 | 4 | 33.0 | 10.9 | 0.33 |
| | 8 | 4 | 54.1 | 23.3 | 0.43 |
| 5 | 1 | 4 | 71.6 | 23.0 | 0.32 |
| | 2 | 8 | 57.5 | 32.2 | 0.56 |
| | 3 | 4 | 30.7 | 14.8 | 0.48 |
| | 4 | 18 | 64.4 | 64.9 | 1.01 |
| 6 | 1 | 16 | 39.0 | 59.2 | 1.52 |
| | 2 | 12 | 48.0 | 18.3 | 0.38 |
| | 3 | 12 | 115.1 | 105.7 | 0.92 |
| | 4 | 20 | 20.5 | 22.2 | 1.08 |
| | 5 | 8 | 40.4 | 24.8 | 0.61 |
| Recommended Target Within-Lot Statistics | | | | 27.1 | 0.74 |

Table 61. Summary of measured within-lot initial smoothness statistics for profile indices (California profilograph) reduced using a 0.0-mm blanking band.

| Project | Lot Number | Number of Samples | Profile Index Mean, mm/km | Profile Index Standard Deviation, mm/km | Coefficient of Variation, % |
|---|---|---|---|---|---|
| 7 | 1 | 120 | 354.6 | 45.1 | 0.13 |
| | 2 | 39 | 55.4 | 11.8 | 0.21 |
| | 3 | 10 | 91.4 | 19.4 | 0.21 |
| 8 | 1 | 14 | 195.0 | 40.9 | 0.21 |
| | 2 | 11 | 158.0 | 69.4 | 0.44 |
| | 3 | 10 | 182.7 | 34.3 | 0.19 |
| | 4 | 11 | 179.4 | 28.2 | 0.16 |
| | 5 | 13 | 194.8 | 67.0 | 0.34 |
| | 6 | 12 | 217.0 | 32.9 | 0.15 |
| | 7 | 16 | 217.8 | 47.1 | 0.22 |
| 9 | 1 | 16 | 233.1 | 36.7 | 0.16 |
| | 2 | 4 | 195.8 | 56.0 | 0.29 |
| | 3 | 4 | 265.1 | 53.7 | 0.20 |
| | 4 | 4 | 144.7 | 59.9 | 0.41 |
| | 5 | 4 | 170.5 | 54.9 | 0.32 |
| 10 | 1 | 4 | 211.5 | 26.0 | 0.12 |
| | 2 | 4 | 186.2 | 51.9 | 0.28 |
| 11 | 1 | 4 | 454.0 | 116.8 | 0.26 |
| | 2 | 4 | 380.0 | 95.9 | 0.25 |
| Recommended Target Within-Lot Statistics | | | | 49.9 | 0.24 |

## SAWCUT DEPTH

### Historical Sawcut Depth Data

A recent study sponsored by the Ontario Ministry of Transportation measured the depth of sawing on 40 cores taken across the longitudinal joint.[34] Sawcut depth ranged from 33 to 67 mm, with a standard deviation of 9 mm. The average and specified sawcut depths were 54 mm and 66 mm, respectively.

Sawcut depth data for one project using an abrasive sawblade were also reported in a recent FHWA study of sawcutting.[35] Sawcut depth for 22 transverse joints averaged 91 mm, with a standard deviation of 8 mm. For the 22 longitudinal joints measured, sawcut depth averaged 94 mm, with a standard deviation of 6 mm.

Factors influencing sawcut depth variability include blade type and the operator's experience. In general, two types of sawblades are used to cut joints in PCC pavements—abrasive and diamond-impregnated. Abrasive blades tend to wear as the

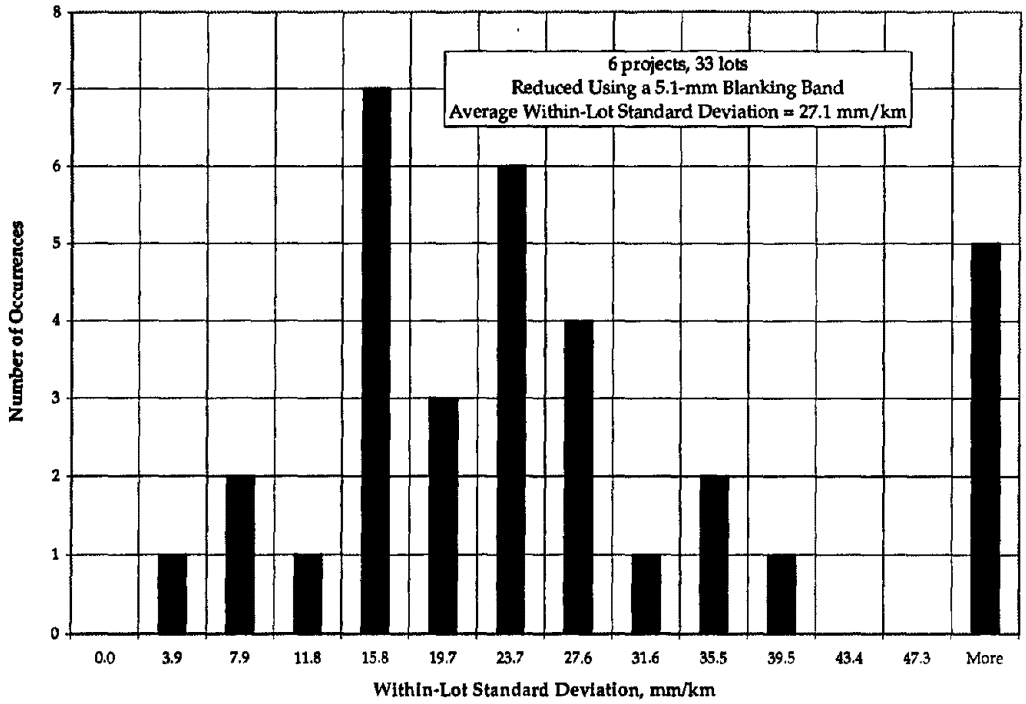



Figure 71. Histogram of the measured within-lot initial smoothness standard deviations for profile indices reduced using a 5.1-mm blanking band.

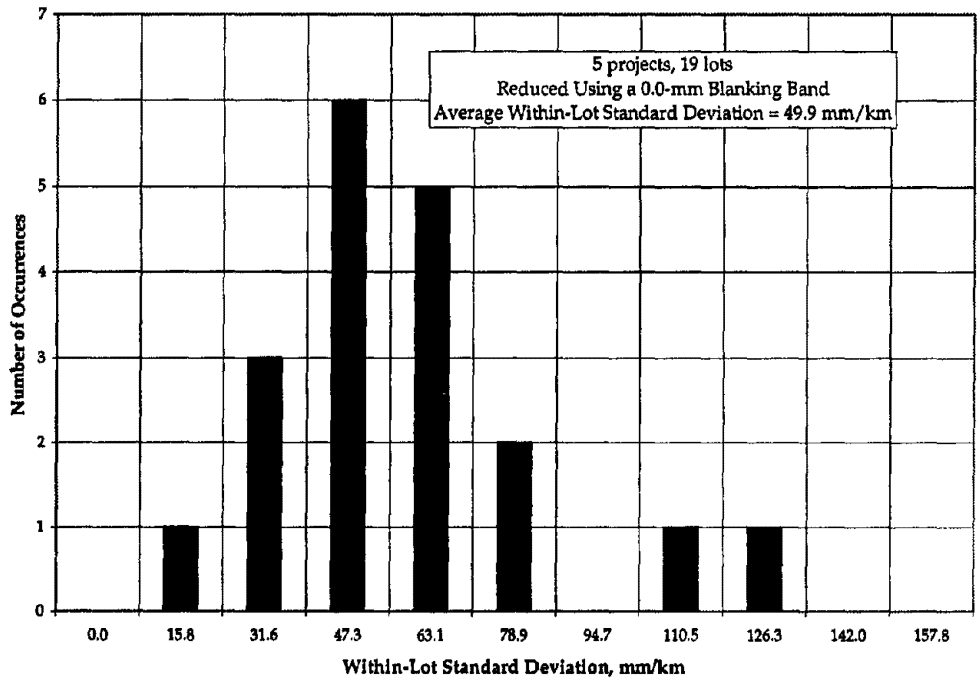



Figure 72. Histogram of the measured within-lot initial smoothness standard deviations for profile indices reduced using a 0.0-mm blanking band.

saw progresses along the pavement. An experienced operator will pay attention to the wearing of the blade and make adjustments in the saw as necessary. If the operator lacks experience, the blade will gradually wear and the joint depth becomes shallower as the saw progresses along the pavement. Diamond-impregnated blades don't wear as the abrasive blades do; however, operator inexperience may affect the initially established blade depth, or allow the saw to rise, as opposed to cutting the full depth intended. Sawcuts deeper than the target depth may result in loss of load transfer at nondoweled transverse joints. It is assumed that sawcut depth does not significantly affect the performance of doweled pavements, since the deflection-reducing benefits of dowel bars are much greater than the contribution from aggregate interlock.

### Field Investigation of the Typical Sawcut Depth Within-Lot Standard Deviation

Sawcut depths were recorded on four new construction projects, as shown in table 45. The data from the Ontario study were also included in this investigation.[34] At the new construction projects, the joint depth was measured directly using a thin steel rule. The within-lot sawcut variation was evaluated for both transverse and longitudinal joints. For transverse joints, the sawcut depth was measured along the joint at 0.30-m intervals and between joints as measured at the outside edge, mid-truck lane, pavement centerline, mid-pass lane, interior edge, and interior wheelpaths. To determine the longitudinal joint depth variation, readings were made approximately 0.30 m from every transverse joint in the lot. For some projects, additional joint depths were measured at the midpoint between transverse joints.

### Summary of Sawcut Depth Variation Results

The collected sawcut depth data are summarized in table 62. All the joints were cut using diamond-impregnated sawblades except for the Ottumwa, IA, transverse joints. The average within-lot transverse joint sawcut depth standard deviation measured 6 mm, but the Ottumwa within-lot transverse joint depth standard deviation averaged 8 mm, and the longitudinal joint depth standard deviation for Ottumwa was 6 mm. A relative increase in standard deviation is likely to occur for those projects in which joints are cut using abrasive sawblades. For both transverse and longitudinal joint sawcut depths, recommended typical standard deviations were computed assuming that the within-lot standard deviations for each lot studied were normally distributed.

## PERCENT CONSOLIDATION AROUND DOWELS

Very little data were found in the literature regarding the within-lot variation of percent consolidation around dowels. In the previous PRS study, reduction factors of 94, 97, and 100 percent were derived to apply to 28-day flexural strength predictions from early age in situ cores.[3] The research team suggested that the performance of doweled joints is dependent on the consolidation level around the dowel bars. It is possible the dowel basket assemblies interfere with the consolidation of the concrete below dowel bars, where the bearing stress will be affected. To incorporate an

Table 62. Summary of within-lot sawcut joint depth standard deviations.

| Joint Type | Project | Lot | Lot Average Joint Depth, mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, mm | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Transverse | Rochelle, IL | 1 | 375 | 1 | 5.8 | 4.1 | 3.4 | 2.9 | 2.6 |
| | Shawano, WI | 1 | 218 | 1 | 3.9 | 2.8 | 2.3 | 2.0 | 1.8 |
| | St. Johns, MI | 1 | 402 | 1 | 3.3 | 2.3 | 1.9 | 1.6 | 1.5 |
| | Ottumwa, IA | 1 | 52 | 2 | 9.8 | 7.0 | 5.7 | 4.9 | 4.4 |
| | | 2 | 75 | 2 | 7.7 | 5.4 | 4.4 | 3.8 | 3.4 |
| | | 3 | 30 | 2 | 5.7 | 4.0 | 3.3 | 2.9 | 2.6 |
| | Ontario, Canada | 1 | 18 | 1 | 7.7 | 5.4 | 4.4 | 3.8 | 3.4 |
| Recommended Target Transverse Joint Depth Standard Deviation | | | | | 6.3 | 4.4 | 3.6 | 3.1 | 2.8 |
| Longitudinal | Shawano, WI | 1 | 112 | 1 | 3.9 | 2.8 | 2.3 | 2.0 | 1.8 |
| | St. Johns, MI | 1 | 21 | 1 | 2.3 | 1.6 | 1.3 | 1.2 | 1.0 |
| | Ottumwa, IA | 1 | 52 | 1 | 7.4 | 5.2 | 4.2 | 3.7 | 3.3 |
| | | 2 | 75 | 1 | 4.9 | 3.5 | 2.8 | 2.5 | 2.2 |
| | | 3 | 30 | 1 | 5.2 | 3.7 | 3.0 | 2.6 | 2.3 |
| | Ontario, Canada | 1 | 26 | 1 | 8.9 | 6.3 | 3.6 | 1.8 | 0.8 |
| Recommended Target Longitudinal Joint Depth Standard Deviation | | | | | 5.4 | 3.8 | 2.9 | 2.3 | 1.9 |

alteration into the current joint faulting model to account for the consolidation around dowel bars, the variation of consolidation level at doweled joints was evaluated at three doweled joint patching projects (Benton Harbor, MI; Des Plaines, IL; Philo, IL) and one new construction project (Ottumwa, IA).

### Field Investigation of the Percent Consolidation Around Dowels Within-Lot Standard Deviation

To evaluate in situ consolidation level variability, core densities were measured. For each project, several joints exhibiting varying degrees of deterioration were selected as part of the study. Cores were drilled through dowel bars, between dowel bars, and away from the joint to evaluate relative densities. Cores drilled through dowel bars were separated into concrete samples from above and below the dowel. To account for a portion of the permeable voids, sample densities were measured in a procedure similar to ASTM C 642, *Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete.* The weight of the "as-received" sample immersed in water was recorded quickly so as not to allow water to penetrate all permeable voids on the sides of the core sample. The sample was then weighed in wet and oven-dried states. Equation 13 was used to determine the density of the concrete samples:

$$\gamma = A/(B - C) * 997.47 \quad (13)$$

where

$\gamma$ = Density of the concrete sample, kg/m$^3$.
A = Weight of oven-dried sample in air, g.
B = Weight of surface-dried sample in air after quick immersion, g.
C = Weight of sample immersed in water, g.

The consolidation of concrete removed from around dowel basket assemblies (above, below, and between dowel bars) was determined as the ratio of the density of the concrete sample to a fully consolidated sample. From all the cores removed away from joints within the lot being evaluated (including cores for thickness, concrete strength, and entrained air content), the core with the highest density was assumed to be 100-percent consolidated. The doweled joint core densities (above, below, and between dowels) were then compared to the 100-percent consolidated density to determine the relative consolidation levels. This procedure incorporates over-consolidation effects when vibration time is extended over basket assemblies or when vibrators contact dowels for longer-than-normal periods.

### Summary of Percent Consolidation Around Dowels Variation Results

The percent consolidation around dowels is summarized in table 63. As shown for the lots included in this investigation, the within-lot standard deviations are generally low, ranging between 0.61 and 2.07 percent. Table 63 also presents recommended target consolidation levels for below, above, and between dowels. A model was developed

Table 63. Summary of within-lot percent consolidation around dowels standard deviations.

| Sample Type | Project | Lot | Lot Average Percent Consolidation, % | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, % | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Below Dowels | Benton Harbor, MI | 1 | 98.0 | 17 | 1.21 | 0.85 | 0.70 | 0.60 | 0.54 |
| | Des Plaines, IL | 1 | 101.2 | 21 | 1.12 | 0.79 | 0.65 | 0.56 | 0.50 |
| | Philo, IL | 1 | 99.7 | 10 | 1.38 | 0.98 | 0.80 | 0.69 | 0.62 |
| | Ottumwa, IA | 1 | 99.4 | 3 | 0.61 | 0.43 | 0.35 | 0.30 | 0.27 |
| | | 2 | 100.3 | 4 | 0.66 | 0.47 | 0.38 | 0.33 | 0.30 |
| | | 3 | 101.4 | 4 | 1.36 | 0.96 | 0.78 | 0.68 | 0.61 |
| Recommended Target Standard Deviation (Below Dowels) | | | | | 1.06 | 0.75 | 0.61 | 0.53 | 0.47 |
| Above Dowels | Benton Harbor, MI | 1 | 98.5 | 17 | 1.03 | 0.73 | 0.59 | 0.51 | 0.46 |
| | Des Plaines, IL | 1 | 100.1 | 22 | 1.08 | 0.77 | 0.63 | 0.54 | 0.48 |
| | Philo, IL | 1 | 98.3 | 10 | 2.07 | 1.46 | 1.19 | 1.03 | 0.93 |
| | Ottumwa, IA | 1 | 99.6 | 3 | 0.91 | 0.64 | 0.52 | 0.45 | 0.41 |
| | | 2 | 100.4 | 4 | 1.26 | 0.89 | 0.73 | 0.63 | 0.57 |
| | | 3 | 100.1 | 4 | 0.62 | 0.44 | 0.36 | 0.31 | 0.28 |
| Recommended Target Standard Deviation (Above Dowels) | | | | | 1.16 | 0.82 | 0.67 | 0.58 | 0.52 |
| Between Dowels | Benton Harbor, MI | 1 | 98.5 | 8 | 1.15 | 0.82 | 0.67 | 0.58 | 0.52 |
| | Des Plaines, IL | 1 | 100.5 | 11 | 1.10 | 0.78 | 0.64 | 0.55 | 0.49 |
| | Philo, IL | 1 | 99.2 | 5 | 1.15 | 0.81 | 0.66 | 0.58 | 0.51 |
| Recommended Target Standard Deviation (Between Dowels) | | | | | 1.13 | 0.80 | 0.66 | 0.57 | 0.51 |

that relates the consolidation of concrete below dowel bars to the percentage of load transfer. While the data correlate well, future studies on consolidation should measure the concrete density by coating the specimen with paraffin prior to immersion in water. This step will increase the accuracy of the density measurement by accounting for the permeable voids in the concrete sample.

## DEVIATION OF TIE BAR PLACEMENT FROM SLAB MID-DEPTH

No information on tie bar depth variability was found in the literature review. Tie bars placed too close to the pavement surface can lead to longitudinal joint spalling from load or corrosion. Tie bar placement problems can also influence the stress transfer effectiveness of tied lanes and concrete shoulders, reducing flexural stresses and increasing fatigue life. No performance models incorporating the effects of tie bar depth or alignment magnitudes of load transfer were identified in the literature survey. The research team proposed evaluation of tie bar depth on multi-lane projects using GPR to determine whether a longitudinal joint spalling model incorporating the effects of tie bar depth could be developed. Other factors, such as load transfer, tie bar size and spacing, deicer chemicals, lane width, climate, subbase, and subgrade were proposed to be included in the model.

The research team also theorized that tie bars placed too high or too low result in a loss of load transfer along the longitudinal joint. A loss of load transfer across the longitudinal joint would cause an increase in the edge stress within the pavement and increased edge stress would directly affect the development of transverse fatigue cracking. The effects of loss of load transfer on the resultant edge stress could then be directly incorporated into the existing transverse cracking (fatigue) distress indicator model. To determine whether it was feasible to develop the relationship, the variation of tie bar depth deviation (from slab mid-depth) was first evaluated.

### Field Investigation of the Within-Lot Standard Deviation of the Deviation of Tie Bar Placement From Slab Mid-Depth

The performance of tied shoulders varies depending on whether shoulders are constructed as part of the mainline pavement, constructed separately from the mainline pavement, or added as a tied concrete shoulder retrofit. While differences are recognized, the research team proposed evaluation of new pavement construction where tie bars were installed by the paver as part of the mainline section. This method of constructing tied joints should result in the greatest variability, as the tie bar inserter on the paver may release the bars at various depths. Other methods of constructing tied joints, such as inserting bent tie bars in the edge of slip-formed pavement or drilling and installing tie bars for retrofit projects, generally utilize equipment or procedures that place the tie bars at a constant depth along the mainline pavement edge.

Two projects (Ottumwa, IA, and Omaha, NE) were evaluated to investigate the variation in deviation of tie bar placement from slab mid-depth. Load transfer was also

measured at one of these projects (Omaha, NE) to determine whether it was feasible to continue with the development of a model incorporating the effect of tie bar depth on the load transfer efficiency of the longitudinal joint. Using GPR, several sections within the project limits were identified with a range of tie bar depths. Stress transfer efficiency was measured directly using a falling-weight deflectometer (FWD), and the effective stress transfer efficiency was computed.

**Summary of the Deviation of Tie Bar Placement From Slab Mid-Depth Variation Results**

The statistical information reduced for the evaluated lots is presented in table 64. The measured within-lot variation shown in the table was calculated as the standard deviation of all tie bar depths (deviation from mid-depth) recorded for each lot. Each tie bar depth-deviation measurement was considered a sample. No replicate measurements were recorded. The average within-lot standard deviation of the deviation of tie bars from slab mid-depth (for all of the investigated lots) was 9 mm. The standard deviation of tie bar depth-deviation was significantly larger for the Ottumwa, IA, project (15 mm) than for the Omaha, NE, project (4 mm). Still, an average standard deviation of 15 mm does not represent poor control by the contractor. At the Ottumwa project, it was very rare to find tie bars placed more than 25 mm from the mid-depth of the 279-mm-thick pavement. Also, while the standard deviation of the tie bar depth deviations was small at the Omaha project, the average overall depth of tie bar placement was 110 mm for the 356-mm-thick pavement. Therefore, for the Omaha project, the tie bars were consistently placed nearly 76 mm above the theoretical slab mid-depth.

Table 64. Summary of within-lot tie bar deviation from mid-depth standard deviations.

| Project | Lot | Lot Average Deviation from Mid-Depth, mm | Number of Samples | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample, mm | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Ottumwa, IA | 1 | -14.6 | 672 | 14.6 | 10.3 | 8.4 | 7.3 | 6.5 |
| | 2 | -4.0 | 1064 | 18.1 | 12.8 | 10.4 | 9.0 | 8.1 |
| | 3 | 7.4 | 968 | 11.9 | 8.4 | 6.9 | 5.9 | 5.3 |
| Omaha, NE | 1 | 71.1 | 244 | 3.9 | 2.8 | 2.2 | 1.9 | 1.7 |
| | 2 | 69.2 | 255 | 3.4 | 2.4 | 1.9 | 1.7 | 1.5 |
| | 3 | 67.2 | 262 | 6.0 | 4.2 | 3.5 | 3.0 | 2.7 |
| | 4 | 62.7 | 244 | 3.7 | 2.6 | 2.1 | 1.8 | 1.6 |
| Recommended Target Standard Deviation | | | | 8.8 | 6.2 | 5.1 | 4.4 | 3.9 |

The recommended tie bar depth-deviation standard deviation was calculated in the same manner as the previously discussed AQC's. The measured tie bar depth-

deviation samples were assumed to be normally distributed within each investigated lot. The recommended average standard deviation for the case without replicates (n = 1) was found to be 8.8 mm (as shown in table 64).

## ACCEPTANCE QUALITY CHARACTERISTIC VARIABILITY SUMMARY

The target standard deviations for pavement AQC's currently achieved in the field were presented in this appendix. Much of the data used in determining recommended typical AQC within-lot standard deviations was collected from 11 projects (new construction and transverse joint patching projects) during the 1995 and 1996 construction seasons. An overall summary of the recommended target within-lot standard deviations for each investigated AQC (including computed values representing different numbers of replicate samples) is presented in table 65.

Table 65. Summary of target within-lot standard deviations for PCC pavement acceptance quality characteristics.

| Acceptance Quality Characteristic | Sample Type | Within-Lot Standard Deviations for Different Numbers of Replicates (n) per Sample | | | | |
|---|---|---|---|---|---|---|
| | | n = 1 | n = 2 | n = 3 | n = 4 | n = 5 |
| Compressive Strength, MPa | 3-day Cylinder | 3.13 | 2.21 | 1.81 | 1.56 | 1.40 |
| | 3-day Core | 2.10 | 1.48 | 1.21 | 1.05 | 0.94 |
| | 14-day Cylinder | 3.37 | 2.38 | 1.95 | 1.69 | 1.51 |
| | 28-day Cylinder | 3.10 | 2.19 | 1.79 | 1.55 | 1.39 |
| | 28-day Core | 2.63 | 1.86 | 1.52 | 1.31 | 1.18 |
| Splitting Tensile Strength, MPa | 3-day Cylinder | 0.55 | 0.39 | 0.32 | 0.28 | 0.25 |
| Flexural Strength, MPa | 3-day Beam | 0.41 | 0.29 | 0.24 | 0.20 | 0.18 |
| | 14-day Beam | 0.43 | 0.30 | 0.25 | 0.21 | 0.19 |
| | 28-day Beam | 0.30 | 0.21 | 0.18 | 0.15 | 0.14 |
| | Predicted 28-day | 0.21 | 0.15 | 0.12 | 0.10 | 0.09 |
| Slab Thickness, mm | Cores | 8.0 | 5.7 | 4.6 | 4.0 | 3.6 |
| | Radar | 9.5 | 6.7 | 5.5 | 4.7 | 4.2 |
| Entrained Air Content, % | Before Paver | 0.75 | 0.53 | 0.43 | 0.37 | 0.33 |
| | After Paver | 0.78 | 0.55 | 0.45 | 0.39 | 0.35 |
| | Linear Traverse | 1.90 | 1.34 | 1.10 | 0.95 | 0.85 |
| Initial Smoothness, mm/km | Profile Index—5.1-mm blanking band | 27.1 | 19.2 | 15.6 | 13.6 | 12.1 |
| | Profile Index—0.0-mm blanking band | 49.9 | 35.3 | 28.8 | 25.0 | 22.3 |
| Joint Sawcut Depth, mm | Transverse | 6.3 | 4.4 | 3.6 | 3.1 | 2.8 |
| | Longitudinal | 5.4 | 3.8 | 2.9 | 2.3 | 1.9 |
| Percent Consolidation Around Dowels, % | Below Dowels | 1.06 | 0.75 | 0.61 | 0.53 | 0.47 |
| | Above Dowels | 1.16 | 0.82 | 0.67 | 0.58 | 0.52 |
| | Between Dowels | 1.13 | 0.80 | 0.65 | 0.57 | 0.51 |
| Tie Bar Deviation from Mid-Depth, mm | Ruler | 8.8 | 6.2 | 5.1 | 4.4 | 3.9 |

# APPENDIX E—DISTRESS INDICATOR PREDICTION MODELS

## INTRODUCTION

The validity and defensibility of a performance-related specification (PRS) depends on the accurate identification and prediction of pavement performance. Pavement performance is described in terms of distress indicators (i.e., transverse cracking, transverse joint faulting, transverse joint spalling, and pavement smoothness). The implementation of a PRS requires considerable knowledge of how these distress indicators develop over time, as well as the construction and material acceptance quality characteristics (AQC's) that influence them. One goal of this research study was to establish new (or improve existing) distress indicator prediction models associated with the AQC's to be included in the PRS for jointed plain concrete pavements (JPCP). Each of these prediction models must be associated with one or more AQC's that are measurable and under the control of the contractor.

A comprehensive literature search was conducted to identify previous studies in which the effects of concrete pavement AQC's (construction- and material-related) on pavement performance were documented. The information was evaluated so that the present investigation could target those AQC's for which the effect on pavement performance had not been well-documented. To establish additional information on the relationship of quality level to pavement performance, a field testing program was developed for the identified AQC's. A comprehensive work plan was developed and carried out during this study.

A discussion of the sampling and testing procedures, as well as a review of the results of the field/laboratory investigations for each of the targeted AQC's, is provided in this appendix.

## IDENTIFICATION OF PROJECTS

In general, data for model development were collected from projects investigated during the variability portion of this research study. A summary of the information collected at each of the projects is shown in table 66.

## SUMMARY OF LABORATORY AND FIELD STUDIES

### Prediction of 28-day Flexural Strength

The prototype PRS recommends using core testing as an indirect measure of in situ flexural strength.[1,2,3] Cores are taken at some reference maturity equivalent (for example, 3 days at 22 °C). The mean compressive or splitting tensile strength of the cores from each sublot is adjusted to a 28-day strength under standard laboratory-cured conditions. This equivalent mean 28-day compressive or splitting strength is then converted to a third-point loading flexural strength using an approved interstrength

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Table 66. Data collected at each project evaluated for the development of new (or calibration of existing) distress indicator models.

| Project | Model/AQC Studied | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Strength Prediction Model | | | | Spalling Model | | Faulting Model | | Cracking Model | |
| | 3-day Cyl. Compressive | 3-day Core Compressive | 3-day Cyl. Splitting Tensile | 3-day Core Splitting Tensile | Entrained Air Content | Joint Edge Spalling | Consolidation Around Dowels | Load Transfer Efficiency | Tie Bar Depth | Load Transfer Efficiency |
| Meade, KS—Meade Municipal Airport | √ | √ | √ | √ | | | | | | |
| Shewano, WI—Route 29 East | √ | √ | √ | √ | | | | | | |
| St. Johns, MI—Route 27 North | √ | √ | √ | √ | | | | | | |
| Ottumwa, IA—Route 23 South | √ | √ | √ | √ | | | | | | |
| Des Plaines, IL—Route 58 West | | | | | √ | √ | √ | √ | | |
| Benton Harbor, MI—I-94 West | | | | | √ | √ | √ | √ | | |
| Philo, IL—Route 130 South | | | | | √ | √ | √ | √ | | |
| Omaha, NE—I-80 East & West | | | | | | | | | √ | √ |

relationship developed (prior to construction) from the specific concrete mixture and project-approved aggregates, cement, and admixtures used during construction. The procedure was roughly outlined in the previous PRS study and evaluated during the prediction variability portion of this investigation, as described in appendix D of this report.[1,2,3] The developed prediction procedure is necessary because the concrete fatigue model used in the PRS is based on 28-day, standard cure flexural strength.

The effects of flexural strength on fatigue life performance are well-documented in the literature. Therefore, while no further field investigations of the effects of flexural strength on pavement performance were performed during the current project, a laboratory study was completed to better explore the errors associated with estimating a 28-day standard cure flexural strength from early age strengths.

Testing Procedure

The previous PRS study compared core and cylinder compressive strength tested at ages of 7, 14, and 28 days for specimens cured under identical curing conditions.[1,2,3] The study concluded that there is no statistical difference between core and cylinder compressive strengths at any of the three investigated maturities. However, the study did not include data from specimens tested at an equivalent 3-day standard laboratory maturity. Therefore, additional laboratory testing was performed to establish whether there is a statistical difference between 102-mm diameter core and 152-mm diameter cylinder compressive and splitting tensile strength at 3 days. Different coarse aggregate types and cements were sampled from each of the four projects used for the strength prediction model study (see table 66). Each coarse aggregate was used with two cement contents to give a total of eight mixes. The concrete mixes were batched in accordance with the American Society for Testing and Materials (ASTM) C 192, *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*. Similar to the previous PRS study, 12 cylinders were cast for each mix and moist-cured for 3 days. Six cylinders were cored just before testing. The core lengths were trimmed to meet the required length-to-diameter ratio of 2 to 1 established by ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete*. Along with this standard, strength testing was performed in the laboratory in accordance with the following ASTM specifications:

- Compressive Strength—ASTM C 39, *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*.
- Splitting Tensile Strength—ASTM C 496, *Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens*.

Compressive strength (three cores and three cylinders) and splitting tensile strength (three cores and three cylinders) were determined on concrete made from the same batch and cured under identical conditions. In addition to the laboratory data collected, results of field samples obtained during the variability investigation were also

summarized. For some of the field projects evaluated, cylinders were cast and cored to obtain a cylinder-to-core relationship. Because curing conditions were identical, the relationship is similar to the lab study. In addition, cores were drilled from the field pavement at some sampling locations. Because the variability of this relationship is slightly increased due to inconsistencies between specific batch constituents, the average cylinder and core strengths were evaluated and presented.

Summary of Strength Results

The results of the cylinder-to-core relationships are plotted in figures 73 and 74 for the compressive and splitting tensile strength data, respectively. On each graph, the best-fit linear equations are also presented. As described in the previous section, relationships were made between cylinders and cored cylinders, cylinders and cored slabs, and overall data sets. All three equations are given in the figures.

In the previous PRS study, it was concluded that there is no significant difference between core and cylinder compressive strengths, irrespective of mix design, coarse aggregate hardness, and coarse aggregate geometry.[1,2,3] Since the data again appear to be clustered around the line of equality (1 to 1), paired t-tests were done to determine if there is a statistically significant difference between matched pairs of core and cylinder compressive strengths. Two-tailed t-tests were performed on data sets of 44 cored cylinders, 44 cored slabs, and 88 overall pairs. In the analyses, the t-values were calculated to be -0.049, -0.222, and -0.099 for the cored cylinders, cored slabs, and overall data, respectively. Therefore, the null hypothesis that there is no statistically significant difference between core and cylinder compressive strengths could not be rejected at the 5-percent significance level for any of the compressive strength data sets.

For completeness, paired t-tests were also performed to confirm that there is a statistically significant difference between matched pairs of core and cylinder splitting tensile strengths. Two-tailed t-tests were performed on data sets of 43 cored cylinders, 36 cored slabs, and 79 overall pairs. In the analyses, the t-values were calculated to be 3.209, 8.631, and 5.569 for the cored cylinders, cored slabs, and overall data, respectively. Therefore, the null hypothesis that there is no statistically significant difference between core and cylinder compressive strengths was rejected at the 5-percent significance level for all of the splitting tensile strength data sets.

Examination of the overall distribution of the compressive strength cylinder-to-core ratios showed an approximately bell-shaped distribution around 1.0. The 88-point relative frequency histogram is shown in figure 75. However, as expected, an examination of the splitting tensile strength cylinder-to-core ratios showed a very skewed curve distributed around 0.75. The 79-point relative frequency histogram is shown in figure 76.

The current transverse joint faulting, transverse joint spalling, and transverse cracking distress indicator models require a 28-day flexural strength. These current

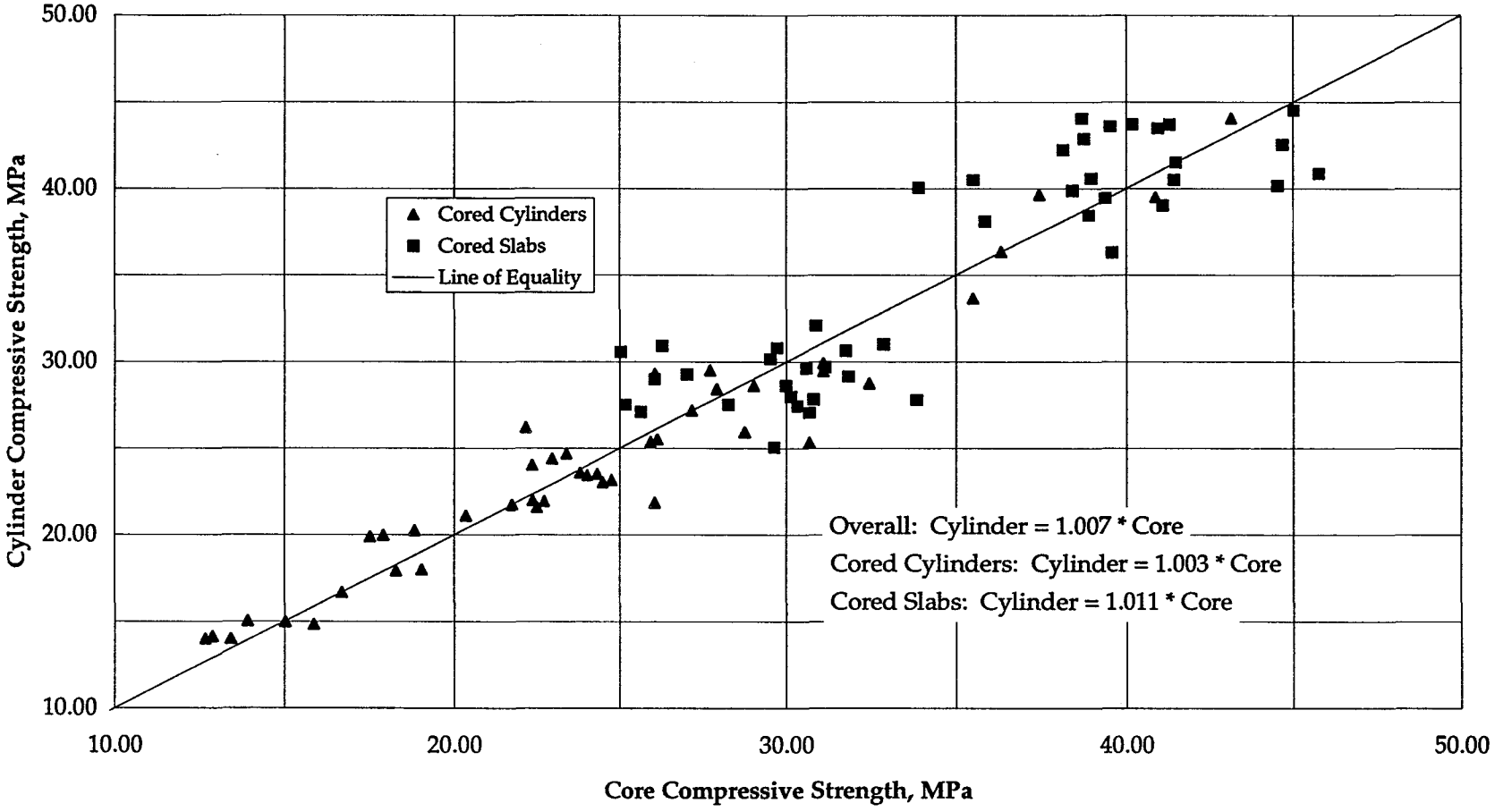



Figure 73. Results of the cylinder compressive strength versus core compressive strength study.

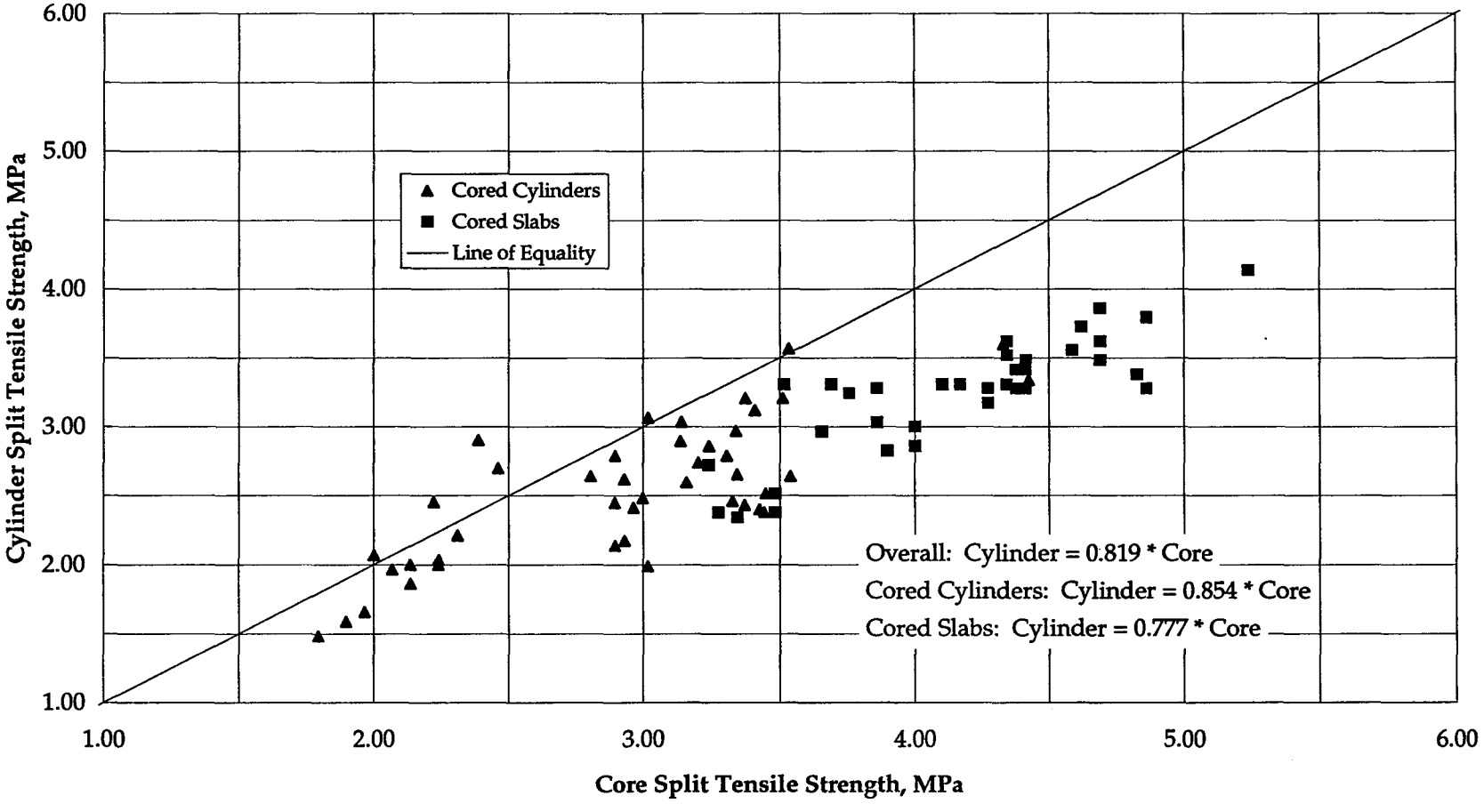



Figure 74. Results of the core split tensile strength versus cylinder split tensile strength study.

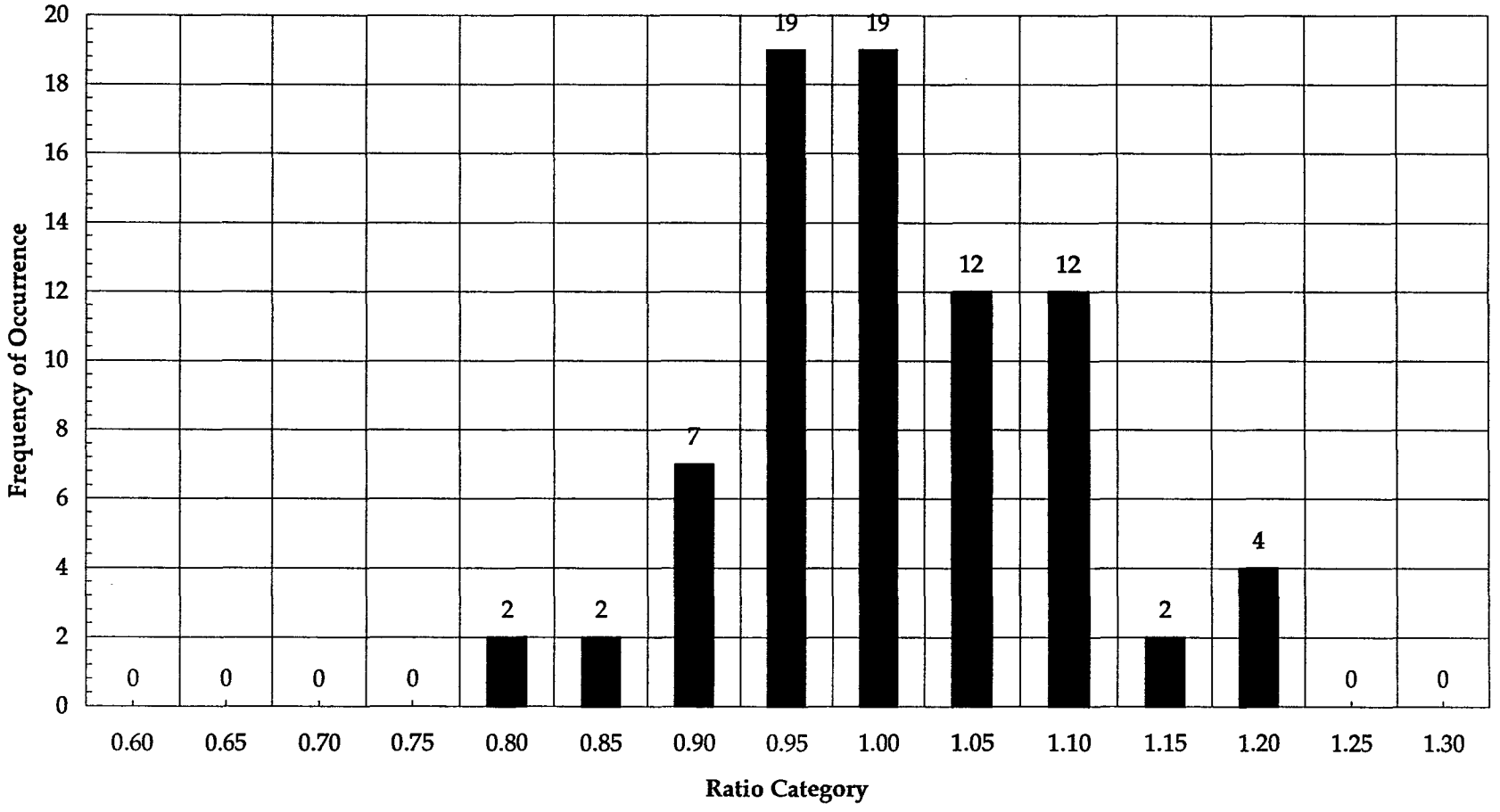



Figure 75. Relative frequency histogram of cylinder-to-core compressive strength ratios.

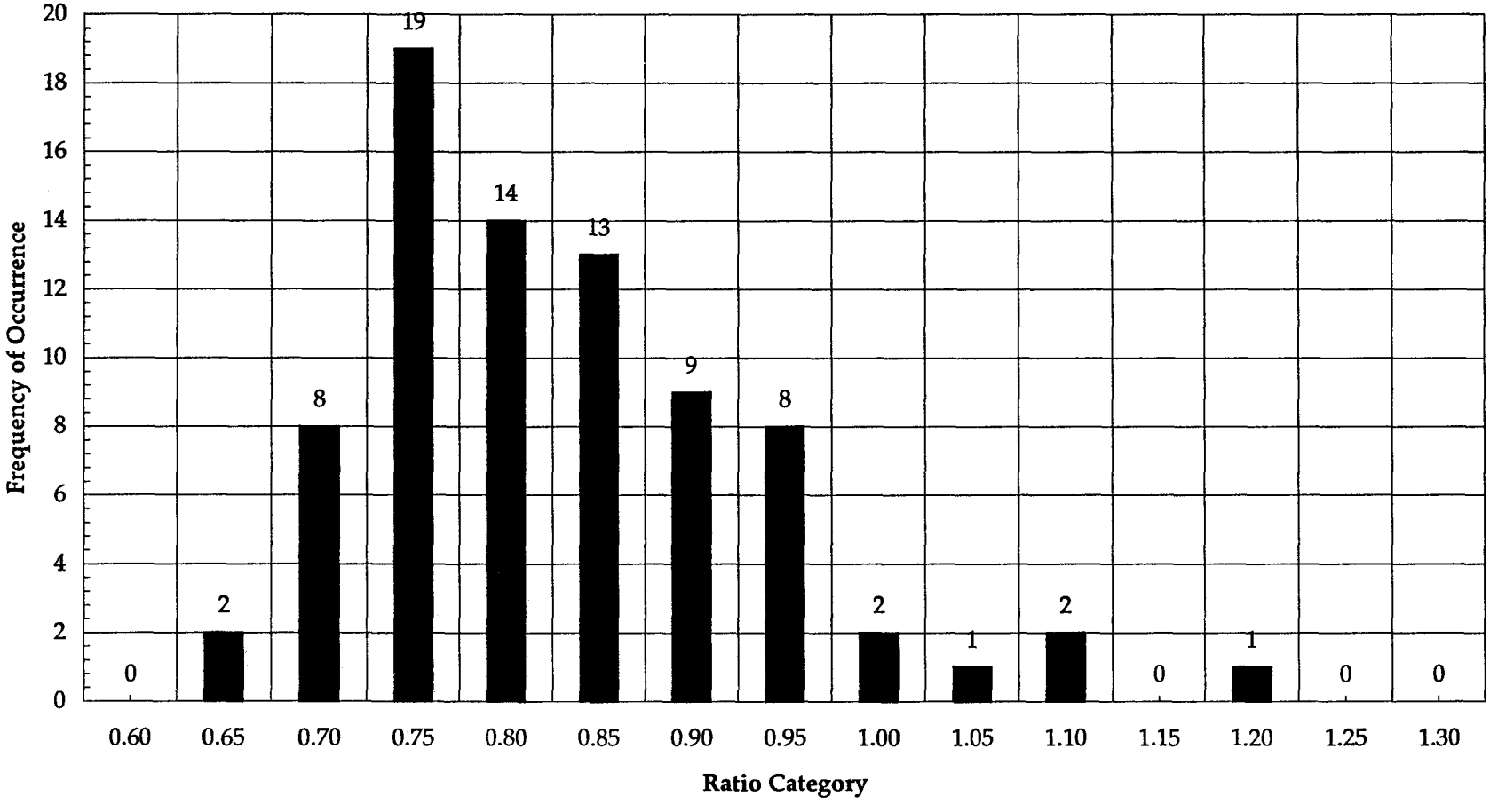



Figure 76. Relative frequency histogram of cylinder-to-core splitting tensile strength ratios.

distress indicator models are described in detail in the section titled *Summary of Current Distress Indicator Models.*

**Effect of Entrained Air Content on Transverse Joint Spalling**

Introduction

The effects of entrained air content on pavement performance were not well-documented in the collected literature. However, under a previous PRS study, a laboratory materials study was conducted to evaluate the effects of air void system parameters on transverse joint scaling/spalling percentages.[1,2,3] That research resulted in three different relationships in which joint spalling was a function of different combinations of entrained air content, void spacing factor, compressive strength, freeze-thaw cycles (at a depth of 76 mm below the pavement surface), and presence of deicing salt. (Note: These relationships and their inputs are presented in English units.) The first model was a function of entrained air content (not void spacing factor), as not all agencies collect hardened air content data. This developed relationship is presented in equation 14.[1,2,3]

$$SPALL = 22.6 + 75.1 * SALT * \log(FTC_{76}) - 78.0 * SALT - 11.7 * AIR * SALT - 0.00478 * f'c \quad (14)$$

where

SPALL = Joint spalling, percent of joint length.
SALT = 0 if no calcium chloride is present, 1 if calcium chloride is present.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.
AIR = Measured air content of the fully consolidated specimen, percent.
f'c = Measured compressive strength mean, psi.

The second model estimated joint spalling as a function of both entrained air content and void spacing factor. This relationship is presented in equation 15.[1,2,3]

$$SPALL = 45.0 + 77.0 * SALT * \log(FTC_{76}) - 29.3 * AIR * SALT - 0.001 * AIR * f'c - 1955 * L * SALT - 0.439 * L * f'c \quad (15)$$

where

SPALL = Joint spalling, percent of joint length.
SALT = 0 if no calcium chloride is present, 1 if calcium chloride is present.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.
AIR = Measured air content of the fully consolidated specimen, percent.
f'c = Measured compressive strength mean, psi.

L = Void spacing factor, 1/in.

The third model estimated joint spalling as a function of void spacing factor and not entrained air content. This relationship is presented in equation 16.[1,2,3]

$$SPALL = 14.1 + 74.9 * SALT * \log(FTC_{76}) - 137.1 * SALT + 1727 * L * SALT - 0.003 * f'c \quad (16)$$

where

SPALL = Joint spalling, percent of joint length.
SALT = 0 if no calcium chloride is present, 1 if calcium chloride is present.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.
L = Void spacing factor, 1/in.
f'c = Measured compressive strength mean, psi.

In order to use these relationships with confidence, they need to be verified under actual field conditions. Under the preceding PRS study, two alternatives were recommended for additional field investigation to accomplish this task.[1,2,3] The first option recommended using in-service pavement sections that exhibited joint spalling due to deficient air void parameters and consolidation levels. The various air void parameters, effects of deicer salts, and the estimated number of freeze-thaw cycles could then be correlated with joint spalling. This information could be used to modify or verify the previous PRS study performance models. The advantage of this type of study is that a field-calibrated performance model could be incorporated into the PRS.

The second option involved the construction and long-term monitoring of specially constructed test sections. The advantage of this option is that the developed performance model would be based on long-term field performance. The disadvantage is that results would not immediately be available (due to long-term monitoring) and certain pavement sections are essentially sacrificed ("destined" to fail because extremely low air contents would be used in order to observe accelerated deterioration).

The research team decided to verify (or calibrate) the entrained air content-based joint spalling relationships using field site evaluations. The testing procedures and model development are described in the following sections.

Sampling and Testing Procedures

For this research study, in-service pavements were used to calibrate the spalling performance models developed in a preceding PRS study.[1,2,3] Sampling was performed at three pavement joint repair projects, as shown in table 66. The same three projects were used for the consolidation and air content studies for variability. A number of joints from each project, exhibiting varying degrees of joint spalling, were selected for the evaluation. Joint repair projects were investigated because they allowed the

retrieval of a sufficient number of 102-mm-diameter cores. At each joint evaluated, a sufficient number of cores were removed from between the dowel bars so that laboratory analyses of hardened air void parameters could be performed. The tests were run in accordance with ASTM C 457, *Standard Test Method for Microscopical Determination of Parameters of the Air Void System in Hardened Concrete.* In addition, cores were removed to determine the compressive strength of the concrete by ASTM C 42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.*

As seen in equations 14 through 16, the laboratory-developed relationships were functions of the presence of deicer salt (calcium chloride).[1,2,3] However, all three projects used to verify the relationship were in areas where deicer salts were regularly applied. Therefore, the SALT variable was effectively removed from the current investigation. In the previously developed equation, the use of salt was incorporated as "1" and the non-use of salt was included as "0," thus eliminating the salt term. For the current study, SALT was always "1", so the SALT multiplier was simply added to the equation constant.

The final factor necessary to develop/calibrate the joint spalling model was the number of freeze-thaw cycles experienced by the pavement. Because the previous study was controlled in the laboratory, the exact number of freeze-thaw cycles was determined with the use of thermocouples placed 76 mm below the surface of the deicer blocks. All the field sites evaluated in the current study were constructed more than 20 years ago, and the exact number of freeze-thaw cycles was unknown. Therefore, an estimate of the number of freeze-thaw cycles was calculated using the climatic model incorporated into the Climatic-Materials-Structural (CMS) computer program.[64]

Atmospheric data obtained from weather stations located near the three projects for the past 3 years were entered into the CMS program. The program output includes the number of freezing and thawing cycles that occur at any depth in the concrete based on the amount of weather station data input. To mirror the previous laboratory study, a freeze occurred for each pavement when the temperature calculated at a depth of 76 mm below the surface stayed below 0 °C for at least 48 hours. The pavement is assumed to be thawed when the temperature at 76 mm below the surface rises above 0 °C. The CMS program was then used to predict the number of freeze-thaw cycles that occurred for each project for the years 1994, 1995, and 1996. Based on studies published by Dempsey, the resulting number of annual pavement freeze-thaw cycles for the projects evaluated seem reasonable. This average annual number of cycles was then multiplied by the age of the pavements studied to compute the total number of cumulative freeze-thaw cycles for each project.

Transverse Joint Spalling Model Calibration

The collected data were used to calibrate the previously developed model predicting the percent of joint that is spalled as a function of compressive strength, freeze-thaw cycles, and air void system parameters. For simplicity, the model incorporating

entrained air content is preferred over the models based on other air void system parameters. The air content model would allow the use of plastic air content data to be used as opposed to costly linear traverse testing.

The collected data for the model calibration is summarized in table 67. Figure 77 shows a plot of the predicted joint spalling using the previously developed equation with entrained air content as the only air void parameter (equation 14) versus the field-measured joint spalling. Three new air void parameter-based regression equations were derived using the data collected for this investigation.

The first calibrated model, giving joint spalling as a function of entrained air content only, is presented as equation 17.

$$SPALL = 115 - 9.29 * AIR - 0.0114 * f'c + 0.118 * FTC_{76} \quad (17)$$

where

SPALL = Joint spalling, percent of joint length.
AIR = Measured air content of the fully consolidated specimen, percent.
f'c = Measured compressive strength mean, psi.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.

The collected data used in the development of equation 17 are presented in figure 78. A plot showing the sensitivity of equation 17 (function of only entrained air content) is presented in figure 79.

The second calibrated model, giving joint spalling as a function of both entrained air content and void spacing factor, is presented as equation 18.

$$SPALL = 96.1 - 8.45 * AIR + 747 * L - 0.00991 * f'c + 0.114 * FTC_{76} \quad (18)$$

where

SPALL = Joint spalling, percent of joint length.
AIR = Measured air content of the fully consolidated specimen, percent.
L = Void spacing factor, 1/in.
f'c = Measured compressive strength mean, psi.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.

The collected data used in the development of equation 18 are presented in figure 80.

The third calibrated model, estimating joint spalling as a function of void spacing factor and not entrained air content, is presented as equation 19.

Table 67. Data collected for variables related to percent joint spalling.

| Project | Year Built | Entrained Air Content, % | Void Spacing Factor, mm | Compressive Strength, MPa | Number of Freeze/Thaw Cycles | Percentage of Joint Spalled, % |
|---|---|---|---|---|---|---|
| Benton Harbor, MI | 1971 | 10.6 | 0.1016 | 47.85 | 375 | 75 |
| | | 7.1 | 0.1016 | 46.82 | 375 | 28 |
| | | 7.8 | 0.1778 | 45.44 | 375 | 3 |
| | | 7.6 | 0.1016 | 46.61 | 375 | 12 |
| | | 5.6 | 0.2032 | 51.37 | 375 | 25 |
| | | 5.4 | 0.1524 | 51.09 | 375 | 28 |
| Des Plaines, IL | 1967 | 5.7 | 0.3048 | 38.13 | 551 | 69 |
| | | 4.5 | 0.2540 | 48.26 | 551 | 52 |
| | | 6.7 | 0.1270 | 50.12 | 551 | 25 |
| | | 5.9 | 0.1778 | 46.75 | 551 | 34 |
| | | 4.9 | 0.2286 | 47.30 | 551 | 77 |
| Philo, IL | 1970 | 2.9 | 0.2032 | 40.82 | 208 | 37 |
| | | 6.7 | 0.2540 | 35.37 | 208 | 21 |
| | | 7.1 | 0.1778 | 29.03 | 208 | 24 |
| | | 5.7 | 0.3048 | 38.54 | 208 | 26 |
| | | 4.3 | 0.3556 | 32.75 | 208 | 45 |

Note: The number of freeze-thaw cycles was determined using air temperature data in Dempsey's CMS computer program.[64]

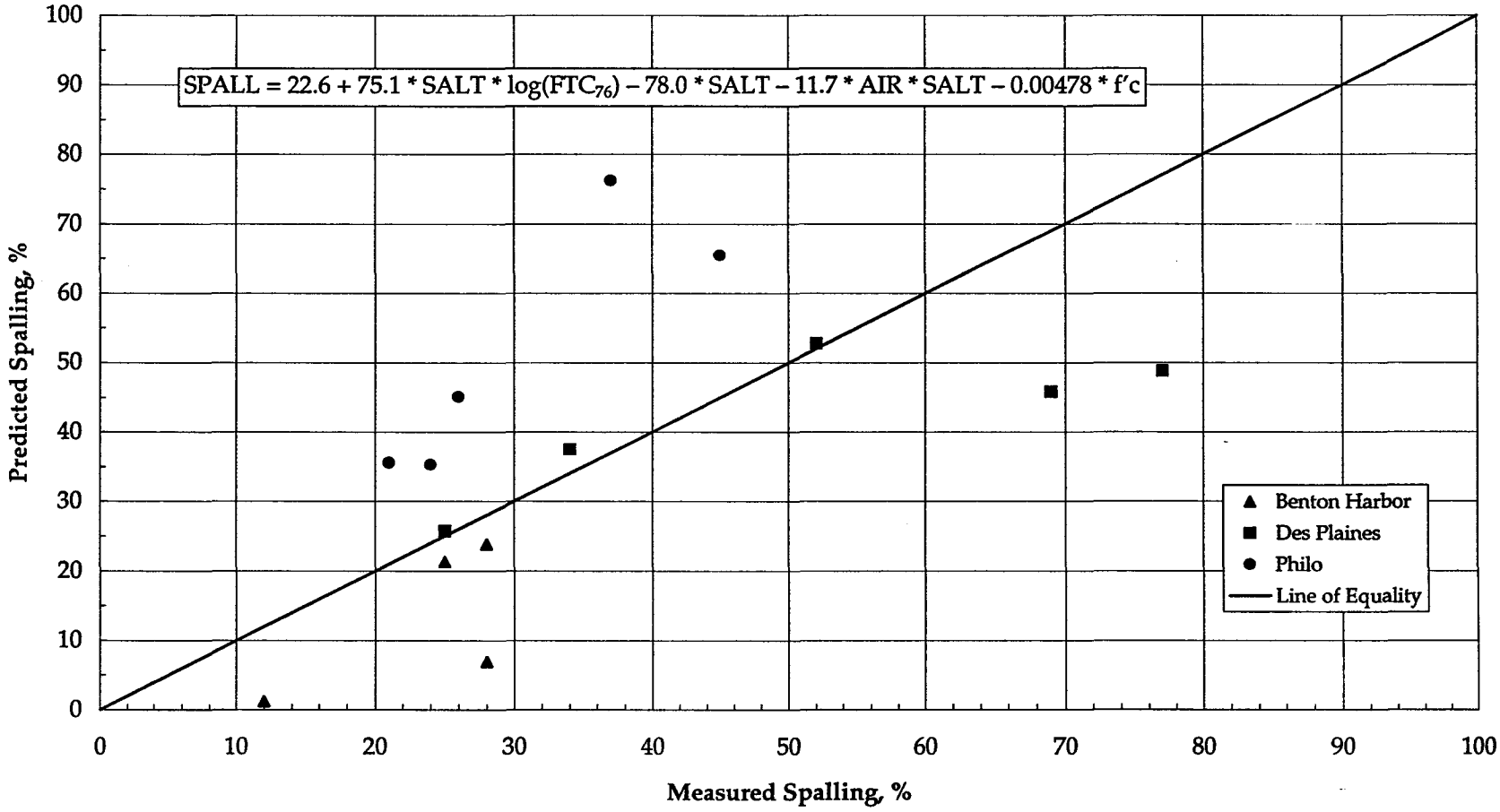



Figure 77. Predicted versus measured transverse joint spalling using the "entrained air content only" model developed under a previous PRS project.[1,2,3]

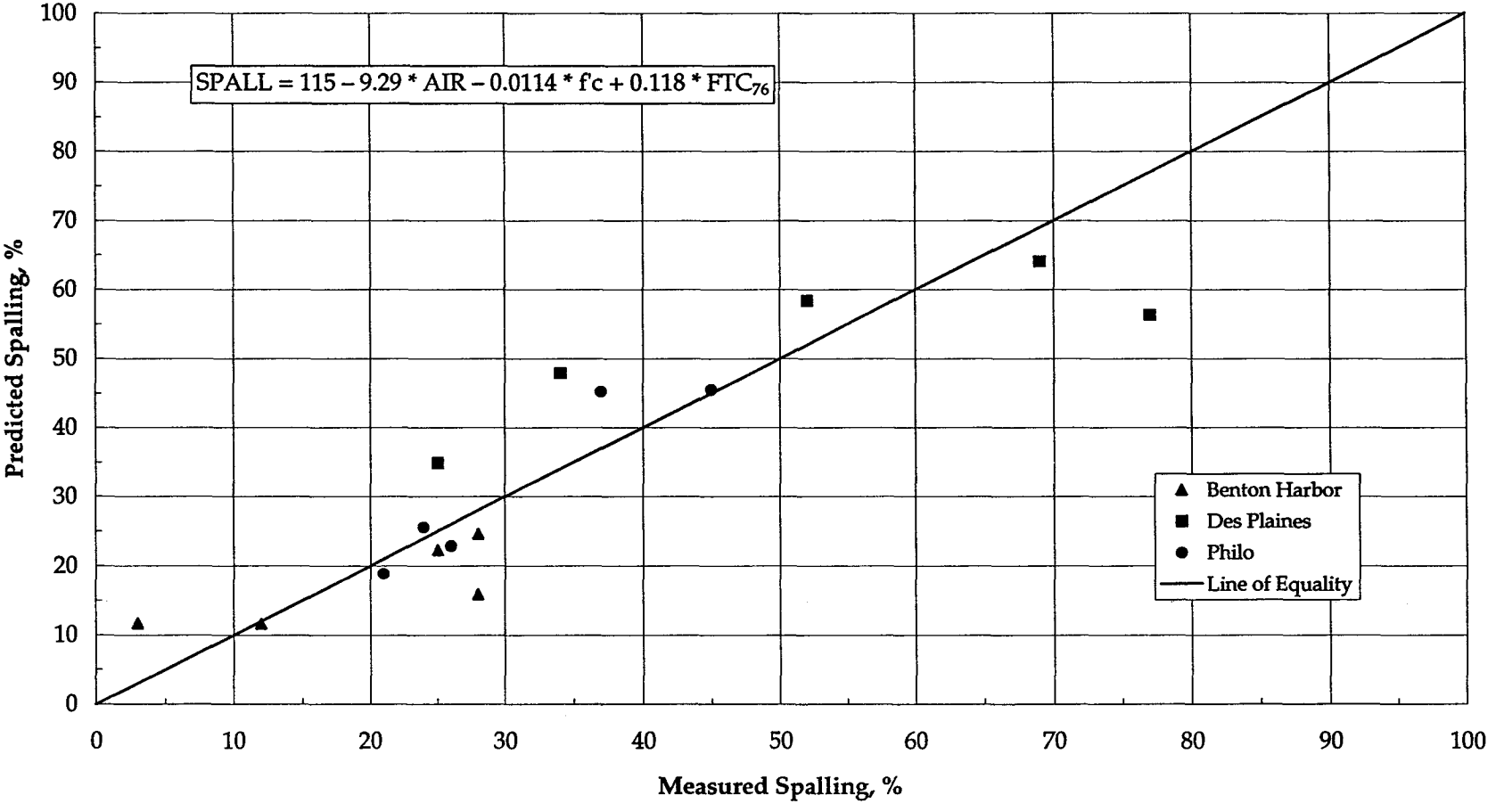



Figure 78. Predicted versus measured transverse joint spalling using the calibrated "entrained air content only" model.

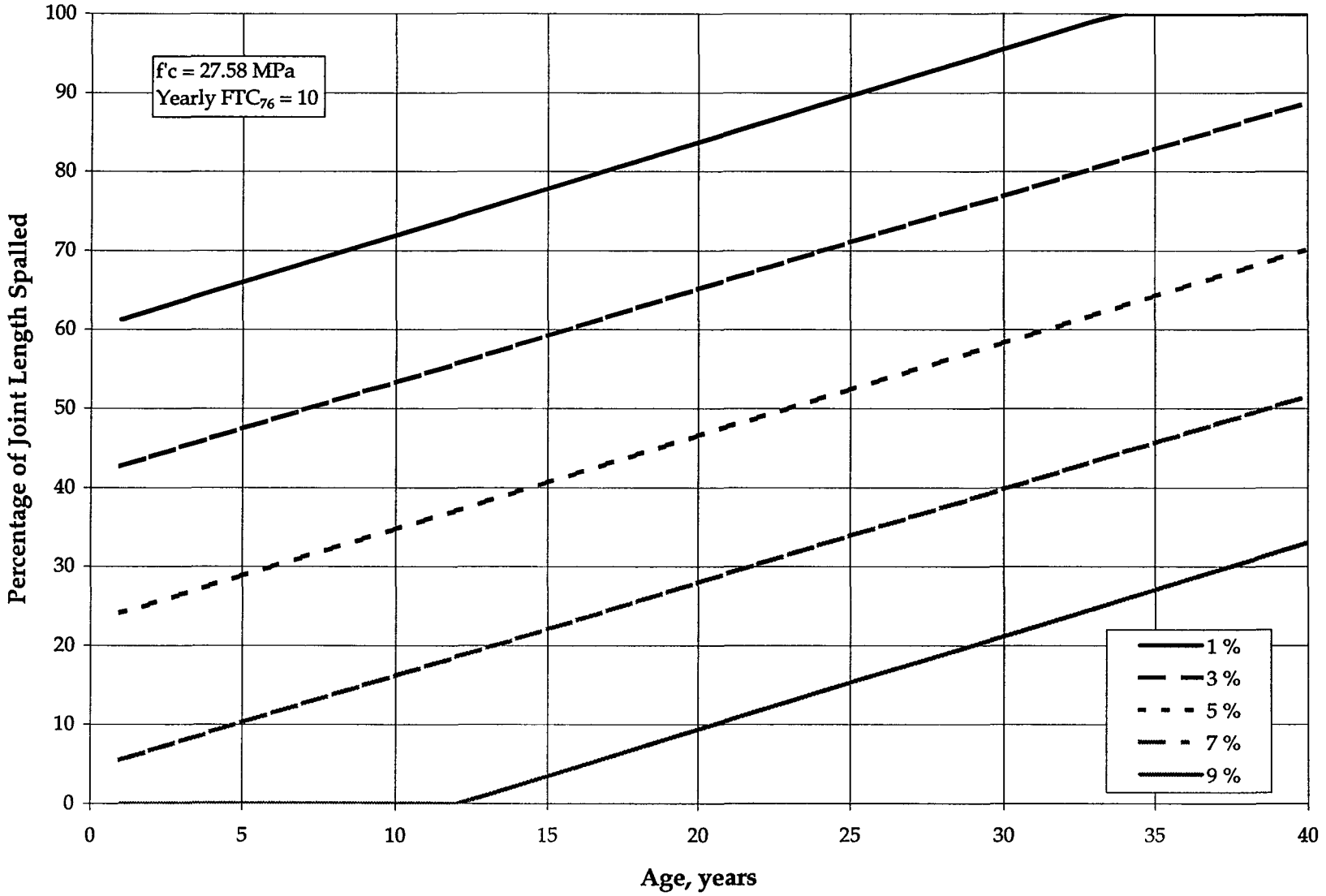



Figure 79. Sensitivity of the calibrated "entrained air content only" spalling model (percentage of joint length spalled versus age for different entrained air content percentages).

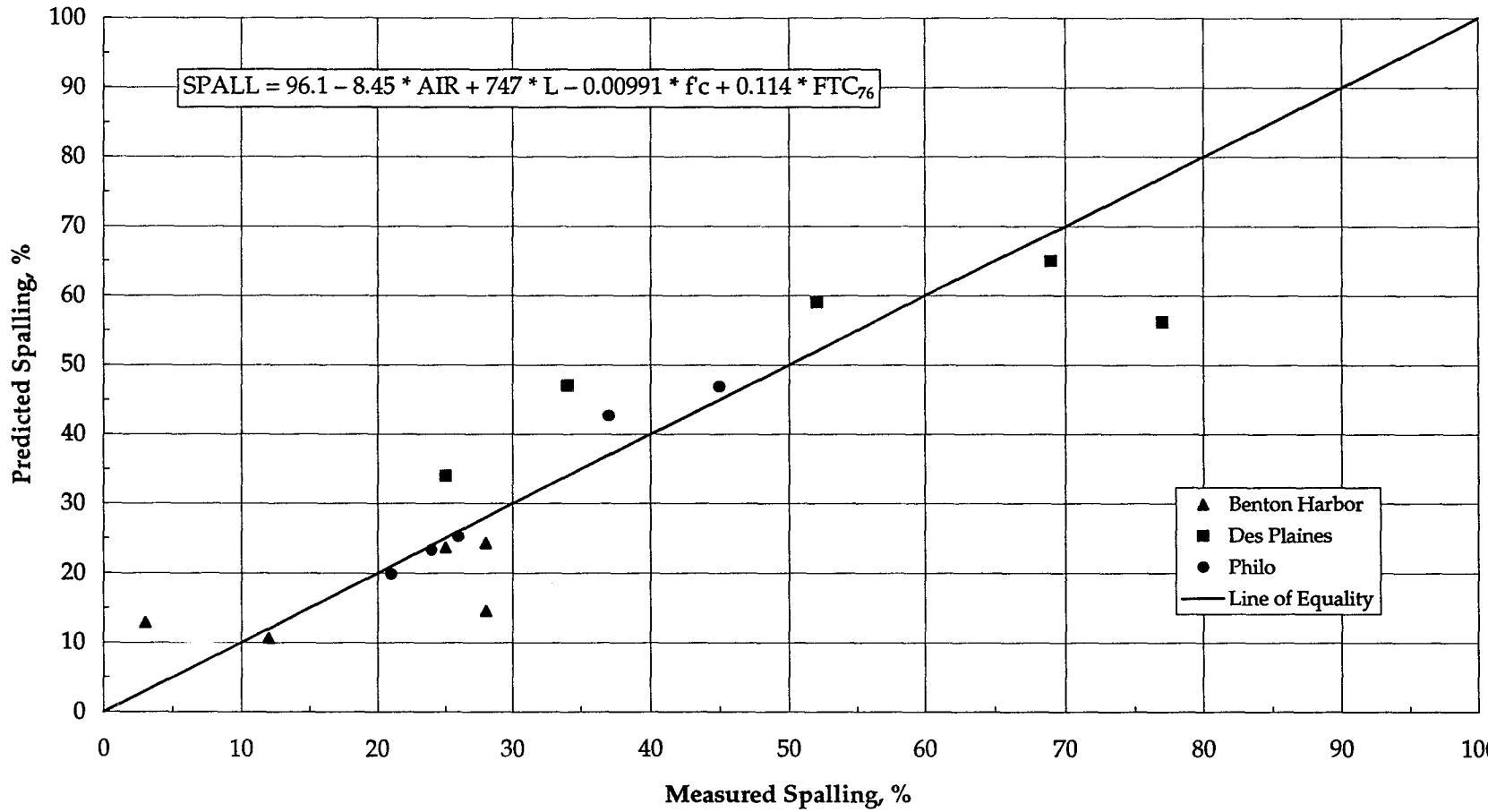



Figure 80. Predicted versus measured transverse joint spalling using the calibrated "entrained air content and void spacing factor" model.

$$SPALL = -7.3 + 3646 * L - 0.00403 * f'c + 0.0963 * FTC_{76} \quad (19)$$

where

SPALL = Joint spalling, percent of joint length.
L = Void spacing factor, 1/in.
f'c = Measured compressive strength mean, psi.
$FTC_{76}$ = Cumulative number of estimated freeze-thaw cycles at 76 mm below the pavement surface.

The collected data used in the development of equation 19 is presented in figure 81.

The complete procedure making up the current transverse joint spalling model is described in detail in the section titled *Summary of Current Distress Indicator Models.*

**Effect of Percent Consolidation Around Dowels on Transverse Joint Faulting**

Introduction

The effects of percent consolidation level around dowels on transverse joint faulting are not well-documented in the literature. The three projects evaluated for the calibration of the spalling prediction model were also evaluated for consolidation level at doweled joints as shown in table 66. Different sections within each project exhibiting a range of distress (faulting or loss of load transfer) were selected for the evaluation of the effect of consolidation level on load transfer across doweled joints. The relationship was integrated into the current faulting prediction model, which is a function of concrete bearing stress (BSTRESS).

Sampling and Testing Procedure

To measure the in situ consolidation level of concrete pavements at doweled joints, density was measured from drilled cores as previously detailed in the consolidation variability study. For each project, several joints exhibiting varying degrees of deterioration were selected as part of the study. Cores were drilled through dowel bars, between dowel bars, and away from the joint to evaluate relative variability. Core densities were measured in a saturated surface-dried condition in accordance with ASTM C 642, *Standard Test Method for Specific Gravity, Absorption, and Voids in Hardened Concrete,* to evaluate variability in consolidation incorporating the effects of basket assemblies and dowel bars (above and below). The core removed away from a joint with the highest density within a sampled lot was assumed to be 100-percent consolidated. The remaining core densities were compared to the 100-percent consolidated density to determine the relative consolidation levels. This procedure incorporates over-consolidation effects when vibration time is extended over basket assemblies or when vibrators contact dowels for longer-than-normal periods. Load transfer efficiency was directly measured using a falling-weight deflectometer (FWD), and the effective stress transfer efficiency was computed.

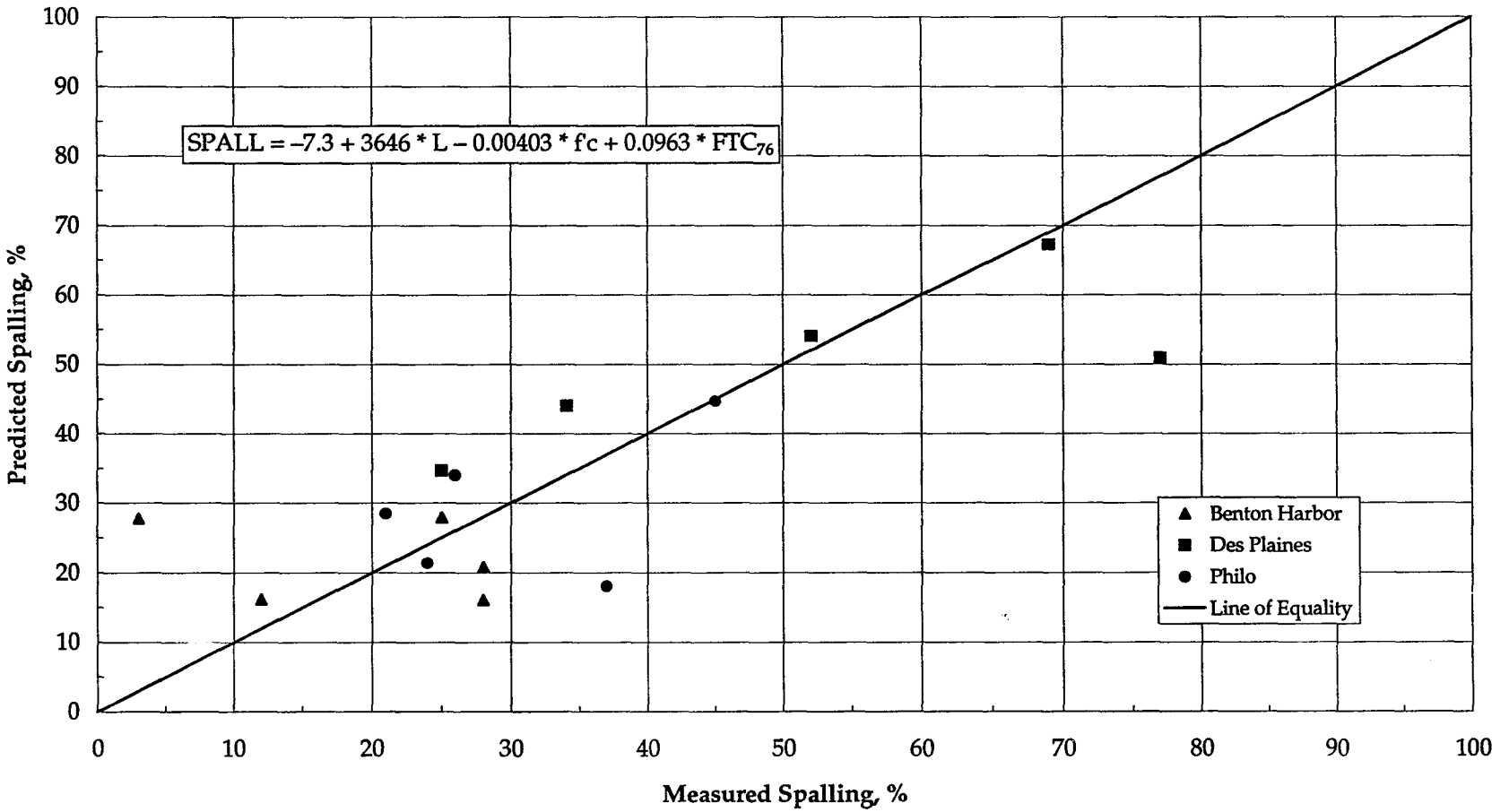



Figure 81. Predicted versus measured transverse joint spalling using the calibrated "void spacing factor only" model.

Summary of Results

The variation of consolidation level was determined to be fairly minor in the variability portion of this investigation. Therefore, further projects were not evaluated for the collection of additional data. However, based on the data collected for the three projects, a load transfer efficiency (LTE) versus percent consolidation relationship was determined and is presented in equation 20.

$$LTE = 10.855 * (\%CON) - 1021.7 \quad (20)$$

where

LTE = Load transfer efficiency, %.
%CON = Computed percent consolidation around dowels, %.

The data used to determine this relationship are plotted in figure 82. Also shown on the graph is the derived regression equation for the relationship. This relationship was incorporated into the current joint faulting model (described in the section titled *Summary of Current Distress Indicator Models*).

**Tie Bar Depth Versus Load Transfer Efficiency**

To evaluate the feasibility that a model could be developed relating the tie bar depth to joint LTE, an FWD was used to measure longitudinal joint load transfer at the Omaha, Nebraska site as shown in table 66. As discussed in the section on tie bar variability, the tie bars at the Omaha project were consistently placed nearly 76 mm above slab mid-depth. However, as shown in figure 83 (for a consistent tie bar depth), the load transfer varied from as low as 30 percent to about 100 percent. Also shown in the figure are five locations where tie bars were placed significantly lower than average. The LTE was approximately 95 percent for these locations. Therefore, the collection of additional tie bar depth data from other construction sites was aborted after data collected at the Omaha project suggested that a reliable relationship was not likely.

## SUMMARY OF CURRENT DISTRESS INDICATOR MODELS

The current PRS approach expresses JPCP performance in terms of any or all of the following distress indicators:

- Transverse fatigue cracking.
- Transverse joint spalling.
- Transverse joint faulting.
- Pavement smoothness—Present Serviceability Rating (PSR) or International Roughness Index (IRI).

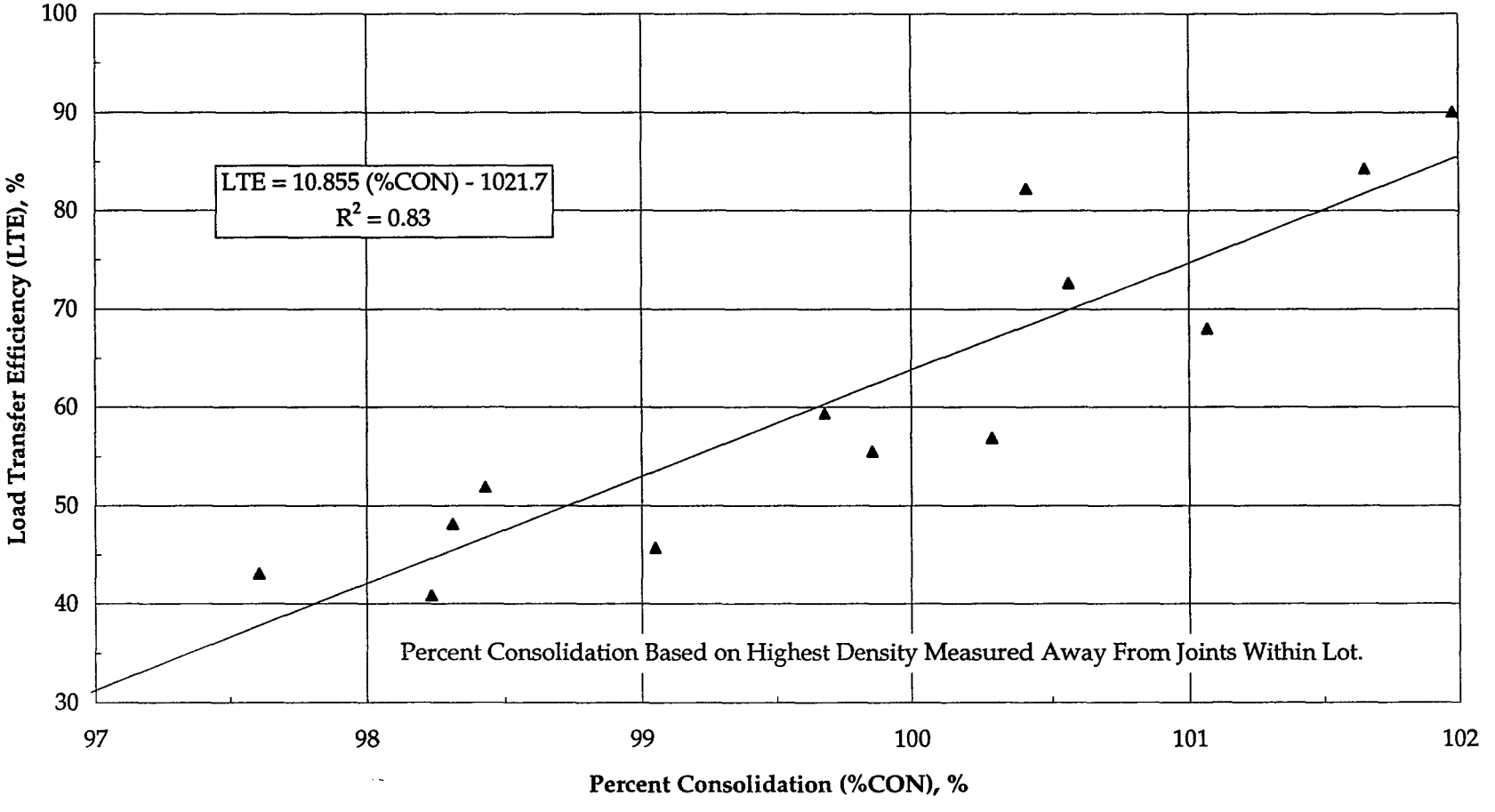



Figure 82. Plot of load transfer efficiency as a function of percent concrete consolidation.

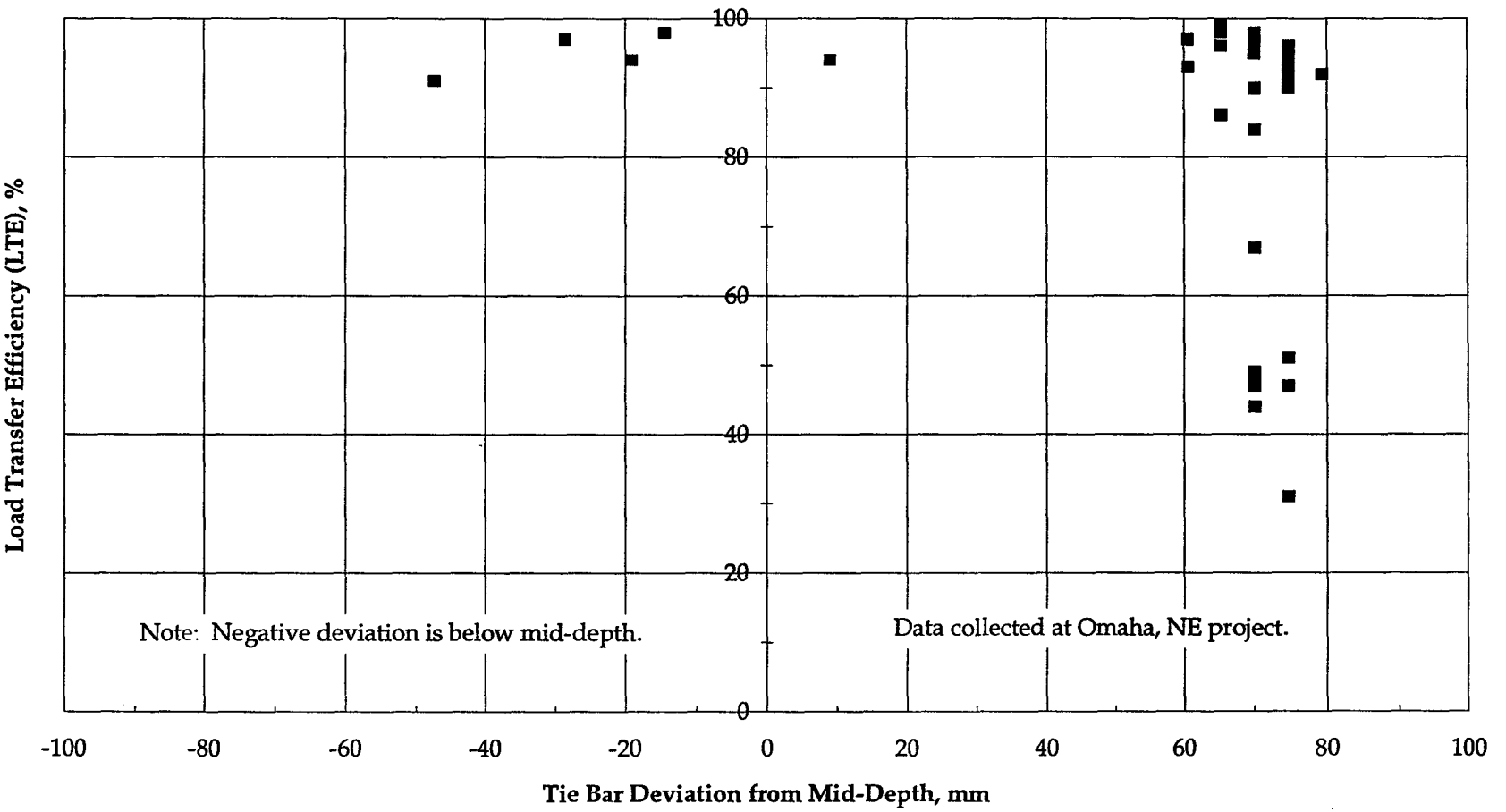



Figure 83. Load transfer efficiency versus tie bar deviation from slab mid-depth.

The best available set of these models was selected and is included in the revised PRS computer software (PaveSpec 2.0). This section includes detailed discussions of each model, including the model source, model form and specific procedures, and some discussion of the model sensitivity. The details of each model are described separately. (Note: Each of these models was originally developed in terms of English units; therefore, they are presented in this section in English units. Metric equivalents are given when appropriate.)

**Transverse Slab Cracking**

The current transverse slab-cracking model is based on a fatigue-cracking model developed by Smith et al. in 1990.[65] Specifically, the model is based on a fatigue-consumption approach that assumes a concrete pavement has a finite life and can only withstand a maximum allowable number of 80-kN equivalent single-axle loading (ESAL) repetitions (N). The details of the chosen transverse slab-cracking model are discussed below.

Transverse Slab-Cracking Model Characteristics

*Model Source*—Smith, K.D., A.L. Mueller, M.I. Darter, and D.G. Peshkin, *Performance of Jointed Concrete Pavements, Volume II—Evaluation and Modification of Concrete Pavement Design and Analysis Models*.[65]

*Model Output Units*—Percentage of slabs cracked.

*Dependency on Other Model Outputs*—None.

Transverse Slab-Cracking Model

$$\%CRAKD = 1 / [0.01 + 0.03 * (20^{-\log(n/N)})] \quad (21)$$

where

%CRAKD = Slabs cracked (transverse cracking), %.

n = Actual number of 80-kN ESAL applications at slab edge.
= (0.05 * *TotESAL's*) for pavements without widened lanes.
= (0.001 * *TotESAL's*) for pavements with widened lanes.

TotESAL's = Cumulative total number of measured 80-kN ESAL applications.

N = Allowable 80-kN ESAL applications.
$= 10^{[2.13*(1/Ratio)^{1.2}]}$

Ratio = The ratio of computed edge stress ($\sigma_{EDGE}$) to 28-day modulus of rupture ($MR_{28}$).

$\sigma_{EDGE}$ = Total resultant stress in the longitudinal direction at the bottom of the PCC slab edge when the wheel load is located at the slab edge.

$= \sigma_{TRAFFIC} + (R) * \sigma_{CURLING}$

$\sigma_{TRAFFIC}$ = Stress at bottom of PCC slab edge when load is located at slab edge (no thermal curling stress). This value is computed using equation 22.

$$= [P/18.0 * h^2] * [17.35783 + (0.07801 * ES) - (0.05388 * h^3/k) + 7.41722 * \log_{10}(h^3/k)] \quad (22)$$

$\sigma_{CURLING}$ = Stress at bottom of PCC slab edge caused by curling of slab due to thermal gradient (no traffic load). This value is computed using equation 23.

$$= [(G * ET) / (5 \times 10^{-6})] * [(0.06712 * k) + (79.07391 * \log_{10}(k)) + (11.72690 * L) - (0.00720 * k * L) - (3.22139 * L * \log_{10}(k)) - (0.06883 * L * ES) - (0.59539 * ES * \log_{10}(k)) - (204.39477 * h/k) - (38.08854 * L/h) - (8.36842 * h * \log_{10}(k)) + (0.07151 * ES * h) + (0.05691 * L * ES * \log_{10}(k)) + (0.20845 * L * h * \log_{10}(k)) + (0.00058 * L * h * k) - (0.00201 * L * ES * \log_{10}(k))] \quad (23)$$

R = Adjustment factor for $\sigma_{CURLING}$ so that it can be combined with $\sigma_{TRAFFIC}$ to give correct $\sigma_{EDGE}$. This value is computed using equation 24.

$$= 0.48039 + (0.01401 * h) - (0.00427 * ES) - (0.27278 * G) - (0.00403 * L) + (0.19508 * \log_{10}(k)) + (0.45187 * G * \log_{10}(h)) - (0.00532 * G^2) + (0.01246 * G * L) - (0.00622 * G * L * \log_{10}(k)) + (8.7872 * \log_{10}(h^3/k)/h^2) + (0.00104 * G * ES) - (0.11846 * G * \log_{10}(h^3/k)) + (0.07001 * \log_{10}(ES + 1.0)) - (0.01331 * G * \log_{10}(ES + 1.0)) \quad (24)$$

P = Total applied load, lb (assumed to be 9,000 lb).

$h_{PCC}$ = Slab thickness, in.

ES = Erodability of support along slab edge, in (assumed to be zero).

k = Modulus of subgrade reaction, psi/in.

ET = Thermal coefficient of contraction of PCC (assumed to be 0.0000055).

L = slab length (joint spacing), ft.

$MR_{28}$ = 28-day modulus of rupture (third-point loading flexural strength), psi.

G = Thermal gradient through slab (estimated from table 68), °F/in.

Table 68. Average daytime thermal gradients (based on slab thickness and climatic zone), °F/in.[65]

| Slab Thickness, mm | Wet-Nonfreeze Climatic Zone | Dry/Wet-Freeze Climatic Zones | Dry-Nonfreeze Climatic Zone |
|---|---|---|---|
| 203 | 1.40 | 1.13 | 1.41 |
| 229 | 1.30 | 1.05 | 1.31 |
| 254 | 1.21 | 0.96 | 1.21 |
| 279 | 1.11 | 0.87 | 1.10 |
| 305 | 1.01 | 0.79 | 1.00 |

Procedure for Using the Transverse Cracking Model

Transverse fatigue cracking is predicted for both the as-designed and as-constructed pavements using the procedure outlined below. The variables that differ between the as-designed and as-constructed pavements include slab thickness and 28-day flexural strength (28-day modulus of rupture). The 28-day flexural strength may be translated from a 28-day compressive strength using an equation of the form $MR = A * (f'c)^{0.5}$.

The step-by-step procedure for calculating the cracking (percentage of slabs cracked) for either the as-designed or as-constructed pavement is as follows:

1. Compute $\sigma_{TRAFFIC}$.
2. Compute $\sigma_{CURLING}$.
3. Compute R (adjustment factor).
4. Compute $\sigma_{EDGE}$ (function of $\sigma_{TRAFFIC}$, $\sigma_{CURLING}$, and R).
5. Compute "Ratio" (function $\sigma_{EDGE}$ of 28-day flexural strength).
6. Compute "N" (N = number of *allowable* cumulative ESAL's).
7. Compute the "n/N" ratio for each year (n/N = [actual cumulative ESAL's /allowable cumulative ESAL's]).
8. Compute the "% cracked slabs/mile" for each year.
9. Translate "% cracked slabs/mile" into "total # cracked slabs per/mile" using equation 25.

$$\text{total \# cracked slabs per/mile} = ([\%\ \text{cracked slabs/mi}] / 100) * \text{integer}(5280/[\text{joint spacing}]) \quad (25)$$

where joint spacing is expressed in units of ft.

The translation is required because the PSR model uses "total # cracked slabs per/mile," whereas the IRI model uses "% cracked slabs/mile."

Sensitivity of Transverse Cracking Model

Plots showing the sensitivity of the model to several key pavement design inputs are included in this section. (Note: All of the sensitivity plots are presented in English units.)

Figure 84 contains a plot showing the effect of shoulder type on transverse slab cracking. For the three different shoulder types investigated, the sections with a tied PCC shoulder and widened outer traffic lane both exhibited very little (if any) transverse cracking over 40 million ESAL applications. The section with only an asphalt shoulder displayed a significant amount of transverse cracking.

A plot showing the sensitivity of transverse cracking to changes in transverse joint spacing is included in figure 85. This chart clearly shows a positive influence of shorter slabs on reducing transverse cracking. The difference in predicted slab cracking between slabs with lengths of 6.1 m, 4.6 m, and 3.0 m is quite significant.

Figure 86 provides the results of a sensitivity analysis of the transverse cracking model with respect to slab thickness. The plot shows that the 203-mm slab exhibits extensive slab cracking very early in its life. Increasing slab thickness from 203 mm to 254 mm greatly reduces the development of fatigue cracking. Likewise, increasing slab thickness from 254 mm to 305 mm essentially reduces the transverse slab cracking to zero.

**Transverse Joint Spalling**

Two different procedures are used to estimate transverse joint spalling, depending on whether entrained air content is considered (the relationships using void spacing factor are not currently available in PaveSpec). Regardless of the procedure chosen, the base transverse joint spalling model is that developed by Yu et al. in 1997.[66] If entrained air content is not considered, transverse joint spalling is predicted directly using the base model. If entrained air content is considered, the transverse joint spalling (predicted using the base model) is then altered based on a comparison of as-constructed to as-designed mean entrained air content. Both model procedures are presented separately below. (Note: The model outputs and inputs are presented in English units.)

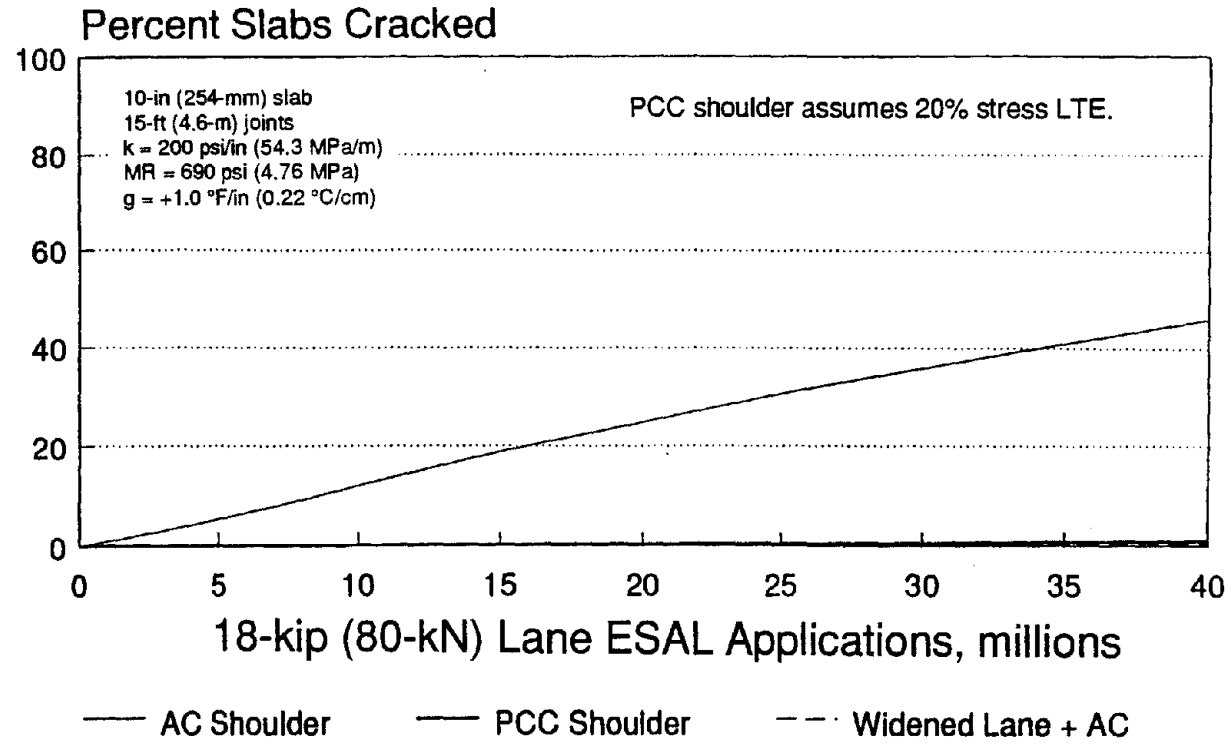
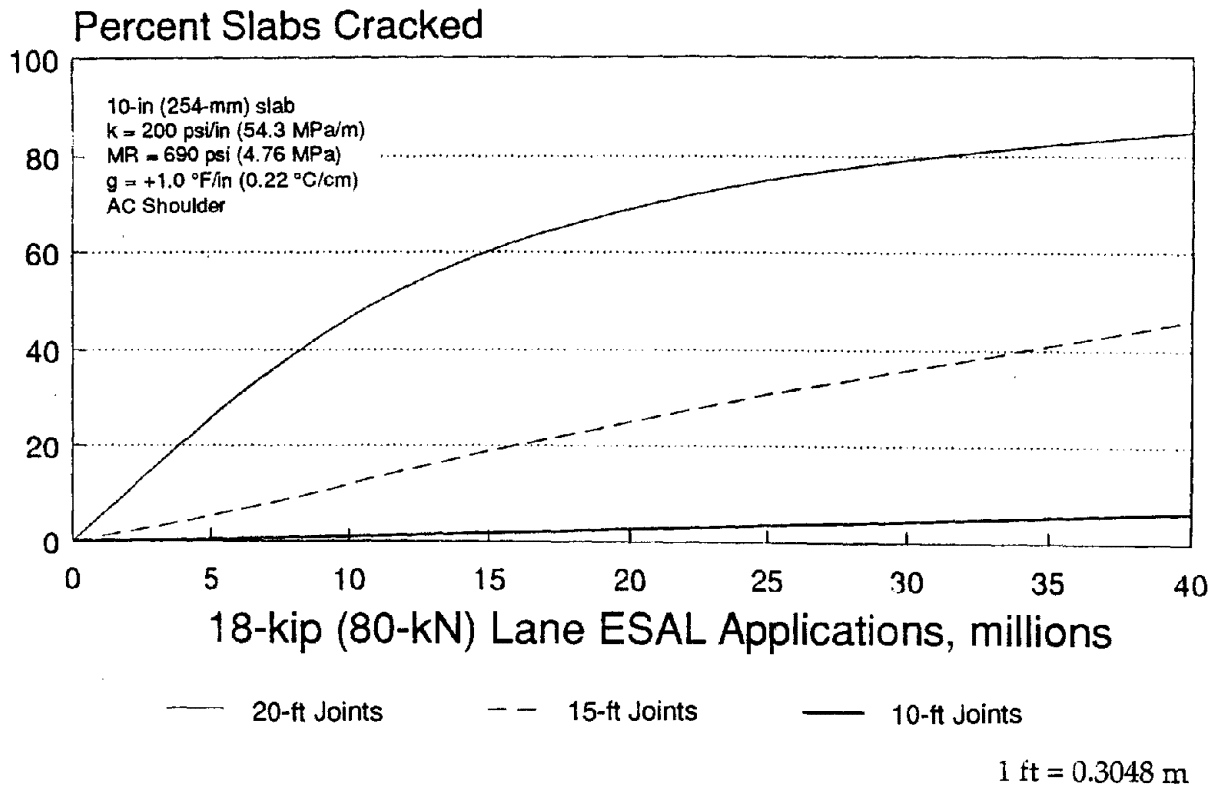
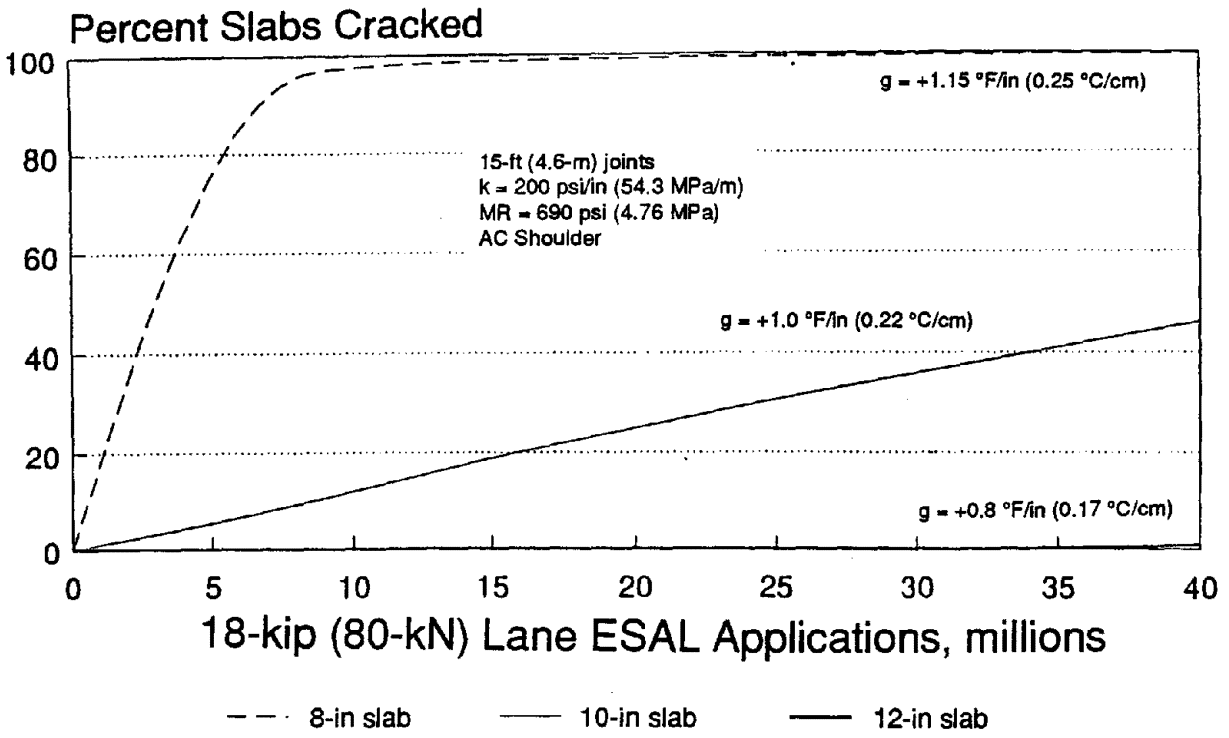



Figure 84. Sensitivity of transverse cracking model to shoulder type.[65]




1 ft = 0.3048 m

Figure 85. Sensitivity of transverse cracking model to transverse joint spacing.[65]




1 in = 25.4 mm

Figure 86. Sensitivity of transverse cracking model to slab thickness.[65]

Tranverse Joint Spalling Base Model Characteristics

*Model Source*—Yu, H.T., K.D. Smith, M.I. Darter, J. Jiang, and L. Khazanovich, *Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance.*[66]

*Model Output Units*—Percentage of medium- and high-severity spalled joints.

*Dependency on Other Model Outputs*—None.

Joint Spalling Model (Not Including Entrained Air Content)

$$\%SPALL = AGE^2 * 10^{-4} * JTSPACE * [551.6 - 847.3 * (LIQSEAL + PREFSEAL) + (0.936 * (Days90)^3 * 10^{-3}) + (364 * DOWELCOR) + (2.783 - 1.40 * LIQSEAL - 2.368 * PREFSEAL - 0.676 * SILSEAL) * FI] \quad (26)$$

where

%SPALL = Percentage of medium- and high-severity spalled joints where entrained air content is not considered.

AGE = Number of years since original construction.

DOWELCOR = Dowel corrosion (probably always assume = 0).
= 0, if no dowels exist or dowels are protected from corrosion.
= 1, if dowels are not protected from corrosion.

JTSPACE = Mean transverse joint spacing, ft.

LIQSEAL = 1, if liquid sealant exists in joint; otherwise, 0.

PREFSEAL = 1, if preformed sealant exists in joint; otherwise, 0.

SILSEAL = 1, if silicon sealant exists in joint; otherwise, 0.

FI = Mean annual freezing index, degree-days.

Days90 = Number of days with temperature greater than 90 $^{o}$F.

Joint Spalling Model and Procedure (Including Entrained Air Content)

The procedure for the case where entrained air content is included is a four-phased procedure. The details of each phase are defined below.

*Phase 1—Calculate the As-Designed Transverse Joint Spalling*

The first step in computing the estimated yearly as-constructed transverse joint spalling is to determine the yearly as-designed transverse joint spalling values. The as-designed transverse joint spalling is assumed to be equal to the spalling computed for the case where entrained air content is not considered (i.e., the spalling computed directly using equation 26).

*Phase 2—Calculate the Relative Difference Between the As-Designed and As-Constructed Spalling (due to the effects of air content and compressive strength)*

This relative difference is due directly to differences between as-constructed and as-designed entrained air content and concrete strength. This as-constructed to as-designed (AC-AD) ratio is computed based on equations that are functions of entrained air content, compressive strength, freeze-thaw cycles (in the pavement), and age. Different equations are used depending on whether deicing salt is present. Each of these scenarios is described separately below.

Case 1—Salt is Present

If salt is present, an AC-AD ratio is computed based on the relationship calibrated under the current research project and presented as equation 17. This equation is calculated for both the as-constructed and as-designed pavements (using representative as-constructed and as-designed compressive strength and entrained air content means), as shown in equations 27 and 28, respectively.

$$\text{AC-SPALL} = 115 - (9.29 * \text{AIR}_{\text{AS-CON}}) - (0.0114 * f'c_{\text{AS-CON}}) + (0.118 * \text{FTC}_{76}) \quad (27)$$

$$\text{AD-SPALL} = 115 - (9.29 * \text{AIR}_{\text{AS-DES}}) - (0.0114 * f'c_{\text{AS-DES}}) + (0.118 * \text{FTC}_{76}) \quad (28)$$

where

AC-SPALL = As-constructed joint spalling number (function of as-constructed air content and f'c), percentage of joint length.
AD-SPALL = As-designed joint spalling number (function of as-designed air content and f'c), percentage of joint length.
$\text{FTC}_{76}$ = Cumulative number of estimated freeze thaw-cycles at 76 mm below the pavement surface.
$\text{AIR}_{\text{AS-CON}}$ = Measured as-constructed air content, percent.
$\text{AIR}_{\text{AS-DES}}$ = As-designed air content, percent.
$f'c_{\text{AS-CON}}$ = Measured as-constructed compressive strength, psi.
$f'c_{\text{AS-DES}}$ = As-designed compressive strength, psi.

Finally, the AC-AD ratio is computed as a function of AC-SPALL and AD-SPALL using equation 29.

$$\text{AC-AD ratio} = \text{AC-SPALL}/\text{AD-SPALL} \quad (29)$$

Case 2—Salt is Not Present

If salt is not present, equation 14 (developed under the previous PRS project) is used.[1,2,3] Without the presence of salt, equation 14 simplifies to be a function of compressive strength only. Therefore, the effects of entrained air content are no longer considered directly, but instead are considered indirectly through compressive strength. The equation is calculated for both the as-constructed and as-designed pavements (using representative as-constructed and as-designed compressive strength means), as shown in equations 30 and 31, respectively.

$$\text{AC-SPALL} = 22.6 - 0.00478 * f'c_{\text{AS-CON}} \quad (30)$$

$$\text{AD-SPALL} = 22.6 - 0.00478 * f'c_{\text{AS-DES}} \quad (31)$$

where

AC-SPALL = As-constructed joint spalling number (function of as-constructed air content and f'c), percentage of joint length.

AD-SPALL = As-designed joint spalling number (function of as-designed air content and f'c), percentage of joint length.

$f'c_{\text{AS-CON}}$ = Measured as-constructed compressive strength, psi.

$f'c_{AS\text{-}DES}$ = As-designed compressive strength, psi.

Finally, the AC-AD ratio is again computed using equation 29.

*Phase 3—Determine the As-Constructed Spalling Values Using the Phase 2 AC-AD Ratio*

The phase 3 as-constructed spalling values are calculated by multiplying the phase 1 as-designed spalling values by the computed phase 2 AC-AD yearly ratios.

*Phase 4—Adjustment of Phase 3 Spalling Values to the Assumed Boundary Conditions*

Finally, the phase 3 as-constructed spalling values are adjusted to the assumed boundary conditions in phase 4. The following general assumptions are used to define the boundary conditions:

- The as-designed and as-constructed curves are never allowed to cross.
- As-designed and as-constructed transverse joint spalling values are both equal to zero at an age equal to zero.
- The predicted transverse joint spalling value must increase (or stay constant) as time increases.
- If the as-constructed curve is above the as-designed curve (poorer quality than the as-designed), the as-constructed curve is assumed to go through the origin at an age of zero.

The following step-by-step procedures are used to determine the as-constructed spalling curve values required to match the boundary conditions. The values in question are near an age of zero. The procedure differs depending on whether the as-constructed values are greater than or less than the as-designed values. The as-designed and as-constructed values at the end of the analysis period are used to determine which scenario is used. Each scenario is explained in detail below.

Scenario 1: The as-constructed value is less than the as-designed value at the end of the analysis period.

Scenario 1 represents the case where the as-constructed pavement is constructed with better quality than specified for the as-designed pavement. When this occurs, the following procedure is used to ensure boundary conditions:

1. Calculate the yearly slope changes starting at the last year of the analysis period.
2. Compare year-to-year slopes starting at the last year of the analysis period and working backwards (i.e., moving toward an age of zero).

3. Identify the first yearly interval (trigger interval) where the slope stops decreasing and begins increasing (Trigger Interval = $SLOPE_{(n)} = SPALL_{YEAR(n)} - SPALL_{YEAR(n-1)}$).

4. If the slopes begin increasing, use the last decreasing slope in the series ($SLOPE_{(n+1)}$) as the assumed slope for the early age slopes (SLOPE = 0 to n).

5. One final check is to make sure that the as-constructed values are less than or equal to the as-designed values.

Scenario 2: The as-constructed value is greater than the as-designed value at the end of the analysis period.

Scenario 2 represents the case where the as-constructed pavement is constructed with poorer quality than that specified for the as-designed pavement. When this occurs, the following procedure is used to ensure boundary conditions:

1. Calculate the yearly slope changes starting at the last year of the analysis period.

2. Determine the projected y-intercept associated with each calculated slope.

3. Compare year-to-year projected y-intercepts starting at the last year of the analysis period and working backwards (i.e., moving toward an age of zero).

4. Identify the first yearly interval (trigger interval) where the y-intercept becomes greater than or equal to zero (Trigger Interval = $SLOPE_{(n)} = SPALL_{YEAR(n)} - SPALL_{YEAR(n-1)}$).

5. Determine the difference between the as-constructed and as-designed values at year n ($DIFFERENCE_{(n)} = AC\text{-}SPALL_{(n)} - AD\text{-}SPALL_{(n)}$).

6. As (n) moves toward zero, the difference between the spalling values decreases linearly. For example, if the difference is determined to be 6 (100 percent of the observed difference) at a time of n=10 years, 50 percent of the observed difference will be applied at a time of n/2 (therefore, difference = 3 at 5 years). This difference progresses until a difference of zero is applied at a time equal to zero.

Sensitivity of Transverse Joint Spalling Base Model

A sensitivity analysis of the transverse joint spalling base model was conducted in the study by Yu et al.[66] The results of this sensitivity analysis are provided in this section. (Note: All of the sensitivity plots are presented in English units.)

Figures 87 through 90 show the sensitivity of the joint spalling model to age, sealant type, and climatic conditions. These plots were constructed by fixing the climatic

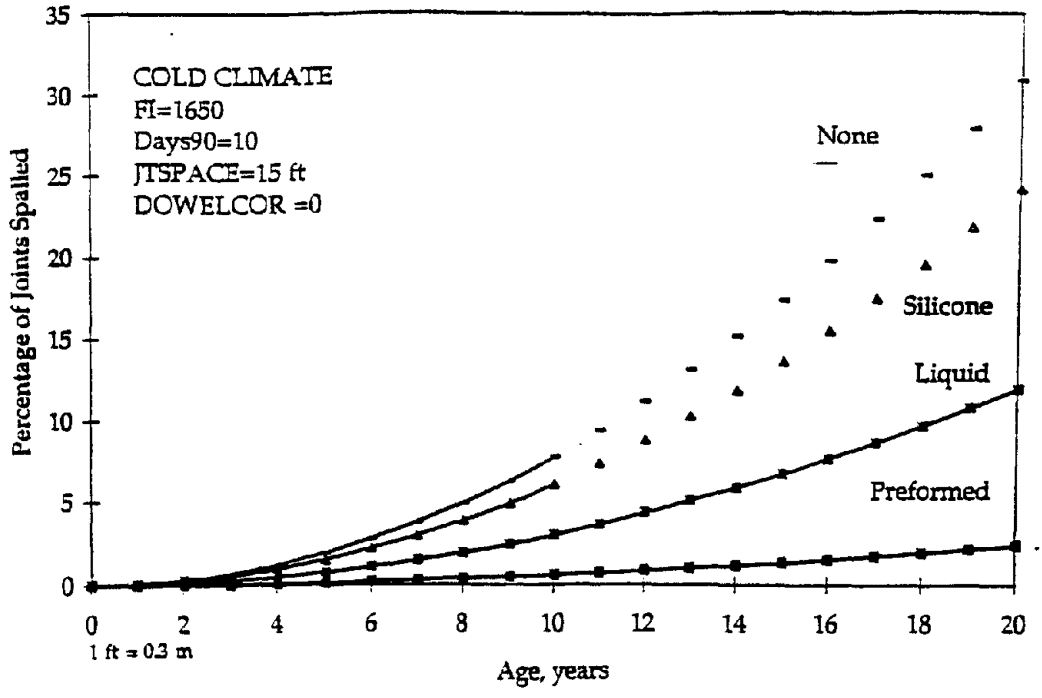



Figure 87. Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 1650).[66]

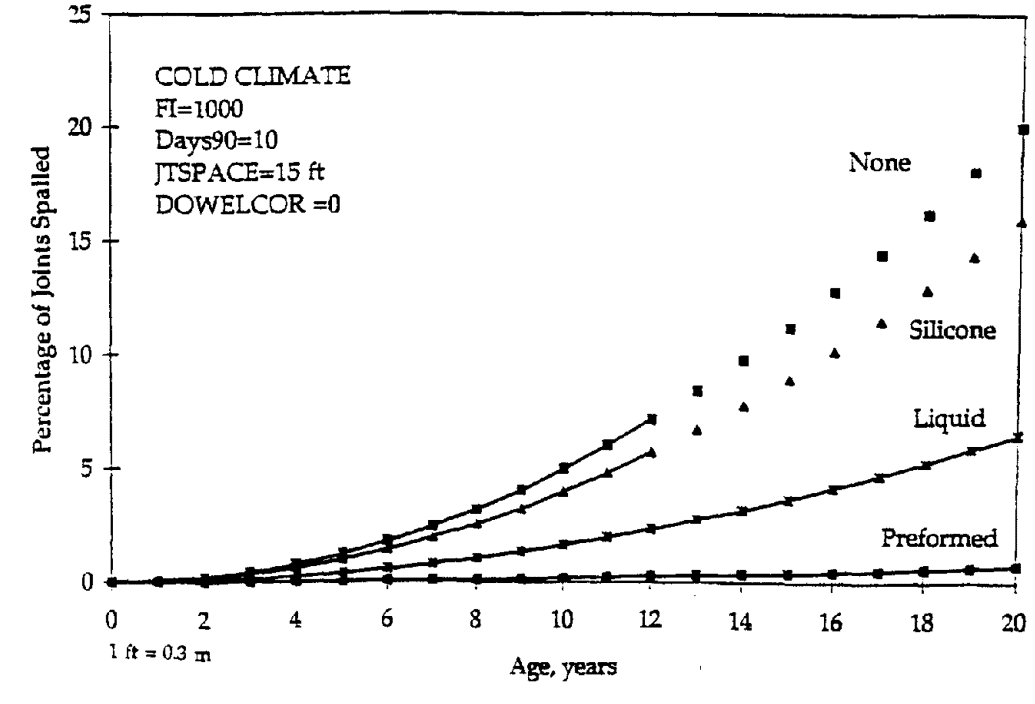



Figure 88. Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 1000).[66]

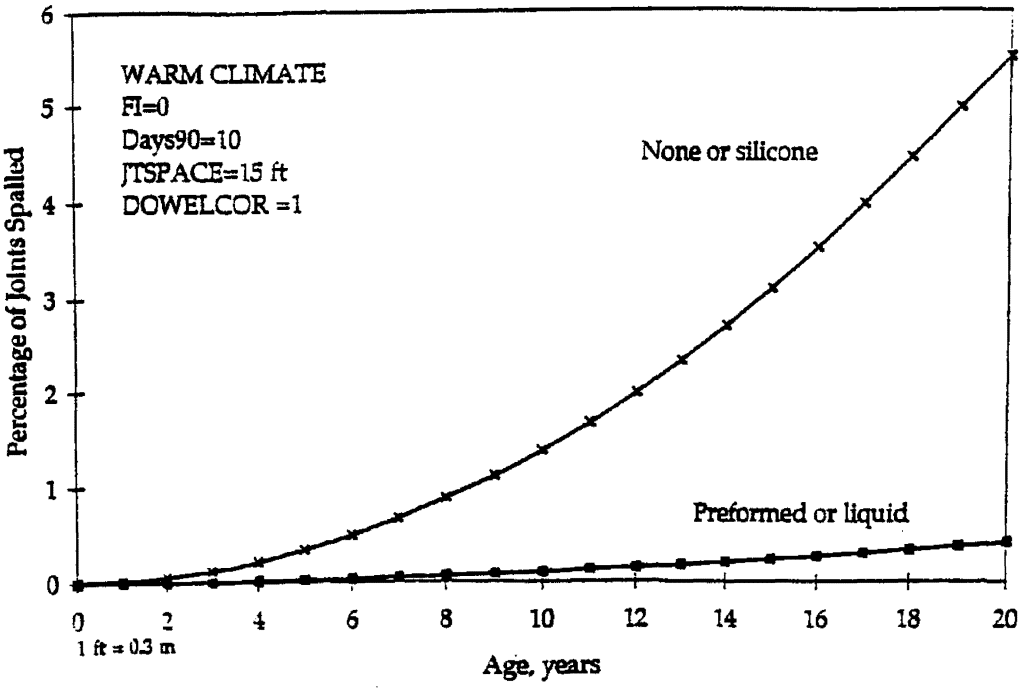



Figure 89. Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 0, Days90 = 10).[66]

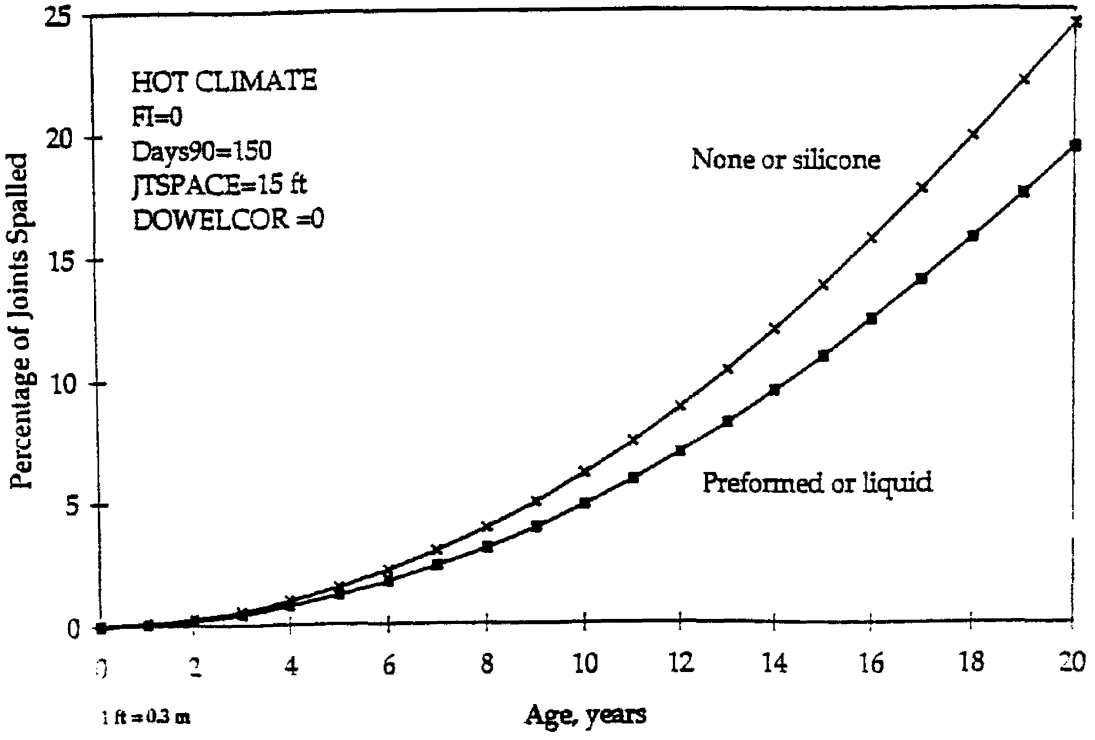



Figure 90. Sensitivity of the JPCP transverse joint spalling base model to age, sealant type, and climatic conditions (FI = 0, Days90 = 150).[66]

variables, Days90 and FI, at constant typical values for cold, warm, and hot regions. It is assumed that no dowel corrosion exists, but similar trends were observed for sections with dowel corrosion. Each of these plots shows that the amount of joint spalling (and the rate of the increase in spalling) always increases with age.

Figure 91 shows results of the sensitivity analysis of age and potential dowel corrosion on tranvserse joint spalling. This plot illustrates the effect of dowel corrosion in a cold climate (FI = 500, Days90 = 10). The plot shows that dowel corrosion significantly increases the percentage of spalled joints. Therefore, corrosion protection of dowels is very important.

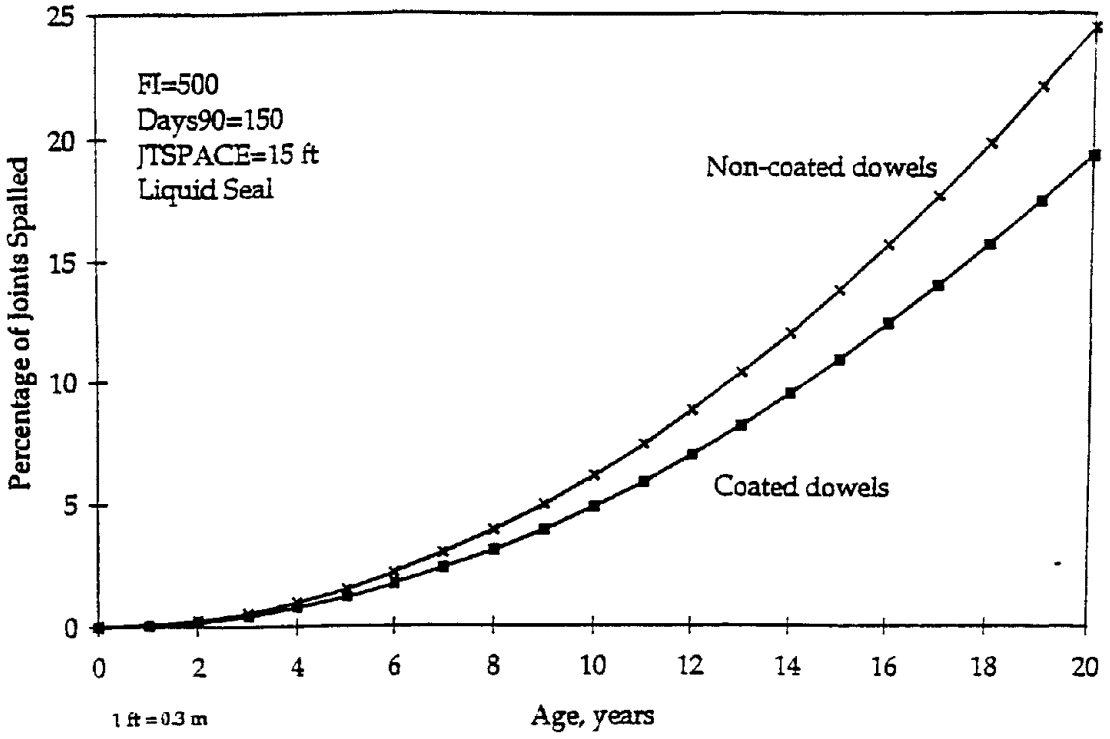



Figure 91. Sensitivity of the JPCP transverse joint spalling base model to age and dowel protection.[66]

As summarized in the report by Yu et al., the three-dimensional plots in figures 92 through 95 show the effect of the interaction between climate conditions and pavement age on the performance of each type of sealant. The number of days with temperatures above 90 °F (32 °C), Days90, is fixed at 10, and the variation of FI from 0 to 2000 represents variation in climate from mild to cold. The percentage of spalled joints increases with age and freezing index. It is predicted that significant spalling of sealed joints does not begin until about 10 or 15 years after construction, but rapidly increases after that, especially in cold climates. This effect can be explained by the increase in the number of freeze-thaw cycles to which the pavements are exposed.[66]

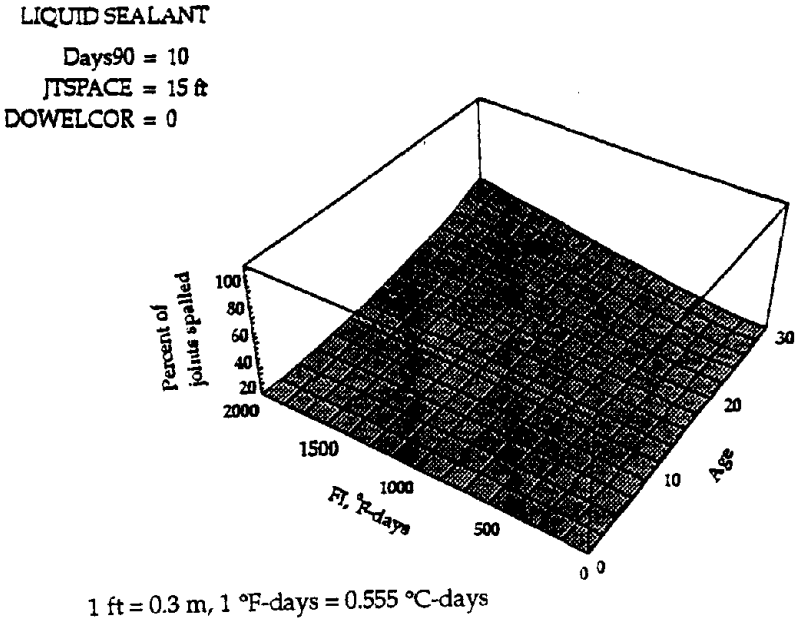



Figure 92. Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (liquid sealant).[66]

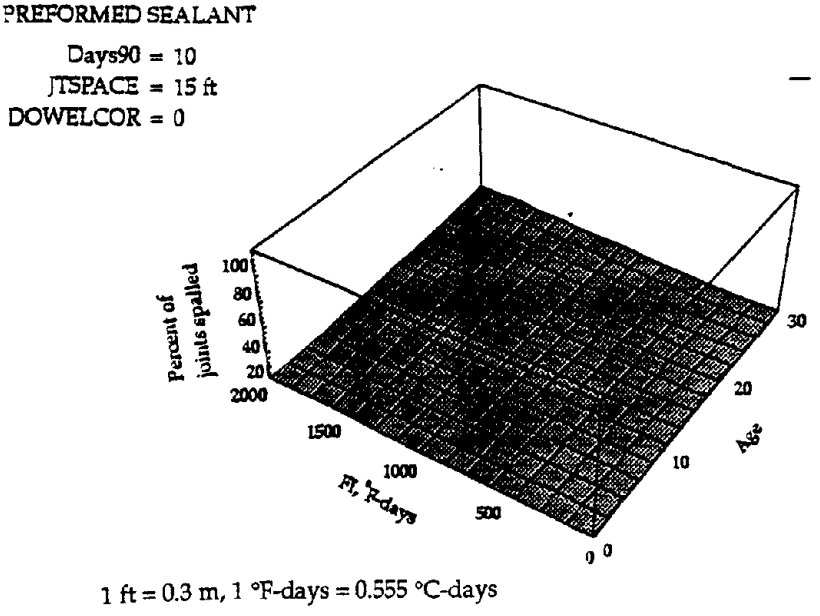



Figure 93. Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (preformed sealant).[66]

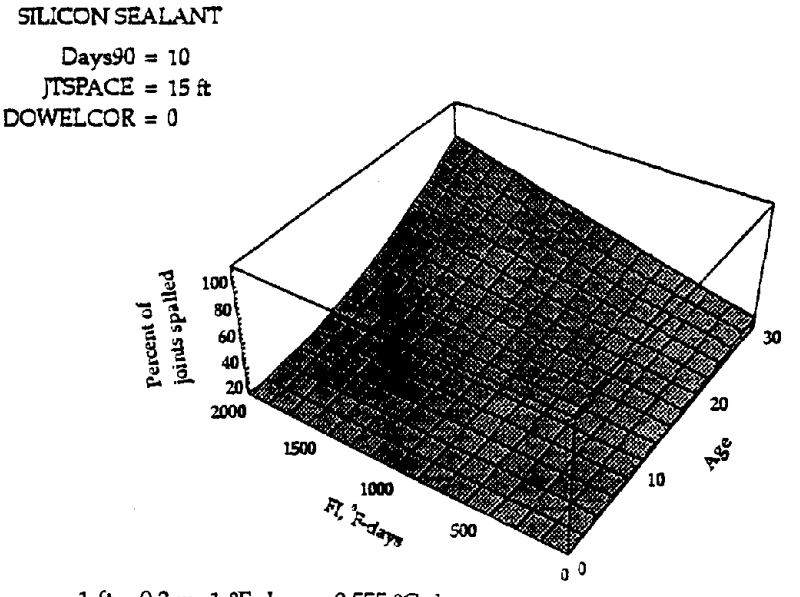



Figure 94. Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (silicone sealant).[66]

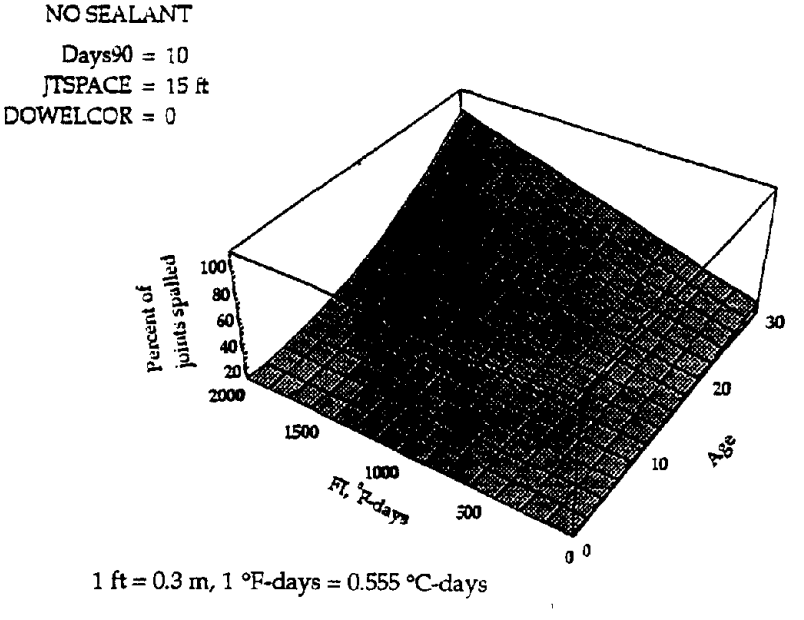



Figure 95. Sensitivity of the JPCP transverse joint spalling base model to age and freezing index (no sealant).[66]

Based on the results provided by the JPCP joint spalling model, the following conclusions were drawn in the report by Yu et al.:[66]

- Tranverse joint spalling increases with the pavement age to the second power.
- Significant transverse joint spalling does not begin until about 10 or 15 years after construction.
- Preformed joint sealant reduces transverse joint spalling more effectively than the other sealant types considered.
- Transverse joints without sealant exhibit the largest amount of spalling.
- Silicone sealants perform worse than liquid sealants, presumably because of the silicone pulling on the sidewalls of the young concrete before it has gained sufficient strength.
- An increase in joint spacing increases the percent of joints spalled.
- The amount of spalling is greater in very cold climates and in very hot climates, but lower in moderate climates.
- Appropriate protection of dowels from corrosion (e.g., epoxy coating) reduces joint spalling.

**Transverse Joint Faulting**

Two different procedures are used to estimate transverse joint faulting, depending on whether percent consolidation around dowels is considered. Regardless of the procedure chosen, the base transverse joint faulting model is that developed by Yu et al. in 1998.[67] If percent consolidation around dowels is not considered, transverse joint faulting is predicted directly using the base model. If percent consolidation around dowels is considered, the transverse joint faulting values (predicted using the base model) are then altered by adjusting the AGG term of the model. The procedures making up both situations are discussed in detail in the following sections. (Note: The model outputs and inputs are presented in English units.)

Base Transverse Joint Faulting Model Characteristics

*Model Source*—Yu, H.T., L. Khazanovich, S.P. Rao, M.I. Darter, and H. Von Quintus, *Guidelines for Subsurface Drainage Based on Performance, Appendices.*[67]

*Model Output Units*— Mean transverse joint faulting, in.

*Dependency on Other Model Outputs*—None.

Transverse Joint Faulting (Not Including Percent Consolidation Around Dowels)

$$FAULT = DAMAGE^{0.2475} * [0.2405 - 0.00118 * Days90 + 0.001216 * WETDAYS - 0.04336 * BASETYPE - (0.004336 + 0.007059 * (1 - DOWEL)) * LCB] \quad (32)$$

where

FAULT = Average transverse joint faulting for JPCP, in.

BASE = Base type (0 if nonstabilized or lean concrete base [LCB]; 1 if stabilized with asphalt [ATB] or cement [CTB]).

LCB = Lean concrete base (1 if lean concrete base, 0 otherwise).

WETDAYS = Average number of wet days per year.

Days90 = Number of days per year with the maximum temperature greater than 90 °F (32 °C).

DOWEL = Presence of dowels (0 if no dowels, 1 otherwise).

DAMAGE = $n_{MILLIONS}/N_{MILLIONS}$

$n_{MILLIONS}$ = *Actual* number of applied cumulative ESAL's, millions.

$N_{MILLIONS}$ = *Allowable* number of applied cumulative ESAL's, millions.

Intermediate variables include the following:

$$\text{Log(N)} = 0.785983 - 0.92991 * (1 + 0.40 * \text{PERM} * (1 - \text{DOWEL})) * \text{Log (DE)} \quad (33)$$

and

$$\text{AGG*} = (\text{AGG/kL}) = 2.3\ \text{Exp}(1 - 1.987 * \text{JTSPACE} / \text{L} + 3.48 * \text{DOWELDIA}^{3.56}) \quad (34)$$

where

AGG* = Nondimensional aggregate interlock stiffness.
k = Modulus of subgrade reaction.
L = Slab's radius of relative stiffness.
JTSPACE = Slab length (joint spacing), ft.
DOWELDIAM = Dowel diameter, in.
PERM = Base permeability (0 = not permeable, 1 = permeable).

Procedure for Using the Transverse Joint Faulting Model (Not Including Percent Consolidation Around Dowels)

To determine the transverse joint faulting if percent consolidation around dowels is not included, the following parameters must be known:

$h_{PCC}$ = Slab thickness, in.

$E_{PCC}$ = PCC modulus of elasticity, psi.

$\mu$ = PCC Poisson's ratio (assumed to be equal to 0.15).

k = Modulus of subgrade reaction, psi/in.

DOWELDIAM = Dowel diameter, in.

JTSPACE = Joint spacing (ft), if joint spacing is not greater than 40 ft (12.2 m); 40 otherwise.

BASE = Base type (0 if nonstabilized or lean concrete base [LCB]; 1 if stabilized with asphalt [ATB] or cement [CTB]).

LCB = 1 if lean concrete base, 0 otherwise.

PERM = Base permeability (0 = not permeable, 1 = permeable).

WIDENLANE = Widened lane (0 = not widened, 1 = widened).

Next, the model is executed using the following step-by-step procedure.

1. Calculate the radius of relative stiffness (L) using equation 35.

$$L = [(E_{PCC} * h_{PCC}) / (12 * (1 - \mu^2) * k)]^{0.25} \quad (35)$$

2. Calculate AGG* using equation 34.

3. Calculate the maximum corner deflections of the loaded and unloaded slabs assuming that the 80-kN ESAL is located 305 mm from the corner (914 mm from the corner if widened slab is used). The following series of steps are used:

   a. Use equation 36 to calculate the free-edge corner deflection (no load transfer to the adjacent slab) assuming that the load is placed at the slab corner.

$$w_{fe,0} = (0.0000864 * L^2 + 0.002824 * L + 0.2953) * 18000/kL^2 \quad (36)$$

   b. Use equation 37 to calculate the free-edge corner deflection (no load transfer to the adjacent slab) assuming that the load is placed at the transverse joint 914 mm away from the slab corner.

$$w_{fe,914} = (0.0000648 * L^2 + 0.003934 * L - 0.02548) * 18000/kL^2 \quad (37)$$

   c. Use equation 38 to calculate the corresponding LTE for a load placed at the slab corner.

$$LTE_{x=0}(\%) = 1 / [0.01 + 0.012 * (AGG^*)^{-0.849}] \quad (38)$$

d. Use equation 39 to calculate the corresponding LTE at the slab corner for a load placed 914 mm away from the slab corner.

$$LTE_{x=914}(\%) = 1 / [0.01 + 0.003483 * (AGG^*)^{-1.13677}] \quad (39)$$

e. Calculate the corner deflection of the unloaded slab for a load placed at 0 and 914 mm from the slab corner using equations 40 and 41, respectively.

$$w_{UL,0} = w_{fe,0} * (LTE_{x=0})/(1 + LTE_{x=0}) \quad (40)$$

$$w_{UL,91.4} = w_{fe,91.4} * (LTE_{x=91.4})/(1 + LTE_{x=91.4}) \quad (41)$$

f. Use equation 42 to calculate the corner deflection of the unloaded slab for an axle load located at a transverse joint in the wheelpath (305 mm from the slab corner for a non-widened slab).

$$w_{UL} = w_{UL,0} + (w_{UL,914} - w_{UL,0}) * x/914 \quad (42)$$

This equation interpolates between the unloaded slab deflections at x = 0 and 914 mm, where x is the distance from the wheelpath to the slab corner. The variable "x" is calculated as the additional width provided by the widened lane plus 305 mm (this sum is limited to 914 mm). For example, if the chosen widened lane width is 4.3 m, and the regular lane width is 3.7 m, the added width is 609 mm. The variable "x" is then calculated as 609 mm + 305 mm to give x = 914 mm.

g. Use equation 43 to calculate the LTE at the slab corner for an axle load located at the transverse joint in the wheelpath (305 mm from the slab corner for a non-widened slab).

$$LTE = LTE_0 + (LTE_{914} - LTE_0) * x/914 \quad (43)$$

This equation interpolates between the LTE's computed for x = 0 and 914 mm.

h. Use equation 44 to calculate the loaded corner deflection for the load located in the wheelpath.

$$w_L = w_{UL} / LTE \quad (44)$$

i. Use equation 45 to calculate the differential energy density, DE.

$$DE = 0.5 * k * (w_L^2 - w_{UL}^2) \quad (45)$$

4. Calculate the allowable number of ESAL applications (N) using equation 33.

5. Calculate predicted faulting for both the as-designed and as-constructed cases using equation 32.

Procedure for Using the Transverse Joint Faulting Model (Including Percent Consolidation Around Dowels)

The only difference between the procedure including percent consolidation and the case without percent consolidation is the change in the value used for the AGG* term. For the case including percent consolidation around dowels, the following procedure is used to alter the AGG* and redefine it as $AGG^*_{NEW}$. (Note: The altered $AGG^*_{NEW}$ is then used in all of the subsequent calculation steps.) The following is the updated step-by-step procedure using $AGG^*_{NEW}$.

1. Calculate the radius of relative stiffness (L) using equation 35.

2. Calculate AGG* using equation 34.

3. Calculate $AGG^*_{NEW}$ using equation 46.

$$AGG^*_{NEW} = 10^{[\text{Log}(AGG^*) + 0.27 * (\%CON - 100)]} \quad (46)$$

4. Calculate the maximum corner deflections of the loaded and unloaded slabs assuming that the 80-kN ESAL is located 305 mm from the corner (914 mm from the corner if widened slab is used). The following series of steps are used:

   a. Use equation 36 to calculate the free-edge corner deflection (no load transfer to the adjacent slab) assuming that the load is placed at the slab corner.

   b. Use equation 37 to calculate the free-edge corner deflection (no load transfer to the adjacent slab) assuming that the load is placed at the transverse joint 914 mm away from the slab corner.

   c. Use equation 47 to calculate the corresponding LTE for a load placed at the slab corner.

$$LTE_{x=0}(\%) = 1 / [0.01 + 0.012 * (AGG^*_{NEW})^{-0.849}] \quad (47)$$

   d. Use equation 48 to calculate the corresponding LTE at the slab corner for a load placed 914 mm away from the slab corner.

$$LTE_{x=914}(\%) = 1 / [0.01 + 0.003483 * (AGG^*_{NEW})^{-1.13677}] \quad (48)$$

e. Calculate the corner deflection of the unloaded slab for a load placed at 0 and 914 mm from the slab corner using equations 40 and 41, respectively.

f. Use equation 42 to calculate the corner deflection of the unloaded slab for an axle load located at a transverse joint in the wheelpath (305 mm from the slab corner for a non-widened slab). This equation interpolates between the unloaded slab deflections at x = 0 and 914 mm, where x is the distance from the wheelpath to the slab corner.

g. Use equation 43 to calculate the LTE at the slab corner for an axle load located at the transverse joint in the wheelpath (305 mm from the slab corner for a non-widened slab). This equation interpolates between the LTE's computed for x = 0 and 914 mm.

h. Use equation 44 to calculate the loaded corner deflection for the load located in the wheelpath.

i. Use equation 45 to calculate the differential energy density, DE.

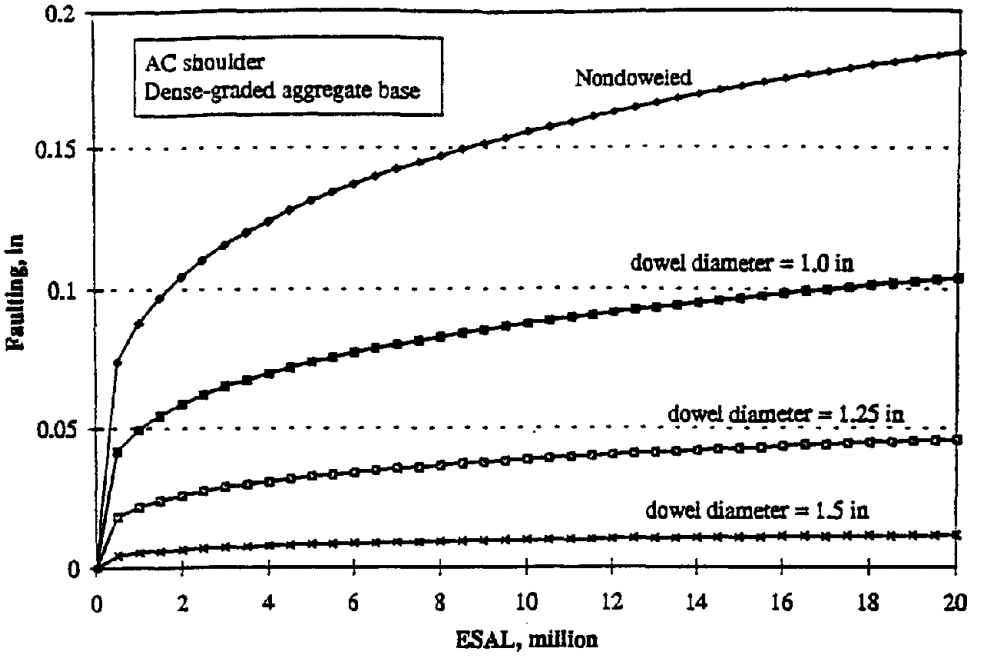
5. Calculate the allowable number of ESAL applications (N) using equation 33.

6. Calculate predicted faulting for both the as-designed and as-constructed cases using equation 32.

Sensitivity of Transverse Joint Faulting Base Model

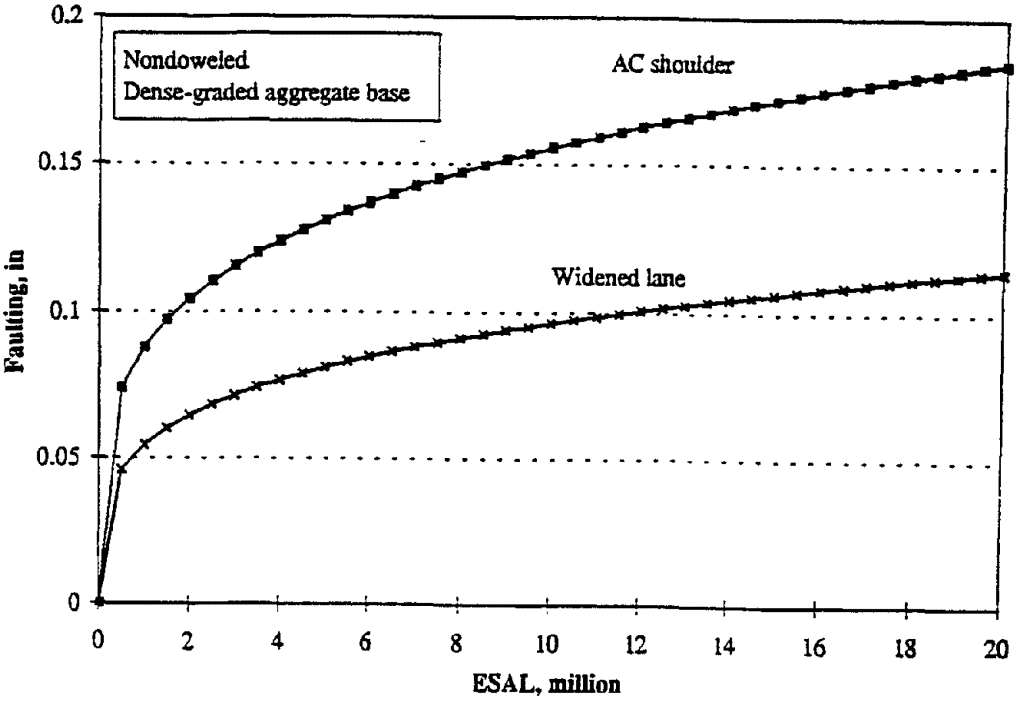
A sensitivity analysis of the transverse joint faulting base model was conducted in the study by Yu et al.[67] The results of this sensitivity analysis are provided in this section. Each plot is developed by fixing all other variables at a constant value while varying the selected variables over practical ranges. (Note: All of the sensitivity plots are presented in English units.)

Figure 96 shows the sensitivity of the joint faulting model to cumulative ESAL's and dowel diameter. The chart shows that dowels decrease faulting and faulting decreases with an increase in dowel diameter. Figures 97 and 98 show that widened lanes decrease faulting for both doweled and nondoweled pavements. Figure 99 shows the effects of traffic and base permeability on the faulting of nondoweled pavements. Figures 100 and 101 show the sensitivity of the faulting model with respect to changes in climatic conditions. Figure 102 shows the sensitivity of the joint faulting model to cumulative ESAL's and percent consolidation around dowels.




1 in = 25.4 mm

Figure 96. Sensitivity of the JPCP transverse joint faulting base model to traffic and dowel diameter.[67]




1 in = 25.4 mm

Figure 97. Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and widened lanes.[67]

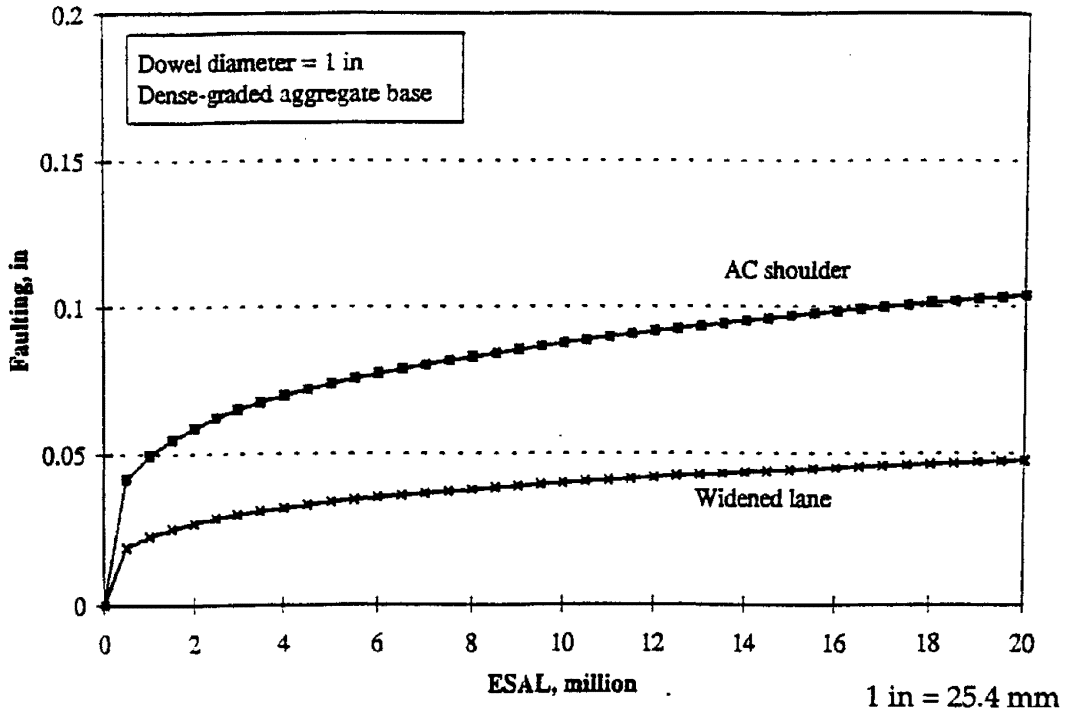



Figure 98. Sensitivity of the JPCP transverse joint faulting base model (for doweled joints) to traffic and widened lanes.[67]

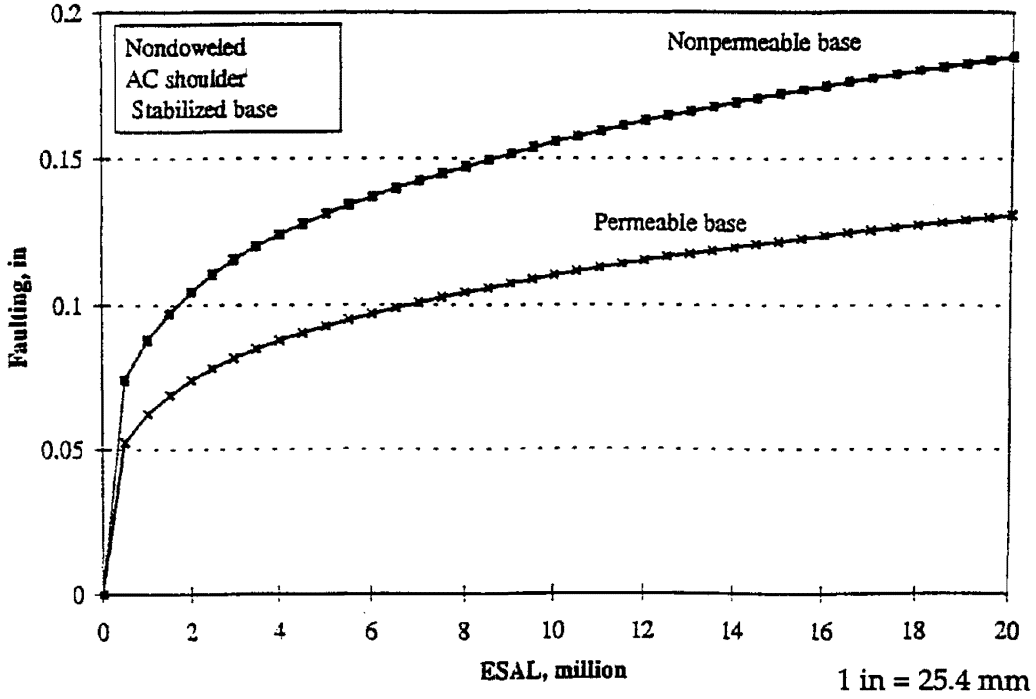



Figure 99. Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and base permeability.[67]

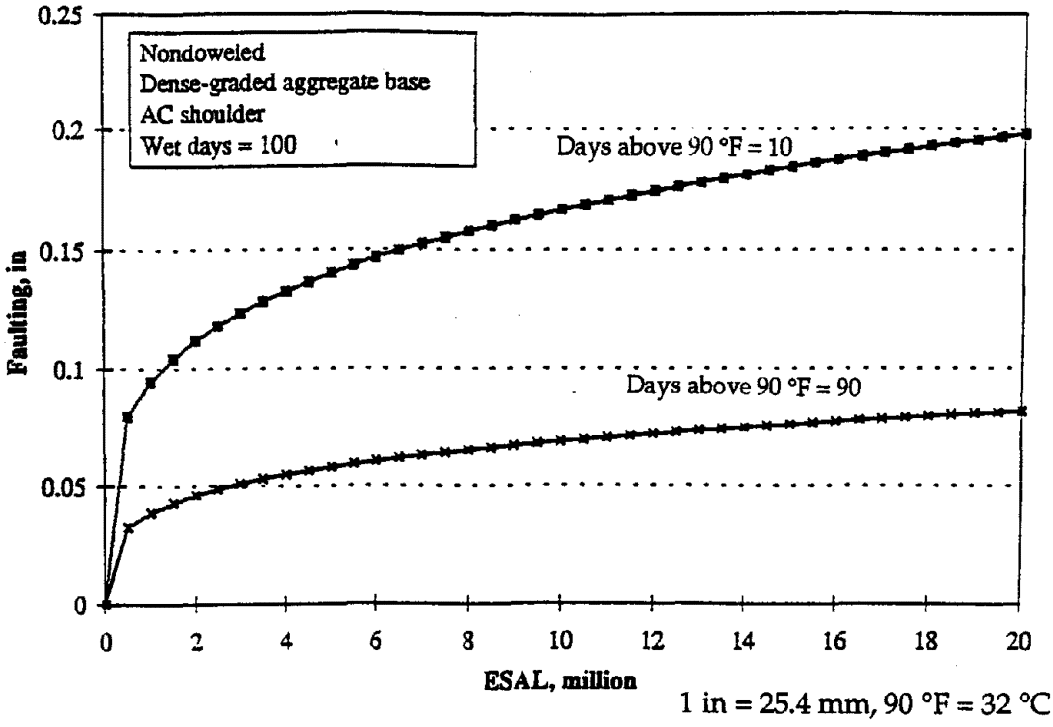



Figure 100. Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and number of days above 32 °C.[67]

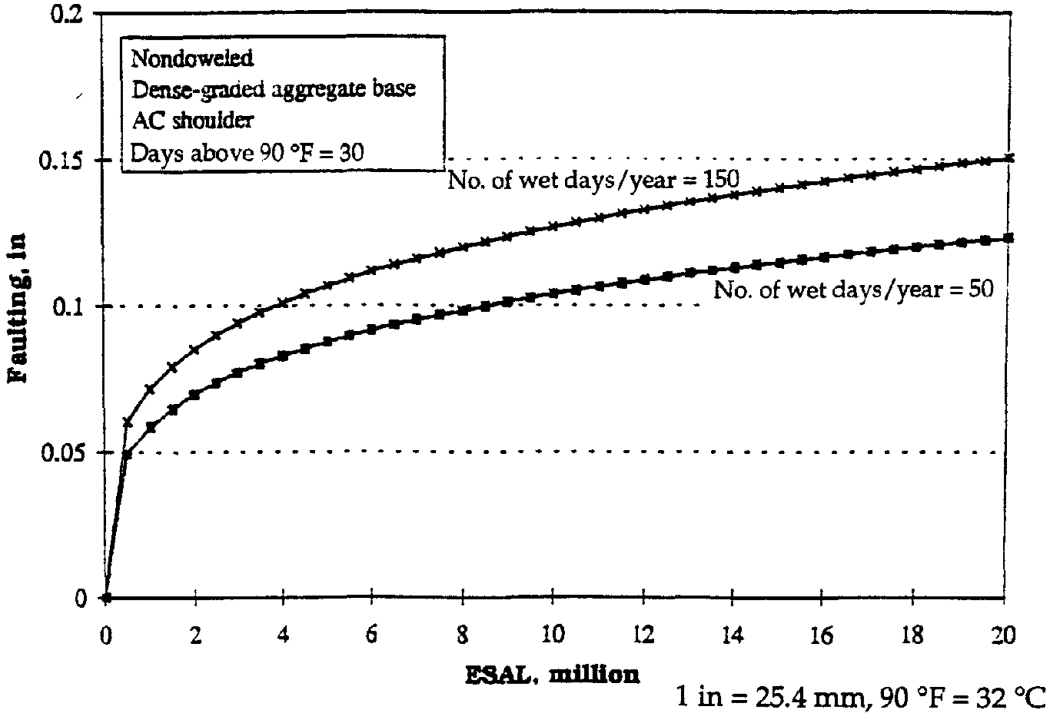
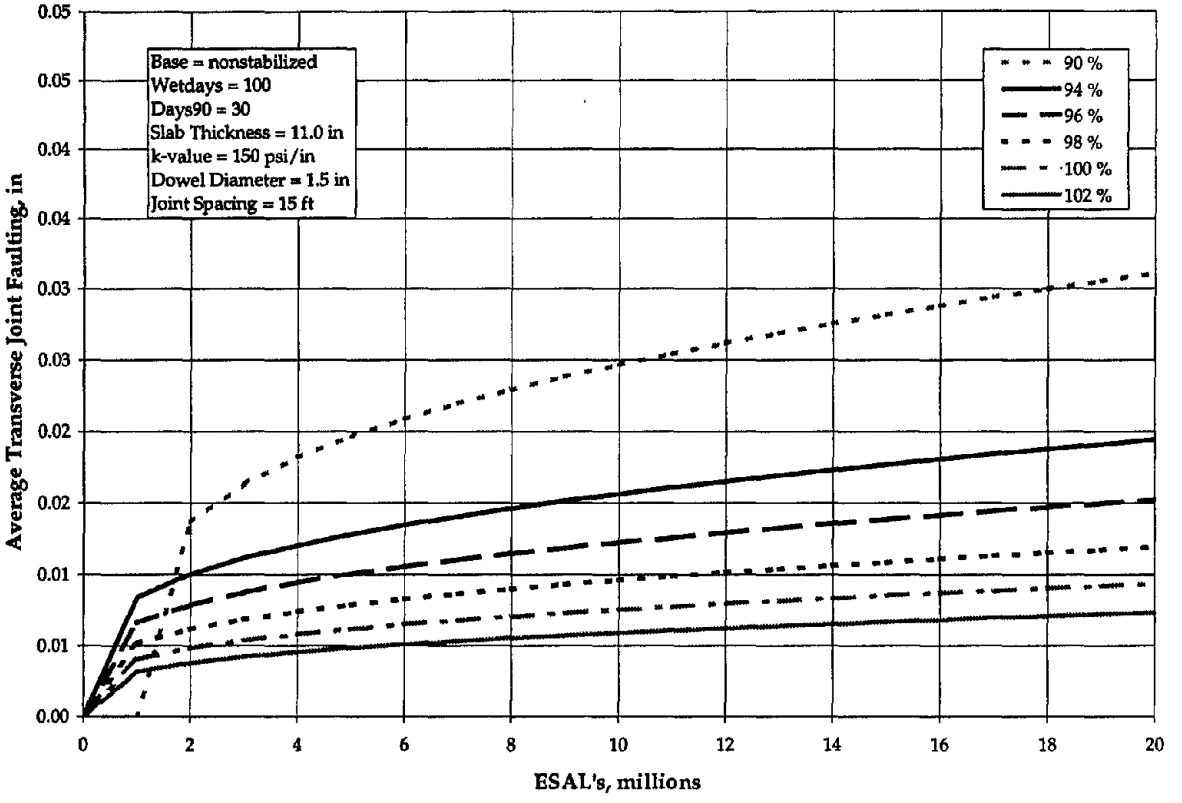



Figure 101. Sensitivity of the JPCP transverse joint faulting base model (for nondoweled pavements) to traffic and number of wet days.[67]




1 in = 25.4 mm, 1 ft = 0.305 m, 1 psi/in = 2.7 kPa/cm

Figure 102. Sensitivity of the JPCP transverse joint faulting base model to traffic and percent consolidation around dowels (faulting versus ESAL's for different percentages of consolidation around dowels).

**Present Serviceability Rating (PSR)**

When developing a PRS, the user may choose to predict smoothness over time in terms of PSR. PSR is predicted using the following model developed by Yu et al. in 1997.[66] (Note: The model outputs and inputs are presented in English units.)

Present Serviceability Rating Model Characteristics

*Model Source*—Yu, H.T., M.I. Darter, K.D. Smith, J. Jiang, and L. Khazanovich, *Performance of Concrete Pavements, Volume III—Improving Concrete Pavement Performance.*[66]

*Model Output Units*—Present serviceability rating (rating between 0 and 5).

*Dependency on Other Model Outputs*—Function of transverse slab cracking, transverse joint faulting, transverse joint spalling, and longitudinal cracking.

Present Serviceability Rating Model

$$PSR = 3.95 - 0.010276 * FaultTT - 0.001014 * T\text{-}Crack - 0.009421 * Spall - 0.003911 * L\text{-}Crack^{0.5} \quad (49)$$

where

PSR = Present serviceability rating (rating between 0 and 5).

FaultTT = Total joint faulting per mile, in/mi.
= (FAULT) * Integer(5280/JTSPACE)

T-Crack = Total number of transverse crack, number/mi.
= (%CRAKD/100) * Integer(5280/JTSPACE)

Spall = Percentage of joints spalled.
= %SPALL (used directly)

L-Crack = Longitudinal cracking, ft/mile (assumed to equal zero).

Therefore, the final usable form of this model becomes:

$$PSR = INITPSR - 0.010276 * [(FAULT) * Integer(5280 / JTSPACE)] - 0.001014 * [(\%CRAKD / 100) * Integer(5280 / JTSPACE)] - 0.009421 * [\%SPALL] \quad (50)$$

where

PSR = Present serviceability rating (rating between 0 and 5).

INITPSR = Initial present serviceability rating (rating between 0 and 5). Estimated as a function of initial profile index.

FAULT = Average transverse joint faulting for JPCP, in.

JTSPACE = Mean transverse joint spacing, ft.

%CRAKD = Percentage of slabs cracked (transverse cracking).

%SPALL = Percentage of medium- and high-severity spalled joints.

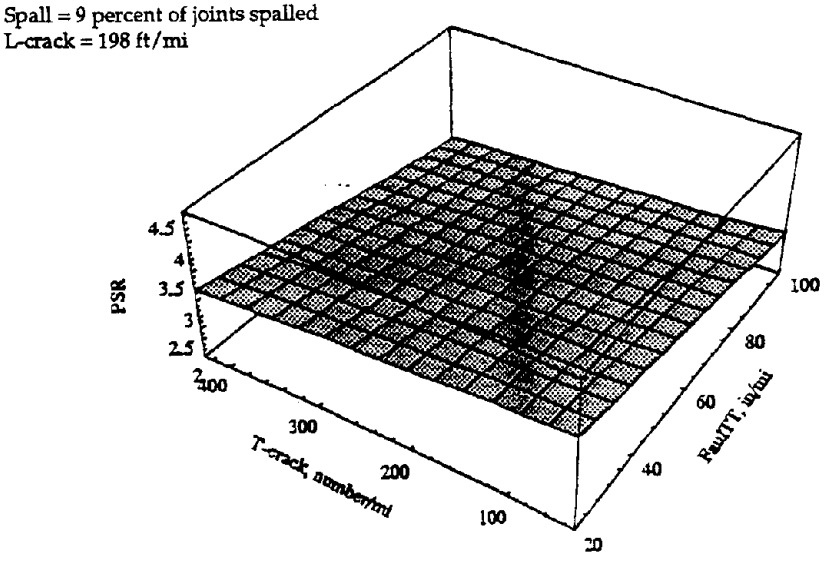
Note that the coefficient of 3.95 (in equation 49) has been replaced with an initial PSR variable "INITPSR." The user must define a correlation between initial PSR and initial profile index (measured using a 5.1-mm blanking band) in the PaveSpec computer software.

Procedure for Using the PSR Model

The PSR is predicted for both the as-designed and as-constructed pavements. The first step in either case is to predict values for all of the other distress indicators at each year. Next, these predicted distress indicator values are used in equation 50 to predict yearly values for PSR. This procedure is repeated using both as-designed and as-constructed pavement characteristics.

Sensitivity of PSR Model

A sensitivity analysis of the PSR model was conducted in the study by Yu et al.[66] Figures 103 through 106 show the sensitivity of the PSR model to changes in the other predicted distress indicators. (Note: All of the sensitivity plots are presented in English units.)

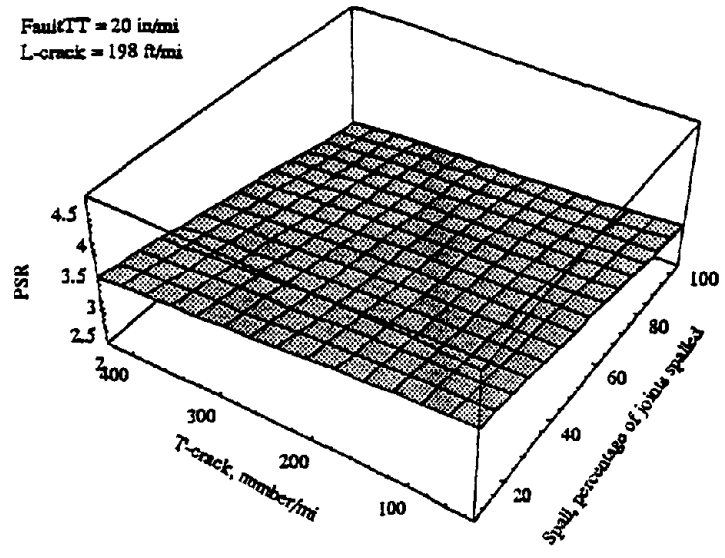



1 mi = 1.6 km; 1 in = 25.4 mm; 1 ft = 0.3 m

Figure 103. Sensitivity of the JPCP PSR model to transverse joint faulting and transverse slab cracking.[66]

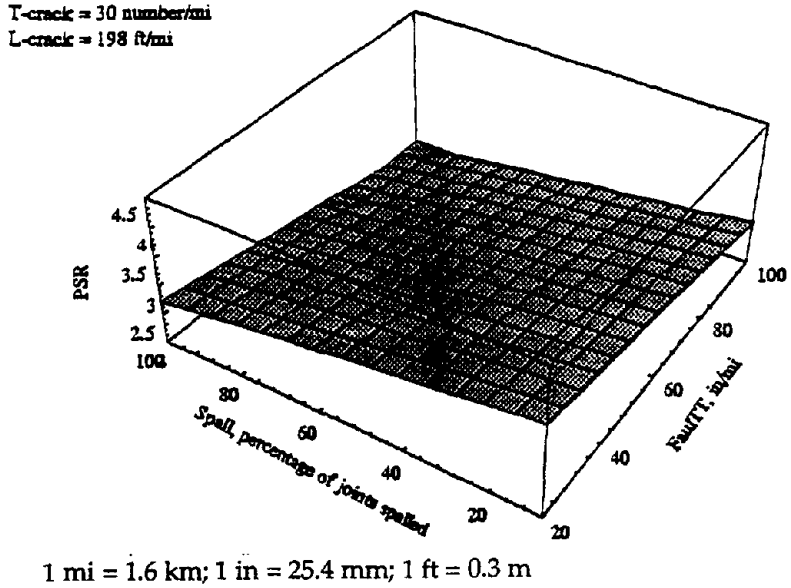
**International Roughness Index (IRI)**

The current PRS approach allows pavement smoothness to be expressed in terms of IRI. A new IRI prediction regression model was developed under the current research project based on data obtained from the Long Term Pavement Performance (LTPP)



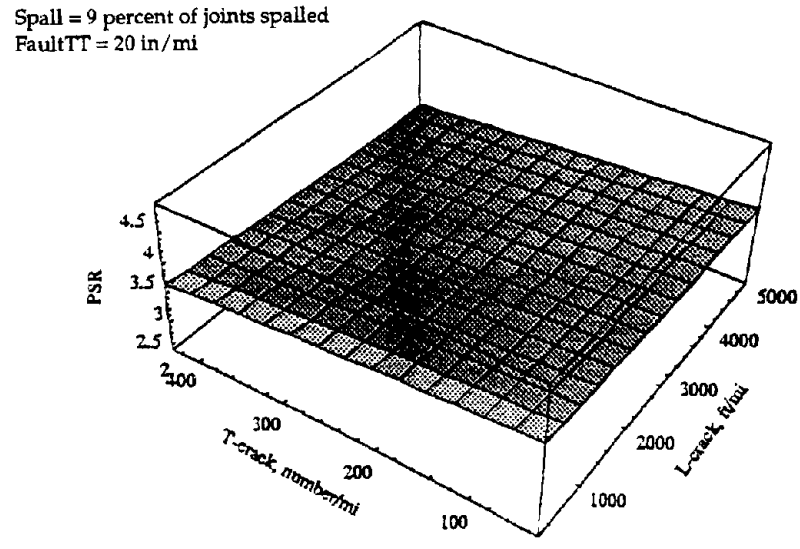

1 mi = 1.6 km; 1 in = 25.4 mm; 1 ft = 0.3 m

Figure 104. Sensitivity of the JPCP PSR model to transverse slab cracking and transverse joint spalling.[66]




1 mi = 1.6 km; 1 in = 25.4 mm; 1 ft = 0.3 m

Figure 105. Sensitivity of the JPCP PSR model to transverse joint faulting and transverse joint spalling.[66]




1 mi = 1.6 km; 1 in = 25.4 mm; 1 ft = 0.3 m

Figure 106. Sensitivity of the JPCP PSR model to transverse slab cracking and longitudinal cracking.[66]

performance database. Details of the developed model are provided in the following sections. (Note: The model outputs and inputs are presented in English units.)

International Roughness Index Model Characteristics

*Model Source*—Developed under the current PRS research project.

*Model Output Units*—International roughness index, in/mi.

*Dependency on Other Model Outputs*—Function of transverse slab cracking, transverse joint faulting, transverse joint spalling.

International Roughness Index Model

$$IRI = INITIRI + (4.8389 * Age) - (0.047 * Age * INITIRI) + (0.7165 * \%CRAKD) + (359.956 * FAULT) + (0.3828 * \%SPALL) \quad (51)$$

where

IRI = International roughness indicator, in/mi.
INITIRI = Initial IRI, in/mi.
Age = Number of years since construction, years.
%CRAKD = Percentage slabs cracked.

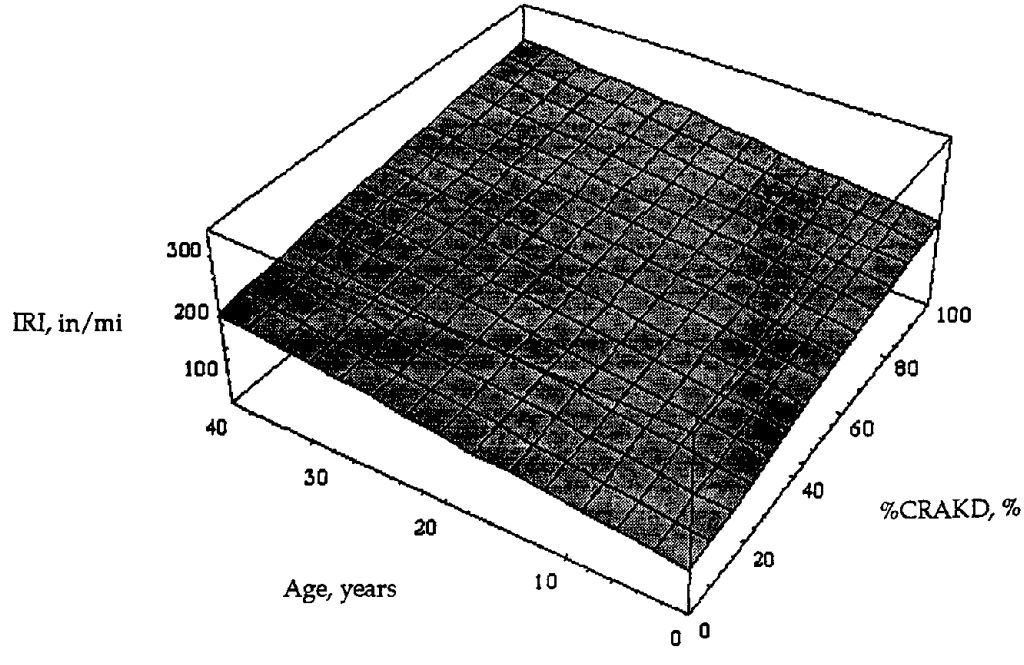
FAULT = Average joint faulting per joint, in.
%SPALL = Percentage of medium- and high-severity spalled joints.

Procedure for Using the IRI Model

The IRI is predicted for both the as-designed and as-constructed pavements. First, values for all of the other distress indicators are predicted at each year. Next, these predicted distress indicator values are used in equation 51 to predict yearly values for IRI. This procedure is repeated using both as-designed and as-constructed pavement characteristics.

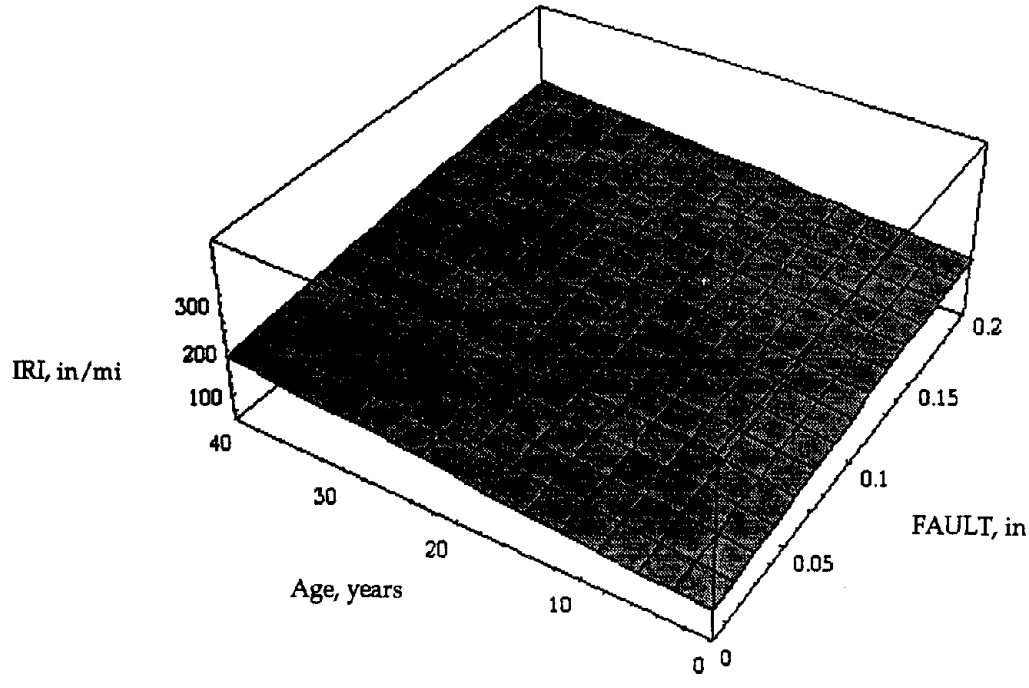
Sensitivity of IRI Model

A sensitivity analysis of the IRI model was conducted under the current study. Figures 107 through 109 show the sensitivity of the IRI model to changes in the other predicted distress indicators. (Note: All of the sensitivity plots are presented in English units.)



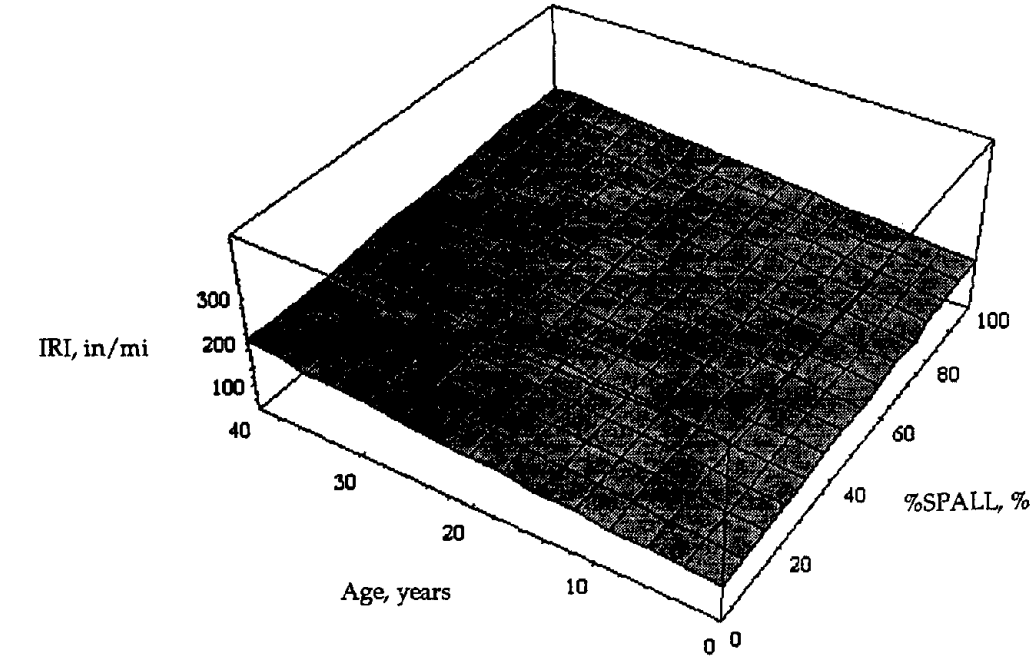

1 in = 25.4 mm, 1 mi = 1.61 km; 1 in/mi = 15.8 mm/km

Figure 107. Sensitivity of the JPCP IRI model to age and transverse slab cracking.




1 in = 25.4 mm, 1 mi = 1.61 km; 1 in/mi = 15.8 mm/km

Figure 108. Sensitivity of the JPCP IRI model to age and transverse joint faulting.




1 in = 25.4 mm, 1 mi = 1.61 km; 1 in/mi = 15.8 mm/km

Figure 109. Sensitivity of the JPCP IRI model to age and transverse joint spalling.

# APPENDIX F—ANNOTATED BIBLIOGRAPHY

**Abdun-nur, E.A., and R.C. Mielenz, *Scaling of Air-Entrained and Non-Air-Entrained Concrete*, Highway Research Board Special Report No. 106, Highway Research Board, 1970, pp. 53-56.**

Case histories and weather data are presented for four pavement sections constructed during different seasons with the same concrete mixture possessing the same properties obtained from the same ready-mixed concrete supplier. Photographs were taken of areas of the pavements at different stages of scaling, and petrographic core examinations were made. Area 80 constituted well-air-entrained concrete, but the topmost section of the pavement was susceptible to disruption by freezing and thawing because of high absorptivity resulting from lack of adequate curing and early drying and carbonation of the cement paste matrix. The action of freezing was aggravated by the presence of deicing salts. Areas 92, 93, and 98 comprised non-air-entrained concrete that was not resistant to freezing and thawing, especially in the presence of deicing salts. Sections constructed with low water–cement ratio concrete were susceptible to rapid disintegration during repeated freezing. Incipient cement–aggregate reactions involving dolomites, chalcedonic cherts, and cherty phases of dolomites occurred, but no deleterious effects were indicated by the petrographic examination. These tests indicate that air entrainment will not ensure adequate durability of exposed concrete in the absence of good concreting practices. The defect leading to failure of the concrete in area 80 was the result of inattention to curing and protection of the concrete from early drying. The defect leading to failure of the pavement surfaces in the other areas was the absence of a proper entrained air void system in the concrete.

**Adkins, D.F., and V.T. Christiansen, "Freeze-Thaw Deterioration of Concrete Pavements," *Journal of Materials in Civil Engineering*, 1(2): 97-104.**

This discussion of how concrete freezes and thaws under natural conditions focuses on the thermal and hydraulic stresses involved in surface scaling. The mechanics of surface scaling are described, and freezing within the paste is discussed, as are temperature and saturation differentials. A series of laboratory and field tests were conducted to establish the validity of the theory that surface scaling could result from freezing temperatures converging upon a zone of greater saturation. Conclusions drawn from the study are presented and discussed.

**Al-Omari, B., and M.I. Darter, *Relationships Between IRI and PSR*, Interim Report, Publication No. UILU-ENG-92-2013, Illinois Department of Transportation, September 1992, 71 pp.**

This report documents the work accomplished on a study to develop relationships between the International Roughness Index (IRI) and the Present Serviceability Rating (PSR) for pavement types included in the Highway Performance Monitoring System

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(HPMS) database (flexible, rigid, and composite pavement types). PSR is defined as the mean user panel rating for rideability.

Relationships between IRI and PSR were analyzed for Louisiana, Michigan, New Jersey, New Mexico, Ohio, and Indiana. Data for all six States were analyzed using a statistical software package, and the following nonlinear model was developed:

$$PSR = 5 * e^{(\alpha * IRI)} \quad (52)$$

Regression analysis was conducted for all possible sets of data considering different States and pavement types. It was determined that there was no significant difference between the models for different States or pavement types; therefore, the following model is recommended:

$$PSR = 5 * e^{(-0.0041 * IRI)} \quad (53)$$

where IRI is in units of in/mi, or:

$$PSR = 5 * e^{(-0.26 * IRI)} \quad (54)$$

where IRI is in units of mm/m.

**Al-Suleiman, T.I., K.C. Sinha, and V.L. Anderson, "Effect of Routine Maintenance on Pavement Roughness," *Transportation Research Record* 1205, Transportation Research Board, 1988, pp. 20-28.**

This paper presents a study of the relationship between routine maintenance expenditure level and pavement roughness. A database by contract section was developed for the State highway system of Indiana. Covariance analysis was performed to test the effect of climatic region. Regression models were developed to examine the effect of routine maintenance expenditure level on rate of change in pavement roughness. Two highway classes and three pavement types were considered in the analysis. The database included a total of 550 pavement contract sections. The results can be used to develop an effective maintenance program.

**American Association of State Highway and Transportation Officials, *Implementation Manual for Quality Assurance*, AASHTO Highway Subcommittee on Construction, 1996, 92 pp.**

This manual presents important considerations for developing and implementing a quality assurance program within a highway agency. Topics discussed include management and industry support, program goals, an implementation plan, and general principles for developing specifications.

**American Association of State Highway and Transportation Officials, *Quality Assurance Guide Specification*, AASHTO Highway Subcommittee on Construction, 1996, 46 pp.**

This document is a companion to the *Implementation Manual for Quality Assurance.* It contains procedures and requirements for sampling and testing, validation, construction requirements, quality control, and acceptance of newly constructed pavements.

**American Association of State Highway and Transportation Officials, *Standard Recommended Practice for Acceptance Sampling Plans for Highway Construction*, AASHTO Designation: R 9-90.**

This recommended practice provides guidance in the preparation of statistical acceptance plans for transportation materials and construction items. The procedures outlined are based on percent defective (or percent within limits) as the quality parameter. The various discussions, tables, and examples in this guideline assume that the acceptance procedures will be used to accept or reject the work. Although it is logical to extend this methodology to the development of acceptance procedures with adjusted pay schedules, that application is not addressed in this guideline.

**American Concrete Institute, *Guide for Construction of Concrete Pavements and Concrete Bases*, Publication No. ACI 325.9R-91, December 1991, 27 pp.**

This report is intended as a guide for the construction of concrete pavements and concrete bases. References are made to specifications, but only as a guide to enable a selection of requirements suitable for a particular location or class of work. The intended users of this guide are agencies other than State highway departments or turnpike and airfield authorities. The report covers topics such as sampling and testing of materials for quality assurance; subgrade or subbase preparation and forms; installation of joints and reinforcement; high-early-strength concrete; mixing, placing, finishing, and curing concrete; and cold and hot weather concreting.

**American Concrete Institute Committee 214, *Guide for Obtaining Cores and Interpreting Compressive Strength Results*, May 1995, 44 pp.**

Core testing is the most direct method for determining the compressive strength of concrete in a structure. Generally, cores are obtained to assess whether suspect concrete in a new structure complies with strength-based acceptance criteria, or to evaluate the structural capacity of an older structure based on the in-place concrete strength. In either case, the process of obtaining core specimens and interpreting the strength test results is often confounded by various factors that affect either the in-place strength of the concrete or the observed strength of the test specimen. The scatter in strength test data, which is inevitable given the inherent randomness of in-place concrete strengths

and the additional uncertainty attributable to the preparation and testing of the specimen, may further complicate compliance and evaluation decisions.

This guide summarizes current practices for obtaining cores and interpreting concrete strength test results. Factors that affect the in-place concrete strength are reviewed so that locations for sampling can be selected that are consistent with the objectives of the investigation. Strength correction factors, which account for the size and condition of the core test specimen, permit the strengths observed on non-standard specimens to be converted to the strength of equivalent standard specimens. Methods for checking strength compliance of concrete in new and old structures are summarized.

**American Concrete Institute Committee 325, *Accelerated Concrete Paving Techniques*, Draft Report, October 1995, 59 pp.**

This report covers the state of the art of accelerated concrete paving techniques, often referred to as "fast-track" concrete paving. Accelerated concrete paving techniques are appropriate for roadways, airfields, and other paved surfaces where quick access is required. Accelerated concrete paving considerations include planning, concrete materials and properties, jointing and joint sealing, curing and temperature control, concrete strength testing, and opening-to-traffic criteria. Applications and uses of accelerated concrete paving are discussed.

**Amsler, D.E., and J.E. Bryden, *Effects of the Clary Screed and Tube Float on Rigid Pavement Construction*, Publication No. NYSDOT-ERD-75-RR-32, New York State Department of Transportation, August 1975, 24 pp.**

Effects of the Clary screed and the tube float (used as finishing devices on portland cement concrete pavement placed with a slipform paver) were examined on six construction projects. Alternate sections were finished with and without the Clary screed on three projects. Sections making up the remaining three projects were finished with different numbers of passes (0, 2, 4, or 6) with a tube float. Pavement roughness was measured with a California profilograph, and surface mortar samples were tested for compressive strength. The screed (used without additional water) resulted in a reduction of roughness and was not considered detrimental to the surface mortar. The tube float (normally used with a fog-spray of water) had little effect on roughness and resulted in a substantial reduction in the compressive strength of the surface mortar.

**Anderson, S.M., and R.L. Carrasquillo, *The Effects of Withholding Mixing Water and Retempering on Properties of Concrete*, Interim Report, Publication No. FHWA/TX-88+1117-1, Texas State Department of Highways & Public Transportation, February 1988, 128 pp.**

An experimental program examined the effects of withholding mixing water at initial batching (followed by retempering of the mix at the job site) on the properties of the

concrete produced for highway applications. Additionally, the effects of redosage with more water than that specified in the mix design were examined. Tests were performed to determine the effects on slump, air content, unit weight, compressive strength, flexural strength, abrasion resistance, and freeze-thaw resistance. The effects of varying the withholding amount, withholding time, and cement content on the fresh and hardened concrete properties mentioned above were examined. The concrete examined was produced at a ready-mix concrete facility to duplicate (as closely as possible) job site conditions arising in typical concrete construction. The results of the study show that significant detrimental effects occur when mixing water is withheld and concrete is retempered at a later time. Slump, air content, abrasion resistance, and freeze-thaw resistance are all adversely affected. The effects were found to vary with variations in both withholding time and cement content. The strength was not affected when water was withheld and concrete was retempered, but a reduction in strength accompanied an increase in water-cement ratio above design values at redosage. The changed properties lead to concrete with reduced quality and questionable performance.

**Ansari, F., "In-Place Air Content of Concrete by Fiber Optics,"** ***Strategic Highway Research Program Products: Proceedings of a Specialty Conference Sponsored by the Highway Division of the American Society of Civil Engineers and the Federal Highway Administration,*** **April 1991, pp. 41-43.**

A new fiber-optic apparatus to determine the air content of freshly mixed-in-place concrete is described. Such a device may eventually be used to facilitate acceptance or rejection in the field. The results of this research included the following:

- A methodology for the rapid and nondestructive monitoring of concrete air void content during mixing, transporting, and placement.
- Development of a hand-held portable device based on the developed methodology.
- Laboratory and field experimentation of the new device, as well as a comparison of results with conventional testing methods.

**Ardani, A.,** ***PCCP Texturing Methods,*** **Interim Report, Publication No. CDOT-DTD-R-95-2, Colorado Department of Transportation, February 1995.**

This report describes the testing and construction details of nine test sections with varying textural characteristics. The impacts of various surface textures (sawed versus tined, longitudinal versus transverse, and random versus uniform spacing) on the frictional and noise characteristics of pavement surface were examined. Skid numbers were acquired at different combinations of tire type (ribbed and smooth) and speed (64.4, 80.5, and 104.6 km/h) for all the test sections. Six different types of texture measuring devices were used to compare the amount of texture in each test section. To examine the noise properties of the test sections, noise data were acquired in three

different locations: (1) inside the test vehicle, (2) 7.62 m from the center line, and (3) near the right rear tire of the test vehicle (away from the exhaust pipe).

**Armstrong, J.P., "Accelerated Strength Tests for Quality Control of Concrete Pavements," *Proceedings: Fourth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, Publication No. FHWA-RD-89-208, April 1989, pp. 519-530.**

This paper presents the results of a laboratory-based effort to evaluate potential methods of obtaining flexural strength for pavement quality control using accelerated test methods (within 24 h). Current U.S. Army Corps of Engineers pavement design procedures specify 28- and 90-day flexural strengths for roads and airfields, respectively. Quality control of pavement concrete strengths use correlated 7- and 14-day flexural strengths to identify problems as early as possible; however, compliance with specifications still depends on the 28- and 90-day strengths. These lengthy test times are not adequate to identify pavement strength problems, particularly with today's rapid construction techniques. A literature review indicates that accelerated test methods employing elevated curing temperatures are available for evaluating compressive strength (ASTM C 684), but comparable procedures for the more sensitive flexural strength have not been published or accepted. Some success has been achieved in predicting long-term flexural strength using the boiling method (ASTM C 684, Procedure B). More recent work indicates that a warm water curing method (ASTM C 684, Procedure A) is safer, may be more reliable, and causes fewer deleterious effects to the specimen; therefore, it was pursued in this study.

To determine the most effective 1-day measure of concrete quality, accelerated (24 h in an elevated temperature water bath) and 14-day flexural, splitting tensile, and compressive strengths were compared to 28- and 90-day flexural strengths. Three concrete mixtures were prepared by varying water-cement ratios to provide a wide strength range. Ten batches of each mixture were cast into beams and cylinders (a total of 1,061 specimens) for testing. Based on the coefficient of determination ($R^2$) for regression equations, the warm water method ($R^2$ = 0.894 to 0.945) provides reliability equal to, if not greater than, the 14-day tests ($R^2$ = 0.785 to 0.920) in predicting 28- and 90-day flexural strength.

**Ayton, G., J. Cruickshank, E. Haber, and H. Richard, *Concrete Pavement Manual: Design and Construction*, Edition 2, Revision 3, New South Wales Roads and Traffic Authority, 1994.**

This manual documents the Australian experience with concrete pavement construction. It discusses factors that are both external (e.g., traffic, climate) and internal (e.g., thickness, construction quality) to the pavement in order to demonstrate the importance of construction standards and structural detailing on the performance of concrete pavements. According to the authors, construction standards and structural

detailing account for all but a few of the premature failures in pavements constructed in New South Wales in the past 15 years.

**Bartlett, F.M., and J.G. MacGregor, "Equivalent Specified Concrete Strength From Core Test Data," *Concrete International*, March 1995, pp. 52-59.**

The purpose of this paper is to present a simple procedure for determining a value of concrete strength from core strength data to be used for structural evaluation. In practice, the number of cores obtained for testing is usually limited by budgetary constraints and, in some cases, by concern about unduly weakening the structure. It is convenient for the resulting strength value to be equivalent in definition to the specified strength of concrete, $f'_c$.

**Bock, B.T., and P.A. Okamoto, "Evaluation of Dowel Placement Using a Dowel Bar Inserter," *Proceedings: Fourth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, Publication No. FHWA-RD-89-208, April 1989, pp. 641-649.**

This paper presents the results of several field investigations to evaluate the effectiveness of automatic dowel bar inserter equipment. The results summarize three studies in which ground-penetrating radar (GPR) was used to measure dowel bar misalignment. The studies used a commercially available radar system capable of locating steel embedded in concrete. The system produces a real-time graphic recording that indicates the horizontal position and relative depth of the embedded steel. Cores were obtained to calibrate the graphic recordings by obtaining the actual embedded depth of the steel bars. In two of the three studies, GPR was used to measure the alignment of dowel bars placed using Guntert and Zimmerman dowel bar inserters, as well as those placed on wire basket assemblies. Overall, the dowel bar inserters performed well compared to the basket assemblies. Dowel depth, misalignment (vertical and horizontal), and longitudinal displacement are generally comparable.

**Burati, J.L., Jr., M.G. Beeson, and H.S. Hill, Jr., "Comparative Analysis of Dowel Placement in Portland-Cement Concrete Pavements," *Transportation Research Record 924*, Transportation Research Board, 1983, pp. 19-27.**

No statistically valid proof has been found to indicate that dowel bar alignment is better or worse when the basket assembly or implanted dowel placement method is used. Also, no significant difference has been identified in the amount of joint-related distress caused by the two methods. There is no evidence that the joint-related distress on the projects studied can be directly linked with the type of dowel misalignment studied, because alignment error does not appear to be the sole determining factor of the distress. Statistical tests and comparison procedures were employed to analyze collected alignment data, and attempts were made to relate the alignment results to

distress data. Conclusions were drawn indicating that neither placement method appears to be superior, but statistical proof could not be established. Many uncontrollable variables, such as work crew difference, equipment difference, and inspection, may be as important to dowel alignment as the examined variables. Alignment data regarding dowel depth were unobtainable due to the electronic detection method of data collection employed and a prohibition against coring. Further research should address this limitation.

**Burg, G.R.U., "Slump Loss, Air Loss, and Field Performance of Concrete," *American Concrete Institute Journal*, 80(4): 332-339.**

Field and laboratory experience demonstrates that when concrete loses slump, there is generally a loss of entrained air content. Results obtained from tests made in conjunction with a large concrete roadway and parking lot paving project may give the answer to the often-asked question, "Is slump loss due to the loss of air, or is air loss due to the loss of slump?" The purpose of these tests was to study the following:

- The accuracy of current recommendations for durable concrete pavement.
- The history of air-entrained pavement-type concrete as it matures from the time it was freshly mixed to after it has been in service for several years.
- The degree of confidence that can be expected when using standard ASTM test methods to obtain certain selected concrete qualities.

Tests were made on randomly selected truckloads of freshly mixed concrete at three different times: at the central mix plant (as mixed), at the job site (as received), and at the job site (as retempered).

**Chamberlin, W.P., *Performance-Related Specifications for Highway Construction and Rehabilitation*, Synthesis of Highway Practice 212, National Cooperative Highway Research Program, 1995, 48 pp.**

The objectives of this synthesis are to identify current use of PRS by highway agencies and to describe the development and present status of PRS for highway materials and construction.

With the exception of the New Jersey Department of Transportation's specifications for portland cement concrete (PCC) strength and its experimental specifications for PCC pavement, no examples or operational PRS that conform to the definition adopted for this study were identified among highway agencies. Further, it was concluded from questionnaire responses that although the concepts of PRS are well-understood within the research community, they have not been communicated effectively to the highway construction community at large.

Continuation of PRS development at the national level and parallel efforts at local levels are recommended to increase awareness of PRS concepts and encourage the development of experimental specifications. Immediate preparation of educational and instructional materials on current and planned adjustable payment acceptance plans and their operating characteristics is also recommended as a means of addressing the issue of their fairness to both owners and contractors.

**Cole, L.W., and P.A. Okamoto, "Flexural Strength Criteria for Opening Concrete Roadways to Traffic," *Transportation Research Record 1478*, Transportation Research Board, 1995, pp. 53-61.**

Various criteria have been used for opening concrete roadways to traffic. Most criteria are based on the accumulated judgment of specifying agencies or other authorities. Little or no engineering analysis exists to substantiate most opening-to-traffic criteria currently in use. On the basis of Miner's hypothesis of accumulated fatigue, flexural strength opening criteria are presented for concrete roadways (municipal and highway) subjected to construction and public traffic. The criteria are appropriate for new construction, reconstruction, and concrete overlays except bonded concrete overlays.

***Concrete Pavement Construction - Inspection at the Paving Site*, Publication No. EB085.01P, Portland Cement Association, 1980, 19 pp.**

Methods for producing high-quality, durable, smooth-riding concrete pavements are described, and the responsibilities and duties of the inspector at the paving site are outlined. Tips for good inspection at the paving site are presented.

**"Concrete Pavement Joints: Should They Be Warranteed?" *Better Roads*, 53(8): 26-28.**

Because most concrete pavement failures are joint-related, many design engineers, public works agencies, and governmental agencies would welcome joints warranteed for several years with free replacement of failures. A few engineering departments now get such warranties for up to 10 years. Implementation of a warranteed contract that avoids joint problems requires cooperation between design engineer, contractor, and materials supplier. This article discusses the various factors and requirements involved.

**"Construction Practices and Inspection Pavements. Recommendations for Construction of Concrete Pavements and Concrete Bases," *ACI Manual of Concrete Practice 1990. Part 2.*, Publication No. ACI 316R-82, American Concrete Institute, 1990, 26 pp.**

This recommended practice covers construction of concrete pavements and concrete bases without attempting to include inflexible specifications for procedures, materials, or equipment. References are made to specifications, but only as a guide to enable a selection of requirements suitable for a particular location or class of work. The

document is slanted to some degree toward use by agencies other than State highway departments or turnpike authorities, which have presumably large and experienced engineering staffs. Sections are devoted to specifying, sampling, and testing materials, and to the possible influence of materials on skid resistance, economy, and durability. Subgrades and subbases are treated only as a final preparation for paving. Recommendations for forms are included, as are recommendations for projects using slipforms. Arrangement of joints is described, and references are given for guidance in using reinforcement. Sections on normal and high-early-strength concrete proportioning rely heavily on reference to the report of ACI Committee 211, but point out the special problems connected with pavement concretes and the use of admixtures.

**Cumbaa, S.L., "Correlation of Profile-Based and Response-Type Roughness Devices for Louisiana's Highway Performance Monitoring System," *Transportation Research Record 1260*, Transportation Research Board, 1990, pp. 99-105.**

Relationships were developed to meet and facilitate roughness reporting and calibration requirements of the Highway Performance Monitoring System (HPMS) in Louisiana. Pavement roughness statistics obtained from a Face Dipstick, K. J. Law Model 8300 Roughness Surveyor, and Mays Ride Meter were correlated to enable Louisiana to satisfy these requirements. On the basis of the results of this research and previously established relationships between the Mays Ride Meter and the AASHO serviceability index (SI), the international roughness index (IRI) and SI were compared for flexible and rigid pavements. A correlation was established between the IRI values obtained with the Face Dipstick and those obtained with the Model 8300 Roughness Surveyor for five flexible and four rigid pavement test sections. Correlations from field testing of the Model 8300 Roughness Surveyor and Mays Ride Meter on 20 flexible and 19 rigid test sections resulted in a distinct relational equation for each pavement type. However, results relating IRI and SI indicated that this relationship was the same for all pavement types.

**"Curing of Concrete Pavements," *Transportation Research Circular*, Issue 208, Transportation Research Board, June 1979, 11 pp.**

The *Current Road Problem Series* was developed by the Highway Research Board during World War II as a series of emergency pamphlets. The Committee on Curing of Concrete Pavements prepared one such pamphlet, first published in 1942 as *Wartime Road Problems No. 1*, which contained the then-current recommended practices for curing concrete pavements. It was revised in October 1952 as *Current Road Problems No. 1-R, Curing of Concrete Pavements*, which also contained suggested specifications and test methods. It was again revised and updated in May 1963 and titled *Current Road Problems No. 1-2R, Curing of Concrete Pavements*. The current publication updates the recommended practices and specifications of the 1963 edition.

**Darter, M.I., "Variations of Concrete Properties Related to Rigid Pavement Design and Performance," Term Report for CE 393, University of Illinois, 1972.**

The purpose of this paper was to summarize the types and causes of variations of concrete properties, to obtain an estimate of their probability frequency distribution and expected standard deviations, and to discuss the design and specification implications. Since concrete properties such as modulus of elasticity and strength have a significant effect on concrete pavement performance, any variability in these factors will affect the performance of the pavement. A probabilistic design approach (as opposed to a deterministic approach) makes it possible to consider the means and relative variations of design factors in the design process. To use a probabilistic approach, the designer must know the type, distributions, and magnitudes of these uncertainties. This paper attempts to relate the types and causes of concrete pavement variations to aggregates, paste, proportioning, construction, and environment. Measured variations were reported for compressive, flexural, and tensile strength; modulus of elasticity; fatigue; joint load transfer; serviceability index; and slab thickness. The normal magnitude of these variations was found to have a considerable effect on pavement performance, and the variations should be considered quantitatively in pavement design.

**Darter, M.I., K.D. Smith, and D.G. Peshkin, "Field-Calibrated Mechanistic-Empirical Models for Jointed Concrete Pavements," *Transportation Research Record 1307*, Transportation Research Board, 1991, pp. 143-153.**

Field-calibrated mechanistic-empirical models have been developed for key performance indicators of jointed concrete pavements. Performance data from nearly 500 in-service pavements were used along with mechanistic and empirical variables to develop improved prediction models for joint faulting, slab cracking, joint spalling, and present serviceability rating. The models should prove extremely valuable in checking the performance capabilities of various pavement designs determined by other means and in determining the relative impact of different design variables on concrete pavement performance. However, the models must be used with care and applied judiciously because it is imperative that they not be extended beyond the conditions for which they were developed.

**Darter, M.I., M. Abdelrahman, P.A. Okamoto, and K.D. Smith, *Performance-Related Specifications for Concrete Pavements, Volume I: Development of a Prototype Performance-Related Specification*, Publication No. FHWA-RD-93-042, Federal Highway Administration, November 1993, 219 pp.**

This study continued the development of performance-related specifications for concrete pavements. Drawing upon previous work, a prototype PRS was developed that considers the expected life-cycle cost of the as-constructed pavement as the overall measure of quality. The approach calls for measurement of in situ concrete properties and explicitly considers variability and multiple quality characteristics in the

determination of pay adjustments. Extensive laboratory testing was conducted to determine material relationships needed in the prototype PRS, and a detailed test plan was developed for the evaluation of construction variables that significantly affect concrete pavement performance, but are not currently accounted for in the specification. A computer program, PaveSpec, was developed for use with the specification in simulation and in generating pay adjustments.

This volume describes the development of the prototype PRS. Concrete strength, slab thickness, entrained air content, and initial smoothness are included in the specification as the key quality characteristics. Both cost models and distress prediction models are used to compute life-cycle costs. The difference between the life-cycle costs of the target, as-designed pavement and the actual, as-constructed pavement is used to determine the pay adjustment. Numerous examples on the use and sensitivity of the specification are presented. A summary of the laboratory testing results that were used in the specification is given, along with a test plan, for the evaluation of quality characteristics not currently included in the specification.

**Darter, M.I., M. Abdelrahman, T. Hoerner, M. Phillips, K.D. Smith, and P.A. Okamoto, *Performance-Related Specifications for Concrete Pavements, Volume II: Appendix A—Prototype Performance-Related Specification, Appendix B—PaveSpec Users Guide, and Appendix C—Annotated Bibliography,* Publication No. FHWA-RD-93-043, Federal Highway Administration, November 1993, 177 pp.**

This volume includes supporting documentation for the research study. The prototype PRS is included in its entirety in appendix A. This specification is complete for pilot testing, verification, and validation on simulated and actual construction projects. Appendix B contains a users guide for the PaveSpec computer program, and appendix C presents an annotated bibliography of literature pertinent to construction specifications.

**Demos, M., N.G. Gharaibeh, and M.I. Darter, *Evaluation of Potential Applications of End-Result and Performance-Related Specifications, White Paper Presentations—Part II,* Chapter V—Portland Cement Concrete Pavements, Publication No. FHWA-IL-UI-256, Illinois Department of Transportation, December 1995, 85 pp.**

The primary objective of this study is to evaluate, based on the best available technology, the potential applications of end-result specifications (ERS) and performance-related specifications (PRS) to asphalt concrete paving, portland cement concrete paving, and structural concrete construction. For each area, the following tasks were performed:

- Review of current literature on ERS and PRS.
- Comprehensive evaluation of other State practices to include parameters used, lot size, and payment schedules.

- Review and evaluation of current Illinois DOT specifications, practices, and policies.
- Identification of relevant IDOT data that have potential application.
- Evaluation of the feasibility of relating materials and construction data to facility performance.
- Determination of the quality of available data and their appropriateness for data analysis.
- Statistical analysis of selected variables within available data and determination of their suitability for developing ERS and PRS.

The study is organized into two parts. Part I discusses the basic concepts and the development of ERS and PRS, and quality acceptance plans. Part II discusses the framework of ERS and PRS, current practice, and data analyses for each of the three areas studied. The report also contains a literature review, summary of other State practices, review and evaluation of the IDOT specification, analyses of MISTIC data, potential linking of MISTIC and Illinois Pavement Feedback System data, and recommendations for improvements to current QC/QA specifications for structural concrete construction.

**Dempsey, B.J., W.A. Herlache, and A.J. Patel, *The Climatic-Materials-Structural Pavement Analysis Program, User's Manual*, Publication No. FHWA-RD-84-115, Federal Highway Administration, February 1985, 208 pp.**

This publication is the User's Manual for the Climatic-Materials-Structural Pavement Analysis Program (CMS program), which is described in detail in the report entitled *Volume 3. Environmental Effects on Pavements—Theory Manual* (FHWA-RD-84-115).

The CMS program is a climatic model used to analyze multi-layered flexible pavement systems. It simulates field conditions by accounting for the fact that climatic characteristics (e.g., minimum and maximum air temperature, sunshine, wind velocity, precipitation) vary with geographical location in many States. The climatic model incorporated in the CMS program takes climatic and material inputs and calculates temperature and moisture profiles as they vary with time. This information is then used in the material model to calculate the asphalt, base, subbase, and subgrade stiffness characteristics. This output can then be combined with load data and input into various structural analysis and performance models. It is believed that this type of analysis will give a more reliable indication of pavement performance than will a model that ignores the effects of climate.

**"Development of Pavement Performance Measures in the Contract Equation," *Summary of Workshop Proceedings*, Transportation Research Board, Washington, DC, December 1995.**

The purpose of this workshop was to identify items in the construction contract for which specifications to encourage improved pavement performance may be applicable; to develop a consensus on useful measures of performance; and to set out research needs in measuring performance. The workshop brought together a wide range of individuals from throughout the United States to discuss the current status of performance measures and to help chart a course to advance the state of the practice. The main products of the workshop were the action plans for advancing pavement performance measures and their use as tools in achieving quality, encouraging innovation, and improving efficiency in highway construction programs.

**Devore, J.J. and M. Hossain, *An Automated System for Determination of Pavement Profile Index and Location of Bumps for Grinding From the Profilograph Traces*, Publication No. K-TRAN: KSU-93-2, The Kansas State University, May 1994, 83 pp.**

The objective of this research was to develop an automated system to reduce traces generated by a manual profilograph. The tasks included locating and marking the bumps that would require grinding and determining the profile index of the new riding surface. The objective was accomplished following the guidelines provided in Kansas Test Method KT-46I. This research resulted in a compact and easy-to-use system capable of reducing any profilogram. The program allows the operator to select values for parameters such as bump template height and blanking band width to conform to existing specifications. The KSCAN version uses a hand-held scanner and straight edge guide and is designed to reduce single sections of trace 0.16 km ± 76.2 m in length. The PROSCAN version uses a motorized paper feed and will automatically reduce traces of any length, giving results in 0.16-km sections.

**Dhir, R.K., P.C. Hewlett, E.A. Byars, and J.P. Bai, "Estimating the Durability of Concrete in Structures," *Concrete*, 28(6): 25-30.**

This article discusses the results of an investigation into the assessment of concrete durability. Conclusions of the research include the following:

- The established methods of measuring permeation properties of concrete (absorption, permeability, and vapor diffusion) relate well to one another.

- Concrete durability can be classified in terms of initial surface absorption, but allowance must be made for the binder type when the aspect of durability considered is other than purely physical, such as for chloride diffusion or carbonation.

- Concrete durability can be predicted from mix constituents and their preparation, duration, temperature of moist curing, and initial surface absorption. This gives an opportunity to build further confidence in durability predictions.

- Existing preconditioning techniques for permeation testing are unsuitable for in situ compliance testing. A method of standardizing the moisture content of concrete using a hard vacuum has been developed, applicable over a range of initial moisture contents from dry (28 days air curing) to saturated (28 days water curing).

- Vacuum equipment for preconditioning the moisture content of concrete has been developed as a permeability index test (VPIT) in its own right. The test is nondestructive, is attached by vacuum alone, and is sensitive to the same parameters as other permeation tests.

**Di Cocco, J.B., *Quality Assurance for Portland Cement Concrete*, Publication No. FHWA-RD-73-077, Federal Highway Administration, September 1973, 157 pp.**

Quality assurance has been used successfully in most industries, but the construction industry and State agencies are only now beginning to recognize its importance. The work reported here results from this recognition and is one attempt to show how the elements of quality assurance can be applied to concrete. Chapter I deals with concrete uniformity and compliance with specifications requirements. Under the current degree of process control and inspection, both uniformity and compliance are found to be poor. Chapter II deals with techniques that producers should use to achieve compliance with concrete requirements and includes three case studies to show that process control does indeed lead to compliance. It is also stressed that process control is the responsibility of producers, but recognizing that inspecting agencies (having done most of the testing in the past) may have more pertinent information than producers, guidelines are suggested for producers to use until they accumulate data of their own.

The next four chapters deal with acceptance sampling (inspection). Chapter III discusses the basic concepts of acceptance sampling. In chapter IV, current inspection schemes are reviewed, with particular attention to those used in New York. It is concluded that these schemes afford little protection to concrete buyers, and statistically sound sampling plans are suggested for inspection of fresh and hardened concrete. The advantages and shortcomings of the suggested plans are discussed in chapter V, along with the monetary value of such plans as applied to concrete. Finally, recognizing the trend toward the improper use of acceptance control charts for inspection of construction materials, these tools are reviewed in chapter VI. It is concluded that concrete acceptance control charts are inappropriate and should not be used in concrete inspection.

**Di Cocco, J.B., and P.J. Ballair, *Acceptance Sampling Plans for Rigid Pavement Thickness*, New York State Department of Transportation, Federal Highway Administration.**

In 1967, New York State initiated studies of quality assurance for portland cement concrete. The ultimate goal is to institute a quality assurance program in New York for all concrete properties. While such a program requires extensive preparatory studies for some concrete properties, historical data available for concrete pavements can be used as a first step toward this goal. The shortcomings of the present sampling procedure for inspecting concrete pavement thickness are explained, and two considerably more efficient acceptance sampling plans are developed. These sampling plans take into consideration the producer's and consumer's risks, the nature of the product, its criticality, and the available historical data. Operating characteristic curves are given for both the present inspection procedure and the derived sampling plans.

**Di Cocco, J.B., and W.W. Nuzzo, *Concrete Pavement Thickness Measured by Two Methods*, Publication No. 45-1, New York State Department of Transportation.**

On two paving projects, slab thickness was measured at 25 test locations by coring and by a single manufacturer's ultrasonic method, and the results were compared for precision. First, the difference between the thickness calculated from ultrasonic readings and that measured from the core was determined and compared for each test location. Then, all measurements obtained by each method were grouped and compared for each project. The first comparison showed that differences can be large and may occur often, but nothing could be concluded about relative precision. Comparing the grouped data proved more meaningful, and inferences concerning precision could be drawn. The sonic thickness for each location represents the average thickness of an area of about 2 ft (0.6 m) in diameter. The core represents the average thickness of a much smaller 4-in- (101.6-mm-) diameter area. Thus, core lengths should include more extreme values, and their standard deviations should be larger if the two methods are equally precise. Since the standard deviations of sonic thicknesses were equal to or greater than those of corresponding groups of cores, it is concluded that sonic measurements are less precise than coring. The costs of measuring with the two methods were also compared, and coring was found to be less expensive.

**Dierstein, P.G., *A Study of PCC Pavement Texturing Characteristics in Illinois*, Publication No. FHWA/IL/PR-82-095, Illinois Department of Transportation, February 1985, 59 pp.**

Seven textures (transverse tine, transverse broom, artificial turf, transverse roller, artificial turf/transverse tine combination, longitudinal tine, and longitudinal broom) were formed in the plastic surface of a continuously reinforced concrete pavement. Construction observations indicated that separate machines for texturing and for applying curing compound are preferred since the timing of the two operations is

sometimes incompatible. During texturing, care must be taken to avoid overlapping transverse textures, edge damage, and surface deformation caused by the pressure of the device. Friction tests indicated that grooved textures are superior to broom and artificial turf textures, with the artificial turf/transverse tine combination being the best. A Macrotexture Index, based on both treaded-tire and smooth-tire friction numbers, shows promise as a surrogate texture-depth indicator. The index can determine, with a high degree of certainty, whether a surface has a coarse, medium, or fine texture. Smoothness tests verified that surface texture can influence roughness, and that transverse-grooved textures are rougher than longitudinal textures. Most motorists can easily detect when they are on the transverse roller texture, because its wider and deeper grooves result in a humming noise like that produced by rumble strips; therefore, the transverse roller texture was eliminated as a final finish candidate. During winter storms, natural crosswind and vehicle-generated wind caused blowing snow to collect more on longitudinal tined and artificial turf textures than on the others.

**Donnelly, D.E., *Consolidation of Portland Cement Concrete Pavement Long Term Performance*, Publication No. CDOH-DTP-R-80-1, Colorado Department of Highways, 1980.**

In 1970, three concrete paving projects were selected to evaluate various parameters to improve the consolidation of concrete pavements. Prior to this time, core drilling to verify pavement thickness revealed an unusual number of voids in spite of field attempts to hold the slump at 1 in (25.4 mm). In Colorado, two main types of concrete mixes are used, one containing a fine aggregate mix and the other containing a coarse aggregate. In addition to aggregate size, other parameters investigated included internal vibrators on the paver, vibrator eccentric diameter, speed, angle, height, paver speed, and slump of the mixes.

**Dossey, T., H. Castedo, and B.F. McCullough, "Prediction of Concrete Properties Using Coarse Aggregate Chemical Composition Data," *Transportation Research Record 1284*, Transportation Research Board, 1990, pp. 1-7.**

Regression models that predict the compressive strength, tensile strength, modulus of elasticity, and drying shrinkage of concrete made with several coarse aggregates are presented in this paper. The findings are part of a comprehensive research study for determining the effect of aggregates on the performance of concrete pavements. Using statistical analysis of laboratory test data from concrete samples made with eight commonly used aggregates of known chemical composition, regression models were developed to predict the concrete properties. The predicted values were then compared with the laboratory test results, and the models were used to predict the properties of 11 additional untested coarse aggregates. The predicted concrete properties were within the pavement concrete range reported for these types of concrete mixtures. These prediction models can estimate preliminary information for coarse aggregates or coarse aggregate blends from chemical composition data before casting and testing of concrete

samples. This will allow for initial screening of proposed new sources of coarse aggregate or tentative blends of new or existing aggregate sources of known chemical composition.

**ELE International, Inc. *Micro Covermeter User Manual*, 1994, 12 pp.**

The ELE CT-4950A Micro Covermeter was designed to locate and size reinforcement bars, as well as to determine concrete coverage, making it a useful tool in evaluating in-place concrete structures. This manual contains a brief description of the Micro Covermeter and provides instructions for its use.

**ERES Consultants, Inc. Technical Memorandum for the Draft Report of *Measurement and Specification of Construction Quality: Literature Survey Report for PCC Pavements*, (for FHWA Contract No. DTFH61-94-C-00078), April 1995, 35 pp.**

This memorandum presents the results of a literature search performed as a preliminary step to investigating the levels of variability in PCC pavement construction. The intent of the literature survey was to identify information related to the following eight topics of interest:

- The level of quality in today's construction.
- Which quality measurements can be assumed to follow a normal distribution.
- The presence and quantification of longitudinal, transverse, and vertical variability in construction.
- The effect of erroneously assuming the existence of normally distributed measurements.
- How lot sizes are selected to ensure measurements are normally distributed.
- What current acceptance sample sizes are large enough to provide sufficiently accurate measures of quality.
- The effect of variability on pavement performance.
- Interrelationships between quality characteristics.

***Fast-Track Concrete Pavements*, American Concrete Pavement Association, 1994, 31 pp.**

To build a fast-track project, both the contractor and the agency must make some changes to traditional construction specifications and processes. Often, this entails high early strength (fast-track) concrete, but it can also include revising opening criteria, construction staggering, joint construction, and worker responsibilities. This document discusses background information and details for these changes.

**Fernando E.G., R.S. Walker, and R.L. Lytton, "Evaluation of the Siometer as a Device for Measurement of Pavement Profiles," *Transportation Research Record 1260*, Transportation Research Board, 1990, pp. 112-124.**

Highway engineers have always been concerned with providing pavements of acceptable serviceability. The serviceability of a highway segment, which is largely a function of pavement roughness, is a widely used criterion for deciding when pavements are in need of rehabilitation. For this application, various statistics are currently used as indicators of pavement serviceability, the most common being the present serviceability index. These statistics are largely determined from measurements of pavement roughness. Various devices and procedures have been developed for accomplishing these measurements. Of practical necessity, devices for measuring pavement roughness must provide repeatable measurements at normal highway speeds. In addition, devices should not require difficult calibration procedures; should be able to field-process collected data; and should be relatively inexpensive to own, operate, and maintain. The Siometer, which is currently used by the Texas State Department of Highways and Public Transportation (SDHPT) for evaluation of pavement riding quality, holds promise as an instrument for the routine collection of profile data on a network-wide scale. The Texas SDHPT has recently begun investigating the profile-measuring capability of the Siometer. A unique feature of this device is the statistical modeling procedure for characterizing the vehicle on which it is installed, which lends portability to the Siometer. In it, the parameters of the statistical model are determined in a self-calibration procedure that is run before profile data are collected. To evaluate the applicability of the Siometer as a device for profile measurements, profile measurements with the Siometer were compared with those from a profilometer.

**Flude, S.T., and D.J. Janssen, "Comparison of Air-Void Characterization for Various Concretes," Prepared for the 69th Annual Meeting of the Transportation Research Board, January 1990, 40 pp.**

At present, the most widely accepted predictor of durability is the spacing factor, which is a parameter related to the maximum distance from any point in the cement paste to the boundary of an air void. At best, this factor is a loose indication of durability, since the durability depends on more than just the paste content, the air content, and the void frequency. Several studies have shown that parameters such as the specific surface and distribution of air voids, as well as the void frequency, the Philleo factor, and the air void content, can influence the durability of laboratory concrete. Field samples were used to investigate these and other parameters. Through such an investigation, the authors hoped to determine any additional parameters that may be used in conjunction with the spacing factor to more accurately predict durability. Since no totally satisfactory method has been found to correlate the air void system to the prediction of durability of concrete, this paper provides improved air void characterization and methods for evaluating the durability of marginal concretes.

**Ford, R.W., and T.L. Shelly, *Water-Based Concrete Curing Compounds*, Publication No. 604185, California Department of Transportation, June 1982, 33 pp.**

Proprietary water-based curing compounds were compared with solvent-based compounds now used by Caltrans. Water-based compounds protect concrete during the curing period, as well as solvent-based compounds, at a slightly lower cost. A tentative specification for water-based curing compounds is included in this report.

**Fowler, G., and W. Gulden, *Investigation of Location of Dowel Bars Placed by Mechanical Implantation*, Georgia Department of Transportation, Office of Materials and Research, May 1983, pp. 1-31.**

This report describes a phase of a study concerned with the restoration of load transfer to existing jointed concrete pavements. Five projects were selected to determine the location of dowel bars placed by mechanical implantation and basket assemblies. A 1-percent stratified random sample of bars was selected for coring to determine depth, horizontal and vertical rotation, and vertical alignment. Measurements were also made with a metal detector on additional bars. The study found substantial problems with the mechanically placed dowels with respect to alignment and rotation. Much better results were obtained with the basket assemblies. It was noted that sawcut location affected longitudinal alignment to a large extent on both basket and implanted dowels. No pavement distress related to dowel bar misplacement had occurred after 3 years of traffic, even for locations with dowel bars with extremely large horizontal and vertical rotation.

**Friggle, T., "Results of an Analysis of a Ride Quality Specification for Hot-Mix Asphaltic Pavements and Portland Cement Concrete Pavements in Texas," Prepared for Transportation Research Board 75th Annual Meeting, January 1996, 21 pp.**

In 1993, the Texas Department of Transportation (TxDOT) initiated the use of a ride quality specification for hot-mix asphalt concrete (HMAC) and portland cement concrete (PCC) pavements. The specification allows payment incentives and disincentives based on the profile index, defined as the ratio of vertical deviations in the surface over a length of pavement. This paper documents an analysis of the data collected after initial use of the specification and modifications made to the specification based on the analysis. The results indicated a high probability of a smooth ride with HMAC pavement, while a smooth ride on concrete pavement was less likely to occur. Certain types of concrete placement, such as hand-finished concrete or very short segments, were unlikely to produce a surface smooth enough to satisfy the specifications.

**Fuchs, F., "An Overview of European Practice," Prepared for Transportation Research Board 73rd Annual Meeting, January 1994, 7 pp.**

This paper addresses noise, skid resistance, and comfort levels provided by different pavement textures. In Europe, an exposed aggregate finish is considered the most advantageous method for obtaining optimum surface characteristics for safety, ride

quality, and noise. It is, however, important to use hard and polish-resistant aggregate in the surface layer. Furthermore, specifications can be adapted to allow agencies to factor in local priorities, climatic conditions, and budgetary constraints.

**Gerhardt, B.B., *The Effect of Vibration on the Durability of Unreinforced Concrete Pavement*, Publication No. CDOH-P&R-R&SS-75-2, Colorado Department of Highways, May 1975, pp. 12-53.**

Because a noticeable difference in durability of concrete pavements in Colorado seemed to be associated with differences in consolidation and void content of the concrete, an effort was made to determine the relationship. After extensive laboratory work to determine the basic characteristics of the designed mixes, test sections were laid out at four different locations. These test sections had internal vibration by vibrators having various diameter eccentrics and vibration speeds. In addition, the angle of the vibrators was changed, as was the height of the vibrators, the paver speed, and the slump of the mixes. Performance of these test sections was observed from May 1970 until May 1975. Wear in the wheelpaths due to studded tires has been approximately 0.1 in (2.5 mm) per year in the heavily traveled non-test areas of the experimental sections. Wear in the test sections has averaged 0.07 in (1.8 mm) per year, but it has not varied directly with the consolidation effort. Although statistical evidence indicates that good field control and reasonable vibration will ensure at least 97-percent consolidation and sound concrete, there does not seem to be a direct relationship between vibration effort and resistance to surface abrasion. Other factors such as curing, mix design, water-cement ratio at the surface, and aggregate type appear to control surface hardness.

***Glossary of Highway Quality Assurance Terms*, Transportation Research Circular 457, Transportation Research Board, April 1996, 18 pp.**

Highway quality assurance, like many other specialized subject areas, has a unique language of technical terms and expressions with very specific meanings. Some of these terms are not well-understood, and their use is subject to a variety of interpretations. Moreover, highway quality assurance terminology is continually changing to keep pace with advances in quality assurance. This document contains terms of common usage and accepted practice.

**Goulias, D.G., "Development of Performance-Related Specification Pay Schedules," Prepared for Transportation Research Board 74th Annual Meeting, January 1995, 12 pp.**

In recent years, highway agencies and constructors have used improved construction techniques and materials for better pavement quality. However, newly constructed asphalt and portland cement concrete pavement surfaces are not always being built to the desired level of ride quality. In 1984, AASHTO conducted a survey of smoothness specifications to recommend a draft smoothness specification for State highway use.

Since then, several agencies have undertaken the development of new smoothness specifications. One of the objectives of these specifications is to provide rational and defensible pay schedules.

A method for defining rational and defensible performance-related pay schedules for smoothness of flexible pavements is presented in this paper. Based on the level of smoothness achieved by the contractor, rewards for superior pavement quality and penalties for inferior quality are defined. Such rewards were defined by examining the influence of initial pavement roughness on pavement performance and the resultant economic effects on the users and the agency. Furthermore, the effect of design parameters such as design period, traffic and reliability level, material properties, and cost was examined through a sensitivity analysis. The possibility of defining stepped and gradually increasing payment schedules is presented with recommendations.

**Goulias, D.G., "Conceptual Development of a Performance-Based Specification for Pavement Smoothness," Prepared for 1997 Transportation Research Board Annual Meeting, January 1996, 11 pp.**

This paper presents the methodology for the development of a performance smoothness specification. The methodology takes into consideration the deviation of the as-built pavement for the design quality (i.e., target value) and evaluates the consequences on pavement performance. Then, based on the achieved level of smoothness by the contractor, incentive plans for superior pavement quality and penalties for inferior quality are defined. Such pay schedules are defined by examining the influence of pavement roughness on user and agency costs (in terms of vehicle operating cost, pavement maintenance, and rehabilitation costs). Furthermore, the effect of design parameters, such as design period, traffic and reliability level, material properties, and cost, on the incentive plans was examined through a sensitivity analysis.

The methodology for defining a performance specification for pavement smoothness identified in this study is applicable to any condition and may be used by highway agencies elsewhere.

**Grady, J.E., and W.P. Chamberlin, "Groove-Depth Requirements for Tine-Textured Pavements," *Transportation Research Record 836*, 1981, pp. 67-76.**

This paper discusses the depth required for grooves on new tine-textured concrete pavements in order to ensure an adequate skid resistance over their entire design life. It is based on measures of texture depth and skid resistance, with both ribbed and smooth tires, made on new to 5-year-old pavements in New York. Initial groove depth needs of 3/16-in (4.8-mm) minimum were calculated from two values estimated from the study data: (1) minimum groove depth (0.050 in [1.3 mm]) to ensure adequate skid resistance with a minimum legal tire tread and (2) mean groove wear rate (0.013 in [0.3 mm] /million vehicle passes). Groove depth measurements on new concrete pavements and

bridge decks indicated 21 and 14 percent compliance, respectively, with the proposed new standard of 3/16-in (4.8-mm) minimum, and 60 and 44 percent compliance with the current standard of 2/16-in (3.2-mm) minimum. Prospects for improving the compliance rate were judged to be most promising in two areas—increasing the awareness and motivation of construction personnel and improving the design of tining rakes over those now in use. Although the findings of this study are specific to standards and conditions in New York, the methodology should be of general interest.

**Gray, R.J., "Variability of Compressive Strength Test Results," *Concrete Technology Today*, June 1990, 11(2): 1-4.**

This article focuses on the variability in compressive test results obtained from different laboratories for the same batch of concrete. An estimate of this between-laboratory variability is derived from data obtained in interlaboratory or comparative testing programs that are operating in the three largest metropolitan areas of British Columbia and Alberta, Canada. Within-laboratory information is also presented. It was concluded that both the within- and between-laboratory precision of the standard test method for determining the compressive strength of cylindrical portland cement concrete test specimens vary linearly with the compressive strength of the test concrete. Results of the within-laboratory study imply that on the basis of test error alone, the difference in absolute value of two test results obtained in the same laboratory on the same material will exceed 10 percent of the average compressive strength of the test concrete about 5 percent of the time. There is reason to question one or both of the test results only if such a difference is larger than this value. Results of the between-laboratory study imply that on the basis of test error alone (including both the within- and between-laboratory components), the difference in absolute value of two test results obtained in different laboratories on the same material will exceed 15 percent of the average compressive strength of the test concrete about 5 percent of the time. There is reason to question one or both of the test results only if such a difference is larger than this value.

**Greer, W.C., Jr., "Evaluation of Strength Tests and Acceptance of Concrete Pavements," *Proceedings: Fourth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, Publication No. FHWA-RD-89-208, April 1989, pp. 375-383.**

One of the most troublesome phases of a concrete pavement construction project can be the determination of whether the strength of the concrete meets the project specifications. This determination can be complicated by the fact that the flexural strength test is often specified for acceptance, and there is often little or no understanding of the procedures and precision of this test. The quick placement of large volumes of concrete for pavement construction projects means that a million dollars or more worth of concrete can be "on the ground" before the first acceptance test is ever performed. If questions arise regarding the validity of the acceptance tests, then

an understanding of the inherent variations in the test is necessary for proper evaluation of concrete acceptability.

This paper presents analyses to determine the statistical parameters for flexural strength test results for more than 1 million $yd^3$ (0.765 million $m^3$) of concrete pavement constructed at the Atlanta International Airport. Parameters for within-test statistical variations are presented so that the quality of the testing agency personnel can be evaluated. Parameters for the between-test statistical variations are presented so that the quality of production by the contractor can be evaluated. The results of the analyses indicate that the within-test variations can approach the between-test variations, and this greatly contributes to confusion and misunderstandings. The analyses also show that two flexural strength test results from the same batch of concrete can be expected to vary by as much as 100 psi (689.5 kPa) and not be considered suspect 95 percent of the time. The difference between the average flexural strengths of each of two batches of the same concrete mix in the laboratory can differ by as much as 75 psi (517.1 kPa) and not be considered suspect 95 percent of the time. Procedures for adjusting the actual flexural strength test results upward based on additional thickness over the design thickness, or downward based on deficient thickness from the design thickness, are given.

**Grove, J.D., "Blanket Curing to Promote Early Strength Concrete," *Transportation Research Record 1234*, Transportation Research Board, 1989, pp. 1-7.**

Fast-track concrete has proven to be successful in obtaining high early strengths; however, this benefit does not come without cost. Special Type III cement and insulating blankets to accelerate the cure add to the expense. This research was intended to determine the benefit derived from the use of insulating blankets to accelerate strength gain in three concrete mixes using Type I cement. The goal was to determine mixes and curing procedures that would result in a range of opening times. This determination would allow the most economical design for a particular project by tailoring it to a specific time restraint. Three mixes of various cement contents were tested in the field. Flexural beams were cast for each mix and tested at various ages. Two test sections were placed for each mix, one section being cured with the addition of insulating blankets and the other being cured with only conventional curing compound. Concrete with fast-track proportions, Type I cement, and insulating blankets reached the required strength in approximately 36 h. A standard mix using the blankets reached the required strength in approximately 48 h, and the fast-track proportions with Type I cement without blankets reached that strength in about 60 h. The results showed a significant improvement in early strength gain with the use of insulating blankets.

**Hall, M. "Early Strength Testing of Concrete Cores and Cylinders," *Transportation Research Record 1478*, Transportation Research Board, 1995, pp. 82-89.**

During the 1989 construction season in Wisconsin, six projects built by four different concrete paving contractors were studied to assess the in situ early strength of concrete pavements, the early strength of lightly insulated field-cured concrete cylinders, and the 28-day strength of paving concrete. The projects were also studied to establish the relationship between cylinder and in situ strength typical on paving projects in Wisconsin. Testing results indicated that most pavements constructed in warm weather attain compressive strengths of 20.7 MPa (3,000 psi) to 24.1 MPa (3,500 psi) in 3 days or less, approximately 95 percent of the paving grade concrete has a 28-day compressive strength of more than 27.9 MPa (4,050 psi), and the compressive strength of lightly insulated cylinders cured in the field provides a reasonable measure of the in situ compressive strength of the pavement as measured in core tests.

**Hankins, K.D., *A Review of Tines Texturing of Portland Cement Concrete Paving*, Publication No. FHWA/TX-86/43+187-10, Texas State Department of Highways and Public Transportation, 1984, 34 pp.**

This report describes the tests conducted by the Transportation Planning Division, Research Section of the State Department of Highways and Public Transportation, to review the methods and effectiveness of tine-texturing of PCC pavements in Texas. It includes average texture values, percent by texture groups, wear measurements, operator variations, and surface variations within a project. The opinions, concerns, and comments of several engineers in various Districts were also discussed. It was concluded that the majority of the Districts have experienced trouble in obtaining the required tine texture levels. They are also concerned with the damage and possibility of structural strength loss caused by tining and do not believe the possible reduction in wet weather accidents achieved by tines is worth the possible reduced loss in structural life. Recommendations were made to reduce the average texture depth requirements and delete the minimum texture depth requirement. Minimum cross slope requirements and alternate methods of finishing were suggested. Also, revisions to the roughness requirements were suggested, as was additional study of concrete finishing techniques.

**Harrison, R., C. Bertrand, and W.R. Hudson, "Measuring the Smoothness of Newly Constructed Concrete Pavement for Acceptance Specifications," *Proceedings: Fourth International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, Publication No. FHWA-RD-89-208, April 1989, pp. 601-614.**

This paper presents a brief historical background and reports the key results of an AASHTO survey to ascertain States' practices for measuring and evaluating pavement smoothness. A modification to the AASHTO guide specification is then discussed, as are implications of the bonus and penalty categories. A high-resolution profiling device, the Dipstick, has been tested with other roughness devices, and the results are presented. Finally, a California-type Profilograph was employed on newly constructed reinforced concrete to test the proposed specification prior to its trial adoption.

Problems with the Profilograph were identified that could make the proposed standards difficult to implement. It was concluded that a combination of Profilograph and Dipstick instruments, together with a modification to the categories within the Texas draft specification, would result in a more equitable and enforceable standard.

**Haviland, J.E., and R.W. Rider, "Construction Control of Rigid Pavement Roughness," *Highway Research Record 316*, Highway Research Board, 1970, pp. 15-32.**

Results of a 2-year study of cause-effect relationships involved in roughness of concrete pavements are reported. Data were derived from analog measurements obtained in each wheelpath within hours after concrete placement on randomly selected pavements, as well as from qualitative observations of paving methods. Sampled construction consisted of 184 sections of 1- and 2-lane pavement built under 62 different contracts with 8 different form-type finishing machines and 3 different slipform pavers. Statistical analysis was held to a minimum by uncontrolled interactions, but five factors were found to be significant in relation to roughness: backing up of the last finishing machine, absence of a float, use of fewer than three screeds, use of a crown section as compared to a uniformly sloping section, and lane-at-a-time paving.

Nine other construction phenomena producing roughness, common to many projects but found less frequently than these five, are also covered in some detail.

**Hegmon, R.R., "A Close Look at Road Surfaces," *Public Roads*, Summer 1993, pp. 4-7.**

To most people, including the majority of readers of *Public Roads*, road surfaces are just gray areas stretching for miles and miles. Road surfaces are expected to provide safe driving conditions in dry and wet weather, provide a smooth and quiet ride all the time, minimize splash and spray during rain, provide good visibility under adverse conditions, and have a long service life.

A close look at the surface reveals many features, including texture, which is needed to provide skid resistance, reduce splash and spray in heavy rain, and reduce headlight glare in night driving. But texture may increase noise and reduce the life of both pavement and tire. Further, as roads age and deteriorate from the effect of heavy truck traffic and weather, signs of distress appear. Road roughness is one sign of distress and is detrimental to both pavement life and ride quality. This article discusses only road roughness, how roughness is measured, and the effect of roughness on the highway user and on pavement life.

**Henley, R.G., D. Malkemus, D.W. Fowler, and A.H. Meyer, "Evaluation of Chace Air Indicator," *Transportation Research Record 1062*, Transportation Research Board, 1986, pp. 55-62.**

An evaluation of the Chace Air Indicator (CAI) was made for use in PCC construction. The CAI indicated higher values than the pressure method at low air content and lower values at high air content. The CAI readings corrected for mortar content and the Chace factors produced values approximately 15 percent higher than the pressure method over all ranges of air content. A regression analysis procedure was used to determine a curve correction to account for the difference between the Chace factor- and mortar-corrected CAI readings and those of the pressure meter. An indication of the reliability of the results was represented by confidence intervals. The CAI does not have sufficient accuracy to measure the air content of concrete for job control purposes.

**Heston, D., and J. Wojakowski, *Portland Cement Concrete Pavement Joint Faulting on US-69, Miami Co., Kansas*, Kansas Department of Transportation, July 1992, 19 pp.**

This paper presents the results of a joint faulting survey performed on U.S. 69 in Miami County, Kansas. Twenty-three test sections were placed with various adjustments to vibrator settings, concrete admixtures, and other special features. The results showed that faulting increased with pavement age and traffic loadings. The results also showed that pavements with greater concrete density tended to fault less.

**Huft, D.L., "Analysis and Recommendations Concerning Profilograph Measurements on F0081(50)107 Kingsbury County," *Transportation Research Record 1348*, Transportation Research Board, 1992, pp. 29-34.**

In 1990, the South Dakota DOT noted significant discrepancies between its ride-quality measurements and those taken by a contractor paving a PCC project. The contractor's measurements were consistently smoother than SDDOT's and would have generated incentive payments approximately twice as large. About half of the observed difference could be attributed to increased pavement roughness after paving, but the rest appeared to result from differences between the department's manual profilograph and the contractor's computerized unit. Analysis revealed that a numerical filtering algorithm used by the computerized profilograph strongly attenuated profile features with wavelengths shorter than 10 ft (3.0 m). Such attenuation was observed directly on the computerized unit's profile traces. Because of the attenuation, SDDOT considered the computerized measurements unsuitable for calculating incentive payments. However, SDDOT could not use its own measurements as a basis for payment because they were not taken within the specified 48-h period after paving.

To estimate a fair incentive payment, SDDOT developed a correlation between the computerized and manually interpreted profile indexes for the project. Using the correlation, SDDOT awarded an incentive payment approximately midway between its original estimate and the contractor's. SDDOT has suspended use of computerized profilographs pending improvement of the filtering algorithm. Preliminary experiments indicate that although the computerized profilograph's first-order filter attenuates profiles too strongly and produces artificially low profile indexes, a third-

order filter might generate higher profile indexes than does a manual interpreter. This suggests that a second-order filter might best approximate a human's visual interpretation of the profile. Further research is needed to confirm this hypothesis and to establish a foundation for standard filtering procedures.

**Hughes, C.S., *Variability in Highway Pavement Construction*, National Cooperative Highway Research Program, Synthesis of Highway Practice 232, 1996, 38 pp.**

The quality of highways has always been a major concern to highway engineers and contractors. The AASHTO Quality Assurance Guide Specifications use the variability of material and construction processes as one of the measures to assess quality. This synthesis of information defines several measures of variability, but concentrates on the use of standard deviation as the usual variability measure. The synthesis updates typical variabilities found in material and construction processes. Also included are discussions of current research activities as related to variability, how variability can be used in the development of specification limits, the state of the practice of incentives and disincentives in specifications, and the need for additional information on the variability of several material and construction processes.

Of the many important findings of the AASHO Road Test, one was a revelation as to the greater-than-expected magnitude of the variabilities encountered in the construction of the pavements in this major study. Since the findings of this study were reported, many reports of studies on typical variability in highway construction have been conducted by both the FHWA and various DOT's. More recently, with greater use of computers to accumulate data from materials quality control and acceptance testing, both DOT's and contractors have developed bases for quantifying material and construction variability and other important properties.

From the data gathered in this synthesis, it is obvious that variability has a relatively wide range of values for each test procedure. One of the factors that influences this variability is the period of time, distance, area, or quantity of material over which the variability is measured. This information was not always available in the references found. It is important that in the development of a specification, data be used prudently. Verification of the variability under conditions of proposed usage is encouraged.

**Hughes, C.S., A.L. Simpson, R. Cominsky, K. Maser, and T. Wilson, *Measurement and Specification of Construction Quality, Volume I*, Publication No. FHWA-RD-98-077, Federal Highway Administration, May 1998, 183 pp.**

This study consisted of testing six projects, three hot-mix asphalt concrete (HMAC) and three portland cement concrete (PCC). The primary objectives were to: (1) determine how current quality control test results vary in construction projects and how this variability affects pavement performance and (2) assess the suitability of current

methods of quantifying materials and construction quality and quality variability, and develop and improve methods that minimize current shortcomings.

Volume I of this report contains information about the data analysis. Volume II provides information about the data collected and use of GPR to determine pavement thickness. This volume also contains univariate data on each of the tests performed on samples taken from the projects used in this study.

**Huschek, S., *Measurements on Test Stretches*, Final Report, Publication No. 240/7, Technische Universitat Berlin, May 1994, 22 pp.**

The Technical University of Berlin prepared this report about the measurements of texture, tire/road noise, and skid resistance on two German test stretches built for the BRITE/EURAM project BE 3415, "Optimization of the Surface Properties of Concrete Roads in Accordance With Environmental Acceptance and Traffic Safety." The "Garlstorf" test stretch includes 13 sections of different textures, and the "Wittstock" test stretch includes 29 sections of different textures. Both were investigated a few weeks after opening, and the "Garlstorf" test stretch was investigated a second time approximately 1 year after the first measurements.

The texture measurement results showed that the demand for high amplitudes in the short macrotexture wavelength and low amplitudes in the megatexture is met only in a few cases. Trailer measurements using the World Road Association (PIARC) tire favor longitudinally structured cement concrete surfaces.

The positive influence of the shortwave macrotexture and the negative influence of the megatexture on the acoustical road surface properties cannot be proven by the regression analysis of the spectral texture measures and the different tire/road noise measurement results. Regression analysis shows no correlation between the acoustical and the frictional properties of road surfaces.

**Idorn, G.M., "Concrete Durability & Resource Economy," *ACI Compilation 24*, American Concrete Institute, 1993, pp. 22-27.**

Concrete of certified, long-term durability, tailored to its performance requirements, will become a basic element in the development of resource economy policies worldwide. The article discusses concrete materials and concrete processing and testing, and points out the need for innovation in modern instrumentation and test methods for site monitoring of effective compaction. Quality control should be based on scientific, physico-chemical monitoring methods. It is expected that during the next decade, on-site monitoring of the processing of fresh concrete and of early curing will completely replace conventional quality control by means of sample testing. Comments are made on research sponsored by litigation and forensic engineering. The Strategic

Highway Research Program is noted as having created more cooperation in the planning and management of research in concretes than ever before.

*Influence of Design Characteristics on Concrete Durability*, **Missouri Highway and Transportation Department, March 1990, 23 pp.**

The primary objective was to investigate the influence of cement factor, percent air, maximum coarse aggregate size, and percent of total aggregate as sand on the performance of concrete subjected to freeze-thaw cycling. An important secondary objective was to document air void system characteristics of hardened concrete with these same design factors. Data in this report include durability and expansion of freeze-and-thaw beams, air void system characteristics, flexural strength, and compressive strength in conjunction with concrete design factors and fresh concrete characteristics.

Resistance to freeze-and-thaw increased as maximum size of coarse aggregate decreased. Neither compressive strength nor flexural strength changed with maximum size of coarse aggregate. Based on the results of this study, a desirable concrete pavement mix design using D-crack-prone coarse aggregate of the materials tested would be: 0.5-in (12.7-mm) maximum size coarse aggregate, 38 percent of aggregate as sand, 6.5 sacks of cement/$yd^3$ (5.0 sacks/$m^3$), 6.0 ±1.5 percent air by volume, and 2.5-in (63.5-mm) maximum slump.

*Innovative Contracting Practices*, **Transportation Research Circular 386, Transportation Research Board, December 1991.**

Historically, transportation agencies in the United States have been conservative in their construction contracting practices and have tended to continue the status quo rather than assume the risks associated with change. Innovative contracting techniques have developed more in foreign countries than in the United States. Before U.S. Federal and State agencies can make greater use of these techniques, the issues, options, potential advantages and disadvantages, steps required, and legal and economic impacts must be better understood.

In order to help develop this understanding and facilitate greater innovation in government and industry, TRB formed its Task Force on Innovative Contracting Practices. The task force examined the processes, as they affect quality and cost, under which U.S. and foreign agencies contract for construction. This report summarizes the findings of the task force, as well as some governmental initiatives that have already taken place as a result of its deliberations.

*An Investigation of Frictional Properties of Wire-Combed PCC Pavement Surfaces*, **Publication No. FHWA/MO-78/3, Missouri Highway and Transportation Department, September 1982, 80 pp.**

This report involves PCC pavements constructed with surface textures produced by a wide variety of wire combs. A small amount of testing was also performed on PCC pavements with other textures as a follow-up of previous research on frictional properties. A limited amount of noise evaluation by rating panels and sound level meters was conducted to assist in selection of an optimum tine spacing for wire combs on future PCC pavement projects. This report includes a brief description of each major project with summaries of friction number tests, sand patch texture depth tests, and sound-level measurements.

**Irick, P.E., S.B. Seeds, M.G. Myers, and E.D. Moody, *Development of Performance-Related Specifications for Portland Cement Concrete Pavement Construction*, Publication No. FHWA-RD-89-211, Federal Highway Administration, May 1990, 271 pp.**

The primary product of this study was a demonstration PRS for PCC pavement construction. The system is designed to consider three key factors (PCC strength, slab thickness, and initial serviceability) in assessing an as-constructed pavement delivered by a contractor and calculating an appropriate reward or penalty.

Many pavement performance prediction relationships and PCC property prediction equations were evaluated to develop the performance-related aspects of the new system. In addition, a rather intensive experimental laboratory study of PCC material properties was conducted to develop better multi-factor prediction relationships.

The demonstration PRS was developed using a computerized spreadsheet program. It was designed to be parallel to the demonstration PRS system developed under NCHRP Project 10-26A for asphalt concrete pavements. As a result of the study, recommendations were made for further research in several key areas related to PRS and field and laboratory studies.

**Janoff, M.S., *Pavement Smoothness*, Information Series 111, National Asphalt Pavement Association, 1991.**

This publication describes the benefits of smooth pavements and the adverse effects of pavement roughness. In addition, examples of methods to construct smooth hot-mix asphalt (HMA) pavements are provided, along with a discussion of the economics of smooth HMA pavements.

The benefits of pavement smoothness include increased roadway safety, vehicle operating speeds, ride quality, and drive comfort; improved long-term pavement performance and vehicle behavior (e.g., braking, steering, and control); and reduced annual maintenance costs, vehicle operating costs, and driver fatigue. Methods to improve pavement smoothness include proper specifications, incentives and disincentives, an HMA product designed for the specific application, properly trained

and utilized personnel, better transportation of the HMA from the mixing facility to the site, and a high-quality paving process.

**Jensen, C.C., *Pavement Smoothness*, Irving F. Jensen, 1987.**

This article discusses Jensen Co.'s experience in using the profilometer, including some "best practices" types of recommendations. The article also discusses profilometer specifications and recommends the following for "quality control through the profilometer specification":

- Incentive or bonus clauses.
- Elimination of 3/10 must-grinds on otherwise superior quality pavement.
- Uniform, stable padline.
- Base construction or soil modification on bad subgrades.
- A range of numbers, such as recommended by the American Concrete Pavement Association, to facilitate realistic quality control under the varying conditions of subgrade and specification design requirements.
- Contractors should own and use profilometers daily to improve their results.

**Johnson, J.L., D.G. Zollinger, and S. Yang, *Guidelines for the Construction of Fast-Track Concrete Pavement Intersections*, Publication No. FHWA-TX-94-1385-1F, Texas Department of Transportation, June 1994, 152 pp.**

This report documents a review of Texas Department of Transportation (TxDOT) design and construction practices with regard to accelerated concrete pavement construction of heavily traveled intersections. The review focuses on the design and construction measures taken by TxDOT to address the impact of reconstruction of such pavement facilities. The review consisted of a field investigation that considered current construction specifications, as well as the development of guidelines based on an assessment of the impact present practices may have on performance of the constructed pavements.

It was determined that the use of fast-track paving techniques requires coordinated construction planning to take advantage of the benefits offered by this construction process. Concrete mix proportioning can be supplemented by special admixtures to accelerate the time of set, but this must also meet the workability requirements associated with the transportation, placement, finishing, and curing of concrete.

It is also beneficial to the construction supervisor to qualify the concrete mixture with respect to strength gain for a given set of weather conditions. Laboratory work was conducted to support the development of the mix qualification process.

**Klemens, T.L., "Ultra-Smooth Slab Boosts Contractor's Profit," *Highway and Heavy Construction*, 133(3): 44-46.**

In mid-1987, Delaware began one of its largest highway projects in recent years, and the first using the State's new smoothness standards. Under those guidelines, payment for concrete paving is tied to the smoothness of the finished roadway. The contract also provides for a bonus to be paid for very smooth pavements. The contractor is using a special grade-control device to place the ultra-smooth concrete slab. A mobile stringline mounted on an extended wheelbase rolls along the centerline side of the paver, averaging out slight grade irregularities. The smoothness of the pavement is measured by a California-type profilograph.

**Komlos, K., F. Kruml, and Z. Homolka, "Judgment of Concrete Quality in Transportation Structures," *Transportation Research Record* 652, Transportation Research Board, 1977, pp. 76-80.**

This paper deals with the present Czechoslovakian method for judging the quality of fresh and hardened concrete used in transportation structures. Standard methods and criteria are presented for estimating concrete properties for different types of structures. In the case of hardened concretes, destructive and nondestructive testing methods are analyzed and evaluation techniques are given. The problems of quality control of cements and aggregates are studied. The judgment of the acceptability of fresh concretes in relation to their composition and workability is also analyzed. Requirements are presented for such properties as concrete strength and properties related to short- and long-term deformation. Statistical evaluation is emphasized, and acceptability criteria for safety, homogeneity, and economy are presented.

**Kopac, P., *Performance-Related Specifications*, Publication No. FHWA-SA-97-098, Federal Highway Administration, 1998.**

This brochure provides an overview of PRS. It explains the background and elements of PRS, lists some general characteristics of good specifications, and discusses the future in research and development of PRS.

**Kosmatka, S.H., "Compressive Versus Flexural Strength for Quality Control of Pavements," *Concrete Technology Today*, December 1985, 6(4): 4-5.**

This article discusses the advantages of using compression-test cylinders as opposed to flexural-test beams for the control of concrete. A relationship between flexural and compressive strength for various aggregate combinations is presented.

**Kraft, L.M., *A Study of Concrete Consolidation*, Publication No. 930-052, Alabama State Highway Department, Bureau of Public Roads.**

In some instances, concrete pavements in Alabama have developed an undesirably high number of air and water pockets (as found in pavement cores). The presence of these

pockets has been attributed to inadequate compaction of the fresh concrete. It is the purpose of this study to establish the effect that several of the variables that influence the compactability have on the quantity of entrapped air in the vibrated concrete. The variables studied were as follows: (1) three coarse aggregates (varying in size and shape), (2) two vibrator frequencies with three amplitudes (spud-type vibrators), (3) variation in vibrating time, and (4) three thicknesses of concrete (6, 10, and 18 in [152.4, 254.0, and 457.2 mm]). The quality of the vibration effort was measured by the entrapped air content of the vibrated concrete; the less entrapped air, the better the vibration. For stiff mixes, the results of this investigation demonstrate the following:

- The percentage of entrapped air is not significantly reduced as the vibration time is increased beyond 5 seconds.
- The percentage of entrapped air can be greatly influenced by the characteristics of the coarse aggregate in the mix.
- An increase in the vibrator amplitude or frequency does not significantly reduce the percentage of entrapped air.
- The percentage of entrapped air decreases as the thickness of concrete increases from 6 to 18 in (152.4 to 457.2 mm).
- The compressive strength of concrete specimens with large volumes of entrapped air were generally greater than 3,000 psi (20.7 MPa); however, higher strengths could be obtained if the entrapped air content could be reduced, thus offering more efficient use of the cement in the mix.

**Kraft, L.M., "Compaction of Concrete Slabs by Vibration," *American Concrete Institute Journal & Proceedings*, 68(6): 462-467.**

Large air and water pockets have been observed in the cores taken from well-proportioned vibrated concrete pavements. Several parameters that may influence the volume of entrapped air in thin concrete sections compacted with spud vibrators are studied. Laboratory test results suggest that it is difficult to eliminate the entrapped air content in concrete slabs of stiff concrete compacted with spud-type vibrators and for moderate vibration times and vibrator amplitudes and frequencies. The use of a surcharge may be beneficial in reducing the entrapped air content, and a crushed stone concrete mix may yield lower entrapped air content than a rounded stone mix.

**Ksaibati, K., S. Asnani, and T. Adkins, "Factors Affecting the Repeatability of Pavement Longitudinal Profile Measurements," *Transportation Research Record 1410*, Transportation Research Board, 1993, pp. 59-66.**

When looking at the accuracy of profilometers, most agencies are mainly concerned with hardware precision rather than the errors caused by the human operators or

environmental factors. The Wyoming Transportation Department and the University of Wyoming conducted a joint research project to determine the effect of these two factors on the accuracy and repeatability of roughness and rut-depth measurements. The Wyoming Transportation Department's road profiler, which is a duplicate of the South Dakota road profiler, was utilized in this study. Thirty-six test sections were tested by three different operators to determine the effect of human factors on measurement repeatability. In addition, a concrete test section was monitored and tested several times in the 1991 testing season to examine the effect of different combinations of environmental factors on the measured roughness. The data collected were then tabulated and statistically analyzed. This paper summarizes the design of the experiment, describes the data collected, and provides specific conclusions with regard to the effect of human and environmental factors on the accuracy of roughness and rut-depth measurements.

**Kulakowski, B.T., and C. Lin, "Effect of Design Parameters on Performance of Road Profilographs," *Transportation Research Record 1311*, Transportation Research Board, 1991, pp. 9-14.**

Road profilographs are commonly used to measure roughness of new and newly surfaced pavements. Because new pavements are usually smooth, profilographs must have high measuring sensitivity, particularly in the range of profile wavelengths responsible for dynamic pavement loading applied by heavy trucks and for ride comfort. Simple analytical and computer models were developed to examine the effects of basic design parameters on the performance of California and Rainhart profilographs. It was found that the quality of measurements obtained with the two profilographs could be improved by modifying some of the design parameters (e.g., length of main truss and number and spacing of supporting wheels). The improvement should be more significant for the California profilograph. The design parameters of the existing Rainhart profilograph are close to the values recommended in the paper.

**Kunt, M.M., and B.F. McCullough, *Improved Design and Construction Procedures for Concrete Pavements Based on Mechanistic Modeling Techniques*, Publication No. FHWA/TX-92+1169-5F, Texas Department of Transportation, June 1992, 94 pp.**

This report describes an improved set of concrete pavement design and construction procedures. In developing these improvements, the researchers used a systems approach to incorporate material characterization subsystems and mechanistic techniques into the JRCP-5 computer program. The data obtained from such an approach were then used to develop, analyze, evaluate, and implement the best procedure for designing concrete pavement reinforcement and for determining pavement sawing time and depth. Whereas the original models used to characterize concrete properties could not distinguish the effect of coarse aggregate types (CAT), the improved model now has this capability. Because several aggregate sources are used in Texas, incorporation of the effect of CAT substantially improved the prediction models.

Additionally, the researchers updated the transverse reinforcement formula according to the findings of CTR Project 459 and developed a probabilistic sawing depth and time prediction model, with all improvements and updates subsequently input into the JRCP-5 computer program. The report concludes with recommendations for improving the prediction accuracy of the models.

**Lane, D.S., and G.C. Clemena, *Determination of Concrete Quality Characteristics for the Development of a Performance-Related Specification for Concrete Pavements*, Work Plan, Virginia Transportation Research Council, April 1995, 6 pp.**

In 1995, Virginia DOT specifications for hydraulic cement concrete for rigid pavement construction were described as an "inconsistent blend of performance and prescriptive requirements." Quality assurance procedures rely primarily on flexural testing of beams by centerpoint loading, a test with high variability. Consequently, the properties of the pavement concrete are less well-known than is desirable, and virtually nothing is known about the mechanical properties of the actual pavement, which is, in fact, the most crucial piece of information for determining acceptability.

This work plan describes the details of a project being conducted by the Virginia Transportation Research Council (in cooperation with FHWA). The purpose and scope of the project is to evaluate the strength properties of concrete paving mixtures in the laboratory, to determine the relationships between compressive strength, tensile strength, flexural strength, and stress-wave propagation. These relationships were to then be verified by field testing on construction projects using the same concrete mixtures evaluated in the laboratory. The ultimate goal of the project is to collect the data necessary to develop a PRS, thereby improving the specifications and quality assurance procedures for hydraulic cement concrete pavements.

**Larson, R.M., *Proposed 1994 Field Testing and Updated Work Plan*, Technical Working Group on PCC Surface Texturing, March 1994.**

The field study referenced in this memorandum would address depth of transverse and longitudinal tining and evaluate possible non-tining alternatives to minimize noise while providing adequate friction characteristics. The memo contains an updated work plan and guidelines for evaluating test sections planned for 1994 construction or for previously constructed sections, as well as a tentative scope of work for NCHRP Synthesis Topic 26-05, Long-Term Noise Characteristics of Pavement Surface Texture.

**Larson, R.M., and B.O. Hibbs, *PCC Surface Texturing Issues*, Federal Highway Administration, April 1995, 24 pp.**

This paper presents a status report on the FHWA's re-evaluation of existing PCC surface texture guidelines. It presents preliminary findings and recommendations from several ongoing studies, including the detriment of poor quality concrete mix designs,

drainage and safety considerations that must be taken into account, and alternative surface treatments to improve skid resistance. Life-cycle cost analysis is also discussed briefly.

**Legge, W.A., *Slip Form Concrete Pavement Consolidation Study*, Kansas Department of Transportation, 1982, 22 pp.**

The purpose of this investigation was to determine if consolidation of slip-formed pavement could be improved by alterations to the system normally used on Kansas DOT projects. Variations in the vibratory system, mix design, and pavement system were evaluated by taking densities with a nuclear meter. Not all tests were conclusive, but overall results confirmed that consolidation could be improved.

**Loeffler, M.D., C.G. Papaleontiou, A.H. Meyers, and D.W. Fowler, "Moisture Retention Tests and Agitation for Membrane-Forming Curing Compounds for Portland Cement Concrete," *Transportation Research Record 1110*, 1987, pp. 46-59.**

Discussed in this paper are the relative merits of Texas specifications (Tex-219-F) and ASTM specifications (ASTM C 156-80) for testing moisture retention by liquid membrane-forming curing compounds. A comparison is also presented of the effectiveness of four motorized agitation devices to be used through drum bungholes. Preliminary work toward the development of a new moisture retention test to replace Tex-219-F and ASTM C 156-80 is also outlined, in addition to suggestions for the direction of continuing research. Recommendations are presented concerning the 6-month curing compound shelf life in effect at the time of the study and the possibility of extending this shelf life. Research is also reported on the effects of altering application rates and pattern on moisture retention. In addition, the use of optical reflectance as a measure of application rate is examined. Finally, findings are presented on the usefulness of in-line testing samples as compared with the usefulness of samples from store drums.

**Loo, K., and D.A. Lange, *Evaluation of Potential Applications of End-Result and Performance-Related Specifications, White Paper Presentations—Part II, Chapter VI—Structural Concrete*, Publication No. FHWA-IL-UI-257, Illinois State Department of Transportation, December 1995, 72 pp.**

The primary objective of this study is to evaluate, based on the best available technology, the potential applications of end-result specifications (ERS) and performance-related specifications (PRS) to asphalt concrete paving, portland cement concrete paving, and structural concrete construction. For each area, the following tasks were performed:

- Review of current literature on ERS and PRS.

- Comprehensive evaluation of other State practices to include parameters used, lot size, and payment schedules.
- Review and evaluation of current Illinois DOT specifications, practices, and policies.
- Identification of relevant IDOT data that have potential application.
- Evaluation of the feasibility of relating materials and construction data to facility performance.
- Determination of the quality of available data and their appropriateness for data analysis.
- Statistical analysis of selected variables within available data and determination of their suitability for developing ERS and PRS.

The study is organized into two parts. Part I discusses the basic concepts and development of ERS and PRS, and quality acceptance plans. Part II discusses the framework of ERS and PRS, current practice, and data analyses for each of the three areas studied. The report also contains a literature review, summary of other State practices, review and evaluation of the IDOT specification, analyses of MISTIC data, potential linking of MISTIC and Illinois Pavement Feedback System data, and recommendations for improvements to current QC/QA specifications for structural concrete construction.

**Magura, D., and G. Crawford, "New Concrete Test Device a Fresh Idea," *Roads & Bridges*, October 1995, p. 12.**

This article describes the air void analyzer (AVA). Developed in Denmark in the early 1990's, the AVA tests a sample of fresh concrete to determine the amount of air voids and the spacing between them. This allows agencies to test trial mixes in the laboratory, as well as to provide quick feedback during construction. Being able to compare test results before and after placement will allow for an improved quality assurance program with a better measure of entrained air.

**Mahone, D.C., K.H. McGhee, J.G.G. McGee, and J.E. Galloway, "Texturing New Concrete Pavements," *Transportation Research Record 652*, Transportation Research Board, 1977, pp. 1-10.**

Several texturing experiments on heavily traveled portland cement concrete pavements in Virginia are described. Included in the experiments were textures imparted by using a heavy burlap drag, metal tines (transverse and longitudinal striations), sprinkled aggregate, mortar removal, and imprinting. All textures were imparted to concrete in the plastic state. Some of the problems encountered in achieving the desired textures

are discussed. Evaluations of the effectiveness and the acceptability of the textures included noise, roughness, and skid-resistance studies. These studies resulted in the rejection of several textures for future use for one or more reasons. Consideration of all factors gave strong indication that transversely tined grooves spaced 19 mm (0.75 in) apart (center-to-center) would be preferable on tangent roadway sections, whereas longitudinally tined grooves spaced 19 mm apart in combination with transverse grooves spaced 76 mm (3 in) apart would be preferable on curves.

**Majidzadeh, K., and G.J. Ilves, *Correlation of Quality Control Criteria and Performance of PCC Pavements*, Publication No. FHWA-RD-83-014, Federal Highway Administration, March 1984, 147 pp.**

This report summarizes the results of a study of the interrelationships between quality indicators and performance of PCC pavements. Historical and construction data relative to selected quality variables were collected for 104 concrete pavement projects in 5 States. These projects were then subjected to a pavement condition evaluation. A number of statistical analyses were performed to establish relationships between the performance rating and the quality indicator data, and 55 models were developed and tested. The report illustrates the types of performance and quality indicator data required to develop statistically reliable relationships, the types of results that can be obtained from such analyses, and the impact of missing data on model development and reliability.

**Majidzadeh, K., G.J. Ilves, M. Luther, and P. Kopac, "Correlation of Quality-Control Data and Performance of PCC Pavements," *Transportation Research Record 924*, Transportation Research Board, 1983, pp. 93-99.**

The relationship between concrete pavement quality indicators and pavement performance is presented. A literature review was conducted to help identify pavement quality indicators, such as water/cement ratio, strength, slump, air content, and so forth. A detailed field investigation was carried out in five States to collect quality indicator data. A pavement condition rating (PCR) procedure was developed to collect PCR data for various pavement sections. Linear and nonlinear statistical analyses were conducted to develop models relating quality control data with PCR data. The results of the statistical analyses and the nature of the models developed are discussed in detail.

**Marquart, M., *Evaluation of Tining Widths to Reduce Noise of Concrete Roadways*, Experimental Study No. ND-94-09, North Dakota Department of Transportation, February 1995, 17 pp.**

The object of this experimental project is to evaluate the effectiveness of various tining widths to reduce noise. This study is expected to provide the NDDOT with the quietest and safest tining spacing for its concrete highways.

**Mattaacchione, A., and L. Mattacchione, "Correlation Between 28-Day Strength and Density," *Concrete International*, March 1995, pp. 37-41.**

This article discusses the relationship between 28-day concrete strength and concrete density as studied during construction of a housing project. The authors found that many factors that have long been thought to be useful indicators actually have very low correlations with 28-day strength. For example, although the 28-day strength generally increased as the water-to-cement (w/c) ratio decreased, the large variability in the 28-day strength for any w/c ratio showed this factor to be, in itself, an unreliable predictor of concrete strength. However, a significant correlation was shown between 28-day strength and concrete density, and the authors suggest that density may be a useful indicator of potentially low 28-day concrete strengths.

**McBride, J.C., *Predicting the 28-Day Strength of Portland Cement Concrete by Nuclear Densities*, Final Report, Publication No. UDOT-MIR-77-1, Utah Department of Transportation, Materials and Research Section, November 1976, 31 pp.**

The report covers a preliminary investigation into the possible use of concrete nuclear densities as a substitute for 28-day concrete specifications. During the investigation, five variables (slump, entrained air, cylinder strength, core strength, and nuclear density) from three previously constructed concrete pavement projects were evaluated. The evaluation indicated a high correlation between cylinder strength, core strength, and nuclear density. The confidence limits were too great to allow nuclear density readings to replace the 28-day cylinder strength requirements. The study recommended further research to incorporate a density requirement as a construction control of consolidation and not of concrete strength.

**McCullough, B.F., D. Zollinger, and B.T. Allison, *Preliminary Research Findings on the Effect of Coarse Aggregate on the Performance of Portland Cement Concrete Paving*, Publication No. FHWA-TX-94-1244-5, Federal Highway Administration, Texas Department of Transportation, October 1993, 112 pp.**

This report focuses on aggregates used in the construction of continuously reinforced concrete pavements (CRCP). Specific chapters address past research regarding aggregates used in asphalt concrete paving, field and laboratory investigations of asphalt concrete pavements, and CRCP behavior and distress. Also addressed are research activities related to aggregates used in asphalt cement concrete pavements. The report reviews such topics as spalling and punchout distresses in continuously reinforced and jointed pavements, aggregate shape characterization using fractals, and the determination of sawcut depth using fractal analysis. Some of these activities relate directly to improving pavement performance, regardless of aggregate type used for construction. Early recommendations based on the project's significant findings have already been presented to the Texas Department of Transportation.

**McFarland, W.F., *Benefit Analysis for Pavement Design Systems*, Research Publication No. 123-13, Texas Highway Department, April 1972, 88 pp.**

The Texas Pavement Design System of the Texas Highway Department includes computer programs that compare pavement design strategies on the basis of effectiveness and cost. The effectiveness of pavements is measured primarily by the pavement serviceability index. This report presents an investigation to determine whether motorist benefits can be related to the pavement serviceability index and whether these benefits can be used in the comparison of pavement design strategies. The motorist benefits that are considered are those associated with travel time, vehicle operating costs, accident costs, and discomfort. A model for relating motorist benefits to the pavement serviceability index is presented, and preliminary estimates of benefits are developed.

**McMahon, T.F., W.J. Halstead, W.W. Baker, E.C. Granley, and J.A. Kelly, *Quality Assurance in Highway Construction*, Publication No. FHWA-TS-89-038, Federal Highway Administration, October 1990, 70 pp.**

This report contains reprints of six articles on the subject of quality assurance that have appeared in past issues of *Public Roads* magazine. The articles are divided into the following:

- Introduction and Concepts
- Quality Assurance of Embankments and Base Courses
- Quality Assurance of Portland Cement Concrete
- Variations of Bituminous Construction
- Summary of Research for Quality Assurance of Aggregate
- Control Charts

**Meininger, R.C., F.T. Wagner, and K.W. Hall, "Concrete Core Strength—The Effect of Length-to-Diameter Ratio," *Journal of Testing and Evaluation*, 5(3).**

ASTM C 42-68, Method for Obtaining and Leasing Drilled Cores and Sawed Beams of Concrete, currently contains length-to-diameter ratio correction factors for soaked cores, but not for dried cores. However, it is widely recognized that for structures that will be substantially dry in service, dried cores are more appropriate. It is sometimes difficult (because of constraints of aggregate size, reinforcing bar placement, and member dimensions) to obtain drilled cores of the preferred dimensions, with a length twice the diameter. Data were collected in two laboratories by drilling and testing a large number of 4-in (102-mm) diameter soaked and dried cores with length-to-diameter ratios of 1.0, 1.25, 1.5, 1.75, and 2.0, taken from six structural-size members. Research results indicated that the same correction factors can be used for both moisture conditions and that the factors now in ASTM Method C 42-68 should be lowered slightly. In addition, data were gathered on the effect of the end preparation procedure

on core strength. For the methods used (three capping procedures and grinding of the ends), no important differences in measured strength were noted.

**Mitchell, M.F., C.J. Semmelink, and A.L. McQueen, "Statistical Quality Assurance in Highway Engineering in South Africa," *Transportation Research Record 652*, Transportation Research Board, 1977, pp. 58-65.**

This paper examines the first large-scale application of process and acceptance control plans to a major road construction project in South Africa. The acceptance control scheme used and its background are outlined, and certain controversial features of the scheme are discussed. The variability of typical South African construction materials and processes is indicated. Some economic consequences of the use of the plan are also reported. Because the average quality of the work was well above the minimum standard required, a fully conclusive assessment of the financial advantages or disadvantages of the scheme is not possible. Because of this, a comparison was made between the acceptance decisions of the specific scheme discussed, and those of the engineering judgment approach. It is concluded that the use of the statistical method leads to more consistent interpretation of results, and the continued use of this scheme on highway projects is recommended.

**Moody, E.D., "PCCP Performance Trends Based on LTPP Program GPS Data," Prepared for the 1996 Region 8 Concrete Pavement Workshop, Salt Lake City, Utah, February 1996, 8 pp.**

This project focuses on a subset of the LTPP database related to the primary traffic and environmentally induced distresses that occur in all PCC pavements (JPCP, JRCP, and CRCP), as well as roughness (IRI). The objective of the project is to utilize the LTPP data to:

- Develop statistical models to predict the primary distresses that occur in PCC pavements (cracking, faulting, and spalling).
- Develop statistical models to predict changes in IRI.
- Evaluate existing and develop new mechanistic-empirical models for predicting the occurrence of distress in PCC pavements.

This work identifies numerous preliminary findings related to both PCC pavement distresses and roughness measurements. These preliminary findings may necessitate some changes in the approach to analyzing data, as well as the nature of future models used in pavement design. This presentation also addresses some of the unique characteristics of the LTPP PCC pavement data.

**Muenow, R., "A Sonic Method to Determine Pavement Thickness," *Journal of the PCA Research and Development Laboratories*, 5, No. 3, Portland Cement Association, September 1963, pp. 8-21.**

This paper presents the theoretical and practical aspects of a technique for determining the thickness of concrete slabs by nondestructive means. The longitudinal resonant frequency wavelength, which is equal to twice the thickness, can be determined from values of pulse velocity and resonant frequency. The data indicate that the difference between the nondestructive determination and the thickness as measured on one core is approximately 5 percent for any one point on the slab, and between 2 and 3 percent if a number of comparisons are made along the slab. Surface cracks, other defects, and construction features may, however, increase the differences.

**Nagi, M.A., and D.A. Whiting, "Achieving and Verifying Air Content in Concrete," *PCA R&D Serial No. 1975*, Portland Cement Association, 1994, 76 pp.**

The objective of this research program was to resolve discrepancies in measurement of concrete air content between fresh and hardened states. In the first phase, data related to air content measurement were collected. A database was developed using studies where air contents were determined in the fresh and hardened states on the same batches of concrete. A questionnaire was also circulated in order to gather information from practical applications. In the second phase, a laboratory investigation was conducted to evaluate the effects of air-entraining admixture, cement alkali level, temperature, and retempering on changes in concrete air contents between fresh and hardened states. Following the completion of Phase II, a field-simulation study was conducted in order to examine the influence of field conditions on air content measurements. A statistical analysis was conducted to define the significance of each factor affecting air content measurements.

This study indicated that none of the factors evaluated have a significant effect on air content discrepancy. Although air content of hardened concrete may differ from air content of fresh concrete measured with the pressure meter, in most cases, these differences are insignificant.

**Neal, B.F., *An Evaluation of PCC Pavement Tine Texturing Patterns*, Interim Report, Publication No. FHWA/CA/TL-85/05;54-633366, California Department of Transportation, November 1985, 22 pp.**

This report provides a limited comparison of transverse and longitudinal tine texturing of PCC pavements. Skid test results and accident data are provided and analyzed. The advantages and disadvantages of each method and the reasons for California's adoption of the longitudinal tining procedure are discussed. It is concluded that although the skid values of transverse textured surfaces are slightly higher than those textured

longitudinally, both types perform satisfactorily. Thus, because of the lower cost, it is recommended that longitudinal tining continue to be specified by Caltrans.

**Neaman, D., and J.G. Laguros, "Statistical Quality Control in Portland Cement Concrete Pavements," *Transportation Research Record 184*, Transportation Research Board, 1967, pp. 1-12.**

Quality control of PCC pavements and their component parts was statistically studied in a field project approximately 8 mi (12.9 km) long. Standard field tests on fresh concrete and standard laboratory tests on hardened concrete, coarse and fine aggregate, and cement were run on an adequate number of samples. Ninety-five pavement thickness measurements were taken, and 400 concrete cylinders were tested. For other characteristics, such as slump, air content, gradations, durability, Los Angeles loss, sand equivalent, fineness, and percent passing the No. 200 sieve, 200 observations were made. The typical statistical parameters (testing, sampling and material variances, standard deviation, and arithmetic mean) were calculated, and frequency distribution curves were drawn.

In nearly all cases, the arithmetic mean of the measured characteristic complied well with the specifications. However, the relatively high values of standard deviation and of the testing variance raise serious questions about the philosophy underlying the existing acceptance–rejection procedures in PCC pavements. Upper and lower control limits, especially those based on average values, show conclusively that unfit material is sometimes accepted. Also, large values of the testing variance $\sigma_t$ suggest that standard tests need some refinement, if not a complete modification, to reduce their inherent variance.

**Neville, A. "Is Our Research Likely to Improve Concrete?" *Concrete International*, March 1995, pp. 45-47.**

This editorial criticizes concrete research as a whole, as well as the ways in which that research is incorporated into practice. According to the author, many research projects are necessarily confined to studying the effects of changing one or two variables at a time using a very limited range of concrete materials, and are often conducted at room temperature. However, because real-life conditions seldom conform to these conditions, it is very difficult to make useful generalizations based on the aggregation of these research projects. Moreover, there is real danger that the conclusions drawn from this type of research may become embedded in the literature as if they were generally valid.

**Okamoto, P.A., C.L. Wu, S.M. Tarr, M.I. Darter, and K.D. Smith, *Performance-Related Specifications for Concrete Pavements, Volume III: Appendix D—Laboratory Testing Procedures and Testing Results and Appendix E—Review of Recent Studies and***

*Specifications*, **Final Report, Publication No. FHWA-RD-93-044, November 1993, 143 pp.**

Drawing upon previous work, a prototype PRS was developed that considers the expected life-cycle cost of the as-constructed pavement as the overall measure of quality. The approach calls for measurement of in situ concrete properties and explicitly considers variability and multiple quality characteristics in the determination of pay adjustments. Extensive laboratory testing was conducted to determine material relationships needed in the prototype PRS, and a detailed test plan was developed for the evaluation of construction variables (e.g., dowel misalignment) that significantly affect concrete pavement performance, but are not currently accounted for in the specification. A computer program, PaveSpec, was developed for use with the specification in simulation and in generating pay adjustments.

This volume contains supporting documentation for the research study. Appendix D describes the laboratory testing that was conducted under this study for use in the development of the prototype PRS. The various laboratory tests are described, and a summary of the data is included in both tabular and graphical form. Appendix E contains a summary of several key studies that have been conducted on performance-related specifications and presents a summary of specifications that incorporate PRS concepts to some extent.

**Okamoto, P.A., P.J. Nussbaum, and K.D. Smith, "Guideline Recommendations for Timing Contraction Joint Sawing of PCC Highway Pavements,"** ***Proceedings: Fifth International Conference on Concrete Pavement Design, Volume 2,*** **Purdue University, April 1993, pp. 39-52.**

Timing of contraction joint sawing is essential to avert raveling at sawcut edges and uncontrolled cracking of concrete pavements. The objectives of this study were to determine the "near" and "far" joint sawcutting window of opportunity and to evaluate nondestructive testing methods and strength indicators that will assist in determining when sawcutting should be initiated. To determine "near" sawing strength indicators, sawing tests were made on large-scale concrete slabs with different aggregates and cement contents. Relationships were developed between surface raveling, sawcut acceptance rating, and compressive strength. Recommended minimum concrete strengths were developed as a function of cement content and aggregate type. Observations of sawcutting and strength development at several highway projects provided comparisons between laboratory and construction site strength development and sawcut quality. Restraint axial and curling (bending) stresses were also calculated using early age temperature histories of concrete pavements. Observations of sawcutting and strength development at several highway projects provided comparisons between construction site strength development and monitoring of crack development below sawcuts. Early age strength properties developed as part of the laboratory investigation and strengths measured in the field were used with calculated

restraint stresses to develop guidelines that can be used to determine the "far" sawcutting limit of opportunity.

**Okamoto, P.A., P.J. Nussbaum, K.D. Smith, M.I. Darter, T.P. Wilson, C.L. Wu, and S.D. Tayabji, *Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume I*, Publication No. FHWA-RD-91-079, Federal Highway Administration, February 1994, 292 pp.**

A study was conducted to provide guidelines for timing of contraction joint sawcutting to avert uncontrolled pavement cracking and to provide guidelines for early loading of pavements by construction traffic. A laboratory study of early age (4 to 24 hours) and early pavement loading (1 to 28 days) concrete strength properties for a range of highway concrete mixes was made. Sawcutting tests were made to determine earliest contraction joint sawcutting. Earliest sawcut timing was correlated on the basis of sawcut ratings to concrete strength properties and nondestructive test results that can be used to determine earliest sawcutting time. Concrete pavement placement and joint sawcutting were observed at three highway construction sites to verify test results. Latest sawcutting time was targeted on the basis of buildup of restraint stresses attributable to slab cooling. Guidelines for sawcut timing are presented to facilitate construction site decision-making based on nondestructive test methods. Early loading by construction traffic was analyzed using ILLI-SLAB finite element models. Load tests were made at two pavement sites to verify that analytical model results are applicable to new pavements. Guidelines are presented to facilitate construction site decision-making for early trafficking of new pavements based on nondestructive test methods.

**Okamoto, P.A., P.J. Nussbaum, K.D. Smith, M.I. Darter, T.P. Wilson, C.L. Wu, and S.D. Tayabji, *Guidelines for Timing Contraction Joint Sawing and Earliest Loading for Concrete Pavements, Volume II: Appendix*, Publication No. FHWA-RD-91-080, Federal Highway Administration, February 1994, 224 pp.**

This is the second volume of the report described above. It contains the appendixes to the final report.

**Okamoto, P.A., "Field Evaluation of Dowel Placement Along a Section of Interstate 45 in Texas," *Transportation Research Record 1186*, Transportation Research Board, 1988, pp. 16-34.**

This report presents the results of a field investigation conducted to determine the effectiveness of an automatic dowel bar inserter at properly placing dowel bars in rigid pavements. The study, which was sponsored by the FHWA Demonstration Projects Division, was conducted during February and March 1987 along a section of I-45, south of Dallas, Texas. A commercially available radar system, capable of quickly and easily locating reinforcing bars and other embedded steel in concrete, was used to examine the experimental dowel placement. This system produces a real-time graphic recording

that gives both location and relative depth of the embedded steel. Cores were taken to calibrate the graphic recordings so that the actual embedded depths of the steel bars could be obtained. Along one portion of the test section, a dowel bar inserter was used to place dowel bars in the plastic concrete. For the remainder of the project, dowel basket assemblies were used to place dowels at transverse joints. The radar system was then used to evaluate dowel placement along 51 basket assembly and 52 inserter joints. Results indicate that dowel bar alignment at inserter joints is as good as or better than that at the basket assembly joints.

**Oluokun, F.A., E.G. Burdette, and J.H. Deatherage, "Rates of Development of Physical Properties of Concrete at Early Ages,"** ***Transportation Research Record 1284,*** **Transportation Research Board, 1990, pp. 16-22.**

This paper presents an investigation into the gain of physical properties of concrete with time and experimental results on the development of compressive strength, splitting tensile strength, modulus of elasticity, and Poisson's ratio. The results show that the elastic modulus developed much faster than either the tensile or the compressive strength, which developed at approximately the same rate. Poisson's ratio did not change substantially with either richness of mix or age of concrete and did not vary appreciably with time after 6 h. The period within the first 12 h after casting was observed to be the period of fastest development of physical properties.

**Ozyildirim, C.,** ***4 x 8 Inch Concrete Cylinders Versus 6 x 12 Inch Cylinders,*** **Virginia Highway & Transportation Research Council, May 1984, 25 pp.**

Laboratory and field investigations were conducted to compare the compressive strengths obtained for 4- x 8-in (100- x 200-mm) cylinders with those for standard 6- x 12-in (150- x 300-mm) cylinders, both made with aggregate having a nominal maximum size of 1 in (25 mm). In the laboratory, in addition to the effect of specimen size on strength, factors such as the mold type, aggregate type, and strength level were considered. The results of the laboratory work indicate that the two sizes of cylinders yield equal compressive strength values at a strength level of about 3,200 psi (22.0 MPa). Above this level, the small specimen values between the two sizes increase with the strength level. The standard deviation of strength values was higher for the small specimens. For equal precision, more tests are needed for the small specimens than for the larger ones.

The field investigation included an examination of the effect of different specimen sizes when different types of mold and capping procedures were used for each size. The results of the field tests comparing specimen sizes were similar to those of the laboratory tests in terms of strength and variability. It was concluded that the results of tests on three small cylinders cast in plastic molds and tested with neoprene pads in steel end caps can be used to predict the strengths of A3 and A4 concretes. The variability of small cylinders compared to that of larger ones prepared in the field was

slightly higher, although not statistically significant; therefore, the use of three small cylinders (rather than two larger ones) is recommended.

**Parcells, W.H., Jr., and M. Hossain, "Kansas Experience With Smoothness Specifications for Concrete Pavements," Prepared for Transportation Research Board 73rd Annual Meeting, January 1994, 21 pp.**

There is a growing interest in the industry in attaining smoother and smoother pavement surfaces. Smoothness specifications for PCC pavements now in effect in Kansas have evolved through applications over the last 80 years. Pavement profiles of short wavelengths and smaller amplitudes than the industry-accepted 5.1 mm (0.2 in) can adversely affect the ride quality of pavements. This experience has led KDOT to eliminate the blanking band width in the profilograph trace reduction process. The implementation of this "zero" or "null" blanking band was successful and has resulted in better quality pavements. The latest proposed specifications will increase the amount of bonus that can be achieved, but might result in more grinding unless the PCC pavement pavers were able to improve the pavement smoothness in the middle (full-pay/grind) ranges. An analysis of the effect of as-constructed smoothness on the roughness history of pavements has shown that the ride quality over the service life of pavements is highly dependent on the smoothness achieved during construction. Limited cost analysis has shown an increasing amount of bonus achieved in PCC pavement construction over the last few years, indicating quality paving in Kansas.

**Parcells, W.H., Jr., *Control of Pavement Trueness in Kansas*, Interim Report, Kansas Department of Transportation, Bureau of Materials and Research, January 1992.**

The smoothness of new pavement has shown significant improvement in the 6 years that the profilograph has been required in Kansas. Changing to the zero blanking band will allow better measurement of the quality of pavement smoothness with existing equipment in the years to come.

**Patel, A.J., M.R. Thompson, and S.H. Carpenter, *Evaluation of Potential Applications of End-Result and Performance-Related Specifications, White Paper Presentations—Part II, Chapter IV—Asphalt Concrete Pavements*, Publication No. FHWA-IL-UI-255, Illinois Department of Transportation, December 1995, 185 pp.**

The primary objective of this study is to evaluate, based on the best available technology, the potential applications of end-result specifications (ERS) and performance-related specifications (PRS) to asphalt concrete paving, portland cement concrete paving, and structural concrete construction. For each area, the following tasks were performed:

- Review of current literature on ERS and PRS.

- Comprehensive evaluation of other State practices to include parameters used, lot size, and payment schedules.
- Review and evaluation of current Illinois DOT specifications, practices, and policies.
- Identification of relevant IDOT data that have potential application.
- Evaluation of the feasibility of relating materials and construction data to facility performance.
- Determination of the quality of available data and their appropriateness for data analysis.
- Statistical analysis of selected variables within available data and determination of their suitability for developing ERS and PRS.

Part II of this study discusses the framework of ERS and PRS, current practice, and data analyses for each of the three areas studied. The report also contains a literature review, summary of other State practices, review and evaluation of the IDOT specification, analyses of MISTIC data, potential linking of MISTIC and Illinois Pavement Feedback System data, and recommendations for improvements to current QC/QA specifications for structural concrete construction.

**Pechlivanidis, C., C.G. Papaleontiou, A.H. Meyer, and D.W. Fowler, *The Effectiveness of Membrane Curing Compounds for Portland Cement Concrete Pavements*, Final Report, Publication No. FHWA-TX-89-1118-1F, Texas State Department of Highways & Public Transportation, November 1988, 66 pp.**

Membrane curing compounds are widely used to cure concrete in highway construction. The function of these compounds is to form a membrane that helps retain the moisture in the concrete slab that is otherwise lost through evaporation. The amount of evaporation loss varies as a function of the environmental conditions and the temperature of the concrete mass during the curing period. This report provides an evaluation of the performance of membrane curing compounds as related to concrete material properties such as tensile and flexural strength, stiffness, surface durability, and density. In addition to traditional testing methods, the nondestructive, in situ Spectral Analysis of Surface Waves (SASW) method is also used to observe and measure material properties as a function of time. Testing can start at initial set or when the modulus of elasticity for concrete is about 10,000 psi (69.0 MPa).

**Pitt, J.M., R.A. Carnazzo, J. Vu, and M. Seshadri, *Control of Concrete Deterioration Due to Trace Compounds in Deicers*, Final Report, Publication No. Iowa DOT Project HR-299, Iowa Department of Transportation, Highway Division, May 1992, 88 pp.**

This report contains work on three facets of rock salt deicer action on freeze-thaw resistance of portland cement concrete. The first deals with improvement of mortars, where it is demonstrated that a 10- to 15-percent replacement of cement with fly ash can more than double the life of concrete by the reduction of porosity and stabilization of calcium hydroxide. Excessive fly ash was found to counteract this benefit. Second, this research defines behavior and performance of aggregates displaying different service lives in concrete-subjected deicers. Freeze-thaw in water produced failure in the aggregate, whereas in deicers, damage was exclusive to the mortar-aggregate interface or the mortar. Aggregate porosity appeared to be a good, but not infallible, predictor of concrete service life. Low-porosity aggregates were best. The third feature of this research was development of a test method capable of modeling the freeze-thaw process and predicting life performance. This was done by adaptation of the ASTM C 666 test to include a reliability-based design. Essential to this test method was the objective definition of failure and a realistic model linking laboratory tests to the temperature observed in the field.

The methodology was compared to performance of a pavement in central Iowa. The model predicted life of 25 years, and the pavement actually failed at 26 years. This predictive method was also used to contrast the life of a pavement subjected to different deicing materials.

**Poblete, M., P. Ceza, J. David, R. Espinosa, A. Garcia, and J. Gonzalez, "Model of Slab Cracking for Portland Cement Concrete Pavements,"** ***Transportation Research Record 1307,*** **Transportation Research Board, 1991, pp. 154-161.**

A predictive model of slab cracking has been developed by considering the actual behavior of Chile's PCC pavements. The approach used is mechanistic-empirical and is based on a structural modeling of the jointed pavement using a finite elements program. The model incorporates realistic boundary conditions of upward concave slabs under yearly cyclic moisture warping and daily cyclic temperature curling obtained from measurements in the climatic zone of central Chile. Fatigue consumption in the slabs was evaluated for the actual traffic loads passing during each group of hours in each season to determine the different levels of upward concavities. The accumulated fatigue consumption of all pavements was calculated and plotted against the percentage of cracked slabs observed in the field. The resulting model is a unique S-shaped curve that is useful for indicating the effect of many design variables on cracking.

**Raczon, F., "Pavement Smoothness Goal of PCC Slipform Paving,"** ***Roads & Bridges,*** **April 1989, pp. 73-74.**

Smoothness has become an important test for PCC pavements in recent years as agencies strive to obtain scientific evidence to support road users' complaints. Profilograph measurements and other related technology are becoming more common

in end-result specifications, and the demand for pavement smoothness is greater than ever before. Faced with increased pressure to produce the smoothest pavements possible, slipform paver operators need to balance a variety of factors in order to achieve the desired concrete pavement quality.

**Rix, G.J., J.A. Bay, and K.H. Stokoe II, "Assessing In Situ Stiffness of Curing Portland Cement Concrete With Seismic Test," *Transportation Research Record 1284*, Transportation Research Board, 1990, pp. 8-15.**

In situ measurement of surface-wave velocity using the Spectral Analysis of Surface Waves (SASW) method shows that this type of seismic measurement offers a reliable alternative to conventional penetration resistance and cylinder compression testing for determining the stiffness of PCC during curing. Both surface-wave velocity and penetration resistance exhibit similar rates of increase during the initial stages of curing, suggesting that in situ measurements of wave velocity are potentially useful for assessing the degree of curing that has occurred. At later stages of curing, values of Young's moduli calculated from in situ seismic tests agree well with values of Young's moduli from cylinder compression tests for similar curing histories. Advantages of seismic tests such as the SASW test are that unlike penetration resistance and cylinder compression tests, they: (1) require no samples, (2) can be performed directly on the concrete slab to evaluate spatial variability, and (3) can be performed repeatedly at the same locations at different times during the curing process.

**Ross, F.R., and T.S. Rutkowski, *A Designed System of Load Transfer Dowels for Joints in PCC Pavement*, Final Report, Applied Research Section C.O. Materials, Division of Highways and Transportation Services, Wisconsin Department of Transportation, January 1989, 40 pp.**

The intent of the research was to determine the need for load transfer dowels at contraction joints in plain concrete pavement on moderately loaded facilities (10,000 average daily traffic [ADT] and 10 to 14 percent heavy vehicles), as well as to determine if a modified dowel system with a reduced amount of steel is an adequate replacement for the standard Wisconsin dowel system. A plain concrete pavement was built containing three separate test sections, intended to represent three levels of cost. Section A had skewed, random-spaced contraction joints with no load transfer dowels. Section B had right-angle joints spaced at 20 ft (6.1 m), with the modified dowel system. Section C was similar to B except that it had the standard dowel system.

Measurements were made of initial horizontal and vertical dowel alignment, and periodic monitoring of load transfer efficiency (LTE), pavement serviceability index (PSI), pavement distress, seasonal horizontal joint movement, and faulting was performed. Pavement temperature, joint width, and load transfer efficiency were related. The LTE drops sharply for joint openings greater than 0.040 in (1.0 mm), which occurs at slab temperatures less than 75 °F (23.9 °C). At 32 °F (0.0 °C), the LTE of

doweled joints is 95 percent compared to 75 percent for non-doweled joints. The measured LTE of non-doweled transverse joints was significantly less than that of doweled transverse joints. The modified dowel design had an incidence of faulting equal to that of the standard dowel system and significantly less than the non-doweled system in both quantity and severity. Undoweled joints have a 60-percent probability of developing faulting, compared to 15 percent for doweled joints. The modified dowel section had less spalling than the other two sections and a relatively equal incidence of cracking; the severity of these characteristics could not be attributed to the presence of lack of joint transfer. The present serviceability index (PSI) of the three test sections has remained relatively equal over the 12-year life of the pavement and has not approached the traditional threshold value at which rehabilitation actions would be considered.

None of the measured characteristics had an apparent effect on the pavement performance. It is recommended that moderately loaded pavements be constructed with non-doweled, random skewed, plain pavement systems, or modified dowel systems using joint spacings of 30 ft (9.1 m) or greater.

**Roy, D.M., *Concrete Microstructure*, Quarterly Report, Strategic Highway Research Program, July 1989.**

This report delineates curing conditions with potential risk of damage to the concrete. For concrete mixtures where sulfate-resistant cement is used, no curing problem will occur if concrete temperatures are between 50 and 80 °F (10.0 and 26.7 °C) and constant air temperatures are between freezing and 100 °F (37.8 °C). For concrete mixtures with ordinary portland cement, the problem-free curing conditions are similar. As expected, the safe intervals for concrete mixtures using hardening cement are smaller; for mixtures with up to 620 lb/yd$^3$ (367.8 kg/m$^3$) of concrete, temperatures between 40 and 80 °F (4.4 and 26.7 °C) and air temperatures between 40 and 100 °F (4.4 and 37.8 °C) are required.

**Sandberg, U., "Mobile Contactless Profiling of Road Texture Saves Time and Money," *Selcom*, Swedish Road and Transport Research Institute, 1995.**

This short article discusses the development and probable approval of a standardized method for determination of pavement surface texture. At the same time, PIARC was conducting an experiment to compare texture measurement devices and to ascertain whether texture values can help to clarify frictional characteristics of the pavement/tire interaction. The results showed that the texture measuring devices correlated well with each other, and some correlated well with friction.

**Saraf, C.L., and B.F. McCullough, "Controlling Longitudinal Cracking in Concrete Pavements," *Transportation Research Record 1043*, Transportation Research Board, 1985, pp. 8-13.**

The objective of the study reported in this paper was to investigate the development of longitudinal cracks in wide concrete pavements (two or more lanes in one direction) and to develop a model to estimate the depth of sawcut needed to control these cracks within the groove. The model developed uses the concepts of variability in the material properties of the concrete (tensile strength), pavement thickness (as constructed in the field), and depth of sawcut groove. It was observed that estimates of longitudinal cracking have a reasonable match with field observations. It was observed that the longitudinal cracking of concrete pavements (two or more lanes in one direction) was dependent on the type of aggregate used in the concrete mix. Two types of aggregates were investigated. Uniformity of concrete mix strength (tensile) represented by standard deviation (tensile strength) affected the development of longitudinal cracks. A lower value of standard deviation obtained using lime rock aggregate in the mix was responsible for confining more cracks within the sawcut compared with the mix using river gravel aggregate. A sensitivity analysis of the model indicated that a substantial reduction in sawcut depth can be achieved if the variability of concrete strength during construction can be reduced.

**Sayers, M.W., "Profiles of Roughness," *Transportation Research Record 1260*, Transportation Research Board, 1990, pp. 106-110.**

Road roughness is normally characterized by a summary index that applies over a length of road. Summary index measures are obtained most directly by measuring the longitudinal profile and then applying a mathematical analysis to reduce the profile to the roughness statistic. The moving average smoothing filter can be used to obtain a profile of one such roughness measure—the international roughness index (IRI). The roughness profile provides another dimension to the description of roughness, showing with maximum detail how the roughness is distributed over the length of the road. The base length used for IRI averaging must be considered. Specifying the base length becomes particularly important when specifications for road quality are formulated, or when profiling accuracy is prescribed. When reporting instrument accuracy or writing roughness specifications, it should be taken into account that the variation in IRI found over the length of a road is more extreme when the base length is short. Specifically, the accuracy of high-speed profiling systems should be specified according to base length.

**Schrader, E.K., "A Solution to Cracking and Stresses Caused by Dowels and Tie Bars," *Concrete International*, 13(7): 40-45.**

This report discusses using square dowel bars with a clip-on plastic sheath having compressible material on its sides as a solution to cracking and stresses caused by dowels and tie bars. The use of square bars instead of round bars provides total vertical load transfer while allowing a compressible material to be attached to the side of the bar. By using the clip-on sheaths, as a joint opens, the side material simply compresses rather than forcing the bar to bend. Square bars also provide more resistance to

bending, so they provide more resistance to edge and corner curling for the same amount of steel. By using square bars, the tensile splitting stresses in a slab induced by round bars are avoided. Square dowels may be used as tie bars if they are altered to resist pullout. Corrosion protection is also improved by using the plastic sheaths.

**Scofield, L.A., *Profilograph Limitations, Correlations, and Calibration Criteria for Effective Performance-Based Specifications*, NCHRP Project 20-7, National Cooperative Highway Research Program, 1993.**

The purpose of this study was to assess the state of the practice in the use of profilographs for measurement of pavement smoothness. The critical objectives were to evaluate the nature and extent of problems and to recommend research to accomplish solutions to these problems. A survey of States and industry identified problems and determined the state of the practice. A literature search was performed, and a limited analysis of the automated profilograph filters was conducted.

The results of the survey indicated that 90 percent of the respondents believe that smoother pavements reduce life-cycle costs and that smoothness requirements will increase in the future. Currently, the majority of the States use the profilograph for acceptance of pavement smoothness. Approximately 25 percent of the profilographs used by the States are computerized. The five highest ranked problems, as reported in the State survey, include comparing profilographs to other roughness measurement devices, trace reduction repeatability, effect of short wavelengths on profile index, interpretations of profilograph traces, and production rate of testing.

The literature search revealed that the variability of mechanical profilographs increases with pavement roughness. The variability of the computerized profilographs is constant and typically ranges between a standard deviation of 0.5 to 1.0 in/mi (7.9 to 15.8 mm/km). For pavement roughness levels below 5 in/mi (78.9 mm/km), the mechanical and computerized profilographs exhibited similar variability. The variability of mechanical profilographs is attributable to trace reduction. High degrees of correlation exist between mechanical and computerized profilograph test results. While Ames and California-style profilographs produce similar results, Rainhart and California-style profilographs often do not.

The results of this study indicate a strong need to develop national standards for both the mechanical and computerized test procedures.

**Scofield, L.A., S.A. Kalevela, and M.R. Anderson, "Evaluation of California Profilograph," *Transportation Research Record 1348*, Transportation Research Board, 1992, pp. 1-7.**

The Arizona DOT evaluates PCC pavements by testing with mechanical as well as electronic profilographs. The precision of the two types of profilograph was evaluated.

More than 100 profilograph runs were conducted on a selected pavement section. The range of replicate readings of pavement profile index could be as much as 2.0 in/mi (31.6 mm/km) for a rough pavement. Electronic profilographs adjusted to operate at low filter settings gave lower profile index values than those obtained with the same profilographs at higher filter settings.

**Scofield, L.A., S.A. Kalevela, M.R. Anderson, and A. Hossain, *A Half Century With the California Profilograph,* Publication No. FHWA-AZ-SP9102, Federal Highway Administration, February 1992, 58 pp.**

This study was performed to establish equipment and operator variability for mechanical and computerized California profilographs. Future work, based on testing conducted during this study, should develop precision and bias statements for profilographs.

The research consists of two phases. Phase 1 included a literature review, field testing, and statistical analysis. The historical development of the profilograph and California test procedures and specifications were evaluated in relationship to today's incentive/disincentive specifications. Additionally, equipment parameters that influence test variability were reviewed.

**Segner, E.P., Jr., and J.R. Cobb, *A Study of Misaligned Dowels in Concrete Pavements,* HPR Report 32, Alabama Highway Department, August 1967, pp. 1-42.**

The primary objectives of this report were to study the effects of misaligned dowel bars across transverse contraction joints in concrete pavement and to determine the maximum dowel bar alignment error that can be tolerated without seriously impairing the effectiveness of the joint. The conclusions in this report are based on results obtained from testing simulated concrete pavement specimens containing dowel bars aligned with various alignment errors.

**Semenov, V.A., "Quality Control in Highway Construction and Maintenance When the Measurement Parameters Are Highly Nonuniform," *Transportation Research Record 1126,* Transportation Research Board, 1987, pp. 28-36.**

In this paper, an original method is presented for quality control based on Weibull's law for the distribution of random quantities with variable parameters. The nomographs obtained for determining the extremal values of the parameters and the defectiveness index can be used for various cases in the statistical reduction of research results. The proposed method can be used for quality control for both highly uniform data (in the United States, the measured values are assumed to have a normal distribution for this purpose) and highly nonuniform data. The method described was widely used in the Soviet Union for quality control in the construction and maintenance of roads.

**Sharaf, E.A., M.Y. Shahin, and K.C. Sinha, "Analysis of the Effect of Deferring Pavement Maintenance," *Transportation Research Record 1205*, Transportation Research Board, 1988, pp. 29-35.**

This paper presents a methodology for quantifying increased maintenance and rehabilitation (M&R) costs due to delaying M&R action. The methodology used data collected from several U.S. Army installations. These data included: (1) pavement performance measured in terms of the Pavement Condition Index (PCI), (2) structural history, and (3) detailed costs. The methodology is based on applying life-cycle cost analysis to determine the Equivalent Uniform Annual Cost (EUAC) for various M&R alternatives. This was repeated at different pavement condition levels. The EUAC analysis for each M&R alternative included pavement surface preparation cost, initial cost of the M&R alternative, and future annual routine maintenance cost. The results of this study showed that considerable savings can be achieved if pavement sections are maintained while they are in good condition and are not allowed to deteriorate into poor condition.

**Shook, J.F., and M.A. Diaz, *Performance-Related Specifications for Asphalt Concrete—Phase II, Proposed Plans for Accelerated Pavement Test Track Study*, Publication No. FHWA-RD-91-071, Federal Highway Administration, January 1992, 62 pp.**

The plan for an accelerated pavement test track study described in this report is part of a Federal Highway Administration (FHWA) program to develop PRS for an asphalt concrete pavement construction by: (1) conducting laboratory studies of the relationships between materials and construction (M&C) variables and fundamental response variables, as well as the response variables and pavement performance indicators, and (2) developing a detailed plan (experimental design, construction details, and data collection and analysis) for an accelerated field test at a test track facility. The objectives of the accelerated field experiment are:

- To verify (or modify, if necessary) the prediction equations developed in the laboratory phase of the project, "Performance-Related Specifications for Asphalt Concrete—Phase II," under realistic field conditions.

- To relate the predicted performance of test sections obtained from the equations to actual performance defined by pavement ride quality, or distress observed on the test sections when subjected to real traffic and environmental conditions.

- To develop data that can be used to extend the findings to traffic and environmental conditions not included in the original study.

- To develop data that can be used to extend the findings to materials not included in the original study.

- To develop data that may be used to confirm or modify existing relationships between mixture properties and pavement performance variables.

- To use and test, during construction of the facility, the applicability of methods for determining pay factors.

**Shook, J.F., M.A. Diaz, M. Stroup-Gardiner, and S.B. Seeds, *Performance-Related Specifications for Asphalt Concrete—Phase II*, Publication No. FHWA-RD-91-070, Federal Highway Administration, January 1992, 230 pp.**

The laboratory study performed for this research project focused on the development of secondary prediction relationships, which are equations that establish the relationship between M&C variables and fundamental response variables. The results of the laboratory study indicate that compaction level had more influence on mixture properties than any other variable in the experiment. A technique for estimating compaction effects using measurable specimen properties was found and resulted in a compaction index equation. The prediction equations can be used with an estimated compaction index to relate measured mixture properties to optimum properties. These equations can also be used to predict relative effects of proposed changes in materials and construction specifications on performance-related mixture properties. When used with equations that relate performance-related mixture properties to pavement performance, the equations derived in this study can be used to establish penalties for non-conformance to specification limits. An automated version (Lotus 1-2-3 spreadsheet) of the conceptual PRS system is included.

**Sipherd, C., "The Role of Internal Vibration and the Extrusion Process Slipform Paver in Consolidation: Portland Cement Concrete Pavement Construction," Prepared for Transportation Research Board Annual Meeting, January 1995, 21 pp.**

These presentation notes provide practical recommendations for construction using slipform paving and internal vibration.

**Smith, G.L., and B.R. McWaters, "Partnering for Performance—Iowa's Experience With Design and Construction Enhancements for Quality Improvement of Concrete Pavements," *Transportation Research Record 1478*, Transportation Research Board, 1995, pp. 62-66.**

Partnering is a new paradigm for owner and contractor relations that emphasizes up-front team building, a clear definition of common objectives, synchronized customs for rapid issue resolution, and joint evaluation of partnership effectiveness. Construction partnering has existed in a limited form for years, but such relationships have found some difficulty in a system in which the low bidder wins the contract, the taxpayer dollar is at stake, and the government may be sensitive to criticism of not spending the

money wisely. Today, new partnering concepts to protect the public interest have been developed by contracting authority and contractor alliances with both parties. Most partnering has been between a contracting authority and contractor with a mutual intent to construct a specific project. This paper describes a more general partnership among the Iowa DOT, other Iowa contracting authorities, and the Iowa Concrete Paving Association. In 1992, Iowa contracting agencies and the concrete paving industry began an Iowa initiative for continuing quality improvement of concrete pavements. Through the Iowa partnership, changes have been made in standard designs, specifications, materials use, and construction techniques to enhance concrete pavement quality.

**Snell, L.M., "Assessing Concrete Technicians," *Concrete International*, March 1995, pp. 42-44.**

The quality of testing is dependent on the ability of the technician. Managers should use an established procedure to assess technicians and determine if problems are likely to occur. The procedures described in this paper present a methodology for identifying testing problems and allow mangers to identify if a technician needs to improve testing procedures. This procedure can help to improve the overall testing and provide positive feedback so that a technician and testing laboratory can assess and improve testing services.

**Soriano, A., *Dowel Bar Placement and Displacement*, Publication No. SK91-09-X, South Dakota Department of Transportation, March 1995, 26 pp.**

This report describes research performed by the SDDOT to investigate the problem of dowel bar misalignment in PCC pavements. More than 400 dowel bars from three construction projects were inspected and compared to the dowel bar alignment specifications. This study found that the dowel bars tended to conform to the alignment specifications even though, in the opinion of the researcher, the specifications were too rigid. It was noted that sawing of the contraction joint was the governing factor for the longitudinal displacements. Although no pavement distress was observed in areas containing dowel bar misalignments, it should be noted that only 2 years had elapsed between project construction and the field survey. It is the opinion of the researcher that this is insufficient time for pavement distress to manifest itself, since even undoweled PCC pavement may not show distress for 5 years or more.

**Steele, G.W., S.B. Hudson, and C.J. Van Til, "The Statistical Approach to Realistic Highway Specifications," Prepared for 25th Annual Convention of SASHO, September 1996, 25 pp.**

This paper presents the step-by-step procedure whereby statistical concepts are employed to incorporate statistical parameters, based on West Virginia research data, into specification text. Numerous figures, flowcharts, and tables are used to illustrate the procedures and to demonstrate that only knowledge of arithmetic is needed to

design an acceptance plan. The statistical approach is extended to the derivation of a penalty system that will permit acceptance of marginal material at a reduced price on a formal contract basis.

**Strauss, P.J., and M.F. Mitchell, "The Performance of Jointed Concrete Pavements as Affected by Factors Other Than Traffic-Induced Stress,"** ***Proceedings: Fourth International Conference on Concrete Pavement Design and Rehabilitation,*** **Purdue University, Publication No. FHWA-RD-89-208, April 1989, pp. 467-472.**

Design and construction techniques developed overseas have been used in Southern Africa on PCC pavements with little variation based on local experience. This paper describes the methodology by which jointed unreinforced PCC pavements are being evaluated, with the aim of improving local design and construction techniques. Indications are that failures found on the pavements can often be attributed to inadequate construction techniques and control. Although the pavements are performing very well, it is felt that an improvement in performance is possible if subgrades are compacted properly to reduce settlement; non-erodible subbases are constructed; load transfer at joints is enhanced by using low-shrinking concrete; joints are kept properly sealed; and PCC-paved shoulders are used.

**"Surveying the Many Curing Methods and Materials,"** ***Concrete Construction,*** **October 1976, pp. 488-492.**

It would be difficult to overemphasize the importance of curing in concrete construction. The watertightness, hardness, durability, and strength of concrete all depend, to a great extent, on curing practices. This article presents a short summary of the types and characteristics of curing methods available, including water, burlap, and materials that retain moisture.

**Szalay, T.W., "City of Windsor Concrete Pavement Design and Construction Experience,"** ***Proceedings: 1991 TAC Annual Conference, Volume 1,*** **1992.**

The City of Windsor has prepared a presentation regarding the culmination of its 25 years of experience in concrete pavement design and construction. This report highlights the specific features of the equipment utilized and the construction methods used. The report also features a discussion of state-of-the-art components of concrete pavement technology, such as slipform paving, newer load transfer basket designs, joint spacing and preparation, finishing with trowels and bump-cutter techniques, texturing and curing, sawcutting joints and sealing, profilograph measurements for smoothness, and testing procedures.

**Tang, T., D.G. Zollinger, and B.F. McCullough,** ***Field Tests and Analyses of Concrete Pavement in Texarkana and La Porte, Texas,*** **Publication No. FHWA-TX-94-1244-7, Texas Department of Transportation, May 1994, 310 pp.**

This report summarizes the research results obtained from two field tests for PCC pavements. One field test was carried out on test sections of jointed plain concrete pavement in Texarkana, Texas, and the other was carried out on test sections of continuously reinforced concrete pavement in La Porte, Texas. Laboratory tests and theoretical analysis were also performed to help in understanding and analyzing the field observation. A closed-form solution is proposed for thermal stresses in a concrete slab when it is curled up. This solution is for the case where the temperature decrease in the concrete slab exceeds a limit so that a gap forms between the slab and the foundation. This case was not addressed in the Westergaard solution. Fracture tests were applied to the concrete made with different coarse aggregates used for Texas pavements and have shown that fracture mechanics is a powerful tool with which to judge the quality of the pavement. With these efforts, a method based on fracture mechanics is proposed to determine the depth and spacing of the jointed plain concrete pavement. This method has successfully been applied to the test section in Texarkana.

**Tayabji, S.D., "Dowel Placement Tolerances for Concrete Pavements," *Transportation Research Record 1062*, Transportation Research Board, 1986, pp. 47-54.**

The results of an investigation conducted to develop placement tolerances for dowels at concrete pavement joints are presented. A theoretical analysis of dowel misalignment was attempted. The purpose of the analysis was to compute restraint stresses induced in the concrete pavement for different levels of dowel misalignment. However, because of the complexity of correctly incorporating the three-dimensional nature of dowel misalignment, the theoretical analysis was not completed. The effect of dowel misalignment was then investigated in the laboratory by conducting pullout tests on sections of concrete slabs incorporating a joint and dowels with different levels of misalignment. Test results indicate that pullout loads were relatively low for dowel misalignment levels of less than 1 in per 18-in (25.4 mm per 457.2-mm) length of dowel bar and a maximum joint opening of 0.25 in (6.4 mm).

**Tayabji, S.D., and P.A. Okamoto, "Field Evaluation of Dowel Placement in Concrete Pavements," *Transportation Research Record 1110*, Transportation Research Board, 1987, pp. 101-109.**

Presented in this paper are the results of a laboratory and field investigation conducted to determine the effectiveness of the radar device for evaluating dowel bar misalignment and to evaluate the effectiveness of an automatic dowel bar inserter to properly place dowel bars in rigid pavements. A commercially available radar system was used. The system produces a real-time graphic recording that indicates the location and the relative depth of the embedded steel. Cores are taken to calibrate the graphic recordings to obtain the actual embedded depth of the steel bars. The laboratory study indicated that dowel bars placed about 5 in (127 mm) below the concrete surface could be located reasonably accurately by the radar system. In the laboratory, the standard

deviation obtained for the differences between actual and measured individual readings was 0.24 in (6.1 mm). The field evaluation was conducted during June 1986 along a section of Interstate 86 in Idaho. At this project, an inserter was used to place the dowel bars in the plastic concrete. Dowel placement along 16 transverse joints was evaluated with the radar system, and results indicate that the radar system can determine the location of dowel bars placed in concrete pavements. However, the degree of accuracy is operator-dependent and test results must be considered in statistical terms.

**Tayabji, S.D., and D. Whiting, "Field Evaluation of Concrete Pavement Consolidation," *Transportation Research Record 1110*, Transportation Research Board, 1987, pp. 90-100.**

Inadequate consolidation of concrete can result in weak concrete that may lead to premature failure or loss of serviceability of the pavement. Presented in this paper is the result of a comprehensive study on concrete pavement consolidation conducted by Construction Technology Laboratories, Inc., and sponsored by the Federal Highway Administration. The scope of the work consisted of laboratory testing, evaluation of nuclear gauges, development of model acceptance plans for concrete consolidation, and field implementation of the acceptance plans. In laboratory tests, consolidation was found to have a strong influence on compressive strength, bond of concrete to reinforcing steel, and permeability of concrete. There was a lesser effect of consolidation on resistance to freezing and thawing. A loss of 30 percent was sustained in compressive strength for every 5-percent decrease in consolidation. A variety of nuclear density gauges was evaluated for use in monitoring consolidation of concrete. Information was obtained from literature sources and State highway agencies. Typical precision on field concrete ranges from 1 to 2 $lb/ft^3$ (16 to 32 $kg/m^3$) for most gauge types. A combination of techniques, such as the consolidation monitoring device and commercial direct transmission gauges, shows promise as a means of monitoring consolidation during the paving process. A model acceptance sampling plan for concrete consolidation is proposed. The plan is of the inspection-by-variables type and requires a sample size of eight per lot. The plan provides for buyer's and seller's risks of 5 percent. A field trial of a model acceptance plan carried out along a section of I-86 in Idaho indicated that it is practical and cost-effective to monitor concrete consolidation in the field.

**Temple, W.H., and S.L. Cumbaa, "Serviceability Index Base for Acceptance of Jointed Concrete Pavements," *Transportation Research Record 1196*, Transportation Research Board, 1988, pp. 251-256.**

This paper describes the techniques and relationships developed to design a Serviceability Index (SI)-based measurement system for acceptance of jointed concrete pavement construction in Louisiana. Pavement roughness statistics obtained from Mays Ride Meter equipment, a Surface Dynamics Profilometer, and a Chloe

Profilometer were regressed to establish an AASHO Road Test-based SI measurement system for concrete pavements with 20-ft (6.1-m) joint spacings (SI JCP 20). A 1986 panel rating of 25 concrete pavements confirmed the validity of the model. Field testing of 50 newly constructed concrete pavement test sections provided a relationship between the SI JCP 20 model and profile statistics from rolling profilograph equipment and a 10-ft (3-m) rolling straightedge. The research resulted in the development of a rational method of providing specification limits for profilograph equipment that relate to pavement rideability. Specification limits in terms of profile statistics are provided to indicate the quality of paving necessary to construct a jointed concrete pavement with a Serviceability Index of 4.5.

**Thompson, M.R., and B.J. Dempsey, *Durability Testing of Stabilized Materials*, Publication No. UILU-ENG-74-2010, Illinois Department of Transportation, June 1974, 65 pp.**

The primary purpose of IHR-401, "Durability Testing of Stabilized Materials," was to develop a satisfactory and realistic procedure for evaluating the freeze-thaw durability of partially cemented highway materials (e.g., soil-cement, lime-flyash-aggregate, lime-soil mixtures). The following accomplishments were achieved during the project:

Phase 1. A heat-transfer model was developed and utilized to quantitatively characterize field pavement freeze-thaw conditions for Illinois.

Phase 2. An automatic freeze-thaw testing apparatus and a test procedure were developed. An accelerated vacuum saturation test procedure and a cured strength correlation relation were established for predicting freeze-thaw durability properties.

Phase 3. The equipment, concepts, and procedures established in Phases 1 and 2 were utilized to conduct an extensive laboratory freeze-thaw testing program with typical Illinois stabilized materials.

Phase 4. A durability evaluation procedure based on the residual strength concept was proposed. The procedure recognizes three distinct durability zones in Illinois.

Phase 5. Specific recommended procedures for considering freeze-thaw durability are outlined in the report.

To aid in refining certain aspects of the proposed durability evaluation procedure, some areas will require further investigation (field-mixing efficiency, construction quality control, field curing, minimum tolerable strengths, and field studies of freeze-thaw durability performance).

**Weed, R. M., *Composite Pay Equations: A General Approach*, New Jersey Department of Transportation, March 1994, 23 pp.**

Highway construction specifications involving the acceptance testing of several different quality characteristics are sometimes confusing and difficult to administer. A procedure is developed by which multiple quality measures may be combined in a rational manner in a single, composite pay equation. This approach is scientifically sound and may be applied to virtually any construction specification for which a relationship between quality and performance is known or can be approximated. An example based on portland cement concrete pavement is presented to illustrate the practicality of this method.

**Weed, R.M., "Equitable Graduated Pay Schedules: An Economic Approach," *Transportation Research Record 691*, Transportation Research Board, 1978, pp. 27-29.**

An approach for establishing graduated pay schedules that are realistic, equitable, and legally defensible is presented. The method consists of determining the present worth of the extra expense anticipated as a result of work of deficient quality. An appropriate pay schedule is developed on the premise that it would be justifiable to withhold this amount from the contract price. The method is applicable for construction items for which data are available that relate quality to performance. An example is given in which concrete pavement is evaluated in terms of compressive strength.

**Whiting, D.A., and S.D. Tayabji, *Relationship of Consolidation to Performance of Concrete Pavements*, Publication No. FHWA/RD-87/095, Federal Highway Administration, February 1988, 144 pp.**

A study was made of the influence of consolidation on properties of portland cement concrete. Consolidation was found to have a strong influence on compressive strength, bond of concrete to reinforcing steel, and permeability of concrete. There is a lesser effect of consolidation on resistance to freezing and thawing. There is a loss of about 30 percent in compressive strength for every 5-percent decrease in consolidation. A variety of nuclear density gauges were evaluated for use in monitoring consolidation of concrete. Use of these gauges has remained fairly constant, at relatively low levels, since 1977. A combination of techniques, such as a consolidation monitoring device (CMD) and commercial direct-transmission gauges, shows promise as a means of monitoring consolidation during the paving process. A model-acceptance sampling plan for concrete consolidation is proposed. The plan is of the inspection-by-variables type and requires a sample size of eight per lot. The plan provides for buyer and seller risks of 5 percent. Field testing in Idaho and Iowa indicated that monitoring concrete pavement consolidation is practical and economically feasible.

**Willenbrock, J.H., and P.A. Kopac, "Development of Price-Adjustment Systems for Statistically Based Highway Construction Specifications," *Transportation Research Record 652*, Transportation Research Board, 1977, pp. 52-58.**

This paper presents a methodology that can be used to develop price adjustment systems for use in statistically based highway construction specifications. Three approaches are proposed for the development of a price adjustment system: the serviceability approach, the cost-of-production approach, and the operating characteristic curve approach. The three approaches are discussed and compared, and their most appropriate applications are recommended. A fourth approach, the cost-of-quality control approach, is also discussed, but it is not fully developed because of the limited cost data available.

**Willenbrock, J.H., and P.A. Kopac, *A Methodology for the Development of Price Adjustment Systems for Statistically Based Restricted Performance Specifications*, Publication No. FHWA-PA-74-27(1), Pennsylvania Department of Transportation, October 1976, 169 pp.**

This report presents a methodology that can be used to develop price adjustment systems suited for statistically based, restricted performance specifications. An example of the use of this methodology for a specific situation, that of bituminous concrete paving for the Pennsylvania DOT, is also presented.

Three approaches are proposed for the development of a price adjustment system: the performance-of-serviceability approach, the cost-of-production approach, and the trial-and-error operating characteristic (OC) curve approach. A cost-of-quality control approach is also examined, but cannot be fully developed at this time because of the limited cost data available.

**Wood, S.L., *Evaluation of the Long-Term Properties of Concrete*, Research and Development Bulletin RD102T, Portland Cement Association, 1992, 34 pp.**

Variations in concrete compressive strength, flexural strength, and modulus of elasticity with time are discussed. Concrete specimens made from five types of portland cement, portland blast-furnace slag cement, portland-pozzolan cement, and air-entraining cement were tested at ages between 1 day and 34 years. Relationships between the compressive strength at various ages and the 28-day compressive strength are developed for specimens stored in moist and dry environments. Flexural strength and modulus of elasticity are related to the compressive strength.

**Woodstrom, J.H., *Measurements, Specifications, and Achievements of Smoothness for Pavement Construction*, Synthesis of Highway Practice 167, National Cooperative Highway Research Program, 1990, 34 pp.**

Smoothness-measuring equipment currently used in new pavement construction includes straightedges (static and rolling), profilographs, response-type road-roughness-measuring systems, and inertial profilometers. The fundamentals of operation, cost, and appropriateness to address a specific need vary considerably.

Certain devices are far better suited than others to the purpose of controlling the smoothness of newly constructed pavements. Therefore, it is important for those concerned with obtaining smoothness in construction to be aware of the equipment best suited for that purpose and the relationship of that equipment to the entire spectrum of smoothness-measuring devices.

Numerous research efforts, as well as symposia and workshops, have been directed toward providing information on the issue of smoothness-measuring equipment. Although there are vast differences in equipment types and their ultimate application, the relationships of several smoothness indexes have been compared and are reasonably well-defined.

A survey of practices in use in the United States and Canada revealed great diversity in the use of smoothness specifications and equipment. However, emphasis on smoothness by specifying agencies, together with strong support form the construction industry, has led to the attainment of increasingly smoother pavements.

**Wu, S., and H.M. Hearne, Jr., "Performance of Concrete Pavement With Econocrete Base," *Proceedings, 4th International Conference on Concrete Pavement Design and Rehabilitation*, Purdue University, 1989, pp. 683-695.**

A three-lane, 16-mi (25.7-km) section of I-85 in Randolph and Davidson Counties, North Carolina, was opened to traffic in 1984. The spring following the opening of this section of highway, reddish-brown stains were visible on the concrete shoulders. Water was determined to be flowing under the slabs and blowing out through weak points along the joints. The data collected in an attempt to understand this phenomenon included horizontal and vertical movements of the slabs, static and dynamic pavement deflections, temperature gradients of the pavement system, location of dowels in the slabs, concrete and soil material properties, and field observations to identify water movement. Possible causes of the problems were identified, and methods to minimize damage to the pavement were recommended.

**Zia, P., and M.R. Hansen, "Durability of High-Performance Concrete," *Pacific Rim TransTech Conference Proceedings. Volume II*, American Society of Civil Engineers, 1993, pp. 398-404.**

This is a summary of a study of the freeze-thaw (F/T) durability and resistance to chloride penetration of high-performance concrete (HPC) developed for highway applications. The tests covered here include the freeze-thaw test, rapid chloride permeability test (RCPT), and impedance test. It was found that the F/T resistance of the HPC exceeded the enhanced F/T durability requirement by a considerable margin. This can be achieved if the concrete has a minimum of 5 percent well-entrained air and the coarse aggregate used in the concrete is F/T-resistant. Despite its high F/T

resistance, HPC may indicate quite high coulomb values in the RCPT because of the additional ions introduced into the concrete by the various admixtures.

**Zollinger, D.G., T. Tang, and D. Xin, "Sawcut Depth Requirements for Concrete Pavement Based on Fracture Mechanics Analysis," Prepared for the 72nd Annual Meeting of the Transportation Research Board, January 1993, 52 pp.**

The efficient control of slab cracking is important to concrete pavement performance. From the viewpoint of engineering analysis and the design of pavements, it is desirable to control pavement cracking to joint locations to decrease the possibility of *uncontrolled* cracking. The purpose of this paper is to suggest an approach to estimate appropriate sawcut depths and placement timing using fracture mechanics for jointed concrete systems. Early age sawcutting, as one form of crack induction, has been applied to concrete pavement surfaces at specific contraction joint locations. This paper presents a mechanics-based approach to the determination of sawcut depth and spacing requirements using fracture mechanics analysis. The stress field in a concrete slab induced by thermal and shrinkage gradients is based on curling and warping analysis, which also leads to sawcut spacing criteria. Fracture parameters Klf and Cf defined by the size effect law are obtained from laboratory-notched beam fracture tests for specific coarse aggregate types. Modified linear elastic fracture mechanics is applied to determine a sufficient notch or sawcut depth to ensure controlled cracking. The theoretical sawcut depth, as determined by fracture analysis, is significantly less than the conventional d/3 or d/4, where d is the slab thickness. Recent pavement surveys have verified this conclusion.

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