LATERITE AND LATERITIC SOILS AND OTHER PROBLEM SOILS OF THE TROPICS

AN ENGINEERING EVALUATION AND
HIGHWAY DESIGN STUDY FOR
UNITED STATES AGENCY FOR
INTERNATIONAL DEVELOPMENT

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TABLE OF CONTENTS

Acknowledgements 1

Historical Background 3

Summary 7

Chapter 1. Engineering Implications of Tropical Weathering and Laterization 11
- Weathering 11
- Profiles of Weathering 12
- Hydrated and Dehydrated Laterite 12
- Conclusion 12

Chapter 2. Regional Settings 15
- Introduction 15
- Summary of the Geology and Physical Geography 15
- Climate and Vegetation 27
- References Cited or Consulted 35

Chapter 3. Tropical Soil Development 37
- Introduction 37
- Clay Mineralogy and Clay Materials 37
- Tropical Soil Formation 39
- Soil Profile and Soil Identification Properties 47
- Tropical Red Residual Soils 51
- Tropical Black Clays 69
- References 82

Chapter 4. Geotechnical Properties of Red Tropical Soils 85
- Introduction 85
- Moisture-Sensitive Soils of Volcanic Regions 85
- The Red Soils of South America and Central America 92
- Red Soils of Africa 109
- Red Soil Groups of Southeast Asia 121
- Similarities and Differences Among Red Soils of the Three Continents 125
- References 127

Chapter 5. Test Procedures and Standards 129
- Introduction 129
- Test Procedures 129
- Summary 134
- References 134

Chapter 6. Pavement Performance Study 135
- Introduction 135
- Scope 135
- Deflection and Pavement Performance 135
- Deflection and Pavement Strength 146
- Conclusions 156
- References 158

Chapter 7. Design of Flexible Pavements in the Tropics 159
- Introduction 159
- Structural Design Curves 159
- Structural Design Process 161
- Flexible Pavement Design 163
- Design Tables 165
- Design Limitations 167
- Pavement Overlay Concepts 167
- Asphalt Equivalents 167
- Design of Unpaved Roads 171

Drainage Considerations 171
Conclusions 173
References 175
Chapter 8. Durability and Repetitive Load Tests for Ironstone and Concretionary Gravel 177
Introduction 177
Durability Studies in South America 177
Durability Studies in Southeast Asia 182
Repetitive Load Testing 188
Conclusions 192
References 192
Chapter 9. Stabilization of Selected Lateritic Gravels and Soils 193
Introduction 193
Stabilization Studies in Africa 193
Stabilization Studies in Southeast Asia 208
Stabilization Studies in South America 214
Conclusions 215
References 220
Chapter 10. Tropical Black Clays 221
Introduction 221
Literature Review 221
Test Procedures and Results 226
Discussion of Properties 231
Design Considerations for Roads Over Tropical Black Clays 239
Recommended Design Procedure 241
References 243
Chapter 11. Air Photo Interpretation 246
Introduction 246
Previous Important Work 246
Identification of Laterites and Other Tropical Soils 250
General Conclusions Regarding Terrain Evaluations 251
References 251
Chapter 12. Suggested Specifications for Materials and Construction in Tropical Climates 253
Introduction 253
Specifications for Subbase Base and Surface Course Materials 256
Specifications for Excavation of Borrow Areas, Compaction Equipment and Compaction Requirements 260
Specifications for Materials and Construction in Tropical Climates 262
References 262

Appendix for Chapter 4 263
Appendix for Chapter 6 288
Appendix for Chapter 7 307
Appendix for Chapter 8 326
Appendix for Chapter 9 337
Appendix for Chapter 10 346
Appendix for Chapter 12 354

LIST OF TABLES

Table No.	TITLE	Page No.
3.1	Classification of the Principal Clay Minerals	38
3.2	Soil Horizon Nomenclature	47
3.3	Types of Soils Structure	50
3.4	Cation Exchange Capacities for the Clay Minerals and Other Materials	50
3.5	Selected Suborder Nomenclature for the New Soil Taxonomy	53
3.6	Former U.S. Soil Classification and the Approximate Equivalent Orders and Suborders	54
3.7	Approximate Correlation of the FAO Classification System with those of the U.S. Soil Taxonomy and the French Classification System	56
3.8	Chemical Composition of Selected Laterites and Lateritic Soils	57
3.9	Silica-Sesquioxide Ratios of South African Soils	57
3.10	Chemical Composition of the Clay Fraction of Some Indonesia Black Clays	74
3.11	Partial Chemical Analyses of South African Black Clays	74
3.12	Partial Chemical Analyses of the Clay Fraction of a Soil Profile at Ata, India	75
4.1	Properties of Soils over Volcanic Rocks, Central America, Java and New Guinea	91
4.2	Mean Values of Properties, Red Tropical Soils of South America	96
4.3	Minerals Identified in Each of the Major Soil Groups Expressed in Percentage of Occurrence	96
4.4	Typical Soil Test Results for Ferruginous Soils of Africa	111
4.5	Typical Soil Test Results for Ferrallitic Soils of Africa	111
4.6	Typical Soil Test Results for Ferrisols of Africa	111
4.7	Mean Soil Properties, Ferrallitic Soils, Ferrisols and Ferruginous Soils of Africa	112
4.8	Mean Soil Properties, Groups and Subgroups, Africa	113
4.9	Engineering Properties of Typical Concretionary Gravels, Thailand	122
5.1	Change in Atterberg Limits after Drying at Various Temperatures and Time Periods	130
6.1	Unit Equivalent Standard Axle Loading Factors	141
6.2	Equivalent Standard Axle Loading Factors for DNER and DER Classification Systems	141
6.3	Average Values of Deflections for Design Criteria	145
6.4	Permissible Tensile Strains under Different Load Applications	147
6.5	Structural Coefficients	152
7.1	Pavement Coefficients for Flexible Pavement Design	161
7.2	Recommended Type and Thickness (cm) of Surface Course	163
7.3	Comparison of Overlay Procedures	167
7.4	Equivalent Pavement Designs	171
8.1	Physical Properties of Samples Tested	180
8.2	Results of Durability Tests	181
8.3	Artificial Sample Gradations Used in Durability Studies	183
8.4	Natural Sample Gradations used in Durability Studies	183
8.5	Summary of Test Results, Southeast Asia	187
8.6	Summary of Test Data, South America	189
9.1	Properties of Soils Investigated in Africa	196
9.2	Design Content of Lime and Cement Stabilized Materials, Africa	204
9.3	Stabilization Test Data, Africa	205
9.4	Properties of Soils Investigated in South America	216
9.5	Percent Cement Required Based on pH Test	217
9.6	Cement and Lime Stabilization Results, South America	217
9.7	Unconfined Compressive Strengths, South America	217
9.8	Applicability of Stabilization Methods	219
9.9	Broad Comparison of Stabilization Techniques	219
10.1	Distribution of Dark Clay Soils	222
10.2	Properties of Tropical Black Clays	224
10.3	Chemical Properties, Tropical Black Clay	227
10.4	Physical Properties, Tropical Black Clays	229

10.5 Swelling Properties, Tropical Black Clays 230
10.6 Data for Making Estimates of Probable Volume Change for Expansive Soils 238
10.7 Test Data on Lime Stabilization of Black Clay Soils 241
10.8 Value of Factor F with Depth D from the Relation D = 20 log F 242

11.1 Airphoto Interpretation of Tropical Soils - Summary 248
11.2 Main Characteristics Used to Determine Physiography 249
11.3 Drainage Characteristics 249

12.1 Specifications, Base and Subbase Materials 256
12.2 Recommended Moisture Ranges for Evaluation of Design CBR Values 259
12.3 Recommended Gradations 260
12.4 Recommended Compaction Requirements for Component Structural Layers and Natural Subgrade 262

A4.1 Chemical and Geological Properties of Red Tropical Soils of South America 263
A4.2 Physical and Engineering Properties of the Red Tropical Soils of South America 266
A4.3 Chemical Analysis 269
A4.4 Mineral Identified by X-Ray Diffraction 271
A4.5 Properties of African Soils 274
A6.1 Rating, Deflection and Traffic 288
A6.2 Physical and Engineering Properties of Individual Structural Layers of Test Pits 291
A6.3 Structural Properties 303
A7.1 Design Table – DBST – Subbase CBR = 25 309
A7.2 Design Table – AC 5 cm, Subbase CBR = 25 310
A7.3 Design Table – AC 10 cm, Subbase CBR = 25 311
A7.4 Design Table – AC 15 cm, Subbase CBR = 25 312
A7.5 Design Table – DBST – Subbase CBR = 30 313
A7.6 Design Table – AC 5 cm, Subbase CBR = 30 314
A7.7 Design Table – AC 10 cm, Subbase CBR = 30 315
A7.8 Design Table – AC 15 cm, Subbase CBR = 30 316
A7.9 Design Table – DBST – Subbase CBR = 35e 317
A7.10 Design Table – AC 5 cm, Subbase CBR = 35 318
A7.11 Design Table – AC 10 cm, Subbase CBR = 35 319
A7.12 Design Table – AC 15 cm, Subbase CBR = 35 320
A7.13 Design Table – DBST – Subbase CBR = 40 321
A7.14 Design Table – AC 5 cm, Subbase CBR = 40 322
A7.15 Design Table – AC 10 cm, Subbase CBR = 40 323
A7.16 Design Table – AC 15 cm, Subbase CBR = 40 324
A7.17 Comparison of Design Procedures 325
A9.1 Stabilization Test Data 337
A10.1 Table for Converting Proving Ring Reading to PVC Category and Approximate Plasticity Index 348
A12.1 Specifications for Rapid-Curing (RC) Liquid Asphalt 354
A12.2 Specifications for Medium-Curing (MC) Liquid Asphalt 355
A12.3 Specifications for Anionic Emulsified Asphalts 356
A12.4 Specifications for Cationic Emulsified Asphalts 357
A12.5 Specifications for Stabilization 362
A12.6 Conversions for Figure A12.1 361
A12.7 Conversions for Figure A12.2 365
A12.8 Conversions for Figure A12.3 366
A12.9 Spreading Temperatures for Cutback Asphalts 367
A12.10 Spreading Temperatures for Emulsified Asphalts 367

LIST OF ILLUSTRATIONS

FIG. No.	TITLE	Page No.
2.1	Major Physiographic Features of South America	16
2.2	Major Physiographic Features of Central America	18
2.3	A Generalized Map of the Tectonic Basins of Africa with Intervening Swells, Plateaus and Rift Valleys	20
2.4	Rift Valley System in Africa	22
2.5	Major Physiographic Features of Southeast Asia	24
2.6	Mean Annual Precipitation in South America	26
2.7	Major Vegetation of South America	28
2.8	Mean Annual Precipitation in Central America	29
2.9	Major Vegetation Regions or Central America	30
2.10	Mean Annual Precipitation in Africa	31
2.11	Major Vegetation Regions of Africa	32
2.12	Mean Annual Precipitation in Southeast Asia	33
2.13	Major Vegetation Regions of Southeast Asia	34
3.1	Sketch of Formation of Weathering Mantle in Areas That Are Tectonically Inactive	41
3.2	Eletron Micrograph of a Soil from Ethiopia	43
3.3	Eletron Micrograph of a Soil from Ethiopia	44
3.4	Outline of Weathering in the Tropical Soils of the Hawaiian Islands Showing Primary, Intermediate, and End Products of Pedological Weathering	45
3.5	Progressive Types of Clay Development in Hawaiian Soils Under a Climate Having Alternating Wet and Dry Seasons	46
3.6	Progressive Types of Clay Development in Hawaiian Soils Under Continuously Wet Climate	46
3.7	Transformations During Weathering	46
3.8	A Hypothetical Soil Profile Having all the Soil Horizons	48
3.9	Red Tropical Soils of South America	60
3.10	Red Tropical Soils of Central America	61
3.11	Red Tropical Soils of Africa, French System	62
3.12	Red Tropical Soils of Africa, FAO System	63
3.13	Red Tropical Soils of Southeast Asia	64
3.14	Red Tropical Soils of Australia	65
3.15	Red Tropical Soils of India and Ceylon	66
3.16	Areas with Ironstone and Concretionary Gravel in South America	67
3.17	Areas with Ironstone and Concretionary Gravel in Africa	68
3.18	Areas with Ironstone and Concretionary Gravel in Southeast Asia	70
3.19	Occurrences of Ironstone and Concretionary Gravel in Australia	71
3.20	Occurrences of Ironstone and Concretionary Gravel in India	72
3.21	Water Balance Diagram for Some Brazilian and African Locations	73
3.22	Major Occurrences of Black Clays in South America	76
3.23	Major Occurrences of Black Clays in Central America	77
3.24	Major Occurrences of Black Clays in Africa	78
3.25	Major Occurrences of Black Clays in Southeast Asia	79
3.26	Major Occurrences of Black Clays in Australia	80
3.27	Major Occurrences of Black Clays in India	81
4.1	Distribution of Andosols and Ash Mixed Soils	87
4.2	Relationship of Halloysite	90
4.3	Locations of Some of the Volcanic Soils on the Casagrande Chart	93
4.4	Location of Tropical Red Soil Samples in Brazil	94
4.5	Sample Locations for Phase III, South America	95
4.6	Ferralsols of South America Gradation Limits	98
4.7	Acrisols of South America Gradation Limits	98
4.8	Arenosols of South America Gradation Limits	99
4.9	Luvisols of South America Gradation Limits	99

4.10 Nitosols of South America Gradation Limits 100
4.11 Ferralsols on the Casagrande Plasticity Chart 101
4.12 Acrisols on the Casagrande Plasticity Chart 102
4.13 Arenosols on the Casagrande Plasticity Chart 102
4.14 Luvisols on the Casagrande Plasticity Chart 103
4.15 Nitosols on the Casagrande Plasticity Chart 103
4.16 Locations of the Tropical Red Soils of South America on the Casagrande Chart 104
4.17 Relationship Between Optimum Moisture Content and Plastic Limit, South American Soils 104
4.18 Relationship Between Optimum Moisture and Clay Content, South America 105
4.19 Relationship Between Specific Gravity and Fe_2O_3 Content (%) South American Soils 105
4.20 General Correlation Between CBR and AASHO Classification, for Red Tropical Soils of South America 106
4.21 CBR vs Plasticity Index x Percentage Passing No. 200, South American Soils 107
4.22 CBR vs Plasticity Index x Percentage Passing No. 40, South American Soils 108
4.23 Location of Soil Samples in Africa 110
4.24 Ferruginous Soil Group-Gradation Limits, Africa 114
4.25 Ferrallitic Soil Group-Gradation Limits, Africa 114
4.26 Ferrisol Soil Group-Gradation Limits, Africa 115
4.27 Locations of the Tropical Red Soils of Africa on the Casagrande Chart 116
4.28 Relationship Between Maximum Dry Density and Optimum Moisture Content, Africa 117
4.29 CBR vs Plasticity Index x Percentage Passing No. 40, Africa 118
4.30 CBR vs Plasticity Index x Percentage Passing No. 200, Africa 119
4.31 Suitability of Material as Base Course on the Basis of Maximum Dry Density, at AASHO Modified Density, and Granulometric Modulus of CBR Specimen 120
4.32 Plasticity Index vs Liquid Limit – Data Points from Lateritic Soils Southeast Asia 126
5.1 Comparison of Physical Properties Tested With and Without Pre-Drying 130
5.2 Wet Test Set up With Mechanical Sieve Shaker 132
5.3 Comparison Between 5 Points Liquid Limit and Liquid Limit Determined by Single Point Method 133
6.1 Measurement of Deflection Parameters MRD and TAN ϕ 137
6.2 Adjustment Factor Curves for Use in Correcting Beam Deflection to a Standard Temperature of 21°C 138
6.3 Frequency of the Load Distribution on Axles of Commercial Vehicles in Brazil, in Percent 140
6.4 General Relation Between Total Daily Number of Vehicles and Equivalent 18 KSAL-Curves Based on Vehicle and Load Distribution Obtained from Loadmeter Study 142
6.5 Relationship Between Deflection and Performance 144
6.6 Percent Variation Within the Test Section 145
6.7 Relation Between Design Deflection and Traffic for the Practical Range of Coefficients of Variation 147
6.8 Relationship Between the Slope of the Deflection Basin and Cracking of the Pavement Surface for Various Standard Axle Applications 148
6.9 Stress Distribution Beneath the Surface of a Flexible Pavement 150
6.10 Principles of Pressure Distribution 152
6.11 Critical CBR Values With Depth Below Pavement Surface Based on Field Analysis 153
6.12 Comparison Between Measured Deflections and Calculated Deflections, Surface Treatment Pavements 154
6.13 Structural Coefficients Versus Thickness of Asphalt Surfacing 154
6.14 Effect of Asphalt on Pavement Strength (SI) for Various Thicknesses of Asphalt and Unbound Soil Layer Strengths. 155
6.15 Comparison Between Measured Deflections and Calculated Deflections, Asphalt Concrete Pavements 157
6.16 Relationship Between Slope Index and Thickness of Asphalt Concrete for the Practical Range of Base Course CBR Values 157
7.1 Comparison Between Measured Deflections and Calculated Deflections 159
7.2 Structural Design Curves for Determining the Required Structural Index of the Standard Payment Section 160
7.3 Minimum Design Thicknesses for All Pavement Courses Based on CBR Values of Underlying Layers 161
7.4 Subgrade Support and Minimum Thickness of Cover Based on Subgrade CBR Values 162
7.5 Combined Strength of Asphalt and Unbound Soil Layers, 5 cm Asphalt Concrete 164
7.6 Combined Strength of Asphalt and Unbound Soil Layers, 10 cm Asphalt Concrete 164
7.7a Combined Strength of Asphalt and Unbound Soil Layers, 15 cm Asphalt Concrete 164
7.7b Combined Strength of Asphalt and Unbound Soil Layers, 20 cm Asphalt Concrete 165
7.7c Combined Strength of Asphalt and Unbound Soil Layers, 25 cm Asphalt Concrete 165
7.8 Daily Intensity of Standard Axle Loading vs Total Standard Axle Loadings 166

7.9 Reduction in Deflection Achieved by Overlays of Different Thicknesses Placed on an Existing Surface Treatment 168
7.10 Reduction in Deflection Achieved by Overlay Placed on an Existing Asphalt Concrete Surface 168
7.11 Effect of Increasing Thickness of Asphalt on High Strength Pavements 169
7.12 Substitution Ratios for Various Base Course Materials 170
7.13 Percent Runoff Entering Pavement Cracks of Various Width 173
7.14 Typical Cross Sections-Drainage 174
8.1 Location of Durability Samples in Brazil 178
8.2 Critical Dimensions of Slake-Durability Test Equipment 179
8.3 A Suggested Durability Classification 179
8.4 Relationship Between Percentage Retained and Number of Slaking Cycles Including a Gravel Classification Based on Observed Performance 181
8.5 Relationship Between Loss in Percent Versus Number of Slaking Cycles 182
8.6 Relationship Between Slake Durability and Observed Performance 182
8.7 Variation of Coarse Durability (Dc) With Respect to Grading 184
8.8 Effect of Petrologic Composition on Dc and L.A.A. Test Results 185
8.9 Variation of Los Angeles Abrasion (L.A.A.) With Respect to Grading 186
8.10 Standard vs Non Standard L.A.A. Tests on the Same Material 187
9.1 Location of Stabilization Samples in Ghana 194
9.2 Location of Stabilization Samples in Brazil 195
9.3 Flow Chart for Lime and Cement Stabilization in Africa 196
9.4 Stabilization Test Results, Africa 198
9.5 Stabilization Test Results, Africa 199
9.6 Stabilization Test Results, Africa 200
9.7 Stabilization Test Results, Africa 201
9.8 Dry Density and Unconfined Compressive Strength of Laterite Soils 202
9.9 Comparison Between Split-Tensile and Unconfined Compressive Strength, Cement 206
9.10 Comparison Between Split-Tensile and Unconfined Compressive Strengths, Lime 206
9.11 Comparison Between Compressive Strength and CBR, Cement 207
9.12 Comparison Between Compressive Strength and CBR, Lime 207
9.13 Results of Compressive Strength Tests on Cement and Cement-Lime Stabilized Specimens, Variable Moist Cure Time Series 209
9.14 Strength of Cement and Cement-Lime Stabilized Aggregates at 30 Days and at 4 to 6 Months 210
9.15 Results of Compressive Strength Tests on Cement & Cement-Lime Stabilized Specimens, Wet-Dry Cycle Series . 211
9.16 Stabilometer Value Before and After Moisture Vapor Susceptibility Test 212
9.17 Cohesiometer Value Before and After Moisture Vapor Susceptibility Test 212
9.18 Permeability and Swell Test Results 213
10.1 Gradation Limits for Black Clays 228
10.2 Moisture-Density-CBR Relationships, African Black Clays 232
10.3 Relationships Between Expansion-Pressures and Dry Densities at Various Initial Moisture Contents, African Black Clays 233
10.4 Effect of Surcharge Load on Volume Change 234
10.5 Plasticity Chart for Black Clays of South America and Africa 234
10.6 Relationship Between Compacted Density and Liquid Limit 235
10.7 Relationship Between Compacted Density and PI x Percentage of Fines 236
10.8 Relationship Between Optimum Moisture Content and Plastic Limit 237
10.9 Potential Expansiveness of Soils 237
10.10 Minimum Water Content Required For Soil Liquid Limit 239
12.1 Relationship Between Liquid Limit and CBR at Various Percentages of Material Passing No. 200 Sieve, African Data 254
12.2 Relationship Between Plasticity Index and CBR at Various Percentages of Material Passing the No. 200 Sieve, African Data 254
12.3 Relationship Between Plasticity Index, Percent Passing No. 200 Sieve and Soaked CBR, South America Data . . 254
12.4 Relationship Between Liquid Limit, Percent Passing No. 200 Sieve and Soaked CBR, South America Data 255
12.5 Relationship Between CBR and Granulometric Modulus, South America Data 255
12.6 Classification of Base Course Materials 257
12.7 Relationship Between Optimum Moisture Content and In-Situ Moisture Content-Base Courses 258
12.8 Relationship Between Optimum Moisture Content and In-Situ Moisture Content, Subbase Courses 258
12.9 Annual Rainfall vs Coefficient of Equilibrium Moisture, South America 259

12.10 Required Density With Depth Below Surface of Pavement 261
A6.1a Typical Results of Laboratory Testing 300
A6.1b Typical Results of Laboratory Testing 301
A6.1c Typical Results of Laboratory Testing 302
A8.1 Relationship Between Resilient Modulus and Number of Load Applications 326
A8.2 Relationship Between Resilient Modulus and ϕ 331
A8.3 Classification of Gravels Based on Resilient Modulus After 100,000 Load Applications 336
A10.1 Pictures of Equipment – PVC 346
A10.2 Proving Ring Calibration From Federal Housing Administration Publication No. 701 347
A10.3 Swell Index vs Potential Volume Change From Federal Housing Administration Publication No. 701 347
A10.4 Swell Index vs Plasticity Index From Federal Housing Administration Publication No. 701 347
A10.5 Fixed-Ring Consolidometer 350
A10.6 Load-Expansion Curves 353
A12.1 Single Pass Surface Treatment 364
A12.2 Double Surface Treatment 365
A12.3 Triple Surface Treatment 366
A12.4a Selection of Bituminous Binder for Surface Treatments or Seal Coats in Tropical Climates Determined by Surface Temperatures and Size of Cover Aggregate 368
A12.4b Selection of Bituminous Binder for Surface Treatments or Seal Coats in Tropical Climates Determined by Surface Temperatures and Size of Cover Aggregate 369

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Some countries participated in this study locally. Public Works Engineers in Peru, Paraguay, Guyana, Nicaragua and Costa Rica ran special tests or collected special samples for the study.

The names of those who assisted were:

Paraguay: Mr. P.S. O'Shaughnessy, USAID Engineer; Ing. Juan H. Pettengill, Director-General for Highways.

Guyana: Mr. Quentin Wildman, AID Engineer; Mr. P. Allsopp, Chief Engineer of Works and Communications; Dr. Sobharam Singh, Commissioner of the Geological Survey.

Bolivia: Ing. Hasan An Hasan, AID; Ing. Grover Carranza, AID; Mr. Dan Protopopeasch, Coas Ing., Consultants; Ing. Mario Antezana, Chief Engineer of Research and Constructions, National Highway Department; Eng. Erwin Moreno Anez, Chief Santa Cruz District, National Highway Department.

Colombia: Mr. Hector Camargo S., AID; Dr. Pablo Lederman, Chief of Materials and Engineering; Dr. Ellis Knox, Soil Scientist; Dr. Carlos Luna Z., Soil Scientist.

Peru: Mr. Dallas Fowler, AID; Ing. Mario Quiroga, AID; Ing. Fernando Alba Manfredi, Subdirector of Special Studies.

Panama: Mr. A. Donato, AID; Mr. Virgilio Hinojosa, AID; Prof. Amador Hasel, University of Panama; Mr. Tony Mann, Soils Engineer, Panama Canal Company; Mr. Giorgio Resky, Chief of the Mineral Resources Department.

Nicaragua: Mr. Fred Alvarez, AID; Mr. Roberto J. Atha R., AID; Ing. J.J. Bustos, Chief, Materials Testing Section; Ing. Mario Gutierres, Nicaragua Highway Department.

Costa Rica: Mr. Heriberto Rodrigues, AID; Ing. Miguel Larrod T., Chief Engineer, Materials Testing Section; Ing. Ezequiel Vieto Solis, Soils Engineer; Ing. Adrian Ramirez, Soil Engineer.

Guatemala: Mr. F.J. Maiolo, AID; Ing. Ankerman, Chief of the Soils Laboratory; Ing. Roberto Lou Wah, University of San A. Carlos.

El Salvador: Ing. George Hewit, Cadessa Engineering Consultants; Ing. Melecio Eduardo Rivera, Rivera-Harrouch and Associates; Ing. Moises Harrouch, Rivera-Harrouch and Associates.

Honduras: Mr. Figueroa, AID; Mr. W. Nichol, Brown and Root.

Finally, appreciation is also expressed for the assistance and support received during the seminars which were held in Rio de Janeiro, Brazil and Lima, Peru near the conclusion of the study. Our hosts on these occasions were the DNER and the Peruvian Soils Mechanics Committee respectively, and support was given by Petroleos Del Peru who were kind enough to furnish their auditorium for the seminar in Lima.

This has only been a partial list of participants in this important landmark study of the engineering properties and uses of tropical soils. We were pleased for the opportunity to meet and work with our contemporary engineers in Latin America and Africa. It simply would not have been possible to conduct this study in such a broad spectrum without their able assistance and support.

Frank E. Lyon Jr., President
LYON ASSOCIATES, INC.

HISTORICAL BACKGROUND

One of the highest priorities of the tropical countries is the development of transportation and communication systems. The basic need in transportation is an adequate road system.

Commerce and communication between countries is essential to stimulate regional development. Feeder roads are urgently needed to allow development of raw materials and agricultural potential. Existing roads have to be strengthened and improved to withstand the present and projected volumes of traffic.

Due to the increasing high cost of road construction it is essential that available funds be utilized in an efficient manner. The design of pavements must be economical for the soil conditions and the traffic loads. This point is readily conceded. Two others however should be emphasized because they are so often neglected. Proper maintenance must be exercised as a never-ending effort. Too many road facilities have been lost or seriously impaired because funds have been diverted to more obvious, tangible works. All road facilities, no matter how well designed, required adequate maintenance. The second factor is construction control. This principle is also fundamental, but often neglected. Many countries do not have adequate staff, trained personnel, or necessary equipment to verify that the construction materials and techniques are following the design specifications. In order to insure that allocated funds are not wasted on sub-standard construction, control and supervision must have a high priority.

Pavement design practices in the tropics have evolved, by and large, from American and European experience. These procedures were devised through years of experience, but with temperate-zone soils. The adequacy of these procedures have never really been evaluated for the climatic conditions, the soils, and the traffic patterns of tropical countries.

Organizations such as the Road Research Laboratory of the United Kingdom, and the Centre Experimental de Recherches et d'Études du Batiments et des Travaux Publics of France have made major contributions in modifying the usual empirical procedures and these modifications have been adopted with, generally, considerable success.

Nevertheless, the question of the available materials, their quality and their adequacy in road structures have never been fully investigated. Therefore the United States Agency for International Development funded a worldwide research study of lateritic materials which are the most plentiful tropical soil. The purpose of this major research effort was to provide satisfactory engineering definitions, classifications and utilization criteria for these materials to insure adequate but not overly conservative design and construction.

Accordingly, a contract was awarded in June, 1967 to Soil and Pavement Consultants of Southeast Asia for Phase I of the program. Phase I was conducted in Bangkok, Thailand with the assistance of the Royal Thailand Department of Highways. The area under study was southeast Asia, with emphasis on Thailand.

The contract for Phase II was awarded in June, 1968 to Lyon Associates, Inc. Phase II was based in Kumasi, Ghana and the area studied included all of tropical Africa, with emphasis on Ghana. The Building and Road Research Institute, of the Ghana Council of Scientific and Industrial Research, actively participated in the study with their staff engineers and geologist. They provided laboratory technicians and assistants as well as laboratory and office facilities.

It was decided that the study should be truly regional, incorporating as many countries in Africa as time and travel would allow. In this particularly important aspect, the scope of work was carefully followed from the beginning. The consultants and staff engineers for Lyon Associates considered an in-depth study of the Ghanaian lateritic materials of limited value to other tropical African countries. The broader regional study was largely possible because of the amount of work already performed in Ghana by the Building and Road Research Institute, since without this background of locally developed knowledge, it would not have been possible to devote as much time on regional investigations.

The contract for Phase III was awarded in June, 1972 to Lyon Associates, Inc. Phase III was based in Rio de Janeiro, Brazil and the area studied included tropical South America with emphasis on Brazil. The National Highway Department of Brazil actively participated in the study with their staff engineers and geologist. They provided the services of their local and state laboratories. The cooperation of the National Highway Department made it possible to increase the scope of the study which contributed greatly to the correlations of engineering properties and the development of the Flexible Pavement Design Procedures for the Tropics.

The objectives of the program were well described in the Terms of Reference for the Phase II study. These prestudy objectives and some mid-study alterations are summarized below.

1. To develop an engineering definition of laterite and lateritic soils and other problem tropical soils. The major emphasis was on lateritic soil and tropical black clays, often termed black cotton soils. The black clays are undoubtedly the most difficult soils for road building in Africa.
2. To propose a suitable classification system for lateritic materials that adequately describes their engineering properties.
3. Evaluate the engineering characteristics of these soils and determine the range of properties for each soil classification.
4. To determine suitable test procedures that properly evaluate these materials.
5. To study methods for stabilizing substandard lateritic and other problem soils of the tropics.
6. To develop suitable specifications, design standards and criteria for the satisfactory use of these

materials in highways and airfields. This study was limited to highways because the design procedures, etc., are essentially similar for airfield and specifications and standards proposed for the one could be adopted for the other.

7. To evaluate the adequacy of these lateritic materials by assisting in the construction of test roads. Construction funds precluded the construction of test roads. This was not considered particularly detrimental, however, since test roads are difficult to design and construct in such a way that all extraneous variables are held constant and only those under investigation isolated and varied for evaluation. Usually a number of test sections are required to evaluate such variables as subgrade soil, climate and traffic conditions. Consequently it was felt that "test on roads" were superior to "test roads". This program of tests on roads was conducted as the pavement performance survey.

8. To appraise the various methods presently in use in the tropics for pavement and roadway design. The pavement condition survey provided a suitable means for evaluating these procedures and proposing modifications or changes.

9. To prepare a usage manual for laboratory and field use for lateritic materials. The manual was originally considered as a useful summary of the project for senior engineers and technicians. This was subsequently altered in the Phase II study when a more important audience was visualized, namely, junior technicians and inexperienced engineers who are unfamiliar with soil testing practices. The soil testing manual originally designed by Harvard University, under the direction of A. Casagrande and S. Wilson, for the U.S. War Department in 1945 (TB 5-253-1), and subsequently adopted by the American Road Building Association, was considered ideal for this purpose. Large sections were copied with added material generally limited to testing procedures peculiar to lateritic materials.

10. To study the potential application of lateritic soils to other engineering construction. This was necessarily limited to reviewing the literature and recounting the practical experience of the consultants and engineers in a number of countries. The other applications considered were: earth dams, slopes, foundations and building materials.

The Phase II report was divided into 18 chapters with each covering a specific topic included in the Terms of Reference. Chapters 2 through 14 dealt with tropical and lateritic soils and Chapters 15 through 17 dealt with tropical black clays. A review of the pertinent literature was provided for both types of soils and the physical and engineering properties of both were also described. The genesis or laterization process was discussed in detail in a sepatate chapter. Recommended test procedures and descriptions of modified or non-standard tests were provided. The results of stabilization studies were reported for both lateritic soils and tropical black clays.

One of the most important aspects of the Phase II study was the pavement condition survey, described in Chapter 12. This study allowed the evaluation of the material specifications and provided a basis for a pavement design procedure based on the results of the study.

Such a detailed analysis was not possible on other problem soils, but design procedures and considerations were included for roads over tropical black clays, Chapter 17. These procedures facilitate the construction of adequate roads by minimizing volume change or swelling to tolerable limits.

The usage manual had been printed separately in booklet form and included many illustrations. This facilitated its use both in the field and in the laboratory. Test procedures and descriptions were straight-forward and comprehensible to junior and inexperienced technicians and engineers who were required to set up and operate laboratory facilities in the field.

The objectives of the Phase III study are described in the Terms of Reference as:

A. STEP ONE (lateritic soils)

1. Redefine laterite clarifying the differences between the two earlier reports insofar as practical. Develop equivalency factors for the various types of qualities of laterite and lateritic materials that may be used in road construction. These equivalency factors will be suitable for use with the design curves for determining the various thicknesses of pavement structure components. Design curves will be based on CBR values.

2. Establish geological and engineering properties and determine if comparable groups such as ferruginous soils, etc., occur in Brazil with predictable ranges of properties and determine which soils are moisture-sensitive. Utilize agricultural soils maps, geology and rainfall information plus basic soils and mineralogical tests and perform parametric correlations.

3. Organize and train state laboratories to perform pavement evaluation on road sections covering the range of properties for each soil group plus the range of traffic and road conditions. Develop material standards for use in design and construction of pavements. With CBR values evaluate previous design curves and modify or establish new ones.

4. Determine suitability of using lime and cement and possibly other local economic materials to stabilize lateritic soils and gravels. Plan test sections for design following completion of stabilization testing. The study will include investigation analysis and recommendations of all various components of the highway pavement structure (i.e. sub-base, base and wearing surface). The study will also include recommendations for laterite for subgrade, including various qualities of lateritic material and, where poor qualities are involved, give various recommendations for treatment, if considered economical. Make recommendations for surfacing of various qualities of laterite. Con-

sideration will also be given to surfacing requirements where economics dictate stage construction employing laterite as surfacing material.

5. Perform dynamic tests with long-term repetitive loads to duplicate traffic loads on selected representative samples (say 12) covering range of properties of each major soil group.
6. Providing air photos are released by the Brazilian Government, develop location techniques for lateritic soils.
7. Establish specifications for laterite and lateritic materials for use in construction contracts.

B. STEP TWO (Principal Problem Soils)

1. Determine the extent, nature, geological and engineering properties of expansive and volcanic soils based on geological, climatic and pedological information and, through physical, mineralogical and swell tests, determine which are moisture-sensitive.
2. Establish stabilization requirements through an appropriate test program.
3. Establish standards for design and construction of roads over these materials including stabilization requirements based on Step Two tests and Step One pavement evaluation plus experience in literature.

The final report of the study which includes the results of all three Phases is prepared in two volumes. Volume I is divided into 12 chapters covering the specific topics included in the Terms of Reference of the Phase III Study. The formation and engineering properties of tropical soils are given in Chapter 1 through 4. Testing methods are given in Chapter 5. Chapters 6 and 7 deal with the development of the pavement design procedure. Chapter 8 covers durability of the natural aggregates. Chapter 9 deals with stabilization of tropical soils. Chapter 10 covers the black clays of tropical regions. Terrain evaluation is covered in Chapter 11. Specifications are presented in Chapter 12.

Volume II is a technical manual to be used by field engineers and technicians. The information is presented in a direct form to be used as a guide in the engineering evaluation of tropical soils and in the design of flexible pavements in the tropics.

SUMMARY

This is a summary of the final report on tropical soils and their engineering uses. The final report culminates a five years worldwide research study that resulted in documenting the following useful information for engineers.

- Comprehensive DESCRIPTION OF TROPICAL SOILS including their chemical, mineralogical and physical properties and engineering behaviors, as well as their appropriate classification.
- A new PAVEMENT DESIGN PROCEDURE developed in the tropics for tropical applications.
- A practical range of strength values attainable by STABILIZING TROPICAL SOILS.
- SPECIFICATIONS for common tropical materials used in highway construction.
- Methods of working with and using PROBLEM SOILS, such as, black clay and volcanic soils.

This engineering information and related information is outlined briefly below and detailed in the text. Chapters 1, 2 and 3 of the text concern background information about the location, development and classification of tropical soils. Chapters 4 and 5 concern the identification and measurement of geotechnical properties, especially the physical properties which are an important engineering consideration.

Chapters 6 and 7 concern the development and use of the new pavement design procedure. Chapters 8 and 9 concern the durability and stabilization of the structural base materials of a pavement. Chapters 10 and 11 are specialty chapters which concern the problems of black clay soils and the use of remote sensing techniques in locating and classifying soils respectively. Finally, Chapter 12 concerns the use and specification of tropical materials commonly used in construction. A list of findings, conclusions and recommendations follow:

- All tropically weathered RED RESIDUAL SOILS were included in this study. Some of these were true lateritic soils under the chemical criterion. Others are presently undergoing or have undergone laterization, but leaching of the silica and bases is incomplete. There is no practical means of distinguishing between the two.

  Further, the engineering properties of both groups are quite similar.
  Many lateritic soils contain an appreciable clay fraction which generally displays properties that are similar to those of other tropical soils which are only partially laterized.
- The more descriptive PEDOLOGICAL CLASSIFICATIONS are preferred, i.e. ferruginous soils, ferrallitic soil, ferrisol, ferralsols, acrisols, luvisols, cambisols and nitosols are recommended in lieu of lateritic soil.
- Variations in properties among groups in DIFFERENT CLASSIFICATION SYSTEMS should be established in each country. Possible variations within each group (among the subgroups) should also be investigated.
- CORRELATIONS OF PROPERTIES OF RED TROPICAL SOIL are established that can be used in identification of soil type and preliminary assessments of engineering properties. The following useful relationshipes were established:

  a) PI = 0.71 LL – 8.50 for ferruginous soils
  PI = 0.57 LL – 3.62 for ferrallitic soils
  PI = 0.50 LL – 1.50 for ferrisols
  PI = 0.58 LL – 4.54 for all tropical red soils – Africa
  PI = 0.45 LL – 3.50 for ferralsols
  PI = 0.74 LL – 3.80 for acrisols
  PI = 0.82 LL – 13.95 for arenosols
  PI = 0.75 LL – 12.70 for luvisols
  PI = 0.23 LL + 7.96 for nitosols and
  PI = 0.49 LL – 5.25 for all tropical red soils – South America.

  b) Figure 4.20 shows a general correlation between the AASHO classification system and California Bearing Ratio values for the tropical red soils.
- Special pretreatments and TEST PROCEDURES are required when soils over volcanic rocks are tested. Standard procedures in which the sample is predried can only be used if it is certain that the soil will be dried during construction. If drying is not feasible, the laboratory testing should be performed on the samples in the as-received condition. Otherwise, the laboratory results will apply only to the altered material and not to the material that is used in construction.
- The ONE-POINT LIQUID LIMIT TEST can be used for tropical soils.
  The following equation:
  $$LL = W \frac{(N)^{0.150}}{25}$$
  can be applied, where: W = moisture content at N blows, percent
  N = the number of blows.
- Each point on the COMPACTION CURVE should be determined with new material. This prevents excessive breakdown during compaction, which occurs when the same material is reused. The soil should be "cured" overnight at its compaction moisture content prior to compaction.
- A minimum of three points should be determined to establish the CBR accurately at the optimum moisture content or any other particular moisture content.
- MAXIMUM DEFLECTION VALUES for the practical range of standard axle applications are established as a result of an analysis of the performance and deflection characteristics of flex-

ible pavements in Africa and South America. Pavement evaluation are based on riding comfort which basically reflects the roughness of the pavement. Roughness, in turn, is generally attributed to pavement deformations particularly those occurring in the subgrade and reflected in the pavement surface. Therefore, the deflection criteria are established to limit the vertical strain in the subgrade to acceptable limits. The critical slope of the deflection basin for the practical range of standard axle applications that prevent excessive pavement cracking are also established from the analysis. Pavement cracking is caused by excessive tensile strain in the asphalt layer due to vehicle loading. Therefore, critical slope of the deflection basins are established which limit the tensile strain in the asphalt layer to acceptable limits.

- STRUCTURAL COEFFICIENTS are established based on the relationships between deflection and strength of the pavement component layers. The structural coefficients are based on the CBR of the unbound structural layer as well as the layers position in the pavement structure. A strength factor for asphalt layers are established based on the thickness of the asphalt layer and the strength of the soil layers beneath the asphalt. A relationship between the slope of the deflection basin, asphalt thickness, and the base course CBR is established. This relationship defines the minimum thickness of asphalt required to prevent excessive pavement cracking within a given traffic period.
- A FLEXIBLE PAVEMENT DESIGN PROCEDURE was developed from two basic relationships (1) between deflection and performance and (2) between deflection and pavement strength.
- DESIGN TABLES are presented which simplifies the design procedure. Sixteen Design Tables have been developed which provide various combinations of pavement thicknesses, material strengths and pavement surfaces. Three thicknesses of asphalt concrete surfacing as well as surface treatment are included in the Design Tables. Four base course thicknesses in combination with six base course sgrengths are included. The sub-base thickness varies depending upon the CBR of the subgrade. Four levels of sub-base strength are provided in the tables.
- An OVERLAY DESIGN PROCEDURE is presented which has been developed from calculated values using the asphalt strength increase factors established in the study.
- Recommendations are also made for the DESIGN OF UNPAVED ROADS. For unpaved roads the minimum thickness of surfacing and base is established based on the CBR of the subgrade.
- THE LOS ANGELES ABRASION AND AGGREGATE IMPACT TESTS do not distinguish between those materials that perform adequately and those that do not. The hardness index is not sufficiently discriminatory. The breakdown that occurs with compaction does not appear to be related to quality; the breakdown may actually improve the gradations in some materials.
- The SLAKE-DURABILITY TEST is tentatively recommended to evaluate durability. The test is relatively simple and the required equipment is not particularly expensive. A minimum slake-durability value of 97 appears to include only high quality material while values above 94 exclude the poor materials.
- The D_c AND D_f TEST procedures correlate well with the observed performance of the materials tested, as well as with numerous other lateritic and non-lateritic materials.
- REPETITIVE TRIAXIAL LOAD TESTS carried to a large number of load repetitions (N = 100,000 or more) hold some promise as a basis of classifying laterite aggregate as to quality. If E_R is determined at several values of θ the following preliminary classification is indicated.

  E_R above 21 ksi, material performance should be good.
  E_R range 19 to 21 ksi, material performance should be fair.
  E_R below 19 ksi, material performance should be poor.
- Sandy soils or those with high contents of iron will be most effectively STABILIZED WITH CEMENT. This would include most of the luvisols and arenosols of South America, the ferruginous soils and sandy ferrallitic soil of Africa and the comparable soils of Southeast Asia.
- The acrisols and ferralsols of South America and the ferrallitic soils and ferrisols of Africa and the comparable soils of Southeast Asia will usually be more effectively STABILIZED WITH LIME.
- The results of the ASPHALT STABILIZATION tests in Africa and Southeast Asia indicate that asphalt is an effective and practical stabilizer in relatively clean sands.
- The TROPICAL DESIGN PROCEDURE can be applied if the stabilized materials do not exceed reasonable limits. Therefore engineers can confidently continue the practice of using stabilization as a method of improving low strength soils and using them as an alternative structural material when high strength soils are not available.
- The properties of the African BLACK CLAYS are similar to those reported in the literature on black cotton soils and also to other expansive soils. The expansion pressure vary considerably, depending on the clay and on the test employed.
- Potential expansiveness of a BLACK CLAY SOIL can be estimated from its PI and clay content. The free swell test and PVC test are recommended as routine tests when investigating tropical black clays or other expansive tropical soils. Lime

stabilization is a practical means of increasing bearing capacity and reducing expansion pressure.

Moisture changes in BLACK CLAY SUBGRADES can be minimized by:

(1) Stripping the vegetation and allowing the soil to fallow for a year. Adding a granular cover is particularly effective in increasing the moisture content of the subgrade. (2) Compacting the subgrade at Standard AASHO density and optimum moisture content. (3) Providing adequate drainage facilities, and by locating drainage ditches well away from the shoulders.

It is feasible to use aerial photography as well as the imagery obtainable from other types of REMOTE SENSORS to classify the terrain. Map units can be developed from the imagery and correlated with the characteristics of soil profiles which are important to engineering projects, especially those related to transportation.

SPECIFICATIONS are presented for sub-base, base and surface gravels as well as compaction requirements, equipment and borrow pit excavation procedures.

SPECIFICATIONS FOR SURFACE TREATMENT PAVEMENT are adopted from a paper entitled "Bitumen Emulsions and Cut-Back for Surface Treatment and Soil Stabilization".

CHAPTER 1
ENGINEERING IMPLICATIONS OF TROPICAL WEATHERING *AND LATERIZATION*

The engineering behavior of soils, whether formed under arctic, temperate or tropical, conditions, is determined by certain physical characteristics designated as engineering properties. Once the engineer has ascertained which properties are the significant ones with respect to a particular type of construction at a particular site, he may at least in principle perform suitable tests to evaluate the properties quantitatively and may base his design on the findings. This fundamental procedure is at the root of all sound geotechnical design.

In practice, the apparently straightforward fundamental procedure can rarely be used without modification. For example, index properties which are indicative of the engineering characteristics required for design are used rather than determining actual engineering properties because indexes are simpler and cheaper to evaluate and because experiences gained with various soils classifications can be used when encountering similar conditions in new localities or on new projects.

It has long been appreciated that the engineering classification of soils is greatly facilitated by taking into account the soil-forming processes by which nature has created the various types of soil conditions. Similar combinations of soil-forming processes in different parts of the world have been found to lead to materials of similar index properties and similar engineering characteristics. Therefore, in any study such as the present one, the identification of localities in which the soil-forming processes have been similar is a prime objective.

The consequences of tropical weathering have been of overriding significance in the formation of the soils of equatorial regions. Hence, no system of classification or no attempt to identify the significant engineering characteristics can succeed if not based on an appreciation of tropical weathering processes. These processes have been the subject of a vast amount of research and are described in an impressive body of literature.

For the most part, however, the literature of tropical weathering is concerned with the chemistry, morphology, and mineralogy of the process. The complexity of the subject is reflected not only in the extent of the literature, but also in wide differences of opinion even over such basic questions as the definition of laterite. The civil engineer may easily find himself diverted from his more limited aims by the complexity of the literature. The detailed scientific knowledge is relevant to the civil engineer only in the same way as knowledge of the chemistry of cement is relevant to the development of the principles and practice of reinforced concrete design.

Those aspects of the phenomena of tropical weathering of primary significance to engineers are summarized in the remainder of this section. In succeeding sections scientific background is introduced where required in connection with specific engineering questions.

WEATHERING*

Rocks when exposed at or near the earth's surface find themselves in a physical and chemical environment often quite different from that in which they were originally formed. The minerals which constitute the rocks may react chemically with the rainwater, groundwater, and dissolved solids and gases of the new near-surface environment to form new minerals which are more nearly in equilibrium with the surface conditions. Some materials may be carried away by solution in groundwater. The end result of these changes is to convert the upper portion of the rock into a residual debris more soil-like than rock-like in character and with chemical, mineralogical, and physical properties entirely different from those of the original rock.

The changes which take place in the rock by these near-surface processes are encompassed by the geological term *weathering*. One form of weathering, denoted as *physical*, includes the effects of such mechanical processes as abrasion, expansion, and contraction. Physical weathering produces end-products consisting of angular blocks, cobbles, gravel, sand, silt, and even clay-sized rock flour. The mineral constituents of all these products are exactly like those of the original rock. *Chemical* weathering, on the other hand, results in the decomposition of the rock and the formation of new minerals. The near-surface agents of weathering which enter into chemical reactions with the primary minerals of the rock include water, oxygen, carbon dioxide, and organic acids derived from vegetation. The various processes include hydration, hydrolysis, oxidation, solution, and carbonation. All of these may operate simultaneously, some more rapidly than others and some more effectively in the alteration of one mineral than another.

The chemical changes operating in the primary minerals of the rocks in temperate or semitropical zones tend to produce end-products consisting of clay minerals predominantly represented by kaolinite and occasionally by halloysite and by hydrated or anhydrous oxides of iron and aluminum. Quartz experiences slight solution but remains essentially unchanged, although it may undergo some comminution.

Chemical weathering is favored by warm humid climates, by the presence of vegetation, and by gentle slopes. Thus, tropical and subtropical regions of low relief with abundant rainfall and high temperatures are the most susceptible to chemical alterations. Deep, strongly leached

* This subsection is taken from P. Jimenez-Quinones, "The Compaction Characteristics of Tropically Weathered Soils", Ph. D Thesis, Univ. of Illinois, 1963, and from lectures by Prof. Don U. Deere in his regular courses at the Univ. of Illionois and as condensed in guest lectures at Harvard University in 1968.

red, brown, and yellow profiles are manifestations of the effects of severe chemical weathering.

Under conditions favorable to tropical weathering, the weathering processes may be so intense and may continue so long that even the clay minerals, which are primarily hydrous aluminum silicates, are destroyed; in the continued weathering, the silica is leached and the remainder consists merely of aluminum oxide such as gibbsite, or of hydrous iron oxide such as limonite or goethite derived from the iron. This process is known as laterization.

The extent to which a residual soil has been laterized may be measured by the ratio of silica, SiO_2, remaining in the soil (except for discrete pebbles of free quartz that may remain) to the amount of Fe_2O_3 and Al_2O_3 that has accumulated. The silica: sesquioxide ratio

$$\frac{SiO_2}{R_2O_3} = \frac{SiO_2}{Fe_2O_3 + Al_2O_3}$$

has served as a basis for classification of residual soils. Ratios less than 1.33 have sometimes been considered indicative of true laterites, those between 1.33 and 2.00 of lateritic soils, and those greater than 2.00 of nonlateritic tropically weathered soils.

Determination of even the relatively simple silica: sesquioxide ratio requires a chemical analysis not likely to be available to the civil engineer. Furthermore, experience has indicated that the ratio is by no means an unique indicator of the index properties or engineering characteristics required by the engineer. Many factors are of greater importance.

PROFILES OF WEATHERING

The alteration of rock by the processes of chemical weathering takes place progressively through a series of events and stages which result in a profile of weathering. Those who have worked with tropically weathered residual soils have noted the frequent occurrence of an upper clayey zone a few feet thick, underlain by a silty or sandy zone which, in turn, passes through a very irregular transition into weathered and finally into sound rock. The thickness of each member of the profile varies greatly from site to site because of the complexity of the interrelationships among the controlling soil-forming factors of rainfall, temperature, time, character of parent rock, topography and vegetation, Moreover, at a given site there may be great differences in the depth and thickness of the weathered rock within lateral distances of only a few feet. The differences arise because of differences in lithology, such as the presence of a dike or a contact, or because of differences in the degree of jointing or extent of shearing. The latter features increase the permeability and the depth to which weathering can proceed.

Similar weathering profiles develop under similar conditions as reflected by the soil-forming factors mentioned above. Hence, a convenient means for regional description of soil properties for engineering purposes is the determination of the engineering characteristics of the various members or zones in typical profiles of weathering. Such investigations constitute a framework by means of which information and further experience can be coordinated.

HYDRATED AND DEHYDRATED LATERITE

In continuously wet regions the end-products of the process of laterization are likely to be characterized by high natural water contents, high liquid limits, and irreversible changes upon drying. Inasmuch as limonite and geothite are hydrated oxides of iron, and gibbsite is a hydrated oxide of aluminum, drying alters the characteristics of the material markedly. The plasticity decreases and the grain-size increases such that much of the clay-sized material agglomerates to the size of silt. When dealing with such materials, the engineer may need to determine the index properties and the engineering properties on the basis of tests performed at the natural water content and again at various degrees of air drying in order to evaluate fully the range of properties associated with the physical conditions that may prevail on the job. Serious engineering and construction problems have arisen through failure to recognize materials likely to experience irreversible changes as a consequence of desiccation.

On the other hand, in regions subject to distinct wet and dry seasons, the products of laterization are likely to be characterized by low natural water contents, low plasticity, and the presence of concretions and cemented horizons. The alternate upward and downward movement of water causes concentration of the iron or of the alumina. Laboratory tests run from the natural water content or from the air-dried state lead to essentially the same results.

The difference in behavior of the hydrated and dehydrated forms of lateritic soils deserves attention because of the danger of being misled by reports in the literature in which the significance of the distinction may not be realized.

CONCLUSION

Progress in dealing with tropically weathered soils will depend primarily on the selection of appropriate physical properties indicative of engineering behavior and the establishment of suitable tests for their investigation. These tests may be expected to differ in some respects from those that have been found suitable in temperate regions. The results of such investigations can be systematized by the identification and physical testing of the various materials in typical profiles of weathering. The profiles may be grouped and classified, at least as a first approximation, on a regional and local basis according to the similarities or differences among the important soil-forming factors.

The setting of geotechnical problems in Equatorial regions is described in Chapter 2 as a background for all subsequent discussion. Chapter 3 is concerned with those aspects of soil genesis needed to establish and make use of procedures for a general classification of soils and profiles of weathering in various parts of the tropics. This information is followed by a summary of what is currently known about the engineering characteristics of the products of

laterization and of other deeply weathered tropical soils as determined in the present investigations as well as in the preceding investigations in Africa and Southeast Asia.

The foregoing discussion indicates that all the products of tropical weathering with red or reddish color have much in common, with respect to both origin and properties. It is also evident that an almost unlimited variety of such materials, having at least slightly different chemistry, morphology, and physical properties, can be found. The differences, often subtle, have led to much controversy over terminology and definition. To avoid such controversy, entirely sterile from the engineering point of view, it would be preferable to refer to all such materials simply as *reddish tropically weathered* materials, irrespective of the details of the degree of their weathering, and irrespective of the presence or absence of ferruginous crusts or concretions. Even transported soils cemented by ferruginous matter would be included. Nevertheless, because of the widespread use of the term laterite, in this report all such reddish tropically weathered materials are for convenience referred to as *lateritic soils*. This usage, therefore, carries no implication with respect to the definition or properties of laterite as such. The distinctions useful for engineering purposes are based on differences in physical or engineering properties.

CHAPTER 2
REGIONAL SETTINGS

INTRODUCTION

There are a number of regional features which significantly affect the soil development process. Among these features physiography, geology, climate and vegetation are the most important in the development of tropical soils.

A basic knowledge of a region's geology provides a background for determining the probable type of parent materials which would be encountered and the probable duration of the weathering process. A knowledge of the physiography, namely the topographic description of the region, when coupled with a knowledge of the climate, namely the amount and seasonal distribution of rainfall lead to a fundamental understanding of tropical weathering processes. Finally, a knowledge of the region's vegetation may be important to the understanding of the chemical weathering process.

The climate of a region is perhaps the most important element in weathering and the soil formation process. A knowledge of the geology is an important factor during the early stages of soil development and identification of the physiography is important in understanding the stage of topographic development and drainage conditions. Vegetation is of lesser importance since it largely reflects present-day climatic conditions.

A brief description of these regional factors in South America, Central America, Africa and Southeast Asia is provided below as background information for succeeding discussions.

SUMMARY OF THE GEOLOGY AND PHYSICAL GEOGRAPHY

South America

South America is the fourth largest continent, after Eurasia, Africa and North America. It covers approximately 18,130,000 square kilometers (7,000,000 square miles) of land mass between latitudes 10° north and 60° south. The widest portions of the continent lie in the equatorial regions. The continent tapers southward dramatically from the tropical zone to Tierra Del Fuego at the southern limit of the continent. See Figure 2.1.

Tropical South America can be divided into four major geotectonic regions (FAO, 1971): the Andean cordilleras along the western coast and northern coast of Venezuela, extending into Trinidad; the Brazilian shield or highland, along the east coast, south of the Amazon basin; the Guyana shield or highlands, extending north from the Amazon basin into northern Brazil, southern Venezuela, Guyana and Surinam; and the Subandean depression which lies east of the Andes and includes the Orinoco basin of Venezuela, the Amazon basin and Chaco-Pampa basin of Paraguay and Argentina. The important physiographic features are shown on Figure 2.1

The Andean Cordilleras

The Andes are one of the great mountain systems of the world. Only the Himalayas are higher. The system is made up of more or less parallel ranges extending over 7,250 kilometers (4,500 miles) along the entire western coast of the continent as well as northern Venezuela and Trinidad. The system is at its widest in Bolivia and southern Peru where the ranges diverge around the altiplano basin. This basin has been elevated to a plateau with elevations of over 3,050 m (10,000 ft). It has no external drainage and the rivers drain to Lake Titicaca and Lake Poopo, which is saline; large salt flats are found in the southern altiplano. The system is at its narrowest in Ecuador where the two ranges have been compressed into a width of only 100 km (60 miles). Within Ecuador each cordillera has over 15 peaks that exceed 4500 m (14,760 ft). Generally, there are two cordilleras, an eastern, named Cordillera Oriental and a western, named Cordillera Occidental. In Peru and again in Colombia, there are three; the third is called Cordillera Central. These diverge in Colombia; Cordillera Occidental dies out in northwestern Colombia but is believed to be related to the mountains in Panama. Cordillera Central also dies out in northern Colombia. Cordillera Oriental turns northeastward through Venezuela and into Trinidad.

The Food and Agricultural Organization (FAO, 1971) has described the lithologies of various sections of the Andes which will be briefly discussed here. Further details have been added from the Handbook of South American Geology (Jenks, 1956). The Caribbean coastal ranges are made up of Mesozoic sedimentary and metamorphic rocks. Tertiary continental shales and sandstones also occur. The Venezuelan Andes contain some Paleozoic and younger sediments with Tertiary sediments predominating. The Cordillera Oriental of Colombia is the mountain system which lies east of the Magdalena river. It is made up mostly of sedimentary rocks, with no volcanics. Cretaceous marine sediments are the most important. The Cordillera Central of Colombia and Oriental of Ecuador are composed principally of metamorphic and igneous rocks of Paleozoic and Mesozoic ages and important Tertiary and Quaternary volcanics. The western cordilleras of Colombia and Ecuador have a core of granodiorite covered with volcanics and Jurassic and Creataceous sediments. The eastern cordillera of Peru, Bolivia and Argentina are composed of various sedimentary rocks of Precambrian to Tertiary age, some of which have suffered local metamorphism. The young volcanic Andes of western Peru and Chile and the Cordillera Principal of Argentina and Chile have thick covers of volcanic rocks of various types from flows to ejecta. The composition varies from rhyolite to basalt with perhaps andesite and trachyte predominating. The maximum width of 170 kilometers (105 miles) occurs in Chile. The altiplano basin is filled with Tertiary and Quaternary continental sediments and pyroclastics, with the volcanics predominant in the upper section. The western cordillera of Peru and

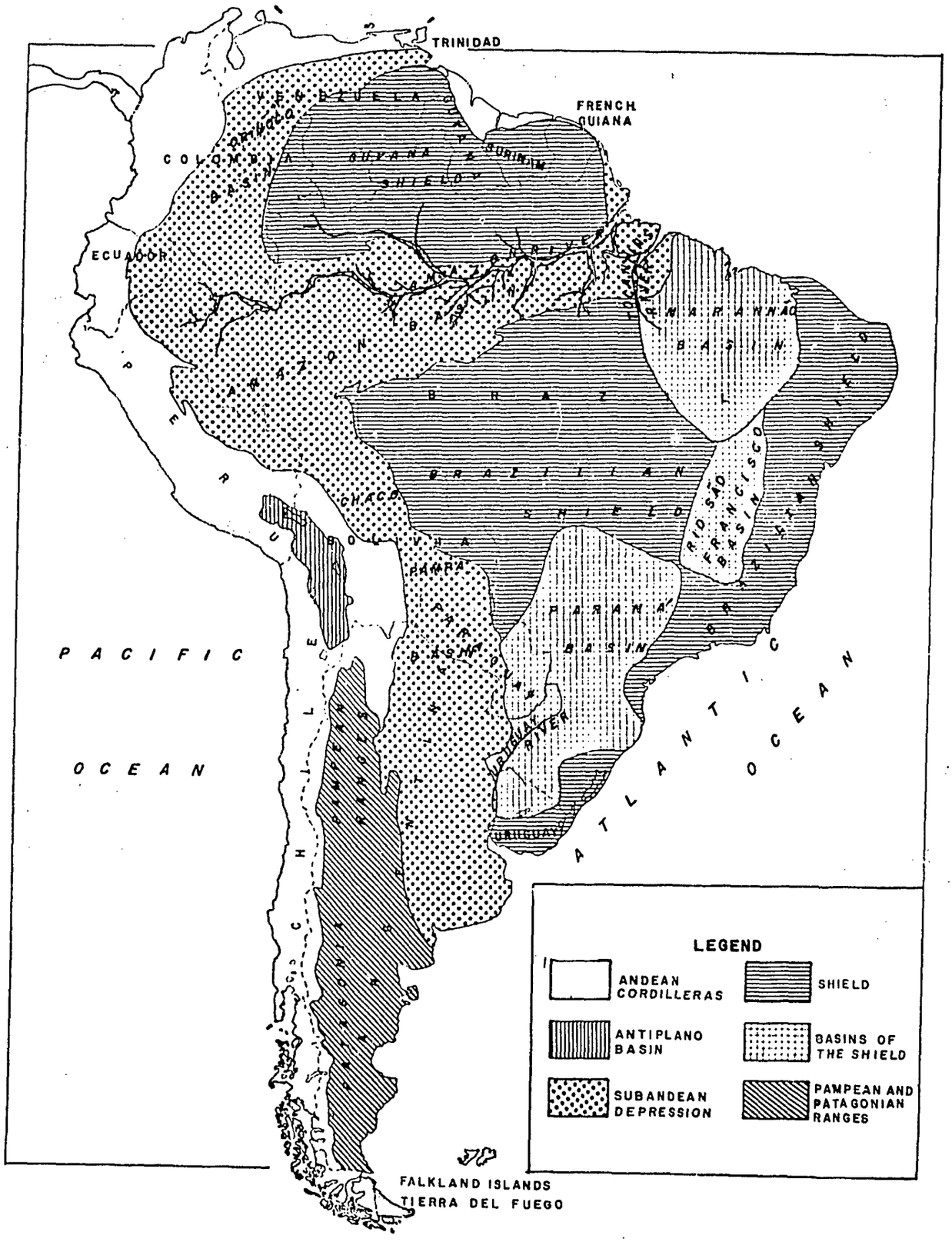

FIGURE 2.1 – MAJOR PHYSIOGRAPHIC FEATURES OF SOUTH AMERICA

Chile is composed of a large elongated batholith. It forms the western edge of the Andes from northern Peru to Tierra del Fuego. Granite, granodiorite, quartz diorite and diorite are the predominant rock types. The Pampa del Tamarugal in northern Chile is of interest. It is a down-faulted depression between the western Córdillera and Cordillera Principal which has a cover of Tertiary and Quaternary sediments and volcanics. Various surficial deposits of chloride, nitrate and sulfate salts are extensive. The Central Valley of Chile is also of interest. It, too is a down faulted block covered with reworked volcanic material. The hummocky sufaces indicate slides and mud flows are common. Glacial moraines are found in the southern part of the valley. The remaining Andes in Argentina are made up of sedimentary rocks of various ages, principally Mesozoic.

The FAO (1971) Soil Map of the World shows that tropical red soils have developed throughout the Andes. The Caribbean coastal range, the intermontane valleys of Colombia, northern Colombia and the western coastal regions have all types of tropical red soils. Some types have also developed in the mountainous regions of Ecuador, Bolivia, Peru and as far south as northern Chile.

The early Andean mountains began forming in late Cretaceous with further uplift and folding continuing into Tertiary time. The modern Andes were formed more recently in Pleistocene time. Great uplifts, up to 4,300 m (14,100 ft) and tight folding were characteristic. Elevated shorelines and stream deposits indicate perhaps two or three pauses.

THE GUYANA AND BRAZILIAN SHIELDS

The Guyana and Brazilian shields constitute the ancient Precambrian continental core, or craton. The Amazon basin separates the two. The Brazilian shield is by far the largest, extending from the Amazon basin to the Rio de La Plata in the south and from the Atlantic to the Rio Paraguay and central Bolivia where it disappears beneath the lowland sediments. Most of the area is an elevated plateau with many separate unconnected mountain ranges. The Guyana shield extends from the Amazon basin through northern Brazil, southern Surinam and Guyana into Venezuela where the Precambrian passes under the sediments of the Orinoco Basin. This shield is also an elevated plateau with several mountain ranges, including the Pacaraima mountains containing Mount Roraima whose peak (2,770 m; 9,090 ft) forms the common point for Brazil, Guyana and Venezuela.

Both shields have a series of more or less level areas which represent erosional surfaces. Four such surfaces have been recognized in Brazil, separated from one another by steep scarps. Four have also been recognized in Guyana. A fifth surface is believed to be beginning in Brazil. Most of the soils developed on these surfaces are tropical red soils.

The Precambrian rocks of the Brazilian shield are exposed all along the east coast, from northern Brazil to Argentina and in the west and central parts of the shield. Sedimentary basins separate the two areas. The Paraná basin covers the shield in most of southern Brazil. It is a huge basin in which various Paleozoic and Mesozoic sediments have been largely covered by basaltic or andesitic lava flows. As many as 14 separate flows have been recognized. One drill hole in the state of São Paulo penetrated 1,400 m (4,590 ft) of volcanic rocks. The Maranhão basin covers the shield in most of the states of Ceará and Piauí, extending as far west as the Rio Tocantins in the state of Pará. This basin is also filled with Paleozoic and Mesozoic sediments, principally limestone, sandstones and shales. In between the Paraná and Maranhão basins lies the Rio São Francisco basin. This basin also contains Paleozoic and Mesozoic sediments. The most widely occurring are limestones, but sandstones and shales are found as well.

The Precambrian has been broken down into three major divisions in Brazil (Jenks, 1956): Early, Middle and Late. The oldest is Early Precambrian or the Mantiqueira series of strongly metamorphosed banded gneisses. The Middle Precambrian, or pre-Minas series is an assemblage of metasedimentary rocks. Between Barbacena and Lafaiete in Minas Gerais, the series is composed of quartz mica schists and gneisses. North of Lafaiete, a wide variety of rock types occur. Among these are conglomerates; quartzites; dolomites; several iron formations; phyllites, some graphitic; and metavolcanics. These extend through the Quadrilátero Ferrífero (a name given to an area where iron deposits are surrounded by mountain ranges forming a quadrangle).

The Late Precambrian is further divided into two series. The Minas series is widespread and is composed of quartzites, slates, iron formations, dolomites, phyllites and conglomerates. The Itacolomi series of eastern Brazil is made up of quartzite, phyllite and less commonly conglomerate. There are many intrusive bodies of various ages but the most common are granite and granodiorite.

The Pacaraima mountains dominate the central part of the Guyana shield. These are largely composed of sandstone, quartzites, shales and conglomerates of the Roraima formation that have been elevated. These have been intruded by basic sills hundreds of feet thick. Two erosional surfaces exist, covering large areas, which are capped with a crust of laterite. According to Bleackley (1964), these elevations are at 395 - 490 m (1,300 - 1,600 ft) and 670 - 760 m (2,200 - 2,500 ft).

The Intermediate Peneplain lies between the mountains and the coastal plain. This peneplain is composed of granites, gneisses, schists and metamorphosed sedimentary rocks and volcanic rocks. Two levels of erosional surfaces, also largely capped with laterite, have been recognized. These lie at 107 - 137 m (350 - 450 ft) and at 305 m (1,000 ft). The lateritic crusts somewhat resemble karst topography in carbonate rocks. The permeable crusts allow rainfall to drain freely causing underlying drainage systems to form which later collapse.

A relatively shallow coastal plain extends along the Atlantic coast boundary of the Guyana shield. The sediments are Tertiary to Recent in age and are largely sands.

The rocks exposed in the Venezuela part of the shield are similar to those in Guyana. The oldest rocks are granite gneiss, hornblende gneiss and mica schist. These have been intruded by granites, pegmatites, gabbros and other basic rocks. These are overlain by the Imataca series of ferruginous quartzites. The uppermost series is the Roraima series which has the same composition as in Guyana.

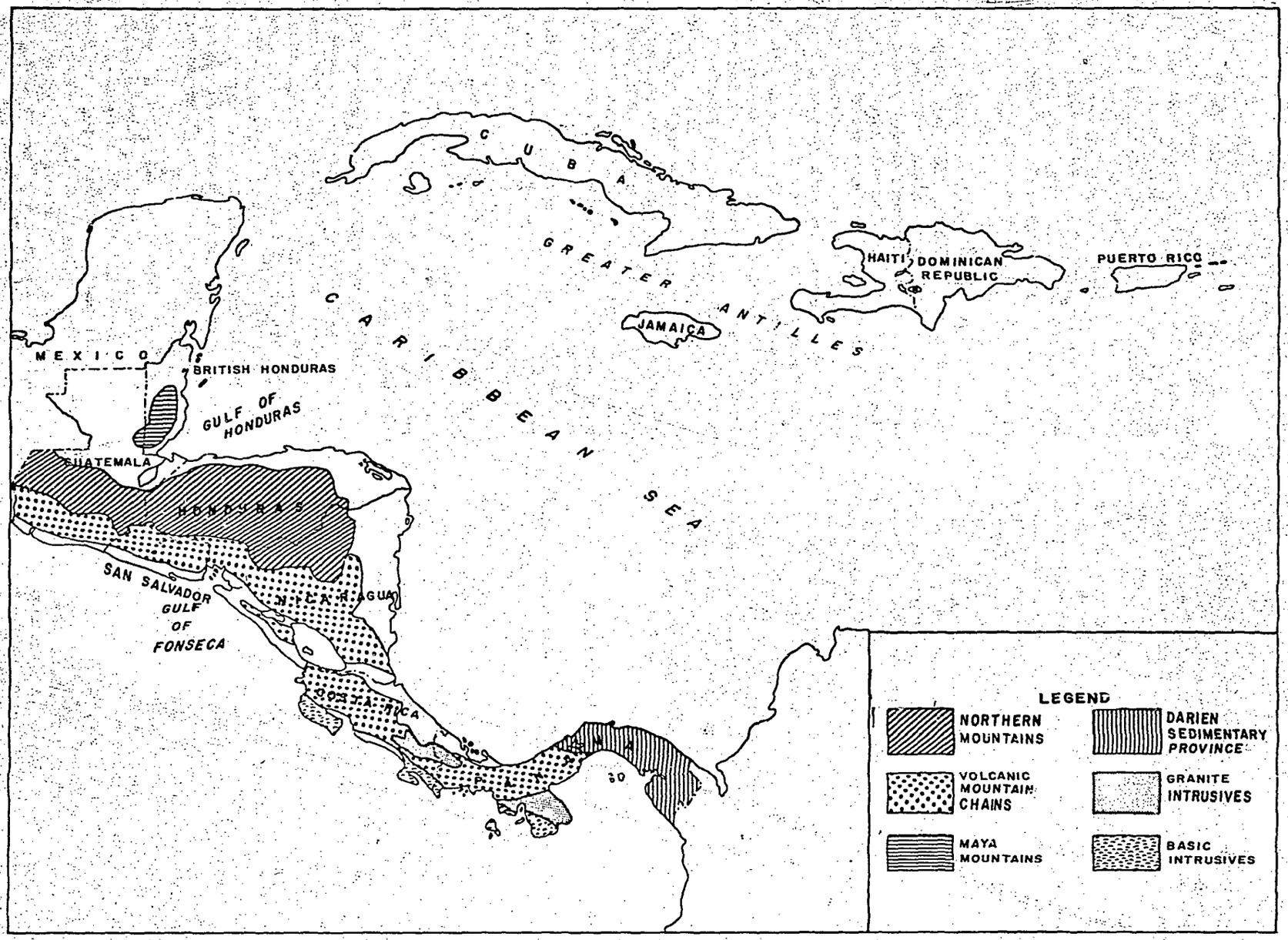

FIGURE 2.2 – MAJOR PHYSIOGRAPHIC FEATURES OF CENTRAL AMERICA

THE SUB-ANDEAN DEPRESSION

The various basins of the Sub-Andean depression will be discussed individually. The Orinoco basin is filled with thick deposits of Tertiary and Quaternary sediments. The Quaternary formation covers a large part of the basin in the Venezuela and Colombia llanos (plains). The sediments are mostly fluvial and gravel terraces are found in the valleys and in the piedmont. The Amazon basin has been a geosyncline since early Paleozoic time. Over 4,000 m (13,100 ft) of sediment has accumulated. The oldest rocks which now outcrop in narrow strips along the northern and southern flanks, are metamorphosed sandstones. Most of the Amazon basin has a cover of Tertiary sediments which are largely made up of fines. Coarser sediments are found in the western part of the basin along with volcanic ejecta from the Andean eruptions. Quaternary deposits include clays and silts. Fanglomerates have formed in the piedmont region, composed of various volcanic material, clays, sands and gravels. Recent deposits are almost entirely fine-grained and occupy a relatively narrow band along the existing Amazon river. The Chaco-Pampa basin of Bolivia, Paraguay and Argentina contains a thick sequence of Paleozoic to Recent sediments of marine and continental origin.

Central America

Central America is usually considered to be the five republics of Costa Rica, El Salvador, Guatemala, Honduras and Nicaragua, as well as British Honduras and the isthmus of Panama. The major features are shown in Figure 2.2.

THE NORTHERN MOUNTAINS

Two mountain systems make up the spine of Central America. The first is found in Guatemala, British Honduras, Honduras and extending into northern Nicaragua. This system has a general east-west orientation, counter to most North and South American mountain-belt orientations, and has a similar geological composition as the Caribbean islands to the east. The System has a core of Precambrian or Paleozoic rocks covered by more recent volcanic materials. The second system is a series of two Pliocene to Recent volcanic mountain chains. The first chain extends in several gentle arcs about 560 kilometers (350 miles) from southern Mexico southeastward through Guatemala and El Salvador to the Gulf of Fonseca in Honduras. The second chain extends from the Gulf of Fonseca southeastward also in several gentle arcs through Nicaragua and Costa Rica into Panama. Together they contain over 100 large volcanoes and another 150 or so minor ones.

These volcanic arcs are highly active seismic zones. Earthquakes are caused both by faulting and by volcanic eruptions. Antigua, Guatemala was destroyed by an earthquake in 1773. Guatemala City was severely damaged in 1917-1918. Managua, Nicaragua was heavily damaged in 1931 and again in December, 1972.

Considerable recent attention (Dillon et al., 1973; Pinet, 1971, 1972) has been centered on the Gulf of Honduras. In the new geological concept of plate tectonics, the North American plate has been faulted away from the Caribbean plate along the east-west line passing through Guatemala and the Gulf of Honduras. It has been postulated (Dillon et al. 1973) that a sphenochasm, or triangular opening, has resulted as first the Yucatan peninsula separated from Honduras by rotation and second by transcurrent faulting along what is now called the Cayman fault zone. The limestone plain of northern Guatemala lies north of the fault zone, or within the North American plate. South of this, the east-west trending mountains of southern Guatemala to Honduras are within the Caribbean plate.

The northern mountains are the oldest in Central America. They are often called the Northern Highlands and are composed of igneous and metamorphic rocks which have been highly folded and faulted. These ranges are believed to be the western extension of the Greater Antilles orographic system. The Maya mountains of British Honduras are considered an outlier, perhaps separated by the sphenochasm postulated above. Tertiary volcanic debris covers most of these older rocks in Guatemala, Honduras and Nicaragua. In composition the volcanics are mainly rhyolites and andesites, but basalts have also been reported.

The second system of mountains, the Pleistocene to Recent volcanoes, occupy a thin belt along the Pacific coast. A number are still active. All lie within 80 kilometers (50 miles) of the coast. Many are strikingly symmetrical composite cones.

Between the Northwestern Highlands and the Southeastern Highlands of Costa Rica and Panama is a wide structural depression. This depression extends from northwestern Nicaragua through Lake Managua, Lake Nicaragua and along the San Juan River valley to the Caribbean. This has been considered a possible canal route across the isthmus.

THE SOUTHEASTERN HIGHLANDS

The Southeastern Highlands of Costa Rica and Panama are considerably different in geological detail. Some consider the isthmus a continuation of the Colombian Andes. The coastal chain of recent volcanoes continues southeastward from Nicaragua into central Costa Rica, where the Cordillera Central turns eastward near San José. A southern range, Cordillera de Talamance, is a granite intrusive block. Volcanoes reappear in western Panama but intrusives occur as well and predominate in eastern Panama as well as along the Pacific coast. The peninsulas along the Pacific coast in Panama and Costa Rica are composed of a series of granite hills.

The land bridge between South America and Central America was closed in late Tertiary time as a result of the accumulation of volcanic material along a submerged arch. Uplift came with igneous, mainly granite, intrusions.

Africa

GENERAL PHYSIOGRAPHY

Africa is the second largest continent, after Eurasia. It has an area of 30,420,000 square kilometers (11,700,000 square miles). It is almost everywhere a plateau and has

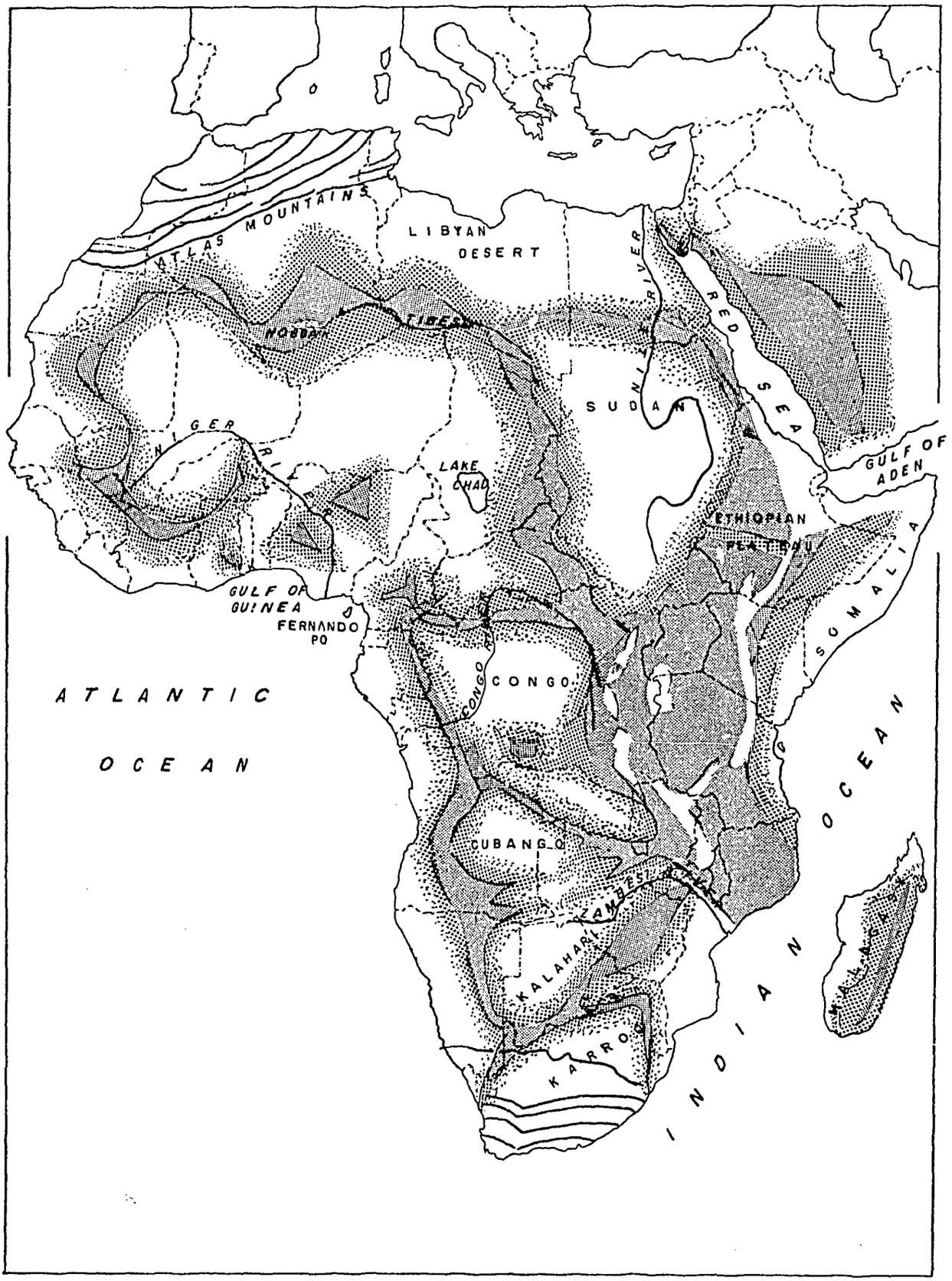

FIGURE 2.3 – A GENERALIZED MAP OF THE TECTONIC BASINS OF AFRICA WITH INTERVENING SWELLS, PLATEAUS AND RIFT VALLEYS (after Holmes, 1965)

been described by D'Hoore (1904, p. 24) as "a huge crystalline mass surrounded by a sedimentary fringe."

The major part of the continent consists of broad swells or plateaus separated by wide, elevated basins, Figure 2.3. Precambrian rocks outcrop in about one third of the continental area, on the plateaus and swells. Except for the Karroo, all basins are both tectonic and topographic features. They are filled with thick deposits of continental sediments derived from the surrounding highlands.

The surface of the African continent is, on the whole, unusually level. Mountains of folded sedimentary rocks occur in the extreme north where the Atlas mountains form part of the Alpine Mountain system, and in the extreme south in the Cape Ranges. Within the tropical zone, there are only a few mountainous areas. These are the Guinea Highlands; Togo Mountains; Cameroon Highlands; Marra Mountains of western Sudan; the Ethiopia and Somalia Plateaus; and other plateaus and volcanic mountains associated with the Rift Valley.

The continental plateau slopes from elevations of over 1,200 m (3,940 ft) in the southeast to 300 m (980 ft) in the Sahara. A number of erosional surfaces have resulted from repeated uplifting of the continental block. Mid-Tertiary surfaces are supposedly the most extensive; they are characterized by many ferruginous crusts (D'Hoore, 1964).

The Rift Valley system of eastern Africa, Figure 2.4, is the most important structural and physiographic feature of tropical Africa. It extends circuitously, with several branches, from Mozambique through Ethiopia to the Red Sea. Holmes (1965) states that the total length of the Rift Valley to the Red Sea exceeds 2,900 kilometers (1,800 miles). The total distance is over 4,800 kilometers (2,980 miles) if the continuation through the Red Sea, Dead Sea and Lake Tiberias is included.

One-third of the continent has no drainage to the oceans or Mediterranean Sea. Most of the remainder is drained by a few major rivers: the Nile, the only perennial stream which flows into the Mediterranean from Africa; the Senegal, Niger and Volta Rivers in West Africa; the Congo of Central Africa; and the Zambesi of southeast Africa.

GEOLOGY

The geology of tropical Africa is, generally speaking, as unvaried as the physiography. The major part of the continent has been emergent and relatively stable since Precamorian.

The rock types that occur are a result of the stable nature of the continent. The large expanse of Precambrian rocks indicates a long history of erosion and denudation. Continental-type deposits are generally limited to the large tectonic basins. The limited extent of marine deposits is evidence that only small areas were ever submerged, and most of these areas, with the exception of the Sahara, were limited to the continental margins.

Geological maps depicting the stratigraphy of Africa show a wide expanse of Precambrian, extending from Capetown to the northern 28 th parallel, well within the Sahara Desert, and from Senegal to Somalia. Over half the island of Malagasay is Precambrian. Continental sediments occupy the basins shown in Figure 2.3.

Paleozoic sediments are widespread in northern Africa, but some of the various formations extend south into western Africa. These are mainly sandstone, shales and limestones. These deposits cover over half of Ghana and parts of Togo and Dahomey; large areas of Mauritania, Mali, Senegal and Guinea are also covered. The only other significant remnants of early Paleozoic formations are limited to South Africa.

A deep succession of continental deposits were laid down during Permian and Triassic times which is called the Karroo Systems after the type locality in South Africa. These deposits are found throughout the southern part of the continent, in Malagasay and in tectonic troughs of eastern Africa (Furon, 1963). The lower beds are alluvial in origin, followed by clayey shales and topped with sandstones and shales (D'Hoore, 1964).

The Triassic culminated with volcanic activity. Basalt flows occurred in northern Africa and to a much larger extent in southern Africa. Basic intrusive bodies of this age are found in western Africa, in Senegal, Sierra Leone, Liberia and Mali.

Jurassic deposits made up of both continental and marine sediments are widespread. Limestone, sandstone and gypsiferous clays were laid down when seas invaded the Sahara from the north and from the Gulf of Guinea and Nigeria. Marine deposits also occur in Tanzania, Somalia and Ethiopia.

Tertiary deposits of various sediments were laid down along much of the coast of western Africa and parts of eastern Africa. Remnants of Tertiary surfaces exist today in southern West Africa and in central Africa which are capped with ferruginous crusts.

Important volcanic activity commenced in the Cretaceous in western Africa and in the Tertiary in eastern Africa. Flood basalts were extruded in Ethiopia and eastern Africa as far south as Malawi. These flood basalts were associated with the uplift in the eastern part of the continent that preceded the down-faulting of the extensive Rift Valley system. Cretaceous and Tertiary basalts were extruded in the Cameroons, Nigeria, Tibesti (northern Chad) and the islands in the Gulf Guinea.

The Quaternary deposits are varied and widespread. They have been summarized by D'Hoore (1964) into four groups: (1) eolian sand deposits found in the Sahara and southwest African deserts and the Kalahari extending north into the Congo basin; (2) alluvial deposits of various ages, some of which have been altered by wind surface erosion; (3) colluvial mantles, often thick enough to have complete profile development and often including talus and debris from ferruginous crusts; and (4) volcanic rocks, usually basaltic, a continuation of the volcanic activity that began in the Tertiary, associated with the Rift Valley system, the chain of volcanic mountains in Equatorial Guinea and Cameroon, and in smaller, isolated areas of central Nigeria, Tibesti, Hoggar (southern Algeria) and central Libya.

The eastern section of the continent has been subjected to a long period of volcanism, uplifting and faulting, which has culminated in the high plateaus, the African Rift system and its associated volcanic rocks and mountains. The oldest fault scraps are of Karroo age (Holmes, 1965), but the most important movements were in Tertiary and Quaternary time. Eruptions of the basalts started in early Tertiary,

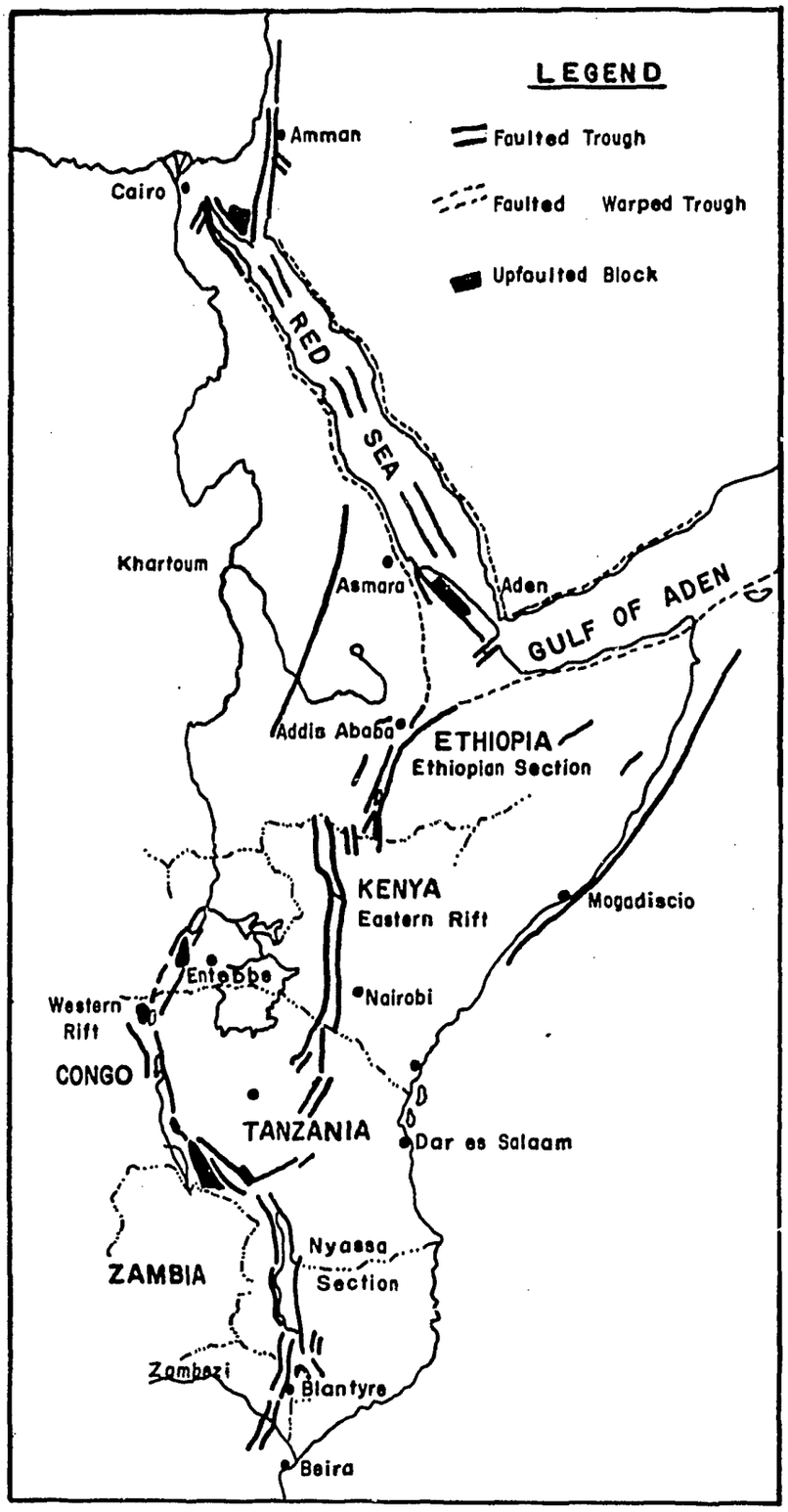

FIGURE 2.4 – RIFT VALLEY SYSTEM IN AFRICA (after Mohr, 1962)

prior to uplifting of the plateaus. Renewed volcanic activity coincided with further uplifting in mid-Tertiary. Major faulting also occurred in the Rift system at this time. The present topographic form results from additional uplifting and faulting in Quaternary time, in mid-Pleistocene. The last phase is still in progress.

Furon (1963) recognizes a Rift Valley type trough in Cameroon extending northeastward from the Gulf of Guinea towards Chad. He also recognizes a tectonic line, made up of a number of parallel faults that lie within the Cameroonian trough. The volcanic mountains of Mount Cameroon, Mount Manengouba and Mount Bambouto erupted along this line, as did the volcanic islands of Fernando Po, Principe, São Thomé and Annobon. Furon believes the line extends to the northeast, passing into the Chad Basin toward the Tibesti region in northern Chad. Volcanic activity also began in the Cretaceous at Mount Cameroon. The mountain, which is over 3,970 m (13,000 ft) in elevation, is still active. The latest eruption occurred in February, 1959. All of the volcanic mountains, including the volcanic islands are composed mainly of basalts.

THE PLATEAUS AND BASINS

The plateaus and basins of Africa have been subjected to repeated uplift and erosion through geological time. These are described in detail by Holmes (1965). The exposed rocks of the swells and plateaus are Precambrian which were formed at considerably lower elevations. Chad basin, containing thick deposits of continental sediments, is an arid region without external drainage. Despite this, Lake Chad is not saline, although evaporation in high; it drains underground to oasis about 720 kilometers (450 miles) northeastward in the southern Tibesti region, perhaps along Furon's tectonic line. The Kalahari and Cubango basins in the southern hemisphere are other examples of internal drainage; both drain into brackish swamps.

The Congo basin includes thick continental sediments of Karroo age overlain by sandy soils. These sediments dip inward from the surrounding swells and plateaus, as a result of Tertiary uplift and warping. The Congo river has incised the western rim of this basin at Stanley Pool, where it flows through a series of cataracts to Matadi, the port for ocean vessels on the Congo river.

The Sudan basin is also of interest. The large areas of sediments in this basin are largely lacustrine. The lake level rose until it breached the northern rim, along the present course of the Nile, forming the series of cataracts between Khartoum and Aswan. The southern section of this basin, where the White Nile flows northward, contains a large swampland of floating vegetation known as the Sudd.

Other basins are the Somalia, with thick sediments of Jurassic, Tertiary and Quaternary age; the Karroo basin of South Africa, now elevated as part of the southern plateau; and the El-Juf or Taoudene basin of West Africa. These are shown on Figure 2.3.

The swells and uplands adjacent to the basins include many areas with ferruginous crusts. These are usually on Tertiary surfaces or colluvium from Tertiary surfaces. D'Hoore (1964 p. 26) states:

> "If it is admitted that the greater part of the outstanding features of present day African geomorphology, such as alpine folding of the north, the great faults, and the marked subsidence of the continental depressions date backward toward the end of the Tertiary, one must believe that during the Tertiary the African relief was even more level than now. This very unusual topography diminished the possibilities of external drainage and engendered soil formation where temporary or permanent water-logging played a predominant role, so long as the climate was not too arid. It is in such conditions that the phenomena of mobilization, precipitation and accumulation of free iron are most intensive".

Southeast Asia

The geographical area usually considered to comprise Southeast Asia consists of the Indochinese and Malay peninsulas and the Malay archipelago. The countries of mainland Asia which are included are Burma, Thailand, Maylasia, Singapore (although actually an island off the Malay peninsula), Laos, Cambodia, North Vietnam and South Vietnam. The Malay archipelago, the largest of all island groups in the world, is made up of the Greater Sunda islands (Sumatra, Borneo, Java and the Celebes); the Lesser Sunda Islands (Bali, Lombok, Sumbawa, Sumba, Flores, Timor, etc.); the Moluccas, or Spice islands, which lie between the Celebes and New Guinea; New Guinea; and the Philippines. New Guinea is usually not included in Southeast Asia and therefore is not discussed in this section. The main features of this region are shown in Figure 2.5.

INDOCHINA

The Indochinese peninsula which now includes the countries of Cambodia, Laos, North Vietnam and South Vietnam has a rough S-shape. The Cardamom Mountains occur in the extreme southwest, bordering the Gulf of Siam. In the East, bordering on the Gulf of Tonkin lie the Annamite Mountains. These are mainly igneous rocks with basalts covering several plateaus. The area between the mountains is largely lowland, including the wide flood plains of the Mekong River. The Mekong lowland is thickly wooded and is largely swampland. The Mekong Delta begins just south of Phnom Penh where the river divides into two branches. The Tonkin Highlands abut China to the north. These are made up principally of sandstone and limestone. The Red River, which flows through the Tonkin Highlands originates in China and empties into the Gulf of Tonkin at Haiphong.

THAILAND

Thailand is composed of five geographic regions. Northeast Thailand is often called a plateau but is actually an undulating plain less than 300 m (980 ft) in elevation. The hills in the southeast are extensions of the Cardamom Mountains in Cambodia. The north is highland country, bordering Laos and Burma. The western border with Burma

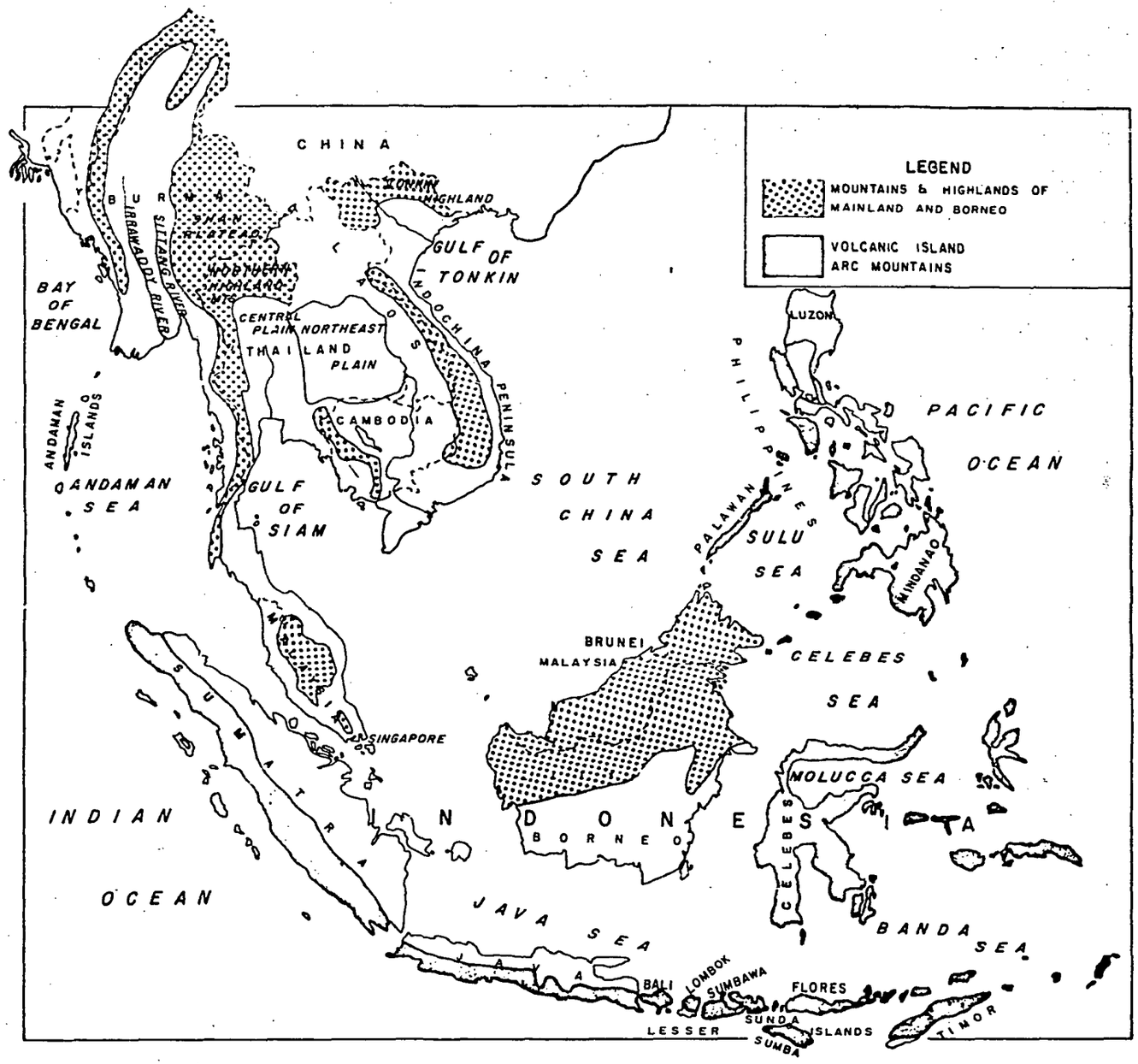

FIGURE 2.5 – MAJOR PHYSIOGRAPHIC FEATURES OF SOUTHEAST ASIA

lies along the divide of the Tenasserim Mountains which continue south forming the backbone of the Malay peninsula. The central plain is the north-south central valley in which the Chao Phraya is the principal river with a number of tributaries from the north and west. Southern Thailand is that part extending along the Malay peninsula to the border with Malaysia.

The geology of Thailand has largely controlled the physiography. The Northeast undulating, slightly elevated plain is composed of Cretaceous sandstone, shale and siltstone with thick beds of rock salt and gypsym. Underlying this and exposed around the periphery of the plain are Jurassic and Triassic sandstones, conglomerates and shales. The hills in the southeast, extensions of the Cardamon Mountains of Cambodia, have a core of Precambrian gneiss and schist, Palezoic shales and sandstones intruded by Triassic and Tertiary granites. The northern highland mountains are a complex series of Paleozoic, Triassic and Jurassic sediments with Paleozoic and Triassic intrusives, mainly granites, but also including ultramafics. Precambrian rocks also occur. The more resistant limestone and quartzite have formed the ridges whereas the sandstone and shales have formed the valleys. The Central Plain is possibly a block which was downfaulted during late Tertiary and which has been accumulating Quaternary and Recent sediments of mainly fine gravel. The peninsular area has Paleozoic, and, to a lesser extent, Mesozoic rocks of sandstone, shale and limestone which have been intruded by granite bodies of various ages.

BURMA

Burma is composed of three geographical regions. The Arakan Yoma Mountains form the western topographic barrier to India. The Shan plateau occupies the eastern part of the country narrowing to continue southward into the Malay peninsula as the Tenasserim Mountains. The central basin lies between the two and contains smaller hills elongated north-south. The major river in the basin is the Irrawaddy, with a number of important tributaries. The Sittang is a second important river which runs north-south, east of the Irrawaddy. The Arakan Yoma mountain range has a core of crystalline rocks, probably including volcanics and metavolcanics, with folded sedimentary rocks on either side of the north-south orientation. These are all of Tertiary age, but Mesozoic rocks are also known to occur, including Cretaceous serpentine intrusions which were apparently associated with intense faulting. The Shan plateau is made up mainly of gneisses, some of Precambrian age, and a thick, resistant Peleozoic limestone. The Tenasserim mountains are granite blocks which have intruded older sedimentary rocks of uncertain age. The central basin contains thick sequences of Tertiary sedimentary rocks with overlying recent sediments.

MALAY ARCHIPELAGO

The Arakan Yoma mountains of western Burma continue south as islands separating the Andaman Sea from the Bay of Bengal and the Indian Ocean. These islands are Preparis, Great Coco, Andaman, Nicobar and Sumatra. From Sumatra the mountain chain turns east through Java, Bali and other Lesser Sunda Islands to the Moluccas. The entire system is known as the Parade of Mountains. The island system contains a double arc of mountains. The inner arc is volcanic and is made up of the islands of Sumatra, Java, Bali, Lomblok, Sumbawa and Flores. Mesozoic sediments, late Tertiary sediments and granitic intrusions are found in the folded mountains which are interspersed with volcanic centers whose rocks are mainly andesite to basalt in composition. The mountain chain in Sumatra is along the western coast. The eastern coast is largely a flat alluvial plain with many large rivers.

The remainder of the Greater and Lesser Sunda Islands, with the possible exception of Borneo, and including the Molluca Islands, form a geological active complex series of island arcs with active volcanoes and frequent earthquakes which separate continental Asia from Australia. In the new theory of plate tectonics, this is the area where the Southeast Asian plate is passing over the Indian plate (which includes Australia, New Guinea and much of the Indian Ocean as well as India) which is being destroyed. The Southeast Asian plate is also colliding with the Indian plate in the west. All of the islands are mountainous and most are volcanic. Sumatra alone has 12 active volcanoes among a total of over 90.

The outer arc of islands is made up of much smaller, scattered islands. These are not volcanic and no crystalline basement has been exposed (Wylie, 1971). These islands are composed of tighly folded marine sediments.

The Greater Sunda Islands are largely made up of Tertiary sediments although metamorphosed Paleozoic and older rocks as well as intrusive bodies are also found. Precambrian, Jurassic and Cretaceous rocks are exposed in Sumatra's western range of mountains. Granites and basalts of various ages are also present. In Java, Cretaceous rocks also occur, strongly folded in the central part of the island, but Tertiary sequences of sandstone, shale, limestone, and marl are prevalent over most of the island. Many volcanoes are located along the central spine of the island. Volcanic activity is believed to have begun in Miocene time and to have reached a peak of activity in the Quaternary. The Lesser Sunda and Molucca islands have similar geology, all dominated by volcanism.

Borneo is still not well understood nor fully explored. The great spinal range of mountains lie in the north, in the narrow part of the island. Borneo is made up principally of sedimentary rocks which are mostly of Tertiary and Quaternary age. Sandstones and limestones predominate. Granite and other intrusive bodies of various ages also occur. The major mountain range between Indonesian Borneo and Sarawak (East Malaysia) is composed of schists with sandstones and slates. Great winding valleys have formed mainly in the southern part of the island where Quaternary and recent sediments are found.

The Philippine Islands portion of the Malay archipelago is highly irregular in shape and width when compared to other volcanic island arcs in the western Pacific volcanic chain. The major islands are Luzon in the north and Mindanao in the south. All are mountainous with narrow coastal plains. Although volcanic rocks again dominate over all other materials, all rock types can be found. Several mountain ranges are contained in the islands, especially on Luzon and Mindanao. These are made up of thick sedimen-

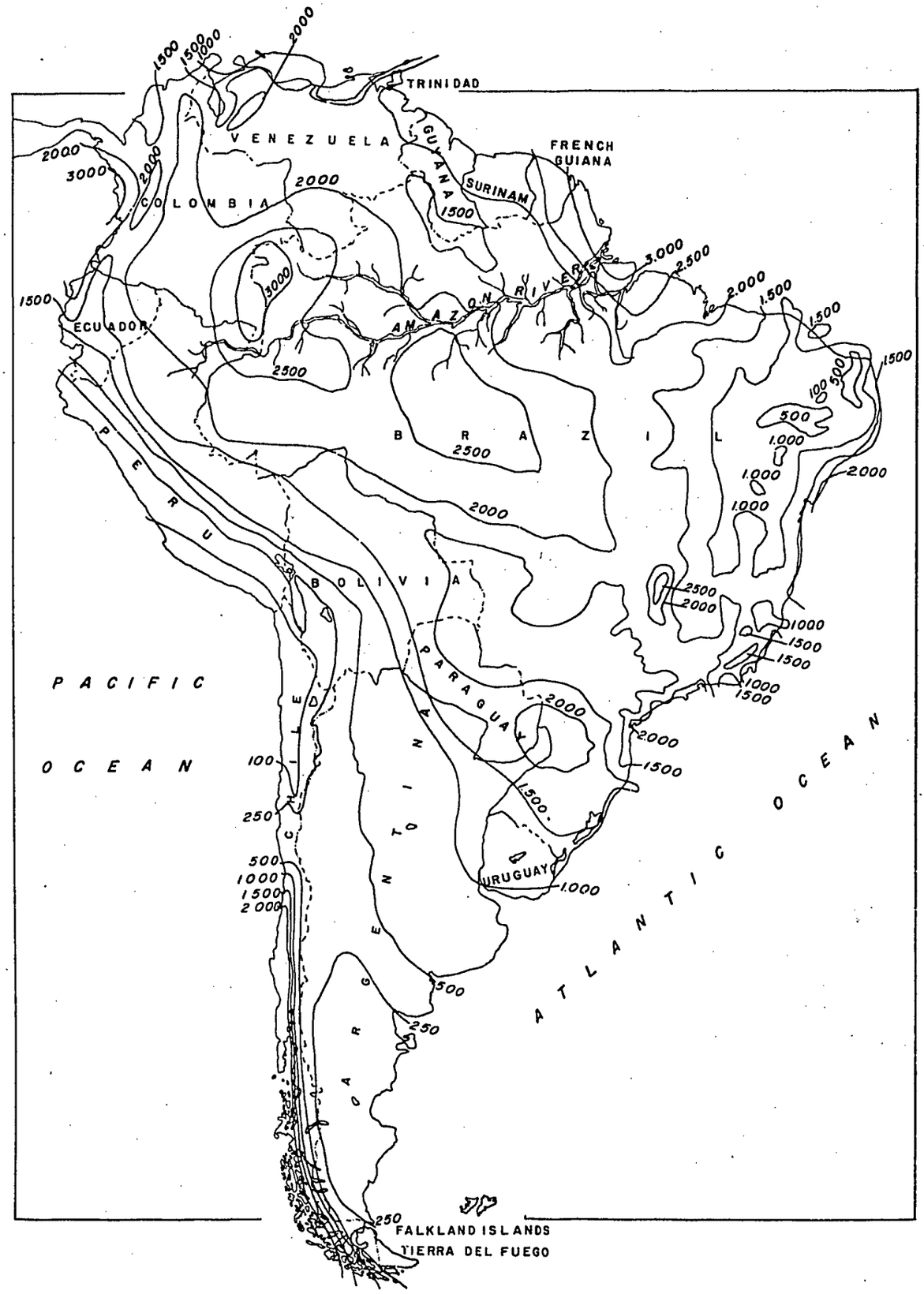

FIGURE 2.6 – MEAN ANNUAL PRECIPITATION IN SOUTH AMERICA (in millimeters)

tary sequences. Folding occurred in Tertiary and Quaternary producing mountain chains with the same orientation as the islands. A number of recent uplifts have been recognized and mountain building is still believed to be in progress.

CLIMATE AND VEGETATION

Most of South and Central America and Southeast Asia have hot and humid climates. There are exceptions where certain areas lie in rain shadows and have drier or arid climates. An example is the west coast of South America where the Atacama Desert is one of the driest areas in the world. Other areas with rain shadows are northeast Thailand, central Burma, the western Celebes and Lesser Sunda Islands. Northeast Brazil has erratic rainfall with occasional devastating droughts.

Most of these areas have high temperatures which vary little year-round. Elevation and prevailing winds modify temperatures, however; climages in the Andes and Southeast Asian Mountains can be temperate and even alpine.

In South and Central America and Southeast Asia, tropical rainforests predominate. Monsoonal, or seasonal rain forest occupy smaller areas and areas covered by savannas are relatively small.

Africa receives less rainfall. There are large desert regions and tropical rainforests cover smaller areas. Much of the continent is savanna with reduced rainfall.

South America

South America contains the largest continental area of humid tropical climates (FAO, 1971). Most of the continent lies within the equatorial latitudes. Although the continent extends further south than Africa or Australia, it narrows wedge-like to the Tierra del Fuego. Ocean currents moderate summer and winter extremes of temperature.

Temperature varies around the continental periphery. Within the Amazon Basin the mean annual temperature is about 27°C (80°F) and seasonal variations are small. Winter temperatures in Patagonia drop below freezing regularly and freezing also occurs as far north as southern Brazil at elevations of over 650 m (2,100 ft). Temperate and polar climates are found in the Andes, depending on elevations. The Humboldt current lowers the temperatures along the west coast, south of Colombia.

The rainfall pattern in South America reflects the prevailing winds and the topography. Over most of South America east of the Andes the prevailing winds come from the northeast to the southeast from the Atlantic ocean, often carrying moisture. The mean annual rainfall at the mouth of the Amazon exceeds 3,000 mm (120 in) as it also does along the western coast of Colombia. The inner Amazon Basin, southern Chile, some areas of southern Brazil and northeastern South America, from the Amazon to Venezuela have over 2,500 mm (100 in). Almost the entire remaining tropical region has over 1,000 mm (40 in) with the exception of those areas in the Andean rain shadow, that is the Atacama Desert of northern Chile, western Bolivia and western Peru which has under 250 mm (10 in); small areas in northern Venezuela and Colombia, along the coast (250 to 1,000 mm; 10 to 40 in); and northeastern Brazil (250 to 1,000 mm; 10 to 40 in). Northeastern Brazil is occasionally subject to long periods of drought. The mean annual rainfall for South America is shown in Figure 2.6.

There are five maior vegetation patterns in South America, if the Andes ranges with their temperate climates and the coastal plains with their wetlands are omitted (Figure 2.7). Equatorial rain forest covers almost all of the Amazon basin and the Guyana highlands. Tropical seasonal, or monsoonal, climate occurs in southern coastal Brazil, extending into Paraguay and bordering the equatorial rain forest to the north, extending into eastern Bolivia, and in the narrower belt along the northern fringes of the rain forest. Tropical savannas, known as cerrado in Brazil, occupy a wide area between the inland and coastal seasonal forests, from Paraguay to northeast Brazil. Central Venezuela and eastern Colombia also have savanna grasslands. Semi-arid and arid woodland and shrub are found in northeast Brazil, in Patagonia and the Argentine Pampas, where they extend into western Paraguay.

Central America

The prevailing climatic type in Central America is dependent on the elevation. The lowlands, under 760 m (2,500 ft) or so, have tropical climates. Above 760 m (2,500 ft) the climate is temperate and above 1,830 m (6,000 ft) it is classified as cold. Although temperatures vary with elevation, variations during the year are relatively slight. Mean annual rainfall is shown in Figure 2.8.

Rainfall is seasonal. The rainy season, which is from June to October, generally is considered the winter season. The extreme northwest receives the greatest rainfall, over 4,000 mm (160 in). Most of the remainder has between 1,000 and 3,000 mm (40 and 120 in) rainfall, but coastal regions bordering the Caribbean locally report as much as 6,000 mm (240 in).

Tropical rain forests predominate along the Caribbean coast in Panama and Costa Rica in a band 80 to 160 kilometers (50 to 100 miles) wide through Central America to British Honduras, up to elevations of 760 m (2,500 ft). Tropical seasonal rain forests or savannas covers much of the Pacific lowlands and mountain slopes again up to about 760 m (2,500 ft). The vegetation pattern is shown on Figure 2.9.

Africa

The tropical region of Africa with its minor topographical relief has a monotonous progression of climates north to south that is slightly modified by the high plateaus and mountains of eastern Africa. On entering the tropics from the north one passes through desert, sub-arid, semi-humid and finally humid at the equatorial region. This reverses on continuing south of the equator. In eastern Africa, the highlands receive more rain, but along the coast of the Indian ocean, arid and semi-arid conditions persist far south of the equator.

Temperatures vary with elevation and prevailing winds.

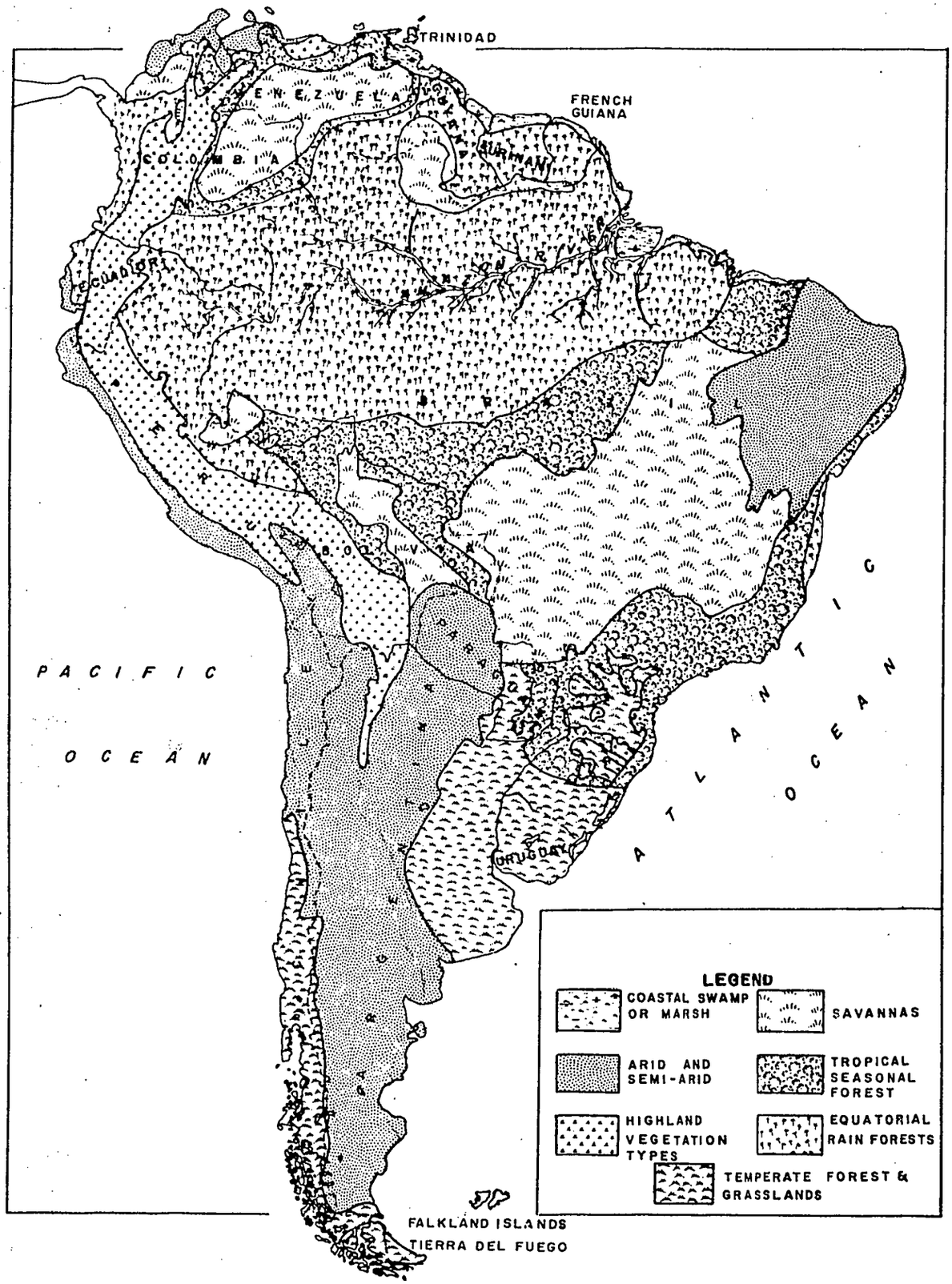

FIGURE 2.7 – MAJOR VEGETATION OF SOUTH AMERICA (after FAO-UNESCO, 1971)

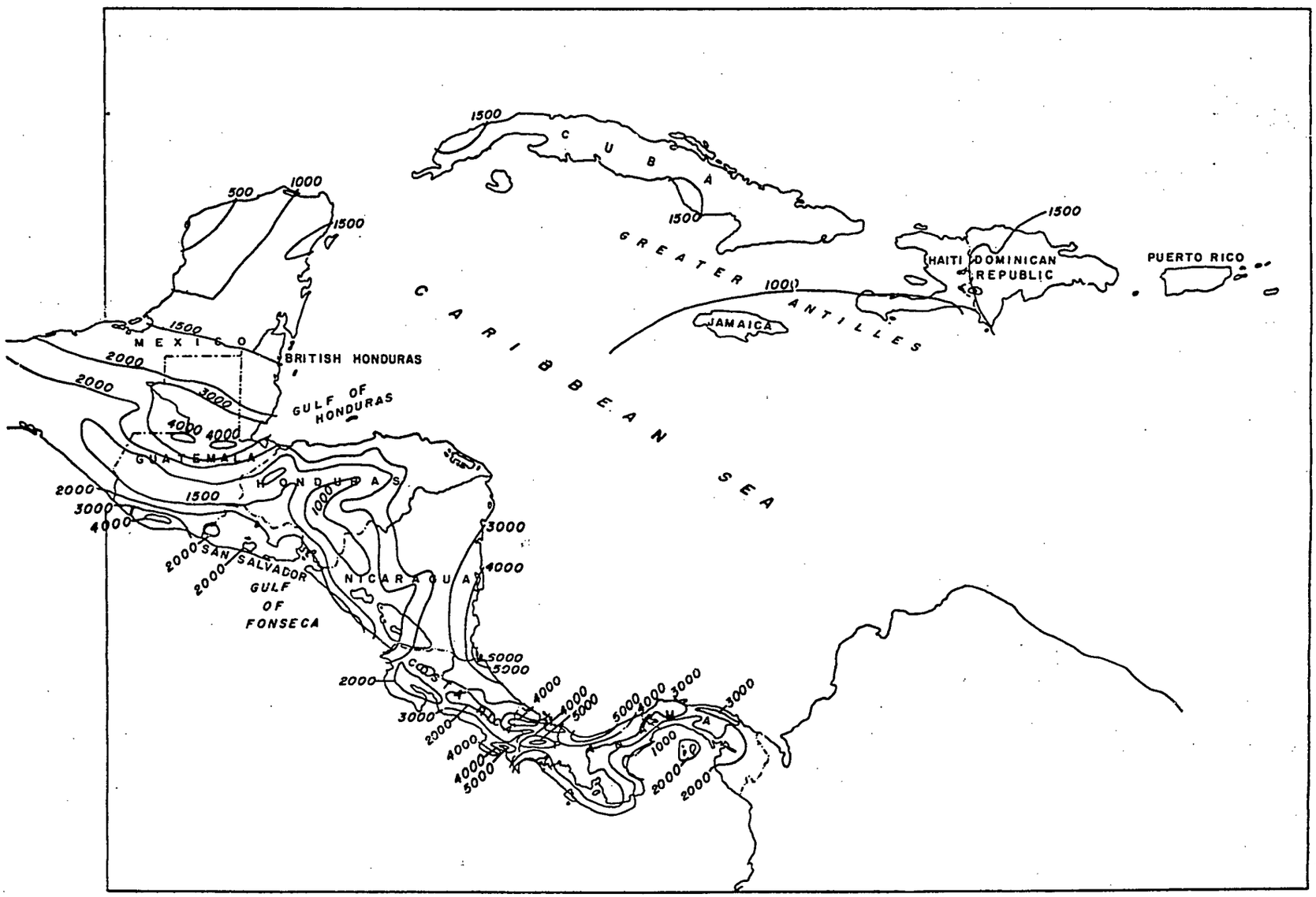

FIGURE 2.8 – MEAN ANNUAL PRECIPITATION IN CENTRAL AMERICA (in millimeters)

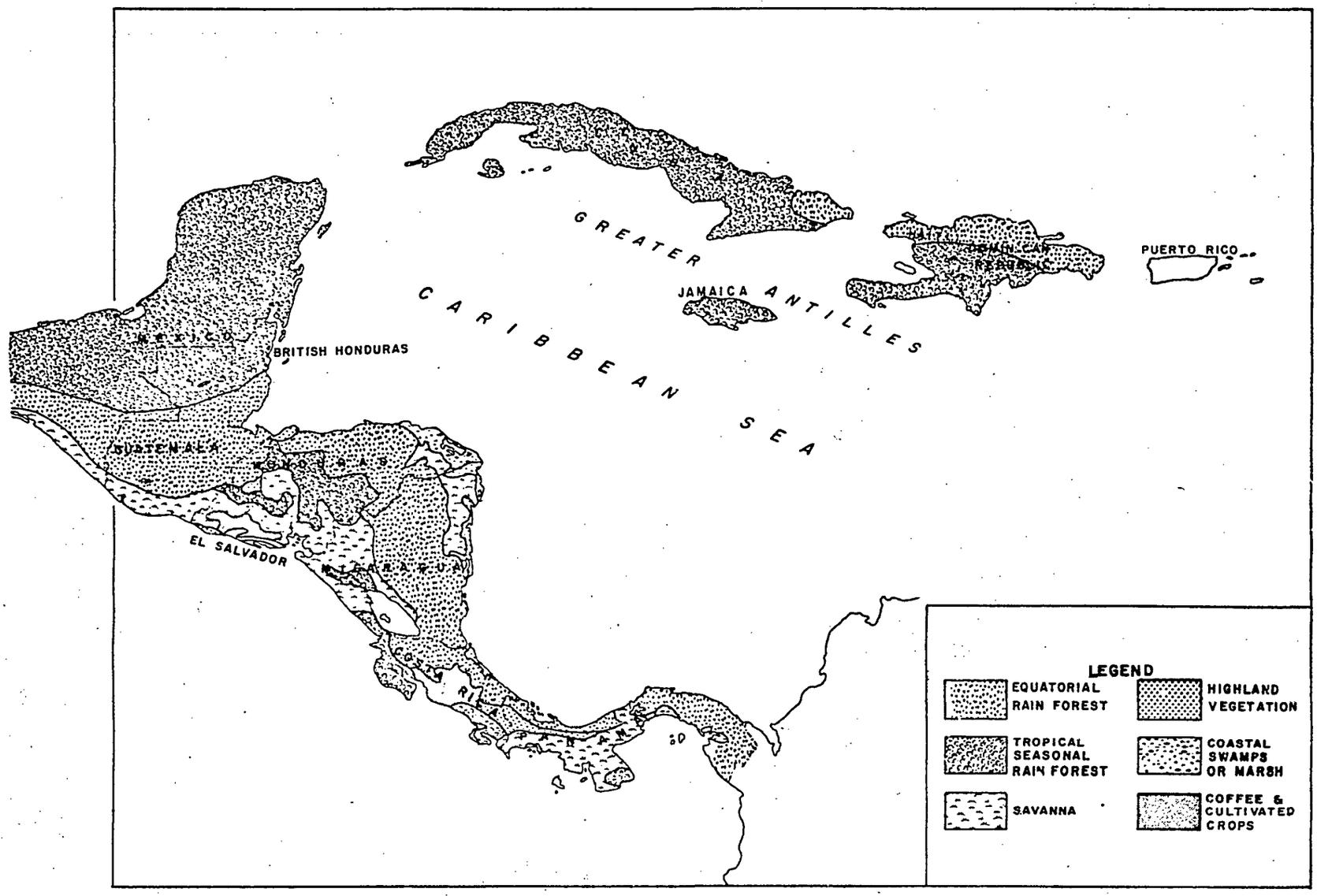

FIGURE 2.9 – MAJOR VEGETATION REGIONS OF CENTRAL AMERICA

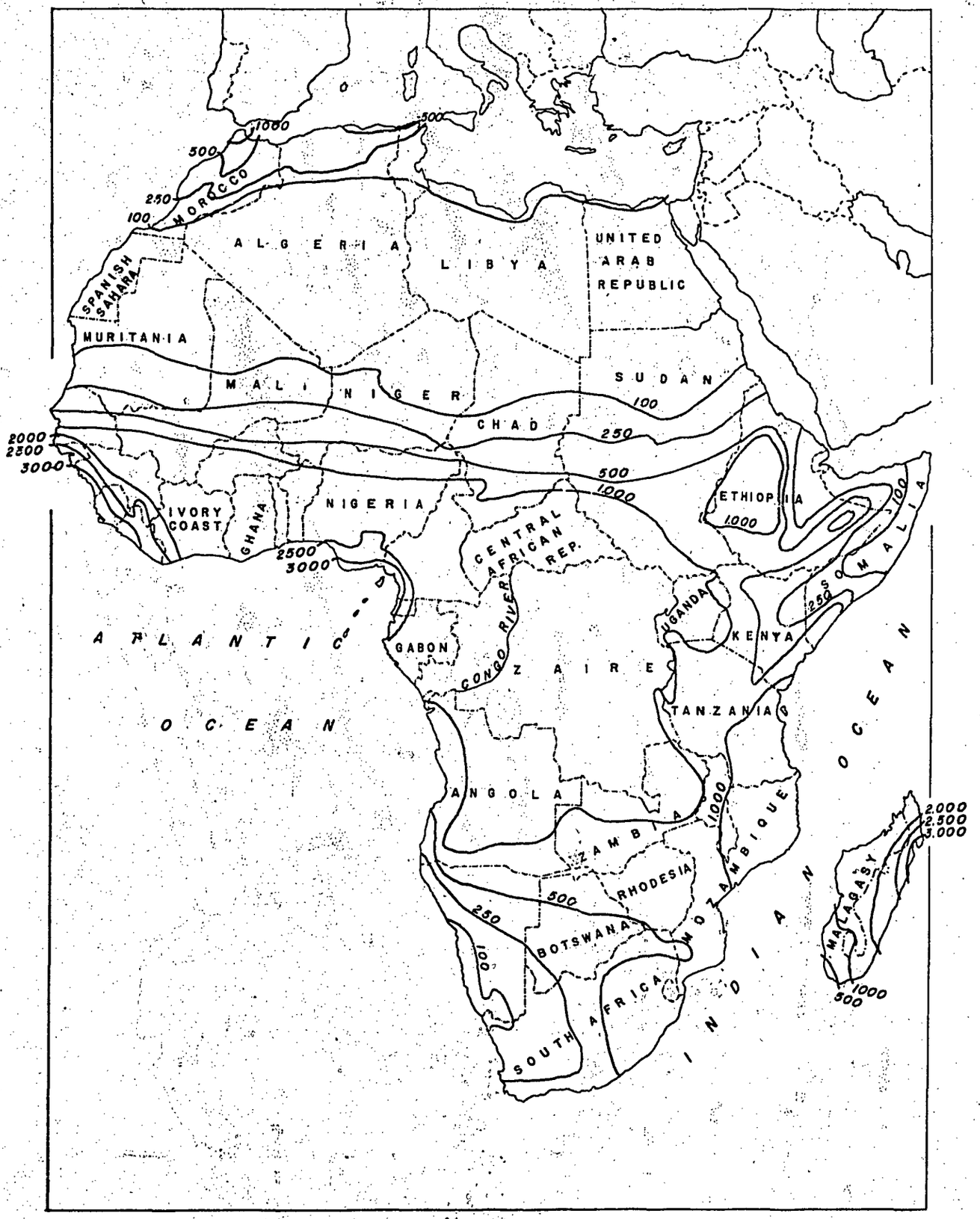

FIGURE 2.10 – MEAN ANNUAL PRECIPITATION IN AFRICA (In millimeters)

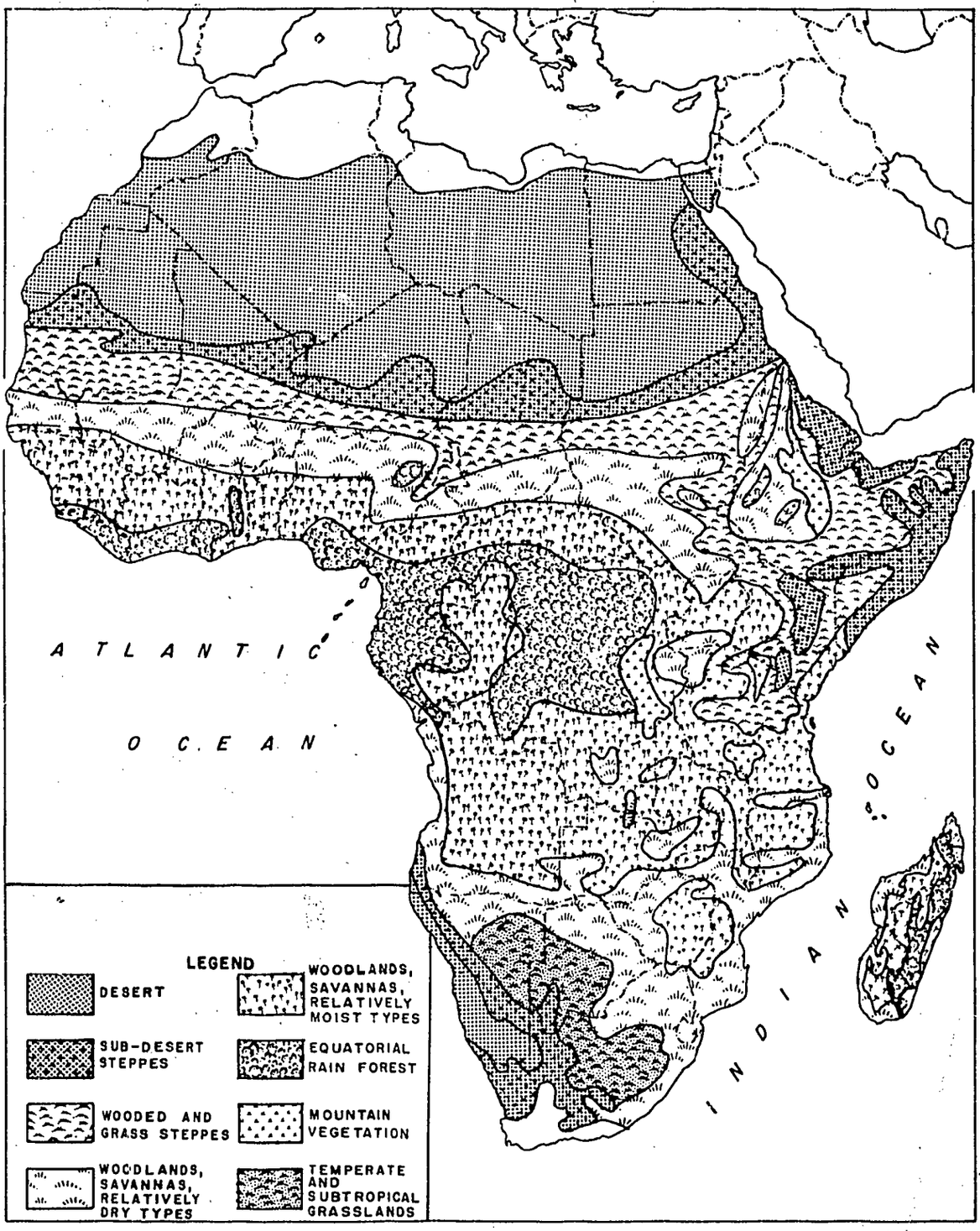

FIGURE 2.11 – MAJOR VEGETATION REGIONS OF AFRICA

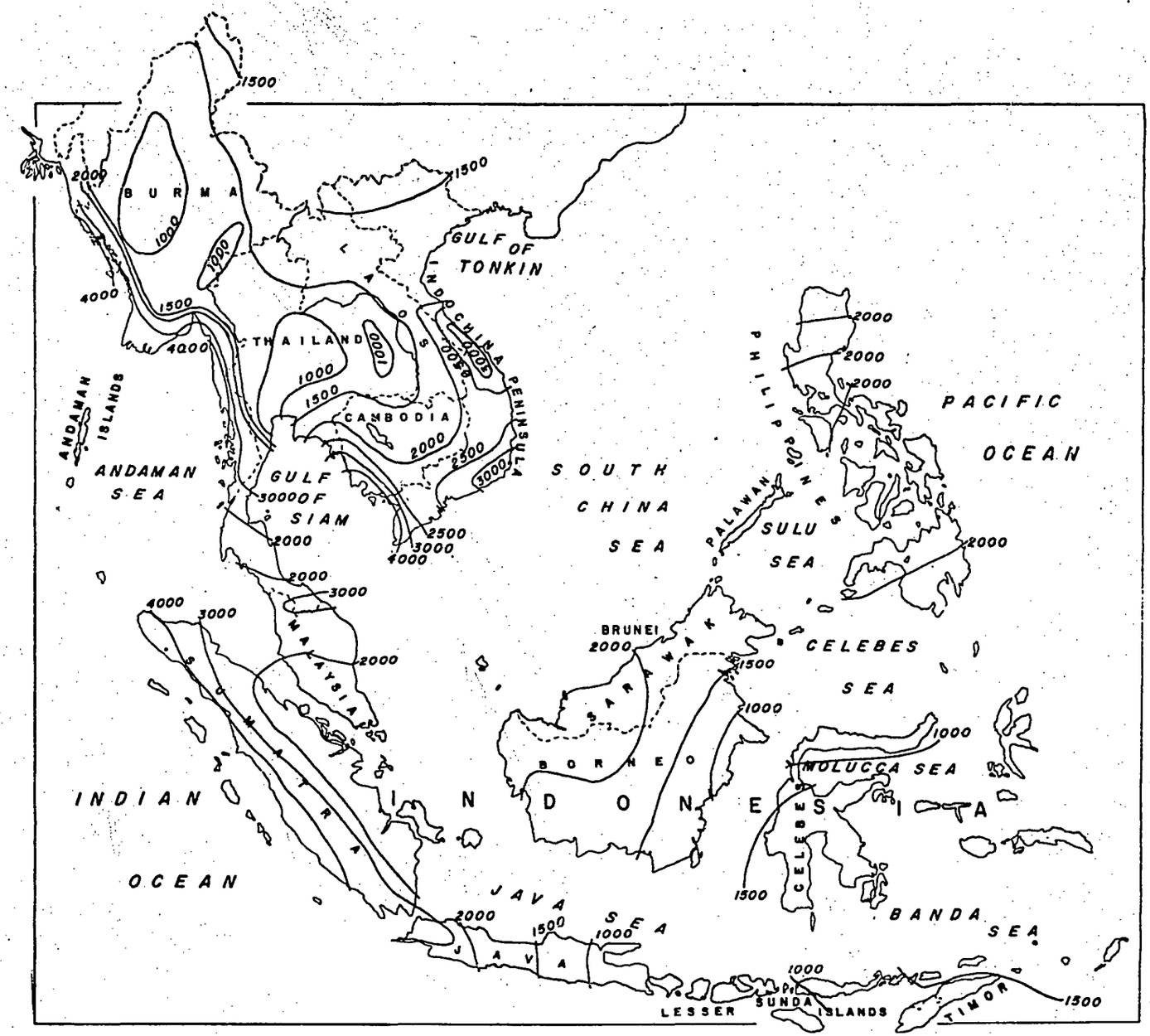

FIGURE 2.12 — MEAN ANNUAL PRECIPITATION IN SOUTHEAST ASIA (in millimeters)

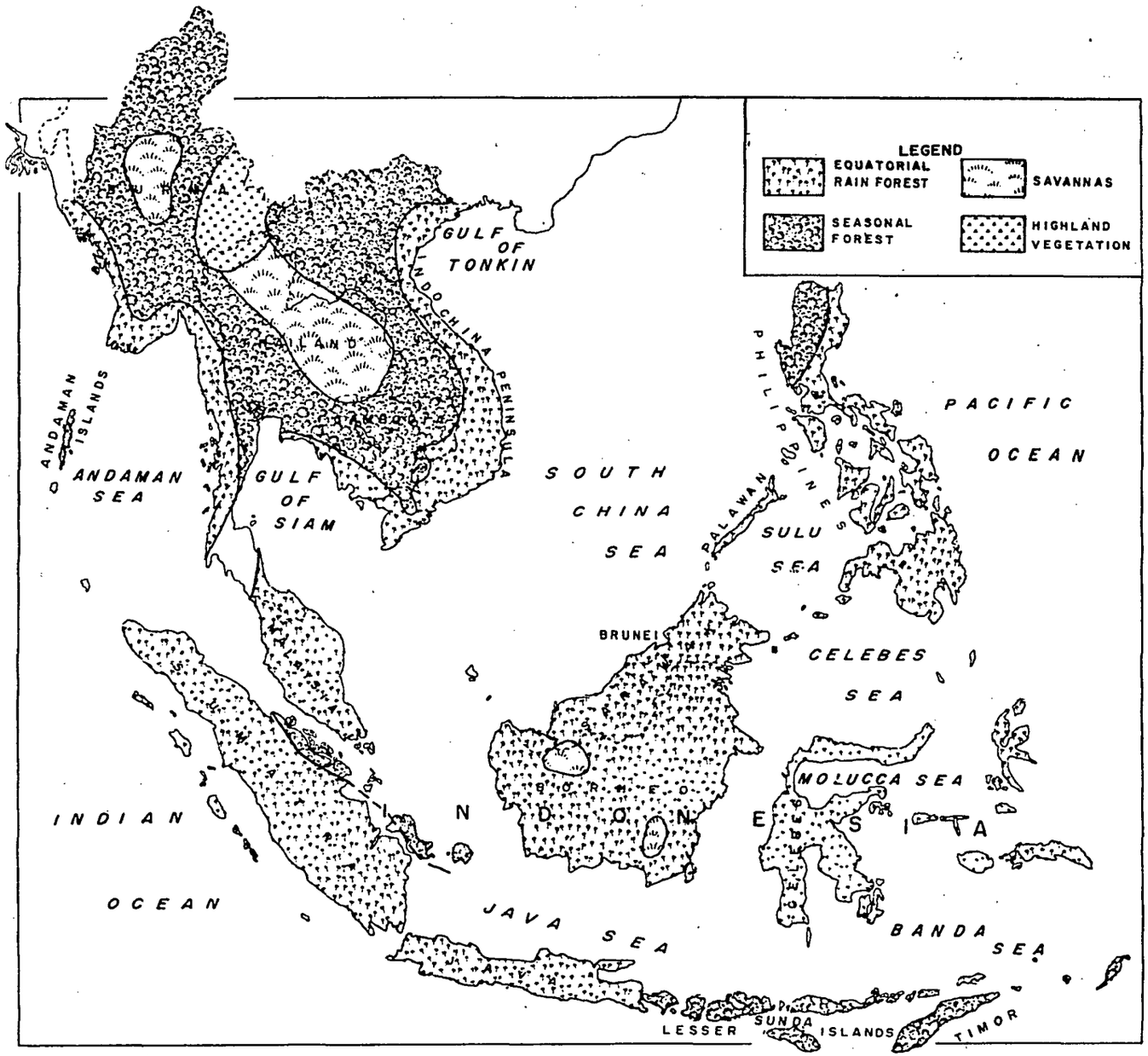

FIGURE 2.13 – MAJOR VEGETATION REGIONS OF SOUTHEAST ASIA

They are not as uniform as in South America and Southeast Asia. This is primarily due to seasonal winds blowing from desert regions, reducing the coastal cooling effect.

Annual mean rainfall varies from under 100 mm (4 in) to over 4,000 mm (160 in). A rainfall map is shown in Figure 2.10. The highest rainfall occurs along parts of the south coast of West Africa. Over 1,000 mm (40 in) falls in much of Central Africa. The remainder of the continent has over 200 mm (8 in) with the exception of the Sahara regions; and portions of the east coast of Central Africa and west coast of South Africa. The rainfall commonly occurs in two seasons, a major or "big rains" and a minor "small rains".

The type of vegetation reflects the rainfall as can be seen in Figure 2.11. Relatively dry to relatively moist types of woodlands and savannas are found in regions with 200 to 1,600 mm (8 to 65 in) rainfall. Moist forest (equatorial rain forest) are restricted to areas with over 1,600 mm (65 in), a relatively small part of Africa, generally restricted to parts of West Africa, along the coast of Gulf of Guinea, the interior Congo and eastern Malagasay.

Southeast Asia

The more variable climate in Southeast Asia is that of mainland Asia. The Malay archipelago has little variation, most of which is associated with elevation.

Indochina is transitional between the continental extremes to the north and the humid equatorial region to the south. The cool dry season is from September to March, when the prevailing winds are from the north. The humid hot season is from May to September. Vietnam has less than 1,500 mm (60 in) in the extreme north and over 4,000 mm (160 in) in the extreme south. With few exceptions equatorial rain forests have developed in over half of the region, with cultivated fields and some savanna in the remainder. Mean annual rainfall is shown in Figure 2.12 and vegetation in Figure 2.13.

The climate of Thailand varies considerably, particularly the rainfall, depending of location and exposure. The average annual temperature is fairly constant in all of Thailand, between 26° and 30° C (78° and 86° F). The rainfall varies from a low of 800 mm (30 in) in the northeast to 1,000 to 1,500 mm (40 to 60 in) for most of the remainder of non-peninsula Thailand. Isolated pockets occur with over 2,500 mm (100 in). The extreme southeast, bordering Cambodia, has 4,000 mm (160 in). Peninsula Thailand has over 3,000 mm (120 in) along the Burma border and as much along the Malaysia border, and over 2,000 mm (80 in) in between.

As in the other regions the vegetation largely reflects the precipitation. Much of non-peninsula Thailand has savanna-type vegetation. The west, extreme southeast and most of the Thailand peninsula have tropical monsoon climate with heavy annual rainfall and a short dry season. Equatorial rain forest, with year-round humidity, is found only along the east coast of Thailand in the southern peninsula and in most of Malaysia.

Burma is also included in the great monsoonal region of Asia, but the topography strongly moderates the climate. There are three seasons: cool (November-February); hot (March-May) and rainy (June-October). The temperature range is quite small near the coast, but increases by 11° C (20° F) or more in the northern part of the country. The two major mountain ranges have nearly 5,000 mm (200 in) of rainfall. The Shan plateau has moderate to high rainfall. The heart of Burma lies in a rain shadow and while Rangoon has 2,500 mm (100 in) per year, the interior has under 1,000 mm (40 in).

The vegetation depends on the elevation and the rainfall. There are occasional frosts above 900 m (2,950 ft) and the vegetation is mostly forests of evergreen oak and open grasslands. Equatorial rainforest occur below 900 m (2,950 ft) if the rainfall is over 2,000 mm (80 in); monsoonal (deciduous) forest occur if the rainfall is between 1,000 to 2,000 mm (40 to 80 in) and scrubland or savanna if the rainfall is under 1,000 mm (40 in).

Along the wide expanse of the Malay archipelago, the major differences in climate are due to elevation and rainfall. The temperatures are high, averaging 27° C (80° F) with variations slight except in the more northerly latitudes of the Philippines. The rainfall varies considerably, however. The mountains of western Sumatra have well over 4,000 mm (160 in) and most of the remainder of the archipelago has over 2,000 mm (80 in). A rain shadow covers eastern Borneo, the northern Celebes and most of the Lesser Sunda Islands. Most of these areas have between 500 and 1,000 mm (20 and 40 in), annually. Tha rainfall in most of the islands is well distributed throughout the year but east of Java, as the total annual rainfall becomes less, the dry season becomes longer, to as much as 6 months. This pattern is also true in the Philippines, from Mindanao northward. In all of the islands, elevations of the mountains cause many local variations.

The vegetation in the Malay archipelago is almost everywhere equatorial rain forest The monsoonal (deciduous) forests and savannas occur only in the drier areas.

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CHAPTER 3
TROPICAL SOIL DEVELOPMENT

INTRODUCTION

Soil development in the tropics varies in many respects from soil development in temperate regions. For example, the weathering processes tend to be more rapid and intense in warm, humid climates with abundant vegetation. Therefore the chemical weathering processes predominate in tropical regions.

Weathering is defined in the American Geological Institute's Glossary of Geology (1972) as "the physical disintegration and chemical decomposition of rock that produce an in-situ mantle of waste and prepare sediments for transportation". There are two processes, mechanical and chemical. Mechanical weathering includes disintegration through the action of temperature changes, wind, frost and ice, and root growth. Chemical weathering essentially involves decomposition and dissolution.

The weathering agents which cause the decomposition and dissolution of rock are water with dissolved oxygen, carbon dioxide and organic acids. The effectiveness of these agents depends upon their concentration, the temperature of the rock or soil material and the type of vegetation cover.

Chemical changes that take place during weathering are solution, oxidation, hydrolysis and carbonation. Certain minerals, such as carbonates and sulfates, are water soluble and are taken directly into solution. Some elements are susceptible to oxidation. Iron, for example, exists as the uncombined element, as the ferrous iron (Fe^{++}) and, with further oxidation, as the ferric iron (Fe^{+++}) common as goethite, FeO(OH) and as hematite, Fe_2O_3. Many elements are susceptible to hydrolysis or combination with $(OH)^-$. Carbonation, or the formation of carbonates, occurs through changes involving carbon dioxide.

The susceptibility of minerals to chemical attack has been shown by Goldich (1938) to be the reverse of Bowen's reaction series. The last-formed minerals in igneous rocks have the greatest stability. Some minerals in their order of decreasing stability are:

Quartz	
Muscovite	
Potash feldspar	
	Biotite
Alkalic plagioclase	
Alkalic-calcic plagioclase	Hornblende
Calcic-alkalic plagioclase	
	Augite
Calcic plagioclase	
	Olivine

This series is not a reverse reaction series; olivine does not revert to augite, etc., but it is the least stable mineral in the series.

Various clay minerals are the ultimate products of chemical weathering as opposed to discrete mineral grains which are the usual products of mechanical disintegration. One of the most straightforward chemical reactions is the formation of kaolinite from orthoclase:

$$\underset{\text{orthoclase}}{4K\,AlSi_3O_8} + 2CO_2 + 4H_2O \rightarrow \underset{\text{kaolinite}}{Al_4Si_4O_{10}(OH)_8} + 8SiO_2 + 2K_2CO_3$$

The water and carbon dioxide react with orthoclase feldspar to produce kaolinite and the silica and potassium carbonate are carried away in solution. Most reactions in the formation of clay minerals are considerably more complex. The desilication of feldspars as well as other primary minerals may be a step-process with new minerals formed as additional silica is removed.

CLAY MINERALOGY AND CLAY MATERIALS

The clay minerals that form in the tropics are principally of four groups: kaolin, halloysite, montmorillonite and illite. Two other varieties, vermiculite and chlorite, are also constituents of some tropical soils. Each group has a number of mineralogical variations reflecting differences in mineral structure or chemical composition. Most of these variation are rarely important insofar as engineering behavior is concerned. One exception is hydrated halloysite. A classification of the principal clay minerals is given in Table 3.1.

It should be noted that the clay minerals include not only the minerals with crystalline structure listed in Table 3.1 but other cryptocrystalline or amorphous varieties as well. The amorphous varieties constitute the allophane group.

The size of the mineral particles is generally under 2 microns and almost always under 5 microns (0.002 and 0.005 mm). Particulate mineral matter other than the clay minerals which also occurs in this size range is considered a clay material. In tropical soils, many of the fine particles within the clay size fraction are no longer clay minerals. Thus the distinction is important.

The clay minerals are essentially hydrous aluminum silicates. Hydrous silicates of magnesium and iron are less common.

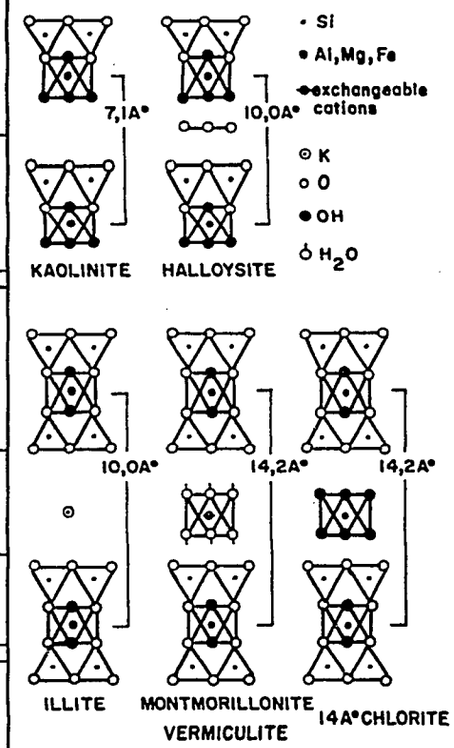
The hydrated oxides of iron and aluminum are common materials in the clay fraction of tropical red soils. The hydrated aluminum oxides include the minerals gibbsite, cliachite, boehmite and diaspore with perhaps the first two being the most common. The hydrated iron oxides include the minerals goethite and limonite. Limonite is considered a mineraloid because of its amorphous state and compositional variations.

Gibbsite is the most widely recognized hydrated aluminum mineral in red tropical soils. It has the chemical formula $Al(OH)_3$. Another form has been described as cliachite, with the formula $Al_2O_3(H_2O)_X$, which is usually associated with gibbsite. This was considered by many to be an amorphous mineraloid, but Bleackley (1964) found that the bauxites of Guyana contain large amounts of this material in cryptocrystalline form. The chemical composition and the lattice structure are the same as gibbsite.

TABLE 3.1
Classification of Principal Clay Minerals
(after Degens, 1965)

LAYERS	EXPANSION	GROUP	SPECIES	CRYSTALLOCHEMICAL FORMULA
TWO-SHEET (1:1)	NON-SWELLING	KAOLINITE	KAOLINITE	$Al_4(OH)_8[Si_4O_{10}]$
			DICKITE	
			NACRITE	
	NON-SWELLING and SWELLING	HALLOYSITE	HALLOYSITE	$Al_4(OH)_8[Si_4O_{10}]\cdot(H_2O)_4$
			METAHALLOYSITE	$Al_4(OH)_8[Si_4O_{10}]\cdot(H_2O)_2$
	SWELLING	MONTMORILLONITE (SMECTITE)	MONTMORILLONITE	$\{(Al_{2-x}Mg_x)(OH)_2[Si_4O_{10}]\}^{-x}Na_x\cdot nH_2O$
			BEIDELLITE	$\{Al_2(OH)_2[(Al,Si)_4O_{10}]\}^{-x}Na_x\cdot nH_2O$
			NONTRONITE	$\{(Fe_{2-x}Mg)(OH)_2[Si_4O_{10}]\}^{-x}Na_x\cdot nH_2O$
THREE-SHEET (2:1)	NON-SWELLING	ILLITE (HYDROMICA)	ILLITE-VARIETIES	$(K,H_3O)Al_2(H_2O,OH)_2[AlSi_3O_{10}]$
	SWELLING	VERMICULITE	VERMICULITE	$(Mg,Fe)_3(OH)_2[(AlSi_3O_{10}]Mg\cdot(H_2O)_4$
THREE-SHEET + One-Sheet (2:2)	NON-SWELLING	14A-CHLORITE (NORMAL CHLORITE)	CHLORITE-VARIETIES	$(Al,Mg,Fe)_3(OH)_2[(Al,Si)_4O_{10}]Mg_3(OH)_6$

STRUCTURE (SCHEMATIC)



Limonite is a general term for a group of amorphous hydrated iron oxides with varying composition, physical properties and chemical properties. It is not possible to identify positively because of these variations and its amorphous state. The chemical formula is $H_2FeO_4(H_2O)_x$ and like cliachite, it may contain a variable amount of water. The evaporation of carbonic acid usually present in the water is one way in which iron is separated. A second is by means of bacteria which absorb iron from the ground water and subsequently deposit it as ferric hydroxide.

Goethite, with the chemical formula $Fe_2O_3 \cdot H_2O$ forms with dehydration of limonite. Whereas limonite is amorphous, the loss of water results in a crystalline structure and more constant chemical composition that is goethite. Goethite is very common product of tropical weathering.

Hematite, Fe_2O_3, is a common iron mineral in tropical red soils that have been subjected to additional oxidation, usually through drying. It rarely occurs in other tropical soils, and is uncommon or a minor constituent of hydrated red soils.

TROPICAL SOIL FORMATION

Tropical soils are the products of chemical weathering. The various soil forming factors that constitute the chemical weathering processes are briefly discussed in this section. More detailed discussions are available in the literature (Grim, 1968; Loughnan, 1969; and Mohr et al., 1972).

Grim has described the various soil forming factors which control weathering processes. The composition and texture of the parent rocks are important in the initial stages of weathering but become less important with time. Climatic features such as the quantity of rainfall, and particularly the seasonal distribution of rainfall, determines the intensity of the weathering processes. Topography effects vertical water movement and consequently the rate of removal of the soluble materials. On steep slopes run-off may be as active in eroding the weathered material as infiltration is in forming it. The type and amount of vegetation can be important in the formation of organic acids and in the assimilation of silica. Finally, time is a controlling factor, for example, in warm, humid climates typical of the tropics, the time required to chemically alter rock material is considerably less than in temperate climates.

The Soil Forming Factors

Of the primary factors which influence the soil forming processes, climate is considered the most important. Hot, humid climates of the tropical regions provide the essential degree of weathering for the formation of iron and aluminum and leaching of soluble silica that is characteristic of the red tropical residual soils. It must be remembered, however, that not all existing soils were formed under present-day climates.

TEMPERATURE

Heat has an important influence on the geological weathering processes, particularly on the chemical reactions that occur during the soil forming processes. For each increase in 10°C, the rate of chemical reactions increases 2.5 times.

The effect of temperature in tropical weathering is rather indirect. Primarily it helps either the destruction or accumulation of humus. Mohr et al., (1972) have shown the relation between these two and temperature. They have found that at soil temperatures above 30°C, humus destruction resulting from bacterial activity is predominant, whereas below 20°C humus accumulation is predominant. Thus, in tropical lowlands, a bare soil which is adequately moistened by rain will exhibit a flora which is predominantly bacterial while in a mountainous area with a dense forest vegetation, at an elevation where the soil temperature is about 20°C, the soil will show an accumulation of humus. This is important from the point of view of the clay-mineral content of the final product.

The occurrence of laterite is prevalent where temperatures are warm or are believed to have been warm at the time of their formation. Though there have been few studies on the thermal conditions of laterite formation. Crowther (1930) demonstrated that SiO_2/Al_2O_3 increases as temperature rises at constant humidity. Maignien (1966) suggested that contemporary lateritic red tropical soils have developed at mean temperatures of around 25°C.

RAINFALL

Red tropical soils are believed to always correspond to climate in which the wet period is warm; this applies to semi-humid tropical and equatorial climates. On the other hand, subtropical climates in which precipitation occurs in the cold season do not seem to favor laterization even if the temperature is above 20°C.

There seems to be no upper limit of precipitation under which lateritic soils may form (Morin and Ayetey, 1971). In lower Guinea, for example, where the climate is tropical, lateritic soil is well defined at an annual precipitation of more than 6,000 mm (240 in). With regard to incrustation itself, it seems to be closely related to precipitation insofar as it affects the water regime of the soils.

The lower limit for the formation of red tropical soils is about 750 mm (30 in). According to Tricart (1972) as this level of rainfall is approached in West Africa, red soils disappear (except for paleosols); desert varnish appears; kaolinite becomes less abundant while montmorillonite becomes more abundant; and savanna vegetation transition to grass land vegetation occurs. A more important factor perhaps than mean annual rainfall in the soil forming process is the moisture diagram of a given area. This will be discussed again later in this chapter.

The effect of rainfall on the chemical composition and mineralogy of some West African red tropical soils have

been analyzed (Morin and Ayetey, 1971). Their findings show that, in general, as the rainfall increases, the amount of kaolinite decreases, as do both silica-alumina and silica sesquioxide ratios, whereas the amount of gibbsite increases. Also the lowest silica-sesquioxide ratios are usually found in association with the highest percentages of gibbsite.

Hay and Jones (1972) studied the weathering of basalt that is 10,000 to 17,000 year old on the island of Hawaii. The present-day rainfall is from 250 to 6,350 mm (10 to 250 in) per year. They found the degree of leaching corresponds closely with the rainfall. Gibbsite is found in the areas with high rainfall, whereas calcite and montmorillonite are found in the driest areas, where little leaching occurs.

A series of weathering environments may exist within one profile. Figure 3.1 shows schematically the possible mineralogical components and environmental conditions with depth in various climatic zones.

VEGETATION

The role of vegetation in soil formation includes several important factors (Mohr, et al., 1972). The topsoil is protected from excessive desiccation during dry seasons and from erosion during intensive rainfall. It produces organic matter that decomposes to humus and reacts with the constituent minerals in the soil and thereby promotes biological activity within the soil profile.

It has also been appreciated recently that some types of vegetation are extremely effective in removing certain elements from the soil. Some savanna grasses, for example, contain as much as 5 to 7 percent silica which they have absorbed from the minerals in the soil profile. When the grass is burned or dies, the colloidal silica is easily leached or wasted away with surface water.

PARENT MATERIAL

Results of investigations all over the world have shown that red tropical residual soils are found overlying various kinds of rocks. They are found over basic igneous rocks such as basalt, norite, and diabase as well as over acidic rocks such as granite, granulites, gneisses, phyllites, and sericitic schists. Stephens (1946) and Marbut (1932) have reported the occurrence of lateritic soils on shales, sandstones, limestones and other sedimentary rocks and in aluvium in association with a groundwater table.

The effect of the parent rock material is more pronounced at the initial stages of weathering than at the later stages when the other conditions of weathering predominate. Soils over various sedimentary and igneous rocks in Puerto Rico were studied by Lohnes et al., (1971) who concluded that pedologic factors outweigh the geologic parent material in determining the properties of the residual soils.

TOPOGRAPHY AND DRAINAGE

Topography has an important effect on the rate of weathering by largely controlling the amount of water that moves through the weathering zone and the quantity deflected as run-off. The slope and quantity of run-off determines the amount of erosion. Obviously, vegetation also plays an important role. In the mountainous regions of eastern Africa, where vegetation is generally scant or shallow-rooted, erosion is rapid and deep profiles have not formed. On the other hand, along the Serra do Mar, the coastal mountains of Brazil, which have a heavy vegetative cover and high rainfall, very deep soil profiles have formed.

Tropical Soil Development

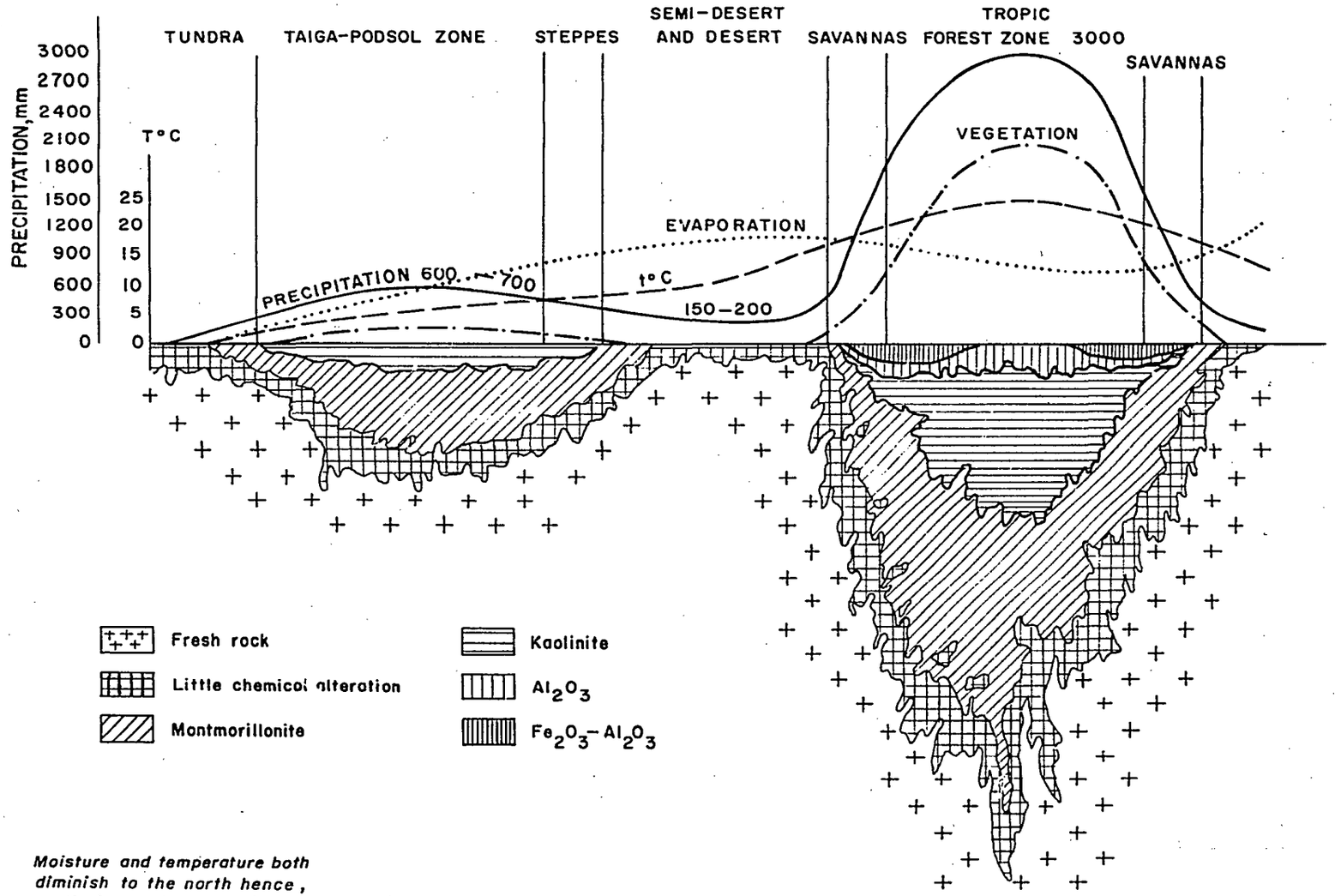
Loughnan (1969) has recently described the chemical weathering of various rock types. The basic crystalline rocks, such as basalt, gabbros, dolerites, and metamorphic rocks with low silica contents contain the most easily weatherable minerals such as olivine and pyroxene and calcic plagioclase. Olivine may weather to serpentine or directly to mortmorillonite. Pyroxene initially weathers to chlorite, calcite or montmorillonite and then to kaolinite and ferric oxides. The accessory minerals, magnetite and ilmenite form hematite, maghemite and goethite.

Granites, granite gneisses, rhyolites and other acid crystalline rocks are composed of quartz and the potash and soda feldspars with accessory micas and amphiboles. The feldspars are more resistant to weathering than the more basic feldspars but eventually they weather to kaolinite or halloysite. Quartz is extremely resistant to leaching and does not convert to any other mineral form. It is slowly dissolved. The amphiboles probably weather in the same way as the pyroxenes. The micas first form vermiculite or montmorillonite and then weather further to kaolinite and gibbsite.

Argillaceous sediments include all sedimentary rocks containing clay minerals such as shales, claystones, clayey sands and sandstones, and such low-grade metamorphic rocks as argillites. The most common clay mineral in these rock types is illite. Montmorillonites are not usually common constituents of sedimentary rocks with the exception of certain shales and volcanic sediments.

The type of soil that develops under tropical conditions, as well as under temperate conditions is essentially a reflection of its mineralogical composition. If the soil has a high clay-size fraction, then the composition of this fraction largely determines the soil properties. Of the various clay minerals that form, montmorillonite and kaolinite or halloysite are the most important in tropical soils. Other clay minerals which are often reported include illite, chlorite and mica, but usually in subordinate amounts.

Kaolinite and halloysite are two-layer clay minerals (see Table 3.1). These are the most stable of the clay minerals, although halloysite readily alters to meta-



Moisture and temperature both diminish to the north hence, weathering is slow, organic matter decomposition is slow, and either the slight precipitation (polar desert) or permafrost (tundra) inhibit leaching of mobile constituents

High moisture promotes high leaching, relatively low temperature results in relatively low rate of organic matter decomposition

High temperature aids chemical weathering, but low moisture inhibits vegetation organic-matter buildup, and leaching of mobile constituents

High moisture and temperature result in rapid weathering and high rate of leaching of mobile constituents

FIGURE 3.1 — SKETCH OF FORMATION OF WEATHERING MANTLE IN AREAS THAT ARE TECTONICALLY

(after Strakhou, 1967 Comments on Soil Orders after Birkeland, 1974)

halloysite, and with time to the more organized structural form of kaolinite. The other three-layer clay minerals are less stable and can weather through loss of silica to kaolinite. For this reason these are called the "weatherable minerals" when soil profiles are discussed.

Kaolinite can revert to gibbsite with continued leaching and loss of silica. However, this reaction has been shown to be completely reversible and gibbsite can readily absorb silica to revert to kaolinite.

The parent material must provide the constituent elements from which the clay minerals are formed. Alumina must be present, for example, before kaolinite can result. The parent rock may be a dominant factor for a protracted period of time or may be only transitory significance (Keller, 1956) but the influence of the parent material is usually short-lived under the weathering conditions that prevail in the warm, humid climate common in the tropics.

Ross and Hendricks (1945) and Kerr (1955) have concluded that montmorillonite forms in an alkaline environment where there is adequate supply of magnesium, silica and alumina. Kaolinite forms under oxidizing conditions in extremely wet, warm climates,

> "by insuring rapid weathering and oxidizing conditions, probably favor formation of kaolinite and lateritic types of soils from parent materials, that, under other conditions, might have montmorillonite-type clays." (Ross and Hendricks, 1945, p. 60).

Hosking (1940) also found that montmorillonite forms from basic rocks in an alkaline environment. Granite and acidic rocks do not form montmorillonite or black clay soils. Hardon and Favejee (1939) found similar environmental conditions required for montmorillonite formation in Java.

The kind of alkali or alkaline earth is also considered important. Rocks that do not contain alkalines can only produce kaolin-type clay minerals or lateritic weathering products (Grim, 1968). The transformation of montmorillonite to kaolinite has been well documented. Morin and Parry (1971) found montmorillonite forms first over the basalts of the Ethiopian plateau, but this mineral is altering to kaolinite when the annual rainfall exceeds 1,000 mm (40 in) and where drainage is unimpeded, as illustrated in electron micrographs, Figures 3.2 and 3.3, in which hexagonal plates of kaolinite can be seen forming from the fluffy montmorillonite. Similar findings have been reported in South Africa (Van der Merwe and Heysteck, 1955). Ferguson (1954) believes that the black montmorillonite soils of Australia alter to red kaolinite soils through loss of silica and alkali, and iron. This change from montmorillonite to kaolinite has been noted by others.

Keller (1964) believes that many field occurrences of the crystalline clay mineral halloysite, the amorphous clay mineral allophane and the clay sized aluminum mineral gibbsite attest to a transformation series. Precipitated hydrous alumina may crystallize to gibbsite which in turn may be transformed to allophane. When silica becomes available, under wetting and drying conditions, it reorients to form a silicate sheet, wich coordinates with the gibbsite sheet and results in a kaolin-type material. Keller believes that halloysite may go on to kaolinite. Humbert (1948) states that as alkalinity is reduced, gibbsite is transformed to kaolinite in the presence of silica. Harrison (1934) found in British Guiana (now Guyana) that gibbsite forms over basic and intermediate rocks and he observed gibbsite forming directly from plagioclase feldspar whereas acidic rocks in that locale form more or less quartziferous and impure kaolins.

Bates, (1960, 1962) in his studies of halloysite and gibbsite formation in Hawaii found that gibbsite is produced by: (1) removal of silica from halloysite, (2) dehydration of Al – gel, and (3) precipitation from groundwater solution. Gibbsite can form directly from feldspar, but he considers halloysite an intermediate crystal form in Hawaii. The lack of kaolinite he attributed to the absence of fine-grained mica as an intermediate weathering product. Consequently, feldspar → potassium mica → kaolinite is the weathering sequence. Bates distinguished three broad mineral groups (1962, p. 318):

> "(1) dry area where leaching has not yet removed all bases are characterized by the presence of montmorillonite-group clay materials; (2) areas sufficiently wet to permit leaching of bases but not all silica will have abundant halloysite and allophane; and (3) areas continually wet or alternately wet and dry for long enough periods of time to allow removal of silica will consist either of hydrated alumina and aluminum-iron gel material, or laterite and gibbsite if dry periods have been involved. Considerable overlap is, of course, expected and found owing to the abundance of halloysite and allophane in area 1 and continous gradation from 1 to 3; but only an unusual set of circumstances would produce gibbsite in area 1 or montmorillonite in area 3".

Aomine and Wada (1962) have found hydrated halloysite formed over volcanic ash at Mt. Aso, Japan. They advance the following weathering sequence:
volcanic glass → allophane → hydrated halloysite.

Sherman (1952) concluded that not only the amount of rainfall but also the intensity and seasonal distribution of rainfall controlled the intensity of the weathering process. Also the length of time that the materials have been subjected to weathering is important. The weathering cycle he derived is reproduced in Figure 3.4. The kaolinite shown is actually reported as kaolin-like clays and may therefore include halloysite. Sherman also determined the progressive formation of clay development. These are reproduced in Figures 3.5 and 3.6. The range in rainfall in which montmorillonite, kaolinite and oxide form is shown as well as the percentage of clay that develops. Montmorillonite forms first and persists only where rainfall is relatively light.

Bates (1962) lists five factors which affect the rate and amount of weathering in Hawaii: (1) age, or length of exposure, (2) composition of soils and (3) texture of the rocks; (4) amount and seasonal distribution of rainfall; and (5) the nature of the intermediate and final products of weathering. This is given in Figure 3.7. Plagioclase is seen to weather to halloysite and gibbsite; glass to allophane and gibbsite or Fe-oxides; and olivine to montmorillonite and Fe-oxides.

These findings do not conflict with Sherman since Bates traces the weathering of the component minerals of the parent basalt. Referring again to Goldich's stability series, it is apparent that olivine will weather first and consequently will provide the first clay mineral formed, as Sherman has indicated. However, Bates does not show the

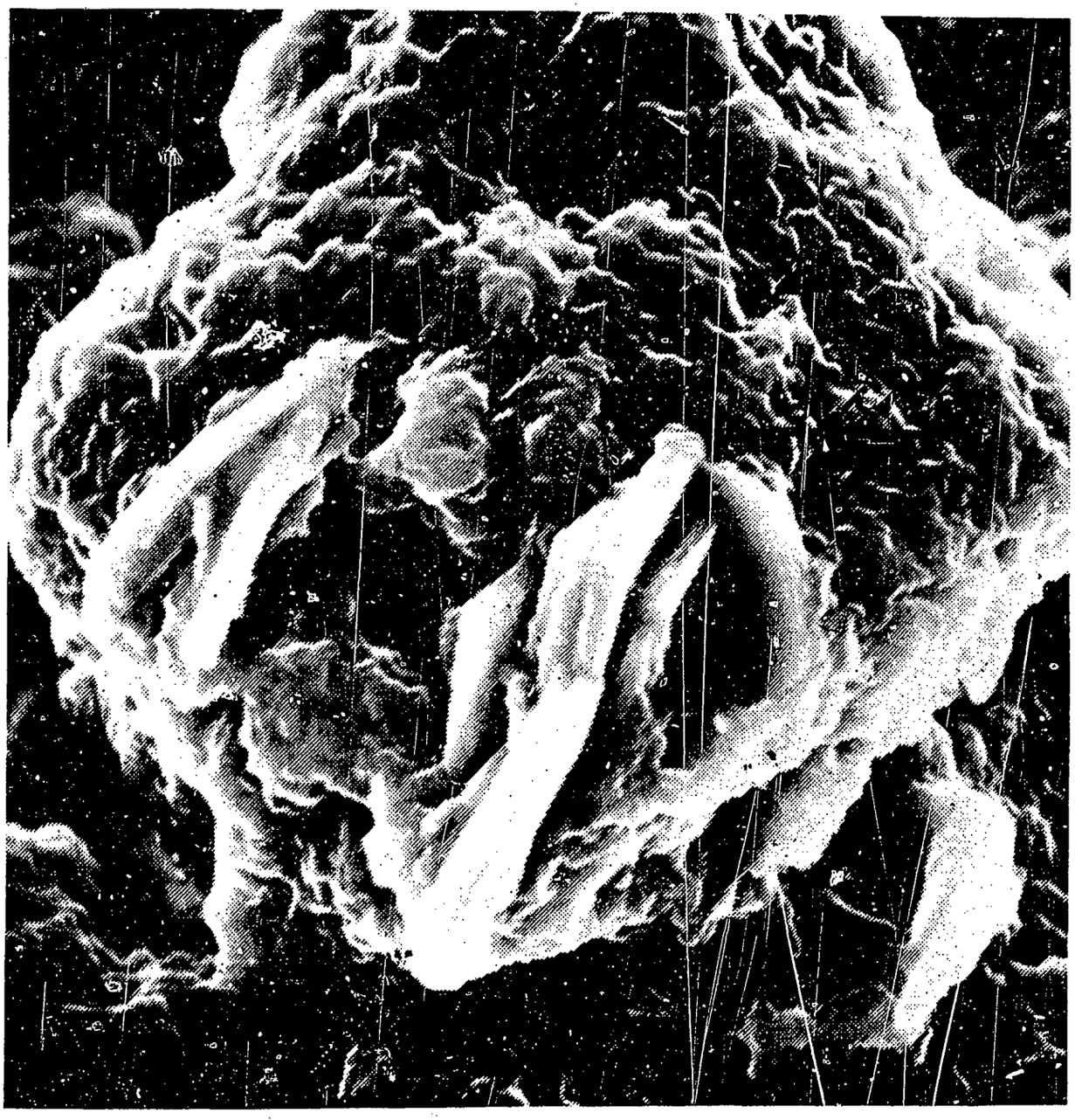


FIGURE 3.2 – ELETRON MICROGRAPH OF SOIL FROM ETHIOPIA (after Morin and Parry, 1971)

FIGURE 3.3 – ELETRON MICROGRAPH OF A SOIL FROM ETHIOPIA (after Morin and Parry, 1971)

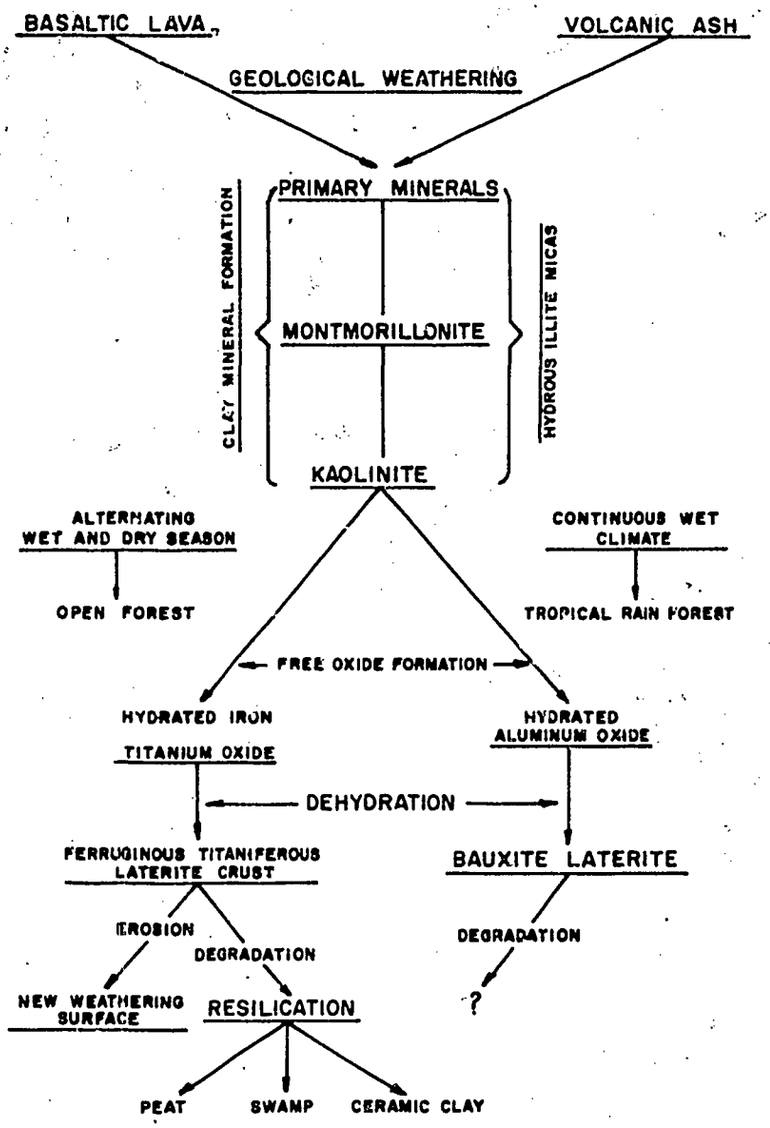


FIGURE 3.4 – OUTLINE OF WEATHERING IN THE TROPICAL SOILS OF THE HAWAIIAN ISLANDS SHOWING PRIMARY, INTERMEDIATE, AND END PRODUCTS OF PEDOLOGICAL WEATHERING

(after Sherman, 1952)

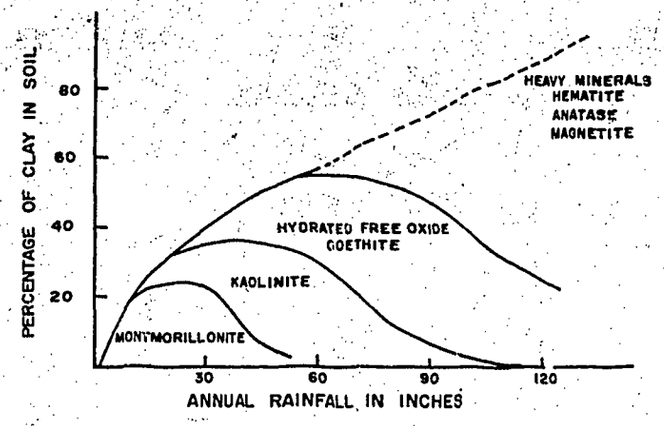


FIGURE 3.5 – PROGRESSIVE TYPES OF CLAY DEVELOPMENT IN HAWAIIAN SOILS UNDER A CLIMATE HAVING ALTERNATING WET AND DRY SEASONS

(after Sherman, 1952)

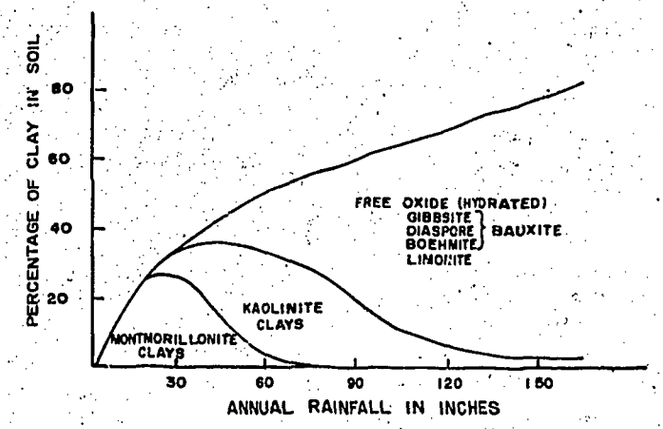


FIGURE 3.6 – PROGRESSIVE TYPES OF CLAY DEVELOPMENT IN HAWAIIAN SOILS UNDER CONTINUOUS WET CLIMATE (after Sherman, 1952)

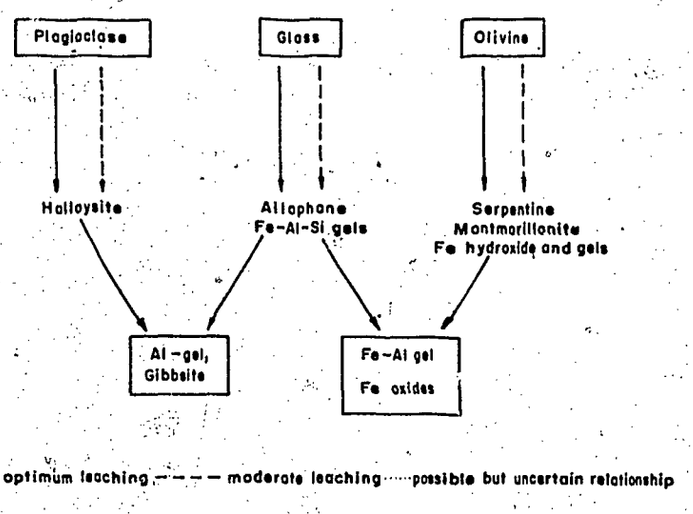


FIGURE 3.7 – TRANSFORMATIONS DURING WEATHERING
(after Bates, 1962)

transformation of montmorillonite to kaolinite as does Sherman.

The following points summarize the general findings reviewed:

1. basic rocks → montmorillonite → kaolinite-type minerals → gibbsite, etc., or goethite, etc., or both.
2. acid rocks → kaolinite-type minerals → as in 1.
3. Fe-rich or Al-rich (and in combination) soils may form indurated crusts as they approach the end products of weathering and silica is largely removed. Harder crusts result with time and added crystallinity.
4. Degree of weathering is controlled by time, characteristics of parent rock, amount and pattern of rainfall, and drainage conditions.

SOIL PROFILE AND SOIL IDENTIFICATION PROPERTIES

Soil Profile

A soil profile is a vertical cross-section of all the soil horizons from the surface down to the fresh, unaltered bedrock. Four or five general horizons are recognized: O,A,B,C, and D. The O horizon, often considered the Ao horizon, is the surface layer of organic debris. The A horizon, the layer underlying the O horizon, is the leached or eluviated layer from which material is removed. This horizon is usually less than 30 cm (12 in) in thickness. The B horizon is the zone of illuviation or the zone in which the dissolved material is deposited. This horizon is usually less than 1 m (3.3 ft) in temperate regions but in humid tropical regions the thickness can be several meters or even tens of meters. The C horizon is the weathered rock (saprolite) or parent material. Finally, the D horizon is the underlying, fresh unaltered bedrock. A hypothetical profile is shown in Figure 3.8.

The Soil Conservation Service introduced a new terminology in 1960 and in subsequent years in order to allow greater precision in profile description since investigators were using the terms A horizon, B horizon, etc., differently. This is now known as the new Soil Taxonomy but has been called the Seventh Approximation for several years. While greater precision has been achieved, it has been at a price. Laboratory analysis may be required to distinguish the various horizons, a drawback to the field worker. A more serious drawback, perhaps, is the new nomenclature which includes terms which many find unintelligible. For example, surface horizons are broken down into several "epipedons". These terms nevertheless have received acceptance among soil scientists, and with use the logic of the terminology becomes apparent. Birkeland (1974) has combined the new terminology with the traditional. His soil horizon nomenclature, which should be very useful to both geologists and engineers, is given in Table 3.2.

TABLE 3.2
Soil Horizon Nomenclature*
(After Birkeland, 1974)

MASTER HORIZONS

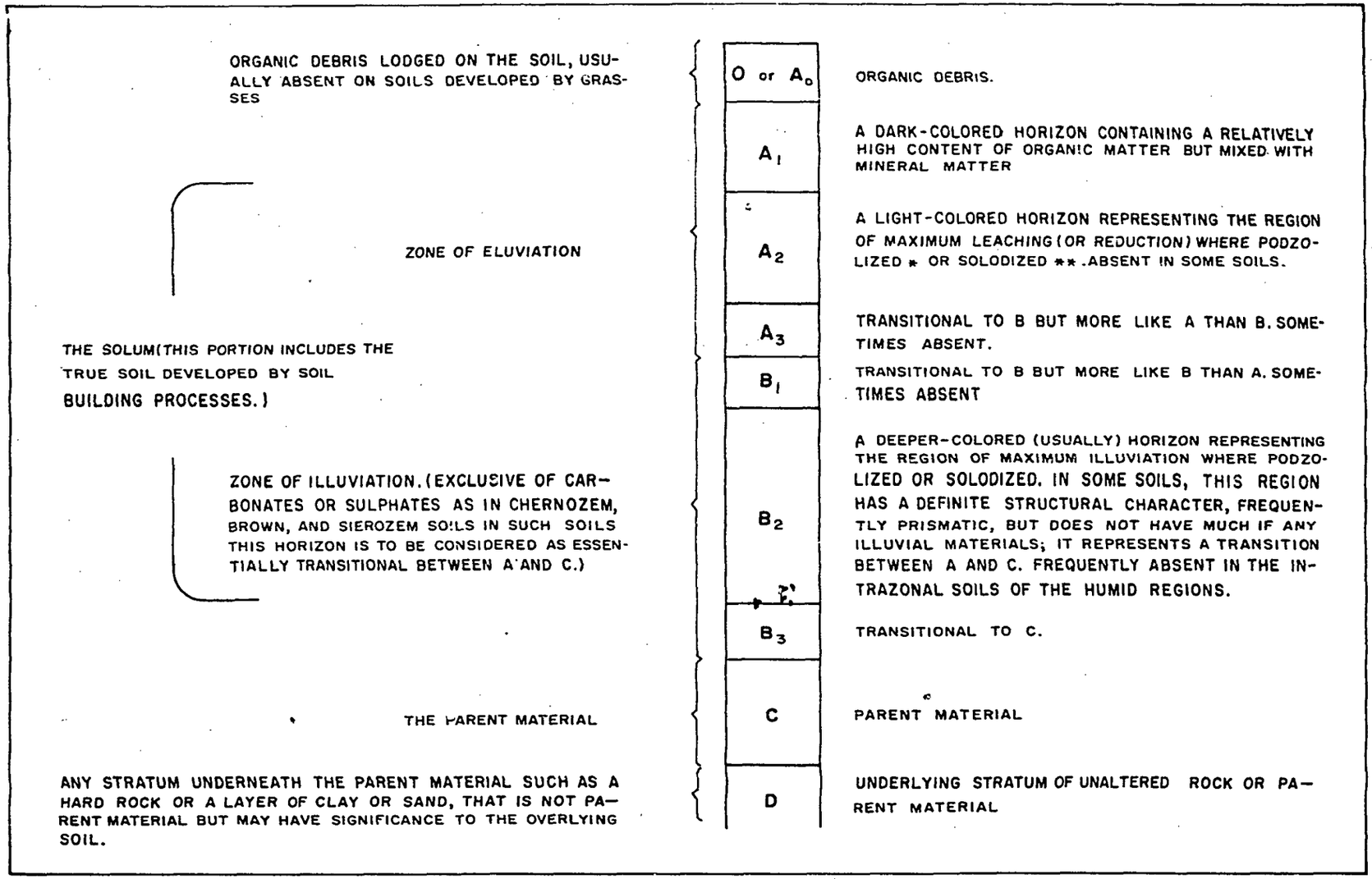
O horizon Surface accumulation of organic material overlying a mineral soil. Lower limits are 30 percent organic matter if the mineral fraction contains more than 50 percent clay or 20 percent organic matter if the mineral fraction has no clay. The horizon is 01 if most vegetative matter is recognizable, 02 if the original form of plant or animal matter is not recognizable.

A horizon Accumulation of humified organic matter mixed with mineral fraction. Occurs at the surface or below an O horizon. Organic matter contents are less than those required for the O horizon. If data are available, the A horizon can be subdivided into the following:
Mollic A horizons are dark colored (chroma of 4.0 or less, value darker than 3.5, when moist), contain at least 1 percent organic matter (0.58 percent organic carbon), and have a base saturation of over 50 percent. Generally associated with grassland vegetation.
Umbric A horizons are similar to mollic A horizons except that the base saturation is less than 50 percent. Generally associated with forest vegetation.
Ochric A horizons are too light in color and low in organic matter to be mollic or umbric A horizons. Generally associated with young soils and/or semi-arid vegetation.

E horizon Underlies an O or A horizon and is characterized by less organic matter and/or less sesquioxides, and/or less clay than the underlying horizon. Horizon is light colored due mainly to the color of primary mineral grains because secondary coatings on grains are absent. Also known as an A2 or albic horizon.

B horizon Underlies an O, A, or E horizon and shows little or no evidence of the original rock structure.
Argillic B horizons have more silicate clay than the A or E horizon and/or the assumed parent material. In other words, silicate clays have been translocated into the B from overlying horizons, or they have formed in place within the B, or both. Clay translocation is recognized in the field by oriented clay films that coat either mineral grains or small channels or ped surfaces. Clay films can be destroyed during subsequent pedogenesis.
Natric B horizons meet the requirements of the argillic B horizons, have columnar or prismatic structure, and more than 15 percent saturation with exchangeable sodium in some subhorizons.
Spodic B horizons generally occur beneath an E horizons and are characterized by a concentration of organic matter and sesquioxides that have been translocated downward from the E horizon.

* In the new classification there are restrictions on some horizons located beneath the A horizon that require a minimum thickness for the horizon in question or a minimum content of various salts. Because many of these restrictions seem to serve little purpose and are inappropriate for many geomorphological studies, they are not used here.



* PROCESS OF WATER LEACHING DOWNWARD THROUGH A AND B HORIZONS

** PROCESS OF ACCUMULATING SURFACE MINERALS THROUGH LEACHING UPWARD, PRODUCED BY EVAPORATION IN AREAS OF LOW RAINFALL CAUSING MOISTURE MOVEMENTS TO BE TOWARD THE SURFACE.

FIGURE 3.8 – A HYPOTHETICAL SOIL PROFILE HAVING ALL THE SOIL HORIZONS

(after PCA Soil Primer)

Table 3.2 (Continued)

Oxic B horizons are highly weathered subsurface horizons that are characterized by hydrated oxides of iron and aluminum, 1:1 lattice clays, and a low cation-exchange capacity. Few primary silicate minerals remain with the exception of quartz, which is quite resistant to weathering.

Cambric B horizons lie in the position of the B horizon and are characterized by at least enough pedogenic alteration to eradicate most rock structure, form some soil structure, and remove or redistribute primary carbonate. Their color has higher chromas or redder hues than does the color of the underlying horizons. Similar horizons can be found in other soil profiles, for example, below argillic B horizons; these horizons do not qualify as cambic B horizons, however, because they are not in the position of the B horizon. Although there is some difficulty in identifying the cambic B horizon, it is a useful term for those B horizons that are characterized primarily by the development of soil structure and/or fairly intense oxidation.

K horizon A subsurface horizon so impregnated with carbonate that its morphology is determined by the carbonate. Authigenic carbonate coats or engulfs all primary grains in a continous medium and makes up 50 percent or more by volume of the horizon. The uppermost part of the horizon commonly is laminated. If cemented, the horizon corresponds with some caliches and calcretes.

C horizon A subsurface horizon, excluding bedrock, like or unlike material from which the soil formed or is presumed to have formed. Lacks properties of A and B horizons, but includes weathering as shown by mineral oxidation, accumulation of silica, carbonates, or more soluble salts, and gleying.

R horizon Consolidated bedrock underlying soil.

SELECTED SUBORDINATE DEPARTURES

The following symbols are used with the master horizon designation to denote special features. They follow the master horizon designation, as well as any numbers (e.g. B2t).

b Buried soil horizon. May be deeply buried and not affected by subsequent pedogenesis or shallow and part of a younger soil profile.

ca Accumulation of carbonates of alkaline earths, usually calcium, in amounts greater than the parent material is presumed to have had. Occurs in A, B, C, or R horizons.

cs Accumulation of gypsum in amounts greater than the parent material is presumed to have had. Occurs in A, B, C, or R horizons.

g Horizon is characterized by strong gleying or reduction of iron, so that colors approach neutral, with or without mottles. Occurs in A, B, or C horizons.

h Illuvial concentration of humus, appearing as coatings on grains of as silt-size pellets.

ir Illuvial concentration or iron, appearing as coatings on grains or as silt-size pellets. A spodic B horizon characterized by both illuvial humus and iron is designated Birh.

m Strong irreversible cementation, for example by accumulation of iron, calcium carbonate, or silica.

sa Accumulation of salts more soluble than gypsum, in amounts greater than the parent material is presumed to have had. Occurs in A, B, C or R horizons.

si Cementation by silica, as nodules or as a continous medium. If cementation is continous the notation sim is used. Such horizons are also known as duripans or silcrete.

t Accumulation of translocated clay, such as in an argillic B horizon.

x Denotes subsurface horizon characterized by a bulk density greater than that of the overlying soil, hard to very hard consistence, and seemingly cemented when dry.

ox, n,
In many unconsolidated Quaternary deposits, the C horizon consists of an oxidized C overlying a seemingly unweathered C. The oxidized C does not meet the requirements of the cambic B horizon. In stratigraphic work it is important to differentiate between these two kinds of C horizons. It is suggested that Cox be used for oxidized C horizons, and Cn for unweathered C horizons.

cn An accumulation of concretions, usually of iron or manganese and iron.

v Denotes spherically shapped voids or vesicles in ochric A horizons in desert regions.

Most tropical profiles display a more or less continuous line of angular or subangular rock fragments known as the stone line. This stone line separates the residual soil below from the transported material above. The overlying material is usual colluvium which has moved down-slope due to gravity and rainwash.

Identification Properties

COLOR

Color is one of the most obvious soil properties but, must be used with caution. The color varies with the moisture content and as a consequence the moisture content must always be mentioned when describing color. The color of mottled clays must be described at the natural or in-situ moisture content and without any undue manipulation which would tend to blend the colors.

Color may be a clue in indicating the presence of certain elements or compounds. Dark brown, dark grey and black colors usually indicate organic matter, except in tropical residual black clays. They have very low organic contents, usually 2 percent or less and the color is due to the manner in which the finely divided organic matter is held by the clay minerals, particularly montmorillonite. Red, yellow and some brown colors are often the results of chemical weathering. Deep red shades indicate iron oxides whereas lighter shades of yellow and yellowish brown indicate hydrated iron oxides. Yellow colors can also be

due to hydrated aluminum oxides, to a mixture of aluminum and iron oxides, to allophane-rich soils and even to some organic compounds. Grey, dark grey and mottled colors generally indicate poor drainage; light greys are due to leaching.

Color has three characteristics (Hunt, 1972). They are hue (the color of the spectrum), value (the amount of light reflected) and chroma (the purity of the color). The Munsell system of color identification is the one most widely used.*

TEXTURE

Texture, which is one of the most important properties of a soil profile, commonly refers to the particle size distribution. It also refers to the particle shape and arrangement. The first is determined by mechanical analysis whereas the others require special examination, which, for the finer sizes, requires a magnifying lens or microscope.

STRUCTURE

Hunt (1972) provides descriptions of eight principal kinds of soil structure, some of which are borrowed from the U.S. Department of Agriculture Soil Survey Manual, (Soil Survey Staff, 1951). Individual aggregates, which are called peds, are classified according to shape. These terms, which are given in Table 3.3 are widely used and should be useful for field descriptions.

TABLE 3.3
Types of Soils Structure
(After Hunt, 1972)

1. Loose single grains without cohesiveness.
2. Crumb: small, soft, porous aggregates.
3. Platy: particles or partings arranged in planes, generally at or near the ground surface and parallel to it: plates generally less than 10 mm (0,4 in) thick.
4. Prismatic or columnar: particles or partings arranged in columns; partings planar or curved; columns generally 2 to 5 mm (0.08 to 0.2 in) in diameter and normal to the surface. In many kinds of ground the prisms or columns end upward at the base of a layer having platey structure.
5. Blocky: aggregated, closely packed clumps, commonly 2 to 3 mm (0.08 to 0.12 in) in diameter and roughly equidimensional but having very irregular surface, corners round or angular.
6. Granular: similar to 4 but aggregates small, mostly less than 5 mm (0.2 in).
7. Nodular: like 4 and 5 but nodules not closely packed and commonly widely spaced; nodules generally differ in composition from the matrix.
8. Tubular: irregular tubelike filling; cast of foreign material filling former cavities such as those developed from decaying roots or burrows of rodents or insects.
9. Massive, or structureless.

* Available from the Munsell Color Company, Inc.; 2441 Calvert Street, Baltimore, Maryland.

ORGANIC MATTER

The organic content affects the physical properties of the soil profile. The water-holding capacity is increased with organic matter. The soil consistency is often affected. Weathering is enhanced by the organic acids that are produced.

Organic matter in the soil profile is composed of a variety of material which ranges from decomposing plant and animal matter to humus. The carbon content is generally used as a measure of the organic content of the soil; the organic content is considered to be 1.724 times the organic carbon content. The carbon to nitrogen ratio, C:N, indicates the amount of decomposition that has occurred in the original organic material (Birkeland, 1974).

CATION EXCHANGE CAPACITY AND BASE SATURATION

The clay minerals and colloidal materials usually carry an electrical charge on their surface to which various anions and cations are attracted. These ions are held in an exchangeable state and may be replaced by other anions or cations. The total negative charge on the clay mineral surface is termed the cation exchange capacity. The amount of this charge depends on the clay mineral or the colloidal material and also on the exchangeable ion. The cation exchange capacities of the common clay minerals and other materials are given in Table 3.4.

TABLE 3.4
Cation Exchange Capacities for the Clay Minerals and Other Materials
(After Grim, 1968 and Birkeland, 1974)

Material	Cation exchange capacity (me/100 g)
kaolinite	3–15
metahalloysite ($2H_2O$)	5–15
hydrated halloysite ($4H_2O$)	40–50
illite	10–40
montmorillonite	80–150
chlorite	10–40
vermiculite	100–150
allophane	25–70
hydrous oxides of aluminum and iron	4
organic matter	150–500

The exchange capacities are given in milli-equivalents per 100 grams of dry soil. The exchangeable cations that are among the most common in clay materials are Ca^{++}, Mg^{++}, H^+, K^+, NH_4^+, Na^+. The common anions or bases include SO_4^{--}, Cl^-, PO_4^{---}, and NO_3^- (Grim, 1968). The percent of base ions that make up the total exchangeable cations is known as base saturation (Birkeland, 1974).

TROPICAL RED RESIDUAL SOILS

Red tropical soils which have been described in the literature are reviewed in this section. The review is limited to red (or yellowish-red, brownish-red) residual soils, including the so-called lateritic soils. The black (or dark gray, dark brown) clay soils, often referred to as "black cotton" soils are reviewed in a subsequent section. The red soils are by far the most widespread in the tropics. The black clay soils, while also common, are generally restricted geographically.

Background of the Terms

Laterite and lateritic soils must be included in any discussion of tropical soils, particularly red tropical soils, in spite of the fact that these terms do not have generally accepted definitions. The word "laterite" is usually attributed to Buchanan (1807) who used it to describe the brick-red soils of southern India that are soft enough to cut into blocks with crude instruments, and which harden on exposure. The word is derived from the Latin *latericius*, meaning brick. The type locality is Angadipuram, India and Stephens (1961) described the material as a mottled, vermicular and indurated horizon in a red soil formed over gneiss.

A great many modifications to the term have developed through the years, of which only the most significant will be discussed here. Bauer (1898) was the first to establish the chemical character of laterite, low silica content and high alumina and iron oxide contents (Sivarajasingham et al., 1962). The work of Holland (1903) revealed that deposits in different tropical regions varied from those rich in iron with little alumina to those rich in alumina with little iron and including many intermediate combinations. Femor (1911) also adopted this chemical criteria although he originally considered restricting the use to only soft materials that harden on exposure. Maignien (1966) summarizes the work of Walther (1916, 1915, 1889) who considered the term laterite as referring to the red brick-like color and consequently extended the definition to include all red alluvial and eluvial products, or more specifically, all tropical red earths. The chemical criteria were again adopted by Martin and Doyne (1927, 1930), among others. They proposed the silica-alumina ratio $\frac{SiO_2}{Al_2O_3}$ of the clay fraction as the criterion with true laterite having a ratio of under 1.33 and lateritic soil having ratios between 1.33 and 2.00, above 2.00, the soil is considered non-lateritic. This was later broadened by Joachim and Kandiah (1941) to the silica-sesquioxide ratio $SiO_2/Al_2O_3 + Fe_2O_3$. These ratios still were popularly adopted and are usually considered as the chemical criteria. Pendleton (1936) however, objected to the adoption of these ratios, claiming they are only indicative and not absolute criteria. He advocated returning to Buchanan's original concept of induration.

More recent attempts have not resulted in an accepted definition. Indeed, popular usage has include such a diversity of materials that many writers believe the term is no longer descriptive. The principal reason for its proliferation is that many disciplines have an interest in these materials. This was borne out during the Specialty Session on the Engineering Properties of Lateritic Soils during the Seventh International Conference on Soil Mechanics and Foundation Engineering held in Mexico City in 1969. Some participants favored a general definition while others favored one restricting all but indurated and concretionary varieties and those capable of indurating on exposure to wetting and drying.

Recent Pedological Terms

Kellog's Laterite and Latosols; Great Soil Groups:

Recent pedological definitions are more restrictive. Alexander and Cady (1962, p. 1) define laterite as:

> "a highly weathered material rich in secondary oxides of iron, aluminum or both. It is nearly void of bases and primary silicates, but it may contain large amounts of quartz and kaolinite. It is either hard or capable of hardening on exposure to wetting and drying. Except hardening, all of these characteristics are possessed by some soils or soil-forming materials. In the past the term "laterite" has been used in many ways, but it is now generally accepted as a name for a material whose composition and properties are within the limits described."

An earlier definition was proposed by Kellog (1949) who confined the word to (1) soft mottled clays that change irreversibly to hard pans and crusts when dried; (2) cellular and mottled hard pans and crusts; (3) concretions and concretion-rich horizons which still may be unconsolidated; and (4) consolidated concretions.

The term "latosol" was proposed by Kellog (1949) to cover those soils excluded by the definition above, such as "yellowish-brown lateritic soil", "reddish-brown lateritic soils", and "red lateritic soil". They comprise all the zonal soils having the characteristic properties associated with low silica sesquioxide ratios of the clay fraction, such as low base exchange capacities, low activities of the clay, low content of most primary minerals, low content of soluble constituents, a high degree of aggregate stability and usually some red color. The excluded soils were then termed "yellowish-brown latosol", "reddish-brown latosol", and "red latosol" respectively.

These terms received fairly wide acceptance among pedologists. Many countries in South America and Southeast Asia adopted these terms and they are found on many pedological maps of these areas.

D'HOORE'S CLASSIFICATION

The French pedological classification was adopted by D'Hoore for his compilation of the Soil Map of Africa (1964). This system was followed by Lyon Associates, Inc. et al. (1971) in their engineering study of African tropical soils, as the terms correlated reasonably well with the physical properties. The D'Hoore map is still the only regional pedological map available for Africa. Accordingly, these terms will be described in some detail here.

The D'Hoore terminology contains three units for the red tropical soils: ferruginous soils, ferrallitic soils and ferrisols. Ferruginous soils generally have ABC profiles which frequently show a marked separation of free iron oxides either leached out of the profile or precipitated within the profile in the form of spots or concretions. The reserve of weatherable minerals is often appreciable although the silt/clay ratio (20/2 microns) is generally above 0.15*. Kaolinite is the predominant clay mineral, but there may be small amounts of 2:1 lattice clays. Gibbsite is generally absent. The SiO_2/Al_2O_3 ratio is near 2 or somewhat higher, but the $SiO_2/Al_2O_3 + Fe_2O_3$ ratio is below 2. Ferruginous soils are found in the lower rainfall areas, generally under 1,830 mm (72 in) per year in areas with pronounced dry seasons. They show lower clay contents and Atterberg limits and higher compacted densities and CBR values then the other two groups. Ferruginous soil profiles are seldom thicker than 250 cm (8.2 ft) and the colors are not as bright as the ferrallitic soils.

Ferrallitic soils are often deep with only slightly diffentiated horizons. The silt/clay ratio is generally less than 0.25 in the B and C horizons. Clay minerals are principally the 1:1 lattice type and often occur with large amounts of iron oxides. Hydrated oxides of aluminum generally occur although the crystalline form gibbsite is not an essential constituent. The SiO_2/Al_2O_3 ratio is sometimes near 2 but usually below. Ferrallitic soils occur in the more humid areas with over 1,500 mm (60 in) of rain per year and supporting dense vegetation.

Ferrisols have profiles closely resembling those of ferrallitic soils. The reserve of weatherable minerals is generally low but may exceed 10 percent of the 50–250 micron fraction. The silt-clay ratio is generally above 0.20 over igneous rocks and metamorphic rocks. The clay fraction consists almost entirely of kaolinite, free iron oxides and amorphous gels sometimes with amounts of 2:1 lattice clays and gibbsite. The SiO_2/Al_2O_3 ratio is near 2, or slightly below 2. Ferrisols differ from the other soil groups in that surface erosion prevents normal profile development and forces the profile to develop at a lower depth in the less weathered parent materials. The ferrisols occur in intermediate to high rainfall areas, between 1,250 and 2,750 mm (50 and 108 in) per year.

THE NEW SOIL TAXONOMY

The Soil Conservation Service of the U.S. Department of Agriculture introduced a comprehensive soil classification system in 1960, which until recently has been called the Seventh Approximation. Supplements were issued in 1967, 1968, and 1970. Details on these are provided by Philipson et al. (1973). Some of the new terms for horizons were discussed earlier. Not all terms of the New Soil Taxonomy will be discussed here as the nomenclature is complex; only those that may apply to tropical conditions will be defined.

The terms laterite, lateritic soil and latosol are abandoned. Plinthite is the new term that replaces laterite. This is defined as a material that indurates irreversibly on exposure to wetting and drying. This has the advantage of eliminating the ambiguity which occurs with using the term laterite.

The term "oxisol" replaces most applications of lateritic soils and latosol. This term is defined as containing an oxic horizon, or one that is highly weathered, with hydrated oxides of iron or aluminum or both and with variable amounts of 1:1 lattice clays and quartz. The profiles are usually over 30 cm (12 in) thick, contain more than 15 percent clay and commonly have a horizon of ironstone nodules or concretions. These soils display low clay activity, high permeability and low erodibility. These characteristics are usually limited to soils of very old stable surfaces (Philipson et al., 1973).

Ten main soil orders are recognized in the New Soil Taxonomy. These are:

1. Entisols – Soils without layering or diagnostic horizons and not containing plinthite or plinthitic gravel in the upper 30 cm (12 in).
2. Vertisols – Soils with over 30 percent clay which is usually expansive and at least 50 cm (20 in) deep.
3. Inceptisols– Immature soil with various surface horizons and including a cambic horizon but no oxic, spodic, argillic or natric horizon exposed and no plinthite in the upper 30 cm (12 in).
4. Aridosols – Soils having low content of organic matter in the A horizon and salt or silica accumulations at depth.
5. Mollisols – Soils with a dark, organic-rich surface horizon and without an oxic, spodic, argillic or natric horizon and no plinthite in the upper 30 cm (12 in). These are soils of grassland areas found mainly in temperate regions.
6. Spodosols – Soils with a spodic horizon in the upper 2 m (6.6 ft); organic matter and sesquioxides may be concentrated; these are mainly coarse-textured soils which occur in humid areas.
7. Alfisols – Soils with an argillic or natric horizon of clay accumulation; no spodic horizon and no plinthite or plinthic gravel in upper 30 cm (12 in) but minerals have weathered to kaolinite clays and free sesquioxides.
8. Ultisols – Soils more mature than alfisols and less mature than oxisols; have an argillic

* There appears to be a correlation between silt content and weatherable minerals in tropical soils. For example, 0.25 is generally considered the maximum value for latosols above that figure the soils are less mature and probably not sufficiently laterized to be considered "lateritic" or "latosol".

horizon; no spodic horizon; no plinthite in the upper 30 cm (12 in).

9. Oxisols – Soils with an oxic horizon within the upper 2 m (6.6 ft) or with plinthite in the upper 30 cm (12 in). These have no spodic or argillic horizon over the oxic horizon.

10. Histosols – Organic soils at least 40 cm (16 in) thick, usually found in swamps and bogs.

Each soil order is further subdivided into suborders. The suborder names are meant to be descriptive of common properties with the final syllable indicating the soil order. For example: aquents denote wet soils of entisols order; plinthic tropadults indicate an ultisol (order) of the trop (continually warm climate) and ud (soil moisture regime has less than 3 months dry period) suborders while plinthic indicates a plinthic horizon. Selected suborder nomenclature is given in Table 3.5.

TABLE 3.5

Selected Suborder Nomenclature for the New Soil Taxonomy (after Birkeland, 1974 and Philipson et al., 1973)

Prefix	Diagnostic Property of Suborder	Suffix	Order
acr	extreme weathering; used only with oxisols	ent	Entisol
alb	presence of E horizon	ept	Inceptisol
and	pyroclastic materials	id	Aridosol
aqu	wetness; temporary saturation	oll	Mollisol
arenic	sandy surface, 50-100 cm (20-40 in) thick	od	Spodosol
		alf	Alfisol
arg	presence of argillic B horizon	ulf	Ultisol
dystr	relates to lower base saturation	ox	Oxisol
eutr	relates to higher base saturation	ert	Vertisol
ferr	high iron content; or iron-cemented nodules in argillic horizon	ist	Histosol
fluv	recent fluvial deposits		
gibbs	cemented gravel-size aggregates with 30 percent or more gibbsite		
hum	high content of organic matter		
hydr	thixotropic in some horizon; may have clays irreversibly dehydrate to silt and sand size aggregations; low bulk density and high water content.		
ochr	presence of an ochric A horizon		
orth	common horizon development		
pale	older soil of stable surfaces; may have plinthite horizon		
petroferric	hard plinthite crust within upper 1 m (3.3 ft)		
plinth	plinthite continuous or more than 50% of soil matrix in upper 1.25 m (4.1 ft); plinthic used when more than 5% plinthite by volume in upper 1.5 m (5.0 ft)		
psamn	sandy texture below 25 cm (10 in)		
quartz	sand fraction has 95% or more quartz or insoluble mineral; used with "psamn"		
trop	associated with continually warm climates		
ud	soil moisture regime dry less than 90 days per year; humid climates		
ust	associated with wet-dry climates		
vitr	large amounts of volcanic ash; used with "and"		

The properties of several suborders important to the tropics have been summarized by the Committee of Tropical Soils of the National Research Council (National Academy of Sciences, 1972). These are summarized below.

The alfisol suborders include three that are found in the tropics. Aqualfs are alfisols in saturated regions that were formely called low-humic gley soils. Udalfs are brownish soils that are exposed to short dry periods which were known before as gray-brown podzolic soils. Ustalfs occur in warm areas with long dry seasons. They are red or reddish brown in color and correspond to the ferruginous soils of the French system or the reddish-brown soil of the Great Soil Groups. Other important suborders include paleudalfs, paleustalfs and rhodustalfs.

Four ultisol suborders are described here. Aquults are ultisols of wet areas that were formerly called humic-gley soils. Humults are ultisols with large accumulations of organic matter; these were formerly called humic latosols. Udults are found in humid tropical areas where the soil is dry for only short periods during the year. These were a part of the red yellow podzol great soil groups. Ustaults are ultisols of warm areas with long dry periods. These also were formerly considered red-yellow podzols.

Ultisols may have a plinthic horizon in soils which are associated with mid-Pleistocene or older surfaces. The other suborders are younger.

The following suborders of oxisols are found on very old erosional remnants or associated with existing ground water tables. Aquox are oxisols which occur in areas with high water table and were formerly known as ground water laterite. Humox are oxisols with a large amount of organic carbon, equivalent to the humic latosols. Orthox occur in warm humid areas with short dry seasons. They correspond to ferrallitic soils of the French system and also to red latosols of the Great Soil Group.

A comparison of the former Great Soil Group Classification System and the approximate equivalent orders and suborders of the new classification is given in Table 3.6.

THE FAO-UNESCO COMPROMISE

In May, 1961 a joint project of FAO (Food and Agriculture Organization of the United Nations) and UNESCO (United Nations Educational, Scientific and Cultural Organization) was organized to prepare a Soil Map of the World. In the ensuing years several meetings of soil scientists were held to establish a terminology to be adopted for the maps. These scientists were from several countries which represented several classification systems. The resulting classification is a compromise which in some respects makes it less desireable than the French system. On the other hand these terms will be used on all the sheets of the Soil Map of the World. To date only Volume IV, South America (2 sheets at 1:5 000 000) have been published but other Volumes should be available in the near future.

TABLE 3.6

Former U.S. Soil Classification and the Approximate Equivalent Orders and Suborders of the New Soil Taxonomy (after Birkeland, 1974)

Former U.S. Classification				The New Classification, 7th Approximation	
Order		Suborder	Great Soil Groups	Suborder	Order
ZONAL	Pedalfer	Soils of forested warm-temperate and tropical regions	Red-Brown Laterite Red-Yellow Podzolic Low-Humic Latosol Latosol Humic Latosol Laterite	Humult, Udalf, Udult Udult Tropept, Ustox Tropept, Humult, Andept, Ustult Humult, Humox, Andept Orthox	Oxisol, Ultisol, Alfisol, Inceptisol
		Soils of cold regions	Polar Desert Artic Brown, Tundra Alpine Turf	Aquept, Ochrept, Umbrept, Andept	Inceptisol
		Soils of forested cool-temperate regions	Podzol Brown Podzolic Gray-Brown Podzolic Gray Wooded Sols Bruns Acides Western Brown Forest	Orthod, Humod Orthod, Andept, Ochrept Udalf, Udult Boralf Ochrept, Umbrept	Spodosol, Ultisol, Alfisol, Ince Inceptisol
		Soils of forest-grassland transition	Degraded Chernozem Noncalcic Brown	Boralf, Boroll Xeralf, Ochrept	Alfisol, Mollisol Inceptisol
	Pedocal	Dark-colored soils of semiarid, subhumid, and humid grasslands	Reddish Prairie Prairie (Brunizem) Chernozem Reddish Chestnut Chestnut	Ustoll Udoll, Boroll, Xeroll, Ustoll Boroll, Ustoll, Xeroll Ustalf, Ustoll Xeroll, Ustoll, Boroll	Mollisol, Alfisol
		Light-colored soils of arid regions	Reddish-Brown Brown Sierozem Red Desert Desert Polar Desert	Ustalf, Orthid, Argid Ustoll, Xeroll, Argid Orthid, Boroll Argid, Orthid Argid, Orthid Argid, Orthid Argid, Orthid	Aridisol, Mollisol, Alfisol
INTRAZONAL		Hydromorphic soils in areas of imperfect drainage of high water table	Humic Gley Low-Humic Gley Alpine Meadow Bog Half Bog Planosol Ground-Water Podzol Ground-Water Laterite	Aquoll, Aquept, Aquult, Aqualf Aquult, Aquent, Aquept, Aqualf Aquod, Aquoll, Umbrept Suborders of Histosol Aquept, Aquoll, Aqualf Aqualf, Alboll Aquod Aquult, Udult, Usult	Inceptisol, Mollisol, Alfisol, Spodosol, Ultisol, Entisol
		Halomorphic soils (saline and alkali) in areas of imperfect drainage in arid and coastal areas	Solonchak Solonetz Soloth	Orthid, Aquept Natric great groups of Alfisol, Mollisol, Arid Natric subgroups of Mollisol and Alfisol	Inceptisol Aridisol, Mollisol, Alfisol
		Calcimorphic soils formed from calcareous parent materials	Brown forest Rendzina	Ochrept, Xeroll, Udoll Rendoll	Inceptisol, Mollisol
AZONAL			Lithosol Regosol Alluvial		Entisol, Inceptisol, Mollisol

There are 25 soil units, but many have limited distribution. Traditional pedological terms such as chernozems, podzols, planosols, solonetz, solonchak, rendzinas, rhegosols and lithosols have been retained. Recent terms which have become popular are also retained, such as vertisols, rankers, andosols and ferralsols. As with the Seventh Approximation, the terms laterite and lateritic soils have been eliminated.

Definitions are given below only for those units that represent the red tropical soils, the black clays and the volcanic soils of the tropics. These definitions are taken from World Resource Reports 33 and 37 (Dudal, 1968 and 1969).

1. Arenosols – Soils which have oxic, argillic or cambic horizons; coarser than sandy loam and with clay contents of 18 percent or less. (Not all sandy soils are arenosols, however). Albic arenosols are sands composed predominantly of quartz.
2. Andosols – Soils formed from materials rich in volcanic extrusives, especially glass and having a very dark surface horizon. These are common soils in volcanic mountainous regions; generally the bulk density is low; moisture content is high and there may be considerable amorphous material.
3. Vertisols – Black clay soils and black, gray or brown heavy-textured soils that shrink and swell with changes in moisture content.
4. Planosols – Soils formed in flat areas with poor drainage.
5. Cambisols – Soils in which changes in color, structure and consistency have occurred due to weathering of the soil profile.
6. Luvisol – Soils with an accumulation of clay in the B horizon. Includes some former red-brown mediterranean soils, brown mediterranean soils and reddish-brown lateritic soils but ferric luvisols and rhodic luvisols are red to deep red, the former corresponding to the ferruginous soils of the French system. Plinthic luvisols contain a plinthic horizon within 1.25 m (4.1 ft) of the surface.
7. Acrisols – Very acid soils; soils with illuvial accumulation of clay in the B horizon; certain types (plinthic acrisols) have a plinthic horizon within 125 cm (49 in) of the surface.
8. Nitosols – Soils with an oxic horizon, shiny ped surfaces and some illuvial clay in the B horizon; soils with an oxic horizon overlying a deep argillic B horizon. Nitosols include the ferrisols and some depleted ferrallitic soils of the French system as well as many of the reddish-brown lateritic soils of South America. These soils may be deeply weathered but have not reached the degree of maturity in weathering as have the ferralsols.
9. Ferralsols – Soils containing a high sesquioxide content in the clay fraction. May be largely made up of kaolinite and contain silica. A plinthite horizon may occur in the profile. This term replaces most applications of "lateritic soil" and includes those iron-rich materials which may harden on exposure to wetting and drying; soils with concretionary or nodular horizons of ironstone; soils with indurated plinthite; and, generally, red or yellow tropical residual soils.

Most tropical red soils of South America are ferralsols. Arenosols, acrisols and luvisols are also common. Cambisols and nitosols are geographically restricted.

An approximate correlation of the FAO with the U.S. new Soil Taxonomy and the French classification system is given in Table 3.7.

Recommended Terminology for Engineers

Pedological maps are of value to soils engineers and geologists as well as agricultural scientists. Such maps are available in many areas. The Great Soil Groups classification has been used for many of these, at least in South America and Southeast Asia. The French system has been used in Africa. The new Soil Taxonomy has also been adopted in some localities. The summary definitions above should be of some assistance in utilizing these maps.

The Soil Map of the World now under preparation, has introduced the compromise terminology. It is expected that these terms and maps will be widely used. These terms should be adopted wherever possible. If a soil is named a ferralsol, no other distinguishing term is needed. The term plinthite is also recommended for soils which harden on exposure to repeated wetting and drying. Lateritic gravel should be called concretionary gravel. Hardened crusts should be termed ironstone.

The old term "lateritic soil" may be necessary when pedological maps and details are unavailable. Under these circumstances, both "laterite" and "lateritic soils" may be the most suitable terms. The geological definition of these terms should be adopted for such usage. The American Geological Institute (1972) defines laterite as:

> "A highly weathered, red subsoil or material rich in secondary oxides of iron, aluminum, or both, nearly void of bases and primary silicates, and maybe containing large amounts of quartz and kaolinite. It develops in a tropical or forested warm to temperate climate, and is a residual or end product of weathering. Laterite is capable of hardening after a treatment of wetting and drying. . ."

TABLE 3.7

Approximate Correlation of the FAO Classification System with those of the U.S. Soil Taxonomy and the French Classification System (modified from National Academy of Sciences, 1972)

FAO	U.S. Soil Taxonomy	French Classification
FLUVISOLS	Fluvents	Juvenile soils on recent alluvium and colluvium
REGOSOLOS	Psamments Orthents	Juvenile soils on recent eolian deposits and weakly developed soils
ARENOSOLS Ferralic A.	Oxic Quartzi-psamments	Ferrallitic soils on loose sandy sediments
GLEYSOLS Eutric G. Dystric G. Humic G. Plinthic G.	 Tropaquepts Humaquepts Plinthaquepts	Mineral hydromorphic soils Hydromorphic soils with an accumulation of iron or a plinthite horizon.
ANDOSOLS	Andepts	Eutropic brown soils of tropical regions on volcanic ash
PLANOSOLS		
CAMBISOLS Dystric C.	Dystropepts	Ferrallitic soils, rejuvenated;
Eutric C.	Eutropepts	Ferruginous or ferrallitic soils, rejuvenated;
Humic C.	Humitropepts	Ferrallitic soils, humic, rejuvenated
LUVISOLS	Tropudalfs Paleudalfs	Ferruginous tropical soils
ACRISOLS	Paleustalfs	Yellowish-brown ferrallitic soils
Rodic A.	Rhodudults Rhodudults	Ferrallitic soils
FERRASOLS	Oxisols	
LITHOSOLS	Lithic subgroups	Lithosols and lithic soils
NITOSOLS (some cambisols)	Udafs (?)	Ferrisols
VERTISOL	Vertisols	Vertisols

The Geological definition of lateritic soil is (AGI 1972):

> "A soil containing laterite; also any reddish tropical soil developed from much weathering."

The last term is highly generalized but it is not always possible to be more precise in the field.

Characteristics and Occurrences

Various red tropical soils display a wide variation in morphological, chemical and mineralogical characteristics. This is true for the indurated deposits of plinthite as well as for the unconsolidated soils.

Ironstones vary from friable to those which are difficult to break with a hammer. The degree of hardness is apparently dependent upon the sesquioxide content, especially the iron content. In addition, the age and the structural arrangement of the constituent materials or their crystallinity are significant factors (Maignien, 1966). The structure is either continuous or discontinuous. Grant (1974) states that the initial bonding of the ironstones in Australia is due to hydrated aluminum oxides. Hematite forms a secondary bond as the ironstone ages. Du Preez (1949) lists the following bonding forms: vesicular, concretionary, cellular, vermicular, slaglike, pisolitic and massive, concrete-like.

The most common cyrstalline minerals are hematite, goethite, gibbsite and boehmite. The amorphous minerals limonite and cliachite are also common; allophane is common is certain soils. Kaolinite is usually the predominant clay mineral and is followed by halloysite. Illite is fairly common. Montmorillonite and vemiculite are only infrequently identified. They are restricted to less mature soils. Quartz which is frequently reported is mostly derived from underlying acidic rock.

Thicknesses of the crusts vary from a few centimeters to well over 9 meters (30 ft). Most indurated or concretionary horizons, however, are less than 3 m (10 ft) and many are less than 1.5 m (5 ft). Lateritic soils have developed thicknesses of up to 20 m (66 ft) (Maignien, 1966).

Chemically, plinthic materials are composed of a high percentage of sesquioxides. In most occurrences iron predominates. Selected chemical analyses are given in Table 3.8.

The criteria of the silica-alumina and silica-sesquioxide ratios bears further examination. Van der Merwe and Heysteck (1952) have analyzed a number of South African soils and calculated both ratios in an attempt to classify them as either laterite or lateritic soils. A few of these are given in Table 3.9. The soils are predominantly clayey and no indurated crusts occur, although the gray and reddish-brown ferruginous lateritic soil, as termed by the authors, include extensive concretions. They consider the high SiO_2 - Al_2O_3 ratio for the gray ferruginous lateritic soil is due to quartz and mica but the presence of gibbsite indicates that weathering of the clay minerals had occurred to free alumina. Vine (1949) and Hardy and Rodrigues (1939) believe the high sesquioxide contents are limited to particular parent rocks. Castagnol (1952) reports chemical analyses for a laterite crust and for a red soil above and below the crust. Curiously the silica-sesquioxide ratio averages 1.41 above the crust, 1.49 below the crust while the crust itself averages 2.00.

It appears that each ratio only indicates the degree of weathering. Silica in the form of quartz and mica may persist in certain profiles where the parent rock is acidic even though the soil has been sufficiently leached to be classified as plinthite.

TABLE 3.8

Chemical Composition of Selected Laterites and Lateritic Soils

Location	Guinea (1)	Grenada (2)	Grenada (2)	Dahomey (3)	Hawaii (4)	South Vietnam (5)	Kenya (6)	Guyana (7)	Brazil
Type of deposit (8)	crust	laterite	red earth	hard laterite	concretionary horizon	ferruginous crust	hydrated lithomarge (9)	laterite cap	ferralsol
Parent rock	dunite	andesite	andesite	granite	basalt	basalt	volcanic	gabbro	basalt
SiO_2	2.70%	18.3%	32.8%	31.37%	7.14%	36.1%	35.60%	8.57%	28.91%
Al_2O_3	9.60	36.2	30.4	19.22	5.82	11.16	30.75	30.65	25.07
Fe_2O_3	74.00	23.4	19.3	38.51	56.69	30.80	15.55	40.35	30.20
FeO	–	–	–	–	2.44	–	–	–	–
CaO	tr	1.4	0.3	0.10	-0.00	7.86	–	–	0.07
MgO	0.72	–	2.2	–	0.46	6.16	–	–	0.49
Na_2O	–	–	–	–	tr	0.10	–	–	0.24
K_2O	–	–	–	–	-0.00	2.90	–	–	5.09
TiO_2	0.68	2.5	1.3	1.12	10.64	–	1.40	4.06	–
MnO	–	–	–	–	0.08	–	–	–	–
P_2O_5	–	–	–	–	0.93	–	–	–	–
Cr_2O_3	0.45	–	–	–	–	–	–	–	–
H_2O	12.40	18.0	12.7	–	–	–	–	–	–
Loss on Ignition	–	–	–	9.10	15.86	7.0	15.03	16.43	10.12
	100.55	99.8	99.0	99.52	100.06	102.8	98.33	100.06	99.99

(1) Bonifas (1959)
(2) Hardy and Rodrigues (1939)
(3) Alexander and Cady (1962)
(4) Walker (1964)
(5) Castagnol (1952)
(6) partial chemical analysis by Robertson in Terzaghi (1958)
(7) Bleackley, (1964)
(8) Authors' descriptions
(9) Robertson describes hydrated lithomarge as a material intermediate between parent rock and laterite.

TABLE 3.9

Silica-Sesquioxide Ratios of South African Soils (after Van der Merwe and Heysteck 1952)

Parent Material	Clay (%) Fraction	$SiO_2 - R_2O_3$	$SiO_2 - Al_2O_3$	Authors' Classification
dolomite	68.7	0.70	1.01	laterite
diabase	59.1	1.60	2.22	lateritic red earth
granite	35.7	1.69	1.89	lateritic red earth
sandstone	53.9	1.21	1.72	lateritic red earth
granite	17.7	1.95	2.09	gray ferruginous laterite soil
dolomite	26.2	1.87	3.00	reddish-brown ferruginous laterite soil

The characteristics of the lateritic soils have been well summarized by Sombroek (1966, p.66):

> "The fundamental characteristics . . . . lie in the nature and constitution of the mineral soil mass, indicating a thoroughly weathered soil. It consists of sesquioxides, silicate clay-minerals having a 1:1 lattice, quartz and other minerals that are highly resistant to weathering. Primary silicate minerals with less resistance to weathering are either absent or are present in only a small amount. The same applies to clay-minerals having a 2:1 lattice, and those amorphous gels of Si and Al that have high base exchange capacities. Free aluminum oxides are often present, but not always. The silt content of the samples in the solum is generally low."

FORMATION OF IRONSTONE

There have been many conflicting theories proposed concerning the formation of ironstone and lateritic soils, principally due to the variability in their occurrences and chemical composition.

The hardening process has been investigated by Alexander and Cady (1962) and Alexander, et al. (1956) who have found that the most significant characteristic of hard ironstone is the greater crystallinity of the iron-oxide minerals. The minerals goethite and hematite form a network of crystals which form the indurated framework, and iron enrichment is necessary before material will harden. In addition to enrichment, two processes appear to be involved in hardening. They are crystallization and dehydration.

Ironstone may be formed from rock by different processes of weathering and mineral transformation. All processes involve the removal of almost all bases and much of the combined silica. Sivarajasingham et al. (1962) believe that the various hypotheses on the weathering processes have been influenced by the techniques employed for identification of the minerals. However, many of the initial weathering products were not identified. Evidence has shown that under certain conditions gibbsite is the first crystalline material to form. Under other conditions kaolinite may be the first mineral to form. Gibbsite forms from kaolinite by desilication and the reverse alteration may also occur. A sequence from primary mineral to disorganized clay to kaolinite is not the only possible process. The mass of amorphous materials are the initial products of the weathering processes and these materials become the parent material for the crystalline clay minerals or other crystalline clay materials. This material may lose or gain iron or alumina and be subjected to desilication and possible resilication. Any or all of these events may occur and it is extremely difficult if not impossible to reconstruct the exact course of events.

The formation of the iron mineral goethite also takes place from unidentified material considered to be amorphous ferruginous gels by Bonifas (1959). Hematite may form by dehydration of goethite or by alteration of the primary minerals magnetite, chromite and ilmenite (Maignien, 1966).

Campbell (1917) was the first to suggest that ironstone is formed by the addition of sesquioxides brought in by ground water from other parts of the drainage basin. Lateral enrichment does occur, as at the toe of hills, but this type of development is far less important than residual concentrations.

Harrassowitz (1926, 1930) considered the formation of ironstone to be due to the concentration of iron and aluminum at the surface due to evaporation of capillary water rich in sesquioxides. He believed that such formations could only form in savannas where there are long dry seasons to promote the evaporation.

The work of Marbut (1932) indicated that most ironstones developed at the surface of fluctuating ground water tables. If the zone of fluctuating water tables is deep, no enrichment in sesquioxides occurs because there is insufficient oxygen. Ironstone develops at the top of this horizon after the overlying layers have been eroded and the surface exposed. Crusts of ironstone on dissected plateaus, which no longer have shallow water tables, are common in Africa (Tertiary surfaces) and in South America and are considered to be fossil. This process of formation does not require alternating wet and dry seasons.

Marbut's observations have been confirmed in Asia by Thorp and Baldwin (1940); in Australia by Prescott and Pendleton (1952); in Africa by D'Hoore (1954); and by Mohr et al., (1972), who describe three stages in the formation of ironstone. The first stage is the formation, through weathering of the parent rock, of a freely draining material or soil. The second stage is the appearance and disappearance of the water table which causes alternating, reducing and oxidizing conditions. The formation of ironstone originates during this stage. The final stage occurs when the base level or erosion is approached and drainage can no longer remove the water so that a permanent ground water table is established. During this stage, only reducing conditions prevail. Drying, usually through erosion and exposure, causes dehydration and crystallization of the amorphous iron and aluminum into crystalline oxides and hydroxides.

Tricart (1972) does not agree. His reasoning appears to follow that of Harrassowitz. He states (p. 175):

> "At the present time it is almost universally admitted that they form in climates with a long dry season, even though they are sometimes found under forest, as in Amazonia or in Ivory Coast. But in the forest of Ivory Coast, since a decade or so, Rougerie has patiently demonstrated that they are relict formations, inherited from epochs going back to the Tertiary, during which morphogenic conditions were different from the present. True, in certain particular cases, cuirasses*can be found that are presently forming under forest, especially under semi-evergreen seasonal forest."

If one considers the time required for the formation of ironstone, it is almost certain that several different climates have been operative. Leneuf and Aubert (1960) calculated that 50,000 years would be required for the desilication of granite one meter thick. An oxisol profile 4 meters (13 ft) in thickness and containing ironstone would require about 120,000 years. These numbers are highly theoretical but they do illustrate the period of time required. It is obvious

* A French term for hard plinthite or indurated laterite crust.

that most existing profiles with ironstone have been subjected to several climatic variations since we know such variations have occurred during the Quaternary epoch.

Various writers have stated that when tropical forests and vegetation are cleared, hardening of the soil occurs very rapidly. McNeil (1964) stated that at one location in Brazil, Yata, an agricultural community had to be abandoned after five years because the soil became a virtual pavement of brick. Sombroek also noted considerable hardening in a road cut in Brazil within 2 – 3 years. Other cases have been mentioned in the literature; one may go as far back as Buchanan (1807) in his paper on the building bricks.

Only plinthithes indurate on exposure. These are not sufficiently widespread to cause extensive brick pavements on removal of vegetation despite what has been printed in the literature. It is now believed that only 5% of central Brazil, [illegible] of tropical Africa, and 7% of tropical India contain plinthite at or near the surface that will harden on exposure (Sanchez and Bual, 1975). In Southeast Asia, the area covered is less than 5%.

The propensity to harden on exposure can rarely be advantageously utilized in engineering design. However, the possibility does exist that roadway subgrades may be improved by clearing the alignment of vegetation, exposing the soil to air and occasionally passing a disc harrow or other device to expose the deeper soils. Some success has been claimed for this procedure in Australia.

The statement that laterites, or plinthites harden irreversibly on exposure to repeated wetting and drying has been included by many writers in describing the hardening process. Ironstone may be extremely durable, but decomposition can take place. Alexander et al., (1956) describe cases in West Africa where they observed the breaking up or softening of ironstone crusts. Sivarajasingham, et al., (1962) cite other examples. Mohr et al., (1972) state that reforestation leads to the disappearance of indurated horizons.

OCCURRENCES OF TROPICAL RED SOILS

Red tropical or lateritic soils, with and without concretions or crusts occur throughout the tropical regions of the world and even in some temperate regions where they are products of former climates and weathering conditions. These locations are shown on several maps covering most of the world's tropical regions.

The tropical red soils of South America are shown on Figure 3.9. The FAO-UNESCO compromise terminology has been used on this map, as it was taken directly from the Soil Map of the World, Volume IV. Almost all of the tropics are covered with some type of lateritic soil with the exception of the Andes Ranges and the Subandean Depression in Paraguay and Argentina. Even the intermontane valleys within the Andes contain many areas with red tropical soils. Ferralsols cover the largest area by far, including most of the Amazon Basin. Acrisols and arenosols are also common. The northeast and coastal regions have more complex soil groups and associations.

The tropical soils of Central America are shown in Figure 3.10. Various types of red soils extend throughout the region, from the border of Colombia and Panama into the Yucatan Peninsula of Mexico. Much of this region is or has been recently volcanic and consequently, many of the soils are derived from volcanic ejecta. Andosols are frequently included within the red tropical soils.

There are two soil maps of Africa, Figures 3.11 and 3.12. On the first, the D'Hoore classification is shown whereas the FAO classification is given on the second. Both of these maps, as well as those of the other regions, have been highly generalized. The distribution in eastern Africa is quite complex with associations of two or more of the soil groups contained within one area. For more specific information about such areas, it is necessary to secure copies of more detailed pedological maps.

Most of Central Africa is covered with ferrallitic soils. There is a relatively narrow fringe of ferruginous soils both to the north and south and along the eastern part of the continent. Ferrisols are found in relatively restricted areas in eastern and also in western Africa.

The soils of Southeast Asia are given in Figure 3.13. It represents a compilation of soils information from several sources. The soils of this region are complex due to the interplay of climate, topography and the extensive volcanoes which are a striking feature of the Malay Archipelago. As in Central America, many areas shown with red tropical soil also contain andosols.

The locations of red tropical soils of Australia are given in Figure 3.14. Those for India and Ceylon are given in Figure 3.15. These maps are included in order to provide a complete world-wide reference to continental land areas with tropical red soils.

The locations of ironstone and concretionary gravel deposits are shown in another series of maps. These have been compiled from various sources, including field work in South and Central America and Africa. Again, these maps are highly generalized; the areas shown are believed to include locations where deposits of these materials occur, although not continuously. Only a few locations could be verified in the field.

Figure 3.16 shows those areas with ironstone and concretionary gravel in South America. The major areas are the Amazon Basin, the Guyana Shield and the interior parts of the Brazilian Shield.

Concretionary gravel within the Amazon Basin is commonly found in several physiographic locations (Sombroek, 1966). A fossil concretionary gravel with an underlying horizon of plinthite generally occurs in eastern Amazonia. It is generally found beneath a clay formation (called Beltera clay) and on old erosional surfaces called the Amazon Planalto. The surfaces appear like terraces 100 m (330 ft) or more above the river levels. On the Planalto in western Amazonia the concretionary horizon is apparently less widespread and the underlying plinthite horizon is rarely found. Concretionary gravels and plinthites are very common on the upland savanna of Amapa Territory. Fossil concretionary gravels are common on the Tertiary and Cretaceous surface within the Amazon Basin. The Cretaceous surfaces generally occur at elevations of 400 to 600 m (1300 to 1970 ft) whereas the Tertiary surfaces occur at 250 to 400 m (820 to 1300 ft).

No major deposits of ironstone or concretionary gravels are known in Central America. This is probably due to a more recent geological environment including a geologically recent volcanic cover.

African deposits are outlined in Figure 3.17. The deposits tend to be concentrated in Central Africa and

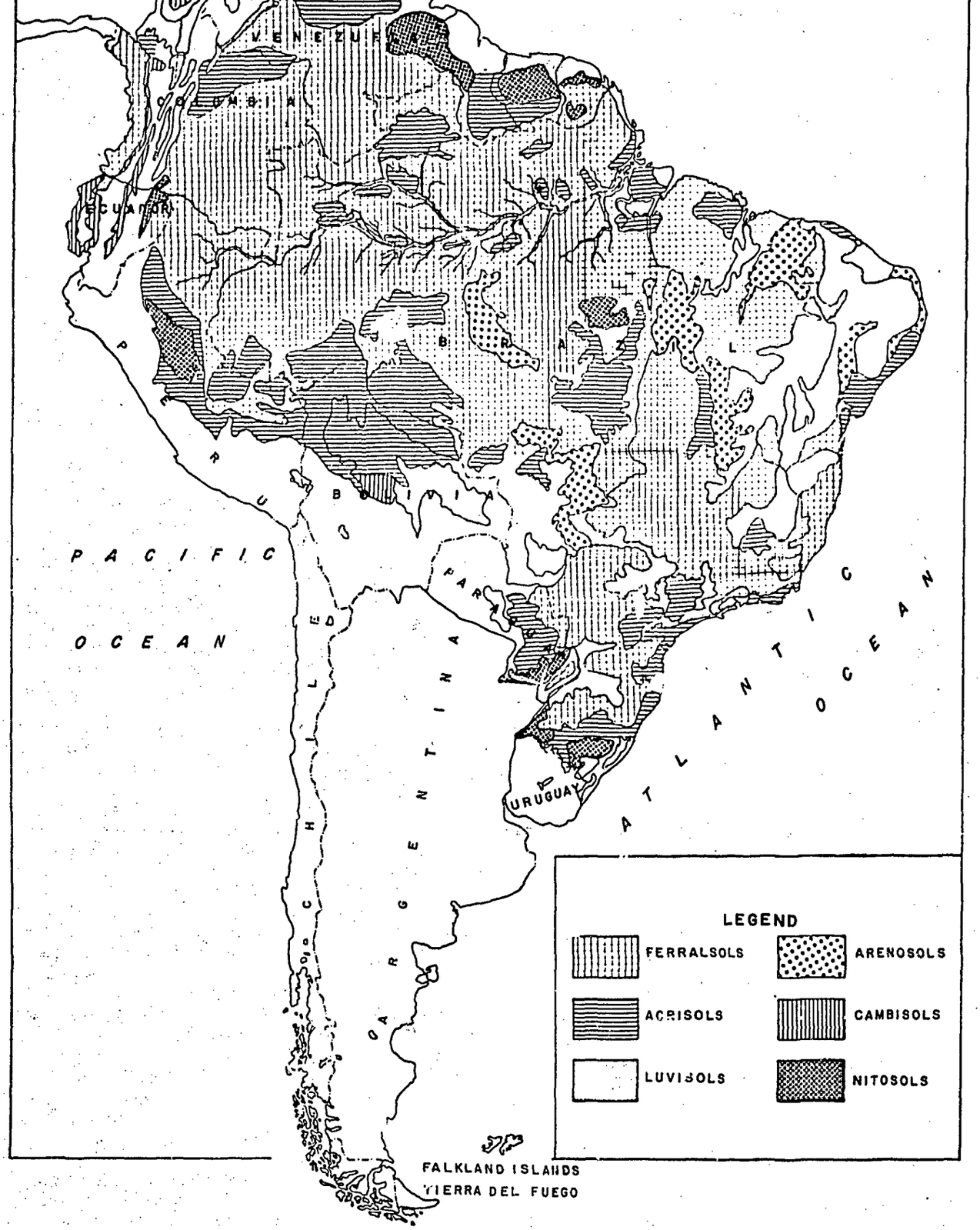


FIGURE 3.9 — RED TROPICAL SOILS OF SOUTH AMERICA

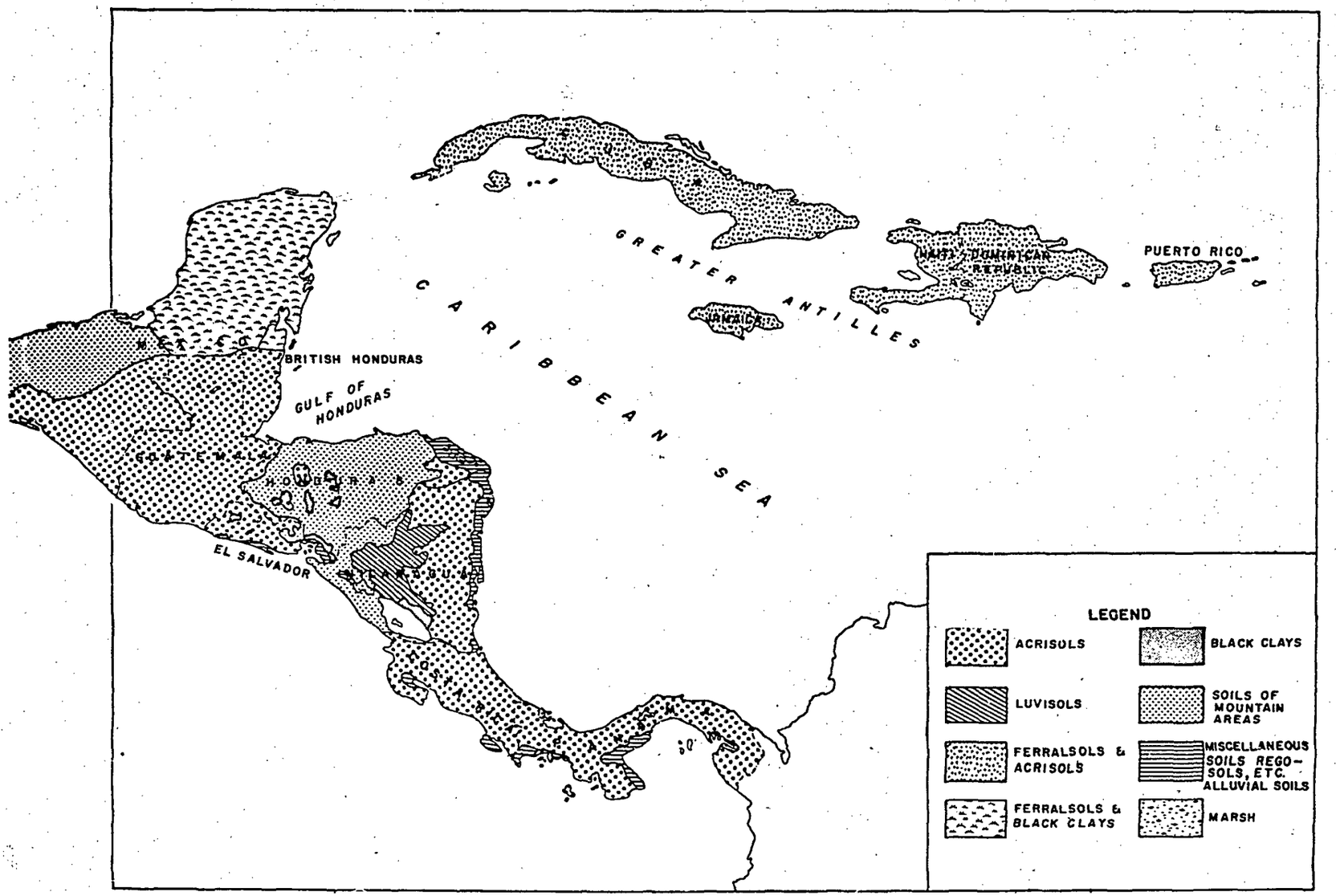


FIGURE 3.10 – RED TROPICAL SOILS OF CENTRAL AMERICA

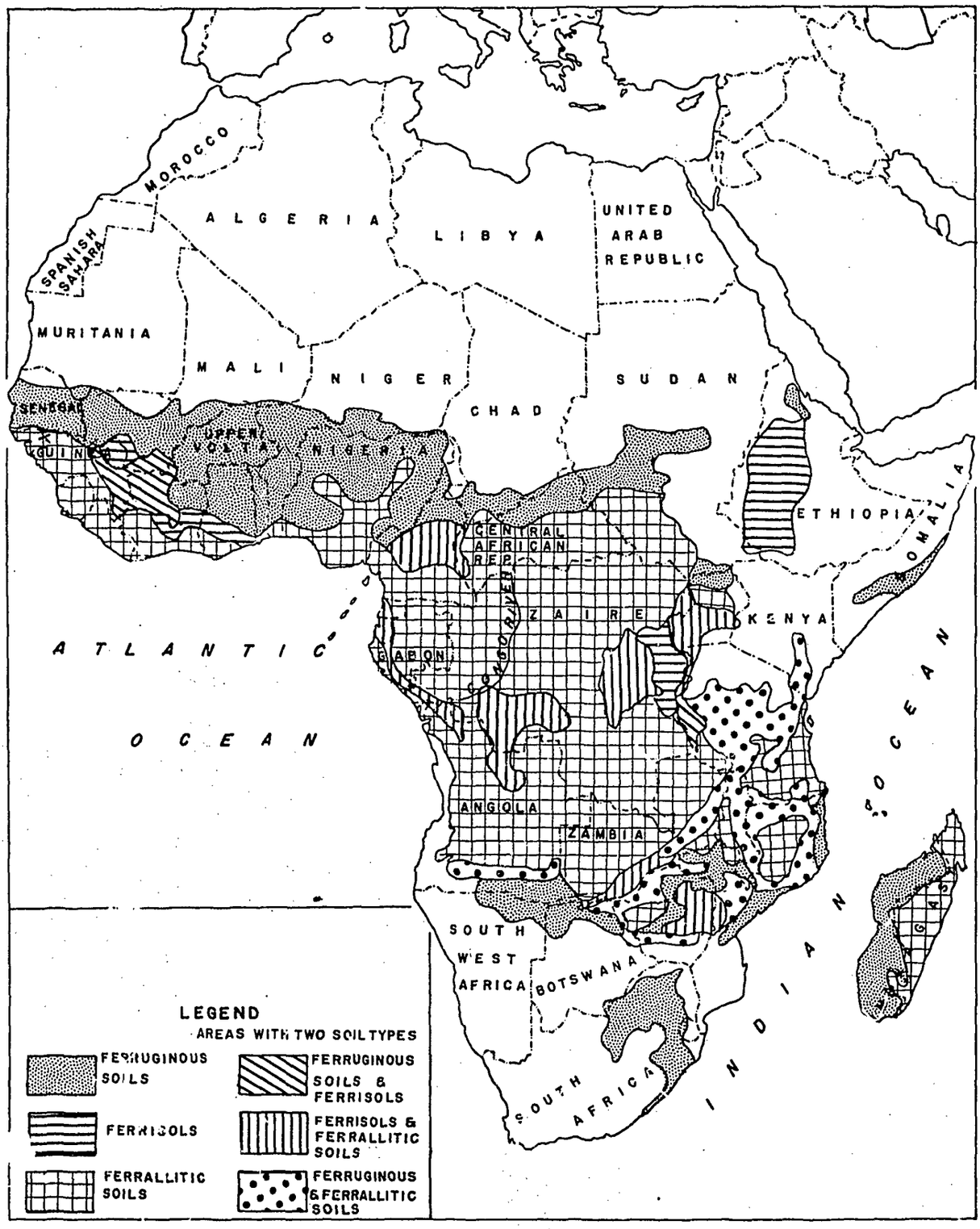


FIGURE 3.11 – RED TROPICAL SOILS OF AFRICA, FRENCH SYSTEM

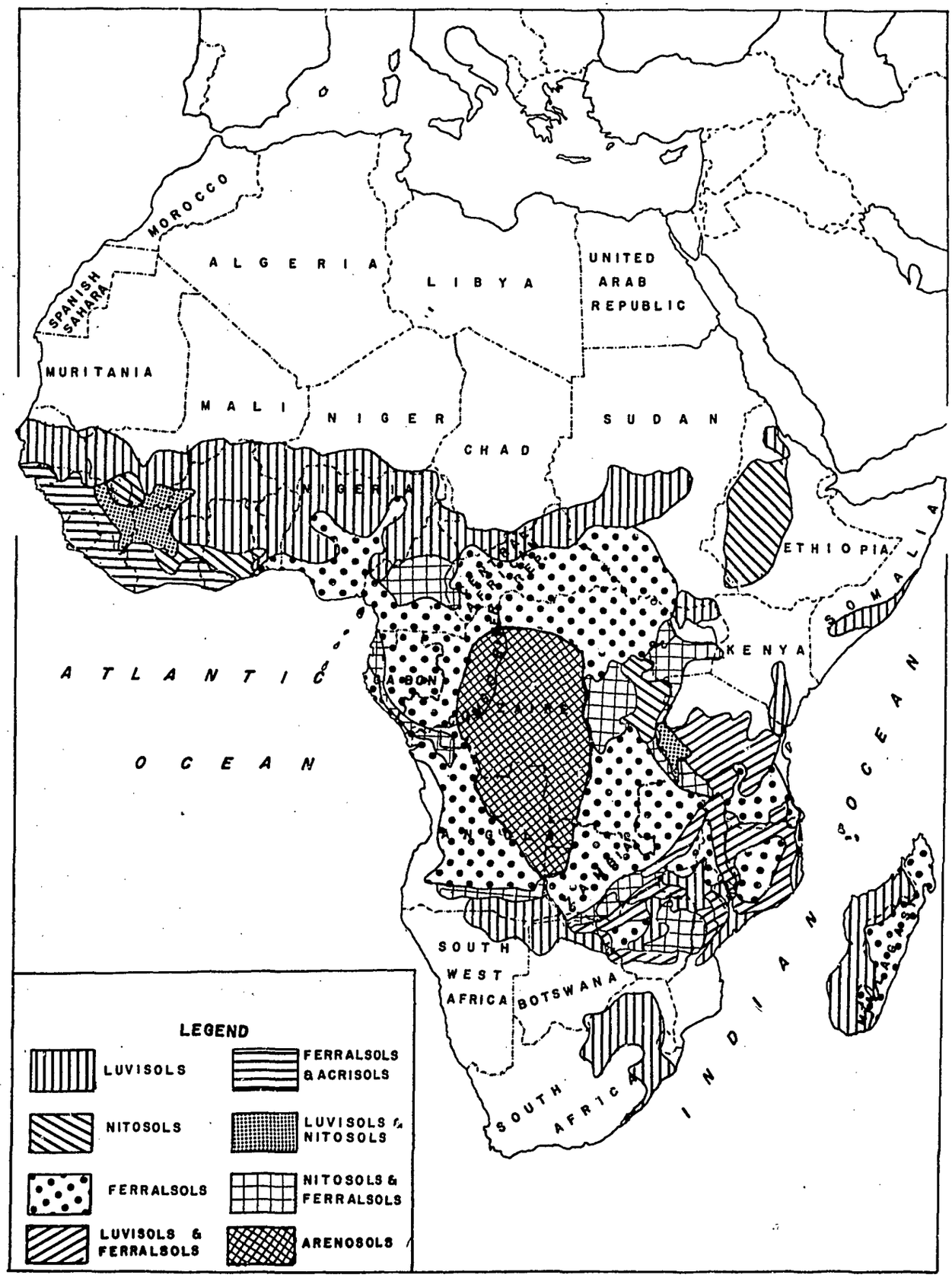


FIGURE 3.12 — RED TROPICAL SOILS OF AFRICA, FAO SYSTEM

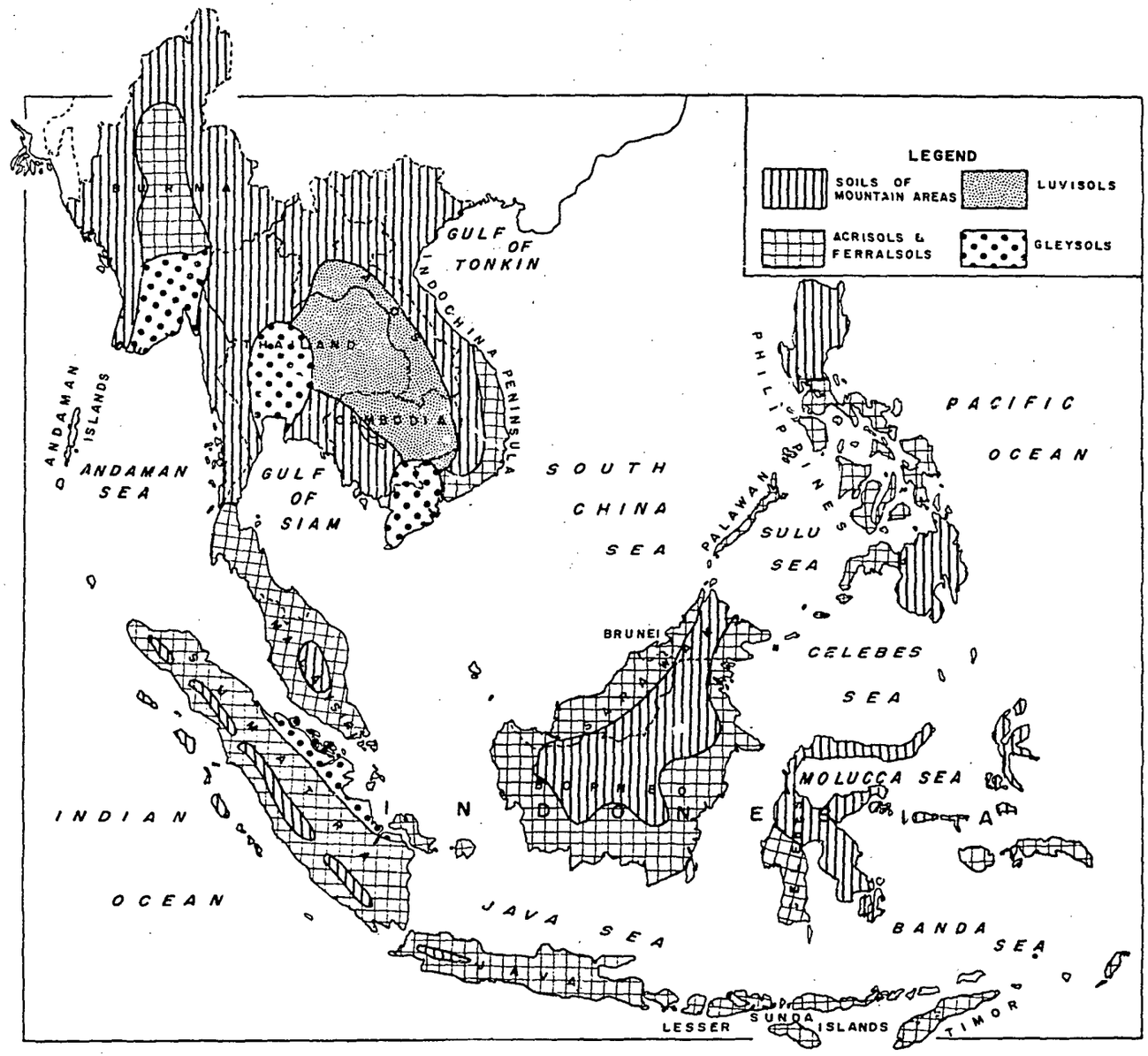


FIGURE 3.13 – RED TROPICAL SOILS OF SOUTHEAST ASIA

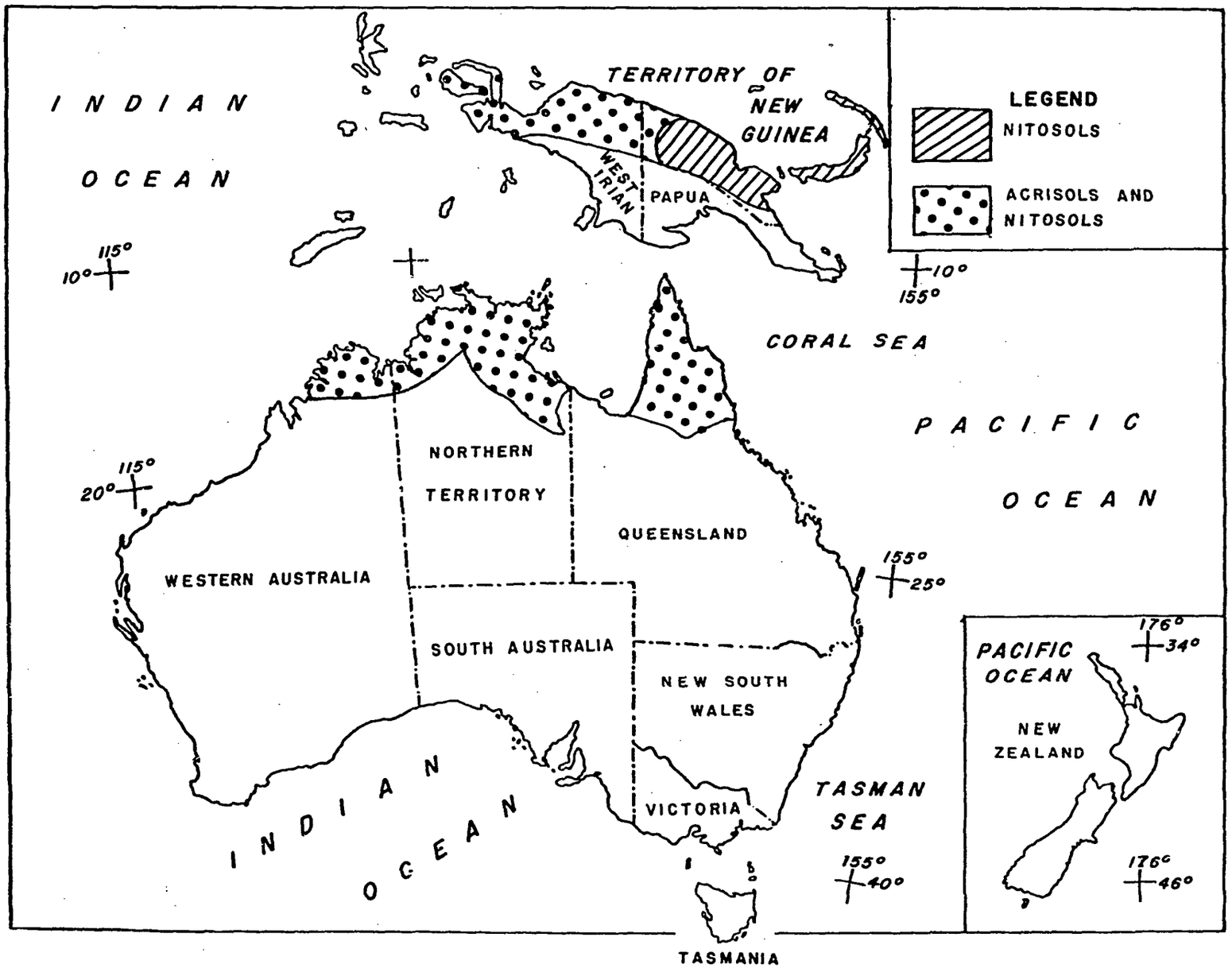


FIGURE 3.14 – RED TROPICAL SOILS OF AUSTRALIA

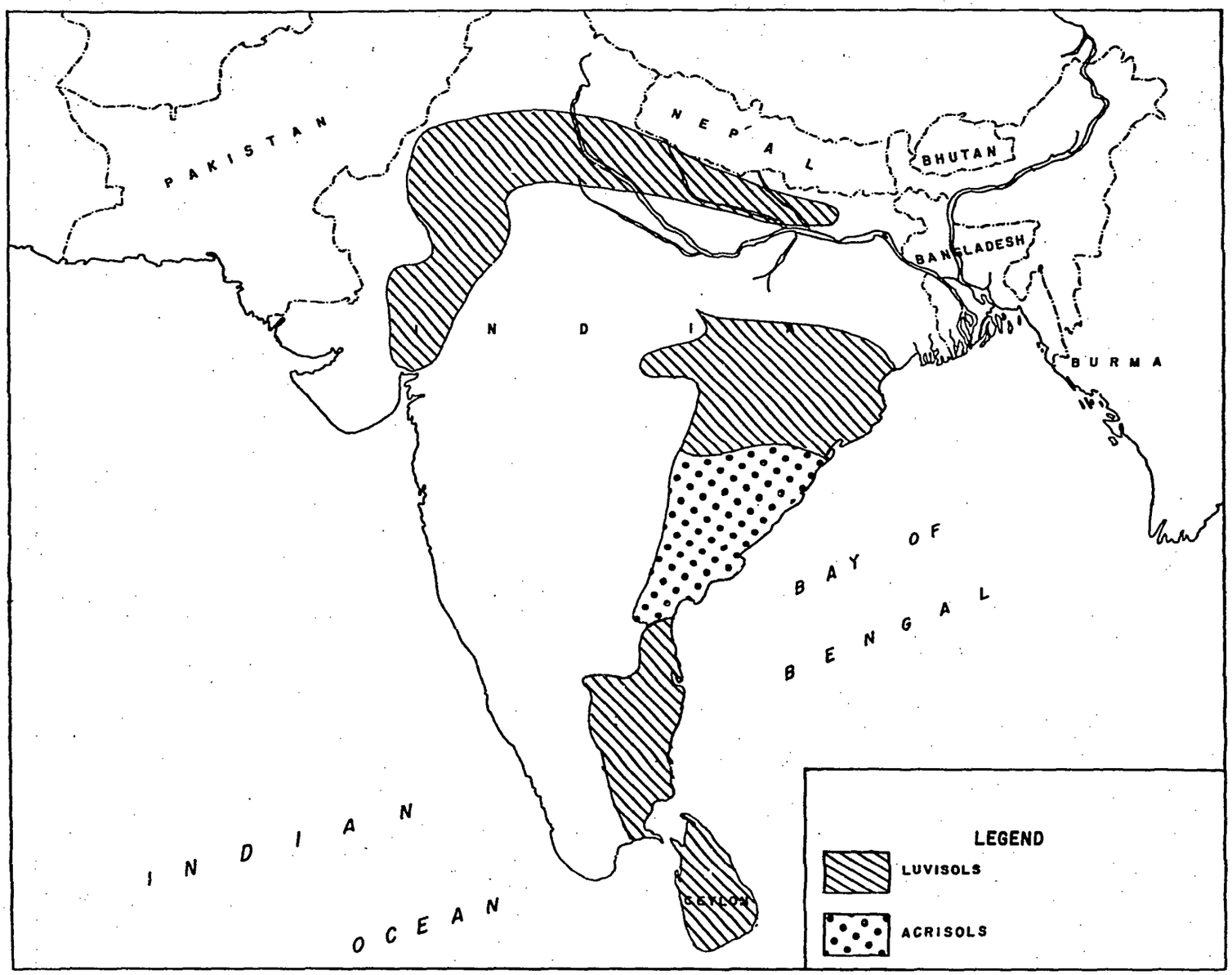


FIGURE 3.15 – RED TROPICAL SOILS OF INDIA AND CEYLON

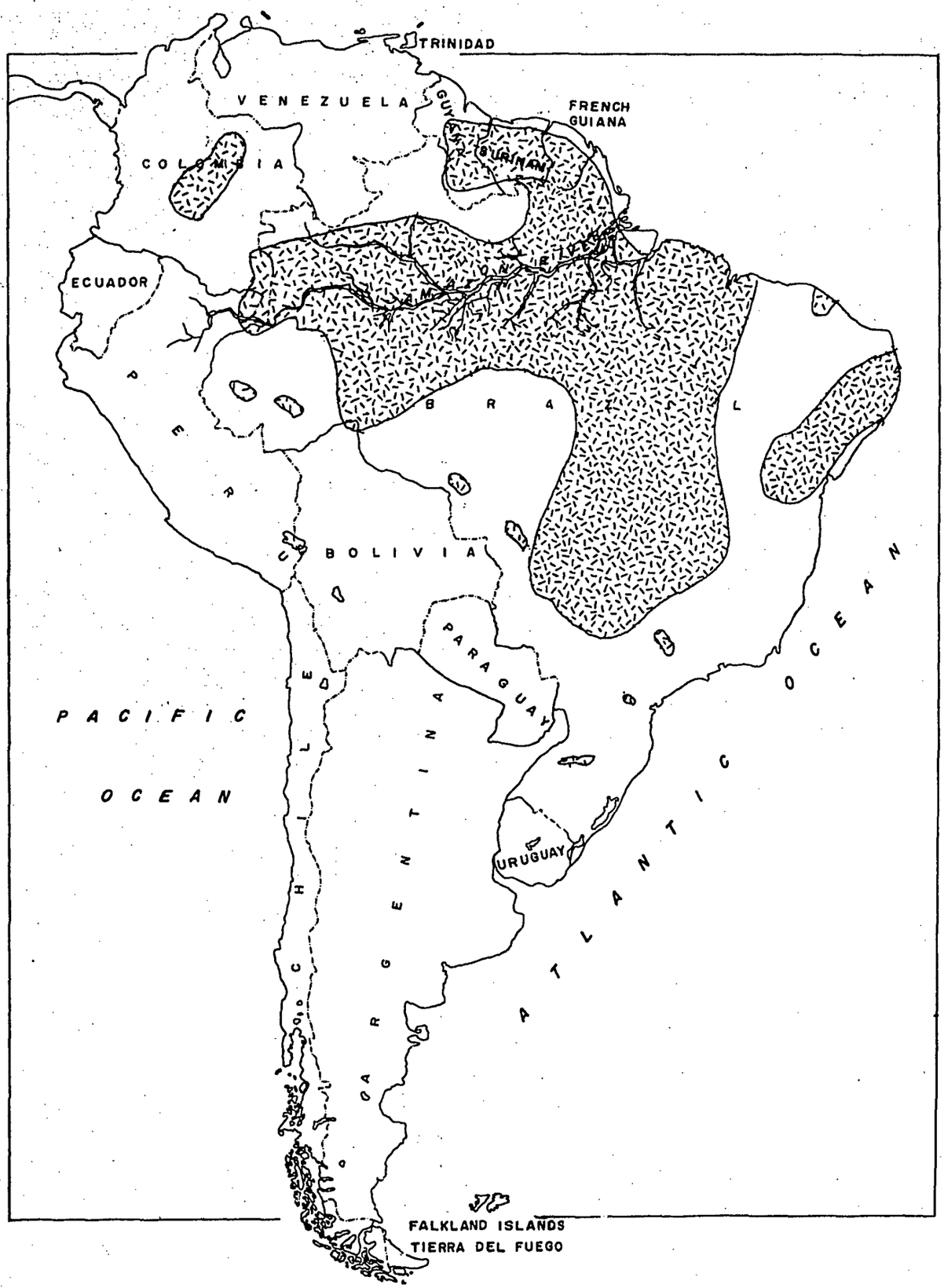


FIGURE 3.16 – AREAS WITH IRONSTONE AND CONCRETIONARY GRAVEL IN SOUTH AMERICA

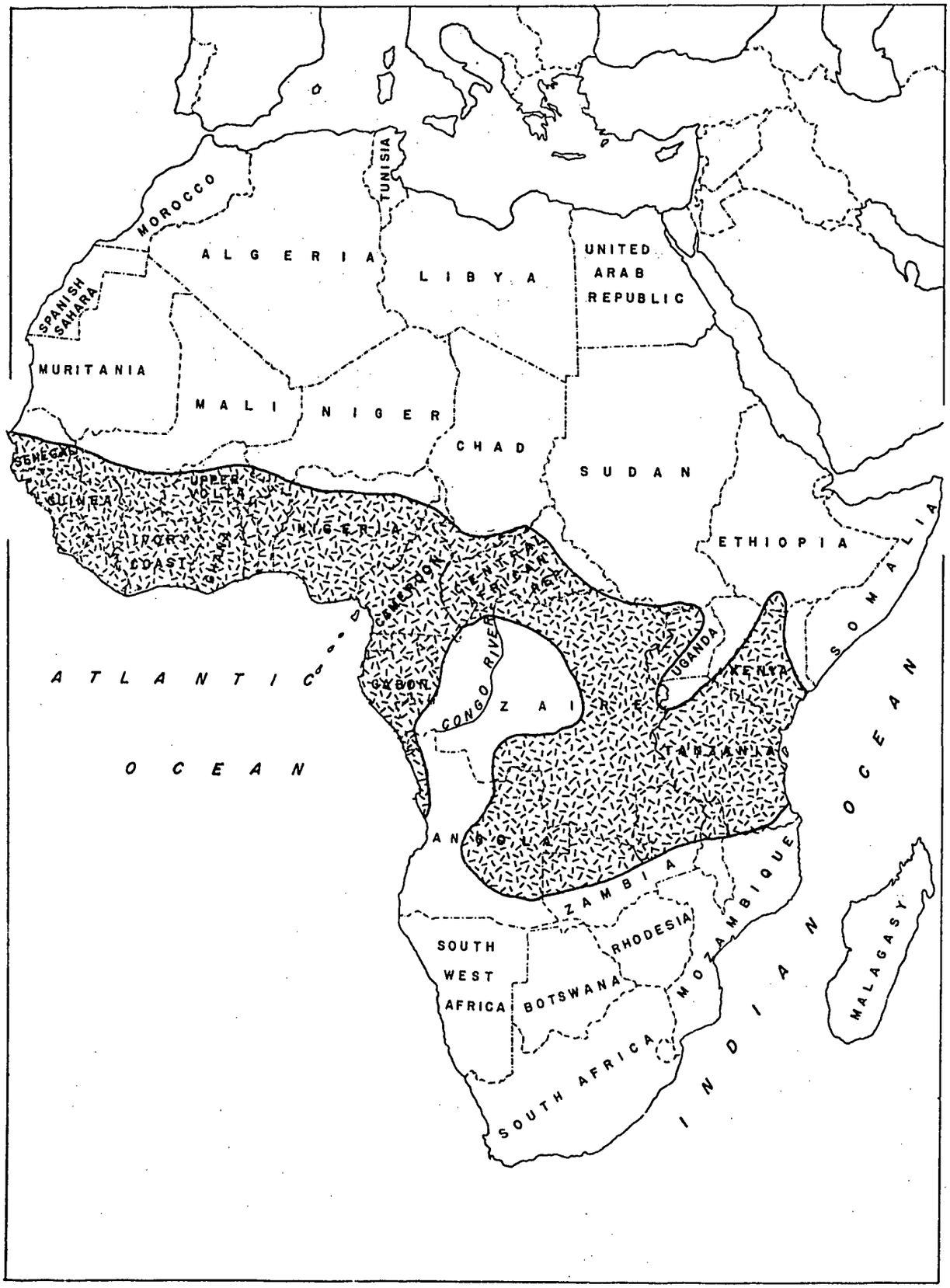


FIGURE 3.17 – AREAS WITH IRONSTONE AND CONCRETIONARY GRAVEL IN AFRICA

along the coast of western Africa in the savanna regions. Deposits also occur along the central eastern coast in Tanzania and along the eastern coast of Malagasay Republic.

Figure 3.18 illustrates those parts of Southeast Asia believed to contain concretionary gravel or ironstone. Again, the relatively recent geological environment of the volcanically active archipelago has not been conductive to the formation of these materials. However, scattered deposits are found in Burma, Thailand, Laos and Vietnam.

Dudal and Moorman (1964, p. 27) provide details of the occurrences in Southeast Asia:

> "Laterite forms extensive horizons in many great soil groups of Southeast Asia. Most of the grey podzolic and low humic gley soils occupying old river terraces and peneplains in continental Southeast Asia contain continuous layers of soft laterite which is commonly used as a building stone. In these soils, hard concretionary forms also occur in association with lowered watertables; these forms may occur close to or at the surface. Similar laterites, mostly of the hard variety, develop in red-yellow podzolic soils, but those found in the subsoils of Ceylon's wet zone are soft, continuous or irregular. Latosols derived from old plateau basalts in south Vietnam and Cambodia, and from sedimentary material in Borneo, Bangka, and Belitung* have extensive layers of hard laterite concretions. In soils evolved from young basalts, the laterite concretions and continuous hard pans near the surface are usually confined to the lower, hydromorphic parts of the landscape. In other great soil groups, laterite is less important in extent and volume; scattered concretions are frequent, however, in alluvial soils and in certain soils of the drier regions such as red-brown earths, non-calcic brown soils and their hydromorphic associations."

The ironstone deposits of Australia are given in Figure 3.19. When this map is compared with present day climatic patterns of Australia, it is obvious that these must have formed under earlier, wetter climates.

Deposits in India are given on Figure 3.20.

Moisture diagrams which were discussed in the first part of this chapter are illustrated for several cities in South America and Africa in Figure 3.21. These diagrams consider precipitation, distribution, evaporation and transpiration in these locations.

In terms of a moisture region, the luvisols (ferruginous soils) occur in semi-arid to moist sub-humid regions. The moisture diagram, Figure 3.21, indicates that the regions in which these soils occur show three moisture cycles including periods when: (1) there is no water in the soil (water deficiency); (2) the water needs exceed precipitation (moisture utilization); and (3) the precipitation exceeds water needs (moisture recharge).

Ferralsols and acrisols (ferrallitic soils) occur in moist sub-humid to very humid regions. The moisture diagram in these zones shows a complete cycling of moisture conditions including periods when (1) there is no water in the soils; (2) the water needs exceed precipitation; (3) the precipitation exceeds water needs; and (4) the precipitation exceeds both water needs and soil capacity (water surplus). This water surplus cycle appears to be a distinctive hydrological feature of ferralsols.

Nitosols (ferrisols) are found in moist sub-humid regions. Information is lacking, but the moisture diagram for nitosols (ferrisols) is assumed to be similar to that given for the ferralsol group, except that there may be a shorter period when precipitation exceeds both the soil needs and capacity.

* Bangka and Belitung are islands between Sumatra and Borneo.

TROPICAL BLACK CLAYS

Tropical black soils are often called "black cotton" soils. The term is believed to have been first used in India where the areas with black or dark gray soils are favorable locations for growing cotton. Apparently related or similar soils have been given many other names, such as the "regur" soils of India, the "black turfs" of Africa and the "margalitic" soils of Indonesia. Bal (1935, p. 261) has defined black cotton as:

> "Soils derived from the weathering of trap rock, in particular, which are black, heavy and climatologically suited to the growth of cotton, are known as black cotton, or regur soils."

Mohr et al., (1972) have adopted the term vertisol from the new Soil Taxonomy which they describe as dark to very dark clays which shrink or swell considerably with changes in moisture content. During the drying process shrinkage cracks up to 25 cm (10 in) in width develop. They sometimes extend a few meters into the soil. The soil expands on wetting and the cracks close.

Whatever local terminology is applied to black soil, or black clays, they display common characteristics which allow them to be grouped together. Tamhane and Sen (1954), believe that black soil of temperate regions do not display similar characteristics. There does not appear to be a basis for this statement as black clays in the tropics are often chemically and mineralogically similar to temperate black clays. The principal characteristics common among almost all black soils are: high clay content; dark color, tendency to expand and shrink with moisture, and presence of appreciable quantities of montmorillonite.

Characteristics and Occurrences

The black color has been attributed by some authors to the organic content and by others to the titanium content of the soil. It has been found that the quantity of organic matter was negligible in some samples which, nevertheless, had characteristic coloring. It is doubtful that the titanium content would account for the black color since titanum oxide is white. In their study of the black turf soils in South Africa, Theron and Van Niekerk, (1934, p. 734) state:

> "The black color is a direct result of the method of weathering in respect to both the organic and mineral constituents. The latter contain in high percentage of silica and the clay fraction and consequently a relatively small quantity of black organic matter can impose its color on the entire soil mass."

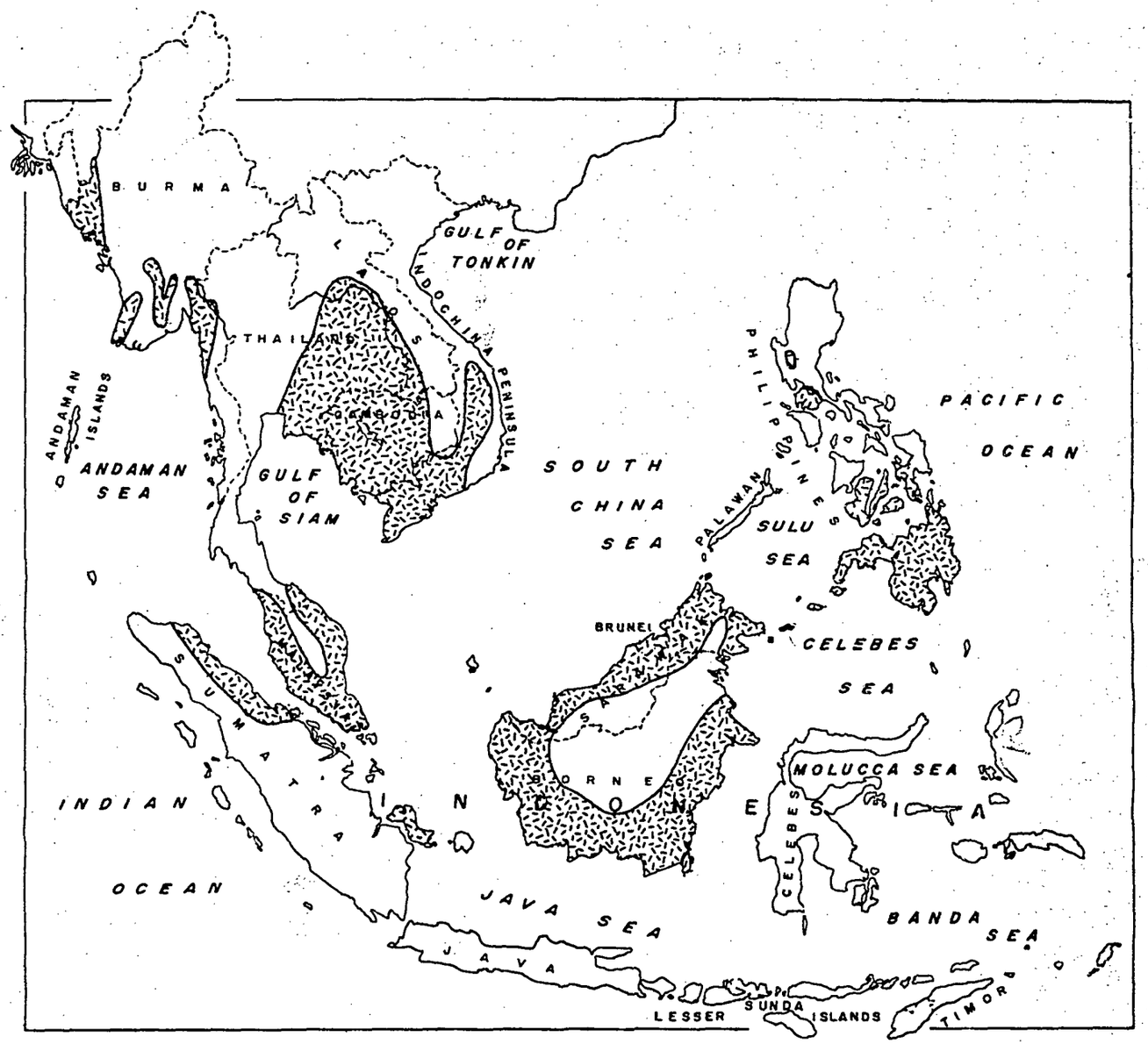


FIGURE 3.18 – AREAS WITH IRONSTONE AND CONCRETIONARY GRAVEL IN SOUTHEAST ASIA

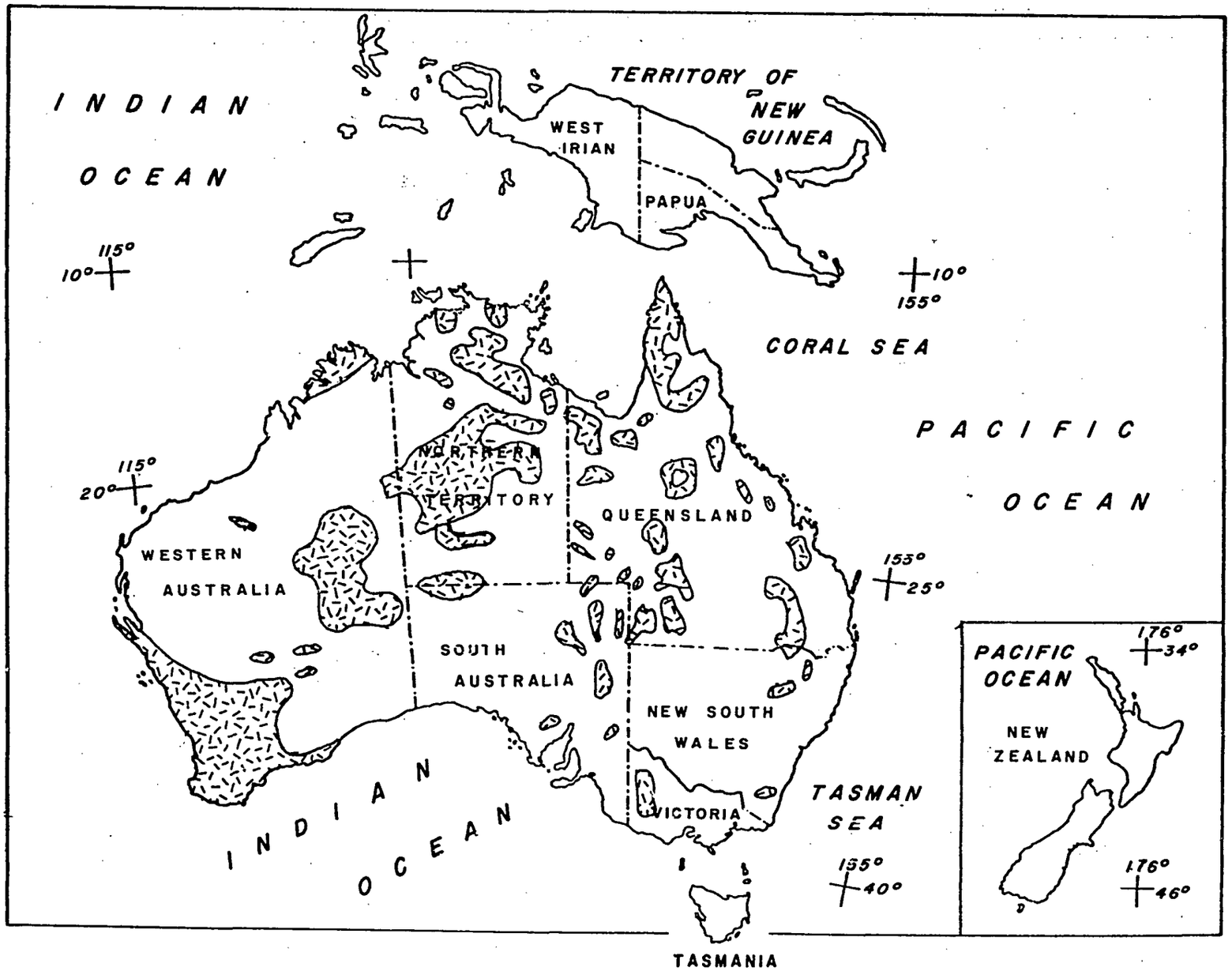


FIGURE 3.19 – OCCURRENCES OF IRONSTONE AND CONCRETIONARY GRAVEL IN AUSTRALIA

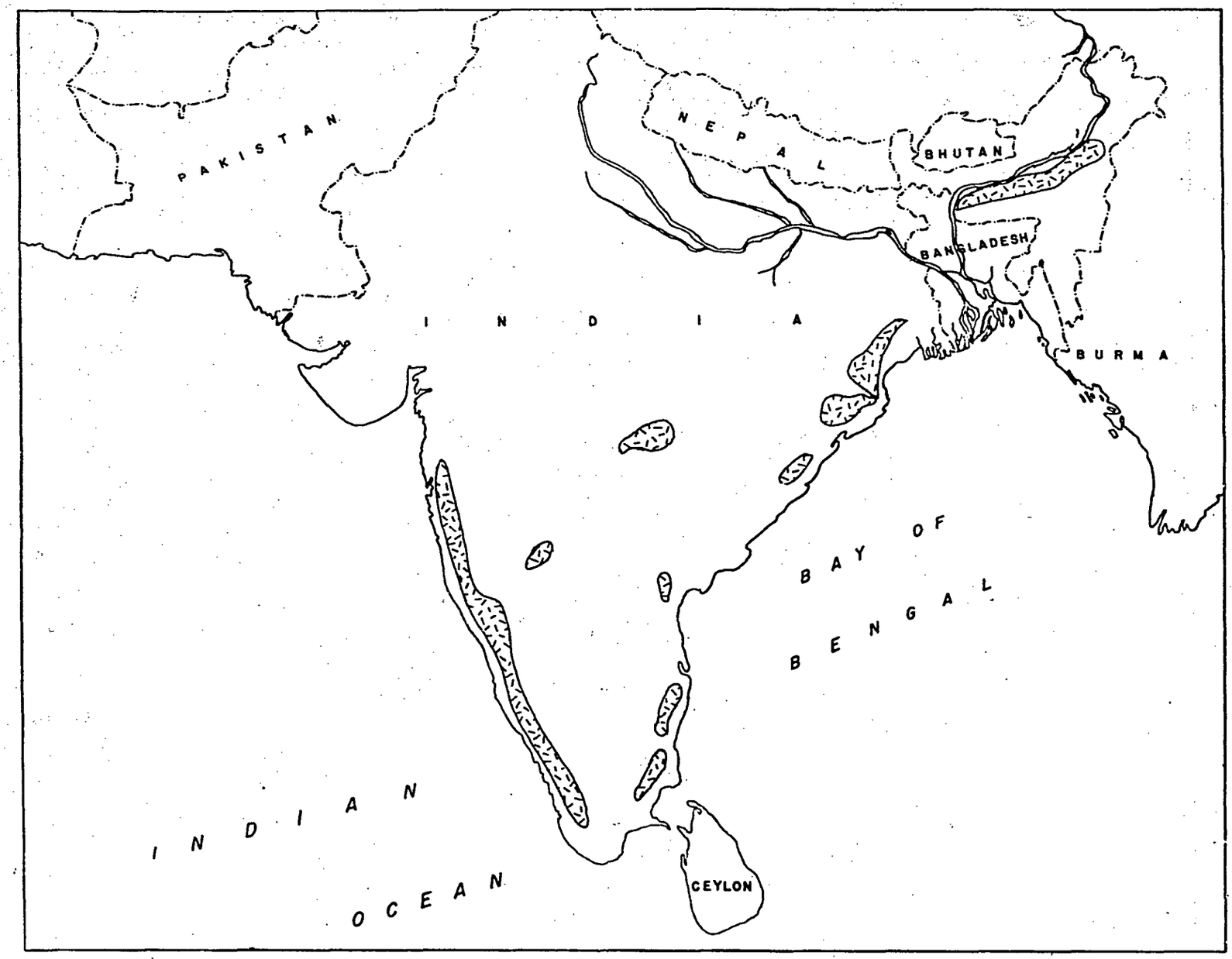


FIGURE 3.20 – OCCURRENCES OF IRONSTONE AND CONCRETIONARY GRAVEL IN INDIA

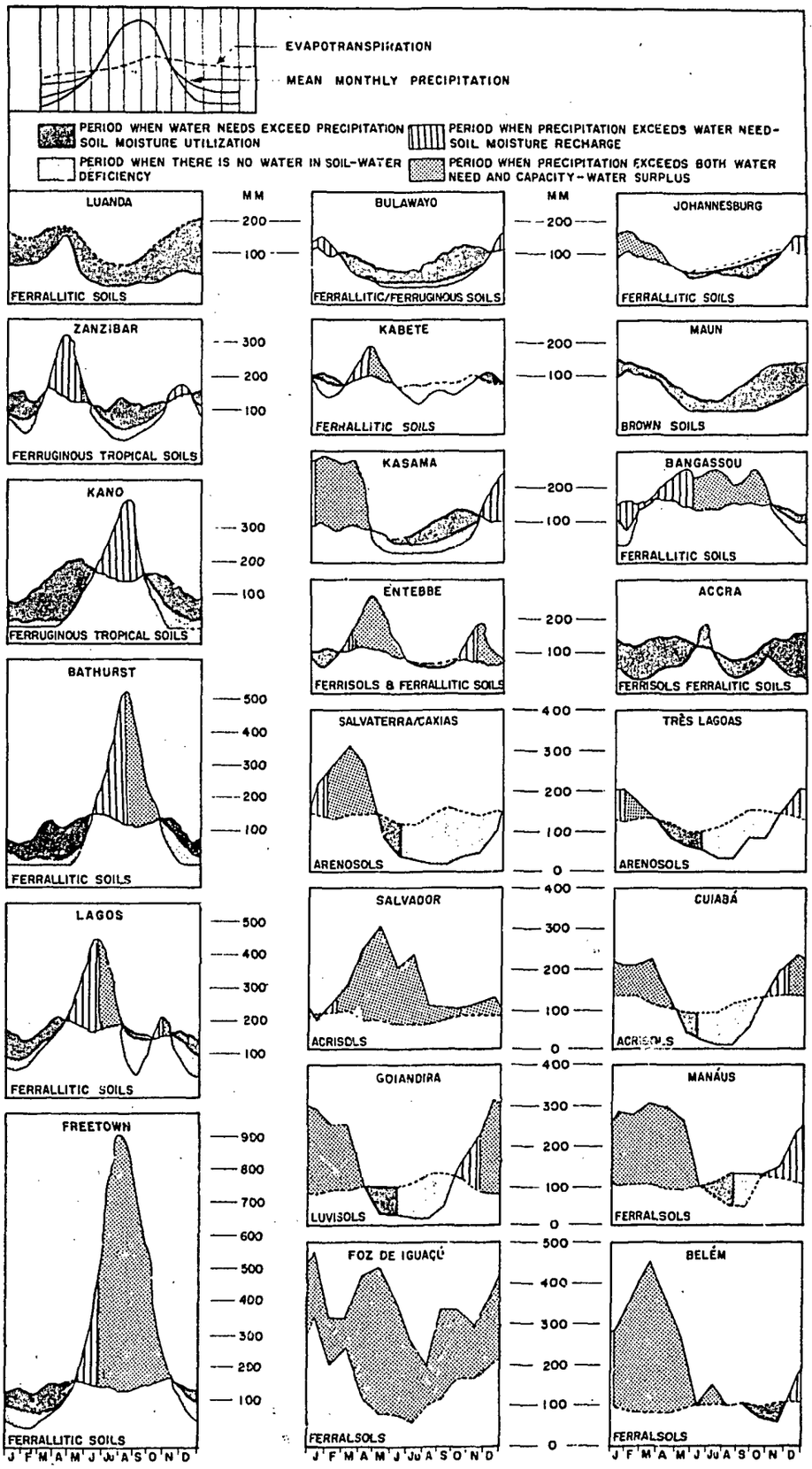


FIGURE 3.21 WATER BALANCE DIAGRAM FOR SOME BRAZILIAN AND AFRICAN LOCATIONS (African locations from Oxford Regional Economic Atlas of Africa, 1967)

These soils rarely develop thick profiles. Van der Merwe and Heysteck (1955) describe South African occurrences that vary from 35 cm (14 in) to 1.8 m (6 ft). Agarwal and Mukerji (1949) report a depth of 1.8 m (6 ft) in India.

Chemical analyses of the clay fraction of some selected black clay soils from Indonesia are included in Table 3.10 while a partial chemical analyses of South African black clays are listed in Table 3.11. The chemical composition varies considerably from the tropical red clays. For example, silica content is high and the iron content is relatively low. The silica-sesquioxide ratios are above 2.50 and the silica-alumina ratios are above 3.40; both are considerably higher than those for the red soils.

The mineralogical composition of the clay fraction invariably includes montmorillonite. Kaolinite is occasionally reported but always in minor quantities. Other clay minerals have also been reported.

The mineralogical composition of the fine sand fraction of various black cotton soils of India was determined by Tamhane and Sen (1954). They found that the sand-size derived from basalt is principally composed of augite with appreciable quantities of magnetite and limonite.

TABLE 3.10

Chemical Composition of the Clay Fraction of some Indonesian Black Clays (after Mohr et al., 1972)

	Sample			
	12843	12461	33193	30277
SiO_2	42.8	36.8	40.6	38.9
Al_2O_3	19.0	17.1	20.2	17.7
Fe_2O_3	8.7	12.2	11.0	5.5
TiO_2	tr	0.6	0.8	0.7
MnO	tr	tr	0.1	tr
CaO	1.7	2.2	2.4	4.6
MgO	1.8	1.8	1.1	0.7
K_2O	0.1	0.2	0.2	0.4
Na_2O	0.8	3.4	0.5	0.5
P_2O_5	0.1	tr	0.1	0.1
CO_2	–	–	0.1	1.0
H_2O	24.[illegible]	23.6	21.8	23.0
Organic Matter	0.9	1.3	1.6	1.5
$SiO_2 - R_2O_3$ ratio	2.9[illegible]	2.52	1.52	3.25
$SiO_2 - Al_2O_3$ ratio	3.82	3.66	3.40	3.82

TABLE 3.11

Partial Chemical Analyses of South African Black Clays (after Van der Merwe and Heysteck, 1955)

Parent Material	Depth (in)	Clay Fraction (%)	SiO_2	Fe_2O_3	Al_2O_3	SiO_2 / R_2O_3	SiO_2 / Al_2O_3
norite	0-9	60.1	55.87	5.62	25.38	3.27	3.73
	21-26	65.6	57.25	6.02	22.81	3.63	4.25
	46-54	12.6	56.57	9.34	22.40	3.34	4.28
basalt	0-7	46.5	55.57	13.[illegible]	22.52	3.02	4.18
	16-28	41.7	52.16	15.74	24.08	2.59	3.67
	28-38	25.8	51.98	15.96	23.56	2.61	3.74
dolerite	18-28	57.2	56.06	14.07	19.98	3.23	4.66
	34-54	41.1	55.65	14.17	20.09	3.24	4.70

TABLE 3.12

Partial Chemical Analysis of the Clay Fraction of a Soil Profile at Ata, India (after Agarwal and Mukerji, 1949)

	Depth (inches)					
	0-12	12-28	28-42	42-56	56-60	60-72
SiO_2	44.03	43.85	44.26	45.10	44.90	44.89
Al_2O_3	21.30	23.35	19.60	20.10	22.15	–
Fe_2O_3	16.40	17.60	18.80	19.20	19.60	–
MgO	2.46	2.46	2.17	2.57	3.17	3.77
K_2O	2.36	2.01	2.90	2.61	2.80	2.80

PROFILE DEVELOPMENT

Profiles in the tropical black clays are less developed than in the red soils. Bal (1935) found few marked horizons in the black cotton soil profiles in India. Similarly Muir (1951) found little evidence of profile development over the basalts in Syria. Tamhane and Sen (1954) found the chemical properties change very little with depth which explains the low "horizontation" characteristics of the black soils of India. Morin and Parry (1971) and Mohr et al., (1972) state that basic rocks usually change abruptly into soil; total transformation takes place within a layer of a few millimeters or even tenths of millimeters. A partial chemical analysis of the clay fraction of a profile in India is provided by Agarwal and Mukerji (1949) and given in Table 3.12. It can be seen that the composition varies but little with depth.

OCCURRENCES

A number of authors have described the Indian occurrences of black cotton soils. Tamhane and Sen (1954) found that the black cotton soils of India occur mainly over the basic volcanic rocks of the Deccan basalt series. They are not produced from acidic rocks in climatic conditions that prevail in India. Bal (1935) found, however, that they occasionally occur over gneissic rock.

Uppal (1965, p. 166) offers the following descriptions:

> "the black cotton soil is one of the major soil groups of India and covers an area of 200,000 square miles (518,000 square kilometers). It derives its name from its black color and also from the fact that cotton grows on it very well. The soil is hard as long as it is dry but loses its stability almost completely when wet. On drying the soil cracks very badly and in the worst cases, the cracks are almost 6 inches (15 cm) wide and travel 9 to 10 feet (2.7 to 3.0 m) deep..."

Other tropical and subtropical areas of black soils have been described. Van der Merwe and Heysteck (1955), in their study of South African black clays, found that these soils are chemically similar even though they are derived from different rocks. However, they also note (p. 158),

> "under all circumstances, mature or immature, the mountain soils derived from basalt are black and the predominant clay mineral is montmorillonite. Under subtropical conditions, however, the soils from basalt are either red or black and the clay minerals accordingly are different... In all cases the initial weathering of basalt seems to produce montmorillonite, which under certain influences weathers further to soil kaolin, releasing iron oxide to produce a red clay."

Australian occurrences have been described by Hallsworth (1952), Hosking (1935, 1940), and others. Hosking emphasized the importance of climate on the development of soil types, including mean annual rainfall, its seasonal occurrence, and efficacy of evaporation. In his later work, he found the black clay soils apparently form in poorly drainage areas over basaltic rocks were climatic conditions would ordinarily cause red loam to form if the drainage were good. Under the influence of the present climatic conditions in New South Wales, Hallsworth (1952) believes the two soils are due to differential leaching, i.e. the dark soils are those less subjected to the leaching processes.

Other occurences of tropical black soils have been described. Tropical black soils from the Gold Coast (now Ghana) were investigated by Stephen (1953) who found that the soils derived from pre-Cambrian hornblende-garnet gneiss are typical black cotton soils. Dark brown to black soils have developed over basalt in Syria (Muir, 1951). Aubert (1941) described black clay soils over basalt in Madagascar; gray or black soils over volcanic rocks in the Itasy volcanic region of West Africa; and black clay soils developed over basalt in South Vietnam. Dark brown to black soils overlying basic igneous rocks occur in Cuba (Bennett and Allison, 1928) and over basic volcanics in various areas in Central America. They have also formed over the basalts of the Parana basin in Southern Brazil and Uruguay.

The distribution of tropical black clay soils in tropical regions is shown on several maps. The limited occurrences in South America and Central America are illustrated in Figure 3.22 and 3.23 respectively. The more extensive deposits of Africa are shown in Figure 3.24. Southeast Asia has many deposits of expansive black clays, but these deposits are never extensive; generalized locations are given in Figure 3.25. Large areas of expansive clays occur in Australia, Figure 3.26, as well as in India, Figura 3.27.

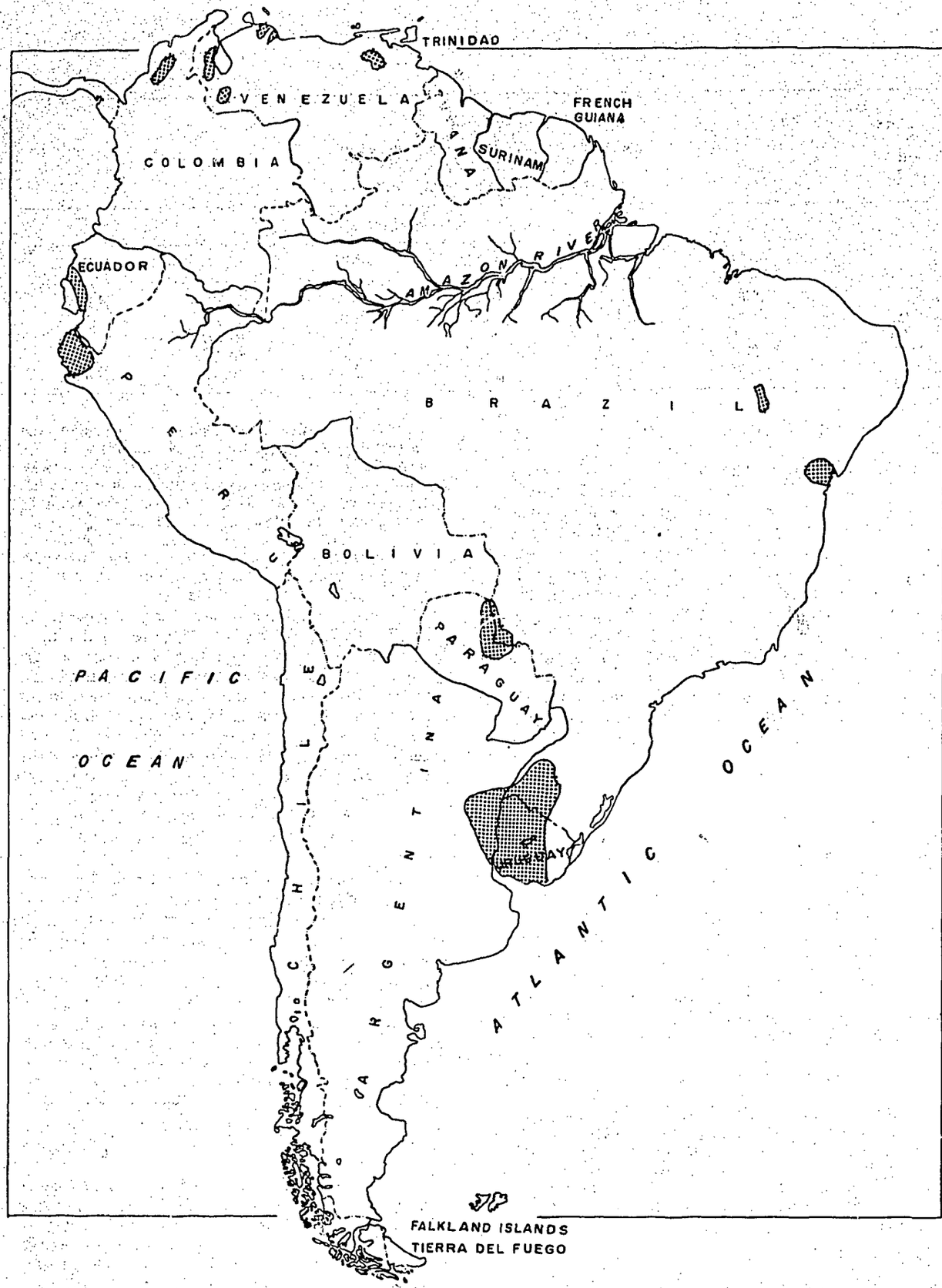


FIGURE 3.22 – MAJOR OCCURRENCES OF BLACK CLAYS IN SOUTH AMERICA

(after Dudal, 1965)

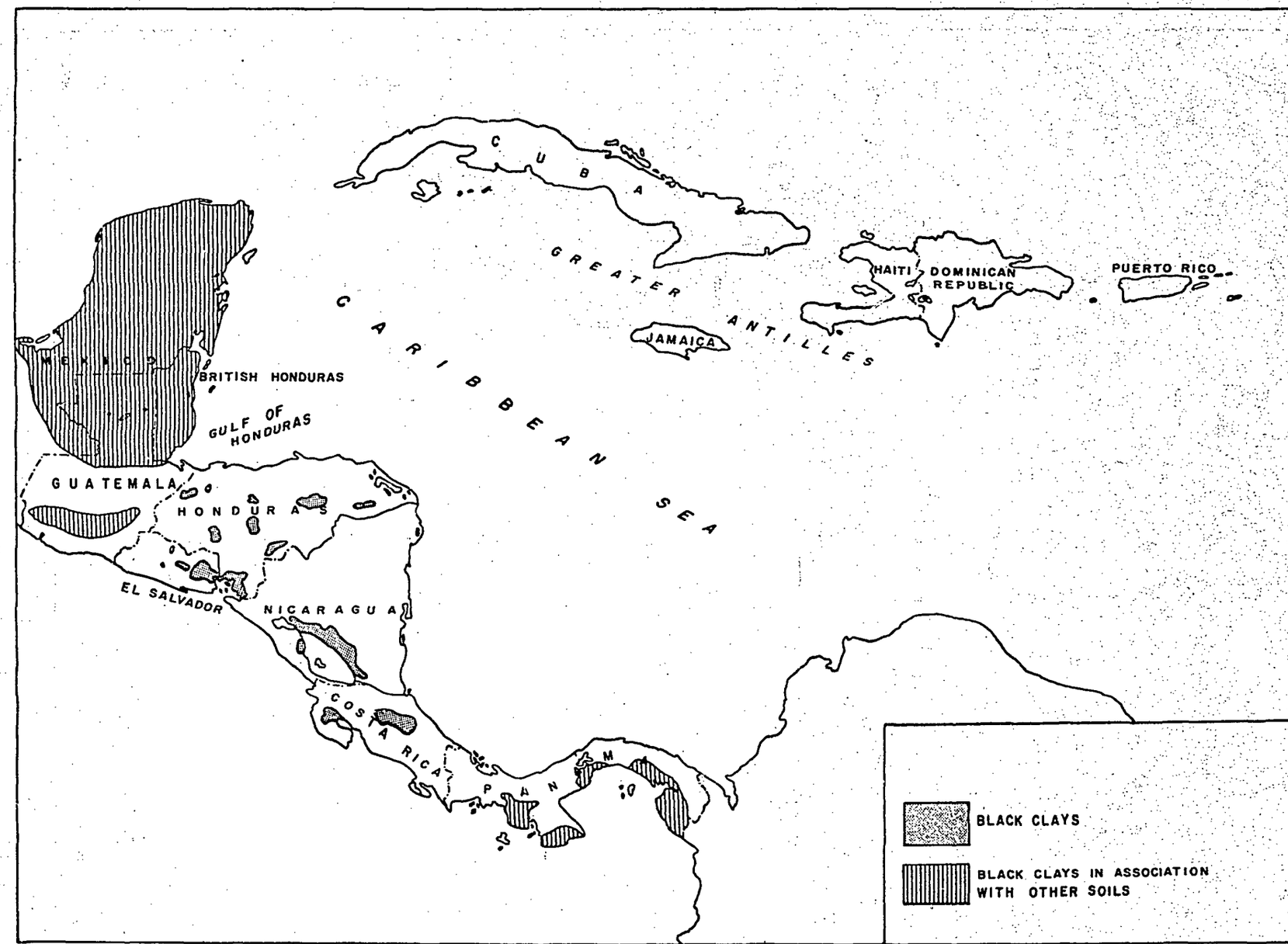


FIGURE 3.23 – MAJOR OCCURRENCES OF BLACK CLAYS IN CENTRAL AMERICA

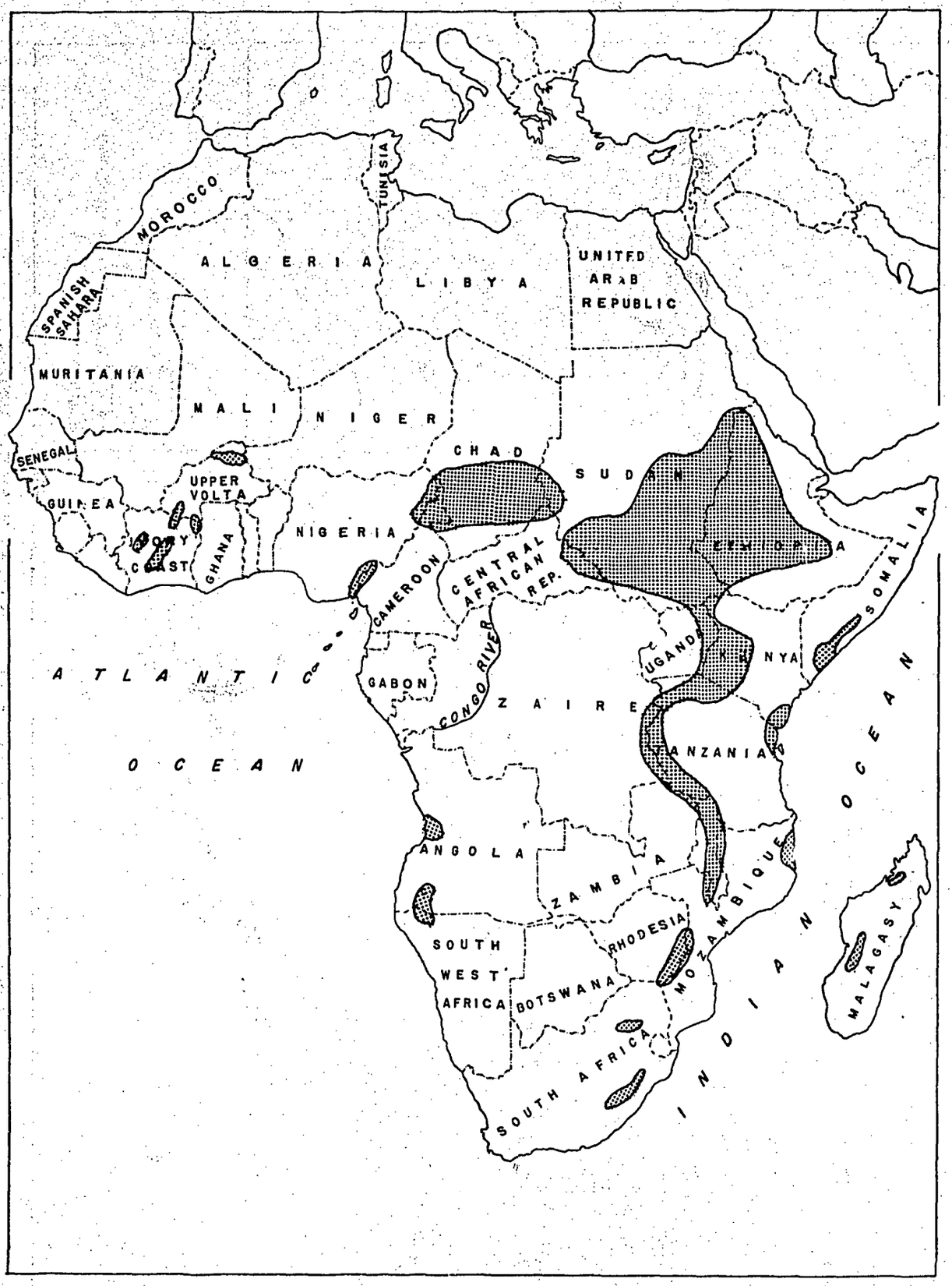


FIGURE 3.24 – MAJOR OCCURRENCES OF BLACK CLAYS IN AFRICA

(after Dudal, 1965)

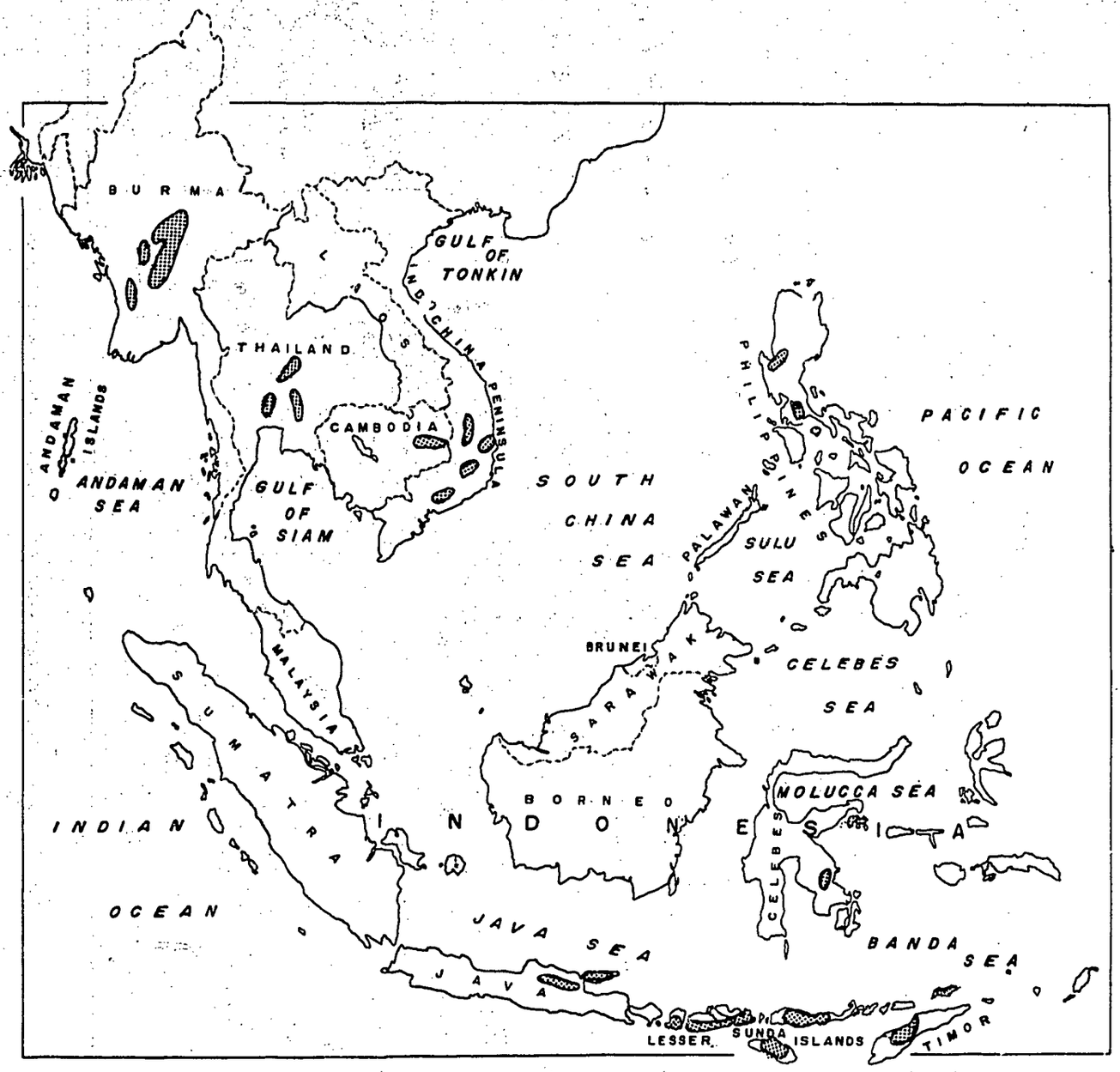


FIGURE 3.25 — MAJOR OCCURRENCES OF BLACK CLAYS IN SOUTHEAST ASIA

(after Dudal, 1965)

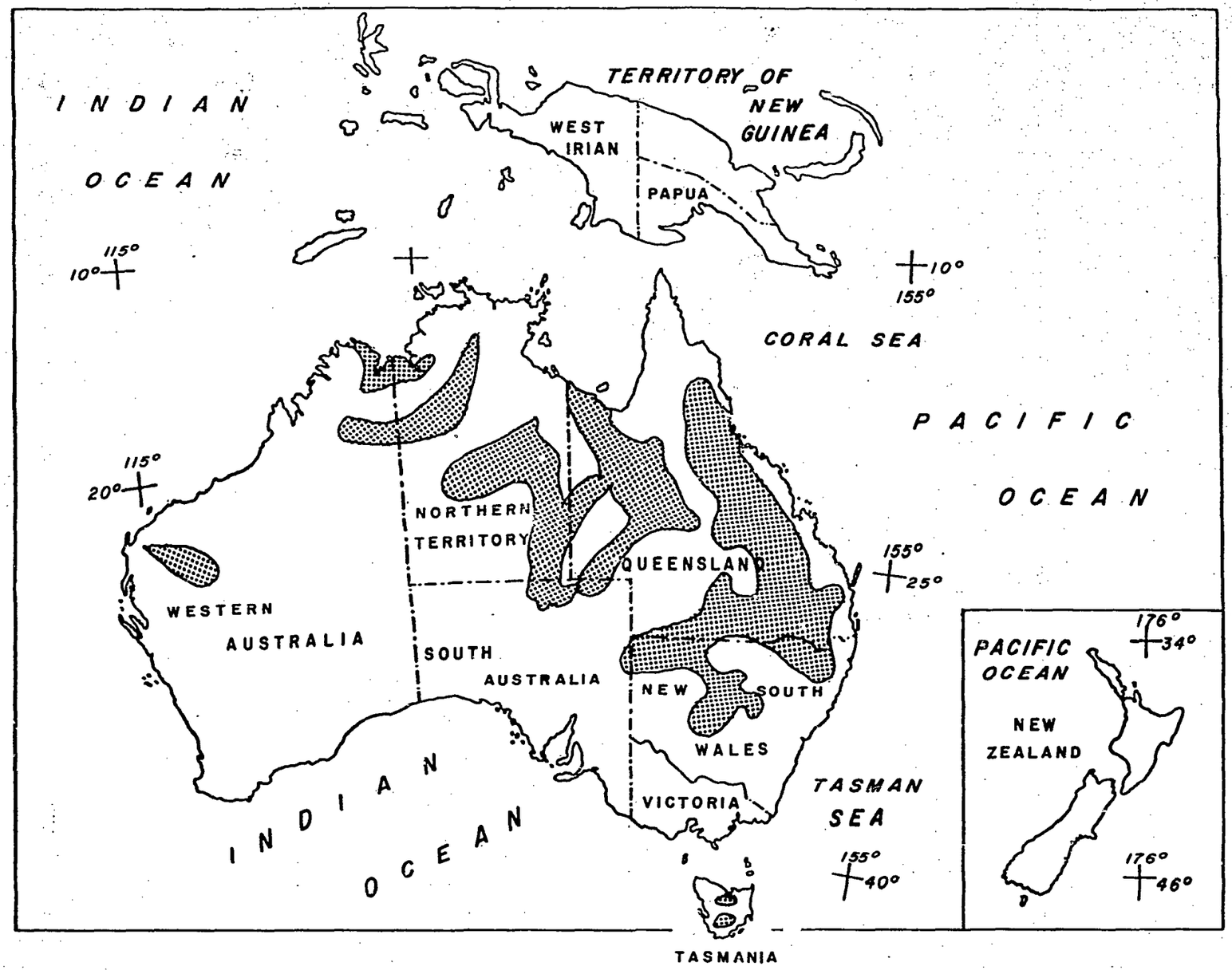


FIGURE 3.26 – MAJOR OCCURRENCES OF BLACK CLAYS IN AUSTRALIA

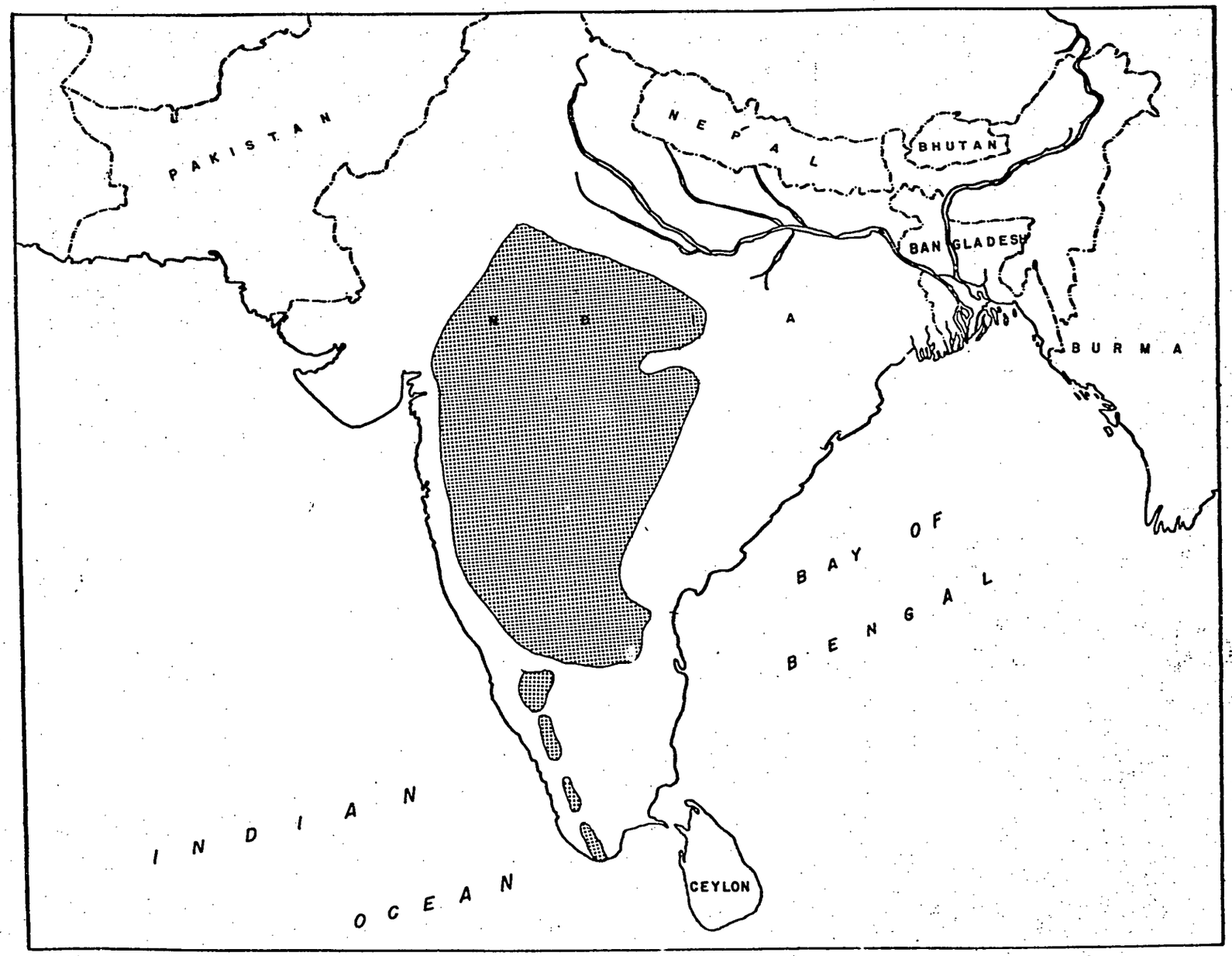


FIGURE 3.27 — MAJOR OCCURRENCES OF BLACK CLAYS IN INDIA

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CHAPTER 4
GEOTECHNICAL PROPERTIES OF RED TROPICAL SOILS

INTRODUCTION

General

The geotechnical properties of red, tropically weathered soils are presented in this chapter. There are no practical means from the engineering point of view of distinguishing between true lateritic materials and those which are only partially laterized inasmuch as the only differentiating criteria are their chemical and mineralogical properties. Both types of materials are utilized in highway construction without distinction. The properties of both are sufficiently similar, and separating them has little engineering significance. The exceptions are those moisture-sensitive soils which change properties upon drying. Such soils are usually the andosols, the less sensitive, highly laterized ferralsols or the ferrallitic soils which occur in areas with high rainfall. The former are restricted to areas of relatively recent volcanic activity.

The investigation of the geotechnical properties of red tropical soils was an important aspect of the South American phase of the tropical soils study for two reasons. First, it was not known if the findings of earlier phases of the study applied in South America; and secondly the new FAO Soil Map of the World was introduced in the interim and its new compromise terminology made further investigations essential. The principal objectives of this investigation were three-fold. They were: 1) to determine the typical properties of a wide selection of geographically dispersed tropical red residual soils; 2) to determine the group into which the soils could be placed for purposes of simplifying terminology, and 3) when possible, to indicate the engineering behavior of each group.

A summary of the prior work in Africa, Phase II, and in Southeast Asia, Phase I, follows. Details for the African study are given in Lyon Associates et al. (1971). In that study most tropical countries of Africa were visited and soils samples from each were collected and tested. However, the most intense sampling by necessity was undertaken in the Republic of Ghana. In Southeast Asia (Vallerga, et al., 1971) samples were collected mostly from Thailand, although soils from Laos, Cambodia, Vietnam and Malaysia were also examined. Other published and unpublished data from other Southeast Asian countries has been tabulated and is included in this report as well.

In South America, Phase III, pedological descriptions and soils maps were utilized during sampling and testing as these were the only means of determining the areal distribution of the various soil types and selecting appropriate sample locations. It was found in Africa that the pedological classification of the red tropical soils are reflected at least in part in their engineering properties. Therefore the pedological terms were adopted for this report since they are more descriptive than the term "lateritic soil". The pedological terms are always used unless the soil type is unknown. The laterites and lateritic soils of South America are included in the broad groups designated by the FAO Soil Map as ferralsols, acrisols, luvisols, arenosols, nitosols and some cambisols. In Africa they are included in the groups designated by D'Hoore as ferruginous soils, ferrallitic soils and ferrisols.

Chemical, mineralogical and engineering properties were evaluated. The chemical analyses provided details on the amount of leaching that has occurred. Mineralogical analyses indicated the type of clay minerals and clay-sized materials present in the soils, which is another indication of soil maturity in the laterization process. The analysis of the engineering properties allowed for the correlations of the first two with the more useful testing program utilized in engineering design. Comparisons were also attempted with the red tropical soils of Africa and Southeast Asia.

The emphasis in this chapter is on Brazil where most of the samples were collected. Samples were collected from the 17 states, from the Federal District and from 3 territories. Brazil was an ideal study area since almost all soil types found on the continent are represented in the large land area of the Republic. The only exceptions are several of the Andean soil types and andosols. Except for the latter, the FAO Soils Map shows that all major soil group occurring on the continent, occur in Brazil. However, a few samples were collected in other South and Central American countries because some subgroups of the major soils types had unique locations outside Brazil. These samples were subjected to a testing program to determine differences or similarities with the same soil group in Brazil. The andosols samples were obtained in Central America and Colombia.

Andosols are not usually considered to be lateritic although they may occur in close proximity to soils that are lateritic. Andosols are often referred to as volcanic soils and some have been referred to as "peculiar soils" of volcanic regions. This soil type will be discussed at some length prior to the discussions of other lateritic soils. Andosols are closely related, usually immature soils, and may cause engineering problems in rather restricted areas. Similar problems may occur in the more laterized soils, although they are likely to be far less pronounced. An understanding of the problem of andosols will provide for a better appreciation of the properties of the more mature lateritic soils.

MOISTURE-SENSITIVE SOILS OF VOLCANIC REGIONS

General Statement

It has been known for a number of years that certain soils found over volcanic materials displayed unusual properties. Many of these soils contain abnormally high moisture contents and yet display surprisingly high strengths. Some even appear granular, but after manipulation become highly plastic, saturated clays. Most of these soils change properties with air or oven drying and actually become granular or "granulate". Such changes are generally believed to be irreversible.

The phenomenon of change in properties with drying is not restricted to volcanic soils. It occurs as well with other soils but the changes in other soils are generally not very large. Those soils which are highly moisture-sensitive are invariably found over volcanic or basic intrusive rocks. Age and precipitation are factors. For example, young soils are more susceptible to change; and high rainfall is the most important factor in preventing the dehydration which causes the change.

Andosols develop over volcanic materials, particularly pyroclastics or material ejected violently from volcanic vents, as opposed to more placid lava flows. Andosols are not restricted to the tropics. They are found in the volcanic regions bordering the Pacific basin along the Andean Cordilleras; the volcanic mountains of Central America; the chain of volcanic mountains along the western coast of the United States; Alaska and the Aleutian Archipelago; the Philippine Archipelago; as well as the Republic of Japan, Indonesia, New Guinea, and New Zealand. Islands within the Pacific basin, such as the Hawaian chain, also have andosols. There are also fairly sizeable deposits in Eastern Africa, Madagascar and there are possibly limited occurrences in western Africa. The latter is associated with the chain of volcanic mountains in Cameroon. Finally there are small, scattered deposits in Europe and Iceland. The worldwide distribution of andosols is shown on Figure 4.1.

These soils are restricted to humid regions, either temperate, sub tropical or tropical. According to Mohr et al., (1972, p. 398): "Soils developed on volcanic ash such as the regosols and other mineral soils in arid regions and the organic soils in very humid and superhumid environments, do not comply with the modal concept of andosols". In some areas they are found only in the higher elevations. Such is the case in Africa and Indonesia, where andosols are found only above 1,100 or 1,200 m (3,600 or 3,950 ft). In Kenya they are found at elevations above 2,150 m (7,000 ft) on the plateaus or mountains. The higher rainfalls and lower temperatures found in these regions are apparently required for andosols to retain their hydrated condition. On the other hand, they are found at lower, but perhaps more humid, areas of Ecuador and Colombia as well.

The degree of hydration of the andosols is at least partly the reason why these soils do not always display changes in properties. Their occurrences in temperate regions where they are in a less hydrated state are not known to have caused construction problems. However, these soils have caused significant problems in the tropical areas of Central America, Hawaii, Java, New Guinea and in some locations in East Africa.

The parent material over which andosols form is almost always a type of unconsolidated volcanic ejecta. Apparently the most important constituent in andosols is volcanic glass. The tuff in volcanic ejecta or other volcanic material can be acid, intermediate or basic in composition, but the basic variety more commonly develops into andosol. However, a soils profile was examined in Guatemala which had the appearance and properties of andosol but in this case the parent material was serpentine.

The changes in properties with drying is discussed below. It was taken in part from the African Phase II Report and subsequently published at the Fifth Regional Conference for Africa on Soil Mechanics and Foundation Engineering (Morin and Ayetey, 1971). In addition, samples collected from Central America during Phase III were subjected to x-ray diffraction and differential thermal analysis. The data from these analyses and the findings of earlier reports provide the following background on past and current research into these soils.

Effects of Drying on Properties

There are two factors which cause the changes in properties of andosols and some types of red tropical soils when they are dried. These are: (1) the tendency of the soils to form aggregations and (2) the loss of water in the hydrated minerals in the soils. Although these are related factors the phenomena are different. Hirashima (1948) is believed to be the first to describe the soil on the island of Hawaii which changes properties on drying. In a later paper (1951) he provided more details on the soil, which formed over volcanic ash. His samples from Hawaii displayed moisture contents of up to 200% and yet they were not freedraining. The rainfall in the area varies from 2,500 to over 8,000 mm (100 to over 300 in) per year. There are cut slopes as steep as 1/4 to 1, which are stable in spite of the high moisture content. However, once disturbed, these soils lose their consistency. If this manipulation is excessive, it may even flow. This indicates that the water is held structurally in an amount which is higher than the liquid limit. Upon partial drying of the surface layer, which on Hawaii can vary from 15 to 45 cm (6 to 18 in) thick, the soil irreversibly changes from its highly plastic state to a relatively non-plastic material that will support automobiles and light trucks.

The Sasamua Dam in Kenya has received a lot of notoriety after Terzaghi (1958) first described the red clays that occur in the area over volcanic material. He considered the high angle of internal friction and low compressibility abnormal because of the high clay content (70%-100%) and the high liquid limit. He explained this anomalous behavior by assuming that the clay existed in clusters of interconnected clay particles. The clay was considered strongly aggregated, a state which was unlikely to be changed during construction.

The aggregation effect of the iron oxide in the Sasamua clay was later demonstrated by Newill (1961). He removed the iron chemically and found the clay aggregations had been dispersed. These did not reform as had been the case after mechanical manipulation. The Atterberg limits were determined before and after chemically removing the free iron oxide. As a result it was found that the liquid limit and plasticity index increased considerably.

The dehydration of other clay materials also causes changes in properties. Tateishi (1967) described variations in the values of maximum density tests. These variations depend on whether the tests were performed after drying or without air drying, where the latter is accomplished by determining the points backward along the moisture-density curve as the soil dries. This was confirmed by Quinones (1963) on a variety of lateritic soils from Hawaii, Puerto Rico and Liberia.

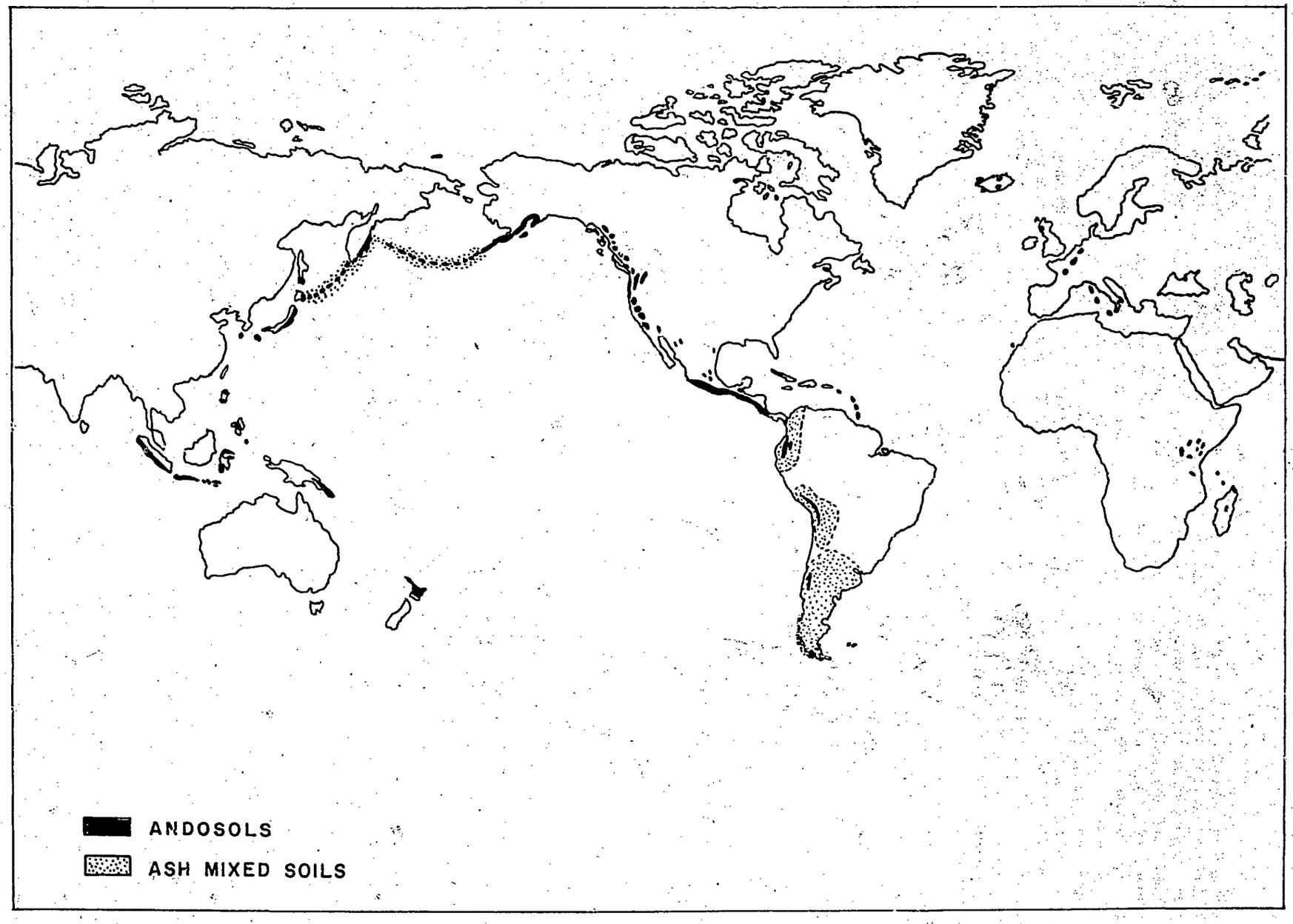

FIGURE 4.1 DISTRIBUTION OF ANDOSOLS AND ASH MIXED SOILS (After Mohr et al., 1972)

Recently Wesley (1973a) disputed the "cluster hypothesis" for the tropical red clays in Java. He agreed, that clusters, or aggregations, exist after drying, but he found that mechanical dispersion readily breaks down such clusters in soils tested at their natural moisture contents. He believes the strength of the natural soils is not due to the clusters as Terzaghi proposed. Newill (1961) also tested the Sasamua clay without air drying and found that the clay content was 79%, which is considerably higher than the 50% reported by Terzaghi or the 47% which Newill himself observed after air-drying. He had little trouble dispersing the clay that had not been air-dried. There is, however, no doubt that these soils readily aggregate on drying and consequently they undergo associated change in properties.

The clay materials most susceptible to changes of properties with drying are considered to be allophane, halloysite, and cliachite or gibbsite. These will be discussed individually.

Allophane

Allophane, a term used to describe amorphous material with various percentages of silica and water is found in andosols. The composition varies from $SiO_2 . Al_2O_3 . H_2O$ called allophane B by Fields (1955) to $2SiO_2 . Al_2O_3 . 3H_2O$. The latter form, termed allophane A by Fields, is found in older soils. It is believed that allophane A crystallizes to halloysite which eventually alters to kaolinite. There is no doubt that either type of allophane would lose much of its water on drying which would result in changes in its properties. Gradwell and Birrell (1954), for example, describe a soil containing a large percentage of allophane in its clay fraction. It displayed a liquid limit of 225 when tested with its natural water and 100 when it had been air dried prior to testing.

Fields (1955) and later Keller's (1964) description of the transformation of volcanic soil minerals in New Zealand, from allophane to kaolinite, suggest a possible reason for the unusually high strengths and moisture contents of soils with allophane. Hydrous alumina which is precipitated out of solution, crystalizes to form the gibbsite sheet with the aluminum contained within the hydroxyl sheets. Partial dehydration removes some hydroxyls from the gibbsite and the hydroxyls are replaced by oxygen atoms associated with silica tetrahedra. Silica tetrahedra are absorbed at random cross-linking octahedral sheets of alumina and they form a random structure full of channels which are relatively rigid. This form of silicated alumina corresponds to allophane A (allophane A of Fields).

Allophane was identified in the samples collected from Guatemala and Nicaragua and is suspected in the sample from Costa Rica. Allophane is amorphous and usually does not diffract X-rays. It displays a large endothermic peak at about 100° C, during differential thermal analyses. However, the allophane may not be detected if it exists with hydrated halloysite and gibbsite, as is often the case in tropical soils formed over volcanics in high rainfall areas. Townsend et al., (1971) compared the thermograms of a sesquioxide-free sample from Panama with that of an untreated sample. Allophane often occurs as a "co-precipitate" with the iron minerals. When the iron minerals are removed the DTA endothermic peak no longer exists or is greatly subdued. Therefore the mineral which causes the peak is allophane. The treated sample from Panama did contain allophane.

Allophane was not reported in any of the samples collected during the African study, including the samples of volcanic soils obtained in Ethiopia, Kenya and Cameroon. It is very possible that pretreatment prior to analysis of these samples removed or altered the allophane. However, none of the volcanic soils displayed the characteristic high moisture contents, nor did they show large changes of properties with drying. This may indicate that the samples became partially desiccated at some time prior to testing. If allophane does exists in the African soils it is probably restricted to those soils over volcanics in the high rainfall areas such as the high volcanic mountains of East Africa or southwestern Cameroon. Since these areas are relatively small in size, allophanes soils are not considered a general problem in Africa.

The formation of allophane is not restricted to volcanic materials. It has been reported over other rock types as well, but such occurrences are not believed to be common (Grim, 1968). The composition of the mineral when not found over pyroclastic materials apparently varies considerably.

Allophane is not the only amorphous mineral in tropical soils. Limonite and cliachite are others that are believed to be fairly common, at least in the higher rainfall areas. Silica gels also exists as a temporary weathering product. Gibbsite and limonite should be identifiable from differential thermal analysis, but the silica gels would probably not be detected except by chemical analysis. Weaver (1974) used chemical procedures to analyze some soils from Brasilia. Amorphous material was detected which be believed could be considered as "unstable allophane". However, he did not believe it should be applied in this case because of probable confusion over the term. The amount of amorphous material he detected was sizeable and in line with the findings of other chemical analyses of highly weathered soils.

As far as engineering applications are concerned it is useful to consider amorphous materials to be allophane only in volcanic soils where the most problems arise. The amorphous material in the Brasilia samples, for example, has no discernible effect on the engineering properties and need not be considered allophane. It is useful, however, to recognize that all soils that occur over the volcanics in question may have allophane whether hydrated or dehydrated. The hydrated variety should be suspected at all times when dealing with soils over volcanics in high rainfall areas. The first indication of the presence of allophane in these areas is the characteristicly high natural moisture content of the soil.

Halloysite

Halloysite has been identified in Central American and African soils and in both locations it was found to change properties with drying. The changes in the Kenyan Sasamua clay as first described by Terzaghi (1958), is the best documented. Newill (1961), in his detailed laboratory

investigation of two red clays from Kenya, determined that for a Sasamua sample which contained hydrated halloysite, air drying increased the maximum density and lowered the optimum moisture content; but for a Kabete sample which contained metahalloysite, air drying did not significantly affect the moisture density curve. The change in density and optimum moisture content was attributed to the interlayer water in the hydrated halloysite. Lamb and Martin (1955) proposed a correction for the interlayer water in halloysite. However, the correction can not be used without first knowing the percentage of halloysite involved.

Halloysite exists in two forms, hydrated halloysite, $Al_2O_3 . 2SiO_2 . 4H_2O$ and metahalloysite, $Al_2O_3 . 2SiO_2 . 2H_2O$. The stability curve for halloysite, shown in Figure 4.2 indicates that this mineral only forms under water or in high humidity environments. With a loss of water in a drier environment, it readily alters to metahalloysite which can be difficult to distinguish from kaolinite. In fact it is highly likely that much of the kaolinite reported in connection with the weathering of feldspars is actually metahalloysite which was formed by the desiccation of hydrated halloysite (Loughnan, 1969). Hydrated halloysite dehydrates when the relative humidity drops below 40% or the moisture content drops below 10% (Lamb and Martin, 1955). Therefore, hydrated halloysite can exist only in a permanently moist environment.

Halloysite has been reported in a red clay found on volcanic extrusive rocks on the island of Fernando Po (Salas, 1963). Changes of property with drying were also noted in this location: the mineral form is undoubtedly hydrated because it changes properties upon drying and because it has a high natural moisture content, over 40%.

Metahalloysite was identified in the following African soils during the course of the Phase II study. They were soils found over basic volcanic rocks from West Cameroon, Ethiopia and Kenya. A soil found over volcanic ash in southern Tanzania was not analyzed but was believed to contain halloysite and possibly allophane. It was reported that laboratory tests did not reveal the same soil properties which were evident during construction.

Halloysite was positively identified in several of the volcanic soils of Central America. Although not positively identified halloysite probably occurs in the ferralsols and acrisols in Brazil and South America. Its location is most likely where the ferralsols and acrisols have formed over volcanic materials such as the basalts of the Parana Basin. Apparently, hydrated halloysite is relatively rare in the tropics over rocks that are not volcanic.

Halloysite can be considered an engineering problem only when it exists in its hydrated form which is the only form associated with changes of properties. The mineral usually occurs only over basic rocks, the basic intrusives and extrusives of southern Brazil, the Andes, Central America, the Caribbean Islands, Cameroon and eastern Africa. In these areas the susceptibility of the soils to changes in moisture should be ascertained, especially in locations where the rainfall exceeds 1,500 mm (60 in) per year.

It has been reported recently that andosols from Java contain allophane and display large changes of properties with drying whereas the latosols, in which halloysite is the principal mineral, show only relatively slight changes of properties with drying (Wesley, 1973b). It may well be that earlier literature reports of samples which exhibited large changes of properties were actually composed of undetected allophane as well as halloysite. Such samples were subject to mineralogical analyses which would not detect the amorphous material. Allophane can be identified through chemical analysis. However, due to its variation in composition and the harsh treatment during the analysis, some investigators even question the reliability of chemical tests.

Gibbsite

Gibbsite is a common clay material found in lateritic soils. Frost (1967) has attributed the changes in properties of soils obtained in New Guinea and New Zealand to this mineral. Gibbsite has the chemical formula $Al_2(OH)_3$. It is possible that the loss of hydroxyl radicals occurs in moist soils with air-drying or perhaps gibbsite has attached but non crystalline water which is lost on air drying. However, it is doubtful if there would be sufficiently large quantities to cause large changes in properties. It is believed that changes reported by Frost are not due only to gibbsite, but to associated hydrated halloysite and allophane.

Gibbsite was identified in only a few African soils. These are all ferrallitic soils. Nonetheless is believed to occur in all ferrallitic soils even though it is undetected. This agrees with D'Hoore (1964) in his definition of ferrallitic soils, and also with the findings of Delvigne (1965).

Considerable testing was performed on the ferralsols of Brazil and ferrallitic soils of Africa to determine if changes in properties occur. Some of these samples may have been at an insufficient depth to include material that has not been subject to seasonal air drying. However, many samples were recovered from auger borings to depths of 2.5 m (8.2 ft) or more. Most of the changes were not particularly significant. Those samples displaying significant changes were mostly from areas of over 1,500 mm (60 in) annual rainfall. Gibbsite, or partially hydrated gibbsite may be the cause of these changes or aggregation with dehydration of both iron and alumina minerals may be the cause.

Gibbsite was identified in about 75% of all the ferralsols and acrisols analyzed. It was also detected in most of the luvisols which may indicate that many of these soils were misclassified. It was far less common in the arenosols and nitosols, as would be expected.

Gibbsite occurs in Central America, the Caribbean Islands and Southeast Asia, in soils which are comparable to the ferralsols and acrisols. It is commonly found in red residual soils in areas which have over 1,500 mm (60 in) of rainfall per year.

Properties and Discussion

The properties of some soils formed over volcanic rocks in Central America, Java and New Guinea are given in Table 4.1. The samples collected in Central America during the course of the Phase III study were augmented by

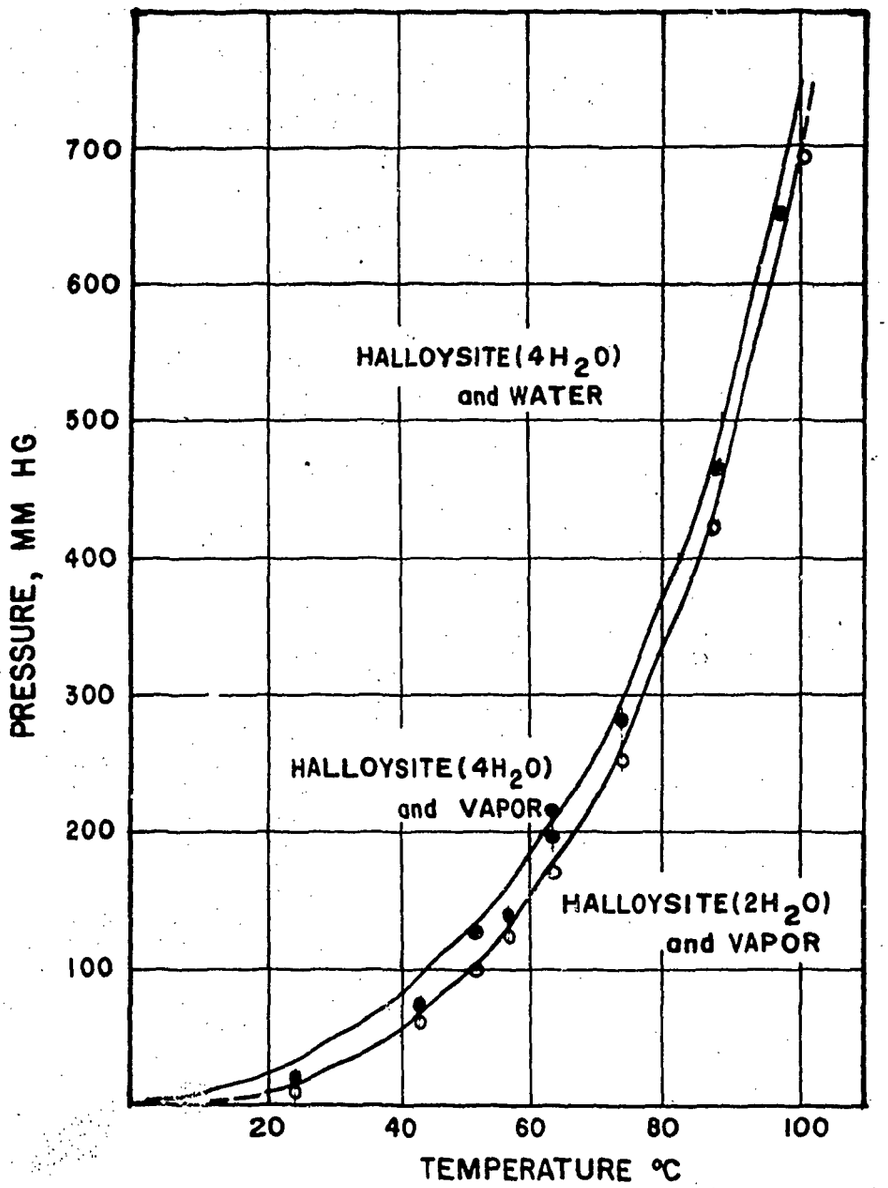

FIGURE 4.2 – RELATIONSHIP OF HALLOYSITE

upper curve is vaporization curve of water
lower curve is boundary between halloysite ($4H_2O$) and halloysite ($2H_2O$) (after Bates, 1952)

TABLE 4.1

Properties of Soils Over Volcanic Rocks, Central America, Java and New Guinea
(Samples Tested Without Air Drying Given First;
Samples Tested After Air Drying Given Second in Parentheses)

Location	*Geology*	*Soil Type*	Moisture Content	– No 200 Sieve	– 2μ (Interpreted)	*LL*	*PI*	*SG*	MDD gm/cm³/pcf	*OMC*	*Mineralogy*
GUATEMALA											
near Parulha	serpentine	andosol	73%		50 (23)	64 (60)	14 (3)				gibbsite, halloysite,
near Antigua	pyroclastics	?			17 (14)	41 (NP)	11 (NP)				allophane (?) goethite
HONDURAS(1)											
near Lagoa de Yojoa	pyroclastics	andosol	70-110			(40-69)	(3-25)				
NICARAGUA											
near Santo Tomas	volcanics	latosol			57 (36)	86 (70)	45 (33)				
COSTA RICA(2)											
San Juan	pyroclastics	andosol	66	97 (90)		81 (56)	29 (19)		.920(1.207)/57.41(75.32)	69.5(40.4)	halloysite, gibb. allophane?
San Juan	pyroclastics	andosol	79	97 (94)		93 (66)	25 (19)		.905(1.103)/56.47(68.83)	69.2(56.9)	halloysite, gibb. allophane?
PANAMA											
east of Panama City	basalt, ash	?			43 (27)	48 (44)	15 (5)				
NEW GUINEA(3)											
Highlands of ‡1	volcanics		130			135 (NP)	55 (NP)				hydrated halloysite with
	with ash	*andosol*				*145 (NP)*	*70 (NP)*				*gibbsite; allophane?*
Papua ‡2	volcanics										allophane & vermiculite
	with ash	andosol	85			160 (NP)	85 (NP)				allophane & vermiculite
‡3	volcanics										
	with ash	andosol	130			110 (NP)	35 (NP)				gibbsite w/hydrated
‡4	limestone										halloysite; allophane
	with ash	andosol	90								
NEW GUINEA(4)											
OTHERS											
Bagema	volcanics	latosol?	46		49 (48)	99 (84)	66 (48)				mostly halloysite
Mati	volcanics	?	72		52 (37)	123 (76)	72 (26)		OTHER PROPERTIES		mostly halloysite
Garoka 30 cm (12 in)	volcanics	?	44			97 (73)	55 (31)		IMPROVED AS WELL		mostly halloysite
1.2 m (48 in)			60			95 (70)	41 (11)				mostly halloysite
Mendi 15 cm (6 in)	volcanics	andosol?	52		20 (3)	NP	NP		DISTURBE AS LITTLE		allophane & gibbsite
23 cm (9 in)			89		36 (2)	133 (NP)	54 (NP)		AS POSSIBLE DURING		allophane & gibbsite
40 cm (16 in)			102		28 (1)	81 (NP)	15 (NP)		CONSTRUCTION		allophane & gibbsite
JAVA(5)											
near Bandung, R_1	andesitic conglomerate	latosol	31	95 (95)	73 (73)	76 (71)	40 (36)	2.78(2.78)	1.313(1.340)/81.93(83.62)	37(35.5)	halloysite & kaolinite
near Bandung, R_2	andesitic conglomerate	latosol & andosol	51	98 (98)	77 (84)	94 (85)	42 (34)	2.76(2.76)	1.084(1.135)/67.64(70.82)	51.5(48)	halloysite & kaolinite
near Jakarta, R_3	tuffaceous sandstone	latosol	50	98 (98)	83 (86)	101 (102)	57 (58)	2.75(2.73)	1.170(1.200)/73.01(74.88)	46.5(42)	halloysite & kaolinite
near Bandung, Y	tuff	andosol	146	89 (67)	* (17)	159 (110)	27 (15)	2.76(2.71)	0.555(0.692)/3463(43.18)	127(95)	allophane & halloysite
near Bandung, Y_2	ash?	andosol	180	96 (14)	* (2)	184 (79)	38 (6)	2.75(2.73)	0.545(0.875)/34.01(54.60)	135(68)	allophane & halloysite
near Semarang, Y_3	ash	andosol	128	97 (29)	* (6)	213 (86)	46 (8)	2.58(2.65)	0.450(0.880)/28.08(54.91)	160(70)	allophane & halloysite
JAVA (OTHER)(6)											
Tjiponund jang Dam TJ1	tuff	andosol	128	96 (37)	65 (11)	165 (60)	46 (7)	2.80(2.78)	0.61(0.96)/38.06(59.90)	120(55)	allophane rich
Dam TJ2	tuff	andosol	68	96 (94)	76 (69)	95 (74)	30 (17)	2.88(2.89)	0.965(1.084)/60.22(67.64)	64(53)	halloysite rich

(1) Arrivillaga, 1963
(2) Courtesy of Min. of Public Works
(3) Wallace, 1973
(4) Frost, 1968
(5) Wesley, 1973b
(6) Wesley, 1974
* difficult to disperse

reports in the literature on similar soils containing allophane from Honduras, Java and New Guinea. All of these are believed to be andosols. Test results on an andosol from Costa Rica were provided by the Materials Laboratory of the Ministry of Public Works in Costa Rica.

Latosols from Java are included for purposes of comparison. It is apparent that, among the soils reported, major property changes occurred with those containing allophane.

Altitude is important in the formation of allophane over volcanic material. Andosols with allophane occur above 1,100 or 1,200 m (3,600 or 3,950 ft) over volcanic rocks of andesitic composition in Java. There is far less allophane between 600 and 1,100 m (1,960 and 3,600 ft) and the soils are called latosols. From 300 to 600 m (980 to 1,960 ft) the amount of amorphous material is reduced considerably and below 300 m, (980 ft) kaolinite type clays predominate (Wesley, 1973b). Soil sequences over acidic volcanics do not show property-changes with drying, nor do those over basic volcanics with long dry seasons.

Some andosols, called "humic andosols", have high organic contents. The FAO Map shows several large areas in the Andes with humic andosols, mainly Peru, Colombia and Ecuador. These have a highly organic A horizon, 15% or even more. Material from this horizon is sometimes dried for fuel since it is similar to peat.

The moisture contents and index properties do not provide an accurate picture of the strength properties that the non-humic andosols display. They are surprisingly stable in cut slopes and in highway embankments provided they are not disturbed. Although the sensitivities are not usually very high, construction equipment bogs down easily.

From Table 4.1 it can be seen that the moisture contents are usually well over 50% for the andosols and generally moisture contents are under 50% for the latosols. Andosols apparently contain far more allophane than halloysite while the reverse is true in the latosols reported over volcanic rocks. It is also obvious that the changes in properties with drying are generally far greater in the andosols.

The Casagrande chart, Figure 4.3, illustrates the great changes which occur, in the andosols. The known latosols generally display less change and these tend to be parallel to the A-line. The only sample that lies above the A-line is a latosol.

Almost all volcanic soils containing appreciable halloysite and allophane are plotted below the A-line. The more allophane, the greater the distance becomes. This is a distinguishing characteristic since these soils are mostly highly plastic clays with little organic matter. On the other hand inorganic silts or organic clays are the soils that generally fall below the A-line. This indicates the strong tendency for these soils to form aggregations and behave as silts.

Highway construction procedures for such soils in Hawaii have been described by Hirashima, (1951). Shallow excavation should be handled by bulldozer, only when it is possible to do so in one pass. All other excavations should be handled by drag-line or shovel. Exposed subgrade should be covered with a minimum of 45 cm (18 in) of select material. This should serve as a travel-way for construction vehicles and also as the sub-base. Embankments should be constructed with a 45 cm (18 in) rocky layer placed after every 1.5 m (5 ft) of soil to serve as a travel-way. The compaction should only be that which occurs from construction traffic. End-dumping should be permissible for low, short embankments only.

THE RED SOILS OF SOUTH AMERICA AND CENTRAL AMERICA

General

Most of the Phase III study samples were collected in Brazil. The locations of over 200 samples collected in Brazil are shown in Figure 4.4 In addition the location of another 25 samples collected in other South American countries are shown in Figure 4.5. Most samples were collected from the B horizon at depths of 1.5 m to 2.5 m (5 ft to 8 ft) in an attempt to sample the non-desiccated soils below the zone of seasonal moisture fluctuations.

Many of these were tested for physical properties without air-drying and again after air drying. Samples were sealed in plastic bags and returned to the various district laboratories for testing. A fraction of each sample was sent to the central DNER laboratory in Rio for additional testing and for dispatching to the "Comissão de Pesquisas e Recursos Minerais" for chemical testing. An additional fraction was sent to the University of Florida for chemical and mineralogical testing.

Only a few index properties were determined for the samples obtained from other South American countries. Some mineralogical analyses were also performed to establish similarities and differences with the Brazilian counterpart soil types.

Chemical and geological properties for the ferralsols acrisols, arenosols, luvisols and nitosols are given in the Appendix to this chapter. The physical and engineering properties are given in the appendix as well (Tables A4.1 and A4.2). The mean values for the individual soil groups are provided in Table 4.2. The symbols for the soil subgroups are as follows: ferralsols are identified as Fa, Fh, Fr, Fo and Fx; acrisols as Ao and Ap; arenosols as Q_f; luvisols as Lc and Lf; and nitosols as Ne and Nd. The first letter indicates the soil group while the second indicates the subgroup.

Chemical Properties

Partial chemical analyses were performed by X-ray fluorescence. The silica, alumina, iron, and base contents were determined. Total silica and free silica (quartz) were determined from many samples and the loss on ignition (water of crystallization) was determined for about one half of the total samples. Chemical analyses were performed on the total sample instead of the more frequently used procedure of analyzing the clay fraction separately. This was done in order to expedite the testing of the large number of samples involved. This analysis of the total sample is as an acceptable practice for fine grained soils but the analyses of the clay fraction is generally preferred for

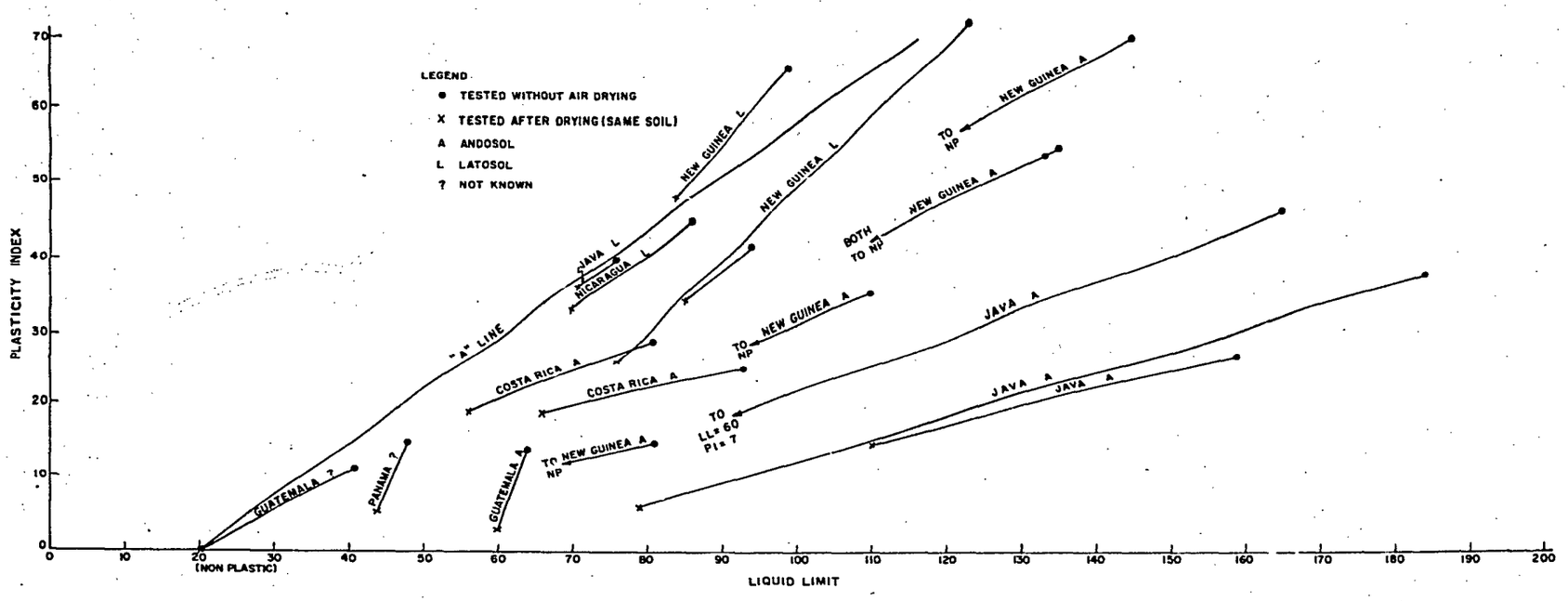

FIGURE 4.3 – LOCATIONS OF SOME OF THE VOLCANIC SOILS ON THE CASAGRANDE CHART

FIGURE 4.4 – LOCATION OF TROPICAL RED SOIL SAMPLES IN BRAZIL

FIGURE 4.5 – SAMPLE LOCATIONS FOR PHASE III, SOUTH AMERICA

TABLE 4.2
Mean Values of Properties, Read Tropical Soils of South America

	Ferralsol	Acrisols	Arenosol	Luvisol	Nitosols
LL	47	42	26	28	56
PL	30	26	18	22	34
PI	18	16	8	8	21
– 200	62	59	26	48	73
– 2μ	35	35	11	27	43
SILT/CLAY*	0.84	0.91	0.98	0.90	0.68
OMC	18	15	9	11	23
MDD**	1,974	1,885	2,105	1,980	1,669
CBR	24	23	84	37	23
Activity	0.51	0.46	0.73	0.30	0.49
SG	2.86	2.73	2.80	2.75	2.75
RAIN (mm)	1502	1447	1172	1178	–
Moisture Deficient Months	1.5	3.4	6.1	6.5	–
p^H	4.82	4.11	4.41	4.73	4.56
K_R ***	0.93	1.48	1.50	1.46	–
SiO_2/Al_2O_3	1.22	1.83	1.66	1.75	–
FREE Si	27.4	47.4	51.4	43.5	–
Total Si	45.7	60.3	65.5	60.2	–
Fe_2O_3	18.2	9.05	15.0	8.8	–
Al_2O_3	20.8	16.8	10.9	16.3	–
WPI ****	– 47.8	–21.5	–16.4	–18.6	–41.5

* Silt-clay ratio
** AASHO Modified Maximum Dry Density
*** Silica Sesquioxide Ratio
**** Weathering Potential Index

TABLE 4.3
Minerals Identified in Each of the Major Soil Groups (Expressed in Percentage of Occurrence), Red Tropical Soils of South America

	Kaolinite	Montmorillonite	Illite	Chlorite	Quartz	Feldspar	Gibbsite	Goethite	Hematite	Magnetite
ACRISOLS (9 samples analyzed)	100	11	56	22	100	0	78	22	22	0
ARENOSOLS (12)	92	17	8	33	100	25	17	25	33	0
FERRASOLS (26)	100	13	13	38	96	0	73	50	77	23
LUVISOLS (10)	80	20	30	30	100	60	70	20	40	30

granular soils. This is particularly true for those soils formed over quartz – rich rocks.

Silica-alumina and silica-sesquioxide ratios SiO_2/Al_2O_3 and $SiO_2/Al_2O_3 + Fe_2O_3$ are given in Table 4.2 and in the appendix to this chapter. (Tables A4.3 and A4.4). In calculating these, it was necessary to assume that the total silica is distributed over each soil fraction in proportion to the total quantity of soil retained in that fraction. For example, if a soil contained 25% clay, then 25% of the total silica is assumed to be in clay fraction. While this is an over-simplification, it is considered more indicative of the silica-sesquioxide ratio of the true clay fraction than utilizing the silica content of the total sample.

The mean values of the silica-sesquioxide ratio ranged from 0.93 for the highly weathered ferralsols to 1.50 for the arenosols. Silica-alumina ratios display greater values. These ranged from 1.22 for the ferralsols to 1.83 for the acrisols. These values are in general agreement with those reported in the literature.

The high silica-alumina and silica-sesquioxide ratios for the acrisols appear unusual since in many other respects the acrisols are similar to the ferralsols. This indicates that acrisols, perhaps because of their acidity, retain considerable silica and contain a large amount of gibbsite.

The iron and alumina enrichment was slight in many samples. This was the case with the soils formed over the acidic, quartz-rich rocks, particularly of northeast Brazil. Again, these results are based on the total sample, not the clay fraction; the latter may well have shown a greater concentration of iron and alumina.

Ferralsols and acrisols, as well as some luvisols and arenosols, can be highly enriched with iron or alumina. Examples are the bauxite deposits found in various parts of Brazil and ironstone which are so rich in iron that they could qualify as low-grade iron ores.

The silica contents are, in many cases, extremely high which reflects the high quartz content of the parent material. In almost all cases where the percentages of SiO_2 are over 50%, the rock over which the samples developed is rich in quartz. Typical quartz-rich rocks are granites and sandstones.

Another relationship, which has been called the Weathering Potential index, was also investigated:

$$WPI = \frac{100 \times \text{mols}\,(K_2O + Na_2O + CaO + MgO - H_2O)}{\text{Mols}\,(SiO_2) + Al_2O_3 + Fe_2O_3 + CaO + MgO + Na_2O + K_2O)}$$

The Index was first proposed by Reiche (1950) and later modified by Short (1961). It was recently applied to soil profiles by Plaster and Sherwood (1971). They state that unstable, immature soils have higher percentages of the mobile oxides and lower water contents. As the oxides are removed and the amount of water increases, the WPI becomes less, eventually becomes negative. The greater the negative number, the more mature the soil.

Results of these calculations are provided in Table 4.2. The percentage of bases which may be seen in the full analysis in the Appendix to this chapter is not repeated in Table 4.2. The values shown are those for all analyses in which a loss on ignition was reported.

The high negative mean value for the ferralsols, Table 4.2 indicates that this group is the most highly weathered. On the other hand the nitosol group also appears to be highly weathered, more so than the acrisol, luvisol and arenosol groups even though the nitosol group represents less mature soil. Therefore, the WPI does not appear to be more descriptive than the silica-sesquioxide ratio.

In fact, neither the WPI nor the silica-sesquioxide ratio correlate well with each other or with other properties. There appears to be no particular advantage to using these relationships to classify tropical soils and since these indices require chemical analyses which are costly and time-consuming. It is recommended that they not be used in routine engineering analyses.

Also of interest in Table 4.2 are the mean values of rainfall and the number of months in which there is a moisture deficiency. The statistics show that ferralsols and acrisols are found in areas that have substantially higher rainfall and few moisture-deficient months.

Mineralogy

The mineralogy of the clay fraction was determined by X-ray diffraction, and, for a few samples, by differential thermal analyses. No attempts were made at providing quantitative estimates.

A summary of the occurrences of each of the various minerals within the major soil groups is given in Table 4.3. Quartz is ubiquitous, even over such basic rocks as basalt. As quartz is readily identifiable by X-ray diffraction and is seldom confused with other minerals, it is reasonable to believe that the interpretations are correct. Kaolinite not surprisingly is almost as ubiquitous as quartz. Goethite is more common in the ferralsols than in the other groups, although it would be expected to be more common in the luvisols. This is true as well of hematite. The hematite must represent near-surface or exposed soil that has become desiccated.

A summary of all X-ray diffractions performed during this part of the study are included in the Appendix to this chapter (Table A4.4). Diffraction patterns are not included; only the interpretation as provided by Professor Eades of the University of Florida are presented.

Physical and Engineering Properties

The mean values of the physical and engineering properties are also given in Table 4.2. The results are for analyses of gradation, Atterberg limits, activity, silt-clay ratio, specific gravity, moisture-density relationships and CBR. Although there is considerable scatter and overlap, differences are detectable between the soil groups. The ferralsols and acrisols have similar mean values of LL, PL, PI, -200, -2 μ and CBR. The arenosols have mean values similar to the luvisols for LL, PL, PI although they contain far less fines and have far greater CBR values.

The grading envelopes for each soil groups are shown in Figures 4.6, 4.7, 4.8, 4.9 and 4.10 for the ferralsols, acrisols, arenosols, luvisols and nitosols respectively.

There is very little difference in the mean values of the activity and the silt-clay ratio among most of the soil groups. Activity provides some discrimination between the

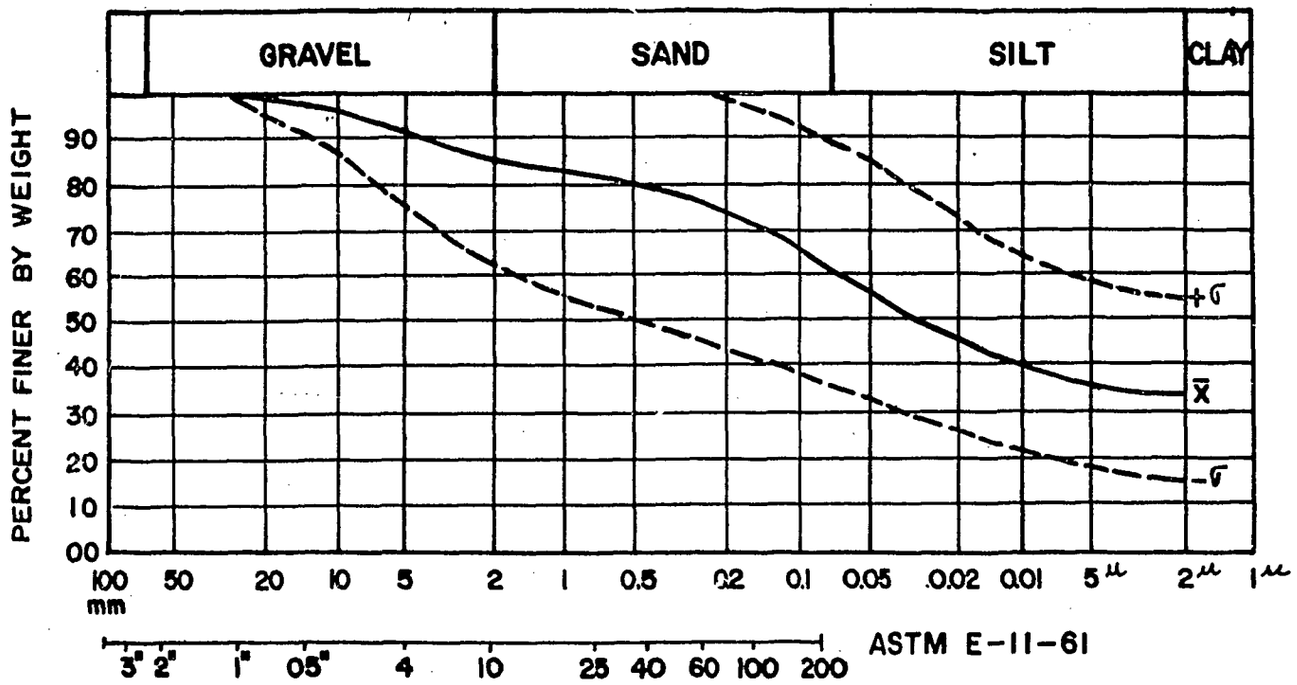

FIGURE 4.6 – FERRASOLS OF SOUTH AMERICA GRADATION LIMITS

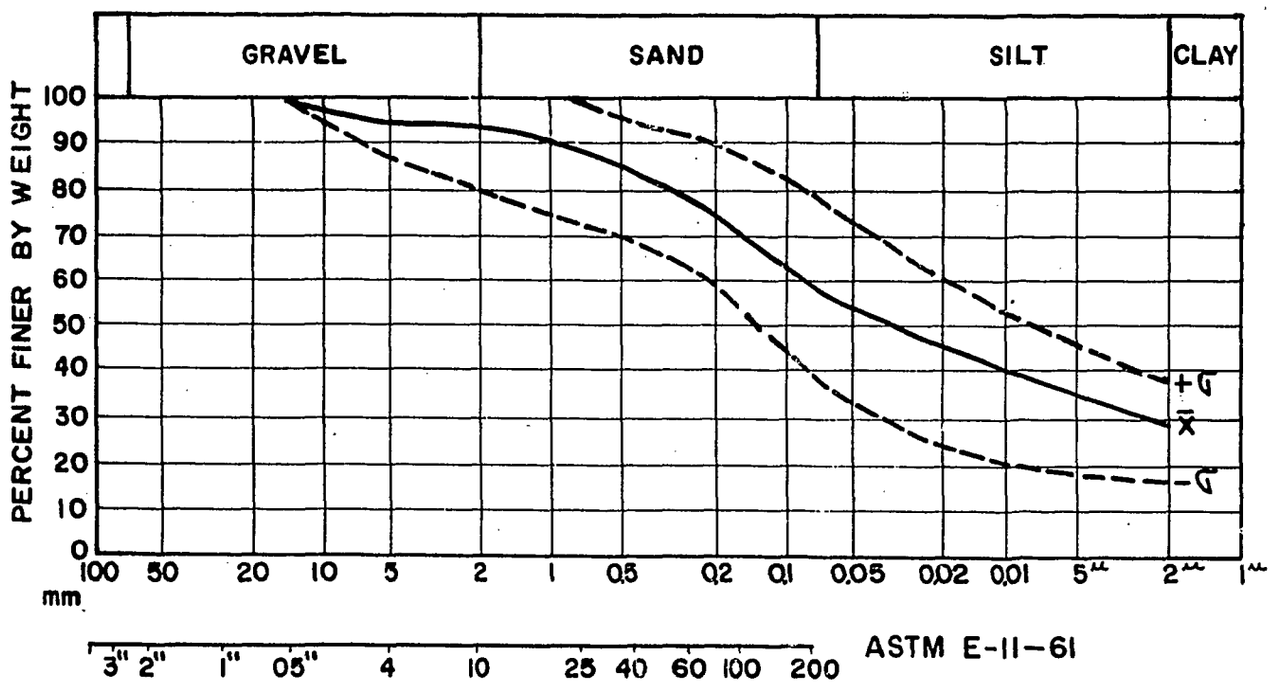

FIGURE 4.7 – ACRISOLS OF SOUTH AMERICA GRADATION LIMITS

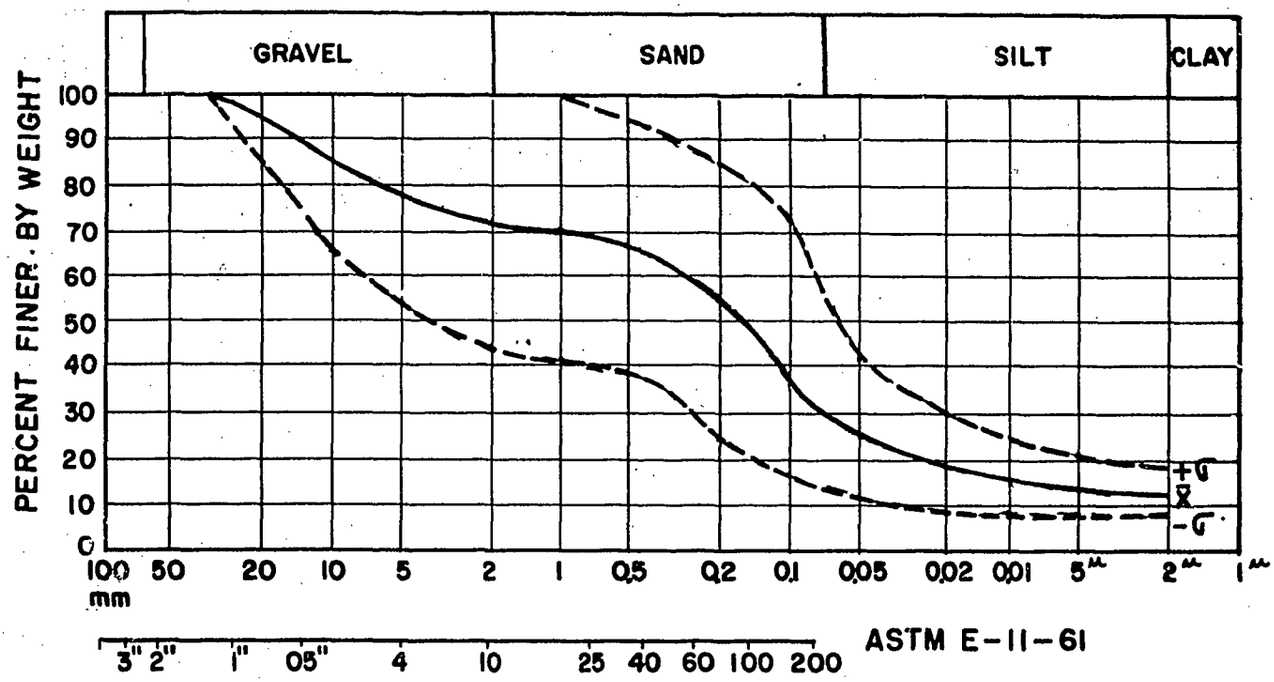

FIGURE 4.8 – ARENOSOLS OF SOUTH AMERICA GRADATION LIMITS

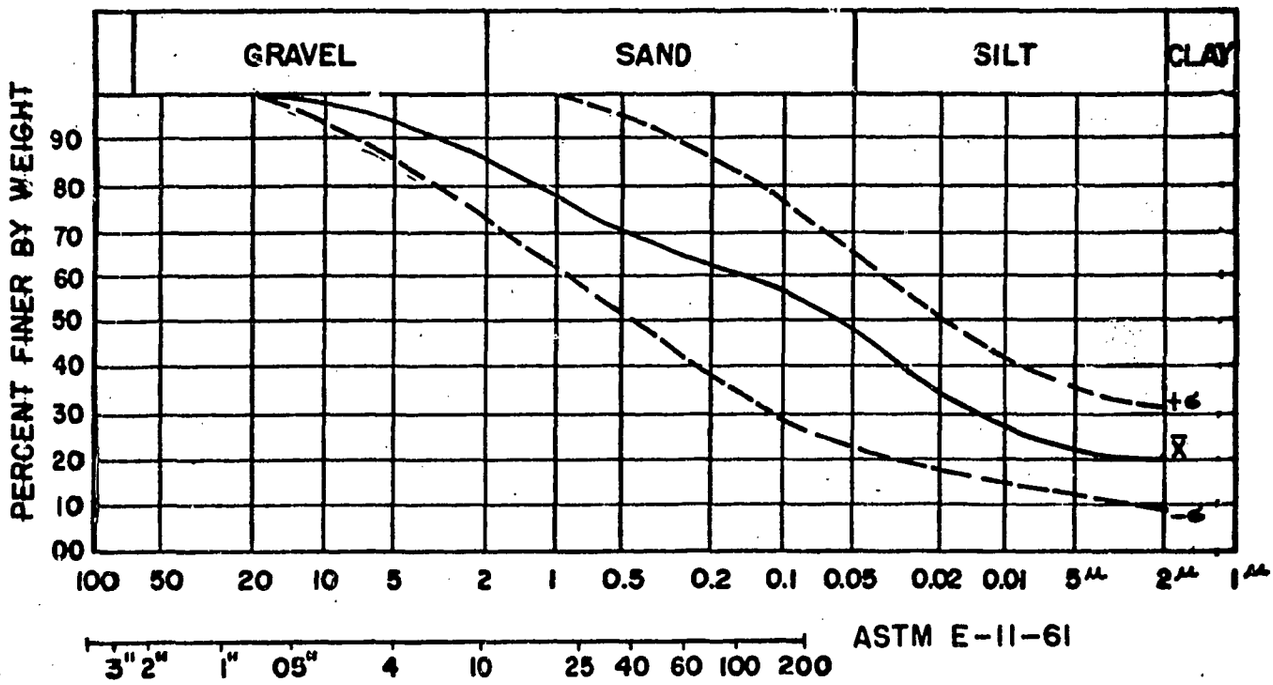

FIGURE 4.9 – LUVISOLS OF SOUTH AMERICA GRADATION LIMITS

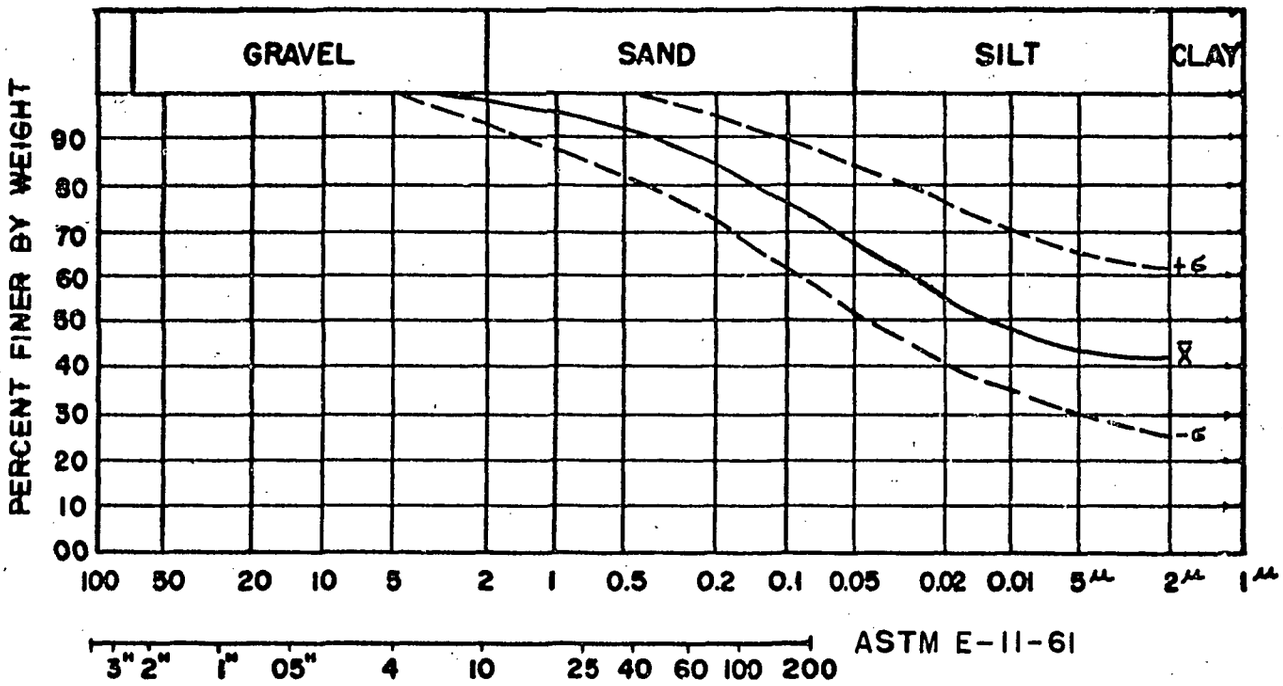

FIGURE 4.10 – NITOSOLS OF SOUTH AMERICA GRADATION LIMITS

nitosols and the remaining groups, but this is all, Also the mean values of the silt-clay ratios are extremely high considering that values reported by pedologists are usually about 0.3 or less.

Silt-clay ratios are determined by means of a different test procedure than the mechanical analysis which is normally employed by engineers. Soils prepared for mechanical analysis are dispersed with a dispersing agent and agitated in a high-speed mixer. This is done only once. Many tropical red soils are difficult to disperse, especially those containing allophane. Many soils apparently re-floculate in spite of the dispersant and the mixing. This is the reason that clay determinations are often unreliable and difficult to reproduce.

Pedologists use repeated dispersions after which the soil particles are allowed to settle until only clay particles are left in the supernatant suspension in the upper 10 or 20 cc or so of the 1,000 ml graduated cylinder. This amount is siphoned off and the procedure repeated several times until virtually all the clay particles have been removed. This procedure is often used for separating clay particles for x-ray or differential thermal analyses. Complete dispersion may even require (Jackson, 1964) removal of the soluble salts which cause flocculation effects; flocculating effects of divalent cations; cementation effects of organic matter; and, at times, removal of the iron oxides which cause cementation.

While this procedure for dispersing the clays is tedious, it is not particularly difficult and may have merit as an indicator of weathering and, very likely, engineering behavior. It is apparent from the mean ratios in Table 4.2 that much of the clay fraction remains undispersed during the standard mechanical analysis, and as a consequence, the ratios calculated from undispersed tests are not meaningful.

Discussions of Results and Correlations

A discriminant analysis was performed on the test results of the major soil groups. The discription of this analysis and the computer program used in the analysis is included in the Appendix to this chapter. The results of the analysis were contradictory. Therefore the test data was reexamined carefully for inconsistencies and the obviously misclassified samples were removed. The data was then subjected to an analysis of variance as described by Liu and Thornburn (1965).

The results of the analysis of variance indicate that ferralsols are significantly different from the arenosols and, in most respects, different from the luvisols, but similar to the acrisols. The luvisols were similar to the arenosols only in their Atterberg limits.

Some relationships and correlations were found to exist among the engineering properties of various soil groups. These were established by regression analyses. These relationships may be useful to the engineer in the selection of materials for highway construction because preliminary conclusions can be drawn based on routine classification tests.

The first relationship to be examined is that between liquid limit and plasticity index of the tropical red soils. The relation is customarily shown on the Casagrande plasticity chart. The results are shown on Figures 4.11,

4.12, 4.13, 4.14, and 4.15 for the ferralsols, acrisols, arenosols, luvisols and nitosols, respectively. All of the soils have been plotted together on Figure 4.16 to illustrate the correlation between these groups. The general equation of the line of "best fit" through regression is as follows:

PI = 0.49 LL - 5.25
Standard deviation (σ) = ± 3.98
Coefficient of correlation (γ) = 0.87
Number of samples (N) = 206

The equations of the individual soil groups are as follows:

PI = 0.45 LL - 3.50 for the ferralsols
PI = 0.47 LL - 3.80 for the acrisols
PI = 0.82 LL - 13.95 for the arenosols
PI = 0.75 LL - 12.70 for the luvisols
PI = 0.23 LL + 7.96 for the nitosols

Most of the soils plot above the A-line or immediately below the A-line. Those that plot far below the A-line are mostly ferrasols and nitosols which have formed over basalt. The implication of this relationship was discussed in the first part of this chapter.

The relationship between optimum moisture content and plastic limit was established and is shown in Figure 4.17. The general equation is:

OMC = 0.61 PL - 0.84
Standard deviation (σ) = ± 3.38
Coefficient of correlation (γ) = 0.858
Number of samples (N) = 157

The relationship between OMC and the clay content was established and is given in Figure 4.18. The general equation is:

$$OMC = 0.34\ (2\mu) + 6.16$$

An increase in specific gravity occurs with increasing iron content. This was predictable and has been demonstrated with lateritic soils. This relationship is shown in Figure 4.19. The trend in Figure 4.19 is apparent even though the results are somewhat scattered.

Figure 4.20 shows the ranges in CBR values for a 95% level of confidence. The use of this figure allows a quick means of providing a rough estimate of the bearing value of a soil when only the classification is known. While the range is very broad for A-6 and A-7-6, it is quite narrow for the coarser grained soils.

Finally, relationships between CBR (at OMC – MDD using AASHO modified compaction with a four day soaking period) and gradation and between CBR and plasticity index are shown in Figures 4.21 and 4.22 respectively. There is no well defined relationship between the CBR and the product (plasticity index times percent passing the No. 200 or 40 sieves). The results can be presented in a broad envelope which indicates higher CBR's at lower values of the product. In Figure 4.21 there is an apparent lower limit for the arenosols while in Figure 4.22 there is an apparent upper limit for acrisols and ferralsols.

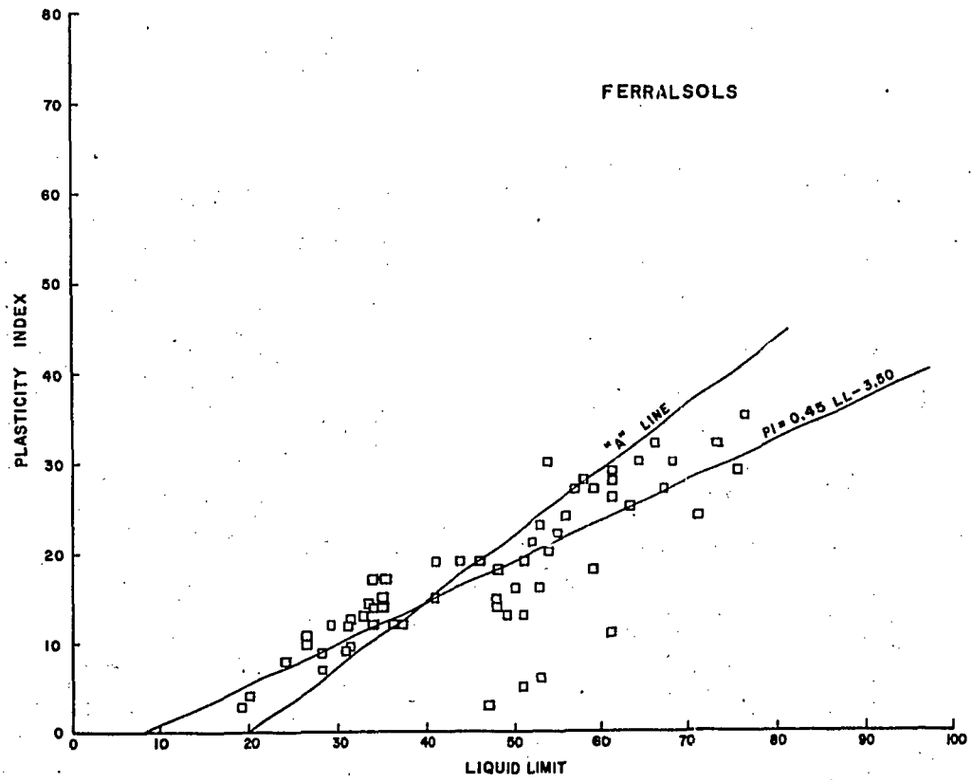

FIGURE 4.11 – FERRALSOLS ON THE CASAGRANDE PLASTICITY CHART, South America

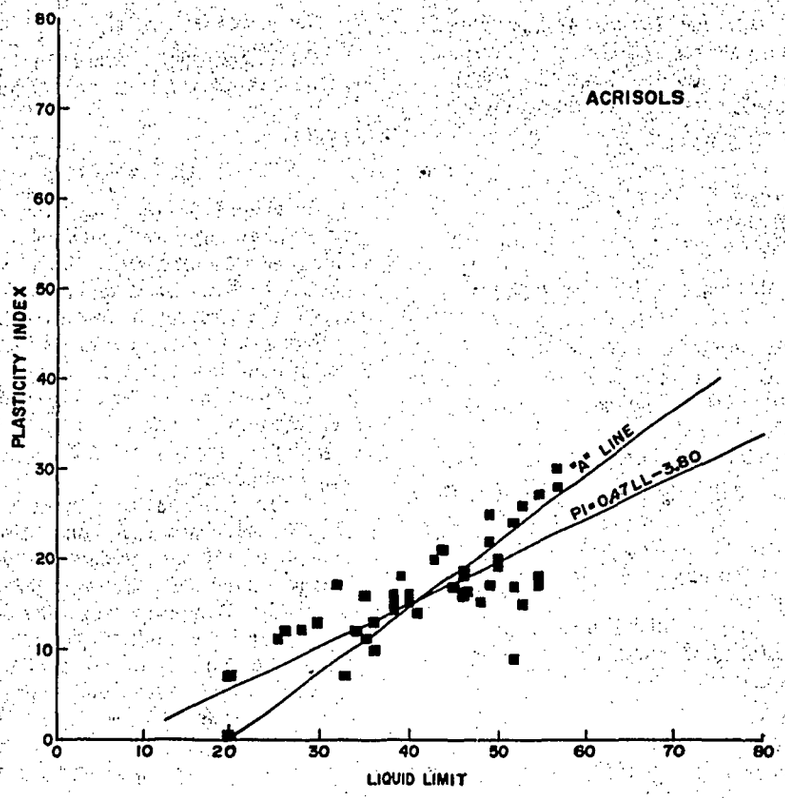

FIGURE 4.12 – ACRISOLS ON THE CASAGRANDE PLASTICITY CHART, South America

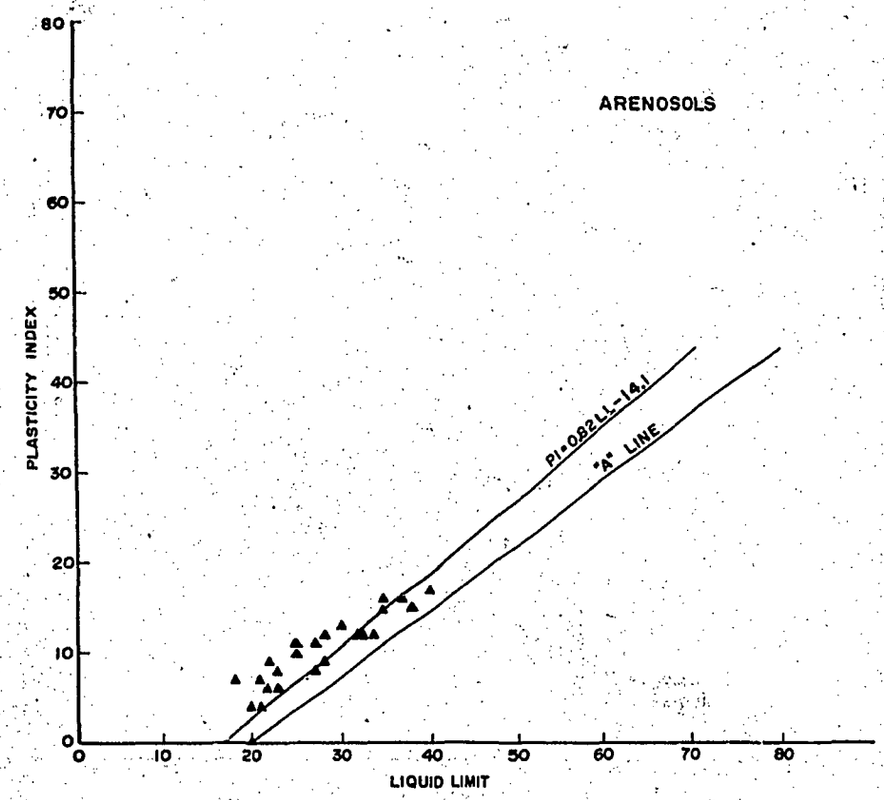

FIGURE 4.13 – ARENOSOLS ON THE CASAGRANDE PLASTICITY CHART, South America

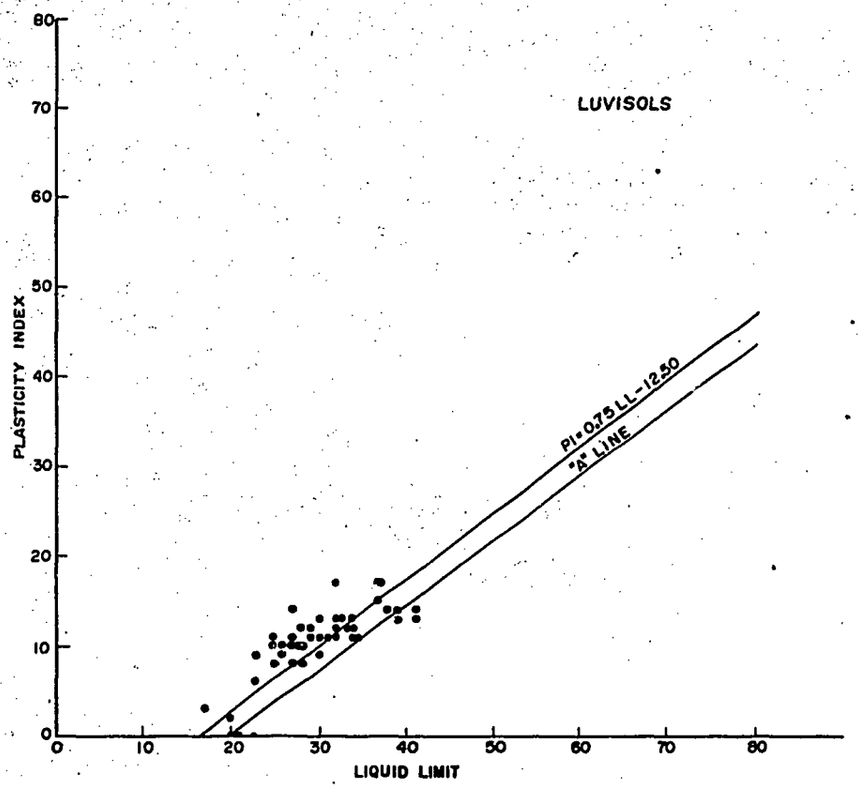

FIGURE 4.14 – LUVISOLS ON THE CASAGRANDE PLASTICITY CHART, South America

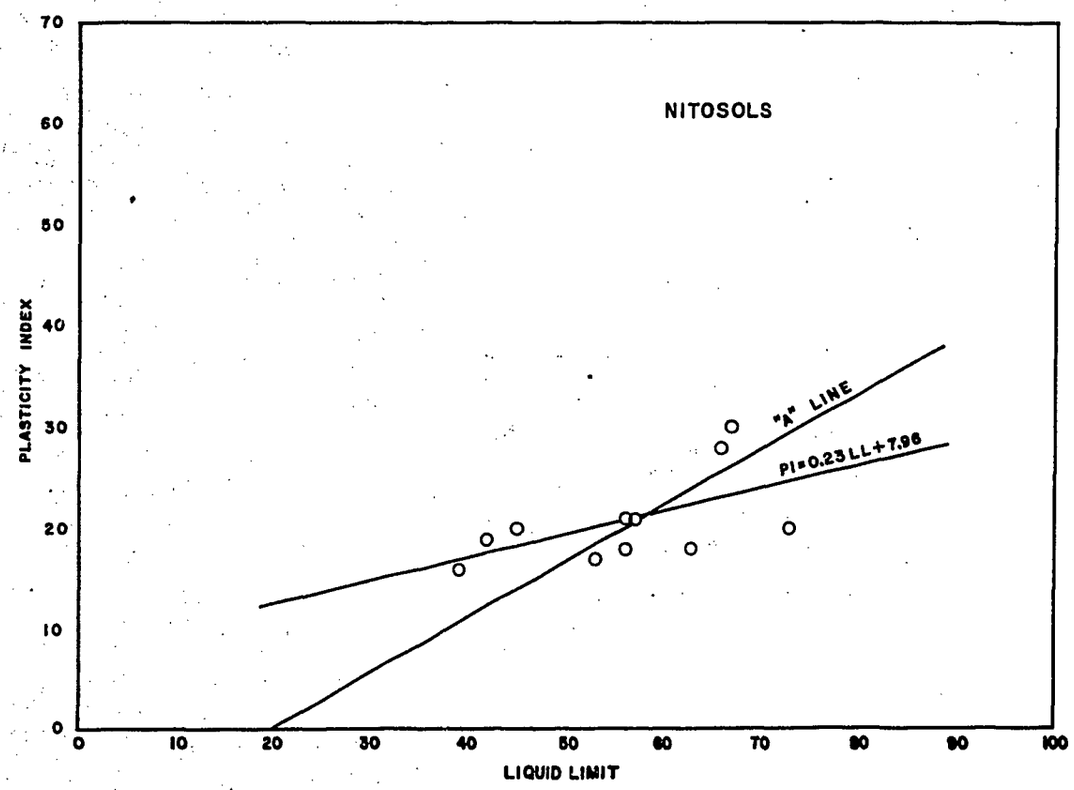

FIGURE 4.15 – NITOSOLS ON THE CASAGRANDE PLASTICITY CHART, South America

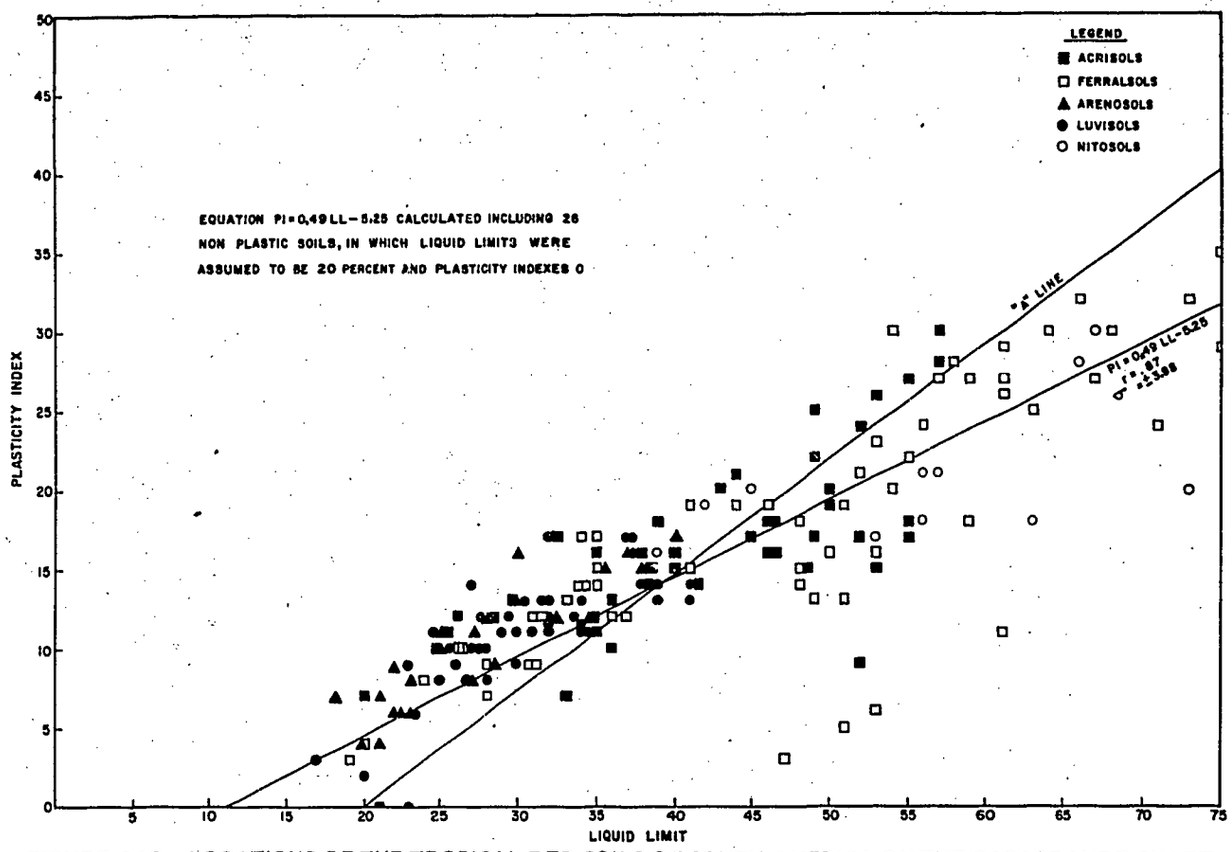

FIGURE 4.16 – LOCATIONS OF THE TROPICAL RED SOILS OF SOUTH AMERICA ON THE CASAGRANDE CHART

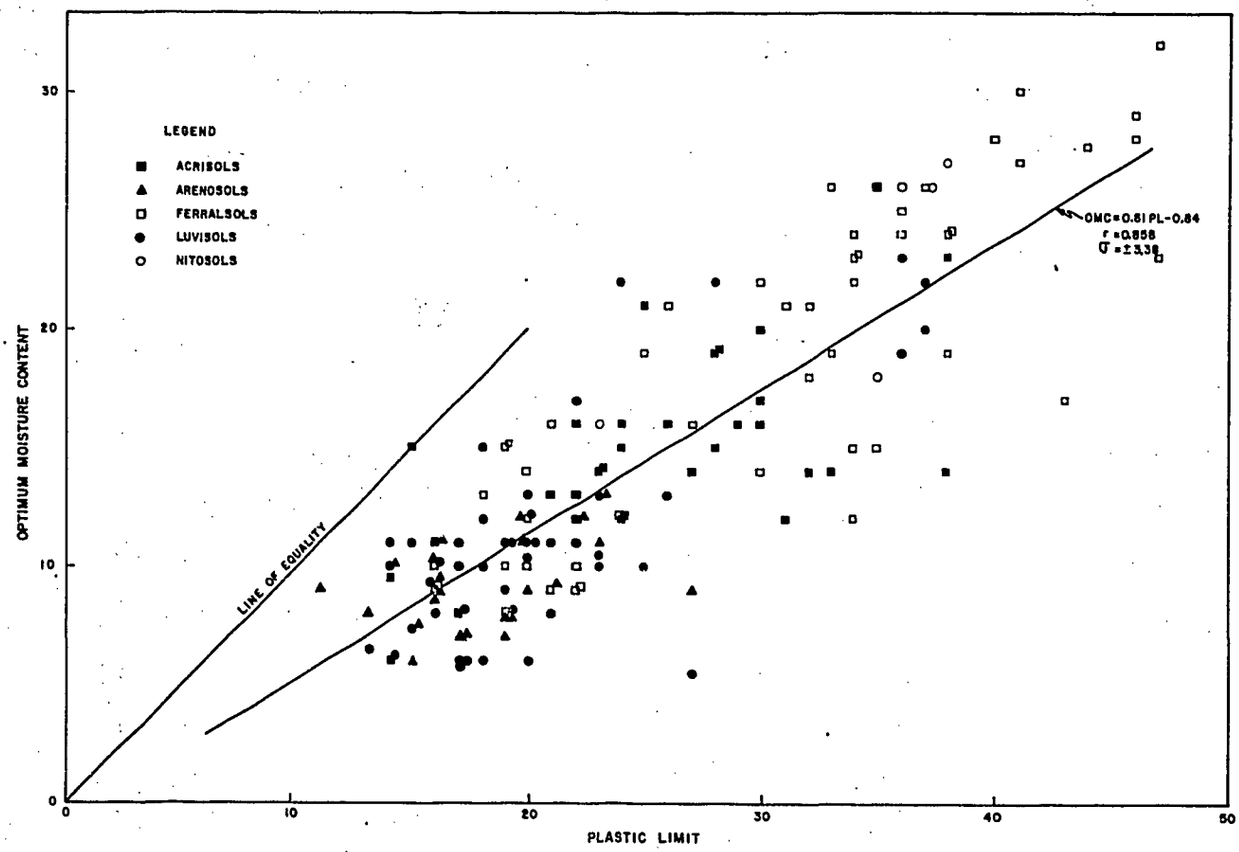

FIGURE 4.17 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND PLASTIC LIMIT, SOUTH AMERICAN SOILS

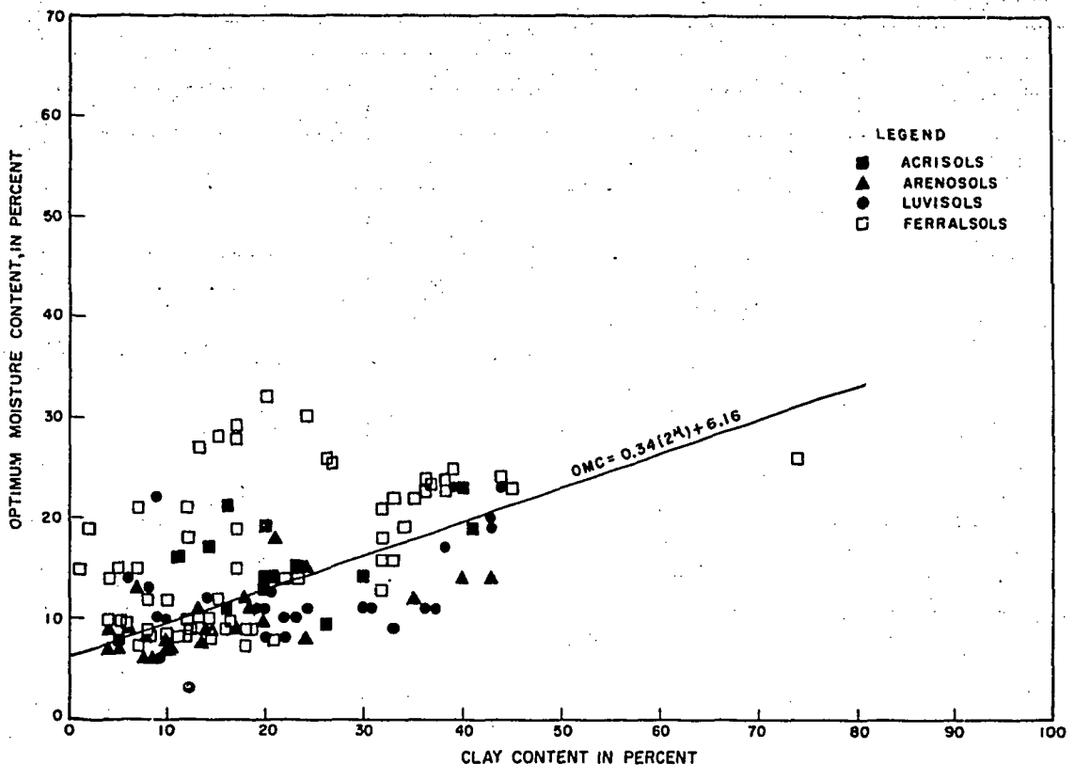

FIGURE 4.18 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE AND CLAY CONTENT, SOUTH AMERICA

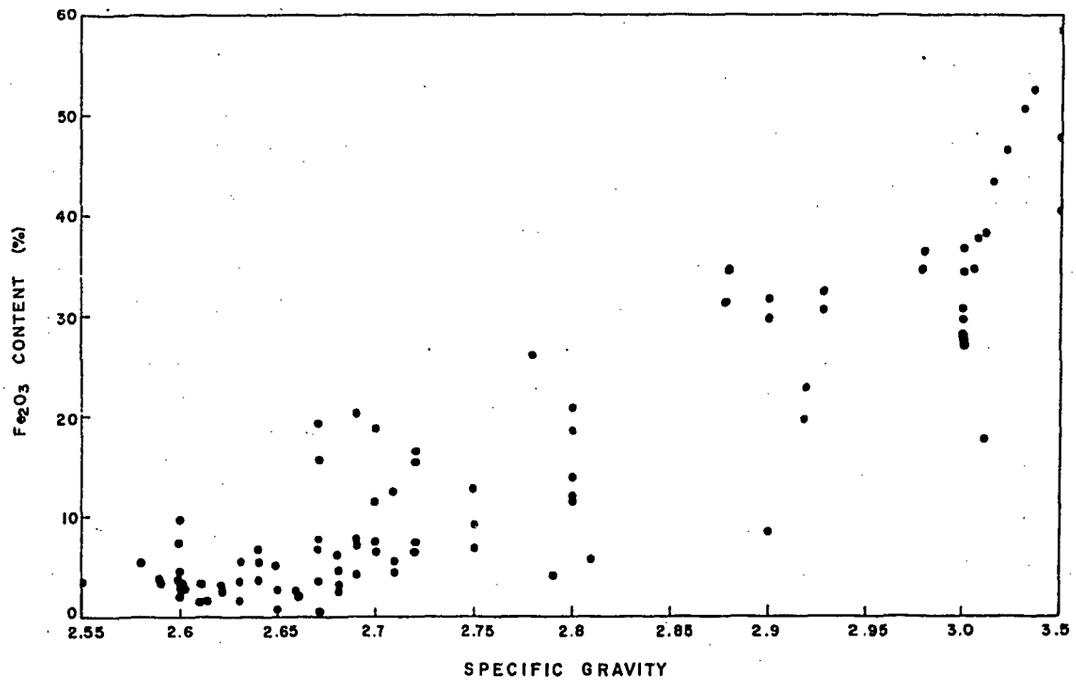

FIGURE 4.19 – RELATIONSHIP BETWEEN SPECIFIC GRAVITY AND Fe_2O_3 CONTENT (%) SOUTH AMERICAN SOILS

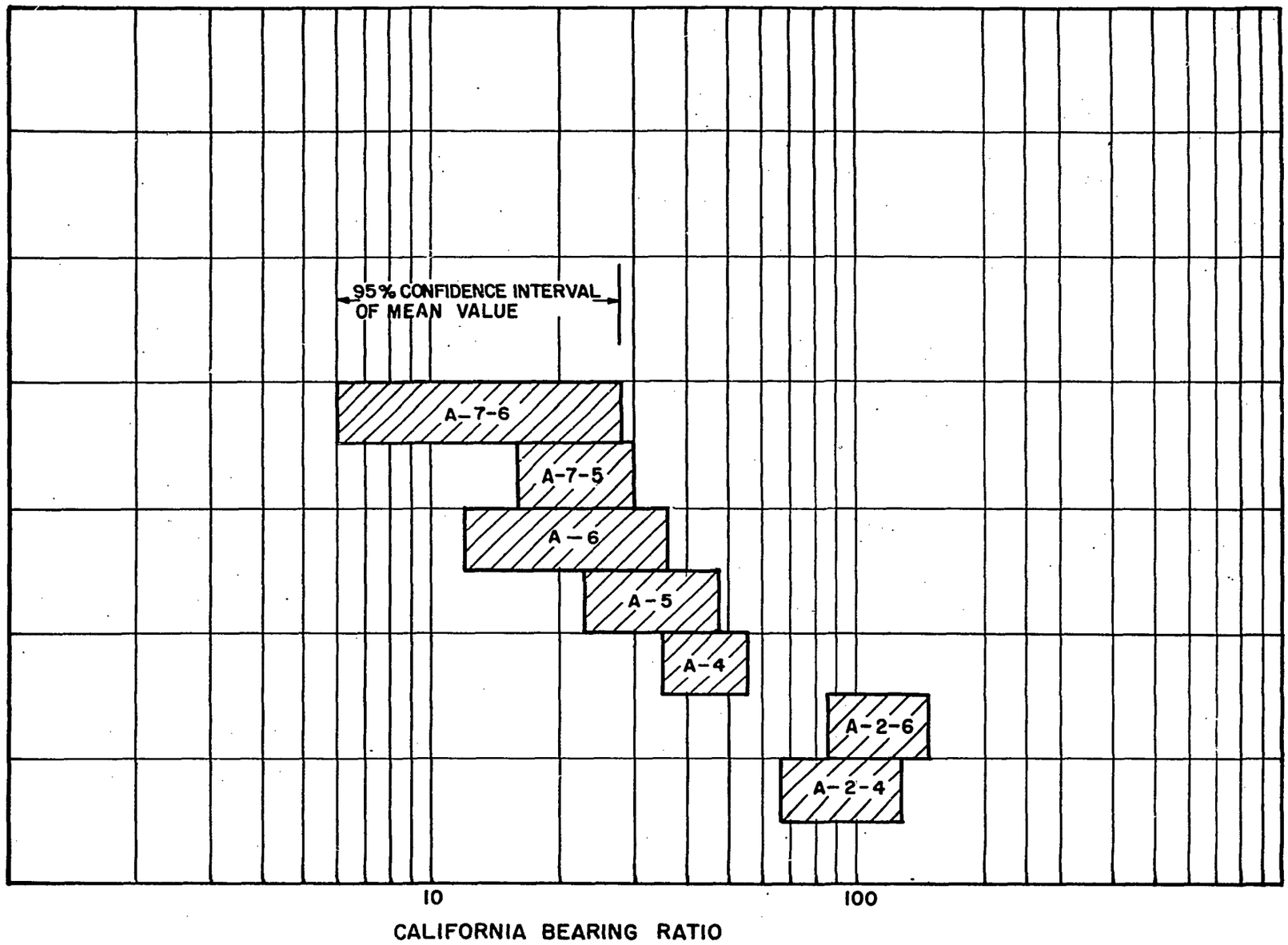

FIGURE 4.20 – GENERAL CORRELATION BETWEEN CBR AND AASHO CLASSIFICATION, FOR RED TROPICAL SOILS OF SOUTH AMERICA

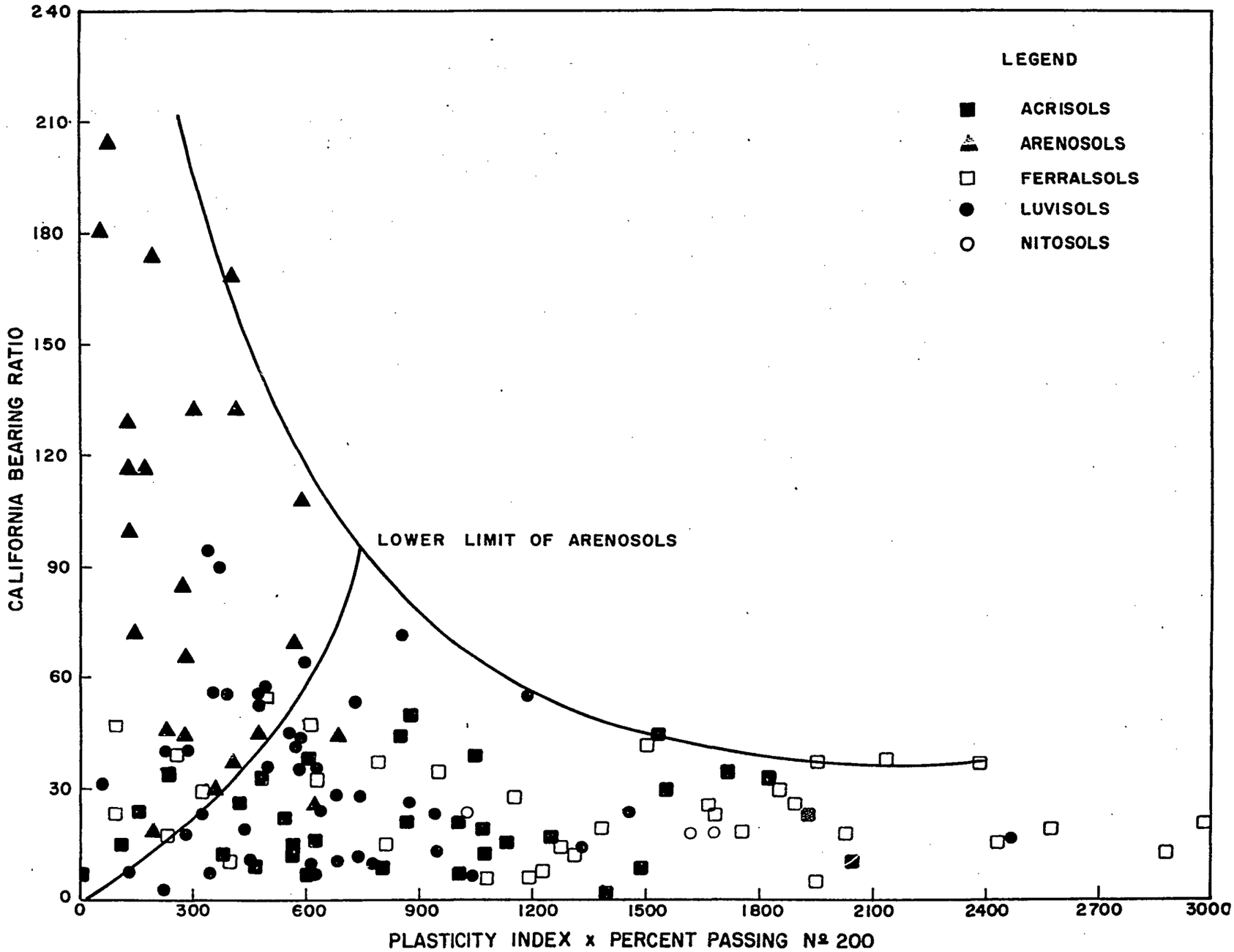

FIGURE 4.21 — CBR vs PLASTICITY INDEX x PERCENTAGE PASSING No. 200, SOUTH AMERICAN SOILS

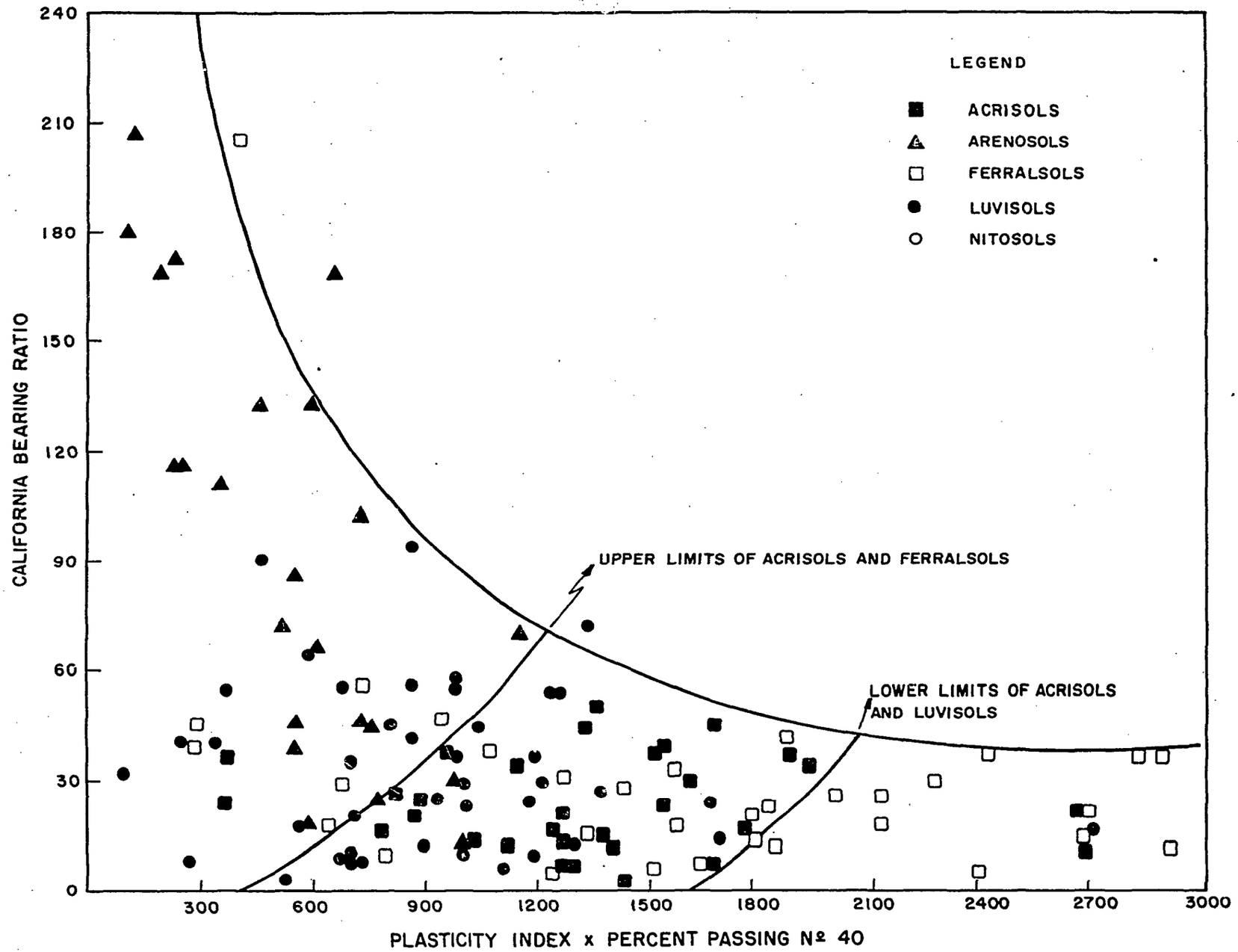

FIGURE 4.22 — CBR vs PLASTICITY INDEX x PERCENTAGE PASSING No. 40, SOUTH AMERICAN SOILS

RED SOILS OF AFRICA

General

It was found during the Phase II Study in Africa (Lyon Associates, Inc. et al., 1971) that the red tropical soils could be divided into three very broad groups. One group is the ferruginous soils which form in warm climates where the annual rainfall ranges from 600 to 1,800 mm (24 to 70 in) and is usually under 150 mm (60 in). This group is typically found in regions where a long dry season prevails. The pH in this group is slightly acidic; the soil environment is oxidizing and organic matter is rapidly destroyed. The alumina minerals are rare in ferruginous soils or do not occur at all. In Africa, regions with ferruginous soils are usually coincident with savanna-type vegetation.

A second group is ferrallitic soils which form where the leaching is intense due to a warm climate with a high annual rainfall of over 1,500 mm (60 in). The pH in this group is more acidic and the soil environment varies from oxidizing to reducing. In the ferrallitic group the alumina minerals are concentrated. In addition the iron minerals are lost in reducing environments and concentrated in oxidizing environments. Ferrallitic soils are more or less coincident with equatorial rain forest or tropical rain forest regimes.

The third group are ferrisols which are the least mature soils of the three groups. These are formed in areas where erosion more or less keeps pace with profile development. Thus clay minerals such as montmorillonite are not unusual as in the ferrisols over phyllites in Ghana. The influence of the parent rocks persists in ferrisols whereas in the ferruginous soils and ferrallitic soils, the influence is far less pronounced. Ferrisols require over 1,250 mm (50 in) of rainfall to form (below this rainfall, black clays may be the predominant soil type over certain rocks).

Samples were collected from most tropical countries of Africa as shown in Figure 4.23. Most of the samples were from West Africa, particularly Ghana.

Summary of Findings

The mean values of the liquid limit, plasticity index, gradation, compaction and CBR, indicate that the ferruginous soils, ferrallitic soils and ferrisols represent distinct soil groups with a characteristic range of properties. A discriminant analysis revealed that the groups were distinguishable on the basis of these properties. It was found that the ferruginous soils are lower in plasticity and the compacted densities at AASHO Modified compaction are higher, which results in higher CBR values than the other two groups. The ferrisols are similar, to some extent, to the ferrallitic soils for they have higher plasticity and lower CBR values than the ferruginous soils.

The D'Hoore classification for the African soils divides the three major groups into several subgroups. The symbols for the subgroups are as follows: ferrallitic soils are identified as La, Lb, Lc, Li, Lm, Ls, Lt and Lx; ferruginous soils as Ja, Jb, Jc and Jd; and ferrisols as Ka, Kb and Kc.

The results of the complete test program are listed in the appendix to this chapter (Table A4.5). Typical properties of the major subgroups are shown in Table 4.4 for the ferruginous group, Table 4.5 for the ferrallitic group and Table 4.6 for the ferrisol group. The ferruginous soils are all within the AASHO A-2 group. The ferrallitic soils are mostly in the A-6 and A-7 groups. However, two others are found. They are A-2-6 and A-2-7 (meaning the fines constitute less than 35% of the sample but the fines are similar to A-6 and A-7 soils in behavior). The one A-2-4 sample (La subgroup) from the Congo is not typical of the ferrallitic group. The ferrisols are mostly A-7-5 or A-7-6 and are similar to the ferrallitic soils.

The mean values for the major soil groups are given in Table 4.7. The grading envelopes for the major soil groups are shown in Figures 4.24, 4.25 and 4.26.

The average values of the various properties, i.e., liquid limit, plastic index, gradation, compaction and CBR, indicate that the ferruginous soils, ferrallitic soils and ferrisols represent distinct soil groups with a characteristic range of properties. A discriminant analysis revealed that the groups were distinguishable on the basis of physical and engineering properties. The discriminant analysis and an abbreviated program are described in the appendix to Chapter 4. The computer program used in this analysis is also included. The ferruginous soils are lower in plasticity and the compacted densities at AASHO Modified compaction are higher, with higher CBR values, than the other two groups. The ferrallitic soils are similar, to some extent, to the ferrisols, exhibiting higher plasticity and lower CBR values than ferruginous soils.

The average index properties of the major subgroups are shown in Table 4.8. There are major differences between soils of the ferruginous group and the soils of the ferrallitic and ferrisol groups. The data indicate that there is an apparent similarity in the physical properties of the ferrallitic and ferrisol soil groups.

There are no major differences within the ferruginous soil subgroups. The differences that exist between the subgroups indicate that although they have distinguishable properties, there are degrees of association.

There are no major differences within the ferrisol subgroups. The Ka and Kb subgroups are similar to each other and are also different from soils of other groups.

The ferralllitic soils group has one subgroup which shows major differences from other soils within the group. This is La – yellow ferrallitic soil on loose sandy sediments. This subgroup is widespread in the Congo and Cubango Basins. With few exceptions these soils in Africa are restricted in the northern hemisphere to the sandy deposits of the coastal fringe. This subgroup has become the arenosol of the new FAO classification.

Other than the subgroups discussed, the analyses suggest that the soils within the major groups (subgroups) are different, but exhibit an association within their major groups The previous results have shown that the pedological definitions of ferruginous soils, ferrallitic soils or ferrisols represent distinct soil groups with a characteristic range of properties and interrelationships between the liquid limit, plastic limit and minus 200. There are considerable variations in the CBR values for each of the soil groups as given in Tables 4.4 through 4.6.

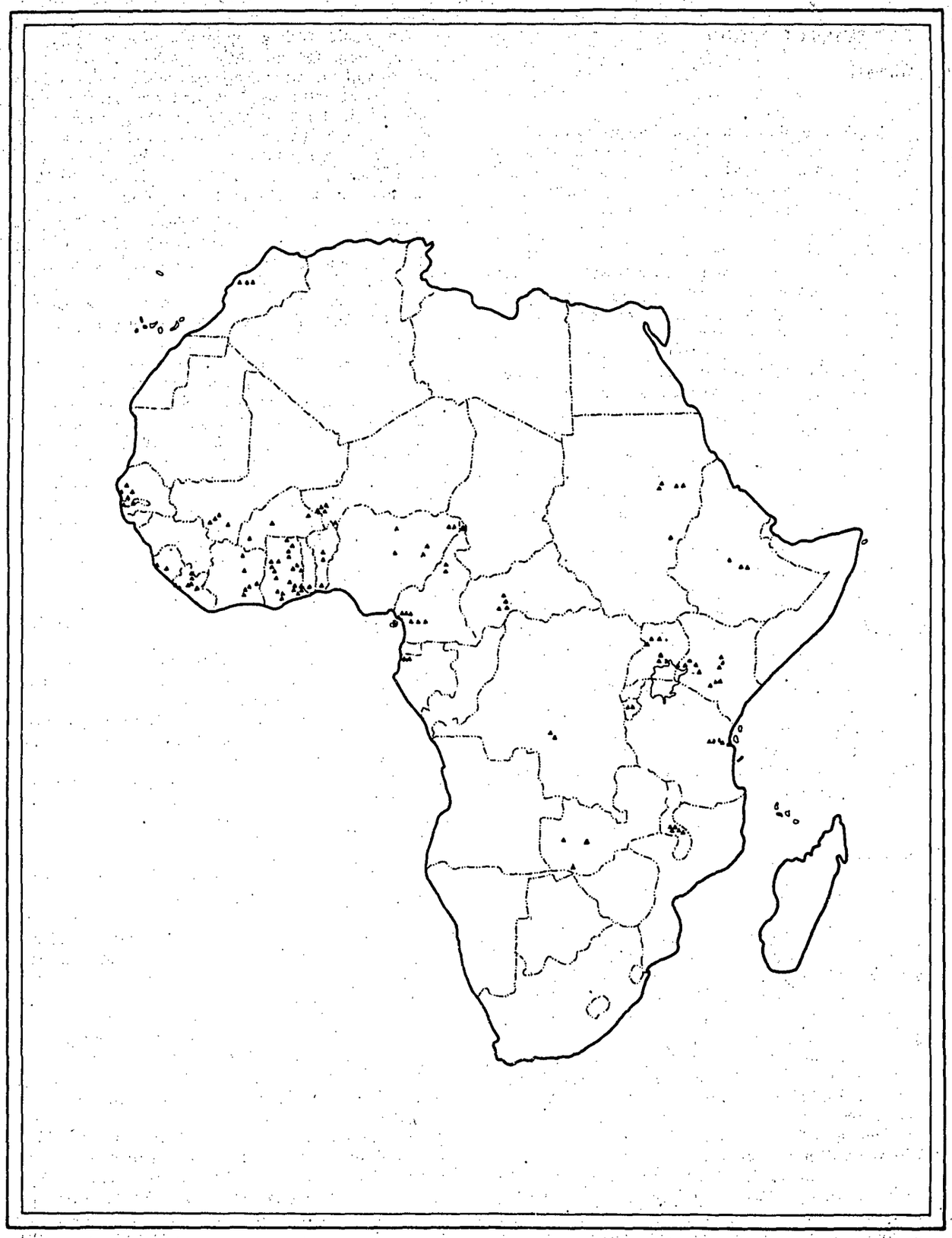

FIGURE 4.23 – LOCATION OF SOIL SAMPLES IN AFRICA

TABLE 4.4
Typical Soil Test Results for Ferruginous Soils of Africa

COUNTRY	D'HOORE	AASHO	GI	LL	PL	PI	I	3/4	1/2	3/8	4	10	40	200	AASHO OMC	MOD MDD	SOAK CBR
Ghana	Bcjd	A-2-6	0	26	15	11							45	20	12	130	150
Senegal	Ja	A-2-7	0	39	20	19	95	91	80	68	46	33	27	20			
Upper Volta	Jc	A-2-6	1	38	14	24	100	97	88	80	66	51	38	25	14	117	35
Niger	Ja	A-2-4	0	21	11	10	97	89	79	73	64	55	40	25	9	134	22
Tanzania	Ja	A-2-6	6	34	19	15	100	100	100	100	100	100	93	61			
Kenya	Jb	A-2-7	0	45	31	14	100	100	97	94	88	52	40	28	19		52
Uganda	Jd	A-2-6	2	38	17	22	100	100	99	96	83	61	51	34	13		19
Sudan	Ja	A-2-4	0	21	12	9	100	100	100	100	100	98	57	27			
Gambia	Jd	A-2-6	0	36	16	20	98	77	60	53	42	34	28	22			

TABLE 4.5
Typical Soil Test Results for Ferrallitic Soils of Africa

COUNTRY	D'HOORE	AASHO	GI	LL	PL	PI	I	3/4	1/2	3/8	4	10	40	200	AASHO OMC	MOD MDD	SOAK CBR
Congo	La	A-2-4	8	26	17	8	100	100	100	100	100	97	88	33	10	128	
Ghana	Lx	A-6	6	38	18	20	100	100	100	100	95	81	67	47	14	120	15
Liberia	Lx	A-2-7	2	56	29	27	100	95	84	72	57	41	36	27			
Sierra Leone	AbLm	A-2-6	1	55	31	24	100	98	95	90	68	37	29	27			
Burundi	BcLsLt	A-6		31	16	16	100	100	96	92	84	76	74	70			
Dahomey	Li	A-7-6	5	45	21	24	100	100	100	99	85	72	55	39			
Ivory Coast	Lm	A-7-6	22	62	31	31	100	100	100	100	100	99	88	69	19	108	
Mali	Li	A-6	3	35	21	14	100	100	99	89	67	55	51	40	16	116	
Uganda	Lx	A-6	2	39	19	20	100	100	96	91	82	73	53	38			

TABLE 4.6
Typical Soil Test Results for Ferrisols of Africa

COUNTRY	D'HOORE	AASHO	GI	LL	PL	PI	I	3/8	1/2	2/8	4	10	40	200	AASHO OMC	MOD MDD	SOAK CBR
Ghana	Kc	A-7-5	3	53	34	19							50	37	17	111	45
Niger	BcKc	A-2-6	0	28	16	12	100	100	100	98	82	68	39	26			
Ivory Coast	Kc	A-7-6	18	48	24	24	100	99	92	83	65	60	50	40	17	110	12
Mali	BcKb	A-7-5	3	55	31	24	100	100	100	89	61	51	43	38	15	120	9
Uganda	Kc	A-2-7	0	46	21	25	100	100	97	91	56	31	24	20	14		16
Kenya	Ka	A-7-5	27				100	100	100	100	100	99	98	91			
Cameroons	Kc	A-7-5	19	65	37	27	100	100	100	100	98	97	88	66			
Ethiopia	Kc	A-7-5	19	68	33	35	100	100	100	98	84	63	62	58	28	96	12
Ghana	Kc	A-7-6	12	57	25	32							65	50	19	109	15

TABLE 4.7
Mean Soil Properties, Ferrallitic Soils, Ferrisols and Ferruginous Soils of Africa

ATTERBERG LIMITS

	Ferrallitic Ghana			Ferrisols Ghana			Ferruginous Ghana		
	LL	PL	PI	LL	PL	PI	LL	PL	PI
$\bar{x}$ (mean)	42	24	19	46	23	23	31	18	14
σ (standard deviation)	12	7	7	11	6	7	11	3	9
N (number of samples)	(41)	(41)	(41)	(46)	(46)	(46)	(47)	(47)	(47)

	Other Countries			Other Countries			Other Countries		
	LL	PL	PI	LL	PL	PI	LL	PL	PI
$\bar{x}$	47	24	23	55	29	27	33	12	15
σ	13	8	7	17	9	12	11	6	8
N	(35)	(35)	(35)	(12)	(12)	(12)	(32)	(32)	(32)

GRADATION

Ferrallitic Ghana				Ferrisols Ghana				Ferruginous Ghana			
Sieve	$\bar{x}$	σ	N	Sieve	$\bar{x}$	σ	N	Sieve	$\bar{x}$	σ	N
1	99	2	(20)	1	95	9	(16)	1	99	2	(19)
3/4	96	6	(20)	3/4	94	10	(16)	3/4	98	3	(19)
1/2	92	10	(20)	1/2	90	12	(16)	1/2	96	5	(19)
3/8	86	14	(20)	3/8	86	14	(16)	3/8	93	8	(19)
4	70	19	(20)	4	73	21	(16)	4	75	16	(19)
10	54	22	(20)	10	52	29	(16)	10	51	19	(19)
40	46	18	(20)	40	40	23	(16)	40	46	22	(19)
200	34	16	(20)	200	37	20	(16)	200	30	16	(19)
2 m	19	8	(20)	2 m	25	16	(11)	2 m	13	7	(15)

Other Countries				Other Countries				Other Countries			
Sieve	$\bar{x}$	σ	N	Sieve	$\bar{x}$	σ	N	Sieve	$\bar{x}$	σ	N
1	99	3	(36)	1	100	0	(13)	1	99	1	(35)
3/4	97	6	(36)	3/4	99	1	(13)	3/4	98	5	(35)
1/2	94	10	(36)	1/2	97	4	(13)	1/2	94	9	(35)
3/8	90	13	(36)	3/8	92	8	(13)	3/8	89	12	(35)
4	80	21	(36)	4	74	18	(13)	4	76	20	(35)
10	70	28	(36)	10	61	26	(13)	10	65	24	(35)
40	56	25	(36)	40	51	28	(13)	40	51	21	(35)
200	40	20	(36)	200	44	27	(13)	200	32	15	(35)
2 m	26	14	(18)	2 m	24	20	(13)	2 m	16	8	(35)

AASHO MODIFIED COMPACTION-CBR

	Ferrallitic Ghana			Ferrisols Ghana			Ferruginous Ghana		
	OMC	MDD	CBR	OMC	MDD	CBR	OMC	MDD	CBR
$\bar{x}$	12	129	46	14	122	42	10	133	75
σ	4	13	30	5	14	26	3	8	53
N	(38)	(38)	(33)	(43)	(43)	(39)	(40)	(40)	(40)

TABLE 4.7 (continued)

	Other Countries			Other Countries			Other Countries		
	OMC	MDD	CBR	OMC	MDD	CBR	OMC	MDD	CBR
$\overline{x}$	13	117	24	21	108	14	13	124	33
σ	4	9	17	7	14	11	4	11	23
N	(12)	(3)	(11)	(9)	(7)	(9)	(19)	(19)	(17)

TABLE 4.8
Mean Soil Properties, Groups & Subgroups (Africa)

Group	Liquid Limit		Plastic Index		Percent Passing No. 200 Sieve		Number of Tests
	$\overline{x}$	σ	$\overline{x}$	σ	$\overline{x}$	σ	
FERRUGINOUS	34	10	16	7	30	17	235
Ja	33	12	16	7	29	13	32
Jb	43	3	18	5	18	14	2
Jc	39	10	18	8	36	19	100
Jd	33	7	17	6	20	14	40
	30	8	13	7	27	13	29
FERRISOLS	50	10	26	8	37	21	93
Ka	65	11	25	5	52	30	3
Kb	48	6	22	8	53	29	9
Kc	50	10	27	8	35	19	77
FERRALLITIC	44	14	18	10	36	19	205
La	25	14	4	8	17	18	24
Lc	55	11	23	6	46	20	37
Lc	41	11	19	9	37	18	12
Lm	41	12	18	7	29	16	19
Ln	38	9	20	7	36	21	5
Lx	43	11	20	8	39	16	80
LxHb	60	0	30	0	32	6	2
LsHa	51	5	24	3	30	18	8

Data from samples tested in kumasi and extracted from roadway soil survey reports from various countries in Africa.

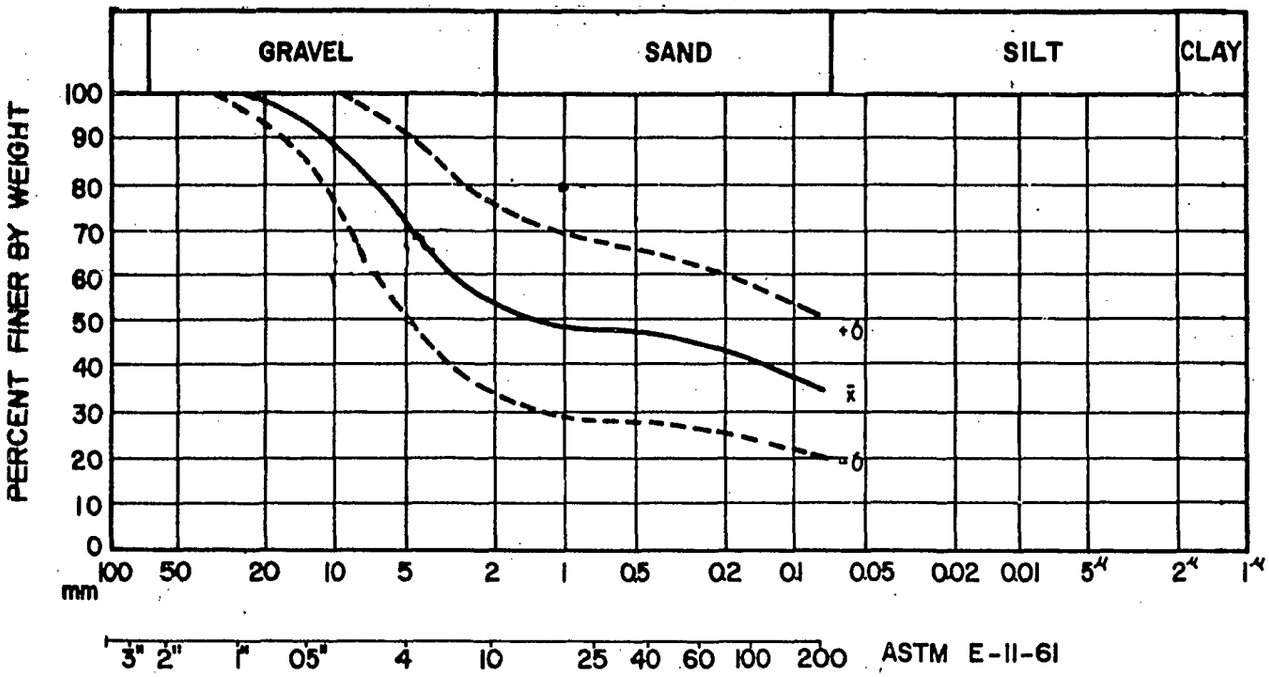

FIGURE 4.24 — FERRUGINOUS SOIL GROUP-GRADATION LIMITS, AFRICA

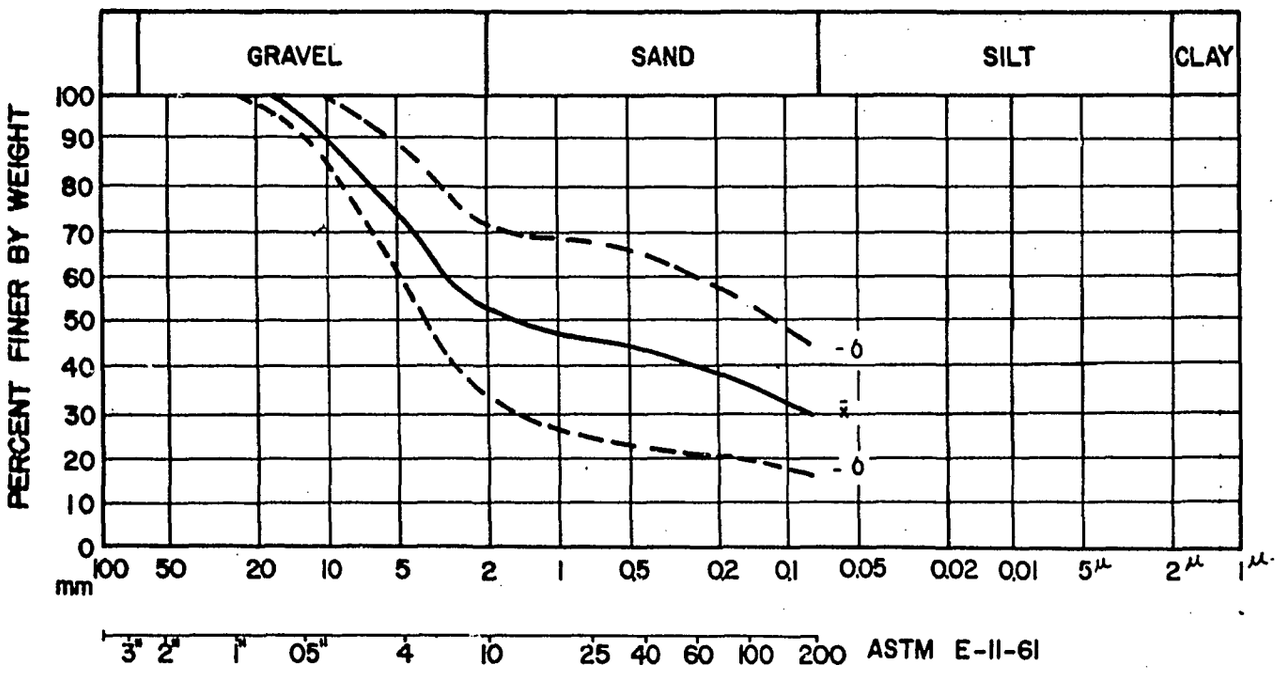

FIGURE 4.25 — FERRALLITIC SOIL GROUP-GRADATION LIMITS, AFRICA

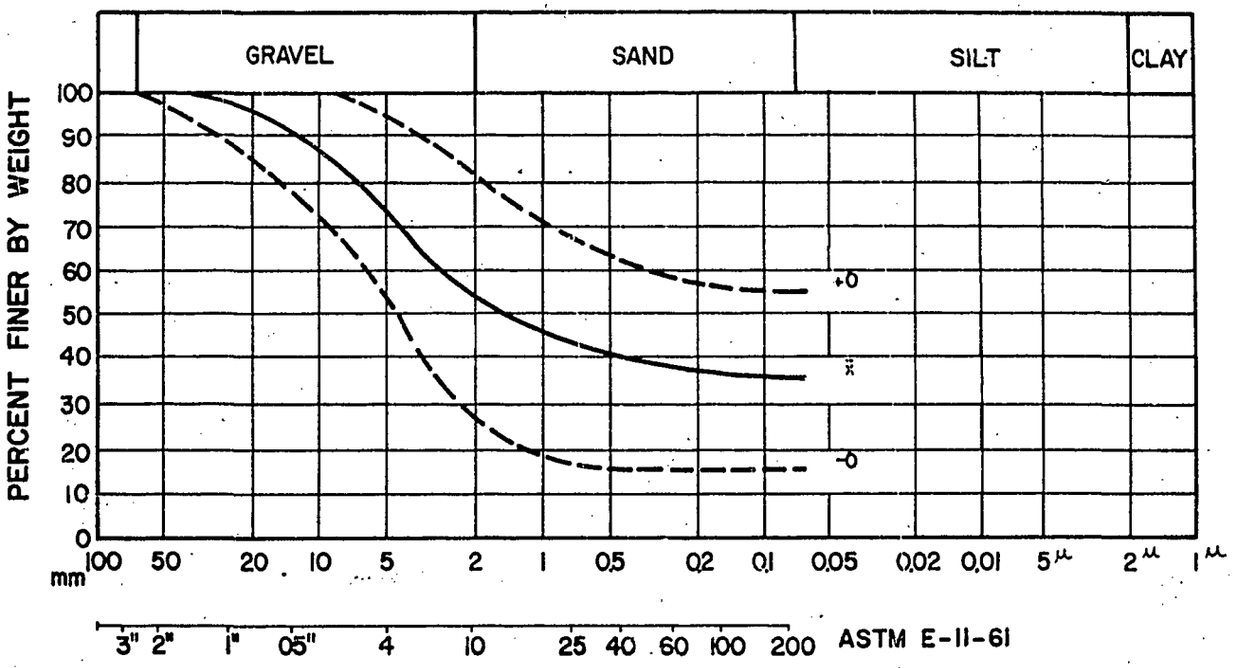

FIGURE 4.26 – FERRISOL SOIL GROUP-GRADATION LIMITS, AFRICA

Correlation of Physical and Engineering Properties

Correlations of Atterberg limits, gradation and CBR values have been found to exist among various temperate zone soils. The physical and engineering properties of the tropical soils were similarly analyzed during the African study by means of regression analyses to establish the existence of useful relationships to assist the engineer in the selection of materials for highway construction on the basis of routine classification tests.

The first relationship is that between the liquid limit and plasticity index of the tropical red soils. The relation is shown on the Casagrande plasticity chart, Figure 4.27. The general regression is as follows:

$$PI = 0.58\ LL - 4.54$$

Standard Deviation $(\sigma) = \pm 4.2$
Coefficient of correlation $(r) = 0.886$
Number of samples $(N) = 212$

The equations of the individual soil types are:

$PI = 0.71\ LL - 8.50$ (ferruginous)
$PI = 0.57\ LL - 3.62$ (ferrallitic)
$PI = 0.50\ LL - 1.50$ (ferrisols)

The positions of these soils are generally above the A-line with the exceptions being samples obtained in Liberia, Sierra Leone, Cameroon and Ghana. The majority of these are ferrallitic soils or ferrisols formed over basic rocks. The implications of this were discussed in the first section of this chapter.

The relationships between moisture, density, index properties and CBR were examined also. The relationship between optimum moisture and maximum dry density at Modified AASHO compaction is shown in Figure 4.28. The general equation is as follows:

$$MDD = 160 - 2.78\ OMC$$

Standard Deviation $(\sigma) = \pm 5.5$ lbs.
Coefficient of correlation $(r) = -0.841$
Number of samples $(N) = 81$

Relationships between CBR (at OMC-MDD, using AASHO modified compaction with a four day soaking period) and gradation as well as plasticity index are shown in Figures 4.29 and 4.30. There is no well defined relationship between the CBR and the product (plasticity index times percent passing the No. 200 or 40 sieves). The result can be presented in a broad envelope which indicate higher CBR's at lower values of the product. In both groups of data there is an apparent limit for ferruginous soils and for the ferrisols and ferrallitic soils.

A relationship which could be used as a guide in the selection of base course materials is shown in Figure 4.31. It has been generally accepted that high quality base course materials should have a CBR in excess of 75, under the moisture conditions which will prevail during the expected life of the pavement. The data shown in Figure 4.31 have been used to classify the soils into three suitability groups: (1) soils which can be used as base course materials under moisture conditions which exceed the optimum moisture content; (2) soils which can be used as base course materials under moisture conditions which will not exceed the optimum moisture content; and (3) soils which do not have a suitable CBR value under either condition. The relationship is shown as a function of maximum dry density at Modified AASHO compaction and the granulometric modulus, which is the accumulated percentage passing the 1", 3/4", 1/2", 3/8", No. 4, No. 10, No. 40 and No. 200 sieves of the laboratory CBR sample prior to compaction. The soils which conform to the first suitability group are enclosed by the points A, B, B', A. The second group is enclosed by B, C, C', B', B, and the unsuitable soils by A', C, C', A'.

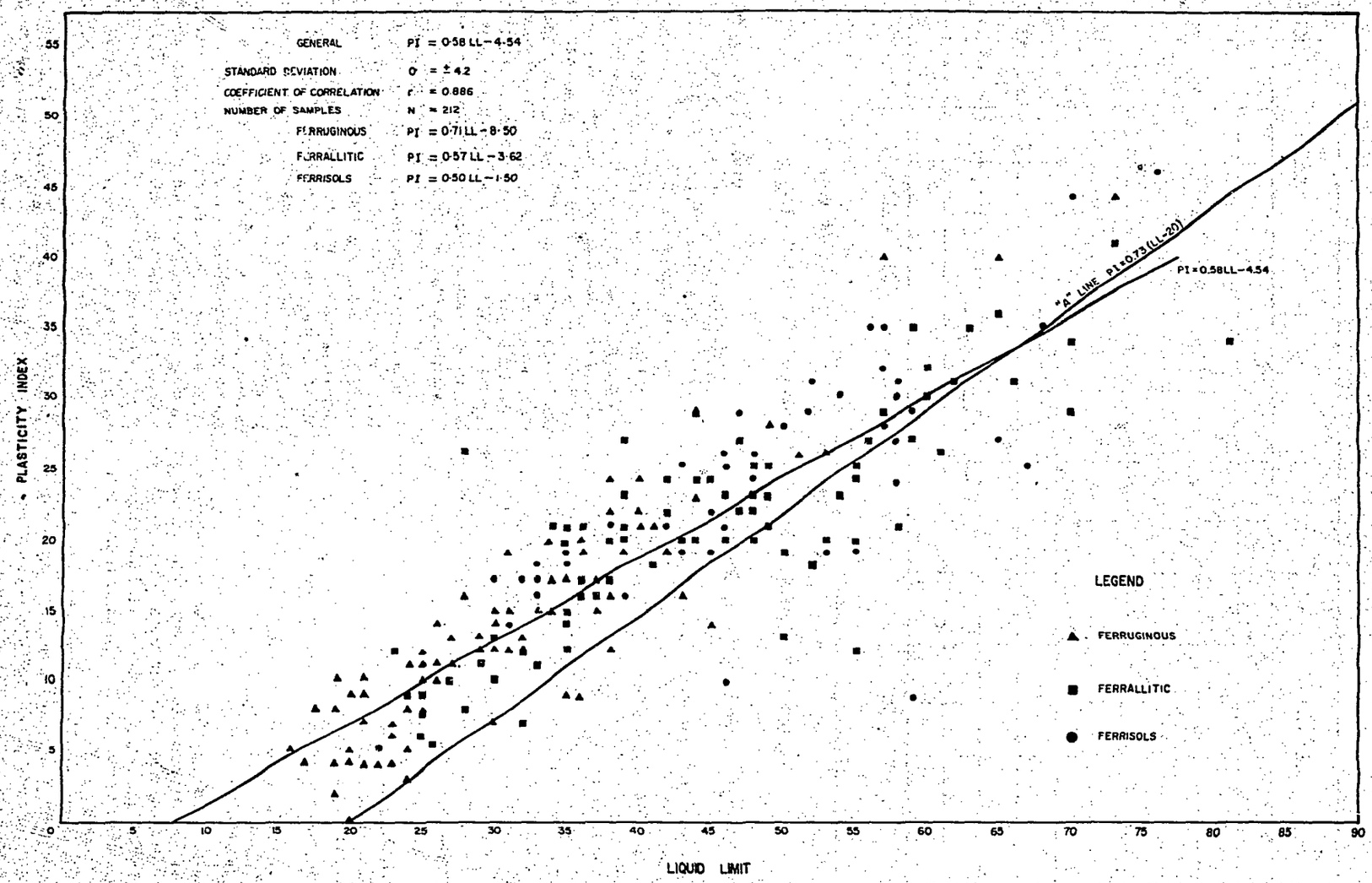

FIGURE 4.27 – LOCATIONS OF THE TROPICAL RED SOILS OF AFRICA ON THE CASAGRANDE CHART

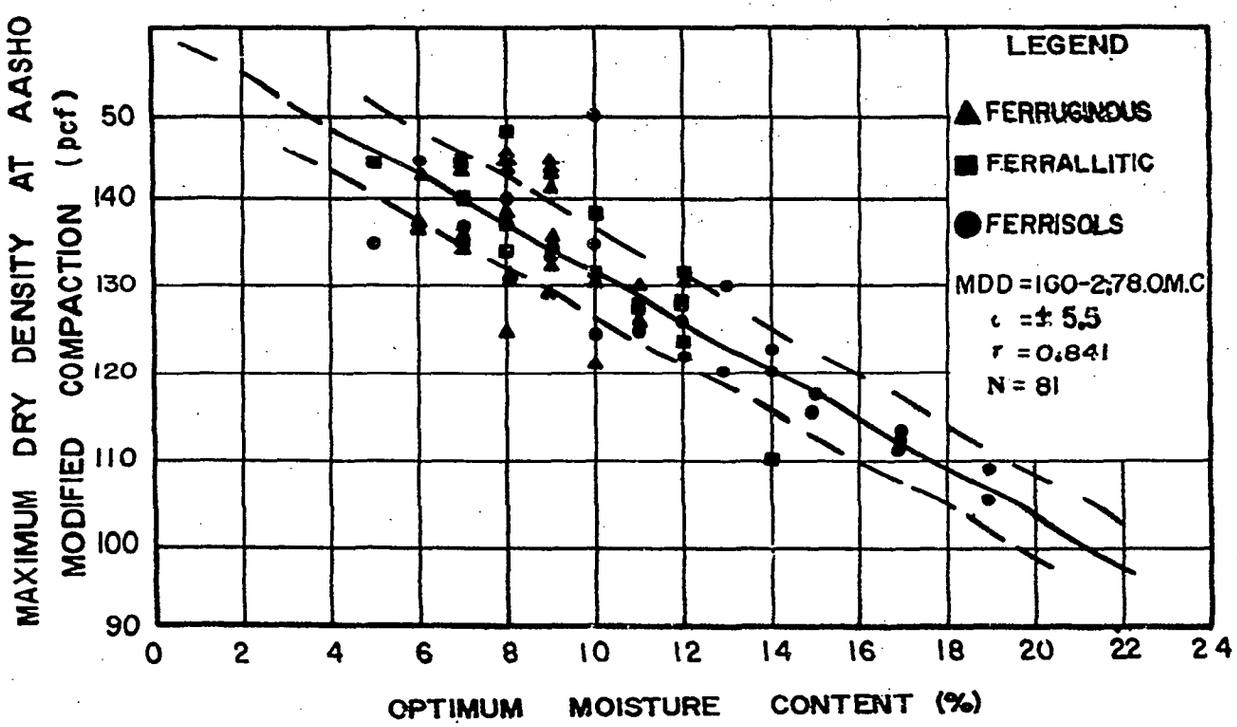

FIGURE 4.28 – RELATIONSHIP BETWEEN MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT, AFRICA

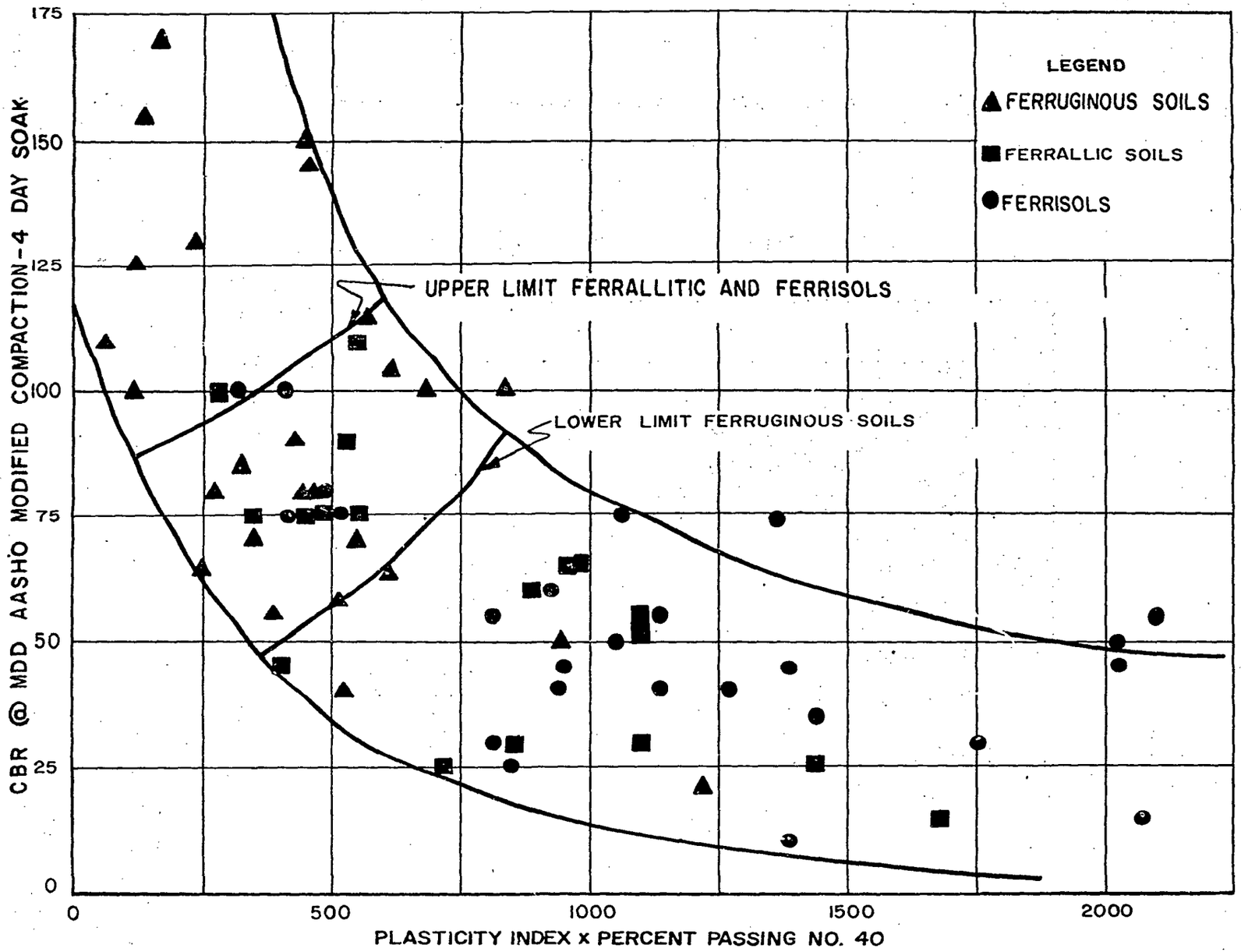

FIGURE 4.29 – CBR vs PLASTICITY INDEX x PERCENTAGE PASSING No. 40. AFRICA

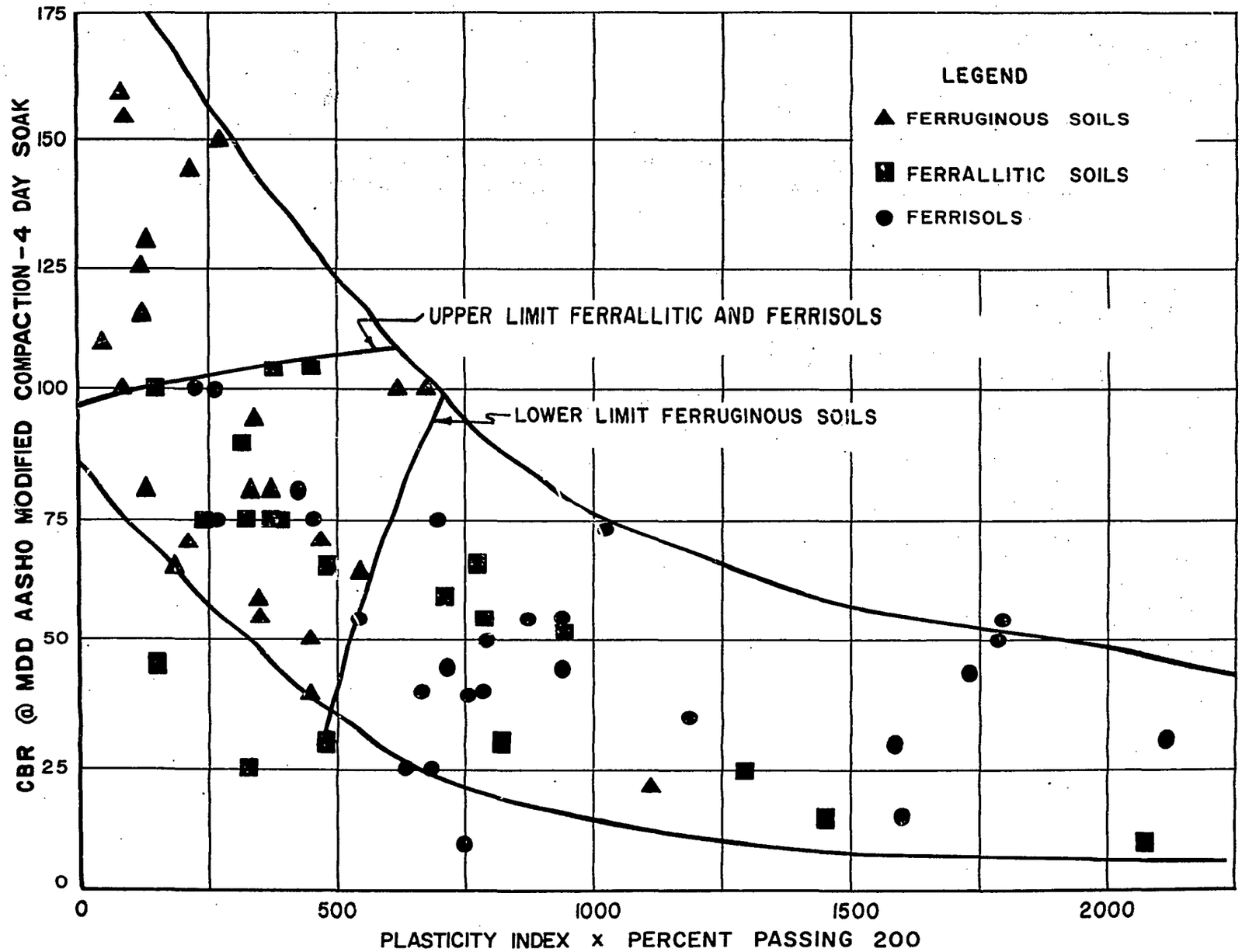

FIGURE 4.30 — CBR vs PLASTICITY INDEX x PERCENTAGE PASSING No. 200, AFRICA

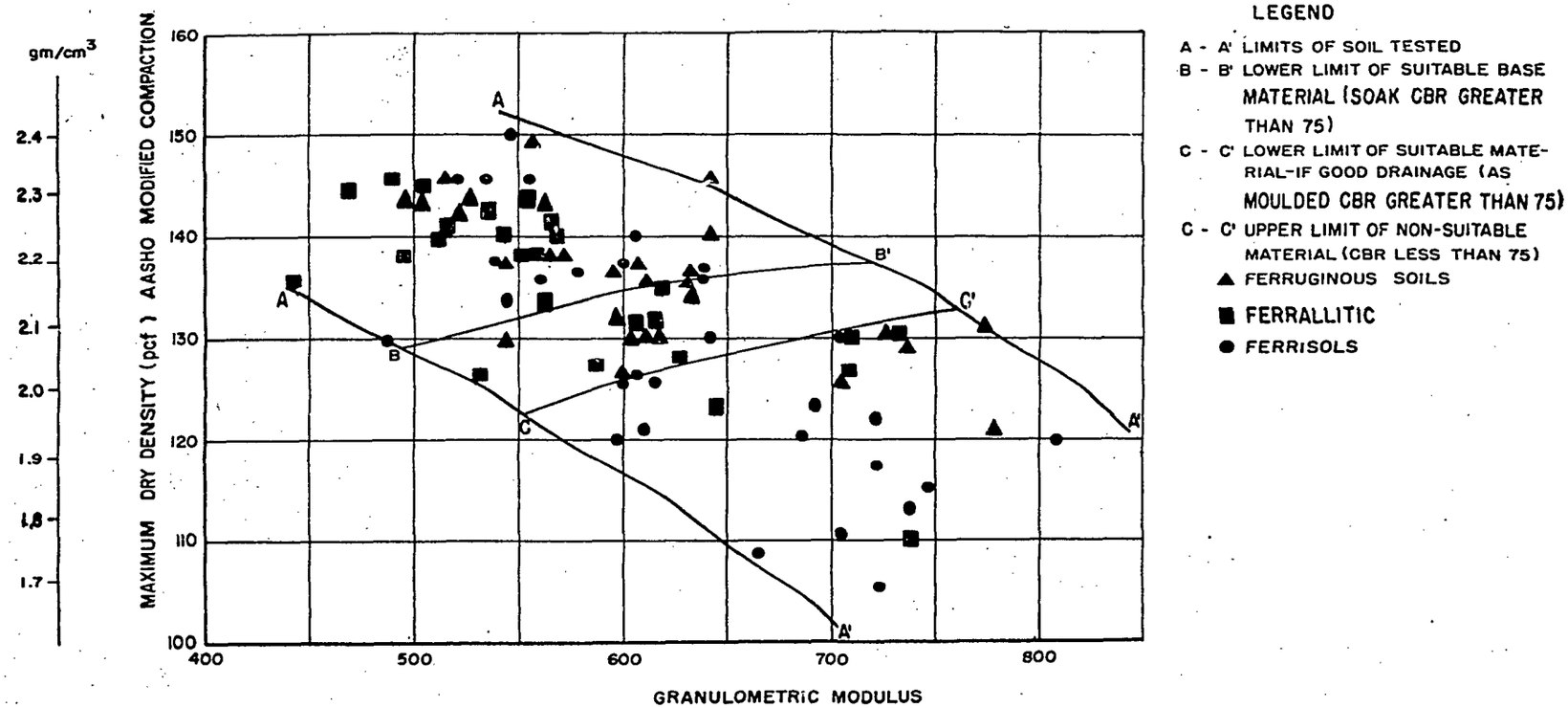

FIGURE 4.31 – SUITABILITY OF MATERIAL AS BASE COURSE ON THE BASIS OF MAXIMUM DRY DENSITY, AT AASHO MODIFIED DENSITY. AND GRANULOMETRIC MODULUS OF CBR SPECIMEN

RED SOIL GROUPS OF SOUTHEAST ASIA

General

This section is largely taken from the Phase I study in Thailand (Vallerga, et al., 1969).

Ironstone and concretionary gravel occur in Laos, Vietnam, Cambodia, Malaysia, Thailand and Burma. During the course of the study for Phase I, all of these countries were visited, and the following discussion, unless otherwise noted, pertains to laterite in all of them. In some areas of these countries and in Indonesia and the Philippines, the great soil groups which may contain laterite are extensive; however, they commonly contain only plinthite because the process of laterization has not advanced sufficiently to yield laterite. In many of these areas it is unlikely that laterite will form, either because the climate does not allow sufficient cyclic wetting and drying of the soils, or because the soil profiles are deficient in the necessary sesquioxides of aluminum and iron.

It was observed that the great soil group classifications provide an excellent key to the geographic areas in which laterite may form. The general soil maps of the countries listed give a good outline of the areas in which laterite or plinthite may exist. Soil maps of a larger scale may be available from local agricultural agencies. The following section provides more specific information on the association of laterite with the great soil groups.

General Engineering Properties of the Various Great Soil Groups

Numerous field correlations were made between the agricultural soil maps and laterites observed in-situ throughout Southeast Asia. Based on these observations, some general statements may be made concerning the engineering characteristics of the lateritic soils formed in the various great soil groups, and the utility of using agricultural soil maps for preliminary material source surveys. Table 4.9 presents results of tests on samples taken at several depths in typical soil profiles in Thailand containing laterite.

It has been observed that lateritic gravel formed in the four most appropriate great soil groups may be generally ranked in order of quality as road construction materials as follows:

Great Soil Groups	Quality	Comments
Grey Podzolic	fair to excellent	may be quite sandy
Red Yellow Podzolic	very poor to good	frequently weak aggregate and excessive clay
Latosols	very poor to fair	in many instances have very little laterite aggregate and may be clayey
Low humic gley	very poor to fair	laterite aggregate, is frequently floating in plastic clay

Some general guideline characteristics which may be associated with lateritic gravels in these great soil groups are as follows:

Grey Podzolic Soils:

1. Laterite or lateritic gravel of fair to excellent overall quality for road construction may occur in one or more strata at shallow depth. The strata may be interbedded with alluvial sediments.
2. The overburden which must be stripped to reach the lateritic gravel is usually less than one meter thick. Frequently the usable gravel deposit is no thicker than the overburden.
3. The gradation is often moderately coarse and open, with 100 percent passing the 1 to 1/2 inch sieve and the sand size fraction (between the No. 10 and No. 200 sieve) all but missing. The minus 200 materials are generally composed of very fine sand, silt, and kaolinitic clay.
4. The Atterberg limits of the minus No. 40 fraction generally varies from non-plastic to moderately plastic. The limits increase with depth. Frequently there is a marked increase in clay content and plasticity immediately beneath the lateritic gravel strata.
5. The durability as measured by their ability to resist mechanical abrasion in the presence of water varies from low to very high. The concretions composed of iron-cemented, pre-existing fine sand and silt appear to have the highest durability. Iron-cemented clays or gravels appear to have lower durability. The cemented materials are generally the same as those in the adjacent uncemented soil matrix.
6. These materials frequently have low to moderate absorption, and moderate sensitivity to moisture content during compaction.
7. Test results are generally not sensitive to manipulation or drying prior to testing and they do not increase appreciably in hardness due to additional wetting and drying.
8. The difference in strength between a lateritic gravel sample compacted at optimum moisture content and one that has subsequently been saturated is generally slight to moderate.

Red Yellow Podzolic Soils:

1. Concretionary gravel of very poor to good overall quality for road construction may occur throughout the soil profile. Though laterite predominates in the concretions near the surface, plinthite frequently predominates at depth.
2. Very little or no overburden must be stripped to reach concretionary gravel deposits, which may be many meters in thickness.
3. The gradation is frequently fine and open. One hundred percent may pass the 3/8 inch sieve, and the No. 10 to No. 200 fraction may be all but missing. The minus No. 200 material is predominantly kaolinitic clay.

TABLE 4.9

Engineering Properties of Typical Concretionary Gravels—Thailand

Locations	Depth Below Surface (cm)	Description	Great Soil Group Class	LAA[1]	LL	PL	PI	SL	pH	Sieves, Percent Passing 1 1/2"	3/4"	3/8"	4	8	16	30	50	100	200
69.4 km E Udon on Rt 22	60	Fine Cl Si Sa[2]	Grey Podzolic		38	21	17	13	5.2	—	—	—	—	100	98	97	93	78	57
	90	Med hard[3] - rounded - VF[4] Hem w/Lim[5] center nodules in Cl Si			43	24	19	15	6.0	—	100	63	35	25	22	21	20	16	14
	140	Iron stained Cl with some poorly formed nodules			59	28	31	14	4.8	—	100	99	93	85	78	74	69	60	49
	200	White Clay with slight iron stain traces			52	23	29	14	4.8	—	—	100	99	96	93	90	85	78	67
34 km SW Sakin Nakon on Rt 213	10	Fine Cl Sa Si	Red-Yellow Podzolic		27	20	7	7	5.4	—	—	100	99	96	92	91	90	70	41
	20	Soft to Hard - rounded - VF Hem nodules in Cl Si			54	27	27	20	5.2	—	100	91	68	41	32	30	28	23	16
	30	Soft-rounded - VF - Lim w/Hem staining nodules in Cl Si (sandstone at 45 cm.)			48	25	23	21	5.4	100	93	75	58	45	39	37	36	31	22
7 km S Kalisin on Rt 213	10	Cl Si Sa w/some small - hard-round - VF - Hem nodules	Low Humic Gley		NP	NP	NP	NP	5.0	—	—	—	—	—	100	98	93	74	48
	80	Fine alluvial gravel and soft to med - irreg - VF - Hem and Lim nodules in Cl Si			41	22	19	15	5.4	100	98	69	36	23	21	20	19	17	14
	166	Some alluvial gravel w/soft to med hard - irreg - VC - Hem and Lim in Cl Si			34	18	16	14	5.4	100	96	66	46	35	31	28	26	23	19
	175	Si Cl with some fine alluvial gravel			43	20	23	15	5.4	—	100	96	87	80	75	71	67	58	49
1 km N Selaphum on Rt 2046	50	Cl Si w/fine alluvial gravel	Grey Podzolic		NO	NO	NO	NO	5.7	NO	NO	NO	NO	NO	NO	NO	NO	NO	NO
	100	Hard-massive to irreg - VC Hem and Lim			X	X	X	X	6.8	X	X	X	X	X	X	X	X	X	X
	120	Alluvial river gravel and soft to med hard - irreg - VC - Hem and Lim nodules in Cl Si			26	16	10	NO	5.8	100	98	81	57	42	36	34	32	25	26
	150	Fine gravel w/a few fine nodules in Cl Si			33	17	16	NO	5.3	—	100	94	83	60	48	41	36	26	18
	200	Soft-massive to irreg- VC - Hem and Lim			X	X	X	X	5.2	X	X	X	X	X	X	X	X	X	X
	250	Med Hard-massive to irreg - VC - Hem and Lim			X	X	X	X	5.1	X	X	X	X	X	X	X	X	X	X
	300	Soft - irreg - VC - Hem and Lim nodules in Cl Si			31	15	16	NO	5.0	—	100	96	87	73	67	63	58	46	36

TABLE 4.9 (continued)

Locations	Depth Below Surface (cm)	Description	Great Soil Group Class	LAA[1]	LL	PL	PI	SL	pH	Sieves, Percent Passing 1 1/2"	3/4"	3/8"	4	8	16	30	50	100	200
6 km E Yasothorn on Rt 216	100	Cl Si	Grey Podzolic		22	15	7	11	5.0	—	100	98	95	92	89	82	77	53	32
	150	Med Hard - rounded – VF - Hem nodules in Cl Sa Si		51	26	18	8	17	5.8	100	67	44	33	28	24	22	19	14	16
	230	Med Hard - rounded - VF - Hem w/Lim staining nodules in Cl Si		51	41	23	19	24	6.0	100	67	47	38	32	28	25	22	16	12
	350	Si Cl with weak nodules - partially formed			35	20	15	17	5.4	100	79	71	60	54	49	41	39	30	22
	450	White fine Sa Si			19	12	7	7	6.6	—	100	99	97	97	96	94	89	66	44
5 km E of Saraburi on Rt 2	120	Hard - angular - Hem nodules nodules in silty clay	Red-Yellow Podzolic	25	76	37	39	39	4.6	100	89	65	34	19	16	15	13	11	9
	210	Iron stained clay and soft poorly formed nodules			79	42	37	19	5.0	100	93	80	74	70	68	66	62	54	44
	420	Grey white iron stained clay			66	42	24	22	—	—	100	82	74	67	62	57	50	39	31
21 - 600 km S.E. Choke Chai on Rt 2071	100	Soft to med-angular - Lim nodules in Cl Si	Grey Podzolic	34	51	31	20	20	5.5	—	100	81	50	39	17	15	13	10	8
21 - 800 km S.E. Choke Chai on Rt 2071	100	Soft to med - angular - Lim nodules in Cl Si	Grey Podzolic	34	52	32	20	26	5.5	—	100	71	43	25	21	17	16	12	7
23 km N. khon kaen on Rt. 2	30	Fine Si Sa	Grey Podzolic		NP	NP	NP	NP	5.8	—	—	—	—	—	100	95	85	52	31
	90	Some alluvial gravel w/soft-irreg - 2M - Hem nodules in Sa Si (100 cm thick)		44	26	17	9	15	5.8	100	80	43	27	19	16	15	14	11	8
	140	Some alluvial gravel w/soft-irreg - 2M - Hem nodules in Sa Si (100 cm thick)		44	32	15	17	16	5.6	—	100	81	51	36	30	29	27	23	18
	180	Alluvial Sa Gr w/some soft-irreg - Hem nodules			43	15	28	12	6.0	—	100	99	90	74	69	65	61	50	38
86 km N. Khon Kaen on Rt. 2	30	Fine Cl Sa Si	Grey Podzolic		27	18	9	16	5.2	—	—	—	—	—	100	97	92	74	51
	80	Soft to med - irreg - Hem nodules in Cl Si (60 cm thick)			31	18	12	14	5.4	100	65	30	13	9	8	8	7	7	5
	148	Small - soft to hard - rounded - Hem nodules in Si Cl			53	30	23	15	5.2	—	100	99	93	79	66	1	56	49	41

TABLE 4.9 (continued)

Locations	Depth Below Surface (cm)	Description	Great Soil Group Class	LAA[1]	LL	PL	PI	SL	pH	Sieves, Percent Passing 1 1/2"	3/4"	3/8"	4	8	16	30	50	100	200
7.8 km E. Udon on Rt. 22	15	Cl Si w/some very smal hard rounded - VF - Hem nodules	Low Humic Gley		NP	NP	NP	NP	5.6	—	—	100	99	92	84	82	80	73	58
	70	Soft to med - rounded - VF Hem nodules in Cl Si			70	32	38	18	5.0	—	100	94	59	46	28	25	24	23	20
	150	Soft-rounded - F - Hem nodule in Cl Si			36	21	15	8	6.0	100	93	69	39	27	21	20	18	15	13
22 km E. Udon on Rt. 22	15	Very fine Si Sa with some Cl	Grey Podzolic		NP	NP	NP	NP	5.4	—	100	97	95	94	92	91	90	80	40
	70	Med to Hard - irreg - VF - Hem w/Lim centers nodules in Cl Si			28	19	9	17	5.6	100	85	55	30	22	19	18	18	17	12
	100	Med to Hard - rounded - VF - Hem w/Lim centers nodules in Si Cl			46	22	23	16	5.4	—	100	90	52	30	20	19	18	17	14
47.5 km E. Udon on Rt. 22	30	Cl Si.	Grey Podzolic		NP	NP	NP	NP	5.2	—	100	98	97	97	95	93	91	76	49
	80	Soft to hard - irreg - Hem w/Lim centers nodules in Si Cl		26	32	18	13	12	5.8	100	83	54	36	30	26	25	24	22	16
	120	Med to hard - rounded - irreg - Hem w/Lim center nodules Si Cl			50	27	23	16	5.2	100	67	46	28	22	18	18	17	15	14
37 km E. Udon on Rt. 217	—	Fine Si Sa	Grey Podzolic		SAMPLE NOT OBTAINED														
	30	Hard-rounded - VF - Hem nodules in Cl Si		24	42	25	17	19	5.2	NO	NO	NO	NO	NO	100	96	83	74	65
	80	Cl with some poorly formed nodules			49	26	23	NO	5.4	—	100	92	89	86	83	80	78	74	69
47.6 km E. Udon on Rt. 217	—	Fine Si Sa	Grey Podzolic		SAMPLE NOT OBTAINED														
	NO	Hard-rounded - VF - Hem nodules in Cl Si		30	53	30	23	20	6.6	100	96	89	54	36	33	31	30	28	24
	NO	Cl Si			SAMPLE NOT OBTAINED														
9 km W. of Katburi towards Chon Bung on Rt. 3090	NO	Fine Sa Si	Grey Podzolic		SAMPLE NOT OBTAINED														
	NO	Hard-rounded - VF - Hem nodules in Cl Si		22	27	20	7	NO	6.5	—	100	94	55	16	15	13	12	11	10
	NO	NO			SAMPLE NOT OBTAINED														

(1) Los Angeles abrasion, "L" grading, 10 ball charge, 560 revolutions.
(2) Cl=clay: Si=silt; Sa=sand; Gr=gravel, in order of increasing contents.
(3) Resistance of modules to crumbling under light blow of hammer.
(4) VF=sand grains less than No. 100 sieve; F=from No. 100 to about No. 40; C=No. 40 to about No. 16; and VC wears greater than No. 16.
(5) Hem. hematite or Goethite; Lim=limonite.

4. The Atterberg limits of the minus No. 40 fraction generally varies from moderate to high. Generally the Atterberg limits and clay content increase gradually with depth.
5. The durability is very poor to fair.
6. These concretionary gravels frequently have high to very high absorption, and may be very sensitive to moisture content during compaction.
7. Test results are slightly to moderately affected by manipulation and drying prior to testing. They may increase in hardness with dehydration or repeated wetting and drying.
8. These gravels, when compacted and tested for strength at optimum moisture content, may experience a great loss in strength upon saturation.

Latosol Soils:

The latosols which were observed frequently did not contain significant amounts of laterite, and, when present, were overlain by several meters of overburden. The general characteristics of these soils were not sufficiently well studied to justify further detailed comment. A few samples from central Malaya were tested and found to be similar to the red yellow podzolic soils described above.

Low Humic Gley* Soils:

1. Concretionary gravels of generally poor to fair quality for road construction may be obtained from throughout the soil profile. This frequently grades into plinthite with depth.
2. Very little or no overburden must be stripped to reach deposits which may be several meters thick. These deposits are frequently in rice paddies, low alluvial terraces, or other marsh lands.
3. The gradation is highly variable and always open. The No. 10 to No. 200 fraction is generally missing. The coarse laterite concretions are essentially floating in kaolinitic clay.
4. The Atterberg limits of the minus No. 40 fraction generally vary from moderate to very high. These limits normally increase slightly with depth.
5. The durability, as measured by their resistance to abrasion in the presence of water, is very poor to fair.
6. These gravels normally have low to moderate absorption, and are not particularly sensitive to moisture content during compaction.
7. Results of tests on the fine fractions (minus No. 200) may be moderately to highly sensitive to manipulation and drying prior to testing. The coarse aggregate may increase in hardness if dehydrated or subjected to repeated wetting and drying.
8. The strength is generally low, regardless of whether a sample compacted at optimum moisture content is tested at this moisture content or is saturated prior to testing.

* Gley means "mottled" as in an iron-stained or iron-mottled clay.

It should be stressed that concretionary gravels are highly variable even within the same pit. The characteristics listed above merely represent trends noted in the respective soil types.

The relationship between plasticity index and liquid limit is shown on the plasticity chart, Figure 4.32. This includes 492 test results from tropical red soils from Laos, Thailand and Malaysia. Most points are above, or near the A-line; but several plot far below..

SIMILARITIES AND DIFFERENCES AMONG RED SOILS OF THE THREE CONTINENTS

General

Most of the testing for the African study was conducted in one laboratory with a limited number of technicians performing all of the tests. This procedure eliminates variability in testing which could be caused by different operators with different levels of experience using different testing techniques.

The testing for the South American study was performed in 19 district laboratories by different technicians. Recent research has shown that test results on one soil by many different technicians may vary enormously, as much as 100% or more in extreme cases. In light of these findings it would not have been surprising if the soils groups in Brazil were not as distinguishable as they were in Africa. Under these circumstances it was gratifying to find significant distinctions in Brazil. Further discrimination between groups and perhaps subgroups will be possible when the information presented in this chapter is correlated with future data generated from new highway projects in Brazil.

The results of the analysis of variance among the African and South American groups was often contradictory to Table 3.7. This analysis considered liquid limit; plastic limit; plasticity index; minus 2μ; OMC; MDD; and CBR. Not all of these are reproduced reliably such as the mechanical analysis.

The analysis of variance of the Atterberg limits was in agreement with Table 3.7. The ferralsols can be grouped with the ferrallitic, as their definitions imply. The acrisols are similar to both and the ferrisols are less so. The nitosols were not included in the analysis as only a few samples were tested.

Summary of Findings Regarding Major Groups

Luvisols and ferruginous soils occur in the more arid extremes for lateritic soils, in areas with pronounced dry seasons. They have formed over all rock types: igneous, metamorphic and sedimentary. Dehydration of the iron minerals and less abundant aluminum minerals has occurred in all except the soils which are sufficiently moist so that seasonal dehydration does not occur. Gibbsite is generally uncommon in these soils. Similarly halloysite or allophane do not occur, even over volcanic rocks. The absence of these minerals and the state of oxidation (dehydration) precludes significant changes of properties with air-drying. The fact that these soils display lower Atterberg limits,

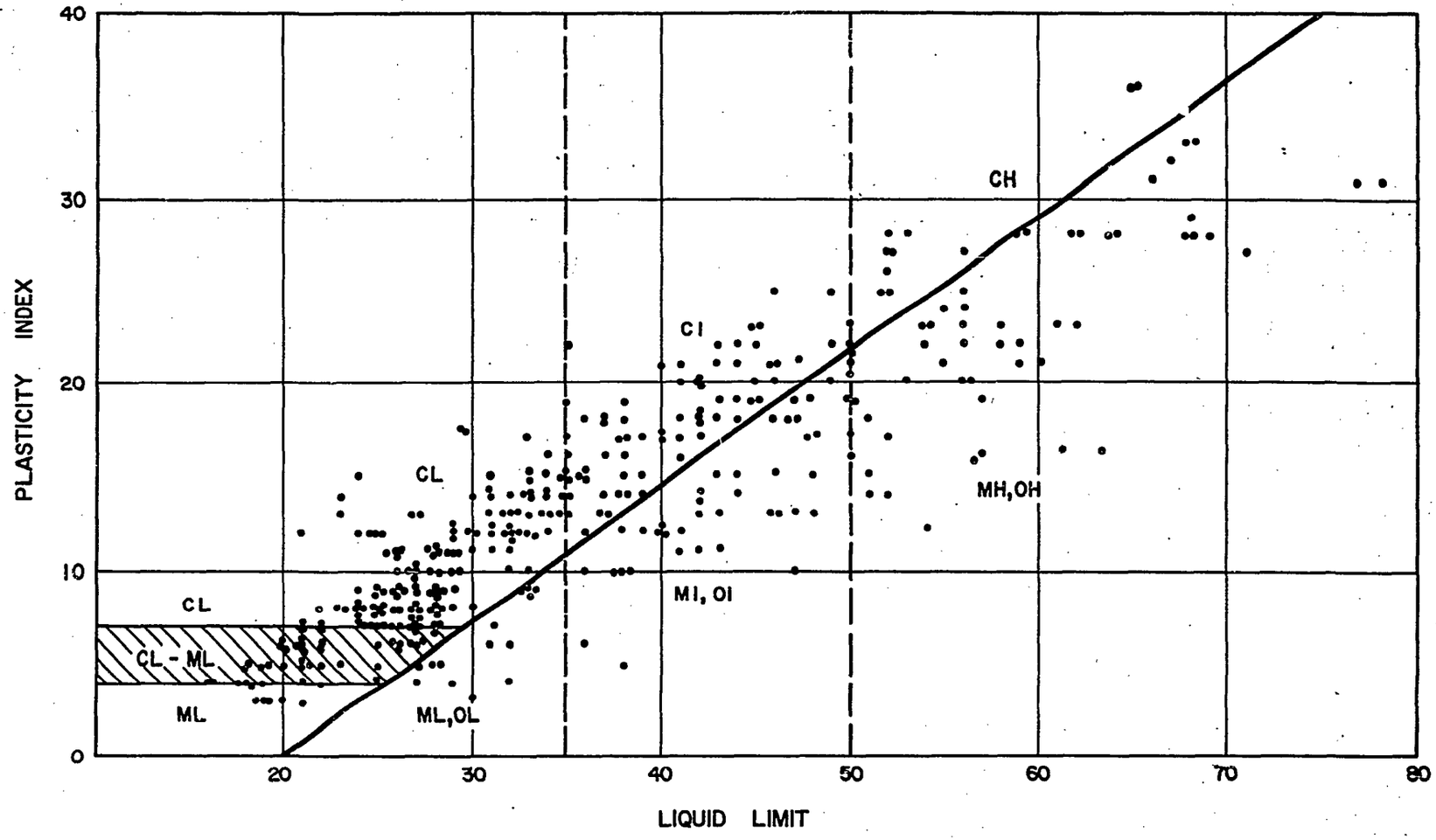

FIGURE 4.32 – PLASTICITY INDEX vs LIQUID LIMIT – DATA POINTS FROM LATERITIC SOILS SOUTHEAST ASIA

higher densities and CBR values could all be explained by dehydration and aggregation. The hydrated minerals would be expected to display greater plasticity and liquid limits.

Ferralsols, acrisols and ferrallitic soils occur in the more humid extremes for lateritic soils and in areas with dense vegetation. These soils have also formed over all rock types. Gibbsite is a common clay material and other hydrated forms of alumina occur as well as hydrated iron minerals. Halloysite is fairly common over volcanic rocks and to a lesser extent over basic intrusives. Changes in properties with drying occur due to hydrated halloysite (generally restricted to continually humid areas) and, to a lesser extent, due to gibbsite. Ferruginous crusts or hardpans are also relatively common. These have not always been subjected to complete dehydration as have the crusts in ferruginous regions and consequently they have not attained the same hardness or durability. However, nodular concretions in relatively shallow horizons can be hard and durable as those in ferruginous soils. Ferrallitic soils may contain plinthite which hardens on exposure. Dehydration is normally prohibited by the dense vegetative cover, but with exposure this process is no longer prevented.

Nitosols, ferrisols and perhaps some cambisols have formed over all types of rocks in intermediate to high rainfall areas where erosion has kept pace with profile development. Kaolinite predominates, but goethite also occurs in these soils. The degree of hydration of the clay materials is high, causing the properties to be similar to ferrallitic soils. Changes in properties with dehydration are considered minor when due to aggregation of the iron minerals, but changes in properties can be significant over volcanic rocks when hydrated halloysite is a constituent.

Changes in properties in ferruginous soils were only detected in a few soils and these changes were minor. All of these were from Ghana, below depths of 1.50 m (5 ft). Gibbsite is not a normal weathering product in ferruginous soils, so these changes can all be attributed to aggregation caused by dehydration of the iron minerals. Near-surface soils have been aggregated naturally during pronounced dry seasons. The depth of seasonal desiccation of course varies, but in Ghana this is apparently 1.50 m (5 ft) or more.

No changes in properties were detected in the luvisols. Only a few were properly tested in the undried condition, but these were believed to be similar to the ferruginous soils and therefore they were not very susceptible.

Changes in ferrisols are usually minor also, except for those formed over volcanic rocks, which contain hydrated halloysite, and perhaps allophane. In the majority of ferrisols, changes can be attributed to aggregation of the iron minerals.

Nitosols were not tested without air drying. Only a few samples were collected as these soils are not found in extensive areas. There are a few deposits over the basalts in southern Brazil which may contain halloysite and possibly change properties on drying.

Andosols contain appreciable allophane and can change properties greatly. They contain considerable structurally held water but display remarkable strength until disturbed. Such soils, with few exceptions, occur over pyroclastics, particularly those with a high content of volcanic glass, in highly humid areas.

Changes in properties with air drying can be significant in construction. Laboratory testing, if performed on air-dried samples only, may produce test results on altered soils. If the material will not be similarly air-dried during construction, erroneous values for the Atterberg limits, gradation, and density may be applied. It is believed that materials subject to changes in property are: (1) andosols, (2) ferralsols, acrisols and ferrallitic soils in areas with over 1,500 mm (60 in) of rain per year and where air-drying during the construction season may not be feasible or (3) to all red soils in areas with volcanic soils such as Southern Brazil, Central America, Ethiopia, East Africa, Cameroon, Indonesia and the Philippines. In these areas, it is strongly recommended that the susceptibility to change with air-drying be investigated. The appropriate laboratory procedures are described in the following chapter. If changes with drying occur that are significant and air-drying cannot be assured during construction, then test results on as-received samples should be applied.

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CHAPTER 5
TEST PROCEDURES AND STANDARDS

INTRODUCTION

Well established testing procedures used in engineering evaluations of temperate soils are not always suitable for evaluating tropical soils. Sometimes modifications of standard tests are necessary in order to obtain proper evaluations. For example, experience with tropical soils has shown that the manipulating and pre-heating preparations of temperate soils will change the properties of tropical soils unless the procedure is altered.

The changes in engineering properties that occur with pre-heating prior to testing are usually irreversible. The gradation, Atterberg limits and the moisture-density relationship are all affected. An example of such changes are shown in Figure 5.1. The sample was obtained from a construction site at the Santa Maria International Airport, San Jose, Costa Rica. The testing was conducted by the departamento Laboratorio De Materiales, Ministerio de de Obras Publicas y Transportes, San Jose. A special study was undertaken since the contractor was unable to obtain the specified compaction which had been determined in the laboratory. The data shown on the left side of Figure 5.1 are the original test results while the data on the right are the test results of the special study. In the latter test the sample was not dried prior to testing. The compaction curve on the right represents the moisture-density relationship of the material as it exists in the field. The subgrade had a natural moisture content of 70%, therefore the difficulty in obtaining the specified laboratory density in the field is obvious. However, it should be noted that the problem was restricted only to the compaction since the four-day soaked CBR for both samples was the same. It is not known if the as-molded CBR would have been the same for both samples but it is known that the in-situ CBR's at the site were in the order of 16.

It is important that such moisture-sensitive soils be identified in preliminary investigations in order to avoid delays during the construction phase. Tateishi (1967) has recommended the use of an "aggregation index" to determine the propensity of a soil to change after dehydration. This index is defined as the sand equivalent value of the soil in its natural state divided by the sand equivalent of the oven-dried sample. An index of two indicates a moderately sensitive soil and an index of 12 indicates a highly sensitive soil.

The effect of drying on the Atterberg limits for several soils tested during the African study is illustrated in Table 5.1. Tests were conducted initially without drying (at the field moisture content), with air drying and with oven drying at various temperatures and drying time. The data appears to indicate that temperature causes the greatest change and the time of drying is secondary.

Variations also occur in the Atterberg limits depending upon the amount of manipulation of the sample prior to testing. Excessive manipulation prior to testing leads to breakdown of the soil structure and disaggregation. Both consequences produce fines which results in higher liquid limit values.

The hydrometer analysis is a particularly difficult test in that it is often difficult to reproduce results. The strong tendency of tropical soils to aggregate or flocculate presents a problem in dispersing the soils prior to testing.

TEST PROCEDURES

The following are the recommended testing procedures to be used in engineering evaluation of tropical soils. Recommendations and modifications are suggested in view of the nature of these soils mentioned above.

Test	AASHO	British Std.
1) *Dry Preparation of Soil Samples*	T 87-70	1377 Part 2 Sec 4

Comments: It is recommended that the soil be air dried regardless of relative humidity and that all drying apparatus be avoided.

Test	AASHO	British Std.
2) *Wet Preparation of Soil Samples*	T 146-49	1377

Comments: It is recommended that the soil be air dried and all drying apparatus be avoided.

Test	AASHO
3) *Preparation of Soil Samples at Natural Moisture Content*	NA

Comments: This procedure has not been standardized by either AASHO or ASTM. It is a special preparation procedure and was developed to facilitate the testing of tropical soils at their natural moisture content. This was necessary because some tropical soils, particularly andosols, exhibit changes in engineering properties with drying. Two factors which cause the change in properties with drying are: (1) the tendency to form aggregation on drying and (2) the loss of water in hydrated minerals. The first generally results in an increase in strength while this is not necessarily true with the second effect.

The following is the procedure for preparing a sample for the Atterberg limit tests:

Wet Method:

1) Break up the required amount of material with rubber-covered pestle or rolling pin.
2) Transfer sample to saucepan and cover with water. Let soak until all material is disintegrated. This may require 2 to 12 hours.
3) Place a No. 40 sieve in a saucepan and transfer entire soaked sample into the sieve. Wash any material still adhering to the soaking pan into the sieve by squirting water from a battery filler.
4) Pour clean water into pan containing sieve until level of water is about 1/2 inch above mesh in sieve.
5) Agitate the sieve with one hand without lifting the sieve. Concurrently, stir material with the other hand until all fine material appears to have passed through sieve.

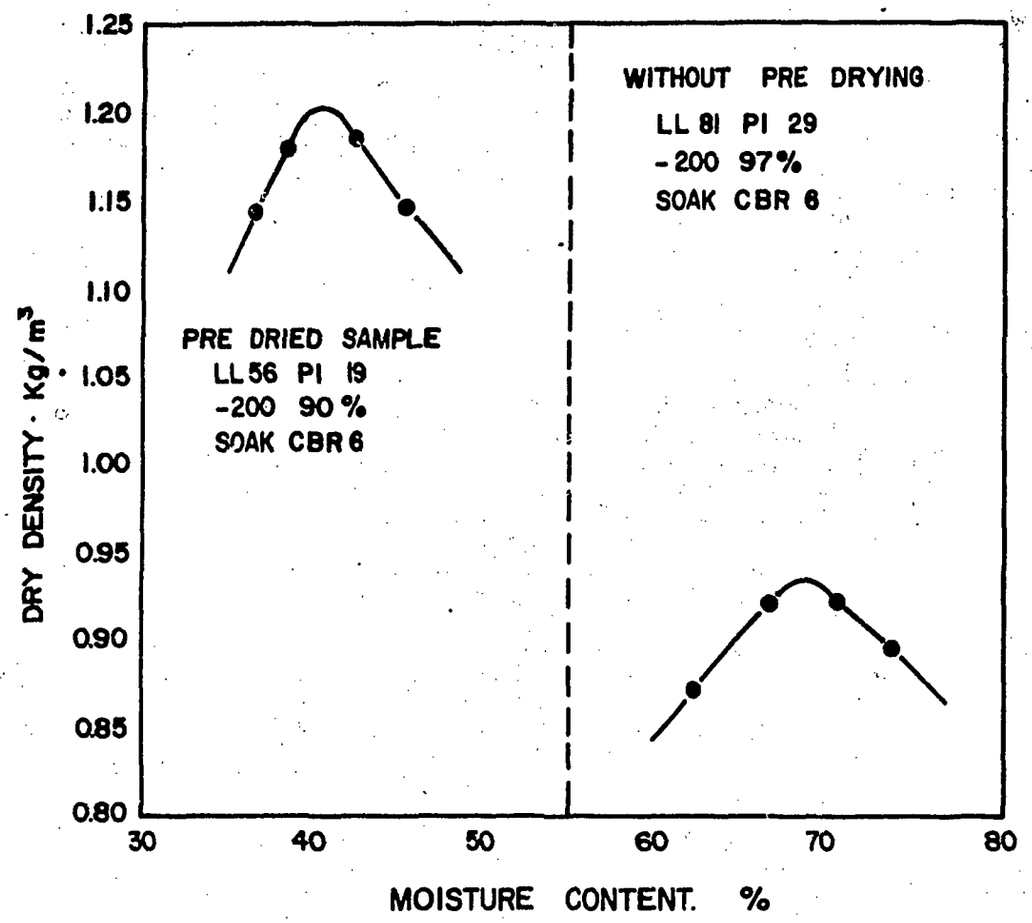

FIGURE 5.1 – COMPARISON OF PHYSICAL PROPERTIES TESTED WITH AND WITHOUT PRE-DRYING
(courtesy of Departamento Laboratorio de Materiales, Ministerio de Obras Publicas y Transportes, San Jose, Costa Rica)

TABLE 5.1
Change in Atterberg Limits After Drying at Various Temperatures and Time Periods

Soil		As Received	Air Dried	6 hours at 50°C	24 hours at 50°C	6 hours at 105°C	24 hours at 105°C
IV-5	LL	63.4	62.2	60.1	60.7	57.3	55.1
	PL	39.1	31.2	27.2	28.7	27.4	28.4
G12-1	LL	54.3	47.6	47.1	44.8	41.8	41.9
	PL	22.4	22.9	22.2	24.1	23.1	22.4
DAH-3	LL	51.8	44.6	44.7	45.5	42.8	42.9
	PL	29.8	26.8	23.9	22.9	19.8	20.3
G11-4A	LL	45.2	40.0	40.5	41.9	41.0	37.6
	PL	21.7	21.0	21.6	21.8	24.8	20.7
G11-4B	LL	36.5	34.5	36.1	36.8	36.0	35.2
	PL	38.2	30.2	31.4	29.1	26.3	28.2
G11-6B	LL	29.0	26.5	26.2	26.5	25.8	24.7
	PL	21.4	16.6	15.6	14.8	14.1	13.8
OKODEE	LL	65.0	62.9	58.3	58.5	49.4	46.0
	PL	27.2	27.8	29.4	28.5	24.4	25.9
BEKWAI	LL	61.6	53.4	54.4	53.9	44.4	42.7
	PL	28.2	25.5	26.2	24.8	23.6	23.8
TAMALE 5	LL	45.7	44.2	44.4	44.4	43.4	44.4
	PL	23.2	23.5	25.9	25.9	26.2	26.1

6) Hold sieve slightly above water surface in pan and squirt water from battery filler into sieve until retained particles and the sieve are clean. Discard material retained in sieve.
7) Place pan where it will not be disturbed and block it up on one side so water on the other side barely reaches rim of pan. Allow soil to settle for several hours.
8) Pour off liquid slowly by gradually increasing tilt of pan until cloudy layer overlaying the sediment reaches rim of pan.
9) Air-dry material to a smooth paste consistency and put in small mixing dish.

For most lateritic soils, material in suspension will settle out. If there is no indication of this after several hours, the following method may be used.

Place filter paper in a funnel and place wet soil inside the funnel in a jar or other container and allow to stand until all the excess water is filtered off.

The procedure of conducting the Atterberg test is the same with the exception that the low count (high moisture) is established first. The material is allowed to dry and the second point is established. This procedure is followed until a flow-curve is developed that will define the moisture content at 25 blows. The Plastic Limit is then determined after the Liquid Limit Test.

For many of these tropical soils that change properties the amount of +40 material is such a minor constituent that it is often unnecessary to sieve the material prior to conducting the liquid limit test.

The following is an outline of the procedure to follow when preparing a sample at its natural moisture content for establishing the moisture-density relationship.

(1) The sample at natural moisture content is passed through a 3/4 inch sieve.

(2) The sample is split into five more or less equal parts each of which are sufficient for compaction in a six inch mold.

(3) Two percent moisture is added to one sample and allowed to cure in a plastic bag or sealed container for a minimum of 12 hours prior to compaction; 18 to 24 hours would coincide better with normal working hours.

(4) A second sample is then compacted at this natural moisture content with the appropriate compactive effort and prepared for CBR testing if required.

(5) The remaining three samples are permitted to air dry for different periods of time prior to compacting the sample.

Test	AASHO	British Std.
4) *Particle Size Analysis*	T 88-70	1377 – Test 6

Comments: It is not recommended to dry the soil prior to testing. It is recommended that the mechanical analysis be performed at the natural moisture content. A moisture correction must be applied prior to computations. The appropriate apparatus for performing the sieve analysis is shown in Figure 5.2.

Sodium hexametaphosphate should be used as the dispersing agent in the hydrometer analysis. The dispersing time should be 15 minutes. Experience has shown that sedimentation tests are difficult to perform on tropical soils. It should be remembered that Stokes' law of sedimentation does not give the actual diameter of the particles but only the diameter of an equivalent sphere. The diameter of a clay plate can be five times greater than the one determined from Stokes' law. Any sedimentation test, no matter how accurately performed, gives only a general indication of the size and quantity of soil particles.

Test	AASHO	British Std.
5) *Liquid Limit*	T 89-68	1377 Test 2A

Comments: The "Single-point Method" evaluated during the African study can be used. Figure 5.3 shows the comparison of test results of the South American soils obtained in the standard laboratory procedure plotted against the results obtained with the African equation

$$LL = W \left(\frac{N}{25}\right)^{.15}$$

where:
W = water content at N blows percent
N = number of blows.

A maximum of five minutes mixing time is recommended because tropical soils are susceptible to breakdown with manipulation.

Test	AASHO	British Std.
6) *Plastic Limit and Plasticity Index*	T 90-70	1377 Test 6

Test	ASSHO
7) *Moisture Density Relations*	T 99-70 T 180-70

Comments: After mixing the samples with the various percentages of water the sample should be sealed in an air-tight container and allowed to cure for a period of 12 hours to insure a homogenous mixture prior to compaction. When significant amounts of gravel size materials are present which are hard and impermeable large moisture samples are necessary.

The following quantities are recommended for moisture determinations:

10 grams for minus No. 40 material
200 grams for minus No. 4 material
1,000 grams for minus 3/8 inch material
2,000 grams for minus 3/4 inch material

Test	ASSHO	British Std.
8) *Specific Gravity*	T 100-70	1377 Test 5

Comments: When the moisture density relation is established without drying the specific gravity should also be determined without allowing the sample to dry prior to testing. A moisture correction is used prior to computations.

Test	ASSHO
9) *California Bearing Ratio*	T 192-63

Comments: The recommendations given for AASHO T 99 and T 180-70 should be followed in compaction. If initial testing indicates that the soil is moisture sensitive, the compaction should be accomplished without predrying.

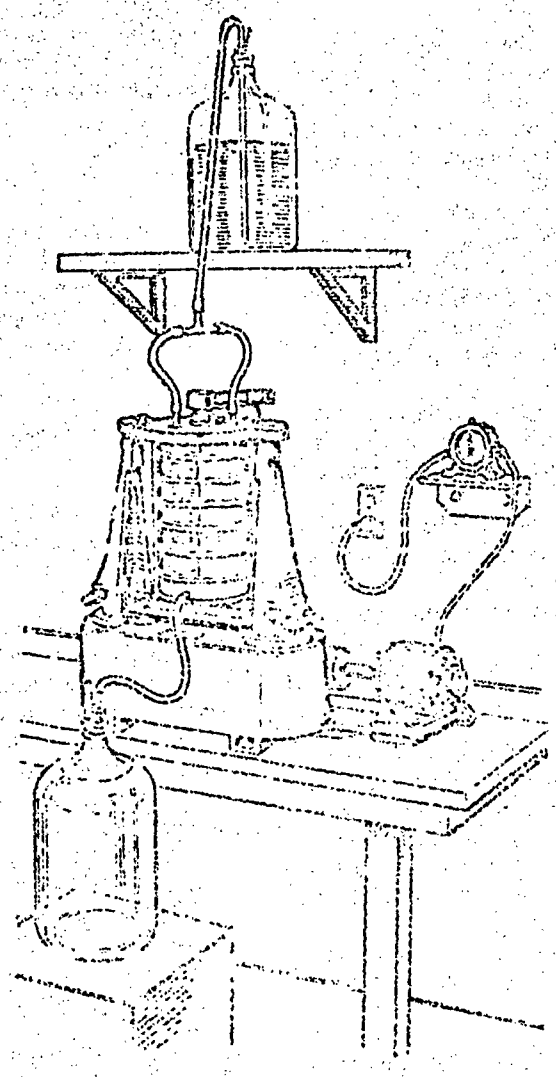

FIGURE 5.2 – WET TEST SET UP WITH MECHANICAL SIEVE SHAKER (ASTM, 1969)

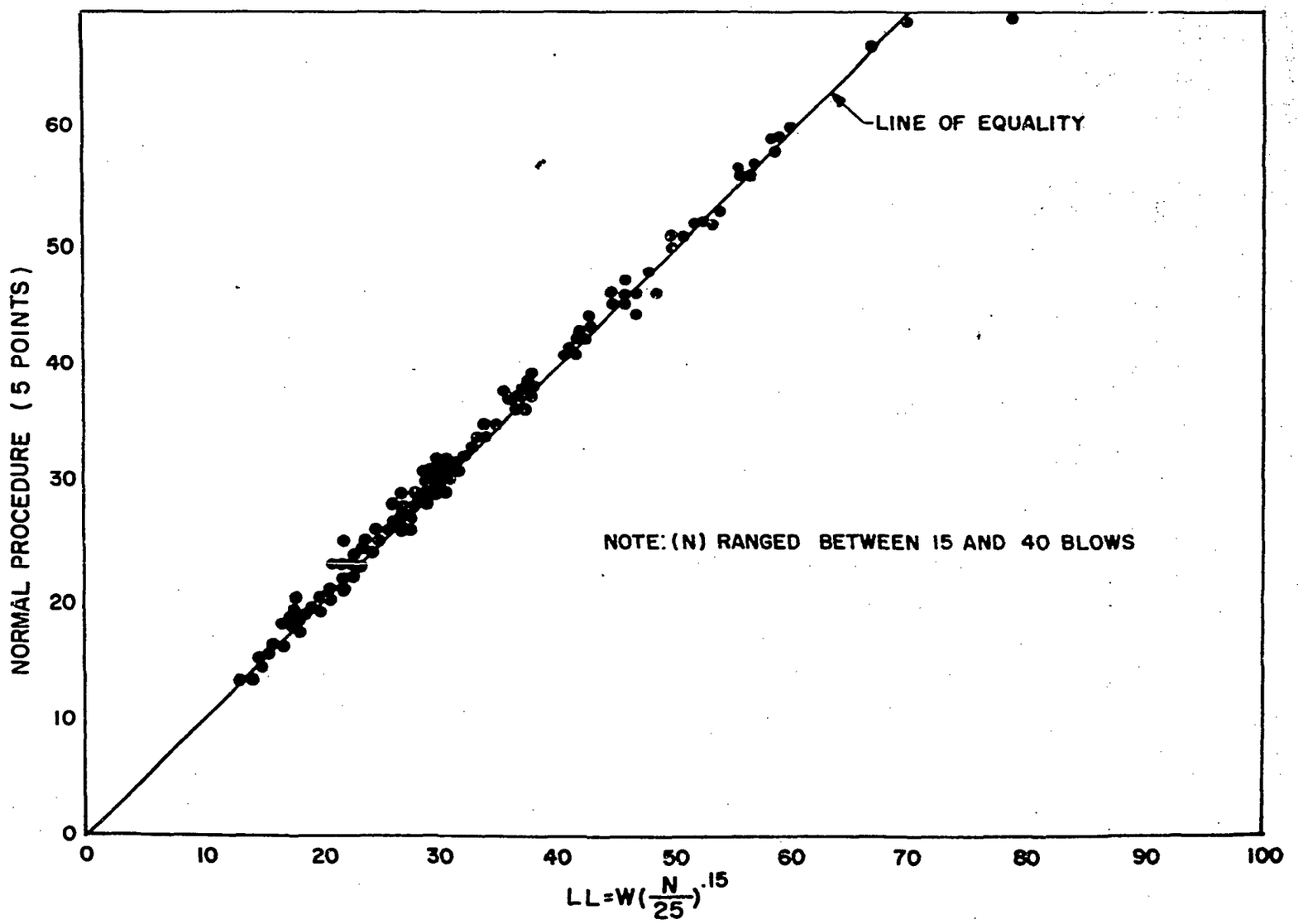

FIGURE 5.3 – COMPARISON BETWEEN 5 POINTS LIQUID LIMIT AND LIQUID LIMIT DETERMINED BY SINGLE POINT METHOD

Test	AASHO
10) *Sand Equivalent Value*	T 176-70

Comments: The susceptibility to change of engineering properties due to dehydration can be estimated by the aggregation index, the ratio of the oven dried sand equivalent value to the sand equivalent value at the field moisture content. The sand equivalent test is run in accordance with AASHO T 176-70 except for the field moisture sample which is not dried prior to testing. An aggregation index value greater than 2 indicates the soil may be susceptible to change in engineering properties upon drying.

SUMMARY

The recommended testing procedures for engineering evaluations of tropical soils are outlined above. The standard procedures are referenced to their source for further details. The modifications required to obtain the proper evaluations are noted.

A procedure of identifying the tropical soils which change properties is also presented. In addition special preparation procedures are outlined for these soils. Special preparation is necessary prior to determining the moisture-density relationship, gradation, Atterberg limits and CBR values.

The rationale for a single point method of determining the liquid limit is included. A coefficient established during the African Study is shown in the following equation:

$$LL = W\left(\frac{N}{25}\right)^{.15}$$

A comparison of the single point method and the standard laboratory 5 point procedure shows that the single point method can be used for tropical red soils.

A moisture-curing time is introduced prior to establishing moisture-density relationships and compaction of CBR samples for laterite gravels. The 12 hour curing time is required to insure a homogeneous moisture content throughout the sample prior to compaction. Each point on the compaction curve should be determined with new material.

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CHAPTER 6
PAVEMENT PERFORMANCE STUDY

INTRODUCTION

A pavement performance study was undertaken in order to analyze existing pavements and identify the relationships between the performance of flexible pavements and the structural components of the pavements section. The ultimate objective was to develop a pavement design procedure based on the interrelationship of performance, traffic, deflection and pavement strength.

Design procedures can be formulated either by analysis of pavement behavior models or by measurement of the performance of pavements under actual service conditions. Pavement behavior models require knowledge of various parameters or at least sufficient experience to estimate the parameters. Unfortunately data on the performance of flexible pavements in the tropics is limited. Developing countries in the tropical areas are and have been engaged in highway construction programs that leave little or no time for research.

The second alternative is to measure the performance of pavements under actual service conditions; this alternative provides a method by which empirical relationships can be established. These empirical relationships require observance of the pavement over long periods of time. The AASHO Road Test is an example of the development of a sound pavement design procedure through statistical analysis of empirical relationships. However, it was a restrictive development procedure which was based on the environmental conditions, soil materials and traffic volumes in the region where the tests were conducted. An analytical approach which has a broader application places greater emphasis on determining the interrelationships between pavement strength and deflection. This analytical approach, sometimes referred to as a "rational design method", has been adopted in this project.

The basic data were obtained through pavement condition surveys of existing pavements in Brazil, South America; Ghana and Ivory Coast, West Africa; and in Thailand, Southeast Asia. The data from Thailand were collected during Phase I of this worldwide study of tropical soils; the data from Ghana and the Ivory Coast were collected during Phase II; and the data for the final Phase III were collected from Brazil.

The analysis and development of the interrelationship between deflection, performance, and pavement strength is presented in this chapter. Details of the pavement condition survey conducted in Africa (Lyon Associates, Inc., et al., 1971) and Southeast Asia (Vallerga et al., 1969) have been provided previously and only the results will be discussed.

SCOPE

Deflection and performance studies of flexible pavements were performed in Thailand, Africa and South America. The conclusions are based on results of the more extensive study in South America where deflection tests were conducted on more than two hundred test sections in Brazil. The results of the African study have been published previously (Todor and Yeboa, 1971).

The first part of this chapter presents a relationship between the deflection characteristics and (1) the subjectively rated performance of the pavement sections and (2) cracking of the pavement surface. The last part presents the structural evaluations, including (1) the establishment of a suitable structural coefficient for each layer for use in design calculations and (2) the analysis of the relationship between deflection characteristics and structural components of the pavement section.

DEFLECTION AND PAVEMENT PERFORMANCE

Pavement deflection studies have been used to evaluate the performance of flexible pavements for over 20 years. Recent researchers, Kung (1967), Todor and Yeboa (1971), Huang (1972), and Miura (1972) to mention a few, transformed the use of Benkelman beam into a valuable method of studying the behavior of flexible pavements under imposed vehicle loads. Lai (1972), Kung (1967) and Huang (1972) have provided relationships between pavement strength and deflection characteristics. Miura (1972) has used deflection as a means to calculate the resilient modulus of the pavement section:

$$\text{Resilient Modulus} = \frac{2(1-\mu)^2 pa}{t} \quad \ldots\ldots\ldots\ldots (6.1)$$

In this equation p is the tire pressure (kg/cm^2); a is the radius of load (cm); t is the Benkelman beam deflection (cm); and μ is the Poisson' ratio = (0.25). In addition, a relation between deflection, CBR and thickness was used to establish the relative strength coefficients of flexible pavement components in a study of flexible pavements in Africa (Lyon Associates, Inc. et al., 1971).

Deflection measurements also provides an evaluation of the structural uniformity of pavement sections. The coefficient of variation is given by the following:

$$\text{Coeficient of Variation} = \frac{\text{Standard deviation}}{\text{Average deflection}} \quad \ldots (6.2)$$

Irick and Hudson (1964) suggested a maximum variation of 25 percent for the selection of existing pavement sections or in construction control specifications for new pavements to be used in the AASHO Satellite Study Program. The 25 percent variation represents a degree of control for research study and is probably the exception rather than the rule under normal quality control practice. In this study a variation of 35 percent has been adopted to define the maximum variation to be expected when generally accepted construction quality control is employed in construction.

Deflection Characteristics Study

The deflections were obtained at six stations which are 30 m (98 ft) apart, within a 150 m (490 ft) test section. Measurements were obtained for both inside and outside

wheel paths. Deflections were measured at various distances from the point of loading. These latter values provided the data necessary to compute the parameters of the deflection basin.

The slope of the deflection basin is determined by the method proposed by Kung (1967) in which the maximum $\tan\phi$ value is used to define the slope of the deflection basin (see Figure 6.1). The rebound deflection and the slope of the deflection basin are evaluated to define the maximum value of each parameter that insures a satisfactory pavement performance for a given volume of traffic.

EQUIPMENT

Deflection beams used in a majority of the deflection tests have a lever arm ratio of 4:1. The micrometer gage graduation is in inches and the dial divisions read to the nearest .001 of an inch. Therefore, the pavement deflection for each division is. 004 inches. The free-span (probe beam toe to front supports) is 256 cm (100 in) which allows deflection basins up to 512 cm (200 in) before movement of the front supports are affected.

The load-vehicles used were the two axle variety which have a single rear axle equipped with dual wheel assemblies. The trucks were loaded to provide a 780 kg (18,000 lb) rear axle loading. Rear tire pressures were maintained between 583 and 617 kN/m^2 (85 and 90 psi).

PROCEDURE

The rebound procedure was used in determining the deflection characteristics of the pavements (Benkelman et al., 1962). The loaded vehicle is placed over the test location. Two deflection beams are positioned behind the rear axle with each beam toe between and in the center of one of the dual wheel assemblies. An initial zero reading is recorded. The truck is advanced a distance of 30 cm (12 in) and a second reading is recorded with the beam remaining in the original position on the pavement. The procedure is repeated at distances of 45, 60, 90, 120, 180 and 600 cm (18, 24, 35, 47, 70 and 236 in). A number of the test sections included a measurement at 256 cm (100 in) which is the distance from the toe of the beam to the front supports of the beam. Plots of the deflection basins indicated that the supports were never within the basin.

TEMPERATURE CORRECTION

All deflection measurements were corrected to a standard deflection temperature of 21°C. It was realized this temperature is unrealistically low to be used as a standard in the tropics, where pavement temperature is about 36°C ± 9°. Nevertheless, the standard temperature of 21°C has been used to provide a means of comparison with deflection data in the literature.

The temperature correction recommended by the Asphalt Institute (Kingham, 1969) has been applied to the test data. This is shown in Figure 6.2. The two curves were formulated from the analysis of the data obtained by six independent research agencies including the AASHO Road Test and the Road Research Laboratory (U.K.) studies. Curve A includes data primarily from pavements over granular base materials and represents strong support to the thin asphalt layer, less than 10 cm (4 in). Curve B represents data from thick asphalt pavements, 10 cm (4 in) or more, laid directly on weak subgrade. Kingham recommends the use of curve A in most circumstances. This is the procedure used in the Phase III tests.

The correction of the slope of the deflection basin is somewhat more complicated than the temperature correction applied to the rebound deflection. Lai (1972) has examined the effect of temperature on the deflection beneath each of the five sensors of the Lane-Wells Dynaflect.* The results of his study illustrate the complex nature of the effect of temperature on the curvature of the deflection basin. The effect of temperature diminishes as the distance from the point of loading increases. Therefore, two methods are used to correct the curvature ($\tan\phi$ of the deflection basin) to the standard temperature. In the first method the temperature correction factor is applied to both deflection points used in the calculation of $\tan\phi$. In the second method the correction factor is applied to only the deflection at the point of loading. The "true" corrected curvature lies between the two values. The average of the two corrected values is used as the "best" corrected value. The following equation is used in applying the temperature correction:

$$\tan\phi = \frac{2\Delta_1 Tc - \Delta_2(1 + Tc)}{2L} \quad \ldots\ldots\ldots\ldots\ldots (6.3)$$

In the equation Δ_1, is the maximum magnitude of deflection; Tc is the temperature correcting factor; Δ_2 is the magnitude of deflection at the point of inflection and L is the distance from the point of maximum deflection to the point of inflection.

The temperature of the pavement was determined by driving a small diameter spike into the pavement, filling the hole with oil, inserting a thermometer and recording the temperature after the oil had reached equilibrium with the pavement temperature.

Traffic Analysis for Test Sections

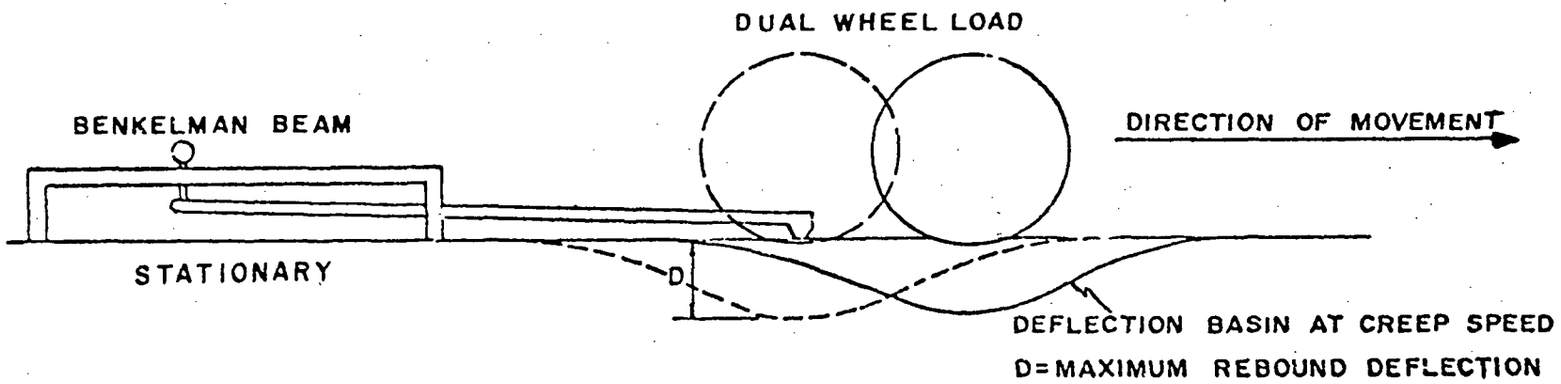
Average and Total Traffic Analysis:

The traffic data available included both the Brazilian National Highway Department (DNER) and the individual State Highway Department (DER) traffic counts. The DNER counts included the following vehicle classifications:

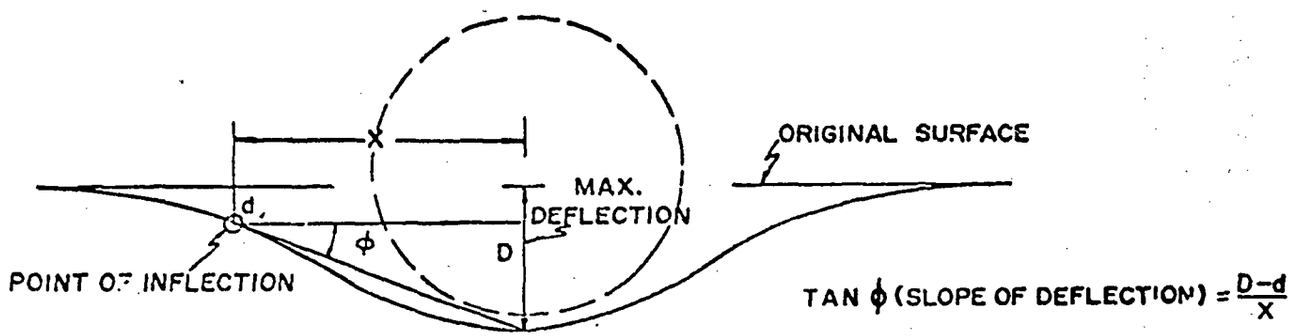
1. Automobiles
2. Buses
3. Light trucks
4. Medium single rear axle trucks
5. Heavy dual rear axle trucks
6. Semitractor-trailer units

The DER classification included the following groups:

* The Dynaflect comprises a dynamic force generator together with five deflection sensing devices mounted on a trailer and a deflection measuring system which is carried in the towing vehicle. More details are given by Lai (1972).



A. MEASUREMENT OF MAXIMUM REBOUND DEFLECTION (MRD)



B. MEASUREMENT OF THE SLOPE OF THE DEFLECTION CURVE (after Kuang-Yualn Kung. 1967)

FIGURE 6.1 – MEASUREMENT OF DEFLECTION PARAMETERS MRD AND TAN ϕ

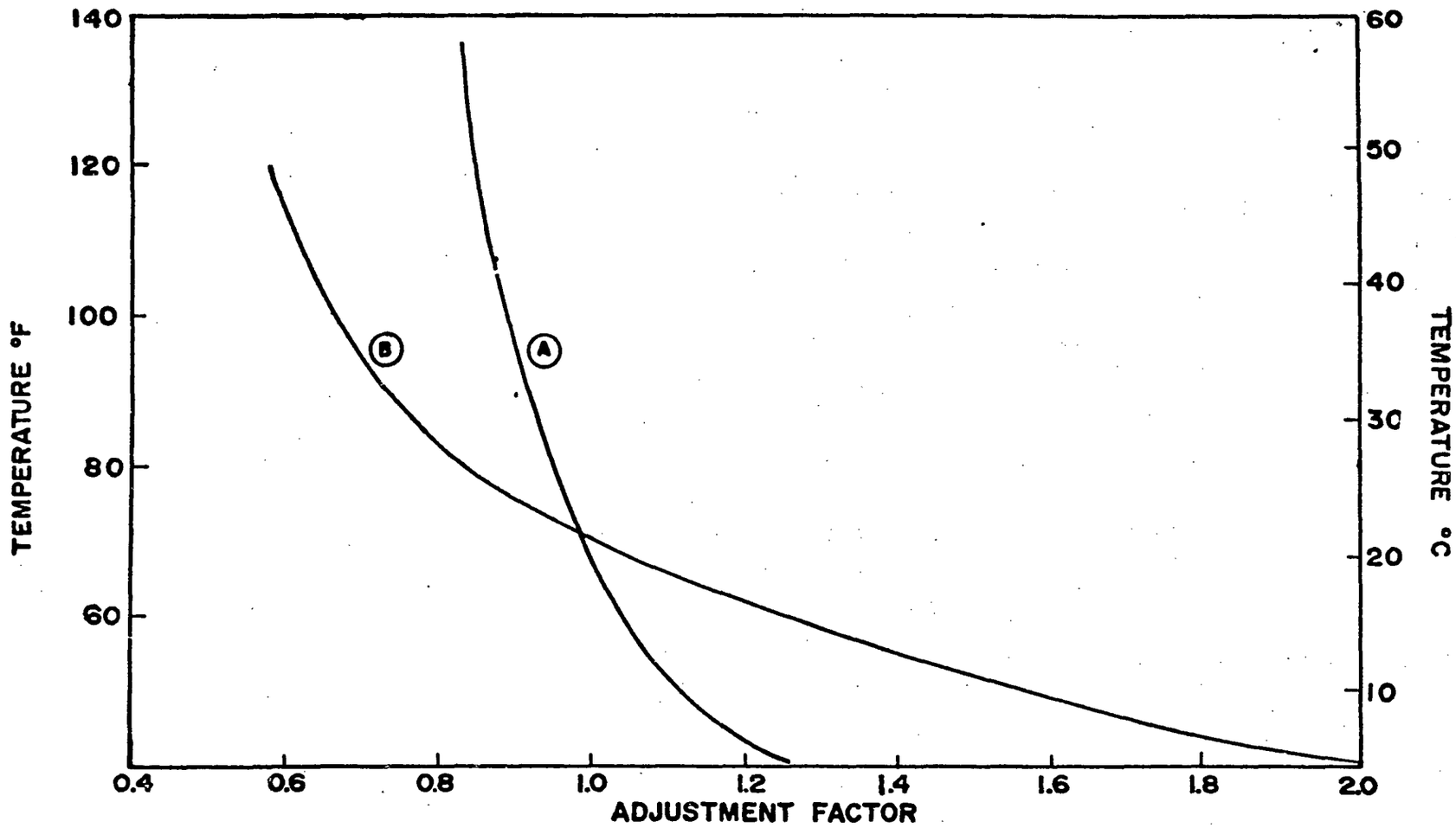


FIGURE 6.2 – ADJUSTMENT FACTOR CURVES FOR USE IN CORRECTING BEAM DEFLECTION TO A STANDARD TEMPERATURE OF 21°C

1. Automobiles
2. Buses
3. Trucks

The total number of vehicles within each group that has traversed the test sections since construction or last pavement overly was determined from available traffic data. In some cases, traffic data were available for each year of pavement life. In other cases, the data was extrapolated to obtain the total number of vehicle applications.

TRAFFIC EQUIVALENCE FACTOR

A loadometer study which was performed on two major highways in the state of Minas Gerais by DNER (1970) was analyzed to establish the loading patterns of commercial vehicles. This was needed to determine the traffic equivalence factor (TEF) for each of the truck-unit classifications. The commercial axle weights as documented in the DNER study are the weights of loaded single, tandem and triple axle vehicles. The axle-load distribution and frequency which are shown in Figure 6.3 are from samples of vehicles traveling in both directions. Therefore, based on the DNER data the following axle weights are selected to represent typical loaded commercial vehicles in Brazil.

Single Axle		
Average weight	29 kN	(6 600 lb)
Characteristic weight	88 kN	(19 800 lb)
Tandem Axle		
Average weight	147 kN	(33 000 lb)
Characteristic weight	182 kN	(41 000 lb)
Triple Axle		
Average weight	225 kN	(50 600 lb)
Characteristic weight	245 kN	(55 000 lb)

For the above, characteristic weight is the average plus one standard deviation.

The traffic equivalence factors (TEF) were calculated in accordance with the AASHO Interim Guide (1972) for each of the truck classifications. The TEF with triple axles vehicles were extrapolated from the relation of single and tandem axle equivalencies. The unit equivalence factor (UEF) for the various truck classifications are shown in Table 6.1. The UEF for each classification was calculated using the percentage of loaded and unloaded trucks for each classification. The UEF for other vehicles which were not included in the loadometer study was calculated utilizing information obtained from manufacturers and commercial users of the equipment; for example, the UEF for a commercial bus was based on weights furnished by local transportation companies.

The percentages of loaded and unloaded trucks indicated in the loadometer study were assumed to exist throughout Brazil. The loaded weights were also assumed to be representative of the loads throughout Brazil. The UEF used in Phase III for the DNER and DER traffic classification systems are shown in Table 6.2. A general relationship was derived between average daily number of vehicles and the corresponding daily equivalent 80 kN (18 kip) single axle load applications (DE18KSAL) for various percentages of truck traffic. The relationship is shown in Figure 6.4. It is recommended as a guide in Brazil to determine the average daily equivalent 80 kN (18 kip) axle load applications when loadometer studies are not available. For convenience the 80 kN (18 kip) loading is referred to as a "standard load" in the remainder of this chapter.

ACCUMULATED EQUIVALENT STANDARD AXLE LOAD APPLICATIONS

The traffic analysis of the loadometer study were used to determine the accumulated equivalent "standard" axle load applications experienced by each test section. The average daily percentage of each vehicle group was multiplied by its corresponding unit traffic equivalence factor. The groups were summed up to give the combined traffic equivalence factor which in turn were multiplied by 365 (days); by the number of years of pavement existence and the average daily traffic to obtain the accumulated equivalent "standard" axle loadings;

$$AE18KSAL's = 365(N)(ADT) \sum_{\text{Tractor-trailer units}}^{\text{Cars}} (n)(UE18KSAL) \quad \ldots (6.4)$$

In this equation N is the age of pavement in years; ADT is the average daily traffic; n is the percent of ADT of each group; and UE18DSAL is the unit equivalent "standard" axle load. The above equation is similar to the equation suggested by Deacon and Deen (1969).

Present Serviceability Rating

GENERAL

The success or failure of a pavement design is ultimately determined by the highway users, drivers and passengers.

The method of pavement evaluation used in Phase III was a subjective rating of the riding quality of the pavement as a vehicle traverses the selected site at a constant speed of 80 kph (50 mph). Standard vehicles were used in the evaluations. In Phase III, standard DNER Chevrolet Carryalls were used, except in two or three locations where this type of vehicle was not available. Individual ratings were obtained by a minimum of two persons. The average of the individual ratings was assigned to the section. This rating is often referred to as the present serviceability index; or more appropriately, the riding comfort index (RCI) as used in Canada.

PAVEMENT RATING

The following was used as a rating scale to assess the riding comfort of the pavement sections:

Rating
9. Excellent
8. Very Good
7. Good
6. Satisfactory
5. Borderline
4. Unsatisfactory
3. Poor
2. Very Poor
1. Excessive Deterioration

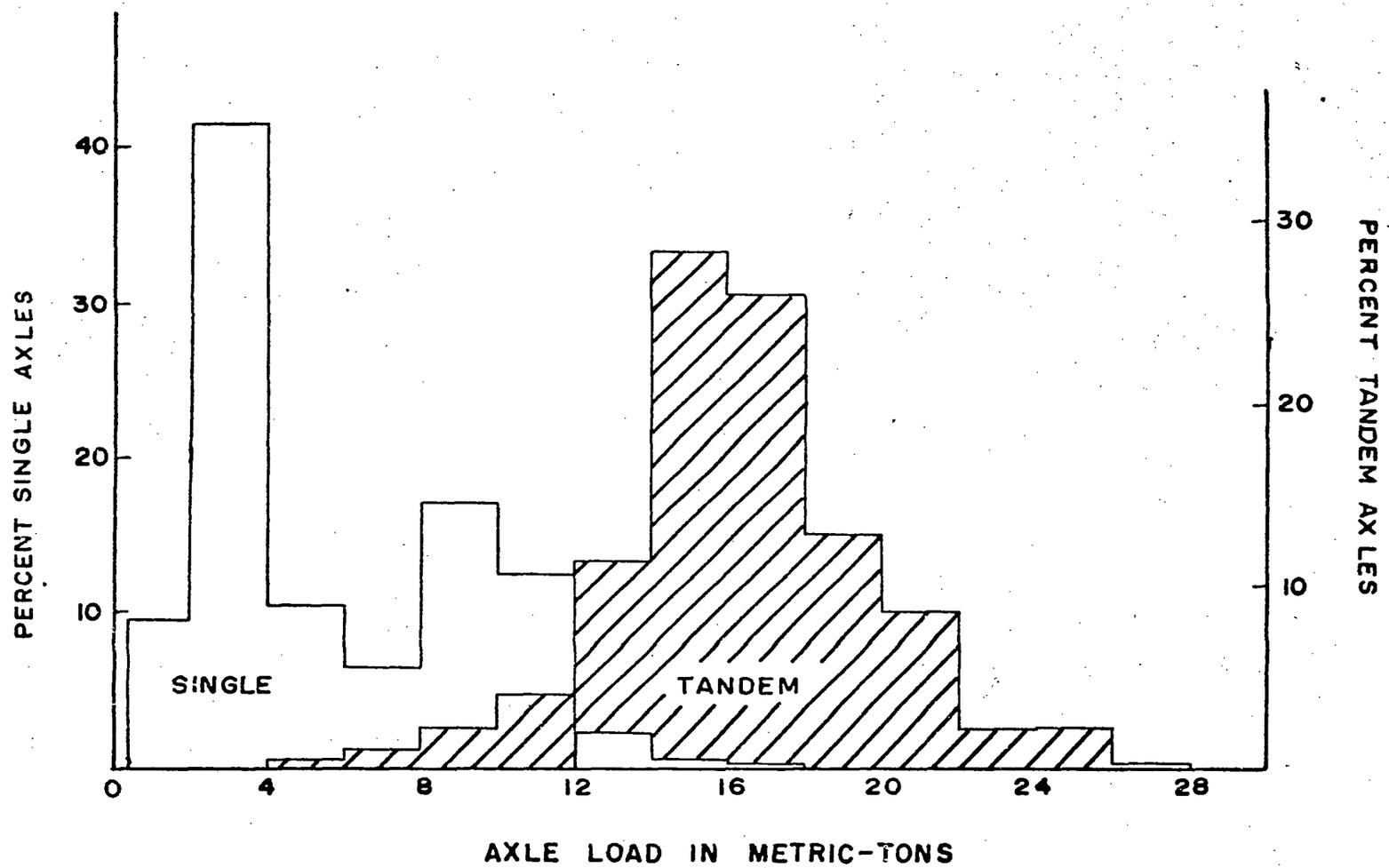


FIGURE 6.3 – FREQUENCY OF THE LOAD DISTRIBUTION ON AXLES OF COMMERCIAL VEHICLES IN BRAZIL, IN PERCENT

TABLE 6.1
Unit Equivalent Standard Axle Loading Factors

Code	Loaded	Empty	UE18KSAL[a]
Automobiles			0.0002
Buses			0.102
Light Trucks	0.0309	0.0006	0.019
Medium Single Rear Axle Trucks	2.129	0.0011	1.236
Heavy Dual Rear Axle Trucks	1.474	0.0046	1.018
Semitractor-Trailer Units			
2 Axle Tractor Single Axle Trailer	5.641	0.0170	3.222
2 Axle Tractor 2 Axle Trailer	5.233	0.0176	2.991
2 Axle Tractor 3 Axle Trailer	4.054	0.0185	2.319
3 Axle Tractor 2 Axle Trailer	2.992	0.0185	1.713
3 Axle Tractor 3 Axle Trailer	1.810	0.0181	1.040
2 Axle Truck 2 Axle Trailer	8.049	0.0051	4.589
3 Axle Truck 2 Axle Trailer	2.792	0.0072	1.594
3 Axle Truck 3 Axle Trailer	2.454	0.0073	1.402

a Calculated from percentages of loaded and unloaded trucks.
NOTE: UE18KSAL = unit equivalent 18 kip (80kN) single axle load.

TABLE 6.2
Equivalent Standard Axle Loading Factors for DNER and DER Classification Systems.

Code	UE18KSAL	DNER	DER Classification
Automobiles	0.0002	0.0002	0.0002
Buses	0.102	0.102	0.102
Light Trucks	0.019	0.019	
Medium Single Rear Axle Trucks	1.236	1.236	1.194[b]
Heavy Dual Rear Axle Trucks	1.018	1.018	
Semitractor-Trailer Units		2.901[a]	

a b – Calculated from values shown in TABLE 1 using the frequency distribution of each unit.
NOTE: UE18KSAL = unit equivalent 18 kip (80 kN) single axle load.

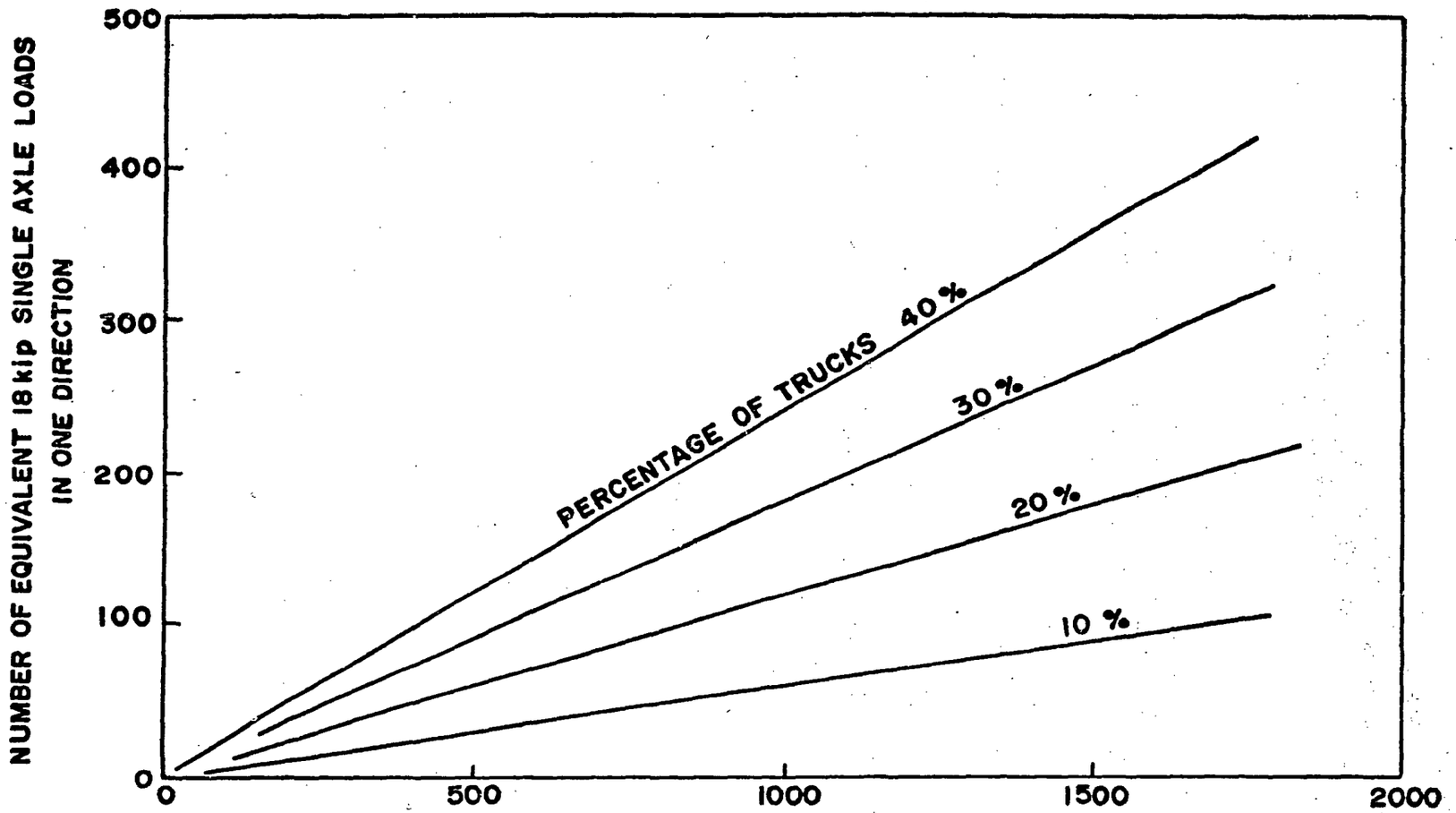


FIGURE 6.4 – GENERAL RELATION BETWEEN TOTAL DAILY NUMBER OF VEHICLES AND EQUIVALENT 18 KSAL-CURVES BASED ON VEHICLE AND LOAD DISTRIBUTION OBTAINED FROM LOADMETER STUDY

Pavements with ratings above five were considered as performing satisfactorily, regardless of intended life. Pavements with ratings of five or less were considered as terminated or in need of major repair with respect to design life. It was recognized that individual opinions introduce variations; however, in most cases the variation of assigned values were not very large. In Phase III ratings differed by plus or minus one point from the average.

Results and Discussion

DEFLECTION AND PERFORMANCE

The relationship between deflection and performance was established for the outside wheel path (OWP). The OWP controls the performance of the pavement since it is generally the weakest area of the pavement. The OWP offers less support to the wheel loads than the inside wheel path (IWP) due to the greater moisture ingress and the lower lateral support offered by the shoulder material. High deflections occasionally experienced in the IWP are generally associated with construction deficiencies of thickness and compaction.

A generally accepted practice is to correlate the performance of the pavement to the representative deflection. The representative deflection is the value of the mean plus two standard deviations. This provides a deflection level that, theoretically, is exceeded in only two percent of the length of the pavement section (Kingham, 1969). Although limited in area, the weaker sections which display larger deflections control the performance of the pavement. Huculak (1966) states that the average deflections do not provide a true indication of the structural strengths of a pavement section unless the variability within the section is also known. Two sections may have the same average deflection but if one has a much greater standard deviation it exhibits greater weakness and will not sustain as much traffic loading as the other.

The relationship between representative deflection and performance is shown in Figure 6.5. The data includes the results from Southeast Asia (Vallerga et al., 1969), Africa (Lyon Associates, Inc. et al., 1971) and South America. The recommended limits for deflection based on the Phase II and III studies are shown. The limiting deflection criterion results adopted by the Asphalt Institute (Kingham, 1969) is also shown.

The Asphalt Institute criterion was established with deflection data obtained in areas of temperate climate where deflection tests are usually conducted during the spring thaw period. Subgrades, subbases and base courses are at their lowest strengths, with a corresponding higher pavement deflection. Canada (Ontario) uses a factor of 1.8 to convert fall deflections to spring deflections. These high deflections occur during a few months of the year; for example: the high spring deflections occurred during March, April, and May at the AASHO Road Test (Benkelman, 1972).

Although seasonal variations in deflections have not been established in Brazil, there is little likelihood that they are as great (80 percent) as those encountered in temperate zones. Seasonal variations in the tropics are due to the wet and dry periods; however, recent studies of moisture changes beneath pavements in tropical countries indicate that seasonal variations in subgrades are small. Phase III deflection testing in Brazil was conducted during the rainy season.

If one uses the short-term spring thaw deflections to establish the deflection-performance criteria, as the Asphalt Institute does, the permissible deflections will be greater than those established by more uniform seasonal deflections. The higher permissible deflections of the Asphalt Institute are apparent at the lower traffic values when comparing the two deflection criteria shown in Figure 6.5. As traffic increases pavement damage increases during the critical thaw season. Therefore, the permissible deflection was reduced more steeply than the recommended curve for tropical regions. The two criteria are in closer agreement at the higher traffic values.

The recommended criterion shown in Figure 6.5 does not exclude all the Phase II and III sections with unsatisfactory ratings. It does represent a confidence level of approximately 95 percent, i.e. only five percent of the pavements meeting the deflection criterion were rated unsatisfactory. Since the sections were rated on riding comfort, the unsatisfactory pavements located below the curve have low deflections, as seen in Figure 6.5. The uneven surfaces in these cases may be due to initial construction deficiencies.

PERMISSIBLE DEFLECTION AND DESIGN DEFLECTION

The curves shown in Figure 6.5 represent deflection criteria which do not contain sufficient information to use in design. The design deflection should be an average deflection in which the mean plus two standard deviations does not exceed the value indicated by the curve. Therefore, the design deflection is governed by the degree of uniformity obtained in the final pavement structure; and that uniformity depends upon the variations in the natural subgrade, borrow materials, construction practices and the effectiveness of the quality control program.

The test sections in Brazil were examined to determine the amount of variation that occurred under normal Brazilian construction procedures. The variations were plotted in the form of a bar graph in Figure 6.6. The average coefficient of variation is 24 percent; 50 percent of the sections are lower than 24 percent and 50 percent are greater. The maximum variation is as high as 83 percent. An analysis of all test sections constructed or overlayed in the past five years indicates an average variation of 20 percent which shows that pavement uniformity has improved during this period. It is believed that the extreme variations analyzed were the result of overlays which were placed on existing pavements with already high variations. Overlays do not necessarily reduce the percent variation although the average deflection and the extreme deflections may be reduced. Huculak (1966) considers that a coefficient of variation of up to 35 percent implies adequate quality control during construction. Since the design is generally based on the assumption that adequate controls will be exercised during construction a maximum variation of 35 percent was used in determining the recommended deflection criterion (Table 6.3).

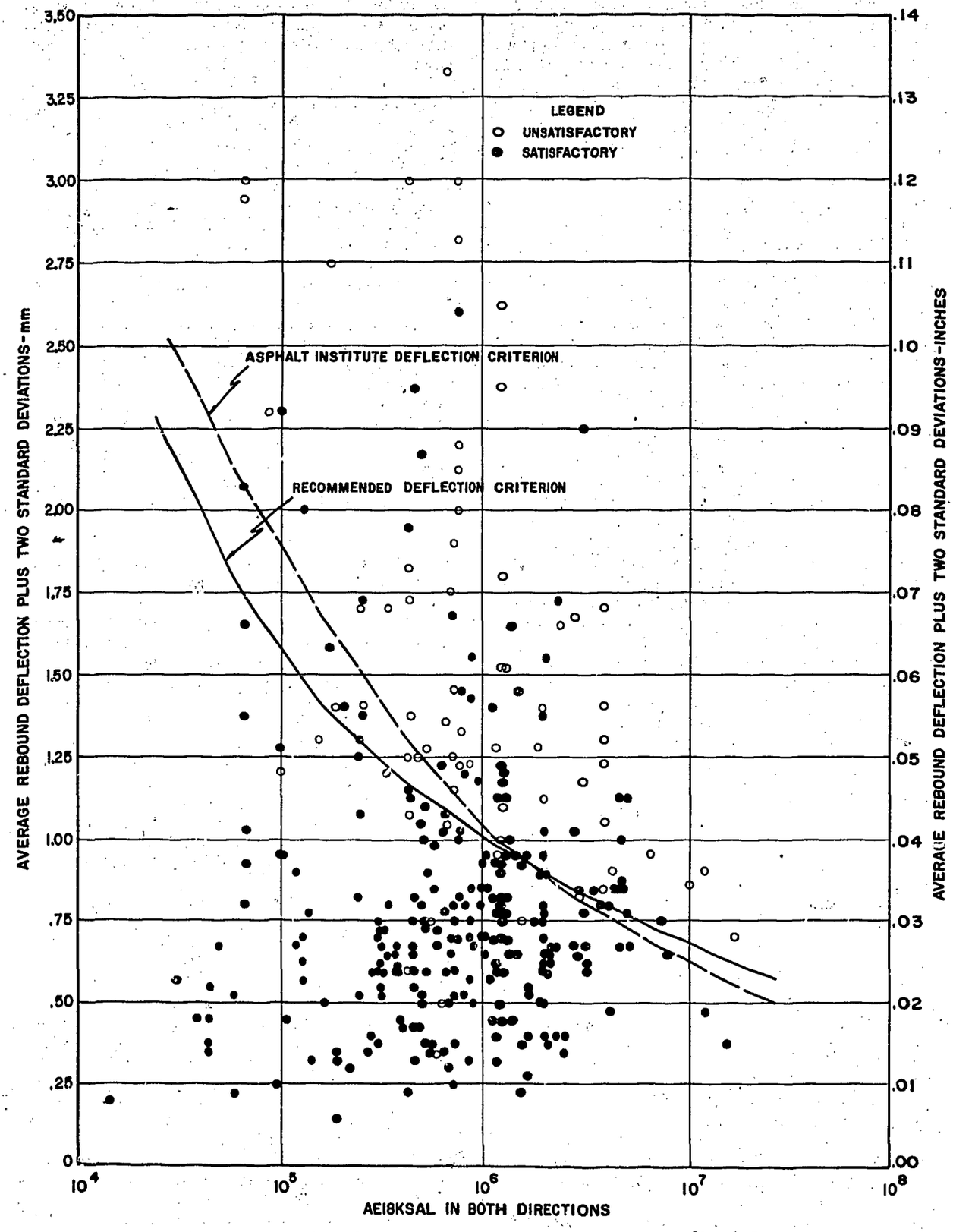


FIGURE 6.5 – RELATIONSHIP BETWEEN DEFLECTION AND PERFORMANCE

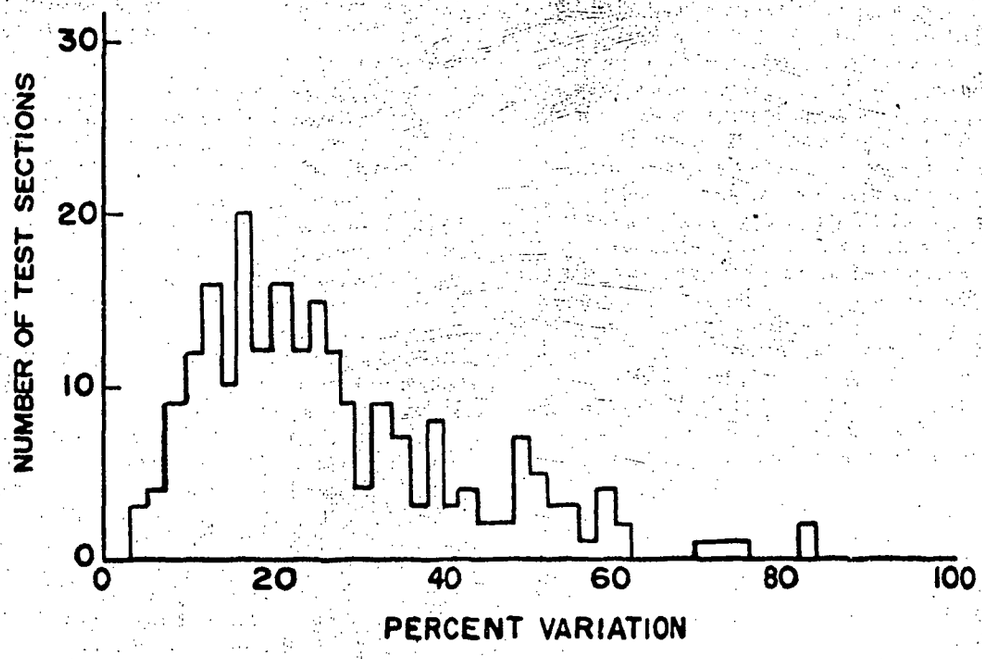


FIGURE 6.6 — PERCENT VARIATION WITHIN THE TEST SECTION

TABLE 6.3
Average Values of Deflections for Design Criteria

AE18KSAL (Both Directions)	African Data	Asphalt Institute a	Recommended b
10^4			1.671 mm (.066 in)
10^5	0.914 mm (.036 in)	1.371 mm (.054 in)	0.965 mm (.038 in)
10^6	0.787 mm (.031 in)	0.762 mm (.030 in)	0.609 mm (.024 in)
10^7	0.686 mm (.027 in)	0.457 mm (.018 in)	0.432 mm (.017 in)
10^8			0.254 mm (.010 in)

a $\sigma = 0.20\,\bar{X}$
b $\sigma = 0.35\,\bar{X}$
NOTE: σ = standard deviation; $\bar{X}$ = average value
AE18KSAL = accumulated equivalent 18 kip (80 kN) single axle loadings.

A comparison of design deflection in terms of average deflection corresponding to traffic (in both directions) is shown in Table 6.3. Although deflection criteria varies considerably among other researchers, there is reasonable agreement between the Phase II, African, data (Todor and Yeboa, 1971) and the Asphalt Institute data for the 500,000 to 1,000,000 traffic range. The Phase III study in Brazil, supports the overall lower deflection requirements of the Asphalt Institute for the higher traffic range (5,000,000 to 10,000,000 standard axle loadings). Nineteen percent of the Brazilian sections which meet the African criterion for this range of traffic were rated unsatisfactory. Studies in Senegal, West Africa indicated that a linear extrapolation of the African data resulted in an under design of the pavement for the higher traffic volumes (Varallyay, 1973). The higher permissible deflections at the lower traffic range given by the Asphalt Institute are not justified by the Phase III data. Recommended deflection criteria, based on the combined deflection studies in Africa and Brazil are also given in Table 6.3. The Asphalt Institute values are based on a variation in deflection of 20 percent.

Figure 6.7 shows the relation between the coefficient of variation and the design deflection. The design curves demonstrate the importance of uniformity in relation to performance. As an example, if two pavements after construction had an average deflection of 0.864 mm (.034 in) but very different coefficients of variation (0.2 and 0.4) the performance of the two pavements will be far different. The pavement with the lower coefficient of variation (0.2) will have an expected pavement life of 350,00 standard axle loading while the pavement with the higher coefficient of variation (0.4) will only have an expected life of 110,000 standard axle loadings. The pavement with the higher value will require an overlay or an increase in the thickness of the base course to reduce the average deflection to 0.685 mm (0.027 in) in order to extend the expected pavement life to 350,000 standard axle loadings. Increasing the thickness of a pavement to accommodate excessive variations is not a very satisfactory approach to pavement design. Improving material evaluation techniques, construction practices and quality control so that a higher degree of uniformity is obtained is a far more economical approach.

The recommended deflection criteria (Table 6.3) were based on an analysis of all the Phase III, Brazilian data. Once a designer determine that a lower degree of variation can be obtained an adjustment of the recommended design deflection can be made using the curves shown in Figure 6.7.

The deflection criteria shown in Figure 6.7 are given in terms of accumulated traffic in both directions. For one direction the traffic is divided by two, allowing the use of the curves for multi-lane design.

SLOPE OF THE DEFLECTION BASIN AND PAVEMENT CRACKING

Deflection measurements included the determination of the slope of the deflection basin (tan ϕ). The Phase II deflection analysis indicated that the slope of the deflections basin, which is similar to the curvature of deflection, is not related to the total strength of the pavement section. Papers presented at the 1972 Third International Conference on the Structural Design of Asphalt Pavements support this conclusion. An example is the conclusion drawn by Huang (1972)

> "It was found that the curvature-tensile strain ratio depends strongly on the thickness of the asphalt layer, but to a much less degree on the modulus ratio, especially when the asphalt layer is thick."

In an earlier paper Huang (1971) concludes that the tensile strain in the asphalt layer is related to the curvature and the thickness of the asphalt layer. Therefore, the tensile strain in the asphalt layer is essentially independent of the thickness of the untreated base course and the properties of the structural courses.

Kung (1967) indicates that the slope of the deflection basin has a direct relationship with the development of fatigue cracks in asphalt pavements.

The critical tensile strains which cause the development of fatigue cracks in asphalt mixtures after various load applications have been determined from laboratory tests conducted by Heukelon and Klomp (1962). Critical strain values corresponding to a given number of standard axle load applications, as shown in Table 6.4, have been formulated from these results (Dorman and Metcalf, 1965). The critical tensile strains have been converted to slopes of the deflection basins (tan ϕ). Considerations are given to temperature corrections and conversion from single wheel deflections to dual wheel deflections.

Pavements (asphalt concrete) with noticeable cracks are plotted in Figure 6.8. The figure also shows the Dorman and Metcalf criterion for critical tensile strains in terms of tan ϕ. The sections were plotted in reference to accumulated traffic and tan ϕ (average tan ϕ plus two standard deviations). The symbols represent various stages of crack development, as explained in the figure. The critical tan ϕ values in Figure 6.8 are satisfactory criteria of crack initiation for various levels of traffic. Some cracking is tolerable within the life of a pavement. Therefore, the designer could accept the higher tan ϕ values represented by the suggested limits shown in Figure 6.8.

The tabulated results of the deflection and pavement performance study conducted during Phase III are given in Table A6.1 of the appendix to this chapter.

DEFLECTION AND PAVEMENT STRENGTH

A relationship between deflection and pavement strength provides the basic requirements for a structural design procedure for flexible pavements. The measurement of the thickness of individual structural layers and the evaluation of the strength of each component layer is required to establish such a relationship. The measurement of the individual structural layers is straightforward. The evaluation of the strength of the individual layers is accomplished by any one of several methods (CBR, R-value or triaxial compression tests, etc.). The strength parameter provides an index of the layer's ability to transfer the vehicle load to the next lowest layer at a lower stress level. The lateral distribution of the vertical load (load-spreading characteristic) of each soil layer depends on the magnitude and concentration of the applied stress and the shear resistance of the material. This load-spreading characteristic

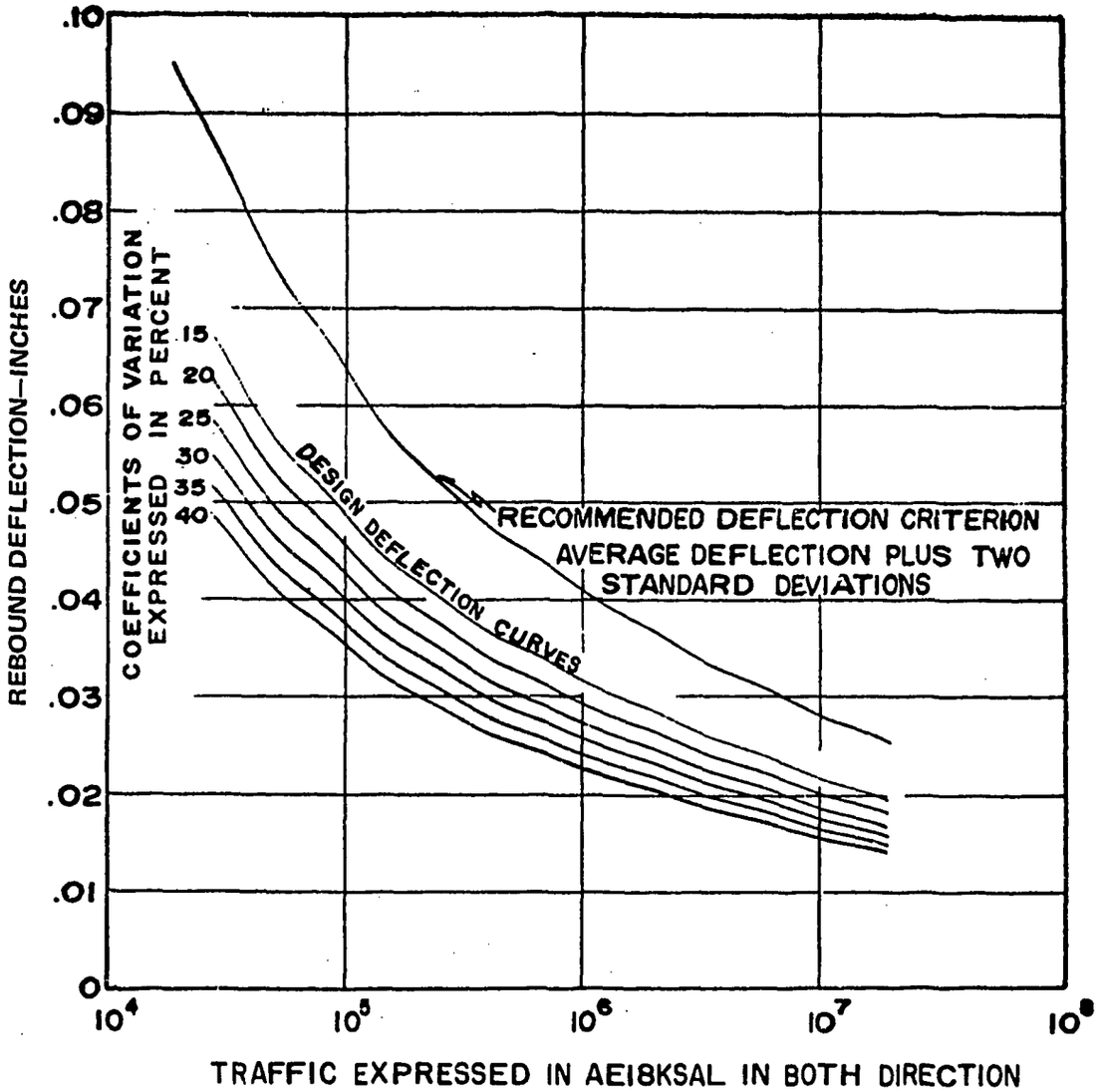


FIGURE 6.7 – RELATION BETWEEN DESIGN DEFLECTION AND TRAFFIC FOR THE PRACTICAL RANGE OF COEFFICIENTS OF VARIATION

TABLE 6.4
Permissible Tensile Strains Under Different Load Applications
(After Dorman & Metcalf, 1965)

Load (Standard Axles)	Strain (e)
10^5	2.30×10^{-4}
10^6	1.45×10^{-4}
10^7	9.20×10^{-5}
10^8	5.80×10^{-5}

NOTE: e in units per unit.

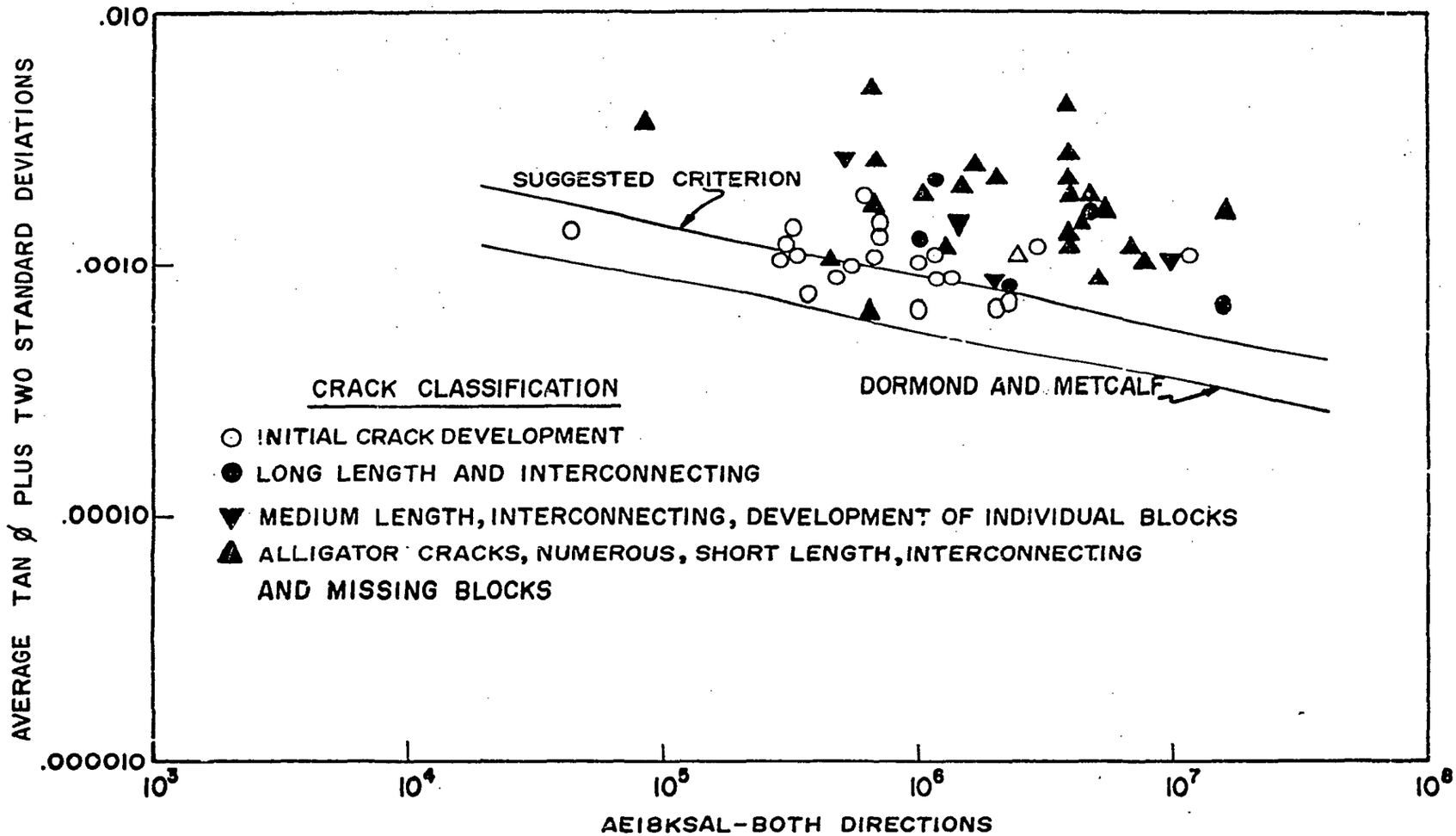


FIGURE 6.8 – RELATIONSHIP BETWEEN THE SLOPE OF THE DEFLECTION BASIN AND CRACKING OF THE PAVEMENT SURFACE FOR VARIOUS STANDARD AXLE APPLICATIONS

of a given layer depends upon the strength of the material and its position within the pavement structure.

The most common method of determining the relative strength of soils in flexible pavement design is the California Bearing Ratio. This method is used more than all others combined. The term "relative strength" is preferred since the test provides a ratio of the resistance to penetration of a piston by the soil in question to that of a crushed rock. Basically it is a bearing capacity test. Housel and Ito (1972) demonstrate that perimeter shear and developed pressure can be expressed by a linear equation for bearing capacity. These two stress reactions are the controlling factors in the structural design of flexible pavements. Thus, the CBR test provides a simple, well known and inexpensive test for evaluating the relative strength of unbound soil layers. In view of this Phase II and III pavement evaluations were based on the results of CBR tests on unbound soil layers within the pavement structure. No evaluation of bound or chemically stabliized soil layers are included since such pavement structures were avoided.

The structural evaluation was originally scheduled for each test section where deflection tests were conducted. However, it was possible to complete only 170 sections. Laboratory tests were conducted to determine the index properties and the CBR-density-moisture relationship were determined from laboratory tests conducted on samples from each soil layer within the excavated depth of 90 cm (36 in).

Structural Evaluation of the Test Section

Test pit locations were selected through analysis of the deflection results. The pits were excavated at the station where the deflection most nearly approached the average deflection of the entire section.

The test pits were excavated to a width which was sufficient to perform the required tests. The pits were one traffic lane wide and 90 cm (36 in) deep. Excavation to this depth provides complete examination of the structural layers which are affected by vehicle loadings.

The stress distribution of a 40-kN (9,000 lb) dual wheel assembly loading is shown in Figure 6.9. The distributed vehicle load at a depth of 90 cm (36 in) is less than one percent of the applied vehicle load. The stress distribution shown is that for a homogeneous soil; the distribution beneath pavement sections varies depending upon the thickness and strength of the component layers. However, at a depth of approximately 64 cm (25 in) the stress levels under all flexible pavements are not significantly different. The stresses, with increasing depth, become more independent of the composition of the pavement structure and more dependent of the total thickness of pavement (Yang, 1972).

The thickness of each structural course was measured at the inside wheel path as well as the outside wheel path. The measurements included the thicknesses of the surface, base and subbase. In some sections, there were as many as five or six different material layers encountered within the excavated depth. The thickness of each layer was measured.

Density determinations were conducted in both inside and outside wheel paths of each soil layer. When layer thicknesses exceeded 20 cm (8 in) a density determination was made in the top half of the layer and in the bottom half of the layer.

Samples for laboratory testing were obtained from each of the soil layers encountered during the test pit excavation. Each sample was tested and classified according to the AASHO classification system (M145-66), which is based on particle size distribution, liquid limit and plasticity index. The soils within each group were evaluated by means of the "Group Index".

A moisture-density-CBR relationship was established for each soil layer component of the pavement structure. The CBR analysis of the layer was determined from tests conducted on three samples compacted at the field moisture content. In this study three compactive efforts were used: AASHO Modified Compaction (T180-70); AASHO Standard Compaction (T99-70); and the Brazil standard, about mid-way in compactive effort between the other two. The results provided a CBR-density relationship at the field moisture content. The CBR value selected for each layer was one that corresponded to the density obtained in the field test. Typical results are shown in Figures A6.1a, A6.1b and A6.1c in the appendix to Chapter 6. The tabulation of the engineering and structural properties are shown in Tables A6.2 and A6.3 of the appendix to this

Structural Coefficients

A simple CBR-coefficient relation cannot be used because the performance of a given layer depends on the magnitude of stress imposed and the shear resistance of the material. For example, a layer with a CBR of 70 is used either as a base or as a subbase; obviously the stress levels will not be the same in each case nor will the load-spreading characteristics of the layer be the same under both conditions. Structural coefficients are developed to relate the load-spreading characteristics of a given component layer with regard to its CBR and position in the pavement structure.

Structural coefficients developed during the African study for base, subbase and subgrade layers (Lyon Associates, Inc. et., al., 1970), were used initially in analyzing the Brazilian test sections. However, the multiple layer systems encountered in Brazil illustrate an ambiguity of the terms base, subbase and subgrade when defining the depth of each layer in the pavement structure.

The African coefficients were not used in their original form to analyze the multiple layer systems. These were modified to represent structural coefficients for various depths beneath the pavement surface rather than by layer description. The modified coefficients are given in Table 6.5. The basic structural equation in which the coefficients are used is:

Pavement Structural Index (SI) $= a_1 t_1 + a_2 t_2 \ldots a_n t_n$ ..... (6.5)

In this equation $a_1, a_2, .. a_n$ are the structural coefficients in dimensionless units per cm and $t_1, t_2, .. t_n$ are the thicknesses of the component layers (in cm). The structural index is computed to a depth of 90 cm (36 in). The structural index (SI) is related to the pavement deflection by the following equations:

$$SI = (0.039\ RD)^{-1} \quad \ldots\ldots\ldots\ldots\ldots \quad (6.6a)$$

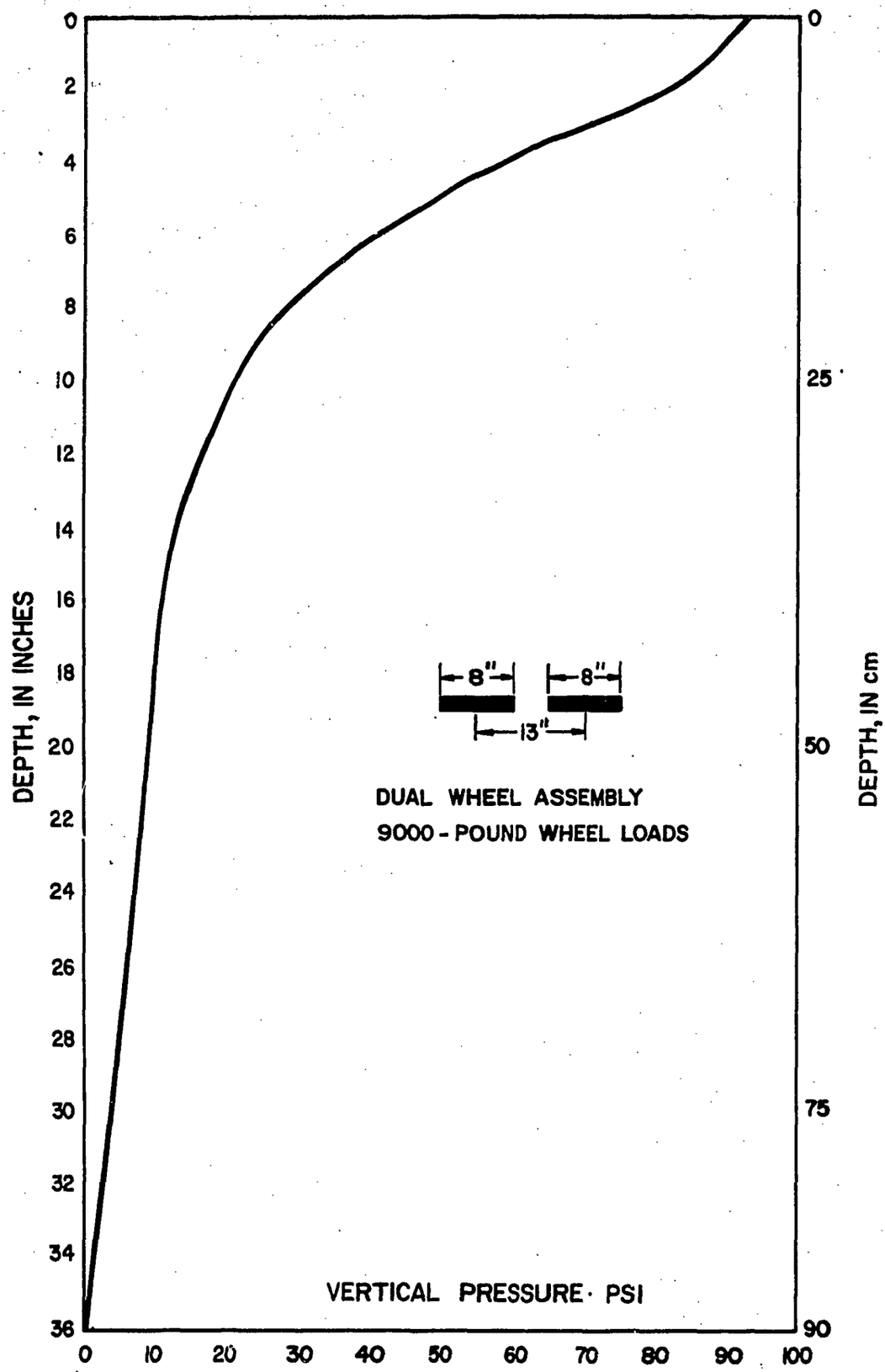


FIGURE 6.9 – STRESS DISTRIBUTION BENEATH THE SURFACE OF A FLEXIBLE PAVEMENT
(after Martin and Wallace, 1958)

TABLE 6.5
Structural Coefficients

Depth 0-25 cm	
CBR	
+100	1.394 (max.)
90	1.232
85	1.167
80	1.102
75	1.037
70	0.940
60	0.552
50	0.383 (min.)
Dept 25-50 cm	
CBR	
+40	0.576 (max.)
35	0.290
30	0.205
25	0.075 (min.)
Dept 50-90 cm	
CBR	
+20	0.481 (max.)
15	0.357
10	0.212
9	0.183
8	0.133
7	0.084
6	0.053
5	0.033
4	0.020
3	0.015
2	0.010 (min.)

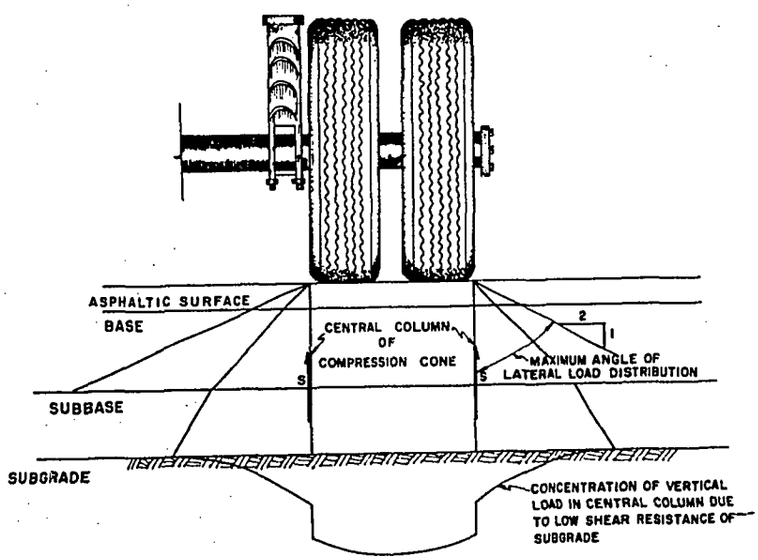


FIGURE 6.10 – PRINCIPLES OF PRESSURE DISTRIBUTION (after Housel and Ito, 1972)

In this equation RD is the measured Benkelman Beam deflections in mm or, for corresponding English units:

$$SI = (RD)^{-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6.6b)$$

In this equation RD is the measured Benkelman Beam deflection in inches.

The structural index is a dimensionless number equal to the numerical value of the reciprocal of the rebound deflection measured in inches or measured in millimeters and reduced by the conversion factor 0.039.

MAXIMUM AND MINIMUM STRUCTURAL COEFFICIENTS

The maximum and minimum structural coefficients, Table 6.5, represent two fundamental principles of the pressure distribution beneath a flexible pavement which are illustrated in Figure 6.10. Housel and Ito (1972) summarized these pressure distribution principles as follows:

> "In the first place there is a definite maximum angle of lateral distribution which defines the compression cone outside of which there is no measurable vertical pressure from the applied load. Second, at increased loads exceeding the shearing resistance of the soil, concentration of vertical pressure develops in the central column of the compression cone resulting from inability of the soil mass to sustain the shearing stresses acting on the perimeter plane."

The first case represents the maximum structural coefficient whereas the second represents the minimum structural coefficient. The maximum and minimum coefficients for each structural depth shown in Table 6.5 were initially determined by judgement and then refined by trial and error computations with the use of those sections displaying extreme ends of the scale of CBR values.

The results of an analysis of test sections with CBR values below the minimum value given in Table 6.5 are shown in Figure 6.11. The deflections of these sections were calculated using equations 6.5 and 6.6 a or b but without assigning structural coefficients to the layers with the low CBR values. The solid points represent sections where the calculated values of deflection are equal to or very close to the measured deflection. The open circles represent sections where the calculated values of deflection are much less than the measured deflections. This implies that the solid points represent layers that have poor load distribution qualities but do not cause excessive elastic deflection in the system. On the other hand, the open circles represent layers that have poor load-spreading qualities and cause excessive elastic deflection in the pavement system. Based on this analysis a minimum thicknesses of cover for various CBR values were established. These are discussed in Chapter 7.

Calculated Deflections vs. Benkelman Beam Deflections

SURFACE TREATMENT PAVEMENTS

The basic equation used in calculating the deflection of a given pavement section with single or multiple surface treatments is combined from equations 6.5 and 6.6.

$$(RD)^{-1} = a_1 t_1 + a_2 t_2 \ldots a_n t_n \quad \ldots\ldots\ldots \quad (6.7)$$

A comparison between the calculated deflection using equation 6.7 and the measured Benkelman Beam deflection is shown in Figure 6.12. Of the sixty five sections available for this analysis seven sections were eliminated due to low CBR values and three pavement sections were eliminated due to low densities within the upper structural layers. The shaded area represents the precision of the Ames dial on the Benkelman Beam which is usually assummed to be one-half the smallest division on the micrometer dial. As the smallest division on the dial is 0.001 inch, the precision is ± 0.0005 inch and the actual deflection for a 4:1 lever arm is ± .002 inch. From a practial point of view any calculated deflection within the shaded band is essentially equal to the measured deflection. The extremes of the band represents plus or minus one dial division on the micrometer. The majority of the points plot within these limits. The exceptions are the open circles which represents sections from the northeast of Brazil which are discussed below. The remainder of the data show a good correlation between the measured deflection and the calculated deflection when considering the precision of the field measurements. A statistical analysis of the data (except for the open circles) gives a standard error of estimate of ± .00291 inches which is very close to the precision of the deflection beams. Some of the scatter can be attributed to the difficulty of "fixing" the toe of the beam on a surface treatment pavement.

ASPHALT CONCRETE PAVEMENTS

The Phase II Africa study did not include any asphalt concrete pavements because there were none suitable for study within the country where the study was conducted (Ghana), nor were there any readily accessible from Ghana. Analysis of the asphalt pavements in Phase III, Brazilian study, determined the structural strength that can be attributed to the asphalt layer.

An asphalt layer provides two structural benefits to the pavement system. First, the asphalt layer provides its own strength and load spreading characteristic and second, it increases the load-spreading characteristics of the underlying unbound structural layers. This second benefit was substantiated by analysis of the test sections. In this analysis the strength of the unbound underlying soil layers were computed from equation 6.5 and compared to the composite strength which was determined from measured field deflections and equation 6.6. In the first analysis, the difference in these strengths (residual strength) is attributed to the asphalt layers. However, when the difference in strength is converted to structural coefficients, as shown in Figure 6.13, the results do not provide an adequate relationship between the calculated structural coefficient and asphalt thickness for use in a design procedure.

In the second analysis it was assumed that for a certain depth beneath the asphalt layer the load-spreading characteristics of the soil layers increase. To substantiate this, the same test sections used in the analysis shown in Figure 6.13 were separated into groups with average asphalt thicknesses of 5,10 and 15 cm (2,4 and 6 in). These curves are shown in Figure 6.14. These curves show that there is a good relationship between increases in the strength of the unbound materials down to a depth of 50 cm for the 5 cm

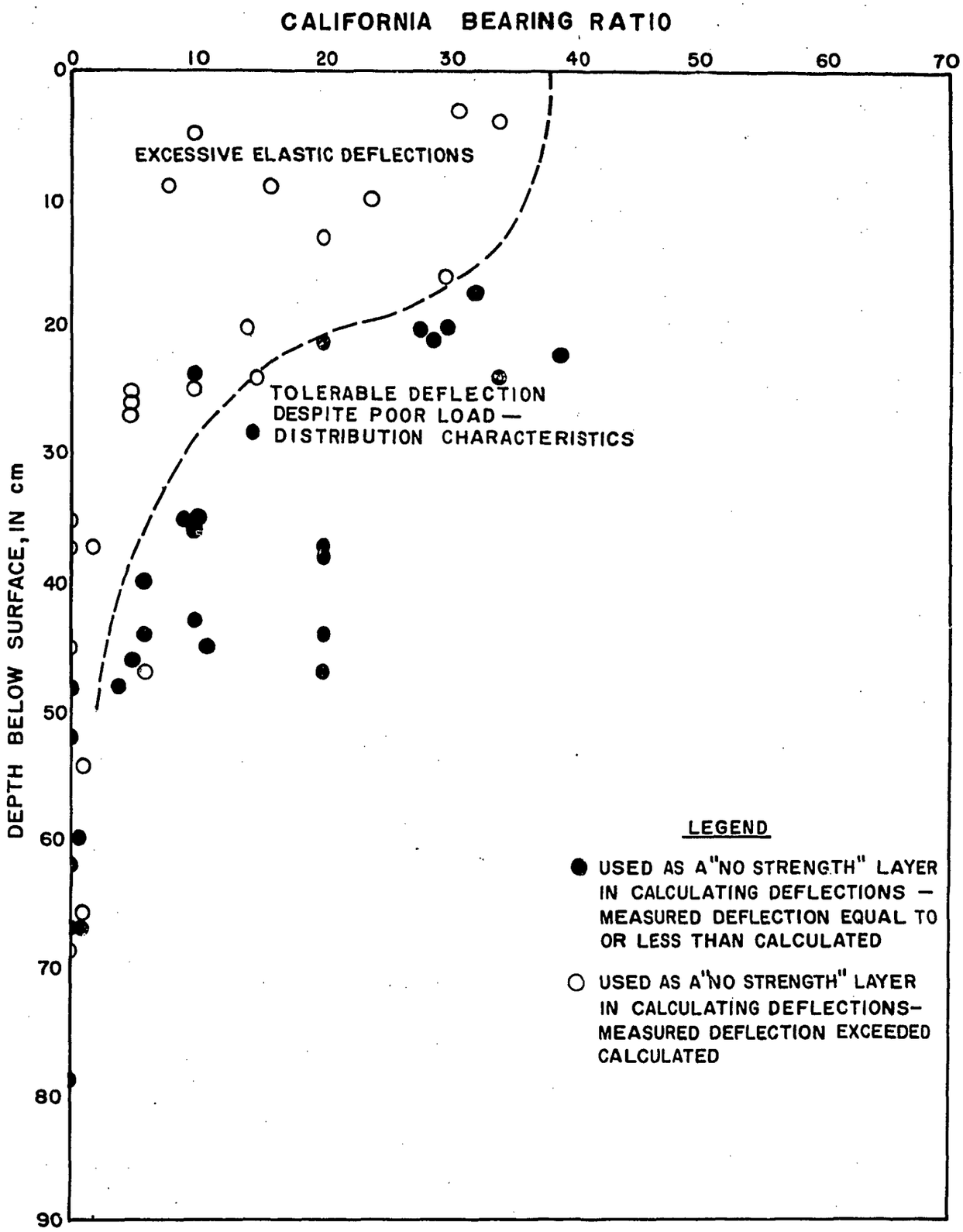


FIGURE 6.11 — CRITICAL CBR VALUES WITH DEPTH BELOW PAVEMENT SURFACE BASED ON FIELD ANALYSIS

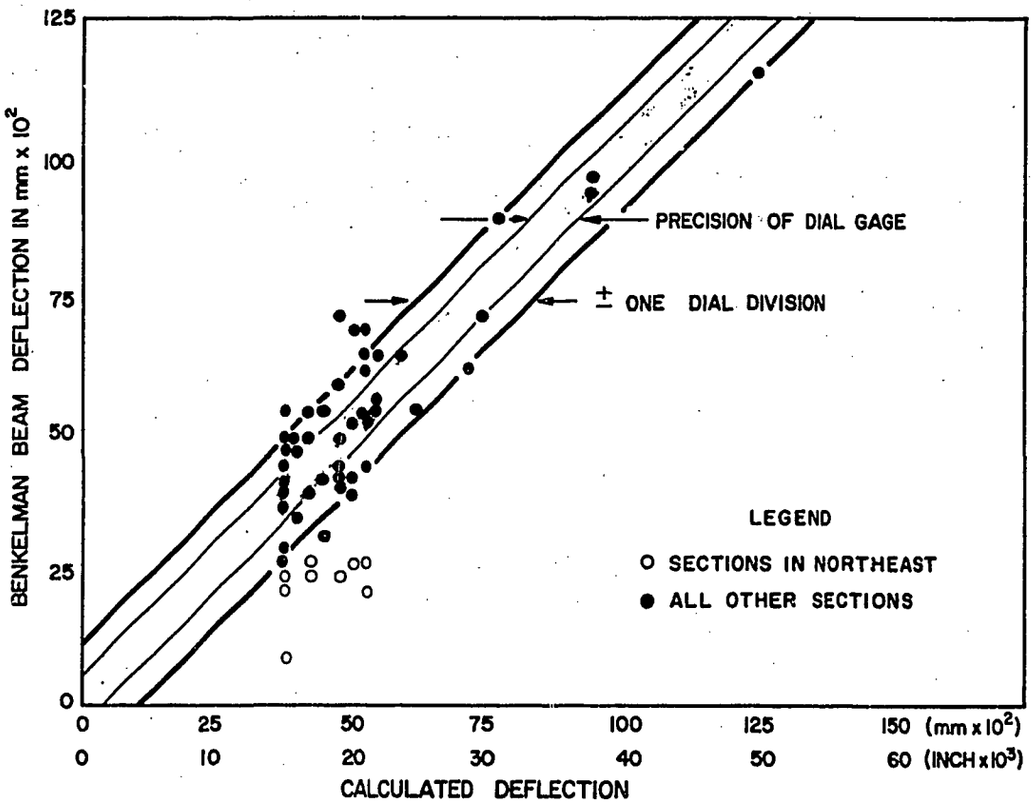
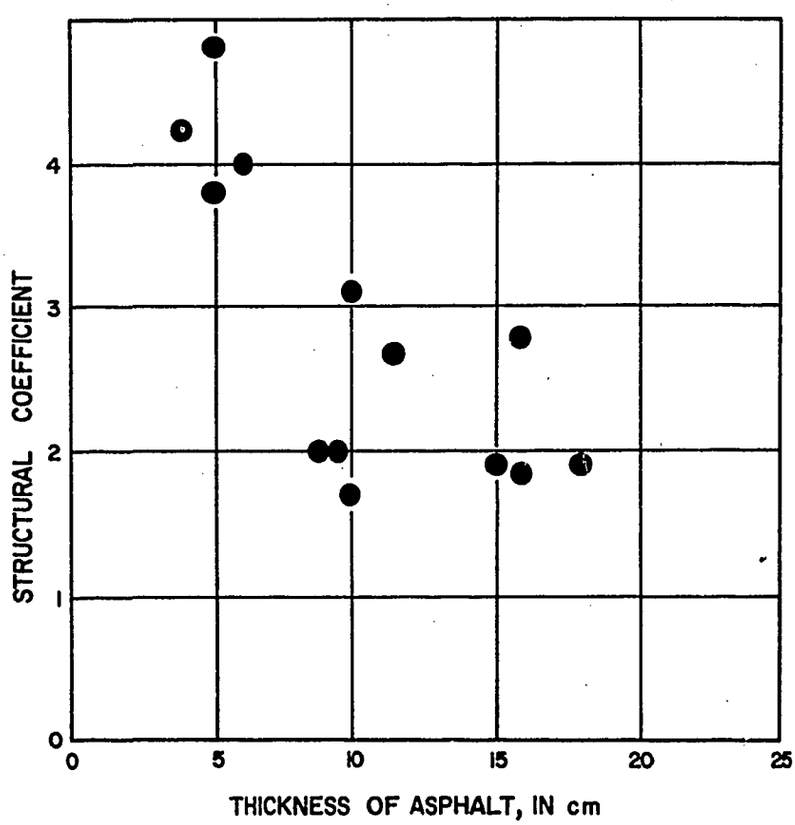


FIGURE 6.12 – COMPARISON BETWEEN MEASURED DEFLECTIONS AND CALCULATED DEFLECTIONS, SURFACE TREATMENT PAVEMENTS



6.13 – STRUCTURAL COEFFICIENTS VERSUS THICKNESS OF ASPHALT SURFACING

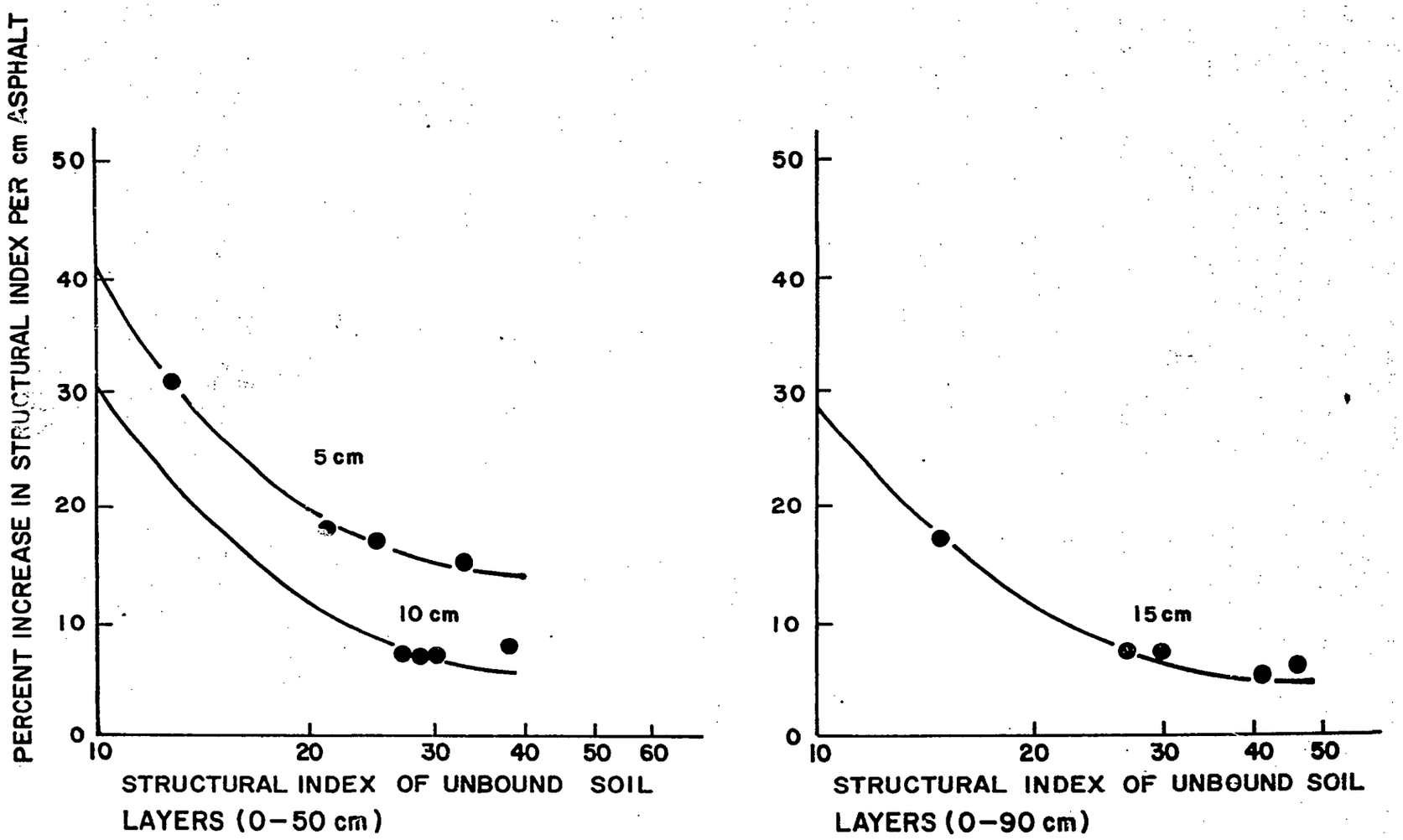


FIGURE 6.14 – EFFECT OF ASPHALT ON PAVEMENT STRENGTH (SI) FOR VARIOUS THICKNESSES OF ASPHALT AND UNBOUND SOIL LAYER STRENGTHS

(2 in) asphalt group. The curve for the 10 cm (4 in) asphalt group is extrapolated to parallel the 5 cm (2 in) asphalt curve. A good relationship also exists for the 15 cm (6 in) asphalt group, as for the 5 cm group, except the increases are based on a depth of 90 cm (36 in). Therefore, it follows that the residual strength is partially attributable to the asphalt layer and partially attributable to the increase in the load-spreading characteristics of the underlying soil layers.

The strength increase factors were used to calculate the deflection of 65 of the 87 test sections with asphalt surfacing. These are shown in Figure 6.15. Seven of the sections were eliminated due to the deterioration of the asphalt surface in the form of severe cracking; 11 sections were eliminated due to low CBR values; 2 sections were eliminated due to low density and 2 other sections were eliminated due to obvious discrepancies in the reported data. The shaded zone represents the precision of the dial gauges. The open circles represents sections from the northeast of Brazil which are discussed in the next section. The remainder of the data showed a good correlation between the measured deflection and the calculated deflection when considering the precision of the field measurements.

Natural Cementing-Slab Action

Twenty sections (open circles in Figures 6.12 and 6.15) exhibited deflections which were significantly lower than the deflection calculated by equation 6.7. The calculated structural indexes of these sections were high, with an average of 63 or, in accordance with Equation 6.6, an average calculated deflection of 0.4 mm (.016 in). The CBR's are not any higher or lower than encountered in other sections; therefore, adjusting the structural coefficients to satisfy these sections only resulted in an unsatisfactory shift for the results of the other sections. It was concluded that the CBR test does not measure the strength these layers exhibit in the field.

The majority of these sections are located in the northeast of Brazil. The area is known for its long dry seasons which allows natural hardening of certain types of tropical red soils. A natural cementing of any of the component soil layers would explain the abnormally low deflections.

Additional information requested from the Districts' personnel where these sections are located indicated that during excavation it was observed that one or more layers within these test sections were naturally cemented. The naturally cemented layers coincided with an older pavement structure or with a component layer of the newer pavement that had been unsurfaced for a period of time.

An analysis of petrification test results for these sections proved to be inconclusive. The cementing of the layers did not correlate with the petrification results.

There is a need for additional research on this phenomenon in the northeast of Brazil. The sections investigated are performing as if one or more of the layers is stabilized. The cementing of these layers is undoubtedly time-dependent. Deflection tests conducted on newly constructed unsurfaced roads over a period of several years will provide information with regard to the time involved in the natural cementing process. Equally important will be the determination of the uniformity of the cementing process.

Special laboratory studies could be devised to determine the increased strength of the layers as a function of time. The time element could be accelerated by processing the sample through a given number of wetting and drying cycles prior to testing. Through such a program, design criteria could be developed with the objective of using the natural cementing process as a means of reducing the overall required pavement thickness. However, if it is found that the cementing of the layers is not uniform, then the phenomenon, which seems to occur primarily in the northeast of Brazil, can only be incorporated as a safety factor in the design procedure.

Slope of the Deflection Basin and Structural Strength

The relationship between the slope of the deflection basin and pavement cracking is shown in Figure 6.8. A pavement design procedures should include recommendations concerning the minimum thickness of asphalt required to reduce pavement cracking to within tolerable limits within a given design period. These recommendations should be established from a relationship between the thickness of the asphalt layer and the slope of the deflection basin. However, an analysis of the Phase III data indicates that the slope of the deflection basin ($\tan \phi$) is not dependent on the thickness of the asphalt layer alone. Figure 6.16 is a plot of the reciprocal of the slope of deflection basin (slope index) versus the thickness of the asphalt concrete. A suitable relationship cannot be established without indicating the CBR of the base course material. When the base course CBR's are added, as shown in Figure 6.16, the data becomes discernible. A minimum slope index for a given thickness of asphalt for the various base course strengths (CBR) are shown in the figure. These are indicated by curves A, B and C. The full possible range in base course CBR values are not covered; however, a recommended range of base course CBR values is given in the figure. The curves will be used in Chapter 7 as the minimum asphalt thicknesses for various traffic periods depending upon the CBR of the base course material.

These recommendations establish minimum thicknesses to prevent excessive cracking of the asphaltic pavement. It is not a thickness requirement for structural considerations.

CONCLUSIONS

Maximum deflection values for the practical range of standard axle applications were established as a result of an analysis of the performance and deflection characteristics of flexible pavements in Africa and South America. Pavement evaluations were based on riding comfort which basically reflects the roughness of the pavement. Roughness, in turn, is generally attributed to pavement deformations particularly those occurring in the subgrade and reflected in the pavement surface. Therefore, the deflection criteria were established to limit the vertical strain in the subgrade to acceptable limits. The critical slopes of the

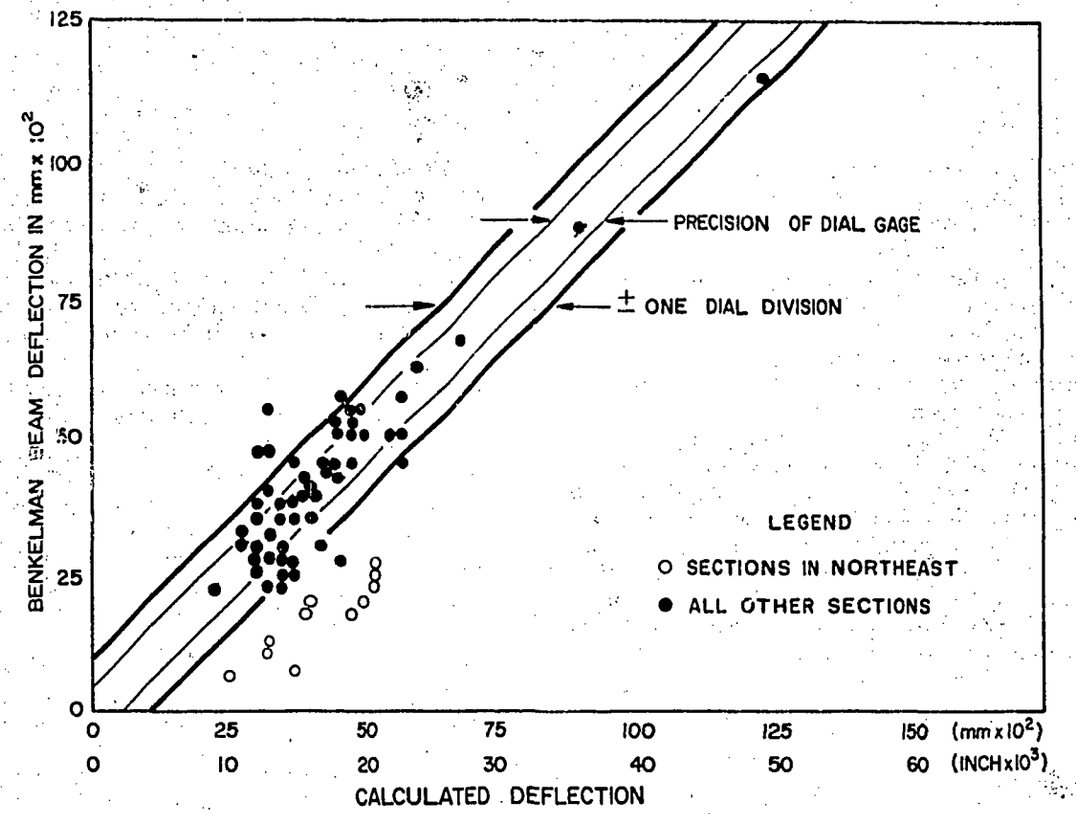


FIGURE 6.15 – COMPARISON BETWEEN MEASURED DEFLECTIONS AND CALCULATED DEFLECTIONS, ASPHALT CONCRETE PAVEMENTS

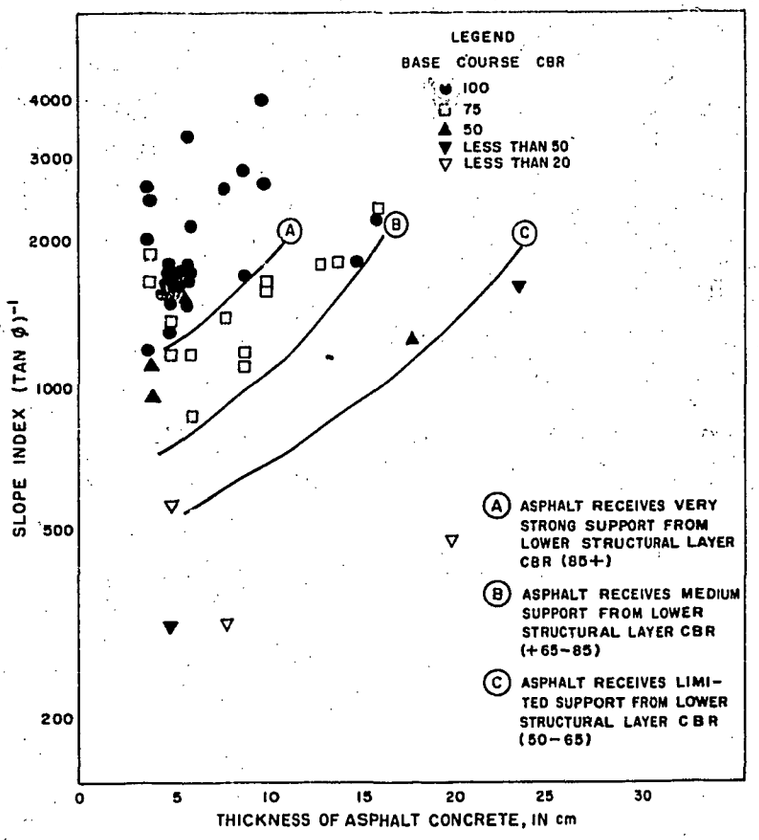


FIGURE 6.16 – RELATIONSHIP BETWEEN SLOPE INDEX AND THICKNESS OF ASPHALT CONCRETE FOR THE PRACTICAL RANGE OF BASE COURSE CBR VALUES

deflection basin for the practical ranges of standard axle applications that prevent excessive pavement cracking were also established from the analysis. Pavement cracking is caused by excessive tensile strain in the asphalt layer due to vehicle loading. Therefore, critical slopes of the deflection basins were established which limit the tensile strains in the asphalt layers to acceptable limits.

Structural coefficients were established based on the relationships between deflection and strength of the pavement component layers. The structural coefficients were based on the CBR of the unbound structural layers as well as the position of the layers in the pavement structure. A strength factor for asphalt layers was established based on the thickness of the asphalt layer and the strength of the soil layers beneath. A relationship between the slope of the deflection basin, asphalt thickness, and base course CBR was established. This relationship defines the minimum thickness of asphalt required to prevent excessive pavement cracking within a given traffic period.

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CHAPTER 7
DESIGN OF FLEXIBLE PAVEMENTS IN THE TROPICS

INTRODUCTION

The pavement behavior relationships established under actual service conditions during the pavement performance study are synthesized and restated in this chapter, since these relationships are basic to developing pavement design procedures for flexible pavements in the tropics.

Historically, the design procedures used in tropical countries have been adopted from those developed in temperate climates. Such procedures take into account the characteristics of the climatic conditions and materials in the location where the design is developed. For example, the AASHO Road Test, noted earlier, represents one of the largest experimental projects ever undertaken to develop empirical criteria for highways. Yet many State Highway Departments found that their local conditions made it necessary to undertake additional research prior to adopting the AASHO procedures. In some cases the undisturbed samples of materials used at the AASHO Road Test were sent to the local researchers in order to correlate the studies (Sowers, 1965). Therefore, the probability is only slight that direct correlations can be established in the tropics where local conditions are very different from those found at the site of the AASHO Road Test. Nonetheless some basic concepts developed during the AASHO Road Test are applicable in the development of any design procedure and these were adopted for this study.

The design procedure described in this chapter was developed from the analysis of pavement sections in a tropical environment. The procedure was principally derived from first, the establishment of the relationships between performance and deflections, and secondly, from the establishment of the relationships between deflection and the structural strength of each component layer within the pavement structure.

The design procedure developed from this study has been termed "Tropical Design Procedure For Flexible Pavements" because it was developed principally for red tropical soils which form the basic structural layers, i.e. base, sub-base and subgrade. However, the procedure was developed from an analytical approach, not an approach which correlates empirical data and uses curve fitting statistical methods. Therefore, the procedure has a wider application and can be used is most temperate climates where pavements are not subjected to frost penetration.

STRUCTURAL DESIGN CURVES

The Structural Design Curves is a basic element of the flexible pavement design procedure described below. It illustrates the relationship between the design traffic and the structural index (a measure of pavement strength). It was developed from two relationships established as a result of the pavement performance study. A description of the development of the curve, its use and limitations follows.

First, the relationships between design deflection and traffic were established for selected coefficients of variations during the pavement performance study. These relationships are illustrated in Figure 6.7. In addition, the relationship between deflection and structural index (SI) was established in this study.

In the latter case an equation has been developed whereby the deflection of a pavement section can be calculated (equation 6.7). The relationships between calculated deflections and measured deflections are illustrated in Figures 6.12 and 6.15. The data in both these figures (except for the open circles) are combined and shown in Figure 7.1.

7.1, undertaken to show the limitations of equation 6.7, indicates that the standard error of estimate is ± 0.081 mm (.003 in). The error is the result of both the limitations of the Benkelman Beam and the methodology used to derive the structural coefficient. Since that portion of the error attributable to the structural coefficient effects the development of the design curves, the design deflection data (Figure 6.7) were adjusted during the translation of deflection to structural index to account for the errors. The adjusted values provide a more conservative relationship for design purposes.

The structural design curves, Figure 7.2 were developed from the above data and illustrate the relationship between design traffic and structural index. In the figure the design traffic is given in terms of standard axle applications in both directions and can be determined in Brazilian designs from Figure 6.4 by multiplying the design daily equivalent axle loading by two and again by the appropriate design period in days. The coefficient of variation is a reflection of local construction practices and is determined by the analysis of the actual deflections of existing roads in the proposed design area. The structural index (SI) obtained from Figure 7.2 represents the minimum required strength for a standard 90 cm (36 in) pavement section and, once determined, it forms the basis for a flexible pavement design.

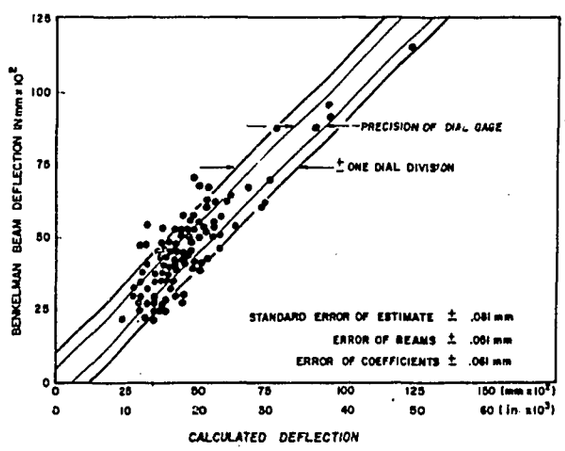


FIGURE 7.1 – COMPARISON BETWEEN MEASURED DEFLECTIONS AND CALCULATED DEFLECTIONS

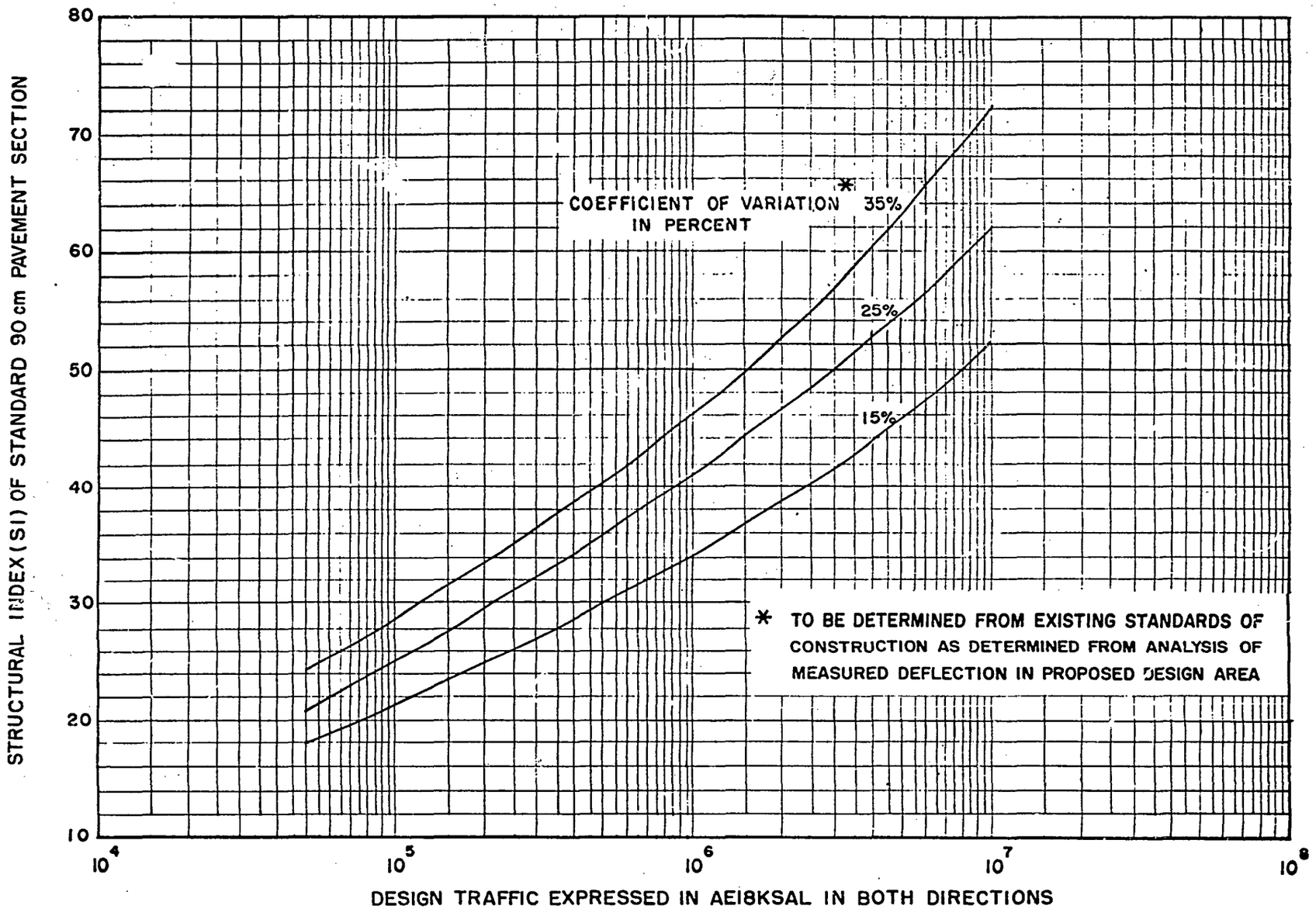


FIGURE 7.2 – STRUCTURAL DESIGN CURVES FOR DETERMINING THE REQUIRED STRUCTURAL INDEX OF THE STANDARD PAVEMENT SECTION

STRUCTURAL DESIGN PROCESS

The objective of the structural design process is to use the structural index to determine the thickness of each layer of a pavement section i.e., the surface, base course and sub-base course. All that is required to use the procedure are the design traffic, the CBR of the base, sub-base and subgrade, the coefficient of variation and the figures and tables provided in this chapter.

The structural design equations (6.5) were developed in the preceding chapter and are repeated below. The equations show that the structural index (SI) of a pavement section is determined by summing the structural indexes of each layer in a standard 90 cm (36 in) pavement section. The equations are:

1) Structural Index (SI) $= a_1 t_1 + a_2 t_2 \ldots a_n t_n$ and; (7.1)

2) $t_1 + t_2 + \ldots\ldots\ldots\ldots t_n = 90$ cm (36 in) (7.2)

In the first equation a_1, a_2, etc. are the structural coefficients of the materials used in succeeding layers of the pavement section. Structural coefficients for various unbound soils, developed in the preceding chapter are summarized in Table 7.1. In addition, Table 7.1 shows the structural coefficients for crushed stone, cement-treated soils and lime-treated soils. The coefficient for crushed stone is determined from an analysis of test sections with macadam base course. The coefficients reflect the load-spreading characteristics of the material. The table shows that the structural coefficient of crushed stone (open graded) is less than the higher quality concretionary gravels. The better load-spreading characteristics of the concretionary gravel (with CBR's greater than 80) are due to cohesion. The structural coefficients for cement and lime-treated soils are estimated from relationships given in the AASHO Interim Design Guide (1972).

In Equation 7.1, t_1, t_2, etc., represent the thickness of each succeeding pavement layer of a standard 90 cm (36 in) design section. Design thicknesses are linked to CBR values of the underlying pavement layer, except for the subgrade. The design CBR-thickness relationship first given in Figure 6.11 is replotted in Figure 7.3. The minimum pavement thickness requirements used by three highway departments have also been plotted on Figure 7.3.

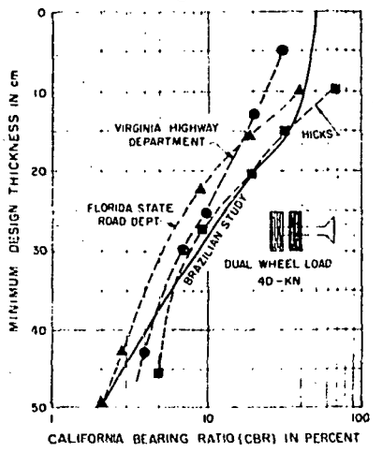


FIGURE 7.3 – MINIMUM DESIGN THICKNESSES FOR ALL PAVEMENT COURSES BASED ON CBR VALUES OF UNDERLYING LAYERS

TABLE 7.1
Pavement Coefficients for Flexible Pavement Design

Pavement Component	Strength Coefficient
Surface Course	
Asphalt Concrete-Sand Asphalt	
5 cm	Figure 7.5
10 cm	Figure 7.6
15, 20 and 25 cm	Figure 7.7 a, b, & c
Note: Bituminous treated macadam included in asphalt layer thickness	
Base Course	
Crushed Stone (Macadam hydraulic)	1.037 open graded
	1.394 graded
Cement Treated	
Compressive strength 7 days	
650 psi or more	2.400 *
400 psi to 650 psi	2.100 *
400 psi or less	1.600 *
Lime Treated	1.4 – 1.6 *
Concretionary Gravels	
CBR (Design)	
+100	1.394
90	1.232
85	1.167
80	1.102
75	1.037
70	0.940
60	0.552
50 (min) (1)	0.383
Subbase Course	
CBR (Design)	
+40	0.576
35	0.290
30	0.205
25 (min)	0.075
Subgrade Layer	
CBR (Design)	
+20	0.481
15	0.357
10	0.212
9	0.183
8	0.133
7	0.084
6	0.053
5	0.033
4	0.020
3	0.015
2 (min)	0.010

Design Coefficient Limits
Base Course refers to materials to a depth of 25 cm.
Subbase Course refers to material layers between 25-50 cm.
Subgrade Layer refers to material layer between 50-90 cm.
* Values estimated from structural coefficient relationships given in AASHO Interim Guide for Design of Pavement Structures, 1972
(1) Material with a CBR of 40 can be used between the depth intervals of 10 and 25 cm and assigned the same coefficient.

The strength of the subgrade, initially expressed as the subgrade CBR, is a key element in the design of the pavement structural layers, e.g., surface, base and sub-base. Its importance in the design is demonstrated in Figure 7.4. This figure which is partially replotted from Figure 7.3, shows that the thickness of cover (the pavements structural layers) varies inversely with subgrade CBR values. The figures also shows the relationship between subgrade CBR and structural index. Since the structural index of the subgrade influences the design thickness of the structural layers it is referred to simply as subgrade support. Therefore, higher strength subgrades furnish greater support and permit the use of thinner structural layers. The combined structural index for the structural layers (surface, base and sub-base) is equivalent to the required structural index (from Figure 7.2) less the subgrade support.

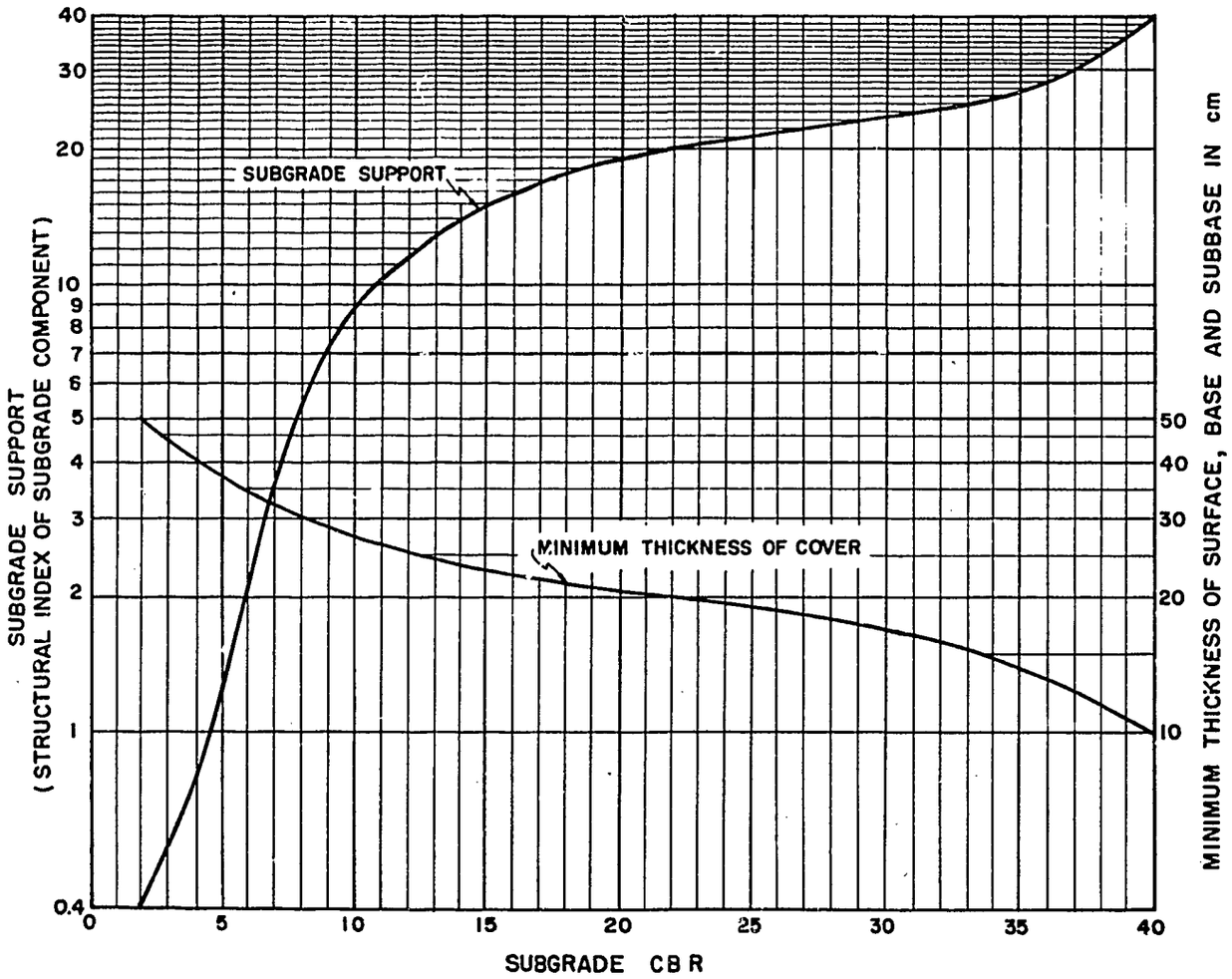


FIGURE 7.4 — SUBGRADE SUPPORT AND MINIMUM THICKNESS OF COVER BASED ON SUBGRADE CBR VALUES

FLEXIBLE PAVEMENT DESIGN

It may take more than one trial to design a pavement using the flexible pavement design process outlined herein. The first trial begins with the selection of a surface type and concludes with the determination of the base and sub-base thickness.

The surface type and thickness is determined from Table 7.2 and is based on the base course CBR values and the design traffic.

The minimum design thickness of the unbound structural layers (base and sub-base) in a DBST pavement design is equal to the minimum thickness of cover as derived from Figure 7.4. The thickness of the DBST pavement course is not considered since it does not contribute strength to the pavement section. However, an AC surface course provides a load-spreading characteristics so it is considered part of the minimum thickness of cover. Therefore, in an AC pavement design the minimum design thickness of the unbound structural layers equals the minimum thickness of cover less the thickness of AC.

The minimum base course thickness is determined from Figure 7.4 and is based on the CBR of the sub-base materials. The sub-base course is the remaining thickness of cover. Neither the base nor the subbase courses should be designed for less than 10 cm (4 in), since it is impractical to construct a thinner layer. Therefore, it may be necessary to over-design the base or sub-base course in order to account for this practical problem. Finally, the subgrade thickness for design purposes is the difference between the standard 90 cm (36 in) design section and the thickness of cover.

The adequacy of this first trial design section to carry the design traffic is determined by summing the structural indexes of each pavement layer (equation 7.1) and comparing it to the required structural index from the Structural Design Curves, Figure 7.2. The structural index of each pavement layer is equivalent to the product of its thickness and its structural coefficient (Table 7.1) to the extent that its thickness and/or CBR value fall within the strength and design coefficient limits provided in the table. Those portions of the pavement section that do not fall within these limits do not provide strength to the pavement and therefore are excluded from the derivation of the structural index of the entire pavement section. The thickness of those portions, however are included as part of the standard 90 cm (36 in) section.

DBST pavement design is the least complex flexible pavement design since the DBST surface does not impart any strength to the pavement. Design example 1 in the appendix to this chapter utilizes the principles, figures and tables described above to solve a simple pavement design problem.

The AC pavement design is more complex because the AC surface influences the design. As explained in the preceeding chapter, the AC surface provides for a better load-spreading characteristic in the underlying unbound

TABLE 7.2
Recommended Type and Thickness (cm) of Surface Course

Total Equivalent Standard Axle Load Applications Both Directions	Strength of Base Course – CBR							
	+100	90	85	80	75	70	60	50
100,000	ST	ST	ST	ST	ST	ST	ST	ST
200,000	ST	ST	ST	ST	ST	ST	ST	10
300,000	ST	ST	ST	ST	ST	ST	15	15
400,000	ST	ST	ST	ST	ST	10	15	15
500,000	ST	ST	ST	ST	10	10	15	15
600,000	ST	ST	ST	10	10	10	15	15
700,000	ST	ST	5	10	10	10	20	20
800,000	ST	5	5	10	10	10	20	20
900,000	ST	5	5	10	10	10	20	20
1,000,000	ST	5	5	10	10	10	20	20
2,000,000	5	5	5	10	10	10	20	20
3,000,000	5	5	5	15	15	15	20	20
4.000,000	5	5	5	15	15	15	20	20
5,000,000	10	10	10	15	15	15	20	20
6,000,000	10	10	10	15	15	15	20	20
7,000,000	10	10	10	15	15	15	20	20
8,000,000	10	10	10	15	15	15	20	20
9,000,000	10	10	10	15	15	15	20	20
10,000,000	10	10	10	20	20	20	25	25

Notes: ST denotes a double bituminous surface treatment, numbers are thicknesses of asphalt concrete surface required.
☐ To be used only if higher quality base material is not available and stablilization or modification proves to be too expensive.

soils layers. Consequently, the strength component of the underlying layers does not have to be as great as it would it it supported a DBST surface. The benefit of the AC load-spreading characteristic is illustrated in Figures 7.5, 7.6, 7.7a, 7.7b, and 7.7c for 5, 10, 15, 20 and 25 cm (2, 4, 6, 8 and 10 in) AC surfaces respectively. It should be noted that the load-spreading benefit extends only 50 cm (20 in) below the surface for the 5 and 10 cm (2 and 4 in) AC surfaces, whereas, it extends throughout the entire 90 cm (36 in) design section for the 15, 20 and 25 cm (6,8 and 10 in) AC surface. Design example 2 in the appendix to this chapter shows how these figures are utilized.

The design examples show pavement designs that meet the structural index requirement. However, if the calculated structural index had exceeded the required index this would have indicated that the section was overdesigned. In such an event it may be desirable to redesign the pavement for economic reasons, otherwise the design is considered satisfactory. Should a redesign be undertaken for an overdesigned pavement section it is possible that some savings may be realized by using materials with lower CBR values. Such a redesign must provide 1) that the minimum design thicknesses (Figure 7.3) are met; and 2) that the redesign does not result in a required change from DBST surface to AC surface or to an increase of the AC surface thickness, since such changes would result in a higher cost. It should also be noted that if the CBR of the lower strength materials fall below those shown in Table 7.1, then these materials can not be assigned a structural coefficient for the purpose of determining the structural index.

If the computed structural index in the design exam-

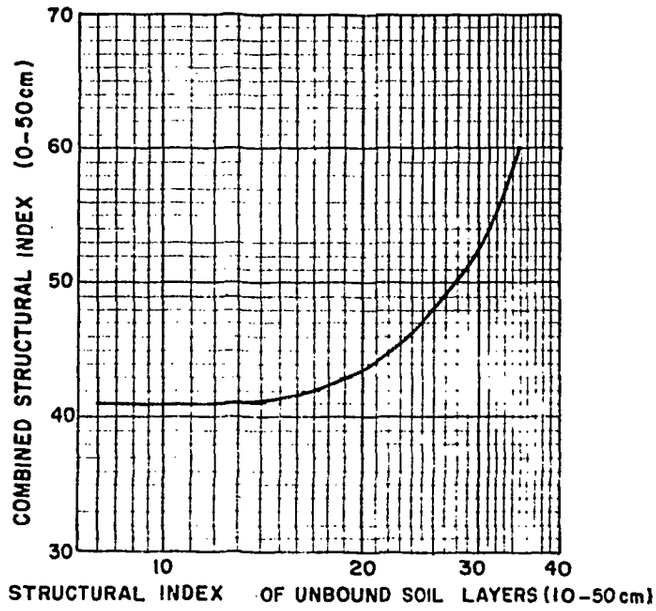


FIGURE 7.6 – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS, 10 cm ASPHALT CONCRETE

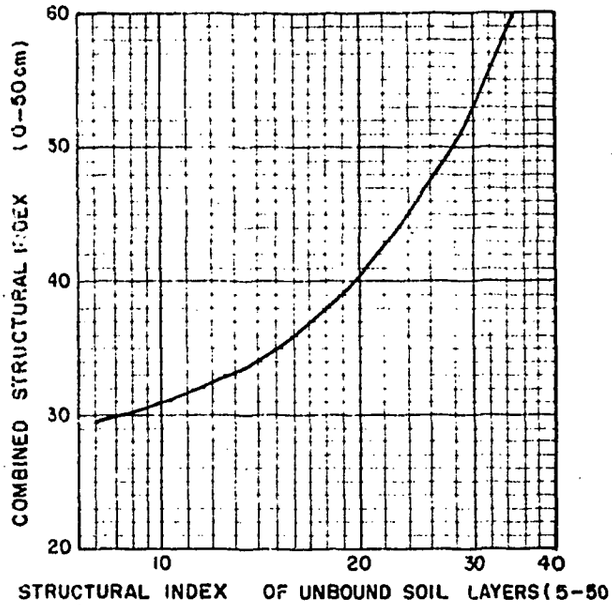


FIGURE 7.5 – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS, 5 cm ASPHALT CONCRETE

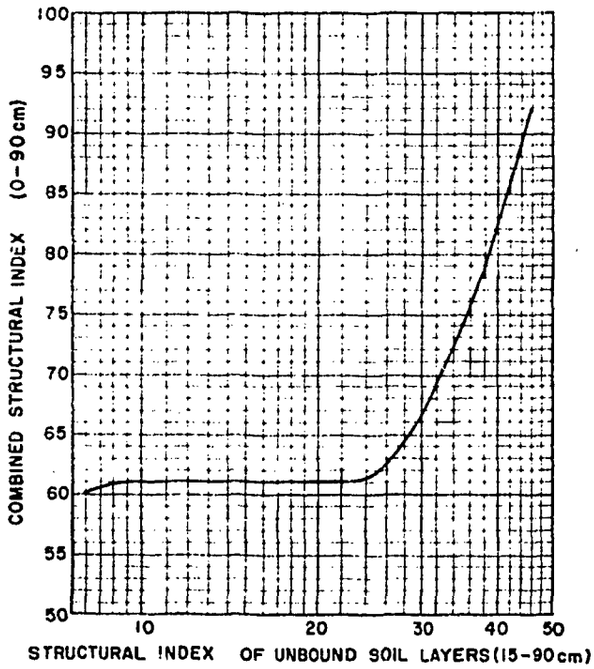


FIGURE 7.7a – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS, 15 cm ASPHALT CONCRETE

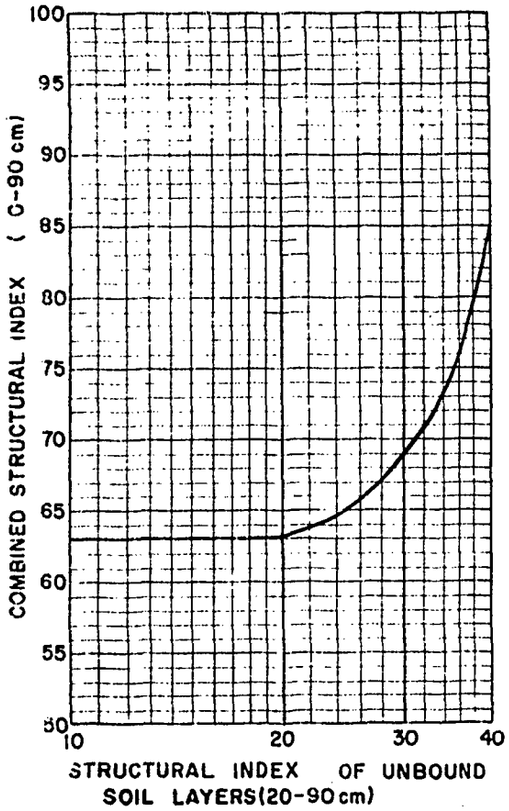


FIGURE 7.7b – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS, 20 cm ASPHALT CONCRETE

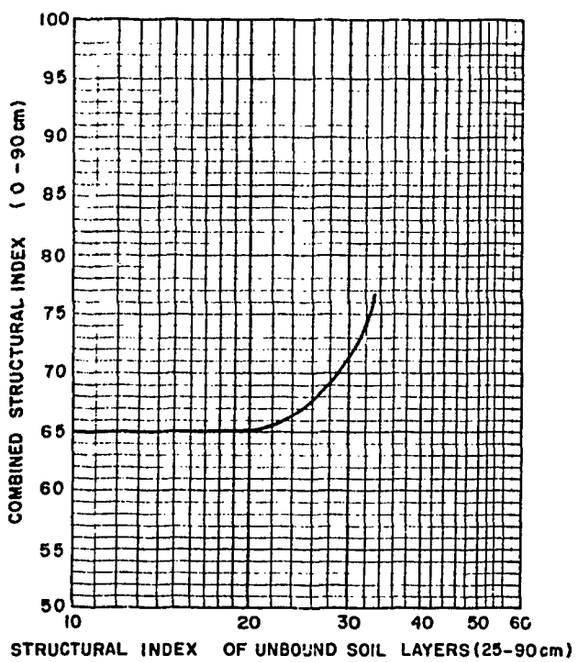


FIGURE 7.7c – COMBINED STRENGTH OF ASPHALT AND UNBOUND SOIL LAYERS, 25 cm ASPHALT CONCRETE

ples were lower than the required structural index (Figure 7.2) then five basic alternatives are available to the design engineer to improve the strength of the pavement design. They are:

- Increase the sub-base thickness. This increases the minimum thickness of cover but does not affect the surface and base courses.
- Increase the base thickness and reduce the sub-base by a like amount. This design retains the minimum thickness of cover.
- Use an AC surface instead of a DBST surface or increase the AC surface thickness and reduce the thickness of the base or sub-base course by a like amount. This design also retains the minimum thickness of cover.
- Stabilize the base course to increase the base course coefficient. This may retain all of the first trial thicknesses unless the AC thickness is adjusted to account for the increase in strength of the base course.
- Any combination of the above.

The engineering decision to undertake one of the above alternatives will be based on economic considerations, local conditions and the availability of the materials required to increase the structural index.

DESIGN TABLES

Design tables have been developed using procedures described above. The appendix to this chapter contains 16 Structural Design Tables which provide for the easy design of pavement sections for various combinations of surface types and subgrade, sub-base and base course CBRs. Four standard thicknesses of base course and three standard thicknesses of asphalt are provided. The thickness of the sub-base is fixed by the required minimum thickness of pavement (Figure 7.4). For a given thickness of asphalt a corresponding thickness of base course is shown where the combined thickness of asphalt and base course is equal to 25 cm (10 in). Since the structural coefficients change at a depth of 25 cm (10 in), an additional thickness of base course material would be uneconomical in design. A high quality sub-base material would do as well.

Numerous design combinations are available in the tables. The tables are based on minimum thickness of cover; therefore, the strength of many sections can be increased by an addition of an incremental thickness of sub-base. An exception is the 15 cm (6 in) asphalt surface tables. In all the tables the pavement combinations represent a reduction in deflection through a reduction in stress. However in the 15 cm (6 in) tables this apparently reaches a maximum value as additional increase in strength for the sections in the lower half of these tables can only be obtained by increasing the shearing resistance of the component layers, i.e., by moving upward in the table (stabilization of the subgrade) and then to the right (stabilization of the base course). Examples of using the Structural Design Tables are given in the appendix to this chapter. A comparison of various design procedures is also given in the appendix to this chapter.

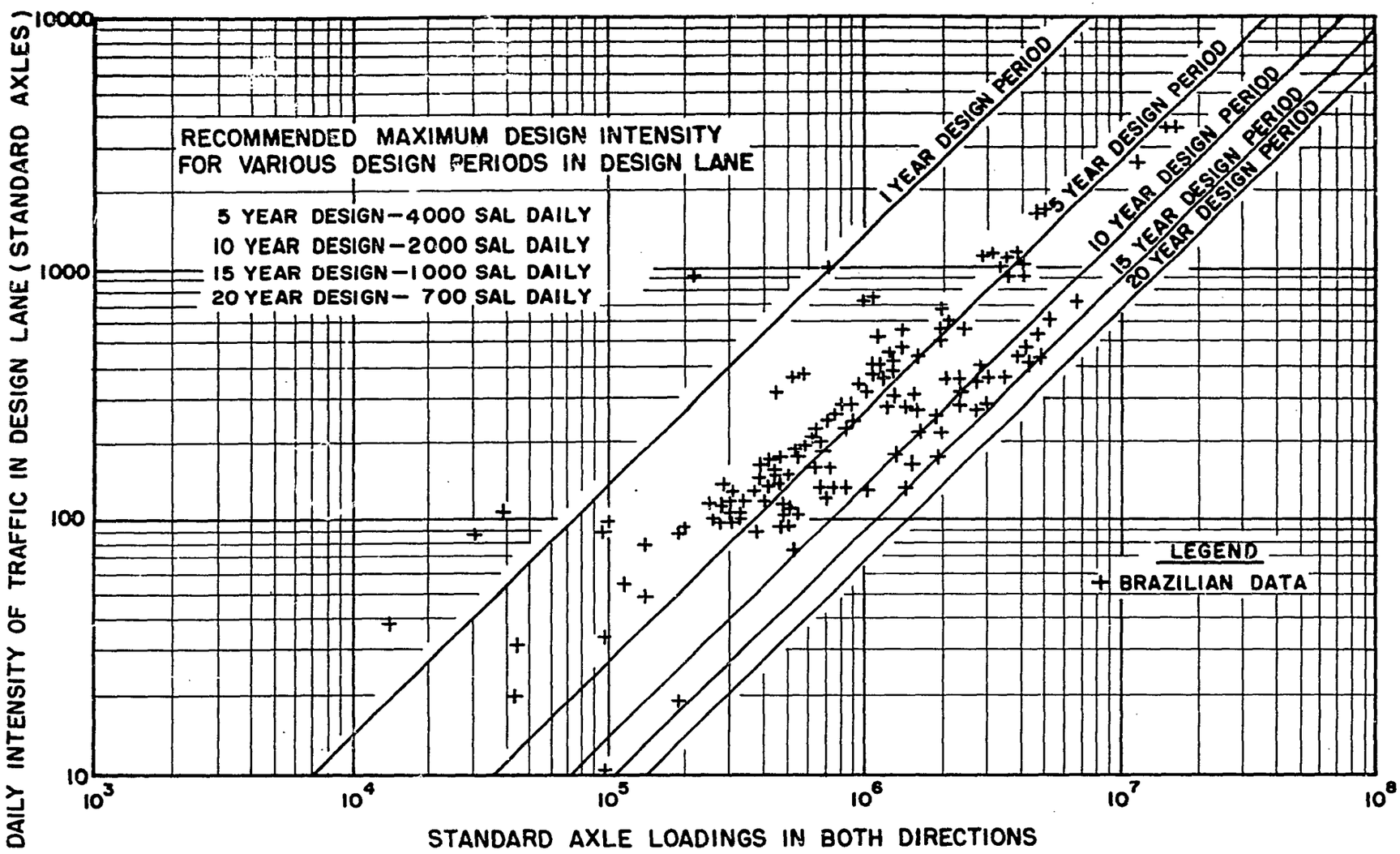


FIGURE 7.8 — DAILY INTENSITY OF STANDARD AXLE LOADING vs TOTAL STANDARD AXLE LOADINGS

TABLE 7.3
Comparison of Overlay Procedures

Method	Deflection Before Overlay			
	0.50 mm	0.75 mm	1.00 mm	1.25 mm
	Deflection After 5 cm Overlay			
Tropical	0.48	0.58	0.65	0.68
Lister	0.38	0.53	0.70	0.85
Average	0.43	0.56	0.66	0.76

DESIGN LIMITATIONS

Experience has shown that pavement performance is not governed by total axle loads alone but also by the number of loads applied within a given time period. Several maximum intensities of standard axle loads are recommended. These are provided in Figure 7.8. If higher maximum intensities are required in design, consideration should be given to adding a second traffic lane.

PAVEMENT OVERLAY CONCEPTS

A field evaluation of the effects of pavement overlays on deflections was not possible during the course of this study, but the reductions due to additional thicknesses of asphalt were analyzed using the structural coefficients and asphalt strength factors established in the study. The deflections of several hypothetical sections were analyzed before and after a specified thickness of asphalt was added to the pavement structure. The percent reduction in deflection for a given overlay thickness is dependent upon whether the pavement receiving the overlay has a surface treatment or an asphalt concrete surface. The results of this analysis are plotted in Figures 7.9 and 7.10.

The curves in Figures 7.9 and 7.10 are recommended for use in designing pavement overlays. A comparison was made between these results and another overlay design procedure which was developed from field testing (Lister, 1972). The comparison is shown in Table 7.3. The average difference between the two methods is 0.090 mm (.0035 in). The two methods were developed in different environments which may account for some of the differences but it would be doubtful if one could measure the difference between the two methods in large scale field studies.

In any major overlay program where the combined thickness of asphalt after overlay is equal to or greater than 15 cm (6 in), it is advisable to evaluate the strength of the existing pavement prior to the final decision of overlay thickness. Occasionally, with such thicknesses, the addition of an overlay will cause an increase in the deflection. The possibility of such an occurrence should be considered. This phenomenon is generally restricted to high strength pavement sections. Overlays are usually applied in such cases to improve the riding quality or for maintenance purposes.

A typical example is given in Figure 7.11 where the initial unpaved (no asphalt) section has a calculated deflection of 0.508 mm (.020 in). An additional five cm (2 in) of asphalt decreases the deflection to 0.382 mm (.015 in). However, with increasing thicknesses of asphalt the deflection increases. This is the result of the elasticity of the asphalt. Assuming the increase in deflection is attributed to the increased thickness of asphalt, and it would be difficult to attribute it elsewhere, then the computed modulus of elasticity would range from 1.31×20^4 to 1.87×10^4 kg/cm^2 (212,121 to 280,000 psi) or an average of approximately 1.67×10^4 kg/cm^2 (250,000 psi). This is a figure commonly used as E for asphalt concrete. The increase in deflection due to the asphalt layer does not indicate an increase in strain in the lower layers. The additional deflection due to 15 cm (6 in) of asphalt ($E = 1.67 \times 10^4$ kg/cm^2) amount to .050 mm (.002 in). A correction has been applied to the curves in Figures 7.7a, b and c to compensate for the deflection attributed solely to the asphalt. No correction has been applied to Figure 7.5 and 7.6 since the additional deflection due to the asphalt is insignificant.

ASPHALT EQUIVALENTS

The Asphalt Institute (1970) recommends the use of a "Substitution Ratio" of 2 for high-quality bases where CBR is 100 or more and 2.7 for low-quality bases where CBR is more than 20. Such equivalency factors or "Substitution Ratios" could not be established directly from field measurement because a proper test pavement sections did not exist. Figure 7.12 illustrates the requirements for such an analysis, which are: uniform subgrade; uniform thickness of asphalt concrete; and a range of base course thicknesses where the combination of CBR and thickness would result in equal deflections for all sections. A control section composed entirely of asphalt concrete would also be necessary. Obviously this type of investigation could only be accomplished by designed test sections.

Asphalt equivalents were developed, however, using the coefficients established in this study for the hypothetical sections shown in Figure 7.12. These provide asphalt equivalents that are in the same range as those recommended by the Asphalt Institute.

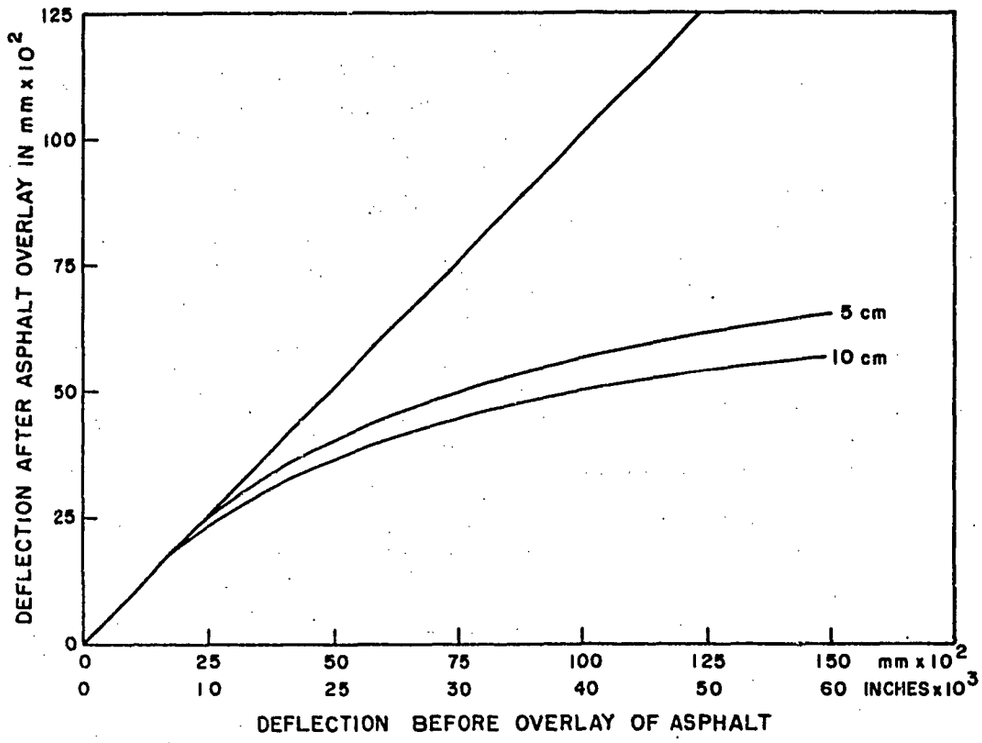


FIGURE 7.9 · REDUCTION IN DEFLECTION ACHIEVED BY OVERLAYS OF DIFFERENT THICKNESSES PLACED ON AN EXISTING SURFACE TREATMENT

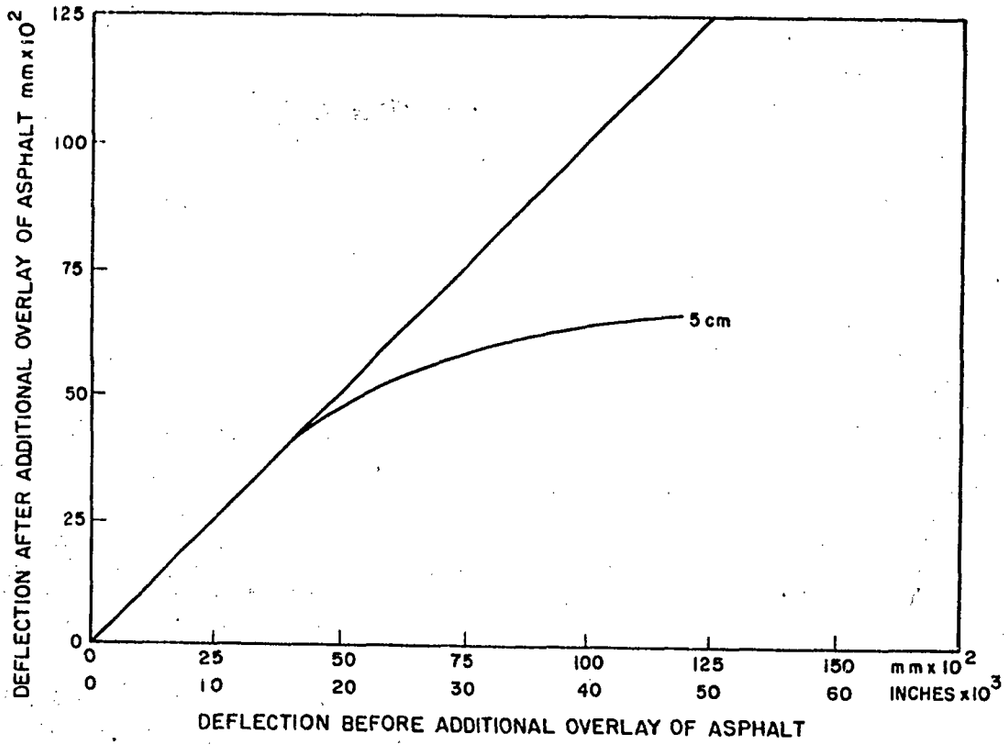


FIGURE 7.10 – REDUCTION IN DEFLECTION ACHIEVED BY OVERLAY PLACED ON AN EXISTING ASPHALT CONCRETE SURFACE

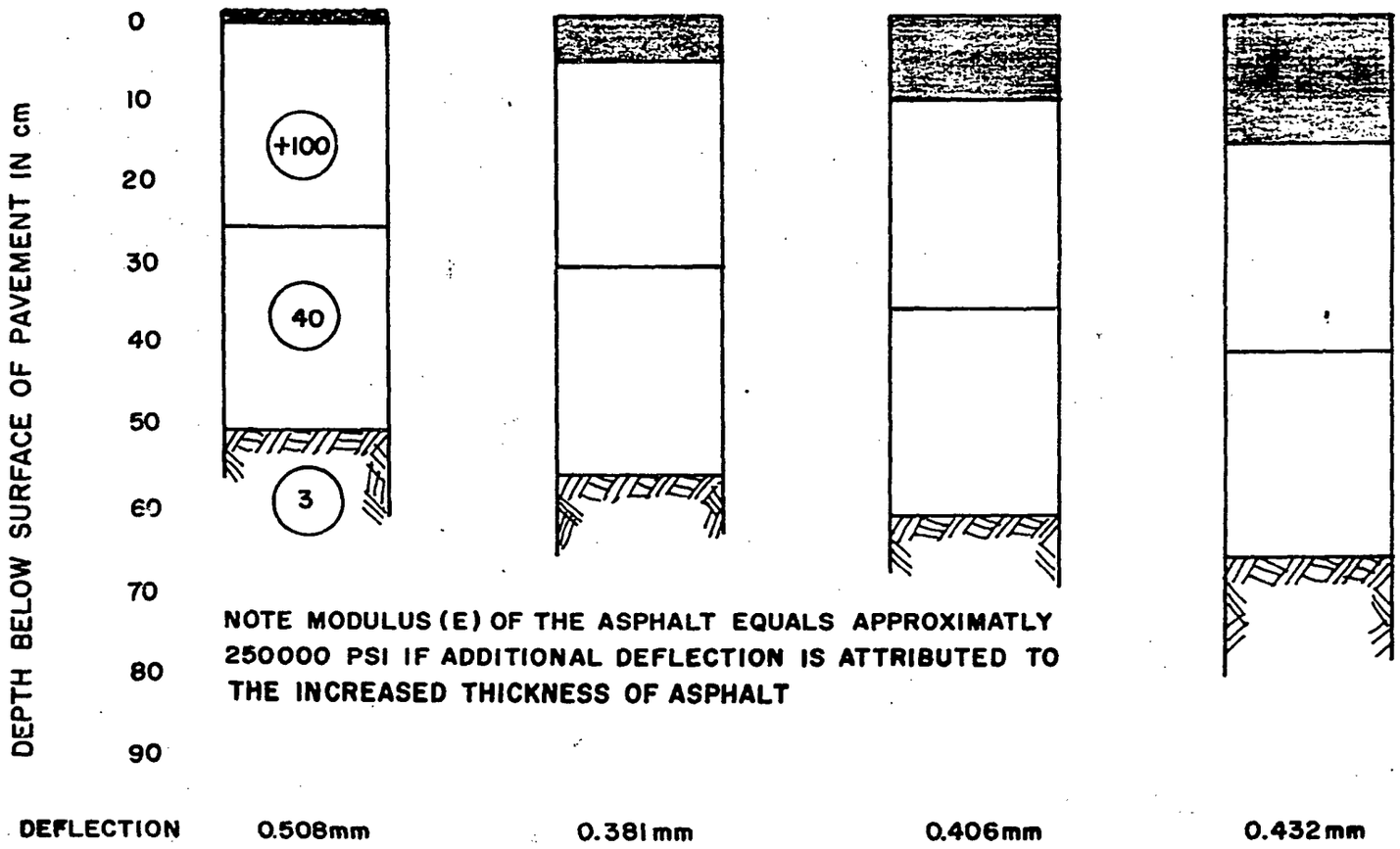
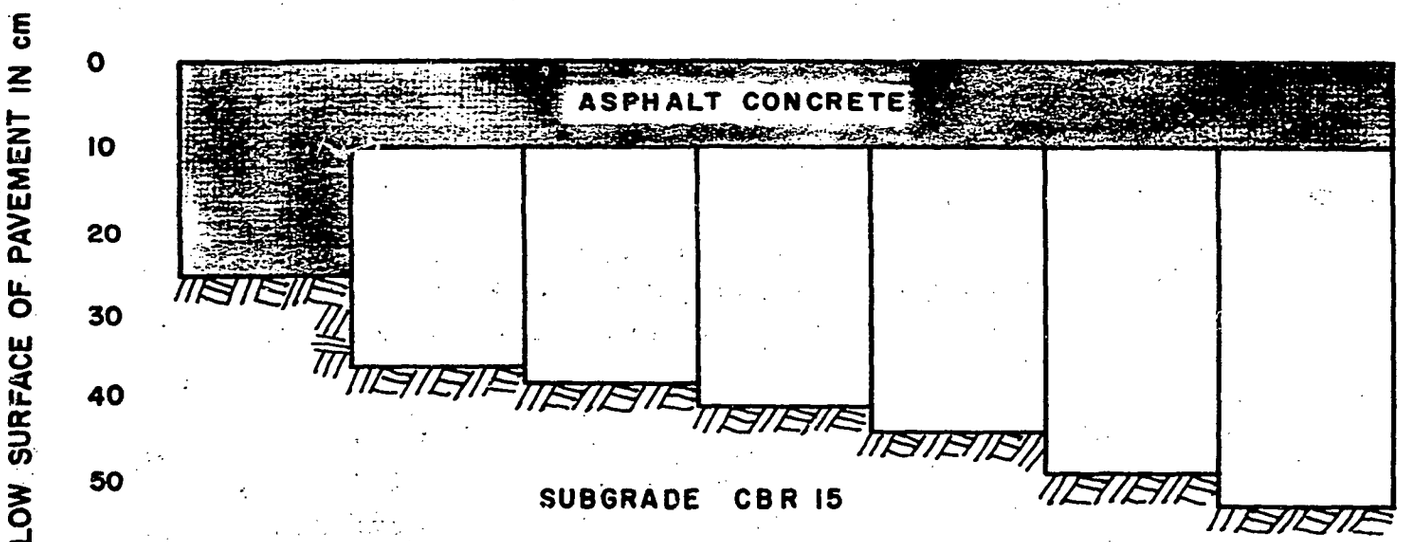


FIGURE 7.11 — EFFECT OF INCREASING THICKNESS OF ASPHALT ON HIGH STRENGTH PAVEMENTS



BASE CBR	+100	100	90	80	70	65
SUBSTITUTION RATIO	1.7	1.8	2.0	2.2	2.5	2.7

FIGURE 7.12 – SUBSTITUTION RATIOS FOR VARIOUS BASE COURSE MATERIALS

TABLE 7.4
Equivalent Pavement Designs

Design	Surface Thickness cm	Structural Index of Base
1	25	0
2	15	22
3	10	28

Subgrade CBR 15; Design traffic = 3.75×10^6 SAL.

Asphalt equivalents are not constant for all design schemes. The Asphalt Institute states:

"Analytical studies of extensive data available from road tests, laboratory experiments, and theoretical analyses show that there is no simple, constant factor for converting a given thickness of asphalt layer into a thickness of untreated granular base that will provide equivalent load-spreading capacity. This conversion is a variable that depends principally on the amount of traffic, the magnitude of the wheel loads, and the strength properties of the untreated granular base and subgrade".

However, the use of material equivalents may be appropriate in the planning and design stages. For example, equivalent pavement designs are shown in Table 7.4 with three different thicknesses of asphalt surfacing for a given design period and subgrade CBR. The equivalents are in terms of the structural index which can be readily computed for various traffic and subgrade conditions. A cost analysis can be made to determine the most economical section. The thickness of the base and sub-base would depend upon the availability of materials, whereas the minimum thickness of asphalt is governed by the base course CBR and traffic (Table 6.1).

DESIGN OF UNPAVED ROADS

The deterioration of unpaved roads results from the loss of surface material; formation of corrugations and potholes; softening of the surface during rain; and the loss of fines as dust during the dry season. It would be impossible to design an unpaved road that required no maintenance. A correlation of the performance of unpaved roads to the maintenance required rather than to a thickness-CBR relationship would be more appropriate.

Maintenance can be held to a minimum by the proper selection of surface materials and by controlling the deflection by selecting an adequate thickness of material based upon the strength of the subgrade. One observation that was made in the study of unpaved roads in Africa is that very little attention has been given to the thickness of the section above the subgrade. In many cases thickness requirements are determined from maintenance records of the loss of surface material during a given time period. The thickness then simply determines the interval between resurfacing. Hveem (1955) observed that the difficulty in maintaining untreated gravel roads was related to the subgrade soil and the thickness of select material above the subgrade. This indicates that a structural thickness requirement is appropriate in designing unpaved roads. The minimum thicknesses for various subgrade CBR values shown in Figure 7.4 are recommended for design of unpaved roads. It is recommended that the CBR of the select material be 40 or above.

Material desirable for the surface course of an unpaved road will not always be suitable as a base course under a paved road. Experience in Africa indicates that material with far higher Atterberg limits than normally acceptable for base courses under paved roads perform better as surface courses of unpaved roads.

If stage construction is desirable, the surface material should comply with specifications for the base course of a paved road. Two alternatives are available: one is to use the more suitable surface material and cover it with an adequate thickness of base material prior to adding an asphalt concrete surface; and the other is to stabilize the surface material with an additive prior to applying the asphalt concrete surfacing. In Nigeria, for example, it has been the practice to remove the top six inches and replace it with a more suitable material or stabilize the top six inches with lime prior to placing an asphalt concrete surface.

Experience in other countries has shown that 10-15 cm (4-6 in) of untreated surface material will last four to five years for traffic in the range of 100 vehicles per day. The loss of material can be taken into consideration in the design where the loss would be replaced with suitable base material prior to paving. In this case the design grade of the unpaved road would be the design grade of the base course in the second stage. Therefore, drainage structures such as side ditches can be initially incorporated into the first stage without major modification during the second phase.

DRAINAGE CONSIDERATIONS

General

Considerable attention has been given to the removal, or more precisely, the interception of water prior to its entry into the pavement sections. Trenches with perforated pipe are used to intercept lateral seepage and to lower the water table. Side ditches carry surface run-off away from the pavement section. Other methods have been designed to remove infiltrated water from pavement sections.

Recent studies have shown that previous estimates of the amount of water that infiltrates into the pavement were low. This led to the erroneous conclusion that drainage layers composed of coarse sands were adequate for the removal of the water. Such studies conducted at the University of Maryland show that a surprisingly large percentage of rain water enters the underlying soil layers through the pavement surface (Cedergren and Godfrey, 1974). Figure 7.13 shows typical results from this study. Therefore, pavement cracks that might otherwise be acceptable in a pavement surface can admit water into the pavement structure and reduce the strength of the section.

Reduction in pavement strength is due to build up of pore water pressures under traffic loads which reduce internal friction and lower shear resistance. This results in a loss of the load-spreading capacity of the unbound structural layers, increasing the applied pressure transmitted to the subgrade. There is a further loss of strength when the water enters the subgrade and reduces its shear resistance.

Design Considerations

This section is taken largely from Cedegren, OBrien and Arman (1972), "Guidelines for the Design of Subsurface Drainage Systems for Highway Structural Section," a research program conducted for the Federal Highway Administration.

The permeabilities of asphalt surfaces vary considerably. A recommended practice is to assume the infiltration rate (mm or inches/hour) to be between 33 and 67 percent of the design precipitation rate (mm or inches/hour) for asphalt concrete pavements. The percentage of infiltration would depend upon the intensity of pavement cracking as well as the width of the cracks. The criterion presented in this study limits the amount of pavement cracking to a minimum during the design period (Table 7.2).

It probably would be more economical to apply a seal coat during the later years of the design period than to design the subdrainage system on the basis of the above percentages for asphalt concrete pavements. The design of the subdrainage system could then be based on permeability tests conducted on samples of standard asphalt mixes. Infiltration rates so established, based on appropriate precipitation rates, would be reasonable. Such information could be obtained in laboratory studies of pavement sections cut from existing pavements.

The design precipitation rate which can be determined from the rainfall records for the given area, should only be exceeded during a small portion of the year. The average precipitation rate plus two standard deviations would provide a reasonable design value. In areas with two rainy seasons, the season with the highest rainfall should be applied.

The drainage layer should be designed using Darcy's law.

$$Q = k i A$$

In this equation k is the permeability, i is the hydraulic gradient and A is the thickness of the base. The drainage capacity of the layer (kA) must be equal to the design infiltration rate (Q) divided by the hydraulic gradient of the drainage layer (i). Design charts are available in several references: Cedergren, O'Brien, and Arman (1972) and the Asphalt Institute Manual Series No. 15 (1966) among others. These can be used for determining thickness and permeability combinations which would provide the required drainage capacity. Design systems include drainage layers, filter course, ditches and pipes (Figure 7.14).

Drain Course

The drain course consists of an open graded base immediately below the pavement surface. The aggregate should be hard, durable and stabilized when necessary with an asphalt binder (2.4%). The desirable thickness of these layers are 8 cm (3 in); however, 4 cm to 5 cm (1.5 to 2 in) layers can be used, with special care that the layers do not become contaminated. The discharge capabilities of the drainage layer must equal the total expected inflows estimated from surface infiltration. The required permeability can be determined using Darcy's law. Cedergren, O'Brien and Arman (1972) recommended that the permeability of the drainage layer as tested in the laboratory should not be less than 3000 m/day (10,000 ft/day).

Filter Protection

The open graded bases must be protected against clogging. The filter material should have the following characteristics:

To prevent clogging of the filter material:

$$\frac{D_{15} \text{ of filter material}}{D_{85} \text{ of protected soil}} \leq 5$$

and

$$\frac{D_{50} \text{ of filter material}}{D_{50} \text{ of protected soil}} \leq 25$$

To assure adequate capacity:

$$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of protected soil}} \geq 5$$

Collector Drains

Collector drain pipes are installed as shown in Figure 7.14. If ground water is to be removed the rectangular trench (right section) should be of sufficient depth to maintain ground water at least 1 m (3 ft) below the bottom of the structural section. Perforated pipes 100-200 mm (4-8 in) in diameter have proved most satisfactory. Filter material within the trench should be properly compacted.

Pipe Outlet

Lateral [illegible] every 30-45 m (100-150 ft) to provide [illegible] drainage layer and collector pipes. In areas of [illegible] the pipe outlets should be spaced at closer intervals to avoid a loss of stability at the shoulder area due to increased hydrostatic pressure. The open end of the outlet pipe should be 30 cm (12 in) above the flow line of the side ditch. The splash area should be protected against erosion. The trench for the outlet pipe should be

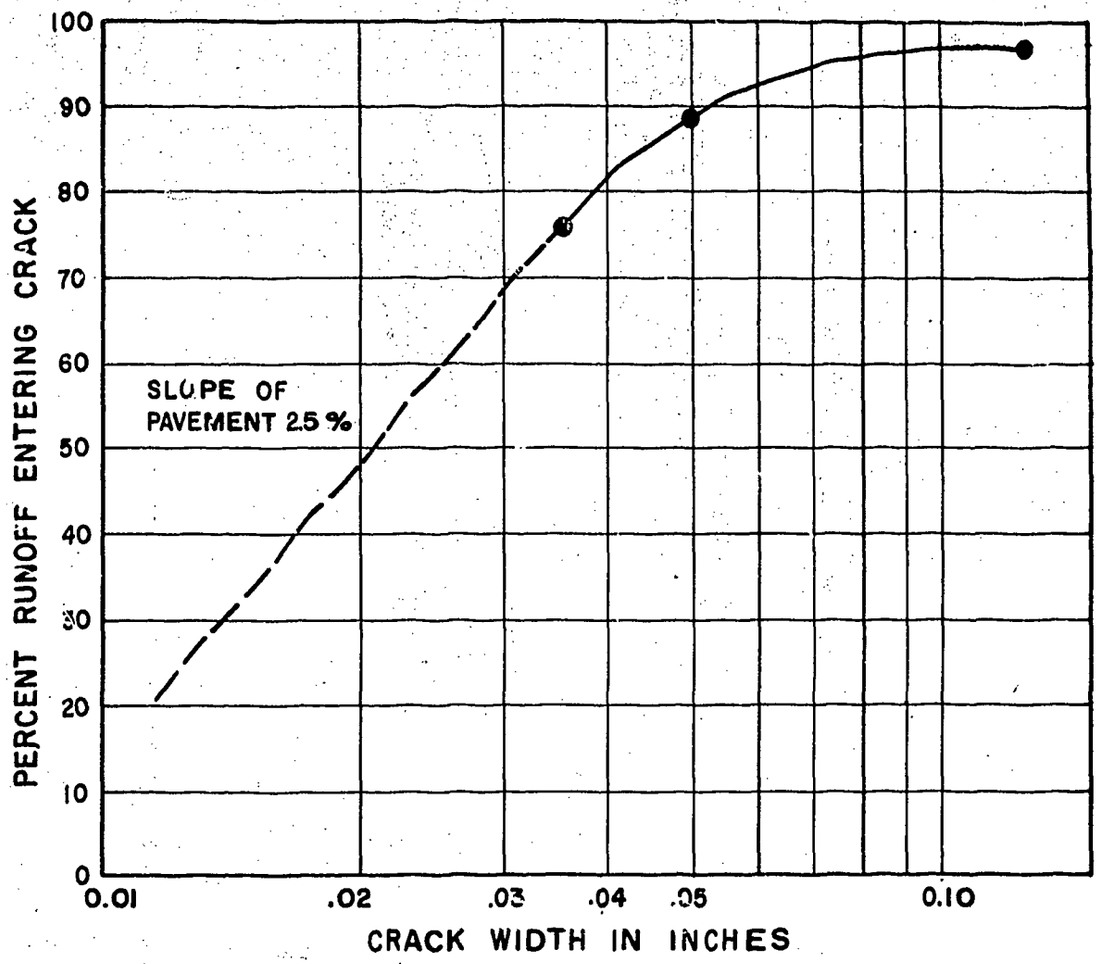


FIGURE 7.13 – PERCENT RUNOFF ENTERING PAVEMENT CRACKS OF VARIOUS WIDTHS

backfilled with a low permeability material or an impervious diaphragm should be used to prevent piping. Typical cross sections are shown in Figure 7.14.

CONCLUSIONS

A flexible pavement design procedure described herein was developed during the Laterite Study Program. The design procedure was developed from two basic relationships: (1) between deflection and performance; and (2) between deflection and pavement strength.

Structural design curves are shown for design traffic and various degrees of construction uniformities. The deflection design curves presented in Chapter 6 are converted to structural design curves by the relationship established between deflection and pavement strength.

A design equation is utilized in the basic design procedure to compute the structural index of trial sections. Structural coefficients are selected for unbound soil layers and are a function of the CBR value of the layer and position within the pavement structure. Asphalt concrete surfacing increases the strength of the pavement sections when it is used in the design. Asphalt strength curves are given for five standard asphalt concrete thicknesses. They are 5, 10, 15, 20 and 25 cm (2, 4, 6, 8 and 10 inches). Design asphalt thicknesses are given for various ranges of traffic and base course CBR values.

Design Tables are presented which simplify the design procedure. Sixteen Design Tables have been developed which provide various combinations of pavement thicknesses, material strengths and pavement surfaces. Three thicknesses of asphalt concrete surfacing as well as surface treatment are included in the Design Tables. Four base course thicknesses in combination with six base course strengths are included. The sub-base thickness varies depending upon the CBR of the subgrade. Four levels of sub-base strength are provided in the tables.

An overlay design procedure is presented which has been developed from calculated values using the asphalt strength increase factors established in the study. The procedure agrees favorably with results from full-scale studies (Lister, 1972).

Recommendations are also made for the design of unpaved roads. For unpaved roads the minimum thickness of surfacing and base is established based on the CBR of the subgrade.

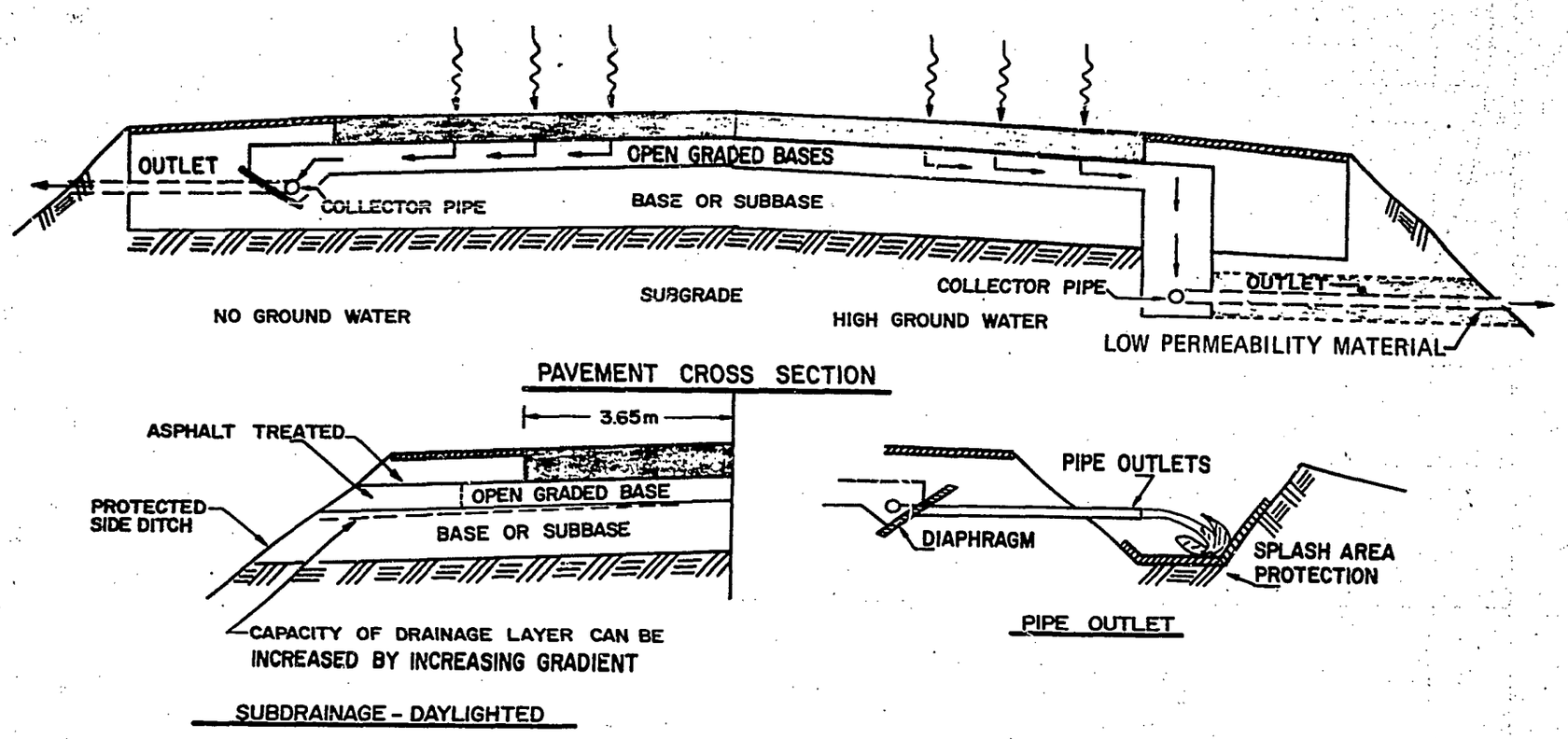


FIGURE 7.14 – TYPICAL DRAINAGE CROSS-SECTIONS (after Cedergren, O'Brien and Armon, 1972)

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CHAPTER 8

DURABILITY AND REPETITIVE LOAD TESTS FOR IRONSTONE AND CONCRETIONARY GRAVEL

INTRODUCTION

The quality of ironstones and concretionary gravels should be evaluated before these materials are utilized for any of the upper structural courses in a highway particularly surface courses. Several tests are presently available for assessing aggregate durability. The Los Angeles abrasion (LAA) test (AASHO T 96-70) which is commonly applied is considered to be too severe.

The Portuguese LNEC (Laboratório Nacional de Engenharia Civil) has modified the LAA test when investigating concretionary gravels by performing it without the normal charge of steel spheres rotating within the drum (Novais Ferreira and Meireles, 1965). Other durability tests include the Deval abrasion, sodium sulfate soundness, freezing and thawing and others. The equipment for the Deval abrasion is not commonly available and the sodium sulfate soundness is a slow, cumbersome test of doubtful applicability in humid tropical environments. Certainly a freezing and thawing test is not applicable.

Ironstones and concretionary gravels vary in quality just as natural gravels and each potential source must be appraised just as natural gravels must be appraised. The fact that no truly applicable test has been developed for lateritic materials does not preclude this requirement. One of the most important factors in assessing the quality of lateritic materials is determining the durability.

Durability is defined here as a resistance of the material to degradation under vehicular loads. It usually is a function of the strength, load intensity and load duration. It is impossible to devise a test that examines all of these factors in a realistic way, particularly since the loading history cannot always be controlled.

Rigid control of vehicular loads are particularly important if durability evaluations are to be meaningful. For example, an ironstone may serve satisfactorily for the design life of the pavement if axle load limitations are enforced but the same aggregate may fail after only a few excessive load applications. This has been the experience the world over. Ore trucks or logging trucks carrying overloads of 50 percent or more have damaged roads within one year in Southeast Asia and Africa.

Drainage is also important but water does not significantly alter the durability of lateritic materials unless they are subject to softening or breakdown in water. Such materials should not be used. The excess hydrostatic pressure generated by dynamic wheel loads reduces the bearing capacity of the structural courses but seldom causes significant degradation.

DURABILITY STUDIES IN SOUTH AMERICA

Several types of durability tests were investigated. These included (1) repetitive load tests, (2) slake durability, (3) Los Angeles abrasion, (4) hardness index as described by Novais Ferreira and Correia, 1965, and (5) the aggregate impact test. The California durability could not be performed because the equipment was not available. The California durability test during the Phase I investigation was found to be the most descriptive durability test for lateritic materials in Southeast Asia. The results of that study are included in this chapter.

A weathering durability test was also attempted which involved repeated boiling, drying and compacting but was found to be too time-consuming and severe and was abandoned.

Methods of Investigation

Samples of ironstones or concretionary gravel were collected from each major tropical soil group that contain these materials. The quality of these materials were rated by local engineers. Those which have performed very satisfactorily in pavement structures were rated good; those that have performed adequately, but with some breakdown, were rated fair, while those which have been found to be unsatisfactory were rated poor.

The 20 samples, whose locations are shown in Figure 8.1, were collected from 10 states. Fractions of all samples were sent to the University of Illinois for the repetitive load tests and to the University of Florida for the slake-durability test. Equipment for both of these tests was not available in Brazil.

Test Procedures

The Los Angeles Abrasion test was performed in accordance with AASHO T 96-70. In this test a sample of approximately 5,000 grams is placed in a steel cylinder 71 cm (28 in) in diameter with one 8.9 cm (3.5 in) wide steel shelf which lifts the charge of up to 12 steel spheres as the cylinder rotates. These spheres fall against the sample as the shelf approaches the upper vertical position. The cylinder is rotated at a speed of 30 to 33 rpm for 500 revolutions. The loss in weight after passing the sample through a 1.68 mm (No. 12) sieve is called the L.A. wear, which is:

$$\text{L.A. wear} = \frac{A - B}{A} \times 100$$

in which A = original oven dried weight of sample and B = weight of the sample after testing. A value of 40 is often specified as a maximum for base courses.

The Hardness Index was proposed as a modification of the Los Angeles abrasion test by Novais-Ferreira and Correia (1965) for ironstone materials. The test is performed in a Los Angeles abrasion machine but with only 200 revolutions and without the charge of steel spheres. The sum of the percentages retained on the 1 in, 3/4 in; 1/2 in; 3/8 in; No. 4; No. 10; No. 40; and No. 200 sieves (which is termed the granulometric modulus) for the natural material is divided by the sum of the percentages retained after the 200 revolutions. This is termed the hardness index and it is

FIGURE 8.1 – LOCATION OF DURABILITY SAMPLES IN BRAZIL

claimed that a minimum value of 0.80 excludes soft ironstones that do not perform satisfactorily in Angola (Novais-Ferreira and Correia, 1965).

The non-standard Brazilian aggregate impact test is a variation of the British Standard Aggregate Impact test. With the British Standard a 13.6 - 14.1 kg (30 - 31 pound) steel hammer is allowed to fall on an aggregate sample which approximately fills a steel cup 102 mm (4 in) in diameter and 51 mm (2in) deep. The hammer is 100 mm (3 15/16) in diameter and slides freely down vertical guides from a height of 381 mm ± 6.5 mm (15 in ± 1/4 in). The sample consists of dried material which passes the 13 mm (1/2 in) sieve and is retained on the 10 mm (3/8 in) sieve. After 15 blows the aggregate impact value is:

$$\text{AIV (aggregate impact value)} = \frac{B}{A} \times 100$$

where A = weight of oven dried aggregate sample and B = weight after testing of the fraction passing the 2 mm sieve (No. 10 US sieve).

Aggregates with impact values of less than 35 are considered excellent, those between 33 and 40 are considered good and those above 40 are considered fair to poor.

The slake-durability test is a recently developed procedure whose primary objective is to evaluate shales, mudstones, siltstones and other sedimentary rocks with appreciable clay content (Franklin and Chandra, 1972). The test was subsequently adopted as a suggested method for rock testing by the Commission for Standardization of Laboratory and Field Tests of the International Society for Rock Mechanics. The apparatus is shown in Figure 8.2.

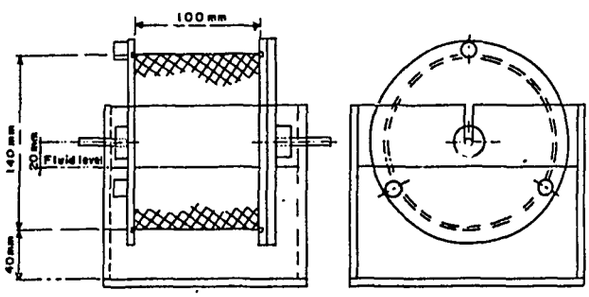

FIGURE 8.2 – CRITICAL DIMENSIONS OF SLAKE-DURABILITY TEST EQUIPMENT

The equipment consists of one to four drums 140 mm (5.5 in) in diameter and 100 mm (3.9 in) in length made up of 2 mm (0.0787 in) mesh screen, which are approximately half submerged in a slaking liquid and rotated at 20 rpm for 10 minutes. The sample remaining in the drum and the drum are then oven-dried. A modification was proposed by Deere and Gamble (1971) in which a second cycle is completed. They found the second cycle reveals more breakdown in clay-bearing rocks but that beyond the second cycle the loss is almost unchanged with the number of cycles. Two cycles were adopted for this study. The suggested procedure for determining the slake-durability index is given in the appendix to this chapter.

Frankling and Chandra (1972) investigated the effects of various slaking liquids including tap water and distilled water at various temperatures and sodium hexametaphosphate. Tap water was as effective a dispersant as any other and was adopted. Impurities in local tap water may require the use of distilled water. Slaking was sensitive to the water temperature, however and 20°C was specified for the standard test. The original classification of slake-durability was modified from Franklin by Gamble (1971) and is reproduced in Figure 8.3.

The repetitive load test was performed to determine if a prolonged dynamic loading pattern would provide an indication of the susceptibility of an aggregate to breakdown under traffic loading. This testing was performed at the University of Illinois and is discussed in the second half of this chapter.

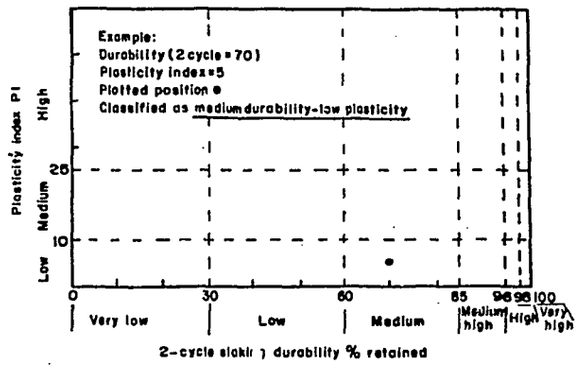

FIGURE 8.3 – A SUGGESTED DURABILITY CLASSIFICATION (after Gamble, 1971)

Test Results and Discussion

Results for the various tests are given in Table 8.1 and 8.2. The index properties, compaction characteristics and CBR values are given in Table 8.1. The Los Angeles abrasion, hardness index, aggregate impact and slake durability results are shown in Table 8.2, together with the qualitative rating provided by the local engineers.

It is apparent that there is little agreement between the qualitative ratings and the Los Angeles abrasion, hardness index and aggregate impact. Los Angeles abrasion results varied from 35 to 80. The aggregate impact results show a similar spread, from 30 to 72. Although such an abbreviated testing program justifies only highly tentative conclusions, these two tests do not appear to provide reliable indications of the performance of ironstone and concretionary gravel.

The hardness index does not appear to be sufficently discriminating. The two low values may indicate that low quality ironstone can be excluded but these results are hardly conclusive. On the other hand, there is almost no distinction between the fair and good quality materials.

The slake durability does show considerable promise. The results are given in Figure 8.4, in which the amount of the material retained is plotted against the number of slaking cycles. The Deere and Gamble (1971) ratings are also shown. Only 14 tests were run at the University of Florida as the remaining 6 samples were damaged in shipment. Figure 8.5 shows the percentage loss versus the number of cycles. It appears that with ironstones as well as with shales the loss becomes linear after the second cycle. Slake durability values are shown plotted against the qualitative ratings in Figure 8.6. A tentative classification is provided. A minimum value of 97 is suggested for high quality ironstones and concretionary gravels whereas 94 is suggested for medium quality ironstone and concretionary gravel materials. These values are slightly different from those recommended by Deere and Gamble.

Additional research is recommended. A far greater number of samples should be tested including 20 to 30 from individual deposits to determine the sensitivity of the test to variability of deposits. The test should be followed through several construction projects in which lateritic materials are utilized.

TABLE 8.1
PHYSICAL PROPERTIES OF SAMPLES TESTED

Location	Sample No.	Aggregate Type Description	Gradation % Before Gravel	Before C. Sand	Before F. Sand	Before −200#	After Gravel	After C. Sand	After F. Sand	After −200#	Compaction AASHO Mod. MDD (pcf)	OMC	CBR
Pará	J. Lazaro	Fx3 - 2a Yellow sandy lateritic Soil	51	9.4	20.2	19.3	–	–	–	–	134	8.8	112
Pará	J. Nordeste	Fx3 - 2a Yellow sandy lateritic Soil	81	7.4	4.98	6.63	58.8	15.19	9.8	16.0	130	9.5	97
Maranhão	Peritoro	Lfg - 3b Ref sandy Lat. Gravel.	73.8	6.0	8.4	11.5	59.3	8.7	9.4	22.4	119	9.1	133
Piauí	No. 1	Qf14 - 1a Yellow sandy Lat. Gravel.	76	4.5	6.2	13.61	62.3	10	6	21.7	138	8.1	11
Piauí	No. 2	Qf14 - 1a Yellow Lat. Gravel not sandy	74.5	9.5	6	9.96	72.3	6.3	6.4	14.8	126	9.2	75
Ceará	No. 2	Ao2 - 2a Red Sandy Clay Lat. Gravel.	58.4	4.1	5	32.0	–	–	–	–	128	12.3	16
R.G. Norte	Rio Taborda	Qf11 - 1a Gray Lat. Gravel	71	10	10	8.8	70	13	7.5	8.7	139	9.7	76
Paraíba	Cabo Branco	Qf11 - 1a Clayey Lat. Gravel.	60	9.3	10	20	36	13	16	33.8	127	11	63
Paraíba	Mari-Guabira	Lf22 - 3b Red Lat. Gravel	69	11.6	8.6	11.17	34	23	16	26.6	143	8.0	180
Pernambuco	Brejinho	Lc2 - 3b Yellow sandy Lat. Gravel.	64	11	10.4	14.6	65	10	9	16	143	9.4	49
Bahia	Guandu	Lf10 - 3b Yellow Lat. Gravel	65	11	11.2	12.7	48	15	18	18	147	10.5	39
Bahia	Vitória da Conquista	Lf23-2/3a Quartz Gravel Lateritic	49	17	12	22	54	13	11	21	133	6.7	156
Bahia	Esplanada	Ao28-3a Red Sandy Lat. Gravel.	62	10	16.	11	53	15	19	14	139	7.7	41
Goiás	Anápoles	Fa2 - 3a Yellow Clayey Gravel	64	6	4.0	25	55	10	6	29	116	16	149
Goiás	Santa Helena	Fa2 - 3a Fri-3a Lat. Gravel	47	29	5	18	72	5	4	18	129	14.4	52
Brasília	No. 2	Fa2 - 3a Lat. Gravel.	52	4	12	32	62	4	9	25	128	11	163
Minas Gerais	BR - 262 Km 262	Fa2 - 3a Lat. Gravel	66	3	19	12	50	6	26	18	136	7.2	129
Minas Gerais	BR - 262 Km 275	Fa2 - 3a Lat. Gravel	61	4	5	30	60	6	9	25	125	15	55
Mato Grosso	Rio da Casca	Ao15 - 3a	64	7	7	22	–	–	–	–	128	16.5	24
Mato Grosso	Rio Itiquira	Fa2 - 3a	73	6	12	9	–	–	–	–	130	11	216

TABLE 8.2
RESULTS OF DURABILITY TESTS

Sample Location	State	Soil Type	Rating (*)	L.A. Abrasion	Hardness Index	Impact Aggregates %	Slake Durability %
Rio Itiquira	Mato Grosso	Ferralsol Fa2-3b	G	43	100	63	97.7
Mari/Guarabira	Paraíba	Luvisol Lf 22-3b	G	36	100	30	–
Rio Taborda	R.G.Norte	Arenosol Qf 11-1a	G	49	100	46	99
No. 1	Piauí	Arenosol Qf 14-1a	G	39	(**)	37	96.6
Anápoles	Goiás	Ferrasol Fa 2-3a	G	54	100	58	–
Santa Helena	Goiás	Ferralsol Fr 11-3a	G	39	100	(**)	–
Esplanada	Bahia	Acrisol Ao 28-3a	G(?)	80	100	72	97.7*
Campos Altos (a)	Minas Gerais	Ferralsol Fa 2-3a	G	49	90	57	–
Campos Altos (b)	Minas Gerais	Ferralsol Fa 2-3a	G	44	90	61	–
Brejinho	Pernambuco	Luvisol LC 2-3b	G	(**)	90	31	97
Cabo Branco	Paraíba	Arenosol	F/G	53	80	47	92.9
Rio da Casca	Mato Grosso	Acrisol Ao 15-3a	F	43	100	35	98.5
No. 2	Brasília	Ferralsol Fa 2-3a	F	(**)	100	43	–
Peritoro	Maranhão	Luvisol Lf 9-3b	F	47	(**)	49	95
Jazida Nordeste	Pará	Ferralsol Fx 3-2a	F	48	100	57	94.5
No. 2	Piauí	Arenosol Qf 14-1a	F	55	100	53	96
Vitória da Conquista	Bahia	Luvisol Lf 23-2/3a	(?)	35	100	(**)	94.5
No. 2	Ceará	Acrisol Ao 2-2a	P	57	80	56	90.5
Jazida Lazaro	Pará	Ferralsol Fx 3-2a	P	54	70	42	91
Guandu BR-101	Bahia	Luvisol Lf 10-3b	P(?)	73	100	58	90.8

* G = Good; F = Fair; P = Poor
* * Not Performed

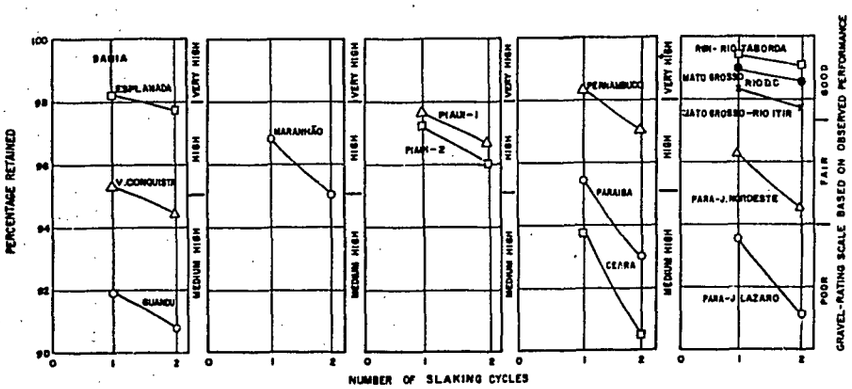

FIGURE 8.4 – RELATIONSHIP BETWEEN PERCENTAGE RETAINED AND NUMBER OF SLAKING CYCLES INCLUDING A GRAVEL CLASSIFICATION BASED ON OBSERVED PERFORMANCE

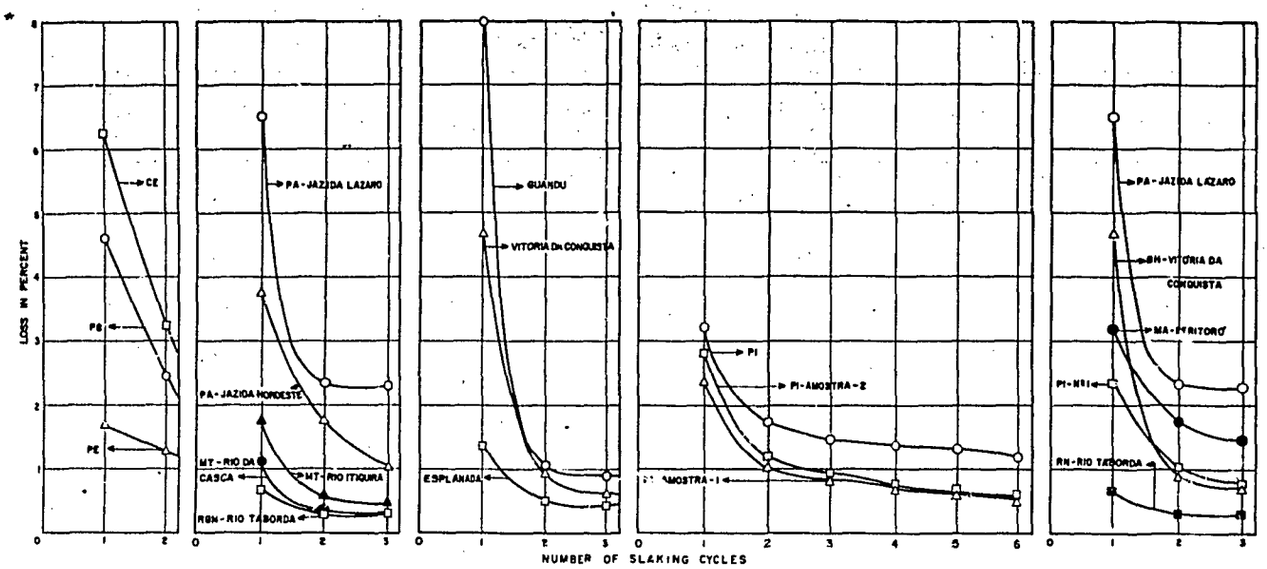

FIGURE 8.5 – RELATIONSHIP BETWEEN LOSS IN PERCENT VERSUS NUMBER OF SLAKING CYCLES

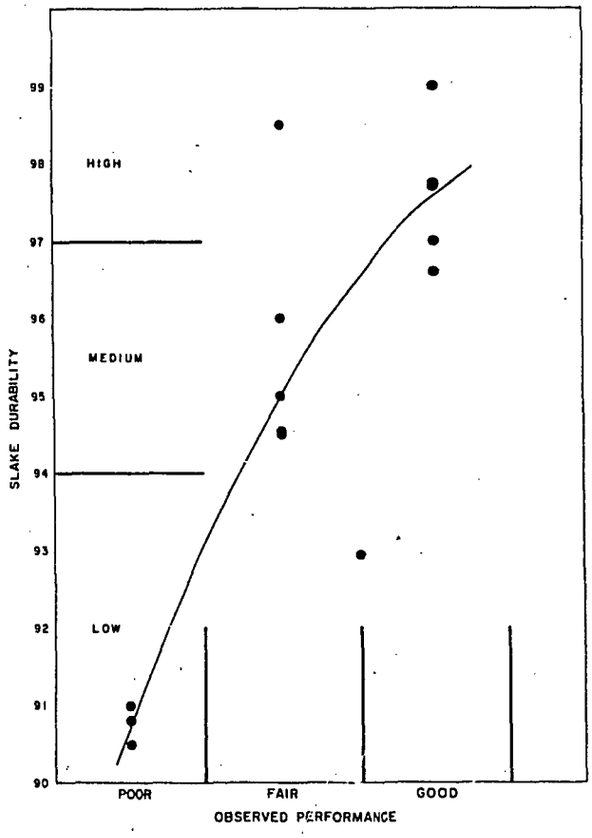

FIGURE 8.6 – RELATIONSHIP BETWEEN SLAKE DURABILITY AND OBSERVED PERFORMANCE

DURABILITY STUDIES IN SOUTHEAST ASIA

This section is taken largely from Vallerga et al (1969). Lateritic materials are reported to have properties ranging from those of a soft clay to those of a coarse stone. In addition, its hardness is reported to change over a wide range when subjected to drying or when subjected to cycles of wetting and drying. In order to evaluate the engineering significance of these variations it is necessary, first, to have suitable methods by which to evaluate the changes occurring in lateritic soils due to wetting and drying. In general, the term durability means resistance to degradation from any cause, including mechanical, thermal and chemical actions. These special studies on durability of lateritic materials were conducted in three phases: selection of test materials, evaluation of test methods; and evaluation of wetting and drying on engineering properties.

In order to make the results of this study more useful and widely applicable, careful consideration was given to the selection of materials. Since the principal uses of lateritic soils in Southeast Asia are in the construction of roads and airfields, the potential use of a material for this purpose was the major criteria for selection. Materials were selected to cover the range of uses in such construction, i.e. from select fill (poorer quality) to aggregate bases (best quality).

Before making the selection, over one hundred lateritic gravel sources throughout Thailand were sampled. These samples were tested for gradation, plasticity, and durability. On the basis of these test results, seven lateritic soils were selected to represent the range of engineering properties (especially mechanical durability and plasticity) from very good to very poor. Large samples were obtained from these sources for use in the durability studies as well as in the stabilization studies described in Chapter 9. A good, petrologically homogeneous limestone and a siliceous river gravel were also obtained for control purposes.

Evaluation of Test Methods

The Los Angeles Abrasion test (LAA) (ASTM, C131-64T) and the California Coarse (D_c) and Fine (D_f) Durability tests (Calif. 229-C) were studied to determine their suitability for evaluating the mechanical durability of lateritic gravel aggregates. It was hypothesized that if these were suitable, the actual mechanical durability of the materials as observed in service, would correlate with the test results. The LAA test yields low values for tests on durable materials, while the D_c and D_f tests give high values for the same materials.

Many lateritic gravels throughout Southeast Asia have been observed to indurate to some extent upon excavation and exposure to the atmosphere. Because of the nature of their formation, concretions in these gravels are, in general, physically and chemically heterogeneous. This is evident particularly from the differences in properties between the various concretion sizes within the same material, with the particles in the coarser fractions being denser and more durable than the particles in the finer fractions. Ferruginous concretions may be essentially absent in fraction smaller than about the 600 μm (No. 30) sieve; the finer fractions being composed predominantly of siliceous minerals. In addition to the predominance of indurated ferruginous concretions, which form the basis for calling material lateritic gravel, sound quartzitic river gravels or other materials may be present in varying amounts in some deposits.

Observations of road construction practices with lateritic gravels throughout Southeast Asia have indicated that materials are usually used or rejected on the basis of tests on pit-run samples. Road construction programs may not provide sufficient funds or time to allow processing of the material to improve its quality.

The facts mentioned in the two preceding paragraphs, led to the decision to compare the standard test procedures and the effects of these three variables on the LAA and D_c test results: gradation, petrologic composition, and predrying at constant grading.

Four aggregates were tested in a wide range of gradations in order to evaluate the sensitivity of the tests to gradation. These different aggregates were used to cover the range of results which may be associated with lateritic gravels of widely varying quality. The gradations selected represent the extreme range of possible gradations, as well as the standard test grading, and the natural (pit-run) grading. The artificial gradings used are presented in Table 8.3, and the natural gradings in Table 8.4. All tests were run according to standard procedures with the exception of the LAA test on samples with non-standard gradings in which 10 steel balls were used regardless of grading.

TABLE 8.4
NATURAL SAMPLE GRADATIONS USED IN DURABILITY STUDIES

Sieve Fraction	Aggregate Number							
	1	2	3	4	5	6	7	8
3/4" x 3/8"	18.0	32.2	44.8	50.0	14.0	38.3	35.1	39.8
3/8" x #4	50.2	41.4	37.8	36.4	47.5	45.3	31.7	42.6
#4 x #8	31.8	26.4	17.4	13.6	38.5	16.4	33.2	17.6

In order to evaluate the effect of petrologic composition on the LAA and the D_c test results at constant grading, a very weak, soft laterite (Aggregate 4) was combined with a hard durable crushed limestone (Aggregate 8) in various proportions. All of the samples were graded with the natural grading of Aggregate 4. The proportion of the two materials present in any given sample was constant in all fractions. The percent of each aggregate present in each of the five samples was as follows:

Aggregate	8	100	90	50	10	0
Aggregate	4	0	10	50	90	100

Possible effects of predrying the coarse lateritic gravel aggregate in preparation for testing were investigated by running the LAA standard C grading on Aggregates 1, 5, and 6 at saturated surface dry, air dry, oven dry, and high temperate (180°C) oven-dry conditions. The material was sampled moist and retained moist until washed clean of fines, fractionated, and predried to the prescribed test condition.

TABLE 8.3
ARTIFICIAL SAMPLE GRADATIONS USED IN DURABILITY STUDIES

Key Word(s)	3/4" x 3/8"	3/8" x #4	#4 x #8
Very Coarse	100.0	0.0	0.0
Coarse	80.0	10.0	10.0
Intermediate	0.0	100.0	0.0
Well	33.3	33.3	33.3
Fine	10.0	10.0	80.0
Very Fine	0.0	0.0	100.0
Standard B*	100.0	0.0	0.0
Standard C*	0.0	100.0	0.0
Standard D_c*	77.5	22.5	0.0

* The Standard B and C Gradings refer to the ASTM Standards of the same letters for the LAR test. The Standar D_c refers to the unbiased California Standard Grading for the D_c tests.

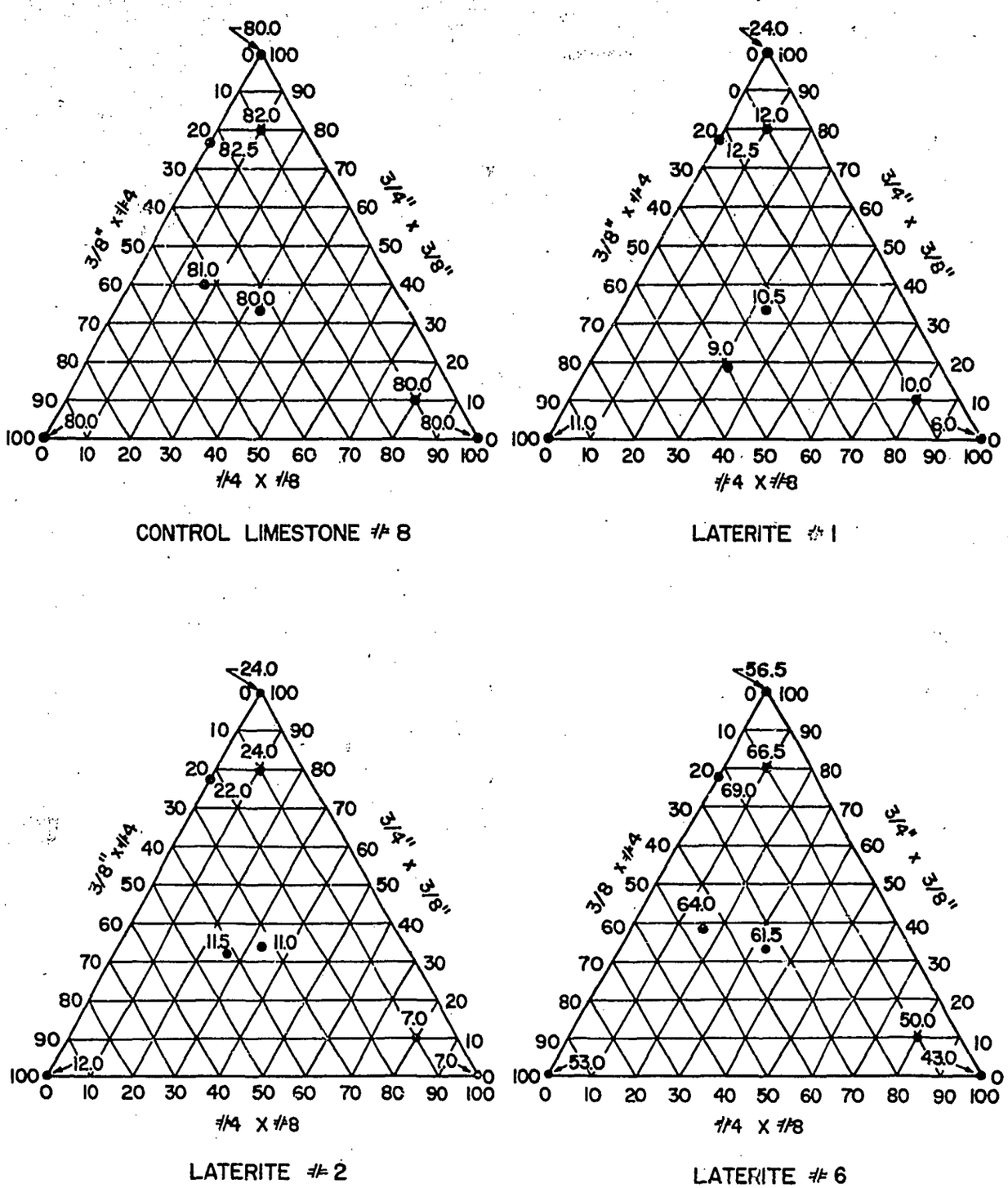

FIGURE 8.7 – VARIATION OF COARSE DURABILITY (Dc) WITH RESPECT TO GRADING
(number on figures are Dc test results)

The D_f test was run by the standard procedure, with the exception that the samples were air dried at 60 to 70° C instead of oven dried at 105° C. This was done to preclude the possible irreversible dehydration of halloysite ($4H_2O$) to halloysite ($2H_2O$), should it be present in the sample. This mineral is included in the kaolinite clay mineral group which composes most of the clay minerals observed in lateritic gravels in Thailand, and the change from halloysite ($4H_2O$) to halloysite ($2H_2O$) is known to affect the engineering properties of the clay.

COARSE DURABILITY TEST:

For materials such as limestone which are petrographically homogeneous throughout all particle sizes, the gradation of the sample apparently has little or no affect on the D_c test results (Figure 8.7). However, with heterogeneous materials such as lateritic gravels, the effect of changing the grading is essentially that of changing its petrologic composition. Changing both variables simultaneously produces the results shown in Figure 8.7. The points of the triangles represent the D_c of each component fraction. Note the differences in these points for each of the four materials. The difference between results is indicative of the range of variability which may be expected in lateritic gravels from Thailand. Figure 8.8 indicates typical results of varying composition at constant gradation. It appears that the D_c test is very sensitive to the presence of poor materials even in relatively small quantities. Such small quantities of material could cause the failure of an otherwise good base course if the poor material degrades to silts and clays while in service.

LOS ANGELES RATTLER TEST:

The LAA test appears to be slightly more affected by grading than does the D_c test. It is suggested that the cushioning effect of more fine material may result in a reduction in the percentage loss. This could account for the apparently conflicting results for the D_c and LAA tests, indicated by a comparison of the lower right corners of Figures 8.7 and 8.9 for Laterite No. 1. Visual comparison of the materials tends to substantiate the evaluation of quality given by the D_c test results.

Though plotted points generally represent an average of the results of two or more tests, the normal experimental deviation could account for the observable variation. Figure 8.8 indicates the variability which may be expected with the LAA test when run with 10 balls regardless of grading. Figure 8.10 illustrates the results obtained with 11 and 8 ball test charges, respectively, which tend to bracket the non-standard, natural graded results with a 10 ball test charge. Data indicate that there is no apparent correlation between the loss after 100 revolutions and the loss after 500 revolutions, and that there is no apparent advantage, in terms of elimination of grading effect or improved repeatability, to be gained by using a sieve other than the 1.68 mm (No. 12) to compute percent loss.

The effect of petrologic composition on the LAA test and D_c test results is shown in Figure 8.8. A major difference between the D_c and LAA tests appears to be that the LAA test does not evaluate the nature of the fines produced during disintegration of the sample. Though a shale and a coarse sandstone may give similar LAA results, the nature of the fines produced during disintegration are very different. If incorporated in a pavement, these materials might yield considerably different performance under similar conditions. The in-service degradation of the shale to silts and clays would have a more detrimental effect than would the degradation of the coarse sandstone to sand. The tendency of an aggregate to produce silt or clay when abraded in the presence of water is evaluated in the D_c test, but not in the LAA test.

It has been suggested by various investigators that the method of sample preparation may greatly affect the results of tests run on lateritic soils. In order to examine this potential effect on coarse aggregate, the LAA test was made on specimens at several moisture contents. All samples were tested with 10 balls and natural grading. New material was used for each test. The results of these tests are shown in Table 8.5.

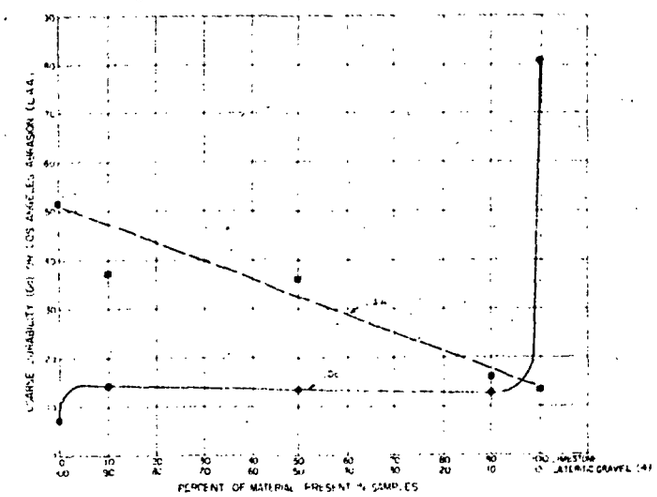

FIGURE 8.8 – EFFECT OF PETROLOGIC COMPOSITION ON Dc AND LAA TEST RESULTS

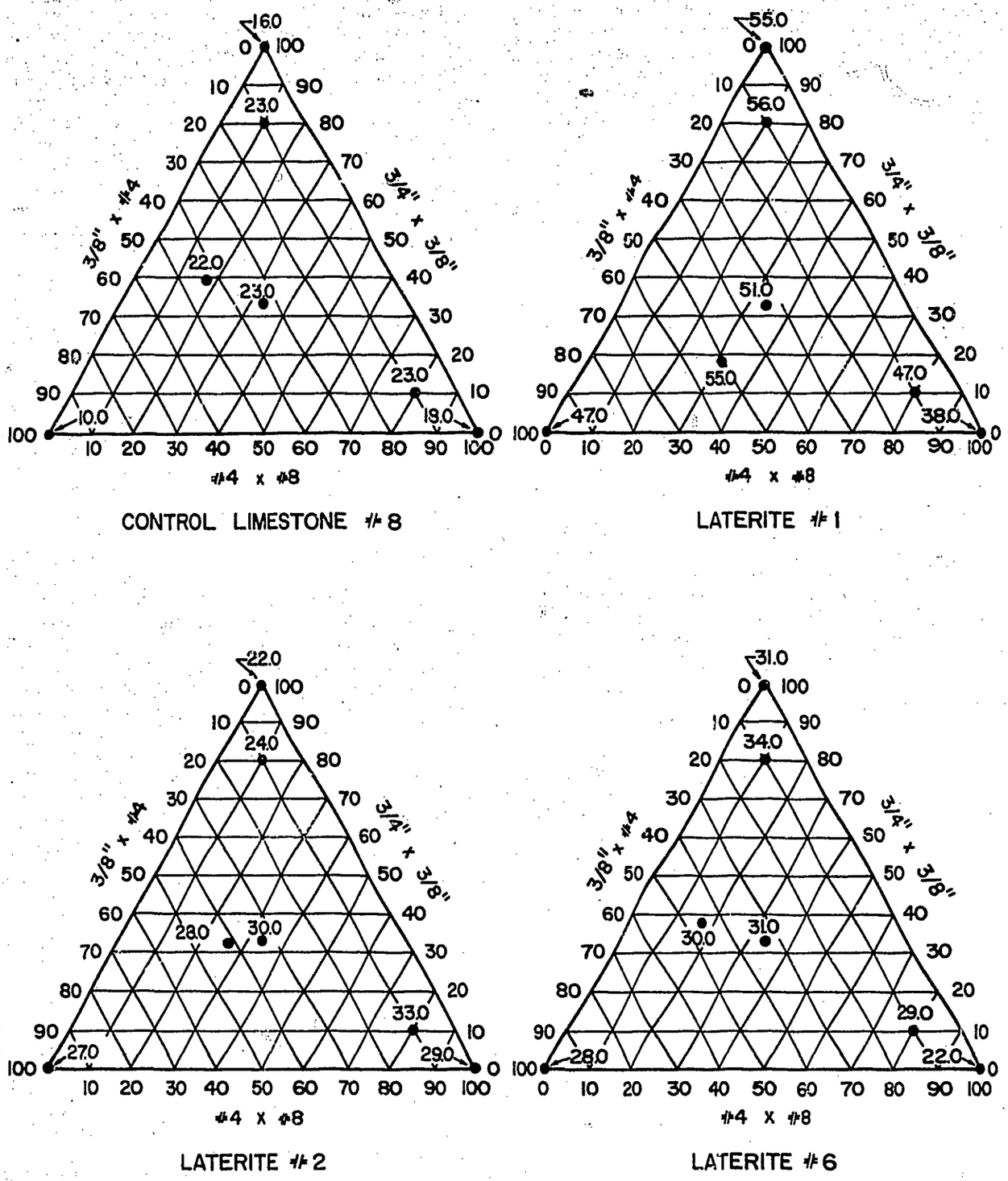

FIGURE 8.9 — VARIATION OF LOS ANGELES ABRASION (LAA) WITH RESPECT TO GRADING
(number on Figures are LAA test results)

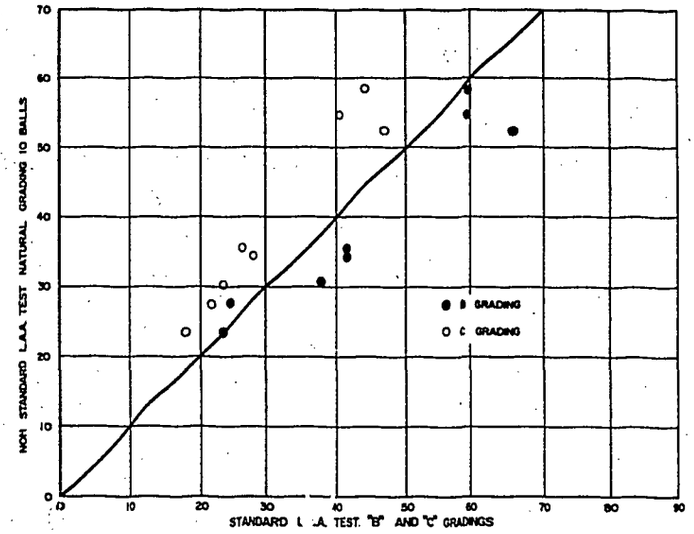

FIGURE 8.10 – STANDARD vs NON STANDARD LAA TESTS ON THE SAME MATERIAL

TABLE 8.5
Summary of Test Results, Southeast Asia
(After Vallerga, et al., 1969)

	Modified Los Angeles Abrasion Test Data			
Aggregate Number	Saturated Surface Dry	Air Dry	Oven-dry at 105° C	Oven-dry at 180° C
1	11	46	44	43
5	16	48	47	47
6	13	25	23	23
53*	18	–	22	–
47B*	40	–	51	–
49B*	37	–	44	–
50*	12	–	34	–
44A*	10	–	24	–

* These results were obtained on materials examined during preliminary testing.

It appears that the three methods of drying these lateritic gravels at the relatively low temperatures results in very little difference in LAA test results. The moist fines generated from the saturated surface dry samples tended to adhere together and coat other particles during testing, possibly reducing the effective impact energy of the steel balls; thus raising questions as to the validity of the test under these conditions. The D_C test can be satisfactorily run on moist samples, although for some lateritic gravels high in plinthite, the result of a moist D_C test may be considerably lower than that of a dry test. However, these materials are so obviously poor that they would not be used in the upper layers of a pavement, and durability testing would not be applicable.

Though low temperature drying apparently has little effect on the coarse aggregate, it is possible that the heating of lateritic aggregates to temperatures on the order of 400°C could cause a considerable change in the durability of the material, since goethite and limonite dehydrate and crystallize to form the harder mineral hematite at temperatures between about 300 and 400°C. Also, some expansive clay minerals dehydrate at lower temperatures. If all fractions of a lateritic gravel were processed within this temperature range, such as by means of a hot-mix asphalt plant, the overall quality of the material for highway construction might be improved sufficiently to justify the cost of treatment. This was beyond the scope of this investigation, but is suggested as a possible subject for future study.

Conclusions Regarding Test Methods

The test procedures investigated appear to be generally suitable for evaluating the mechanical durability of lateritic materials. However, the following modifications of the standard procedures are recomended in order to arrive at more meaningful test results:

1. The natural pit run or the "as used" grading of the material coarser than the 2 mm (No. 10) sieve should be used in preparing samples for the D_C and LAA tests. Since lateritic gravels are petrologically heterogeneous, an artificial laboratory granding may not give results representative of the material as it will be used.
   The 2 mm (No. 10) sieve, rather than the 4.75 mm (No. 4) was selected as the minimum sieve size because most lateritic gravels are "gap-graded", with the particle sizes between about the 2 mm (No. 10) and 75 mm (No. 200) sieves frequently missing. The result is a gravel "floating" in silt or clay. Use of the 2 mm (No. 10) sieve will result in including the smaller nodules, frequently the weakest, in the durability test.
2. Oven drying of samples prior to conducting the LAA, the D_C, or the D_f test is not recommended, primarily because of mineralogic changes which may occur. For good quality coarse-grained materials oven drying appears to have no significant effect on the LAA or D_C test results. However, for coarse-grained lateritic materials containing a high

proportion of weak laterite or plinthite (such as laterites from poorly drained tropical rain forest areas) oven drying may cause particle cementation, mineralogic alteration of clays, and alteration of iron minerals. If such changes occur, they will affect the LAA, and especially, the D_c and D_f test results. The effect will be toward indicating a higher durability than actually exists.

3. It is recommended that the LAA test be run with 10 standard steel balls, regardless of sample grading. The effect of varying the number of balls was observed to be more severe than the effect of changing the grading. Changing both gradation and the number of balls may lead to widely differing results on the same material. In general, the LAA test run with 10 balls on the naturally graded material yielded results midway between the standard "B" (11 ball) and "C" (8 ball) grading results for the same material. Furthermore, it is suggested that the natural grading and 10 ball procedure be called the "L" grading.

The LAA test results for lateritic gravels from Thailand may range from 20 to 60, when tested with the "L" grading. The D_c may vary from 5 to 80 and the D_f from 15 to 70. Thus, lateritic gravel aggregates range in quality from very weak to comparable to good limestone. The data indicate that both the LAA and D_c tests correlate well with the apparent mechanical durability of laterite gravels in service. However, the D_c and D_f tests on the air-dried natural (or as-used) grading, are believed to provide a better measure of the relative durability of these materials than does the LAA. These tests measure not only the resistance of the material to rubbing abrasion in the presence of water, but also evaluate the quality and quantity of the fines produced by this abrasion. Furthermore, both the D_c and D_f tests results correlate well with the disintegration of the material under the action of repeated loading.

The two tests together measure the relative mechanical durability of the entire material from 19 mm (3/4 in) through the 75 mm. (No. 200) sieve.

Though these tests appear to provide good and valid information, there is some problem with the repeatability of the D_f test when run on some lateritic soils.

It is recommended that duplicate D_f tests be run on lateritic soils and the results averaged. Although the D_c test is affected by the same factors which affect the D_f test, the test portion is over ten times as large and the coarser fractions of laterite generally are better formed and more homogeneous than the finer fractions. This accounts for the fact that the repeatability of the D_c test is much better than that of the D_f test.

The California Division of Highways, which developed the D_c and D_f tests, has concluded that no direct correlation exists between these and the LAA test. An examination of the data obtained during the present investigation confirms this conclusion.

REPETITIVE LOAD TESTING

Repetitive loading and durability studies were conducted on 10 ironstone aggregates. The sample covered a range in characteristics, (as classified on the basis of performance) from poor to good.

The compaction tests were performed in Brazil where the ironstone and concretionary gravel samples were crushed to minus 1.3 cm (1/2 in) in a laboratory jaw crusher. Test results are given in Table 8.6. The remaining tests were performed at the University of Illinois. It was not always possible to achieve the densities obtained in Brazil. Especially low densities were observed (as shown in Table 8.6) with samples 2B (Pará), 4 (Cabo Branco), 5 (Paraíba) and 9 (Brasília).

Specimen Preparation and Testing

It was not possible to follow the durability procedures reported in the report for Phase I (Vallerga, et al., 1969) since the comparisons of degradation they discussed were based primarily on studies of artificially graded aggregates. In order to evaluate the amount of degradation as accurately as possible the following procedure was adopted:

1. The full amount of material available was dry sieved and separated into fractions on a sieve series varying from 12.5 mm (1/2 in) to 75 μm (No. 200) as listed in Table 8.6.
2. The fractions were then recombined in the air-dry state to produce two samples one of about 700-900 grams and another of about 1,500 to 2,000 grams.
3. For samples 1A, 1B, 2A, and 4, the smaller fraction was throughly dry mixed and then subjected to a wet sieve analysis. These results are designated in Table 8.6 as "Before Testing". The remaining samples were not subjected to a wet sieve analysis until after the repetitive load tests.
4. The larger fraction was thoroughly dry mixed and used to form two test specimens as described in a later section on repetitive loading.
   a. One of these specimens was used as a trial to reproduce as nearly as possible the maximum AASHO T-180 dry density. After compaction the complete specimen was subjected to a wet sieve analysis. These results are designated in Table 8.6 as "After Compaction".
   b. The other specimen was also compacted to essentially 100% AASHO T-180 dry density and then subjected to repetitive loading tests. After approximately 100,000 load applications (except for sample 2A) at a principal stress ratio of 3, the specimen was also subjected to a wet sieve analysis. These results are indicated on Table 8.6 as "After Testing".

TABLE 8.6
SUMMARY OF TEST DATA, SOUTH AMERICA

Sample Identification	Fraction	Wet Sieve Analysis: Before Testing	After Comp.	After Testing	Moisture-Density (Brazil): γ_d and w	Trial Specimen: γ_d and w	Tested Specimen: γ_d and w	Modulus of Resilience, ksi at $\sigma_1/\sigma_3 = 3$; Number of Load Applications (N): 1	10	100	1000	10,000	100,000
1A	1/2 - 3/8	6.9	8.2	8.1	118.2pfc	115.8	119.5	12.3	15.0	16.6	17.7	16.6	15.2
Maranhão -	3/8 - #4	21.4	21.0	19.4	9.1%	10.4%	9.0%						
Peritoro	#4 - #8	12.9	14.3	14.4									
	#8 - #16	8.7	9.5	9.6									
Rating: Fair	#16 - #30	6.6	6.0	7.5									
	#30 - #50	4.5	4.9	5.3									
	#50 - #100	6.7	7.1	6.9									
	#100 - #200	9.5	9.7	10.0									
	Pass #200	22.8	19.3	18.3									
1B	1/2 - 3/8	14.7	14.5	15.6	138.4	130.4	134.7	14.7	15.2	16.3	17.3	17.6	22.6
Rio G. do Norte-	3/8 - #4	30.8	29.3	30.5	9.7%	10.6%	10.6%						
Rio Taborda	#4 - #8	11.8	11.2	11.2									
	#8 - #16	4.5	6.3	5.3									
Rating: Good	#16 - #30	2.7	6.2	5.5									
	#30 - #50	6.2	7.7	8.3									
	#50 - #100	12.5	11.1	11.4									
	#100 - #200	6.7	6.5	5.7									
	Pass #200	9.9	7.3	6.7									
2A	1/2 - 3/8	15.8	9.9	12.0	127.5	129.4	129.3	14.7	16.4	20.8	Sample failed at N = 200		
Ceara - No. 2	3/8 - #4	47.7	41.9	41.0	12.3%	12.4%	12.4%						
	#4 - #8	13.2	13.2	12.4									
Rating: Poor	#8 - #16	5.3	5.9	5.9									
	#16 - #30	4.[illegible]	5.8	5.7									
	#30 - #50	2.[illegible]	4.8	4.9									
	#50 - #100	3.4	5.3	5.6									
	#100 - #200	2.9	4.1	4.2									
	Pass #200	4.7	9.1	8.4									
4	1/2 - 3/8	5.4	2.5	5.6	126.8	110.0	108.7	11.4	12.3	14.3	14.7	15.8	18.0
Paraíba -	3/8 - #4	16.8	10.4	13.5	10.8%	15.6%							
Cabo Branco	#4 - #8	10.8	6.6	7.1									
	#8 - #16	13.0	8.3	9.0									
Rating: Fair	#16 - #30	11.4	10.9	12.6									
	#30 - #50	12.4	13.5	14.7									
	#50 - #100	12.2	14.6	14.6									
	#100 - #200	7.4	9.8	8.8									
	Pass #200	10.7	23.4	14.1									
4B	1/2 - 3/8	4.3		4.0	129.2		113.6		15.1	15.4	17.9	17.9	18.9
Pará	3/8 - #4	12.3		13.8	8.8		9.5						
	#4 - #8	6.0		6.3									
J. Lazaro	#8 - #16	4.3		3.7									
	#16 - #30	5.1		4.1									
Rating: Poor	#30 - #50	13.9		11.0									
	#50 - #100	29.8		23.4									
	#100 - #200	9.5		12.9									
	Pass #200	14.8		20.8									
5	1/2 - 3/8	11.9		11.0	142.8		116.7		17.6	18.0	19.2	19.7	23.1
Paraíba	3/8 - #4	29.1		25.1	8.0		10.3						
	#4 - #8	15.7		11.2									
Mari -	#8 - #16	10.4		10.8									
Guarabira	#16 - #30	7.7		9.3									
	#30 - #50	6.7		8.6									
Rating: Good	#50 - #100	6.6		8.6									
	#100 - #200	4.3		5.4									
	Pass #200	7.6		10.0									
6	1/2 - 3/8	7.8		6.6	141.3		138.5		141.3	16.6	Sample failed at N = 200		
Piauí	3/8 - #4	32.7		33.9	8.1		8.7						
	#4 - #8	19.8		20.2									
No. 1	#8 - #16	8.8		8.4									
	#16 - #30	4.7		4.6									
Rating: Good	#30 - #50	2.5		2.5									
	#50 - #100	2.5		2.5									
	#100 - #200	5.3		4.8									
	Pass #200	15.9		16.5									
7	1/2 - 3/8	5.4		5.4	125.7		124.5		15.9	17.6	19.2	19.8	20.2
Piauí	3/8 - #4	32.8		29.8	9.2		10.9						
	#4 #8	20.6		20.0									
No. 2	#8 - #16	11.3		11.4									
	#16 - #30	7.0		6.9									
Rating: Fair	#30 - #50	4.0		3.7									
	#50 - #100	4.0		4.2									
	#100 - #200	5.2		6.0									
	Pass #200	9.7		12.6									
Brasília	1/2 - 3/8	8.6		5.0	127.3		119.1		17.6	19.8	20.2	20.7	22.8
	3/8 - #4	29.6		31.2	11.0		12.9						
No. 2	#4 - #8	20.9		21.7									
	#8 - #16	6.3		9.2									
Rating: Fair	#16 - #30	2.4		5.2									
	#30 - #50	4.1		4.7									
	#50 - #100	10.0		6.9									
	#100 - #200	9.7		6.3									
	Pass #200	12.6		9.8									
11	1/2 - 3/8	8.6		8.2	129.0				19.4	20.3	22.5	22.5	22.8
Goiás	3/8 - #4	44.7		40.8	14.5								
	#4 - #8	21.2		21.7									
St. Helena	#8 - #16	5.0		8.8									
	#16 - #30	1.7		5.9									
Rating: Fair	#30 - #50	3.5		4.0									
To Good	#50 - #100	7.1		4.6									
	#100 - #200	3.5		2.6									
	Pass #200	4.7		3.4									

Results of Durability Testing

A comparison of the sieve analysis summarized in Table 8.6 does not permit any significant conclusions to be drawn regarding the effect of compaction and loading on the increase in the amount of fines. Samples 1A (Peritoro), 1B (Rio Taborda), 9 (Brasília) and 11 (Goiás) actually show a decrease in fines after testing. This is believed to be due, at least in part, to the loss of fines by sticking to the sides of the mixer prior to compaction.

Samples 6 and 7 (Piauí) show practically no change in gradation with testing.

Sample 5 (Paraíba) shows an increase in fines passing the 150 μm (No. 100) sieve of about 3.5 percent which is about 1.5 percent below the increase noted in Sample 2A (Ceará) and Sample 4 (Cabo Branco).

Sample 2B (Pará) showed a significant increase in fines (minus No. 100) or more than 9 percent. It is perhaps significant that this specimen when compacted for testing had a dry density 240 kg/m^3 (15 lb/cu ft) below the maximum obtained in Brazil and that the aggregate was rated as poor.

Sample 2A (Ceará No. 2) had nearly double the fines after compaction and testing than before testing. Since the sample failed after only 200 load applications this perhaps significant degradation appears to be due primarily to the compaction process. It is impossible to predict whether further load repetitions would have caused significant degradation. Much more significant from the standpoint of evaluating the material as a road building material is the fact that the sample did fail after only 200 load applications.

Sample 4 (Cabo Branco) also appears to show significant degradation under compaction and repetitive loading. However, the results do not indicate conclusively that any degradation was due to repetitive loading, but seems to be associated primarily with the compaction process. Reference to the modulus of resilience data, however, seems to indicate that this sample was not of as high quality as Sample 1B (Rio Taborda) and probably not quite as good as 1A (Peritoro). Again the fact that it was not possible to produce the AASHO T-180 density makes the above comparisons and the qualitative rating inconclusive.

Specimen Preparation for Repetitive Loading

For each of the four samples, enough material was obtained to compact two 6.4 cm (2.5 in) diameter by 12.7 cm (5 in) high cylindrical specimens. The material was dry-mixed in a Lancaster mixer to insure uniformity. Sufficient water was then slowly added to the gravel mixture to bring it to approximately the optimum moisture content as determined by AASHO Procedure T-180 and the mixture thoroughly blended The modified compaction (AASHO T-180) values for optimum moisture and maximum dry density are summarized in Table 8.6 as supplied from Brazil.

Two specimens were then compacted. A split mold, lined with a rubber triaxial membrane was used to facilitate specimen removal. The specimens were compacted in five equal layers with a full-faced compaction hammer that had a 1.8 kg (4 lb) hammer falling 30.5 cm (12 in). Sufficient blows were placed on each layer to obtain the desired density. In general, this required about 50 blows per layer.

Table 8.6 summarizes the moisture and density conditions obtained for the specimens prepared from the 10 samples. With the exception of the specimens made from the material from samples 2B, 4, 5, and 9, the moisture and density conditions obtained were very similar to the modified compaction values previously supplied.

The moisture-density values obtained with the Sample 4 specimens substantially vary from the target values. However, it was found that when compacted at the 10.8 percent moisture content recommended, the gravel material was very dry and obviously substantially below optimum moisture conditions. The moisture content was thus adjusted so that visually, the material had a similar moisture condition and a consistency as the other samples when they were at optimum.

For the two specimens prepared for samples 1A, 1B, 2A, and 4, one specimen was subjected to a wet sieve analysis immediately after compaction to determine the amount of degradation that occurred during compaction, while the other specimen was immediately prepared for the repeated load triaxial test.

Repeated Triaxial Testing

For the specimen to be tested in the repeated load triaxial cell, a loading base and loading cap were carefully placed on the ends of the specimen and a second rubber triaxial membrane placed around the specimen and secured with rubber bands.

The specimen was then placed in a triaxial cell and the cell placed between a load actuator and a loading table. The specimens were not saturated but were tested at the compaction moisture content.

The desired magnitude of confining pressure (non-repeated) was applied to the specimens with compressed air which was carefully controlled by means of a regulator. The repeated axial stress was applied by means of compressed air and a Bellofram cylinder. The magnitude of axial specimen stress was controlled electrically with a 3-way solenoid valve. The duration of each load pulse was 0.060 second and 30 loads per minute were applied to the specimen.

The deformation that occurred when each load was applied was monitored with an LVDT and recorded on a strip recorder.

The repeated load testing sequence for each specimen consisted of the following:

1. Apply a confining pressure = 1.2 kg/cm^2 (17 psi).
2. Apply 1,000 applications of a deviator stress, σ_D = 2.4 kg/cm^2 (34 psi) i.e. σ_1 =3.6 kg/cm^2 (51 psi) and σ_1/σ_3 = 3 and monitor the resilient or elastic axial deformation at various intervals. The resilient axial deformation was converted to a Resilient Modulus, E_R, defined as the applied deviator stress divided by the resilient axial strain.

3. At N = 1,000, the stress-dependent behavior of the Resilient Modulus was determined for a range in θ (defined as $\theta = \sigma_1 + 2\sigma_3$) from 0.7 to 6.6 kg/cm² (10 to 93 1/2 psi). Upon completion this, σ_1/σ_3 was reset to 3 (i.e. 51/17) and additional loads applied.
4. At N = 5,000; 50,000; and 100,000 the resilient deformation was again monitored.
5. At N = 100,000 Step 3 was repeated and the influence of θ on E_R was again ascertained.

Following this testing sequence, each specimen was removed from the triaxial cell and a wet sieve analysis conducted to determine degradation during the repeated load testing.

Results of Repetitive Loading

The results from the repeated load triaxial testing of the samples are summarized in Figures A8.1 and A8.2 in the appendix to this chapter.

The data presented in Figure A8.1 summarize the relation between resilient modulus, E_R, and number of axial load applications, N, for each of the samples at an applied axial stress of $\sigma_1 = 3.6$ kg/cm² (51 psi) and a constant confining pressure of $\sigma_3 = 1.2$ kg/cm² (17 psi) ($\sigma_1/\sigma_3 = 3$).

Data presented in Figure A8.2 of the appendix to this chapter summarize the effect of total principal stress, θ ($\theta = \sigma_1 + 2\sigma_2$ for triaxial cell) on the resilient behavior of the samples. The range of σ_1 for these tests was 0.28 to 4.2 kg/cm² (4 to 60 psi), and σ_3 ranged 0.133 to 1.2 kg/cm² (2 to 17 psi).

It should be pointed out that the first repeated load test specimen made from Sample 1A failed in a classical shear failure after about 5,000 load applications. At the time of failure, σ_3 was equal to 1.2 kg/cm² (17 psi) and σ_1 was equal to 4.8 kg/cm² (68 psi). Therefore, a new specimen was made and during subsequent testing, the maximum value of σ_1 was limited to about 4.2 kg/cm² (60 psi).

Also, the repeated load test specimen for Sample 2A displayed a classical shear failure at N = 200 with $\sigma_1 = 3.6$ kg/cm² (51 psi) and $\sigma_3 = 1.2$ kg/cm² (17 psi). It was decided not to try another specimen from Sample 2A, since this specimen had failed at a principal stress ratio of 3.

EFFECT OF LONG TERM LOADING ON RESILIENCE:

The results from the repeated load testing of the concretionary gravels reveal that, in general, the materials display a slight increase in resilient modulus, E_R, with an increased number of axial stress applications. Sample 1A was an exception however, since it showed an increased E_R until about N = 1,000 then a slight reduction thereafter.

However, the E_R value for Sample 1A at N = 100,000 was only about 13% less than the peak E_R value of 1.209 kg/cm² (17.2 psi) at N = 1000.

Samples 1B and 4 displayed E_R values at N = 100,000 about 50-60 percent higher than those at the beginning of the test (i.e. at N = 1). The reason for Sample 1A not showing a similar strengthening trend is not known. The increased E_R values exhibited by the remaining samples from the beginning of each test to N = 100,000 averaged about 32 percent with a range of 22.5 to 38.8 percent.

STRESS DEPENDENT BEHAVIOR:

The resilient modulus of most high quality unbound granular materials has been found to be stress dependent. By this is meant that E_R is not constant, but rather is dependent on the state of stresses that is imposed on the material.

A convenient method of expressing the combined axial and confining pressures that are imposed on a triaxial specimen is by assuming the principal stresses.

The term θ is often used to denote this sum and is defined as $\theta = \sigma_1 + 2\sigma_3$ for a triaxial test.

The general response of high quality granular materials is such that higher E_R values are related to higher θ values.

Figure A8.2 in the appendix shows the general trend of E_R as a function of θ for the nine gravel samples at N = 1,000 and N = 100,000 load applications. Analysis of the data shows that the trend between E_R and θ for these six samples is inconsistent. E_R may either increase, remain fairly constant, or decrease with an increase in θ depending on the particular gravel sample and number of load applications.

Of significance is a comparison of the $E_R - \theta$ data for each of the gravel materials with the response of typical high quality granular materials. Typical $E_R - \theta$ relations for a gravel (Reference 1) and a crushed granite with high fines content (Reference 2) have been plotted on Figure A8.2. It can be noted that E_R of these high quality granular materials is greatly influenced by θ. A comparison of the $E_R - \theta$ results for the nine samples to these high quality granular materials indicates the following.

1. The E_R values of samples are not as greatly affected by θ as are other quality granular materials.
2. In general, the E_R values displayed by these nine samples are substantially less for comparable values of θ than higher quality granular materials.
3. On the other hand, there is some correspondence between E_R (when measured at approximately 100,000 load repetitions) and the qualitative rating of the materials (Figure A8.3).

Sample 1B (Rio Taborda) rated as good, has consistent E_R values below 1406 kg/cm² (20 ksi).

Sample 4 (Cabo Branco) rated as fair to good, has E_R values ranging from about 914 to 1336 kg/cm² (13 to 19 ksi) with most in the range of 1055 to 1336 kg/cm² (15 to 19 ksi).

Sample 1A (Peritoro) rated fair, shows E_R values ranging from about 773 to 1266 kg/cm² (11 to 18 ksi), but all but one are below 1055 kg/cm² (15 ksi).

Sample 2A (Ceará No. 2) rated poor, demonstrated its lack of quality by failing after about 200 load repetitions.

Sample 5 (Paraíba) rated good, E_R range is 1336 to 2030 kg/cm² (19 to 29 ksi), with most in the range 1547 to 1687 kg/cm² (22 to 24 ksi).

Sample 6 Piauí No. 1) rated good, E_R range is 1336 to 1828 kg/cm² (19 to 26 ksi), most in the range 1406 to 1547 kg/cm² (20 to 22 ksi).

Sample 11 (Goiás) rated fair to good, E_R range is 1406 to 1758 kg/cm² (20 to 25 ksi), with most in the range 1406 to 1617 kg/cm² (20 to 23 ksi).

Sample 9 (Brasília) rated fair, E_R range is 1265 to 1687 kg/cm² (18 to 24 ksi), most in the range 1406 to 1617 kg/cm² (20 to 23 ksi).

Sample 7 (Piauí No. 2) rated fair, E_R range is 914 to 1546 kg/cm² (13 to 22 ksi), most in the range 1125 to 1406 kg/cm² (16 to 20 ksi).

Sample 2B (Pará) rated poor, E_R range is 1054 to 1687 kg/cm² (15 to 24 ksi), most in the range 1054 to 1336 kg/cm² (15 to 19 ksi).

CONCLUSIONS

The durability of ironstones and concretionary gravels were evaluated through several tests and compared with the qualitative ratings provided by local engineers who have experience with these materials in highway construction. Samples displaying performance characteristics ranging from poor to good were selected from each of the major soil groups.

Results of this study indicate that the Los Angeles abrasion and aggregate impact tests do not provide a means of distinguishing between those structural layer materials that perform adequately and those that do not. The hardness index is not sufficiently discriminatory. No correlation could be established between breakdown resulting from compaction and aggregate quality. In fact evidence indicates that the breakdown may actually improve the gradations in some materials.

The slake-durability is tentatively recommended to evaluate durability. The test is relatively simple and the required equipment is not particularly expensive. A minimum slake-durability value of 97 appears to include only high quality material while values above 94 exclude the poor materials.

Lateritic soils which are sufficiently well developed and of good enough quality to be considered for use in airfields or highways, exhibit mechanical durability properties which are comparable to those of non-lateritic materials. Therefore, the D_c and D_f test procedures correlate well with the observed performance of the materials tested, as well as with numerous other lateritic and non-lateritic materials.

The findings of this study of natural laterite aggregates indicate that it is not worthwhile to carry on further durability studies with repetitive load tests in order to measure the relative amount of breakdown in materials. The lack of definite results is due, at least in part, to the fact that identical samples cannot be prepared for comparative purposes. However, the tests show that the degradation which does occur is primarily the result of the compaction procedure and that little or no further breakdown occurs during repetitive loading.

The only conclusions that could be drawn from the tests of ten ironstone and concretionary gravel samples without any replicates is that repetitive triaxial load tests carried to a large number of load repetitions (N = 100,000 or more) hold some promise as a basis of classifying laterite aggregate as to quality.

If E_R is determined at several values of θ the following preliminary classification is indicated.

E_R above 1476 kg/cm² (21 ksi) material performance should be good.

E_R range 1336 to 1476 kg/cm² (19 to 21 ksi) material performance should be fair to good.

E_R below 1336 kg/cm² (19 ksi) material performance should be poor to fair.

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CHAPTER 9

STABILIZATION OF SELECTED LATERITIC GRAVELS AND SOILS

INTRODUCTION

In countries where economic development is based primarily on agriculture, it is important to have a complete network of major highways, secondary roads, and low-cost feeder roads. Economy requires the maximum use of local materials in the construction of these roads. Earthwork is a major road construction cost item and local lateritic soils are a potentially excellent cost saving material. However, experience has shown that the use of untreated lateritic materials have presented many problems in road construction and maintenance in the past. Consequently, for the three decades stabilization of lateritic materials by admixtures has been practiced and studied in tropical countries.

The purpose of stabilizing a soil is to alter its physical properties, increase its strength and increase its durability in order to provide a satisfactory foundation material. The admixtures most commonly used today in road construction throughout the world are cement, lime, asphalt and sand. Cement appears to be the most common additive used to date in Africa and South America. Lime has been used as well, but not as commonly as cement. The use of asphalt has been limited to lateritic soils of a sandy nature. Stabilization by admixture of sand has been investigated and successfully used in locations where sand is readily available such as in coastal regions.

Scope

To evaluate the use of lime, cement and asphalt as stabilizers for lateritic materials, a series of studies was performed. A total of 20 representative soil materials were selected for this purpose from Africa, 15 samples from South America and 8 samples from Southeast Asia. Four samples from Africa were sent to the Asphalt Institute Laboratories in College Park, Maryland for asphalt stabilization testing.

A quick and simple method has been devised by Eades and Grim (1966) for determining lime and cement requirements for soil stabilization. It was adopted for the study in Africa to investigate its applicability for lateritic soils. It was later modified for the South American study.

The results from the African study have been taken largely from the Lyon Associates Report (LAI, 1971) while the results from Southeast Asia have been taken largely from the report by Vallerga et al. (1969). The results of the South America tests are reported, with general conclusions, in the final sections of this chapter.

Materials

Three soils from each of the lateritic soil groups were selected from various borrow pits in Africa and Brazil. The locations are shown in Figures 9.1 and 9.2 respectively. The two Congo soils were selected from sandy materials that were being assessed for use as a base course in a highway design study. A clayey sample from Ivory Coast was selected as a subgrade-type material. All other materials were selected from convenient borrow pits in countries designated in Figure 9.1.

The lime and cement used in the study were all obtained from local sources when possible. The cement, type I Portland cement, was made locally and met ASTM specifications for mortar strength and setting time. Lime is not produced locally in Ghana so an imported brand was used for the lime stabilization in that country. Locally produced lime was used in Brazil and Thailand.

STABILIZATION STUDIES IN AFRICA

Lime and Cement Stabilization

Air dried soils were used in all of the stabilization tests. The flow chart for the lime and cement tests is presented in Figure 9.3. The physical and chemical properties of the tested materials are listed in Table 9.1.

COMPACTION

Moisture-density relationships were investigated in accordance with ASTM D1557-70 Method B. Recent studies (Arman and Saifan, 1967) have shown that the density and strength show a considerable decrease if there is a delay between mixing and compaction. This decrease may occur if the delay is longer than 2 hours for soil-cement mixtures and 24 hours for lime-soil mixtures. Therefore, it was decided to compact specimens containing either stabilizer immediately after mixing.

The required amount of stabilizer was blended with the soil in a mechanical mixer. Water was then added and thoroughly mixed with the soil and stabilizer in the mechanical mixer. Immediately after mixing, the soil was compacted in a mold. All excess material was sealed until needed for compaction. After being weighed, the sample was extracted from the mold, placed on a pallet, sealed in a plastic bag and then placed in the curing room. Water was added to each bag to maintain approximately 100 percent relative humidity.

CBR

After determination of the optimum moisture content (OMC) for each stabilizer content, two CBR samples were prepared in accordance with ASTM D 1557-70 Method D at the optimum moisture content. The samples were placed on a layer of moist sand in a storage box. Moist sand was then placed in the collar extension of the molds. One sample was cured for 7 days and the other for 28 days. After their respective curing periods, the molds were taken from the curing room and soaked in water for 4 days under a 4.5 kg (10 pound) surcharge weight. Readings were taken before and after soaking to determine swell. After the soaking period the CBR's were determined in an automatic testing machine at a strain rate of 1.3 mm per minute (0.05 inches per minute).

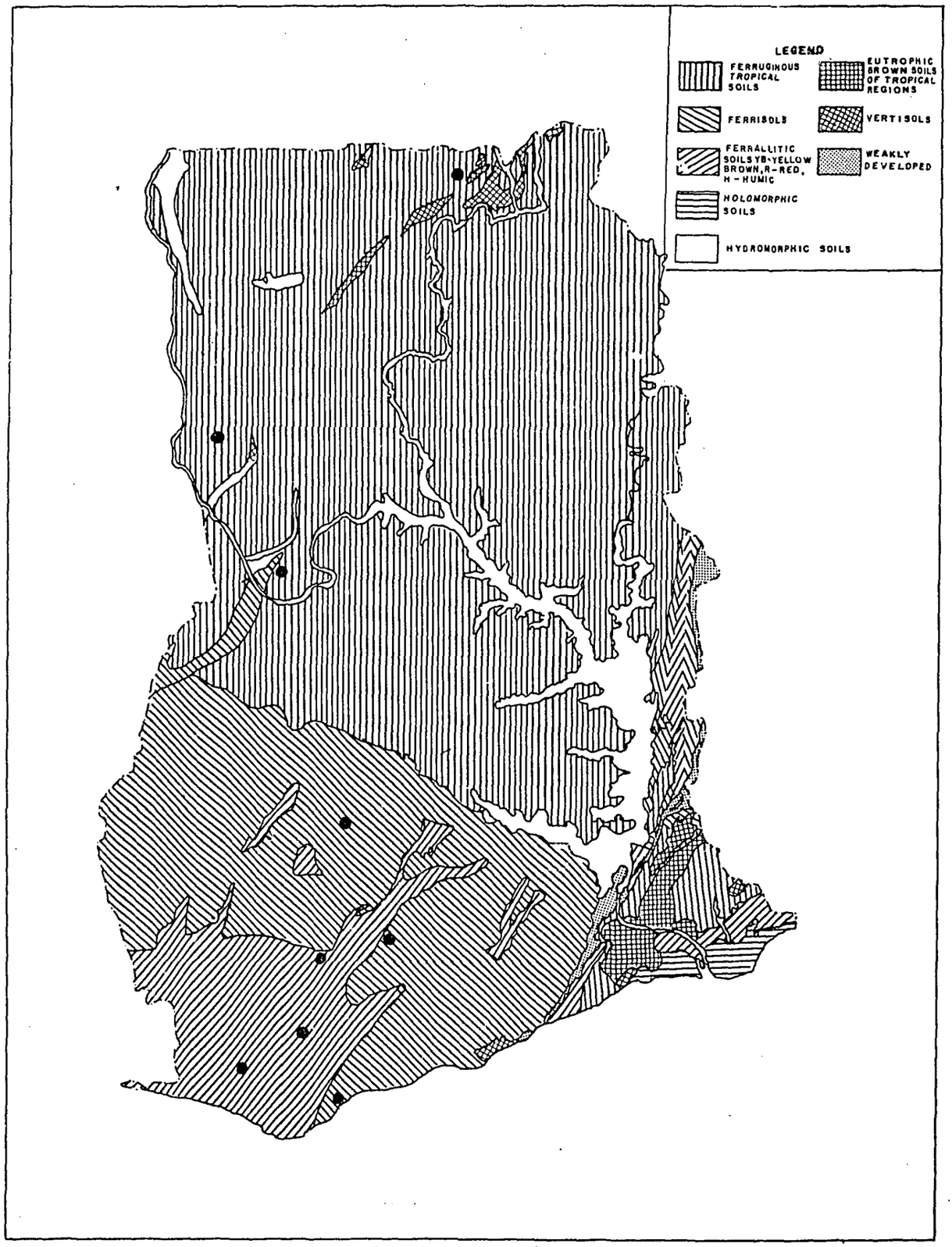


FIGURE 9.1 – LOCATION OF STABILIZATION SAMPLES IN GHANA

FIGURE 9.2 – LOCATION OF STABILIZATION SAMPLES IN BRAZIL

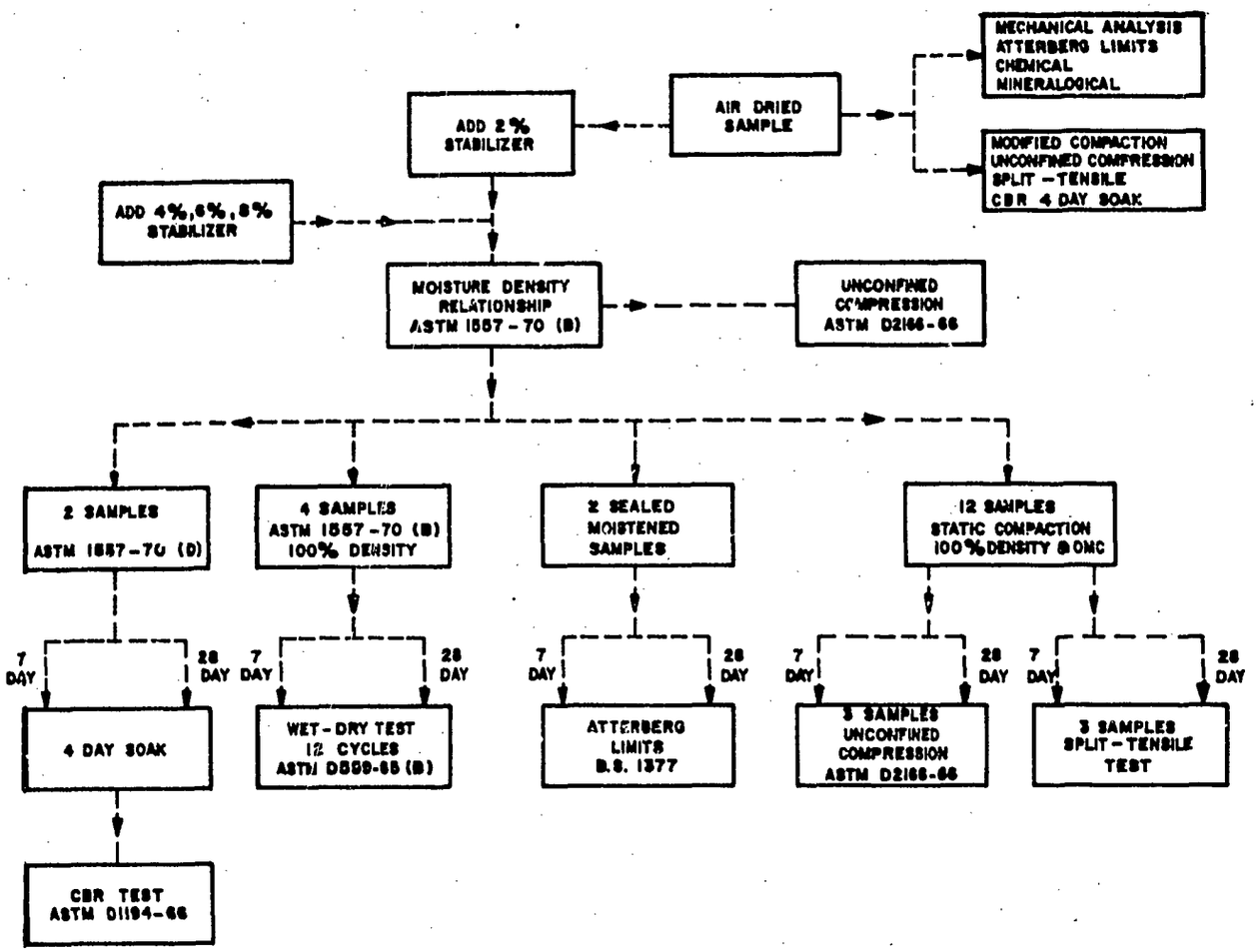


FIGURE 9.3 – FLOW CHART FOR LIME AND CEMENT STABILIZATION IN AFRICA

TABLE 9.1

Properties of Soils Investigated in Africa

Properties	G5-1	G9-3	G1-14	G12-2	G2-1	G8-1	G8-2	G8-3	G12-1	IV-5	K-4	U-4	CON-1	CON-2	DAH-3	NIA-1	Tormabum	Freetown	Kabala	Batkanu
D'HOORE CLASS.	BcJd	Jc	JdBc	kc	kc	kc	Lx	Lx	Lc	Lm	Ja	BcLm	La	La	Li	Ja	Lc	Lm	Lc	LcNa
GRAVEL	45	27	30	43	35	43	45	53	63	0	8	51	0	0	15	36	0	16	39	50
SAND	35	53	60	37	35	37	24	24	14	31	46	25	53	72	46	39	76	35	33	29
SILT	12	12	5	9	10	6	15	8	16	23	16	8	11	8	12	11	10	16	14	8
CLAY (0.002 mm)	8	8	5	11	20	14	16	15	7	46	30	16	36	20	27	14	14	33	14	13
LL %	29	33	23	42	44	38	55	54	48	62	44	36	38	21	45	21	24	59	64	56
PL %	18	17	12	20	23	21	32	27	26	31	15	21	19	12	21	11	14	39	45	38
PI	11	16	11	22	21	17	23	27	22	31	29	15	19	9	24	10	10	20	19	18
SPEC. GRAVITY[a]	2.87	2.98	2.90	2.92	2.85	2.90	2.91	2.78	2.92	2.83	2.85	2.75	2.66	2.68	2.54	2.58				
MDD g/cm³ (pcf)	2.18 (136)	2.15 (134)	2.24 (140)	2.16 (135.0)	2.04 (127)	2.18 (136)	2.00 (125)	2.02 (126)	2.05 (128)	1.73 (108)	1.96 (122)	2.00 (125)	1.91 (119)	2.07 (129)	1.81 (113)	2.15 (134)	2.08 (130)	1.70 (106)	1.87 (116)	2.10 (131)
OMC[b] %	8.9	9.1	7.2	10.0	13.5	8.2	11.0	10.2	11.4	18.7	12.2	12.3	13.6	9.2	16.6	8.7	9.2	25.0	15.0	13.6
AASHO CLASS.	A-2-4	A-2-6	A-2-4	A-2-7	A-2-7	A-2-7	A-2-7	A-2-7	A-2-7	A-7-5	A-7-6	A-2-7	A-6	A-2-4	A-7-6	A-2-4	A-2-4	A-7-5	A-2-7	A-2-7(7)
GROUP INDEX	(0)	(0)	(0)	(1)	(2)	(1)	(1)	(1)	(1)	(19)	(8)	(2)	(5)	(0)	(4)	(4)	(7)	(7)	(1)	(0)
pH	6.5	6.8	6.5	6.2	4.5	6.3	4.5	4.7	4.7		4.6	5.3	5.1		5.8	5.0				
CATION EXC. CAPACITY Meq/100 gm.	7.23	8.11	4.37	8.81	9.07	9.63	9.63	7.23	5.94	4.43	7.71	7.94	4.32	1.74						

a. On minus No. 10 Fraction
b. AASHO Modified

WET-DRY TESTS

Samples for wet-dry tests were prepared and tested in accordance with ASTM D1559-65 Method B. Two samples each were prepared for a 7-day and 28-day curing period. After extraction from the molds, each sample was placed on a pallet, sealed in a plastic bag and then placed in the curing room. Water was added to each bag to maintain approximately 100 percent relative humidity. Both samples of a pair were subjected to 12 cycles of wetting and drying; one sample was used for the brushing test and the other was used to measure volume change.

UNCONFINED COMPRESSION AND SPLIT-TENSILE TESTS

Past research (Thompson, 1965) has shown that the split-tensile strength correlates closely with unconfined compressive strength for lime-soil mixtures. To determine if this correlation would be similar for lateritic stabilized soils, split-tensile tests were performed along with unconfined compressive tests.

All specimens were prepared in 5 by 10 cm (2 by 4 in) molds by static compaction to the modified AASHO maximum density at optimum moisture content for each stabilizer content.

Twelve samples were prepared for each stabilizer content: three specimens each for unconfined and split-tensile tests at 7 and 28 day curing periods. The specimens were placed on a pallet, sealed in a plastic bag and then placed in the curing room for their respective curing periods. Water was added to each bag to maintain approximately 100 percent relative humidity. Loading strips 6.4 mm (0.25 in) wide, which gave a width to diameter ratio of 1/8, were used for the split-tensile test. Both tests were conducted at a strain rate of 1.3 mm per minute (0.05 inches per minute).

ATTERBERG LIMITS

Two samples were prepared at each stabilizer content and sealed in plastic bags. One sample prepared for the 7 day and the other for the 28 day curing periods. The samples were prepared at moisture contents close to their plastic limit. Water contents were checked during the curing periods. Water was added when necessary to allow proper action between the stabilizer and soil. A minimum of three plastic limit determinations were made for each sample. Liquid limits were determined in accordance with British Standard 1377.

All samples were cured at approximately 100 percent relative humidity at temperatures between 21°and 23° C.

pH METHOD

The pH determination for lime and cement requirements was performed at the University of Illinois and the University of Florida. The test measures the amount of cement or lime used up by a soil after one hour. The pH of cement or lime soil mixtures is measured to determine the minimum percent of stabilizer required to attain a pH of 12.40 for lime mixtures and 12.20 for cement mixtures.

Results of Lime and Cement Stabilization

COMPACTION

The change in maximum density and OMC for the soils stabilized with lime and cement are similar to those reported for soils from temperate zones and other lateritic soils. The maximum density decreased with increases in lime content. With the addition of 2 percent lime, the maximum density decreased 0.016 to 0.096 g/cm^3 (1 to 6 pcf), with additional percentages of lime the maximum densities decreased further, in some cases 0.128 g/cm^3 (8 pcf) less than the maximum density of the untreated soil. The OMC increased from 1 to 2 percent with 2 percent lime, but for most of the soils further addition of lime produced little increase in the OMC.

The changes in maximum density and OMC for soil-cement were not as great as those for lime. The most significant change took place with the addition of 2 percent cement, but with increasing cement contents the changes were small. Except for three soils (G9-3, G12-2, Kabala), the maximum densities and OMC did not vary by more than 0.048 g/cm^3 (3 pcf) and 2 percent in moisture content.

The maximum density-optimum moisture relationships for all of the soils tested are tabulated in the appendix to this chapter.

ATTERBERG LIMITS

The results of the Atterberg limit tests are shown in the appendix to this chapter and the change in the plasticity indexes for selected tests are given in Figures 9.4 through 9.7.

Almost all research on lime stabilization has indicated that lime alters the plasticity of the soil. The addition of lime reduces the plasticity index. The amount of reduction varies directly with the quantity of lime added and with the type of clay mineral and organic matter present in the soil. The liquid limit normally decreases with increasing amounts of lime (Herrin & Mitchell, 1961), although the converse has also been reported for some soils (Diamond & Kinter, 1965).

In the tests performed for this study, almost all the lateritic samples tested experienced a decrease in liquid limit with the addition of lime. Only two soils showed an increase of their liquid limit these were soils which had formed over sandy parent material. However, the increase of the liquid limit was not as great as the accompanying increase in plastic limit and the plasticity index decreased accordingly. Increases in the liquid limit occurred in the majority of soils stabilized with cement.

Clare and Cruchley (1957) reported that an increase in the liquid limit of a soil stabilized with lime may be due to clacium saturation of the clay particles. This explanation could apply to the lateritic soils studied. The phenomenon may be due to flocculation and cementing of the individual particles, whereupon the soil becomes silty and stiffens so

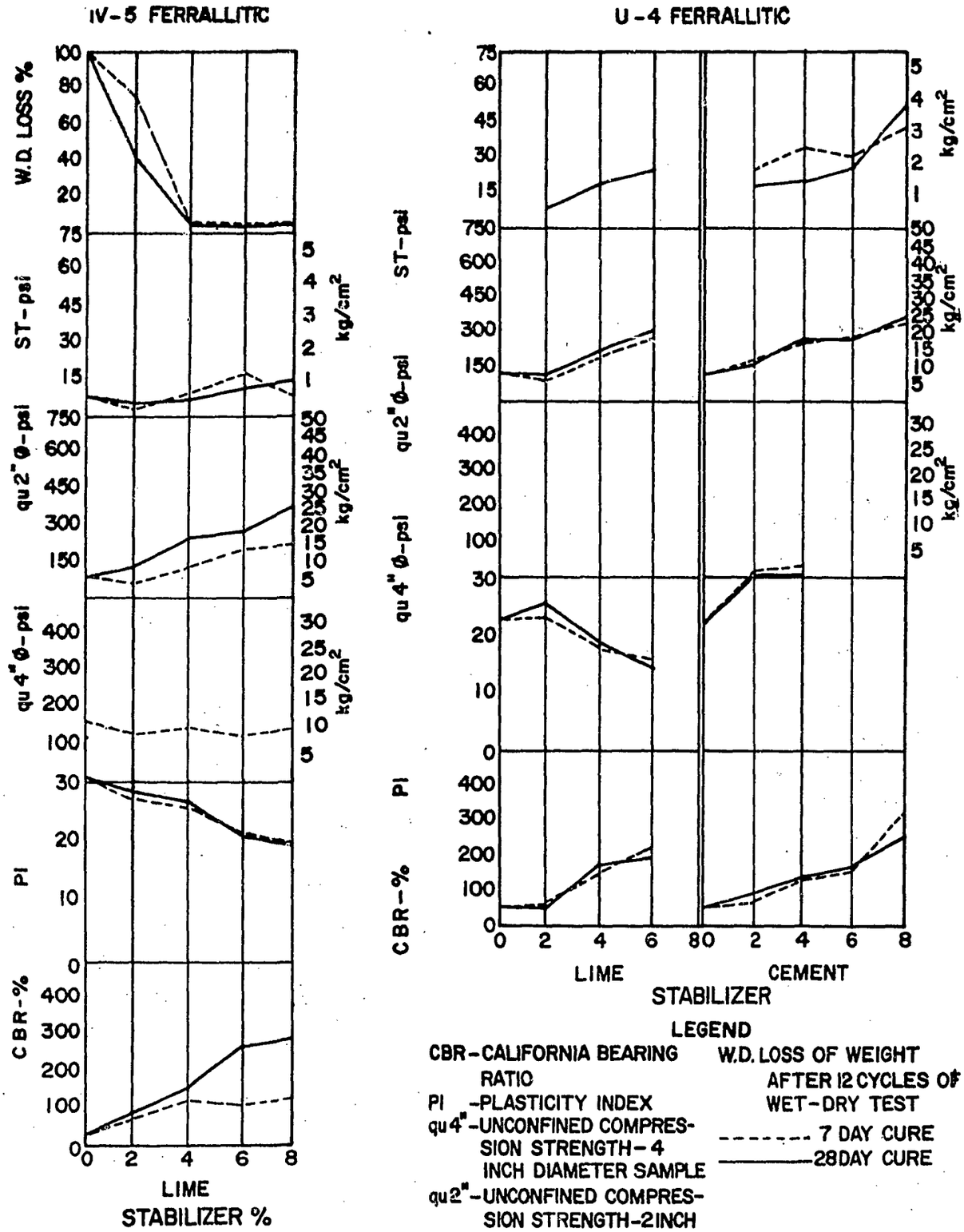


FIGURE 9.4 – STABILIZATION TEST RESULTS, AFRICA

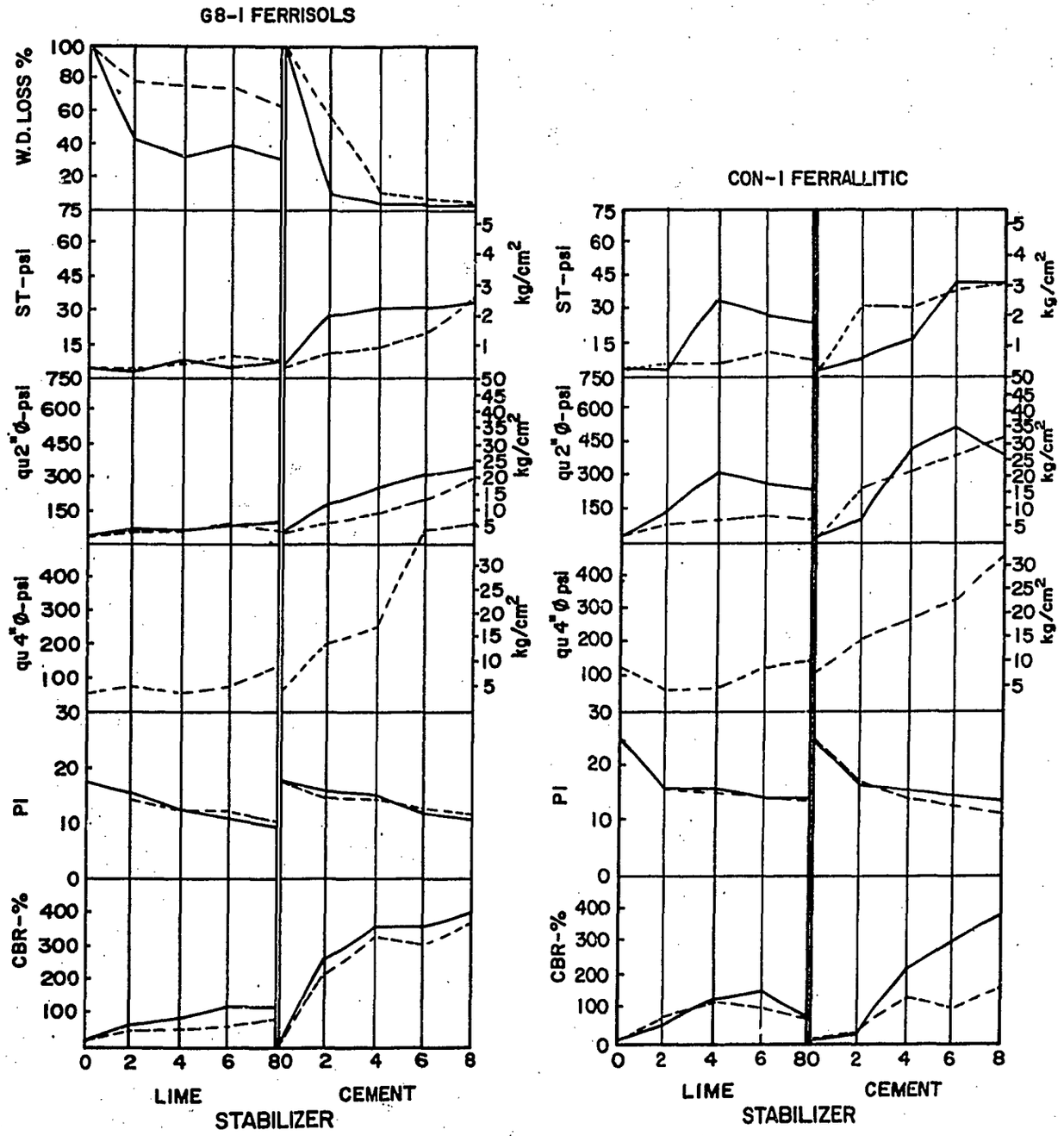


FIGURE 9.5 – STABILIZATION TEST RESULTS, AFRICA

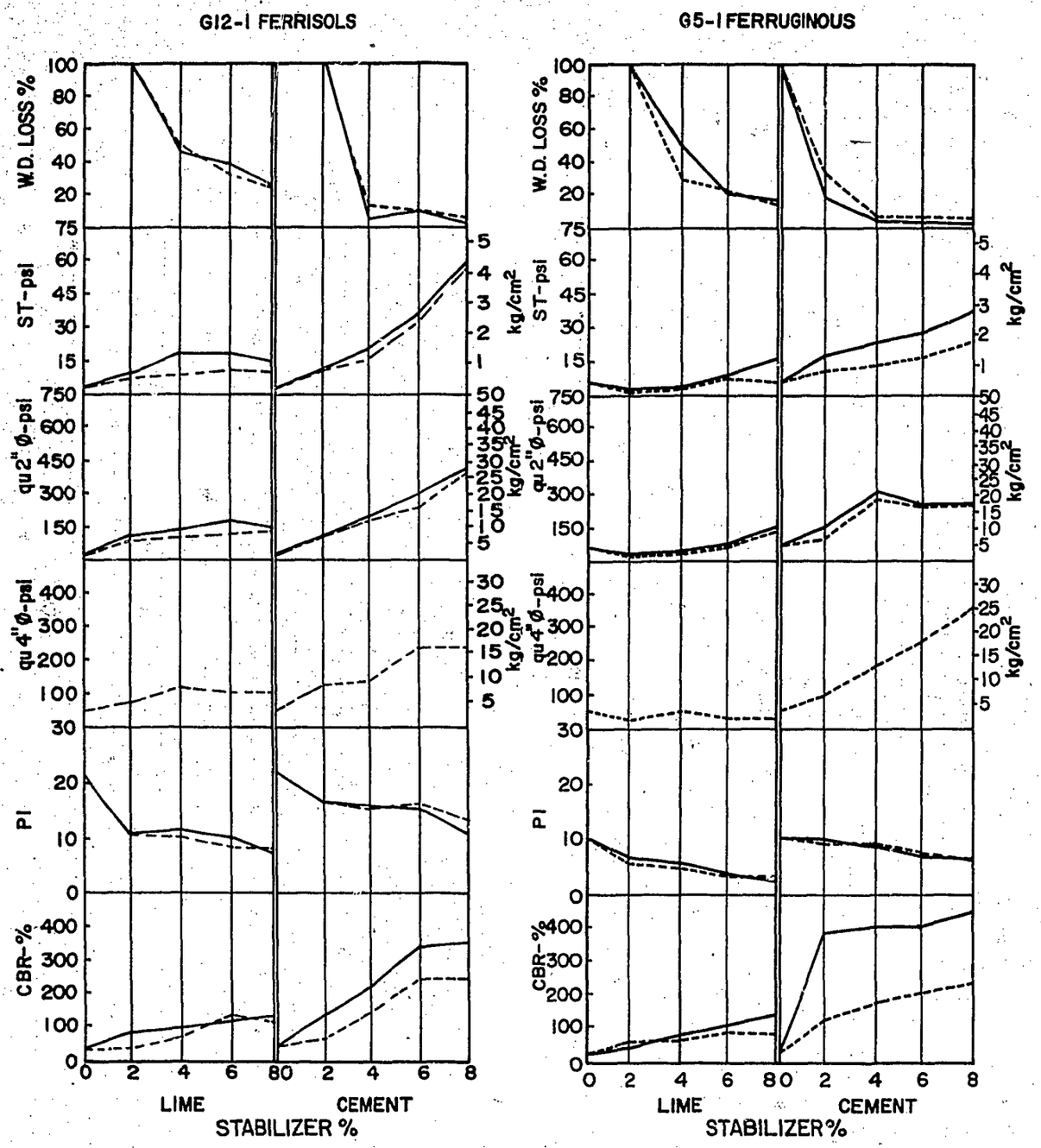


FIGURE 9.6 — STABILIZATION TEST RESULTS, AFRICA

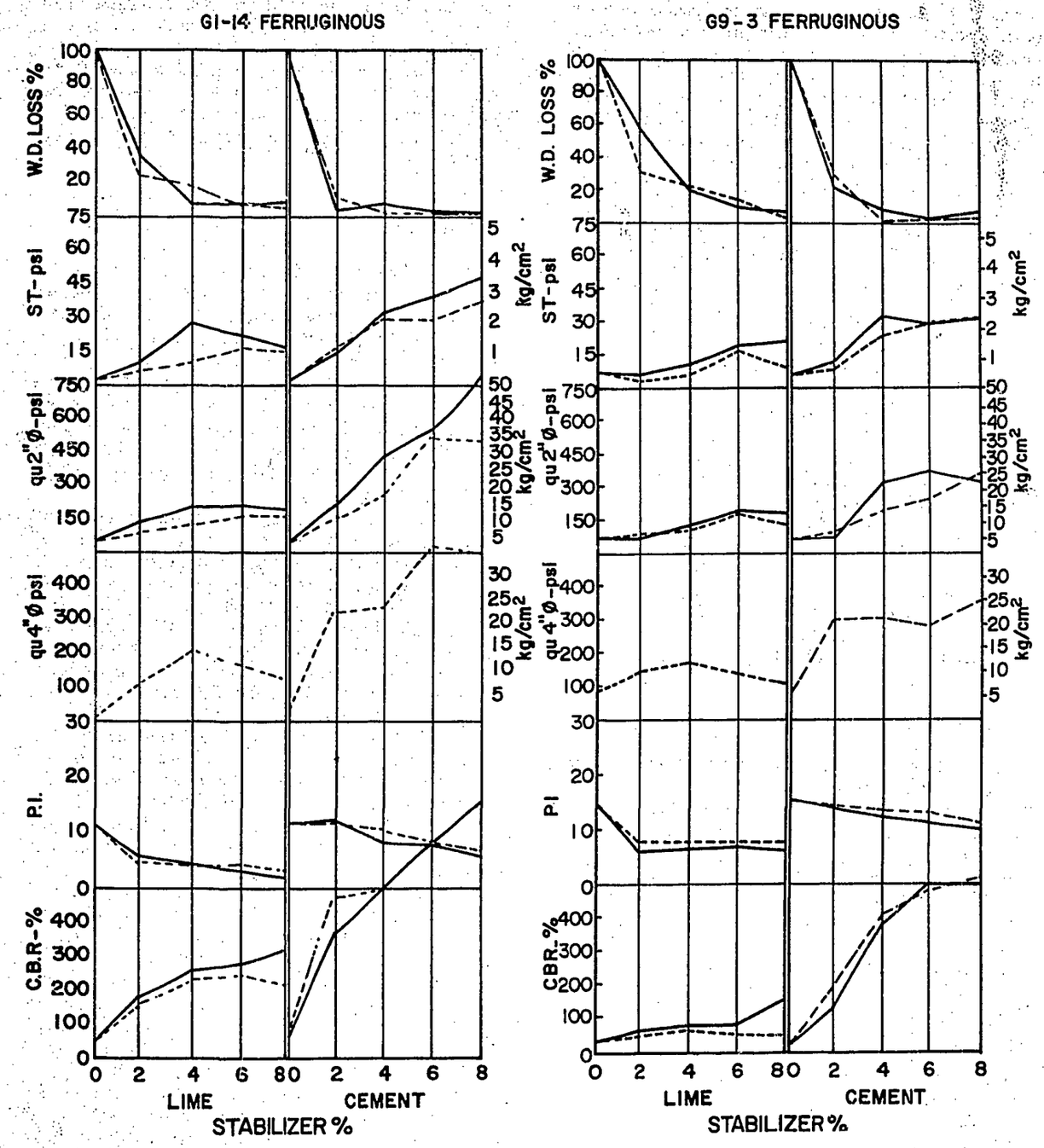


FIGURE 9.7 – STABILIZATION TEST RESULTS, AFRICA

that it flows with greater difficulty in the liquid limit cup.

There was very little difference in liquid limit between the 7 and 28 day curing periods for lime or cement-stabilized soils.

In almost all cases where comparative data were obtained, a greater reduction in the plasticity index occurred with the addition of lime than with cement. This finding is in agreement with previous research with both types of stabilizers. Lime in comparison with an equal weight of cement has more calcium available for cationic exchange. This reaction will alter the water holding capacity of the clay to form a less plastic soils.

The plasticity index of the untreated soils ranged from 10 to 34. The kabala soil from Sierra Leone was the only one to attain a non-plastic state. All the other materials tested showed a decrease of the plasticity index to less than 15, except soil IV-5 which exhibited a reduction from 31 to only 20 with 8 percent lime.

The ferruginous soils and soils derived from sandy parent materials all showed a great reduction in the plasticity index with the addition of 2 percent lime. Higher percentages of lime resulted in lesser reductions. The other soils showed a steady reduction in the plasticity index with increasing lime content. All cement treated materials showed a steady reduction.

In previous research, Cartmell and Berch (1958) showed for a Zambian "laterite" the steady reduction from an original plasticity index of 19 with increasing quantities of lime to zero at 5 percent lime. Hayter and Cairns (1966) showed that the plasticity index of a Nigerian lateritic soils with an initial value of 29 only decreased to 14 with the addition of up to 8 percent lime. Wiggington (1967) reported a reduction of the plasticity index from 20 to 10 with 2 percent lime. However, higher percentages of lime did not reduce the plasticity index to less than 9.

WET-DRY DURABILITY

In most tropical countries the durability requirements of soil-cement mixtures are assumed to be satisfactory if the stabilized soil is in conformance with their minimum strength requirements; a minimum value of 17 kg/cm^2 (250 psi) is often applied. Moh et al., (1967) discussed some soils in Thailand, stabilized with cement, for which the 7-day compressive strength was 10 kg/cm^2 (150 psi) which possessed adequate stability to withstand the wet-dry durability test. The assessment of the wet-dry durability properties of lime-soil mixtures by means of laboratory tests is difficult. Results from standard wet-dry tests using brushed samples when compared to criteria developed for soil-cement mixtures, indicate that lime-soil mixtures have little resistance to weathering. However, field research has shown that lime-soil mixtures when protected by a bituminous wearing surface are more durable than indicated by most laboratory durability tests (Herrin and Mitchell, 1961).

Nevertheless, in the study of durability all of the samples were subjected to the wet-dry test. The results of the study are tabulated in Table A9.1 in the appendix to this chapter and shown in Figures 9.4 through 9.7. Based on the Portland Cement Association (PCA) criteria of 14 percent, 10 percent or 7 percent loss of weight depending upon the AASHO soil classification, the experimental data shows that lateritic gravels can be made durable with the addition of 4 percent cement. One soil, G8-2, required 6 percent cement to meet the criterion. The lime-soil mixtures had a greater loss of weight when compared to the soil-cement mixtures. Even with 8 percent lime, most of the soils did not meet the PCA criteria.

The volumetric-change specimens that were molded for each test condition and stabilized did not experience any extensive volume or length changes. These specimens were subjected to the 12 cycles of wetting and drying without brushing.

STRENGTH

The California Bearing Ratio (CBR) test and the unconfined compression tests are the most commmon procedures used to assess strength gains in stabilized soils. These have been extensively reported in research on temperate and tropical soils and are used as design parameters in most African countries. However, the tensile strength characteristics of base courses, subbases and subgrades are also important considerations in pavement design. Thompson (1965) and Kennedy and Hudson (1968) have shown that the split-tensile tests can be used to evaluate stabilized materials.

CBR: There did not seem to be any distinction in CBR values for the lime-treated samples of the various soil groups. There was some distinction in CBR values for the cement-treated soils, as will be discussed subsequently. The CBR values for cement treated soil were much higher than for lime treated soil. In most cases the CBR value of the lime treated samples were less than the 28 day CBR for 2 percent cement treated samples. In no case was the soil-lime CBR values more than the 28 day CBR value for 4 percent cement treated samples.

The CBR value for most of the lime treated soils increased linearly with time for the lower lime contents, but with higher lime contents the increases were smaller and even a slight decrease in strength was noted. The highest CBR values were attained with contents of 4 to 6 percent lime. There were only small increases in strength between samples cured at 7 and 28 days.

The low increase in strength may be due to the lack of pozzolanic material in the natural soil. Usually the aluminous and siliceous materials in soil react with the lime to form a cementation of the soil particles. Lateritic soils are usually low in siliceous materials, which might explain the low increase in strength with longer curing periods. Lime is known to react more quickly with montmorillonitic clays than with kaolinitic clays that are the most common in lateritic soils. The difference in reaction time has been reported to be a few weeks (Ingles and Metcalf, 1972). Herrin and Mitchell (1961) state that neither base exchange capacity, the pH of the soil, nor the plasticity characteristics seem to be satisfactory indicators as to whether or not a soil will gain high strengths with the addition of lime. Further research may be necessary to determine whether the addition of a pozzolan, such as fly ash, might be helpful to aid the cementing process in lateritic soils. Irwin (1958)

found that after 26 weeks of curing, lateritic gravels from Uganda stabilized with 4 percent of lime were approaching the 7 day strength of the same gravels stabilized with 4 percent of cement. Hayter and Cairns (1966) reported a very marked gain in strength between 7 and 28 days for a lime treated lateritic soil. They did not give any explanation for this reaction. The long time required for the gain in strength is due to the slow cementing process associated with lime. The time may be accelerated with the addition of pozzolans.

Most of the cement-treated soils in the study developed high CBR values with low percentages of cement. The values generally increased linearly with increase of cement content, although some soils exhibited smaller increases at 4 to 6 percent. Sherwood and Sullivan (1960) observed smaller increases when testing a Gambian lateritic gravel at optimum moisture content. They showed that the decreased rate was due to insufficient water available for complete hydration of the larger quantities of cement. When the gravel was tested at higher moisture contents, the unconfined compressive strength increased linearly with an increase in cement content.

For all cement treated soils except one, the CBR values were over 200 percent. The exception was a ferrallitic sandy clay from Freetown which had a CBR value of 137 percent at 4 percent cement content. The three gravels from the ferruginous group all had CBR values of over 400 percent at 4 percent cement content. Some of the soils from the ferrisol and ferrallitic groups also attained very high CBR values at the higher cement contents. Generally the test was stopped when CBR values exceeded 500; values greater than 100 have little meaning.

Many countries in Africa use, as design criteria, a minimum CBR value of 180 percent for cement and lime treated soils when tested at 100 percent modified AASHO compaction. O'Reilly and Millard (1969) recommended a CBR value of 80-100 percent at the density to which the soil will be compacted in the field. This requirement is for tests conducted after a 7 day period, comprising 3 days moist curing followed by 4 days immersion in water.

Unconfined Compression: Unconfined compression tests were performed on the extracted 10.2 by 11.4 cm (4 by 4.5 in) specimens. The compressive strengths of each of the samples were plotted against the molded moisture content. The strength corresponding to the OMC was selected as the unconfined compression strength of the sample. Figure 9.8 shows a typical compaction curve with corresponding unconfined compressive strengths. It will be observed that a small variation in the OMC causes a considerable change in the compressive strength.

The 5 cm (2 in) diameter cement treated samples had higher unconfined compressive strengths than those for lime treated soils.

Most of the cement-treated soils had strength gains in proportion to the cement content. Except for the Batkanu soil from Sierra Leone, all soils attained a 28-day strength of 17 kg/cm² (250 psi) at 4 to 6 percent cement.

Most of the lime-treated soils did not attain 28-day strengths of 17 kg/cm² (250 psi) even at the higher lime contents. There was very little or no gain in s rength from 7 to 28 days. The one exception was IV-5 which showed gains from 50 to over 100 percent. The strengths generally

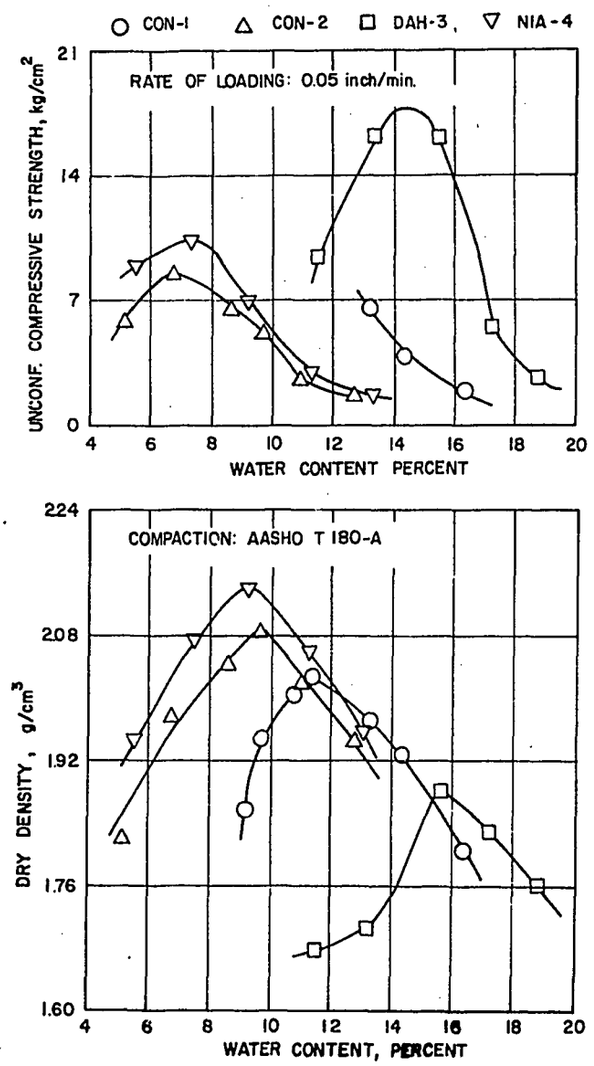


FIGURE 9.8 – DRY DENSITY AND UNCONFINED COMPRESSIVE STRENGTH OF LATERITE SOILS

increased linearly with an increase in lime content but some soils exhibited peak values at 4 to 6 percent and then decreased in strength at the higher lime contents.

A decrease in compressive strength at higher lime contents as well as complete failures with high percentages of lime were reported for lateritic soils by Winterkorn and Chandrasekharan (1951). Hayter and Cairns (1966) also reported little or no further gain in strength beyond a certain percentage, usually in the range of 4 to 7 percent.

No distinction could be made between the D'Hoore soil groups based on the unconfined compression test results, although this intuitively appears unreasonable. One would expect one group to be more reactive with lime than cement and the reverse with another group. Research by Eades (personal communication) has shown that the chemical reaction between lime and soil is time and temperature dependent. Therefore, there is an optimum lime percent for a given curing period and laboratory exercises are not necessarily indicative of true conditions in the field.

Split-Tensile Strength: The split-tensile test procedure (ASTM C496-64T) is primarily used to evaluate the tensile strength of concrete. However, it appears to be a practical method to evaluate the tensile strength of lime, cement and asphalt-treated materials.

Thompson (1965) reported that the ratio of split-tensile strength to compressive strength (ST/qu) showed small but significant differences for different soil types, and that the best estimate of the ST/qu ratio for lime soil mixture was 0.13, ranging from 0.113 to 0.155. The test results, Figures 9.9 and 9.10, show that for lime-treated soils the ST/qu ratio has an average value of 0.096 with a range of 0.041 to 0.153. The cement treated soils have an average value of 0.097 with a range of 0.040 to 0.164. The ST/qu ratios were similar for all soil groups. There were no major differences in the ST/qu ratio between the lime and cement treated soils. It is apparent that the changes in compressive and split-tensile strengths are similar for lime and cement within the range of additive contents and curing periods used in the program.

On the basis of the results of the study it appears that the split-tensile test has application in the evaluation of stabilization for lateritic soils. An ST/qu ratio of 0.10 for lime and cement treated mixtures is appropriate for tropical soils.

Strenght Relationships: The Road Research Laboratory (O'Reilly and Millard, 1969) established for five soil types a correlation between the CBR and unconfined compressive strength for soil-cement mixtures. A similar correlation has been investigated in this study for both lime and cement-treated samples. This is shown in Figure 9.11 for cement-soil mixtures and in Figure 9.12 for lime-soil mixtures. Medina (1964) reported the same relationship for laterite gravels from Ghana as the Road Research Laboratory did for concretionary gravels. He believes that the slopes of the lines differ from each other because of the absence of lateral pressure during the unconfined compression test whereas such pressure exists in the CBR test.

pH METHOD

The minimum stabilizer contents needed to maintain the required pH for the lime or cement mixtures are listed in Table 9.2. Also listed are the design contents determined from test results.

The design contents of the soil-cement mixtures were based on attaining a minimum CBR value of 180 and an unconfined compressive strength of 10 kg/cm^2 (150 psi). Most of the lime-soil mixtures exhibited peak values for the range of lime contents studied. The design lime content was considered to correspond to the peak value even if this value was lower than the minimum strength required for base materials.

TABLE 9.2

Design Content of Lime and Cement Stabilized Materials, Africa

Sample	Lime Design Content %		Cement Design Content %	
	By pH Method	By Test Method	By pH Method	By Test Method
G9-3	4	6	6	4
G1-14	4	4	6	3
5-1	4	6	6	3
12-2	4	8	6	6
2.1	4	5	6	3
8-1	4	6	9	4
8-2	4	6	6	5
8-3	4	4	6	3
12-1	4	4	6	3
IV-5	3	5	NA	NA
K-4	4	4	NA	NA
U-4	4	6	NA	NA
C-1	–	–	10	4
C-2	–	–	9	3

The pH method for lime-soil mixtures indicated design contents lower than those found by the unconfined compression test method. Most soils had a pH lime design content of 4 percent, while the test method (unconfined compression tests) showed from 4 to 6 percent. One soil, IV-5, had a pH-method content of 3 percent compared to a test method content of 4 percent. Four soils showed a design content of 4 percent for both materials.

For soil-cement mixtures, the pH method gave higher design contents than the unconfined compression test method. The pH method indicated contents of 6 percent with 2 soils at 9 percent and 1 at 10 percent. The test method indicated design contents from 2 to 4 percent with most soils. The two Congo sandy soils had pH-method design contents of 9 to 10 percent compared to 3 and 4 percent respectively for the test method.

Test results for DAH-3 and NIA-1, stabilized with lime and cement respectively, at the pH design content are shown in Table 9.3.

The strength increases of NIA-1, stabilized with 4 percent cement, were high. The CBR increased from 78 to 500 with 4 percent cement after 7 days curing. The unconfined compressive strength increased from 5.6 to 25.3 kg/cm^2 (80 to 360 psi) in 7 days. There was a slight reduction of the PI and maximum dry density, and a small increase in the OMC.

The increase of strength was less for DAH-3, stabilized with 5 percent lime. The CBR increased from 16 to 129; the unconfined compressive strength increased from 7.7 to 16.2 kg/cm^2 (110 to 231 psi) after 28 days. The plasticity index reduced from 24 to 16, while there was a slight decrease in the maximum density and slight increase in the OMC.

Asphalt Stabilization

Asphalt stabilization studies on four sandy lateritic soils were conducted by The Asphalt Institute in College Park, Maryland. Three ferrallitic soils were submitted, two from the Congo Basin and one from Dahomey. One sandy ferruginous soil from Niamey, Niger was also submitted.

The data indicates that all four soils can be successfully stabilized and waterproofed with bituminous binders. However, the soils with appreciable content of fines required special studies. As with all types of stabilization, the type and amount of binder have to be established by means of laboratory tests involving actual project soils.

Conclusions for the African Program

One of the goals of this stabilization investigation was to determine if soils of the three major groups would react differently to stabilization. From the results of this study, it appears that some differences occur in the reaction of the different soil groups to stabilization. The ferruginous gravels when stabilized with cement showed large gains in strength at small percentages of cement, higher than the materials in any of the other soil groups. The ferruginous and ferrallitic soils from sandy parent materials showed a great reduction in the plasticity index with a small percentage of lime, but with further increases in the lime content the decrease in the plasticity index was slight. The other soils showed a linear decrease with increasing lime content.

In almost all cases the strength and durability of lime and cement treated materials increased with increased stabilizer content. For each soil tested, the soil-cement strength and durability were greater than the lime-soil strength and durability.

For the cement treated soils, the recommended minimum values of unconfined compressive and CBR strengths were attained with 3 to 6 percent cement. Because the experimental test data included a wide variety of lateritic gravels, it can be concluded that lateritic gravels can generally be stabilized with 3 to 6 percent cement and used as base course.

From the same experimental data it can be seen that lime stabilized gravels would have limited use as base course materials *if* the previously mentioned criteria is accepted. Most lime treated soils showed maximum strength values for percentages of lime between 4 and 6. Higher percentages of lime resulted in a reduction of strength. Lime has value as an admixture for subbase material or to improve subgrade soils where lower CBR's are acceptable. Again this is based on the previously mentioned criteria for base courses.

TABLE 9.3

Stabilization Test Data
(Stabilizer Content Determined From pH Test)

	NIA – 1			DAH – 3		
	0%	4% Cement		0%	5% Lime	
		7 day	28 day		7 day	28 day
LL	21.0	29.6	33.6	44.6	46.3	47.4
PL	11.0	21.5	23.3	20.8	30.2	29.8
PI	10.0	8.1	10.3	23.8	16.1	17.6
MDD	2.15	2.13	–	1.81	1.80	–
g/cm^3 (pcf)	(134)	(133)	–	(113)	(112.5)	–
OMC	8.7	9.5	–	16.6	17.0	–
5cm (2 in) unconfined	5.6	25.	36.	7.1	12.4	16.2
kg/cm^2 (psi)	(80)	(360)	(511)	(110)	(177)	(231)
CBR	77.5	500	500 +	16.2	98.7	129.0
Swell	0.18	0	0	0.7	0	0

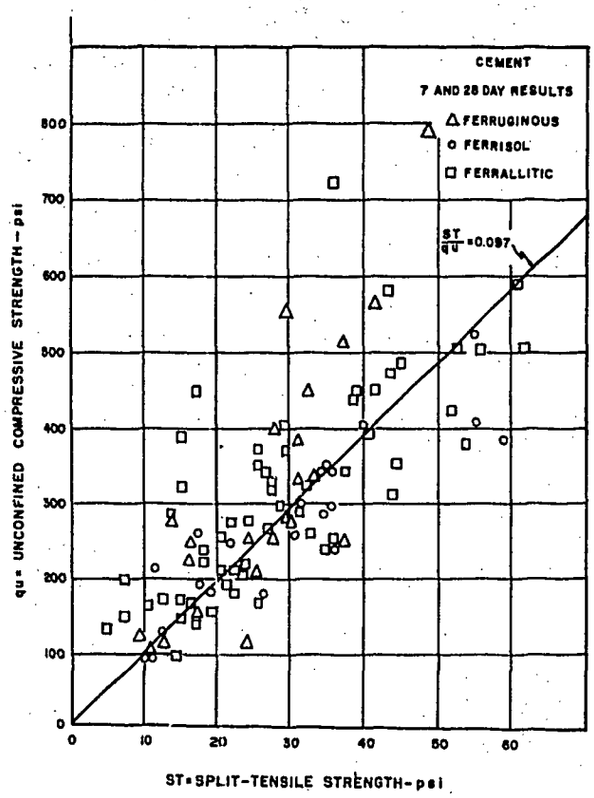


FIGURE 9.9 – COMPARISON BETWEEN SPLIT-TENSILE AND UNCONFINED COMPRESSIVE STRENGTH, CEMENT

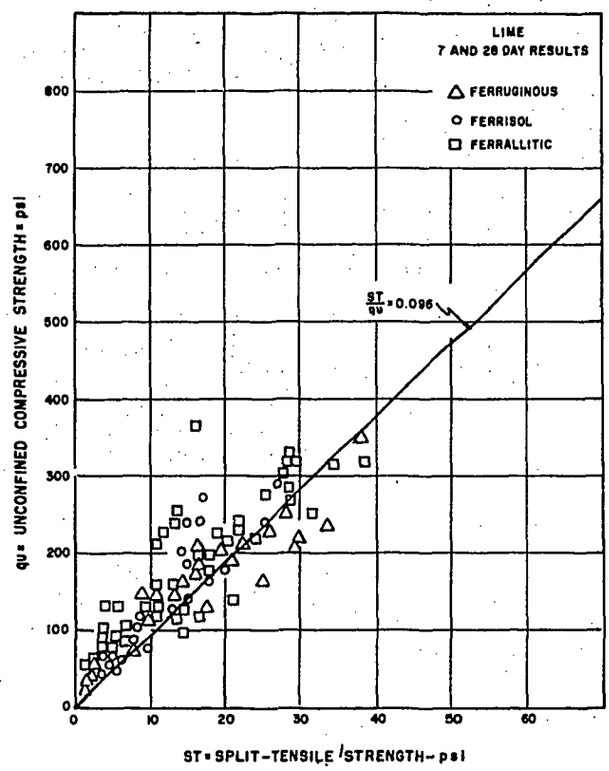


FIGURE 9.10 – COMPARISON BETWEEN SPLIT-TENSILE AND UNCONFINED COMPRESSIVE STRENGTHS, LIME

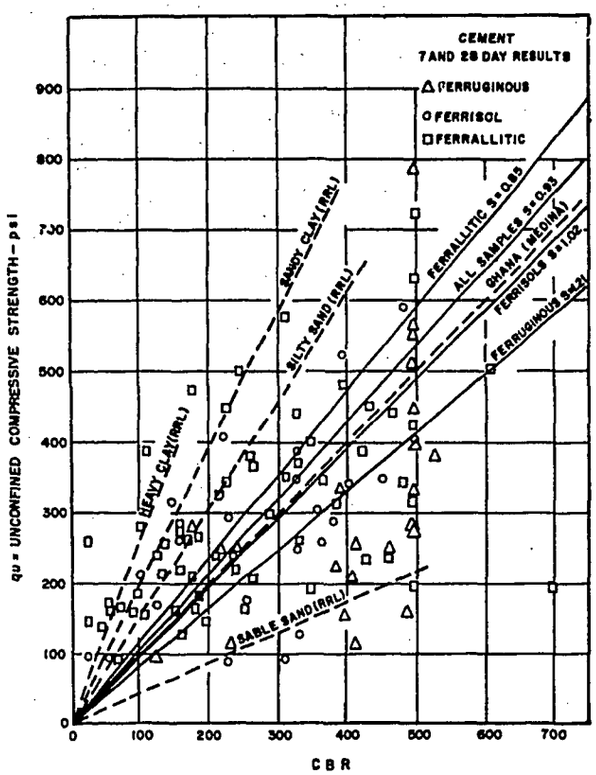


FIGURE 9.11 – COMPARISON BETWEEN COMPRESSIVE STRENGTH AND CBR, CEMENT

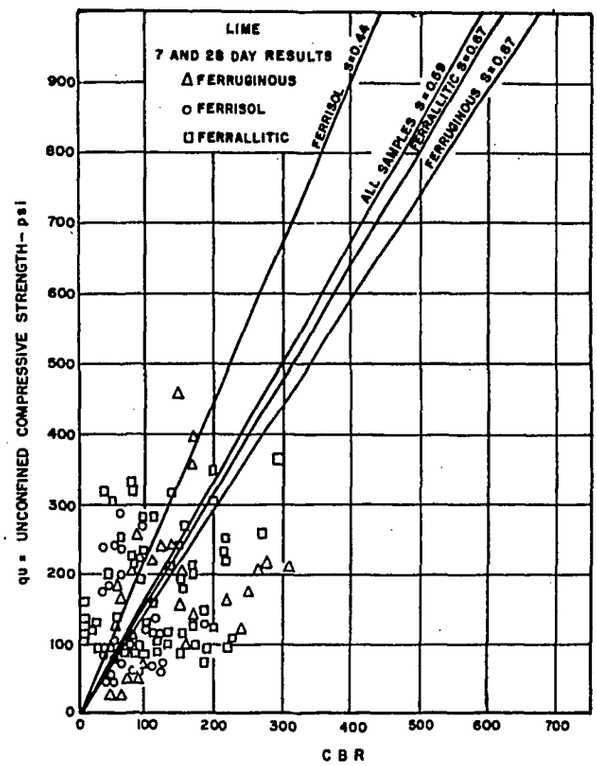


FIGURE 9.12 – COMPARISON BETWEEN COMPRESSIVE STRENGTH AND CBR, LIME

The recommended PCA values for the loss of weight after 12 cycles of the wet-dry durability test were attained with 4 percent cement in all soil-cement mixtures. The recommended PCA values were generally not attained with the lime-soil mixtures even at the higher lime contents. This test may not be a true indication of the actual required durability of gravels with a surface covering.

In almost all cases the lime or cement-treated materials did not reach a non-plastic state. In the lime-soil mixtures the plasticity index was reduced considerably more than in the soil-cement mixtures. The plasticity indexes of all the lime-treated gravels were reduced to below 15.

In the soil-cement mixtures there were large increases in strength between 7 and 28 days. With lime-soil mixtures there was only a slight increase in strength between 7 and 28 days, but lime-soil reactions with kaolinite clays are known to be slow, requiring weeks.

The split tensile test appears to be a good method to evaluate the tensile strength properties of lime and cement--treated laterite materials. The split-tensile strength increases proportionately with the unconfined compressive strength. The lime-soil and soil-cement mixtures both showed an average value for the ST/qu ratio of 0.10.

STABILIZATION STUDIES IN SOUTHEAST ASIA

Experiment Design

Three aggregates were selected for the major portion of the laboratory study program. Aggregate No. 1, with a generally poor service record, was selected as a typical "poor" lateritic soil; and Aggregate No. 6, with a generally good record, represented the typical "good" lateritic soils. They were selected to give ressonable extremes in lateritic soil qualities. Aggregate No. 9, a siliceous gravel, was selected as a relatively good quality, non-lateritic control material. Less extensive testing was done on other lateritic materials designated as Aggregates No. 2, 3, 4, 5, and 7.

Stabilizing admixtures used were cement, cement with lime, and asphalts of two types; a slow setting emulsion and a rapid curing cutback. For control purposes, tests were also performed on the aggregates with no admixture added.

Testing

CEMENT AND CEMENT-LIME TESTS

Unconfined compressive strength was selected as the means of evaluating the effectiveness of the various levels of cement and cement-lime stabilization treatments. Two major series of tests, based on different methods of curing before testing, were undertaken. In one series the specimens were moist-cured and tested at different ages, and in the second series the specimens were tested after various numbers of wet-dry cycles. Cement used in this series was a normal portland cement generally used locally, and the lime used was an ordinary hydrated lime, also processed and commonly used locally. All specimens were prepared with aggregates at the natural grading and at optimum moisture content. After mixing with the designated quantity of admixture, the specimens were compacted in five layers in molds 10.2 cm (4 in) in diameter and 12.7 cm (5 in) in height with 25 blows per layer of 4.5 kg (10 lb) hammer falling 0.45 m (18 in). Specimens were extruded from the molds immediately after compaction, and were capped on both ends about two hours before testing.

ASPHALT TESTS

Tests selected to evaluate the effectiveness of the various levels of asphalt emulsion and cutback stabilization treatments were Hweem Stability (including swell), moisture vapor susceptibility, Hveem cohesiometer value and permeability. Mixing of aggregates and asphalt was done with both materials at room temperature. Mixtures were aerated to reduce moisture or the content of volatiles to a pre-computed level before compaction. All mixes were compacted at room temperature with 25 preliminary tamping blows of the kneading compactor at 3.5 kg/cm^2 (500 psi). Two specimens were tested at 60°C 140°F for Stabilometer ("S") Value, and immediately following, specimens were tested for Cohesiometer ("C") Value. These tests were repeated on a second set of specimens after they had been subjected to the moisture vapor susceptibility (MVS) test.

Summary of Test Results

The results of all tests performed in these studies of stabilization of lateritic soils are presented in Figures 9.13 through 9.18. In the discussions below, this data is interpreted and expressed in terms of general statements of findings, and brief conclusions are presented regarding the significance and future applications of the findings.

CEMENT AND CEMENT-LIME STABILIZATION

The major findings represented by the results of tests in the variable curing time series are as follows:

1. With few exceptions, the strength at all ages increased with increased cement, or cement and lime content: and the strength of the specimens with 6 percent cement was higher than those with 4 percent cement and 2 percent lime. (Figure 9.13).
2. There was a general increase from 3 and 7 day strengths to 30 day strength. Most specimens also increase somewhat more in strength from the 30 day period to the 4 to 6 month period, but this was usually a small increase, and some specimens actually showed a small decrease in strength with this further curing (Figure 9.14).
3. Aggregates Number 1, 3, and 4 were the only ones which showed an appreciable reaction to lime, with strengths on Numbers 3 and 4 (after the longer curing period) being the only values over 70 kg/cm^2 (1,000 psi). The strength of Aggregate Number 1 increased from just over 21 kg/cm^2 (300 psi) with 4 percent cement only, to about

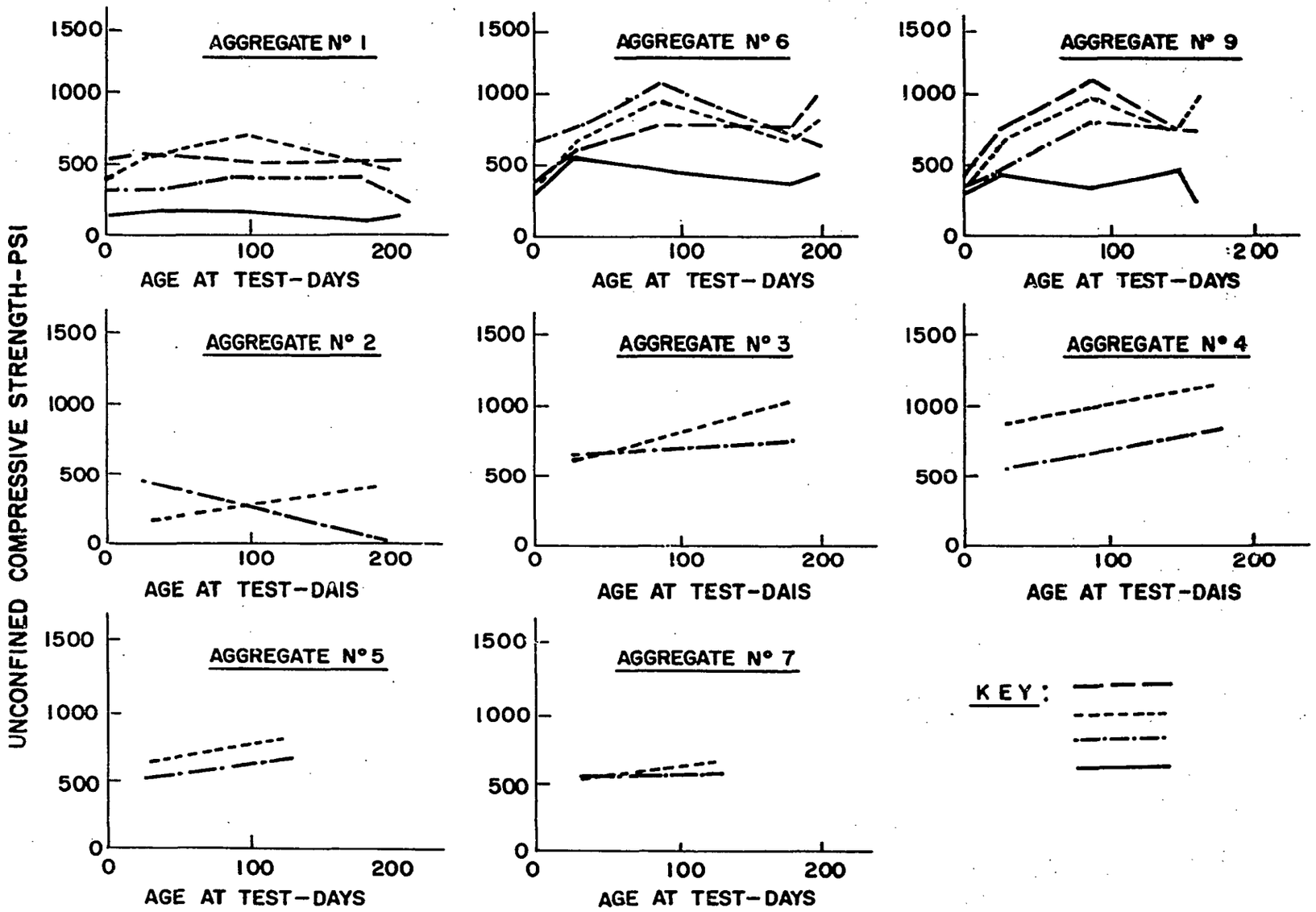


FIGURE 9.13 – RESULTS OF COMPRESSIVE STRENGTH TESTS ON CEMENT AND CEMENT-LIME STABILIZED SPECIMENS, VARIABLE MOIST CURE TIME SERIES

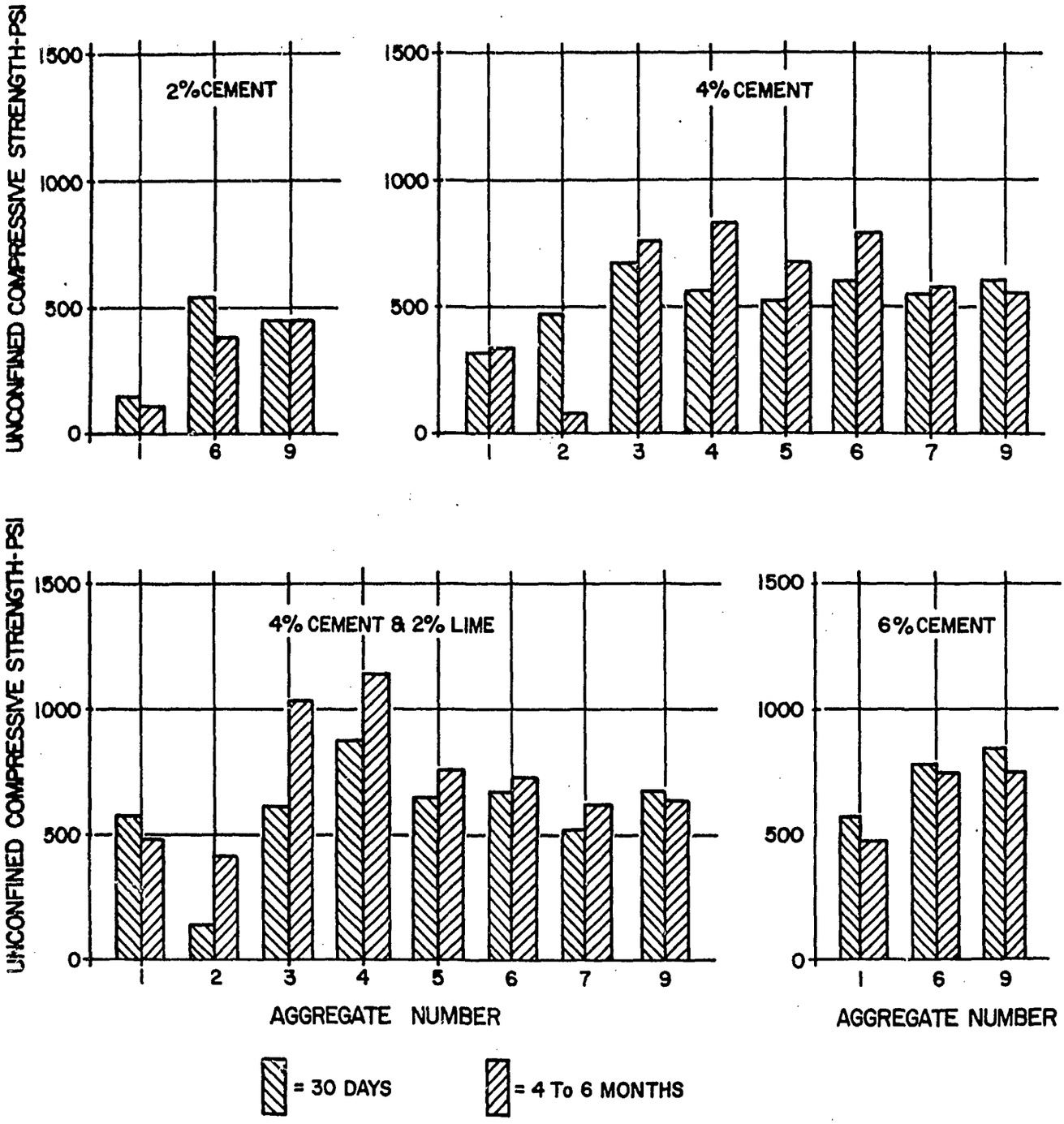


FIGURE 9.14 – STRENGTH OF CEMENT AND CEMENT-LIME STABILIZED AGGREGATES AT 30 DAYS AND AT 4 TO 6 MONTHS

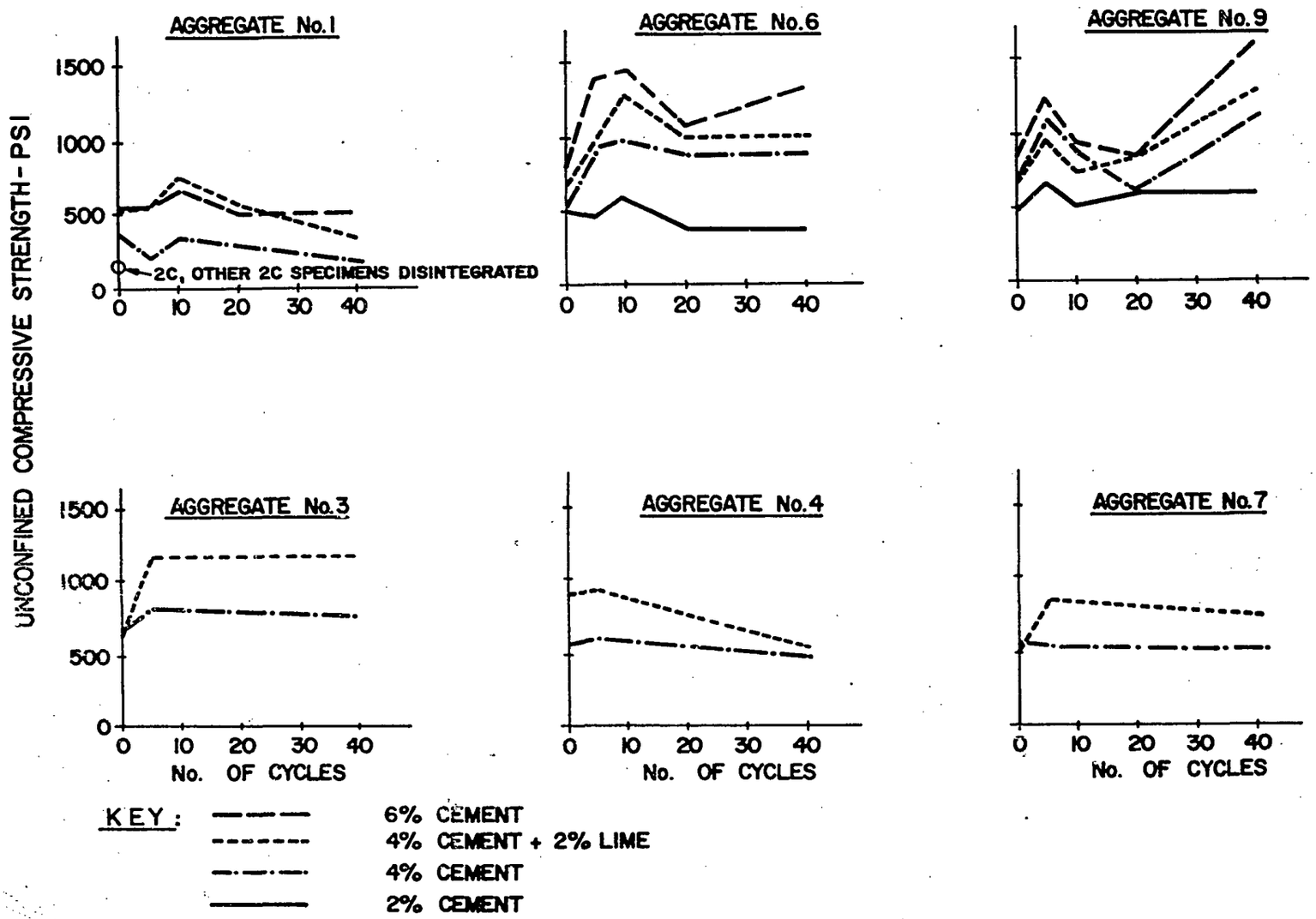


FIGURE 9.15 – RESULTS OF COMPRESSIVE STRENGTH TESTS ON CEMENT AND CEMENT-LIME STABILIZED SPECIMENS, WET-DRY CYCLE SERIES

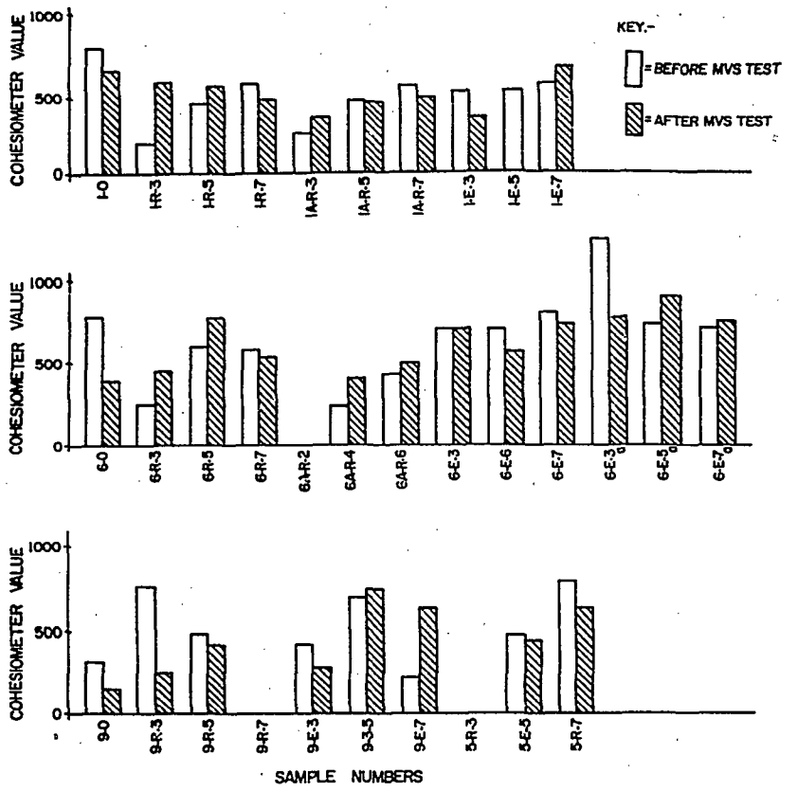


FIGURE 9.17 – COHESIOMETER VALUE BEFORE AND AFTER MOISTURE VAPOR SUSCEPTIBILITY TEST

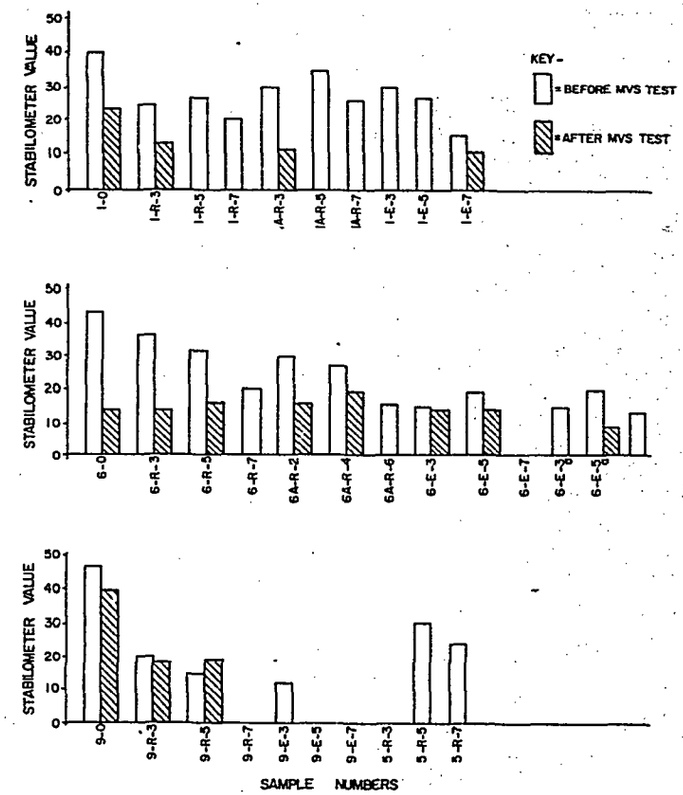


FIGURE 9.16 – STABILOMETER VALUE BEFORE AND AFTER MOISTURE SUSCEPTIBILITY TEST

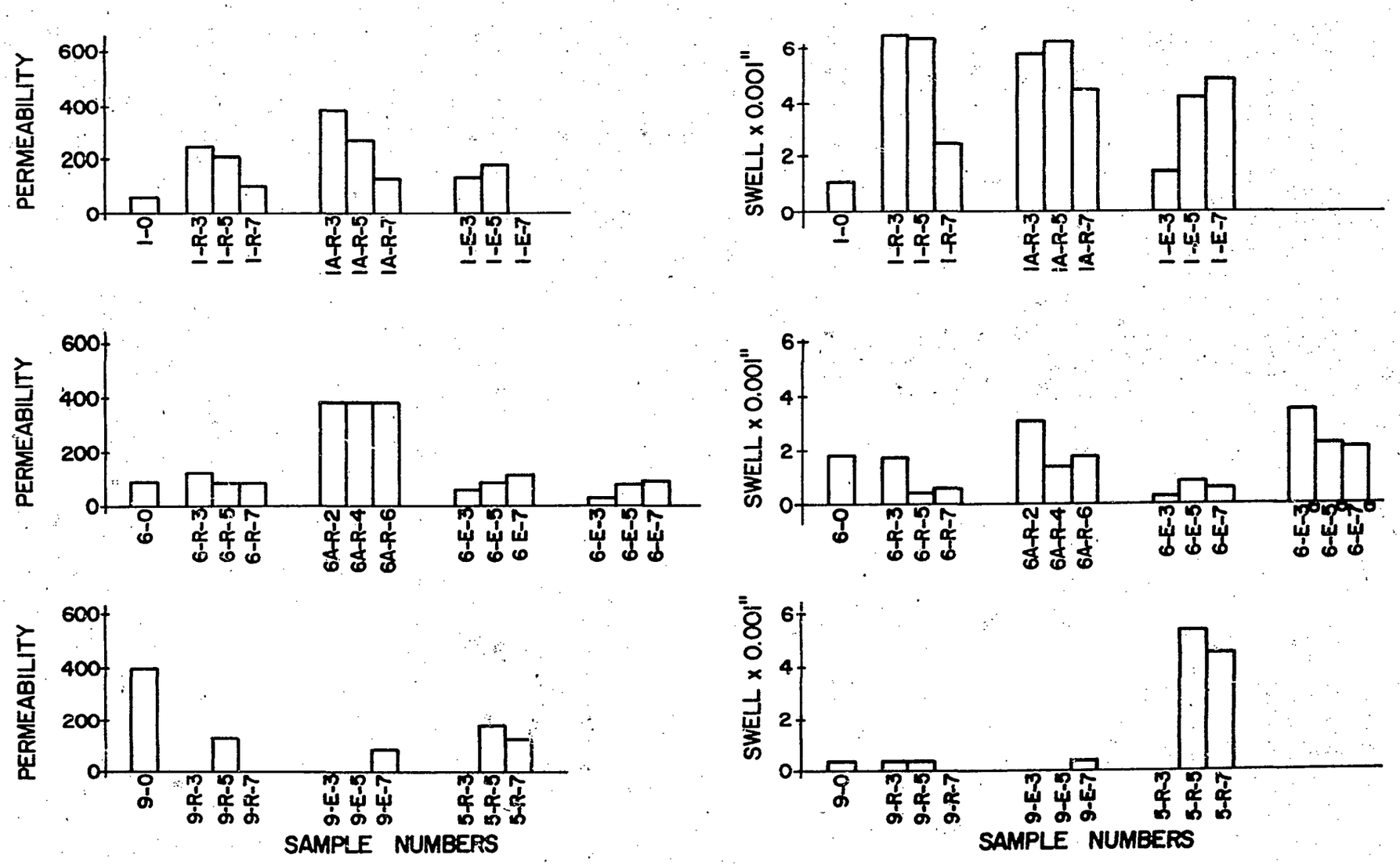


FIGURE 9.18 — PERMEABILITY AND SWELL TEST RESULTS

35 kg/cm² (500 psi) with 4 percent cement with 2 percent lime; this same strength, however, was obtained with 6 percent cement with no lime.

4. If the criteria of a strength of at least 32 kg/cm² (450 psi) at 7 days is applied, indications are that Aggregates Number 9 and 6 would be satisfactory at 4 percent cement, and Aggregate Number 1 at 6 percent. None would meet this requirement with 4 percent cement and 2 percent lime.
5. If overall increase in strength after long-time curing is used as the criteria, the aggregates tested could be ranked in order from best to worst about like this: 4, 3, 6, 9, 7, 1 and 2.
6. Aggregate Number 2 appears to be the least reactive to stabilization with these additives—it is the only which failed to attain a strength of 32 kg/cm² (450 psi) with 4 percent cement and 2 percent lime, even after curing over six months.

The following major findings are represented by the results of tests in the Wet-Dry Cycles series:

1. As would be expected, specimens with the higher percentages of cement and cement-lime were less detrimentally affected by the wet-dry cycling than were those with the lower percentages. This was particularly evident in the case of Aggregate Number 1, where specimens with 2 percent cement disintegrated and were unable to be tested (Figure 9.15).
2. If all specimens with only 2 percent cement are not considered, the strengths of Aggregate Number 9 (the siliceous gravel) and of Number 6 (the "better" lateritic gravel) are higher after the total 40 cycles of wetting and drying than after 28 day curing and no wet-dry cycles. Aggregate Number 1 (the "poorer" lateritic gravel), together with Numbers 3, 4, and 7, on which limited tests were conducted, tended to be slightly reduced in strength or remain about the same after the total 40 cycles.

It is concluded that both cement and cement-lime were demonstrated to be effective in stabilizing lateritic gravels, although the amount of additive required, the magnitude of the benefit derived, and the effect of wetting and drying on the mixture varies considerably with different materials. Thus, for some soils, the amount of additive required may be so great that stabilization would be uneconomical.

Since the advantages of lime treatment are in its reaction on the clay portion of a soil, cement-lime may be most effective in improving the soils, which most need improvement, i.e., those with the higher percentage of clay.

ASPHALT STABILIZATION

The major findings are represented by the results of tests in the asphalt emulsion and asphalt cutback stabilization series are as follows:

1. Generally, the Hveem Stabilometer Value at the highest asphalt content in each series of these tests is lower, or about equal to, that at the lowest asphalt content. At the intermediate asphalt content, this value is always higher than at the highest asphalt content and sometimes higher than at the lowest asphalt content. The three values, together with the value for no asphalt, could be plotted as a relatively typical stability curve for asphalt mixes, with stability decreasing with increasing asphalt content (Figure 9.16).
2. The Hveem Stabilometer Value after the MVS test is generally appreciably lower than the initial value in the cases of the two lateritic materials (Numbers 1 and 6). For Aggregate Number 9, the non-lateritic control material, there appears to be a much less reduction, although the data are not as complete.
3. The Cohesiometer Value tends to follow the expected trend of increasing with increasing asphalt content, although this is not a clear-cut trend. (Figure 9.17). It should be noted here that it is very difficult to obtain a homogeneous mixture of asphalt with a material which has an appreciable amount of fines, particularly if these fines are plastic. This difficulty in mixing would tend to make test results erratic, especially for a test like the Cohesiometer Test.
4. The Cohesiometer Value appears to be little affected by the MVS Test. The values after MVS are about equally divided between those higher and those lower, with a few about the same.
5. Permeability tends to be reduced with increased asphalt content, and increased asphalt also seems to contribute to a reduction in swell (Figure 9.18).
6. The data show no advantage or disadvantage for either the asphalts used — the emulsion or the cutback.

The results of the asphalt stabilization studies were inconclusive as far as determining whether it is practical or economical to stabilize lateritic soils with asphalt. The stabilometer value on the lateritic materials was shown to be reduced after MVS tests, which indicates some potential detrimental effects of water on the mixtures. However, more intimate mixing of the asphalt and soil might have lessened this effect. As an aid to obtaining better mixing in the field, it is suggested that the aggregate be premixed with water before adding asphalt. For cutbacks, the aggregate should contain little free water, however, considerably more water may be used before mixing with emulsions. In both cases, the mixtures must still be dried to the proper liquid content before compacting. However, if improved mixing cannot be obtained in the field, it is doubtful that there are any advantages of asphalt stabilization of lateritic materials, other than relatively clean lateritic sands and gravels.

STABILIZATION STUDIES IN SOUTH AMERICA

A very limited stabilization study was undertaken in South America. In view of the results obtained in Africa and Southeast Asia, it was felt that only a limited amount of testing was required; only the unconfined compressive strength and CBR tests were considered necessary. Only hydrated lime and Portland cement additives were examined.

The original intention had been to collect samples whose properties fell loosely into three groups. The first would comprise those materials which had CBR values around 50, or which indicated that, untreated, they could be considered as low quality base materials. The second group would have CBR values between 20 and 40 and that, untreated, could be considered to be suitable as subbase material. Finally, the third group of materials would be those with CBR values of under 20 which would be considered as subgrade material. Representative samples from arenosols, ferralsols and luvisols were selected from the testing conducted for Chapter 4. Two each of the arenosols, ferralsols and luvisols were found for each category.

It proved to be difficult to collect all of these samples, and to have them shipped to Rio de Janeiro for testing and forwarding to the University of Florida.

The properties of the soils investigated are given in Table 9.4. The results from Nicaragua were provided by the Ministry of Public Works. Only one sample had appreciable gravel, but many had considerable sand contents. Only one sample was non-plastic. In the remainder, the liquid limits ranged from 20 to 93 while the plasticity indexes varied from 8 to 59.

The minimum lime and cement contents were determined by the quick pH test of Eades and Grim (1966), but, as this test yielded lower and higher percentages of additives (depending upon the additive) than the unconfined compression test in the Phase II study a new pellet pH method was devised by Eades (personal communication). In this procedure, 20 grams of air dried soil passing the 0.425 mm sieve is mixed with a given percentage of lime or cement (several different contents of each are tested) and enough water to bring the mixture to its plastic limit is added, mixed thoroughly, and poured into a 3.6 cm (1.4 in) by 6.4 cm (2.5 in) mold and compacted by applying 19 blows with a 0.6 kg (1.4 lb) hammer falling 30 cm (12 in). This compactive effort is equivalent to the Standard AASHO compactive effort of 1,715 kg – m (12,400 ft–lbs). The compacted pellet is extruded from the mold and sealed in a plastic bag for four hours and cured at 120°C. The pellet is then crushed with a rubber pestle until it is disaggregated, mixed with 100 ml of water, stirred and allowed to sit for one hour. The pH is measured with an electrode calibrated with a buffer solution having a pH of 12.00. The pH for each of the lime-soil and cement-soil mixtures is recorded. The lowest percentage of lime that gives a pH of 12.4 is the minimum amount of lime required to stabilize the soil. The lowest percentage of cement that gives a pH of 12.2 is the minimum amount of cement required to stabilize the soil.

The pH method is believed to indicate that all soil-lime and soil-cement reactions, such as those involving the additive and organic matter or sulphate must occur before reactions between the additive and the clay materials will occur. This is indicated when the soil-lime mixture gives a pH of 12.4 (the pH of lime) and when the soil-cement mixture gives a pH of 12.2 (the pH of cement). The pellet method is believed by Eades to give a more reliable determination because the lime and cement are placed in more intimate contact with the soil and cured for a longer period of time.

The pellet test was compared with the ordinary quick test and the quick test after curing the mixture at 120°C. The results are shown in Table 9.5. It can be seen that the quick test indicates for all the samples tested a minimum of 8 percent cement is required and a maximum of 10 percent. The pellet test indicates that most soils required 10 percent. The reasons for such high cement contents are not clear, but perhaps soil-cement reactions occur quickly before other reactions are satisfied. The more realistic values shown for lime may indicate that this is not the case for soil-lime reactions.

Cement and lime stabilization results are given in Table 9.6. The cement and lime contents added are given in the column on the left. These were the amounts indicated by the pellet test for lime, performed at the University of Florida. The unconfined compression strengths reported are for 28 days of curing. CBR values are also shown for a few of the samples. The unconfined compression results reported are the average of three tests.

Although the percentages of cement and lime are those that were indicated from the lime pellet test, the strengths reported in Table 9.6 were greater for cement than for lime in about 46 percent of the samples tested. The unconfined compressive strengths for the various percentages of cement are shown in Table 9.7. According to Ingles and Metcalf (1972) the strength of the stabilized materials should not exceed 56 kg/cm^2 (800 psi) as measured in the unconfined compression test without soaking, if flexible behavior is to be maintained. Based on this criteria both lime and cement are good additives for improving the strength of the component layers of a flexible pavement.

A soil such as Ceara 3 is highly reactive with cement. The reactivity of the sample with 5 percent cement is 15 kg/cm^2 (214 psi). The reactivity is defined as the difference between the untreated strength and the strength after treatment. With 6 percent cement it has an unconfined compressive strength of 32 kg/cm^2 (454 psi); in Table 9.6 this soil shows an unconfined compressive strength of 16.8 kg/cm^2 (239 psi), with 5 percent cement. With lime this value is 9.1 kg/cm^2 (130 psi), although lower, indicates this soil is reactive as well with lime.

Lime appears to be more reactive than cement in 7 of the samples (Table 9.6) while cement appears to be more reactive in the remaining 6 samples. All the samples which show higher strengths with cement have plasticity indexes under 12.

CONCLUSIONS

There has been a certain amount of controversy between those advocating stabilization with cement and those advocating stabilization with lime. This is true in temperate climates involving both residual and transported soils. It is also true in tropical climates where the soils are mostly residual. Those that believe only in cement stabilization state that there is little in tropical residual soils with which the lime can react, and unless there are clay-lime reactions, there is no benefit in adding lime. Those who believe in lime stabilization state that most clayey materials are more effectively stabilized with lime than cement. There are also those who believe in a combination of both

TABLE 9.4

Properties of Soils Investigated in South America

	Pernambuco 8	Rio Grande do Norte	Paraíba	Bahia 6	Brasília 4	Pernambuco 6	Goiás 3	Bahia 8	Rio Grande do Sul 4	Ceará 3	Bahia 5	Minas Gerais	Maranhão 3	Nicaragua Red Clay	Mato Grosso 3
GRAVEL	0	7	1	2	3	2	42	0	0	3	0	0	1	–	0
SAND	69	57	46	45	11	18	45	42	21	57	24	18	23	–	57
SILT	31	36	53	4	86	70	4	8	17	9	24	29	24	–	43
CLAY	–	–	–	39	–	–	9	50	62	31	52	53	52	–	43
LL %	20	23	27	31	50	46	NP	33	48	28	64	47	57	84	27
PL %	9	15	19	20	33	29	NP	20	30	17	45	33	30	41	15
PI	11	8	8	11	17	17	NP	13	18	11	19	14	27	43	12
SPECIFIC GRAVITY (a)	–	–	–	2.68	–	–	2.91	2.65	2.61	2.58	2.82	2.66	2.59	–	2.67
MDD g/cm³ (pcf)	2.130 (132.9)	2.068 (128.4)	2.010 (125.4)	1.916 (119.6)	1.381 (86.2)	1.785 (11.4)	2.095 (130.7)	1.775 (110.8)	1.550 (96.7)	1.910 (119.2)	1.442 (90.0)	1.412 (88.1)	1.508 (94.1)	1.220 (76.1)	1.78 (111)
OMC % (b)	7.5	9.0	11.0	11.2	26.0	19.0	13.3	16.5	24.0	12.3	29.4	29.0	25.0	37.5	13.2
ASSHO Class.	A-2-6	A-4	A-4	A-6	A-7-5	A-7-6	A-1-6	A-6	A-7-5	A-2-6	A-7-5	A-7-6	A-7-5	A-7-5	A-6
GROUP INDEX	(0)	(0)	(3)	(1)	(18)	(12)	(0)	(5)	(15)	(0)	(19)	(14)	(24)	–	(1)
CBR	44	35	30	27	25	24	24	15	14	13	11	11	2	2.3	16
CBR Swell	0.2	–	0.1	0.1	0.05	1.2	0.3	0.2	0.2	0.1	1.0	0.1	2.9	5.3	–
FAO Class. (c)	Q	Q	Q	L	F	L	F	Q	F	L	L	L	L	–	Q

(a) on minus No. 10 fraction (b) AASHO Modified (c) Solid Groups: A = Arenosols; L = Luvisols; F = Ferralsols

TABLE 9.5

Percent Cement Required Based on pH Tests

Sample Name	Quick Test	Quick Test & Curing at 120°C	Pellet
Brasília 4	10%	8%	10%
Maranhão 3	8%	6%	10%
Rio Grande do Sul 4	10%	8%	10%
Pernambuco 6	8%	6%	10%
Ceará 3	8%	6%	10%
Pernambuco 8	8%	6%	9%
Bahia 8	8%	6%	8%
Bahia 6	10%	6%	10%
Bahia 5	8%	6%	8%
Paraíba 9	8%	5%	8%
Rio Grande do Norte 1	8%	6%	8%
Minas Gerais	8%	5%	12%

TABLE 9.6

Cement and Lime Stabilization Results, South America

Sample	Soil Type	% Additive	Unconfined Compressive Strength			California Bearing Ratio	
			Untreated	Lime	Cement	Untreated	Lime
Brasília 4	F	6	1.7 (24)	7.6 (108)	3.8 (54)		
Maranhão 3	L	5	2.4 (34)	6.0 (85)	3.7 (53)	02	11
R.G.Sul 4	F	7	2.5 (35)	5.6 (80)	4.4 (62)		
Pernambuco 6	L	3	1.7 (24)	6.4 (91)	3.8 (54)		
Ceará 3	L	5	1.8 (25)	9.1 (130)	16.8 (239)		
Pernambuco 8	Q	3	0.4 (6)	1.4 (20)	3.6 (51)		
Bahia 8	Q	4	1.8 (26)	9.0 (128)	6.7 (96)		
Bahia 6	L	4	2.1 (30)	8.6 (122)	14.8 (210)		
Bahia 5	L	6	4.4 (62)	7.7 (109)	3.1 (44)	11	39
Paraíba 9	Q	3	1.6 (23)	14.8 (210)	6.0 (85)		
Mato Grosso 3	Q	3	2.0 (28)	5.8 (83)	11.0 (151)		20
Nicaragua	Red Clay	4	--	-	–	02	16
R.G.Norte 1	Q	2	1.5 (21)	3.6 (51)	8.1 (116)		
Minas Gerais		4	1.7 (24)	4.7 (67)	5.6 (80)	11	10

NOTE: Unconfined Compressive Strength at 28 day curing period in kg/cm^2 and (psi)

TABLE 9.7

Unconfined Compressive Strengths, South America

Sample	Percentages of Cement			
	6%	8%	10%	12%
Brasília 4	3.8 (54)	6.6 (94)	8.2 (116)	10.4 (148)
Maranhão 3	4.5 (64)	10.4 (148)	13.0 (185)	9.0 (128)
R.G.Sul 4	6.1 (87)	10.2 (145)	20.1 (286)	22.6 (321)
Pernambuco 6	23.0 (326)	11.2 (159)	28.4 (404)	24.2 (344)
Ceará 3	32.0 (454)	36.7 (522)	43.3 (615)	39.7 (565)
Pernambuco 8	12.8 (182)	18.7 (266)	23.4 (333)	30.8 (438)
Bahia 8	22.4 (318)	20.0 (285)	20.0 (283)	18.7 (266)
Bahia 6	18.0 (255)	23.8 (339)	34.1 (486)	30.2 (430)
Bahia 5	3.1 (44)	5.1 (72)	6.0 (84)	10.2 (145)
Paraíba 9	8.7 (123)	32.0 (455)	52.3 (744)	41.8 (594)
Mato Grosso 3	20.0 (285)	24.1 (343)	29.4 (418)	47.0 (669)
R.G.Norte 1	13.6 (193)	19.1 (271)	61.6 (876)	49.7 (707)
Minas Gerais	8.7 (123)	13.1 (186)	14.8 (210)	12.4 (177)

NOTE: Unconfined Compressive Strengths at 28 day curing period in kg/cm^2 and (psi)

lime and cement for tropical red soils. Studies have found many highly plastic soils which do not increase in strength upon the addition of lime but which always show a reduction in plasticity. The soils become more friable and easier to mix creating ideal soils for cement stabilization.

Although both cement and lime were used individually to stabilize most of the selected soil samples in these studies, it must be kept in mind that these were laboratory programs. Regardless of the plasticity and water content, laboratory-size soil samples can be dried and disaggregated to the desired size fraction. Dry soil irrespective of its grain size and plasticity can be blended with any dry admixture before adding water. The mixture can be compacted immediately before chemical reactions take place.

Under field conditions it is very hard and expensive to dry and break down a plastic soil mechanically. In a high rainfall area it is impossible. It has been recommended that cement be used to stabilize those soils having a plasticity index of 15 or less and a percentage passing the No. 200 sieve of no more than 25 percent. Soils with these physical characteristics can usually be mixed with most available construction equipment. It is often practical to cement stabilize even heavy clays after pre-conditioning with hydrated lime.

Lime is almost always more practical to use when the plasticity index is greater than 15 and the percent passing the No. 200 sieve is greater than 25 percent. However, lime has little effect in highly organic soils, or in soils with little reactive clay content. If the clay material is mostly kaolinitic the reaction with lime may be very slow and if the clay material is mostly iron minerals, lime may not be reactive. Cement should be used in those cases. However, soils with a plasticity index greater than 15 tend to "ball up" when a pulvermixer is used and lime may improve the workability. To insure that a soil will be properly blended, at least 60 percent of the soil binder must pass a No. 4 sieve after final mixing. This insures that the cement and the soil has formed a homogeneous mixture. The mixture must be compacted within a two hour period after the cement is added. This is done so that the mixture does not set before compaction. If the mixture does set and is disturbed after bonding begins there will be an unnecessary loss in strength.

Cementing reactions that take place between the lime and soil proceed slowly and, therefore, the final mixing can be delayed for considerable time. If the soil is very plastic, the lime can be mixed very expeditiously at first and compacted only enough to protect the lime from carbonation and subgrade from becoming saturated in case of a heavy rain until the lime has time to react with the clays. At the time of final mixing the pH of the soils should be determined and if needed enough lime added to bring the pH to 12.4 before compaction.

If the material to be stabilized has a plasticity index greater than 15 and laboratory strength tests show that the lime treated soil will not give a bearing value which will meet the design standards, lime and cement should be used in combination. The soil or base material should be mixed with enough lime to give a pH of 11.0 ± 0.2. The lime should be added first to disaggregate the soil by reducing the plasticity. At the time of final mixing, the cement should be added. The mixing and compaction of the cement and lime treated soil should be accomplished in the two hour time limit.

The general applicability of all stabilization methods is given in Table 9.8 taken from Ingles and Metcalf (1972). The general ranges for cement and lime are given along with those for bitumen, polymeric-organic materials and mechanical stabilization. Thermal stabilization is a technique applied mainly to expansive clays, usually for localized foundation problems.

A broad comparison of the stabilization techniques available in highway construction is given in Table 9.9. This may be used as a general guide, but other factors should be considered for tropical soils. The reactivity of the soils must be checked with each project.

Generally the sandy soils or those with high contents of iron will be most effectively stabilized with cement. This would include most of the luvisols and arenosols of South America, the ferruginous soils and sandy ferrallitic soils of Africa and the comparable soils of Southeast Asia. The acrisols and ferralsols of South America and the ferrallitic soils and ferrisols of Africa and the comparable soils of Southeast Asia will usually be more effectively stabilized with lime. With tropical soils a few chemical and mineralogical analysis are always advisable.

Lime needs clay minerals to react with, and kaolinite – lime reactions may be very slow. Lime is reactive with the aluminum minerals which are common in the ferralsols and acrisols. If none of these materials are present, lime is of little value outside possibly to help dry the soil during construction. These minerals or materials are commonly present in ferralsols and acrisols of South America and the ferrallitic soils of Africa and the comparable soils of Southeast Asia.

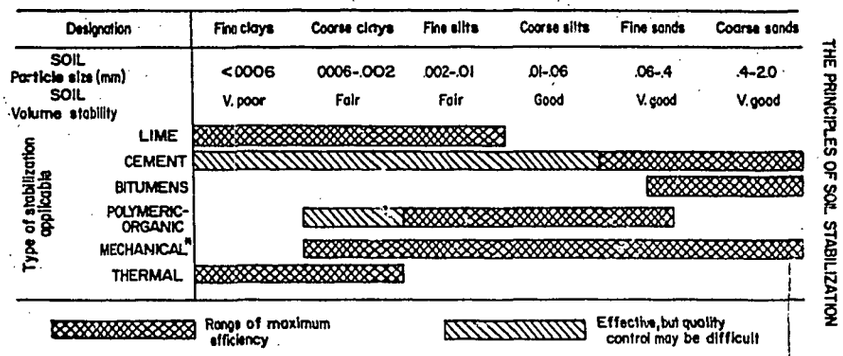
The results of the asphalt stabilization tests in Africa and Southeast Asia indicate that asphalt is an effective and practical stabilizer in relatively clean sands.

The Tropical Design Procedure can be applied if the stabilized materials do not exceed reasonable limits. Structural coefficients are given in Chapter 7 for CBR values greater than 100; however, there is no distinction made between these higher CBR values, since CBR values over 100 are meaningless. In using the design procedure outlined in Chapter 7 appropriate maximum CBR values for the design of stabilized layers would be the maximum values shown in Table 7.1 for the various depth intervals. Stabilization can be used to improve a material from a CBR of 50 to a CBR of 80. Or the designer can use stabilization as a means of improving materials that are otherwise unsatisfactory. As an example, a material which exhibits a CBR of 30 can be used as a base course material if, through stabilization, the CBR can be increased to over 50. Increased strengths with CBR values in the range of 100 and over should be evaluated in terms of unconfined compressive strengths.

The results of such tests can then be applied in a design procedure which takes into consideration the difference between the load-spreading characteristics of stabilized layers and unbound layers. The design procedure outlined in Chapter 7 can accommodate stabilized base courses on the basis of unconfined compressive strength, but these coefficients are conservative values.

One of the more advanced flexible pavement design procedures which incorporates stabilization of the component layers is founded on the basic concept that a reduction in the overall required thickness of pavement is possible due to its acquiring higher flexural strength with

TABLE 9.8
Applicability of Stabilization Methods
(After Ingles and Metcalf 1972)



THE PRINCIPLES OF SOIL STABILIZATION

TABLE 9.9
Broad Comparison of Stabilization Techniques
(After Ingles and Metcalf, 1972)

In situ Material	Pavement Thickness	Mechanical	Cement	Lime	Bitumen
Natural gravel	Min. 10 cm (4 in.)	Fines may be needed to prevent ravelling	Probably not necessary except if plastic. 2-4 per cent	Not necessary except if plastic. 2-4 per cent	Not necessary unless lacking fines. 3 per cent residual bitumen. Use medium or-slow curing cut-back or emulsion
Clean sand	Min. 10 cm (4 in.)	Coarse material for strength and fines to prevent ravelling	Unsuitable; produces brittle material	Unsuitable; no reaction	Most suitable 3 per cent residual bitumen Rapid curing cut-backs may be used Add 2 per cent lime for wet sand
Clayey sand loam	15-25 cm (6-10 in.)	Coarse material for strength and seal adhesion	4-8 per cent	May be suitable, depending on clay content*	May be suitable 3-4 per cent
Sandy clay	15-35 cm (6-14 in.)	Not usually suitable	4 - 12 per cent	4-8 per cent depending on clay content	May be suitable for light traffic 3-4 per cent
Heavy clay	25 cm (10 in.)	Unsuitable	Unsuitable. Mixing may be assisted by pre-treatment with 2 per cent lime then 8-15 per cent cement	Most suitable. 4-8 per cent depending on clay content	Not usually suitable

*Lime may render the material non-plastic if clay content is low. Usually requires less than 4 percent lime.

stabilization(McDowell, 1972). The original thickness is determined assuming the structural layers to be unbound material. Stabilization reduces the required thickness. The amount of reduction depends upon the tensile strength of the stabilized layer and the original required thickness.

The common African CBR requirement of 180 is an example of an arbitrary value which is not linked to pavement design criteria which takes advantage of flexural strength development. The new procedure is one of the few procedures which does provide for reduction in pavement thickness based on developed flexural strength resulting from stabilization. This is a relatively new concept which could add a significant dimension to engineering designs of flexible pavement.

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CHAPTER 10
TROPICAL BLACK CLAYS

INTRODUCTION

Tropical black clays, often called black cotton soils, are the major problem soils of the tropics. They are an unsuitable material to use in highway or airfield construction. They are unsuitable because they contain a large percentage of plastic clay and because they swell and expand as they absorb water.

In many areas where these soils occur, there are no suitable natural gravels or aggregates. Most tropical black clay deposits cover such large areas that avoiding or by-passing them is not feasible. Few roads constructed through these soils have proved satisfactory. Some, in Africa and Southeast Asia, have failed completely.

Most of the tropical black clays are formed residually by weathering of basic rocks such as basalts. Alluvial deposits also occur but, they are much more restricted in size except in the African Sudan. The Lake Chad Basin, also in Africa, is the only extensive lacustrine deposit of black clay soils known.

Black clays formed over calcareous rocks are not common in South America and Africa but they have been reported in other parts of the world, such as the western United States and Israel. Calcareous rocks are rich in the bases Ca and Mg; they have an alkaline pH and plentiful silica; and montmorillonite will form as a product of weathering.

The physical properties of the tropical black clays are distinctly different from those of arenosols and luvisols but there can be some overlap in properties with ferralsols and nitosols. This is particularly true over geologically recent volcanic rocks. In such cases there has been insufficient time to allow transformation of most of the clay minerals from montmorillonite to kaolinite to gibbsite. The exceptions are those areas of high rainfall where erosion has not proceeded as rapidly as soil development. Over other rock types, most of which are geologically older, the index properties of the ferralsols and nitosols still overlap those of the black clays. However, the differences in engineering properties are generally distinguishable. Over older basic rocks, leaching has altered almost all of the montmorillonite to kaolinite and gibbsite, which results in a greater iron content as well.

Most of the soils investigated were selected from the major tropical black clay deposits in Africa. Minor residual and alluvial deposits of limited area are not included as other soils are generally available and their use may be more economical. Moreover, the properties of these limited deposits can be expected to fall within the range described in this chapter. The emphasis here is on larger deposits, where construction difficulties have occurred and where alternative sources of material are restricted or absent.

Additional deposits from South America were also investigated. The areal distribution of black clays is far more restricted in South America and many deposits maintain sufficiently elevated moisture contents as to be non-critical.

This chapter is mostly taken from the Phase II report (Lyon Associates, et ali., 1972). Sections of that report have been published elsewhere (Morin, 1971). Recent research findings have been incorporated as have the test results from South America.

The distribution of the black clays in the tropics is given in Table 10.1 and on several maps in Chapter 3.

The objectives of the laboratory investigation of the tropical black clay were to determine typical properties of various deposits, to explain these properties with regard to the chemical and mineralogical composition, and to select tests that suitably describe the engineering behavior.

The samples were collected from areas selected from the Soils Maps, after considering the geology of the area. Soils over recent volcanic rocks were augmented by samples in poorly drained areas over various rocks including Precambrian, and over recent alluvial sediments.

LITERATURE REVIEW

Data from the literature have been compiled in Table 10.2. These show that the development of black clay soils follow a fairly characteristic pattern. Parent rocks for the residual soils are usually basic, most commonly basalt. Rainfall is generally under 1,270 mm (50 in) and drainage conditions are usually poor. All pH values reported are alkaline. Under these conditions, montmorillonite is the principal clay mineral that forms. Kaolinite and illite are common in subordinate amounts. Halloysite has been reported only in Ethiopia. The principal exchangeable cations are calcium and magnesium with potassium and sodium usually in minor amounts. Sodium is the principal ion in the Burma soil. Cation exchange capacities are within the normal ranges of montmorillonitic soils.

The gradation of the soils varies considerably. There is little or no gravel size in most soils and those which show appreciable percentages, such as those in India and Ethiopia, may include rock fragments not yet decomposed. The amount of sand varies from 0 to 39 percent and silt from 2 to 47 percent. The clay size reported (the upper limit for the clay size is rarely stated) ranges from 13 to 100 percent.

Almost all liquid limits are over 40 percent with most over 50 percent; most plasticity indexes fall between 20 and 60. Shrinkage limits range from 7 to 28 percent. The AASHO classification for most of the soils is A-7-5 or A-7-6. Organic contents are usually fairly low.

Compaction and CBR data are limited. CBR values are low when reported after the normal 4 day soaking period.

The expansiveness of tropical black clays is notorious and has been studied by a number of investigators. Uppal and Pallit (1959) found that swell pressures: (1) increase as the dry density increases; (2) increase with the height of the specimen being tested; (3) increase with increasing ranges of initial and final moisture; (4) are not significant until moisture exceeds the shrinkage limit; and (5) increase as the area of confinement increases (that is swelling pressures are greater under the central part of the roadway or building, etc.). Deb and Joshi (1960) found that the swelling pressure under a circular plate in the field reduced as the diameter of

TABLE 10.1
Distribution of Dark Clay Soils
(After Dudal, 1965)

Country	Area	Estimated coverage in round figures (Million hectares)
Angola	Valleys of the Cunene and Cubango, region of Catete and Southwestern part of the country	0.5
Argentina	Mainly in the northeastern part of the country in the province of Entre Rios, in the northeastern department of Buenos Aires, in south Corrientes, in Santa Fé and the Eastern Chaco	6.0
Australia	Mainly in Queensland (Darling Downs), northern New South Wales (Liverpool Plains), east-central part and coastal areas of the Northern Territories, patches in South Australia (near Adelaide), northwestern Australia and in Tasmania[1]	70.5
Basutoland	Drakensberg	2.3
Bechuanaland	West of Livingstone and Tuli; probably large areas around the Okovanbo and Makarikari swamps	0.5
Bolivia	Mainly in the eastern part of the country, Gran Chaco and Santa Cruz plains; scattered patches in depressional areas	2.0
Brazil	Southwestern and western regions of Rio Grande do Sul (Bagé, Uruguaiana, Alegrete and D. Pedrito districts); in the western parts of the country bordering Uruguay, Paraguay and Bolivia; in limited areas of the northeastern Brazil	4.5
Cameroon	Logone Chari basin and part of the Chad basin; peneplain of Kaélé and Marona region	1.2
Chad	Chad basin and scattered patches	16.5
Chile	Depressional areas associated with Noncalcic Brown soils in Santiago and O'Higgins provinces and also associated with Prairie and Chestnut soils in Magallanes Province	0.5
Congo	Ruzizi Plain, valleys of the Semliki and Lufira	0.3
Dahomey	Northern part of the country and in the region of Divo	0.1
Ecuador	Hilly lowland and valley bottoms in western part of the country (provinces of Guayas, Manabi, Esmeraldas)	1.0
Ethiopia	Rift valley and Ethiopian plateau	10.0
Ghana	Mainly Accra, Ho-Keta and Wineba Plains; scattered patches near Kpandu, Kwamen and Kwesi	0.2
India	Central and south-central Deccan plateau (parts of Bombay, Hyderabad and Madya Pradesh states)	60.0
Indonesia	Mainly in Central Java (Semarang, Demak, Bodjonegoro, Surabaya area), East Java and Lesser Sunda Islands (Lombok, Timor, Sumbawa and Flores)	1.8
Ivory Coast	In northern part of the country and in the region of Divo	0.2
Kenya	Athi Plains near Nairobi and other areas	2.0
Madagascar	Some valley depressions in the western part of the country and on uplands in the western and northwestern part	0.8
Malawi	Chyre valley and areas around Nyasa and Chilwa lakes	1.6
Mali	Niger valley, and a large area along the borders with Mauritania	0.7
Morocco	Mainly in the northwest (Gharb) and in the Doukkalas (south of Casablanca); scattered spots	0.2
Mozambique	Alluvial plains of the Limpopo and Inkomati rivers and surrounding uplands; Zambezi valley upstream of Tete; coastal area north of Mozambique city	1.1
Niger	Central Niger valley and several scattered patches	0.1
Nigeria	Northeast Bornu and Benoué river basin	4.0
Paraguay	Depressional areas in basaltic and limestone plateaus of the eastern region; in the Paraguay river basin and large areas in the Chaco region	1.5
Portugal	Depressional areas on diorite and limestone in Alentejo	0.1
Rhodesia, Southern	Part of the valley of the Zambezi; livingstone area and southern part of the country	1.0
Senegal	Senegal valley, lowland of the northwest; on the Cape Verde peninsula	0.2
Somalia	In the plains extending between the Giuba and Scebeli rivers	0.8
South Africa	Bush Veld and Springbok Flats (Transvaal), Highveld (Orange Free State and Transvaal)	2.1
South West Africa	Probable occurrence in the Caprivi Strip along the Cubango river and around the "Etosha Pan"	0.7
Sudan	Region between the White and Blue Nile extending east of the Blue Nile into Ethiopia and covering an area west of the White Nile; widespread in South Sudan, Bahr el Gasal, Upper Nile and Equatoria Province	40.0
Swaziland	Mainly in the Middelveld, eastern Lowveld and Lebombo areas	0.2
Tanzania	Valleys of the Mayowosi and Malagarasi, Great Ruaha valley areas near Tendigo swamps and Rukwa lake; scattered upland areas	7.0
Togo	In the northern part of the country and in the Mono valley	0.1

TABLE 10.1 (Continuation)

Country	Area	Estimated coverage in round figures (Million hectares)
Uganda	Valley of the Semliki, areas near George, Albert and Edward Lakes, areas in the east and northeast of the country	1.7
Upper Volta	Souron Valley; poorly-drained basin deposits spread over the country	0.4
Uruguay	In the southwest, northwest and southcentral part of the country and along the border with Brazil (Rio Grande do Sul, Bage area)	1.0
Zambia	Valleys of the Luangwa, Lukushashi and Zambezi; Kafue flats	0.9
Occurrence of dark clays has also been reported from: Algeria, Bulgaria, Burma, Cambodia, Ceylon, Colombia, Costa Rica, Honduras, Mexico, New Caledonia, Nicaragua, Panama, Philippines, Principe, São Tomé, Thailand, Tunisia, Venezuela, Viet-Nam	The extension of dark clay soils in the respective countries is relatively small and cannot be listed separately. The total surface occupied is estimated at	3.5

1 The figures available for Australia are 83 million hectares of Black Earths and 75 million hectares of Gray and Brown soils of heavy texture. While Black Earths consist mainly of dark clay soils, it is believed that only a part of the Gray and Brown soils of heavy texture belongs to the dark clays. The proportion is not know; a figure of 50 percent of the total hectare coverage has been tentatively adopted here.

the plate increased. Mehra and Uppal (1964) found that the swell pressure increases as the degree of compaction increases and as the difference between initial moisture content and moisture content at saturation increases. They measured swelling pressures of up to 14,700 kg/m^2 (3,000 psf) in India. Wooltorton (1951) measured up to 19,600 kg/m^2 (4,000 psf) in Burma. In an earlier paper on Burma (1936), Wooltorton suggested a minimum loading of 9,800 kg/m^2 (2,000 psf). Annual vertical movements of over 10 cm (4 in) were measured in the black turf soils in Central Transvaal, South Africa (de Bruijn, 1967).

A number of other Indian workers have studied the swelling pressures of Indian black cotton soils. Deb (1961) reviewed laboratory and field studies of swelling pressure versus bearing capacity. Mohan and Goel (1960) determined the swelling pressures and volume expansions of 20 soils. Nagarajan and Sundaraman (1960) evaluated different methods of laboratory measurements of swelling pressures. Palit (1953) describes a method of measuring swelling pressures with a simple loading frame and proving ring. Ranganatham and Satyanarayana (1965a) also describe a procedure for measuring swelling pressures in a consolidation cell. They later (1965b) advocated the use of the shrinkage index (liquid limit minus shrinkage limit) to predict the swell potential. Gupta et al. (1967) state that the montmorillonite content is mainly responsible for swelling, but other clay minerals such as illite and beidellite may also cause swelling. They believe that the base-exchange capacity "is a good indicator". Katti (1965) and Katti and Sadasivan (1967) state that a superimposed nonswelling soil layer is very effective in resisting the swelling pressure; the shear characteristics of a non-swelling soil contribute to a great extent in resisting the swelling pressure. Jennings and Knight (1957-58), of South Africa, devised the double oedometer test to determine the total heave that will occur. The test takes into account the degree of desiccation of the soil, the overburden and applied load, and the activity of the soil.

Swelling clays and clay shales are found in many parts of the world in temperate as well as tropical regions. The causes are usually high clay content with montmorillonite as the principal clay mineral. Therefore, work on swelling clays in temperate areas can be applied, except for unusual cases, to tropical black clays. Gizienski and Lee (1965), determined that expansion decreases with increased surcharge loads, decreasing density and increasing moisture content. However, actual heave measured in the field was much less than the amount indicated by the laboratory tests. Ho (1967) found that swelling pressure can be reduced by allowing some expansion to occur.

Kormornik and David (1969) statistically analyzed over 200 disturbed and undisturbed samples from Israel and found that the logarithm of the swelling pressure should be based on:

$$\log P = \bar{2}.132 + 0.0208\,(LL) + 0.000665\,(nd)$$
$$= 0.0269\,(w)$$

where P = swelling pressure in kg/cm^2
LL = liquid limit
nd = natural dry density of the soil in kg/m^3
w = natural moisture content

The correlation coefficient between log P and each of the single variables was low, but collectively it was 0.60. This suggests that swelling pressure depends not only on the properties but also on initial placement conditions, or density, water content and soil structure.

Foundations for structures in black cotton soils have received some attention by South African and Indian authors. Donaldson (1963, 1962) described the foundation problems that arise in South Africa and design methods that have been adopted. Changes in moisture content that bring about heaving in the black clays have been studied by Simons and Williams (1963) and Williams (1959). Deb and Joshi (1960) describe various factors which influence the bearing capacity of expansive clays. They believe designing bearing pressures to counteract swelling pressures may lead to bearing capacity failures since swelling pressure is bound

TABLE 10.2
Properties of Tropical Black Clays

Location	Author's Description of Material	Depth m (ft)	Parent Material	Rainfall mm (in.) yr	Drainage Characteristics	pH	Clay Mineralogy	Cation Ex. Capacity m.e/100 gm	Cation
Darling Downs Australia[1]	black earth	1(3)	basalt	<1250 (50)					Ca; Mg; Na
Burma[2]	black cotton		basalt	500-900(20-35)			illite; montmorillonite		Na;Mg
India (various)[3]	black cotton		trap. basalt			8-9	montmorillonite; some illite	40-50	Ca; Na; K
India (various)[4]	black cotton						–	44-72	
India (various)[5]	black cotton	1-4.5(3-15)	mostly trap				mainly montmorillonite		
Hyderabad. India[6]	black clay				poor		montmorillonite; tr. of kaolinite		
Nagpur India[6]	black clay			<1250(50)	poor		montmorillonite; some illite; tr. of kaolinite		
Palestine[7]	black cotton		probably chalk	860(34)		8.2			
Ethiopia[8]	black clay		basalt, trachyte	230-135(9-53)	poor to good	7.2-8.4	montmorillonite; kaolinite; halloysite	42-95	Ca;Na; K
Kenya[9]	black cotton		basic extrusive						
Uganda[10]	black clay								
Rhodesia[11]	black cotton			<1250(50)	poor		montmorillonite; with kaolinite; illite		
Basutoland[12]	black clay		basalt, dolerite						
S. Africa[13]	black clay	1.2(4)	norite	60(24)		7.7	montmorillonite		
Nigeria[14]	black cotton			<1250(50)	poor				
Nigeria[10]	black clay-cotton								
Ghana[6]	black clay	1-1.5(3- 5)	hornblende garnet-gneiss	<1250(50)	poor	7.9[15]	montmorillonite; tr. of kaolinite		Ca;Mg;Na[15]
Morocco[16]	tirs-		alluvial				montmorillonite; chlorite; tr. of kaolinite		
British Honduras[10]	black clay		chalk or marl		poor	8.4	montmorillonite		

TABLE 10.2 (Continuation)

Location	Gravel	Gradation Sand	Silt	Clay	Liquid Limit	Plast. Index	Shrink. Limit	AASHO Classific.	Specific Gravity	Organic Content %	Max. Density g/cm^3 (pcf)	Optimum Moisture,%	CBR
Darling Downs Australia[1]				50-64									
Burma[2]				48-61	58-81	38-57	9-11		2.68-2.70	little			
India (various)[3]		3-10	70-100		40-100	20-60							1.6
India (various)[4]				33-100	53-325	21-60	7-20		2.62-2.90	0.4-4.0			
India (various)[5]	0-10	6-35	15-19	42-74	46-97	22-49	11-14		2.66-2.73	0.4-2.4			
Hyderabad, India[6]		30	34	36	45	22	11	A-7-6(13)		2.7			
Naspur, India[6]		17	30	53	69	35	10	A-7-5(20)		0.6			
Palestine[7]		16	26	58	69	42	10	A-7-6(20)	2.79	1.0	1.426(89) (British Std.)	31	4
Ethiopia[8]	0-50	3-33	9-47	13-75	37-88	11-48	7-28		2.66-2.94	–	1.105-1.618 (69-101) (AASHO Std.)	40-17	2-8
Kenya[9]	1	4	28	67	103	60		A-7-5(20)	2.69	2.3	1.330(83) Modified (AASHO)	32	11 (at opt.)
Uganda[10]		39	33	28	46	22	17	A-7-6(11)	2.71	5.3			
Rhodesia[11]		34	11	55	72	48	11	A-7-6(17)	2.73	1.5			
Basutoland[12]	1	1	2	91	57	34		A-7-6(19)			1.330(83) (British Std.)	31	13 (at opt.)
S. Africa[13]				45	85	51							
Nigeria[14]		1	23	76	82	44	12	A-7-5(20)	2.61				
Nigeria[10]		6	26	68	66	41	13	A-7-6(20)	2.68	0.6			
Ghana[6]		10	40	50	99	70	11	A-7-6(20)		1.3			
Morocco[16]		5/10	25/30	60/70	60	40	10			2.0	1.426-1.458 (89-91) (AASHO Std.)		3-6 (in-place)
British Honduras[10]		2	23	75	126	68	25	A-7-5(20)	2.64	1.8	–		

1 Hosking (1935)
2 Woolforton (1951)
3 Mehra and Uppal (1964)
4 Katti, et al. (1966)
5 Mohan (1957)
6 Clare (1957)
7 Road Research Laboratory (1947)
8 Morin and Parry (1971)
9 Williams (1961)
10 Evans (1957)
11 Grainger (1952)
12 Pollard (1964)
13 D.H. Van der Merwe (1967)
14 Grainger (1951)
15 Stephen (1953)
16 Moussaoui (1967)

to dissipate with time while the moisture content does not change. Bearing capacity against shear failure is often considerably less than swelling pressure.

"Under-reamed" pile foundations in black cotton soils are commonly employed in India and elsewhere (Chandra, 1962; Chandra and Khepar, 1964). These are bell-shaped piles that resist upward forces. Double under-reamed piles are also utilized for increased bearing capacity (Subhash and Khepar, 1964). Mohan and Rao (1965) state that under-reamed pile foundations are satisfactory if carried to depths of 12 feet or more, at depths where moisture variations are not significant.

Strongman (1963) utilized black cotton soil successfully in a 25 foot embankment in Kenya by placing a protective blanket of weathered phonolite which prevented moisture loss in the black cotton soil.

Ranganatham and Satyanarayana (1967) have proposed an approximate method of analysis for the determination of earth pressures in expansive (black cotton) soils.

Road construction in black cotton soils has been described by Nageswara (1945) for India; by Wooltorton (1959) for Kenya; by Moussaoui (1967) for Morocco and for South Africa by Weinert (1968); Williams (1965); Williams and Simons (1963); and Williams and Jennings (1960). Moussaoui states that granular material is placed as a buttress to embankments of black clay soils; this prevents shoulder settlement caused by alternating wetting and drying. Uppal (1965) found that the moisture content of black cotton as subgrade soil under pavements varied from only 13 to 16 percent from dry to wet seasons, whereas, under the uncovered shoulders the variation was between 6 and 23 percent.

Black clay soils have been stabilized in the laboratory with hydrated lime. Clare (1957) found that the addition of 4 percent lime eliminated all plasticity in a Southern Rhodesian black cotton soil. Mehra and Uppal (1964) state that 4 percent lime improves the texture of an Indian black cotton soil sufficiently to permit its use in construction. The soil properties were improved from liquid limit of 90 percent to 55 percent; plasticity index of 44 to 5; and CBR of 1.6 to 17. The increase in CBR occurred with only 2 percent lime. Clauss (1967) describes the use of pH determinations of fresh and weathered dolerite as indicators of the stabilization requirements. Fossberg (1965) has investigated the engineering properties of a lime-stabilized clay.

Soils with montmorillonite and degraded weathering products retard the hydration and hardening of cement; such soils are more suitable for stabilization with lime than cement (Croft, 1967). Cement stabilization of Indian black cotton requires 17 percent cement (Mehra and Uppal, 1964).

Other research on stabilization has been performed in India. Katti and Kulkarni (1962) studied the effect of trace inorganic chemicals on lime stabilization of an expansive soil from Poona, India. Very small quantities of calcium chloride, sodium hydroxide, sodium silicate and potassium permanganate were used to improve the early strength characteristics of black cotton soil-lime mixtures. Katti and Sudhar (1962) and Katti et al. (1966) investigated stabilization with combinations of cement and lime. Mehra and Chadda (1957) also studied stabilization with gypsum. Misty (1957) gives the details of an experimental road where black cotton soil was mechanically stabilized. Cement stabilization is further described by Venkatesen and Bapaye (1955). Clare (1957) found that heating a Southern Rhodesian Black cotton soil to 500°C eliminated all plasticity and that vitrified aggregate was produced between 500°C and 700°C. Similar findings are reported by Mehra and Uppal (1964). The Indian soil was made non-plastic by heating at 500°C for two hours. Grainger (1951) found that a Nigerian black clay became nonplastic after heating for one hour above 500°C. The heated soil broke down on impact to give a material whose particle-size distribution was similar to a sandy gravel. Thermal stabilization was also attempted by Radhakrishnan, et al. (1967). It appears doubtful that such treatment would be practical or economic for highways, but could prove advantageous where localized stabilization is desirable.

The influence of soil mineralogical composition on stabilization has been studied by Croft (1967). His findings may be summarized as follows: (1) kaolinitic and illitic soils are more suitable for cement stabilization than soils with expansive clays; (2) heavy clay soils are difficult to stabilize with cement; if expansive montmorillonite is present in quantities sufficient to control the behavior of the soil, cement will probably be uneconomical and lime should be used; (3) cement reduces the plasticity indexes and increases the shrinkage limits of the clay minerals; the changes in expansive clay minerals are not enough to stabilize the clay and are less than changes produced with lime; (4) one of the problems with cement stabilization is the difficulty in obtaining an intimate mixture with the soil; cement does not improve the soil workability as does lime and consequently must be effective with the lump-size material that is available, and (5) the addition of lime before cement will assist stabilization by reducing the size of the clods.

TEST PROCEDURES AND RESULTS

Test results for the black clays sampled in Africa and South America are given in Tables 10.3, 10.4 and 10.5. Chemical and mineralogical data are given in Table 10.3. Index properties and engineering properties are shown in Table 10.4 and swelling tests are reported in Table 10.5.

The samples are believed to be fairly representative of tropical black clays. Both residual and alluvial soils are represented. The residual soils tested were mostly over volcanic rocks, but a few were over basic intrusives and metamorphic rocks. The parent material for each soil tested is given in Table 10.3.

Samples were collected from several locations in Brazil and Uruguay and from the following countries in Africa; Morocco; Ghana; Nigeria; Cameroon; Chad; Sudan; Ethiopia; Kenya; Uganda; Tanzania; and Zambia. Results are included from published and unpublished reports from Colombia, Peru, Ethiopia, Kenya, Sudan and Morocco.

The mineralogy of the samples is basically similar. Montmorillonite is the principal clay mineral reported, but kaolinite, halloysite and illite are also common. Only alluvial soils contain illite and only soils over volcanic rocks contain halloysite. The cation exchange capacities range from 30.07 to 64.90 milli-equivalents per 100 grams. The principal exchangeable cation in most cases is calcium. Magnesium is present in significant quantities, but potas-

TABLE 10.3
Chemical and Mineralogical Properties Tropical Black Clays

Location	Sample No.	Parent Material	Clay Mineralogy	Cation Exchange Capacity in M.E./100 Gram	Exchangeable Cations	pH	Organic Content
Chad Basin	CB-1	Alluvium		34.90	mostly Ca; with Mg; minor K and Na	8.4	<1.0%
Chad Basin	CB-2	Alluvium		32.83	mostly Ca; with Mg; minor K and Na	6.3	<1.0%
Chad Basin	CB-3	Alluvium	kaolinite, illite, montmorillonite				
Chad Basin	CB-4	Alluvium	quartz, kaolinite montmorillonite			6.3	
Nigeria	NBC-1	Calcereous Rocks	montmorillonite, quartz, kaolinite	44.08	mostly Ca; with Mg; minor K and Na		<1.0%
Ethiopia	ETH-1	Basalt	montmorillonite, kaolinite, halloysite	64.90	mostly Ca; with Mg; minor K and Na	7.2	<1.0%
Kenya	KEN-6	Basalt	montmorillonite, kaolinite			8.8	
Kenya	KEN-9	Basalt	montmorillonite, kaolinite	45.02	mostly Ca; with Mg; minor K and Na	7.2	
Zambia	ZAM-1	Alluvium		32.16	mostly Ca; with Mg; minor K, Na, Mn		1.0-1.5%
Zambia	ZAM-2	Basalt		33.34	mostly, Ca; with Mg; minor K, Na, Mn	6.4	<1.0%
Uganda	U-12	Basalt		56.74	mostly Ca; minor Mg, K and Na	7.4	1.0%
Tanzania	TAN-X-1	Alluvium		30.07	mostly Ca; with Mg; minor K and Na	9.2	<1.0%
Tanzania	TAN-3A	Alluvium					
Morocco	MOR-2	Alluvium from basalt	montmorillonite chlorite, kaolinite				
Morocco	MOR-3	Alluvium from shales	montmorillonite chlorite, kaolinite				
Sudan	SU-1	Alluvium from basalt	montmorillonite, kaolinite, illite, quartz, calcite			8.9	
Sudan	SU-2	Alluvium from basalt	montmorillonite illite, kaolinite calcite				
Sudan	SU-3	Alluvium from basalt	montmorillonite kaolinite, illite quartz, calcite				
Sudan	SU-5	Alluvium from basalt	montmorillonite kaolinite, illite quartz, calcite				
Sudan	SUD-D	Unknown	montmorillonite vermiculite, misc.				
West Cameroon	WC-2	Basalt	vermiculite metahalloysite				
Colombia[1]			montmorillonite, some kaolinite quartz	53.5	Na, Ca, K	5.6	
Peru[2]	–	Quaternary sediments	montmorillonite, kaolinite		Ca, Na	7.8-8.2	
Brazil	Minas Sertãozinho Patos		montmorillonite			4.6	
Brazil	Bahia 1	Clay-shale	kaolinite, gibbsite, goethite, hematite			6.85	
	Bahia 2	Clay-shale	montmorillonite, kaolinite, chlorite, gibbsite, goethite, hematite.				

TABLE 10.3 (Continuation)

Location		Parent Material	Clay Mineralogy	Cation Exchange Capacity in M.E./100 Gram	Exchangeable Cations	pH	Organic Content
	Bahia 3	Clay-shale	montmorillonite, kaolinite, illite, chlorite				7.05
	Bahia 4	Clay-shale	–			7.35	
Uruguay	1	quaternary alluvium	quartz, feldspar, illite				0.36
	2	quaternary alluvium	quartz, feldspar, montmorillonite				0.54%
	3	quaternary alluvium	quartz, feldspar, illite				0.46%

(1) Gil, 1969, 1973.
(2) Colombia Ministerio de Obras Publicas.

sium, sodium and manganese occur only in minor amounts. Clays with calcium cations display lower volume change than clays with sodium cations, but they display greater expansion pressures (Donaldson, personal communication). Most of the soils are basic, with pH values varying from 6.30 to 9.20.

There is considerable variation in the index properties shown in Table 10.4. The silt and clay content varies from 58 to 95 percent, while the clay fraction varies from 40 to 75 percent. The mean gradation is given in Figure 10.1. Liquid limits range from 22 to 149 percent and plasticity indexes vary from 11 to 93. Most shrinkage limits are fairly high; in fact these may be suspect since high shrinkage limits indicate little volume change in the material as it approaches the plastic limit. Activities are all below 1.70 and most are under 1.0. Thus, most of the clay would be considered on this basis as "normal" by Skempton (1953). Almost all of the soils are either A-7-5 or A-7-6. The recent revised AASHO procedure (M 145-66) for calculating the group indexes was followed. With this expanded range, the group index values vary from 13 to 89.

Engineering properties include moisture-density relationships, California Bearing Ratios and swelling properties. The first two are listed in Table 10.4, while the various swelling tests results are given in Table 10.5. Both Modified AASHO and Standard AASHO densities and moisture contents are reported for most soils. The Modified AASHO maximum densities vary from 1,367 kg/m^3 (85.3 pcf) to 1,918 kg/m^3 (119.7 pcf) while the Standard AASHO densities range from 1,076 kg/m^3 (67.2 pcf) to 1,630 kg/m^3 (101.8 pcf). The residual black clays over volcanic rocks display noticeably lower densities and higher optimum moisture contents. In this respect they resemble the ferralsols and nitosols soils formed over volcanic rocks.

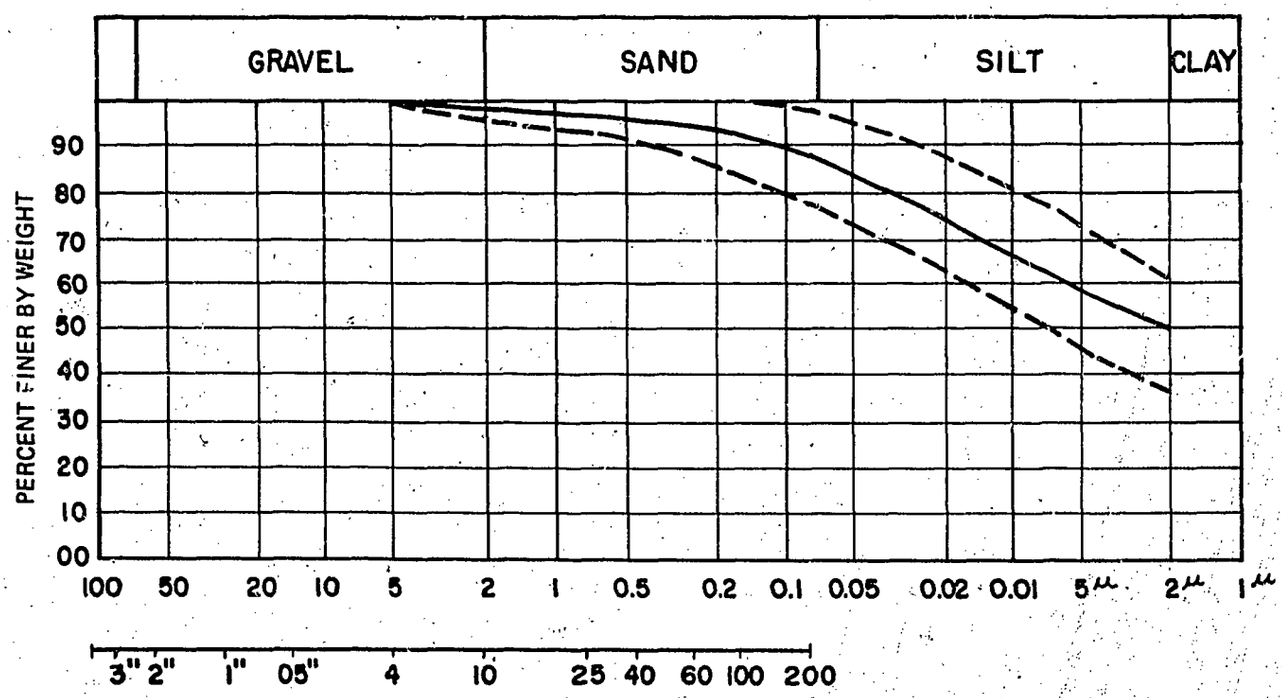


FIGURE 10.1 — GRADATION LIMITS FOR BLACK CLAYS

TABLE 10.4
Physical Properties, Tropical Black Clays

Location	Sample No.	Gradation (%)				Atterberg Limits (%)				AASHO Classifications	Group Index	Activity	Specific Gravity	Standard AASHO			Modified AASHO		
		GRAVEL	SAND	SILT	CLAY	LL	PL	PI	SL					MDD g/cm³ (pcf)	OMC%	CBR	MDD g/cm³ (pcf)	OMC	CBR
Chad Basin	CB 1	0	10	20	70	58	16	42	12	A-7-6	40	0.60	–	–	–	–	1.719(107.3)	20.2	1.6
Chad Basin	CB 2	0	19	28	53	44	14	30	9	A-7-6	24	0.57	–	–	–	–	1.918(119.7)	13.0	
Chad Basin	CB 3	0	14	34	52	56	30	26	15	A-7-6	20	0.50	–	1.432(89.4)	18.9	–	1.660(103.6)	18.6	–
Chad Basin	CB 4	0	10	30	60	52	19	33	15	A-7-5	25	0.55	–	1.437(89.7)	19.3	–	–	–	–
Nigéria	NBC-1	2	11	30	57	86	20	46	–	A-7-6	43	0.81	2.52	1.628(101.6)	24.0	–	1.690(105.5)	20.0	1.5
Ethiopia	ETH-1	2	4	38	56	109	28	81	14	A-7-6	89	1.45	–	1.208(75.4)	32.5	0.5	1.485(92.7)	23.0	1.0
Kenya	KEN-6	8	22	8	62	104	34	70	–	A-7-5	52	1.13	2.28	1.076(67.2)	37.2	–	1.366(85.3)	32.2	1.0
Kenya	KEN-9	0	8	37	55	72	24	48	–	A-7-6	50	0.87	2.47	1.168(72.9)	24.3	–	1.515(94.6)	23.7	0.9
Zambia	ZAM-1	0	14	35	51	49	16	33	16	A-7-6	38	0.65	2.58	1.530(95.5)	18.8	–	1.802(112.5)	16.0	–
Zambia	ZAM-2	0	30	20	50	50	17	33	15	A-7-6	21	0.66	2.61	1.517(94.7)	17.8	–	–	–	–
Uganda	U-12	6	6	32	56	87	24	63	18	A-7-6	61	1.13	2.33	1.328(82.9)	31.4	0.8	1.455(90.8)	24.0	0.8
Tanzania	TAN-X-1	2	19	31	48	60	30	30	–	A-7-5	26	0.63	–	1.610(100.5)	20.21	–	1.735(108.3)	18.7	–
Tanzania	TAN-3A	4	6	50	40	58	16	42	15	A-7-5	40	1.05	–	1.599(99.8)	21.1	0.7	1.830(114.2)	15.3	–
Morocco	MOR-2	0	14	32	54	56	24	32	–	A-7-6	30	0.59	–	1.631(101.8)	24.8	–	–	–	–
Morroco	MOR-3	0	19	25	56	59	31	28	–	A-7-5	25	0.50	–	1.607(100.3)	25.3	–	1.720(107.4)	19.4	–
Sudan	SU-1	–	–	–	–	70	29	41	–	–	–	–	–	–	–	–	–	–	–
Sudan	SU-2	–	–	–	–	58	30	28	–	–	–	–	–	–	–	–	–	–	–
Sudan	SU-3	–	–	–	–	30	22	8	–	–	–	–	–	–	–	–	–	–	–
Sudan	SU-5	–	–	–	–	40	17	23	–	–	–	–	–	–	–	–	–	–	–
Cameroon	CAM-2	1	41	58	–	64	37	27	–	A-7-5	13	1.70	–	–	–	–	–	–	–
Cameroon	WC-2	0	19	43	38	62	35	27	–	A-7-5	26	0.71	–	–	–	–	–	–	–
Cameroon	WC-5	–	–	–	–	85	64	21	–	–	–	–	–	–	–	–	–	–	–
Ghana	GH-4-7	–	–	–	–	47	18	29	–	–	–	–	–	–	–	–	–	–	–
Ghana	GH-4-13	–	–	–	–	22	6	16	–	–	–	–	–	–	–	–	–	–	–
Morroco	NOR-M	–	5-10	25-30	60-70	60+	20+	40+	10	–	–	–	–	1.554(97-100)	–	–	–	–	–
Sudan	SUD-D	–	–	–	–	61	25	36	11	–	–	–	2.80	–	–	–	–	–	–
Ethiopia	ETH-T	–	–	–	–	96-124	–	54-74	10-17	–	–	–	2.87	–	–	–	1298-1362(81-85)	20-31	–
Kenya	KEN-T	–	–	–	75	94	38	55	11.3-16.0	–	–	–	2.63	1.330(83.0)	32.5	–	1.540(94.2)	26.6	–
Colombia	Galapa	0	5	30	65	103	36	67	70	A-7-5	43	1.03	2.73	1.482(92.5)	26.0	2.0	–	–	–
Peru	Talara-Piura	0	6-7	19-36	57-75	118-149	35-56	82-93	20-21	–	–			–	–	–	–	–	–
Brazil	RGS-1	2	2	54	42	80	40	46	12	A-7-5	48	0.95	–	1.309(61.7)	–	–	–	–	–
Brazil	RGS-2	0	5	57	38	58	29	29	16	A-7-5	32	0.76	–	–	–	–	–	–	–
Brazil	RGS-3	0	22	46	22	93	37	56	14	A-7-5	49	1.75	–	–	–	–	–	–	–
Brazil	Bahia-1	8	3	89	89	62	35	27	18	A-7-5	29		2.77	–	–	–	–	–	–
Brazil	Bahia-2	12	3	85	85	74	37	37	13	A-7-5	38	–	2.82	–	–	–	–	–	–
Brazil	Bahia-3	0	0	108	108	55	34	21	12	A-7-5	30	–	2.74	–	–	–	–	–	–
Brazil	Bahia-4	1	15	84	84	51	27	24	14	A-7-6	22	–	2.70	–	–	–	–	–	–
Brazil	Paraná 116	0	33	67	67	64	30	34	–	A-7-5	20	–	2.55	1.572(98.1)	2.26	1	–	–	–
Brazil	Sertãozinho	0	3	97	97	87	53	34	18	A-7-5	46	–	2.94	–	–	–	–	–	–
Uruguay	1	1	15	35	49	56	26	30	16	A-7-6	28	0.61	2.56	–	–	–	–	–	–
Uruguay	2	0	14	39	47	62	33	29	13	A-7-5	30	0.61	2.70	–	–	–	–	–	–
Uruguay	3	0	6	61	33	36	22	14	20	A-7-6	14	0.42	2.91	–	–	–	–	–	–
Uruguay	4	0	7	48	45	48	23	25	11	A-7-6	26	0.55	2.70	–	–	–	–	–	–
Nicarágua	BC	–	–	–	–	93	34	59	–	A-7-5	36	–	–	1.300(81.1)	36	2.4	–	–	–

TABLE 10.5
Swelling Properties, Tropical Black Clays

Location	Sample No.	Expansion Pressure kg/m² (psf)		Swell Index (PVC) kg/m² (psf)		Volume Change %		CBR Swell %		Free Swell %
		At AASHO Mod. MDD AND OMC	At AASHO Std. MDD AND OMC	At AASHO Mod. MDD AND OMC	At AASHO Std. MDD AND OMC	Under 15 kg/cm² 1 (psi)	Load to Retain Vol. kg/m² (psf)	At Std. Dens.	At Mod Dens.	
Chad Basin	CB 1	–	–	28.570(5850)	16.110(3300)	–	–	–	5.3	70
Chad Basin	CB 2	4370(895)	–	–	–	–	–	–	–	55
Chad Basin	CB 3	–	–	21.730(4450)	15.630(3200)	–	–	–	–	50
Chad Basin	CB 4	–	–	29.300(6000)	16.360(3350)	–	–	–	–	50
Nigéria	NBC-1	5030(1030)	2860(585)	25.760(5275)	16.110(3300)	9.0	34.180(7000)	–	5.2	72
Ethiopia	ETH-1	7130(1460)	5620(1150)	34.200(7000)	22.100(4525)	15.8	28.130(16.000)	11.4	12.4	88
Kenya	KEN-6	5490(1125)	3150(646)	34.200(7000)	16.850(3450)	–	–	–	11.5	92
Kenya	KEN-9	7420(1520)	–	26.800(5487)	12.210(2500)	–	–	–	13.4	87
Zambia	ZAN-1	3910(800)	2000(410)	24.300(4975)	14.530(2975)	5.5	19.530(4000)	–	–	57
Zambia	ZAN-2	880(1800)	–	21.360(4375)	9.900(2025)	–	–	–	–	58
Uganda	U-12	7570(1550)	1390(284)	30.030(6150)	21.360(4375)	–	–	14.6	14.5	82
Tanzania	TAN-X-1	3170(650)	1880(385)	19.900(4075)	11.480(2350)	–	–	–	5.7	65
Tanzania	TAN-3A	6020(1232)	1900(390)	24.380(4975)	10.620(2175)	5.2	19.530(4000)	7.0	7.0	62
Sudam	SUD-D	–	–	–	–	–	–	–	–	100
Colombia	(1)Galapa	–	–	–	–	14.0	–	–	–	–
Peru	(2)Talara-Piura	–	–	–	–	–	–	–	–	92
Brazil	Sertãozinho-MG	–	–	17.800(3650)	11.900(2430)	4.0	244(500)	–	–	70
	RGS-1	–	–	–	–	7.4	12.210(2500)	2.9	–	150
	RGS-2	–	–	–	–	23.4	34.180(7000)	–	–	120
	RGS-3	–	–	–	–	–	–	–	–	150
	Bahia-1	–	–	–	–	–	–	–	–	70
	Bahia-2	–	–	–	–	–	–	–	–	120
	Bahia-3	–	–	–	–	–	–	–	–	80
	Bahia-4	–	–	–	–	–	–	–	–	70
	Paraná-6	–	–	–	–	–	–	4.1	–	–
Uruguay	URU-2	–	–	–	–	4.3	12.210(2500)	–	–	75
	URU-4	–	–	–	–	3.7	7.320(1500)	–	–	50
Nicaragua	BC	–	–	–	–	–	–	5.0	–	–

CBR values are all very low and reflect the instability of the black clays. The range is from 0.5 to 0.8 when compacted at AASHO Standard density and from 0.8 to 1.6 when compacted at AASHO Modified density. Relationship between density, moisture content and CBR are shown in Figure 10.2. for three soils. In all cases the CBR values increase with increasing moisture contents, and swelling decreases.

Swelling was determined by means of four different tests: CBR swell, free swell, expansion-pressure and P.V.C. (Potential Volume Change). Expansion-pressure tests were performed in accordance with AASHO T-190-61 with slight modifications as adopted by the Utah State Department of Highways. Expansion-pressures were determined at various densities and moisture contents. The figures reported in Table 10.5 are for tests conducted at both maximum densities and corresponding optimum moisture contents. The results range from 880 to 7,150 kg/m^2 (180 to 1,460 psf) for samples compacted at Modified AASHO density and from 1,390 to 5,640 kg/m^2 (284 to 1,150 psf) for those compacted at Standard AASHO density.

Relationships between expansion pressures and dry densities at initial molding moisture contents are shown in Figure 10.3 for four soils. The increase in expansion pressure with increasing density and decreasing initial moisture content is apparent.

Free swell is reported for most soils. This is an extremely simple test and it was desirable to compare the results with those from other swelling tests. The free swell is determined by slowly pouring 10 cc of air-dried soil passing the 0.425 mm (No. 40) sieve into a gratuated cylinder containing 100 cc of water. After the soil comes to rest at the bottom of the cylinder, the expanded volume is measured. It is calculated as follows:

$$\text{Free swell, \%} = \frac{\text{Final Volume} - \text{Initial Volume x 100}}{\text{Initial Volume (10 cc)}}$$

The results ranged from 50 to 100 percent.

PVC (potential volume change) tests were conducted following the procedure described by Lambe (1960) and Henry and Dragoo (1965), who developed the apparatus for the Federal Housing Administration for a rapid determination of the expansiveness of soils. As these reports may not be readily accessible, the test procedure is reproduced in Appendix 10.1 to this chapter. The apparatus is now available commercially from various suppliers of soils laboratory equipment. The swell indexes were determined at both maximum densities and corresponding moisture contents. This also is a relatively simples test which can be easily incorporated in testing programs. Results varied from 19,970 kg/m^2 (4,075 psf) to 34,300 kg/m^2 (7,000 psf) for soils compacted at Modified AASHO densities, and 9,910 kg/m^2 (2,025 psf) to 22,170 kg/m^2 (4,525 psf) for soils compacted at Standard AASHO densities.

A few volume change tests were performed employing oedometers. This is a very time-consuming test if a series of samples at varying initial moisture contents and surcharge loads is tested. The tests were performed only at Standard AASHO maximum density and optimum moisture content. Expansion was allowed under the normal 15 kg/cm^2 (1 psi) load until equilibrium was reached. The sample was then loaded in increments until the original volume was restored. Volume change varied from 5.2 to 15.8 percent. The results of one volume change test are shown in Figure 10.4. Volume change is plotted against surcharge load; a load of 4,720 kg (10,300 lbs) was required to restore the sample its initial volume.

Since the purpose of the expansion study was to evaluate suitable tests to adopt for highway design, no undisturbed testing was performed. The double oedometer test, devised by Jennings and Knight (1957-58), has been useful is estimating heave under buildings, but undisturbed samples are required.

DISCUSSION OF PROPERTIES

The classifications of the black clays reported in Table 10.4 are similar to those given in the literature. A plasticity chart, Figure 10.5 shows similar properties and both the correlation line and Casagrande's A-Line are shown. The correlation equation is:

$$PI = 0.59\,LL - 2.14$$

A correlation performed by Kassiff, et al. (1969) for Israeli black clays produced a similar equation:

$$PI = 0.75\,LL - 8.9$$

Most of the clays plot above the A-Line. One that plots far below is from Cameroon and is formed over volcanic extrusives.

A few other significant correlations were established. Figure 10.6 shows the relationship between liquid limit and both Modified and Standard AASHO maximum density. The relationship between Modified density and the amount passing the 0.075 (No. 200) sieve multiplied by the PI is shown in Figure 10.7.

The relationship between optimum moisture content and plastic limit is shown in Figure 10.8. Both optimum moisture contents at Modified and Standard AASHO densities are included. Most optimum moisture contents are either near or above the plastic limit.

A modification to Skempton's colloidal activity (1953) was proposed by Williams (1957-58) to classify the soils into very high, medium and low degrees of potential expansiveness. The black clays were plotted on the basis of this classification (Figure 10.9). All but one fall within the very high to high degrees of potential expansiveness.

Swelling in soils depends on the initial moisture content and the subsequent amount of moisture the soil is allowed or able to absorb. This point is considered in greater detail later. However, if a soil is completely desiccated it will exert high expansion pressures and volume change as it approaches saturation. When highways are constructed over expansive black clays, the amount of expansion and swelling pressure depends on the moisture content existing at the time of construction.

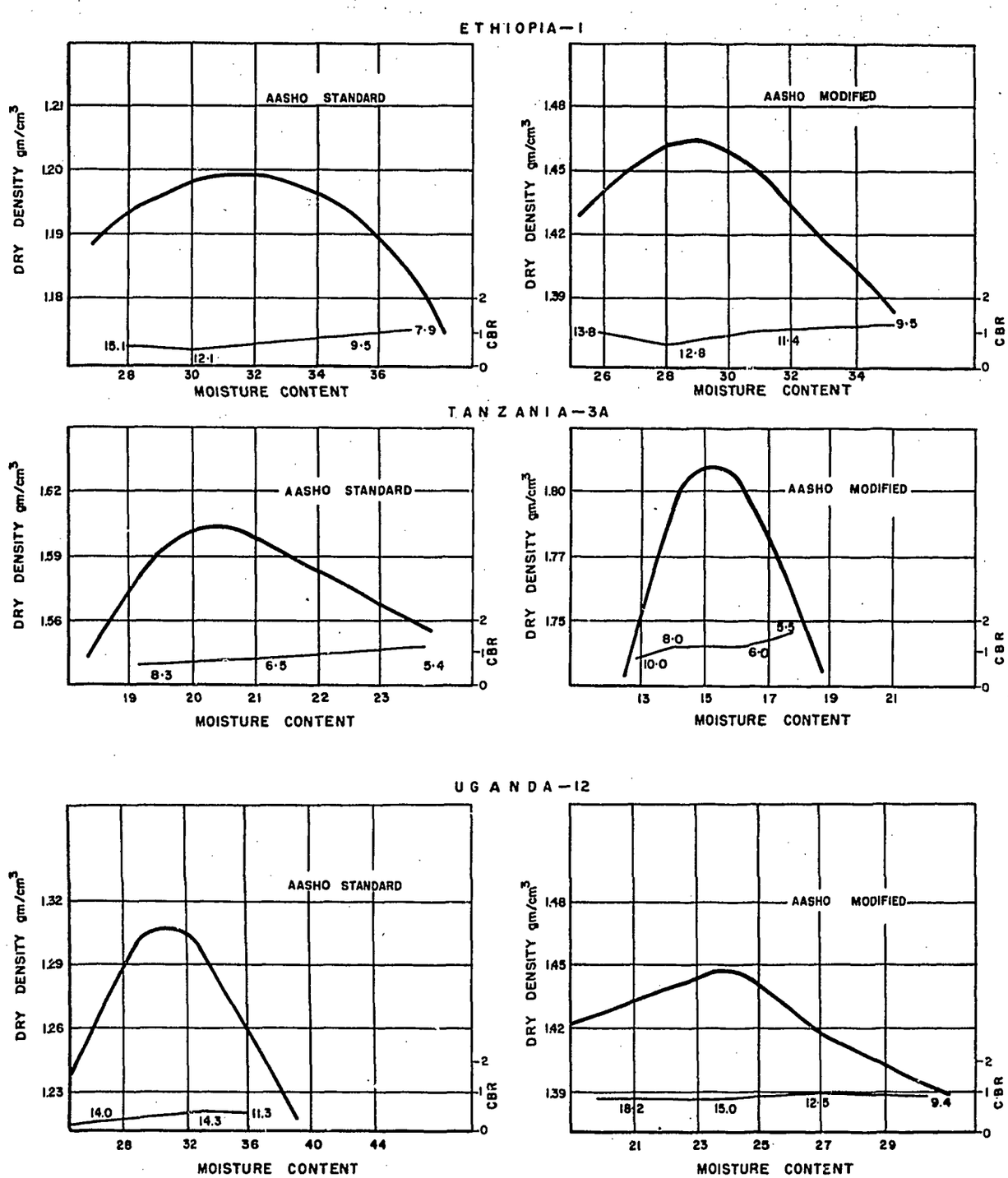


FIGURE 10.2 – MOISTURE-DENSITY-CBR RELATIONSHIPS, AFRICAN BLACK CLAYS (CBR swell shown below CBR curve)

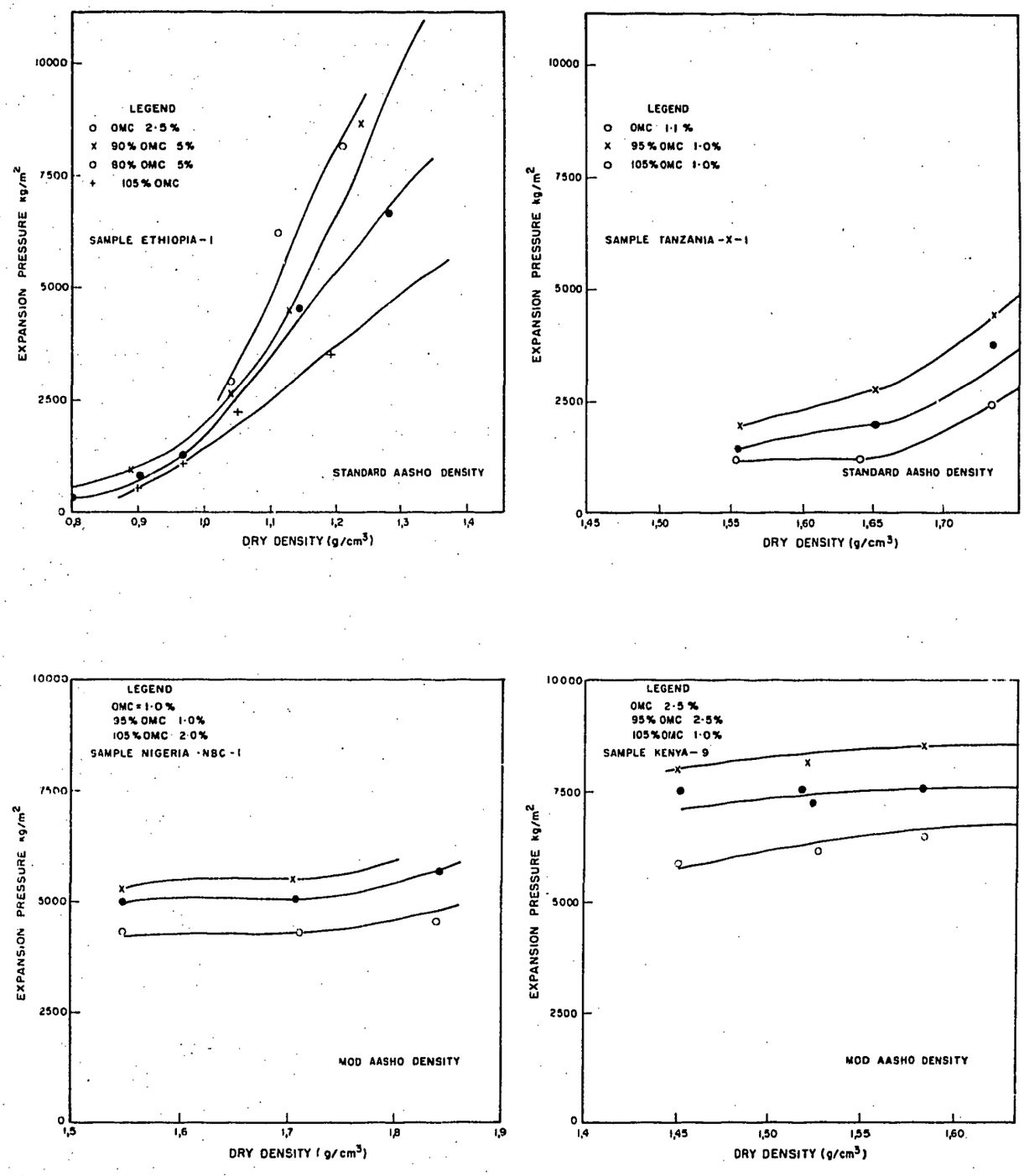


FIGURE 10.3 – RELATIONSHIP BETWEEN EXPANSION-PRESSURES AND DRY DENSITIES AT VARIOUS INITIAL MOISTURE CONTENTS, AFRICAN BLACK CLAYS

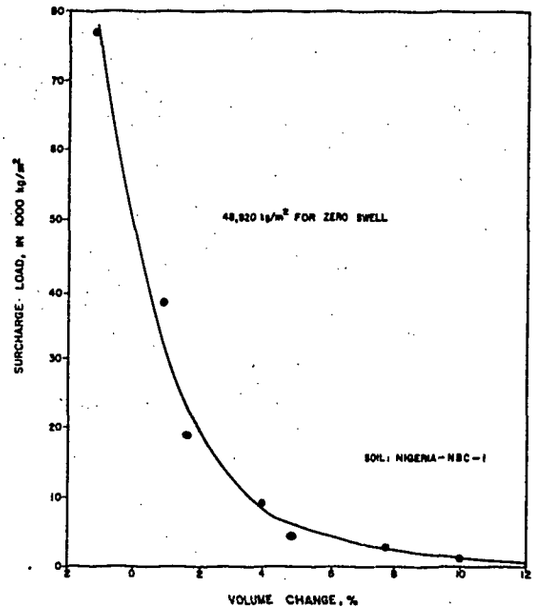


FIGURE 10.4 – EFFECT OF SURCHARGE LOAD ON VOLUME CHANGE

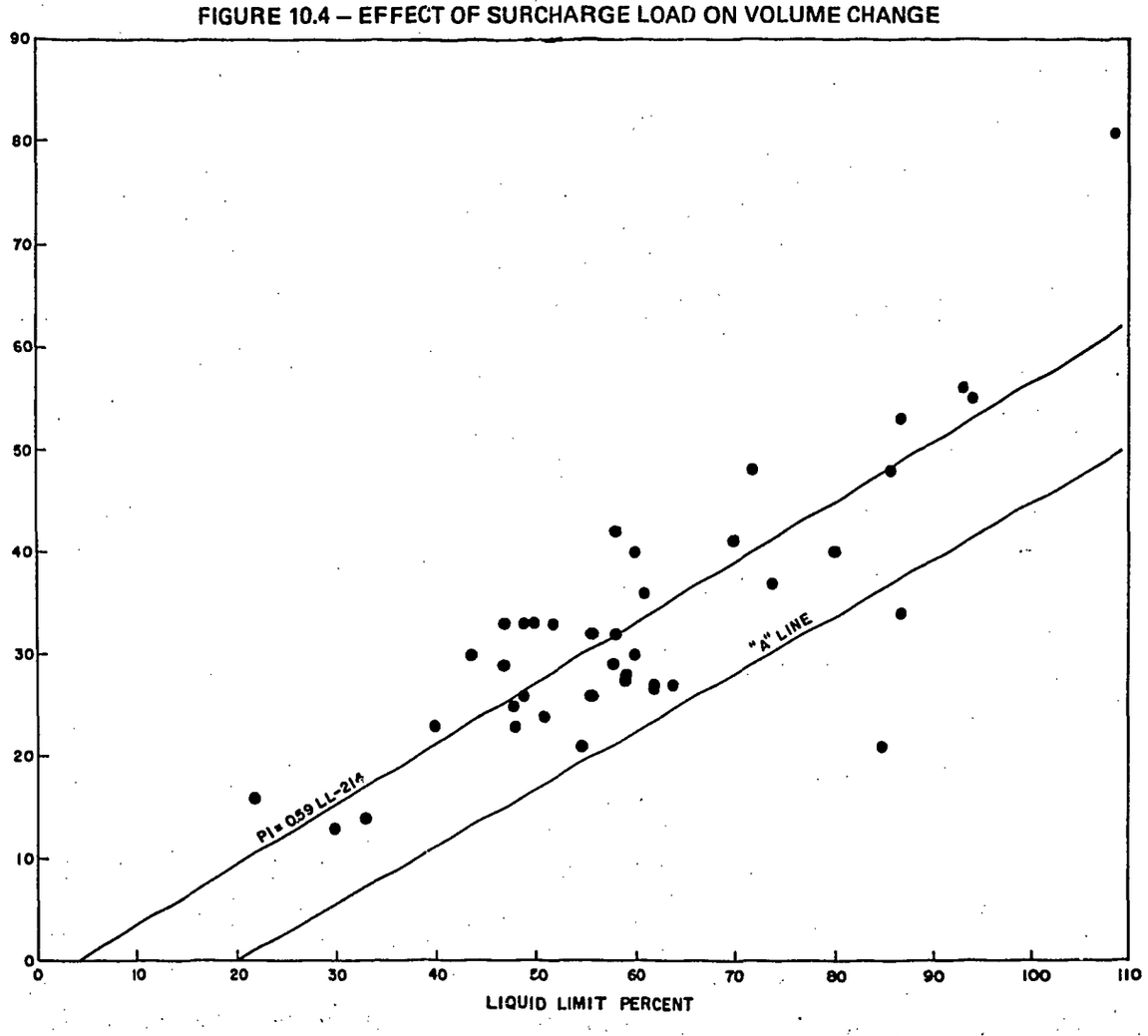
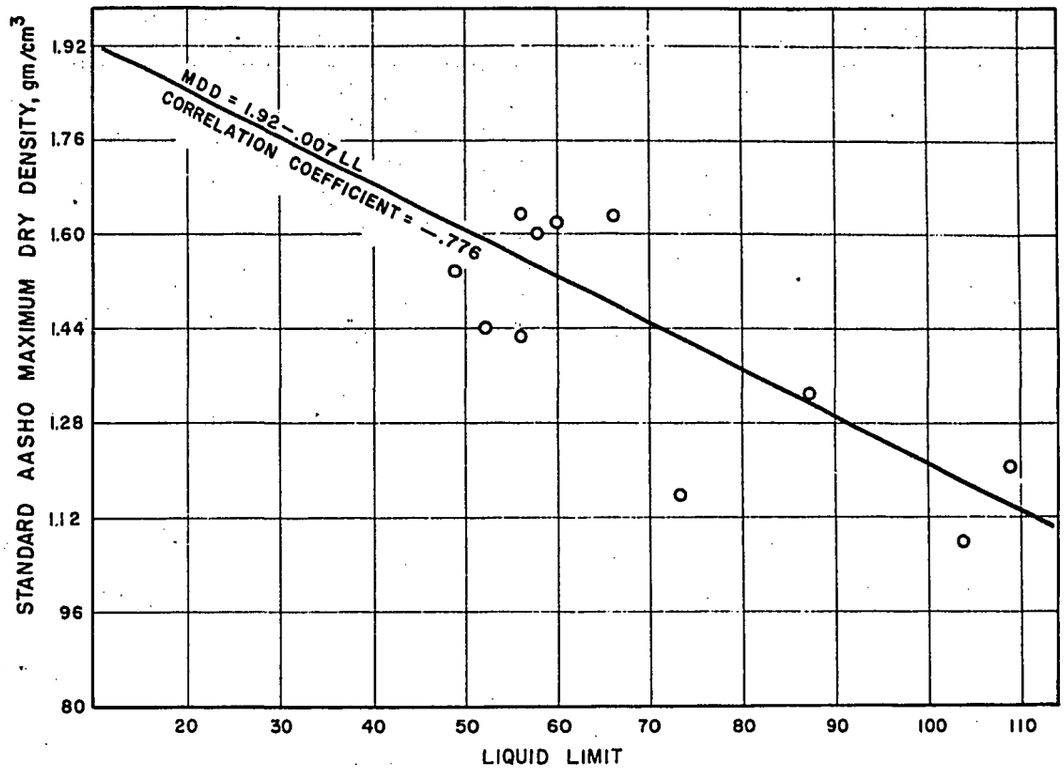


FIGURE 10.5 – PLASTICITY CHART FOR BLACK CLAYS OF SOUTH AMERICA AND AFRICA



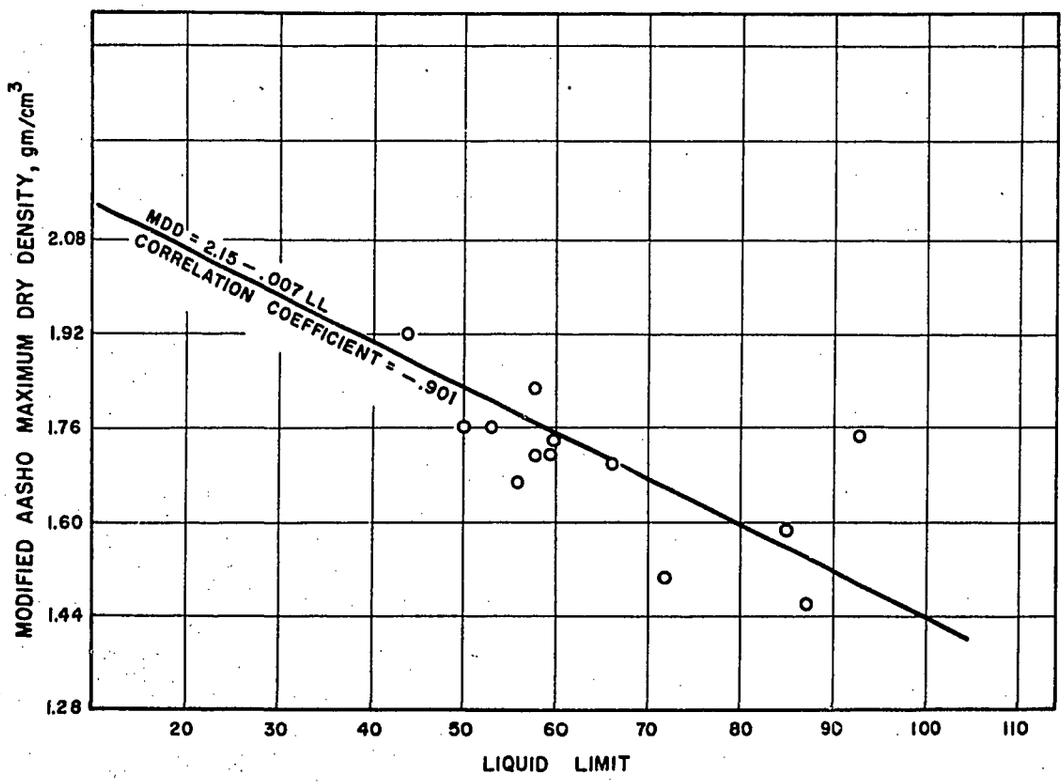


FIGURE 10.6 – RELATIONSHIP BETWEEN COMPACTED DENSITY AND LIQUID LIMIT

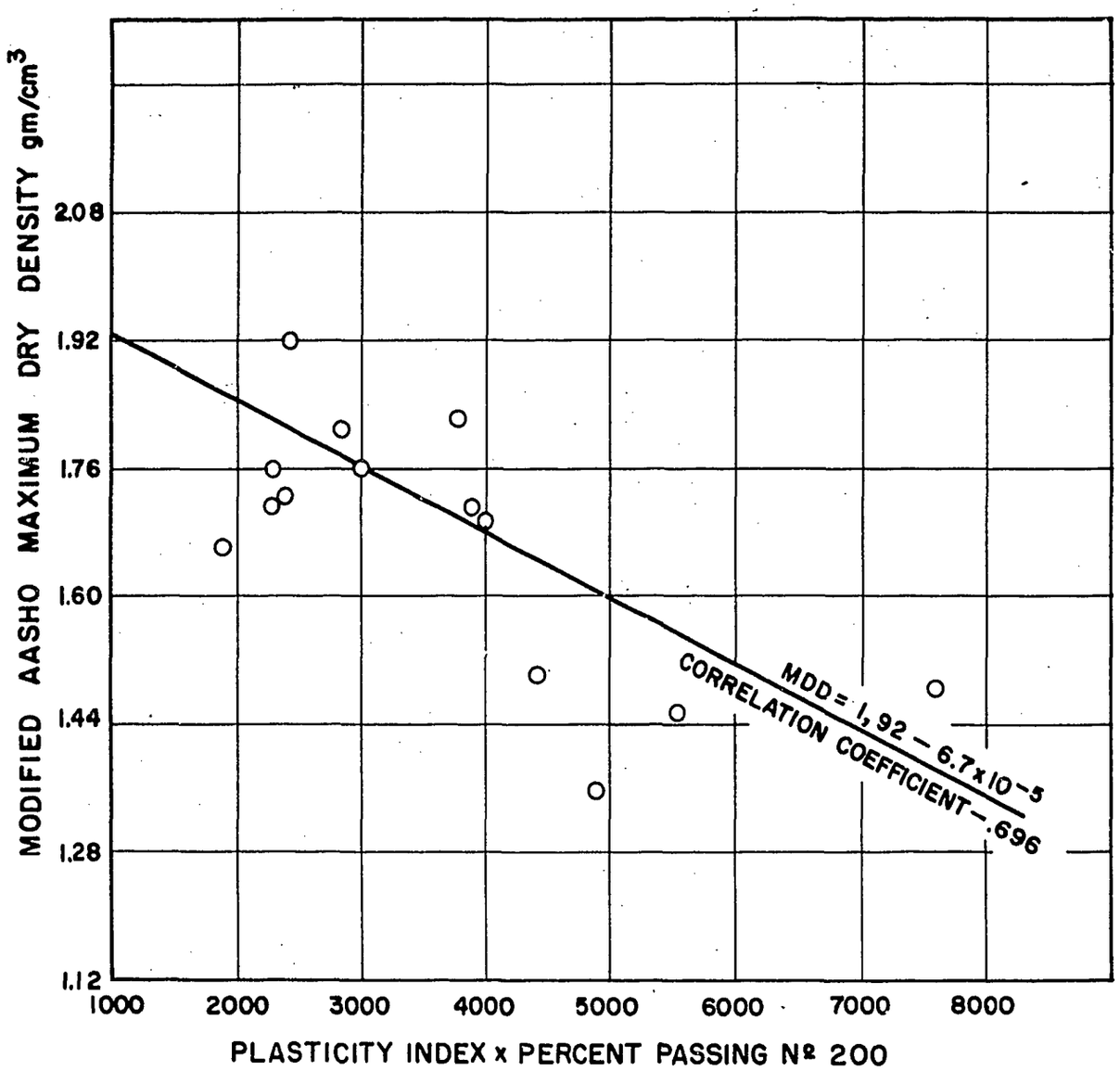


FIGURE 10.7 – RELATIONSHIP BETWEEN COMPACTED DENSITY AND PI x PERCENTAGE OF FINES

The expansion-pressure and volume change tests in the African study were performed on soils at either Modified or Standard densities and optimum moisture contents. These are reasonable initial moisture contents at which to conduct swelling tests for highway purposes, as they should be the moisture-density condition that will be applicable during construction.

Of the swelling tests investigated in Africa, free swell and Potential Volume Change (PVC) at Modified AASHO are considered the best indicators. Correlation coefficients between free swell and liquid limit and plasticity index were 0.669 and 0.652 respectively while correlation coefficients between PVC at Modified AASHO and liquid limit and plasticity index were 0.801 and 0.803 respectively. The relationships between PVC at Modified AASHO and Group Index and between free swell and Group Index displayed fair correlation. The first was 0.772 and the second, 0.782. The expansion pressure at Modified AASHO also correlated well with liquid limit, plasticity index, group index, activity, and plasticity index multiplied by the amount passing the 75 μ m (No.200) sieve. However, correlations are not as significant as with the free swell and PVC at Modified AASHO density.

Neither expansion-pressure nor PVC at Standard density showed very high correlations with the index tests. Expansion pressures at lower densities are evidently more

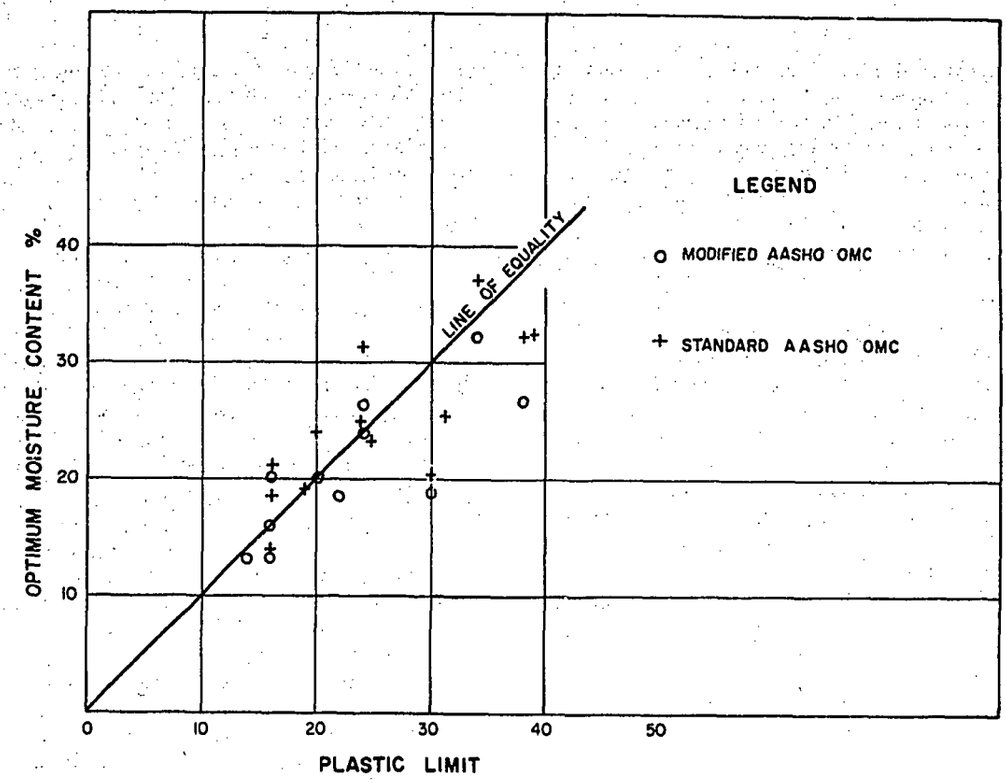


FIGURE 10.8 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND PLASTIC LIMIT

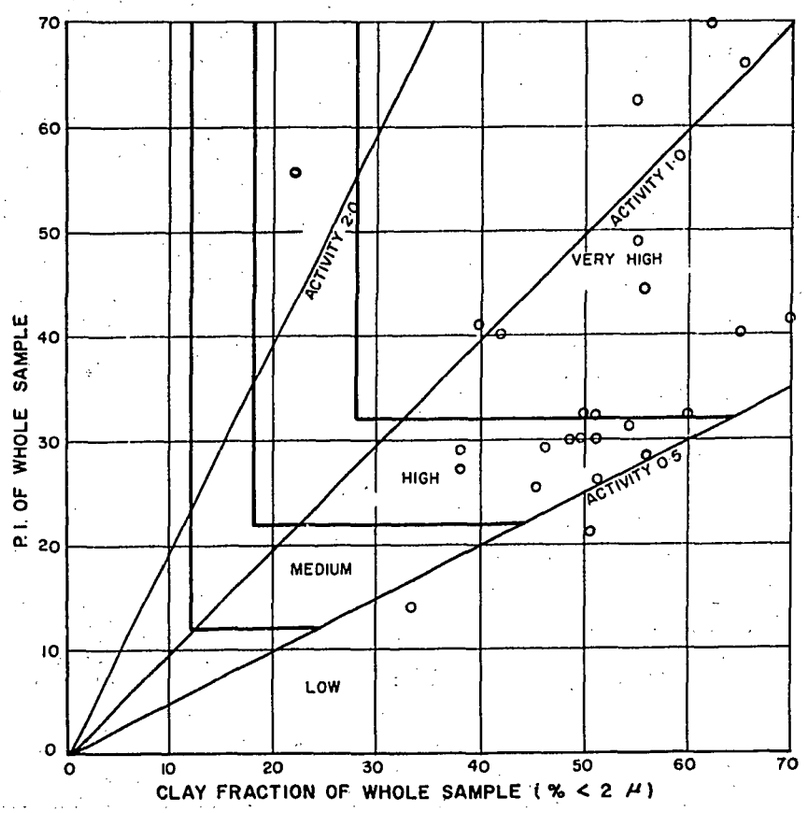


FIGURE 10.9 – POTENTIAL EXPANSIVENESS OF SOILS (after Williams, 1957-58)

erratic. Compacting the soil in the PVC device at Standard AASHO is accomplished with one layer and this may produce a non-uniform density and irregular expansion within the samples during the two hour testing period.

The expansion-pressure test is not considered as practical as the PVC. It requires a considerably longer time, 24 to 48 hours as opposed to 2 hours. Also the 11.3 cm (4.5 in) height of the specimen leads to a considerably longer time for saturation than the 1.9 cm (0.75 in) PVC specimen. As a consequence the expansion pressures are considerably lower than the PVC swell indexes.

The effect of time on expansion pressure was investigated by Seed et al. (1962). They advocate a 7-day testing period to develop complete expansion. Testing one sample, Kenya-9, indicated that 30 days were required to develop complete expansion, and the final expansion pressure was more than double that developed in 24 hours. In fact, in 5 other samples tested, water did not percolate through the specimen in a testing period of 7 days. The final expansion pressures in 4 of the 5 samples was approximately double the 24 hour value. In one, it was 3 times the 24-hour value. From 24 to 30 days were required to develop the full expansion pressure.

A single PVC test on the Nigerian black cotton soil indicated that 8 hours were required to attain full expansion and that the full swell pressure was over twice the 2 hour value. Nevertheless, the standard swell index test is preferred for classification purposes.

It has been well documented that volume change and expansion pressures in swelling soils can be reduced in a number of ways. If a small amount of expansion is allowed, a considerable reduction in expansion pressure and volume change result. Expansion is reduced considerably by surcharge loading and by an amount out of proportion to the intensity of the load. Expansion is also dependent on the initial density and moisture content. These considerations indicate that accepting lower densities and higher moisture contents may be advisable as a means of reducing volume change and expansion pressure.

No single test evaluates all these possibilities. It is felt that the PVC at Modified AASHO density provides an approximation that can be useful for design purposes. A copy of the procedure is included in the appendix to this chapter. Densities and moisture contents can be varied to investigate these effects. The PVC device allows a small amount of expansion to occur (as does the expansion-pressure device) in deflecting the proving bar or proving ring. However, the change in volume is probably considerably less than the amount that would normally be tolerable in a highway subgrade. Surcharge loading cannot be evaluated in the PVC device. The volume change test in an oedometer is believed to be the best means for evaluating this factor.

A suggested procedure that has been proposed by Holtz (1970) is recommended for design purposes. Three specimens are required; the first is saturated and loaded sufficiently to restore it to its original volume. The second is saturated and allowed to expand completely before loading. The third specimen is allowed to shrink through air-drying. These tests can be performed at the moisture contents that apply in the profile in question or after compacting to the required density. It is likely that both would be advisable for the deeper profiles. The shrinkage specimen may not always be necessary for highway design purposes, but it does provide a measure of the total potential expansion of the soil when compared to the other test specimens. A copy of the tentative procedure is provided in Appendix 10.2 to this chapter.

Holtz and Gibbs (1956) provide an estimate of probable expansion based on the index tests, shrinkage limit, plasticity index and the amount smaller than 1 micron. These estimates are reproduced in Table 10.6.

For most black clays, the shrinkage limit was found to be a difficult test to conduct properly and the results correlated poorly with those of the swelling tests. The approximation proposed by Williams (1957-58) and shown in Figure 10.9 is considered more applicable.

The free swell test is also recommended for soils suspected of being expansive. Correlations with index tests are good and the test is extremely simple to conduct. It provides no quantitative measure of either expansion pressure or volume change, but nevertheless is useful in identifying problem soils. In Rhodesia it is used to identify those subgrade soils which have an adverse effect on pavement performance; those over 60 percent are considered critical (van der Merwe and Ahronovitz, 1973).

Other useful relationships include liquid limit, dry density and natural moisture content. Vijayvergiya and Ghazzaly (1973) propose correlations through which swell can be estimated; the first is between liquid limit and

TABLE 10.6
Data for Making Estimates of Probable Volume Change for Expansive Soils (after Holtz and Gibbs, 1956)

Data from Index Test			Estimated[1] Probable Expansion in % (Dry to Saturated)	Degree of Expansion
Colloid Content (% Minus 1 Micron)	Plasticity Index	Shrinkage Limit		
>27	>32	>10	>30	very high
18-37	23-45	6-12	20-30	high
<17	<20	<13	<10	low

1 Based on a vertical loading of 15 kg/cm^2 (1.0 psi) in the one-dimensional consolidation testing machine.

moisture content and the second between liquid limit and dry unit weight. Bara (1969) found that the moisture contents for given liquid limits indicated whether soils are critically expansive or not, Figure 10.10. If a soil has a natural moisture content of 25 percent, the liquid limit can be almost 50 before the soil would be considered critical. Stated another way, if the soils developed over the basalts of the Paraná Basin of southern Brazil were located in the arid northeast, they would be far more expansive.

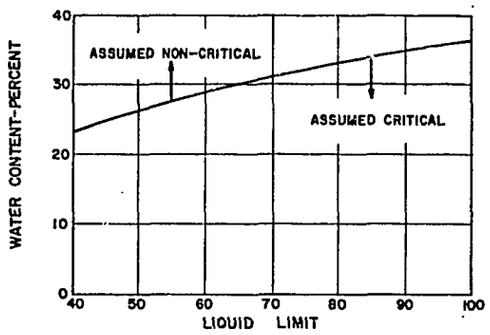


FIGURE 10.10 -- MINIMUM WATER CONTENT REQUIRED FOR SOIL LIQUID LIMIT (after Bara, 1969)

DESIGN CONSIDERATIONS FOR ROADS OVER TROPICAL BLACK CLAYS

General

The tropical black clays (or "black cotton" soils) have been a persistent problem in highway and airfield construction in those parts of the tropics where they are widespread. This is particularly true of those deposits formed residually over basic intrusive rocks such as gabbros or, more commonly, over basic extrusives, such as basalts. Weathering of these rocks is rapid because their constituents are among the minerals least stable under chemical attack. Without resistant mineral crystals to form sand-size material, the rocks tend to weather directly to clay minerals. The predominant clay mineral formed in the early stages, is montmorillonite.

Three problems are associated with black clay soils in highway construction. First, the material is predominantly clay, i.e. principally montmorillonite, which is potentially expansive and may have a very low bearing capacity. Secondly, in the larger residual deposits there is a lack of a suitable natural aggregate where these soils occur, although alluvial deposits generally include layers of sandy materials within the profile. Finally, road construction alters the moisture pattern in the subgrade soils in all but very arid or very humid climates. Surface evaporation is prevented or reduced and the moisture content of the subgrade soil invariably rises. In a montmorillonitic soil, this causes swelling. The swelling, in turn, reduces the bearing capacity. A pavement structure that is adequately designed for the as-constructed subgrade density may fail as the density is reduced through expansion.

There are no inexpensive means of utilizing these soils satisfactorily. At least, no easy way has been found for dealing with these or any other expansive soils that occur throughout the world. The obvious solution is to avoid them wherever possible, or if shallow, to excavate and waste the material. This is rarely possible in large deposits.

A very great thickness of base and surface is usually required, as dictated by low CBR values even in the absence of swelling. Economizing on the thickness of the pavement structure invariably results in greater expenditures later. This has been the experience all too often in many countries. Inadequate designs have resulted in extensive failures; large sections have had to be rebuilt within one or two years.

The various means of dealing with expansive black clays are discussed in this section. Recent design practices are summarized from the literature reviewed and from the interviews conducted during the course of this study. The applications of these practices are discussed for tropical black clays.

Recent Design Practices

Recent work on expansive soils, summarized in the literature review has shown that swelling of expansive clays can be reduced by: (1) accepting lower densities and higher moisture contents; (2) allowing some initial expansion to occur; (3) by surcharging; and (4) preventing changes in moisture content.

Pavement design practices over expansive clays in the U.S. have been described by Sallberg and Smith (1965), and more recently, in the Workshop Proceedings on Expansive Clays and Shales in Highway Design and Construction, sponsored by the U.S. Federal Highway Administration (Lamb and Hanna, 1973). There are four major approaches:

1. Avoid expansive clays by either realignment or undercutting and backfilling.
2. Reduce expansive characteristics.
3. Confine expansive clays under embankments.
4. Minimize moisture changes in the expansive clay after paving.

The first, avoiding the clays, is not often possible in many of the locations where these soils occur. However, a routine swell test, such as the PVC, would at least provide a basis for classifying the expansive sections.

Reducing expansive characteristics has been attempted by compaction control and by stabilization. It has been well documented that compacting to lower densities reduces swell and to compact wet of optimum further reduces swell. The method of compaction that produces greater dispersion also causes less swell.

Ponding water over or pre-wetting the subgrade soil has been successful in the U.S. and South Africa. McDowell (1965) found that ponding water for 30 days over an expansive Texas soil was sufficient to reduce the heave to under one inch. The water did not penetrate deeper than 1.25 m (4 ft) in 24 days but the moisture content at a depth of 6 m (20 ft) began to increase after a few days and the zone of increase progressed upward to 1.25 m (4 ft) within 24 days. McDowell states "All of the reasons why the subsoil can be wetted in spite of ponded water failing to penetrate deeply is not known; however, ponding seems to accomplish the results desired regardless of whether or not gravity water can penetrate deeply". Pre-wetting expansive

clays may provide a solution to prevent localized heaving around culverts. Pre-wetting has also been employed by the Bureau of Reclamation in California (Bara, 1969), where troublesome soils are identified by the relation between their water content and liquid limit. The loss of strength and highly plastic condition of the soil after ponding may require lime treatment of the upper 15 to 20 cm (6 to 12 in) to provide a working platform.

Possible treatments for expansive subgrades have been described by Williams (1965). Clearing the vegetation and allowing the soil to "fallow" reduces water loss due to evapotranspiration and allows moisture to accumulate from rainfall. The moisture content of the subgrade is raised significantly and a large part of the swelling occurs prior to covering with structural courses of the highway. Pre-wetting for some months, with a grid of auger holes, is advisable for culvert sites.

Fallowing to increase the subgrade moisture content and induce heave is a simple and inexpensive procedure to eliminate a major part of the problem of expansive soils. It is particularly adaptable to highways and stage construction. Placing a layer of filter material or a subbase course is particularly beneficial. de Bruijn (1967) found that a 15 cm (6 in) sand over black clay induced most of the heave in one season by permitting the moisture content to increase. The sand prevented evapotranspiration and evaporation and conserved the moisture from precipitation. Sand was found to be more effective than a plastic cover. Kassiff el al. (1965) also found a sand filter to be effective in reducing drying at the edges of the pavement by retaining rainwater and reducing evaporation through capillary rise. Jennings (personal communication) believes 5 or 6 years are required for heave to reach equilibrium under structures. However, he considers that allowing the soil to fallow for one year, particularly with a granular cover, can accelerate heave by conserving moisture, and that it is quite likely that most of the heave will occur in 1 or 2 years.

Reducing expansive characteristics by chemical stabilization has been attempted successfully. In plastic clays, hydrated lime is usually more successful than cement. Lime improves the workability of the soil while cement does not; moreover, montmorillonitic clays retard the hydration of cement. If the black clays have appreciable organic matter, about 5 percent or more, stabilization may not be possible. Williams (1965) believes lime-stabilization is beneficial for a number of reasons: (1) it reduces potential swelling in the top layer of the subgrade which would contribute most toward total heave; (2) more water has to be added than for the untreated soil, which helps to raise the moisture content nearer to final equilibrium moisture content; (3) the treated layer is more impermeable, so moisture variations are reduced; and (4) the workability of the soil is improved, particularly during the wet seasons. The limited stabilization study on black clays is reported further below.

Deep-plow stabilization with lime has been effectively used on recent highway projects (Thompson, 1972). Stabilized depths of 60 cm (24 in) have been successfully completed and it may be possible to go as deep as 90 cm (36 in). Quality control is important on such projects.

No other chemical is as effective as lime for expansive soils. However, all trademarked chemicals have not proved to be of particular value as stabilizers.

The third point, confining clays under embankment material, assumes that suitable embankment material is readily available. On the Ethiopian Plateau in Africa, for example, suitable material is scarce and much of the readily available embankment material may itself be expansive. Surcharge does reduce swell by an amount greater than its weight. Williams (1965) estimates that heave can be reduced by about 30 percent for every 1 m (3.3 ft) of fill.

Minimizing moisture changes is undoubtedly a very important aspect. A recent investigation by the Road Research Laboratory on pavement design over tropical soils (O'Reilly et al., 1968) showed the importance of eliminating any design features which allow the accumulation of moisture in the pavement structure. Adequate provisions must be made for surface drainage; cracks must be filled and the surface maintained.

Drainage is particularly important in highways over black clays. If drainage is poor, seasonal variations in the subgrade moisture are unavoidable and heaving results. However, if drainage ditches are too deep, or too close to the pavement structure, seasonal drying or partial desiccation will occur along the shoulders. Kassiff et al. (1969) recommend extending the shoulders to a width equal to the depth of the active zone and locating the surface drainage as far away from the pavement as practicable. Recent studies (Sokolov and Amir, 1973) have shown by means of the linear diffusion equation that seasonal moisture changes penetrate under the pavement as much as 3/4 of the thickness of the swelling layer. This has been confirmed by field measurements.

The compactive effort applied to expansive soils and the corresponding optimum moisture content should correspond to Standard AASHO (or British Standard) rather than to higher density standards. O'Reilly et al. (1965) state that:

> "Field measurements indicated that either the optimum moisture content in the British Standard compaction test or the plastic limit would provide a useful guide to the moisture conditions under bituminous surfaced pavements when these conditions were not governed by the water-table or where the drainage arrangements for the pavement layers were not defective."

They recommend compaction to at least 90 percent B.S. maximum dry density and preferably 95 to 100 percent. Kassiff et al. (1969) state that swelling can be minimized by compacting clay above the plastic limit to 90 percent Modified AASHO density.

The final moisture content beneath pavements has been discussed by Kassiff et al. (1969). Soil conditions are divided into three categories:

1. groundwater level close to ground level.
2. deep groundwater level and a yearly rainfall of over 250 mm (10 in).
3. arid climatic conditions and a yearly rainfall of less than 250 mm (10 in).

In the first category are pavements over clay subgrades with a groundwater depth of 5 to 6 m (17 to 20 ft). In this case the water table controls the moisture conditions in the subgrade. For clays in the second category, field measurements have shown that the final moisture contents are close to the plastic limit or to the optimum moisture content at

Standard AASHO compaction, which is usually close to the plastic limit. In the final category are those areas that are so arid that there is no moisture accumulation in the subgrade due to capillary rise. Similarly, in those areas with very humid climates, the subgrade is usually close to the equilibrium moisture content.

The use of membranes as impervious blankets has increased in recent years in the western U.S. with generally good results. Catalytically blown asphalt about 4.8 mm (3/16 in) thick is extended all the way across the subgrade from ditch line to ditch line.

Lime stabilization also serves the same purpose of waterproofing. Such treatments should also extend completely across the shoulders. Otherwise seasonal settlement of the shoulders due to shrinkage causes longitudinal cracks to form which readily admit water.

It has been stated that the CBR values for black clay soils are always low; the results of tests in this study after the four-day soaking period are all under 2. It has been found in South Africa that the in-situ CBR values improve with time. Measurements on a number of black clay subgrade soils in roads up to 15 years old were always 6 or over (Williams, personal communication). An explanation of this gain in strength with time is that the initial water content introduced during compaction partially fills voids and is adsorbed on mineral and soil particle surfaces. With time, some of the adsorbed water becomes absorbed or structurally-held within the clay mineral lattice. Also, adsorbed water, being a dipole molecule, becomes electrically oriented with time and, once oriented, is more firmly held on the clay mineral surfaces. In either event, the strength increases. The tendency to gain strength appears to be another advantage to stage construction, whereby subgrade CBR values can be expected to improve somewhat with time.

Stabilization

A limited stabilization study was conducted on a few black clay soils. Lime was the only additive investigated because experience indicates that lime is far better for clay soils than is cement. The lime content was determined by the quick pH test procedure devised by Eades and Grim (1966). In most soils, 6 percent lime was found to be the optimum content for stabilization. Two soils required 7 percent and two only 4 percent.

It can be seen from the results given in Table 10.7 that in all cases the strengths improved considerably. The unconfined compressive strength of one sample, NBC-1, decreased with lime and with curing time, but the CBR value increased. Decreases in unconfined compressive strength of treated clay soils can be attributed to the loss of cohesion; this is not uncommon. In these cases, the CBR test is a better representation of the true gain in strength.

Expansiveness was reduced in all samples. CBR swell values were 0 or slightly over 0. Expansion pressures after stabilization were 10 percent or less than the comparable untreated values. Although the expansion pressures were reported after only 14 days, all samples had reached equilibrium.

RECOMMENDED DESIGN PROCEDURE

It is recommended that the following additions be made to the normal soil survey procedures for highways over black clay soils and to the design considerations.

The depth of the troublesome clay layer should be determined at sufficiently close intervals to provide a reasonably accurate picture of its stratigraphic position. This may be accomplished by normal hand or machine auger borings. The expansiveness of the clay should be determined along the length of alignment and with depth.

The index properties can be used for this purpose, using Williams' chart (Figure 10.9) as an approximation of the potential expansiveness. The expansiveness of a few selected samples can be checked by means of the free swell and PVC tests.

TABLE 10.7
Test Data on Lime Stabilization of Black Clay Soils

Sample	Compaction S = Standard AASHO M = Modified AASHO	CBR Untreated	Percentage Lime	CBR After 7 Days	CBR After 28 Days	CBR After Swell	Unconfined Compressive Strength Untreated	Unconfined Compressive Strength 7 Days	Unconfined Compressive Strength 28 Days	Expansion Pressure 14 Days
NBC-1	S	–	6	19.8	54.0	0	6.0(85)	4.8(68)	3.9(56)	210(43)
ETH-1	M	1.0	6	22.5	25.0	0.05	2.1(30.4)	3.8(54)	4.7(66.3)	703(144)
U-12	S	0.8	6	13.2	19.2	0.05	0.8(11.5)	1.4(19.4)	3.8(53.7)	576(118)
CB-3	M	–	6	27.4	99.2	0	3.2(45.6)	7.3(104)	7.2(103)	–
TAN-3A	M		6	70.4	79.4	0	6.0(85.0)	7.5(106)	–	146(30)
KEN-9	S	–	6	–	–	–	1.3(18.5)	–	6.7(96)	0(0)
KEN-6	M	–	7	–	–	–	2.2(33.2)	–	10 (143)	–
RGS-1	S	–	7	–	–	–	8.0(114)	–	20.5(292)	–
Paraná 116	S	1	4	–	–	–	8.0(114)	–	23.8(339)	–
Nicaragua BC	S	2.4	4	17	–	–	–	–	–	–

Unconfined Compressive Strength kg/cm^2 (psi)
Expansion Pressure kg/m^2 (psf)

The natural moisture contents and water table depths should be determined. These are important in all soil surveys but particularly important in expansive soils. Investigations for the water table should be carried to depths of at least 6 m (20 ft). Low lying, poorly-drained areas with high water tables should be avoided.

The potential total heave can be approximated in accordance with the method proposed by D. H. Van der Merwe (1964) which makes use of Williams' chart. The potential unit heave at ground surface is assumed to be 25 mm (1 in) for each 30 cm (1 ft) of depth of very highly expansive soil; 12.5 mm (0.5 in) for each 30 cm (1 ft) of highly expansive soil; 6.2 mm (0.25 in) for each 30 cm (1 ft) of medium expansive soil; and 0 mm for low or non-expansive soil. The potential heave is given approximately by:

$$H = \sum_{D=30\text{cm}}^{D=n} F_D (\text{P.E.})$$

where P.E. = potential expansiveness from Figure 10.9.
F_D = the factor by which heave decreases with depth.

The values of F are given in Table 10.8.

The potential expansiveness of all the soil layers are then added to determine the total potential heave. As an example, if only depths 0.3 to 1.2 m (1 to 4 ft) are of concern and the soil is highly expansive, the total heave could be estimated by:

0.3-0.6 m, F_2 = 0.842 x 12.5 = 10.525
0.6-0.9 m, F_3 = 0.750 x 12.5 = 9.375
0.9-1.2 m, F_4 = 0.668 x 12.5 = 8.350
Total heave = 28.25 mm

This approximation does not take into account the natural moisture content or what the ultimate moisture content might be. If the water table is below 6 m (20 ft) and the natural moisture content is near optimum, the approximation is too conservative. Nevertheless, a total potential heave of over 2.5 cm (1 in) should be considered detrimental for highways and special design procedures or relocation of the alignment should be investigated.

Stage construction and allowing the subgrade to lie fallow for one year should be adopted whenever possible. A granular cover should also be provided not only to conserve moisture, but also to act as a filter. Kassiff et al., (1969) have found that quarry material in the 1 cm (3/8 in) to 2 mm (No. 10) sieve size is the most suitable filter size over cohesive soils.

Standard AASHO requirements for compaction and optimum moisture content, rather than Modified AASHO requirements, should be specified. The lower density and higher moisture content will reduce potential swelling. The lower density does not appreciably affect bearing capacity. The CBR values at both compactive efforts are very low. Moisture contents above optimum do not cause lower CBR values. Therefore, there is no advantage in specifying greater compaction.

The Tropical Design Procedure described in Chapter 7 can be applied to black clay soils. The minimum CBR value in these curves is 2. From the pavement performance survey, it was concluded that subgrades with CBR's less than 2 contribute no appreciable strength to the pavement structure.

For expansive soils the minimum cover should be 1 m (3.3 ft). The minimum thickness design then applies only to the subbase, base and surface thicknesses in those cases, or to the structural courses. For example, a non-expansive subgrade exhibiting a design CBR of 2 requires a minimum cover of 50 cm (20 in). However, an additional 50 cm (20 in) of select material is required as the subgrade is expansive. The select material may be of any quality as long as it is not expansive. The minimum thicknesses of the overlying courses are then based on the CBR value of the select material which are included as part of the minimum 1 m (3.3 ft) of cover.

Very coarse material should not be placed directly on expansive soil, or any plastic clay for that matter. An overlying sand layer should be placed to prevent pumping of the fines into the structural layers. The sand should meet the normal filter requirements in grading.

TABLE 10.8
Value of Factor F With Depth D From the Relation D = 20 log F

Depth in m.	Depth in ft.	Mean Value of F		Depth in m.	Depth in ft.	Mean Value of F	
0-0.3	0-1	F_1	0.943	4.6-4.9	15-16	F_{16}	0.168
0.3-0.6	1-2	F_2	0.842	4.9-5.2	16-17	F_{17}	0.150
0.6-0.9	2-3	F_3	0.750	5.2-5.5	17-18	F_{18}	0.133
0.9-1.2	3-4	F_4	0.668	5.5-5.8	18-19	F_{19}	0.119
1.2-1.5	4-5	F_5	0.595	5.8-6.1	19-20	F_{20}	0.106
1.5-1.8	5-6	F_6	0.531	6.1-6.4	20-21	F_{21}	0.094
1.8-2.1	6-7	F_7	0.473	6.4-6.7	21-22	F_{22}	0.084
2.1-2.4	7-8	F_8	0.422	6.7-7.0	22-23	F_{23}	0.075
2.4-2.7	8-9	F_9	0.376	7.0-7.3	23-24	F_{24}	0.067
2.7-3.0	9-10	F_{10}	0.335	7.3-7.6	24-25	F_{25}	0.060
3.0-3.3	10-11	F_{11}	0.298	7.6-7.9	25-26	F_{26}	0.053
3.3-3.6	11-12	F_{12}	0.266	7.9-8.2	26-27	F_{27}	0.047
3.6-4.0	12-13	F_{13}	0.237	8.2-8.5	27-28	F_{28}	0.042
4.0-4.3	13-14	F_{14}	0.211	8.5-8.8	28-29	F_{29}	0.038
4.3-4.6	14-15	F_{15}	0.188	8.8-9.1	29-30	F_{30}	0.034

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CHAPTER 11
AIR PHOTO-INTERPRETATION

INTRODUCTION

The technique of interpreting imagery obtained from remote sensors is becoming more important as the pace of worldwide development increases. The technique is particularly useful in developing generalized maps of existing ground conditions in regions where either existing documented data is not available or where ground surveys are impractical because of time and budgetary constraints.

Clearly documented data is preferable. For example, standard geological maps and agricultural (pedological) soils maps along with their supporting reports are useful to the engineer, since he can rely on past experience to identify the probable engineering and construction problems associated with the different ground conditions identified on these maps. However, when these maps are not available the engineer must depend upon generalized reports to establish his preliminary plans and to set up his detailed surveys. Under these conditions imagery from various remote sensors can be very useful to the engineer, especially if the imagery is to a fairly large scale and it is already available at a relatively low cost.

Types of Imagery

The imagery that has been most often used in the past and will undoubtedly continue to be the most widely used by soil engineers for mapping purposes has been aerial photography. Most of the aerial photography available today is the monochrome or black and white format. It is usually available in a variety of scales ranging from 1:5,000 to 1:100,000. Scales on the order of 1:15,000 to 1:25,000 are generally the most useful for detailed interpretation of terrain characteristics. Nevertheless, smaller scale photography in the range of 1:50,000 can also be utilized to great advantage if nothing larger is available. Numerous publications have appeared in the last 15 to 20 years on the interpretation of black and white aerial photography. One of the most useful is now being revised. It is the *Manual of Photo-Interpretation* published by the American Society of Photogrammetry.

Color photography has become more widely used also, particularly in the last ten years. In some areas of the world today, color photography may be available at about the same scales as mentioned above. The *Manual of Color Aerial Photography* has also been published by the American Society of Photogrammetry. Where color photography is not already available, it may be flown for a specific project for often its cost is only slightly higher than panchromatic, because a major portion of the aerial photography cost is related to the cost of the aircraft and crew time expended in obtaining the photography.

Color infrared (IR) of "false color" photography is also frequently used. There are indications that in many cases IR photography is of greater value than natural color for the interpretation of the geologic situation. IR photography is usually exposed in approximately the same range of scales as panchromatic also, but the film is not readily available in most areas of the world. Therefore, it would be necessary to contract for this special photography.

Cloud cover severely limits all types of photography in tropical countries especially during the long wet season. Thus, the periods when aerial photographs can be taken are limited and may be at variance with project requirements. It is undoubtedly for this reason that recently, more and more use has been made of radar imagery since it can be obtained in all but the most severe weather conditions. The Radam project in Brazil has been working for several years with radar imagery at a scale of 1:250,000. In some respects, radar imagery interpretation is similar to aerial photography interpretation. It is especially useful in making topographic maps where cloud cover exists during much of the year. The extensive use of the information obtained from their Radam Project in Brazil will be discussed again in another section of this chapter.

Another relatively new type of imagery is now available from the multispectral scanner located in the Earth Resources Technological Satellite (ERTS-1) which is currently in polar orbit around the earth. The multispectral scanner operates in various wave bands and scans continuously as it orbits the earth. It produces tremendous amounts of data which can be utilized for various interpretation purposes. Each image scanned by the satellite covers an area about 100 by 100 nautical miles (185 by 185 kilometers) so that the scale of the imagery is on the order of 1:750,000. This imagery, however, can usually be enlarged several times with little loss of definition. Furthermore, the available information can be analyzed without actually being transformed into an image. This is accomplished by working directly with the computer compatible magnetic tape (CCT). The scanner cannot penetrate cloud formation so much of the imagery coming back from ERTS-1 is useless as far as terrain analysis is concerned. On the other hand, all portions of the earth's surface are scanned every 18 days. In addition there is also a considerable overlap of adjacent scanning orbits, thus the opportunity to obtain almost cloud-free imagery even during the rainy season is possible. The utility of this imagery is limited primarily by the ability to retrieve it from the satellite and to process it. At the present time, only United States and Canada have receiving stations which can retrieve the information and make it available to the rest of the world. However, Brazil is now in the process of building a receiving station while Italy and perhaps central Africa will have stations in the near future. These additional stations will greatly speed the processing time of imagery in all regions. It can therefore, be anticipated that more and more emphasis will be placed on interpretation of satellite imagery, especially as its capabilities are recognized outside of the United States and Canada.

PREVIOUS IMPORTANT WORK

Numerous terrain classifications based primarily on terrain evaluation procedures have been developed in the

last decade or so. All of these depend heavily upon the interpretation of remotely sensed imagery, usually aerial photography. For example, in South Africa, the National Institute of Road Research (1971) has developed a system of land classification largely patterned after the British scheme. The Institute is collecting information in the form of a data bank to correlate with the various land classification patterns. Similar work has been carried on in Australia under what has been called a "pattern, unit, component evaluation" (PUCE) again utilizing primarily a land classification scheme based on the study of aerial photographs with particular emphasis on vegetation and rock type (Aitchison and Grant, 1968). Lawrence (1972) has also described the use of aerial photography in the tropical situation of West Malaysia.

In addition to the Radam project previously mentioned, a great deal of work has been done which pertains directly to terrain evaluation in South America. Ray (1963) cited its usefulness in making geologic maps. Vera (1964) described the experience on an aerial photographic project conducted through the cooperation of the Pan-American Union in Chile. Liang (1964) summarized the characteristics for the interpretation of tropical soils in a special report for the United States Air Force. The system developed by Liang is shown in Table 11.1. Ricci and Petri (1965), described the principles of airphoto interpretation from the geologic standpoint and Condori (1967) gave an up-to-date description of the principles of photo-interpretation. One of the most recent publications is that of the Brazilian Coffee Institute, Photo-Interpretation Service of the Executive Group of Rationalization of Coffee Culture (IBC-GERCA, 1972). The techniques which are commonly employed in the photo-interpretation procedure are outlined very well in this latter publication and for this reason will be only briefly reviewed here.

Techniques Employed

In the procedure commonly followed, the first step of the geologic analysis was to look at the photo mosaic of the area and to delineate regions of similar geomorphology, drainage patterns and land use, since these are normally related to pedologic aspects. In the pilot area of Minas Gerais (IBC-GERCA), photos were 1:60,000 panchromatic.

The second step was to sketch the major lithologic units which correlated with known soil types. This was done by studying all available geologic literature for the area, mapping out the major units from the characteristics mentioned above; and then locating appropriate field sampling sites from visible outcrops and situations which require boundary clarifications. The IBC group stated that particular emphasis was given to geomorphic features, drainage patterns, structural features, evidences of lithology, such as stratification of sedimentary rock, and other criteria, such as type of vegetation and methods of cultivation.

The third step was to take the map into the field and revise it so that it correlates with field observations and findings of rock samples analysis.

Finally, the fourth step was to prepare a geologic map initially at a scale of approximately 1:50,000, and later reduced to 1:200,000 on the basis of the correlations between the field studies, the photo-interpretative mapped units and the information concerning the geologic periods.

The second analysis in the process was to prepare a sketch of the vegetation situation. First, various vegetative areas were identified on the 1:60,000 photography. The four units mapped were the forest; the cerrado, which is essentially a twisted or stunted forest and grass; the grassland areas; and the cultivated fields. These were delineated on the basis of image structure, image texture, qualities of the shapes of the boundaries, color tones, and the topographic position. Finally on the basis of the field investigation, the vegetation map was prepared and finally corrected.

The final analysis was to prepare the soils interpretation map. Photo pattern elements, particularly those observed with stereo-pairs, were utilized in order to identify the following map units in the pilot area: (1) soils with a latosolic B, (2) soils with an argillic B, (3) weakly developed soils, (4) soil associations and (5) a lowland complex. Each unit was described in accordance with Tables 11.2 and 11.3 which are commonly used in photo-interpretation to describe the geomorphology, drainage and pedology of the area.

Next, a preliminary soil legend was prepared through a soil exploratory survey which was made along all existing roads and lanes throughout the area under investigation. Numerous soil profiles were studied, samples were taken of both soil and rock and then brought to the laboratory for testing.

Next the corrected soil map was prepared by revising the preliminary map based on photo-interpretation techniques and then by delineating the new map units on the 1:60,000 photography.

Finally, all data was transferred to an uncontrolled mosaic, again to an approximate scale of 1:57,000. From this a manuscript map at a scale of 1:100,000 was prepared.

Significant conclusions were drawn on the basis of this pilot study. They are very similar to those which have been previously enumerated:

1) photo interpretation allows for an almost global planning of field activities;
2) the field control conducted with the use of aerial photographs and of photo-interpretation techniques is highly satisfactory and allows for the preparation of a considerably detailed survey map;
3) time saving and precision which are gained in delineating the mapping units have been adequately confirmed through field investigations;
4) rock and vegetation photo-interpretation provide useful information for mapping the soil;
5) the interaction of photo-interpretation with field and laboratory activities lead to a sound mapping precision.

The authors of the pilot study concluded finally,

"It is our opinion that the methodology here used has presented sound results and therefore met the proposed objectives for its application."

TABLE 11.1
Airphoto Interpretation of Tropical Soils — Summary
(after Liang, 1964)

Soil Groups	Topography (land Form)	Drainage	Erosion	Gray Tone	Vegetation	Remarks
1. Laterite Crust LC	Cap rock (relatively flat hilltops): Ledges on Slopes	Little surface drainage	Little; rockfalls on edges of cap	Light	Grass or low shrubs	
2. Laterite Gravel LG	Flat hilltops to rolling ground	Little surface drainage	Little; gullying erosion starts on slopes below the gravel layer	Light	Grass or low shrubs	
3. Laterite (Hardening upon exposure) LH	Subdued to rolling ground intervened by flat valleys at high to intermediate slopes	Little surface drainage	Little erosion	Light	Variable; Plantations; Swidden agriculture	General absence of landslides; suitable for golf course or cemetery development; possibility of excavation pits
4. Lateritic Soil LS	Rolling ground, at high to intermediate slopes	Variable; less surface drainage with more advanced stage of and deeper laterization	Variable; less erosion with more advanced stage of and deeper laterization	Light to medium	Variable; Plantations; Swidden agriculture common; Intensive agriculture in high rainfall areas	Termite hills are common
5. Red and Brown Clays CR	Variable; all forms except in basins	Considerable surface drainage development	Gullying erosion; depth of gullying indication of depth of soil to rock; slides	Medium to dark	Variable; intensive agriculture common	
6. Black Clays CB	Gentle lower slopes and flat depressions	Little surface drainage; high water table	Little erosion	Dark	Intensive agriculture	Quality of clay depends on source of material and mode of deposition
7. Desert Soils SD	Variable	Dry drainage channels	Wind erosion	Light	General absence	
8. Saline Soils SS	Low, coastal land; water table near surface	Tidal drainage channels	Little erosion	Light patches (dark in special cases)	General absence	
9. Swamps SW	Low, flat land; water table at surface	Tidal drainage channels	Little erosion	Dark	Mangrove and other swamp vegetation	
10. Alluvium SA	Low, flat land bordering stream	Little surface drainage	Little erosion	Light to medium	Recent—little vegetation; Old—intensive agriculture	
11. Rock and Thin Soils SR	Rolling to hilly, at high slopes	Variable	Variable	Light to medium	Variable; no agriculture development	

TABLE 11.2
Main Characteristics Used to Determine Physiography

Relief	Geomorphology	Drainage Pattern		Depth of Weathering	Probable Texture	Permeability	Probable Lithology	Vegetation	Land Use
		Standard	Characteristics	Cover					
Flat	Hill	Basic	Integration degree	Deep	Coarse	High	Igneous	Field	Little – used
Subdued to rolling	Ridge	Modified	Density	Medium	Fine	Medium	Sedimentary	Savannah	Moderate
Rolling	Plain	Special	Degree of uniformity	Shallow	Medium	Low	Metamorphic	Forest	Intense
Strongly rolling	Valley	Internal	Orientation	Outcrops	–			Meadow	
Mountainous	Basin		Degree of control						
	Mountainous		Angle of junction						
	Plateau		Angularity						

TABLE 11.3
Drainage Characteristics

Systems	Drainage Patterns					
Basic	Dendritic	Parallel	Rectangular	Trellis	Pectinate	Circular
Modification	Contoured Dichotomic	Pincer like Assymetric	Sub-dendritic Angular	Pinnate Subparallel	Pectinate Colinear	–
Special features	Disorderly	Lagoonal Braided	Illusory Anabranched	Kettle-hole –	Yazoo stream –	–
Internal	Sink-hole	Infiltration pits	–	–	–	–

IDENTIFICATION OF LATERITES AND OTHER TROPICAL SOILS

The highly weathered lateritic soils occur in a part of the world where most of the other soils have high clay contents. Thus, these soils containing concretionary material are distinctive because of their generally coarser texture, greater permeability, and inability to retain water and nutrient ions. Areas with such soils are usually suitable agriculturally as poor pasture or are not farmed at all. Surrounding areas may be forested but the coarse-grained or indurated area will only support scattered scrubby trees and sparse grass. In addition the high permeability characteristic applies even to well-indurated lateritic crusts.

True laterite crust, LC (Table 11.1) often appears in association with the remnants of ancient peneplain surfaces. Dowling (1968) describes several occurrences in northeastern Nigeria where the soils have developed over very different geological parent materials. Liang (1964) states that this association is a general characteristic of lateritic crusts and cites examples with illustrations in Brazil, Uganda and Australia. Nearly all of the peneplain remnants appear to be of rather small size. At the most they are a few square miles in area. Bates (1962) suggests that as erosion encroaches upon the peneplain remnant, it permits a substantial lateral flow of groundwater and results in the segregation of iron and aluminum oxides. The iron is transported in solution to the exposed sides of the remnant where it is precipitated as the groundwater enters a strongly oxidizing environment. The interior of the remnant is found to be less consolidated and to contain less iron oxide and more aluminum oxide than the outside fringe. This observation is confirmed by Brammer (1962) in discussing the agricultural soils of Ghana. Apparently, laterite crust occurs typically as a ring surrounding a flat-topped peneplain remnant rather than as a caprock covering the peneplain surface. Airphoto frequently shows only sparse vegetation and no surface drainage features because remnant surface soils have a high permeability. Slopes leading downhill away from the mesa usually present concave profiles. Encroaching gullies appear suddenly, often at the bottom of almost vertical headwalls, tens of feet in height. Liang (1964) notes the danger of confusing the LC pattern with that of basalt or sandstone; which often form a caprock over a base of less permeable, more easily eroded material.

The fact that each peneplain surface was formed only a few stages in geologic time is another clue to the identification of terrain areas with possible laterite crusts. Often the approximate elevations of these surfaces may be known or can be inferred from elevations of the surrounding areas. For example, the Voltaian basin of east-central Ghana has several surfaces, which are discontinuous, but similar enough in elevation to make a reliable correlation. The most noteworthy correlations occurs in the higher hills i.e. elevation of 42o to 760 m (1,400 to 2,500 ft) above sea level (Brush, 1962).

Ferricrete is a laterite-like material formed by the cementation of the sands and gravels in stream banks and terraces. The formation of ferricrete is a continuing process which takes place in areas close to present day stream levels, which is where iron oxide is precipitated from emerging groundwater. Ferricrete may also be associated with terrace systems located above present day stream systems, where the previously stream regimen was stable for a long period of time. In the Voltaian basin terraces are frequently found at levels of 18-23 m (60-75 ft), 30-36 m (100-120 ft) and 75-91 m (250-300 ft) above present base levels. The two lower terrace levels often contain well-cemented ferricrete deposits.

Laterite gravel, LG, like the laterite crust, may occur as a result of weathering in-situ or by other processes which produce accumulations of particles high in iron oxide. LG is often found on high-lying ancient erosion surfaces where the formation of LC is not complete. Erosion surfaces tend to be flat-topped and show evidence of high surface permeability, local aridity and infertility. The sharply defined break at the heads of encroaching gullies which is common in LC areas is usually subdued or missing in LG areas as is the concave profile of the slopes.

In northern Ghana some concretionary gravel is usually present in the subsoil a foot or two below the surface in welldrained areas. In such areas, the vegetative cover is usually grass and shrub rather than forest. A concentration of gravel, analogous to a laterite crust, may be found near the upper convex portion of hillsides. If a crust is present, an accumulation of detached talus material may be present. Apparently, concentrations of gravel are least likely to be found near mid-slope. Near the bottoms of the slopes, soil aggregations may be enlarged or cemented by additions of iron which has been precipitated from groundwater. If the underlying material is quite impermeable, the enriched material may remain permanently moist and become plinthite, LH. At the bottom of the slope, deposits containing lateritic detritus are usually buried by fine-grained colluvium, sheet wash material or alluvium.

Even those lateritic soils which do not contain appreciable coarse material are generally poor agricultural soils. Liang (1964) states that such areas are often devoted to "swidden" agriculture, in which an area is cleared, farmed for a few years, and then abandoned to native vegetation for many years. The resulting land pattern is often quite apparent in aerial photography which shows scattered small fields, some of which are under cultivation and others which are in various natural vegetation regrowth stages. In contrast, some nearby uncultivated areas are not farmed because of their coarse lateritic soils and still other areas show patterns of intensive perennial agriculture use because of their weathered clay or alluviated soils.

Although s[illegible]ces of sound gravel are rare near the ground surface [illegible] the tropics, materials other than laterite may sometimes be found. For example, layers of quartz fragments concentrated at or near the ground surface can be found in terrain where there has been a deep residual weathering of rocks which contain veins or inclusions of quartz. In spite of its convenient location, it is unfortunately, often too thin and scattered to be of much use.

If a fairly thick layer is present it can sometimes be identified in aerial photography. It appears in areas of tonal lightness and in areas with discontinuous vegetative cover. Identification is easier on sloping terrains to identify where sheet erosion has washed the finer material from the surface. It is also easier when the location of underlying country rock which contains large veins or lenses of quartz

is known. Occurrences of this type are more likely to be present in youthful terrains.

Surface drainage conditions are often of considerable engineering importance. Natural color aerial photography provides one useful means for evaluating drainage systems especially when combined with considerations of the topographic position of the drainage system. Also variations in the state of oxidation and hydration of iron compounds are strongly reflected in soil color. Therefore, soil colors should be observed where land has been cleared for agricultural use and where other occasional bare spots are found. Brammer (1962) reports a change in color in typical forest soils of Ghana from red for the best drained soils in which the iron is completely oxidized and not hydrated; to brown and yellow in imperfectly drained soils in which the iron becomes increasingly hydrated; and finally to gray or white in poorly drained soils which are perennially waterlogged so that the iron has been reduced to the ferrous state or completely leached from the soil.

Tropical black clays rich in montmorillonite have attracted considerable interest because of the special engineering problems which they present. They are discussed in detail in Chapter 10. Where black clays occur as residual soils, they are nearly always associated with basic bedrock materials. Topographically, occurrences are generally located in positions where soil drainage tends to be impeded. Thus, black clays are found in areas of gently rolling topography or in flat depressions, but seldom in areas of mountainous relief. Natural vegetative cover is usually savannah and not forest even if the soil is located in a region where forest cover is common. In many tropical areas these soils are farmed intensively. Even though shrinkage cracks may attain considerable size under dry conditions, Liang reports that they were not apparent in photographs at a scale of 1:20,000 of areas in Colombia and Burma. In areas of Africa with longer dry seasons and sparser vegetation photo identification may be easier. Certainly the use of larger-scale color photography should make it easier to locate black clay soils.

In most tropical areas unweathered country rock is seldom found near the surface, but there are exceptional occurrences of isolated rock outcrops of inselbergs. Such features frequently appear as isolated "rock castles" which rise many feet above the surrounding terrain and present a relatively unweathered surface of bare bedrock. In most cases, these features seem to bear little relation to the surrounding deeply weathered terrain. They are useful sources of aggregates and coarse material if they can be found. Most inselbergs can be easily identified in aerial photography by their light appearance which constrasts with their shadows.

GENERAL CONCLUSIONS REGARDING TERRAIN EVALUATIONS

The preceeding discussions indicate that a considerable amount of work is required in order to perform an adequate terrain evaluation of a particular area. The three major steps are the interpretation of the remote sensor imagery, a review of all available bibliographic information regarding the soils and geology of the region, and a selective field investigation of the geologic, vegetation and the pedologic situation of the region.

In the conduct of the present investigation, it was impossible to carry out such a comprehensive procedure. It is important, however, to note that it can and has been done. The degree of the detail involved will always depend upon the purpose for which the evaluation is being conducted. As far as engineering requirements are concerned it will not be necessary to make a detailed pedologic survey, such as that reported by the IBC. Nevertheless, the survey must be sufficiently detailed to allow for the sampling of typical soils and for the verification of the map units. Once a map is completed a great deal would remain to be done in actually correlating typical patterns as seen in the imagery with such on-site items as the type of rock, depth of residual soil, location of laterite crust, location of lateritic gravels, and the identification of soil profiles which have special implications from the engineering performance standpoint.

In this respect, the great significance of field correlation is indicated by the statement which appeared in several of the Radam project reports:

> "Initially, intention was to use data of previous work and then to transpose the information to the radar imagery. However, the complete failure of this procedure led to direct field investigation to determine the correspondence of imagery-geologic features."

From the beginning of this present project, it was hoped that more time might be devoted to developing procedures of photo interpretation through the correlation of available imagery, particularly, black and white panchromatic aerial photography with both available geologic data and field investigation. Some IR photography was also available for study. Unfortunately, it was impossible to carry out this preliminary objective because of the short time available and because of the difficulties in obtaining necessary background data, photography and field mapping. Thus, the present project has depended primarily on the available geologic and pedologic maps as a basis of locating and classifying laterite materials. There seems no doubt, however, in view of the Radam report, the success of the Radam project and the work done by the Brazilian Coffee Institute in cooperation with the Brazilian Ministry of Agriculture, that it is feasible to use aerial photography as well as the imagery obtainable from other types of remote sensors to classify the terrain. Furthermore, various map units can be developed from the imagery and correlated with the characteristics of soil profiles which are important to engineering projects, especially those related to transportation.

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CHAPTER 12
SUGGESTED SPECIFICATIONS FOR MATERIALS AND CONSTRUCTION IN TROPICAL CLIMATES

INTRODUCTION

The uses of specifications are to insure a quality product and to minimize or eliminate any basis of disputes between designers and contractors. Ideally, specifications should be used as a guide, for both the engineer and contractor in the performance of work outlined in the contract documents.

The specifications for base and sub-base materials presented in this section have been developed through analysis of the performance of pavement sections evaluated during this study. Specifications for lime, cement and asphalts have been taken from AASHO and Asphalt Institute (USA) standard specifications. Specifications for surface treatment surfacing have been adopted from recommendations presented at the First African Highway Conference, Addis Ababa, by Pullen and Prevost (1969). Specifications for excavation and compaction equipment were taken from experience gained by the U.S. Army Corps of Engineers in working with laterites and lateritic soils of Vietnam (1968) and experience gained in Africa and South America.

AASHO Specifications

The American Association of State Highway Officials (AASHO) provides specifications for base and sub-base materials. The specifications have been developed from experience with temperate soils and reflect the performance of these soils under temperate climates. The specifications provide a means of selecting ideal materials to be used in the construction of highway pavements. The selection of materials is based on Atterberg limits and gradation. These tests provide a simple means of selecting materials by engineers, technicians or in fact anyone possessing only the skill of conducting these tests.

More and more engineers are realizing that the simplicity has a price. Simplification through elimination of a strength (CBR) requirement has necessitated restricting the use of materials to those which exhibit physical properties that fit into well defined limits. As these ideal materials become harder to find in temperate regions, it may become necessary to modify the AASHO specifications. This is even more true in the tropics where soils similar to temperate soils are almost impossible to find. The specifications presented in this section have broader limits for the physical properties and include the CBR value of the material.

DISCRIMINANT ANALYSIS

A basis of modification of the AASHO specifications was established by an analysis of the performance of selected test sections. All of the sections (34) used in the analysis incorporated CBR values which were greater than the maximum values shown in Table 6.5. Through this discriminant selection the effect of the CBR in the upper layers was eliminated. The sections were divided into two groups, one included sections where all the component layers exhibited physical properties within the range recommended by the AASHO specifications; the second group included sections where the physical properties of the component layers were outside of the limits recommended by AASHO specifications. The analysis compared the relationship of thickness and deflection between the two groups. Basically, the discriminant analysis assumes that a distinction can be made between the two groups through the difference of the thickness-deflection relationships of each group. The results showed that there was no difference between the two groups. Thus, the results indicated that AASHO specifications eliminates materials that would otherwise perform as satisfactorily as the "in-specification" materials and, therefore, eliminates the use of material that would have otherwise been satisfactory for flexible pavement construction.

FACTORIAL ANALYSIS

The design procedure outlined in Chapter 7 incorporates layer strength (CBR) in the determination of thickness requirements which is in contrast to the more conventional approach of assuming all materials to have adequate strength provided the material has certain physical properties (PI, LL and percent passing the 200 sieve) within well defined limits.

It would be extremely difficult to evaluate the effects of the various engineering and physical properties on the performance of the various test sections. A complete factorial analysis of all the possible levels of all the variables involved would require a designed experiment outside the scope of this study. However, a factorial analysis was performed to establish the relative importance of physical and engineering properties of the subgrade in the performance (deflection) of flexible pavements. Only sections that had a minimum combined thickness of 60 cm (24 in) were used in the analysis to isolate the variables of composition for the surface, base and sub-base layers. As the total thickness of pavement (surface, base and sub-base) are increased, the deflection of the section becomes more dependent upon the combined thickness of these layers and the properties of the subgrade, rather than the properties of the upper pavement layers. The properties of the subgrade studies were the percentage of fines, plasticity and CBR.

The analysis showed that the most important subgrade variable with respect to the deflection of the test section was the CBR, followed by plasticity and percentage of fines.

Atterberg Limits, Percentage of Fines and CBR

During the African study attempts were made to establish a relationship between the individual physical

properties used in the AASHO specifications and CBR. A meaningful relationship could not be developed. However, the results showed a useable relationship between a combination of the Atterberg limits and percentage of fines and CBR. The results are shown in Figures 12.1 and 12.2. The product of the liquid limit or plasticity index times the percentage of fines provides a reasonable approximation of the lower limit for the CBR values. Similar plots are shown for the soils tested during the South American Study in Figures 12.3 and 12.4. Included in these figures are the limits for the respective properties given in the AASHO specifications. Many of the soils exhibiting suitable CBR values (4 day soak) would be eliminated from use in pavement construction if AASHO specifications were to govern the selection of materials.

Specifications proposed in the African Study incorporated the product of the values rather than individual physical properties. The specifications presented in this report are also in terms of the product of the values.

A parameter which was not investigated in the African Study, the granulometric modulus, is also proposed. The granulometric modulus is defined as the sum of the accumulated percentages of material passing the 1", 3/4", 1/2", 3/8", No. 4, No. 10, No. 40 and No. 200 sieves. The relationship between CBR and the granulometric modulus is shown in Figure 12.5. The plotted curve represents the minimum CBR value that one can expect for a given value of the granulometric modulus.

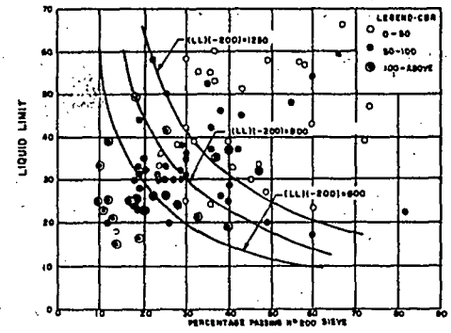


FIGURE 12.1 RELATIONSHIP LIQUID LIMIT AND CBR AT VARIOUS PERCENTAGES OF MATERIAL PASSING Nº 200 – AFRICAN DATA

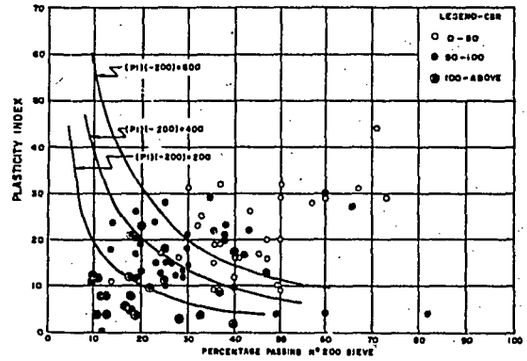


FIGURE 12.2 RELATIONSHIP PLASTICITY INDEX AND CBR AT VARIOUS PERCENTAGES OF MATERIAL PASSING THE Nº 200 SIEVE – AFRICAN DATA

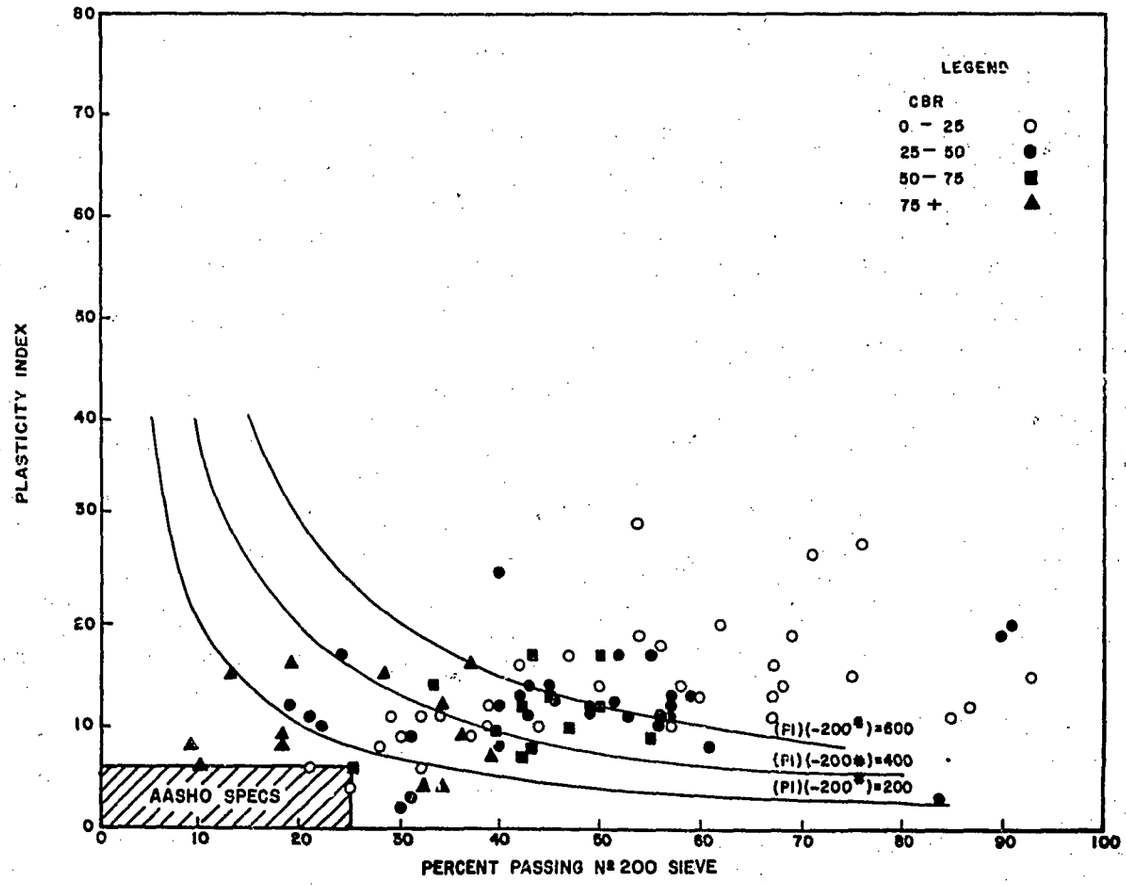


FIGURE 12.3 – RELATIONSHIP BETWEEN PLASTICITY INDEX, PERCENT PASSING No. 200 SIEVE AND SOAKED CBR, SOUTH AMERICAN DATA

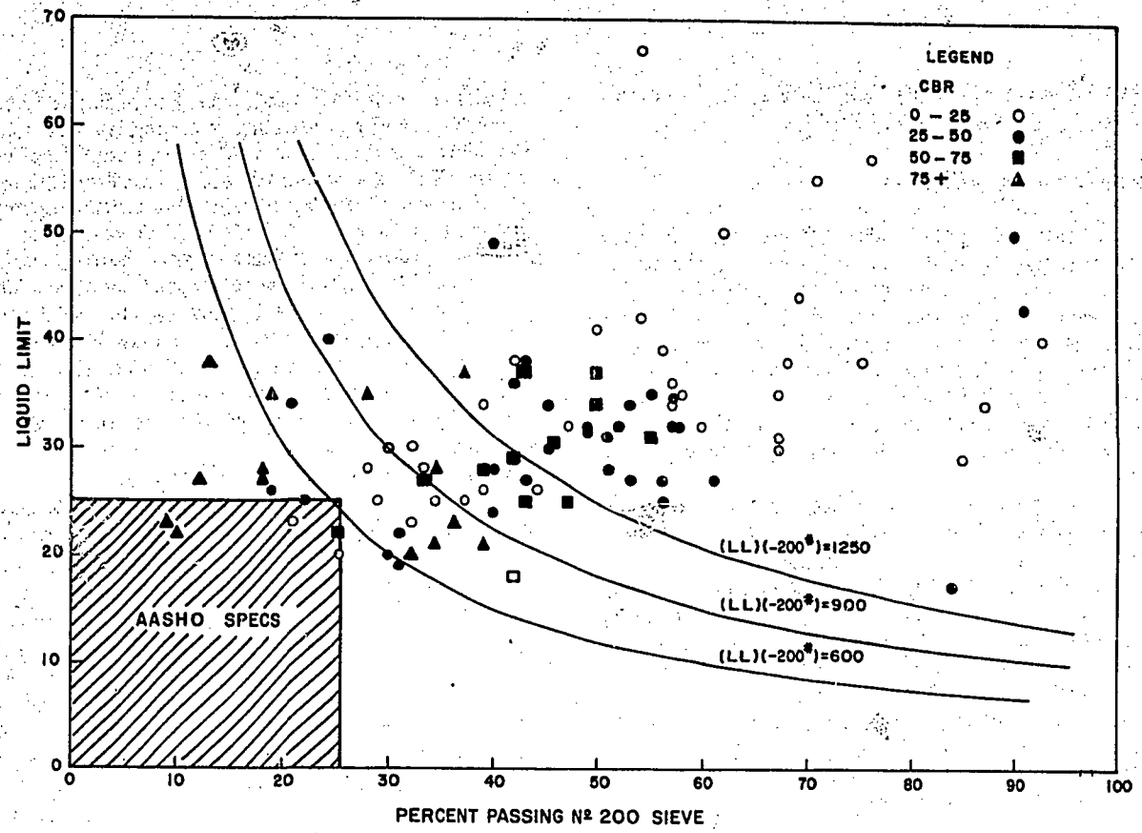


FIGURE 12.4 – RELATIONSHIP BETWEEN LIQUID LIMIT, PERCENT PASSING No. 200 SIEVE AND SOAKED CBR, SOUTH AMERICAN DATA

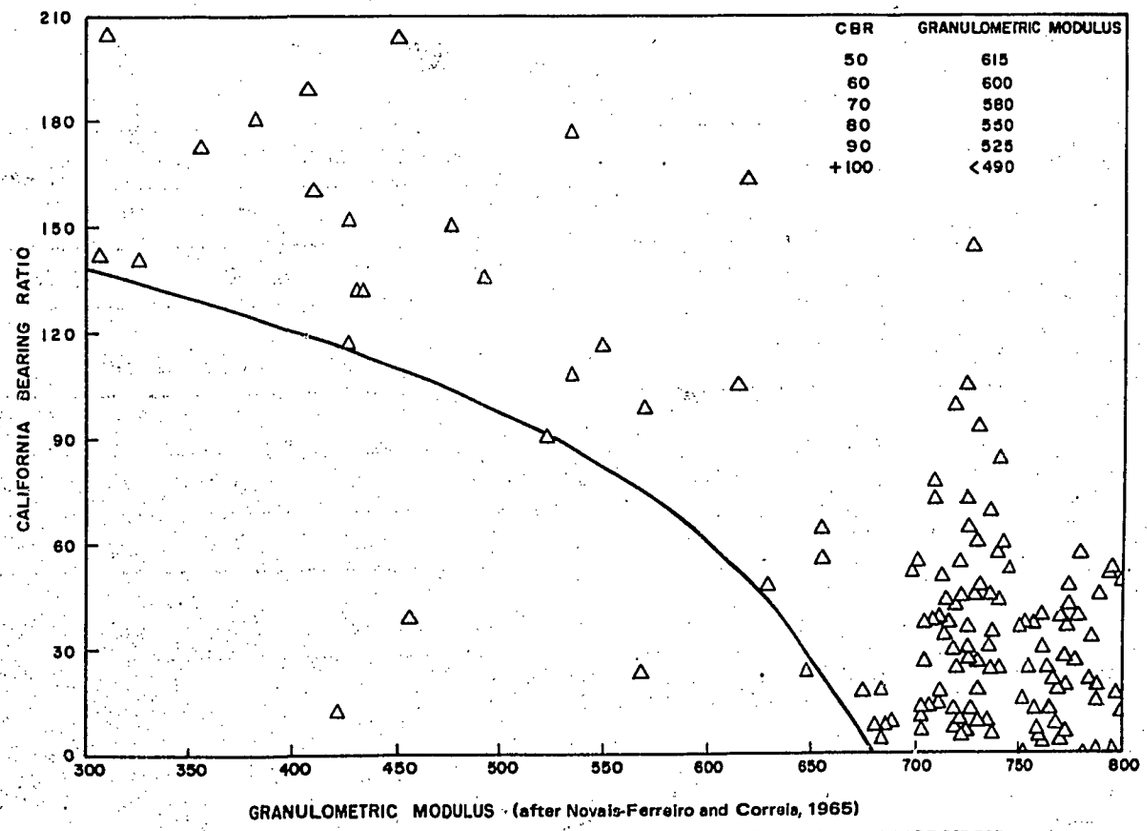


FIGURE 12.5 – RELATIONSHIP BETWEEN CBR AND GRANULOMETRIC MODULUS, SOUTH AMERICAN DATA

SPECIFICATIONS FOR SUB-BASE; BASE AND SURFACE COURSE MATERIALS

Sub-base and Base Courses

Recommended specifications for sub-base and base course materials are given in Table 12.1. Six classifications of base course materials are shown in the table. The recommended base classification is determined by the design period which is the accumulated equivalent standard axle loadings in both directions (single lane design) used in the design computations. The six classifications provide for efficient use of available materials since thickness requirements are dependent upon the CBR of the component layers.

Limits for grading and Atterberg limits are shown for each classification. These are used as criteria in construction control testing. If a surface treatment is used in design it is recommended that the durability requirements given for Surface Course Materials be included in the specifications for the base course material. There is no reason to believe that the durability requirement is necessary if an asphalt concrete surface course is used in design.

The design CBR is determined by a statistical analysis of samples obtained from the proposed borrow area. An illustrated example is shown in Figure 12.6. The percentage of the samples greater than a given CBR value is computed and plotted as shown on the lower right of Figure 12.6. An acceptable practice of selecting an appropriate CBR value is that which corresponds to a 90 percent confidence limit, i.e., the CBR value where 90 percent of all samples tested are greater than the selected value. For statistical purposes the minimum number of tests is 6. However, more tests may be required to evaluate a given borrow area. The number of tests required would depend upon variations of material and size of the borrow pit. In the illustrated example the CBR would be 55 or a Class VI base course. An alternative would be to use "selective" excavation which would eliminate areas exhibiting the lower CBR values. The borrow area would then receive a higher classification.

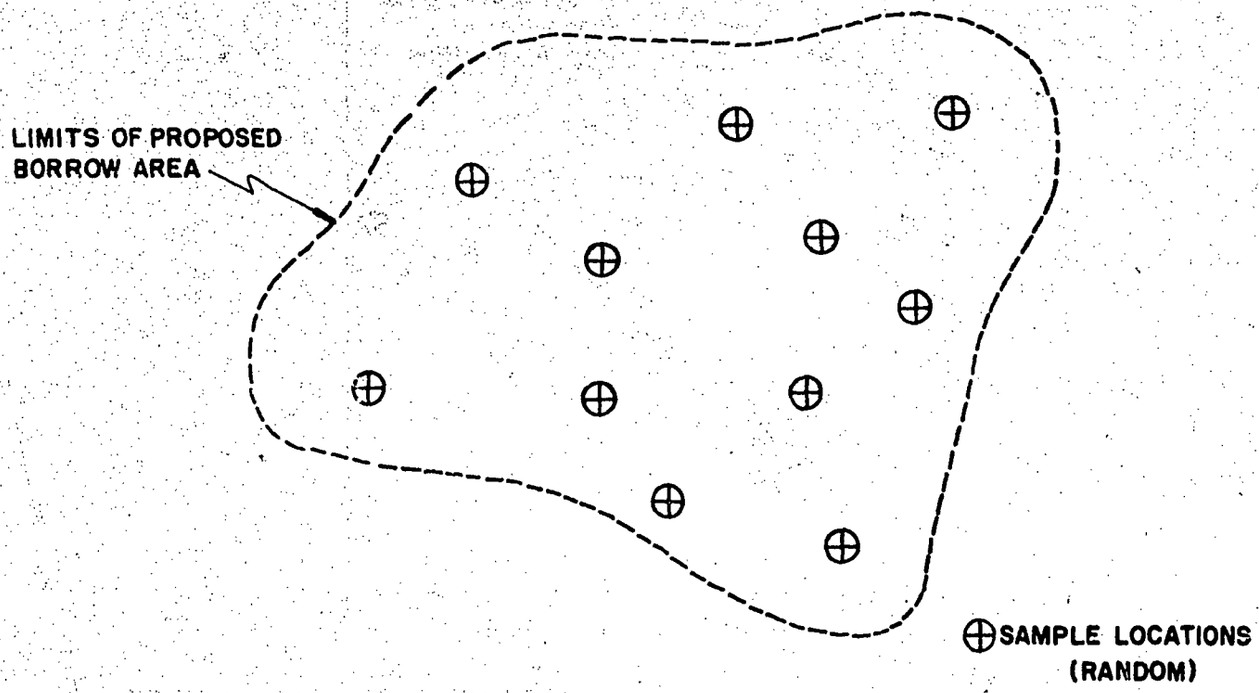
DETERMINATION OF CBR VALUES

A common practice is to soak the CBR sample for four days prior to testing. The reasoning is that the CBR is then determined at the worst possible condition that would exist during the design period. In areas of frost penetration with the associated accumulation of water in the pavement layers during the spring thaw period, such a pre-treatment prior to testing is certainly warranted. The in-situ moisture contents within the various component layers were examined during the South American study. Figures 12.7 and 12.8 show the relationship between optimum moisture content at AASHO Modified compaction and in-situ moisture content for base, and subbase layers. The annual

TABLE 12.1
Specifications, Base and Sub-base Materials

Criteria	Base Classification						Subbase
	I	II	III	IV	V	VI	
Design CBR	+100	90	80	70	60	50	25 – 40
Maximum traffic AESALBD	10^7	10^7	10^7	2×10^6	9×10^5	5×10^5	–
Gradation	1	1	1	2	2	2	3
Maximum LL X (– 200)	600	900	900	900	1 250	1 250	1 600
Maximum PI X (– 200)	200	400	400	400	600	600	800
Maximum Granularmetric Modulus	490	525	550	580	600	615	630

AESALBD – Accumulated Equivalent Standard Axle Loadings in Both Directions
Granularmetric Modulus – Percent passing 1, 3/4, 1/2, 3/8, 4, 10, 40, 200 – sieves.

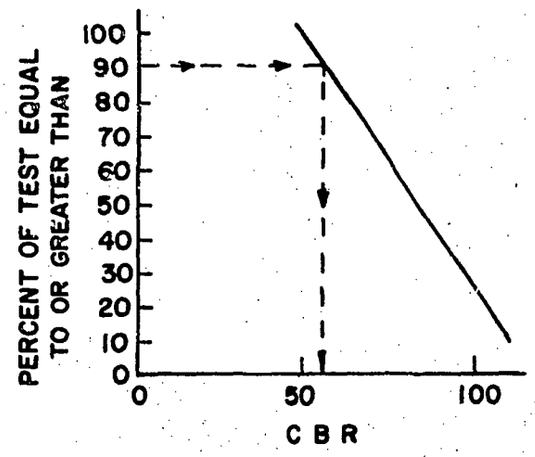


RESULTS OF LABORATORY TESTS

CBR = 50-60-65-70-75-80-80-90-90-90-110

STATISTICAL TEST

CBR	NUMBER	PERCENT ≥
50	11	100
65	9	82
70	8	3
75	7	64
80	6	55
90	4	30
110	1	9



CBR = 55 ~ CLASS VI

FIGURE 12.6 – CLASSIFICATION OF BASE COURSE MATERIALS

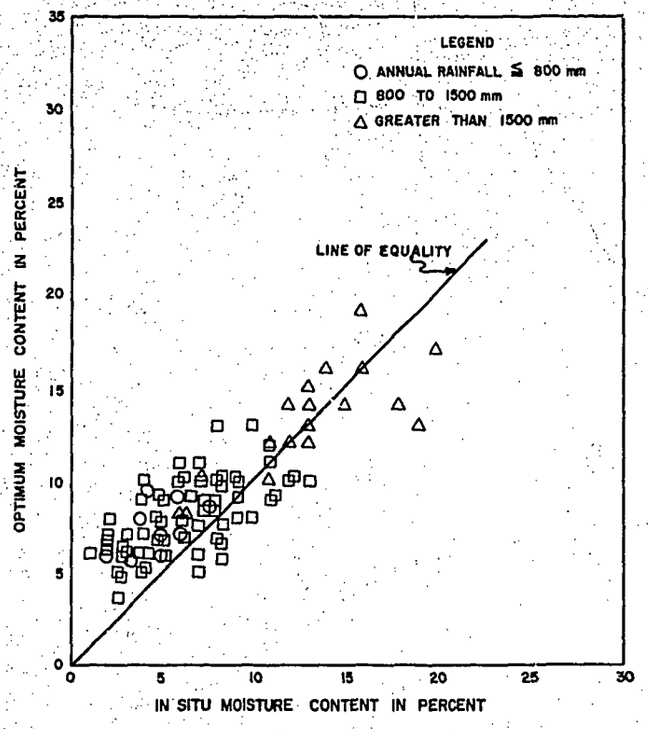


FIGURE 12.7 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND IN SITU MOISTURE CONTENT-BASE COURSES

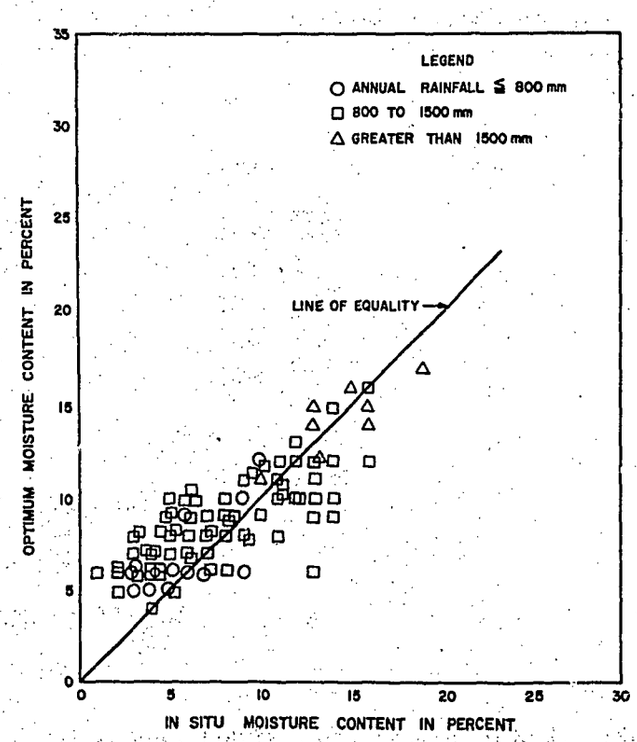


FIGURE 12.8 – RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND IN SITU MOISTURE CONTENT, SUBBASE COURSES

rainfall has been indicated for the individual test section. Examination of these plots shows that the in-situ moisture contents of the base materials does not significantly exceed the optimum moisture content of the material until the annual rainfall exceeds 1500 mm (60 in). The in-situ moisture content of the subbase materials does not significantly exceed the optimum moisture content of the material until the annual rainfall exceeds 800 mm (32 in). Figure 12.9 shows the relationship between annual rainfall and coefficient of equilibrium moisture content (c) for the subgrade materials. The equilibrium moisture content represents the amount of moisture the soil can accumulate plastic limit. The coefficient of equilibrium moisture content (c) represents a function of the plastic limit which relates the equilibrium moisture to the plastic limit and the annual rainfall.

Equilibrium moisture = (c) (PL) . . . . . (12.1).

The band in Figure 12.9 represents the range of moisture contents which should be considered in the determination of the design CBR of the subgrade. Table 12.2 gives the range of moisture contents to be used in examining the CBR value of the respective layers for design purposes.

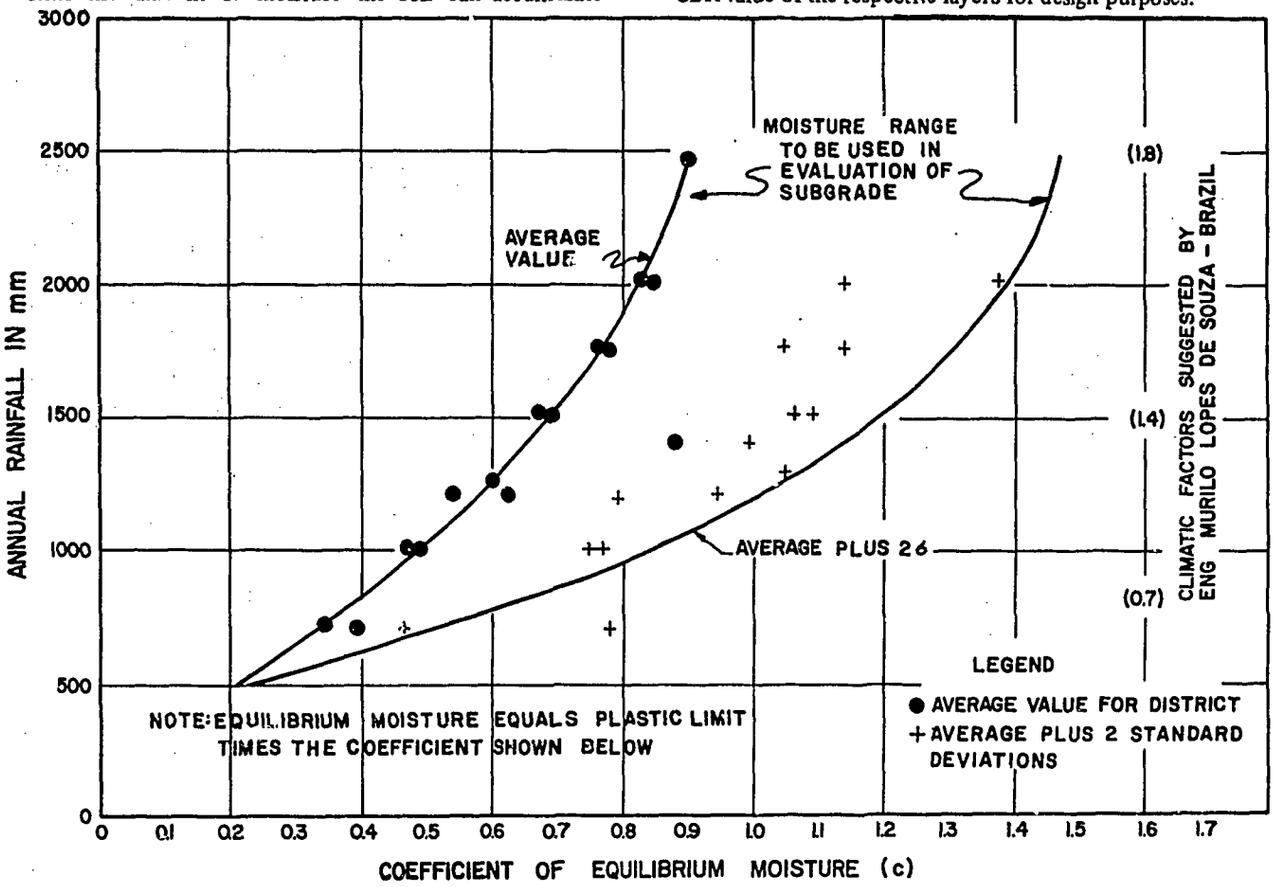


FIGURE 12.9 – ANNUAL RAINFALL vs COEFFICIENT OF EQUILIBRIUM, SOUTH AMERICA

TABLE 12.2

Recommended Moisture Ranges for Evaluation of Design CBR Values

Structural Layer	Annual Rainfall		
	< 800 mm 30 in	800 to 1,500 mm 30 to 60 in	> 1,500 mm 60 in
Base	OMC	OMC – 1.25 OMC	4 day soak
Sub-base	OMC	OMC – 1.5 OMC	4 day soak
Subgrade	0.4 – 0.6 PL	0.7 – 1.2 PL	0.9 – 1.5 PL

Conditions: Water table at least 1 meter below pavement surface and good surface drainage.

Surface Course Materials

The selection of surface course materials is governed by gradation. Table 12.3 shows the various gradings used in selecting materials for the component layers. Grading 4 represents the recommended gradation for untreated surface course materials. The grading envelope was arrived at by studies of reports covering the performance of untreated gravel roads. The minimum CBR value for surface course materials is 40. The physical properties given for Class VI base courses or subbases can be used for control purposes.

The quality of ironstones and concretionary gravels should be evaluated before these materials are utilized for surface course materials.

Several durability tests were evaluated in the course of this study (Chapter 8). The most promising of these is the slake durability test. A tentative specification of a minimum slake durability value of 95 is recommended for ironstones and concretionary gravels when these materials are to be used in the surface course.

Another promising test is the repetitive load test described in Chapter 8. The equipment is highly specialized and therefore would not be available in all laboratories. Should the equipment be available for testing, a minimum value of 1476 kg/cm^2 (21 ksi) is recommended for the resilient modulus after 100,000 load applications. In testing σ_1 is 3.58 kg/cm^2 (51 psi) and σ_3 is 1.21 kg/cm^2 (17 psi).

SPECIFICATIONS FOR EXCAVATION OF BORROW AREAS, COMPACTION EQUIPMENT AND COMPACTION REQUIREMENTS

Excavation of Borrow Areas*

In developing the layout of a borrow site, area utilization and drainage (especially during the rainy season) are key factors to consider. If scrapers are to be used, the borrow pit should be excavated from an uphill position down, and the furrows made by the scrapers should be continuous and provide for drainage away from the pit. However, if power shovels are to be used, the pit should be excavated from a down hill position up. This technique will permit natural drainage and prevent local ponding. All stripped soil should be placed in a location outside the borrow area. While the ironstone or concretionary gravel is being excavated, care should be taken to prevent excavation into underlying clayey silts. If silt is mixed in with the borrow material during excavation, it must be washed out prior to placement, otherwise serious local failures may be expected. Most lateritic soil can be excavated with a scraper or a scraper pushed by a bulldozer. However, the ironstone must be excavated with a bulldozer with a 30 to 45 cm (12 to 18 in) ripper tooth. Blasting to excavate laterite is impractical because the laterite has a high natural porosity.

* From U. S. Army Engineers (1968).

TABLE 12.3
Recommended Gradations

Sieve		1	2	3	4
ISO* 50 mm	ASTM-E11 2 in	100	100	100	100
37.5	1 1/2	80 – 100	90 – 100	100	100
25	1	50 – 100	70 – 100	85 – 100	100
19	3/4	45 – 100	60 – 100	75 – 100	100
12.5	1/2	35 – 100	45 – 100	65 – 100	90 – 100
9.5	3/8	30 – 90	40 – 100	55 – 100	80 – 100
4.75	No. 4	20 – 70	30 – 75	45 – 80	55 – 85
2	10	10 – 50	25 – 60	35 – 65	35 – 65
425μm	40	5 – 35	20 – 45	30 – 45	20 – 40
75	200	0 – 25	15 – 35	25 – 40	15 – 30

* ISO – International Standards Organization, Geneva, Switzerland.

Compaction Equipment*

1. IRONSTONE OR CONCRETIONARY GRAVEL (LATERITE

During excavation, transportation, and compaction, an effort should be made to prevent unnecessary structural degradation of the laterite; therefore, compaction should be light and shaping kept to a minimum to avoid high shear stresses. The reference to "light compaction" is to indicate that lighter rollers and good moisture control should be employed to obtain the specified density and that heavy compactors should not be used as a panacea to all compaction processes. For the wormhole laterite, the 7258 to 9720 kg (8 to 10 ton) vibratory steel wheel roller gives the best result. For pellet laterite, the 4536 to 7258 kg (5 to 8 ton) steel wheeled or pneumatic tire rollers are the most effective.

2. LATERITIC SOILS

In Thailand, a variation of a sheepsfoot roller was used effectively in compacting a lateritic soil. The roller is similar to a sheepsfoot roller except that the feet are flatter and have a larger surface area. It has the capability of compacting thick lifts; for example, a 30 cm (12 in) loose lift can be compacted to a 15 cm (6 in) compacted thicknesses. However, the loose lift thickness is usually limited to 15 cm (6 in) and is compacted to a thickness of 8 to 10 cm (3 to 4 in). Contrary to popular opinion, the sheepsfoot roller can be used effectively if the weight of the roller is reduced (usually by only half filling the drum) and the roller is pulled slowly to avoid high shear stresses. The 45360 kg (50 ton) roller can be used, but the load and tire pressure are critical; they must be adjusted to approximately 22680 kg (25 tons) and 6.3 to 7.7 kg/cm^2 (90 to 110 psi) respectively. A versatile roller which provides the "compaction action" of both a steel wheel and a sheepsfoot is the "Hyster" Grid Roller. This type of roller should prove to be very effective in the compaction of both laterite and lateritic soils.

Lateritic soil compacted on the wet side of OMC often gives a spongy section instead of a suitable compacted layer. A good rule of thumb for the field is to apply water at 2 percent less than the lab optimum moisture content.

Compaction Requirements

The specified compaction requirements have been developed through analysis of the deflection and compaction relationships shown in Figure 12.10. Sections which exhibited deflections slightly higher than predicted by calculations were plotted against the percentage of AASHO modified compaction. In all sections the CBR of the various layers provided sufficient strength so the excessive deflection could not be attributed to low strength of the layer. The plotted curve represents the density at a particular depth where excessive deflections were not measured at

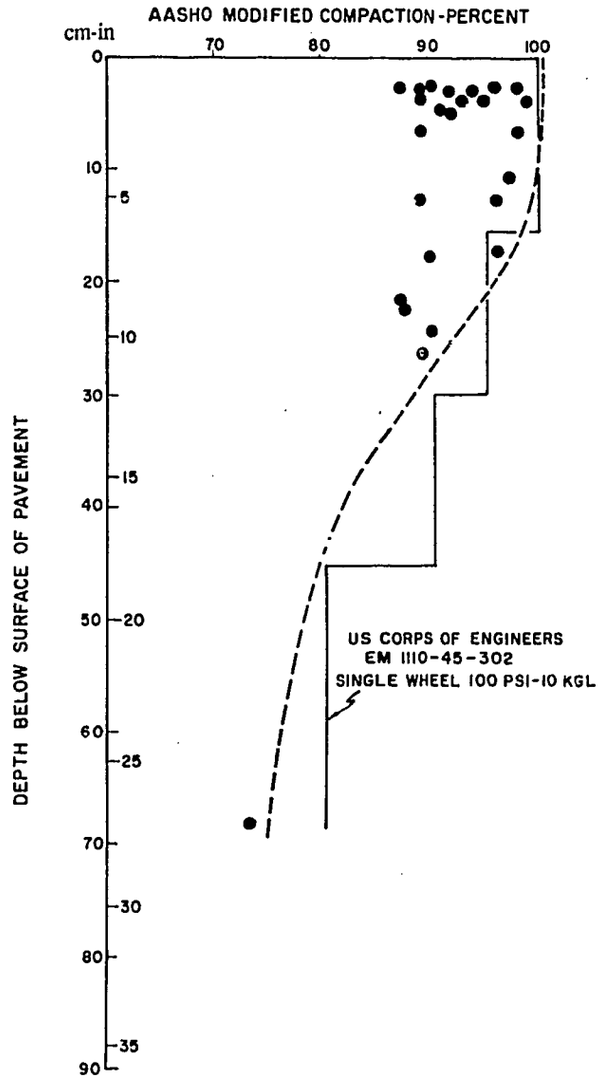


FIGURE 12.10 – REQUIRED DENSITY WITH DEPTH BELOW SURFACE OF PAVEMENT

higher density values. There is a definite relationship between required compaction and depth beneath the pavement surface. Most specifications require a specified degree of compaction for base, subbase and subgrade layers. However, these do not define a specific compaction-depth requirement. A compaction-depth requirement used by the US Corps of Army engineers is also shown in Figure 12.10. The recommended density requirements shown in Table 12.4 are expressed as a function of the depth beneath the surface of the pavement. The requirements are applicable to natural subgrades in which case the required compaction will prevent excessive deflection of the layer but does not necessarily provide for the deelopment of shear strengths required in the stability of high fills.

* From U.S. Army Engineers (1968)

TABLE 12.4
Recommended Compaction Requirement for Component Structural Layers and Natural Subgrade

Depth Below Surface	Compaction
0 – 25 cm (0 – 10 in)	100% AASHO MOD.
25 – 45 cm (10 – 18 in)	95% AASHO MOD.
45 – 60 cm (18 – 24 in)	100% AASHO STD
60 – 90 cm (24 – 36 in)	95% AASHO STD

SPECIFICATIONS FOR MATERIALS AND CONSTRUCTION IN TROPICAL CLIMATES

The standard specifications for asphalt, Portland cement and lime are given in the appendix to this chapter.

Specifications for surface treatment pavements are also presented in the appendix to this chapter. These have been taken directly from a paper presented at the First African Highway Conference, Addis-Ababa, October 1969 by Pullen and Prevost.

Specifications for asphalt concrete surfacing are not presented since evaluation of composition and mix design of these surfaces were not in the scope of this study nor did the review of the literature provide information similar to that presented for surface treatment pavement design in the tropics.

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APPENDIX TO CHAPTER 4

TABLE A4.1

Chemical and Geological Properties of Red Tropical Soils of South America

Sample Number	Soil Group *	Location State *	Rainfall, mm	Moisture Deficience Months	W.P.I. *	SiO^2 Total	SiO^2 Free	Fe^2O^3	Al^2O^3	R^2O^3	$\frac{Sio^2}{Al^2O^3}$	pH	Mineralogy Clay
6	A	GO	1,500	–	–	72.2	57.8	4.8	12.4	1.58	1.98	4.10	–
1	A	ES	1,150	0	–	75.5	59.3	6.1	11.3	2.02	2.72	3.95	–
5	A	RS	1,410	2	– 21,2	60.9	51.4	4.0	10.7	–	–	3.7	Qz, K
6	A	PR	1,300	0	– 20.8	64.8	48.5	11.6	14.6	2.10	3.16	4.10	Qz, He, Gib, K
5	A	SP	1,750	0	– 15.16	76.2	68.4	4.1	10.1	2.14	2.69	3.8	K, Qz Cl Gib Go
1	A	CE	1,250	6	–	70.9	51.2	3.0	17.9	1.70	1.88	4.35	–
2	A	CE	1,250	6	– 45.4	45.4	20.6	22.7	21.6	.30	.50	4.30	Qz I K Gib He
8	A	CE	1,300	6	–	57	47.2	4.3	16.5	.60	.70	3.8	Qz F K M I
6	A	MG	1,000	–	–	61.9	48.8	6.9	20.2	1.07	1.30	4.05	K Qz I Gib
12	A	MG	1,500	–	–	52	46.6	12.5	21.8	1.33	1.82	4.20	–
2	A	PE	1,900	–	–	58	32.6	5.3	22.2	1.57	1.82	4.10	–
1	A	MT	1,450	5	– 17.86	63.9	38.3	3.4	17.4	2.55	2.87	–	Qz K I Gib
2	A	MT	1,250	5	– 17	64	45.2	13.6	15.1	1.92	3.02	4.2	Qz Go Gib Cl I K
3	A	PA	2,200	–	–	22.9	–	34.4	23.5	0.41	0.79	4.4	–
4	A	BA	1,500	4	– 12.8	–	–	–	–	–	–	4.5	K Qz Gib
7	Q	PE	800	–	–	78.3	66.5	1.4	8.9	1.49	1.64	4.85	–
8	Q	PE	750	–	–	78.3	64	1.4	11.4	1.95	2.10	3.70	–
4	Q	MA	1,450	7	–	28.7	13.4	46.3	13.8	–	0.14	4.60	–
6	Q	MA	1,350	7	–	94.4	88.4	0.6	2.8	–	4.58	4.50	–
7	Q	MA	1,150	–	–	45.6	34.8	32.2	11.9	0.24	0.65	4.20	–
8	Q	MA	1,100	–	–	36.9	14.5	31.4	18.3	0.13	0.27	4.15	–
11	Q	BA	650	7	–	85.9	80	1.7	6.4	2.63	3.08	4.20	Qz M K
1	Q	PI	1,250	7	– 31.03	48.2	36.3	30.6	13.4	0.15	1.10	4.35	Qz He Go K
2	Q	PI	1,200	–	–	42	31.2	37.6	11.9	0.18	0.54	4.05	–
3	Q	PI	1,200	–	– 29.81	41.5	27.5	38.0	12.6	0.25	0.73	3.65	Qz He Go I K
4	Q	PI	1,300	–	–	20.2	8.1	58.5	13.0	–	0.13	4.15	–
6	Q	PI	1,000	–	–	29.5	14.2	43.1	15.8	–	0.22	4.00	–
8	Q	PI	1,100	–	–	59.1	6.1	19.1	13.5	0.55	1.04	3.90	Qz Go K
1	Q	RN	1,050	4	–	81.5	68.5	2.6	10.6	2.03	1.82	4.30	Qz He K
2	Q	RN	1,100	4	– 13.19	66.3	54.6	4.0	15.1	–	–	4.20	Qz F Cl K
3	Q	RN	650	11	– 14.15	53.6	43.7	7.4	15.0	0.60	0.79	4.90	Qz F Cl He M
7	Q	CE	1,100	8	– 3.13	92.5	87.4	0.4	5.0	2.69	2.83	6.75	–
1	Q	PB	900	–	–	54.5	36.0	3.4	22.5	1.05	1.15	5.00	Qz Cl F K
2	Q	PB	1,500	5	– 23.8	60.7	41.1	9.7	18.7	0.91	1.21	4.25	Qz Cl K
9	Q	PB	1,800	5	– 21.84	78.6	67.0	2.9	12.3	2.26	2.60	4.15	Qz Gib K
8	Q	GO	1,550	–	– 2.93	86.1	78.8	2.8	6.3	–	3.94	4.80	–
9	Q	GO	1,650	–	–	93.2	88.7	2.6	2.8	2.84	4.52	5.05	–
8	Q	SP	1,160	–	–	86.7	–	5.43	4.12	3.41	5.71	3.75	–
3	Q	CG	1,350	–	–	91.9	86.9	1.9	3.0	–	5.20	4.50	Qz Gib K
6	Q	CE	1,000	–	– 14.22	84.2	75.7	2.3	8.3	2.78	3.27	4.30	K Qz
2	Q	CG	1,350	–	–	85.1	70.4	3.6	5.1	2.34	3.40	4.30	–
5	L	PR	1,000	–	–	59.9	43	3.4	18.9	1.59	1.78	4.65	–
6	L	PR	750	–	–	51.4	28.9	6.7	25.2	1.27	1.49	4.90	–
9	L	PR	550	–	–	61.9	48	3.1	15.4	1.35	1.50	5.30	–
1	L	MA	1 850	5	–	34.7	18.7	40.2	14.8	0.20	0.53	4.15	–

*notations given below.

TABLE A4.1 (continued)

Sample Number	Soil Group *	Location State *	Rainfall,mm	Moisture Deficience Months	W.P.I. *	SiO^2 Total	SiO^2 Free	Fe^2O^3	Al^2O^3	R^2O^3	$\frac{Sio^2}{Al^2O^3}$	pH	Mineralogy Clay
2	L	MA	1,800	5	–	60.5	36.0	6.4	22.3	1.71	2.03	4.10	–
5	L	MA	1,600	–	–	29.9	18.9	47.8	11.6	0.11	0.39	4.60	–
6	L	BA	1,100	5	– 17.85	73.6	28.7	6.5	12.4	2.72	3.63	4.25	Qz K He Go Gib
7	L	BA	900	11	– 13.7	66.6	53.8	3.6	14.4	1.83	2.12	5.25	Qz F Gib K
10	L	BA	950	4	– 3.57	77.7	60.5	3.7	11.3	–	–	3.75	Qz K Go Gib
5	L	RN	750	–	– 3.0	56.8	41.7	5.7	13.0	1.45	1.85	5.30	Qz F Mg Cl I M
6	L	RN	700	–	– 18.35	74.9	61.7	2.9	12.6	2.99	3.43	4.80	Qz F K
7	L	RN	800	–	– 88.60	54.7	32.3	5.4	20.9	1.07	1.24	4.90	Qz F Gib Cl M K
3	L	CE	1,250	9	–	65.6	51.7	3.4	15.9	1.72	1.96	4.10	–
4	L	CE	1,200	6	– 3.10	44.4	25.8	6.4	18.7	0.50	0.60	–	Qz F Mg He I
5	L	CE	1,200	8	–	60.1	35.3	7.2	15.5	1.42	1.84	–	–
10	L	CE	850	8	– 8.31	43.0	22.8	9.0	21.0	0.65	0.83	4.90	Qz K F Cl I
3	L	PB	1,300	5	– 18.2	75.9	61.4	3.0	13.5	2.67	3.05	5.25	–
4	L	PB	1,300	–	– 20.60	75.0	60.0	2.6	14.2	1.93	2.15	4.20	–
5	L	PB	1,400	6	– 19.34	79.1	65.7	2.3	12.0	–	3.36	3.90	–
8	L	PB	1,800	–	– 11.85	58.6	45.8	5.2	16.7	1.09	1.31	4.65	–
10	L	PB	900	6	– 23.4	52.1	28.0	15.6	17.4	0.45	0.71	4.65	–
11	L	PB	550	–	–	55.3	40.8	7.4	15.9	1.00	1.30	5.20	–
13	L	MG	1,350	–	–	42.1	21.8	12.8	27.7	0.88	1.13	5.00	–
3	L	GO	1,750	–	–	25.6	18.1	18.4	33.5	0.33	0.44	5.50	Qz Mg He Gib K
12	L	SP	1,750	–	–	75	60	4.4	12.5	1.16	1.43	4.55	–
13	L	SP	1,170	–	–	78.8	68	3.9	9.3	2.95	3.74	4.55	–
5	L	RJ	–	–	–	–.	–	–	–	–	–	–	–
8	L	RJ	–	–	–	–	–	–	–	–	–	–	–
5	L	MG	1,250	–	–	79.1	78.4	4.3	10.4	1.83	2.32	5.10	Qz He Ba Gib K
7	L	SP	1,200	–	–	72.4	64.0	5.7	9.6	1.86	2.56	5.45	–
8	N	RS	–	–	–	–	–	–	–	–	–	–	–
1	F	PA	2,500	–	–	46.2	–	35	9.52	0.30	1.0	4.20	–
2	F	PA	2,290	–	–	–	–	–	–	–	–	4.20	–
5	F	PA	1,800	–	–	79.9	–	4.06	10.4	2.08	2.61	4.10	–
6	F	PA	2,250	–	–	–	–	–	–	–	–	4.10	–
7	F	PA	1,950	–	–	41.3	–	15.3	28.8	0.80	1.20	4.15	–
1	F	DF	1,650	4	–	20.6	2.4	50.5	14	–	–	5.20	–
2	F	DF	1,650	4	–	32.9	14.6	36.2	17.8	0.16	0.38	5.60	–
3	F	DF	1,650	4	–	38	19.9	36.7	12.9	0.84	0.75	5.05	–
4	F	DF	1,650	4	–	21.3	5.6	15.4	33.7	0.61	0.79	5.85	–
2	F	PR	2,100	0	– 38.77	37.8	–	19.5	28.2	1.26	1.82	4.40	Qz He Cl K
4	F	PR	1,250	0	– 71.3	22.4	0.7	30.3	25	10.51	0.91	5.10	Qz Mg He Gib Cl K
5	F	PR	1,200	0	– 53.05	31.0	4.9	27.6	20.8	1.15	1.87	5.70	Qz He Bo K
8	F	PR	1,750	0	–	29.6	0.9	27.2	26.2	10.67	1.11	4.30	M K Go He
9	F	PR	1,400	0	– 20.65	23.8	5.9	28.1	26.5	10.71	1.19	4.90	Qz Mag He Go Gib Cl K
7	F	MG	1,370	–	–	31.3	14.8	16.2	32.3	0.45	0.59	5.80	–
8	F	MG	1,500	–	–	40.5	22.6	11.4	29.6	0.82	0.88	4.85	Qz Go Gib K
9	F	MG	1,600	–	–	21	10.1	29.8	25.5	0.34	0.60	6.00	Qz Mg He Gib K
2	F	GO	1,500	–	–	30.8	29.2	17.8	30	0.43	0.59	6.10	Qz Gib Mg He Cl K
4	F	GO	1,450	–	–	21	8.7	29.5	25	0.37	0.64	5.40	Qz K He Gib
5	F	GO	1,500	–	–	76.6	65.8	3.1	12.2	2.02	2.34	5.30	K Qz Go Gib M
7	F	GO	1,750	–	–	28.4	26.8	18.7	30.6	0.45	0.63	6.30	–

*notations given below.

TABLE A4.1 (continued)

Sample Number	Soil Group *	Location State *	Rainfall,mm	Moisture Deficience Months	W.P.I. *	SiO^2 Total	SiO^2 Free	Fe^2O^3	Al^2O^3	R^2O^3	$\frac{Sio^2}{Al^2O^3}$	pH	Mineralogy Clay
1	F	PR	2,100	0	– 81.80	27.3	6.5	20.7	29.8	0.82	1.17	5.65	Qz Cl He Go Gib K
3	F	PE	1,850	–	–	63.1	45.9	4.6	16.7	1.20	1.41	3.90	–
4	F	PE	1,450	–	–	63.9	38.4	3.3	21.9	1.49	1.63	3.70	–
2	F	ES	1,200	4	–	57	36.6	7.5	21.5	1.40	1.71	4.60	–
3	F	ES	1,250	0	–	57.6	38.8	7.1	21.5	1.05	1.27	4.00	Qz Gib I K
5	F	ES	1,100	1	– 31.39	57.9	45.3	8.5	19.8	0.94	1.19	6.10	Qz He Cl K
6	F	ES	1,100	–	–	56.5	32.7	7.7	19.3	1.34	1.69	4.65	–
9	F	MA	1,150	–	–	37.2	18.8	34.6	15.6	0.23	0.57	4.20	–
10	F	MA	1,750	–	–	71.2	50.2	5.1	15.4	2.07	2.50	4.85	–
2	F	BA	950	6	– 38.81	36	4.4	20.0	28.3	0.49	0.71	5.15	Qz He Go Gib Cl M K
1	F	BA	1,650	–	– 51.80	37.8	9.9	26	22.8	0.68	1.18	4.40	Qz He Go Gib K
7	F	PI	800	–	–	26.1	13.5	52.3	12.1	0.47	–	4.30	Qz Gib He Go K
9	F	PI	1,100	–	–	46.1	34.8	34.2	11.7	0.23	0.67	4.00	Qz He Go K
10	F	PI	1,100	–	–	43	30.5	34.7	13.5	0.16	0.43	4.45	–
4	F	RN	–	–	– 27.29	56.4	35.4	5.7	21.4	1.07	1.25	5.15	Qz K He Gib
4	F	RS	1,600	0	– 51.75	39.2	26	7.9	21.9	1.11	0.37	3.70	Qz K Gib He
6	F	RS	1,780	0	–	57.5	35.3	6.3	18.2	1.76	2.14	3.90	–
7	F	RS	1,850	0	– 41.86	54.5	16.2	11.9	24.6	1.21	1.56	3.90	Qz He Go K
1	F	MG	1,500	–	–	42.9	19.2	11.7	28	0.90	1.14	4.70	–
2	F	MG	1,000	–	–	71	55	5.3	15.6	1.43	–	5.15	Qz Gib Cl I K
4	F	MG	1,200	–	–	48.1	23.4	11.8	26	0.89	1.15	4.70	–
1	F	CG	1,450	0	– 64.68	25	3.7	31.6	23	0.41	0.78	4.90	Qz Mg He Go Gib K
6	F	SP	1,250	–	–	39.8	22.4	29.1	18.9	0.68	1.36	4.90	–
9	F	SP	1,250	–	–	83.5	73.9	5.1	6.5	1.72	–	5.25	–
11	F	SP	950	–	–	85.9	98.1	2.9	7.0	1.49	–	3.75	–
1	F	SP	1,250	–	–	53.2	36.7	5.6	23.6	0.80	0.92	4.35	–
3	F	SP	1,320	–	–	54.9	35.4	7.1	20.9	1.47	1.78	4.65	Qz Mg He Go Gib Cl K
4	F	SP	1,300	–	–	75.9	63	4.9	12.1	1.35	1.70	4.45	Qz Go K
4	F	CG	1,600	–	–	50.5	42.9	8.2	19.2	1.12	1.42	4.70	Qz M He Gib K
1	F	RJ	–	–	–	–	–	–	–	–	–	–	–
4	F	MT	–	–	–	–	–	–	–	–	–	–	Qz Cl I k

NOTATION: (Symbols used)

Qz	– Quartz	Ba	– Bayerite
Mg	– Magnetite	Gib	– Gibbsite
He	– Hematite	Cl	– Chlorite
Go	– Goethite	I.	– Illite
Bo	– Boehmite	M	– Montmorillonite
K	– Kaolinite	F	– Feldspar

SOIL GROUPS:
A = Acrisols
Q = Arenosols
L = Luvisols
F = Ferralsols

WIP – Weathering Potential Index

RS	– Rio Grande do Sul	PB	– Paraíba	MT	– Mato Grosso
PR	– Paraná	RN	– Rio Grande do Norte	CG	– Campo Grande
SP	– São Paulo	CE	– Ceará	RR	– Roraima
RJ	– Rio de Janeiro	PI	– Piauí	RO	– Rondonia
ES	– Espírito Santo	MA	– Maranhão	AM	– Amazonas
MG	– Minas Gerais	PA	– Pará	AP	– Amapá
BA	– Bahia	GO	– Goiás		
PE	– Pernambuco	DF	– Distrito Federal		

TABLE A4.2

Physical and Engineering Properties of the Red Tropical Soils of South America

Sample Number	Soil Group	AASHO (GI)	%			% passing nº				%	g/cm³	CBR	SG
			LL	PL	PI	10	40	200	<2μ	OMC	MDD		
GO-6	AO14-25	A-6 (0)	28	16	12	100	96	40	20	11	2.036	34	2.68
ES-1	AO1-3a	A-2-7 (2)	46	28	18	93	76	49	24	15	1.900	50	2.68
RS-5	AO1-3a	A-7-6 (19)	43	23	20	99	97	91	46	14	1.924	34	
PR-6	AO1-1a	A-7-5 (9)	46	30	16	99	96	66	42	17	1.832	39	2.8
SP-5	AO6-3b	A-7-6 (4)	48	33	15	100	87	40	21	14	1.896	7	2.69
CE-1	AO2-2a	A-6 (7)	39	21	18	97	86	56	20	13	1.915	23	2.68
CE-2	AO2-2a	A-2-6 (1)	40	24	16	–	41	29	14	16	2.020	–	2.68
CE-8	AO2-2a	A-2-4 (0)	33	26	7	77	52	24	–	–	2.080	24	2.71
MG-6	AO7-3b	A-7-5 (5)	49	32	17	99	78	50	26	14	1.920	44	2.75
MG-12	AO7-3b	A-7-5 (20)	55	38	17	100	99	90	45	23	1.670	45	2.71
PE-2	AO24-3b	A-7-6 (20)	55	28	27	100	98	71	41	19	1.810	22	2.63
MT-1	AO15-3b	A-6 (16)	40	25	15	100	96	93	46	21	1.630	2	2.60
MT-2	AO15-3a	A-7-6 (16)	46	28	18	96	93	83	42	19	1.630	8	2.80
PA-3	Ap8-2a	A-7-5 (24)	55	37	18	100	99	97	48	–	–	–	2.88
BA-4	Ao28-3a	A-2-6 (0)	25	14	11	100	94	34	26	10	2.110	13	2.65
RJ-2	Ao6-3b	A-6 (4)	32	15	17	98	75	47	–	15	1.842	8	–
RJ-3	Ao6-3b	A-6 (3)	38	24	14	99	69	43	–	12	2.039	38	–
RJ-4	Ao6-3b	A-6 (1)	35	24	11	95	76	38	–	12	1.995	26	–
RJ-6	AO1-3b	A-2-4 (0)	20	NP	0	94	58	30	–	8	1.990	7	–
RJ-7	Ao6-3b	A-6 (3)	38	22	16	97	80	42	–	13	1.948	14	–
RJ-9	AO1-3b	A-2-4 (0)	20	NP	0	96	65	26	–	7	2.03[illegible]	7	–
PERU2	Ap6-2a	A-7-6 (11)	52	28	24	100	100	80	40	–	–	–	–
PERU-4	Ao26-3c	A-7-6 (14)	57	27	30	100	99	84	42	–	–	–	–
AC-1	Ao17-2/3a	A-7-6 (7)	53	27	26	97	96	92	46	–	–	–	–
AC-2	Ao17-2/3a	A-7-6 (5)	44	23	21	64	63	42	21	–	–	–	–
PR-2	Ap8-2a	A-4 (3)	20	13	7	100	85	63	36	–	–	–	–
VEN. 1	Ao11-3c	A-7-6 (8)	45	28	17	99	89	70	35	–	–	–	–
VEN. 2	Ao11-3c	A-7-6 (7)	49	27	22	90	82	65	32	–	–	–	–
TP-1	ACRISOL	A-6 (2)	36	23	13	97	69	42	–	13	1.760	25	–
TP-2	ACRISOL	A-7-6 (3)	41	27	14	96	88	50	–	14	1.675	16	–
TP-3	ACRISOL	A-7-5 (14)	52	35	17	99	95	91	–	26	1.550	30	–
TP-4	ACRISOL	A-4 (4)	36	26	10	80	–	57	–	16	1.810	16	–
TP-5	ACRISOL	A-7-5 (4)	53	38	15	96	75	58	–	14	1.825	12	–
TP-6	ACRISOL	A-6 (5)	30	17	13	74	68	67	–	8	2.025	22	–
TP-7	ACRISOL	A-6 (1)	34	22	12	94	–	39	–	16	1.825	9	–
TP-8	ACRISOL	A-7-6 (5)	49	24	25	70	61	40	–	15	1.910	38	–
TP-9	ACRISOL	A-7-5 (12)	50	30	20	95	89	62	–	20	1.730	17	–
TP-10	ACRISOL	A-7-5 (7)	46	30	16	93	88	67	–	16	1.825	12	–
TP-11	ACRISOL	A-5 (9)	52	43	9	94	88	69	–	17	1.800	16	–
TP-12	ACRISOL	A-6 (8)	38	23	15	98	92	75	–	14	1.825	16	–
TP-13	ACRISOL	A-6 (5)	35	19	16	87	79	67	–	15	1.940	20	–
TP-14	ACRISOL	A-7-6 (11)	57	29	28	100	96	76	–	16	1.800	11	–
TP-15	ACRISOL	A-7-5 (14)	50	31	19	100	99	90	–	12	1.675	35	–
TP-16	ACRISOL	A-2-6 (0)	26	14	12	–	32	19	–	6	2.175	35	–
DF-3	Fa2-3a	A-2-6 (0)	35	20	15	–	–	19	15	12	1.830	–	3.01
DF-4	Fa2-3a	A-7-5 (22)	55	33	22	97	96	86	74	26	1.381	25	2.72
PR-2	Fr4-3b	A-7-5 (25)	59	41	18	100	99	97	80	30	1.532	17	2.92
PR-4	Fr4-3b	A-4 (0)	51	46	5	100	100	97	60	29	1.556	–	3.0
PR-5	Fr4-3b	A-7-5 (38)	73	41	32	99	95	93	74	27	1.614	20	3.0
PR-8	Fh2-3a	A-7-5 (40)	75	46	29	100	100	99	58	28	1.557	12	3.0
PR-9	Fh1-3a	A-7-5 (33)	67	40	27	100	99	95	78	28	1.567	20	3.0
MG-7	Fo4-3b	A-7-5 (24)	66	34	32	98	88	73	36	22	1.730	–	2.72
MG-8	Fo4-3b	A-7-5 (21)	63	38	25	98	91	74	38	24	1.600	30	2.70
MG-9	Fr1-3a	A-7-5 (16)	50	34	16	100	93	86	43	24	1.710	20	2.90
GO-2	Fo1-3a	A-7-6 (12)	44	25	19	100	97	69	34	19	1.830	12	3.10
GO-4	Fr1-3a	A-7-5 (23)	57	36	27	100	99	90	45	25	1.660	15	3.0
GO-5	Fa2-2a	A-6 (2)	34	20	14	100	91	45	22	14	1.900	31	2.61
GO-7	Fa22-3a	A-7-5 (18)	52	31	21	91	87	80	40	21	1.670	23	2.70
PR-1	Fh1-3a	A-4 (0)	53	47	6	100	98	97	75	32	1.462	–	2.8
PE-3	Fx8-3b	A-4 (0)	–	–	–	99	80	44	22	14	1.795	14	2.60
PE-4	Fx8-3b	A-7-5 (12)	48	30	18	100	84	66	33	14	1.860	6	2.60
ES-2	Fo4-3b	A-7-5 (18)	71	47	24	100	83	69	38	23	1.618	26	2.60
ES-3	Fo4-3b	A-7-5 (5)	48	34	14	97	77	56	28	15	1.817	38	2.7
ES-5	Fx1-2a	A-7-5 (8)	61	35	26	94	69	49	24	15	1.822	14	2.90
ES-6	Fx1-2a	A-7-5 (17)	61	32	29	98	82	67	34	18	1.789	5	2.67
MA-9	Fa2-2a	A-2-6 (10)	34	22	12			24	14	9	2.102		2.98
MA-10	Fa22-3b	A-6 (6)	35	18	17	94	92	55	32	13	1.927	34	2.65

TABLE A4.2 (continued)

Sample Number	Soil Group	AASHO (GI)	%			% passing nº				%	g/cm³	CBR	SG
			LL	PL	PI	10	40	200	<2μ	OMC	MDD		
BA-2	Fo1-2a	A-7-6 (11)	46	27	19	96	86	64	33	16	1.887	7	2.69
BA-1	Fx5-2a	A-7-5 (14)	51	38	13	99	95	83	42	19	1.768	5	2.78
PI-7	Fx2-2a	A-2-4 (0)	28	19	9	–	–	–	–	10	2.278	–	3.36
PI-9	Fo2-2a	A-2-7 (0)	41	22	19	–	–	18	10	12	1.880	–	3.01
PI-10	Fo2-2a	A-2-4 (0)	31	22	9	–	–	16	8	10	2.198	–	3.04
RN-4	Fo1-2a	A-4 (3)	31	22	9	86	81	55	22	9	1.964	55	2.71
RS-4	Fh1-3a	A-7-5 (24)	61	34	27	97	89	79	45	23	1.640	38	–
RS-6	Fh2-3a	A-7-5 (16)	53	37	16	100	96	79	40	26	1.470	–	–
RS-7	Fh2-3a	A-7-5 (14)	49	36	13	100	97	85	42	24	1.640	–	–
MG-1	Fo4-3b	A-7-5 (28)	68	38	30	100	94	79	44	24	1.600	37	–
MG-2	Fo4-3b	A-6 (5)	36	24	12	77	74	59	–	12	1.980	–	–
MG-4	Fo4-3b	A-7-5 (8)	64	34	30	98	96	65	36	23	1.640	37	2.8
CG-1	Fr3-3a	A-5 (11)	47	44	3	99	95	84	42	26	1.730	39	2.9
SP-6	Fr1-3a	A-7-5 (13)	48	33	15	100	96	77	38	19	1.776	28	–
SP-9	Fo1-2a	A-2-4 (0)	19	16	3	99	99	31	16	10	2.015	46	–
SP-11	Fo9-2a	A-6 (2)	34	17	17	100	99	41	20	–	–	–	–
SP-1	Fo1-3c	A-7-5 (4)	61	50	11	79	65	48	24	18	1.798	–	–
SP-3	Fo1-3a	A-7-5 (17)	51	32	19	99	98	79	40	21	1.675	42	–
SP-4	Fr1-3a	A-2-4 (0)	28	21	7	94	90	33	16	9	2.052	17	–
CG-4	Fa2-3a	A-6 (5)	35	21	14	100	96	58	32	16	1.840	15	–
RJ-1	Fo1-3c	A-6 (3)	31	19	12	100	78	51	–	15	2.131	47	–
MT-4	Fa2-3a	A-7-5 (17)	58	30	28	100	99	93	–	–	–	–	–
MT-5	Fa2-3a	A-6 (6)	34	20	14	100	98	74	–	–	–	–	–
MT-6	Fa14-3b	A-4 (4)	26	16	10	100	98	70	–	–	–	–	–
IQUITOS	Fx3-2a	A-7-6 (6)	54	24	30	99	94	57	28	–	–	–	–
RO-1	Fx3-2a	A-7-6 (12)	41	26	15	99	99	93	46	–	–	–	–
AP-1	Fx3-2a	A-2-6 (0)	29	17	12	–	–	18	9	–	–	–	–
AP-2	Fx3-2a	A-6 (4)	56	32	24	97	83	60	35	–	–	–	–
AM-1	Fx6-3a	A-7-5 (19)	76	41	35	100	96	86	56	–	–	–	–
RR-1	Fo14-3b	A-6 (3)	37	25	12	80	76	55	28	–	–	–	–
GUIANA 1	Fo16-2a	A-7-5 (3)	59	32	27	–	–	46	23	–	–	–	–
PE-7	Qf12-1a	A-4-(1)	25	15	10	74	40	22	11	6	2.180	46	2.61
PE-8	Qf12-1a	A-2-4 (0)	22	13	9	100	84	31	18	8	2.130	44	2.61
MA-4	Qf5-1a	A-2-4 (0)	23	15	8	18	17	9	4	7	2.038	205	3.22
MA-6	Qf5-1a	A-2-4 (0)	20	NP	0	100	99	16	8	7	1.973	37	2.65
MA-7	Qf3-1a	A-2-6 (0)	35	19	16	30	29	19	10	7	2.168	132	2.93
MA-8	Qf3-1a	A-2-4 (0)	27	19	8	33	33	18	8	8	2.214	116	2.88
BA-11	Qf12-1a	A-2-4 (0)	20	NP	0	100	99	27	13.5	8	2.072	105	2.63
PI-1	Qf14-1a	A-6 (2)	37	21	16	47	46	37	18	9	2.092	108	2.93
PI-2	Qf14-1a	A-2-4 (0)	28	19	9	28	27	18	9	8	2.180	116	3.08
PI-3	Qf14-1a	A-2-6 (1)	40	23	17	36	33	24	13	11	2.228	38	3.10
PI-4	Qf14-1a	A-1-a (0)	22	16	6	20	19	10	5	9	2.323	180	3.52
PI-6	Qf14-1a	A-2-6 (0)	38	23	15	17	16	13	7	13	2.172	172	3.15
PI-8	Qf3-1a	A-2-6 (1)	35	20	15	43	41	28	14	9	2.108	132	2.67
RN-1	Qf3-1a	A-4 (0)	20	NP	0	93	76	36	18	9	2.068	35	2.65
RN-2	Qf13-1a	A-6 (1)	32	20	12	–	–	–	–	12	1.950	36	2.79
RN-3	Qf13-1a	A-2-4 (0)	20	NP	0	92	82	26	13	9	2.070	–	2.69
CE-7	Qf1-1a	A-2-4 (0)	20	NP	0	100	95	17	9	–	–	–	2.67
PB-1	Qf16-1a	A-6 (5)	34	22	12	99	84	57	28	12	1.943	44	2.60
PB-2	Qf11-1a	A-6 (2)	30	17	13	100	89	45	22	9	2.061	70	2.60
PB-9	Qf11-1a	A-6 (3)	32	20	12	96	82	49	24	11	2.040	30	2.62
GO-8	Qf3-1a	A-2-4 (0)	21	17	4	97	88	34	17	7	2.138	100	2.60
GO-9	Qf3-1a	A-2-4 (0)	20	NP	0	100	84	15	8	6	2.078	50	2.66
SP-8	Qf7-1a	A-2-4 (0)	23	17	6	100	99	32	16	7	2.120	18	–
CG-3	Qf17-1a	A-2-4 (0)	20	NP	0	100	98	20	10	8	2.090	58	–
TP1	ARENOSOL	A-2-4 (0)	20	NP	0	98	86	35	–	7	2.075	160	–
TP2	ARENOSOL	A-6 (1)	27	16	11	56	52	43	–	10	2.150	45	–
TP3	ARENOSOL	A-1-b (0)	20	NP	0	38	37	19	–	8	2.210	230	–
TP4	ARENOSOL	A-2-4 (0)	20	16	4	56	50	32	–	9	2.170	168	–
TP5	ARENOSOL	A-2-4 (0)	20	NP	0	97	–	29	–	8	2.150	110	–
TP6	ARENOSOL	A-4 (0)	21	14	7	87	81	39	–	10	2.075	85	–
TP7	ARENOSOL	A-4 (0)	18	11	7	94	88	42	–	9	2.125	65	–
TP8	ARENOSOL	A-2-6 (0)	28	16	12	62	55	34	–	11	2.000	168	–
TP9	ARENOSOL	A-6 (3)	25	14	11	72	71	56	–	10	2.025	25	–
TP10	ARENOSOL	A-3 (0)	20	NP	0	100	91	4	–	7	1.808	42	–
TP11	ARENOSOL	A-2-4 (0)	20	NP	0	98	94	18	–	6	2.060	88	–
PA-1	Fx3-2a	A-1-b (0)	20	16	4	–	–	25	12	21	1.712	23	–
PA-2	Fx3-2a	A-2-6 (0)	31	19	12	73	–	23	12	8	2.134	–	–
PA-5	Fx4-3a	A-2-4 (0)	26	16	10	96	80	39	20	9	2.135	11	–
PA-6	Fx3-2a	A-4 (0)	24	16	8	96	86	40	20	9	2.088	29	–
PA-7	Fx3-2a	A-7-5 (23)	53	30	23	92	92	88	44	22	1.642	18	–

TABLE A4.2 (continued)

Sample Number	Soil Group	AASHO (GI)	%			% passing nº				%	g/cm³	CBR	SG
			LL	PL	PI	10	40	200	<2μ	OMC	MDD		
DF-1	Fa2-3a	A-2-7 (0)	54	34	20	–	–	–	–	12	1.765	–	3.30
DF-2	Fa2-3a	A-2-6 (0)	33	20	13	–	–	16	12	10	1.900	–	2.98
PE-5	Lc3-3b	A-6 (4)	32	19	13	94	79	60	33	9	2.025	10	2.55
PE-6	Lf22-3b	A-7-5 (16)	–	36	19	96	88	76	43	19	1.785	24	2.64
PE-9	Lc2-3b	A-4 (3)	26	16	10	90	72	44	22	10	2.050	19	2.59
MAR-1	Lf9-3b	A-2-6 (0)	39	25	14	36	34	27	14	10	2.135	90	2.79
MAR-2	Lf9-3b	A-6 (11)	34	22	12	94	93	87	44	17	1.790	6	2.70
MAR-5	Lf9-3b	A-2-4 (0)	27	17	10	30	29	12	9	6	2.192	–	2.66
BAHIA-6	Lc2-3b	A-6 (4)	34	21	13	90	78	53	36	11	2.066	29	2.67
BAHIA-7	Lf23-2a	A-6 (3)	27	16	11	88	79	53	27	8	2.132	41	2.63
BAHIA-10	Lc2-3b	A-4 (6)	32	15	17	96	81	52	37	11	2.009	27	2.64
RGN-5	Lc9-3b	A-6 (1)	34	22	12	97	79	50	25	11	2.012	64	2.81
RGN-6	Lf2-3b	A-6 (0)	31	20	11	86	82	67	34	13	1.859	12	2.62
RGN-7	Lf16-3b	A-6 (4)	32	20	11	97	85	57	28	12	1.882	25	2.58
CEARÁ-3	Lc2-3b	A-4 (1)	27	17	10	98	81	56	28	10	2.080	45	2.67
CEARÁ-4	Lc2-3b	A-2-4 (0)	30	21	9	85	63	30	15	8	2.235	17	2.72
CEARÁ-5	Lf2-2a	A-6 (8)	34	23	11	71	61	57	28	10	2.035	8	2.69
CEARÁ-10	Lc2-3b	A-6 (3)	39	26	13	97	92	47	24	13	1.895	10	2.75
PB-3	Lf22-3b	A-6 (2)	32	20	12	96	82	49	32	11	2.035	36	2.60
PB-4	Lf22-3b	A-6 (2)	30	17	13	98	81	45	24	11	2.010	44	2.68
PB-5	Lf22-3b	A-6 (5)	37	20	17	96	79	50	30	11	2.015	72	2.60
PB-8	Lc2-3b	A-6 (3)	37	20	17	88	73	43	22	10	2.020	54	2.64
PB-10	Lc9-3b	A-2-4 (0)	20	NP	0	96	81	29	14	14	1.875	6	2.67
PB-11	Lc9-3b	A-6 (1)	29	17	12	98	82	42	22	8	2.230	58	2.72
MG-13	Lf4-3b	A-7-5 (31)	–	36	25	99	98	97	44	23	1.610	–	2.75
GO-3	Lf3-3b	A-6 (8)	38	24	14	97	93	68	34	22	1.677	13	2.80
SP-12	Lfl-1a	A-2-6 (0)	25	14	11	100	93	29	14	10.8	2.032	23	–
SP-13	Lf1-1a	A-6 (3)	28	16	12	100	100	51	26	9	1.926	36	–
RJ-5	Lf5-2b	–	–	37	23	91	74	57	–	20	1.674	14	–
RJ-8	Lf5-2b	–	–	37	29	96	93	85	–	22	1.663	17	–
TITICACA1	Lc10-3a	–	37	21	16	99	96	90	45	–	–	–	–
MG-5	Lf1-1a	A-4 (0)	23	14	9	100	97	36	18	10	2.060	94	–
SP-7	Lf23-2/30	A-4 (0)	28	18	10	100	98	39	20	15	1.860	56	–
TP-1	Lc2-3b	A-2-4 (0)	20	18	2	91	54	30	–	6	1.915	32	–
TP-2	Lc2-3b	A-1-b (0)	23	17	6	55	46	21	–	6	1.950	8	–
TP-3	Lc9-3b	A-4 (0)	26	17	9	91	77	37	–	6	1.836	8	–
TP-4	Lc9-3b	A-2-4 (0)	28	20	8	95	67	28	–	6	1.813	3	–
TP-5	Lc9-3b	A-7-6 (3)	41	27	14	76	52	50	–	5.5	1.820	11	–
TP-6	Lc2-3b	A-2-6 (1)	27	13	14	83	63	33	–	6.5	2.190	55	–
TP-7	Lc2-3b	A-1-b (1)	20	NP	0	70	47	24	–	6.0	2.170	55	–
TP-8	Lc2-3b	A-2-4 (0)	20	NP	0	76	51	24	–	7.0	2.235	50	–
TP-9	Lc2-3b	A-2-4 (0)	20	NP	0	68	53	27	–	7	2.140	50	–
TP-10	Lc7-3b	A-2-6 (0)	30	19	11	91	64	32	–	8	2.100	10	–
TP-11	Lc9-3b	A-2-4 (0)	20	NP	0	92	70	32	–	7	2.080	40	–
TP-13	Lc9-3b	A-2-4 (0)	20	NP	0	72	56	31	–	6	2.165	85	–
TP-14	Lc9-3b	A-2-4 (0)	20	NP	0	76	59	26	–	6.3	2.140	135	–
TP-15	Lc9-3b	AA-2-4 (0)	25	15	10	91	68	47	–	7.5	2.135	55	–
TP-16	Lc9-3b	A-4 (1)	20	NP	0	83	49	23	–	6.0	2.160	105	–
TP-17	Lf22-3b	A-1-b (0)	20	NP	0	99	67	13	–	7.0	2.060	60	–
TP-18	Lc9-3b	A-2-4 (0)	20	NP	0	67	50	18	–	7.0	2.075	90	–
TP-19	Lc9-3b	A-1-b (0)	23	NP	0	47	40	21	–	9.0	2.025	42	–
TP-20	Lc2-3b	A-1-b (0)	34	23	11	40	31	21	–	10.5	1.990	40	–
TP-21	Lc2-3b	A-2-6 (1)	21	NP	0	66	50	25	–	7.5	2.125	74	–
TP-22	Lf16-3b	A-4 (4)	17	14	3	90	86	84	–	6.0	2.160	40	–
TP-23	Lc9-3b	A-6 (12)	29	18	11	99	99	85	–	12	1.953	23	–
TP-24	Lc9-3b	A-4 (1)	25	18	8	52	47	43	–	10	2.115	56	–
TP-25	Lc9-3b	–	–	–	–	58	52	48	–	13	1.993	6	–
TP-26	Lc9-3b	A-2-4 (0)	20	NP	0	92	88	28	–	5	1.894	37	–
TP-27	Lc9-3b	A-6 (3)	32	19	13	95	94	57	–	11	1.972	29	–
TP-28	Lc9-3b	A-4 (4)	27	19	8	91	89	61	–	11	2.025	35	–
TP-29	Lf4-3b	–	–	–	–	98	97.5	97	–	23	1.600	14	–
TP-30	Lf4-3b	–	–	–	–	97	93	86	–	21	1.725	60	–
TP-31	Lf4-3b	A-7-6 (4)	41	28	13	99	96	92	–	22	1.625	55	–
TP-32	Lf1-1a	–	–	–	–	99	98	98	–	23	1.660	30	–
RS-8	Nd1-3a	A-7-5 (11)	67	37	30	97	96	54	24	26	1.562	24	–
RS-9	Nd1-3a	A-6 (10)	42	23	19	100	99	54	–	16	1.847	31	–
ES-4	Nell/3c	A-7-5 (15)	56	35	21	99	76	68	8	18	1.728	18	–
PR-5	Nel-3b	A-7-5 (38)	56	38	18	98	97	93	74	27	1.614	20	–
SP-2	Ne1-3b	A-7-5 (19)	53	36	17	99	98	88	42	26	1.596	–	–
Varginha	Ne1-3b	A-7-5 (19)	57	36	21	99	93	78	64	–	–	–	2.80
SP-51	Ne1-3b	A-6 (8)	39	13	16	96	94	61	32	–	–	–	2.68
RS-55	Nd2-3c	A-7-6 (10)	45	25	20	95	82	61	38	–	–	–	2.70
RS-57	Nd2-3c	A-7-5 (20)	66	38	28	99	80	66	45	–	–	–	2.69
RS-53	Nd1-3a	A-7-5 (25)	73	53	20	99	86	83	52	–	–	–	2.98
RS-54	Nd1-3a	A-7-5 (25)	63	45	18	99	98	95	53	–	–	–	2.70

TABLE A4.3

Chemical Analysis
By X-Ray Fluorescence Weight, Per Cent

No.	Total SiO_2	FREE SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	K_2O	TiO_2	LOI*	Undet
Bahia										
1.	40.0	9.9	23.8	23.3	.1	.2	.6	2.0	10.0	.0
2.	60.0	4.4	11.0	19.7	.1	.2	.9	.9	7.3	.0
3.	58.2	28.2	6.0	25.4	.5	.2	.2	.7	8.9	.0
4.	68.5	34.2	2.2	9.5	.1	.1	.3	.6	3.6	15.0
5.	53.8	28.9	11.5	23.9	.1	.3	.9	.9	9.0	.0
6.	69.6	28.7	6.3	13.8	.1	.4	.9	.7	4.8	3.4
7.	66.3	53.8	3.2	15.1	.1	.2	4.8	.6	3.6	6.1
9.	57.5	29.7	7.9	18.2	2.4	.7	2.4	.8	5.4	7.1
10.	74.7	60.5	3.8	13.9	.3	.2	.4	.6	1.3	5.5
11.	85.9	80.0	1.7	6.4	.2	.4	1.2			
Brasilia										
1.	20.6	2.4	50.5	14.0	.1	.1	.1			
2.	32.9	14.6	36.2	17.8	.2	.1	.1			
3.	38.0	19.9	36.7	12.9	.1	.1	.1			
4.	21.3	5.6	15.4	33.7	.2	.1	.1			
Campo Grande										
1.	28.7	3.7	30.2	25.1	.1	.5	.2	5.1	10.1	.0
3.	85.0	86.9	2.6	4.8	.4	.8	.3	.7	1.6	3.8
4.	59.6	42.9	8.2	21.6	.2	.5	.3	1.2	8.4	.0
5.	55.0	26.7	5.4	22.2	.2	1.5	2.7			
6.	75.7	61.7	4.4	13.2	.2	.1	.6			
Ceará										
1.	70.9	51.2	3.0	17.9	.2	.1	.2			
2.	52.7	20.6	11.2	24.3	.2	.3	.4	1.4	9.4	
3.	65.6	51.7	3.4	15.9	.8	.4	2.5			
4.	55.2	25.8	7.3	9.2	3.6	1.7	2.5	1.2	3.2	16.1
5.	60.1	35.3	7.2	15.5	.9	2.5	2.8			
6.	81.0	75.7	2.2	8.0	.1	.2	.3	1.0	4.0	3.3
7.	92.5	87.4	.4	5.0	.2	.1	.06			
8.	66.2	47.2	4.3	16.4	.8	.4	3.6	.6	2.4	5.3
9.	88.2	82.9	1.2	6.7	.1	.2	1.7	.2	.5	5.2
10.	52.8	22.8	9.6	22.1	.8	1.3	1.5	1.1	5.1	5.7
Cuiabá										
1.	67.2		4.7	17.3	1.0	.5	3.0	.7	5.6	.0
2.	58.1		10.6	13.5	.2	.5	1.4	.9	4.8	.0
3.	90.7	66.2	1.3	4.6	.2	.5	.7			
4.	65.8	40.1	6.0	18.8	.1	.4	2.2	1.0	5.8	.0
5.	65.0	49.3	5.2	21.0	.2	.4	.05			
6.	58.0	43.9	7.2	22.4	.1	.3	.2	1.3	10.5	.0
7.	26.3	8.0	9.0	36.4	.2	.4	.05			
Espírito Santo										
1.	75.5	59.3	6.1	11.3	.1	.1	.06			
2.	57.0	36.6	7.5	21.5	.1	.1	.1			
3.	62.8	38.3	6.5	23.1	.1	.4	.9	.9	1.5	3.7
4.	54.2	22.3	7.3	24.0	.2	.4	.4	.9	9.0	3.5
5.	61.4	45.3	8.1	18.7	.1	.2	.3	1.2	7.0	3.0
6.	56.5	32.7	7.7	19.3	.2	.1	.1			
Goiás										
1.	60.8	52.8	9.6	18.6	.2	.3	.7			
2.	37.2	29.2	17.7	27.2	.1	.3	.3	3.4	13.7	.0
3.	29.2	19.1	18.7	32.0	.2	.6	.9	3.9	14.6	.0
3.A	46.3	39.5	12.0	26.0	.3	.5	.9	2.1	11.9	.0
4.	23.2	8.7	29.2	26.6	.3	.7	.3	7.7	12.1	.0
5.	63.7		5.2	20.1	.3	.5	.3	1.8	8.0	.0
Minas Gerais										
1.	42.9	19.2	11.7	28.0	.2	.1	.1			
2.	67.3	55.0	6.1	18.5	.2	.2	1.2	.8	5.6	.0
3.	39.9	32.2	33.5	16.2	.2	.5	2.4			
4.	48.1	23.4	11.8	26.0	.2	.4	.4			
5.	76.5	74.4	4.8	11.6	.3	.4	.3	1.2	5.0	.0
6.	67.7	48.8	6.1	18.6	.07	.2	1.0	.9	5.5	.0
7.	31.3	14.8	16.2	32.3	.2	.1	.02			
8.	49.2	22.6	10.4	26.4	.2	.3	.3	.4	11.9	.0
9.	26.9	10.1	26.8	25.1	.3	.5	.3	7.6	12.5	.0

Table A4.3 (continued)

No.	Total SiO_2	FREE SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	K_2O	TiO_2	LOI*	Undet
10.	79.6	70.2	7.2	6.8	.2	.1	.02			
11.	36.6	19.2	31.1	15.0	.1	.2	.4	3.5	13.1	.0
12.	52.0	46.6	12.5	21.8	.2	.1	1.2			
13.	42.1	21.8	12.8	27.7	.2	.1	.06			
Paraíba										
1.	57.1	36.0	3.3	24.4	.2	.2	4.6	.4	5.9	3.8
2.	75.6	41.1	3.3	14.5	.3	.3	.1	.9	5.1	.0
3.	75.9	61.4	3.0	13.5	.1	.1	.1			
4.	69.5	60.0	3.2	14.0	.1	.2	.3	1.0	5.3	6.4
5.	74.7	65.7	3.8	14.7	.2	.4	.4	.7	5.2	.0
6.	60.0	28.1	8.0	14.9	2.2	1.3	3.0	.8	6.5	3.4
7-1	67.8		7.4	16.5	.6	.5	1.2	.8	5.3	.0
7-2	64.1		7.1	15.4	.9	.6	1.1	.7	5.8	4.4
8.	64.9	58.6	5.6	17.7	.2	.4	6.7	.6	3.8	.0
9.	81.3	78.6	3.6	12.3	.2	.4	.3	.9	1.0	.0
10.	54.4	52.1	15.7	17.9	.2	.8	1.5	3.0	6.5	.0
11.	55.3	40.8	7.4	15.9	3.5	1.6	.9			
Paraná										
1.	30.2	6.5	19.5	31.8	.4	.4	.1	4.1	13.4	.0
2.	37.8		19.5	28.2	.4	.5	.1	3.3	10.2	.0
3.	84.8	76.0	3.4	7.3	.2	.2	.1		2.8	
4.	23.4	.7	28.4	27.0	.1	.4	.3	5.0	10.6	4.7
5.	33.8	4.9	25.8	23.3	.2	.7	.4	6.7	9.2	.0
6.	62.0	48.5	9.8	15.6	.2	.4	.5	2.1	5.1	4.4
7.	58.6	44.9	6.6	21.9	.1	.2	.7	1.4	8.6	1.9
8.	29.6	.9	27.2	26.2	.4	.1	.1			
9.	27.5	5.9	29.0	31.1	.1	.4	.4	5.4	3.4	2.7
Pernambuco										
1.	79.7	68.9	2.5	7.9	.4	.4	.3	.6	3.0	5.2
2.	56.9	32.6	5.4	23.8	.1	.3	.3	.9	9.4	3.1
3.	68.0	45.9	4.1	18.0	.1	.4	3.9	.5	4.9	.0
4.	63.9	38.4	3.3	21.9	.2	.1	1.6			
5.	65.3	43.0	3.5	19.9	.3	.4	4.0	.6	6.2	.0
6.	51.4	23.9	6.7	25.2	.2	.2	.8			
7.	78.1	66.5	2.1	11.4	.4	.2	2.3	.3	2.2	2.9
8.	79.3	64.0	1.4	11.4	.2	.1	1.0			
9.	62.8	48.0	3.9	19.1	.4	.2	5.5	.6	4.3	3.3
Piauí										
1.	52.6	36.3	26.5	12.2	.2	.3	.5	1.0	6.7	.0
2.	42.0	31.2	37.6	11.9	.2	.1	.2			
3.	49.4	27.5	29.4	13.0	.2	.4	1.0	.7	6.1	.0
4.	20.2	8.1	58.5	13.0	.2	.1	.3			
5.	35.1	4.4	22.7	27.5	.3	.5	.3	1.9	11.8	.0
6.	29.5	14.2	43.1	15.8	.3	.1	.3			
7.	30.8	13.5	48.9	11.3	.2	.5	.3	2.4	5.5	.0
8.	64.0	6.1	14.9	13.6	.1	.1	.7	.9	5.8	.0
9.	48.4	34.5	31.2	11.6	.1	.4	.4	.9	7.1	.0
10.	43.0	30.5	34.7	13.5	.2	.1	.2			
Rio Grande do Norte										
1.	77.0		2.8	12.4	.3	.4	.1	.5	3.9	2.6
2.	65.6		4.0	16.5	.1	.2	3.8	.4	4.6	5.7
3.	54.2		8.9	13.4	1.3	.4	3.2	.3	2.2	16.1
4.	62.1		5.8	22.0	.4	.5	1.8	.6	6.8	.0
5.	55.0		6.5	13.8	3.4	1.4	1.2	.5	4.0	14.4
6.	70.7		3.1	13.4	.3	.4	1.8	.5	4.0	5.8
7.	50.6		4.6	19.9	.2	.3	3.1	.5	20.8	.0
8.	47.8		14.4	22.8	.1	.1	.3	1.8	9.1	3.6
Rio Grande do Sul										
1.	65.6	25.3	7.8	15.2	.8	.7	.8	1.1	8.0	.0
2.	62.2	24.6	9.2	14.9	1.3	.4	.8	1.5	9.7	.0
3.	61.5	37.3	8.7	13.8	.6	.4	1.0	1.6	7.9	4.5
4.	54.3	25.0	8.4	23.8	.1	.4	.6	.8	9.2	2.3
5.	73.2	61.4	4.0	12.9	.1	.2	.4	.8	5.0	3.3
6.	57.5	35.3	6.3	18.2	.2	.3	.2			
7.	43.6	16.2	21.4	21.9	.1	.3	.4	2.6	9.7	.0
8.	49.3	12.8	12.8	25.1	.1	.4	.3	1.9	10.0	.0
9.	71.5	57.0	6.0	15.0	.3	.4	.4	1.2	5.2	.0
São Paulo										
1.	58.2	36.7	5.6	23.6	.4	.2	.1			
2.	45.6	19.4	13.8	26.4	.1	.4	.4	2.6	10.8	.0
3.	55.2	35.4	7.0	22.2	.1	.2	.3	1.9	9.4	3.2
4.	79.8	63.0	3.3	11.0	.3	.4	.3	1.0	4.0	.0
5.	62.2		2.6	8.5	.1	.4	.2	1.4	4.1	20.6

* Loss on ignition (600°C)

TABLE A4.4
Minerals Identified By X-Ray Diffraction

	Kaol.	Montmor.	Illite	Chlorite	Gibbs.	Geothite	Bayerite	Boehmite	Hematite	Magnet.	Diaspore	Feldspar	Quartz
BAHIA													
1	✓				✓	✓			✓				✓
2	✓	✓		✓	✓	✓			✓				✓
3	✓	✓	✓	✓									✓s
4	✓				✓								✓s
5	✓ss				✓								✓s
6	✓s				✓	✓			✓				✓s
7	✓				✓							✓s	✓s
9	✓W										✓	✓	✓s
10	✓s				✓	✓							✓s
11	✓	✓											✓s
CAMPO GRANDE													
1	✓				✓	✓			✓	✓			✓W
3	✓				✓								✓s
4	✓	✓W			✓				✓				✓s
CEARÁ													
2	✓		✓W		✓				✓				✓s
4			✓						✓	✓		✓	✓
6	✓s												✓
8	✓	✓	✓									✓	✓s
9	✓	✓W											✓s
10	✓W		✓	✓								✓	✓W
CUIABÁ													
1	✓		✓		✓								✓s
2	✓		✓	✓	✓	✓							✓s
4	✓		✓	✓									✓
6	✓				✓								✓s
E. SANTO													
3	✓		✓	✓	✓								✓s
4	✓												✓s
5	✓								✓				✓

Table A4.4 (continued)

	Kaol.	Montmor.	Illite	Chlorite	Gibbs.	Geothite	Bayerite	Boehmite	Hematite	Magnet.	Diaspore	Feldspar	Quartz
GOIÁS													
(Formosa)													
2	√			√	√s				√	√			√s
3	√				√				√	√W			√s
3A	√				√								√s
4	√W				√				√				√W
5	√s	√			√	√							√
MINAS GERAIS													
2	√		√	√	√								√s
5	√				√		√		√				√s
6	√s		√s		√								√s
8	√				√	√							√s
9	√				√				√	√			√
11	√s				√	√							√s
PARAÍBA													
1	√			√W								√	√s
2	√			√W									√s
4	√			√	√								√s
5	√												√
6		√s	√W		√							√	√s
7-1		√			√							√	√s
7-2				√?	√							√	√
8	√		√		√							√	√s
9	√				√								√s
10	√	√W	√s		√W							√	√s
11	√	√	√									√	√
PARANÁ													
BR-116													
1	√			√W	√	√			√				√W
2	√			√					√				√W
3	√								√				√s
4	√			√	√				√	√W			√W
5	√							√	√				√s
6	√				√				√				√s
7	√	√W		√	√	√							√s
8	√	√W				√			√				
9	√		√	√	√	√			√	√			√s

TABLE A.4 (Continuation)

	Kaol.	Montmor.	Illite	Chlorite	Gibbs.	Geothite	Bayerite	Boehmite	Hematite	Magnet.	Diaspore	Feldspar	Quartz
PIAUÍ													
1	√					√			√				√s
3	√		√			√			√				√s
5	√					√							√s
7	√				√W	√			√				√W
8	√					√							√s
9	√					√			√W				√s
RIO G. NORTE													
1	√								√W				√s
2	√			√								√	√s
3		√		√W					√			√	√s
4		√		√						√W			√s
5		√	√	√						√		√	√s
6	√											√	√
7	√	√		√	√							√	√s
8	√			√W					√				√s
RIO G. SUL													
1		√	√										√s
2			√	√								√	√s
3		√										√	√s
4	√s				√				√				√s
5	√												√s
7	√					√			√				√s
8	√s	√?							√				√s
9	√	√?			√				√				√s
SÃO PAULO													
2	√	√W		√					√	√			√s
3	√			√	√	√			√	√			√s
4	√					√							√
5	√s			√	√	√							√s

w – weak
s – strong

TABLE A.4.5
Properties of African Soils

LAB NO.	SOIL GRP	AASHO	GI	LL	PL	PI	4	10	40	200	2μ	OMC	(1) MDD	(2) CBR	(3) CBR	SG	pH
GO920	JB1	7		41	20	21	54	23	13	8	4						6.2
GO461	JC1	4		22	17	5	93	90	75	24	5						7.2
GO150	JC1	7	0	44	15	29	70	28	10	5							6.8
GO520	JC1			32	20	12	83	49				16	1.875		22	2.74	
GO910	JC1	7	4	57	16	40	74	49	42	35	16						6.95
GO940	JC1	7	2	53	27	26	84	62	46	30	17						5.8
GO950	JC1	6	1	31	12	19	88	68	46	27	12	16	1.971		7		6.6
GO112	JC1	6	0	40	18	22	72	42	20	5		16	1.875		23		7.2
GO114	JC1	4		20	11	9	59	34	13	5							
G116A	JC1	10	9	35	18	17	93	77	74	67	23	15	1.811		16	2.44	6.
G116B	JC1	10	2	29	16	13	90	67	63	45	18	12	2.019		10	2.77	7.2
G116C	JC1	10	4	27	14	13	96	74	67	50	16	13	1.955		15	2.58	6.25
G117A	JC1	7	4	49	21	28	65	47	38	34	22	11	2.115		25	2.71	5.4
G117B	JC1	12	15	65	25	40	88	57	53	50	28	13	1.843		3	2.51	5.85
GO511	BCJD	4	0	28	12	16	71	43	14	6		12	2.051		30	2.56	
GO530	BCJD			36	17	19	64	30				13	2.147		20	2.65	
GO550	BCJD			40	16	24	54	27				13	2.115		24	2.55	
GO6[illegible]0	BCJD	6	0	34	17	17	52	33	22	15	6					2.53	
GO620	BCJD	4	0	23	16	7	61	28	19	15	4					2.50	
GO930	BCJD	7	2	42	23	19	72	44	34	26	11	10	2.164		35	2.98	6.9
GO96A	BCJD	6	1	34	14	20	50	37	27	18	8						6.8
GO96A	BCJD	6	1	30	15	15	92	69	45	26	12	9	2.083		31		6.8
G1010	KB2	12	15	45	26	19	100	100	98	76	59	23	1.603		16	2.74	
GS100	KC2	12	8	43	19	25	84	64	60	47	35	17	1.779		22		
GO21P	KC2	7	0	48	22	26	44	24	12	3		15	1.875		8		
GO23P	KC2	12	18	58	28	30	100	95	75	65		26	1.506		7		
GO315	KC2	7	2	57	22	35	54	34	29	23	15					2.26	
GO32L	KC2	7	3	52	21	31	85	64	42	32	25	21	1.603		18		
GO33L	KC2	7	0	56	21	35	43	16	8	6	4	17	1.859		6	2.41	
GO630	KC2	6	1	38	17	21	82	40	30	20	15	17	1.859			2.73	
GO700	KC2	11	16	58	31	27	93	80	75	63	40	27	1.538			2.56	
GO810	KC2	7	0	48	24	24	60	32	25	23	16	12	1.987		3	2.76	
G1020	KC2	7	0	47	21	26	40	21	15	12	7	22	1.875		21	2.90	
G1030	KC2	9	3	46	36	10	100	100	67	50	27	18	1.731		23	2.56	
GS100	KC2	12	08	43	19	25	84	64	60	47	35	17	1.779		22		
GO820	LC3	7	4	49	24	25	53	30	25	23	13				17	2.56	
G115A	LC3	11	13	58	37	21	69	46	45	40	17	20	1.570		7	2.68	4.5
G115B	LC3	11	10	55	35	20	91	86	80	75	30					2.68	
G115C	LC3	11	3	54	31	23	89	68	59	50	19					2.74	3.9
G131A	LH3	6	0				54	36	31	24	8	11	2.196		25	3.05	
G132A	LH3	7	0	55	43	12	54	26	25	23	14	14	2.034		7	2.89	
G132B	LH3	12	6				78	49	48	42	22	20	1.747			2.96	
GO830	LX3	7	0	63	28	35	33	16	15	14	10				33	2.71	
GO840	LX3	7	3	65	29	36	58	31	28	25	14					2.81	
G111A	LX3	12	8	44	20	24	83	64	56	41	30	13	2.051		8	2.77	
MAL01	J1	10	0	25	14	11	58	44	41	39	11	10			66	2.87	4.15
NIA06	J1	6	1	26	11	14	81	70	36	31	15	9	2.099		10	2.93	
SU003	J1	4	0	30	23	7	75	52	27	10						2.63	
NIA01	JA1	4	0	21	11	10	64	55	40	25	14	9	2.147		22	2.58	
NIA03	JA1	4	0	19	11	8	91	88	76	29	16	8	2.083		15	2.69	
SEN01	JA1	6	0	39	20	19	46	33	27	22	12					2.56	
SEN02	JA1	10	6	30	16	14	100	100	77	43	22					2.55	
SEN06	JA1	6	3	40	19	21	62	44	47	27	17					2.55	
SU004	JA1	4	0	21	12	9	100	98	57	27	19					2.76	6.95
TA1A3	JA	10	6	34	19	15	100	100	93	61						2.73	
TA1A4	JA	12	30	73	29	44	100	99	89	68		16				2.58	
TA1A2	JA	6	1	33	15	18	100	100	75	29							
SEN03	JAJD	6	0	24	13	11	51	34	29	18	9					2.57	
SEN05	JAJD	10	2	27	16	11	100	100	86	47	23					2.60	
UOO10	JB1	7	0	45	31	14	88	52	40	28	19	19			52	2.95	5.4
DAH01	JC1	10	1	38	26	12	81	63	51	40	19	15	1.843			2.64	
DAH1A	JC1	12	3	44	21	23	85	62	50	39	16				56	2.91	
IV001	JC1	7	1	44	24	20	73	60	42	30	16	14	1.939		15	2.79	4.1
IV003	JC1	6	1	37	21	16	63	57	45	29	18	10	2.035		29	2.88	4.9
MAL04	JC1	4	0	36	27	9	59	51	42	34	8	15	1.843		75	2.63	
NIA05	JC1	6	1	27	16	11	52	36	25	18	8	8	2.260		63	2.85	5.1

(1) g/cm^3
(2) as molded
(3) soaked 4 days

TABLE A4.5 (Continuation)

LAB. NO.	SOIL GRP	AASHO	GI	LL	PL	PI	4	10	40	200	2μ	OMC	(1) MDD	(2) CBR	(3) CBR	SG	pH
NIA11	JC1	4	0	19	12	10	98	93	45	19	1						3.65
NIA12	JC1	1	0	19	15	4	54	44	35	15	6						4.7
NIA13	JC1	4	0	21	13	7	62	48	38	30	12						6.
NIA14	JC1						52	40	35	22	10						
UV001	JC1	12	9	43	24	19	90	82	71	58	34	22	1.651		20	2.80	
UVO1A	JC1	12	9	43	27	16	95	88	78	64	38	19	1.795		6	2.80	
UV002	JC1	6	1	38	14	24	66	51	38	25	17	14	1.875		35	2.61	
GB002	JD1	6	0	36	16	20	42	34	28	22	11					2.60	
MAL02	JD1						40	30	20	12	4	16	2.003		51	2.92	
U0002	JD1	6	2	38	17	22	83	61	51	34	24	13			19	2.66	5.6
TA2E0	JD						100	100	74	18						2.92	
U0009	JD1	10	10	39	19	20	91	87	77	61	37	17			20	2.60	7.1
NIA02	ABJD	4	0	16	11	5	80	73	57	26	10	12	2.131		7	2.66	
NIA04	ABJD	4	0	24	16	8	70	48	38	20	10	10	2.212			2.79	
U0011	KA2	11	27				100	99	98	91	60	29			7	2.76	5.1
1V004	KB2	7	2	61	35	26	60	39	34	30	13	14	1.987		22	2.97	
MAL03	BBKB	11	3	55	31	24	61	51	43	38	22	15	1.923		9	2.93	
MAL05	BCKB	7	0	45	32	13	46	31	26	24	7	14	1.923		39	2.79	
CAM02	KC2	11	19	65	37	27	98	97	88	66	12					2.63	
CAM03	KC2			0	0	0	67	40	20	10	4					3.29	
CAM05	KC2	11	4	67	42	25										3.10	
ET002	KC2	12	12	73	24	49	66	55	49	45	27	23	1.603		3	2.72	
ET003	KC2	11	19	68	33	35	84	63	62	58	43	28	1.538		12	2.85	
ET004	KC2	11	50	76	35	46	100	100	98	95	63	31	1.426		5	2.63	
TP04B	KC	7	02	58	27	31			45	030	746	15	1.843	050	045	2.86	
TP07A	KC2	6	0	35	17	18			45	30	606	8	2.244	090	055	2.94	
TP07C	KC	12	12	52	23	29			70	060	686	14	1.923	065	045	3.00	
TP08A	KC2	10	06	30	13	17			75	045	702	11	2.083	100	040	2.86	
TP08B	KC	12	20	47	18	29			95	073	969	17	1.795	060	030	2.86	
TP09A	KC2	7	01	42	21	21			40	030	578	11	2.180	035	025	2.93	
TP09B	KC	12	01	46	25	21			50	038	638	11	2.164	110	050	2.85	
TP09C	KC	12	12	57	25	32			65	050	663	19	1.747	030	015	2.83	
TP10B	KC2	6	10	31	17	14			35	30	535	6	2.324	110	80	2.95	
TP10C	KC	12	14	57	29	28			63	057	691	14	1.971	050	030	2.92	
TP11C	KC2	11	18	59	32	27			75	066	737	17	1.811	070	050	2.88	
TP15B	KC2	12	04	49	26	23			40	038	544	10	2.404	060	060	3.04	
TP18A	KC2	10	3	33	17	16			58	41	642	8	2.083	90	40	2.91	
TP19C	KC2	12	05	45	23	22			52	043	600	11	2.003	055	055	2.91	
TP36A	KC2	06	00	32	19	13			31	020	553	08	2.324	130	075	2.98	
TP36C	KC	12	06	60	29	32			45	037	599	14	1.923	050	035		
TP36D	KC	11	02	55	36	19			43	036	605	12	2.019	040	030	2.83	
TP38A	KC2	07	04	52	24	29			47	035	614	10	2.003	074	074	2.90	
TP38B	KC	10	02	39	22	16			71	040	808	13	1.923	045	040	2.83	
TP40A	KC2	06	00	35	17	19			36	020	634	07	2.180	105			
TP40B	KC	06	00	30	19	12			26	018	520	07	2.324	120	100		
TP41A	KC2	10	04	33	17	16			87	047	723	12	1.955	040	010		
TP42A	KC2	12	10	43	25	19			65	060	684	10	2.164				
TP42B	KC	07	01	58	34	21			24	022	486	13	2.083	075	075		
TP32A	LC	12	03	46	26	20			48	039	601	12	2.099	115	065	3.15	
TP32B	LC	06	00	36	20	16			33	024	514	08	2.372			3.15	
TP33B	LC3	7	00	49	28	21			26	018	499	10	2.212	110	110	3.05	
TP34C	LC3	11	03	55	30	25			44	033	589	11	2.035	070	030		
TP34D	LC	11	22	66	35	31			79	067	737	14	1.763	060	010		
TIV1A	LM	04	00	21	17	04			30	013	505	05	2.308	135			
TIV1B	LM	06	00	32	19	13			32	029	513	08	2.244	075	075		
TIV2A	LM	06	01	28	26	26			38	019	543	07	2.244	100	065		
TIV2B	LM	06	00	38	20	17			29	019	490	08	2.324	075	075		
TIV3A	LM	8	00	25	17	08			36	018	469	07	2.308	100	100		
TIV3B	LM	10	06	37	20	17			52	042	619	08	2.147	120	060		
TIV3C	LM	10	00	39	19	20			72	065	703	11	2.019	065	025		
TIV3D	LM	12	06	48	28	20			55	047	629	12	2.051	070	052		
TIV4A	LM	04	00	25	15	09			30	013	524	08	2.292	105			
TIV4B	LM	07	00	44	24	20			27	019	516	09	2.260	075	075		
TIV4C	LM	10	00	29	18	11			36	014	536	08	2.292	075	045		
TIV6B	LM	07	01	41	22	18			34	025	557	08	2.212	105	105		
TIV6C	LM	12	02	42	19	22			50	036	614	10	2.099	070	055		
TP14B	LX3	12	11	57	28	29			58	050	649	12	1.971	055	015	2.92	
TP16B	LX3	8	0	24	15	9			78	36	704	8	2.083	40	25	2.93	
TP17A	LX3	6	0	30	17	13			40	24	568	7	2.244	90	90	2.98	
TP17B	LX	08	02	27	17	10			83	049	732	10	2.083	040	030		
TP35B	LX3	04	00	30	20	10			34	025	559	08	2.292	085	075	2.95	

TABLE A4.5 (Continuation)

LAB. NO.	SOIL GRP	AASHO	GI	LL	PL	PI	4	10	40	200	2μ	OMC	(1) MDD	(2) CBR	(3) CBR	SG	pH
GI11B	LX3	7	1	43	23	20	59	51	45	32	19	17	2.051			2.68	
GI12A	LX3	6	0	32	20	12	40	35	30	24	11	10	2.083		17	2.70	
GI12B	LX3	12	7	42	18	24	82	68	66	57	18	18	1.763		15	2.66	
GI13A	LX3	6	0	35	20	15	56	39	34	28	19	14	1.955			2.68	
GI13B	LX3	12	10	49	26	23	82	69	61	54	26	21	1.683		59	2.71	
GI13C	LX3	11	8	61	35	26	87	68	55	50	33	19	1.763		40	2.74	
GI13D	LX3	11	4	50	37	13	82	65	58	47	20	16	1.955			2.82	
GI14A	LX3	10	6	38	18	20	95	81	67	47	26	14	1.923		15	2.63	4.8
GI14B	LX3	6	0	37	21	16	62	45	36	25	15	14	1.907		025	2.67	4.4
GI14C	LX3	8	7	32	25	7	100	98	72	37	6	11	2.019		13	2.66	4.55
TP30A	JC	06	00	31	19	12			27	019		07	2.292	120	085	3.15	
TP30B	JC	08	04	20	16	04			63	049		08	2.099	180	065	2.97	
TP30C	JC	08	05	22	18	04			98	082		10	1.939	070	055	3.00	
TP30D	JC	10	01	29	17	12			45	040		09	2.244	080	070		
TP05A	BCJD	06	00	26	15	11			45	025		12	2.083	150	150	2.86	
TP05B	BCJD	8	1	25	15	10			95	40		9	2.067	100	90	2.86	
TP06A	BCJD	4	0	26	16	10			46	22		8	2.196	160	145	3.06	
TP06B	BCJD	4	0	20	20	0			95	12		8	2.003	063	063	2.63	
TP20A	BCJD	6	2	31	16	15			31	23		8	2.324	110	80	2.97	
TP21A	BCJD	10	3	37	20	17			49	40		9	2.147	130	100	2.99	
TP21B	BCJD	10	1	37	22	15			41	36		9	2.115	180	64	2.94	
TP21C	BCJD	12	01	51	25	26			47	043		11	2.019	080	022	2.95	
TP21D	BCJD	06	01	38	22	16			33	028		11	2.083	045	040		
TP22A	BCJD	2	0	23	18	4			17	11		8	2.292	140	110	3.00	
TP22D	BCJD	6	0	30	18	12			36	29		6	2.212	120	90		
TP23A	BCJD	4	0	25	17	8			22	12		6	2.292	230	170	2.95	
TP23B	BCJD	4	0	20	15	5			55	26		7	2.180	100	80	2.86	
TP24A	BCJD	06	01	30	18	12			42	027		06	2.196	080	058	2.86	
TP25A	BCJD	04	00	21	18	04			62	033		07	2.164	130	130	2.93	
TP26B	BCJD	06	00	25	13	12			48	010		09	2.164	115	115	2.70	
TP27A	BCJD	06	01	33	18	15			32	025		08	2.212	100	080	2.94	
TP27B	BCJD	08	02	17	13	04			87	060		08	2.083	160	070	2.92	
TP28A	BCJD	02	00	24	19	05			26	018		08	2.308	220	160	3.15	
TP28B	BCJD	04	00	24	21	03			41	028		08	2.180	210	210	2.95	
TP29A	BCJD	02	30	23	17	06			23	020		09	2.276	125	125	3.15	
TP29B	BCJD	2	0	20	16	4			26	19		9	2.292	180	180		
TP31A	BCJD	10	03	32	19	13			53	047		09	2.324	100	100	3.17	
TP31B	BCJD	08	01	19	17	02			59	040		10	2.083	100	100	2.88	
TP01A	KC2	12	16	54	24	30			70	060		15	1.875	075	055	2.74	
TP02A	KC2	7	0	50	22	28			38	25		9	2.131	75	75	2.79	
TP02B	KC	12	32	70	26	44			77	071		19	1.683	060	045	2.79	
TP03A	KC	6	0	32	15	17			24	14		5	2.164	200	100	2.78	
TP038	KC	11	03	53	34	19			50	037		17	1.179	065	045	2.86	
TP04A	KC2	6		33	15	18						7	2.196	90	80	2.86	
IV002	KC2	12	18	48	24	24	65	60	50	40	27	17	1.763		12	2.81	
U0007	KC2	7	0	46	21	25	56	31	24	20	8	14			16	2.90	5.7
DAH02	BCKC	6	0	25	13	12	83	68	30	24	10					2.66	
DAH2A	BCKC	6	0	28	16	12	82	68	39	26	12					2.92	
DAH03	L13	12	4	48	21	24	85	72	55	39	27				16	2.84	
GB001	L13	6	0	35	18	17	56	47	42	27							
LIB8A	L3						59	24	20	15	10					2.85	
LIB8B	L3	6	0	35	23	12	64	52	39	28	11					3.01	
LIB8C	L3	7	1	53	33	20	68	50	36	29	21					2.90	
MAL06	L13	10	3	35	21	14	67	55	51	40	20	16	1.859		60	2.84	3.85
SEN04	L13	10	7	32	19	13	100	100	93	64	41						
CAM01	LA3	11	3	50	30	19	100	95	61	38	32					2.81	
CAM04	LA3	11	39	81	45	34	100	[illegible]	98	88						3.05	
U0001	LA3	10	5	35	14	21	98	94	78	46	29	12			10	2.74	4.2
U0003	BCLA	7	3	46	19	27	79	56	48	33	22	14			11	2.65	4.9
GA001	LC3	7	1	48	26	22	27	24	23	23						2.70	
GA002	LC3	12	16	46	23	23	100	99	98	82						2.77	
LIB05	LC3	7	3	70	40	29	61	42	37	32	22						
LIB06	LC3	11	10	52	34	18	99	95	85	56	20						
SL001	LCNA	11	15	70	36	34	85	69	67	60	44					2.69	
CAR01	LN3	12	6	47	20	27	66	56	50	44						2.90	
TA4A1	LN	10	8	34	14	21	100	98	84	55					1	2.72	
TA4A2	LN	10	8	35	15	20	99	98	86	56						2.68	
TA4C0	LN	7	0	48	23	25	54	33	19	15		9			22	2.82	
TAX02	LN	4		28	20	8	75	61	28	12					43	2.96	
IV005	LM3	11	22	62	31	31	100	99	88	69	46	17	1.731		23	2.83	
SL002	ABLM	7	1	55	31	24	68	37	29	27	15					2.71	

TABLE A4.5 (Continuation)

LAB. NO.	SOIL GRP	AASHO	GI	LL	PL	PI	4	10	40	200	2μ	OMC	(1) MDD	(2) CBR	(3) CBR	SG	pH
BUR01	BCLS	10	9	31	16	16	84	78	74	70						3.07	
LIB01	LX3	7	0	48	25	23	32	18	17	14							
LIB02	LX3	10	11	36	19	17	98	81	73	48							
LIB03	LX3	7	3	60	30	30	66	46	36	27						2.20	
LIB04	LX3	7	2	56	29	27	57	41	36	27	17						
LIB07	LX3	7	0	59	32	27	53	23	19	14	6					2.90	
TA2B1	LX	12	7	59	24	35	98	95	70	39		11				2.77	
TA2B2	LX	7	5	68	27	41	99	90	48	29		10				2.82	
TA2B3	LX	12	7	44	15	29	100	99	72	42		13				2.76	
TA1C0	LX	6	2	39	18	21	100	98	65	29		10				3.01	
TAT2A	LX	6	1	32	20	13	100	97	58	24		10	2.019		34	2.84	
U0005	LX3	10	2	39	19	20	82	73	53	38	22	14			23	2.78	5.6
U0006	LX3	12	21	47	25	22	100	97	88	76	62	21			22	2.75	6.

APPENDIX DO CHAPTER FOUR
DISCRIMINANT ANALYSIS

The method of discriminant analysis was used to provide an unbiased estimate of differences between the major soil groups–ferrisol, ferrallitic and ferruginous–and their respective subgroups.

Basically, discriminant analysis assumes that various materials can be classified into major groups according to certain dominant characteristics, and then tested to see if the same classification is valid using secondary characteristics. In this study, soil groups were determined by a pedological classification and then tested according to physical properties.

MATHEMATICAL BACKGROUND

The method of discriminant analysis as developed by R. A. Fisher, defines the variable Z_1

$$Z = \lambda_1 X_1 + \lambda_2 X_2 + \ldots\ldots\ldots + \lambda_k X_k \quad (1)$$

where Z is an index used to differentiate between the two groups of materials based on the secondary parameters $X_1, X_2 \ldots\ldots X_k$, and the coefficients $\lambda_1, \lambda_2 \ldots\ldots, \lambda_k$. If there is any relationship between the primary and the secondary characteristics, then with the proper choice of k coefficients, one can reasonably expect the frequency distribution of the variable Z to have a bimodal characteristic.

The samples grouped about the group means $\bar{Z}_1$ and $\bar{Z}_2$, separated on the basis of the secondary parameters, are likewise correctly separated on the basis of the primary characteristics. That is, a sample of primary class 1 will be found to have a value of Z, based on secondary characteristics, near $\bar{Z}_1$, while a sample from primary class 2 will have a value of Z near $\bar{Z}_2$.

The object of the method is to choose the parameters $X_1, X_2, \ldots\ldots, X_k$ such that the discriminant nature of Z-distribution is at a maximum–that is, the distance between the group means $\bar{Z}_1$ and $\bar{Z}_2$ is as great as possible, while the variation within each group is as small as possible.

Since Z is a defined rather than a measured variable, calculation of the coefficients $\lambda_1, \lambda_2, \ldots\ldots\ldots, \lambda_k$ cannot be obtained through direct regression. To circumvent this problem, the ratio of the differences between groups, as given by the group means, $\bar{Z}_1$ and $\bar{Z}_2$, to the differences within the groups, as represented by the standard deviations, $T\bar{Z}_1$ and $T\bar{Z}_2$ is maximized.

Both the maximization of bimodal differences and solutions for the coefficients $\lambda_1, \ldots\ldots \lambda_k$ can be obtained by differentiating the function:

$$G = \frac{(\bar{Z}_1 - \bar{Z}_2)^2}{\sum_{i=1}^{2} \sum_{j=1}^{ni} (Z_{ij} - \bar{Z}_i)^2} \quad \ldots \quad (2)$$

with $\bar{Z}_i$ = mean of ith group
$\bar{Z}_{ij}$ = value of Z in the ith group evaluated with j^{th} set of parameters.

In terms of the individual parameters, Z_{ij} can be written

$$Z_{ij} = \lambda_1 X_{1ij} + \lambda_2 X_{2ij} + \ldots\ldots\ldots + \lambda_k X_{kij} \quad (3)$$

Equation (3) can be resolved into the form:

$$G = \frac{\sum_{p=1}^{k} \sum_{q=1}^{k} \lambda_p \lambda_q d_p d_q}{\sum_{p=1}^{k} \sum_{q=1}^{k} \lambda_p \lambda_q S_{pq}} = \frac{A}{B} \quad (4)$$

with $d_p = \bar{X}_{p1} - \bar{X}_{p2}$ the difference between the mean value of the pth parameter for each group.

$$S_{pq} = \sum_{i=1}^{2} \sum_{j=1}^{ni} (X_{pij} - \bar{X}_{pi})(X_{qij} - \bar{X}_{qi})$$

Maximization of G occurs by differentiation with respect to the coefficient λ_r and setting the equivalent expression equal to zero.

$$\frac{\delta G}{\delta \lambda_r} = \frac{\frac{B\delta A}{\delta \lambda_r} - \frac{A\delta B}{\delta \lambda_r}}{B^2} = 0$$

$$(r = 1, \ldots\ldots, k)$$

Simplifying $$\frac{\delta B}{\delta \lambda_r} = \frac{1}{G} \frac{\delta A}{\delta \lambda_r} \quad \ldots \quad (5)$$

For each coefficient the following relationships hold:

$$\frac{\delta B}{\delta \lambda_r} = 2(\lambda_1 S_{r1} + \lambda_2 S_{r2} + \ldots\ldots \lambda_k S_{rk}) \quad (6)$$

and $$\frac{A}{\lambda_r} = 2(\lambda_1 d_1 + \lambda_2 d_2 + \ldots\ldots + \lambda_k d_k)\, dr \quad (7)$$

Inserting (6) and (7) into (5), and simplifying yields

$$\lambda_1 S_{r1} + \lambda_2 S_{r2} + \ldots\ldots + \lambda_k S_{rk} = cdr \quad (8)$$

where C is $(\lambda_1 d_1 + \lambda_2 d_2 + \ldots\ldots + \lambda_k d_k)/G$ and independent of r. For convenience C is set equal to one.

Since (8) defines a linear equation for each coefficient, k sets of equations are evolved which can be written in matrix form.

$$\begin{bmatrix} S_{11} & S_{12} \cdots S_{1k} \\ \vdots & \vdots \quad \vdots \\ S_{k1} & S_{k2} \cdots S_{kk} \end{bmatrix} \begin{bmatrix} \lambda_1 \\ \vdots \\ \lambda_k \end{bmatrix} = \begin{bmatrix} d_1 \\ \vdots \\ d_k \end{bmatrix} \qquad (9)$$

The terms S_{ij} and d_i are numerical quantities and can be readily calculated. Once the matrix (S) and the vector (d) have been computed, standard reduction techniques such as a Gaussian reduction can be used to solve for the individual coefficients.

TEST OF SIGNIFICANCE

After computing the coefficients $\lambda_1, \lambda_2, \ldots, \lambda_k$ the sum of the squares "between" and "within" groups are determined.

For "between" groups

$$D_B = \lambda_1 d_1 + \lambda_2 d_2 + \ldots\ldots + \lambda_k d_k \qquad \ldots (10)$$

while "within" groups

$$D_W = \left(\frac{n_1 n_2}{n_1 + n_2}\right) D_B^2 \qquad \ldots (11)$$

where n_1, n_2 = the number of data sets in groups one and two respectively.

The number of degrees of freedom for "between" and "within" groups is computed.

$DF_B = k$, the number of parameters

$DF_W = n_1 + n_2 - k - 1$

Dividing each sum of squares by its number of degrees of freedom, yields its mean square. Subsequent division of the mean square between groups by the mean square within groups gives the F value.

Once the F ratio has been calculated, a standard set of statistical tables are consulted to test the significance of the results. Three outcomes are possible.

(1) The computed F value is less than the statistical value at both the one and five percent significance levels.

(2) The computed F value is greater than the statistical value at the one percent but less than the value at the five percent level.

(3) The computed F value is greater than the statistical values at both the one and five percent levels.

Interpretation of the above situations is as follows:

(1) Neither group can be distinguished from the other, based on the given information.

(2) While the groups can be distinguished at the one percent level, they can not be distinguished at the five percent level. In the absence of more data, the result is inconclusive.

(3) At the five percent level both groups can be distinguished and the hypothesis of difference is accepted.

Of course, as the number of samples is increased and the number of parameters extended, the results become more conclusive.

PROGRAMMING

Due to extensive calculations, discriminant analysis becomes inapplicable, in large scale studies, without access to an electronic computer. A program was written to process discriminant analysis data and its listing is included at the end of this section.

Written in FORTRAN II programming language, the program runs on an IBM 1620-II computer having a 60-K storage, card reader and typewriter output. Neither external storage nor an on-line printer are required.

Programming features include:

(1) Internal storage for two groups of data with a maximum of one hundred data sets, five parameters per data set.

(2) Internal cycling and select feature to compare two groups based on from one to five parameters which need not be sequential.

A simplified flow chart is shown below.

APPLICATION AND EXAMPLE

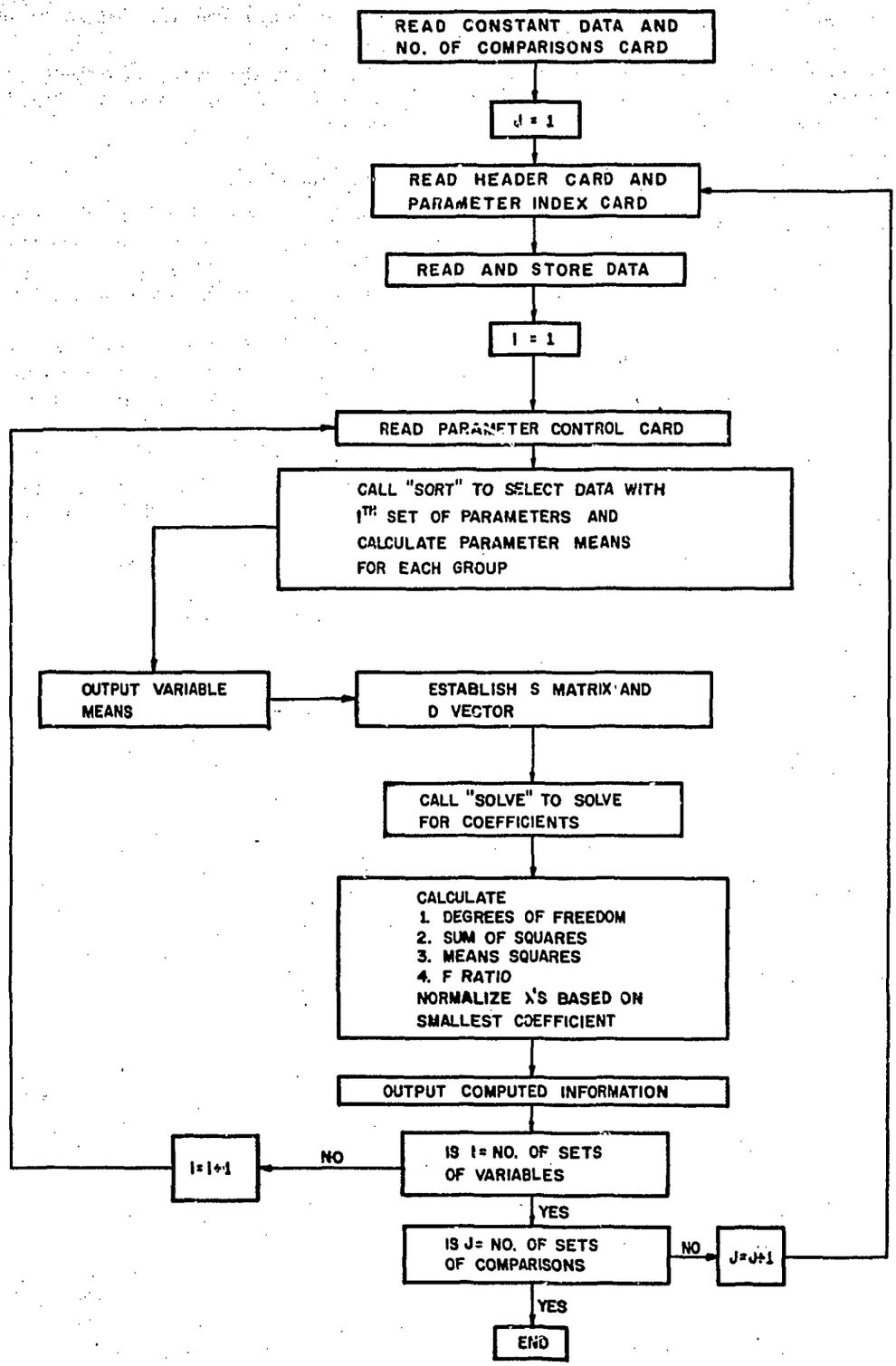
Sample input, sample output and an input sheet are included together with a program listing.

As an illustration of the use of the program two groups of soils, ferruginous and ferrallitic, each with liquid limit (LL), plastic index (PI) and maximum dry density (MDD) parameters are to be tested. Initially, both groups are to be tested using all three parameters (LL, PI, MDD) and subsequently tested using only liquid limit and plastic index (LL/PI) parameters. Although only twenty data sets for each group are presented here, substantially more were used in the final testing.

Input

The input cards are prepared in the following sequence.

1. Constant data card – This feature allows alphabetic representation of the characters LL, PI and MDD to appear on the output. The initials, LL, PI and MDD are entered in columns one, five and nine respectively.
2. Comparison card – only one comparison, namely that between ferruginous and ferrallitic soils, is being made; hence a one is entered.
3. Header Card – A written description of the problem


READ CONSTANT DATA AND NO. OF COMPARISONS CARD
J = 1
READ HEADER CARD AND PARAMETER INDEX CARD
READ AND STORE DATA
I = 1
READ PARAMETER CONTROL CARD
CALL "SORT" TO SELECT DATA WITH I^TH SET OF PARAMETERS AND CALCULATE PARAMETER MEANS FOR EACH GROUP
OUTPUT VARIABLE MEANS
ESTABLISH S MATRIX AND D VECTOR
CALL "SOLVE" TO SOLVE FOR COEFFICIENTS
CALCULATE
1. DEGREES OF FREEDOM
2. SUM OF SQUARES
3. MEANS SQUARES
4. F RATIO
NORMALIZE X'S BASED ON SMALLEST COEFFICIENT
OUTPUT COMPUTED INFORMATION
IS I = NO. OF SETS OF VARIABLES
NO
I = I+1
YES
IS J = NO. OF SETS OF COMPARISONS
NO
J = J+1
YES
END

Ferruginous–Ferrallitic Sample Test is entered starting in column one.

4. Parameter index card – since comparison is being made on two sets of parameters – first, all three parameters together, and secondly, only liquid limit and plastic index – a two is entered.
5. Data – both sets of data are entered with ferruginous group preceding the ferrallitic group. Each set of data is proceeded by a card indicating the number of points within each group, which in this case is twenty. followed by the data. The data is specified first by an identification code, of the user's choice, entered in columns one through six, followed by the parameters liquid limit, plastic index and maximum dry density starting in columns seven, fifteen and twenty-three Although the input sheet has been abridged to show only the first and last cards, twenty cards are entered for each group.
6. Parameter control cards – this card selects the number and the individual parameter to be used in testing. There *must* be as many parameter control cards as the number *entered* on the parameter index card. In this case there must be two parameter control cards. On the first card, a three, indicating all three parameters are to be used, must be entered in column three followed by the numbers: one (LL) in column six, two in column nine (PI) and three in column twelve (MDD). For the second z comparison, a second card is used with a two entered in column three, indicating that only two parameters are to be used, followed by a one in column six (LL) and a two in column nine (PI).

The test has been so defined that group one is ferruginous, group two is ferrallitic, parameter one is liquid limit, parameter two is plastic index and parameter three is maximum dry density.

Output

Where more than one set of parameters is chosen as a basis for comparison of two groups, the output becomes multiphased. In this case, two phases occur. Since both phases have the same output format, only one need be discussed.

Initially, the number of sets in each group used in testing is presented. Seven sets of ferruginous soil and seven sets of ferrallitic soil were used. These numbers differ from twenty, since some elements in each group did not have maximum dry density values. Only those sets possessing all three parameters are allowable.

The mean values of each parameter, liquid limit, plastic index and maximum dry density are listed with each group, for those groups which were defined earlier as $\bar{X}_{p1}$ and $\bar{X}_{p2}$.

Two sets of coefficients are given for the discriminant equation. The first set, under the heading LAMBDA define the discriminant equation:

$$Z = -4.83\times10^{-2}(LL) + 2.88\times10^{-2} + -3.07\times10^{-3}(MDD)$$

After computing the various sum of squares required, the coefficients are normalized by dividing the equation by the coefficient with the least absolute value. In this case, -3.07×10^{-3}. This operation yields the normalized equation:

$$Z^1 = -15.743(LL) + 9.376\ PI - (MDD)$$

Since Z is only an arbitrary measure of difference, the sense of the equation is unaltered. The quantities DELTA and PRODUCT refer to the differences between the mean values of each parameter ($\bar{X}_{p1} - \bar{X}_{p2}$) and the product of this difference and the "unnormalized" lambda ($\lambda_i d_i$).

Lastly, the sum of squares (SS), degrees of freedom (DF) and mean square (MS) are given for "between" and "within" groups. Following these is the F value.

In both cases the computed F values of 12.7 and 29.5 are greater than the statistical values of 6.55 and 4.36 at the five percent level of significance. The two groups can then be distinguished, one from the other, according to F criteria.

BIBLIOGRAPHY

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FERRALLITIC SOILS

	Sample Code	LL	PI	MDD
1	LA	40	22	
2	LC	61.3	15.9	93.4
3	LC	57.8	18.4	93.4
4	LC	58.7	15.9	86.0
5	LC	38.9	10.8	
6	LA	42.0	26.0	
7	LC	63	33	
8	LC	60	27	
9	LC	30	8	
10	LC	76	35	
11	LC	58	18	
12	LC	56	21	
13	LC	64	24	83.5
14	LC	54	20	88
15	LC	49	26	120
16	LC	72	31	
17	LC	83	40	
18	LCNA	64	29	
19	LCNA	70	34	103
20	LCNA	63	29	

FERRUGINOUS SOILS

	Sample Code	LL	PI	MDD
1	JA	30	17.7	124.5
2	JA	42	22.8	108.6
3	JA	43.6	22	108.8
4	JA	18	8	125.7
5	JA	21	10	
6	JA	18	8	123
7	JA	26	8	131.8
8	JA	39	19	
9	JA	42	17	
10	JA	41	21	
11	JA	45	25	
12	JA	30	14	
13	JA	37	19	
14	JA	34	12	119.3
15	JA	39	9	
16	JAJD	26.9	13.6	
17	JAJD	26.8	15.1	
18	JAJD	23	10	
19	JAJD	28	16	
20	JAJD	38	21	

FERRUGINOUS–FERRALLITIC SAMPLE TEST

Variable	Group 1 = 7 Mean 1	Group 2 = 7 Mean 2
LL	30.228	59.257
PI	14.071	22.028
MDD	120.242	95.328

Variable	Lambda	Normalized	Delta	Product
LL	-4.8288E-02	-15.743	-29.028	1.4017E-00
PI	2.8760E-02	9.376	- 7.957	-2.2884E-01
MD	-3.0672E-03	- 1.000	24.914	-7.6417E-02

	SS	DF	MS	F
Between Groups	4.20786E-00	3	1.40262E-00	12.7
Within Groups	1.09647E-00	10	1.09647E-01	

Variable	Group 1 = 20 Mean 1	Group 2 = 20 Mean 2
LL	32.415	58.035
PI	15.410	24.200

Variable	Lambda	Normalized	Delta	Product
LL	-8.0400E-03	-1.533	-25.620	2.0598E-01
PI	5.2440E-03	1.000	- 8.790	-4.6095E-02

	SS	DF	MS	F
Between Groups	2.55653E-01	2	1.27826E-01	29.5
Within Groups	1.59891E-01	37	4.32139E-03	

```
*1008
C       DISCRIMINANT ANALYSIS PROGRAM
        DIMENSION  A%10,11¤
        DIMENSION  PROD%10¤
        DIMENSION  PSUM%2,25¤,  DELTA%5¤,  FLMDA%5¤
        DIMENSION  PBAR%2,5¤,  PARM%2,5,100¤,  DSET%2,5,100¤,  N%2¤
        DIMENSION  NSET%2¤,  NSORT%5¤
        DIMENSION  FLMDC%5¤
        DIMENSION  ITITLE%20¤
        DIMENSION  NAME%5¤
        COMMON  A,  PARM,  DSET,  PBAR,  NSORT,  NSET,  N
        READ 1111,NAME
        READ 100,NCOMBS
        DO 2000 JPM#1,NCOMBS
        READ 1050,ITITLE
        PRINT 1051,ITITLE
        READ 100,NCOMBV
        DO 1 J#1,2
        READ 100,NSET%J¤
        INDEX#NSET%J¤
        DO 1 K#1,INDEX
    1   READ 102,%DSET%J,M,K¤,M#1,5¤
        DO 2000 KLM#1,NCOMBV
        READ 100,NVAR,%NSORT%M¤,M#1,NVAR¤
        CALL SORT%NVAR¤
        PRINT 700,N%1¤,N%2¤
        PRINT 702
        DO 701 J#1,NVAR
        M#NSORT%J¤
  701   PRINT 703,NAME%M¤,PBAR%1,J¤,PBAR%2,J¤
        NVARSQ#NVAR*NVAR
        DO 3 JKL#1,2
        DO 3 J#1,NVAR
        INDEX#N%JKL¤
        DO 3 K#1,INDEX
    3   PARM%JKL,J,K¤#PARM%JKL,J,K¤—PBAR%JKL,J¤
        DO 4 J#1,2
        DO 4 K#1,NVARSQ
    4   PSUM%J,K¤#O
        J#1
        L#O
        DO 5 JKL#1,NVARSQ
        L#L&1
        DO 6 LLL#1,2
        INDEX#N%LLL¤
        DO 6 K#1,INDEX
    6   PSUM%LLL,JKL¤#PSUM%LL,JKL¤&PARM%LLL,J,K¤*PARM%LLL,L,K¤
        A%J,L¤#PSUM%1,JKL¤&PSUM%2,JKL¤
        IF%NVAR-L¤5,50,5
   50   L#O
        J#J&1
    5   CONTINUE
        DO 8 J#1,NVAR
        DELTA%J¤#PBAR%1,J¤—PBAR%2,J¤
    8   A%J,NVAR&1¤#DELTA%J¤
        CALL SOLVE%NVAR¤
        DO 220 J#1,NVAR
  220   FLMDA%J¤#A%J,NVAR&1¤
        FMIN#ABS%FLMDA%1¤¤
```

```
      DO 221 J#2,NVAR
      IF%FMIN—ABS%FLMDA%J¤¤¤221,221,222
  222 FMIN#ABS%FLMDA%J¤¤
  221 CONTINUE
      DELS#O
      DO 223 J#1,NVAR
      FLMDC%J¤#FLMDA%J¤/FMIN
      PROD%J¤#; ELTA%J¤*FLMDA%J¤
  223 DELS#DELS&PROD%J¤
      FONE#N%1¤
      FTWO#N%2¤
      SSWG#DELS
      SSBG#FONE*FTWO/%FONE&FTWO¤*DELS*DELS
      NDFBG#NVAR
      NDFWG#N%1¤&N%2¤—NVAR—1
      FDFBG#NDFBG
      FDFWG#NDFWG
      FMSWG#SSWG/FDFWG
      FMSBG#SSBG/FDFBG
      FTLST#FMSBG/FMSWG
      PRINT 600
      DO 601 J#1,NVAR
      M#NSORT%J¤
  601 PRINT 602,NAME%M¤,FLMDA%J¤,FLMDC%J¤,DELTA%J¤,PROD%J¤
      PRINT 603
      PRINT 604,SSBG,NDFBG,FMSBG,FTEST
      PRINT 605,SSWG,NDFWG,FMSWG
 2000 CONTINUE
  600 FORMAT%/,9X,8HVARIABLE,6X,6HLAMBDA,5X,10HNORMALIZED,5X,5HDELTA,17X,7HPRODUCT¤
  602 FORMAT%11X,A4,5X,E11.4,F11.3,F13.3,4X,E11.4¤
  603 FORMAT%/,32X,2HSS,9X,2HDF,9X,2HMS,12X,1HF¤
  604 FORMAT%9X,14HBETWEEN GROUPS,4X,E12.5,16,4X,E12.5,4X,F5.1¤
  605 FORMAT%9X,14HWITHIN GROUPS,4X,E12.5,16,4X,E12.5¤
  700 FORMAT%///,25X,9HGROUP 1 #, 13,6X,9HGROUP 2 #,13¤
  702 FORMAT%16X,8HVARIABLE,13X,6HMEAN 1,14X,6HMEAN 2¤
  703 FORMAT%18X,A4,F21.3,F20.3¤
  100 FORMAT%613¤
  102 FORMAT%6X,5F8.0¤
 1050 FORMAT%20A4¤
 1051 FORMAT%///,20A4¤
 1111 FORMAT%5A4¤
      END

*1008

      SUBROUTE SORT%NVAR¤
      DIMENSION  A%10,11¤
      DIMENSION  PBAR%2,5¤, PARM%2,5,100¤, DSET%2,5,100¤, N%2¤
      DIMENSION  NSET%2¤, NSORT%5¤
      COMMON  A, PARM, DSET, PBAR, NSORT, NSET, N
      DO 1 K#1,2
      N%K¤#O
      DO 1 J#1,5
    1 PBAR%K,J¤#O
      DO 2 JPM#1,2
      INDEX#NSET%JPM¤
      INC#1
      DO 2 JKL#1,INDEX
      NOTE#O
```

```
      DO 3 K#1,NVAR
      NTVAR#NSORT%K¤
      IF%DSET%JPM,NTVAR,JKL¤¤4,3,4
    4 NOTE#NOTE&1
    3 CONTINUE
      IF%NOTE-NVAR¤2,5,2
    5 DO 6 J#1,NVAR
      NTVAR#NSORT%J¤
      PARM%JPM,J,INC¤#DSET%JPM,NTVAR,JKL¤
    6 PBAR%JPM,J¤#PBAR%JPM,J¤&PARM%JPM,J,INC¤
      INC#INC&1
      N%JPM¤#N%JPM¤&1
    2 CONTINUE
      DO 7 J#1,2
      DO 7 K#1,NVAR
      DIV#N%J¤
    7 PBAR%J,K¤#PBAR%J,K¤/DIV
      RETURN
      END

*1008
      SUBROUTE SOLVE%NROWS¤
      DIMENSION  A%10,11¤
      COMMON  A
C
C     SIMULTANEOUS EQUATION SOLUTION
C
      NEND#NROWS&1
      DO 1 J#2,NROWS
      NBEG#J-1
      DO 1 L#J,NROWS
      IF%A%L,J-1¤¤4,1,4
    4 CON#A%J-1,J-1¤/A%L,J-1¤
      DO 1 K#NBEG,NEND
      A%L,K¤#CON*A%L,K¤-A%J-1,K¤
    1 CONTINUE
      NEND#NROWS
      NSTOP #NROWS&1
      DO 2 J#2,NROWS
      NEND#NEND-1


      DO 2 L#1,NEND
      IF%A%L,NEND&1¤¤5,2,5
    5 CON#A%NEND&1,NEND&1¤/A%L,NEND&1¤
      DO 2 K#L,NSTOP
      A%L,K¤#CON*A%L,K¤-A%NEND&1,K¤
    2 CONTINUE
      M#NROWS&1
      DO 3 J#1,NROWS
      A%J,M¤#A%J,M¤/A%J,J¤
    3 A%J,J¤#1
      RETURN
      END
```

DISCRIMINANT ANALYSIS
(F – 3)

CONSTANT DATA CARD

1	5	9	13	17
LL	PI	NOO		

COMPARISON CARD

1
1

HEADER CARD

1
FERRUGINOUS-F

PARAMETER INDEX CARD

1
2

CARDS IN DATA SET ONE (1-100)

1
20

DATA SET ONE

1 IDENTITY	7 PARAMETER 1	15 PARAMETER 2	23 PARAMETER 3	31 PARAMETER 4	39 PARAMETER 5
LA	30.	17.7	124.5		
JAJO	36.	21.			

CARDS IN DATA SET TWO (1-100)

1
20

DATA SET TWO

1 IDENTITY	7 PARAMETER 1	15 PARAMETER 2	23 PARAMETER 3	31 PARAMETER 4	39 PARAMETER 5
LA	40.	22.			
LCNA	63.	29.			

PARAMETER CONTROL CARDS (SAME AS NUMBER OF CARDS IN PARAMETER INDEX)

1	4	7	10	13	16
	3	1	2	3	
	2	1	2		

APPENDIX TO CHAPTER 6

TABLE A6.1

Rating, Deflection and Traffic

District Section	Rating	Class. of Pavement Cracking	Average Owp Def. (inch)	Test Pit Location		Traffic A18KSAL x 10^5
				Deflection	TAN ϕ	
Esp. Santo						
1	5	–	.031	.038	.00054	18.6
2	8	–	.022	.024	.00110	41.4
3	8	–	.018	.018	.00078	–
4	9	–	.010	.011	.00051	2.43
5	8		.026	.025	.00139	4.64
6	6	–	.019	.020	.00072	4.64
7	6	–	.055	.062	.00307	4.64
8	7	–	.020	.020	.00068	45.5
9	7	–	.017	.020	.00088	45.5
10	6	–	.020	.021	.00091	19.3
Pernambuco						
1	8	–	.008	.009	.00026	4.71
2	8	–	.014	.014	.00038	4.71
3	8	–	.009	.010	.00024	4.71
4	8	–	.015	.015	.00052	4.71
5	8	1	.019	.017	.00035	4.71
6	8	–	.009	.012	.00055	4.71
7	6	–	.017	.016	.00086	13.90
8	6	–	.013	–	–	13.90
9	8	–	.008	.010	.00055	23.5
10	6	1	.012	.013	.00051	23.5
11	7	–	.008	–	–	16.5
12	8	–	.016	.017	.00048	16.5
Maranhão						
1	7	–	.015	–	–	5.44
2	7	–	.045	.043	.00172	–
3	7	–	.010	–	–	5.44
4	5	4	.036	.035	.00084	5.44
5	8	1	.023	.029	.00120	3.04
6	7	1	.011	.012	.00090	3.04
7	7	–	.021	.021	.00085	5.26
8	6	1	.018	.020	.00064	5.26
9	5	4	.009	.010	.00040	6.41
10	6	1	.011	–	–	6.41
11	8	–	.005	.007	.00043	4.42
M. Gerais						
1	7	–	.018	.019	.00126	53.5
2	7	2	.020	.023	.00047	48.1
3	7	–	.020	.023	.00096	–
4	7	2	.023	.023	.00074	10.4
5	8	–	.016	.016	.00050	10.2
6	7	–	.021	.021	.00130	10.0
7	7	–	.027	.027	.00139	5.82
8	7	1	.017	.013	.00077	29.2
9	7	–	.031	.031	.00103	2.53
10	7	–	.031	.031	.00092	1.00
11	7	–	.047	.048	.00100	–
12	7	–	.037	.037	.00190	15
13	7	–	.020	.019	.00098	7.5
14	7	–	.012	.012	.00064	5.9
15	6	2	.010	.011	.00018	21

TABLE A6.1 (Continuation)

District Section	Rating	Class. of Pavement Cracking	Average Owp Def. (inch)	Test Pit Location		Traffic A18KSAL x 10^5
				Deflection	TAN ϕ	
Paraíba						
1	7	–	.008	.009	.00025	2.21
2	6	–	.004	.004	.00029	15.3
3	7	–	.009	.009	.00059	12.7
4	7	–	.008	.007	.00043	6.97
5	7	–	.010	.010	.00057	5.50
6	8	1	.013	.013	.00058	7.00
7	7	–	.014	.014	.00025	3.14
8	6	1	.014	.015	.00049	2.99
9	7	–	.008	.008	.00055	1.95
10	6	–	.007	.007	.00026	1.95
Ceará						
1	8	–	.013	–	–	4.72
2	6	–	.016	.018	.00084	11.3
3	5	4	.066	.102	.00352	6.57
4	8	1	.014	–	–	7.05
5	7	–	.006	.007	.00027	7.05
6	6	–	.017	.013	.00077	3.2
7	7	–	.016	.016	.00059	3.2
8	6	–	.017	.017	.00076	3.2
9	7	4	.020	.020	.00310	6.66
10	6	1	.029	.025	.00311	6.66
11	5	4	.026	–	–	6.66
12	8	–	.020	.020	.00060	8.27
13	6	–	0.34	–	–	8.27
Rio Grande do Norte						
1	6	–	0.13	–	–	8.65
2	8	–	.012	.010	.00055	5.10
3	8	–	.015	.014	.00057	5.10
4	7	3	.045	.042	.00179	5.10
5	8	–	.029	.029	.00119	5.10
6	7	–	.011	.018	.00089	5.10
7	8	–	0.20	.019	.00081	1.20
8	6	–	.018	–	–	1.20
9	5	–	.023	.017	.00082	1.0
10	8	–	.006	.004	.00014	1.95
11	8	–	.021	.021	.00065	6.01
12	7	–	.020	–	–	6.01
13	7	–	.010	.008	0.0025	8.65
Pará						
1	8	–	.006	–	–	0.616
2	7	–	.018	–	–	0.616
3	8	–	.009	–	–	5.31
4	6	–	.008	.004	.00028	7.29
5	5	–	.008	.008	.00026	–
6	7	–	0.11	.010	.00028	15.6
7	7	–	.009	.009	.00060	–
8	6	–	.010	.010	.00057	–
9	6	–	.013	–	–	16.5
10	7	–	.009	.008	.00057	16.5
Brasília						
2	6	–	.017	.016	.00143	3.86
4	5	–	.025	.028	.00286	12.4
6	6	–	.024	.023	.00192	12.4
7	7	2	.027	.028	.00345	10.4
10	7	–	.019	.021	.00221	3.15

TABLE A6.1 (Continuation)

District Section	Rating	Class. of Pavement Cracking	Average Owp Def. (inch)	Test Pit Location		Traffic A18KSAL x 10^5
				Deflection	TAN ϕ	
Brasília						
12	6	–	.020	.021	.00217	3.15
13	6	–	.020	.018	.00170	32.8
15	6	–	.019	.019	.00147	32.8
16	8	1	.017	.017	.00055	21.9
20	7	–	.014	.015	.00157	0.444
São Paulo						
1	8	2	.011	.011	.00045	160
2	7	–	.026	.027	.00099	– 1.4
4	3	4	.029	.027	.00113	39.6
6	6	–	.022	.021	.00062	50
7	7	4	.021	.023	.00062	50
9	3	4	.041	.042	.00204	39
10	4	4	.022	.026	.00086	39
11	6	4	.024	.022	.00067	39
12	3	–	.029	.028	.00085	39
13	3	4	.037	.039	.00132	39
Goiás						
1	7	–	.020	.022	.00080	35.2
2	6	3	.018	.019	.00161	20.5
3	6	4	.050	.053	.00401	20.5
4	5	3	.024	.022	.00183	42
5	7	–	.015	.018	.00147	42
6	7	1	.034	.040	.00254	29.12
7	4	3	.050	.053	.00354	29.1
8	7	–	.019	.022	.00175	30
9	4	1	.022	.028	.00123	30
10	6	–	.022	.025	.00157	9.10
11	8	–	.018	.020	.00083	9.10
12	8	–	.034	.038	.00252	7.91
13	7	–	.026	.027	.00106	15.4
Piauí						
1	8	1	.018	.017	.00102	0.446
2	9	–	.005	.003	.00021	0.141
3	7	–	.017	.016	.00065	0.317
4	6	–	.009	.010	.00039	2.71
5	8	–	–	.012	–	0.397
7	7	–	.00030	.015	–	0.984
8	8	–	.00061	.015	.00084	1.07
9	8	–	.00054	.010	.00057	1.45
10	7	1	.00076	.015	.00051	3.32
11	6	11	.00041	.018	.00065	3.79
Paraná						
1	6	–	.034	.040	.00077	–
4	8	–	.014	.014	.00042	20.1
5	7	–	.010	.011	.00036	25.8
7	8	–	.039	.038	.00112	5.34
8	8	–	.021	.021	.00085	5.34
9	7	4	.018	.018	.00074	4.67
10	8	–	.017	.018	.00067	12.2
11	7	–	.012	.011	.00029	12.2
12	6	1	.017	.016	.00057	12.2
13	7	1	.017	.017	.00058	12.2
14	7	1	.022	.020	.00057	12.2
15	8	–	.009	.011	.00045	12.2

TABLE A6.1 (Continuation)

District Section	Rating	Class. of Pavement Cracking	Average Owp Def. (inch)	Test Pit Location		Traffic A18KSAL x 10^5
				Deflection	TAN ϕ	
Rio Grande do Sul						
1	6	4	.032	.033	.00242	16.3
2	7	3	.022	.025	.00123	14.5
3	8	2	.041	.041	.00176	11.8
4	5	4	.041	.046	.00250	24.6
5	8	–	.025	.027	.00054	14.5
6	6	4	.039	.040	.00069	24.0
7	5	4	.030	.038	.00079	68.7
8	7	–	.019	.017	.00036	76.2
9	6	4	.021	.021	.00055	–
10	7	4	.022	.021	.00089	–
11	8	–	.019	.018	.00063	–
12	7	1	.023	.022	.00061	–
13	7	–	.021	.021	.00070	–
14	7	–	.016	.016	.00048	–
15	8	–	.020	.023	.00079	–
Mato Grosso						
1	7	–	.012	.011	.00059	–
3	6	–	.013	.014	.00049	–
5	4	–	.049	.054	.00229	–
6	3	–	.035	.038	.00132	–
8	7	2	.008	.009	.00044	–
10	7	4	.016	.019	.00060	–
13	8	–	.009	.009	.00058	–
14	7	1	.012	.012	.00038	–
15	8	–	.006	.005	.00044	–

TABLE A6.2
Physical and Engineering Properties of Individual Structural Layers of Test Pits

State	Section	Layer	LL	PI	AASHO Mod.			% Swell	Percent Passing Sieve								Port. Swell	Sand Equivalent	Petrifaction
					MDD	OMC	CBR		1"	3/4	1/2	3/8	4	10	40	200			
Espírito Santo																			
	1	1	14	0	2.15	6	+120	.01	–	–	–	–	100	96	55	17	–	–	.5
	–	2	41	9	2.08	9	58	.42	–	–	–	100	93	88	53	36	–	–	.6
	–	3	52	19	1.80	17	32	.06	–	–	–	100	99	96	75	61	–	–	.6
	2	1	18	2	2.05	9	120	.03	–	–	–	–	100	96	60	18	–	–	–
	–	2	54	14	1.93	13	80	.21	–	–	–	–	100	95	66	39	–	–	.7
	–	3	47	15	1.87	16	40	.01	–	–	–	100	99	95	79	57	–	–	.5
	3	1	21	4	1.98	10	70	.09	–	–	–	100	97	90	56	24	–	–	.5
	–	2	13	9	2.00	12	85	.15	–	–	100	98	94	89	59	20	–	–	.5
	–	3	55	24	1.88	14	60	.10	–	–	–	–	–	100	77	47	–	–	.6
	5	1	28	6	2.10	7	70	.20	–	100	99	97	85	64	34	19	–	–	.4
	–	2	23	3	2.10	8	110	.01	–	100	95	93	85	70	29	17	–	–	.5
	–	3	27	6	2.07	9	90	0	–	–	–	91	81	58	37	31	–	–	.5
	6	1	25	5	2.05	9	+120	.02	–	100	99	98	92	73	22	8	–	–	.5
	–	2	39	15	2.00	10	+120	0	–	–	–	100	93	76	24	9	–	–	.4
	–	3	20	4	2.02	9	+120	.02	–	100	96	92	90	81	35	9	–	–	–
	7	1	24	4	2.08	8	105	–	–	100	97	94	93	82	44	20	–	–	.5
	–	2	43	13	1.92	12	50	.17	–	–	–	100	98	93	58	38	–	–	.5
	–	3	53	20	1.87	13	30	.03	–	–	–	–	100	96	71	59	–	–	.5
	–	4	53	18	1.80	16	25	.02	–	–	–	–	100	97	73	59	–	–	.5
	8	1	32	9	2.10	9	70	.04	–	–	–	–	100	93	59	30	–	–	.4
	–	2	31	9	2.08	9	50	.03	–	–	–	100	99	92	57	28	–	–	.4
	9	1	21	3	2.10	8	100	.08	–	–	–	100	97	94	59	26	–	–	.4
	–	2	31	7	1.98	11	30	.03	–	–	–	100	98	95	60	37	–	–	.5
	–	3	39	10	1.90	11	40	.02	–	–	–	–	100	97	61	38	–	–	.2

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Espírito Santo																			
	10	1	24	6	2.05	7	+ 120	.03	–	–	–	100	98	86	34	15	–	–	.4
	–	2	49	13	1.98	12	50	.06	–	–	–	100	98	93	57	38	–	–	.4
	–	3	62	20	1.90	15	30	.61	–	–	–	–	100	96	66	51	–	–	.5
Pernambuco																			
	1	1	20	NP	2.20	5	105	.02	–	–	–	100	99	83	53	26	4	–	.5
	–	2	20	NP	2.25	5	180	.00	–	–	–	92	76	62	44	22	3	–	.4
	–	3	20	NP	2.22	6	80	.06	–	–	–	99	88	69	48	25	5	–	.5
	–	4	27	14	2.20	6	60	.02	–	–	–	96	90	83	63	33	6	–	.4
	2	1	20	NP	2.20	6	110	.00	–	–	–	100	94	88	56	26	4	–	.7
	–	2	20	NP	2.16	6	225	.02	–	–	–	–	100	93	63	27	3	–	.7
	–	3	20	NP	2.13	6	35	.00	–	–	–	94	84	75	55	29	4	–	.5
	–	4	20	NP	2.18	4	110	.1	–	–	–	90	79	70	47	24	2	–	.6
	3	1	20	NP	2.30	5	165	.1	–	–	–	96	85	74	43	18	4	–	.5
	–	2	20	NP	2.20	6	100	.08	–	–	–	93	81	69	44	25	5	–	.4
	–	3	25	12	2.37	9	–	.03	–	–	–	92	80	66	46	27	6	–	.6
	–	4	31	13	2.00	8	25	.1	–	–	–	92	79	70	48	33	–	–	.6
	–	5	20	NP	2.20	7	65	.03	–	–	–	94	86	76	51	24	3	–	.6
	4	1	20	NP	2.25	6	85	.02	–	–	–	90	73	60	33	15	6	–	.3
	–	2	20	NP	2.20	7	50	.03	–	–	–	92	77	66	44	27	8	–	.6
	4	3	20	NP	2.18	7	30	.1	–	–	–	90	77	65	42	24	65	–	.3
	–	4	20	NP	2.20	8	40	.3	–	–	–	90	81	68	53	27	10.7	–	.5
	5	1	20	NP	2.22	6	70	.1	–	–	–	87	77	70	46	18	1.7	–	.5
	–	2	20	NP	2.30	4	95	.3	–	–	–	94	83	77	53	20	11	–	.4
	–	3	20	NP	2.18	7	80	.1	–	–	–	86	76	64	44	19	2.3	–	.5
	–	4	30	11	2.10	8	10	1,0	–	–	–	–	–	91	64	32	15	–	.3
	6	1	20	NP	2,25	6	75	.00	–	–	–	93	85	76	56	28	0.7/1.9	–	A/.5
	–	2	20	NP	2.20	4	160	.7	–	–	–	93	86	76	31	22	1.7	–	.5
	–	3	27	8	2.20	6	165	.03	–	–	–	–	–	94	67	33	4.6	–	.5
	–	4	29	16	2.20	5	165	.03	–	–	–	97	90	75	57	39	1.0	–	.4
	7	1	20	NP	2.20	6	100	.00	–	–	–	91	64	42	25	16	6.9	–	.5
	–	2	20	NP	2.20	6	55	.00	–	–	–	80	58	52	40	27	3.0	–	1
	–	3	20	NP	2.10	7	30	.00	–	–	–	98	95	87	65	34	1.8	–	1
	–	4	20	NP	2.10	6	95	.00	–	–	–	97	90	81	63	27	3.3	–	1
	–	5	20	NP	2.10	7	45	.02	–	–	–	00	97	92	70	32	0.9	–	.9
	8	1	20	NP	2.25	4	150	.04	–	–	–	82	64	52	35	21	2.6	–	.9
	–	2	20	NP	2.20	5	75	.02	–	–	–	90	76	66	46	27	2.3	–	.7
	–	3	20	NP	2.23	6	40	.2	–	–	–	90	77	67	46	24	2.9	–	.8
	9	1	20	NP	2.15	6	55	.01	–	–	–	95	85	68	45	26	4.3	–	.5
	–	2	20	NP	2.18	6	60	.02	–	–	–	91	77	66	46	25	1.9	–	.8
	–	3	20	NP	2.18	6	100	.02	–	–	–	94	84	75	57	28	4.2	–	.4
	–	4	20	NP	2.15	7	110	.02	–	–	–	93	81	72	56	31	1.3	–	.5
	10	1	20	NP	2.25	5	+ 120	.04	–	–	–	89	75	65	48	30	2.2/3.0	–	.4/.5
	–	2	20	NP	2.17	6	160	.03	–	–	–	88	74	65	45	24	4.3/3.0	–	.5/.6
	–	3	22	NP	2.21	7	115	.02	–	–	–	98	93	86	59	29	6.8	–	.5
	–	4	20	NP	2.14	6	+ 120	.02	–	–	–	90	82	76	59	26	1.3	–	.5
	11	1	20	NP	2.25	5	75	.1	–	–	–	93	77	63	40	21	7.9	–	.5
	–	2	20	NP	2.20	6	12	.1	–	–	–	89	69	58	40	23	7.2	–	.5
	–	3	20	NP	2.17	6	115	.07	–	–	–	86	62	58	42	25	8.8	–	.4
	–	4	25	10	2.14	8	35	.06	–	–	–	–	98	91	68	47	3.6	–	.5
	12	1	20	NP	2.17	7	125	.00	–	–	–	–	98	87	54	27	2.4	–	.5
	–	2	20	NP	2.10	6	95	.00	–	–	–	99	97	85	56	28	4.3	–	.4
	–	3	26	11	2.02	9	36	.2	–	–	–	99	92	76	55	41	8	–	.8
	–	4	20	NP	2.16	6	125	.00	–	–	–	98	94	83	49	23	5.2	–	.7
Maranhão																			
	1	1	29	18	2.13	10	40	.03	100	–	–	86	63	53	50	38	10.1	7	–
	–	2	30	13	2.10	11	46	.08	100	–	–	88	67	56	54	42	11.5	4	–
	2	1	23	3	2.18	10	100	.03	100	–	–	86	64	45	35	26	10.8	17	–
	–	2	20	NP	2.18	8	182	.04	100	–	–	89	64	45	38	28	9.7	15	–
	–	3	29	11	1.95	11	23	2.32	–	–	–	–	100	99	99	85	17.6	2.0	–
	3	1	20	3	2.19	7	102	.1	100	–	–	74	50	39	36	28	6.4	11	–
	–	2	27	8	2.14	10	37	.03	100	–	–	94	75	55	50	46	4.6	10	–
	4	1	30	11	2.76	9	110	.14	100	–	–	88	62	43	38	34	12.4	13	–
	4	2	26	8	2.15	9	97	.02	100	–	–	84	60	42	38	34	13.9	1.3	–
	–	3	25	8	2.11	10	56	.17	100	–	–	87	66	52	47	43	12.7	11	–
	5	1	20	NP	2.14	7	176	.02	100	–	–	86	65	52	45	23	7.0	17	–
	–	2	23	6	2.18	9	74	.02	98	–	–	82	60	45	38	26	11.4	13	–
	–	3	58	32	1.99	13	6	4.97	100	–	–	93	72	58	52	48	28.8	6	–
	6	1	20	NP	2.96	8	170	.03	100	–	–	92	67	44	33	17	7.1	22	–
	–	2	20	NP	2.27	8	211	.02	100	–	–	81	51	32	27	17	8.9	21	–

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Maranhão																			
	–	3	20	NP	1.89	5	37	.01	100	–	–	98	94	92	88	28	2.0	33	–
	7	1	24	9	2.17	9	138	.00	94	–	–	74	58	45	40	25	5.2	13	–
	–	2	32	13	1.97	11	29	2.13	99	–	–	97	96	95	94	57	5.2	3	–
	8	1	23	8	2.20	8	102	.01	90	–	–	48	37	32	30	23	6.5	10	–
	–	2	22	7	2.13	10	18	.09	100	–	–	83	69	59	54	36	9.3	7	–
	–	3	27	8	2.02	11	34	.37	100	–	–	98	94	91	89	61	11	2	–
	9	1	21	5	2.26	9	74	.05	99	–	–	86	68	53	46	27	6.9	11	–
	–	2	19	20	2.12	9	58	.04	98	–	–	93	84	75	71	40	7.0	8	–
	–	3	20	NP	2.05	9	100	.07	100	–	–	99	98	97	96	54	11	3	–
	10	1	20	NP	2.06	8	148	.05	92	–	–	61	47	41	39	7	0	14	–
	–	2	20	NP	2.02	6	90	.01	99	–	–	94	79	72	64	9	0	33	–
	–	3	20	NP	1.81	7	42	.01	–	–	–	–	–	100	91	4	0	69	–
	11	1	22	7	2.05	9	70	.00	99	–	–	73	57	48	43	19	7.2	14	–
	–	2	17	2	2.20	8	140	.01	92	–	–	59	44	39	36	15	4.3	9	–
	–	3	20	NP	2.06	6	88	.01	–	–	–	100	99	98	94	18	1.0	17	–
Minas Gerais																			
	1	1	22	6	1.88	8	81	–	96	–	–	84	77	68	43	21	7.7	16	.4
	–	2	20	5	1.87	6	88	–	91	–	–	74	66	57	36	17	6.3	16	.1
	–	3	42	15	1.72	13	35	–	96	–	–	92	90	87	76	43	7.1	5	.2
	2	1	30	10	1.85	8	104	–	100	–	–	98	96	77	31	17	2.0	24	.3
	–	2	29	9	1.81	7	193	–	100	–	–	97	94	73	26	12	12.9	29	.4
	–	3	53	18	1.62	16	22	–	100	–	–	100	100	99	88	56	3	8	.7
	–	4	36	13	1.77	13	26	–	100	–	–	100	100	97	69	42	5.2	14	.4
	3	1	29	9	1.81	8	85	–	100	–	–	100	99	94	44	21	0	23	–
	–	2	20	NP	1.64	10	66	–	100	–	–	100	99	93	35	10	5	34	.5
	–	3	41	14	1.68	15	16	–	100	–	–	100	99	96	88	50	4.1	4	.2
	4	1	37	14	1.87	11	140	–	97	–	–	89	81	65	33	20	3.6	20	.4
	–	2	29	9	1.88	9	243	–	96	–	–	91	87	70	29	16	6.1	23	.5
	–	3	31	11	1.85	9	83	–	100	–	–	94	87	76	54	29	6.3	15	.5
	–	4	71	24	1.48	21	31	–	100	–	–	100	100	99	91	69	7.5	4	.4
	5	1	27	6	2.03	9	114	0	90,8	–	–	56	41,5	32	29	15	2	26	.4
	–	2	48	16	1.93	15	50	0	100	–	–	83	62	42	38	35	6	17	.6
	–	3	44	11	1.72	18	86	1	100	–	–	85	77	68	64	59	7.1	13	.4
	–	4	30	11	2.04	9	88	9	97	–	–	82	72	59	44	31	8.4	14	.7
	–	5	61	19	1.66	22	26	2.0	–	–	–	100	100	99	99	98	4.6	2	.5
	6	1	16	1	2.10	7	+120	0	86	–	–	69	58	46	41	17	.5	15	.2
	6	2	48	16	1.98	9	34	1	91	–	–	83	67	45	37	30	10.4	21	.5
	–	3	41	11	1.7	19	36	1	–	–	–	100	100	99	99	68	1.8	17	.8
	7	1	33	8	2.09	11	+120	1	92	–	–	63	50	39	33	25	5.3	13	.6
	–	2	33	9	2.10	11	84	0	–	–	–	87	68	52	41	29	.6	15	.3
	–	3	36	11	2.10	8	+120	1	98	–	–	82	66	48	36	23	4.1	13	.4
	–	4	50	13	1.57	25	20	0	–	–	–	100	100	99	97	82	2	12	.6
	8	1	20	5	2.25	7	+120	0	100	–	–	86	71	55	48	22	1.2	11	.5
	–	2	30	6	2.24	11	+120	0	98	–	–	81	62	46	37	23	5	20	.4
	–	3	21	3	2.08	10	94	0	–	–	–	–	100	99	97	36	.6	11	.6
	9	1	36	9	2.12	10	+120	1	91	–	–	69	59	46	29	20	6	15	.4
	–	2	29	6	2.23	9	61	7	100	–	–	99.1	98	91	55	33	.7	17	.3
	–	3	52	17	1.56	26	30	1	–	–	–	100	100	99	95	91	2.7	11	.7
	10	1	29	9	2.01	7	+120	1	98	–	–	93	89	74	39	21	2.8	20	.4
	–	2	27	9	2.08	8	+120	1	100	–	–	99	97	88	51	25	4.3	19	.3
	–	3	35	11	2.06	8	+120	0	97	–	–	87	82	71	41	23	1.5	16	.5
	–	4	52	18	1.79	17	28	–	100	–	–	99	99	98	82	67	15.5	5	.3
	11	1	33	7	1.84	13	62	0	96	–	–	82	74	65	61	41	1.9	17	.7
	–	2	20	NP	1.88	12	+120	0	94	–	–	84	78	71	67	30	.8	12	.3
	–	3	64	17	1.65	19	9	5	100	–	–	97	93	88	82	75	10.9	2	.3
	12	1	34	12	2.11	7	58	2	90	–	–	75	65	50	40	32	–	9	.3
	–	2	37	12	2.10	12	52	0	96	–	–	80	65	52	50	37	–	8	.8
	–	3	62	26	1.61	23	13	0	–	–	–	100	99	98	98	97	3.6	2	.7
	13	1	34	12	2.00	12	72	1	100	–	–	96	87	69	61	41	3.7	11	.6
	–	2	41	13	2.02	11	78	1	100	–	–	89	71	49	39	29	3.4	12	.4
	–	3	35	12	2.02	14	72	0	96	–	–	86	73	58	47	34	–	12	.6
	–	4	52	16	1.78	20	44	5	100	–	–	99	99	97	93	86	5.1	3	.7
	14	1	44	16	2.07	10	57	0	95	–	–	79	64	44	35	31	–	17	.4
	–	2	37	13	2.01	12	50	1	97	–	–	82	72	60	56	41	5.7	6	.5
	–	3	41	13	1.63	22	57	2	–	–	–	100	99	99	96	92	1.7	10	.6
	15	1	23	7	2.18	6	+120	1	88	–	–	72	66	57	50	27	3.3	8	.4
	–	2	20	NP	2.11	7	+120	0	–	–	–	100	99	96	90	25	6	7	–

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Rio Grande do Sul																			
	1	1	30	12	2.04	8	90	.05	100	–	–	100	89	68	36	21			
		2	32	13	2.01	9	101	.03	100	–	–	100	97	77	47	28			
		3	51	23	1.86	14	59	.39	100	–	–	100	98	83	59	45			
		4	42	23	1.96	13	43	.31	100	–	–	100	99	94	80	55			
	12	1	26	10	2.08	8	82	.06	98	–	–	91	87	72	35	20			
		2	62	29	1.82	18	37	.04	100	–	–	100	98	82	62	43			
		3	49	21	1.87	14	48	.09	100	–	–	97	95	76	56	48			
	3	1	33	14	1.97	10	86	.27	98	–	–	98	96	79	40	25			
		2	22	8	2.05	10	56	.12	100	–	–	100	100	97	91	35			
		3	36	20	2.04	11	60	.07	97	–	–	93	91	88	69	48			
	4	1	24	9	2.06	8	150	.06	97	–	–	89	85	70	32	16			
		2	46	22	2.00	11	38	.41	92	–	–	88	84	74	55	42			
	5	1	36	15	2.13	7	87	.29	100	–	–	96	90	75	37	22			
		2	36	15	2.13	7	87	.29	100	–	–	96	90	75	37	22			
		3	36	16	2.14	8	70	.51	100	–	–	96	90	75	45	28			
		4	26	12	2.19	7	27	0	89	–	–	72	62	54	32	19			
	6	1	47	22	2.07	9	15	1.79	100	–	–	87	83	76	50	33			
		2	48	17	1.99	10	7	5.19	100	–	–	100	99	94	57	35			
	7	1	52	20	1.59	25	28	.15	100	–	–	100	99	99	92	82			
	8	1	50	14	1.57	27	44	.49	100	–	–	100	100	100	97	94			
		2	61	26	1.55	27	42	.50	100	–	–	100	100	100	98	95			
	9	1	54	20	1.56	24	47	.48	100	–	–	97	97	96	91	87			
	10	1	64	28	1.56	28	48	.42	100	–	–	100	100	99	96	90			
		2	64	29	1.49	29	–	.36	100	–	–	100	100	100	98	92			
	11	1	63	23	1.54	29	30	.46	100	–	–	97	97	96	90	60			
	12	1	44	18	1.84	17	66	.24	86	–	–	69	61	55	49	39			
		2	74	33	1.55	27	15	2.66	98	–	–	96	95	93	90	81			
	13	1	47	16	1.74	20	36	.13	100	–	–	99	99	99	97	65			
		2	37	12	1.79	16	41	.07	63	–	–	55	51	46	38	25			
	14	1	32	14	1.92	14	37	.16	100	–	–	100	100	100	99	50			
		2	34	11	1.86	15	74	.09	76	–	–	53	44	35	30	18			
	15	1	39	19	1.83	17	28	.99	100	–	–	96	96	95	94	91			
Mato Grosso																			
	1	1	NP	NP	2.30	10	165	0	93	88	–	65	45	36	32	10		13	.7
		2	NP	NP	2.03	9	56	0	–	–	–	–	–	100	98	24		12	.7
		3	NP	NP	2.09	8	48	0	–	–	–	–	–	100	99	26		11	.8
		4	NP	NP	2.09	8	48	0	–	–	–	–	–	100	99	22		12	.5
	3	1	NP	NP	2.20	7	115	0	96	93	–	93	69	58	45	14		15	.3
		2	19	5	2.18	7	91	0	96	92	–	81	65	54	44	16		14	.7
		3	NP	NP	2.11	6	105	0	–	–	–	–	–	100	98	23		16	.8
		4	NP	NP	2.09	7	84	0	–	–	–	–	–	100	98	20		17	.9
	5	1	49	18	1.66	19	17	0	79	74	–	72	69	65	55	43		13	.2
		2	37	12	1.89	17	27	3	100	100	–	97	83	74	67	50		14	.4
		3	47	21	1.77	22	19	.09	–	–	–	–	–	100	96	74		14	.4
		4	40	16	–	–	–	0	–	–	–	–	–	100	96	73		12	.7
	6	1	NP	NP	2.01	13	65	.01	72	66	–	58	51	45	33	25		14	.3
		2	41	16	1.73	22	27	.15	–	–	–	–	–	100	97	79		4	.4
		3	49	20	1.76	21	20	.05	–	–	–	–	–	100	97	80		5	.4
		4	43	15	–	–	–	–	–	–	–	–	–	100	97	79		3	.5
	8	1	21	6	2.17	9	89	0	100	88	–	73	53	41	34	15		14	.5
		2	NP	NP	2.05	7	74	0	–	–	–	–	–	100	97	17		14	.5
		3	NP	NP	2.13	9	40	0	–	–	–	–	–	100	97	19		13	.6
		4	32	14	2.00	13	40	.02	–	–	–	–	–	100	99	57		9	.2
	10	1	20	9	2.22	7	60	0	98	93	–	80	65	57	51	21		11	.3
		2	NP	NP	2.10	8	88	0	–	–	–	–	–	100	99	24		30	.4
		3	31	13	1.89	17	22	1.8	–	–	–	–	–	100	99	50		3	.3
		4	44	20	1.84	16	15	2.5	–	–	–	–	–	100	99	70		4	.5
		5	39	19	–	–	–	–	–	–	–	–	–	100	98	69		5	.3
	13	1	NP	NP	2.19	9	145	0	71	64	–	53	47	44	37	11		13	.3
		2	NP	NP	2.06	8	58	.01	–	–	–	–	–	100	100	2		11	.5
		3	NP	NP	2.03	10	44	0	–	–	–	–	–	100	100	27		11	.4
		4	NP	NP	–	–	–	–	–	–	–	–	–	100	100	24		13	.8
	14	1	NP	NP	2.16	9	135	0	76	70	–	59	52	50	45	11		19	.4
		2	NP	NP	2.11	8	50	0	–	–	–	–	–	100	100	24		13	.8
		3	33	11	1.88	16	38	.25	–	–	–	–	–	100	96	63		9	.4
		4	36	14	–	–	–	–	–	–	–	95	86	83	80	55		10	.4
	15	1	NP	NP	2.11	8	104	0	88	78	–	86	52	47	42	18		11	.3
		2	NP	NP	2.10	8	72	.01	–	–	–	–	–	100	99	27		13	.5
		3	NP	NP	2.08	7	44	1.1	–	–	–	–	–	100	99	15		13	.4
		4	43	17	1.71	22	32	1.25	–	–	–	–	–	100	95	75		9	.7

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Pará																			
	4	1	15	4	2.05	10	45	0	95	90	–	78	73	71	67	18	–	–	.4
	–	2	20	8	2.17	9	+ 120	0	93	91	–	81	74	68	58	20	–	–	.4
	5	1	28	14	2.00	13	78	01	94	–	–	77	57	46	41	25	–	–	.3
	–	2	17	4	2.05	10	12	–	100	100	–	99	98	97	94	32	–	–	.4
	6	1	21	6	2.01	11	110	0	95	94	–	82	67	59	52	21	–	–	.4
	–	2	12	3	2.05	7	+ 120	0	100	100	–	100	100	100	91	23	–	–	.3
	7	1	44	17	1.90	17	45	0	95	87	–	74	60	51	46	37			
	8	1	37	16	1.82	19	30	0	97	96	–	87	78	72	65	52			
		2	17	4	2.05	9	25	–	100	100		100	99	98	92	33			
	9	1	33	17	1.77	13	5	.02	–	92	–	74	70	68	63	45			
	10	1	24	9	1.92	13	50	–	–	–	–	–	–	56	–	26			
Goiás																			
	1	1	24	8	2.20	9	180	0	91	–	–	69	51	41	32	18	3.0	–	.4
		2	28	9	2.15	8	60	0	98	–	–	87	71	60	47	28	6.4	–	.5
		3	34	13	1.82	16	42	0	100	–	–	100	100	99	90	59	5.6	–	.4
	2	1	25	7	2.20	10	+ 120	0	92	–	–	62	44	36	27	15	4.2	–	–
		2	28	10	2.20	11	120	0	94	–	–	61	40	31	24	15	7.6	–	.4
	3	1	26	8	2.12	9	+ 120	0	99	–	–	94	77	60	50	29	5.5	–	.3
		2	27	5	2.20	7	110	–	91	–	–	75	56	43	34	23	5.5	–	.4
		3	34	14	1.85	14	26	0	100	–	–	100	100	99	97	73	2	–	.8
		4	34	14	–	–	–	–	–	–	–	–	–	99	–	73	–	–	–
		5	34	14	–	–	–	–	–	–	–	–	–	99	–	73	–	–	–
	4	1	21	6	2.20	6	+ 120	0	78	–	–	59	48	41	32	14	2.4	16	.4
		2	33	13	2.12	8	86	0	88	–	–	70	51	35	25	15	6.5	17	.5
		3	36	15	1.95	13	50	0	100	–	–	99	96	92	82	51	7.6	10	.5
	5	1	29	9	2.15	9	+ 120	0	98	–	–	94	70	53	37	21	4.5	29	.4
		2	29	10	2.15	7	+ 120	0	8	–	–	68	53	46	33	20	4.7	15	.5
		3	34	12	1.90	14	38	0	100	–	–	100	99	99	88	52	3.4	20	–
		4	34	13	1.85	14	42	0	100	–	–	100	100	100	89	51	1.6	19	.9
	6	1	26	7	2.15	9	+ 120	0	86	–	–	75	67	59	43	23	3.4	15	.6
		2	34	14	1.98	15	40	0	100	–	–	100	100	99	95	58	3.4	12	.6
		3	34	14	–	–	–	–	–	–	–	–	–	99	–	58	1.7	–	.5
		4	34	14	–	–	–	–	–	–	–	–	–	99	–	58	–	–	–
	7	1	26	8	2.12	8	164	0	98	–	–	86	75	65	51	27	2.4	14	.5
		2	32	12	1.90	16	32	0	100	–	–	100	100	99	97	74	1.4	16	1
	8	1	27	10	2.18	7	+ 120	0	98	–	–	75	44	29	20	11	2.7	27	1
		2	31	12	2.25	8	+ 120	0	92	–	–	69	54	46	37	21	4.5	18	.7
		3	40	13	1.70	22	40	0	100	–	–	99	98	96	91	68	5.2	19	1
		4	40	13	1.70	21	–	–	–	–	–	–	–	96	–	68	–	–	–
	9	1	26	9	2.22	8	90	0	93	–	–	81	59	40	30	16	5.6	42	.7
		2	28	9	2.20	7	+ 120	0	92	–	–	77	68	58	40	20	2.2	22	.6
		3	25	11	2.15	6	90	0	83	–	–	47	39	32	25	14	5.1	17	.4
		4	39	13	1.72	23	24	0	100	–	–	100	100	99	94	71	4.7	21	1
	10	1	31	10	2.20	10	100	0	94	–	–	62	44	32	18	13	5.8	30	–
		2	30	9	2.22	11	60	0	96	–	–	72	54	42	29	19	4.4	21	–
		3	35	12	2.20	10	+ 120	0	90	–	–	55	37	28	21	15	3.7	21	1
	11	1	32	10	2.15	9	98	0	95	–	–	78	64	52	38	26	7.7	21	1
		2	34	10	2.15	10	90	0	89	–	–	55	37	29	22	16	6.7	20	1
		3	36	13	1.94	15	42	0	100	–	–	100	89	67	64	42	3.5	15	.6
		4	35	12	1.90	14	40	0	100	–	–	93	85	79	70	47	8.8	13	.4
	12	1	28	9	2.20	6	+ 120	0	97	–	–	67	57	49	33	16	4.7	23	.5
		2	31	11	2.25	5	+ 120	0	95	–	–	67	54	45	30	17	5.2	18	.5
		3	36	11	2.05	11	104	0	91	–	–	86	74	64	51	36	7.3	15	.6
		4	37	13	2.05	11	40	0	96	–	–	85	80	74	67	48	3.9	12	.6
	13	1	23	6	2.06	12	80	0	91	–	–	67	51	44	40	29	1.7	20	.5
		2	53	19	1.53	26	22	03	100	–	–	100	100	100	98	95	7.8	10	.9
	14	1	25	8	2.20	8	84	0	86	–	–	63	45	36	29	14	5.9	21	.3
		2	39	13	1.80	16	90	0	86	–	–	66	58	53	49	40	3.0	15	.5
		3	42	10	1.85	13	100	0	96	–	–	58	42	36	33	28	1.1	12	1
		4	48	17	1.50	26	20	0	100	–	–	100	100	99	99	94	5.1	22	.7
Piauí																			
	1	1	20	6	2.20	10	90	0	–	–	–	57	39	32	28	16	0.9	12	
		2	27	10	2.94	10	+ 120	. 18	97	–	–	82	71	65	59	38	5.9	8	
		3	20	NP	2.10	7	+ 120	. 01	–	–	–	–	100	99	94	23	0.0	11	
	2	1	19	3	2.20	8	+ 120	. 0	–	–	–	74	52	39	34	18	4.4	14	
		2	20	NP	2.21	7	+ 120	. 2	–	–	–	83	55	43	38	20	0.0	16	
		3	20	NP	2.08	7	+ 120	. 3	–	–	–	–	99	98	86	35	0.7	11	

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
	3	1	20	2	2.30	9	140	.0	98	–	–	75	40	26	23	11	0.3	15	
		2	23	7	2.29	8	+120	.02	–	–	–	78	49	34	29	17	7.7	14	
		3	22	7	2.20	9	70	.01	–	–	–	82	62	51	47	39	10.5	9	
		4	27	11	2.15	10	42	.08	–	–	–	87	66	56	52	43	8.6	7	
	4	1	20	NP	2.17	6	+120	.3	95	–	–	68	49	44	39	18	0.2	16	
		2	20	NP	2.37	9	+120	.0	98	–	–	77	45	30	26	14	0.6	16	
		3	20	NP	2.30	8	+120	.25	–	–	–	83	54	40	35	20	3.7	14	
		4	20	NP	2.20	8	232	.10	98	–	–	71	46	38	37	19	4.0	10	
	5	1	23	6	2.12	9	116	1.12	–	–	–	85	65	55	50	34	5.1	8	
		2	24	8	2.17	9	200	.3	–	–	–	92	65	49	42	29	9.6	12	
		3	25	7	2.05	10	200	1	–	–	–	86	66	56	51	38	7.6	10	
		4	20	4	2.17	9	160	.5	–	–	–	91	70	56	50	32	4.3	13	
	6	1	20	NP	2.15	8	200	.0	98	–	–	74	59	52	48	15	–	10	
		2	23	8	2.12	9	180	.0	95	–	–	69	49	39	31	19	–	9	
	7	1	20	NP	2.20	8	236	.3	–	–	–	90	73	61	55	21	8.8	18	
		2	22	6	2.20	9	212	.0	–	–	–	90	76	64	56	30	2.3	14	
		3	20	NP	2.15	8	108	.0	–	–	–	99	98	97	94	29	0.4	16	
	8	1	22	5	2.30	9	246	.1	–	–	–	80	56	43	33	16	9.4	17	
		2	26	8	2.17	11	148	.0	–	–	–	63	47	38	33	19	9.0	12	
		3	21	7	2.17	10	80	.0	–	–	–	94	90	87	81	39	4.4	12	
	9	1	27	10	2.25	9	232	.1	97	–	–	67	51	43	36	17	2.2	15	
		2	28	11	2.17	10	90	–	97	–	–	76	55	46	40	23	6.9	10	
		3	18	7	2.12	9	64	.1	–	–	–	96	95	94	88	42	3.6	5	
	10	1	25	10	2.07	11	92	1.2	96	–	–	71	57	51	48	33	0.5	10	
		2	16	3	2.25	9	128	.0	–	–	–	67	45	42	38	21	2.0	14	
		3	28	12	2.00	11	+120	5	98	–	–	85	72	62	55	34	8.7	10	
	11	1	18	7	2.22	8	118	.10	93	88	–	63	50	43	37	13	0.4	8	
		2	20	NP	2.27	7	156	.20	96	81	–	63	52	47	44	24	1.3	11	
		3	25	11	2.02	10	24	2.6	–	–	–	97	82	72	71	56	9.3	3	
São Paulo																			
	1	1	20	2	2.07	5	77	–	–	–	–	–	–	82	–	18	.0		.1
		2	20	1	2.10	7	+120	–	–	–	–	–	–	65	–	14	.3		.3
		3	35	14	1.90	12	32	–	–	–	–	–	–	99	–	44	2.4		.3
	2	1	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–		–
		2	–	–	–	–	–	–	–	–	–	–	–	–	–	..	–		–
		3	72	36	1.45	37	10	–	–	–	–	–	–	100	–	76	4.2		.5
	4	1	29	8	2.10	7	62	.01	–	90	–	84	79	61	31	19	7.6		.4
		2	22	8	2.10	8	65	.08	–	85	–	80	74	66	43	21	7.9		.4
		3	36	10	1.82	17	15	.39	–	99	–	94	88	80	60	57	14.1		.5
	5	1	25	10	2.10	6	75	0	–	95	–	87	77	58	23	12	10.5		.4
		2	29	9	2.15	7	93	–	–	86	–	76	67	57	40	23	7.8		.5
		3	37	12	1.83	15	34	.4	–	–	–	99	–	96	86	58	13.8		.4
	6	1	19	2	2.16	6	68	0	100	100	–	93	87	68	62	45	1.0		.4
		2	32	10	2.19	8	74	0	100	100	–	91/98	83/03	69/77	64/61	49/46	5.7		.4
		3	53	15	1.82	15	12	0	100	100	–	100	98	96	75	58	15.9		.5
	7	1	27	9	2.15	7	46	0	90	86	–	81	75	63	40	26	6.7		.3
		2	28	5	2.00	16	89	0	100	100	–	99	99	89	80	52	8.9		.6
		3	30	13	2.03	8	22	1.17	100	100	–	100	93	74	68	67	14.4		.4
	8	1	29	10	2.12	8	56	.4	–	86	–	79	73	63	31	17	8.2		.5
		2	27	7	2.05	8	60	0	–	87	–	76	68	60	35	20	4.7		.5
		3	34	12	1.82	16	9	0	–	–	–	99	97	94	84	39	13.5		.5
	9	1	40	9	2.05	11	45	–	100	96	–	93	88	74	32	14	9.2		.4
		2	29	13	2.12	10	46	.8	93	87	–	82	76	68	49	29	8.9		.4
		3	49	25	1.91	15	27	1	92	89	–	81	77	70	61	40	8.3		.6
	10	1	28	6	2.08	8	80	0.7	100	99	–	95	92	78	37	18	14.8		.5
		2	25	6	2.11	9	54	0	100	95	–	87	83	74	49	26	9.2		.5
		3	50	20	1.74	20	17	.36	100	100	–	98	97	95	89	63	14.1		.5
	11	1	30	5	2.10	9	82	0	100	100	–	97	93	80	39	19	8.4		.4
		2	20	2	2.15	9	42	0	90	84	–	75	69	64	42	19	5.9		.6
		3	46	16	1.82	16	14	.8	100	100	–	98	96	93	88	67	7.7		.5
	12	1	30	7	2.09	8	85	–	–	96	–	87	79	64	35	17	4.9		.5
		2	26	9	2.10	9	24	–	96	89	–	84	79	72	44	20	7.6		.5
		3	52	9	1.80	17	18	–	100	99	–	96	95	94	88	69	12.9		.4
	13	1	29	6	2.10	8	86	–	–	–	–	94	88	72	45	19	9.0		.4
		2	NP	NP	2.09	9	57	0	88	81	–	76	72	67	37	16	5.0		.5
		3	38	15	1.83	14	17	–	100	100	–	100	99	98	89	75	11.8		.5
	15	1	NP	NP	2.03	9	82	.2	100	94	–	54	37	24	11	4	5.6		–
		2	25	7	2.12	9	60	.9	–	–	–	84	73	56	31	14	4.0		–
		3	35	16	1.94	15	20	3	–	99	–	96	89	87	79	67	11.7		.5

TABLE A6.7 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Ceará																			
	1	1	22	7	2.17	9	43	.05	94	–	–	85	75	66	56	25	3.7	13	–
	–	2	–	–	2.07	12	65	.1	–	–	–	–	–	–	–	–	8.2	–	–
	2	1	–	–	2.07	7	59	.1	–	–	–	–	–	–	–	–	0	–	–
	–	2	–	–	2.01	9	75	.05	–	–	–	–	–	–	–	–	4.2	–	–
	–	3	–	–	1.90	13	35	.07	–	–	–	–	–	–	–	–	4.7	–	–
	3	1	20	NP	2.19	7	50	.08	94	–	–	70	49	30	16	3	1.2	30	–
	–	2	–	–	2.00	10	16	.3	–	–	–	–	–	–	–	–	5.1	–	–
	–	3	–	–	2.05	9	28	.15	–	–	–	–	–	–	–	–	5.5	–	–
	4	1	18	4	2.16	8	120	.04	98	–	–	76	59	44	35	18	1.7	16	–
	–	2	–	–	2.14	7	120	.12	–	–	–	–	–	–	–	–	6.6	–	–
	–	3	–	–	2.12	8	108	.00	–	–	–	–	–	–	–	–	6.3	–	–
	5	1	19	8	2.20	8	42	.01	98	–	–	78	65	56	36	13	0,1	23	–
	–	2	–	–	2.18	6	80	.07	–	–	–	–	–	–	–	–	3.2	–	–
	–	3	–	–	2.16	7	100	.07	–	–	–	–	–	–	–	–	3.0	–	–
	6	1	30	10	212	8	80	.01	–	–	–	86	73	54	36	23	8.4	20	–
	–	2	–	–	2.05	10	18	.03	–	–	–	–	–	–	–	–	8.8	–	–
	–	3	–	–	2.09	9	14	.12	–	–	–	–	–	–	–	–	12.6	–	–
	7	1	28	9	2.07	10	49	.05	–	–	–	87	76	64	52	30	4.9	14	–
	–	2	–	–	2.08	9	30	.1	–	–	–	–	–	–	–	–	7.6	–	–
	–	3	–	–	2.10	7	110	.04	–	–	–	–	–	–	–	–	7	–	–
	–	4	–	–	2.14	8	78	.02	–	–	–	–	–	–	–	–	3	–	–
	8	1	22	7	2.07	9	37	.04	–	–	–	87	81	73	57	37	5.8	19	–
	–	2	–	–	2.03	10	26	.03	–	–	–	–	–	–	–	–	9.8	–	–
	–	3	–	–	2.07	9	15	.26	–	–	–	–	–	–	–	–	10.3	–	–
	9	1	20	NP	2.25	7	108	.04	–	–	–	96	86	72	37	12	5.5	32	–
	–	2	–	–	2.12	8	30	.11	–	–	–	–	–	–	–	–	2,7	18	–
	–	3	–	–	2.02	10	33	.14	–	–	–	–	–	–	–	–	2.5	–	–
	–	4	–	–	2.10	8	54	.14	–	–	–	–	–	–	–	–	2.7	–	–
	–	5	–	–	2.12	8	21	.11	–	–	–	–	–	–	–	–	4.1	–	–
	10	1	26	7	2.14	7	78	.04	–	–	–	95	78	52	34	12	10.6	27	–
	–	2	–	–	2.15	6	104	.01	–	–	–	–	–	–	–	–	5.4	–	–
	–	3	–	–	2.14	7	36	.02	–	–	–	–	–	–	–	–	8.3	–	–
	–	4	–	–	2.14	7	47	.06	–	–	–	–	–	–	–	–	3	–	–
	12	1	–	–	2.15	6	89	.03	–	–	–	–	–	–	–	–	3.9	–	–
	–	2	–	–	2.15	7	67	.16	–	–	–	–	–	–	–	–	2.7	–	–
	–	3	–	–	2.14	7	26	.14	–	–	–	–	–	–	–	–	1.3	–	–
	13	1	–	–	2.07	14	22	.01	–	–	–	–	–	–	–	–	11.1	–	–
	–	2	–	–	2.10	9	114	.03	–	–	–	–	–	–	–	–	–	–	–
Rio Grande do Norte																			
	1	1	19	4	–	–	–	–	–	97	–	65	44	27	28	8	1.7	–	.7
	–	2	24	10	–	–	–	–	94	84	–	59	44	34	28	5	4.2	–	.8
	–	3	36	14	–	–	–	–	–	–	–	–	–	–	–	–	7.4	–	.4
	2	1	20	5	2.05	6	25	–	98	96	–	36	79	70	50	29	4.2	–	.3
	–	2	16	NP	1.9	6	49	–	98	94	–	81	75	69	42	25	3.0	–	.1
	–	3	20	2	1.92	6	36	–	–	–	–	98	97	91	54	30	12.6	–	.2
	3	1	18	NP	2.0	6	44	–	98	94	–	71	56	47	34	18	1.6	–	–
	–	2	18	NP	1.97	6	40	–	94	78	–	66	56	–	42	23	1.2	–	.3
	–	3	23	6	1.95	6	8	–	–	96	–	77	65	55	46	21	0.7	–	.3
	4	1	19	4	2.05	6	32	–	–	92	–	77	64	52	34	13	2.1	–	–
	–	2	32	10	1.95	6	68	–	–	98	–	88	76	56	37	29	11.2	–	.5
	–	3	26	9	1.82	6	8	–	–	99	–	97	95	91	77	37	10.1	–	.2
	5	1	19	4	2.05	5	43	–	–	95	–	81	67	52	34	13	2.6	–	.2
	–	2	28	9	2.00	6	23	–	–	98	–	85	73	57	36	22	95	–	.2
	–	3	28	8	1.80	6	3	–	–	–	–	100	99	95	68	28	12.1	–	.3
	7	1	20	NP	1.92	6	34	–	94	90	–	83	74	61	36	17	1.9	–	.4
	–	2	20	NP	1.79	6	60	–	–	–	–	100	100	95	60	17	1.5	–	–
	–	3	18	6	1.80	6	36	–	–	99	–	98	95	89	52	22	0.9	–	–
	10	1	20	4	2.00	5	50	–	–	97	–	83	64	43	27	15	3.5	–	.4
	–	2	20	6	1.92	6	17	–	–	97	–	88	81	74	47	19	1.3	–	.3
	–	3	20	NP	1.79	6	29	–	–	–	–	99	99	97	58	10	0	–	–
	11	1	18	–	2.03	6	–	–	–	93	–	66	50	36	21	10	2.8	–	.1
	–	2	20	NP	1.83	6	–	–	–	–	–	100	99	97	64	13	0	–	–
	–	3	23	7	–	–	–	–	–	–	–	100	99	98	79	27	4.7	–	.5
	–	4	22	7	1.80	6	34	–	–	–	–	100	99	98	79	27	0	–	–
	12	1	18	4	2.06	6	7	–	98	83	–	68	56	46	35	13	2.1	–	.6
	–	2	20	NP	1.80	6	26	–	–	–	–	100	100	96	50	11	0.9	–	–
	–	3	19	4	–	–	–	–	–	–	–	100	100	99	72	22	1.3	–	.6
	–	4	20	NP	1.82	7	34	–	–	–	–	–	100	99	76	22	0	–	–
	13	1	22	6	2.02	6	43	–	97	95	–	73	67	61	47	29	2.1	–	.6
	–	2	21	5	2.02	6	23	–	–	93	–	68	60	52	43	24	0.6	–	.6
	–	3	20	NP	1.77	6	30	–	–	–	–	100	100	99	69	9	6.8	–	.6

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
Distrito Federal (Brasília)																			
	2	1	36	12	1.72	14	32	.13	94	77	–	55	38	27	21	18		21	
		2	40	14	1.74	14	86	0	89	86	–	66	51	35	28	25		21	
		3	24	8	1.94	12	56	.08	100	100	–	100	100	98	92	62		12	
	4	1	41	15	1.80	11	100	0	86	78	–	57	41	29	24	22		18	
		2	25	8	1.83	11	175	0	95	87	–	77	69	60	55	39		15	
		3	24	6	1.73	9	62	0.6	100	99	–	93	88	81	75	54		9	
	6	1	44	14	1.70	15	95	0.1	86	82	–	70	61	51	43	42		15	
		2	41	12	1.73	15	115	0	85	76	–	68	58	49	44	41		19	
		3	40	13	1.74	14	133	0	77	67	–	53	44	32	28	23		24	
		4	26	10	1.80	9	33	0.1	100	99	–	90	83	74	60	53		7	
	7	1	26	7	1.92	10	70	0.1	94	91	–	78	62	44	40	30		17	
		2	28	9	1.90	10	88	0.1	100	100	–	84	61	43	40	31		17	
		3	33	11	1.76	12	125	0	100	96	–	79	59	44	38	32		23	
		4	47	16	1.34	26	39	0.2	100	100	–	100	100	99	97	94		22	
	10	1	43	14	1.74	16	116	0	100	96	–	74	60	46	41	39		22	
		2	44	14	1.76	16	45	0	100	93	–	77	62	47	42	39		20	
		3	44	11	1.39	25	18	0	100	100	–	100	100	99	98	93		25	
	12	1	47	17	1.72	14	150	0.1	94	91	–	80	68	57	52	49		18	
		2	48	18	1.63	16	140	0	93	80	–	71	61	50	45	42		20	
		3	46	13	1.33	28	33	0.2	100	100	–	100	100	98	96	92		14	
	13	1	39	15	1.84	11	165	0	80	72	–	59	50	36	27	24		24	
		2	39	12	1.85	11	198	0	95	88	–	72	58	41	32	29		27	
		3	41	13	1.75	13	96	0	88	76	–	53	39	31	26	24		25	
		4	45	13	1.52	19	36	0	100	100	–	89	85	81	77	73		17	
	15	1	46	16	1.74	14	120	0	98	90	–	66	42	31	24	21		28	
		2	41	13	1.82	13	67	0	88	77	–	52	37	28	23	19		22	
		3	38	10	1.75	12	160	0	93	91	–	71	49	34	24	20		30	
		4	46	13	1.45	20	30	0.1	100	100	–	100	100	95	88	67		18	
	16	1	39	11	1.84	12	95	0.1	100	98	–	74	51	39	29	23		30	
		2	44	16	1.70	16	18	0	96	91	–	70	43	34	30	26		22	
		3	49	18	1.55	16	50	0.3	100	100	–	96	89	80	70	64		20	
	20	1	39	12	1.72	17	60	0	93	83	–	69	51	40	31	26		23	
		2	37	14	1.72	14	100	0.1	89	86	–	70	54	44	35	28		20	
		3	38	13	1.73	13	80	0	94	85	–	70	53	44	37	30		20	
		4	52	10	1.52	17	12	0.5	100	100	–	91	87	84	82	79		22	
Paraíba																			
	1	1	21	4	2.14	7	152	0	100	100	–	100	99	92	48	15	0.3	20	.7
		2	20	5	2.09	9	80	0	100	100	–	98	96	91	54	22	3.3	18	.7
		3	17	3	2.11	8	92	0	100	100	–	97	95	87	57	8	4.7	20	1
		4	NL	NP	1.90	8	59	0	100	100	–	100	100	99	65	11	0	20	1
	2	1	24	6	2.17	7	47	.02	100	100	–	100	97	82	51	22	0	18	.7
		2	21	5	2.08	9	27	0	100	100	–	100	100	98	59	23	0.4	16	1
		3	NL	NP	1.83	6	43	0	100	100	–	100	100	100	67	6	0.5	65	1
	3	1	27	10	2.16	7	40	.08	77	54	–	47	35	26	17	13	1.7	23	.4
		2	21	7	2.16	10	21	.1	–	98	–	91	87	83	54	26	3.6	43	.8
		3	27	12	2.12	8	49	.04	100	100	–	100	99	86	47	26	5.1	28	.6
		4	30	9	1.96	11	30	.18	–	–	–	99	99	98	74	39	1.6	17	1
	4	1	24	7	2.07	9	52	.31	59	50	–	41	35	32	22	12	6.6	24	1
		2	23	8	2.04	9	18	.09	49	41	–	42	38	35	23	12	5.3	19	.6
		3	20	7	2.09	9	55	.05	66	57	–	45	38	33	19	11	8.4	19	.6
		4	NL	NP	2.00	6	79	.06	–	–	–	96	95	94	65	19	6.8	28	1
	5	1	18	2	2.19	7	96	.08	100	96	–	87	75	62	40	19	0.8	25	.5
		2	26	7	2.03	13	42	0	100	100	–	93	82	72	60	38	5.6	18	1
		3	NP	NP	2.06	7	74	.1	100	100	–	100	100	99	67	13	0.1	38	1
	6	1	N	NP	2.21	6	120	.1	68	59	–	39	30	27	17	6	1.6	34	.5
		2	NL	NP	2.14	6	127	.08	61	55	–	48	40	34	23	9	3.6	29	.6
		3	NL	NP	2.10	7	56	.0087	95	91	–	84	65	49	30	13	5.0	33	.5
		4	NL	NP	2.07	7	90	.017	90	–	–	83	76	67	50	18	2.4	26	.2
	7	1	20	11	2.16	8	86	.13	48	39	–	31	24	21	15	8	4.7	19	.4
		2	20	8	2.10	8	13	.15	83	77	–	66	57	50	35	18	5.6	20	.4
		3	23	0	2.02	9	43	.10	58	55	–	52	50	47	40	21	10.5	26	.5
	8	1	17	0	2.27	7	85	.18	63	62	–	54	46	36	31	18	1.8	14	.4
		2	24	6	2.10	10	23	.13	74	–	–	70	63	57	44	29	15/12.1	13	.3
		3	34	11	2.00	11	31	.22	56	–	–	49	45	40	31	21	14	13	.5
	9	1	NL	NP	2.10	7	82	.17	42	39	–	34	28	25	18	9	0.8	29	.6
		2	NL	NP	2.17	6	58	.14	92	85	–	81	70	58	37	18	5.5	31	.5
		3	20	–	2.02	9	88	.05	64	–	–	60	56	52	37	16	5.8	23	.5
		4	21	–	2.13	7	74	051	77	75	–	73	70	66	50	25	4.9	19	.5/.2
	10	1	18	4	2.13	5	86	.32	92	91	–	89	82	67	28	15	6.0	24	.4

TABLE A6.2 (Continuation)

State	Section	Layer	LL	PI	AASHO Mod. MDD	AASHO Mod. OMC	AASHO Mod. CBR	% Swell	Percent Passing Sieve 1"	3/4	1/2	3/8	4	10	40	200	Port. Swell	Sand Equivalent	Petrifaction
		2	18	5	2.23	5	66	.16	–	–	–	–	98	88	85	83	5.6	25	.5
		3	17	6	2.16	7	28	.04	100	100	–	100	100	94	60	34	1.7	22	.4
		4	17	3	2.16	6	44	.1	–	–	–	–	98	90	86	84	8.8	21	.2
Paraná																			
	1	1	20	5	2.20	6	200	.06	92	86	–	73	62	52	37	19		21	
		2	38	10	1.72	17	9	.3.45	100	100	–	100	99	98	95	72		3	
	4	1	54	27	2.18	14	168	.07	98	96	–	85	73	60	43	24		22	
		2	31	10	1.96	12	60	.03	100	100	–	100	100	100	96	38		43	
		3	57	28	1.81	17	12	2.90	100	100	–	100	100	100	96	76		5	
	5	1	50	19	1.68	24	41	.61	100	100	–	100	100	100	99	90		10	
	6	1	50	19	1.79	17	34	.95	100	100	–	100	100	99	96	58		4	
	7	1	57	13	1.59	27	95	.14	97	97	–	92	86	77	75	73		19	
		2	36	6	1.75	20	123	.15	100	100	–	100	98	91	89	85		26	
	8	1	54	12	1.58	25	137	.05	89	82	–	62	48	38	35	32	–	23	–
	–	2	50	11	1.66	22	117	.09	86	80	–	62	46	33	30	26	–	21	–
	9	1	72	20	1.54	29	39	.31	100	100	–	100	100	99	97	93	–	8	–
	–	2	65	23	1.60	26	21	1.86	100	100	–	100	99	98	95	90	–	6	–
	10	1	60	20	1.57	27	27	2.50	100	100	–	100	100	99	94	89	–	14	–
	11	1	64	17	1.58	28	46	.65	97	95	–	94	95	92	90	86	–	32	–
	12	1	52	15	161	27	57	.07	86	82	–	78	75	74	71	68	–	15	–
	13	1	46	12	1.73	20	64	.50	90	84	–	59	60	50	45	34	–	14	–
	–	2	62	22	1.63	25	21	2.00	100	100	–	100	100	98	95	80	–	22	–
	14	1	44	9	1.62	25	54	1.20	85	80	–	67	59	54	52	47	–	7	–
	–	2	51	16	1.66	25	28	.11	100	100	–	100	100	100	99	94	–	14	–
	15	1	55	22	1.65	25	25	.06	100	100	–	100	100	100	98	96	–	17	–

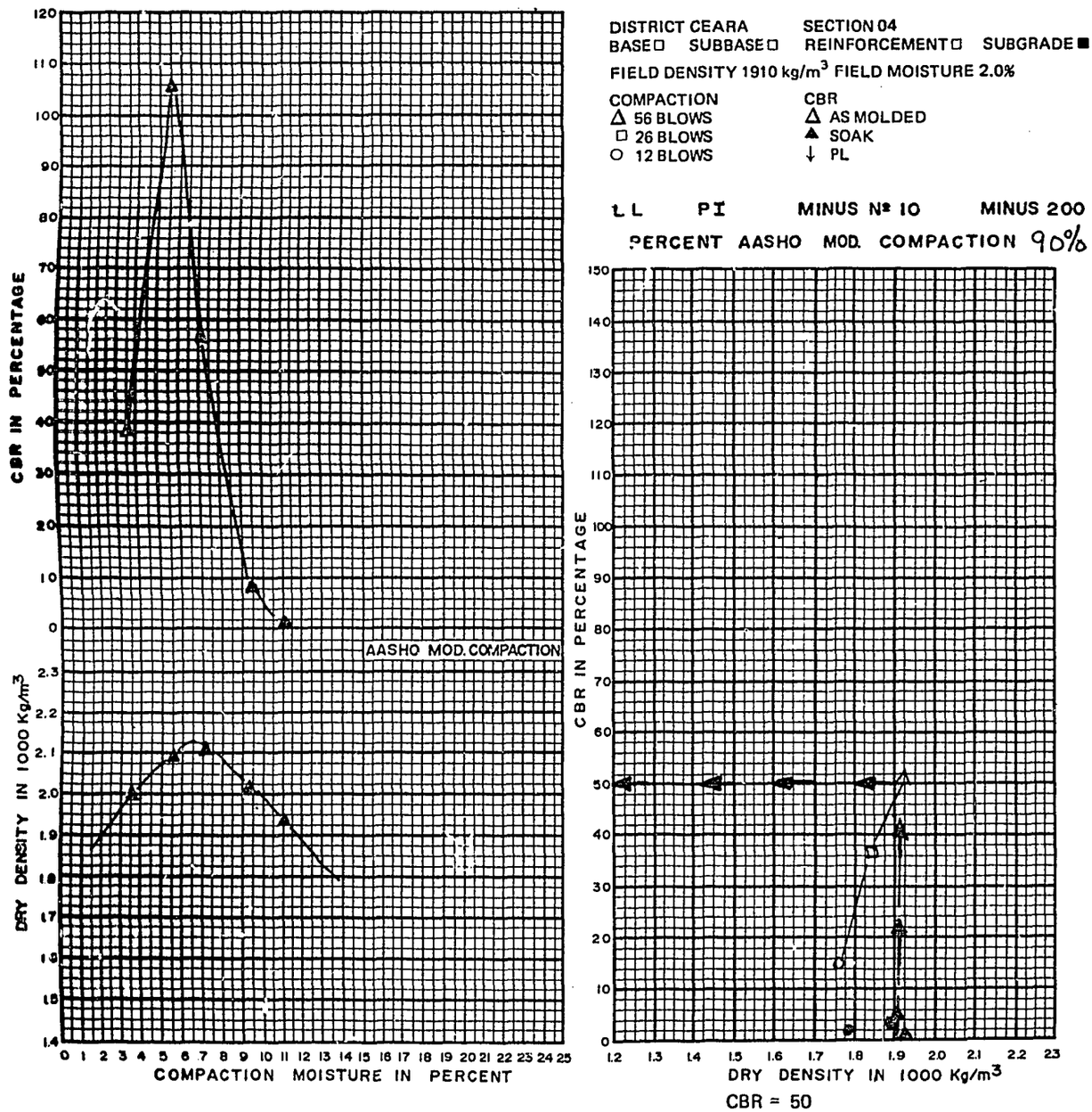
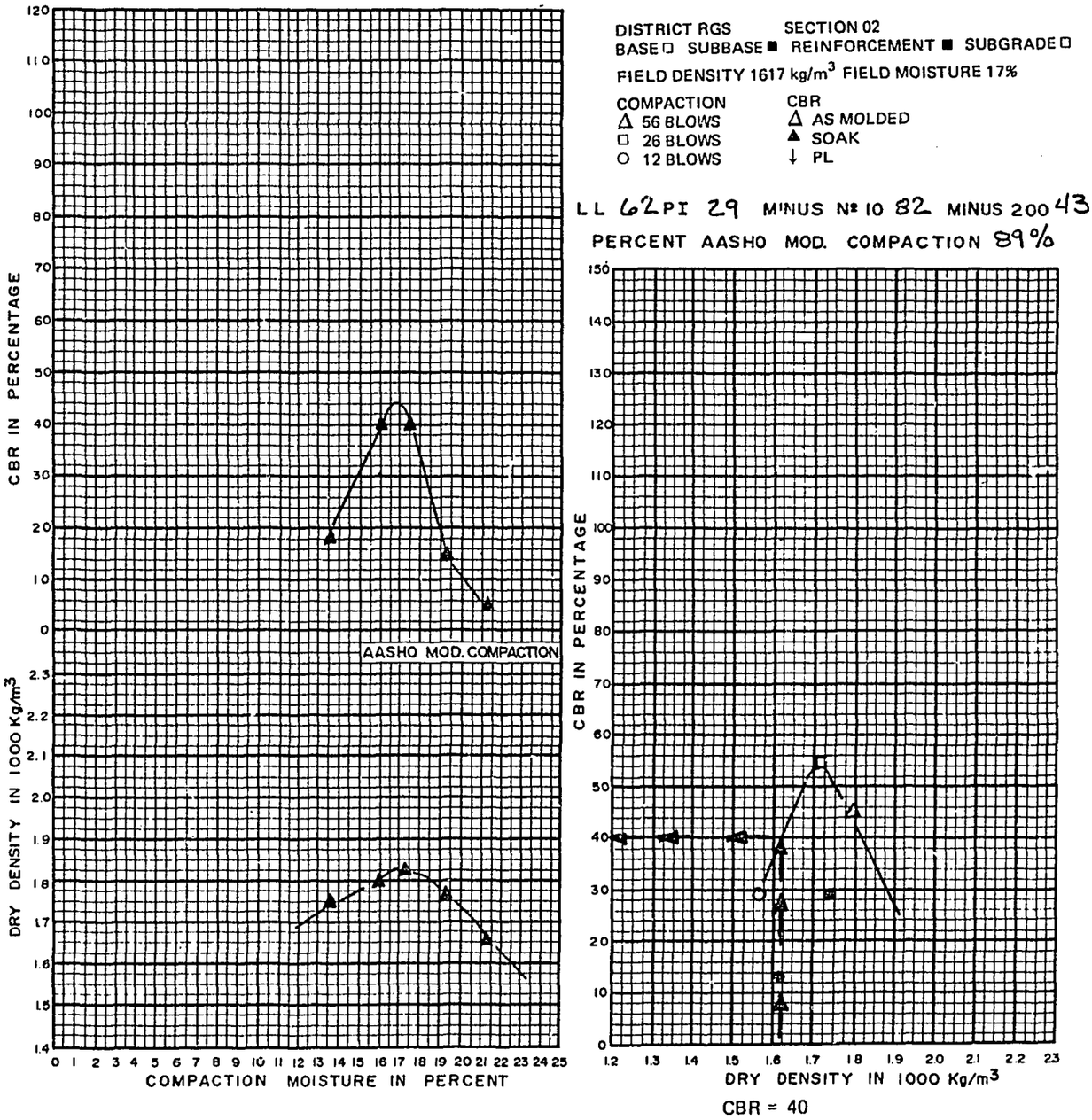
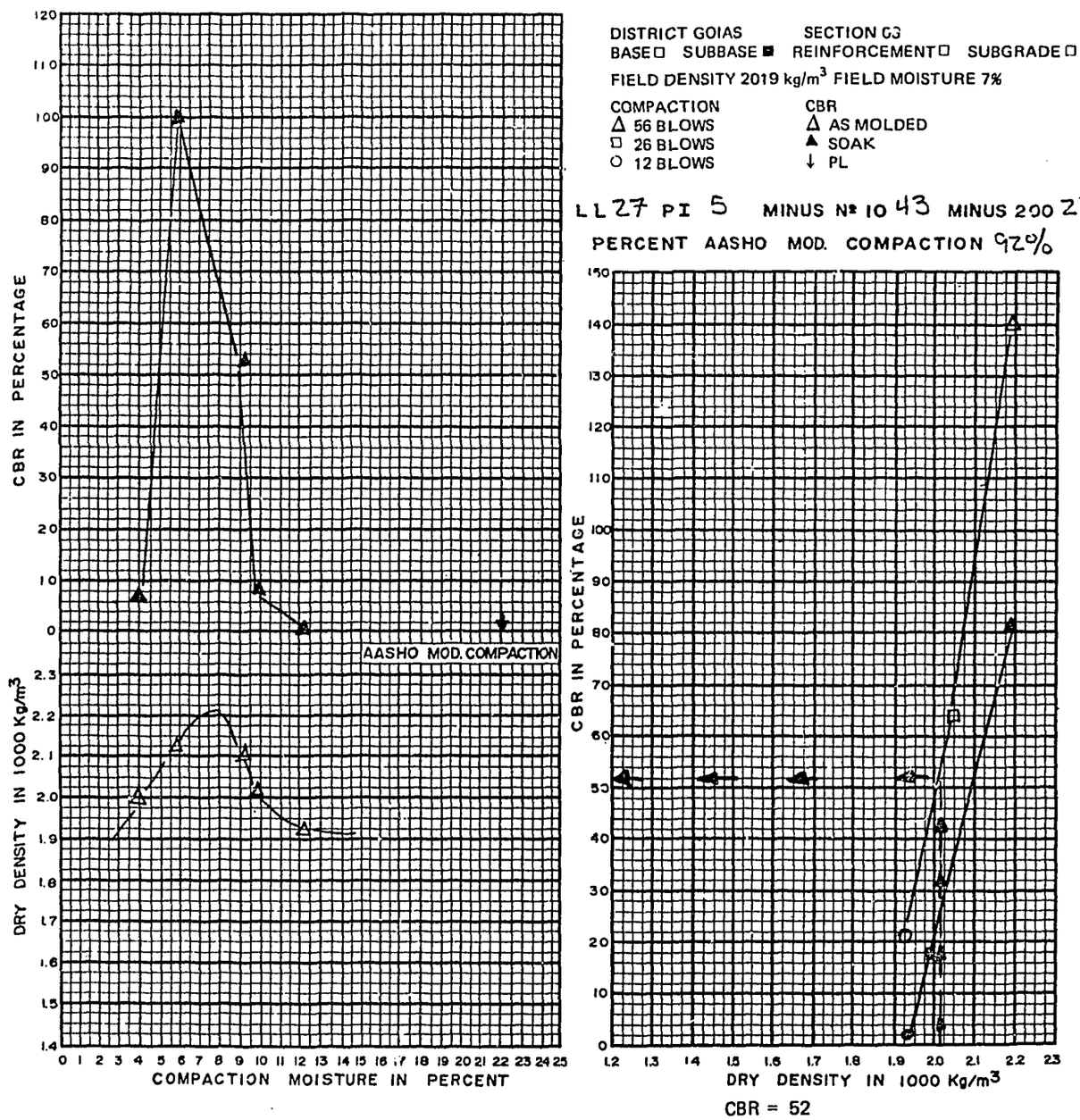


FIGURE A6.1a – TYPICAL RESULTS OF LABORATORY TESTING



MOISTURE-DENSITY-CBR SUMMARY SHEET

FIGURE A6.1b — TYPICAL RESULTS OF LABORATORY TESTING



MOISTURE-DENSITY-CBR SUMMARY SHEET

FIGURE A6.1c – TYPICAL RESULTS OF LABORATORY TESTING

TABLE A6.3
Structural Properties

Section	Surface Type	Thickness	Base Thickness	CBR	% Compaction	Subbase Thickness	CBR	% Compaction	Reinforcement Thickness	CBR	% Compaction	Subgrade I Thickness	CBR	% Compaction	Subgrade II CBR	% Compaction
Espírito Santo																
1	ST	2	15	96	93	18	32	95	–	–	–	27	8	88	0	77
2	ST	3	18	72	94	26	87	92	22	24	90	22	0	73	–	–
3	ST	3	20	44	91	18	35	85	–	–	–	49	30	86	–	–
5	ST	3	23	60	94	28	68	94	–	–	–	36	+ 50	95	–	–
6	ST	2	20	80	93	13	50	88	–	–	–	55	30	88	–	–
7	ST	2	20	87	97	15	76	95	–	–	–	53	0	87	–	–
8	ST	3	22	58	92	16	60	94	Bed Rock				–	–	–	–
9	ST	3	17	65	94	27	30	96	28	10	77	15	5	75	–	–
10	ST	4	30	60	93	11	25	87	–	–	–	45	32	88	–	–
Pernambuco																
1	ST	3	42	+100	92	25	75	89	–	–	–	20	3	75	–	–
2	ST	4	38	+100	97	25	+100	97	–	–	–	23	15	86	–	–
3	ST	4	19	+100	91	19	50	92	10	10	78	12	15	90	+40	90
4	ST	4	20	+100	94	20	34	87	20	20	84	26	9	85	–	–
5	ST	4	15	41	85	40	95	93	16	50	93	15	13	93	–	–
6	AC	6	67	+100	94	–	–	–	–	–	–	17	100	92	–	–
7	ST	4	17	+100	97	23	75	91	–	–	–	23	90	90	25	90
9	ST	4	20	75	88	23	90	94	19	80	92	24	1	78	–	–
10	ST	4	19	+100	97	28	100	99	–	–	–	22	18	93	2	73
12	ST	5	45	+100	99	16	25	96	–	–	–	24	+ 50	99	–	–
Maranhão																
2	SAC	5	20	10	90	–	–	–	–	–	–	65	5	87	–	–
4	SAC	4	20	20	96	–	–	–	–	–	–	20	4	85	2	80
5	SAC	4	20	45	91	20	15	86	–	–	–	46	22	81	–	–
6	SAC	3	20	55	95	15	35	87	–	–	–	52	55	99	–	–
7	ST	3	20	90	92	20	40	87	–	–	–	47	60	92	–	–
8	SAC	4	15	30	91	15	10	86	–	–	–	56	13	79	–	–
9	SAC	4	15	45	88	11	+100	95	–	–	–	60	52	88	–	–
11	SAC	3	15	70	88	40	68/30	91/86	–	–	–	32	+ 50	92	–	–
Minas Gerais																
1	AC	11	59	+100	97	–	–	–	–	–	–	20	62	91	–	–
2	AC	7	18	72	98	11	+100	97	9	45	100	45	50	100	–	–
3	AC	5	13	80	–	13	64	97	29	52	98	30	5	94	–	–
4	AC	13	36	+100	96	21	100	100	–	–	–	20	16	99	–	–
5	ST	3	15	64	88	19	46	89	20	40	87	33	+ 40	96	–	–
6	ST	4	41	74	93	25	34	95	–	–	–	20	18	90	–	–
7	ST	3	16	100	92	14	+100	94	14	92	94	43	6	86	–	–
8	AC	5	20	90	90	15	+100	100	–	–	–	30	25	85	17	85
9	ST	3	22	+100	96	16	40	90	–	–	–	49	70	87	–	–
10	ST	3	19	53	87	10	39	87	15	+100	97	43	+ 40	91	–	–
11	ST	5	8	+100	94	22	20	89	29	80	98	26	25	87	–	–
12	ST	4	11	60	95	9	94	92	22	10	83	44	2	69	–	–
13	ST	3	43	+100	93	24	30	90	–	–	–	20	6	79	–	–
14	ST	3	26	98	94	8	3	84	28	30	100	25	25	86	–	–
15	ST	3	15	+100	94	23	60	90	–	–	–	49	15	81	–	–

TABLE A6.3 (Continuation)

Section	Surface Type	Thickness	Base Thickness	CBR	% Compaction	Subbase Thickness	CBR	% Compaction	Reinforcement Thickness	CBR	% Compaction	Subgrade I Thickness	CBR	% Compaction	Subgrade II CBR	% Compaction
Ceará																
2	AC	6	20	46	96	25	35	81	–	–	–	39	25	76	–	–
3	CMC	9	16	75	94	20	10	86	–	–	–	45	0	74	–	–
5	CMC	7	19	60	88	18	65	86	23	27	81	23	0	70	–	–
6	ST	3	16	31	85	26	46	88	16	44	89	29	20	82	–	–
7	ST	3	18	80	87	18	78	88	20	30	81	31	14	79	–	–
8	ST	4	20	34	85	16	75	93	25	15	81	25	6	77	–	–
9	CMC	8	14	4	69	13	17	85	16	25	88	20	43	96	20	90
10	CMC	5	18	43	89	21	+100	94	–	–	–	20	4	80	0	73
12	SAC	5	26	AM	–	16	+100	92	15	52	85	28	10	73	–	–
Rio Grande do Norte																
2	AC	5	25	+100	96	–	–	–	–	–	–	20	95	100	+50	93
3	AC	5	30	+100	100	–	–	–	–	–	–	55	95	93	–	–
4	AC	5	15	+100	95	15	38	100	–	–	–	55	0	86	–	–
5	AC	4	25	+100	95	41	30	100	–	–	–	20	26	99	–	–
6	SAC	4	13	+100	100	12	60	100	–	–	–	61	14	90	–	–
7	ST	3	20	+100	98	20	100	98	–	–	–	47	50	100	–	–
9	SAC	3	17	+100	100	35	28	100	–	–	–	35	18	95	–	–
10	SAC	8	10	+100	100	11	95	97	–	–	–	61	75	100	–	–
11	ST	4	20	+100	99	20	+100	100	27	+100	100	19	35	90	–	–
13	ST	3	15	+100	100	15	31	91	20	+100	100	20	30	97	10	87
Pará																
4	SAC	5	20	+100	96	20	26	78	–	–	–	45	+ 40	90	–	–
5	SAC	6	35	+100	92	–	–	–	–	–	–	49	0	79	–	–
6	SAC	4	26	19	86	20	58	86	–	–	–	40	+ 40	75	–	–
7	SAC	4	25	31	83	25	8	74	–	–	–	36	36	84	–	–
8	SAC	6	30	60	84	27	23	84	–	–	–	27	8	72	–	–
10	AC	11	20	65	94	20	18	81	–	–	–	39	8	75	–	–
Goiás																
1	ST	2	17	90	91	22	68	89	20	29	84	29	9	70	–	–
2	AC	5	22	+100	91	20	66	89	15	50	86	28	30	82	–	–
3	AC	3	10	63	45	10	52	92	20	22	81	47	10	75	–	–
4	ST	4	9	62	89	9	94	89	16	39	89	52	20	85	–	–
5	ST	3	12	54	85	26	84	93	20	45	83	29	10	77	–	–
6	ST	3	32	50	89	20	37	89	20	17	88	15	8	90	–	–
7	ST	3	10	72	91	10	38	87	20	16	84	20	10	77	1	67
8	ST	2	23	90	93	14	64	88	20	25	80	20	36	84	0	69
9	ST	2	17	+100	92	14	68	90	12	28	88	20	11	80	0	68
10	ST	4	22	74	94	13	44	88	15	58	89	36	7	80	–	–
11	ST	3	18	+100	96	21	20	88	20	46	88	28	22	81	–	–
12	ST	3	13	90	95	14	30	91	20	49	90	23	100	97	3	77
13	ST	3	13	+100	96	14	54	91	–	–	–	60	29	87	–	–
Piauí																
1	SAC	4	31	59	88	29	39	87	–	–	–	26	18	84	–	–
2	SAC	5	19	55	88	18	48	90	–	–	–	48	+100	99	30	86
3	ST	3	24	61	86	20	70	86	20	20	83	23	25	79	–	–
4	CMC	4	20	98	92	40	+100	100	–	–	–	26	+ 50	95	–	–
5	SAC	6	32	75	87	20	+100	91	23	30	81	9	+ 40	91	–	–
7	AC	7	31	+100	89	14	100	92	–	–	–	33	+ 50	90	–	–

TABLE A6.3 (Continuation)

Section	Surface Type	Thickness	Base Thickness	CBR	% Compac-tion	Subbase Thickness	CBR	% Compac-tion	Reinforce-ment Thickness	CBR	% Compac-tion	Subgrade I Thickness	CBR	% Compac-tion	Subgrade II CBR	% Compac-tion
Piauí																
8	SAC	5	20	65	86	54	+100	94	–	–	–	11	8	70	–	–
9	SAC	5	22	55	82	30	98	88	–	–	–	33	22	83	–	–
10	SAC	4	33	72	90	15	35	86	–	–	–	23	25	86	40	86
11	SAC	5	24	30	89	25	+100	91	–	–	–	36	19	75	–	–
São Paulo																
1	AC	16	55	+100	100	–	–	–	–	–	–	19	28	91	–	–
2	ST	5	30	M	–	17	36	90	–	–	–	38	0	73	–	–
4	AC	10	44	+100	100	–	–	–	–	–	–	36	5	85	–	–
6	AC	20	25	+100	98	20	65	99	–	–	–	25	4	82	–	–
7	AC	27	20	42	96	20	25	85	–	–	–	23	1	76	–	–
9	AC	20	10	14	81	20	20	85	–	–	–	40	50	86	–	–
10	AC	13	30	100	100	27	+100	98	–	–	–	20	40	90	–	–
11	AC	10	51	+100	103	14	18	91	–	–	–	15	6	87	–	–
12	AC	16	20	94	96	30	72	95	–	–	–	24	20	98	–	–
13	AC	8	60	+100	104	11	59	92	–	–	–	11	6	81	–	–
Rio Grande do Sul																
1	AC	10	28	24	87	10	48	91	12	25	93	30	1	66	–	–
2	AC	10	28	+100	97	20	40	89	–	–	–	32	22	84	–	–
3	AC	15	40	60	94	19	80	90	–	–	–	16	3	89	–	–
4	ST	3	16	60	93	23	35	89	15	10	90	33	5	75	–	–
5	AC	14	14	AM	–	15	78	94	15	32	95	32	+ 30	95	–	–
6	AC	37	19	M	–	16	20	87	–	–	–	18	0	86	–	–
7	AC	15	35	M	–	20	30	95	–	–	–	20	18	89	–	–
8	AC	9	47	M	–	15	22	91	–	–	–	19	2	85	–	–
9	AC	13	39	M	–	–	–	–	–	–	–	38	2	84	–	–
10	AC	9	39	M	–	–	–	–	–	–	–	42	4	85	–	–
11	AC	10	17	M	–	48	+100	–	–	–	–	15	2	86	–	–
12	AC	10	36	M	–	13	5	86	–	–	–	31	2	80	–	–
13	AC	8	30	M	–	22	45	94	–	–	–	30	0	80	–	–
14	AC	18	27	45	96	–	–	–	–	–	–	45	12	95	–	–
15	AC	18	30	–	–	21	27	95	–	–	–	21	0	82	–	–
Mato Grosso																
1	ST	2	18	+100	93	20	90	98	20	48	89	30	28	88	–	–
3	AC	4	68	+100	97	–	–	–	–	–	–	18	90	91	–	–
5	MIP	9	14	16	93	–	–	–	–	–	–	67	22	89	–	–
6	MIP	10	22	40	95	20	28	95	–	–	–	38	2	78	–	–
8	ST	5	59	+100	95	–	–	–	–	–	–	26	48	89	–	–
10	ST	5	15	73	92	18	+100	96	15	47	90	37	23	79	–	–
13	AC	9	35	+100	99	20	52	94	–	–	–	26	96	99	–	–
14	AC	8	50	+100	100	20	54	105	–	–	–	12	3	65	–	–
15	AC	8	45	+100	103	15	60	90	–	–	–	22	25	97	–	–
Brazilia																
2	ST	3	12	+100	100	12	100	100	–	–	–	63	100	89	–	–
4	ST	3	15	55	96	15	+100	100	15	+100	95	42	0	83	–	–
6	ST	3	15	60	90	20	+100	96	20	+100	100	32	+100	100	–	–
7	ST	3	20	+100	100	43	98	96	–	–	–	24	1	97	–	–
10	ST	2	14	75	100	15	+100	100	20	65	100	38	5	91	–	–
12	ST	3	18	+100	96	35	29	93	–	–	–	34	15	95	–	–
13	ST	3	35	+100	100	20	73	99	15	25	88	17	4	88	–	–

TABLE A6.3 (Continuation)

Section	Surface Type	Thickness	Base Thickness	CBR	% Compaction	Subbase Thickness	CBR	% Compaction	Reinforcement Thickness	CBR	% Compaction	Subgrade I Thickness	CBR	% Compaction	Subgrade II CBR	% Compaction
Brazilia																
15	ST	3	17	84	100	20	+100	95	17	85	97	42	32	94	–	–
16	AC	15	17	100	100	15	85	98	15	10	88	28	0	74	–	–
20	ST	3	15	70	97	50	+100	100	–	–	–	22	5	93	–	–
Paraíba																
1	SAC	10	15	+100	97	16	+100	98	26	+100	100	15	85	100	15	89
2	SAC	7	20	+100	93	21	85	92	21	76	98	21	26	90	–	–
3	SAC	4	21	84	94	10	70	96	20	90	100	20	72	95	8	77
4	ST	2	18	+100	100	24	20	72	21	100	100	25	40	100	–	–
5	ST	3	11	+100	100	22	+100	95	25	58	88	39	10	76	–	–
6	AC	6	19	+100	100	20	+100	99	20	76	95	25	+100	100	–	–
7	AC	6	20	76	95	25	–	88	39	25	100	–	–	–	–	–
8	ST	2	20	+100	91	20	75	97	20	18	87	23	0	73	–	–
9	ST	2	22	100	89	17	+100	97	23	78	97	25	+ 50	88	–	–
10	ST	2	21	+100	95	16	+100	95	20	+100	97	31	+ 50	86	–	–
Paraná																
1	AC	3	12	M	–	22	85	97	–	–	–	53	2	76	–	–
4	AC	16	14	M	–	16	76	93	–	–	–	44	21	93	–	–
5	AC	10	30	PM	–	–	–	–	–	–	–	50	6	86	–	–
7	AC	6	9	PM	–	11	M	–	28	5	87	36	1	78	–	–
8	AC	6	9	PM	–	13	M	–	23	2	86	39	5	88	–	–
9	AC	5	10	PM	–	8	M	–	40	6	88	27	8	91	–	–
10	AC	6	10	AM	–	12	M	–	12	UM	–	50	0	77	–	–
11	AC	6	9	AM	–	10	M	–	30	UM	–	35	35	92	–	–
12	AC	5	10	AM	–	13	M	–	35	UM	–	27	8	81	–	–
13	AC	5	9	AM	–	21	M	–	31	10	87	24	14	84	–	–
14	AC	6	7	AM	–	15	M	–	–	–	–	62	3	97	–	–
15	AC	6	8	M	–	13	M	–	–	–	–	63	7	85	–	–

NOTE:
(1) Thickness in cm
(2) % of AASHO Modified Compaction
(3) ST – Surface Treatment
(4) AC – Asphalt Concrete
(5) SAC – Sand Asphalt Concrete
(6) CMC – Colo Mix Asphalt Concrete
(7) AM – Asphalt Macapam
(8) M – Macapam
(9) MIP – Mix in-place Asphalt Concrete
(10) PM – Premixed Macadam
(11) UM – Ungraded Macadam

DESIGN EXAMPLES

Design 1

Design traffic period 125,000 SAL (both directions) assume a construction variation of 35%.

Step 1:

From Figure 7.2 required structural index = 30.

Step 2:

From Figure 7.4 (subgrade CBR = 9) the minimum thickness of cover = 30 cm. The subgrade support index = 7. Therefore, the surface, base and subbase must provide a structural index of 23. Required SI (30) -- subgrade SI (7) = 23.

MATERIALS AVAILABLE FOR BASE AND SUBBASE

	CBR	Structural Coefficients (Table 7.1)		
		0-25 cm	25-50 cm	50-90 cm
Base	80	1.102	0.576	0.481
Subbase	30	NA → 17 → 0 cm	0.205	0.481

From Figure 7.3 the minimum thickness of cover for a CBR of 30 is 17 cm. The material can only be used below a depth of 17 cm because Table 7.1 shows the minimum CBR to which a structural coefficient can be assigned between the depths of 0 and 25 cm is 50. Therefore, the coefficient for a CBR of 30 is shown as NA between the depths of 0 and 17 cm. The material can be used between the depths of 17 and 25 cm; however, no structural strength is assigned.

Step 3:

From Table 7.2 the recommended surfacing for a base course CBR of 80 and a designed traffic period of 125,000 SAL (both directions) is a DBST (double bituminous surface treatment).

Step 4:

From Figure 7.3 the minimum thickness of cover for a CBR of 30 is 17 cm. Rounding off, the required base course thickness is 20 cm. To provide the minimum cover over the subgrade (30 cm) the required thickness of the subbase is 10 cm.

Step 5:

The design equation:

$$SI = a_1 t_1 + a_2 t_2 - + a_n t_n$$

is used to determine if the required SI can be obtained within the minimum design thickness.

$$SI = (1.102)(20) + (0)(5)^{(1)} + (.205)(5) + (0)(20)^{(2)} + (.183)(40) = 30$$

$$t = 20 + 5 + 5 + 20 + 40 = 90 \text{ cm}$$

(1) The subbase material (CBR 30) is not assigned a coefficient between the depths of 20 and 25 cm.

(2) The subgrade material (CBR 9) is not assigned a coefficient between the depths of 30 and 50 cm.

Design: DBST, 20 cm base and 10 cm subbase.

Design 2

Design traffic period 1,500,000 SAL (both directions).

Step 1:

From Figure 7.2 required structural index = 50. Base, subbase and subgrade materials the same as in Design 1, therefore, the surface, base and subbase must provide a structural index of 43.

Step 2:

Required SI (50) – Subgrade SI (7) = 43.

Step 3:

From Table 7.2 the recommended surfacing for a base course CBR of 80 and a designed traffic period of 1,500,000 SAL (both directions) is 10 cm of asphalt concrete.

Step 4:

From Figure 7.6, the required structural index of the unbound soil layers between the depths of 10 and 50 cm is 19.

As determined in Design 1, the minimum thickness of cover for the subbase is 20 cm (rounded up from 17 cm). The asphalt concrete provides 10 cm, the minimum base course thickness required is 10 cm. To provide the minimum cover over the subgrade (30 cm) the required thickness of the subbase is 10 cm.

Step 5:

Compute the SI of the unbound soil layers between the depth of 10 and 50 cm.

$$SI_{10\text{-}50} = (1.102)(10) + (0)(5) + (.205)(5) + (0)(20) = 12 < 19$$

Try a base course thickness of 15 cm

$$SI_{10\text{-}50} = (1.102)(15) + (.205)(5) + (0)(20) = 17.56 \sim 18$$

Checking; Figure 7.6 the combined structural index is 42.5.

$42.5 + 7 = 49.5 \sim 50$

Design: 10 cm AC, 15 cm base and 5 cm subbase. In this case the subbase thickness would be increased to 10 cm to facilitate construction of the layer. The increase in structural index would not warrant redesign.

Design 3

The design requirements are the same as Design 1. In order to determine the required thickness, the Design Tables will be used.

The type of surfacing and the subbase CBR determines the table to be used. For a DBST surfacing and a subbase material with a CBR of 30 use Design Table A7.5.

Step 1:

From Figure 7.2 required SI = 30.

Step 2:

From Table 7.2 use DBST.

Step 3:

Locate the subgrade CBR in the first column on the left, move to the right to the columns captioned by a CBR of 80 for the base course. The structural index is indicated for various applicable thicknesses of base course layers. Locate a structural index equal to or slightly greater than

30. Move up the column to determine the base course thickness (20 cm), move to the right to the columns captioned as subbase thickness stopping at the previously determined base course thickness (20 cm). Move down the column to a subgrade CBR of 9. The number (10) indicates a 10 cm thickness of subbase which is based on the minimum thickness of cover for the subgrade CBR value.
Design: DBST, 20 cm base and 10 cm subbase.

Design 4

The design requirements are the same as Design 2.

In order to determine the required thickness the Design Tables will be used.

Step 1:

From Figure 7.2 required SI = 50.

Step 2:

From Table 7.2 use 10 cm asphalt concrete.

Step 3:

Design Table A7.7 is used for surfacing of 10 cm of asphalt concret and a subbase material with a CBR of 30. Follow the procedure outlined in Step 1 of Design 3.

Design: 10 cm AC, 15 cm base and 5 cm subbase. In this case the subbase thickness would be increased to 10 cm to facilitate construction of the layer. The increase in structural index would not warrant redesign.

Design 5

Design traffic period 350,000 SAL (both directions).

Step 1:

From Figure 7.2 required SI = 38.

Step 2:

From Table 7.2 use DBST.

Step 3:

As shown in Design Table A7.5, the maximum SI is 36 for minimum thickness design. An addition SI of 2 is required to satisfy the requirements in Step 1. A decision is made to increase the SI of the section by increasing the subbase thickness.

Step 4:

An increase in subbase thickness of 10 cm greater than the thickness required by minimum thickness design (5 cm) will increase the structural index by 2 (0.205 x 10 cm = 2.05). Check with structural equation.

$$SI = (1.102)(25) + (.205)(15) + (0)(10) + (.183)(40) = 37{,}95 \sim 38$$

Design: DBST, 25 cm base and 15 cm subbase.

Design 6

Design traffic period of 2,100,000 SAL (both directions).

Step 1:

From Figure 7.2, required structural index = 53. Base, subbase and subgrade the same as in Design 1.

Step 2:

From Table 7.2, the recommended surfacing for a base course CBR of 80 and designed traffic period of 2,100,000 SAL (both directions) is 10 cm of asphalt concrete.

Step 3:

As shown in Design Table A7.5 the maximum SI is 50 for minimum thickness design. An additional SI of 3 is required to satisfy the requirements in Step 1. A decision is made to increase the SI of the section by increasing the subbase thickness.

Step 4:

Required SI (53) – Subgrade SI (7) = 46.

Step 5:

From Figure 7.6, the required structural index of the unbound soil layers between the depth of 10 and 50 cm is 24. Minimum thickness design (Design 2) provides a structural index of 17.56. An additional thickness of subbase of 20 cm provides an increase in the structural index of 4.1 (.205 x 20 cm). The total SI of the unbound soil layers between 10 and 50 cm would be 21.66 (rounding off) 22. From Figure 7.6 the combined structural number would be 45 ∿ 46.

Design: 10 cm AC, 15 cm base and 25 cm subbase.

DESIGN TABLE A7.1
DBST – Subbase CBR-25

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR 50				CBR 60				CBR 70				CBR 80				CBR 90				CBR+100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40																												
30																									–	–	–	–
20			28	30			31	34			39	43			42	47			45	51			48	55	10	5	–	–
15			22	24			25	28			33	38			36	42			39	45			42	49	13	8	3	–
10			16	18			20	23			28	32			31	36			33	40			37	44	19	14	9	4
9			15	17			19	21			26	31			30	35			32	38			36	43	20	15	10	5
8				15			17	20			25	29			28	33			30	37			34	41	21	16	11	6
7								17			22	27			26	31			28	34			31	38	23	18	13	8
6								17			22	26			25	30			28	34			31	38	25	20	15	10
5								16			21	26			24	30			27	33			30	37	28	23	18	13
4								16			21	26			24	30			27	33			30	37	30	25	20	15
3								16			21	26			24	30			27	33			30	37	34	29	24	19
2								16			21	26			24	30			27	33			30	37	40	35	30	25

Structural index for pavement sections with various base course materials and subgrade CBR values along with required subbase thickness.

DESIGN TABLE A7.2

AC 5 cm – Subbase CBR-25

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR 50				CBR 60				CBR 70				CBR 80				CBR 90				CBR+100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40																									–	–	–	–
30																									2	–	–	–
20			46			46	50			53	58			56	63			57	65			60	65		5	–	–	–
15			44			44	45			49	54			51	57			52	60			55	64		8	3	–	–
10			38			38	41			43	48			46	51			47	55			50	58		14	9	4	–
9			37			37	39			41	47			45	50			46	54			49	57		15	10	5	–
8			35			35	37			40	45			43	49			44	52			47	56		16	11	6	1
7			33			33	35			38	43			41	47			43	50			45	54		18	13	8	3
6			32			32	34			37	42			40	46			42	49			44	52		20	15	10	5
5			31			31	33			37	41			39	45			41	48			44	52		23	18	13	8
4			31			31	33			36	41			38	45			41	48			44	52		25	20	15	10
3			31			31	33			36	41			39	45			41	48			44	52		29	24	19	14
2			31			31	34			36	42			39	45			41	48			44	52		35	30	25	20

Subbase CBR 25

Structural index for pavement sections with various base course materials and subgrade CBR values along with required subbase thickness.

DESIGN TABLE A7.3
AC 10cm – Subbase CBR-25

SG	Base Course CBR and Thickness																								Sub-base Thickness			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR +100				Basethickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 63																											
40	63	63			64	64			67	70			69	73			70	76			72	79			–	–	–	–
30	60	60			60	60			60	62			61	64			62	65			62	67			–	–	–	–
20	–	–			–	60			60	61			60	61			60	62			61	63			–	–	–	–
15	–	–			–	55			55	56			55	56			55	57			56	59			3	–	–	–
10	–	–			–	50			50	50			50	51			50	51			50	53			9	4	–	–
9	–	–			–	48			48	49			48	49			48	50			49	51			10	5	–	–
8	–	–			–	45			46	47			46	47			46	48			47	50			12	7	2	–
7	–	–			–	44			44	45			44	45			44	46			45	48			13	8	3	–
6	–	–			–	43			43	44			43	44			43	45			44	46			15	10	5	–
5	–	–			–	42			42	43			42	44			42	44			43	46			18	13	8	3
4	–	–			–	42			42	42			42	43			42	44			42	46			20	15	10	5
3	–	–			–	42			42	42			42	43			42	44			42	46			24	19	14	9
2	–	–			–	41			41	42			41	43			42	44			42	46			30	25	20	15

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.4
AC 15 cm — Subbase CBR-25

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR+100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 63																											
30	Asphalt Placed Directly on Subgrade S1 = 76																											
40																									–	–	–	–
30																									–	–	–	–
20	61				62				64				67				67				69				–	–	–	–
15	61				61				62				63				64				64				–	–	–	–
10	61				61				61				61				61				62				4	–	–	–
9	61				61				61				61				61				61				5	–	–	–
8	61				61				61				61				61				61				6	1	–	–
7	–				61				61				61				61				61				8	3	–	–
6	–				61				61				61				61				61				10	5	–	–
5	–				61				61				61				61				61				13	8	3	–
4	–				–				61				61				61				61				15	10	5	–
3	–				–				61				61				61				61				19	14	9	4
2	–				–				61				61				61				61				25	20	15	10

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.5

DBST — Subbase CBR-30

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR+100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/	/
30	/	/	32	34	/	/	35	38	/	/	43	48	/	/	46	52	/	/	49	55	/	/	52	59	7	2	–	–
20	/	/	28	30	/	/	31	34	/	/	39	43	/	/	42	47	/	/	45	51	/	/	48	55	10	5	–	–
15	/	/	22	24	/	/	25	28	/	/	33	38	/	/	36	42	/	/	39	45	/	/	42	49	13	8	3	–
10	/	/	17	19	/	/	20	23	/	/	28	33	/	/	31	37	/	/	34	40	/	/	37	44	19	14	9	4
9	/	/	16	18	/	/	19	22	/	/	27	32	/	/	30	36	/	/	33	39	/	/	36	43	20	15	10	5
8	/	/	/	16	/	/	18	20	/	/	25	30	/	/	29	34	/	/	31	37	/	/	34	41	21	16	11	6
7	/	/	/	15	/	/	16	19	/	/	24	29	/	/	27	33	/	/	29	36	/	/	33	40	23	18	13	8
6	/	/	/	/	/	/	15	18	/	/	23	28	/	/	26	32	/	/	28	35	/	/	32	39	25	20	15	10
5	/	/	/	/	/	/	15	18	/	/	23	27	/	/	26	31	/	/	28	35	/	/	32	39	28	23	18	13
4	/	/	/	/	/	/	15	18	/	/	23	27	/	/	26	31	/	/	28	35	/	/	32	39	30	25	20	15
3	/	/	/	/	/	/	15	18	/	/	23	28	/	/	27	32	/	/	29	35	/	/	32	39	34	29	24	19
2	/	/	/	/	/	/	17	19	/	/	24	29	/	/	28	33	/	/	30	36	/	/	33	40	40	35	30	25

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.6

AC 5 cm — Subbase CBR-30

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = +100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40																									–	–	–	–
30		51	52			53	56			59	64			61	68			64	72			67	76		2	–	–	–
20		–	46			46	50			53	58			56	63			57	65			60	65		5	–	–	–
15		–	44			44	45			49	54			51	57			52	60			55	64		8	3	–	–
10		–	38			39	42			44	49			47	53			49	56			52	61		14	9	4	–
9		–	37			38	39			42	47			45	51			47	54			50	59		15	10	5	–
8		–	36			36	38			41	45			43	49			45	53			47	57		16	11	6	1
7		–	34			34	36			39	44			42	48			44	51			46	55		18	13	8	3
6		31	33			33	35			38	43			41	47			43	50			46	55		20	15	10	5
5		31	32			33	35			38	43			40	47			42	50			46	55		23	18	13	8
4		30	30			33	35			38	43			40	47			43	50			46	54		25	20	15	10
3		31	32			33	36			39	44			41	48			44	51			47	55		29	24	19	14
2		32	33			34	37			40	45			43	49			45	52			48	57		35	30	25	20

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.7
AC 10 cm – Subbase CBR-30

SG	Base Course CBR and Thickness																								Subbase Thickness			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = +100				Base Thick			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 63																								–	–	–	–
40	63	63			64	64			67	70			69	73			70	76			72	79			–	–	–	–
30	60	60			60	60			60	62			61	64			62	65			62	67			–	–	–	–
20	–	–			–	60			60	61			60	61			60	62			61	63			–	–	–	–
15	–	–			–	55			55	56			55	56			55	57			56	58			3	–	–	–
10	–	–			–	50			50	50			50	51			50	52			50	53			9	4	–	–
9	–	–			–	48			48	49			48	50			48	50			49	52			10	5	–	–
8	–	–			–	46			46	47			46	48			47	49			47	50			12	7	2	–
7	–	–			–	44			44	45			44	46			45	47			45	48			13	8	3	–
6	–	–			–	43			43	44			43	45			43	46			44	47			15	10	5	–
5	–	42			42	42			42	43			42	44			43	45			43	47			18	13	8	3
4	–	42			42	42			42	43			42	44			42	45			43	47			20	15	10	5
3	–	42			42	42			42	43			42	44			43	46			43	47			24	19	14	9
2	41	41			41	41			42	43			42	44			43	46			44	48			30	25	20	15

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.8
AC 15 cm – Subbase CBR-30

SG	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
CBR	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = +100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 63																								–	–	–	–
30	Asphalt Placed Directly on Subgrade S1 = 76																								–	–	–	–
																									–	–	–	–
40																									–	–	–	–
30	64				67				71				73				75				77				–	–	–	–
20	61				62				64				67				67				69				–	–	–	–
15	61				61				62				63				64				64				–	–	–	–
10	61				61				61				61				61				62				4	–	–	–
9	61				61				61				61				61				61				5	–	–	–
8	61				61				61				61				61				61				6	1	–	–
7	61				61				61				61				61				61				8	3	–	–
6	61				61				61				61				61				61				10	5	–	–
5	61				61				61				61				61				61				13	8	3	–
4	–				61				61				61				61				61				15	10	5	–
3	–				61				61				61				61				61				19	14	9	4
2	–				61				61				61				61				61				25	20	15	10

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.9

DBST – Subbase CBR-35

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = +100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40																												
30		30	32	34		33	35	38		38	43	48		41	46	52		43	49	55		45	52	59	7	2	–	–
20		26	28	30		28	31	34		34	39	43		36	42	47		39	45	51		41	48	55	10	5	–	–
15		20	22	24		23	25	28		28	33	38		31	36	42		33	39	45		35	42	49	13	8	3	–
10		15	17	19		18	21	23		24	28	33		26	32	37		28	34	40		31	38	45	19	14	9	4
9		15	16	18		17	20	23		23	28	32		25	31	36		27	33	40		30	37	44	20	15	10	5
8				17		15	18	21		21	26	31		24	29	35		26	32	38		28	35	42	21	16	11	6
7				15			17	19		20	25	29		22	28	33		24	30	36		27	34	41	23	18	13	8
6							16	19		19	24	29		22	27	33		24	30	36		26	33	40	25	20	15	10
5							16	19		19	24	29		22	27	33		24	30	36		26	33	40	28	23	18	13
4							16	19		19	24	29		22	27	33		24	30	36		26	33	40	30	25	20	15
3				16			17	20		20	25	30		23	28	34		25	31	37		27	34	41	34	29	24	19
2			15	17		16	19	21		22	26	31		24	29	35		26	32	38		29	36	43	40	35	30	25

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.10

AC 5 cm — Subbase CBR-35

Asphalt Surfacing at 5 cm

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR= + 100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40																									–	–	–	–
30	50	51	52		50	53	56		52	59	64		56	61	68		61	64	72		63	67	76		2	–	–	–
20	–	–	46		–	46	50		40	53	53		51	56	63		52	57	65		54	60	65		5	–	–	–
15	–	–	44		–	44	45		45	49	54		46	50	57		47	54	60		48	61	64		8	3	–	–
10	–	–	38		–	39	41		39	44	49		41	46	53		42	48	55		44	51	59		14	9	4	–
9	–	37	37		–	38	40		39	43	48		40	46	52		41	48	55		43	49	59		15	10	5	–
8	–	35	36		–	36	39		37	41	46		38	44	51		39	46	53		41	48	57		16	11	6	1
7	–	33	34		33	34	38		35	40	45		37	42	48		38	45	52		40	47	56		18	13	8	3
6	–	32	33		32	34	36		35	39	44		36	42	48		37	44	51		39	47	56		20	15	10	5
5	31	32	33		31	33	36		35	39	45		36	42	49		37	44	51		39	47	56		23	18	13	8
4	31	32	33		31	34	36		34	39	45		36	42	48		38	44	52		39	48	57		25	20	15	10
3	31	32	34		32	34	37		36	40	46		38	43	50		38	46	53		40	49	58		29	24	19	14
2	32	34	35		33	36	39		37	42	48		39	43	52		40	47	56		42	51	61		35	30	25	20

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.11
AC 10 cm – Subbase CBR-35

SG CBR	Base Course CBR and Thickness																								Subbase Thickness			
	CBR 50				CBR 60				CBR 70				CBR 80				CBR 90				CBR + 100				Base Thick			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade 63																											
40	63	63			64	64			67	70			69	73			70	76			72	79			–	–	–	–
30	60	60			60	60			60	62			61	64			62	65			62	67			–	–	–	–
20	–	–			–	60			60	61			60	61			60	62			61	63			–	–	–	–
15	–	–			–	55			55	56			55	56			55	57			56	58			3	–	–	–
10	–	–			–	50			50	50			50	51			50	52			51	53			9	4	–	–
9	–	–			–	48			48	49			48	50			49	51			49	52			10	5	–	–
8	–	46			–	46			46	47			46	48			47	50			47	51			12	7	2	–
7	–	44			–	44			44	45			45	46			45	47			45	49			13	8	3	–
6	–	43			43	43			43	44			43	45			44	46			44	48			15	10	5	–
5	–	42			42	42			42	43			43	45			43	46			44	48			18	13	8	3
4	42	42			42	42			42	43			42	46			43	46			43	48			20	15	10	5
3	42	42			42	42			42	44			43	46			43	47			44	49			24	19	14	9
2	42	42			42	42			42	45			43	47			43	48			44	51			30	25	20	15

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.12
AC 15 cm – Subbase CBR-35

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = + 100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 76														76										–	–	–	–
30	Asphalt Placed Directly on Subgrade S1 = 62														62									–	–	–	–	–
40																									–	–	–	–
30	64				67				71				73				75				77				–	–	–	–
20	61				62				64				67				67				69				–	–	–	–
15	61				61				62				63				64				64				–	–	–	–
10	61				61				61				61				61				62				4	–	–	–
9	61				61				61				61				61				61				5	–	–	–
8	61				61				61				61				61				61				6	1	–	–
7	61				61				61				61				61				61				8	3	–	–
6	61				61				61				61				61				61				10	5	–	–
5	61				61				61				61				61				61				13	8	3	–
4	61				61				61				61				61				61				15	10	5	–
3	61				61				61				61				61				61				19	14	9	4
2	61				61				61				61				61				61				25	20	15	10

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.13
DBST – Subbase CBR-40

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = +100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	43				45	46	47		49	51	54		51	54	58		52	56	60		53	58	63		–	–	–	–
30	31	31	32		33	33	35		36	39	43		38	42	46		39	44	49		41	46	52		7	2	–	–
20	28	28	28		29	30	31		33	36	39		35	38	42		36	40	45		38	43	48		10	5	–	–
15	23	23	23		25	26	26		29	31	34		30	34	37		32	36	40		33	38	43		13	8	3	–
10	20	20	20		22	23	24		26	29	32		28	31	35		29	33	37		30	36	41		19	14	9	4
9	20	20	20		21	22	23		25	28	31		27	31	34		28	33	37		30	35	40		20	15	10	5
8	18	18	18		20	21	22		24	27	30		26	29	33		27	31	35		28	34	39		21	16	11	6
7	18	18	18		19	20	21		23	26	29		25	28	32		26	31	35		28	33	38		23	18	13	8
6	17	17	17		19	20	21		23	26	29		25	28	32		26	31	34		28	33	38		25	20	15	10
5	18	18	18		20	21	22		24	27	30		26	29	33		27	31	35		29	34	39		28	23	18	13
4	19	19	19		21	22	22		25	27	30		26	30	33		28	32	36		29	34	39		30	25	20	15
3	21	21	21		21	23	24		25	27	29		28	32	36		30	34	38		31	36	41		34	29	24	19
2	24	24	24		26	27	28		30	33	36		32	35	39		33	37	41		34	40	45		40	35	30	25

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.14

AC 5 cm — Subbase CBR-40
Asphalt Surfacing at 5 cm

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = + 100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	62	\	\	\	64	65	66	\	69	72	77	\	71	76	81	\	72	79	86	\	75	83	90	\	–	–	–	–
30	50	51	52	\	51	53	56	\	54	59	64	\	56	61	68	\	57	64	72	\	59	67	76	\	2	–	–	–
20	–	–	46	\	–	46	50	\	51	53	58	\	53	56	63	\	54	57	65	\	55	60	65	\	5	–	–	–
15	–	–	44	\	44	44	45	\	47	50	54	\	49	52	57	\	50	54	60	\	51	57	64	\	8	3	–	–
10	40	40	40	\	41	41	42	\	44	47	50	\	46	50	54	\	47	52	57	\	49	55	61	\	14	9	4	–
9	38	38	38	\	40	41	41	\	44	47	50	\	45	49	53	\	47	51	57	\	48	55	60	\	15	10	5	–
8	37	37	37	\	38	39	40	\	42	45	50	\	44	48	52	\	45	50	55	\	47	53	60	\	16	11	6	1
7	36	36	36	\	38	38	39	\	41	44	48	\	43	47	51	\	44	50	55	\	47	52	60	\	18	13	8	3
6	36	36	36	\	37	38	39	\	41	44	48	\	43	47	52	\	45	50	55	\	47	53	60	\	20	15	10	5
5	37	37	37	\	38	39	40	\	42	46	49	\	44	49	53	\	46	51	57	\	48	54	62	\	23	18	13	8
4	37	37	37	\	39	40	43	\	43	47	50	\	45	50	55	\	47	52	58	\	49	56	63	\	25	20	15	10
3	39	39	39	\	41	42	45	\	46	49	53	\	48	52	57	\	49	55	62	\	52	59	67	\	29	24	19	14
2	43	43	43	\	45	46	49	\	50	54	58	\	52	57	63	\	54	60	67	\	56	64	72	\	35	30	25	20

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.15

AC 10 cm — Subbase CBR-40

Asphalt Surfacing at 10 cm

SG CBR	Base Course CBR and Thickness																								Subbase Thickness			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = + 100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade 63																											
40	63	63			64	64			67	70			69	73			70	76			72	79			–	–	–	–
30	60	60			60	60			60	62			61	64			62	65			62	67			–	–	–	–
20	–	–			–	60			60	61			60	61			60	62			61	63			–	–	–	–
15	–	–			–	55			55	56			55	56			55	57			56	59			3	–	–	–
10	50	50			50	50			50	50			50	51			51	53			51	54			9	4	–	–
9	48	48			48	48			49	49			49	50			50	52			50	53			10	5	–	–
8	46	46			46	46			47	48			47	49			48	50			49	52			12	7	2	–
7	44	44			44	44			45	46			46	47			46	49			47	51			13	8	3	–
6	43	43			43	43			44	45			45	47			45	48			47	50			15	10	5	–
5	42	42			43	43			44	46			45	47			46	49			47	52			18	13	8	3
4	42	42			43	43			44	46			45	48			46	50			47	53			20	15	10	5
3	43	43			43	44			46	48			47	50			48	53			49	55			24	19	14	9
2	44	44			46	46			48	51			50	54			51	57			53	61			30	25	20	15

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

DESIGN TABLE A7.16

AC 15 cm — Subbase CBR-40

Asphalt Surfacing at 15 cm

SG CBR	Base Course CBR and Thickness (cm)																								Subbase Thickness (cm)			
	CBR = 50				CBR = 60				CBR = 70				CBR = 80				CBR = 90				CBR = + 100				Base Thickness			
	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25	10	15	20	25
40	Asphalt Placed Directly on Subgrade S1 = 76														76										–	–	–	–
30	Asphalt Placed Directly on Subgrade S1 = 62														62										–	–	–	–
40	76				79				85				88				91				94				–	–	–	–
30	64				67				71				73				75				77				–	–	–	–
20	61				62				64				67				67				69				–	–	–	–
15	61				61				62				63				64				64				–	–	–	–
10	61				61				61				61				61				62				4	–	–	–
9	61				61				61				61				61				62				5	–	–	–
8	61				61				61				61				61				62				6	1	–	–
7	61				61				61				61				61				61				8	3	–	–
6	61				61				61				61				61				61				10	5	–	–
5	61				61				61				61				61				61				13	8	3	–
4	61				61				61				61				61				62				15	10	5	–
3	61				61				61				62				62				62				19	14	9	4
2	61				61				62				63				63				64				25	20	15	10

Structural Index for Pavement Sections with various base course materials and subgrade CBR values along with subbase thickness required for minimum cover over the subgrade.

TABLE A7.17
Comparison of Design Procedures

Design Procedure	Thickness (cm)			DE18KSAL 20 Year Analysis	CBR Values
	Asphalt	Base	Subbase		
Tropical	5	10	25	100* (15)	
AASHO	5	10	25	10	
Tropical	10	10	25	112 (43)	
AASHO	10	10	25	50	
AI	10	10	25	28*	
Tropical	15	10	25	384 (260)	Base + 100**
AASHO	15	10	25	210	SB 50
AI	15	10	25	1,856*	SG 5
Tropical	5	25	25	100* (100*)	
AASHO	5	25	25	45	
Tropical	10	25	25	432 (135)	
AASHO	10	25	25	200	
AI	10	25	25	28*	
Tropical	15	25	25	786 (636)	
AASHO	15	25	25	650	
AI	15	25	25	1,856*	
Tropical	10	15	18	295	Base 500 PSI
AASHO	10	15	18	100	SB 30
NLA	10	15	18	547	SG 5
Tropical	DBST	25	–	7	Base 50
AASHO	DBST	25	–	1	SG 20
NLA	DBST	25	–	5	
Tropical	DBST	20	15	14	Base + 100
AASHO	DBST	20	15	2	SB 30
NLA	DBST	20	15	14-20	SG 6
Florida	DBST	20	15	7-41	

* Limited by thickness of asphalt
** AASHO Summer CBR Values
( ) Using AASHO SPRING CBRs in calculations
NLA – National Lime Association
AI – Asphalt Institute

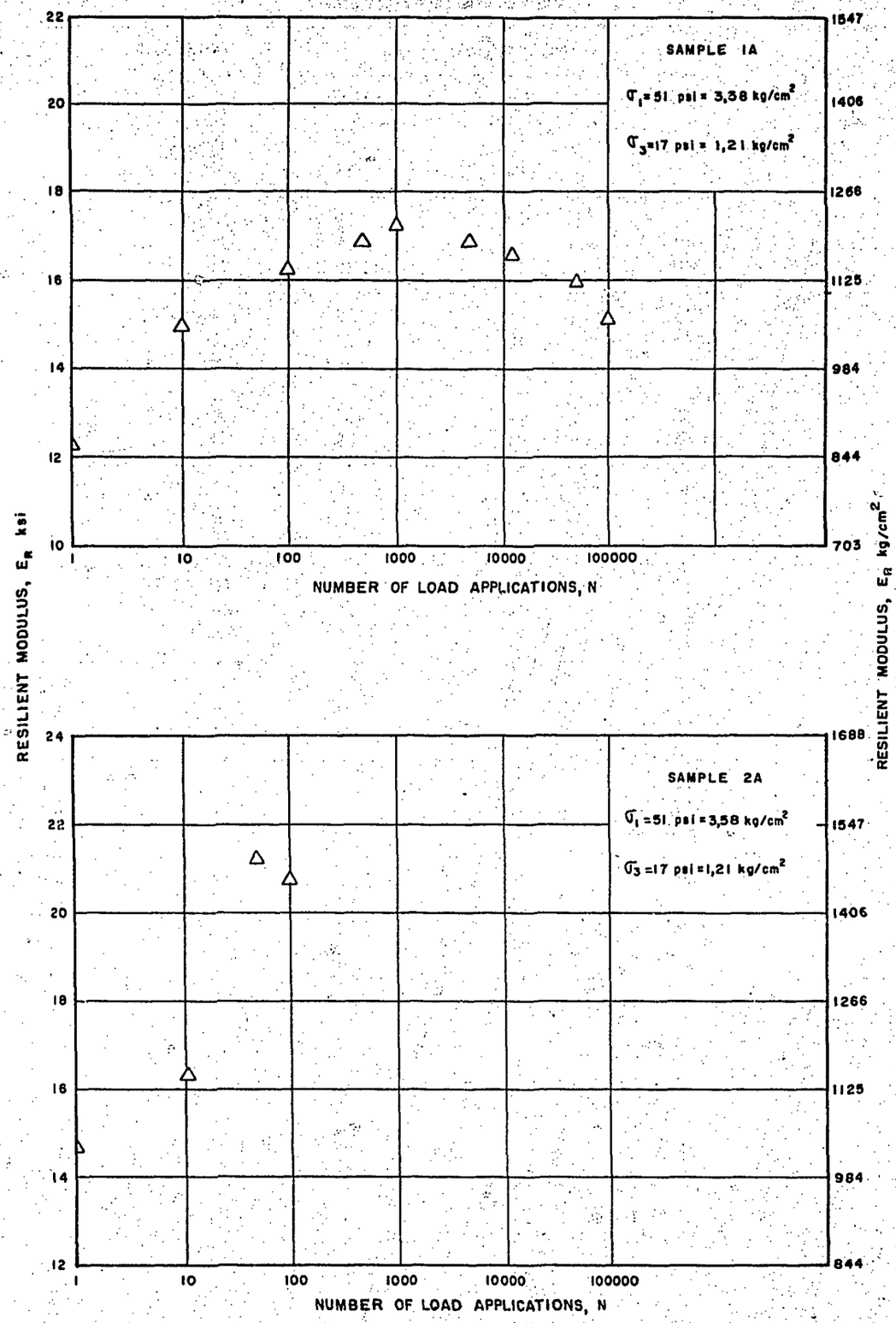


FIGURE A8.1 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND NUMBER OF LOAD APPLICATIONS

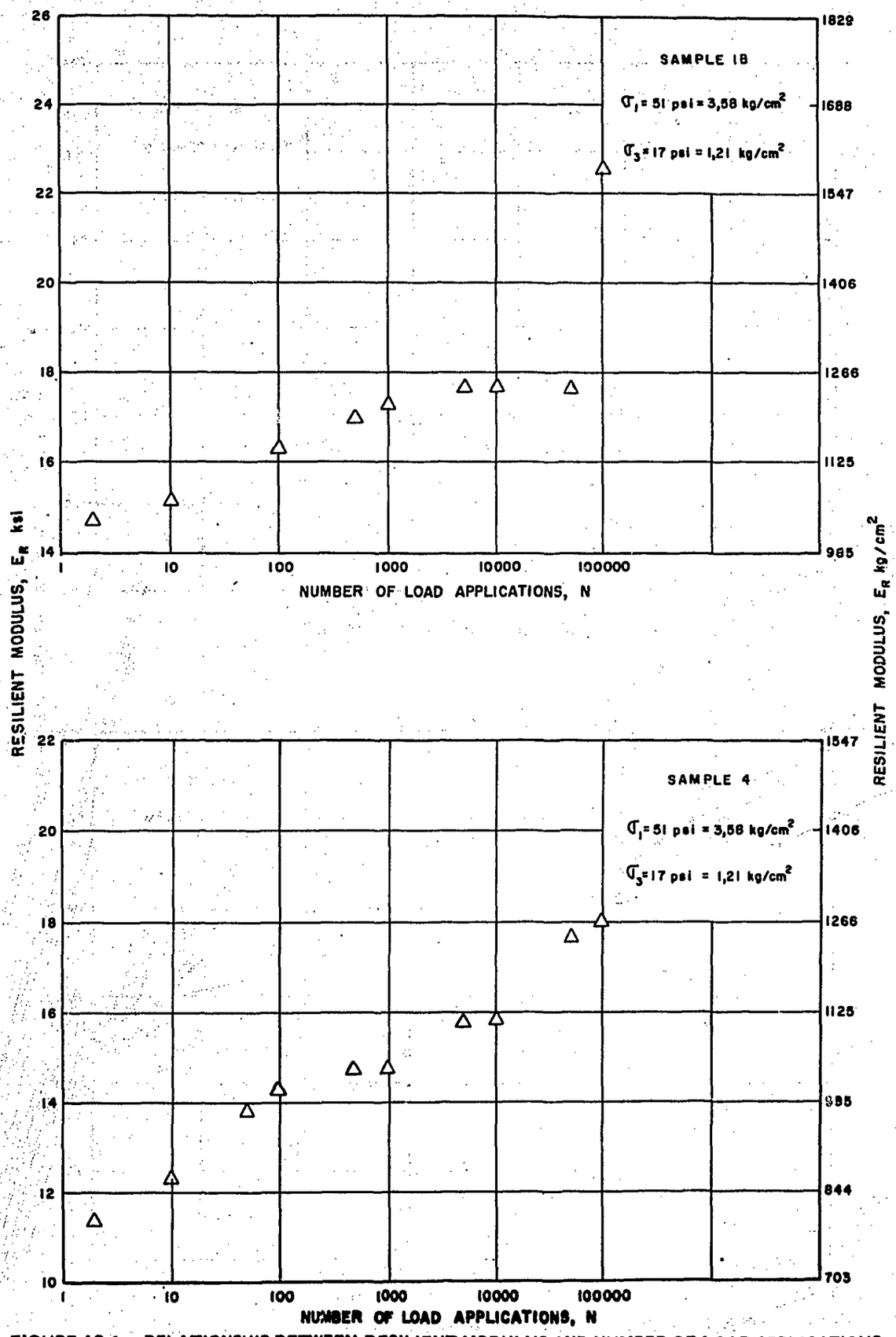


FIGURE A8.1 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND NUMBER OF LOAD APPLICATIONS (continued)

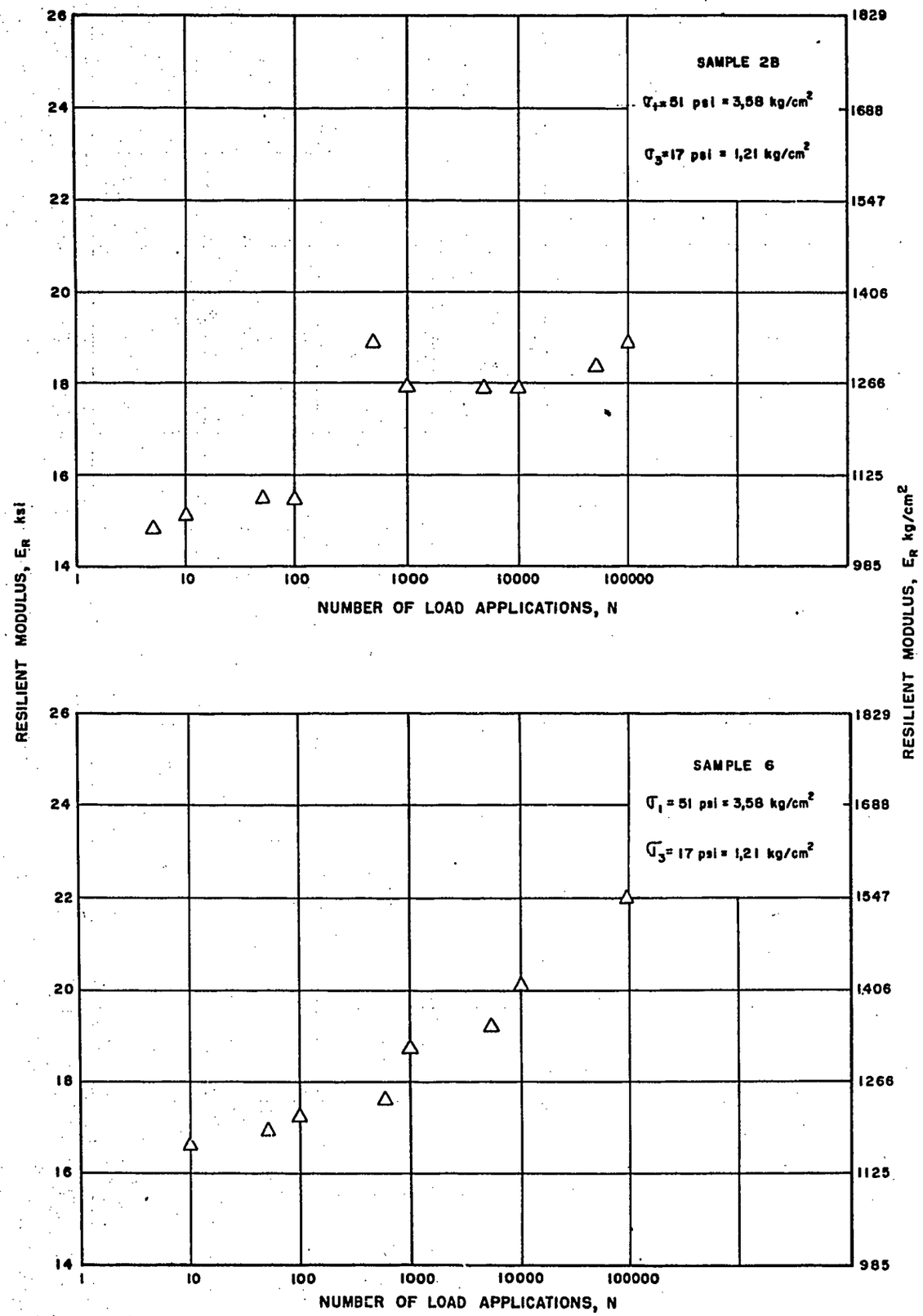


FIGURE A8.1 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND NUMBER OF LOAD APPLICATIONS (continued)

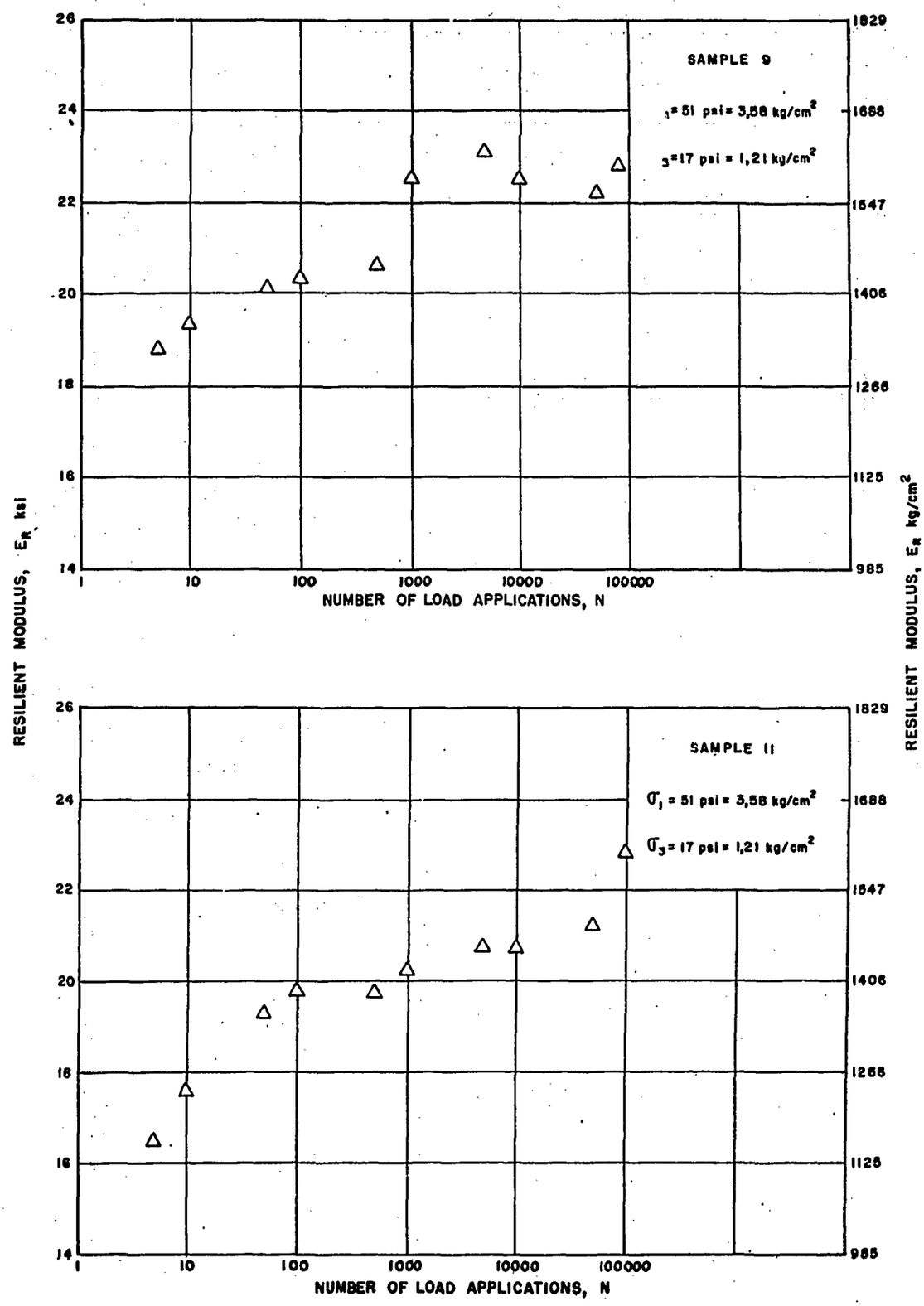


FIGURE A8.1 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND NUMBER OF LOAD APPLICATIONS (continued)

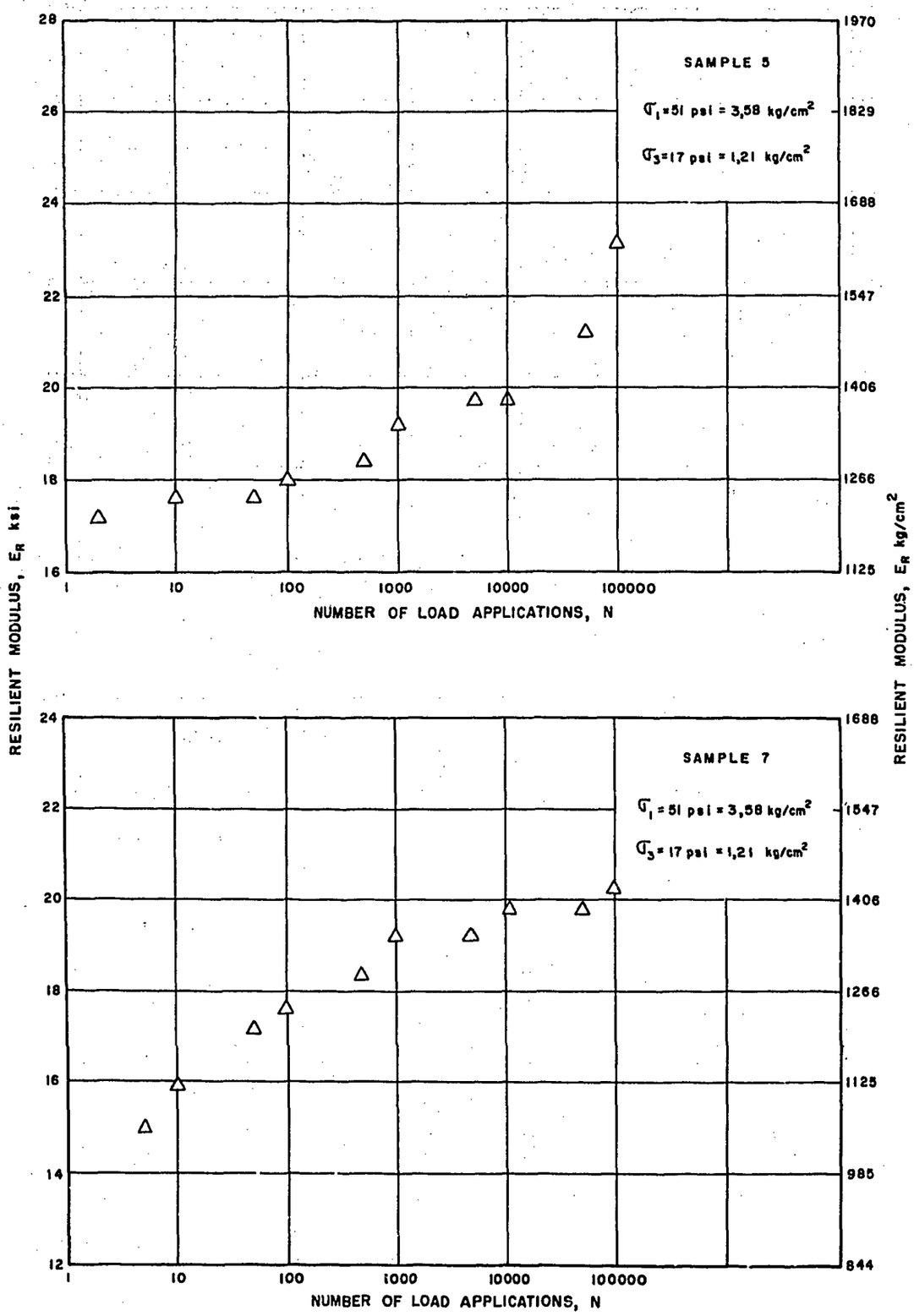


FIGURE A8.1 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND NUMBER OF LOAD APPLICATIONS (continued)

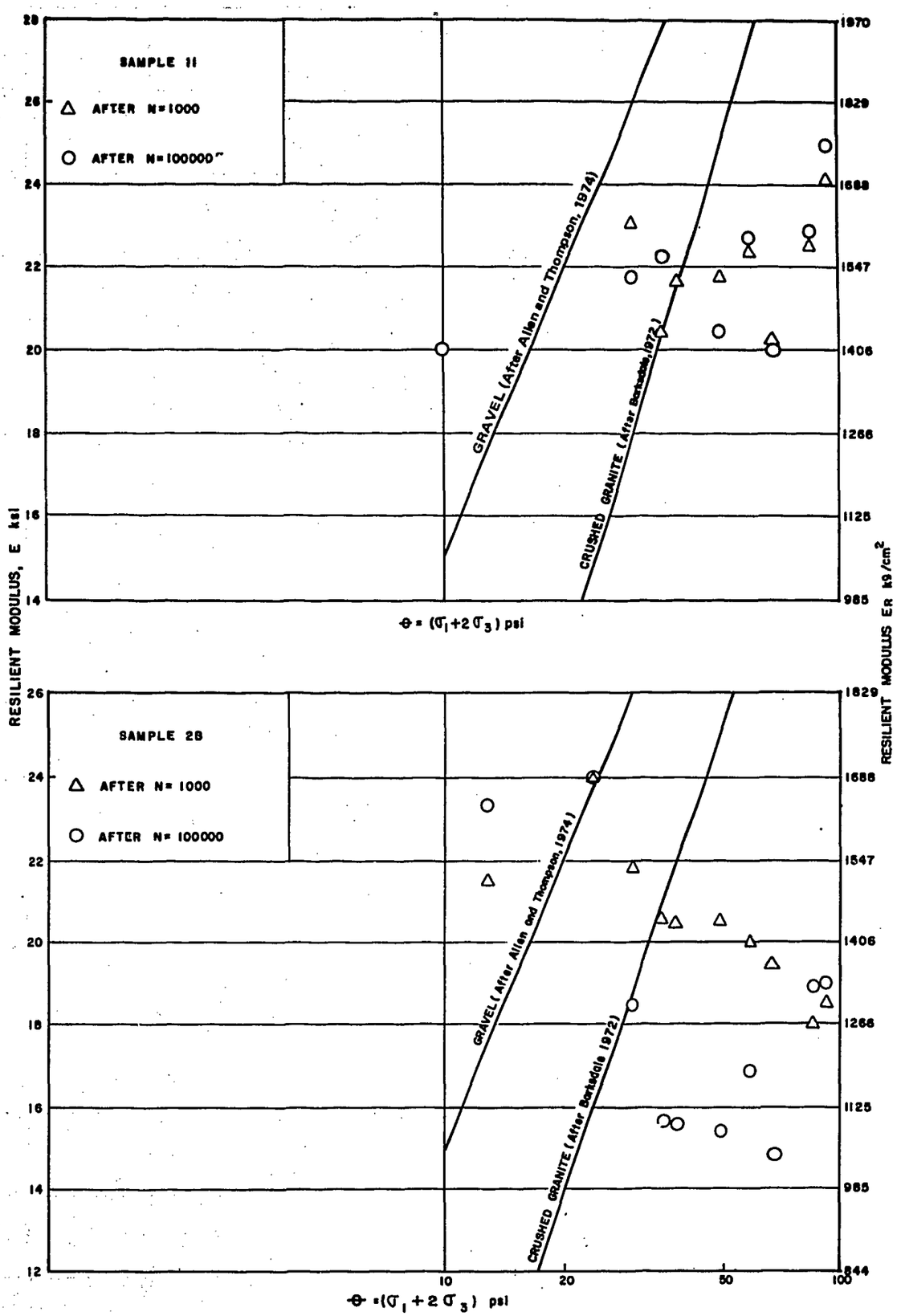


FIGURE A8.2 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND ϕ

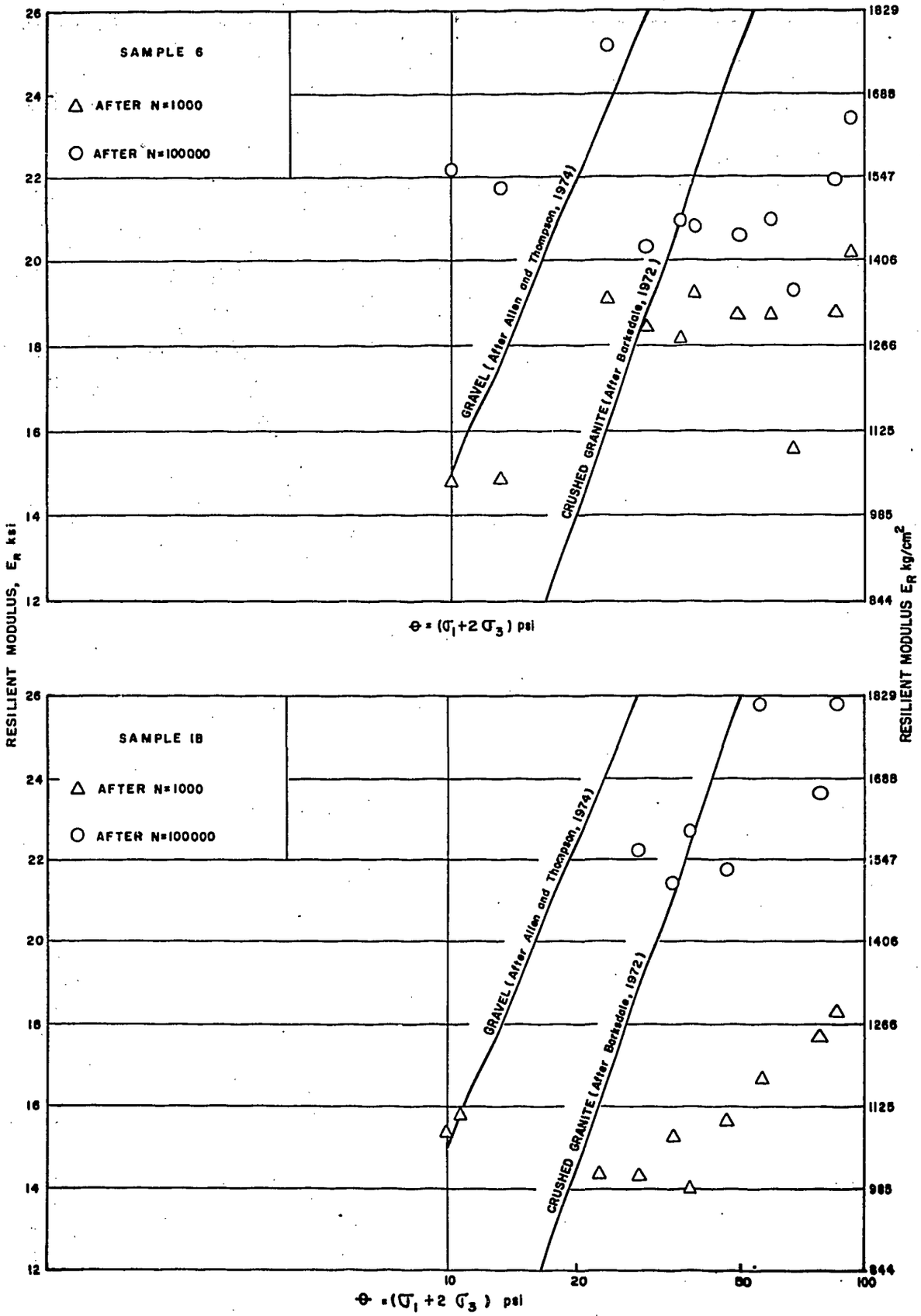


FIGURE A8.2.— RELATIONSHIP BETWEEN RESILIENT MODULUS AND ϕ (continued)

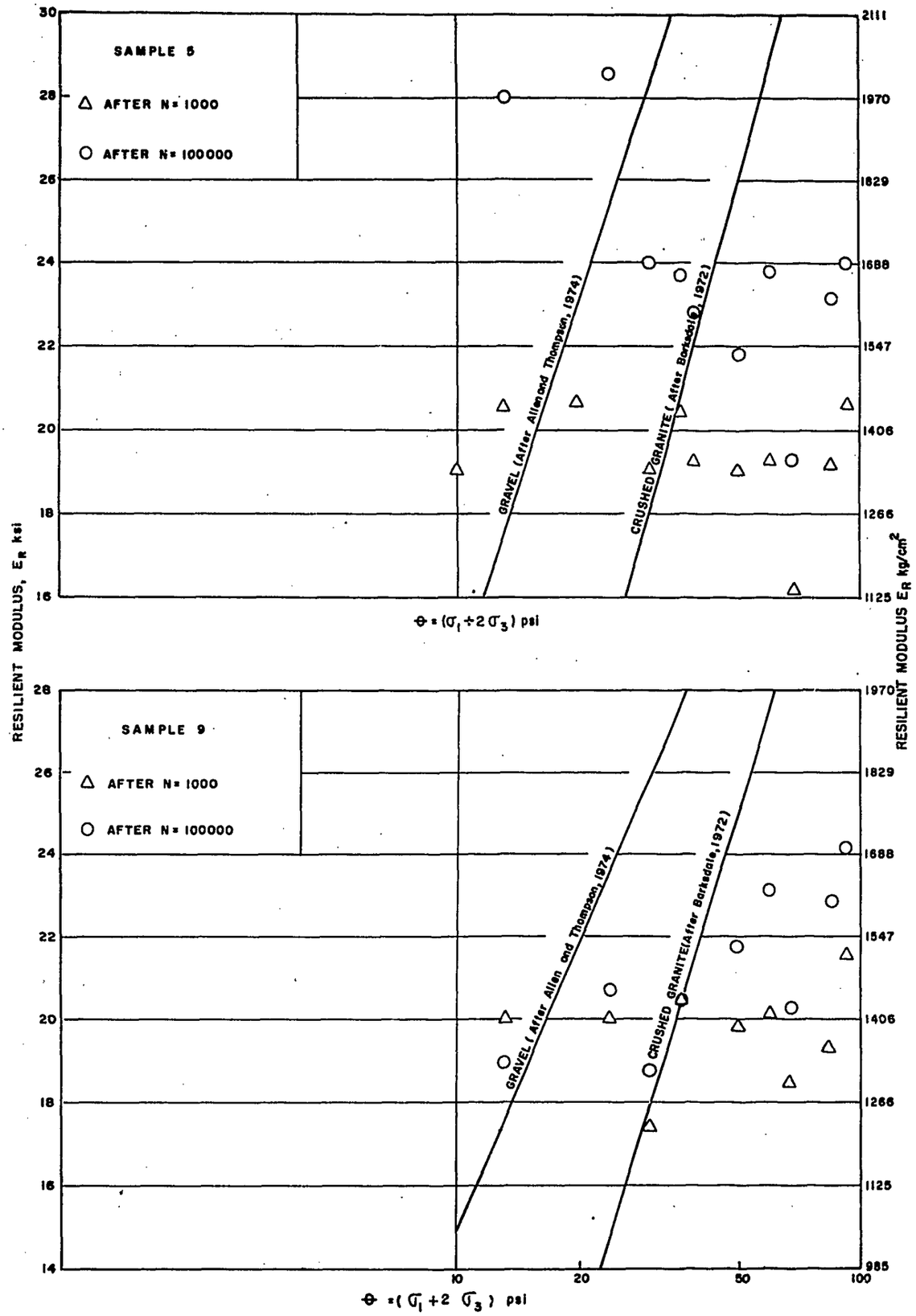


FIGURE A8.2 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND ϕ (continued)

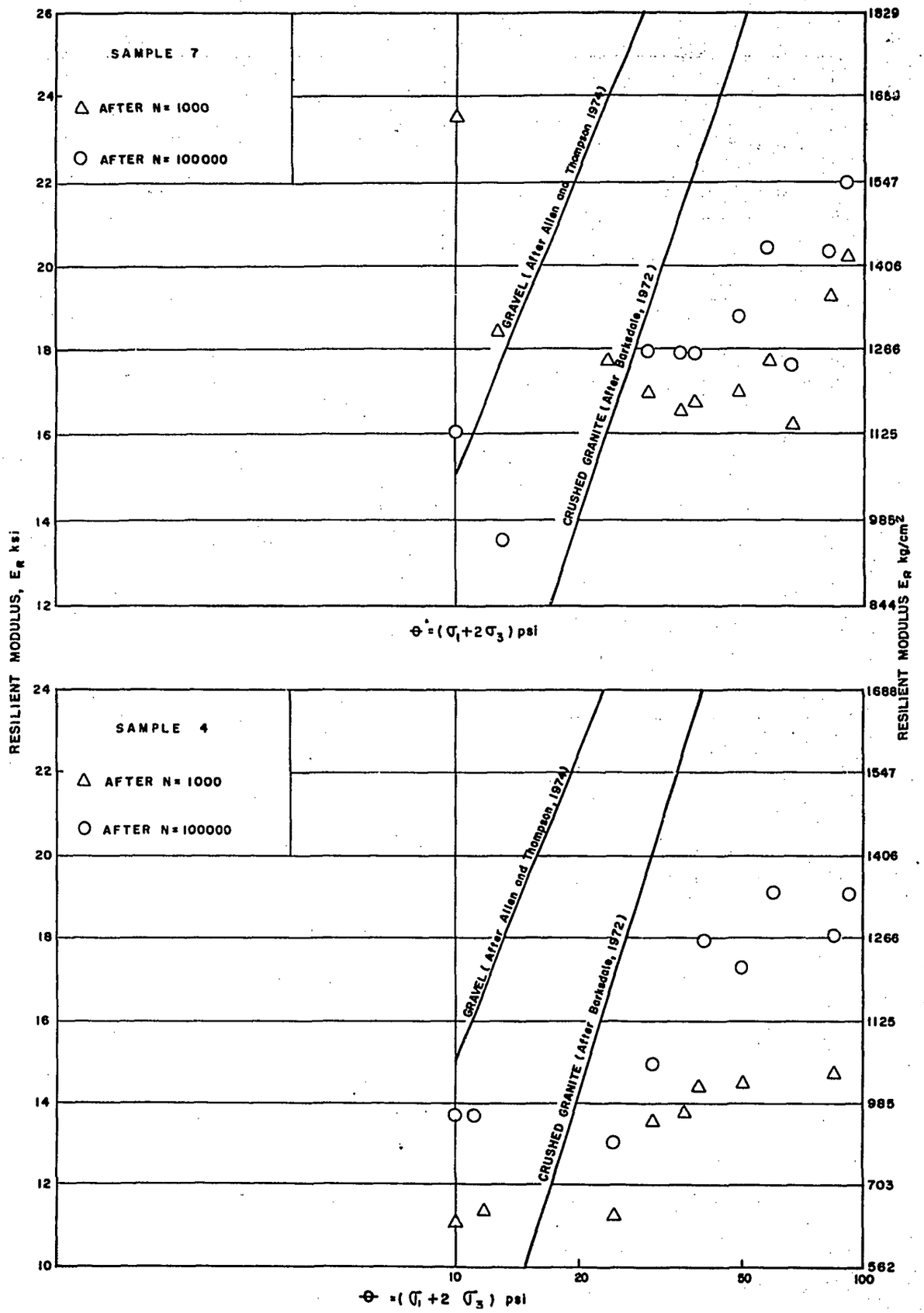


FIGURE A8.2 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND ϕ (continued)

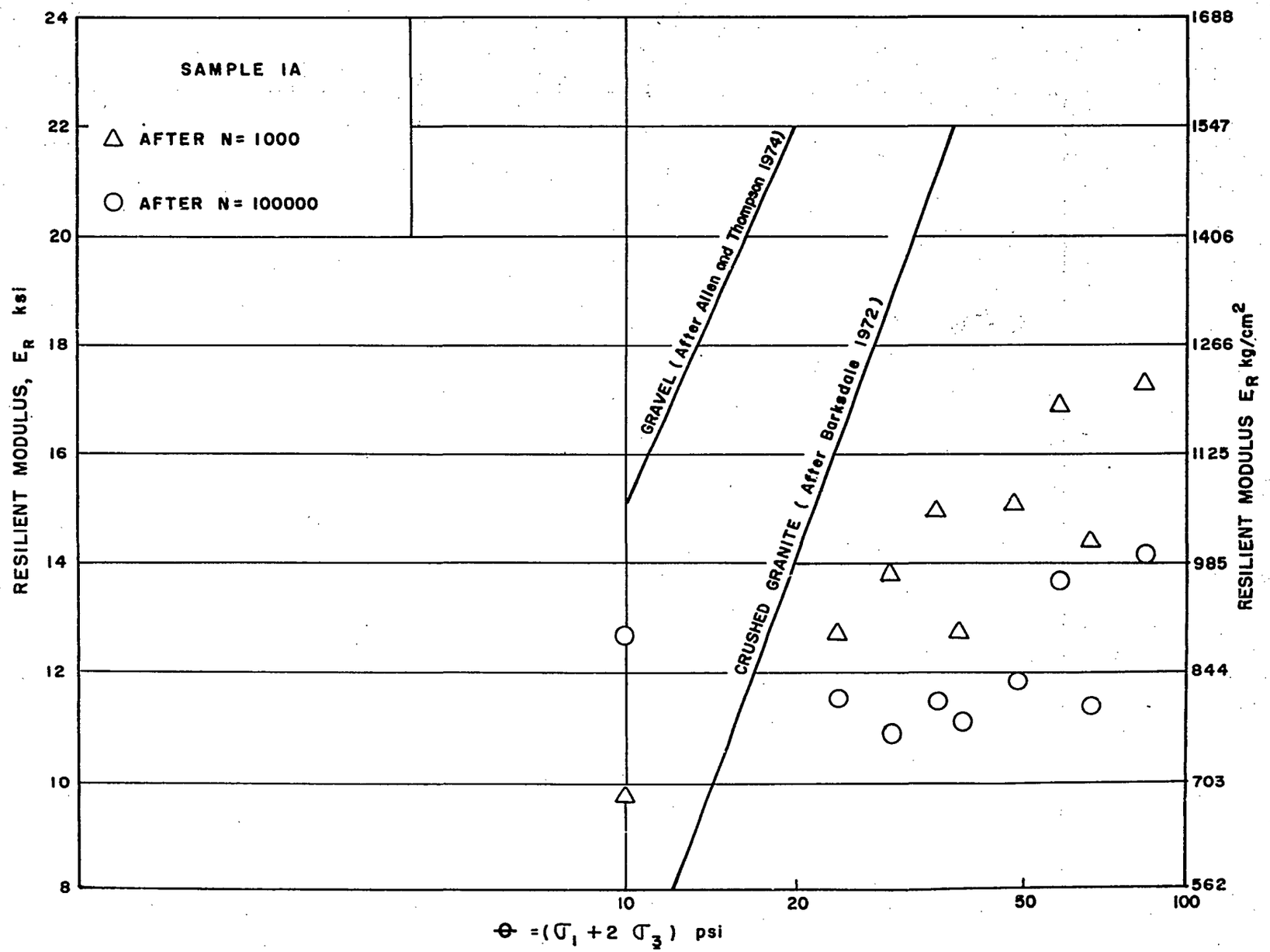


FIGURE A8.2 – RELATIONSHIP BETWEEN RESILIENT MODULUS AND ϕ (continued)

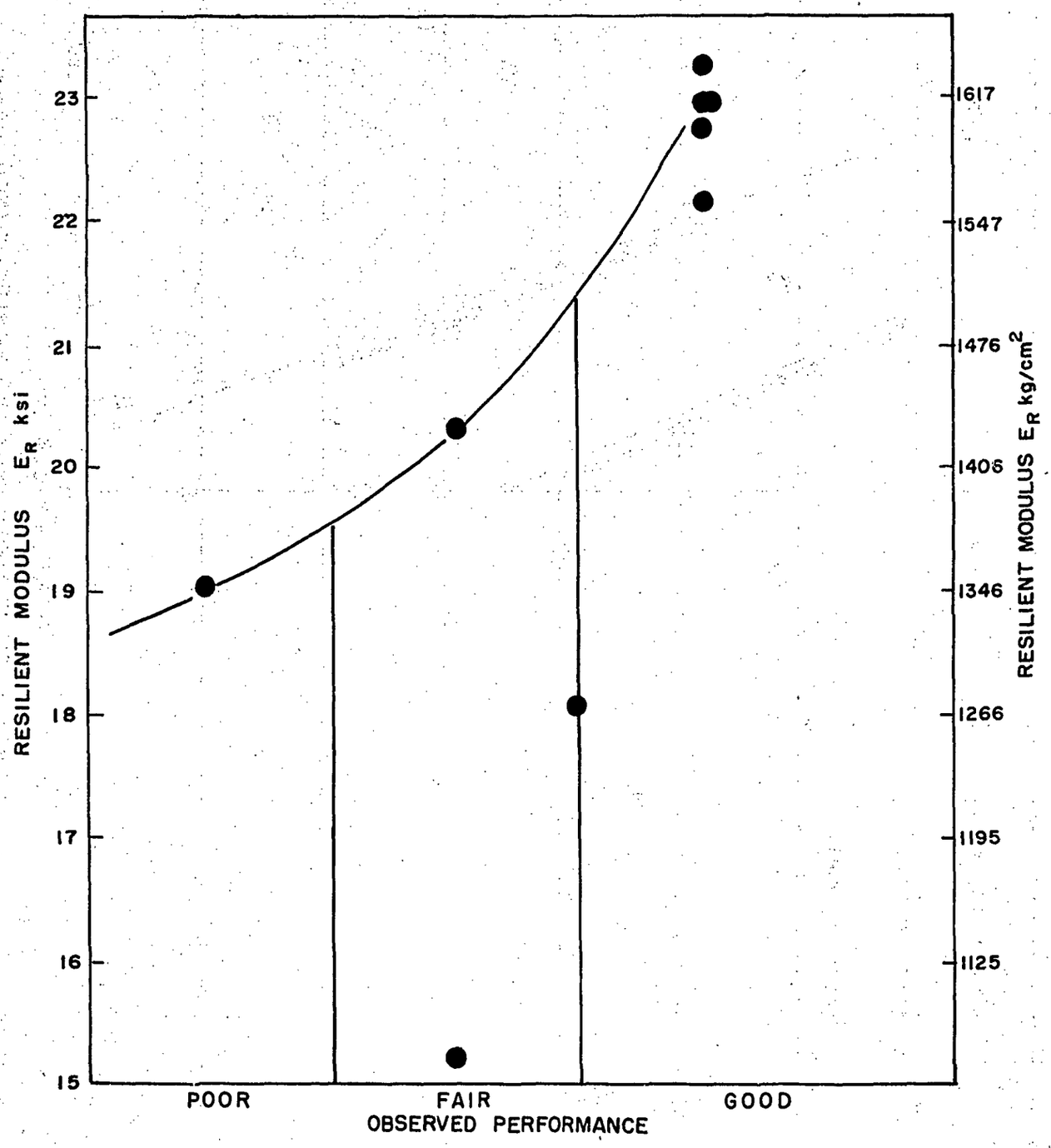


FIGURE A8.3 – CLASSIFICATION OF GRAVELS BASED ON RESILIENT MODULUS AFTER 100,000 LOAD APPLICATIONS

TABLE A9.1

Stabilization Test Data

Sample No.	G5-1																
Type of Stab		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	28.8	25.4	23.4	24.8	24.5	26.3	23.9	25.4	25.5	34.3	32.9	36.6	38.1	36.7	37.6	37.3	39.2
PL	18.4	19.8	16.8	20.7	18.7	22.5	19.6	22.0	22.5	24.7	22.8	27.1	29.7	29.1	29.6	31.0	32.6
PI	10.4	5.6	6.6	4.1	5.8	3.8	4.3	3.4	3.0	9.6	10.1	9.5	8.4	7.6	7.0	6.3	6.6
MDD pcf	136.3	134.2		133.0		132.1		130.6		135.5		136.5		136.3		137.0	
OMC-%	8.9	9.1		9.3		9.7		10.2		8.8		8.6		8.9		8.8	
4" ϕ qu psi	56	30		51		35		35		102		195		264		365	
2" ϕ qu psi	64	24.8	26.3	47.7	51.0	69.7	74.3	129.0	153.2	94.5	153.5	275.0	312.5	249.0	253.5	247.5	249.3
ST - psi	6.2	1.9	2.2	3.1	3.7	7.5	8.5	6.1	16.7	11.1	17.9	14.4	24.2	17.3	27.9	24.7	37.8
ST/qu	9.7	7.7	8.3	8.6	7.2	10.7	11.4	5.0	10.5	11.8	11.6	5.2	7.8	7.0	11.0%	10.0	15.2
CBR-%	30.2	64.8	55.0	77.8	89.0	98.5	122.0	91.1	152.0	135.0	400.0	183.0	416.0	219	424.3	246	467.0
SWELL-%	0.7%																
WDLOSS %	100	100	100	31.4	51.5	24.0	22.0	16.4	18.1	35.4	20.5	8.7	5.7	7.5	4.0	4.9	2.9

Stabilization Test Data

Sample No.	G9-3																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	33.0	27.0	28.5	28.3	28.4	26.0	30.0	29.1	29.7	33.3	34.5	35.4	36.3	35.6	36.7	36.4	34.4
PL	17.0	19.0	22.4	20.2	22.0	20.1	23.0	21.0	23.3	17.5	19.3	21.6	23.5	22.3	25.0	24.5	23.9
PI	16.0	8.0	6.1	8.1	6.4	7.9	7.0	8.1	6.4	15.8	15.2	13.8	12.8	13.3	11.7	11.9	10.5
MDD pcf	134.0	132.6		130.6		130.7		129.5		130.8		130.0		131.1		131.0	
OMC-%	9.1	10.5		9.5		12.4		11.5		10.0		11.2		11.2		10.6	
4" ϕ qu psi	93	150		175		149		130		315		315		298		372	
2" ϕ qu psi	76.0	92.6	75.5	103.0	113.7	182.6	202.4	147.0	198.0	118.6	112.0	207.8	334.2	277.7	396.9	380.0	328.0
ST psi	7.7	4.1	8.1	6.6	10.6	17.0	19.7	9.8	21.1	9.7	13.0	25.8	33.9	29.4	28.5	31.5	31.4
ST/qu	10.1	4.5	10.7	6.7	9.3	9.3	9.7	6.7	10.7	8.2	11.6	12.4	10.1	10.6	7.2	8.3	9.6
CBR-%	34.7	52.4	60.7	72.4	80.0	60.0	85.0	60.6	153.0	197.5	236.8	412.0	393.3	495.0	501.0	533.0	500.0
SWELL-%	0.5	0.5	0.1	0.2													
W.D.LOSS %	100	34.0	59.0	23.0	20.0	15.0	10.0	4.1	6.4	32.5	23.2	3.7	8.1	2.4	2.5	3.3	8.8

Table A9.1 (continued)

Stabilization Test Data

Sample No.	G1-14																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	23.4	22.8	23.7	23.6	23.6	22.9	23.7	24.0	23.3	31.6	32.4	35.3	34.5	35.7	34.5	34.1	35.1
PL	12.0	17.4	17.8	19.3	19.1	18.8	20.8	20.5	21.6	19.9	20.6	25.3	26.6	27.8	26.9	27.1	29.2
PI	11.4	5.4	5.9	4.3	4.5	4.1	2.9	3.5	1.7	11.7	11.8	10.0	7.9	7.9	7.6	7.0	5.9
MDD pcf	140.0	134.7		135.5		135.7		132.0		137.1		139.0		139.1		140.0	
OMC-%	7.2	8.6		8.3		8.4		9.0		7.4				7.5		7.3	
4" ϕ qu psi	23	119		232		186		139		334		343		530		500	
2" ϕ qu psi	63.2	99.7	144.7	124.3	206.0	173.0	214.0	164.0	208.7	156.7	222.9	272.7	445.7	551.3	563.0	510.0	785.3
ST psi	3.9	7.2	11.1	10.1	29.6	16.8	22.9	15.4	16.5	18.2	17.4	30.5	33.2	29.9	41.9	37.5	48.5
ST/qu	6.2	7.2	7.6	8.1	11.4	9.7	10.7	9.4	7.8	11.6	7.8	10.4	7.4	5.4	7.5	7.4	6.2
CBR-%	40.9	164.0	171.0	241.0	265	250.0	276	220.0	310	490	387	500	500	500+	500+	500+	500+
SWELL-%																	
W.D..WDLOSS %	100	26.8	38.4	21.1	10.3	8.7	8.7	5.0	9.5	15.2	6.7	5.5	9.3	3.0	4.2	1.7	1.7

Stabilization Test Data

Sample No.	G12-2																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	42.0	40.3	39.4	42.2	41.6	40.5	40.7	40.8	40.0	39.2	40.0	40.5	40.5	40.9	37.5	38.0	35.4
PL	20.3	29.6	29.9	31.8	30.5	32.4	31.2	33.4	33.1	23.4	24.5	26.1	25.8	26.0	23.9	25.9	26.1
PI	21.7	10.7	10.5	10.4	11.1	8.1	9.5	7.4	6.9	15.8	15.5	14.1	14.70	14.8	13.60	12.1	9.3
MDD pcf	135.0	132.0		133.0		129.5		127.0		130.5		131.0		133.0		133.5	
OMC-%	10.0	11.8		11.6		12.2		12.6		11.5		12.2		11.0		11.9	
4" ϕ qu psi	50	75		111		102		102		121		147		225		235	
2" ϕ qu psi	23.8	87.7	104.7	103.5	130.0	117.8	170.8	119.5	141.2	96.9	94.8	165.7	182.8	236.0	295.8	406	384.5
ST psi	2.8	7.7	8.6	8.8	17.9	10.4	18.6	9.6	150	106	10.5	15.6	19.5	36.6	36.2	55.5	59.5
ST/qu	11.7	8.8	8.3	8.5	13.7	8.9	10.9	8.1	10.6	11.0	11.1	9.4	10.7	15.5	12.2	13.7	15.4
CBR-%	33.0	40.0	80.6	71.2	89.0	126.5	111.0	106.5	120.5	65.2	130	131.0	207	238.0	330	231.0	340
SWELL-%	.79%																
W.D. WDLOSS %	100	100	100	51.0	47.0	33.0	38.1	23.4	24.1	100	100	11.9	4.5	7.6	8.0	4.0	2.7

Table A9.1 (continued)

Stabilization Test Data

Sample No	G2-1																
Type of Stab		Lime								Cement							
% Stab	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	44.0	43.0	42.0	41.6	43.0	41.7	42.6	42.3	42.4	42.6	44.0	43.3	44.4	46.3	46.6	45.6	48.3
PL	23.5	26.5	26.2	26.7	28.2	27.7	28.8	28.4	30.8	22.0	23.8	24.3	25.9	28.6	28.5	28.9	31.3
PI	20.5	16.5	15.8	14.9	14.8	14.0	13.8	13.9	11.6	20.6	20.2	19.0	18.5	17.7	18.1	16.7	17.0
MDD pcf	127.0	123.5		123.8		122.0		121.4		127.0		127.0		127.5		127.5	
OMC-%	12.5	12.7		12.8		14.4		14.0		11.7		12.7		12.6		12.4	
4" ϕ qu psi	37	162		181		186		167		150		278		390		397	
2" ϕ qu psi	186	187	184	242	240.0	287.7	272	244.2	200	212.3	259.7	247.7	346.3	344.7	522.5	401.7	590.0
ST psi	18.3	14.9	20.4	25.0	25.5	27.2	17.3	17.0	14.4	12.2	18.2	21.9	35.6	36.4	55.1	39.7	61.0
ST/qu psi	9.9	8.0	11.1	10.4	10.6	9.5	6.4	7.0	7.2	5.8	7.1	8.8	10.1	10.5	10.5	9.9	
CBR-%	11.6	39.7	50.8	39.8	63.4	65.6	100.0	61.3	70.2	103.3	163.7	333.3	333	453.3	400	500.0	488
SWELL-%																	
WDLOSS %	100	32.3	42.1	19.8	19.7	18.9	14.8	10.3	11.9	46.5	79.2	0.7	1.9	2.7	3.1	8.4	6.9

Stabilization Test Data

Sample No	08-1																
Type of Stab		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	38.0	36.0	37.0	37.0	34.2	37.9	34.6	36.0	34.5	36.5	36.4	35.6	37.6	24.9	36.9	35.6	36.8
PL	21.0	21.0	21.5	24.0	21.3	24.8	23.3	24.8	24.2	21.0	20.4	20.5	21.9	21.6	24.6	23.2	25.6
PI	17.0	15.0	15.5	13.0	12.9	13.1	11.3	11.2	10.3	15.5	16.0	15.1	15.7	13.3	12.3	12.4	11.2
MDD pcf	135.7	129.0		128.2		128.4		128.5		133.2		134.0		135.2		135.7	
OMC-%	8.2	10.8		11.2		10.7		10.5		9.0		9.4		9.2		9.0	
4" ϕ qu psi	60	84		75		84		139		204		265		562		575	
2" ϕ qu psi	38.6	49.5	53.3	55.2	62.1	77.6	74.6	63.8	67.2	89.6	176,	127.2	257.0	193.3	304	285.9	338.5
ST psi	4.0	4.6	4.5	6.2	6.5	9.9	4.0	5.2	5.0	10.9	27.3	13.4	31.0	18.4	31.9	35.2	346
ST/qu	10.4	9.1	8.5	11.2	10.4	12.2	5.4	8.2	7.4	12.2	15.5	10.5	12.1	9.5	10.5	12.2	10.2
CBR-%	12.0	49.0	59.1	55.3	87.8	72.3	128.0	96.2	128.0	237.0	261	333.0	363.5	311	362.5	384.0	403
SWELL-%	0.3																
WDLOSS	100	77.7	41.8	76.0	31.6	76.0	39.0	64.2	31.4	58.3	16.4	10.7	3.3	8.3	3.1	6.0	4.2

Table A9.1 (continued)

Stabilization Test Data

Sample No	G8-2																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	55.5	54.4	49.3	54.8	51.2	52.4	51.2	50.4	50.8	55.4	53.1	54.2	53.8	54.0	55.4	55.5	55.8
PL	32.2	32.6	29.3	36.9	35.2	36.8	36.4	37.8	39.6	32.0	30.6	31.7	31.2	33.0	35.0	35.5	36.3
PI	23.3	22.8	20.0	17.9	17.0	15.6	14.8	12.6	11.2	23.4	22.5	22.5	22.6	21.0	20.4	20.0	19.5
MDD pcf	125.2	123.6		123.5		122.0		122.0		125.0		125.8		125.2		126.0	
OMC-%	11.0	11.8		11.6		11.2		11.2		11.6		11.6		11.6		11.6	
4" ϕ qu psi	33	74		119		163		195		135		252		277		306	
2" ϕ qu psi	158.5	215.0	196.0	192.3	227.0	231.5	244.3	275.6	281.7	164.3	163.5	187.0	189	235	311.7	340.0	449.0
ST psi	16.7	20.7	18.0	17.4	19.3	21.6	22.1	25.4	28.4	17.0	16.9	22.1	22.0	19.0	44.0	27.0	41.9
ST/qu	10.5	9.6	9.2	9.1	8.5	9.4	9.1	9.3	10.1	10.4	10.3	11.8	11.6	8.1	14.1	8.0	9.3
CBR-%	20.8	91.0	93.1	98.0	100.7	101.6	108.4	105.6	123	60.1	149.0	102.7	354.0	463.3	386.6	483.3	437.0
SWELL-%	0.45																
WDLOSS %	100	100	55.6	40.2	22.4	37.7	19.5	27.6	16.9	100	100	19.1	15.6	9.1	12.6	8.8	5.1

Stabilization Test Data

Sample No	G8-3																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	54.5	49.0	50.4	49.7	45.0	45.1	41.2	42.8	40.9	53.8	53.5	57.7	54.6	57.5	55.8	56.0	55.2
PL	27.3	26.9	28.7	32.8	27.3	33.2	28.4	33.4	32.3	26.6	32.1	31.2	36.1	32.9	38.9	35.0	39.5
PI	27.2	22.1	21.7	16.9	17.7	11.9	12.8	9.4	8.6	27.2	21.4	26.5	18.5	24.6	16.9	21.0	15.7
MDD pcf	126.0	125.0		124.0		123.8		123.0		126.5		127.3		127.0		127.0	
OMC-%	10.2	11.1		11.7		11.0		11.2		11.1		11.4		11.3		11.3	
4" ϕ qu psi	30.0	79		149		130		120		51		65		111		270	
2" ϕ qu psi	133.3	117.5	132.0	121.5	157.2	127.5	128.5	108.8		90.6	130.5	163.0	386.2	220.6	296.7	319.7	349.0
ST psi	16.1	16.4	11.5	11.1	13.5	14.8	11.6	11.3		14.9	15.4	11.2	15.8	23.3	17.7	28.0	19.0
ST/qu	12.1	14.0	8.7	9.1	8.6	11.5	9.1	10.4		16.4	11.8	6.9	4.1	10.5	6.0	8.8	5.5
CBR-%	20.3	63.4	77.8	87.5	89.1	85.6	85.6	89.0	93.3	71.3	163.5	186.7	436	248	500	500	500
SWELL-%	0.4	0.2	0.1														
WDLOSS %		100	100	54.6	23.5	21.1	16.1	12.4	13.2	100	100	16.1	9.4	11.0	13.6	7.7	9.0

Table A9.1 (continued)

Stabilization Test Data

Sample No	G12-1																
Type of Stab.	Lime									Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	47.6	50.7	48.3	51.6	48.5	48.8	46.0	49.4	44.3	47.0	49.0	49.0	48.6	48.5	48.5	48.5	48.3
PL	25.8	32.8	31.4	35.2	33.4	33.5	32.8	36.0	34.0	25.9	28.8	31.2	31.3	32.7	33.6	34.5	34.1
PI	21.8	17.9	16.9	16.4	15.1	15.3	13.2	13.4	10.3	21.1	20.2	17.7	17.3	15.8	14.9	14.0	14.2
MDD pcf	127.5	124.0		124.5		125.0		122.6		129.0		127.5		128.5		129.0	
OMC-%	11.4	12.9		12.5		12.0		12.5		12.8		13.6		13.0		14.0	
4" ϕ qu psi	84	185		232		175		181		223		186		204		162	
2" ϕ qu psi	214.5	194	203.0	317.0	321.5	303.0	252	328	320	138.5	280	156.5	325	271.0	367	368.4	437
ST psi	21.4	18.0	16.5	29.5	29.3	28.0	31.8	28.9	38.6	18.0	14.4	6.0	27.9	22.7	30.3	25.8	38.8
ST/qu	10.0	9.3	8.2	9.3	9.1	8.9	12.6	8.8	12.1	13.0	5.2	10.2	8.6	8.4	8.3	7.0	8.9
CBR-%	17.3	36.0	48.9	47.7	54.5	51.8	65.1	79.7	86.1	50.0	107.0	113.3	224.6	187.3	284.0	273.3	330.0
SWELL-%	0.9	0.7	.03														
WDLOSS %	100	100.0	100.0	57.7	44.5	46.3	49.0	33.9	21.8	100	100	12.4	3.9	9.2	1.9	3.9	3.2

Stabilization Test Data

Sample No	IV-5																
Type of Stab.	Lime																
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	62.2	56.0	57.0	58.0	56.6	58.0	58.0	57.5	58.1								
PL	31.2	29.9	30.2	33.4	31.8	35.7	35.8	37.7	38.5								
PI	31.0	26.1	26.8	24.6	24.8	22.3	22.2	19.8	19.6								
MDD pcf	107.6	104.0		104.5		106.5		106.2									
OMC-%	18.7	20.8		22.2		20.0		20.4									
4" ϕ qu psi	162	139		149		139		149									
2" ϕ qu psi	87.7	84.0	133.8	103.3	237.0	177.7	255.2	210.0	363.8								
ST psi	9.2	4.3	6.2	10.6	13.9	18.4	14.2	11.0	16.3								
ST/qu	10.5	5.1	4.7	10.3	5.8	10.3	5.6	5.3	4.5								
CBR-%	22.5	75.5	83.4	126.0	150.0	116.7	270.0	138.4	290.0								
SWELL-%		.8%	0.2%	0.2%													
WDLOSS %	100	74.6	40.8	6.5	5.1	4.7	5.2	6.0	6.3								

Table A9.1 (continued)

Stabilization Test Data

Sample No	K-4																
Type of Stab.	Lime																
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	44.3	41.5	38.4	44.0	44.5	46.3	49.2	47.2	49.5								
PL	15.1	21.2	21.7	26.3	29.0	29.7	35.0	32.2	37.5								
PI	29.2	20.3	16.6	17.7	15.5	16.6	14.2	15.0	12.0								
MDD pcf	122.0	122.5		121.0		121.0		121.0									
OMC-%	12.2	12.0		12.4		13.0		13.5									
4" ϕ qu psi	130	163		181		218		195									
2" ϕ qu psi	99.5	165.5	253.0	217.3	237.3	239.0	353.3	451.5	393.3								
ST psi	15.2	25.2	28.3	30.3	33.8	25.8	381		72.3								
ST/qu	15.3	15.2	11.2	13.9	14.2	19.8	10.8		16.0								
CBR-%	37.8	67.1	90.4	111.5	138.5	127.0	166.7	170.0	149.0								
SWELL-%	0.7	0.2															
WDLOSS %																	

Stabilization Test Data

Sample No	U-4																
Type of Stab.	Lime																
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	46.8	41.4	41.8	40.7	39.4	39.8	40.4	36.4	36.8								
PL	15.1	24.4	25.6	26.8	25.6	26.8	28.1	29.4	28.6								
PI	31.7	17.0	16.2	13.9	13.7	11.7	12.3	7.0	8.2								
MDD pcf	125.3	124.6		124.2		121.9		122.0									
OMC-%	12.3	12.8		13.0		13.4		13.5									
4" ϕ qu psi	35	60		98		116		125									
2" ϕ qu psi	10.4	95.4	132.5	95.4	128.2	116.6	200.6	142.9	220.0								
ST psi	1.9	7.9	10.1	14.8	13.0	16.8	17.3	21.8	24.2								
ST/qu	18.2	8.3	7.6	15.5	10.1	14.4	8.7	15.2	11.0								
CBR-%	37.2	92.0	102.3	96.0	169.5	142.5	173.0	102.2	219.0								
SWELL-%	.04%	0.3%															
WDLOSS %																	

Table A9.1 (continued)

Stabilization Test Data

Sample No	CON-1																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	37.5	36.5	36.0	36.5	39.6	36.2	36.8	36.7	36.7	39.7	39.4	37.4	41.4	40.7	40.5	42.3	41.4
PL	18.5	20.9	19.6	21.4	24.3	22.0	22.6	23.6	23.1	22.3	23.3	23.4	25.3	27.7	28.1	31.0	30.4
PI	19.3	16.6	16.4	15.1	15.3	14.2	14.2	13.1	13.6	17.4	16.1	14.0	16.1	13.0	12.4	11.3	11.0
MDD pcf	118.8	117.5		117.0		116.5		116.0		120.5		120.5		120.0		120.5	
OMC-%	13.6	14.3		14.2		14.4		14.8		13.5		13.7		14.0		14.0	
4" ϕ qu psi	139	79		84		139		167		223		297		343		483	
2" ϕ qu psi	30.4	87.3	132.7	104.3	315.5	115.9	265.3	100.8	225.5	261.0	147.0	318.0	446.7	386.0	579.3	473.5	482.0
ST psi	2.6	6.2	4.6	6.1	34.6	13.8	29.1	7.3	26.7	33.4	15.8	32.9	17.8	41.2	43.6	44.3	45.4
ST/qu	8.6	7.1	3.5	5.9	11.0	11.9	11.0	7.3	11.8	12.0	9.3	10.4	4.0	10.6	7.5	9.4	9.4
CBR-%	6.8	88.6	59.4	132.0	142.7	112.0	158.0	75.6	83.3	32.0	32.1	151.8	238.0	116.8	315.0	179.0	400.0
SWELL-%	0.2%	0.1%															
WDLOSS %																	

Stabilization Test Data

Sample No	CON-2																
Type of Stab.		Lime								Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	21.0	22.6	20.5	21.8	22.5	21.4	22.0	22.6	22.0	25.0	25.0	28.6	30.0	30.2	31.5	30.3	34.0
PL	12.5	16.4	14.4	17.1	16.6	17.4	17.8	19.3	19.4	19.3	18.9	23.5	24.4	26.6	26.3	28.3	29.2
PI	8.5	6.2	6.1	4.7	5.9	4.0	4.2	3.3	2.6	5.7	6.1	5.1	5.6	3.6	5.2	2.0	4.8
MDD pcf	128.8	126.5		126.1		125.0		125.0		128.8		130.0		130.2		130.5	
OMC-%	9.2	10.5		10.2		10.3		10.3		9.5		9.5		9.5		9.6	
4" ϕ qu psi	56	88		93		130		93		195		344		427		500	
2" ϕ qu psi	55.6	56.4	77.9	75.4	87.2	73.5	156.9	98.1	118.1	168.0	207.5	275.0	400.5	348.7	443.0	724.3	629.3
ST psi	2.9	2.3	5.5	3.9	6.7	5.5	10.9	4.3	16.7	15.8	23.3	24.6	29.3	26.1	39.6	36.1	75.2
ST/qu	5.2	4.1	7.1	5.2	7.7	7.5	7.0	4.4	14.0	9.4	11.2	9.0	7.3	7.5	8.9	5.0	11.9
CBR-%	31.6	82.0	101.5	98.0	115.0	117.5	117.5	171.0	155.5	80.6	182.1	165.5	353.0	368.0	474.0	500.0	+500
SWELL-%	0.04																
WDLOSS %																	

Table A9.1 (continued)

Stabilization Test Data

Sample No	Tormabum																	
Type of Stab.		Lime								Cement								
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%		
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	30	26	26	26	28	33	34			28	34	44	39				
PL	15	16	15	17	18	20	20			18	20	23	26				
PI	15	10	11	9	10	13	14			10	14	21	15				
MDD pcf	130.2	130.2		127.7		127.6				131.3		131.1		132.2		132.8	
OMC-%	9.2	9.3		9.8		10.2				9.2		9.7		9.4		9.1	
4"ϕqu psi																	
2"ϕqu psi	47	74	107	89	96	91	95			145	171	207	298	252	503	422	875
ST psi	4	8	9	8	9	6	9			8	13	21	29	37	62	52	109
ST/qu	8.5	10.8	8.4	9.0	9.4	6.6	9.5			5.5	7.6	10.1	9.8	14.7	12.3	12.3	12.4
CBR-%	37	190	230	153	223	190	157			200	256	270	290	333	617	500	658
SWELL-%																	
WDLOSS %																	

Stabilization Test Data

Sample No	Freetown																	
Type of Stab.		Lime								Cement								
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%		
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	65	58	57	54	49	51	46			56	60	55	48				
PL	43	35	32	34	31	35	32			33	32	34	31				
PI	22	23	25	17	18	16	14			23	28	21	17				
MDD pcf	106.0	104.0		102.2		101.3				106.8		107.6		106.5		107.8	
OMC-%	25.0	26.3		25.1		26.2				24.8		24.6		25.3		24.5	
4"ϕqu psi																	
2"ϕqu psi	145	92	100	178	211	250	301			168	156	239	254	286	263	350	370
ST psi			12		21		27			26	20	36	21	31	27	45	54
ST/qu			12.0		9.9		9.0			15.4	12.8	15.0	8.3	10.8	10.3	12.8	14.4
CBR-%	53	64	58	158	169	218	200			66	95	132	137	163	173	312	266
SWELL-%	12																
WDLOSS %																	

Stabilization Test Data

Sample No.	Kabala																
Type of Stab.	Like									Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	73	51	50	51	48	54	43			52	58	51	51				
PL	39	34	34	35	33	36				34	34	36	33				
PI	34	17	16	16	15	18	NP			18	24	15	18				
MDD pcf	116.5	121.7		120.4		118.8				119.8		122.2		124.5		123.4	
OMC-%	15.0	15.0		15.8		15.0				15.3		15.5		14.3		13.4	
4" ϕ qu psi																	
2" ϕ qu psi	128	73	75	148	190	230	345			219		341		497		500	
ST psi		8	10	17	18	24	30			19		38		56		53	
ST/qu		10.9	13.3	11.5	95	10.4	8.7			8.7		11.1		11.3		10.6	
CBR-%	55	109	110	191	153	215	200			163	153	230	279	253	484	320	517
SWELL-%																	
W.D.LOSS %																	

Stabilization Test Data

Sample No.	Batkanu																
Type of Stab.	Lime									Cement							
% Stab.	0%	2%		4%		6%		8%		2%		4%		6%		8%	
Curing Time	0	7	28	7	28	7	28	7	28	7	28	7	28	7	28	7	28
LL	60	49	53	50	48	54	45			56	51	51	52				
PL	38	33	30	33	34	35	34			37	32	34	34				
PI	22	16	23	17	14	19	12			19	19	17	18				
MDD pcf	130.8	128.8		128.0		125.3				133.8		133.4		133.6		134.0	
OMC-%	13.6	15.5		15.1		16.2				13.7		14.2		13.5		12.8	
4" ϕ qu psi																	
2" ϕ qu ST psi	51	78		143		122				215		240		231		188	
ST/qu																	
CBR-%	93	109	100	169	179	198	176			138	200	220	326	432	457	700	300
SWELL-%																	
W.D.LOSS %																	

APPENDIX TO CHAPTER 10 (APPENDIX 10.1)

USING THE FHA SOIL PVC METER

(Reproduced from "Guide to Use of the FHA Soil PVC Meter, by G. F. Henry and M. C. Dragoo, Federal Housing Administration, FHA No. 595, Jan. 1965, Washington).

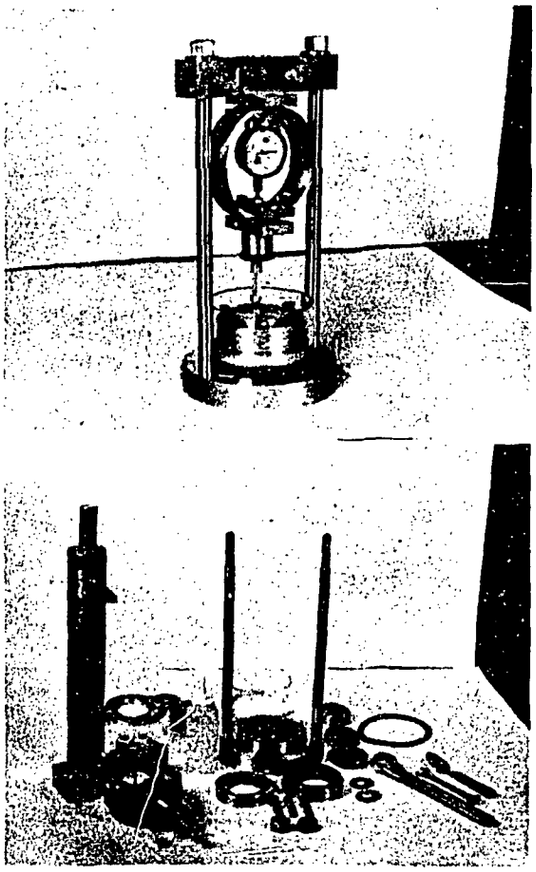


FIGURE A10.1 — PICTURES OF EQUIPMENT - PVC

A. General

The FHA Soil PVC Meter is used to perform a swell index test. This test is essentially a measurement of the pressure exerted by a sample of compacted soil when it swells against a restraining force after being wetted. The FHA Soil PVC Meter, in addition to yielding PVC values, can be used to estimate the plasticity index and shrinkage behavior of soils. These values are determined by comparing the results of the swell index test with appropriate values contained in Figures A10.2, A10.3, A10.4 and Table A10.1 in this guide and reading the corresponding extrapolations.

The following categories of PVC have been established:

PVC Rating	Category
Less than 2	Noncritical
2 to 4	Marginal
4 to 6	Critical
Greater than 6	Very critical

These ratings were established on the basis of the swelling and shrinking behavior of the soil.

B. Equipment

1. PVC Meter
2. Spacers, plate, and clamp for alternate compaction method
3. No. 10 Sieve
4. Teaspoon
5. Compaction Hammer and Sleeve
6. Two Dry Porous Stones
7. Knife, (preferably serated)
8. Straight Edge
9. Water in Squirt Bottle with Pointed End
10. Wrenches

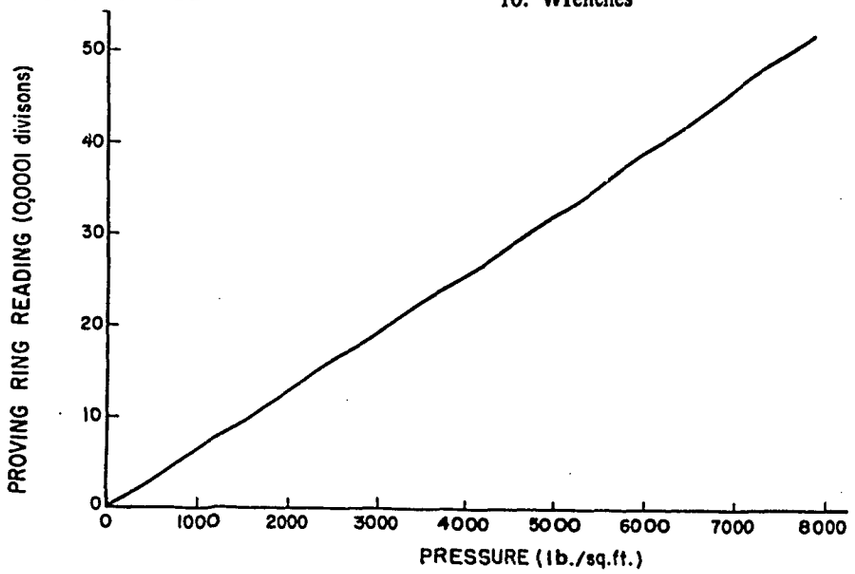


FIGURE A10.2 — PROVING RING CALIBRATION FROM "FHA SOIL PVC METER PUBLICATION" FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

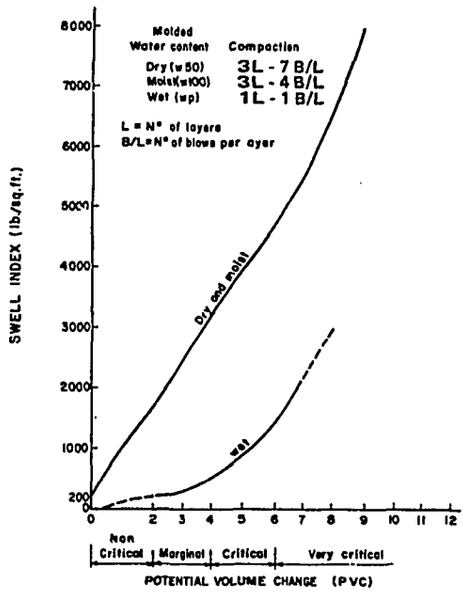


FIGURE A10.3 — SWELL INDEX vs POTENTIAL VOLUME CHANGE FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

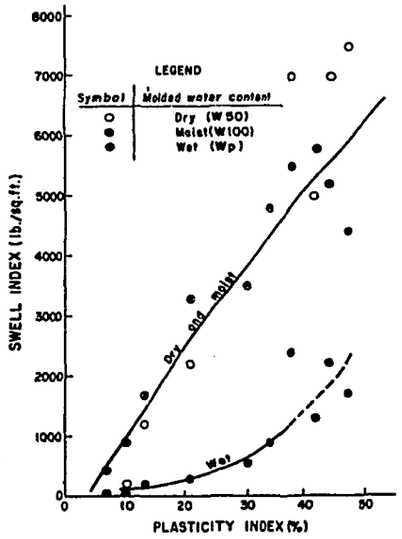


FIGURE A10.4 — SWELL INDEX vs PLASTICITY INDEX FROM FEDERAL HOUSING ADMINISTRATION PUBLICATION No. 701 (Lambe, 1960)

C. Preparation of Sample

For the test sample, take about a pint of soil from the soil layer in which the foundation member will rest. Although samples can be tested at three relative water contents (dry, moist, or wet), it is suggested that those being tested for FHA purposes be tested in the air dried condition only. The samples can be sufficiently air dried by breaking the soil into small lumps and leaving it in the sun for a few hours. The following procedures are for soil in the air dried condition. For information about soil in other conditions, see Lambe (1960).

D. Preparation for Compaction

Disassemble the PVC Meter with exception of the rods which can remain screwed into the base. Place proving ring and top bar where it will not be jarred during compaction. Wipe equipment with clean cloth.

E. Compaction

Definitions:

Compaction ring — largest ring; identified by letter "c" etched on outside periphery.

Spacer ring — smallest ring; identified by letter "s" etched on outside periphery.

1. To assemble meter for compaction, fit compaction ring on base so that "c" is backwards and at the top. Align bolt holes with those in base. Place spacer ring on compaction ring so that "s" is at the top (radial grooves are at top). Align bolt holes with those in base. Insert the 3 bolts through both the rings and the base and tighten firmly to base.

2. The soil sample is to be placed in the ring assembly in 3 layers of equal amounts. Each layer is to be compacted separately. Compaction is accomplished by use of the hammer, which is a tamping device encased in a metal sleeve.

3. Compact each layer of the sample in the following manner:

a. Place 3 heaping teaspoonsfull of sample in ring assembly and smooth lightly with hammer to firm up the surface before applying the blows (This reduces the amount of soil "jumping" out of the mold during compaction.). Place apparatus on a solid level floor.

b. Before each blow, lift sleeve 1/8 inch from soil and hold firmly against the inside of the spacer ring. *Make sure sleeve of hammer rests inside rings so that hammer does not damage them in falling.* Be sure to hold sleeve and hammer perpendicular and in line with supporting rods. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer. (See F for Alternate Compaction Method).

TABLE A10.1

Table for Converting Proving Ring Readings to PVC Category and Approximate Plasticity Index

Proving Ring Reading	Swell Index (#/SF)	PVC Category	Plasticity Index (%)
5	775	0.8	8.5
6	925	1.0	9.5
7	1,075	1.2	10.7
8	1,250	1.4	11.7
9	1,375	1.6	12.7
10	1,550	1.8	13.8
10.8	1,675	2.0	14.6
11	1,700	2.0	14.8
12	1,875	2.2	15.8
13	2,025	2.4	17.0
14	2,175	2.65	18.0
15	2,350	2.85	19.0
16	2,500	3.05	20.0
17	2,675	3.3	21.5
18	2,800	3.45	22.5
19	2,975	3.7	23.8
20	3,150	3.9	25.0
20.3	3,200	4.0	25.5
21	3,300	4.1	26.0
22	3,450	4.3	27.5
23	3,600	4.5	28.5
24	3,775	4.75	29.8
25	3,925	4.95	30.8
26	4,075	5.15	31.8
27	4,225	5.4	33.0
28	4,375	5.55	34.0
29	4,525	5.75	35.3
30	4,700	5.95	37.0
30.2	4,725	6.00	37.1
31	4,850	6.2	38.0
32	4,975	6.35	39.0
33	5,125	6.5	40.4
34	5,275	6.7	41.7
35	5,425	6.9	43.4
36	5,575	7.1	44.2
37	5,725	7.25	45.5
38	5,850	7.4	46.6
39	6,000	7.5	48.0
40	6,150	7.65	49.5
40.5	6,225	7.7	50.0

Prepared by the Architectural Section, Federal Housing Administration Insuring Office San Antonio, Texas.

4. At completion of the compaction of both the first and second layers, scratch the top surface of the layer with a knife to assure proper bond with the next layer. After compaction, the last layer should extend approximately 1/4 inch into the spacer ring. If it is significantly below this point, remove entire sample and recompact.

5. Put assembly on table and remove the 3 bolts. Rotate spacer ring (to break bond between ring and soil) and remove carefully from base. Remove compaction ring containing sample in same way. Do not tilt compaction ring or spill soil.

6. Trim top of the sample with a knife. Hold knife against the compaction ring at all times during trimming to avoid dislodging sample. Trim in a sawing motion taking off only a small amount of soil a at a time. Rotate the ring as you trim. Work from the edge toward the center. When sample is almost level, do final leveling by drawing a metal straight edge over sample.

7. The final surface of the soil sample should be firm and smooth. Any voids should be filled by pressing additional soil into them with the knife or spoon.

8. Clean soil from base and from all holes in rings and base. Remove soil in the groove of the spacer ring and from the holes in the spacer ring and the compaction ring with a toothpick or paperclip.

F. Alternate Compaction Method

1. After fitting rings to base as explained in E, paragraph 1, place one spacer on each rod, then set the plate on the spacers. Bolt these securely to the rods. Attach the clamp to the sleeve so that the sleeve extends about 1/4 inch insider the spacer ring. Place the soil sample in the ring assembly in the same manner as explained in E, paragraphs 1 and 3a.

2. Before each blow, turn the "foot" of the clamp so that it points in the direction of the spot to be compacted. The sleeve and hammer must be held perpendicular and in line with the supporting rods. To assure this, the sleeve should be held firmly against the inside of the plate and the spacer ring. Raise hammer to top of sleeve and let it fall free (not striking sides of sleeve). Space blows evenly over surface of sample by shifting hammer after each blow. Compact the first two layers with 7 blows each of the compaction hammer and the last with 8 blows. Repeat this process for each layer.

3. The remaining compaction process is the same as E, paragraphs 4 through 8.

G. Swelling

1. Place spacer ring on base with "s" (and radial grooves) on top. Align bolt holes with those on base. Place *thoroughly dry* porous stone in spacer ring. Move assembled base to edge of working table. Place thumb under base and other fingers over spacer ring and stone, holding them firmly in place. Turn base upside-down retaining firm hold on stone and spacer ring. Pick up compaction ring containing sample – trimmed side up – and place flush against porous stone in spacer ring aligning bolt holes in the two rings. Move compaction ring with as little disturbance of sample as possible. Turn base with rings, stone, and sample rightside-up. Bolt rings tightly to base.

2. Place a dry porous stone on top of sample inside compaction ring. Place the rubber O-ring on the base and screw the lucite container onto it tightly to insure water seal. Place metal cover on porous stone with the center identation at the top.

3. Place top bar with proving ring on the steel rods (Be sure that the adjustable rod which extends down from the proving ring dial does not strike the cover.). Add washers and nuts and tighten firmly.

4. Set proving ring dial to zero by moving the band around the dial. Tighten dial with the screw on band. Push up on proving ring dial to see that it appears to work properly. Turn adjustable rod exactly into the center of the indentation on top of the cover. *Be sure that the cover is centered exactly over the stone.* Tighten lock nut on adjustable rod firmly. Be sure adjustable rod does not stick in cover (receptable for adjustable rod may require slight enlargement). Turn adjustable rod until dial reads one division past zero. Tighten lock nut firmly again until adjustable rod has no play.

5. Record the time and the proving ring reading. Add water to sample by squeezing from squirt bottle into the holes located at the top of compaction ring until water level in lucite container has covered the spacer ring and tops of the bolts. (This procedure is used to reduce the amount of air entrapped in the ring assembly and thus insures that the sample has uniform access to water over its entire top and bottom surfaces.)

H. Reading

1. Allow soil to expand until completely stabilized or for a maximum of 2 hours, then read dial to obtain PVC swell index value. On the dial the number 1 equals 10 divisions, the number 2 equals 20, etc.

2. Next, find the number corresponding to the proving ring dial reading on Figure A10.2 and subtract the one division that registered on the dial prior to swell. Read horizontally to intersection with sloping line. From point of intersection, read downward to baseline which indicates pressure in lbs./sq.ft.

3. Take this figure to Figure A10.3. Find the number corresponding to it on left hand side of the chart. Read horizontally to intersection with the sloping line marked "Dry and Moist." From point of intersection, read downward to the baseline, which indicates PVC category.

4. Take the reading in lbs./sq.ft. to Figure A10.4 to determine the plasticity index.

5. It is also possible to obtain the approximate PVC category and plasticity index by taking the reading from the proving ring dial directly to Table A10.1.

REFERENCE

Lambe, T. W., 1960. The character and identification of expansive soils. Federal Housing Admin. Publ. No. 701, Washington.

APPENDIX TO CHAPTER 10 (APPENDIX 10.2)

SUGGESTED METHOD OF TEST FOR ONE-DIMENSIONAL EXPANSION AND UPLIFT PRESSURE OF CLAY SOILS

(Reproduced from "Special Procedures for Testing Soil and Rock for Engineering Purposes", 5th ed., ASTM Special Technical Publication 479, June 1970).

1. Scope

1.1 This method explains how to make expansion tests on undisturbed or compacted clay soil samples that have no particle sizes greater than 3/16 in. (passing the No. 4 standard ASTM sieve[3]). The test is made to determine (*1*) magnitude of volume change under load or no-load conditions, (*2*) rate of volume change, (*3*) influence of wetting on volume change, and (*4*) axial permeability of laterally confined soil under axial load or no-load during expansion. Saturation (no drainage) takes place axially. Permeant water is applied axially for determining the effect of saturation and permeability. The specimens prepared for this test may also be used to determine the vertical or volume shrinkage as the water content decreases. Total volume change for expansive soils is determined from expansion plus shrinkage values for different ranges of water content.

1.2 Expansion test data may be used to estimate the extent and rate of uplift in subgrades beneath structures or in structures formed from soils, and shrinkage tests may be used to estimate the volume changes which will occur in soils upon drying, provided that natural conditions and operating conditions are duplicated.

2. Significance

2.1 The expansion characteristics of a soil mass are influenced by a number of factors. Some of these are size and shape of the soil particles, water content, density, applied loadings, load history and mineralogical and chemical properties. Because of the difficulty in evaluating these individual factors, the volume-change properties cannot be predicted to any degree of accuracy unless laboratory tests are performed. When uplift problems are critical, it is important to test samples from the sites being considered.

2.2 The laboratory tests described herein are primarily intended for the study of soils having no particles larger than the No. 4 standard sieve size (3/16 in). If the test is made on the minus No. 4 fraction of soils containing gravel material (plus No. 4), some adjustment is required in any analysis. Gravel reduces volume change because it replaces the more active soil fraction.

3. Apparatus

3.1 *Consolidometer* – Conventional laboratory consolidometers are used for the expansion test. Consolidometers most used in the United States are of the fixed-ring and floating-ring types. Figure A10.5 illustrates the fixed-ring type. Either of these is suitable. Both types are available

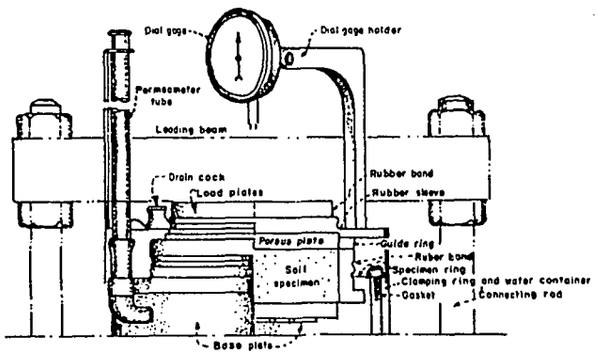


FIGURE A10.5 – FIXED-RING CONSOLIDOMETER

commercially. In the fixed-ring container, all specimen movement relative to the container is upward during expansion. In the floating-ring container, movement of the soil sample is from the top and bottom away from the center during expansion. The specimen containers for the fixed-ring consolidometer and the floating-ring consolidometer consist of brass or plastic rings, and other component parts. Sizes of container rings most commonly used vary between 4 1/4 in. diameter by 1 1/4 in. deep and 2 1/2 in. diameter by 3/4 in. deep, although other sizes are used. However, the diameter should be not less than 2 in. and the depth not greater than three tenths of the diameter, except that the depth must not be less than 3/4 in. for specimens of small diamter. Lesser depths introduce errors caused by the magnitude of surface disturbance, while large depths cause excessive side friction. For expansion tests the larger diameter consolidation rings are preferred as they restrain the soil action to a lesser degree. In a test using the floating-ring apparatus, the friction between the soil specimen and container is smaller than with the fixed-ring apparatus. On the other hand, the fixed-ring apparatus is more suitable for saturation purposes and when permeability data are required. Porous stones are required at the top and bottom of the specimen to allow application of water. The apparatus must allow vertical movement of the top porous stone for fixed-ring consolidometers, or vertical movement for top and bottom porous stones for floating-ring consolidometers, as expansion takes place. A ring gage machined to the height of the ring container to an accuracy of 0.001 in. is required; thus, the ring gage for 1 1/4 in. high specimens will have a height of 1.250 in. Measure the diameter of the specimen container ring to 0.001 in.

3.2 *Loading Device* – A suitable device for applying vertical load to the specimen is required. The loading device may be platform scales of 1000 to 3000-lb capacity mounted on a stand and equipped with a screw jack attached underneath the frame. The jack operates a yoke which extends up through the scale platform and over the specimen container resting on the platform. The yoke is forced up or down by operating the jack, thus applying or releasing load to the soil specimen. The desired applied pressure, which is measured on the scale beam, becomes fully effective when the beam is balanced.

3.2.1 Another satisfactory loading device utilizes weights and a system of levers for handling several tests simultaneously. Hydraulic-piston or bellows-type loading apparatus are also very satisfactory if they have adequate capacity, accuracy, and sensitivity for the work being performed. Apparatus such as described in ASTM Method D 2435, Test for One-Dimensional Consolidation Properties of Soils,[4] is satisfactory and may be used.

3.3 *Device for Cutting Undisturbed Specimens* – This apparatus consists of a cutting bit of the same diameter as the ring container of the consolidometer, a cutting stand with bit guide, and knives for trimming the soil. Wire saws of trimming lathes may be used if a uniform tight fit of the specimen to the container is obtained.

3.4 *Device for Preparation of Remolded Specimens* – Compacted soil specimens are prepared in the consolidometer ring container. In addition to the container, the apparatus consists of an extension collar about 4 in. in depth and of the same diameter as the container. A compaction hammer of the same type required in Method A of ASTM Method D 698, Test for Moisture-Density Relations of Soils, Using 5.5-lb Rammer and 12-in. Drop.[4]

4. Procedure-Expansion Test

4.1 *Preparation of Undisturbed Specimens* – Perform the tests on hand-cut cube samples or core samples of a size that will allow the cutting of approximately 1/2 in. of material from the sides of the consolidometer specimen. (Alternatively, obtain a core of a diameter exactly the same as the diameter of the consolidometer specimen container, and extrude the core directly into the container. This procedure is satisfactory provided that the sampling has been done without any sidewall disturbance and provided that the core specimen exactly fits the container. Place the undisturbed soil block or core on the cutting platform, fasten the cutting bit to the ring container, and place the assembly on the sample in alignment with the guide arms. With the cutting stand guiding the bit, trim the excess material with a knife close to the cutting edge of the bit, leaving very little material for the bit to shave off as it is pressed gently downward. (Other suitable procedures to accommodate guides for wire saws, trimming lathes, or extrusion devices may be used in conformance with the use of alternative apparatus and samples). In trimming the sample, be careful to minimize disturbance of the soil specimen and to assure an exact fit of the specimen to the consolidometer container. When sufficient specimen has been prepared so that it protrudes through the container ring, trim it flush with the surface of the container ring with a straightedge cutting tool. Place a glass plate on the smooth, flat cut surface of the specimen, and turn the container over. Remove the cutting bit, trim the specimen flush with the surface of the container ring, and cover it with a second glass plate to control evaporation until it is placed in the loading device.

4.2 *Preparation of Remolded Specimens* – Use about 2 lb of representative soil that has been properly moistened to the degree desired and processed free from lumps and from which particles or aggregations of particles retained by a 3/16 in. (No. 4) sieve have been excluded. Compact the specimen to the required wet bulk density after adding the required amount of water as follows: Place the extension collar on top of the container ring and fasten the bottom of the container ring to a baseplate. Weigh the exact quantity of the processed sample to give the desired wet density when compacted to a thickness ¼ in. greater than the thickness of the container ring. Compact the specimen to the desired thickness by the compaction hammer. Remove the extension collar and trim the excess material flush with the container ring surface with a straightedge cutting tool. Remove the ring and specimen from the baseplate and cover the specimen surfaces with glass plates until the specimen is placed in the loading device. If, after weighing and measuring the specimen and computing the wet density, as described below, the wet density is not within 1.0 lb/ft^3 of that required, repeat the preparation of the remolded specimen until the required accuracy is obtained.

4.3 *Calibration of Dial Gage for Height Measurements* Prior to filling the container ring with the soil specimen, place a ring gage in the specimen container with the same arrangement of porous plates and load plates to be used when testing the soil specimen. Place the assembly in the loading machine in the same position it will occupy during the test. After the apparatus has been assembled with the ring gage in place, apply a load equivalent to a pressure of 0.35 psi (or 0.025 kgf/cm^2) on the soil specimen. The dial reading at this time will be that for the exact height of the ring gage. Mark the parts of the apparatus so that they can be matched in the same position for the test.

4.4 *Initial Height and Weight of Soil Specimen* – Clean and weigh the specimen container ring and glass plates and weigh them to ± 0.01 g before the ring is filled. After filling and trimming is completed, weigh the soil specimen, ring, and glass plates to ± 0.01 g. Determine the weight of the soil specimen. Assemble the specimen container and place it in the loading device. If the specimen is not to be saturated at the beginning of the test, place a rubber sleeve around the protruding porous plates and load plates to prevent evaporation. Apply the small seating load of 0.35 psi (or 0.025 kgf/cm^2) to the specimen. By comparing the dial reading at this time with the dial reading obtained with the ring gage in place, determine the exact height of the specimen. Use this information to compute the initial volume of the specimen, the initial density, void ratio, water content, and degree of saturation. The true water content of the specimen will be determined when the total dry weight of the specimen is obtained at the end of the test.

4.5 *Saturation and Permeability Data* – To saturate the specimen attach the percolation tube standpipe, fill it with water, and wet the specimen. Take care to remove any air that may be entrapped in the system by slowly wetting the lower porous stone and draining the stone through the lower drain cock. After the specimen is wetted, fill the pan in which the consolidometer stands with water. After saturation has been completed, permeability readings can be taken at any time during the test by filling the percolation tube standpipe to an initial reading and allow the water to percolate through the specimen. Measure the amount of water flowing through the sample in a given time

4 *Annual Book of ASTM Standards*, Part 11.

by the drop in head.

4.6 *Expansion Test:*

4.6.1 *General Comments* – The expansion characteristics of an expansive-type soil vary with the loading history, so that it is necessary to perform a separate test or several specimens for each condition of loading at which *exact* expansion data are required. However, one procedure is to test only two specimens: (*1*) loaded-and-expanded, and (*2*) expanded-and-loaded. From these data, an estimate of expansion can be made for any load condition as shown by Curve C, Figure A10.6, in which Specimen No. 1 was loaded and expanded by saturation with water, (Curve B) and Specimen No. 2 was expanded by saturation with water and then loaded (Curve A).

4.6.2 *Loaded and Expanded Test* – To measure expansion characteristics where the soil specimen is saturated under full load and then allowed to expand, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 1, and secure initial dial readings. Then saturate the soil specimen as described in 4.5. (The permeameter tube head should be sufficiently low so that the specimen is not lifted.) As the specimen begins to expand, increase the load as required to hold the specimen at its original height. Then reduce the load to 1/2, 1/4, and 1/8 of the maximum load and finally to the seating load of 0.35 psi (or 0.025 kgf/cm^2) and measure the height with each load. Use a greater number of loadings if greater detail in the test curve is required. Maintain all loads for 24 h, or longer if needed, to obtain constant values of height. Remove the specimen from the ring container and weigh it immediately and again after drying to 105 C. From the water content, dry bulk density, and specific gravity of the specimen, calculate the volume of air and, assuming it to be the same as the volume of air following the determination of permeability, calculate the water content and degree of saturation.

4.6.3 *Expanded and Loaded Test* – To measure expansion characteristics where the soil is allowed to expand before loading, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 2, and secure initial dial gage readings. Then saturate the specimen as described in 4.5. Allow the specimen to expand under the seating load for 48 h or until expansion is complete. Load the specimen successively to 1/8, 1/4, 1/2 and 1 times the maximum load found in 4.6.2, to determine the reconsolidation characteristics of the soil. Use a greater number of loadings, if greater detail in the test curve is required. Follow the procedures specified in 4.6.2 for making loadings and all measurements and determinations.

4.6.4 *Individual Load-Expansion Test* – When it is desired to perform separate expansion tests for other conditions of loading apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to the specimen and measure the initial height. Then load the specimen to the desired loading, saturate the specimen as described in 4.5, and allow the specimen to expand under the applied load for 48 h, or until expansion is complete. Measure the height of the expanded specimen. Reduce the load to that of the seating load. Allow the height to become constant and measure; then remove the specimen from the ring and make the determination specified in 4.6.2.

5. Procedure – Shrinkage Test

5.1 *Specimen Preparation* – When measurements of shrinkage on drying are needed, prepare an additional specimen as described in 4.1 or 4.2. Cut this specimen from the same undisturbed soil sample as the expansion specimens, or remolded to the same bulk density and water content conditions as the expansion specimens. Place the specimen in the container ring, and measure the initial volume and height as described in 4.4. Determine the water content of the soil specimen by weighing unused portions of the original sample of which the specimen is a part, drying the material in an oven to 105 C, and reweighing it.

5.2 *Volume and Height Shrinkage Determinations* – To measure volume shrinkage, allow the specimen in the ring to dry in air completely or at least to the water content corresponding to the shrinkage limit (ASTM Method D 427, Test for Shrinkage Factors of Soils). After the specimen has been air-dried, remove it from the ring container, and obtain its volume by the mercury-displacement method.

5.2.1 To perform the mercury displacement measurement, place a glass cup with a smoothly ground top in an evaporating dish. Fill the cup to overflowing with mercury, and then remove the excess mercury by sliding a special glass plate with three prongs for holding the specimen in the mercury over the rim. Pour the excess mercury into the original container and replace the glass cup in the evaporating dish. Then immerse the air-dried soil specimen in the glass cup filled with mercury using the special glass plate over the glass cup to duplicate the initial mercury volume determination condition. (See Method D 427 for general scheme of test and equipment.) Transfer the displaced mercury into a graduated cylinder, and measure the volume. If the shrinkage specimen is cracked into separate parts, measure the volume of each part, and add the individual volumes to obtain the total. (A paper strip wrapped around the specimen side and held by a rubber band is effective in holding the specimen intact during handling.)

5.2.2 If the height of the air-dried specimen is desired, place the specimen and ring container in the loading machine. Apply the seating load of 0.35 psi (or 0.025 kgf/cm^2), and then read the dial gage.

6. Calculations

6.1 *Expansion Test Data* – Calculate the void ratio as follows:

$$e = \frac{\text{volume of solids}}{\text{volume of solids}} = \frac{h - h_0}{h_0}$$

Where:

e = void ratio,
h = height of the specimen, and
h_o = height of the solid material at zero void content

Calculate the expansion, as a percentage of the original height, as follow:

$$\Delta_e \text{ percent} = \frac{h_2 - h_1}{h_1} \times 100$$

where:

Δ = expansion in percentage of initial volume,
h_1 = initial height of the specimen, and
h_2 = height of the specimen under a specific load condition.

6.2 *Permeability Test Data* – Calculate the permeability rate by means of the following basic formula for the variable head permeameter:

$$k = \frac{A_p \times L_s}{A_s \times 12} \times \frac{1}{t} \ln \frac{H_i}{H_f}$$

where:

k = permeability rate, ft/year,
A_p = area of standpipe furnishing the percolation head, in.^{2},
A_s = area of the specimen, in.^{2},
L_s = length of the specimen, in.,
H_i = initial head, difference in head between headwater and tailwater, in.,
H_f = final head, difference in head between headwater and tailwater, in., and
t = elasped time, years.

6.3 *Shrinkage Test Data* – Calculate the volume shrinkage as a percentage of the initial volume as follows:

$$\Delta_e = \frac{V_i - V_d}{V_i} \times 100$$

where:

Δ_s = volume shrinkage in percentage of initial volume,
V_i = initial volume of specimen (height of specimen times area of ring container), and
V_d = volume of air-dried specimen from mercury displacement method.

Calculate the shrinkage in height as follows:

$$\Delta_{hs} = \frac{h_i - h_d}{h_i} \times 100$$

Where:

Δ_{hs} = height of shrinkage in percentage of initial height,
h_i = initial height of specimen, and
h_d = height of air-dried specimen.

6.3.1 To calculate the total percentage change in volume from "air-dry to saturated conditions," add the percentage shrinkage in volume on air drying Δ_s to the percentage expansion in volume on saturation Δ_e, as described in 6.1. This value is used as an indicator of total expansion but is based on initial conditions of density and water content. Since expansion volume data are determined for several conditions of loading, the total volume change can also be determined for several conditions of loading.

6.3.2 To calculate the total percentage change in height from saturated to air-dry conditions, add the percentage shrinkage in height Δ_{hs} to the percentage expansion Δ when the specimen is saturated under specific load conditions.

7. Plotting Test Data

7.1 *Expansion Test* – The test data may be plotted as shown on Fig. 10.6

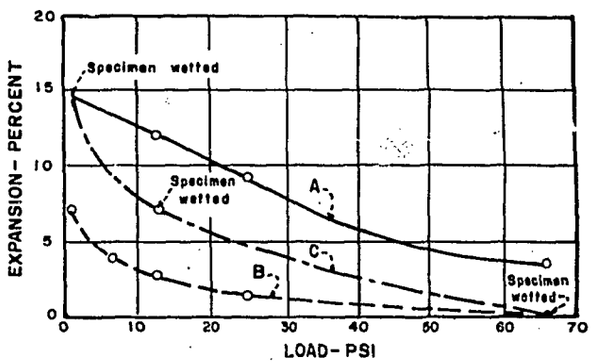


FIGURE A 10.6 LOAD-EXPANSION CURVES

FIGURE A10.6 – LOAD-EXPANSION CURVES

8. Reports

8.1 *Expansion Test* – Include the following information on the soil specimens tested in the report:

8.1.1 Identification of the sample (hole number, depth, location).

8.1.2 Description of the soil tested and size fraction of the total sample tested.

8.1.3 Type of sample tested (remolded or undisturbed; if undisturbed, describe the size and type, as extruded core, hand-cut, or other).

8.1.4 Initial moisture and density conditions and degree of saturation (if remolded, give the comparison to maximum density and optimum water content (see Methods D 698).

8.1.5 Type of consolidometer (fixed or floating ring, specimen size), and type of loading equipment.

8.1.6 A plot load versus volume change curves as in Figure A10.6. A plot of void ratio versus log of pressure curve may be plotted if desired.

8.1.7 A plot log of time versus deformation if desired.

8.1.8 Load and time versus volume change data in other forms if specifically requested.

8.1.9 Final water content, bulk dry density, and saturation degree data.

8.1.10 Permeability data and any other data specifically requested.

8.2 *Shrinkage Test* – For the report on shrinkage, include data on the decrease in volume from the initial to air-dried condition and, if desired, other information such as the total change in volume and total change in height. Report the load conditions under which the volume change measurements were obtained. Include also Items 8.1.1 through 8.1.5 and 8.1.9.

TABLE A12.1

Specifications for Rapid-Curing (RC) Liquid Asphalts-Asphalt Institute

Characteristics	AASHO Test Method	ASTM Test Method	Grades			
			RC-70	RC-250	RC-800	RC-3000
Kinematic Viscosity at 140°F., cs	T 201	D2170	[illegible]	250-500	800-1600	3000-6000
Flash Point (Tag. Open Cup), °F	T79	D1310	.....	80 +	80 +	80 +
Distillation						
Distillate (% by Volume of Total Distillate to 680°F.)						
To 374°F.			10 +	.....	.....	.....
To 437°F.	T 78	D402	50 +	35 +	15 +	.....
To 500°F.			70 +	60 +	45 +	25 +
To 600°F.			85 +	80 +	75 +	70 +
Residue from Distillation to 680°F., % Volume by Difference			55 +	65 +	75 +	80 +
Tests on Residue from Distillation						
Penetration, 77°F., 100 g., 5 sec.	T 49	D5	80-120	80-120	80-120	80-120
Ductility, 77°F., cms.	T 51	D113	100 +	100 +	100 +	100 +
Solubility in Carbon Tetrachloride, %[1]	T 44	D2042	99.5 +	99.5 +	99.5 +	99.5 +
Water, %	T 55	D95	0.2 –	0.2 –	0.2 –	0.2 –

General Requirement--The material shall not foam when heated to application temperature recommended by the Asphalt Institute.

TABLE A12.2

Specifications for Medium-Curing (MC) Liquid Asphalts-Asphalt Institute

Characteristics	AASHO Test Method	ASTM Test Method	Grades				
			MC-30	MC-70	MC-250	MC-800	MC-3000
Kinematic Viscosity at 140°F., cs.	T 201	D2170	30-60	70-140	250-500	800-1600	3000-6000
Flash Point (Tag. Open Cup), °F.[1]	T 91	D1310	100+	100 +	150 +	150 +	150 +
Distillation							
Distillate (% by Volume of Total Distillate to 680°F).							
To 437°F.			25-	20-	0-10	. . . . .	
To 500°F.	T 70	D402	40-70	20-60	15-55	35-	15-
To 600°F.			75-93	65-90	60-87	45-80	15-75
Residue from Distillation to 680°F.,% Volume by Difference			50 +	55 +	67 +	75 +	80 +
Tests on Residue from Distillation							
Penetration, 77°F., 100 g., 5 sec.	T 49	D5	120-250	120-250	120-250	120-250	120-250
Ductility, 77°F., cms[2]	T 51	D113	100 +	100 +	100 +	100 +	100 +
Salubility in Carbon Tetrachloride, %[3]	T 44	D2042	99.5 +	99.5 +	99.5 +	99.5 +	99.5 +
Water, %	T 55	D95	0.2 –	0.2 –	0.2 –	0.2 –	0.2 –

General Requirement - The material shall not foam when heated to application temperature recommended by the Asphalt Institute.

TABLE A12.3
Specifications for Anionic Emulsified Asphalts – Asphalt Institute

Characteristics	AASHO Test Method	ASTM Test Method	Grades				
			Rapid[2] Setting		Medium Setting	Slow[3] Setting	
			RS-1	RS-2	MS-2	SS-1	SS-1h
Tests on Emulsion[1]							
Furol Viscosity at 77°F., sec.			20-100	.....	100 +	20-100	20-100
Furol Viscosity at 122°F., sec.			.....	75-400	.....	.....	.....
Residue from Distillation, % by weight			57 +	62 +	62 +	57 +	57 +
Settlement, 5 days, % difference	T 59	D244	3 –	3 –	3 –	3 –	3 –
Demulsibility							
35ml. of 0.02 N $CaCl_2$, %			60 +	50 +	.....	.....	.....
50ml. of 0.10 N $CaCl_2$, %			.....	.....	30 –		
Sieve Test (Retained on N°20), %			0.10 –	0.10 –	0.10 –	0.10 –	0.10 –
Cement Mixing Test, %			.....	.....	.....	2.0 –	2.0 –
Test on Residue							
Penetration, 77°F, 100 g., 5 sec.	T 49	D5	100-200	100-200	100-200	100-200	40-90
Solubility in Carbon Tetrachloride, %	T 44	D4	97.5 +	97.5 +	87.5 +	97.5 +	97.5 +
Ductility 77°F., cms.	T 51	D113	40 +	40 +	40 +	40 +	40 +

Note: 1 To be used with calcareous materials only.
2 Rapid setting emulsions for surface treatments.
3 Slow setting emulsions for slurry-seal.

TABLE A12.4
Specifications for Cationic Emulsified Asphalts — Asphalt Institute

Characteristics	AASHO Test Method	ASTM Test Method	Grades					
			Rapid Setting		Medium Setting		Slow Setting[2]	
			RS-2K	RS-3K	SM-K	CM-K	SS-K	SS-Kh
Tests on Emulsion[1]								
Furol Viscosity at 77°F., sec	T 59	D244					20-100	20-100
Furol Viscosity at 122°F, sec	T 59	D244	20-100	100-400	50-500	50-500		
Residue from Distillation:								
Residue, % by weight	T 59	D244	60 +	65 +	60 +	65 +	57 +	57 +
Oil Distillate, % by volume of Emulsion	T 59	D244	5 –	5 –	20 –	12 –		
Settlement, 7 days, % differ.	T 59	D244	3 –	3 –	3 –	3 –	3 –	3 –
Sieve Test (Retained on N°20), %	T 59	D244	0.10 –	0.10 –	0.10 –	0.10 –	0.10 –	0.10 –
Aggregate Coating – Water Resistence Test		D244						
Dry Aggregate (Job), % Coated					80 +	80 +		
Wet Aggregate (Job), % Coated					60 +	60 +		
Cement Mixing Test, %	T 59	D244					2 –	2 –
Particle Charge Test	T 59A	D244	Positive	Positive	Positive	Positive		
pH	T 200	E70					6.7 –	6.7 –
Tests on Residue								
Penetration, 77°F. 100g. 5 sec.	T 49	D5	100-250	100-250	100-250	100-250	100-200	40-90
Solubility in Carbon Tetrachloride, %	T 44	D4	97.0 +	97.0 +	97.0 +	97.0 +	97.0 +	97.0 +
Ductility, 77°F., cm.	T 51	D113	40 +	40 +	40 +	40 +	40 +	40 +

Note: 1 Rapid setting emulsions for surface treatment.
2 Slow setting emulsions for Slurry-seal.

STANDARD SPECIFICATION FOR

PORTLAND CEMENT

AASHO DESIGNATION: M 85-70

SCOPE

1.1 This specification covers five types[1] of portland cement, as follows:

Type I. For use in general concrete construction when the special properties specified for types II, III, IV and V are not required.

Type II. For use in general concrete construction exposed to moderate sulfate action, or where moderate heat of hydration is required.

Type III. For use when high early strength is required.

Type IV. For use when a low heat of hydration is required. (Note 1).

Type V. For use when high sulfate resistance is required. (Note 1).

Basis of Purchase

2.1 The purchaser should specify the type or types desired. When no type is specified, the requirements of type I shall govern.

DEFINITION

3.1 Portland cement is the product obtained by pulverizing clinker consisting essentially of hydraulic calcium silicates to which no additions have been made subsequent to calcination other than water and/or untreated calcium sulfate, except that additions of other nondeleterious materials may be added at the option of the manufacturer in an amount not to exceed 0.1 per cent.

CHEMICAL LIMITS

4.1 Portland cement of each of the five types shown in Section 1 shall conform to the requirements prescribed in Table 1.

PHYSICAL REQUIREMENTS

5.1 Portland cement of each of the five types shown in Section 1 shall conform to the requirements prescribed in Table 2.

PACKAGING AND MARKING

6.1 When the cement is delivered in packages, the name and brand of the manufacturer and the type shall be plainly identified thereon, except that, in the case of type I cement, the type need not be identified. When the cement is delivered in bulk this information shall be contained in the shipping advices accompanying the shipment. A bag shall contain 94 lb. net. A barrel shall consist of 376 lb. net. All packages shall be in good condition at the time of inspection.

STORAGE

7.1 The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weathertight building that will protect the cement from dampness and minimize warehouse set.

INSPECTION

8.1 Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. The following periods from time of sampling shall be allowed for completion of testing:

1-day test	6 days
3-day test	8 days
7-day test	12 days
28-day test	33 days

REJECTION

9.1 The cement may be rejected if it fails to meet any of the requirements of this specification.

9.2 Cement remaining in bulk storage at the mill, prior to shipment, for a period greater than six months, or cement in bags in local storage in the hands of a vendor for more than three months, after completion of the tests may be retested and may be rejected if it fails to conform to any of the requirements of this specification.

9.3 Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

9.4 Cement failing to meet the test for soundness in the autoclave may be accepted if it passes a retest, using a new sample, at any time within 28 days thereafter. The provisional acceptance of the cement at the mill shall not deprive the purchaser of the right to reject on a retest of soundness at the time of delivery of the cement to the purchaser.

1 Attention is called to the fact that cements conforming to the requirements for type IV and type V are not usually carried in stock. In advance of specifying their use, purchasers or their representatives should determine whether these types of cement are, or can be made available.

TABLE 1
Chemical Requirements

	Type I	Type II	Type III	Type IV[1]	Type V[1]
Silicon dioxide (SiO_2), min, per cent	...	21.0	...	...	...
Aluminum oxide (Al_2O_3), max, per cent	...	6.0	...	...	...
Ferric oxide (Fe_2O_3), max, per cent	...	6.0	...	6.5	...
Magnesium oxide (MgO), max, per cent	5.0	5.0	5.0	5.0	4.0
Sulfur trioxide (SO_3), max, per cent:					
When $3CaO{\cdot}Al_2O_3$ is 8 per cent or less	3.0	3.0	3.5	2.3	2.3
When $3CaO{\cdot}Al_2O_3$ is more than 8 per cent	3.5	...	4.5	...	...
Loss on ignition, max, per cent	3.0	3.0	3.0	2.5	3.0
Insoluble residue, max, per cent	0.75	0.75	0.75	0.75	0.75
Sodium and potassium oxide ($Na_2O + 0.658\ K_2O$), max, per cent[2]	0.6	0.6	0.6	0.6	0.6
Tricalcium silicate ($3CaO.SiO_2$), [3] max, per cent	...	55	...	35	...
Dicalcium silicate ($2CaO{\cdot}SiO_2$), [3] min. per cent	...	...	...	40	...
Tricalcium aluminate ($3CaO{\cdot}Al_2O_3$), [3] max, per cent	15	8	15[4]	7	5
Tetracalcium aluminoferrite plus twice the tricalcium aluminate [3] ($4CaO{\cdot}Al_2O_3 + 2(3CaO{\cdot}Al_2O_3)$, or solid solution ($4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3 + 2CaO{\cdot}Fe_2O_3$), as applicable, max, per cent	...	...	...	...	20.0

1 See Note 1, p. 358

2 This requirement applies only when the Engineer specifies "low-alkali cement." Such cement should be specified only when alkali-reactive aggregates are to be used in the concrete. The maximum value of 0.6% may be reduced when the experience of the Engineer indicates that such action is desirable.

3 The expressing of chemical limitations by means of calculated assumed compounds does not necessarily mean that the oxides are actually or entirely present as such compounds.

When the ratio of percentages of aluminum oxide to ferric oxide is 0.64 or more, the percentages of tricalcium silicate, dicalcium silicate, tricalcium aluminate and tricalcium aluminoferrite shall be calculated from the chemical analysis as follows:

Tricalcium silicate = (4.071 x per cent CaO) − (7.600 x per cent SiO_2) − (6.718 x per cent Al_2O_3) − (1.430 x per cent Fe_2O_3) − (2.852 x per cent SO_3)

Dicalcium silicate = (2.867 x per cent SiO_2) − (0.7544 x per cent C_3S)

Tricalcium aluminate = (2.650 x per cent Al_2O_3) − (1.692 x per cent Fe_2O_3)

Tetracalcium aluminoferrite = 3.043 x per cent Fe_2O_3

When the alumina-ferric oxide ratio is less than 0.64, a calcium aluminoferrite solid solution (expressed as ss ($C_4AF + C_2F$) is formed. Contents of this solid solution and of tricalcium silicate shall be calculated by the sollowing formulas:

ss ($C_4AF + C_2F$) = (2.100 x per cent Al_2O_3) + (1.702 x per cent Fe_2O_3)

Tricalcium silicate = (4.071 x per cent CaO) − (7.600 x per cent SiO_2) − (4.479 x per cent Al_2O_3) − (2.859 x per cent Fe_2O_3) − (2.852 x per cent SO_3).

No tricalcium aluminate will be present in cements of this composition. Dicalcium silicate shall be calculated as previously shown.

In the calculation of C_3A, the values of Al_2O_3 and Fe_2O_3 determined to the nearest 0.01 per cent shall be used. Values for C_3A and for the sum of $C_4AF + 2C_3A$ shall be reported to the nearest 0.1 per cent. Values for other compounds shall be reported to the nearest 1 per cent.

4 When moderate sulfate resistance is required for type III cement, tricalcium aluminate may be limited to 8 per cent. When high sulfate resistance is required, the tricalcium aluminate may be limited to 5 per cent.

5 When moderate heat of hydration is required, a limit of 58 per cent on the total tricalcium silicate and tricalcium aluminate shall apply for type II cement.

METHODS OF SAMPLING AND TESTING

10.1 The sampling and testing of portland cement shall be in accordance with the following standard methods of the American Association of State Highway Officials:

Sampling . . . . . T 127
Chemical analysis . . . . . T 105

Note: Sodium and potassium oxides may be determined by either the flame photometry method outlined in T 105, Part II, or the gravimetric method outlined in T 105, Part I.

Fineness:
- Turbidimeter . . . . . T 98
- Air permeability . . . . . T 153

Soundness . . . . . T 107

Time of setting:
- Gillmore needles . . . . . T 154
- Vicat . . . . . T 131

Air content of mortar . . . . . T 137
Normal consistency . . . . . T 129
Tensile strength . . . . . T 132
Compressive strength . . . . . T 106
False set . . . . . T 186

TABLE 2
Physical Requirements

	Type I	Type II	Type III	Type IV (1)	Type V (1)
Fineness, specific surface, sq. cm per g. (alternate methods): (2)					
Turbidimeter test:					
Average value, min.	1,600	1,600	...	1,600	1,600
Min. value, any one sample	1,500	1,500	...	1,500	1,500
Average value, max.	2,200	2,200	...	2,200	2,200
Max. value, any one sample	2,300	2,300	...	2,300	2,300
Air permeability test:					
Average value, min.	2,800	2,800	...	2,800	2,800
Min. value, any one sample	2,600	2,600	...	2,600	2,600
Average value, max.	4,000	4,000	...	4,000	4,000
Max. value, any one sample	4,200	4,200	...	4,200	4,200
Soundness:					
Autoclave expansion, max., per cent	0.50	0.50	0.50	0.50	0.50
Time of setting (alternate methods): (3)					
Gillmore test:					
Initial set, min., not less than	60	60	60	60	60
Final set, hr., not more than	10	10	10	10	10
Vicat test (T131):					
Set, min., not less than	45	45	45	45	45
Air content: (5)					
Air content of mortar prepared and tested in accordance with Method T 137, max. per cent by volume	12.0	12.0	12.0	12.0	12.0
Tensile strength, psi: (4)					
The average tensile strength of not less than three standard mortar briquets, prepared in accordance with Method T 132, shall be equal to or higher than the values specified for the ages indicated below:					
1 day in moist air	...	...	275	...	...
1 day in moist air, 2 days in water	150	125	375	...	...
1 day in moist air, 6 days in water	275	250	...	175	250
1 day in moist air, 27 days in water	350	325	...	300	325
Compressive strength, psi.: (4)					
The average compressive strength of not less than three mortar cubes, prepared in accordance with Method T 106, shall be equal to or higher than the values specified for the ages indicated below:					
1 day in moist air	...	...	1,700	...	...
1 day in moist air, 2 days in water	1,200	1,000	3,000	...	...
1 day in moist air, 6 days in water	2,100	1,800	...	800	1,500
1 day in moist air, 27 days in water	3,500	3,500	...	2,000	3,000
False set, final penetration, min. per cent (6)	50	50	50	50	50

1 See Note 1, p. 358.

2 Either of the two alternate fineness methods may be used at the option of the test laboratory. However in case of dispute or when the sample fails to meet the requirements of the Blaine meter, the Wagner turbidimeter shall be used, and the requirements in Table 2 for this method shall govern.

3 The purchase should specify the type of setting time test required. In case he does not so specify, or in case of dispute, the requirement of the Vicat test only shall govern.

4 The purchaser shall specify the type of strength test desired. In case he does not so specify the requirements of the compressive strength test only shall govern. Unless otherwise specified, the strength tests for Types I and II cement will be made only at 3 and 7 days. The strength at any age shall be higher than the strength at the next preceding age.

5 Cement producing an air content of mortar between 12 and 16 percent may be accepted at the discretion of the purchaser when it is to be used in air-entraining concrete and the air content of this concrete is controlled at the mixer.

6 This requirement applies only when specifically requested.

STANDARD SPECIFICATION FOR

LIME FOR SOIL STABILIZATION

AASHO DESIGNATION: M 216-68

SCOPE

1.1 This specification covers two types, including 3 grades each, of lime to be used for soil stabilization as follows:

1.1.1 Type I. High calcium hydrated lime containing a maximum magnesium content, calculated as magnesium oxide, of 4 percent by weight. Compliance with chemical composition requirements shall be determined by use of AASHO T 219.

1.1.2 Type II. Magnesium or dolomitic lime containing magnesium, calculated as magnesium oxide, greater than 4 but no more than 36 percent by weigt. Compliance with chemical composition requirements shall be determined by use of ASTM C25-58 (Chemical Analysis of Limestone, Quicklime and Lime).[1]

Note: No attempt is made to present requirements for waste lime, commercial lime slurry, etc. Specification requirements for these materials could be better determined on a local basis.

BASIS OF PURCHASE

2.1 The purchaser and contractor should agree upon the grade desired before construction begins. When no grade is agreed upon the requirements of Grade A shall govern. When Grades B and C are used, plan quantities for Grade A shall be increased, without additional cost to the purchaser, as follows:

Type	Grade B	Grade C
I	6%	20%
II	2%	5%

CHEMICAL LIMITS

3.1 Type I lime when sampled and tested by procedures prescribed herein shall conform to the following requirements:

	Grade A	Grade B	Grade C
Min. Hydrate alkalinity, percent by weight $Ca(OH)^2$	90	85	75
Max. Unhydrated lime content, percent by weight CaO	7	8	9
Max. "Free Water" content, percent by weight H_2O	3	3	2

3.2 Type II lime when tested in accordance with ASTM Designation C 25 shall conform to the following requirements:

	Grade A	Grade B	Grade C
Calcium and magnesium oxide content of ignition residue, minimum percent[a]	98	96	94
Carbon dioxide (as received basis) maximum percent	3	4	8
Unhydrated calcium oxide (as received basis) maximum percent	7	8	9
"Free Walter" content, maximum percent	3	3	2

"Ignition to constant weight shall be performed utilizing an electric muffle furnace operating at 1000-1100°C (1800-2000 F).

PHYSICAL REQUIREMENTS

4.1 Type I and Type II lime shall conform to the following particle size requirements when tested according to AASHO T 192 (modified):

	Grade A	Grade B	Grade C
Max. residue retained on a No. 30 (0.600 mm) sieve, percent by weight	2	3	4
Max. residue retained on a No. 200 (0.075 mm) sieve, percent by weight	12	14	18

PACKAGING AND MARKING

5.1 When the lime is delivered in bags, the name and brand of the manufacturer and type shall be plainly identified thereon. A bag shall contain a nominal weight of 50 pounds (25 kg) and all bags shall be in good condition at time of inspection. When lime is delivered in bulk, information regarding type and manufacturer shall be contained in the invoice accompanying the shipment.

INSPECTION

6.1 Every facility shall be provided the purchaser for careful sampling and inspection of the lime at either the plant or at the site of the work as may be specified by the purchaser.

REJECTION

7.1 The lime shall be rejected if it fails to meet any of the requirements of these specifications.

7.2 Lime remaining in storage that becomes difficult to handle or that fails to conform to the requirements of this specification when resampled and tested, or both, may be rejected.

7.3 In the case of bag lime, bags varying more than 5 percent from the specified weight may be rejected. If the average weight of bags in any shipment as shown by

weighing 50 bags taken at random is less than that specified, the entire shipment may be rejected.

METHODS OF SAMPLING AND TESTING

8.1 The sampling and testing of hydrated lime shall be in accordance with the following standard methods of AASHO or ASTM:

Sampling – AASHO T 218
Chemical Analysis for Type I lime – AASHO T 219
Chemical Analysis for Type II lime – ASTM C 25[a]
Particle Size Analysis – AASHO T 219

a Except that Section 2, "Samples for Analysis" is excluded.

TABLE A 12.5
SPECIFICATIONS FOR STABILIZATION
A. ASPHALT STABILIZATION

Sieve Size	% Passing
1"	100
4	50-100
30	25-100
100	10-65
200	5-25
by weight of total mix	3-15% asphalt

The above represents the normal gradation limits of materials recommended for stabilization with asphalt. However, by proper selection of types and grades of asphalt materials soils outside of these limits can be stabilized.

B. CEMENT STABILIZATION

AASHO Soil Group	Range of Cement Percent by Weight
A-1-a	3-5
A-1-b	5-8
A-2-4	5-8
A-2-5	5-8
A-2-6	5-9
A-2-7	5-9
A-3	7-11
A-4	7-12
A-5	8-13
A-6	9-15
A-7	10-16

The pasticity of the material governs the ability of mixing the cement into the soil. It is generally recognized that the mixing operation is one if not the most important phase of a cement stabilization project. It is recommended that materials displaying plastic index above 10 be pre-treated with lime prior to mixing the cement into the material. Cement has been known to stabilize heavy clays when used in combination with a pre-treatment of lime.

C. LIME STABILIZATION

Lime stabilization is suitable for a broad range of soil types. Lime can be used in combination with a pozzolan for stabilization of sandy soils. Lime does not generally react with silts. The best criteria to use in establishing the suitability of lime as a stabilizing agent is the changes which occur due to the introduction of lime into the soil. The principal changes occurring during lime stabilization are as follows:

1. Reduction in PI and volume change.
2. Flocculation of clay particles, making the soil more friable.
3. Increase in optimum moisture content, thereby permitting compaction under wetter conditions; soils dry out more rapidly.
4. Increase in strength and stability through cementing action.
5. Resistence to water absorption and capillary rise.

SUGGESTED SPECIFICATIONS FOR MULTI-LAYER SURFACE TREATMENTS IN TROPICAL CLIMATES USING LIQUID ASPHALTS (AFTER PULLEN AND PREVOST 1969)

1. *Prime coat with a single seal coat*

Prime M. C. 30	Spray at 0.12 to 0.20 glns/sq.yd.
Seal coat: 80/100 Pen or M.C. 3000	Spray at 0.18 to 0.22 glns/sq.yd
Cover aggregate	$\frac{1}{2}$ inch nominal size, spread at 90 sq. yds/ton.

The Prime coat must be left to dry before applying the seal coat and the rate of application will depend on the condition of the road surface and the speed of drying to a uniform film.

2. *Prime coat with two coat surface treatment*

Prime M.C. 30	Spray at 0.12 to 0.20 glns/sq.yd.
1st Seal: 80/100 Pen. or M. C. 3000	Spray at 0.18 to 0.22 glns/sq.yd.
Cover aggregate	3/4 inch nominal size at 70 sq.yd/ton
2nd Seal: 80/100 Pen. or M. C. 3000	Spray at 0.22 to 0.26 glns/sq.yd.
Cover aggregate	3/8 inch nominal size at 110 sq.yd/ton

3. *Prime coat and two coat surface treatments* with second coat applied after road has been open to traffic for several months.

Prime: M. C. 30	Spray at 0.12 to 0.20 glns/sq.yd.
1st Seal: 80/100 Pen. or M. C. 3000	Spray at 0.15 to 0.20 glns/sq.yd.
Cover aggregate	3/8 inch nominal size at 110 sq.yd/ton.
2nd Seal (after several months) 80/100 Pen. or M. C. 3000	Spray at 0.20 to 0.24 glns/sq.yd.
Cover aggregate	1/2 inch nominal size at 90 sq.yds/ton.

In East Africa the following specifications have been noted.

4. *No Prime Coat*

1st Seal: 180/200 Pen or R. C. 2	With 3/8 inch aggregate 0.20 to 0.22 glns/sq.yd. With 1/2 inch aggregate 0.22 to 0.25 glns/sq.yd.
2nd Seal: 180/200 Pen or 80/100 Pen	0.33 glns/sq.yd. with 3/4 inch aggregate.
Cover aggregate application:	
3/8 inch nominal size:	90 to 140 sq.yd/cubic yd.
1/2 inch nominal size:	65 to 95 sq.yd/cubic yd.
3/4 inch nominal size:	40 to 65 sq.yd/cubic yd.

5. *Prime and Triple Seal*

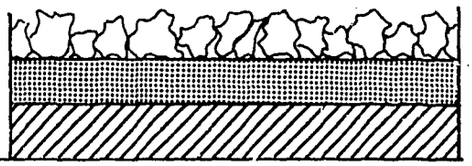
Prime coat: M. C. 2	0.10 to 0.12 glns/sq. yd (left for seven days)
1st Seal Coat: 80/100 Pen:	0.22 glns/sq. yd. with 3/4 inch aggregate at 55 sq.yds/cu yd.
2nd Seal Coat: 80/100 Pen:	0.28 glns/sq. yd. with 1/2 inch aggregate at 85 sq.yds/cu yd.
3rd Seal Coat: 80/100 Pen:	0.10 glns/ sq.yd with 1/4 inch to dust at 300 sq.yds/cu yds.

AGGREGATE – 12.5/18 mm 14 L/m²

EMULSION 60 % 2.100 kg /m²

OR

EMULSION 70 % 1.800 kg/m²

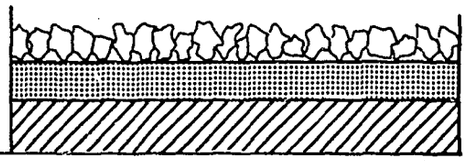


AGGREGATE – 8/12.5 mm 9 L/m²

EMULSION 60 % 1.500 kg/m²

OR

EMULSION 70 % 1.300 kg m²

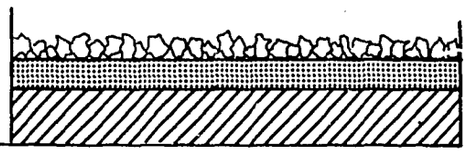


AGGREGATE – 5/8 mm 6 L/m²

EMULSION 60 % 1.200 kg/m²

OR

EMULSION 70 % 1.050 kg /m²



AGGREGATE 2.5/5 mm 4 L/m²

EMULSION 60 % 0.900 kg/m²

OR

EMULSION 70 % 0.800 kg/m²

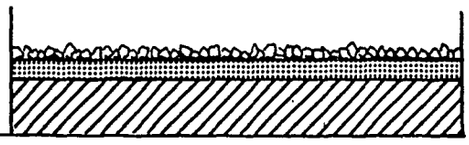


FIGURE A12.1 – SINGLE PASS SURFACE TREATMENT (after Pullen and Prevost, 1969)

TABLE A12.6
Conversions for Figure A12.1

12.5/18 mm	#	1/2 / 0.709 in.
14 1/m²	#	2.576 Imp. Gal. (3.08 US gal.) per sq. yd.
2.1 kg/m²	#	3.870 lbs/sq. yd.
1.8 kg/m²	#	3.317 lbs/sq. yd.
8/12.5 mm	#	0.315/ 1/2 in.
9 1/m²	#	1.656 Imp. Gal. (2 US gal.) per sq. yd.
1.5 kg/m²	#	2.765 lbs/sq. yd.
1.3 kg/m²	#	2.400 lbs/sq. yd.
5/8 mm	#	0.200/0.315 in.
6 1/m²	#	1.104 Imp. Gal. (1.320 US gal.) per sq. yd.
1.2 kg/m²	#	2.210 lbs/sq. yd.
1.05 kg/m²	#	1.935 lbs/sq. yd.
2.5/5 mm	#	0.100/0.200 in.
4 1/m²	#	0.736 Imp. Gal. (0.880 US gal.) per sq. yd.
0.9 kg/m²	#	1.660 lbs./sq. yd.
0.8 kg/m²	#	1.475 lbs./sq. yd.

US GALLON x 4.951 x 10^{-3} = Cubic Yards

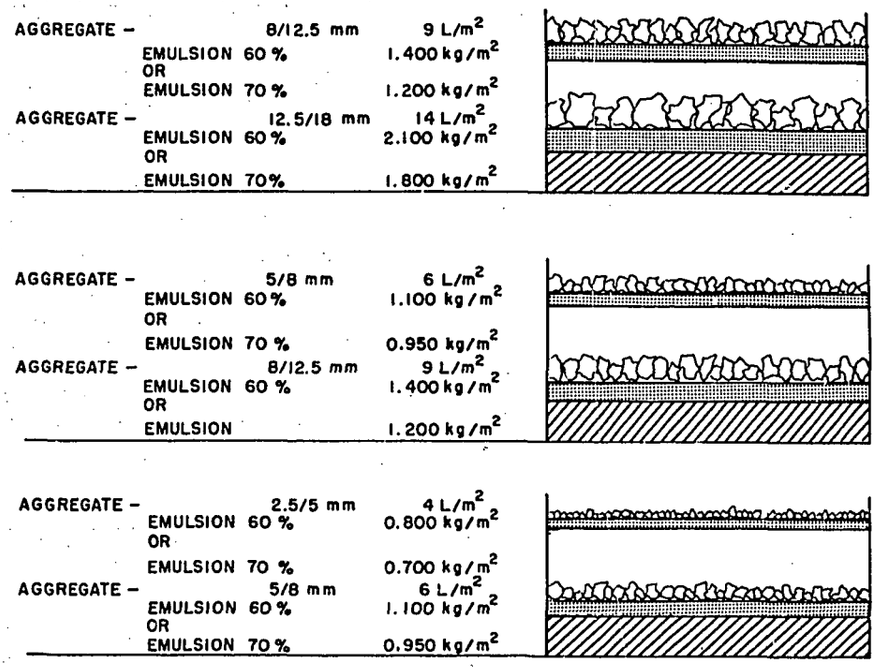


FIGURE A12.2 – DOUBLE SURFACE TREATMENT (after Pullen and Prevost, 1969)

TABLE A12.7
Conversions for Figure A12.2

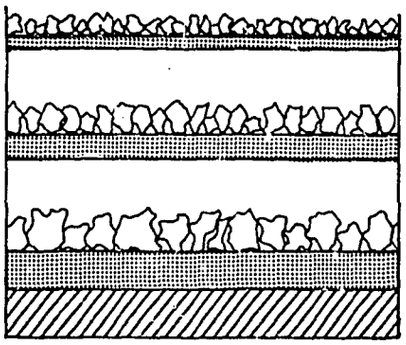
8/12.5 mm	#	0.315 / 1/2 in.
9 1/m²	#	1.656 Imp. Gal. (2 US gal.) per sq. yd.
1.4 kg/m²	#	2.580 lbs/sq. yd.
1.2 kg/m²	#	2.210 lbs/sq. yd.
12.5/18 mm	#	1/2 / 0 709 in.
14 1/m²	#	2.576 Imp. Gal. (3.08 US gal.) per sq. yd.
2.1 kg/m²	#	3.870 lbs/sq. yd.
1.8 kg/m²	#	3.317 lbs/sq. yd.
5/8 mm	#	0.200/0.315 in.
6 1/m²	#	1.104 Imp. Gal. (1.320 US gal.) per sq. yd.
1.1 kg/m²	#	2.030 lbs/sq. yd.
0.950 kg/m²	#	1.750 lb / sq. yd.
8/12.5 mm	#	0.315 / 1/2 in.
9 1/m²	#	1.656 Imp. Gal. (2 US gal.) per sq. yd.
1.4 kg/m²	#	2.580 lbs/sq. yd.
1.2 kg/m²	#	2.212 lbs/sq. yd.
2.5/5 mm	#	0.100/0.200 in.
4 1/m²	#	0.736 Imp. Gal. (0.880 US gal.) per sq. yd.
0.8 kg/m²	#	1.475 lb/sq. yd.
0.7 kg/m²	#	1.290 lb/sq. yd.
5/8 mm	#	0.200/0.315 in.
6 1/m²	#	1.104 Imp. Gal. (1.320 US gal.) per sq. yd.
1.1 kg/m²	#	2.030 lbs/sq. yd.
0.950 kg/m²	#	1.750 lb/sq. yd.

US GALLON x 4.951 x 10^{-3} = Cubic Yards

AGGREGATE - 5/8 mm 6 L/m²
EMULSION 60% 0.800 kg/m²
OR
EMULSION 70% 0.700 kg/m²

AGGREGATE 8/12.5 mm 9 L/m²
EMULSIONS 60% 1.300 kg/m²
OR
EMULSION 70% 1.150 kg/m²

AGGREGATE - 12.5/18 mm 14 L/m²
EMULSION 60% 2.000 kg/m²
OR
EMULSION 70% 1.700 kg/m²



ALTERNATE

AGGREGATE - 5/8 mm 6 L/m²
EMULSION 60% 1.600 kg/m²
OR
EMULSION 70% 1.400 kg/m²

AGGREGATE - 8/12.5 mm 9 L/m²
EMULSION 60% 1.400 kg/m²
OR
EMULSION 70% 1.200 kg/m²

AGGREGATE 12.5/18 mm 14 L/m²
EMULSION 60% 1.100 kg/m²
OR
EMULSION 70% 0.950 kg/m²

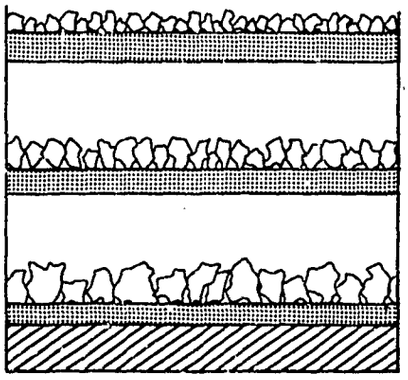


FIGURE A12.3 – TRIPLE SURFACE TREATMENT (after Pullen and Prevost, 1969)

TABLE A12.8
Conversions for Figure A12.3

5/8 mm	#	0.200/0.315 in.
6 1/m²	#	1.104 Imp. Gal. (1.320 US gal.) per sq. yd.
0.8 kg/m²	#	1.475 lb/sq. yd.
0.7 kg/m²	#	1.290 lb/sq. yd.
8/12.5 mm	#	0.315 / 1/2 in.
9 1/m²	#	1.656 Imp. Gal. (2 US gal.) per sq. yd.
1.3 kg/m²	#	2.400 lbs/sq. yd.
1.150 kg/m²	#	2.120 lbs/sq. yd.
12.5/18 mm	#	1/2 / 0.709 in.
14 1/m²	#	2.576 Imp. Gal. (3.08 US Gal.) per sq. yd.
2 kg/m²	#	3.685 lbs/sq. yd.
1.7 kg/m²	#	3.135 lbs/sq. yd.
5/8 mm	#	0.200/0.315 in.
6 1/m²	#	1.104 Imp. Gal. (1.320 US gal.) per sq. yd.
1.6 kg/m²	#	2.950 lbs/ sq. yd.
1.4 kg/m²	#	2.580 lbs/sq. yd.
8/12.5 mm	#	0.315 / 1/2 in.
9 1/m²	#	1.656 Imp. Gal. (2 US gal.) per sq. yd.
1.4 kg/m²	#	2.580 lbs/sq. yd.
1.2 kg/m²	#	2.210 lbs/sq. yd.
12.5/18 mm	#	1/2 /0.709 in.
14 1/m²	#	2.576 Imp. Gal. (3.08 US gal.) per sq. yd.
1.1 kg/m²	#	2.030 lbs/sq. yd.
0.950 kg/m²	#	1.750 lbs/sq. yd.

US GALLON x 4.951×10^{-3} = Cubic Yards

TABLE A12.9

Spreading Temperatures for Cutback Asphalt (After Pullen and Prevost 1969)

U. S. Grade	Kinematic Viscosity	Recommended Spraying Temperature
Cutbacks RC-MC	Centistokes at 140°C	Ranges °F and °C Low Viscosity / High Viscosity
30	30 – 60	83–125°F/118–158°F 28– 52°C/ 48– 70°C
70	70 – 140	120–165°F/145–190°F 49– 74°C/ 63– 88°C
250	250 – 500	162–208°F/183–234°F 72– 98°C/ 84–112°C
800	800 – 1600	197-247°F/218–268°F 92–119°C/103–131°C
3000	3000 – 6000	233–283°F/249–300°F 112–139°C/120–149°C

TABLE A12.10

Spreading Temperatures for Emulsified Asphalts Recommended by the Asphalt Institute

ANIONIC EMULSIONS		
RS-1	75 – 130°F	24 – 55°C
RS-2	110 – 160°F	43 – 71°C
MS-2	100 – 160°F	38 – 71°C
SS-1	75 – 130°F	24 – 55°C
SS-1 h	75 – 130°F	24 – 55°C
CATIONIC EMULSIONS		
RS-2K	75 – 130°F	24 – 55°C
RS-3K	110 – 160°F	43 – 71°C
CM-K	100 – 160°F	38 – 71°C
SM-K	100 – 160°F	38 – 71°C
SS-K	75 – 130°F	24 – 55°C
SS-K h	75 – 130°F	24 – 55°C

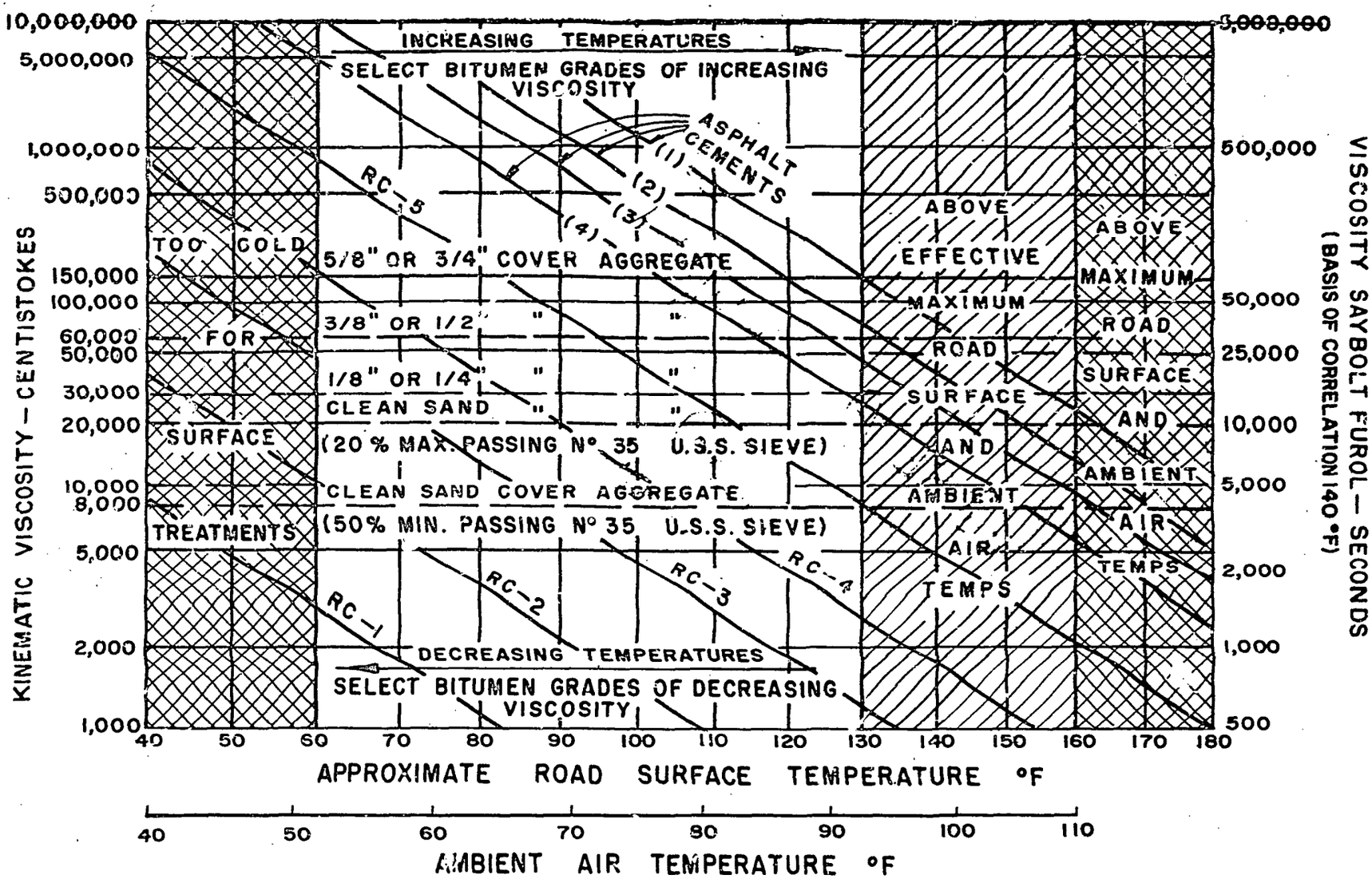


FIGURE A12.4a – SELECTION BITUMINOUS BINDER FOR SURFACE TREATMENTS OR SEAL COATS IN TROPICAL CLIMATES DETERMINED BY SURFACE TEMPERATURES AND SIZE OF COVER AGGREGATE *(after Pullen and Prevost, 1969)*

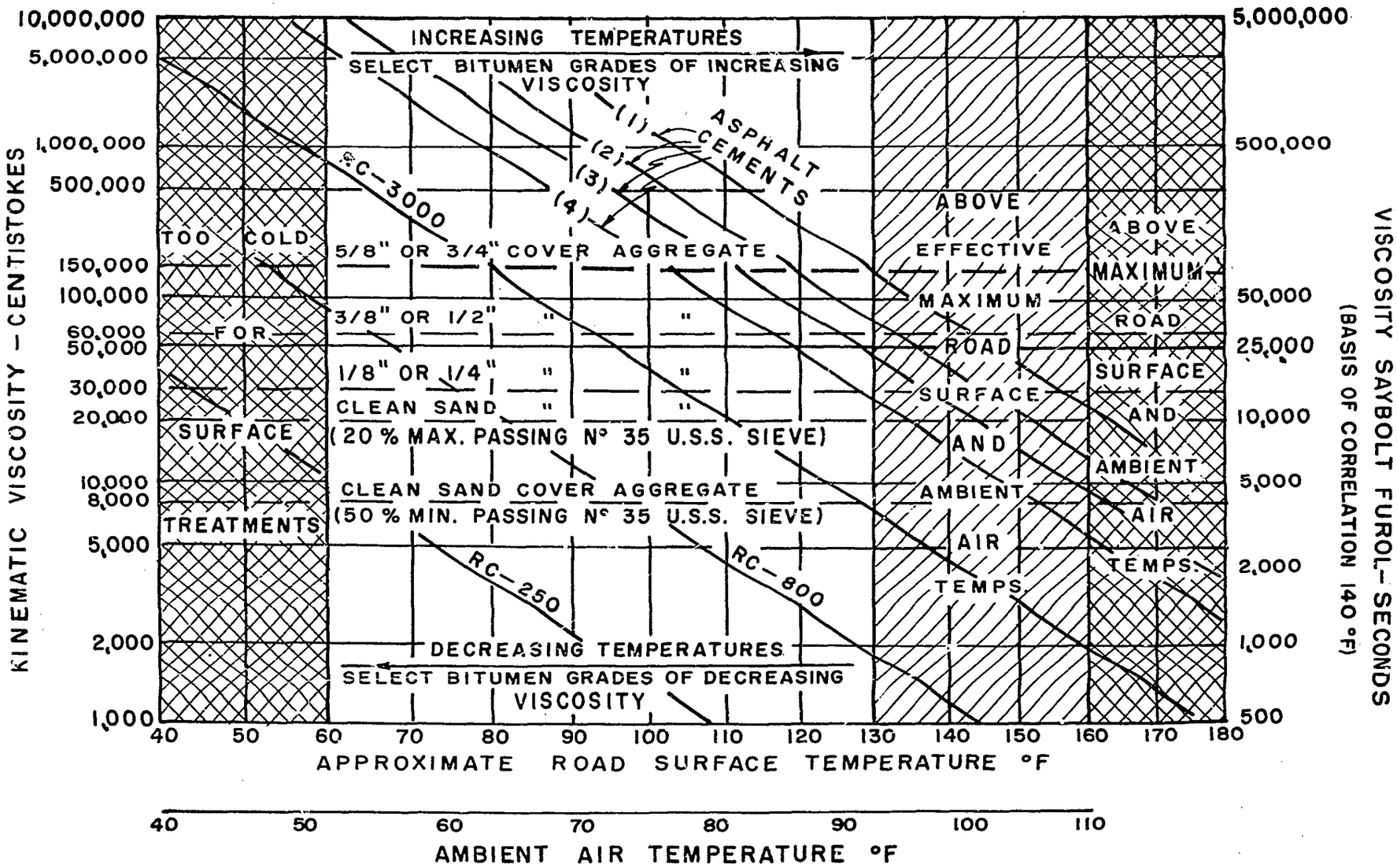


FIGURE A12.4b – SELECTION BITUMINOUS BINDER FOR SURFACE TREATMENTS OR SEAL COATS IN TROPICAL CLIMATES DETERMINED BY SURFACE TEMPERATURES AND SIZE OF COVER AGGREGATE (after Pullen and Prevost, 1969)