Report No. CDOT-DTD-R-95-10

High Performance Hot Mix Asphalt Pavements for Intersections

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Final Report
April, 1995

Prepared in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

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

Acknowledgements

The authors wish to thank the Colorado Department of Transportation for providing the location for this demonstration project and allowing the Colorado Asphalt Pavement Association significant latitude in the design and construction process. In particular, thanks are extended to Robert Clevenger, former Chief Engineer of CDOT, Bill Vidal, former Director of Region 6 whose vision allowed this project to proceed, and to the current Region 6 Construction Engineer, Stan Ihlenfeldt, for agreeing to pay for part of the cost.

Thanks are due Skip Bettis, Bill Brown, John Cheever, Ron Snyder, and Lou Chapman of Bituminous Roadways (BR) of Colorado for fitting this project into an already jammed schedule, and to Bill Lauer of BR for agreeing to do the construction at their cost.

Much of the cost of this project was also absorbed by Sinclair Refining, Koch Materials, and Conoco Asphalt by donating the AC-10 and AC-20P asphalt cement used in one quadrant of the intersection. Thanks to Brannan Sand and Gravel and Frank C. Klein for donating trucks and personnel to help reduce costs further.

The SUPERPAVE Level I Mix Design was performed by the Asphalt Institute in Lexington, Kentucky. The European tests were performed by Nava Far and Dave Price (CDOT-Staff Materials), and the SHRP asphalt cement tests were performed by Benja Bemelen. Werner Hutter performed the life cycle cost analysis.

The research study panel was Steve Horton and Bob LaForce (CDOT-Staff Materials), Sid Motchan (CDOT-Region 6), Ken Wood (CDOT-Region 4), Donna Harmelink (CDOT-Research), John Cheever and Bill Brown (Bituminous Roadways of Colorado), Richard Zamora (FHWA-Colorado Division), Frank Rich (FHWA-Region 8), and Dave Potter (City of Denver).

Technical Report Documentation Page

1. Report No. CDOT-DTD-R-95-10	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle High Performance Hot Mix Asphalt Pavements for Intersections		5. Report Date April 1995
		6. Performing Organization Code File No. 10.12
7. Author(s) Tim Aschenbrener & Scott Shuler		8. Performing Organization Rpt.No. CDOT-DTD-R-95-10
9. Performing Organization Name and Address Colorado Department of Transportation 4201 East Arkansas Avenue Denver, Colorado 80222		10. Work Unit No.(TRAIS)
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Colorado Department of Transportation 4201 E. Arkansas Avenue Denver, Colorado 80222		13. Type of Rpt. and Period Covered Final Report
		14. Sponsoring Agency Code

15. Supplementary Notes
Prepared in Cooperation with the U.S. Department of Transportation Federal Highway Administration

16. Abstract
High traffic intersections have become a popular location for substituting portland cement concrete (PCC) pavements in place of hot mix asphalt (HMA). There are four principle reasons for this trend in the U.S.: 1) marginal performance of HMA pavements, 2) a lack of understanding regarding how to eliminate failures in HMA pavements at intersections, 3) a highly effective marketing campaign by the portland cement industry, and 4) an attitude by some in the asphalt industry to let this market go to PCC.

A program has begun in Colorado to combat the reasons shown above to prove that HMA pavements can be effectively constructed at intersections and provide significant benefits to the owner. To many highway agencies, performance, construction speed, and cost are the three key issues to consider when rehabilitating or reconstructing high traffic intersections. Therefore, demonstration projects were organized by the Colorado Asphalt Pavement Association (CAPA) to demonstrate performance, construction speed and cost advantages over PCC pavements at high traffic intersections in Colorado.

This paper documents the planning, design, and construction processes that were followed to successfully build an HMA pavement at an intersection carrying over 7.7 million 18 kip (8200 kg) ESALs for a 20-year design. Results of this work indicate that some of the new technologies from the Strategic Highway Research Program (SHRP) in conjunction with certain so-called "torture tests" from European experience are important factors in determining which HMA mixtures will perform best under the difficult conditions at intersections. The project was designed to evaluate both conventional and polymer modified asphalt cements, and thin overlay and thick rehabilitation strategies.

17. Key Words high traffic intersections, asphalt intersections, SUPERPAVE, European testing equipment, case history	18. Distribution Statement No Restrictions: This report is available to the public through the National Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report) Unclassified	20. Security Classif.(of this page) Unclassified	21. No. of Pages 71	22. Price

Table of Contents

1.0 Introduction 1

2.0 Planning and Design of the Demonstration Project 3
- 2.1 Project Location 3
- 2.2 Objectives and Approach 4

3.0 Pavement Management Strategy 6
- 3.1 Traffic and Environment 6
- 3.2 Sampling 6
- 3.3 Laboratory Tests 7
- 3.4 Materials and Structure Problems 7
- 3.5 Repair and Construction Plan 9

4.0 Mixture Designs 11
- 4.1 Surface Course. 11
  - 4.1.1 Aggregate Test Results. 11
  - 4.1.2 Asphalt Cement Test Results. 15
  - 4.1.3 Mixture Test Results. 18
- 4.2 Large Stone Base Course. 19

5.0 European "Torture Tests" 21
- 5.1 French Rutting Tester 21
- 5.2 Hamburg Wheel-Tracking Device 22
- 5.3 Thermal-Stress, Restrained-Specimen Test (TSRST) 23

6.0 Field Verification 25
- 6.1 Trial Runs Prior to Project Work 25
  - 6.1.1 Laboratory Mixed 25
  - 6.1.2 Plant Produced Field Trial 25
- 6.2 Actual Intersection Mixture 28
  - 6.2.1 French Rutting Tester 28
  - 6.2.2 Hamburg Wheel-Tracking Device 28
  - 6.2.3 TSRST 29
- 6.3 Summary 30

7.0 Construction 31
- 7.1 Milling and Base Course Paving 31
- 7.2 Surface Course Paving 31
- 7.3 Summary 31

8.0 Economy 33
- 8.1 Life Cycle Cost 33

8.1.1 HMA Cost 33
8.1.2 Life Cycle Cost Analysis 33
8.1.3 Full-Depth Treatment 35
8.1.4 Surface Treatment 35
8.2 Summary 35

9.0 Field Performance 36

10.0 Summary 37
10.1 Maximize Performance 37
10.2 Minimize Inconvenience 37
10.3 Optimize Economy 37

11.0 References 38

Appendix A: SUPERPAVE Aggregate Specifications
Appendix B: SUPERPAVE Gradation "Master Ranges"
Appendix C: SUPERPAVE Gyratory Gyration Specifications
Appendix D: SUPERPAVE Volumetric Specifications
Appendix E: Profilograph Traces

List of Tables

Table 1. Selecting Asphalt Cement Grade for Traffic and Temperature. 6
Table 2. Material Properties of Base and Subgrade. 7
Table 3. Conventional Properties of AC-10. 15
Table 4. SUPERPAVE Properties of AC-10 and AC-20P. 16
Table 5. Task Force 31 Test Results for AC-20P. 17
Table 6. SUPERPAVE Volumetric Properties. 18
Table 7. Volumetric Properties of Large Stone Base Course. 19
Table 8. French Rutting Tester Results for the Laboratory Mixed Samples. 25
Table 9. French Rutting Tester Results for the Field Trial. 26
Table 10. Hamburg Wheel-Tracking Device Results for the Field Trial. 28
Table 11. French Rutting Tester Results for the Intersection. 28
Table 12. Hamburg Wheel-Tracking Device Results for the Intersection. 29
Table 13. TSRST Test Results. 29
Table 14. Life Cycle Cost Analysis for the Intersection. 34

List of Figures

Figure 1. Location of Demonstration Project. 5
Figure 2. Rutting Prior to Construction. 5
Figure 3. Rehabilitated Pavement Sections. 10
Figure 4. Plan View of the Intersection. 10
Figure 5. Aggregate Gradation for the Surface Course Mixture. 13
Figure 6. Gradation of Large Stone Base Course. 20
Figure 7. Typical Hamburg Wheel-Tracking Device Output. 24
Figure 8. French Rutting Tester Results at 60°C for Surface Course. 27
Figure 9. Hamburg Wheel-Tracking Results at 45°C for Surface Course. 27
Figure 10. Appearance of Surface After Milling and Base Course Placement. 32
Figure 11. Appearance of Surface Course Mixture. 32

1.0 Introduction

There appears to be a growing trend in the U.S. to substitute portland cement concrete (PCC) pavements for hot mix asphalt (HMA) pavements at high traffic intersections when rehabilitation is necessary. Discussions with owner-agencies responsible for reconstruction indicates the switch is due to an increase in permanent deformation in HMA pavements at intersections. Unfortunately, this lack of confidence is often derived from experience with poor life-cycle performance of HMA pavements in high traffic areas.

Asphalt technologists argue that HMA pavements have historically performed well under the most demanding traffic conditions and that rutting is due to a lack of understanding of the technology or a misuse of known principles. Whatever the reason for early distress, when HMA pavements do not perform as expected, there is a tendency to substitute products with no risk of rutting failure, whatever the cost.

This apparent trend in the U.S. to substitute PCC pavements for HMA pavements at intersections has become an issue for many in the asphalt industry. This is true even though some asphalt producers argue that construction in intersections is difficult, represents small tonnages, and therefore, is not worth pursuing. To the rest of the industry, however, this issue has become a challenge to prove that HMA pavements will perform under the most demanding conditions, emphasizing what many in the industry already know and wish to share.

Members of the Colorado Asphalt Pavement Association (CAPA) and Colorado Department of Transportation (CDOT) met in 1993 to discuss construction of a project to demonstrate the ability of HMA pavements to perform without premature rutting at an intersection. The project was intended to demonstrate several factors:

- Achieving Maximum Performance
- Minimizing Agency/Public Inconvenience
- Optimizing Economy with Life Cycle Costs

These three factors are the basis of any properly conceived engineering project. However, they become more important when working in intersections because of the potential impact to businesses in urban areas and the high visibility associated with construction in these locations.

2.0 Planning and Design of the Demonstration Project

2.1 Project Location

The location of the demonstration project was selected based on two factors:

1) historical rutting distress, and
2) very high traffic loading.

These two factors were important in the selection process to demonstrate that the technologies evaluated had the ability to perform under the most adverse conditions even though materials and methods utilized in the past had not.

The location selected is the intersection of US-85 and 104th Avenue in Adams County, Colorado shown in Figure 1.

This intersection is one of the highest trafficked pavements in Colorado carrying over 7.7 million 18 kip (8200 kg) equivalent single axle loads (ESALs) in a 20 year design period. The average daily traffic (ADT) is 24,200 with 8.5% trucks. This level of traffic volume was considered an excellent location to evaluate the performance of new HMA paving materials and construction techniques. The adverse environment provided by the very high traffic at this location provides an opportunity to evaluate pavement performance under the most demanding conditions in a relatively short time period. Positive results from this demonstration project can then be used to make decisions regarding use of HMA materials in other high traffic applications.

An initial site visit was made on April 19, 1994 with CDOT and CAPA representatives to determine if the intersection at US-85 and 104th Avenue would be suitable for a demonstration project. Based on the high truck traffic observed and the rutting in the wheel paths shown in Figure 2, the intersection appeared to be acceptable. Rut depths were typically 2 inches (50 mm) deep.

2.2 Objectives and Approach

The objectives of the project were three-fold: 1) Maximize Performance, 2) Minimize Inconvenience, and 3) Optimize Economy. The details behind each of the three major objectives of the demonstration project were accomplished by following the steps shown in the outline below:

I. Achieving Maximum Performance

- A. Determine a Construction Strategy
    1. Forensic Analysis
        - a. Observation
        - b. Sampling
        - c. Laboratory Tests
        - d. Materials and/or Structural Problems
    2. Develop a Repair/Construction Plan
        - a. Match Materials/Structure to Site
        - b. Mixture Design/Analysis
        - c. Structural Design
    3. Build the Design

II. Minimize Agency/Public Inconvenience

- A. Night Construction-Weekdays
- B. Minimize Duration of Construction
- C. Minimize Disturbance of Traffic

III. Optimize Economy

- A. Initial Costs
- B. Life-Cycle Costs
- C. User Costs

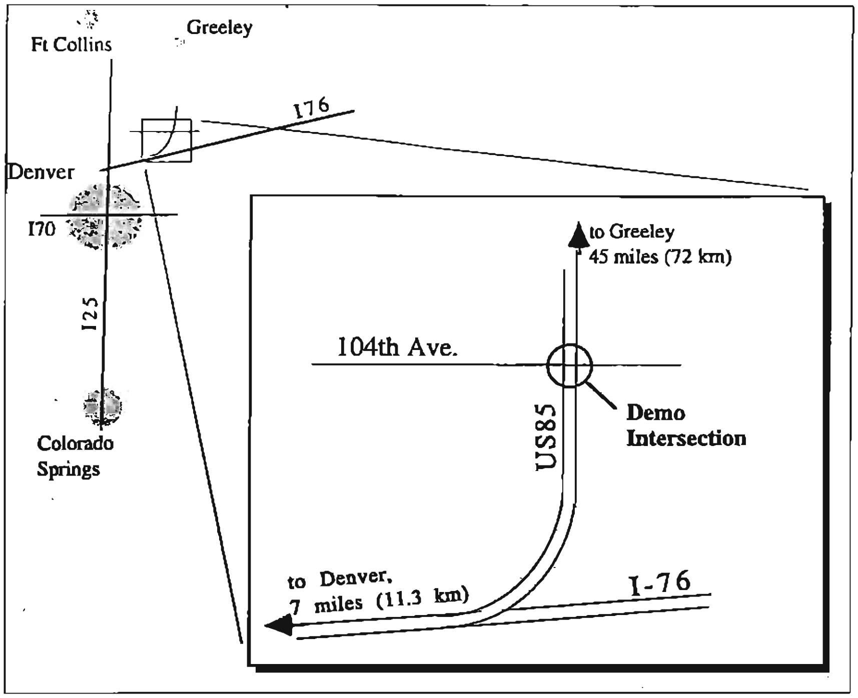

Figure 1. Location of Demonstration Project.

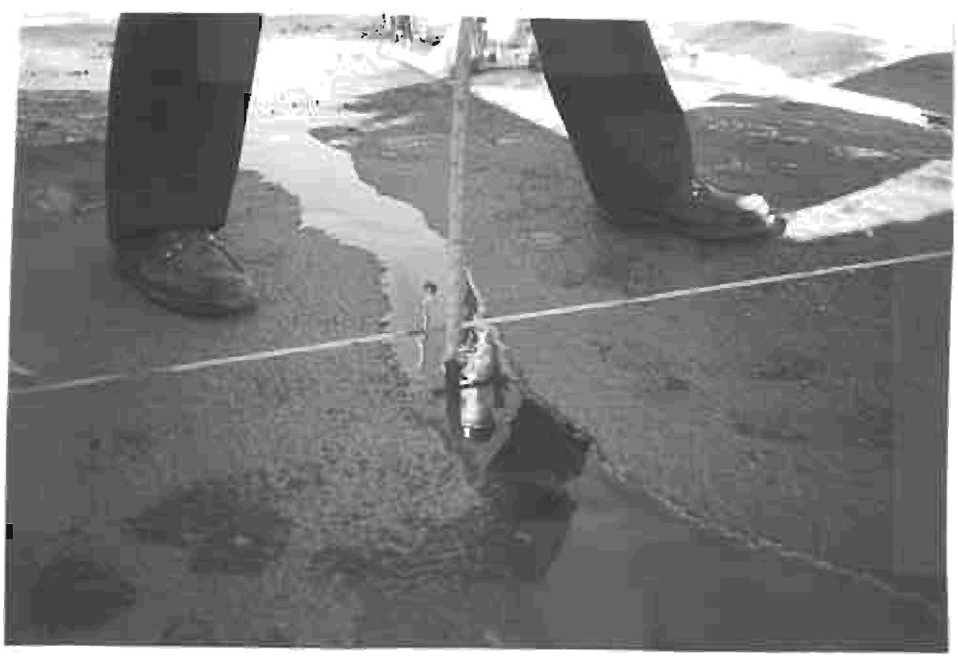

Figure 2. Rutting Prior to Construction.

3.0 Pavement Management Strategy

3.1 Traffic and Environment

This intersection is one of the highest trafficked pavements in Colorado carrying over 7.7 million 18 kip (8200 kg) equivalent single axle loads (ESALs) in a 20 year design period.

The high and low temperature environments for the area were determined from data compiled in the SHRP weather data base. The nearest weather station was in Denver at Stapleton Airport; the data is shown in Table 1.

Table 1. Selecting Asphalt Cement Grade for Traffic and Temperature.

Reliability	Temperatures (°C)			Asphalt Cement Grade (PG)
	Maximum Pavement	Minimum Air	Design Air	
50 %	54	-24	34	58-28
98 %	56	-32	34	58-34

For slow moving or stop-and-go traffic, SUPERPAVE recommends 1 to 2 grades stiffer on the high temperature side of the specification.

3.2 Sampling

Samples of the existing HMA pavement were obtained by cutting ten, 4-inch diameter (100 mm) cores from the driving and passing lanes of the intersection. The cores were used to determine the thickness of the existing pavement. A continuous-flight, power auger was used to obtain sub-pavement samples of base course and subgrade soils. The existing HMA pavement was 8-inches (200 mm) thick. Observation of the cores indicated that rutting distress had occurred in the upper 2 inches (50 mm) of the HMA surface course.

3.3 Laboratory Tests

Base course and subgrade soils were evaluated and tested by the CDOT and results are shown in Table 2. The resilient modulus of the base course and subgrade soils were calculated from the R-value. The correlation is reported in Chapter 600 of the CDOT Roadway Design Manual and is shown below:

$$S = [(R - 5) / 11.29] + 3$$

$$\log (M_R) = (S + 18.72) / 6.24$$

where:

R = R-value from AASHTO T 190
S = soil support value, and
M_R = resilient modulus (psi).

Table 2. Material Properties of Base and Subgrade.

Location	Material	Depth,in (mm)	- No. 200	PI	M_R, psi (kPa)	R	Classif.
1	HMA	8 (203)	na	na	na	na	na
	Gravel	40 (1015)	13	4	34,000 (2.3E+05)	79	A-1-b(0)
	Silty Clay	40 + (1015 +)	54	30	4000 (0.3E+05)	13	A-7-6(12)
2	HMA	8 (203)	na	na	na	na	na
	Med Sand	24 (610)	22	np	15,000 (1.0E+05)	54	A-1-b(0)
	Clay	36-60 (914-1524)	53	22	4000 (0.3E+05)	13	A-6(8)

na - not applicable
np - not plastic

3.4 Materials and Structure Problems

Physical tests were not conducted on the core samples taken from the pavement; only visual observations were made. However, the appearance of the HMA cores indicated that permanent deformation was occurring in approximately the upper 2 inches (50 mm) of the surface course mixture. The cores contained many round aggregate particles and an apparently high fine aggregate fraction, which may have contributed to the rutting distress. No evidence of moisture damage was present.

It should be noted that <u>it is highly recommended to perform tests on cores to determine the rehabilitation strategy</u>. Tests should include the profile of the air voids in the core versus depth. Extractions should be performed so the aggregate gradation, coarse aggregate angularity, and fine aggregate angularity can be measured. Additionally, some type of moisture susceptibility testing should be performed, preferably AASHTO T 283. Tests were not performed because of the time involved. This intersection was not originally scheduled to have the "correct" engineered fix, only a temporary maintenance fix. The change to make it a "correct" engineering fix occurred at the last minute, so engineering judgement had to be used in lieu of testing.

Material properties and thicknesses shown in Table 2 were used with traffic information to estimate the design structural thicknesses for each layer using the AASHTO method of pavement design (*1*). Based on this information the design thickness of the pavement was judged adequate for the materials present. Therefore, since there was an adequate structural design, there was a high probability that the rutting did not occur in the subbase and subgrade materials.

It was further believed that the structure was adequate because of the observed distresses. No longitudinal or alligator cracking existed prior to rehabilitation. The lack of these types of cracks indicated the structure had not failed; the rutting was likely in the HMA pavement.

It is believed that the rutting distress was caused by failure of the HMA pavement, not due to over-stressing of the subgrade. The three reasons are: 1) visual observation of the cores, 2) an adequate pavement structure was present, and 3) lack of longitudinal and alligator cracking observed prior to rehabilitation.

3.5 Repair and Construction Plan

Based on the information summarized above the minimum repair required removing the top 2 inches (50 mm) of the surface course HMA pavement and replacing it with a material resistant to rutting under the high traffic loads. In addition to this repair, an additional 6 inches (150 mm) of HMA pavement was removed and replaced with a large stone HMA pavement base course in the right-hand driving lanes. A schematic of the rehabilitation is shown in Figure 3.

One additional variable was added to the demonstration by using two different asphalt cements. A conventional AC-10 asphalt cement was used for the base course and surface course mixtures in the northbound quadrant of the intersection. The AC-10 was selected because it is the asphalt cement grade that is typically used in the Denver Metropolitan area. A polymer modified AC-20P asphalt cement was used in the surface course mixture of the southbound quadrant. The AC-20P was selected because it is typically used when higher quality asphalt cements are required. The AC-20P exceeded the SUPERPAVE asphalt cement recommendations for the intersection.

A plan view of the intersection is shown in Figure 4. Since traffic in the left and right driving lanes is approximately equal, four combinations of pavement sections can be evaluated as follows:

Northbound, left lane (AC-10) (Section 1, Figure 4)	2-inch (50mm) thick Surface Course over 6-inch (150mm) thick Original HMA Pavement
Northbound, right lane (AC-10) (Section 2, Figure 4)	2-inch (50mm) thick Surface Course over 6-inch (150 mm) thick Base Course
Southbound, left lane (AC-20P) (Section 3, Figure 4)	2-inch (50mm) thick Surface Course over 6-inch (150mm) thick Original HMA Pavement
Southbound, right lane (AC-20P) (Section 4, Figure 4)	2-inch (50mm) thick Surface Course over 6-inch (150mm) thick Base Course

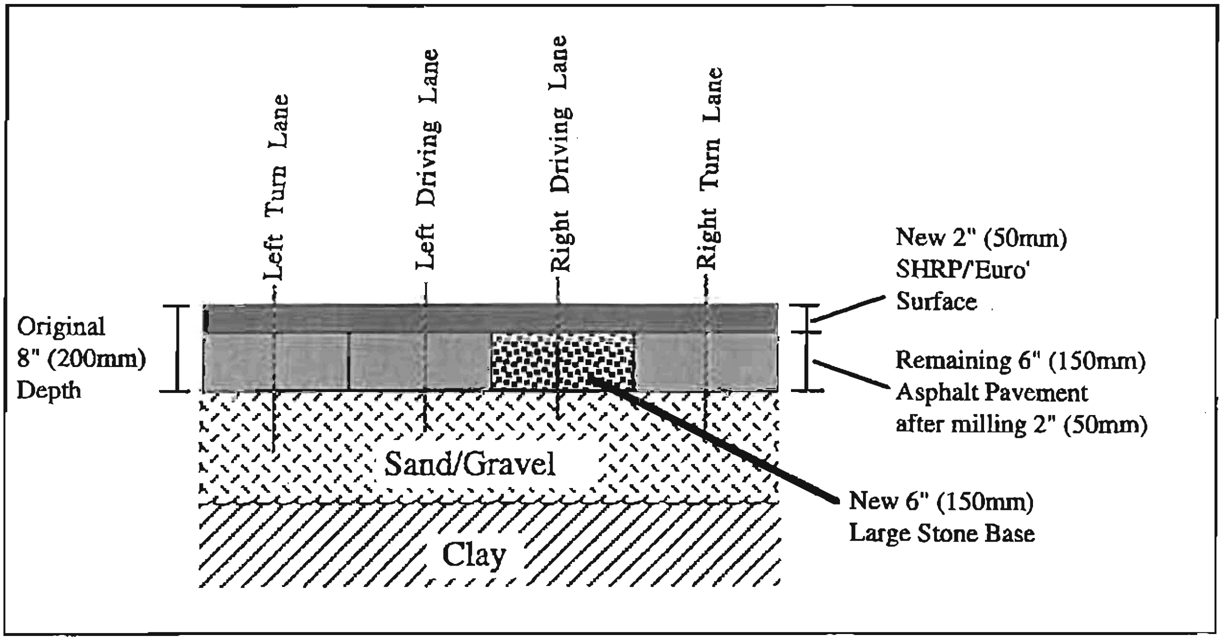

Figure 3. Rehabilitated Pavement Sections.

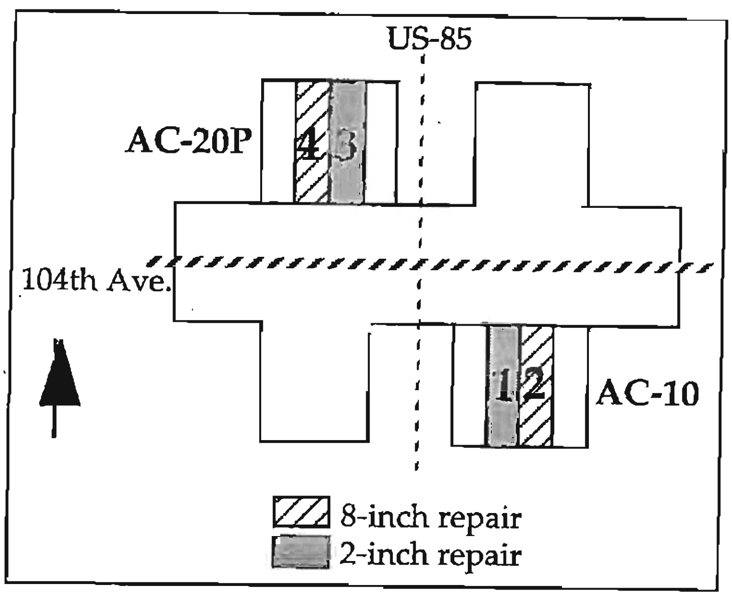

Figure 4. Plan View of the Intersection.

4.0 Mixture Designs

4.1 Surface Course.

The HMA mixture to be used in the surface course was designed using the new Strategic Highway Research Program (SHRP) SUPERPAVE™ Level I technology (2). SUPERPAVE includes careful selection of aggregates and asphalt cements. Additionally, it includes a mixture design using volumetric properties to judge the adequacy of asphalt paving mixtures. The volumetric properties are determined with the SUPERPAVE gyratory compactor that was developed based on the French *Laboratoire Central des Ponts et Chaussees* (LCPC) gyratory compactor.

For this study, the aggregates and volumetric properties were specifically selected to meet the SUPERPAVE requirements. The asphalt cements were selected based on the asphalt cements that were commonly available. One of the asphalt cements (AC-20P) happened to exceed the SUPERPAVE requirements, and the other asphalt cement (AC-10) happened to fail the SUPERPAVE requirements.

4.1.1 Aggregate Test Results.

Under the SUPERPAVE specifications, there are several aggregate requirements. The primary requirement is gradation. In addition there are consensus and source aggregate properties. Consensus properties have wide agreement in the test and specified value. These properties include coarse aggregate angularity, fine aggregate angularity, flat or elongated particles, and clay content. Source properties have wide agreement in the test, but the specified value might vary from location to location. These properties include toughness, soundness, and deleterious material.

The specifications for each of these tests are shown in Appendix A. The aggregates used for the intersection were primarily from the Cooley Morrison Quarry. The blend was 48% of 19.0 mm (3/4 in.) rock, 20% of 12.5 mm (1/2 in.) rock, and 21% granite sand. Additionally, there was 10% washed concrete sand and 1% hydrated lime was used as an anti-stripping additive.

4.1.1.1 Gradation. The 0.45 power gradation chart is used to define the permissible gradation. The recommended gradation should lie within the control points, and it is recommended the gradation lie outside the restricted zone. The recommended "Master Ranges" for the various top-size aggregate gradations are shown in Appendix B. The aggregate gradation obtained for this intersection is shown in Figure 5.

4.1.1.2 Coarse Aggregate Angularity. This property ensures a high degree of internal friction in the coarse aggregate. The recommended test procedure is Colorado Procedure 45 (CP-45) as shown in Appendix A. It is measured by the weight of aggregates larger than 4.75 mm (No. 4) that have one or more fractured faces. For this intersection, the recommended specification was a minimum of 85% with one or more fractured faces and 80% with two or more fractured faces. For this intersection 100% of the aggregates larger than 4.75 mm (No. 4) had two or more fractured faces.

4.1.1.3 Fine Aggregate Angularity. This property ensures a high degree of internal friction in the fine aggregate. The recommended test procedure is AASHTO TP 3. The fine aggregate angularity is defined as the percent air voids present in loosely compacted aggregates smaller than 2.36 mm (No. 8). The higher void content indicates more angular fine aggregate. For this intersection, the recommended specification was a minimum of 45.0%, and the test result was 48.2%.

4.1.1.4 Flat or Elongated Particles. This property is intended to eliminate aggregates that may have a tendency to break during construction and under traffic. The recommended test procedure is ASTM D 4791, and it is performed on coarse aggregate larger than 4.75 mm (No. 4). The flat or elongated particles are defined as the weight of coarse aggregates that have a maximum to minimum dimension of greater than 5. For this intersection, the recommended specification was a maximum of 10%, and the test result was 1%.

Sieve Size		Passing
mm	opening	%
25.40	1	100
19.00	3/4	100
12.50	1/2	78
9.50	3/8	56
4.75	4	29
2.36	8	23
1.18	16	17
0.60	30	13
0.30	50	9
0.15	100	6
0.08	200	4

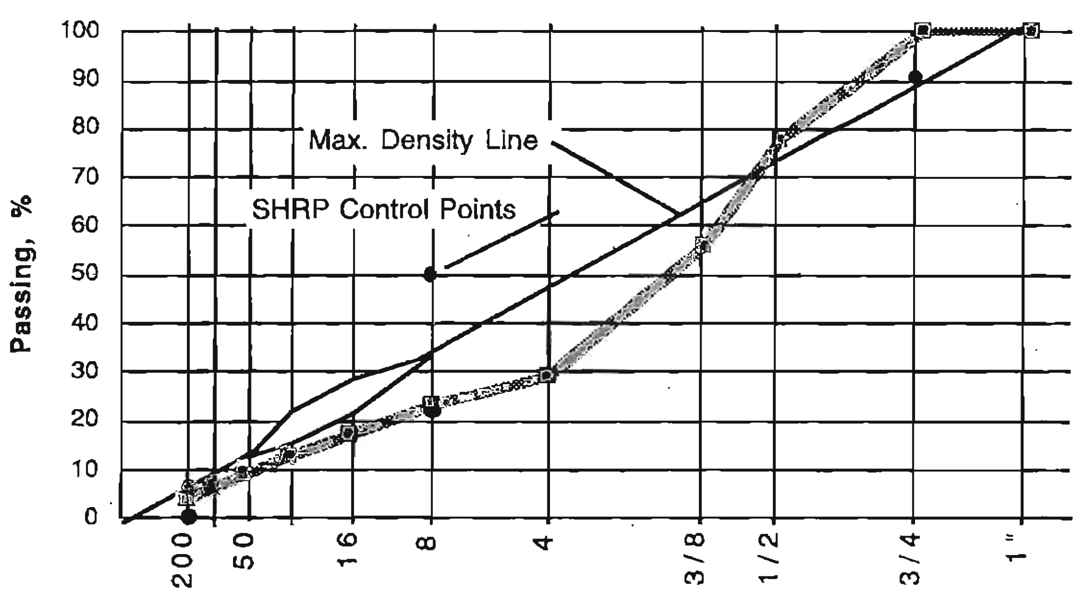

Figure 5. Aggregate Gradation for the Surface Course Mixture.

4.1.1.5 Clay Content. This property is intended to eliminate the presence of clay materials. The clay content is measured by the sand equivalent test as defined in AASHTO T 176. For this intersection, the recommended specification was a minimum of 45%, and the test result was 74%.

4.1.1.6 Toughness. This property measures resistance of the coarse aggregate to abrasion and mechanical degradation. The toughness is measured using AASHTO T 96 (Los Angeles Abrasion). For this intersection, the recommended specification should be a maximum value of 35%, or the maximum value could be as high as 45%. The test result was 24%.

4.1.1.7 Soundness. This property is used to measure the resistance of the aggregate to weathering. It is measured by using either the Sodium or Magnesium Sulfate soundness test in AASHTO T 104. A maximum of 10 to 20% for five cycles is recommended. This test was not performed for the intersection, but the source has a history of approximately 2%.

4.1.1.8 Deleterious Material. This property is used to measure the quantity of contaminants such as shale, wood, mica and coal in blended aggregates. The test used is AASHTO T 112. The recommended specification should be a maximum value between 0.2 to 10%. This test was not performed for the intersection. The CDOT requirement for deleterious material is based upon visual inspection, and no deleterious material is allowed. Based on visual inspection; there was no deleterious material observed in the aggregates.

4.1.2 Asphalt Cement Test Results.

It is believed the type of asphalt cement used in the production of HMA can influence performance of the HMA pavement. Therefore, two asphalt cements were used to produce the HMA for the surface course in an effort to quantify this difference in field performance. The first asphalt cement was a conventional AC-10 supplied by the Sinclair Refinery in Sinclair, Wyoming. The second asphalt cement was an AC-10 supplied by Conoco Asphalt in Denver, Colorado which was modified into an AC-20P by Koch Materials in Pueblo, Colorado using the Styrelf process.

The properties of the AC-10 were evaluated using conventional asphalt technology, and SUPERPAVE technology was used to evaluate both the AC-10 and AC-20P. The SUPERPAVE technology includes the SUPERPAVE tests from the dynamic shear and bending beam rheometers. Results of this testing are shown in Tables 3 and 4.

Table 3. Conventional Properties of AC-10.

Property	Results	Criteria
Viscosity, P 140°F (60°C)	1070	800 -1200
Penetration, 0.1mm 77°F (25°C)	85	80, min
Viscosity After TFOT, P 140°F (60°C)	1979	5000, max

The high temperature performance grade is based on the test results from the dynamic shear rheometer. The performance grade is in increments of 6°C and represents the highest average 7-day pavement temperature (in Celsius) for which the asphalt cement should be used. The high temperature performance grades are: 46, 52, 58, 64, 70, 76, and 82. For example, the highest temperature where the AC-20P meets the minimum requirement from the dynamic shear rheometer is 78.3°C. This asphalt cement would be acceptable for a pavement that had a highest temperature of 76°C but unacceptable for 82°C. Therefore, the AC-20P has a high temperature performance grade of 76.

The low temperature performance grade is based on the test results primarily from the bending beam rheometer. The performance grade is in increments of 6°C and represents the lowest pavement temperature (in Celsius) for which the asphalt cement should be used. The low temperature performance grades are: -10, -16, -22, -28, -34, -40, and -46. For example, the lowest test temperature where the AC-20P meets the requirement for the bending beam rheometer is -21.6°C. A value of 10°C must be subtracted from the test results from the bending beam rheometer. This is done using time-temperature superposition theory in order that the laboratory testing time could be accelerated. After subtracting 10°C, the test result would be -31.6°C. This asphalt cement would be acceptable for a pavement that had a lowest temperature of -28°C but unacceptable for -34°C. Therefore, the AC-20P has a low temperature performance grade of -28.

Table 4. SUPERPAVE Properties of AC-10 and AC-20P.

Test, Unaged		**Criteria**	**AC-10**	**AC-20P**
Brookfield Visc, 135°C, Pa-s		3.0, max	0.28	1.34
G*/sin delta, 10 rad/sec	**Temperature for:**	1.0 kPa, min	58°C	79.8°C
Test, RTFO Residue				
Mass Loss, %		1.0, max	0.006	< 1.0
G*/sin delta, 10 rad/sec	**Temperature for:**	2.2 kPa, min	58°C	78.3°C
Tests on PAV (after RTFO); 100°C				
G* x sin delta, 10 rad/sec	**Temperature for:**	5000 kPa, max	25°C	13.7°C
Creep Stiffness @ 60 s	**Temperature for:**	300 kPa, max	-12°C	-25°C
Slope, m, @ 60 s	**Temperature for:**	0.3 min	-12°C	-21.6°C
High Temperature Grade			58	76
Low Temperature Grade			-22	-28

The results from the SUPERPAVE tests indicate the asphalt cements are quite different. The AC-10 meets the minimum requirements for a SUPERPAVE PG 58-22, while the AC-20P meets the minimum requirements for a SUPERPAVE PG 76-28. This means the AC-10 should perform well for pavement temperatures from 58°C in the summer to -22°C in the winter, while the AC-20P should perform over a much wider range from 78°C to -31°C, summer to winter, respectively.

In addition to the SUPERPAVE tests, more conventional technology was used to evaluate the AC-20P, as well. An alternative specification (*3*) using conventional test methods has been developed by Task Force 31 of AASHTO, ARTBA (American Road and Transportation Builders Association), and AGC (Associated General Contractors). The results of this testing are shown in Table 5.

Table 5. Task Force 31 Test Results for AC-20P.

Property	Criteria	Results
Penetration, 77°F (25°C), 0.1mm	40-75	67
R & B Softening Point, °F (°C)	140, min	155 (68)
Separation, 2-day, R&B diff, °F	4	-0.30
Absolute Visc, P, 140°F (60°C)	5000, min	32,625
Kinematic Visc, cSt, 275°F (135°C)	2000, max	1481

4.1.3 Mixture Test Results.

4.1.3.1 Recommended Gyratory Revolutions. When using the SUPERPAVE gyratory compactor, there are three different gyrations that have a specified level of compaction. The number of gyrations are the initial (N_{init}), design (N_{des}), and maximum (N_{max}) gyrations. The specification at N_{des} is 3 to 5% air voids. For this project, N_{des} was selected as 109 gyrations based on traffic and environment, and 4% air voids were targeted to select the optimum asphalt content. The specification at the N_{init} and N_{max} gyrations are shown in Table 6. The recommended number of gyrations for each of the three different levels is shown in Appendix C.

Table 6. SUPERPAVE Volumetric Properties.

Property	Test Result	Criteria
Air Voids, % @ 109 gyrations	4.0	4.0
Asphalt, %	4.6	na
VMA, %	14.2	13.0, min
VFA, %	72.0	65-75
Tensile Strength Ratio, %	85	80, min
Relative Density @ 8 gyrations, %	85.7	89, max
Relative Density @ 174 gyrations, %	97.6	98, max
Dust Proportion	0.9	0.6-1.2

4.1.3.2 Volumetric Properties. The volumetric properties of the sample compacted in the SUPERPAVE gyratory compactor are measured. There are specifications for air voids (3 to 5%), voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). For the intersection, the specified values are shown in Table 6. The recommended specifications for VMA and VFA are shown in Appendix D.

4.1.3.3 Dust Proportion. The dust proportion is calculated as the dust to asphalt ratio. It is computed as the percentage by weight of aggregate finer than the 0.075 mm (No. 200) sieve size to the effective asphalt content expressed as a percent by total weight of mix. The acceptable ratio is between 0.6 and 1.2. This mix had a ratio of 0.9.

4.1.3.4 Moisture Susceptibility. The purpose of this test is to define if the mixture is susceptible to moisture damage. The test used is AASHTO T 283. A minimum ratio 0.80 is recommended. This mix had a ratio of 0.85.

4.2 Large Stone Base Course.

The design of the large stone base course used more conventional technology than the surface course. Colorado has had much experience with these mixtures and has developed a procedure for design based on the California Kneading Compactor that relates to performance very well. The design is based on aggregate gradation and volumetrics of the laboratory compacted mixture. The gradation of the mixture conformed to the requirements for the CDOT Grading G as shown in Figure 6 and volumetric properties shown in Table 7

Table 7. Volumetric Properties of Large Stone Base Course.

Property	Test Result	Criteria
Air Voids, %	3.7	3-5
Asphalt, %	4.0	na
VMA, %	12.8	11.0, min
VFA, %	71	65-75
Tensile Strength Ratio, %	107	80, min
Dust Proportion	1.14	0.6-1.2

Sieve Size		Passing	CDOT 'G' Spec	
mm	opening	%	lower	upper
38.10	1-1/2	100	100	100
25.40	1	93		
19.00	3/4	82	63	85
12.50	1/2	64	46	78
9.50	3/8	55		
4.75	4	43	22	54
2.36	8	33	13	43
1.18	16	25		
0.60	30	18	4	22
0.30	50	11		
0.15	100	7		
0.08	200	4.8	1	8

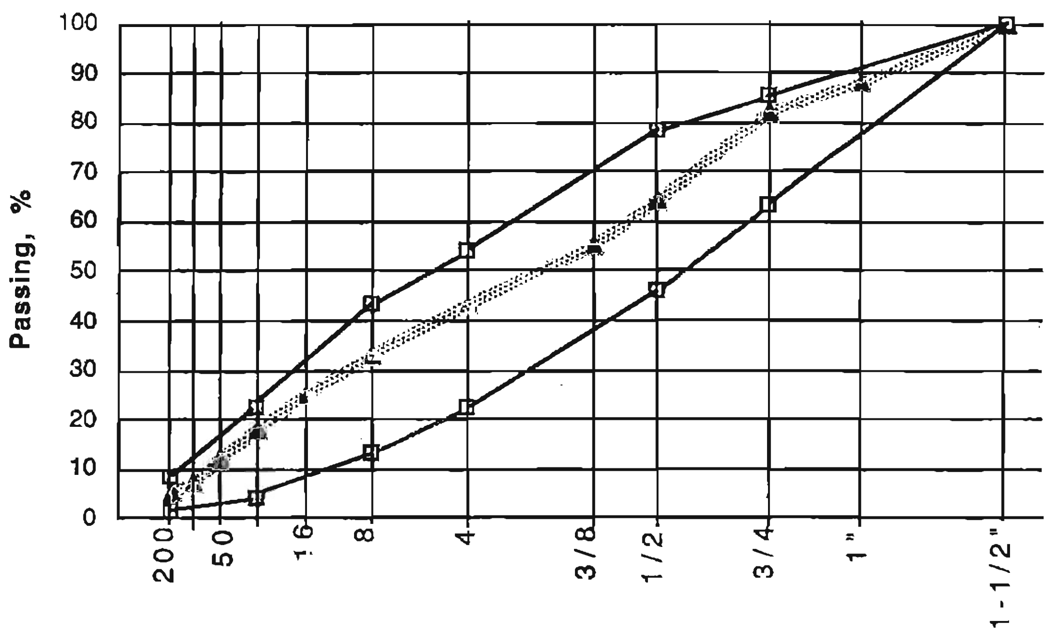

Figure 6. Gradation of Large Stone Base Course.

5.0 European "Torture Tests"

Certain European-based performance-related laboratory tests were used to verify the performance of the SUPERPAVE Level I mixture design. The European tests are sometimes referred to as "torture tests" because of the severe loading and environmental conditions which they subject the test specimens. Positive results from these tests have been correlated to well-performing HMA pavements in the field.

5.1 French Rutting Tester

The French Rutting Tester manufactured by the *Laboratoire Central des Ponts et Chaussees* (LCPC) was used to evaluate the resistance of the surface course mixture to permanent deformation.

Specimens are 500 x 180 mm (19.7 x 7.1 in.) and can be 50 or 100 mm (2 or 4 in.) thick. Two samples can be tested simultaneously.

Specimens are repeatedly loaded by a tire rolling back and forth over the surface at elevated temperatures. The samples are loaded with 5000 N (1124 lbs.) by a pneumatic tire inflated to 0.6 MPa (87 psi). The tires load each sample at 1 cycle per second; one cycle is two passes. The chamber is heated to 60°C (140°F) but can be set to any temperature between 35° and 60°C (95° and 140°F).

When a test is performed on a laboratory compacted sample, it is aged at room temperature for as long as 7 days. It is then placed in the apparatus and loaded with 1000 cycles at room temperature to "zero" the device. The sample is then heated to the test temperature for 12 hours before the test begins. Rutting depths are measured after 100, 300, 1000, 3000, 10,000, and 30,000 cycles. The rutting depth is reported as a percentage of the sample thickness. After a given number of cycles, the percentage is calculated as the average of 15 measurements (five locations along the length and three along the width) divided by the original slab thickness.

HMA surface course mixtures are considered resistant to rutting on high traffic pavements if the rutting depth after the test is less than or equal to 10% of the slab thickness after 30,000 cycles. The results are plotted on a log-log graph paper. The slope and intercept (at 1000 cycles) are calculated using linear regression. The equation is:

$$Y = A\ (X/1000)^{B}$$

where:

Y = rutting depth (%),

X = cycles,

A = intercept of the rutting depth at 1000 cycles, and

B = slope of the curve

5.2 Hamburg Wheel-Tracking Device

The Hamburg wheel-tracking device was used to evaluate the resistance of the SHRP Level I mixture to moisture damage. It is manufactured by Helmut-Wind Inc. in Hamburg, Germany.

Test specimens measure 260 mm (10.2 in.) wide, 320 mm (12.6 in.) long, and 40 mm (1.6 in.) deep. Test specimens are compacted in a linear kneading compactor to an air void content of 6% $\pm$ 1%. The mass of the resulting specimen is approximately 7.5 kg (16.5 lbs.). The samples were submerged in water and tested at 45°C (113°F) and 50°C (122°F). A steel wheel, 47 mm (1.85 in.) wide, loads the samples with 705 N (158 lbs.). The wheel makes 50 passes per minute over each of two samples. The maximum velocity of the wheel is 34 cm/sec (1.1 ft/sec) in the center of the sample.

Each sample is loaded for 20,000 passes or until 20 mm of deformation occurs.

The results from the Hamburg wheel-tracking device include the creep slope, stripping slope and stripping inflection point defined by Hines (*4*) and shown in Figure 7.

The creep slope relates to rutting from plastic flow. It is the inverse of the rate of deformation in the linear region of the deformation curve, after post compaction effects have ended and before the onset of stripping. The stripping slope is the inverse of the rate of deformation in the linear region of the deformation curve, after stripping begins and until the end of the test. It is the number of passes required to create a 1 mm impression from stripping. The stripping slope is related to the severity of moisture damage. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope. It is related to the resistance of the HMA to moisture damage.

5.3 Thermal-Stress, Restrained-Specimen Test (TSRST)

The Thermal-Stress, Restrained-Specimen Test (TSRST), developed as part of the SHRP research program (*5*), is based on earlier research by Arand (*6*). The test evaluates the thermal shrinkage resistance of HMA by constraining the ends of a cylindrical specimen while lowering the temperature of the specimen until tensile failure occurs.

Test specimens were produced using plant mixed HMA but compacted in the laboratory using the linear kneading compactor. The compacted HMA was then aged for 120 hours (5 days) at 85°C (185°F) in a forced draft oven. Short term aging was not conducted in the laboratory since the samples were plant produced. Samples tested were 50 mm (2 in.) diameter and 250 mm (10 in.) long.

The test specimen is cooled at the rate of 10°C (18°F) per hour. Liquid Nitrogen is used to provide the cooling. The sample is not allowed to contract during the cooling period. The sample length is monitored with LVDTs and the use of invar steel rods. Tensile stresses develop in the specimen since the ends of the sample are constrained, and not allowed to contract. A closed-loop servo system keeps the sample at a constant length. When the developed stress exceeds the strength of the sample, fracture occurs. The temperature and stress at fracture are recorded.

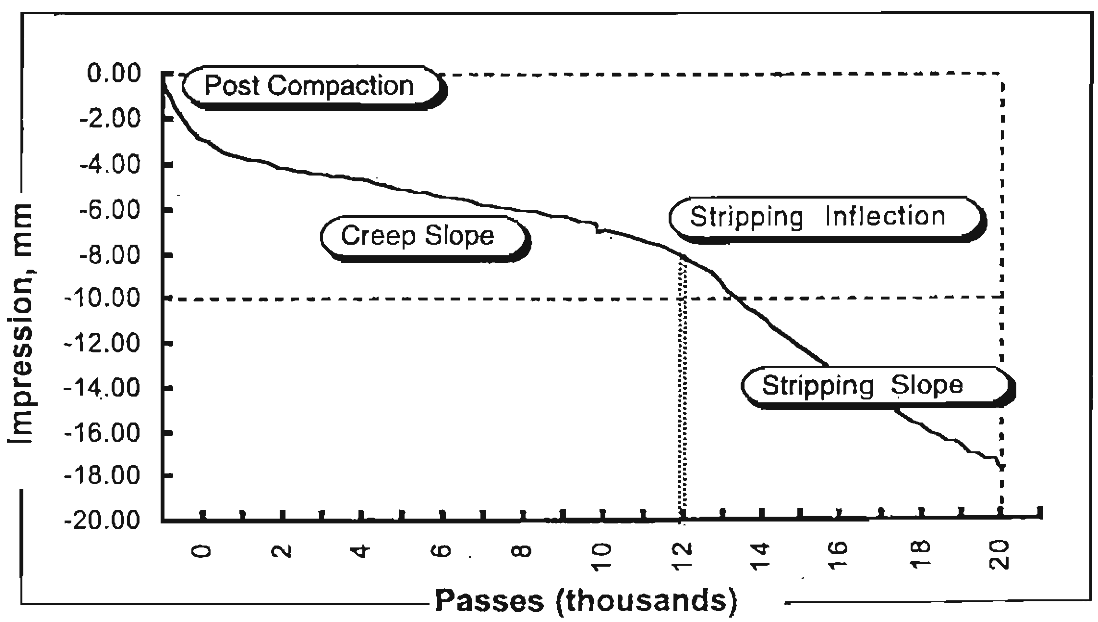

Figure 7. Typical Hamburg Wheel-Tracking Device Output.

6.0 Field Verification

6.1 Trial Runs Prior to Project Work

The job-mix formula used for the mix design was duplicated for testing in the European equipment by: 1) preparing mixtures in the CDOT laboratory and 2) producing approximately 150 tonnes of the mixture in the hot mix plant that was used during construction. Mixtures produced by both of these methods were evaluated for water sensitivity in the Hamburg wheel-tracking device and for rutting potential in the French rutting tester.

6.1.1 Laboratory Mixed

The asphalt contents examined in the laboratory were higher than that recommended in the laboratory mix design. The asphalt content recommended in the mix design was 4.6% (Table 6). This was considered very low. Therefore, higher asphalt contents were tested in the laboratory environment to examine the mixture's sensitivity to rutting from plastic flow.

Results of the tests conducted on the laboratory mixed and compacted samples are shown in Table 8. Only the French rutting tester was used, and the results were excellent.

Table 8. French Rutting Tester Results for the Laboratory Mixed Samples.

Test Temperature	Asphalt Cement Type	Asphalt Content	Rut Depth	Specification
60°C	AC-20P	5.0 %	3.9 %	≤ 10%
		5.5 %	4.3 %	

6.1.2 Plant Produced Field Trial

After the laboratory testing indicated that the mixture was very rut resistant, the contractor produced approximately 150 tonnes of the mixture as a field trial. This material was not placed at the intersection. The material was produced to examine the effect of plant production on the material properties. The field trial had the potential to gain a valuable insight prior to construction of the intersection.

Results from the French rutting tester are shown in Table 9 and Figure 8. The French rutting tester results were 8.8% and acceptable: less than the 10% maximum. However, the results were close enough to the maximum criteria that a slightly lower asphalt content was recommended for the actual intersection. It is not uncommon for the plant produced material to be less rut-resistant than the laboratory mixed material. It is precisely because of that phenomena that the field trial was conducted. The asphalt content was verified with extraction testing.

Table 9. French Rutting Tester Results for the Field Trial.

Test Temperature	Asphalt Cement Type	Asphalt Content	Rut Depth	Specification
60°C	AC-10	5.3%	8.8 %	≤ 10%

The Hamburg wheel-tracking results are shown in Table 10 and Figure 9. Note that the AC-10 mixtures exceed the 4 mm allowable impression specified by the City of Hamburg, Germany. This standard applies to laboratory produced mixtures. CDOT has found that field produced mixtures often provide higher impressions than the same mixtures produced in the laboratory. The guide being used for field produced mixtures is a maximum of 10 mm impression.

The recommended test temperature for the Denver metropolitan area is 45°C, and the maximum rut depth is 10 mm. The test result at this temperature was acceptable. An additional test was performed at 50°C since that is the test temperature recommended in Hamburg. This test result was presented for research purposes.

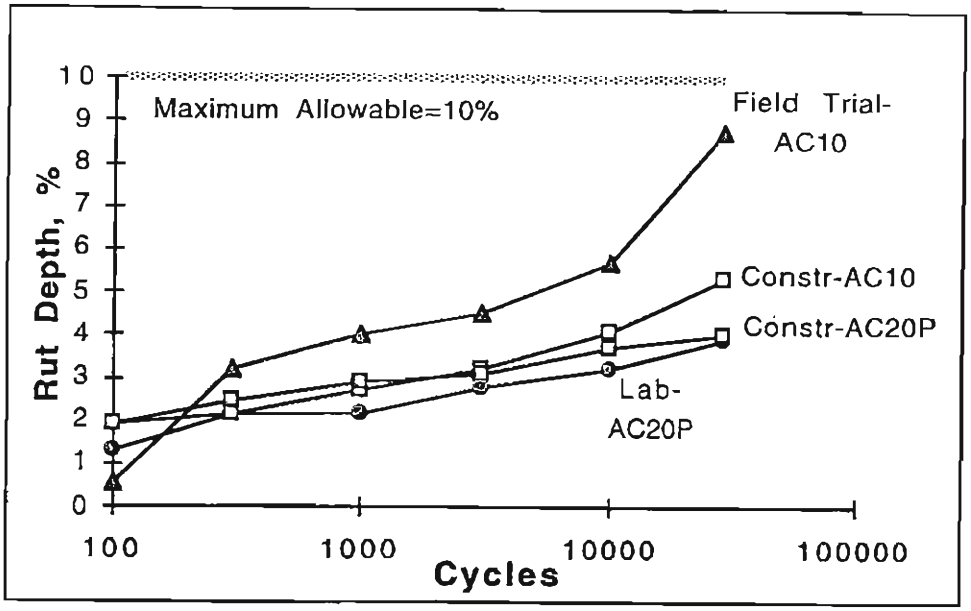

Figure 8. French Rutting Tester Results at 60°C for Surface Course.

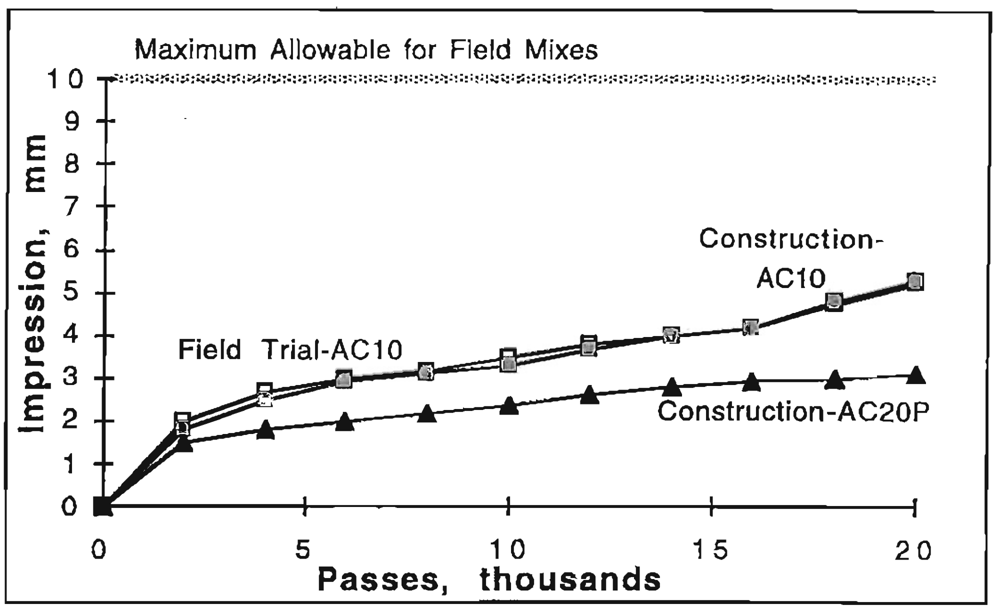

Figure 9. Hamburg Wheel-Tracking Results at 45°C for Surface Course.

Table 10. Hamburg Wheel-Tracking Device Results for the Field Trial.

Test Temperature	Asphalt Cement Type	Asphalt Content	Rut Depth	Specification
45°C	AC-10	5.3 %	5.2 mm	< 10 mm
50°C	AC-10	5.3 %	> 20 mm	

6.2 Actual Intersection Mixture

6.2.1 French Rutting Tester

The French rutting tester results are shown in Table 11 and Figure 7. The results were acceptable for both the AC-10 and AC-20P mixtures.

Table 11. French Rutting Tester Results for the Intersection.

Test Temperature	Asphalt Cement Type	Asphalt Content	Rut Depth	Specification
60°C	AC-10	5.0 %	5.5 %	≤ 10%
	AC-20P	5.0 %	4.1 %	

6.2.2 Hamburg Wheel-Tracking Device

The Hamburg wheel-tracking results are shown in Table 12 and Figure 8. The results were acceptable for the AC-20P mixture, even when tested at extremely high temperatures (55°C). The AC-10 mixture was acceptable at the recommended 45°C. The 50°C test results were not acceptable, but were presented only for research purposes.

Table 12. Hamburg Wheel-Tracking Device Results for the Intersection.

Test Temperature	Asphalt Cement Type	Asphalt Content	Rut Depth	Specification
45°C	AC-10	5.0 %	5.3 mm	< 10 mm
50°C		5.0 %	> 20 mm	
50°C	AC-20P	5.0%	3.1 mm	
55°C		5.0%	7.2 mm	

6.2.3 TSRST

The TSRST results used to quantify the thermal cracking performance of the HMA mixture are shown in Table 13. The AC-20P mixture had superior performance in terms of fracture temperature and fracture strength when compared to the AC-10 mixture. More importantly, the AC-10 mixture failed in the TSRST before the lowest temperature expected to be encountered in the field (-32°C for 98% reliability) was encountered, and barely surpassed the 50% reliability (-24°C). Based on the TSRST results, the AC-20P has approximately a 98% reliability of having its lowest performing temperature exceeded.

Table 13. TSRST Test Results.

Asphalt Type	Fracture	
	Temperature (°C)	Strength (kPa)
AC-10	-24.5	3140
AC-20P	-31.5	4250

As a point of interest, the bending beam rheometer (BBR) test on the asphalt cement was compared to the TSRST mixture test. For the AC-10, the BBR had a lowest performance temperature of -22°C, approximately 2°C warmer than the TSRST. For the AC-20P, the BBR had a lowest performance temperature of -31.6°C, virtually identical to the TSRST.

6.3 Summary

Results from the French rutting tester and the Hamburg wheel-tracking device, for the plant produced HMA mixture, indicated the mixture would be adequate for service in the intersection relative to rutting performance and water damage. Based on these test results, and the volumetric properties obtained from the SHRP Level I mixture design, the mixtures should provide good performance in the intersection in the future.

The objectives of the project were three-fold: 1) Maximize Performance, 2) Minimize Inconvenience, and 3) Optimize Economy. These test results indicate the first objective, maximizing performance, has a high probability of being achieved. Only field performance will indicate if this objective was actually accomplished. However, at the time of construction, the laboratory test results were excellent.

7.0 Construction

7.1 Milling and Base Course Paving

The construction sequence consisted of cold-milling and removal of the top 2 inches (50 mm) of the pavement surface in all four lanes in each direction. This occurred in one night beginning at 7:30 PM on August 30, until 1:00 AM the following morning.

After milling was completed, removal of an additional 6 inches (150 mm) of HMA pavement in the right driving lanes was accomplished in two, 3-inch (75 mm) lifts. A large-stone HMA base course conforming to the requirements of a CDOT Grading G was placed in the 6 inch (150 mm) trench in the right driving lanes. It was placed in two, 3-inch (75 mm) lifts. This phase of the construction was accomplished in the second night, on August 31. The resulting pavement consisting of a cold-milled surface and base course was opened to traffic. Figure 10 is a photograph showing the appearance of the milled and base course surfaces.

7.2 Surface Course Paving

The surface course was placed twelve days after the milling and base course construction. Construction of the surface course also occurred during the third night at 7:30 PM on September 12 and ending at 2:00 AM the following morning. Construction and traffic control was sequenced to allow for placement of both the AC-10 and AC-20P in different quadrants of the intersection. The appearance of the surface course mixture is shown in Figure 11.

7.3 Summary

The objectives of the project were three-fold: 1) Maximize Performance, 2) Minimize Inconvenience, and 3) Optimize Economy. The construction sequencing indicated the second objective, minimizing inconvenience, was achieved. The construction took only three evenings, and the intersection remained open to traffic throughout construction. Additionally, no special equipment was required.

Figure 10. Appearance of Surface After Milling and Base Course Placement.

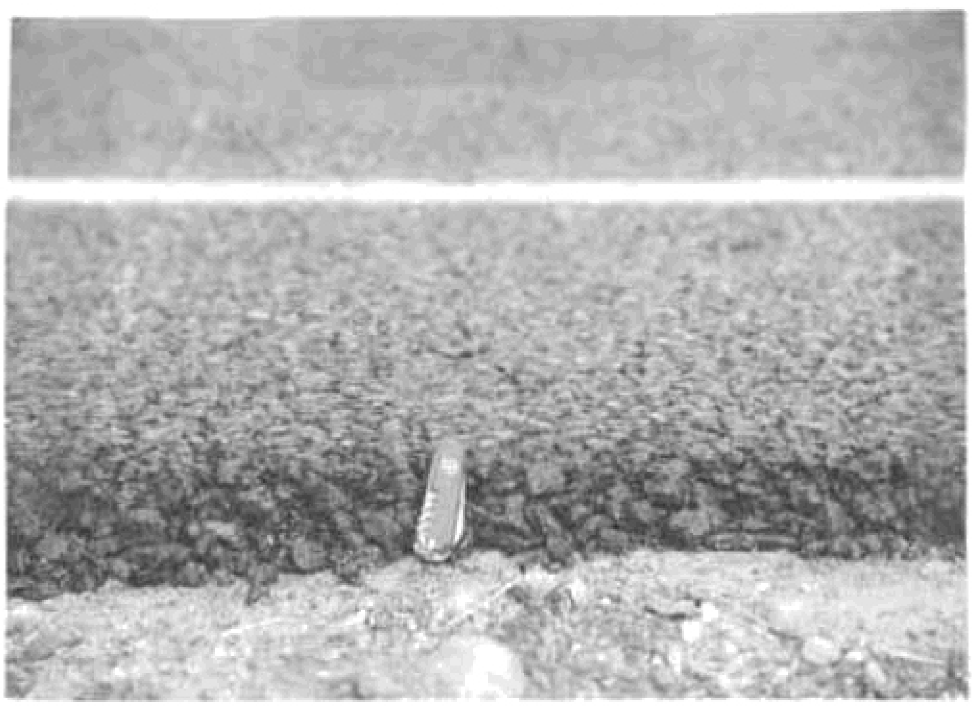

Figure 11. Appearance of Surface Course Mixture.

8.0 Economy

8.1 Life Cycle Cost

8.1.1 HMA Cost

The HMA pavement was placed under a maintenance contract. The special mixture used at this intersection was not the mixture that was bid. The cost of the special mixture exceeded the bid cost; however, the CDOT did not have to pay for any of the additional cost. The additional cost was funded by the contracting industry. The contractor made cost estimates for this special mix assuming they were to bid the mixture on future projects.

The AC-10 mixture was estimated to cost $29 per ton, and the AC-20P mixture was estimated to cost $37 per ton in place. These estimated costs are very similar to the costs of the standard CDOT mixtures.

8.1.2 Life Cycle Cost Analysis

A life cycle cost analysis was performed using DARWin 2.01 (Pavement Design, Analysis, and Rehabilitation for Windows) that was developed by ERES Consultants for AASHTO using the 1993 AASHTO design guide for pavements. For this study, the life cycle cost was based on a net present value (NPV) analysis. Five different pavement alternatives (three PCC and two HMA) were analyzed. Each pavement section and the NPV life cycle cost are shown in Table 14. The assumptions used in the life cycle cost analysis are shown below.

General Assumptions:

20 year design with 30 year life cycle cost analysis
User costs are not included
Mobilization, traffic control, and PE costs included
4 total lanes with a project length of 0.32 km (0.2 miles)

HMA Pavement Assumptions:

7.7 million ESALs over 20 years
10 and 20 year rehabilitation with 2" (50 mm) overlay (milling after 20 year rehabilitation only)
Maintenance of $900 / lane mile / year
Base course cost = $26 / ton
Surface course cost = $29 / ton (without polymer)
Surface course cost = $37 / ton (with polymer)

PCC Pavement Assumptions:

10.8 million ESALs over 20 years
19.7 million ESALs over 30 years
20 year rehabilitation with 3" (75 mm) overlay
Maintenance of $300 / lane mile / year for years 1 through 20
Maintenance of $600 / lane mile / year for years 21 through 30
PCC cost = $2.00 / square yard × inch

Table 14. Life Cycle Cost Analysis for the Intersection.

Pavement Section Alternatives	Life Cycle Cost* (Net Present Value)
1A. Mill and fill 2" (50 mm) HMA with polymer	$160,371
1B. Mill and fill 2" (50 mm) HMA without polymer	$137,988
2A. 6" (150 mm) HMA base course under 2" (50 mm) HMA surface course with polymer	$245,775
2B. 6" (150 mm) HMA base course under 2" (50 mm) HMA surface course without polymer	$222,402
3. 5" (125 mm) PCC white topping	$157,827
4. 9.5" (225 mm) PCC reconstruction (20-year design)	$259,203
5. 10.5" (250 mm) PCC reconstruction (30-year design)	$264,994

* Total intersection cost.

The thicknesses used in the life cycle cost analysis were determined based upon the component method of design. If a life cycle cost analysis were actually used to select between HMA or PCC pavement, then it is recommended to perform a deflection testing analysis with the falling weight deflectometer (FWD).

8.1.3 Full-Depth Treatment

For full-depth treatment, the HMA sections (alternatives 2A and 2B) were more cost effective than the PCC sections (alternatives 4 and 5). Alternative 4 with PCC pavement was 17% more expensive than Alternative 2 with HMA pavement. For a high volume intersection, such as the one investigated in this report, a full-depth treatment would be the recommended rehabilitation treatment.

8.1.4 Surface Treatment

Surface treatments were analyzed for information. The intersection was constructed with a thin HMA surface treatment that consisted of milling and filling; however, it is not known if this HMA surface treatment will provide adequate performance. The HMA surface treatments (Alternative 1A and 1B) were very similar in cost to the PCC surface treatment of white topping (Alternative 3).

8.2 Summary

The objectives of the project were three-fold: 1) Maximize Performance, 2) Minimize Inconvenience, and 3) Optimize Economy. The cost per ton of the HMA and the life cycle cost analysis of the full-depth treatment indicated the third objective, optimizing economy, was achieved. The AC-10 mixture was estimated to cost \$29 per ton, and the AC-20P mixture was estimated to cost \$37 per ton in place. These estimated costs are very similar to the costs of the standard CDOT mixtures.

9.0 Field Performance

Construction of the demonstration project was completed in September of 1994 and, to date, there is no evidence of rutting distress. Although more time will be required before a judgment can be made regarding rutting or water susceptibility of this system, the very high traffic volume using this intersection should provide an indication in a relatively short time period.

Transverse profilograph reading were taken in March of 1995, and the traces are shown in Appendix E. These readings will serve as a baseline of rut depth measurements for future evaluations. Future field evaluations will include crack mapping and rut measurements.

10.0 Summary

A demonstration project was constructed at a very high traffic (7.7 million 18 kip ESALs) intersection near Denver, Colorado to measure the ability of an HMA mixture to resist permanent deformation and moisture damage. The objectives of the project were three-fold: 1) Maximize Performance, 2) Minimize Inconvenience, and 3) Optimize Economy.

Pavement sections evaluated consist of 2 inches (50 mm) of surface course over the cold-milled existing pavement and 2 inches (50 mm) of surface course over 6 inches (150 mm) of large stone base course. Both conventional AC-10 and AC-20P asphalt cements were used for the HMA surface course.

10.1 Maximize Performance

The surface course mixture was designed using new SUPERPAVE technology and verified using the Hamburg wheel-tracking device, French rutting tester and TSRST technology.

Results from these tests indicated the first objective, maximizing performance, had a high probability of being achieved. Only field performance will indicate if this objective was actually accomplished. However, at this time, the laboratory test results were excellent.

10.2 Minimize Inconvenience

The construction sequencing indicated the second objective, minimizing inconvenience, was achieved. The construction took only three evenings, and the intersection remained open to traffic throughout construction. Additionally, no special equipment was required.

10.3 Optimize Economy

The cost per ton of the HMA and the life cycle cost analysis of the full-depth treatment indicated the third objective, optimizing economy, was achieved. The AC-10 mixture was estimated to cost $29 per ton, and the AC-20P mixture was estimated to cost $37 per ton in place. These estimated costs are very similar to the costs of the standard CDOT mixtures.

11.0 References

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2. "SUPERPAVE Asphalt Mixture Design and Analysis," National Asphalt Training Center, Demonstration Project 101, FHWA and Asphalt Institute, March 1994.

3. "AASHTO-ARTBA-AGC Joint Task Force 31, "Guidelines for Polymer Modified Asphalts," January 1992.

4. Hines, Mickey, "The Hamburg Wheel-Tracking Device," Proceeding of the Twenty-Eighth Paving and Transportation Conference, The University of New Mexico, Albuquerque, NM, 1991.

5. Vinson, T.S., V.C. Janoo, and R.C.G. Haas, "Summary Report on Low Temperature and Thermal Fatigue Cracking," SHRP-A/IR-90-001, June 1989.

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Appendix A:

SUPERPAVE Aggregate Specifications

COARSE AGGREGATE ANGULARITY

Coarse Aggregate Angularity:		
Traffic,	Depth from Surface	
ESALs	< 100 mm	> 100 mm
$< 3 \times 10^5$	55/-	-/-
$< 1 \times 10^6$	65/-	-/-
$< 3 \times 10^6$	75/-	50/-
$< 1 \times 10^7$	85/80	60/-
$< 3 \times 10^7$	95/90	80/75
$< 1 \times 10^8$	100/100	95/90
$> 3 \times 10^8$	100/100	100/100
Note: "85/80" denotes that 85 % of the coarse aggregate has one fractured face and 80 % has two fractured faces.		

FINE AGGREGATE ANGULARITY

Fine Aggregate Angularity:		
Traffic,	Depth from Surface	
ESALs	< 100 mm	> 100 mm
$< 3 \times 10^5$	-	-
$< 1 \times 10^6$	40	-
$< 3 \times 10^6$	40	40
$< 1 \times 10^7$	45	40
$< 3 \times 10^7$	45	40
$< 1 \times 10^8$	45	45
$> 3 \times 10^8$	45	45
Note: Criteria are presented as percent air voids in loosely compacted fine aggregate.		

FLAT AND ELONGATED PARTICLES

Flat, Elongated Particles	
Traffic, ESALs	Percent
$< 3 \times 10^5$	-
$< 1 \times 10^6$	-
$< 3 \times 10^6$	10
$< 1 \times 10^7$	10
$< 3 \times 10^7$	10
$< 1 \times 10^8$	10
$> 3 \times 10^8$	10
Note: Criteria are presented as maximum percent by weight of flat and elongated particles.	

CLAY CONTENT

Clay Content	
Traffic, ESALs	Sand Equivalent, minimum
$< 3 \times 10^5$	40
$< 1 \times 10^6$	40
$< 3 \times 10^6$	40
$< 1 \times 10^7$	45
$< 3 \times 10^7$	45
$< 1 \times 10^8$	50
$> 3 \times 10^8$	50

COLORADO PROCEDURE 45-90

FOR DETERMINING PERCENT OF PARTICLES WITH ONE OR MORE FRACTURED FACES

SCOPE

1.1 This method describes the procedure for determining the percentage of crushed particles in an aggregate sample.

NOTE 1: If the test is performed in conjunction with a sieve analysis test such as CP-31, save the plus No. 4 portion and reduce, if desired, by splitting to the test size shown in the table in 3.2 and proceed as in 4.2.

APPARATUS

2.1 Balance, having sufficient capacity and sensitive to 1 gram.

2.2 Sieve, No. 4 with square openings conforming to AASHTO M 92.

2.3 Sample splitter, for the selection of a representative specimen.

2.4 Drying equipment, such as a stove or oven.

SAMPLE AND TEST SPECIMEN SIZE

3.1 The minimum required weight of the total sample shall conform to the requirements of the table as shown in CP-30 or to the applicable table in CP-42 if the test is to be determined on the extracted aggregate.

3.2 The minimum weight of the total specimen shall be sufficient to yield a plus No. 4 test specimen conforming to the following table:

SIZE OF PLUS NO. 4 TEST SPECIMEN

Nominal Maximum Aggregate Size	Minimum Weight of Specimen, grams
3/8 in.	100
1/2 in.	200
3/4 in., or over	300

PROCEDURE

4.1 Sieve the total unwashed specimen over the No. 4 sieve and discard the minus No. 4 material. Wash the retained material and dry at 230° F $\pm$ 9. When dry, sieve it over a No. 4 sieve (Note 1).

4.2 Weigh the plus No. 4 specimen and spread onto a work table large enough so the individual particles may be inspected.

4.3 Separate the particles with one or more fractured faces from those without. A rounded particle with a small chip broken off shall not be counted as having a fractured face. A particle is counted if 25% or more of the surface area appears to be fractured.

4.4 Weigh the particles with one or more fractured faces and record as "weight of fractured aggregate."

CALCULATIONS

5.1 Determine the percentage of particles with one or more fractured faces by dividing the weight of the fractured aggregate by the total weight of the plus No. 4 test specimen and calculate:

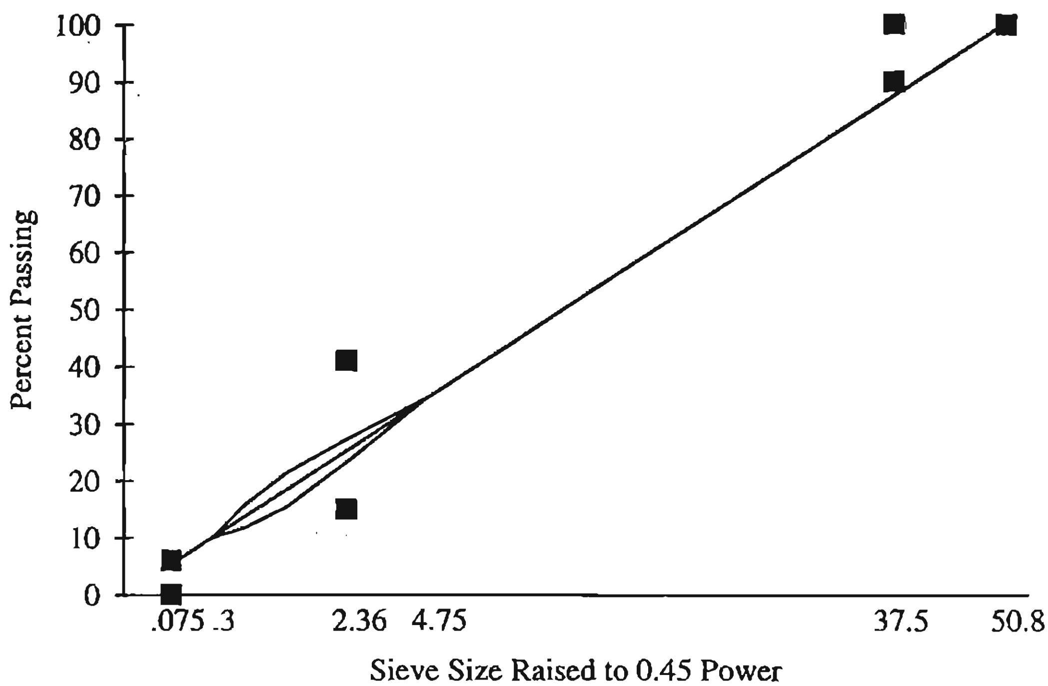
$$\textit{Percent of Particles with one or more fractured faces} = \frac{\textit{weight of fractured aggregate}}{\textit{total weight of specimen}} \times 100$$

Appendix B:

SUPERPAVE Gradation "Master Ranges"

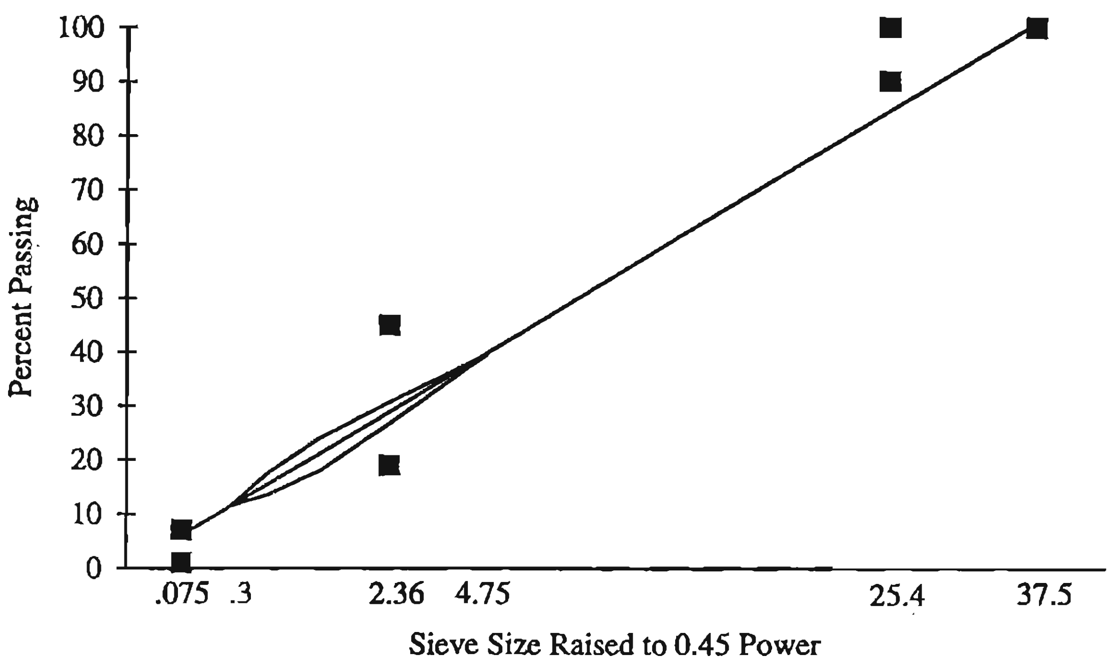
37.5 MM NOMINAL SIZE

Sieve					0.45 chart	Restricted Zone	
mm	µm	0.45	Control Points		Max Dens	Minimum Boundary	Maximum Boundary
50	50000	130		100.0	100.0		
37.5	37500	114	100.0	90.0	87.9		
25.4	25000	95			73.2		
19.00	19000	84			64.7		
12.50	12500	70			53.6		
9.50	9500	62			47.4		
4.75	4750	45			34.7	34.7	34.7
2.36	2360	33	41.0	15.0	25.3	23.3	27.3
1.18	1180	24			18.5	15.5	21.5
0.60	600	18			13.7	11.7	15.7
0.30	300	13			10.0	10.0	10.0
0.15	150	10			7.3		
0.075	75	7	6.0	0.0	5.4		



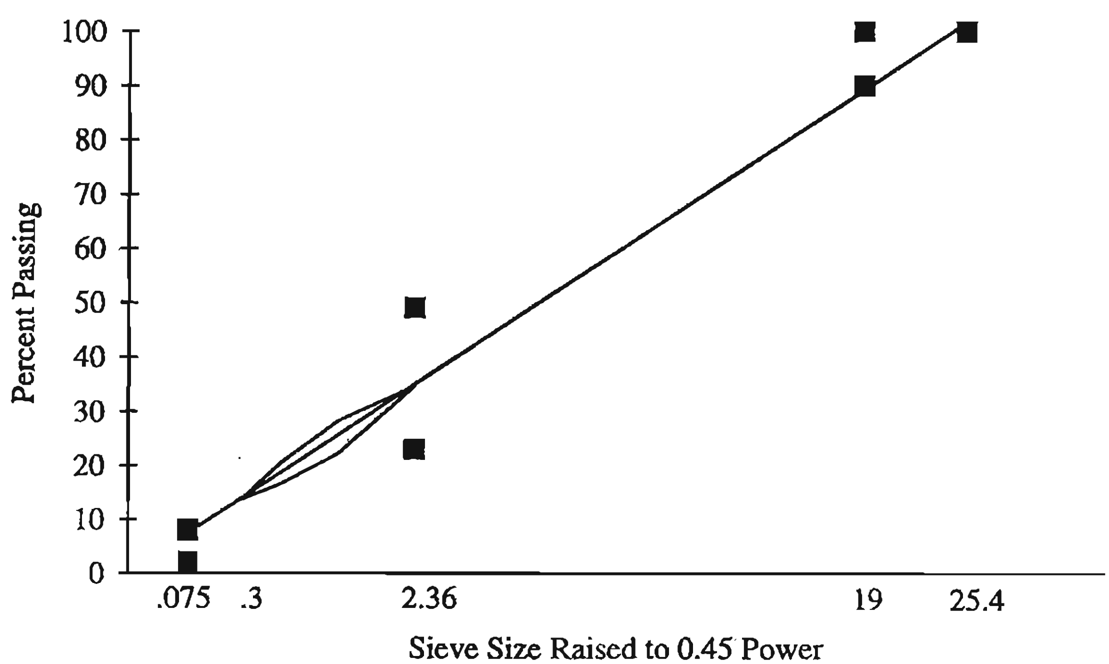
25 MM NOMINAL SIZE

Sieve						Restricted Zone	
					0.45 chart	Minimum	Maximum
mm	µm	0.45	Control Points		Max Dens	Boundary	Boundary
37.5	37500	114		100.0	100.0		
25.4	25000	95	100.0	90.0	83.3		
19.00	19000	84			73.6		
12.50	12500	70			61.0		
9.50	9500	62			53.9		
4.75	4750	45			39.5	39.5	39.5
2.36	2360	33	45.0	19.0	28.8	26.8	30.8
1.18	1180	24			21.1	18.1	24.1
0.60	600	18			15.6	13.6	17.6
0.30	300	13			11.4	11.4	11.4
0.15	150	10			8.3		
0.075	75	7	7.0	1.0	6.1		



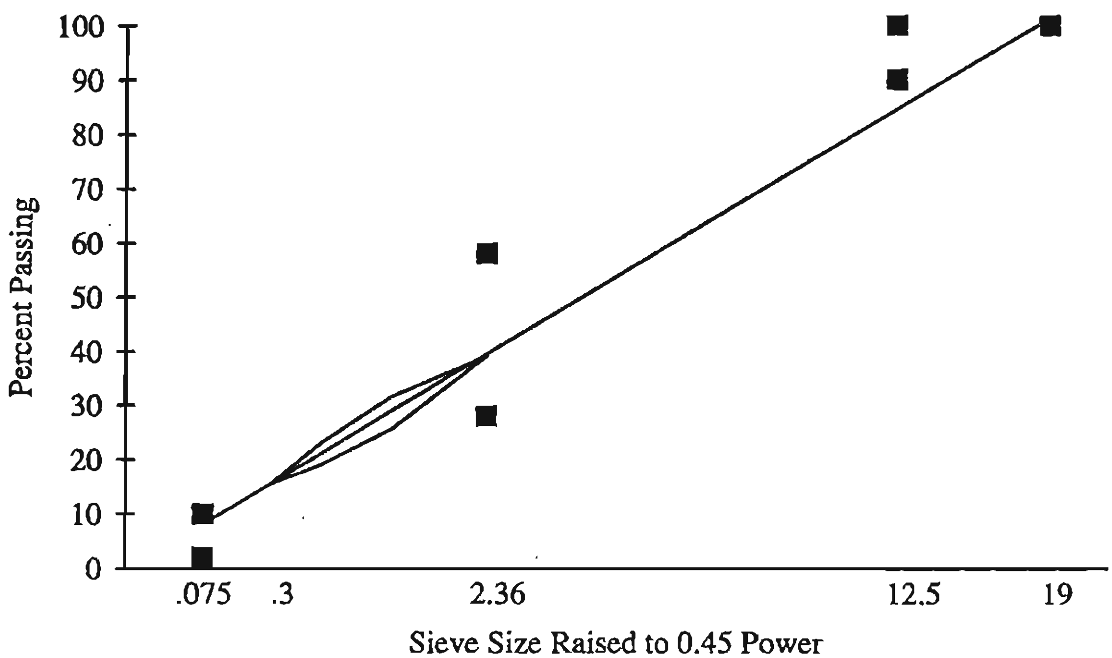
19 MM NOMINAL SIZE

Sieve mm	µm	0.45	Control Points		0.45 chart Max Dens	Restricted Zone Minimum Boundary	Restricted Zone Maximum Boundary
25.4	25000	95		100.0	100.0		
19.00	19000	84	100.0	90.0	88.4		
12.50	12500	70			73.2		
9.50	9500	62			64.7		
4.75	4750	45			47.4		
2.36	2360	33	49.0	23.0	34.6	34.6	34.6
1.18	1180	24			25.3	22.3	28.3
0.60	600	18			18.7	16.7	20.7
0.30	300	13			13.7	13.7	13.7
0.15	150	10			10.0		
0.075	75	7	8.0	2.0	7.3		



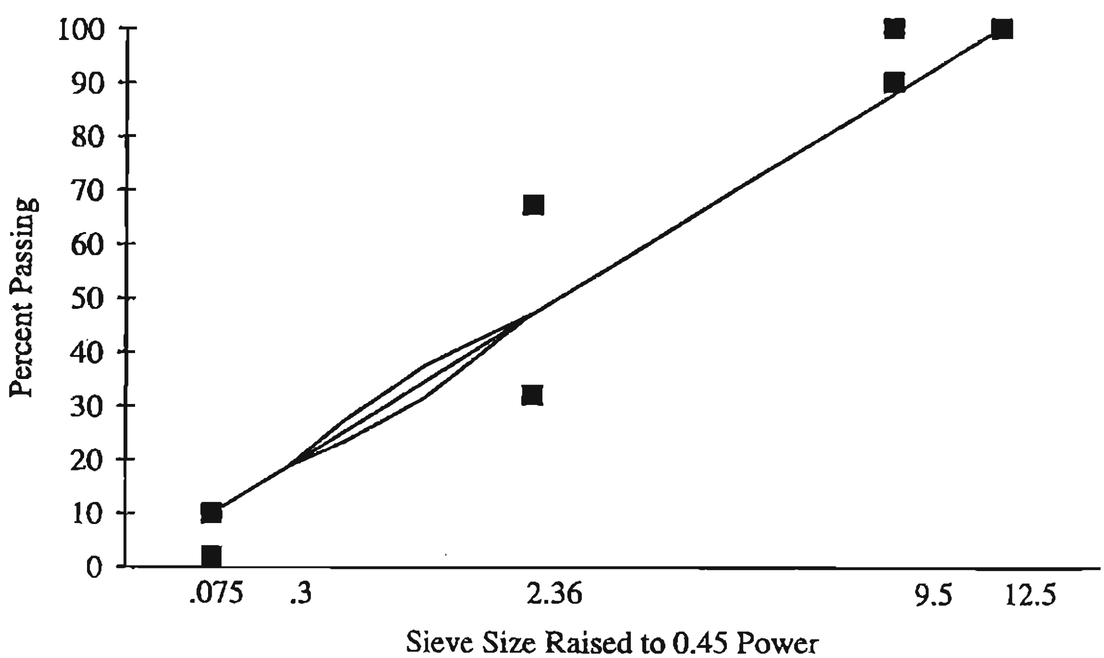
12.7 MM NOMINAL SIZE

Sieve mm	µm	0.45	Control Points		0.45 chart Max Dens	Restricted Zone Minimum Boundary	Restricted Zone Maximum Boundary
19.00	19000	84		100.0	100.0		
12.50	12500	70	100.0	90.0	82.8		
9.50	9500	62			73.2		
4.75	4750	45			53.6		
2.36	2360	33	58.0	28.0	39.1	39.1	39.1
1.18	1180	24			28.6	25.6	31.6
0.60	600	18			21.1	19.1	23.1
0.30	300	13			15.5	15.5	15.5
0.15	150	10			11.3		
0.075	75	7	10.0	2.0	8.3		



9.5 MM NOMINAL SIZE

Sieve mm	µm	0.45	Control Points		0.45 chart Max Dens	Restricted Zone Minimum Boundary	Restricted Zone Maximum Boundary
12.50	12500	70		100.0	100.0		
9.50	9500	62	100.0	90.0	88.4		
4.75	4750	45			64.7		
2.36	2360	33	67.0	32.0	47.2	47.2	47.2
1.18	1180	24			34.6	31.6	37.6
0.60	600	18			25.5	23.5	27.5
0.30	300	13			18.7	18.7	18.7
0.15	150	10			13.7		
0.075	75	7	10.0	2.0	10.0		



Appendix C:

SUPERPAVE Gyratory Gyration Specifications

N-Design

ESAL (10^6)	Average 7-Day High Air Temperature (°C)			
	<39	39-40	41-42	43-44
<0.3	68	74	78	82
0.3-1	76	83	88	93
1-3	86	95	100	105
3-10	96	106	113	119
10-30	109	121	128	135
30-100	126	139	146	153
>100	142	158	165	172

N-Maximum

ESAL (10^6)	Average 7-Day High Air Temperature (°C)			
	<39	39-40	41-42	43-44
<0.3	104	114	121	127
0.3-1	117	129	138	146
1-3	134	150	158	167
3-10	152	169	181	192
10-30	174	195	208	220
30-100	204	228	240	253
>100	233	262	275	288

N-Initial

ESAL (10^6)	Average 7-Day High Air Temperature (°C)			
	<39	39-40	41-42	43-44
<0.3	7	7	7	7
0.3-1	7	7	7	8
1-3	7	8	8	8
3-10	8	8	8	9
10-30	8	9	9	9
30-100	9	9	9	10
>100	9	10	10	10

Appendix D:

SUPERPAVE Volumetric Specifications

Minimum VMA Requirements

Nominal Maximum Aggregate Size	Minimum VMA, %
9.5 mm	15.0
12.5 mm	14.0
19 mm	13.0
25 mm	12.0
37.5 mm	11.0

Acceptable Range of Voids Filled with Asphalt

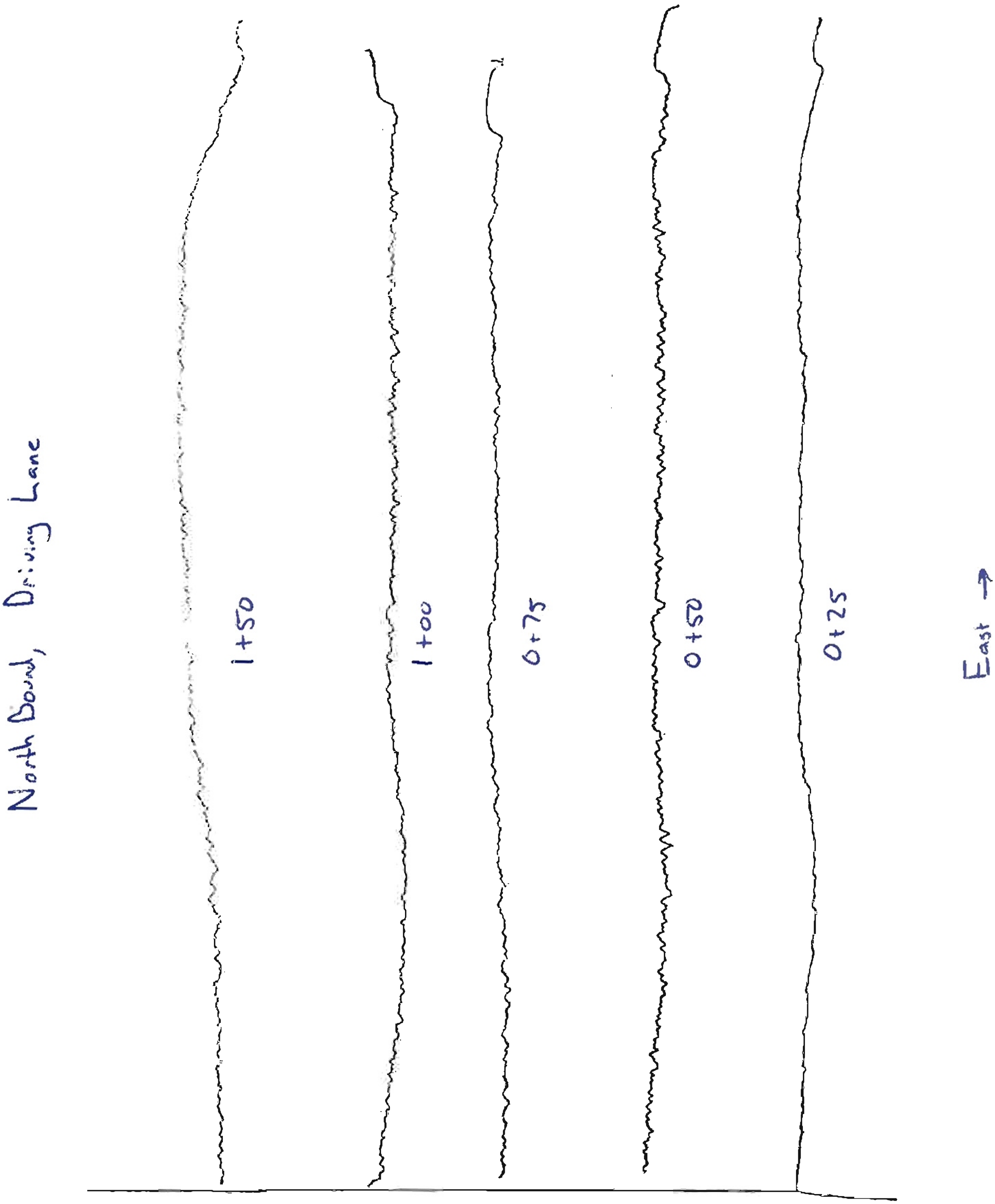
Traffic, ESALs	Design VFA, %
$< 3 \times 10^5$	70 - 80
$< 1 \times 10^6$	65 - 78
$< 3 \times 10^6$	65 - 78
$< 1 \times 10^7$	65 - 75
$< 3 \times 10^7$	65 - 75
$< 1 \times 10^8$	65 - 75
$> 3 \times 10^8$	65 - 75

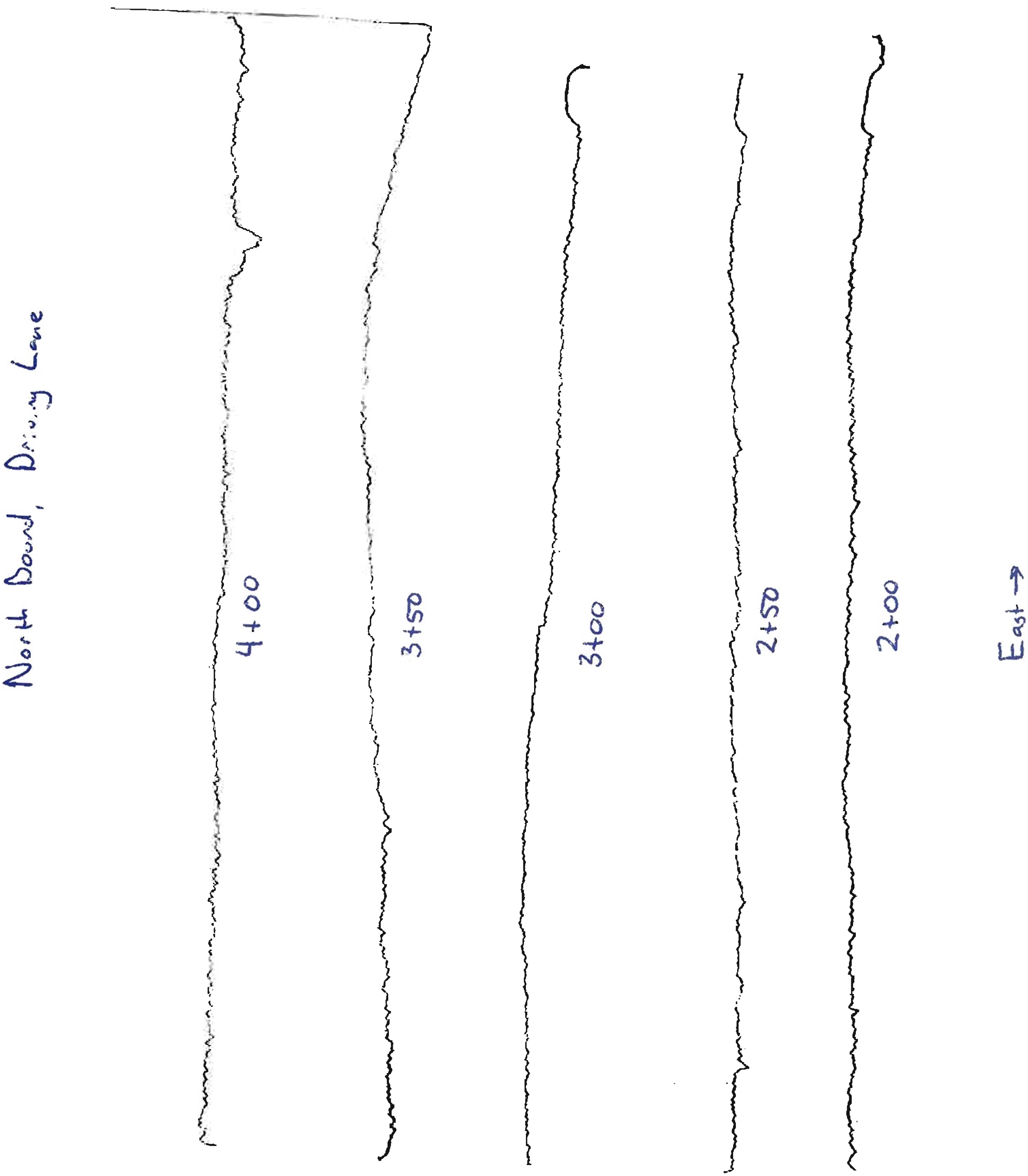
Appendix E:

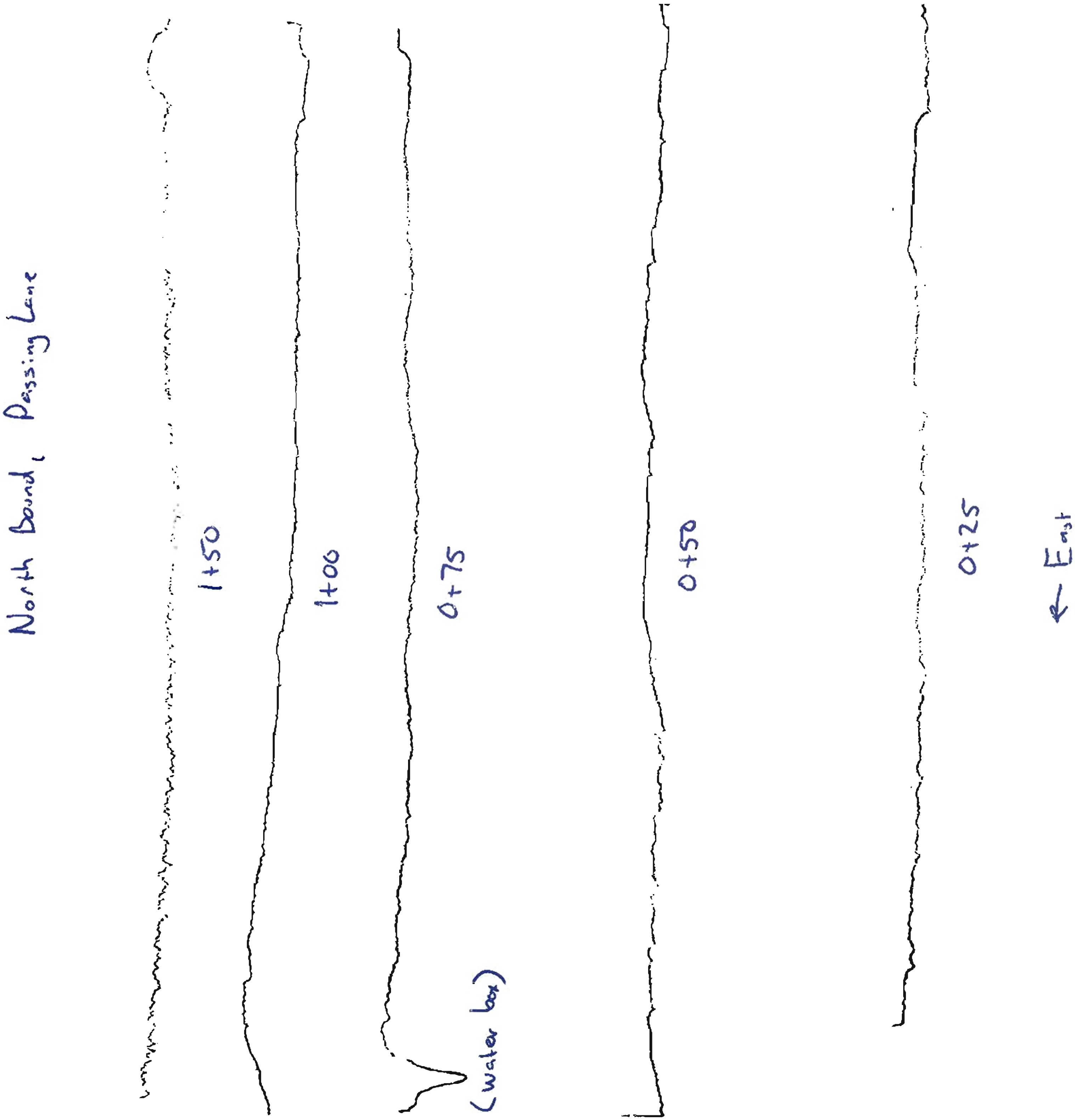
Profilograph Traces

Scaling

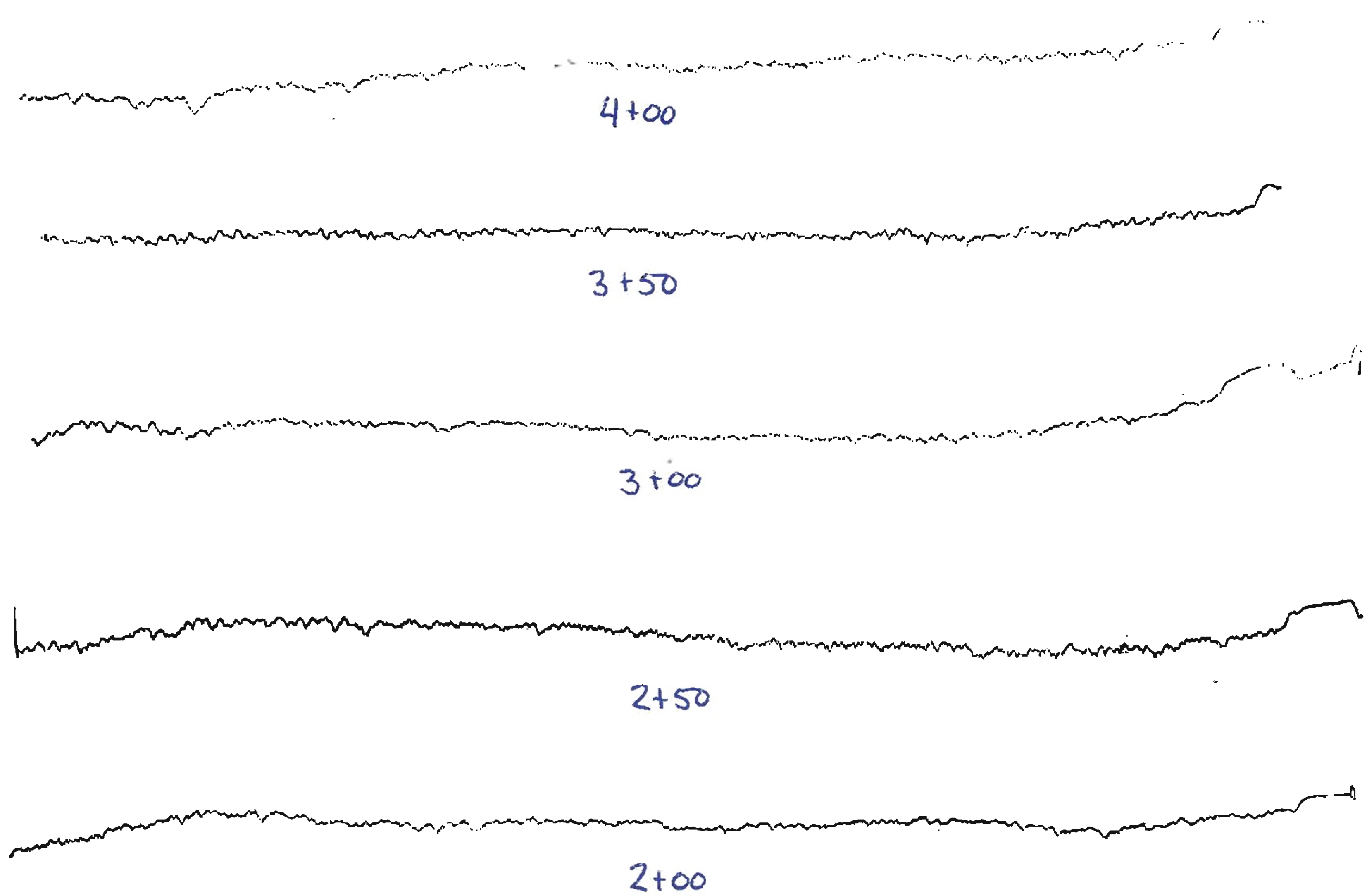
Horizontal: 1" = 1.4'
Vertical: 1" = 1.05"

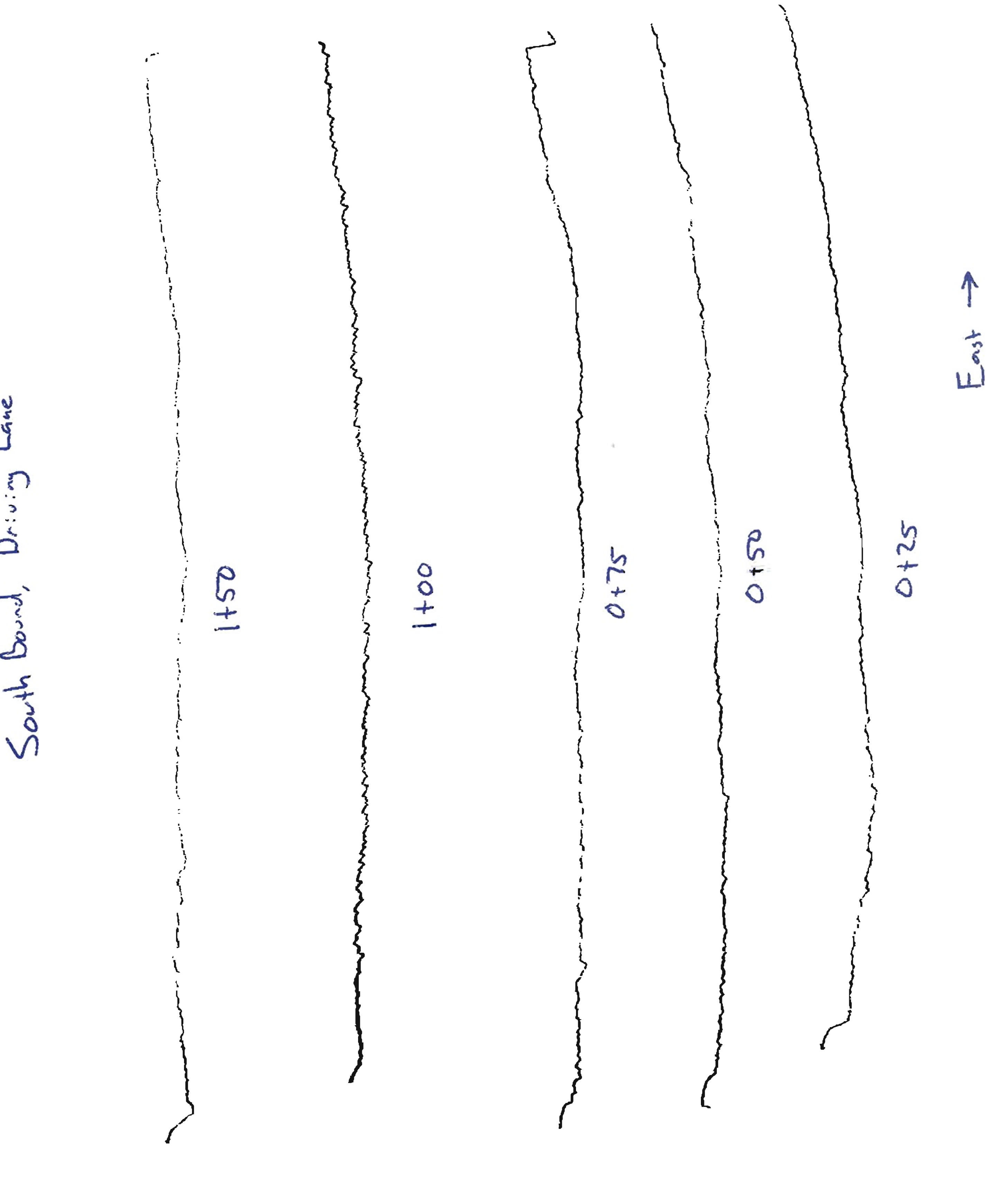

North Bound, Driving Lane
1+50
1+00
0+75
0+50
0+25
East →

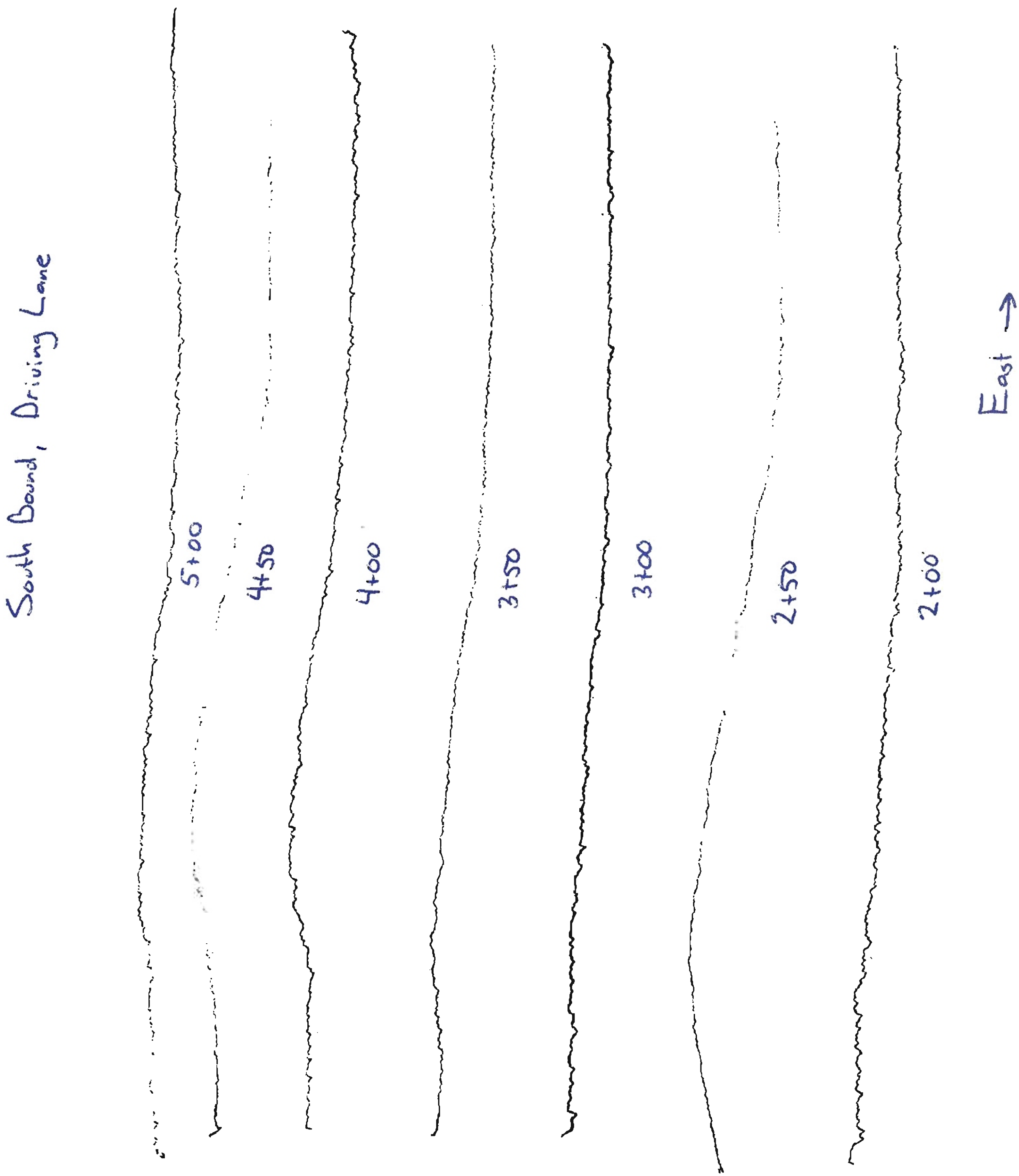

North Bound, Driving Lane
4+00
3+50
3+00
2+50
2+00
East →


North Bound, Passing Lane
1+50
1+00
0+75
(water box)
0+50
0+25
← East

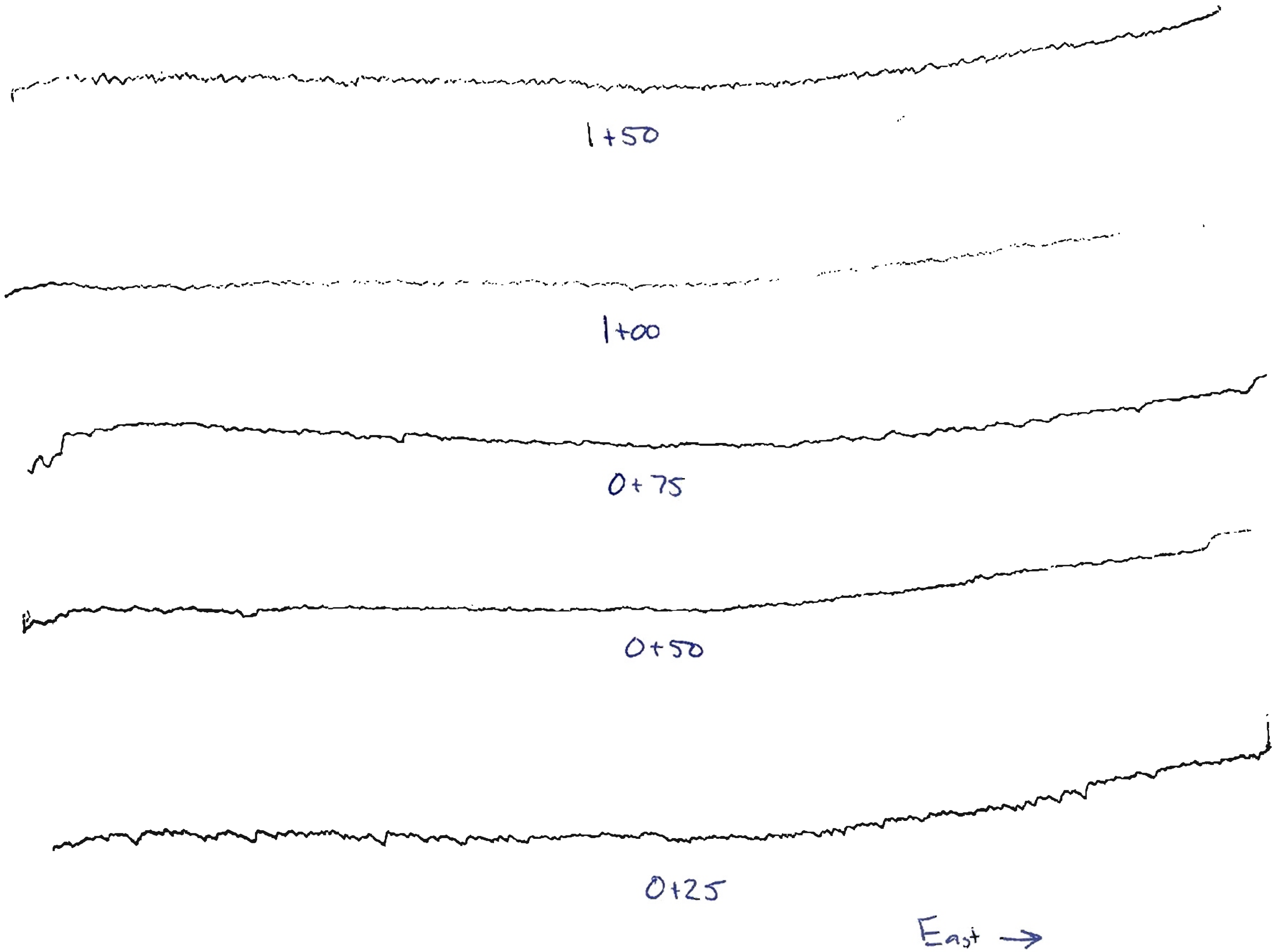
North Bound, Passing Lane



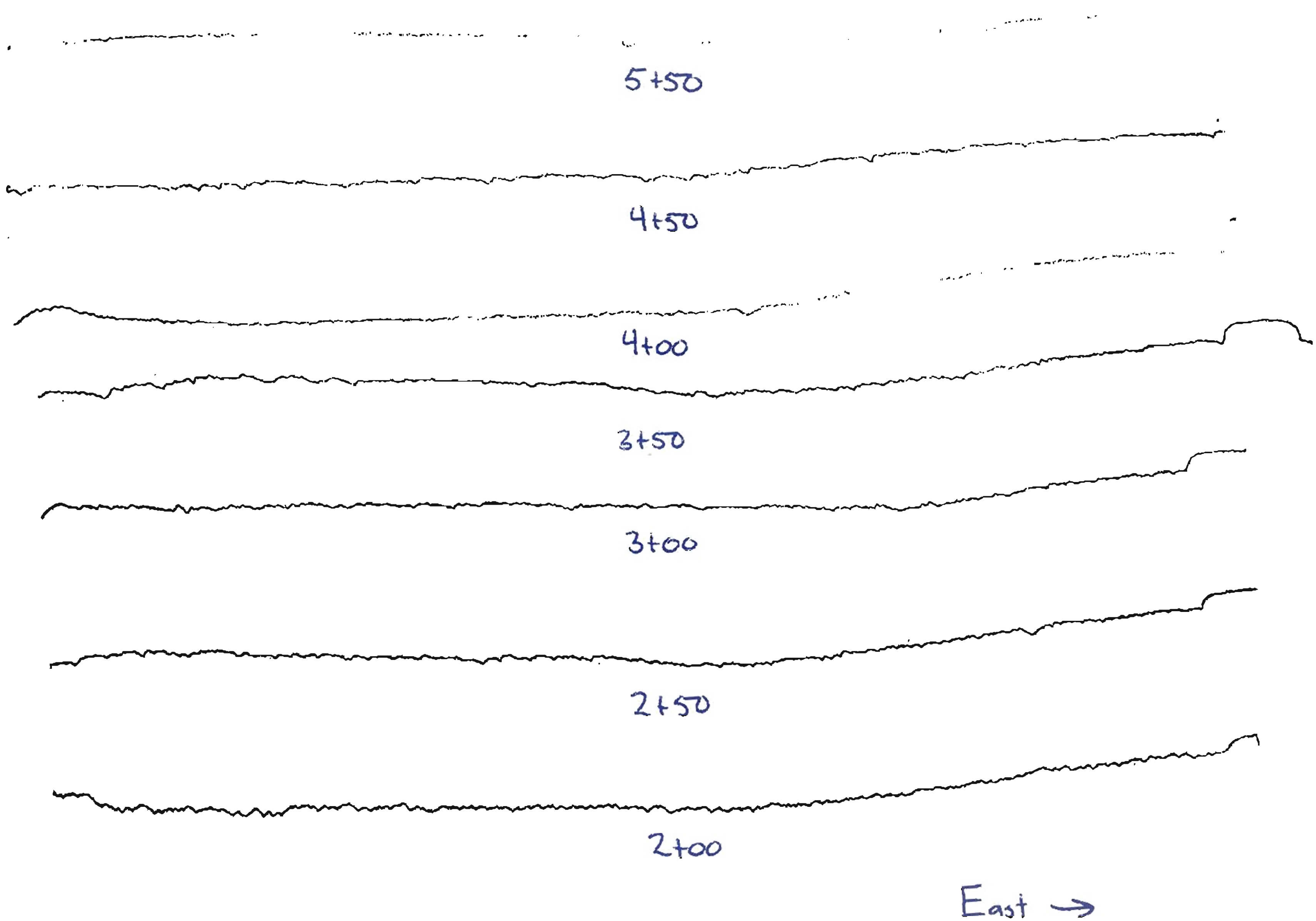

South Bound, Driving Lane
1+50
1+00
0+75
0+50
0+25
East →


South Bound, Driving Lane
5+00
4+50
4+00
3+50
3+00
2+50
2+00
East →

South Bound, Passing Lane



South Bound, Passing Lane



FIELD NOTES

Date: March 23, 1995

Study: Asphalt Intersection

Loc: US 85 MP 227.3 (Jct w/ SH 44 - 104th ave.)

By: Skip Outcalt, Research Branch

On Monday March 13, 1995 I took a Rainhart transverse profilograph, borrowed from C. T. L. Thompson, to US 85 at 104th Ave. north of Commerce City. There are test sites on both north and southbound US 85 at the intersection.

The intersection is controlled by a traffic light. US 85 has a high volume of heavily loaded trucks which had caused ruts up to two inches deep at the intersection. During September of 95, with contributions from several contractors, the surface was milled and paved from the intersection upstream about 500′ on the northbound side and about 600′ on the southbound sides. To reduce the inconvenience to the public, the passing lane was milled 2" and the driving lane milled 6" during night operations. The driving lane was paved with two lifts of Gr G, then the site was left with the milled surface exposed in the passing lanes for 12 days. A final lift of 2" of coarse graded asphalt with AC-10 was placed over both lanes northbound, and coarse graded asphalt with AC-10 and polymer on both southbound lanes.

C. T. L. Thompson loaned us the Rainhart transverse profilograph so we could accurately monitor the progression of ruts as they develop in the new pavement. The Rainhart device is a square steel box beam, about 14′ in overall length, that stands on legs at both ends. To make a trace of the pavement surface, a carrier which holds a paper wrapped drum and rides above the beam on rollers is pushed across the lane. A vertical rod is mounted to the carrier in bearings so it can move up and down as a small wheel on the bottom follows the surface of the pavement. Mounted to the vertical rod is a pen holder which can be adjusted to the desired location on the paper on the drum. (I easily made seven traces on one piece of 8-1/2" X 17" paper.) As the operator pushes the carrier along the beam, the pen is moved vertically in a direct ratio by the surface of the road, and the drum is turned by a cable and pulley mechanism to make a trace. The trace has a horizontal scale of 1:12 and a vertical scale of 1:1. For example, a road 12 feet wide with a 1 inch rut will make a trace 12 inches long with a 1 inch deep depression corresponding to the rut.

The intersection is not at a right angle so the first trace site is located about 25' (25' on the northbound side and 30' on the southbound side) back from the stop bar painted on the pavement. In the northbound lanes, the intersection of the stop bar and the painted shoulder stripe is defined as 0+00. Traces were made in both lanes at 0+25, 0+50, 0+75, 1+00, 1+50, 2+00, 2+50, 3+00, 3+50, and 4+00. In the southbound lanes, a point on the shoulder stripe 5' upstream from the stop bar is defined as 0+00. Traces were taken at 0+25, 0+50, 0+75, 1+00, 1+50, 2+00, 2+50, 3+00, 3+50, 4+00, 4+50, and 5+50. The sites at 3+00 and 4+00 will probably not be used in the future - we will take ten tests in each direction. The crew drove PK nails into the asphalt in the edge of the paint stripes (both shoulders and the skip stripe) at each trace site so traces in the future could be made as close as possible to the original location.

The traces show no indication of rutting but the surface is concave in some locations and convex in others. These profiles show why measurements taken with a straight edge could be misleading as to the actual depth of ruts even in a situation where there is no shoving. Tim Aschenbrener, the study manager, plans to have profilograph readings taken during the winter or early spring for the next several years to monitor the site.

Distribution:
Tim Aschenbrener, Materials
Donna Harmelink, Research
Steve Horton, Materials
Bob LaForce, Materials
Scot Shuler, CAPA
file